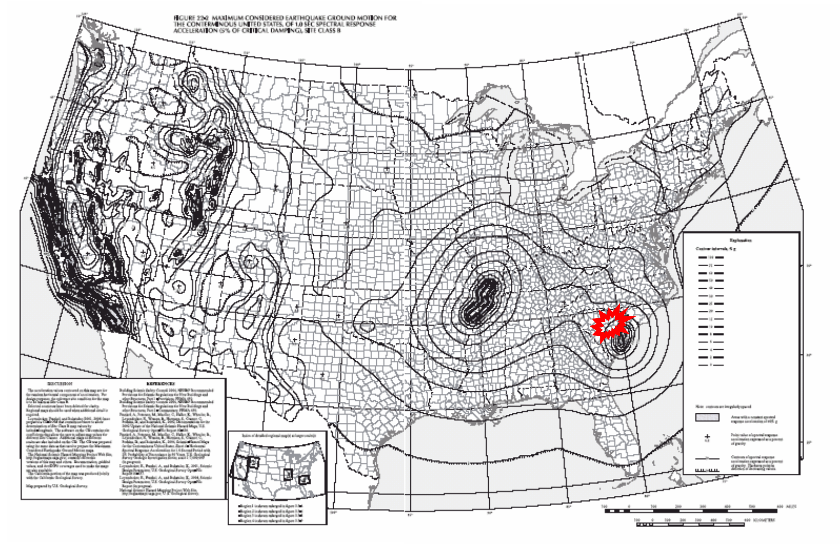
SEISMIC LOAD ANALYSIS



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This topic focuses on seismic load analysis. In the topic, the manner in which ground accelerations impart design loads into the structure are described from a code perspective. Also discussed are modeling approaches, drift computation and acceptance criteria, and p-delta effects.

The topic is presented from the perspective of the seismic portion of ASCE 7-05, which is based on the 2003 *NEHRP Recommended Provisions*. When *Provisions* requirements are different, the differences are indicated.

Topic Objectives

- Selection of method of analysis
- Description of analysis techniques
- Modeling considerations
- System regularity
- Load combinations
- Other considerations
- Drift computation and acceptance criteria
- P-delta effects

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The bulleted items are the main objectives of the topic.

Load Analysis Procedure
(ASCE 7, *NEHRP Recommended Provisions*)

1. Determine building occupancy category (I-IV)
2. Determine basic ground motion parameters (S_S, S_1)
3. Determine site classification (A-F)
4. Determine site coefficient adjustment factors (F_a, F_v)
5. Determine design ground motion parameters (S_{dS}, S_{d1})
6. Determine seismic design category (A-F)
7. Determine importance factor
8. Select structural system and system parameters (R, C_d, Ω_o)

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The topic will basically follow the procedure outlined in the visual. As mentioned earlier, ASCE 7-05 is emphasized. When there are significant differences between ASCE 7 and the 2003 *NEHRP Recommended Provisions*, these differences are noted and discussed.

Load Analysis Procedure
(Continued)

9. Examine system for configuration irregularities
10. Determine diaphragm flexibility (flexible, semi-rigid, rigid)
11. Determine redundancy factor (ρ)
12. Determine lateral force analysis procedure
13. Compute lateral loads
14. Add torsional loads, as applicable
15. Add orthogonal loads, as applicable
16. Perform analysis
17. Combine results
18. Check strength, deflection, stability

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Continuation of previous slide.

Occupancy Category (ASCE 7)

I) Low risk occupancy
- Agricultural facilities
- Temporary facilities
- Minor storage facilities

II) Normal hazard occupancy
- Any occupancy not described as I, III, IV

III) High hazard occupancy
- High occupancy (more than 300 people in one room)
- Schools and universities (various occupancy)
- Health care facilities with < 50 resident patients
- Power stations
- Water treatment facilities
- Telecommunication centers
- Other….

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The first step in any seismic load analysis is the determination of occupancy or "use". ASCE 7 has four Occupancy Categories (I -IV), whereas NEHRP has three Seismic Use Groups (I-III). The relationship between the ASCE 7 and the NEHRP categories is presented on the next slide. Importance factors, which are tied to the occupancy and use, are described later.

Occupancy Category (ASCE 7, continued)

IV) Essential facilities

Hospitals or emergency facilities with surgery
Fire, rescue, ambulance, police stations
Designated emergency shelters
Aviation control towers
Critical national defense facilities
Other….

Note: *NEHRP Recommended Provisions* has Occupancy Categories I-III;
ASCE 7 I+II = NEHRP I, ASCE 7 III = NEHRP II, ASCE 7 IV = NEHRP III

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The relationship between ASCE 7 Occupancy Groups and *NEHRP Recommended Provisions* Seismic Use Groups (SUGs) is shown in the table.

Hazard Maps → Design Ground Motions

- Provide 5% damped firm rock (Site Class B) spectral accelerations S_s and S_1 or 2% in 50 year probability or 1.5 times deterministic peak in areas of western US
- Modified for other site conditions by coefficients F_v and F_a to determine spectral coefficients S_{MS} and S_{M1}
- Divided by 1.5 to account for expected good performance. This provides the design spectral coordinates S_{DS} and S_{D1}.

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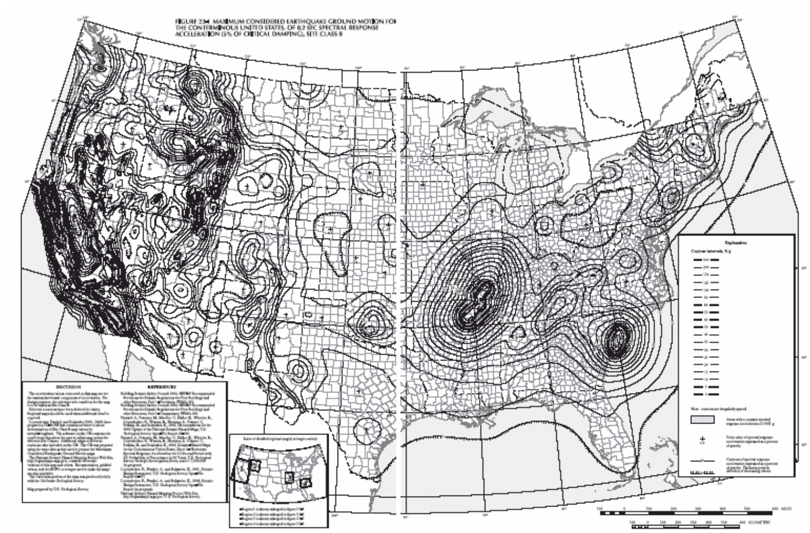
The seismic hazard analysis topic provided a complete review of the hazard maps and described in some detail how the maps were developed. The USE of the maps is presented in this topic.

Regarding the third bullet, those involved in the code deliberations indicate that “the division by 1.5 was to preserve the existing design procedures, specifically including the values for R, even though the design limit state was changed from life safety to collapse prevention.

The *NEHRP Commentary* puts this in quite different terms, stating that the "the design ground motions are based on a lower bound estimate of the margin against collapse inherent in structures designed according to the *Provisions*. This lower bound was judged, based on experience, to correspond to a factor of about 1.5 on ground motion. Consequently, the design ground motion was selected as a shaking level that is 1/1.5 (2/3) times the maximum considered ground motion." A note has been provided to clarify this.

T = 0.2 Spectral Accelerations (S_s) for Conterminous US

(2% in 50 year, 5% damped, Site Class B)



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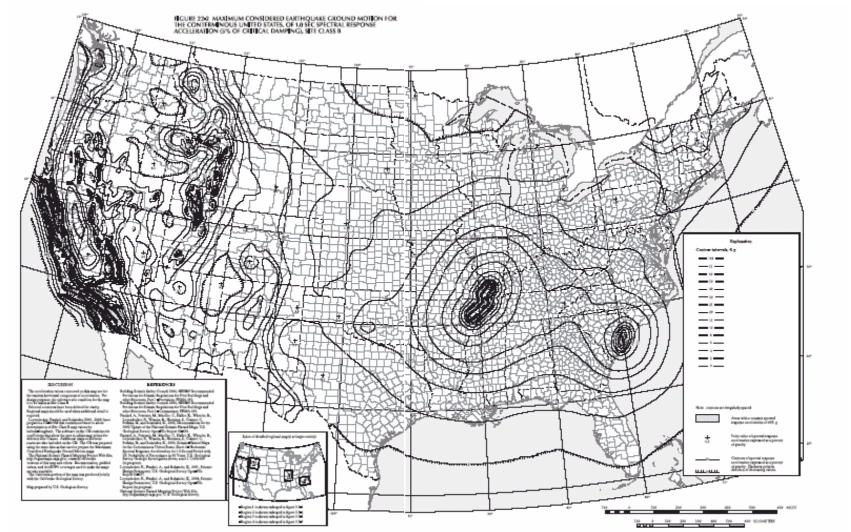
This is one of two maps used by ASCE 7 and the *NEHRP Recommended Provisions* determine the seismic hazard. The development of these maps is covered in detail in the topic on seismic hazard analysis. This map is the 0.2 second spectral pseudo acceleration map for a 2% in 50 year probability of exceedance. Damping is 5% critical, and the basic Site Class is D.

Note that this map is different from the USGS map, principally in California.

Note also that there is no PGA map.

T = 1 Spectral Accelerations (S_1) for Conterminous US
(2% in 50 year, 5% damped, Site Class B)



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This is the map for 1.0 second spectral acceleration.

SITE CLASSES

A Hard rock v_s > 5000 ft/sec

B Rock: 2500 < v_s < 5000 ft/sec

C Very dense soil or soft rock: 1200 < v_s < 2500 ft/sec

D Stiff soil : 600 < v_s < 1200 ft/sec

E V_s < 600 ft/sec

F Site-specific requirements

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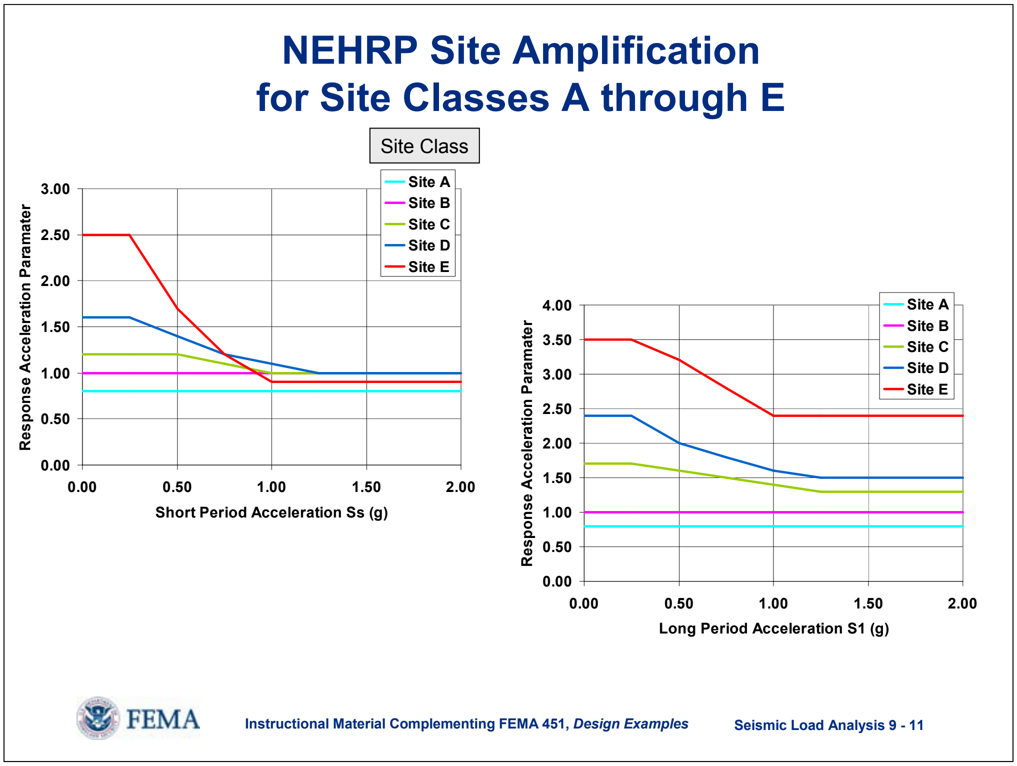
These at the basic site class definitions based on shear wave velocity. Chapter 20 of ASCE 7-05 provides details. The same information is provided in Chapter 3 of the 2003 *NEHRP Recommended Provisions.*



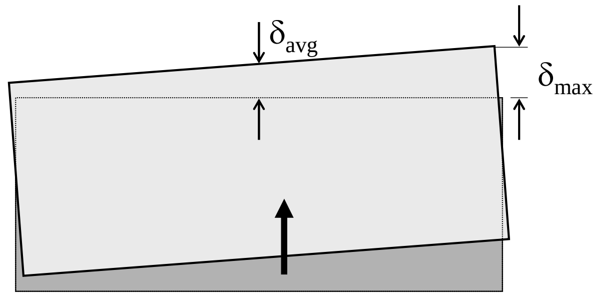
The provisions provide the site amplification coefficient F_v and F_a in tabular form. These have been presented graphically on this slide.

Note that all values in each chart are 1.0 when the Site Class is B. The coefficients are generally greater than one for softer soil sites, with the exception of F_v coefficients for Site Class E when the period is long.

If no site investigation is done, the default Site Class is D.

Horizontal Structural Irregularities

1a) and 1b) Torsional Irregularity



$\delta_{max} < 1.2\delta_{avg}$ No irregularity

$1.2\delta_{avg} \leq \delta_{max} \leq 1.4\delta_{avg}$ Irregularity

$\delta_{max} > 1.4\delta_{avg}$ Extreme irregularity

Irregularity 1b is NOT PERMITTED in SDC E or F.

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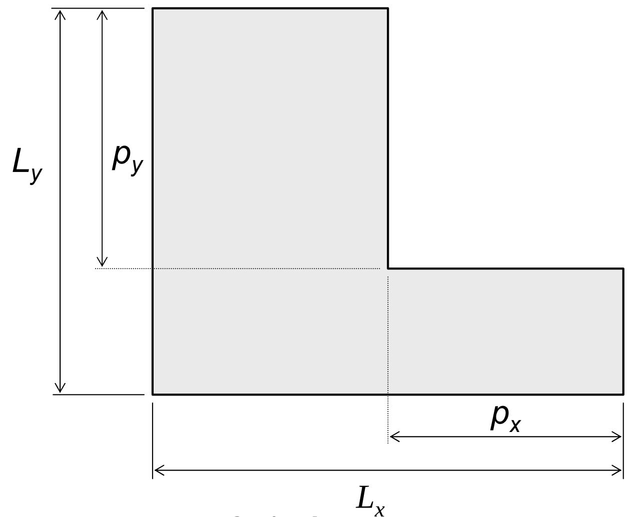
Several design requirements are tied to the notion of regularity. The more regular the system in terms if its lateral load resisting system’s load path, stiffness, and strength the better. When irregularities exist, special requirements often kick in, depending on Seismic Design Category and use.

There are two types of irregularity, horizontal and vertical. The torsional irregularity, when it exists, is due to an inherent torsional response in the system. The analysis described is used to determine if the irregularity exists. Note that the applied loads must be applied with a 5% eccentricity as required for accidental torsion calculations.

