2009 NEHRP Recommended Seismic Provisions:

Training and Instructional Materials

FEMA P-752 CD / June 2013

FEMA nehrp

4

Structural Analysis

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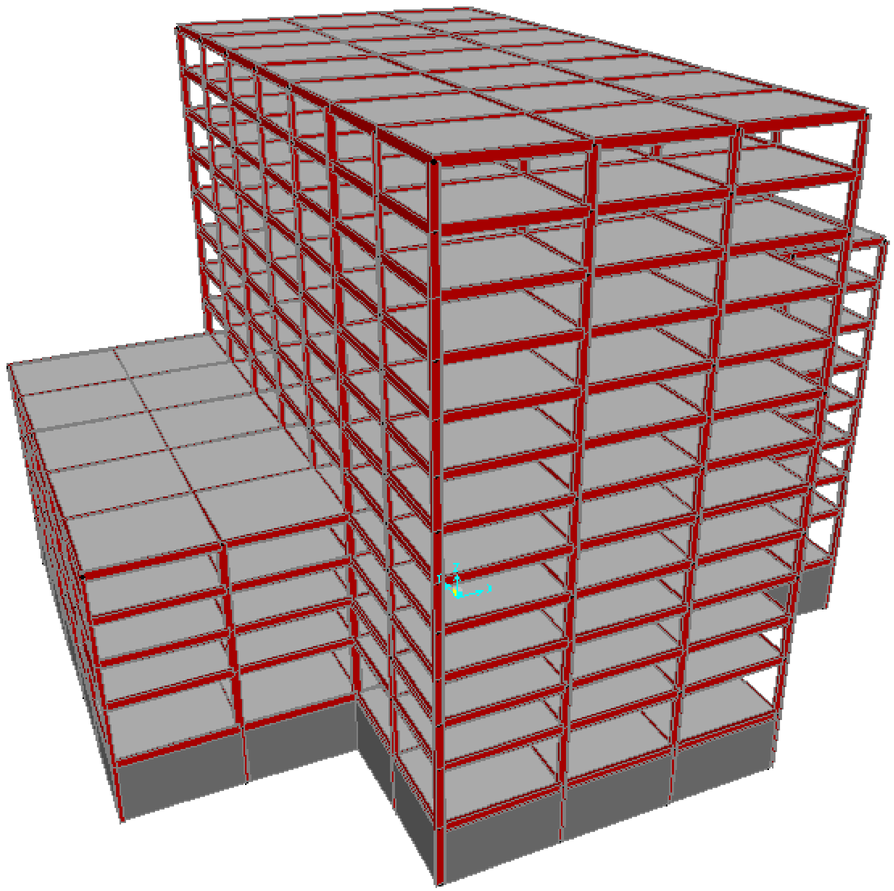
Structural Analysis: Example 1
Twelve-story Moment Resisting Steel Frame

Analysis of a 12-Story Steel Building In Stockton, California



Building Description

- 12 Stories above grade, one level below grade
- Significant Configuration Irregularities
- Special Steel Moment Resisting Perimeter Frame
- Intended Use is Office Building
- Situated on Site Class C Soils

Analysis Description

- Equivalent Lateral Force Analysis (Section 12.8)
- Modal Response Spectrum Analysis (Section 12.9)
- Linear and Nonlinear Response History Analysis (Chapter 16)

Overview of Presentation

- Describe Building
- Describe/Perform steps common to all analysis types
- Overview of Equivalent Lateral Force analysis
- Overview of Modal Response Spectrum Analysis
- Overview of Modal Response History Analysis
- Comparison of Results
- Summary and Conclusions

Note: The majority of presentation is based on requirements provided by ASCE 7-05. ASCE 7-10 and the 2009 NEHRP Provisions (FEMA P-750) will be referred to as applicable.

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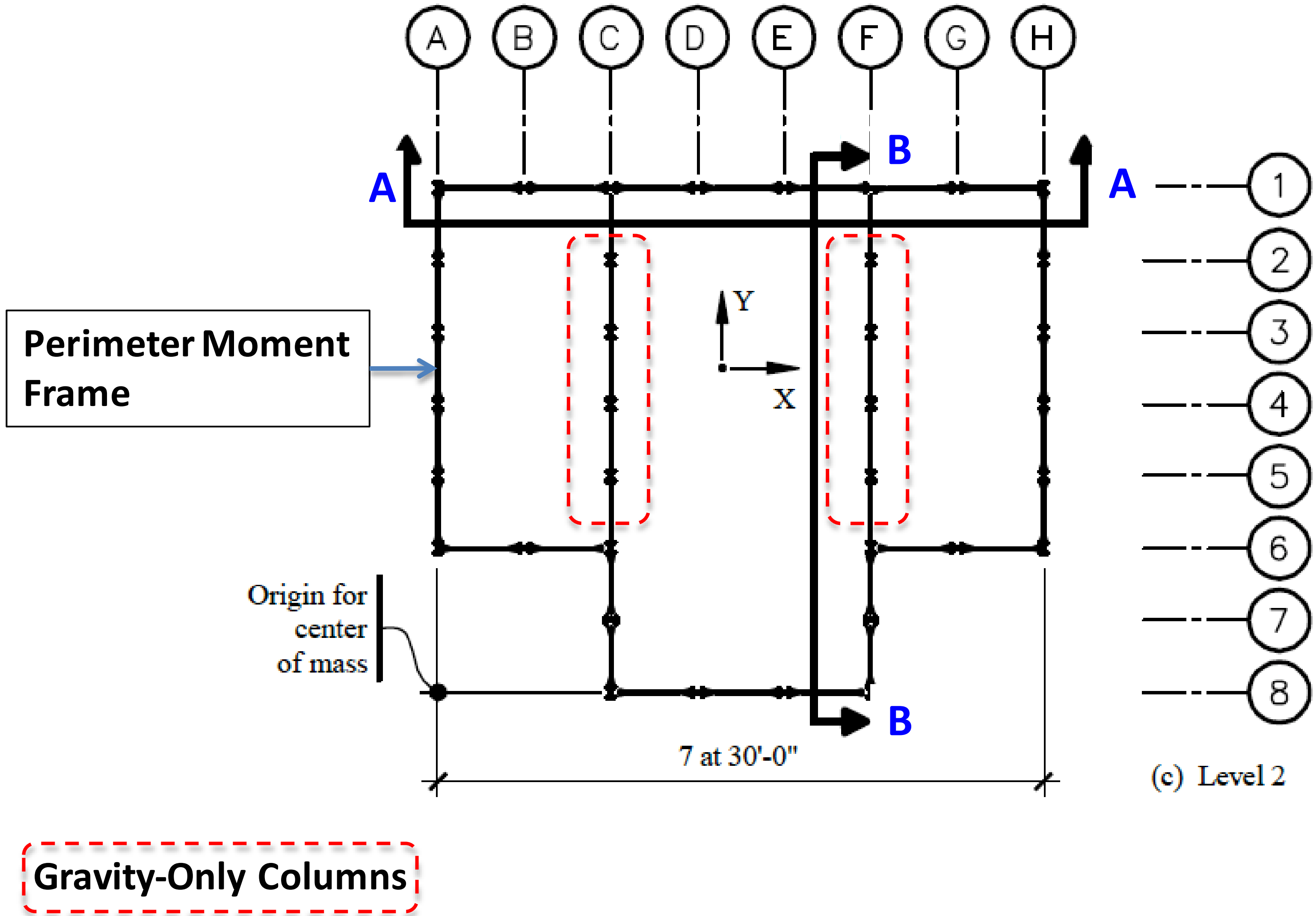
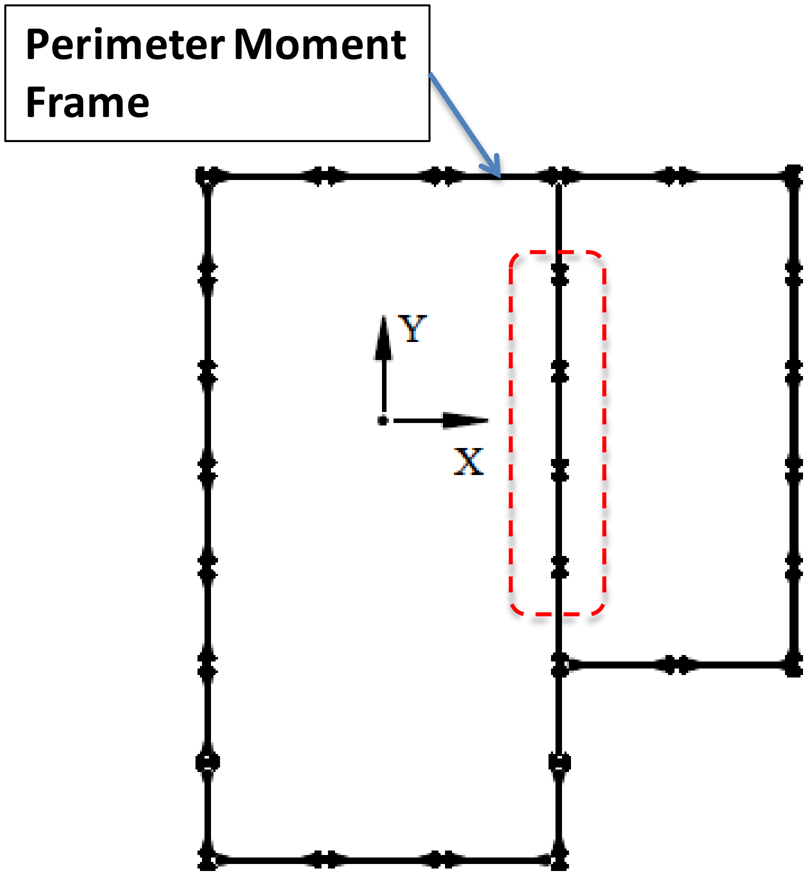
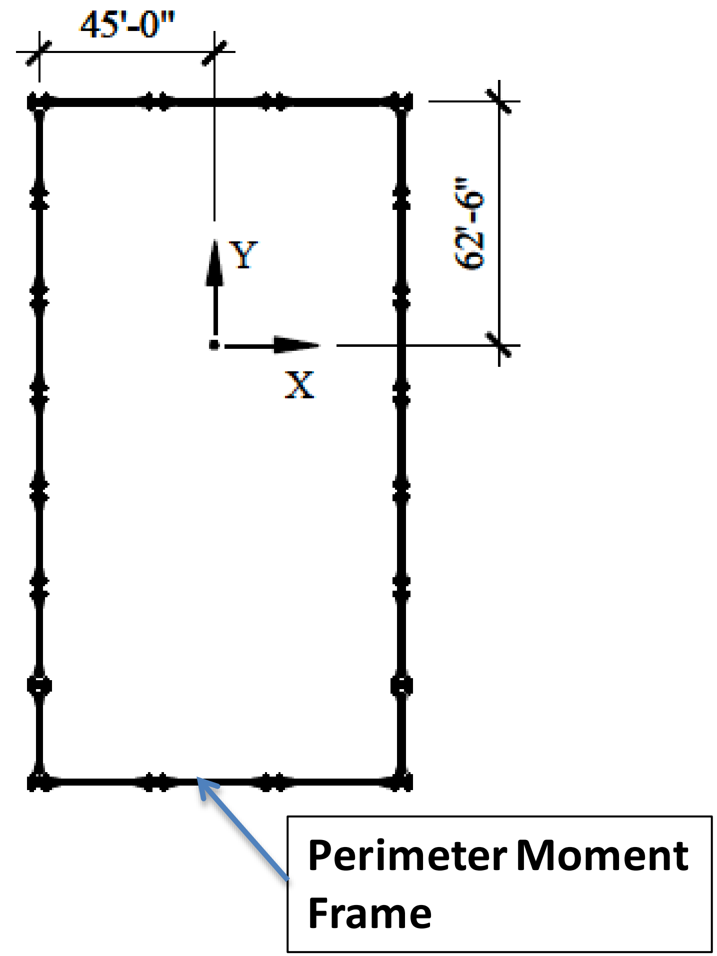
Plan at First Level Above Grade



Plans Through Upper Levels



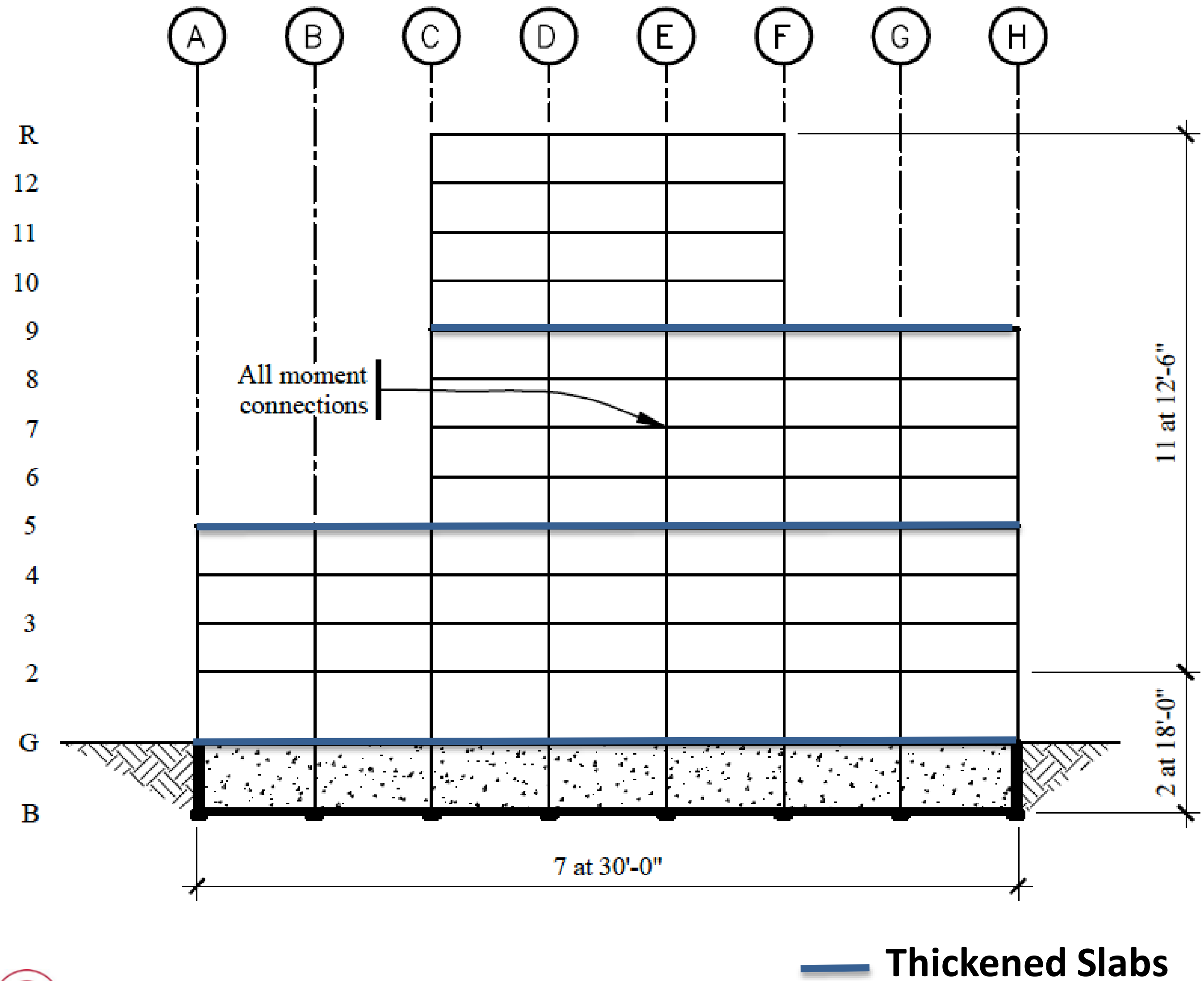
Above Level 5



Above Level 9

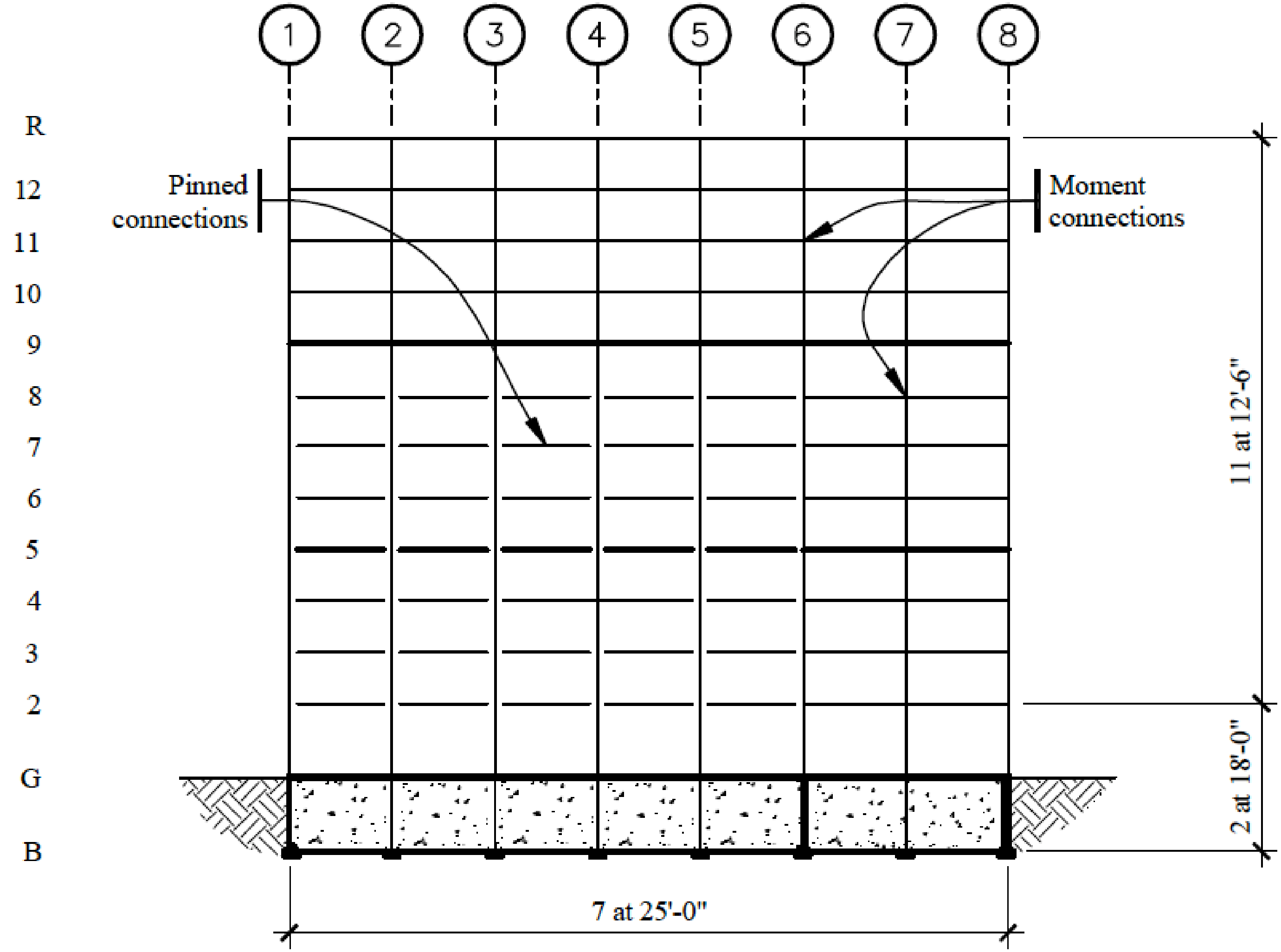
Gravity-Only Columns

Section A-A



A
B
C
D
E
F
G
H
R
12
11
10
9
8
7
6
5
4
3
2
G
B
All moment connections
11 at 12'-6"
2 at 18'-0"
7 at 30'-0"
Thickened Slabs

Section B-B


1
2
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7
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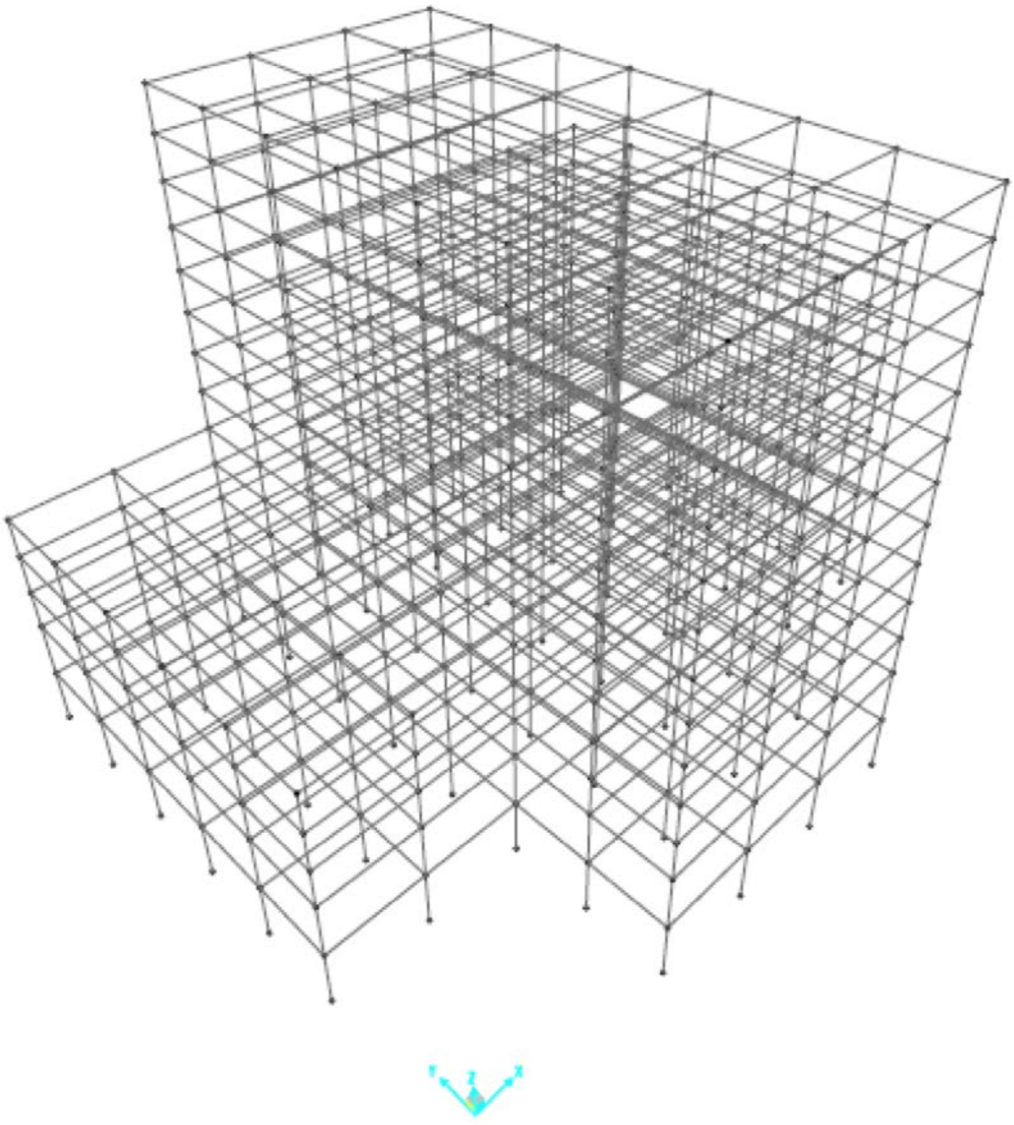
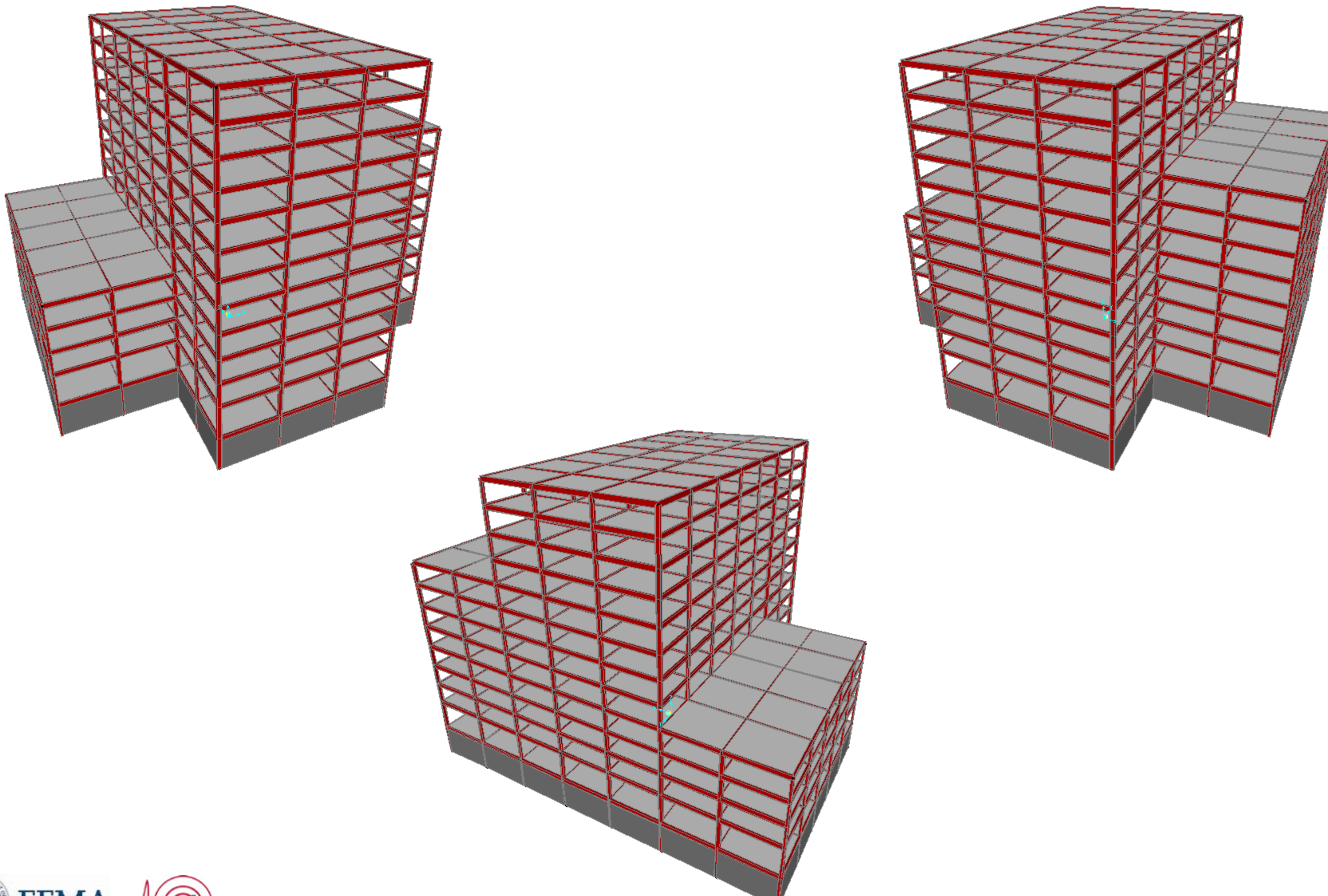
R
12
11
10
9
8
7
6
5
4
3
2
G
B
Pinned connections
Moment connections
11 at 12'-6"
2 at 18'-0"
7 at 25'-0"

3-D Wire Frame View from SAP 2000



Perspective Views of Structure (SAP 2000)



Overview of Presentation

- Describe Building
- **Describe/Perform steps common to all analysis types**
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Seismic Load Analysis: Basic Steps

1. Determine Occupancy Category (Table 1-1)
2. Determine Ground Motion Parameters:
- S_S and S_1 USGS Utility or Maps from Ch. 22)
- F_a and F_v (Tables 11.4-1 and 11.4-2)
- S_{DS} and S_{D1} (Eqns. 11.4-3 and 11.4-4)
3. Determine Importance Factor (Table 11.5-1)
4. Determine Seismic Design Category (Section 11.6)
5. Select Structural System (Table 12.2-1)
6. Establish Diaphragm Behavior (Section 11. 3.1)
7. Evaluate Configuration Irregularities (Section 12.3.2)
8. Determine Method of Analysis (Table 12.6-1)
9. Determine Scope of Analysis [2D, 3D] (Section 12.7.2)
10. Establish Modeling Parameters

Determine Occupancy Category

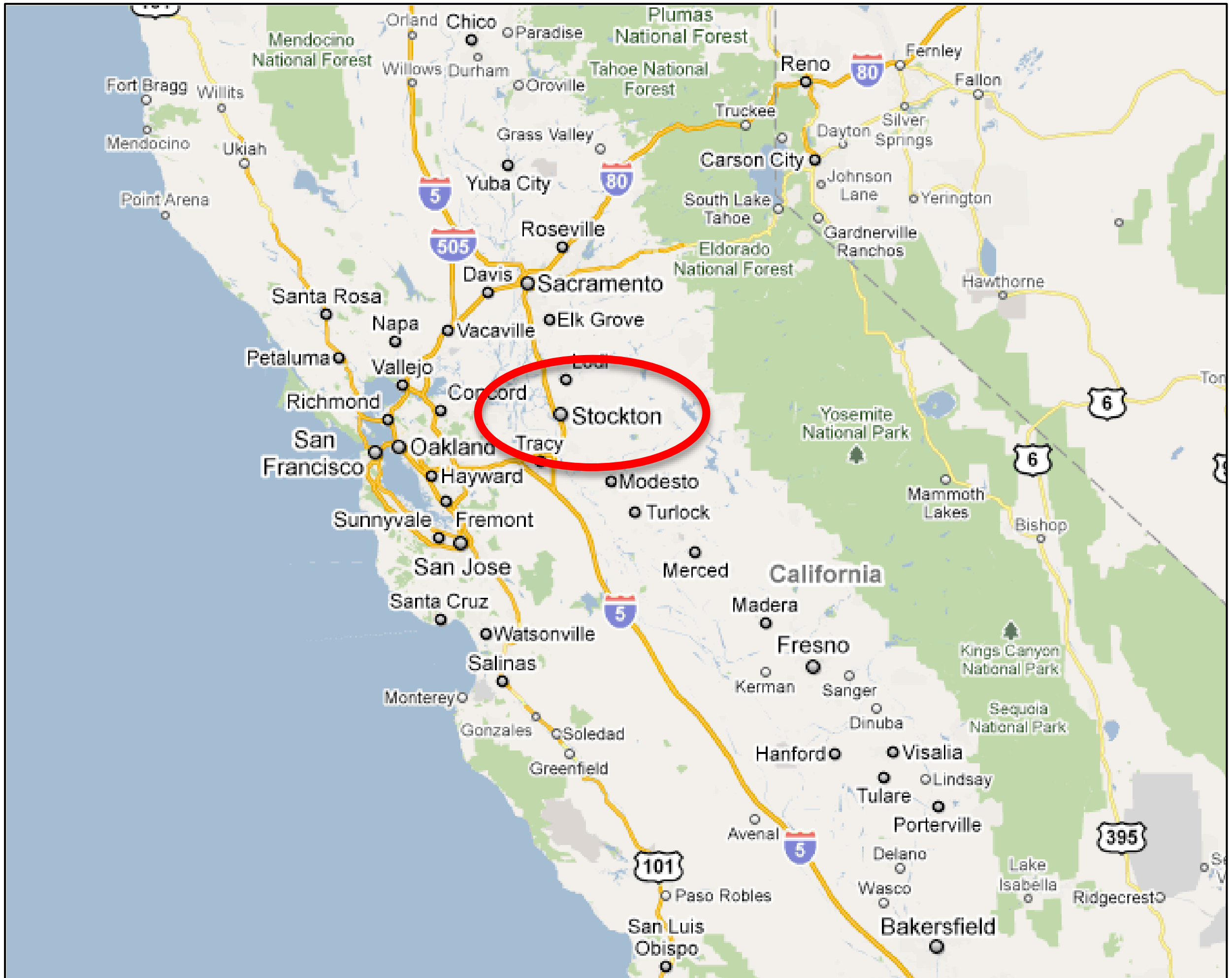
TABLE 1-1 OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES FOR FLOOD, WIND, SNOW, EARTHQUAKE, AND ICE LOADS

Nature of Occupancy	Occupancy Category
Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to: • Agricultural facilities • Certain temporary facilities • Minor storage facilities	I
All buildings and other structures except those listed in Occupancy Categories I, III, and IV	II
Buildings and other structures that represent a substantial hazard to human life in the event of failure, including, but not limited to: • Buildings and other structures where more than 300 people congregate in one area • Buildings and other structures with daycare facilities with a capacity greater than 150 • Buildings and other structures with elementary school or secondary school facilities with a capacity greater than 250 • Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities • Health care facilities with a capacity of 50 or more resident patients, but not having surgery or emergency treatment facilities • Jails and detention facilities Buildings and other structures, not included in Occupancy Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure, including, but not limited to: • Power generating stations[a] • Water treatment facilities • Sewage treatment facilities • Telecommunication centers Buildings and other structures not included in Occupancy Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released. Buildings and other structures containing toxic or explosive substances shall be eligible for classification as Occupancy Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the toxic or explosive substances does not pose a threat to the public.	III
Buildings and other structures designated as essential facilities, including, but not limited to: • Hospitals and other health care facilities having surgery or emergency treatment facilities • Fire, rescue, ambulance, and police stations and emergency vehicle garages • Designated earthquake, hurricane, or other emergency shelters • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response • Power generating stations and other public utility facilities required in an emergency • Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Occupancy Category IV structures during an emergency • Aviation control towers, air traffic control centers, and emergency aircraft hangars • Water storage facilities and pump structures required to maintain water pressure for fire suppression • Buildings and other structures having critical national defense functions Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing highly toxic substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction. Buildings and other structures containing highly toxic substances shall be eligible for classification as Occupancy Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the highly toxic substances does not pose a threat to the public. This reduced classification shall not be permitted if the buildings or other structures also function as essential facilities.	IV

[a]Cogeneration power plants that do not supply power on the national grid shall be designated Occupancy Category II.

Occupancy Category = II (Table 1-1)

Ground Motion Parameters for Stockton



S_S=1.25g

S_1=0.40g

Determining Site Coefficients

TABLE 11.4-1 SITE COEFFICIENT, F_a

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

F_a=1.0

TABLE 11.4-2 SITE COEFFICIENT, F_v

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

F_a=1.4

Determining Design Spectral Accelerations

- S_{DS}=(2/3)F_aS_S=(2/3)x1.0x1.25=0.833
- S_{D1}=(2/3)F_vS_1=(2/3)x1.4x0.40=0.373

Determine Importance Factor, Seismic Design Category

TABLE 11.5-1 IMPORTANCE FACTORS

Occupancy Category	*I*
I or II	1.0
III	1.25
IV	1.5

I = 1.0

TABLE 11.6-1 SEISMIC DESIGN CATEGORY BASED ON SHORT PERIOD RESPONSE ACCELERATION PARAMETER

Value of S_{DS}	Occupancy Category		
	I or II	III	IV
$S_{DS} < 0.167$	A	A	A
$0.167 \leq S_{DS} < 0.33$	B	B	C
$0.33 \leq S_{DS} < 0.50$	C	C	D
$0.50 \leq S_{DS}$	D	D	D

TABLE 11.6-2 SEISMIC DESIGN CATEGORY BASED ON 1-S PERIOD RESPONSE ACCELERATION PARAMETER

Value of S_{D1}	OCCUPANCY CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067$	A	A	A
$0.067 \leq S_{D1} < 0.133$	B	B	C
$0.133 \leq S_{D1} < 0.20$	C	C	D
$0.20 \leq S_{D1}$	D	D	D

Seismic Design Category = D

Select Structural System (Table 12.2-1)

Building height (above grade) = 18+11(12.5)=155.5 ft

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]
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Special steel moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Special steel truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Intermediate steel moment frames	12.2.5.6, 12.2.5.7, 12.2.5.8, 12.2.5.9, and 14.1	4.5	3	4	NL	NL	35[h,i]	NP[h]	NP[i]
4. Ordinary steel moment frames	12.2.5.6, 12.2.5.7, 12.2.5.8, and 14.1	3.5	3	3	NL	NL	NP[h]	NP[h]	NP[i]
5. Special reinforced concrete moment frames	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Special composite steel and concrete moment frames	12.2.5.5 and 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Intermediate composite moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP
11. Ordinary composite moment frames	14.3	3	3	2½	NL	NP	NP	NP	NP

Select Special Steel Moment Frame: R=8, C_d=5.5, Ω_0=3

Establish Diaphragm Behavior and Modeling Requirements

12.3.1 Diaphragm Flexibility.

The structural analysis shall consider the relative stiffness of diaphragms and the vertical elements of the seismic force–resisting system. *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., semi-rigid modeling assumption).*

12.3.1.2 Rigid Diaphragm Condition.

Diaphragms of concrete slabs or concrete filled metal deck with span-to-depth ratios of 3 or less in *structures that have no horizontal irregularities are permitted to be idealized as rigid.*

Due to horizontal irregularities (e.g. reentrant corners) the diaphragms must be modeled as semi-rigid. This will be done by using Shell elements in the SAP 2000 Analysis.

Determine Configuration Irregularities
Horizontal Irregularities

TABLE 12.3-1 HORIZONTAL STRUCTURAL IRREGULARITIES

		Irregularity Type and Description	Reference Section	Seismic Design Category Application
?	1a.	**Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.8.4.3 12.7.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F C, D, E, and F B, C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
?	1b.	**Extreme Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D B, C, and D
√	2.	**Reentrant Corner Irregularity** is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
	3.	**Diaphragm Discontinuity Irregularity** is defined to exist where there are diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
X	4.	**Out-of-Plane Offsets Irregularity** is defined to exist where there are discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.	12.3.3.4 12.3.3.3 12.7.3 Table 12.6-1 16.2.2	D, E, and F B, C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
√	5.	**Nonparallel Systems-Irregularity** is defined to exist where the vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic force–resisting system.	12.5.3 12.7.3 Table 12.6-1 Section 16.2.2	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

Irregularity 2 occurs on lower levels. Irregularity 3 is possible but need not be evaluated because it has same consequences as irregularity 3. Torsional Irregularities will be assessed later.

Determine Configuration Irregularities
Vertical Irregularities

TABLE 12.3-2 VERTICAL STRUCTURAL IRREGULARITIES

		Irregularity Type and Description	Reference Section	Seismic Design Category Application
X	1a.	**Stiffness-Soft Story Irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	Table 12.6-1	D, E, and F
	1b.	**Stiffness-Extreme Soft Story Irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	12.3.3.1 Table 12.6-1	E and F D, E, and F
✓	2.	**Weight (Mass) Irregularity** is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	Table 12.6-1	D, E, and F
✓	3.	**Vertical Geometric Irregularity** is defined to exist where the horizontal dimension of the seismic force–resisting system in any story is more than 130% of that in an adjacent story.	Table 12.6-1	D, E, and F
X	4.	**In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity** is defined to exist where an in-plane offset of the lateral force-resisting elements is greater than the length of those elements or there exists a reduction in stiffness of the resisting element in the story below.	12.3.3.3 12.3.3.4 Table 12.6-1	B, C, D, E, and F D, E, and F D, E, and F
X	5a.	**Discontinuity in Lateral Strength–Weak Story Irregularity** is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 Table 12.6-1	E and F D, E, and F
X	5b.	**Discontinuity in Lateral Strength–Extreme Weak Story Irregularity** is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 12.3.3.2 Table 12.6-1	D, E, and F B and C D, E, and F

Irregularities 2 and 3 occur due to setbacks. Soft story and weak story irregularities are highly unlikely for this system and are not evaluated.

Selection of Method of Analysis (ASCE 7-05)

TABLE 12.6-1 PERMITTED ANALYTICAL PROCEDURES

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Analysis Section 12.8	Modal Response Spectrum Analysis Section 12.9	Seismic Response History Procedures Chapter 16
B, C	Occupancy Category I or II buildings of light-framed construction not exceeding 3 stories in height	P	P	P
	Other Occupancy Category I or II buildings not exceeding 2 stories in height	P	P	P
	All other structures	P	P	P
D, E, F	Occupancy Category I or II buildings of light-framed construction not exceeding 3 stories in height	P	P	P
	Other Occupancy Category I or II buildings not exceeding 2 stories in height	P	P	P
	Regular structures with $T < 3.5T_s$ and all structures of light frame construction	P	P	P
	Irregular structures with $T < 3.5T_s$ and having only horizontal irregularities Type 2, 3, 4, or 5 of Table 12.2-1 or vertical irregularities Type 4, 5a, or 5b of Table 12.3-1	P	P	P
	All other structures	NP	P	P

NOTE: P: Permitted; NP: Not Permitted

Not applicable

System is not "regular"

Vertical irregularities 2 and 3 exist

ELF is not permitted:

Must use Modal Response Spectrum or Response History Analysis

Selection of Method of Analysis (ASCE 7-10)

Table 12.6-1 Permitted Analytical Procedures

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Analysis, Section 12.8[a]	Modal Response Spectrum Analysis, Section 12.9[a]	Seismic Response History Procedures, Chapter 16[a]
B, C	All structures	P	P	P
D, E, F	Risk Category I or II buildings not exceeding 2 stories above the base	P	P	P
	Structures of light frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 160 ft in structural height	P	P	P
	Structures exceeding 160 ft in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures not exceeding 160 ft in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12-2 or vertical irregularities of Type 4, 5a, or 5b in Table 12-3	P	P	P
	All other structures	NP	P	P

[a]P: Permitted; NP: Not Permitted; $T_s = S_{D1}/S_{DS}$.

ELF is not permitted:

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Overview of Presentation

- Describe Building
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Comments on use of ELF for *This System*

ELF is NOT allowed as the *Design Basis Analysis*.

However, ELF (or aspects of ELF) must be used for:

- Preliminary analysis and design
- Evaluation of torsion irregularities and amplification
- Evaluation of system redundancy factors
- Computing P-Delta Effects
- Scaling Response Spectrum and Response History results

Determine Scope of Analysis

12.7.3 Structural Modeling.

A mathematical model of the structure shall be constructed for the purpose of determining member forces and structure displacements resulting from applied loads and any imposed displacements or P-Delta effects.

The model shall include the stiffness and strength of elements that are significant to the distribution of forces and deformations in the structure and represent the spatial distribution of mass and stiffness throughout the structure.

Note: P-Delta effects should not be included directly in the analysis. They are considered indirectly in Section 12.8.7

Determine Scope of Analysis (Continued)

Continuation of 12.7.3:

Structures that have horizontal structural irregularity Type 1a, 1b, 4, or 5 of Table 12.3-1 shall be analyzed using a 3-D representation.

Where a 3-D model is used, a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis shall be included at each level of the structure.

Where the diaphragms have not been classified as rigid or flexible in accordance with Section 12.3.1, the model shall include representation of the diaphragm's stiffness characteristics and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response.

Analysis of structure must be in 3D, and diaphragms must be modeled as semi-rigid

Establish Modeling Parameters

Continuation of 12.7.3:

In addition, the model shall comply with the following:

a) Stiffness properties of concrete and masonry elements shall consider the effects of cracked sections.

b) For steel moment frame systems, the contribution of panel zone deformations to overall story drift shall be included.

Modeling Parameters used in Analysis

1) The floor diaphragm was modeled with shell elements, providing nearly rigid behavior in-plane.

2) Flexural, shear, axial, and torsional deformations were included in all columns and beams.

3) Beam-column joints were modeled using centerline dimensions. This approximately accounts for deformations in the panel zone.

4) Section properties for the girders were based on bare steel, ignoring composite action. This is a reasonable assumption in light of the fact that most of the girders are on the perimeter of the building and are under reverse curvature.

Modeling Parameters used in Analysis (continued)

5) Except for those lateral load-resisting columns that terminate at Levels 5 and 9, all columns of the lateral load resisting system were assumed to be fixed at their base.

6) The basement walls and grade level slab were explicitly modeled using 4-node shell elements. This was necessary to allow the interior columns to continue through the basement level. No additional lateral restraint was applied at the grade level, thus the basement level acts as a very stiff first floor of the structure. This basement level was not relevant for the ELF analysis, but did influence the MRS and MRH analysis as described in later sections of this example

7) P-Delta effects were not included in the mathematical model. These effects are evaluated separately using the procedures provided in section 12.8.7 of the Standard.

Equivalent Lateral Force Analysis

1. Compute Seismic Weight, W (Sec. 12.7.2)
2. Compute Approximate Period of Vibration T_a (Sec. 12.8.2.1)
3. Compute Upper Bound Period of Vibration, $T=C_uT_a$ (Sec. 12.8.2)
4. Compute "Analytical" Natural periods
5. Compute Seismic Base Shear (Sec. 12.8.1)
6. Compute Equivalent Lateral Forces (Sec. 12.8.3)
7. Compute Torsional Amplification Factors (Sec. 12.8.4.3)
8. Determine Orthogonal Loading Requirements (Sec. 12.8)
9. Compute Redundancy Factor ρ (Sec. 12.3.4)
10. Perform Structural Analysis
11. Check Drift and P-Delta Requirements (Sec. 12.9.4 and 12.9.6)
12. Revise Structure in Necessary and Repeat Steps 1-11 [as appropriate]
13. Determine Design-Level Member Forces (Sec. 12.4)

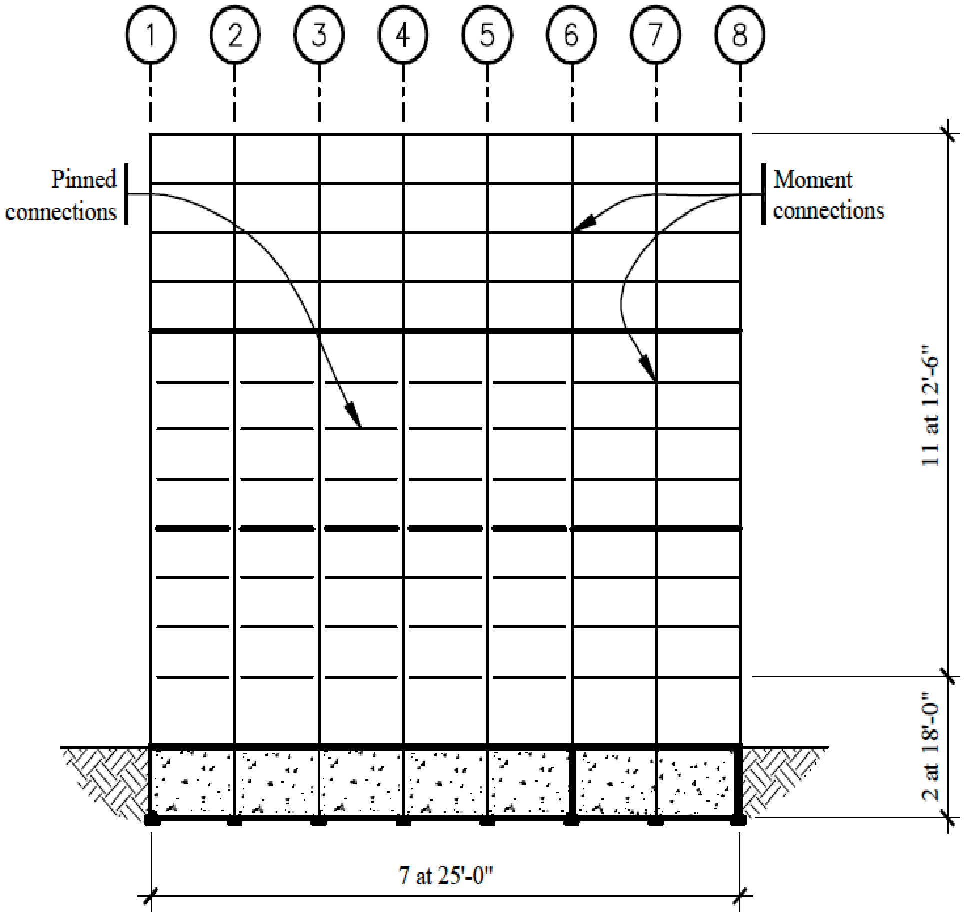
Notes on Computing the Period of Vibration

T_a (Eqn.12.8-7) is an approximate lower bound period, and is based on the measured response of buildings in high seismic regions.

$T=C_uT_a$ is also approximate, but is somewhat more accurate than T_a alone because it is based on the “best fit” of the measured response, and is adjusted for local seismicity. Both of these adjustments are contained in the C_u term.

C_uT_a can only be used if an analytically computed period, called $T_{computed}$ herein, is available from a computer analysis of the structure.

Using Empirical Formulas to Determine T_a



$$T_a = C_t h_n^x$$

From Table 12.8.2:

C_t=0.028

x=0.80

h_n=18+11(12.5)=155.5 ft

$$T_a = 0.028(155.5)^{0.8} = 1.59 \text{ sec}$$

Applies in Both Directions

Adjusted Empirical Period $T=C_uT_a$

TABLE 12.8-1 COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD

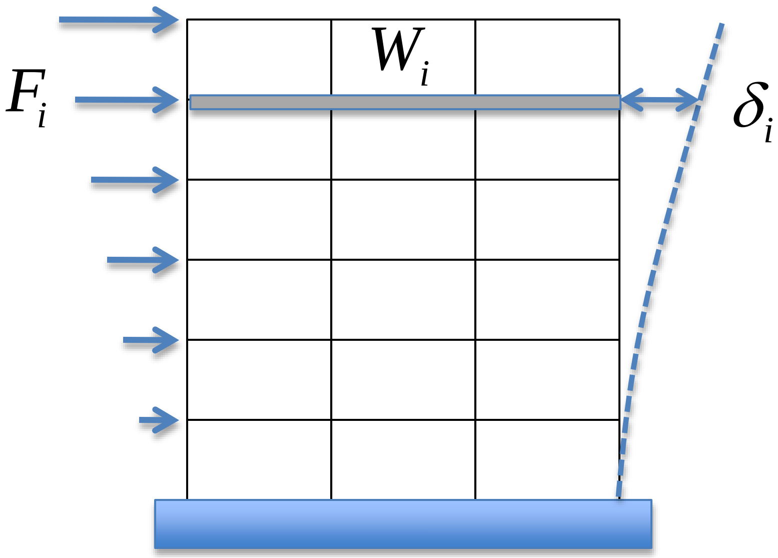
Design Spectral Response Acceleration Parameter at 1 s, S_{D1}	Coefficient C_u
≥ 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤ 0.1	1.7

S_{D1}=0.373
Gives C_u=1.4

$$T = 1.4(1.59) = 2.23 \text{ sec}$$

Applies in Both Directions

Use of Rayleigh Analysis to Determine $T_{computed}$



Building has n Levels

$$T_{computed} = \frac{2\pi}{\omega_{computed}}$$

$$\omega_{computed} = \sqrt{\frac{g\sum_{i=1}^{n}\delta_i F_i}{\sum_{i=1}^{n}\delta_i^2 W_i}}$$

Use of Rayleigh Analysis to Determine $T_{computed}$

Table 4.1-9 Rayleigh Analysis for X Direction Period of Vibration

Level	Drift, δ(in.)	Force, F (kips)	Weight, W (kips)	δF (in.-kips)	$\delta^2 W/g$ (in.-kips-sec^2)
R	6.67	186.9	1657	1247	191
12	6.35	154.0	1596	979	167
11	5.90	129.9	1596	767	144
10	5.34	107.6	1596	575	118
9	4.73	186.3	3403	881	197
8	4.15	100.8	2331	418	104
7	3.52	77.0	2331	271	75
6	2.87	56.2	2331	162	50
5	2.24	71.4	4324	160	56
4	1.71	31.5	3066	54	23
3	1.17	16.6	3066	19	11
2	0.64	6.3	3097	4	3
Σ				5536	1138

$\omega = (5536/1138)^{0.5} = 2.21$ rad/sec. $T = 2\pi/\omega = 2.85$ sec. 1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

X-Direction $T_{computed}$ = 2.85 sec.
Y-Direction $T_{computed}$ = 2.56 sec.

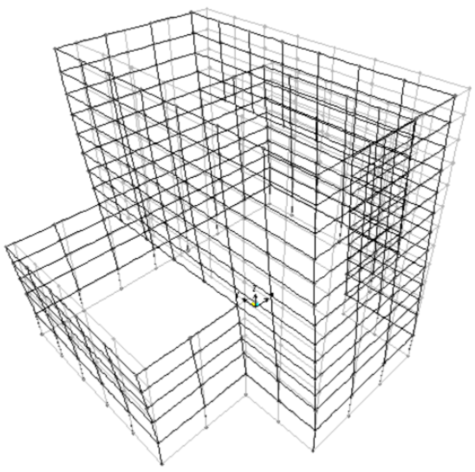
(see Text)

Periods Computed Using Eigenvalue Analysis

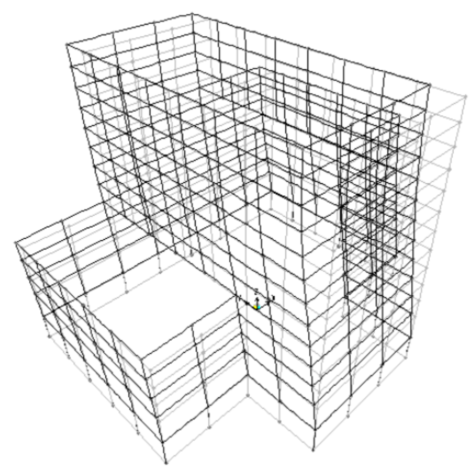
$$K\Phi = M\Phi\Omega^2$$

$\Omega =$ Diagonal matrix containing circular frequencies ω

$\Phi =$ Mode Shape Matrix



Mode 1 T=2.87 sec



Mode 2: T:2.60 sec

Range of Periods Computed for This Example

T_a=1.59 sec

$C_u T_a$=2.23 sec

$T_{computed}$ = 2.87 sec in X direction

2.60 sec in Y direction

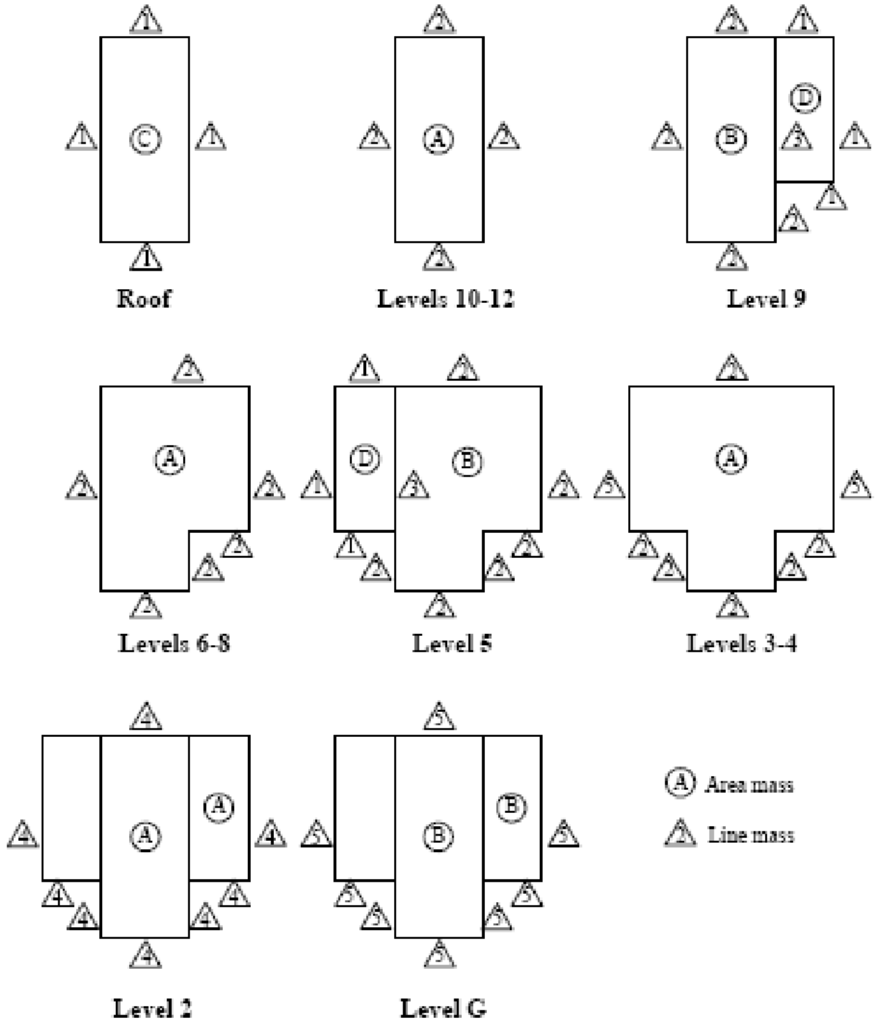
Periods of Vibration for Computing Seismic Base Shear (Eqns 12.8-1, 12.8-3, and 12.8-4)

if $T_{computed}$ is not available use T_a

if $T_{computed}$ is available, then:

- if $T_{computed} > C_u T_a$ use $C_u T_a$
- if $T_a <= T_{computed} <= C_u T_a$ use $T_{computed}$
- if $T_{computed} < T_a$ use T_a

Area and Line Weight Designations



Area and Line Weight Values

Table 4.1-1 Area Weights Contributing to Masses on Floor Diaphragms

Mass Type	Area Weight Designation				
	A	B	C	D	E
Slab and Deck (psf)	50	75	50	75	75
Structure (psf)	20	20	20	20	50
Ceiling and Mechanical (psf)	15	15	15	15	15
Partition (psf)	10	10	0	0	10
Roofing (psf)	0	0	15	15	0
Special (psf)	0	0	0	60	25
Total (psf)	95	120	100	185	175

See Figure 4.1-4 for mass location. 1.0 psf = 47.9 N/m^2.

Table 4.1-2 Line Weights Contributing to Masses on Floor Diaphragms

Mass Type	Line Weight Designation				
	1	2	3	4	5
From Story Above (plf)	60.0	93.8	93.8	93.8	135.0
From Story Below (plf)	93.8	93.8	0.0	135.0	1350.0
Total (plf)	153.8	187.6	93.8	228.8	1485.0

See Figure 4.1-4 for mass location. 1.0 plf = 14.6 N/m.

Weights at Individual Levels

Table 4.1-3 Floor Weight, Floor Mass, Mass Moment of Inertia, and Center of Mass Locations

Level	Weight (kips)	Mass (kip-sec^{2}/in.)	Mass Moment of Inertia (in.-kip-sec^{2}/radian)	X Distance to C.M. (in.)	Y Distance to C.M. (in.)
R	1657	4.287	2.072×10^{6}	1260	1050
12	1596	4.130	2.017×10^{6}	1260	1050
11	1596	4.130	2.017×10^{6}	1260	1050
10	1596	4.130	2.017×10^{6}	1260	1050
9	3403	8.807	5.309×10^{6}	1638	1175
8	2331	6.032	3.703×10^{6}	1553	1145
7	2331	6.032	3.703×10^{6}	1553	1145
6	2331	6.032	3.703×10^{6}	1553	1145
5	4320	11.19	9.091×10^{6}	1160	1206
4	3066	7.935	6.356×10^{6}	1261	1184
3	3066	7.935	6.356×10^{6}	1261	1184
2	3097	8.015	6.437×10^{6}	1262	1181
G	6525	16.89	1.503×10^{7}	1265	1149
Σ	36912				

Total Building Weight=36,912 k.

Weight above grade = 30,394 k.

