



2020 West El Camino Avenue, Suite 800
Sacramento, CA 95833
hcai.ca.gov



Hospital Building Safety Board Structural and Nonstructural Regulations Committee

AGENDA

July 29, 2026

10:00 a.m. – 4:00 p.m.

The Committee may not discuss or act on any matter raised during the public comment section that is not included on this agenda, except to place the matter on a future meeting agenda. (Government Code §§ 11125, 11125.7, subd. (a).)

Locations:

[2020 West El Camino Ave, Conference Room 930, Sacramento, CA 95833](#)

[355 South Grand Avenue, Conference Room 2000, Los Angeles, CA 90071](#)

[Teams Meeting Access](#); Meeting ID: 214 524 335 948 07; Passcode: gG7nx75Y

Call in: (916) 535-0978; Phone Conference ID: 136 393 606#

Item #1

Call to Order and Welcome

*Facilitator: Jim Malley, SE, Senior Principal, Degenkolb Engineers;
Committee Chair (or designee)*

Item #2

Roll Call and Meeting Advisories/Expectations

Facilitator: Veronica Yuke, HCAI; Executive Director (or designee)

Item #3 Update on Wood-Frame Standard Details prepared by the HBSB
Codes and Processes Committee

- Discussion and public input

*Facilitator: Gary Dunger, Executive Director, Design and
Construction, Cedars-Sinai Health System (or designee)*

Item #4

Update on OSHPD Preapproved Details (OPD): Standard Partition Wall

- Comparison between design of partition walls using American Society of Civil Engineers (ASCE) 7-16 and ASCE 7-22
- Applicability of 2013 OPD to the 2025 California Building Code (CBC)
- Committee to endorse and/or provide feedback on modifications presented
- Discussion and public input

*Facilitator: Chris Davis, SE, District Structural Engineer, HCAI
(or designee)*

OSHPPD Preapproved Partition Wall Detail Updates

By

Chris Davis & Roy Lobo

To HBSB – SNSR Committee

7/29/2026

Motivation for this Study

- Demands on partition walls in hospital buildings have doubled in some cases
- Correspondingly stud sizes and anchorage top and bottom has increased resulting in higher costs
- For high seismic areas would have demands outside the existing OPD tables requiring custom details
- Partition wall damage/cracks are generally associated with inter-story drifts
- Full height or braced partial height partition wall failures have generally been linked to screw pullout between gypsum board and the studs due to out of plane seismic forces.
- Out of plane failures have been observed for unbraced or ceiling supported partition walls.

Outline of Presentation

- *Background on seismic design force F_p for design of nonstructural components in ASCE 7-22*
- *Partition wall design impacts caused by the new seismic design forces*
- *How was the new force equation in ASCE 7-22 developed?*
- *Floor accelerations used in the F_p equation for demands on Nonstructural Components (NSC)*
- *Floor accelerations for various building types that influence the design of NSC*
- *Summary areas of conservatism in the current force demands on Partition Walls*
- *Impact of new ASCE 7-22 requirements on stud sizes and anchor types for Partition Wall design*

ASCE 7-16 vs 7-22 Nonstructural Horizontal Seismic Component Equation Comparison

F_p = Horizontal seismic component design force equation comparison
(similarities)

$$F_p = \frac{0.4 a_p S_{DS} W_p}{\left(\frac{R_p}{I_p} \right)} \left(1 + 2 \frac{z}{h} \right) \quad (13.3-1)$$

ASCE 7-16

VS

$$F_p = 0.4 S_{DS} I_p W_p \left[\frac{H_f}{R_\mu} \right] \left[\frac{C_{AR}}{R_{po}} \right] \quad (13.3-1)$$

ASCE 7-22

Portions remain
unchanged

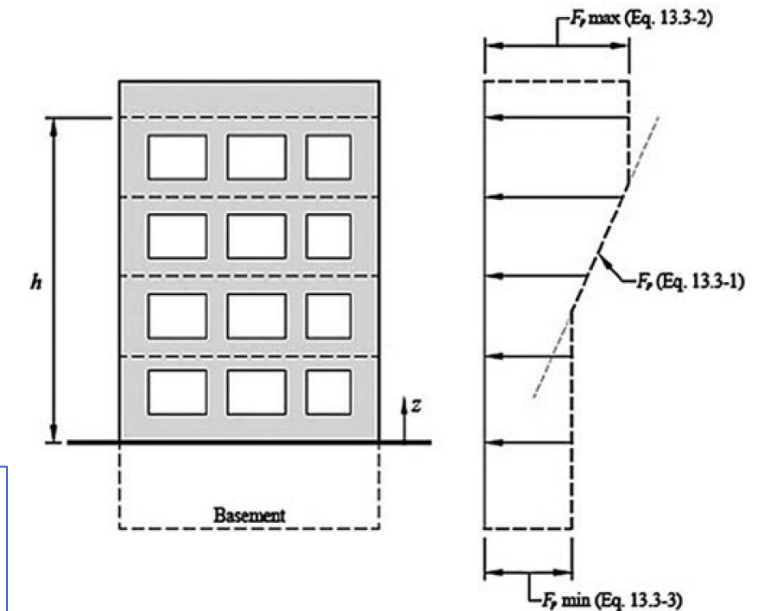
Upper and lower bounds
remain unchanged

F_p is not required to be taken as greater than

$$F_p = 1.6 S_{DS} I_p W_p \quad (13.3-2)$$

and shall not be taken as less than

$$F_p = 0.3 S_{DS} I_p W_p \quad (13.3-3)$$



F_p is *now* a function of the **Building Seismic Force Resisting Systems (SFRS)**

$$F_p = 0.4 S_{DS} I_p W_p \left[\frac{H_f}{R_\mu} \right] \left[\frac{C_{AR}}{R_{po}} \right]$$

$$R_\mu = [1.1 R (I_e \Omega_0)]^{1/2} \geq 1.3 \quad (13.3-6)$$

R_μ = Structure *ductility* reduction factor, based upon the SFRS

R = Response modification coefficient, SFRS in Table 12.2-1, 15.4-1, or 15.4-2

I_e = Importance Factor

Ω_0 = Overstrength factor as defined in Table 12.2-1, 15.4-1, or 15.4-2



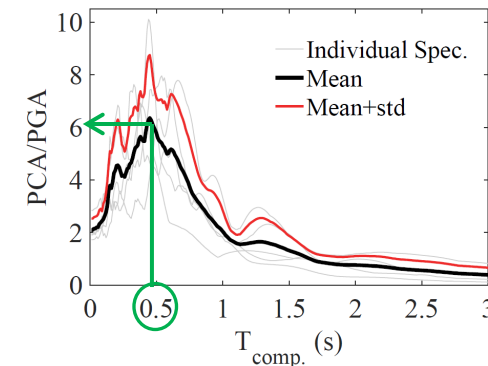
Concrete Shear wall



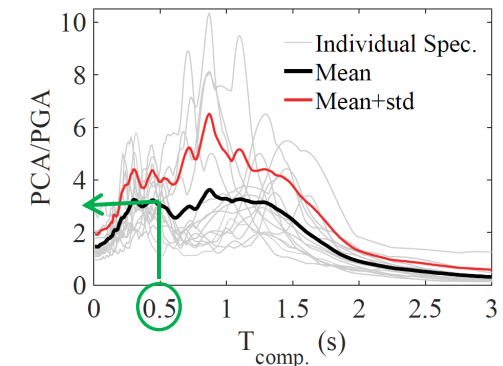
Moment Frame



Steel Braced Frame



(a) normalized roof spectra for relatively stiff five-to-seven story buildings (a sample size of seven)



(b) normalized roof spectra for relatively flexible five-to-seven story buildings (a sample size of 24)

Effect of building stiffness on PCA/PGA from instrumental recording.

Source: NIST GCR 18-917-43

Current requirements in ASCE 7-22

- *Design force F_p from a Nonlinear Response History Analysis Section 13.3.1.5*

$$F_p = I_p W_p a_i \left[\frac{C_{AR}}{R_{po}} \right] \quad (13.3-7)$$

$$F_p = 0.4 S_{DS} I_p W_p \left[\frac{H_f}{R_\mu} \right] \left[\frac{C_{AR}}{R_{po}} \right] \quad (13.3-1)$$

- *Upper and lower limits shall apply*

F_p is not required to be taken as greater than

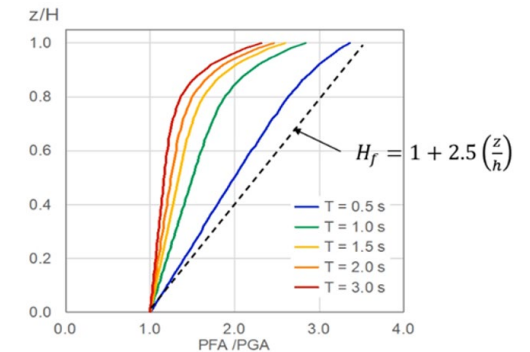
$$F_p = 1.6 S_{DS} I_p W_p \quad (13.3-2)$$

and shall not be taken as less than

$$F_p = 0.3 S_{DS} I_p W_p \quad (13.3-3)$$

$$H_f = 1 + a_1 \left(\frac{z}{h} \right) + a_2 \left(\frac{z}{h} \right)^{10}$$

$$H_f = 1 + 2.5 \left(\frac{z}{h} \right) \text{ Default}$$



Partition Wall Design Impacts from ASCE 7-22 Updates

- F_p equation has changed from ASCE 7-16 to ASCE 7-22, along with the non-structural response coefficients. This has resulted in large force level increases in walls with a height over 9'

What has Changed?

ASCE 7-16

$$F_p = \frac{0.4 a_p S_{DS} W_p}{\left(\frac{R_p}{I_p} \right)} \left(1 + 2 \frac{z}{h} \right)$$

ASCE 7-22

$$F_p = 0.4 S_{DS} I_p W_p \left[\frac{H_f}{R_\mu} \right] \left[\frac{C_{AR}}{R_{po}} \right]$$

Table 13.5-1 Coefficients for Architectural Components

Architectural Component	a_p^a	R_p	Ω_0^b
Interior nonstructural walls and partitions ^c			
Plain (unreinforced) masonry walls	1	1½	1½
All other walls and partitions	1	2½	2

Table 13.5-1. Coefficients for Architectural Components.

Architectural Component	C_{AR}		R_{po}	Ω_{op}^a
	Supported at or below grade plane	Supported above grade plane by a structure		
Interior nonstructural walls and partitions ^b				
Light frame ≤ 9 ft (2.74 m) in height	1	1	1.5	2
Light frame > 9 ft (2.74 m) in height	1.4	1.4	1.5	2

2.33 increase in the component amplification factor partition walls > 9'

Impact of the Change

- Partition wall demands using ASCE 7-22 have increased **1.5** times the demands for partition walls $< 9'$ and **2.1** times the demands for partition walls $> 9'$ when the default building type is chosen
- Is the change justified?



How was the Force Equation in ASCE 7-22 Developed?

Newton's Second Law: $F = ma$

Component acceleration = Floor Acceleration x Component Response Factor

Force Equation in ASCE 7-22 can be written as

$$F_p = 0.4S_{DS}I_pW_p \left[\frac{H_f}{R_\mu} \right] \left[\frac{C_{AR}}{R_{po}} \right]$$

The diagram shows the force equation $F_p = 0.4S_{DS} \left[\frac{H_f}{R_\mu} \right] \left[\frac{C_{AR}}{R_{po}} \right] I_p W_p$ with several terms circled in red and labeled with blue arrows:

- $0.4S_{DS} \left[\frac{H_f}{R_\mu} \right]$ is circled in red, with an arrow pointing to it from the label "Floor Acceleration / g".
- $\left[\frac{C_{AR}}{R_{po}} \right]$ is circled in red, with an arrow pointing to it from the label "Component Response Factor".
- I_p is circled in red, with an arrow pointing to it from the label "Component Importance Factor".
- W_p is circled in red, with an arrow pointing to it from the label "Equipment mass x g".

Floor Accelerations a_i used for Calculation of Demands on NSC

- Given a_i in equation 13.3-7 for design of NSC, the a_i component can be extracted from Eq. 13.3-1 and Eq. 12.10-4 and compared for different building types and Risk Categories, as shown next.
- Note $R_s = 1$ when used in these comparisons

$$F_p = I_p W_p a_i \left[\frac{C_{AR}}{R_{po}} \right] \quad (13.3-7)$$

$$F_p = 0.4 S_{DS} I_p W_p \left[\frac{H_f}{R_\mu} \right] \left[\frac{C_{AR}}{R_{po}} \right] \quad (13.3-1) \quad \longrightarrow \quad a_i = 0.4 S_{DS} \left[\frac{H_f}{R_\mu} \right]$$

$$F_{px} = \frac{C_{px}}{R_s} w_{px} \quad (12.10-4) \quad \longrightarrow \quad a_i = C_{px}$$

Floor accelerations a_i for various building types

- Special Reinforced Concrete Moment Frames, RC-II Bldgs

$$F_p = I_p W_p a_i \left[\frac{C_{AR}}{R_{po}} \right] \quad (13.3-7)$$

$$F_p = 0.4 S_{DS} I_p W_p \left[\frac{H_f}{R_\mu} \right] \left[\frac{C_{AR}}{R_{po}} \right] \quad (13.3-1)$$

$$F_{px} = \frac{C_{px}}{R_s} W_{px} \quad (12.10-4)$$

$$H_f = 1 + a_1 \left(\frac{z}{h} \right) + a_2 \left(\frac{z}{h} \right)^{10} \quad (13.3-4)$$

$$H_f = 1 + 2.5 \left(\frac{z}{h} \right) \quad (13.3-5)$$

where

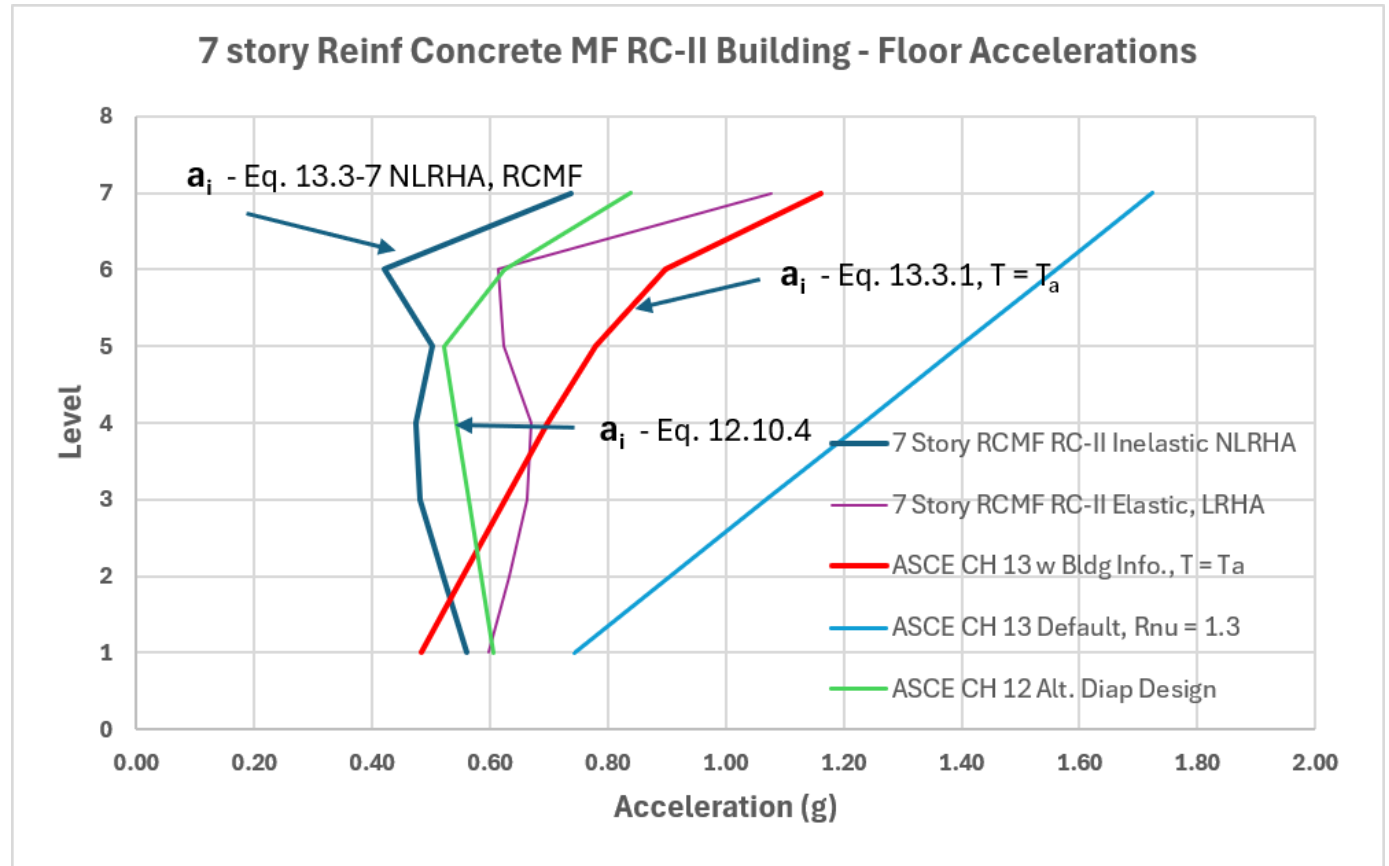
$$a_1 = 1/T_a \leq 2.5;$$

$$a_2 = [1 - (0.4/T_a)^2] \geq 0;$$

z = Height above the base of the structure to the point of attachment of the component. For items at or below the base, z shall be taken as 0. The value of $\frac{z}{h}$ need not exceed 1.0;

h = Average roof height of structure with respect to the base; and

T_a = Lowest approximate fundamental period of the supporting building or nonbuilding structure in either orthogonal direction. For structures with combinations of seismic force-resisting systems (SFRSs), the SFRS that produces the lowest value of T_a shall be used.



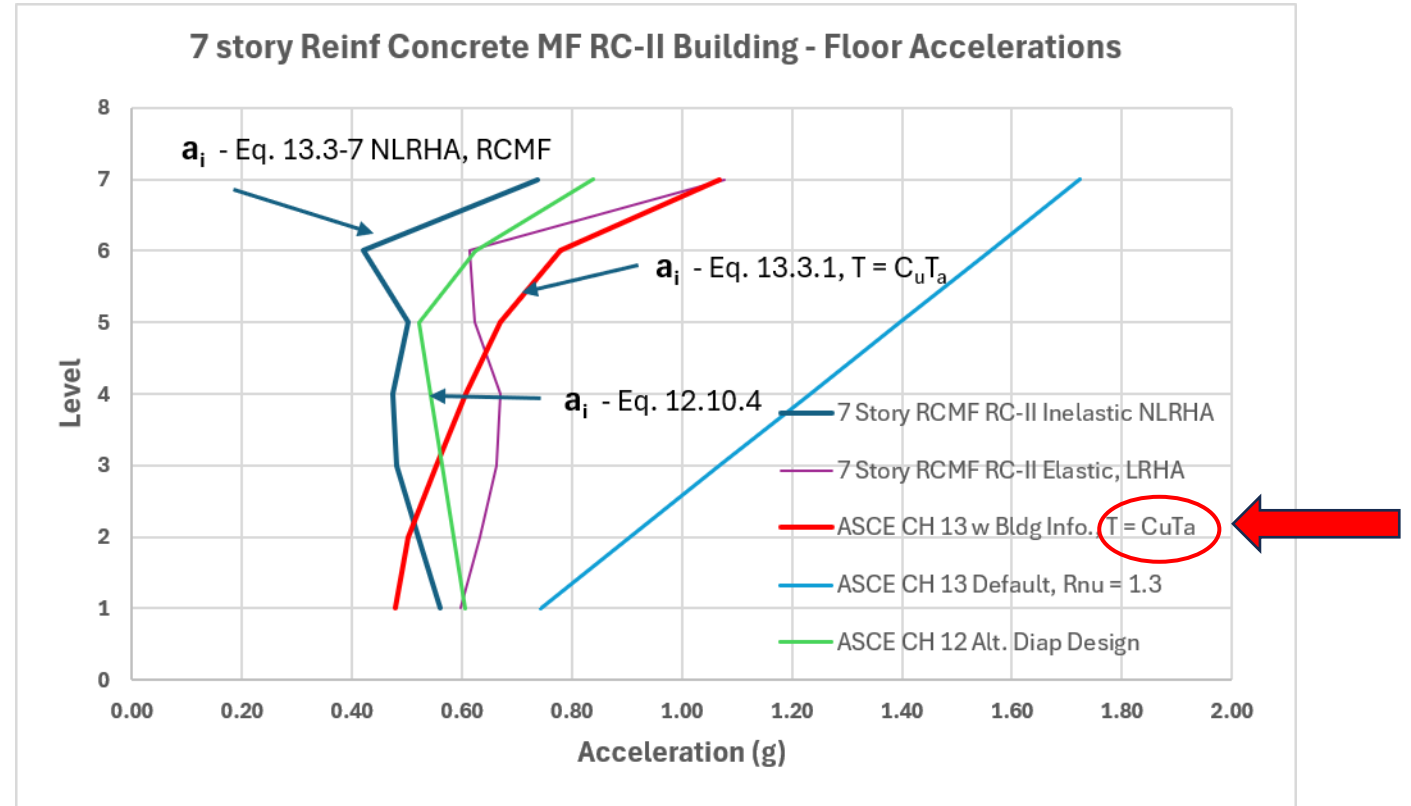
Impact of building period on a_i in Eq. 13.3-1

- Special Reinforced Concrete Moment Frames, RC-II Bldgs

$$F_p = I_p W_p a_i \left[\frac{C_{AR}}{R_{po}} \right] \quad (13.3-7)$$

$$F_p = 0.4 S_{DS} I_p W_p \left[\frac{H_f}{R_\mu} \right] \left[\frac{C_{AR}}{R_{po}} \right] \quad (13.3-1)$$

$$F_{px} = \frac{C_{px}}{R_s} W_{px} \quad (12.10-4)$$



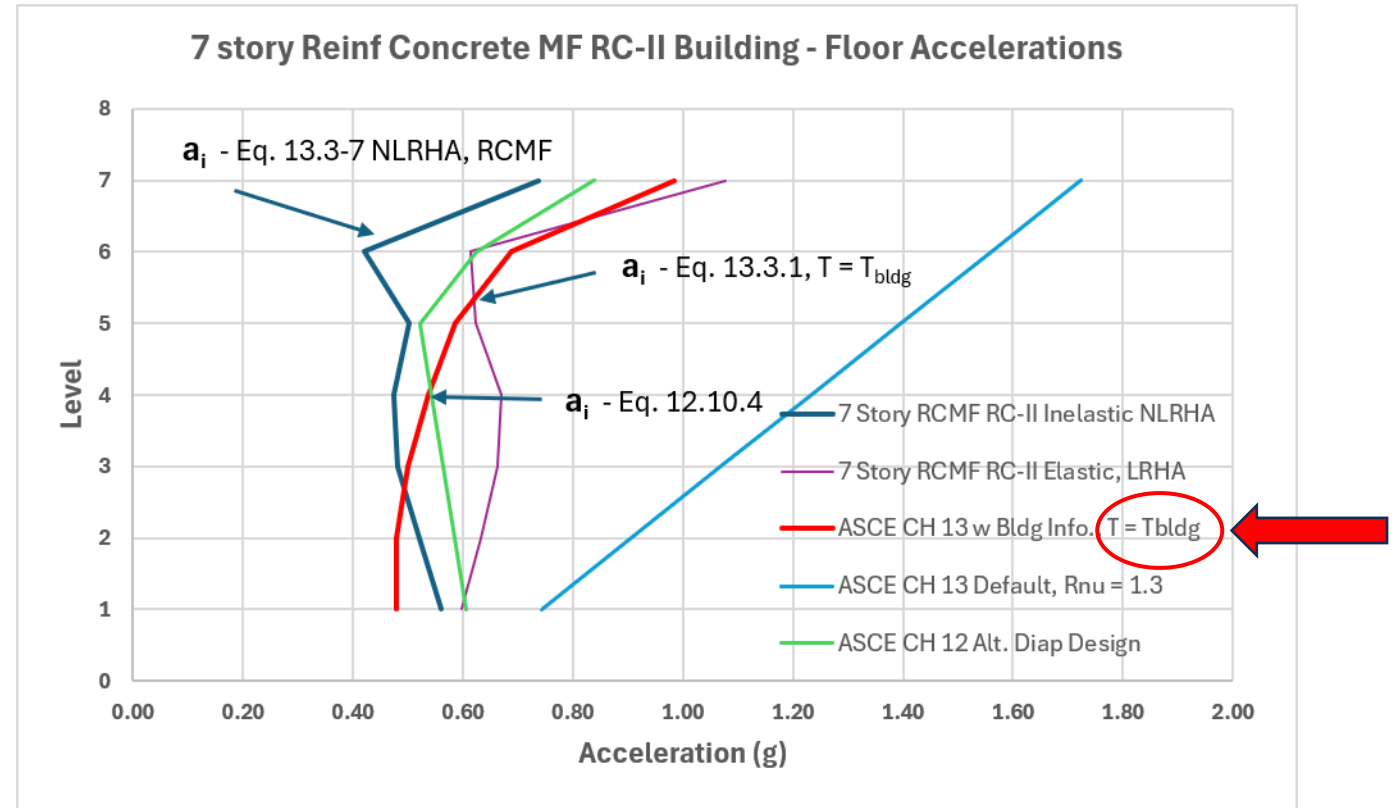
Impact of building period on a_i in Eq. 13.3-1

- Special Reinforced Concrete Moment Frames, RC-II Bldgs

$$F_p = I_p W_p \left[\frac{C_{AR}}{R_{po}} \right] \quad (13.3-7)$$

$$F_p = 0.4 S_{DS} I_p W_p \left[\frac{H_f}{R_\mu} \right] \left[\frac{C_{AR}}{R_{po}} \right] \quad (13.3-1)$$

$$F_{px} = \frac{C_{px}}{R_s} W_{px} \quad (12.10-4)$$



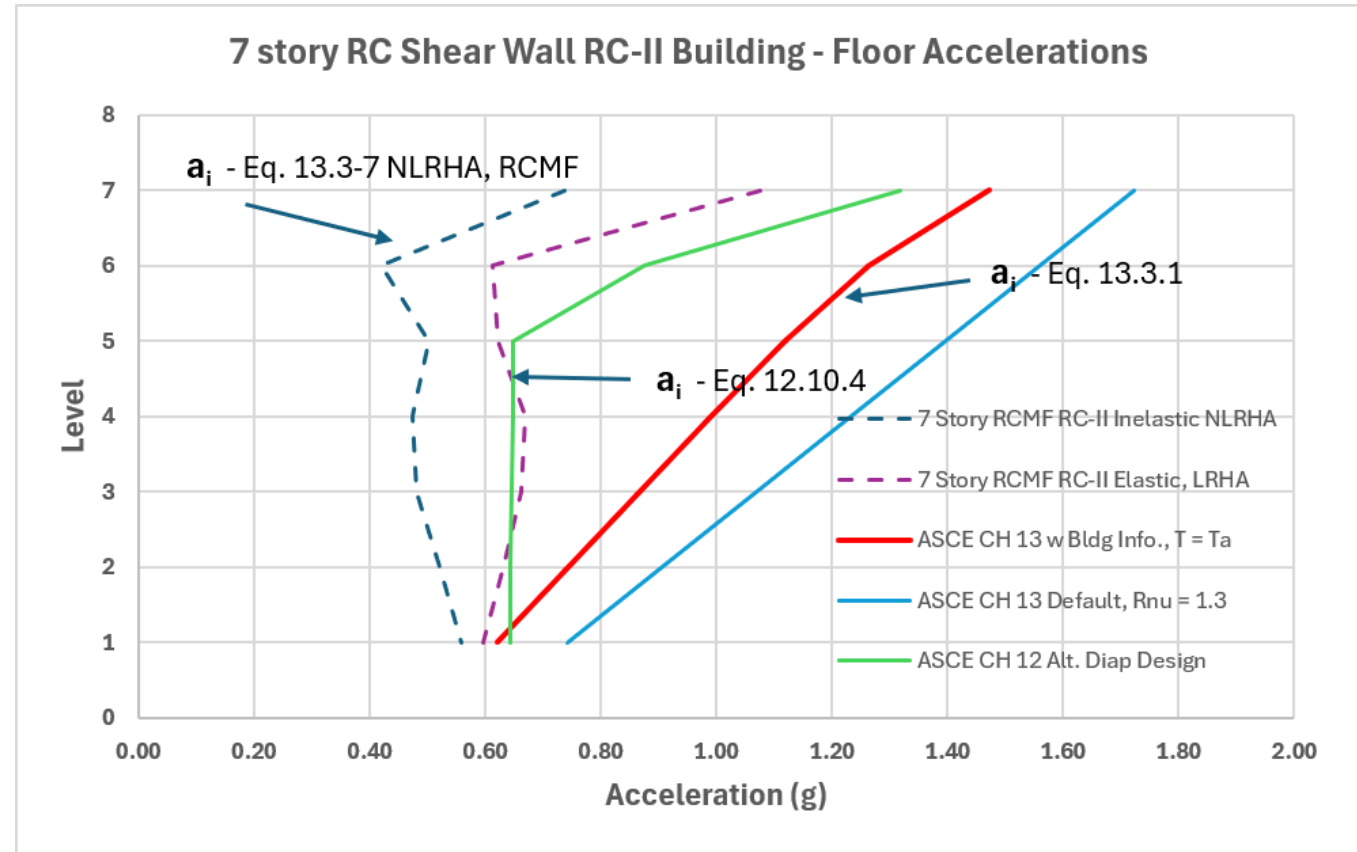
Floor accelerations a_i for various building types

- Special Reinforced Concrete Shear Walls, RC-II Bldgs

$$F_p = I_p W_p a_i \left[\frac{C_{AR}}{R_{po}} \right] \quad (13.3-7)$$

$$F_p = 0.4 S_{DS} I_p W_p \left[\frac{H_f}{R_\mu} \right] \left[\frac{C_{AR}}{R_{po}} \right] \quad (13.3-1)$$

$$F_{px} = \frac{C_{px}}{R_s} W_{px} \quad (12.10-4)$$



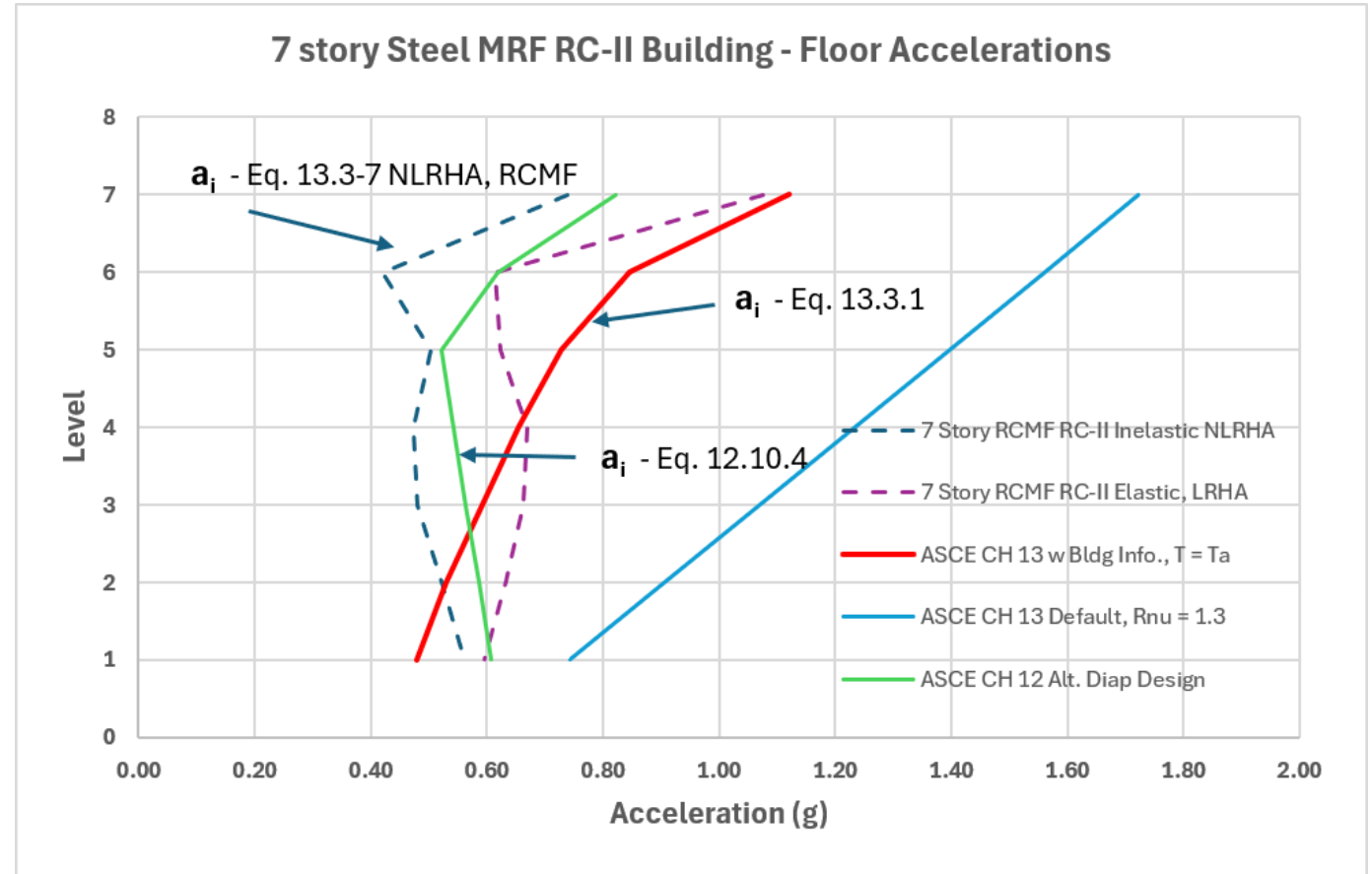
Floor accelerations a_i for various building types

- Steel Special Moment Resisting Frame, RC-II Bldgs

$$F_p = I_p W_p \left(a_i \right) \left[\frac{C_{AR}}{R_{po}} \right] \quad (13.3-7)$$

$$F_p = 0.4 S_{DS} I_p W_p \left(\frac{H_f}{R_{\mu}} \right) \left[\frac{C_{AR}}{R_{po}} \right] \quad (13.3-1)$$

$$F_{px} = \frac{C_{px}}{R_s} W_{px} \quad (12.10-4)$$



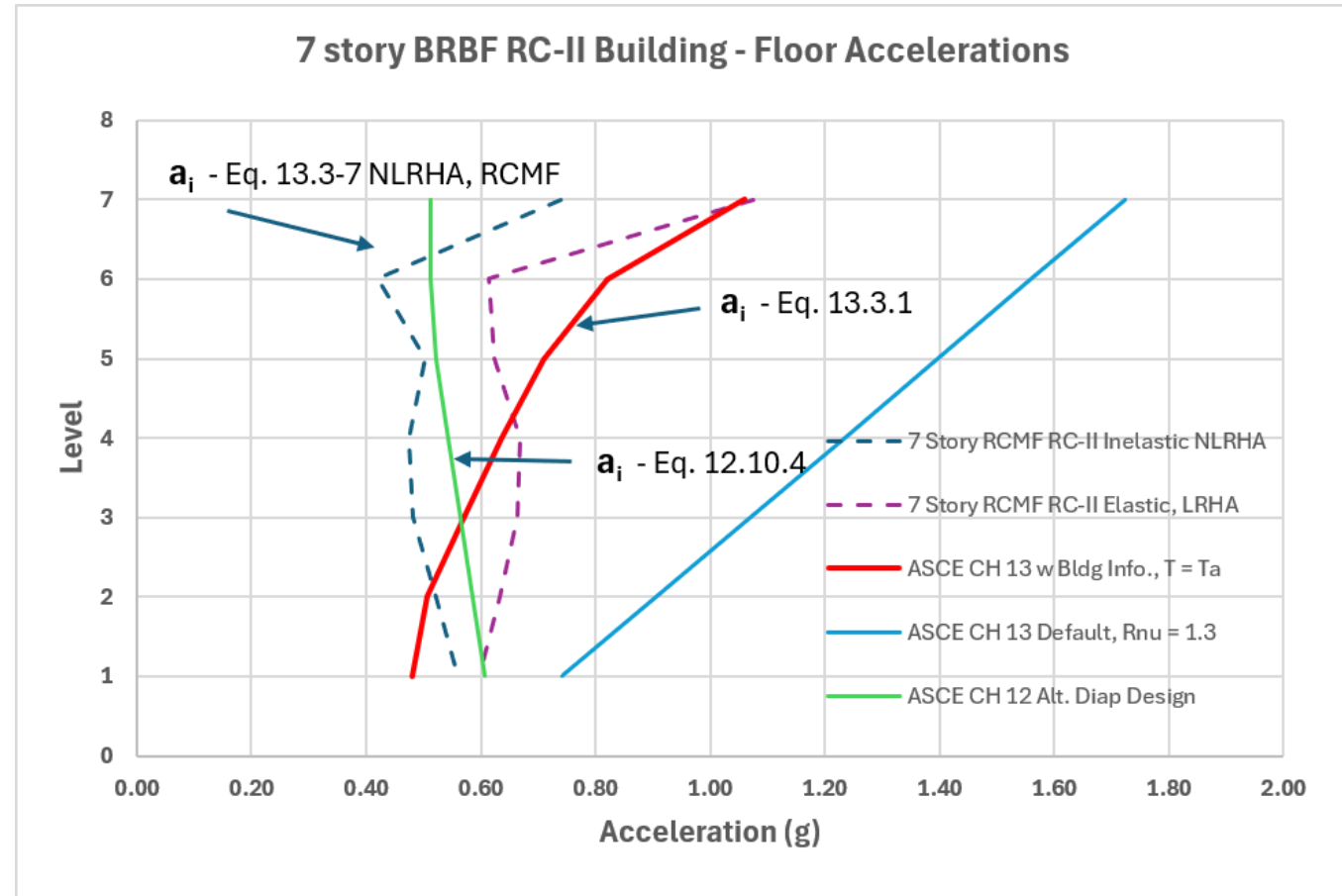
Floor accelerations a_i for various building types

