UEFA BERKELEY DESIGN

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CE 244 – REINFORCED CONCRETE STRUCTURES

UNIVERSITY OF CALIFORNIA, BERKELEY
DEPT. OF CIVIL AND ENVIRONMENTAL ENGINEERING
PROF. EYITAYO OPABOLA
GSI HECTOR GARCIA

UNIVERSITY OF CALIFORNIA, BERKELEY

1950 OXFORD ST. RESIDENTIAL BUILDING

Design Documents, Drawings and Calculations
Friday, December 12, 2025

UEFA
BERKELEY
DESIGN ®

Letter of Transmittal

December 12, 2025
University of California, Berkeley
1950 Oxford St, Berkeley, CA 94704

Subject: Structural Design Documents, Drawings, and Calculations for the 1950 Oxford St. Residential Building

Dear Professor Eyitayo Opabola and Mr. Hector Garcia,

The structural design team at UEFA Berkeley Design is honored to have been allowed to contribute to the new 1950 Oxford St. building at the U.C. Berkeley campus. As U.C. Berkeley alumni, our team of experienced structural engineers found this project both intellectually engaging and creatively fulfilling. Located just west to the Hayward Fault, it is an opportunity to deliver a safe and visually compelling structure that replicates that of the greater Berkeley area.

Per client specifications, we are pleased to present the design of a new nine-story, reinforced concrete residential building. The facility will feature a lobby, eight floors of residential space, and a total square footage of 112,128 square feet. The client-consultant relationship is one of significant importance for UEFA Berkeley Design. Hence, the team placed an emphasis on the ease of construction, cost-effectiveness, and seismic resilience. This inspired the building's gravity and lateral systems to consist of a post-tensioned flat-plate system supported by columns and walls at each level and coupled concrete shear walls along the building height, respectively. Furthermore, the shear wall system is placed around the central elevator and stair core, integrating it into the spacious and stunning interior.

Our team of inspired engineers have worked iteratively and meticulously to ensure the best design possible to allow for quality building performance and economy. This involves sufficient yielding of the structure in the event of an earthquake, ample design of gravity members, and steering clear of overdesign. Ultimately, this design embodies that of the surrounding community: unique, resilient, and outstanding.

Further information, including a basis of design, structural calculation package, and building/section details can be found in the following attached report. We are proud of the outcome of the design and are sincerely grateful for the opportunity and your continued support throughout the entire process.

Sincerely,
Felipe Delgado, Facundo L. Pfeffer, Carlos Quezada, Everardo Solis-Correa at UEFA Berkeley Design

Contents

1 Part I: Basis of Design

1.0.1 Project Description

This term project develops and documents the preliminary design of a reinforced–concrete residential building located just west of the UC Berkeley campus[1]. The notional building is nine stories tall with a basement; the basement plan measures 160 ft × 160 ft (see **Fig. 1.0.1**).

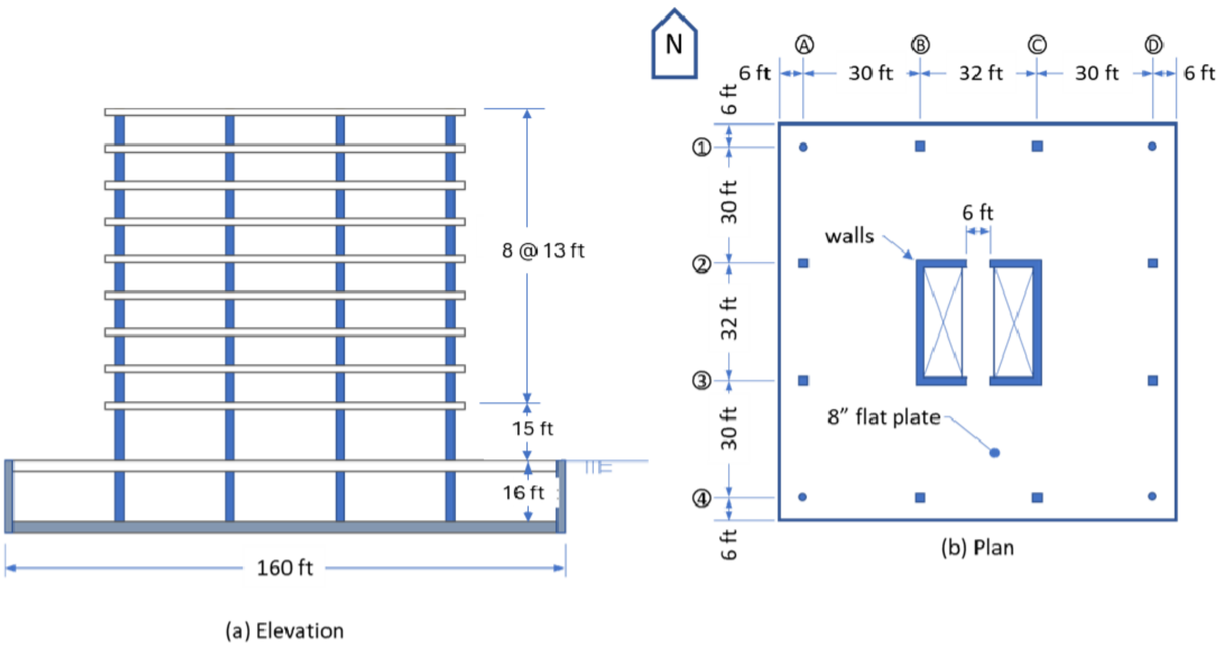


Figure 1.0.1: (a) Elevation and (b) Typical Floor Plan View.

1.0.2 Site and Geotechnical Conditions

The site is in Berkeley at approximately 37.8730°, −122.2665°. A preliminary geotechnical summary indicates stiff sandy clay extending to considerable depth. For seismic design the site shall be taken as *Site Class C* per ASCE 7–22 Table 20.3-1[2].

1.1 Structural Analysis

1.1.1 Foundation

The geotechnical report is being prepared by a preliminary geotechnical engineer. Preliminary information from the report indicates that the site is underlain by very stiff to hard clays with varying sand content and layers of medium dense to very dense clayey sand. Groundwater was not observed within the depth of the foundation. Therefore, the foundation system will not need to be designed for hydrostatic uplift.

Based on these findings, the tower and podium can be supported on a mat foundation, ranging in thickness. The mat foundation is out of the scope of this academic design excercise as is thus not designed.

1.1.2 Gravity System

The above-grade structure consists of a 12” thick mild-reinforced concrete flat-plate slab at ground level supported by concrete columns and walls. At residential floors and the roof top, the gravity system consists of PT slabs supported by concrete columns and walls. Columns are located near the perimeter and internally to limit spans to 32 feet or less. Shear stud rails can be provided as a design enhancement at all columns but are not needed for capacity and thus are not designed. Slab thicknesses are typically 8-inch thick.

1.1.3 Lateral Force-resisting System

The lateral force-resisting system (LFRS) consists of coupled concrete shear walls placed around the central elevator and stair core of the tower. Wall thickness at the critical subsection is estimated to be 21” in the N-S direction and 25” in the E-W direction. Wall thickness is kept constant along the height of the structure to simplify the design and construction process. Wall coupling is provided by diagonally reinforced coupling beams. Consistent with typical residential coupling beams, the coupling beam depth will be 36 inches with span of 72 inches.

Above ground level, the concrete floor slab diaphragm delivers the distributed inertial loads to the core. At ground level, the concrete floor slab diaphragm connects the core to the continuous perimeter basement walls, activating them as shear walls. Note however, that said diaphragms are not within the scope of this academic exercise and thus are not designed.

1.1.4 Structural System Summary

The gravity system consists of an unbonded post-tensioned flat plate slab, nominal thickness 8 in., supported by reinforced–concrete columns and walls at all levels. The Level 1 slab will be thicker (on the order of 12 in.) and conventionally reinforced. Lateral resistance is provided by coupled reinforced–concrete shear walls forming the central elevator/stair core. Wall thickness varies with location. A mat foundation is assumed based on the preliminary geotechnical information.

1.2 Codes and References

The project is designed in accordance with the following building codes and material standards:

1.2.1 Building Codes

- Department of Planning and Development of the City of Berkeley, adopting the 2022 California Building Code (CBC).
- 2022 International Building Code (IBC).
- Minimum Design Loads and Associated Criteria for Buildings and Other Structures, American Society of Civil Engineers, 2022 (ASCE 7-22).

1.2.2 Material Standards

- Reinforced Concrete: American Concrete Institute, Building Code Requirements for Structural Concrete and Commentary, 2025 Edition (ACI 318-25).

1.2.3 Building Code Exceptions

The design generally conforms to the prescriptive provisions of the California Building Code, with the following exceptions approved by the Building Official:

- **Response Modification Coefficient:** $R = 5$ (Bearing Wall System), ASCE 7, Table 12.2-1.
- **Redundancy Factor (ρ):** Per ASCE 7, subsection 12.3.4.2. This factor is set equal to 1.0.
- **Redistribution of Coupling Beam Demands:** For DE design, the shear (and moment) in individual coupling beams is permitted to be redistributed vertically to adjacent floors. The maximum reduction in beam shear does not exceed 20 percent of the design shear obtained from linear analysis.

1.2.4 Governing Codes and Parameters

Design shall follow the requirements of the City of Berkeley (2022 California Building Code adopting and amending the 2021 IBC[3, 4]), with referenced standards including:

- ASCE 7–22[2] *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*.
- ACI 318–25[5] *Building Code Requirements for Structural Concrete and Commentary*.

1.3 Performance Objectives

1.3.1 Overall Building Performance Objectives

The goal of the design is to ensure that the overall building design meets minimum prescriptive design requirements of the California Building Code considering the Design Earthquake (DE) requirements. Based on occupancy considerations, the building has been classified as Risk Category II. The performance of the building when subjected to earthquakes having different intensities is not being checked explicitly as part of this design.

1.3.2 Component Performance Objectives

For all members of the structural system, member actions are categorized as either deformation-controlled or force-controlled.

- **Deformation-controlled actions:** Those expected to undergo nonlinear behavior in response to MCE_R shaking and are detailed in accordance with ACI 318 so that they will be capable of sustaining a series of oscillations into the inelastic range of response without critical deterioration in strength.
- **Force-controlled actions:** Designed using the capacity design method and therefore are not expected to undergo nonlinear behavior in response to earthquake shaking.

Table 1.3.1 lists the structural member actions and their corresponding categorization.

Table 1.3.1: Member Actions and Categories

Member Action	Force-Controlled	Deformation-Controlled
Shear Wall Flexure		X
Shear Wall Shear	X	
Coupling Beam Shear		X
Column Axial/Flexure	X	
Column Shear		X

1.4 Loading Criteria

1.4.1 Gracity Loading

In addition to the self-weight of the structure, the following minimum loadings are applied.

Table 1.4.1: Gravity Loads

Use	Live Loading (psf)	Superimposed Dead Loading (psf)
Corridor/Stairs*	100	15
Lobbies	100	40
Parking	40	5
Residential	40	25
Occupied Roof**	100	75
Roof	20	15

*Corridor/stairs loading applies to the region with the core and extending 4 ft beyond the core on all sides.
**Assume 25% of the roof is Occupied Roof.

Cladding: An exterior cladding load (Precast panel system) of 55 psf of wall area is applied as an equivalent line load along the perimeter of the floor slabs.

1.4.2 Wind Design Criteria

Lateral wind loading was not applicable within the scope of this CE 244 project. Moreover, for a mid-rise building in a high seismicity zone like the Bay Area, seismic criteria generally govern.

1.4.3 Seismic Design Criteria

Seismic loads are in accordance with the ASCE 7[2] requirements and are listed in **Table 1.4.2**.

Parameter	Value
Building Latitude	37.8730° N
Building Longitude	122.2665° W
Risk Category	II
Importance Factor, I_e	1.0
Mapped Spectral Acceleration Values	S_S = 2.49, S_1 = 0.92
Site Class	C - Very Dense Soil and Soft Rock
Spectral Response Coefficients	S_{MS} = 2.59, S_{M1} = 1.28
Design Spectral Response Coefficients	S_{DS} = 1.73, S_{D1} = 0.85
Seismic Design Category	E
Lateral System	Special Reinforced Concrete Shear Wall
Response Modification Factor, R	5
Deflection Amplification Factor, C_d	5
Design Base Shear, V_{NS}, V_{EW}	Provided by Analysis Team
Analysis Procedures Used	DEx; Modal Response Spectrum Analysis (MRSA)

Table 1.4.2: Seismic Design Criteria

1.4.4 Seismic Response Spectra

Procedures per ASCE 7 are used to develop the seismic response spectra for the project. **Figure 1.4.1** presents the standard response spectrum from ASCE 7 at the Design Earthquake (DE) response level.

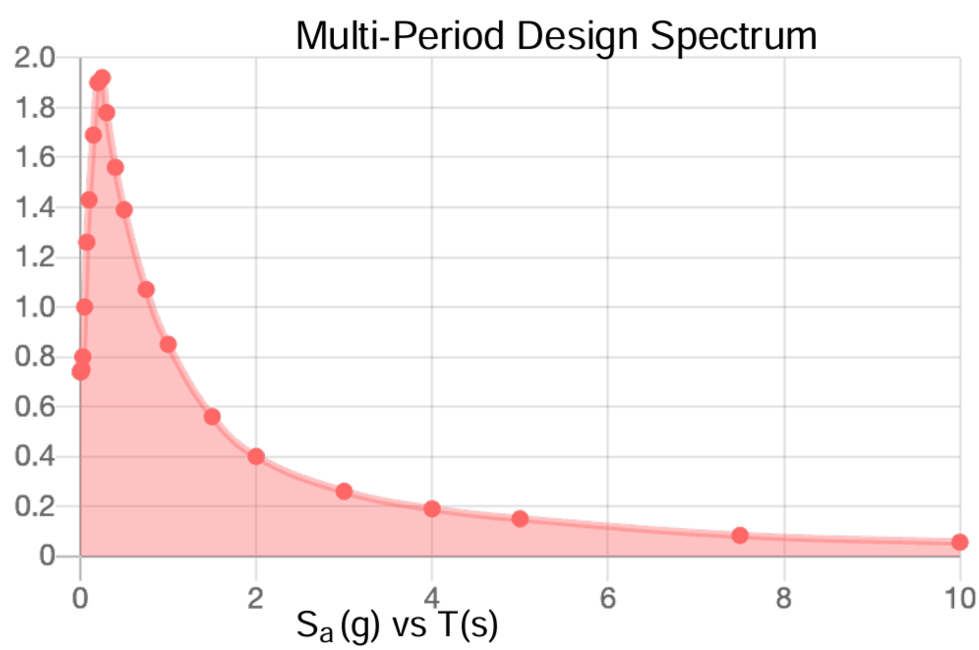


Figure 1.4.1: Multi-Perid Design Spectrum: S_a = 1.1g for T_a = 0.72s. Obtained from ASCE Hazard Tool[2].

1.4.5 Load Combinations

The Design Earthquake (DE) shaking level response is analyzed using the modal response spectrum analysis (MRSA) per ASCE 7. The DE load combinations follow the strength load combinations listed in ASCE 7. In addition to the DE load combinations listed below, the structural design and evaluation for code-specified gravity and wind lateral combinations are performed in compliance with the requirements of ASCE 7 subsection 2.3.6.

- $(1.2 + 0.2S_{DS})D + Q_E + (f_1 L + f_2 S)$
- $(0.9 - 0.2S_{DS})D + Q_E$

Where:
D = dead load
L = reduced live load
f_1 and f_2 = factor per CBC
Q_E = horizontal earthquake load (DE response spectrum)
S = snow load

Seismic directional effects are considered as follows:
$Q_{EX} = \pm 1.0E_{hx} \pm 0.3E_{hy} \pm T_{hx}$
$Q_{EY} = \pm 1.0E_{hy} \pm 0.3E_{hx} \pm T_{hy}$

1.5 Materials

The material properties used for the design include the following:

Member	**Nominal f'_c**
Mat Foundation	6.0 ksi (56 days)
Basement Walls	6.0 ksi
Reinforced Concrete Slabs and Beams**	6.0 ksi
Post–Tensioned Floor Slabs**	6.0 ksi
Columns**	6.0 ksi
Shear Walls**	7.0 ksi

Table 1.5.1: Concrete properties. ** Modulus of Elasticity shall be calculated per ACI 318–19 § 19.2.2.1[5].

Material	**Nominal f_y**
ASTM A615 Grade 60	60 ksi (non–seismic)*
ASTM A706 Grade 60	60 ksi (seismic)

Table 1.5.2: Typical for all rebar, unless specifically noted otherwise. * Non–seismic applications.

Standard	Nominal f_u	Expected f_u
0.5–inch–diameter, 7–wire strand	f_{pu} = 270 ksi	N/A

Table 1.5.3: Post–tensioned tendon properties (strand strength per manufacturer/project specs).

1.6 Analysis and Design

1.6.1 Analysis and Design Software

The computer software shown in **Table 1.6.1** is employed for the analysis and design of the structure.

Table 1.6.1: Analysis and Design Software

Structural Analysis	Computer Software
Wind and Seismic Elastic Analysis	CSI ETABS V20
Concrete Shear Wall	ACSAHE / ETABS / Custom Spreadsheet
Concrete Columns	ACSAHE

1.6.2 Overall Analysis and Design Procedure

Building design is carried out for load combinations including dead, live, and earthquake loads, where earthquake loads are based on ASCE 7 Design Earthquake loading. DE design conforms to all building code provisions except those noted in the Building Code Exceptions subsection of this report; code specified phi factors and nominal material properties are used.

1.6.3 Story Drift

The lateral force-resisting system is proportioned to satisfy code-specified drift limits using MRSA for DE loading.

1.6.4 Shear Wall Coupling Beam Design

Coupling beams are designed using the results of MRSA for DE loading demands. Concrete coupling beams are designed and detailed in accordance with ACI 318 subsection 18.10.7. Redistribution of design moments is permitted per the Building Code Exceptions.

1.6.5 Shear Wall Strength Design

The shear wall is designed to have combined moment and axial load strength in accordance with load combinations including gravity loads and DE loading strength demands. Wall shear demands are obtained from DE loading amplified for overstrength and dynamic effects in accordance with ACI 318.

1.6.6 Shear Wall Confinement

The walls are confined at the critical subsection (seismic base taken as Level 1) per ACI 318 requirements, with confinement extending into the subterranean levels to the foundation mat as above the critical subsection as required by ACI 318. Intermediate-level confinement is provided above these levels except where the vertical reinforcement ratio is less than $400/F_y$.

1.6.7 Slab-Column and Slab-Wall Connections

The slab-column connections are reinforced per ACI 318, subsection 18.14.5.1. Integrity reinforcement (slab bottom reinforcement) is provided at all mild slab-column connections per ACI 318, subsection 8.7.4.2.2, and at all post-tensioned slab-column connections per subsection 8.7.5.6.

1.6.8 Gravity Column Design

Columns will be detailed for ductility within the expected hinge zones, according to subsection 18.7.5 of ACI 318, to protect them during lateral displacements. Further, the columns will be designed in accordance with subsection ACI 318 18.14.3.3.

1.6.9 Elastic Model

Elastic models using ETABS are employed for the Design Earthquake (DE) analysis. The models include the shear walls, coupling beams, floor diaphragms, and basement walls. Diaphragms and walls are modeled using shell elements; coupling beams are modeled using frame elements. For DE analyses, structural elements are modeled using nominal material properties.

Boundary Conditions: The models include the structural elements (shear walls, coupling beams, diaphragms, and basement walls) down to the top of the mat foundation with all analytical nodes pinned at the top of the mat. The mat foundation is not modeled in ETABS but is analyzed separately in SAFE. Soil springs are not included in the elastic models.

Mass: The mass of the entire superstructure building is included in the DE model. Below-grade mass is excluded in DE analysis.

P-Delta Effects: P-Delta effects are accounted for in the analysis models.

Diaphragms: Story diaphragms are modeled as rigid; the sole exception is the podium level diaphragm, which is modelled as semi-rigid.

Modes and Modal Combination: The analysis includes enough modes to obtain a combined modal mass participation of at least 90 percent of the building mass for each principal horizontal direction of response (basement mass is not included in the determination of modal participation). Modal responses are combined using the Complete Quadratic Combination (CQC) method.

Damping: The MRSA includes 5% damping for DE analysis.

Stiffness Assumptions: The stiffness parameters for the various structural elements generally follow the recommendations in ACI 318.

1.7 Acceptance Criteria

1.7.1 DE Acceptance Criteria

- Since the elastic design for DE complies with all building code provisions except those noted in the Building Code Exceptions section of this report, the corresponding detailed acceptance criteria set forth in the referenced codes and standards apply.
- In accordance with Table 12.12-1 of ASCE 7, story drifts for the DE analysis are limited to less than 2.0 percent of story height based on Risk Category II.

LOADING CRITERIA

A. ALL LOADS NOT PROVIDED IN THE GENERAL PROJECT DESCRIPTION WERE DETERMINED FROM TABLES ON ASCE 7-22. THESE ARE AS FOLLOWS:

B. THE REST OF THE LOADS WERE SET PER TABLE 1 OF THE PROJECT DESCRIPTION:

Table 1 Gravity Loading

Use	Live Loading (psf)	Superimposed Dead Loading (psf)
Corridor/Stairs*	100	15
Lobbies	100	40
Parking	40	5
Residential	40	25
Occupied Roof**	100	75
Roof	20	15
Exterior Cladding (Precast panel system)		55 psf of wall area

C. ASSUMED THAT CORRIDOR/STAIRS LOADING APPLIES TO THE REGION WITH THE CORE AND EXTENDING 4 FT BEYOND THE CORE ON ALL SIDES.

D. ASSUMED 25% OF THE ROOF IS OCCUPIED ROOF.

E. ALL LIVE LOADS WERE REDUCED ACCORDING TO LIVE LOAD REDUCTION CODE SPECIFICATIONS FROM SECTIONS 4.7 AND 4.8 OF ASCE 7-22

MATERIAL PROPERTIES

A. THE BUILDING PROJECT IS TO BE CONSTRUCTED OF NORMAL-WEIGHT REINFORCED CONCRETE WITH THE FOLLOWING MATERIALS PROPERTIES IDENTIFIED AS MINIMUM VALUES IN TABLE 2. ALTHOUGH IT WAS ACCEPTABLE FOR THE PROJECT TEAM TO ADOPT HIGHER CONCRETE COMPRESSIVE STRENGTHS AND HIGHER GRADES OF REINFORCEMENT IF PERMITTED BY THE APPLICABLE BUILDING CODES.

Table 2. Concrete Properties

Member	Nominal f_c'
Mat Foundation	6.0 ksi (56 days)
Basement Walls	6.0 ksi
Reinforced Concrete Slabs and Beams**	6.0 ksi
Post-Tensioned Floor Slabs**	6.0 ksi
Columns**	6.0 ksi
Shear Walls**	7.0 ksi

A. REINFORCEMENT PROPERTIES WERE ALSO OUTLINED ON TABLE 3 AS FOLLOWS:

Table 3. Reinforcement Properties

Material	Nominal f_y
ASTM A615 Grade 60	60 ksi (non-seismic) *
ASTM A706 Grade 60	60 ksi (seismic)

Typical for all rebar, unless specifically noted otherwise

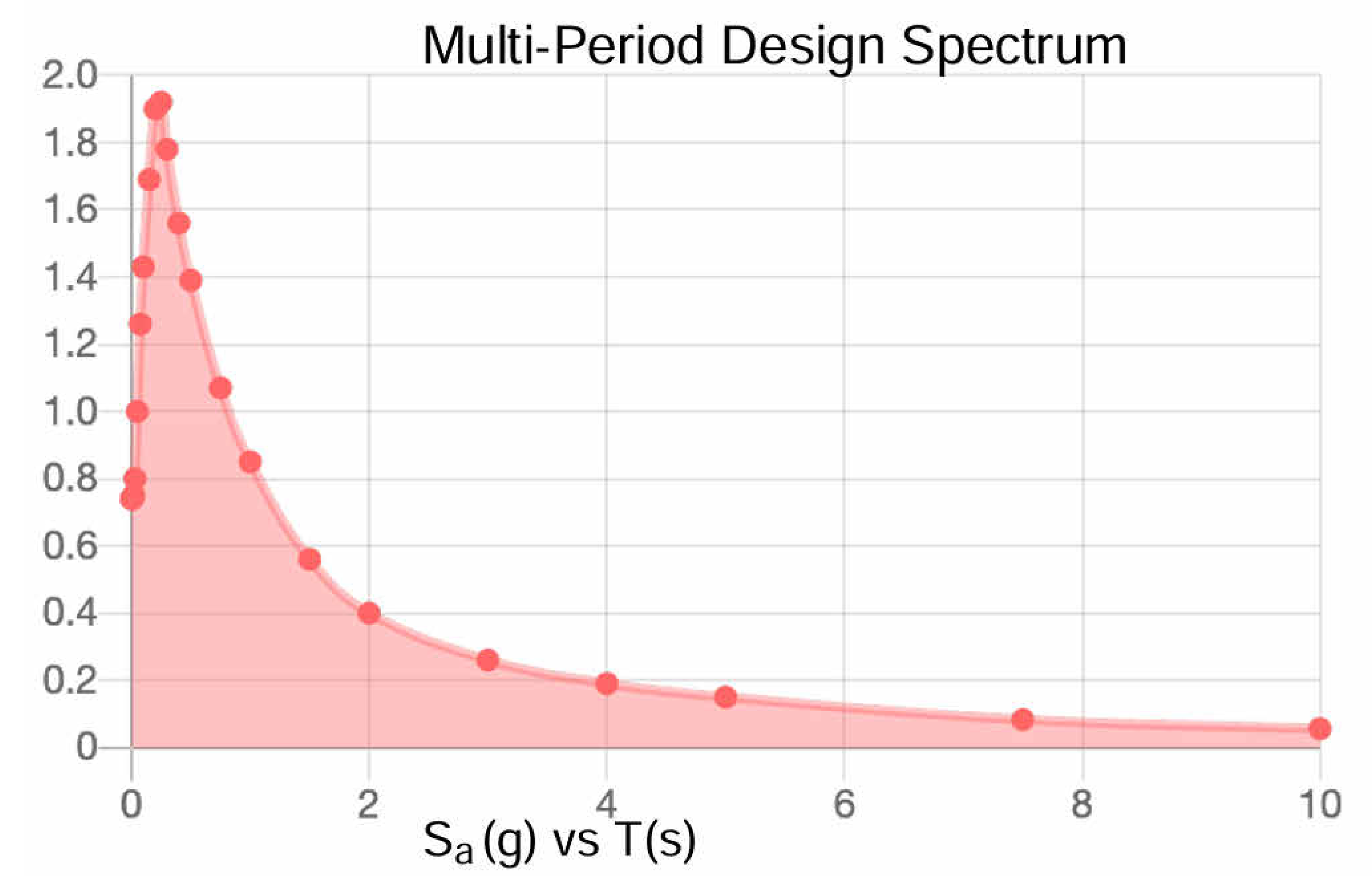
STRUCTURAL ANALYSIS

A. DESIGN EARTHQUAKE (DE) ANALYSIS WILL BE DONE USING ETABS SOFTWARE (COMPUTERS AND STRUCTURES, INC.) USING MODAL RESPONSE SPECTRUM ANALYSIS FOR DE RESPONSE SPECTRA FROM ASCE 7-22.

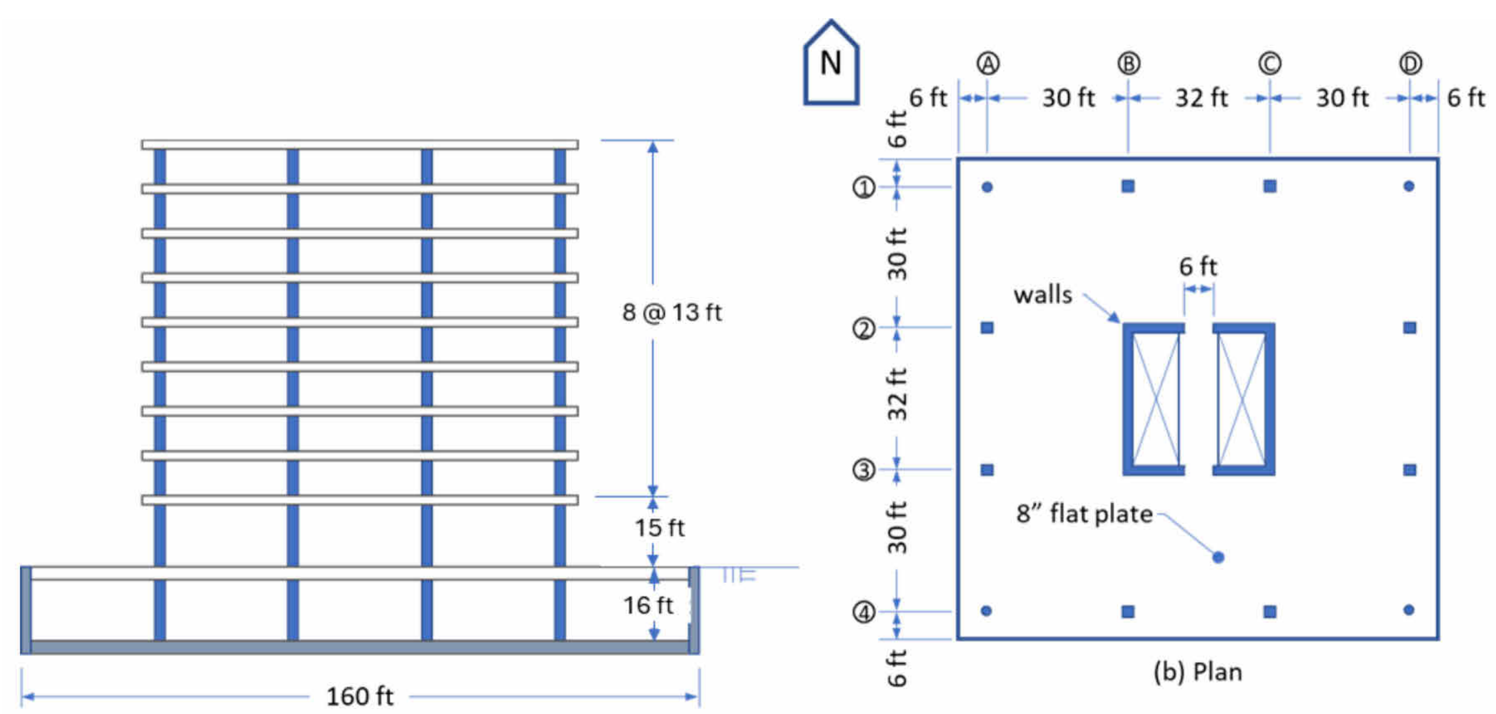
B. ANALYSES WILL BE CARRIED OUT BY AN ANALYSIS TEAM (NOT PART OF THE PROJECT TEAM), WITH KEY ANALYSIS RESULTS SUPPLIED FOR USE IN PROPORTIONING AND DETAILING STRUCTURAL ELEMENTS OF THE GRAVITY AND SEISMIC-FORCE-RESISTING SYSTEM.

DRAWING INDEX

MULTI-PERIOD DESIGN SPECTRUM



BUILDING ELEVATION AND PLAN



SEISMIC DESIGN CONSIDERATIONS

A. BUILDING LOCATION: LATITUDE 37.8730°N, LONGITUDE 122.2665°W

B. SITE CLASS: C - VERY DENSE SOIL AND SOFT ROCK

C. MAPPED ACCELERATION PARAMETER, SDS = 1.73g

D. MAPPED ACCELERATION PARAMETER, SD1 = 0.85g

E. BUILDING SEISMIC IMPORTANCE FACTOR, IE: 1.00

F. OCCUPANCY CATEGORY: III

G. BUILDING RISK CATEGORY: RISK CATEGORY II

H. SEISMIC DESIGN CATEGORY: E

I. SEISMIC DESIGN FACTORS (SPECIAL REINFORCED CONCRETE SHEAR WALL - BUILDING FRAME SYSTEM):
   1. R = 5
   2. CD = 5

J. SEISMIC WEIGHT = 18,773.99 KIPS

K. SEISMIC BASE SHEAR COEFFICIENT, CS = 0.22

L. BASE SHEAR, V = 4,130.28 KIPS

STRUCTURAL TESTING & INSPECTIONS

A. REQUIRED MATERIAL TESTING PER CBC AND ASTM STANDARDS
   1. CONCRETE
      a. SLUMP, AIR CONTENT, TEMPERATURE – ASTM C143, C231, C1064
      b. COMPRESSION CYLINDERS – ASTM C39
      c. UNIT WEIGHT – ASTM C138
   2. REINFORCEMENT STEEL
      a. TENSILE AND BEND TESTS – ASTM A615/A706
      b. MILL CERTIFICATIONS TO BE PROVIDED FOR ALL BARS
      c. VERIFY BAR SIZE, SPACING, COVER, AND GRADE ON SITE

B. SPECIAL INSPECTIONS – PER CBC CHAPTER 17
   1. CONCRETE
      a. VERIFY MIX DESIGN SUBMITTALS
      b. CONTINUOUS INSPECTION DURING PLACEMENT
      c. INSPECTION OF FORMWORK AND REINFORCEMENT PRIOR TO POUR
      d. VERIFY CONSOLIDATION, FINISHING, AND CURING METHODS
   2. REINFORCEMENT STEEL
      a. VERIFY PLACEMENT, SUPPORT, SIZE, AND SPACING
      b. TIE WIRE AND SUPPORT DETAILING TO BE VERIFIED

C. STRUCTURAL OBSERVATION REQUIRED PER CBC 1704.6
   1. OBSERVATION BY LICENSED DESIGN PROFESSIONAL
   2. REQUIRED AT THE FOLLOWING STAGES:
      a. COMPLETION OF FOUNDATION REINFORCEMENT PRIOR TO POUR
      b. PRIOR TO CONCRETE PLACEMENT FOR EACH FLOOR SLAB
      c. AT INSTALLATION OF LATERAL SYSTEM COMPONENTS
      d. FINAL STRUCTURAL SYSTEM COMPLETION REVIEW

GEOTECHNICAL REPORT & SITE CONDITIONS

A. PRELIMINARY INFORMATION FROM THE PROVIDED GEOTECHNICAL REPORT INDICATES THAT THE FOUNDATION WILL BE UNDERLAIN BY STIFF SANDY CLAY SOILS TO CONSIDERABLE DEPTH, WITH A DETERMINATION FROM THE GEOTECHNICAL ENGINEERING OF SITE CLASS C (SEE ASCE 7-22, TABLE 20.3-1)

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No.	Description	Date

The City of Berkeley Council

1950 Oxford St.
Residential Building

GENERAL NOTES

Project Number	CE 244 Semester Project
Date	December 12, 2025
Drawn By	CQ
Checked By	FP, FD, ES

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2 Part II: Summary

Shear Wall and Gravity-System Design Summary of Part III

Part III documents the design workflow for the lateral system (core walls, coupling beams, and boundary detailing) together with supporting gravity framing checks. The sequence followed throughout the chapter is: (i) establishing seismic input and preliminary wall sizing, (ii) designing and verifying the wall flexural reinforcement using interaction diagrams, (iii) designing wall shear reinforcement using amplified seismic shear demands, (iv) designing coupling beams, and (v) checking representative gravity-system actions and detailing (including columns and boundary confinement) [2, 5, 6].

A consolidated set of final element dimensions adopted in Part III is summarized at the end of this section.

2.1 Preliminary Design

The preliminary wall design is organized in three parts: determining the effective seismic weight, computing the design base shear from the design spectrum, and iteratively verifying wall thickness because the assumed thickness influences the self-weight and therefore the seismic demand [2].

2.1.1 Effective Seismic Weight, W, and Base Shear, V_b

The effective seismic weight is taken as the sum of self-weight and applicable superimposed dead loads, which is then used with the seismic response coefficient C_s to determine the base shear, $V_b = C_s W$ [2]. Because the wall thickness affects self-weight, the calculations are iterated until the thickness assumptions and resulting demands are consistent. The final iteration reported a stabilized effective seismic weight and base shear of $\mathbf{W = 18{,}773.99}$ **kips** and $\mathbf{V_b = 4{,}130.28}$ **kips**.

2.1.2 Preliminary Wall Thickness Verification

Wall thickness is verified using a first-order shear-capacity check of the form $V_u \leq 4\phi\sqrt{f'_c}\, l_w\, t_w$, rearranged to solve for t_w and repeated as W (and thus V_b) updates with the assumed thickness [5]. The converged preliminary thicknesses adopted are $\mathbf{t_{wNS} = 21}$ **in** for the N–S direction and $\mathbf{t_{wEW} = 25}$ **in** for the E–W direction.

2.2 C-Shape Flexural Design (P–M Interaction)

The wall flexural design proceeds by extracting governing design force states from analysis results at the wall base (Story 1), selecting a preliminary vertical reinforcement layout, and verifying (and refining) the design using interaction diagrams consistent with ACI 318-25 [5]. The interaction checks are generated using ACSAHE [7].

2.2.1 P-M Design Forces

	1.4D			1.2D+0.5L+(Ex±0.3Ey)/(R/IE)			1.2D+0.5L+(0.3Ex±Ey)/(R/IE)			0.9D+(Ex±0.3Ey)/(R/IE)			0.9D+(Ex±0.3Ey)/(R/IE)		
Level	P [kip]	My [kip ft]	Mx [kip ft]	P [kip]	My [kip ft]	Mx [kip ft]	P [kip]	My [kip ft]	Mx [kip ft]	P [kip]	My [kip ft]	Mx [kip ft]	P [kip]	My [kip ft]	Mx [kip ft]
Roof	-743	596	199	-1100	387	1404	-926	511	3199	-726	204	1480	-552	329	3275
Story 8	-1429	487	283	-2173	-1342	3756	-1694	-119	10757	-1604	-1437	3864	-1125	-214	10864
Story 7	-2117	497	326	-3426	-3308	6787	-2516	-694	21282	-2662	-3420	6911	-1753	-807	21407
Story 6	-2805	499	353	-4838	-5404	10402	-3386	-1325	34075	-3881	-5515	10537	-2429	-1436	34209
Story 5	-3492	505	368	-6387	-7556	14517	-4296	-1968	48788	-5235	-7670	14657	-3145	-2082	48928
Story 4	-4177	508	376	-8047	-9891	19045	-5239	-2670	65284	-6702	-10005	19189	-3895	-2784	65427
Story 3	-4861	529	382	-9788	-12616	23861	-6205	-3476	83448	-8251	-12737	24006	-4668	-3597	83593
Story 2	-5543	462	385	-11550	-16421	28745	-7177	-4679	103165	-9821	-16504	28891	-5448	-4762	103311
Story 1	-6283	654	387	-13218	-25422	33801	-8154	-7212	127330	-11285	-25635	33948	-6221	-7425	127478
Subfloor	-7193	442	265	-14189	-1574	24586	-9048	-179	89938	-11981	-1712	24687	-6839	-317	90039

	1.2D+1.6L			1.2D+0.5L+(-Ex±0.3Ey)/(R/IE)			1.2D+0.5L+(-0.3Ex±Ey)/(R/IE)			0.9D+(-Ex±0.3Ey)/(R/IE)			0.9D+(-0.3Ex±Ey)/(R/IE)		
Level	P [kip]	My [kip ft]	Mx [kip ft]	P [kip]	My [kip ft]	Mx [kip ft]	P [kip]	My [kip ft]	Mx [kip ft]	P [kip]	My [kip ft]	Mx [kip ft]	P [kip]	My [kip ft]	Mx [kip ft]
Roof	-1323	686	-208	-603	744	507	-777	618	2931	-229	561	583	-404	436	3006
Story 8	-2065	387	-296	-803	2156	2706	-1283	930	10441	-234	2061	2813	-714	835	10549
Story 7	-2806	444	-342	-825	4169	6031	-1736	1548	21056	-61	4057	6156	-973	1436	21180
Story 6	-3546	443	-370	-685	6265	10145	-2141	2175	33998	273	6153	10279	-1183	2063	34132
Story 5	-4283	451	-385	-408	8427	14921	-2503	2825	48909	744	8313	15061	-1352	2712	49049
Story 4	-5019	451	-394	-16	10764	20375	-2830	3525	65682	1329	10651	20518	-1486	3412	65826
Story 3	-5752	479	-400	460	13529	26583	-3132	4366	84264	1997	13408	26728	-1595	4245	84410
Story 2	-6483	345	-403	961	17168	33732	-3424	5396	104660	2690	17085	33878	-1695	5313	104807
Story 1	-7262	795	-405	1269	26670	43606	-3809	8412	130271	3201	26456	43753	-1876	8199	130419
Subfloor	-8301	516	-278	519	2415	29946	-4636	1017	91546	2727	2278	30047	-2427	880	91647

Figure 2.2.1: C-shaped core-wall segment loads considered for flexural design and sign convention for internal forces provided by analysis team.

The governing factored design actions used to design the flanges of the section are:

$$M_u = 130{,}419 \text{ k-ft}, \qquad P_u = 1{,}876 \text{ k.Compression}$$

The governing factored design actions used to design the web of the section are:

$$M_u = 26{,}670 \text{ k-ft}, \qquad P_u = -1{,}269 \text{ k.Tension}$$

2.2.2 Preliminary Longitudinal Reinforcement

A preliminary design based on worst case loading of the section and estimated compressive zones is performed.