Horizontal Structural Irregularities

2) Re-entrant Corner Irregularity

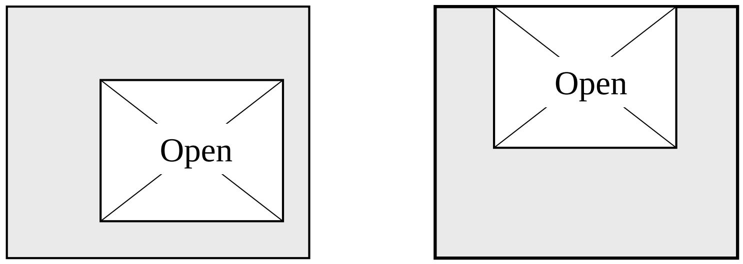


Irregularity exists if $p_y > 0.15L_y$ and $p_x > 0.15L_x$

The re-entrant corner irregularity, if it exists, can cause distress at the reentrant corner. More is said about this in the topic on “configuration.”

Horizontal Structural Irregularities

3) Diaphragm Discontinuity Irregularity



Irregularity exists if open area > 0.5 times floor area OR if effective diaphragm stiffness varies by more than 50% from one story to the next.

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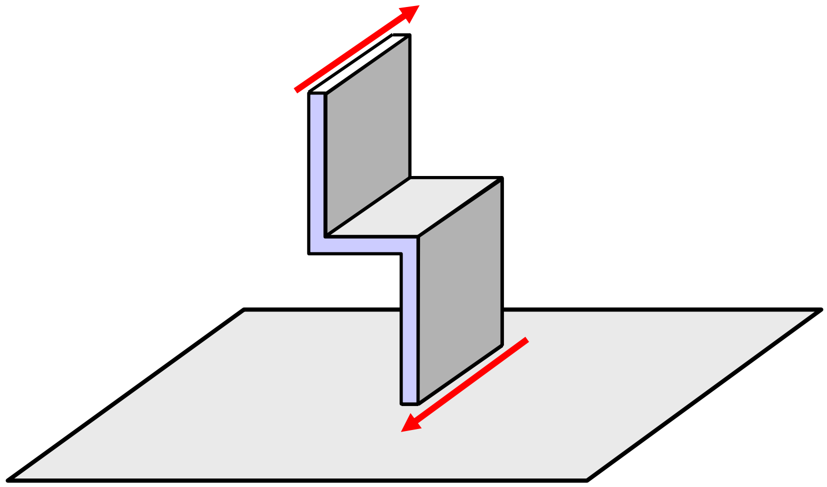
The provisions are not specific on how effective diaphragm stiffness is to be computed.

Horizontal Structural Irregularities

4) Out of Plane Offsets



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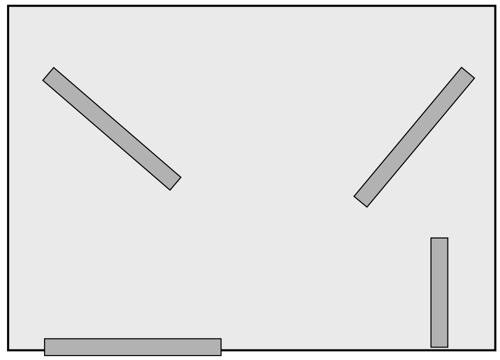
The out of plane offset should be avoided.

Horizontal Structural Irregularities

5) Nonparallel Systems Irregularity



Nonparallel system Irregularity exists when the vertical lateral force resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic force resisting system.

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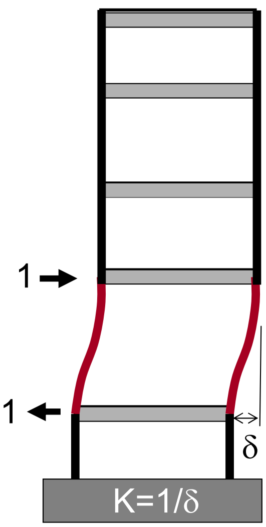
Self explanatory.

Vertical Structural Irregularities

1a, 1b) Stiffness (Soft Story) Irregularity



Irregularity (1a) exists if stiffness of any story is less than 70% of the stiffness of the story above or less than 80% of the average stiffness of the three stories above.

An extreme irregularity (1b) exists if stiffness of any story is less than 60% of the stiffness of the story above or less than 70% of the average stiffness of the three stories above.

Exception: Irregularity does not exist if no story drift ratio is greater than 1.3 times drift ratio of story above.

Irregularity 1b is NOT PERMITTED in SDC E or F.

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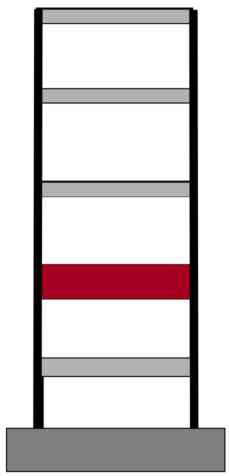
The provisions are not clear on how story stiffness is to be determined. One way to do so is to compute the story displacement under unit forces as shown.

It is recommended that the story drift calculations be made first as this may eliminate the requirement to do the mode detailed story-by-story comparisons.

Vertical Structural Irregularities

2) Weight (Mass) Irregularity



Irregularity exists if the effective mass of any story is more than 150% of the effective mass of an adjacent story.

Exception: Irregularity does not exist if no story drift ratio is greater than 1.3 times drift ratio of story above.

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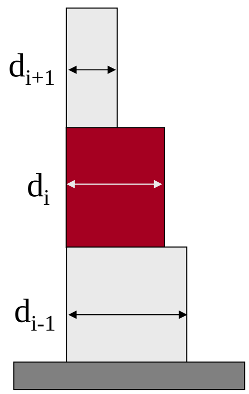
Here, it might be easier to compare the masses.

Vertical Structural Irregularities

3) Vertical Geometric Irregularity



Irregularity exists if the dimension of the lateral force resisting system at any story is more than 130% of that for any adjacent story

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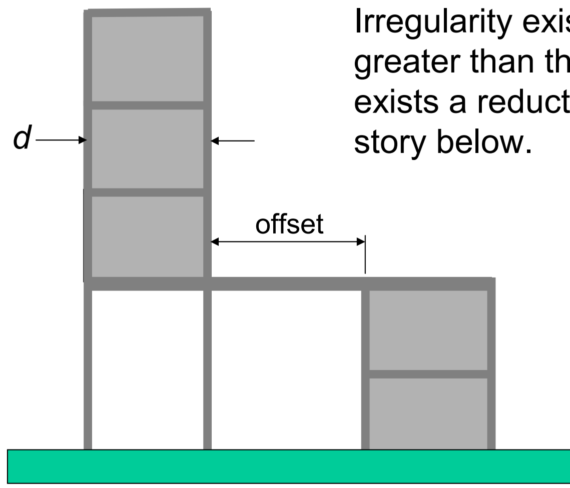
Self explanatory.

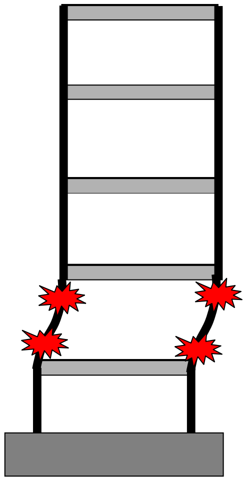
Vertical Structural Irregularities
4) In-Plane Discontinuity Irregularity



Irregularity exists if the offset is greater than the width (*d*) or there exists a reduction in stiffness of the story below.

It is not clear what is meant by “if there exists a reduction in the stiffness of the story below” and how this is different from a soft story irregularity.

Vertical Structural Irregularities
5a, 5b) Strength (Weak Story) Irregularity



Irregularity (5a) exists if the lateral strength of any story is less than 80% of the strength of the story above.

An extreme irregularity (5b) exists If the lateral strength of any story is less than 65% of the strength of the story above.

Irregularities 5a and 5b are NOT PERMITTED in SDC E or F.
Irregularity 5b not permitted in SDC D.

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Determination of the existence of this irregularity is problematic because there is not clear definition of story strength. How does one establish the strength of a story of a moment frame when hinging is expected to occur in the girders? In such frames, story mechanisms are the real problem and that they should be checked even if the columns are nominally stronger than the beams (in concrete systems the beams are usually much stronger than allowed in ACI 318 due to slab reinforcement that doesn't pass through the narrow band at the column really contributing the beam strength anyway).

For shear walls axial force plus bending and shear are both important to be checked, even though in reality it is expected and desired that yielding occurs at the bottom of cantilever wall systems.

Structural Systems

A. Bearing wall systems
B. Building frame systems
C. Moment resisting frame systems
D. Dual systems with SMRF
E. Dual systems with IMRF
F. Ordinary shear-wall frame interactive systems
G. Cantilever column systems
H. Steel systems not detailed for seismic

System Parameters:
Response modification coefficient = $\boldsymbol{R}$
System overstrength parameter = Ω_o
Deflection amplification factor = C_d
Height limitation = by SDC

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ASCE 7-05 lists 15 bearing wall systems, 27 building frame systems, 11 moment resisting frame systems, 22 dual systems, and 7 cantilever systems. Somewhat fewer systems are listed in the 2003 *NEHRP Recommended Provisions* and in ASCE 7-02.

Structural Systems

TABLE 12.2-1 DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC FORCE–RESISTING SYSTEMS

Seismic Force–Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations and Building Height (ft) Limit[c]				
					Seismic Design Category				
					B	C	D[d]	E[d]	F[e]
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls	14.2 and 14.2.3.6	5	$2\frac{1}{2}$	5	NL	NL	160	160	100
2. Ordinary reinforced concrete shear walls	14.2 and 14.2.3.4	4	$2\frac{1}{2}$	4	NL	NL	NP	NP	NP
3. Detailed plain concrete shear walls	14.2 and 14.2.3.2	2	$2\frac{1}{2}$	2	NL	NP	NP	NP	NP
4. Ordinary plain concrete shear walls	14.2 and 14.2.3.1	$1\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{1}{2}$	NL	NP	NP	NP	NP
5. Intermediate precast shear walls	14.2 and 14.2.3.5	4	$2\frac{1}{2}$	4	NL	NL	40[k]	40[k]	40[k]
6. Ordinary precast shear walls	14.2 and 14.2.3.3	3	$2\frac{1}{2}$	3	NL	NP	NP	NP	NP
7. Special reinforced masonry shear walls	14.4 and 14.4.3	5	$2\frac{1}{2}$	$3\frac{1}{2}$	NL	NL	160	160	100
8. Intermediate reinforced masonry shear walls	14.4 and 14.4.3	$3\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{4}$	NL	NL	NP	NP	NP
9. Ordinary reinforced masonry shear walls	14.4	2	$2\frac{1}{2}$	$1\frac{3}{4}$	NL	160	NP	NP	NP
10. Detailed plain masonry shear walls	14.4	2	$2\frac{1}{2}$	$1\frac{3}{4}$	NL	NP	NP	NP	NP
11. Ordinary plain masonry shear walls	14.4	$1\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{1}{4}$	NL	NP	NP	NP	NP
12. Prestressed masonry shear walls	14.4	$1\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{3}{4}$	NL	NP	NP	NP	NP
13. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1, 14.1.4.2, and 14.5	$6\frac{1}{2}$	3	4	NL	NL	65	65	65
14. Light-framed walls with shear panels of all other materials	14.1, 14.1.4.2, and 14.5	2	$2\frac{1}{2}$	2	NL	NL	35	NP	NP
15. Light-framed wall systems using flat strap bracing	14.1, 14.1.4.2, and 14.5	4	2	$3\frac{1}{2}$	NL	NL	65	65	65
B. BUILDING FRAME SYSTEMS									

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This is a portion of a Table 12.2-1 from ASCE 7-05.

The instructor should read off a few values and describe the meaning.

Bearing Wall

- Any metal or wood stud wall that supports more than 100 lbs/ft of vertical load in addition to its own weight
- Any concrete or masonry wall that supports more than 200 lbs/ft of vertical load in addition to its own weight

It appears that almost ANY concrete or masonry wall would be classified as a bearing wall!

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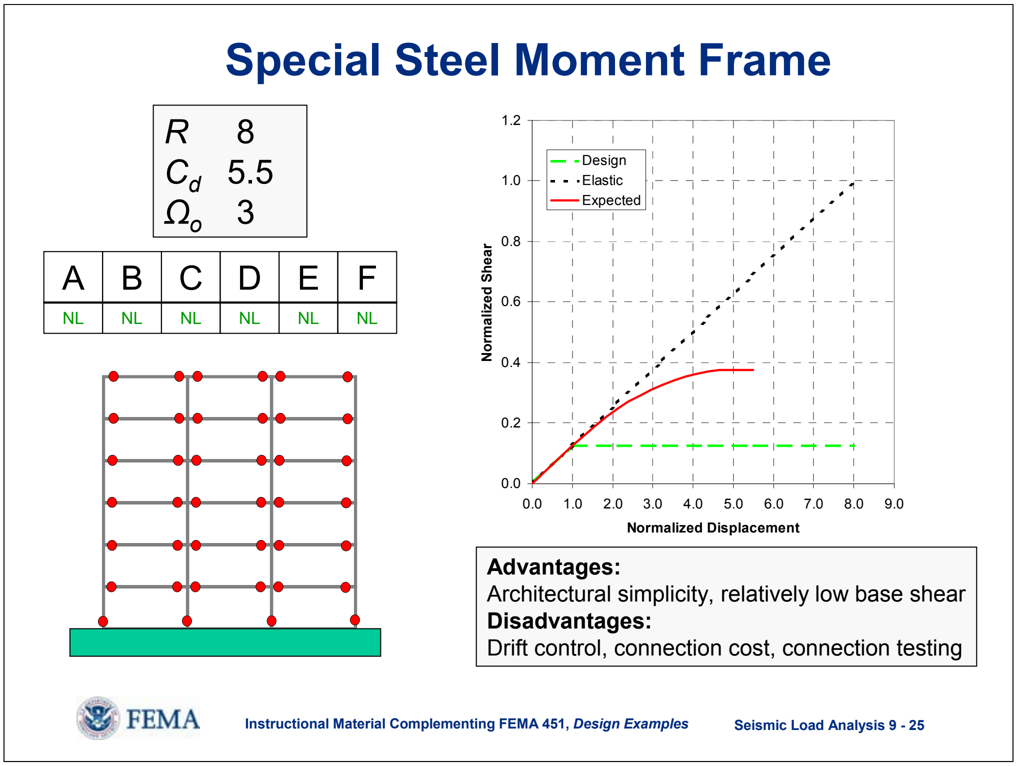
Sometimes it is difficult to determine whether a shear wall is part of a frame system or a bearing wall system as shear walls are included in both classifications.

The definition shown in the slide for bearing wall systems is from ASCE 7-05. The *NEHRP Recommended Provisions* do not provide as detailed a definition.

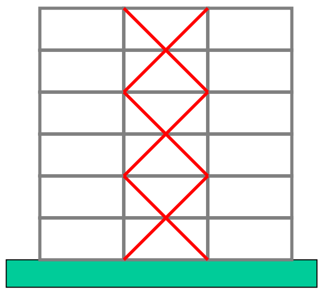


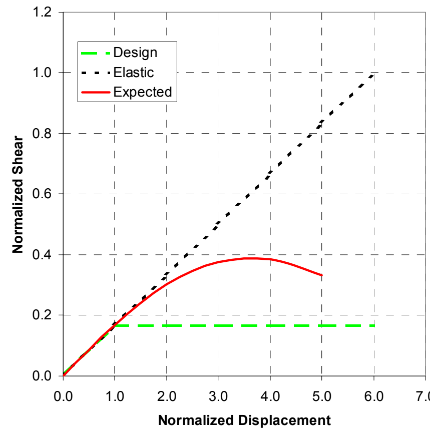
This slide and the two that follow give a more detailed example of the information provided for a particular structural system. Tabulated values are from ASCE 7-05.

