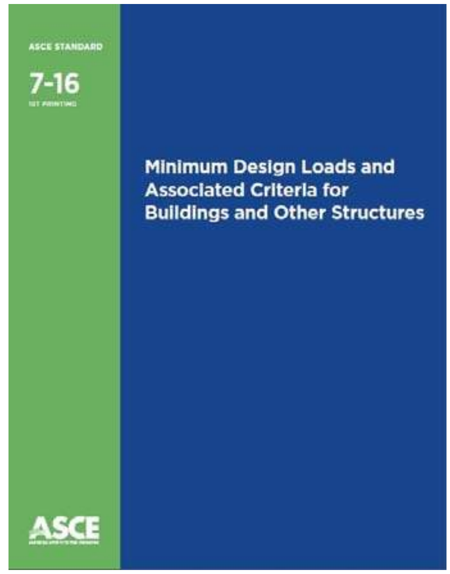
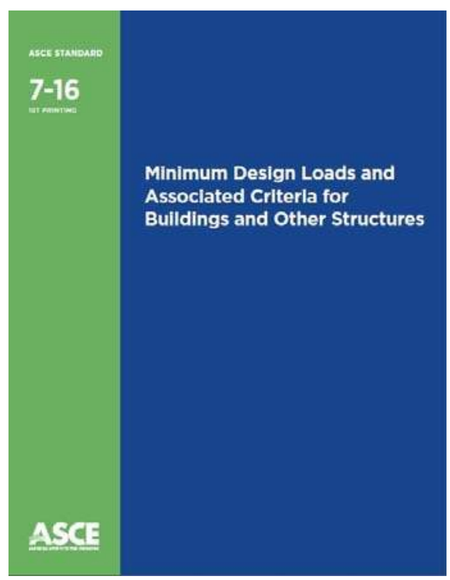
ASCE 7-16

Highlights and Selected Changes
Presented by
SEAU Seismic Committee



ASCE 7-16

Significant Changes to Chapter 11
Presented by
Brent Maxfield



Changes to S_S and S_1

Sample Site in Murray

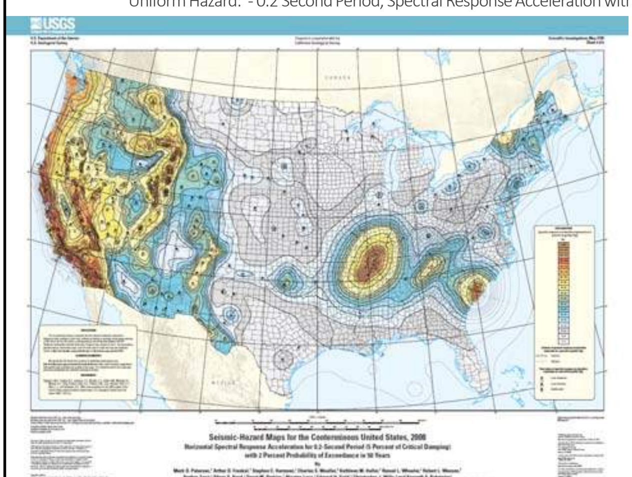
	ASCE 7-10	ASCE 7-16	Change	Percentage
S_S	1.556	1.488	-0.068	-4%
S_1	0.530	0.526	-0.004	-1%

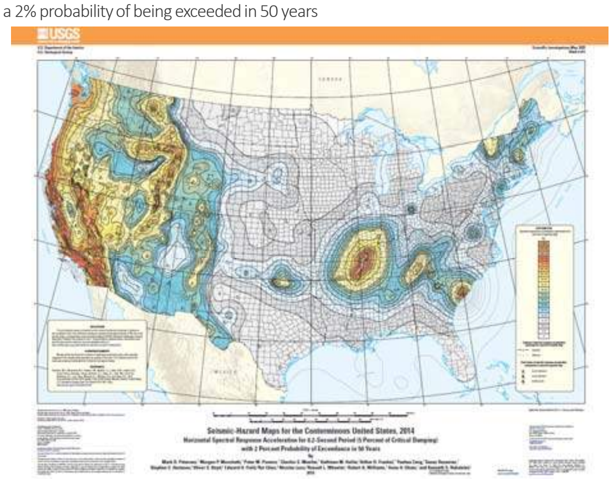
ASCE 7-10 Derived from 2008 USGS Maps

ASCE 7-16 Derived from 2014 USGS Maps

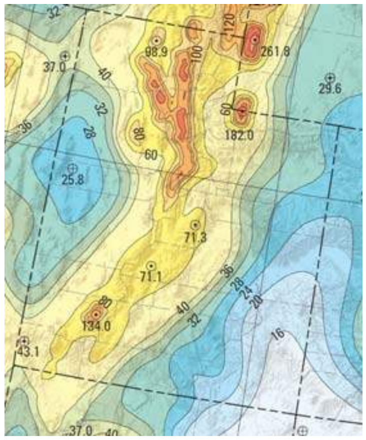
Comparison of USGS National Seismic Hazard Maps

Uniform Hazard: - 0.2 Second Period, Spectral Response Acceleration with a 2% probability of being exceeded in 50 years

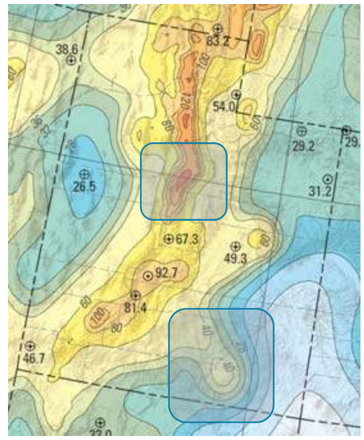




Utah 0.2 Second Response Acceleration

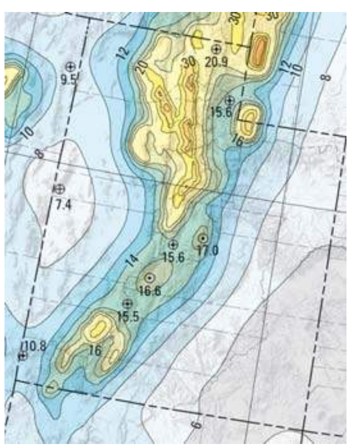


2008

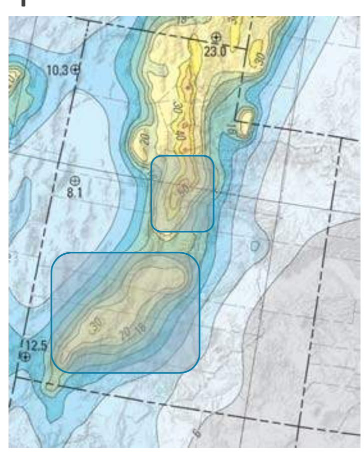


2014

Utah 1.0 Second Response Acceleration



2008



2014

Other Utah Locations – S_S

	ASCE 7-10	ASCE 7-16	Change	Percentage
Logan	0.971	1.058	0.087	9%
Brigham City	1.467	1.372	-0.095	-6%
Ogden	1.373	1.362	-0.011	-1%
Provo	1.144	1.323	0.179	16%
Manti	0.638	0.635	-0.003	0%
Cedar City	0.702	0.777	0.075	11%
St. George	0.499	0.509	0.010	2%
Vernal	0.297	0.317	0.020	7%
Monticello	0.156	0.179	0.023	15%

Other Utah Locations – S_1

	ASCE 7-10	ASCE 7-16	Change	Percentage
Logan	0.311	0.353	0.042	14%
Brigham City	0.521	0.488	-0.033	-6%
Ogden	0.499	0.497	-0.002	0%
Provo	0.427	0.496	0.069	16%
Manti	0.186	0.199	0.013	7%
Cedar City	0.216	0.250	0.034	16%
St. George	0.153	0.165	0.012	8%
Vernal	0.091	0.082	-0.009	-10%
Monticello	0.054	0.057	0.003	6%

One of the Most Significant Changes in ASCE 7-16

Site Coefficients

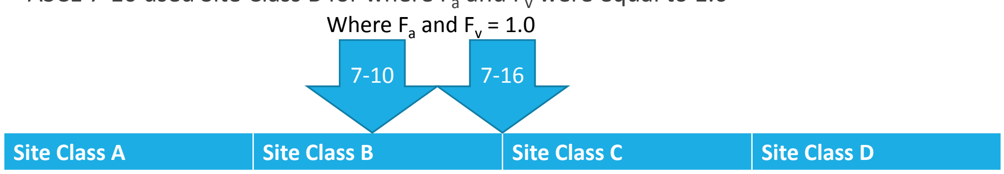
F_a and F_v

Site classifications have not changed.

- A = Hard rock
- B = Rock
- C = Very dense soils and soft rock
- D = Stiff soil
- E = Soft clay soil
- F = Soils requiring a site response analysis

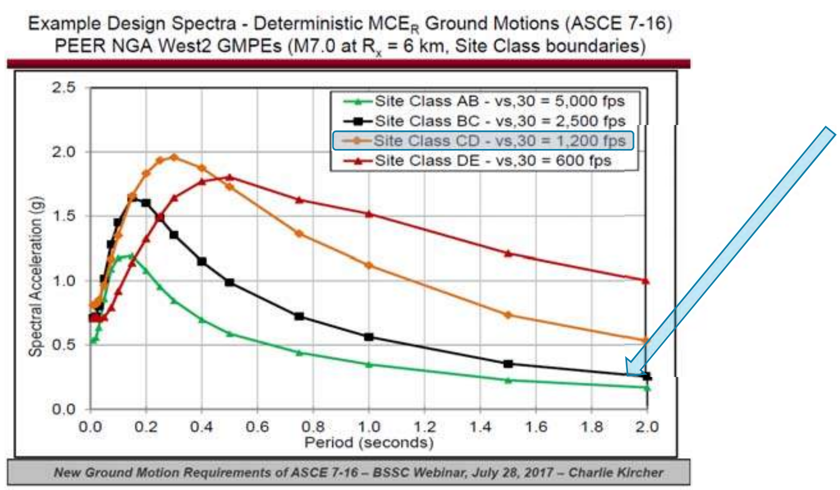
Site Coefficients

- USGS maps are for "Rock" with a shear wave velocity of 760 m/s (2,500 ft/s)
- Corresponds to the boundary between Site Class B and Site Class C (See Commentary C11.4.4)
- At this boundary, Site Coefficients F_a and F_v are both equal to 1.0
- ASCE 7-10 used Site Class B for where F_a and F_v were equal to 1.0

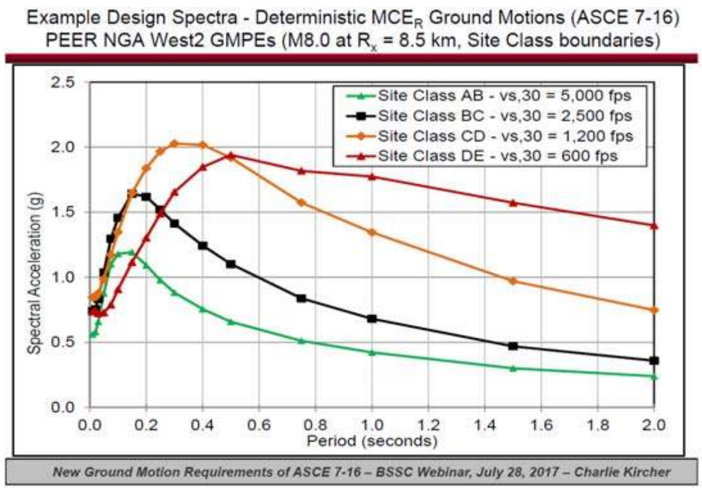


- In ASCE 7-16 Site Coefficients for Site Class B are less than 1.0.