Moment equilibrium requires that the tension force satisfies:

$$T_s = \frac{M_n - P_n(x_e)}{x_s}$$

So that the total steel area needed based on the tension force and the steel yield strength (f_y = 60 ksi) is:

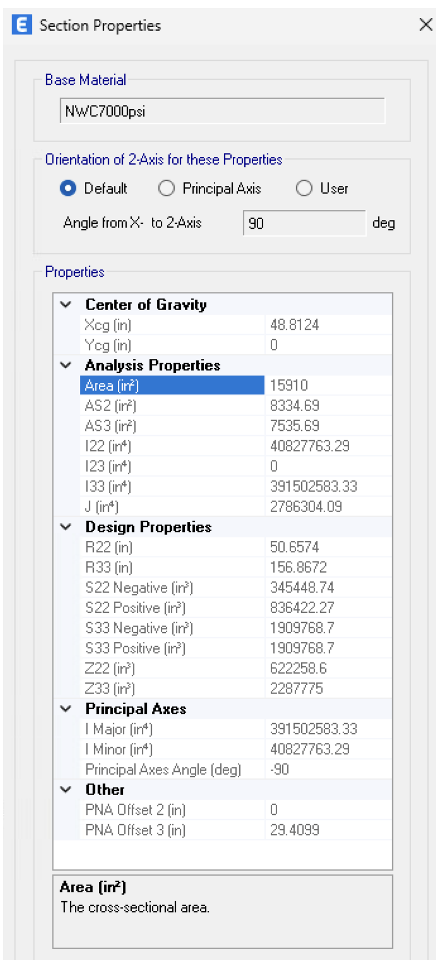
$$A_s = \frac{T_s}{f_y}$$

A trial layout of #10 vertical bars at 6 in. spacing is selected for the flanges and #8 vertical bars at 6 in. spacing for the web as an initial configuration for the core wall [5, 8]. For a full breakdown of calculations and values utilized see Section 3.2.2

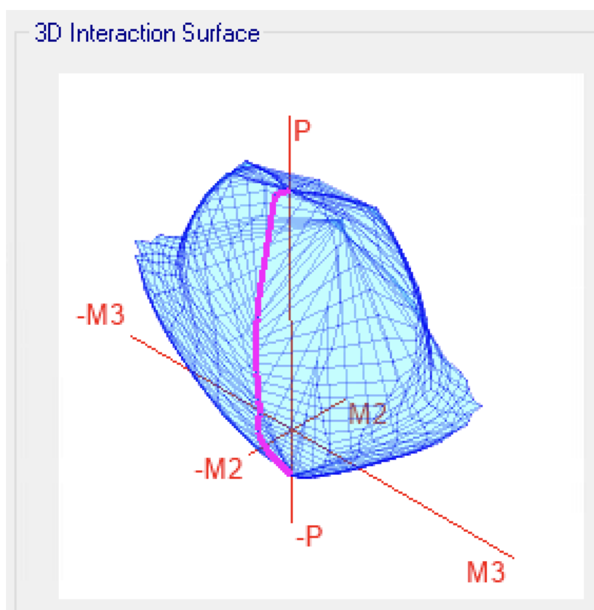
2.2.3 Verification with P–M Interaction Diagrams

The preliminary layout is verified using ETABS and ACSAHE to perform the wall section capacity in axial–flexural interaction (and corresponding P–M–M visualization), with iteration toward a final detailing arrangement that satisfies the governing combinations [5, 7].

Cross Section and Properties of Preliminary Design:



(a) Section properties of preliminary ETABS model.



(b) 3D view of preliminary design in ETABS.

Figure 2.2.2: Preliminary ETABS model: section properties and 3D view.

Looking closer at the principle planes:

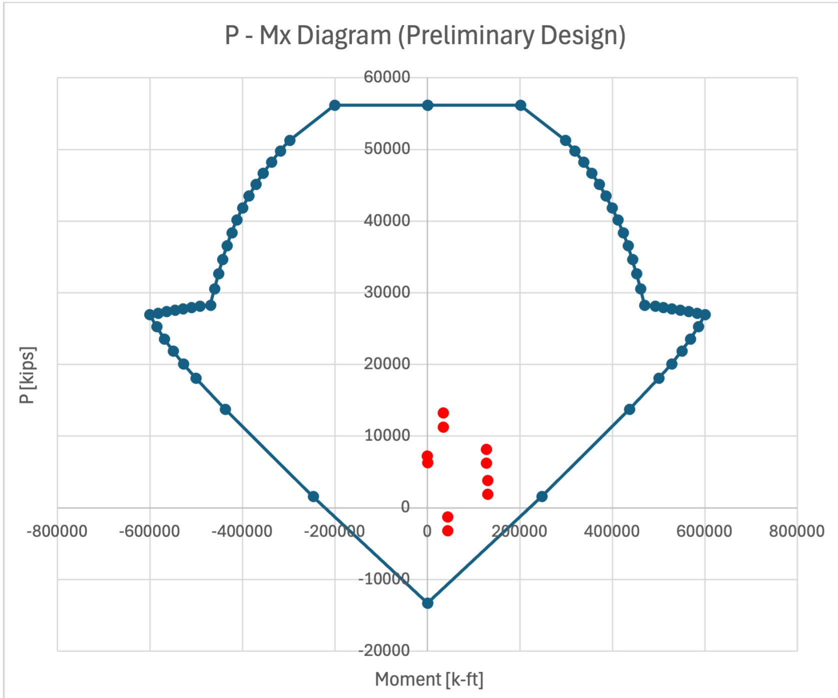


Figure 2.2.3: 0°-180° PM Diagram for core wall section with #10 Bars in the E-W Direction and #8 bars in the N-S Direction. Note Red points represent demands for different load combinations at base story

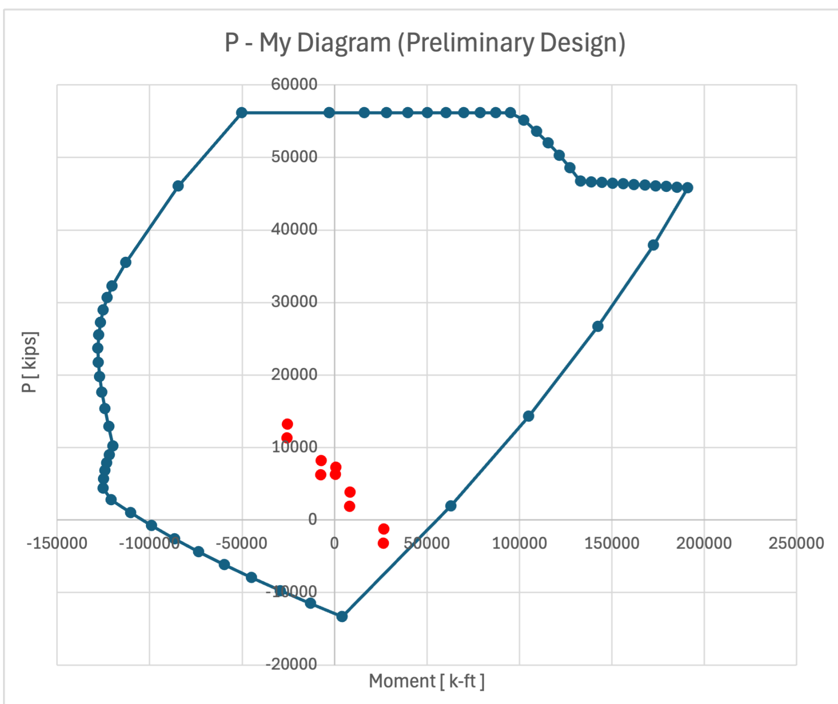


Figure 2.2.4: 90°-270° PM Diagram for core wall section with #10 Bars in the E-W Direction and #8 bars in the N-S Direction. Note Red points represent demands for different load combinations at base story

After analyzing the P-M Diagram for the preliminary section, the reinforcement design can be iterated to produce a more efficient design.

2.2.4 Final Core Wall Longitudinal Reinforcement Design

After several iterations on the reinforcement layout using both ETABS and ACSAHE, the final design for the C-shaped core wall section consists of:

(i) No. 7 reinforcement bars spanning in the y-direction (web reinforcement).

(ii) No. 9 reinforcement bars placed in the flanges, spanning in the x-direction.

2.2.5 ACSAHE: Final Interactive P-M-M-Diagram

The biaxial interaction diagram of the concrete cross section can be found in below, and is available online at: `https://facundo-pfeffer.github.io/ACSAHE.github.io/pages/examples/c_shape.html`. The online visualizations allows the user to interact with the diagram and see the full response spectrum. The full report is available in Section B.

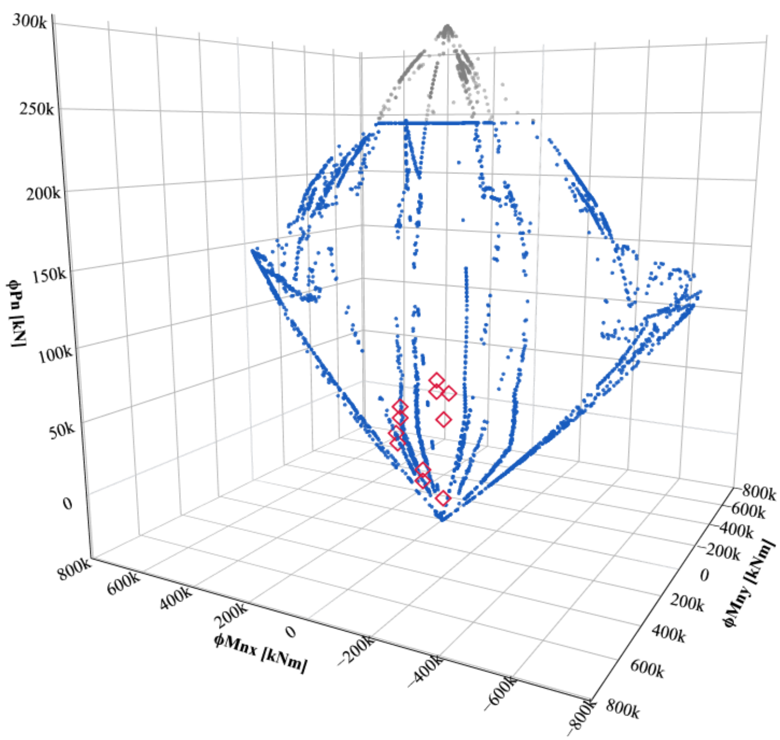
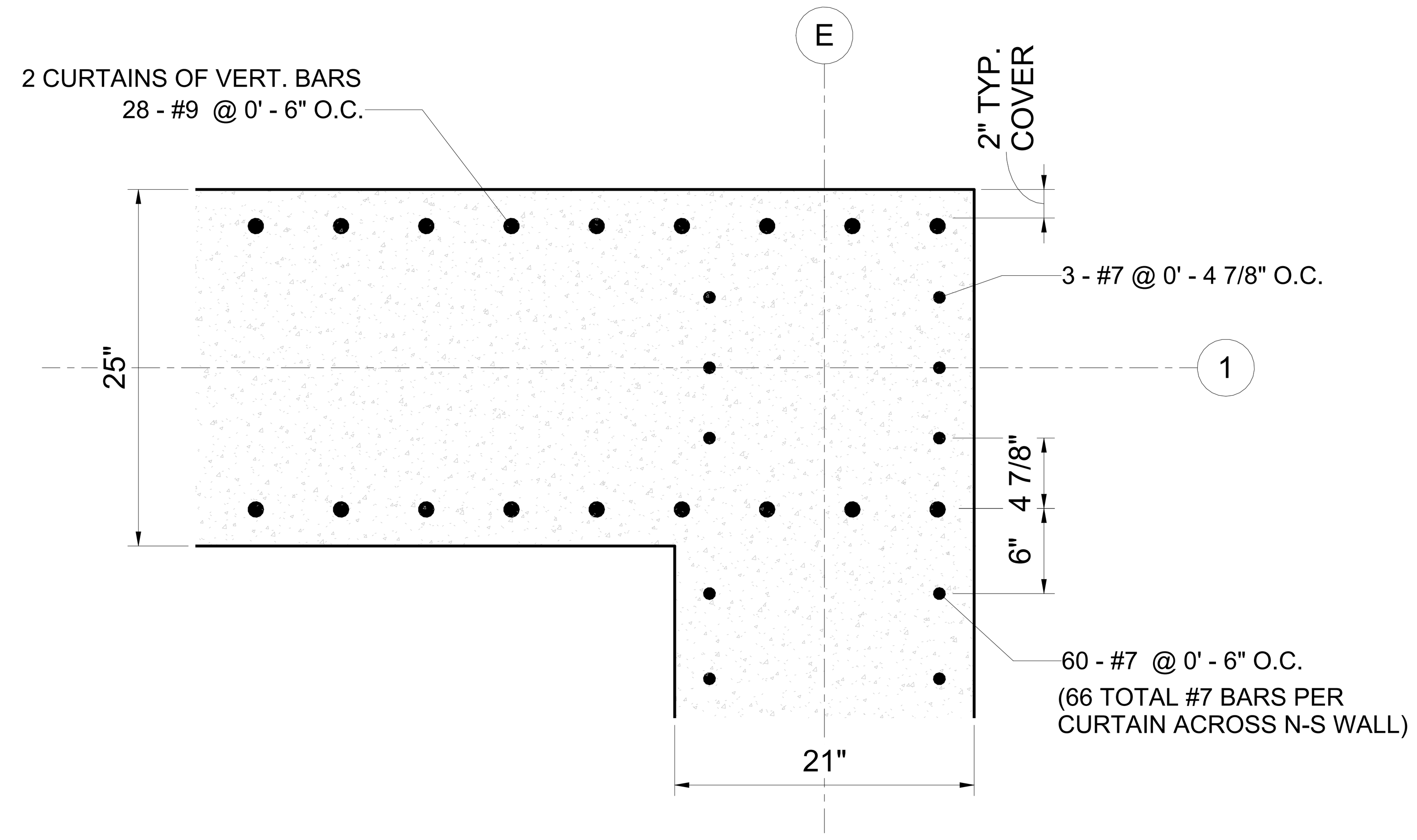
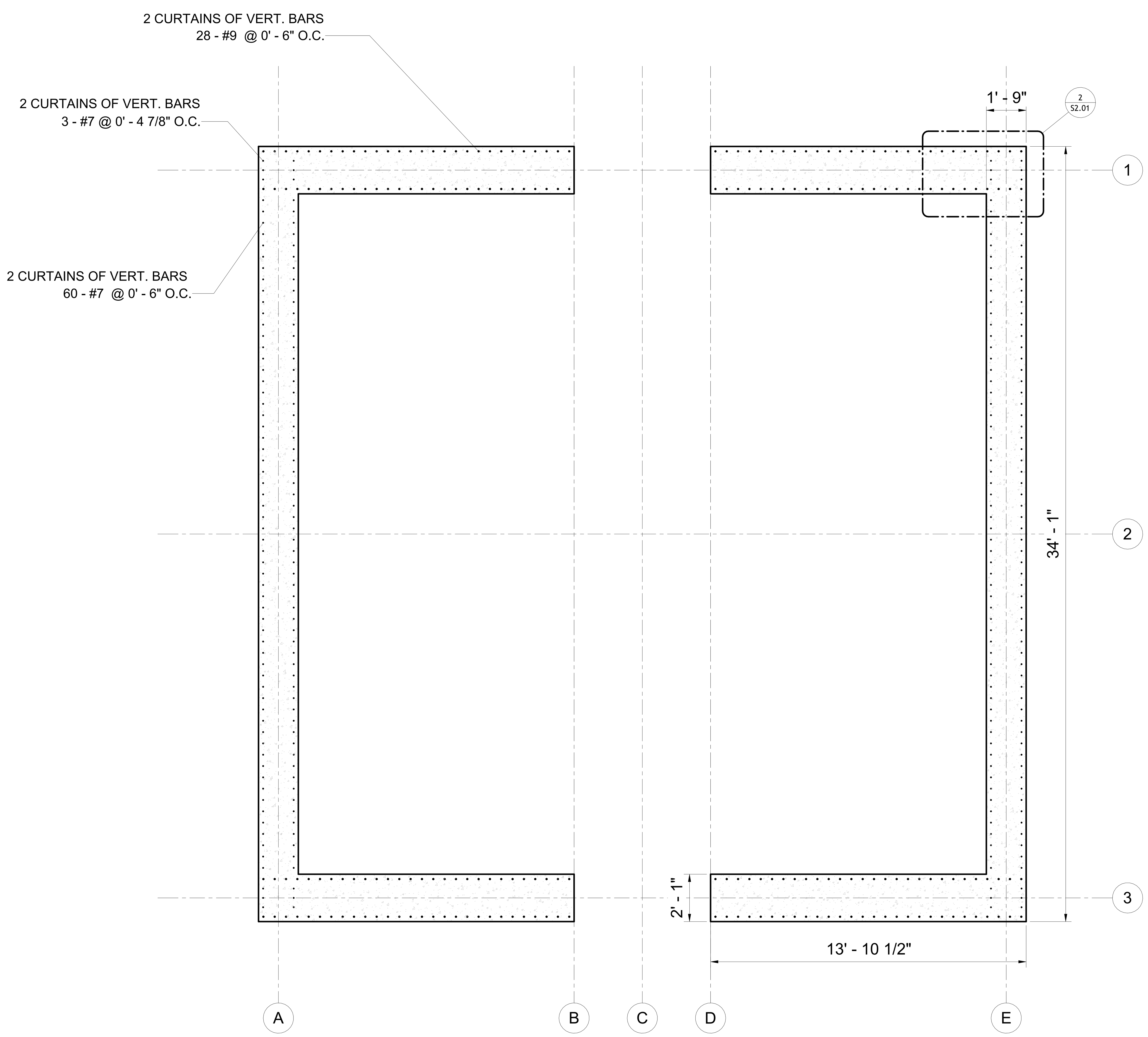


Figure 2.2.5: Final P-M-M diagram. Units are expressed in SI values. No. 9 bars in the Flanges and No. 7 bars in the web. Available online for visualization at this link.

Refer to structural sheet **S2.01** for the final detailing drawings of the core wall longitudinal reinforcement.



2 Core Wall Corner Detail
1 1/2" = 1'-0"



1 Core Wall Section Detail - Plan
3/8" = 1'-0"

N

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No.	Description	Date

The City of Berkeley Council

1950 Oxford St.
Residential Building

FINAL LONGITUDINAL
REINF. DETAIL

Project Number	CE 244 Semester Project
Date	December 12, 2025
Drawn By	CQ
Checked By	FP, FD, ES

S2.01

Scale: As indicated

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2.3 Wall Transverse Shear Reinforcement Design

The shear design sequence is: (i) computing amplified seismic shear demands per ACI 318-25, (ii) checking concrete shear limits to confirm adequacy of wall dimensions, and (iii) determining the required transverse reinforcement ratio using the nominal shear-strength model for special structural walls [5, 9, 1].

2.3.1 Wall Shear Design Forces

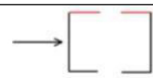
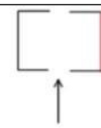
				
	DL (+ LL) ± Ex ± 0.3Ey		DL (+ LL) ± 0.3Ex ± Ey	
	Vx (kips)	Vy (kips)	Vx (kips)	Vy (kips)
Roof	439	162	160	372
Story 8	698	354	228	799
Story 7	946	482	303	1093
Story 6	1129	580	357	1321
Story 5	1283	663	402	1512
Story 4	1419	739	442	1688
Story 3	1541	812	478	1848
Story 2	1634	884	502	1992
Story 1	1700	932	524	2076
Subfloor	1572	676	431	2567

Figure 2.3.1: Shear demands provided by analysis team

2.3.2 Amplified Shear Demands

For special structural walls, the amplified design shear is taken as $V_e = \Omega_v \omega_v V_u \leq 3V_u$, where Ω_v accounts for flexural overstrength and ω_v accounts for higher-mode amplification [5]. For the configuration considered, the amplification parameters are computed and reported.

For walls with $h_{wcs}/\ell_w \geq 2.0$:

$$\Omega_v = 1.5, \qquad \omega_v = 0.8 + 0.09\, h_n^{1/3} \geq 1.0,$$

where h_w is the total height of wall above the critical section and h_n is the building height in feet (or the equivalent number of stories) used in the dynamic amplification expression.

For the building in this assignment, the height parameter $h_n = 119$ ft from the project data gives:

$$\omega_v = 0.8 + 0.09\, h_n^{1/3} \approx 1.24 \geq 1.0,$$

The total amplified shear for one wall in the N-S direction is taken as

$$V_{e,y} = \Omega_v\, \omega_v\, V_y = (1.5)(1.24)(2076) = 3,869.71 \text{ kips}.$$

The total amplified shear for one wall in the E-W direction is taken as

$$V_{e,x} = \Omega_v\, \omega_v\, V_x = (1.5)(1.24)(1700) = 3,169\,\text{kip}$$

2.3.3 Required Wall Thickness

The required wall thickness per code for each direction is verified against the assumed wall thickness from the preliminary design. For an individual wall segment, the maximum permitted design shear strength is taken as:

$$\phi V_{n,\mathrm{max,seg}} = \phi\, \alpha_{sh}\, 10\sqrt{f'_c}\, A_{cv,\mathrm{seg}},$$

and for the entire wall system in a given direction:

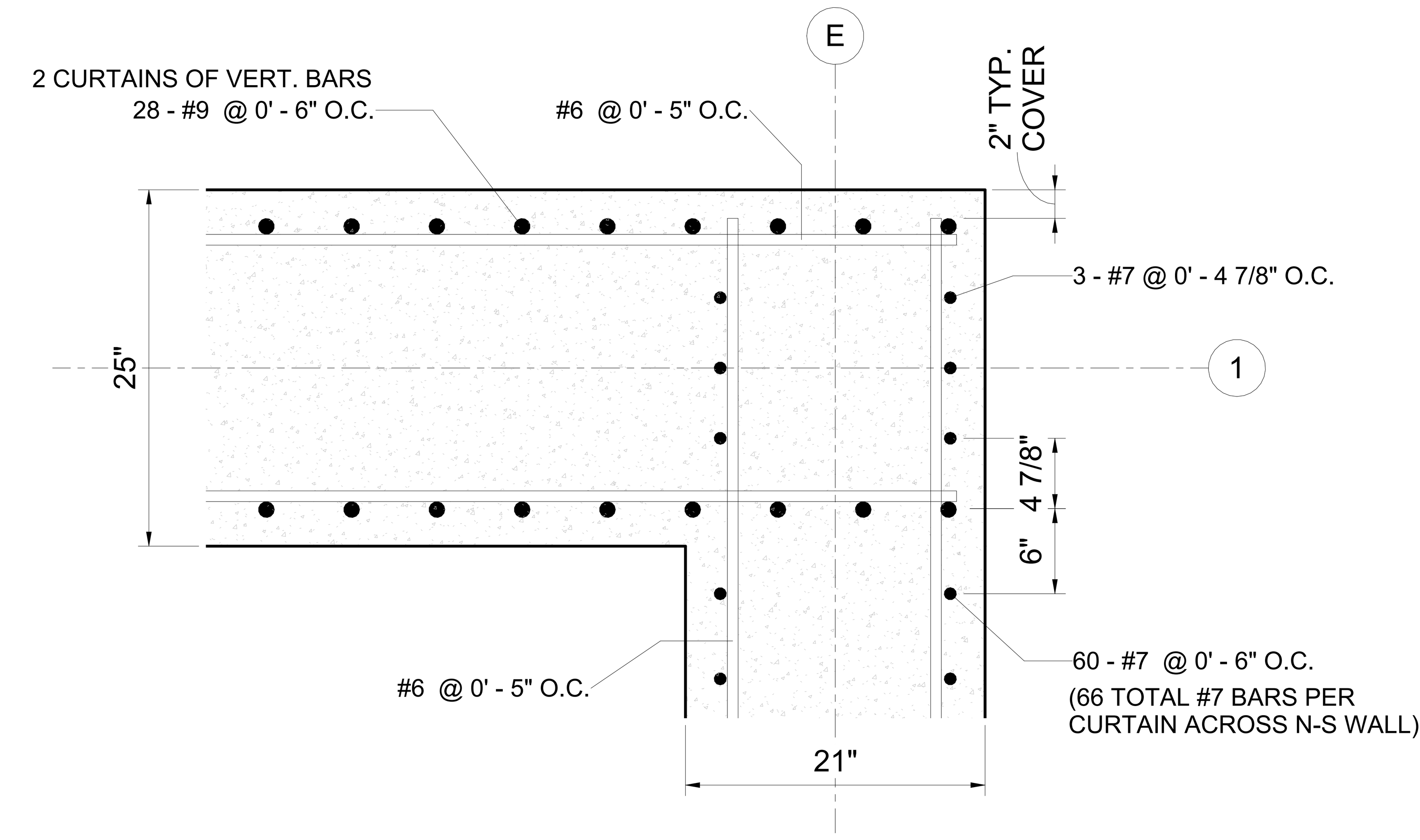
$$\phi V_{n,\mathrm{max,tot}} = \phi\, \alpha_{sh}\, 8\sqrt{f'_c}\, A_{cv,\mathrm{tot}}.$$

In each case, the limit is verified by checking that $\phi V_{n,\mathrm{max}} \geq V_u$.

In all cases, the initially assumed wall thickness of 21 inches in the N-S direction and 25 inches in the E-W direction are shown to be sufficient. For full calculations of procedure see Section 3.3.2

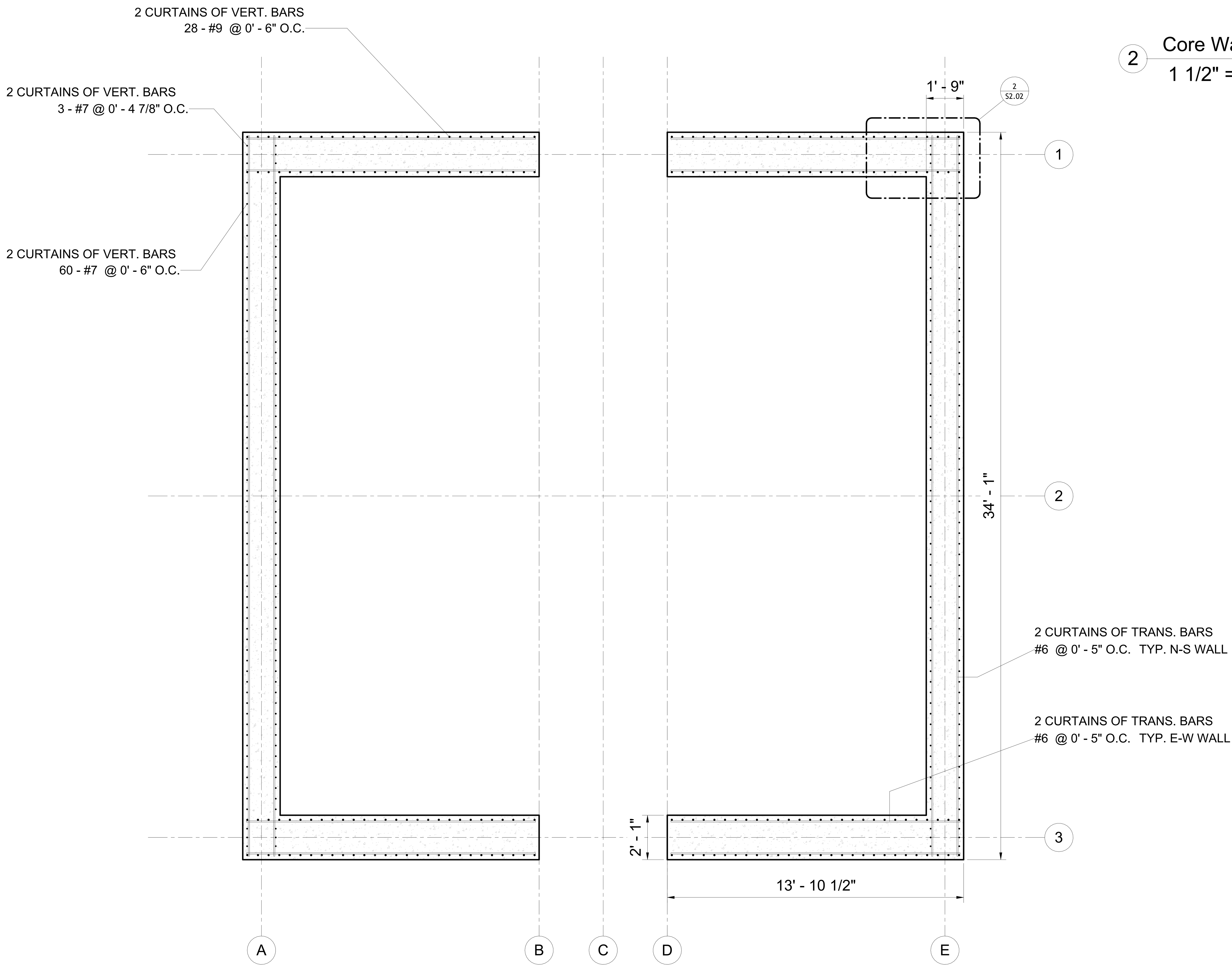
2.3.4 Transverse Reinforcement Final Design

The nominal shear strength model is expressed as $V_n = \left(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_{yt}\right) A_{cv}$ with the design condition $\phi V_n \geq V_u$ and $\phi = 0.75$ [5]. Solving for the required transverse ratio provides $\rho_{t,\mathrm{req}}$; for example, for the E–W wall the effective shear area is taken as $A_{cv} = t_w\, l_{w,\mathrm{eff}} = (25)(2)(166.5) = 8325\ in^2$, leading to $\rho_{t,\mathrm{req}} \approx 0.00567$, governed by shear demand rather than the code minimum. A reinforcement of No. 6 bars at 5 inches on center is selected for all sections of the wall. Refer to structural sheet **S2.02** for the final detail of the core wall shear reinforcement.



2 Core Wall Corner Detail

1 1/2" = 1'-0"



1 Core Wall Section Detail - Plan

3/8" = 1'-0"

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No.	Description	Date

The City of Berkeley Council

1950 Oxford St.
Residential Building

FINAL TRANSVERSE
(SHEAR) REINF. DETAIL

Project Number	CE 244 Semester Project
Date	December 12, 2025
Drawn By	CQ
Checked By	FP, FD, ES

S2.02

Scale: As indicated

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2.4 Core Wall Confinement

Special Boundary Element (SBE) evaluation is documented using the permitted ACI procedures (stress-based and displacement-based), followed by confinement sizing and detailing where required [5, 10, 11]. For the reported wall configuration, the displacement-based check triggers SBEs for the flange-compression case, while other cases do not require SBEs under the stated checks [5, 10]. Boundary confinement is then summarized for right flange, left flange, and web regions in the final design tables. Refer to Section 3.7 for full details in the calculations.

2.4.1 SBE Verification: Criteria, Governing Combinations, and Boundary Locations

Overview and adopted workflow. The need for special boundary elements (SBEs) is evaluated using either of two alternative procedures permitted by ACI 318-25 [5]: (i) the displacement-based criterion of § 18.10.6.2, and (ii) the stress-based (fiber-stress) criterion of § 18.10.6.3. In this project, the stress-based check is reported first as a direct screening of boundary compression demand under factored gravity and seismic actions. When the stress-based check does not trigger confinement, the displacement-based criterion is also evaluated as a secondary verification.

Stress-based (fiber-stress) criterion: ACI 318-25 § 18.10.6.3. At the wall critical section, factored axial load and factored uniaxial bending moments from the governing earthquake combinations are used to compute extreme-fiber compressive stresses assuming linear-elastic response. Special boundary confinement is required wherever the calculated compressive stress exceeds:

$$\sigma_C > 0.20\, f'_c = 1.4 \text{ ksi}.$$

The stresses are evaluated independently for bending about the $x-x$ and $y-y$ axes:

$$\sigma_{x,C} = \frac{P_u}{A_g} + \frac{M_{u,x}}{I_{x-x}}\, y_{\max} \quad < \quad 0.20\, f'_c \tag{2.4.1}$$

$$\sigma_{y,C} = \frac{P_u}{A_g} + \frac{M_{u,y}}{I_{y-y}}\, x_{\max,C} \quad < \quad 0.20\, f'_c \tag{2.4.2}$$

where P_u is taken as negative in compression. The distance $x_{\max,C}$ is defined at the extreme compression fiber for the sign of $M_{u,y}$; because the C-shaped wall is not symmetric about the $y-y$ axis, the compression fiber (and thus $x_{\max,C}$) changes depending on whether the web or the flanges are in compression.

Displacement-based criterion: ACI 318-25 § 18.10.6.2. When the computed compressive stresses from **Eq. (3.7.1)** and **Eq. (3.7.2)** remain below $0.20 f'_c$, the displacement-based criterion is evaluated as:

$$1.50\frac{\delta_u}{h_{wcs}} \geq \frac{l_w}{600c} \tag{2.4.3}$$

where δ_u is the design displacement (from **Table 3.5.1**), h_{wcs} is the height of the wall critical section, l_w is the wall length in the direction under consideration, and c is the neutral-axis depth measured from the extreme compression fiber.

In this work, c is estimated using the empirical expression proposed by Abdullah and Wallace [10]:

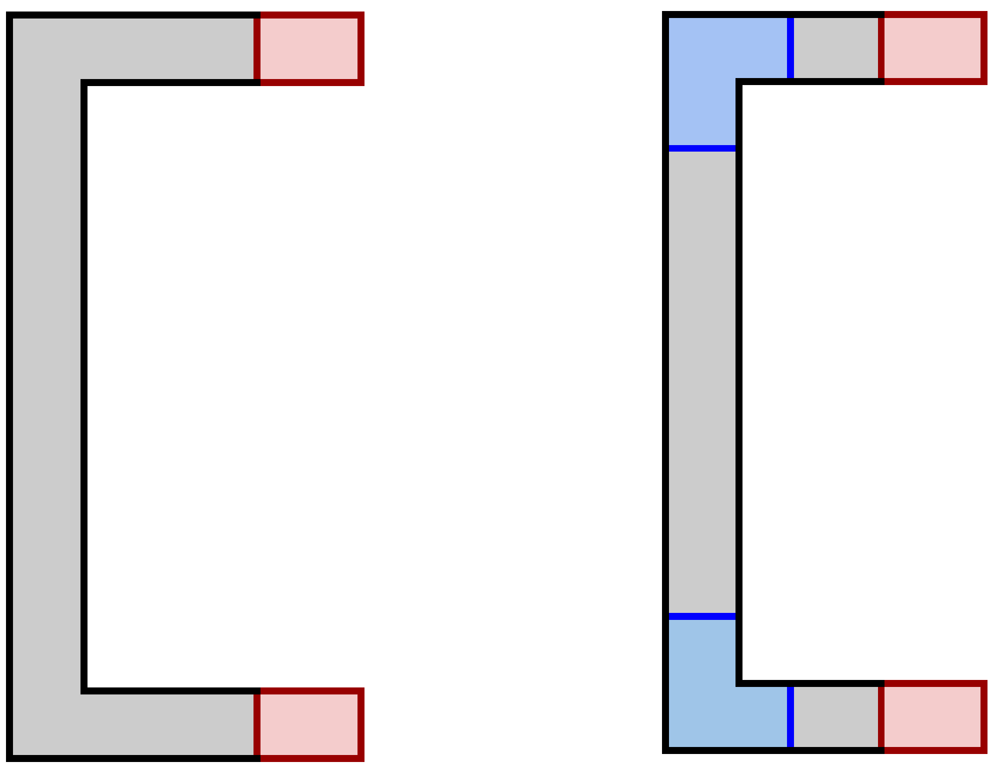
$$\frac{c}{l_w} = k_1 + k_2\frac{-P_u}{A_g f'_c}, \tag{2.4.4}$$

with (k_1, k_2) selected based on the bending axis and the compression region. Two M_y cases are therefore evaluated (web in compression and flanges in compression), each with its corresponding (k_1, k_2) pair.

Governing load combinations used in the checks. Only factored load combinations that include earthquake effects in the horizontal direction under consideration are used for the SBE verification. The ETABS combinations reported in the subsequent tables include orthogonal directional effects through the $\pm 1.0E \pm 0.3E$ combination rule, as shown explicitly in the table combination labels (for example, $(0.3E_x + E_y)$ and $(E_x + 0.3E_y)$). The complete audit trail used in the screening and displacement verification is reported in:

- **Table 3.7.1** for M_x,
- **Table 3.7.2** for M_y with flanges in compression, and
- **Table 3.7.3** for M_y with web in compression.

Boundary regions requiring SBEs and adopted detailing footprint. The verification results indicate that the displacement-based criterion governs the SBE trigger for the critical M_y compression case, resulting in SBEs being required at the flange boundary regions, as highlighted in Fig. 2.4.1a. For constructability and to provide consistent confinement and ductility around the perimeter, symmetric boundary-element detailing is adopted at all wall edges, as shown in Fig. 2.4.1b. The SBE detailing designed for the required flange regions is therefore applied to all boundary locations indicated in Fig. 2.4.1b, after completing the corresponding confinement sizing checks in Section 3.7.3.



(a) (In red) highlighted zones that require SBEs per the verification checks.

(b) (In red and blue) zones where boundary elements are detailed by design choice.

Figure 2.4.1: Special Boundary Element zones

2.4.2 SBEs Design

Having verified the need for SBEs in Section 3.7.1, this section presents the design of the confined boundary regions in accordance with ACI 318-25, Section 18.10.6 [5].

The maximum vertical spacing of transverse reinforcement along the height of the wall, s, is limited by **Eq. (3.7.5)**:

$$s = \min \begin{cases} b/3, \\ 6d_b, \\ s_0 \end{cases} \qquad 4\,\text{in} \le s_0 = 4 + \frac{14 - h_x}{3} \le 6\,\text{in}, \tag{2.4.5}$$

where b is the boundary element width, d_b is the diameter of the longitudinal bars, and h_x is the clear horizontal distance between the outermost longitudinal bars in the boundary element.

The confined region at each boundary is required to extend a distance l_{be} from the extreme compression fiber such that:

$$l_{be} = \max \begin{cases} c/2, \\ c - 0.10\,l_w \end{cases} \tag{2.4.6}$$

The transverse reinforcement area A_{sh} provided in each tie leg is required to satisfy the larger of the two ACI 318-25 expressions in **Eq. (3.7.7)**:

$$A_{sh} \ge \begin{cases} 0.09\,s\,b_c\,\dfrac{f_c'}{f_{yt}}, \\ 0.3\,s\,b_c\left(\dfrac{A_g}{A_{ch}} - 1\right)\dfrac{f_c'}{f_{yt}}, \end{cases} \tag{2.4.7}$$

where b_c is the core dimension measured center-to-center of the outer transverse reinforcement, A_{ch} is the area of the confined concrete core measured to the outside of the transverse reinforcement, and f_{yt} is the specified yield strength of the transverse reinforcement.

The design procedure for the right flange in **Fig. 3.7.1b** is summarized as:

1. Selection of an admissible vertical spacing s using **Eq. (3.7.5)**.
2. Estimation of c using **Eq. (3.7.4)**, followed by the required confinement length l_{be} from **Eq. (3.7.6)**.
3. Definition of the confined region using the resulting l_{be}, determination of A_{ch} and b_c, and evaluation of the required transverse reinforcement A_{sh} from **Eq. (3.7.7)** for both the long and short boundary components.
4. Verification of detailing requirements (bar sizes, number of legs, hoop configuration, and overlap conditions), with adjustments as needed.

The final SBE design is summarized separately for the flange boundary regions (**Table 3.7.4**) and for the web boundary region (**Table 3.7.5**).

2.4.3 Final Design of Core Wall Confinement

The final special boundary element (SBE) detailing is summarized in **Table 2.4.1** for the flange boundary regions (right and left flanges) and for the web boundary region. In all cases, the adopted confinement detailing uses #5 transverse reinforcement at a uniform vertical spacing of $s = 5$ in. The governing confinement length is $l_{be} = 62.5$ in for the flanges and $l_{be} = 59.5$ in for the web. Required tie area per transverse leg is satisfied by #5 bars for both the long and short core dimensions, with 11 legs along the long direction and 5 legs along the short direction in the flange regions, and 11 legs (long) with 4 legs (short) in the web region. The right flange governs the flange confinement demand ($c = 72.94$ in), while the left flange is detailed to match the same confinement length and transverse layout for constructability ($c = 18.87$ in).

Region	Item	Value	Comment
Flanges	Adopted s [in.]	5.00	Same for right/left flange.
	Adopted transverse bar	#5	Used for both directions.
	Adopted l_{be} [in.]	62.50	= $\max(l_{be,\mathrm{min}})$ (right flange); matched on left.
	b_{c1} (long) [in.]	61.00	Core dimension (center-to-center).
	b_{c2} (short) [in.]	22.00	Core dimension (center-to-center).
	# legs across b_{c1}	11	Long-direction transverse legs.
	$A_{sh,\mathrm{req}}$ per leg across b_{c1} [in^2]	0.291	#5 OK ($\max\{3.20, 1.75\}/11$).
	# legs across b_{c2}	5	Short-direction transverse legs.
	$A_{sh,\mathrm{req}}$ per leg across b_{c2} [in^2]	0.231	#5 OK ($\max\{1.16, 0.63\}/5$).
Web	Adopted s [in.]	5.00	Governing spacing.
	Adopted transverse bar	#5	Used for both directions.
	Adopted l_{be} [in.]	59.50	$\geq \max(39.53, 40.21)$.
	b_{c1} (long) [in.]	58.00	Core dimension (center-to-center).
	b_{c2} (short) [in.]	18.00	Core dimension (center-to-center).
	# legs across b_{c1}	11	Long-direction transverse legs.
	$A_{sh,\mathrm{req}}$ per leg across b_{c1} [in^2]	0.277	#5 OK ($\max\{3.05, 2.00\}/11$).
	# legs across b_{c2}	4	Short-direction transverse legs.
	$A_{sh,\mathrm{req}}$ per leg across b_{c2} [in^2]	0.236	#5 OK ($\max\{0.95, 0.62\}/4$).