- Steel Special Buckling Restrained Brace Frame, RC-II Bldgs

$$F_p = I_p W_p \left[\frac{C_{AR}}{R_{po}} \right] a_i \quad (13.3-7)$$

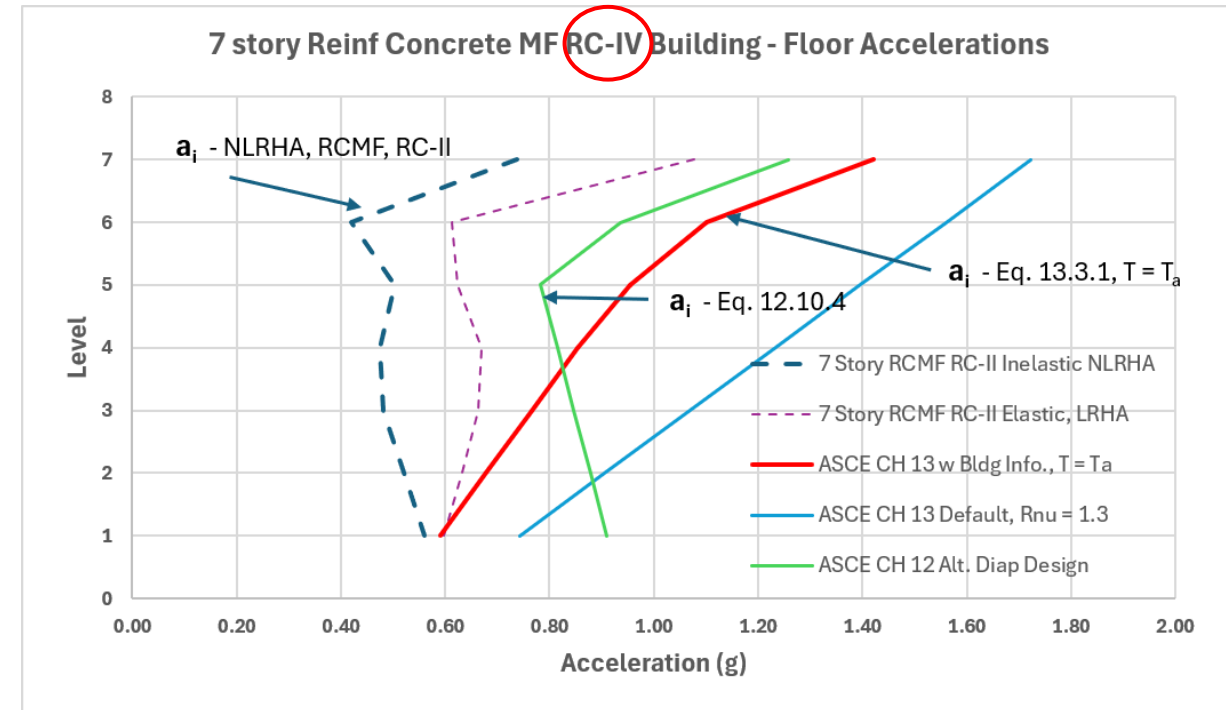
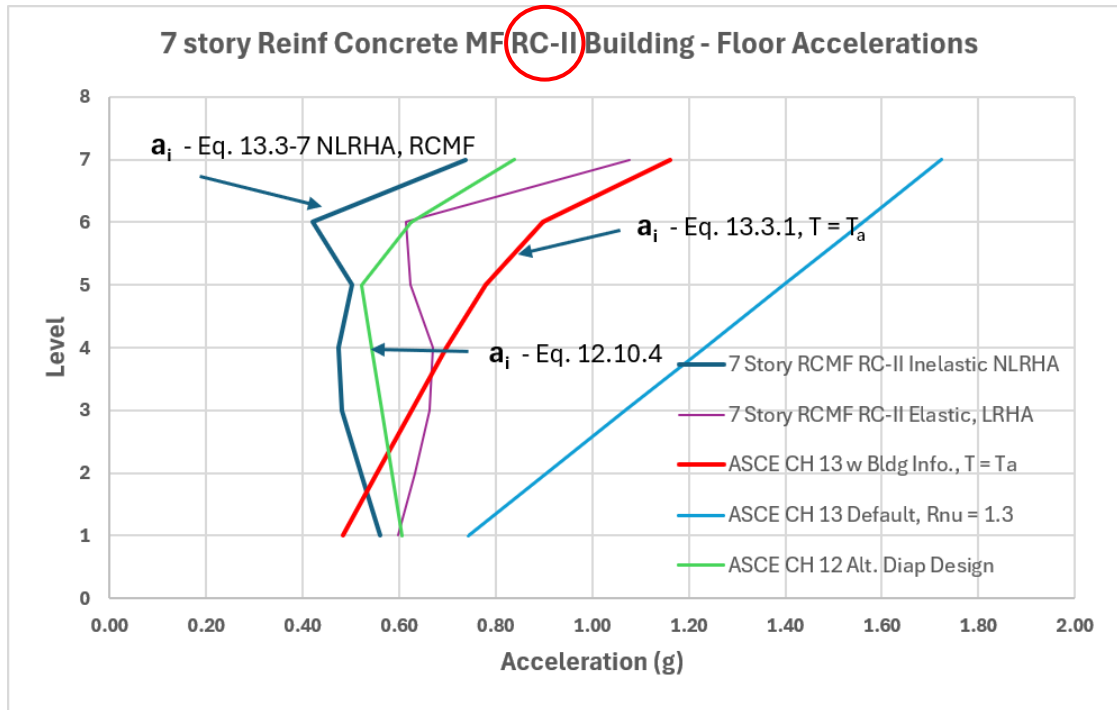
$$F_p = 0.4 S_{DS} I_p W_p \left[\frac{H_f}{R_\mu} \right] \left[\frac{C_{AR}}{R_{po}} \right] a_i \quad (13.3-1)$$

$$F_{px} = \frac{C_{px}}{R_s} w_{px} \quad (12.10-4)$$



Comparison between RC-II and RC-IV demands

- Reinforced Concrete Moment Frame, Comparison RC-II Vs RC-IV Bldg

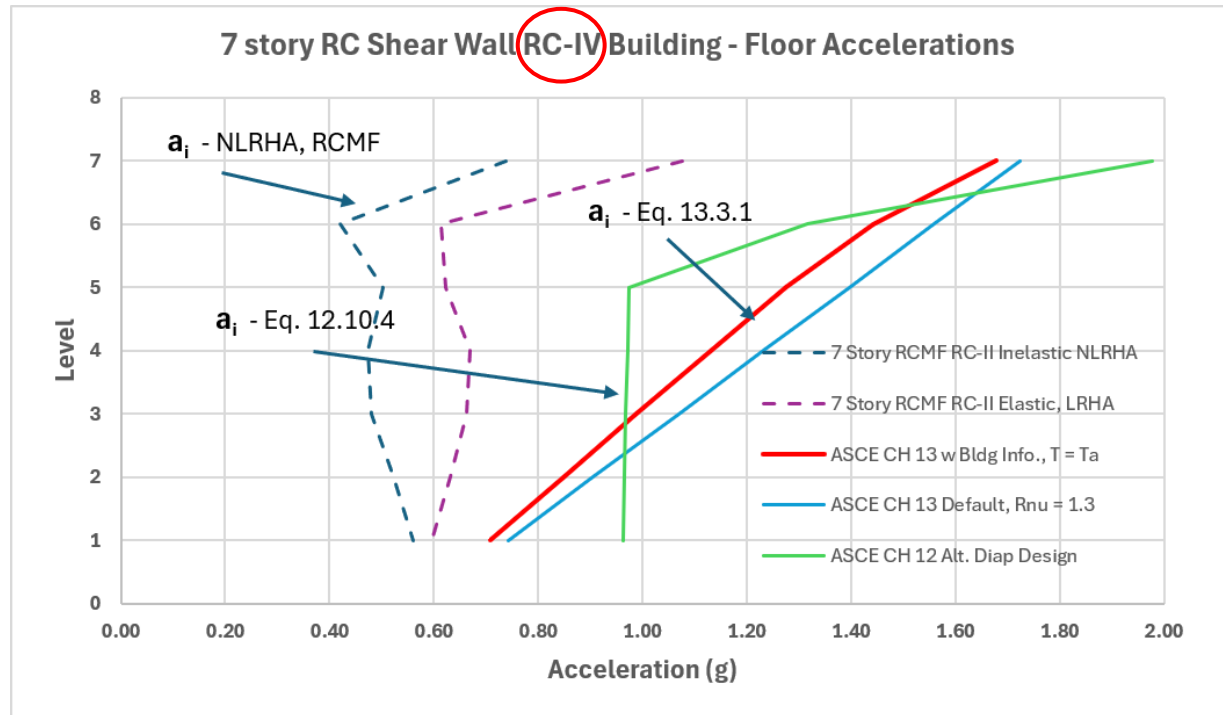
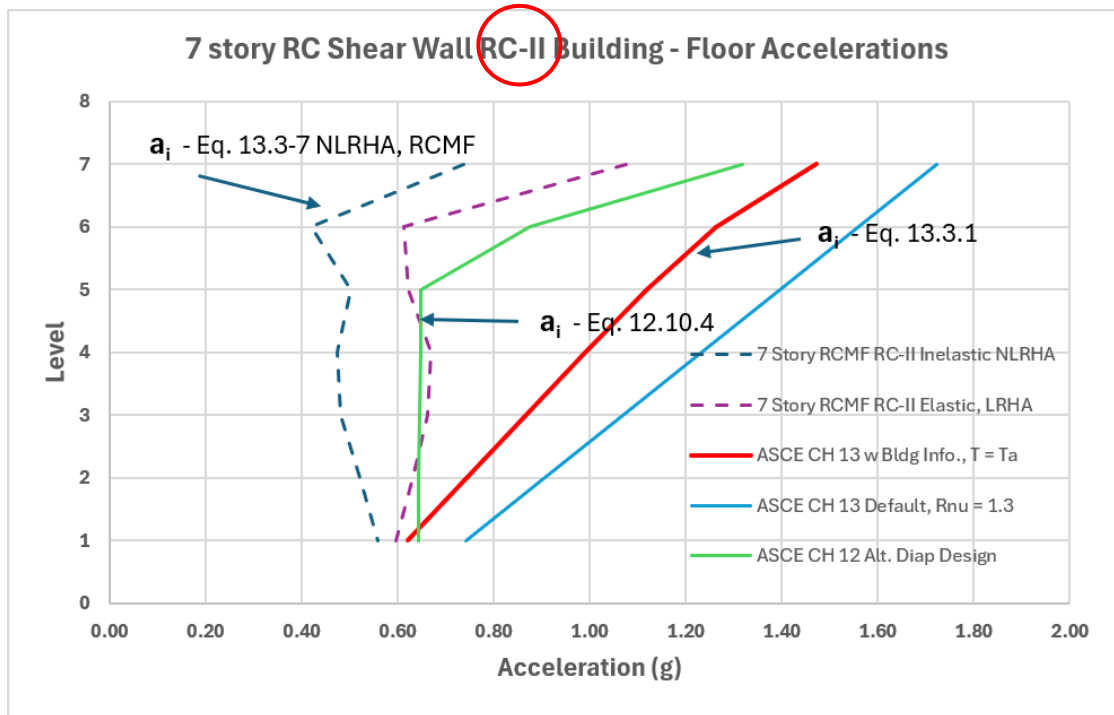


Why the increase in a_i for RC-IV buildings

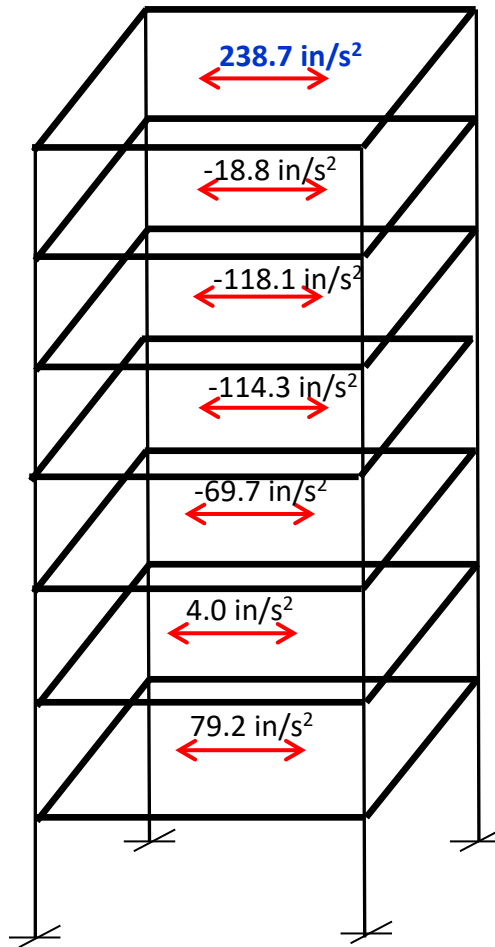
$$R_{\mu} = [1.1R/(I_e\Omega_0)]^{1/2} \geq 1.3 \quad (13.3-6)$$

Comparison between RC-II and RC-IV demands

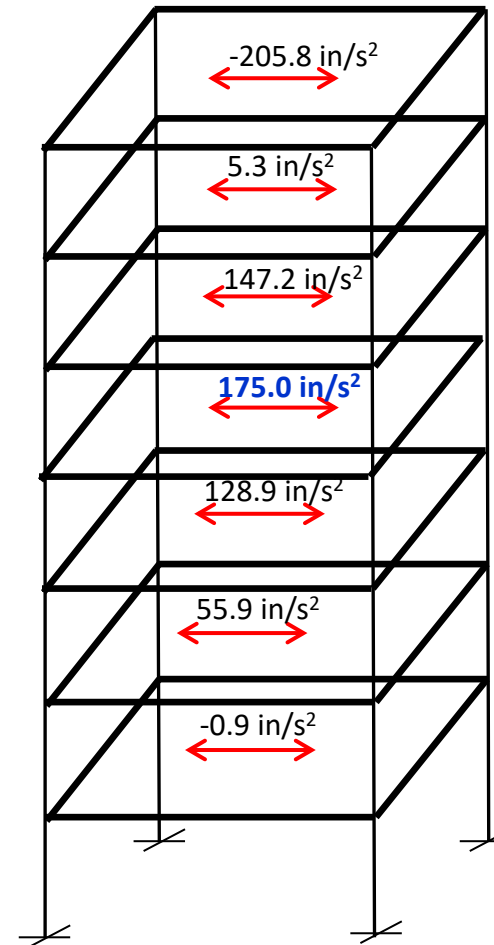
- Reinforced Concrete Shear Wall, Comparison RC-II Vs RC-IV Bldg



Instantaneous Floor Accelerations at a floor max



Floor accelerations at
Max. Roof Acceleration



Floor accelerations at Max.
4th floor Acceleration

Summary Takeaways

- Floor accelerations for RC-II buildings from the alternate diaphragm equation, Eq. 12.10-4 reasonably match the accelerations from a NLRHA for an RC-II building.
- Results from Eq. 13.3-1 appear conservative for RC-II buildings. Using a building period $C_u T_a$ could eliminate some of the conservatism.
- Using Eq. 12.10-4 for RC-IV building accelerations (a_i) double counts the acceleration if used for design of NSC as these are required to be further multiplied by I_p for design.

Areas of Conservatism in ASCE 7-22 Partition Demands

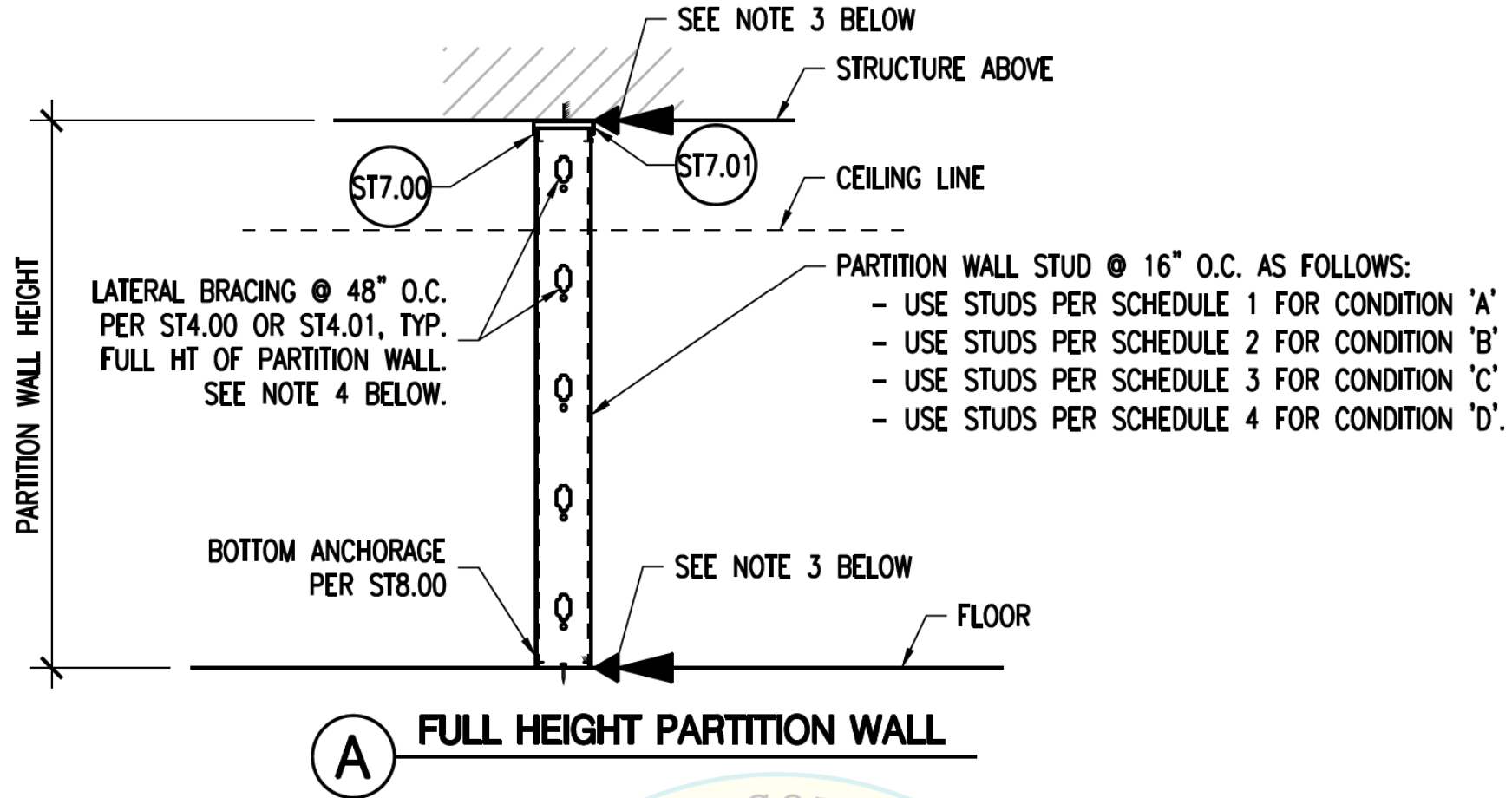
- Floor accelerations used for design of nonstructural components are inherently conservative even when building information is known.
- Results from Eq. 13.3-1 appear conservative for RC-II buildings. Using a building period $C_u T_a$ could eliminate some of the conservatism.
- Floor acceleration **maximums are not aligned** at each floor leading to **conservative** outcomes.
- Partition wall flexibility depends on cross wall spacing.
- Effect of gypsum board on both sides of the studs may not have been considered in the overstrength and component amplification factors.
- Cabinet hung on the wall also provides stiffness.

Comparison of ASCE 7-22 vs 7-16 for current OPDs

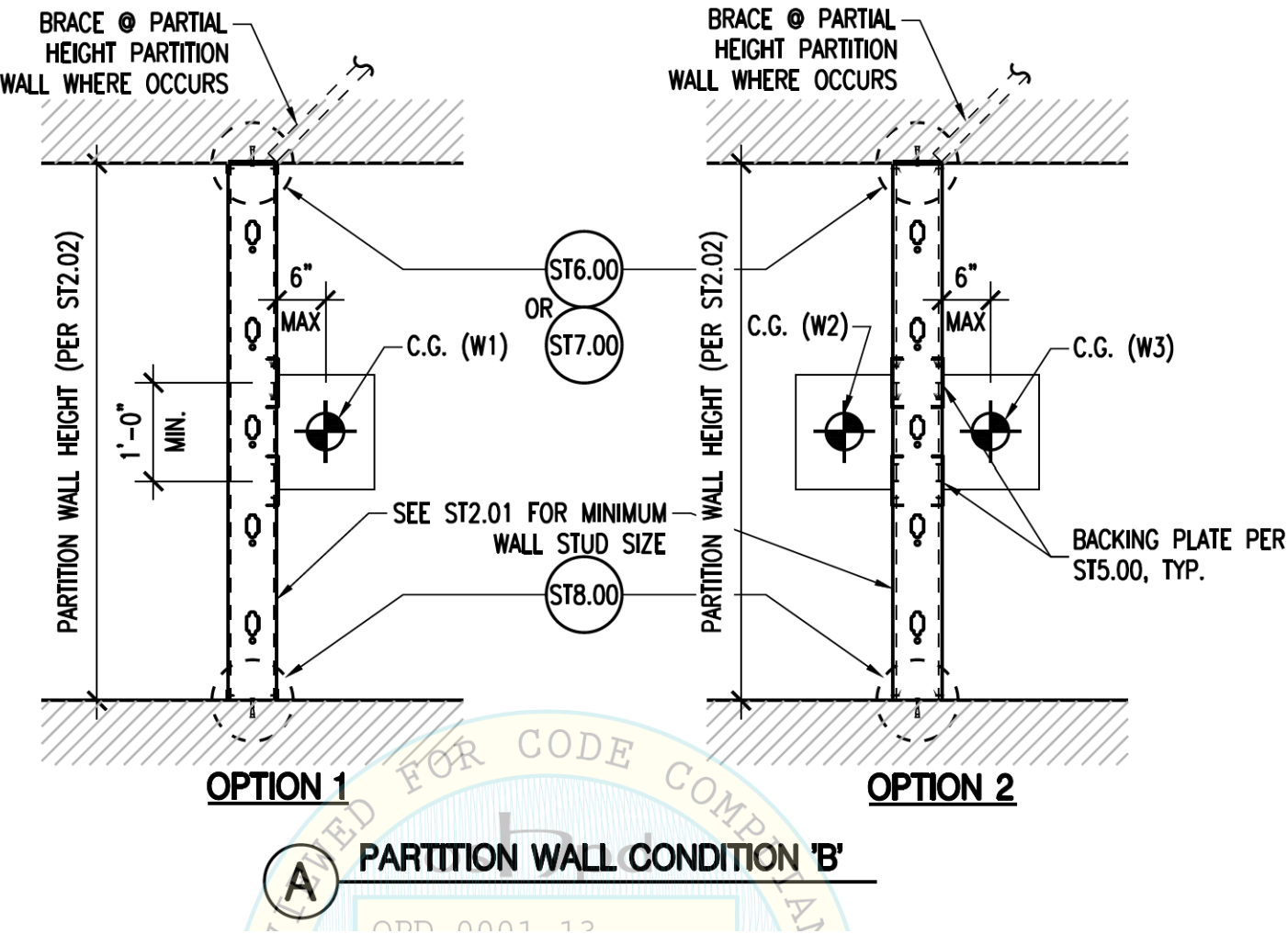
TYPICAL PARTITION WALL CONDITIONS

- CONDITION 'A' – PARTITION WALL WITHOUT ATTACHMENTS. SEE DETAIL ST2.02
- CONDITION 'B' – PARTITION WALL SUPPORTING CABINETS OR EQUIPMENT ON ONE SIDE OR BOTH SIDES OF THE WALL DISTRIBUTING UP TO 50 LB TOTAL VERTICAL LOAD PER STUD (37 PLF). REFER TO GENERAL NOTE 7C ON ST0.01. CENTER OF GRAVITY LESS THAN 6" FROM THE FACE OF THE STUD. SEE DETAIL ST2.03.
- CONDITION 'C' – PARTITION WALL SUPPORTING OVERHEAD AND/OR BASE CABINETS OR EQUIPMENT ON ONE SIDE OR BOTH SIDES OF THE WALL, DISTRIBUTING UP TO 152 LB TOTAL VERTICAL LOAD PER STUD (114 PLF). REFER TO GENERAL NOTE 7C ON ST0.01. CENTER OF GRAVITY WITHIN 6" OF FACE OF THE STUD. SEE DETAIL ST2.04.
- CONDITION 'D' – PARTITION WALL SUPPORTING FULL HEIGHT CABINETS OR EQUIPMENT ON ONE SIDE OR BOTH SIDES OF THE WALL, DISTRIBUTING UP TO 380 LB TOTAL VERTICAL LOAD PER STUD (285 PLF). REFER TO GENERAL NOTE 7C ON ST0.01. CENTER OF GRAVITY WITHIN 6" OF FACE OF THE STUD. SEE DETAIL ST2.04.

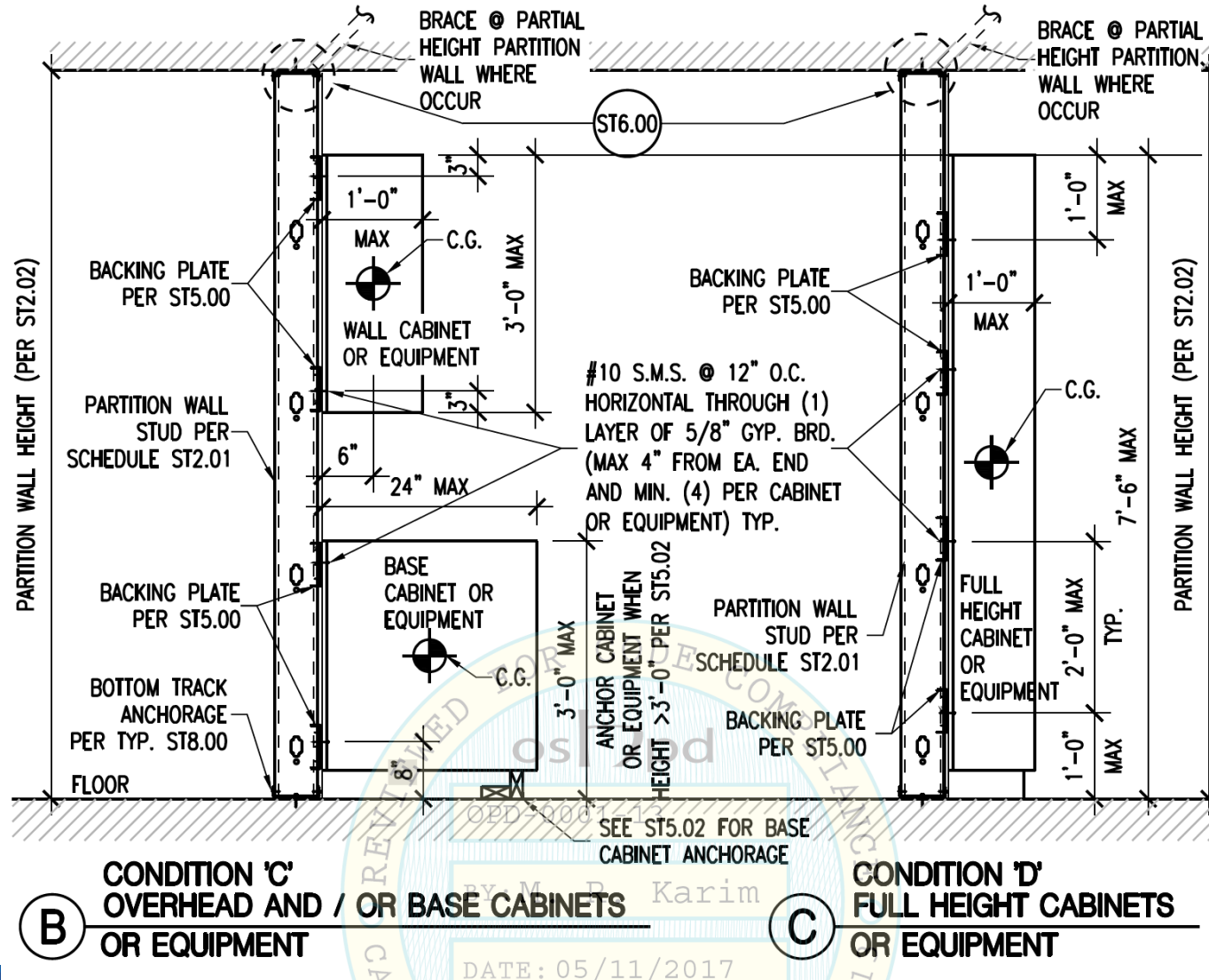
Condition A – 0 lbs Vertical load per stud



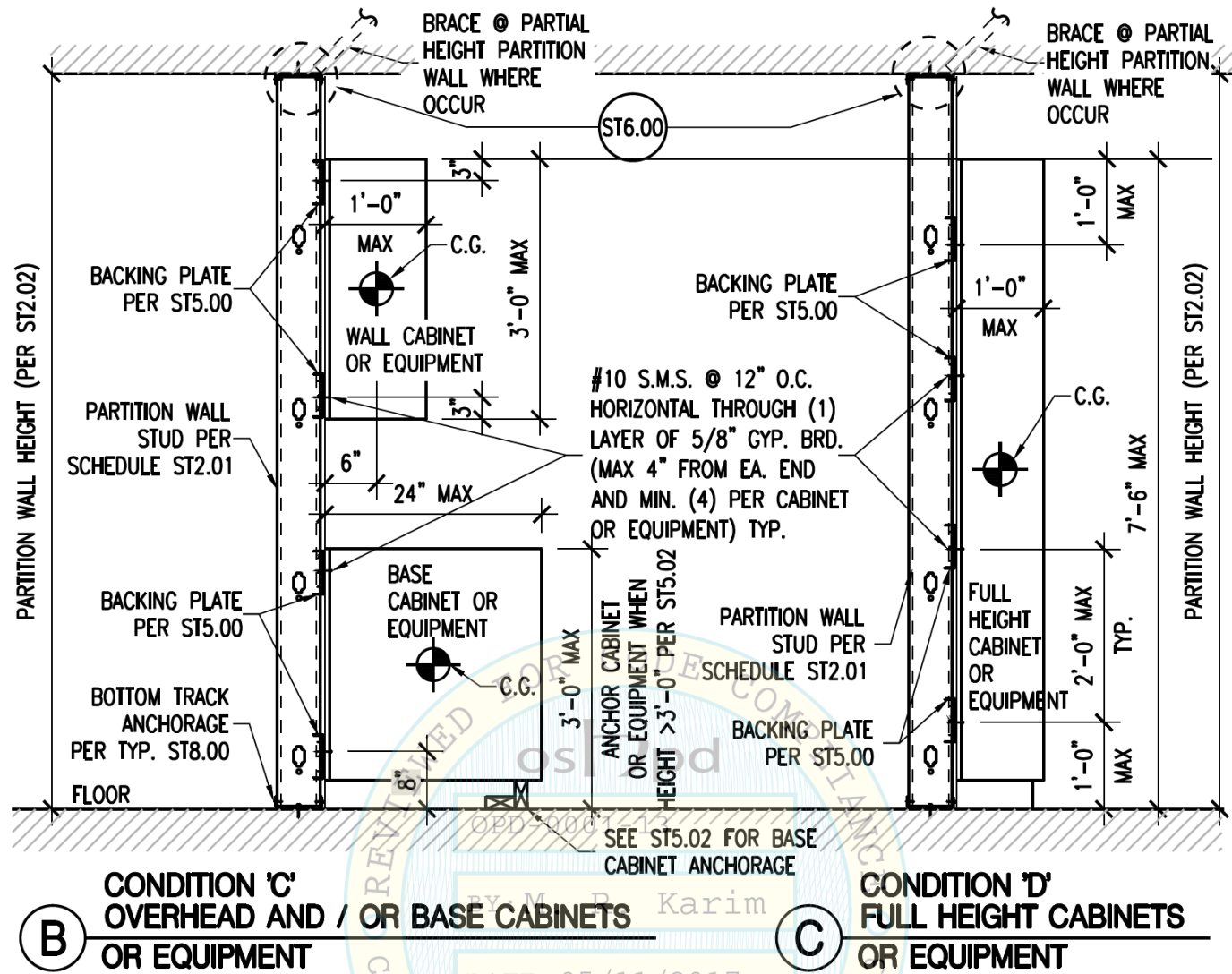
Condition B – 50 lbs Vertical Load per stud



Condition C – 152 lbs Vertical Load per stud



Condition D – 380 lbs Vertical Load per stud



DATE: 05/11/2017

ASCE 7-22 Force Level

Out-of-plane forces on partitions in ASCE 7-22 have increased by as much as 2.1 times the forces from ASCE 7-16.

Increased Stud thicknesses and depths

Light Frame Partitions <= 9 ft

ASCE 7-22				ASCE 7-16
S_{ds}	$F_{p,min}/W_p$	F_p/W_p	$F_{p,max}/W_p$	F_p/W_p
0.99	0.45	1.07	2.38	0.71
1.25	0.56	1.35	3.00	0.90
1.45	0.65	1.56	3.48	1.04
1.95	0.88	2.10	4.68	1.40

Light Frame Partitions > 9 ft

ASCE 7-22				ASCE 7-16
S_{ds}	$F_{p,min}/W_p$	F_p/W_p	$F_{p,max}/W_p$	F_p/W_p
0.99	0.45	1.49	2.38	0.71
1.25	0.56	1.88	3.00	0.90
1.45	0.65	2.19	3.48	1.04
1.95	0.88	2.94	4.68	1.40

↑ 1.5

↑ 2.1

New Stud Table for ASCE 7-22

SCHEDULE 1: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'A')

Fp/Wp Range	9 FT	12 FT	16 FT		
0.00 - > 0.75	362S162-33	362S162-33	362S162-33	400S162-33	600S162-33
0.76 - > 1.50	362S162-33	362S162-33	362S162-33	400S162-33	600S162-33
1.51 - > 2.25	362S162-33	362S162-33	362S200-33	400S162-33	600S162-33
2.26 - > 3.00	362S162-33	362S162-33	362S162-43	400S162-43	600S162-43

SCHEDULE 2: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'B')

Fp/Wp Range	9 FT	12 FT		16 FT		
0.00 - > 0.75	362S162-33	362S162-33	400S162-33	362S162-54	400S162-33	600S162-33
0.76 - > 1.50	362S162-33	362S162-33	400S162-33	362S162-54	400S162-43	600S162-33
1.51 - > 2.25	362S162-33	362S162-43	400S162-43	362S162-54	400S162-54	600S162-43
2.26 - > 3.00	362S162-43	362S162-54	400S200-43	362S200-54	400S162-54	600S162-43

SCHEDULE 3: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'C')

Fp/Wp Range	9 FT		12 FT			16 FT
0.00 - > 0.75	362S162-43	400S162-43	362S162-43	400S162-43	600S162-43	600S162-54
0.76 - > 1.50	362S162-54	400S162-54	362S162-54	400S162-54	600S162-54	600S162-54
1.51 - > 2.25	362S162-68	400S162-54	362S162-97	400S162-68	600S162-68	600S162-68
2.26 - > 3.00	362S250-97	400S162-68	N/A	400S200-97	600S162-68	600S162-68

SCHEDULE 4: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'D')

Fp/Wp Range	9 FT	12 FT	16 FT			
0.00 - > 0.75	362S162-33	400S162-33	600S162-43	362S162-43	400S162-43	600S162-43
0.76 - > 1.50	362S162-54	400S162-54	600S162-54	362S162-54	400S162-54	600S162-54
1.51 - > 2.25	362S162-54	400S162-54	600S162-54	362S200-68	400S162-68	600S162-54
2.26 - > 3.00	362S162-68	400S162-68	600S162-54	362S250-97	400S162-97	600S162-68

Partition Wall Stud Size Updates

SCHEDULE 3: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'C')

Fp/Wp Range	12 FT			16 FT
2.26 - > 3.00	362S137-118	400S137-118	600S137-68	600S137-97
1.46-1.95	NA	NA	600S137-54	600S137-54

2025 OPD 01

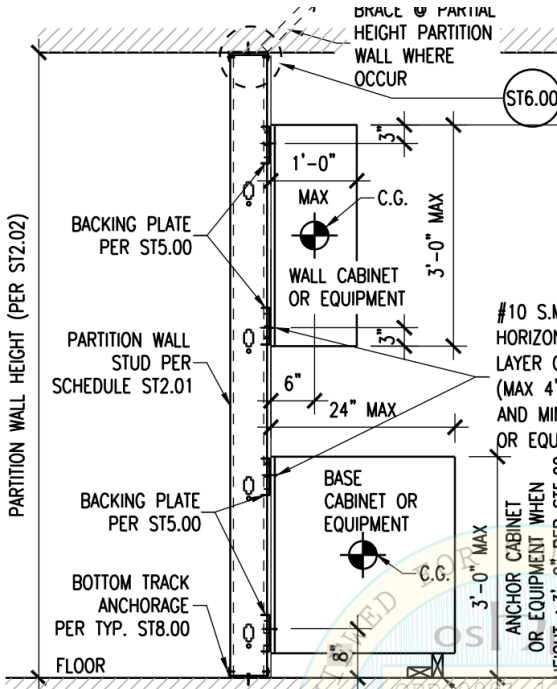
2013 OPD 01

25% ↑ 73% ↑
Weight Increase

Designation Thickness (mil)	Minimum Thickness ¹ (in)
43	0.0428
54	0.0538
68	0.0677
97	0.0966
118	0.1180

26% thicker than 54 mil
80% thicker than 54 mil

S _{ds}	ASCE 7-22 F _p /W _p Range	
	≤ 9 ft	> 9 ft
0.25-0.99	0.27-1.07	0.38-1.49
1.00-1.25	1.08-1.35	1.51-1.88
1.26-1.45	1.36-1.56	1.90-2.19
1.46-1.95	1.57-2.1	2.20-2.94

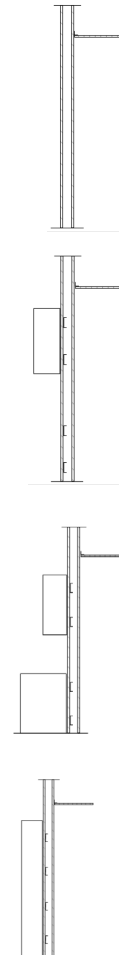


Assumes cabinet weight of 38 lbs per ft³

OPD Update: Stud Size Research

2013 OPD

- Limited flange size to 1-3/8" and maximum thickness of 16 gauge (54 mil).



PARTITION WALL SCHEDULES

SCHEDULE 1: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'A')

S _{ps}	WALL HEIGHT				
	9 FT	12 FT	16 FT		
0.25-0.99	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.00-1.25	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.26-1.45	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.46-1.95	362S137-33	362S137-33	362S137-54	400S137-54	600S137-43

SCHEDULE 2: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'B')

S _{ps}	WALL HEIGHT					
	9 FT	12 FT		16 FT		
0.25-0.99	362S137-33	362S137-43	400S137-43	362S137-54	400S137-43	600S137-43
1.00-1.25	362S137-33	362S137-43	400S137-43	NA	400S137-54	600S137-43
1.26-1.45	362S137-33	362S137-43	400S137-43	NA	400S137-54	600S137-43
1.46-1.95	362S137-33	362S137-54	400S137-43	NA	400S137-54	600S137-43

SCHEDULE 3: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'C')

S _{ps}	WALL HEIGHT					
	9 FT	12 FT			16 FT	
0.25-0.99	362S137-43	400S137-54	362S137-54	400S137-43	600S137-43	600S137-43
1.00-1.25	362S137-43	400S137-54	362S137-54	400S137-54	600S137-43	600S137-54
1.26-1.45	362S137-43	400S137-54	NA	400S137-54	600S137-54	600S137-54
1.46-1.95	362S137-54	400S137-54	NA	NA	600S137-54	600S137-54

SCHEDULE 4: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'D')

S _{ps}	WALL HEIGHT						
	9 FT	12 FT		16 FT			
0.25-0.99	362S137-33	400S137-43	600S137-43	362S137-43	400S137-43	600S137-43	600S137-43
1.00-1.25	362S137-43	400S137-43	600S137-43	NA	400S137-54	600S137-43	600S137-43
1.26-1.45	362S137-43	400S137-43	600S137-43	NA	400S137-54	600S137-43	600S137-54
1.46-1.95	NA	400S137-54	600S137-54	NA	NA	600S137-54	600S137-54

OPD Update: Stud Size Research

Research:

- Kirsten Zeydel reached out to industry partners/manufacturers to get data on common sizes and the impact on stud cost with increasing flange size and thickness.
- 4" members:
 - 1-5/8", 2", and 2-1/2" flange studs
 - Price is comparable to the area of steel
 - 3" flange studs
 - ~15% premium versus smaller flange sizes
 - 3-1/2" flange studs
 - ~25% premium versus smaller flange sizes

PARTITION WALL SCHEDULES

SCHEDULE 1: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'A')

S _{ps}	WALL HEIGHT				
	9 FT	12 FT	16 FT		
0.25-0.99	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.00-1.25	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.26-1.45	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.46-1.95	362S137-33	362S137-33	362S137-54	400S137-54	600S137-43

SCHEDULE 2: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'B')

S _{ps}	WALL HEIGHT					
	9 FT	12 FT		16 FT		
0.25-0.99	362S137-33	362S137-43	400S137-43	362S137-54	400S137-43	600S137-43
1.00-1.25	362S137-33	362S137-43	400S137-43	NA	400S137-54	600S137-43
1.26-1.45	362S137-33	362S137-43	400S137-43	NA	400S137-54	600S137-43
1.46-1.95	362S137-33	362S137-54	400S137-43	NA	400S137-54	600S137-43

SCHEDULE 3: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'C')

S _{ps}	WALL HEIGHT					
	9 FT	12 FT		16 FT		
0.25-0.99	362S137-43	400S137-54	362S137-54	400S137-43	600S137-43	600S137-43
1.00-1.25	362S137-43	400S137-54	362S137-54	400S137-54	600S137-43	600S137-54
1.26-1.45	362S137-43	400S137-54	NA	400S137-54	600S137-54	600S137-54
1.46-1.95	362S137-54	400S137-54	NA	NA	600S137-54	600S137-54

SCHEDULE 4: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'D')

S _{ps}	WALL HEIGHT						
	9 FT			12 FT		16 FT	
0.25-0.99	362S137-33	400S137-43	600S137-43	362S137-43	400S137-43	600S137-43	600S137-43
1.00-1.25	362S137-43	400S137-43	600S137-43	NA	400S137-54	600S137-43	600S137-43
1.26-1.45	362S137-43	400S137-43	600S137-43	NA	400S137-54	600S137-43	600S137-54
1.46-1.95	NA	400S137-54	600S137-54	NA	NA	600S137-54	600S137-54

OPD Update: Stud Size Research

Research:

- 6" members:
 - 1-5/8", 2", 2-1/2", and 3" flange studs
 - Price is comparable to the area of steel
 - 3-1/2" flange studs
 - ~25% premium versus smaller flange sizes
- Thickness and Installation Cost:
 - 43mil (18ga) and 54mil (16ga) framing similar installation time/cost
 - 68mil (14ga) framing has higher installation time/cost
 - 97mil (12ga) framing significantly higher installation time/cost. Most recommended that 12ga should be avoided.