Special Steel Concentrically Braced Frame

R	6
C_d	5
Ω_o	2

A	B	C	D	E	F
NL	NL	NL	160	160	100





Advantages:
Lower drift, simple field connections
Disadvantages:
Higher base shear, high foundation forces, height limitations, architectural limitations

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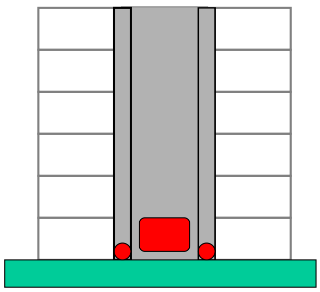
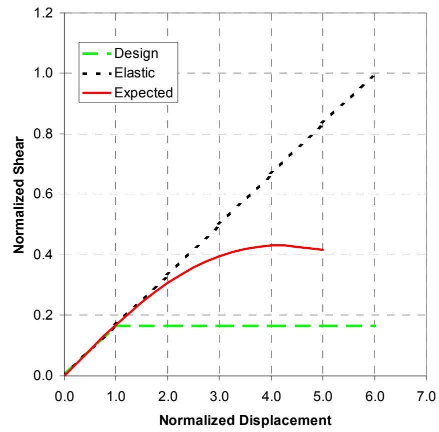
Tabulated values are from ASCE 7-05.

Special Reinforced Concrete Shear Wall

R	6
C_d	5
Ω_o	2.5

A	B	C	D	E	F
NL	NL	NL	160	160	100





Advantages:
Drift control
Disadvantages:
Lower redundancy (for too few walls)

Tabulated values are from ASCE 7-05.

Response Modification Factor *R*

Accounts for:

- Ductility
- Overstrength
- Redundancy
- Damping
- Past behavior

Maximum = 8

Eccentrically braced frame with welded connections
Buckling restrained brace with welded connections
Special moment frame in steel or concrete

Minimum = 1.5 (exclusive of cantilever systems)

Ordinary plain masonry shear walls

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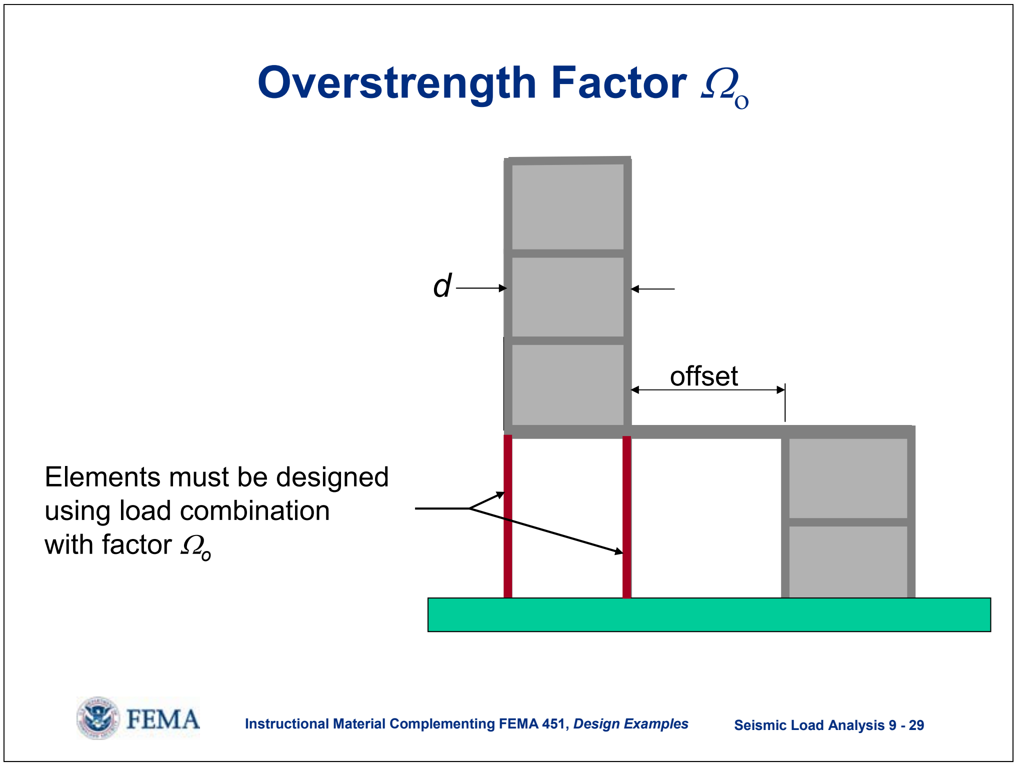
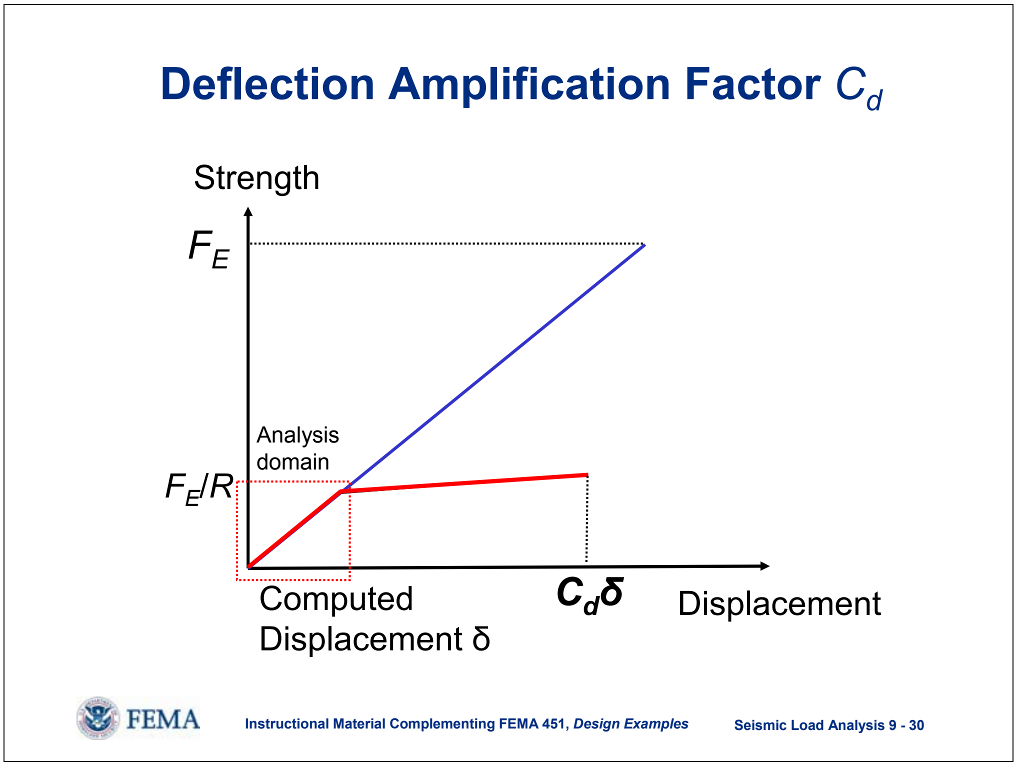
The number of significant digits in R should be considered as an indicator of its accuracy. Note also that politics (competing materials trades) figure into the R factors as well.

Extensive background on the development of the R factor is given in the topic on inelastic behavior of structures.



The overstrength factor is used to protect critical elements and is not necessarily an indicator of the true overstrength of the structure as a whole. The structure shown is only one example of where the overstrength factor comes into play.



The topic on inelastic behavior provides background on the need for and use of the Cd factor. The value of C_d should be considered as more of a true indicator of ductility than is R because of the extra ingredients in R.

Diaphragm Flexibility

Diaphragms must be considered as semi-rigid unless they can be classified as **FLEXIBLE** or **RIGID**.

- Untopped steel decking and untopped wood structural panels are considered **FLEXIBLE** if the vertical seismic force resisting systems are steel or composite braced frames or are shear walls.
- Diaphragms in one- and two-family residential buildings may be considered **FLEXIBLE**.
- Concrete slab or concrete filled metal deck diaphragms are considered **RIGID** if the width to depth ratio of the diaphragm is less than 3 and if no horizontal irregularities exist.

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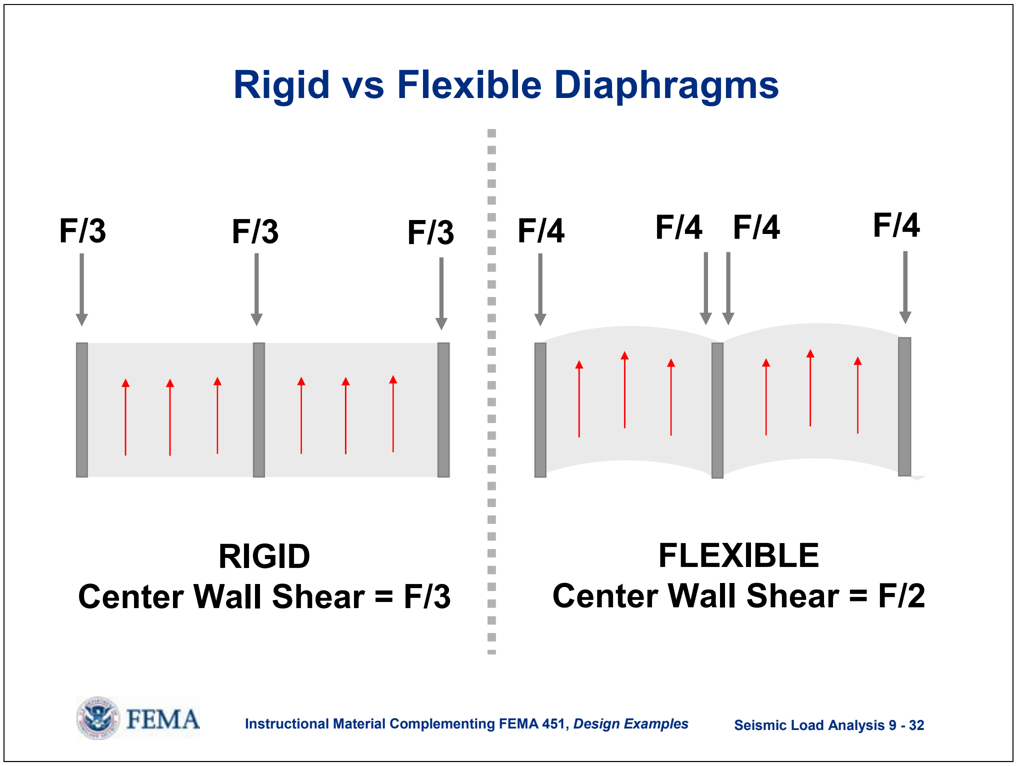
The permission to use a flexible diaphragm model for low rise light frame (residential) is not based upon behavior; it is based on successful past practice.

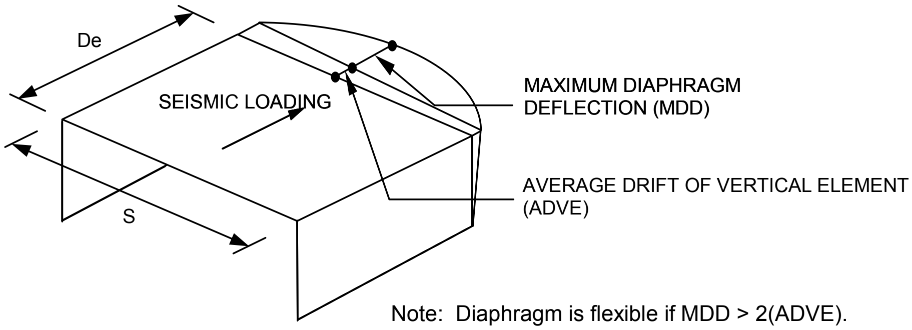


In the system on the left, the walls are assumed to have equal stiffness and the diaphragm is rigid. Hence (in the absence of torsion), each wall attracts 1/3 of the base shear. If the interior wall was twice as stiff as either end wall, the interior wall would attract twice the shear of the end wall.

In the system on the right, the diaphragm is considered fully flexible, and the forces developed in the diaphragm are based on tributary area.

Most diaphragms are semi-rigid, and a finite element analysis would be required to determine an accurate distribution of lateral loads to the walls.

Diaphragm Flexibility



Note: Diaphragm is flexible if MDD > 2(ADVE).

Diagram taken from ASCE 7-05

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The provisions require that an analysis be performed to determine if diaphragms are rigid or flexible. A finite element analysis will generally be required to do this. Loading should be uniformly distributed on the diaphragm, not a concentrated load as shown in this figure from ASCE 7.

When there is some doubt on diaphragm flexibility ant it is desired to avoid explicit modeling of the diaphragm (e.g., using finite elements), a good approach is to run the analysis with both rigid and flexible diaphragms and use the most critical result for the design.

Importance Factors

SUG	Importance Factor
IV	1.50
III	1.25
I, II	1.00

Using ASCE 7-05 Use Groups

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Importance factors are as shown, with higher values for “more critical” structures. Importance factors appear in the denominator of the denominator to indicate that less damage is desired, effectively reducing R.

Others believe that it is more effective to think of the importance factor as a required overstrength which will result in reduced ductility demand for systems designed with any R value.

As shown later, drifts are computed in the absence of I, but the drift limit does depend on use (and, hence, I).

Seismic Design Category = Seismic Use Group + Design Ground Motion

Based on SHORT PERIOD acceleration

Value of S_{DS}	Seismic Use Group*		
	I, II	**III**	**IV**
S_{DS} < 0.167g	A	A	A
0.167g < S_{DS} < 0.33g	B	B	C
0.33g < S_{DS} < 0.50g	C	C	D
0.50g < S_{DS}	D	D	D

*Using ASCE 7-05 Use Groups

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The Seismic Design Category (SDC) is used to determine a variety of aspects of building analysis and design. For example, the required method of analysis, height limitations for various structures, applicability of accidental torsion amplification, and detailing requirements are a function of the SDC.

The value used is the highest (more detailing required) taken from this or the next slide.

Seismic Design Category

Based on LONG PERIOD acceleration

Value of S_{D1}	Seismic Use Group*		
	I, II	**III**	**IV**
$S_{D1} < 0.067g$	**A**	**A**	**A**
$0.067g < S_{D1} < 0.133g$	**B**	**B**	**C**
$0.133g < S_{D1} < 0.20g$	**C**	**C**	**D**
$0.20g < S_{D1}$	**D**	**D**	**D**

Value of S_1	Seismic Use Group*		
	I, II	**III**	**IV**
$S_1 > 0.75g$	**E**	**E**	**F**

*Using ASCE 7-05 Use Groups

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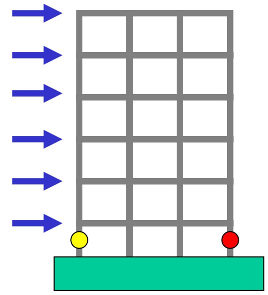
Note that SDC E and F are a function of the mapped spectral acceleration, not the design acceleration (including site response and 1/3 multiplier).

Basic Load Combinations (involving earthquake)

$$1.2D + 1.0E + L + 0.2S$$

$$0.9D + 1.0E$$



Note: 0.5L may be used when L_o < 100 psf (except garages and public assembly)

The load combinations in ASCE 7 and the *Provisions* are the same, but the presentation is quite different. This and the following slides reflect ASCE 7-05 because the treatment there is different from that used in the past by either ASCE 7 or the *Provisions*.

The first combination is intended for use, for example, in "leeward" columns where the added effect of seismic and gravity would be the greatest. Hence, live and snow loads are included here. The second combination is for "windward" columns where the seismic effect is in tension. To produce the largest net tension (or minimum compression), a smaller dead load and no live load is used.

The additive combination usually controls the negative moment strength of beams while the counteracting combination controls the positive moment strength of beams. Both combinations need to be checked in various aspects of shear wall design.

