Harmonizing Performance-Based Seismic Design - A Comparative Study of US and Japanese Practices for a 10-Story RC Archetype

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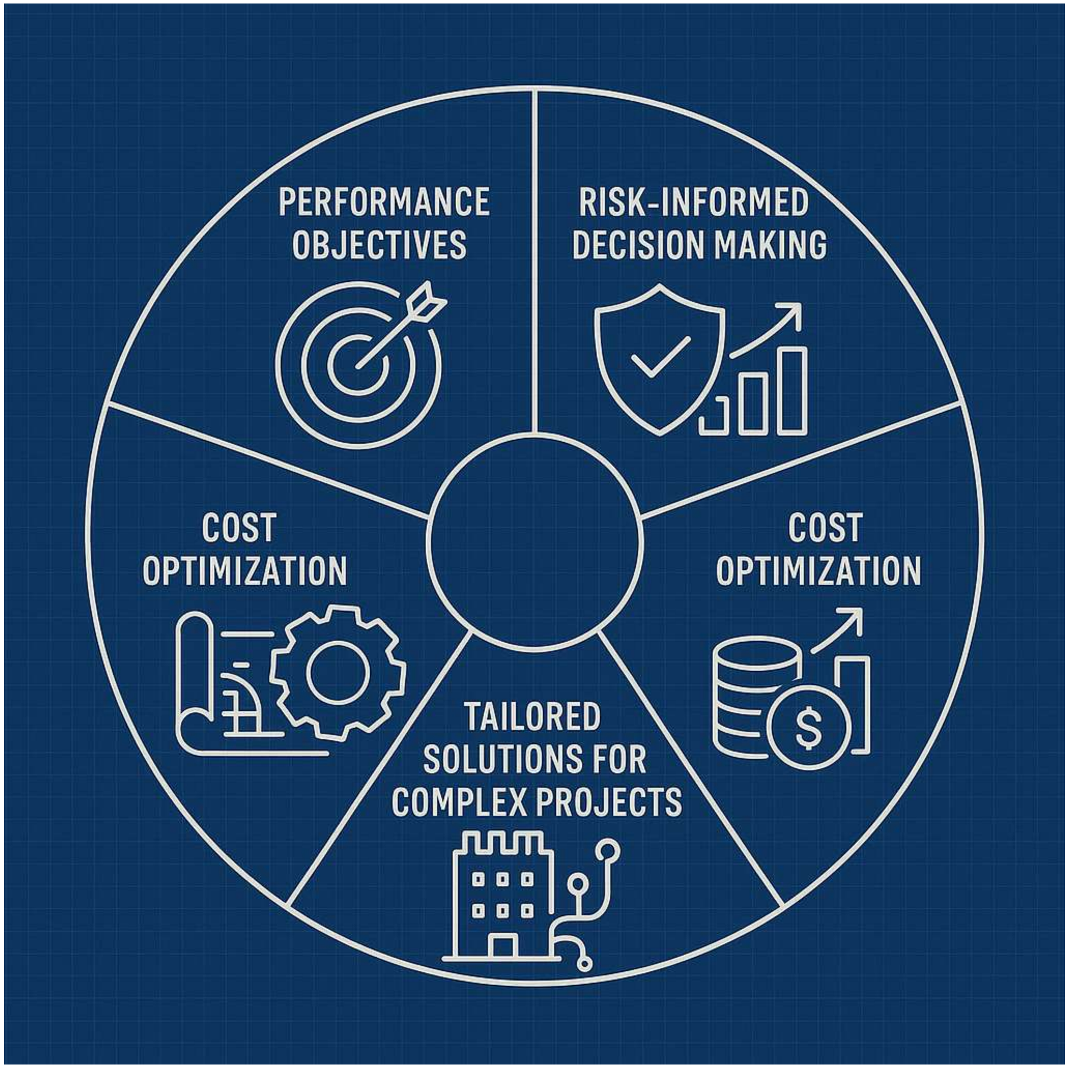
WHY PERFORMANCE-BASED SEISMIC DESIGN?



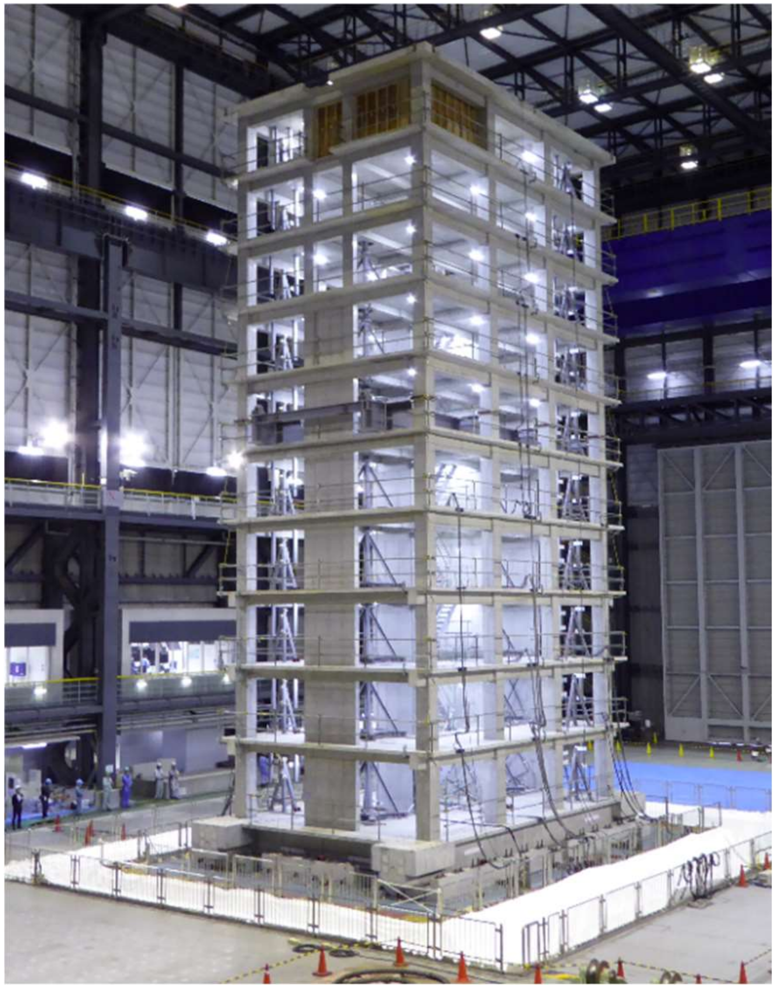
CASE STUDY BUILDING

Continuation of two previous studies . . .

- Kang et al. → Analyzes results from a 10-story building put through E-Defense earthquake simulation
- Unal et al. → Replicates these structures in ETABS to check compliance with ASCE 7-16 and ACI 318-19

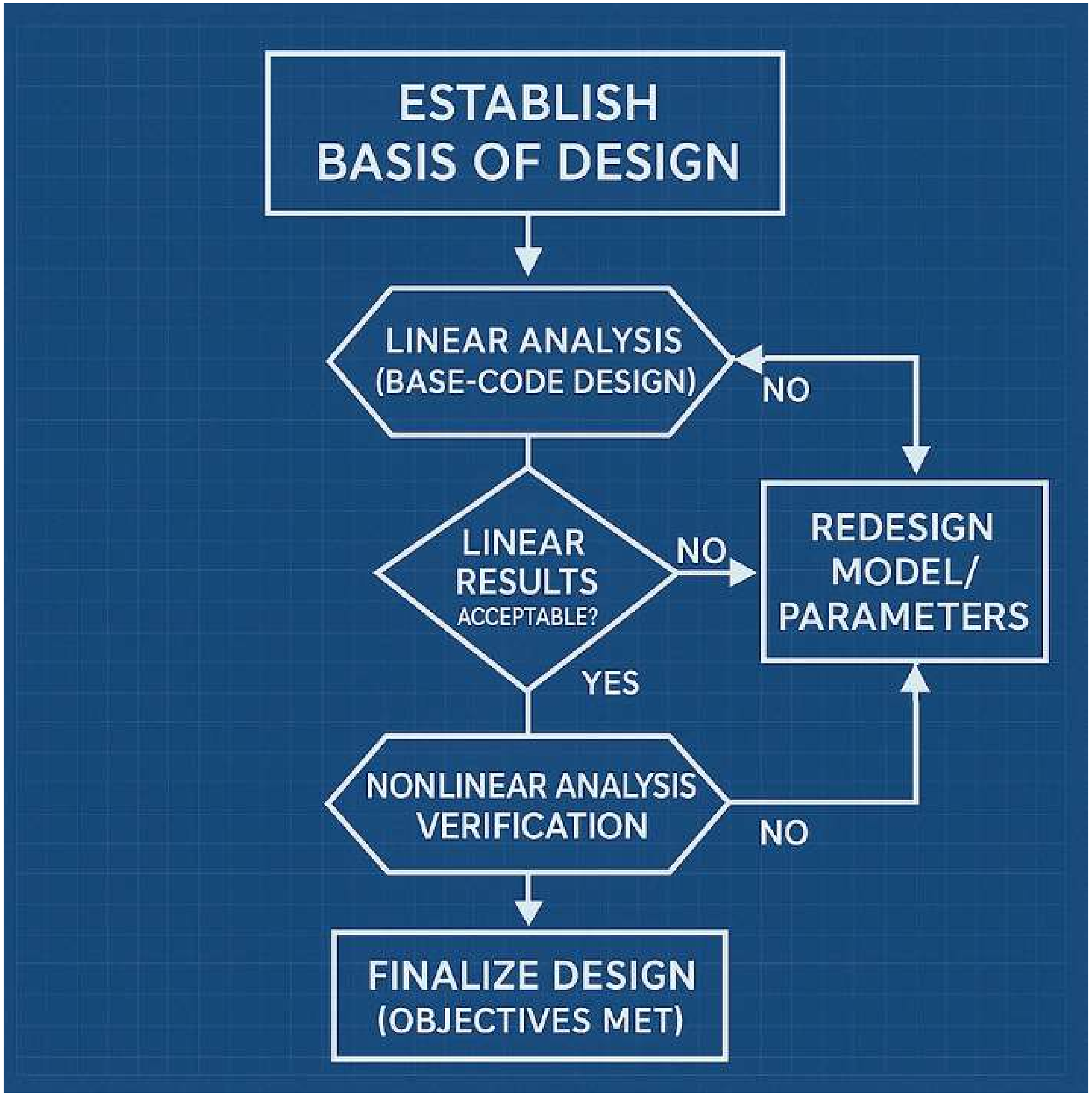
Goal: Compare US and Japan engineering practices and governing building standards. Our task in the past months:

- Design a 10-story RC structure that captures the nature of American building codes, standards, and guidelines



The structure tested in Kang et al.

PROCESS



ESTABLISH
BASIS OF DESIGN
LINEAR ANALYSIS
(BASE-CODE DESIGN)
NO
LINEAR
RESULTS
ACCEPTABLE?
NO
REDESIGN
MODEL/
PARAMETERS
YES
NONLINEAR ANALYSIS
VERIFICATION
NO
FINALIZE DESIGN
(OBJECTIVES MET)

INITIAL CONCEPT: BUILDING DESCRIPTION

Lateral force resisting systems are chosen to match the structures in Kang et al:

- Y-Direction: Special reinforced concrete walls
- X-Direction: Special reinforced concrete moment frames
- Interior gravity columns

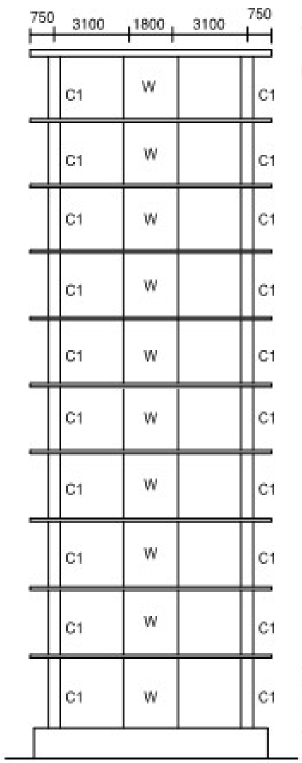
MATERIAL PROPERTIES

Concrete: f'_c = 33 MPa (4.78 ksi)

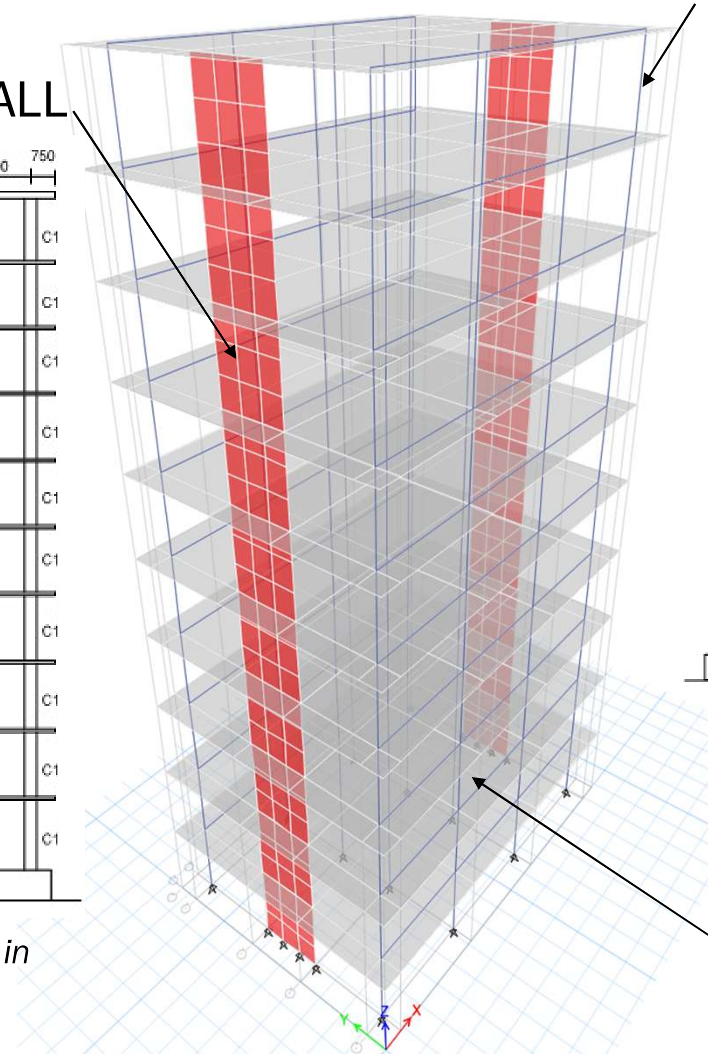
Rebar: SD390 steel, f_y = 390 MPa (56.6 ksi)

In our analysis, we used ASTM Grade 60 rebar

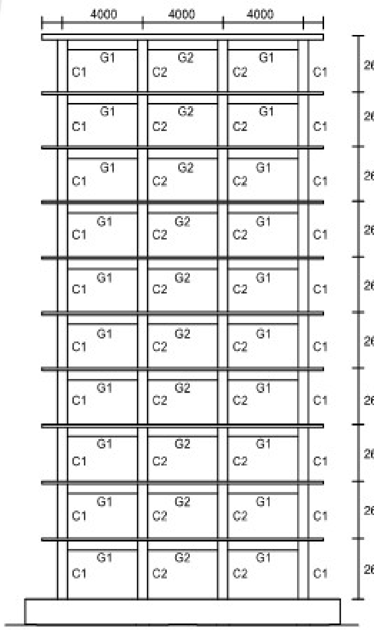
SHEAR WALL



measurements in mm.



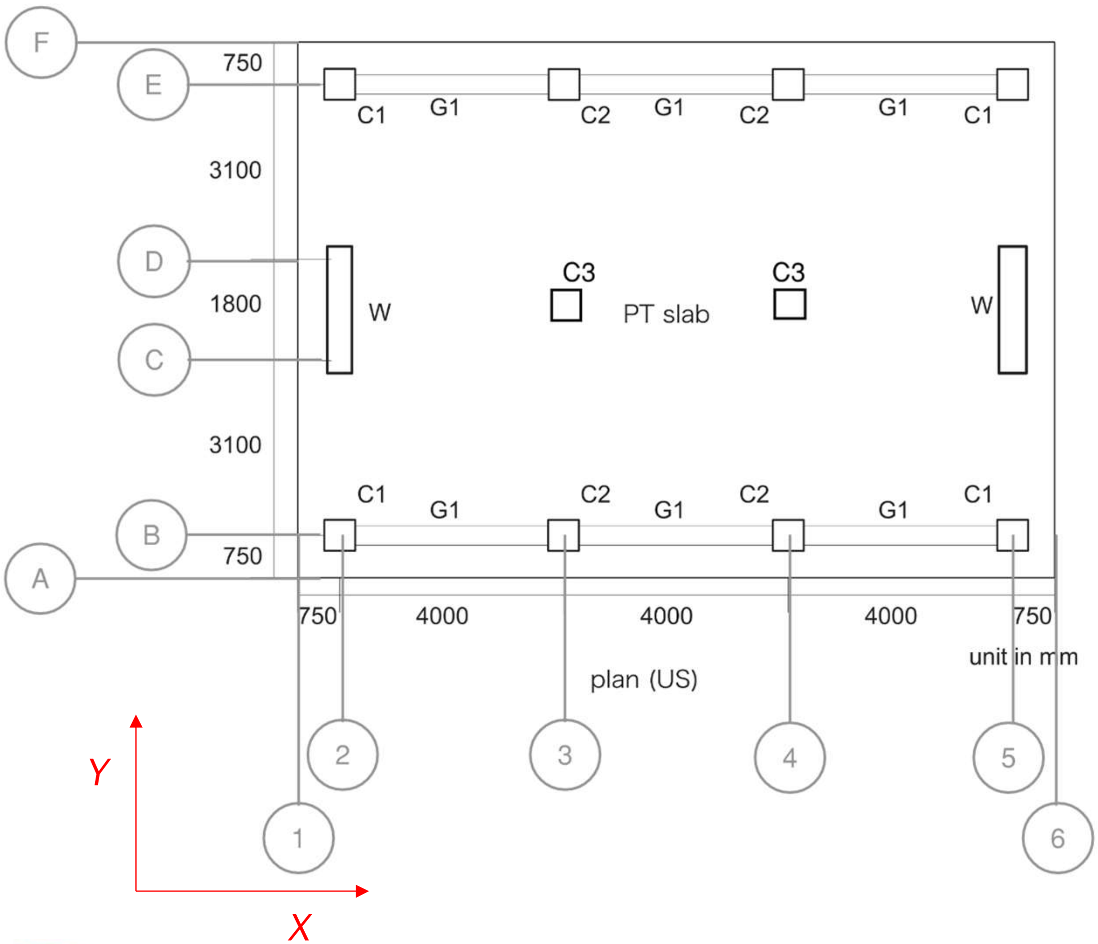
MOMENT FRAME



measurements in mm.

PT SLAB

INITIAL CONCEPT: FLOOR PLAN



MEMBER DIMENSIONS

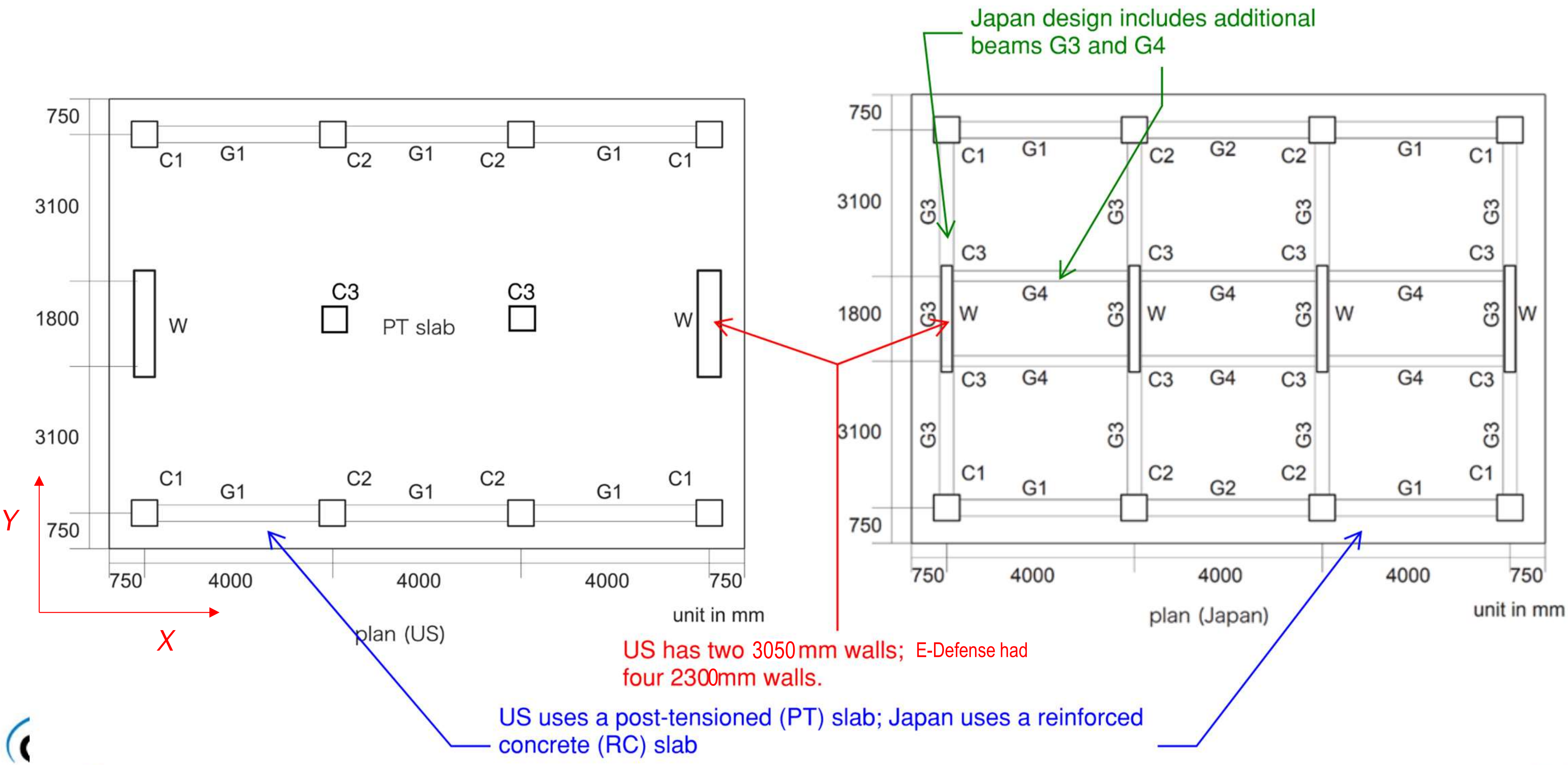
Units in mm (ft.).

Floor Height	Columns (C1, C2, C3)	Beam (G1, G2)	Wall (W)	Slab (S)
	B × D	B × D	t	T
2600 (8.5)	500 × 500 (1.6 × 1.6)	350 × 550 (1.1 × 1.8)	460 (1.5)	120 (0.4)

Note: To account for the overlap between the slab and beam, we applied a reduction factor in the linear analysis model.