PARTITION WALL SCHEDULES

SCHEDULE 1: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'A')

S _{ps}	WALL HEIGHT				
	9 FT	12 FT	16 FT		
0.25-0.99	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.00-1.25	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.26-1.45	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.46-1.95	362S137-33	362S137-33	362S137-54	400S137-54	600S137-43

SCHEDULE 2: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'B')

WALL HEIGHT						
S _{ps}	9 FT	12 FT		16 FT		
0.25-0.99	362S137-33	362S137-43	400S137-43	362S137-54	400S137-43	600S137-43
1.00-1.25	362S137-33	362S137-43	400S137-43	NA	400S137-54	600S137-43
1.26-1.45	362S137-33	362S137-43	400S137-43	NA	400S137-54	600S137-43
1.46-1.95	362S137-33	362S137-54	400S137-43	NA	400S137-54	600S137-43

SCHEDULE 3: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'C')

S _{ps}	WALL HEIGHT					
	9 FT		12 FT			16 FT
0.25-0.99	362S137-43	400S137-54	362S137-54	400S137-43	600S137-43	600S137-43
1.00-1.25	362S137-43	400S137-54	362S137-54	400S137-54	600S137-43	600S137-54
1.26-1.45	362S137-43	400S137-54	NA	400S137-54	600S137-54	600S137-54
1.46-1.95	362S137-54	400S137-54	NA	NA	600S137-54	600S137-54

SCHEDULE 4: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'D')

WALL HEIGHT							
S _{ps}	9 FT			12 FT		16 FT	
0.25-0.99	362S137-33	400S137-43	600S137-43	362S137-43	400S137-43	600S137-43	600S137-43
1.00-1.25	362S137-43	400S137-43	600S137-43	NA	400S137-54	600S137-43	600S137-43
1.26-1.45	362S137-43	400S137-43	600S137-43	NA	400S137-54	600S137-43	600S137-54
1.46-1.95	NA	400S137-54	600S137-54	NA	NA	600S137-54	600S137-54

OPD Update: Stud Size Research

- Stud size is now selected based on area of steel
 - For example, in-lieu of going to a thicker gauge, a larger flange size may be selected.
- Multiple flange sizes are included.

SCHEDULE 1: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'A')

Fp/Wp Range	9 FT	12 FT	16 FT		
0.00 - > 0.75	362S162-33	362S162-33	362S162-33	400S162-33	600S162-33
0.76 - > 1.50	362S162-33	362S162-33	362S162-33	400S162-33	600S162-33
1.51 - > 2.25	362S162-33	362S162-33	362S200-33	400S162-33	600S162-33
2.26 - > 3.00	362S162-33	362S162-33	362S162-43	400S162-43	600S162-43

SCHEDULE 2: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'B')

Fp/Wp Range	9 FT	12 FT		16 FT		
0.00 - > 0.75	362S162-33	362S162-33	400S162-33	362S162-54	400S162-33	600S162-33
0.76 - > 1.50	362S162-33	362S162-33	400S162-33	362S162-54	400S162-43	600S162-33
1.51 - > 2.25	362S162-33	362S162-43	400S162-43	362S162-54	400S162-54	600S162-43
2.26 - > 3.00	362S162-43	362S162-54	400S200-43	362S200-54	400S162-54	600S162-43

SCHEDULE 3: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'C')

Fp/Wp Range	9 FT		12 FT			16 FT
0.00 - > 0.75	362S162-43	400S162-43	362S162-43	400S162-43	600S162-43	600S162-54
0.76 - > 1.50	362S162-54	400S162-54	362S162-54	400S162-54	600S162-54	600S162-54
1.51 - > 2.25	362S162-68	400S162-54	362S162-97	400S162-68	600S162-68	600S162-68
2.26 - > 3.00	362S250-97	400S162-68	N/A	400S200-97	600S162-68	600S162-68

SCHEDULE 4: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'D')

Fp/Wp Range	9 FT			12 FT			16 FT
0.00 - > 0.75	362S162-33	400S162-33	600S162-43	362S162-43	400S162-43	600S162-43	600S162-43
0.76 - > 1.50	362S162-54	400S162-54	600S162-54	362S162-54	400S162-54	600S162-54	600S162-54
1.51 - > 2.25	362S162-54	400S162-54	600S162-54	362S200-68	400S162-68	600S162-54	600S162-54
2.26 - > 3.00	362S162-68	400S162-68	600S162-54	362S250-97	400S162-97	600S162-68	600S162-68

Stud Schedule Comparison

OPD Table – ASCE 7-22

SCHEDULE 1: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'A')

Fp/Wp Range	9 FT	12 FT	16 FT		
0.00 - > 0.75	362S162-33	362S162-33	362S162-33	400S162-33	600S162-33
0.76 - > 1.50	362S162-33	362S162-33	362S162-33	400S162-33	600S162-33
1.51 - > 2.25	362S162-33	362S162-33	362S200-33	400S162-33	600S162-33
2.26 - > 3.00	362S162-33	362S162-33	362S162-43	400S162-43	600S162-43

SCHEDULE 2: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'B')

Fp/Wp Range	9 FT	12 FT		16 FT		
0.00 - > 0.75	362S162-33	362S162-33	400S162-33	362S162-54	400S162-33	600S162-33
0.76 - > 1.50	362S162-33	362S162-33	400S162-33	362S162-54	400S162-43	600S162-33
1.51 - > 2.25	362S162-33	362S162-43	400S162-43	362S162-54	400S162-54	600S162-43
2.26 - > 3.00	362S162-43	362S162-54	400S200-43	362S200-54	400S162-54	600S162-43

SCHEDULE 3: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'C')

Fp/Wp Range	9 FT		12 FT			16 FT
0.00 - > 0.75	362S162-43	400S162-43	362S162-43	400S162-43	600S162-43	600S162-54
0.76 - > 1.50	362S162-54	400S162-54	362S162-54	400S162-54	600S162-54	600S162-54
1.51 - > 2.25	362S162-68	400S162-54	362S162-97	400S162-68	600S162-68	600S162-68
2.26 - > 3.00	362S250-97	400S162-68	N/A	400S200-97	600S162-68	600S162-68

SCHEDULE 4: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'D')

Fp/Wp Range	9 FT			12 FT			16 FT
0.00 - > 0.75	362S162-33	400S162-33	600S162-43	362S162-43	400S162-43	600S162-43	600S162-43
0.76 - > 1.50	362S162-54	400S162-54	600S162-54	362S162-54	400S162-54	600S162-54	600S162-54
1.51 - > 2.25	362S162-54	400S162-54	600S162-54	362S200-68	400S162-68	600S162-54	600S162-54
2.26 - > 3.00	362S162-68	400S162-68	600S162-54	362S250-97	400S162-97	600S162-68	600S162-68

Original OPD Table

SCHEDULE 1: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'A')

Sds	Fp/Wp Range	9 FT	12 FT	16 FT		
0.00-> > 0.99	0.00 - > 0.71	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.00-> > 1.25	0.72 - > 0.90	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.26-> > 1.45	0.91 - > 1.04	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.46-> > 1.95	1.05 - > 1.40	362S137-33	362S137-33	362S137-54	400S137-54	600S137-43

SCHEDULE 2: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'B')

Sds	Fp/Wp Range	9 FT	12 FT		16 FT		
0.00-> > 0.99	0.00 - > 0.71	362S137-33	362S137-43	400S137-43	362S137-54	400S137-43	600S137-43
1.00-> > 1.25	0.72 - > 0.90	362S137-33	362S137-43	400S137-43	N/A	400S137-54	600S137-43
1.26-> > 1.45	0.91 - > 1.04	362S137-33	362S137-43	400S137-43	N/A	400S137-54	600S137-43
1.46-> > 1.95	1.05 - > 1.40	362S137-33	362S137-54	400S137-43	N/A	400S137-54	600S137-43

SCHEDULE 3: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'C')

Sds	Fp/Wp Range	9 FT		12 FT			16 FT
0.00-> > 0.99	0.00 - > 0.71	362S137-43	400S137-54	362S137-54	400S137-43	600S137-43	600S137-43
1.00-> > 1.25	0.72 - > 0.90	362S137-43	400S137-54	362S137-54	400S137-54	600S137-43	600S137-54
1.26-> > 1.45	0.91 - > 1.04	362S137-43	400S137-54	N/A	400S137-54	600S137-54	600S137-54
1.46-> > 1.95	1.05 - > 1.40	362S137-54	400S137-54	N/A	N/A	600S137-54	600S137-54

SCHEDULE 4: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'D')

Sds	Fp/Wp Range	9 FT			12 FT			16 FT
0.00-> > 0.99	0.00 - > 0.71	362S137-33	400S137-43	600S137-43	600S137-43	400S137-43	600S137-43	600S137-43
1.00-> > 1.25	0.72 - > 0.90	362S137-43	400S137-43	600S137-43	N/A	400S137-54	600S137-43	600S137-43
1.26-> > 1.45	0.91 - > 1.04	362S137-43	400S137-43	600S137-43	N/A	400S137-54	600S137-43	600S137-54
1.46-> > 1.95	1.05 - > 1.40	N/A	400S137-54	600S137-54	N/A	N/A	600S137-54	600S137-54

Anchor Updates

Bottom Connection

Deeper embedment and larger diameter anchor options

2013 OPD 01

BOTTOM CONNECTION DEMANDS (PARTITION WALL CONDITIONS 'C' & 'D')				
MAX PARTITION WALL HEIGHT	S _{DS}	MAXIMUM FASTENER SPACING IN INCHES AT BOT		
		PAF	3/8" Ø EXP ANCHOR W/ 2" EMBED	1/2" Ø EXP ANCHOR W/ 2 1/4" EMBED
9 FT	0.25-0.99	-	32	32
	1.00-1.25	-	32	32
	1.26-1.45	-	32	32
	1.46-1.95	-	24	32
12 FT	0.25-0.99	-	32	32
	1.00-1.25	-	32	32
	1.26-1.45	-	24	32
	1.46-1.95	-	16	24
16 FT	0.25-0.99	-	32	32
	1.00-1.25	-	32	32
	1.26-1.45	-	24	32
	1.46-1.95	-	16	24

2025 OPD 01

OPD CONDITION C

MAX PARTITION WALL HEIGHT	F _p /W _p RANGE	MAXIMUM FASTENER SPACING IN INCHES AT BOTTOM CONNECTION		
		3/8" Ø EXP. ANCHOR W/ 2" EMBED	1/2" Ø EXP. ANCHOR W/ 3-1/4" EMBED	5/8" Ø EXP. ANCHOR W/ 2-3/4" EMBED
9 FT	0.00 -> 0.75	32	32	32
	0.76 -> 1.50	24	32	32
	1.51 -> 2.25	16	32	32
	2.26 -> 3.00	8	24	32
12 FT	0.00 -> 0.75	32	32	32
	0.76 -> 1.50	24	32	32
	1.51 -> 2.25	16	32	32
	2.26 -> 3.00	8	24	32
16 FT	0.00 -> 0.75	32	32	32
	0.76 -> 1.50	24	32	32
	1.51 -> 2.25	16	32	32
	2.26 -> 3.00	8	24	32

Spacing reduced

Anchor Updates

Top Connection

2013 OPD tables for ST7.02A. If using ST7.02B then double spacing.

PARTITION WALL CONDITION 'C'

S _{ps} / WALL HEIGHT	PAF SPACING (INCHES O.C.) PER ST7.02 DETAIL A		
	9 FT	12 FT	16 FT
0.25-0.99	6	6	6
1.00-1.25	6	6	6
1.26-1.45	6	6	6
1.46-1.95	6	6	6

FOR DETAIL B THROUGH D (STRAP OR Z-CLIP) DOUBLE THE SPACING INDICATED UP TO 48" OC.

PARTITION WALL CONDITION D'

S _{ps} / WALL HEIGHT	PAF SPACING (INCHES O.C.) PER ST7.02 DETAIL A		
	9 FT	12 FT	16 FT
0.25-0.99	6	6	6
1.00-1.25	6	6	6
1.26-1.45	6	6	6
1.46-1.95	6	6	6

FOR DETAIL B THROUGH D (STRAP OR Z-CLIP) DOUBLE THE SPACING INDICATED UP TO 48" OC.

2025 OPD tables

OPD CONDITION C & D COMBINED				
MAX PARTITION WALL HEIGHT	F _p /W _p RANGE	MAXIMUM PAF SPACING IN INCHES		
		BOTTOM CONNECTION PER ST8.01	TOP CONNECTION PER ST7.02A	TOP CONNECTION PER ST7.02B, ST7.02C-D
9 FT	0.00 -> 0.75	NA	6	12
	0.76 -> 1.50	NA	NA	6
	1.51 -> 2.25	NA	NA	NA
	2.26 -> 3.00	NA	NA	NA
12 FT	0.00 -> 0.75	NA	6	12
	0.76 -> 1.50	NA	NA	6
	1.51 -> 2.25	NA	NA	NA
	2.26 -> 3.00	NA	NA	NA
16 FT	0.00 -> 0.75	NA	6	12
	0.76 -> 1.50	NA	NA	6
	1.51 -> 2.25	NA	NA	NA
	2.26 -> 3.00	NA	NA	NA

ASCE 7-22 Equivalent (S_{DS})

- To determine an equivalent S_{DS} (to allow use of 2013 OPD's), the F_p/W_p is calculated for the ASCE 7-22 force level, and then the ASCE 7-16 F_p equation is used to solve for S_{DS} .
- This equivalent S_{DS} can then be used to determine partition wall designs/anchorage using the 2013 OPD's, where applicable.

ASCE 7-22

ASCE 7-16

$$F_p = 0.4S_{DS}I_pW_p \left[\frac{H_f}{R_\mu} \right] \left[\frac{C_{AR}}{R_{po}} \right] \longrightarrow F_p = \frac{0.4a_pS_{DS}W_p}{\left(\frac{R_p}{I_p} \right)} \left(1 + 2\frac{z}{h} \right)$$

ASCE 7-22 Equivalent S_{DS} : Partition Height ≤ 9 ft

Light Frame Partitions ≤ 9 ft

ASCE 7-16		ASCE 7-22						
		ASCE 7-22 S_{DS} / OPD S_{DS} (%)						
S_{DS}	z/h	Default	SMRF-3	SMRF-10	SRCSW-BF-3	SRCSW-BF-10	SRCSW-BW-3	SRCSW-BW-10
0.25	0.5	69%	93%	128%	71%	102%	69%	100%
	1.0	67%	79%	99%	68%	82%	67%	80%
0.99	0.5	69%	93%	128%	71%	102%	69%	100%
	1.0	67%	79%	99%	68%	82%	67%	80%
1	0.5	69%	93%	128%	71%	102%	69%	100%
	1.0	67%	79%	99%	68%	82%	67%	80%
1.25	0.5	69%	93%	128%	71%	102%	69%	100%
	1.0	67%	79%	99%	68%	82%	67%	80%
1.26	0.5	69%	93%	128%	71%	102%	69%	100%
	1.0	67%	79%	99%	68%	82%	67%	80%
1.45	0.5	69%	93%	128%	71%	102%	69%	100%
	1.0	67%	79%	99%	68%	82%	67%	80%
1.46	0.5	69%	93%	128%	71%	102%	69%	100%
	1.0	67%	79%	99%	68%	82%	67%	80%
1.95	0.5	69%	93%	128%	71%	102%	69%	100%
	1.0	67%	79%	99%	68%	82%	67%	80%

SRCSW-BW-3

This is the ASCE 7-22/ASCE 7-16 S_{DS} ratio.

Ex: For $z/h \leq 0.5$, an ASCE 7-22 site with S_{DS} of $0.69 \times 1.25 = 0.86g$ can use the details for 1.25g.

ASCE 7-16

$S_{DS} = 1.25$

ASCE 7-22

$S_{DS} = 0.86$

Default: Worst Case Lateral Force Resisting System and Building Height

SMRF-3/10: 3/10 Story Steel Special Moment Resisting Frame

SRCSW-BF-3/10: 3/10 Story Special Reinforced Concrete Shear Wall (Bearing Frame)

SRCSW-BW-3/10: 3/10 Story Special Reinforced Concrete Shear Wall (Bearing Wall)

ASCE 7-22 Equivalent S_{DS} : Partition Height > 9ft

Light Frame Partitions > 9 ft

ASCE 7-16		ASCE 7-22						
		ASCE 7-22 S_{DS} / OPD S_{DS} (%)						
S_{ds}	z/h	Default	SMRF-3	SMRF-10	SRCSW-BF-3	SRCSW-BF-10	SRCSW-BW-3	SRCSW-BW-10
0.25	0.5	50%	66%	92%	51%	73%	50%	72%
	1.0	48%	56%	70%	49%	59%	48%	57%
0.99	0.5	50%	66%	92%	51%	73%	50%	72%
	1.0	48%	56%	70%	49%	59%	48%	57%
1	0.5	50%	66%	92%	51%	73%	50%	72%
	1.0	48%	56%	70%	49%	59%	48%	57%
1.25	0.5	50%	66%	92%	51%	73%	50%	72%
	1.0	48%	56%	70%	49%	59%	48%	57%
1.26	0.5	50%	66%	92%	51%	73%	50%	72%
	1.0	48%	56%	70%	49%	59%	48%	57%
1.45	0.5	50%	66%	92%	51%	73%	50%	72%
	1.0	48%	56%	70%	49%	59%	48%	57%
1.46	0.5	50%	66%	92%	51%	73%	50%	72%
	1.0	48%	56%	70%	49%	59%	48%	57%
1.95	0.5	50%	66%	92%	51%	73%	50%	72%
	1.0	48%	56%	70%	49%	59%	48%	57%

SRCSW-BW-3

S_{DS} ratio to convert 2013 OPD S_{DS} to an equivalent ASCE 7-22 S_{DS}

Ex: For $z/h \leq 0.5$, an ASCE 7-22 site with S_{DS} of $0.5 \times 1.25 = 0.63g$ can use the details for 1.25g.

ASCE 7-16

$S_{DS} = 1.25$

ASCE 7-22

$S_{DS} = 0.63$

Default: Worst Case Lateral Force Resisting System and Building Height

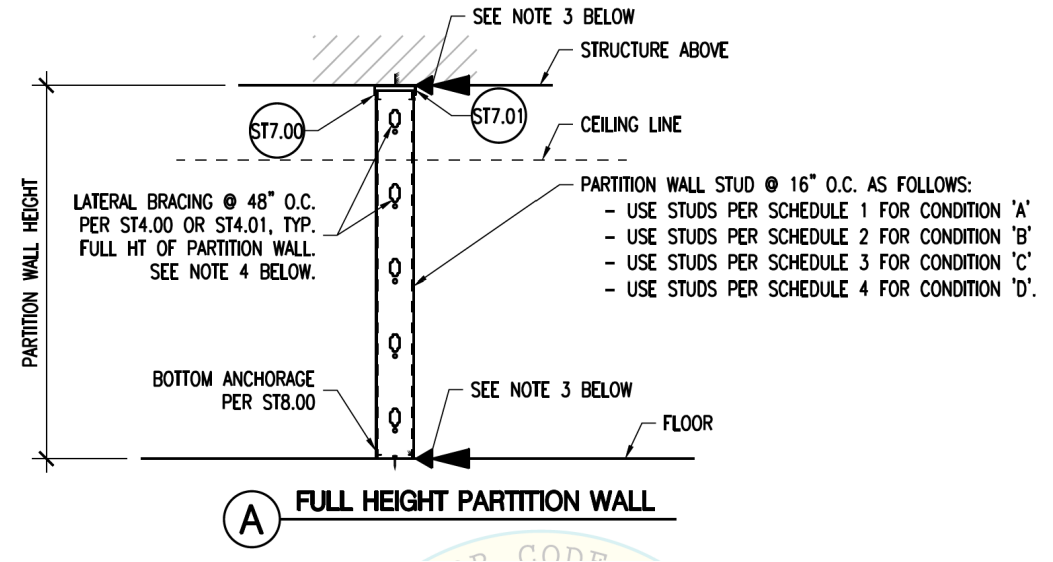
SMRF-3/10: 3/10 Story Steel Special Moment Resisting Frame

SRCSW-BF-3/10: 3/10 Story Special Reinforced Concrete Shear Wall (Bearing Frame)

SRCSW-BW-3/10: 3/10 Story Special Reinforced Concrete Shear Wall (Bearing Wall)

Example Stud and Anchorage Selection – Condition A

- Hospital with $S_{DS} = 1.00g$
- Building lateral system = Concrete Shear Wall (Bearing Frame)
- Height = 7 stories (16' per story)
- Wall height = 16' and no casework/attachments.

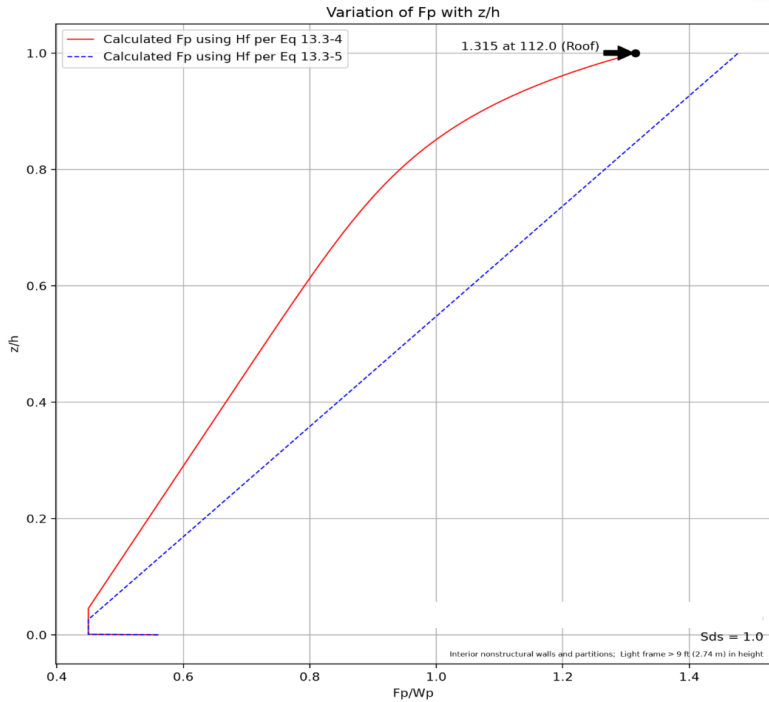


Site Info/ F_p Calculations from [HCAI Website](#)

- $F_p/W_p = 1.315$ ($z/h = 1.0$) [ASCE 7-22](#)
- Equivalent S_{DS} for use with [2013 OPD](#) = 1.83

Governing F_p :

Location, ($S_{ds} = 1.0$)	z	z/h	H_f	F_p/W_p
Roof	112	1	3.1148	1.3148



Equivalent S_{DS} Calculation for OPDs/OPMs approved for previous versions of ASCE 7: ⇄

Equivalent ASCE7-16 Chapter 13 item = Interior nonstructural walls and partitions -All other walls and partitions

$$a_p = 1.0$$

$$R_p = 2.5$$

Equivalent S_{DS} :

Location	z	z/h	Equivalent S_{ds} for OPD/OPM
Roof	112	1	1.8261

Note: All older OPDs/OPMs are approved for $z/h = 1.0$

Example Stud Selection

PARTITION WALL SCHEDULES

SCHEDULE 1: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'A')

S _{DS}	WALL HEIGHT				
	9 FT	12 FT	16 FT		
0.25-0.99	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.00-1.25	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.26-1.43	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33

Stud size based on 2013 OPD and
ASCE 7-16 using $S_{DS} = 1.00g$

PARTITION WALL SCHEDULES

SCHEDULE 1: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'A')

S _{DS}	WALL HEIGHT				
	9 FT	12 FT	16 FT		
0.25-0.99	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.00-1.25	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.26-1.45	362S137-33	362S137-33	362S137-43	400S137-43	600S137-33
1.46-1.95	362S137-33	362S137-33	362S137-54	400S137-54	600S137-43

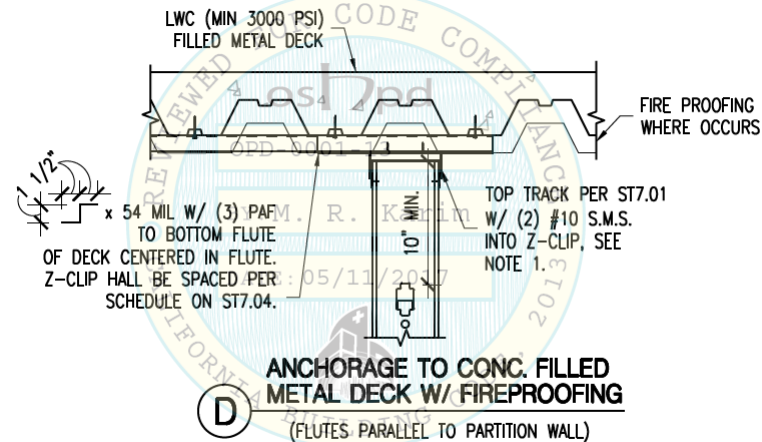
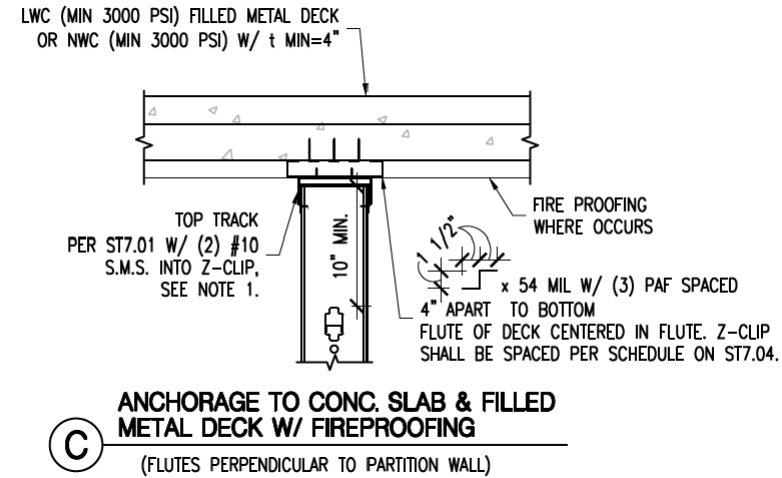
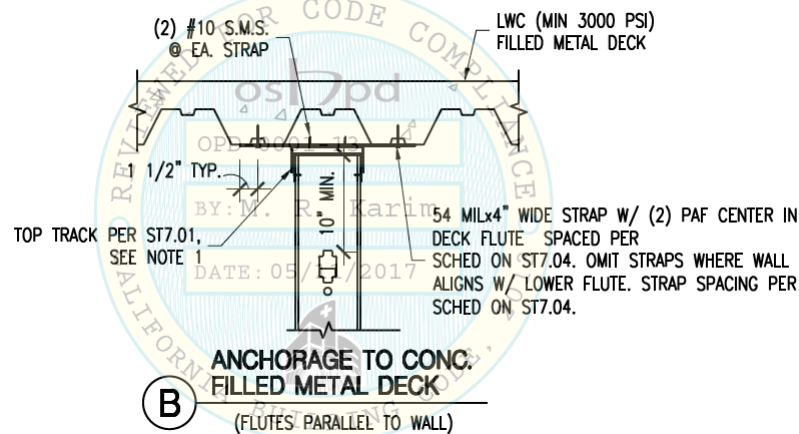
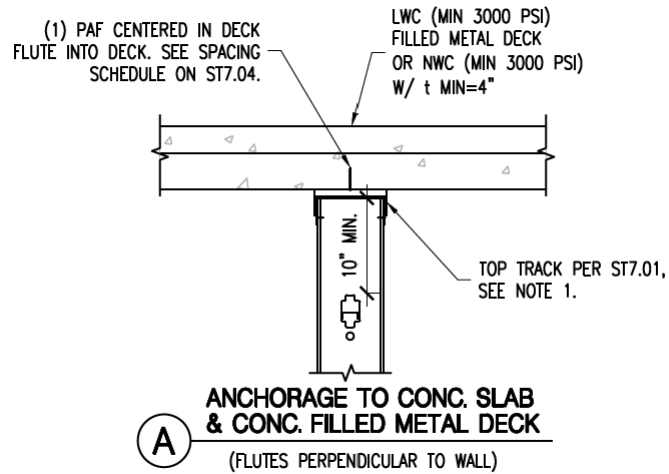
Stud size based on 2013 OPD and
ASCE 7-22 Equivalent $S_{DS} = 1.83$

SCHEDULE 1: MINIMUM PARTITION WALL STUD SIZE (PARTITION CONDITION 'A')

F _p /W _p Range	9 FT	12 FT	16 FT		
0.00 - > 0.75	362S162-22	362S162-22	362S162-22	400S162-22	600S162-22
0.76 - > 1.50	362S162-33	362S162-33	362S162-33	400S162-33	600S162-33
1.51 - > 2.25	362S162-33	362S162-33	362S200-33	400S162-33	600S162-33
2.26 - > 3.00	362S162-33	362S162-33	362S162-43	400S162-43	600S162-43

Stud size based on ASCE 7-22 F_p/W_p
= 1.32 and optimized stud sizes.

Example Top Track Anchorage



Example Top Track Anchorage

TOP TRACK CONNECTION TO LWC FILLED METAL DECK & NWC SLAB (MIN 3000 PSI) FASTENER SPACING SCHEDULES

PARTITION WALL CONDITION 'A'

S _{DS} / WALL HEIGHT	PAF SPACING (INCHES O.C.) PER ST7.02 DETAIL A		
	9 FT	12 FT	16 FT
0.25-0.99	30	24	18
1.00-1.25	24	18	12
1.26-1.45	24	18	12
1.46-1.95	18	12	6

FOR DETAIL B THROUGH D (STRAP OR Z-CLIP) DOUBLE THE SPACING INDICATED UP TO 48" OC.

Anchorage based on 2013 OPD and
ASCE 7-16 using $S_{DS} = 1.00g$

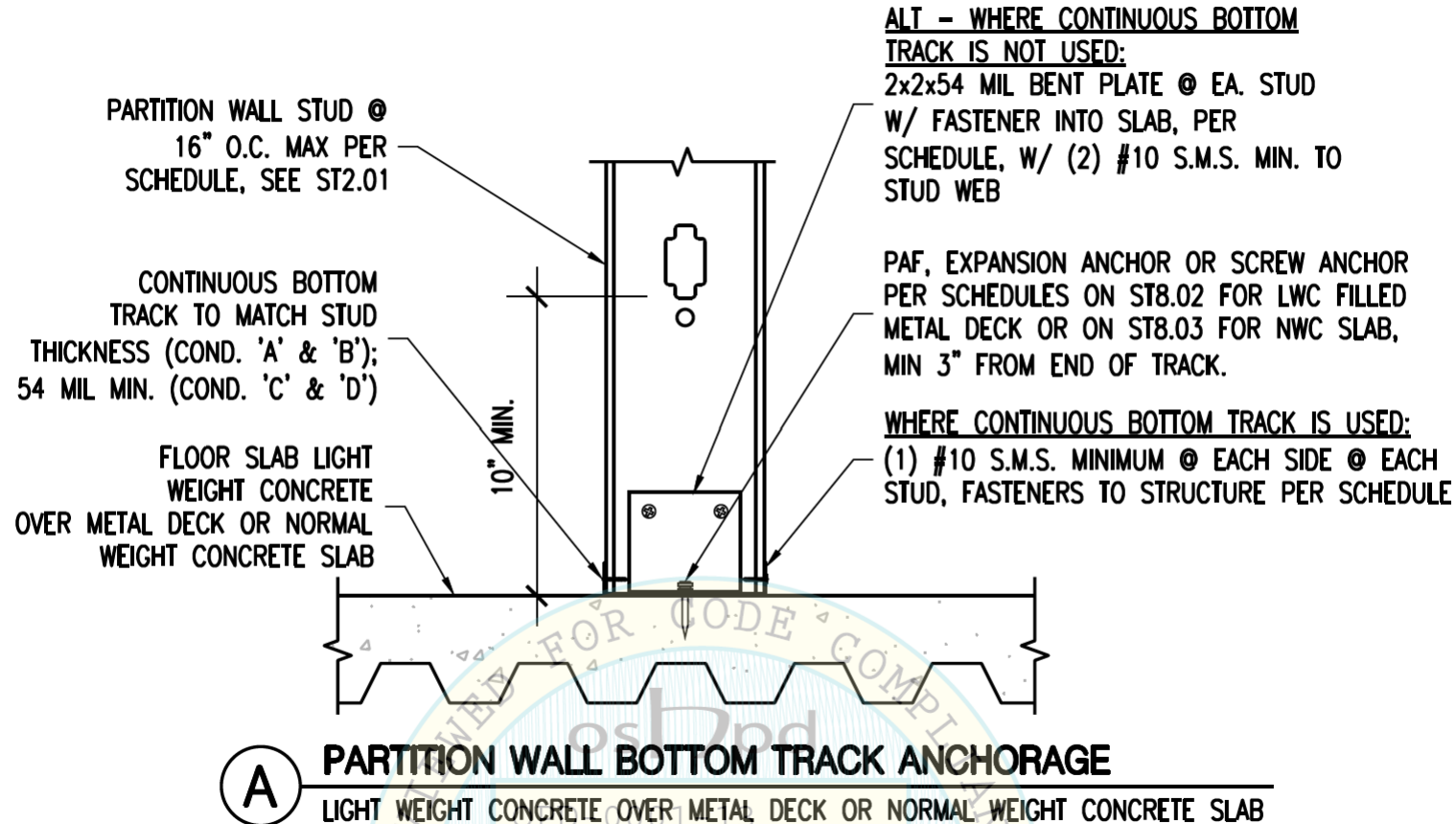
Anchorage based on 2013 OPD and
ASCE 7-22 Equivalent $S_{DS} = 1.83$

PARTITION WALL CONDITION 'A'

MAX PARTITION WALL HEIGHT	F _p /W _p RANGE	MAXIMUM PAF SPACING IN INCHES		
		BOTTOM CONNECTION PER ST8.01	TOP CONNECTION PER ST7.02A	TOP CONNECTION PER ST7.02B, ST7.02C-D
16 FT	0.00 -> 0.75	16	18	42
	0.76 -> 1.50	8	6	18
	1.51 -> 2.25	NA	6	12
	2.26 -> 3.00	NA	NA	6

Anchorage based on ASCE 7-22
 $F_p/W_p = 1.32$

Example Bottom Track Anchorage



Example Bottom Track Anchorage

BOTTOM CONNECTION DEMANDS (PARTITION WALL CONDITION 'A')

MAX PARTITION WALL HEIGHT	S_{DS}	MAXIMUM FASTENER SPACING IN INCHES AT BOTTOM CONNECTION				
		PAF	3/8" EXP ANCHOR W/ 2" EMBED	1/2" EXP ANCHOR W/ 2 1/4" EMBED	3/8" SCREW ANCHOR W/ 2 1/2" EMBED	1/2" SCREW ANCHOR W/ 2 1/2" EMBED
16 FT	0.25-0.99	16	32	32	32	32
	1.00-1.25	8	32	32	32	32
	1.26-1.45	8	32	32	32	32
	1.46-1.95	8	32	32	32	32

Anchorage based on 2013 OPD and
ASCE 7-16 using $S_{DS} = 1.00g$

Anchorage based on 2013 OPD and
ASCE 7-22 Equivalent $S_{DS} = 1.83$

PARTITION WALL CONDITION 'A'

MAX PARTITION WALL HEIGHT	F_p/W_p RANGE	MAXIMUM PAF SPACING IN INCHES		
		BOTTOM CONNECTION PER ST8.01	TOP CONNECTION PER ST7.02A	TOP CONNECTION PER ST7.02B, ST7.02C-D
16 FT	0.00 -> 0.75	16	18	42
	0.76 -> 1.50	8	6	18
	1.51 -> 2.25	NA	6	12
	2.26 -> 3.00	NA	NA	6

Anchorage based on ASCE 7-22
 $F_p/W_p = 1.32$

Conclusions

- New force requirements in ASCE 7-22 have increased the demands on partition walls in hospital buildings which have doubled in some cases
- Partition wall overstrength factor R_{PO} may not have considered the added strength from the gypsum board and demands based on floor accelerations are inherently conservative
- Stud sizes and anchorage at the top and bottom track have increased resulting in higher costs for material and labor
- For high seismic areas demands are outside the existing range of S_{DS} in the OPD tables requiring custom details with 14 gage or even 12 gage studs for conditions C or D
- Partition wall damage/cracks are generally associated with inter-story drifts
- Out-of-plane failures in partition walls have been observed when they are inadequately braced, unbraced or ceiling supported.

Recommendations!

- Keep design of partition walls consistent with ASCE 7-16
- Update design of partition walls consistent with ASCE 7-22
- Other?



Item #5

Minimum anchor qualification requirement for post-installed anchors in concrete

- Background for current provisions in American Concrete Institute (ACI) 355.2-24 and 355.4-24 for post-installed anchors in concrete
- Applicability of the new crack cycling qualification requirements for post-installed anchors in concrete used for anchorage of nonstructural components in hospital buildings
- Committee to endorse and/or provide feedback on modifications presented
- Discussion and public input

Facilitator: Roy Lobo, PhD, SE, Principal Structural Engineer, HCAI (or designee)

Anchor Testing Requirements for Qualification of Post-installed Anchors in Concrete ACI-355.2(24) and ACI-355.4 (24)

By
Roy Lobo, Ph.D., S.E.
To HBSB – SNSR Committee
7/29/2026

Motivation for the study

- New Crack Cycling requirement for anchor qualification will result in reduced anchors design values, added cost of anchors and is not justified due to lack of evidence that this level of qualification applies to Hospital buildings designed as using an importance factor of 1.5 intended to limit structural damage resulting in smaller crack widths in a seismic event.
- Nonstructural components and equipment with anchors in concrete in hospitals are typically floor mounted to the floor slab, slab on grade, on an equipment pad, or hung from the underside of a concrete over metal deck. The research resulting in the new requirements is premised on flexure cracks with anchors installed on the underside of a reinforced concrete moment resisting frame beam, not a suitable location for post installed anchors.

Outline of Presentation

- Background
- New Crack Cycling requirement for anchor qualification
- Application of new qualification requirements for post installed anchors in concrete for anchorage of nonstructural components
- Issues with new crack cycling anchor qualification
- Recommendations

Code Requirement for Anchors in Concrete

- Requirement in ASCE 7

13.4.2.3 Post-Installed Anchors in Concrete and Masonry Post-installed anchors in concrete shall be prequalified for seismic applications in accordance with ACI 355.2 or other approved qualification procedures. Post-installed anchors in masonry shall be prequalified for seismic applications in accordance with approved qualification procedures.

- Requirement in 2025 CBC

1617A.1.19 ASCE 7, Section 13.4 Replace ASCE 7, Sections 13.4.2.3, with the following:

13.4.2.3 Prequalified post-installed anchors and specialty inserts in concrete and masonry.

Post-installed anchors, post-installed reinforcing bars and specialty inserts in concrete that are pre-qualified for seismic applications in accordance with ACI 355.2, ACI 355.4, ICC-ES AC193, ICC-ES AC232, ICC-ES AC308 or ICC-ES AC446 shall be permitted. Post-installed anchors in masonry shall be pre-qualified for seismic applications in accordance with ICC-ES AC01, AC58 or AC106. ||

What prompted this issue?

- ACI 318-25 now adopts ACI 355.2 (24) and ACI 355.4 (24) by reference, and ACI 318-25 is adopted by the 2027 IBC
- ACI 355.2 (24) and ACI 355.4 (24) now have a new testing protocol for anchor qualification that requires good performance of anchors under repeated crack width cycling under sustained tension loads.

Why was Crack Cycling Investigated?

- Concrete cracks
- Cracks open and close during a cyclic event (wind or seismic)
- Reliability of anchors when subject to extreme events like earthquakes
- Current crack cycling requirements (≤ 0.3 mm) and large number of cycles (1000) in ACI 355 not intended to represent seismic conditions
- Crack closure and compressive stress can significantly affect anchor performance
- Other

Background for testing parameters

Behavior and Testing of Anchors in Simulated Seismic Cracks

by Matthew S. Hoehler and Rolf Eligehausen

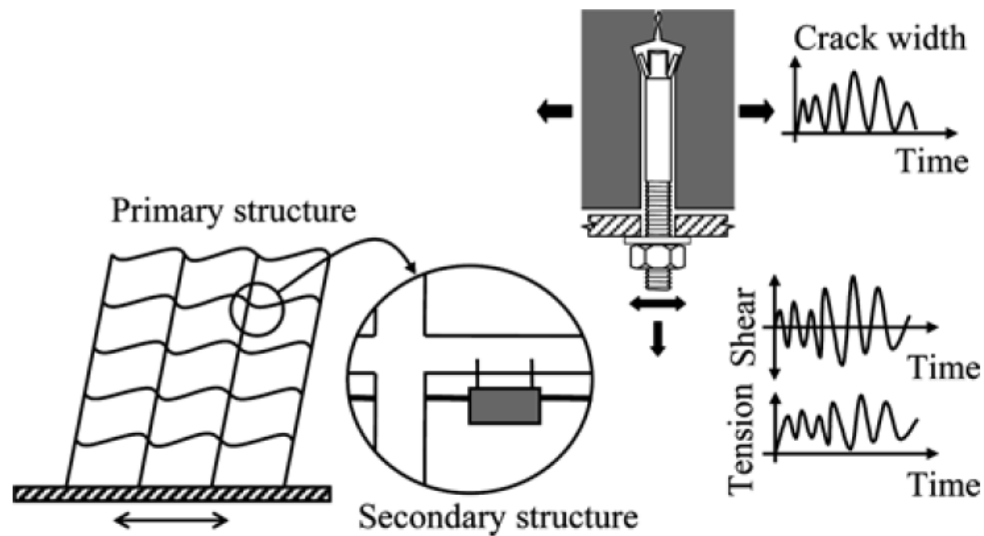


Fig. 1—Actions acting on nonstructural anchorage under earthquake loading.