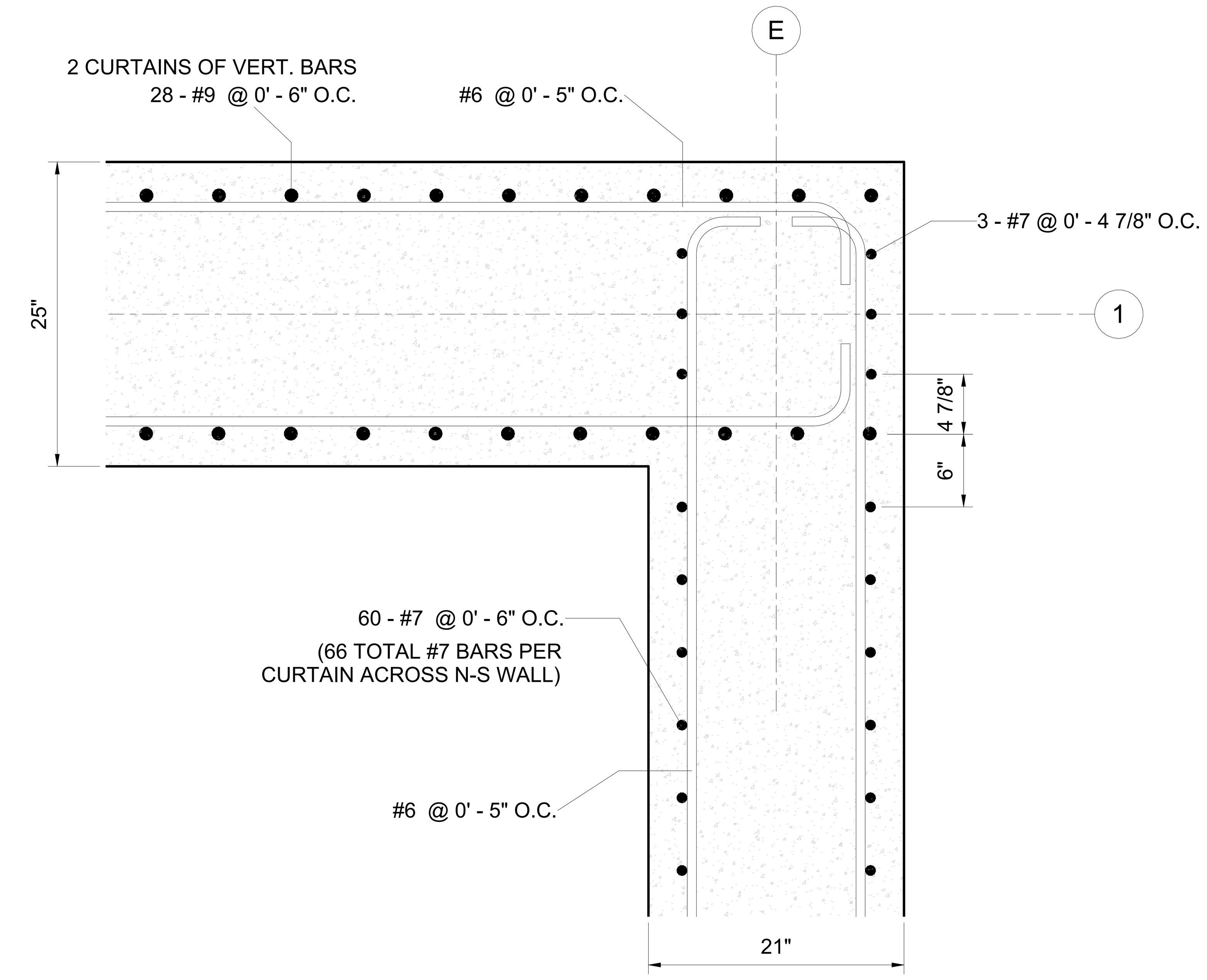
Table 2.4.1: Summary of final SBE confinement dimensions and adopted transverse reinforcement detailing for flange and web boundary regions (See **Table 3.7.4** and **Table 3.7.5**) for full calculation details.

Please note that additional crossties were added outside of the boundary elements throughout the rest of the core wall (in the gray areas of **Fig. 2.4.1b**) to prevent vertical bar buckling. These are not meant to be for confinement and are not part of the SBEs. Thus, we have chosen a No. 4 crosstie for constructability and efficiency.

Refer to structural sheet **S2.03** for the final detail drawing of the core wall including SBEs. Further, see sheet **S2.04** for a close-up detail of each boundary element.

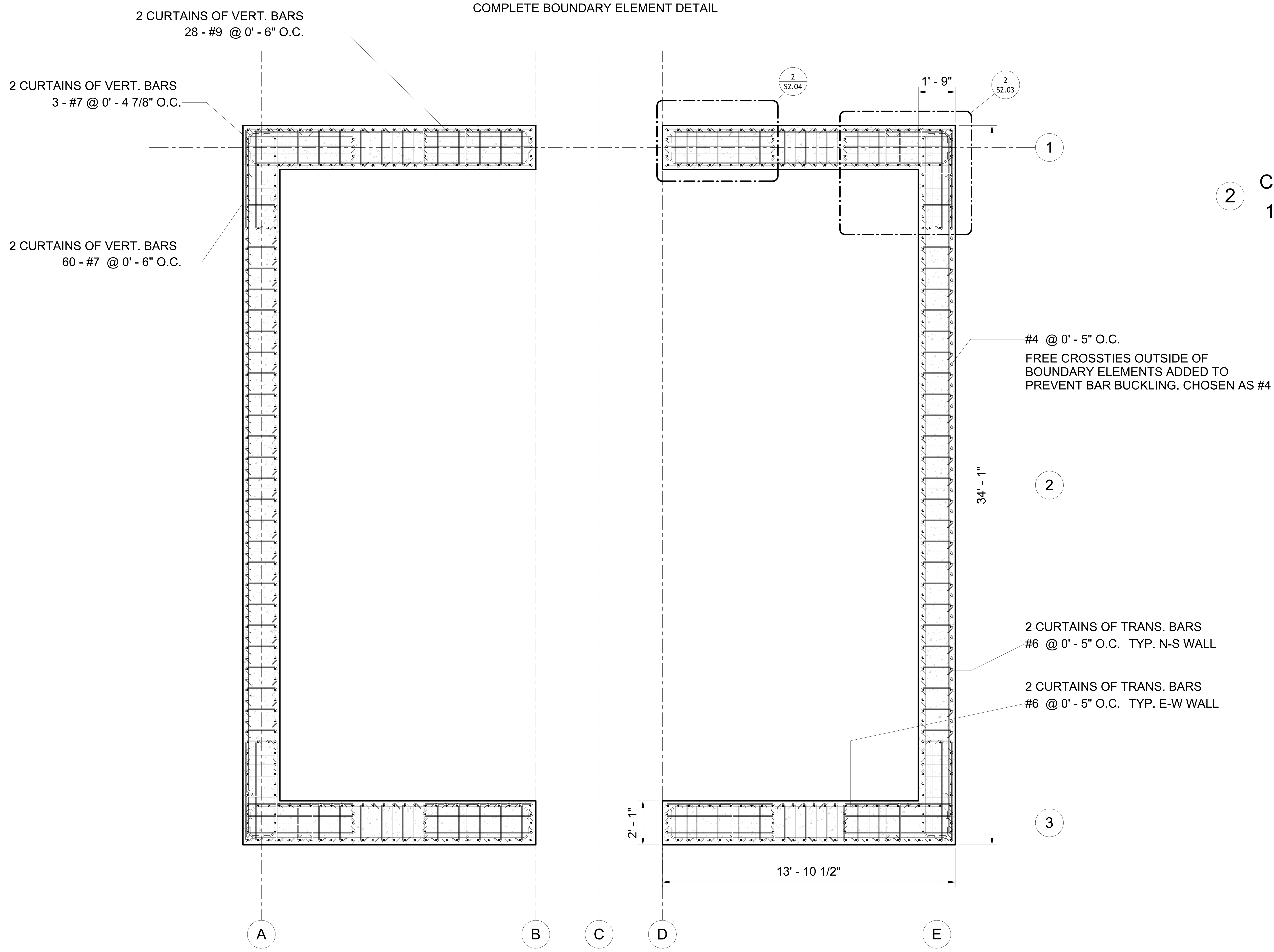
NOTE:
THIS CALLOUT (2) HAS THE CORNER BOUNDARY ELEMENT HIDDEN IN VIEW FOR CLARITY OF THE LONG. AND TRANS. REINF.

PLEASE REFER TO SHEET S2.04 FOR A COMPLETE BOUNDARY ELEMENT DETAIL, INCLUDING THAT OF THIS CORNER



2 **Core Wall Corner Detail**
1 1/2" = 1'-0"

NOTE:
PLEASE REFER TO SHEET S2.04 FOR A COMPLETE BOUNDARY ELEMENT DETAIL



1 **Core Wall Section Detail - Plan**
3/8" = 1'-0"

N

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No.	Description	Date

The City of Berkeley Council

1950 Oxford St.
Residential Building

FINAL
CORE WALL DETAIL

Project Number	CE 244 Semester Project
Date	December 12, 2025
Drawn By	CQ
Checked By	FP, FD, ES

S2.03

Scale	As indicated

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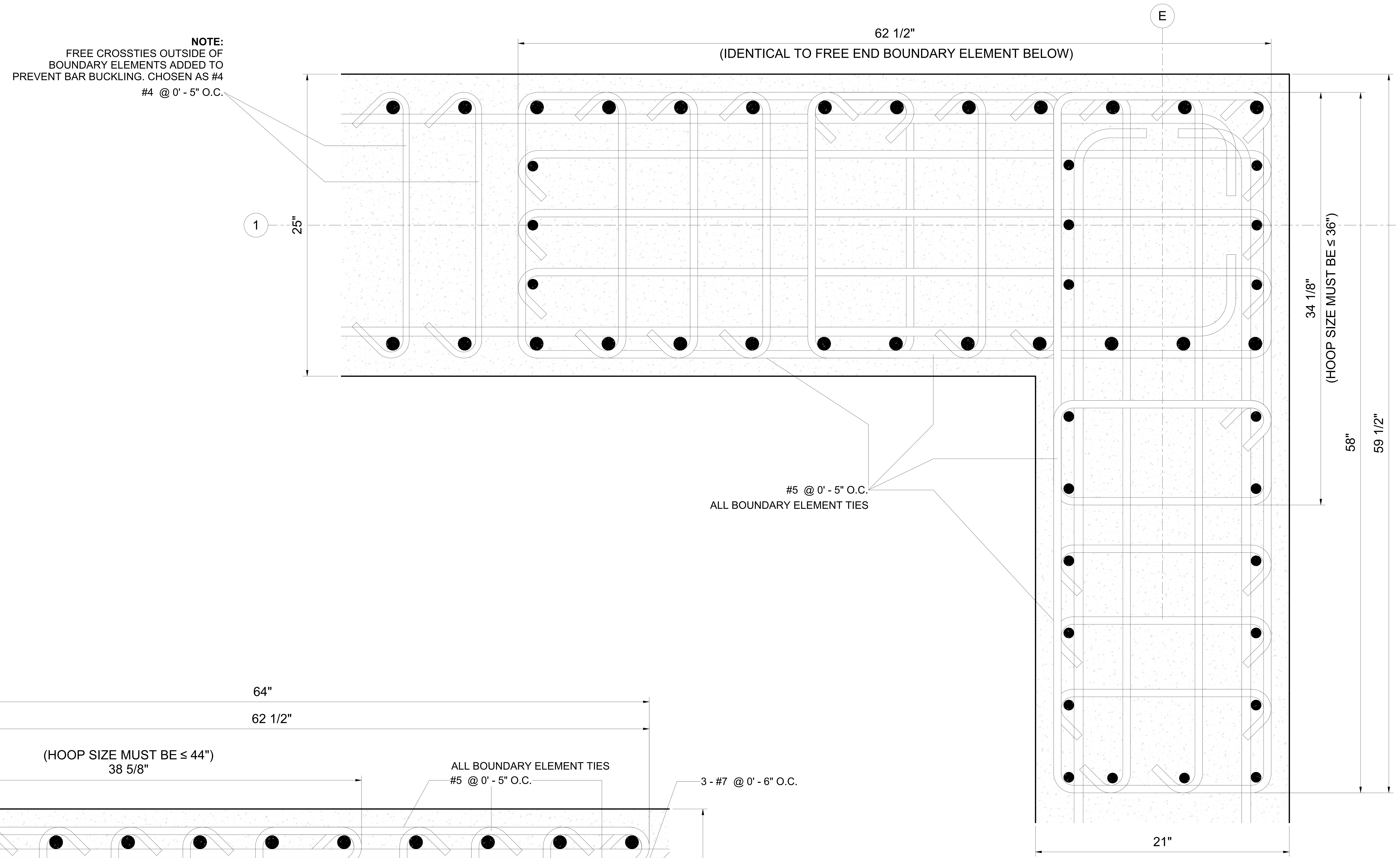
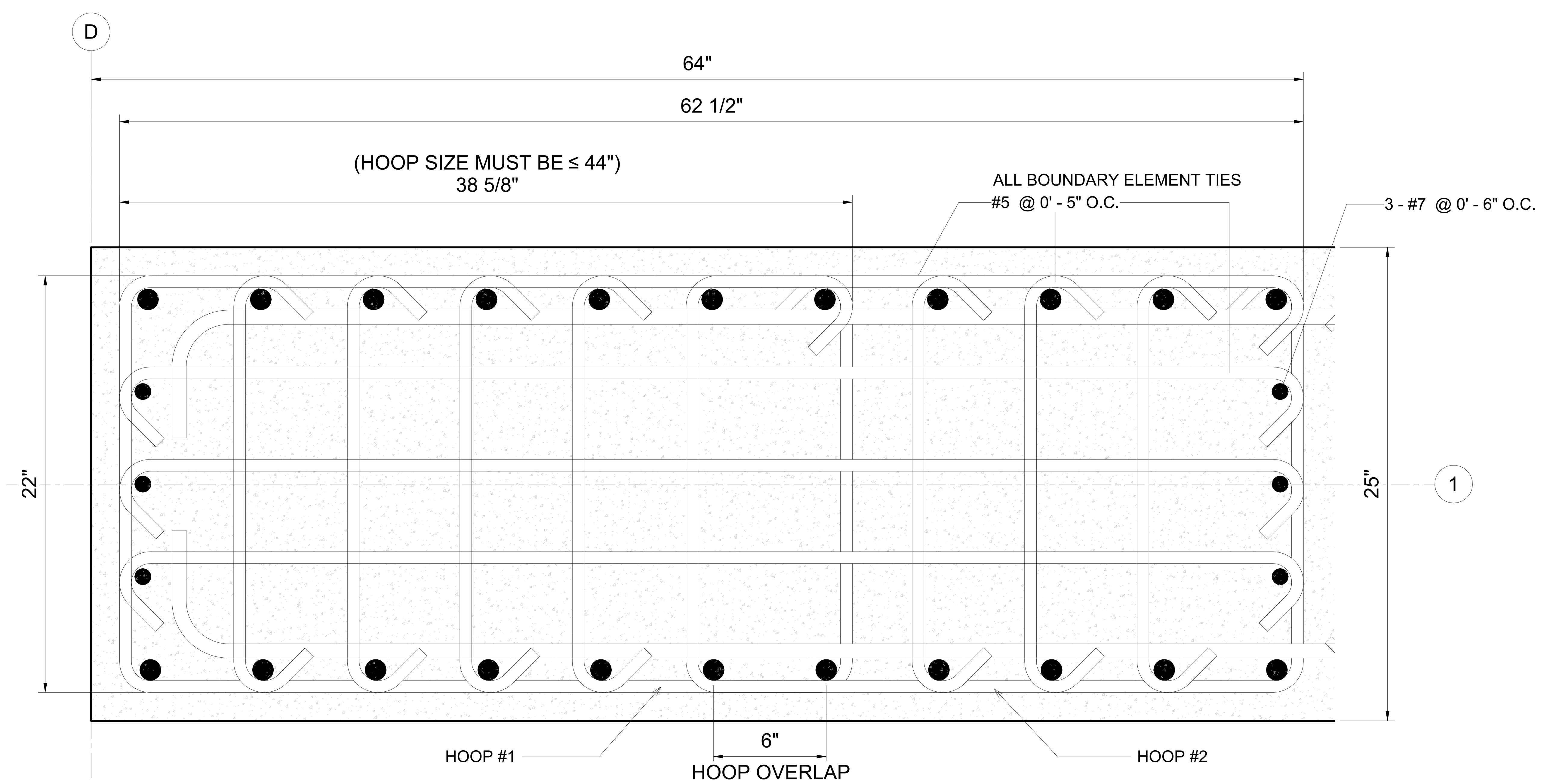


1 Corner Boundary Element
3" = 1'-0"



2 Free End Boundary Element
3" = 1'-0"

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No.	Description	Date

The City of Berkeley Council

1950 Oxford St.
Residential Building

FINAL BOUNDARY
ELEMENT DETAIL

Project Number	CE 244 Semester Project
Date	December 12, 2025
Drawn By	CQ
Checked By	FP, FD, ES

S2.04

Scale	3" = 1'-0"

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2.5 Coupling Beam Design

The objective of this section is to establish the dimensions and reinforcement of the coupling beams in the lateral-force-resisting system. Because the plan geometry, support conditions, and material properties are symmetric (see **Fig. 1.0.1**), designing a single representative beam is sufficient. For this assignment, the coupling beam at Floor #5 is used as the representative design case. The provisions of ACI 318–25 governing special coupling beams are adopted throughout this section [5, 11, 9, 1, 12].

2.5.1 Design shear force and redistribution

Section 18.10.7.5 of ACI 318–25 permits redistribution of design shear among vertically aligned coupling beams when ductile coupling-beam detailing is provided [5]. Redistribution is used here to better reflect the expected inelastic response while maintaining equilibrium of total coupling-beam shear over the building height.

Redistribution is permitted when the following requirements are satisfied [5]:

(a) coupling beams sharing redistributed forces are vertically aligned within a special structural wall,

(b) coupling beams sharing redistributed forces have $\ell_n/h \geq 2$,

(c) the maximum redistribution of V_e from any beam does not exceed 20% of the value obtained from analysis,

(d) $\sum \phi V_n \geq \sum V_e$ for the group of beams sharing redistributed demands.

Requirement (a) is satisfied by the wall configuration. Requirement (b) is satisfied by **Eq. (2.5.1)**. Requirements (c) and (d) are verified by the redistribution audit summarized in **Table 3.4.1**. Four distinct redistributed design shear values appear in the audit; only the group corresponding to Floors #5 and #6 is used in the subsequent design, as requested for academic purposes, giving the representative design shear $\mathbf{V_u}$ = **517** kip.

Level	ETABS CB shear [kip]	Redistributed CB shears [kips]	Relative Change [%]	Diff.[kips]
Roof	181.00	181.00	0.00%	-158.50
8	290.00	339.50	17.07%	0.00
7	389.00	339.50	-12.72%	-177.50
6	479.00	517.00	7.93%	0.00
5	555.00	517.00	-6.85%	-116.50
4	617.00	633.50	2.67%	0.00
3	661.00	633.50	-4.16%	0.00
2	670.00	633.50	-5.45%	0.00
1	586.00	633.50	8.11%	0.00
SUM	**4428.00**	**4428.00**	–	–

Table 2.5.1: Coupling beam shear redistribution along building height (full audit table reproduced from the design calculations).

2.5.2 Beam-type check and need for special coupling-beam detailing

The first check evaluates the clear-span-to-depth ratio. The clear span is $l_n = 72$ in and the beam depth is $h = 36$ in, giving:

$$\frac{l_n}{h} = \frac{72\text{ in}}{36\text{ in}} = 2.0. \tag{2.5.1}$$

Section 18.10.7.2 of ACI 318–25 indicates special coupling-beam detailing is required when the member has a small span-to-depth ratio and develops large shear demand relative to $A_{cw}\sqrt{f_c'}$ [5]. Using $f_c' = 7000$ psi, $\lambda = 1.0$, and $A_{cw} = bh = (25\text{ in})(36\text{ in})$, the shear threshold is:

$$4\lambda\sqrt{f_c'}\,A_{cw} = 4(1.0)\,\sqrt{7000\text{ psi}}\,(25\text{ in})(36\text{ in}) \approx 301.2\text{ kip}.$$

Since $V_u = 517$ kip (Floors #5–#6 group from **Table 3.4.1**) exceeds this value and $l_n/h = 2.0$, the beam is detailed as a diagonally reinforced special coupling beam.

2.5.3 Diagonal reinforcement design (ACI 318–25 18.10.7.4)

Section 18.10.7.4 limits the nominal shear strength for diagonally reinforced coupling beams to:

$$V_n \le 10\sqrt{f_c'}\,A_{cw}.$$

With $f_c' = 7000$ psi and $A_{cw} = (25)(36)$ in^2:

$$10\sqrt{f_c'}\,A_{cw} = 10\sqrt{7000}\,(25)(36) \approx 753\text{ kip}.$$

Since $V_u = 517$ kip, the limit is satisfied.

The diagonal reinforcement demand is obtained from:

$$V_u = \phi\, 2\, A_{vd,\text{req}}\, f_y\, \sin\alpha, \qquad A_{vd,\text{req}} = \frac{V_u}{\phi\, 2\, f_y\, \sin\alpha}.$$

Using $V_u = 517$ kip, $\phi = 0.85$, $f_y = 60$ ksi, and $\alpha = \tan^{-1}\left(\frac{11}{36}\right) \approx 17°$:

$$A_{vd,\text{req}} = \frac{517}{0.85(2)(60)\sin(17°)} \approx 17.34\text{ in}^2.$$

The final diagonal reinforcement selected for each diagonal set is:

$$\textbf{12 \#11}\text{ bars} \quad \Rightarrow \quad \mathbf{A_{vd,\text{prov}}} \approx \mathbf{18.72}\text{ in}^{\mathbf{2}} \;\ge\; \mathbf{A_{vd,\text{req}}}.$$

2.5.4 Development length of diagonal reinforcement

Diagonal bars are developed for $1.25 f_y$ per coupling-beam detailing provisions, and the hooked development length is evaluated per ACI 318–25 25.4.2.3 [5]:

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

Using $\lambda = 1.0$, $\psi_t = \psi_e = \psi_s = 1.0$, $f_y = 60$ ksi, $f_c' = 7000$ psi, and $d_b = 1.41$ in (No. 11 bar):

$$l_{dh} = \left(\frac{1.25(60{,}000)}{20\sqrt{7000}}\right)(1.41) \approx 63.2\text{ in} \;\rightarrow\; 64\text{ in}.$$

The horizontal projection check is:

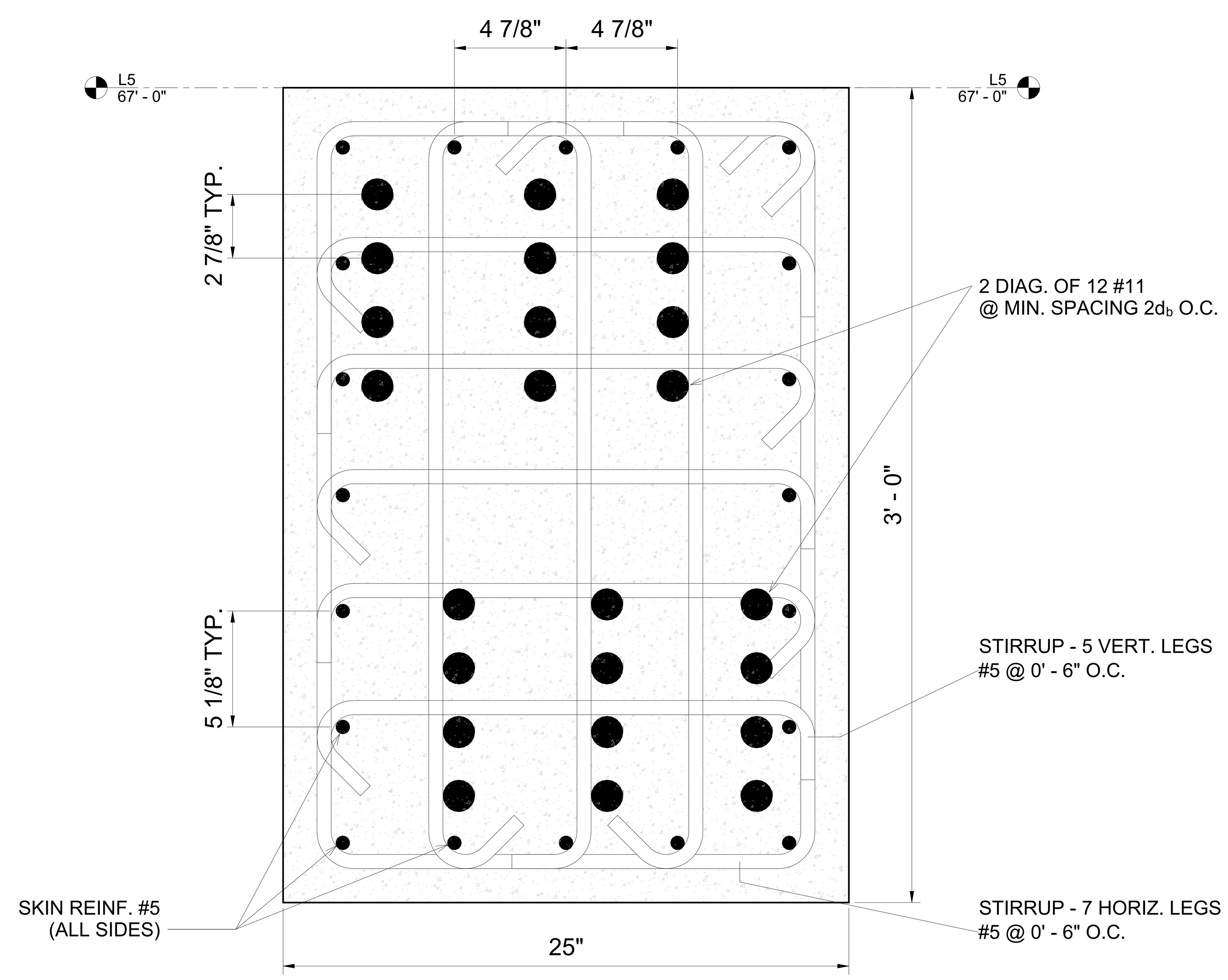
$$l_{dh}\cos\alpha = 64\cos(17°) \approx 61.2\text{ in} < l_{\text{E–W}} = 166.5\text{ in} \qquad \text{OK}.$$

2.5.5 Transverse confinement and skin reinforcement

Transverse confinement is designed per ACI 318–25 18.10.7.4 using:

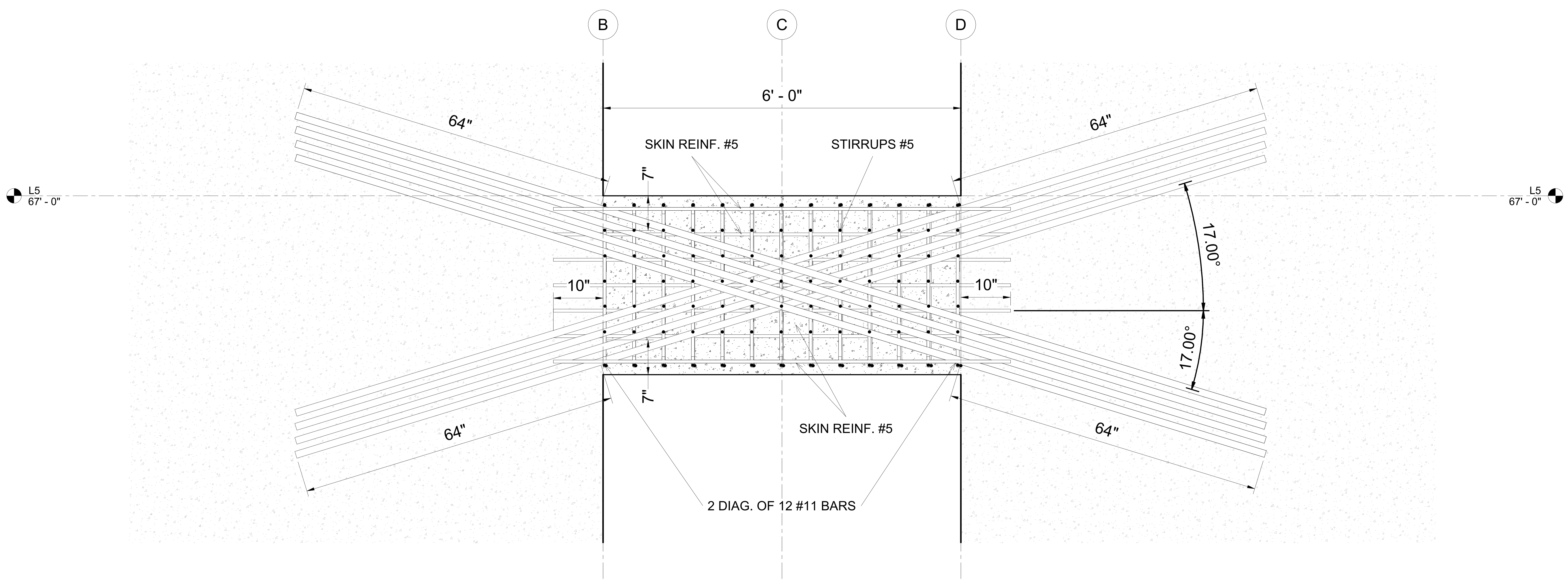
$$A_{sh} \geq \max\left\{0.09\, s\, b_c\, \frac{f'_c}{f_{yt}},\ 0.3\, s\, b_c \left(\frac{A_g}{A_{ch}} - 1\right) \frac{f'_c}{f_{yt}}\right\},$$

evaluated for both confinement directions. Based on the detailed checks, No. 5 transverse reinforcement satisfies the per-leg confinement demands. The final detailing provides full confinement of the coupling-beam core using No. 5 ties, and No. 5 skin reinforcement is provided as needed to satisfy spacing and ductility requirements for special coupling beams [5, 11, 9, 12]. Refer to structural sheet **S2.05** for the final detail drawing of the coupling beam.



1 Coupling Beam Section - Lv. 5

3" = 1'-0"



2 Coupling Beam Detail - Lv. 5

1" = 1'-0"

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No.	Description	Date

The City of Berkeley Council

1950 Oxford St.
Residential Building

FINAL
COUPLING BEAM DETAIL

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2.6 Gravity System Design

2.6.1 Design Displacements and Story Drift

The following table summarizes the displacement, drift, and capacity checks for each story.

Story	Height (in.)	X-disp. (in.)	Y-disp. (in.)	X-drifts (%)	Y-drifts (%)	Avg X-drift (%)	Avg Y-drift (%)	Capacity by Code (%)	Reinf. Needed?
Roof	135	8.04	7.86	0.58	0.64	0.305	0.325	1.0	NO
Story8	122	7.14	6.87	0.61	0.65	0.605	0.645	1.0	NO
Story7	109	6.20	5.85	0.63	0.65	0.625	0.645	1.0	NO
Story6	96	5.23	4.84	0.64	0.64	0.635	0.630	1.0	NO
Story5	83	4.23	3.84	0.64	0.61	0.625	0.605	1.0	NO
Story4	70	3.25	2.89	0.61	0.57	0.600	0.555	1.0	NO
Story3	57	2.30	2.00	0.56	0.50	0.550	0.495	1.0	NO
Story2	44	1.42	1.22	0.49	0.42	0.455	0.395	1.0	NO
Story1	31	0.66	0.56	0.35	0.29	0.250	0.220	1.0	NO
Lobby	16	0.03	0.03	0.01	0.02	0.175	0.145	1.0	NO

Table 2.6.1: Story drift demands, average drifts, and code drift capacity check.

From the Table above it can be shown that shear reinforcement within the slab-column joint is not needed at each level.

2.6.2 Typical Flat-Plate Connection for shear reinforcement

Representative gravity actions are evaluated to confirm compatibility of slab-to-wall connections and gravity framing under combined gravity demands and lateral drift compatibility [2, 5].

The total ultimate vertical shear force (V_u) to be resisted by the column-slab connection is the sum of the factored gravity load from the slab area and the factored cladding load.

$$V_{u,\text{total}} = V_{u,\text{gravity}} + V_{u,\text{cladding}}$$
$$V_{u,\text{total}} = 138.83 \text{ kips} + 34.27 \text{ kips}$$
$$V_{u,\text{total}} = 173.1 \text{ kips}$$

Design Punching Shear Strength Calculation:

$$\phi V_c = \phi \cdot v_c \cdot b_o \cdot d$$

Using $\phi = 0.75$ for shear:

$$\phi V_c = 0.75(295.1 \text{ psi})(124 \text{ in})(7 \text{ in})$$
$$\phi V_c \approx 192110.1 \text{ lbs}$$
$$\phi \mathbf{V_c} = \mathbf{192.11} \text{ kips}$$

It can be noted that punching shear capacity is larger than punching shear demand and thus shear reinforcement is not needed.

The shear reinforcement requirements for drift capcity are governed by the facts that ACI 318 18.14.5.1 need not be satisfied if (a) or (b) is met:

(a) $\Delta_x / h_{sx} \leq 0.005$ for nonprestressed slabs

(b) $\Delta_x / h_{sx} \leq 0.01$ for unbonded post-tensioned slabs with f_{pc} in each direction meeting the requirements of 8.6.2.1

Given that the maximum story drift is of 0.00645 which is $\leq$ 0.01 section 18.14.5.2 is triggered and reinforcement is not needed.

2.6.3 Gravity Column Design

The gravity column design at the base of the building is presented beginning with load takeoff (story-by-story axial demands), followed by selection of a representative interior base-level column, longitudinal reinforcement sizing, transverse reinforcement detailing, and a shear-strength verification [2, 5]. 16 No. 9 longitudinal bars are selected for an adequate design. Additionally transverse reinforcement of No 6. bars at 6 inches on center is selected along the entire height of the column.

The following assumptions govern the design:

- **Column Section:** 24 in. $\times$ 24 in. square section (assumed constant height).
- **Seismic Load (E):** $E = 0$ (Gravity-only column).
- **Moments:** Assumed negligible due to slab overhang balancing interior span moments and low drift ratios.

For a non-prestressed member with tied reinforcement, the design strength is calculated as:

$$\phi P_{n,max} = \phi \cdot 0.80 \cdot P_o \tag{2.6.1}$$

Where P_o is the nominal axial strength at zero eccentricity, defined in ACI Eq. 22.4.2.2 as:

$$P_o = 0.85 f'_c (A_g - A_{st}) + f_y A_{st} \tag{2.6.2}$$

By substituting the strength reduction factor $\phi = 0.65$ (for tied columns) and expanding the equation, we derive the expression for the required design strength. The coefficients below are derived from combining the reduction factors (e.g., $0.65 \times 0.80 \times 0.85 \approx 0.442$):

$$\begin{aligned} P_u &= \phi P_{n,max} \\ P_u &= 0.65(0.80) \left[0.85 f'_c (A_g - A_{st}) + f_y A_{st} \right] \\ P_u &= 0.442 f'_c A_g - 0.442 f'_c A_{st} + 0.52 f_y A_{st} \end{aligned}$$

Rearranging the terms to solve for the required area of steel (A_{st}):

$$A_{st} = \frac{P_u - 0.442 f'_c A_g}{0.52 f_y - 0.442 f'_c} \tag{2.6.3}$$

Calculation of Required Longitudinal Reinforcement

Using the controlling load combination and the section properties defined below:

- **Factored Load (P_u):** 1,948.64 kips ($1,948,640$ lbs)
- **Concrete Strength (f_c'):** 6,000 psi
- **Yield Strength (f_y):** 60,000 psi
- **Gross Area (A_g):** 24 in $\times$ 24 in = 576 in^2

Substituting these values into the derived equation:

$$A_{st} = \frac{1,948,640 - 0.442(6,000)(576)}{0.52(60,000) - 0.442(6,000)}$$

$$A_{st,\text{required}} = \mathbf{14.75}\ \text{in}^{\mathbf{2}}$$

To satisfy the required area of 14.75 in^2, we select 16 No. 9 bars.

- **Bar Area (No. 9):** 1.00 in^2
- **Total Provided Area:** 16×1.00 in^2 = 16.00 in^2

Confinement Length and Reinforcement

Per **ACI 18.7.5.1**, the length of the confinement zone (l_o) shall be the greatest of:

$$\begin{aligned}
l_o &= \max\left(C_1, C_2, \frac{h_n}{6}, 18\text{in}\right)\\
l_o &= \max\left(24\text{ in}, 24\text{ in}, \frac{15\text{ ft} \times 12}{6}, 18\text{ in}\right)\\
l_o &= \max\left(24\text{in}, 24\text{ in}, 30\text{ in}, 18\text{in}\right)\\
\boldsymbol{l_o} &= \mathbf{30}\text{ in}
\end{aligned}$$

- Column Base Dimensions: $C_1 = C_2 = 24$ in
- Column Clear Height Span: 16 ft - 1 ft = 15 ft

Spacing Requirements (s) within confinement zone

Per **ACI 18.7.5.3**, the spacing of transverse reinforcement shall not exceed the smallest of the following. First, we calculate the reference spacing s_o, assuming a horizontal spacing $h_x \leq 8$ in per **ACI 18.7.5.2** :

$$s_o = 4 + \left(\frac{14 - h_x}{3}\right) = 4 + \left(\frac{14 - 8\text{ in}}{3}\right) = 6\text{ in}$$

$$4\text{in} \le s_o \le 6 \text{ in OK}\checkmark$$

Now, determining the maximum allowable spacing s:

$$s = \min\left(\frac{C_1}{4}, \frac{C_2}{4}, 6d_b, s_o\right)$$
$$s = \min\left(\frac{24\text{ in}}{4}, \frac{24\text{ in}}{4}, 6(1.128\text{ in}), 6\text{ in}\right)$$
$$s = \min\left(6\text{ in}, 6\text{ in}, 6.77\text{ in}, 6\text{ in}\right)$$
$$\boldsymbol{s} = \mathbf{6}\text{ in}$$

Required Transverse Area (A_{sh})

Per **ACI 18.7.5.4**, since $P_u > 0.3A_g f'_c$ (1948.64 kips $>$ 1036.8 kips), the area of transverse reinforcement must satisfy the maximum of the three conditions below.

$$0.3A_g f'_c = \frac{0.3(24\text{ in} \times 24\text{ in})(6000\text{ psi})}{1000\frac{\text{lbs}}{\text{kip}}} = 1036.8\text{ kips}$$

Calculations (A_{sh}/sb_c ratios):

Parameters:

- Clear Cover (Outside of Ties): 1.5 in
- Core dimension (b_c): 24 in − 2(1.5 in) = 21 in
- Gross Area (A_g): 24 in × 24 in = 576 in^2
- Core Area (A_{ch}): 21 in × 21 in = 441 in^2
- Concrete Strength (f'_c): 6 ksi
- Yield Strength (f_{yt}): 60 ksi

Condition 1 (Geometry Control):

$$0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}} = \mathbf{0.009}$$

Condition 2 (Min Strength):

$$0.09\frac{f'_c}{f_{yt}} = \mathbf{0.009}$$

Condition 3 (High Axial Load): Requires calculation of factors k_f and k_n:

$$k_f = \frac{f'_c}{25,000} + 0.6 = 0.84 < 1.0 \quad (\text{Take } k_f = 1.0)$$
$$k_n = \frac{n_l}{n_l - 2} = \frac{16}{16 - 2} = 1.143$$

$$0.2k_f k_n \frac{P_u}{f_{yt} A_{ch}} = \mathbf{0.0168}$$

The controlling ratio is **0.0168** (Condition 3). Solving for required area A_{sh}:

$$\begin{aligned}
\frac{A_{sh}}{sb_c} &= 0.0168 \\
A_{sh} &= 0.0168(s)(b_c) \\
A_{sh} &= 0.0168(6\text{ in})(21\text{ in}) \\
\boldsymbol{A_{sh,\text{req}}} &= \mathbf{2.12}\ \text{in}^{\mathbf{2}}
\end{aligned}$$

Selection and Detailing

Inside Confinement Zone (l_o): Required $A_{sh} = 2.12$ in^2. Using **No. 6 bars** ($A_b = 0.44$ in^2):

$$\text{Number of legs} = \frac{2.12}{0.44} = 4.81 \rightarrow \text{Use 5 legs}$$

Since the column is square and symmetrical the calculations above stand for both directions of the column and thus the designed reinforcement is applied equally in both directions.