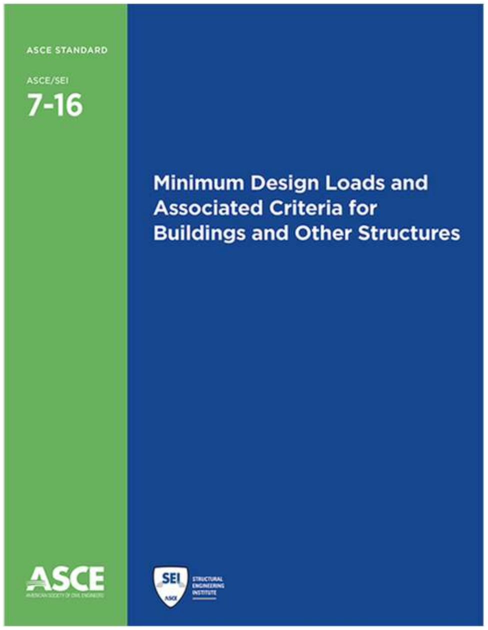
INITIAL CONCEPT: US/JAPAN STRUCTURES

The US and Japanese teams agreed on the following floor plan designs. They have some differences:

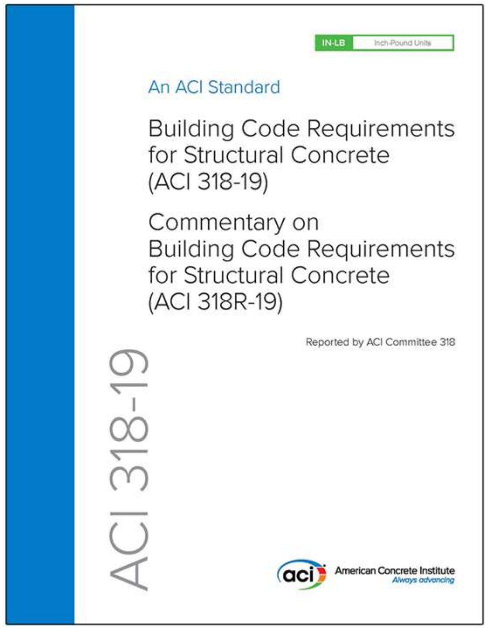


BASIS OF DESIGN

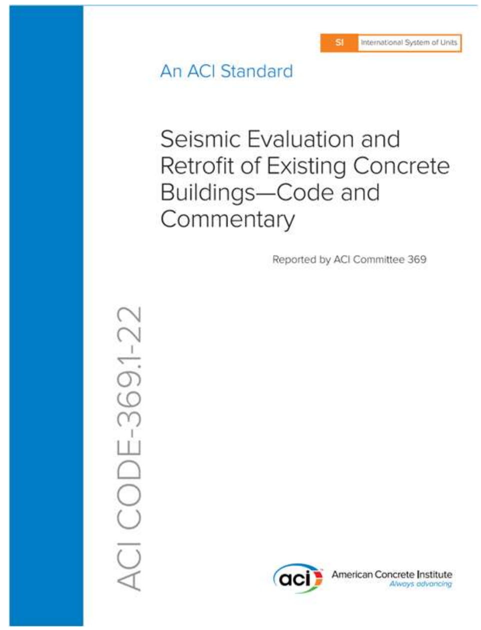
GOVERNING CODES AND STANDARDS



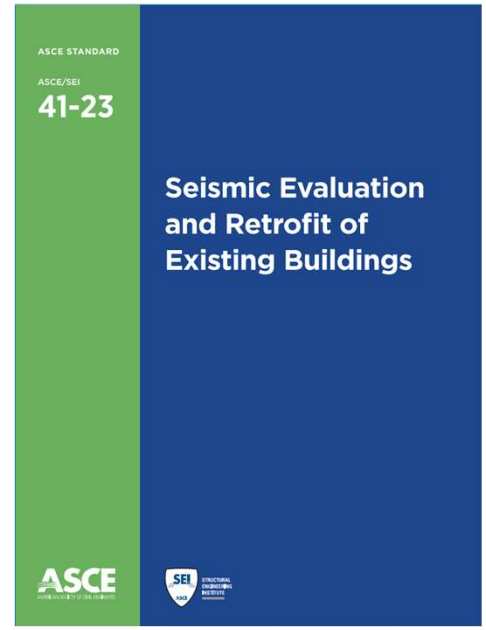
ASCE 7-16



ACI 318-19

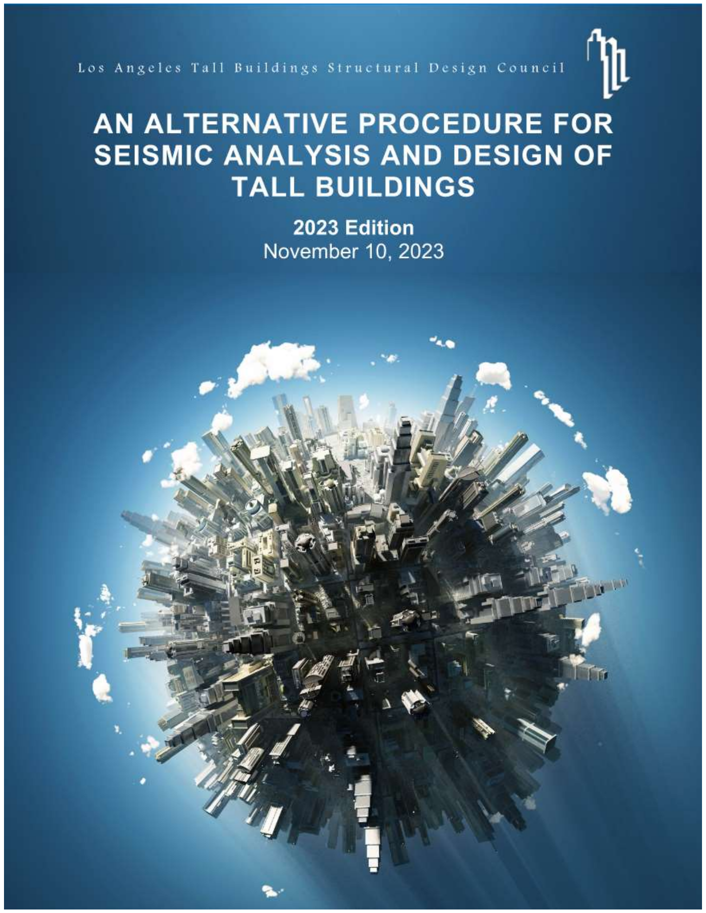


ACI CODE-369.1-22

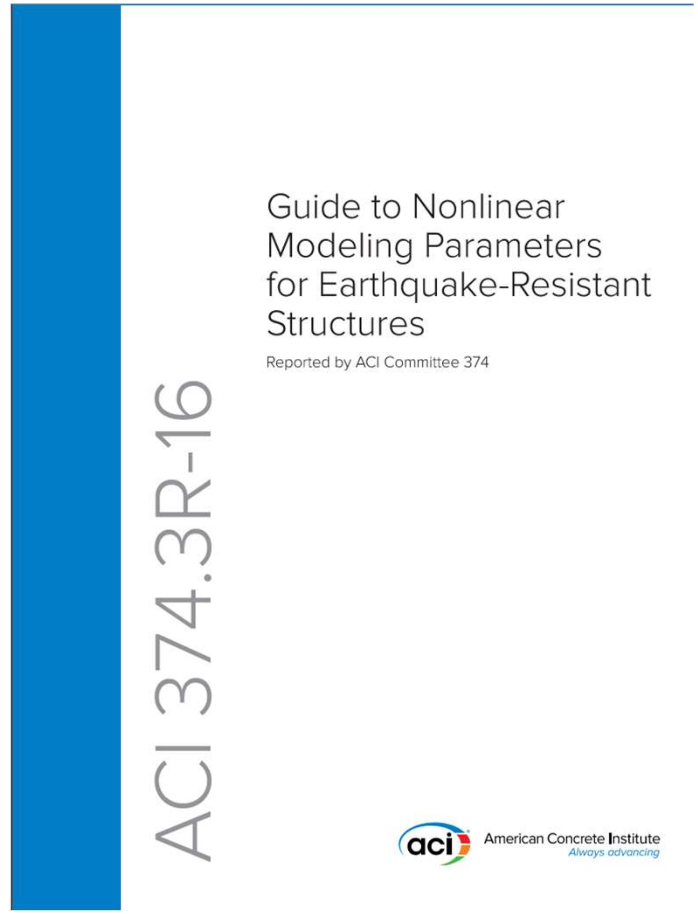


ASCE 41-23

GUIDELINES



LATBSDC



ACI 374.3R-16

APPENDIX A—DESIGN VERIFICATION USING NONLINEAR RESPONSE HISTORY ANALYSIS

A.2.4 All concrete structures designed or verified by this Appendix shall be proportioned and detailed as required by Chapter 18 and the requirements of A.12 when applicable.

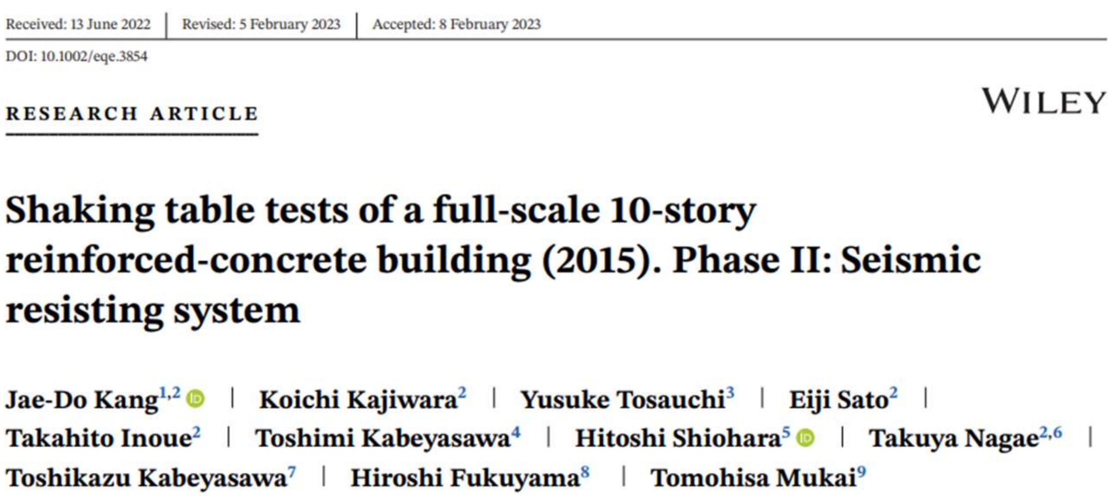
A.2.5 It shall be permitted to use the provisions of Appendix A to demonstrate the adequacy of a structural system as required by 18.2.1.7.

A.2.6 Independent structural design review consistent with A.13 shall be required for use of Appendix A.

A.2.7 The licensed design professional shall provide justification for any interpretation required for the application of Appendix A, and if accepted by the independent structural design reviewers, justification shall be provided to the building official for acceptance.

OTHER REFERENCES: TECHNICAL PAPERS

- *Shaking Table Tests of a Full-Scale 10-Story Reinforced-Concrete Building* by Kang et al., 2022
- *SP-339-11: Recommendations for Modeling the Nonlinear Response of Flexural Concrete Walls Using Perform* by Lowes et al., 2020
- *Assessment of 2015 and 2018 E-Defense 10-Story Reinforced Concrete Buildings Based on ACI 318 and ASCE 7 Provisions* by Unal et al., 2021

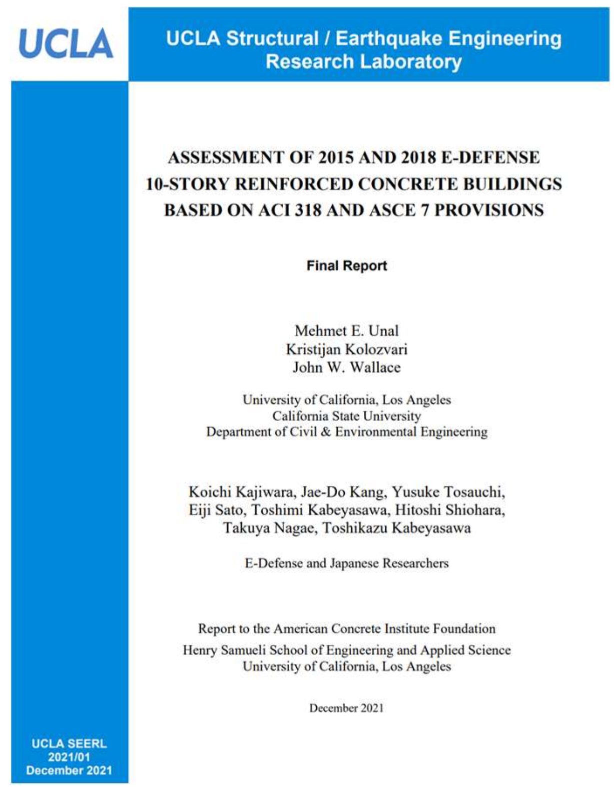

Received: 13 June 2022 | Revised: 5 February 2023 | Accepted: 8 February 2023
DOI: 10.1002/eqe.3854

RESEARCH ARTICLE

WILEY

Shaking table tests of a full-scale 10-story reinforced-concrete building (2015). Phase II: Seismic resisting system

Jae-Do Kang[1,2] | Koichi Kajiwara[2] | Yusuke Tosauchi[3] | Eiji Sato[2] | Takahito Inoue[2] | Toshimi Kabeyasawa[4] | Hitoshi Shiohara[5] | Takuya Nagae[2,6] | Toshikazu Kabeyasawa[7] | Hiroshi Fukuyama[8] | Tomohisa Mukai[9]


UCLA

UCLA Structural / Earthquake Engineering Research Laboratory

ASSESSMENT OF 2015 AND 2018 E-DEFENSE 10-STORY REINFORCED CONCRETE BUILDINGS BASED ON ACI 318 AND ASCE 7 PROVISIONS

Final Report

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Koichi Kajiwara, Jae-Do Kang, Yusuke Tosauchi, Eiji Sato, Toshimi Kabeyasawa, Hitoshi Shiohara, Takuya Nagae, Toshikazu Kabeyasawa

E-Defense and Japanese Researchers

Report to the American Concrete Institute Foundation
Henry Samueli School of Engineering and Applied Science
University of California, Los Angeles

December 2021

UCLA SEERL
2021/01
December 2021

OTHER REFERENCES: ACI 318-19 CODE CASES

From *Concrete* International, Vol. *45, Issue 6*

- Q&A drafted by the ACI Committee 318 that answers questions from groups such as the LA Tall Buildings Structural Design Council regarding ACI 318-19
- Proposes ACI 318 revisions
- Used in this structure's wall design

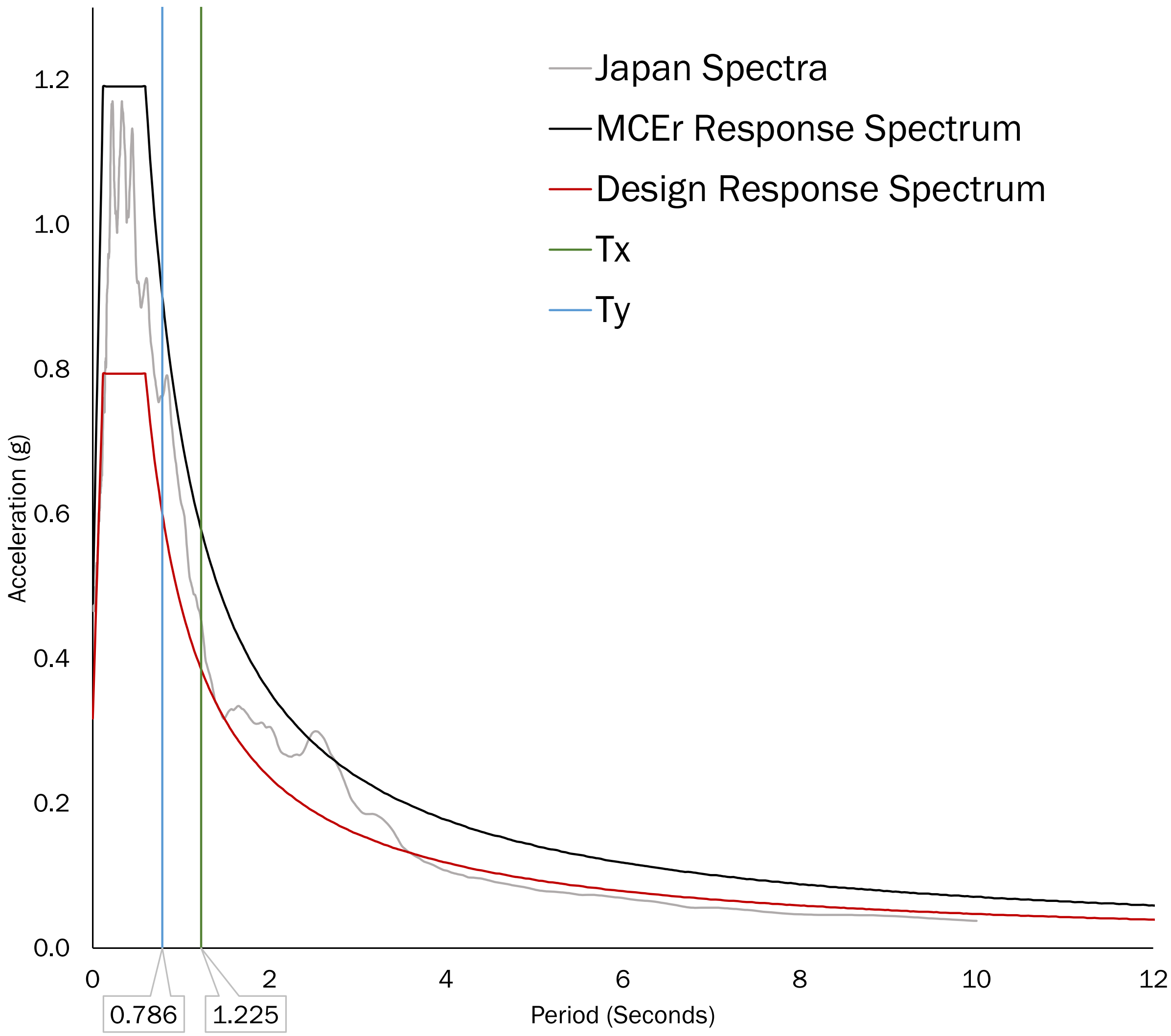
Concrete international

The Magazine of the Concrete Community

COMPONENT ACCEPTANCE CRITERIA

Component	Demands	Analysis Procedure	Action	Action Type	Acceptance Criteria	Capacity
Concrete Walls	ACI 318 A.5	NDP	V	D	ASCE 41-23 Table 7.4.1.1.1a	ACI 318
			M		ASCE 41-23 Table 7.4.1.1.2	
Concrete Columns	ACI 318 A.5	NDP	V	D	ACI 374.3R-16 Table 4.1.1	ACI 318
			M			
			P_{min}			
			P_{max}			
Concrete Beams	ACI 318 A.5	NDP	V	D	ACI 374.3R-16 Table 4.2a	ACI 318
			M			
Concrete Beam-Column Joints	ACI 318 A.5	NDP	V	D	ACI 374.3R-16 Table 4.2b	ACI 318
			M			

SEISMIC AND GEOLOGIC HAZARD (ASCE 7-16 § 11.2)



6 response spectra from JCI

These response spectra were averaged, and a similar U.S. spectrum was identified from Northern California.

The ASCE 7 Hazard Tool was used for seismic response parameters:

	MCE_R (g)	Design (g)
S_S	1.19	0.794
S_1	0.759	0.506

SEISMIC HAZARD AND STRUCTURAL SYSTEMS

The structure is classified based on its site using the ASCE 7 Hazard Tool:

- Risk Category (ASCE 7-16 § 1.5.1) → *The structure is Risk Category II*
- Importance Factor, I_e (ASCE 7-16 Table 1.5-2) → I_e *= 1.0 in this structure*
- Seismic Design Category (ASCE 7-16 § 11.6) → *Seismic Design Category D*

Building Frame System	ASCE 7-16 Table 12.2-1 Factors			
	R	Ω_0	C_d	ρ (ASCE 7-16 § 12.3.4)
Special reinforced concrete wall	6	2 ½	5	1.3
Special reinforced concrete moment frame	8	3	5 ½	1.0

LOADING CRITERIA (ASCE 7-16 § 2.3 and § 12.4)

DEAD LOADS

Only considers weight of materials (self-weight) (ASCE 7-16 § 3.1.2):

- In ETABS, found through member dimensions and concrete weight
- Results verified through manual calculations
- No superimposed DLs

LIVE LOADS

ASCE 7-16 Table 4.3-1 provides live loads based on building's use.

- Office Loads: 50 psf (2.40 kN/m^2)
- Partition Loads as per ASCE 7-16 § 4.3.2: 15 psf (0.72 kN/m^2)

SEISMIC MASS

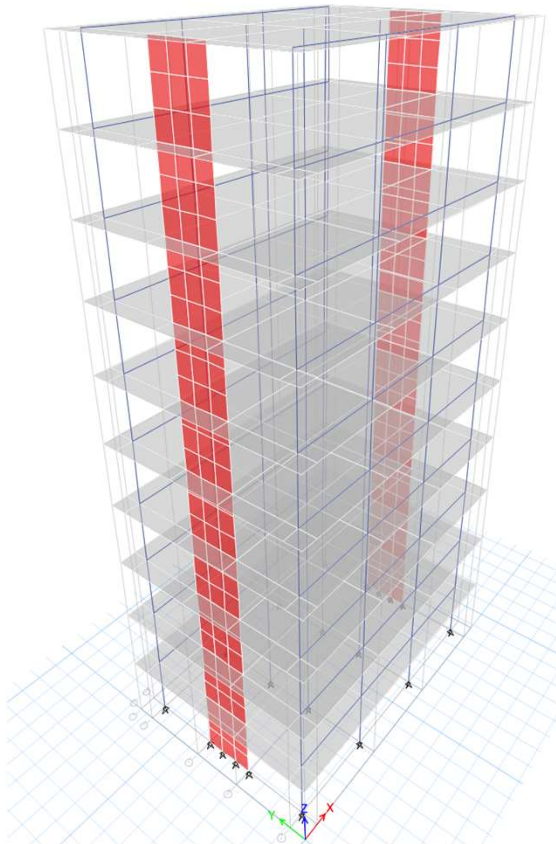
ASCE 7-16 § 12.7.2 states that the seismic mass includes:

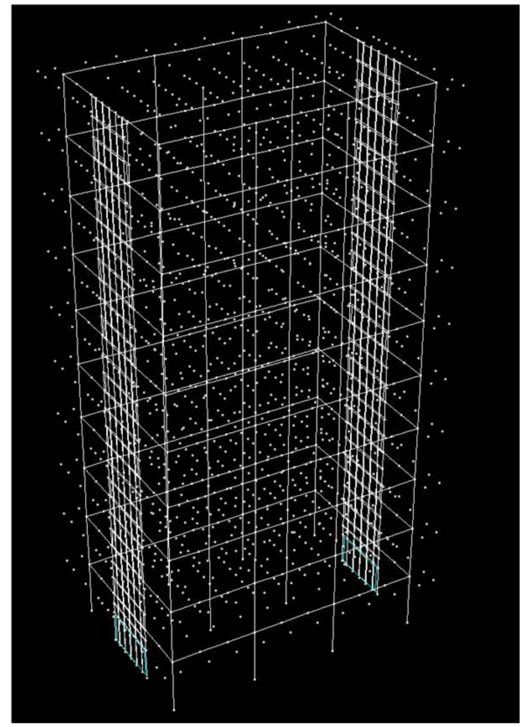
- Dead Loads above the base
- Partition Weight: 10 psf (0.48/m^2)

MATHEMATICAL MODELING

ASCE 7-16 § 12.1.1 requires a mathematical model to confirm adequacy.

A model for linear analysis was assembled, then a model for nonlinear analysis was assembled.





LINEAR

- Uses CSI ETABS 19
- Follows the Equivalent Lateral Force procedure detailed in ASCE 7-16 § 12.8

NONLINEAR

- CSI Perform 3D
- Uses ACI 318-19 Appendix A and ACI 374.3R-16

LINEAR LOAD COMBINATIONS

GOVERNING VERTICAL COMBINATION (ASCE 7-16 § 2.3.1)

$1.2D + 1.6L$

SEISMIC LOADS (ASCE 7-16 § 2.3.6)

$1.2D + E_v + E_h + (0.5)L$

$0.9D - E_v + E_h$

Reduced as allowed by ASCE 7-16 § 2.3.6

Where:

D = Dead Loads

L = Live Loads

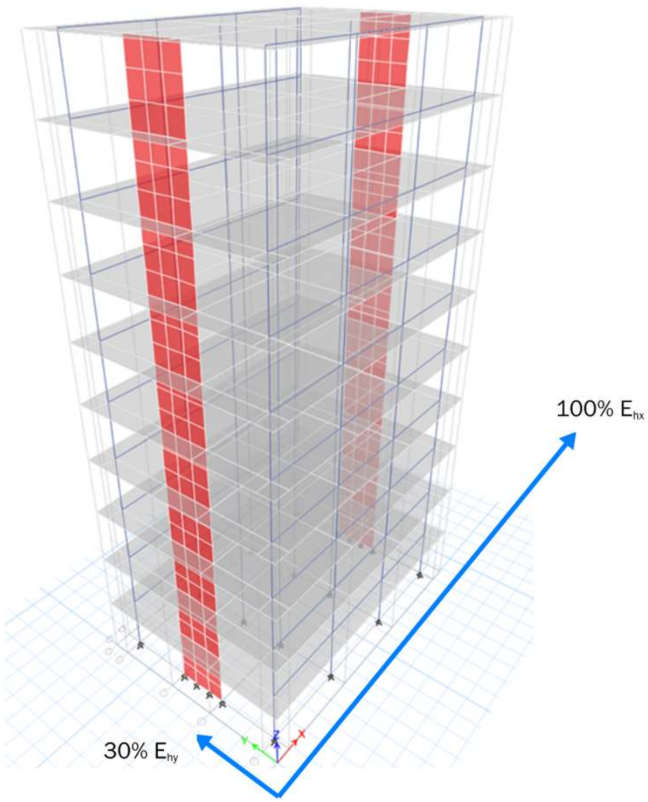
E_v = Vertical Earthquake Loads

E_h = Horizontal Earthquake Loads

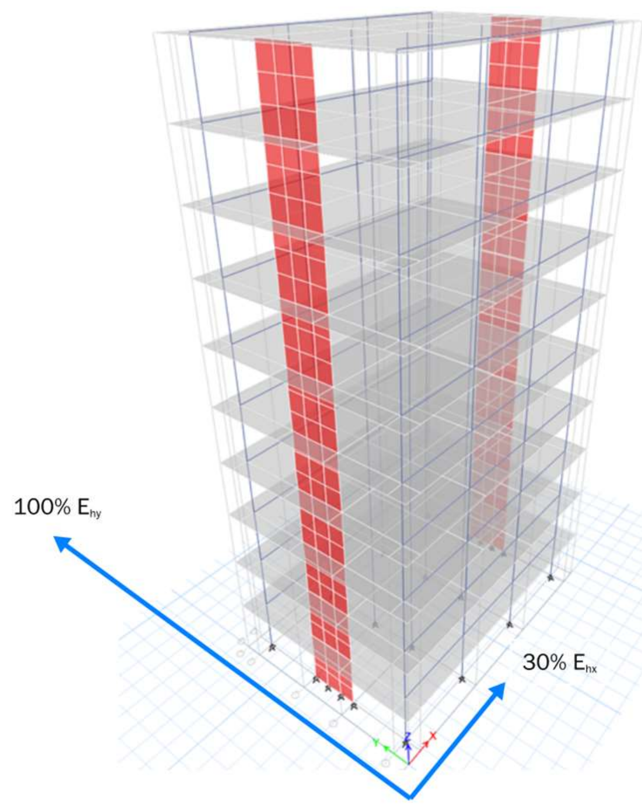
- *Consider **Accidental Torsion** (ASCE 7-16 § 12.8.4.2)*
- *Check if the **30% Rule** is required (ASCE 7-16 § 12.5.3.1)*
- *These considerations replace response history analysis (ASCE 7-16 § C12.5.3)*

30% RULE (ASCE 7-16 § 12.5.3.1)

Horizontal seismic loads, E_h, consider 100% of the seismic load in a direction plus 30% of the seismic load in the perpendicular direction. There are two cases:



$$E_h = E_{hx} + 0.3E_{hy}$$



$$E_h = 0.3E_{hx} + E_{hy}$$

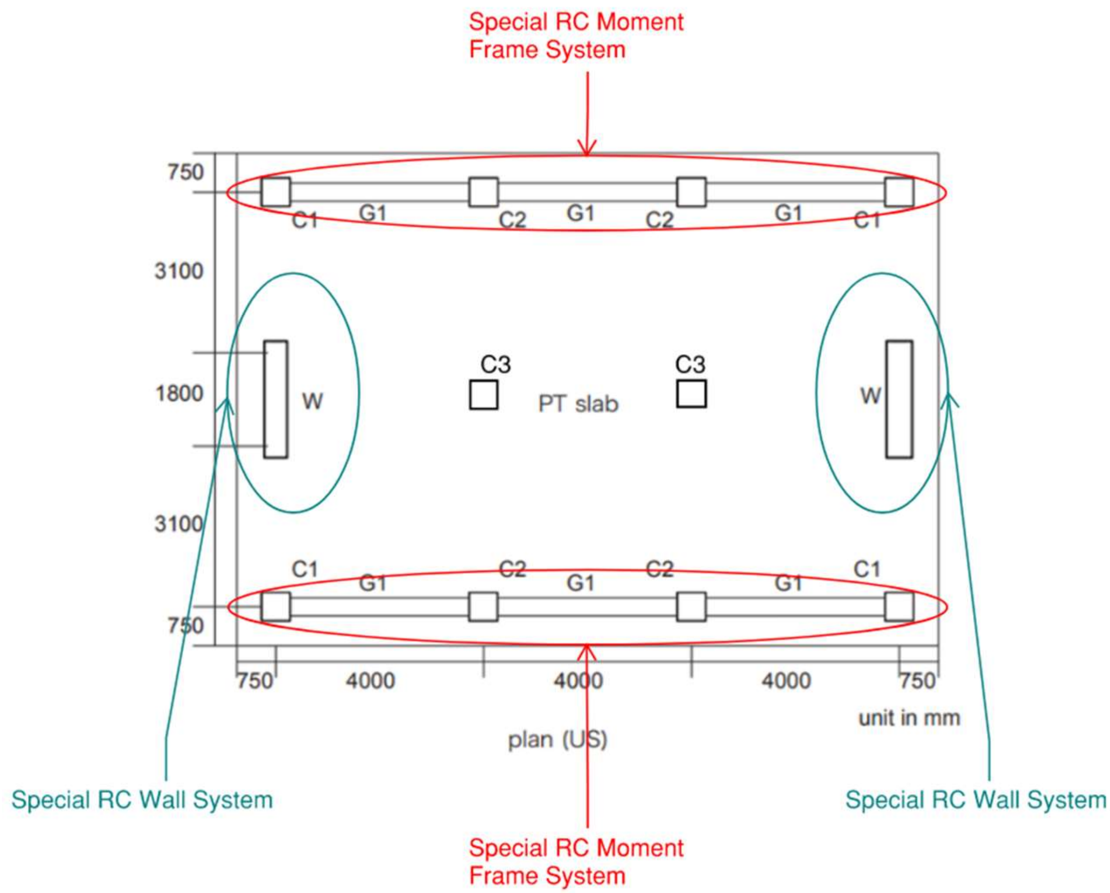
30% RULE (ASCE 7-16 § 12.5.3.1)

Horizontal seismic loads consider 100% of the seismic load in a direction plus 30% of the seismic load in the perpendicular direction.