Combination of Load Effects

Use ASCE 7 basic load combinations but substitute the following for the earthquake effect *E*:

$$E = E_h \pm E_v$$

$$E_h = \rho Q_E \qquad E_v = 0.2 S_{DS} D$$

Resulting load combinations (from this and previous slide)

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

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

Note: See ASCE 7 for combinations including hydrostatic load

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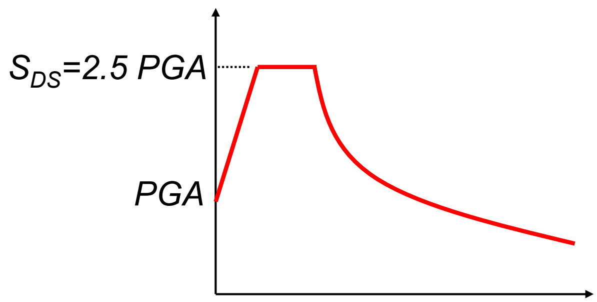
The seismic load effect, E, is split into two parts, a horizontal effect and a vertical effect. The redundancy factor, ρ, is a multiplier on the horizontal effect. The factor 1.2 on gravity loads effectively represents a vertical acceleration of about 0.2 x 2.5 = 0.5 times the peak horizontal ground acceleration. This last point is illustrated in the next slide.

The resulting load combinations are shown exactly as presented in ASCE 7. These combinations were not as explicitly presented in previous versions of the standard.

Vertical Accelerations are Included in the Load Combinations



Vertical acceleration = 0.2(2.5) = 0.5 PGA

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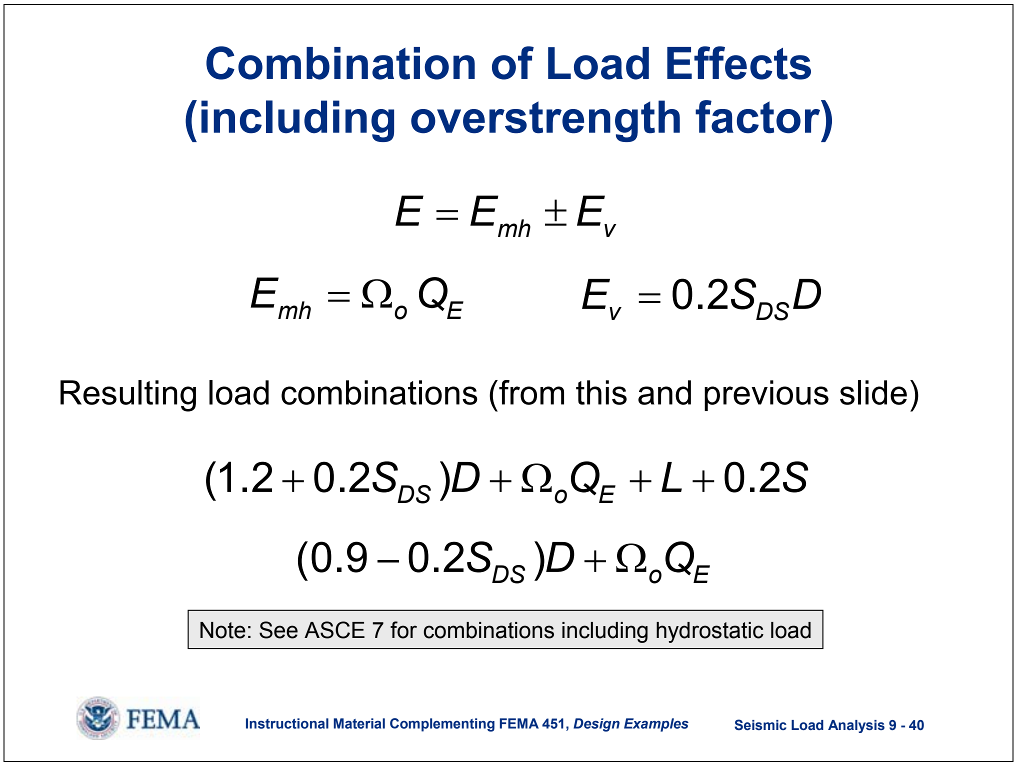
The 0.2 second spectral acceleration is *approximately* 2.5 times the peak ground acceleration. Recall that neither ASCE 7 nor the *Provisions* includes peak horizontal acceleration maps although these are provided by USGS.

The actual peak vertical accelerations recorded in earthquakes can be greater then the horizontal acceleration peaks. Additionally, there may be different spectral amplifications (peak pseudoacceleration divided by peak ground acceleration) for vertically accelerated systems.



The use of the overstrength factor, Ωo, in this combination is intended to protect structural elements where the failure of these elements could initiate a catastrophic building failure. An example of the use of Ωo was given in an earlier slide.

Redundancy Factor ρ

Cases where ρ = 1.0

- Structures assigned to SDC B and C
- Drift and P-delta calculations
- Design of nonstructural components
- When overstrength (Ω_o) is required in design
- Diaphragm loads
- Systems with passive energy devices

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In past versions of the *Provisions*, the redundancy factor was a calculated quantity ranging from 1.0 to 1.5. Now this calculation is eliminated and the value is taken as either 1.0 or 1.3 depending on a semi-quantitative assessment of redundancy.

Redundancy Factor ρ

Cases where ρ = 1.0 for SDC D, E, and F buildings

When each story resisting more than 35% of the base shear in the direction of interest complies with requirements of Table 12.3-3 (next slide)

OR

Structures that are regular in plan at all levels and have at least two bays of perimeter framing on each side of the building in each orthogonal direction for each story that resists more than 35% of the total base shear.

Otherwise ρ = 1.3

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Seismic Load Analysis 9 - 42

Self explanatory.

Redundancy Factor ρ

Requirements for ρ = 1 in SDC D, E, and F buildings

Braced Frames Removal of an individual brace, or connection thereto, would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).

Moment Frames Loss of moment resistance at the beam-to-column connections at both ends of a single beam would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).

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This slide discusses how to determine whether the ρ = 1 value is applicable. Unfortunately it is not clear how these calculations should be made. For a large moment resisting frame, there are hundreds of permutations.

Redundancy Factor ρ

Requirements for ρ = 1 in SDC D, E, and F buildings

Shear Walls

Removal of a shear wall or wall pier with a height-to-length ratio greater than 1.0 within any story, or collector connections thereto, would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).

Cantilever Column

Loss of moment resistance at the base Connections of any single cantilever column would not result in more than a 33% reduction in story strength, nor does the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).

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Continuation of previous slide.

Required Methods of Analysis

The equivalent lateral force method is allowed for all buildings in SDC B and C. It is allowed in all SDC D, E, and F buildings EXCEPT:

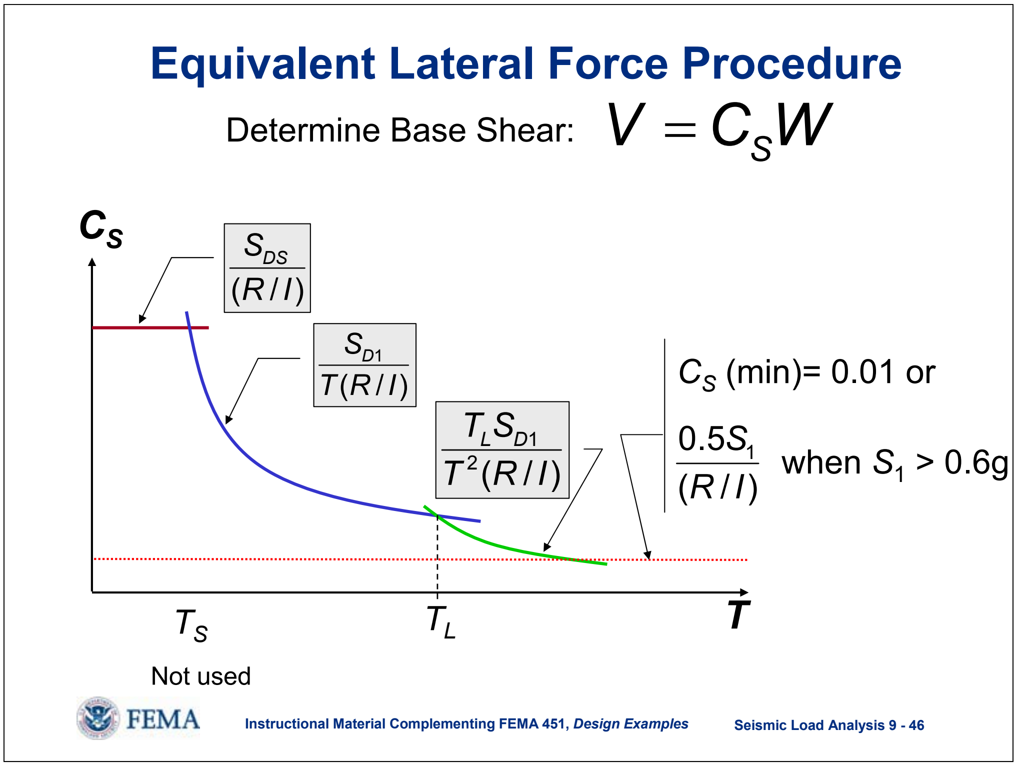
Any structure with $T > 3.5\ T_s$

Structures with $T < 3.5\ T_s$ and with Plan Irregularity 1a or 1b or Vertical Irregularity 1, 2 or 3.

When the ELF procedure is not allowed, analysis must be performed by the response spectrum analysis procedure or by the linear (or nonlinear) response history analysis procedure.

An ELF analysis will likely be used in some form for all structures even though it may not be allowed for final analysis. ELF analysis will almost certainly be used for preliminary design and a percentage of the ELF base shear will be used as a minimum when dynamic analysis is performed.

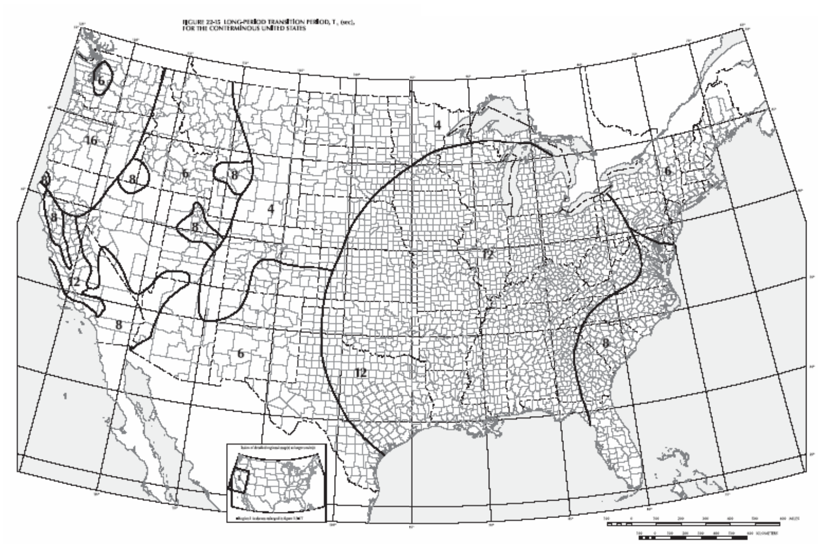
For example, when modal response spectrum analysis is used, the computed base shear must not be less than 85% of the ELF base shear. If the shear from response spectrum analysis is less than 85% of the ELF shears, the forces from the response spectrum analysis would be scaled up. Note, however, that the displacements computed by response spectrum analysis need not be scaled.



Note that for structures 5 stories or less in height with T = 0.5 sec or less, Cs is permitted to be calculated using 1.5 for Ss.

The mapped values for the long period transition are shown in the following slide.

Transition Periods for Conterminous United States



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This map is used to determine the transitional period for the long period region of the response spectrum. Note that the periods are generally very large, indicating that this map would be rarely used in building design.

This map appears as Figures 3.3-16 in the 2003 *Provisions* and as Figure 22-15 in ASCE 7-05.

Effective Seismic Weight *W*

- All structural and nonstructural elements
- 10 psf <u>minimum</u> partition allowance
- 25% of storage live load
- Total weight of operating equipment
- 20% of snow load when “flat roof” snow load exceeds 30psf

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Self explanatory.

Approximate Periods of Vibration

$$\boxed{T_a = C_t h_n^x}$$

C_t = 0.028, x = 0.8 for steel moment frames
c_t = 0.016, x = 0.9 for concrete moment frames
c_t = 0.030, x = 0.75 for eccentrically braced frames
c_t = 0.020, x = 0.75 for all other systems
Note: Buildings ONLY!

$$\boxed{T_a = 0.1N}$$

For moment frames < 12 stories in height, minimum story height of 10 feet. N = number of stories.

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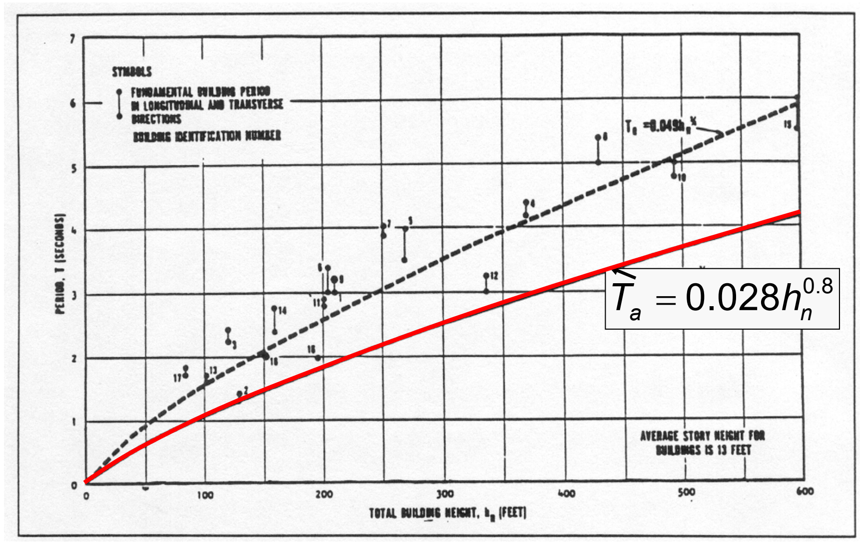
For the ELF method, an approximate period is used. The formula for approximate period is based on the measured low-amplitude response of California buildings that have been subjected to minor earthquakes. Hence, T_a can be considered a lower bound period. Periods of buildings in the central and eastern United States could be significantly longer.

Empirical Data for Determination of Approximate Period for Steel Moment Frames



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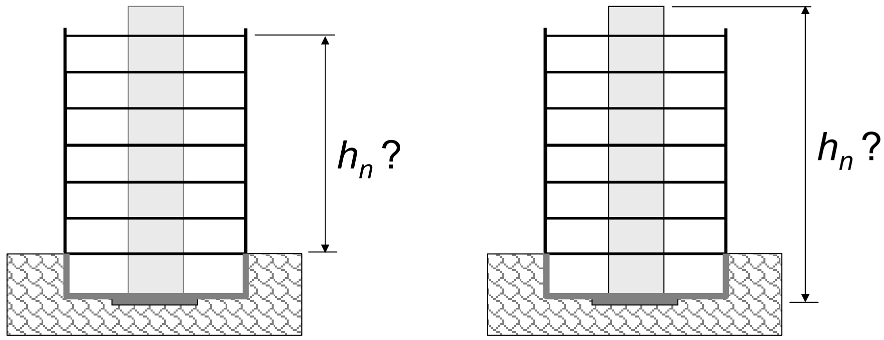
As mentioned in the previous slide, T_a is based on curve-fitting of data obtained from measured response of California buildings after small earthquakes. This figure shows some of the data used to determine the period formula. (Note that this figure is from an old NEHRP *Commentary* and is used only to illustrate the point made.)