Selection: Use No. 6 hoops/crossties at 6 in o.c. both ways within l_o.

Outside Confinement Zone: Per ACI 18.7.5.5, spacing shall be the lesser of $6d_b$ or 6 in.

$$s = \min(6\text{ in}, 6d_b)$$

$$s = \min(6\text{ in}, 6(1.128\text{ in})) = 6\text{ in}$$

Selection: Use No. 6 hoops/crossties at 6 in o.c. both ways due to the same spacing and reasons specified for reinforcement inside the confinement zone (l_o)

Column Shear Reinforcement

The calculations below are done to show transverse reinforcement provided in the previous section is sufficient for shear. The design shear (V_u) is determined using the probable moment strength (M_{pr}) of the column, accounting for the range of axial forces.

Design Shear Force (V_u)

Based on the P-Mpr interaction diagram analysis:

- **Probable Moment (M_{pr}):** 1258 kips-ft
- **Associated Axial Load (P):** 656.09 kips
- **Clear Span (l_n):** 15 ft

The design shear is calculated assuming plastic hinges form at both ends of the clear span:

$$V_u = \frac{2M_{pr}}{l_n}$$

$$V_u = \frac{2(1,258 \text{ kips-ft})}{15 \text{ ft}}$$

$$V_u = 167.73 \text{ kips}$$

Nominal Shear Strength (V_n)

The nominal shear strength of the column is calculated as the sum of the concrete contribution (V_c) and the steel reinforcement contribution (V_s).

The concrete contribution (V_c) Per ACI 25.5.5 V_c is calculated as:

$$V_c = \left[2\lambda\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d \tag{2.6.4}$$

The steel contribution (V_s) provided by the transverse reinforcement is:

$$V_s = \frac{A_v f_{yt} d}{s} \tag{2.6.5}$$

Using the adopted section and reinforcement:

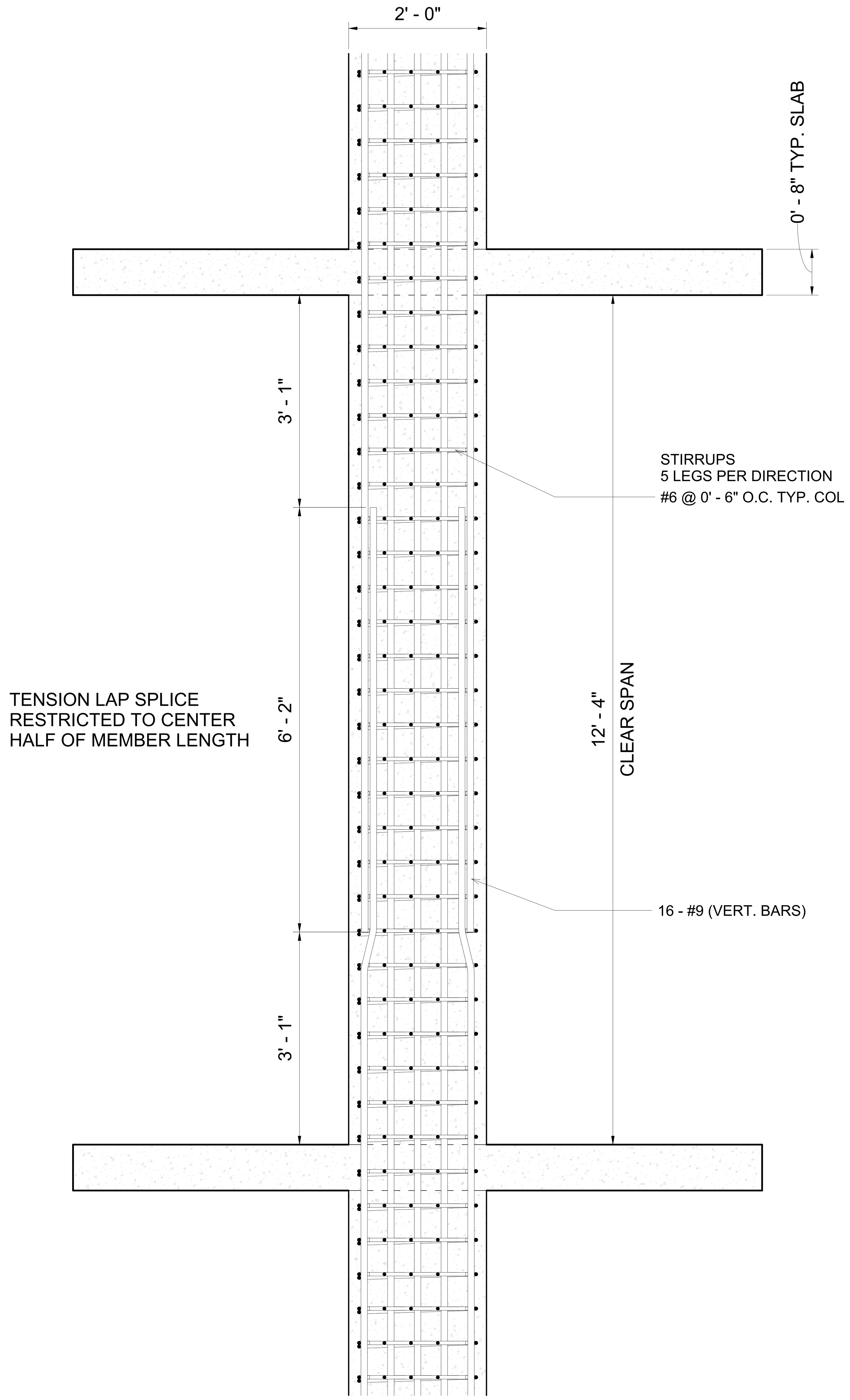
$$\begin{aligned} V_c &= \left(2\lambda\sqrt{f'_c} + \frac{N_u}{6A_g}\right) b_w d = 175.3\,\text{kips}, \\ V_s &= \frac{A_v f_{yt} d}{s} = 466.1\,\text{kips}, \\ \phi V_n &= 0.75\,(V_c + V_s) = 481.0\,\text{kips}. \end{aligned}$$

Therefore:

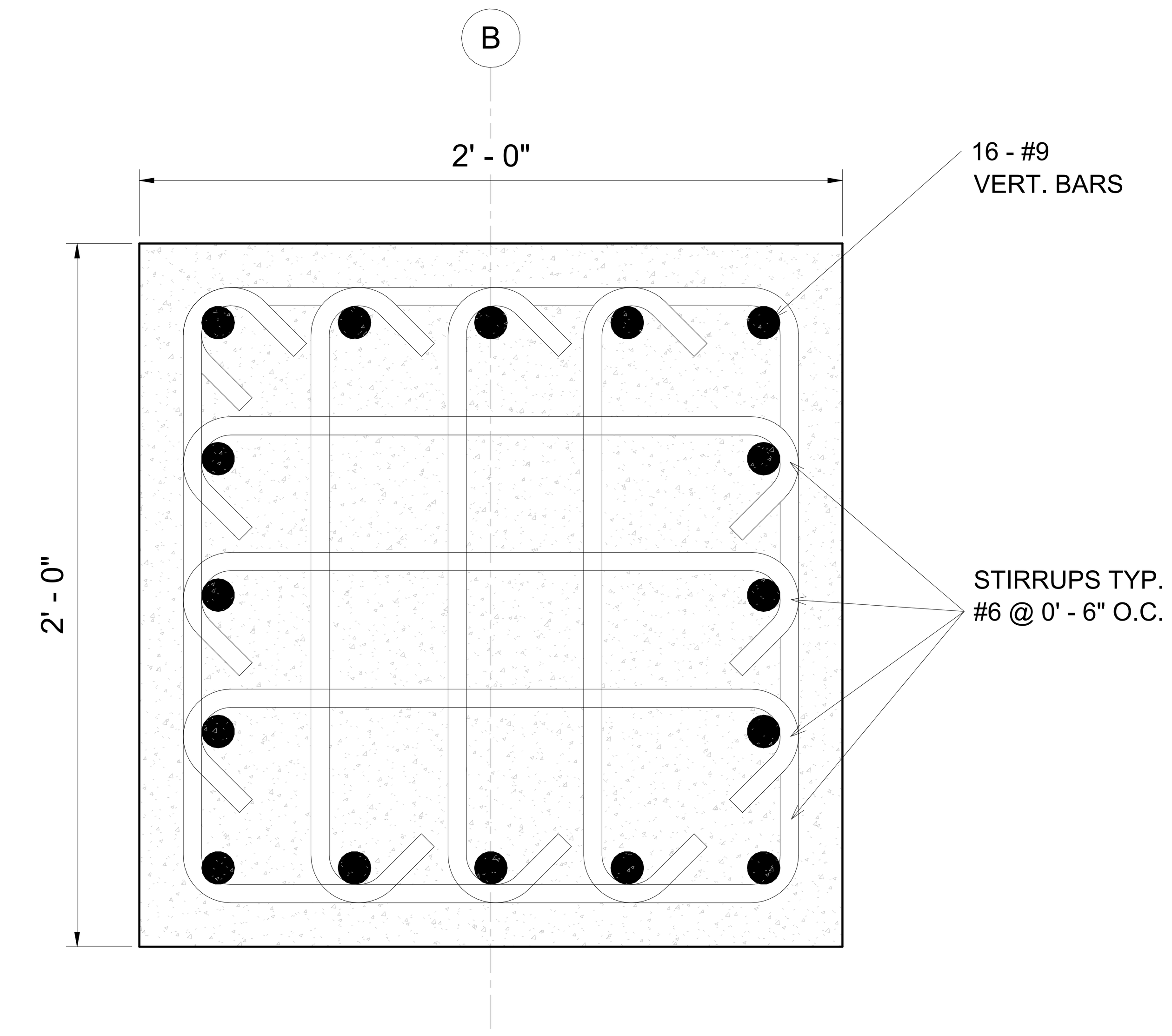
$$V_u = 167.73\,\text{kips} \leq \phi V_n = 481.0\,\text{kips}.$$

For a complete description of calculations see Section 3.6

Refer to structural sheet **S2.06** for the final detail of a typical interior column.



1 Typical Column Detail - Elevation
1" = 1'-0"



2 Typical Column Detail - Plan
3" = 1'-0"

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1950 Oxford St.
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TYP. INTERIOR COLUMN DETAIL

Project Number	CE 244 Semester Project
Date	December 12, 2025
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3 Part III: Specific Homework Submissions

3.1 Shear Wall Preliminary Design

3.1.1 Seismic Design Criteria and Response Coefficients

Parameter	Description / Formula	Value / Calculation	Reference
R	Seismic response modification factor.	5	Given [1]
I_e	Seismic importance factor.	1.0	Given (ASCE 7–22 Sec. 11.5.1[2])
h_n	Structural height from the base to mean roof.	119 ft	ASCE 7–22 Sec. 11.2 (definitions)
C_t	Empirical period coefficient (concrete shear walls).	0.02	
x	Empirical period exponent (concrete shear walls).	0.75	
T_a	Approx. fundamental period, $T_a = C_t\, h_n^x$.	0.72 s	ASCE 7–22 Sec. 12.8.2.1
S_a	Design EQ response spectrum pseudo–acceleration at T_a.	1.1 g	Project multi-period spectrum; see **Fig. 3.1.1**.
$C_{s,NS}$	Seismic response coefficient, $C_s = \dfrac{S_a}{R/I_e}$.	$\dfrac{1.1}{5/1.0} = 0.22$	ASCE 7–22 Eq. (12.8–2)
$C_{s,EW}$	Seismic response coefficient, $C_s = \dfrac{S_a}{R/I_e}$.	$\dfrac{1.1}{5/1.0} = 0.22$	ASCE 7–22 Eq. (12.8–2)

Table 3.1.1: Seismic response parameters and coefficients using the empirical period and $C_s = S_a/(R/I_e)$ per ASCE 7–22[2].

Note: N-S & E-W directions have identical lateral systems, thus having the same seismic response coefficients

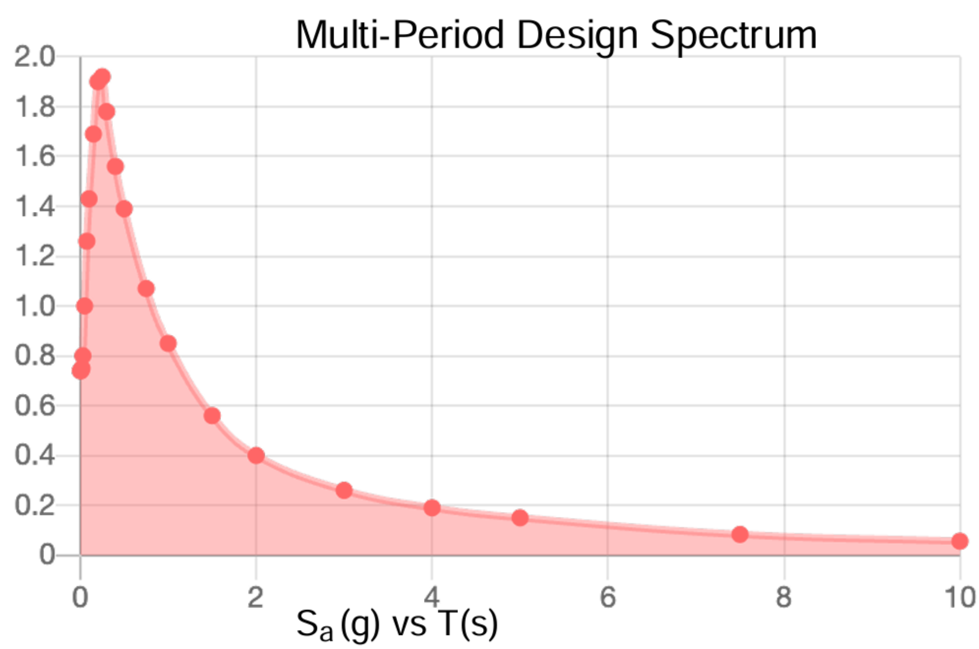


Figure 3.1.1: Multi-Perid Design Spectrum: $S_a = 1.1$g for $T_a = 0.72$s. Obtained from ASCE Hazard Tool[2].

3.1.2 Effective Seismic Weight of the Building

The objective of this section is to correctly define member widths, convert them into member volumes, and compute self-weight and superimposed dead load (SDL) to assemble the effective seismic weight per ASCE 7–22[2]. The seismic weight is fundamental to defining the seismic demands for the structure, as it will be seen Section 3.1.3. The results are presented from Table 3.1.2 to Table 3.1.4.

Considered Member Widths

- **Columns:** 2 ft. A square 24 in × 24 in pier is taken as a 2 ft tributary width for volume.
- **Walls (N/S):** $32\ \text{ft} + t_{\text{wall(E/W)}}$. Full wall width plus the wall thickness so that shared interfaces are not lost when volumes are summed.
- **Walls (E/W):** $\frac{1}{2}(32-6) - \frac{1}{2}t_{\text{wall(N/S)}}$ [ft]. One half-wall minus the 6 ft opening, then subtract half the N/S wall thickness to avoid double counting at the intersection.
- **Slabs:** 104 ft. Net plan width; openings (elevator, shafts) are discounted separately.

Member Volumes: $V\ [\text{ft}^3] = \text{Length} \times \text{Thickness} \times \text{Width}$.

Member Self-weight:

$$W_{\text{member}}\ [\text{kips}] = \frac{V\ [\text{ft}^3] \times \text{Quantity} \times 150\ \text{pcf}}{1000\ \text{lb/kip}}, \qquad W_{\text{self, total}} = \sum_i W_{\text{member},i}.$$

Normal-weight concrete is taken as 150 pcf, as per ACI 318-25[5].

Applied Areas per Floor: $A_{\text{applied}}\ [\text{ft}^2]$ equals the footprint receiving SDL on each participating floor (net of openings).

Superimposed Dead Load:

$$W_{\text{SDL, total}}\ [\text{kips}] = \sum_j \left(\frac{\text{SDL}_j\ [\text{psf}]}{1000\ \text{lb/kip}} \cdot A_{\text{applied},j}\ [\text{ft}^2] \cdot N_{\text{floors},j} \right).$$

Cladding is applied by wall area when specified (psf of wall area).

Effective Seismic Weight (optional summary): $W_{\text{eff}} = W_{\text{self, total}} + W_{\text{SDL, total}}$.

Use	Live Loading (psf)	Superimposed Dead Loading (psf)
Corridor/Stairs [1]	100	15
Lobbies	100	40
Parking	40	5
Residential	40	25
Occupied Roof[2]	100	75
Roof	20	15
Exterior Cladding (Precast panel system)		55 psf of wall area

Table 3.1.2: Gravity loading.

[1] Corridor/stairs loading applies to the core region and 4 ft beyond the core on all sides.
[2] 25% of the roof is assumed as Occupied Roof.

Member	Parameter	Value	Comment
Columns	Uniform Weight [pcf]	150.00	
	Width [ft]	2.00	
	Thickness [ft]	2.00	
	Length [ft]	113.00	
	Volume [ft^3]	452.00	
	Total [kips]	813.60	Assumed as 24 by 24.
Walls (N/S)	Uniform Weight [pcf]	150.00	
	Width [ft]	34.50	
	Thickness [ft]	2.50	
	Length [ft]	113.00	
	Volume [ft^3]	9,746.25	Quantity = 2
	Total [kips]	2,923.88	End to end.
Walls (E/W)	Uniform Weight [pcf]	150.00	
	Width [ft]	11.75	
	Thickness [ft]	2.50	
	Length [ft]	113.00	
	Volume [ft^3]	3,319.38	Quantity = 4
	Total [kips]	1,991.63	Subtracting N/S thickness.
Coupling Beams	Uniform Weight [pcf]	150.00	
	Width [ft]	1.83	
	Thickness [ft]	2.50	
	Length [ft]	6.00	
	Volume [ft^3]	27.50	Quantity = 18
	Total [kips]	74.25	Assume 30” L–beam depth[3].
Slabs	Uniform Weight [pcf]	150.00	
	Width [ft]	104.00	
	Thickness [ft]	0.67	
	Length [ft]	104.00	
	Elevator Area [ft^2]	693.25	Discounted area
	Volume [ft^3]	6,748.50	Quantity = 9
	Total [kips]	9,110.48	Elevator area discounted.
Total Selfweight			**14,913.83 kips**

Table 3.1.3: Member self–weight using normal–weight concrete (150 pcf); volumes reflect listed tributary widths and the elevator area discount.

[3]The values computed subtract the 8 inches corresponding to the slab thickness.

Dead Load	Superimposed Dead Loading (psf)	Applied Area per Floor (ft^2)	Number of Floors for Seismic Demand	Total [lbs]	Total [kips]	Comment
Corridors/Stairs	15.00	1,600.00	8.00	192,000.00	192.00	Floors above ground. Extend 4 ft from floor core.
Lobbies	40.00	9,216.00	0.00	0.00	0.00	Lobbies only on ground level.
Parking	5.00	25,600.00	0.00	0.00	0.00	Assumed below–grade. No seismic demand.
Residential	25.00	9,216.00	8.00	1,843,200.00	1,843.20	Only on floors above ground level.
Occupied Roof	75.00	2,704.00	1.00	202,800.00	202.80	Assumed as 25% of the total roof area.
Roof	15.00	8,112.00	1.00	121,680.00	121.68	Assumed as 75% of the total roof area.
Exterior (1st F) Cladding	55.00	6,240.00	1.00	343,200.00	343.20	No reduction on 1st floor.
Exterior (Other) Cladding	55.00	5,408.00	8.00	2,379,520.00	2,379.52	
Total Superimposed Dead Load					**5,082.40**	

Table 3.1.4: Superimposed dead loads by use; applied areas are per floor and multiplied by the number of floors contributing to seismic mass.

Total Effective Seismic Weight (W):

W = Total Self-weight + Total Superimposed D.L. = 19,996.23 kips

3.1.3 Seismic Base Shear: V_{NS} & V_{EW}

Base Shear:

Note: Since N-S and E-W directions have the same lateral systems and thus the same seismic response coefficients, $V_{NS} = V_{EW} = V_b$

$$V_b = C_s\, W = (0.22)(19,996.23\,\text{kips}) = 4399.17\,\text{kips}$$

3.1.4 Shear Wall Thickness Verification

In order to estimate the required member thicknesses, **Eq. (3.1.1)** serves as a first-order approximation for for this preliminary stage [1, 12].

$$V_u = V_b \leq 4\phi\sqrt{f_c'}\, l_w t_w \tag{3.1.1}$$

Rearranging for t_w

$$t_w = \frac{V_b}{4\phi\sqrt{f_c'}\, l_w}$$

Where l_w is the total length of shear walls in direction of interest

Iteration 1
Assuming 30 inch walls in both directions:

W = 19996.23 Kips,

V_b = 4399.17 Kips

North-South Direction

Note: l_w = 2 * 414" = 828"

$$t_w = \frac{4399170}{4(0.75)\sqrt{7000}\,(828)}$$

$$t_w = 21.17" \approx 22"$$

East-West Direction

Note: l_w = 4 * 171" = 684"

$$t_w = \frac{4399170}{4(0.75)\sqrt{7000}\,(684)}$$

$$t_w = 25.62" \approx 26"$$

The obtained thickness values are smaller than the initial assumed thickness. Therefore, an iterative process is carried out in Section 3.1.5 to optimize the shear wall thicknesses in this preliminary stage. The calculations of the seismic weights, as per Table 3.1.3, are rerun for each iteration, yielding different results for the base shear.

3.1.5 Verification of Shear Wall Thickness Iterations

Iteration 2
Using Wall Thicknesses Found In Iteration 1

W = 18,944.38 Kips,

V_b = 4,167.76 Kips

North-South Direction

Note: l_w = 2* 410" = 820"

$$t_w = \frac{4167760}{4(0.75)\sqrt{7000}\,(820)}$$

$$t_w = 20.25" \approx 21"$$

East-West Direction Note: l_w = 4 * 167" = 668"

$$t_w = \frac{4167760}{4(0.75)\sqrt{7000}\,(668)}$$

$$t_w = 24.86" \approx 25"$$

Iteration 3
Using Wall Thicknesses Found In Iteration 2

W = 18,773.99 Kips,

V_b = 4,130.28 Kips

North-South Direction

Note: l_w = 2* 409" = 818"

$$t_w = \frac{4130280}{4(0.75)\sqrt{7000}\,(818)}$$

$$t_w = 20.12" \approx 21"$$

East-West Direction

Note: l_w = 4 * 166.5" = 666"

$$t_w = \frac{4130280}{4(0.75)\sqrt{7000}\,(666)}$$

$$t_w = 24.71" \approx 25"$$

3.1.6 Summary

As it can be observed in Section 3.1.5, the wall thickness for both directions no longer decreased from Iteration 2 to Iteration 3. The thicknesses of Iteration 3 is considered as the final thicknesses in this preliminary stage, subject to future verification. The Final Building Properties are:

$$W = 18,773.99\ Kips$$
$$V_b = 4130.28\ Kips$$

$$t_{wNS} = 21\text{''}$$
$$t_{wEW} = 25\text{''}$$

3.2 C-Shape Flexural Design

The analysis team [1, 12] has developed a three-dimensional model of the building in ETABS[6] and has completed analyses under the governing load combinations including dead (D), live (L), and earthquake (E) load cases, consistent with ASCE 7 provisions [2]. The resulting axial forces and bending moments for the C-shaped segment of the core wall are summarized in **Fig. 3.2.1**. As required by ASCE 7-22 [2], components that resist earthquake design loads acting in both horizontal and vertical directions are considered.

The flexural design procedure adopted in this section begins by identifying the most demanding axial-force states at the base of the C-shaped wall. In this direction, the required section forces acting at the Story 1 are considered at this design stage.

	1.4D			1.2D+0.5L+(Ex±0.3Ey)/(R/IE)			1.2D+0.5L+(0.3Ex±Ey)/(R/IE)			0.9D+(Ex±0.3Ey)/(R/IE)			0.9D+(Ex±0.3Ey)/(R/IE)		
Level	**P [kip]**	**My [kip ft]**	**Mx [kip ft]**	**P [kip]**	**My [kip ft]**	**Mx [kip ft]**	**P [kip]**	**My [kip ft]**	**Mx [kip ft]**	**P [kip]**	**My [kip ft]**	**Mx [kip ft]**	**P [kip]**	**My [kip ft]**	**Mx [kip ft]**
Roof	-743	596	199	-1100	387	1404	-926	511	3199	-726	204	1480	-552	329	3275
Story 8	-1429	487	283	-2173	-1342	3756	-1694	-119	10757	-1604	-1437	3864	-1125	-214	10864
Story 7	-2117	497	326	-3426	-3308	6787	-2516	-694	21282	-2662	-3420	6911	-1753	-807	21407
Story 6	-2805	499	353	-4838	-5404	10402	-3386	-1325	34075	-3881	-5515	10537	-2429	-1436	34209
Story 5	-3492	505	368	-6387	-7556	14517	-4296	-1968	48788	-5235	-7670	14657	-3145	-2082	48928
Story 4	-4177	508	376	-8047	-9891	19045	-5239	-2670	65284	-6702	-10005	19189	-3895	-2784	65427
Story 3	-4861	529	382	-9788	-12616	23861	-6205	-3476	83448	-8251	-12737	24006	-4668	-3597	83593
Story 2	-5543	462	385	-11550	-16421	28745	-7177	-4679	103165	-9821	-16504	28891	-5448	-4762	103311
Story 1	-6283	654	387	-13218	-25422	33801	-8154	-7212	127330	-11285	-25635	33948	-6221	-7425	127478
Subfloor	-7193	442	265	-14189	-1574	24586	-9048	-179	89938	-11981	-1712	24687	-6839	-317	90039

	1.2D+1.6L			1.2D+0.5L+(-Ex±0.3Ey)/(R/IE)			1.2D+0.5L+(-0.3Ex±Ey)/(R/IE)			0.9D+(-Ex±0.3Ey)/(R/IE)			0.9D+(-0.3Ex±Ey)/(R/IE)		
Level	**P [kip]**	**My [kip ft]**	**Mx [kip ft]**	**P [kip]**	**My [kip ft]**	**Mx [kip ft]**	**P [kip]**	**My [kip ft]**	**Mx [kip ft]**	**P [kip]**	**My [kip ft]**	**Mx [kip ft]**	**P [kip]**	**My [kip ft]**	**Mx [kip ft]**
Roof	-1323	686	-208	-603	744	507	-777	618	2931	-229	561	583	-404	436	3006
Story 8	-2065	387	-296	-803	2156	2706	-1283	930	10441	-234	2061	2813	-714	835	10549
Story 7	-2806	444	-342	-825	4169	6031	-1736	1548	21056	-61	4057	6156	-973	1436	21180
Story 6	-3546	443	-370	-685	6265	10145	-2141	2175	33998	273	6153	10279	-1183	2063	34132
Story 5	-4283	451	-385	-408	8427	14921	-2503	2825	48909	744	8313	15061	-1352	2712	49049
Story 4	-5019	451	-394	-16	10764	20375	-2830	3525	65682	1329	10651	20518	-1486	3412	65826
Story 3	-5752	479	-400	460	13529	26583	-3132	4366	84264	1997	13408	26728	-1595	4245	84410
Story 2	-6483	345	-403	961	17168	33732	-3424	5396	104660	2690	17085	33878	-1695	5313	104807
Story 1	-7262	795	-405	1269	26670	43606	-3809	8412	130271	3201	26456	43753	-1876	8199	130419
Subfloor	-8301	516	-278	519	2415	29946	-4636	1017	91546	2727	2278	30047	-2427	880	91647

Figure 3.2.1: C-shaped core-wall segment considered for flexural design and sign convention for internal forces.

Remark: the design axial forces and moments reported in Figure 3.2.1 are the same for all CE 244 project groups and do not depend on the specific stiffness assumptions adopted for each case. Even if the building model is refined or updated, these section forces are to be treated as fixed inputs for this homework. This simplification is introduced solely for academic purposes.

3.2.1 Comment on Critical Load Combinations

The ETABS results at the base of the C-shaped wall provide factored axial forces and biaxial bending moments for all required strength-level load combinations in accordance with ASCE 7[2]. Each column in the tabulated output corresponds to a different combination of dead, live, and earthquake effects, with different orientations and levels of axial load and moment.

For the purposes of the *preliminary* flexural design in this report, two representative design states are adopted instead of checking every individual combination at this stage. The intent is to capture the extremes of the expected axial–moment interaction envelope and size the longitudinal reinforcement accordingly, with the understanding that a more detailed verification against all load combinations will follow in the final design step (see Section 3.2.4).

- $0.90D + (-0.30E_x \pm E_y)(R/IE)$, which is the combination with the largest factored bending moment about the global x-axis (i.e., maximum $|M_{ux}|$) and negligible moment about the x-axis, representing strong flexure in the global x direction.
- $0.90D + (E_x \pm 0.30E_y)(R/IE)$, with the largest factored bending moment about the global y-axis (i.e., maximum $|M_{uy}|$) and negligible moment about the y-axis, representing strong flexure in the global y direction.

Both selected states are flexure-controlled, as it will be shown in future P-M diagrams. Each state represents an extreme unidirectional bending condition about one of the two principal global axes. Using these two orthogonal flexural extremes as the basis for preliminary P–M interaction checks provides a clear envelope for sizing the longitudinal reinforcement. A subsequent verification against the full set of design combinations will be carried out in Section 3.2.4.

3.2.2 Preliminary Longitudinal Reinforcement Design

Solving for the Centroid The objective of this section is to find the geometric center of the cross-section of the C-shape wall. Since the geometry presents symmetry around X, the centroid of the section will be located at the mid-height of the figure. To solve for the x coordinate, the cross-section is broken down into three simple rectangular areas (A_1, A_2, and A_3) to facilitate the calculation of the cross-section area's centroid. The corresponding areas are:

$$A_1 = A_3 = 25\,\text{in} \times 145.5\,\text{in} = 3637.5\,\text{in}^2 \qquad A_2 = 21\,\text{in} \times 409\,\text{in} = 8589\,\text{in}^2$$

The distance of the centroid of each component area is calculated from a chosen reference axis considered at the leftmost edge.

$$x_1 = x_3 = 21\,\text{in} + \frac{145.5\,\text{in}}{2} = 93.75\,\text{in} \qquad x_2 = \frac{21\,\text{in}}{2} = 10.5\,\text{in}$$

Using first moments of area, the x-coordinate of the composite section's centroid is

$$\bar{X} = \frac{\sum A_i x_i}{\sum A_i} = \frac{A_1x_1 + A_2x_2 + A_3x_3}{A_1 + A_2 + A_3}.$$

With $A_1 = A_3 = 3637.5\,\text{in}^2$, $A_2 = 8589\,\text{in}^2$, $x_1 = x_3 = 93.75\,\text{in}$, and $x_2 = 10.5\,\text{in}$,

$$\bar{X} = \frac{2(3637.5\,\text{in}^2)(93.75\,\text{in}) + (8589\,\text{in}^2)(10.5\,\text{in})}{2(3637.5\,\text{in}^2) + 8589\,\text{in}^2} = 48.67\,\text{in} \approx 4.05\,\text{ft}.$$

As mentioned, section is symmetric about the x-axis, so the y-coordinate of the centroid is one half of the total height:

$$\bar{Y} = \frac{409\,\text{in}}{2} = 204.5\,\text{in} \approx 17.04\,\text{ft}.$$

Preliminary Steel Reinforcement in the N-S (Web) The required steel area A_s in the M_y direction (bending about the vertical y-axis) is evaluated from the following factored actions:

$$M_u = 26{,}670\text{ k-ft}, \qquad P_u = -1{,}269\text{ k}.$$

The required nominal axial strength, obtained by dividing the factored axial load by the strength reduction factor $\phi = 0.9$ according to a tension-controlled section as per ACI 318-25[5], is:

$$P_n = \frac{P_u}{\phi} = \frac{-1{,}269 \text{ k}}{0.9} = -1{,}410 \text{ k}.$$

Similarly, the required nominal flexural strength about the y-axis is;

$$M_n = \frac{M_u}{\phi} = \frac{26{,}670 \text{ k-ft}}{0.9} = 29{,}633.33 \text{ k-ft}.$$

According to **Fig. 3.2.2**, the eccentricity of the axial load with respect to the compression centroid, measured from the section centroid $\bar{X}$, is;

$$x_e = 93.75 \text{ in} - 48.67 \text{ in} = 45.08 \text{ in} = 3.756 \text{ ft}.$$

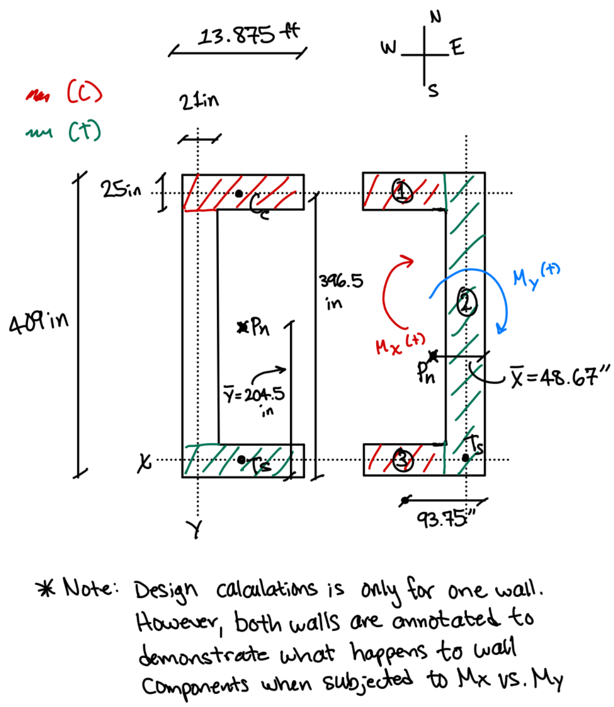


Figure 3.2.2: Wall when subjected to: M_x (Left) and M_y (Right)

The eccentricity of the tensile force provided by the reinforcement, measured from $\bar{X}$ to the line of action of the tensile steel, is;

$$x_s = 93.75 \text{ in} - \frac{21 \text{ in}}{2} = 83.25 \text{ in} = 6.938 \text{ ft}.$$

Moment equilibrium requires that the tension force satisfies:

$$T_s = \frac{M_n - P_n(x_e)}{x_s} = 5{,}034.89 \text{ k}$$

So that the total steel area needed based on the tension force and the steel yield strength ($f_y = 60$ ksi) is:

$$A_s = \frac{T_s}{f_y} = 83.91 \text{ in}^2$$

Distributing the required A_s into a practical number and size of bars is carried out by assuming two curtains and a target spacing along the $L \approx 409$ in N–S wall length. For two curtains, the required area per curtain are taken as:

$$A_{s,\text{curtain}} \approx 41.96 \text{ in}^2.$$

A bar spacing s of 6 inches is adopted. The clear cover l_c, bar diameter d_b (assuming #10 longitudinal bars), and transverse bar diameter d_{tv} (assuming #4 bars) are taken as:

$$l_c = 1.5 \text{ in}, \qquad d_b = 1.27 \text{ in}, \qquad d_{tv} = 0.5 \text{ in}.$$

The maximum number of longitudinal bars that can be placed along the wall length, accounting for cover and bar size, is

$$n_{\text{bars}} = \frac{L - 2\left(l_c + \frac{d_b}{2} + d_{tv}\right)}{s} + 1 = \frac{409 - 2\left(1.5 + \frac{1.27}{2} + 0.5\right)}{6} + 1 = 68.265 \approx 68 \text{ bars}.$$

The corresponding required area per bar for one curtain is

$$A_{st} = \frac{A_{s,\text{curtain}}}{n_{\text{bars}}} = \frac{41.96 \text{ in}^2}{68} \approx 0.617 \text{ in}^2/\text{bar}.$$

Selecting No. 8 longitudinal bars at 6 in. on center, each providing $A_{bar} = 0.79\ in^2$, satisfies the required area per bar ($A_{bar} > A_{st}$).

Preliminary Steel Reinforcement in the E-W Walls (Flanges) Determining the required steel area A_s in the M_x direction (bending about the horizontal x-axis), using expressions analogous to those adopted for the M_y direction.

The factored design actions for this direction are;

$$M_u = 130{,}419 \text{ k-ft}, \qquad P_u = 1{,}876 \text{ k}.$$

The required nominal axial strength, using a strength reduction factor $\phi = 0.9$, is;

$$P_n = \frac{P_u}{\phi} = \frac{1{,}876 \text{ k}}{0.9} = 2{,}084.44 \text{ k}.$$

The required nominal moment capacity about the x-axis is;

$$M_n = \frac{M_u}{\phi} = \frac{130{,}419 \text{ k-ft}}{0.9} = 144{,}910 \text{ k-ft}.$$

The eccentricity of the axial load with respect to the compression centroid, measured from the section centroid $\bar{Y}$, is;

$$x_e = \frac{409 \text{ in}}{2} - \frac{25 \text{ in}}{2} = 192 \text{ in} = 16 \text{ ft}.$$

The eccentricity of the tensile force associated with the longitudinal reinforcement, again measured from $\bar{Y}$, is;

$$x_s = 409 \text{ in} - 25 \text{ in} = 384 \text{ in} = 32 \text{ ft}.$$

The tension force T_s is given by:

$$T_s = \frac{M_n - P_n(x_e)}{x_s}$$
$$T_s = 3485.98 \text{ k}$$

Required Area of Steel (A_s):

$$A_s = \frac{T_s}{f_y} = 58.10 \text{ in}^2$$

Distributing the required A_s into a practical number and size of bars for the M_x-direction flanges is based on the following assumptions for one flange curtain;

$$A_{s,\text{curtain}} \approx 29.05 \text{ in}^2, \qquad s = 6 \text{ in}, \qquad L \approx 166.5 \text{ in},$$
$$l_c = 1.5 \text{ in}, \qquad d_{b\#10} = 1.27 \text{ in}, \qquad d_{tv\#4} = 0.5 \text{ in}.$$

The maximum number of longitudinal bars that can be placed along the flange length, accounting for concrete cover, bar diameter, and transverse reinforcement, is;

$$n_{\text{bars}} = \frac{L - 2\left(l_c + \frac{d_b}{2} + d_{tv}\right)}{S} + 1 = 27.85 \approx 28 \text{ bars}.$$

The corresponding required area per bar in each curtain is;

$$A_{st} = \frac{A_{s,\text{curtain}}}{n_{\text{bars}}} = \frac{29.05 \text{ in}^2}{28} \approx 1.037 \text{ in}^2/\text{bar}.$$

Selecting No. 10 longitudinal bars at 6 in. on center, each providing $A_{\text{bar}} = 1.27 \text{ in}^2$, satisfies the required area per bar ($A_{\text{bar}} > A_{st}$).

3.2.3 Verification of the Design with PM Interaction Diagram

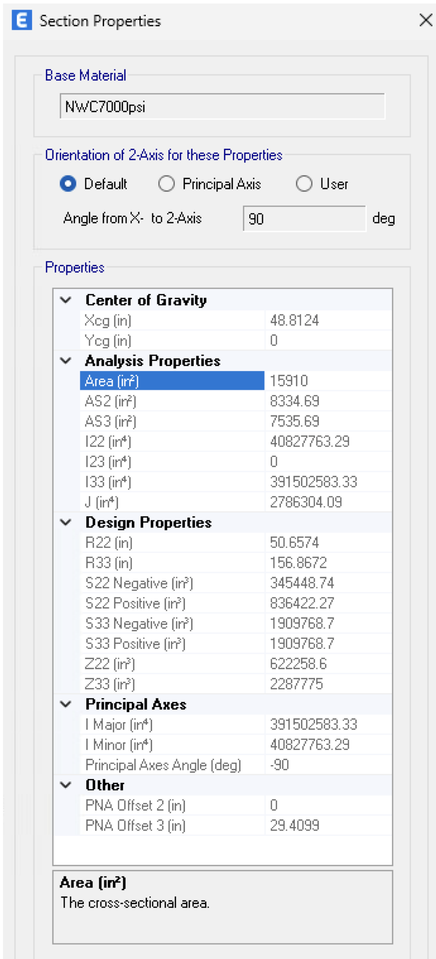
ACSAHE For a first-pass verification of the flexural adequacy of the wall section, the selected load combinations are evaluated against nominal strength interaction curves generated using **ACSAHE**—a self-developed open-source tool authored by one of the team members[7]. ACSAHE generates axial–moment (P–M-M) interaction diagrams for arbitrary reinforced concrete sections following the provisions and assumptions of ACI 318–25 [5]. The tool accommodates arbitrary bar layouts, non-rectangular geometries, and the optional inclusion of prestressing steel. Illustrative examples of typical workflows and outputs are available on the project landing website.[4]

In this assignment, ACSAHE is used to construct section-specific strength envelopes and to compare them directly against the factored demand points associated with the selected extreme flexural loading conditions. This enables rapid iteration and early design calibration prior to full evaluation in ETABS in Section 3.2.3.