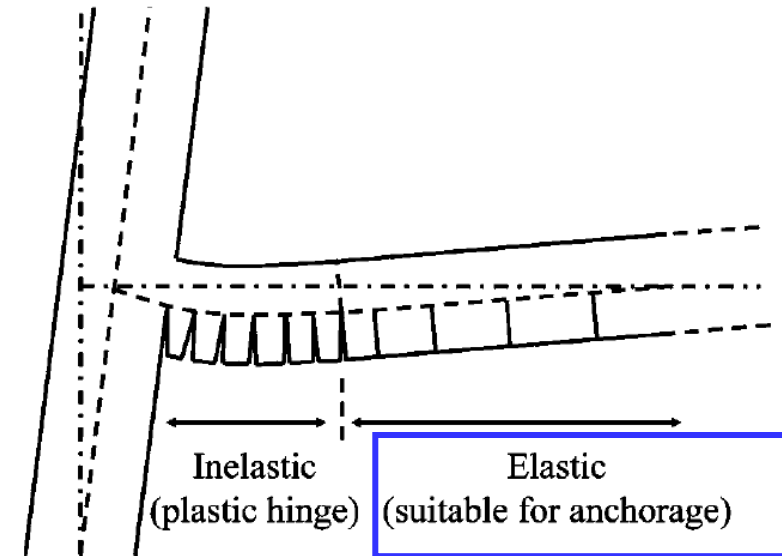


Fig. 2—Idealization of cracking in beam caused by transverse motion of building.

Crack width and cycling

- Extensive numerical studies of crack width in reinforced concrete bending members designed according to Eurocode indicate that the maximum crack width that can be expected to occur at steel yield in unfavorably designed flexural members is approximately 0.8 mm (0.03 in.). This value was determined for frame members subjected primarily to axial and bending deformation and is not valid for shear cracking. (Hoehler et. al.)

Crack Cycling Loading Setup

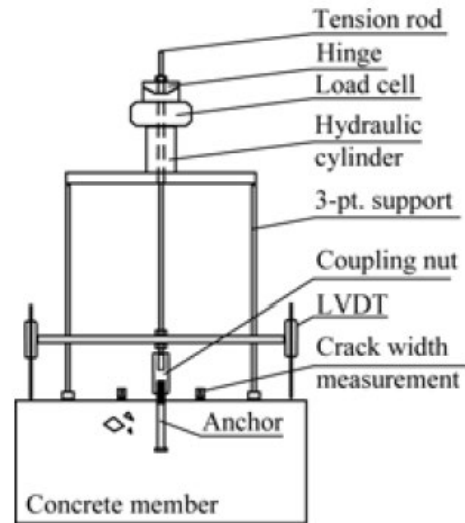


Fig. 7—Anchor loading setup for seismic crack cycling tests.

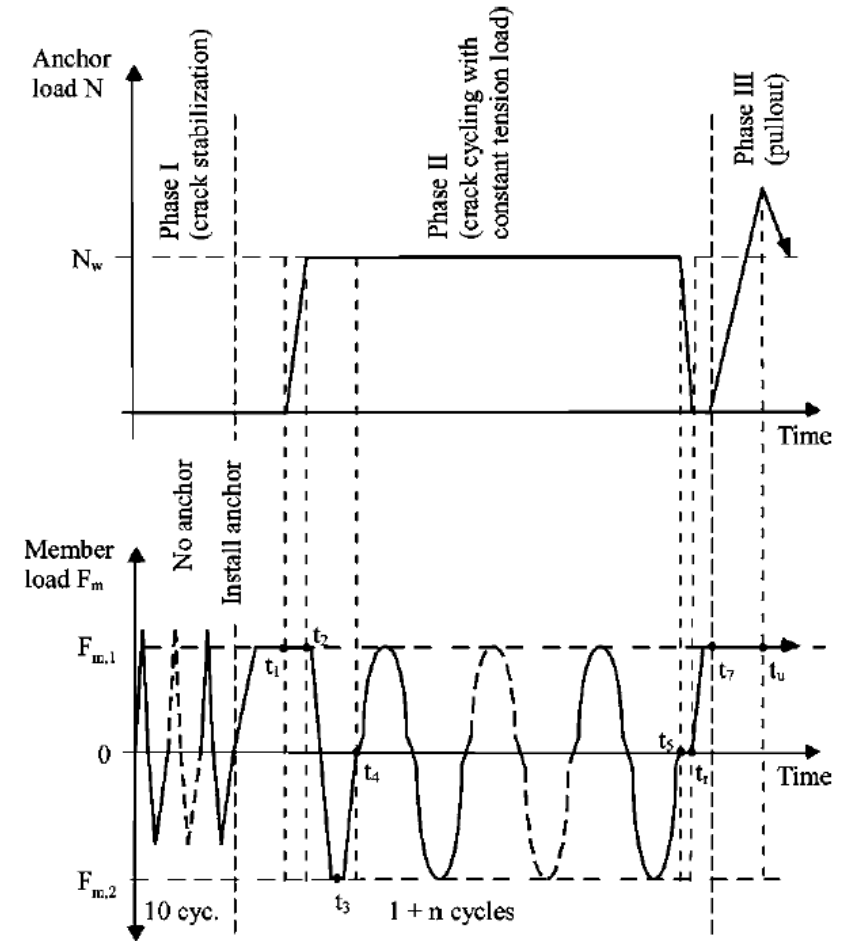


Fig. 8—Schematic loading time-histories for anchor and anchorage component.

Failure Modes

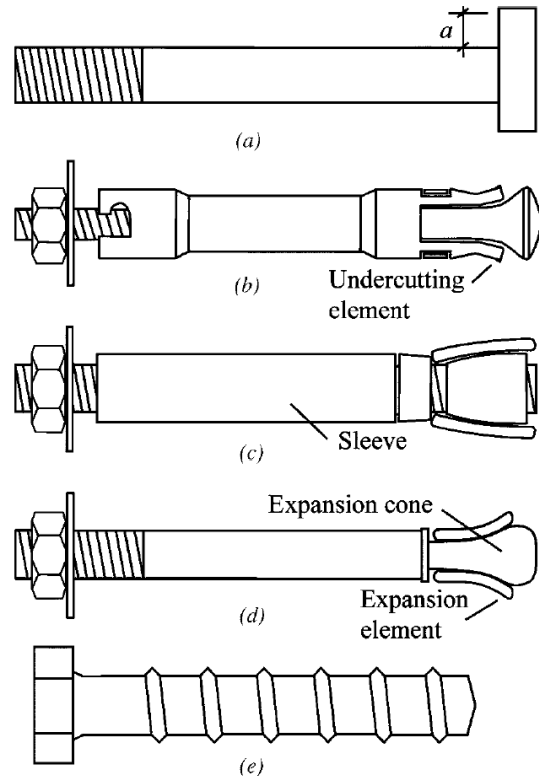


Fig. 3—Investigated anchor types: (a) headed stud; (b) undercut anchor; (c) expansion anchor (sleeve-type); (d) expansion anchor (bolt-type); and (e) screw anchor.

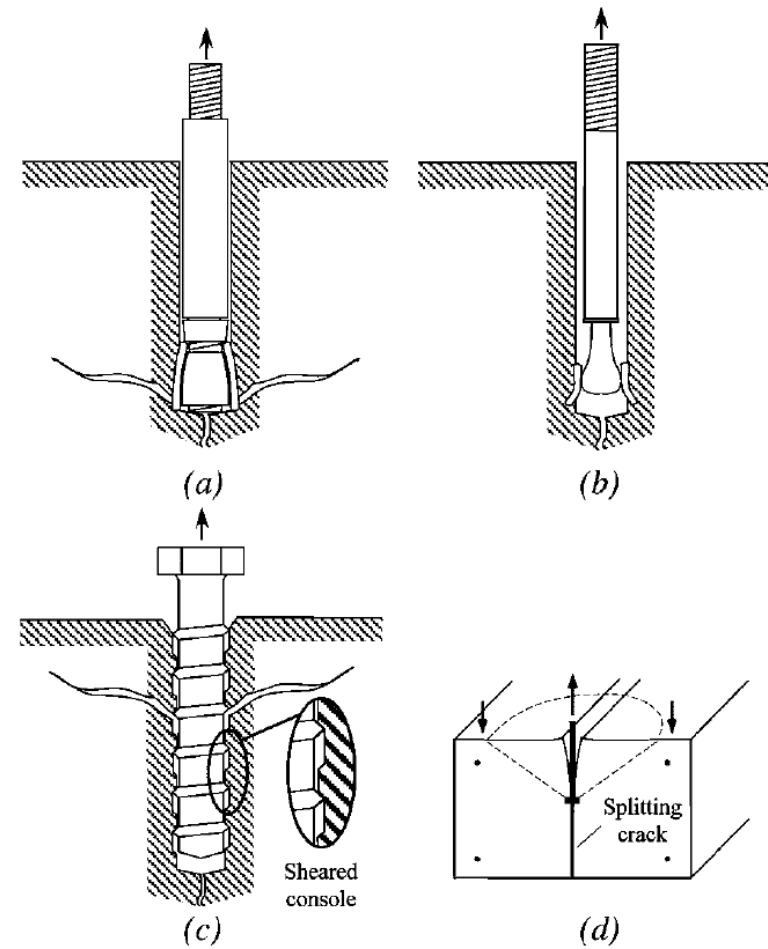


Fig. 10—Failure modes: (a) concrete cone breakout; (b) pull-through; (c) pullout/concrete cone; and (d) splitting.

Pull through failure of Mechanical Anchors

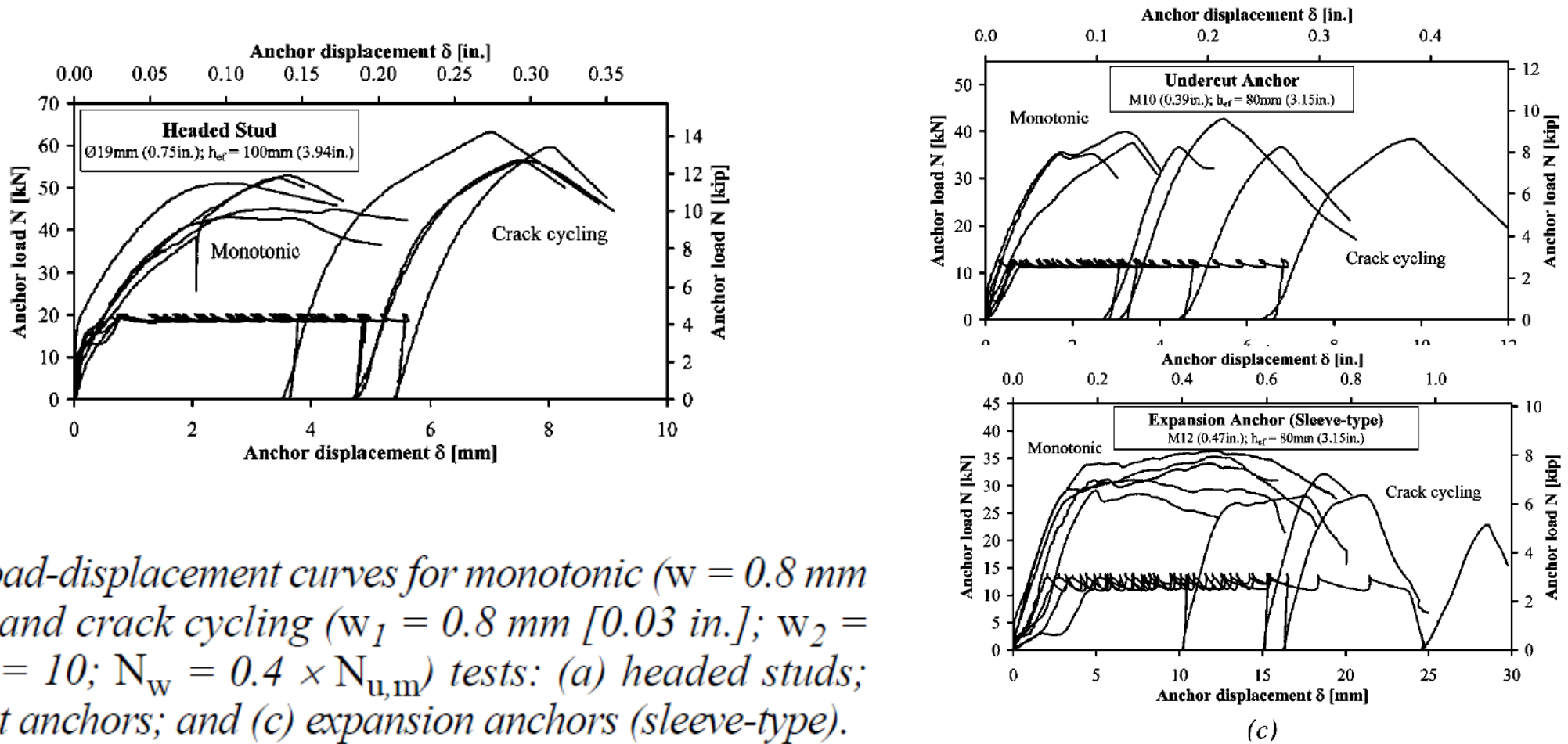


Fig. 12—Load-displacement curves for monotonic ($w = 0.8\text{ mm}$ [0.03 in.]) and crack cycling ($w_1 = 0.8\text{ mm}$ [0.03 in.]; $w_2 = 0.0\text{ mm}$; $n = 10$; $N_w = 0.4 \times N_{u,m}$) tests: (a) headed studs; (b) undercut anchors; and (c) expansion anchors (sleeve-type).

Pull through failure of Mechanical Anchors

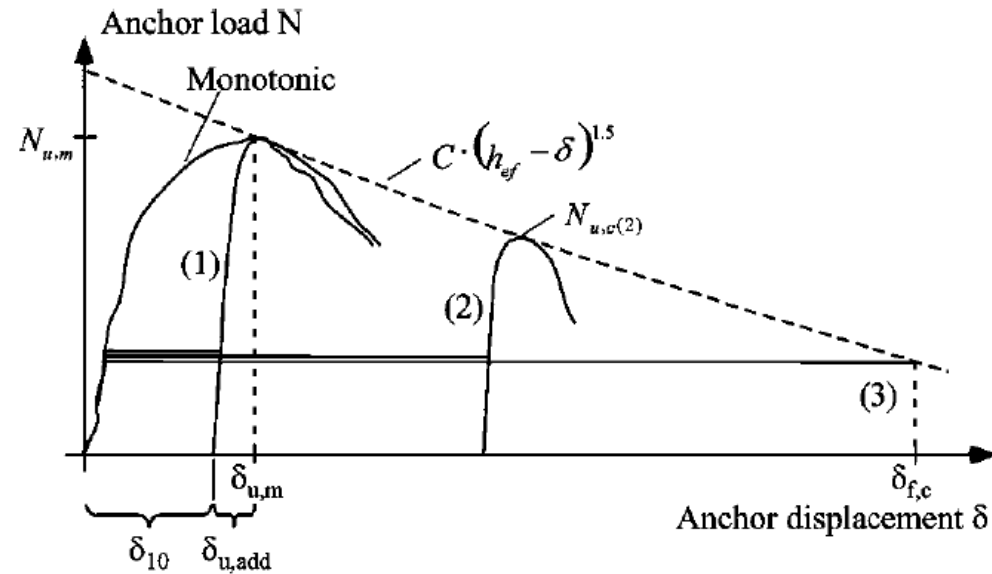


Fig. 13—Schematic load-displacement curves for crack cycling tests in case of concrete cone breakout.

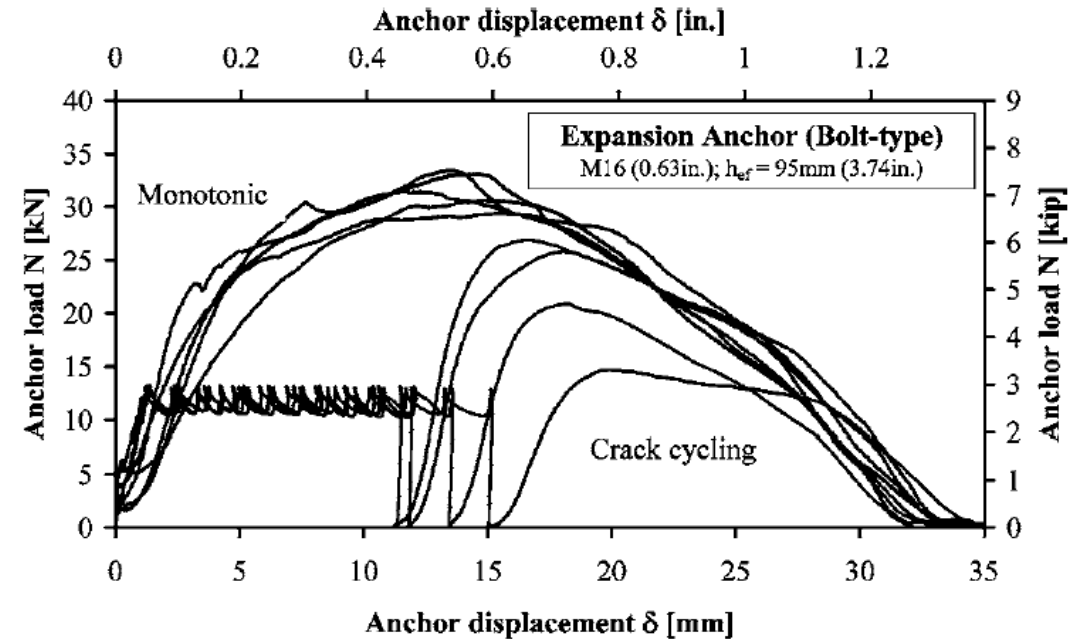


Fig. 14—Load-displacement curves for monotonic ($w = 0.8 \text{ mm}$ [0.03 in.]) and crack cycling ($w_1 = 0.8 \text{ mm}$ [0.03 in.]; $w_2 = 0.0 \text{ mm}$; $n = 10$; $N_w = 0.4 \times N_{u,m}$) tests for pull-through ($\delta_{10} \approx \delta_{u,m}$).

Displacement Conditions during Crack Cycling

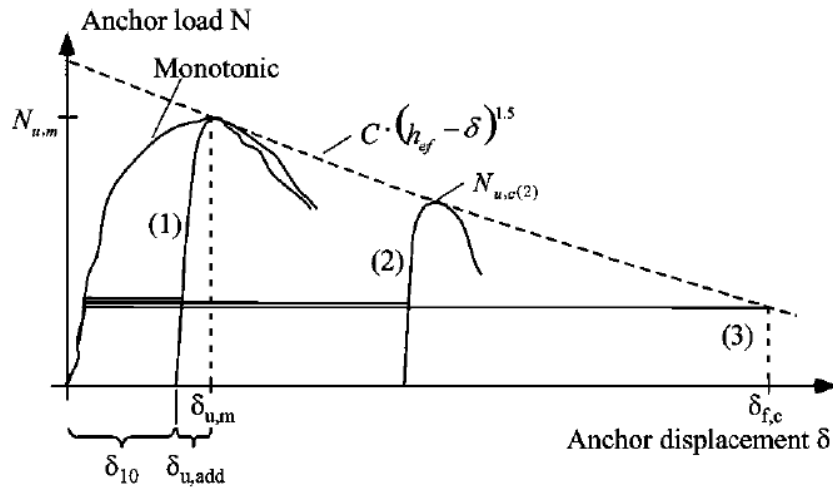


Fig. 13—Schematic load-displacement curves for crack cycling tests in case of concrete cone breakout.

Table 3—Displacement conditions during crack cycling for concrete cone breakout

Condition	Description	Residual strength
(1) $\delta_{10} < \delta_{u,m}$	Displacement during crack cycling is less than that at ultimate load in monotonic reference tests	$N_{u,c} \geq N_{u,m}$
(2) $\delta_{u,m} \leq \delta_{10} < \delta_{f,c}$	Displacement during crack cycling equals or exceeds that at ultimate load in monotonic reference tests but does not reduce effective anchorage depth enough to result in concrete cone breakout at applied constant tension load	$N_{u,c} < N_{u,m}$
(3) $\delta_{10} = \delta_{f,c}$	Displacement during crack cycling reduces effective anchorage depth enough to result in concrete cone breakout at applied constant tension load	Not applicable

Acceptance Criteria

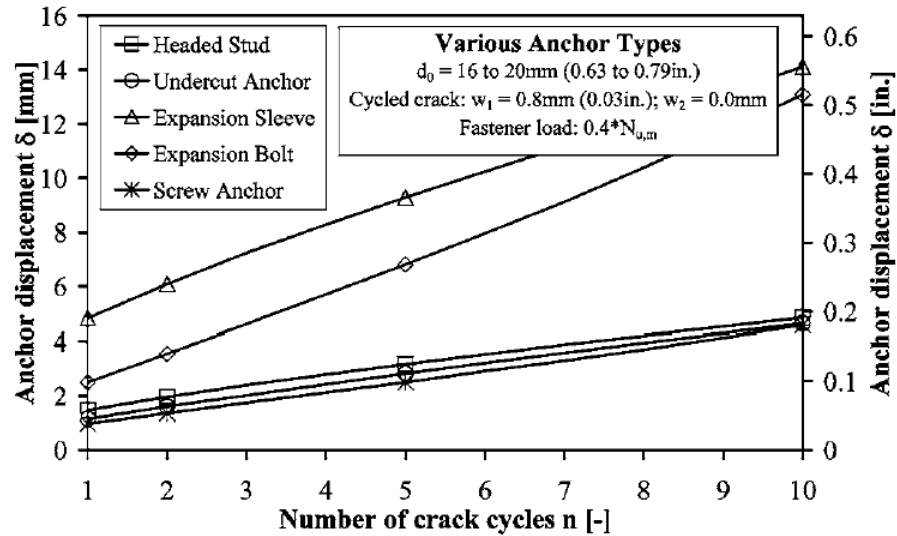


Fig. 17—Anchor displacement as function of number of crack cycles for investigated anchors.

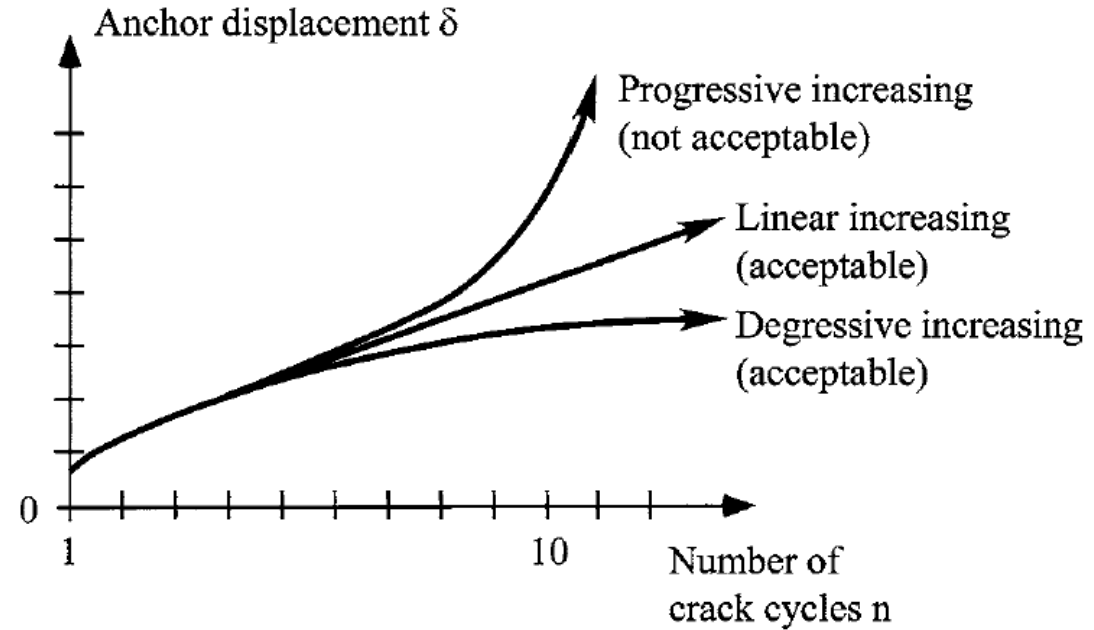


Fig. 18—Schematic anchor displacement for small number of crack cycles (<100) of relatively large amplitude, $w_1 - w_2 \geq 0.5\text{ mm}$ (0.02 in.).

Test Protocol New Anchor Qualification

- ACI 355.2 & 355.4 (24) – Crack cycling tests

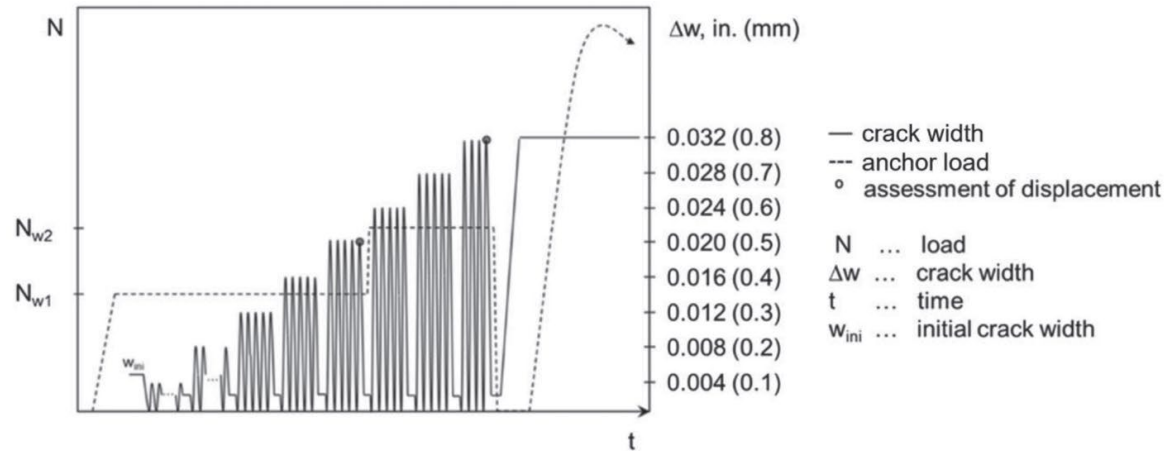


Table 10.6.7.2—Required load amplitude for Table 5.1b , Test 20

Anchor load	Number of cycles	Crack width Δw, in. (mm)
N _{w1}	20	0.004 (0.1)
N _{w1}	10	0.008 (0.2)
N _{w1}	5	0.012 (0.3)
N _{w1}	5	0.016 (0.4)
N _{w1}	5	0.020 (0.5)
N _{w2}	5	0.024 (0.6)
N _{w2}	5	0.028 (0.7)
N _{w2}	4	0.032 (0.8)
Total	59	

Fig. 10.6.7.2—Cyclic crack width and loading protocol for Table 5.1b, Test 20.

$$N_{w1} = 0.40 \cdot N_{u,m,3a} \cdot \left(\frac{f_{c,20}}{f_{c,3a}} \right)^n \quad (\text{lb [N]}) \quad (10.6.7.4b)$$

$$N_{w2} = 0.50 \cdot N_{u,m,3a} \cdot \left(\frac{f_{c,20}}{f_{c,3a}} \right)^n \quad (\text{lb [N]}) \quad (10.6.7.4d)$$

Test Protocol New Anchor Qualification

- ACI 355.2 & 355.4 (24) – Seismic Cycling Tension Test

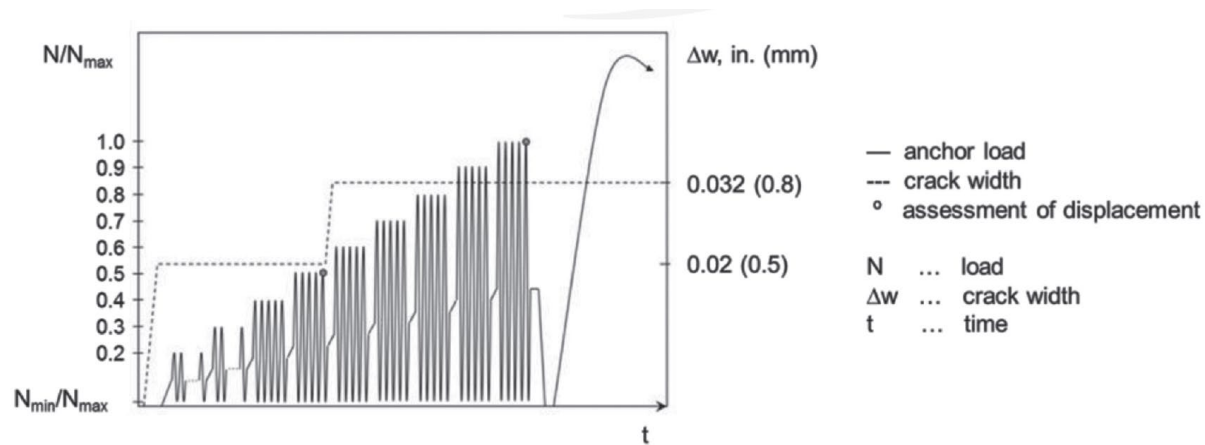


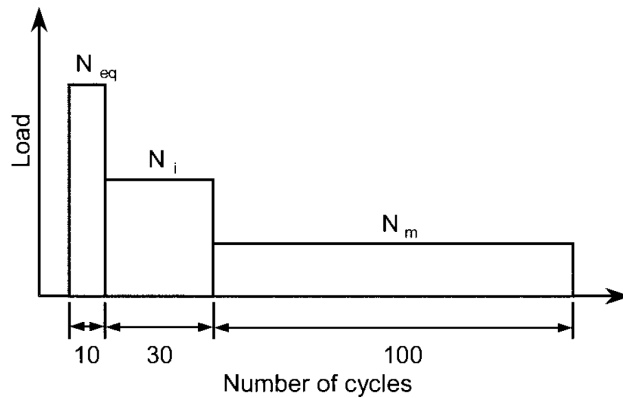
Fig. 10.6.6—Cyclic loading protocol for Table 5.1b, Test 19.

Table 10.6.6—Required load amplitude for Table 5.1b, Test 19

N/N_{max}	Number of cycles	Crack width Δw , in. (mm)
0.2	25	0.02 (0.5)
0.3	15	0.02 (0.5)
0.4	5	0.02 (0.5)
0.5	5	0.02 (0.5)
0.6	5	0.032 (0.8)
0.7	5	0.032 (0.8)
0.8	5	0.032 (0.8)
0.9	5	0.032 (0.8)
1	5	0.032 (0.8)
Total	75	

Test Protocol Current Anchor Qualification

- 355.4 (11) – Simulated Seismic Cycling Test for tension and shear similar in 355.2 & 355.4 (22)

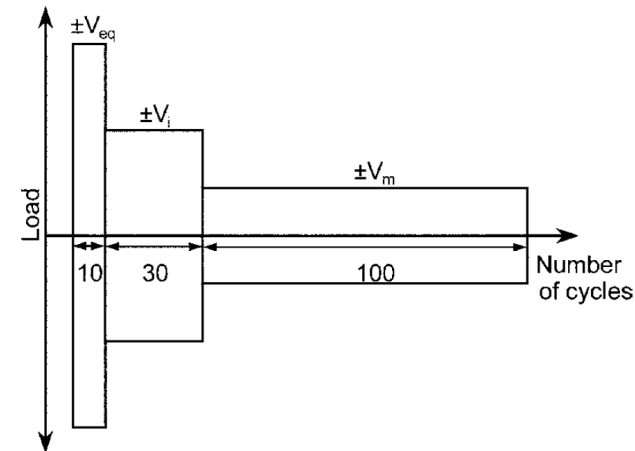


$$N_{eq} = 0.5 \bar{N}_{o,i} \left(\frac{f_{c,test}}{f_{c,test,2}} \right)^n \text{ lb}$$

$$N_m = \frac{N_{eq}}{2} \text{ lb}$$

$$N_i = \frac{N_{eq} + N_m}{2} \text{ lb}$$

Simulated seismic tension test



$$V_{eq} = 0.5 \bar{V}_{o,i} \frac{f_{c,test}}{f_{c,test,2}} \text{ lb}$$

$$V_m = \frac{V_{eq}}{2} \text{ lb}$$

$$V_i = \frac{V_{eq} + V_m}{2} \text{ lb}$$

Simulated seismic shear test

What is Different in the new qualification requirement?

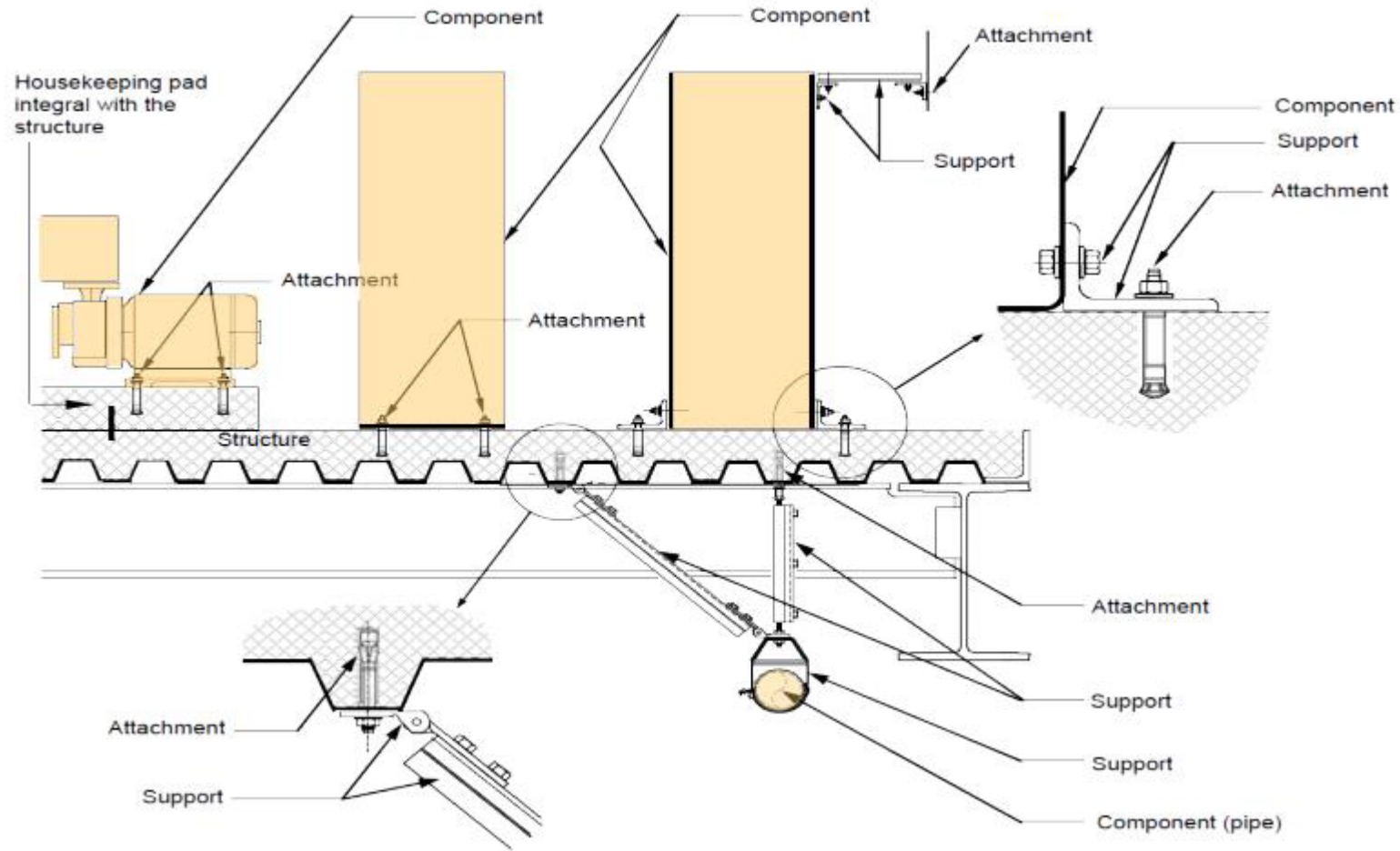
- Crack Protocol: Cycle counting is performed on curvature histories, going from smallest to largest
- Load Protocol: Cycle counting is performed on the oscillating SDOF systems, the input motions of these are affected by the nonlinear behavior of the building
- Integration of a unified protocol to simultaneously consider serviceability and suitability of anchor
- Based on current science and latest FEA tools

Is the loading protocol for anchor qualification reasonable and applicable?

- Thoughts?



Typical Anchorage of Nonstructural Components in Concrete



Issues with crack cycling for post installed anchor qualification

- Anchors for floor mounted equipment are not subject to sustained tension loads.
- Anchors to diagonal braces for seismic bracing of distribution systems are not subject to sustained tension.
- Vertical hangers of brace assemblies could support tributary gravity loads and are in sustained tension, but that load is expected to be small relative to the seismic tension. System could be supported by multiple gravity supports not subject to seismic cyclic loads.
- Location where crack cycling is expected to occur will have reinforcement. Not suitable locations for post installed anchors.
- Max. crack open and tension load occur at different times.
- No evidence provided from tests or past earthquake failures to show how the new protocol prevents failures from earthquakes where the current protocol does not.

Is Crack cycling for anchor qualification in hospital buildings necessary?

- Floor diaphragms are typically concrete over metal deck and are designed essentially elastic thus limiting the crack width or crack cycling that could potentially occur
- There is redundancy in the anchorage not all anchors resisting tension loads will also experience crack width opening
- Anchors in concrete are already qualified for cracked and uncracked concrete under cyclic loads
- Anchors in concrete for NSC are designed for demands using an $I_p = 1.5$ on top of the Omega (Ω) amplification, in conjunction with 0.75% seismic reduction factor, strength reduction factor (ϕ), and the 5% fractile value for the anchor capacity
- Not seen evidence from tests or past earthquake failures that show how the new protocol prevents failures from earthquakes where the current loading protocol for anchor qualification ACI 355.2 (22) and ACI 355.4 (22) does not
- Anchors failures have occurred where there is improper installation and lack of testing

Conclusion

- New anchor qualification adds additional cost of testing the anchors for each size and depth costing millions of dollars without added value for anchors in concrete not in sustained tension.
- Recommend to permit the anchor qualification requirement for anchors not in sustained tension or anchors with very low tension (< 0.2 times the anchor design capacity) subject to seismic cyclic loads, as acceptable, if qualified to the minimum loading protocol as required by ACI 355.2 (22) and ACI 355.4 (22).
- Thoughts?



Item #6

Drift scaling requirement for Equivalent Lateral Force (ELF) and modal response spectrum analysis (MRSA) when ASCE 7-22, Equation 12.8-7 governs the design

- Background research for current drift scaling requirements for long period buildings when $S_1 \geq 0.6$
- Committee to endorse and/or provide feedback on modifications presented
- Discussion and public input

Facilitator: Roy Lobo (or designee)

Drift Scaling for ELF and MRSA when ASCE 7-22 Eq. 12.8-7 Governs

By

Aysegul Gogus, Roy Lobo and Chris Davis

To HBSB – SNSR Committee

7/29/2026

Outline of Presentation

- *Motivation for the study*
- *Background research for current drift scaling requirements for long period buildings when $S_1 \geq 0.6$*
- *Impact of drift scaling on hospital construction – A Case study*
- *Additional Studies showing impact of the change for near fault and non near fault sites*
- *Recommendations from the committee*

Motivation for the Drift Scaling study

- Drift scaling is required for structures where S_1 is equal to or greater than 0.6 and Eq. 12.8-7 governs the minimum design base shear of the building.
- The $S_1 \geq 0.6$ trigger creates a step function for drift for tall buildings when the minimum base shear for long period governs.
- Designs including scaling for drifts could significantly increase cost as this requirement does not include the flexibility of the model which could be inherently flexible relative to the strength.
- Drift scaling is not required when short period governs the response.
- Have improvements in the design ground motions, the multiperiod or two period spectra now made this requirement unnecessary?

Requirements in ASCE 7 for Base Shear Calculation

12.8 EQUIVALENT LATERAL FORCE (ELF) PROCEDURE

12.8.1 Seismic Base Shear The seismic base shear, V , in a given direction shall be determined in accordance with the following equation:

$$V = C_s W \quad (12.8-1)$$

where

C_s = The seismic response coefficient determined in accordance with Section 12.8.1.1, and

W = The effective seismic weight per Section 12.7.2.