What are Site Coefficients?



Coefficients vary with strength of shaking



Compare Site Coefficients

at S_S=1.25 and S_1=0.5

Site Class	S_S=1.25		S_1=0.5	
	7-10	7-16	7-10	7-16
A	0.8	0.8	0.8	0.8
B	1.0	0.9	1.0	0.8
C	1.0	1.2	1.3	1.5
D	1.0	1.0	1.5	1.8*
E	0.9	1.2*	2.4	**

760 m/s Boundary

The * and ** will be discussed in future slides

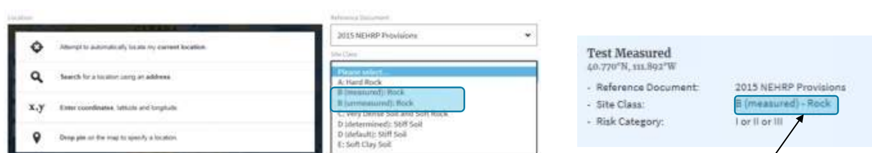
Table 11.4-1 Short–Period Site Coefficient, F_a

	S_S<=0.25		S_S=0.5		S_S=0.75		S_S=1.0		S_S=1.25		S_S>=1.5
	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-16 (New)
A	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
B	1.0	0.9	1.0	0.9	1.0	0.9	1.0	0.9	1.0	0.9	0.9
C	1.2	1.3	1.2	1.3	1.1	1.2	1.0	1.2	1.0	1.2	1.2
D	1.6	1.6	1.4	1.4	1.2	1.2	1.1	1.1	1.0	1.0	1.0
Default	1.6	1.6	1.4	1.4	1.2	1.2	1.1	1.2	1.0	1.2	1.2
E	2.5	2.4	1.7	1.7	1.2	1.3	1.1	1.2*	1.0	1.2*	1.2*

- Note that F_a for Site Class C exceeds F_a for Site Class D, when S_S >= 1.0.
- Section 11.4.4: "Where Site Class D is selected as the default site class per Section 11.4.3, the value of F_a shall not be less than 1.2."
- The * will be discussed in future slides

Measured shear wave velocity for Site Class B

- 11.4.3: "For situations in which site investigations, performed in accordance with Chapter 20, reveal rock conditions consistent with Site Class B, but site-specific velocity measurements are not made, the site coefficients F_a, F_v, and F_{PGA} shall be taken as unity (1.0).
- New Terms: "Measured Rock" vs "Unmeasured Rock"
- In the USGS Website, you now must Select B (Measured) or B (Unmeasured)
- If geotechnical report states Site Class B, you must know if the shear wave velocity was measured.



Screen shots from USGS website

Table 11.4-1: Site Coefficient F_a

Site Class	Spectral Reponse Acceleration Parameter at Short Period					
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S = 1.25$	$S_S \ge 1.50$
A	0.8	0.8	0.8	0.8	0.8	0.8
B (measured)	0.9	0.9	0.9	0.9	0.9	0.9
B (unmeasured)	1.0	1.0	1.0	1.0	1.0	1.0
C	1.3	1.3	1.2	1.2	1.2	1.2
D (determined)	1.6	1.4	1.2	1.1	1.0	1.0
D (default)	1.6	1.4	1.2	1.2	1.2	1.2
E	2.4	1.7	1.3	1.2*	1.2*	1.2*
F	See Section 11.4.7					

Table 11.4-1 Long–Period Site Coefficient, F_V

New

	S_1<=0.1		S_1=0.2		S_1=0.3		S_1=0.4		S_1=0.5		S_1>=0.6
	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-16
A	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
B Measured	1.0	0.8	1.0	0.8	1.0	0.8	1.0	0.8	1.0	0.8	0.8
B Unmeasured	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
C	1.7	1.5	1.6	1.5	1.5	1.5	1.4	1.5	1.3	1.5	1.4
D	2.4	2.4	2.0	2.2*	1.8	2.0*	1.6	1.9*	1.5	1.8*	1.7*
E	3.5	4.2	3.2	**	2.8	**	2.4	**	2.4	**	**

The * and ** will be discussed in future slides.

Design Response Spectrum Using Two Points (S_{DS} and S_{D1})

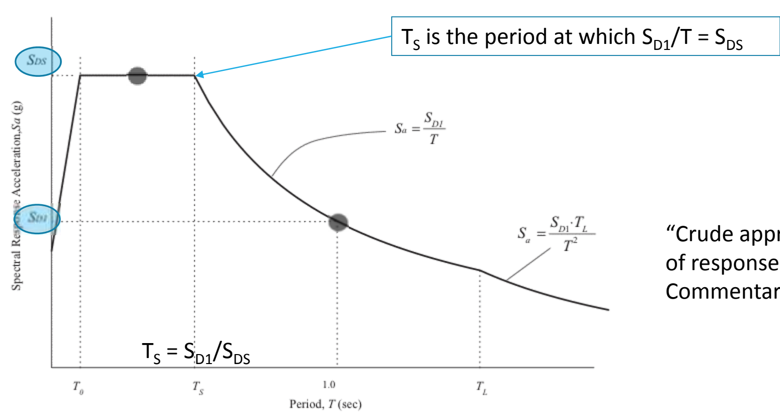
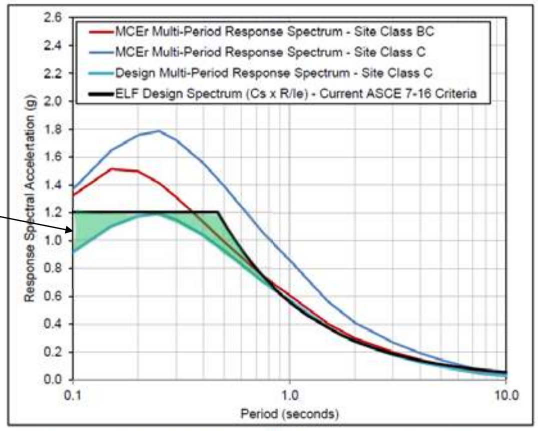


FIGURE 11.4-1 Design Response Spectrum

"Crude approximation to the actual shape of response spectral accelerations"
Commentary C11.4.8

Response spectra if a response point was calculated at multiple periods.

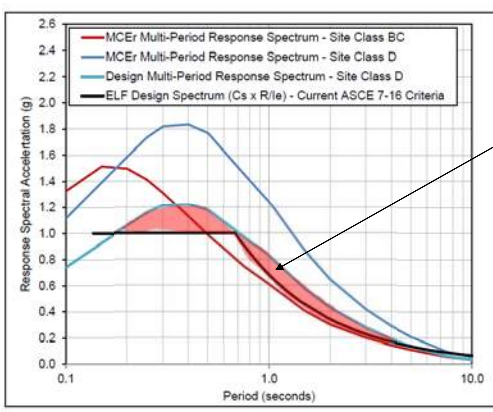
Conservative



Figures from
FEMA P-1050

FIGURE C11.4-1 Comparison of ELF and Multi-Period Design Spectra – Site Class C Ground Motions ($v_{s,30}$ = 1,600 ft/s)

Site Class D



Unconservative

FIGURE C11.4-2 Comparison of ELF and Multi-Period Design Spectra – Site Class D Ground Motions ($v_{s,30}$ = 870 ft/s)

Site Class E

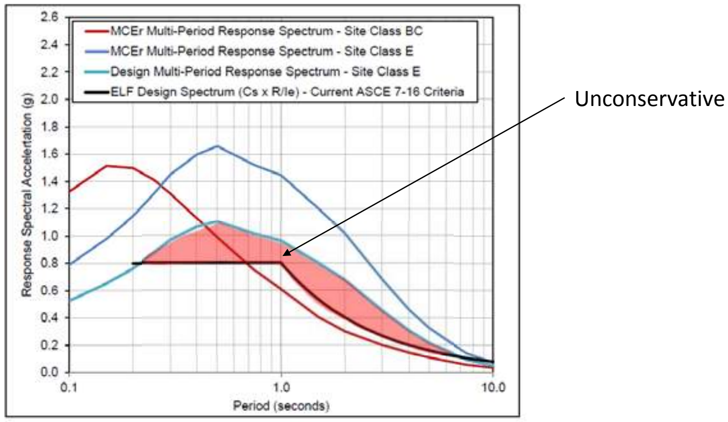


FIGURE C11.4-3 Comparison of ELF and Multi-Period Design Spectra – Site Class E Ground Motions ($v_{s,30}$ = 510 ft/s)

How to Address Unconservative Spectrum?

- Unconservative values occur on Site Class D and E soils at higher ground motions
- ASCE 7-16 resolves the issue by requiring a site-specific ground motion hazard analysis (per Section 21.2, which will generate response points at multiple periods) in the following cases:
  - Structures on Site Class E with S_S greater than or equal to 1.0
  - Structures on Site Class D and E sites with S_1 greater than or equal to 0.2.

F_a	S_s<=0.25		S_s=0.5		S_s=0.75		S_s=1.0		S_s=1.25		S_s>= 1.5
	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-16
A	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
B	1.0	0.9	1.0	0.9	1.0	0.9	1.0	0.9	1.0	0.9	0.9
C	1.2	1.3	1.2	1.3	1.1	1.2	1.0	1.2	1.0	1.2	1.2
D	1.6	1.6	1.4	1.4	1.2	1.2	1.1	1.1	1.0	1.0	1.0
E	2.5	2.4	1.7	1.7	1.2	1.3	1.1	1.2*	1.0	1.2*	1.2*

F_v	S_1<=0.1		S_1=0.2		S_1=0.3		S_1=0.4		S_1=0.5		S_1>= 0.6
	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-16
A	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
B	1.0	0.8	1.0	0.8	1.0	0.8	1.0	0.8	1.0	0.8	0.8
C	1.7	1.5	1.6	1.5	1.5	1.5	1.4	1.5	1.3	1.5	1.4
D	2.4	2.4	2.0	2.2*	1.8	2.0*	1.6	1.9*	1.5	1.8*	1.7*
E	3.5	4.2	3.2	**	2.8	**	2.4	**	2.4	**	**

Note that for Site Class F soils, a "site response analysis" (21.1) is required. This is different from a "ground motion hazard analysis" (21.2)

The * and ** will be discussed in future slides.

A ground motion hazard analysis is defined in Section 21.2

21.2 RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION HAZARD ANALYSIS

Exceptions to Requirement for Site-Specific Ground Motion Hazard Analysis

- 11.4.8 Exception 1: "Structures on Site Class E sites with S_S greater than or equal to 1.0, provided the site coefficient Fa is taken as equal to that of Site Class C."