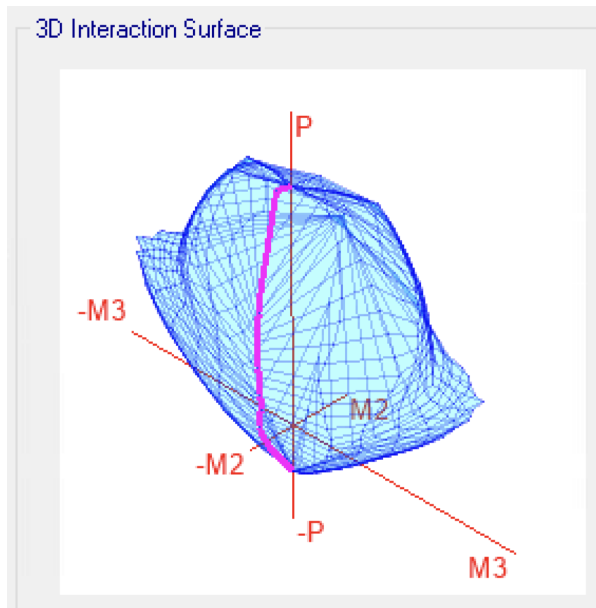
[4] https://facundo-pfeffer.github.io/ACSAHE.github.io/

ETABS An ETABS model was made to perform a section analysis of the core wall. Dimensions of the wall and preliminary rebar sizes were implemented from the initial calculations shown above.

Cross Section and Properties of Preliminary Design:



(a) Section properties of preliminary ETABS model.



(b) 3D view of preliminary design in ETABS.

Figure 3.2.3: Preliminary ETABS model: section properties and 3D view.

Looking closer at the principle planes:

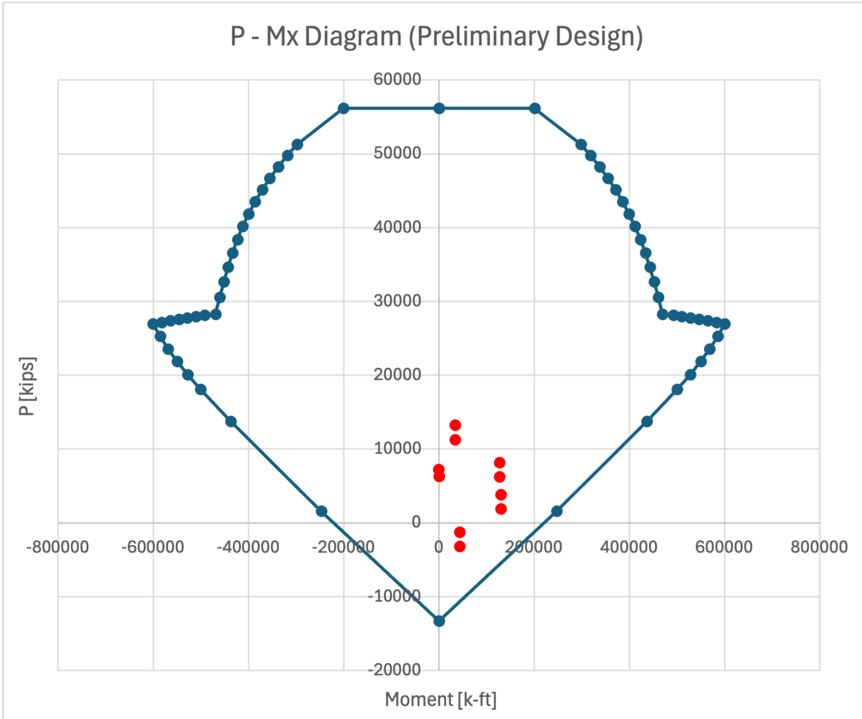


Figure 3.2.4: 0°-180° PM Diagram for core wall section with #10 Bars in the E-W Direction and #8 bars in the N-S Direction. Note Red points represent demands for different load combinations at base story

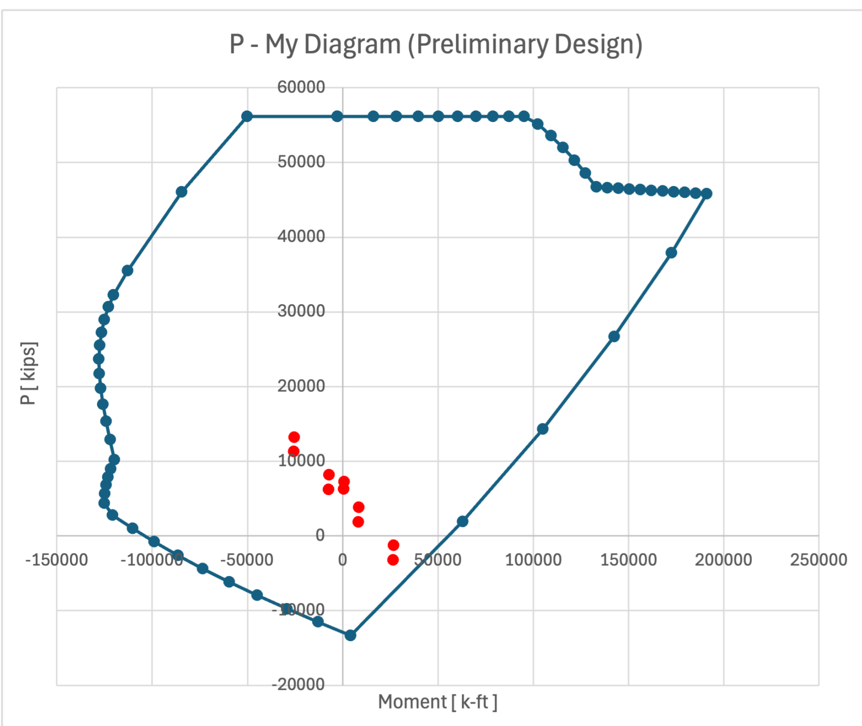


Figure 3.2.5: 90°-270° PM Diagram for core wall section with #10 Bars in the E-W Direction and #8 bars in the N-S Direction. Note Red points represent demands for different load combinations at base story

After analyzing the P-M Diagram for the preliminary section, the reinforcement design can be iterated to produce a more efficient design.

3.2.4 Final Design

After several iterations on the reinforcement layout using both ETABS and ACSAHE, the final design for the C-shaped core wall section consists of:

(i) No. 7 reinforcement bars spanning in the y-direction (web reinforcement).

(ii) No. 9 reinforcement bars placed in the flanges, spanning in the x-direction.

ETABS Verification The P–M diagram results for the final iterations are presented in **Fig. 3.2.6** through **Fig. 3.2.7**, together with all load combinations shown in **Fig. 3.2.1**. Since this only represents a two-dimensional representation of a three-dimensional problem, the 3D plot of the results is computed using ACSAHE and can be found in Section 3.2.4. A URL is provided where the interactive plot can be accessed.

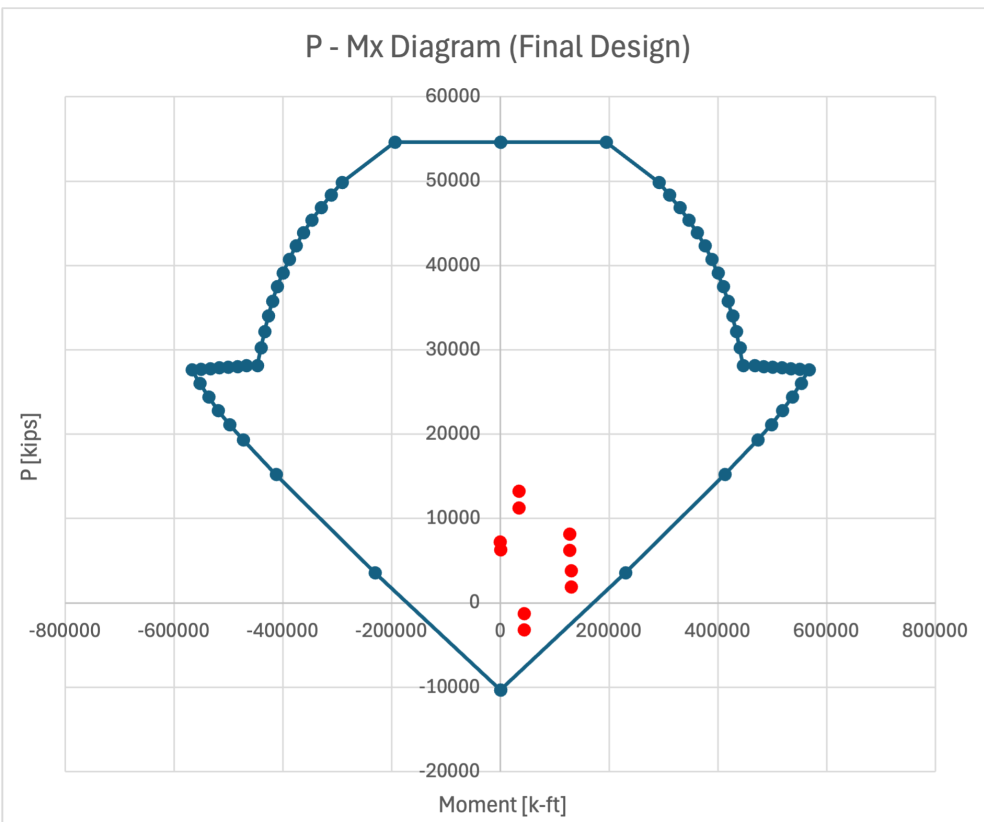


Figure 3.2.6: 0°-180° PM Diagram for core wall section with #9 Bars in the E-W Direction and #7 bars in the N-S Direction. Note Red points represent demands for different load combinations at base story

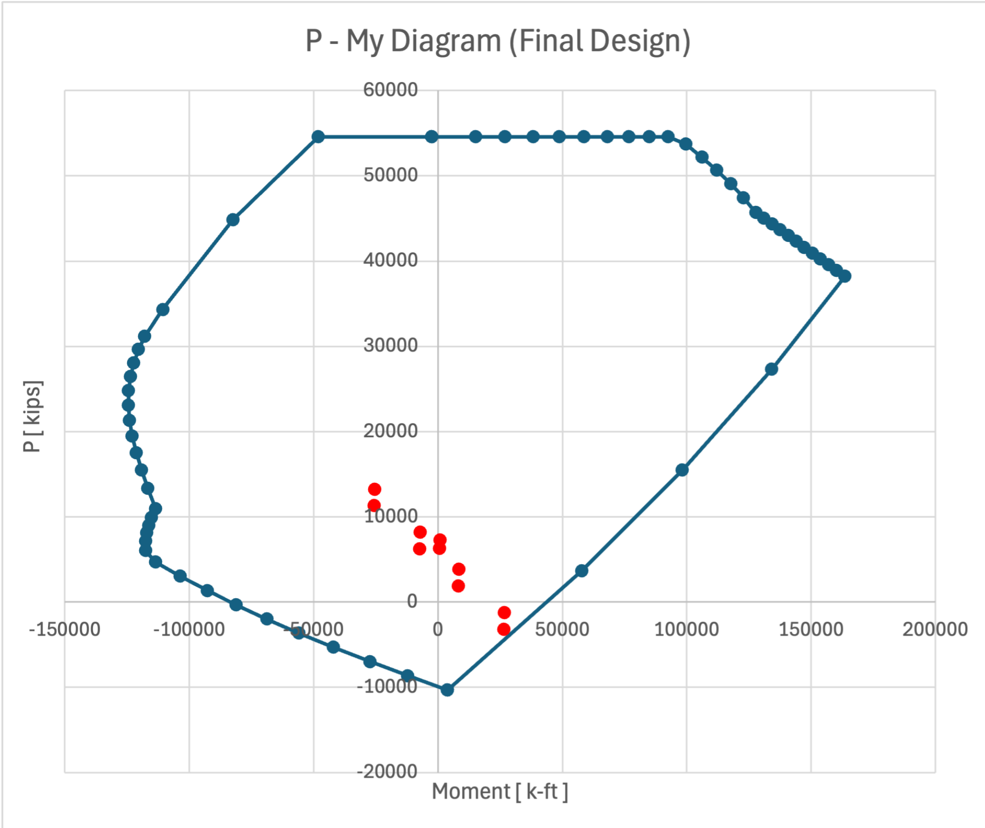


Figure 3.2.7: 90°-270° PM Diagram for core wall section with #9 Bars in the E-W Direction and #7 bars in the N-S Direction. Note Red points represent demands for different load combinations at base story

ACSAHE: Interactive P-M-M-Diagram The biaxial interaction diagram of the concrete cross section can be found in **Fig. 3.2.8**, and is available online at: https://facundo-pfeffer.github.io/ACSAHE.github.io/pages/examples/c_shape.html. The online visualizations allows the user to interact with the diagram and see the full response spectrum. The full report is available in Section B.

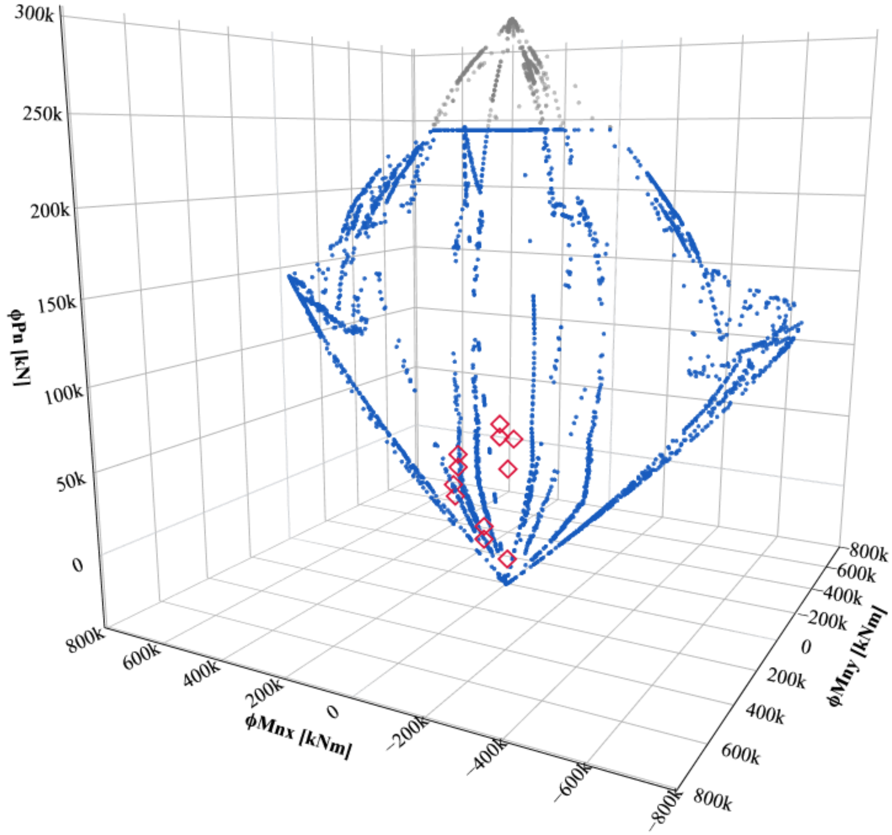
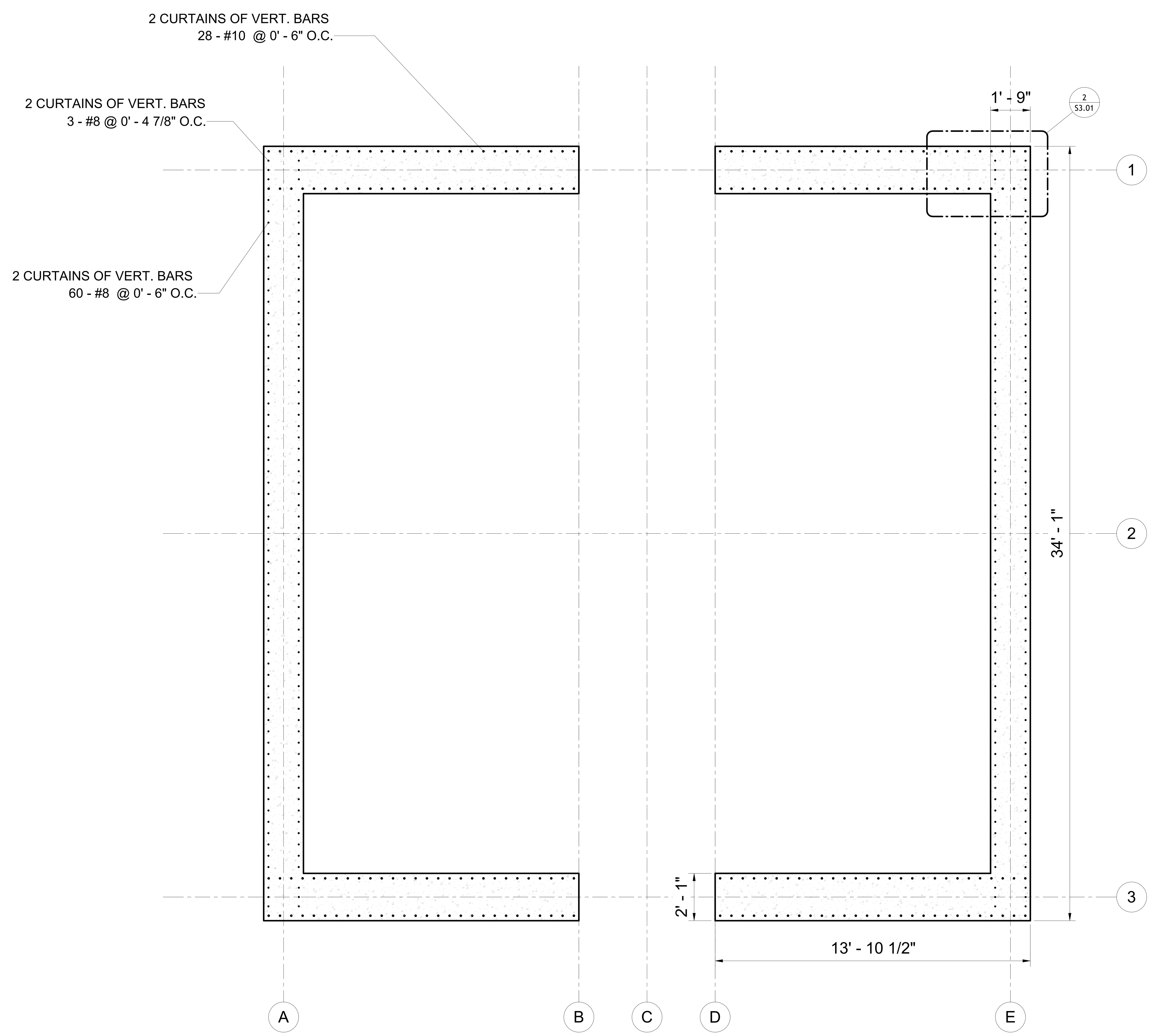
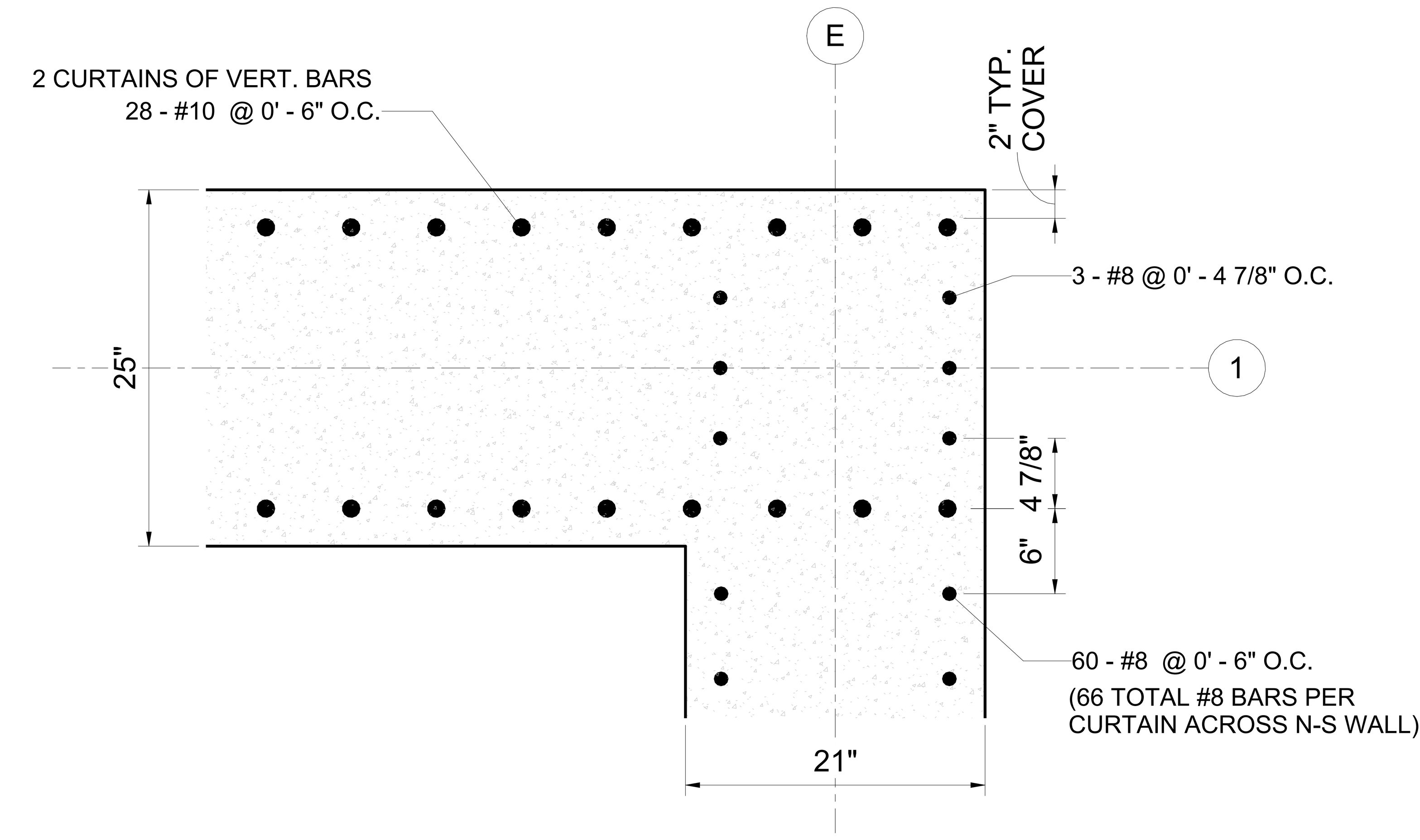


Figure 3.2.8: Final P-M-M diagram for section depicted in **??**. Units are expressed in SI values. Available online for visualization at this link.

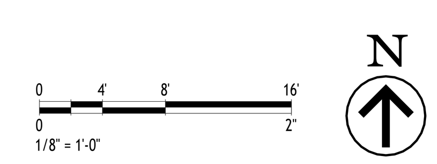
Final Designs Drawings The final cross-section is shown on the following page.



1 Core Wall Section Detail - Plan
3/8" = 1'-0"



2 Core Wall Corner Detail
1 1/2" = 1'-0"



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No.	Description	Date

The City of Berkeley Council

1950 Oxford St.
Residential Building

PRELIMINARY LONG.
REINF. DETAIL

Project Number	CE 244 Semester Project
Date	December 12, 2025
Drawn By	CQ
Checked By	FP, FD, ES

S3.01

Scale: As indicated

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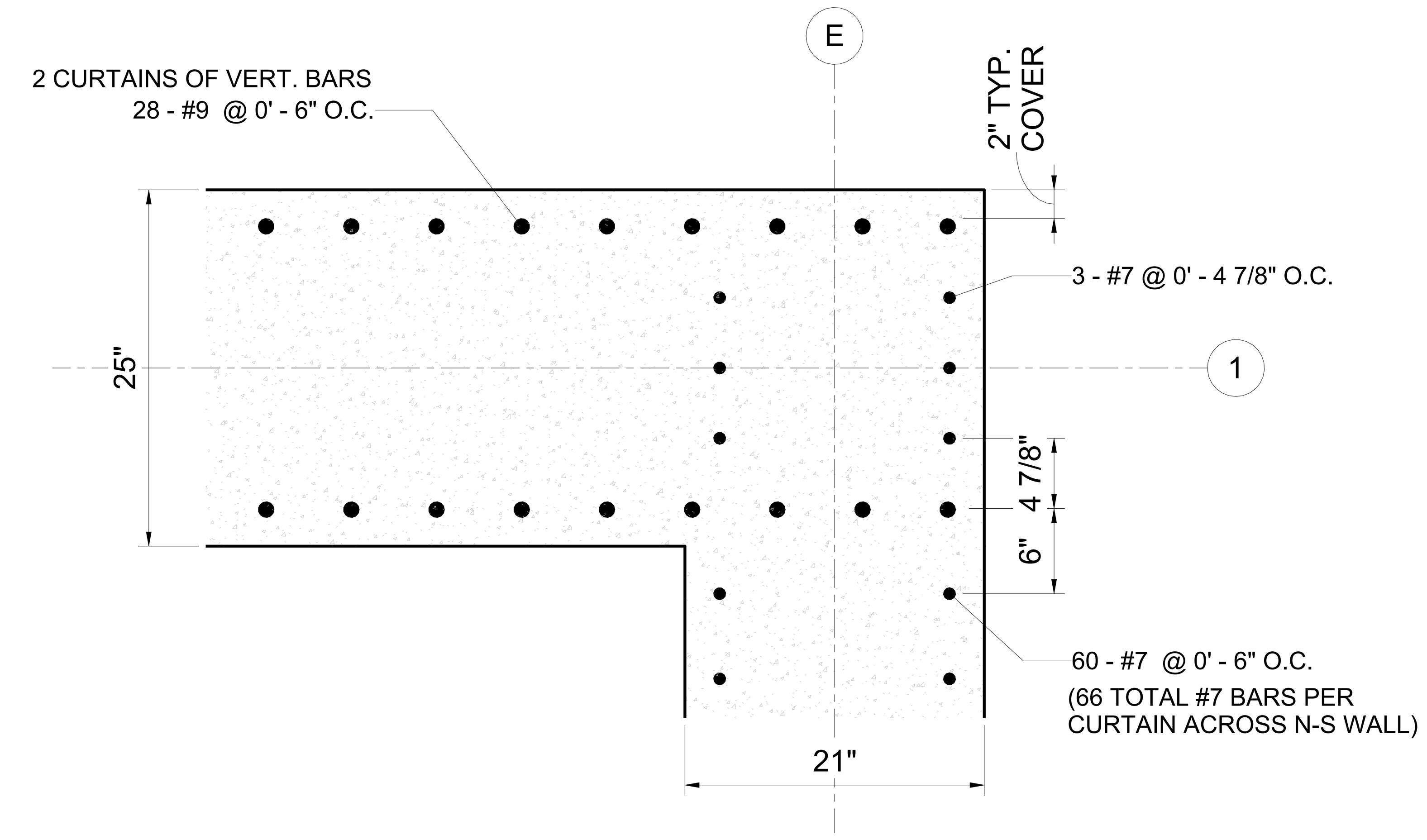
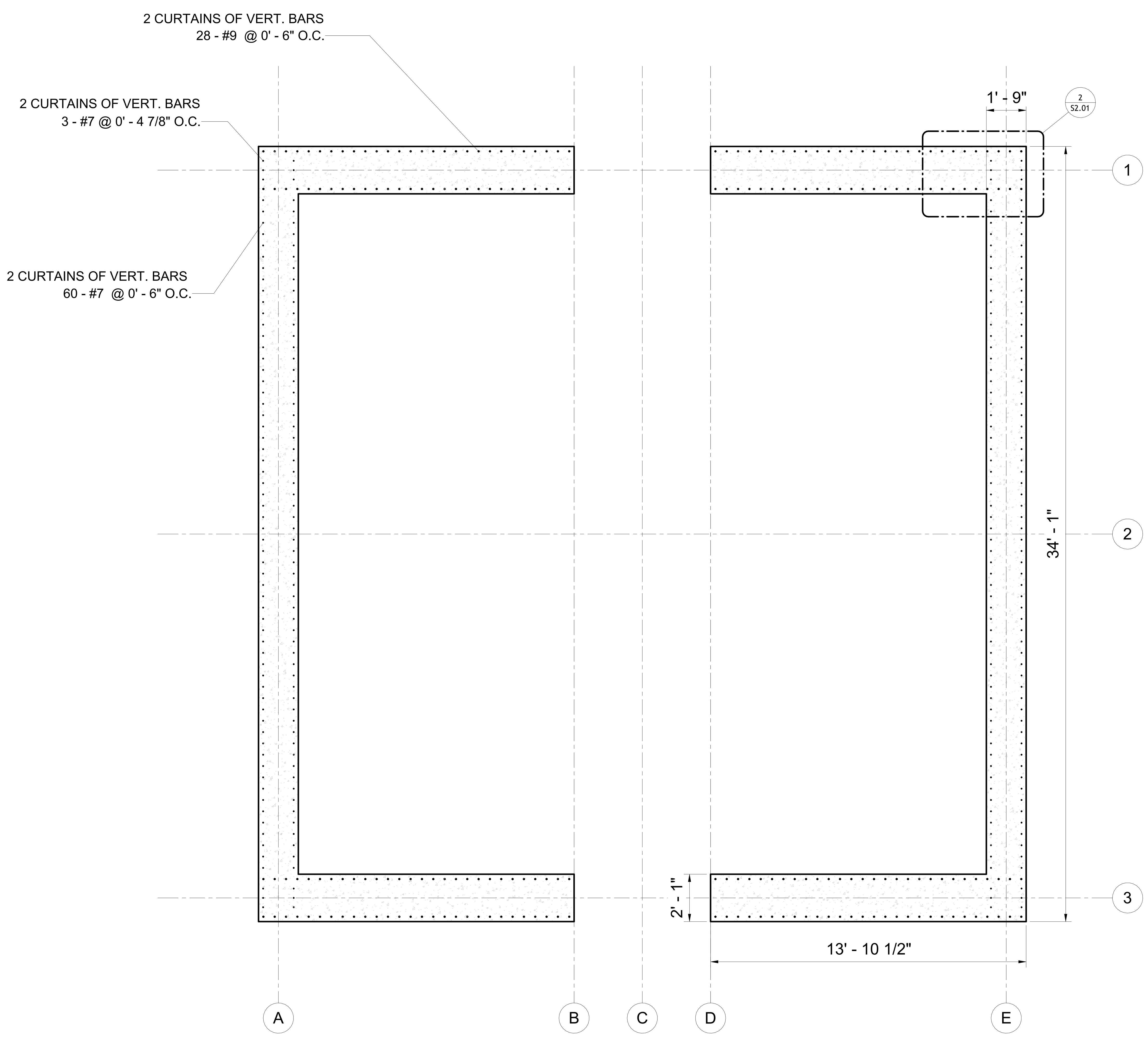


2 Core Wall Corner Detail
1 1/2" = 1'-0"



1 Core Wall Section Detail - Plan
3/8" = 1'-0"

N

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No.	Description	Date

The City of Berkeley Council

1950 Oxford St.
Residential Building

FINAL LONGITUDINAL
REINF. DETAIL

Project Number	CE 244 Semester Project
Date	December 12, 2025
Drawn By	CQ
Checked By	FP, FD, ES

S2.01

Scale: As indicated

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3.3 Wall Transverse Shear Reinforcement Design

The design of transverse shear reinforcement for the structural walls follows the requirements of ACI 318-25 Chapter 18[5]. The procedure consists of three main steps. First, in Section 3.3.1 the factored story shears obtained from structural analysis[1] are amplified in accordance with Section 18.10.3, where the overstrength factor Ω_v approximates the flexural overstrength at the wall critical section and the dynamic factor ω_v accounts for higher–mode amplification that is not directly captured in linear analysis. These amplified design shears form the governing demands for the wall critical sections. Second, in Section 3.3.2 the concrete shear limits are checked to ensure the wall dimensions are adequate. Third, in Section 3.3.3 the required transverse reinforcement ratio is established using the nominal shear strength expression of Section 18.10.4.1, with all parameters defined consistently with the wall geometry and material properties. The following subsections detail the computation of amplified shear demands and the subsequent determination of the required transverse reinforcement.

3.3.1 Amplified Shear Demands

For special structural walls, ACI 318-25 Section 18.10.3 defines an amplified design shear force that accounts for flexural overstrength and dynamic higher–mode effects. The design shear force is taken as:

$$V_e = \Omega_v \, \omega_v \, V_u \le 3V_u, \tag{3.3.1}$$

where:

- Ω_v is an overstrength factor intended to reflect flexural overstrength at the critical section.
- ω_v is a dynamic shear amplification factor associated with higher–mode effects.
- V_u is the factored story shear from structural analysis.

The overstrength factor Ω_v is obtained from Table 18.10.3.3[5]. Copied here for convenience:

Condition (h_w/l_w)	$\boldsymbol{\Omega_v}$	$\boldsymbol{\omega_v}$
$h_w/l_w \le 1.0$	1.0	1.0
$1.0 < h_w/l_w < 2.0$	Linear interpolation	Permitted between 1.0 and 1.5
$h_w/l_w \ge 2.0$	1.5	$0.8 + 0.09h_w^{1/3} \ge 1.0$

Table 3.3.1: Table 18.10.3.3 from ACI 318-25[5]. Factors Ω_v and ω_v

.

For walls with $h_{wcs}/\ell_w \ge 2.0$, the table specifies:

$$\Omega_v = 1.5, \qquad \omega_v = 0.8 + 0.09\,h_n^{1/3} \ge 1.0,$$

where h_w is the total height of wall above the critical section and h_n is the building height in feet (or the equivalent number of stories) used in the dynamic amplification expression[5].

For the building in this assignment, the height parameter $h_n = 119$ ft from the project data gives:

$$\omega_v = 0.8 + 0.09\,h_n^{1/3} \approx 1.24 \ge 1.0,$$

[5]Note that for the studied structure, $h_n = h_w$ because the wall cross section geometry does not change across the entire building height.

which is the value reported as $w_v = 1.24$ in the design spreadsheet. Because the product $\Omega_v \omega_v \approx 1.5 \times 1.24 = 1.86 < 3.0$, the upper limit $3V_u$ in (3.3.1) does not control.

E–W Direction Story shears and wall dimensions are:

$$V_x = 1700 \text{ kips}, \qquad V_y = 932 \text{ kips},$$

$$\ell_w = 333 \text{ in}, \qquad h_w = 1428 \text{ in}, \qquad \frac{h_w}{\ell_w} = 4.29.$$

The aspect ratio $h_{wcs}/\ell_w = 4.29 \geq 2.0$ places the wall in the third row of Table 18.10.3.3, so:

$$\Omega_v = 1.5, \qquad \omega_v \approx 1.24.$$

The amplified shear in each direction follows directly from (3.3.1):

$$V_{e,x} = \Omega_v \, \omega_v \, V_x = (1.5)(1.24)(1700) = 3,169 \, \text{kip}$$

Similarly, for the orthogonal horizontal direction:

$$V_{e,y} = \Omega_v \, \omega_v \, V_y = (1.5)(1.24)(932) = 1737.27 \text{ kips},$$

These values serve as the demand shears for the E–W wall in the subsequent shear-reinforcement design, with $V_{e,x}$ taken as the governing demand.

N–S Direction For the N–S wall, story shears and dimensions are:

$$V_x = 524 \text{ kips}, \qquad V_y = 2076 \text{ kips},$$

$$\ell_w = 409 \text{ in}, \qquad h_w = 1428 \text{ in}, \qquad \frac{h_w}{\ell_w} = 3.49.$$

Again, since $h_w/\ell_w \geq 2.0$:

$$\Omega_v = 1.5, \qquad \omega_v = 1.24.$$

The amplified shears become:

$$V_{e,x} = \Omega_v \, \omega_v \, V_x = (1.5)(1.24)(524) = 976.75 \text{ kips},$$

and:

$$V_{e,y} = \Omega_v \, \omega_v \, V_y = (1.5)(1.24)(2076) = 3,869.71 \text{ kips}.$$

These amplified values constitute the design shears for the N–S wall in the transverse reinforcement calculations that follow in Section 3.3.3 with $V_{e,y}$ taken as the governing demand.

3.3.2 Concrete Shear Limit Checks

ACI 318-25 Section 18.10.4.4 imposes upper limits on the nominal shear strength that may be assigned to individual wall segments and to the entire wall system. As explained in Commentary R18.10.4.4, these limits are intended to control the degree of shear-force redistribution among parallel wall segments. When several walls or several vertical wall segments resist the factored shear at a given story, the average unit shear strength for the total available cross-sectional area may not exceed $\alpha_{sh} 8\sqrt{f'_c}$, and the unit shear strength assigned to any single wall segment may not exceed $\alpha_{sh} 10\sqrt{f'_c}$. These limits ensure that the design does not assign an unrealistically large portion of the story shear to a single segment when multiple vertical wall elements act in parallel. All checks in this subsection adopt $\alpha_{sh} = 1.0$ as permitted by ACI 318-25.

For an individual wall segment, the maximum permitted design shear strength is taken as:

$$\phi V_{n,\text{max,seg}} = \phi\, \alpha_{sh}\, 10\sqrt{f'_c}\, A_{cv,\text{seg}},$$

and for the entire wall system in a given direction:

$$\phi V_{n,\text{max,tot}} = \phi\, \alpha_{sh}\, 8\sqrt{f'_c}\, A_{cv,\text{tot}}.$$

In each case, the limit is verified by checking that $\phi V_{n,\text{max}} \geq V_u$.

E–W Shear Limit Checks

Individual Wall Segment (Maximum Shear Strength) The amplified design shear on a single E–W wall segment is $V_u = 3{,}169$ kips. The corresponding concrete shear limit is:

$$\begin{aligned}
\phi V_{n,\text{max,seg}} &= \phi\, \alpha_{sh}\, 10\sqrt{f'_c}\, A_{cv,\text{seg}} \\
&= 0.75(1.0)\, 10\sqrt{7{,}000 \text{ psi}}\, \left(2 \times 166.5\text{in} \times 25\text{in}\right) \\
&\approx 5{,}224 \text{ kips.}
\end{aligned}$$

Thus:

$$5{,}224 \text{ kips} \geq 3{,}169 \text{ kips} \quad (\text{OK}).$$

All Wall Segments Combined (Total Shear Strength) The total amplified shear demand in the E–W direction is $V_u = 6{,}338$ kips. The corresponding wall-system limit is:

$$\begin{aligned}
\phi V_{n,\text{max,tot}} &= \phi\, \alpha_{sh}\, 8\sqrt{f'_c}\, A_{cv,\text{tot}} \\
&= 0.75(1.0)\, 8\sqrt{7{,}000 \text{ psi}}\, \left(2 \times 166.5\text{in} \times 25\text{in}\right) \times 2 \\
&\approx 8{,}358.2 \text{ kips.}
\end{aligned}$$

Therefore:

$$8{,}358.2 \text{ kips} \geq 6{,}338 \text{ kips} \quad (\text{OK}).$$

N–S Shear Limit Checks

Individual Wall Segment (Maximum Shear Strength) For the N–S wall, the amplified shear on a single segment is $V_u = 3{,}869.71$ kips. The corresponding concrete shear limit is:

$$\begin{aligned}\phi V_{n,\text{max,seg}} &= \phi\,\alpha_{sh}\,10\sqrt{f'_c}\,A_{cv,\text{seg}}\\ &= 0.75(1.0)\,10\sqrt{7{,}000\text{ psi}}\,\big(409\text{in}\times 21\text{in}\big)\\ &\approx 5{,}389.6\text{ kips}.\end{aligned}$$

Thus:

$$5{,}389.6\text{ kips} \geq 3{,}869.71\text{ kips} \quad (\text{OK}).$$

All Wall Segments Combined (Total Shear Strength) The total amplified shear demand in the N–S direction is $V_u = 7{,}739.42$ kips. The corresponding wall-system limit is:

$$\begin{aligned}\phi V_{n,\text{max,tot}} &= \phi\,\alpha_{sh}\,8\sqrt{f'_c}\,A_{cv,\text{tot}}\\ &= 0.75(1.0)\,8\sqrt{7{,}000\text{ psi}}\,\big(409\text{in}\times 21\text{in}\big)\times 2\\ &\approx 8{,}623.3\text{ kips}.\end{aligned}$$

Therefore:

$$8{,}623.3\text{ kips} \geq 7{,}739.42\text{ kips} \quad (\text{OK}).$$

3.3.3 Reinforcement Design

Having verified in Section 3.3.2 that the concrete shear strength limits of ACI 318-25 Section 18.10.4.4 are satisfied for both wall directions, the design of transverse shear reinforcement proceeds using the nominal shear strength expression of ACI 318-25 Section 18.10.4.1[5]. According to this provision, the nominal shear strength of a special structural wall may be taken as:

$$V_n = \big(\alpha_c\,\lambda\sqrt{f'_c} + \rho_t f_{yt}\big)\,A_{cv}, \tag{3.3.2}$$

where $\alpha_c = 2$ for $h_w/\ell_w \geq 2$, $\lambda = 1$ for normal-weight concrete, f'_c is in psi, f_{yt} is in psi, ρ_t is the transverse reinforcement ratio, and A_{cv} is the effective shear area of the wall. The design condition is:

$$\phi V_n \geq V_u, \qquad \phi = 0.75. \tag{3.3.3}$$

Solving (3.3.2) for the required transverse reinforcement ratio yields:

$$\rho_{t,\text{req}} = \frac{1}{f_y}\left(\frac{V_u}{\phi\,A_{cv}} - \alpha_c\,\lambda\sqrt{f'_c}\right). \tag{3.3.4}$$

ACI 318 specifies a minimum transverse reinforcement ratio $\rho_{t,\text{min}} = 0.0025$, requirement that holds if V_u does not exceed $\lambda\sqrt{f'_c}A_{cv}$ as per 18.10.2.1. If this condition is met, the requirement turns to:

$$\rho_t \geq \max\big(\rho_{t,\text{req}},\ \rho_{t,\text{min}}\big). \tag{3.3.5}$$

E–W Wall For the E–W wall, the following design data apply:

$$V_u = 3168840 \text{ lb}, \qquad f'_c = 7000 \text{ psi}, \qquad f_y = 60 \text{ ksi} = 60000 \text{ psi}.$$

The wall is classified as slender in shear, so $\alpha_c = 2$. The effective shear area is taken as[5]:

$$A_{cv} = t_w\, \ell_{w,\text{eff}} = (25 \text{ in})(2)(166.5 \text{ in}) = 8325 \text{ in}^2.$$

Substituting these values in (3.3.4) gives

$$\begin{aligned}\rho_{t,\text{req}} &= \frac{1}{60000}\left(\frac{3168840}{0.75 \times 8325} - 2 \times 1 \times \sqrt{7000}\right)\\ &\approx 0.00567.\end{aligned}$$

The code minimum is

$$\lambda\sqrt{f'_c}A_v = 696.5\,\text{kip} < V_u \quad \Rightarrow \quad \rho_{t,\min} = 0.0025 < \rho_{t,\text{req}},$$

so the required ratio is governed by the shear demand:

$$\rho_t = 0.00567.$$

The transverse reinforcement ratio is related to the area of transverse steel within spacing s by

$$\rho_t = \frac{A_{v,\text{req}}}{s\, t_w}, \tag{3.3.6}$$

so for a chosen spacing $s = 5$ in,

$$\begin{aligned}A_{v,\text{req}} &= \rho_t\, t_w\, s = 0.00567 \times 25 \text{ in} \times 5 \text{ in}\\ &\approx 0.71 \text{ in}^2.\end{aligned}$$

The wall is reinforced with two curtains of transverse bars. Distributing the required steel equally between the two curtains leads to

$$A_{s,\text{req}} = \frac{A_{v,\text{req}}}{2} \approx \frac{0.71}{2} \approx 0.355 \text{ in}^2 \quad \text{per curtain at 5 in o.c.}$$

Adopting No. 6 bars with $A_s = 0.44$ in^2 in each curtain at 5 in on center provides

$$A_s = 0.44 \text{ in}^2 > 0.355 \text{ in}^2,$$

which satisfies the shear reinforcement requirement for the E–W wall.