Method 2.

Method 1: The seismic response coefficient, C_s , shall be determined in accordance with Equation (12.8-2).

$$C_s = \frac{S_a}{\left(\frac{R}{I_e}\right)} \quad (12.8-2)$$

where

be used.

Method 2: The seismic response coefficient, C_s , shall be determined in accordance with the following:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \quad (12.8-3)$$

for $T \leq T_L$,

$$C_s = \frac{S_{D1}}{T \left(\frac{R}{I_e}\right)} \quad (12.8-4)$$

and for $T > T_L$,

$$C_s = \frac{S_{D1} T_L}{T^2 \left(\frac{R}{I_e}\right)} \quad (12.8-5)$$

where I_e and R are as defined in this section;

S_{D1} = Design spectral response acceleration parameter at a period of 1.0 s, as determined from Section 11.4.5 or 11.4.6;

T = Fundamental period of the structure(s) determined in Section 12.8.2; and

T_L = Long-period transition period(s) determined in Section 11.4.6.

Lower Bounds for C_s for Both Methods: C_s shall not be less than

$$C_s = 0.044 S_{DS} I_e \geq 0.01 \quad (12.8-6)$$

In addition, for structures located where S_1 is equal to or greater than 0.6, C_s shall not be less than

$$C_s = 0.5 S_1 / (R/I_e) \quad (12.8-7)$$

Period (T) for calculating Base Shear

substantiated analysis. The fundamental period, T , shall not exceed the product of the coefficient for upper limit on calculated period, C_u , from Table 12.8-1 and the approximate fundamental period, T_a , determined in accordance with Section 12.8.2.1. As an alternative to performing an analysis to determine the fundamental period, T , it is permitted to use the approximate building period, T_a , calculated in accordance with Section 12.8.2.1, directly.

12.8.2.1 Approximate Fundamental Period The approximate fundamental period (T_a), in seconds, shall be determined from the following equation:

$$T_a = C_t h_n^x \quad (12.8-8)$$

where h_n is the structural height as defined in Section 11.2, and the coefficients C_t and x are determined from Table 12.8-2.

$$C_s = \frac{S_{D1}}{T \left(\frac{R}{I_e} \right)} \quad (12.8-4)$$

Table 12.8-1. Coefficient for Upper Limit on Calculated Period.

Design Spectral Response Acceleration Parameter at 1 s, S_{D1}	Coefficient C_u
≥ 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤ 0.1	1.7

Period for Calculating Drift

12.8.6.2 Period for Computing Displacement and Drift For determining displacements and drifts, it is permitted to determine the elastic displacements, δ_e , using seismic design forces based on the computed fundamental period of the structure without the upper limit, $C_u T_a$, specified in Section 12.8.2.

Scaling of Drift - ELF

C12.8.6.1 Minimum Base Shear and Load Combination for Computing Displacement and Drift Except for period limits (as described in Section C12.8.6.2), all of the requirements of Section 12.8 must be satisfied when computing drift for an ELF analysis, except that the minimum base shear determined from applying Equation (12.8-6) does not need to be considered. This equation represents a minimum strength that needs to be provided to a system (Section C12.8.1.1). Equation (12.8-7) needs to be considered, when triggered, because it represents the increase in the response spectrum in the long-period range from near-fault effects.

Scaling of Drift - MRSA

C12.9.1.4.2 Scaling of Drifts Displacements from the modal response spectrum are only scaled to the ELF base shear where V_t is less than $C_s W$ and C_s is determined based on Equation (12.8-7). For all other situations, the displacements need not be scaled, because the use of an overly flexible model will result in conservative estimates of displacement that need not be further scaled. The reason for requiring scaling when Equation (12.8-7) controls the minimum base shear is to be consistent with the requirements for designs based on the ELF procedure.

S_a for Calculating Base Shear & Drift ($T_s < T < T_L$)

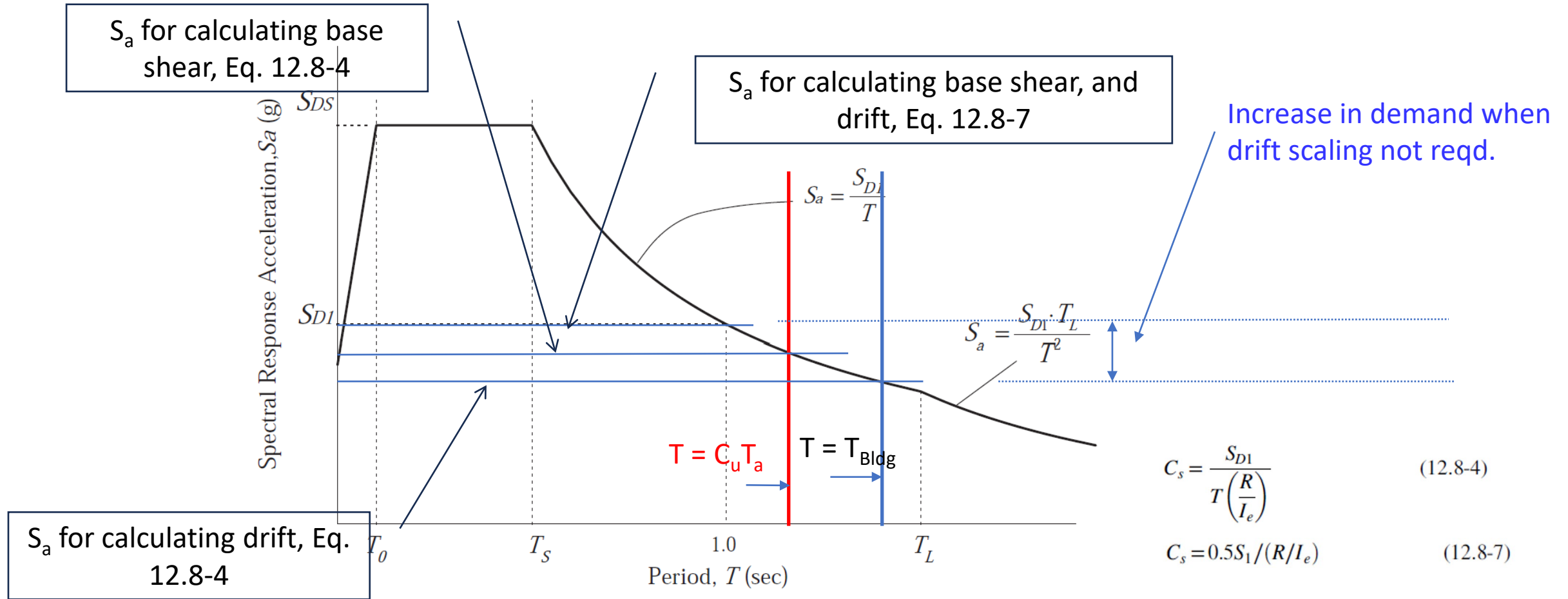


Figure 11.4-1. Two-period design response spectrum.

Genesis of Eq. 12.8-7

- Eq. 12.8-7 is pegged to sites where $S_1 \geq 0.6$. This limit was selected to address **spectral shapes** from near fault records including the Sylmar Converter Station (1971), Jensen Filtration plant (1994) and Newhall Fire Station (1994) records. Similar records were seen in Kobe (1995) and El Centro (1979). (source:- [Ron Hamburger](#))

Is Spectral Shape for drift scaling dependent on Site Class for near fault effects?

- For buildings in the long period range ($T_s < T < T_L$), is drift scaling intended more for buildings in Site Class D and E soils because of the elongated spectra, or in Site Class A, B and C soils?



- Drift scaling is independent of Site Class?

Is Drift Scaling per Eq. 12.8-7 Still Required?

- When Eq 12.8-7 governs the design base shear, is drift scaling still required given the advances made to the design response spectrum. This equation was introduced in the 2000 NEHRP provisions which required F_a and F_v multipliers to estimate the response spectrum. Since then, there have been revisions to how S_{D1} is derived from a response spectrum. This has resulted in S_{D1} being larger than S_{DS} for many site class D and E sites.

Background regarding 1997 UBC Eq. 30-7 and Drift

Many significant investigations of the effects of **near field pulses** had been undertaken especially those by Heaton and Hall of CalTech in the late 1990s. These investigations confirmed the earlier work of the SEAOC Seismology's Strong Ground Motion Subcommittee that in the **near field of large earthquakes, tall structures need to be stiffer as well as stronger if they were to safely withstand the near field pulse effects of large earthquakes**. One way to achieve this objective is to require that the minimum base shear of formula 30-7 be used when calculating drift. (Source: [Kircher et.al.](#))

Background regarding 1997 UBC Eq. 30-7 and Drift

TABLE 16-I—SEISMIC ZONE FACTOR Z

ZONE	1	2A	2B	3	4
Z	0.075	0.15	0.20	0.30	0.40

NOTE: The zone shall be determined from the seismic zone map in Figure 16-2.

TABLE 16-R—SEISMIC COEFFICIENT C_v

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	$Z = 0.075$	$Z = 0.15$	$Z = 0.2$	$Z = 0.3$	$Z = 0.4$
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	See Footnote 1				

¹Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F .

TABLE 16-T—NEAR-SOURCE FACTOR N_v ¹

SEISMIC SOURCE TYPE	CLOSEST DISTANCE TO KNOWN SEISMIC SOURCE ^{2,3}			
	≤ 2 km	5 km	10 km	≥ 15 km
A	2.0	1.6	1.2	1.0
B	1.6	1.2	1.0	1.0
C	1.0	1.0	1.0	1.0

¹The Near-Source Factor may be based on the linear interpolation of values for distances other than those shown in the table.

²The location and type of seismic sources to be used for design shall be established based on approved geotechnical data (e.g., most recent mapping of active faults by the United States Geological Survey or the California Division of Mines and Geology).

³The closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the

TABLE 16-U—SEISMIC SOURCE TYPE¹

SEISMIC SOURCE TYPE	SEISMIC SOURCE DESCRIPTION	SEISMIC SOURCE DEFINITION ²	
		Maximum Moment Magnitude, M	Slip Rate, SR (mm/year)
A	Faults that are capable of producing large magnitude events and that have a high rate of seismic activity	$M \geq 7.0$	$SR \geq 5$
B	All faults other than Types A and C	$M \geq 7.0$ $M < 7.0$ $M \geq 6.5$	$SR < 5$ $SR > 2$ $SR < 2$
C	Faults that are not capable of producing large magnitude earthquakes and that have a relatively low rate of seismic activity	$M < 6.5$	$SR \leq 2$

¹Subduction sources shall be evaluated on a site-specific basis.

²Both maximum moment magnitude and slip rate conditions must be satisfied concurrently when determining the seismic source type.

1630.2.1 Design base shear. The total design base shear in a given direction shall be determined from the following formula:

$$V = \frac{C_v I}{R T} W \quad (30-4)$$

The total design base shear need not exceed the following:

$$V = \frac{2.5 C_a I}{R} W \quad (30-5)$$

The total design base shear shall not be less than the following:

$$V = 0.11 C_a I W \quad (30-6)$$

In addition, for Seismic Zone 4, the total base shear shall also not be less than the following:

$$V = \frac{0.8 Z N_v I}{R} W \quad (30-7)$$

Background regarding 1997 UBC Eq. 30-7 and Drift

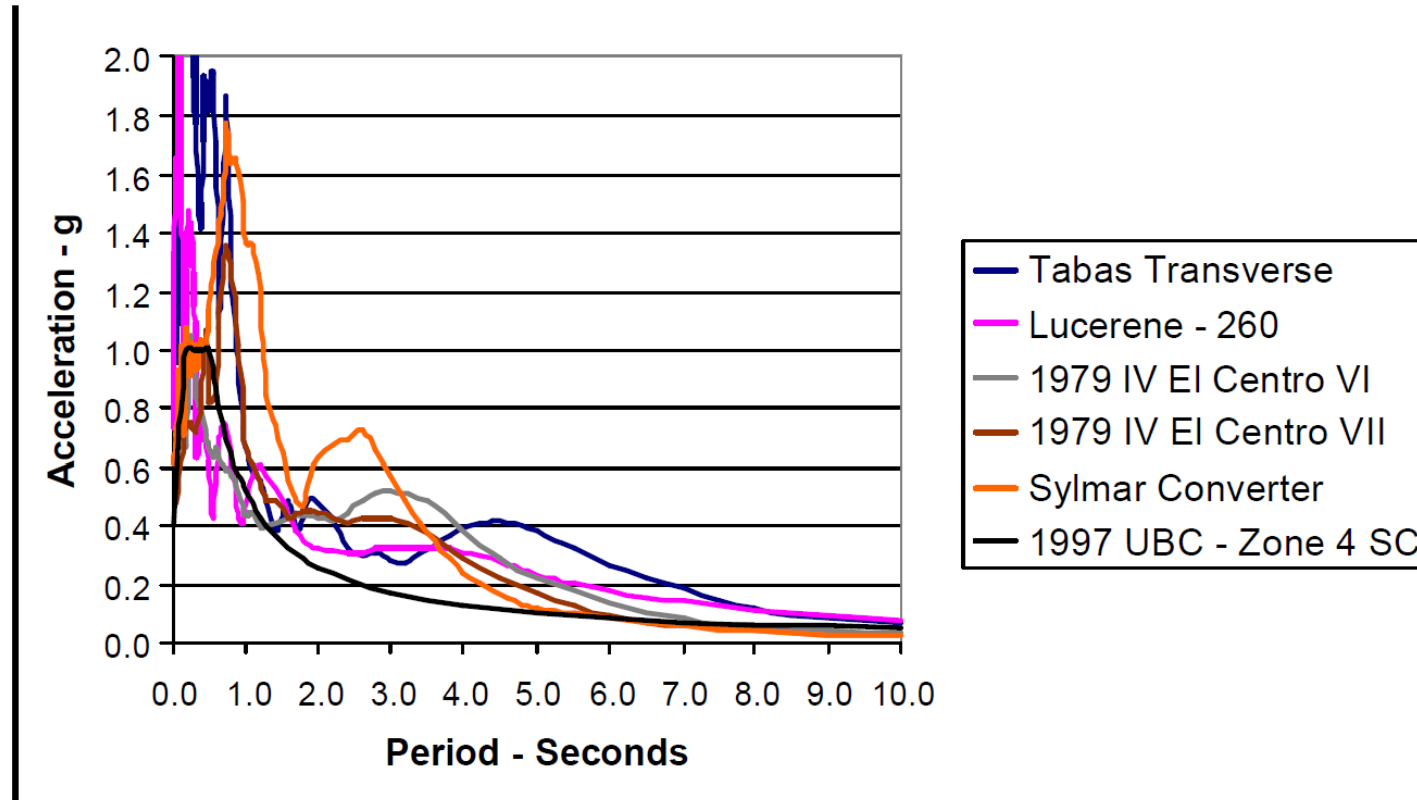


Figure 1 – Comparison of Near Fault Records and 1997 UBC Zone 4 Acceleration Response Spectrum

Background regarding 1997 UBC Eq. 30-7 and Drift

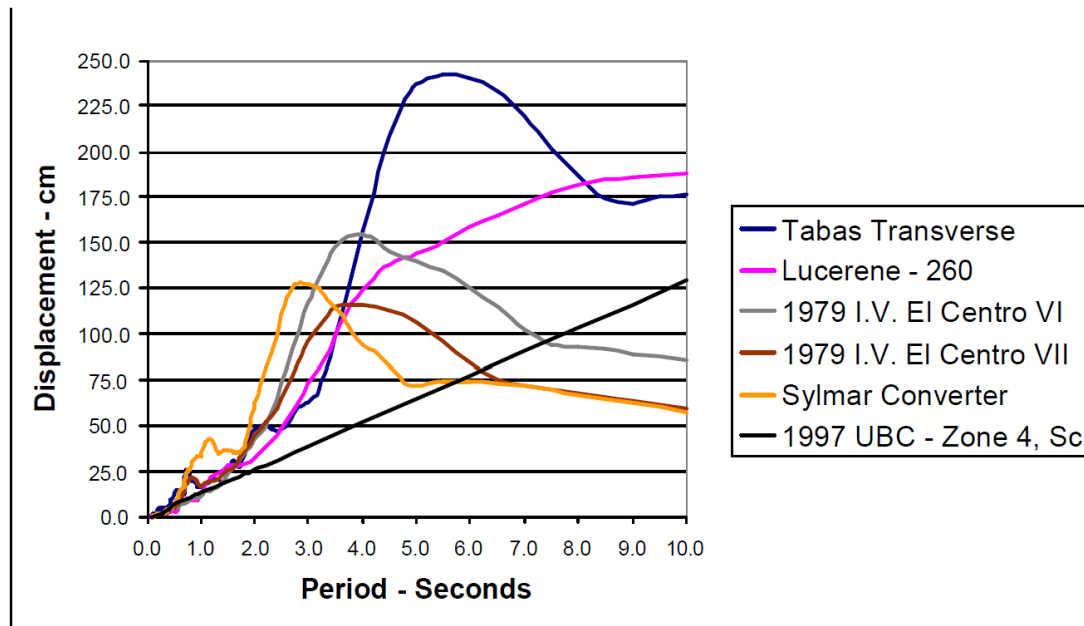


Figure 2 – Comparison of Near Fault Records and 1997 UBC Zone 4 – Displacement Response Spectrum

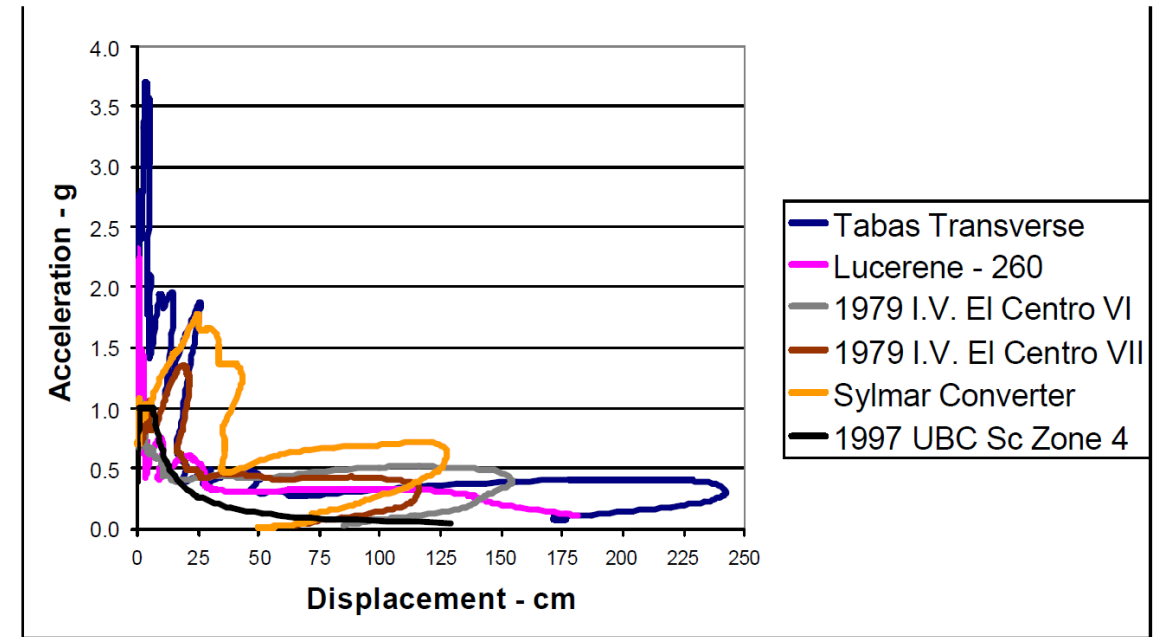


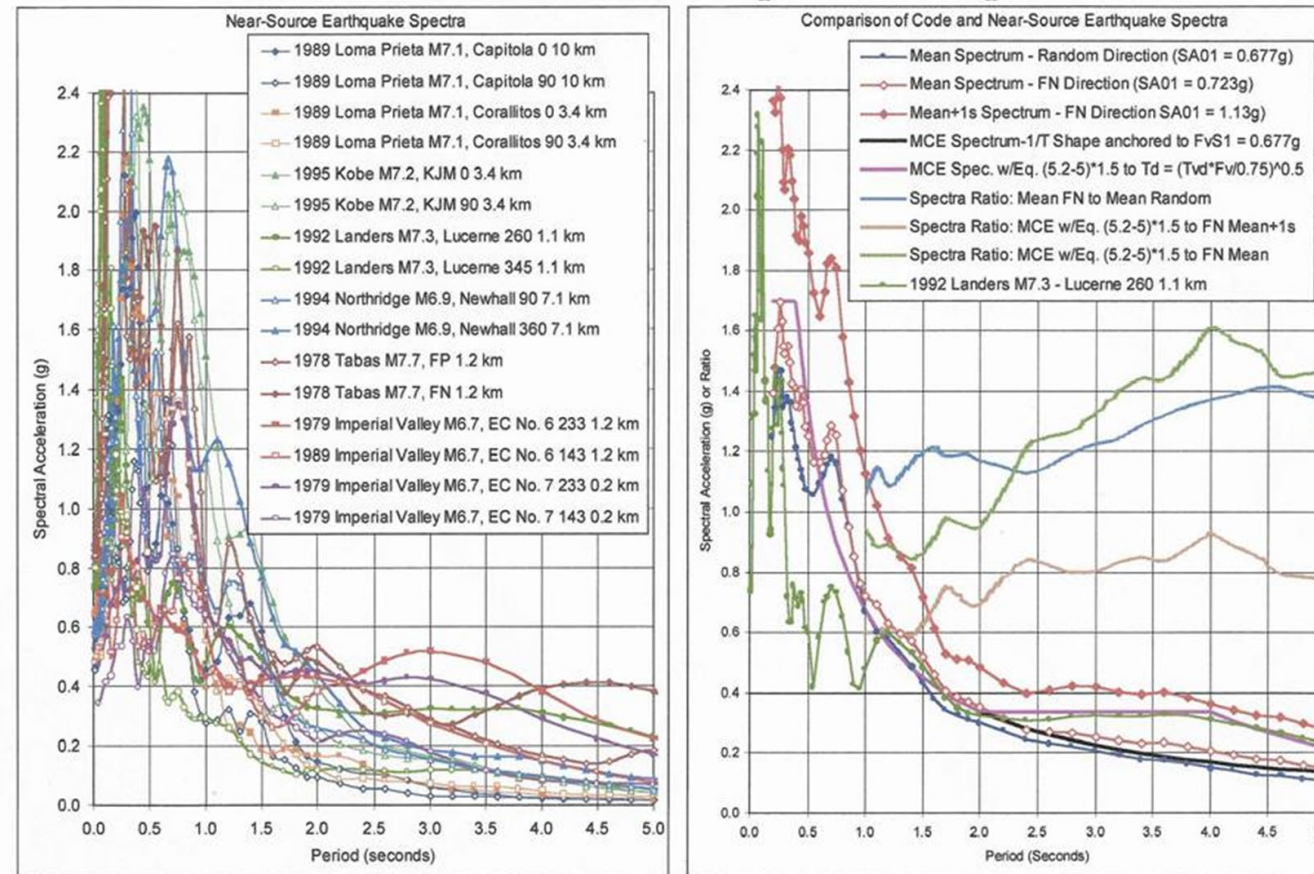
Figure 3 – Comparison of Near Fault Records and 1997 UBC Zone Spectrum in Acceleration – Displacement (ADRS) format

Near Source Factors – Comparison of Spectra

Kircher & Associates
Consulting Engineers

Near-Source Ground Motions
April 2003

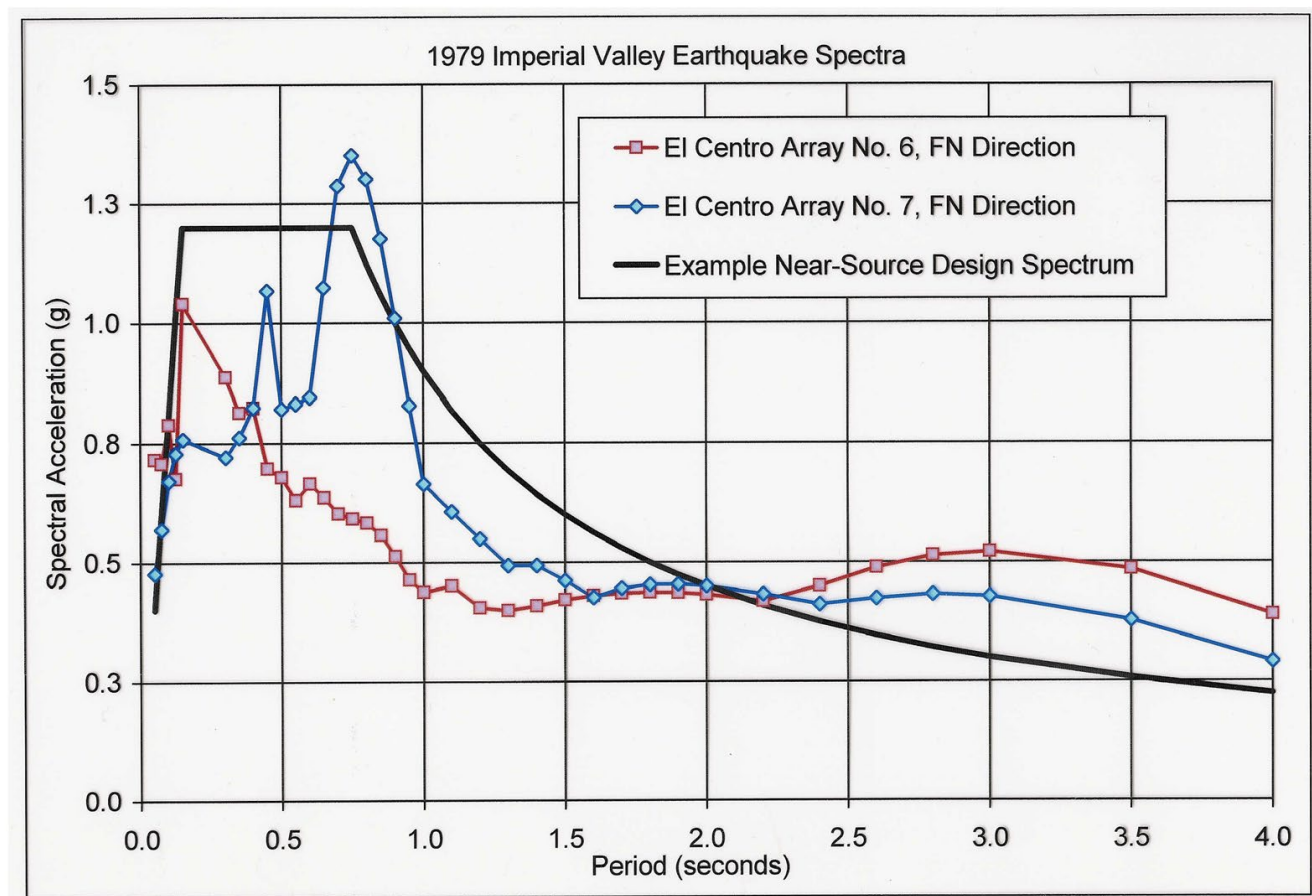
Near-Source Factors - Comparison of Spectra



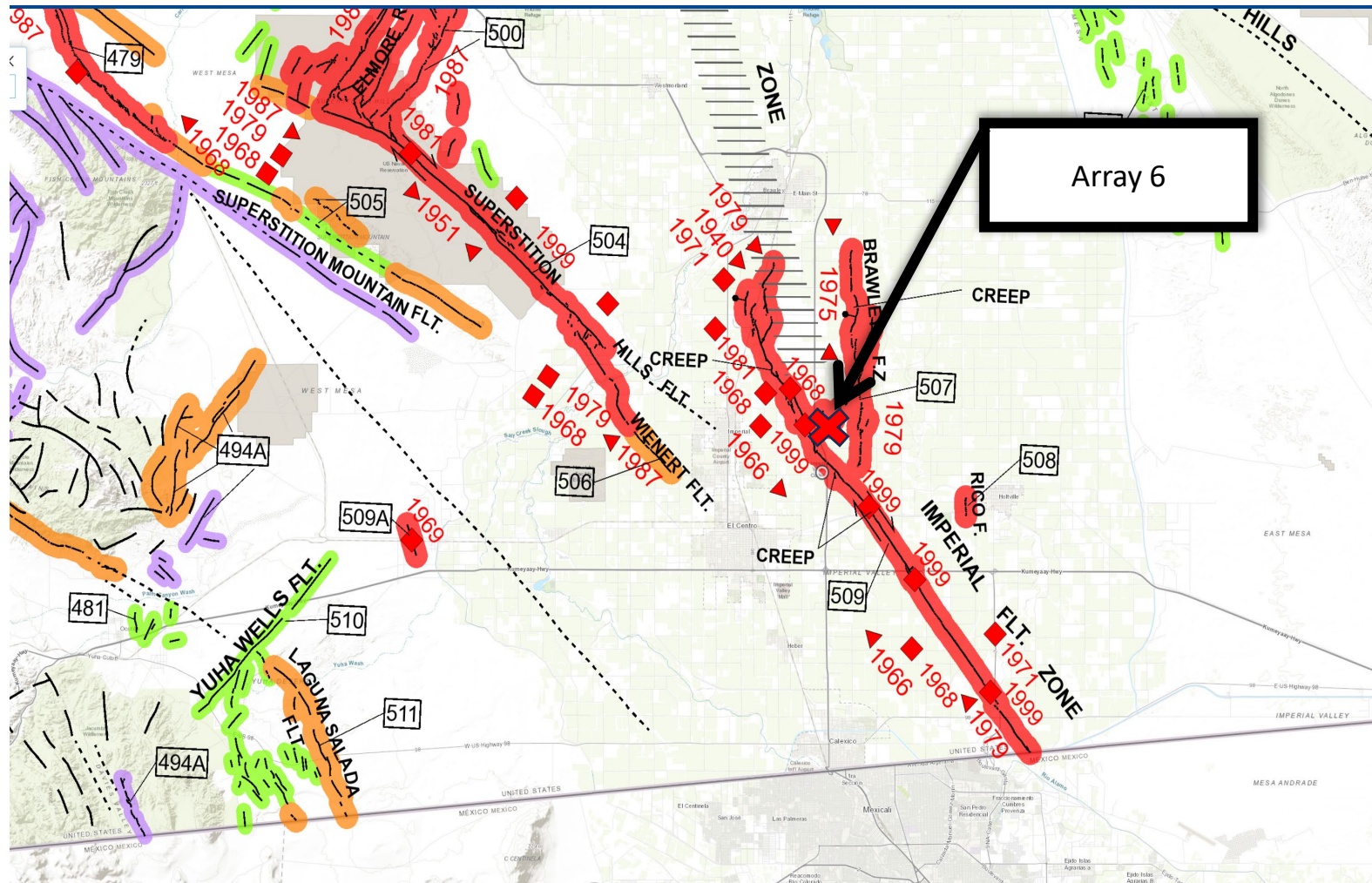
1979 Imperial Valley – Comparison of Spectra

Kircher & Associates
Consulting Engineers

Near-Source Ground Motions
April 2003



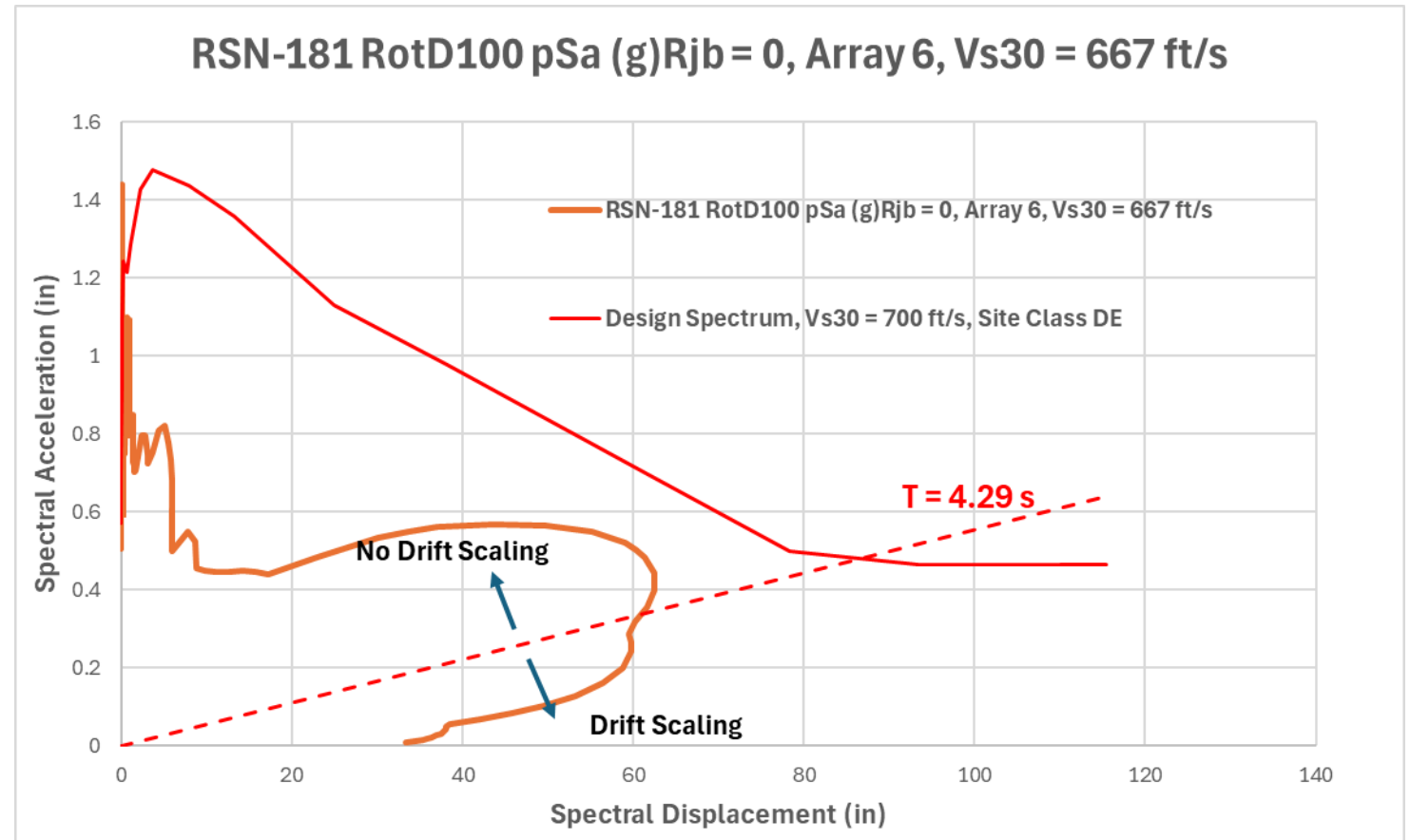
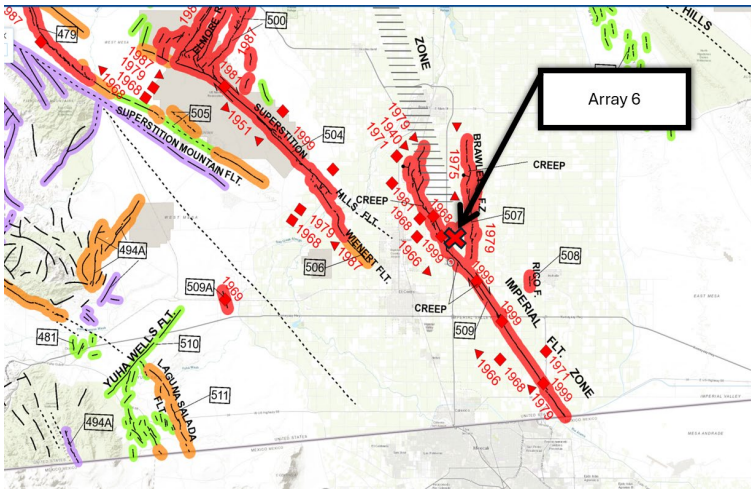
1979 Imperial Valley – Array 6



1979 Imperial Valley – Array 6

Comparison with ASCE 7-22 Design Spectrum

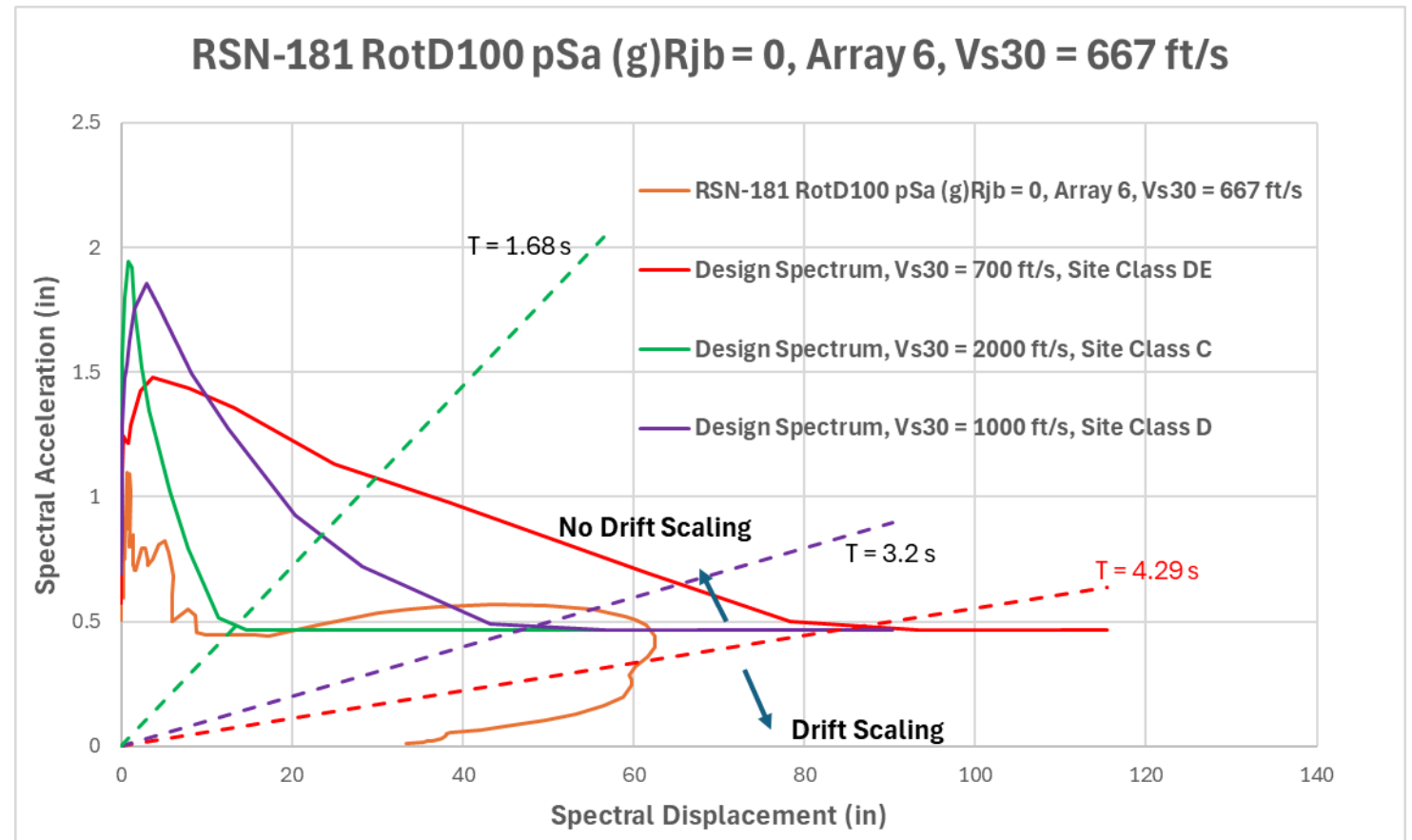
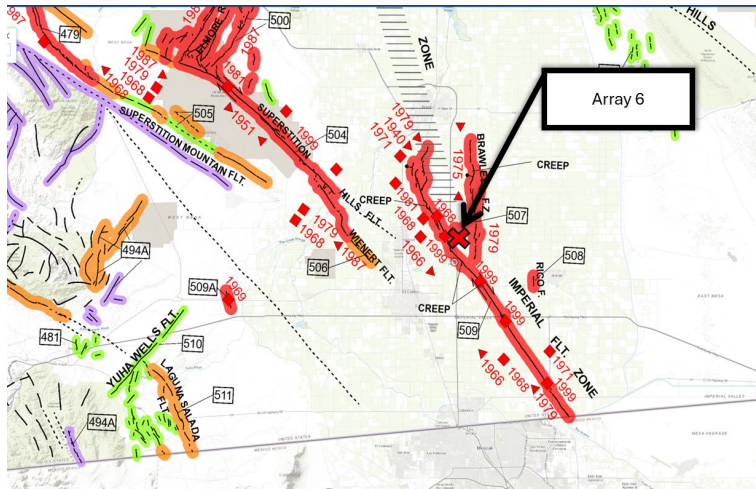
- $S_1 = 0.93$ g
- $S_{Ds} = 1.25$ g
- $S_{D1} = 2.12$ g
- $T \geq 4.29$ s Drift Scaling Required