This should be applied when the structure has:

1. Horizontal structural irregularity (ASCE 7-16 Table 12.3-1)

2. Elements part of 2+ seismic force resisting systems (ASCE 7-16 § 12.5.4)

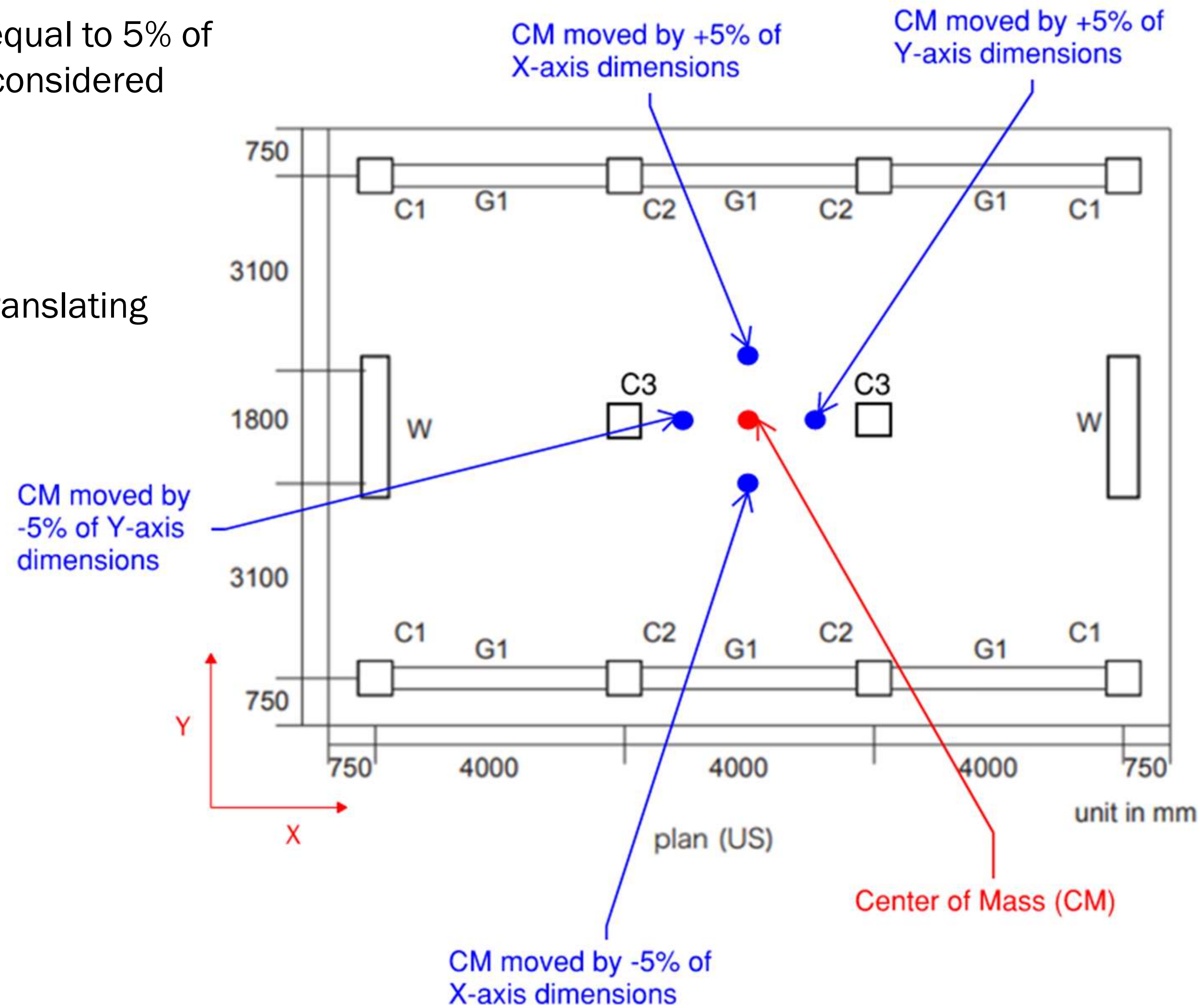


Structural system elements are separate.

ACCIDENTAL TORSION (ASCE 7-16 § 12.8.4.2)

Consider loads to occur at an eccentricity equal to 5% of the structure's width perpendicular to the considered direction.

Considered by translating center of mass:

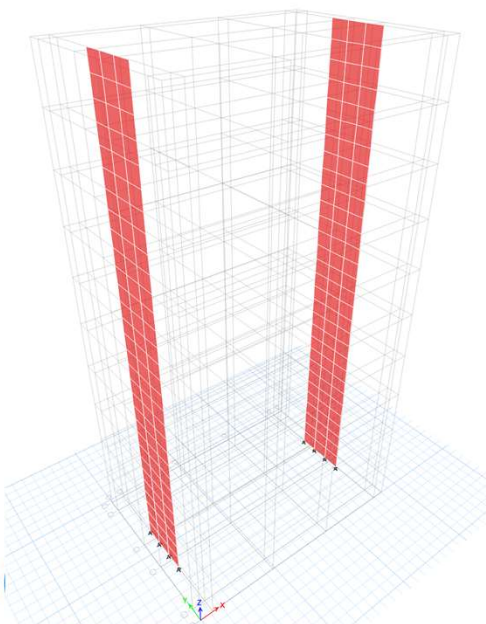


LINEAR ELEMENT STIFFNESS (ACI 318-19 T6.6.3.1.1(a))

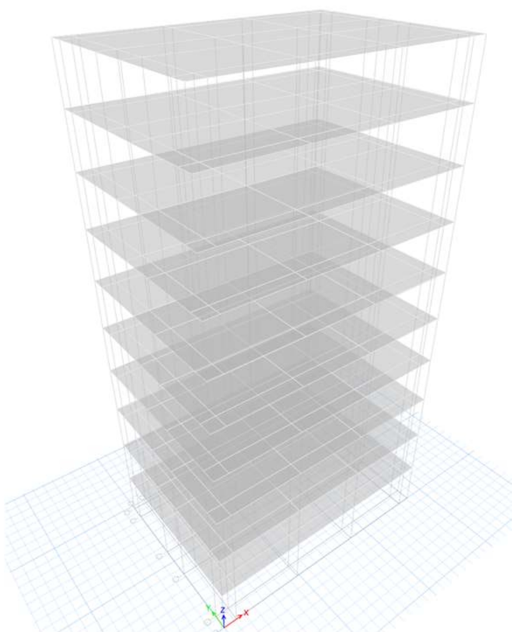
Concrete Member Type	Moment of Inertia
Walls (Cracked)	0.35Ig
Slabs (Out-of-Plane)	0.01Ig
Columns	0.70Ig
Beams	0.35Ig

Stiffness zeroed out to focus on walls' seismic resistance

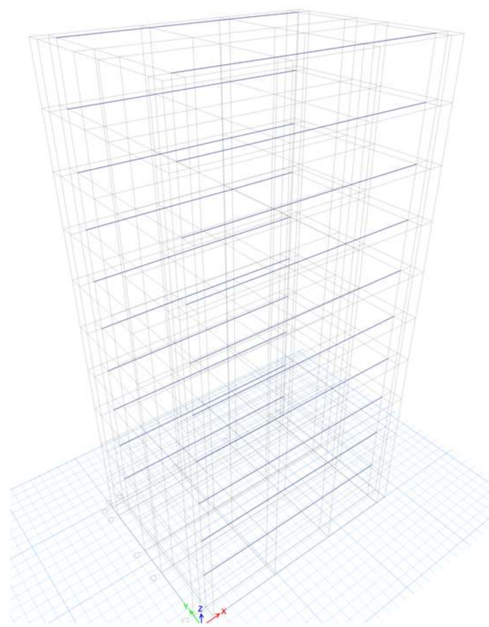
WALLS



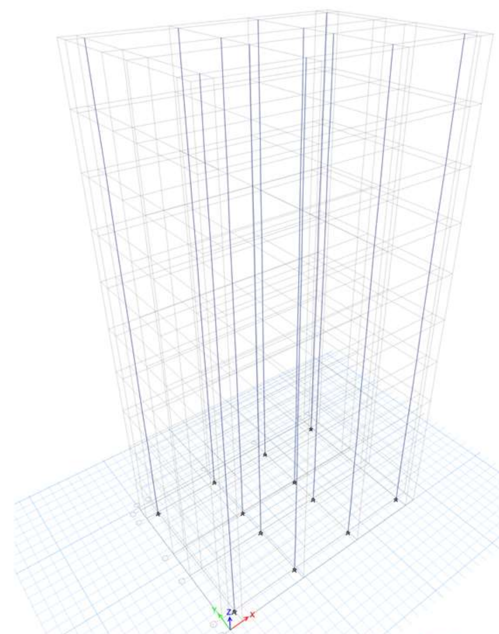
SLABS



BEAMS



COLUMNS



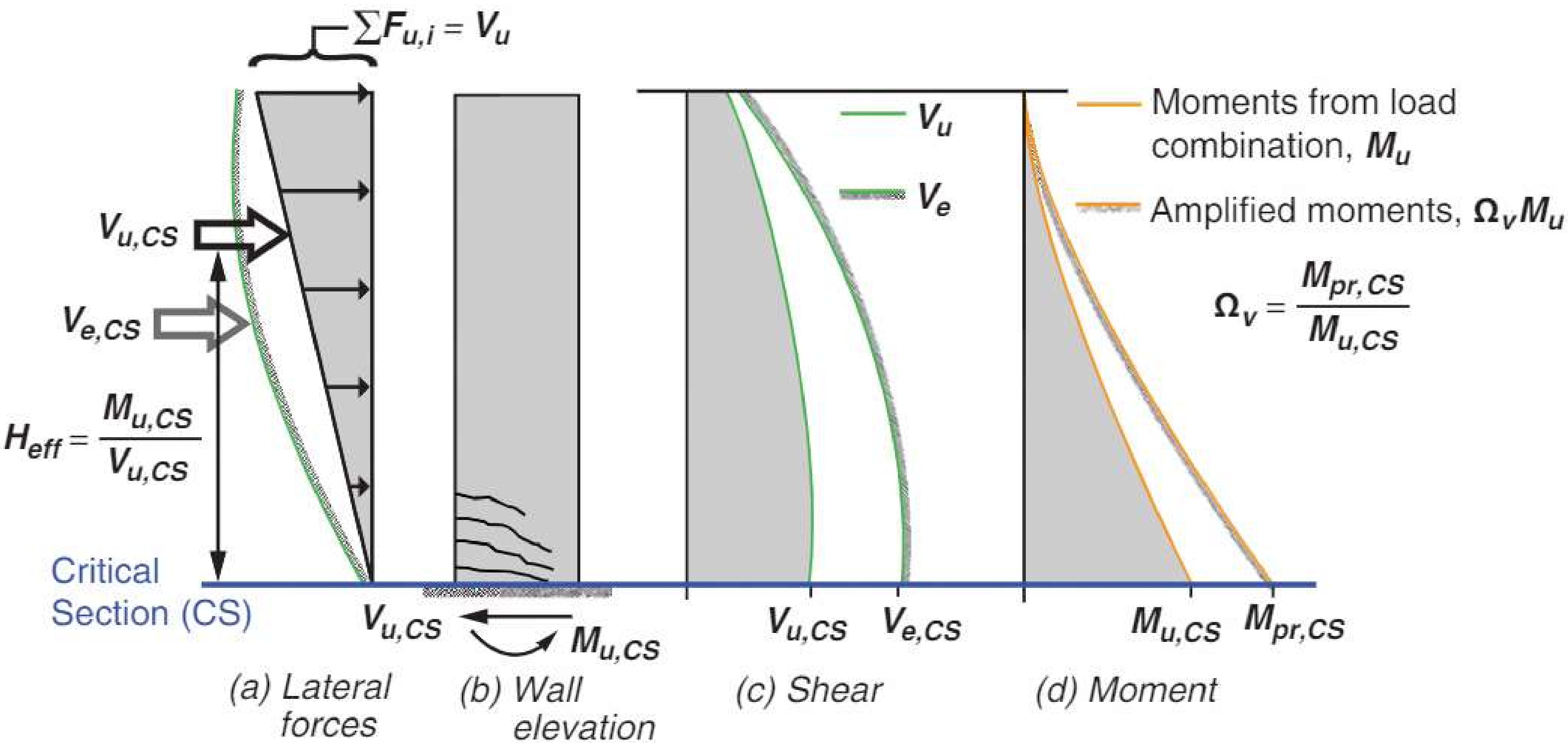


Fig. R18.10.3.1*—Determination of shear demand for walls with* $h_w/\ell_w \geq 2.0$ *(Moehle et al 2011).*

ACI 318 Section 18.10.3.1 provides the following equation to determine and verify V_e, the design shear force, associated with the walls. Note that the coefficient attached to V_u is changed to 2.5, a value taken from the ACI 318-19 Code Cases:

$$V_e = \Omega_v \omega_v V_u \leq 2.5 V_u \qquad \text{(ACI 318 Sec. 18.10.3.1)}$$

5.3 Special Boundary Elements

ACI 318 Section 18.10.6.1 requires a check for special boundary elements; the appropriate check for the designed wall is provided in ACI 318 Section 18.10.6.2 through the following equation:

$$\frac{1.5\delta_u}{h_{wcs}} \geq \frac{l_w}{600c} \qquad \text{(ACI 318 Eqn. 25.4.2.4a)}$$

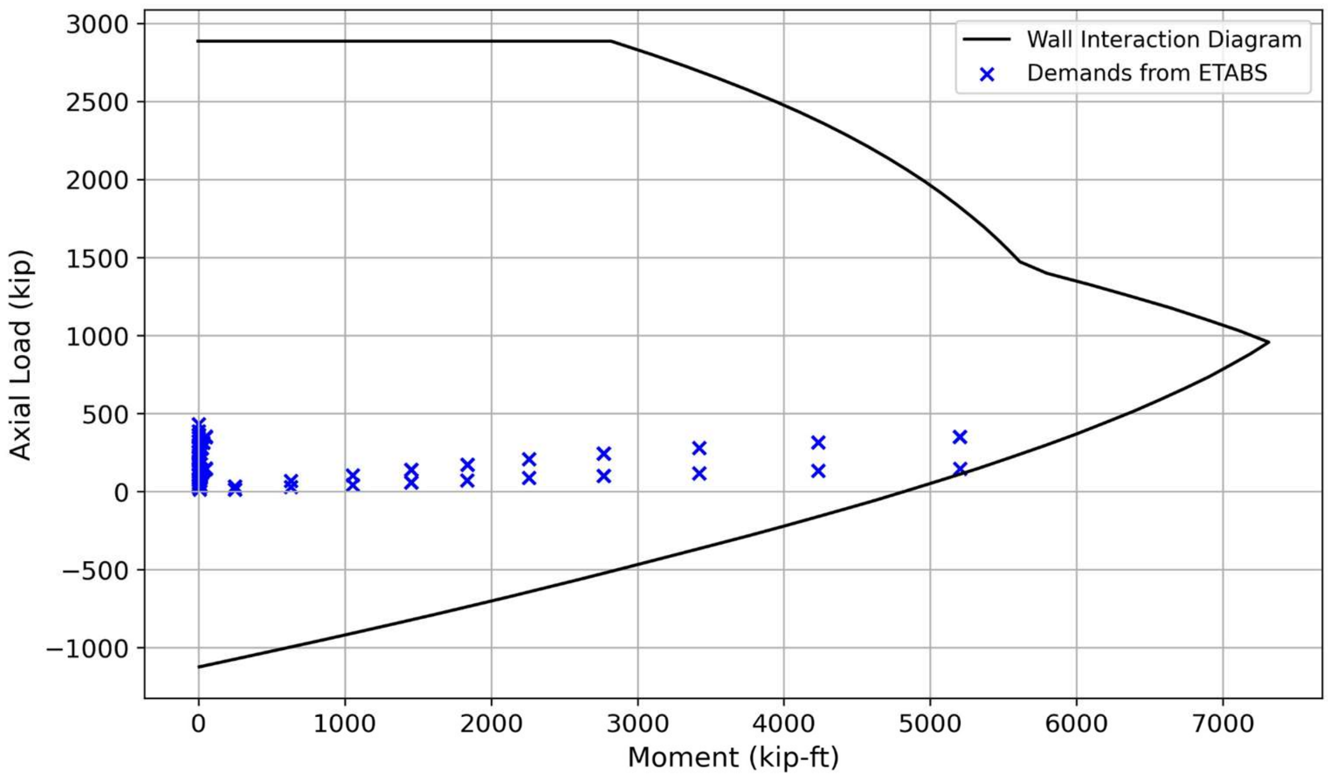
The section assigns a minimum value of 0.005 to δ_u/h_{wcs}. The c value is assumed to be 24 inches, which is a fifth of the wall's length, 120 inches; the assumption is later verified. δ_u is the design displacement, which is defined as ASCE 7 Eqn. 12.8-15:

$$\delta_u = \frac{C_d \delta_{xe}}{I_e} = \frac{(5)(3.52\,\text{inches})}{1} = 17.6\,\text{inches} \qquad \text{(ASCE 7 Eqn. 12.8-15)}$$

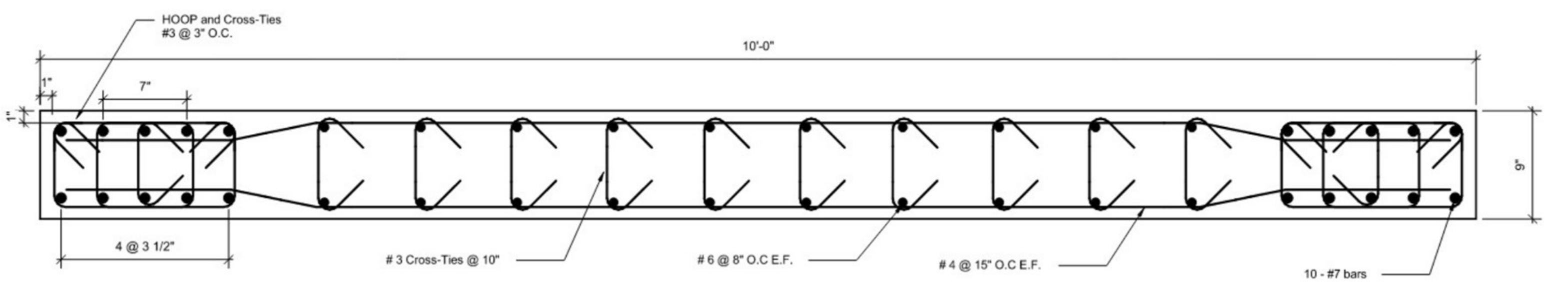
$$\frac{1.5\delta_u}{h_{wcs}} = \frac{1.5(17.6\,\text{inches})}{(1023.62\,\text{inches})} = 0.0258$$

$$\frac{l_w}{600c} = \frac{(120\,\text{inches})}{600(24\,\text{inches})} = 0.008$$

$$0.0258 > 0.008$$

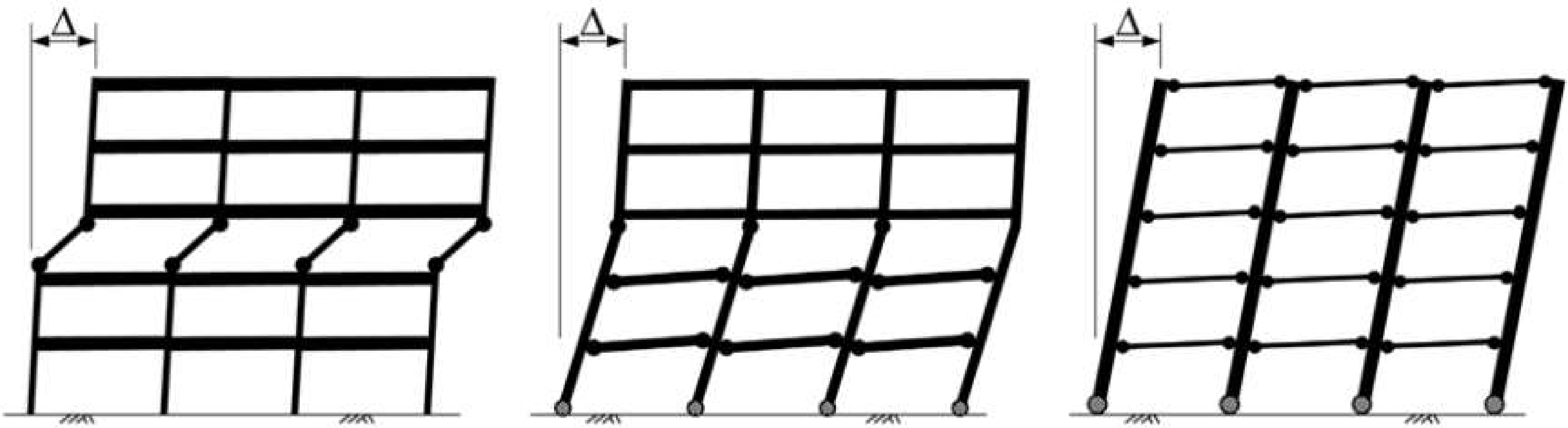


3000
2500
2000
1500
1000
500
0
−500
−1000
Axial Load (kip)
0
1000
2000
3000
4000
5000
6000
7000
Moment (kip-ft)
Wall Interaction Diagram
Demands from ETABS



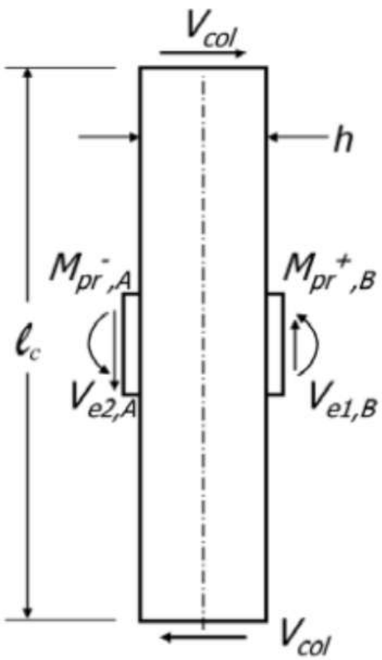
HOOP and Cross-Ties
#3 @ 3" O.C.
10'-0"
1"
7"
1"
9"
4 @ 3 1/2"
3 Cross-Ties @ 10"
6 @ 8" O.C E.F.
4 @ 15" O.C E.F.
10 - #7 bars

Moment Frame Design



(a) Story mechanism (b) Intermediate mechanism (c) Beam mechanism

Avoid Beam and Column Shear Failure w/ Capacity-Based Approach:



$$V_{col} = \left[(M_{pr}^{+}{}_{,B} + M_{pr}^{-}{}_{,A}) + (V_{e2,A} + V_{e1,B})\frac{h}{2}\right] / \ell_c$$

Figure 5-4 – Free body diagram of column used to calculate column shear V_{col}.