N–S Wall For the N–S wall, the design values are:

$$V_u = 3869710 \text{ lb}, \qquad f'_c = 7000 \text{ psi}, \qquad f_y = 60000 \text{ psi},$$

With the same $\alpha_c = 2$ and $\lambda = 1$. The effective shear area is taken as:

$$A_{cv} = t_w\, \ell_{w,\text{eff}} = 21 \text{ in} \times 409 \text{ in} = 8589 \text{ in}^2.$$

Substitution into (3.3.4) yields

$$\begin{aligned}\rho_{t,\text{req}} &= \frac{1}{60000}\left(\frac{3869710}{0.75 \times 8589} - 2 \times 1 \times \sqrt{7000}\right)\\ &\approx 0.00717.\end{aligned}$$

Again,

$$\lambda\sqrt{f_c'}A_v = 71.80\,\text{kip} \quad \Rightarrow \quad \rho_{t,\min} = 0.0025 < \rho_{t,\text{req}},$$

so

$$\rho_t = 0.00717.$$

Using the same bar spacing $s = 5$ in,

$$\begin{aligned}A_{v,\text{req}} &= \rho_t\, t_w\, s = 0.00717 \times 21 \text{ in} \times 5 \text{ in}\\ &\approx 0.75 \text{ in}^2.\end{aligned}$$

With two curtains,

$$A_{s,\text{req}} = \frac{A_{v,\text{req}}}{2} \approx \frac{0.75}{2} \approx 0.376 \text{ in}^2 \quad \text{per curtain at 5 in o.c.}$$

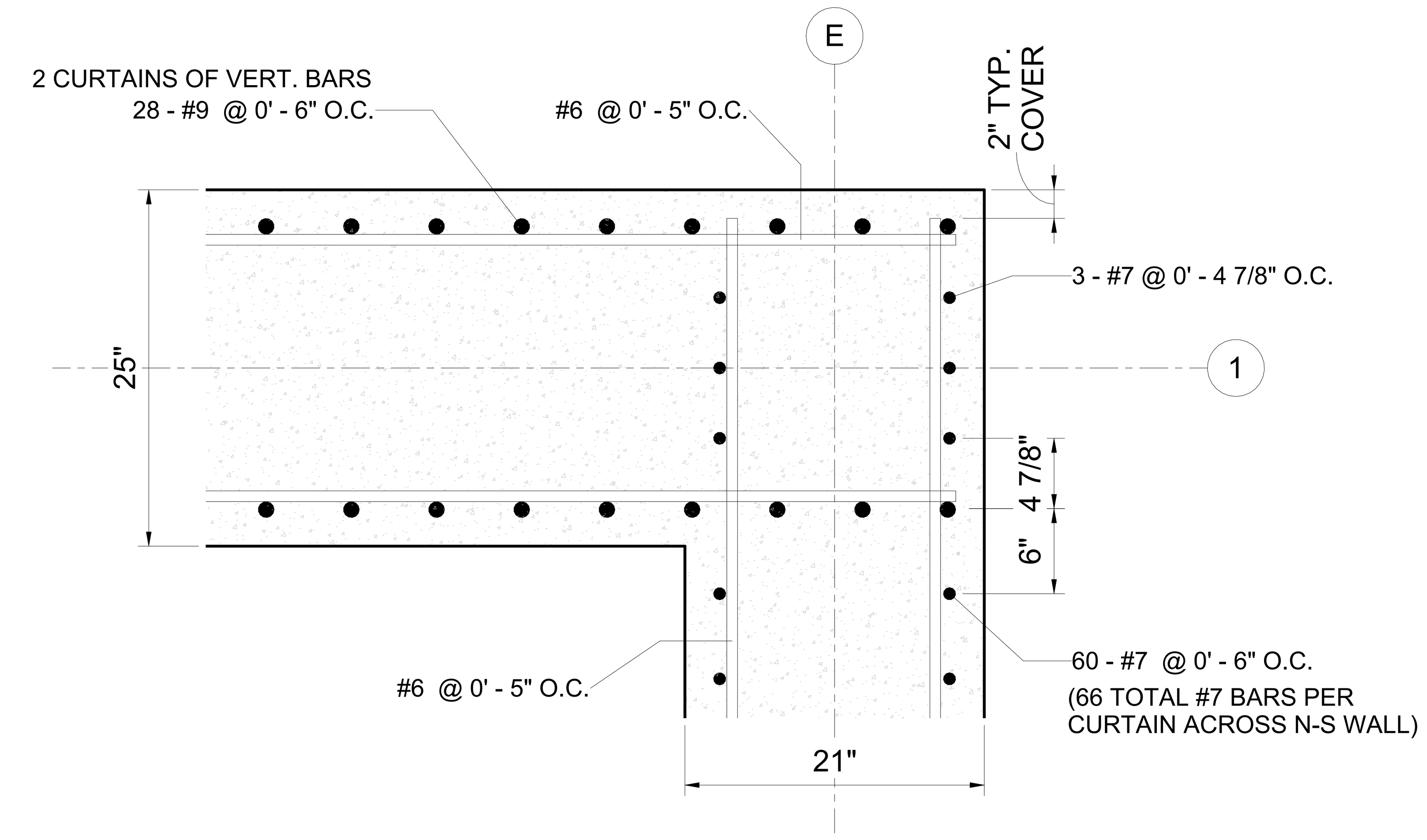
Selecting No. 6 bars with $A_s = 0.44$ in^2 in each curtain at 5 inches on center provides

$$A_s = 0.44 \text{ in}^2 > 0.376 \text{ in}^2,$$

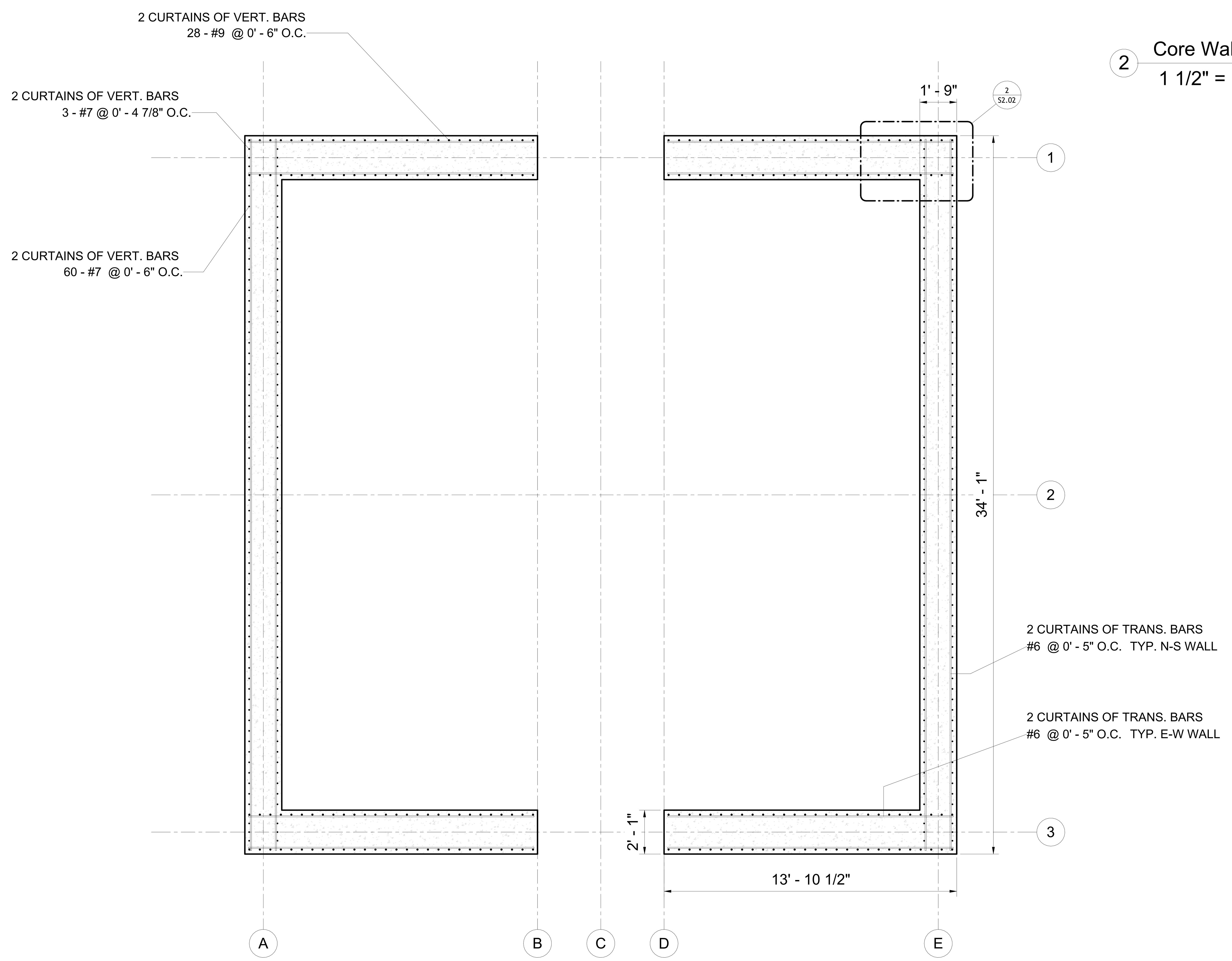
which satisfies the shear reinforcement requirement for the N–S wall.

3.3.4 Final Designs Drawings

The final core wall cross-section is shown on the following page.



2 Core Wall Corner Detail
1 1/2" = 1'-0"



1 Core Wall Section Detail - Plan
3/8" = 1'-0"

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No.	Description	Date

The City of Berkeley Council

1950 Oxford St.
Residential Building

FINAL TRANSVERSE
(SHEAR) REINF. DETAIL

Project Number	CE 244 Semester Project
Date	December 12, 2025
Drawn By	CQ
Checked By	FP, FD, ES

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3.4 Coupling Beam Design

The objective of this section is to establish the dimensions and reinforcement of the coupling beams in the lateral-force-resisting system. Because the plan geometry, support conditions, and material properties are symmetric (see **Fig. 1.0.1**), designing a single representative beam is sufficient. For the purposes of this assignment, only the coupling beam at Floor #5 is designed. The provisions of ACI 318-25[5] governing coupling beam design are adopted throughout Section 3.4.

3.4.1 Design Shear Force: Redistribution

Section 18.10.7.5 of ACI 318-25[5] permits redistribution of shear forces among coupling beams at adjacent floor levels. This is allowed because beams designed in accordance with Section 18.10.7 possess significant plastic rotational capacity and are expected to act as primary yielding mechanisms[5, 12, 9]. In addition to better reflecting expected inelastic behavior, this redistribution can lead to more economical and constructible detailing.

Shear redistribution is permitted when the following requirements are satisfied[5]:

(a) Coupling beams sharing redistributed forces are vertically aligned within a special structural wall.

(b) Coupling beams sharing redistributed forces have $\ell_n/h \geq 2$.

(c) The maximum redistribution of V_e from any beam does not exceed 20% of the value obtained from analysis.

(d) The sum of ϕV_n of the coupling beams sharing redistributed demands is greater than or equal to the sum of V_e in those beams.

Requirement (a) is satisfied by the building configuration, requirement (b) is satisfied using **Eq. (3.4.1)** while requirements (c) and (d) are verified by the calculations summarized in **Table 3.4.1**.

Level	ETABS CB shear [kip]	Redistributed CB shears [kips]	Relative Change [%]	Diff.[kips]
Roof	181.00	181.00	0.00%	-158.50
8	290.00	339.50	17.07%	0.00
7	389.00	339.50	-12.72%	-177.50
6	479.00	517.00	7.93%	0.00
5	555.00	517.00	-6.85%	-116.50
4	617.00	633.50	2.67%	0.00
3	661.00	633.50	-4.16%	0.00
2	670.00	633.50	-5.45%	0.00
1	586.00	633.50	8.11%	0.00
SUM	**4428.00**	**4428.00**	–	–

Table 3.4.1: Coupling beam shear redistribution along building height.

Four different values for the design shear can be identified in **Table 3.4.1**. However, only the second group corresponding to floors #5 and #6 is used in the following sections, as requested for academic purposes.

3.4.2 Beam-type checks and need for a coupling beam design

The first step consists in evaluating the clear-span-to-beam-height ratio for the coupling beam; the clear span is $l_n = 72$ in and the beam height is $h = 36$ in. The length-to-height ratio is therefore:

$$\frac{l_n}{h} = \frac{72\text{ in}}{36\text{ in}} = 2.0. \tag{3.4.1}$$

Section 18.10.7.2 of ACI 318–25[5] specifies that a coupling beam must be designed as a special coupling beam when both of the following conditions are satisfied: (i) $l_n/h<2$ and (ii) the design shear V_u is at least $4\lambda\sqrt{f'_c}\,A_{cw}$, where λ is the lightweight concrete modification factor, f'_c is the specified concrete compressive strength, and A_{cw} is the net vertical area of the coupling-beam cross section. The demand shear on the beam is $V_u = 517$ kip, the concrete strength is $f'_c = 7000$ psi, the modification factor is taken as $\lambda = 1.0$, and the effective area is $A_{cw} = 25\text{ in} \times 36\text{ in}$. The corresponding shear threshold is:

$$4\lambda\sqrt{f'_c}\,A_{cw} = 4(1.0)\,\sqrt{7000\text{ psi}}\,(25\text{ in})(36\text{ in}) \approx 301.2\text{ k}.$$

Since $l_n/h = 2.0 \leq 4$ and $V_u = 517\text{ k} \geq 301.2\text{ k}$, the member is on the limit of the conditions imposed by ACI 318–25 18.10.7.2.

A visual representation of this requirement is defined in terms of the normalized shear demand $V_u/(A_{cw}\sqrt{f'_c})$ and the span-to-depth ratio l_n/h. The denominator of this ratio is first obtained:

$$A_{cw}\sqrt{f'_c} = (25\text{ in})(36\text{ in})\,\sqrt{7000\text{ psi}} \approx 75.3\text{ k},$$

and the normalized shear demand becomes:

$$\frac{V_u}{A_{cw}\sqrt{f'_c}} = \frac{517\text{ k}}{75.3\text{ k}} \approx 6.87.$$

The design point corresponding to this beam is therefore $(l_n/h,\ V_u/(A_{cw}\sqrt{f'_c})) \approx (2.0,\ 6.87)$, which falls in the "diagonally reinforced" region of the coupling-beam design space (see **Fig. 3.4.1**). This graphical check is consistent with the code trigger above and confirms that the member must be detailed as a diagonally reinforced special coupling beam.

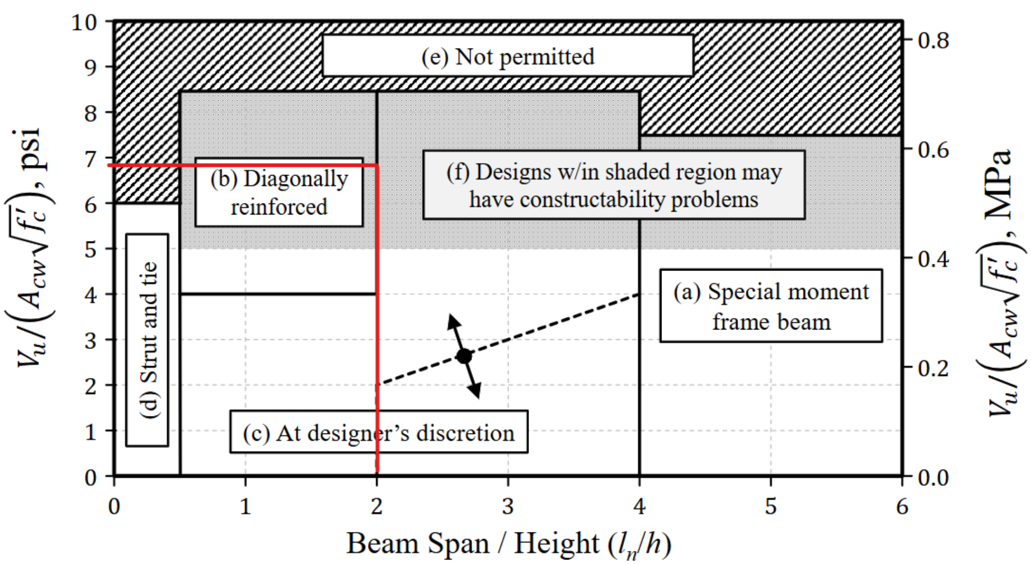


Figure 3.4.1: Coupling beam design space per ACI 318–25 18.10.7.3.

3.4.3 Design of Diagonal Reinforcement per ACI 318–25 18.10.7.4

The design of the diagonal reinforcement starts with verifying that the factored shear demand satisfies the upper limit permitted for diagonally reinforced coupling beams. Section 18.10.7.4 limits the nominal shear strength to $V_n \leq 10\sqrt{f'_c}\,A_{cw}$. Using $f'_c = 7000$ psi and $A_{cw} = 25$ in $\times$ 36 in, the expression becomes:

$$10\sqrt{f'_c}\,A_{cw} = 10\sqrt{7000}\,(25\,\text{in})(36\,\text{in}) \approx 753\text{ k}.$$

Since the demand is $V_u = 517$ k (see **Table 3.4.1**), the limit is satisfied.

The nominal shear capacity of the diagonally reinforced mechanism is expressed as:

$$V_u = \phi V_{n,req} = \phi\, 2\, A_{vd,req}\, f_y\, \sin\alpha,$$

where $A_{vd,req}$ is the total required area of the diagonal reinforcement in one direction, f_y is the yield stress of the diagonal bars, and α is the diagonal inclination measured from the beam axis. Solving for the required diagonal reinforcement area introduces:

$$A_{vd,\text{req}} := \frac{V_u}{\phi\, 2\, f_y\, \sin\alpha}.$$

Substituting the values $V_u = 517$ k, $\phi = 0.85$, $f_y = 60$ ksi, and $\alpha = \tan^{-1}\left(\frac{11\text{ in}}{36\text{ in}}\right) \approx 17°$, the required area becomes:

$$A_{vd,\text{req}} = \frac{517\text{ k}}{0.85\,(2)(60\text{ ksi})\,\sin(17°)} \approx 17.34\text{ in}^2.$$

A practical reinforcement layout is achieved by selecting commercially available bar sizes whose total area meets or exceeds the required value. A possible arrangement satisfying the requirement is illustrated below:

$$12\ \#11\text{ bars} \;\Rightarrow\; A \approx 18.72\text{ in}^2,$$

3.4.4 Development Length of Diagonal Reinforcement

The development length of the diagonal reinforcement, l_{dh}, is evaluated as per ACI 318–25, Section 25.4.2.3[5]:

$$l_{dh} = \left(\frac{f_y\,\psi_t\,\psi_e\,\psi_s}{20\,\lambda\,\sqrt{f'_c}}\right) d_b.$$

The modification factors follow the definitions of Table 25.4.2.5:

- $\lambda = 1.0$. Normal-weight concrete.
- $\psi_t = 1.0$. Bar is not in a top-cast position.
- $\psi_e = 1.0$. Uncoated reinforcement.
- $\psi_s = 1.0$. No. 7 and larger bars.
- $f_y = 1.25\,f_y = 1.25(60,000\text{ psi})$. Bars for coupling beams should be developed for $1.25 f_y$ as per 18.10.2.5(b).

- $d_b = 1.41$ in. Diameter of the diagonal bar.

Introducing these values into the development-length expression gives:

$$l_{dh} = \left(\frac{1.25(60{,}000)\,(1)(1)(1)}{20(1.0)\sqrt{7000}} \right)(1.41 \text{ in}) = 63.2 \text{ in} \xrightarrow{\text{Round up}} 64 \text{ in} = 5.33 \text{ ft}.$$

To confirm that the diagonal reinforcement can be fully developed within each E–W wall, the horizontal projection of the required development length is checked:

$$l_{dh} \cos\alpha = 64 \text{ in } \cos(17^\circ) \approx 61.2 \text{ in} < l_{\text{E–W}} = 166.5 \text{ in} \qquad \text{OK} \checkmark$$

This verifies that the available embedment length in the wall is sufficient to fully develop the diagonal bars.

3.4.5 Transverse Bar Requirements per ACI 318–25, 18.10.7.4

ACI 318–25 18.10.7.4 requires the transverse reinforcement area A_{sh} to satisfy the larger of the following two expressions (evaluated with an assumed spacing $s = 6$ in):

$$A_{sh} \geq \begin{cases} 0.09\, s\, b_c\, \dfrac{f'_c}{f_{yt}}, \\ 0.3\, s\, b_c \left(\dfrac{A_g}{A_{ch}} - 1 \right) \dfrac{f'_c}{f_{yt}}. \end{cases}$$

Vertical Legs Along Beam Width Requirement (i):

$$A_{sh}^{(i)} = 0.09(6)(22)\frac{7 \text{ ksi}}{60 \text{ ksi}} = 1.386 \text{ in}^2.$$

Requirement (ii):

$$A_{sh}^{(ii)} = 0.3(6)(22)\left(\frac{36 \times 25}{33 \times 22} - 1 \right) \frac{7 \text{ ksi}}{60 \text{ ksi}} = 1.11 \text{ in}^2.$$

The governing requirement is therefore:

$$A_{sh,\text{req}} = \max\{A_{sh}^{(i)}, A_{sh}^{(ii)}\} = 1.386 \text{ in}^2.$$

Assuming five transverse legs across the section, the required bar area per leg is:

$$A_{\text{bar,req}} := \frac{1.386 \text{ in}^2}{5} \approx 0.277 \text{ in}^2.$$

A No. 5 bar provides:

$$A_{\#5} = 0.31 \text{ in}^2 > A_{\text{bar,req}},$$

so five No. 5 transverse legs satisfy the transverse reinforcement requirements.

Horizontal Legs Along Beam Height Requirement (i):

$$A_{sh}^{(i)} = 0.09(6)(33)\frac{7\text{ ksi}}{60\text{ ksi}} = 2.079\text{ in}^2.$$

Requirement (ii):

$$A_{sh}^{(ii)} = 0.3(6)(33)\left(\frac{36 \times 25}{33 \times 22} - 1\right)\frac{7\text{ ksi}}{60\text{ ksi}} = 1.66\text{ in}^2.$$

The governing requirement is therefore:

$$A_{sh,\text{req}} = \max\{A_{sh}^{(i)}, A_{sh}^{(ii)}\} = 2.079\text{ in}^2.$$

Assuming seven transverse legs across the section, the required bar area per leg is:

$$A_{\text{bar,req}} := \frac{2.079\text{ in}^2}{7} \approx 0.297\text{ in}^2.$$

A No. 5 bar provides:

$$A_{\#5} = 0.31\text{ in}^2 > A_{\text{bar,req}},$$

so seven No. 5 transverse legs satisfy the transverse reinforcement requirements.

3.4.6 Development Length of Skin Reinforcement

To ensure that the skin reinforcement in the beam does not develop and adversely increases the strength of the coupling beam, the development length is calculated and a smaller length is chosen.

The development length, l_{dh}, is calculated using the equations in Table 25.4.2.3 of ACI 318-25 [5]:

$$l_{dh} = \left(\frac{f_y \psi_t \psi_e \psi_g}{25\lambda\sqrt{f_c'}}\right) d_b$$

Substituting the respective coefficients into the equation $f_y = 60{,}000$ psi, $f_c' = 7{,}000$ psi, bar diameter $d_b = 5/8$ in, $\psi_t = 1$, $\psi_e = 1$, $\psi_g = 1$, and $\lambda = 1$. :

$$l_{dh} = \left(\frac{60000(1)(1)(1)}{25(1)\sqrt{7000}}\right)\left(\frac{5}{8}\right)$$

The calculated development length is:

$$l_{dh} = 17.93\text{ in}$$

A development length less than l_{dh} is chosen for the skin reinforcement, say 10 in.

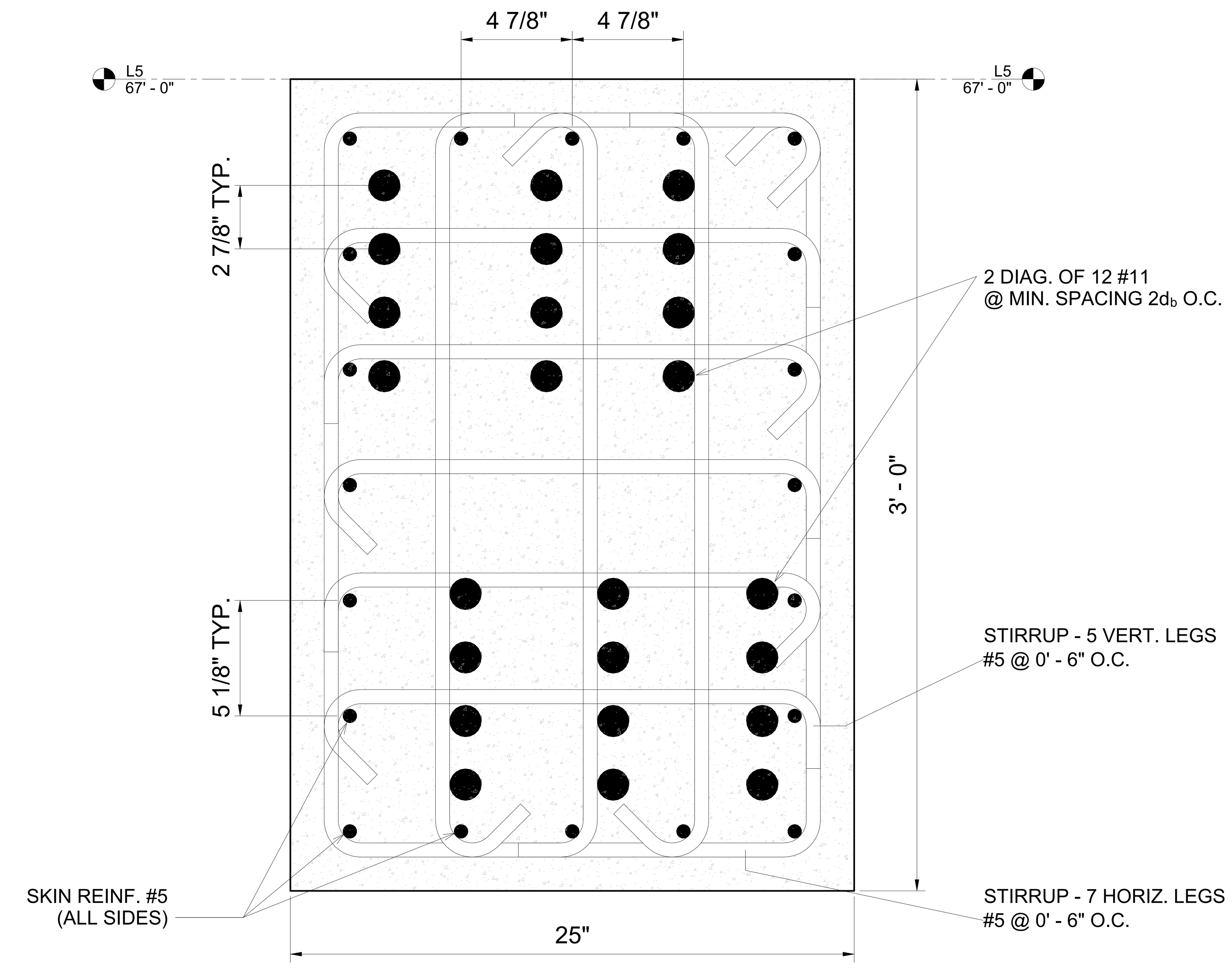
$$\text{Chosen } l_{dh} = 10\text{ in}$$

Verification:

$$10\text{ in} < 17.93\text{ in} \quad \text{OK}$$

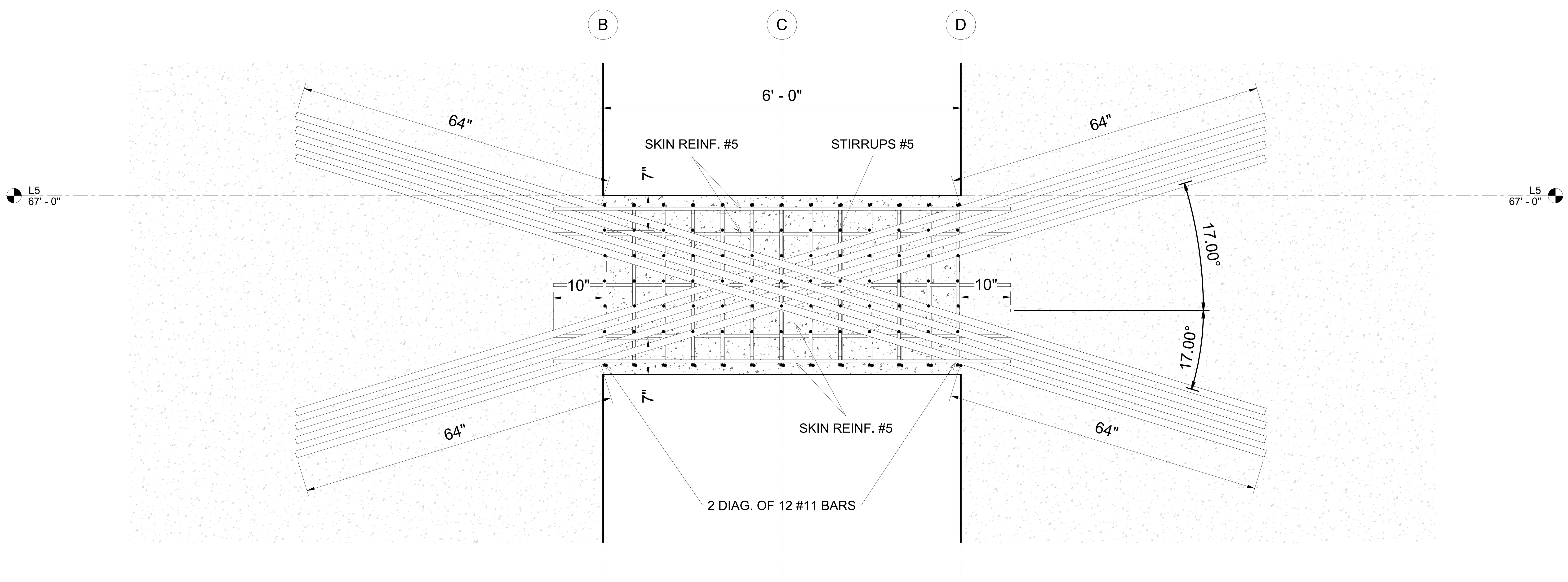
3.4.7 Coupling Beam Detail

The detail of the final design for the coupling beam can be found in the following page.



1 Coupling Beam Section - Lv. 5

3" = 1'-0"



2 Coupling Beam Detail - Lv. 5

1" = 1'-0"

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No.	Description	Date

The City of Berkeley Council

1950 Oxford St.
Residential Building

FINAL
COUPLING BEAM DETAIL

Project Number	CE 244 Semester Project
Date	December 12, 2025
Drawn By	CQ
Checked By	FP, FD, ES

S2.05

Scale	As indicated

12/12/2025 3:27:46 AM

3.5 Gravity Framing

Ultimate Gravity Shear Calculation

3.5.1 Gravity Shear from Floor Loads on Interior Connection (Grid Line B)

The objective of this calculation is to determine the maximum ultimate vertical shear force (V_u) transferred from the above-ground slabs to a typical interior column connection (Grid Line B). By considering the typical case loading on the slabs (residential), this force is used for the punching shear check of the flat-plate floor system.

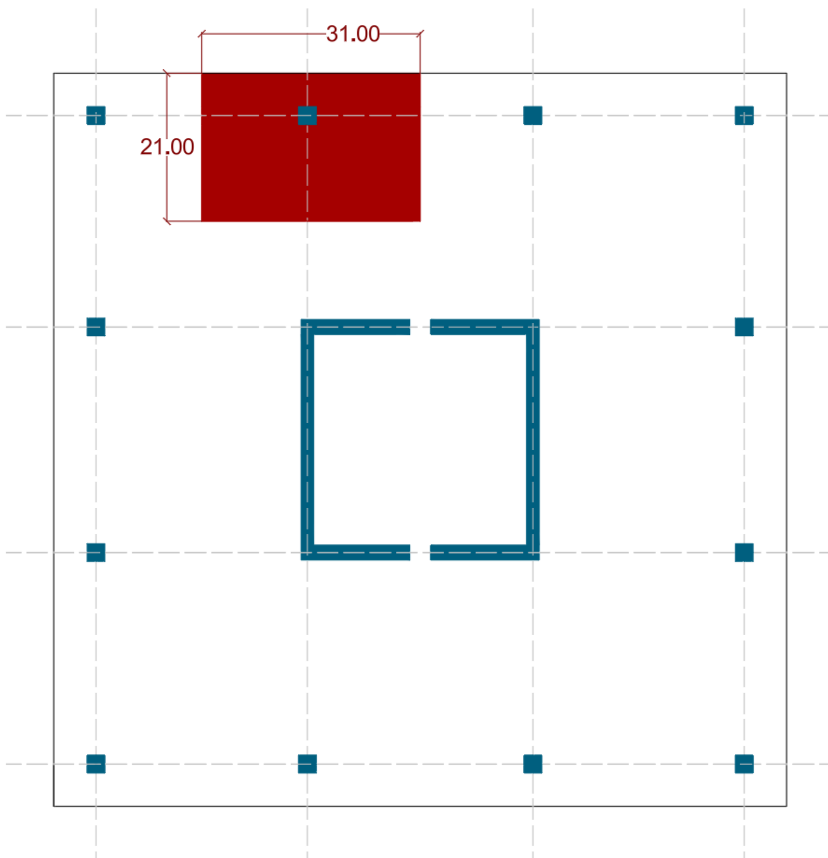


Figure 3.5.1: Tributary Area for Punching Shear Calculation

Given Data and Assumptions:

- Controlling Load Combination: $U = (1.2 + 0.2S_{DS})D + 0.5L$.
- Seismic Design Parameter: $S_{DS} = 1.73$.
- Governing Case (Residential Area):
  - Live Load (L): $L = 40$ psf.
  - Superimposed Dead Load (D): $D = 25$ psf.
- Slab Self-Weight (D): $D_{\text{slab}} = 150 \text{ pcf} \times \frac{8 \text{ in}}{12 \frac{\text{in}}{\text{ft}}} = 100$ psf

Calculate Ultimate Distributed Load (w_u)

$$w_u = (1.2 + 0.2S_{DS})(D + D_{\text{slab}}) + 0.5L$$

$$w_u = (1.2 + 0.2(1.73))(25 \text{ psf} + 100 \text{ psf}) + 0.5(40 \text{ psf})$$

$$w_u = 193.25 \text{ psf} + 20 \text{ psf}$$

$$w_u = 213.25 \text{ psf}$$

Calculate Tributary Area (A_T) The total area of a typical residential floor supported by the interior column is determined using the tributary area method. The dimensions are estimated as shown in **Fig. 3.5.1**.

$$A_T = (6 + 15) \text{ ft} \times (15 + 16) \text{ ft}$$

$$A_T = 651 \text{ ft}^2$$

Calculate Ultimate Gravity Shear ($V_{u,\text{gravity}}$) The factored gravity load transferred to the column from the tributary slab area is:

$$V_{u,\text{gravity}} = w_u \times A_T$$

$$V_{u,\text{gravity}} = 213.25 \text{ psf} \times 651 \text{ ft}^2$$

$$V_{u,\text{gravity}} = 138825.75 \text{ lbs}$$

$$V_{u,\text{gravity}} = 138.83 \text{ kips}$$

3.5.2 Load from Exterior Cladding

This section determines the ultimate load transferred to the column-slab connection from the exterior cladding, assuming the cladding is supported by the slab edge.

Cladding Load Assumptions:

- Cladding Dead Load (D_{cladding}): 55 psf of wall area.
- Tributary Wall Height: 13 ft.

Calculate Ultimate Cladding Load Factor ($w_{u,\text{cladding}}$) Since the cladding is a Dead Load, only the Dead Load factor from the governing load combination is applied.

$$w_{u,\text{cladding}} = (1.2 + 0.2S_{DS}) \times D_{\text{cladding}}$$

$$w_{u,\text{cladding}} = (1.2 + 0.2(1.73)) \times (55 \text{ psf})$$

$$w_{u,\text{cladding}} = 85 \text{ psf}$$

Calculate Tributary Wall Area ($A_{T,\text{wall}}$) The tributary wall area supported by the column is calculated by multiplying the tributary wall length (31 ft) by the tributary wall height (13 ft).

$$A_{T,\text{wall}} = (31\ \text{ft} \times 13\ \text{ft})$$
$$A_{T,\text{wall}} = 403\ \text{ft}^2$$

Calculate Ultimate Cladding Load ($V_{u,\text{cladding}}$) The total factored cladding load supported by the column connection is:

$$V_{u,\text{cladding}} = w_{u,\text{cladding}} \times A_{T,\text{wall}}$$
$$V_{u,\text{cladding}} = 85\ \text{psf} \times (403\ \text{ft}^2)$$
$$V_{u,\text{cladding}} = 34267.1\ \text{lbs}$$
$$V_{u,\text{cladding}} = 34.27\ \text{kips}$$

Total Ultimate Vertical Load The total ultimate vertical shear force (V_u) to be resisted by the column-slab connection is the sum of the factored gravity load from the slab area and the factored cladding load.

$$V_{u,\text{total}} = V_{u,\text{gravity}} + V_{u,\text{cladding}}$$
$$V_{u,\text{total}} = 138.83\ \text{kips} + 34.27\ \text{kips}$$
$$V_{u,\text{total}} = 173.1\ \text{kips}$$

3.5.3 Determine the Design Punching Shear Strength ϕV_c

Because the floor system is post-tensioned, the equations for the punching shear strength of post-tensioned flat-plate connections is used. According to ACI 318-25, section 22.6.5.5, the punching shear strength of such connections is given by the following:

22.6.5.5 For two-way shear in prestressed members conforming to 22.6.5.4, v_c shall be permitted to be the lesser of (a) and (b):

$$v_c = 3.5\lambda\sqrt{f'_c} + 0.3 f_{pc} + \frac{V_p}{b_o d} \tag{3.5.1a}$$
$$v_c = \left(1.5 + \frac{\alpha_s d}{b_o}\right)\lambda\sqrt{f'_c} + 0.3 f_{pc} + \frac{V_p}{b_o d} \tag{3.5.1b}$$

where:

- α_s is given in 22.6.5.3.
- The value of f_{pc} is the average of f_{pc} in the two directions and shall not exceed 500 psi.
- V_p is the vertical component of all effective prestress forces crossing the critical section.
- The value of $\sqrt{f'_c}$ shall not exceed 70 psi.

Note: Equation (3.5.1a) (22.6.5.5a) controls the strength for this problem.

In the equation:

- λ is the factor for lightweight aggregate concrete (= 1.0).
- f_{pc} is the average prestress in both directions (= 167 psi).
- V_p is normally taken equal to 0.0 because the angle of the strands is so small and uncontrollable in construction.