F_a		S_S=1.0		S_S=1.25		S_S>=1.5
	7-16	7-10	7-16	7-10	7-16	7-16
C	1.2	1.0	1.2	1.0	1.2	1.2
E	1.3	1.1	1.2*	1.0	1.2*	1.2*

* Equal to Site Class C unless site-specific ground motion hazard analysis is used

Effect is to increase the S_{DS} Portion of the spectrum

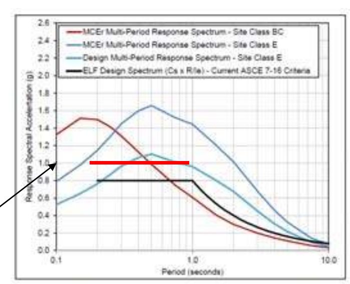


FIGURE C11.4-3 Comparison of ELF and Multi-Period Design Spectra – Site Class E Ground Motions ($v_{s,30}$ = 510 ft/s)

Exceptions to Requirement for Site-Specific Ground Motion Hazard Analysis

- 11.4.8 Exception 3: “Structures on Site Class E sites with S_1 greater than or equal to 0.2, provided that T is less than or equal to T_S and the equivalent static force procedure is used for design.” (IE: Short, stiff buildings, with moderate shaking)

F_v	S_1<=0.1		S_1=0.2		S_1=0.3		S_1=0.4		S_1=0.5		S_1>=0.6
	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-16
E	3.5	4.2	3.2	**	2.8	**	2.4	**	2.4	**	**

** No values are provided here because a site-specific hazard analysis is required.

F_a	S_s<=0.25		S_s=0.5		S_s=0.75		S_s=1.0		S_s=1.25		S_s>=1.5
	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-16
A	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
B	1.0	0.9	1.0	0.9	1.0	0.9	1.0	0.9	1.0	0.9	0.9
C	1.2	1.3	1.2	1.3	1.1	1.2	1.0	1.2	1.0	1.2	1.2
D	1.6	1.6	1.4	1.4	1.2	1.2	1.1	1.1	1.0	1.0	1.0
E	2.5	2.4	1.7	1.7	1.2	1.3	1.1	1.2*	1.0	1.2*	1.2*

If S_S>=1.0, Exception 1 applies.

Exception 3 only applies if S_S is < 1.0

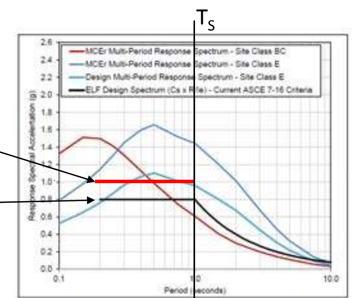


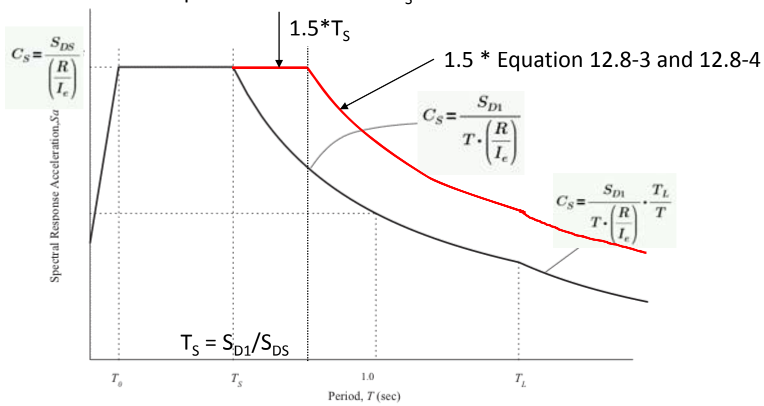
FIGURE C11.4-3 Comparison of ELF and Multi-Period Design Spectra – Site Class E Ground Motions ($v_{s,30}$ = 510 ft/s)

Exceptions to Requirement for Site-Specific Ground Motion Hazard Analysis

- 11.4.8 Exception 2: “Structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_S is determined by Eq. (12.8-2) for values of T <= 1.5*T_S and taken as equal to 1.5 times the value computed in accordance with either: Eq. (12.8-3) for 1.5T_S <= T <= T_L or Eq. (12.8-4) for T > T_L.”

Exception 2

Extend Equation 12.8-2 to 1.5*T_S



T_S ~ .6 Seconds.

This is equivalent to:

- A 6 Story Moment Frame
- A 55 high foot buckling-restrained braced frame or moment frame

▪Structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_S is determined by Eq. (12.8-2) for values of T <= 1.5*T_S

▪ and taken as equal to 1.5 times the value computed in accordance with either:

▪Eq (12.8-3) for 1.5T_S <= T <= T_L or Eq. (12.8-4) for T > T_L.

When building period is longer than T_S, Exception 2 significantly increases base shear

F_v	S_1=0.2		S_1=0.3		S_1=0.4		S_1=0.5		S_1>=0.6
	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-16
D	2.0	2.2*	1.8	2.0*	1.6	1.9*	1.5	1.8*	1.7*
If no Site-Specific Analysis, Multiply C_S by 1.5 for long-period buildings. (Similar to increasing Site coefficients by 1.5)		3.3		3.0		2.85		2.7	2.55
Increase over 7-10		1.3		1.2		1.25		1.2	1.05
Percentage increase		65%		80%		78%		80%	70%

Recap of F_a and F_v

F_a	S_S<=0.25		S_S=0.5		S_S=0.75		S_S=1.0		S_S=1.25		S_S>= 1.5
	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-16
A	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
B Measured	1.0	0.9	1.0	0.9	1.0	0.9	1.0	0.9	1.0	0.9	0.9
B Unmeasured	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
C	1.2	1.3	1.2	1.3	1.1	1.2	1.0	1.2	1.0	1.2	1.2
D	1.6	1.6	1.4	1.4	1.2	1.2	1.1	1.1	1.0	1.0	1.0
Default	1.6	1.6	1.4	1.4	1.2	1.2	1.1	1.2	1.0	1.2	1.2
E	2.5	2.4	1.7	1.7	1.2	1.3	1.1	1.2*	1.0	1.2*	1.2*

F_v	S_1<=0.1		S_1=0.2		S_1=0.3		S_1=0.4		S_1=0.5		S_1>=0.6
	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-10	7-16	7-16
A	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
B Measured	1.0	0.8	1.0	0.8	1.0	0.8	1.0	0.8	1.0	0.8	0.8
B Unmeasured	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
C	1.7	1.5	1.6	1.5	1.5	1.5	1.4	1.5	1.3	1.5	1.4
D	2.4	2.4	2.0	2.2**	1.8	2.0**	1.6	1.9**	1.5	1.8**	1.7**
Default	2.4	2.4	2.0	2.2**	1.8	2.0**	1.6	1.9**	1.5	1.8**	1.7**
E	3.5	4.2	3.2	***	2.8	***	2.4	***	2.4	***	***

- * Site-specific ground motion hazard analysis is required, or use S_S equal to Site Class C (Exception 1)
- ** Site-specific ground motion hazard analysis is required, or extend use of Equation 12.8-2 for building periods up to 1.5*T_S, and use 1.5*Cs for building periods greater than 1.5*T_S (Exception 2). This has the effect of increasing the F_v values as shown.
- *** Site-specific ground motion hazard analysis is required if S_1 is greater than or equal to 0.2. An exception is allowed (Exception 3), but it only uses F_a values and the building period must be less than T_S, and the ELF procedure must be used (I.E.: Short stiff buildings on Site Class E do not require a site-specific ground motion hazard analysis if the ELF procedure is used). No values are provided for F_v because a site-specific hazard analysis is required for building periods greater than T_S.

Sample Changes, S_{DS} (Short Period Buildings)

Location	S_{DS}	A	B Measured	B Unmeasured	C	D	Default	E
Logan	+9%	+8%	-2%	+9%	+29%	+6%	+18%	+40%
Brigham City	-6%	-6%	-16%	-6%	+12%	-6%	+12%	+25%
Murray	-4%	-4%	-14%	-4%	+15%	-4%	+15%	+15%
St. George	+2%	+2%	-8%	+2%	+10%	+1%	+1%	+1%
Monticello	+15%	+16%	+4%	+15%	+25%	+15%	15%	+10%

Sample Changes, S_{D1} (Long Period Buildings)

Location	S_{D1}	A	B Measured	B Unmeasured	C	D	Default	E
Logan (S1 > 0.2)	+14%	+13%	-9%	+14%	+15%	+24% (+59% Base Shear)	+24%	*
Brigham City (S1 > 0.2)	-6%	-6%	-25%	-6%	+8%	+13% (+40% Base Shear)	+13%	*
Murray (S1 > 0.2)	-4%	0%	-20%	-1%	13%	+17% (+43% Base Shear)	+17%	*
St. George (S1 < 0.2)	+8%	+7%	-14%	+8%	-2%	+12%	+12%	+17%
Monticello (S1 < 0.2)	+6%	+3%	-17%	+6%	-7%	+5%	+5%	+26%

* Site-specific ground motion hazard analysis is required

Take Home Points

1. If the geotechnical report lists Site Class B, you must know if site-specific velocity measurements were used to classify the soil. (Measured vs. Unmeasured)
2. Assuming a Site Class D soil without a geotechnical report could be expensive. The base shear will be 9% higher for S_S=1.0 and 20% higher for S_S > 1.25 than if you had a geotechnical report that defined a Site Class D soil.
3. If you are on Site Class D soil (assumed or geotechnical report define) and S_1 >= 0.2, you must have a site-specific ground motion hazard analysis. See exception 2.
4. If you are on Site Class E soil with S_S >= 1.0, you must have a site-specific ground motion hazard analysis. See exception 1.
5. If you are on Site Class E soils with S_1 >= 0.2, you must have a site-specific ground motion hazard analysis. See exception 3 for building period less than T_S. No exception for long-period buildings (building period greater than T_S).

Questions?

Diaphragm Design

THE RIGID, THE FLEXIBLE, AND THE SEMIRIGID

ASCE 7-16 SECTIONS 12.3 &12.10

LUKE BALLING SE

Diaphragm Design Overview

- Flexible Diaphragm Analysis
- Rigid Diaphragm Analysis
- Semirigid Diaphragm Analysis
- Horizontal Structural Irregularities
- Diaphragms, Chords, and Collectors
- 3 Sided Semirigid Diaphragm Design Example

Diaphragm Flexibility

12.3.1 **Diaphragm Flexibility**. "Unless a diaphragm can be idealized as either flexible or rigid in accordance with sections 12.3.1.1, 12.3.1.2, or 12.3.1.3, the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semirigid modeling assumption).

All diaphragms can be analyzed as semirigid even if they can be idealized as rigid or flexible.

Flexible Diaphragm Condition

12.3.1.1 **Flexible Diaphragm Condition**. Diaphragms constructed of untopped steel decking or wood structural panels are permitted to be idealized as flexible if any of the following conditions exist:

- In steel structures where the vertical elements are steel braced frames; steel and concrete composite braced frames; or concrete, masonry, steel, or steel and concrete composite shear walls. (**Moment frames need to be analyzed as rigid or semirigid unless section 12.3.1.3 is satisfied**)
- In one- and two-family dwellings.

Flexible Diaphragm Condition

- In structures of light frame construction where all of the following conditions are met:
  - No topping or only 1 ½" maximum thickness non-structural topping slab (gypcrete)
  - Each line of vertical elements of the seismic force resisting system complies with the allowable story drift of Table 12.12-1 (this requirement is intended as an indicator that the shear walls are substantial enough to share load on a tributary area basis and not require torsional force distribution.)