Figure 3-2 - Shear failure can lead to a story mechanism and axial collapse.

Avoid Beam and Column Shear Failure w/ Capacity-Based Approach:

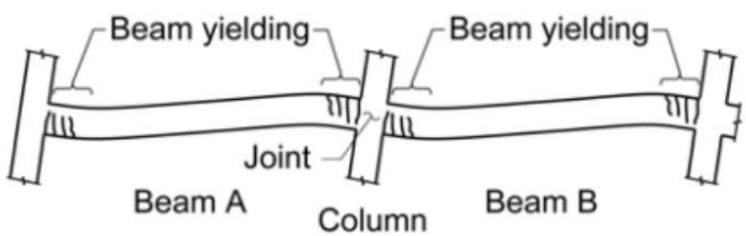
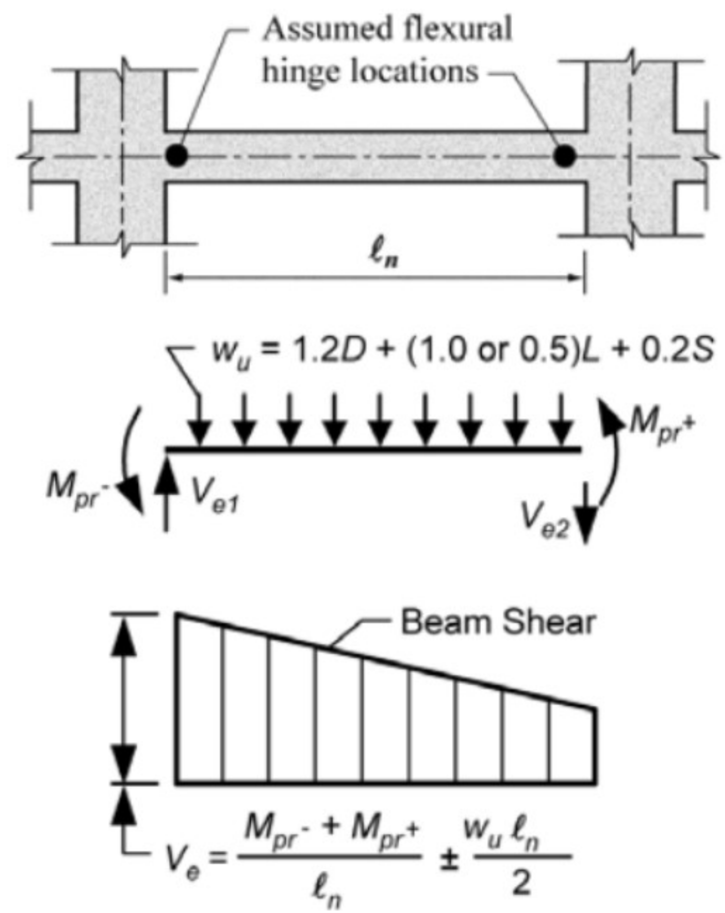


Figure 5-3 – The frame yielding mechanism determines the forces acting on the column and beam-column joint.



Beam-Column Joint Shear

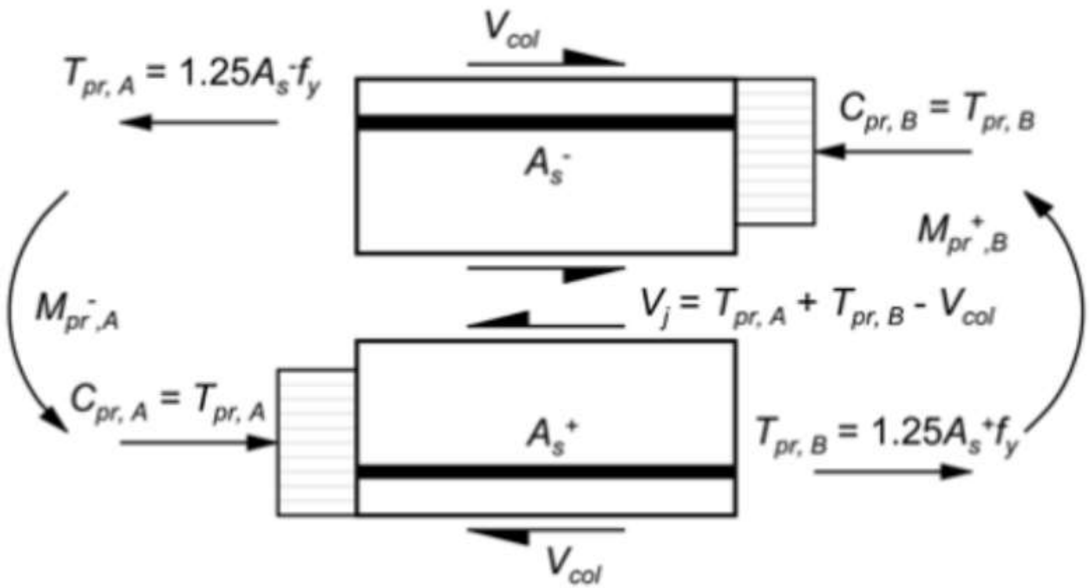


Figure 5-5 - Joint shear free body diagram.

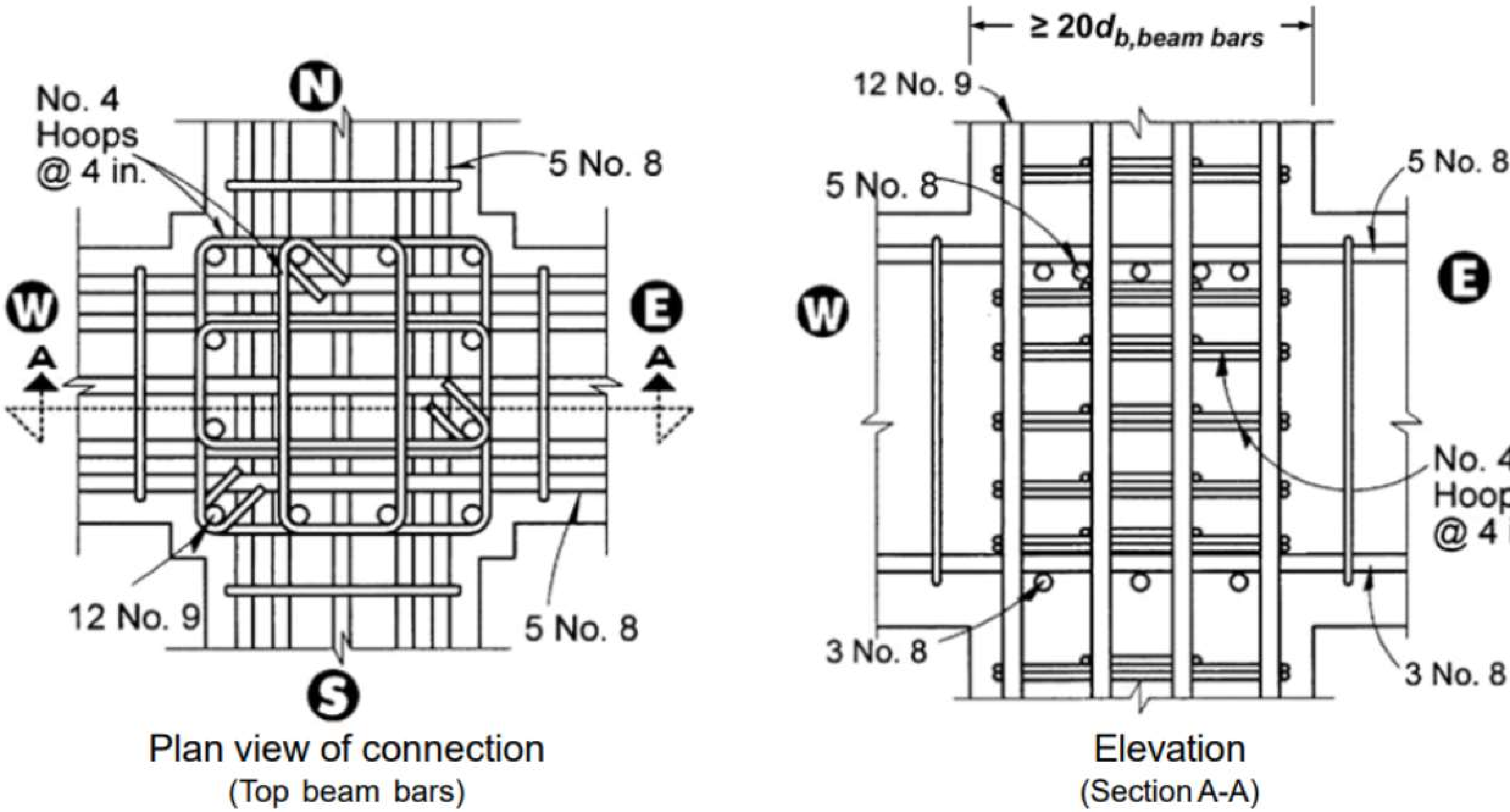
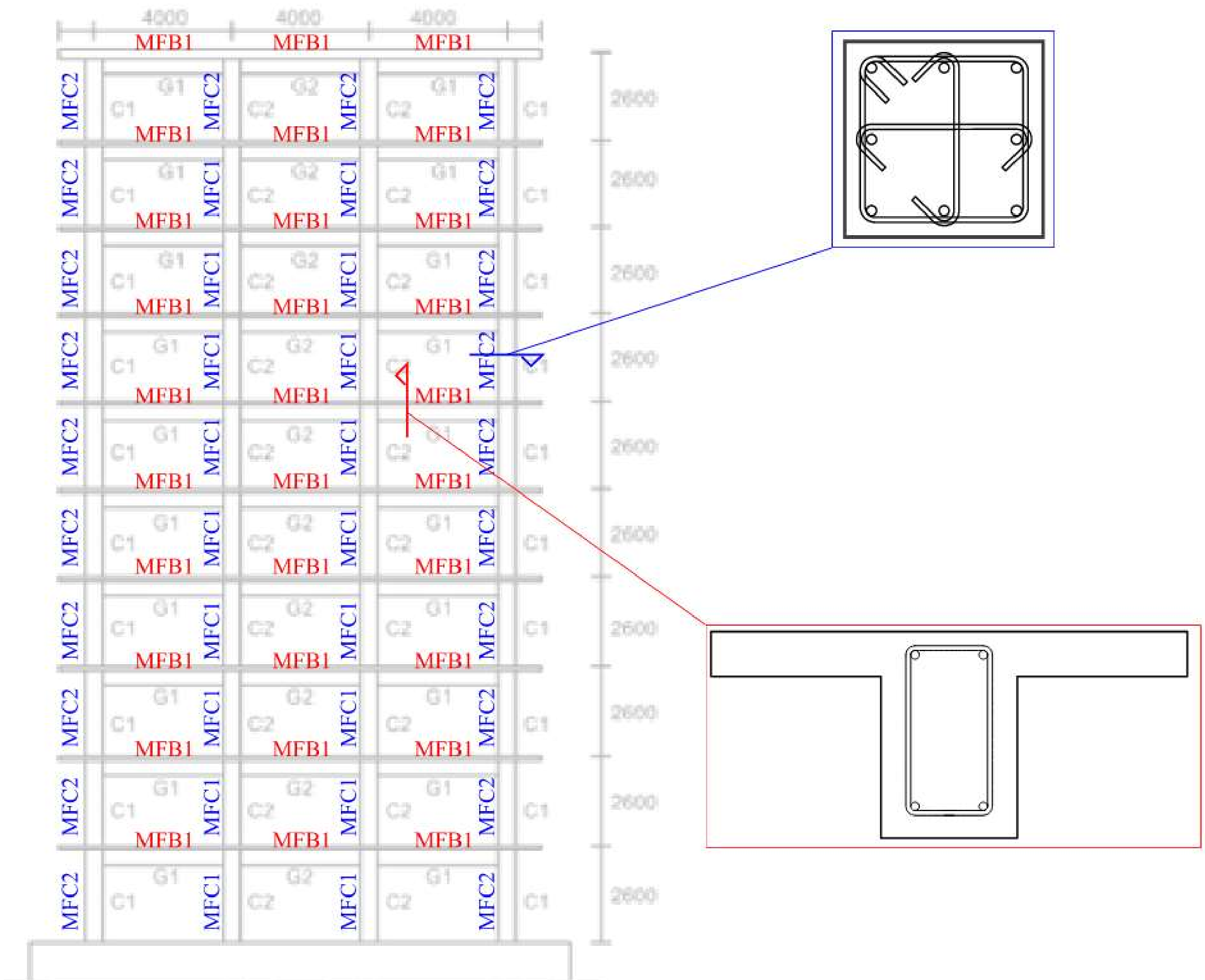
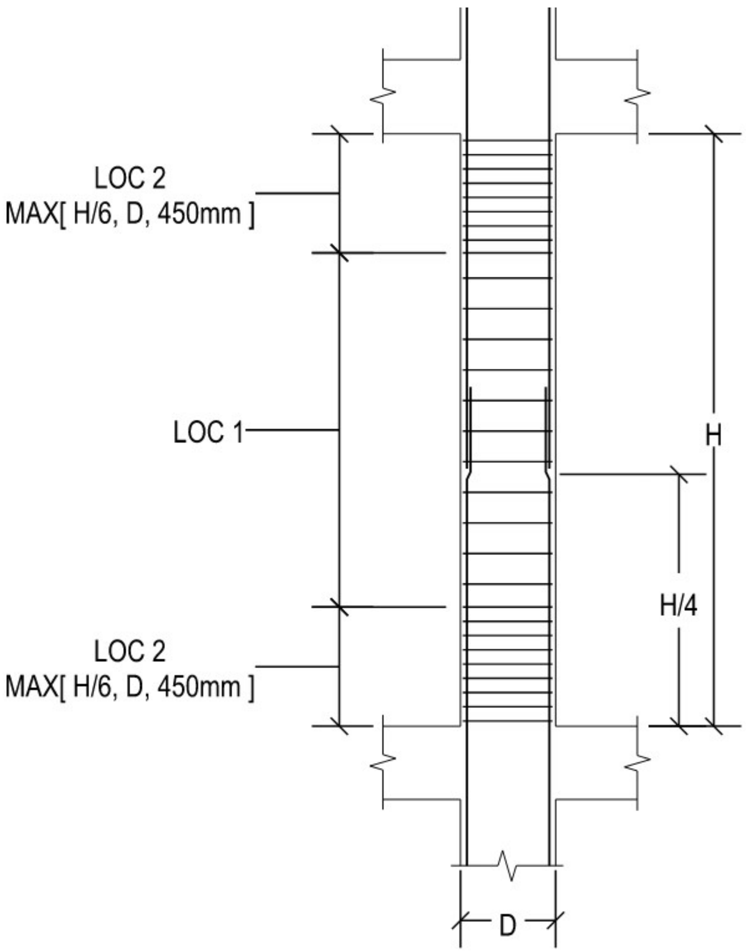


Figure 5-8 – Example interior joint detailing.



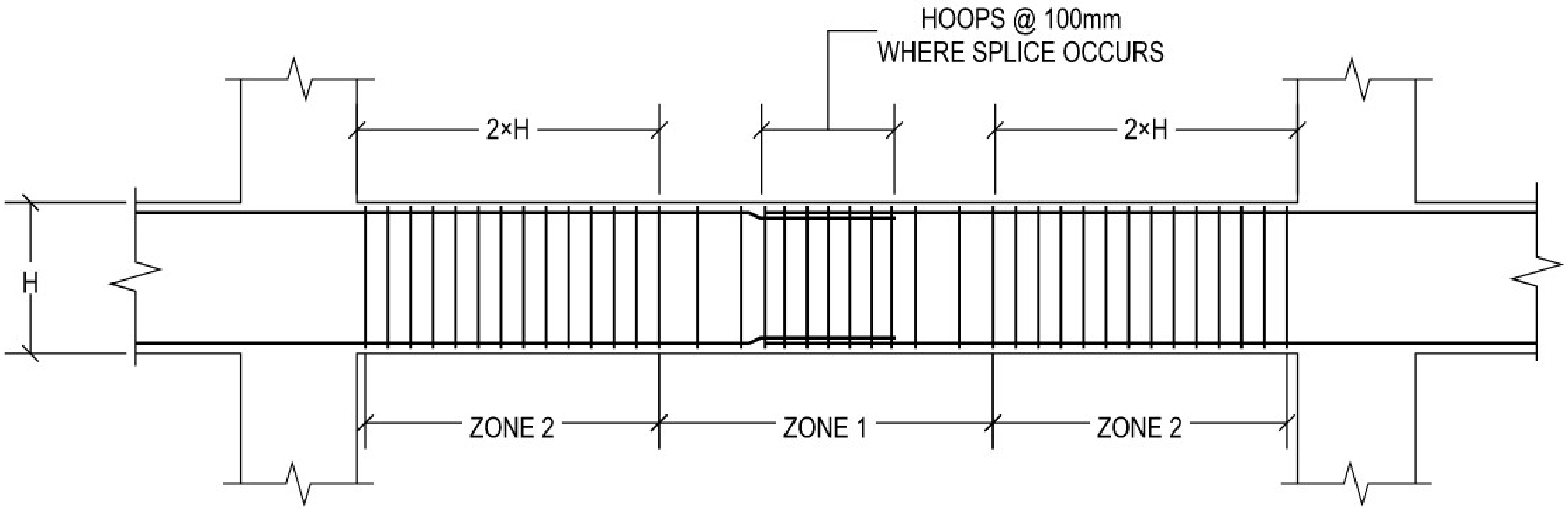
4000
4000
4000
MFB1
MFC2
MFC1
G1
G2
C1
C2
2600

CONCRETE MOMENT FRAME COLUMN SCHEDULE										
MARK	SIZE (SHORTxLONG)	f'c (PSI)	LONGIT fy (KSI)	TRANSV fy (KSI)	VERT REINF	TIE CONFIG	VERT CONFIG	LOC 1 TIES	LOC 2 TIES	REMARKS
MFC1	500mm x 500mm	4,786	60	60	(8) #8	3x3	3x3	#4 @ 115mm	#4 @ 90mm	
MFC2	500mm x 500mm	4,786	60	60	(8) #7	3x3	3x3	#4 @ 115mm	#4 @ 90mm	



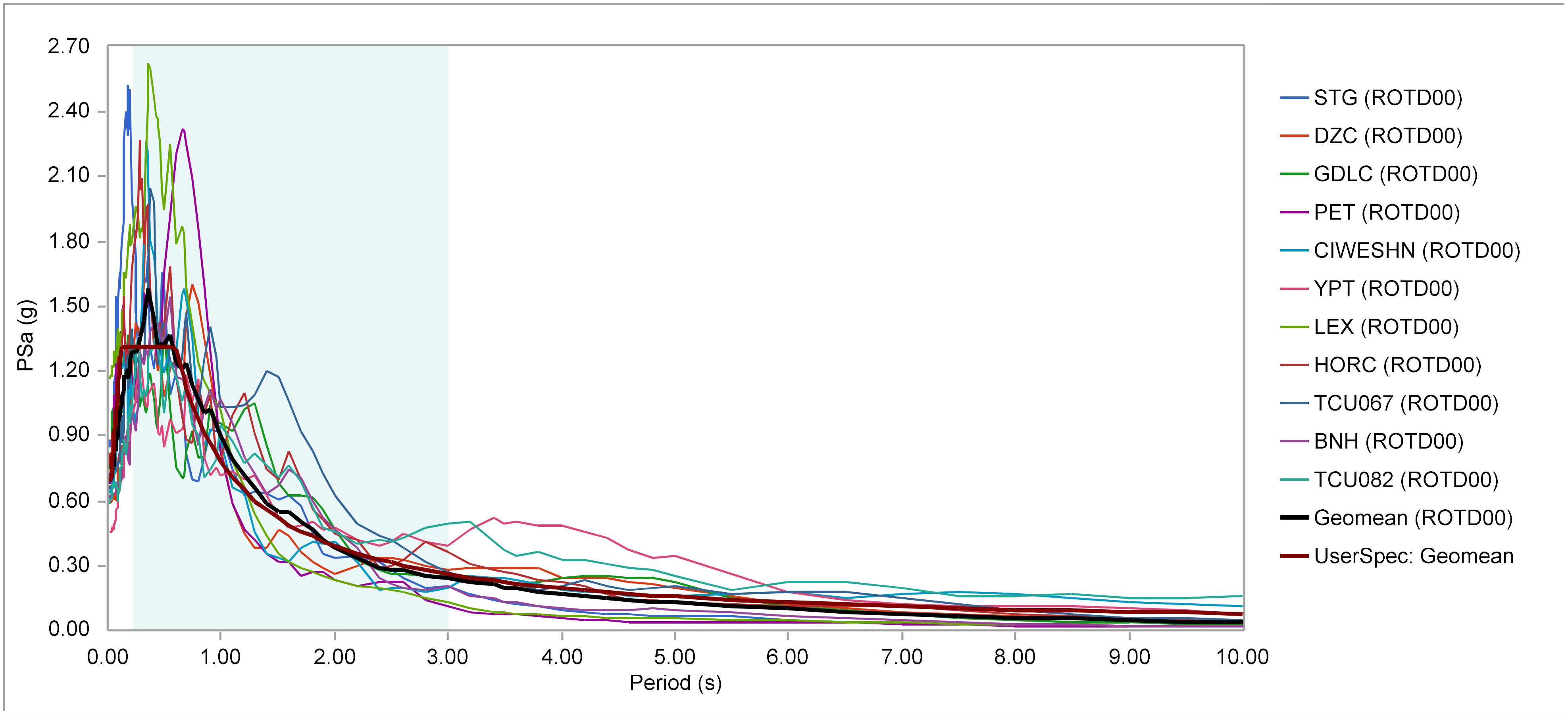
CONCRETE MOMENT FRAME BEAM SCHEDULE

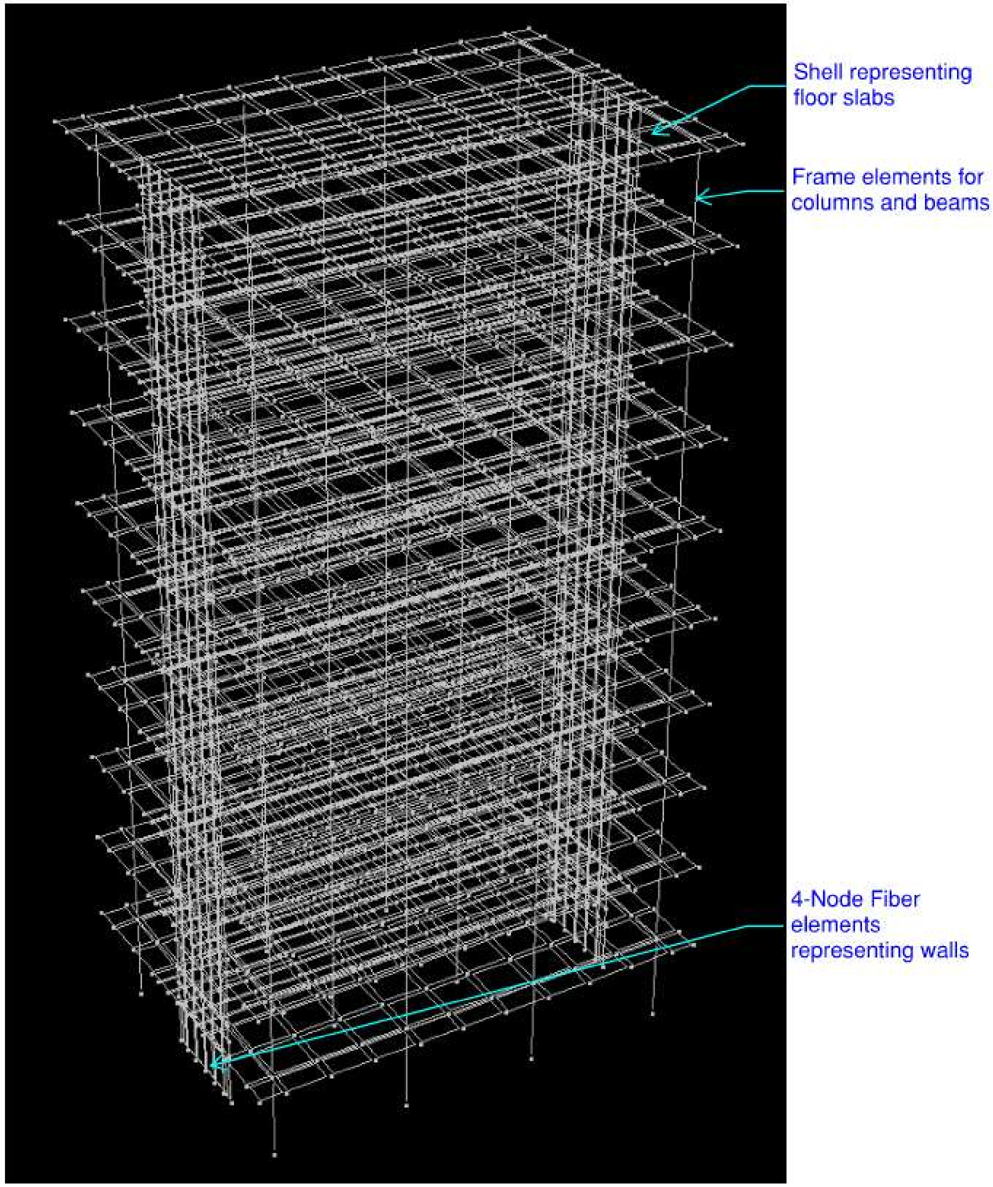
MARK	SIZE (WIDTHxDEPTH)	f'c (PSI)	LONGIT fy (KSI)	TRANSV fy	BOTTOM BARS LEFT	BOTTOM BARS CONTINUOUS	BOTTOM BARS RIGHT	TOP BARS LEFT	TOP BARS CONTINUOUS	TOP BARS RIGHT	HOOPS OR STIRRUPS ZONE 2	STIRRUPS ZONE 1	REMARKS
MFB1	350mm x 550mm	4,786	60	60		(2) #7			(2) #7		#4 @ 115mm [H2]	#4 @ 230mm [S2]	



Ground Motion Scaling

- Suite was selected from a previous project in CA Bay Area and scaled to the MCE_R hazard level
- Japan Ground Motions are only applied in one direction




Shell representing floor slabs
Frame elements for columns and beams
4-Node Fiber elements representing walls

Cracked Stiffness Properties

Concrete Component Type	**Flexural Rigidity (In-Plane)**	**Flexural Rigidity (Out-of-Plane)**	**Shear Rigidity (Each Way)**	**Axial Rigidity**
Beams	$0.30\ E_c I_g$	$0.30\ E_c I_g$	$0.40\ E_c A_g$	$1.00\ E_c A_g$
Columns	Varies based on the axial load[1]	Varies based on the axial load[1]	$0.40\ E_c A_g$	$1.00\ E_c A_g$
Wall Piers	Based on vertical fiber elements[2]	$0.25\ E_c I_g$	$0.40\ E_c A_g$	Based on vertical fiber elements[2]
Slabs	Rigid[3]	$0.01\ E_c I_g$[3]	Rigid	Rigid

1. Linear interpolation between $0.30\ E_c I_g$ and $0.70\ E_c I_g$ for compression caused by design gravity loads between $0.1 A_g f'_c$ and $0.5 A_g f'_c$.
2. See Section 2.4.2.3 of this document.
3. See LATBSDC (2023) Table 3, ASCE 7 Section 12.3.1.2, and ASCE 41 Section 7.2.11.1.