Calculation of ELF Base Shear

$$V = C_S W \qquad \text{(12.8-1)}$$

$$C_S = \frac{S_{DS}}{R/I} = \frac{0.833}{8/1} = 0.104 \qquad \text{(12.8-2)}$$

$$C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.373}{2.23(8/1)} = 0.021 \qquad \text{(12.8-3)}$$

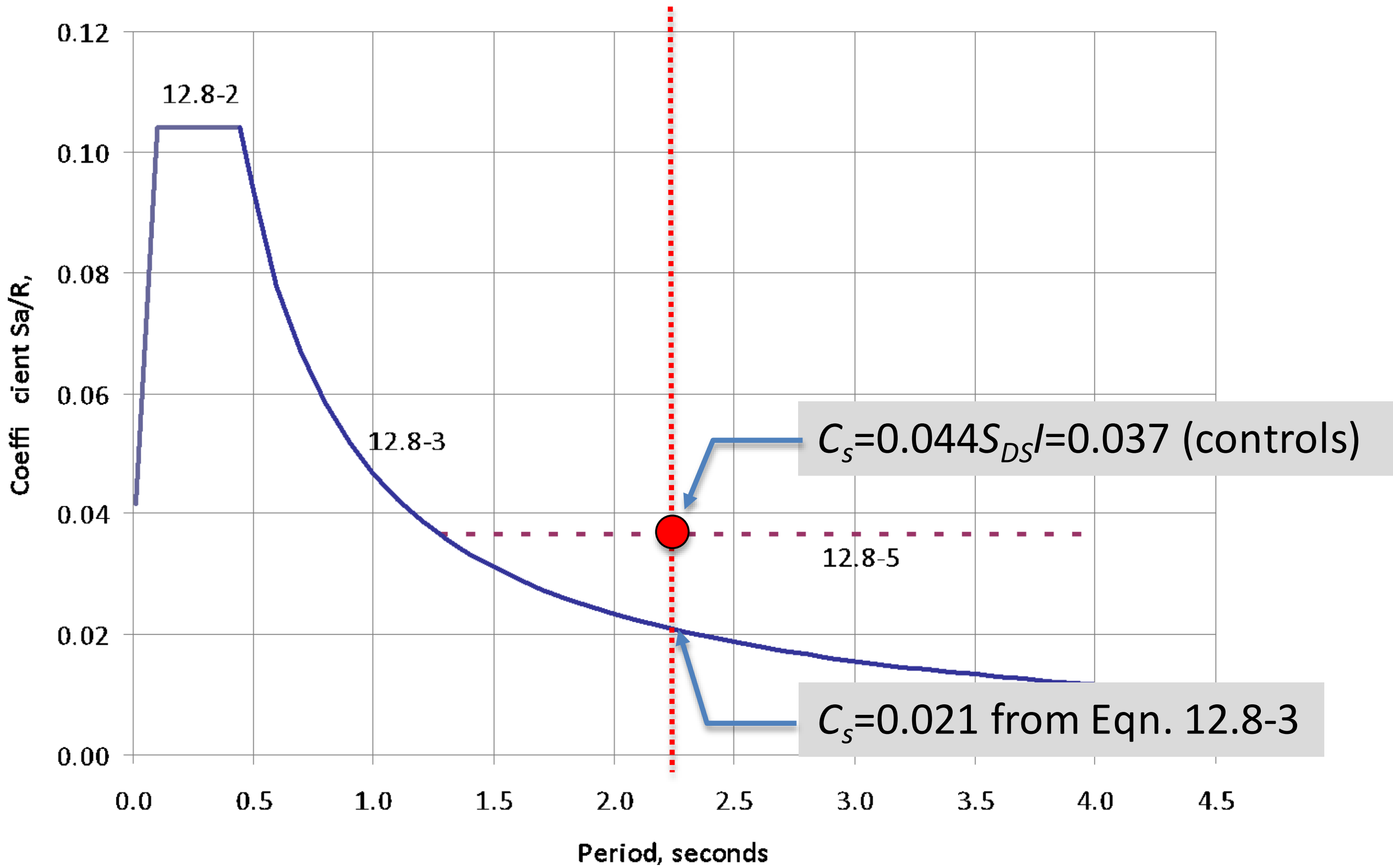
$$\boxed{C_S = 0.044 S_{DS} I = 0.044(0.833)(1) = 0.0307} \qquad \text{(12.8-5)}$$

Controls

$$V = 0.037(30394) = 1124 \text{ kips}$$

Concept of $R_{effective}$

$C_u T_a$=2.23 sec



$$R_{effective} = (0.021/0.037) \times 8 = 4.54$$

Issues Related to Period of Vibration and Drift

12.8.6.1 Minimum Base Shear for Computing Drift

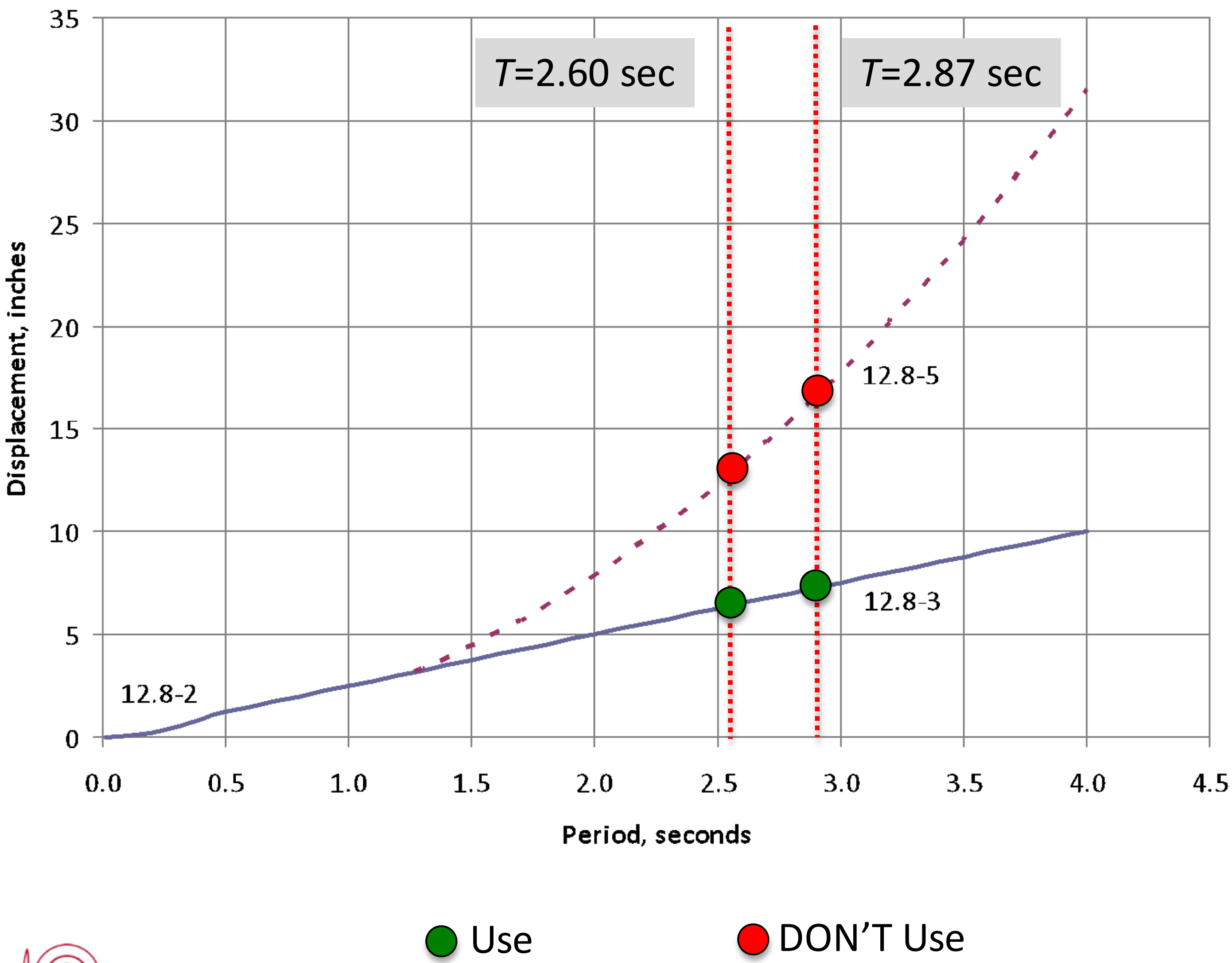
The elastic analysis of the seismic force-resisting system for computing drift shall be made using the prescribed seismic design forces of Section 12.8.

EXCEPTION: Eq. 12.8-5 need not be considered for computing drift

12.8.6.2 Period for Computing Drift

For determining compliance with the story drift limits of Section 12.12.1, it is permitted to determine the elastic drifts, (δ_{xe}), using seismic design forces based on the computed fundamental period of the structure without the upper limit ($C_u T_a$) specified in Section 12.8.2.

Using Eqns. 12.8-3 or 12.8-5 for Computing ELF Displacements



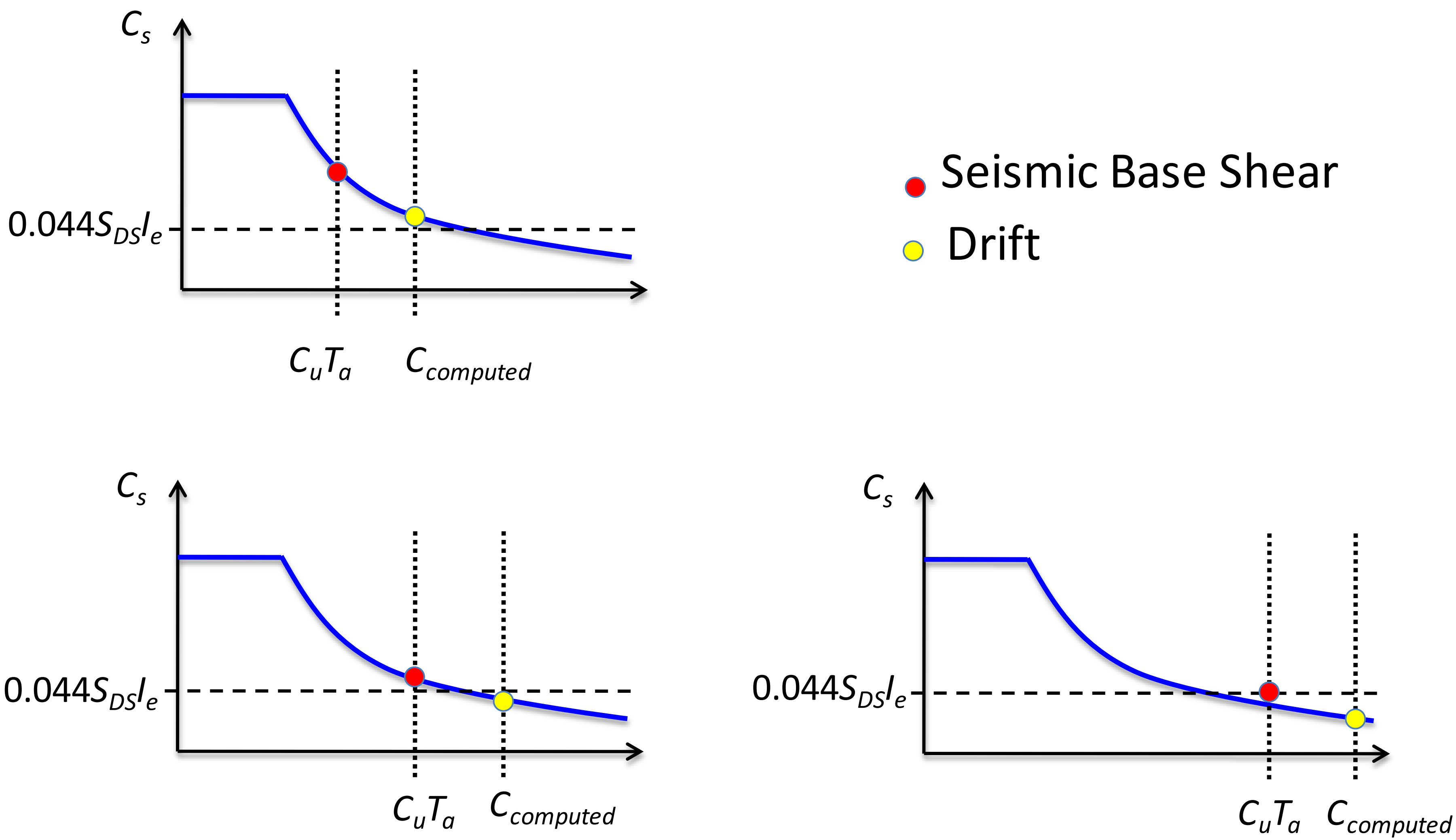
What if Equation 12.8-6 had Controlled Base Shear?

$$C_s = \frac{0.5S_1}{(R/I)}$$

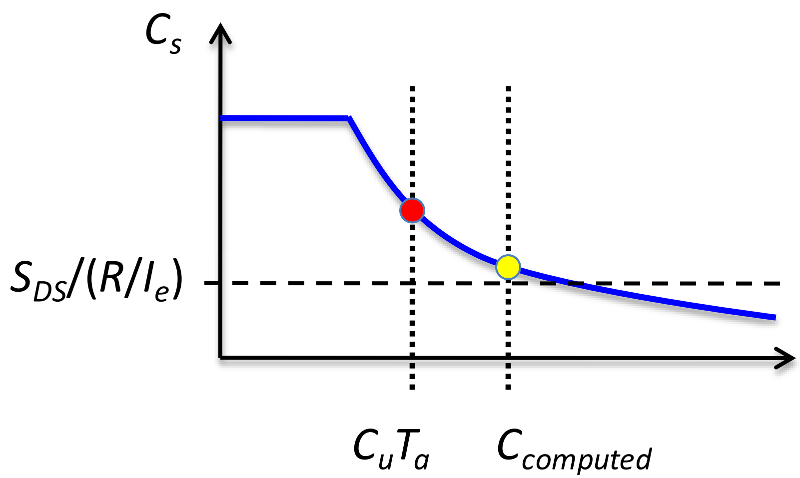
Eqn. 12.8-6, applicable only when S_1 >= 0.6g

This equation represents the “true” response spectrum shape for near-field ground motions. Thus, the lateral forces developed on the basis of this equation must be used for determining component design forces **and** displacements used for computing drift.

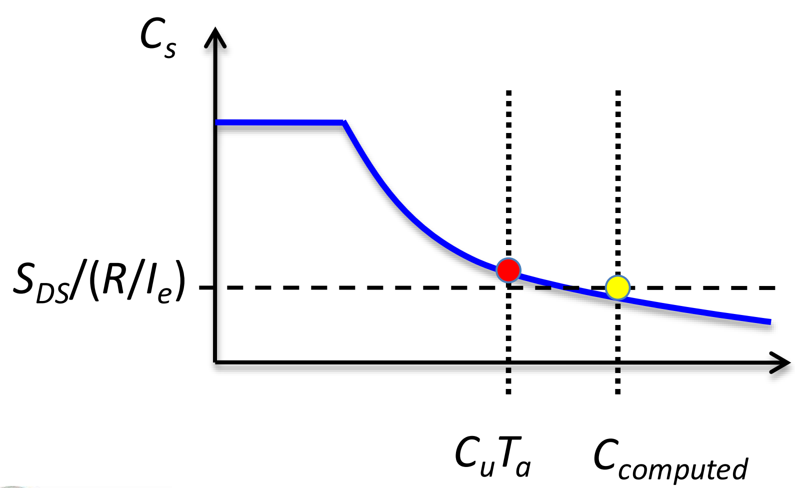
When Equation 12.8-5 May Control Seismic Base Shear (S_1 < 0.6g)

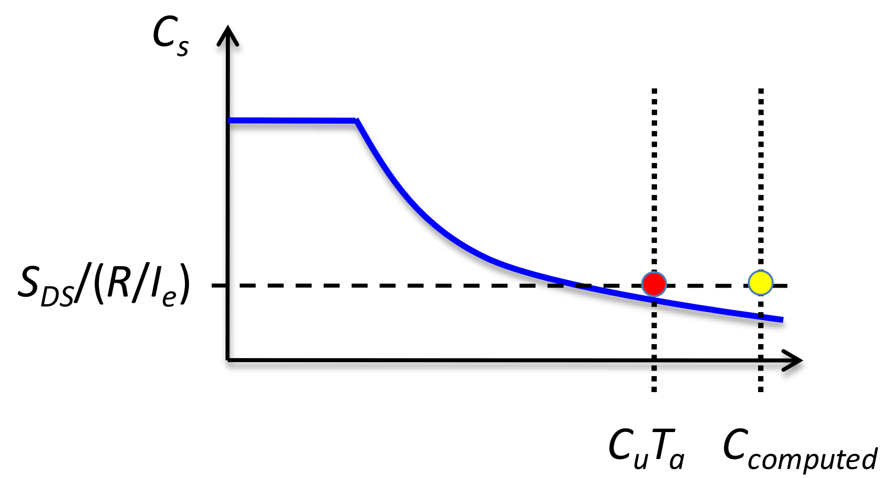


When Equation 12.8-6 May Control Seismic Base Shear (S_1 >= 0.6g)



Seismic Base Shear
Drift

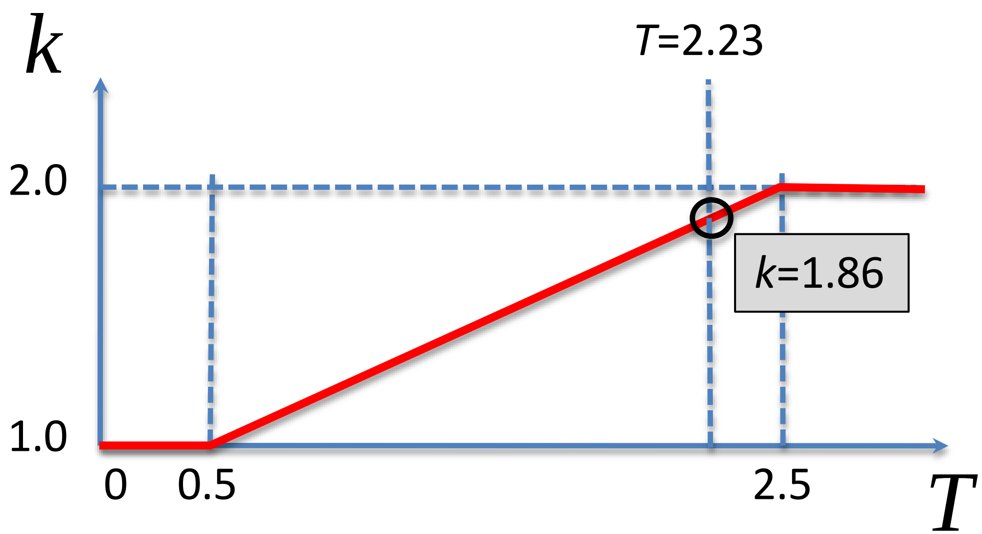




Calculation of ELF Forces

$$F_x = C_{vx} V \qquad \text{(12.8-11)}$$

$$C_{vs} = \frac{w_x h^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \text{(12.8-12)}$$



Calculation of ELF Forces (continued)

Table 4.1-4 Equivalent Lateral Forces for Building Responding in X and Y Directions

Level x	w_x (kips)	h_x (ft)	$w_x h_x^k$	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
R	1657	155.5	20272144	0.1662	186.9	186.9	2336
12	1596	143.0	16700697	0.1370	154.0	340.9	6597
11	1596	130.5	14081412	0.1155	129.9	470.8	12482
10	1596	118.0	11670590	0.0957	107.6	578.4	19712
9	3403	105.5	20194253	0.1656	186.3	764.7	29271
8	2331	93.0	10933595	0.0897	100.8	865.5	40090
7	2331	80.5	8353175	0.0685	77.0	942.5	51871
6	2331	68.0	6097775	0.0500	56.2	998.8	64356
5	4324	55.5	7744477	0.0635	71.4	1070.2	77733
4	3066	43.0	3411857	0.0280	31.5	1101.7	91505
3	3066	30.5	1798007	0.0147	16.6	1118.2	103372
2	3097	18.0	679242	0.0056	6.3	1124.5	120694
Σ	30394	-	121937234	1.00	1124.5		

Values in column 4 based on exponent k=1.865. 1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN.

Inherent and Accidental Torsion

12.8.4.1 Inherent Torsion. For diaphragms that are not flexible, the distribution of lateral forces at each level shall consider the effect of the inherent torsional moment, M_t, *resulting from eccentricity* between the locations of the center of mass and the center of rigidity. For flexible diaphragms, the distribution of forces to the vertical elements shall account for the position and distribution of the masses supported.

Inherent torsion effects are automatically included in 3D structural analysis, and member forces associated with such effects need not be separated out from the analysis.

Inherent and Accidental Torsion (continued)

12.8.4.2 Accidental Torsion. Where diaphragms are not flexible, the design shall include the inherent torsional moment (M_t) (kip or kN) resulting from the location of the structure masses plus the accidental torsional moments (M_{ta}) (kip or kN) caused by assumed displacement of the center of mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied forces.

Where earthquake forces are applied concurrently in two orthogonal directions, the required 5 percent displacement of the center of mass need not be applied in both of the orthogonal directions at the same time, but shall be applied in the direction that produces the greater effect.

Inherent and Accidental Torsion (continued)

12.8.4.3 Amplification of Accidental Torsional Moment. Structures assigned to Seismic Design Category C, D, E, or F, where Type 1a or 1b torsional irregularity exists as defined in Table 12.3-1 shall have the effects accounted for by multiplying M_{ta} at each level by a torsional amplification factor (A_x) as illustrated in Fig. 12.8-1 and determined from the following equation:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \qquad (12.8\text{-}14)$$

where

δ_{max} = the maximum displacement at Level x (in. or mm) computed assuming $A_x = 1$

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x computed assuming $A_x = 1$ (in. or mm)

EXCEPTION: The accidental torsional moment need not be amplified for structures of light-frame construction.

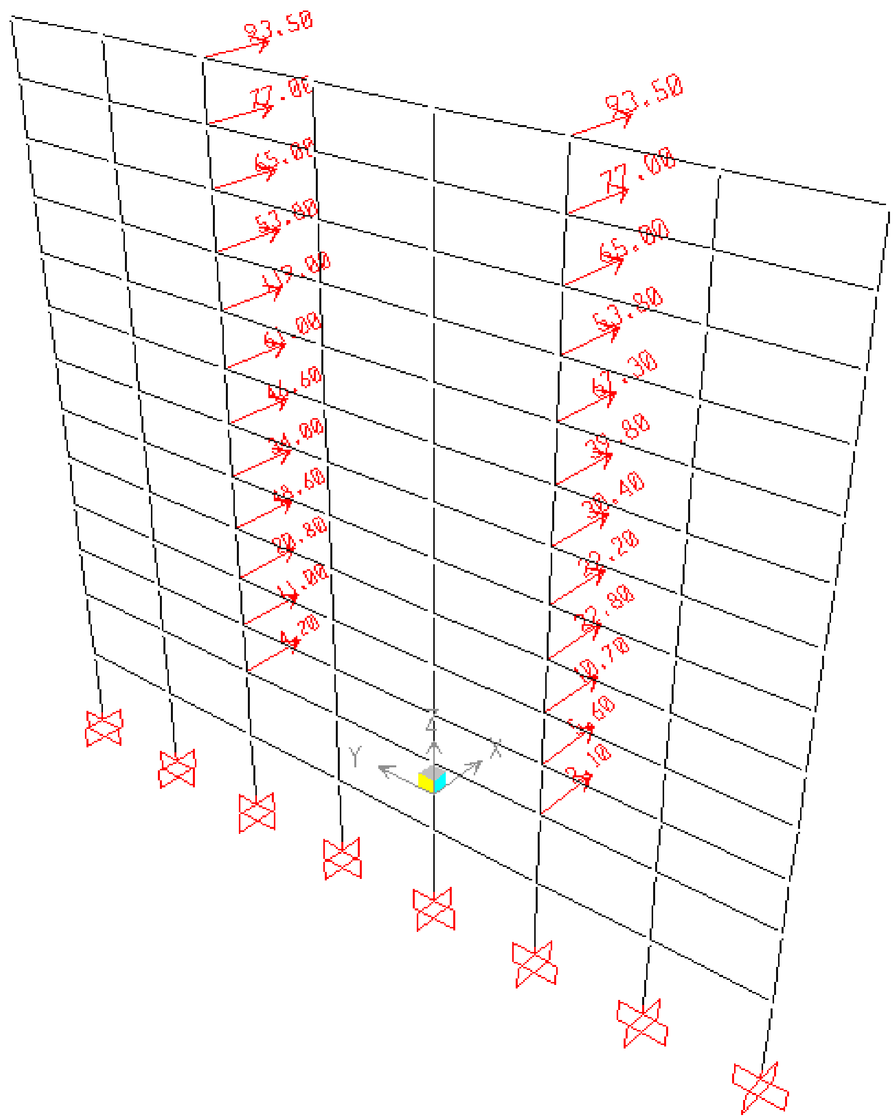
The torsional amplification factor (A_x) is not required to exceed 3.0. The more severe loading for each element shall be considered for design.

Determine Configuration Irregularities
Horizontal Irregularities

TABLE 12.3-1 HORIZONTAL STRUCTURAL IRREGULARITIES

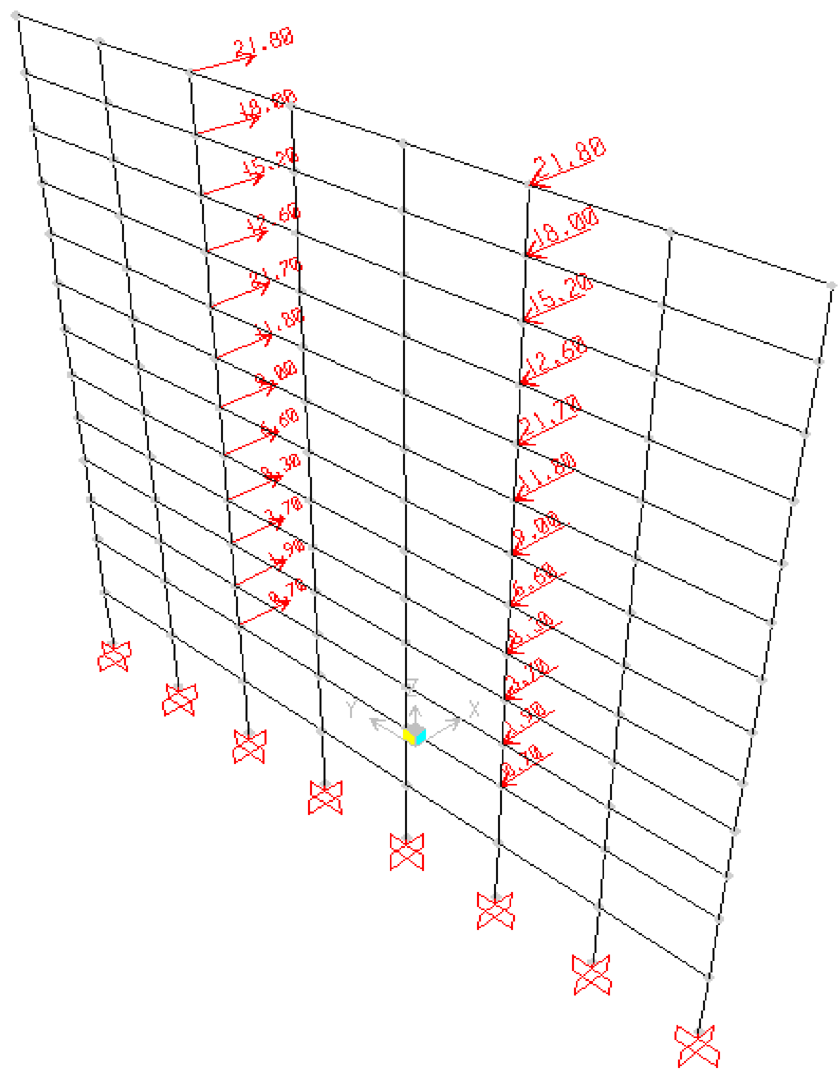
	Irregularity Type and Description	Reference Section	Seismic Design Category Application
1a.	**Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.8.4.3 12.7.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F C, D, E, and F B, C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	**Extreme Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D B, C, and D
2.	**Reentrant Corner Irregularity** is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
3.	**Diaphragm Discontinuity Irregularity** is defined to exist where there are diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
4.	**Out-of-Plane Offsets Irregularity** is defined to exist where there are discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.	12.3.3.4 12.3.3.3 12.7.3 Table 12.6-1 16.2.2	D, E, and F B, C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
5.	**Nonparallel Systems-Irregularity** is defined to exist where the vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic force–resisting system.	12.5.3 12.7.3 Table 12.6-1 Section 16.2.2	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

Application of Equivalent Lateral Forces (X Direction)



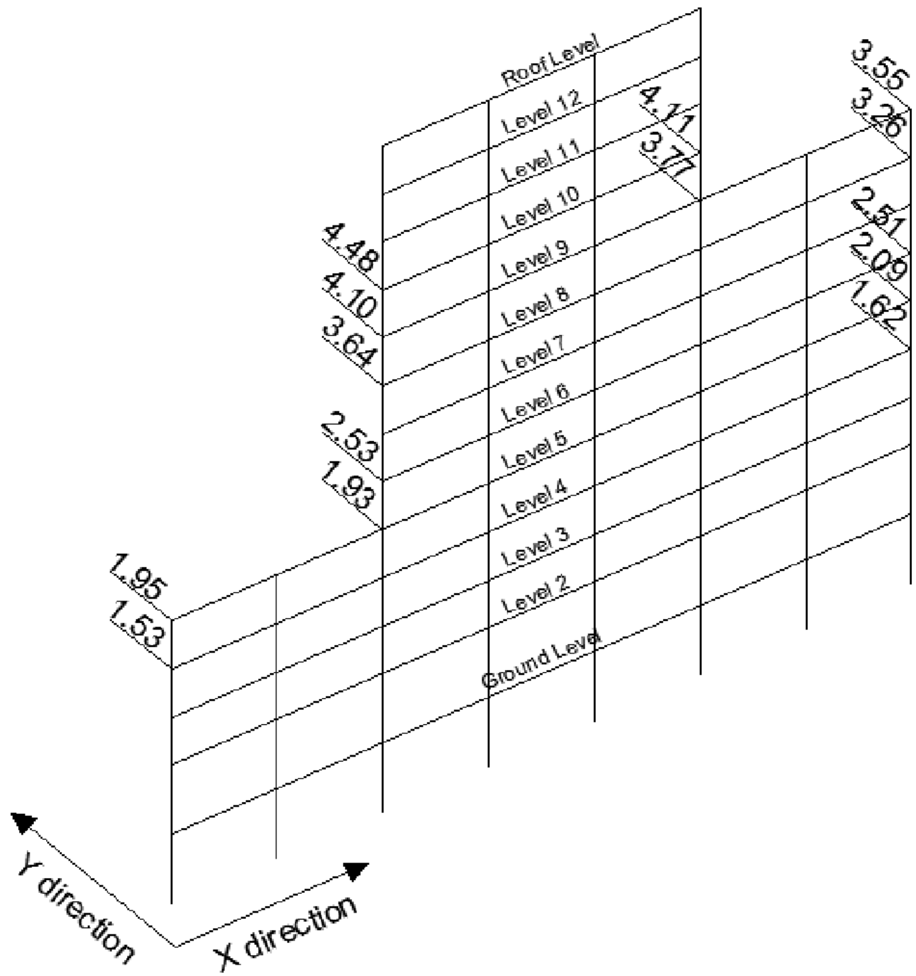
Forces in Kips

Application of Torsional Forces (Using X-Direction Lateral Forces)



Forces in Kips

Stations for Monitoring Drift for Torsion Irregularity Calculations with ELF Forces Applied in X Direction



Results of Torsional Irregularity Calculations For ELF Forces Applied in X Direction

Table 4.1-5a Computation for torsional irregularity with ELF loads acting in X direction and Torsional Moment applied Counterclockwise

Level	$\delta 1$ (in)	$\delta 2$ (in)	$\Delta 1$ (in)	$\Delta 2$ (in)	Δavg (in)	Δmax (in)	$\Delta max/\Delta avg$	Irregularity
R	7.27	6.15	0.34	0.29	0.31	0.34	1.08	None
12	6.93	5.87	0.48	0.42	0.45	0.48	1.07	None
11	6.44	5.45	0.60	0.51	0.55	0.60	1.07	None
10	5.85	4.93	0.66	0.56	0.61	0.66	1.08	None
9	5.19	4.37	0.65	0.54	0.59	0.65	1.10	None
8	4.54	3.84	0.69	0.58	0.64	0.69	1.09	None
7	3.84	3.26	0.70	0.59	0.65	0.70	1.09	None
6	3.14	2.67	0.69	0.58	0.63	0.69	1.09	None
5	2.46	2.09	0.60	0.50	0.55	0.60	1.09	None
4	1.86	1.60	0.59	0.50	0.55	0.59	1.08	None
3	1.27	1.10	0.58	0.49	0.53	0.58	1.08	None
2	0.69	0.61	0.69	0.61	0.65	0.69	1.06	None

1.0 in. = 25.4 mm

Result: There is not a Torsional Irregularity for Loading in the X Direction

Results of Torsional Irregularity Calculations For ELF Forces Applied in Y Direction

Table 4.1-5b Computation for torsional irregularity with ELF loads acting in Y direction, and Torsional Moment applied Clockwise

Level	δ1 (in)	δ2 (in)	Δ1 (in)	Δ2 (in)	Δavg (in)	Δmax (in)	Δmax/Δavg	Irregularity
R	5.19	4.77	0.15	0.14	0.15	0.15	1.03	None
12	5.03	4.63	0.25	0.23	0.24	0.25	1.03	None
11	4.79	4.40	0.31	0.29	0.30	0.31	1.04	None
10	4.48	4.11	0.38	0.34	0.36	0.38	1.06	None
9	4.10	3.77, 3.55	0.46	0.28	0.37	0.46	1.24	Irregularity
8	3.64	3.26	0.54	0.36	0.45	0.54	1.20	None
7	3.09	2.90	0.56	0.39	0.47	0.56	1.18	None
6	2.53	2.51	0.60	0.42	0.51	0.60	1.18	None
5	1.93, 1.95	2.09	0.41	0.47	0.44	0.47	1.06	None
4	1.53	1.62	0.47	0.50	0.48	0.50	1.03	None
3	1.07	1.12	0.47	0.50	0.48	0.50	1.03	None
2	0.60	0.63	0.60	0.63	0.61	0.63	1.03	None

1.0 in. = 25.4 mm

Result: There is a minor Torsional Irregularity for Loading in the Y Direction

Results of Torsional Amplification Calculations For ELF Forces Applied in Y Direction

(X Direction Results are Similar)

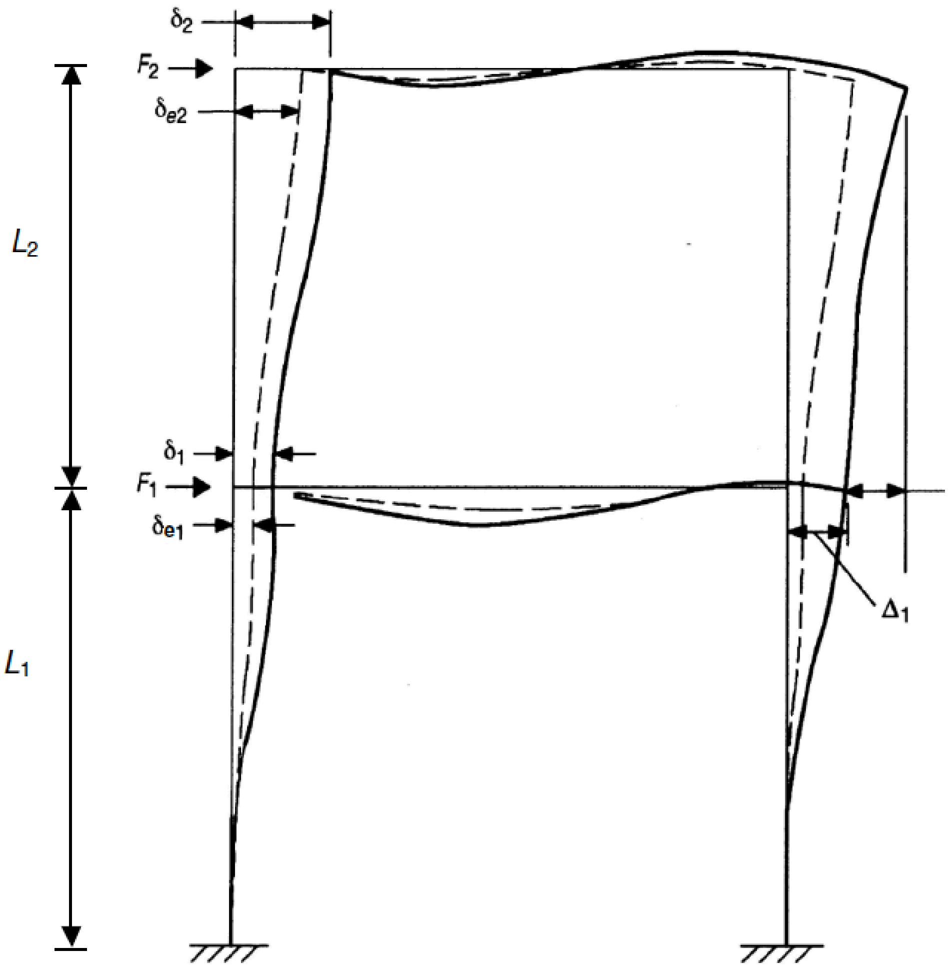
Table 4.1-5d Amplification Factor A_x for Accidental Torsional Moment Loads acting in the Y direction and Torsional Moment applied Clockwise

Level	δ_1 (in)	δ_2 (in)	δ_{avg} (in)	δ_{max} (in)	A_x calculated	A_x corrected
R	5.19	4.77	4.98	5.19	0.75	1.00
12	5.03	4.63	4.83	5.03	0.75	1.00
11	4.79	4.40	4.59	4.79	0.76	1.00
10	4.48	4.11	4.29	4.48	0.76	1.00
9	4.10	3.55	3.82	4.10	0.80	1.00
8	3.64	3.26	3.45	3.64	0.77	1.00
7	3.09	2.90	3.00	3.09	0.74	1.00
6	2.53	2.51	2.52	2.53	0.70	1.00
5	1.95	2.09	2.02	2.09	0.74	1.00
4	1.53	1.62	1.58	1.62	0.73	1.00
3	1.07	1.12	1.10	1.12	0.73	1.00
2	0.60	0.63	0.61	0.63	0.73	1.00

1.0 in. = 25.4 mm

Result: Amplification of Accidental Torsion Need not be Considered

Drift and Deformation



Story Level 2

F_2 = strength-level design earthquake force

δ_{e2} = elastic displacement computed under strength-level design earthquake forces

δ_2 = $C_d\,\delta_{e2}/I_E$ = amplified displacement

Δ_2 = $(\delta_{e2} - \delta_{e1})\,C_d/I_E \leq \Delta_a$ (Table 12.12-1)

Story Level 1

F_1 = strength-level design earthquake force

δ_{e1} = elastic displacement computed under strength-level design earthquake forces

δ_1 = $C_d\,\delta_{e1}/I_E$ = amplified displacement

Δ_1 = $\delta_1 \leq \Delta_a$ (Table 12.12-1)

Δ_i = Story Drift

Δ_i/L_i = Story Drift Ratio

δ_2 = Total Displacement

FIGURE 12.8-2 STORY DRIFT DETERMINATION

Drift and Deformation (Continued)

12.12 DRIFT AND DEFORMATION

12.12.1 Story Drift Limit. The design story drift (Δ) as determined in Sections 12.8.6, 12.9.2, or 16.1, shall not exceed the allowable story drift (Δ_a) as obtained from Table 12.12-1 for any story. For structures with significant torsional deflections, the maximum drift shall include torsional effects. For structures assigned to Seismic Design Category C, D, E, or F having horizontal irregularity Types 1a or 1b of Table 12.3-1, the design story drift, Δ, shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the story under consideration.

Not strictly Followed in this Example due to *very minor* torsion irregularity

TABLE 12.12-1 ALLOWABLE STORY DRIFT, Δ_a[a,b]

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$[c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$\boxed{0.020h_{sx}}$	$0.015h_{sx}$	$0.010h_{sx}$

Drift and Deformation (Continued)

ASCE 7-05 (ASCE 7-10) Similar

12.8.6.2 Period for Computing Drift. For determining compliance with the story drift limits of Section 12.12.1, it is permitted to determine the elastic drifts, (δ_{xe}), using seismic design forces based on the computed fundamental period of the structure without the upper limit ($C_u T_a$) specified in Section 12.8.2.

ASCE 7-10

12.8.6.1 Minimum Base Shear for Computing Drift

The elastic analysis of the seismic force-resisting system for computing drift shall be made using the prescribed seismic design forces of Section 12.8.

EXCEPTION: Eq. 12.8-5 need not be considered for computing drift.

Computed Drifts in X Direction

Table 4.1-7 ELF Drift for Building Responding in X Direction

Level	1 Total drift from SAP2000 (in.)	2 Story drift from SAP2000 (in.)	3 Amplified story drift (in.)	4 Amplified drift times 0.568 (in.)	5 Allowable drift (in.)
R	6.67	0.32	1.74	0.99	3.00
12	6.35	0.45	2.48	1.41	3.00
11	5.90	0.56	3.07	1.75	3.00
10	5.34	0.62	3.39	1.92	3.00
9	4.73	0.58	3.20	1.82	3.00
8	4.15	0.63	3.47	1.97	3.00
7	3.52	0.64	3.54	2.01	3.00
6	2.87	0.63	3.47	1.97	3.00
5	2.24	0.54	2.95	1.67	3.00
4	1.71	0.54	2.97	1.69	3.00
3	1.17	0.53	2.90	1.65	3.00
2	0.64	0.64	3.51	2.00	4.32

Column 4 adjusts for *Standard* Eq. 12.8-3 (for drift) vs 12.8-5 (for strength).
1.0 in. = 25.4 mm.

C_d Amplified drift based on forces from Eq. 12.8-5

Modified for forces based on Eq. 12.8-3

Computed Drifts in Y Direction

Table 4.1-8 ELF Drift for Building Responding in Y Direction

Level	1 Total drift from SAP2000 (in.)	2 Story drift from SAP2000 (in.)	3 Amplified story drift (in.)	4 Amplified drift times 0.568 (in.)	5 Allowable drift (in.)
R	4.86	0.15	0.81	0.46	3.00
12	4.71	0.24	1.30	0.74	3.00
11	4.47	0.30	1.64	0.93	3.00
10	4.17	0.36	1.96	1.11	3.00
9	3.82	0.37	2.05	1.16	3.00
8	3.44	0.46	2.54	1.44	3.00
7	2.98	0.48	2.64	1.50	3.00
6	2.50	0.48	2.62	1.49	3.00
5	2.03	0.45	2.49	1.42	3.00
4	1.57	0.48	2.66	1.51	3.00
3	1.09	0.48	2.64	1.50	3.00
2	0.61	0.61	3.35	1.90	4.32

Column 4 adjusts for *Standard* Eq. 12.8-3 (for drift) versus Eq. 12.8-5 (for strength).
1.0 in. = 25.4 mm.

C_d Amplified drift based on forces from Eq. 12.8-5

Modified for forces based on Eq. 12.8-3

P-Delta Effects

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

Eq. 12.8-16*

The drift Δ in Eq. 12.8-16 is drift from ELF analysis, multiplied by C_d and divided by *I*.

*The importance factor *I* was inadvertently left out of Eq. 12.8-16 in ASCE 7-05. It is properly included in ASCE 7-10.

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

Eq. 12.8-17

The term β in Eq. 12.8-17 is essentially the inverse of the Computed story over-strength.

P-Delta Effects for modal response spectrum analysis and modal response history analysis are checked using the ELF procedure indicated on this slide.

P-Delta Effects

Table 4.1-11 Computation of P-Delta Effects for X Direction Response

Level	h_{sx} (in.)	Δ (in.)	P_D (kips)	P_L (kips)	P_T (kips)	P_X (kips)	V_X(kips)	θ_X
R	150	1.74	1656.5	315.0	1971.5	1971.5	186.9	0.022
12	150	2.48	1595.8	315.0	1910.8	3882.3	340.9	0.034
11	150	3.07	1595.8	315.0	1910.8	5793.1	470.8	0.046
10	150	3.39	1595.8	315.0	1910.8	7703.9	578.4	0.055
9	150	3.20	3403.0	465.0	3868.0	11571.9	764.7	0.059
8	150	3.47	2330.8	465.0	2795.8	14367.7	865.8	0.070
7	150	3.54	2330.8	465.0	2795.8	17163.5	942.5	0.078
6	150	3.47	2330.8	465.0	2795.8	19959.3	998.8	0.084
5	150	2.95	4323.8	615.0	4938.8	24898.1	1070.2	0.083
4	150	2.97	3066.1	615.0	3681.1	28579.2	1101.7	0.093
3	150	2.90	3066.1	615.0	3681.1	32260.3	1118.2	0.101
2	216	3.51	3097.0	615.0	3712.0	35972.3	1124.5	0.095

1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

Marginally exceeds limit of 0.091 using β=1.0. θ would be less than θ_{max} if actual β were computed and used.

Orthogonal Loading Requirements

12.5.4 Seismic Design Categories D through F. Structures assigned to Seismic Design Category D, E, or F shall, as a minimum, conform to the requirements of Section 12.5.3.

12.5.3 Seismic Design Category C. Loading applied to structures assigned to Seismic Design Category C shall, as a minimum, conform to the requirements of Section 12.5.2 for Seismic Design Category B and the requirements of this section. *Structures that have horizontal structural irregularity Type 5* in Table 12.3-1 shall the following procedure [for ELF Analysis]:

Continued on Next Slide

Orthogonal Loading Requirements (continued)

Orthogonal Combination Procedure. The structure shall be analyzed using the equivalent lateral force analysis procedure of Section 12.8 with the loading applied independently in any two orthogonal directions and the most critical load effect due to direction of application of seismic forces on the structure is permitted to be assumed to be satisfied if components and their foundations are designed for the following combination of prescribed loads: ***100 percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction***; the combination requiring the maximum component strength shall be used.

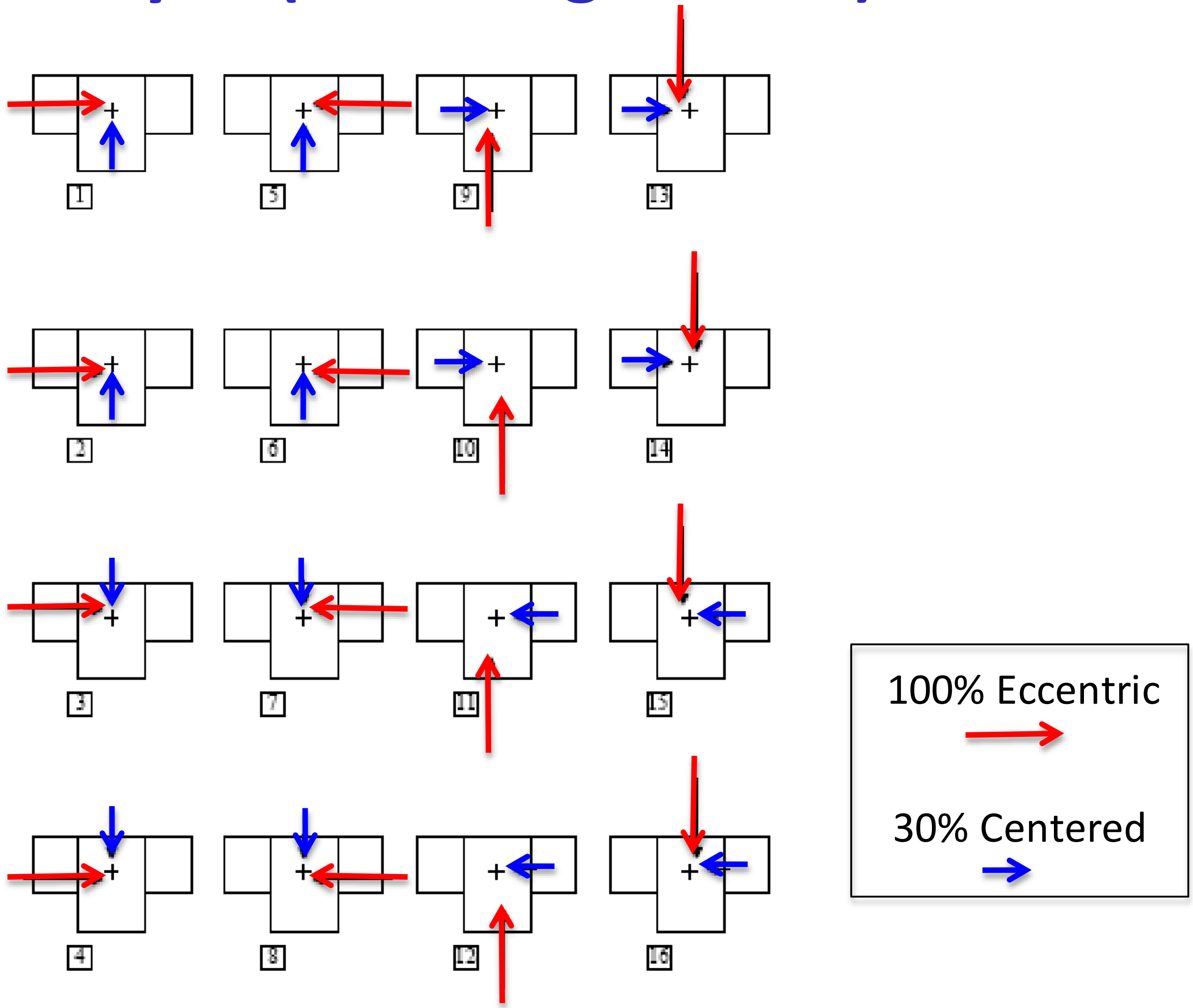
ASCE 7-05 Horizontal Irregularity Type 5

Nonparallel Systems-Irregularity is defined to exist where the vertical lateral force-resisting elements are not parallel to ***or symmetric about*** the major orthogonal axes of the seismic force–resisting system.

The system in question clearly has nonsymmetrical lateral force resisting elements so a Type 5 Irregularity exists, and orthogonal combinations are required. *Thus, 100%-30% procedure given on the previous slide is used.*

Note: The words “or symmetric about” have been removed from the definition of a Type 5 Horizontal Irregularity in ASCE 7-10. Thus, the system under consideration does not have a Type 5 irregularity in ASCE 7-10.

16 Basic Load Combinations used in ELF Analysis (Including Torsion)



Combination of Load Effects

$$1.2D + 1.0E + 0.5L + \cancel{0.2S}$$

$$0.9D + 1.0E + \cancel{1.6H}$$

$$E = E_h + E_v$$

$$E_h = \rho Q_E \qquad (\rho = 1.0)$$

$$E_v = 0.2S_{DS} \qquad (S_{DS} = 0.833g)$$

Redundancy Factor

12.3.4.2 Redundancy Factor, *ρ, for Seismic Design Categories* D through F. For structures assigned to Seismic Design Category D, E, or F, ρ shall equal 1.3 unless one of the following two conditions is met, whereby ρ is permitted to be taken as 1.0:

} See next slide

a) Each story resisting more than 35 percent of the base shear
in the direction of interest shall comply with Table 12.3-3.

b) Structures that are regular in plan at all levels provided that the seismic force–resisting systems consist of at least two bays of seismic force–resisting perimeter framing on each
side of the structure in each orthogonal direction at each
story resisting more than 35 percent of the base shear. The
number of bays for a shear wall shall be calculated as the
length of shear wall divided by the story height or two times

Structure is NOT regular at all Levels.

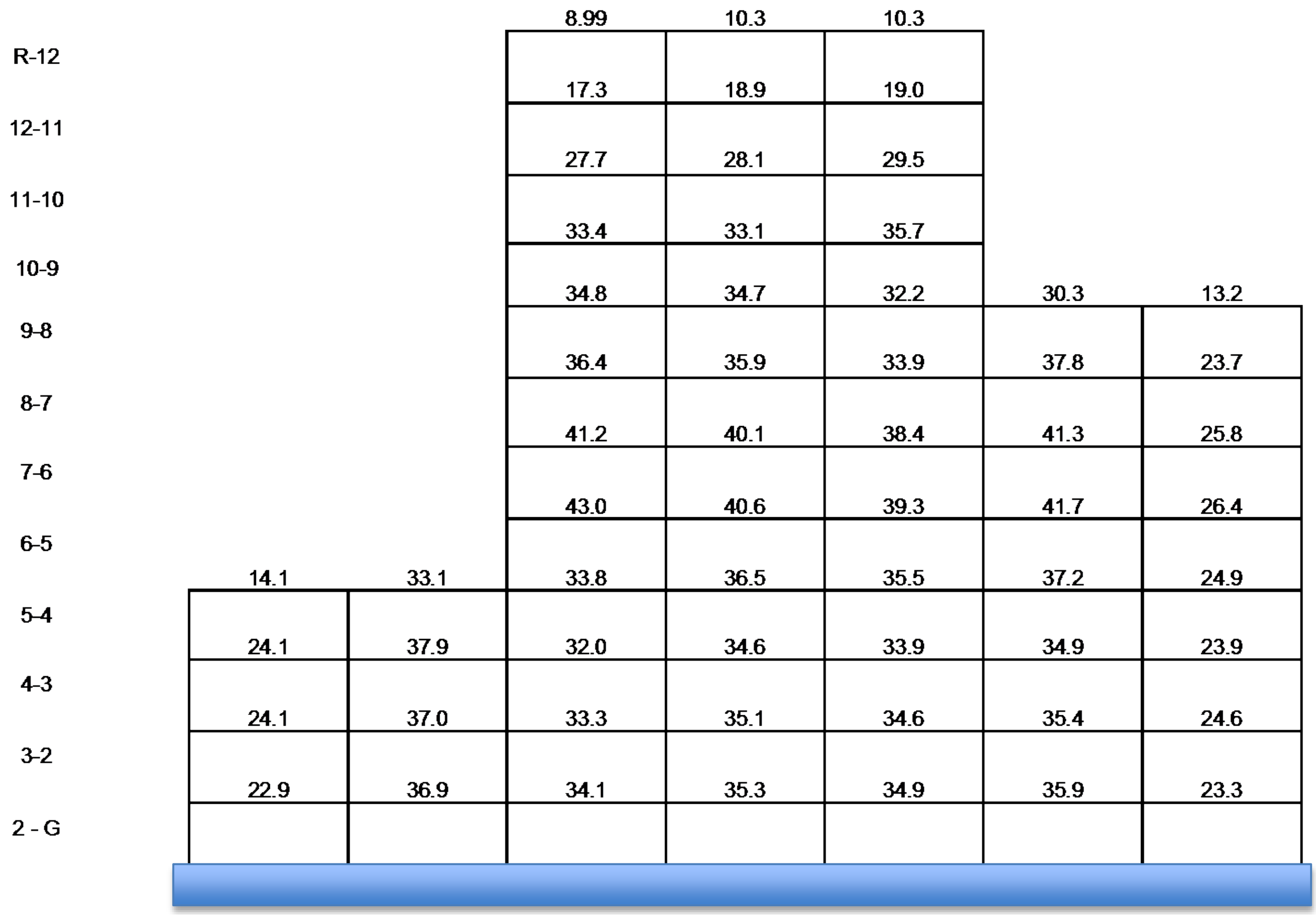
Redundancy, Continued

TABLE 12.3-3 REQUIREMENTS FOR EACH STORY RESISTING MORE THAN 35% OF THE BASE SHEAR

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).

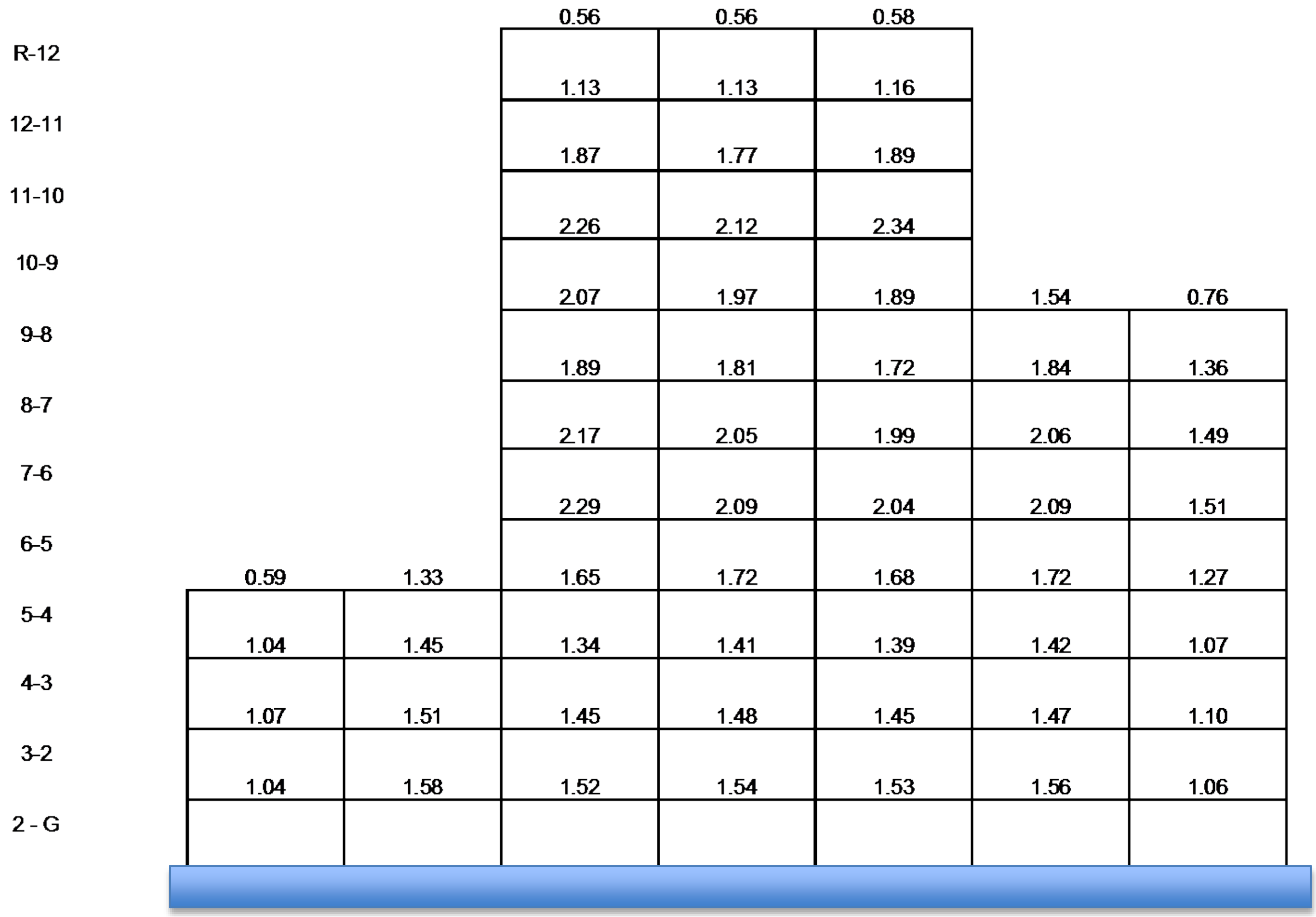
It can be seen by inspection that removal of one beam in this structure will not result in a result in a significant loss of strength or lead to an extreme torsional irregularity. **Hence ρ = 1 for this system**. (This is applicable to ELF, MRS, and MRH analyses).

Seismic Shears in Beams of Frame 1 from ELF Analysis



Seismic Shears in Girders, kips, *Excluding Accidental Torsion*

Seismic Shears in Beams of Frame 1 from ELF Analysis



Seismic Shears in Girders, kips, *Accidental Torsion Only*

Overview of Presentation

- Describe Building
- Describe/Perform steps common to all analysis types
- Overview of Equivalent Lateral Force analysis
- **Overview of Modal Response Spectrum Analysis**
- Overview of Modal Response History Analysis
- Comparison of Results
- Summary and Conclusions

Modal Response Spectrum Analysis
Part 1: Analysis

1. Develop ***Elastic*** response spectrum (Sec. 11.4.5)
2. Develop adequate finite element model (Sec. 12.7.3)
3. Compute modal frequencies, effective mass, and mode shapes
4. Determine number of modes to use in analysis (Sec. 12.9.1)
5. Perform modal analysis in each direction, combining each direction's
   results by use of CQC method (Sec. 12.9.3)
6. Compute Equivalent Lateral Forces (ELF) in each direction (Sec. 12.8.1
   through 12.8.3)
7. Determine accidental torsions (Sec 12.8.4.2), amplified if necessary (Sec. 12.8.4.3)
8. Perform static Torsion analysis

Modal Response Spectrum Analysis Part 2: Drift and P-Delta for Systems *Without* Torsion Irregularity

1. Multiply all dynamic displacements by C_d/R (Sec. 12.9.2).
2. Compute SRSS of interstory drifts based on displacements at center of mass at each level.
3. Check drift Limits in accordance with Sec. 12.12 and Table 12.2-1. Note: drift Limits for Special Moment Frames in SDC D and above must be divided by the Redundancy Factor (Sec. 12.12.1.1)
4. Perform P-Delta analysis using Equivalent Lateral Force procedure
5. Revise structure if necessary

Note: when centers of mass of adjacent levels are not vertically aligned the drifts should be based on the difference between the displacement at the upper level and the displacement of the point on the level below which is the vertical projection of the center of mass of the upper level. (This procedure is included in ASCE 7-10.)

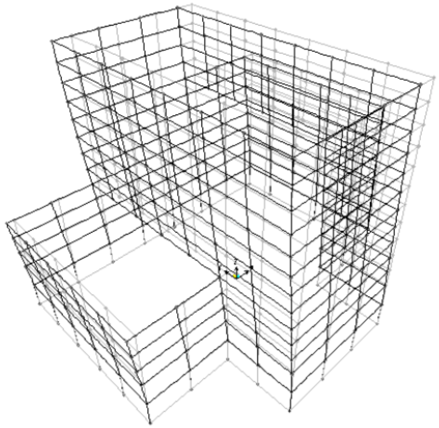
Modal Response Spectrum Analysis Part 2: Drift and P-Delta for Systems *With* Torsion Irregularity

1. Multiply all dynamic displacements by C_d/R (Sec. 12.9.2).
2. Compute SRSS of story drifts based on displacements at the *edge* of the building
3. Using results from the static torsion analysis, determine the drifts at the same location used in Step 2 above. Torsional drifts may be based on the computed period of vibration (without the C_uT_a limit). Torsional drifts should be based on computed displacements
multiplied by C_d and divided by I.
4. Add drifts from Steps 2 and 3 and check drift limits in Table 12.12-1.
Note: Drift limits for special moment frames in SDC D and above must be divided by the Redundancy Factor (Sec. 12.12.1.1)
5. Perform P-Delta analysis using Equivalent Lateral Force procedure
6. Revise structure if necessary

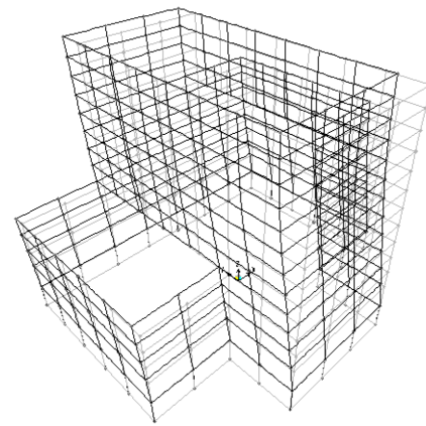
Modal Response Spectrum Analysis
Part 3: Obtaining Member Design Forces

1. Multiply all dynamic force quantities by I/R (Sec. 12.9.2)
2. Determine dynamic base shears in each direction
3. Compute scale factors for each direction (Sec. 12.9.4) and apply to respective member force results in each direction
4. Combine results from two orthogonal directions, if necessary (Sec. 12.5)
5. Add member forces from static torsion analysis (Sec. 12.9.5). Note that static torsion forces may be scaled by factors obtained in Step 3
6. Determine redundancy factor (Sec. 12.3.4)
7. Combine seismic and gravity forces (Sec. 12.4)
8. Design and detail structural components

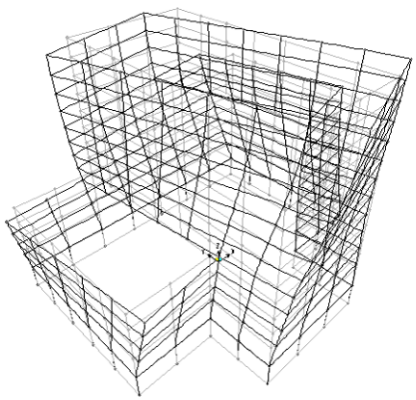
Mode Shapes for First Four Modes



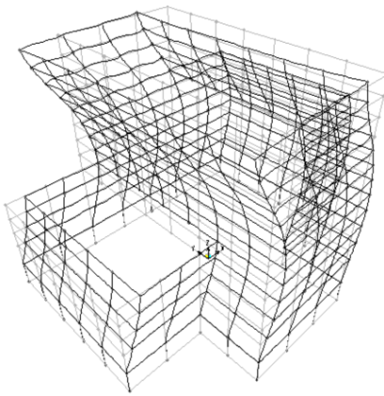
Mode 1 T=2.87 sec
(1st Mode Translation X)



Mode 2: T:2.60 sec
(1st Mode Translation Y)

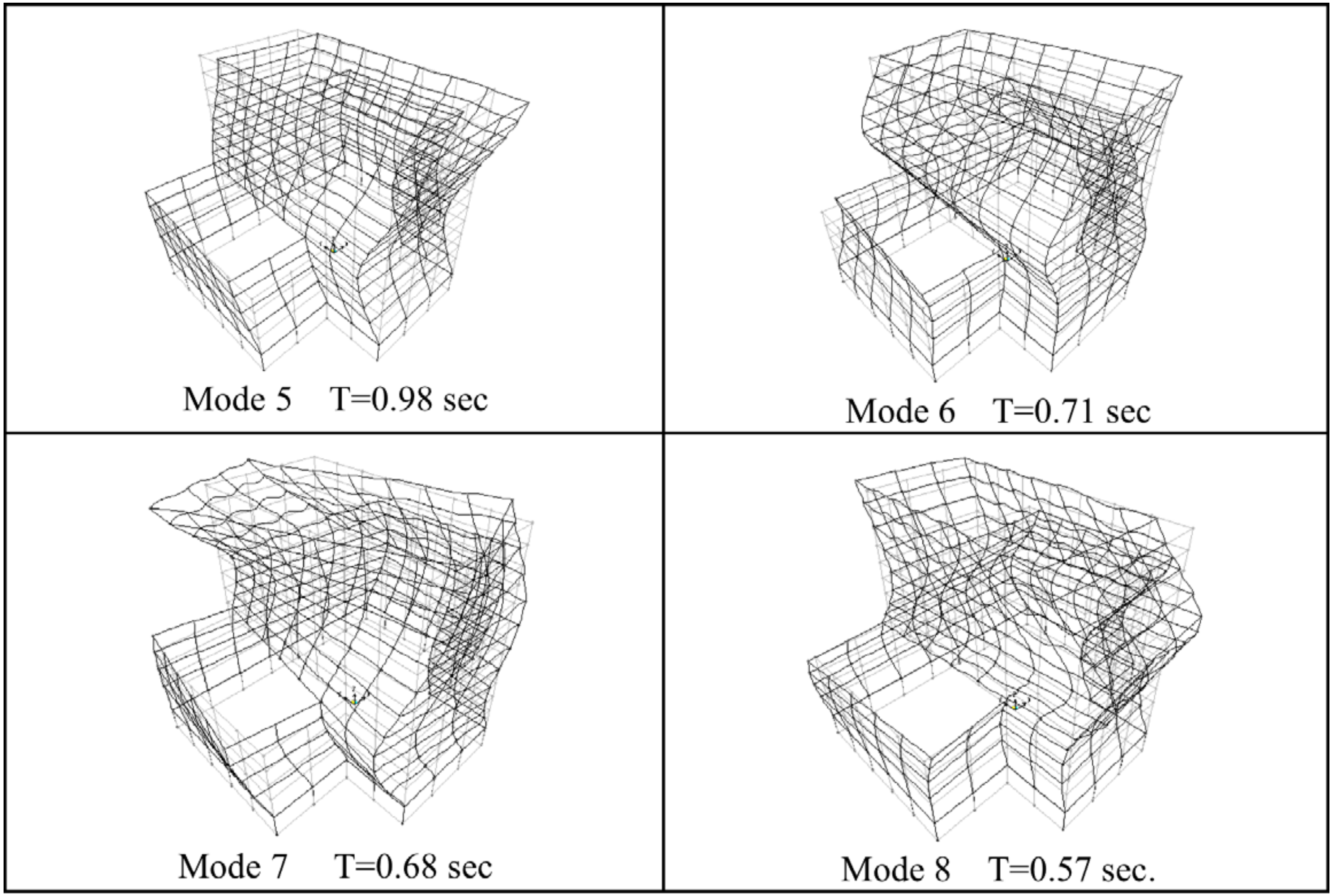


Mode 3 T=1.57 sec
(1st Mode Torsion)



Mode 4 T=1.15 sec
(2nd Mode X)

Mode Shapes for Modes 5-8



Number of Modes to Include in Response Spectrum Analysis

12.9.1 Number of Modes

An analysis shall be conducted to determine the natural modes of vibration for the structure. 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.