What to use as the "height above the base of the building?



When in doubt use the lower (reasonable) value of h_n

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There is some ambiguity in determining building height for use in the period formula. In the case shown, the above-grade height, not including parapet (figure on the left), should be used if the grade-level diaphragm is reasonably rigid in its own plane.

Adjustment Factor on Approximate Period

$$T = T_a C_u \leq T_{computed}$$

S_{D1}	C_u
> 0.40g	1.4
0.30g	1.4
0.20g	1.5
0.15g	1.6
< 0.10g	1.7

Applicable **ONLY** if $T_{computed}$ comes from a "properly substantiated analysis."

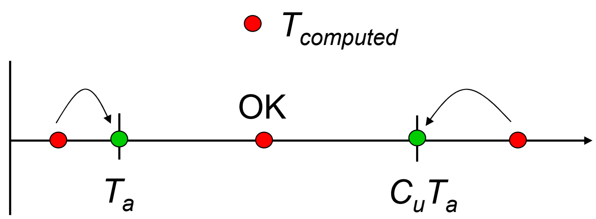
The C_u coefficient eliminates the lower bound conservatism of the empirical period in California and adjusts for the likely lower stiffness in areas of lower seismic hazard. It is very important to note, however, that the coefficient C_u can ONLY be used if a "properly substantiated" analysis (probably using a computer) has also been performed to determine T.

Decisions Regarding Appropriate Period to Use

if $T_{computed}$ is > C_uT_a use C_uT_a

if $T_a < T_{computed} < C_uT_a$ use $T_{computed}$

if $T_{computed} < T_a$ use T_a



If the period from the computer analysis exceeds C_uT_a, then C_uT_a should be used in base shear calculations. If the computer predicts a shorter period than T_a, then T_a can be used (because there is no requirement that an actual period ever be computed as ASCE-7 equation 12.8-2 can be used in all cases). It is recommended that T computed be used if it is less than C_uT_a and greater than T_a.

As noted later, the period from the computer (when greater than C_uT_a) can be used to develop forces used for determination of drift.

It should also be noted that if a computer period (from an analysis program) is significantly different from T_a (say more than 2 times greater or less than half of T_a), the analyst should carefully review the analytical model.

Distribution of Forces along Height

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

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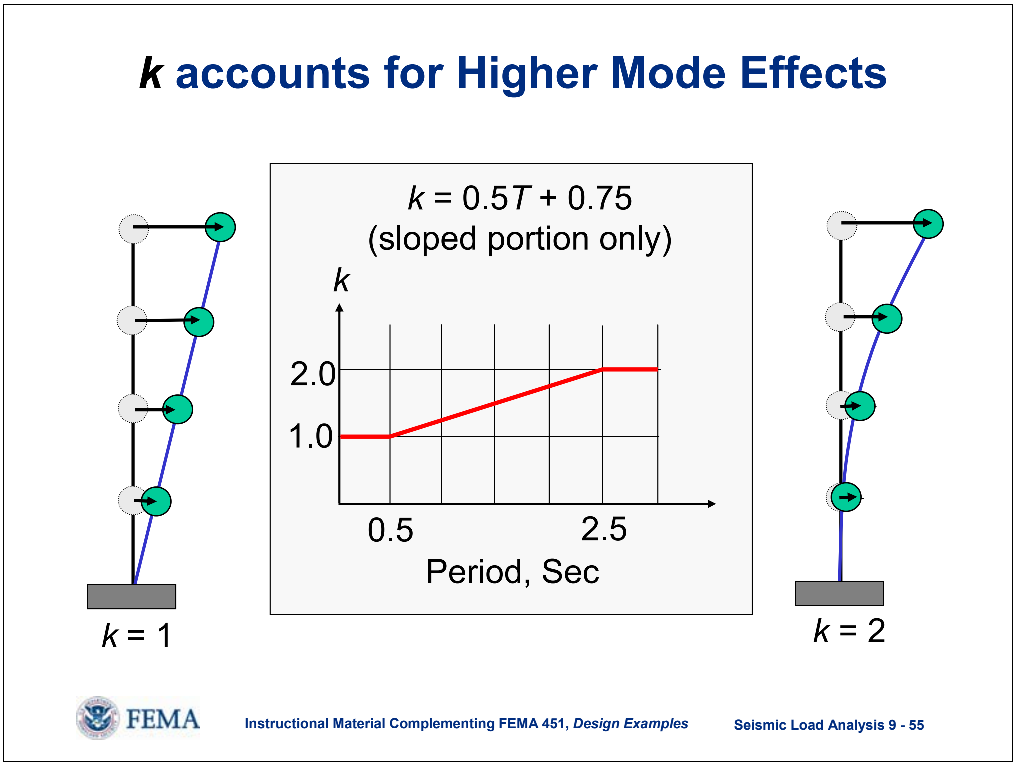
Distribution of the base shear along the height is done through the equation shown. The exponent *k* is used to correct for higher mode effects in longer period buildings.



Self explanatory.

Overturning

The 2003 *NEHRP Recommended Provisions* and ASCE 7-05 allow a 25% reduction at the foundation only.

No overturning reduction is allowed in the above grade portion of the structure.

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As noted in the topic on MDOF dynamics, the ELF method has been "calibrated" to produce story forces that provide story shears that give story shear envelopes similar to those obtained through full dynamic analysis.

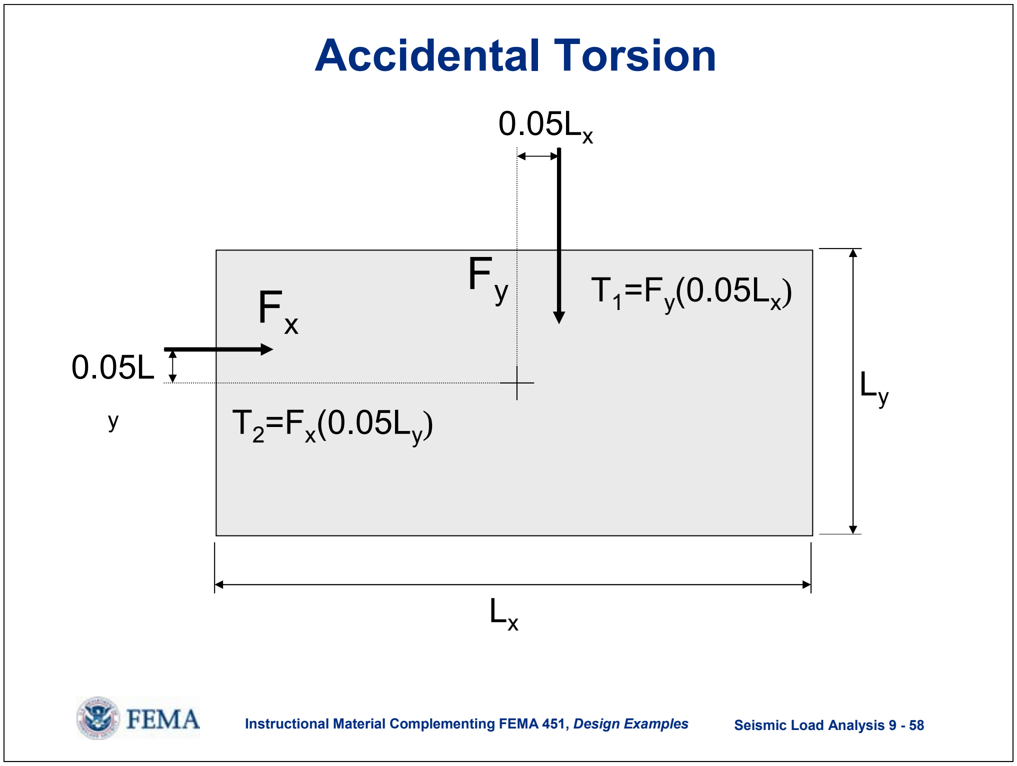
As a result of this, it is possible that the overturning moments that are produced by the ELF forces are overestimated, and hence, a modification factor was allowed. In the past several code cycles this overturning moment reduction was modified consistently. In the 1997 *Provisions*, overturning moment was allowed at the foundation and above grade. In the 2000 and 2003 *NEHRP Recommended Provisions,* it was allowed for the foundation only.

Torsional Effects

ALL	Include inherent and accidental torsion
B	Ignore torsional amplification
C, D, E, F	Include torsional amplification where Type 1a or 1b irregularity exists

For structures with flexible diaphragms, torsional loads need not be considered. For other structures, torsion must be considered as indicated on this slide.

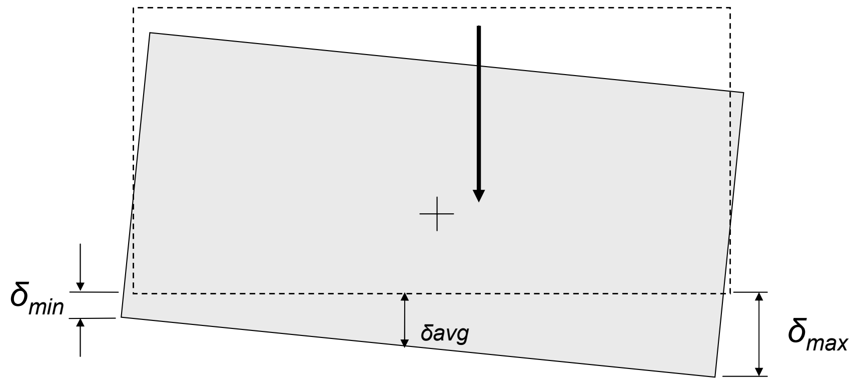
Accidental torsion is always required. Amplification of accidental torsion is required only in the higher SDCs, and then, only when torsional irregularities have already been identified.



Accidental torsion is computed by applying the lateral forces at a 5% eccentricity. The effect of the torsion on the lateral load resisting elements is then added to the direct shear, and the element is designed for the sum of these forces.

Even though the 5% eccentricity seems small, the shears induced due to the accidental torsion can be quite large in some cases.

Amplification of Accidental Torsion

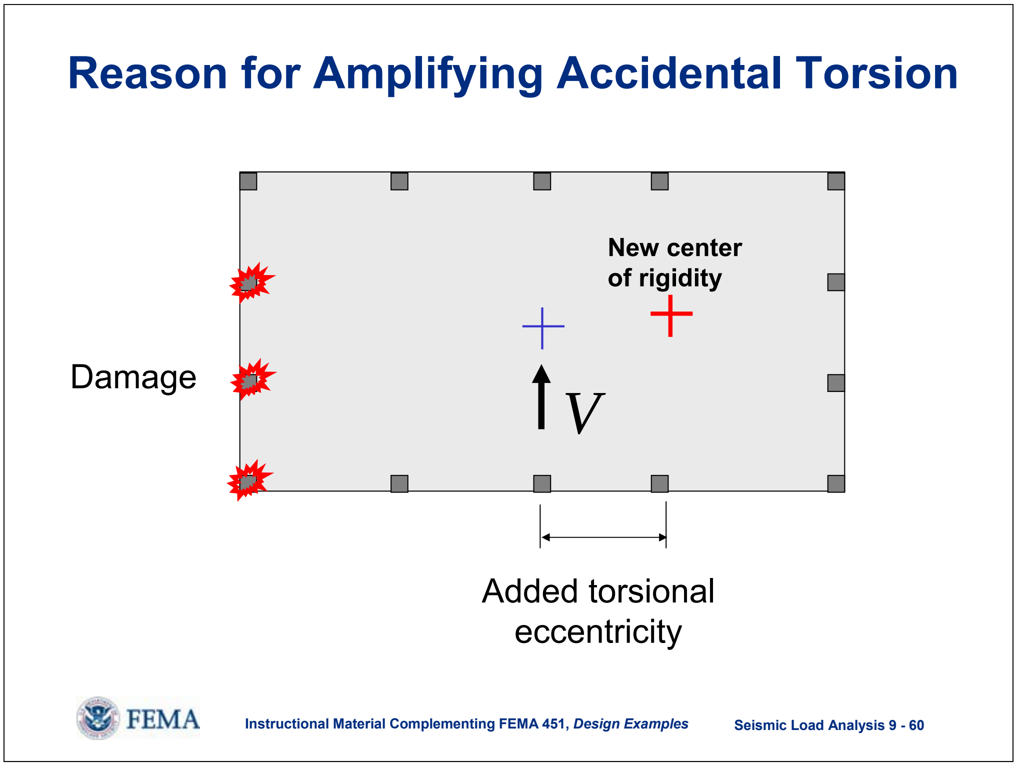


$$A_x = \left(\frac{\delta_{\max}}{1.2\delta_{\text{avg}}} \right)^2$$

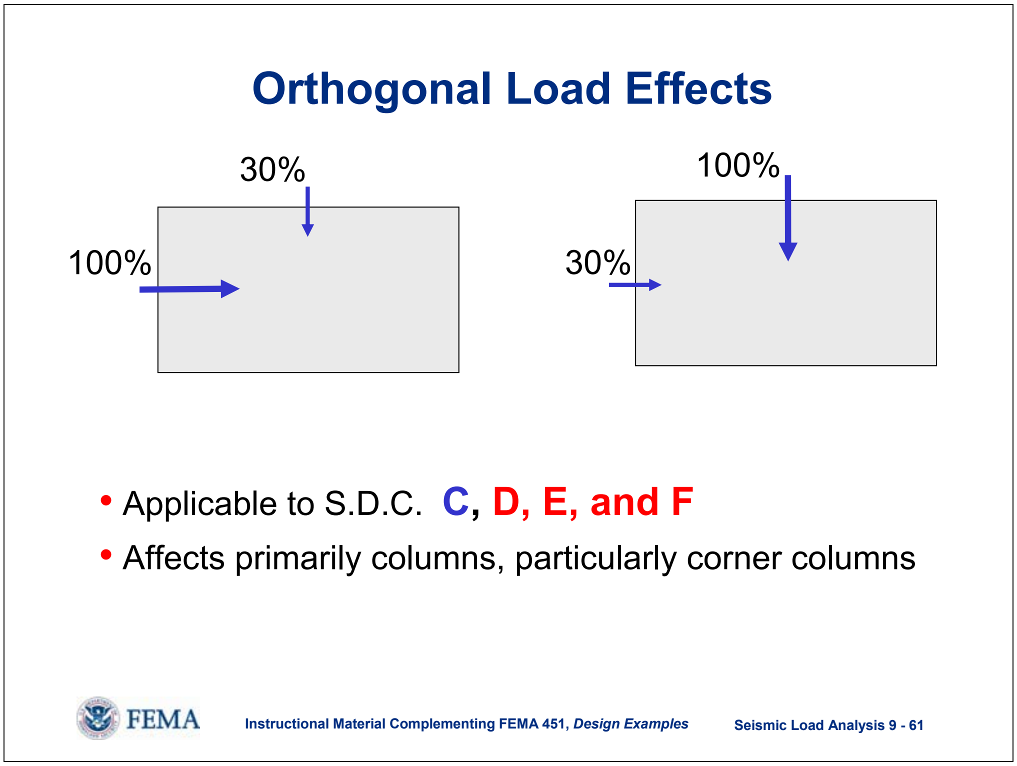
The amplification of accidental torsion is based on the formula shown. Note that the “inherent torsion” need not be amplified as required by earlier editions of the *Provisions*.

In the calculation, the lateral load is applied at a 5% eccentricity.

In fact, it is not clear from the provisions exactly why the torsion is amplified. The commentary to the 2003 *Provisions* indicates that it is to account for dynamic effects, but other references indicate that it is designed to account for eccentricities that occur due to migrating centers of rigidity as the structure yields unsymmetrically.