"The diaphragms in most buildings braced by wood light-framed shear walls are semirigid." ASCE 7-16 Commentary

Flexible Diaphragm Analysis

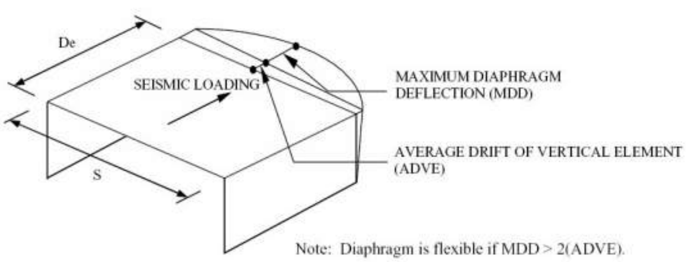


FIGURE 12.3-1 FLEXIBLE DIAPHRAGM

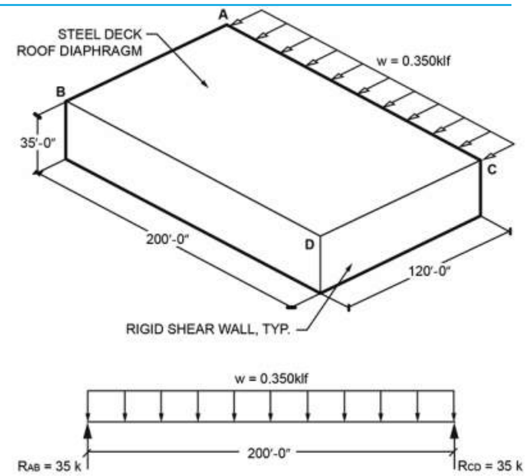
Section 12.3.1.3 **Calculated Diaphragm Condition.**

- Diaphragm not meeting rigid diaphragm conditions are permitted to be idealized as flexible if MDD>2(ADVE).
- This requires a diaphragm analysis to determine deflections or use semirigid analysis to determine diaphragm deflections.

Flexible Diaphragm Analysis

Section 12.8.4.1

- Torsion analysis not required for flexible diaphragms
- The distribution of forces to the vertical elements shall account for the position and distribution of the masses supported.
- Table 12.2-1 footnote b – Where the tabulated value of the overstrength factor, Ω_o , is greater than or equal to 2½, Ω_o is permitted to be reduced by subtracting ½ for structures with flexible diaphragms.



Flexible Diaphragm Story Drift

Section 12.8.6 **Story Drift Determination.**

- "The design story drift shall be computed as the difference in deflections at the center of mass at the top and bottom of the story under consideration."
- Center of mass typically occurs at the center of the diaphragm and therefore; diaphragm deflection needs to be considered for story drifts.

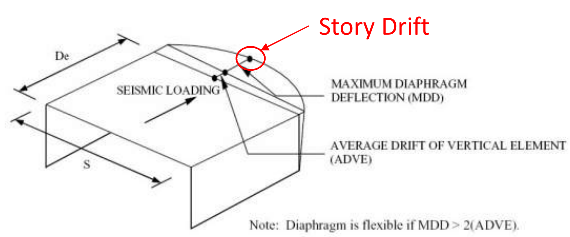
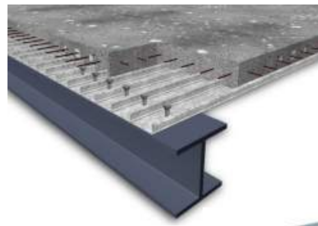
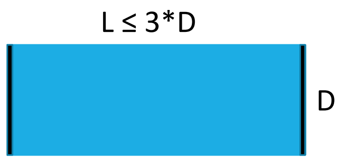


FIGURE 12.3-1 FLEXIBLE DIAPHRAGM

Rigid Diaphragm Condition

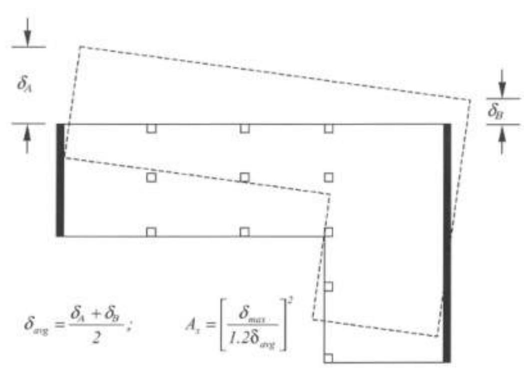
Section 12.3.1.1 **Rigid Diaphragm Condition**. Diaphragms of concrete slabs or concrete filled metal deck with a span-to-depth ratio of 3 or less in structures that have no horizontal irregularities are permitted to be idealized as rigid.





Rigid Diaphragm Analysis

- Inherent torsional analysis per Section 12.8.4.1 (Where center of rigidity and center of mass do not align)
- Accidental Torsion of 5% building length perpendicular to seismic force per Section 12.8.4.2
- Accidental Torsion is not required to be applied for determination of story drifts unless torsional irregularity types 1a and 1b for seismic design categories C-F, but must be applied for torsional irregularity checks and for design forces.
- Amplification of Accidental Torsion Moment for seismic design category C-F for horizontal torsional irregularities Type 1a or 1b.



Semirigid Diaphragm Condition

- Section 12.3.1 "The structural analysis shall explicitly include consideration of the stiffness of the diaphragm."
  - Model roof deck and floor decking stiffness.
  - Model diaphragm chord and drag elements and their stiffness (assign them as lateral members in model)
  - Inherent torsional analysis per Section 12.8.4.1. **Even if analyzing a diaphragm that can be idealized as flexible.**

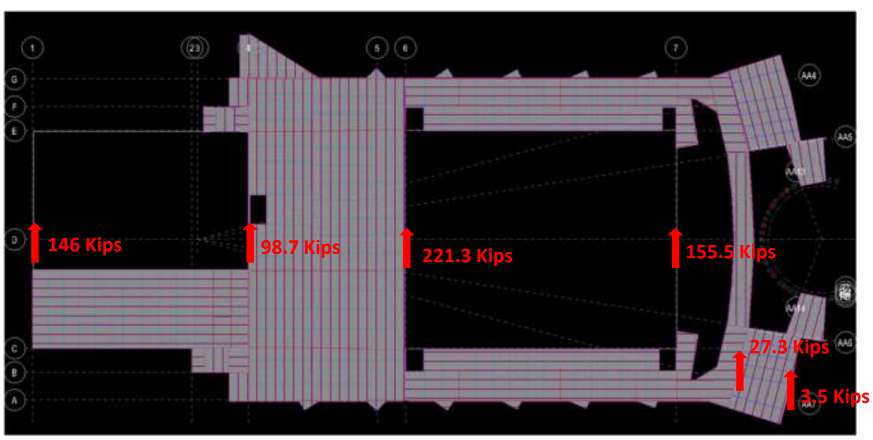
Semirigid Diaphragm Condition

- Accidental Torsion of 5% per Section 12.8.4.2
- Accidental Torsion is not required to be applied for determination of story drifts unless torsional irregularity types 1a and 1b for seismic design categories C-F, but must be applied for torsional irregularity checks and for design forces.
- Amplification of Accidental Torsion Moment for seismic design category C-F for horizontal torsional irregularities Type 1a or 1b.

Semirigid Diaphragm Design Programs

- RAM Structural System (Bentley)
- STAAD (Bentley)
- ETABS (CSI)
- SAP2000 (CSI)
- RISA 3D
- Tekla Structural Designer

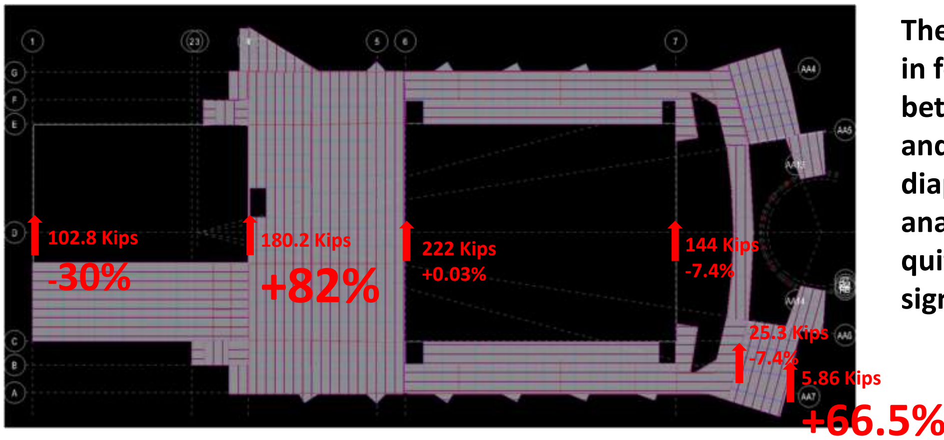
TBSE Project Rigid Diaphragm Forces



Horizontal Irregularities?
- Reentrant corner
- Diaphragm discontinuities

Image Indicates Major Shear Wall and Brace Forces

TBSE Project Semirigid Diaphragm Forces



The difference in forces between rigid and semirigid diaphragm analysis can be quite significant!

Diaphragm Failure In Bhuj Earthquake

Underutilization of Shear Capacity of Elevator Core <u>Due to Improper Diaphragm Action of Slabs</u> Resulted in Failure of an Apartment Building

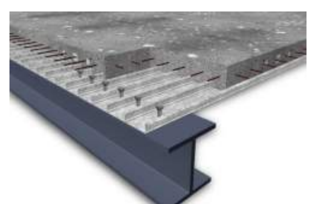
Diaphragm Failure In Anchorage Earthquake

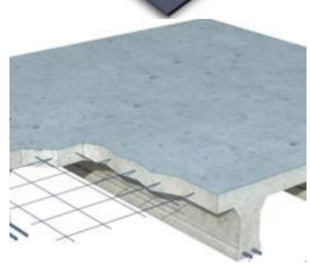
Horizontal diaphragms are not always rigid elements capable of distributing forces between frames. In this Anchorage school a reinforced concrete roof slab has torn like a piece of cardboard.

Semirigid Diaphragm Analysis

Concrete on metal deck and suspended concrete slab stiffness:

- Use thickness of concrete above flutes for concrete on metal deck.
- For concrete suspended slabs and concrete on metal deck use stiffness factors of 0.7 for uncracked concrete or 0.35 for cracked concrete (ACI 318-11 section 10.10.4.1.). Use envelope of worst case forces of both cracked and uncracked factors.





Semirigid Diaphragm Analysis

Metal Roof Decking Stiffness:

Verco and ASC decking provide a flexibility factor(F) in the product catalogs based on deck gauge and attachments. Vulcraft gives G' directly.

$F = \#.\# + \#.\#\,R$ (micro inches per pound)

$R = L_v/L$

L_v= Vertical load span

L= Deck Panel Length

R=1/3 for typical triple span condition

Metal Roof Decking Stiffness Equations:

G'= 1/F = Equivalent Shear Modulus

Effective G = G'/t per SDI

t = deck thickness (inches)

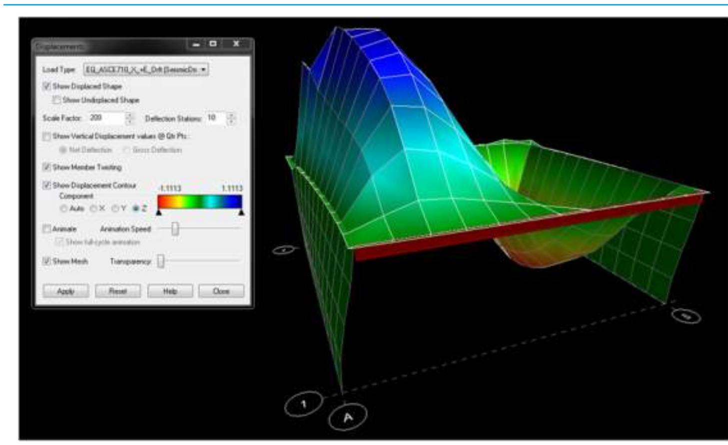
ν = poisson's ratio = 0.3 for steel

$E' = \frac{G'}{t}(2(1+\nu))$ = Effective Young's Modulus

For modeling vertical stiffness of deck it is recommended to use the deck profile thickness and a to reduce E' by actual deck thickness to profile thickness ratio.