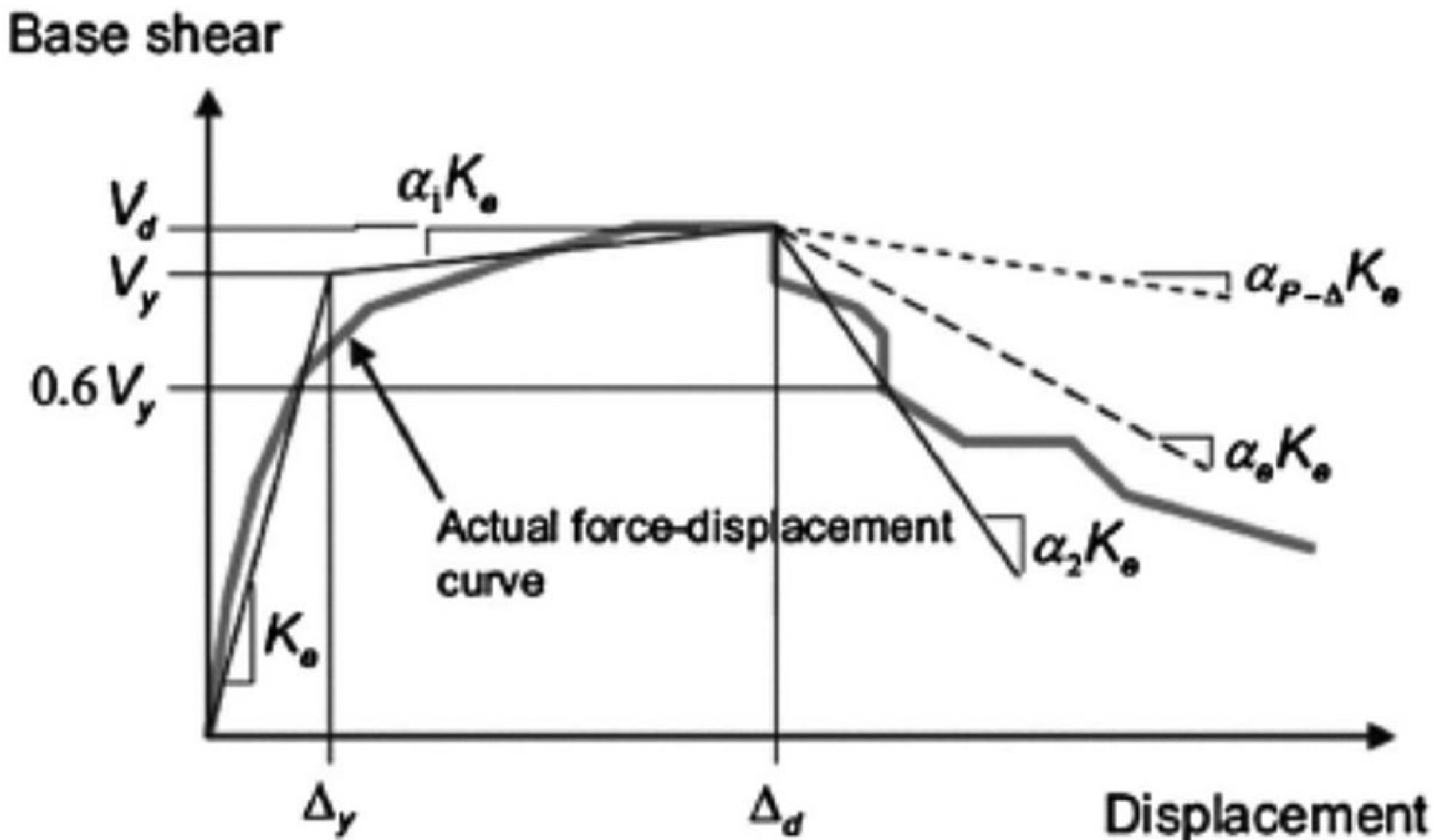
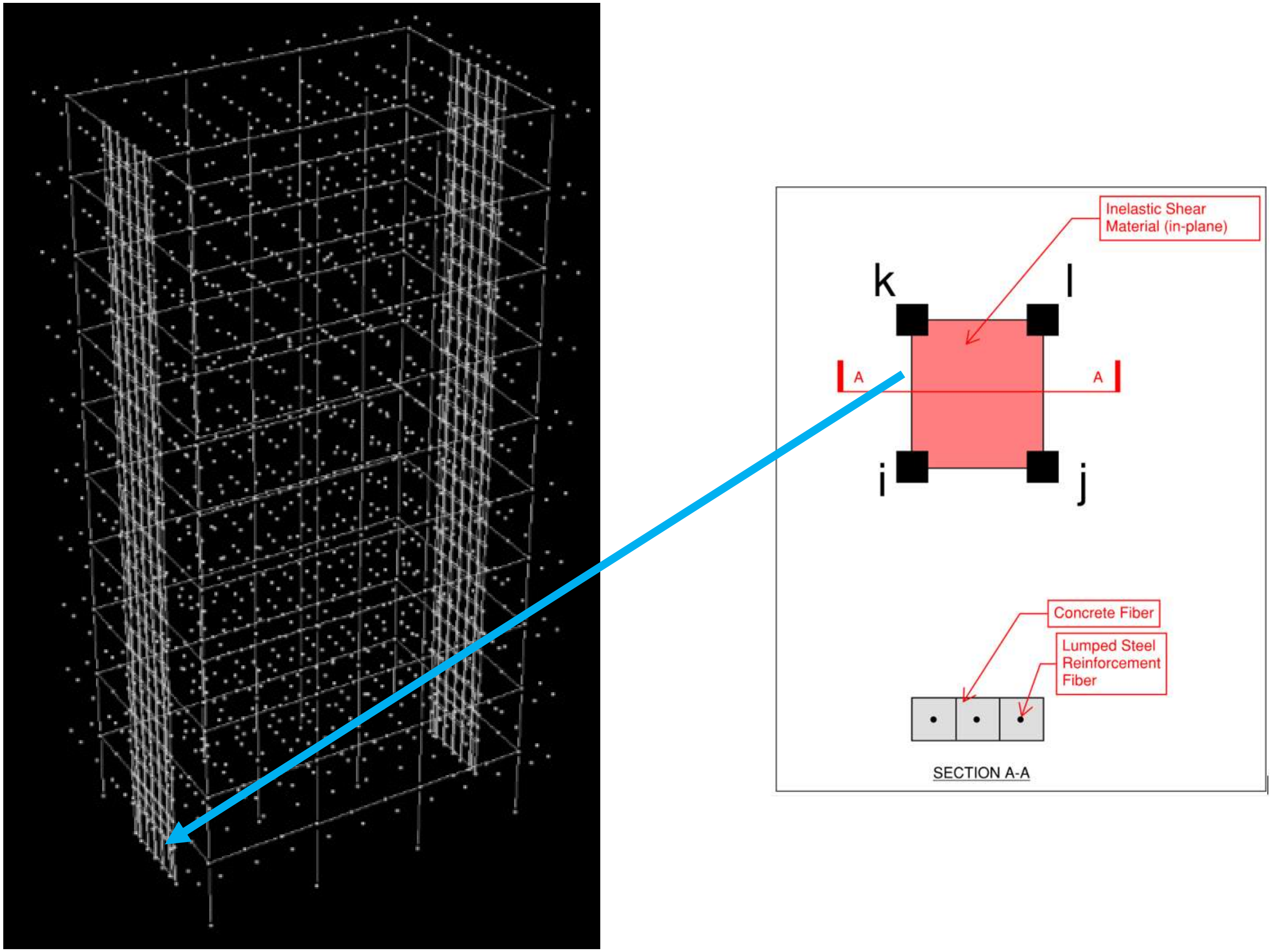
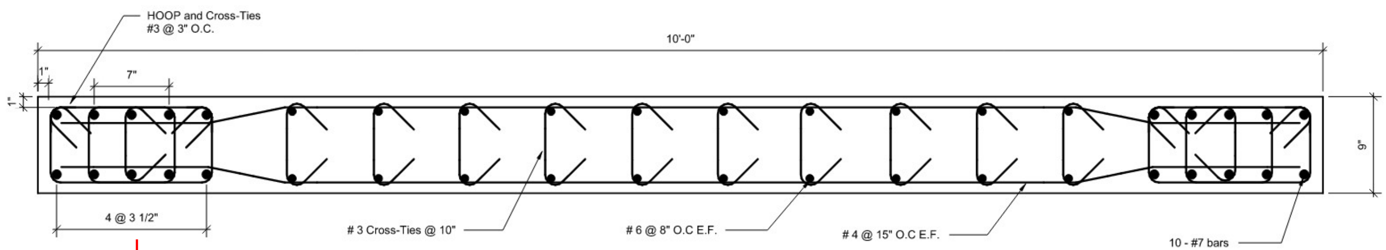
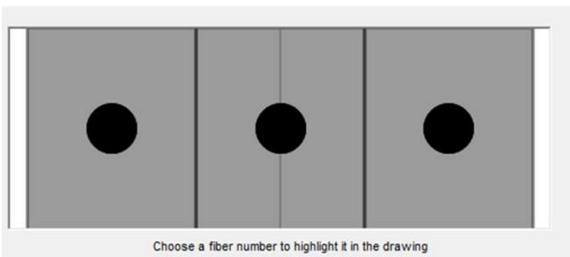


Figure 7-3. Idealized force–displacement curves.


Inelastic Shear Material (in-plane)
k
l
A
A
i
j
Concrete Fiber
Lumped Steel Reinforcement Fiber
SECTION A-A

MODELING Approach (Walls)





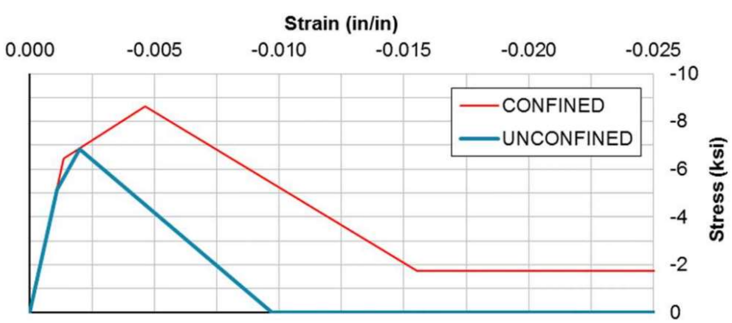


Figure 4: Typical Compressive Stress-Strain Envelopes for Unconfined and Confined Concrete

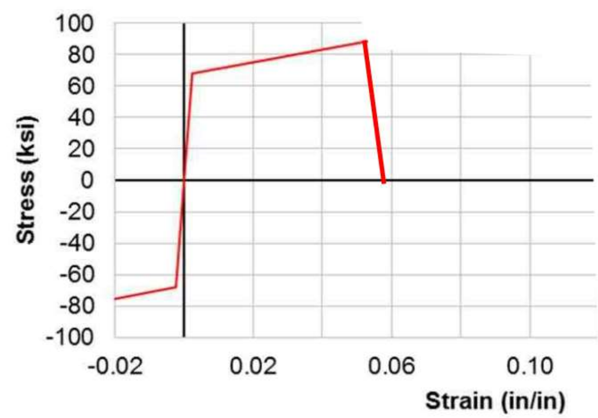
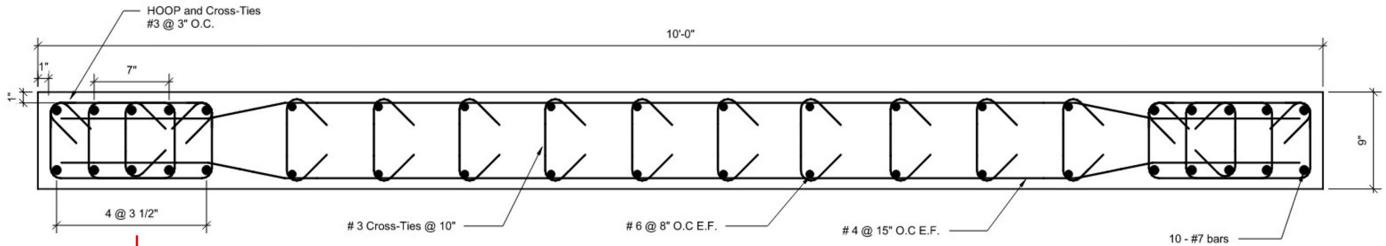
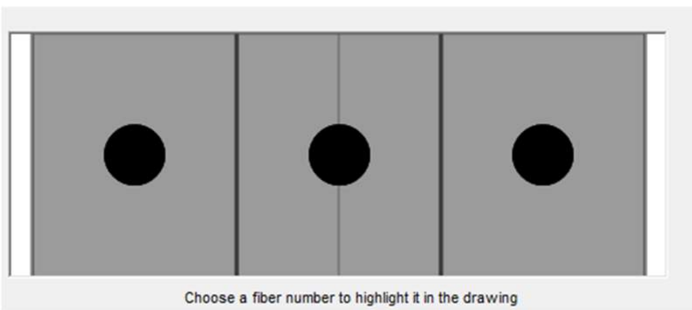


Figure 5: Typical Stress-Strain Reinforcing Steel

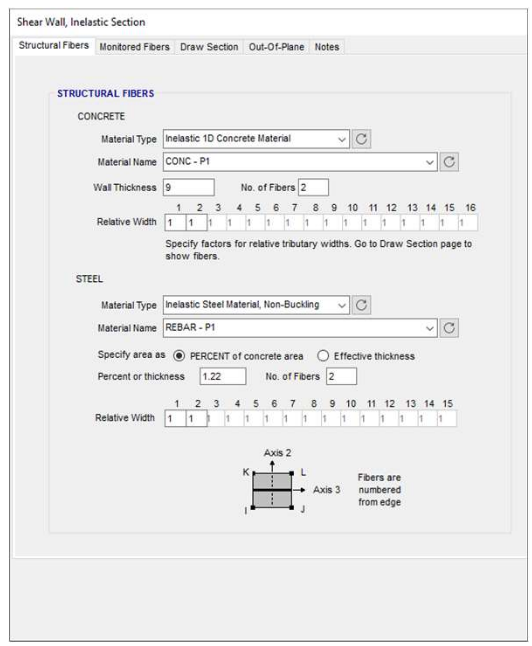
- Materials are regularized per Lowes et al. (2016)
- Steel properties calibrated to match ASCE 41-23 moment-rotation
- Used elastic material for shear with strength section

MODELING Approach (Walls)

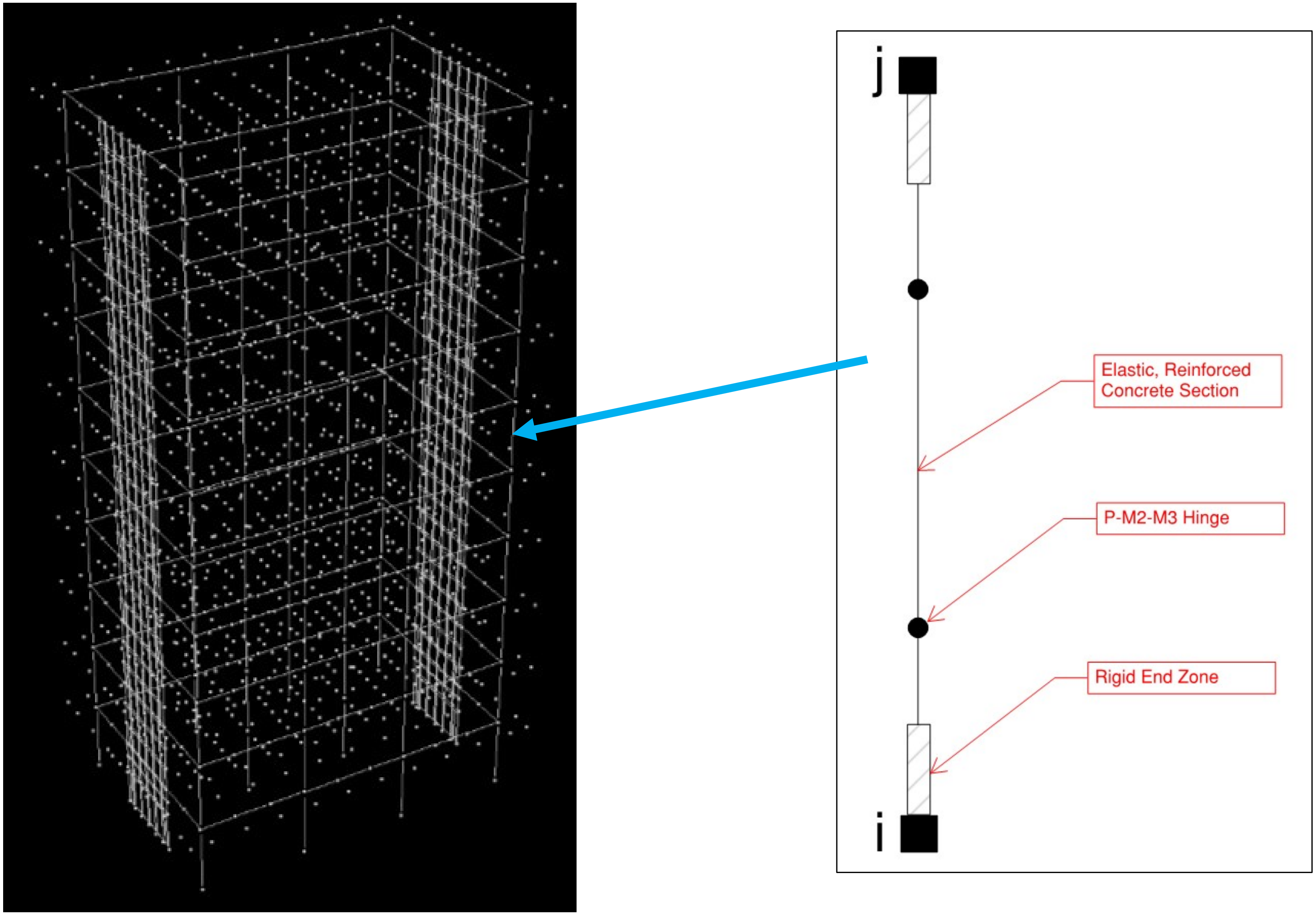




No.	Type	Name	Coordinate	Area	T-Draw
1	Inelastic Steel Mat...	REBARBZ - P1 ...	-5.5	2	
2	Inelastic Steel Mat...	REBARBZ - P1 ...	0	2	
3	Inelastic Steel Mat...	REBARBZ - P1 ...	5.5	2	
4	Inelastic 1D Concr...	CONCBZ - P1	-5.5	36.44	6.625
5	Inelastic 1D Concr...	CONCBZ - P1	0	36.44	6.625
6	Inelastic 1D Concr...	CONCBZ - P1	5.5	36.44	6.625



- Materials are regularized per Lowes et al. (2016)
- Steel properties calibrated to match ASCE 41-23 moment-rotation
- Used elastic material for shear with strength section

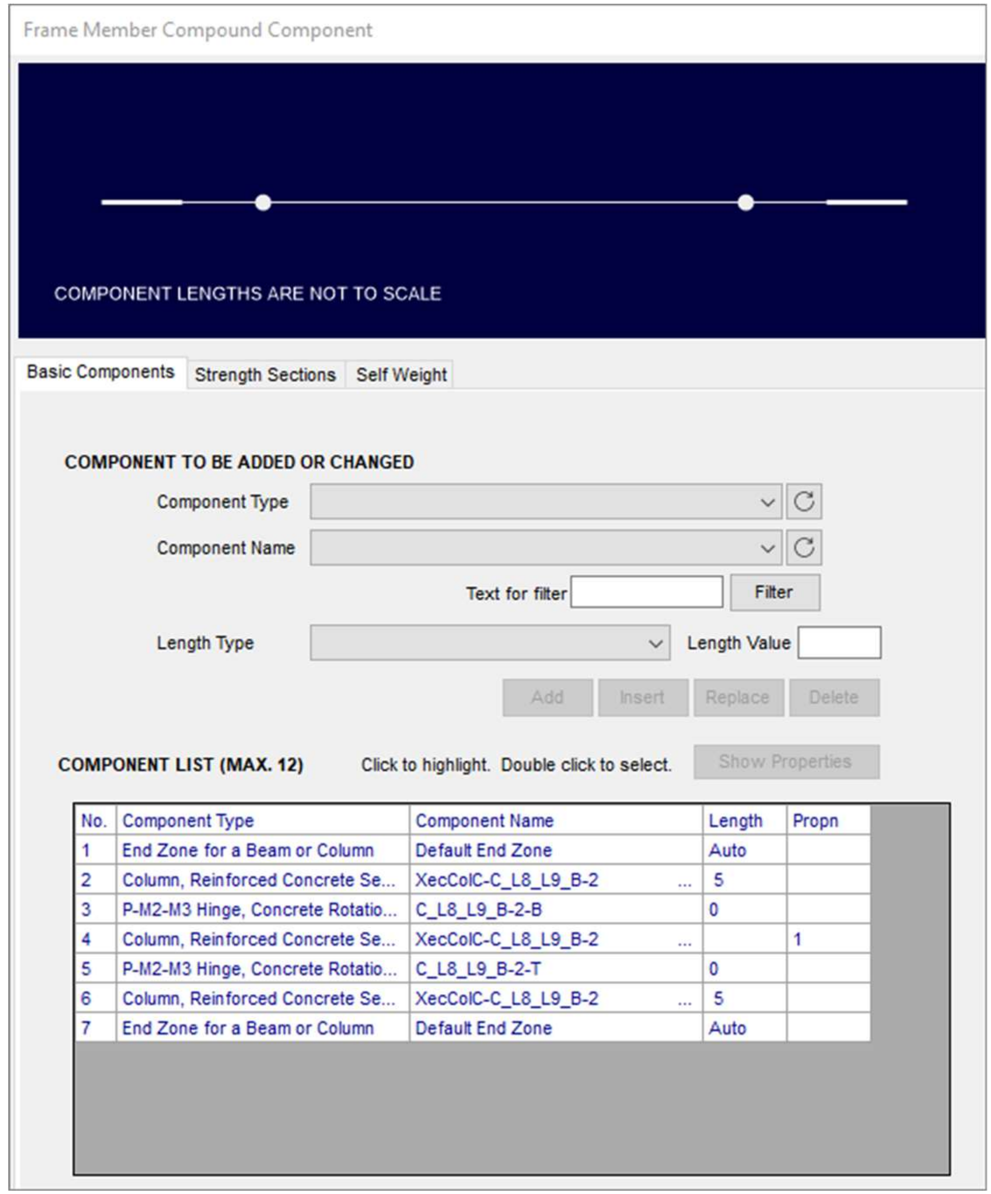

j
Elastic, Reinforced Concrete Section
P-M2-M3 Hinge
Rigid End Zone
i

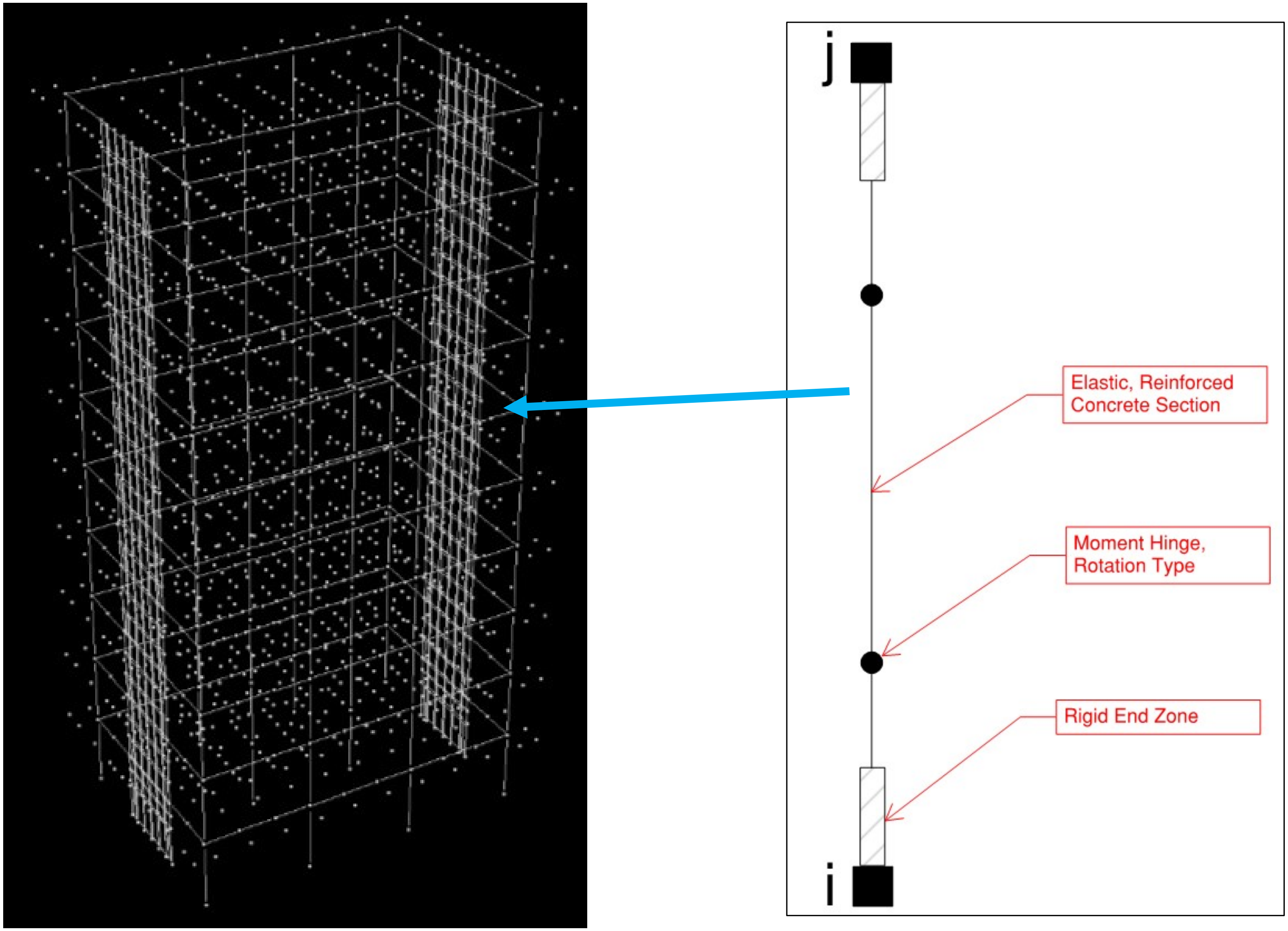
MODELING Approach (Columns)