Effective Masses for First 12 Modes

Table 4.1-13 Computed Periods and Effective Mass Factors (Lower Modes)

Mode	Period (seconds)	Effective Mass Factor, [Accum Mass Factor]		
		X Translation	Y Translation	Z Rotation
1	2.87	0.6446 [0.64]	0.0003 [0.00]	0.0028 [0.00]
2	2.60	0.0003 [0.65]	0.6804 [0.68]	0.0162 [0.02]
3	1.57	0.0035 [0.65]	0.0005 [0.68]	0.5806 [0.60]
4	1.15	0.1085 [0.76]	0.0000 [0.68]	0.0000 [0.60]
5	0.975	0.0000 [0.76]	0.0939 [0.78]	0.0180 [0.62]
6	0.705	0.0263 [0.78]	0.0000 [0.78]	0.0271 [0.64]
7	0.682	0.0056 [0.79]	0.0006 [0.79]	0.0687 [0.71]
8	0.573	0.0000 [0.79]	0.0188 [0.79]	0.0123 [0.73]
9	0.434	0.0129 [0.80]	0.0000 [0.79]	0.0084 [0.73]
10	0.387	0.0048 [0.81]	0.0000 [0.79]	0.0191 [0.75]
11	0.339	0.0000 [0.81]	0.0193 [0.81]	0.0010 [0.75]
12	0.300	0.0089 [0.82]	0.0000 [0.81]	0.0003 [0.75]

12 Modes Appears to be Insufficient

Effective Masses for Modes 108-119

Table 4.1-14 Computed Periods and Effective Mass Factors (Higher Modes)

Mode	Period (seconds)	Effective Mass Factor, [Accum Effective Mass]		
		X Translation	Y Translation	Z Rotation
108	0.0693	0.0000 [0.83]	0.0000 [0.83]	[illegible]
109	0.0673	0.0000 [0.83]	0.0000 [0.83]	0.0000 [0.79]
110	0.0671	0.0000 [0.83]	0.0354 [0.86]	0.0000 [0.79]
111	0.0671	0.0000 [0.83]	0.0044 [0.87]	0.0000 [0.79]
112	0.0669	0.0000 [0.83]	0.1045 [0.97]	0.0000 [0.79]
113	0.0663	0.0000 [0.83]	0.0000 [0.97]	0.0000 [0.79]
114	0.0646	0.0000 [0.83]	0.0000 [0.97]	0.0000 [0.79]
115	0.0629	0.0000 [0.83]	0.0000 [0.97]	0.0000 [0.79]
116	0.0621	0.0008 [0.83]	0.0010 [0.97]	0.0000 [0.79]
117	0.0609	0.0014 [0.83]	0.0009 [0.97]	0.0000 [0.79]
118	0.0575	0.1474 [0.98]	0.0000 [0.97]	0.0035 [0.80]
119	0.0566	0.0000 [0.98]	0.0000 [0.97]	0.0000 [0.80]

Virtually the Same as 12 Modes

118 Modes Required to Capture Dynamic Response of Stiff Basement Level and Grade Level Slab

Effective Masses for First 12 Modes

Table 4.1-13 Computed Periods and Effective Mass Factors (Lower Modes)

Mode	Period (seconds)	Effective Mass Factor, [Accum Mass Factor]		
		X Translation	Y Translation	Z Rotation
1	2.87	0.6446 [0.64]	0.0003 [0.00]	0.0028 [0.00]
2	2.60	0.0003 [0.65]	0.6804 [0.68]	0.0162 [0.02]
3	1.57	0.0035 [0.65]	0.0005 [0.68]	0.5806 [0.60]
4	1.15	0.1085 [0.76]	0.0000 [0.68]	0.0000 [0.60]
5	0.975	0.0000 [0.76]	0.0939 [0.78]	0.0180 [0.62]
6	0.705	0.0263 [0.78]	0.0000 [0.78]	0.0271 [0.64]
7	0.682	0.0056 [0.79]	0.0006 [0.79]	0.0687 [0.71]
8	0.573	0.0000 [0.79]	0.0188 [0.79]	0.0123 [0.73]
9	0.434	0.0129 [0.80]	0.0000 [0.79]	0.0084 [0.73]
10	0.387	0.0048 [0.81]	0.0000 [0.79]	0.0191 [0.75]
11	0.339	0.0000 [0.81]	0.0193 [0.81]	0.0010 [0.75]
12	0.300	0.0089 [0.82]	0.0000 [0.81]	0.0003 [0.75]

12 Modes are Actually Sufficient to Represent the Dynamic Response of the *Above Grade* Structure

Inelastic Design Response Spectrum Coordinates

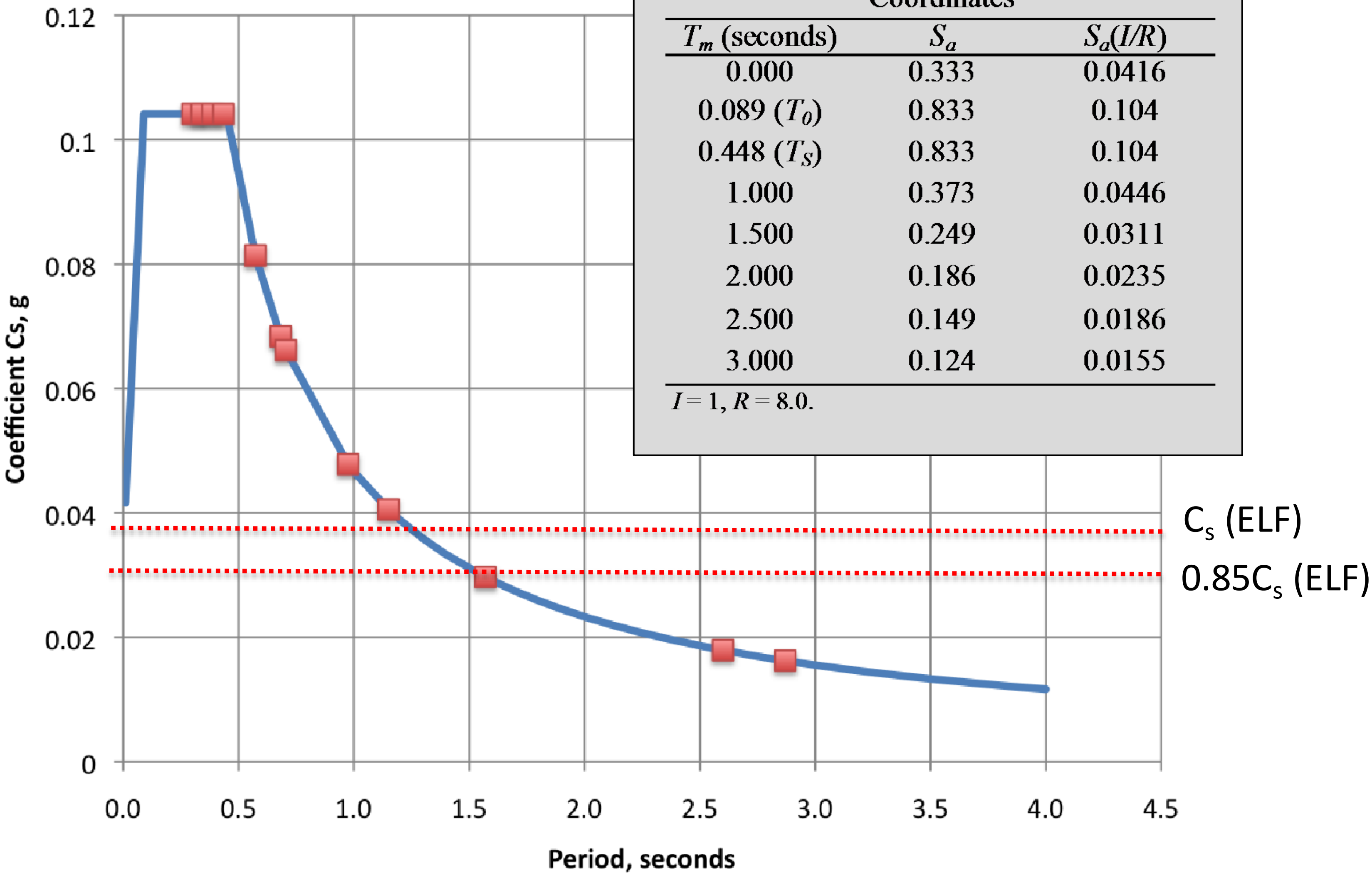


Table 4.1-15 Response Spectrum Coordinates

T_m (seconds)	S_a	$S_a(I/R)$
0.000	0.333	0.0416
0.089 (T_0)	0.833	0.104
0.448 (T_S)	0.833	0.104
1.000	0.373	0.0446
1.500	0.249	0.0311
2.000	0.186	0.0235
2.500	0.149	0.0186
3.000	0.124	0.0155

$I = 1$, $R = 8.0$.

Scaling of Response Spectrum Results (ASCE 7-05)

12.9.4 Scaling Design Values of Combined Response.

A base shear (V) shall be calculated in each of the two orthogonal horizontal directions using the calculated fundamental period of the structure T in each direction and the procedures of Section 12.8, except where the calculated fundamental period exceeds $(C_u)(T_a)$, then $(C_u)(T_a)$ shall be used in lieu of T in that direction. Where the combined response for the modal base shear (V_t) is less than 85 percent of the calculated base shear (V) using the equivalent lateral force procedure, the forces, but not the drifts, shall be multiplied by

$$0.85\frac{V}{V_t}$$

where

- V = the equivalent lateral force procedure base shear, calculated in accordance with this section and Section 12.8
- V_t = the base shear from the required modal combination

Note: If the ELF base shear is governed by Eqn. 12.5-5 or 12.8-6 the force V shall be based on the value of C_s calculated by Eqn. 12.5-5 or 12.8-6, as applicable.

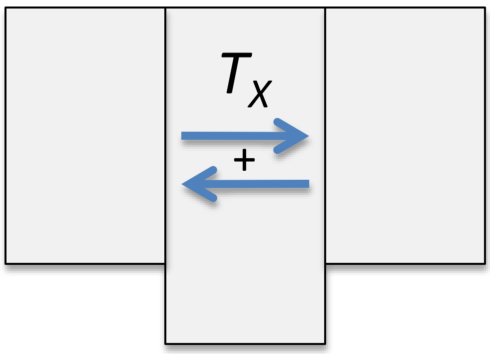
Scaling of Response Spectrum Results (ASCE 7-10)

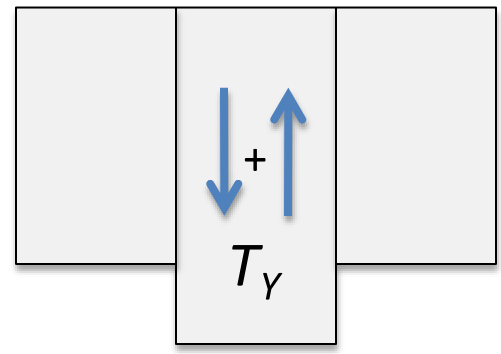
12.9.4.2 Scaling of Drifts

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

$$0.85\frac{C_s W}{V_t}$$

Scaled Static Torsions



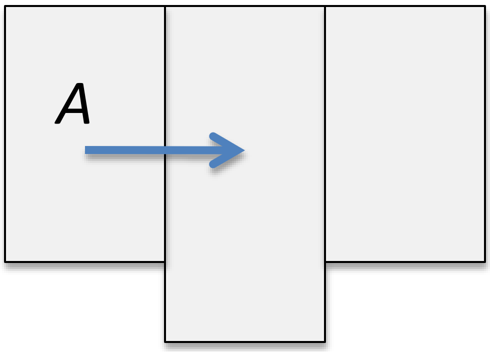


Apply Torsion as a Static Load. Torsions can be Scaled to 0.85 times Amplified* EFL Torsions if the Response Spectrum Results are Scaled.

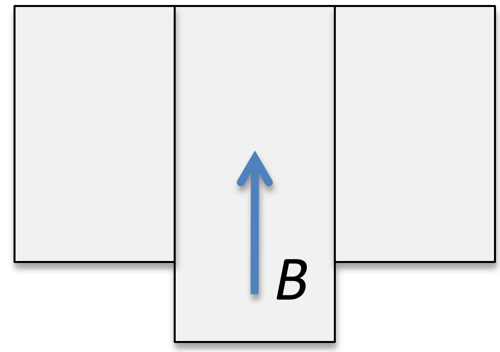
* See Sec. 12.9.5. Torsions must be amplified because they are applied statically, not dynamically.

Method 1: Weighted Addition of *Scaled* CQC'd Results

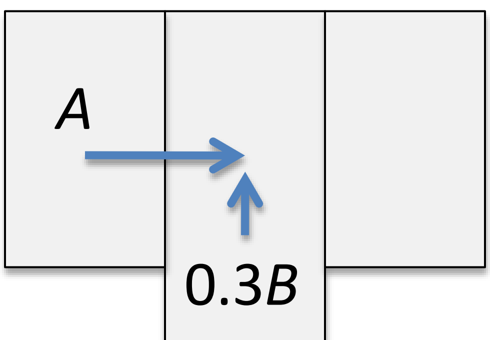
A = *Scaled* CQC'd Results in X Direction



B = *Scaled* CQC'd Results in Y Direction

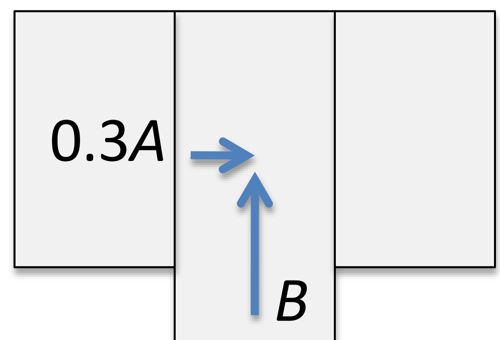


Combination 1



$$A + 0.3B + |T_X|$$

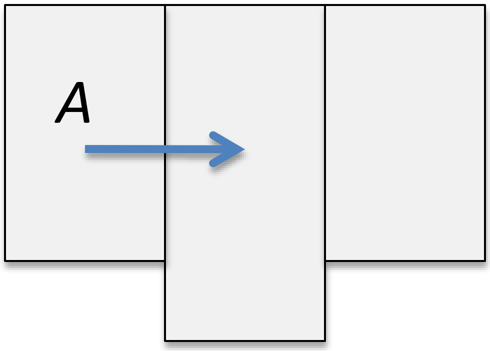
Combination 2



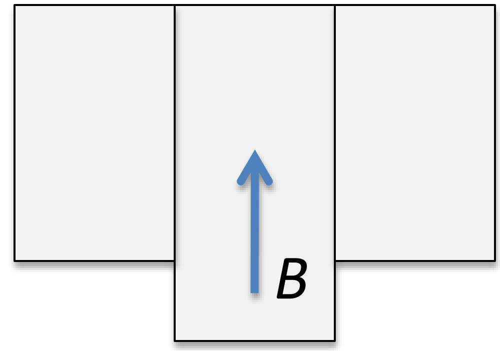
$$0.3A + B + |T_Y|$$

Method 2: SRSS of *Scaled* CQC'd Results

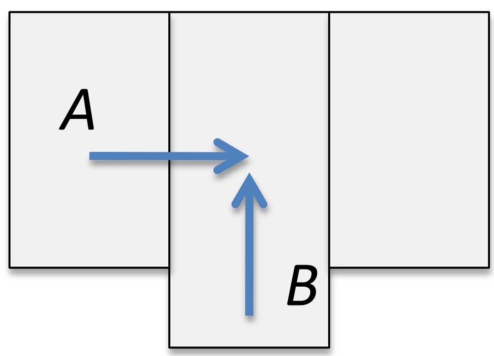
A = *Scaled* CQC'd Results in X Direction



B = *Scaled* CQC'd Results in Y Direction



Combination



$$(A^2+B^2)^{0.5} + \max(|T_X| \text{ or } |T_Y|)$$

Computed Story Shears and Scale Factors from Modal Response Spectrum Analysis

Table 4.1-16 Story Shears from Modal Response Spectrum Analysis

Story	X Direction (SF = 2.18)		Y Direction (SF = 1.94)	
	Unscaled Shear (kips)	Scaled Shear (kips)	Unscaled Shear (kips)	Scaled Shear (kips)
R-12	82.7	180	77.2	150
12-11	130.9	286	132.0	256
11-10	163.8	357	170.4	330
10-9	191.4	418	201.9	392
9-8	240.1	524	265.1	514
8-7	268.9	587	301.4	585
7-6	292.9	639	328.9	638
6-5	316.1	690	353.9	686
5-4	359.5	784	405.1	786
4-3	384.8	840	435.5	845
3-2	401.4	895	462.8	898
2-G	438.1	956	492.8	956

1.0 kip = 4.45 kN.

X-Direction Scale Factor = 0.85(1124)/438.1=2.18

Y-Direction Scale Factor = 0.85(1124)/492.8=1.94

Response Spectrum Drifts in X Direction

(No Scaling Required)

Level	Total Drift from R.S. Analysis (in.)	Story Drift (in.)	Story Drift × C_d (in.)	Allowable Story Drift (in.)
R	2.23	0.12	0.66	3.00
12	2.10	0.16	0.89	3.00
11	1.94	0.19	1.03	3.00
10	1.76	0.20	1.08	3.00
9	1.56	0.18	0.98	3.00
8	1.38	0.19	1.06	3.00
7	1.19	0.20	1.08	3.00
6	0.99	0.20	1.08	3.00
5	0.80	0.18	0.97	3.00
4	0.62	0.19	1.02	3.00
3	0.43	0.19	1.05	3.00
2	0.24	0.24	1.34	4.32

1.0 in. = 25.4 mm

Response Spectrum Drifts in Y Direction

(No Scaling Required)

Level	Total Drift from R.S. Analysis (in.)	Story Drift (in.)	Story Drift × C_d (in.)	Allowable Story Drift (in.)
R	1.81	0.06	0.32	3.00
12	1.76	0.09	0.49	3.00
11	1.67	0.11	0.58	3.00
10	1.56	0.12	0.67	3.00
9	1.44	0.13	0.70	3.00
8	1.31	0.16	0.87	3.00
7	1.15	0.17	0.91	3.00
6	0.99	0.17	0.92	3.00
5	0.92	0.17	0.93	3.00
4	0.65	0.19	1.04	3.00
3	0.46	0.20	1.08	3.00
2	0.26	0.26	1.44	4.32

1.0 in. = 25.4 mm

Scaled Beam Shears from Modal Response Spectrum Analysis

			8.41	8.72	8.91		
R-12			14.9	15.6	15.6		
12-11			21.5	21.6	22.5		
11-10			24.2	24.0	25.8		
10-9			23.3	23.3	21.8	20.0	8.9
9-8			23.7	23.5	22.4	24.5	15.8
8-7			26.9	26.1	25.4	26.7	17.2
7-6			28.4	26.8	26.2	27.3	17.8
6-5	10.1	22.4	23.6	25.3	24.8	25.5	17.0
5-4	17.4	26.6	23.7	24.9	24.6	25.1	17.0
4-3	18.5	27.5	25. 9	26.6	26.4	26.8	18.5
3-2	18.5	29.1	27.8	28.2	28.1	28.7	18.5
2 - G							

Overview of Presentation

- Describe Building
- Describe/Perform steps common to all analysis types
- Overview of Equivalent Lateral Force analysis
- Overview of Modal Response Spectrum Analysis
- **Overview of Modal Response History Analysis**
- Comparison of Results
- Summary and Conclusions

Modal Response History Analysis
Part 1: Analysis

1. Select suite of ground motions (Sec. 16.1.3.2)
2. Develop adequate finite element model (Sec. 12.7.3)
3. Compute modal frequencies, effective mass, and mode Shapes
4. Determine number of modes to use in analysis (Sec. 12.9.1)
5. Assign modal damping values (typically 5% critical per mode)
6. Scale ground motions* (Sec. 16.1.3.2)
7. Perform dynamic analysis for each ground motion in each direction
8. Compute Equivalent Lateral Forces (ELF) in each direction (Sec. 12.8.1 through 12.8.3)
9. Determine accidental torsions (Sec 12.8.4.2), amplified if necessary (Sec. 12.8.4.3)
10. Perform static torsion analysis

*Note: Step 6 is referred to herein as Ground Motion Scaling (GM Scaling). This is to avoid confusion with Results Scaling, described later.

Modal Response History Analysis Part 2: Drift and P-Delta for Systems *Without* Torsion Irregularity

1. Multiply all dynamic displacements by C_d/R (omitted in ASCE 7-05).
2. Compute story drifts based on displacements at center of mass at each level
3. If 3 to 6 ground motions are used, compute envelope of story drift at each level in each direction (Sec. 16.1.4)
4. If 7 or more ground motions are used, compute average story drift at each level in each direction (Sec. 16.1.4)
5. Check drift limits in accordance with Sec. 12.12 and Table 12.2-1. Note: drift limits for Special Moment Frames in SDC D and above must be divided by the Redundancy Factor (Sec. 12.12.1.1)
6. Perform P-Delta analysis using Equivalent Lateral Force procedure
7. Revise structure if necessary

Note: when centers of mass of adjacent levels are not vertically aligned the drifts should be based on the difference between the displacement at the upper level and the displacement of the point on the level below which is the vertical projection of the center of mass of the upper level.(This procedure is included in ASCE 7-10.)

Modal Response History Analysis Part 2: Drift and P-Delta for Systems *With* Torsion Irregularity

1. Multiply all dynamic displacements by C_d/R (omitted in ASCE 7-05).
2. Compute story drifts based on displacements at edge of building at each level
3. If 3 to 6 ground motions are used, compute envelope of story drift at each level in each direction (Sec. 16.1.4)
4. If 7 or more ground motions are used, compute average story drift at each level in each direction (Sec. 16.1.4)
5. Using results from the static torsion analysis, determine the drifts at the same location used in Steps 2-4 above. Torsional drifts may be based on the computed period of vibration (without the C_uT_a limit). Torsional drifts should be based on computed displacements multiplied by C_d and divided by I.
6. Add drifts from Steps (3 or 4) and 5 and check drift limits in Table 12.12-1. Note: Drift limits for special moment frames in SDC D and above must be divided by the Redundancy Factor (Sec. 12.12.1.1)
7. Perform P-Delta analysis using Equivalent Lateral Force procedure
8. Revise structure if necessary

Modal Response History Analysis
Part 3: Obtaining Member Design Forces

1. Multiply all dynamic member forces by *I*/*R*
2. Determine dynamic base shear histories for each earthquake in each direction
3. Determine Result Scale Factors* for each ground motion in each direction, and apply to response history results as appropriate
4. Determine design member forces by use of envelope values if 3 to 6 earthquakes are used, or as averages if 7 or more ground motions are used.
5. Combine results from two orthogonal directions, if necessary (Sec. 12.5)
6. Add member forces from static torsion analysis (Sec. 12.9.5). Note that static torsion forces may be scaled by factors obtained in Step 3
7. Determine redundancy factor (Sec. 12.3.4)
8. Combine seismic and gravity forces (Sec. 12.4)
9. Design and detail structural components

*Note: Step 3 is referred to herein as Results Scaling (GM Scaling). This is to avoid confusion with Ground Motion Scaling, described earlier.

Selection of Ground Motions for MRH Analysis

16.1.3.2 Three-Dimensional Analysis

Where three-dimensional analyses are performed, ground motions shall consist of pairs of appropriate horizontal ground motion acceleration components that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of recorded ground motion pairs is not available, appropriate simulated ground motion pairs are permitted to be used to make up the total number required.

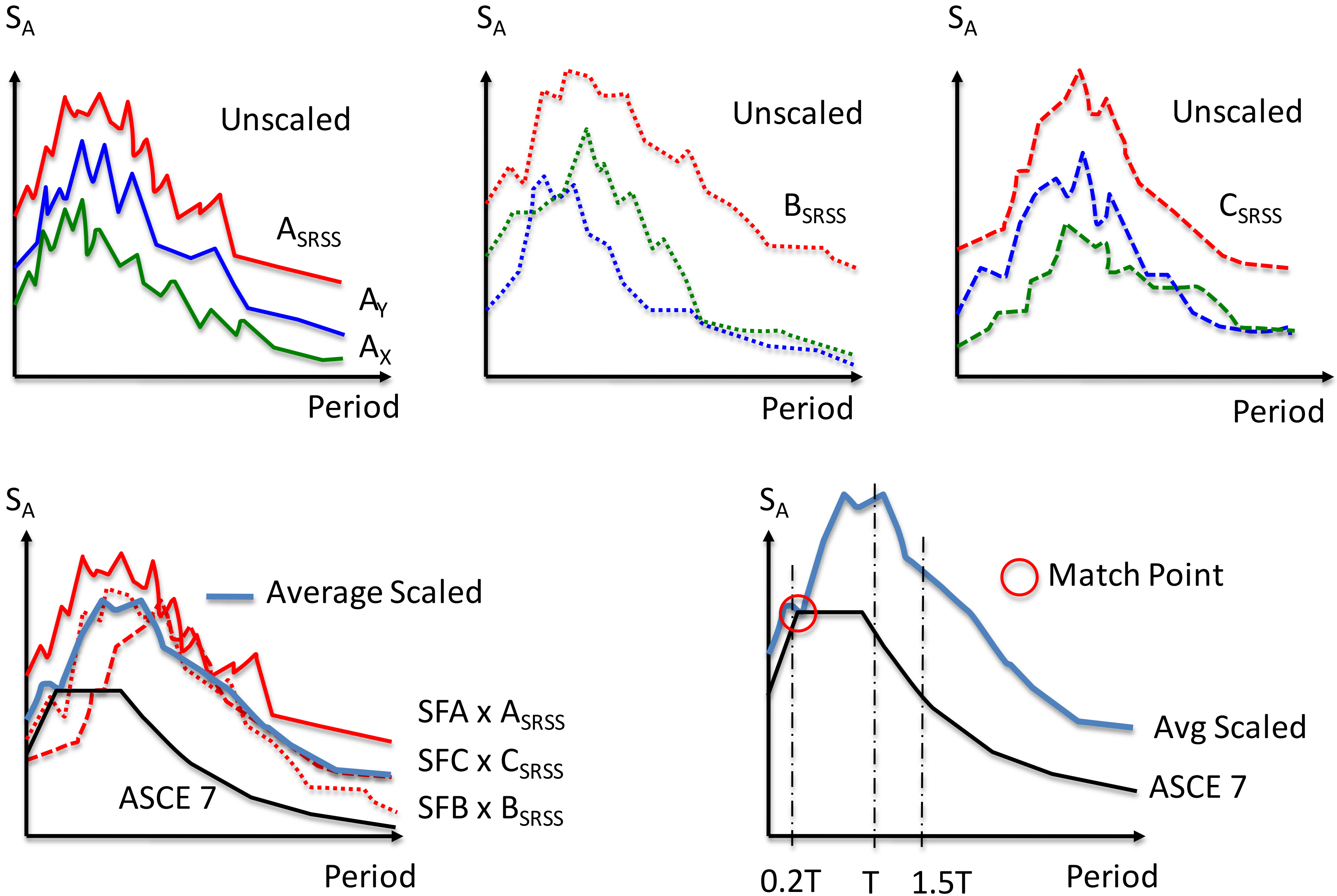
3D Scaling Requirements, ASCE 7-10

For each pair of horizontal ground motion components, a square root of the sum of the squares (SRSS) spectrum shall be constructed by taking the SRSS of the 5 percent-damped response spectra for the scaled components (where an identical scale factor is applied to both components of a pair). Each pair of motions shall be scaled such that in the period range from 0.2*T* to 1.5*T, the average of the SRSS spectra from all* horizontal component pairs does not fall below the corresponding ordinate of the response spectrum used in the design, determined in accordance with Section 11.4.5.

ASCE 7-05 Version:

does not fall below 1.3 times the corresponding ordinate of the design response spectrum, determined in accordance with Section 11.4.5 by more than 10 percent.

3D ASCE 7 Ground Motion Scaling



Issues With Scaling Approach

- No guidance is provided on how to deal with different fundamental
periods in the two orthogonal directions

- There are an infinite number of sets of scale factors that will satisfy the criteria. Different engineers are likely to obtain different sets of scale factors for the same ground motions.

- In linear analysis, there is little logic in scaling at periods greater than the structure's fundamental period.

- Higher modes, which participate marginally in the dynamic response, may dominate the scaling process

Resolving Issues With Scaling Approach

No guidance is provided on how to deal with different fundamental periods in the two orthogonal directions:

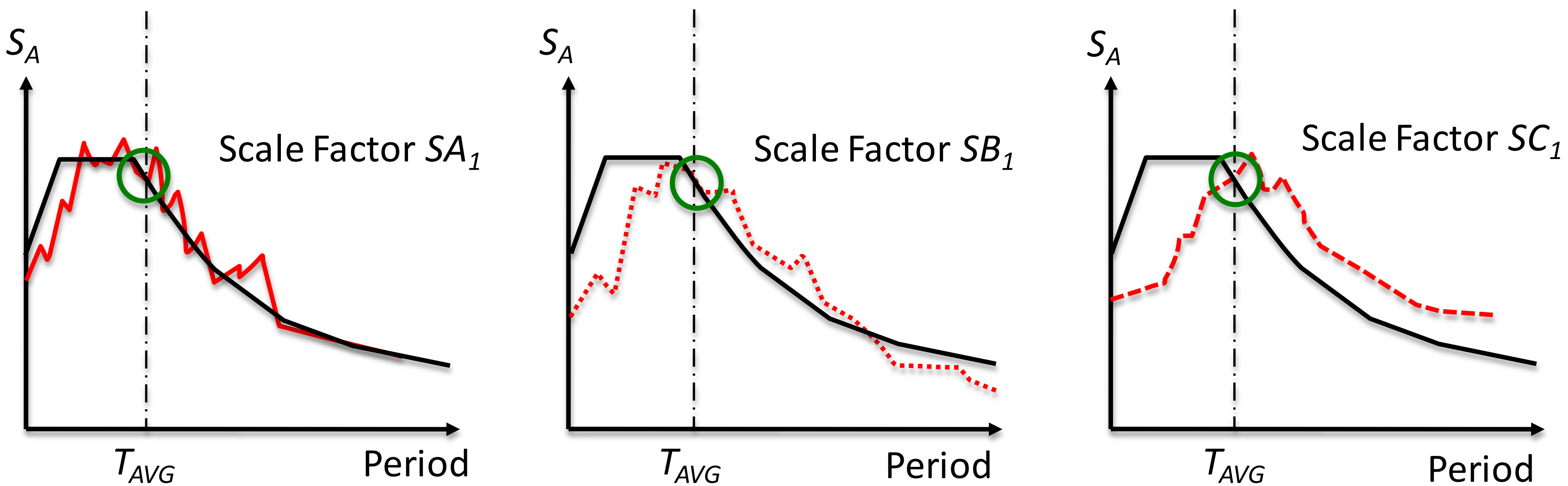
1. Use different periods in each direction (not recommended)
2. Scale to range 0.2 T_{min} to 1.5 T_{max} where T_{min} is the lesser of the two periods and T_{max} is the greater of the fundamental
   periods in each principal direction
3. Scale over the range 0.2T_{Avg} to 1.5 T_{Avg} where T_{Avg} is the average of T_{min} and T_{max}

Resolving Issues With Scaling Approach

There are an infinite number of sets of scale factors that will satisfy the criteria. Different engineers are likely to obtain different sets of scale factors for the same ground motions.

Use Two-Step Scaling:

1] Scale each SRSS'd Pair to the Average Period



Note: A different scale factor will be obtained for each SRSS'd pair

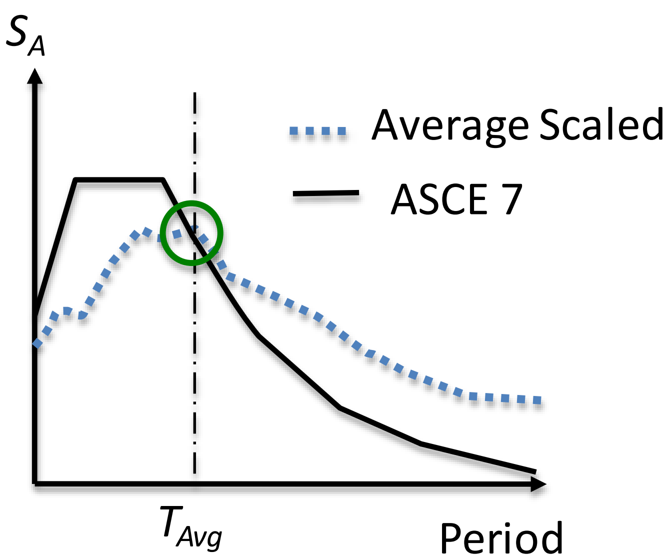
Resolving Issues With Scaling Approach

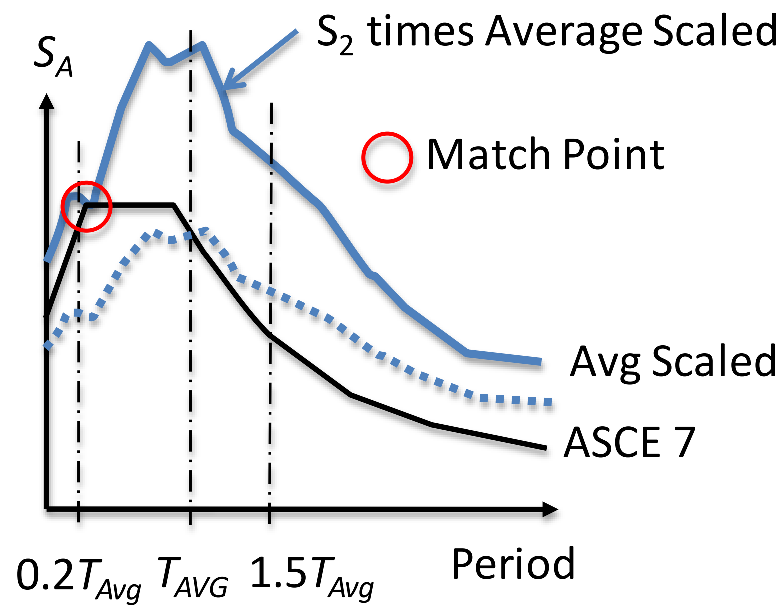
There are an infinite number of sets of scale factors that will satisfy the criteria. Different engineers are likely to obtain different sets of scale factors for the same ground motions.

Use Two-Step Scaling:

1] Scale each SRSS'd Pair to the Average Period

2] Obtain Suite Scale Factor S_2





Note: The same scale factor S_2 Applies to Each SRSS'd Pair

Resolving Issues With Scaling Approach

There are an infinite number of sets of scale factors that will satisfy the criteria. Different engineers are likely to obtain different sets of scale factors for the same ground motions.

Use Two-Step Scaling:

1] Scale each SRSS'd Pair to the Average Period

2] Obtain Suite Scale Factor S_2

3] Obtain Final Scale Factors:

Suite A: $SS_A=S_{A1} \times S_2$

Suite B: $SS_B=S_{B1} \times S_2$

Suite C: $SS_C=S_{C1} \times S_2$

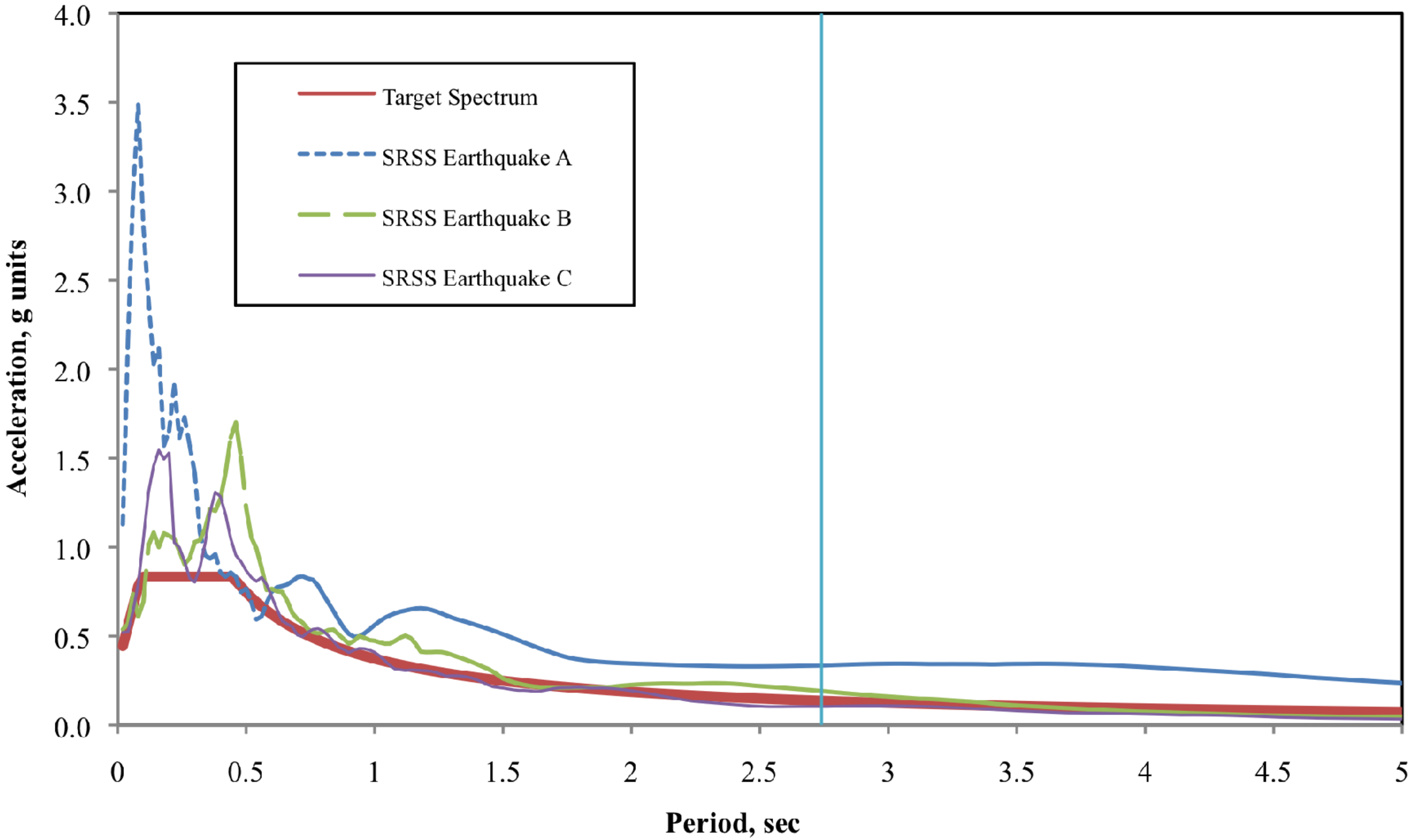
Ground Motions Used in Analysis

Table 4.1-20a. Suite of Ground Motions Used for Response History Analysis

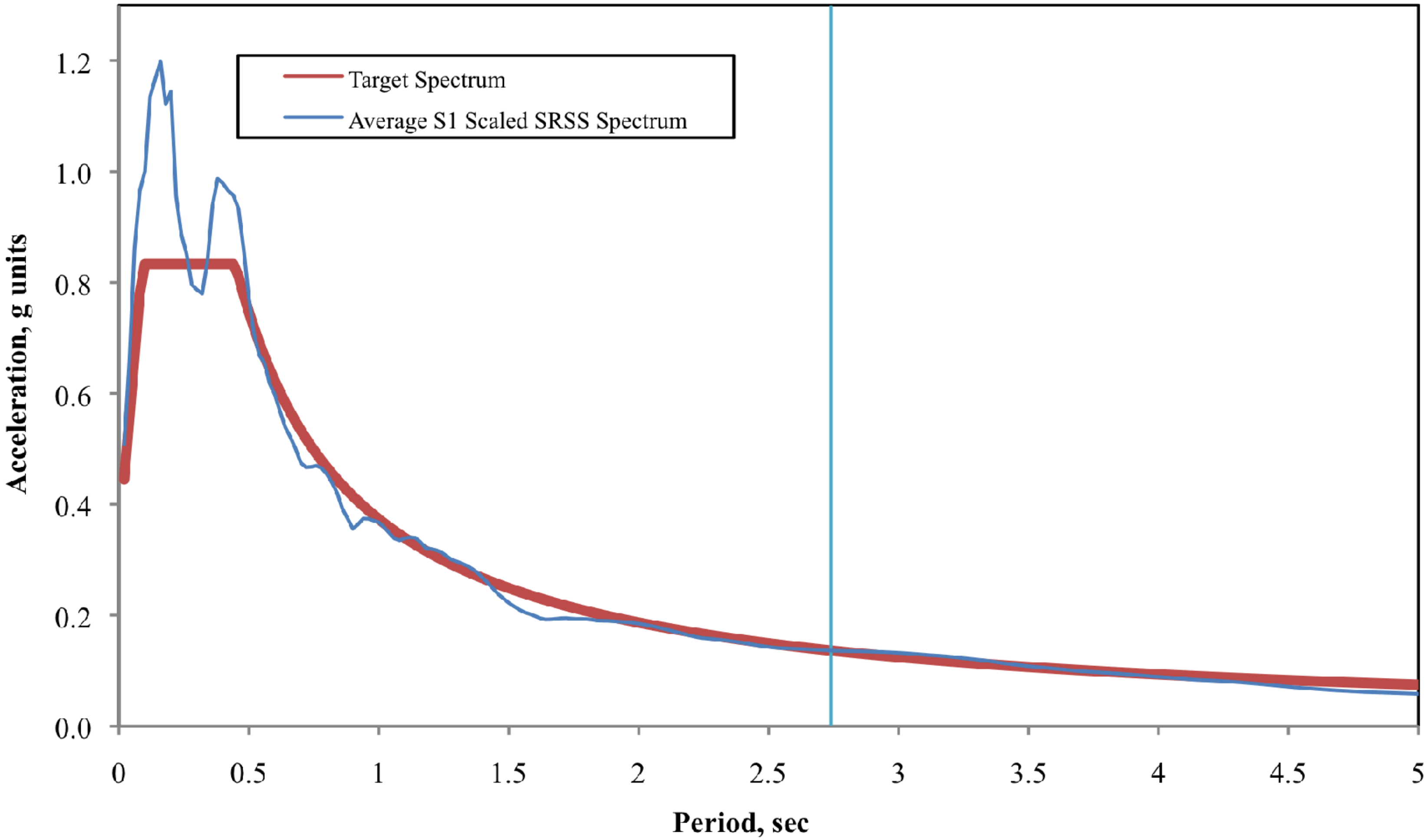
NGA Record Number	Magnitude [Epicenter Distance, km]	Site Class	Number of Points and Digitization Increment	Component Source Motion	PGA (g)	Record Name (This Example)
0879	7.28	C	9625 @ 0.005 sec	Landers/LCN260*	0.727	A00
	[44]			Landers/LCN345*	0.789	A90
0725	6.54	D	2230 @ 0.01 sec	SUPERST/B-POE270	0.446	B00
	[11.2]			SUPERST/B-POE360	0.300	B90
0139	7.35	C	1192 @ 0.02 sec	TABAS/DAY-LN	0.328	C00
	[21]			TABAS/DAY-TR	0.406	C90

* Note that the two components of motion for the Landers earthquake are apparently separated by an 85 degree angle, not 90 degrees as is traditional. It is not known whether these are true orientations, or of there is an error in the descriptions provided in the NGA database.

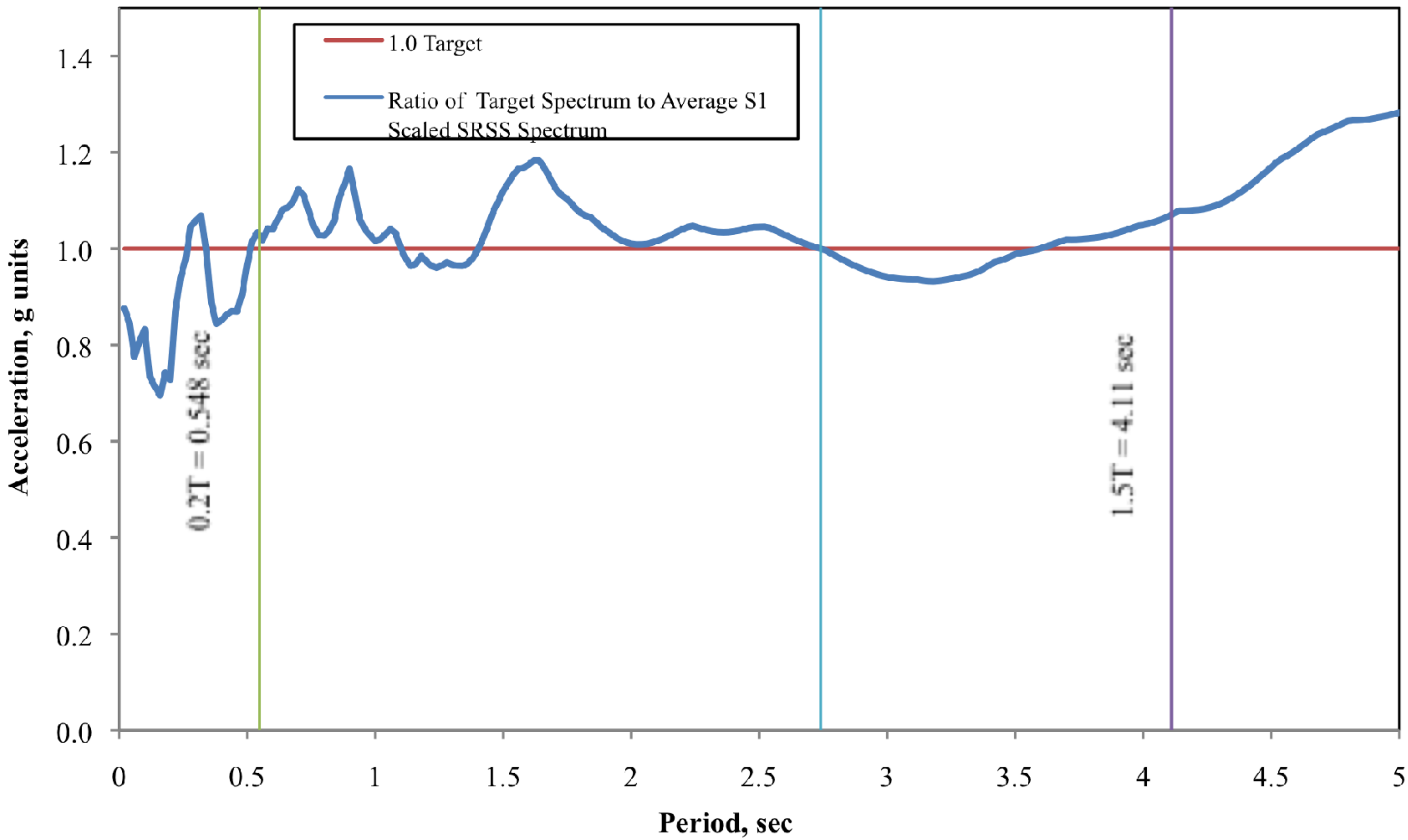
Unscaled Spectra



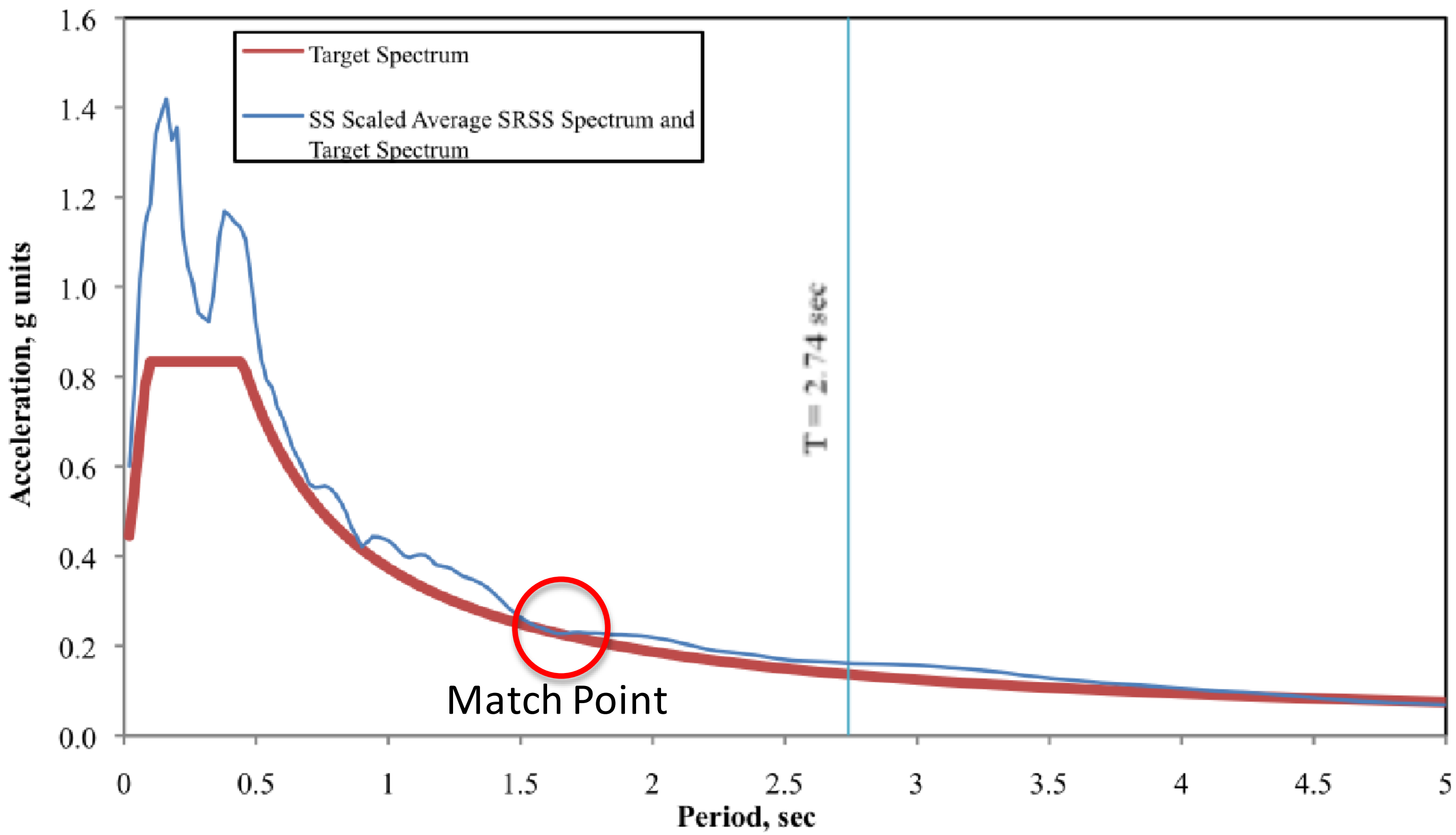
Average S1 Scaled Spectra



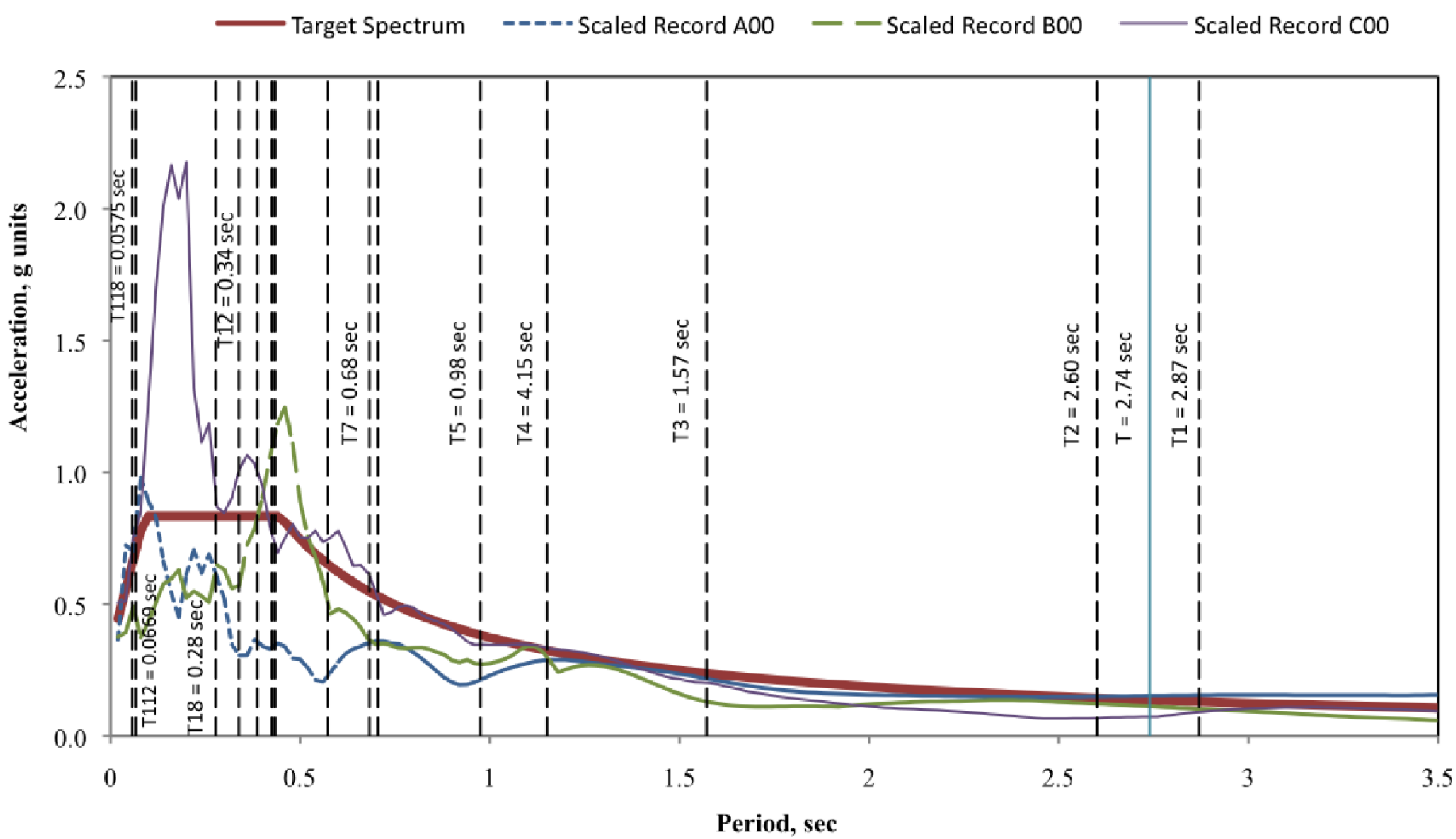
Ratio of Target Spectrum to Scaled SRSS Average



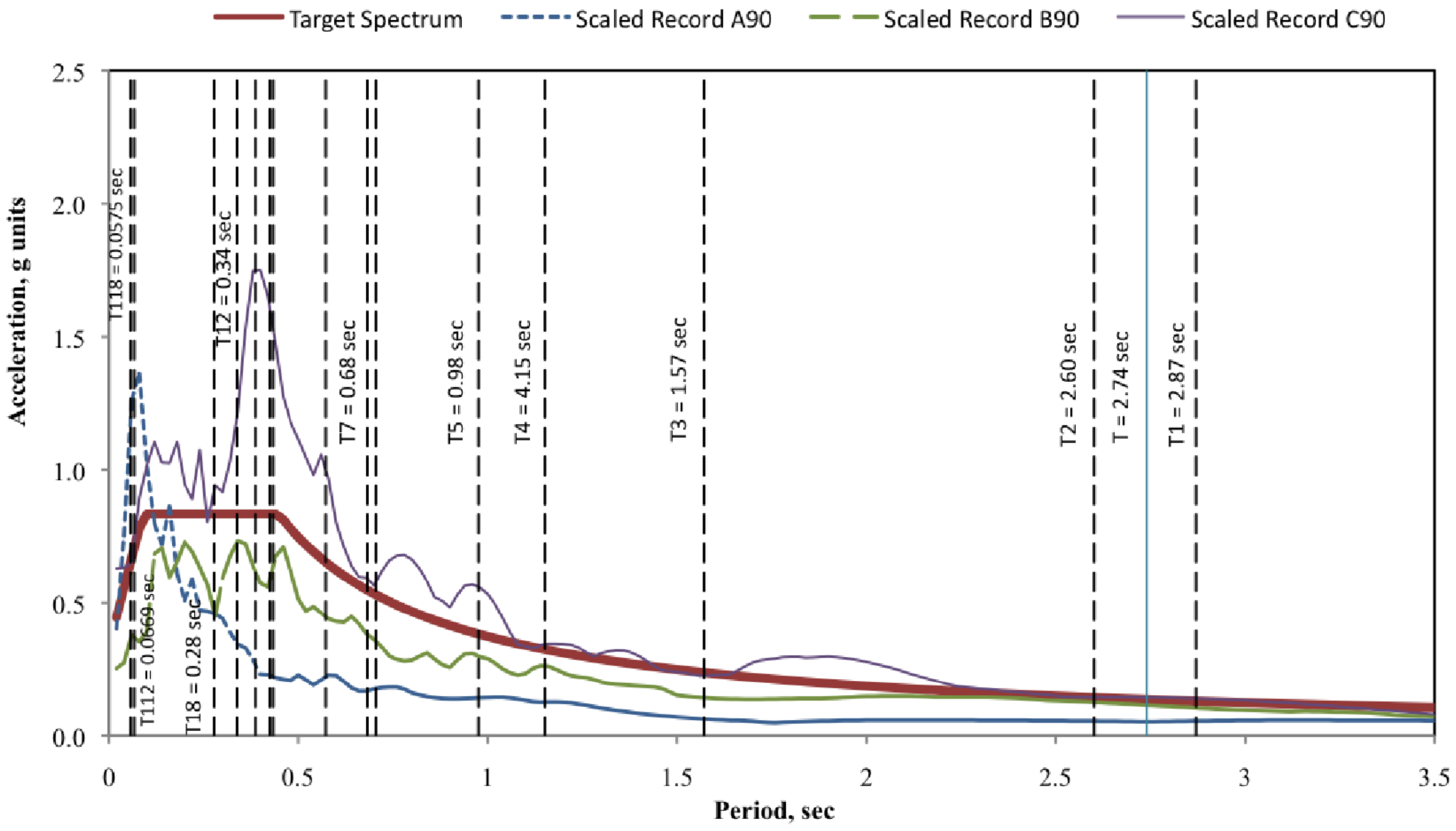
Target Spectrum and SS Scaled Average



Individual Scaled Components (00)



Individual Scaled Components (90)



Computed Scale Factors

Table 4.1-20b. Result of 3D Scaling Process

Set No.	Designation	SRSS ordinate at $T=T_{Avg}$ (g)	Target Ordinate at $T=T_{Avg}$ (g)	S1	S2	SS
1	A00 & A90	0.335	0.136	0.407	1.184	0.482
2	B00 & B90	0.191	0.136	0.712	1.184	0.843
3	C00 & C90	0.104	0.136	1.310	1.184	1.551

Number of Modes for Modal Response History Analysis

ASCE 7-05 and 7-10 are silent on the number of modes to use in Modal Response History Analysis. It is recommended that the same procedures set forth in Section 12.9.1 for MODAL Response Spectrum Analysis be used for Response History Analysis:

12.9.1 Number of Modes

An analysis shall be conducted to determine the natural modes of vibration for the structure. **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.

Damping for Modal Response History Analysis

ASCE 7-05 and 7-10 are silent on the amount of damping to use in Modal Response History Analysis.

Five percent critical damping should be used in all modes considered in the analysis because the Target Spectrum and the Ground Motion Scaling Procedures are based on 5% critical damping.