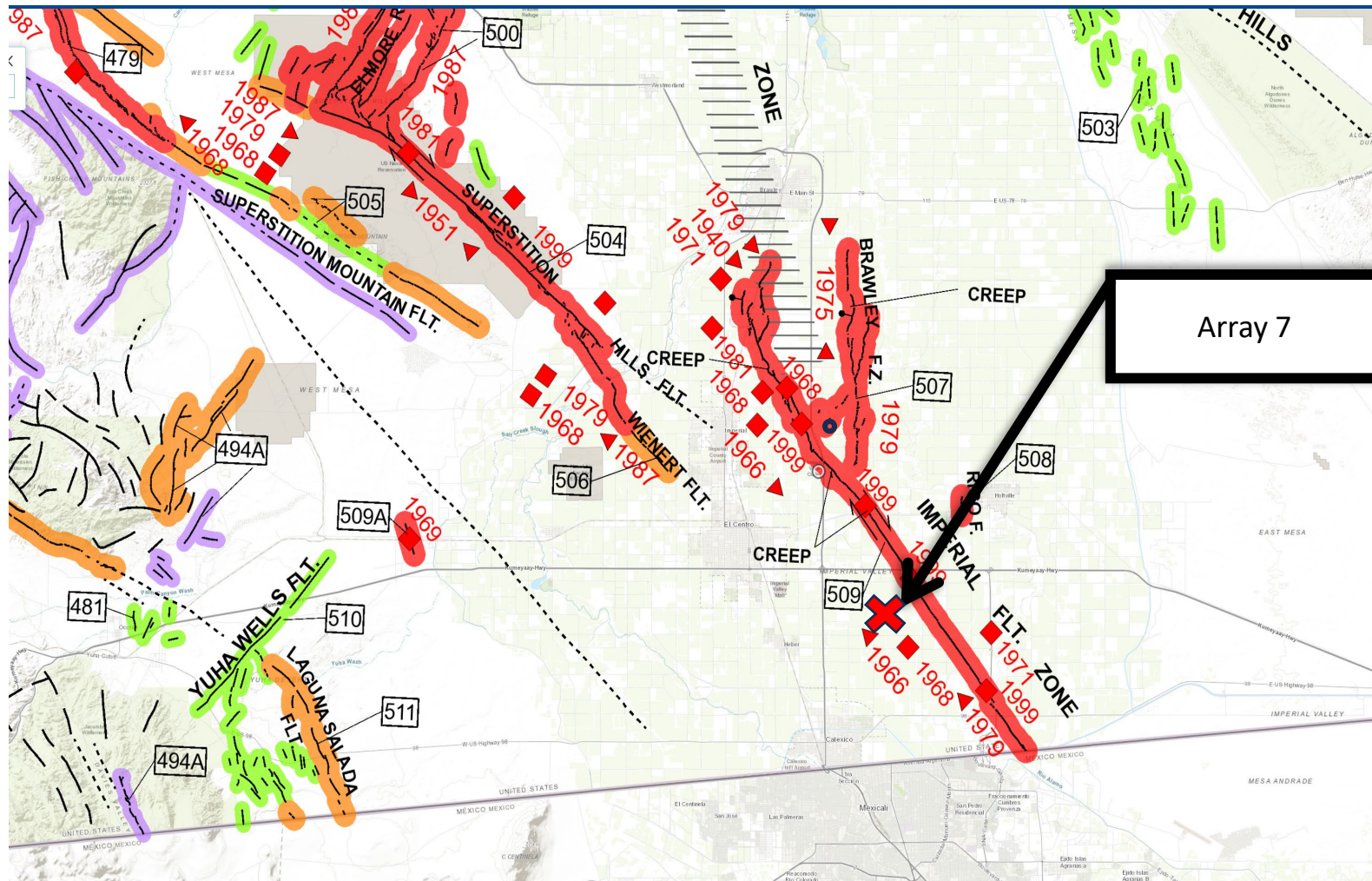
1979 Imperial Valley – Array 6

Comparison with ASCE 7-22 Design Spectrum

- $S_1 = 0.93 \text{ g}$
- $S_{Ds} = 1.25 \text{ g}$
- $S_{D1} = 2.12 \text{ g}$
- $C_u T_a \geq 4.29 \text{ s}$ Drift Scaling Required

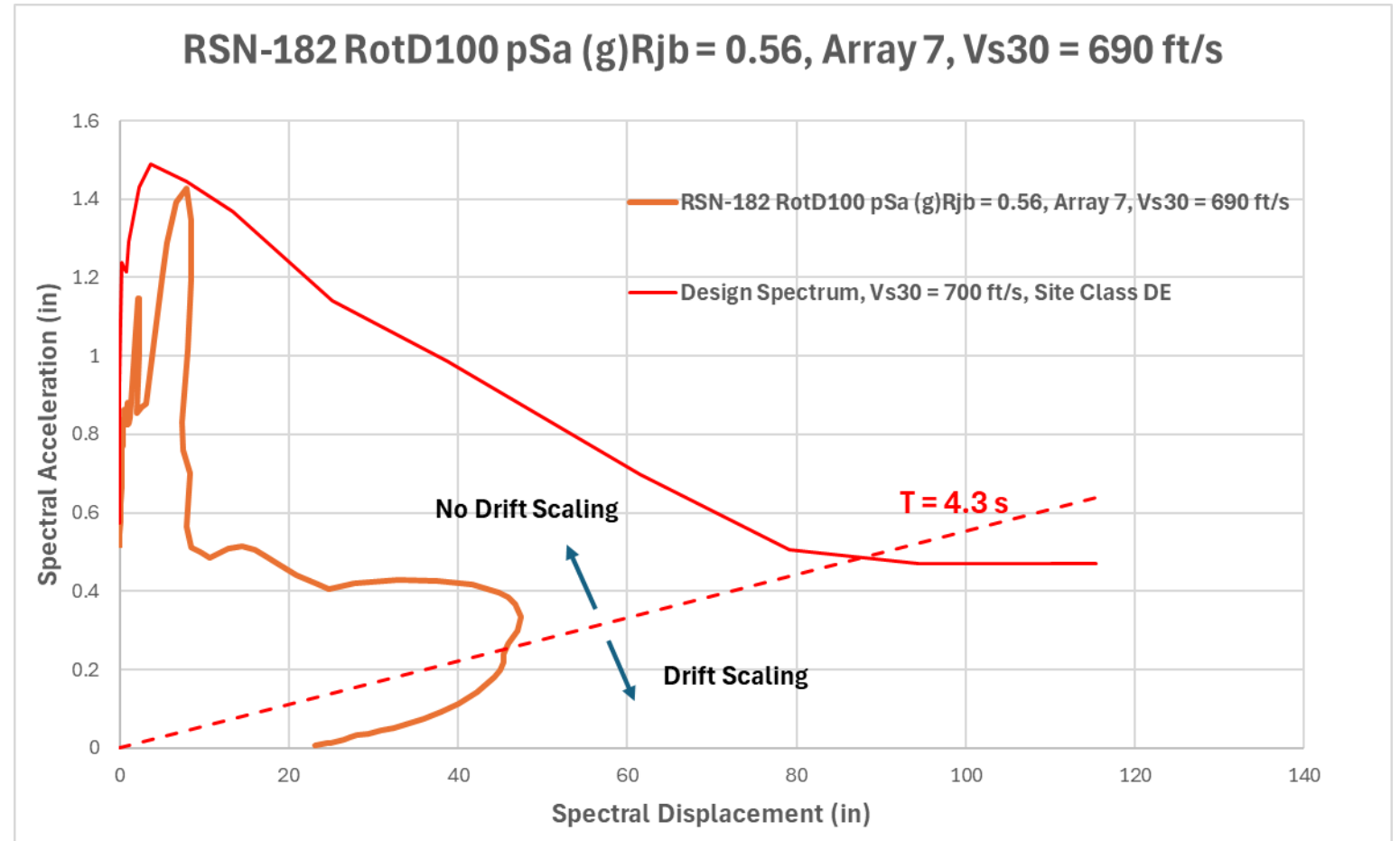
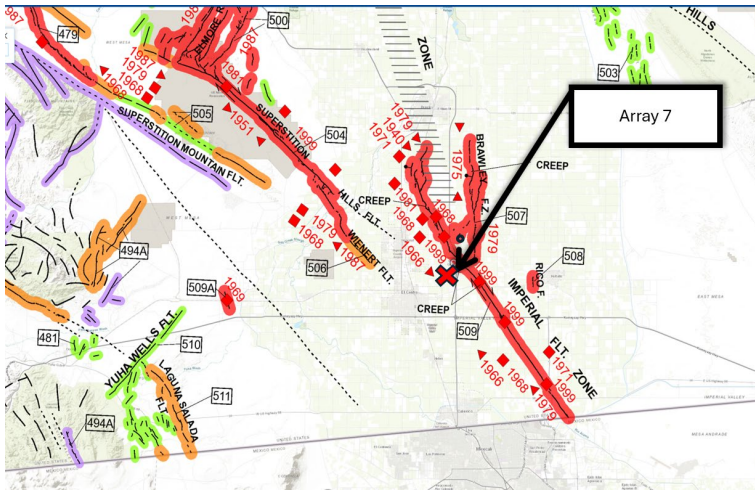


1979 Imperial Valley – Array 7



1979 Imperial Valley – Array 7 Comparison with ASCE 7-22 Design Spectrum

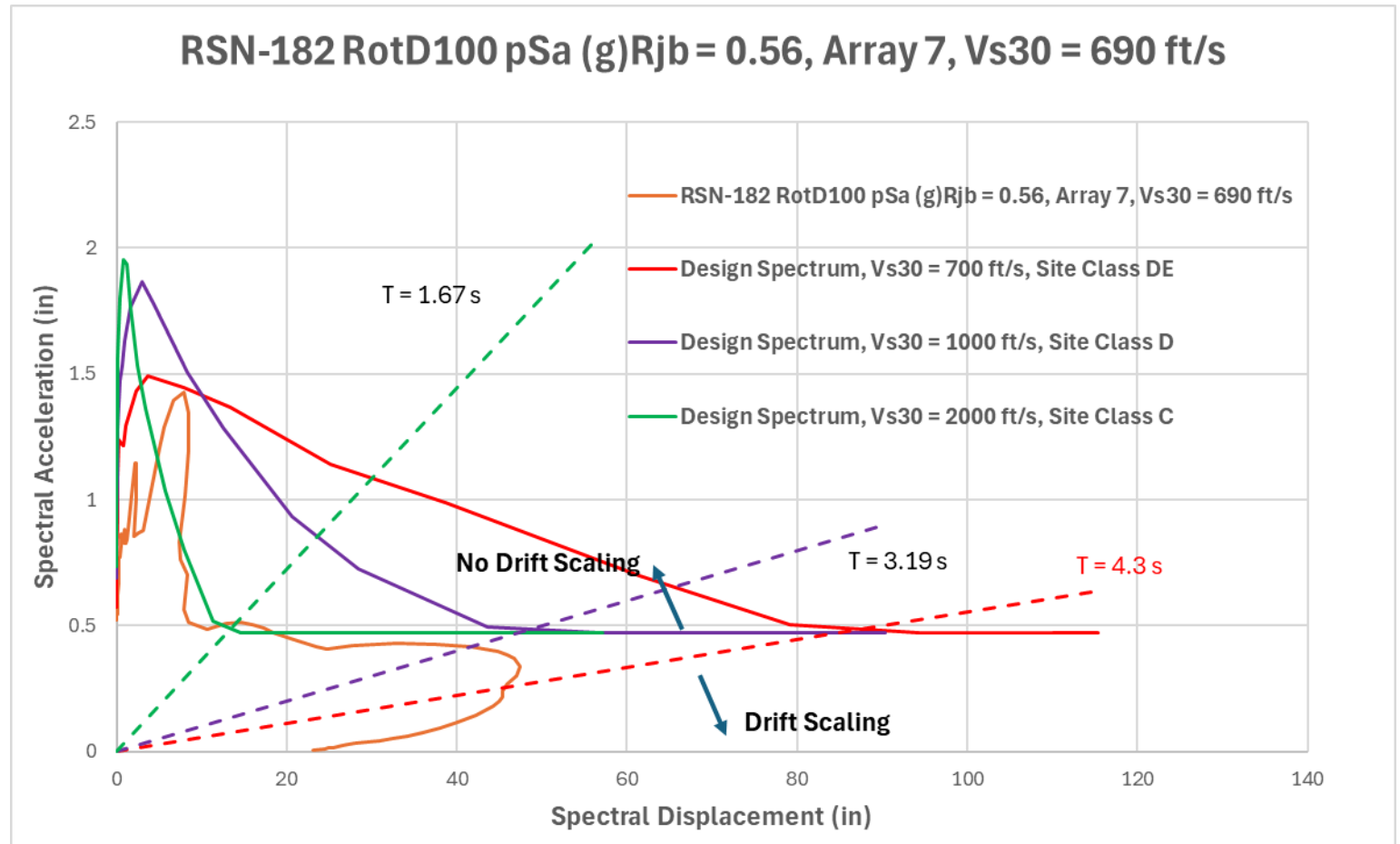
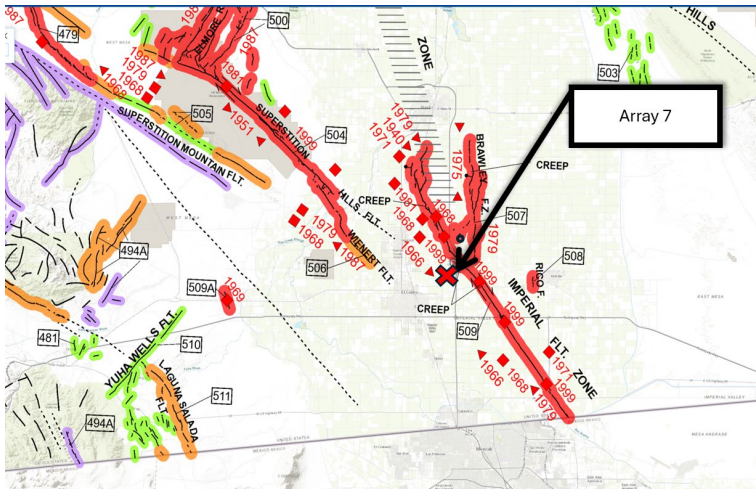
- $S_1 = 0.94 \text{ g}$
- $S_{Ds} = 1.25 \text{ g}$
- $S_{D1} = 2.13 \text{ g}$
- $C_u T_a \geq 4.3 \text{ s}$ Drift Scaling Required



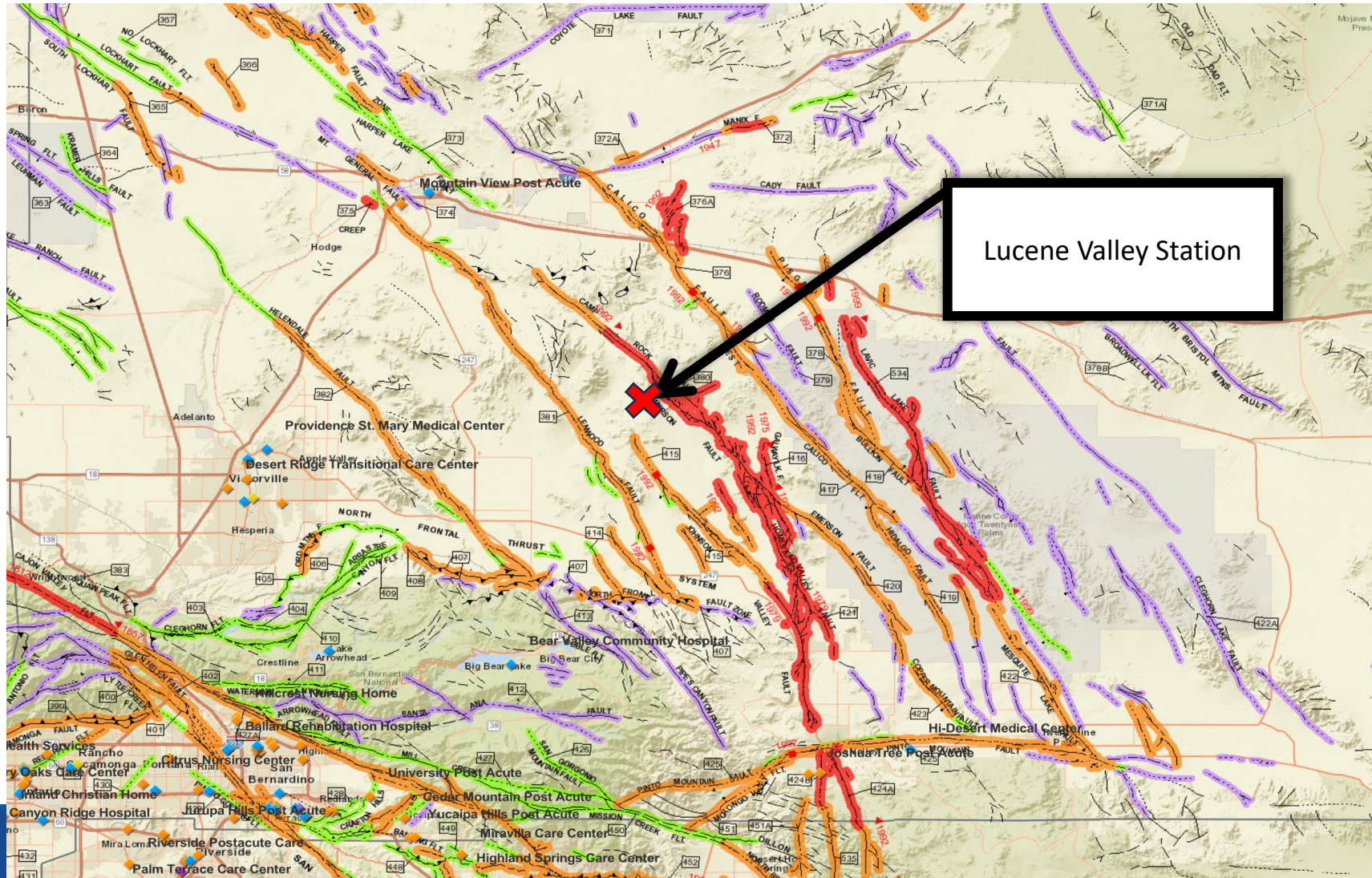
1979 Imperial Valley – Array 7

Comparison with ASCE 7-22 Design Spectrum

- $S_1 = 0.94$ g
- $S_{Ds} = 1.25$ g
- $S_{D1} = 2.13$ g
- $C_u T_a \geq 4.3$ s Drift Scaling Required

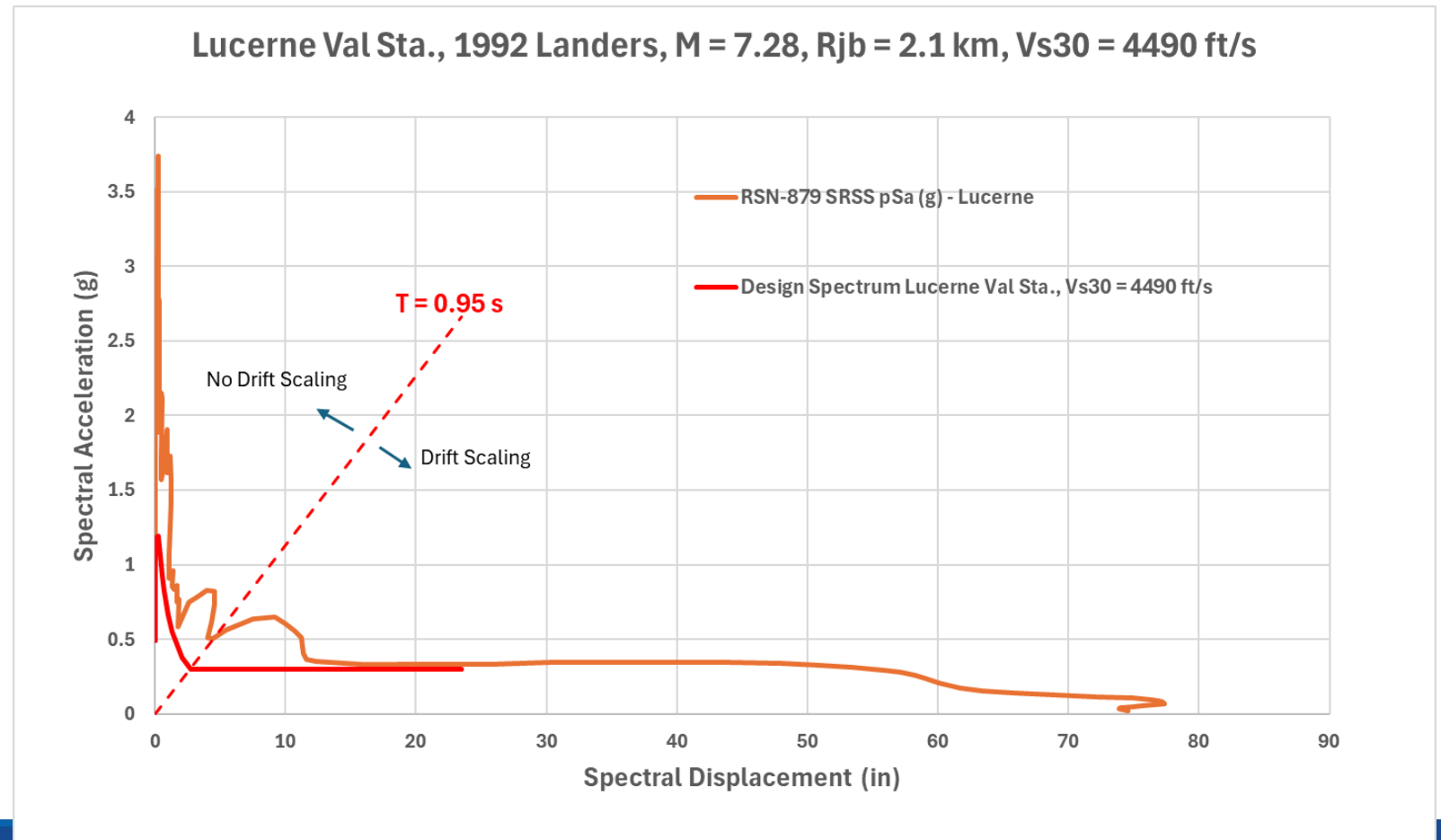
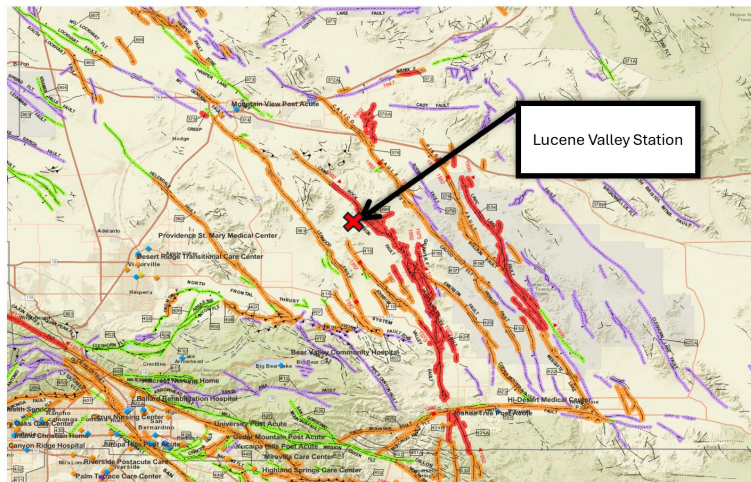


1992 Landers Eqk. – Lucerne Valley Station



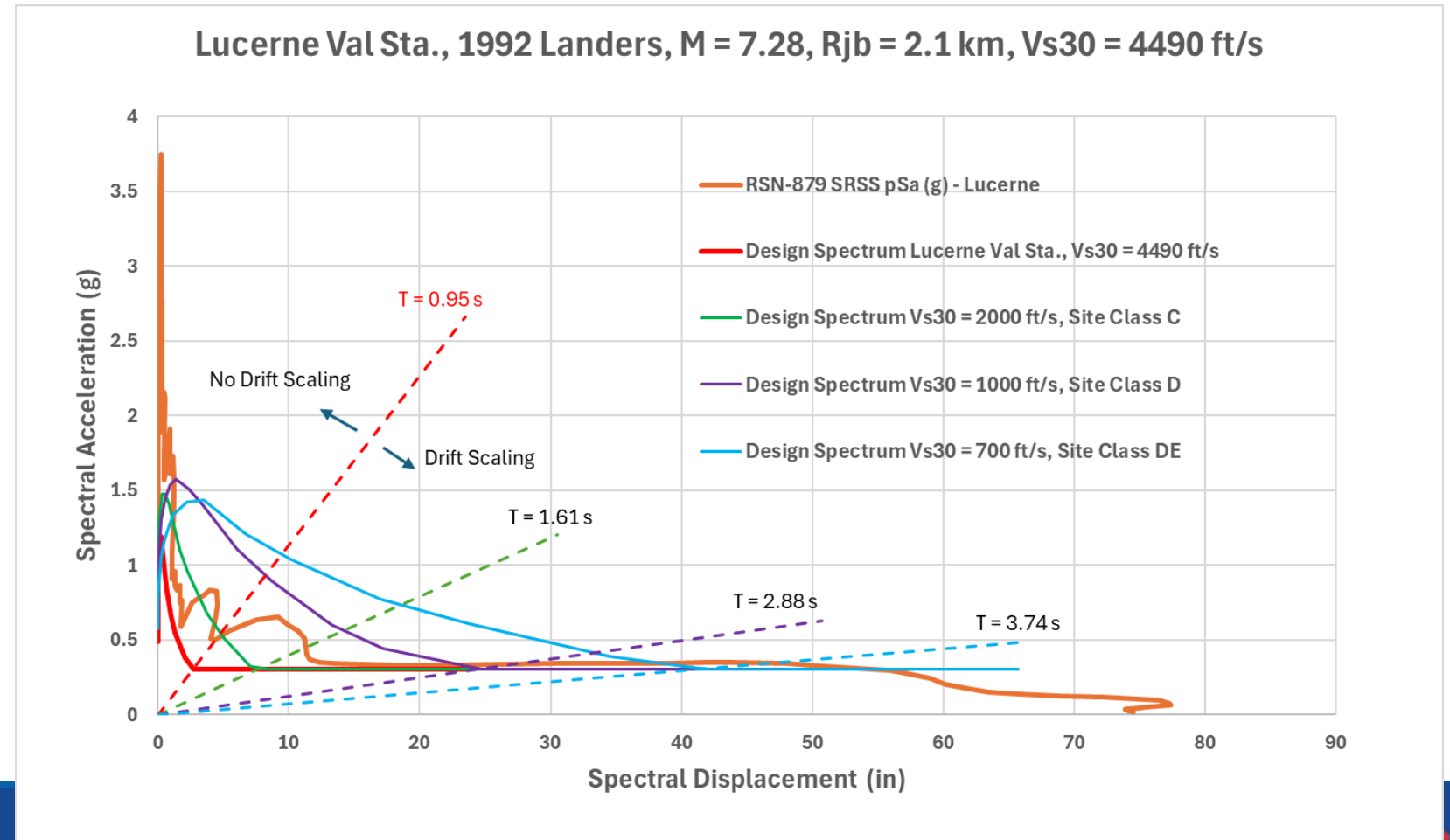
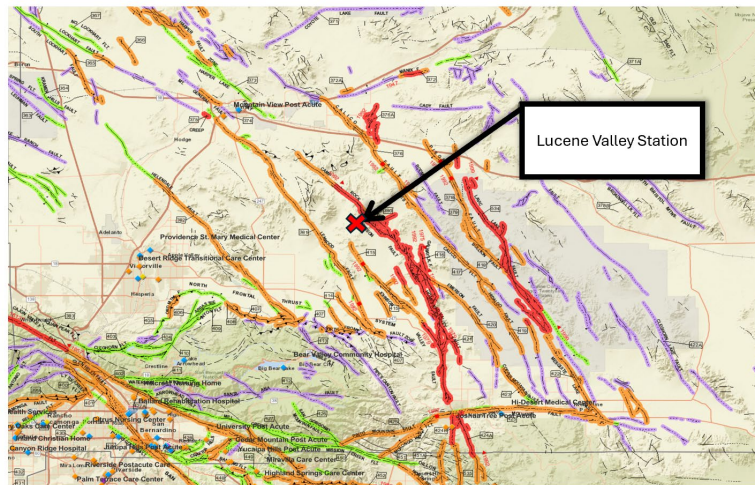
1992 Landers Eeqk. – Lucerne Valley Station Comparison with ASCE 7-22 Design Spectrum

- $S_1 = 0.6 \text{ g}$
- $S_{Ds} = 0.96 \text{ g}$
- $S_{D1} = 0.28 \text{ g}$
- $C_u T_a \geq 0.95 \text{ s}$ Drift Scaling Required



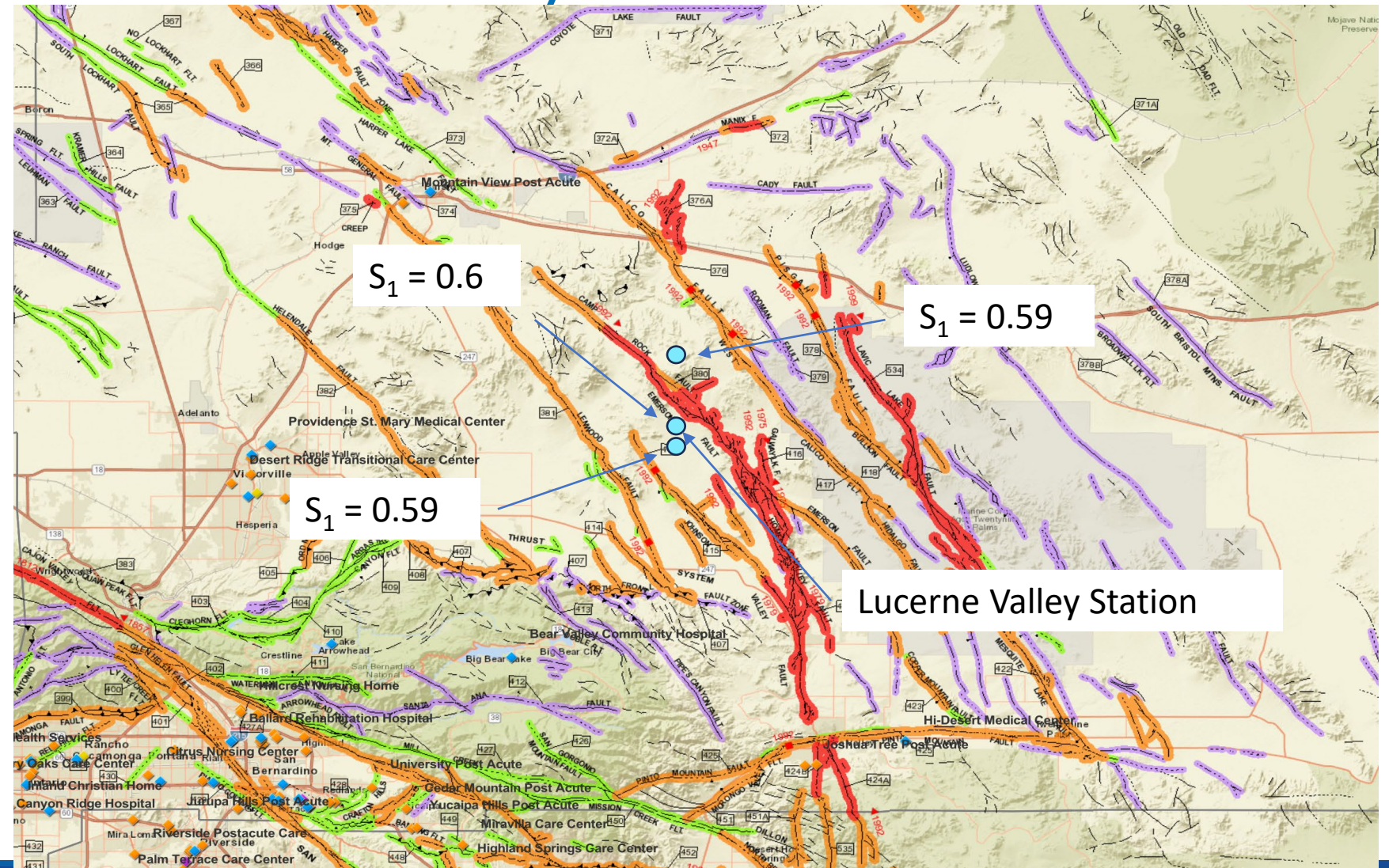
1992 Landers Eeqk. – Lucerne Valley Station Comparison with ASCE 7-22 Design Spectrum

- $S_1 = 0.6 \text{ g}$
- $S_{Ds} = 0.96 \text{ g}$
- $S_{D1} = 0.28 \text{ g}$
- $C_u T_a \geq 0.95 \text{ s}$ Drift Scaling Required

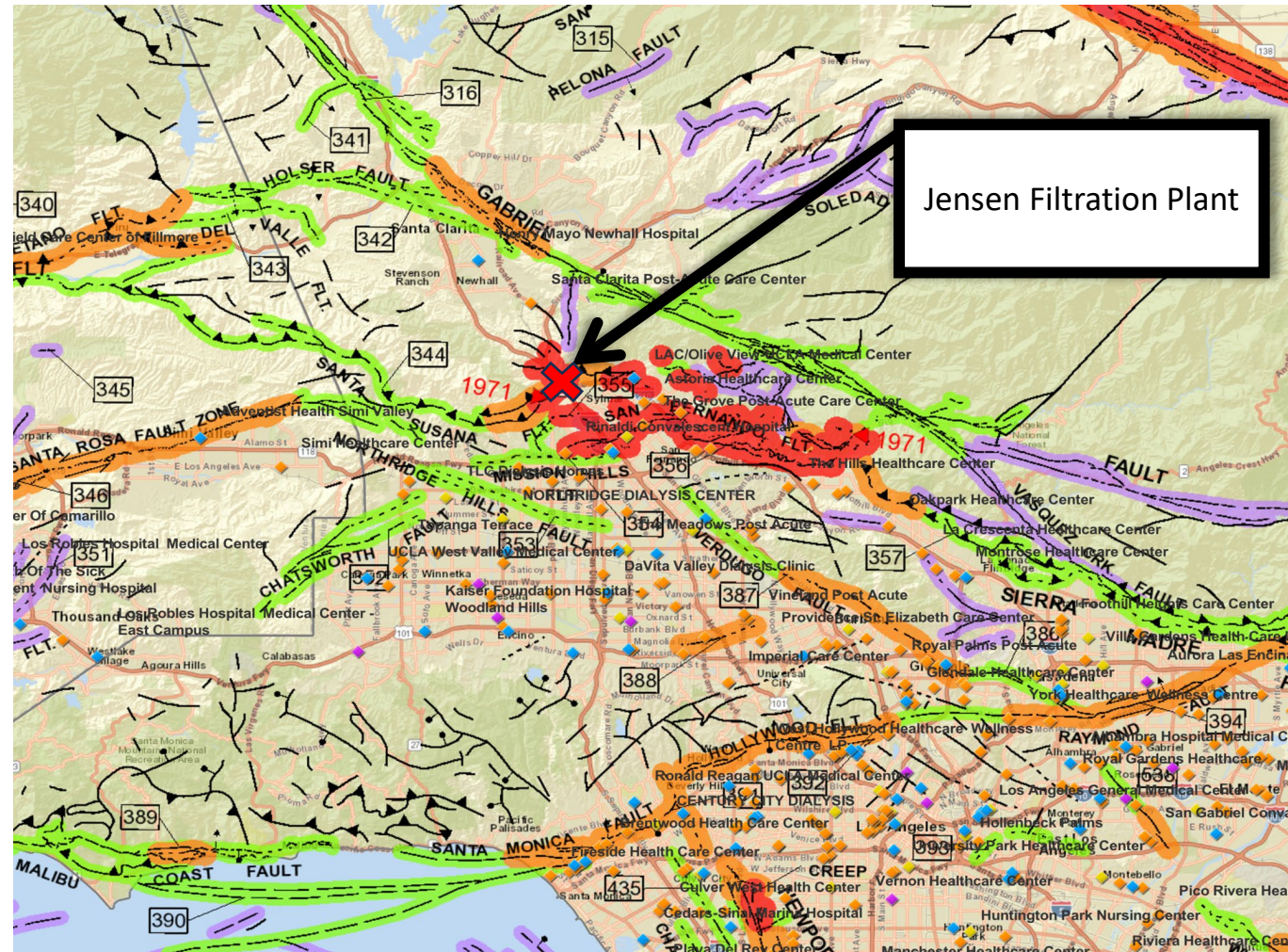


S_1 values near Lucerne Valley Station

- $S_1 < 0.6$ g No drift scaling

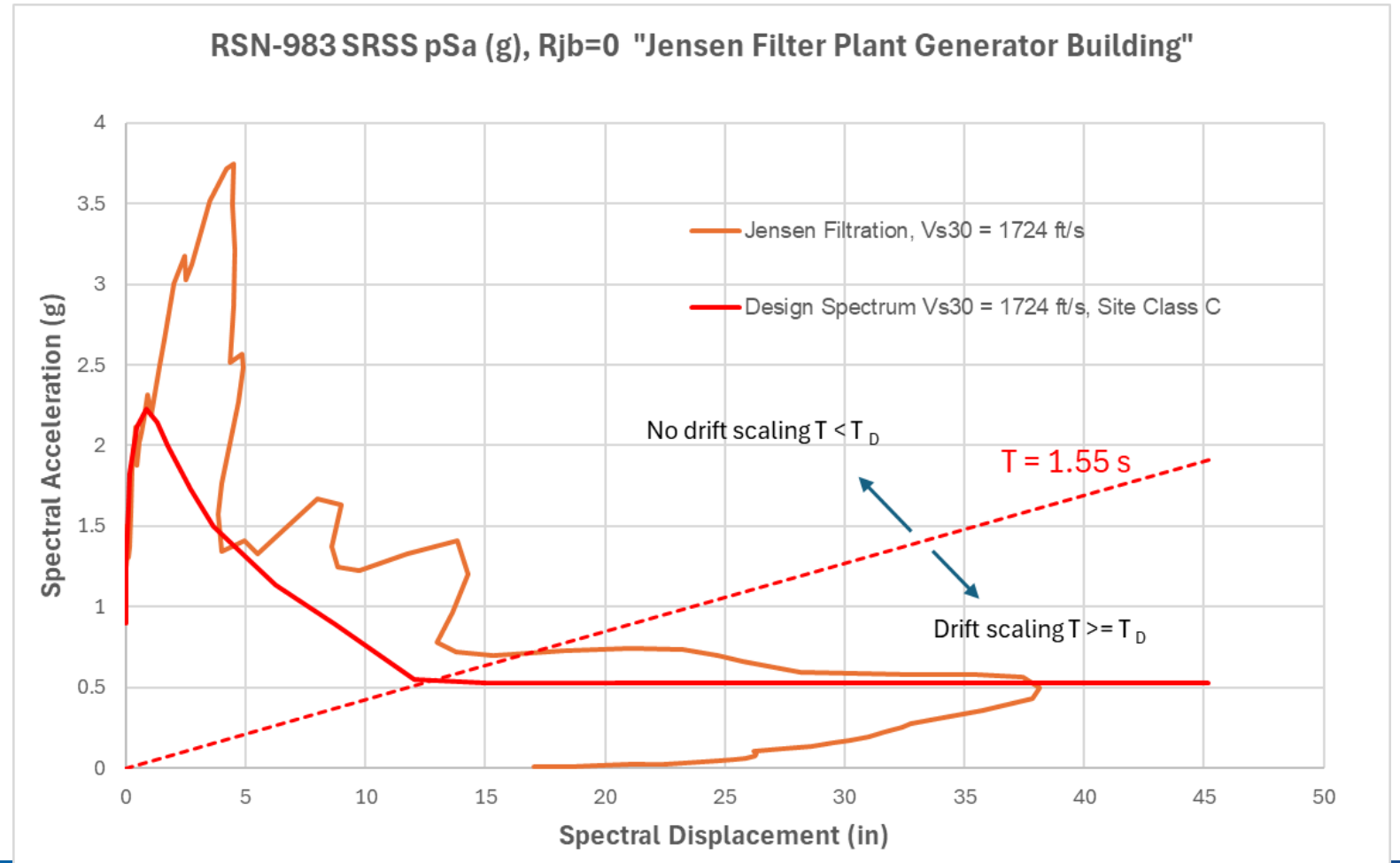
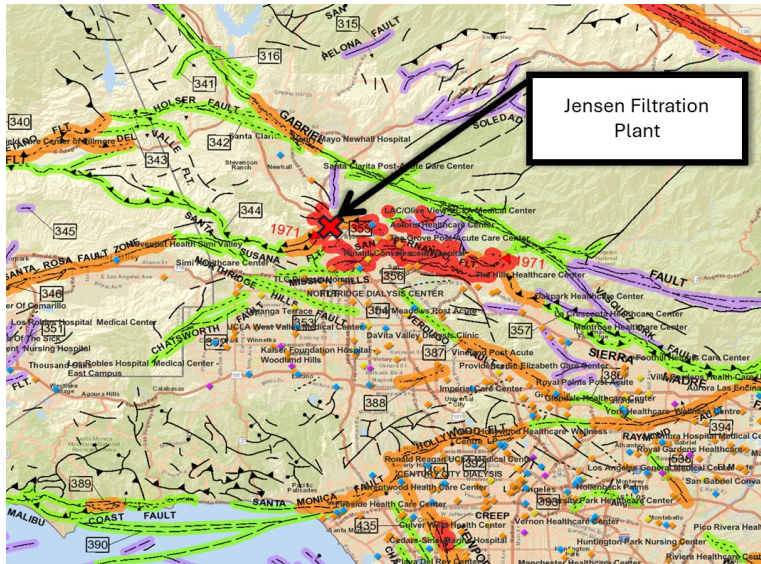