Given Parameters:

- $\lambda = 1.0$
- $f_{pc} = 167$ psi
- $f'_c = 6000$ psi (From **Table 1.5.1**)
- $\sqrt{f'_c} = \sqrt{6000 \text{ psi}} \approx 77.46 \text{ psi} > 70 \text{ psi} \rightarrow \sqrt{f'_c} = 70$ psi according to the limit of ACI 318-25 (22.6.5.5)

Using Governing Equation (3.5.1a):

$$v_c = 3.5(1.0)(70 \text{ psi}) + 0.3(167 \text{ psi}) + 0$$

$$v_c \approx 245 + 50.1 = 295.1 \text{ psi}$$

Geometry properties:

- Column size: 24 in × 24 in
- Assumed slab clear cover: 0.75 in
- Strand diameter assumption: 0.5 in (from **Table 1.5.3**)

Calculate effective depth d:

$$d = 8 \text{ in} - 0.75 \text{ in} - \frac{0.5 \text{ in}}{2} = 7 \text{ in}$$

Calculate critical perimeter b_o:

$$b_o = 2[(c_1 + d) + (c_2 + d)]$$

$$b_o = 2[(24 \text{ in} + 7 \text{ in}) + (24 \text{ in} + 7 \text{ in})] = 2[31 \text{ in} + 31 \text{ in}] = 124 \text{ in}$$

Design Punching Shear Strength Calculation:

$$\phi V_c = \phi \cdot v_c \cdot b_o \cdot d$$

Using $\phi = 0.75$ for shear:

$$\phi V_c = 0.75(295.1 \text{ psi})(124 \text{ in})(7 \text{ in})$$

$$\phi V_c \approx 192110.1 \text{ lbs}$$

$$\boldsymbol{\phi \mathbf{V_c} = \mathbf{192.11} \text{ kips}}$$

3.5.4 Drift Ratio capacity

The drift ratio capacity of a typical connection without shear reinforcement is determined in order to establish a need for shear reinforcement.

ACI 318-25 Code Reference

18.14.5 Slab-column connections

18.14.5.1 For slab-column connections of two-way slabs without beams, slab shear reinforcement satisfying the requirements of 18.14.5.3 and either 8.7.6 or 8.7.7 shall be provided at any slab critical section defined in 22.6.4.1 for the following conditions:

(a) Nonprestressed slabs where $\Delta_x/h_{sx} \geq 0.035 - (1/20)(v_{uv}/\phi v_c)$

(b) Unbonded post-tensioned slabs with f_{pc} in each direction meeting the requirements of 8.6.2.1, where:

$$\frac{\Delta_x}{h_{sx}} \geq 0.040 - \left(\frac{1}{20}\right)\left(\frac{v_{uv}}{\phi v_c}\right)$$

The load combinations to be evaluated for v_{uv} shall only include those with E. The value of (Δ_x/h_{sx}) shall be taken as the greater of the values of the adjacent stories above and below the slab-column connection, v_c shall be calculated in accordance with 22.6.5; and, for unbonded post-tensioned slabs, the value of V_p shall be taken as zero when calculating v_c.

Calculation

Using ACI 318-25 Equation 18.14.5.1(b):

The limiting drift ratio capacity is calculated as:

$$\frac{\Delta_x}{h_{sx}} \geq 0.04 - \frac{1}{20}\left(\frac{V_{uv}}{\phi V_c}\right)$$

Calculated Values:

- $\phi V_c = 192.11$ kips (From Part b)
- $V_{uv} = 173.1$ kips (From Part a)

Substitution:

$$\frac{\Delta_x}{h_{sx}} \geq 0.04 - \frac{1}{20}\left(\frac{173.1\text{ kips}}{192.11\text{kips}}\right)$$

$$\frac{\Delta_x}{h_{sx}} \geq 0.04 - \frac{1}{20}(0.9)$$

$$\frac{\Delta_x}{h_{sx}} \geq 0.04 - 0.045$$

$$\frac{\Delta_x}{h_{sx}} \geq -0.005$$

Result:

$$\frac{\Delta_x}{h_{sx}} \geq -0.005 \quad \text{or} \quad \mathbf{-0.5}\%$$

Note however the equation above is capped if the story drift is below 1 %. ACI 318-25 18.14.5.2 (b) states that if $\Delta_x/h_{sx} \leq 0.01$, shear reinforcement is not needed.

18.14.5.2 The shear reinforcement requirements of 18.14.5.1 need not be satisfied if (a) or (b) is met:

(a) $\Delta_x/h_{sx} \leq 0.005$ for nonprestressed slabs

(b) $\Delta_x/h_{sx} \leq 0.01$ for unbonded post-tensioned slabs with f_{pc} in each direction meeting the requirements of 8.6.2.1

Given that the maximum story drift is of 0.00645 which is $\leq$ 0.01 section 18.14.5.2 is triggered and reinforcement is not needed.

3.5.5 Drift Ratio Capacity Check

The following table summarizes the displacement, drift, and capacity checks for each story.

Story	Height (in.)	X-disp. (in.)	Y-disp. (in.)	X-drifts (%)	Y-drifts (%)	Avg X-drift (%)	Avg Y-drift (%)	Capacity by Code (%)	Reinf. Needed?
Roof	135	8.04	7.86	0.58	0.64	0.305	0.325	1.0	NO
Story8	122	7.14	6.87	0.61	0.65	0.605	0.645	1.0	NO
Story7	109	6.20	5.85	0.63	0.65	0.625	0.645	1.0	NO
Story6	96	5.23	4.84	0.64	0.64	0.635	0.630	1.0	NO
Story5	83	4.23	3.84	0.64	0.61	0.625	0.605	1.0	NO
Story4	70	3.25	2.89	0.61	0.57	0.600	0.555	1.0	NO
Story3	57	2.30	2.00	0.56	0.50	0.550	0.495	1.0	NO
Story2	44	1.42	1.22	0.49	0.42	0.455	0.395	1.0	NO
Story1	31	0.66	0.56	0.35	0.29	0.250	0.220	1.0	NO
Lobby	16	0.03	0.03	0.01	0.02	0.175	0.145	1.0	NO

Table 3.5.1: Story drift demands, average drifts, and code drift capacity check.

From the Table above it can be shown that shear reinforcement within the slab-column joint is not needed at each level.

3.6 Gravity Column Design

3.6.1 Design Basis and Assumptions

Due to the complexity of the gravity framing model, a conservative design approach is adopted. The analysis assumes that demands may exceed design strengths; therefore, ductile detailing will be utilized.

This report analyzes an interior gravity column at the base of the building along grid line B. The following assumptions govern the design:

- **Column Section:** 24 in. $\times$ 24 in. square section (assumed constant height).
- **Seismic Load (E):** $E = 0$ (Gravity-only column).
- **Moments:** Assumed negligible due to slab overhang balancing interior span moments and low drift ratios.

3.6.2 Load Combinations

The design axial force is determined by evaluating the following controlling load combinations [5, 2]:

1. $1.4D$
2. $1.2D + 1.6L$
3. $(1.2 + 0.2S_{DS})D + 0.5L + E$
4. $(0.9 - 0.2S_{DS})D + E$

The summation of loads for the interior gravity column is detailed in **Table 3.6.1**.

	Trib. Area (ft^2)		Self Weight (lbs)		Dead Load (psf)			Total Loads		
Story	**Horiz.**	**Vert.**	**Slab**	**Column**	**Horiz.**	**Vert.**	**L.L (psf)**	**Story DL (lbs)**	**Story LL (lbs)**	**Seismic**
Roof	651	201.5	65,100	7,800	75	55	100	132,807.5	65,100	0
Story 8	651	403.0	65,100	7,800	25	55	40	111,340.0	26,040	0
Story 7	651	403.0	65,100	7,800	25	55	40	111,340.0	26,040	0
Story 6	651	403.0	65,100	7,800	25	55	40	111,340.0	26,040	0
Story 5	651	403.0	65,100	7,800	25	55	40	111,340.0	26,040	0
Story 4	651	403.0	65,100	7,800	25	55	40	111,340.0	26,040	0
Story 3	651	403.0	65,100	7,800	25	55	40	111,340.0	26,040	0
Story 2	651	403.0	65,100	7,800	25	55	40	111,340.0	26,040	0
Story 1	651	434.0	65,100	9,000	25	55	40	114,245.0	26,040	0
Lobby	651	232.5	97,650	9,600	40	55	100	146,077.5	65,100	0
							Total Service Loads	**1,172,510**	**338,520**	**0**
							Total (kips)	$P_D = 1,172.5$	$P_L = 338.5$	$E = 0$

Table 3.6.1: Story Loads Takeoff

Load Combination	Dead (D) (kips)	Live (L) (kips)	Seismic (E) (kips)	S_{DS}	P_u (kips)
$1.4D$	1,172.51	338.52	0	–	1,641.51
$1.2D + 1.6L$	1,172.51	338.52	0	–	**1,948.64**
$(1.2 + 0.2S_{DS})D + 0.5L + E$	1,172.51	338.52	0	1.73	656.09
$(0.9 - 0.2S_{DS})D + E$	1,172.51	338.52	0	1.73	649.57

Table 3.6.2: Factored Load Combination Results

Based on the service loads ($P_D = 1,172.5$ kips, $P_L = 338.5$ kips) and seismic parameter $S_{DS} = 1.73$, the factored loads are calculated in **Table 3.6.2**.

Conclusion: Load Combination 2 ($1.2D + 1.6L$) controls the design of the gravity columns.

$$P_u = 1,948.64 \text{ kips}$$

3.6.3 Design of Column Longitudinal Reinforcement

Objective The objective of this section is to determine the required area of longitudinal steel reinforcement (A_{st}) for the concrete column. The design must satisfy the highest factored axial load (P_u) determined from the load combination analysis.

Code Provisions and Derivation Per **ACI 318 Section 22.4.2.1**, the nominal axial compressive strength (P_n) of a non-prestressed member shall not exceed the maximum allowable strength $P_{n,max}$.

For a non-prestressed member with tied reinforcement, the design strength is calculated as:

$$\phi P_{n,max} = \phi \cdot 0.80 \cdot P_o \tag{3.6.1}$$

Where P_o is the nominal axial strength at zero eccentricity, defined in ACI Eq. 22.4.2.2 as:

$$P_o = 0.85 f'_c (A_g - A_{st}) + f_y A_{st} \tag{3.6.2}$$

By substituting the strength reduction factor $\phi = 0.65$ (for tied columns) and expanding the equation, we derive the expression for the required design strength. The coefficients below are derived from combining the reduction factors (e.g., $0.65 \times 0.80 \times 0.85 \approx 0.442$):

$$\begin{aligned} P_u &= \phi P_{n,max} \\ P_u &= 0.65(0.80)\left[0.85 f'_c (A_g - A_{st}) + f_y A_{st}\right] \\ P_u &= 0.442 f'_c A_g - 0.442 f'_c A_{st} + 0.52 f_y A_{st} \end{aligned}$$

Rearranging the terms to solve for the required area of steel (A_{st}):

$$A_{st} = \frac{P_u - 0.442 f'_c A_g}{0.52 f_y - 0.442 f'_c} \tag{3.6.3}$$

Calculation of Required Reinforcement Using the controlling load combination and the section properties defined below:

- **Factored Load (P_u):** 1,948.64 kips ($1,948,640$ lbs)
- **Concrete Strength (f'_c):** 6,000 psi
- **Yield Strength (f_y):** 60,000 psi
- **Gross Area (A_g):** 24 in $\times$ 24 in = 576 in^2

Substituting these values into the derived equation:

$$A_{st} = \frac{1,948,640 - 0.442(6,000)(576)}{0.52(60,000) - 0.442(6,000)}$$

$$A_{st} = \frac{1,948,640 - 1,527,552}{31,200 - 2,652}$$

$$A_{st} = \frac{421,088}{28,548}$$

$$\boldsymbol{A_{st,\text{required}}} = \mathbf{14.75}\ \text{in}^{\mathbf{2}}$$

Selection and Verification To satisfy the required area of 14.75 in^2, we select 16 No. 9 bars.

- **Bar Area (No. 9):** 1.00 in^2
- **Total Provided Area:** 16×1.00 in^2 = 16.00 in^2

Since 16.00 in^2 > 14.75 in^2, the selection is adequate.

Final Design: Use 16 No. 9 bars.

3.6.4 Design of Base Level Interior Column Transverse Reinforcement

Objective: Determine the column transverse reinforcement required at the column ends to confine the core and support the longitudinal bars. This reinforcement will be applied along the full length of the column as a verified conservative measure.

Confinement Length (l_o) Per **ACI 18.7.5.1**, the length of the confinement zone (l_o) shall be the greatest of:

$$l_o = \max\left(C_1, C_2, \frac{h_n}{6}, 18\text{in}\right)$$
$$l_o = \max\left(24\text{ in}, 24\text{ in}, \frac{15\text{ ft} \times 12}{6}, 18\text{ in}\right)$$
$$l_o = \max\left(24\text{in}, 24\text{ in}, 30\text{ in}, 18\text{in}\right)$$
$$\boldsymbol{l_o} = \mathbf{30}\text{ in}$$

- Column Base Dimensions: $C_1 = C_2 = 24$ in
- Column Clear Height Span: 16 ft - 1 ft = 15 ft

Spacing Requirements (s) Per **ACI 18.7.5.3**, the spacing of transverse reinforcement shall not exceed the smallest of the following. First, we calculate the reference spacing s_o, assuming a horizontal spacing $h_x \leq 8$ in per **ACI 18.7.5.2** :

$$s_o = 4 + \left(\frac{14 - h_x}{3}\right) = 4 + \left(\frac{14 - 8 \text{ in}}{3}\right) = 6 \text{ in}$$

$$4\text{in} \leq s_o \leq 6 \text{ in OK}\checkmark$$

Now, determining the maximum allowable spacing s:

$$s = \min\left(\frac{C_1}{4}, \frac{C_2}{4}, 6d_b, s_o\right)$$
$$s = \min\left(\frac{24 \text{ in}}{4}, \frac{24 \text{ in}}{4}, 6(1.128 \text{ in}), 6 \text{ in}\right)$$
$$s = \min(6 \text{ in}, 6 \text{ in}, 6.77 \text{ in}, 6 \text{ in})$$
$$\boldsymbol{s} = \mathbf{6} \text{ in}$$

Required Transverse Area (A_{sh}) Per **ACI 18.7.5.4**, since $P_u > 0.3A_g f'_c$ (1948.64 kips > 1036.8 kips), the area of transverse reinforcement must satisfy the maximum of the three conditions below.

$$0.3A_g f'_c = \frac{0.3(24 \text{ in} \times 24 \text{ in})(6000 \text{ psi})}{1000\frac{\text{lbs}}{\text{kip}}} = 1036.8 \text{ kips}$$

Parameters:

- Clear Cover (Outside of Ties): 1.5 in
- Core dimension (b_c): 24 in − 2(1.5 in) = 21 in
- Gross Area (A_g): 24 in × 24 in = 576 in^2
- Core Area (A_{ch}): 21 in × 21 in = 441 in^2
- Concrete Strength (f'_c): 6 ksi
- Yield Strength (f_{yt}): 60 ksi

Calculations (A_{sh}/sb_c ratios):
Condition 1 (Geometry Control):

$$0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}} = 0.3\left(\frac{576 \text{ in}}{441 \text{ in}} - 1\right)\frac{6 \text{ ksi}}{60 \text{ ksi}} = \mathbf{0.009}$$

Condition 2 (Min Strength):

$$0.09\frac{f'_c}{f_{yt}} = 0.09\left(\frac{6 \text{ ksi}}{60 \text{ ksi}}\right) = \mathbf{0.009}$$

Condition 3 (High Axial Load): Requires calculation of factors k_f and k_n:

$$k_f = \frac{f'_c}{25,000} + 0.6 = \frac{6,000 \text{ psi}}{25,000} + 0.6 = 0.84 < 1.0 \quad (\text{Take } k_f = 1.0)$$

$$k_n = \frac{n_l}{n_l - 2} = \frac{16}{16-2} = 1.143$$

$$0.2 k_f k_n \frac{P_u}{f_{yt} A_{ch}} = 0.2(1.0)(1.143)\frac{1948.64 \text{ kips}}{60(441\text{in}^2)} = \mathbf{0.0168}$$

The controlling ratio is **0.0168** (Condition 3). Solving for required area A_{sh}:

$$\frac{A_{sh}}{sb_c} = 0.0168$$

$$A_{sh} = 0.0168(s)(b_c)$$

$$A_{sh} = 0.0168(6 \text{ in})(21 \text{ in})$$

$$\boldsymbol{A_{sh,\text{req}}} = \mathbf{2.12} \ \text{in}^{\mathbf{2}}$$

Selection and Detailing

Inside Confinement Zone (l_o): Required $A_{sh} = 2.12$ in^2. Using **No. 6 bars** ($A_b = 0.44$ in^2):

$$\text{Number of legs} = \frac{2.12}{0.44} = 4.81 \rightarrow \text{Use 5 legs}$$

Since the column is square and symmetrical the calculations above stand for both directions of the column and thus the designed reinforcement is applied equally in both directions.

Selection: Use No. 6 hoops/crossties at 6 in o.c. both ways within l_o.

Outside Confinement Zone: Per ACI 18.7.5.5, spacing shall be the lesser of $6d_b$ or 6 in.

$$s = \min(6 \text{ in}, 6d_b)$$

$$s = \min(6 \text{ in}, 6(1.128 \text{ in})) = 6 \text{ in}$$

Selection: Use No. 6 hoops/crossties at 6 in o.c. both ways due to the same spacing and reasons specified for reinforcement inside the confinement zone (l_o)

3.6.5 Column Shear Strength Verification

Objective The objective is to determine whether the transverse reinforcement provided in the previous section is sufficient for shear. The design shear (V_u) is determined using the probable moment strength (M_{pr}) of the column, accounting for the range of axial forces.

Design Shear Force (V_u) Based on the P-Mpr interaction diagram analysis:

- **Probable Moment (M_{pr}):** 1258 kips-ft
- **Associated Axial Load (P):** 656.09 kips
- **Clear Span (l_n):** 15 ft

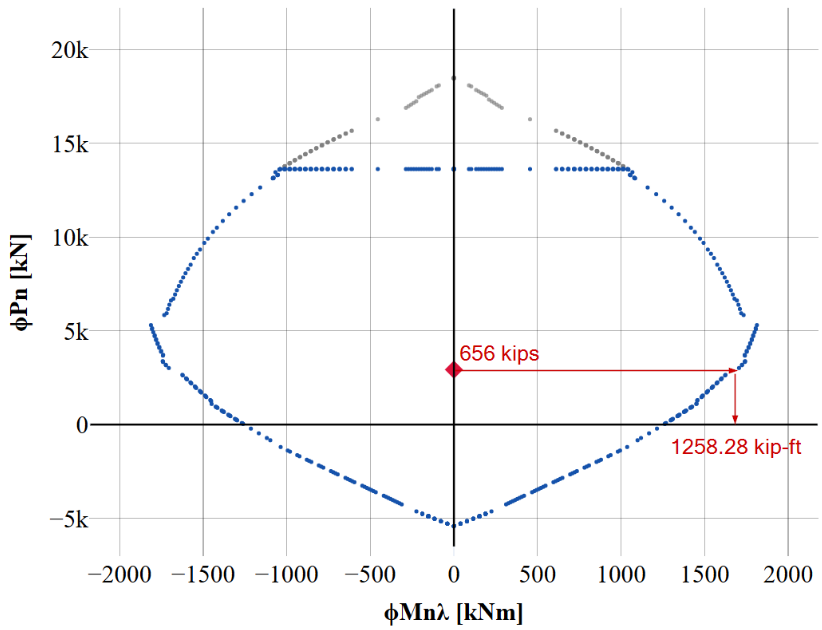


Figure 3.6.1: P-Mpr Diagram for column section. Note ϕ is taken as 1 for probable moment capacity.

The design shear is calculated assuming plastic hinges form at both ends of the clear span:

$$V_u = \frac{2M_{pr}}{l_n}$$

$$V_u = \frac{2(1,258 \text{ kips-ft})}{15 \text{ ft}}$$

$$V_u = 167.73 \text{ kips}$$

Nominal Shear Strength (V_n) The nominal shear strength of the column is calculated as the sum of the concrete contribution (V_c) and the steel reinforcement contribution (V_s).

Concrete Contribution (V_c) Per ACI 25.5.5 V_c is calculated as:

$$V_c = \left[2\lambda\sqrt{f'_c} + \frac{N_u}{6A_g}\right] b_w d \tag{3.6.4}$$

Where:

- $f'_c = 6,000$ psi
- $N_u = 656,090$ lbs (Axial compression)
- $A_g = 24 \text{ in} \times 24 \text{ in} = 576 \text{ in}^2$
- $b_w = 24$ in
- $d = 24 \text{ in} - (1.5 \text{ in} + \frac{6 \text{ in}}{8} + \frac{1.128 \text{ in}}{2}) \approx 21.186$ in (Effective depth)

Steel Contribution (V_s) The shear strength provided by the transverse reinforcement is:

$$V_s = \frac{A_v f_{yt} d}{s} \tag{3.6.5}$$

Using the reinforcement selected in Part C (5 legs of No. 6 bars at 6 in spacing):

- $A_v = 5 \text{ legs} \times 0.44 \text{ in}^2 = 2.20 \text{ in}^2$
- $f_{yt} = 60,000$ psi
- $s = 6$ in

Verification of shear strength is based on:

$$V_u \le \phi V_n = \phi (V_c + V_s), \qquad \phi = 0.75 \text{ (shear)}.$$

Using the adopted section and reinforcement:

$$\begin{aligned}
V_c &= \left(2\lambda\sqrt{f_c'} + \frac{N_u}{6A_g}\right) b_w d = \left(2(1)\sqrt{6000\,\text{psi}} + \frac{656{,}090\,\text{lb}}{6(576\,\text{in}^2)}\right)(24\,\text{in})(21.186\,\text{in}) = 175.3\,\text{kips},\\
V_s &= \frac{A_v f_{yt} d}{s} = \frac{5(0.44\,\text{in}^2)(60{,}000\,\text{psi})(21.186\,\text{in})}{6\,\text{in}} = 466.1\,\text{kips},\\
\phi V_n &= 0.75\,(V_c + V_s) = 0.75(175.3 + 466.1) = 0.75(641.4) = 481.0\,\text{kips}.
\end{aligned}$$

Therefore:

$$V_u = 167.73\,\text{kips} \le \phi V_n = 481.0\,\text{kips}.$$

Going through the ACI 318-25 design limit checks[5]:

- Concrete shear limit:

$$V_c = 175.3\,\text{kips} \le 5\lambda\sqrt{f_c'}\, b_w d = 196.93\,\text{kips} \quad \text{OK} \checkmark$$

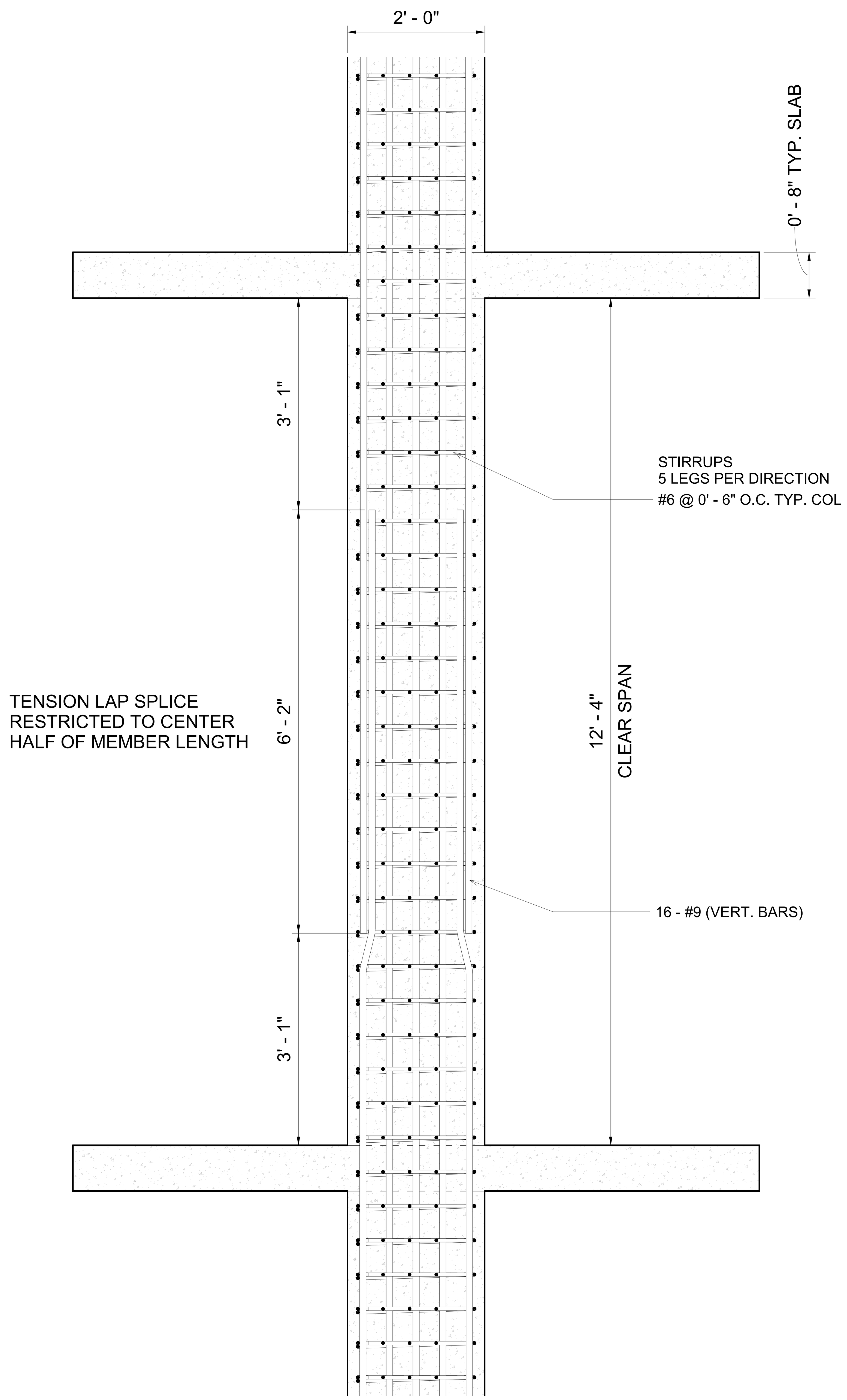
- Axial-stress term:

$$\frac{N_u}{6A_g} \approx 189.84\,\text{psi} \le 0.05 f_c' = 300\,\text{psi} \quad \text{OK} \checkmark$$

Since $\phi V_n = 481.0\,\text{kips} > V_u = 167.73\,\text{kips}$, the shear strength requirement is satisfied. The provided transverse reinforcement is adequate for shear.

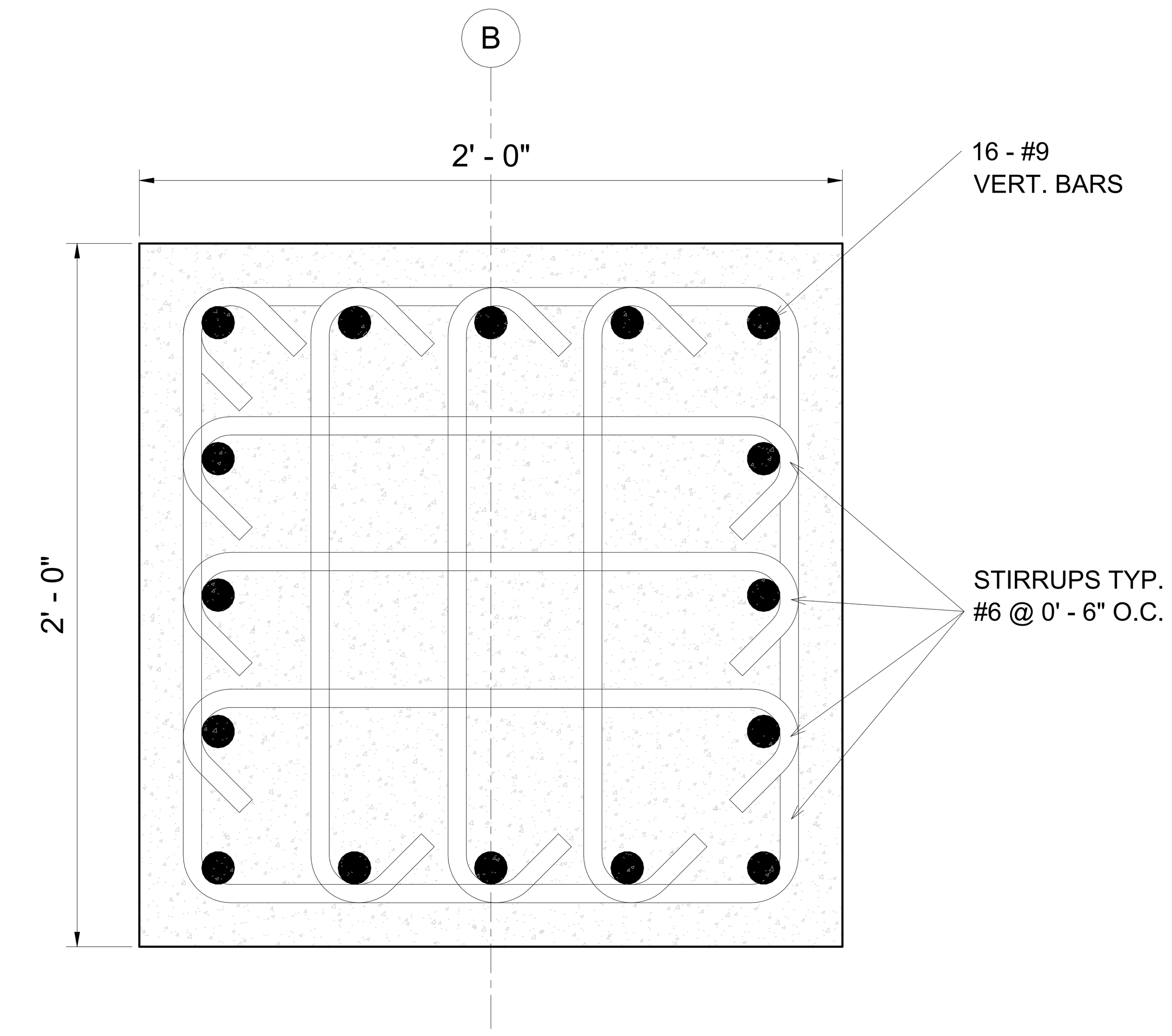
3.6.6 Column Details

Based on the design calculations throughout Section 3.6, the final interior gravity column details are provided on the following page.



1 Typical Column Detail - Elevation

1" = 1'-0"



2 Typical Column Detail - Plan

3" = 1'-0"

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No.	Description	Date

The City of Berkeley Council

1950 Oxford St.
Residential Building

TYP. INTERIOR COLUMN DETAIL

Project Number	CE 244 Semester Project
Date	December 12, 2025
Drawn By	CQ
Checked By	FP, FD, ES

S2.06

Scale	As indicated

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3.7 Special Boundary Elements (SBE)

The objective of this section is to design the Special Boundary Elements (SBE in short) according to ACI 318-25[5]. SBEs are specially confined zones located at the extremes of walls composing the laterally-force resisting system. The section is strengthened by longitudinal and transverse reinforcement is referred to as a boundary element. Where combined seismic and gravity loads result in high compressive demands on the edge, ACI 318 requires a special boundary element. These have closely spaced transverse reinforcement enclosing the vertical boundary bars to increase ductiliyu capacity of core concrete and to restrain longitudinal bar buckling [11, 9, 1, 12].

3.7.1 SBE Requirement Check

The need for special boundary elements (SBEs) may be evaluated using either of two alternative procedures permitted by ACI 318-25 [5]. The first is a displacement-based approach (Section 18.10.6.2), in which boundary detailing is established directly from the expected lateral deformations of the wall. The second is a stress-based (fiber-stress) check (Section 18.10.6.3), which examines the compressive demand that develops at the wall boundaries under combined gravity and seismic actions.

In this project, the stress-based procedure of Section 18.10.6.3 is adopted first for its relative simplicity. When the stress-based criterion does not trigger boundary confinement, the displacement-based check is also reported as a secondary verification.

Compressive Stress Check At the critical section, factored axial load and bending moments from the governing earthquake load combinations are used to compute the extreme-fiber compressive stresses assuming linear-elastic response. Special boundary confinement is required wherever the calculated compressive stress exceeds $0.20 f'_c$, a commonly used threshold intended to identify regions expected to experience significant compression demand under repeated inelastic cycling [5, 11].

For a linear Euler–Bernoulli model subjected to factored axial force and uniaxial bending about one principal axis at a time, the extreme-fiber compressive stresses are evaluated separately for bending about the $x-x$ and $y-y$ axes as:

$$\sigma_{x,C} = \frac{P_u}{A_g} + \frac{M_{u,x}}{I_{x-x}}\, y_{\max} \quad < \quad 0.20\, f'_c = 1.4 \text{ ksi} \tag{3.7.1}$$

$$\sigma_{y,C} = \frac{P_u}{A_g} + \frac{M_{u,y}}{I_{y-y}}\, x_{\max,C} \quad < \quad 0.20\, f'_c = 1.4 \text{ ksi} \tag{3.7.2}$$

where:

- $\sigma_{x,C}$ is the extreme-fiber compressive stress for uniaxial bending about the $x-x$ axis combined with axial force.
- $\sigma_{y,C}$ is the extreme-fiber compressive stress for uniaxial bending about the $y-y$ axis combined with axial force.
- P_u is the factored axial force (compression taken as negative, consistent with **Fig. 3.2.1**).
- A_g is the gross cross-sectional area of the wall.
- $M_{u,x}$ is the factored bending moment about the $x-x$ axis.
- $M_{u,y}$ is the factored bending moment about the $y-y$ axis.

- I_{x-x} and I_{y-y} are the gross second moments of area about the $x-x$ and $y-y$ axes, respectively.
- $y_{\max}$ is the distance from the neutral axis to the extreme compression fiber measured along the y-direction.
- $x_{\max,C}$ is the distance from the centroidal $y-y$ axis to the extreme compression fiber in the x-direction. The subscript C denotes the compression fiber; because the cross-section is symmetric about the $x-x$ axis but not about the $y-y$ axis, the location (and sign) of $x_{\max,C}$ can change with the sign of $M_{u,y}$.
- f'_c is the specified concrete compressive strength, taken as 7 ksi in this project.

Only factored load combinations that include earthquake effects in the horizontal direction under consideration are used to evaluate $\sigma_{x,C}$ and $\sigma_{y,C}$ via **Eq. (3.7.1)** and **Eq. (3.7.2)**, respectively.

Displacement Check When the computed compressive stresses from **Eq. (3.7.1)** and **Eq. (3.7.2)** remain below $0.20 f'_c = 1.4\,\text{ksi}$, the displacement-based criterion of ACI 318-25 Section 18.10.6.2 is evaluated as:

$$1.50 \frac{\delta_u}{h_{wcs}} \geq \frac{l_w}{600c} \tag{3.7.3}$$

where δ_u is the design displacement (from **Table 3.5.1**), h_{wcs} is the height of the wall critical section, c is the neutral-axis depth measured from the extreme compression fiber, and l_w is the wall length in the direction under consideration.

In this work, c is estimated using the empirical expression proposed by Abdullah and Wallace [10]:

$$\frac{c}{l_w} = k_1 + k_2 \frac{-P_u}{A_g f'_c}, \tag{3.7.4}$$

where P_u is the factored axial force (compression taken as negative in this expression), A_g is the gross cross-sectional area, f'_c is the specified concrete compressive strength, and k_1, k_2 are empirical coefficients that depend on wall geometry and the compression region.

The coefficients adopted for the c-estimation in **Eq. (3.7.4)** are summarized below for each bending/compression case. The corresponding section properties and the factored load combinations used in the displacement verification are then reported in **Table 3.7.1**, **Table 3.7.2**, and **Table 3.7.3**:

Case	k_1	k_2
M_x	0.03	1.40
M_y (web compression)	0.03	0.70
M_y (flange compression)	0.20	2.00

y_{max} [in]	$0.2f_c'$ (ksi)	A_g (in^2)	I_{x-x} (in^4)
204.5	1.4	15,864	388,296,000
M_x combinations (compression taken as negative)			
Combination	**P [kip]**	**M_x [kip-ft]**	$P/A_g + M_x/S$
1.2D+0.5L+(0.3Ex+Ey)/(R/IE)	-8,154	127,330	1.32
0.9D+(0.3Ex+Ey)/(R/IE)	-6,221	127,478	1.20
1.2D+0.5L+(-0.3Ex+Ey)/(R/IE)	-3,809	130,271	1.06
0.9D+(-0.3Ex+Ey)/(R/IE)	-1,876	130,419	0.94
1.2D+0.5L+(Ex+0.3Ey)/(R/IE)	-13,218	33,801	1.05
0.9D+(Ex+0.3Ey)/(R/IE)	-11,285	33,948	0.93
1.2D+1.6L	-7,262	-405	0.46
1.2D+0.5L+(-Ex+0.3Ey)/(R/IE)	1,269	43,606	0.20
1.4D	-6,283	387	0.40
0.9D+(-Ex+0.3Ey)/(R/IE)	3,201	43,753	0.07

Table 3.7.1: Section properties and factored combinations for the uniaxial M_x stress check.

Since the stresses in **Table 3.7.1** do not exceed the $0.20f_c'$ limit, the displacement criterion in **Eq. (3.7.3)** is evaluated using c from **Eq. (3.7.4)**:

$$c = \left(0.03 + (1.40)\frac{13{,}218\,\text{kip}}{15{,}864\,\text{in}^2(7\,\text{ksi})}\right)(409\,\text{in}) = 80.426\,\text{in}.$$

$$1.50\frac{\delta_u}{h_{wcs}} = 1.50\frac{0.56\,\text{in}}{180\,\text{in}} = 4.67\times10^{-3} < \frac{l_w}{600c} = \frac{180\,\text{in}}{600(80.426\,\text{in})} = 8.48\times10^{-3},$$

indicating that SBEs are not required for the M_x compression region (which would otherwise include both web and flanges).

$x_{\max,C}$ [in] (distance from centroid to extreme fiber)	**$0.2f'_c$ (ksi)**	A_g (in^2)	I_{y-y} (in^4)
48.6772	1.4	15,864	40,448,100
M_y combinations (flanges in compression), compression taken as negative			
Combination	**P [kip]**	**M_y [kip-ft]**	$P/A_g - M_y/S$
1.2D+0.5L+(Ex+0.3Ey)/(R/IE)	-13,218	-25,422	1.20
0.9D+(Ex+0.3Ey)/(R/IE)	-11,285	-25,635	1.08
1.2D+1.6L	-7,262	-25,422	0.82
1.2D+0.5L+(0.3Ex+Ey)/(R/IE)	-8,154	-7,212	0.62
0.9D+(0.3Ex+Ey)/(R/IE)	-6,221	-7,425	0.50
1.4D	-6,283	654	0.39
1.2D+0.5L+(-0.3Ex+Ey)/(R/IE)	-3,809	8,412	0.12
1.2D+0.5L+(-Ex+0.3Ey)/(R/IE)	1,269	26,670	-0.47
0.9D+(-0.3Ex+Ey)/(R/IE)	-1,876	8,199	0.00
0.9D+(-Ex+0.3Ey)/(R/IE)	3,201	26,456	-0.58

Table 3.7.2: Section properties and factored combinations for the uniaxial M_y stress check (flanges in compression).

Two compression cases are considered for M_y due to the non-symmetry of the cross-section about the $y-y$ axis. For the case with the flanges in compression (**Table 3.7.2**), the estimated neutral-axis depth is:

$$c = \left(0.03 + (0.70)\frac{13{,}218\,\text{kip}}{15{,}864\,\text{in}^2(7\,\text{ksi})}\right)(166.50\,\text{in}) = 18.868\,\text{in},$$

and:

$$1.50\frac{\delta_u}{h_{wcs}} = 1.50\frac{0.66\,\text{in}}{180\,\text{in}} = 5.50 \times 10^{-3} < \frac{l_w}{600c} = \frac{180\,\text{in}}{600(18.868\,\text{in})} = 14.71 \times 10^{-3},$$

indicating that SBEs are not required for this case.

$x_{\max,C}$ **[in] (distance from centroid to extreme fiber)**	**0.2**f'_c **(ksi)**	A_g **(in^2)**	I_{y-y} **(in^4)**
117.8228	1.4	15,864	40,448,100
M_y **combinations (web in compression), compression taken as negative**			
Combination	**P [kip]**	M_y **[kip-ft]**	$P/A_g + M_y/S$
1.2D+0.5L+(Ex+0.3Ey)/(R/IE)	-13,218	-25,422	-0.06
0.9D+(Ex+0.3Ey)/(R/IE)	-11,285	-25,635	-0.18
1.2D+1.6L	-7,262	-25,422	-0.43
1.2D+0.5L+(0.3Ex+Ey)/(R/IE)	-8,154	-7,212	0.26
0.9D+(0.3Ex+Ey)/(R/IE)	-6,221	-7,425	0.13
1.4D	-6,283	654	0.42
1.2D+0.5L+(-0.3Ex+Ey)/(R/IE)	-3,809	8,412	0.53
1.2D+0.5L+(-Ex+0.3Ey)/(R/IE)	1,269	26,670	0.85
0.9D+(-0.3Ex+Ey)/(R/IE)	-1,876	8,199	0.40
0.9D+(-Ex+0.3Ey)/(R/IE)	3,201	26,456	0.72

Table 3.7.3: Section properties and factored combinations for the uniaxial M_y stress check (web in compression).