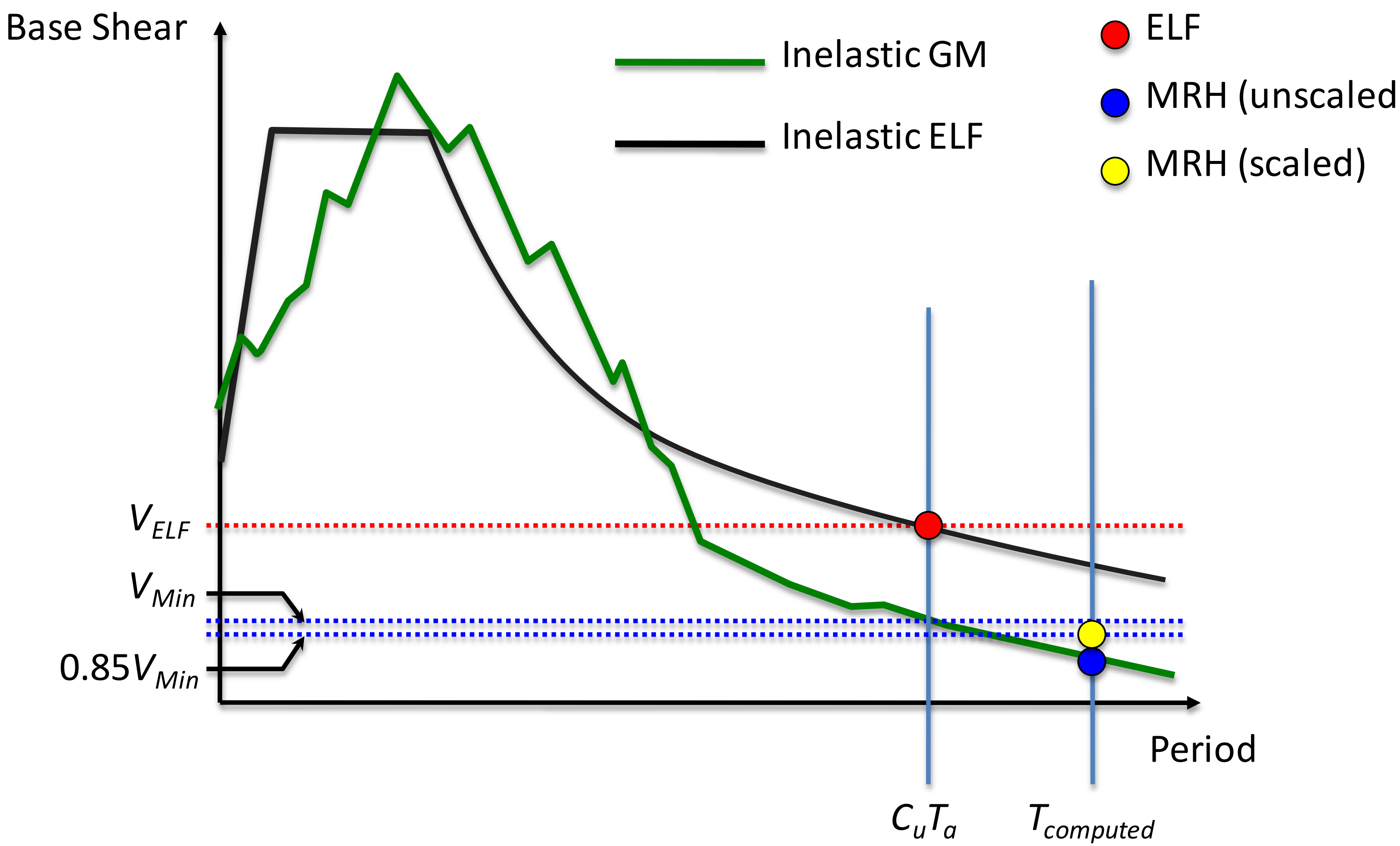
Scaling of Results for Modal Response History Analysis (Part 1)

The structural analysis is executed using the GM scaled earthquake records in each direction. Thus, the results represent the expected elastic response of the structure. The results must be scaled to represent the expected inelastic behavior and to provide improved performance for important structures. ASCE 7-05 scaling is as follows:

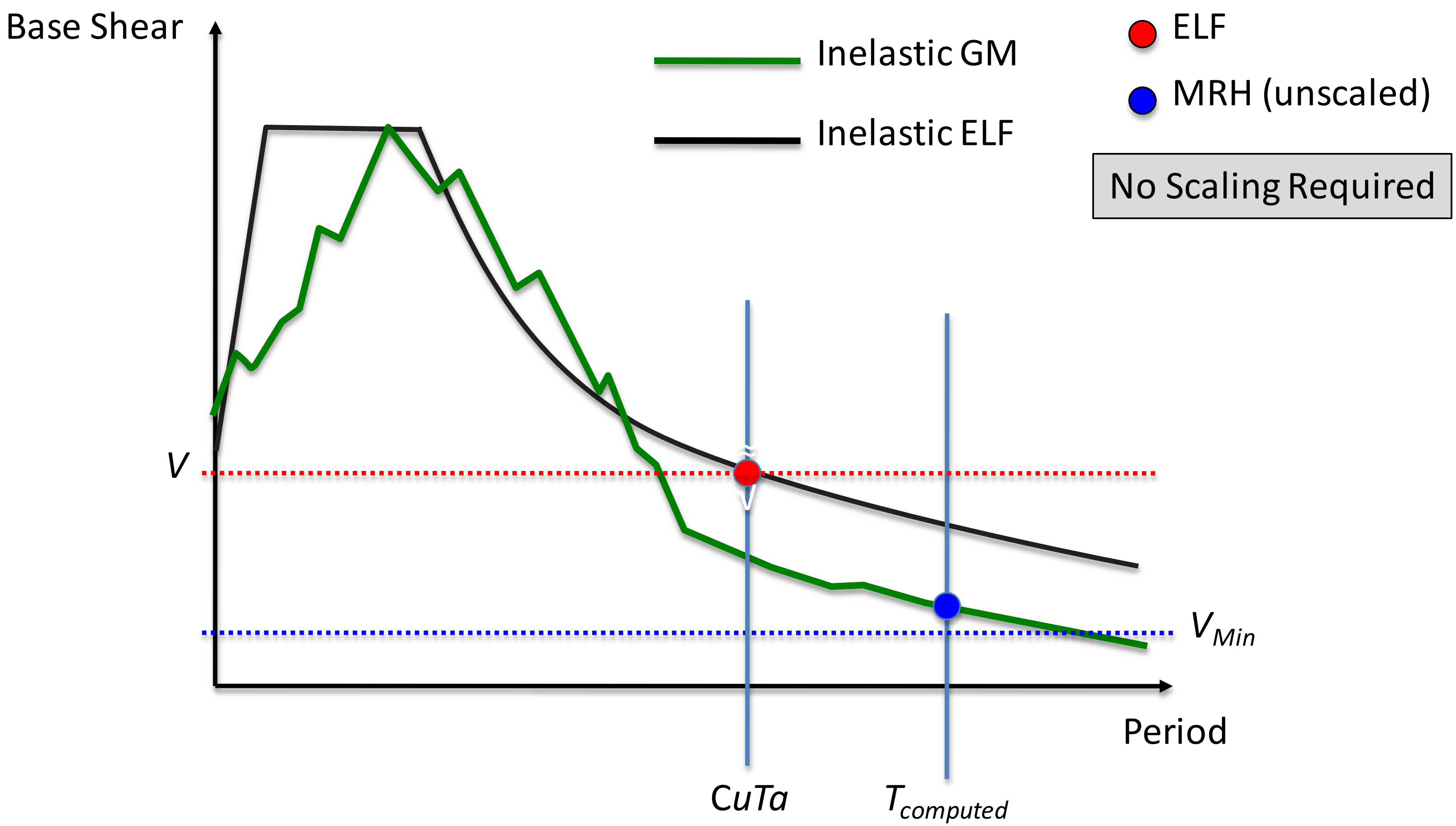
1) Scale all component design forces by the factor (I/R). This is stipulated in Sec. 16.1.4 of ASCE 7-05 and ASCE 7-10.

2) Scale all displacement quantities by the factor (C_d/R). This requirement
was inadvertently omitted in ASCE 7-05, but is included in Section 16.1.4 of ASCE 7-10.

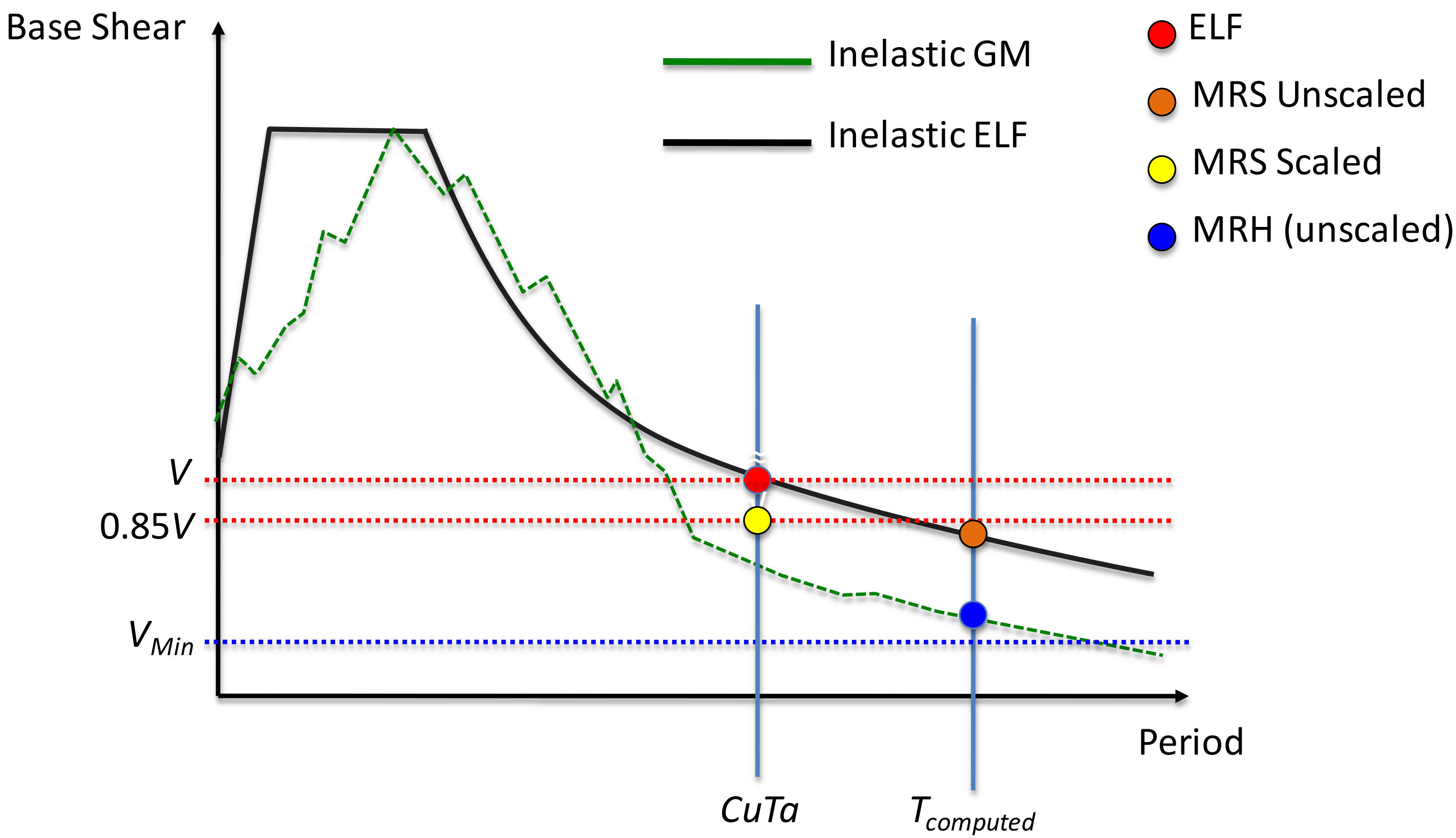
Response Scaling Requirements when MRH Shear is *Less Than* Minimum Base Shear



Response Scaling Requirements when MRH Shear is *Greater Than* Minimum Base Shear



Response Scaling Requirements when MRH Shear is *Greater Than* Minimum Base Shear



12 Individual Response History Analyses Required

1. A00-X: SS Scaled Component A00 applied in X Direction
2. A00-Y: SS Scaled Component A00 applied in Y Direction
3. A90-X: SS Scaled Component A90 applied in X Direction
4. A90-Y: SS Scaled Component A90 applied in Y Direction

5. B00-X: SS Scaled Component B00 applied in X Direction
6. B00-Y: SS Scaled Component B00 applied in Y Direction
7. B90-X: SS Scaled Component B90 applied in X Direction
8. B90-Y: SS Scaled Component B90 applied in Y Direction

9. C00-X: SS Scaled Component C00 applied in X Direction
10. C00-Y: SS Scaled Component C00 applied in Y Direction
11. C90-X: SS Scaled Component C90 applied in X Direction
12. C90-Y: SS Scaled Component C90 applied in Y Direction

Result Maxima from Response History Analysis Using SS Scaled Ground Motions

	Analysis	Maximum base shear (kips)	Time of maximum shear (sec.)	Maximum roof displacement (in.)	Time of maximum displacement (sec.)
	A00-X	3507	11.29	20.28	11.38
	A00-Y	3573	11.27	14.25	11.28
	A90-X	1588	12.22	7.32	12.70
Low >	A90-Y	1392	13.56	5.16	10.80
	B00-X	3009	8.28	12.85	9.39
	B00-Y	3130	9.37	11.20	10.49
	B90-X	2919	8.85	11.99	7.11
	B90-Y	3460	7.06	11.12	8.20
	C00-X	3130	13.5	9.77	13.54
	C00-Y	2407	4.64	6.76	8.58
	C90-X	3229	6.92	15.61	6.98
High >	C90-Y	5075	6.88	14.31	7.80

1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

I/R Scaled Shears and Required 85% Rule Scale Factors

Analysis	(I/R) times maximum base shear from analysis (kips)	Required additional scale factor for $V = 0.85 V_{ELF} = 956$ kips
A00-X	438.4	2.18
A00-Y	446.7	2.14
A90-X	198.5	4.81
A90-Y	173.9	5.49
B00-X	376.1	2.54
B00-Y	391.2	2.44
B90-X	364.8	2.62
B90-Y	432.5	2.21
C00-X	391.2	2.44
C00-Y	300.9	3.18
C90-X	403.6	2.37
C90-Y	634.4	1.51

1.0 kip = 4.45 kN

Response History Drifts for all X-Direction Responses

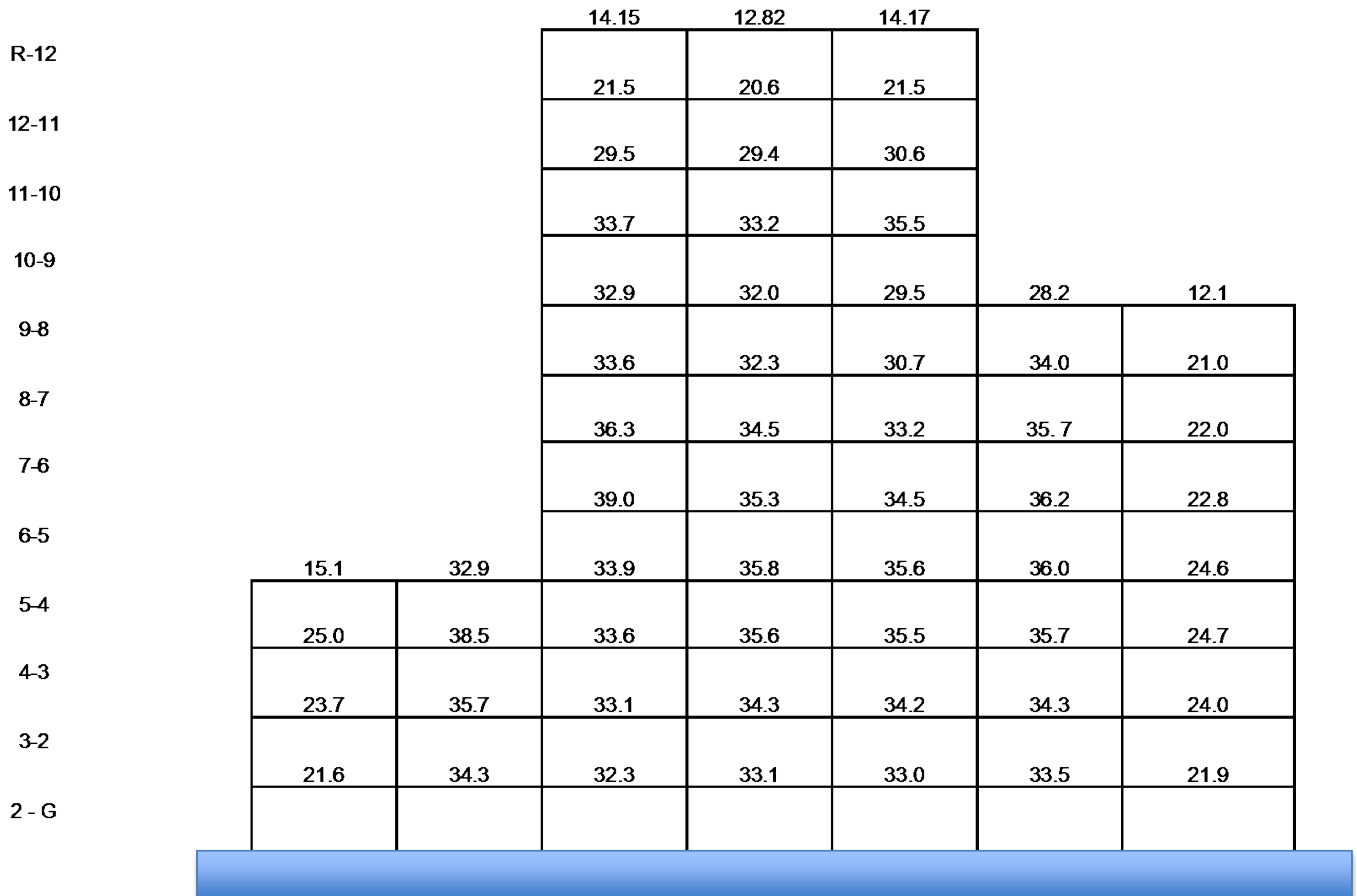
Level	Envelope of drift (in.) for each ground motion A00-X	A90-X	B00-X	B90-X	C00-X	C90-X	Envelope of drift for all the ground motions	Envelope of drift × C_d/R	Allowable drift (in.)
R	1.17	0.49	0.95	0.81	0.91	1.23	1.23	0.85	3.00
12	1.64	0.66	1.22	0.95	1.16	1.27	1.64	1.13	3.00
11	1.97	0.78	1.32	0.99	1.25	1.52	1.97	1.35	3.00
10	2.05	0.86	1.42	1.04	1.20	1.68	2.05	1.41	3.00
9	1.79	0.82	1.26	1.25	0.99	1.41	1.79	1.23	3.00
8	1.83	0.87	1.22	1.42	1.23	1.50	1.83	1.26	3.00
7	1.82	0.83	1.27	1.36	1.21	1.67	1.82	1.25	3.00
6	1.77	0.74	1.36	1.35	1.06	1.94	1.94	1.33	3.00
5	1.50	0.59	1.19	1.21	1.09	1.81	1.81	1.24	3.00
4	1.55	0.62	1.22	1.32	1.23	1.76	1.76	1.21	3.00
3	1.56	0.64	1.24	1.30	1.33	1.60	1.60	1.10	3.00
2	1.97	0.86	1.64	1.58	1.73	1.85	1.97	1.35	4.32

1.0 in. = 25.4 mm.

Load Combinations for Response History Analysis

Load Combination for Response History Analysis					
Earthquake	Load Combination	Loading X Direction		Loading Y Direction	
		Record	Scale Factor	Record	Scale Factor
A	1	A00-X	2.18	A00-Y	5.49
	2	A90-X	-4.81	A90-Y	2.14
	3	A00-X	-2.18	A00-Y	-5.49
	4	A90-X	4.81	A90-Y	-2.14
B	5	B00-X	2.54	B00-Y	2.21
	6	B90-X	-2.62	B90-Y	2.44
	7	B00-X	-2.54	B00-Y	-2.21
	8	B90-X	2.62	B90-Y	-2.44
C	9	C00-X	2.44	C00-y	1.50
	10	C90-X	-2.36	C90-Y	3.18
	11	C00-X	-2.44	C00-Y	-1.50
	12	C90-X	2.36	C90-Y	-3.18

Envelope of Scaled Frame 1 Beam Shears from Response History Analysis



Overview of Presentation

- Describe Building
- Describe/Perform steps common to all analysis types
- Overview of Equivalent Lateral Force analysis
- Overview of Modal Response Spectrum Analysis
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- **Comparison of Results**
- Summary and Conclusions

Comparison of Maximum X-Direction Design Story Shears from All Analysis

Level	ELF	Modal response spectrum	Enveloped response history
R	187	180	295
12	341	286	349
11	471	357	462
10	578	418	537
9	765	524	672
8	866	587	741
7	943	639	753
6	999	690	943
5	1,070	784	1,135
4	1,102	840	1,099
3	1,118	895	1,008
2	1,124	956	956

Comparison of Maximum X-Direction Design Story Drift from All Analysis

Level	X Direction Drift (in.)		
	ELF	Modal response spectrum	Enveloped response history
R	0.99	0.66	0.85
12	1.41	0.89	1.13
11	1.75	1.03	1.35
10	1.92	1.08	1.41
9	1.82	0.98	1.23
8	1.97	1.06	1.26
7	2.01	1.08	1.25
6	1.97	1.08	1.33
5	1.67	0.97	1.24
4	1.69	1.02	1.21
3	1.65	1.05	1.10
2	2.00	1.34	1.35

1.0 in. = 25.4 mm.

Comparison of Maximum Beam Shears from All Analysis

Level	Beam Shear Force in Bay D-E of Frame 1 (kips)		
	ELF	Modal response spectrum	Enveloped response history
R	10.27	8.72	12.82
12	18.91	15.61	20.61
11	28.12	21.61	29.45
10	33.15	24.02	33.22
9	34.69	23.32	32.02
8	35.92	23.47	32.30
7	40.10	26.15	34.53
6	40.58	26.76	35.29
5	36.52	25.29	35.82
4	34.58	24.93	35.65
3	35.08	26.60	34.27
2	35.28	28.25	33.07

1.0 kip = 4.45 kN.

Overview of Presentation

- Describe Building
- Describe/Perform steps common to all analysis types
- Overview of Equivalent Lateral Force analysis
- Overview of Modal Response Spectrum Analysis
- Overview of Modal Response History Analysis
- Comparison of Results
- **Summary and Conclusions**

Required Effort

- The Equivalent Lateral Force method and the Modal Response Spectrum methods require similar levels of effort.

- The Modal Response History Method requires considerably more effort than ELF or MRS. This is primarily due to the need to select and scale the ground motions, and to run so many response history analyses.

Accuracy

It is difficult to say whether one method of analysis is "more accurate" than the others. This is because each of the methods assume linear elastic behavior, and make simple adjustments (using R and C_d) to account for inelastic behavior.

Differences inherent in the results produced by the different methods are reduced when the results are scaled. However, it is likely that the Modal Response Spectrum and Modal Response History methods are generally more accurate than ELF because they more properly account for higher mode response.

Recommendations for Future Considerations

1. Three dimensional analysis should be required for *all* Response Spectrum and Response History analysis.

2. Linear Response History Analysis should be moved from Chapter 16 into Chapter 12 and be made as consistent as possible with the Modal Response Spectrum Method. For example, requirements for the number of modes and for scaling of results should be the same for the two methods.

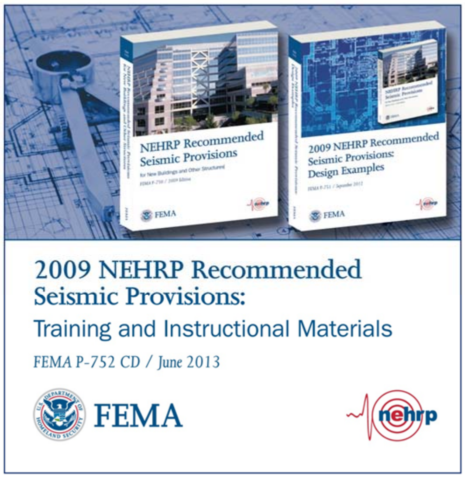
3. A rational procedure needs to be developed for directly including Accidental Torsion in Response Spectrum and Response History Analysis.

4. A rational method needs to be developed for directly including P-Delta effects in Response Spectrum and Response History Analysis.

5. The current methods of selecting and scaling ground motions for linear response history analysis can be and should be much simpler than required for nonlinear response history analysis. The use of "standardized" motion sets or the use of spectrum matched ground motions should be considered.

6. Drift should always be computed and checked at the corners of the building.

Questions



4

Structural Analysis

Finley Charney, Adrian Tola Tola, and Ozgur Atlayan

Structural Analysis: Example 1
Twelve-story Moment Resisting Steel Frame

Title slide

This example demonstrates three linear elastic analysis procedures provided by ASCE 7-05: Equivalent Lateral Force analysis (ELF), Modal Response Spectrum Analysis (MRS), and Modal Response History Analysis. The building is a structural steel system with various geometric irregularities. The building is located in Stockton, California, an area of relatively high seismic activity.

The example is based on the requirements of ASCE 7-05. However, ASCE 7-10 is referred to in several instances.

Complete details for the analysis are provided in the written example, and the example should be used as the “Instructors Guide” when presenting this slide set. Many, but not all of the slides in this set have “Speakers Notes”, and these are intentionally kept very brief.

Finley Charney is a Professor of Civil Engineering at Virginia Tech, Blacksburg, Virginia. He is also president of Advanced Structural Concepts, Inc., located in Blacksburg. The written example and the accompanying slide set were completed by Advanced Structural Concepts. Adrian Tola was a graduate student at Virginia Tech when the example was developed, and served as a contractor for Advanced Structural Concepts.

Building Description

- 12 Stories above grade, one level below grade
- Significant Configuration Irregularities
- Special Steel Moment Resisting Perimeter Frame
- Intended Use is Office Building
- Situated on Site Class C Soils

This building was developed specifically for this example. However, an attempt was made to develop a realistic structural system, with a realistic architectural configuration.

Analysis Description

- Equivalent Lateral Force Analysis (Section 12.8)
- Modal Response Spectrum Analysis (Section 12.9)
- Linear and Nonlinear Response History Analysis (Chapter 16)

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These are the three linear analysis methods provided in ASCE 7.

The Equivalent Lateral Force method (ELF) is essentially a one-mode response spectrum analysis with corrections for higher mode effects. This method is allowed for all SDC B and C buildings, and for the vast majority of SDC D, E and F buildings. Note that some form of ELF will be required during the analysis/design process for all buildings.

The Modal Response Spectrum (MRS) method is somewhat more complicated than ELF because mode shapes and frequencies need to be computed, response signs (positive or negative) are lost, and results must be scaled. However, there are generally fewer load combinations than required by ELF. MRS can be used for any building, and is required for SDC D, E, and F buildings with certain irregularities, and for SDC D, E, and F buildings with long periods of vibration.

The linear Modal Response History (MRH) method is more complex that MRS, mainly due to the need to select and scale at least three and preferably seven sets of motions. MRS can be used for any building, but given the current code language, it is probably too time-consuming for the vast majority of systems.

Overview of Presentation

- Describe Building
- Describe/Perform steps common to all analysis types
- Overview of Equivalent Lateral Force analysis
- Overview of Modal Response Spectrum Analysis
- Overview of Modal Response History Analysis
- Comparison of Results
- Summary and Conclusions

Note: The majority of presentation is based on requirements provided by ASCE 7-05. ASCE 7-10 and the 2009 NEHRP Provisions (FEMA P750) will be referred to as applicable.

The vast majority of the written example and this slide set is based on the requirements of ASCE 7-05. The requirements of ASCE 7-10 are mentioned when necessary. When ASCE 7-10 is mentioned, it is generally done so to point out the differences in ASCE 7-05 and ASCE 7-10.

Overview of Presentation

- **Describe Building**
- Describe/Perform steps common to all analysis types
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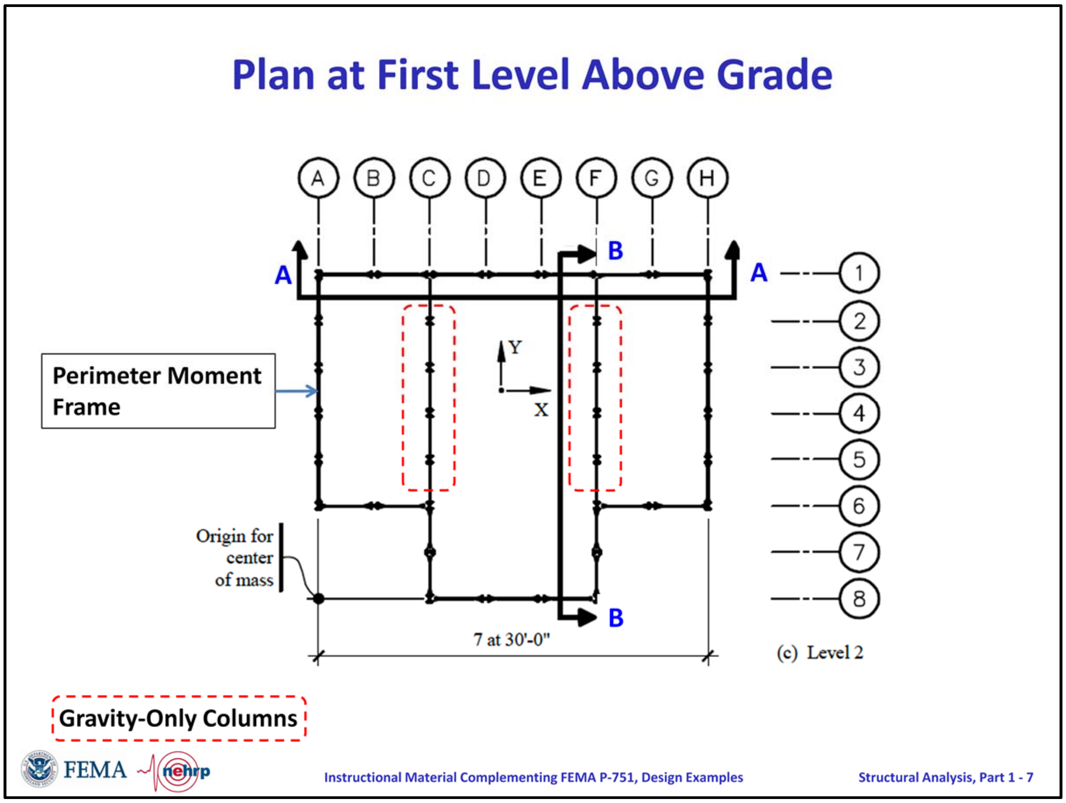
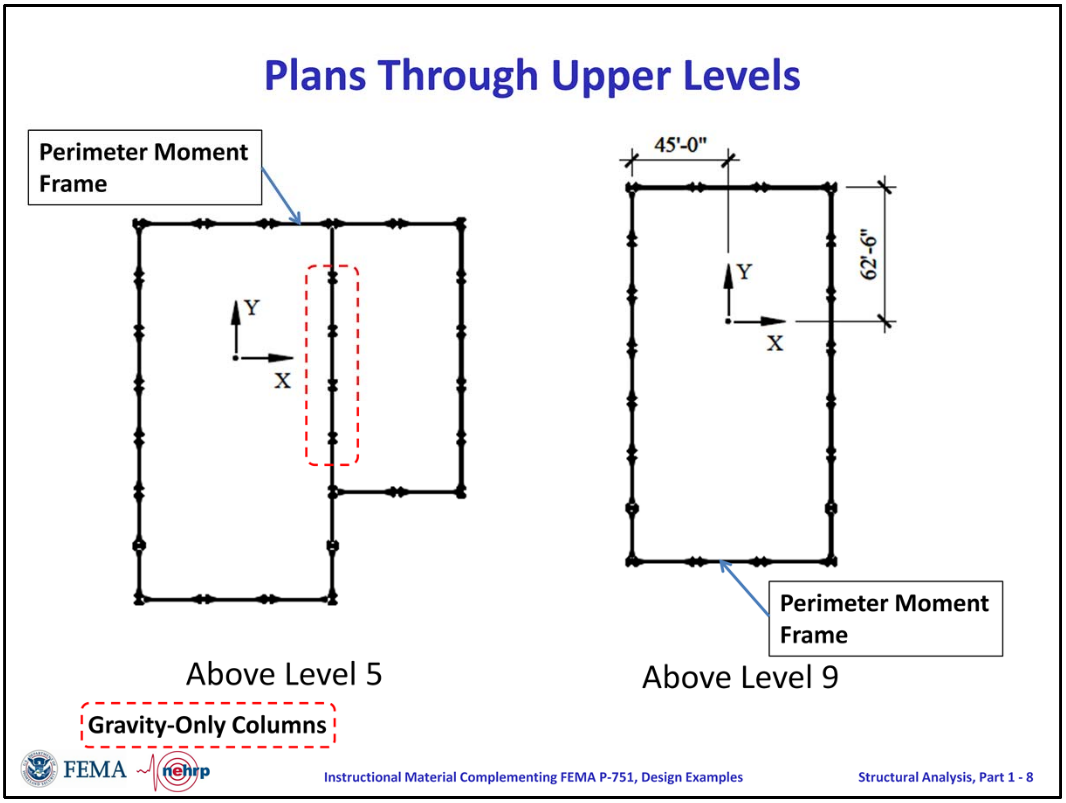
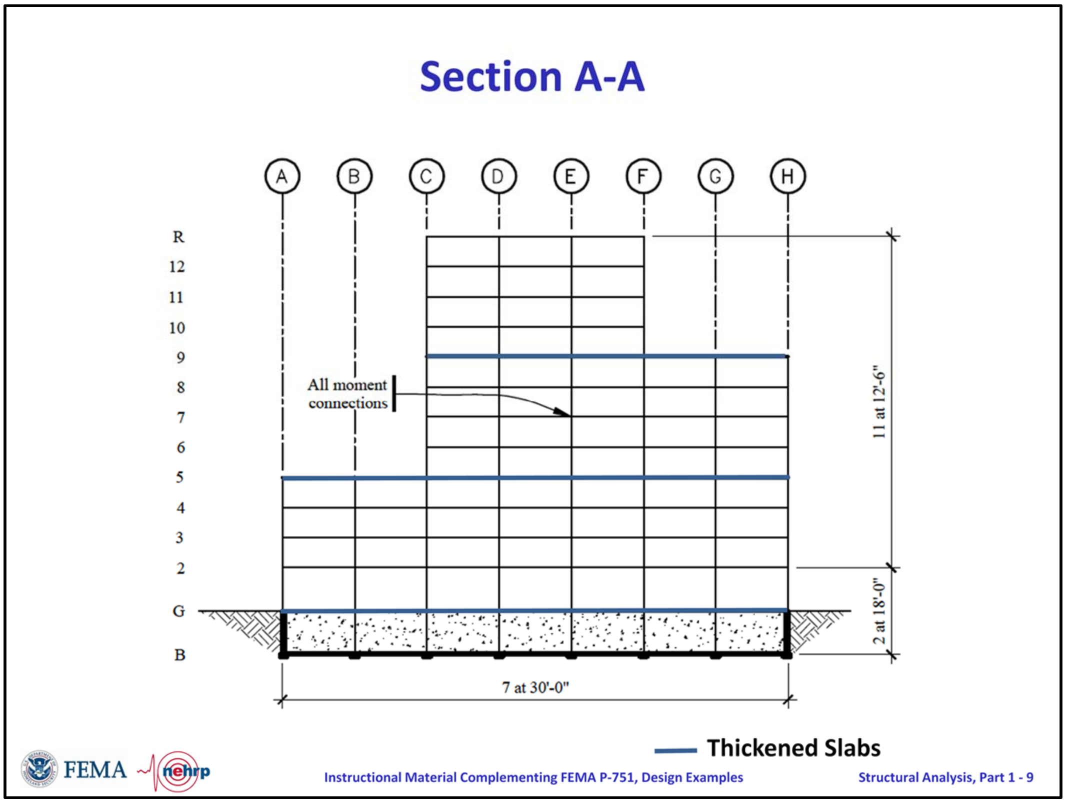
The structure analyzed is a 3-Dimensional Special Steel Moment resisting Space Frame.



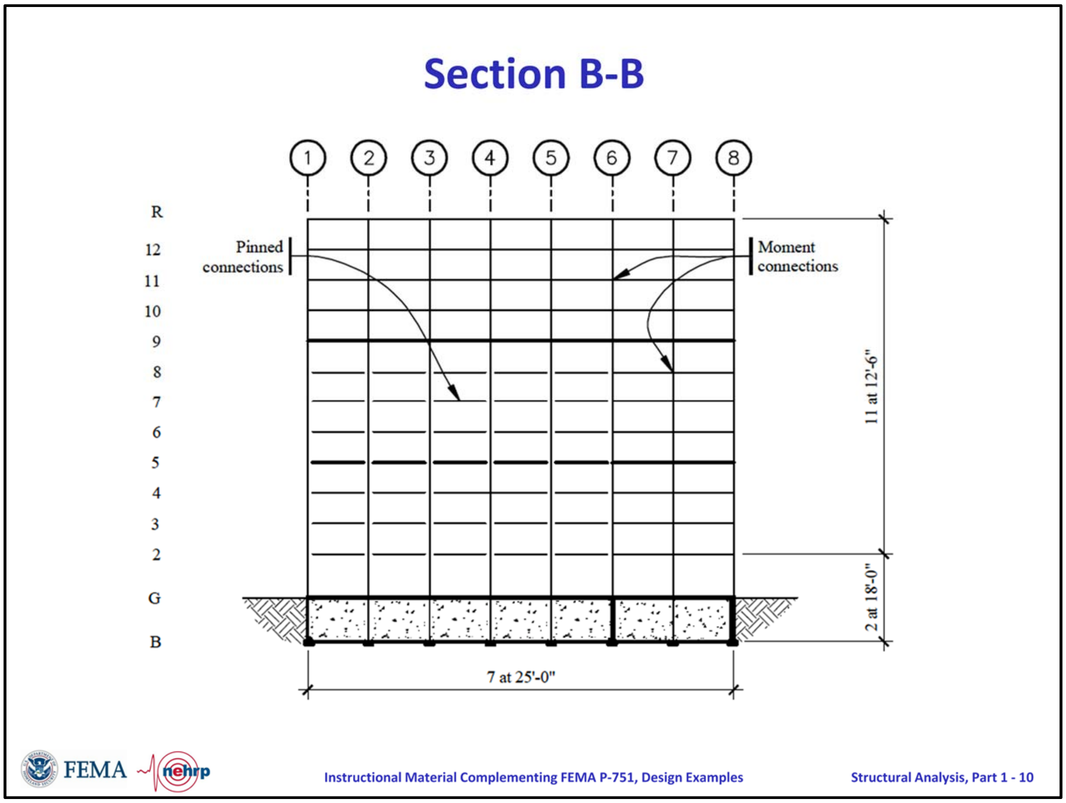
In this building all of the exterior moment resisting frames are lateral load resistant. Those portions of Frames C and F that are interior at the lower levels are gravity only frames.



The gravity-only columns and girders below the setbacks in grids C and F extend into the basement.



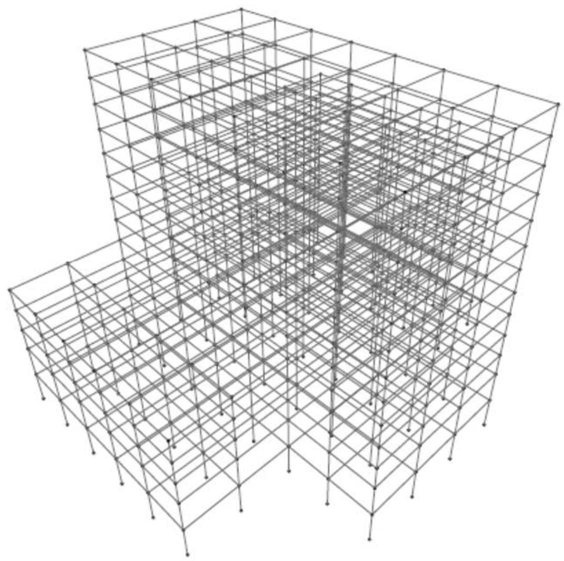
This view show the principal setbacks for the building. The shaded lines at levels 5 and 9 represent thickened diaphragm slabs.



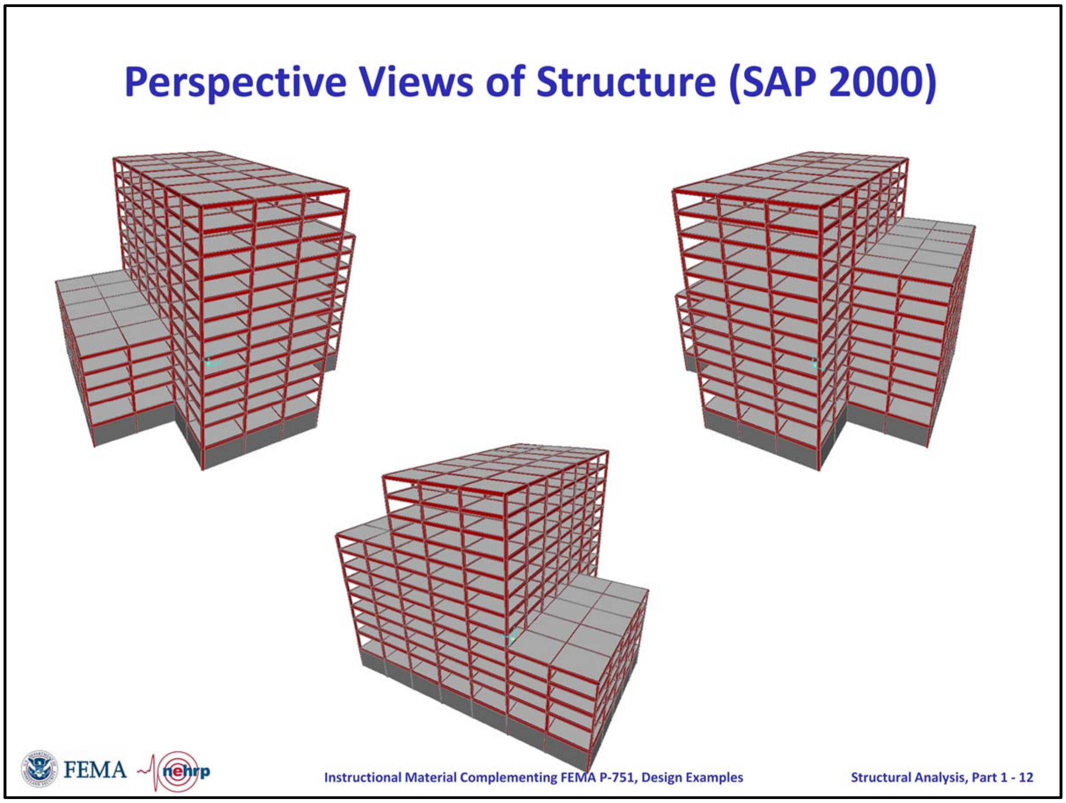
Note that the structure has one basement level. This basement is fully modeled in the analysis (the basement walls are modeled with shell elements), and will lead to complications in the analyses presented later.

All of the perimeter columns extend into the basement, and are embedded in the wall. (The wall is thickened around the columns to form monolithic pilasters). Thus, for analysis purposes, the columns may be assumed to be fixed at the top of the wall.

3-D Wire Frame View from SAP 2000



All analysis for this example was performed on SAP2000. The program ETABS may have been a more realistic choice, but this was not available.



These views show that the basement walls and the floor diaphragms were explicitly modeled in three dimensions. It is the author's opinion that all dynamic analysis should be carried out in three dimensions. When doing so it is simple to model the slabs and walls using shell elements. Note that a very coarse mesh is used because the desire is to include the stiffness (flexibility) of these elements only. No stress recovery was attempted. If stress recovery is important, a much finer mesh is needed.

Overview of Presentation

- Describe Building
- **Describe/Perform steps common to all analysis types**
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The goal of this example is to present the ASCE 7 analysis methodologies by example. Thus, this slide set is somewhat longer than it would need to be if only the main points of the analysis were to be presented.

Seismic Load Analysis: Basic Steps

1. Determine Occupancy Category (Table 1-1)
2. Determine Ground Motion Parameters:
   - S_S and S_1 USGS Utility or Maps from Ch. 22)
   - F_a and F_v (Tables 11.4-1 and 11.4-2)
   - S_{DS} and S_{D1} (Eqns. 11.4-3 and 11.4-4)
3. Determine Importance Factor (Table 11.5-1)
4. Determine Seismic Design Category (Section 11.6)
5. Select Structural System (Table 12.2-1)
6. Establish Diaphragm Behavior (Section 11. 3.1)
7. Evaluate Configuration Irregularities (Section 12.3.2)
8. Determine Method of Analysis (Table 12.6-1)
9. Determine Scope of Analysis [2D, 3D] (Section 12.7.2)
10. Establish Modeling Parameters

The steps presented on this slide are common to all analysis methods. The main structural analysis would begin after step 10. Note, however, that a very detailed "side analysis" might be required to establish diaphragm flexibility and to determine if certain structural irregularities exist. One point that should be stressed is that regardless of the method of analysis selected in step 8 (ELF, MRS, or MRH), an ELF analysis is required for all structures. This is true because ASCE 7-05 and ASCE 7-10 use an ELF analysis to satisfy accidental torsion requirements and P-Delta requirements. Additionally, an ELF analysis would almost always be needed in preliminary design.

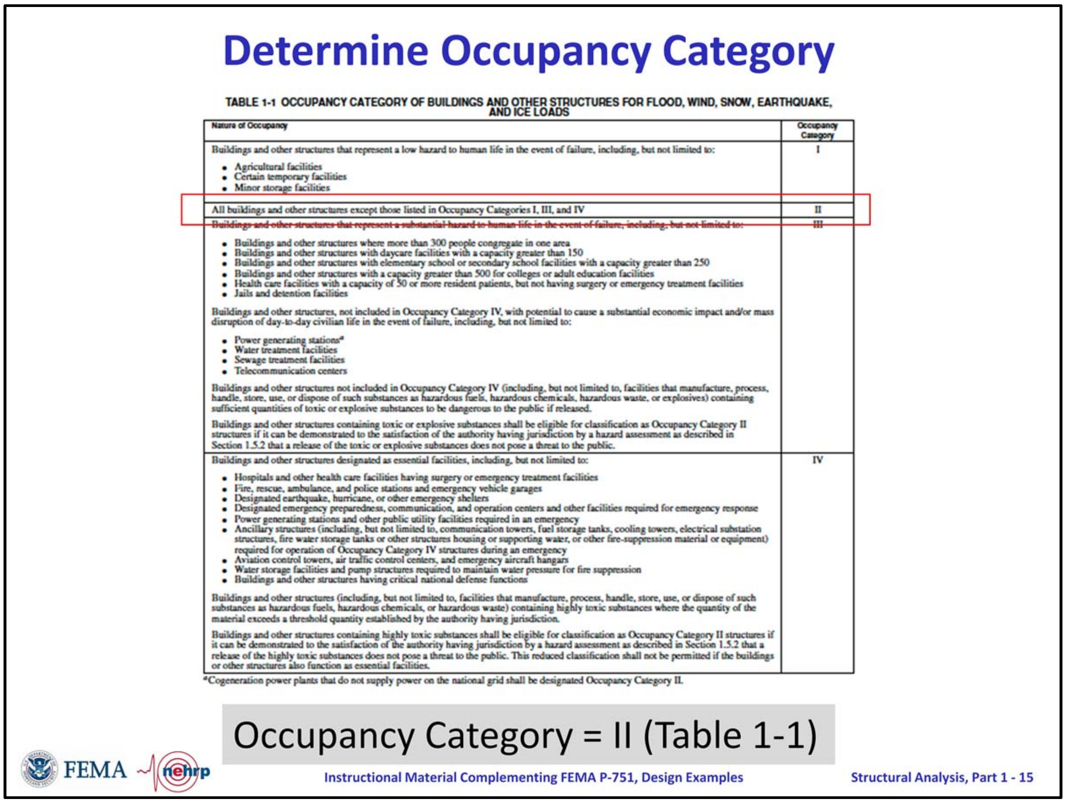


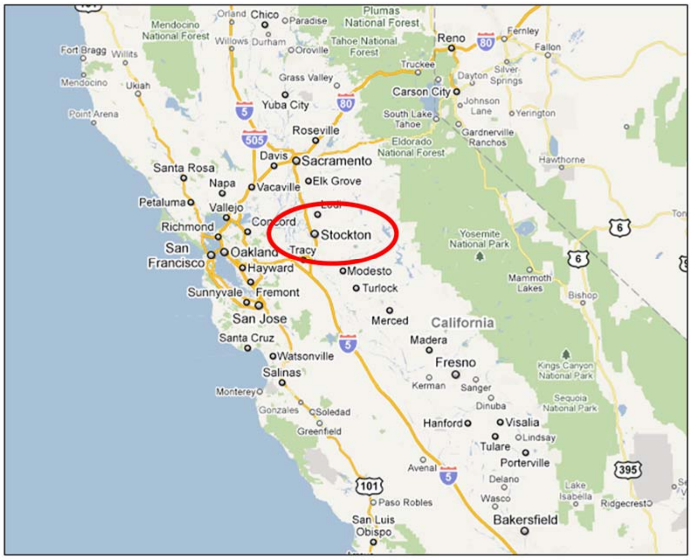
TABLE 1-1 OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES FOR FLOOD, WIND, SNOW, EARTHQUAKE, AND ICE LOADS

Nature of Occupancy	Occupancy Category
Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to: • Agricultural facilities • Certain temporary facilities • Minor storage facilities	I
All buildings and other structures except those listed in Occupancy Categories I, III, and IV	II
Buildings and other structures that represent a substantial hazard to human life in the event of failure, including, but not limited to: • Buildings and other structures where more than 300 people congregate in one area • Buildings and other structures with daycare facilities with a capacity greater than 150 • Buildings and other structures with elementary school or secondary school facilities with a capacity greater than 250 • Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities • Health care facilities with a capacity of 50 or more resident patients, but not having surgery or emergency treatment facilities • Jails and detention facilities Buildings and other structures, not included in Occupancy Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure, including, but not limited to: • Power generating stations[a] • Water treatment facilities • Sewage treatment facilities • Telecommunication centers Buildings and other structures not included in Occupancy Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released. Buildings and other structures containing toxic or explosive substances shall be eligible for classification as Occupancy Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the toxic or explosive substances does not pose a threat to the public.	III
Buildings and other structures designated as essential facilities, including, but not limited to: • Hospitals and other health care facilities having surgery or emergency treatment facilities • Fire, rescue, ambulance, and police stations and emergency vehicle garages • Designated earthquake, hurricane, or other emergency shelters • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response • Power generating stations and other public utility facilities required in an emergency • Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Occupancy Category IV structures during an emergency • Aviation control towers, air traffic control centers, and emergency aircraft hangars • Water storage facilities and pump structures required to maintain water pressure for fire suppression • Buildings and other structures having critical national defense functions Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing highly toxic substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction. Buildings and other structures containing highly toxic substances shall be eligible for classification as Occupancy Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the highly toxic substances does not pose a threat to the public. This reduced classification shall not be permitted if the buildings or other structures also function as essential facilities.	IV

[a]Cogeneration power plants that do not supply power on the national grid shall be designated Occupancy Category II.

This structure is used for an office building, so the Occupancy Category is II. Note that analysts usually need to refer to the IBC occupancy category table which is somewhat different than shown on this slide. It is for this reason that Table 1-1 as shown above has been simplified in ASCE 7-10. It should also be noted that assigning an Occupancy Category can be subjective, and when in doubt, the local building official should be consulted.

Ground Motion Parameters for Stockton



S_S=1.25g

S_1=0.40g

Instructional Material Complementing FEMA P-751, Design Examples

Structural Analysis, Part 1 - 16

These coefficients are not particularly realistic because they were selected to provide compatibility with an earlier version of this example. It is for this reason that Latitude-Longitude coordinates are not given. Students should be advised that Latitude-Longitude is preferable to zip code because some zip codes cover large geographic areas which can have a broad range of ground motion parameters.

Determining Site Coefficients

TABLE 11.4-1 SITE COEFFICIENT, F_a

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	$\boxed{1.0}$
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

F_a=1.0

TABLE 11.4-2 SITE COEFFICIENT, F_v

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	$\boxed{1.4}$	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

F_a=1.4

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Structural Analysis, Part 1 - 17

Note that the site coefficients are larger in areas of low seismicity. This is because the soil remains elastic under smaller earthquakes. For larger earthquakes the soil is inelastic, and the site amplification effect is reduced. Note that for site classes D and E the factor F_v can go as high as 3.5 for smaller earthquakes. Thus, for such sites in the central and eastern U.S., the ground motions can be quite large, and many structures (particularly critical facilities) may be assigned to Seismic Design Category D.

Determining Design Spectral Accelerations

- S_{DS}=(2/3)$F_a S_S$=(2/3)x1.0x1.25=0.833
- S_{D1}=(2/3)$F_v S_1$=(2/3)x1.4x0.40=0.373

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In this slide the intermediate coefficients S_{MS} and S_{M1} are not separately computed. Note that the subscript M stands for Maximum Considered Earthquake (MCE), and the subscript D in S_{DS} and S_{D1} stands for Design Basis Earthquake (DBE). The MCE is the earthquake with a 2% probability of being exceeded in 50 years. In California, the DBE is roughly a 10% in 50 year ground motion. In the Eastern and central U.S. the DBE is somewhere between a 2% and 10% in 50 year event.

Determine Importance Factor, Seismic Design Category

TABLE 11.5-1 IMPORTANCE FACTORS

Occupancy Category	*I*
I or II	1.0
III	1.25
IV	1.5

I = 1.0

TABLE 11.6-1 SEISMIC DESIGN CATEGORY BASED ON SHORT PERIOD RESPONSE ACCELERATION PARAMETER

Value of S_{DS}	Occupancy Category I or II	III	IV
$S_{DS} < 0.167$	A	A	A
$0.167 \leq S_{DS} < 0.33$	B	B	C
$0.33 \leq S_{DS} < 0.50$	C	C	D
$0.50 \leq S_{DS}$	D	D	D

TABLE 11.6-2 SEISMIC DESIGN CATEGORY BASED ON 1-S PERIOD RESPONSE ACCELERATION PARAMETER

Value of S_{D1}	OCCUPANCY CATEGORY I or II	III	IV
$S_{D1} < 0.067$	A	A	A
$0.067 \leq S_{D1} < 0.133$	B	B	C
$0.133 \leq S_{D1} < 0.20$	C	C	D
$0.20 \leq S_{D1}$	D	D	D

Seismic Design Category = D

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Instructional Material Complementing FEMA P-751, Design Examples

Structural Analysis, Part 1 - 19

Note that the SDC is a factor of BOTH the seismicity and intended use. For important buildings on soft sites in the central and Eastern U.S. it is possible to have an assignment of SDC D, which requires the highest level of attention to detailing. A few code cycles ago the same building would have had only marginal seismic detailing (if any).

Select Structural System (Table 12.2-1)

Building height (above grade) = 18+11(12.5)=155.5 ft

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]
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Special steel moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Special steel truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Intermediate steel moment frames	12.2.5.6, 12.2.5.7, 12.2.5.8, 12.2.5.9, and 14.1	4.5	3	4	NL	NL	35[k,i]	NP[h]	NP[i]
4. Ordinary steel moment frames	12.2.5.6, 12.2.5.7, 12.2.5.8, and 14.1	3.5	3	3	NL	NL	NP[h]	NP[h]	NP[i]
5. Special reinforced concrete moment frames	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Special composite steel and concrete moment frames	12.2.5.5 and 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Intermediate composite moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP
11. Ordinary composite moment frames	14.3	3	3	2½	NL	NP	NP	NP	NP

Select Special Steel Moment Frame: R=8, C_d=5.5, Ω_0=3

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We entered this example knowing it would be a special moment frame, so system selection was moot. However, this table can be used to illustrate height limits (which do not apply to the Special Steel Moment Frame). The required design parameters are also provided by the table.

The values of R = 8 and $\Omega\mu\epsilon\gamma\alpha_0$ are the largest among all systems. The ratio of C_d to R is one of the smallest for all systems.

Establish Diaphragm Behavior and Modeling Requirements

12.3.1 Diaphragm Flexibility.

The structural analysis shall consider the relative stiffness of diaphragms and the vertical elements of the seismic force–resisting system. *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., semi-rigid modeling assumption).*

12.3.1.2 Rigid Diaphragm Condition.

Diaphragms of concrete slabs or concrete filled metal deck with span-to-depth ratios of 3 or less in *structures that have no horizontal irregularities are permitted to be idealized as rigid.*

Due to horizontal irregularities (e.g. reentrant corners) the diaphragms must be modeled as semi-rigid. This will be done by using Shell elements in the SAP 2000 Analysis.

The diaphragm is modeled using shell elements in SAP2000. Only one element is required in each bay as all that is needed in the analysis is a reasonable estimate of in-plane diaphragm stiffness. If diaphragm stresses are to be recovered a much finer mesh would be required.

Determine Configuration Irregularities
Horizontal Irregularities

TABLE 12.3-1 HORIZONTAL STRUCTURAL IRREGULARITIES

		Irregularity Type and Description	Reference Section	Seismic Design Category Application
?	1a.	**Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.8.4.3 12.7.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F C, D, E, and F B, C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
?	1b.	**Extreme Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D B, C, and D
√	2.	**Reentrant Corner Irregularity** is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 $\boxed{\text{Table 12.6-1}}$	D, E, and F $\boxed{\text{D,}}$ E, and F
√	3.	**Diaphragm Discontinuity Irregularity** is defined to exist where there are diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 $\boxed{\text{Table 12.6-1}}$	D, E, and F $\boxed{\text{D,}}$ E, and F
X	4.	**Out-of-Plane Offsets Irregularity** is defined to exist where there are discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.	12.3.3.4 12.3.3.3 12.7.3 Table 12.6-1 16.2.2	D, E, and F B, C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
√	5.	**Nonparallel Systems-Irregularity** is defined to exist where the vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic force–resisting system.	12.5.3 12.7.3 Table 12.6-1 Section 16.2.2	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

Irregularity 2 occurs on lower levels. Irregularity 3 is possible but need not be evaluated because it has same consequences as irregularity 3. Torsional Irregularities will be assessed later.

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Torsional irregularities must be determined by analysis, and this is discussed later in the example. The structure clearly has a re-entrant corner irregularity, and the diaphragm discontinuity irregularity is also likely. Note, however, that the consequences of the two irregularities (2 and 3) are the same, so these are effectively the same irregularity.

The structure has a nonparallel system irregularity because of the nonsymmetrical layout of the system. Note that in ASCE 7-10 the words "or symmetric about" in the description of the nonparallel system irregularity have been removed, so this structure would not have a nonsymmetrical irregularity in ASCE 7-10. This is a consequential change because requirements for three dimensional analysis and orthogonal loading are tied to the presence of a type 5 irregularity.

Determine Configuration Irregularities
Vertical Irregularities

TABLE 12.3-2 VERTICAL STRUCTURAL IRREGULARITIES

		Irregularity Type and Description	Reference Section	Seismic Design Category Application
X	1a.	**Stiffness-Soft Story Irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	Table 12.6-1	D, E, and F
	1b.	**Stiffness-Extreme Soft Story Irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	12.3.3.1 Table 12.6-1	E and F D, E, and F
✓	2.	**Weight (Mass) Irregularity** is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	$\boxed{\text{Table 12.6-1}}$	$\boxed{\text{D,}}$ E, and F
✓	3.	**Vertical Geometric Irregularity** is defined to exist where the horizontal dimension of the seismic force–resisting system in any story is more than 130% of that in an adjacent story.	$\boxed{\text{Table 12.6-1}}$	$\boxed{\text{D,}}$ E, and F
X	4.	**In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity** is defined to exist where an in-plane offset of the lateral force-resisting elements is greater than the length of those elements or there exists a reduction in stiffness of the resisting element in the story below.	12.3.3.3 12.3.3.4 Table 12.6-1	B, C, D, E, and F D, E, and F D, E, and F
X	5a.	**Discontinuity in Lateral Strength–Weak Story Irregularity** is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 Table 12.6-1	E and F D, E, and F
X	5b.	**Discontinuity in Lateral Strength–Extreme Weak Story Irregularity** is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 12.3.3.2 Table 12.6-1	D, E, and F B and C D, E, and F

Irregularities 2 and 3 occur due to setbacks. Soft story and weak story irregularities are highly unlikely for this system and are not evaluated.

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The structure in question clearly has the two irregularities noted.

One thing that should be illustrated on this slide (and the previous slide) is that the there are no “consequences” if certain irregularities occur in SDC B and C systems. For example, Vertical Irregularities 1, 2, and 3 have consequences only for SDC D, E, and F, thus the possible occurrence of the irregularities need not be checked in SDC B and C.

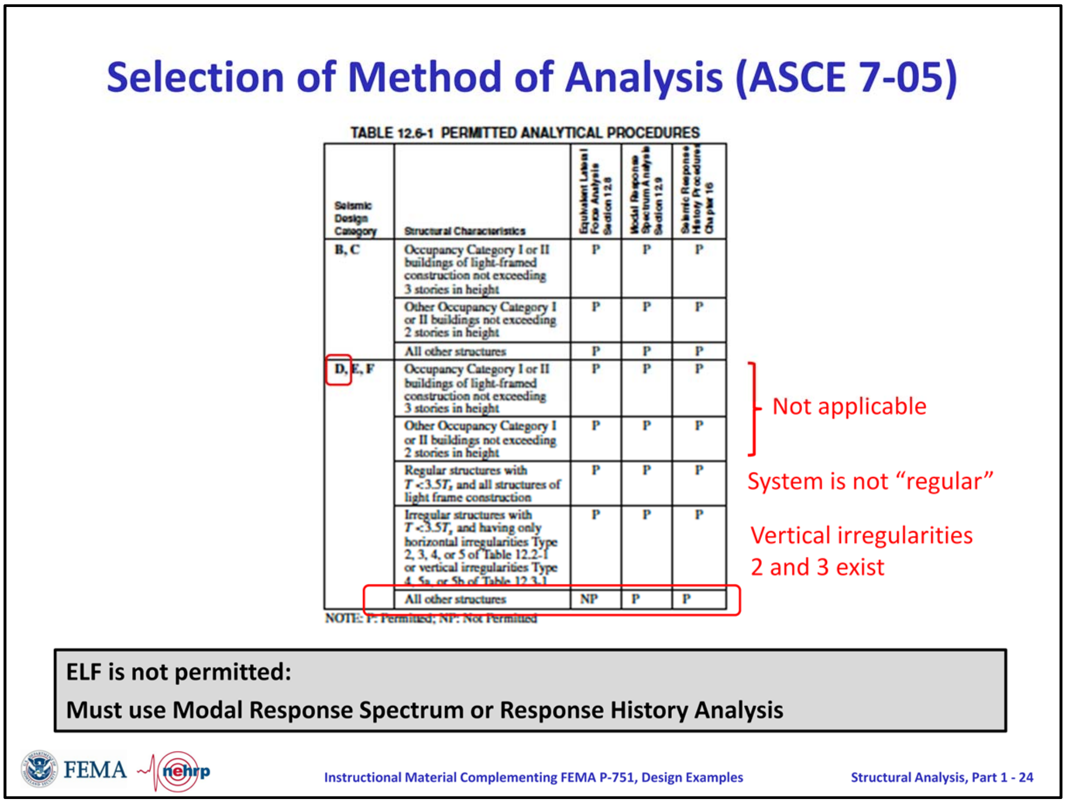

TABLE 12.6-1 PERMITTED ANALYTICAL PROCEDURES

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Analysis Section 12.8	Modal Response Spectrum Analysis Section 12.9	Seismic Response History Procedures Chapter 16
B, C	Occupancy Category I or II buildings of light-framed construction not exceeding 3 stories in height	P	P	P
	Other Occupancy Category I or II buildings not exceeding 2 stories in height	P	P	P
	All other structures	P	P	P
D, E, F	Occupancy Category I or II buildings of light-framed construction not exceeding 3 stories in height	P	P	P
	Other Occupancy Category I or II buildings not exceeding 2 stories in height	P	P	P
	Regular structures with $T < 3.5T_s$ and all structures of light frame construction	P	P	P
	Irregular structures with $T < 3.5T_s$ and having only horizontal irregularities Type 2, 3, 4, or 5 of Table 12.2-1 or vertical irregularities Type 4, 5a, or 5b of Table 12.3-1	P	P	P
	All other structures	NP	P	P

NOTE: P: Permitted; NP: Not Permitted

The ELF method is allowed for the vast majority of systems. The main reason that ELF is not allowed for this system is that (1) it is in SDC D, and (2) it has Reentrant Corner and Diaphragm Discontinuity Irregularities. It is interesting to note that ELF is allowed in higher SDC even when there are stiffness, weight, and weak story irregularities. It seems that this would be more of a detriment to the accuracy of ELF than than would a reenrtant corner.