1994 Northridge – Jensen Filtration Plant Generator Building



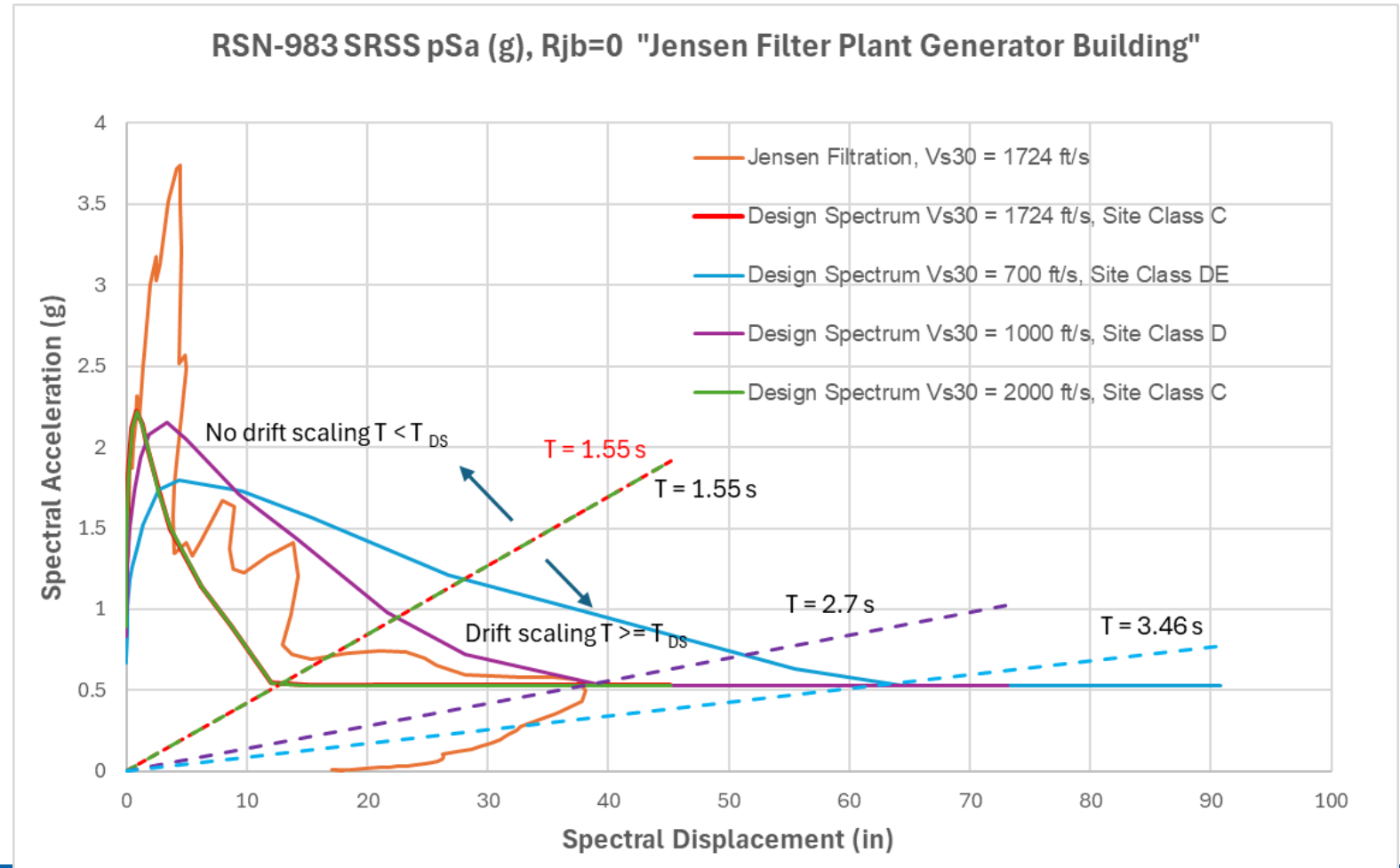
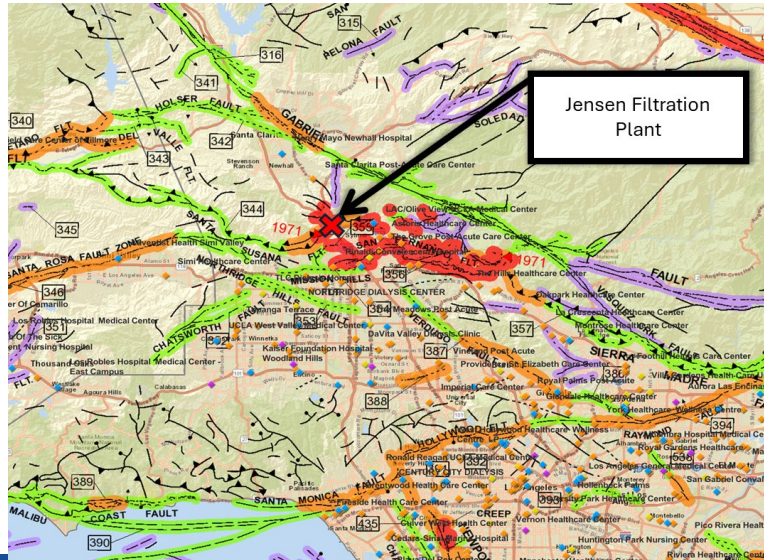
Jensen Filtration Plant Generator Building Comparison with ASCE 7-22 Design Spectrum

- $S_1 = 1.06 \text{ g}$
- $S_{Ds} = 2.08 \text{ g}$
- $S_{D1} = 0.99 \text{ g}$
- $C_u T_a \geq 1.55 \text{ s}$ Drift Scaling Required

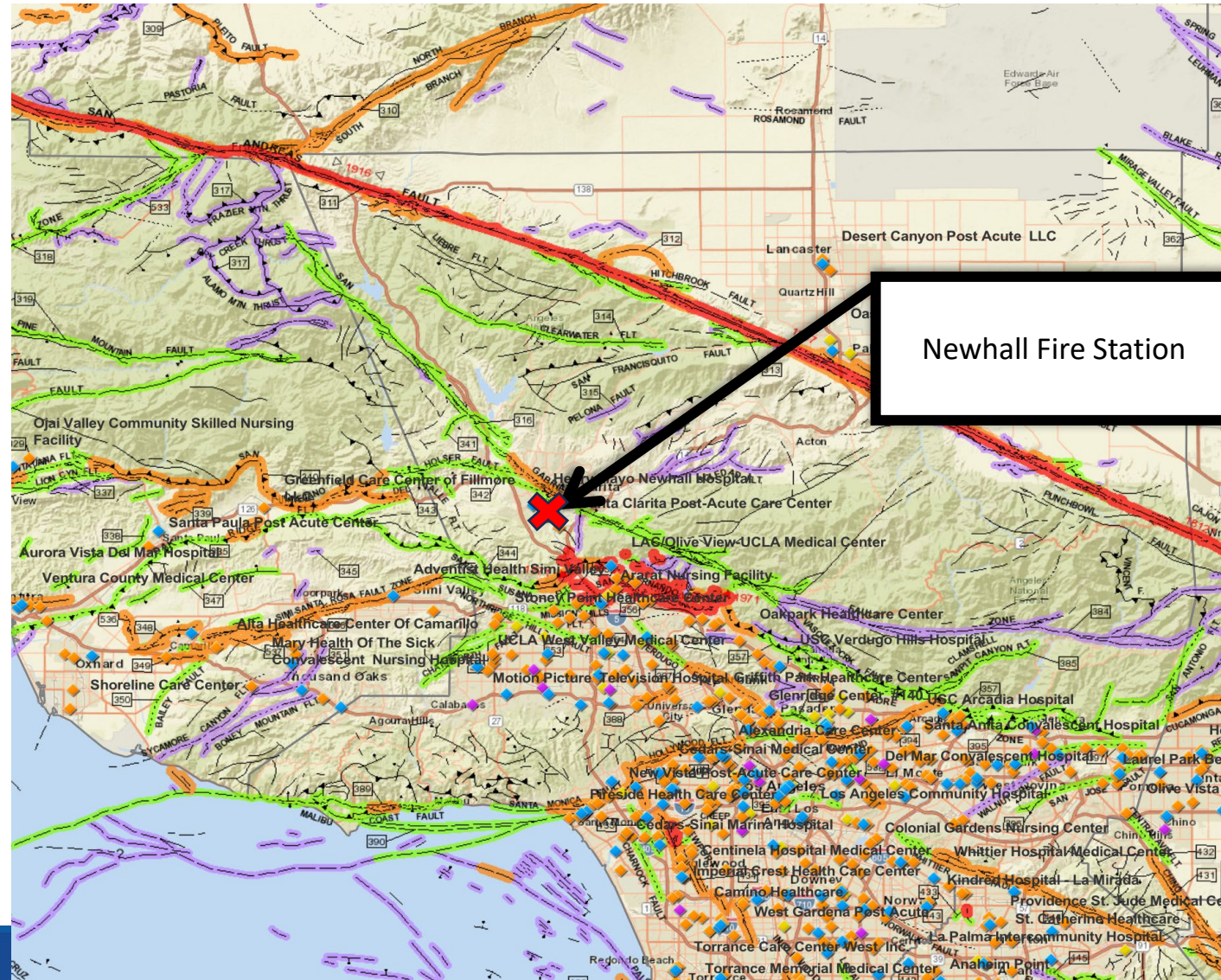


Jensen Filtration Plant Generator Building Comparison with ASCE 7-22 Design Spectrum

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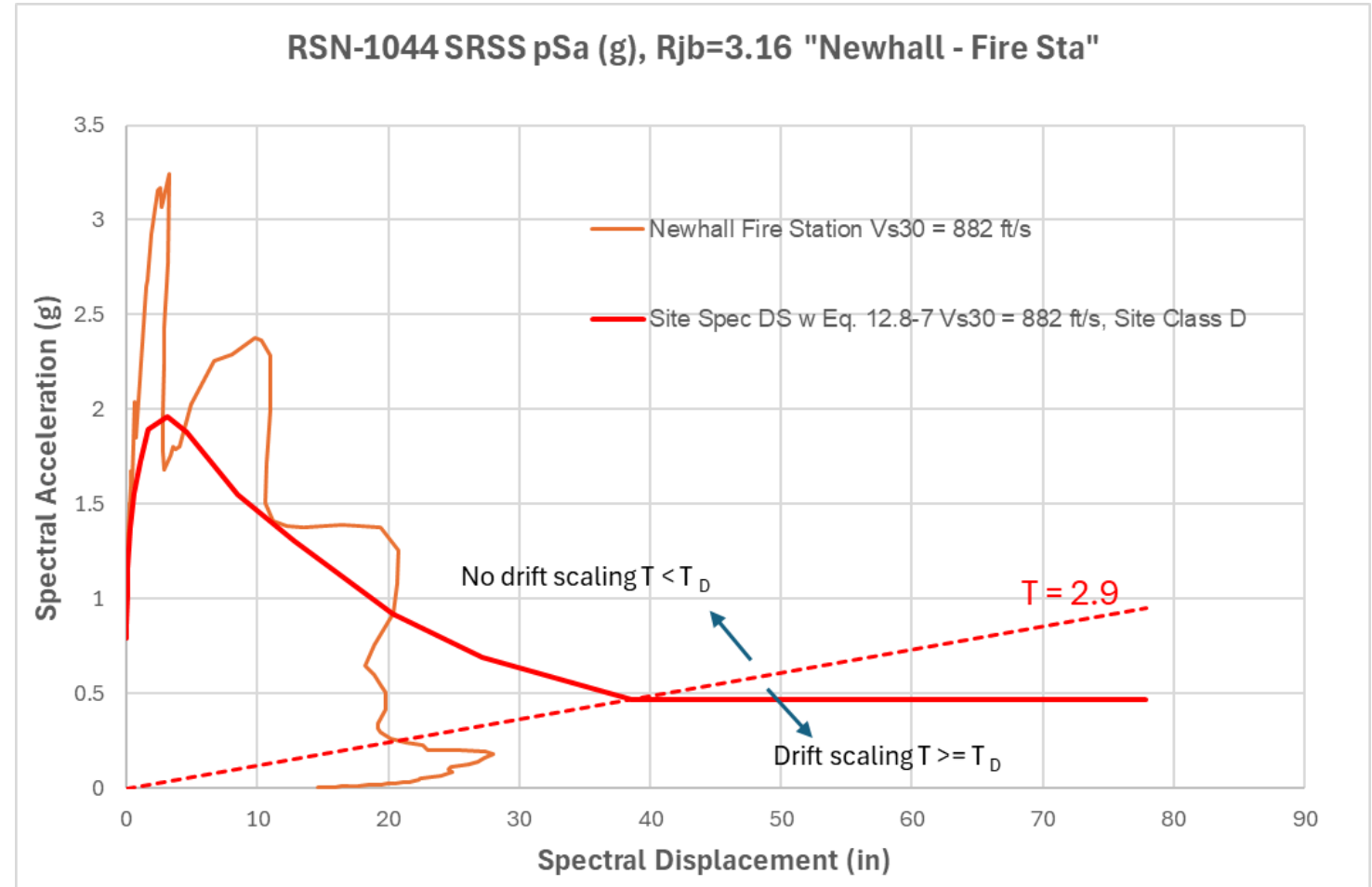
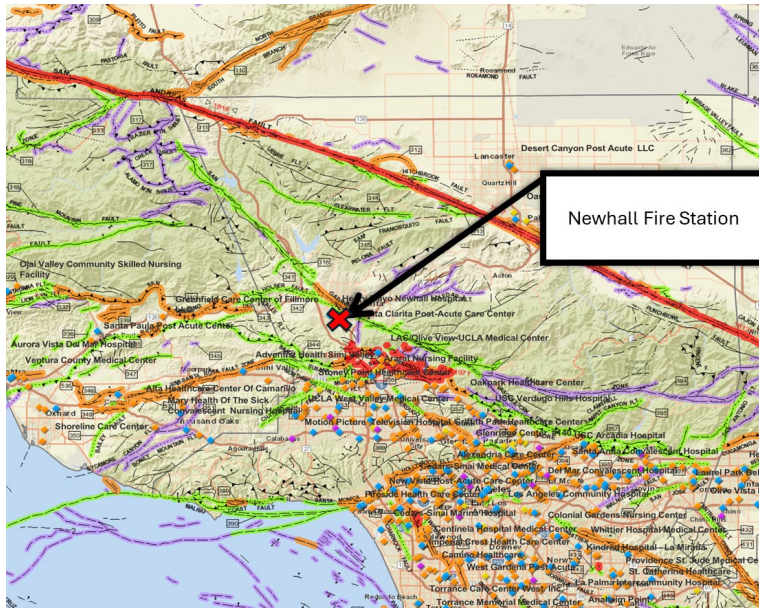


1994 Northridge – Newhall Fire Station



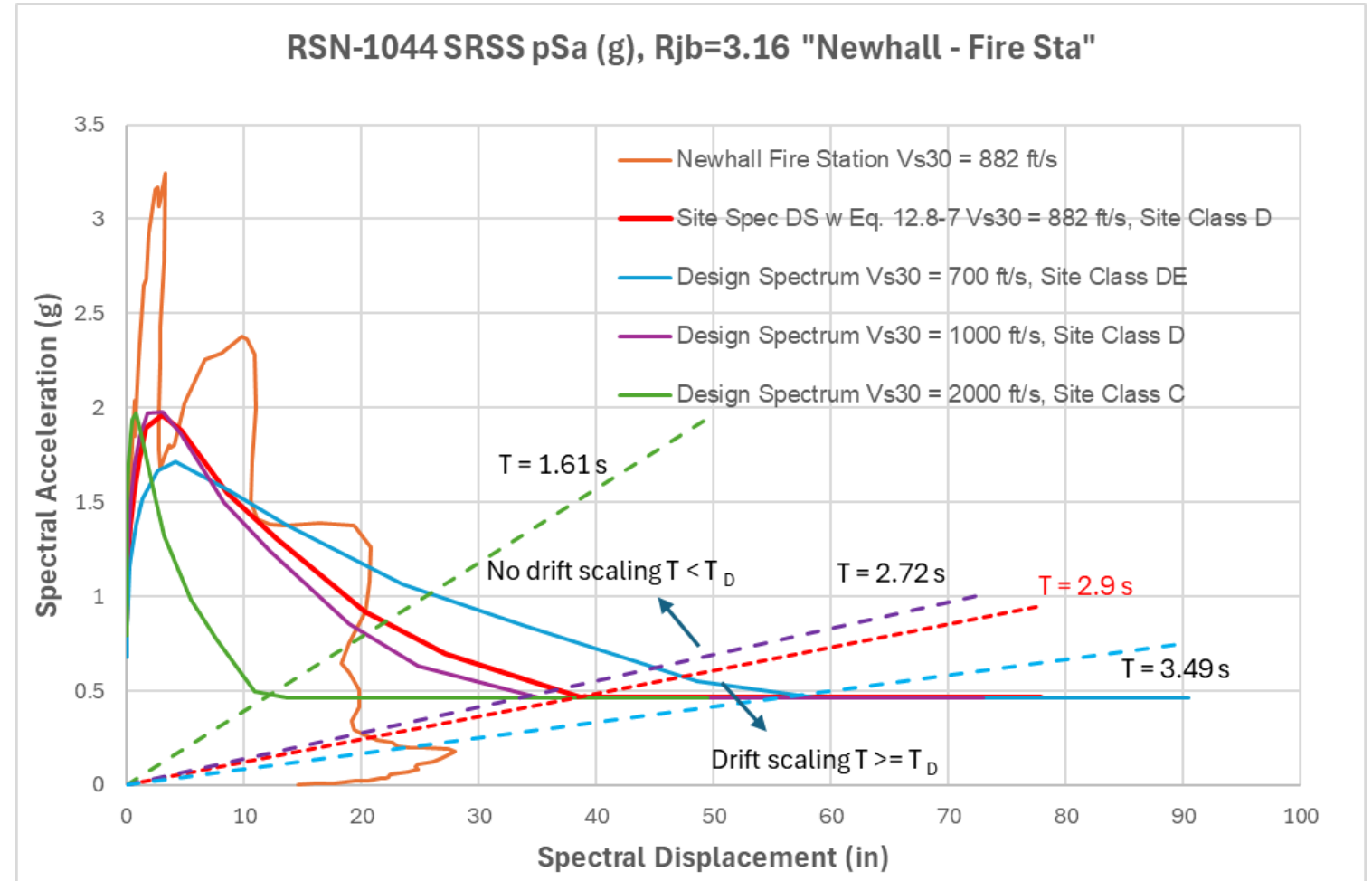
1994 Northridge – Newhall Fire Station Comparison with ASCE 7-22 Design Spectrum

- $S_1 = 0.93 \text{ g}$
- $S_{Ds} = 1.85 \text{ g}$
- $S_{D1} = 0.86 \text{ g}$
- $C_u T_a \geq 2.9 \text{ s}$ Drift Scaling Required

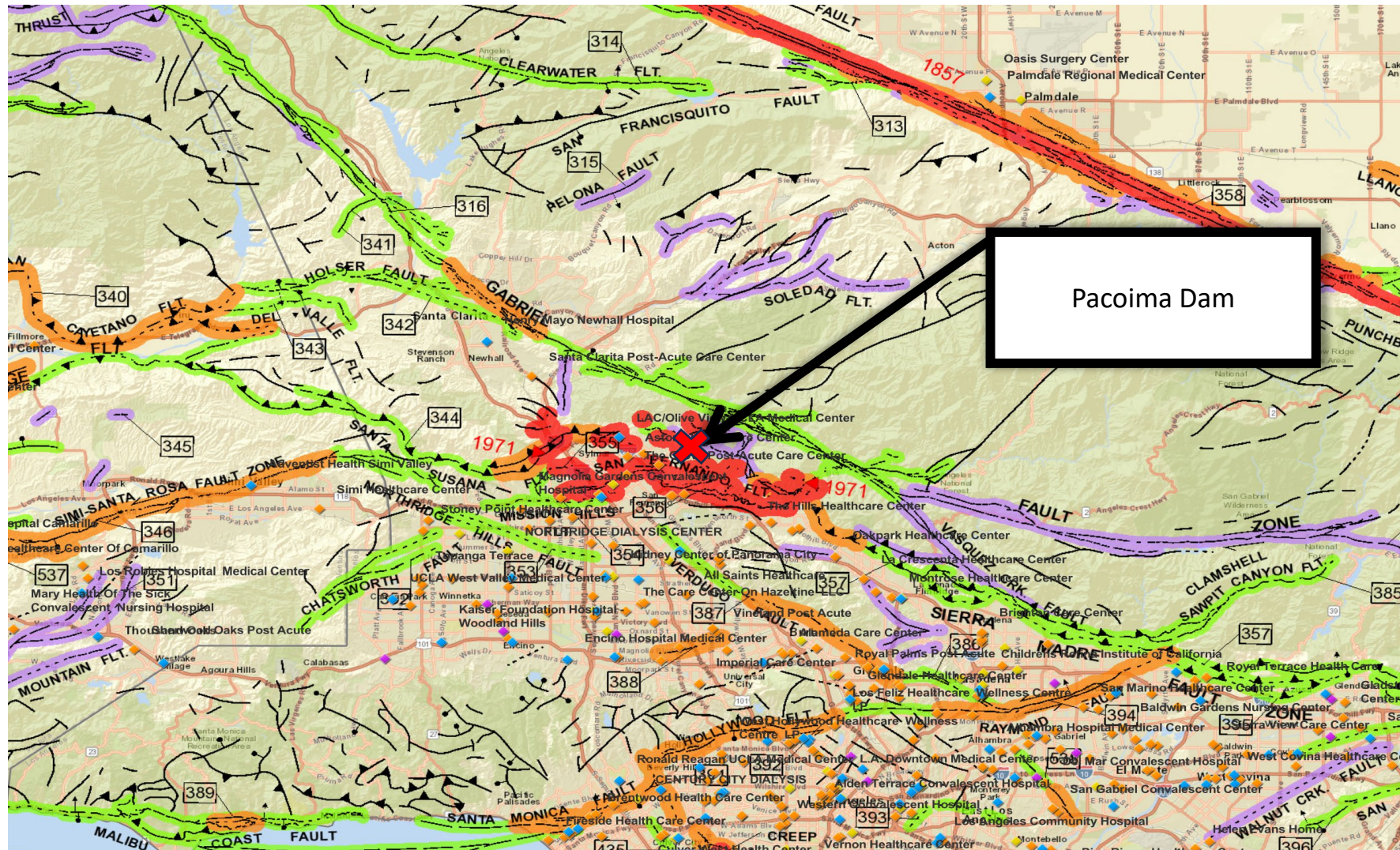


1994 Northridge – Newhall Fire Station Comparison with ASCE 7-22 Design Spectrum

- $S_1 = 0.93 \text{ g}$
- $S_{Ds} = 1.85 \text{ g}$
- $S_{D1} = 0.86 \text{ g}$
- $C_u T_a \geq 2.9 \text{ s}$ Drift Scaling Required

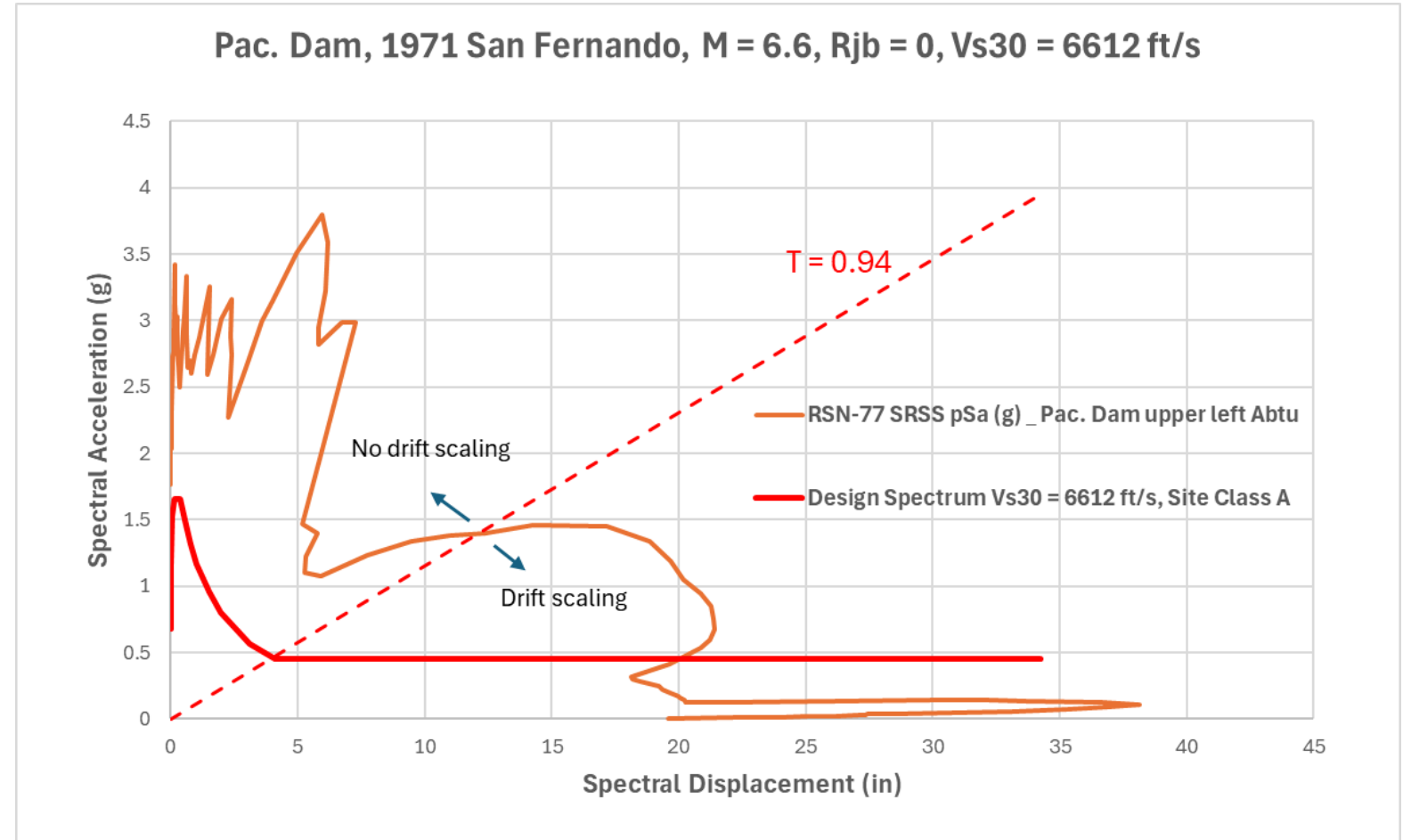
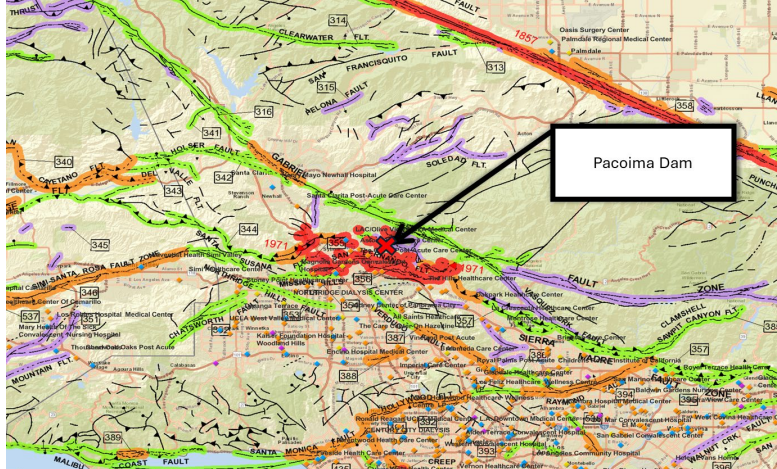


1971 San Fernando – Pacoima Dam



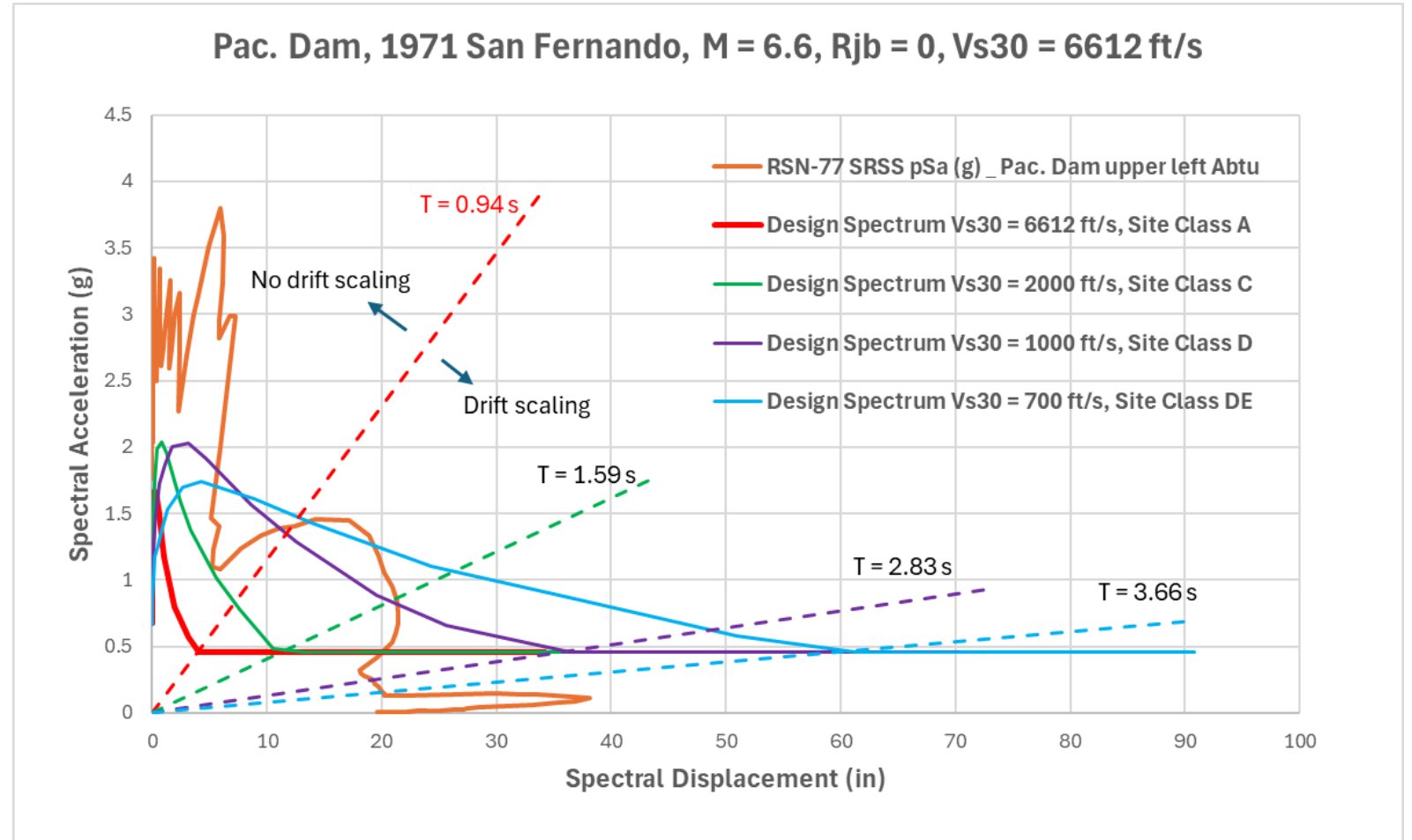
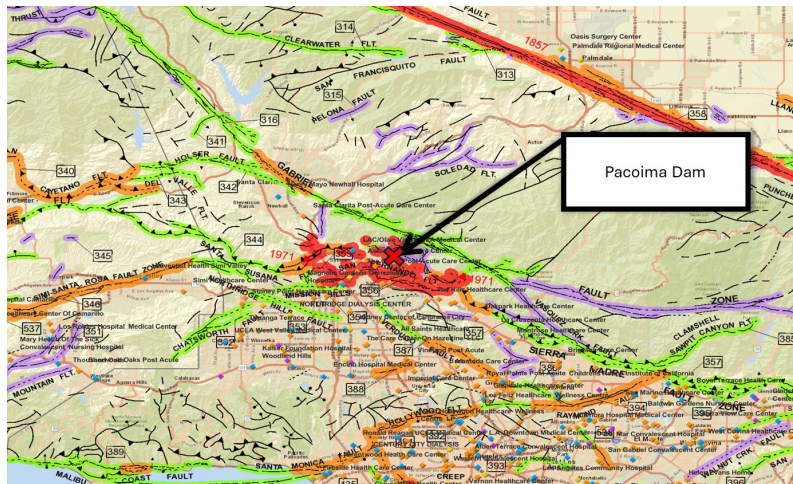
1971 San Fernando – Pacoima Dam Comparison with ASCE 7-22 Design Spectrum

- $S_1 = 0.91$ g
- $S_{Ds} = 1.17$ g
- $S_{D1} = 0.38$ g
- $C_u T_a \geq 0.94$ s Drift Scaling Required



1971 San Fernando – Pacoima Dam Comparison with ASCE 7-22 Design Spectrum

- $S_1 = 0.91 \text{ g}$
- $S_{Ds} = 1.17 \text{ g}$
- $S_{D1} = 0.38 \text{ g}$
- $C_u T_a \geq 0.94 \text{ s}$ Drift Scaling Required



Proposed Change Currently being balloted in ASCE 7-28

- Limit drift scaling to sites classified as near fault in accordance with ASCE 7-22, Section 11.4.



Conclusion

- Design ground motion spectra do not capture near fault effects.
- Stiff sites produce high velocity pulses that could affect long period structures.
- Drift scaling not required for buildings on soft soils with $V_{s30} < 1000$ ft/s or for Site Class D or lower.
- Drift scaling should be considered for buildings in Site Class C or better regardless of the S_1 value when the building site is less than 5 km from a known active fault of Source Type B, $M > 6.5$.

Case Study

PROJECT DESCRIPTION:

- 240-ft tall OSHPD-1 bed tower located in San Diego, California
- Distance from fault < 5km
- $S_s = 1.38g$
- $S_1 = 0.472g$
- $S_{DS} = 1.028g$
- $S_{D1} = 0.444g$
- Building codes:
 - CBC 2016
 - ASCE 7-10
- Risk category IV
- $I_e = 1.5$
- SFRS:
 - Longitudinal direction: Steel special moment resisting frames
 - Transverse direction: Steel buckling restrained braced frames
- $R = 8$
- $C_d = 5.5$ (longitudinal direction)
- $C_d = 5$ (transverse direction)

Case Study

RESPONSE SPECTRUM ANALYSIS (RSA):

ASCE 7-10 Eq (12.8-5) (which is Eq (12.8-6) in ASCE 7-22) governed minimum base shear as S_1 was less than 0.6g.

ASCE 7-10:	ASCE 7-22:
C_s shall not be less than $C_s = 0.044S_{DS}I_e \geq 0.01 \quad (12.8-5)$ In addition, for structures located where S_1 is equal to or greater than 0.6g, C_s shall not be less than $C_s = 0.5S_1/(R/I_e) \quad (12.8-6)$	Lower Bounds for C_s for Both Methods: C_s shall not be less than $C_s = 0.044S_{DS}I_e \geq 0.01 \quad (12.8-6)$ In addition, for structures located where S_1 is equal to or greater than 0.6, C_s shall not be less than $C_s = 0.5S_1/(R/I_e) \quad (12.8-7)$ where S_1 is the mapped maximum considered earthquake spectral response acceleration parameter determined in accordance with Section 11.4.2 or 11.4.4.

This resulted in C_s of 0.068g. Due to the requirement in CBC 2016 §1616A.1.13 (snip below), drifts were also evaluated at a base shear of 0.068W.

Case Study

(Note: §1616A.1.13 provision in CBC 2016 has been removed in CBC 2022. CBC 2022 has no amendments on the drift scaling provision of ASCE 7-16.)

ASCE 7-10:

12.9.4.1 Scaling of Forces

Where the calculated fundamental period exceeds $C_u T_a$ in a given direction, $C_u T_a$ shall be used in lieu of T in that direction. Where the combined response for the modal base shear (V_t) is less than 85 percent of the calculated base shear (V) using the equivalent lateral force procedure, the forces shall be multiplied by $0.85 \frac{V}{V_t}$:
where

V = the equivalent lateral force procedure base shear, calculated in accordance with this section and Section 12.8

12.9.4.2 Scaling of Drifts

Where the combined response for the modal base shear (V_t) is less than $0.85 C_s W$, and where C_s is determined in accordance with Eq. 12.8-6, drifts shall be multiplied by $0.85 \frac{C_s W}{V_t}$

CBC 2016, OSHPD 1, 4 & DSA-SS:

1616A.1.13 ASCE 7, Section 12.9.4. Replace ASCE 7 Section 12.9.4 as follows:

12.9.4 Scaling design values of combined response.
Modal base shears used to determine forces and drifts shall not be less than the base shear calculated using the equivalent lateral force procedure of Section 12.8.

Base shears that correspond to drifts scaled by C_d/I_e , were approximately 0.23W and 0.25W in the longitudinal and transverse directions, respectively.

Case Study

It is also observed that the drift scaling requirement for RSA resulted in DE-level base shears that are greater than the MCE-level base shears obtained through NLRHA.

Hazard	Direction	Base Shears at which Drifts are Evaluated*	
		RSA	NLRHA
DE	Longitudinal	0.23W	0.16W
	Transverse	0.25W	0.14W
MCE	Longitudinal	-	0.21W
	Transverse	-	0.17W

Table below compares RSA drifts, with and without base shear scaling to ELF, of the revised design with 40% less steel SFRS tonnage. It is observed that the drift limit of 1% is still satisfied with the revised design if base shear is not scaled to the ELF base shear.

Level	RSA Drift Limit	Max. RSA Drifts of the Revised Design			
		Base Shear Scaled to ELF		Base Shear not Scaled to ELF	
		Longitudinal Dir.	Transverse Dir.	Longitudinal Dir.	Transverse Dir.
Roof	1%	1.8%	2.4%	0.6%	0.9%
L1 – L15		2.4%	1.8%	0.8%	0.7%

Takeaway from Case Study

- Drift scaling was intended to prevent weak buildings from collapse of buildings subject to pulse period drifts, the NLRHA shows that buildings with sufficient ductility are adequate to accommodate these pulse period drifts and also satisfy the minimum drift requirement.

Additional studies

- Additional studies were performed to investigate the impact of drift scaling for randomly selected near fault and far field sites with high S_{D1} values.