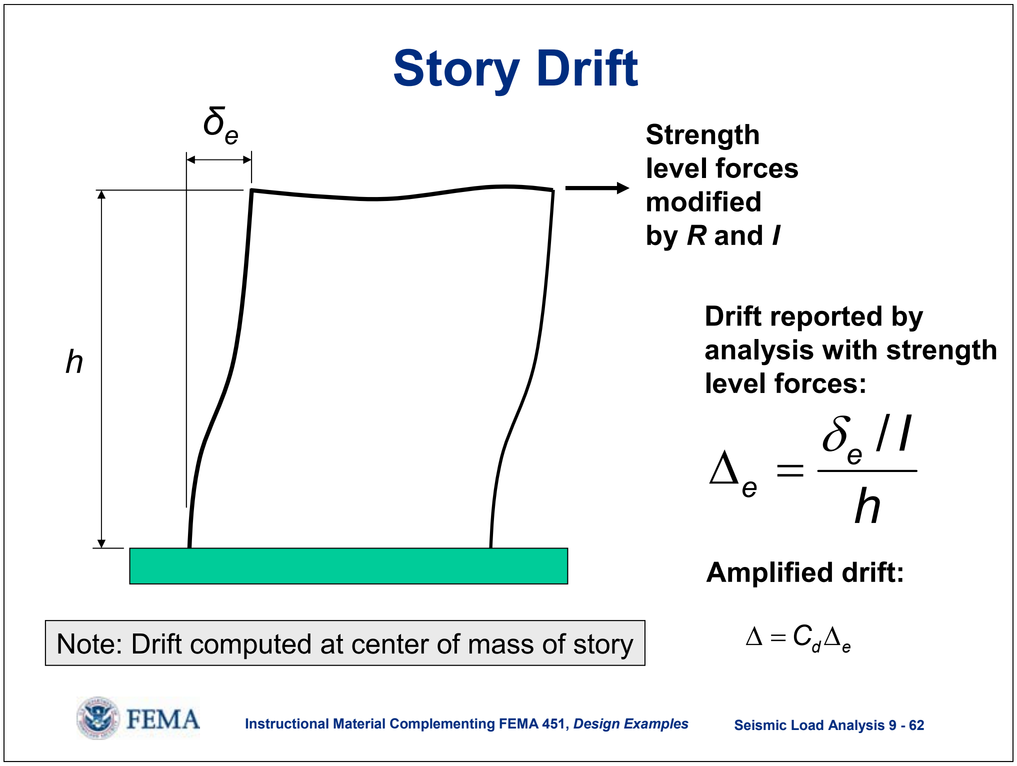


This slide illustrates the point on the previous page. As the columns on one side yield, the center of rigidity migrates to produce increased torsional eccentricities.



Earthquake ground motions may come from any direction and always have components in two orthogonal horizontal directions. The provisions require that the indicated load combinations be used to account for these effects. Orthogonal load effects primarily effect corner columns.

Tall masonry and concrete wall piers sensitive to both in-plane and out-of-plane loading can be affected, as can the reinforcement requirements in cantilever core wall structures.



The slide shows the basic definition of interstory drift for a planar structure. SDCs C through F buildings with Torsional Irregularities 1a and 1b must have drift checked at the edges of the building.

Note that the load used to produce the drift need not include I, ρ, or Ω_o.

Drift Limits

	Occupancy		
	I or II	III	IV
Structures other than masonry 4 stories or less with system Designed to accommodate drift	$0.025h_{sx}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures*	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

* For moment frames in SDC D, E, and F drift shall not exceed tabulated values divided by ρ.

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The note applies to both 2003 *Provisions* and ASCE 7-05. However, no background is provided in the commentary of the *Provisions*.

Story Drift (continued)

For purposes of computing drift, seismic forces may be based on computed building period without upper limit C_uT_a.

For SDC C,D,E, and F buildings with torsional irregularities, drift must be checked at building edges.

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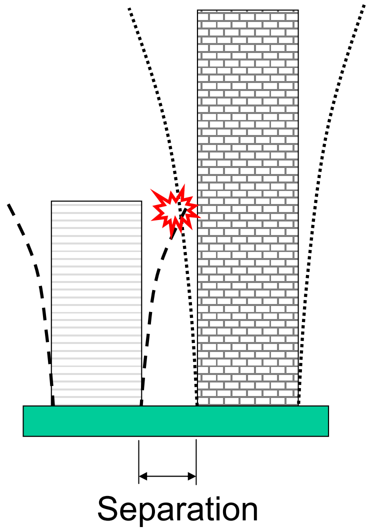
Self explanatory.

Building Separation to Avoid Pounding



Exterior damage to the back (north side) of Oviatt Library during Northridge Earthquake (attributed to pounding).

Source: http://library.csun.edu/mfinley/eqexdam1.html

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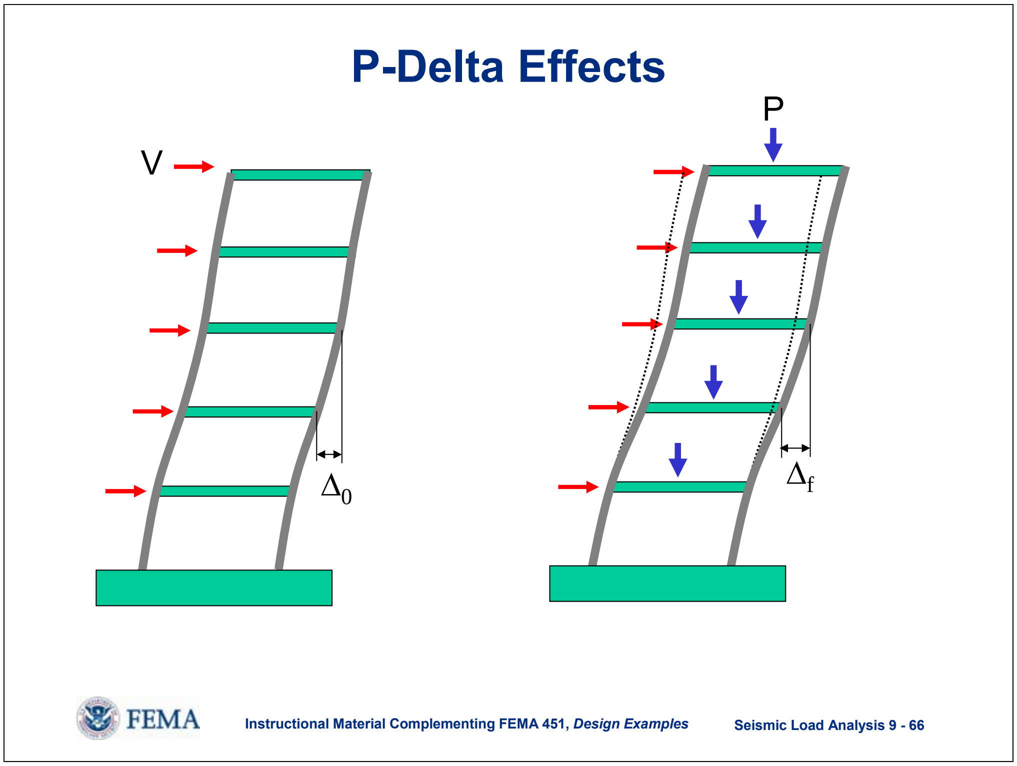
Buildings should be adequately separated to avoid damage or failure due to pounding. If, for example, the 2% drift is attained along the full height of each of two buildings, the building separation must be 4% of the height of the shorter building. This separation would be 4 feet for a 100 foot building.



P-delta effects represent the additional overturning moments produced by the gravity loads acting through the deflections produced by the lateral loads. Analysis that ignores P-delta effects may significantly underestimated the displacements and may also underestimate the apparent lateral force resistance provided by the structure.

There is a significant difference in ASCE 7 and 2003 *Provisions* concerning how P-delta effects are checked. This difference is explained in the following slides.

For elastic systems:

$$\Delta_f = \frac{\Delta_o}{1 - \dfrac{P\Delta_o}{Vh}} = \frac{\Delta_o}{1-\theta}$$

Δ_o = story drift in absence of gravity loads (excluding P-Δ)
Δ_f = story drift including gravity loads (including P-D)
P = total gravity load in story
V = total shear in story
h = story height

Θ is defined as the "story stability ratio"

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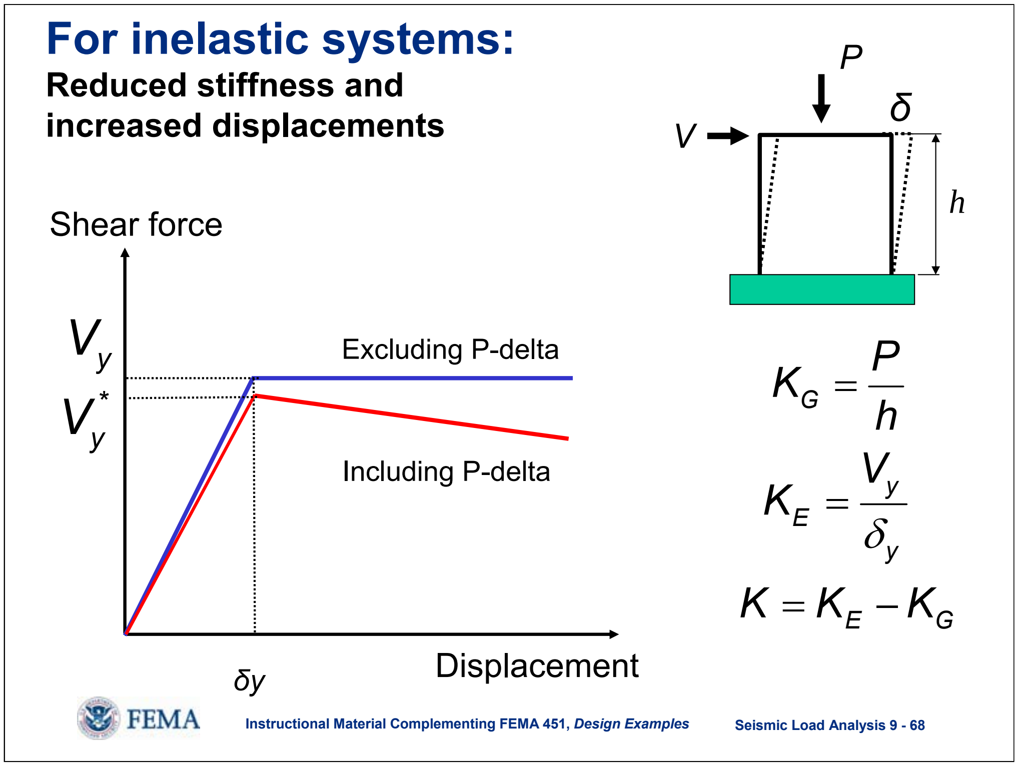
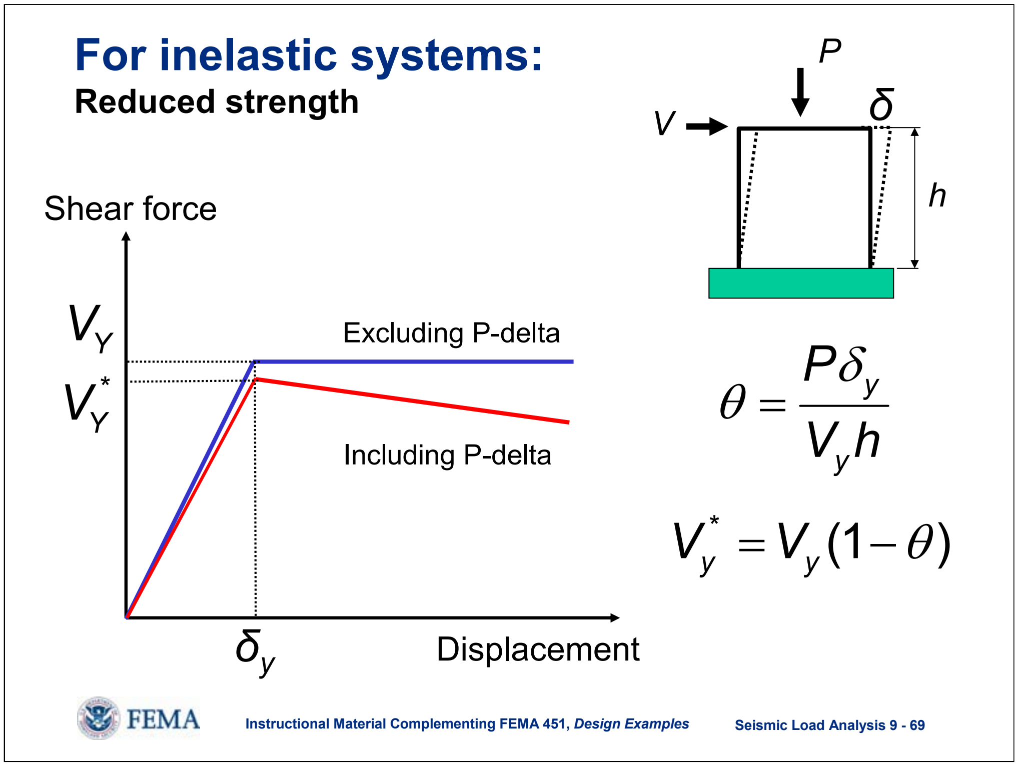
This slide shows the basic calculation for the computation of the final story displacement including P-delta effects, Δ_f, in terms of the displacement without P-delta effects, Δ_o. The stability ratio, θ, is an indicator of how sensitive the structure is to P-delta effects. If $\theta = 1$ (or greater), the structure is unstable.

The formula given in the slide is based on a full elastic analysis.

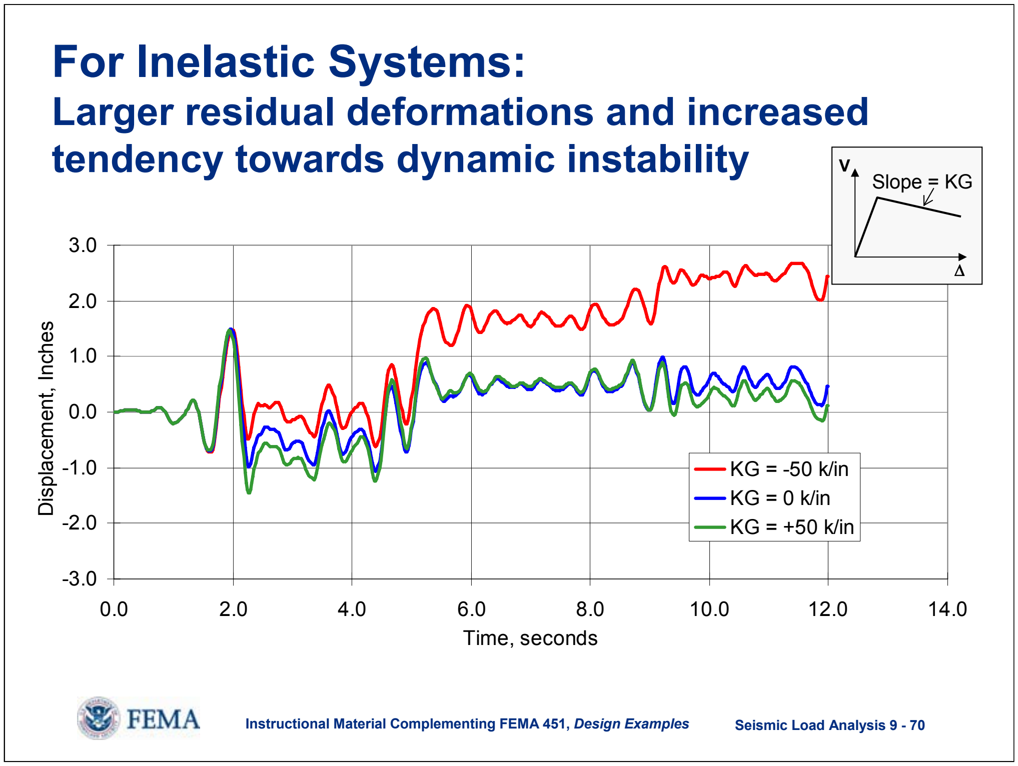


In the simplest sense, P-delta effects lead to a reduction in stiffness and strength of a structure. In this slide a linearized version of the P-delta effect is shown. The term K_G refers to the "linearized geometric stiffness" of the structure. The term LINEAR is used because it is assumed the column has a straight-line deflection for consideration of P-delta effects. Hence, only the rigid-body rotation of the column is considered in the formulation. The actual flexural deformation of the column is not included but, when included, tends to reduce the effective stiffness somewhat more..