Note that Table 12.6-1 as shown in the slide is from ASCE 7-05. The table has been simplified somewhat for ASCE 7-10 (see the next slide), but the basic configurations where ELF are allowed/disallowed are essentially the same.

Selection of Method of Analysis (ASCE 7-10)

Table 12.6-1 Permitted Analytical Procedures

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Analysis, Section 12.8[a]	Modal Response Spectrum Analysis, Section 12.9[a]	Seismic Response History Procedures, Chapter 16[a]
B, C	All structures	P	P	P
D, E, F	Risk Category I or II buildings not exceeding 2 stories above the base	P	P	P
	Structures of light frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 160 ft in structural height	P	P	P
	Structures exceeding 160 ft in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures not exceeding 160 ft in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12-2 or vertical irregularities of Type 4, 5a, or 5b in Table 12-3	P	P	P
	All other structures	NP	P	P

[a]P: Permitted; NP: Not Permitted; $T_s = S_{D1}/S_{DS}$.

ELF is not permitted:
Must use Modal Response Spectrum or Response History Analysis

This is Table 12.6-1 from ASCE 7-10. The main difference with respect to ASCE 7-05 is that building height is the trigger for making decisions, rather than the use of $T < 3.5T_s$. The change was made because there are scenarios under the ASCE 7-05 table that produced illogical results. For example, there were scenarios where a tall building on soft soil in Seattle could use ELF, whereas a shorter building on stiff soil in New York could not.

Overview of Presentation

- Describe Building
- Describe/Perform steps common to all analysis types
- **Overview of Equivalent Lateral Force analysis**
- Overview of Modal Response Spectrum Analysis
- Overview of Modal Response History Analysis
- Comparison of Results
- Summary and Conclusions

Title slide.

Comments on use of ELF for *This System*

ELF is NOT allowed as the *Design Basis Analysis*.

However, ELF (or aspects of ELF) must be used for:

- Preliminary analysis and design
- Evaluation of torsion irregularities and amplification
- Evaluation of system redundancy factors
- Computing P-Delta Effects
- Scaling Response Spectrum and Response History results

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It is important to note that ALL seismic analysis requires ELF analysis in one form or another. The statement that ELF may not be allowed as a "Design Basis" analysis means that the design drifts and element forces may need to be based on more advanced analysis, such as Modal Response Spectrum or Response History analysis.

Determine Scope of Analysis

12.7.3 Structural Modeling.

A mathematical model of the structure shall be constructed for the purpose of determining member forces and structure displacements resulting from applied loads and any imposed displacements or P-Delta effects.

The model shall include the stiffness and strength of elements that are significant to the distribution of forces and deformations in the structure and represent the spatial distribution of mass and stiffness throughout the structure.

Note: P-Delta effects should not be included directly in the analysis. They are considered indirectly in Section 12.8.7

There is a significant inconsistency in the requirement that P-Delta effects be represented in the mathematical model. In fact, such effects should NOT be included in the model because they are evaluated separately in Section 12.8.7. Additionally, direct modeling of the strength of the elements is not required in linear analysis, but of course, would be needed in any form of nonlinear analysis.

Determine Scope of Analysis (Continued)

Continuation of 12.7.3:

Structures that have horizontal structural irregularity Type 1a, 1b, 4, or 5 of Table 12.3-1 shall be analyzed using a 3-D representation.

Where a 3-D model is used, a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis shall be included at each level of the structure.

Where the diaphragms have not been classified as rigid or flexible in accordance with Section 12.3.1, the model shall include representation of the diaphragm's stiffness characteristics and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response.

Analysis of structure must be in 3D, and diaphragms must be modeled as semi-rigid

Three dimensional analysis is required for this system, and the diaphragms must be modeled as semi-rigid because the reentrant corners prohibit classification of the diaphragms as rigid. Regardless of this requirement, it would be virtually impossible to model the example structure in 2 dimensions.

In most cases is is easier to model a structure in three dimensions than in two. This is due to the fact that most modern software makes it easy to generate the model, and assumptions do not need to be made as to the best way to separate out the various elements for analysis. Additionally, the use of rigid diaphragms as a way to reduce the number of DOF is not needed because the programs can analyze quite complex 3D systems in only a few seconds. Semi-rigid diaphragms are easy to model using shell elements, and very coarse meshes may be used if it is not desired to recover diaphragm stresses.

Establish Modeling Parameters

Continuation of 12.7.3:

In addition, the model shall comply with the following:

a) Stiffness properties of concrete and masonry elements shall consider the effects of cracked sections.
b) For steel moment frame systems, the contribution of panel zone deformations to overall story drift shall be included.

No comment required. See the notes on the following slide.

Modeling Parameters used in Analysis

1) The floor diaphragm was modeled with shell elements, providing nearly rigid behavior in-plane.

2) Flexural, shear, axial, and torsional deformations were included in all columns and beams.

3) Beam-column joints were modeled using centerline dimensions. This approximately accounts for deformations in the panel zone.

4) Section properties for the girders were based on bare steel, ignoring composite action. This is a reasonable assumption in light of the fact that most of the girders are on the perimeter of the building and are under reverse curvature.

Most of these points are self-explanatory. It should be noted that the use of centerline analysis in steel moment frames is used because it has been shown that offsetting errors lead to reasonable results. The errors in centerline analysis are that (a) shear deformations in the panel zones are underestimated, and (b) flexural deformations in the panel zones are overestimated. Many programs have models that can directly include panel zone beam column joint deformations. Several programs allow the use of rigid end zones, but this should never be done because it drastically overestimates the lateral stiffness of the structure.

Modeling Parameters used in Analysis (continued)

5) Except for those lateral load-resisting columns that terminate at Levels 5 and 9, all columns of the lateral load resisting system were assumed to be fixed at their base.

6) The basement walls and grade level slab were explicitly modeled using 4-node shell elements. This was necessary to allow the interior columns to continue through the basement level. No additional lateral restraint was applied at the grade level, thus the basement level acts as a very stiff first floor of the structure. This basement level was not relevant for the ELF analysis, but did influence the MRS and MRH analysis as described in later sections of this example

7) P-Delta effects were not included in the mathematical model. These effects are evaluated separately using the procedures provided in section 12.8.7 of the Standard.

The basement was modeled because it was desired to run the interior columns down to the basement slab.

Equivalent Lateral Force Analysis

1. Compute Seismic Weight, *W* (Sec. 12.7.2)
2. Compute Approximate Period of Vibration T_a (Sec. 12.8.2.1)
3. Compute Upper Bound Period of Vibration, $T=C_uT_a$ (Sec. 12.8.2)
4. Compute "Analytical" Natural periods
5. Compute Seismic Base Shear (Sec. 12.8.1)
6. Compute Equivalent Lateral Forces (Sec. 12.8.3)
7. Compute Torsional Amplification Factors (Sec. 12.8.4.3)
8. Determine Orthogonal Loading Requirements (Sec. 12.8)
9. Compute Redundancy Factor ρ (Sec. 12.3.4)
10. Perform Structural Analysis
11. Check Drift and P-Delta Requirements (Sec. 12.9.4 and 12.9.6)
12. Revise Structure in Necessary and Repeat Steps 1-11 [as appropriate]
13. Determine Design-Level Member Forces (Sec. 12.4)

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These are the basic steps required for equivalent lateral force analysis. Each of these points are discussed in the following several slides.

It should be noted that there is a lot of detail in the ELF analysis, and thus this is not a trivial task. There are numerous requirements scattered throughout ASCE 7, and sometimes these requirements are somewhat ambiguous. Anyone attempting an ELF analysis (or any other ASCE 7 analysis for that mater) should read the entire relevant chapters (11 and 12 in this case) before beginning the analysis.

Notes on Computing the Period of Vibration

T_a (Eqn.12.8-7) is an approximate lower bound period, and is based on the measured response of buildings in high seismic regions.

$T=C_uT_a$ is also approximate, but is somewhat more accurate than T_a alone because it is based on the "best fit" of the measured response, and is adjusted for local seismicity. Both of these adjustments are contained in the C_u term.

C_uT_a can only be used if an analytically computed period, called $T_{computed}$ herein, is available from a computer analysis of the structure.

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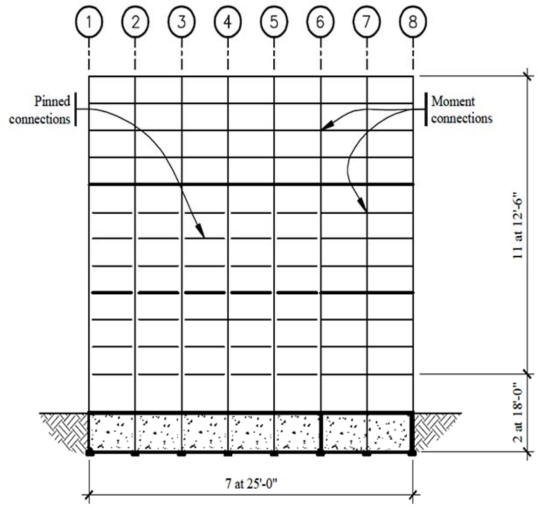
Slide provides comments on computing period of vibration.

Using Empirical Formulas to Determine T_a



$$T_a = C_t h_n^x$$

From Table 12.8.2:
C_t=0.028
x=0.80

h_n=18+11(12.5)=155.5 ft

$$T_a = 0.028(155.5)^{0.8} = 1.59 \text{ sec}$$

Applies in Both Directions

Here the height for period calculations is taken as the height above grade. This is reasonable because the basement walls are very stiff, and because the perimeter columns are embedded in pilasters that are cast with the walls.

Adjusted Empirical Period $T=C_uT_a$

TABLE 12.8-1 COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD

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

S_{D1}=0.373
Gives C_u=1.4

$$T = 1.4(1.59) = 2.23 \text{ sec}$$

Applies in Both Directions

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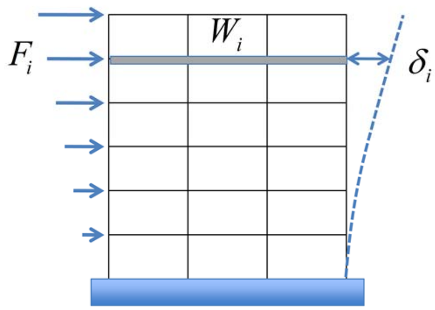
The C_u adjustment to period is allowed only if a rational (Eigenvalue or Rayleigh) analysis is used to compute a period. This adjustment removes an inherent conservatism in the statistics used to derive the empirical formula, and adjusts for seismicity (recognizing that structures in lower hazard areas are likely to be more flexible than structures in high hazard areas). The period used in base shear calculations can not exceed C_uT_a, but drifts may be computed on the basis of the period determined from rational analysis.

Use of Rayleigh Analysis to Determine $T_{computed}$



Building has *n* Levels

$$T_{computed} = \frac{2\pi}{\omega_{computed}}$$

$$\omega_{computed} = \sqrt{\frac{g\sum_{i=1}^{n}\delta_i F_i}{\sum_{i=1}^{n}\delta_i^2 W_i}}$$

If a computer model is available it is easy to estimate the period using this approach. The lateral load pattern should be of the same approximate shape as the first mode shape. An upper triangular pattern or the ELF load pattern will usually suffice.

Use of Rayleigh Analysis to Determine $T_{computed}$

Table 4.1-9 Rayleigh Analysis for X Direction Period of Vibration

Level	Drift, δ(in.)	Force, F (kips)	Weight, W (kips)	δF (in.-kips)	$\delta^2 W/g$ (in.-kips-sec^2)
R	6.67	186.9	1657	1247	191
12	6.35	154.0	1596	979	167
11	5.90	129.9	1596	767	144
10	5.34	107.6	1596	575	118
9	4.73	186.3	3403	881	197
8	4.15	100.8	2331	418	104
7	3.52	77.0	2331	271	75
6	2.87	56.2	2331	162	50
5	2.24	71.4	4324	160	56
4	1.71	31.5	3066	54	23
3	1.17	16.6	3066	19	11
2	0.64	6.3	3097	4	3
Σ				5536	1138

$\omega = (5536/1138)^{0.5} = 2.21$ rad/sec. $T = 2\pi/\omega = 2.85$ sec. 1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

X-Direction $T_{computed}$ = 2.85 sec.
Y-Direction $T_{computed}$ = 2.56 sec.

(see Text)

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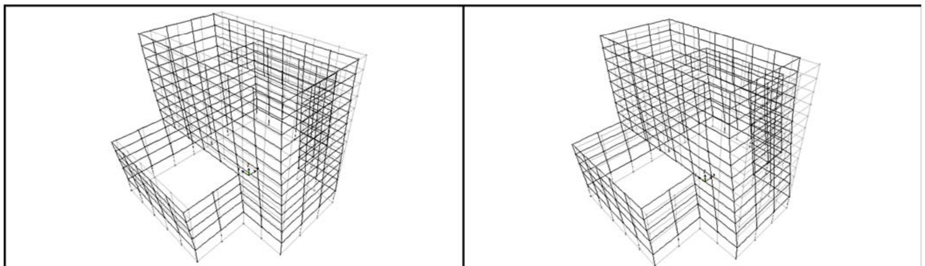
Both of the rationally computed periods exceed C_uT_a, so C_uT_a will be used in the ELF analysis.

Periods Computed Using Eigenvalue Analysis

$$K\Phi = M\Phi\Omega^2$$

$\Omega =$ Diagonal matrix containing circular frequencies ω

$\Phi =$ Mode Shape Matrix



Mode 1 T=2.87 sec

Mode 2: T:2.60 sec

The periods from the Eigenvalue analysis are the most mathematically precise. As seen, these are very close that those produced by the Rayleigh method (see previous slide). Periods computed using the Rayleigh method should generally be close to, but slightly less than those computed from Eigenvalue analysis.

Range of Periods Computed for This Example

T_a=1.59 sec

C_uT_a=2.23 sec

$T_{computed}$ = 2.87 sec in X direction

2.60 sec in Y direction

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This slide simply summarizes the periods found by the three different methods. The distribution of periods shown is not uncommon. It is the author's experience that the computed period is almost always greater than C_uT_a for moment frames.

Periods of Vibration for Computing Seismic Base Shear (Eqns 12.8-1, 12.8-3, and 12.8-4)

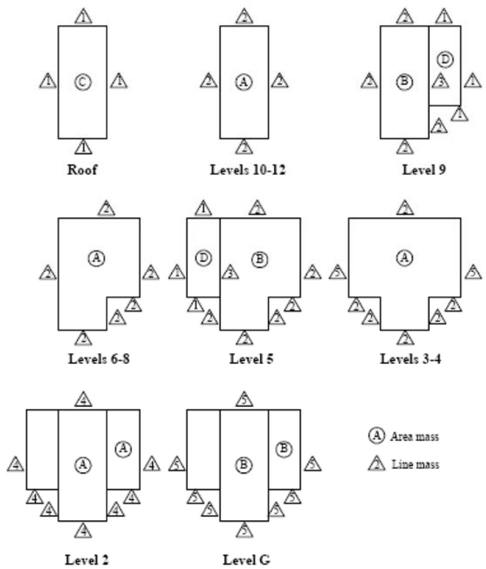
if $T_{computed}$ is not available use T_a

if $T_{computed}$ is available, then:

- if $T_{computed} > C_u T_a$ use $C_u T_a$
- if $T_a <= T_{computed} <= C_u T_a$ use $T_{computed}$
- if $T_{computed} < T_a$ use T_a

This slide provides a simple summary for choosing the period to use for ELF analysis.

Area and Line Weight Designations



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This slide is simply a key for use in describing masses computation (see following slide). Both line masses and area masses were considered.

Area and Line Weight Values

Table 4.1-1 Area Weights Contributing to Masses on Floor Diaphragms

Mass Type	Area Weight Designation				
	A	B	C	D	E
Slab and Deck (psf)	50	75	50	75	75
Structure (psf)	20	20	20	20	50
Ceiling and Mechanical (psf)	15	15	15	15	15
Partition (psf)	10	10	0	0	10
Roofing (psf)	0	0	15	15	0
Special (psf)	0	0	0	60	25
Total (psf)	95	120	100	185	175

See Figure 4.1-4 for mass location. 1.0 psf = 47.9 N/m^2.

Table 4.1-2 Line Weights Contributing to Masses on Floor Diaphragms

Mass Type	Line Weight Designation				
	1	2	3	4	5
From Story Above (plf)	60.0	93.8	93.8	93.8	135.0
From Story Below (plf)	93.8	93.8	0.0	135.0	1350.0
Total (plf)	153.8	187.6	93.8	228.8	1485.0

See Figure 4.1-4 for mass location. 1.0 plf = 14.6 N/m.

Slide shows calculations for computing area and line weights.

Weights at Individual Levels

Table 4.1-3 Floor Weight, Floor Mass, Mass Moment of Inertia, and Center of Mass Locations

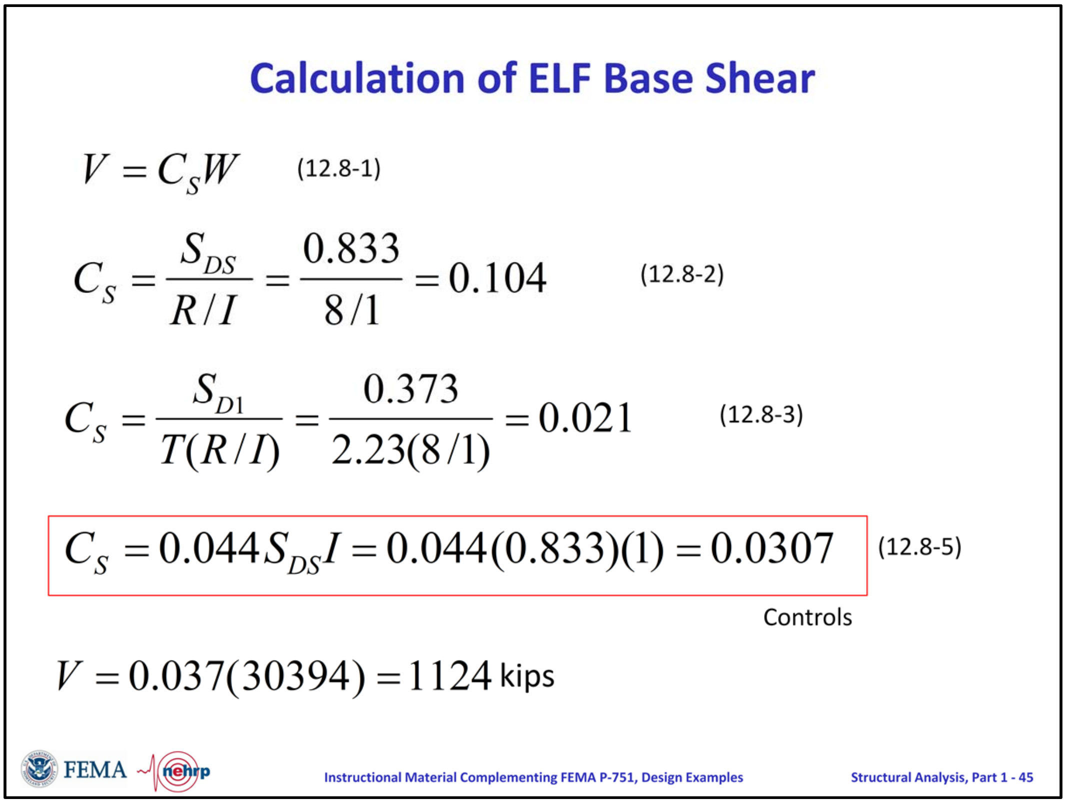
Level	Weight (kips)	Mass (kip-sec^2/in.)	Mass Moment of Inertia (in.-kip-sec^2/radian)	X Distance to C.M. (in.)	Y Distance to C.M. (in.)
R	1657	4.287	2.072×10^6	1260	1050
12	1596	4.130	2.017×10^6	1260	1050
11	1596	4.130	2.017×10^6	1260	1050
10	1596	4.130	2.017×10^6	1260	1050
9	3403	8.807	5.309×10^6	1638	1175
8	2331	6.032	3.703×10^6	1553	1145
7	2331	6.032	3.703×10^6	1553	1145
6	2331	6.032	3.703×10^6	1553	1145
5	4320	11.19	9.091×10^6	1160	1206
4	3066	7.935	6.356×10^6	1261	1184
3	3066	7.935	6.356×10^6	1261	1184
2	3097	8.015	6.437×10^6	1262	1181
G	6525	16.89	1.503×10^7	1265	1149
Σ	36912				

Total Building Weight=36,912 k.

Weight above grade = 30,394 k.

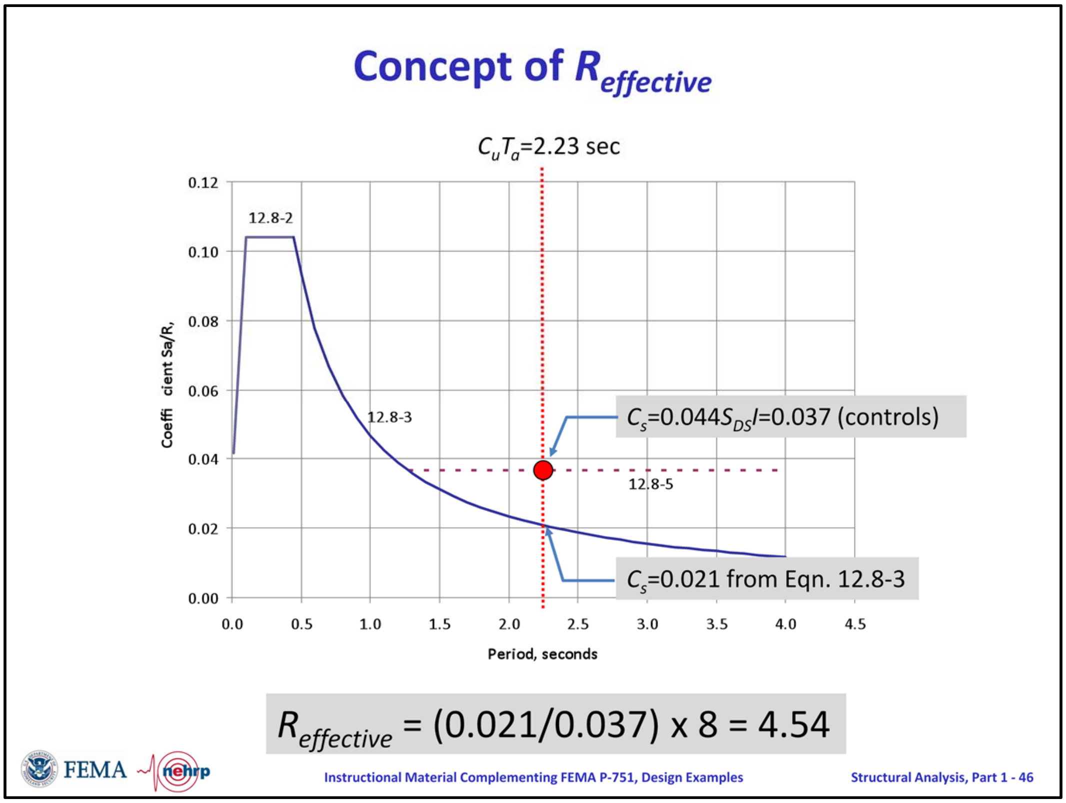
The calculations for determining total seismic weight are shown. The equivalent lateral forces will be based on the weight of the structure above grade (30,394 kips) even though the full structure, including the basement, is modeled.

The location of the CM is needed because the equivalent lateral forces are applied to the CM at each level.



This slide shown the equations that are needed for computing the design base shear. Equation 12.8-4 is not needed because the structures period is less than T_L. Equation 12.6-6 is not needed because S_1 < 0.6g.

Equation 12.8-5 controls the base shear. Note that this equation was originally not used in ASCE 7-05 (where the the minimum was instead taken as 0.01*W*). Equation 12.8-5 as shown above is included in a supplement to ASCE 7-05, and is provided as shown in ASCE 7-10.



This slide shows that the "Effective" *R* value for this structure is 4.54. Thus, the anticipated economy inherent in the use of *R* = 8 has not been realized.

Issues Related to Period of Vibration and Drift

12.8.6.1 Minimum Base Shear for Computing Drift

The elastic analysis of the seismic force-resisting system for computing drift shall be made using the prescribed seismic design forces of Section 12.8.

EXCEPTION: Eq. 12.8-5 need not be considered for computing drift

12.8.6.2 Period for Computing Drift

For determining compliance with the story drift limits of Section 12.12.1, it is permitted to determine the elastic drifts, (δ_{xe}), using seismic design forces based on the computed fundamental period of the structure without the upper limit (C_uT_a) specified in Section 12.8.2.

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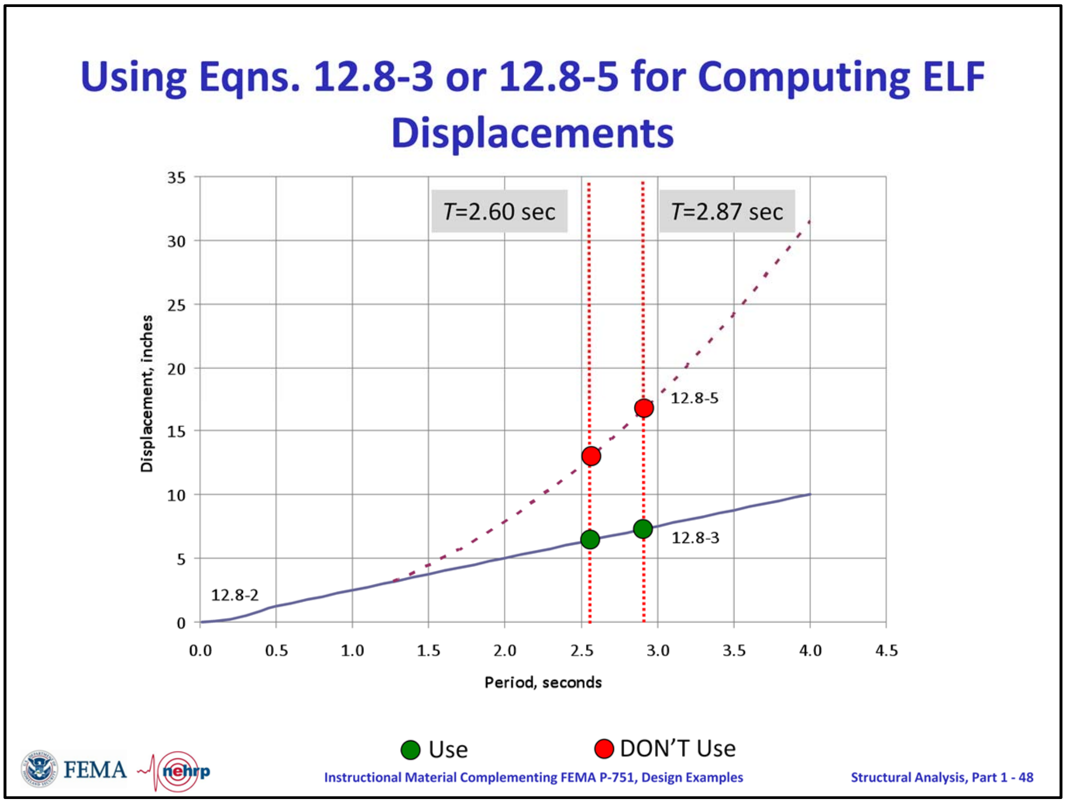
Although base shear may be controlled by Equation 12.8-5, the drifts can be based on the base shear computed from Eqn. 12.8-3, and furthermore, the computed period of vibration may be used in lieu of C_uT_a for drift calculations. This means that a separate set of lateral forces may be computed for the purposes of calculating deflections in the structure.

The exception shown for ASCE 7-10 did not exist in ASCE 7-05, although many analysts used this exception anyway. The reason is shown on the following slide, where the deflections based on Eqn. 12.8-3 and 12.5-5 are compared.



This slide shows Equations 12.8-3 and 12.8-5 in the form of a displacement spectrum. The two periods are from the Eigenvalue analysis. If Equation 12.8-5 is used to compute forces for determining drift, the drifts would increase exponentially, which is not rational. The irrationality is due to the fact that 12.8-5 is a minimum base shear formula, and is NOT a true branch of the response spectrum.

What if Equation 12.8-6 had Controlled Base Shear?

$$C_s = \frac{0.5S_1}{(R/I)}$$

Eqn. 12.8-6, applicable only when S_1 >= 0.6g

This equation represents the "true" response spectrum shape for near-field ground motions. Thus, the lateral forces developed on the basis of this equation must be used for determining component design forces **and** displacements used for computing drift.

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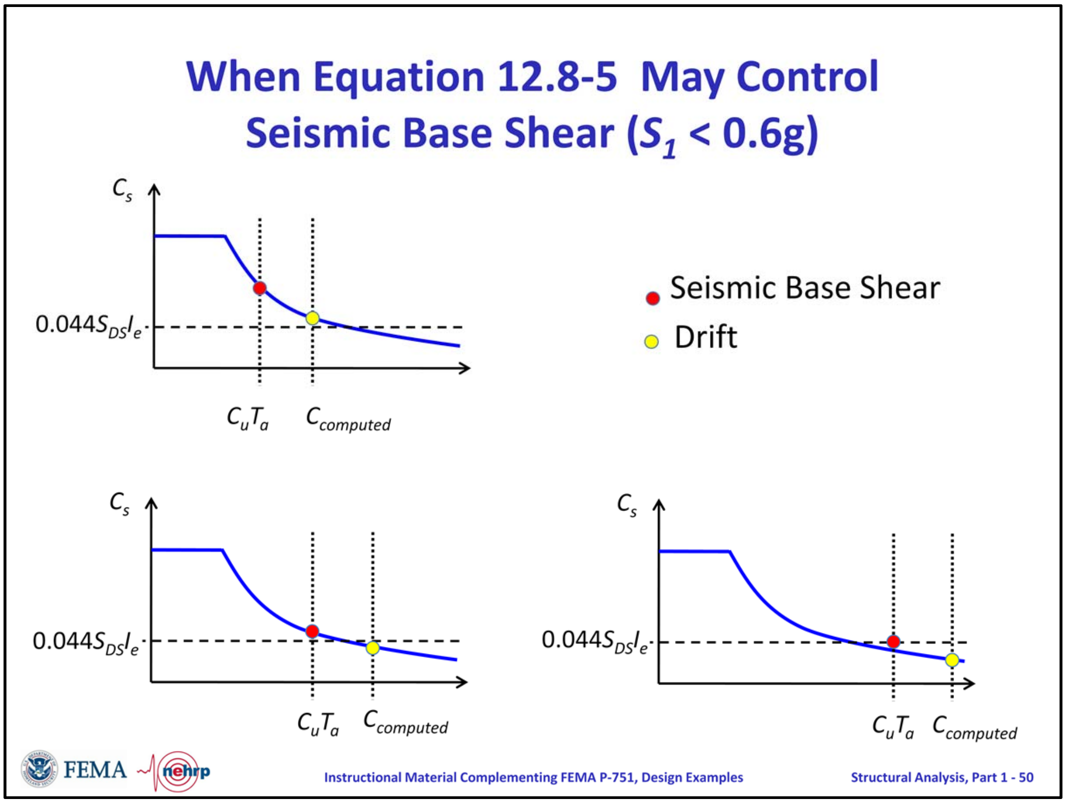
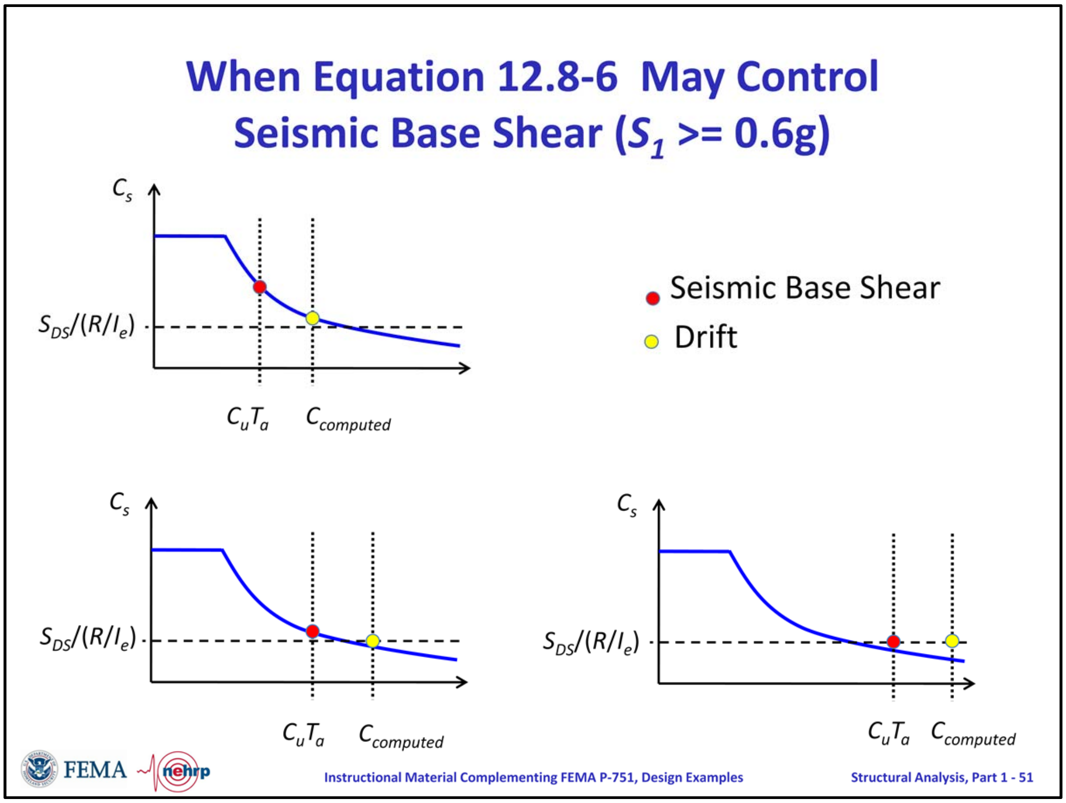
When Eqn. 12.8-6 controls, the drifts must be based on the lateral forces computed from 12.8-6. Note that this formula is not dependent on period.

The argument for requiring that Eqn. 12.8-6 be used for drift calculations is that it represents the the "true" spectral shape… it is not a minimum base shear formula. However, for longer period buildings, Eqn. 12.8-6 can lead to irrationally large displacements because the deflections will increase exponentially with period.



This slide summarizes the use of Equations 12.8-3 and 12.8-5 when computing base shear and drift.

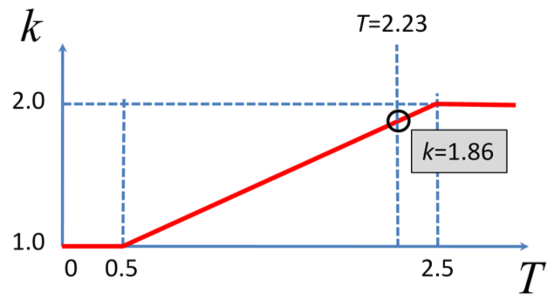


This slide summarizes the use of Equations 12.8-3 and 12.8-6 when computing base shear and drift.

Calculation of ELF Forces

$$F_x = C_{vx} V \quad \text{(12.8-11)}$$

$$C_{vs} = \frac{w_x h^k}{\sum_{i=1}^{n} w_i h_i^k} \quad \text{(12.8-12)}$$



These are the equations for determining the distribution of lateral force along the height. The exponent *k* is determined by interpolation.

Calculation of ELF Forces (continued)

Table 4.1-4 Equivalent Lateral Forces for Building Responding in X and Y Directions

Level x	w_x (kips)	h_x (ft)	$w_x h_x^k$	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
R	1657	155.5	20272144	0.1662	186.9	186.9	2336
12	1596	143.0	16700697	0.1370	154.0	340.9	6597
11	1596	130.5	14081412	0.1155	129.9	470.8	12482
10	1596	118.0	11670590	0.0957	107.6	578.4	19712
9	3403	105.5	20194253	0.1656	186.3	764.7	29271
8	2331	93.0	10933595	0.0897	100.8	865.5	40090
7	2331	80.5	8353175	0.0685	77.0	942.5	51871
6	2331	68.0	6097775	0.0500	56.2	998.8	64356
5	4324	55.5	7744477	0.0635	71.4	1070.2	77733
4	3066	43.0	3411857	0.0280	31.5	1101.7	91505
3	3066	30.5	1798007	0.0147	16.6	1118.2	103372
2	3097	18.0	679242	0.0056	6.3	1124.5	120694
Σ	30394	-	121937234	1.00	1124.5		

Values in column 4 based on exponent k=1.865. 1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN.

The lateral forces are computed using a spreadsheet. Note that the forces in the X and Y directions are the same because both directions are controlled by the same minimum base shear formula, and both have the same period of vibration $C_u T_a$.

ELF Analysis Assumptions

1. The floor diaphragm was modeled with shell elements, providing nearly rigid behavior in-plane.
2. Flexural, shear, axial, and torsional deformations were included in all columns and beams.
3. Beam-column joints were modeled using centerline dimensions. This approximately accounts for deformations in the panel zone.
4. Section properties for the girders were based on bare steel, ignoring composite action. This is a reasonable assumption in light of the fact that most of the girders are on the perimeter of the building and are under reverse curvature.

The basic analysis assumptions for ELF are summarized here. And on the following slide.

ELF Analysis Assumptions (Continued)

5. Except for those lateral-load-resisting columns that terminate at Levels 5 and 9, all columns were assumed to be fixed at their base.

6. The basement walls and grade level slab were explicitly modeled using 4-node shell elements. This was necessary to allow the interior columns to continue through the basement level. No additional lateral restraint was applied at the grade level, thus the basement level acts as a very stiff first floor of the structure. This basement level was not relevant for the ELF analysis, but did influence the MRS and MRH analysis as described in later sections of this example

7. P-Delta effects were not included in the mathematical model. These effects are evaluated separately using the procedures provided in section 12.8.7 of the Standard.

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Assumptions on ELF analysis, continued.

Inherent and Accidental Torsion

12.8.4.1 Inherent Torsion. For diaphragms that are not flexible, the distribution of lateral forces at each level shall consider the effect of the inherent torsional moment, M_t, *resulting from eccentricity* between the locations of the center of mass and the center of rigidity. For flexible diaphragms, the distribution of forces to the vertical elements shall account for the position and distribution of the masses supported.

Inherent torsion effects are automatically included in 3D structural analysis, and member forces associated with such effects need not be separated out from the analysis.

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Previous versions of ASCE 7 required that both accidental and inherent torsion be amplified in higher SDCs when there were significant torsional irregularities. Thus, the inherent torsion needed to be separated out from the results of a 3D analysis. In ASCE 7-05 and ASCE 7-10, the inherent torsion need not be amplified, so inherent torsion need not be separated out when a 3D analysis is used.

If a planar analysis is performed, it will be necessary to determine the inherent torsion loading and transform it into in-plane loads on the frames. Such calculations are not straightforward, thus 3D modeling, which may seem to be complex, may in fact be simpler than 2D analysis.

Inherent and Accidental Torsion (continued)

12.8.4.2 Accidental Torsion. Where diaphragms are not flexible, the design shall include the inherent torsional moment (M_t) (kip or kN) resulting from the location of the structure masses plus the accidental torsional moments (M_{ta}) (kip or kN) caused by assumed displacement of the center of mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied forces.

Where earthquake forces are applied concurrently in two orthogonal directions, the required 5 percent displacement of the center of mass need not be applied in both of the orthogonal directions at the same time, but shall be applied in the direction that produces the greater effect.

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The structure analyzed will require accidental torsion analysis because the diaphragms are not flexible.

Inherent and Accidental Torsion (continued)

12.8.4.3 Amplification of Accidental Torsional Moment. Structures assigned to Seismic Design Category C, D, E, or F, where Type 1a or 1b torsional irregularity exists as defined in Table 12.3-1 shall have the effects accounted for by multiplying M_{ta} at each level by a torsional amplification factor (A_x) as illustrated in Fig. 12.8-1 and determined from the following equation:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \quad (12.8\text{-}14)$$

where

δ_{max} = the maximum $\boxed{\text{displacement}}$ at Level x (in. or mm) computed assuming $A_x = 1$

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x computed assuming $A_x = 1$ (in. or mm)

EXCEPTION: The accidental torsional moment need not be amplified for structures of light-frame construction.

The torsional amplification factor (A_x) is not required to exceed 3.0. The more severe loading for each element shall be considered for design.

Excerpt of ASCE 7 showing requirements for accidental torsion.

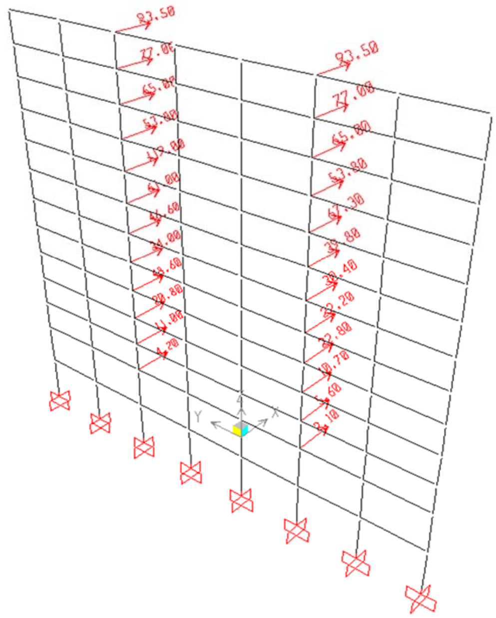
Determine Configuration Irregularities
Horizontal Irregularities

TABLE 12.3-1 HORIZONTAL STRUCTURAL IRREGULARITIES

	Irregularity Type and Description	Reference Section	Seismic Design Category Application
1a.	**Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.8.4.3 12.7.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F C, D, E, and F B, C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	**Extreme Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D B, C, and D
2.	**Reentrant Corner Irregularity** is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
3.	**Diaphragm Discontinuity Irregularity** is defined to exist where there are diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
4.	**Out-of-Plane Offsets Irregularity** is defined to exist where there are discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.	12.3.3.4 12.3.3.3 12.7.3 Table 12.6-1 16.2.2	D, E, and F B, C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
5.	**Nonparallel Systems-Irregularity** is defined to exist where the vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic force–resisting system.	12.5.3 12.7.3 Table 12.6-1 Section 16.2.2	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

Three dimensional structural analysis is required to determine if the structure has torsion irregularities. In the analysis, the ELF loads determined earlier are applied at a 5% eccentricity as required. Note that the torsion irregularity calculations are based on interstory DRIFT, not story displacement. On the other hand, torsional amplification (when required) is based on story displacement, not drift.

Application of Equivalent Lateral Forces (X Direction)



Forces in Kips

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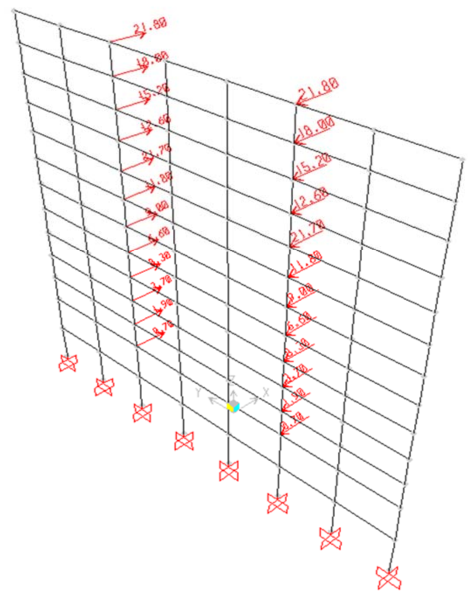
In the analysis the direct lateral load and the torsional loads are applied separately. The direct loading is shown here. These forces have been computed to represent center of mass loading on the diaphragms. A similar set of forces (not shown) were computed in the Y direction.

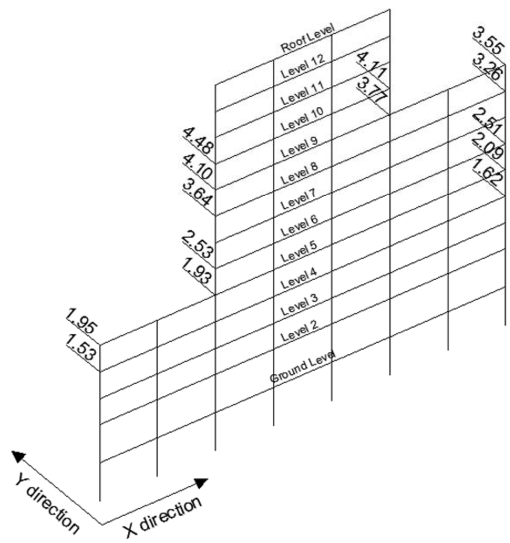
Application of Torsional Forces (Using X-Direction Lateral Forces)



Forces in Kips

These forces represent the accidental torsion due to X-direction forces applied at a 5% eccentricity. A similar set of forces (not shown) were computed for the Y direction loading.

Stations for Monitoring Drift for Torsion Irregularity Calculations with ELF Forces Applied in Y Direction



This slide shows the stations for which displacements were calculated to determine torsional irregularity due to lateral forces applied in the Y direction.

Results of Torsional Irregularity Calculations For ELF Forces Applied in X Direction

Table 4.1-5a Computation for torsional irregularity with ELF loads acting in X direction and Torsional Moment applied Counterclockwise

Level	$\delta 1$ (in)	$\delta 2$ (in)	$\Delta 1$ (in)	$\Delta 2$ (in)	Δavg (in)	Δmax (in)	$\Delta max/\Delta avg$	Irregularity
R	7.27	6.15	0.34	0.29	0.31	0.34	1.08	None
12	6.93	5.87	0.48	0.42	0.45	0.48	1.07	None
11	6.44	5.45	0.60	0.51	0.55	0.60	1.07	None
10	5.85	4.93	0.66	0.56	0.61	0.66	1.08	None
9	5.19	4.37	0.65	0.54	0.59	0.65	1.10	None
8	4.54	3.84	0.69	0.58	0.64	0.69	1.09	None
7	3.84	3.26	0.70	0.59	0.65	0.70	1.09	None
6	3.14	2.67	0.69	0.58	0.63	0.69	1.09	None
5	2.46	2.09	0.60	0.50	0.55	0.60	1.09	None
4	1.86	1.60	0.59	0.50	0.55	0.59	1.08	None
3	1.27	1.10	0.58	0.49	0.53	0.58	1.08	None
2	0.69	0.61	0.69	0.61	0.65	0.69	1.06	None

1.0 in. = 25.4 mm

Result: There is not a Torsional Irregularity for Loading in the X Direction

There is no torsional irregularity for loading in the X direction.

Results of Torsional Irregularity Calculations For ELF Forces Applied in Y Direction

Table 4.1-5b Computation for torsional irregularity with ELF loads acting in Y direction, and Torsional Moment applied Clockwise

Level	δ1 (in)	δ2 (in)	Δ1 (in)	Δ2 (in)	Δavg (in)	Δmax (in)	Δmax/Δavg	Irregularity
R	5.19	4.77	0.15	0.14	0.15	0.15	1.03	None
12	5.03	4.63	0.25	0.23	0.24	0.25	1.03	None
11	4.79	4.40	0.31	0.29	0.30	0.31	1.04	None
10	4.48	4.11	0.38	0.34	0.36	0.38	1.06	None
9	4.10	3.77, 3.55	0.46	0.28	0.37	0.46	1.24	Irregularity
8	3.64	3.26	0.54	0.36	0.45	0.54	1.20	None
7	3.09	2.90	0.56	0.39	0.47	0.56	1.18	None
6	2.53	2.51	0.60	0.42	0.51	0.60	1.18	None
5	1.93, 1.95	2.09	0.41	0.47	0.44	0.47	1.06	None
4	1.53	1.62	0.47	0.50	0.48	0.50	1.03	None
3	1.07	1.12	0.47	0.50	0.48	0.50	1.03	None
2	0.60	0.63	0.60	0.63	0.61	0.63	1.03	None

1.0 in. = 25.4 mm

Result: There is a minor Torsional Irregularity for Loading in the Y Direction

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There is a very minor torsional irregularity a level 9 for loads applied in the Y direction. It would probably be best to redesign the structure to eliminate the irregularity. However, the consequences of the irregularity are not severe.

Note that the double entries for displacements in some locations (Levels 5 and 9) is due to the setbacks. This was discussed on a previous slide that showed the deflection monitoring stations for this loading.

Results of Torsional Amplification Calculations For ELF Forces Applied in Y Direction

(X Direction Results are Similar)

Table 4.1-5d Amplification Factor A_x for Accidental Torsional Moment Loads acting in the Y direction and Torsional Moment applied Clockwise

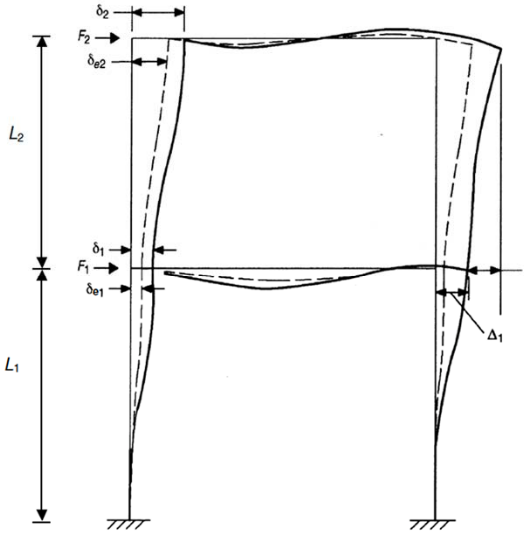
Level	δ_1 (in)	δ_2 (in)	δ_{avg} (in)	δ_{max} (in)	A_x calculated	A_x corrected
R	5.19	4.77	4.98	5.19	0.75	1.00
12	5.03	4.63	4.83	5.03	0.75	1.00
11	4.79	4.40	4.59	4.79	0.76	1.00
10	4.48	4.11	4.29	4.48	0.76	1.00
9	4.10	3.55	3.82	4.10	0.80	1.00
8	3.64	3.26	3.45	3.64	0.77	1.00
7	3.09	2.90	3.00	3.09	0.74	1.00
6	2.53	2.51	2.52	2.53	0.70	1.00
5	1.95	2.09	2.02	2.09	0.74	1.00
4	1.53	1.62	1.58	1.62	0.73	1.00
3	1.07	1.12	1.10	1.12	0.73	1.00
2	0.60	0.63	0.61	0.63	0.73	1.00

1.0 in. = 25.4 mm

Result: Amplification of Accidental Torsion Need not be Considered

No torsional amplification is required for this structure.

Drift and Deformation



Story Level 2

F_2	=	strength-level design earthquake force
δ_{e2}	=	elastic displacement computed under strength-level design earthquake forces
δ_2	=	$C_d\ \delta_{e2}/I_E$ = amplified displacement
Δ_2	=	$(\delta_{e2} - \delta_{e1})\ C_d/I_E \le \Delta_a$ (Table 12.12-1)

Story Level 1

F_1	=	strength-level design earthquake force
δ_{e1}	=	elastic displacement computed under strength-level design earthquake forces
δ_1	=	$C_d\ \delta_{e1}/I_E$ = amplified displacement
Δ_1	=	$\delta_1 \le \Delta_a$ (Table 12.12-1)

Δ_i	=	Story Drift
Δ_i/L_i	=	Story Drift Ratio
δ_2	=	Total Displacement

FIGURE 12.8-2 STORY DRIFT DETERMINATION

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This is directly from ASCE 7. No additional commentary required.

Drift and Deformation (Continued)

12.12 DRIFT AND DEFORMATION

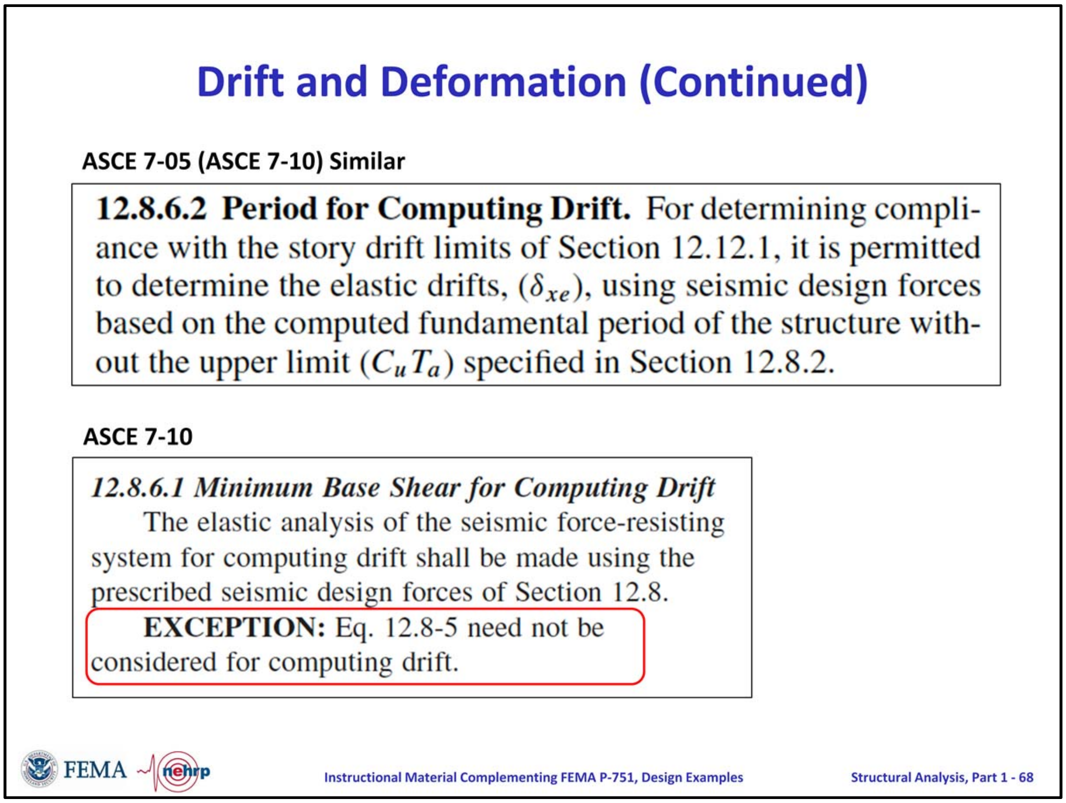
12.12.1 Story Drift Limit. The design story drift (Δ) as determined in Sections 12.8.6, 12.9.2, or 16.1, shall not exceed the allowable story drift (Δ_a) as obtained from Table 12.12-1 for any story. For structures with significant torsional deflections, the maximum drift shall include torsional effects. For structures assigned to Seismic Design Category C, D, E, or F having horizontal irregularity Types 1a or 1b of Table 12.3-1, the design story drift, Δ, shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the story under consideration.

Not strictly Followed in this Example due to *very minor* torsion irregularity

TABLE 12.12-1 ALLOWABLE STORY DRIFT, $\Delta_a^{a,b}$

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}{}^{c}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

ASCE 7 states that for structures with “Significant Torsional Deflections”, the maximum drift shall include torsional effects. This language is vague, because it is not clear what “significant” is, and it is not clear how torsional effects should be included (inherent torsion, inherent plus accidental torsion, inherent plus amplified accidental torsion?). The authors assumed that this structure did not have significant torsional deflections, and thereby did not include accidental torsion loading in the analysis. Inherent torsion was, of course, included in the analysis. Deflections were computed at center of mass, not at the edges of the building. As shown later, this building is relatively stiff, and the drifts are significantly less than allowed. Had the drifts been closer to the allowed drifts, it might have been appropriate to determine the drifts at the edge of the building.



This issue was discussed in earlier slides. In the present analysis drift is computed on the basis of lateral forces computed using Eqn. 12.8-3 with $T = C_u T_a$. Has the drifts from this analysis exceeded the allowable drift, a reanalysis would have been permitted using the periods for Rayleigh or Eigenvalue analysis.

Computed Drifts in X Direction

Table 4.1-7 ELF Drift for Building Responding in X Direction

Level	1 Total drift from SAP2000 (in.)	2 Story drift from SAP2000 (in.)	3 Amplified story drift (in.)	4 Amplified drift times 0.568 (in.)	5 Allowable drift (in.)
R	6.67	0.32	1.74	0.99	3.00
12	6.35	0.45	2.48	1.41	3.00
11	5.90	0.56	3.07	1.75	3.00
10	5.34	0.62	3.39	1.92	3.00
9	4.73	0.58	3.20	1.82	3.00
8	4.15	0.63	3.47	1.97	3.00
7	3.52	0.64	3.54	2.01	3.00
6	2.87	0.63	3.47	1.97	3.00
5	2.24	0.54	2.95	1.67	3.00
4	1.71	0.54	2.97	1.69	3.00
3	1.17	0.53	2.90	1.65	3.00
2	0.64	0.64	3.51	2.00	4.32

Column 4 adjusts for *Standard* Eq. 12.8-3 (for drift) vs 12.8-5 (for strength).
1.0 in. = 25.4 mm.

C_d Amplified drift based on forces from Eq. 12.8-5

Modified for forces based on Eq. 12.8-3

The drifts have been determined on the basis of lateral loads from Eqn. 12.8-5, and have been modified to be consistent with Eqn 12.8-3, which uses C_uT_a as the period of vibration. Note that the computed periods from Eigenvalue analysis could have been used instead, and the resulting drifts would be even lower.

If the drifts had been based on lateral forces consistent with Eqn. 12.8-5, the drifts would have been excessive. However, the computed drifts are significantly less than the limits when the adjustment is made.

Computed Drifts in Y Direction

Table 4.1-8 ELF Drift for Building Responding in Y Direction

Level	1 Total drift from SAP2000 (in.)	2 Story drift from SAP2000 (in.)	3 Amplified story drift (in.)	4 Amplified drift times 0.568 (in.)	5 Allowable drift (in.)
R	4.86	0.15	0.81	0.46	3.00
12	4.71	0.24	1.30	0.74	3.00
11	4.47	0.30	1.64	0.93	3.00
10	4.17	0.36	1.96	1.11	3.00
9	3.82	0.37	2.05	1.16	3.00
8	3.44	0.46	2.54	1.44	3.00
7	2.98	0.48	2.64	1.50	3.00
6	2.50	0.48	2.62	1.49	3.00
5	2.03	0.45	2.49	1.42	3.00
4	1.57	0.48	2.66	1.51	3.00
3	1.09	0.48	2.64	1.50	3.00
2	0.61	0.61	3.35	1.90	4.32

Column 4 adjusts for *Standard* Eq. 12.8-3 (for drift) versus Eq. 12.8-5 (for strength).
1.0 in. = 25.4 mm.

C_d Amplified drift based on forces from Eq. 12.8-5

Modified for forces based on Eq. 12.8-3

The comments on the previous slide apply to this slide as well.

P-Delta Effects

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

Eq. 12.8-16*

The drift Δ in Eq. 12.8-16 is drift from ELF analysis, multiplied by C_d and divided by *I*.

*The importance factor *I* was inadvertently left out of Eq. 12.8-16 in ASCE 7-05. It is properly included in ASCE 7-10.

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

Eq. 12.8-17

The term β in Eq. 12.8-17 is essentially the inverse of the Computed story over-strength.

P-Delta Effects for modal response spectrum analysis and modal response history analysis are checked using the ELF procedure indicated on this slide.

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This slide provides the basic expressions used in P-Delta analysis. Note that the deflections "Delta" in equation 12.8-16 are for the analysis <u>without</u> P-Delta effects included.

P-Delta Effects

Table 4.1-11 Computation of P-Delta Effects for X Direction Response

Level	h_{sx} (in.)	Δ (in.)	P_D (kips)	P_L (kips)	P_T (kips)	P_X (kips)	V_X(kips)	θ_X
R	150	1.74	1656.5	315.0	1971.5	1971.5	186.9	0.022
12	150	2.48	1595.8	315.0	1910.8	3882.3	340.9	0.034
11	150	3.07	1595.8	315.0	1910.8	5793.1	470.8	0.046
10	150	3.39	1595.8	315.0	1910.8	7703.9	578.4	0.055
9	150	3.20	3403.0	465.0	3868.0	11571.9	764.7	0.059
8	150	3.47	2330.8	465.0	2795.8	14367.7	865.8	0.070
7	150	3.54	2330.8	465.0	2795.8	17163.5	942.5	0.078
6	150	3.47	2330.8	465.0	2795.8	19959.3	998.8	0.084
5	150	2.95	4323.8	615.0	4938.8	24898.1	1070.2	0.083
4	150	2.97	3066.1	615.0	3681.1	28579.2	1101.7	0.093
3	150	2.90	3066.1	615.0	3681.1	32260.3	1118.2	0.101
2	216	3.51	3097.0	615.0	3712.0	35972.3	1124.5	0.095

1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

Marginally exceeds limit of 0.091 using β=1.0. θ would be less than θ_{max} if actual β were computed and used.

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For this structure the maximum stability factor of 0.091 is marginally exceeded for the bottom three levels of the structure. However, this is based on conservative estimates of live load, and the "Beta" factor used to compute θ_{max} was taken conservatively as 1.0. Actual values of this factor are likely to be significantly less than 1.0, so the analysis will proceed as if P-Delta provisions are satisfied.

Orthogonal Loading Requirements

12.5.4 Seismic Design Categories D through F. Structures assigned to Seismic Design Category D, E, or F shall, as a minimum, conform to the requirements of Section 12.5.3.