Scaled $E'=E'(t/t_{profile})$

Semirigid model with actual deck thickness

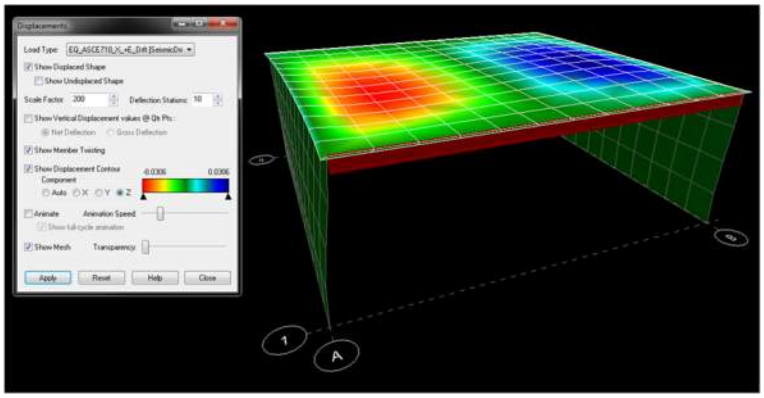


Model with horizontal seismic loads only and using deck gauge thickness and Effective Young's Modulus for semirigid diaphragm input.

Deck is buckling because it has very low vertical stiffness which is not the case for typical roof decking.

Note: Deflections are scaled by a factor of 200.

Semirigid model with profile deck thickness



Model with horizontal seismic loads only and using <u>deck profile thickness t=1.5"</u> and <u>Scaled Effective Young's Modulus</u> for semirigid diaphragm input.

Vertical deck buckling does not occur. <u>Model more accurately models diaphragm deflections.</u>

Horizontal Structural Irregularities

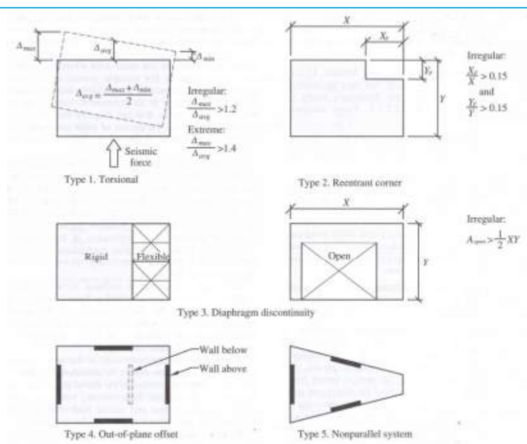


FIGURE C12.3-1 Horizontal Structural Irregularity Examples

If any of these horizontal structural irregularities occur the <u>diaphragm must be modeled as semirigid</u> or idealized as flexible.

Horizontal Structural Irregularities

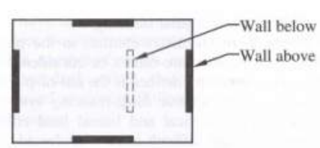
- Section 12.3.3.1 Extreme torsional irregularity type 1b is **prohibited in seismic design categories E-F**
- Section 12.3.3.4 For structures assigned to Seismic Design Categories D-F and having horizontal structural Irregularity of **Type 1a, 1b, 2,3,or 4** the **design forces shall be increased by 25% for the following elements** of the seismic force resisting system:
  - **Connections of the diaphragms** to vertical elements and to collectors
  - **Collectors and their connections**, including connections to vertical elements, of the seismic force resisting system. (Not required if overstrength factor is applied)

"Even where such irregularities are permitted, they should be avoided whenever possible in all structures"
ASCE 7-16 commentary

Diaphragms, Chords, and Collectors

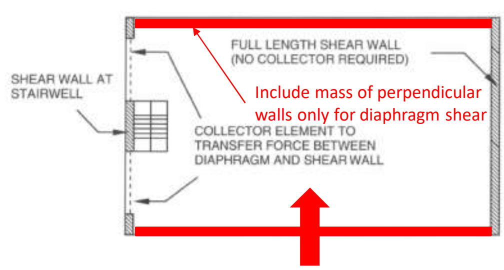
ASCE 7-16 Section 12.10

- Precast concrete diaphragms, chords, and collectors shall be designed per new alternate design provisions under section 12.10.3. Cast-in- place concrete and wood diaphragms may also be designed by this section. **New for ASCE 7-16**
- Diaphragm shear per equation 12.10 -1. Does not include wall mass parallel to seismic force.
- For horizonal irregularity type 4, horizontal shear load shall be increase by overstrength factor in the diaphragm. **Exception:** one- and two- family dwellings of light framed construction.
- Collector elements in seismic design categories C-F shall be designed with overstrength factor **Exception:** wood light-frame shear wall buildings.



Type 4. Out-of-plane offset

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$$



Diaphragms, Chords, and Collectors

ASCE 7-16 Section 12.10

- Section 12.10.1 Diaphragm Design. "At diaphragm discontinuities, such as openings and reentrant corners, the design shall ensure that the dissipation or transfer of edge (chord) forces combined with other forces in the diaphragm is within shear and tension capacity of the diaphragm.

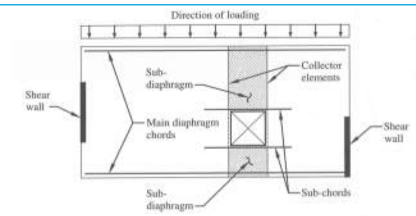


FIGURE C12.10-1 Diaphragm with an Opening

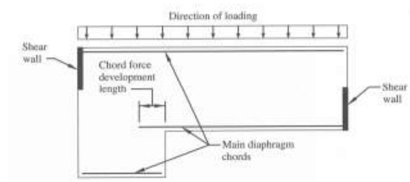
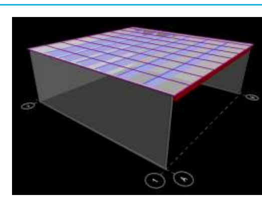


FIGURE C12.10-2 Diaphragm with a Reentrant Corner

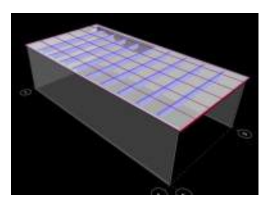
3 Sided Semirigid Diaphragm Examples

Design Assumptions:

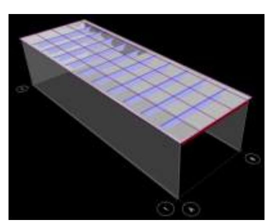
- Investigated three diaphragm aspect ratios, 1:1(42'x42'), 2:1(54'x27'), and 3:1(63'x21'). Each model has same roof area and wall lengths and therefore; same seismic base shear.
- 15' tall building with 8" CMU walls on three sides with 1 ½" type B steel roof decking (3-span minimum) and steel roof joists at 6' o.c.
- Special Reinforced Masonry Shear Walls
  R=5, Ω_o= 2 ½, and C_d=3 ½
- Roof DL = 20 psf, Wall DL = 52 psf
- Seismic Loading: Site Class D soil, S_{DS} = 1.0g, I_e= 1.0



1:1



2:1



3:1

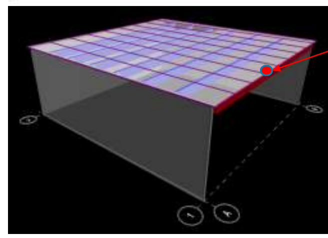
3 Sided Semirigid Diaphragm Examples

ASCE 7-16 Design Requirements:

- All 3 sided diaphragms have extreme torsional irregularity 1b per table12.3-1.
- Increase diaphragm, chord, and collector forces by **25%** per section 12.3.3.4.
- Redundancy factor **ρ= 1.3** for extreme torsional irregularity 1b per section 12.3.4.2
- Amplification of accidental torsional moment per section 12.8.4.3.
- For structures assigned to Seismic Design Category C-F that have horizontal irregularity Type 1a or 1b, the design story drift, Δ, shall be computed as the largest difference of the deflections of vertically aligned points at the top and bottom of the story under consideration along any of the edges of the structure. Section 12.8.4.3
- Limit allowable story drift per table 12.12-1 For masonry walls 0.7%

Table 2. Story drift limits specified by ASCE 7-16

Structural system	Drift limit Risk category I or II	III	IV
Structures, other than masonry wall structures, 4 stories or less above the base with partitions that have been designed to accommodate the story drifts	2.5 %	2.0 %	1.5 %
Masonry cantilever shear wall structures	1.0 %	1.0 %	1.0 %
Other masonry shear wall structures	0.7 %	0.7 %	0.7 %
All other structures	2.0 %	1.5 %	1.0 %



Calculate story drift at worst case, not at center of mass.

3 Sided Semirigid Diaphragm Summary

Model	Δ/Δave	Ax	Ax (%)	Drift (in)	Allowable Drift (in)	Diaphragm Shear (plf)	Deck
1:1	1.960	2.675	13.4%	0.916	1.26	301	20ga 36/5 VSC2 @ 24"
2:1	1.945	2.649	13.2%	1.122	1.26	508	18ga 36/5 VSC2 @ 24"
3:1	1.916	2.569	12.8%	1.199	1.26	599	16ga 36/5 VSC2 @18"

Designs were **governed by diaphragm deflection** and not diaphragm shear.

Note: Stiffness of deck is predominantly governed by span/deck length ratio, deck thickness, and weld/pin pattern. **Sidelap connection has minimal effect on deck stiffness**.

Alternative Design Provision for Diaphragms

Section 12.10.3

- Use for the design of cast-in-place concrete, precast concrete, and wood sheathed diaphragms only.
- Seismic Design Forces for Diaphragms

$F_{px} = (C_{px}/R_S)w_{px} \geq 0.2S_{DS}I_e w_{px}$

C_{px} shall be determined per Figure 12.10.2

Diaphragm Design Force Reduction Factor R_s per table 12.10-1

- Similar to ASCE 41 diaphragm analysis with m- factors.

Table 12.10-1 Diaphragm Design Force Reduction Factor, R_s

Diaphragm System		Shear-Controlled	Flexure-Controlled
Cast-in-place concrete designed in accordance with Section 14.2 and ACI 318	—	1.5	2
Precast concrete designed in accordance with Section 14.2.4 and ACI 318	EDO[a]	0.7	0.7
	BDO[b]	1.0	1.0
	RDO[c]	1.4	1.4
Wood sheathed designed in accordance with Section 14.5 and AWC SDPWS-15	—	3.0	NA

[a]EDO is precast concrete diaphragm elastic design option.
[b]BDO is precast concrete diaphragm basic design option.
[c]RDO is precast concrete diaphragm reduced design option.

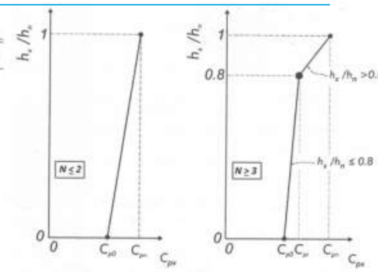


FIGURE 12.10-2 Calculating the Design Acceleration Coefficient C_{px} in Buildings with $N \leq 2$ and in Buildings with $N \geq 3$

Section 14.2.4 indicate detailing requirements for EDO, BDO, and RDO for precast concrete diaphragms.

Figure indicates distribution of story forces along the building height
h_x= Story height
h_n= Total structural height

Summary

- All diaphragms may be modeled as semirigid.
- Flexible diaphragms must meet prescriptive requirements or the deflection criteria of section 12.3.1.3
- Rigid diaphragms must have less than a 3:1 aspect ration and not have any horizontal structural irregularities. Otherwise analyze as semirigid or idealize as flexible.
- Semirigid diaphragm analysis provides an accurate distribution of forces and diaphragm deflections.
- Steel roof deck stiffness equation $E' = \frac{G'}{t}(2(1+\nu))$ = Effective Young's Modulus
- Horizontal irregularities should be avoided, otherwise; there are increases in design forces.
- 3 sided diaphragm designs are typically governed by deflection and diaphragm stiffness is crucial to the design.