For the case with the web in compression (**Table 3.7.3**), the estimated neutral-axis depth is:

$$c = \left(0.20 + (2.00)\frac{13{,}218\,\text{kip}}{15{,}864\,\text{in}^2(7\,\text{ksi})}\right)(166.50\,\text{in}) = 72.937\,\text{in},$$

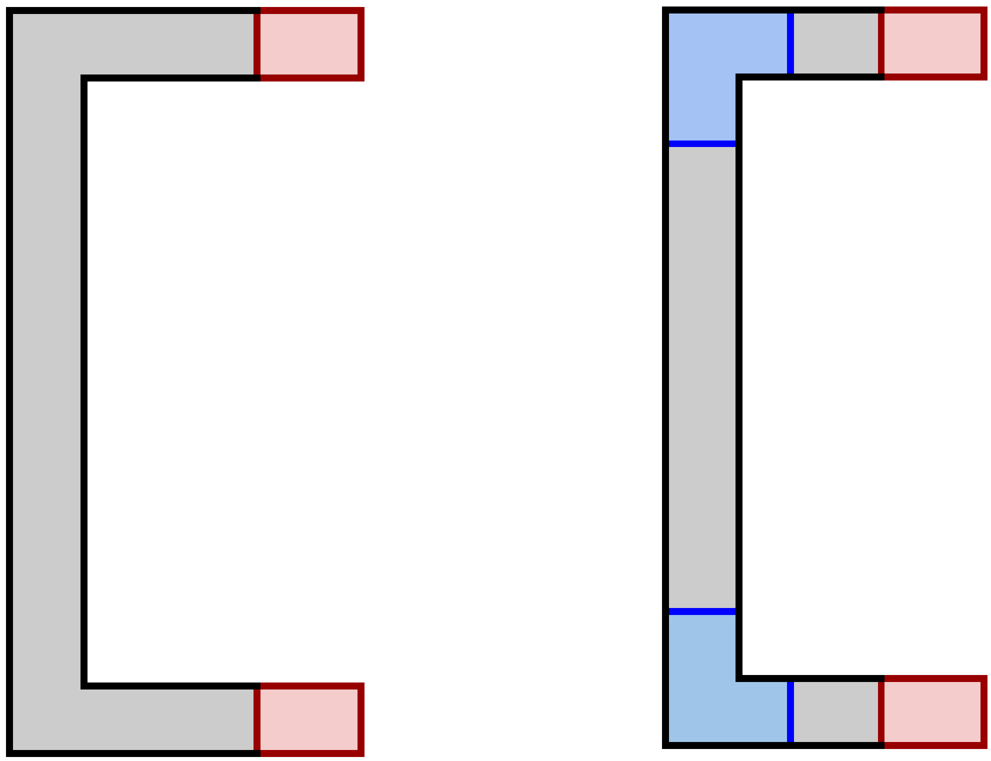
and:

$$1.50\frac{\delta_u}{h_{wcs}} = 1.50\frac{0.66\,\text{in}}{180\,\text{in}} = 5.50\times10^{-3} > \frac{l_w}{600c} = \frac{180\,\text{in}}{600(72.937\,\text{in})} = 3.80\times10^{-3},$$

which triggers the displacement-based requirement for SBEs when the flanges are in compression.

3.7.2 Confinement Criteria

Section 3.7.1 indicates that SBEs are required only in the flange regions, as highlighted in **Fig. 3.7.1a**. However, as a design choice, boundary-element detailing is also provided at the remaining edges of the C-shaped wall, as shown in **Fig. 3.7.1b**. This symmetric detailing is intended to increase confinement and ductility capacity, thereby improving overall seismic performance.



(a) (In red) Highlighted zones that require SBEs per Section 3.7.1.

(b) (In red and blue) Zones where boundary elements are detailed.

Figure 3.7.1: Special Boundary Element zones

For simplicity, the SBE detailing designed for the required flange regions in **Fig. 3.7.1a** is also applied to all boundary locations shown in **Fig. 3.7.1b**, after completing the corresponding verifications in Section 3.7.3.

3.7.3 Special Boundary Element Design

Having verified the need for SBEs in Section 3.7.1, this section presents the design of the confined boundary regions in accordance with ACI 318-25, Section 18.10.6 [5].

The maximum vertical spacing of transverse reinforcement along the height of the wall, s, is limited by **Eq. (3.7.5)**:

$$s = \min \begin{cases} b/3, \\ 6d_b, \\ s_0 \end{cases} \qquad 4\,\text{in} \le s_0 = 4 + \frac{14 - h_x}{3} \le 6\,\text{in}, \tag{3.7.5}$$

where b is the boundary element width, d_b is the diameter of the longitudinal bars, and h_x is the clear horizontal distance between the outermost longitudinal bars in the boundary element.

The confined region at each boundary is required to extend a distance l_{be} from the extreme compression fiber such that:

$$l_{be} = \max \begin{cases} c/2, \\ c - 0.10\,l_w \end{cases} \tag{3.7.6}$$

The transverse reinforcement area A_{sh} provided in each tie leg is required to satisfy the larger of the two ACI 318-25 expressions in **Eq. (3.7.7)**:

$$A_{sh} \geq \begin{cases} 0.09\, s\, b_c\, \dfrac{f_c'}{f_{yt}}, \\ 0.3\, s\, b_c \left(\dfrac{A_g}{A_{ch}} - 1 \right) \dfrac{f_c'}{f_{yt}}, \end{cases} \tag{3.7.7}$$

where b_c is the core dimension measured center-to-center of the outer transverse reinforcement, A_{ch} is the area of the confined concrete core measured to the outside of the transverse reinforcement, and f_{yt} is the specified yield strength of the transverse reinforcement.

The design procedure for the right flange in **Fig. 3.7.1b** is summarized as:

1. Selection of an admissible vertical spacing s using **Eq. (3.7.5)**.
2. Estimation of c using **Eq. (3.7.4)**, followed by the required confinement length l_{be} from **Eq. (3.7.6)**.
3. Definition of the confined region using the resulting l_{be}, determination of A_{ch} and b_c, and evaluation of the required transverse reinforcement A_{sh} from **Eq. (3.7.7)** for both the long and short boundary components.
4. Verification of detailing requirements (bar sizes, number of legs, hoop configuration, and overlap conditions), with adjustments as needed.

The final SBE design is summarized separately for the flange boundary regions (**Table 3.7.4**) and for the web boundary region (**Table 3.7.5**).

Region	Parameter	Value	Comment
Right flange	h_x [in.]	6.00	Boundary thickness.
	b [in.]	25.00	Flange width.
	b_{c1} (long) [in.]	61.00	Core dimension.
	b_{c2} (short) [in.]	22.00	Core dimension.
	$s_{\max}$ [in.]	8.33	= min(·) candidate.
	$s_{\max}$ [in.]	6.77	From # longitudinal bars.
	$s_{\max}$ [in.]	6.67	From 6 in o.c. reference.
	Adopted s [in.]	5.00	Governing; adopted.
	l_w [in.]	166.50	Wall length for this check.
	c [in.]	72.94	From **Eq. (3.7.4)**.
	$l_{be,\min}$ [in.]	56.29	From **Eq. (3.7.6)** criterion (1).
	$l_{be,\min}$ [in.]	36.47	From **Eq. (3.7.6)** criterion (2).
	Adopted l_{be} [in.]	62.50	$\geq \max(56.29, 36.47)$; adopted.
	A_g [in^2]	1,562.50	Boundary gross area.
	A_{ch} [in^2]	1,342.00	Confined core area.
	$A_{sh,\text{eq.1}}$ long b_{c1} [in^2]	3.20	1st expr. in **Eq. (3.7.7)**.
	$A_{sh,\text{eq.2}}$ long b_{c1} [in^2]	1.75	2nd expr. in **Eq. (3.7.7)**.
	# legs (long b_{c1})	11	Transverse legs crossing b_{c1}.
	$A_{sh,\text{req}}$ per leg (long b_{c1}) [in^2]	0.2911	#5 OK.
	$A_{sh,\text{eq.1}}$ short b_{c2} [in^2]	1.16	1st expr. in **Eq. (3.7.7)**.
	$A_{sh,\text{eq.2}}$ short b_{c2} [in^2]	0.63	2nd expr. in **Eq. (3.7.7)**.
	# legs (short b_{c2})	5	Transverse legs crossing b_{c2}.
	$A_{sh,\text{req}}$ per leg (short b_{c2}) [in^2]	0.2310	#5 OK.
	Adopted transverse bar	#5	OK for both directions.
Left flange	h_x [in.]	6.00	Same geometry as right flange.
	b [in.]	25.00	Same geometry as right flange.
	b_{c1} (long) [in.]	61.00	Same geometry as right flange.
	b_{c2} (short) [in.]	22.00	Same geometry as right flange.
	Adopted s [in.]	5.00	Same as right flange.
	l_w [in.]	166.50	Same as right flange.
	c [in.]	18.87	From **Eq. (3.7.4)**.
	$l_{be,\min}$ [in.]	2.22	From **Eq. (3.7.6)** criterion (1).
	$l_{be,\min}$ [in.]	9.43	From **Eq. (3.7.6)** criterion (2).
	Adopted l_{be} [in.]	62.50	Adopted to match detailing.
	A_g [in^2]	1,562.50	Same as right flange.
	A_{ch} [in^2]	1,342.00	Same as right flange.
	$A_{sh,\text{eq.1}}$ long b_{c1} [in^2]	3.20	Same as right flange.
	$A_{sh,\text{eq.2}}$ long b_{c1} [in^2]	1.75	Same as right flange.
	# legs (long b_{c1})	11	Same as right flange.
	$A_{sh,\text{req}}$ per leg (long b_{c1}) [in^2]	0.2911	#5 OK.
	$A_{sh,\text{eq.1}}$ short b_{c2} [in^2]	1.16	Same as right flange.
	$A_{sh,\text{eq.2}}$ short b_{c2} [in^2]	0.63	Same as right flange.
	# legs (short b_{c2})	5	Same as right flange.
	$A_{sh,\text{req}}$ per leg (short b_{c2}) [in^2]	0.2310	#5 OK.
	Adopted transverse bar	#5	OK for both directions.

Table 3.7.4: Summary of SBE confinement sizing and transverse reinforcement selection for the flange boundary regions, showing right–left differences.

Region	Parameter	Value	Comment
Web	h_x [in.]	6.00	Boundary thickness.
	b [in.]	21.00	Web width.
	b_{c1} (long) [in.]	58.00	Core dimension.
	b_{c2} (short) [in.]	18.00	Core dimension.
	$s_{\max}$ [in.]	7.00	= $\min(\cdot)$ candidate.
	$s_{\max}$ [in.]	5.25	From # longitudinal bars.
	$s_{\max}$ [in.]	6.67	From 6 in o.c. reference.
	Adopted s [in.]	5.00	Governing; adopted.
	l_w [in.]	409.00	Wall length for this check.
	c [in.]	80.43	From **Eq. (3.7.4)**.
	$l_{be,\min}$ [in.]	39.53	From **Eq. (3.7.6)** criterion (1).
	$l_{be,\min}$ [in.]	40.21	From **Eq. (3.7.6)** criterion (2).
	Adopted l_{be} [in.]	59.50	$\geq \max(39.53, 40.21)$; adopted.
	A_g [in^2]	1,249.50	Boundary gross area.
	A_{ch} [in^2]	1,044.00	Confined core area.
	$A_{sh,\text{eq.1}}$ long b_{c1} [in^2]	3.05	1st expr. in **Eq. (3.7.7)**.
	$A_{sh,\text{eq.2}}$ long b_{c1} [in^2]	2.00	2nd expr. in **Eq. (3.7.7)**.
	# legs (long b_{c1})	11	Transverse legs crossing b_{c1}.
	$A_{sh,\text{req}}$ per leg (long b_{c1}) [in^2]	0.2768	#5 OK.
	$A_{sh,\text{eq.1}}$ short b_{c2} [in^2]	0.95	1st expr. in **Eq. (3.7.7)**.
	$A_{sh,\text{eq.2}}$ short b_{c2} [in^2]	0.62	2nd expr. in **Eq. (3.7.7)**.
	# legs (short b_{c2})	4	Transverse legs crossing b_{c2}.
	$A_{sh,\text{req}}$ per leg (short b_{c2}) [in^2]	0.2363	#5 OK.
	Adopted transverse bar	#5	OK for both directions.

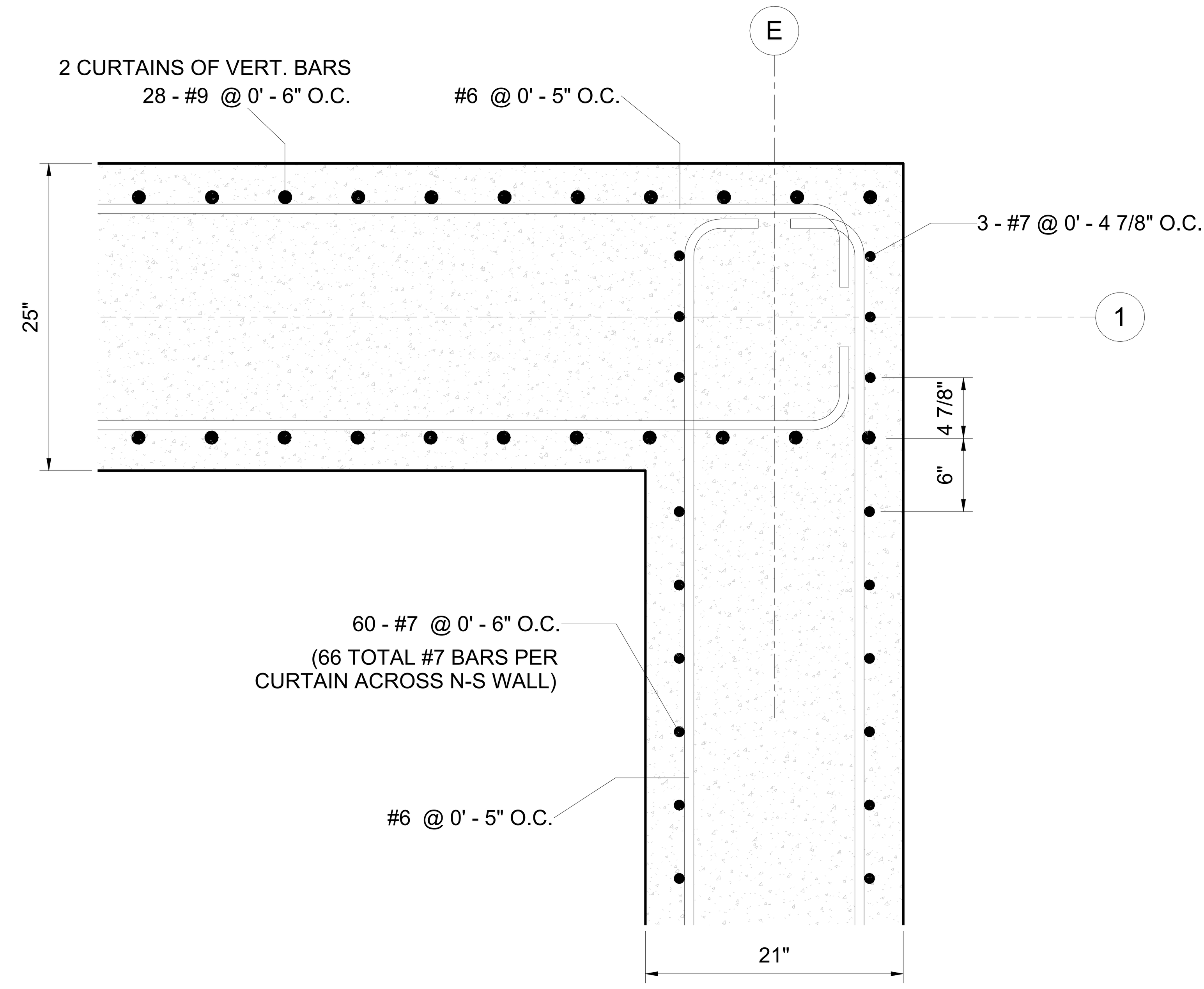
Table 3.7.5: Summary of SBE confinement sizing and transverse reinforcement selection for the web boundary region.

3.7.4 SBEs Details

The details of the Special Boundary Elements designed as part of Section 3.7 can be found in the next page.

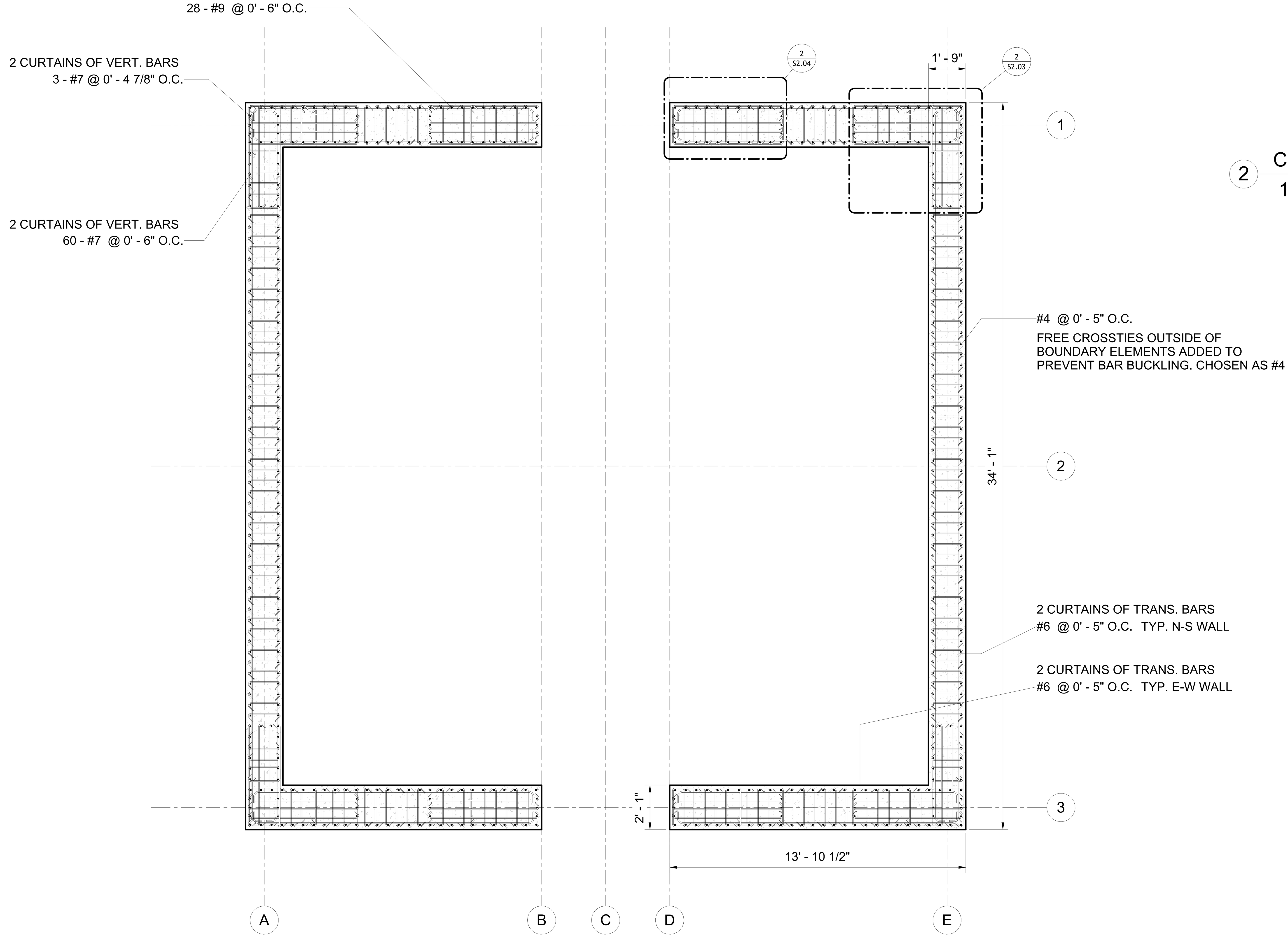
NOTE:
THIS CALLOUT (2) HAS THE CORNER BOUNDARY ELEMENT HIDDEN IN VIEW FOR CLARITY OF THE LONG. AND TRANS. REINF.

PLEASE REFER TO SHEET S2.04 FOR A COMPLETE BOUNDARY ELEMENT DETAIL, INCLUDING THAT OF THIS CORNER



2 Core Wall Corner Detail
1 1/2" = 1'-0"

NOTE:
PLEASE REFER TO SHEET S2.04 FOR A COMPLETE BOUNDARY ELEMENT DETAIL



1 Core Wall Section Detail - Plan
3/8" = 1'-0"

UEFA BERKELEY DESIGN ®

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No.	Description	Date

The City of Berkeley Council

1950 Oxford St.
Residential Building

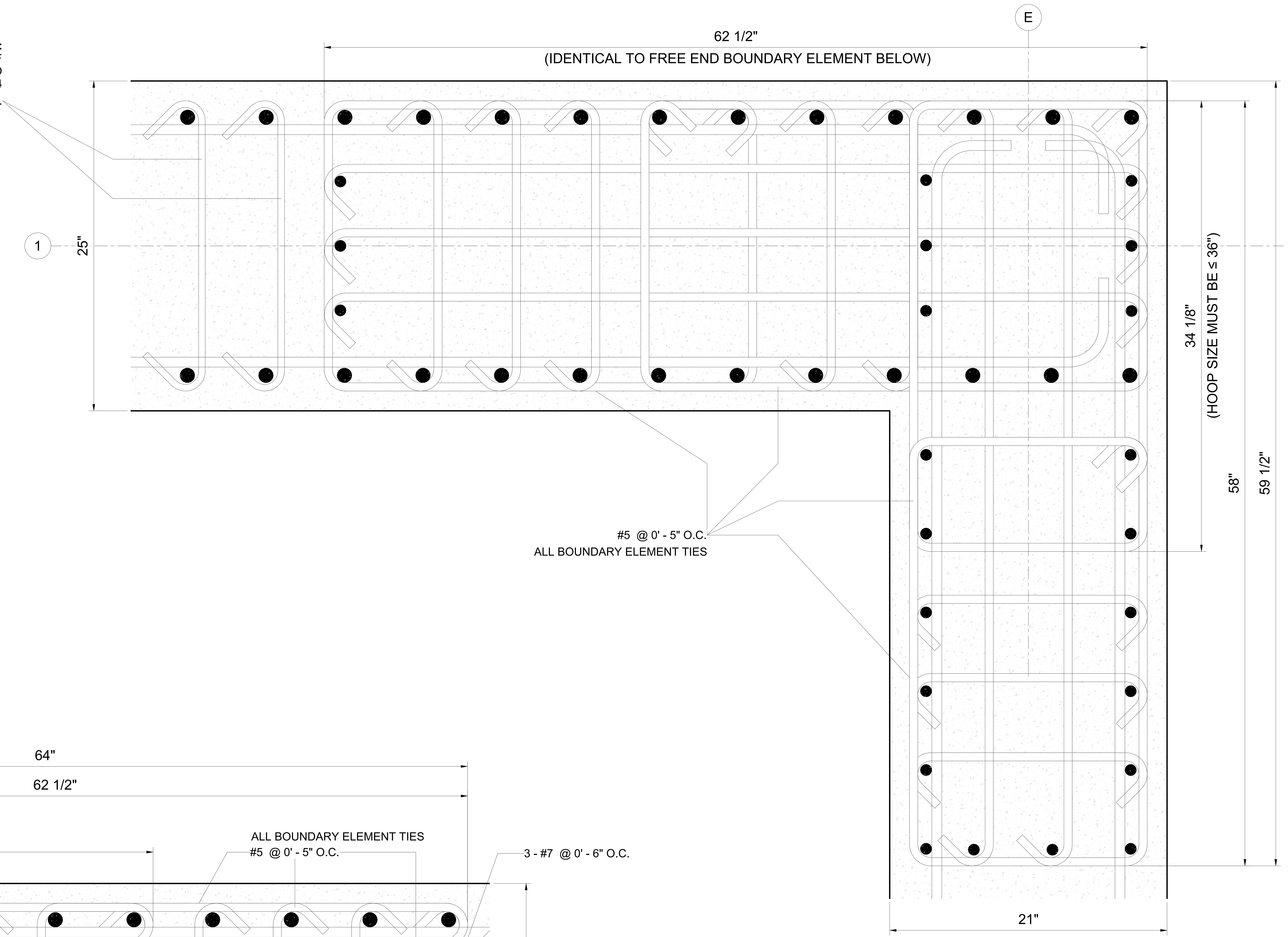
FINAL
CORE WALL DETAIL

Project Number	CE 244 Semester Project
Date	December 12, 2025
Drawn By	CQ
Checked By	FP, FD, ES

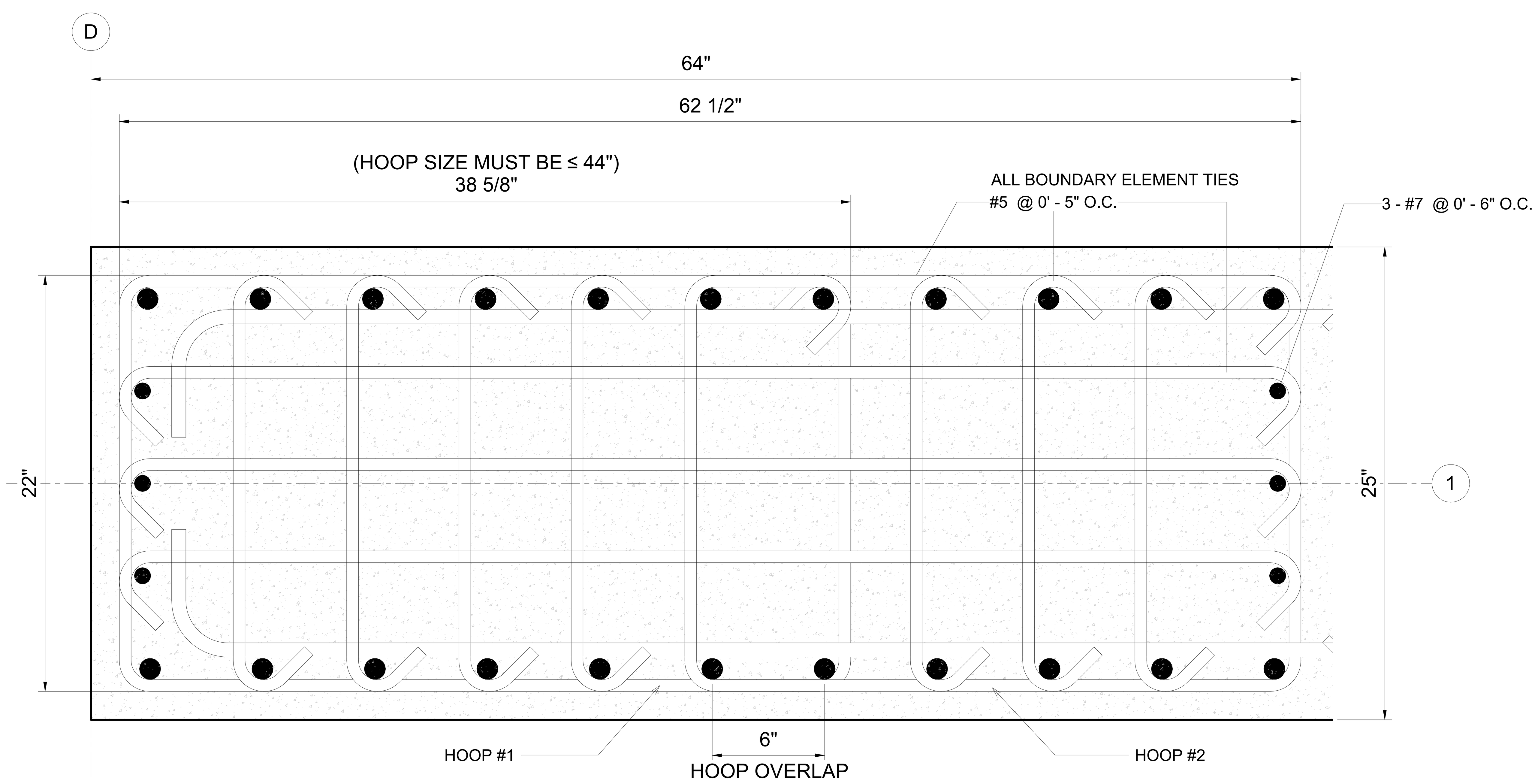
S2.03

Scale	As indicated

12/12/2025 3:28:07 AM



1 Corner Boundary Element
3" = 1'-0"



2 Free End Boundary Element
3" = 1'-0"

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No.	Description	Date

The City of Berkeley Council

1950 Oxford St.
Residential Building

FINAL BOUNDARY
ELEMENT DETAIL

Project Number	CE 244 Semester Project
Date	December 12, 2025
Drawn By	CQ
Checked By	FP, FD, ES

S2.04

Scale	3" = 1'-0"

12/12/2025 3:28:21 AM

4 References

[1] T. Opabola, "Ce 244 – reinforced concrete structures, lectures. semm ms program," 2025. Lecture slides.

[2] American Society of Civil Engineers, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. Reston, Virginia: American Society of Civil Engineers, 2022. ASCE Standard ASCE/SEI 7-22. Library of Congress Control Number: 2021951104.

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[4] International Code Council, "International building code (ibc) 2021," 2021.

[5] ACI Committee 318, *Building Code Requirements for Structural Concrete (ACI 318-25) and Commentary (ACI 318R-25)*. Farmington Hills, Michigan: American Concrete Institute, 2025.

[6] Computers and I. Structures, "Etabs: Building analysis and design software," 2024. WalnutCreek,CA, USA.

[7] F. L. Pfeffer, "ACSAHE: Axial–Curvature Strength Analysis of Heterogeneous Elements." https://github.com/facundo-pfeffer/ACSAHE, 2024. Open-source Python application for reinforced concrete P–M analysis. Available at: https://facundo-pfeffer.github.io/ACSAHE/.

[8] J. K. Wight and J. G. MacGregor, *Reinforced Concrete: Mechanics and Design*. Pearson Education, 6th ed., 2012.

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[10] S. Abdullah and J. Wallace, "Drift capacity of rc structural walls with special boundary elements," *Aci Structural Journal*, vol. 116, pp. 183–194, 01 2019.

[11] J. Moehle, *Seismic Design of Reinforced Concrete Buildings*. New York: McGraw-Hill Education, 1st edition ed., 2015.

[12] H. García, "Ce 244 – reinforced concrete structures, discussion sessions," 2025. Discussion materials.

Appendix A: ASCE Hazards Report

This appendix documents the site seismic hazard parameters used to build the design response spectrum. Using the site data and soil classification summarized in Section 1.0.2, the spectrum was constructed following ASCE/SEI 7–22, Ch. 11[2]:

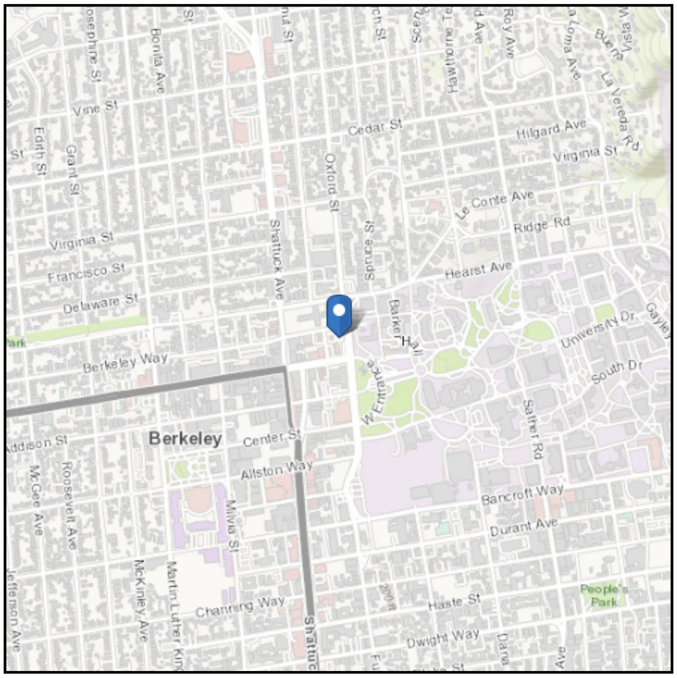
The complete ASCE Hazards Report output (address, coordinates, risk category, soil class, and parameter tables) is appended on the next pages.

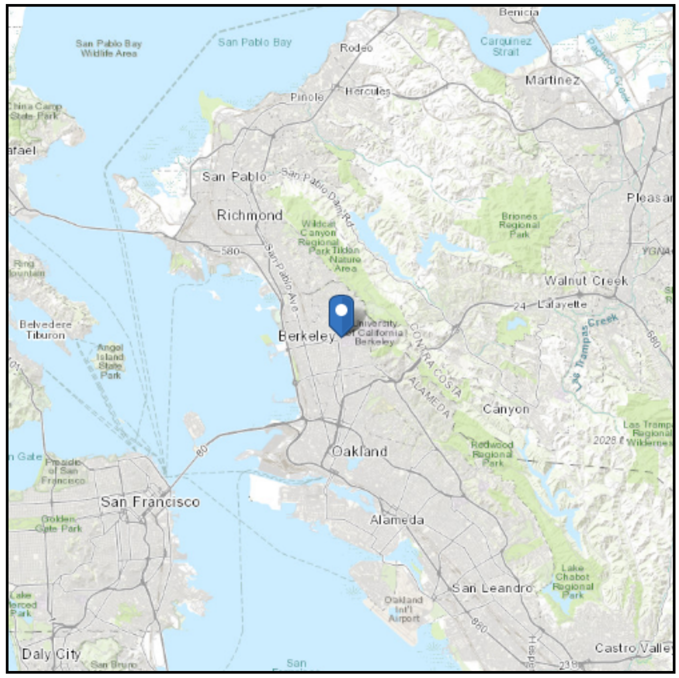
ASCE Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-22
Risk Category: II
Soil Class: C - Very Dense Soil and Soft Rock

Latitude: 37.873
Longitude: -122.2665
Elevation: 214.07919039384095 ft (NAVD 88)





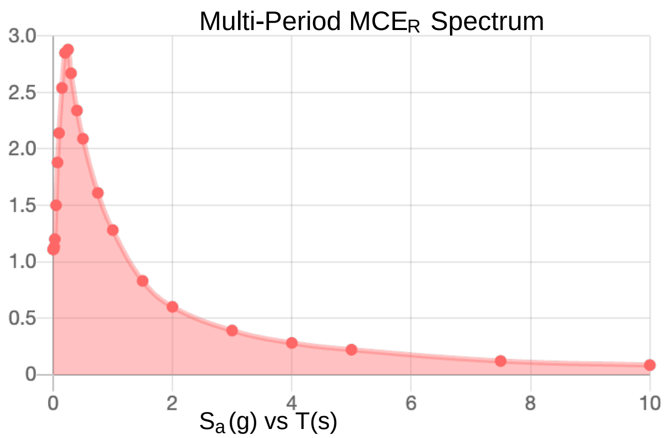
Seismic

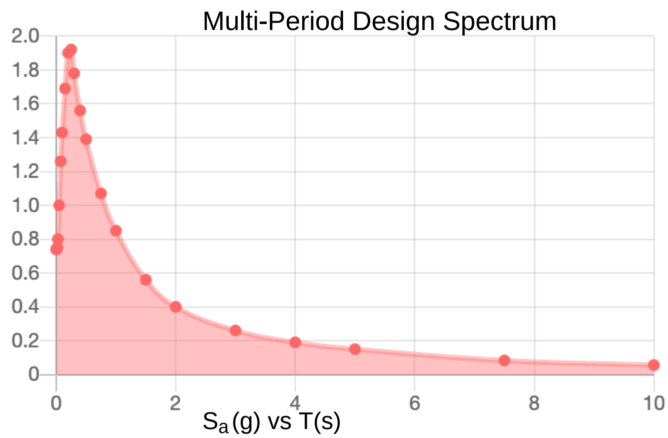
Site Soil Class: C - Very Dense Soil and Soft Rock

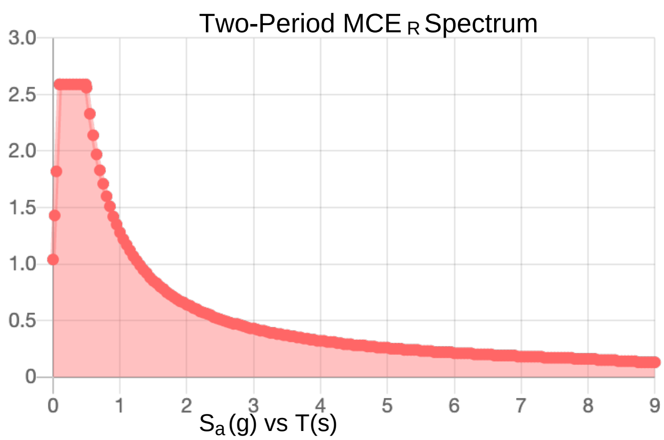
Results:

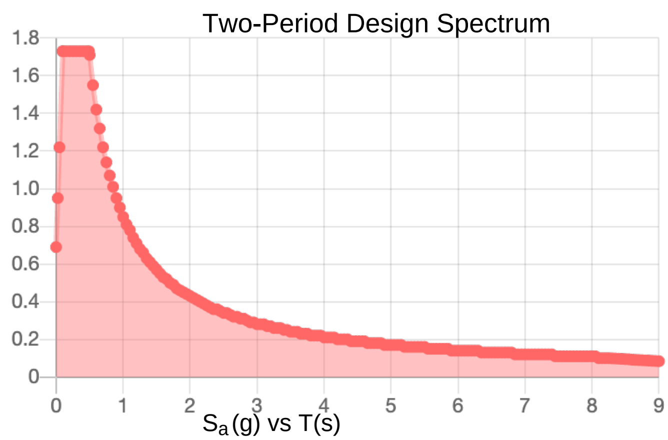
PGA_M :	0.92	T_L :	8
S_{MS} :	2.59	S_S :	2.49
S_{M1} :	1.28	S_1 :	0.92
S_{DS} :	1.73	V_{S30} :	530
S_{D1} :	0.85		

Seismic Design Category: E









MCE$_R$ Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Design Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Data Accessed: **Mon Nov 03 2025**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-22 and ASCE/SEI 7-22 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-22 Ch. 21 are available from USGS.

The ASCE Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

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Appendix B: ACSAHE Report

A report of the results of ACSAHE is included in the next page.

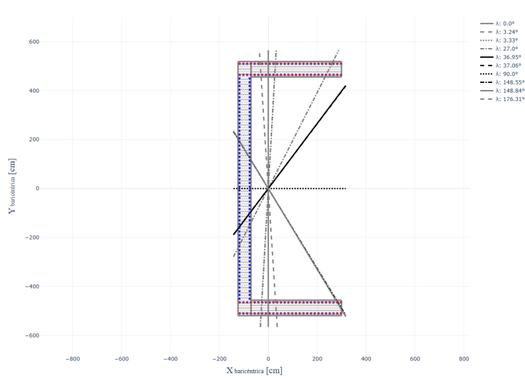
ACSAHE

Final Design Homework 10

MATERIAL CHARACTERISTICS

MATERIAL	DESCRIPTION	VALUE
Concrete	Concrete quality. The number indicates the characteristic compressive strength expressed in ksi.	7
Passive Steel	Type of steel selected for passive reinforcement.	Grade 60
Transverse Reinforcement	Type of transverse reinforcement selected.	Stirrups

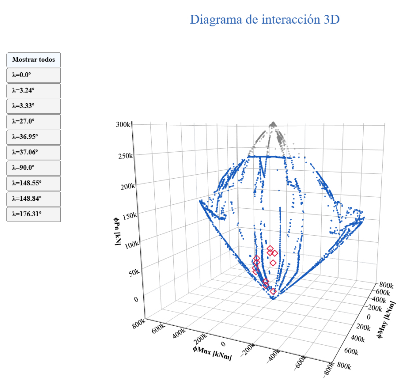
SECTION AND DISCRETIZATION



SECTION PROPERTIES

PROPERTY	DESCRIPTION	VALUE
Area	Gross concrete section area.	15861.5 in^2
Ix	Moment of inertia about the x-axis.	388779.3 in^4
Iy	Moment of inertia about the y-axis.	40515.3 in^4
ρ	Geometric reinforcement ratio for passive reinforcement.	1.63%
Discretization	Type of discretization chosen: Fine.	ΔX=7.39 cm ΔY=20.78 cm

RESULT



ADDRESS
Instituto de Mecánica Aplicada y Estructuras (IMAE).
Riobamba 250 bis, CP2000.
Rosario, Santa Fe, Argentina.

TECHNICAL SUPPORT
facundo.pfeffer@gmail.com [Facundo L. Pfeffer]
(Por favor, incluir planilla utilizada junto con el problema).

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(Por favor, incluir planilla utilizada junto con el problema).