12.5.3 Seismic Design Category C. Loading applied to structures assigned to Seismic Design Category C shall, as a minimum, conform to the requirements of Section 12.5.2 for Seismic Design Category B and the requirements of this section. *Structures that have horizontal structural irregularity Type 5* in Table 12.3-1 shall the following procedure [for ELF Analysis]:

Continued on Next Slide

This structure has a type 5 horizontal irregularity under the provisions of ASCE 7-05, but not under ASCE 7-10. This is because the symmetry requirement included in the nonparallel system irregularity has been eliminated (see Table 12.3-1). As this example was written principally for accordance with ASCE 7-05, orthogonal loading is included. Additionally, this structure uses a perimeter moment frame, and the corner columns will be affected by loading from two directions.

Orthogonal Loading Requirements (continued)

Orthogonal Combination Procedure. The structure shall be analyzed using the equivalent lateral force analysis procedure of Section 12.8 with the loading applied independently in any two orthogonal directions and the most critical load effect due to direction of application of seismic forces on the structure is permitted to be assumed to be satisfied if components and their foundations are designed for the following combination of prescribed loads: ***100 percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction***; the combination requiring the maximum component strength shall be used.

The 100/30 percent loading is used for this structure.

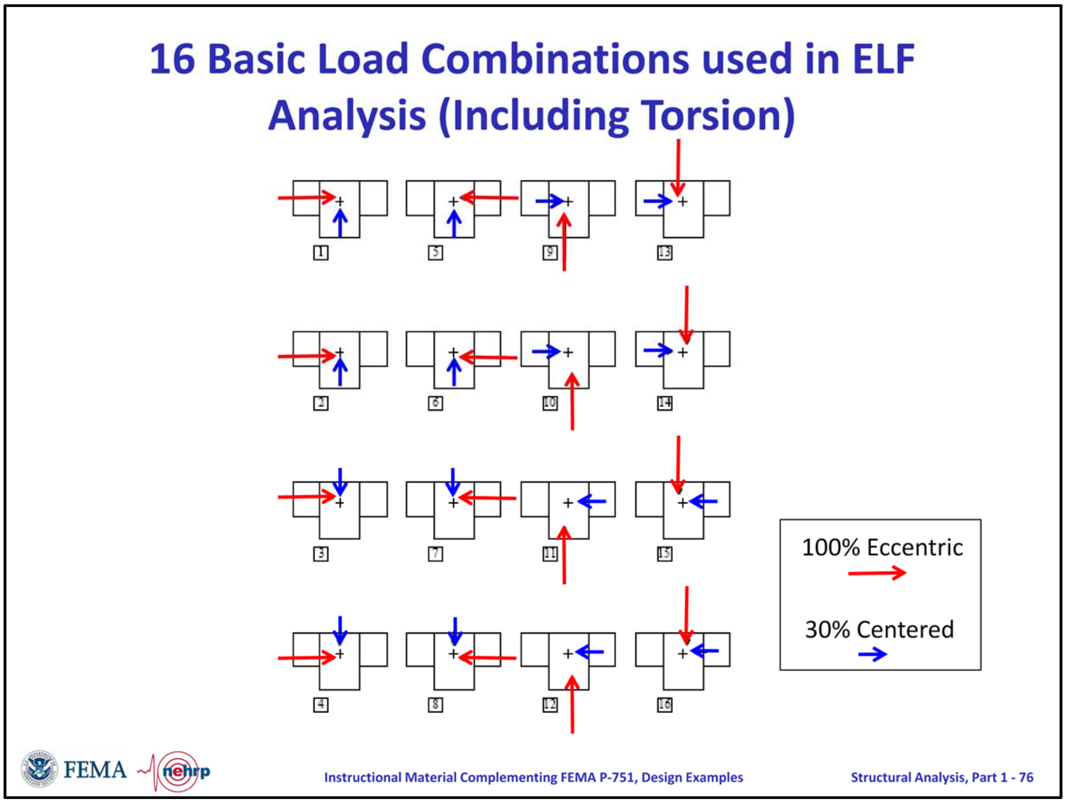
ASCE 7-05 Horizontal Irregularity Type 5

Nonparallel Systems-Irregularity is defined to exist where the vertical lateral force-resisting elements are not parallel to ***or symmetric about*** the major orthogonal axes of the seismic force–resisting system.

The system in question clearly has nonsymmetrical lateral force resisting elements so a Type 5 Irregularity exists, and orthogonal combinations are required. *Thus, 100%-30% procedure given on the previous slide is used.*

Note: The words “or symmetric about” have been removed from the definition of a Type 5 Horizontal Irregularity in ASCE 7-10. Thus, the system under consideration does not have a Type 5 irregularity in ASCE 7-10.

The modification in ASCE 7-10 is significant, because many structures deemed irregular due to nonsymmetric systems in ASCE 7-05 are longer irregular. Thus, orthogonal loading may no longer be required for may SDC D, E, and F structures.



This slide shows the 16 basic seismic loadings that are required when accidental torsion and orthogonal loading requirements are met. When the two basic gravity loadings are included, it is seen that 32 seismic load cases are required.

Combination of Load Effects

$$1.2D + 1.0E + 0.5L + \cancel{0.2S}$$

$$0.9D + 1.0E + \cancel{1.6H}$$

$$E = E_h + E_v$$

$$E_h = \rho Q_E \qquad (\rho = 1.0)$$

$$E_v = 0.2S_{DS} \qquad (S_{DS}=0.833g)$$

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These are the basic gravity plus seismic load combinations. The snow and hydrostatic loads are not applicable, and are crossed out. There would be no requirement to use the similar load combinations including the overstrength factor Ομεγα$_0$, so this is not shown. The two gravity loadings in combination with the 16 seismic loads produce a total of 32 seismic load combinations. This is in addition to the gravity only and gravity plus wind combinations that would be required.

Redundancy Factor

12.3.4.2 Redundancy Factor, *ρ, for Seismic Design Categories* D through F. For structures assigned to Seismic Design Category D, E, or F, ρ shall equal 1.3 unless one of the following two conditions is met, whereby ρ is permitted to be taken as 1.0:

a) Each story resisting more than 35 percent of the base shear in the direction of interest shall comply with Table 12.3-3.

} See next slide

b) Structures that are regular in plan at all levels provided that the seismic force–resisting systems consist of at least two bays of seismic force–resisting perimeter framing on each side of the structure in each orthogonal direction at each story resisting more than 35 percent of the base shear. The number of bays for a shear wall shall be calculated as the length of shear wall divided by the story height or two times the length of shear wall divided by the story height for light framed construction.

Structure is NOT regular at all Levels.

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The structure is not regular, so only subparagraph (a) applies.

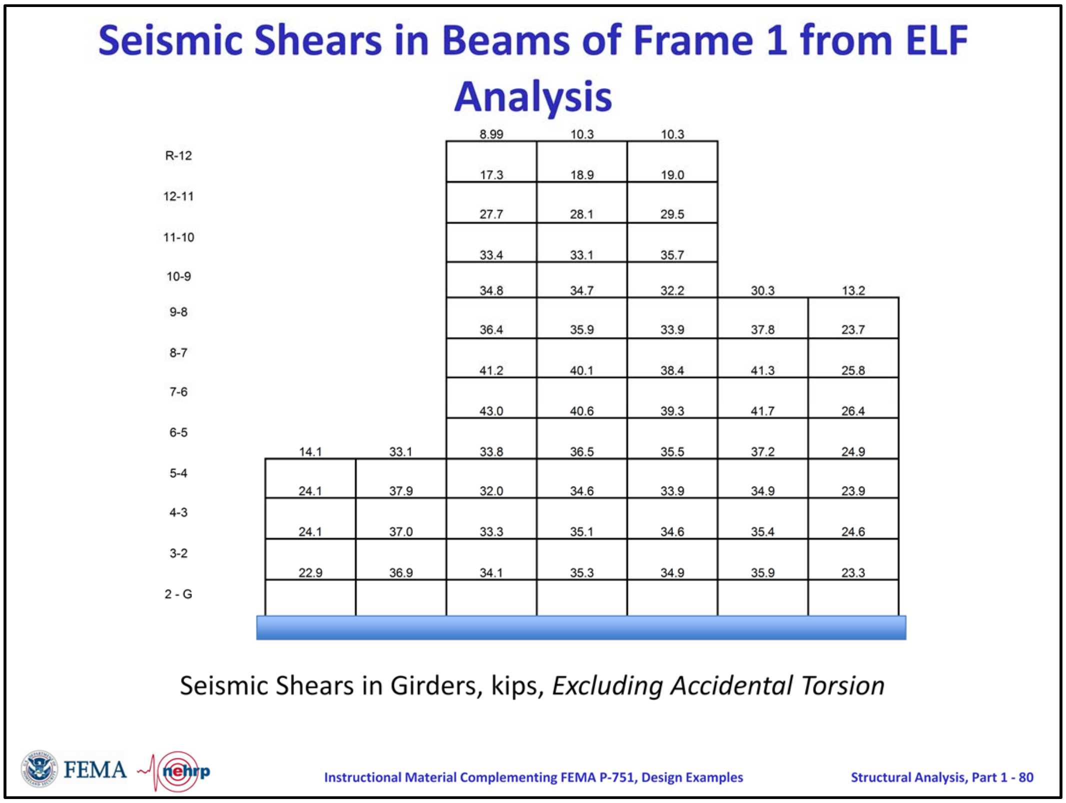
Redundancy, Continued

TABLE 12.3-3 REQUIREMENTS FOR EACH STORY RESISTING MORE THAN 35% OF THE BASE SHEAR

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).

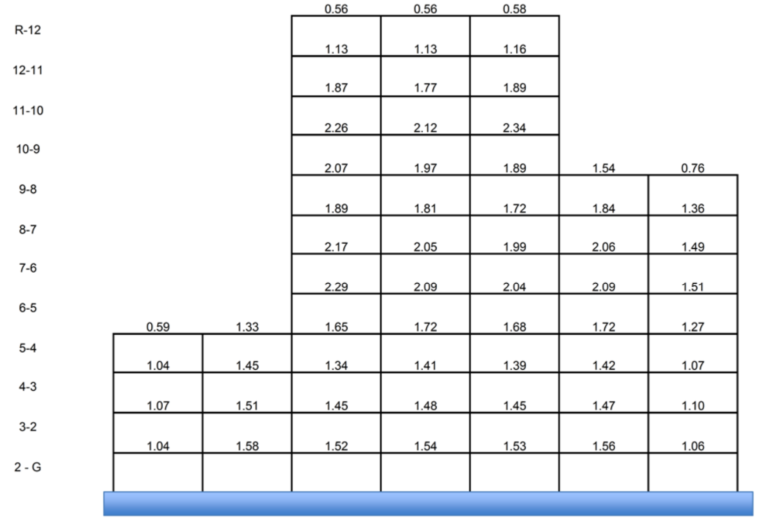
It can be seen by inspection that removal of one beam in this structure will not result in a result in a significant loss of strength or lead to an extreme torsional irregularity. **Hence ρ = 1 for this system**. (This is applicable to ELF, MRS, and MRH analyses).

It is very clear that the removal of a single beam in this highly redundant perimeter moment frame structure would not cause an extreme torsional irregularity or a reduction in strength of more than 33 percent. These redundancy calculations would only be required for systems with only one or two bays of resisting frame in each direction. Thus, for the Stockton building, the ρ factor is taken as 1.0.



This slide provides the maximum beam shears in Frame 1 of the structure. These include lateral loads only, without gravity and without accidential torsion. Accidental torsional forces are included separately (see next slide). Separation of the torsional forces facilitates the comparison of the results from the three methods of analysis. Additionally, the torsional forces determined in the ELF analysis would be used (with possibly some reduction) in the response spectrum and response history calculations.

Seismic Shears in Beams of Frame 1 from ELF Analysis



Seismic Shears in Girders, kips, *Accidental Torsion Only*

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These are the accidental torsion forces on Frame 1. See also the comments for the previous slide.

Note that these forces are applicable to all three analysis methods because both the MRS and the MRH methods apply accidental torsion using the ELF procedure.

Overview of Presentation

- Describe Building
- Describe/Perform steps common to all analysis types
- Overview of Equivalent Lateral Force analysis
- **Overview of Modal Response Spectrum Analysis**
- Overview of Modal Response History Analysis
- Comparison of Results
- Summary and Conclusions

Title slide. No commentary provided.

Modal Response Spectrum Analysis
Part 1: Analysis

1. Develop ***Elastic*** response spectrum (Sec. 11.4.5)
2. Develop adequate finite element model (Sec. 12.7.3)
3. Compute modal frequencies, effective mass, and mode shapes
4. Determine number of modes to use in analysis (Sec. 12.9.1)
5. Perform modal analysis in each direction, combining each direction's results by use of CQC method (Sec. 12.9.3)
6. Compute Equivalent Lateral Forces (ELF) in each direction (Sec. 12.8.1 through 12.8.3)
7. Determine accidental torsions (Sec 12.8.4.2), amplified if necessary (Sec. 12.8.4.3)
8. Perform static Torsion analysis

These are the basic steps in a modal response spectrum analysis. Many of the steps are required for ELF analysis, so the amount of additional work is not substantial, and the additional work that is required (steps 6, 7, and 8) is generally done by the computer.

Modal Response Spectrum Analysis
Part 2: Drift and P-Delta for Systems *Without* Torsion Irregularity

1. Multiply all dynamic displacements by C_d/R (Sec. 12.9.2).
2. Compute SRSS of interstory drifts based on displacements at center of mass at each level.
3. Check drift Limits in accordance with Sec. 12.12 and Table 12.2-1. Note: drift Limits for Special Moment Frames in SDC D and above must be divided by the Redundancy Factor (Sec. 12.12.1.1)
4. Perform P-Delta analysis using Equivalent Lateral Force procedure
5. Revise structure if necessary

Note: when centers of mass of adjacent levels are not vertically aligned the drifts should be based on the difference between the displacement at the upper level and the displacement of the point on the level below which is the vertical projection of the center of mass of the upper level. (This procedure is included in ASCE 7-10.)

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Note that P-Delta effects are handled in exactly the same manner as for ELF. Thus, P-Delta effects should not be included when computing the mode shapes and frequencies.

ASCE 7 requires that drift be checked at the center of mass, but this is not easily done when the masses are not vertically aligned. The new ASCE 7-10 provision addresses the problem. Drifts computed at the corners of the building would be conservative (exceeding the requirements for center of mass calculations) and are much easier to calculate. The vertical alignment approach described in ASCE 7-10 was used in the example.

Modal Response Spectrum Analysis Part 2: Drift and P-Delta for Systems *With* Torsion Irregularity

1. Multiply all dynamic displacements by C_d/R (Sec. 12.9.2).
2. Compute SRSS of story drifts based on displacements at the *edge* of the building
3. Using results from the static torsion analysis, determine the drifts at the same location used in Step 2 above. Torsional drifts may be based on the computed period of vibration (without the $C_u T_a$ limit). Torsional drifts should be based on computed displacements multiplied by C_d and divided by *I*.
4. Add drifts from Steps 2 and 3 and check drift limits in Table 12.12-1.
   Note: Drift limits for special moment frames in SDC D and above must be divided by the Redundancy Factor (Sec. 12.12.1.1)
5. Perform P-Delta analysis using Equivalent Lateral Force procedure
6. Revise structure if necessary

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This procedure would be used for a system with significant torsional displacements. It was not required for the building under consideration.

Modal Response Spectrum Analysis
Part 3: Obtaining Member Design Forces

1. Multiply all dynamic force quantities by I/R (Sec. 12.9.2)
2. Determine dynamic base shears in each direction
3. Compute scale factors for each direction (Sec. 12.9.4) and apply to respective member force results in each direction
4. Combine results from two orthogonal directions, if necessary (Sec. 12.5)
5. Add member forces from static torsion analysis (Sec. 12.9.5). Note that static torsion forces may be scaled by factors obtained in Step 3
6. Determine redundancy factor (Sec. 12.3.4)
7. Combine seismic and gravity forces (Sec. 12.4)
8. Design and detail structural components

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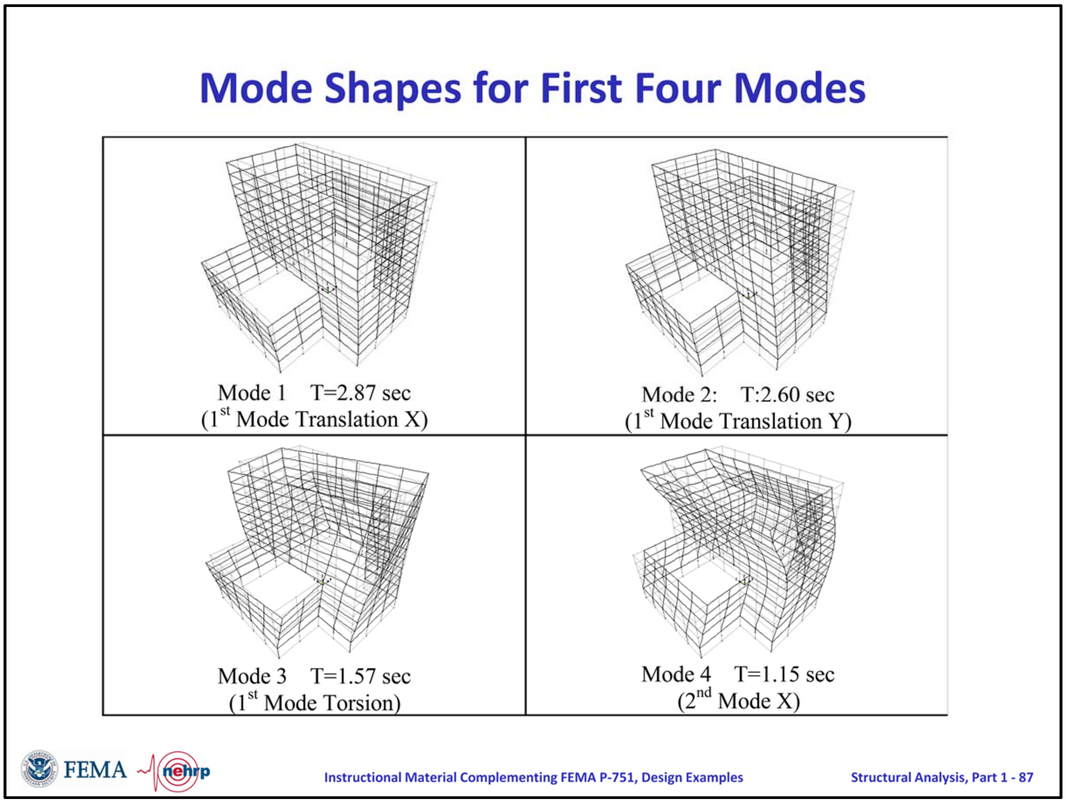
One of the complications to response spectrum analysis is that member forces must generally be scaled up such that the base shear from the response spectrum analysis is not less than 85 percent of the ELF shears. Accidental torsional forces would be scaled using the same factor.

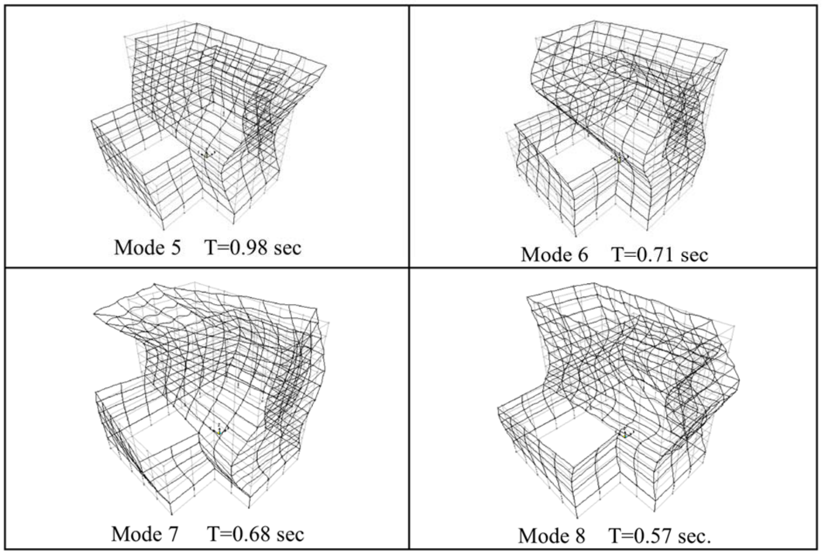
This 85 percent rule provides some incentive for performing MRS analysis because the 15 percent reduction in base shear is usually allowed. This is due to the fact that the computed periods based on Eigenvalue analysis are generally much longer than periods computed using C_uT_a. Note, however, that in the unlikely case that the MRS analysis produces shears greater than those from ELF, there are no provisions for scaling the results down to the ELF forces.

Deflections computed from MRS analysis may be used directly, without scaling. This is consistent with allowing deflections to be based on the computed period, without the C_uT_a limit, in ELF analysis.



This plot simply shows the first four mode shapes and associated periods from the SAP 2000 analysis.

Mode Shapes for Modes 5-8



The next four mode shapes are shown here. There is significant lateral-torsional interaction because of the setbacks.

Number of Modes to Include in Response Spectrum Analysis

12.9.1 Number of Modes

An analysis shall be conducted to determine the natural modes of vibration for the structure. 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.

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This provision is based on the assumption that the heavy basement walls and ground level slab are not modeled in the system. The basement has significant mass, and that mass does not appear until modes 100 and above in this structure. Had the structure been modeled as fixed at the base of the first story columns, only the first dozen or so modes would be required to capture 85 percent of the mass in each direction.

The authors believe that the ASCE 7 language should be modified to account for such problems. Furthermore, a sufficient modes should be used to capture 85 percent of the torsional mass.

Effective Masses for First 12 Modes

Table 4.1-13 Computed Periods and Effective Mass Factors (Lower Modes)

Mode	Period (seconds)	Effective Mass Factor, [Accum Mass Factor]		
		X Translation	Y Translation	Z Rotation
1	2.87	0.6446 [0.64]	0.0003 [0.00]	0.0028 [0.00]
2	2.60	0.0003 [0.65]	0.6804 [0.68]	0.0162 [0.02]
3	1.57	0.0035 [0.65]	0.0005 [0.68]	0.5806 [0.60]
4	1.15	0.1085 [0.76]	0.0000 [0.68]	0.0000 [0.60]
5	0.975	0.0000 [0.76]	0.0939 [0.78]	0.0180 [0.62]
6	0.705	0.0263 [0.78]	0.0000 [0.78]	0.0271 [0.64]
7	0.682	0.0056 [0.79]	0.0006 [0.79]	0.0687 [0.71]
8	0.573	0.0000 [0.79]	0.0188 [0.79]	0.0123 [0.73]
9	0.434	0.0129 [0.80]	0.0000 [0.79]	0.0084 [0.73]
10	0.387	0.0048 [0.81]	0.0000 [0.79]	0.0191 [0.75]
11	0.339	0.0000 [0.81]	0.0193 [0.81]	0.0010 [0.75]
12	0.300	0.0089 [0.82]	0.0000 [0.81]	0.0003 [0.75]

12 Modes Appears to be Insufficient

Only 82 percent of the total lateral mass is captured by mode 12. The third mode is principally torsion, and with 12 modes only 75 percent of the torsional mass is captured.

Effective Masses for Modes 108-119

Table 4.1-14 Computed Periods and Effective Mass Factors (Higher Modes)

Mode	Period (seconds)	Effective Mass Factor, [Accum Effective Mass]		
		X Translation	Y Translation	Z Rotation
108	0.0693	0.0000 [0.83]	0.0000 [0.83]	[illegible]
109	0.0673	0.0000 [0.83]	0.0000 [0.83]	[illegible]
110	0.0671	0.0000 [0.83]	0.0354 [0.86]	0.0000 [0.79]
111	0.0671	0.0000 [0.83]	0.0044 [0.87]	0.0000 [0.79]
112	0.0669	0.0000 [0.83]	0.1045 [0.97]	0.0000 [0.79]
113	0.0663	0.0000 [0.83]	0.0000 [0.97]	0.0000 [0.79]
114	0.0646	0.0000 [0.83]	0.0000 [0.97]	0.0000 [0.79]
115	0.0629	0.0000 [0.83]	0.0000 [0.97]	0.0000 [0.79]
116	0.0621	0.0008 [0.83]	0.0010 [0.97]	0.0000 [0.79]
117	0.0609	0.0014 [0.83]	0.0009 [0.97]	0.0000 [0.79]
118	0.0575	0.1474 [0.98]	0.0000 [0.97]	0.0035 [0.80]
119	0.0566	0.0000 [0.98]	0.0000 [0.97]	0.0000 [0.80]

Virtually the Same as 12 Modes

118 Modes Required to Capture Dynamic Response of Stiff Basement Level and Grade Level Slab

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At mode 108 the lateral mass has only marginally increased. At mode 112 the mass associated with the basement finally appears in the Y direction. This mass shows up at mode 118 in the X direction. The torsional mass has still not reached 85 percent, even at mode 119.

Effective Masses for First 12 Modes

Table 4.1-13 Computed Periods and Effective Mass Factors (Lower Modes)

Mode	Period (seconds)	Effective Mass Factor, [Accum Mass Factor]		
		X Translation	Y Translation	Z Rotation
1	2.87	0.6446 [0.64]	0.0003 [0.00]	0.0028 [0.00]
2	2.60	0.0003 [0.65]	0.6804 [0.68]	0.0162 [0.02]
3	1.57	0.0035 [0.65]	0.0005 [0.68]	0.5806 [0.60]
4	1.15	0.1085 [0.76]	0.0000 [0.68]	0.0000 [0.60]
5	0.975	0.0000 [0.76]	0.0939 [0.78]	0.0180 [0.62]
6	0.705	0.0263 [0.78]	0.0000 [0.78]	0.0271 [0.64]
7	0.682	0.0056 [0.79]	0.0006 [0.79]	0.0687 [0.71]
8	0.573	0.0000 [0.79]	0.0188 [0.79]	0.0123 [0.73]
9	0.434	0.0129 [0.80]	0.0000 [0.79]	0.0084 [0.73]
10	0.387	0.0048 [0.81]	0.0000 [0.79]	0.0191 [0.75]
11	0.339	0.0000 [0.81]	0.0193 [0.81]	0.0010 [0.75]
12	0.300	0.0089 [0.82]	0.0000 [0.81]	0.0003 [0.75]

12 Modes are Actually Sufficient to Represent the Dynamic Response of the *Above Grade* Structure

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Only the first 12 modes were used in the analysis, as this captured more than 90 percent of the mass in each direction.

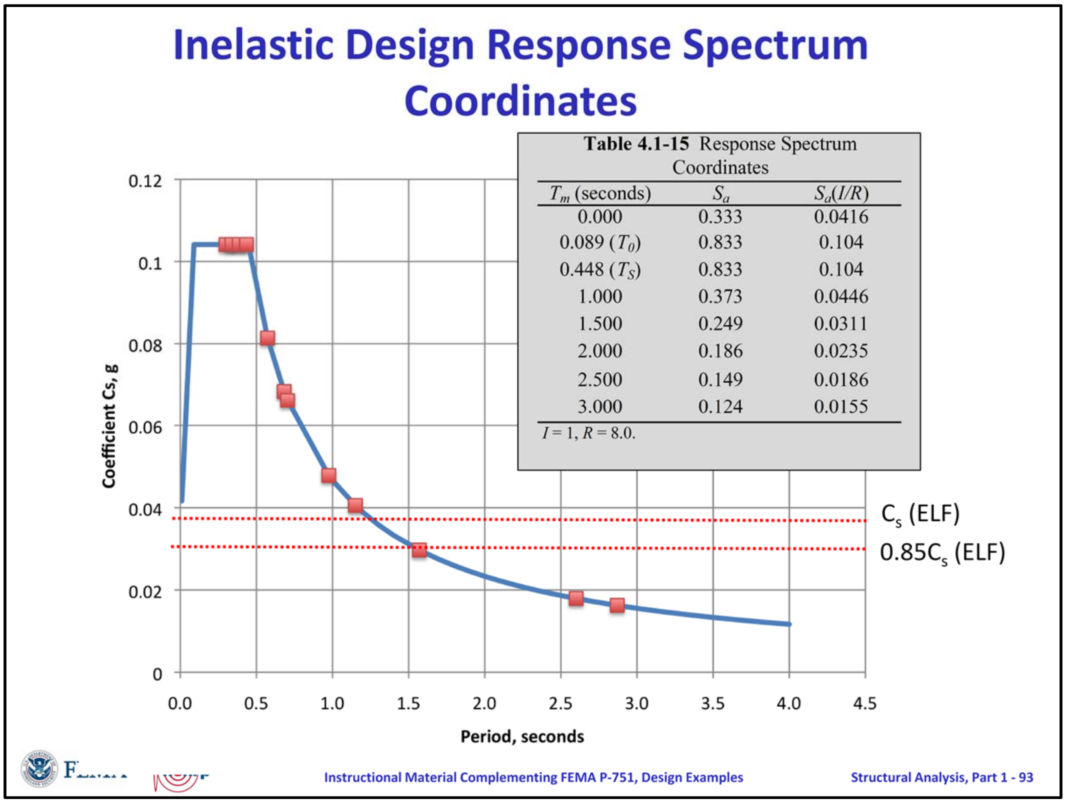

Table 4.1-15 Response Spectrum Coordinates

T_m (seconds)	S_a	$S_a(I/R)$
0.000	0.333	0.0416
0.089 (T_0)	0.833	0.104
0.448 (T_S)	0.833	0.104
1.000	0.373	0.0446
1.500	0.249	0.0311
2.000	0.186	0.0235
2.500	0.149	0.0186
3.000	0.124	0.0155

$I = 1$, $R = 8.0$.

These are the response spectrum ordinates used in the analysis. The *R* factor is included in the spectrum.

Scaling of Response Spectrum Results (ASCE 7-05)

12.9.4 Scaling Design Values of Combined Response.

A base shear (V) shall be calculated in each of the two orthogonal horizontal directions using the calculated fundamental period of the structure T in each direction and the procedures of Section 12.8, except where the calculated fundamental period exceeds $(C_u)(T_a)$, then $(C_u)(T_a)$ shall be used in lieu of T in that direction. Where the combined response for the modal base shear (V_t) is less than 85 percent of the calculated base shear (V) using the equivalent lateral force procedure, the forces, but not the drifts, shall be multiplied by

$$0.85\frac{V}{V_t}$$

where

- V = the equivalent lateral force procedure base shear, calculated in accordance with this section and Section 12.8
- V_t = the base shear from the required modal combination

Note: If the ELF base shear is governed by Eqn. 12.5-5 or 12.8-6 the force V shall be based on the value of C_s calculated by Eqn. 12.5-5 or 12.8-6, as applicable.

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A question arises when the ELF base shear is based on the absolute minimum of 0.01*W*. The *Standard* is not clear on whether the scaling would effectively lower this minimum to 0.0085*W*. In the author's opinion, the scaling of the MRS results should not produce a base shear less than the absolute minimum of 0.01*W*.

Scaling of Response Spectrum Results (ASCE 7-10)

12.9.4.2 Scaling of Drifts

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

$$0.85 \frac{C_s W}{V_t}$$

Drifts need be scaled only if the ELF base shear is based on equation 12.8-6. This is consistent with the requirements of ELF.

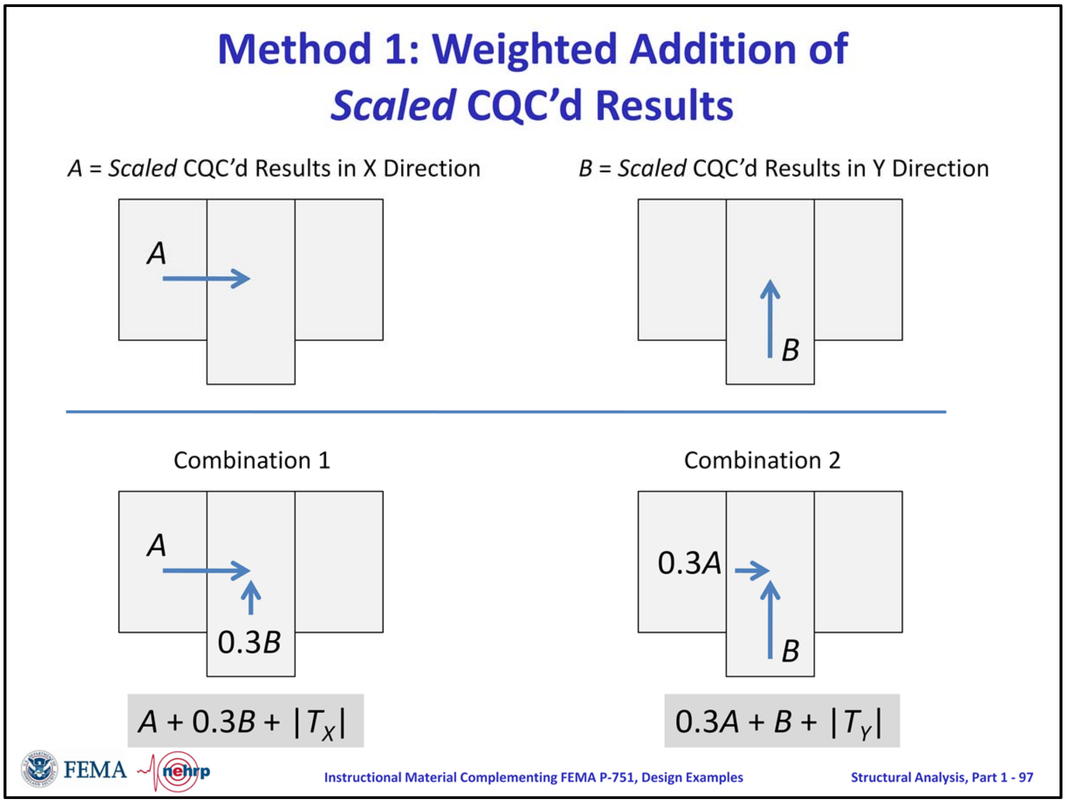
Scaled Static Torsions

Apply Torsion as a Static Load. Torsions can be Scaled to 0.85 times Amplified* EFL Torsions if the Response Spectrum Results are Scaled.

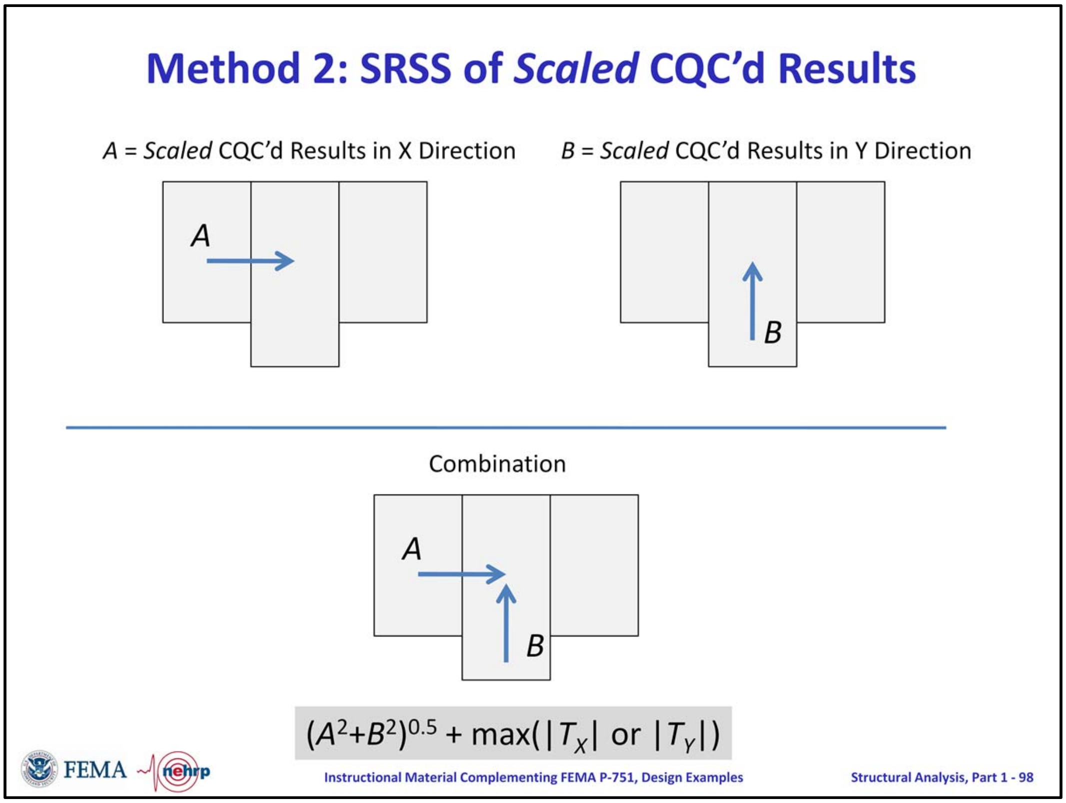
* See Sec. 12.9.5. Torsions must be amplified because they are applied statically, not dynamically.

The MRS analysis automatically accounts for inherent torsion. Accidental torsion is generally included by direct addition of the the ELF static torsion effects, scaled in accordance with the 85 percent rule, if applicable. Note that when static accidental torsions are used, they may need to be amplified in accordance with Section 12.8.4.3.

Accidental torsion need not be amplified if is is included in the dynamic analysis, presumably by physically shifting of the mass eccentricities. See Section 12.9.5 of ASCE 7.



This is one of two approaches to handle orthogonal loading in MRS analysis. The approach shown on the next slide is preferred.



This approach, while not specifically described in ASCE 7, is preferred. This method is somewhat more conservative than the method given on the previous slide because it will provide a uniform resistance for "all possible angles of attack" of the earthquake. Programs like SAP2000 and ETABS can automatically implement this procedure (or the procedure shown on the previous slide).

Computed Story Shears and Scale Factors from Modal Response Spectrum Analysis

Table 4.1-16 Story Shears from Modal Response Spectrum Analysis

Story	X Direction (SF = 2.18) Unscaled Shear (kips)	X Direction (SF = 2.18) Scaled Shear (kips)	Y Direction (SF = 1.94) Unscaled Shear (kips)	Y Direction (SF = 1.94) Scaled Shear (kips)
R-12	82.7	180	77.2	150
12-11	130.9	286	132.0	256
11-10	163.8	357	170.4	330
10-9	191.4	418	201.9	392
9-8	240.1	524	265.1	514
8-7	268.9	587	301.4	585
7-6	292.9	639	328.9	638
6-5	316.1	690	353.9	686
5-4	359.5	784	405.1	786
4-3	384.8	840	435.5	845
3-2	401.4	895	462.8	898
2-G	438.1	956	492.8	956

1.0 kip = 4.45 kN.

X-Direction Scale Factor = 0.85(1124)/438.1=2.18

Y-Direction Scale Factor = 0.85(1124)/492.8=1.94

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This slide shows the modal shears for each level as computed using the MRS approach. The X direction base shear is 438.1 kips, and the Y direction shear is 492.8 kips. Thus, all of the story shears and related member forces need to be scale up to 0.85 times the ELF base shear of 1124 kips. The scale factors are 2.18 and 1.94 in the X and Y directions, respectively.

Response Spectrum Drifts in X Direction

(No Scaling Required)

Level	Total Drift from R.S. Analysis (in.)	Story Drift (in.)	Story Drift × C_d (in.)	Allowable Story Drift (in.)
R	2.23	0.12	0.66	3.00
12	2.10	0.16	0.89	3.00
11	1.94	0.19	1.03	3.00
10	1.76	0.20	1.08	3.00
9	1.56	0.18	0.98	3.00
8	1.38	0.19	1.06	3.00
7	1.19	0.20	1.08	3.00
6	0.99	0.20	1.08	3.00
5	0.80	0.18	0.97	3.00
4	0.62	0.19	1.02	3.00
3	0.43	0.19	1.05	3.00
2	0.24	0.24	1.34	4.32

1.0 in. = 25.4 mm

The modal story drifts in the second column come directly from the analysis, and are not scaled. These drifts already include the effect of *R*, which was included in the response spectrum. The story drifts are generally not equal to the difference in the total drifts, as these are determined individually in each mode and then SRSSed. The story drifts are multiplied by C_d in the fourth column. The final C_d scaled drifts are significantly less than the allowable drifts, indicating that this structure is probably too stiff as currently designed.

These displacements will be compared to the ELF and MRH displacements at the end of this slide set.

Response Spectrum Drifts in Y Direction

(No Scaling Required)

Level	Total Drift from R.S. Analysis (in.)	Story Drift (in.)	Story Drift × C_d (in.)	Allowable Story Drift (in.)
R	1.81	0.06	0.32	3.00
12	1.76	0.09	0.49	3.00
11	1.67	0.11	0.58	3.00
10	1.56	0.12	0.67	3.00
9	1.44	0.13	0.70	3.00
8	1.31	0.16	0.87	3.00
7	1.15	0.17	0.91	3.00
6	0.99	0.17	0.92	3.00
5	0.92	0.17	0.93	3.00
4	0.65	0.19	1.04	3.00
3	0.46	0.20	1.08	3.00
2	0.26	0.26	1.44	4.32

1.0 in. = 25.4 mm

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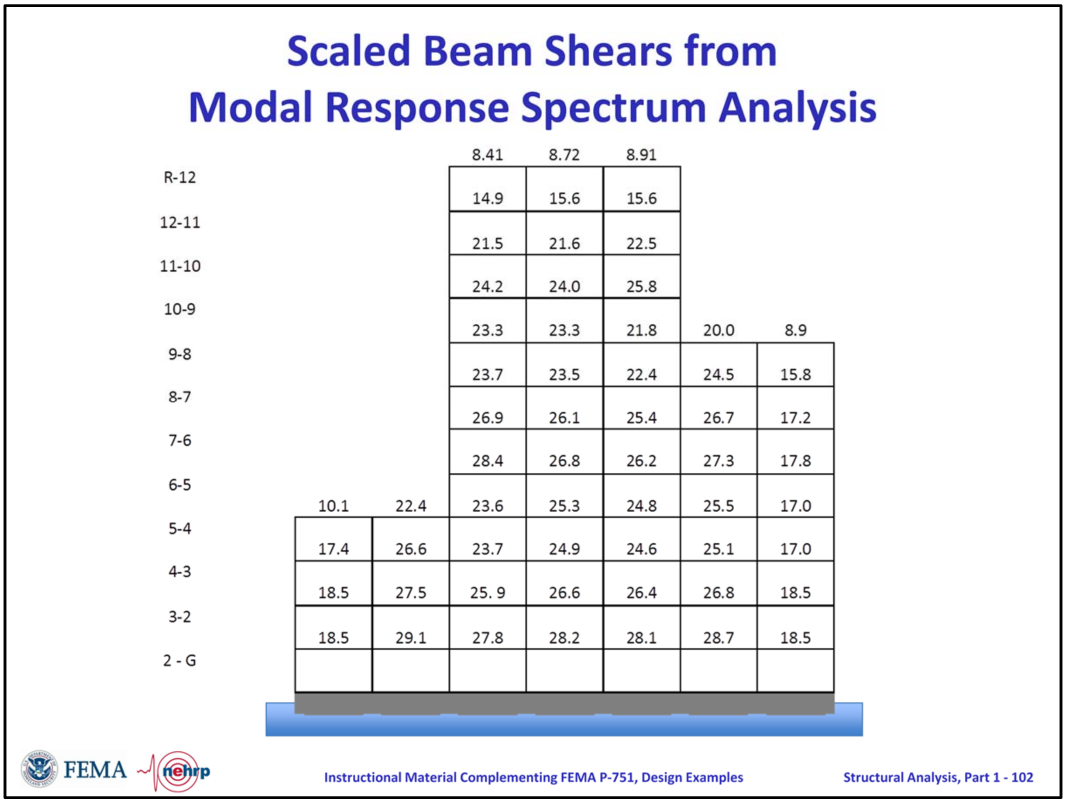
See previous slide for discussion



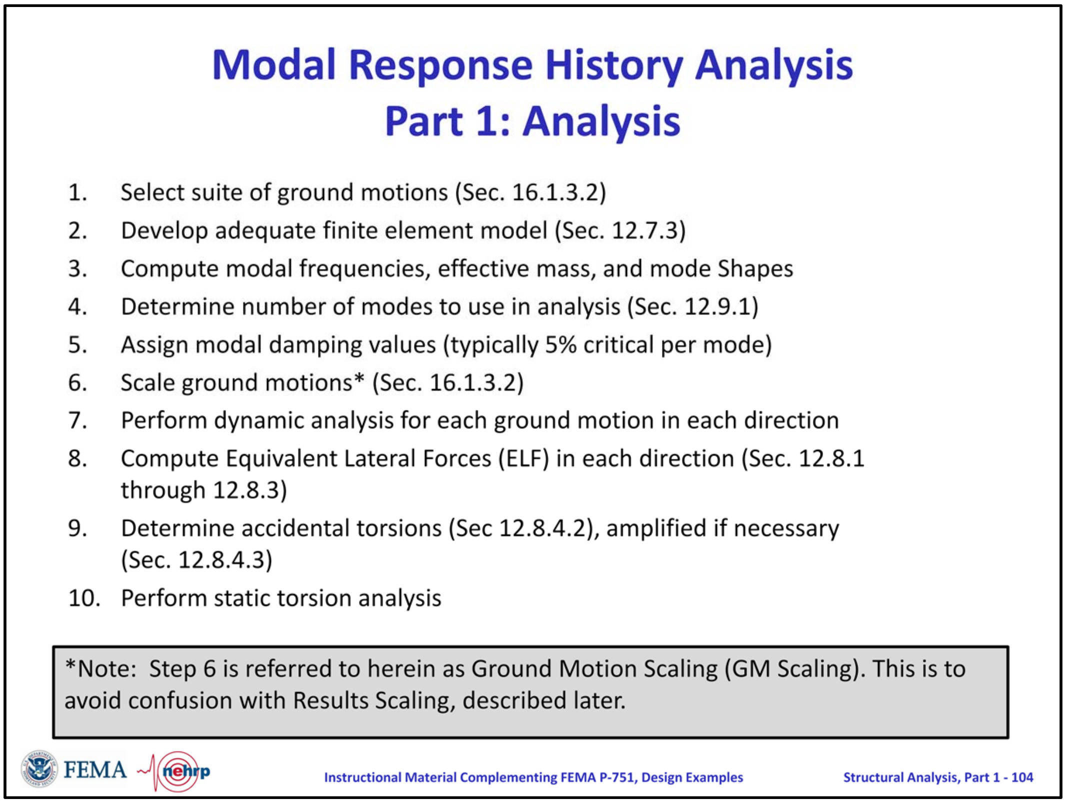
The beam shears are found in each mode, and then combined by SRSS. The shears shown on this slide have been scaled such that they are consistent with (85% scaled) scaled base shears.

These shears will be compared to the ELF and MRH shears at the end of this slide set.

Overview of Presentation

- Describe Building
- Describe/Perform steps common to all analysis types
- Overview of Equivalent Lateral Force analysis
- Overview of Modal Response Spectrum Analysis
- **Overview of Modal Response History Analysis**
- Comparison of Results
- Summary and Conclusions

Title slide.



This slide shows the basic steps in the Modal Response History method. Many of the steps are the same as required for ELF or MRS analysis. The largest “new” item is the selection and scaling of the ground motions, and the running of the dynamic analysis.

Modal Response History Analysis Part 2: Drift and P-Delta for Systems *Without* Torsion Irregularity

1. Multiply all dynamic displacements by C_d/R (omitted in ASCE 7-05).
2. Compute story drifts based on displacements at center of mass at each level
3. If 3 to 6 ground motions are used, compute envelope of story drift at each level in each direction (Sec. 16.1.4)
4. If 7 or more ground motions are used, compute average story drift at each level in each direction (Sec. 16.1.4)
5. Check drift limits in accordance with Sec. 12.12 and Table 12.2-1. Note: drift limits for Special Moment Frames in SDC D and above must be divided by the Redundancy Factor (Sec. 12.12.1.1)
6. Perform P-Delta analysis using Equivalent Lateral Force procedure
7. Revise structure if necessary

Note: when centers of mass of adjacent levels are not vertically aligned the drifts should be based on the difference between the displacement at the upper level and the displacement of the point on the level below which is the vertical projection of the center of mass of the upper level.(This procedure is included in ASCE 7-10.)

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This slide lists the steps required to determine drift. Drifts are taken directly from the analysis, and need not be scaled other than by the ratio of C_d/R. All drifts are calculated at the center of mass.

Note that P-Delta effects are checked using the same procedure as used for the ELF and MRS analysis. Therefore, P-Delta effects should not be included in the dynamic analysis.

Modal Response History Analysis Part 2: Drift and P-Delta for Systems *With* Torsion Irregularity

1. Multiply all dynamic displacements by C_d/R (omitted in ASCE 7-05).
2. Compute story drifts based on displacements at edge of building at each level
3. If 3 to 6 ground motions are used, compute envelope of story drift at each level in each direction (Sec. 16.1.4)
4. If 7 or more ground motions are used, compute average story drift at each level in each direction (Sec. 16.1.4)
5. Using results from the static torsion analysis, determine the drifts at the same location used in Steps 2-4 above. Torsional drifts may be based on the computed period of vibration (without the C_uT_a limit). Torsional drifts should be based on computed displacements multiplied by C_d and divided by *I*.
6. Add drifts from Steps (3 or 4) and 5 and check drift limits in Table 12.12-1. Note: Drift limits for special moment frames in SDC D and above must be divided by the Redundancy Factor (Sec. 12.12.1.1)
7. Perform P-Delta analysis using Equivalent Lateral Force procedure
8. Revise structure if necessary

The only difference between this slide and the previous slide is that when there are significant torsional deflections, the drift should be computed at the corner of the building. This was not done here as the structure did not have a significant torsional response.

Modal Response History Analysis
Part 3: Obtaining Member Design Forces

1. Multiply all dynamic member forces by *I*/*R*
2. Determine dynamic base shear histories for each earthquake in each direction
3. Determine Result Scale Factors* for each ground motion in each direction, and apply to response history results as appropriate
4. Determine design member forces by use of envelope values if 3 to 6 earthquakes are used, or as averages if 7 or more ground motions are used.
5. Combine results from two orthogonal directions, if necessary (Sec. 12.5)
6. Add member forces from static torsion analysis (Sec. 12.9.5). Note that static torsion forces may be scaled by factors obtained in Step 3
7. Determine redundancy factor (Sec. 12.3.4)
8. Combine seismic and gravity forces (Sec. 12.4)
9. Design and detail structural components

*Note: Step 3 is referred to herein as Results Scaling (GM Scaling). This is to avoid confusion with Ground Motion Scaling, described earlier.

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This is the procedure for determining design seismic member forces. The significant point in this slide is that the scaling to 85 percent of the design base shear will be required if the dynamic base shears are less than the 85 percent of the ELF shears.

Selection of Ground Motions for MRH Analysis

16.1.3.2 Three-Dimensional Analysis

Where three-dimensional analyses are performed, ground motions shall consist of pairs of appropriate horizontal ground motion acceleration components that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of recorded ground motion pairs is not available, appropriate simulated ground motion pairs are permitted to be used to make up the total number required.

The ASCE 7-10 requirements for selecting ground motion are shown here. Selecting an appropriate number of records that satisfy the criteria can be challenging because there are few available recordings of design level ground motions.

There is a general consensus that "more is better" when running response history analysis. If fact, ASCE 7 rewards the engineer when seven or more motions are used as the average response among the seven may be used when determining design values. The peak response must be used if less than seven motions are included in the analysis. One must not use fewer than three records under any circumstances.

3D Scaling Requirements, ASCE 7-10

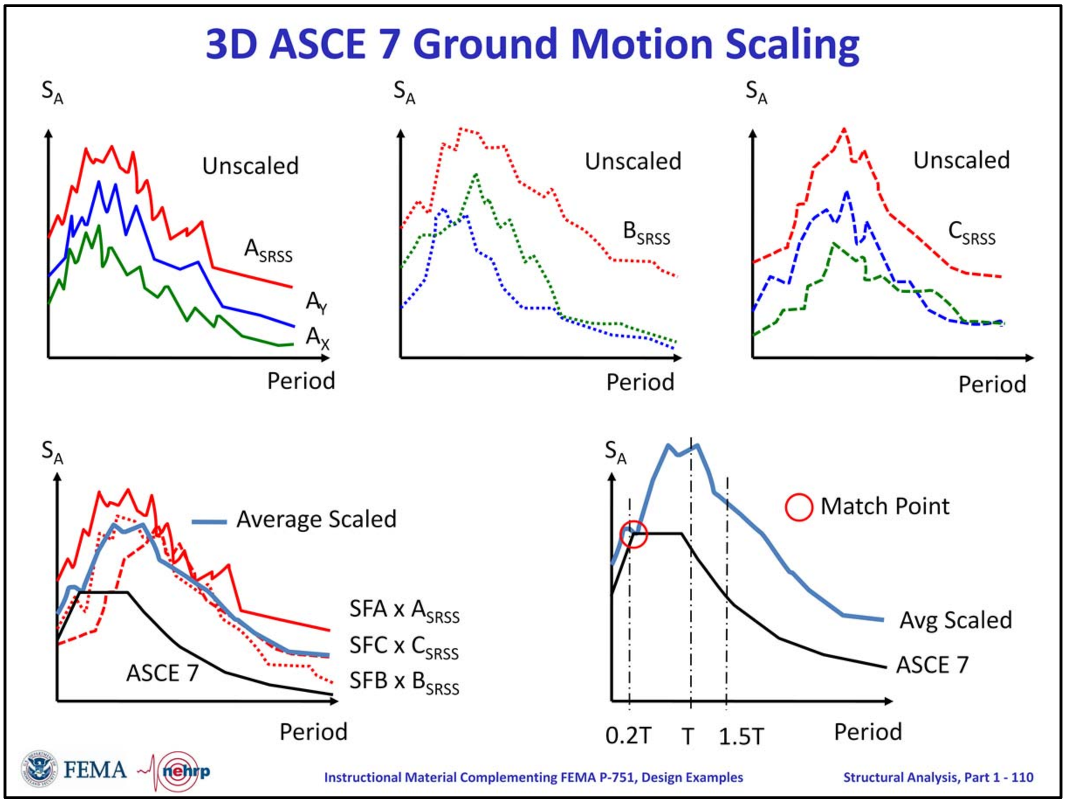
For each pair of horizontal ground motion components, a square root of the sum of the squares (SRSS) spectrum shall be constructed by taking the SRSS of the 5 percent-damped response spectra for the scaled components (where an identical scale factor is applied to both components of a pair). Each pair of motions shall be scaled such that in the period range from 0.2*T* to 1.5*T, the average of the SRSS spectra from all* horizontal component pairs does not fall below the corresponding ordinate of the response spectrum used in the design, determined in accordance with Section 11.4.5.

ASCE 7-05 Version:
does not fall below 1.3 times the corresponding ordinate of the design response spectrum, determined in accordance with Section 11.4.5 by more than 10 percent.

The scaling requirements for the ground motions are based on ASCE 7-10. This results in somewhat lower scale factors than used in ASCE 7-05.

Here it is important to note that that there are several sets of scale factors applied in the analysis:

(1) Scaling by ratio of *I*/*R*
(2) Ground motion scaling as indicated above
(3) Scaling to 85% of ELF base shear



Ground motions must be scaled to be compatible with the design spectrum. There are numerous was to do scaling, and there is no consensus as to which is the best approach.

In ASCE 7-10, the first step in scaling (for 3D analysis) is to take the square root of the sum of the squares of the 5% damped spectra for the two orthogonal components from each earthquake. Next, each of these SRSS spectra are multiplied by a scale factor. Then, the average of the three Scaled Spectra is computed. The chosen scale factors must be established such that the average spectra lies above the design spectra for the period range of 0.2*T* to 1.5*T*, where *T* is the period of vibration of the structure.

In the example, the Match Point is that point at which the scaled average scaled spectrum and the target spectrum have the same ordinate. In the example given, note how the average scaled spectral ordinate is far above the target spectrum at the structures period of vibration. This is one of the consequences in the ASCE 7 method.

Issues With Scaling Approach

- No guidance is provided on how to deal with different fundamental
  periods in the two orthogonal directions
- There are an infinite number of sets of scale factors that will satisfy the criteria. Different engineers are likely to obtain different sets of scale factors for the same ground motions.
- In linear analysis, there is little logic in scaling at periods greater than the structure's fundamental period.
- Higher modes, which participate marginally in the dynamic response, may dominate the scaling process

These points are in addition to the problem discussed in the commentary in the last slide.

Regarding the first point, the authors chose to scale to the average of the two first mode fundamental periods. Another choice would be to scale over the range of 0.2 times the smaller period to 1.5 times the larger period.

To some the second point is not important because it is unlikely that different engineers would use the same set of ground motions. However, the current method allows the designer to apply scale factors in a arbitrary manner, and this allows the designer to scale down "offending" ground motions.

In nonlinear analysis the periods elongate, so it makes sense to consider this when scaling. For linear analysis, the periods do not change, and there is no reason to scale at periods above T (unless one is trying to manage uncertainties related to computing T).

The final point is related to the problem illustrated in the previous slide. The higher modes dominate the scaling, even though they may contribute very little to the dynamic response.

Resolving Issues With Scaling Approach

No guidance is provided on how to deal with different fundamental periods in the two orthogonal directions:

1. Use different periods in each direction (not recommended)

2. Scale to range 0.2 T_{min} to 1.5 T_{max} where T_{min} is the lesser of the two periods and T_{max} is the greater of the fundamental
periods in each principal direction

3. Scale over the range 0.2T_{Avg} to 1.5 T_{Avg} where T_{Avg} is the average of T_{min} and T_{max}

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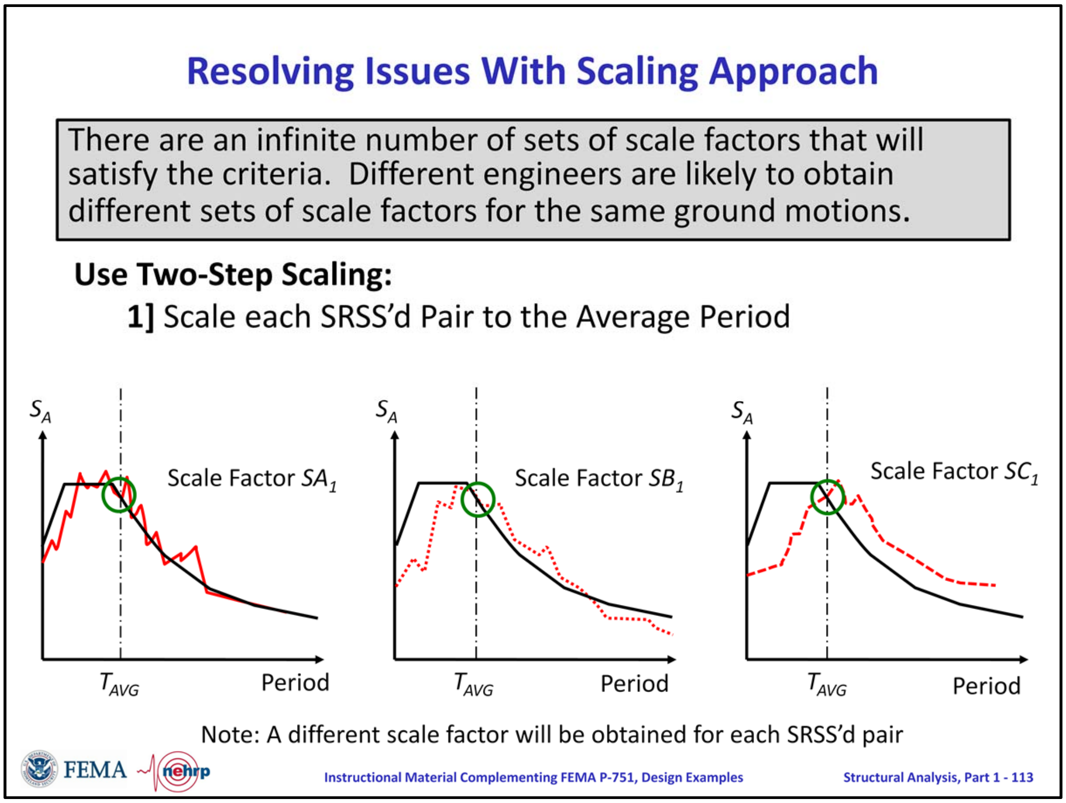
As already mentioned, the third approach was used in this example.



In this example a two-step scaling approach is used. First, the SRSS of each component pair are scaled to match the target spectrum at the period T_{avg}. This factor will be different for each of SRSS spectra.

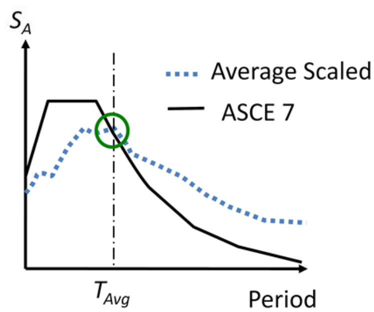
Resolving Issues With Scaling Approach

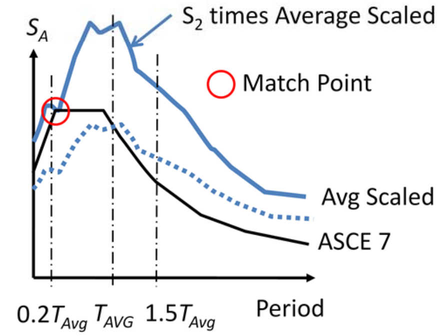
There are an infinite number of sets of scale factors that will satisfy the criteria. Different engineers are likely to obtain different sets of scale factors for the same ground motions.

Use Two-Step Scaling:

1] Scale each SRSS'd Pair to the Average Period

2] Obtain Suite Scale Factor S_2





Note: The same scale factor S_2 Applies to Each SRSS'd Pair

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The average of the scaled spectra will match the target spectrum at T_{avg}. Now a second factor is applied equally to each motion (already scaled once) such that the scaled average spectrum lies above the target spectrum from $0.2T_{avg}$ to $1.5T_{avg}$.

Resolving Issues With Scaling Approach

There are an infinite number of sets of scale factors that will satisfy the criteria. Different engineers are likely to obtain different sets of scale factors for the same ground motions.

Use Two-Step Scaling:

1] Scale each SRSS'd Pair to the Average Period

2] Obtain Suite Scale Factor S_2

3] Obtain Final Scale Factors:

Suite A: $SS_A = S_{A1} \times S_2$

Suite B: $SS_B = S_{B1} \times S_2$

Suite C: $SS_C = S_{C1} \times S_2$

The final scale factor for each motion is the product of the two scale factors. By use of this approach all engineers will arrive at the same scale factors for the same set of motions.