Questions?

Modal Response Spectrum Analysis (MRSA)

ASCE 7-16 SECTION 12.9.1
BY ERIC HOFFMAN, PE

ASCE 7-10

12.9.1 Number of Modes

… The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of *at least 90 percent of the actual mass* in each of the orthogonal horizontal directions of response considered by the model.

ASCE 7-16

12.9.1 Number of Modes

… The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of *100% of the structure's mass.* For this purpose, *it shall be permitted to represent all modes with periods less than 0.05 s in a single rigid body mode that has a period of 0.05 s.*

Exception: Alternatively,... *at least 90 percent of the actual mass* in each of the orthogonal horizontal directions...

100% MASS PARTICIPATION

ASCE 7-10

12.9.4.1 Scaling of Forces

… 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 0.85V / V_t.*

ASCE 7-16

12.9.4.1 Scaling of Forces

… Where the combined response for the modal base shear (V_t) is less than *100 percent* of the calculated base shear (V) using the equivalent lateral force procedure, the forces shall be *multiplied by V/V_t.*

100% OF ELF BASE SHEAR

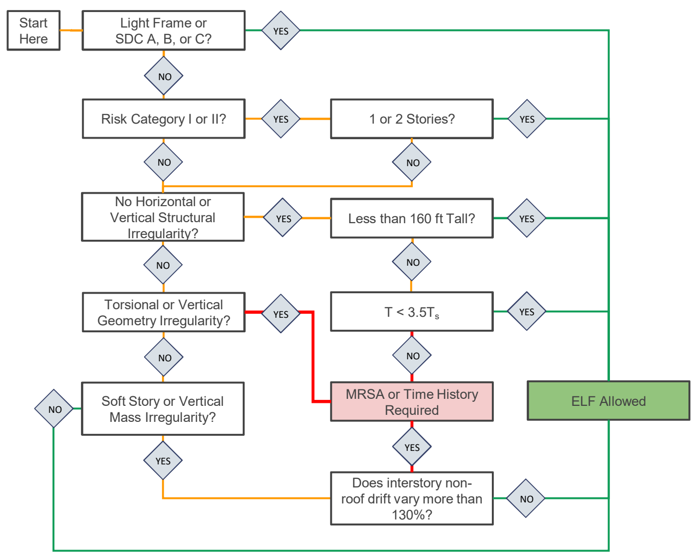
ASCE 7-16 COMMENTARY

C12.9.1.4 Scaling Design Values of Combined Response

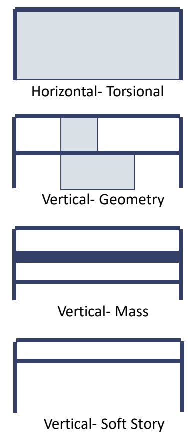
...Recent studies of building collapse performance, such as those of ATC-63, ATC-76 and ATC-84 show that designs based on *the ELF procedure generally result in better collapse performance* than those based on modal response spectrum analysis (MRSA) with the 15% reduction in base shear included. In addition, *many of the designs using scaled MRSA did not achieve the targeted 10% probability of collapse* given MCE ground shaking.

Whereas scaling to 100% of the ELF base shear ... *does not necessarily achieve the intended collapse performance*, it does result in performance that is closer to the stated goals of this standard.

MRSA DOES NOT EQUAL BETTER COLLAPSE PREVENTION.



When do you have to use Time History or Modal Response Spectrum Analysis?



When might you elect to use MRSA?

ASCE 7-16

Maybe use MRSA if-

1. You have heavy floors. ELF tends to “throw” more load to the higher stories. You may get a more inexpensive design using MRSA.
2. As part of an ELF “envelope” style analysis to look for locations of building weakness that ELF may not identify.
3. If your structure’s stiffness and mass distribution is irregular and the assumptions of the ELF procedure do not fit.

Structural Walls and their Anchorage

ASCE 7-16 SECTION 12.11
BY ERIC HOFFMAN, PE

San Fernando Earthquake

February 9th, 1971 – 6:00 am

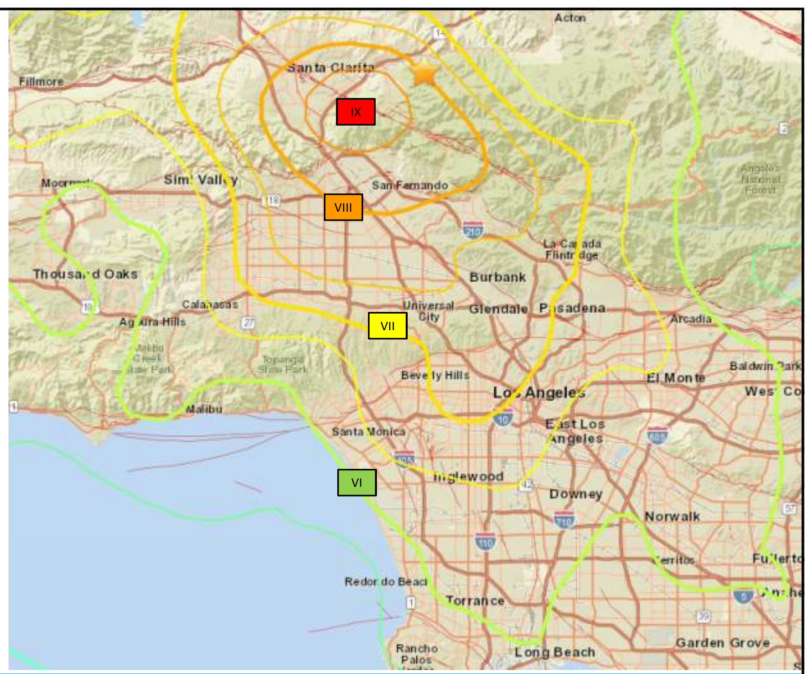
Richter magnitude - M6.6

Max Intensity IX (8.6)

Ss about 2.1g, S1 about 1.7g

58-65 Fatalities

$550 Million in Damages



1971 SAN FERNANDO

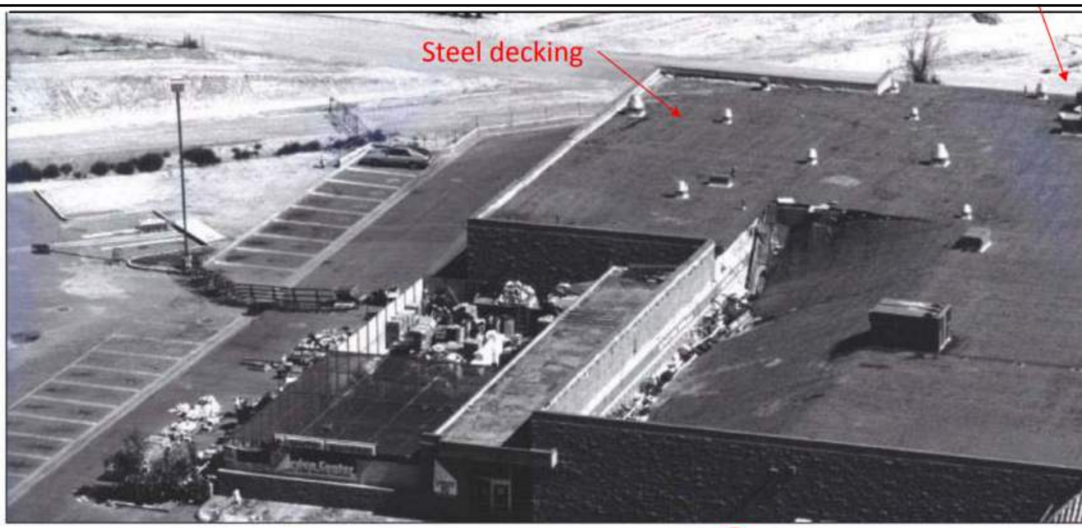
1971 SAN FERNANDO

Major Code Changes after San Fernando

1. Continuous ties required.
2. No wood cross grain bending.
3. Subdiapragms required to transfer forces to ties.
4. Design forces increased by about 50%.

Major Code Changes after Loma Prieta

1. Increase design forces another 50% after Loma Prieta



Failure of Roof Anchorage Due to Inadequate Anchorage and Collector Design (Courtesy Gregg Brandow, Brandow and Johnston)

Masonry Block

1992 LANDERS

1992 LANDERS

Northridge Earthquake

January 17th, 1994 – 4:30 am

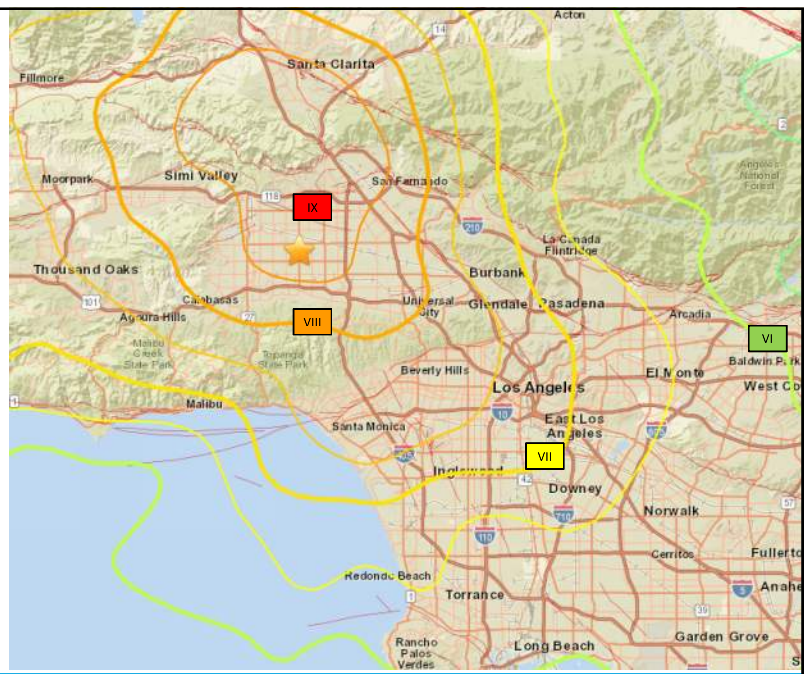
Richter magnitude - M6.7

Max Intensity IX (8.6)

Ss about 1.7g, S1 about 1.1g

57 Fatalities

$13-$44 Billion in Damages



1994 NORTHRIDGE

1994 NORTHRIDGE

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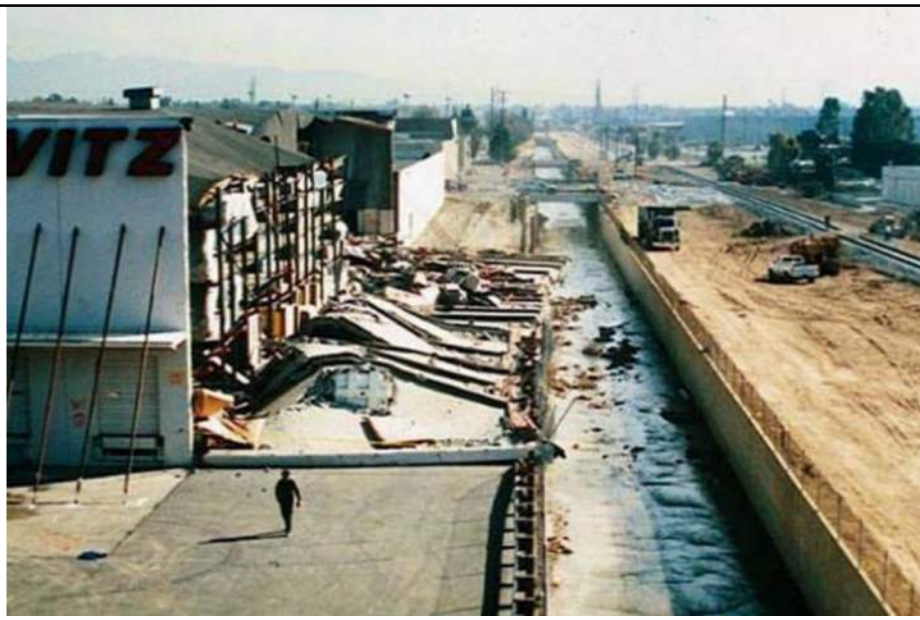