- Used cracked linear sections with lumped hinges
- Used ACI 374.3 proposal for modeling parameters
- Used ACI 369.1-22 for acceptance criteria

Table 4.1.2—Nonlinear modeling parameters for rectangular and circular columns in special moment frames based on database-

Column Shape	Plastic rotation angle, radians		Coefficient of variation Measured/Calculated plastic rotation		Residual strength ratio
	a	b	a	b	c
Rectangular	$0.05 - 0.05\frac{P}{A_g f_c'} + 0.60\rho_t - 0.03\frac{s}{d} \geq 0$	$0.05 - 0.05\frac{P}{A_g f_c'} + 0.55\rho_{lc} - 0.40\rho_t \geq a$	0.30	0.28	0.2
Circular	$0.07 - 0.09\frac{P}{A_g f_c'} + 0.50\rho_{lc} - 0.05\frac{V_p}{V_o} \geq 0$	$0.09 - 0.09\frac{P}{A_g f_c'} + 0.40\rho_{lc} - 0.05\frac{V_p}{V_o} \geq a$	0.36	0.29	0.2




j
Elastic, Reinforced Concrete Section
Moment Hinge, Rotation Type
Rigid End Zone
i

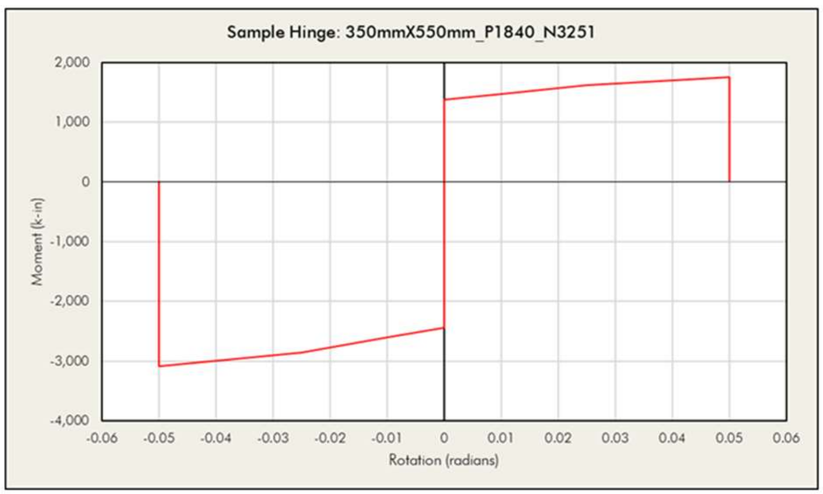
MODELING Approach (Beams)

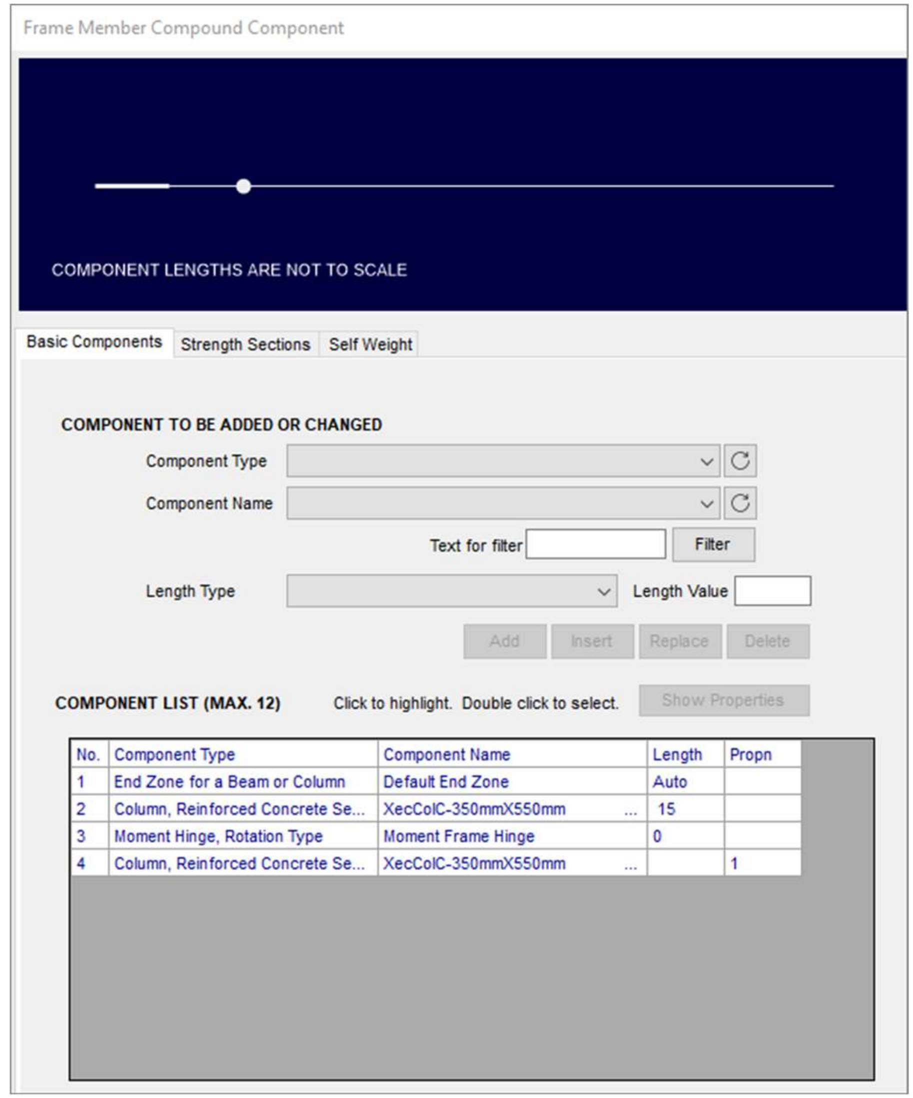
- Used ACI 374.3 for modeling parameters (provided by MKA)
- Used cracked linear sections with lumped hinges

- Beams
  - Elastic frame section, strong-axis bending modifier = 0.3
  - Moment-rotation hinges at 0.2 and 0.8 relative distances along element length:

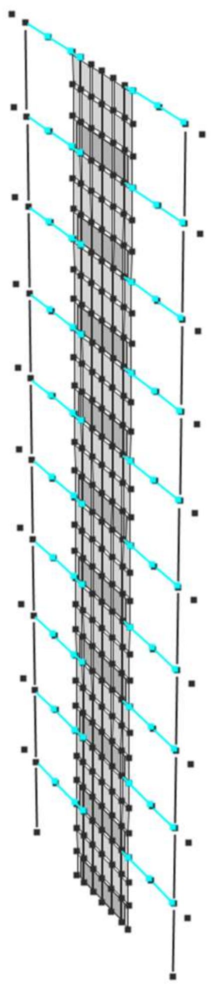
Hinge Definitions

Name	Point	Force (kip-in)	Displacement
350mmX550mm_P1840_N3251	E-	-3,088	-0.05
350mmX550mm_P1840_N3251	D-	-2,861	-0.025
350mmX550mm_P1840_N3251	C-	-2,601	-0.01
350mmX550mm_P1840_N3251	B-	-2,438	0
350mmX550mm_P1840_N3251	A	0	0
350mmX550mm_P1840_N3251	B	1,380	0
350mmX550mm_P1840_N3251	C	1,472	0.01
350mmX550mm_P1840_N3251	D	1,619	0.025
350mmX550mm_P1840_N3251	E	1,748	0.05





MODELING Approach (Outrigger Beams)



Coupling between the core wall and the gravity columns, or between two columns, shall be explicitly modeled in the nonlinear analysis using an equivalent slab-beam (outrigger beam) if any of the following two conditions apply:

1. Column-to-core distance is less than 20 feet.
2. Column-to-column distance is less than 10 feet.

Table 10-15. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Two-Way Slabs and Slab–Column Connections

		Modeling Parameters[a]			Acceptance Criteria[a]		
		Plastic Rotation Angle (radians)		Residual Strength Ratio	Plastic Rotation Angle (radians)		
					Performance Level		
						Secondary	
Conditions		a	b	c	IO	LS	CP
Condition i. Reinforced concrete slab–column connections[b]							
$\frac{V_g}{V_o}$[c]	Continuity reinforcement[d]						
0	Yes	0.035	0.05	0.2	0.01	0.035	0.05
0.2	Yes	0.03	0.04	0.2	0.01	0.03	0.04
0.4	Yes	0.02	0.03	0.2	0	0.02	0.03
≥0.6	Yes	0	0.02	0	0	0	0.02
0	No	0.025	0.025	0	0.01	0.02	0.025
0.2	No	0.02	0.02	0	0.01	0.015	0.02
0.4	No	0.01	0.01	0	0	0.008	0.01
0.6	No	0	0	0	0	0	0
>0.6	No	0	0	0	—[e]	—[e]	—[e]
Condition ii. Post-tensioned slab–column connections[b]							
$\frac{V_g}{V_o}$[c]	Continuity reinforcement[d]						
0	Yes	0.035	0.05	0.4	0.01	0.035	0.05
0.6	Yes	0.005	0.03	0.2	0	0.025	0.03
>0.6	Yes	0	0.02	0.2	0	0.015	0.02
0	No	0.025	0.025	0	0.01	0.02	0.025
0.6	No	0	0	0	0	0	0
>0.6	No	0	0	0	—[e]	—[e]	—[e]

C10.4.4.2.1 Linear Static and Dynamic Procedures.

1. Effective beam width model: Allen and Darvall (1977) provide tables of effective width coefficients for different combinations of plate aspect ratios (l_1/l_2) and column width-to-slab span ratios (c_1/l_1 or c_2/l_1). Research indicates that the effective width of exterior bays should be less than the effective width of interior bays because of the higher flexibility of one-sided slab–column connections at the frame end. Hwang and Moehle (2000) provide equations for effective width that show the relationship between exterior and interior bays is about 1/2.

Eq. (C10-3) can be used instead of tables from Allen and Darvall (1977).

$$\text{For interior bays: } b_{\text{eff}} = 2c_1 + l_1/3 \qquad \text{(C10-2a)}$$

$$\text{For exterior bays: } b_{\text{eff}} = c_1 + l_1/6 \qquad \text{(C10-2b)}$$

where b_{eff} is the effective slab width.

To account for cracking from temperature, shrinkage, or nonlinear response, slab stiffness determined using gross section properties based on the above guidance should be reduced by an effective stiffness factor β_{eff}. There is general agreement that $\beta_{\text{eff}} = 1/3$ is appropriate for non-prestressed slabs (Vanderbilt and Corley 1983). Somewhat higher, yet conservative, values can be obtained using Eq. (C10-4) from Hwang and Moehle (2000):

Acceptance Criteria

Table 1.3-2. Target Reliability (Conditional Probability of Failure) for Structural Stability Caused by Earthquake.

Risk Category	Conditional Probability of Failure Caused by the MCE_R Shaking Hazard (%)
I and II	10
III	5
IV	2.5

Risk-Targeted Maximum Considered Earthquake (MCE_R) **Ground Motion Response Acceleration:** The most severe earthquake effects considered by this standard determined for the orientation that results in the largest maximum response to horizontal ground motions and with adjustment for targeted risk. In this standard, general procedures for determining the MCE_R ground motion values are provided in Section 11.4.4; site-specific procedures are provided in Sections 21.1 and 21.2.

The adequacy of the design and the attainment of acceptable building performance shall be demonstrated using two earthquake ground motion intensities:

A. Serviceable Behavior When Subjected to Frequent Earthquake Ground Motions.

B. Low Probability of Collapse and the Likelihood of Building Repairability when subjected to Extremely Rare Earthquake Ground Motions. The extremely rare earthquake motions shall be taken as the Risk Targeted Maximum Considered

3.5.2. Service Level Design Earthquake

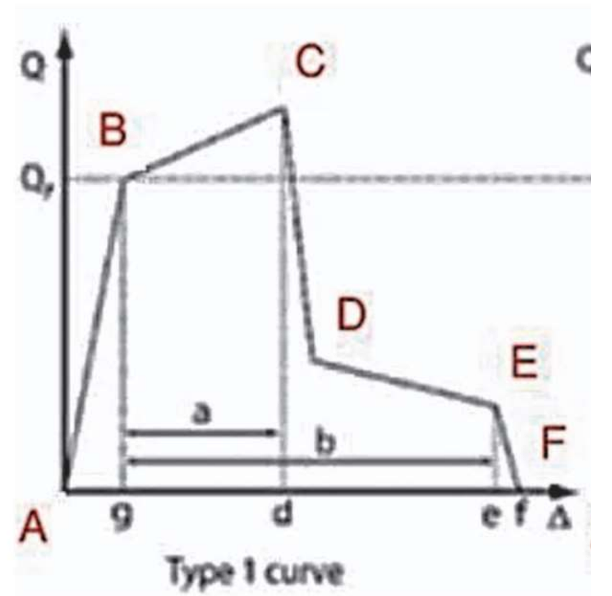
The service level design earthquake shall be taken as an event having a 50% probability of being exceeded in 30 years (43-year return period). The Service Level Design Earthquake is defined in the form of a site-specific, linear, uniform hazard acceleration response spectrum with the damping level determined using Equation (1).

=> Essentially Elastic

16.4.2 Element-Level Acceptance Criteria All element actions shall be classified either as force-controlled or deformation-controlled, in accordance with ACI 318 for reinforced concrete elements or ASCE 41 for elements of other materials.

APPENDIX A—DESIGN VERIFICATION USING NONLINEAR RESPONSE HISTORY ANALYSIS

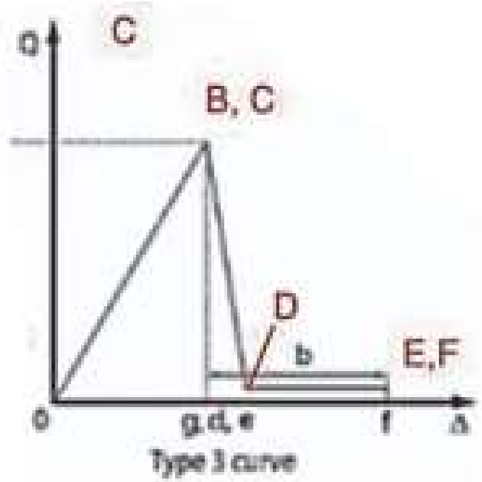
16.4.2.2 Deformation-Controlled Actions The valid range of modeling for deformation-controlled element actions shall be as established in the applicable material design standard. Where the material design standard does not specify the valid range of modeling, this parameter shall be established as the maximum value of the parameter at which the element model is capable of replicating the hysteretic behavior and load-carrying capability observed in laboratory testing of similar elements. Where



16.4.2.1 Force-Controlled Actions Force-controlled actions shall satisfy Equation (16.4-1) and (16.4-2):

$$(1.2 + 0.12S_{MS})D + 0.5L + 1.3I_e(Q_u - Q_{ns}) \leq \phi BR_n \quad (16.4\text{-}1)$$

$$(0.9 - 0.12S_{MS})D + 1.3I_e(Q_u - Q_{ns}) \leq \phi BR_n \quad (16.4\text{-}2)$$



- **Critical action** – A force-controlled action, the failure of which is likely to lead to partial or total structural collapse.
- **Ordinary action** – A force-controlled action, the failure of which is unlikely to lead to structural collapse or it might lead to local collapse comprising not more than one bay in a single story.

Table 4 Force-controlled actions and their categories

	Component	Seismic Action	Category	
			Critical	Ordinary
Reinforced Concrete	Below Grade Perimeter Retaining Walls	Moment		X
		Shear		X
	Below Grade Non-Perimeter / Non-Core Walls	Shear	X	
	Core Walls Above and Below Grade and All Above Grade Walls	Shear	X	
	Diaphragms with Major Shear Transfer	Axial	X	
		Flexure	X**	
		Shear	X	
	Coupling beams without special diagonal reinforcing including steel-fiber reinforced coupling beams*	Shear	X	
	Typical (non-transfer slab) Diaphragm Forces (excludes collectors and shear transfer to vertical element)	Axial		X
		Flexure		X
		Shear		X
	All Drag (Collector) Members	Compression	X	
		Tension	X	
	Vertical Element-to-Diaphragm Connection	Bearing	X	
		Shear Transfer (Shear Friction)	X	
	Gravity Columns and Special Moment Frames (Columns, Beam-Column joints) excluding, Intentional	Axial	X	
		Shear	X	

Table 6-1 Seismic resistance factors, ϕ_s.

Action Type	ϕ_s
Critical force-controlled element	ϕ as specified in the applicable material standard (ACI 318, AISC 360, AISC 341, AISC 358)
Ordinary force-controlled element	0.9

16.4.1.1 Unacceptable Response Unacceptable response to ground motion shall consist of any of the following:

1. Analytical solution fails to converge,
2. Predicted demands on deformation-controlled elements exceed the valid range of modeling,
3. Predicted demands on critical or ordinary force-controlled elements, as defined in Section 16.4.2, exceed the element capacity,
4. Predicted deformation demands on elements not explicitly modeled exceed the deformation limits at which the members are no longer able to carry their gravity loads,
5. Peak transient story drift ratio exceeds 150% of the permissible value of mean transient story drift, as per Section 16.4.1.2, or
6. For structures exceeding 240 ft (73m) in height, the residual story drift for any story exceeds a value of 0.015 h_{sx}.

Unacceptable response to ground motion shall not be permitted.

EXCEPTION: For Risk Category I and II structures, where spectral matching of ground motion is not used, not more than one motion shall be permitted to produce unacceptable response.


L_p

Table 7.4.1.1.1a. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures: Conforming Reinforced Concrete Structural Walls and Associated Components Controlled by Flexure.

Conditions[d]				Acceptance Criteria Performance Level
$\frac{l_w c_{DE}}{b_s^2}$	$\frac{w_v V_{MCultDE}}{A_{cv}\sqrt{f'_{cE}}}$ [c]	Overlapping hoops[a] used?	d_{nl}	IO
≤10	≤4	Yes	0.032	$\theta_{yE} + 0.1(d_{nl} - \theta_{yE})$
≤10	≥6	Yes	0.026	
≥70	≤4	Yes	0.018	
≥70	≥6	Yes	0.014	
≤10	≤4	No	0.032	
≤10	≥6	No	0.026	
≥70	≤4	No	0.012	
≥70	≥6	No	0.011	

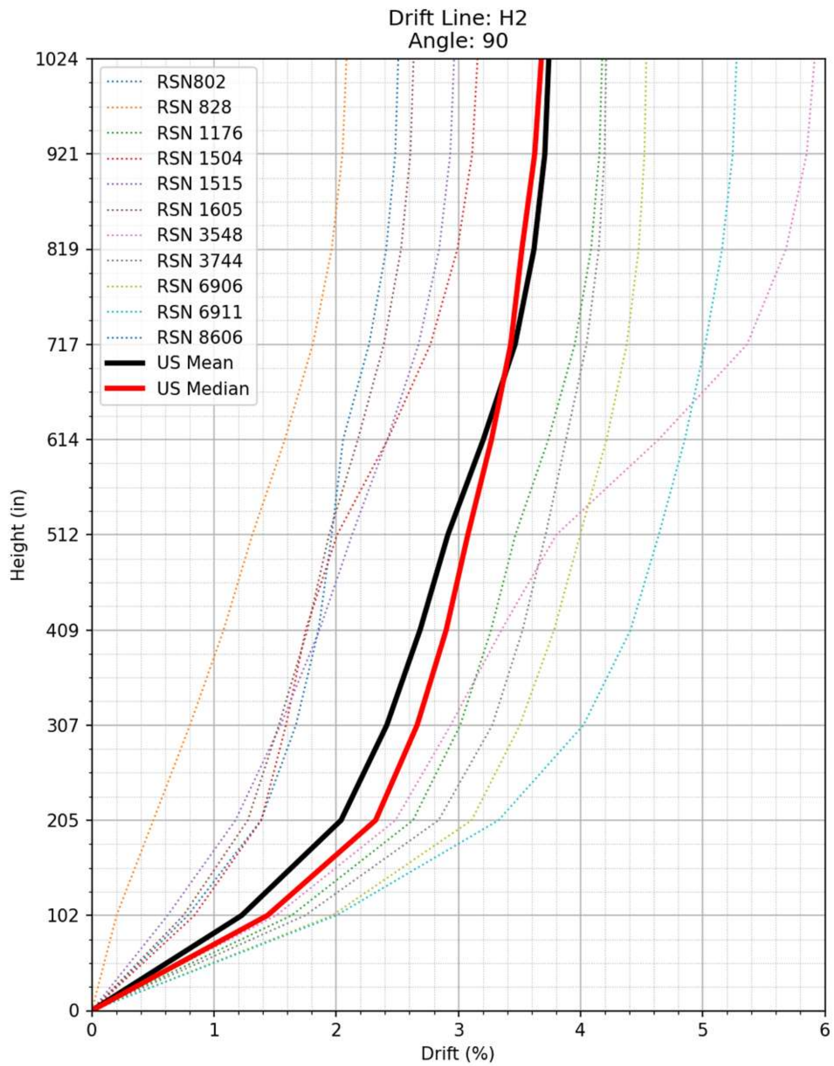
Conditions[d]						Acceptance Criteria Performance Level	
$\frac{l_w c_{GE}}{b_s^2}$	$\frac{N_{UD}}{A_g f'_{cE}}$	c_{nl}	c'_{nl}	d'_{nl} [b]	e_{nl} [b]	LS	CP
≤10	≤0.10	0.5	1.15	0.036	0.040	0.75 e_{nl}	0.85 e_{nl}
≤10	≥0.20	0.1		0.030	0.032		
≥70	≤0.10	0.0		0.018	0.020		
≥70	≥0.20	0.0		0.014	0.014		

[a]Overlapping hoop definition shall be per ACI 318-19.
[b]Parameters d'_{nl} and e_{nl} shall not be taken smaller than parameter d_{nl}.
[c]The shear amplification factor ω_v need not be applied if V_{MCultE} is obtained from nonlinear analyses procedures.
[d]Linear interpolation between the values given in the table shall be permitted; however, interpolation between the values specified for Conforming walls (Table 7.4.1.1.1a) and Nonconforming walls (Table 7.4.1.1.1b) shall not be permitted.
Note: IO = Immediate Occupancy, LS = Life Safety, CP = Collapse Prevention.