P-delta effects also reduce the lateral strength of a structure. In essence, the P-delta effects are imposing a lateral load on the structure, hence, it takes a lower additional lateral load to cause yielding.

If the real action on the structure were a static load V, then the structure would fail at V^*_Y, but because the action is dynamic and displacement controlled, it *may* not fail at that point.



The most profound influence of the P-delta effect on inelastic structures is on the dynamic response. This plot shows the response history of a simple SDOF system with three different assumptions regarding the post-yield stiffness. All three systems have the same yield strength. A slightly decreased KG (to a value of -75 k/in) may have caused a complete dynamic instability of the system.

Note that in this slide the term KG is the total secondary stiffness of a bilinear system, including P-delta effects.

P-Delta Effects

For each story compute:

$$\theta = \frac{P\Delta}{V_x h_{sx} C_d}$$

P_x = total vertical design load at story above level x
Δ = computed story design level drift (including C_d)
V_x = total shear in story
h = story height

If $\Theta < 0.1$, ignore P-delta effects

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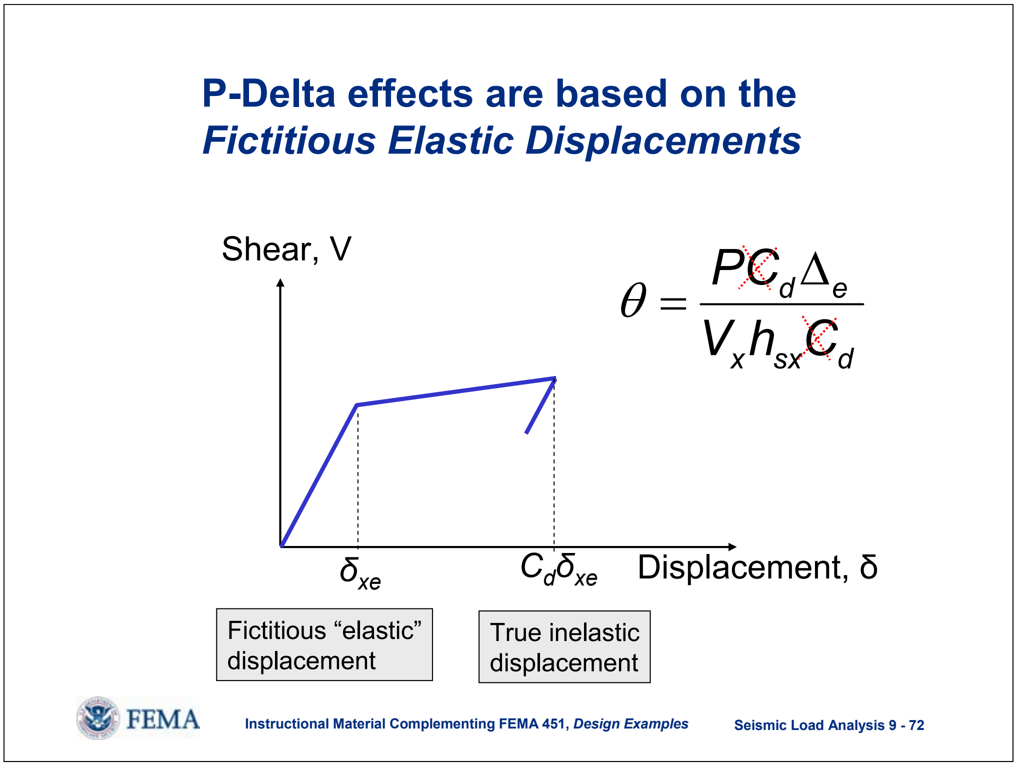
The *Provisions* and ASCE 7 use the equation shown to calculate the story stability ratio for an inelastic system. Note that the term Δ in the numerator INCLUDES the C_d amplifier and, because there is a C_d in the denominator, the C_ds cancel out. As a result, the story stability ratio is based on the entirely fictitious displacement NOT INCLUDING C_d. This is theoretically incorrect.



This slide shows how the C_d terms canel out in the code-based P-delta calculations. Even though this appears to be inconsistent with theory,

the stability equation in ASCE 7 is adequate for determining when stability may be a problem, and for adjusting the strength requirement. However, the procedure provides no protection against the type of behavior given in Slide 70.

P-Delta Effects: ASCE 7-05 approach

If $\theta > 0.1$ then check

$$\theta_{\max} = \frac{0.5}{\beta C_d} < 0.25$$

where β is the ratio of the shear demand to the shear capacity of the story in question (effectively the inverse of the story overstrength). β may conservatively be taken as 1.0 [which gives, for example, $\Theta_{max} = 0.125$ when $C_d = 4$].

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According to both the *Provisions* and ASCE 7, P-delta effects can be ignored if $\theta < 0.1$.

For the *Provisions*, values of θ greater than 0.1 are permitted only if a nonlinear static pushover analysis shows that the post-yield portion of the response is never negative. This requirement is new in 2003.

In ASCE 7-05, the structure is checked as in previous years. If θ exceeds 0.1, then the displacements and the member design forces must be increased by the quantity $1/(1-\theta.)$

In no case is θ allowed to exceed θ_{max} nor 0.25.

P-Delta Effects: ASCE 7-02 approach

If $\theta > 0.1$ and less than $\theta_{\max}$:

Multiply all computed element forces and displacements by:

$$a = \frac{1}{1-\theta}$$

- Check drift limits using amplified drift
- Design for amplified forces

Note: P-delta effects may also be automatically included in the structural analysis. However, limit on θ still applies.

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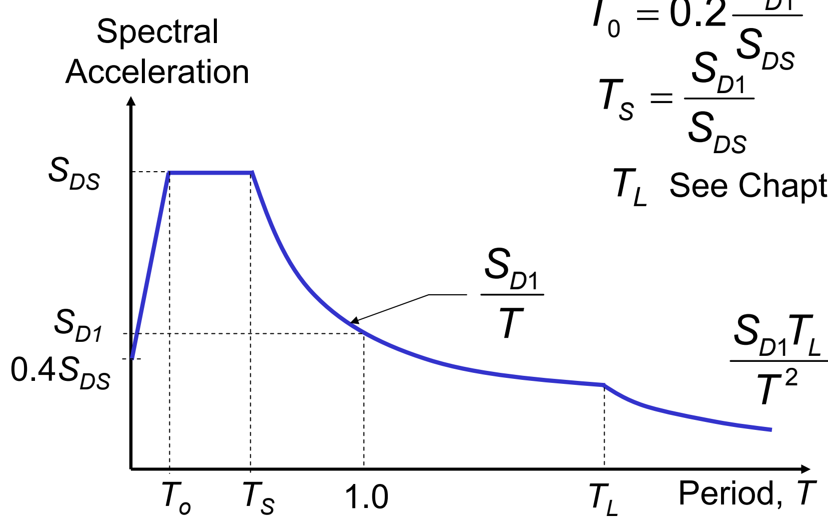
Continuation of previous slide.

Modal Response Spectrum Analysis

$$T_0 = 0.2\frac{S_{D1}}{S_{DS}}$$

$$T_S = \frac{S_{D1}}{S_{DS}}$$

T_L See Chapter 22



Note: Spectrum includes 5% damping

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The modal response spectrum method of analysis usually uses the simplified elastic design response spectrum shown. Note that the spectrum is similar to that used for ELF, except that the R and I factors are omitted (they will be considered later), and that the ELF spectrum does not transition to the PGA at T = 0.

The value of PGA = 0.4SDS is an approximation. Better estimates of PGA may be obtained from the USGS maps (where applicable).

Instead of using the simplified spectrum, a site specific ground motion spectrum may be used. Details for the development of the site specific spectrum may be found in Section 3.4 of the 2003 *Provisions* and in Chapter 21 of ASCE 7-05.

Basic Steps in Modal Response Spectrum (RS) Analysis

1. Compute modal properties for each mode
   - Frequency (period)
   - Shape
   - Modal participation factor
   - Effective modal mass

2. Determine number of modes to use in analysis. Use a sufficient number of modes to capture at least 90% of total mass in each direction

3. Using general spectrum (or compatible ground motion spectrum) compute spectral accelerations for each contributing mode.

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These are the basic steps for modal response spectrum analysis. The procedures in ASCE 7-05 are more succinct than those of those in the 2003 *Provisions*, but the basic idea is the same.

The main difference is that the *Provisions* does not allow the use of the low-period portion of the response spectrum (that goes from S_s to PGA) in the first mode or for any period greater than 0.3 seconds.

Basic Steps in Modal RS Analysis (continued)

4. Multiply spectral accelerations by modal participation factor and by (*I*/*R*)

5. Compute modal displacements for each mode

6. Compute element forces in each mode

7. Statistically combine (SRSS or CQC) modal displacements to determine system displacements

8. Statistically combine (SRSS or CQC) component forces to determine design forces

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The I/R scaling in step 4 is due to the fact that the basic spectrum does not have these factors included.

CQQ is preferred for any three dimensional structure, and is the default for most commercial software. When CQC is used, the analyst must be sure to enter the damping value for each mode, or CQC will reduce to SRSS. SRSS has been shown to be inaccurate for structures with numerous closely spaced modes.

SRSS and CQC will produce exactly the same results for 2D structures regardless of damping.

Basic Steps in Modal RS Analysis (continued)

9. If the design base shear based on modal analysis is less than 85% of the base shear computed using ELF (and $T = T_a C_u$), the member forces resulting from the modal analysis and combination of modes must be scaled such that the base shear equals 0.85 times the ELF base shear.

10. Add accidental torsion as a *static loading* and amplify if necessary.

11. For determining drift, multiply the results of the modal analysis (including the I/R scaling but not the 85% scaling) by C_d/I.

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Step 9 is required to prevent the design of a structure with a “too low” base shear. Note, however, that the displacements need not be scaled accordingly. This is consistent with the rule that allow the use of the computer period (not $C_u T_a$) in checking drift with the ELF method.

Analytical Modeling for Modal Response Spectrum Analysis

- Use three-dimensional analysis
- **For concrete structures, include effect of cracking [req'd]**
- **For steel structures, include panel zone deformations [req'd]**
- Include flexibility of foundation if well enough defined
- Include actual flexibility of diaphragm if well enough defined
- Include P-delta effects in analysis if program has the capability
- Do not try to include accidental torsion by movement of center of mass
- Include orthogonal load effects by running the fill 100% spectrum in each direction, and then SRSSing the results.

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These suggestions are based on the experience of the course developer. Only the two items in bold text are explicitly mentioned by the *Provisions*.

Any analysis which includes P-delta should be run both ways (with and without). If the analysis with P-delta effects produces displacements (inter-story drifts) within 10% of those computed without P-delta effects, P-delta effects may be ignored in subsequent analysis.

The inclusion of accidental torsion by repositioning the center of mass is problematic because the movement of the mass changes the frequencies and mode shapes of the system, and makes it difficult to combine results.

The use of SRSS combinations for the orthogonal load effect comes from Ed Wilson, Professor Emeritus at U.C. Berkeley. Most commercial programs can do this automatically.

Modal Response History Analysis:

uses the natural mode shapes to transform the coupled MDOF equations (with the nodal displacements as the unknowns) into several SDOF equations (with modal amplitudes as the unknowns). Once the modal amplitudes are determined, they are transformed back to nodal displacements, again using the natural mode shapes.

Coupled equations: $\boxed{M\ddot{u} + C\dot{u} + Ku = -MR\ddot{u}_g}$

Transformation: $\boxed{u = \Phi y}$

Uncoupled equations: $\boxed{m_i^* \ddot{y}_i + c_i^* \dot{y}_i + k_i^* y_i = -\phi_i^T MR\ddot{u}_g}$

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There is no place in the *Provisions* or in ASCE 7 that requires response history analysis for traditional buildings (without base isolation or passive energy devices). Hence, there is little advantage in performing a response history analysis, particularly given the difficulties in selecting and scaling appropriate ground motions.

See the topic on MDOF analysis for additional details on how a response history analysis is performed.

Linear Response History Analysis:

Solves the coupled equations of motion directly, without use of natural mode shapes. Coupled equations are numerically integrated using one of several available techniques (e.g., Newmark linear acceleration). Requires explicit formation of system damping matrix *C*.

Coupled equations: $\boxed{M\ddot{u} + C\dot{u} + Ku = -MR\ddot{u}_g}$

There is no reason to perform a fully coupled analysis response history analysis. Modal analysis is just as accurate and is much more computationally efficient.

Advantages of Modal Response History Analysis:

- Each SDOF equation may be solved exactly
- Explicit damping matrix *C* is not required (see below)
- Very good (approximate) solutions may be obtained using only a small subset of the natural modes

$$\ddot{y}_i + 2\xi_i\omega_i\dot{y}_i + \omega_i^2 y_i = -P_i\ddot{u}_g$$

Modal damping ratio

Modal frequency

Modal participation factor

Self explanatory.

Modal and Linear Response History Structural Modeling Procedures

- Follow procedures given in previous slides for modeling structure. When using modal response history analysis, use enough modes to capture 90% of the mass of the structure in each of the two orthogonal directions.
- Include accidental torsion (and amplification, if necessary) as additional static load conditions.
- Perform orthogonal loading by applying the full recorded orthogonal horizontal ground motion simultaneous with the principal direction motion.

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Self explanatory.

ASCE 7-05 Ground Motion Selection

- Ground motions must have magnitude, fault mechanism, and fault distance consistent with the site and must be representative of the *maximum considered ground motion*
- Where the required number of motions are not available simulated motions (or modified motions) may be used

(Parenthesis by F. Charney)

How many records should be used?
Where does one get the records?
How are ground motions scaled?

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One of the most difficult aspects of response history analysis is the selection and scaling of ground motions. This slide asks some of the relevant questions, many of which will be addressed in the next several slides.

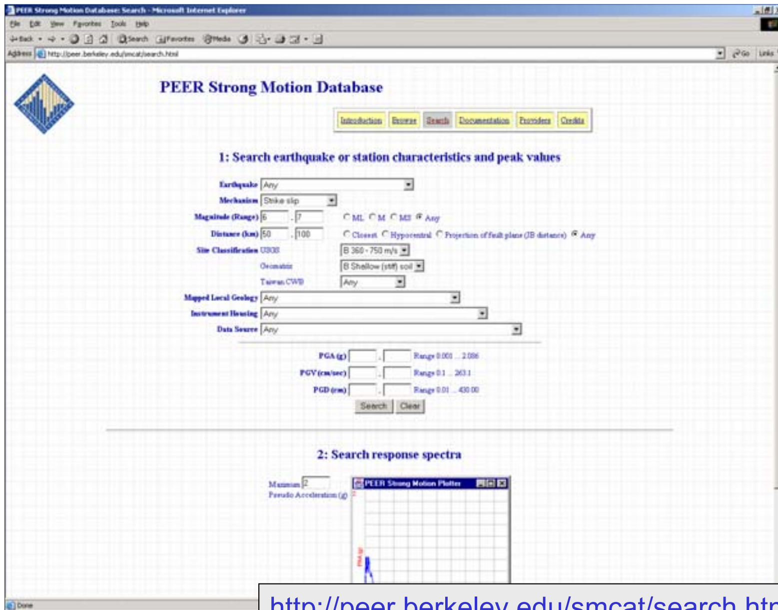
How Many Records to Use?