1994 NORTHRIDGE

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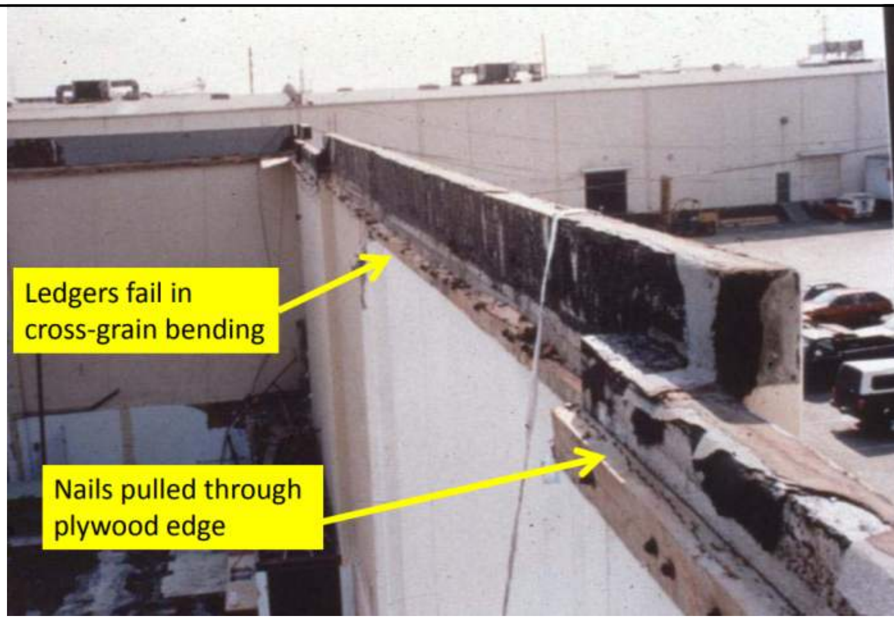
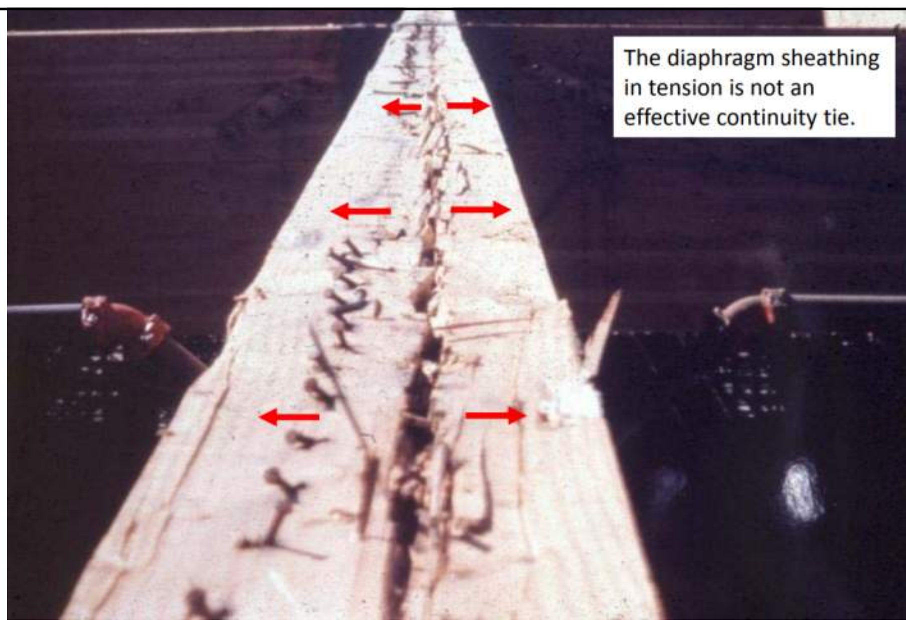


1994 NORTHRIDGE – CROSS GRAIN BENDING/WOOD DIAPHRAGM TENSION

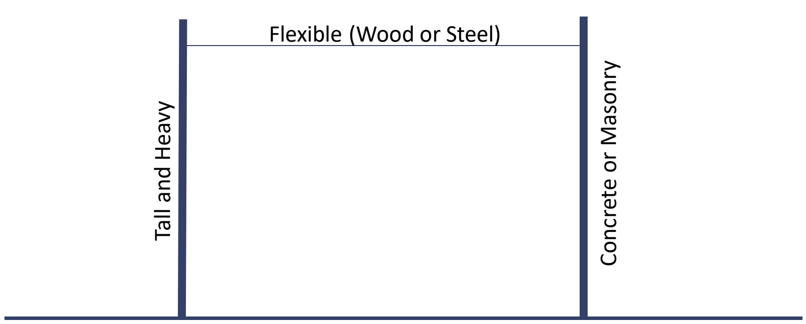


1994 NORTHRIDGE – WOOD DIAPHRAGM TENSION

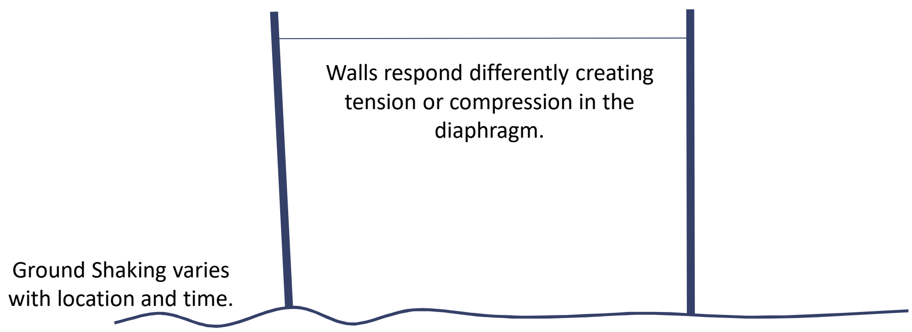
Major Code Changes after Northridge

1. Design forces increased by about another 25%.
2. Increased loading at pilasters.
3. Anchors to be concentrically loaded.
4. An additional 40% increase in forces for steel elements.

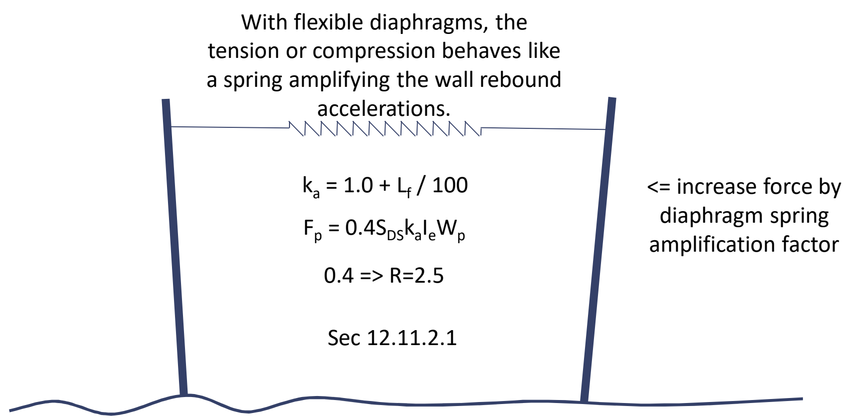
Worst Case-



BUILDINGS MOST AT RISK



WHY WALL ANCHORAGE?

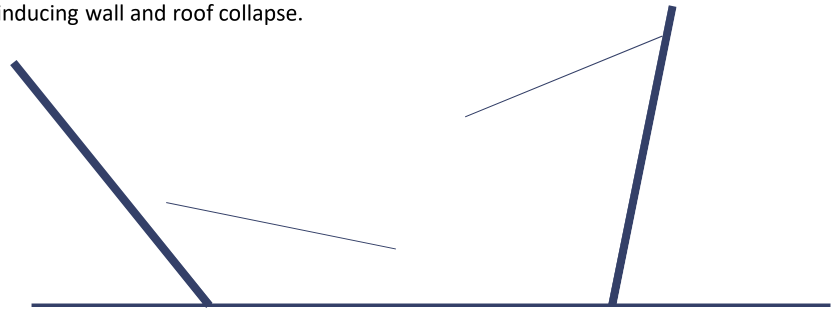


WHY WALL ANCHORAGE?

If not designed properly, walls may -

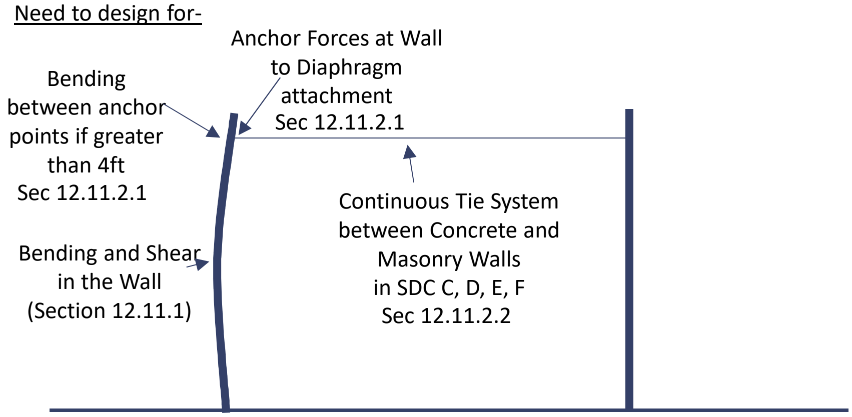
1. Disconnect from the diaphragm or
2. Tear the diaphragm apart

inducing wall and roof collapse.



WHY WALL ANCHORAGE.

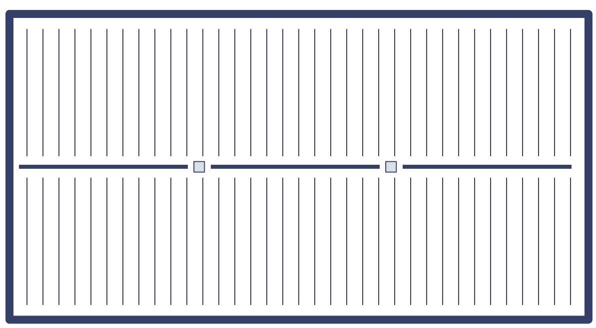
<u>Need to design for-</u>



WHAT NEEDS DESIGNED.

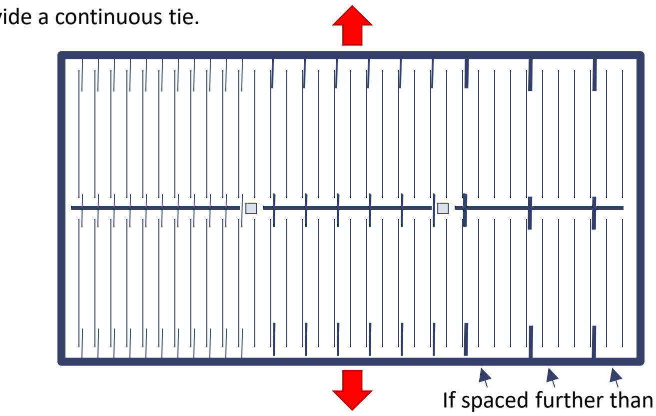
1994 NORTHRIDGE – CONTINUOUS TIES

Rectangular masonry or concrete building with wood sheathing supported by wood framing members. 24” o.c. joists bare on a single post supported girder line.