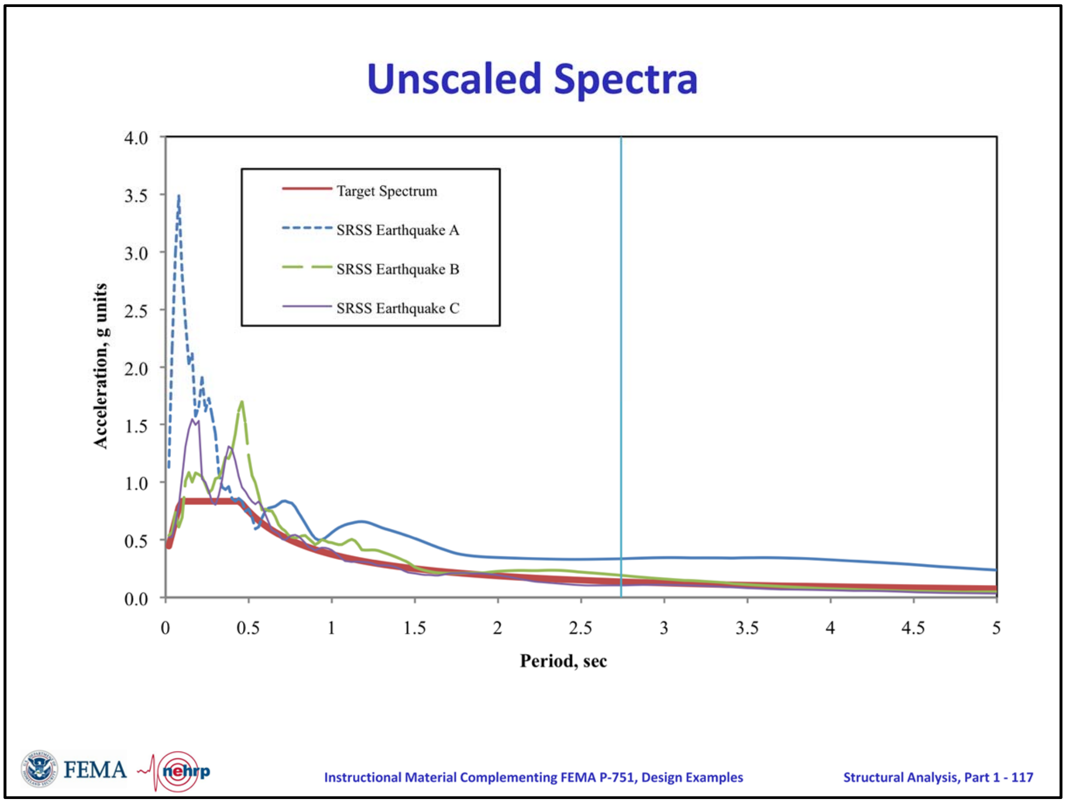
Ground Motions Used in Analysis

Table 4.1-20a. Suite of Ground Motions Used for Response History Analysis

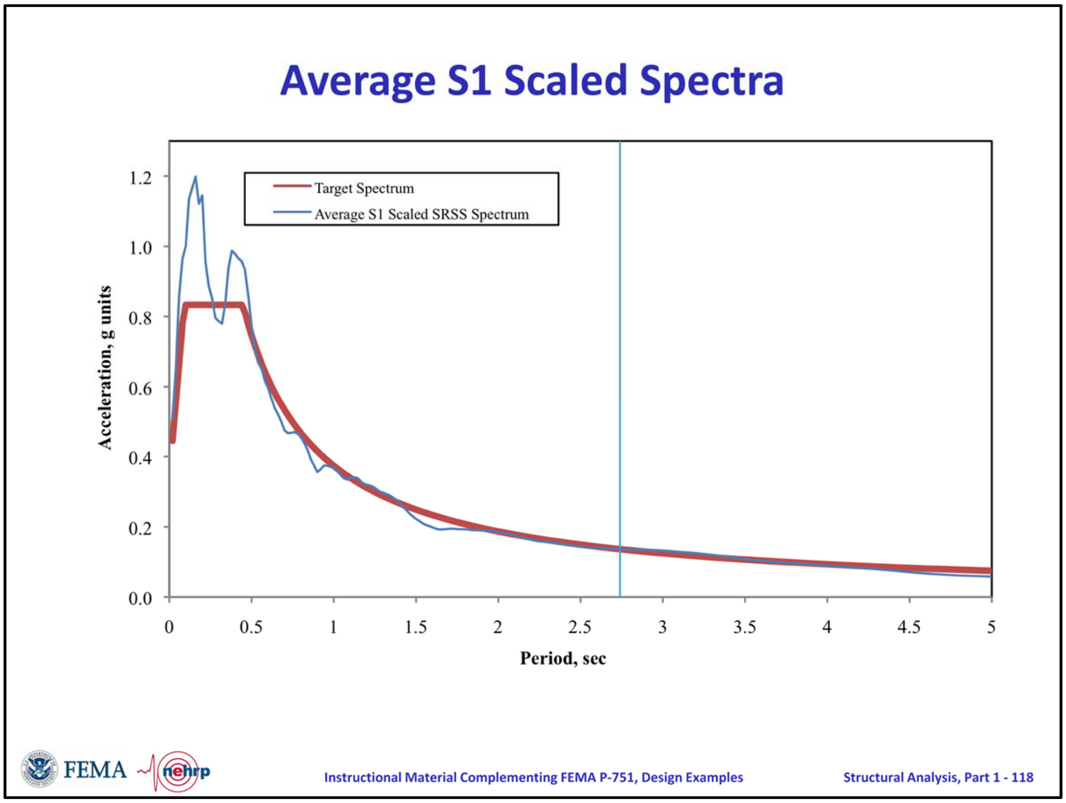
NGA Record Number	Magnitude [Epicenter Distance, km]	Site Class	Number of Points and Digitization Increment	Component Source Motion	PGA (g)	Record Name (This Example)
0879	7.28	C	9625 @ 0.005 sec	Landers/LCN260*	0.727	A00
	[44]			Landers/LCN345*	0.789	A90
0725	6.54	D	2230 @ 0.01 sec	SUPERST/B-POE270	0.446	B00
	[11.2]			SUPERST/B-POE360	0.300	B90
0139	7.35	C	1192 @ 0.02 sec	TABAS/DAY-LN	0.328	C00
	[21]			TABAS/DAY-TR	0.406	C90

* Note that the two components of motion for the Landers earthquake are apparently separated by an 85 degree angle, not 90 degrees as is traditional. It is not known whether these are true orientations, or of there is an error in the descriptions provided in the NGA database.

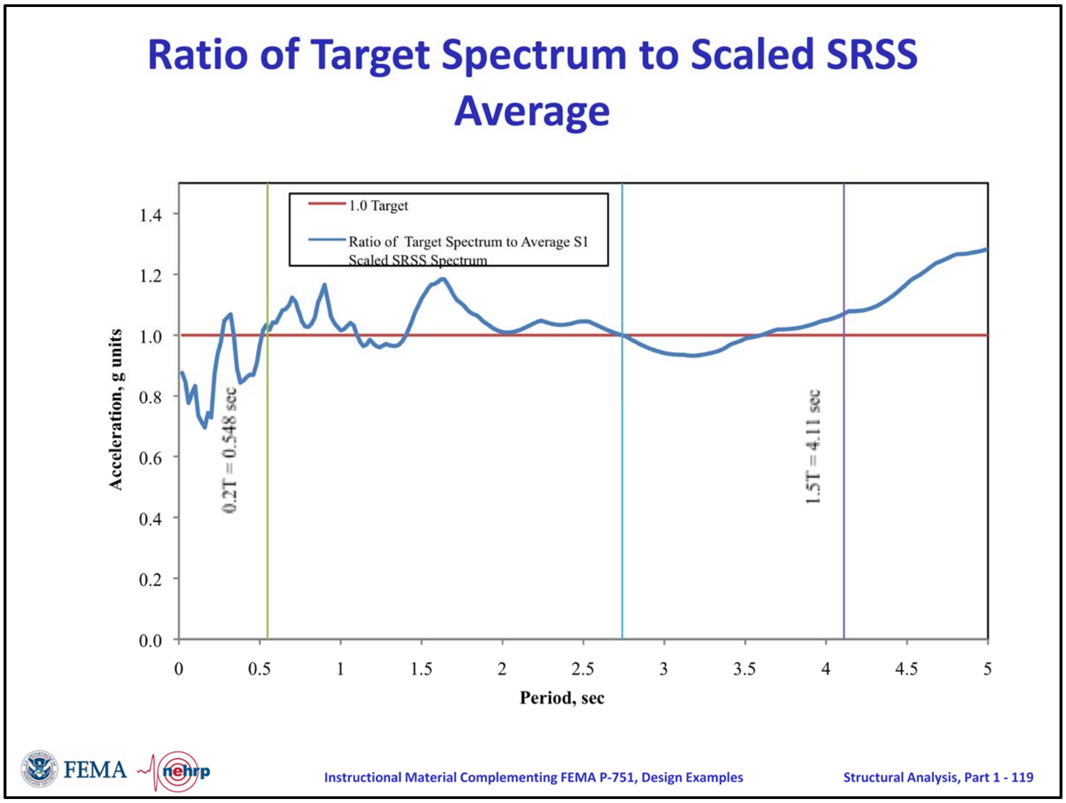
The actual records used form the analysis are shown in this slide. These records came from the PEER NGA database. They are referred to as sets A, B, and C herein.



This slide shows the unscaled SRSS spectra for each motion pair, together with the target spectrum.

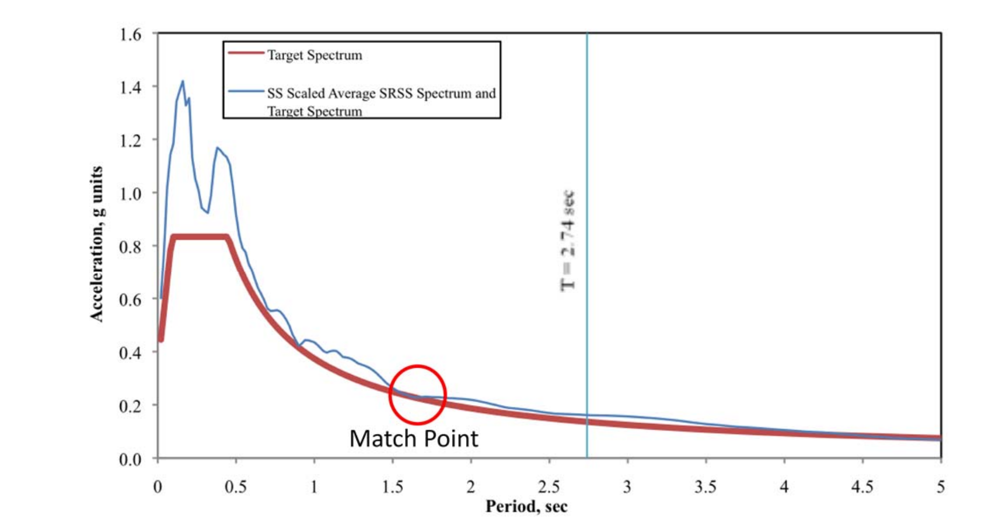


This slide shows the average of the S1 scaled spectra for the three earthquakes. Note the perfect match at the target period.



This slide shows the ratio of the target spectrum to the S1 Scaled spectra over the target period range.

Target Spectrum and SS Scaled Average



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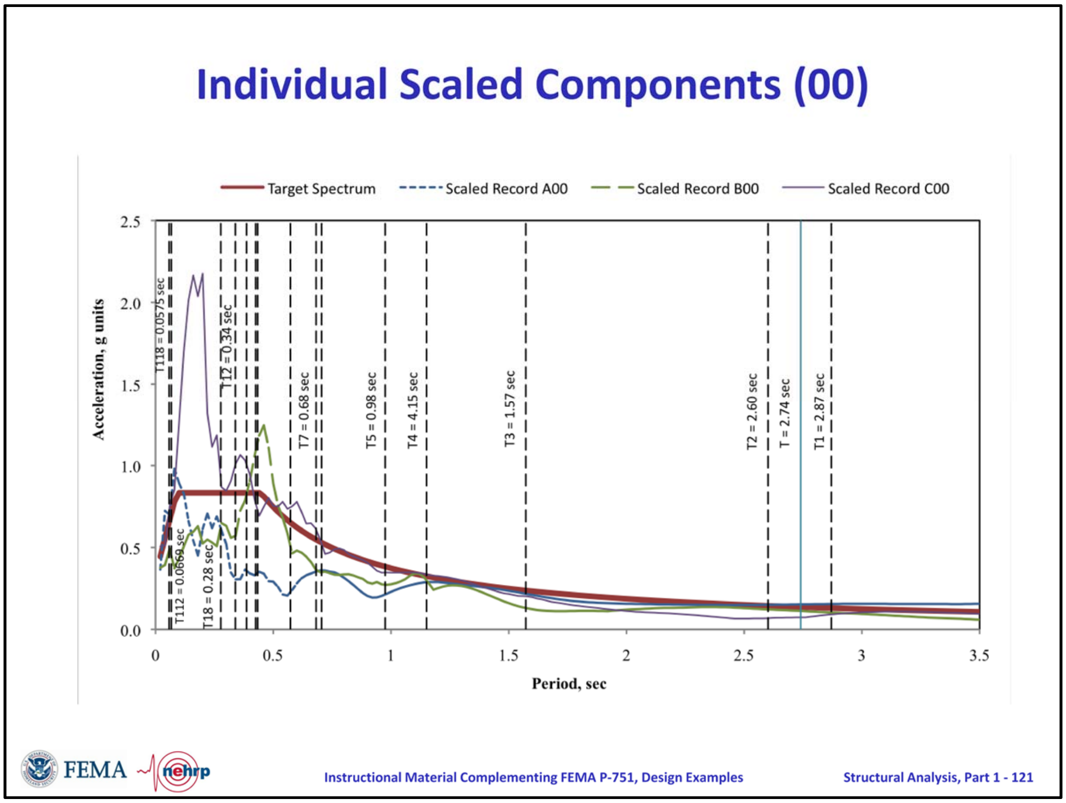
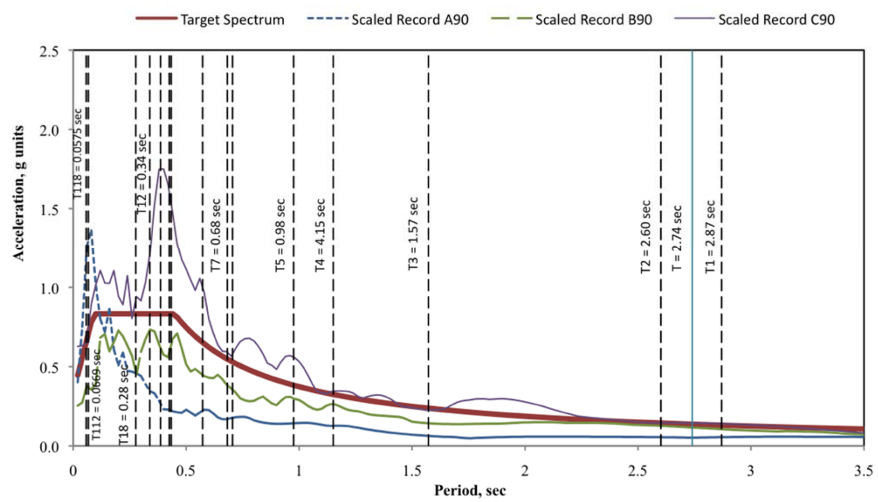
The spectrum final scaled spectrum is compared to the target spectrum here. There is a pretty good match at periods between 0.5 seconds and 5.0 seconds, but the match is not so good in the higher modes.



This plot shows the individual scaled components in the 00 direction. Note that the component spectra fall below the target spectra because the components are not "amplified" by the SRSS procedure. The SRSS of the component pairs would be closer to the target spectrum.

Individual Scaled Components (90)



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See the comment on the previous slide.

Computed Scale Factors

Table 4.1-20b. Result of 3D Scaling Process

Set No.	Designation	SRSS ordinate at $T=T_{Avg}$ (g)	Target Ordinate at $T=T_{Avg}$ (g)	S1	S2	SS
1	A00 & A90	0.335	0.136	0.407	1.184	0.482
2	B00 & B90	0.191	0.136	0.712	1.184	0.843
3	C00 & C90	0.104	0.136	1.310	1.184	1.551

This slide shows the final computed scale factors. Note that each component pair receives its own S1 factor, and all records use the same S2 factor.

Number of Modes for Modal Response History Analysis

ASCE 7-05 and 7-10 are silent on the number of modes to use in Modal Response History Analysis. It is recommended that the same procedures set forth in Section 12.9.1 for MODAL Response Spectrum Analysis be used for Response History Analysis:

12.9.1 Number of Modes

An analysis shall be conducted to determine the natural modes of vibration for the structure. **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.

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Chapter 16 of ASCE 7 does not provide guidance on the number of modes to use in modal response history analysis. It seems logical to follow the same procedures as given in Chapter 12 for modal response spectrum analysis, and this was done for the example building.

Damping for Modal Response History Analysis

ASCE 7-05 and 7-10 are silent on the amount of damping to use in Modal Response History Analysis.

Five percent critical damping should be used in all modes considered in the analysis because the Target Spectrum and the Ground Motion Scaling Procedures are based on 5% critical damping.

Chapter 16 of ASCE 7 does not provide guidance on damping in response history analysis. It seems logical to use 5% damping in each mode as this was used in the development of the response spectra. Thus, 5% was used in the example. Note, however the that use of 5% damping in nonlinear response history analysis is probably unconservative. The use of a lower value, say 2% critical, is generally recommended for nonlinear analysis.

Scaling of Results for Modal Response History Analysis (Part 1)

The structural analysis is executed using the GM scaled earthquake records in each direction. Thus, the results represent the expected elastic response of the structure. The results must be scaled to represent the expected inelastic behavior and to provide improved performance for important structures. ASCE 7-05 scaling is as follows:

1) Scale all component design forces by the factor (I/R). This is stipulated in Sec. 16.1.4 of ASCE 7-05 and ASCE 7-10.

2) Scale all displacement quantities by the factor (C_d/R). This requirement
was inadvertently omitted in ASCE 7-05, but is included in Section 16.1.4 of ASCE 7-10.

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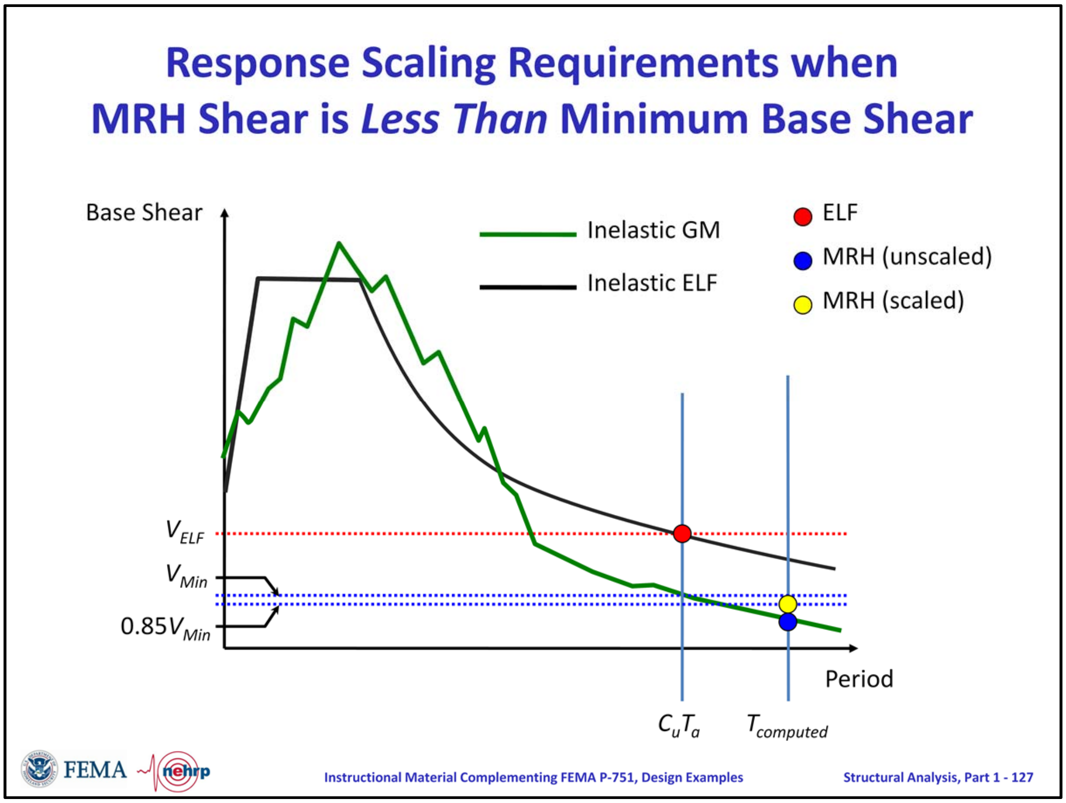
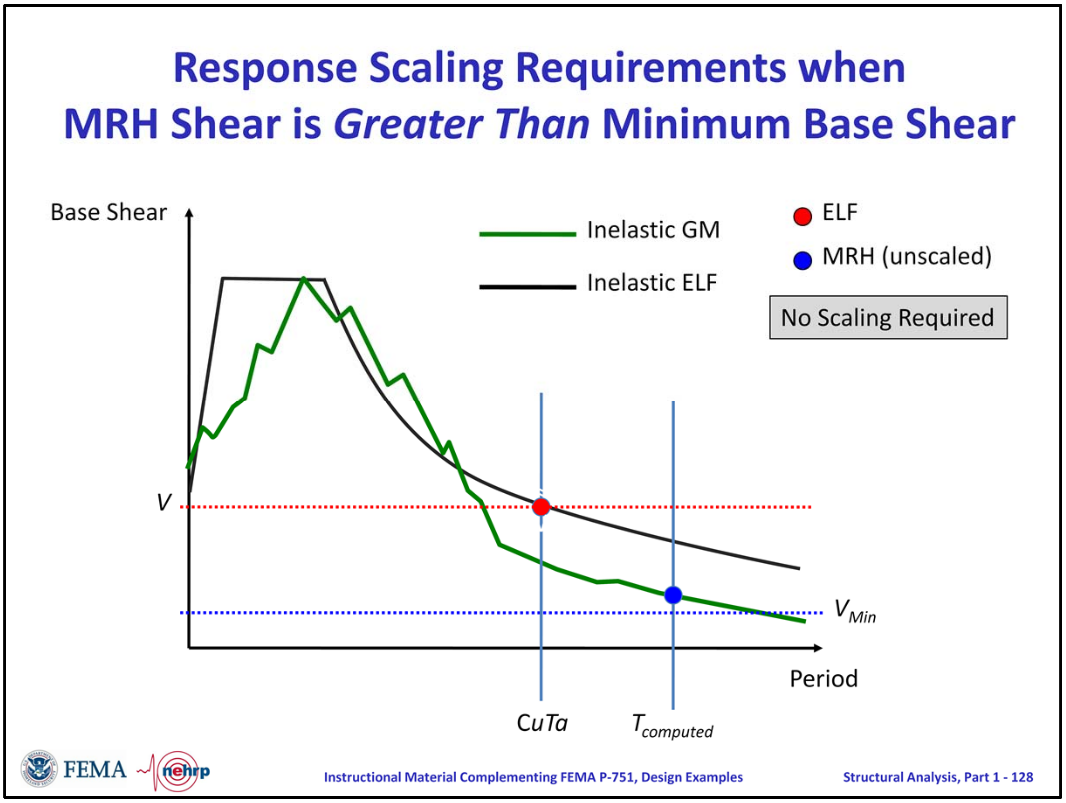
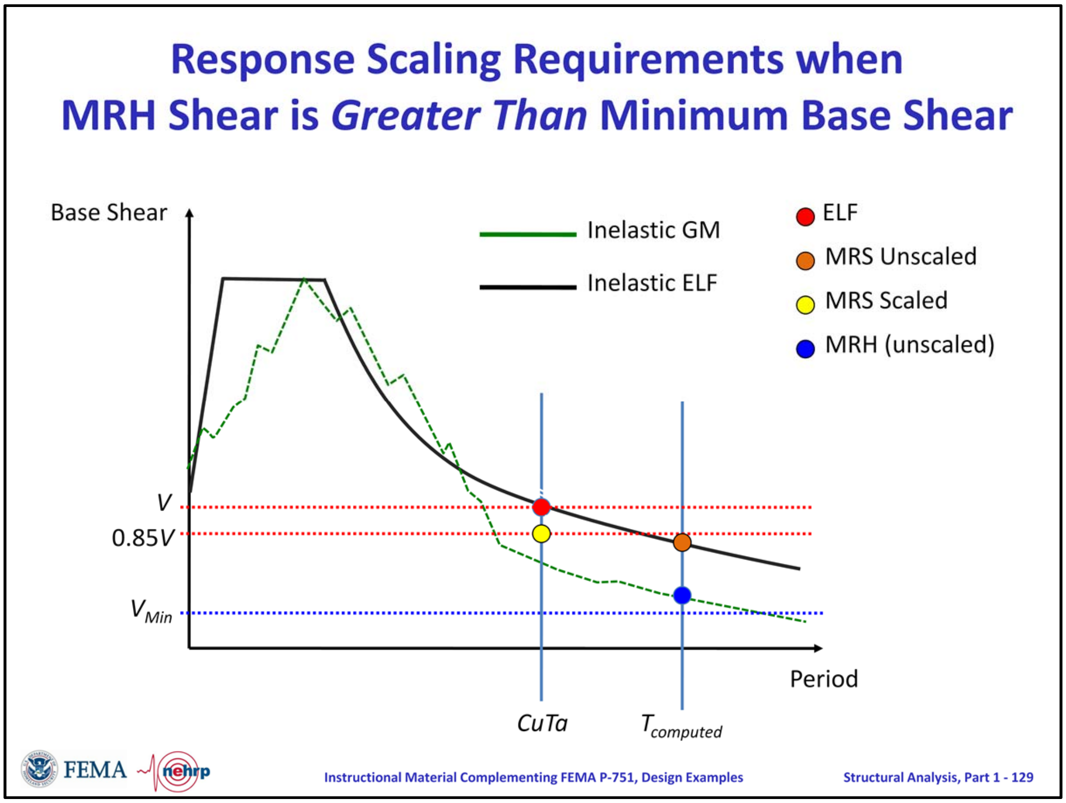
These points are explained in the following slides.



The response history shears should be scaled up to 85% of the minimum base shear.



No scaling is required when the MRH shear is greater than the Minimum Base Shear.



This slide compares response spectrum scaling with response history scaling.

12 Individual Response History Analyses Required

1. A00-X: SS Scaled Component A00 applied in X Direction
2. A00-Y: SS Scaled Component A00 applied in Y Direction
3. A90-X: SS Scaled Component A90 applied in X Direction
4. A90-Y: SS Scaled Component A90 applied in Y Direction

5. B00-X: SS Scaled Component B00 applied in X Direction
6. B00-Y: SS Scaled Component B00 applied in Y Direction
7. B90-X: SS Scaled Component B90 applied in X Direction
8. B90-Y: SS Scaled Component B90 applied in Y Direction

9. C00-X: SS Scaled Component C00 applied in X Direction
10. C00-Y: SS Scaled Component C00 applied in Y Direction
11. C90-X: SS Scaled Component C90 applied in X Direction
12. C90-Y: SS Scaled Component C90 applied in Y Direction

These are the individually scaled GM used in the analyses.

Result Maxima from Response History Analysis Using SS Scaled Ground Motions

	Analysis	Maximum base shear (kips)	Time of maximum shear (sec.)	Maximum roof displacement (in.)	Time of maximum displacement (sec.)
	A00-X	3507	11.29	20.28	11.38
	A00-Y	3573	11.27	14.25	11.28
	A90-X	1588	12.22	7.32	12.70
Low >	A90-Y	1392	13.56	5.16	10.80
	B00-X	3009	8.28	12.85	9.39
	B00-Y	3130	9.37	11.20	10.49
	B90-X	2919	8.85	11.99	7.11
	B90-Y	3460	7.06	11.12	8.20
	C00-X	3130	13.5	9.77	13.54
	C00-Y	2407	4.64	6.76	8.58
	C90-X	3229	6.92	15.61	6.98
High >	C90-Y	5075	6.88	14.31	7.80

1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

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Structural Analysis, Part 1 - 131

This slide shows the maximum response quantities from the SS scaled ground motions. There is a huge variation (considering the fact that all records were scaled in a similar manner to the same target spectrum), with base shears ranging from a low of 1392 kips to a high of 5075 kips. The variation in other response quantities are similar. It is difficult to determine the source of these variations, which include the scaling method, the difference between components, and higher mode effects.

I/R Scaled Shears and Required 85% Rule Scale Factors

Analysis	(I/R) times maximum base shear from analysis (kips)	Required additional scale factor for $V = 0.85V_{ELF} = 956$ kips
A00-X	438.4	2.18
A00-Y	446.7	2.14
A90-X	198.5	4.81
A90-Y	173.9	5.49
B00-X	376.1	2.54
B00-Y	391.2	2.44
B90-X	364.8	2.62
B90-Y	432.5	2.21
C00-X	391.2	2.44
C00-Y	300.9	3.18
C90-X	403.6	2.37
C90-Y	634.4	1.51

1.0 kip = 4.45 kN

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Here the individual scale factors are provided. These factors "normalize" the responses to have the same base shear as given by 85 percent of the ELF base shear. It is notable that all of the ground motions had to be scaled up.

Response History Drifts for all X-Direction Responses

Level	Envelope of drift (in.) for each ground motion A00-X	A90-X	B00-X	B90-X	C00-X	C90-X	Envelope of drift for all the ground motions	Envelope of drift $\times C_d/R$	Allowable drift (in.)
R	1.17	0.49	0.95	0.81	0.91	1.23	1.23	0.85	3.00
12	1.64	0.66	1.22	0.95	1.16	1.27	1.64	1.13	3.00
11	1.97	0.78	1.32	0.99	1.25	1.52	1.97	1.35	3.00
10	2.05	0.86	1.42	1.04	1.20	1.68	2.05	1.41	3.00
9	1.79	0.82	1.26	1.25	0.99	1.41	1.79	1.23	3.00
8	1.83	0.87	1.22	1.42	1.23	1.50	1.83	1.26	3.00
7	1.82	0.83	1.27	1.36	1.21	1.67	1.82	1.25	3.00
6	1.77	0.74	1.36	1.35	1.06	1.94	1.94	1.33	3.00
5	1.50	0.59	1.19	1.21	1.09	1.81	1.81	1.24	3.00
4	1.55	0.62	1.22	1.32	1.23	1.76	1.76	1.21	3.00
3	1.56	0.64	1.24	1.30	1.33	1.60	1.60	1.10	3.00
2	1.97	0.86	1.64	1.58	1.73	1.85	1.97	1.35	4.32

1.0 in. = 25.4 mm.

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Instructional Material Complementing FEMA P-751, Design Examples

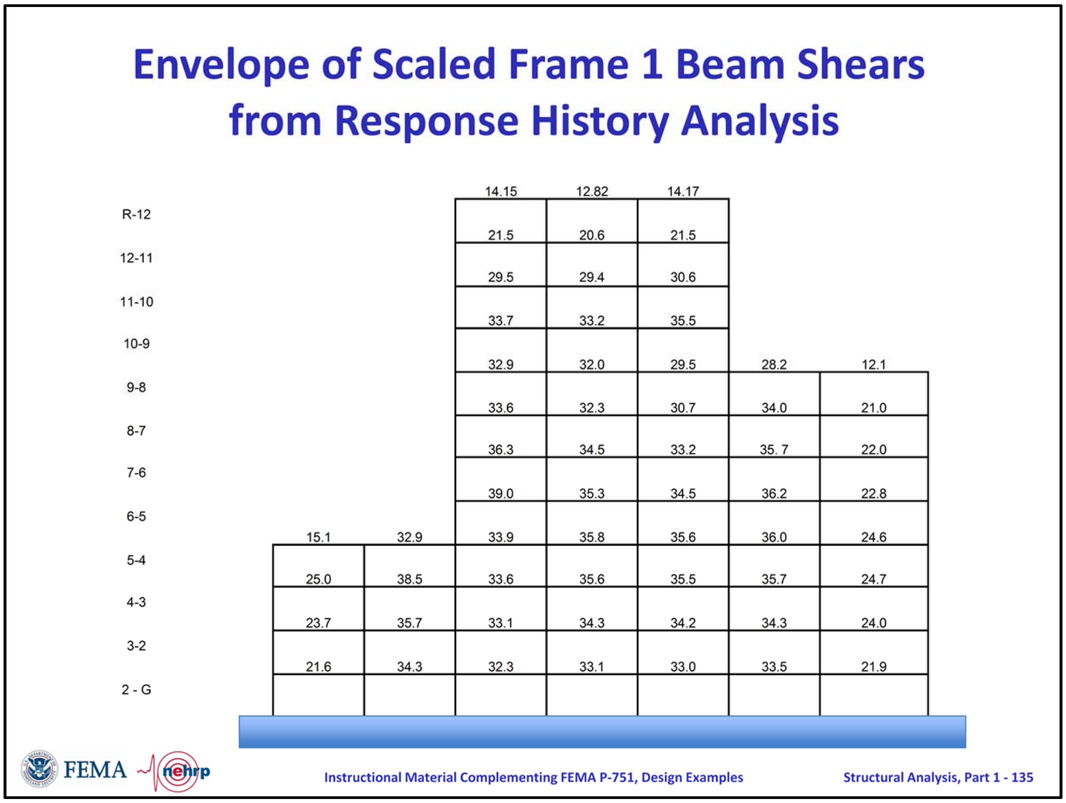
Structural Analysis, Part 1 - 133

The computed drift envelopes are shown here. The drifts shave been scaled by Cd/R, but no "85%" scaling is required. As with the other methods, the drifts appear to be well below the limits, indicating that the structure is probably too stiff.

Load Combinations for Response History Analysis

Load Combination for Response History Analysis					
Earthquake	Load Combination	Loading X Direction		Loading Y Direction	
		Record	Scale Factor	Record	Scale Factor
A	1	A00-X	2.18	A00-Y	5.49
	2	A90-X	-4.81	A90-Y	2.14
	3	A00-X	-2.18	A00-Y	-5.49
	4	A90-X	4.81	A90-Y	-2.14
B	5	B00-X	2.54	B00-Y	2.21
	6	B90-X	-2.62	B90-Y	2.44
	7	B00-X	-2.54	B00-Y	-2.21
	8	B90-X	2.62	B90-Y	-2.44
C	9	C00-X	2.44	C00-y	1.50
	10	C90-X	-2.36	C90-Y	3.18
	11	C00-X	-2.44	C00-Y	-1.50
	12	C90-X	2.36	C90-Y	-3.18

This slide shows the various load combinations. Note that 100 percent of the “85%” scaled motions were applied in each direction.



This slide shows the envelopes of all of the "85%" scaled beam shears on Frame 1. These will be compared to the results from the other methods at the end of the presentation.

Overview of Presentation

- Describe Building
- Describe/Perform steps common to all analysis types
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- Overview of Modal Response History Analysis
- **Comparison of Results**
- Summary and Conclusions

Title slide.

Comparison of Maximum X-Direction Design Story Shears from All Analysis

Level	ELF	Modal response spectrum	Enveloped response history
R	187	180	295
12	341	286	349
11	471	357	462
10	578	418	537
9	765	524	672
8	866	587	741
7	943	639	753
6	999	690	943
5	1,070	784	1,135
4	1,102	840	1,099
3	1,118	895	1,008
2	1,124	956	956

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The story shears are comparable due to the scaling of the MRS and MRH results. However, it seems that he shears in the upper levels are relatively greater in the MRH analysis. This is probably due to the higher spectral acceleration in the higher modes (when compared to the target spectrum).

Comparison of Maximum X-Direction Design Story Drift from All Analysis

Level	X Direction Drift (in.) ELF	Modal response spectrum	Enveloped response history
R	0.99	0.66	0.85
12	1.41	0.89	1.13
11	1.75	1.03	1.35
10	1.92	1.08	1.41
9	1.82	0.98	1.23
8	1.97	1.06	1.26
7	2.01	1.08	1.25
6	1.97	1.08	1.33
5	1.67	0.97	1.24
4	1.69	1.02	1.21
3	1.65	1.05	1.10
2	2.00	1.34	1.35

1.0 in. = 25.4 mm.

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The ELF method produces the largest drifts. However, these drifts were based on a period of C_uT_a, and not on the computed system period. The response history drifts are larger at the upper levels, reflecting the influence of the higher modes.

Comparison of Maximum Beam Shears from All Analysis

Level	Beam Shear Force in Bay D-E of Frame 1 (kips)		
	ELF	Modal response spectrum	Enveloped response history
R	10.27	8.72	12.82
12	18.91	15.61	20.61
11	28.12	21.61	29.45
10	33.15	24.02	33.22
9	34.69	23.32	32.02
8	35.92	23.47	32.30
7	40.10	26.15	34.53
6	40.58	26.76	35.29
5	36.52	25.29	35.82
4	34.58	24.93	35.65
3	35.08	26.60	34.27
2	35.28	28.25	33.07

1.0 kip = 4.45 kN.

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Again, the beam shears are larger in the upper levels when computed using response history. As with drift and story shear, this is attributed to higher mode effects accentuated by high spectral accelerations at lower periods (when compared to the target spectrum).

Overview of Presentation

- Describe Building
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Title slide.

Required Effort

- The Equivalent Lateral Force method and the Modal Response Spectrum methods require similar levels of effort.

- The Modal Response History Method requires considerably more effort than ELF or MRS. This is primarily due to the need to select and scale the ground motions, and to run so many response history analyses.

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Slide comparing relative effort of various methods of analysis.

Accuracy

It is difficult to say whether one method of analysis is "more accurate" than the others. This is because each of the methods assume linear elastic behavior, and make simple adjustments (using R and C_d) to account for inelastic behavior.

Differences inherent in the results produced by the different methods are reduced when the results are scaled. However, it is likely that the Modal Response Spectrum and Modal Response History methods are generally more accurate than ELF because they more properly account for higher mode response.

Slide describes accuracy in analysis.

Recommendations for Future Considerations

1. Three dimensional analysis should be required for *all* Response Spectrum and Response History analysis.

2. Linear Response History Analysis should be moved from Chapter 16 into Chapter 12 and be made as consistent as possible with the Modal Response Spectrum Method. For example, requirements for the number of modes and for scaling of results should be the same for the two methods.

3. A rational procedure needs to be developed for directly including Accidental Torsion in Response Spectrum and Response History Analysis.

4. A rational method needs to be developed for directly including P-Delta effects in Response Spectrum and Response History Analysis.

5. The current methods of selecting and scaling ground motions for linear response history analysis can be and should be much simpler than required for nonlinear response history analysis. The use of "standardized" motion sets or the use of spectrum matched ground motions should be considered.

6. Drift should always be computed and checked at the corners of the building.

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These are the author's opinion and do not necessarily reflect the views of ASCE or BSSC.

Questions

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This slide is intended to initiate questions for the participants.

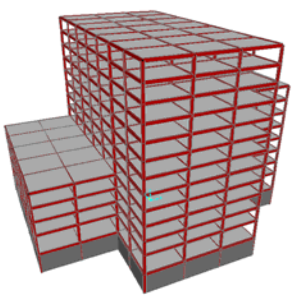
4

Structural Analysis

Finley Charney, Adrian Tola Tola, and Ozgur Atlayan

Structural Analysis: Example 1
Twelve-story Moment Resisting Steel Frame

Analysis of a 12-Story Steel Building In Stockton, California



Building Description

- 12 Stories above grade, one level below grade
- Significant Configuration Irregularities
- Special Steel Moment Resisting Perimeter Frame
- Intended Use is Office Building
- Situated on Site Class C Soils

Analysis Description

- Equivalent Lateral Force Analysis (Section 12.8)
- Modal Response Spectrum Analysis (Section 12.9)
- Linear and Nonlinear Response History Analysis (Chapter 16)

Overview of Presentation

- Describe Building
- Describe/Perform steps common to all analysis types
- Overview of Equivalent Lateral Force analysis
- Overview of Modal Response Spectrum Analysis
- Overview of Modal Response History Analysis
- Comparison of Results
- Summary and Conclusions

Note: The majority of presentation is based on requirements provided by ASCE 7-05. ASCE 7-10 and the 2009 NEHRP Provisions (FEMA P-750) will be referred to as applicable.

Overview of Presentation

- **Describe Building**
- Describe/Perform steps common to all analysis types
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- Overview of Modal Response History Analysis
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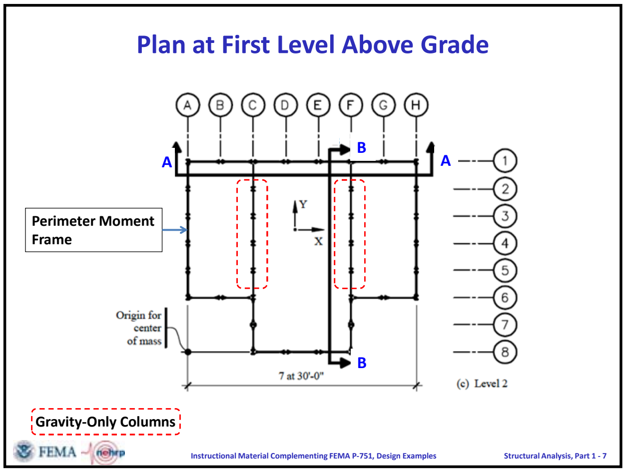
- Summary and Conclusions


Plan at First Level Above Grade
A
B
C
D
E
F
G
H
1
2
3
4
5
6
7
8
Perimeter Moment Frame
Origin for center of mass
Y
X
7 at 30'-0"
(c) Level 2
Gravity-Only Columns
FEMA
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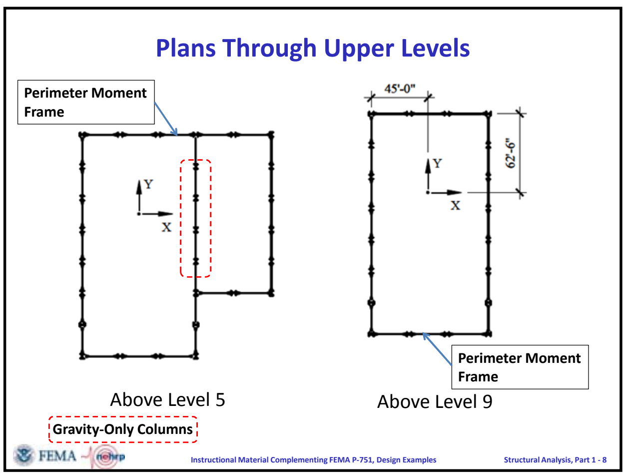
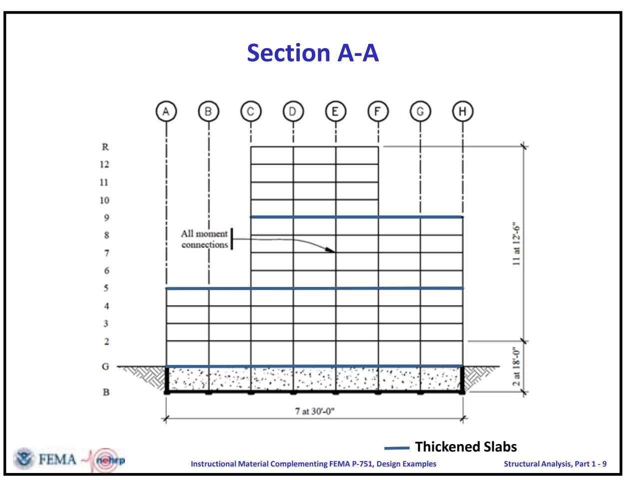
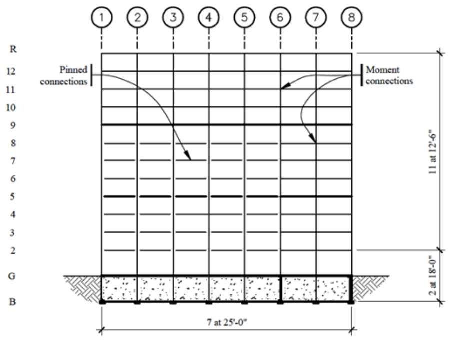
Structural Analysis, Part 1 - 7


Plans Through Upper Levels
Perimeter Moment Frame
45'-0"
62'-6"
Y
X
Perimeter Moment Frame
Above Level 5
Above Level 9
Gravity-Only Columns
FEMA
Instructional Material Complementing FEMA P-751, Design Examples
Structural Analysis, Part 1 - 8


Section A-A
A
B
C
D
E
F
G
H
R
12
11
10
9
8
7
6
5
4
3
2
G
B
All moment connections
11 at 12'-6"
2 at 18'-0"
7 at 30'-0"
Thickened Slabs
FEMA
Instructional Material Complementing FEMA P-751, Design Examples
Structural Analysis, Part 1 - 9

Section B-B



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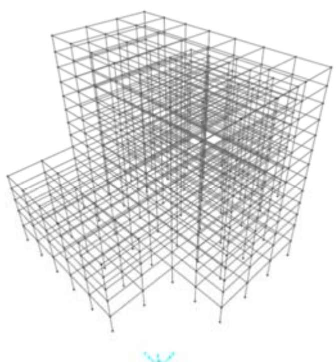
Structural Analysis, Part 1 - 10

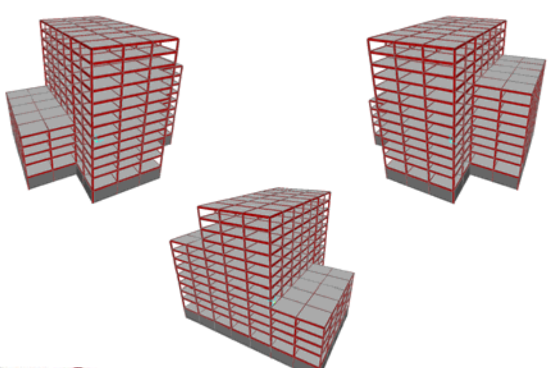
3-D Wire Frame View from SAP 2000



Instructional Material Complementing FEMA P-751, Design Examples

Structural Analysis, Part 1 - 11

Perspective Views of Structure (SAP 2000)



Instructional Material Complementing FEMA P-751, Design Examples

Structural Analysis, Part 1 - 12

Overview of Presentation

- Describe Building
- **Describe/Perform steps common to all analysis types**
- Overview of Equivalent Lateral Force analysis
- Overview of Modal Response Spectrum Analysis
- Overview of Modal Response History Analysis
- Comparison of Results
- Summary and Conclusions

Seismic Load Analysis: Basic Steps

1. Determine Occupancy Category (Table 1-1)
2. Determine Ground Motion Parameters:
   - S_S and S_1 USGS Utility or Maps from Ch. 22)
   - F_a and F_v (Tables 11.4-1 and 11.4-2)
   - S_{DS} and S_{D1} (Eqns. 11.4-3 and 11.4-4)
3. Determine Importance Factor (Table 11.5-1)
4. Determine Seismic Design Category (Section 11.6)
5. Select Structural System (Table 12.2-1)
6. Establish Diaphragm Behavior (Section 11. 3.1)
7. Evaluate Configuration Irregularities (Section 12.3.2)
8. Determine Method of Analysis (Table 12.6-1)
9. Determine Scope of Analysis [2D, 3D] (Section 12.7.2)
10. Establish Modeling Parameters

Determine Occupancy Category

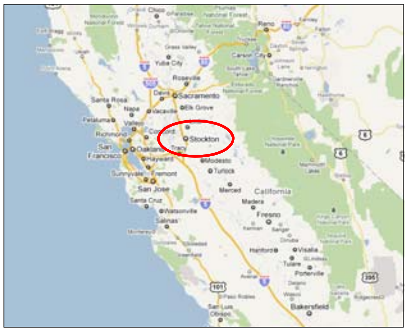
TABLE 1-1 OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES FOR FLOOD, WIND, SNOW, EARTHQUAKE, AND ICE LOADS

Nature of Occupancy	Occupancy Category
Buildings and other structures that represent a low hazard to human life in the event of failure, including, but not limited to: • Agricultural facilities • Certain temporary facilities • Minor storage facilities	I
All buildings and other structures except those listed in Occupancy Categories I, III, and IV	II
Buildings and other structures that represent a substantial hazard to human life in the event of failure, including, but not limited to: • Buildings and other structures where more than 300 people congregate in one area • Buildings and other structures with daycare facilities with a capacity greater than 150 • Buildings and other structures with elementary school or secondary school facilities with a capacity greater than 250 • Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities • Health care facilities with a capacity of 50 or more resident patients, but not having surgery or emergency treatment facilities • Jails and detention facilities Buildings and other structures, not included in Occupancy Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure, including, but not limited to: • Power generating stations* • Water treatment facilities • Sewage treatment facilities • Telecommunication centers Buildings and other structures not included in Occupancy Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released. Buildings and other structures containing toxic or explosive substances shall be eligible for classification as Occupancy Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the toxic or explosive substances does not pose a threat to the public.	III
Buildings and other structures designated as essential facilities, including, but not limited to: • Hospitals and other health care facilities having surgery or emergency treatment facilities • Fire, rescue, ambulance, and police stations and emergency vehicle garages • Designated earthquake, hurricane, or other emergency shelters • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response • Power generating stations and other public utility facilities required in an emergency • Ancillary structures (including, but not limited to, communication towers, fuel storage tanks, cooling towers, electrical substation structures, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Occupancy Category IV structures during an emergency • Aviation control towers, air traffic control centers, and emergency aircraft hangars • Water storage facilities and pump structures required to maintain water pressure for fire suppression • Buildings and other structures having critical national defense functions Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing highly toxic substances where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction. Buildings and other structures containing highly toxic substances shall be eligible for classification as Occupancy Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the highly toxic substances does not pose a threat to the public. This reduced classification shall not be permitted if the buildings or other structures also function as essential facilities.	IV

*Cogeneration power plants that do not supply power on the national grid shall be designated Occupancy Category II.

Occupancy Category = II (Table 1-1)

Ground Motion Parameters for Stockton



S_S=1.25g

S_1=0.40g

Determining Site Coefficients

TABLE 11.4-1 SITE COEFFICIENT, F_a

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	$\boxed{1.0}$
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

F_a=1.0

TABLE 11.4-2 SITE COEFFICIENT, F_v

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	$\boxed{1.4}$	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

F_a=1.4

Determining Design Spectral Accelerations

- S_{DS}=(2/3)$F_a S_S$=(2/3)x1.0x1.25=0.833
- S_{D1}=(2/3)$F_v S_1$=(2/3)x1.4x0.40=0.373

Determine Importance Factor, Seismic Design Category

TABLE 11.5-1 IMPORTANCE FACTORS

Occupancy Category	I
I or II	1.0
III	1.25
IV	1.5

I = 1.0

TABLE 11.6-1 SEISMIC DESIGN CATEGORY BASED ON SHORT PERIOD RESPONSE ACCELERATION PARAMETER

Value of S_{DS}	Occupancy Category		
	I or II	III	IV
$S_{DS} < 0.167$	A	A	A
$0.167 \le S_{DS} < 0.33$	B	B	C
$0.33 \le S_{DS} < 0.50$	C	C	D
$0.50 \le S_{DS}$	D	D	D

TABLE 11.6-2 SEISMIC DESIGN CATEGORY BASED ON 1-S PERIOD RESPONSE ACCELERATION PARAMETER

Value of S_{D1}	OCCUPANCY CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067$	A	A	A
$0.067 \le S_{D1} < 0.133$	B	B	C
$0.133 \le S_{D1} < 0.20$	C	C	D
$0.20 \le S_{D1}$	D	D	D

Seismic Design Category = D

Select Structural System (Table 12.2-1)

Building height (above grade) = 18+11(12.5)=155.5 ft

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]
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Special steel moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Special steel truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Intermediate steel moment frames	12.2.5.6, 12.2.5.7, 12.2.5.8, 12.2.5.9, and 14.1	4.5	3	4	NL	NL	35[h,i]	NP[h]	NP[i]
4. Ordinary steel moment frames	12.2.5.6, 12.2.5.7, 12.2.5.8, and 14.1	3.5	3	3	NL	NL	NP[h]	NP[h]	NP[i]
5. Special reinforced concrete moment frames	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Special composite steel and concrete moment frames	12.2.5.5 and 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Intermediate composite moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP
11. Ordinary composite moment frames	14.3	3	3	2½	NL	NP	NP	NP	NP

Select Special Steel Moment Frame: R=8, C_d=5.5, Ω_0=3

Establish Diaphragm Behavior and Modeling Requirements

12.3.1 Diaphragm Flexibility.
The structural analysis shall consider the relative stiffness of diaphragms and the vertical elements of the seismic force–resisting system. *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., semi-rigid modeling assumption).*

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

Due to horizontal irregularities (e.g. reentrant corners) the diaphragms must be modeled as semi-rigid. This will be done by using Shell elements in the SAP 2000 Analysis.

Determine Configuration Irregularities
Horizontal Irregularities

TABLE 12.3-1 HORIZONTAL STRUCTURAL IRREGULARITIES

		Irregularity Type and Description	Reference Section	Seismic Design Category Application
?	1a.	**Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.8.4.3 12.7.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F C, D, E, and F B, C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
?	1b.	**Extreme Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D B, C, and D
√	2.	**Reentrant Corner Irregularity** is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
√	3.	**Diaphragm Discontinuity Irregularity** is defined to exist where there are diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
X	4.	**Out-of-Plane Offsets Irregularity** is defined to exist where there are discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.	12.3.3.4 12.3.3.3 12.7.3 Table 12.6-1 16.2.2	D, E, and F B, C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
√	5.	**Nonparallel Systems-Irregularity** is defined to exist where the vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic force–resisting system.	12.5.3 12.7.3 Table 12.6-1 Section 16.2.2	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

Irregularity 2 occurs on lower levels. Irregularity 3 is possible but need not be evaluated because it has same consequences as irregularity 3. Torsional Irregularities will be assessed later.

Determine Configuration Irregularities
Vertical Irregularities

TABLE 12.3-2 VERTICAL STRUCTURAL IRREGULARITIES

		Irregularity Type and Description	Reference Section	Seismic Design Category Application
X	1a.	**Stiffness-Soft Story Irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.	Table 12.6-1	D, E, and F
	1b.	**Stiffness-Extreme Soft Story Irregularity** is defined to exist where there is a story in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three stories above.	12.3.3.1 Table 12.6-1	E and F D, E, and F
√	2.	**Weight (Mass) Irregularity** is defined to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	Table 12.6-1	D, E, and F
√	3.	**Vertical Geometric Irregularity** is defined to exist where the horizontal dimension of the seismic force–resisting system in any story is more than 130% of that in an adjacent story.	Table 12.6-1	D, E, and F
X	4.	**In-Plane Discontinuity in Vertical Lateral Force-Resisting Element Irregularity** is defined to exist where an in-plane offset of the lateral force-resisting elements is greater than the length of those elements or there exists a reduction in stiffness of the resisting element in the story below.	12.3.3.3 12.3.3.4 Table 12.6-1	B, C, D, E, and F D, E, and F D, E, and F
X	5a.	**Discontinuity in Lateral Strength–Weak Story Irregularity** is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 Table 12.6-1	E and F D, E, and F
X	5b.	**Discontinuity in Lateral Strength–Extreme Weak Story Irregularity** is defined to exist where the story lateral strength is less than 65% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.	12.3.3.1 12.3.3.2 Table 12.6-1	D, E, and F B and C D, E, and F

Irregularities 2 and 3 occur due to setbacks. Soft story and weak story irregularities are highly unlikely for this system and are not evaluated.

Selection of Method of Analysis (ASCE 7-05)

TABLE 12.6-1 PERMITTED ANALYTICAL PROCEDURES

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Analysis Section 12.8	Modal Response Spectrum Analysis Section 12.9	Seismic Response History Procedures Chapter 16	
B, C	Occupancy Category I or II buildings of light-framed construction not exceeding 3 stories in height	P	P	P	
	Other Occupancy Category I or II buildings not exceeding 2 stories in height	P	P	P	
	All other structures	P	P	P	
D, E, F	Occupancy Category I or II buildings of light-framed construction not exceeding 3 stories in height	P	P	P	Not applicable
	Other Occupancy Category I or II buildings not exceeding 2 stories in height	P	P	P	
	Regular structures with $T < 3.5T_s$ and all structures of light frame construction	P	P	P	System is not "regular"
	Irregular structures with $T < 3.5T_s$ and having only horizontal irregularities Type 2, 3, 4, or 5 of Table 12.2-1 or vertical irregularities Type 4, 5a, or 5b of Table 12.3-1	P	P	P	Vertical irregularities 2 and 3 exist
	All other structures	NP	P	P	

NOTE: P: Permitted; NP: Not Permitted

ELF is not permitted:
Must use Modal Response Spectrum or Response History Analysis

Selection of Method of Analysis (ASCE 7-10)

Table 12.6-1 Permitted Analytical Procedures

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Analysis, Section 12.8[a]	Modal Response Spectrum Analysis, Section 12.9[a]	Seismic Response History Procedures, Chapter 16[a]
B, C	All structures	P	P	P
D, E, F	Risk Category I or II buildings not exceeding 2 stories above the base	P	P	P
	Structures of light frame construction	P	P	P
	Structures with no structural irregularities and not exceeding 160 ft in structural height	P	P	P
	Structures exceeding 160 ft in structural height with no structural irregularities and with $T < 3.5T_s$	P	P	P
	Structures not exceeding 160 ft in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12-2 or vertical irregularities of Type 4, 5a, or 5b in Table 12-3	P	P	P
	All other structures	NP	P	P

[a]P: Permitted; NP: Not Permitted; $T_s = S_{D1}/S_{DS}$.

ELF is not permitted:
Must use Modal Response Spectrum or Response History Analysis

Overview of Presentation

- Describe Building
- Describe/Perform steps common to all analysis types
- **Overview of Equivalent Lateral Force analysis**
- Overview of Modal Response Spectrum Analysis
- Overview of Modal Response History Analysis
- Comparison of Results
- Summary and Conclusions

Comments on use of ELF for *This System*

ELF is NOT allowed as the *Design Basis Analysis*.
However, ELF (or aspects of ELF) must be used for:

- Preliminary analysis and design
- Evaluation of torsion irregularities and amplification
- Evaluation of system redundancy factors
- Computing P-Delta Effects
- Scaling Response Spectrum and Response History results

Determine Scope of Analysis

12.7.3 Structural Modeling.

A mathematical model of the structure shall be constructed for the purpose of determining member forces and structure displacements resulting from applied loads and any imposed displacements or P-Delta effects.

The model shall include the stiffness and strength of elements that are significant to the distribution of forces and deformations in the structure and represent the spatial distribution of mass and stiffness throughout the structure.

Note: P-Delta effects should not be included directly in the analysis. They are considered indirectly in Section 12.8.7

Determine Scope of Analysis (Continued)

Continuation of 12.7.3:

Structures that have horizontal structural irregularity Type 1a, 1b, 4, or 5 of Table 12.3-1 shall be analyzed using a 3-D representation.

Where a 3-D model is used, a minimum of three dynamic degrees of freedom consisting of translation in two orthogonal plan directions and torsional rotation about the vertical axis shall be included at each level of the structure.

Where the diaphragms have not been classified as rigid or flexible in accordance with Section 12.3.1, the model shall include representation of the diaphragm's stiffness characteristics and such additional dynamic degrees of freedom as are required to account for the participation of the diaphragm in the structure's dynamic response.

Analysis of structure must be in 3D, and diaphragms must be modeled as semi-rigid

Establish Modeling Parameters

Continuation of 12.7.3:

In addition, the model shall comply with the following:

a) Stiffness properties of concrete and masonry elements shall consider the effects of cracked sections.

b) For steel moment frame systems, the contribution of panel zone deformations to overall story drift shall be included.

Modeling Parameters used in Analysis

1) The floor diaphragm was modeled with shell elements, providing nearly rigid behavior in-plane.

2) Flexural, shear, axial, and torsional deformations were included in all columns and beams.

3) Beam-column joints were modeled using centerline dimensions. This approximately accounts for deformations in the panel zone.

4) Section properties for the girders were based on bare steel, ignoring composite action. This is a reasonable assumption in light of the fact that most of the girders are on the perimeter of the building and are under reverse curvature.

Modeling Parameters used in Analysis (continued)

5) Except for those lateral load-resisting columns that terminate at Levels 5 and 9, all columns of the lateral load resisting system were assumed to be fixed at their base.

6) The basement walls and grade level slab were explicitly modeled using 4-node shell elements. This was necessary to allow the interior columns to continue through the basement level. No additional lateral restraint was applied at the grade level, thus the basement level acts as a very stiff first floor of the structure. This basement level was not relevant for the ELF analysis, but did influence the MRS and MRH analysis as described in later sections of this example

7) P-Delta effects were not included in the mathematical model. These effects are evaluated separately using the procedures provided in section 12.8.7 of the Standard.

Equivalent Lateral Force Analysis

1. Compute Seismic Weight, W (Sec. 12.7.2)
2. Compute Approximate Period of Vibration T_a (Sec. 12.8.2.1)
3. Compute Upper Bound Period of Vibration, $T=C_uT_a$ (Sec. 12.8.2)
4. Compute "Analytical" Natural periods
5. Compute Seismic Base Shear (Sec. 12.8.1)
6. Compute Equivalent Lateral Forces (Sec. 12.8.3)
7. Compute Torsional Amplification Factors (Sec. 12.8.4.3)
8. Determine Orthogonal Loading Requirements (Sec. 12.8)
9. Compute Redundancy Factor ρ (Sec. 12.3.4)
10. Perform Structural Analysis
11. Check Drift and P-Delta Requirements (Sec. 12.9.4 and 12.9.6)
12. Revise Structure in Necessary and Repeat Steps 1-11 [as appropriate]
13. Determine Design-Level Member Forces (Sec. 12.4)

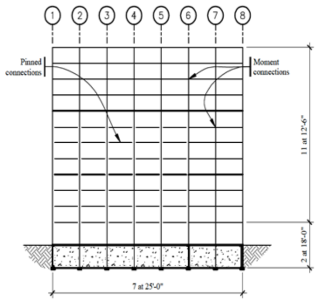
Notes on Computing the Period of Vibration

T_a (Eqn.12.8-7) is an approximate lower bound period, and is based on the measured response of buildings in high seismic regions.

$T=C_uT_a$ is also approximate, but is somewhat more accurate than T_a alone because it is based on the "best fit" of the measured response, and is adjusted for local seismicity. Both of these adjustments are contained in the C_u term.

C_uT_a can only be used if an analytically computed period, called $T_{computed}$ herein, is available from a computer analysis of the structure.