Additional studies

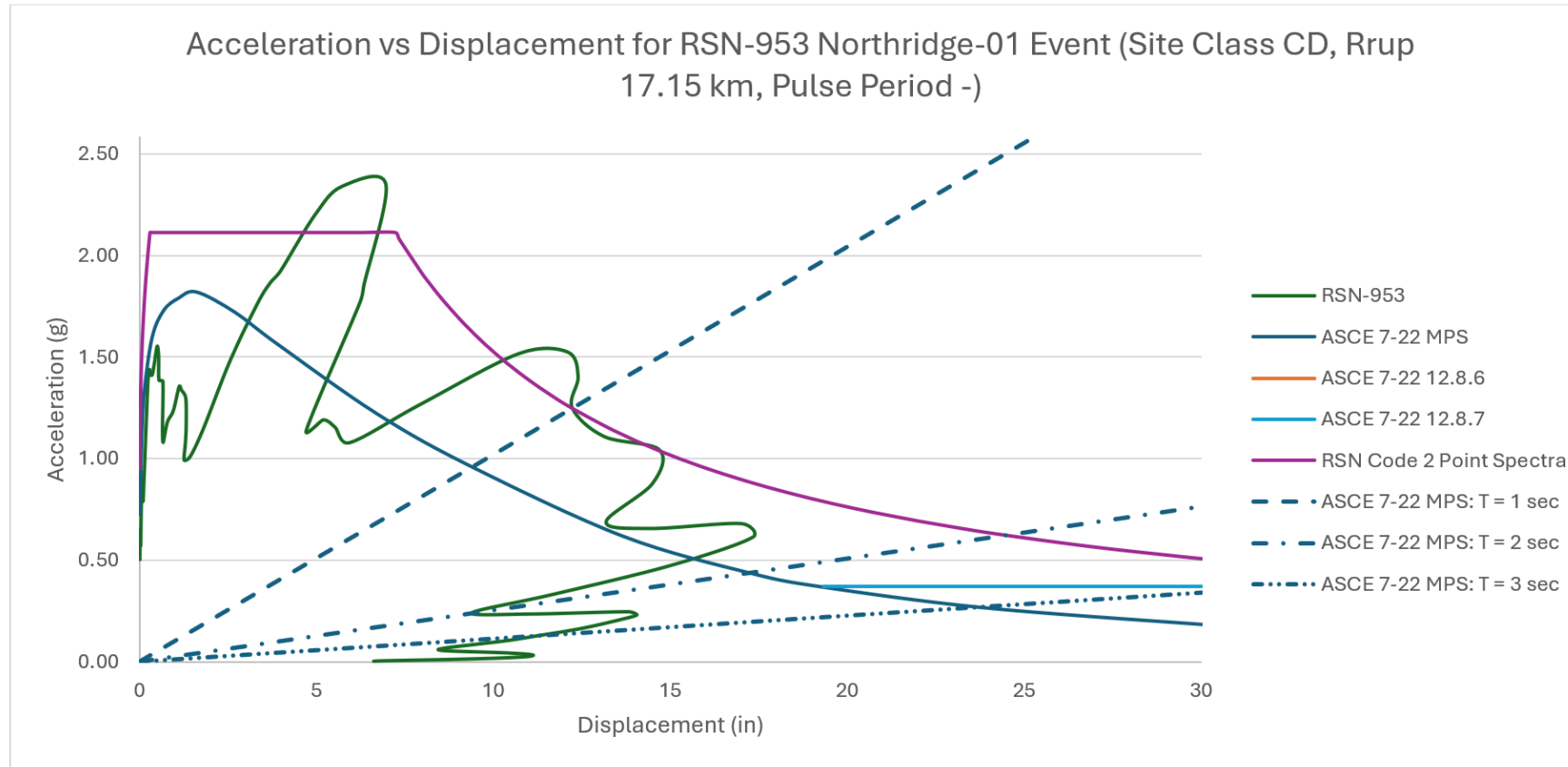


Figure 1: RSN-953, Spectral Acceleration vs Spectral Displacement plot

Mapped S_1 : 0.74

Additional studies

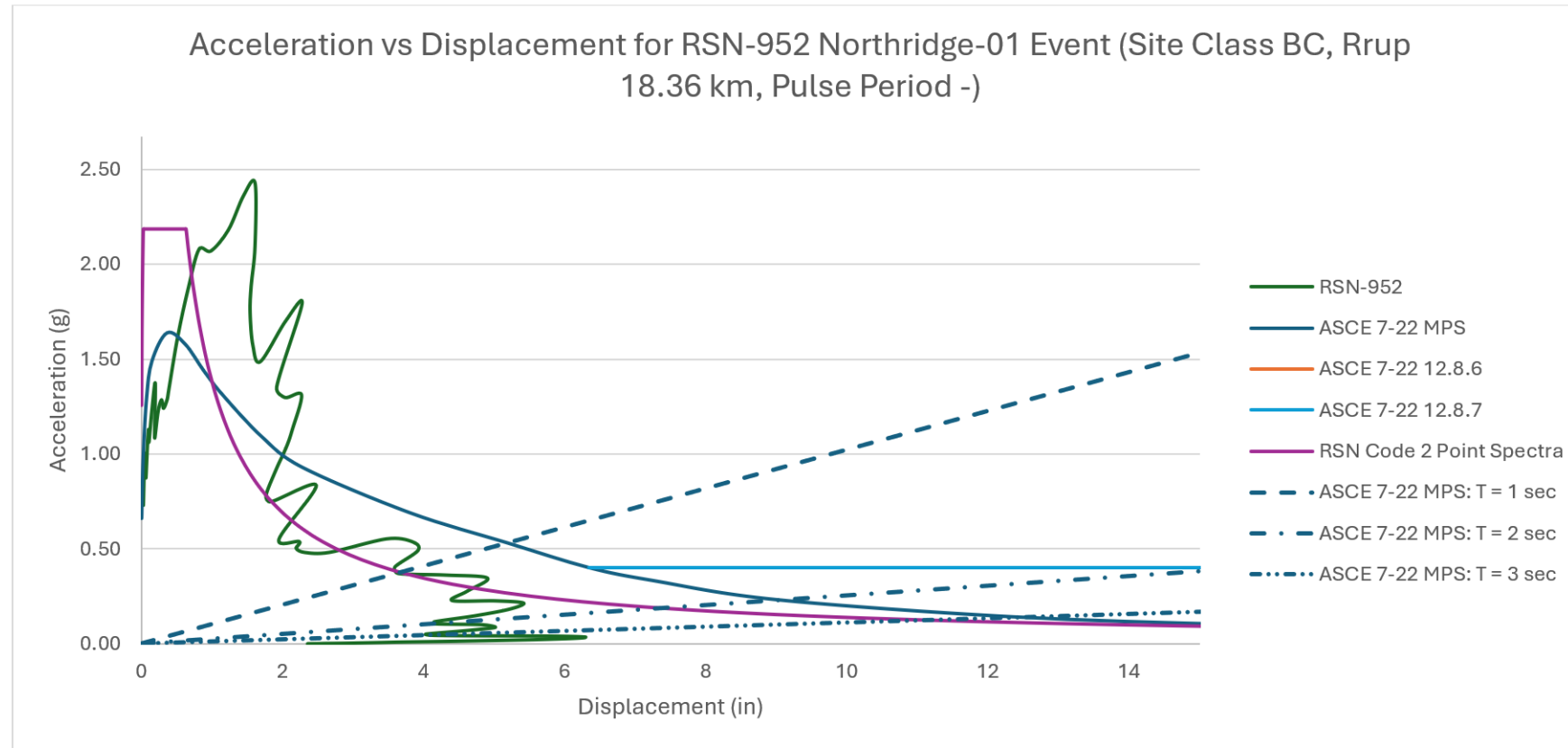


Figure 2: RSN-953, Spectral Acceleration vs Spectral Displacement plot

Mapped S_1 : 0.80

Additional studies

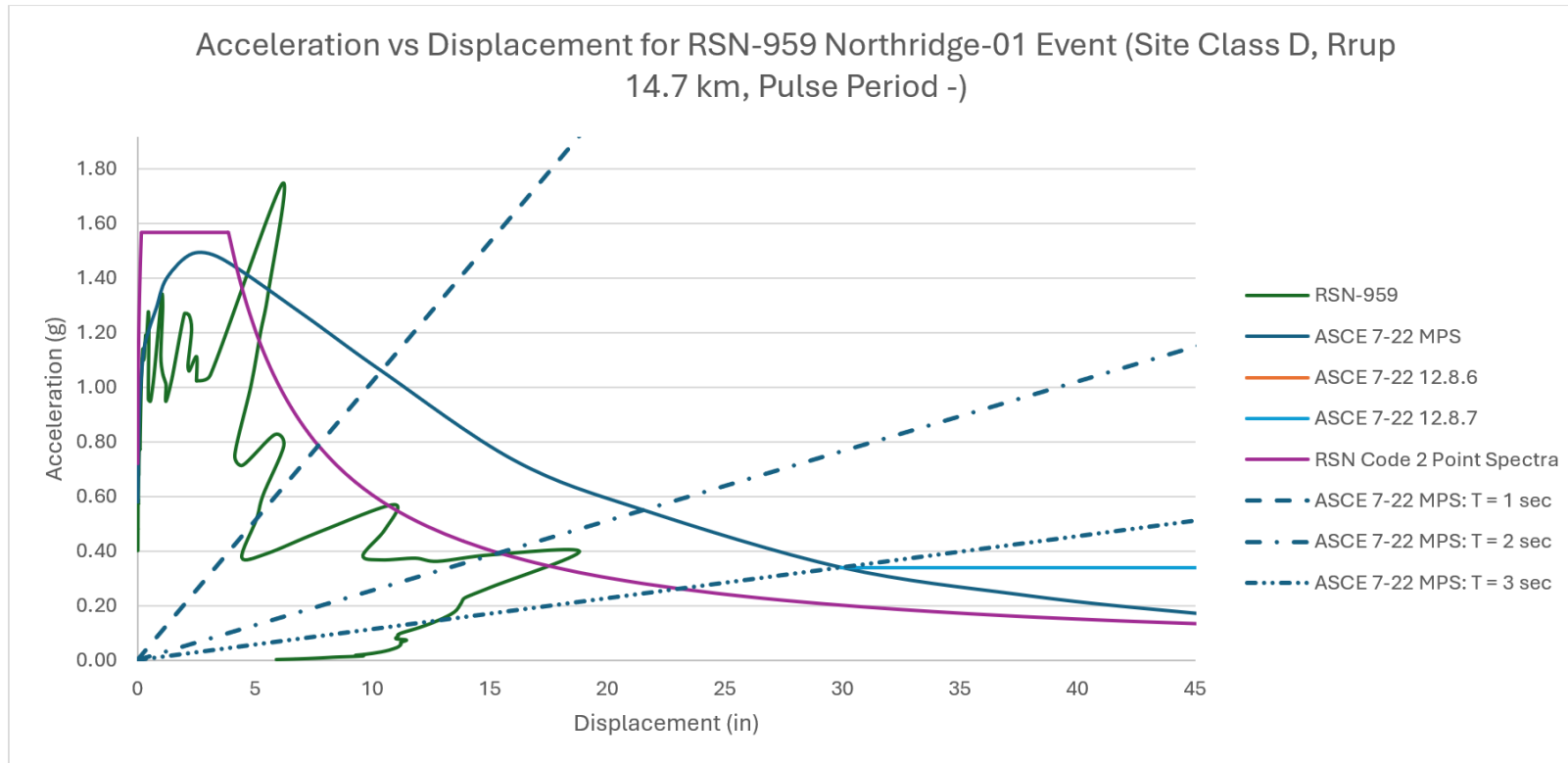


Figure 3: RSN-959, Spectral Acceleration vs Spectral Displacement plot

Mapped S_1 : 0.68

Additional studies

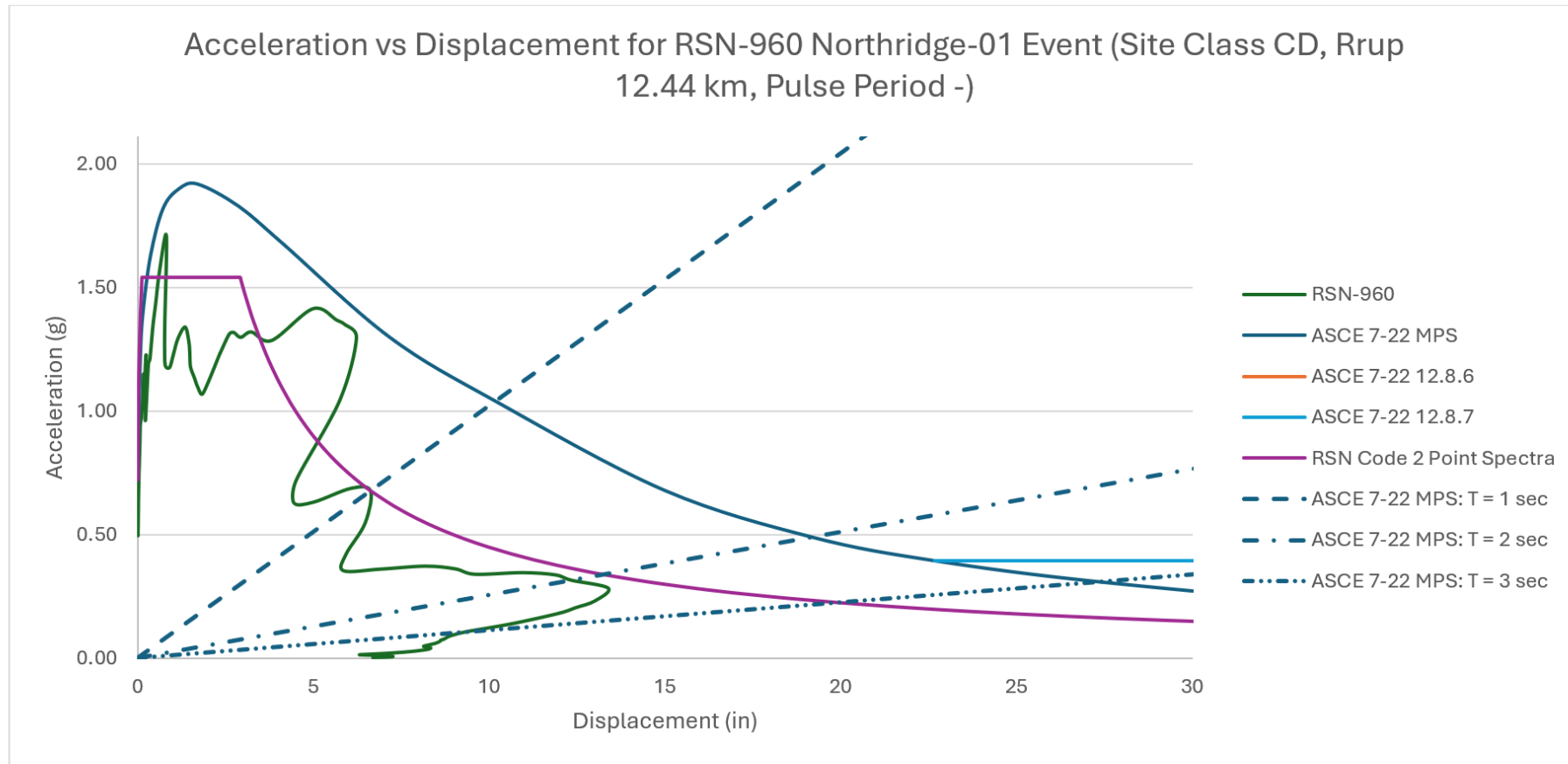


Figure 4: RSN-960, Spectral Acceleration vs Spectral Displacement plot

Mapped S_1 : 0.79

Additional studies

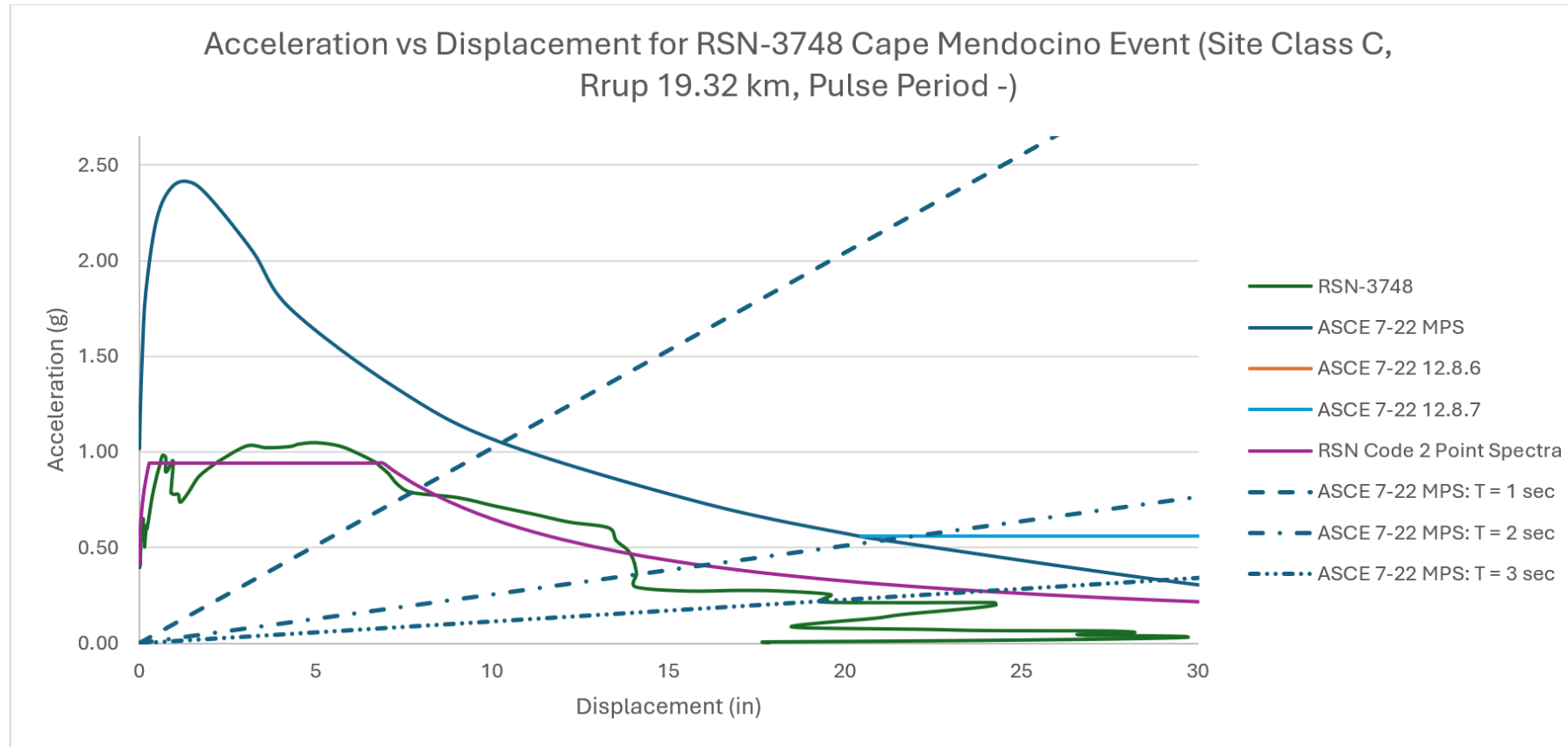


Figure 5: RSN-3748, Spectral Acceleration vs Spectral Displacement plot

Discussion!

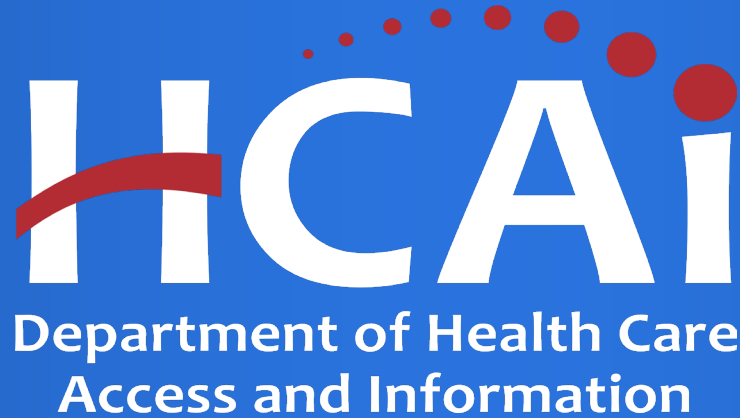
- Drift scaling required for near fault sites, only when triggered, similar to the proposed ballot in ASCE 7-28
- Drift scaling required only for near fault sites only when triggered, in Site Classes CD or C and Better
- Have a minimum floor for design of buildings with period T , where $T_s < T < T_L$ regardless of the value of S_1 and drift scaling for near fault sites in Site Classes CD or C and Better
- Other?

Item #7

Fiber-Reinforced Polymer (FRP) retrofit requirements for reinforced concrete shear walls, diaphragms, spandrels, and collectors

- Background for current provisions in CBC, and ASCE 41 for FRP retrofit for concrete elements
- Recent tests and proposed design criteria approach
- Committee to endorse and/or provide feedback on modifications presented
- Discussion and public input

Facilitator: Ali Sumer, PhD, SE, Supervisor, HCAI (or designee)



Item #7:
**Fiber-Reinforced Polymer (FRP) retrofit requirements for
reinforced concrete shear walls, diaphragms, spandrels, and
collectors**

Ali Sumer, Seismic Compliance Unit

HBSB SNSR Committee Meeting – July 29, 2026

Outline

- What is FRP?
- Why FRP?
- Current code requirements
- Various standards, guides
- Ongoing tests
- Proposal direction

What is Fiber Reinforced Polymer (FRP)?



- Photo credit: Fyfe

What is FRP

FIBER (FABRIC)



POLYMER (EPOXY)



• Photo credit: Fyfe

Why FRP for hospitals?

- FRP is one of the options to retrofit existing old concrete buildings.
(HCAI has been using FRP for decades)

Advantages

- Easier to apply in congested areas.
- Most cases, it is faster to install, can be economical
- Improves seismic capacity

Disadvantages

- Not well understood by the industry
- Limited strength
- No comprehensive standard exists today
- Requires specialized crew to install

Current Code CBC 1911A.3

1911A.3 Concrete strengthening by externally bonded fiber reinforced polymer (FRP). Design and construction of externally bonded FRP systems for strengthening concrete structures shall be in accordance with ACI 440.2R.

Exceptions:

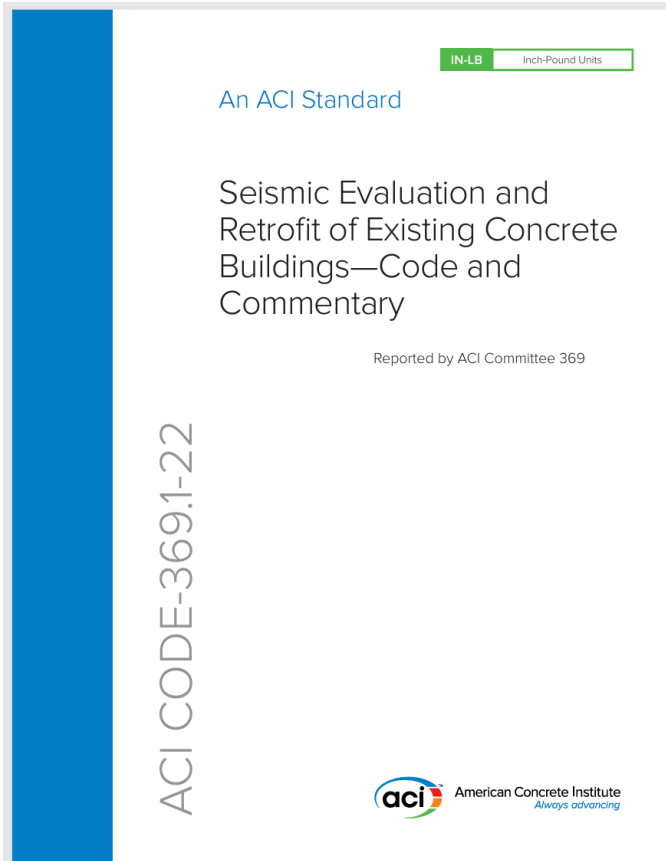
- 1. Near-Surface Mounted (NSM) FRP bars shall not be permitted.*
- 2. Strengthening of shear walls and diaphragms (including chords and collectors) shall be considered as an alternative system.*

Design capacities, reliability, serviceability of FRP materials shall be permitted to be established in accordance with ICC-ES AC 125. Minimum inspection requirements of FRP composite systems shall be in accordance with ICC-ES AC 178.

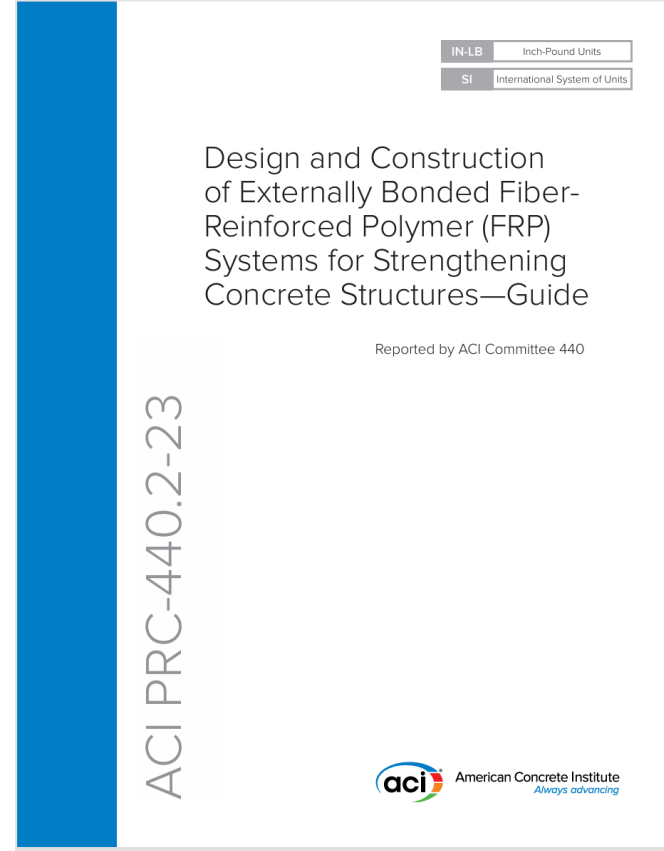
There is no ASCE 41 section that addresses FRP force levels, ductility or detailing.

HCAI has been using Alternative Method of Compliance for all retrofit projects for shear walls, diaphragms etc.

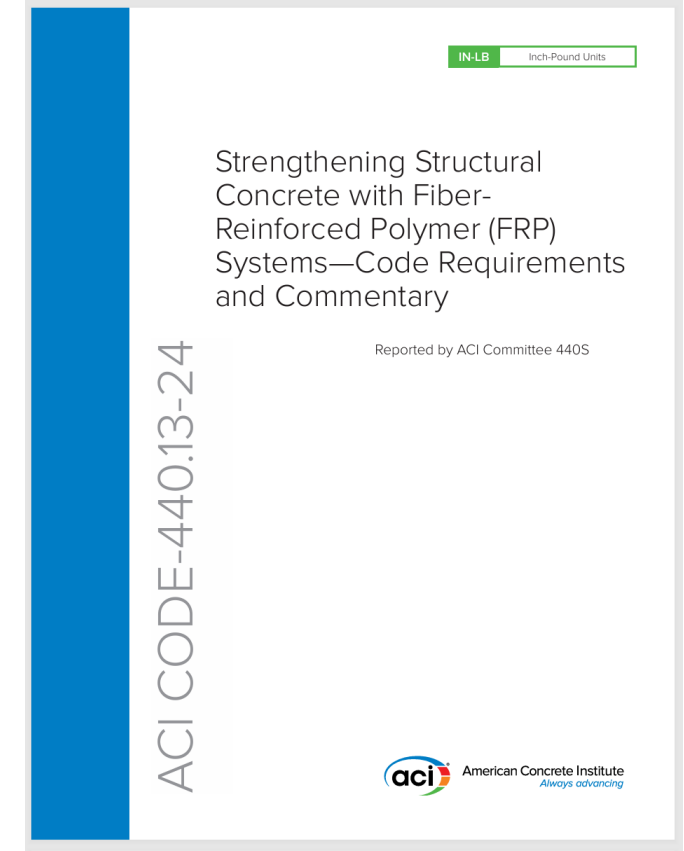
Other Standards: ACI



- FRP section in later versions, future version in the works

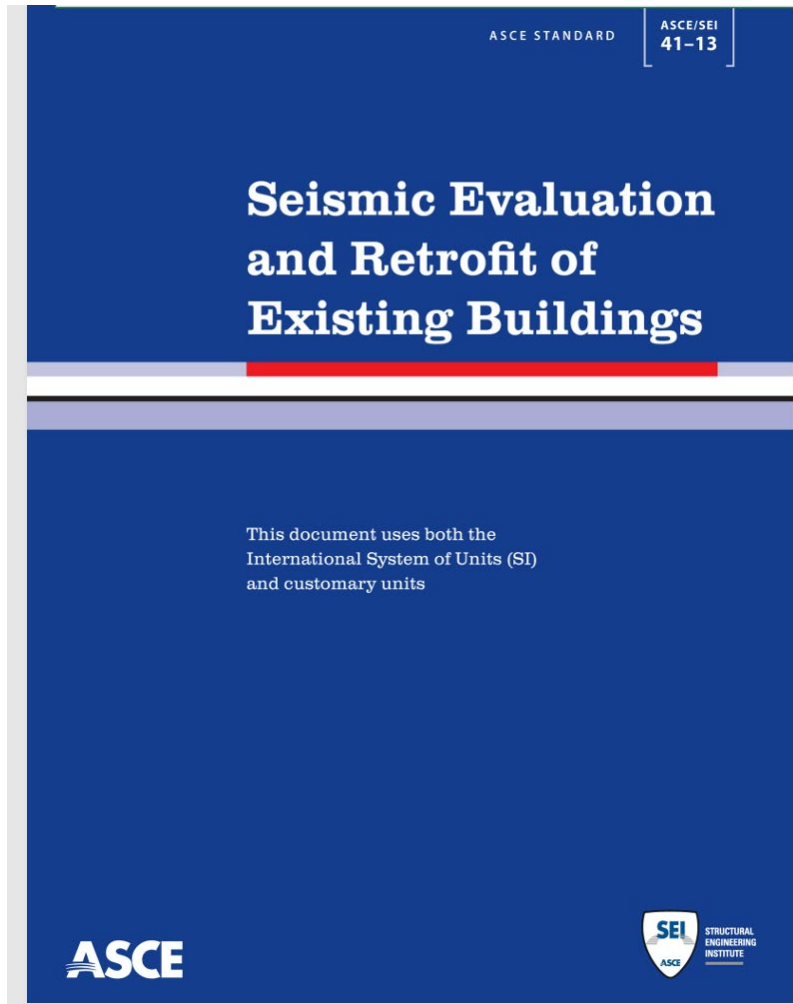


- Limited FRP strengthening criteria exists, future version in the works



- Only gravity, not seismic, future version in the works

ASCE41



- ASCE 41-13, ASCE 41-23 don't have FRP chapters
- ASCE 41-29 will point to ACI future standards

HCAI Current Design Method: AMC

Concrete Shear Wall Design Guidelines

OSHPD DESIGN CRITERIA GUIDELINES – FRP STRENGTHENING

1. Member type to be strengthened – Column, wall, diaphragm, slab or beam.
 - **Concrete Wall**
2. Strengthening type – Shear (columns, walls, beams or diaphragms), confinement (columns only), flexural (walls, beams & slabs).
 - **In-Plane Shear**
3. Whether strengthening resists gravity (i.e. live-loading or sustained loading) and/or seismic forces for each type of strengthening used.
 - **Seismic forces**
4. Whether FRP is bond-critical or contact-critical for each type of strengthening used.
 - **Bond-critical**
5. Method of application – Wet lay-up, pre-cured FRP laminate.

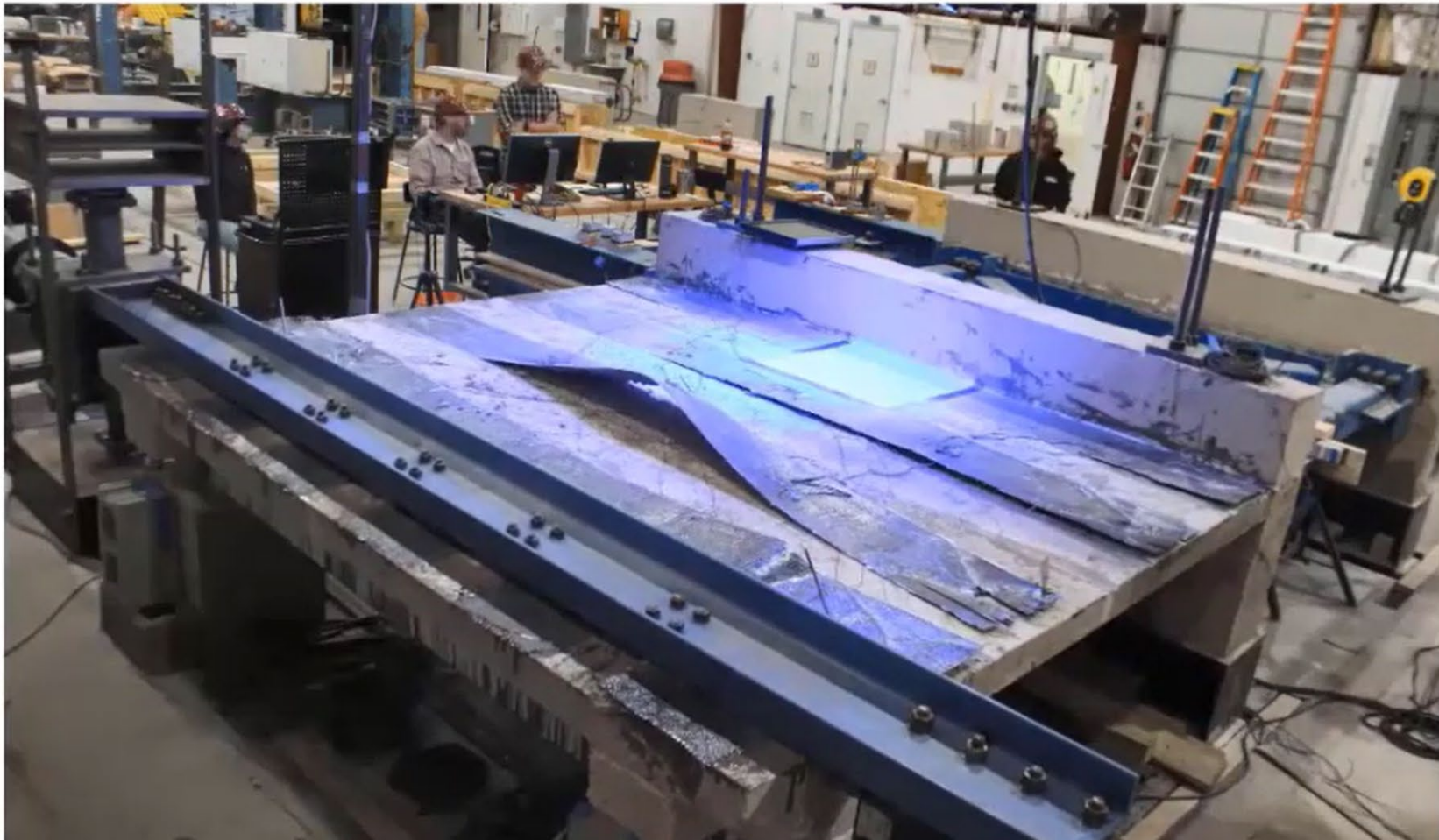
- HCAI has used an Alternate Method of Compliance for the FRP criteria.
- The criteria differ between projects.

PCS program

- Preapproval of FRP design criteria
- Data collection started as part of the PCS program.
- Additional tests were recognized as necessary.
- Manufacturers performed several tests in the last 5 years.
 - Shear Walls
 - Fiber anchors (individual, grouped, spaced)
 - Diaphragm
 - Collectors
 - Slab to Wall connections
 - Contact critical column tests

Virginia Tech – Diaphragm

Specimen CD2 Timelapse



Idaho State University – Slab to Wall connection



a. Debonding failure in SP8



b. Debonding failure of glass fabric SP10



c. Shear failure at connection - SP11



d. Failure at connection - SP12

UT San Antonio – Flexural wall tests

UTSA The University of Texas at San Antonio™

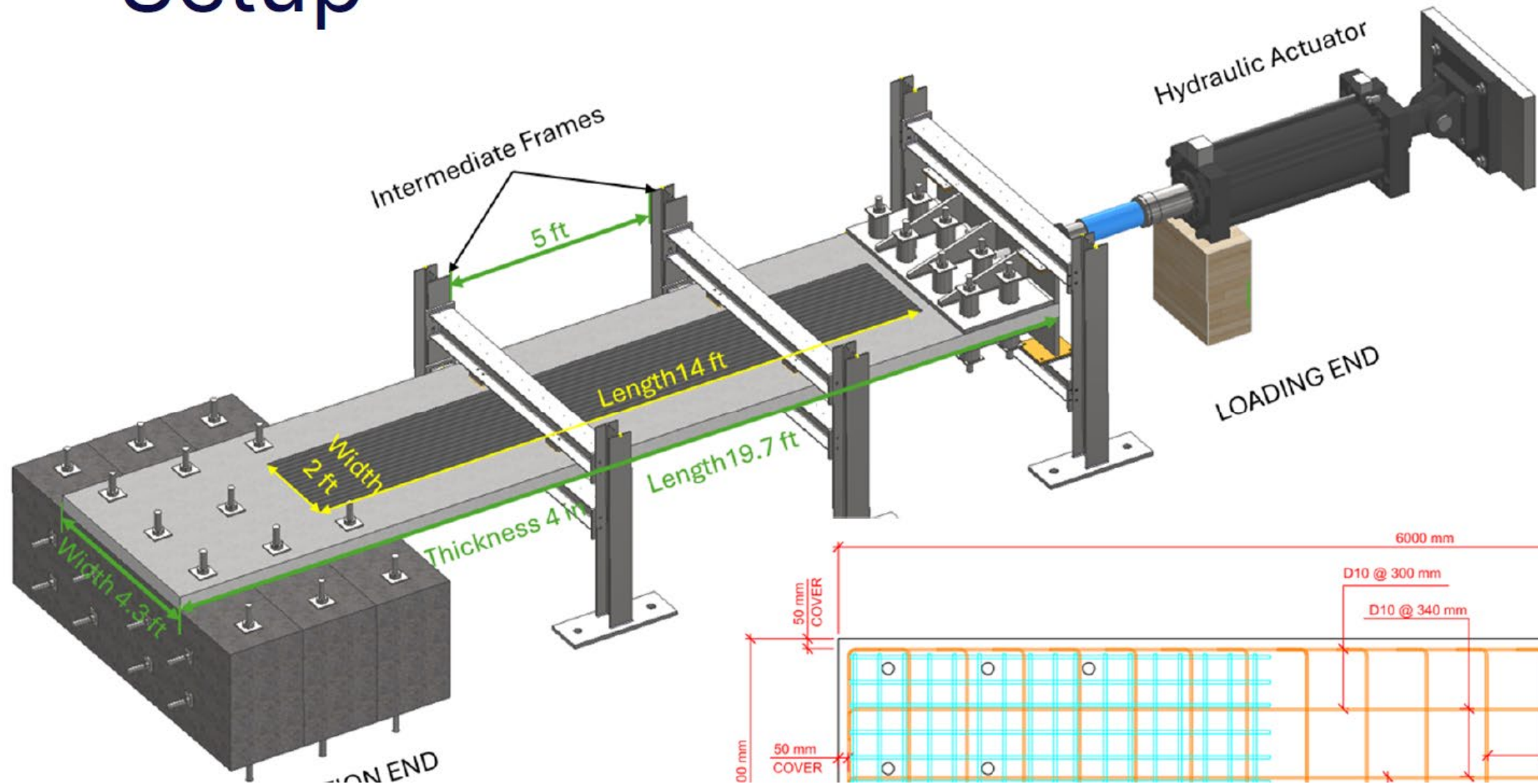
Series 2 – Wall 2 CFRP

Test video



University of Auckland: Collector Tests

Setup



More tests...

- Simpson shear wall tests
 - USC contact critical and low strength column tests
 - Simpson short collector tests with anchors
 - Univ. of Miami – wall to floor tests
 - Several anchor tests
 - And more...
-
- UCLA will be testing concrete shear walls
 - More tests are in discussions...

Code Change Proposal

Where do we place FRP requirements in the code?

- Since FRP material can be applied to concrete and masonry, there is no individual material chapter that can cover all FRP requirements.
- Splitting FRP into multiple code chapters will not be easy to follow
- Recommend a new section - just for FRP - under CEBC 304A.4.
- CEBC 304A.4 will include FRP for masonry and concrete.
- Point/reference from 1911A.3 to CEBC 304A.4.

Proposal for FRP for CONCRETE:

Develop criteria for FRP in Concrete applications for the following cases:

ASCE 41 Overall criteria:

- For existing shear walls, wall segments, and diaphragms with different reinforcement percentages, develop ASCE 41 criteria for force controlled or deformation-controlled actions.
- Develop maximum allowable design strain for shear strengthening.

ASCE 41 Analysis:

- Develop specific “m” values” for linear analysis and rotation values for non-linear analysis.

Detailing:

- Develop maximum anchor spacing and embedment for different applications.
- Develop detailing criteria for FRP collectors with and without anchors

Proposal for FRP for MASONRY:

Develop minimum criteria for FRP in masonry applications for the following cases:

- Shear walls with reinforcing compliant with TMS Requirements
- Shear walls with reinforcing not compliant with TMS but compliant with ASCE 41
- Shear walls with vertical reinforcing compliant and horizontal reinforcing not compliant
- Shear walls with neither vertical nor horizontal reinforcing compliant

Item #8

Reschedule the August 26, 2026, Committee meeting

- Discussion and public input

Facilitator: Jim Malley (or designee)

Item #9

Comments from the Public/Committee Members on Issues not on this Agenda

The Committee will receive comments from the Public/Committee Members. Matters raised at this time may be taken under consideration for placement on a subsequent agenda.

Facilitator: Jim Malley (or designee)

Item #10

Adjournment