EXAMPLE : WOOD ROOF

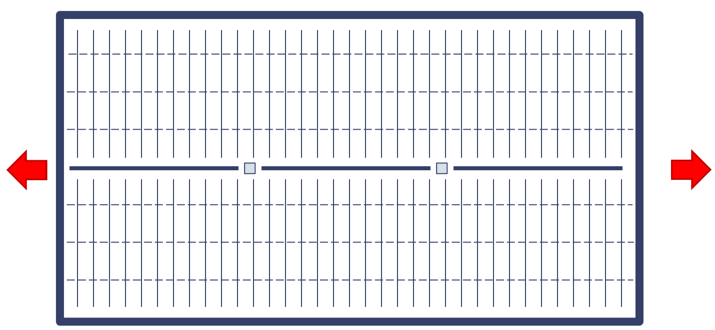
Look at the continuous tie system between the North and South walls. The joists are spaced less than 4ft o.c., so they could each be anchored individually to the wall, but they also need to be strapped across the girder line to provide a continuous tie.



If spaced further than 4' o.c., then the wall must be designed for bending between elements, or subdiaphragms must be provided.

EXAMPLE : WOOD ROOF

Look at the continuous tie system between the East and Walls walls. Wood sheathing CANNOT be used in tension or compression. We could block and strap across the entire building!

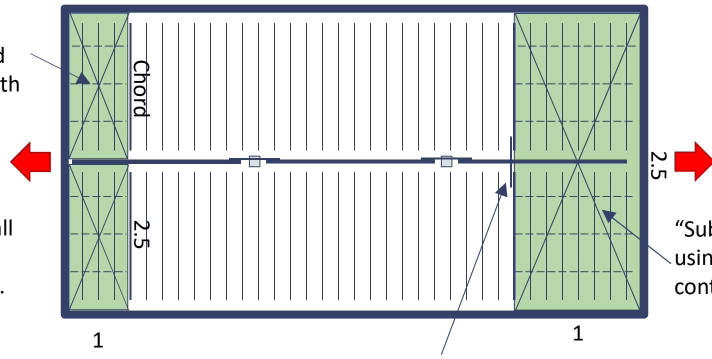


EXAMPLE : WOOD ROOF

Instead we design a "Subdiaphragm" which will transfer the forces to the girder line and the North/South walls which we will use as the continuous ties.

Anchor straps to wall and strap and block the full length of the "Sub-diaphragm"

<= 4ft o.c. or you can design the wall to span between anchors if desired.

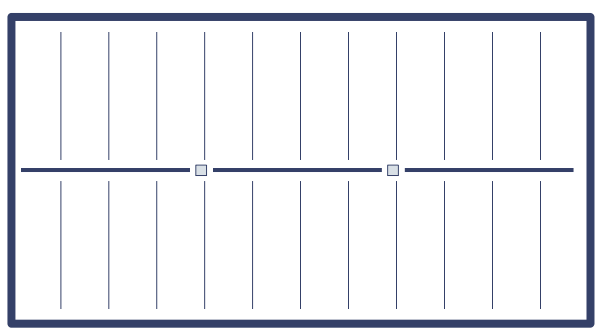


"Subdiaphragm" without using central girderline for continuous tie.

Strap for Chord Continuity

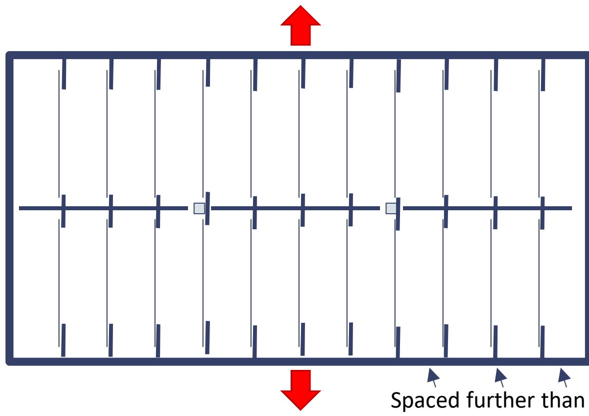
EXAMPLE : WOOD ROOF

Rectangular masonry or concrete building with steel framing at 6' o.c. and steel deck. Steel joists bare on a single post supported girder line.



EXAMPLE : STEEL ROOF

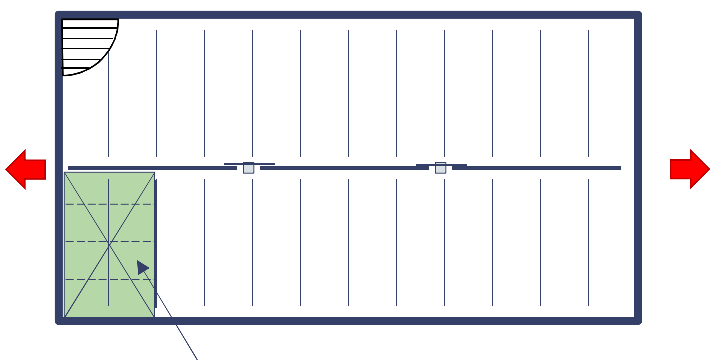
For the North-South direction anchor joists to the wall and provide a continuous load path across the girder line.



Spaced further than 4’ o.c., so the wall must be designed for bending between the joists.

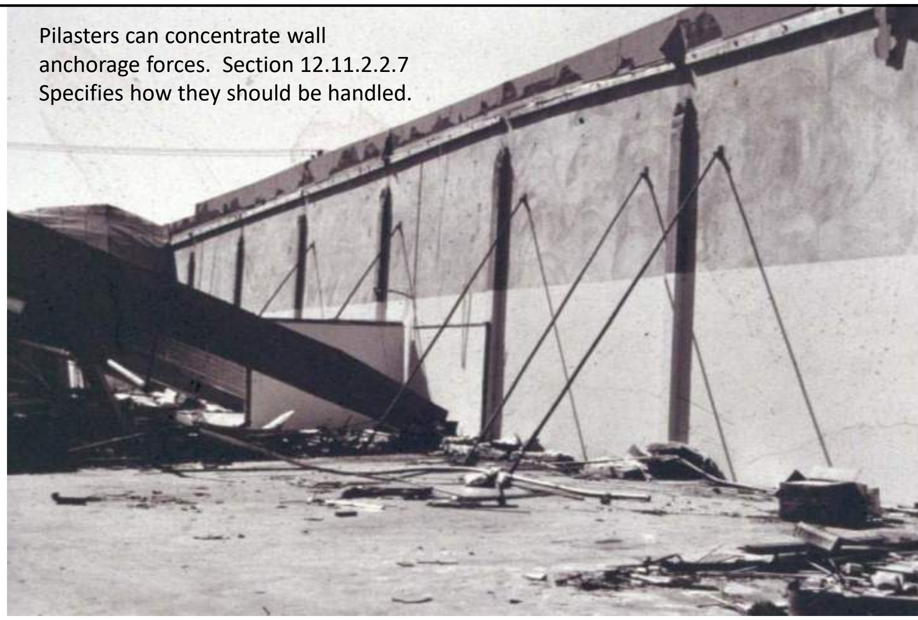
EXAMPLE : STEEL ROOF

For the East-West direction the steel deck can be used if it can handle the tension/compression loads.



If deck can’t handle axial loads, you could design a subdiaphragm and use the girder as a continuous tie.

EXAMPLE : STEEL ROOF



1994 NORTHRIDGE – PILASTER FAILURES

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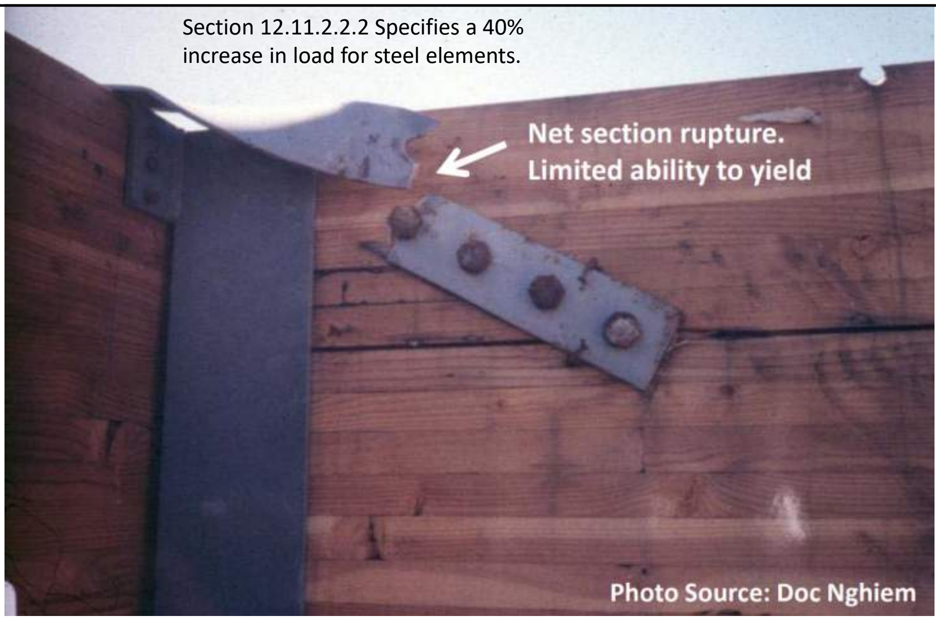
1994 NORTHRIDGE – PILASTER FAILURES

Section 12.11.2.2.2 Specifies a 40% increase in load for steel elements.



1994 NORTHRIDGE – STEEL FAILURES

Important Things to Remember

1. Walls have to be tied together with continuous ties. (Sec 12.11.2.2.1)
2. Anchorage loads to continuous ties spaced more than 4 ft o.c. must be transferred through bending in the wall, or through subdiaphragms. (Sec 12.11.2.1)
3. Subdiaphragms have a maximum aspect of 2.5 to 1 regardless of the material. (Sec 12.11.2.2.1)
4. Wood diaphragms may not resist tension nor compression. (Sec 12.11.2.2.3)
5. Steel deck can only resist tension and compression in the direction parallel to the ribs. Must be designed for those forces. (Sec 12.11.2.2.4)
6. Steel elements shall have anchorage forces increased by 1.4. (Sec 12.11.2.2.2)
7. Embedded straps shall be attached to or hooked around wall reinforcing so as to transfer the force to the reinforcing steel. (Sec 12.11.2.2.5)
8. Be careful with eccentrically loaded anchors.
9. Wall pilasters must be assumed to concentrate the reaction at the roof. Other anchors still must be designed for the full tributary wall load. (Sec 12.11.2.2.7)
10. Continuous tie system has to transfer forces around openings in the diaphragm.
11. Openings near/next to walls may have less than the 2.5 to 1 ratio. Anchoring these walls require the wall to be designed in bending around the opening with concentrated forces at the edges of the opening.
12. Wall anchorage failures account for a large percentage of collapses in past earthquakes. Tall, heavy walls with wood roofs are particularly susceptible and should be given special attention.
13. The code changes are recent and only deemed to comply after the 1997 UBC.
14. The current code provisions haven't been tested by a significant earthquake.

REMEMBER!

Questions?

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