Table 6-2. Recommended deformation limits for deformation-controlled actions

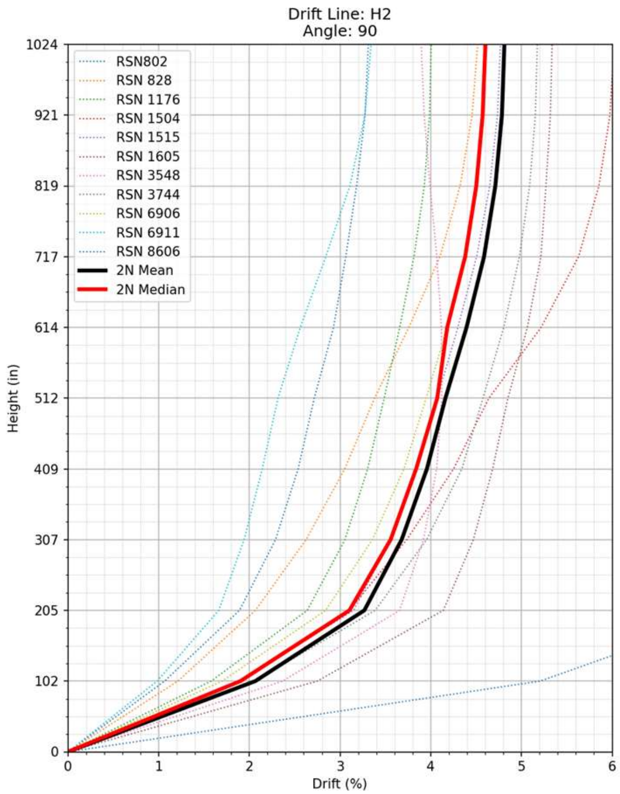
Item		Engineering Demand Parameter	Acceptance Limit
Reinforced concrete walls (outside of primary hinge zone)	No confinement	Concrete compression strain over gage length [1]	$0.001/I_e$
		Steel tension strain over gage length [1]	$2\varepsilon_y/I_e$
	Intermediate confinement per ACI 318-19 18.10.6.5	Concrete compression strain over gage length [1]	$0.003/I_e$
		Steel tension strain over gage length [1]	$0.01/I_e$
	Full confinement per ACI 318-19 18.10.6.4 except provisions of Section 18.10.6.4(i) need not be satisfied [2]	Concrete compression strain over gage length [1]	$0.005/I_e$ ($0.01/I_e$ [3])
		Steel tension strain over gage length[1]	$0.01/I_e$ ($0.05/I_e$ [3])
Reinforced concrete walls (primary hinge zone)	Full confinement of the entire cross section per ACI 318-19 18.10.6.4 [2]	Concrete compression strain over gage length [1]	$0.005/I_e$ ($0.01/I_e$ [3])
		Steel tension strain over gage length [1]	$0.01/I_e$ ($0.05/I_e$ [3])
Coupling beams	Conventionally-reinforced [4]	Total chord rotation	$0.04/I_e$
	Diagonally-reinforced [4]	Total chord rotation	$0.06/I_e$
	Fiber-reinforced [5]	Total chord rotation	$0.04/I_e$
	Steel-reinforced	Total chord rotation	$0.06/I_e$

Wall Performance: Drift

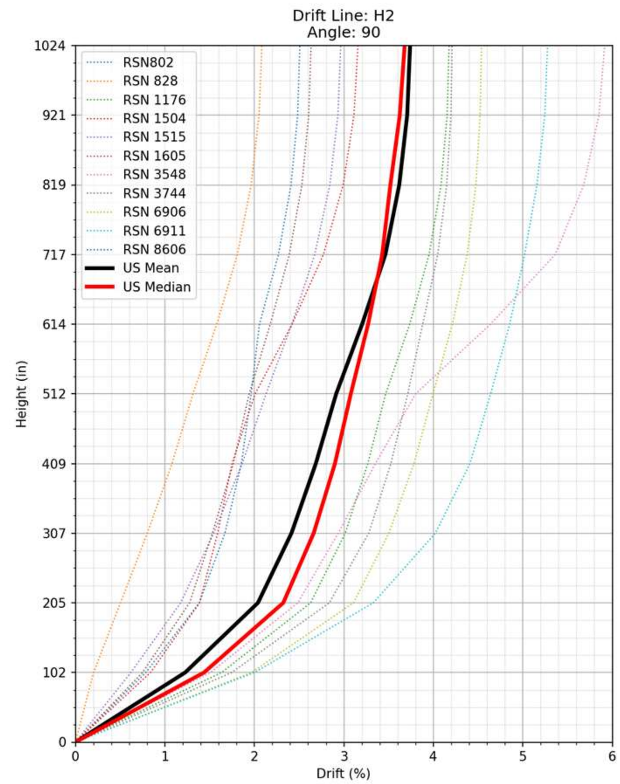


Wall Performance

- Performance can be dependent on the ground motion components

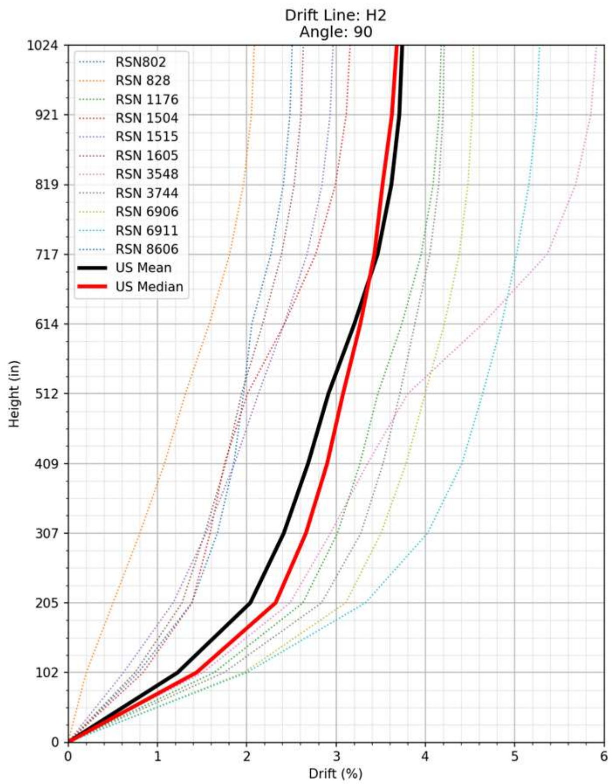


- Condition 1

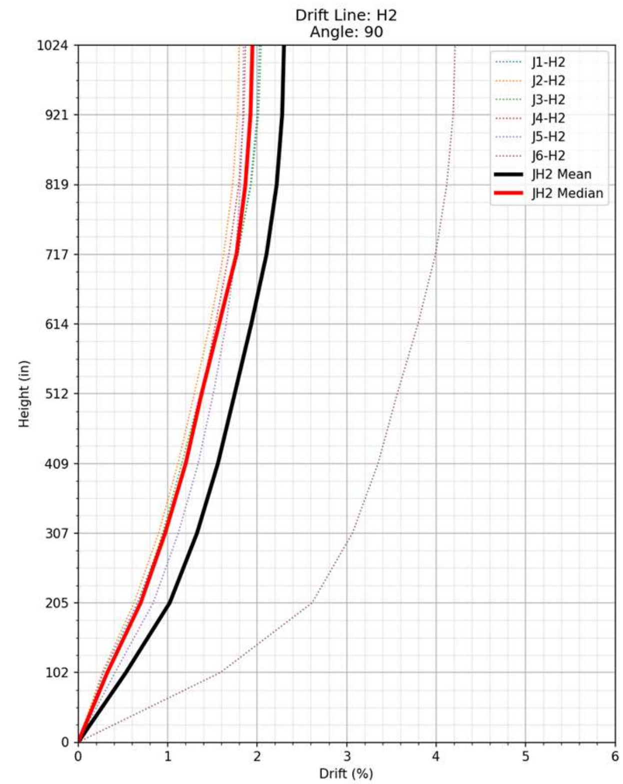


- Condition 2

Wall Performance: Drift



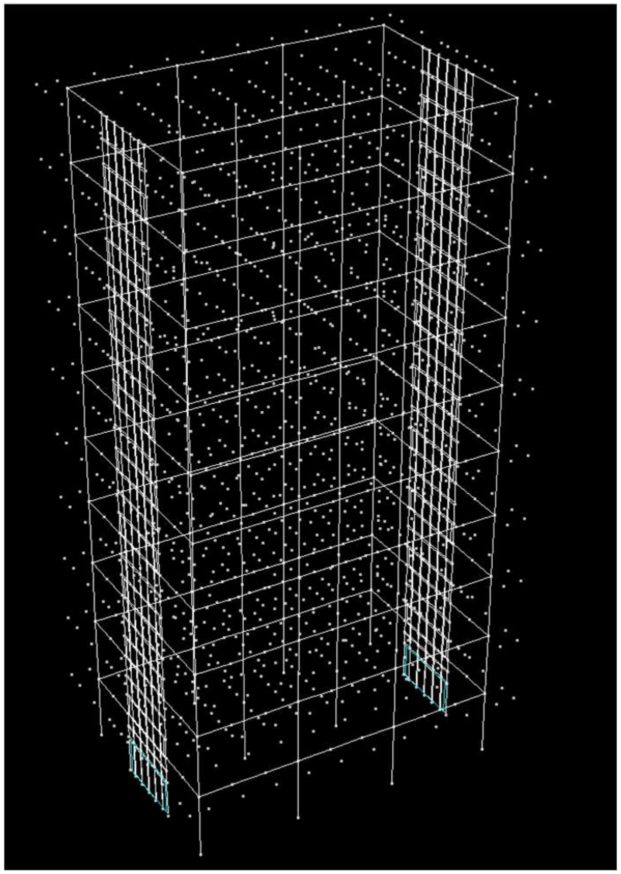
- US Suite



- Japan Suite

Wall Performance

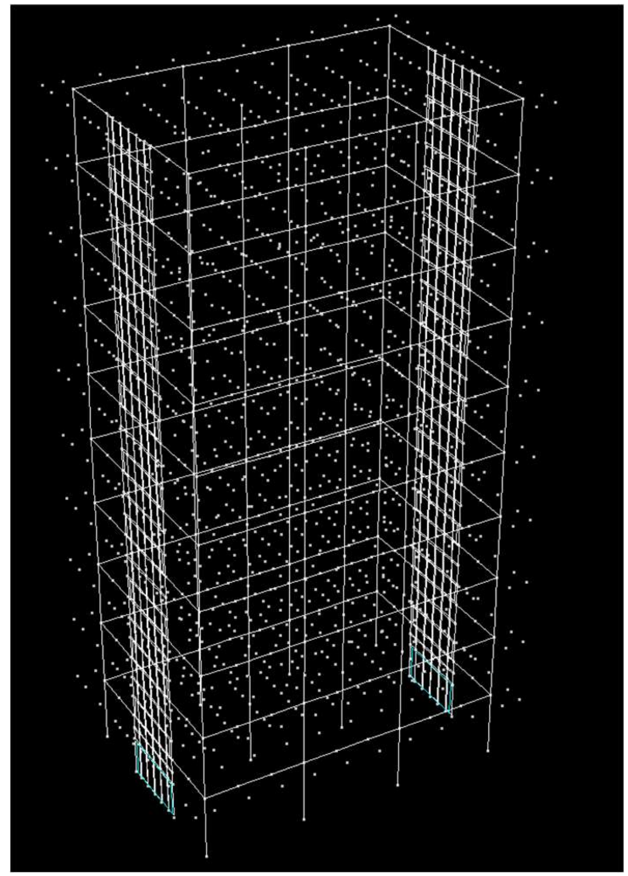
- DCR (Usage ratio per ASCE 41-23 rotation limits)



- Usage Ratio

  = 0.68

- US Suite



- Usage Ratio

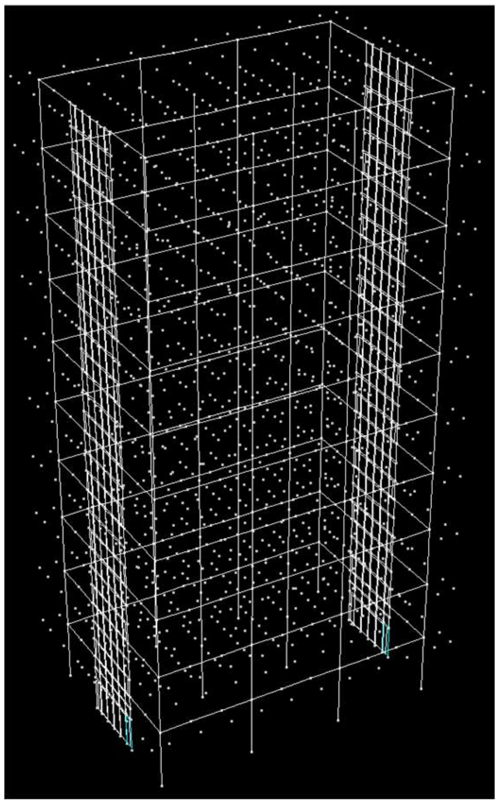
  = 0.25

- Japan Suite

Wall Performance

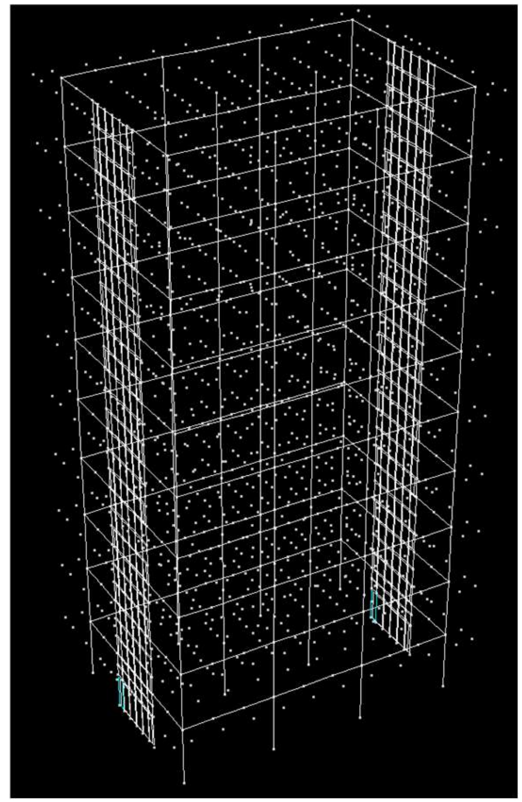
- Usage ratio per LATBSDC rotation limits

Reinforced concrete walls (primary hinge zone)	Full confinement of the entire cross section per ACI 318-19 18.10.6.4 [2]	Concrete compression strain over gage length [1]	$0.005/l_e$ ($0.01/l_e$ [3])
		Steel tension strain over gage length [1]	$0.01/l_e$ ($0.05/l_e$ [3])



- Usage Ratio

  = 0.25



- Usage Ratio

  = 0.4

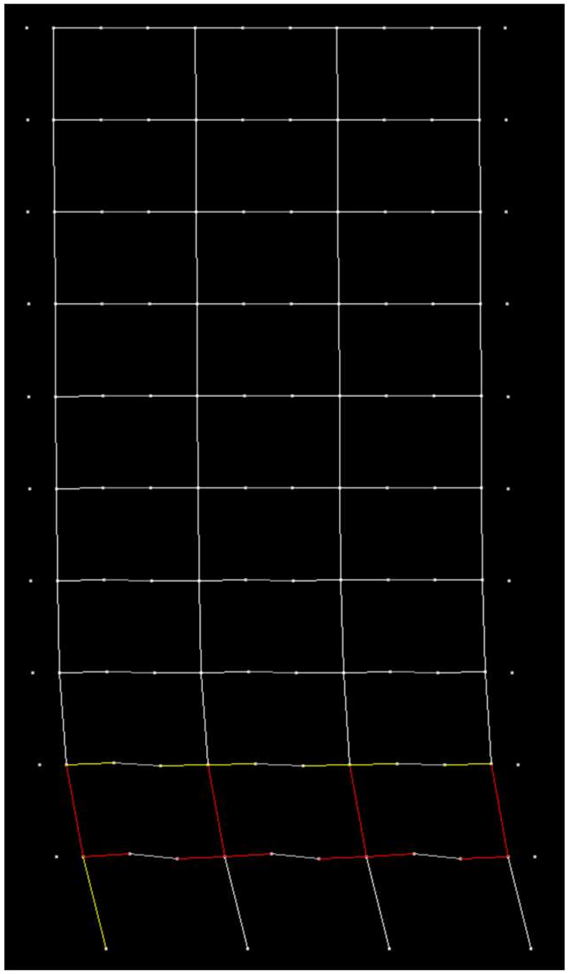
- US-Compression
- US-Tension

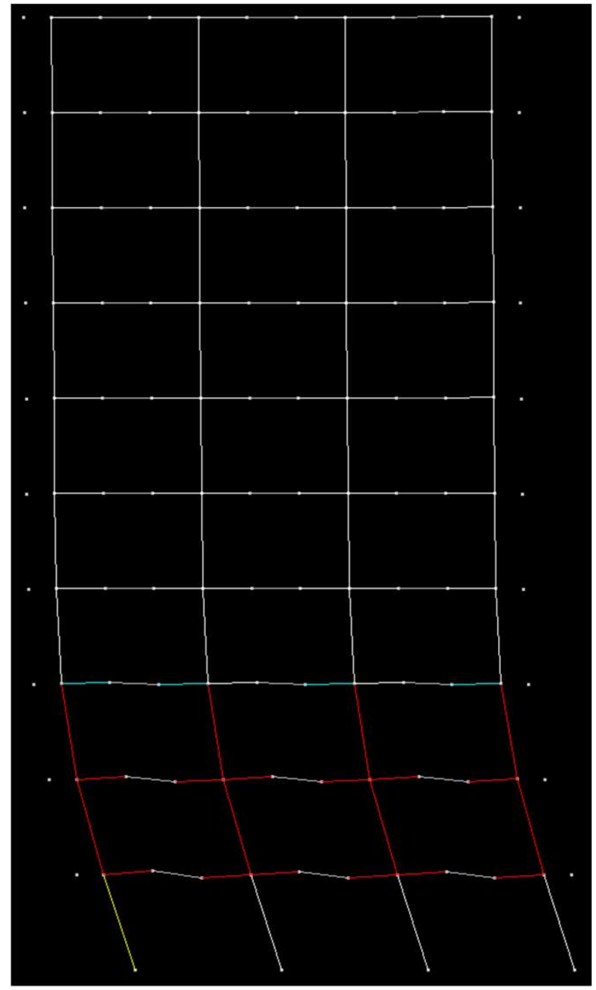
Moment Frame Direction

Table 4.1.2—Nonlinear modeling parameters for rectangular and circular columns in special moment frames based on database-

Column Shape	Plastic rotation angle, radians		Coefficient of variation Measured/Calculated plastic rotation		Residual strength ratio
	a	b	a	b	c
Rectangular	$0.05 - 0.05\frac{P}{A_g f_c'} + 0.60\rho_t - 0.03\frac{s}{d} \geq 0$	$0.05 - 0.05\frac{P}{A_g f_c'} + 0.55\rho_{lc} - 0.40\rho_t \geq a$	0.30	0.28	0.2
Circular	$0.07 - 0.09\frac{P}{A_g f_c'} + 0.50\rho_{lc} - 0.05\frac{V_p}{V_o} \geq 0$	$0.09 - 0.09\frac{P}{A_g f_c'} + 0.40\rho_{lc} - 0.05\frac{V_p}{V_o} \geq a$	0.36	0.29	0.2

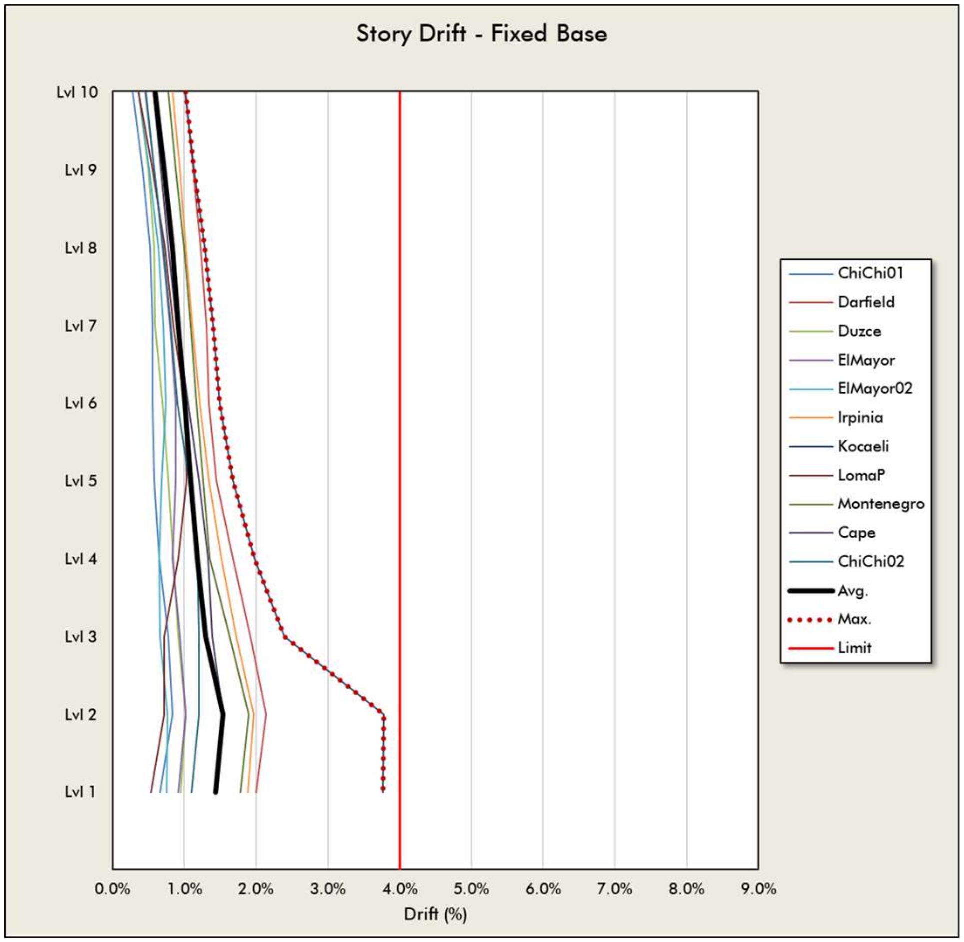
Moment Frame Yielding

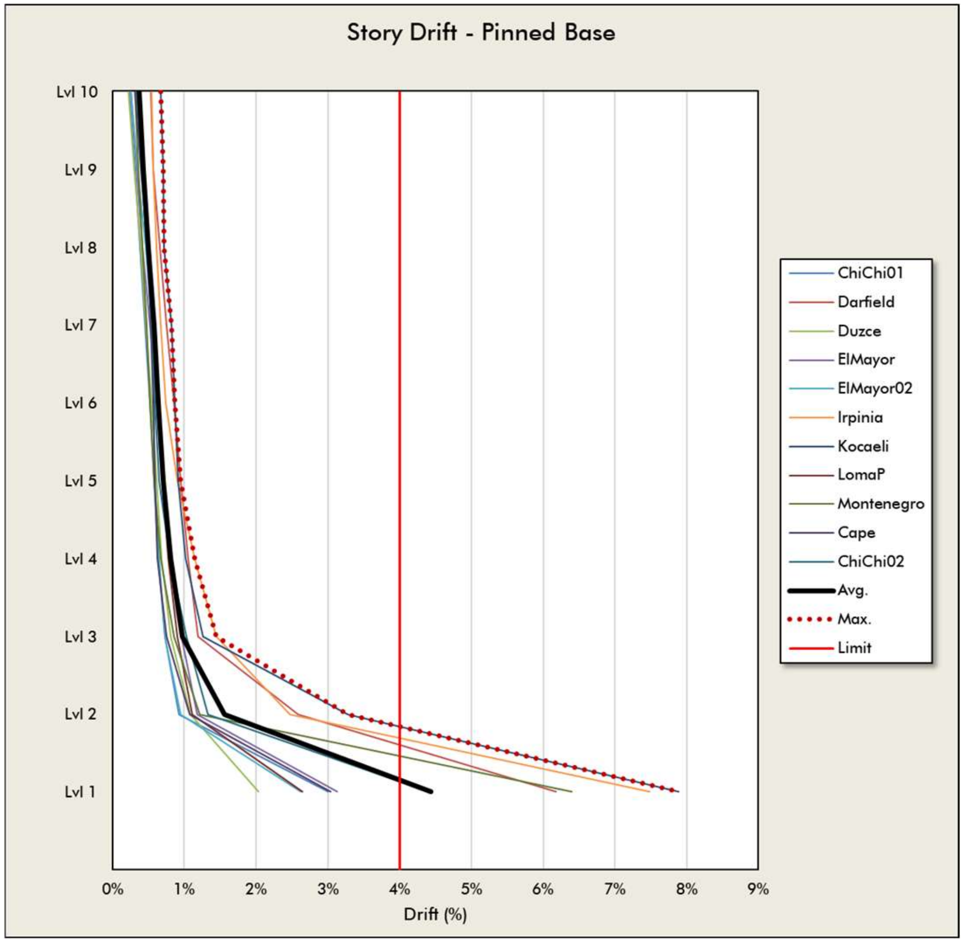




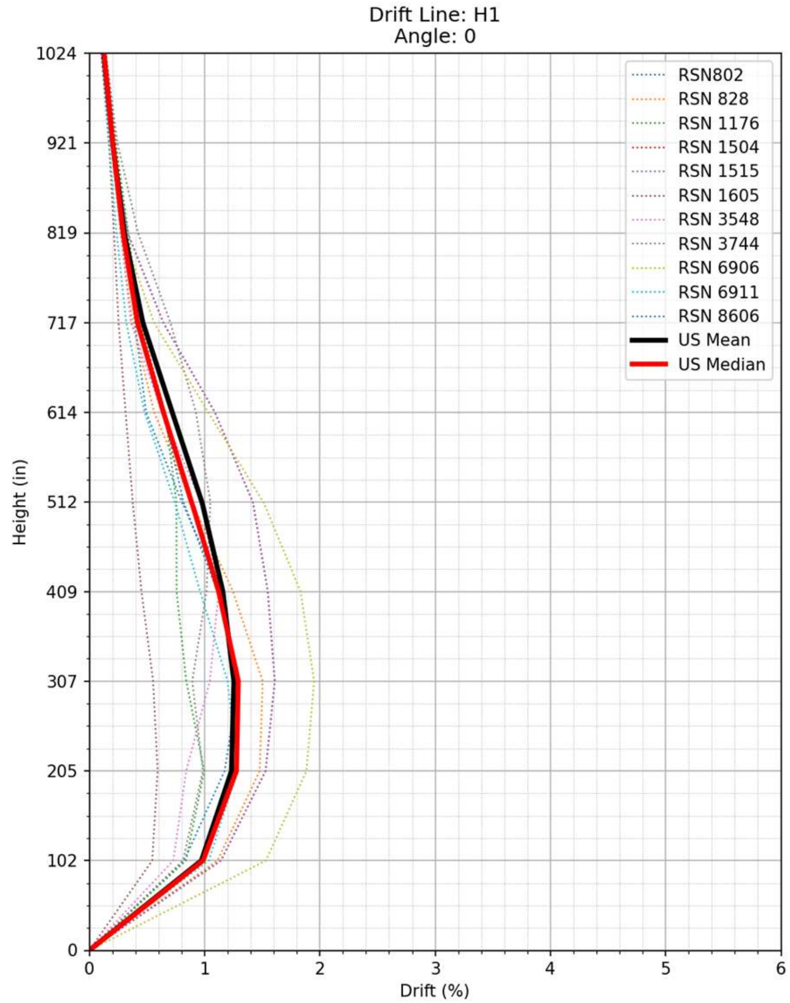
Nonlinear Model Sensitivity Studies

Analyses were compared using fixed (grade beams) versus pinned base assumptions:





Frame Direction



Comparison of Performance based Seismic Design with nonlinear time history response analysis in the US and Japan

– An example of 10 story RC building design according to Japanese practice –

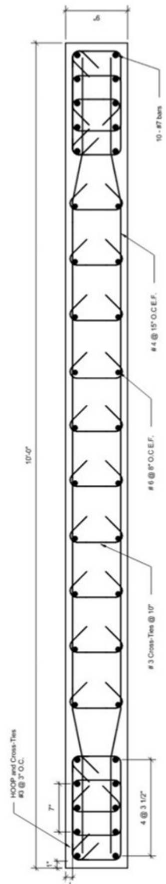
By

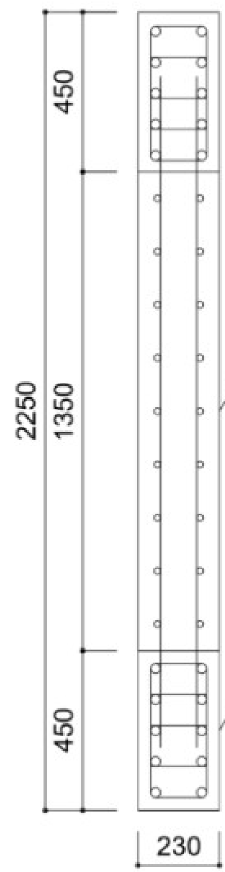
Japan Concrete Institute
Research Committee TC-233A

Table 1-1 Summary of performance engineering parameters and acceptance criteria

Type of Design Criteria	Verification Requirement		Design Criteria	Method to predict seismic response		
	Engineering Parameters	Member		Linear Elastic Frame analysis	Non-linear Pushover Frame Analysis	Non-linear time history response analysis of lump-mass model
Gravity Load	Stress	Column	Less than long term allowable stress	◎	–	–
		Girder	Less than long term allowable stress	◎	–	–
		Joint	Less than long term allowable stress	–	–	–
		Shear wall	Less than long term allowable stress	◎	–	–
Level 1 Earthquake 6 ground motions	Stress in member	Column	Less than short term allowable stress	–	◎	–
		Girder	Less than short term allowable stress	–	◎	–
		Joint	Less than short term allowable stress	–	–	–
		Shear wall	Less than short term allowable stress	–	◎	–
	Member ductility factor		Less than 1.0	–	◎	–
	Story drift ratio		Less than 0.005	–	–	◎
	Story drift ductility factor		Less than 1.0	–	–	◎
	–					
Level 2 Earthquake 6 ground motions	Member Strength	Column	Flexure: For interior column in compression $1.4M_{RU} \le M_u$ For exterior column in compression	–	◎	–
			Shear: $1.5V_{RU} \le V_u$ for column under moderate axial load $1.8V_{RU} \le V_u$ for column under high axial load	–	◎	–
			Axial: $-(3/4)_t N_u \le N_{RU} \le (2/3)_c N_u$	–	◎	–
		Girder	Shear: $1.1V_{RU} \le V_u$	–	◎	–
		Joint	Shear: $1.0V_{RU} \le V_u$	–		–
		Shear wall	Shear: $1.25V_{RU} \le V_u$	–	◎	–
			Axial: $N_{RU} \le {}_c N_u$ for boundary column in compression	–	◎	
	Member ductility factors		Less than 4.0	–	◎	
	Story drift ratio		Less than 0.01	–	–	◎
	Story drift ductility		Less than 2.0	–	–	◎
	...					

Comparative Design ACI Vs. JCI





Wall

Wall Thickness	230
Vertical Reinforcement	D16@150 Double
Horizontal Reinforcement	D16@150 Double

Column

B × D	230 × 450
Main Bar X	5-D29
Main Bar Y	2-D29
HOOP	5/2-U10@100

CONCRETE MOMENT FRAME COLUMN SCHEDULE										
MARK	SIZE (SHORTxLONG)	f'c (PSI)	LONGIT fy (KSI)	TRANSV fy (KSI)	VERT REINF	TIE CONFIG	VERT CONFIG	LOC 1 TIES	LOC 2 TIES	REMARKS
MFC1	500mm x 500mm	4,786	60	60	(8) #8	3x3	3x3	#4 @ 115mm	#4 @ 90mm	
MFC2	500mm x 500mm	4,786	60	60	(8) #7	3x3	3x3	#4 @ 115mm	#4 @ 90mm	

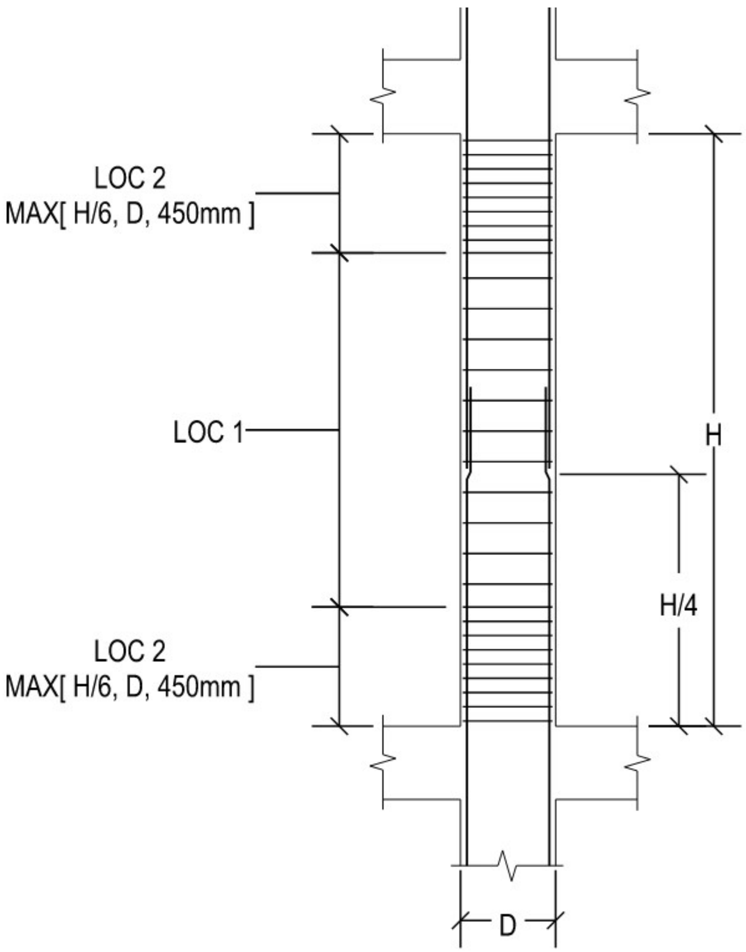
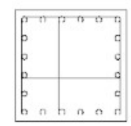
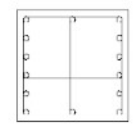
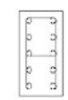
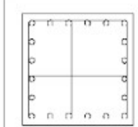
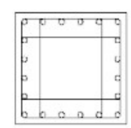
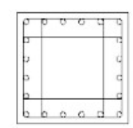
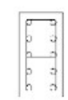
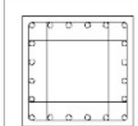
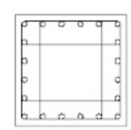
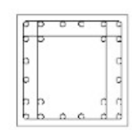
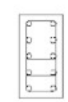
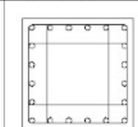
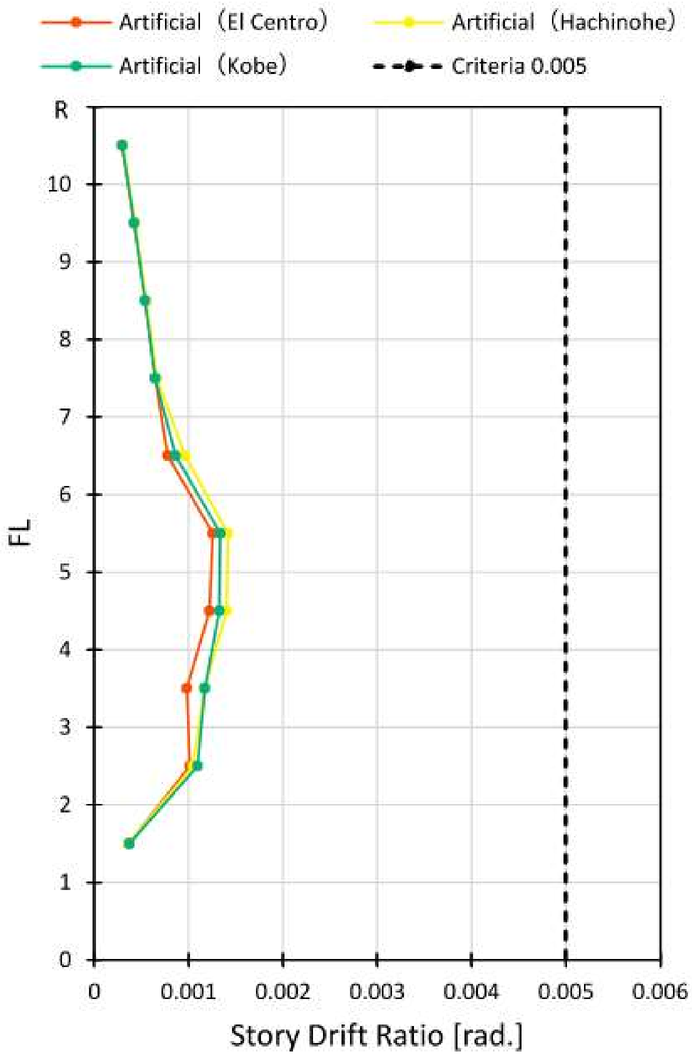
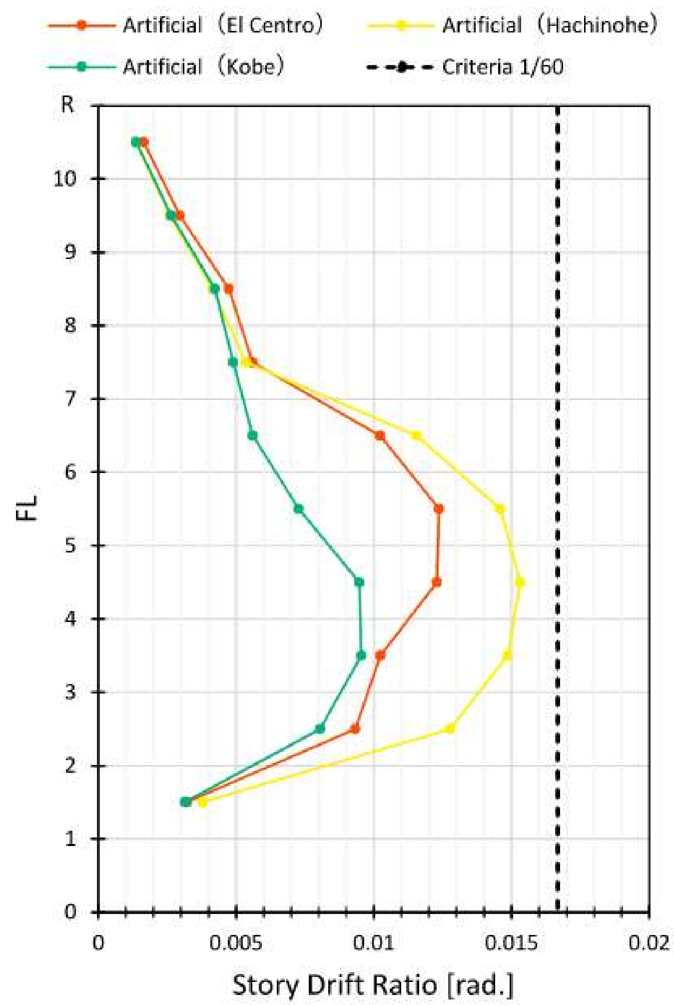


Table 1-4: List of columns and boundary member section

Symbol	C1	C2	C3	C4
8F～10F				
B×D	600×600	600×600	230×450	600×600
Main Bar X	6-D29	6-D29	5-D29	6-D29
Main Bar Y	6-D29	3-D29	2-D29	6-D29
HOOP	3/3-U10@100	3/3-U10@100	3/2-U10@100	3/3-U10@100
5F～7F				
B×D	600×600	600×600	230×450	600×600
Main Bar X	6-D29	6-D29	5-D29	6-D29
Main Bar Y	6-D29	6-D29	2-D29	6-D29
HOOP	4/4-U10@100	4/4-U10@100	3/2-U10@100	4/4-U10@100
1F～4F				
B×D	600×600	600×600	230×450	600×600
Main Bar X	6-D29	6+4-D29	5-D29	6-D29
Main Bar Y	6-D29	6+4-D29	2-D29	6-D29
HOOP	4/4-U10@100	4/4-U10@100	5/2-U10@100	4/4-U10@100


Artificial（El Centro）
Artificial（Hachinohe）
Artificial（Kobe）
Criteria 0.005
R
10
9
8
7
6
5
4
3
2
1
0
FL
0
0.001
0.002
0.003
0.004
0.005
0.006
Story Drift Ratio [rad.]


Artificial（El Centro）
Artificial（Hachinohe）
Artificial（Kobe）
Criteria 1/60
R
10
9
8
7
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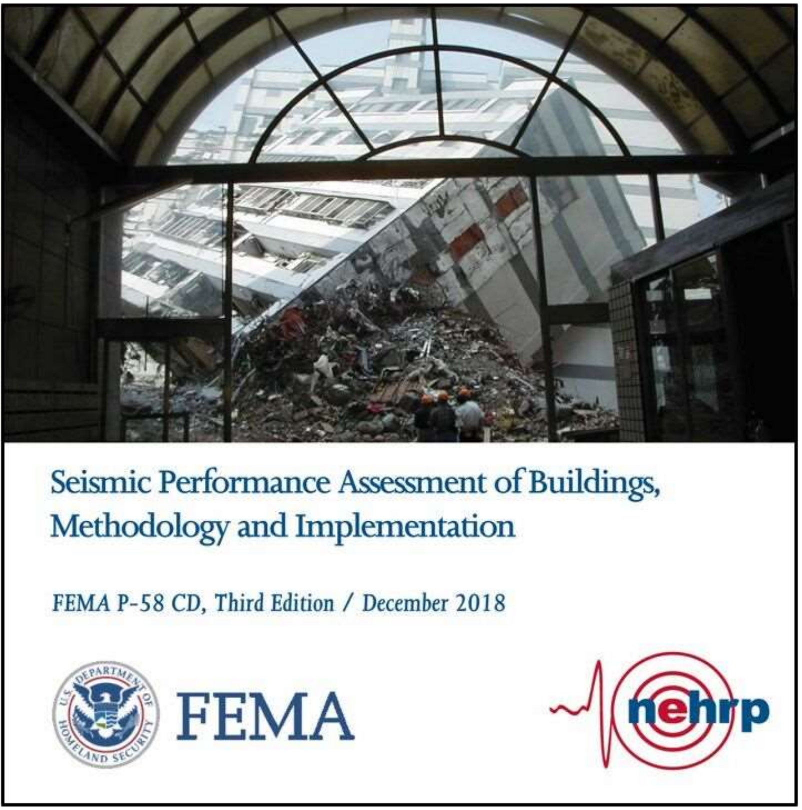
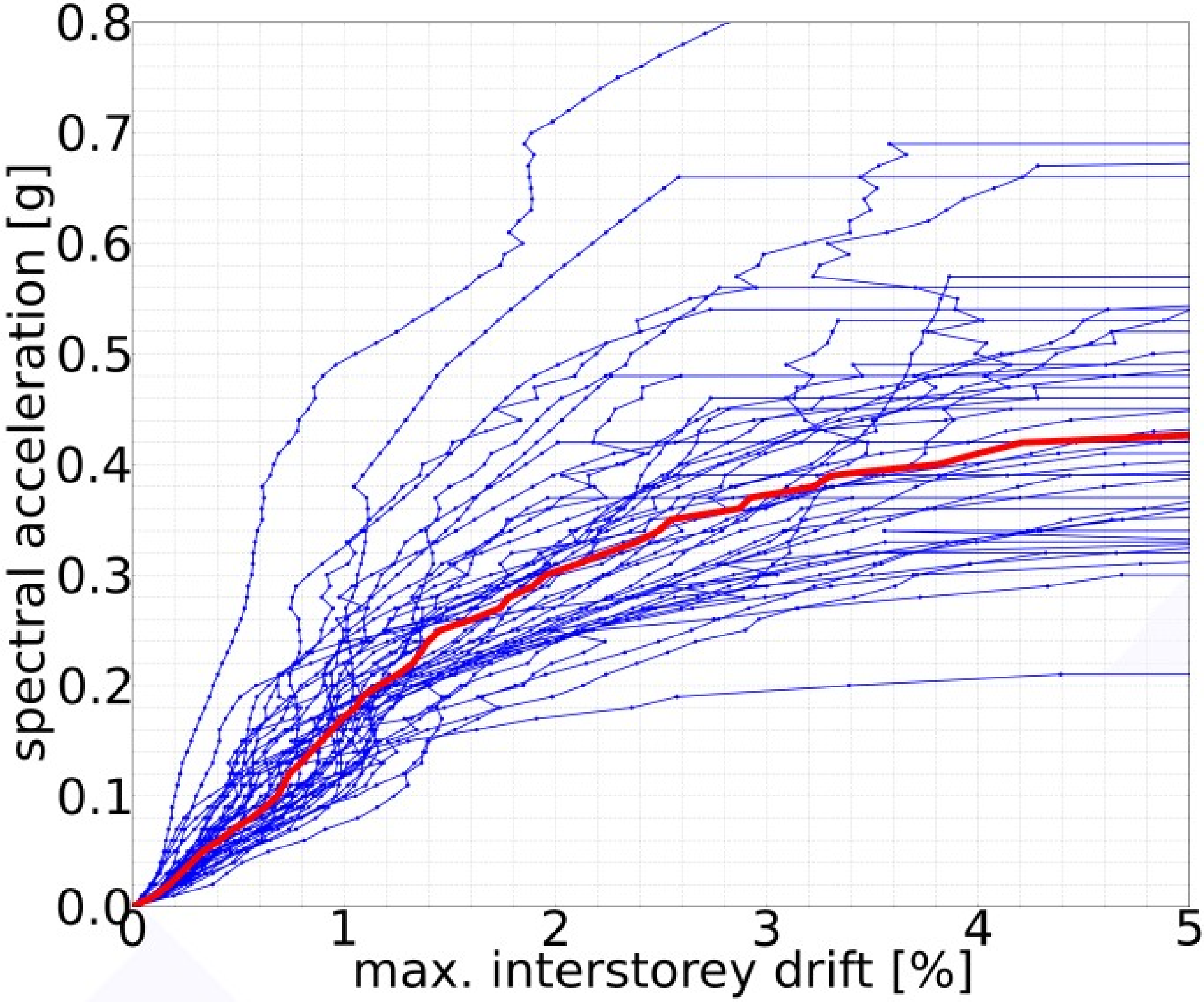
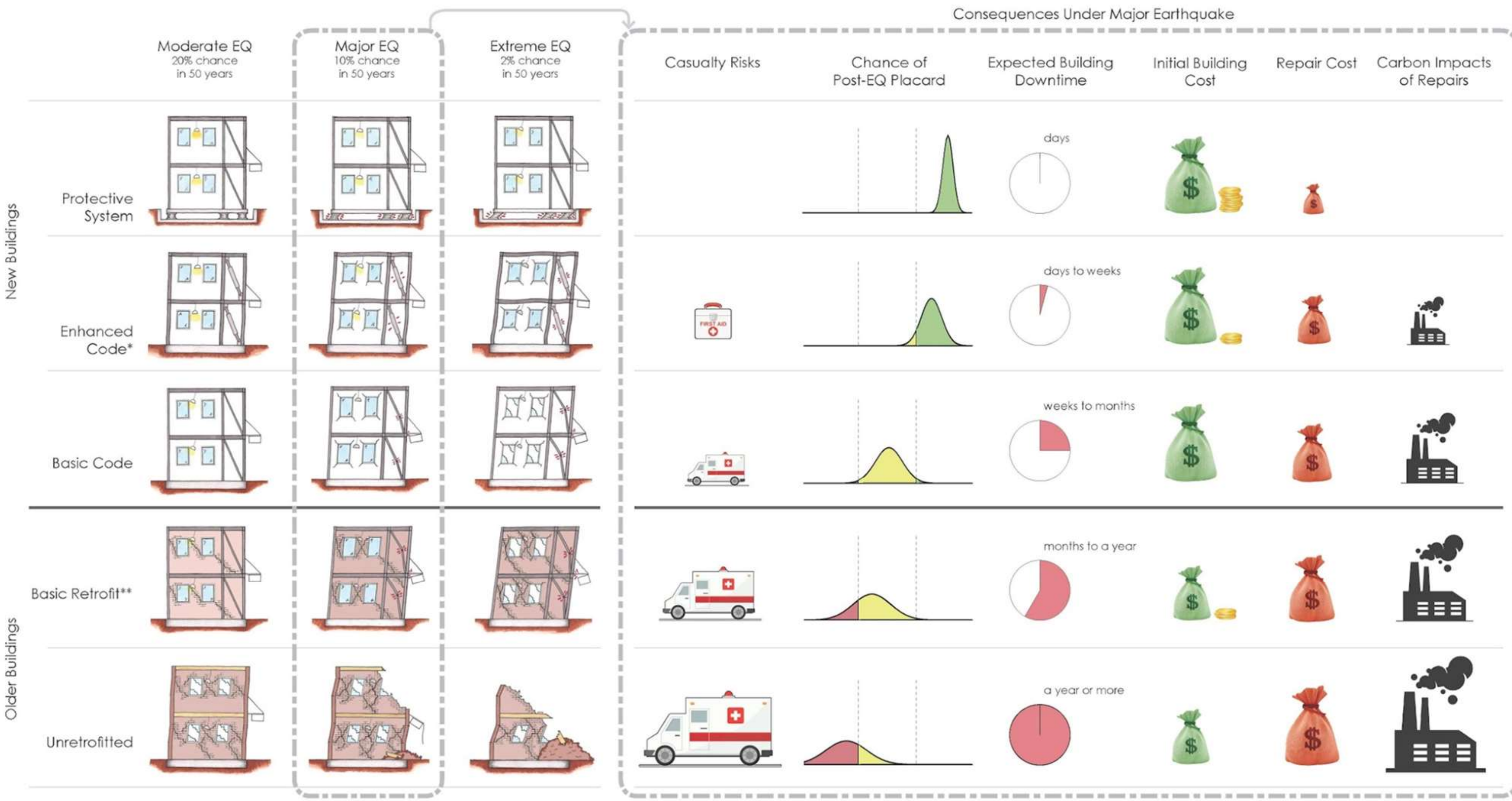
0.02
Story Drift Ratio [rad.]


Seismic Performance Assessment of Buildings,
Methodology and Implementation
FEMA P-58 CD, Third Edition / December 2018
FEMA
nehrp


spectral acceleration [g]
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
max. interstorey drift [%]
0
1
2
3
4
5

OPTIONS FOR EARTHQUAKE-RESISTANT DESIGN

Design Decisions Have Measurable Consequences



Earthquake damage varies depending on the level of shaking experienced and characteristics of the building's structure and other systems. Design choices affect the amount and types of likely damage.

* Includes Risk Category IV and other buildings with enhanced seismic resistance features.

** Some retrofitted buildings can experience much less damage.

New code-compliant buildings have very low risks of casualties in the US. Some older buildings have high safety risks.

Design choices affect the odds of a green tag (no restrictions), yellow tag (restricted entry), or red tag (unsafe, no entry).

Design choices affect the amount of time required before a building can be occupied after an earthquake.

More resilient buildings typically cost slightly more upfront, but result in lower post-earthquake repair costs and consequences.

Building materials require energy to produce. The amount and types of repairs required affect carbon releases and energy usage.

COMPARING DESIGN TRADE-OFFS

	Existing Building	Option A	Option B	Option C
Casualties				
Downtime				
Damage Cost	$$$$	$$	$$	$
Construction Cost		$$	$	$$$
Interior Impact				
Exterior Impact				
Construction Surprise Risk				
LADBS Risk				