2003 *NEHRP Recommended Provisions* and ASCE 7-05:

A suite of not less than three motions shall be used.

The *Provisions* provides this guidance for finding the maximum response values for use in complying with performance requirements (such as interstory drift). The use of seven or more records is recommended (by the author of this unit) because it is less conservative. However, it should be pointed out that most commercial programs provide envelope results wherein they may not provide capabilities to average across runs.

Ground Motion Sources: PEER



http://peer.berkeley.edu/smcat/search.html

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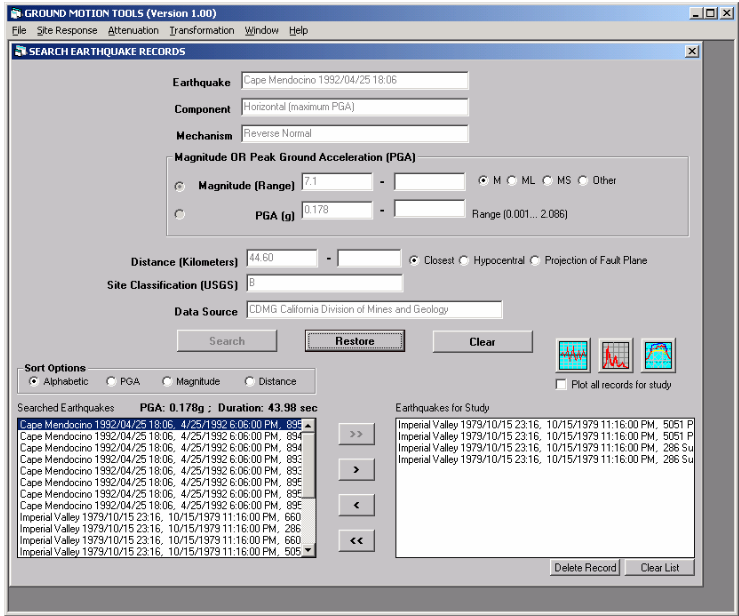
The PEER web site is one source of ground motions. A search engine is available for entering a variety of parameters. The program will provide a list of those ground motions that match the search parameters. The user may view the accelerogram or the response spectrum and may then download the record for use in analysis.

Ground Motion Sources: EQTools



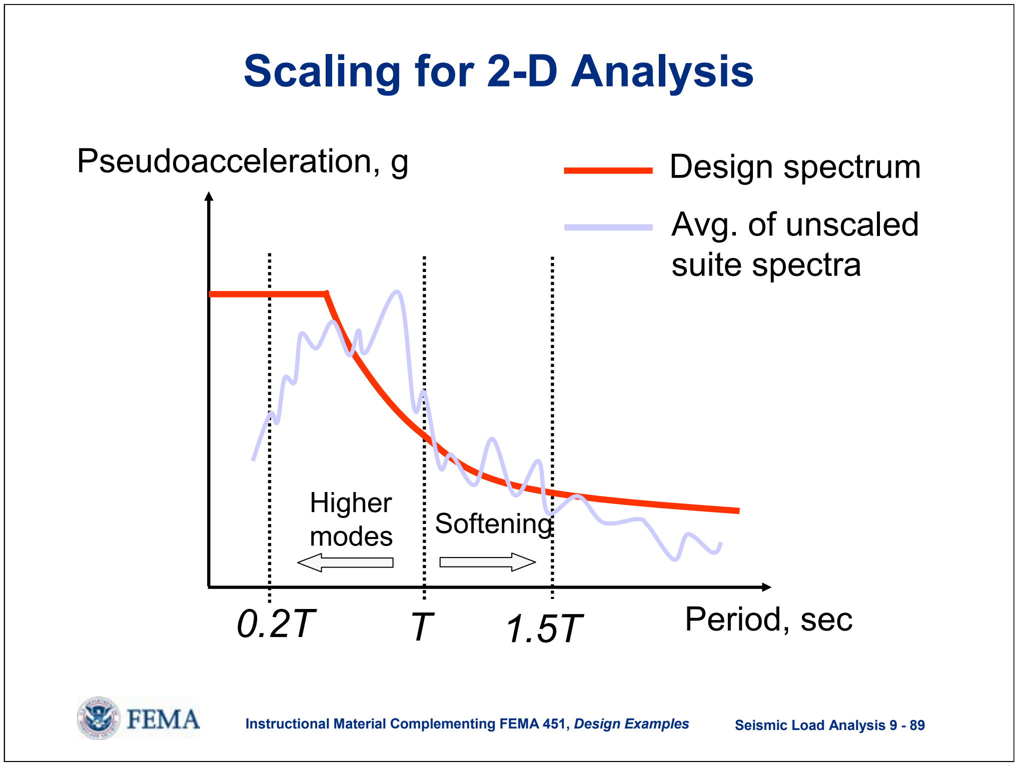
A similar search engine has been developed by F. Charney and S. Riaz at Virginia Tech. This is a standalone searchable data base that contains more the 2000 records as well as a multitude of tools for evaluating the records. This program will be demonstrated during the course.

Ground Motion Scaling

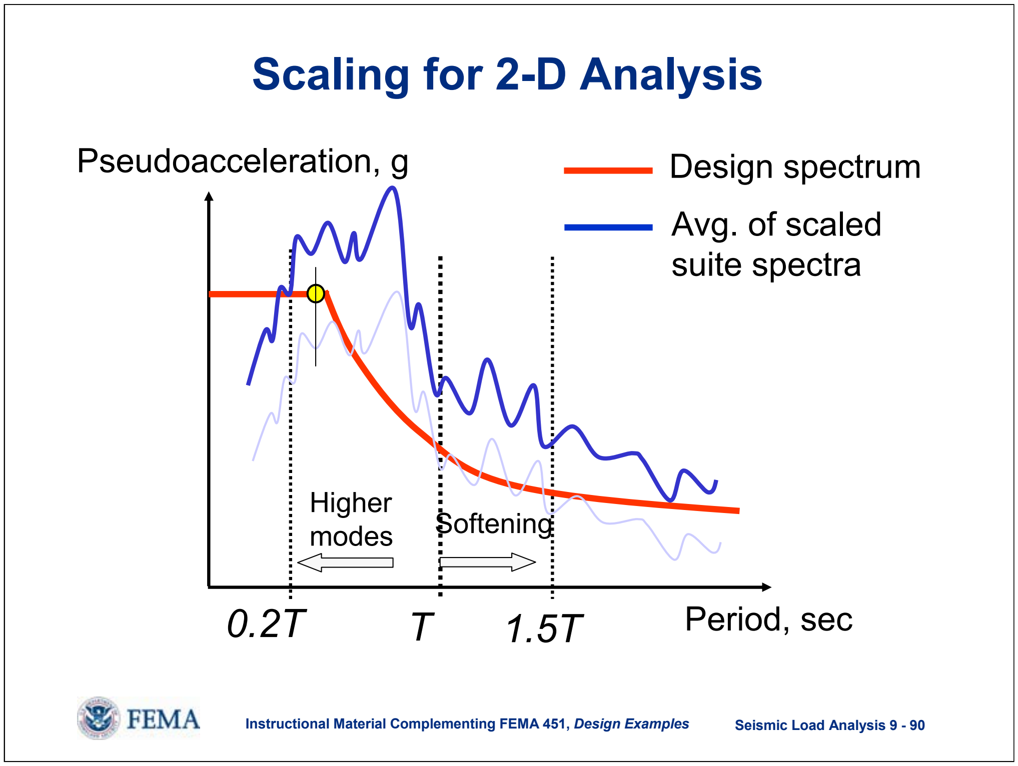
Ground motions must be scaled such that the average value of the 5% damped response spectra of the suite of motions is not less than the design response spectrum in the period range 0.2*T* to 1.5*T*, where *T* is the fundamental period of the structure.

The 2000 *Provisions* (as well as ASCE7-02 and 2003 IBC) provides scaling rules that are a bit difficult to interpret and that may produce some curious results. Different rules are provided for 2D and 3D analysis. The same rules are applied to linear and nonlinear response history analysis.

The idea behind the scaling rules is to capture the effect of period elongation associated with yielding (hence the 1.5*T* requirement), as well as to capture higher mode effects (the 0.2*T* requirement).



This diagram illustrates the average spectra and the code spectra before scaling. In effect, the unscaled spectra will need to be "lifted up" until all its ordinates are greater than or equal to the corresponding ordinates of the code spectra in the period range 0.2 to 1.5*T*.



The scaled spectra is shown in dark blue. Here, the controlling point is at a period of about 0.4*T*, which will definitely be a higher mode response. It seems clear that this scaling approach is extremely conservative in this case -- the spectrum at the principle period of interest (*T*) is about 40% greater than the code spectrum at the same point.

Ground Motion Selection and Scaling

1. The square root of the sum of the squares of the 5% damped spectra of each motion pair (N-S and E-W components) is constructed.

2. Each pair of motions should be scaled such that the average of the SRSS spectra of all component pairs is not less than 1.3 times the the 5% damped design spectrum in the period range 0.2 to 1.5 T.

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Similar rules are applied for cases where a 3D analysis is used. The 1.3 factor compensates for the SRSSing of the orthogonal motion pairs.

Potential Problems with Scaling

- A degree of freedom exists in selection of individual motion scale factors, thus different analysts may scale the same suite differently.
- The scaling approach seems overly weighted towards higher modes.
- The scaling approach seems to be excessively conservative when compared to other recommendations (e.g., Shome and Cornell)

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This slide is self explanatory. The Shome and Cornell method is discussed subsequently.

Recommendations:

- Use a minimum of seven ground motions
- If near-field effects are possible for the site a separate set of analyses should be performed using only near field motions
- Try to use motions that are magnitude compatible with the design earthquake
- Scale the earthquakes such that they match the target spectrum at the structure's initial (undamaged) natural frequency and at a damping of at least 5% critical.

These are the recommendations of the principal author of this topic (Charney). Note: Charney is *not* a seismologist.

Response Parameters for Linear Response History Analysis

For each (scaled) ground motion analyzed, all computed response parameters must be multiplied by the appropriate ratio (*I*/*R*). Based on these results, the maximum base shear is computed.

The ratio of the maximum base shear to total weight for the structure must not be less than the following:

$$V/W = 0.01 \qquad \text{for SDC A through D}$$

$$V/W = \frac{0.5S_1}{R/I} \qquad \text{for SDC E and F when } S_1 > 0.\text{g}$$

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For the response history procedure, there is no equivalent minimum shear of 0.85 times the ELF base shear (as in the modal response spectrum approach) because the ground motion scaling already adjusts for this.

ASCE 7-02 Response Parameters for Linear Response History Analysis (continued)

If at least seven ground motions are used, response quantities for component design and story drift may be based on the *average* quantity computed for all ground motions.

If less than seven ground motions are used, response quantities for component design and story drift must be based on the *maximum* quantity computed among all ground motions.

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Self explanatory.

Nonlinear Response History Analysis is an Advanced Topic and in not covered herein.

Due to effort required, it will typically not be used except for very critical structures, or for structures which incorporate seismic isolation or passive, semi-active, or active control devices.

The principal difficulty with nonlinear response history analysis (aside from the effort required) are the sensitivities of the computed response due to a host of uncertainties.
Such sensitivities are exposed by a systematic analysis approach called incremental dynamic analysis.

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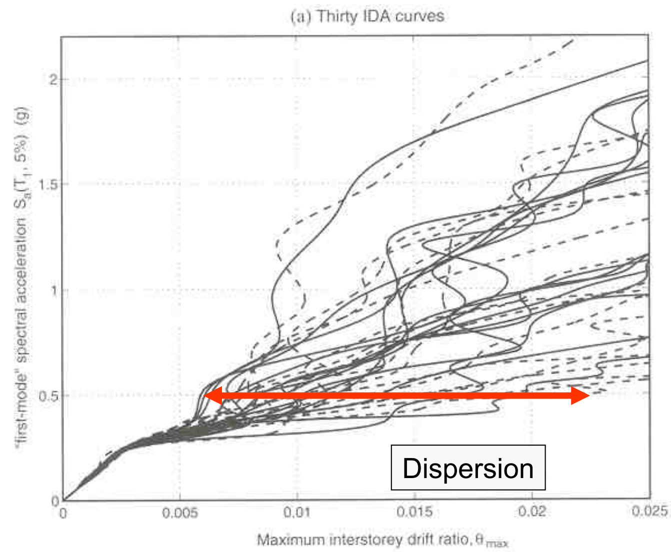
As with the linear response history approach, there is no requirement that nonlinear dynamic analysis ever be used except for systems with base isolation or passive energy. The exception is the use of systems without a predefined *R* value. Here, nonlinear analysis may be used to support the use of a nonspecified system.

A Family of IDA Curves of the Same Building Subjected to 30 Earthquakes [exposing effect of ground motion uncertainty]



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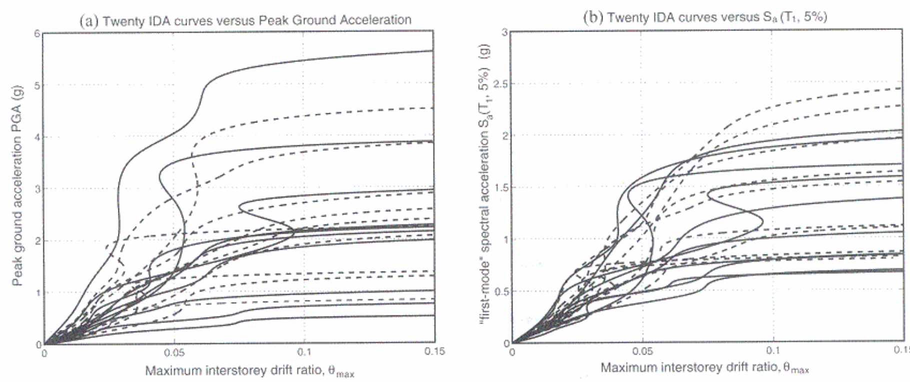
This plot, from Cornell and Vamvatisikos, shows IDA curves for a single structure subjected to 30 different ground motions. All of the systems appear to have the same general behavior up to an intensity of about 0.25g, after which all bets are off. At intensities of 0.5g, the variations in response are huge. The variation in response at a given intensity is referred to as dispersion. The huge dispersion here is characteristic of IDA analysis. If the actual ground motion intensity was 0.5g, how would one determine if the design is appropriate?

It is also useful to note that the dispersion in apparent collapse capacity (vertical on the chart), as opposed to the dispersion in drift at any one level of acceleration, is also very significant. Both dispersions are a measure of how one cannot capture all the meaningful information in an earthquake ground motion with any one parameter.

IDA Curves of the Same Building Subjected to Suite of Earthquakes Where Different Scaling Methods Have Been Used

NORMALIZED to PGA NORMALIZED to S_a



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Here, sets of IDA were performed using two different scaling techniques. On the left, scaling was normalized to PGA. On the right, scaling was based on 5% damped spectral pseudoacceleration at the structure's first mode natural frequency. The variation in dispersion is quite remarkable, with far less dispersion being evident on the right. Other items, such as damage measure, have an effect on dispersion. It would seem that limiting dispersion would be desirable.

Methods of Analysis Described in ASCE 7-05

Nonlinear static pushover analysis

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There are no provisions in ASCE 7 for performing a nonlinear static pushover analysis. Appendix A5 if the 2003 *Provisions* does address this issue.