Using Empirical Formulas to Determine T_a



$$T_a = C_t h_n^x$$

From Table 12.8.2:
C_t=0.028
x=0.80

h_n=18+11(12.5)=155.5 ft

$$T_a = 0.028(155.5)^{0.8} = 1.59 \text{ sec}$$

Applies in Both Directions

Adjusted Empirical Period $T=C_uT_a$

TABLE 12.8-1 COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD

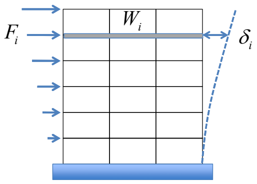
Design Spectral Response Acceleration Parameter at 1 s, S_{D1}	Coefficient C_u
≥ 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
≤ 0.1	1.7

S_{D1}=0.373
Gives C_u=1.4

$$T = 1.4(1.59) = 2.23 \text{ sec}$$

Applies in Both Directions

Use of Rayleigh Analysis to Determine $T_{computed}$



Building has *n* Levels

$$T_{computed} = \frac{2\pi}{\omega_{computed}}$$

$$\omega_{computed} = \sqrt{\frac{g\sum_{i=1}^{n}\delta_i F_i}{\sum_{i=1}^{n}\delta_i^2 W_i}}$$

Use of Rayleigh Analysis to Determine $T_{computed}$

Table 4.1-9 Rayleigh Analysis for X Direction Period of Vibration

Level	Drift, δ(in.)	Force, F (kips)	Weight, W (kips)	δF (in.-kips)	$\delta^2 W/g$ (in.-kips-sec^2)
R	6.67	186.9	1657	1247	191
12	6.35	154.0	1596	979	167
11	5.90	129.9	1596	767	144
10	5.34	107.6	1596	575	118
9	4.73	186.3	3403	881	197
8	4.15	100.8	2331	418	104
7	3.52	77.0	2331	271	75
6	2.87	56.2	2331	162	50
5	2.24	71.4	4324	160	56
4	1.71	31.5	3066	54	23
3	1.17	16.6	3066	19	11
2	0.64	6.3	3097	4	3
Σ				5536	1138

$\omega = (5536/1138)^{0.5} = 2.21$ rad/sec. $T = 2\pi/\omega = 2.85$ sec. 1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

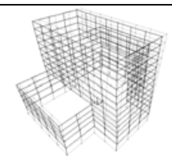
X-Direction $T_{computed}$ = 2.85 sec.
Y-Direction $T_{computed}$ = 2.56 sec.

(see Text)

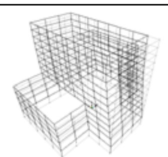
Periods Computed Using Eigenvalue Analysis

$$K\Phi = M\Phi\Omega^2$$

$\Omega =$ Diagonal matrix containing circular frequencies ω
$\Phi =$ Mode Shape Matrix



Mode 1 T=2.87 sec



Mode 2: T:2.60 sec

Range of Periods Computed for This Example

T_a=1.59 sec

C_uT_a=2.23 sec

$T_{computed}$ = 2.87 sec in X direction
2.60 sec in Y direction

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Periods of Vibration for Computing Seismic Base Shear (Eqns 12.8-1, 12.8-3, and 12.8-4)

if $T_{computed}$ is not available use T_a

if $T_{computed}$ is available, then:

- if $T_{computed} > C_uT_a$ use C_uT_a
- if $T_a <= T_{computed} <= C_uT_a$ use $T_{computed}$
- if $T_{computed} < T_a$ use T_a

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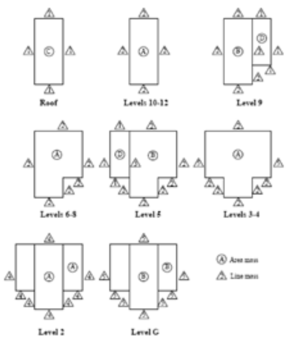
Area and Line Weight Designations



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Area and Line Weight Values

Table 4.1-1 Area Weights Contributing to Masses on Floor Diaphragms

Mass Type	Area Weight Designation				
	A	B	C	D	E
Slab and Deck (psf)	50	75	50	75	75
Structure (psf)	20	20	20	20	50
Ceiling and Mechanical (psf)	15	15	15	15	15
Partition (psf)	10	10	0	0	10
Roofing (psf)	0	0	15	15	0
Special (psf)	0	0	0	60	25
Total (psf)	95	120	100	185	175

See Figure 4.1-4 for mass location. 1.0 psf = 47.9 N/m^2.

Table 4.1-2 Line Weights Contributing to Masses on Floor Diaphragms

Mass Type	Line Weight Designation				
	1	2	3	4	5
From Story Above (plf)	60.0	93.8	93.8	93.8	135.0
From Story Below (plf)	93.8	93.8	0.0	135.0	1350.0
Total (plf)	153.8	187.6	93.8	228.8	1485.0

See Figure 4.1-4 for mass location. 1.0 plf = 14.6 N/m.

Weights at Individual Levels

Table 4.1-3 Floor Weight, Floor Mass, Mass Moment of Inertia, and Center of Mass Locations

Level	Weight (kips)	Mass (kip-sec$^{2/}$in.)	Mass Moment of Inertia (in.-kip-sec$^{2/}$/radian)	X Distance to C.M. (in.)	Y Distance to C.M. (in.)
R	1657	4.287	2.072×10^6	1260	1050
12	1596	4.130	2.017×10^6	1260	1050
11	1596	4.130	2.017×10^6	1260	1050
10	1596	4.130	2.017×10^6	1260	1050
9	3403	8.807	5.309×10^6	1638	1175
8	2331	6.032	3.703×10^6	1553	1145
7	2331	6.032	3.703×10^6	1553	1145
6	2331	6.032	3.703×10^6	1553	1145
5	4320	11.19	9.091×10^6	1160	1206
4	3066	7.935	6.356×10^6	1261	1184
3	3066	7.935	6.356×10^6	1261	1184
2	3097	8.015	6.437×10^6	1262	1181
G	6525	16.89	1.503×10^7	1265	1149
Σ	36912				

Total Building Weight=36,912 k. Weight above grade = 30,394 k.

Calculation of ELF Base Shear

$$V = C_S W \quad \text{(12.8-1)}$$

$$C_S = \frac{S_{DS}}{R/I} = \frac{0.833}{8/1} = 0.104 \quad \text{(12.8-2)}$$

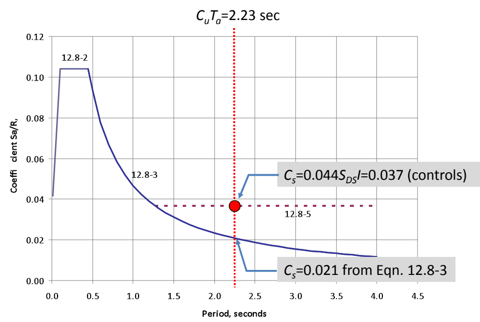
$$C_S = \frac{S_{D1}}{T(R/I)} = \frac{0.373}{2.23(8/1)} = 0.021 \quad \text{(12.8-3)}$$

$$\boxed{C_S = 0.044 S_{DS} I = 0.044(0.833)(1) = 0.0307} \quad \text{(12.8-5)}$$

Controls

$V = 0.037(30394) = 1124$ kips

Concept of $R_{effective}$



$R_{effective}$ = (0.021/0.037) x 8 = 4.54

Issues Related to Period of Vibration and Drift

12.8.6.1 Minimum Base Shear for Computing Drift

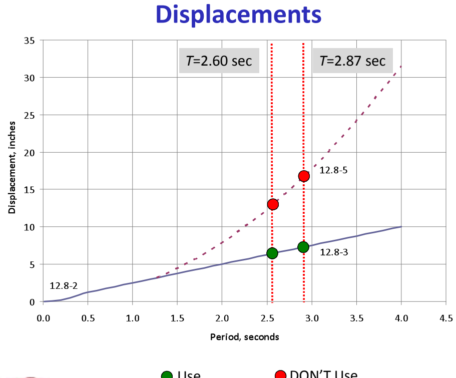
The elastic analysis of the seismic force-resisting system for computing drift shall be made using the prescribed seismic design forces of Section 12.8.

EXCEPTION: Eq. 12.8-5 need not be considered for computing drift

12.8.6.2 Period for Computing Drift

For determining compliance with the story drift limits of Section 12.12.1, it is permitted to determine the elastic drifts, (δ_{xe}), using seismic design forces based on the computed fundamental period of the structure without the upper limit (C_uT_a) specified in Section 12.8.2.

Using Eqns. 12.8-3 or 12.8-5 for Computing ELF Displacements



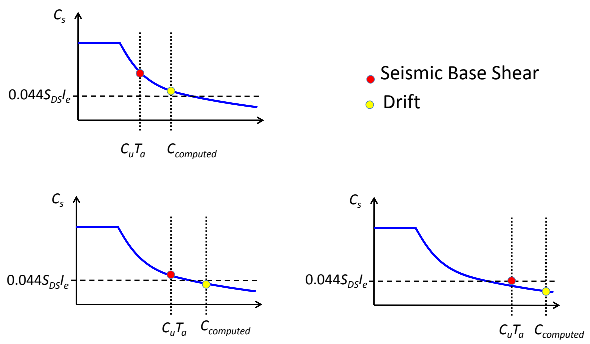
What if Equation 12.8-6 had Controlled Base Shear?

$$C_s = \frac{0.5 S_1}{(R/I)}$$

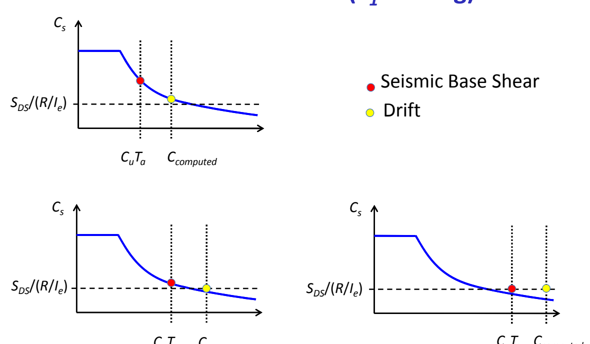
Eqn. 12.8-6, applicable only when S_1 >= 0.6g

This equation represents the “true” response spectrum shape for near-field ground motions. Thus, the lateral forces developed on the basis of this equation must be used for determining component design forces **and** displacements used for computing drift.

When Equation 12.8-5 May Control Seismic Base Shear (S_1 < 0.6g)



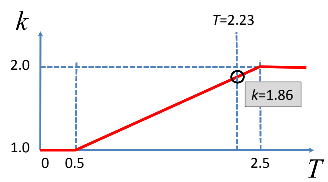
When Equation 12.8-6 May Control Seismic Base Shear (S_1 >= 0.6g)



Calculation of ELF Forces

$$F_x = C_{vx} V \quad \text{(12.8-11)}$$

$$C_{vs} = \frac{w_x h^k}{\sum_{i=1}^{n} w_i h_i^k} \quad \text{(12.8-12)}$$



Calculation of ELF Forces (continued)

Table 4.1-4 Equivalent Lateral Forces for Building Responding in X and Y Directions

Level x	w_x (kips)	h_x (ft)	$w_x h_x^k$	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
R	1657	155.5	20272144	0.1662	186.9	186.9	2336
12	1596	143.0	16700697	0.1370	154.0	340.9	6597
11	1596	130.5	14081412	0.1155	129.9	470.8	12482
10	1596	118.0	11670590	0.0957	107.6	578.4	19712
9	3403	105.5	20194253	0.1656	186.3	764.7	29271
8	2331	93.0	10933595	0.0897	100.8	865.5	40090
7	2331	80.5	8353175	0.0685	77.0	942.5	51871
6	2331	68.0	6097775	0.0500	56.2	998.8	64356
5	4324	55.5	7744477	0.0635	71.4	1070.2	77733
4	3066	43.0	3411857	0.0280	31.5	1101.7	91505
3	3066	30.5	1798007	0.0147	16.6	1118.2	103372
2	3097	18.0	679242	0.0056	6.3	1124.5	120694
Σ	30394	-	121937234	1.00	1124.5		

Values in column 4 based on exponent k=1.865. 1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN.

Inherent and Accidental Torsion

12.8.4.1 Inherent Torsion. For diaphragms that are not flexible, the distribution of lateral forces at each level shall consider the effect of the inherent torsional moment, M_t, *resulting from eccentricity* between the locations of the center of mass and the center of rigidity. For flexible diaphragms, the distribution of forces to the vertical elements shall account for the position and distribution of the masses supported.

Inherent torsion effects are automatically included in 3D structural analysis, and member forces associated with such effects need not be separated out from the analysis.

Inherent and Accidental Torsion (continued)

12.8.4.2 Accidental Torsion. Where diaphragms are not flexible, the design shall include the inherent torsional moment (M_t) (kip or kN) resulting from the location of the structure masses plus the accidental torsional moments (M_{ta}) (kip or kN) caused by assumed displacement of the center of mass each way from its actual location by a distance equal to 5 percent of the dimension of the structure perpendicular to the direction of the applied forces.

Where earthquake forces are applied concurrently in two orthogonal directions, the required 5 percent displacement of the center of mass need not be applied in both of the orthogonal directions at the same time, but shall be applied in the direction that produces the greater effect.

Inherent and Accidental Torsion (continued)

12.8.4.3 Amplification of Accidental Torsional Moment. Structures assigned to Seismic Design Category C, D, E, or F, where Type 1a or 1b torsional irregularity exists as defined in Table 12.3-1 shall have the effects accounted for by multiplying M_{ta} at each level by a torsional amplification factor (A_x) as illustrated in Fig. 12.8-1 and determined from the following equation:

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \qquad (12.8\text{-}14)$$

where

δ_{max} = the maximum displacement at Level x (in. or mm) computed assuming $A_x = 1$

δ_{avg} = the average of the displacements at the extreme points of the structure at Level x computed assuming $A_x = 1$ (in. or mm)

EXCEPTION: The accidental torsional moment need not be amplified for structures of light-frame construction.

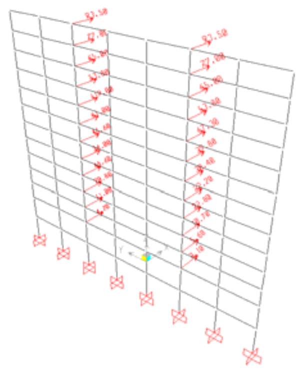
The torsional amplification factor (A_x) is not required to exceed 3.0. The more severe loading for each element shall be considered for design.

Determine Configuration Irregularities Horizontal Irregularities

TABLE 12.3-1 HORIZONTAL STRUCTURAL IRREGULARITIES

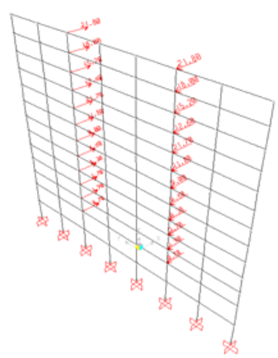
	Irregularity Type and Description	Reference Section	Seismic Design Category Application
1a.	**Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. Torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.4 12.8.4.3 12.7.3 12.12.1 Table 12.6-1 Section 16.2.2	D, E, and F C, D, E, and F B, C, D, E, and F C, D, E, and F D, E, and F B, C, D, E, and F
1b.	**Extreme Torsional Irregularity** is defined to exist where the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure. Extreme torsional irregularity requirements in the reference sections apply only to structures in which the diaphragms are rigid or semirigid.	12.3.3.1 12.3.3.4 12.7.3 12.8.4.3 12.12.1 Table 12.6-1 Section 16.2.2	E and F D B, C, and D C and D C and D D B, C, and D
2.	**Reentrant Corner Irregularity** is defined to exist where both plan projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
3.	**Diaphragm Discontinuity Irregularity** is defined to exist where there are diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50% from one story to the next.	12.3.3.4 Table 12.6-1	D, E, and F D, E, and F
4.	**Out-of-Plane Offsets Irregularity** is defined to exist where there are discontinuities in a lateral force-resistance path, such as out-of-plane offsets of the vertical elements.	12.3.3.4 12.3.3.3 12.7.3 Table 12.6-1 16.2.2	D, E, and F B, C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F
5.	**Nonparallel Systems-Irregularity** is defined to exist where the vertical lateral force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the seismic force–resisting system.	12.5.3 12.7.3 Table 12.6-1 Section 16.2.2	C, D, E, and F B, C, D, E, and F D, E, and F B, C, D, E, and F

Application of Equivalent Lateral Forces (X Direction)



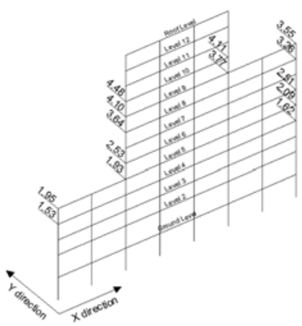
Forces in Kips

Application of Torsional Forces (Using X-Direction Lateral Forces)



Forces in Kips

Stations for Monitoring Drift for Torsion Irregularity Calculations with ELF Forces Applied in X Direction



Results of Torsional Irregularity Calculations For ELF Forces Applied in X Direction

Table 4.1-5a Computation for torsional irregularity with ELF loads acting in X direction and Torsional Moment applied Counterclockwise

Level	δ1 (in)	δ2 (in)	Δ1 (in)	Δ2 (in)	Δavg (in)	Δmax (in)	Δmax/Δavg	Irregularity
R	7.27	6.15	0.34	0.29	0.31	0.34	1.08	None
12	6.93	5.87	0.48	0.42	0.45	0.48	1.07	None
11	6.44	5.45	0.60	0.51	0.55	0.60	1.07	None
10	5.85	4.93	0.66	0.56	0.61	0.66	1.08	None
9	5.19	4.37	0.65	0.54	0.59	0.65	1.10	None
8	4.54	3.84	0.69	0.58	0.64	0.69	1.09	None
7	3.84	3.26	0.70	0.59	0.65	0.70	1.09	None
6	3.14	2.67	0.69	0.58	0.63	0.69	1.09	None
5	2.46	2.09	0.60	0.50	0.55	0.60	1.09	None
4	1.86	1.60	0.59	0.50	0.55	0.59	1.08	None
3	1.27	1.10	0.58	0.49	0.53	0.58	1.08	None
2	0.69	0.61	0.69	0.61	0.65	0.69	1.06	None

1.0 in. = 25.4 mm

Result: There is not a Torsional Irregularity for Loading in the X Direction

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Results of Torsional Irregularity Calculations For ELF Forces Applied in Y Direction

Table 4.1-5b Computation for torsional irregularity with ELF loads acting in Y direction, and Torsional Moment applied Clockwise

Level	δ1 (in)	δ2 (in)	Δ1 (in)	Δ2 (in)	Δavg (in)	Δmax (in)	Δmax/Δavg	Irregularity
R	5.19	4.77	0.15	0.14	0.15	0.15	1.03	None
12	5.03	4.63	0.25	0.23	0.24	0.25	1.03	None
11	4.79	4.40	0.31	0.29	0.30	0.31	1.04	None
10	4.48	4.11	0.38	0.34	0.36	0.38	1.06	None
9	4.10	3.77, 3.55	0.46	0.28	0.37	0.46	1.24	Irregularity
8	3.64	3.26	0.54	0.36	0.45	0.54	1.20	None
7	3.09	2.90	0.56	0.39	0.47	0.56	1.18	None
6	2.53	2.51	0.60	0.42	0.51	0.60	1.18	None
5	1.93, 1.95	2.09	0.41	0.47	0.44	0.47	1.06	None
4	1.53	1.62	0.47	0.50	0.48	0.50	1.03	None
3	1.07	1.12	0.47	0.50	0.48	0.50	1.03	None
2	0.60	0.63	0.60	0.63	0.61	0.63	1.03	None

1.0 in. = 25.4 mm

Result: There is a minor Torsional Irregularity for Loading in the Y Direction

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Results of Torsional Amplification Calculations For ELF Forces Applied in Y Direction

(X Direction Results are Similar)

Table 4.1-5d Amplification Factor A_x for Accidental Torsional Moment Loads acting in the Y direction and Torsional Moment applied Clockwise

Level	δ_1 (in)	δ_2 (in)	δ_{avg} (in)	δ_{max} (in)	A_x calculated	A_x corrected
R	5.19	4.77	4.98	5.19	0.75	1.00
12	5.03	4.63	4.83	5.03	0.75	1.00
11	4.79	4.40	4.59	4.79	0.76	1.00
10	4.48	4.11	4.29	4.48	0.76	1.00
9	4.10	3.55	3.82	4.10	0.80	1.00
8	3.64	3.26	3.45	3.64	0.77	1.00
7	3.09	2.90	3.00	3.09	0.74	1.00
6	2.53	2.51	2.52	2.53	0.70	1.00
5	1.95	2.09	2.02	2.09	0.74	1.00
4	1.53	1.62	1.58	1.62	0.73	1.00
3	1.07	1.12	1.10	1.12	0.73	1.00
2	0.60	0.63	0.61	0.63	0.73	1.00

1.0 in. = 25.4 mm

Result: Amplification of Accidental Torsion Need not be Considered

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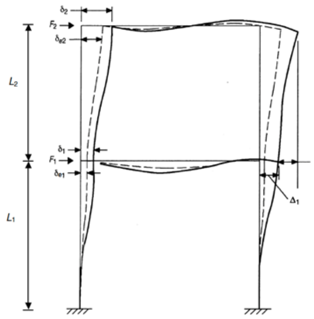
Drift and Deformation



Story Level 2

F_2 = strength-level design earthquake force

δ_{e2} = elastic displacement computed under strength-level design earthquake forces

δ_2 = $C_d\, \delta_{e2}/I_E$ = amplified displacement

Δ_2 = $(\delta_{e2} - \delta_{e1})\, C_d / I_E \leq \Delta_a$ (Table 12.12-1)

Story Level 1

F_1 = strength-level design earthquake force

δ_{e1} = elastic displacement computed under strength-level design earthquake forces

δ_1 = $C_d\, \delta_{e1}/I_E$ = amplified displacement

Δ_1 = $\delta_1 \leq \Delta_a$ (Table 12.12-1)

Δ_i = Story Drift

Δ_i/L_i = Story Drift Ratio

δ_2 = Total Displacement

FIGURE 12.8-2 STORY DRIFT DETERMINATION

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Drift and Deformation (Continued)

12.12 DRIFT AND DEFORMATION

12.12.1 Story Drift Limit. The design story drift (Δ) as determined in Sections 12.8.6, 12.9.2, or 16.1, shall not exceed the allowable story drift (Δ_a) as obtained from Table 12.12-1 for any story. For structures with significant torsional deflections, the maximum drift shall include torsional effects. For structures assigned to Seismic Design Category C, D, E, or F having horizontal irregularity Types 1a or 1b of Table 12.3-1, the design story drift, Δ, shall be computed as the largest difference of the deflections along any of the edges of the structure at the top and bottom of the story under consideration.

Not strictly Followed in this Example due to *very minor* torsion irregularity

TABLE 12.12-1 ALLOWABLE STORY DRIFT, Δ_a[a,b]

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$[c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$\boxed{0.020h_{sx}}$	$0.015h_{sx}$	$0.010h_{sx}$

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Drift and Deformation (Continued)

ASCE 7-05 (ASCE 7-10) Similar

12.8.6.2 Period for Computing Drift. For determining compliance with the story drift limits of Section 12.12.1, it is permitted to determine the elastic drifts, (δ_{xe}), using seismic design forces based on the computed fundamental period of the structure without the upper limit ($C_u T_a$) specified in Section 12.8.2.

ASCE 7-10

12.8.6.1 Minimum Base Shear for Computing Drift

The elastic analysis of the seismic force-resisting system for computing drift shall be made using the prescribed seismic design forces of Section 12.8.

EXCEPTION: Eq. 12.8-5 need not be considered for computing drift.

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Computed Drifts in X Direction

Table 4.1-7 ELF Drift for Building Responding in X Direction

Level	1 Total drift from SAP2000 (in.)	2 Story drift from SAP2000 (in.)	3 Amplified story drift (in.)	4 Amplified drift times 0.568 (in.)	5 Allowable drift (in.)
R	6.67	0.32	1.74	0.99	3.00
12	6.35	0.45	2.48	1.41	3.00
11	5.90	0.56	3.07	1.75	3.00
10	5.34	0.62	3.39	1.92	3.00
9	4.73	0.58	3.20	1.82	3.00
8	4.15	0.63	3.47	1.97	3.00
7	3.52	0.64	3.54	2.01	3.00
6	2.87	0.63	3.47	1.97	3.00
5	2.24	0.54	2.95	1.67	3.00
4	1.71	0.54	2.97	1.69	3.00
3	1.17	0.53	2.90	1.65	3.00
2	0.64	0.64	3.51	2.00	4.32

Column 4 adjusts for *Standard* Eq. 12.8-3 (for drift) vs 12.8-5 (for strength).
1.0 in. = 25.4 mm.

C_d Amplified drift based on forces from Eq. 12.8-5

Modified for forces based on Eq. 12.8-3

Computed Drifts in Y Direction

Table 4.1-8 ELF Drift for Building Responding in Y Direction

Level	1 Total drift from SAP2000 (in.)	2 Story drift from SAP2000 (in.)	3 Amplified story drift (in.)	4 Amplified drift times 0.568 (in.)	5 Allowable drift (in.)
R	4.86	0.15	0.81	0.46	3.00
12	4.71	0.24	1.30	0.74	3.00
11	4.47	0.30	1.64	0.93	3.00
10	4.17	0.36	1.96	1.11	3.00
9	3.82	0.37	2.05	1.16	3.00
8	3.44	0.46	2.54	1.44	3.00
7	2.98	0.48	2.64	1.50	3.00
6	2.50	0.48	2.62	1.49	3.00
5	2.03	0.45	2.49	1.42	3.00
4	1.57	0.48	2.66	1.51	3.00
3	1.09	0.48	2.64	1.50	3.00
2	0.61	0.61	3.35	1.90	4.32

Column 4 adjusts for *Standard* Eq. 12.8-3 (for drift) versus Eq. 12.8-5 (for strength).
1.0 in. = 25.4 mm.

C_d Amplified drift based on forces from Eq. 12.8-5

Modified for forces based on Eq. 12.8-3

P-Delta Effects

$$\theta = \frac{P_x \Delta I}{V_x h_{sx} C_d}$$ Eq. 12.8-16*

The drift Δ in Eq. 12.8-16 is drift from ELF analysis, multiplied by C_d and divided by I.

*The importance factor *I* was inadvertently left out of Eq. 12.8-16 in ASCE 7-05. It is properly included in ASCE 7-10.

$$\theta_{\max} = \frac{0.5}{\beta C_d}$$ Eq. 12.8-17

The term β in Eq. 12.8-17 is essentially the inverse of the Computed story over-strength.

P-Delta Effects for modal response spectrum analysis and modal response history analysis are checked using the ELF procedure indicated on this slide.

P-Delta Effects

Table 4.1-11 Computation of P-Delta Effects for X Direction Response

Level	h_{sx} (in.)	Δ (in.)	P_D (kips)	P_L (kips)	P_T (kips)	P_X (kips)	V_X(kips)	θ_X
R	150	1.74	1656.5	315.0	1971.5	1971.5	186.9	0.022
12	150	2.48	1595.8	315.0	1910.8	3882.3	340.9	0.034
11	150	3.07	1595.8	315.0	1910.8	5793.1	470.8	0.046
10	150	3.39	1595.8	315.0	1910.8	7703.9	578.4	0.055
9	150	3.20	3403.0	465.0	3868.0	11571.9	764.7	0.059
8	150	3.47	2330.8	465.0	2795.8	14367.7	865.8	0.070
7	150	3.54	2330.8	465.0	2795.8	17163.5	942.5	0.078
6	150	3.47	2330.8	465.0	2795.8	19959.3	998.8	0.084
5	150	2.95	4323.8	615.0	4938.8	24898.1	1070.2	0.083
4	150	2.97	3066.1	615.0	3681.1	28579.2	1101.7	0.093
3	150	2.90	3066.1	615.0	3681.1	32260.3	1118.2	0.101
2	216	3.51	3097.0	615.0	3712.0	35972.3	1124.5	0.095

1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

Marginally exceeds limit of 0.091 using β=1.0. θ would be less than θ_{max} if actual β were computed and used.

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Orthogonal Loading Requirements

12.5.4 Seismic Design Categories D through F. Structures assigned to Seismic Design Category D, E, or F shall, as a minimum, conform to the requirements of Section 12.5.3.

12.5.3 Seismic Design Category C. Loading applied to structures assigned to Seismic Design Category C shall, as a minimum, conform to the requirements of Section 12.5.2 for Seismic Design Category B and the requirements of this section. *Structures that have horizontal structural irregularity Type 5* in Table 12.3-1 shall the following procedure [for ELF Analysis]:

Continued on Next Slide

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Orthogonal Loading Requirements (continued)

Orthogonal Combination Procedure. The structure shall be analyzed using the equivalent lateral force analysis procedure of Section 12.8 with the loading applied independently in any two orthogonal directions and the most critical load effect due to direction of application of seismic forces on the structure is permitted to be assumed to be satisfied if components and their foundations are designed for the following combination of prescribed loads: ***100 percent of the forces for one direction plus 30 percent of the forces for the perpendicular direction***; the combination requiring the maximum component strength shall be used.

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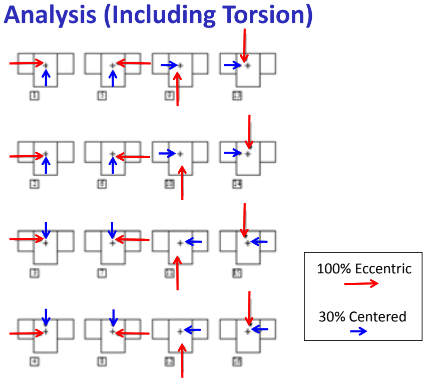
ASCE 7-05 Horizontal Irregularity Type 5

Nonparallel Systems-Irregularity is defined to exist where the vertical lateral force-resisting elements are not parallel to ***or symmetric about*** the major orthogonal axes of the seismic force–resisting system.

The system in question clearly has nonsymmetrical lateral force resisting elements so a Type 5 Irregularity exists, and orthogonal combinations are required. *Thus, 100%-30% procedure given on the previous slide is used.*

Note: The words "or symmetric about" have been removed from the definition of a Type 5 Horizontal Irregularity in ASCE 7-10. Thus, the system under consideration does not have a Type 5 irregularity in ASCE 7-10.

16 Basic Load Combinations used in ELF Analysis (Including Torsion)



Combination of Load Effects

$$1.2D + 1.0E + 0.5L + \cancel{0.2S}$$
$$0.9D + 1.0E + \cancel{1.6H}$$

$$E = E_h + E_v$$
$$E_h = \rho Q_E \quad (\rho = 1.0)$$
$$E_v = 0.2S_{DS} \quad (S_{DS}=0.833g)$$

Redundancy Factor

12.3.4.2 Redundancy Factor, *ρ*, *for Seismic Design Categories* D through F. For structures assigned to Seismic Design Category D, E, or F, ρ shall equal 1.3 unless one of the following two conditions is met, whereby ρ is permitted to be taken as 1.0:

a) Each story resisting more than 35 percent of the base shear in the direction of interest shall comply with Table 12.3-3.

} See next slide

b) Structures that are regular in plan at all levels provided that the seismic force–resisting systems consist of at least two bays of seismic force–resisting perimeter framing on each side of the structure in each orthogonal direction at each story resisting more than 35 percent of the base shear. The number of bays for a shear wall shall be calculated as the length of shear wall divided by the story height or two times

Structure is NOT regular at all Levels.

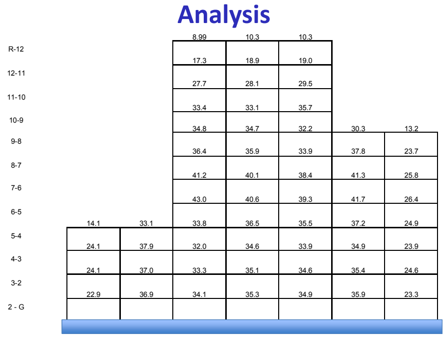
Redundancy, Continued

TABLE 12.3-3 REQUIREMENTS FOR EACH STORY RESISTING MORE THAN 35% OF THE BASE SHEAR

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).

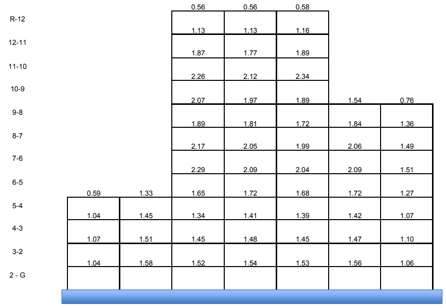
It can be seen by inspection that removal of one beam in this structure will not result in a result in a significant loss of strength or lead to an extreme torsional irregularity. **Hence ρ = 1 for this system**. (This is applicable to ELF, MRS, and MRH analyses).

Seismic Shears in Beams of Frame 1 from ELF Analysis



Seismic Shears in Girders, kips, *Excluding Accidental Torsion*

Seismic Shears in Beams of Frame 1 from ELF Analysis



Seismic Shears in Girders, kips, *Accidental Torsion Only*

Overview of Presentation

- Describe Building
- Describe/Perform steps common to all analysis types
- Overview of Equivalent Lateral Force analysis
- **Overview of Modal Response Spectrum Analysis**
- Overview of Modal Response History Analysis
- Comparison of Results
- Summary and Conclusions

Modal Response Spectrum Analysis Part 1: Analysis

1. Develop ***Elastic*** response spectrum (Sec. 11.4.5)
2. Develop adequate finite element model (Sec. 12.7.3)
3. Compute modal frequencies, effective mass, and mode shapes
4. Determine number of modes to use in analysis (Sec. 12.9.1)
5. Perform modal analysis in each direction, combining each direction's results by use of CQC method (Sec. 12.9.3)
6. Compute Equivalent Lateral Forces (ELF) in each direction (Sec. 12.8.1 through 12.8.3)
7. Determine accidental torsions (Sec 12.8.4.2), amplified if necessary (Sec. 12.8.4.3)
8. Perform static Torsion analysis

Modal Response Spectrum Analysis Part 2: Drift and P-Delta for Systems *Without* Torsion Irregularity

1. Multiply all dynamic displacements by C_d/R (Sec. 12.9.2).
2. Compute SRSS of interstory drifts based on displacements at center of mass at each level.
3. Check drift Limits in accordance with Sec. 12.12 and Table 12.2-1. Note: drift Limits for Special Moment Frames in SDC D and above must be divided by the Redundancy Factor (Sec. 12.12.1.1)
4. Perform P-Delta analysis using Equivalent Lateral Force procedure
5. Revise structure if necessary

Note: when centers of mass of adjacent levels are not vertically aligned the drifts should be based on the difference between the displacement at the upper level and the displacement of the point on the level below which is the vertical projection of the center of mass of the upper level. (This procedure is included in ASCE 7-10.)

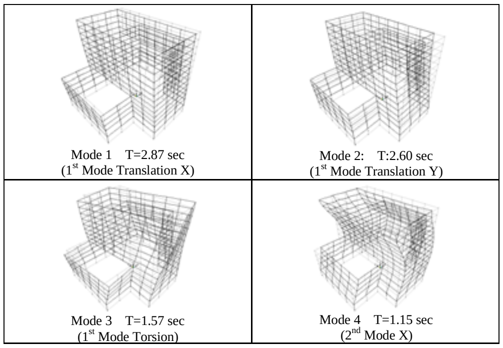
Modal Response Spectrum Analysis Part 2: Drift and P-Delta for Systems *With* Torsion Irregularity

1. Multiply all dynamic displacements by C_d/R (Sec. 12.9.2).
2. Compute SRSS of story drifts based on displacements at the *edge* of the building
3. Using results from the static torsion analysis, determine the drifts at the same location used in Step 2 above. Torsional drifts may be based on the computed period of vibration (without the C_uT_a limit). Torsional drifts should be based on computed displacements multiplied by C_d and divided by *I*.
4. Add drifts from Steps 2 and 3 and check drift limits in Table 12.12-1. Note: Drift limits for special moment frames in SDC D and above must be divided by the Redundancy Factor (Sec. 12.12.1.1)
5. Perform P-Delta analysis using Equivalent Lateral Force procedure
6. Revise structure if necessary

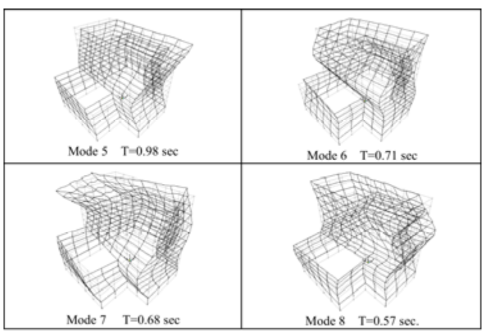
Modal Response Spectrum Analysis Part 3: Obtaining Member Design Forces

1. Multiply all dynamic force quantities by I/R (Sec. 12.9.2)
2. Determine dynamic base shears in each direction
3. Compute scale factors for each direction (Sec. 12.9.4) and apply to respective member force results in each direction
4. Combine results from two orthogonal directions, if necessary (Sec. 12.5)
5. Add member forces from static torsion analysis (Sec. 12.9.5). Note that static torsion forces may be scaled by factors obtained in Step 3
6. Determine redundancy factor (Sec. 12.3.4)
7. Combine seismic and gravity forces (Sec. 12.4)
8. Design and detail structural components

Mode Shapes for First Four Modes



Mode Shapes for Modes 5-8



Number of Modes to Include in Response Spectrum Analysis

12.9.1 Number of Modes

An analysis shall be conducted to determine the natural modes of vibration for the structure. 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.

Effective Masses for First 12 Modes

Table 4.1-13 Computed Periods and Effective Mass Factors (Lower Modes)

Mode	Period (seconds)	Effective Mass Factor, [Accum Mass Factor]		
		X Translation	Y Translation	Z Rotation
1	2.87	0.6446 [0.64]	0.0003 [0.00]	0.0028 [0.00]
2	2.60	0.0003 [0.65]	0.6804 [0.68]	0.0162 [0.02]
3	1.57	0.0035 [0.65]	0.0005 [0.68]	0.5806 [0.60]
4	1.15	0.1085 [0.76]	0.0000 [0.68]	0.0000 [0.60]
5	0.975	0.0000 [0.76]	0.0939 [0.78]	0.0180 [0.62]
6	0.705	0.0263 [0.78]	0.0000 [0.78]	0.0271 [0.64]
7	0.682	0.0056 [0.79]	0.0006 [0.79]	0.0687 [0.71]
8	0.573	0.0000 [0.79]	0.0188 [0.79]	0.0123 [0.73]
9	0.434	0.0129 [0.80]	0.0000 [0.79]	0.0084 [0.73]
10	0.387	0.0048 [0.81]	0.0000 [0.79]	0.0191 [0.75]
11	0.339	0.0000 [0.81]	0.0193 [0.81]	0.0010 [0.75]
12	0.300	0.0089 [0.82]	0.0000 [0.81]	0.0003 [0.75]

12 Modes Appears to be Insufficient

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Effective Masses for Modes 108-119

Table 4.1-14 Computed Periods and Effective Mass Factors (Higher Modes)

Mode	Period (seconds)	Effective Mass Factor, [Accum Effective Mass]		
		X Translation	Y Translation	Z Rotation
108	0.0693	0.0000 [0.83]	0.0000 [0.83]	[illegible]
109	0.0673	0.0000 [0.83]	0.0000 [0.83]	[illegible]
110	0.0671	0.0000 [0.83]	0.0354 [0.86]	0.0000 [0.79]
111	0.0671	0.0000 [0.83]	0.0044 [0.87]	0.0000 [0.79]
112	0.0669	0.0000 [0.83]	0.1045 [0.97]	0.0000 [0.79]
113	0.0663	0.0000 [0.83]	0.0000 [0.97]	0.0000 [0.79]
114	0.0646	0.0000 [0.83]	0.0000 [0.97]	0.0000 [0.79]
115	0.0629	0.0000 [0.83]	0.0000 [0.97]	0.0000 [0.79]
116	0.0621	0.0008 [0.83]	0.0010 [0.97]	0.0000 [0.79]
117	0.0609	0.0014 [0.83]	0.0009 [0.97]	0.0000 [0.79]
118	0.0575	0.1474 [0.98]	0.0000 [0.97]	0.0035 [0.80]
119	0.0566	0.0000 [0.98]	0.0000 [0.97]	0.0000 [0.80]

Virtually the Same as 12 Modes

118 Modes Required to Capture Dynamic Response of Stiff Basement Level and Grade Level Slab

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Effective Masses for First 12 Modes

Table 4.1-13 Computed Periods and Effective Mass Factors (Lower Modes)

Mode	Period (seconds)	Effective Mass Factor, [Accum Mass Factor]		
		X Translation	Y Translation	Z Rotation
1	2.87	0.6446 [0.64]	0.0003 [0.00]	0.0028 [0.00]
2	2.60	0.0003 [0.65]	0.6804 [0.68]	0.0162 [0.02]
3	1.57	0.0035 [0.65]	0.0005 [0.68]	0.5806 [0.60]
4	1.15	0.1085 [0.76]	0.0000 [0.68]	0.0000 [0.60]
5	0.975	0.0000 [0.76]	0.0939 [0.78]	0.0180 [0.62]
6	0.705	0.0263 [0.78]	0.0000 [0.78]	0.0271 [0.64]
7	0.682	0.0056 [0.79]	0.0006 [0.79]	0.0687 [0.71]
8	0.573	0.0000 [0.79]	0.0188 [0.79]	0.0123 [0.73]
9	0.434	0.0129 [0.80]	0.0000 [0.79]	0.0084 [0.73]
10	0.387	0.0048 [0.81]	0.0000 [0.79]	0.0191 [0.75]
11	0.339	0.0000 [0.81]	0.0193 [0.81]	0.0010 [0.75]
12	0.300	0.0089 [0.82]	0.0000 [0.81]	0.0003 [0.75]

12 Modes are Actually Sufficient to Represent the Dynamic Response of the *Above Grade* Structure

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Inelastic Design Response Spectrum Coordinates

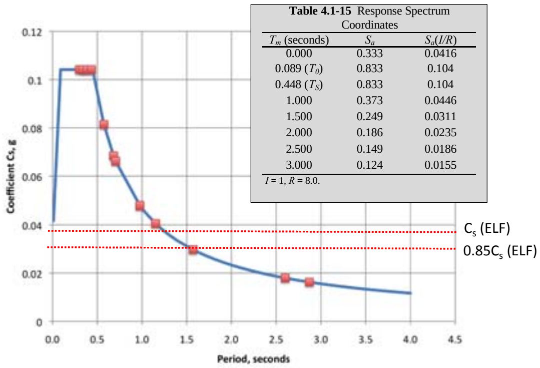


Table 4.1-15 Response Spectrum Coordinates

T_m (seconds)	S_a	$S_a(I/R)$
0.000	0.333	0.0416
0.089 (T_0)	0.833	0.104
0.448 (T_S)	0.833	0.104
1.000	0.373	0.0446
1.500	0.249	0.0311
2.000	0.186	0.0235
2.500	0.149	0.0186
3.000	0.124	0.0155

$I = 1$, $R = 8.0$.

Scaling of Response Spectrum Results (ASCE 7-05)

12.9.4 Scaling Design Values of Combined Response.
A base shear (V) shall be calculated in each of the two orthogonal horizontal directions using the calculated fundamental period of the structure T in each direction and the procedures of Section 12.8, except where the calculated fundamental period exceeds $(C_u)(T_a)$, then $(C_u)(T_a)$ shall be used in lieu of T in that direction. Where the combined response for the modal base shear (V_t) is less than 85 percent of the calculated base shear (V) using the equivalent lateral force procedure, the forces, but not the drifts, shall be multiplied by

$$0.85\frac{V}{V_t}$$

where

- V = the equivalent lateral force procedure base shear, calculated in accordance with this section and Section 12.8
- V_t = the base shear from the required modal combination

Note: If the ELF base shear is governed by Eqn. 12.5-5 or 12.8-6 the force V shall be based on the value of C_s calculated by Eqn. 12.5-5 or 12.8-6, as applicable.

Scaling of Response Spectrum Results (ASCE 7-10)

12.9.4.2 Scaling of Drifts

Where the combined response for the modal base shear (V_t) is less than 0.85 C_sW, and where C_s is determined in accordance with Eq. 12.8-6, ***drifts shall be multiplied by***:

$$0.85\frac{C_sW}{V_t}$$

Scaled Static Torsions

Apply Torsion as a Static Load. Torsions can be Scaled to 0.85 times Amplified* EFL Torsions if the Response Spectrum Results are Scaled.

* See Sec. 12.9.5. Torsions must be amplified because they are applied statically, not dynamically.

Method 1: Weighted Addition of *Scaled* CQC'd Results

A = *Scaled* CQC'd Results in X Direction

B = *Scaled* CQC'd Results in Y Direction

Combination 1

Combination 2

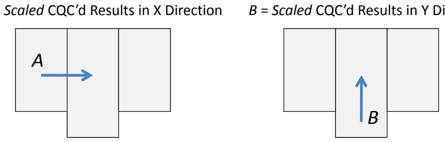
$A + 0.3B + |T_X|$

$0.3A + B + |T_Y|$

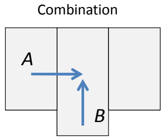
Method 2: SRSS of *Scaled* CQC'd Results

A = *Scaled* CQC'd Results in X Direction

B = *Scaled* CQC'd Results in Y Direction



Combination



$(A^2+B^2)^{0.5} + \max(|T_X| \text{ or } |T_Y|)$

Computed Story Shears and Scale Factors from Modal Response Spectrum Analysis

Table 4.1-16 Story Shears from Modal Response Spectrum Analysis

Story	X Direction (SF = 2.18) Unscaled Shear (kips)	X Direction (SF = 2.18) Scaled Shear (kips)	Y Direction (SF = 1.94) Unscaled Shear (kips)	Y Direction (SF = 1.94) Scaled Shear (kips)
R-12	82.7	180	77.2	150
12-11	130.9	286	132.0	256
11-10	163.8	357	170.4	330
10-9	191.4	418	201.9	392
9-8	240.1	524	265.1	514
8-7	268.9	587	301.4	585
7-6	292.9	639	328.9	638
6-5	316.1	690	353.9	686
5-4	359.5	784	405.1	786
4-3	384.8	840	435.5	845
3-2	401.4	895	462.8	898
2-G	438.1	956	492.8	956

1.0 kip = 4.45 kN.

X-Direction Scale Factor = 0.85(1124)/438.1=2.18

Y-Direction Scale Factor = 0.85(1124)/492.8=1.94

Response Spectrum Drifts in X Direction

(No Scaling Required)

Level	Total Drift from R.S. Analysis (in.)	Story Drift (in.)	Story Drift × C_d (in.)	Allowable Story Drift (in.)
R	2.23	0.12	0.66	3.00
12	2.10	0.16	0.89	3.00
11	1.94	0.19	1.03	3.00
10	1.76	0.20	1.08	3.00
9	1.56	0.18	0.98	3.00
8	1.38	0.19	1.06	3.00
7	1.19	0.20	1.08	3.00
6	0.99	0.20	1.08	3.00
5	0.80	0.18	0.97	3.00
4	0.62	0.19	1.02	3.00
3	0.43	0.19	1.05	3.00
2	0.24	0.24	1.34	4.32

1.0 in. = 25.4 mm

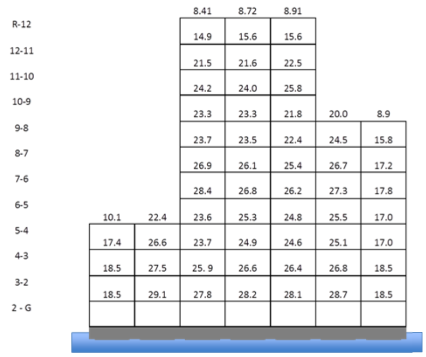
Response Spectrum Drifts in Y Direction

(No Scaling Required)

Level	Total Drift from R.S. Analysis (in.)	Story Drift (in.)	Story Drift × C_d (in.)	Allowable Story Drift (in.)
R	1.81	0.06	0.32	3.00
12	1.76	0.09	0.49	3.00
11	1.67	0.11	0.58	3.00
10	1.56	0.12	0.67	3.00
9	1.44	0.13	0.70	3.00
8	1.31	0.16	0.87	3.00
7	1.15	0.17	0.91	3.00
6	0.99	0.17	0.92	3.00
5	0.92	0.17	0.93	3.00
4	0.65	0.19	1.04	3.00
3	0.46	0.20	1.08	3.00
2	0.26	0.26	1.44	4.32

1.0 in. = 25.4 mm

Scaled Beam Shears from Modal Response Spectrum Analysis



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Overview of Presentation

- Describe Building
- Describe/Perform steps common to all analysis types
- Overview of Equivalent Lateral Force analysis
- Overview of Modal Response Spectrum Analysis
- **Overview of Modal Response History Analysis**
- Comparison of Results
- Summary and Conclusions

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Modal Response History Analysis Part 1: Analysis

1. Select suite of ground motions (Sec. 16.1.3.2)
2. Develop adequate finite element model (Sec. 12.7.3)
3. Compute modal frequencies, effective mass, and mode Shapes
4. Determine number of modes to use in analysis (Sec. 12.9.1)
5. Assign modal damping values (typically 5% critical per mode)
6. Scale ground motions* (Sec. 16.1.3.2)
7. Perform dynamic analysis for each ground motion in each direction
8. Compute Equivalent Lateral Forces (ELF) in each direction (Sec. 12.8.1 through 12.8.3)
9. Determine accidental torsions (Sec 12.8.4.2), amplified if necessary (Sec. 12.8.4.3)
10. Perform static torsion analysis

*Note: Step 6 is referred to herein as Ground Motion Scaling (GM Scaling). This is to avoid confusion with Results Scaling, described later.

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Modal Response History Analysis Part 2: Drift and P-Delta for Systems *Without* Torsion Irregularity

1. Multiply all dynamic displacements by C_d/R (omitted in ASCE 7-05).
2. Compute story drifts based on displacements at center of mass at each level
3. If 3 to 6 ground motions are used, compute envelope of story drift at each level in each direction (Sec. 16.1.4)
4. If 7 or more ground motions are used, compute average story drift at each level in each direction (Sec. 16.1.4)
5. Check drift limits in accordance with Sec. 12.12 and Table 12.2-1. Note: drift limits for Special Moment Frames in SDC D and above must be divided by the Redundancy Factor (Sec. 12.12.1.1)
6. Perform P-Delta analysis using Equivalent Lateral Force procedure
7. Revise structure if necessary

Note: when centers of mass of adjacent levels are not vertically aligned the drifts should be based on the difference between the displacement at the upper level and the displacement of the point on the level below which is the vertical projection of the center of mass of the upper level.(This procedure is included in ASCE 7-10.)

Modal Response History Analysis Part 2: Drift and P-Delta for Systems *With* Torsion Irregularity

1. Multiply all dynamic displacements by C_d/R (omitted in ASCE 7-05).
2. Compute story drifts based on displacements at edge of building at each level
3. If 3 to 6 ground motions are used, compute envelope of story drift at each level in each direction (Sec. 16.1.4)
4. If 7 or more ground motions are used, compute average story drift at each level in each direction (Sec. 16.1.4)
5. Using results from the static torsion analysis, determine the drifts at the same location used in Steps 2-4 above. Torsional drifts may be based on the computed period of vibration (without the C_uT_a limit). Torsional drifts should be based on computed displacements multiplied by C_d and divided by *I*.
6. Add drifts from Steps (3 or 4) and 5 and check drift limits in Table 12.12-1. Note: Drift limits for special moment frames in SDC D and above must be divided by the Redundancy Factor (Sec. 12.12.1.1)
7. Perform P-Delta analysis using Equivalent Lateral Force procedure
8. Revise structure if necessary

Modal Response History Analysis Part 3: Obtaining Member Design Forces

1. Multiply all dynamic member forces by *I*/*R*
2. Determine dynamic base shear histories for each earthquake in each direction
3. Determine Result Scale Factors* for each ground motion in each direction, and apply to response history results as appropriate
4. Determine design member forces by use of envelope values if 3 to 6 earthquakes are used, or as averages if 7 or more ground motions are used.
5. Combine results from two orthogonal directions, if necessary (Sec. 12.5)
6. Add member forces from static torsion analysis (Sec. 12.9.5). Note that static torsion forces may be scaled by factors obtained in Step 3
7. Determine redundancy factor (Sec. 12.3.4)
8. Combine seismic and gravity forces (Sec. 12.4)
9. Design and detail structural components

*Note: Step 3 is referred to herein as Results Scaling (GM Scaling). This is to avoid confusion with Ground Motion Scaling, described earlier.

Selection of Ground Motions for MRH Analysis

16.1.3.2 Three-Dimensional Analysis

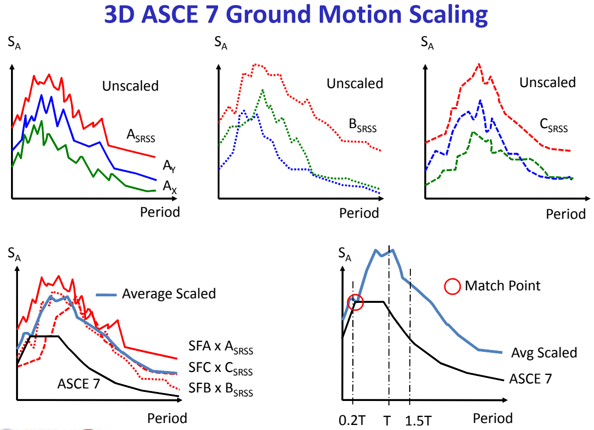
Where three-dimensional analyses are performed, ground motions shall consist of pairs of appropriate horizontal ground motion acceleration components that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the maximum considered earthquake. Where the required number of recorded ground motion pairs is not available, appropriate simulated ground motion pairs are permitted to be used to make up the total number required.

3D Scaling Requirements, ASCE 7-10

For each pair of horizontal ground motion components, a square root of the sum of the squares (SRSS) spectrum shall be constructed by taking the SRSS of the 5 percent-damped response spectra for the scaled components (where an identical scale factor is applied to both components of a pair). Each pair of motions shall be scaled such that in the period range from 0.2*T* to 1.5*T, the average of the SRSS spectra from all* horizontal component pairs does not fall below the corresponding ordinate of the response spectrum used in the design, determined in accordance with Section 11.4.5.

ASCE 7-05 Version:
does not fall below 1.3 times the corresponding ordinate of the design response spectrum, determined in accordance with Section 11.4.5 by more than 10 percent.

3D ASCE 7 Ground Motion Scaling



Issues With Scaling Approach

- No guidance is provided on how to deal with different fundamental periods in the two orthogonal directions
- There are an infinite number of sets of scale factors that will satisfy the criteria. Different engineers are likely to obtain different sets of scale factors for the same ground motions.
- In linear analysis, there is little logic in scaling at periods greater than the structure's fundamental period.
- Higher modes, which participate marginally in the dynamic response, may dominate the scaling process

Resolving Issues With Scaling Approach

No guidance is provided on how to deal with different fundamental periods in the two orthogonal directions:

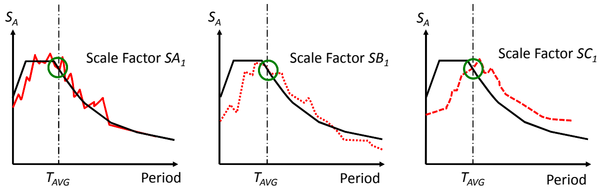
1. Use different periods in each direction (not recommended)
2. Scale to range 0.2 T_{min} to 1.5 T_{max} where T_{min} is the lesser of the two periods and T_{max} is the greater of the fundamental periods in each principal direction
3. Scale over the range $0.2T_{Avg}$ to 1.5 T_{Avg} where T_{Avg} is the average of T_{min} and T_{max}

Resolving Issues With Scaling Approach

There are an infinite number of sets of scale factors that will satisfy the criteria. Different engineers are likely to obtain different sets of scale factors for the same ground motions.

Use Two-Step Scaling:

1] Scale each SRSS'd Pair to the Average Period



Note: A different scale factor will be obtained for each SRSS'd pair

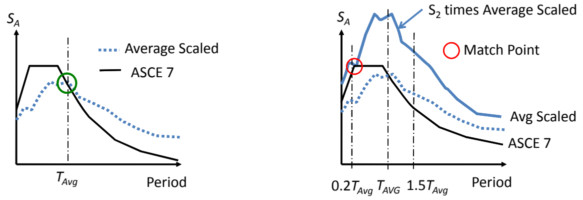
Resolving Issues With Scaling Approach

There are an infinite number of sets of scale factors that will satisfy the criteria. Different engineers are likely to obtain different sets of scale factors for the same ground motions.

Use Two-Step Scaling:

1] Scale each SRSS'd Pair to the Average Period

2] Obtain Suite Scale Factor S_2



Note: The same scale factor S_2 Applies to Each SRSS'd Pair

Resolving Issues With Scaling Approach

There are an infinite number of sets of scale factors that will satisfy the criteria. Different engineers are likely to obtain different sets of scale factors for the same ground motions.

Use Two-Step Scaling:

1] Scale each SRSS'd Pair to the Average Period

2] Obtain Suite Scale Factor S_2

3] Obtain Final Scale Factors:

Suite A: $SS_A=S_{A1} \times S_2$

Suite B: $SS_B=S_{B1} \times S_2$

Suite C: $SS_C=S_{C1} \times S_2$

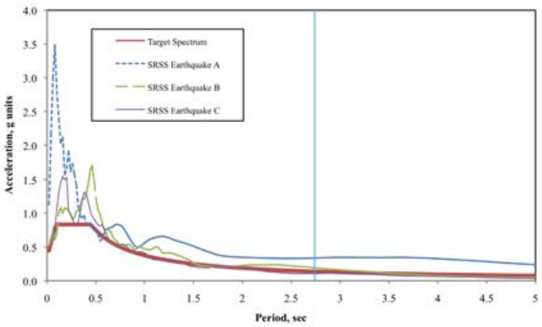
Ground Motions Used in Analysis

Table 4.1-20a. Suite of Ground Motions Used for Response History Analysis

NGA Record Number	Magnitude [Epicenter Distance, km]	Site Class	Number of Points and Digitization Increment	Component Source Motion	PGA (g)	Record Name (This Example)
0879	7.28	C	9625 @ 0.005 sec	Landers/LCN260*	0.727	A00
	[44]			Landers/LCN345*	0.789	A90
0725	6.54	D	2230 @ 0.01 sec	SUPERST/B-POE270	0.446	B00
	[11.2]			SUPERST/B-POE360	0.300	B90
0139	7.35	C	1192 @ 0.02 sec	TABAS/DAY-LN	0.328	C00
	[21]			TABAS/DAY-TR	0.406	C90

* Note that the two components of motion for the Landers earthquake are apparently separated by an 85 degree angle, not 90 degrees as is traditional. It is not known whether these are true orientations, or of there is an error in the descriptions provided in the NGA database.

Unscaled Spectra



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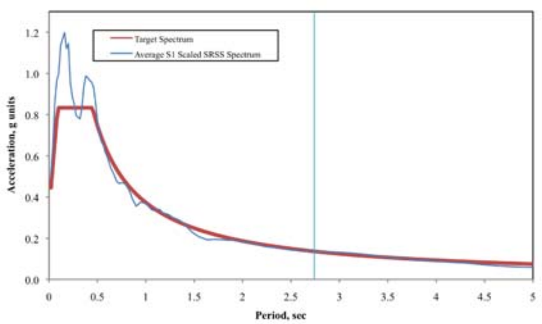
Average S1 Scaled Spectra



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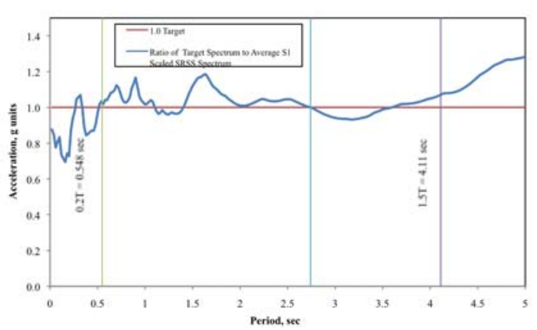
Ratio of Target Spectrum to Scaled SRSS Average



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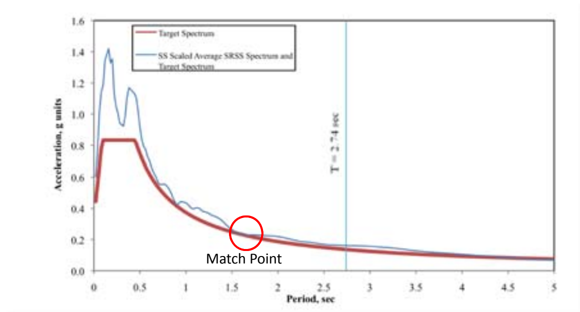
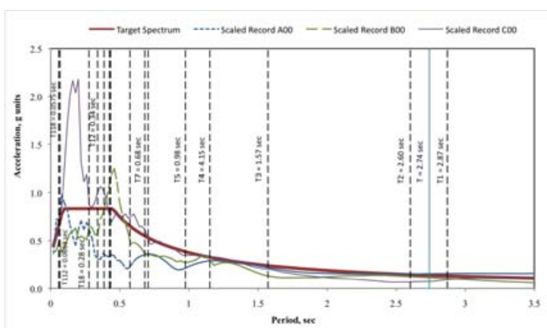
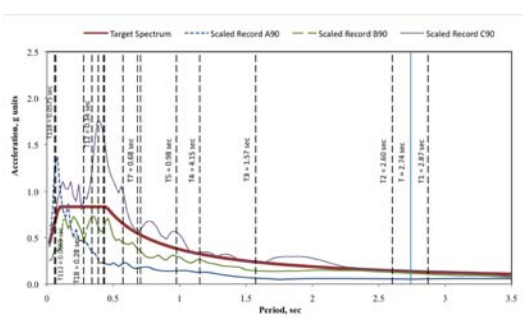
Target Spectrum and SS Scaled Average



Individual Scaled Components (00)



Individual Scaled Components (90)



Computed Scale Factors

Table 4.1-20b. Result of 3D Scaling Process

Set No.	Designation	SRSS ordinate at $T=T_{Avg}$ (g)	Target Ordinate at $T=T_{Avg}$ (g)	S1	S2	SS
1	A00 & A90	0.335	0.136	0.407	1.184	0.482
2	B00 & B90	0.191	0.136	0.712	1.184	0.843
3	C00 & C90	0.104	0.136	1.310	1.184	1.551

Number of Modes for Modal Response History Analysis

ASCE 7-05 and 7-10 are silent on the number of modes to use in Modal Response History Analysis. It is recommended that the same procedures set forth in Section 12.9.1 for MODAL Response Spectrum Analysis be used for Response History Analysis:

12.9.1 Number of Modes

An analysis shall be conducted to determine the natural modes of vibration for the structure. **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.

Damping for Modal Response History Analysis

ASCE 7-05 and 7-10 are silent on the amount of damping to use in Modal Response History Analysis.

Five percent critical damping should be used in all modes considered in the analysis because the Target Spectrum and the Ground Motion Scaling Procedures are based on 5% critical damping.

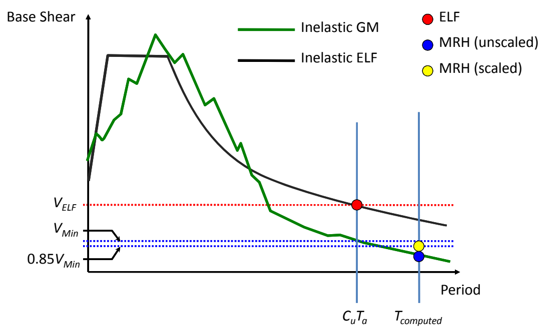
Scaling of Results for Modal Response History Analysis (Part 1)

The structural analysis is executed using the GM scaled earthquake records in each direction. Thus, the results represent the expected elastic response of the structure. The results must be scaled to represent the expected inelastic behavior and to provide improved performance for important structures. ASCE 7-05 scaling is as follows:

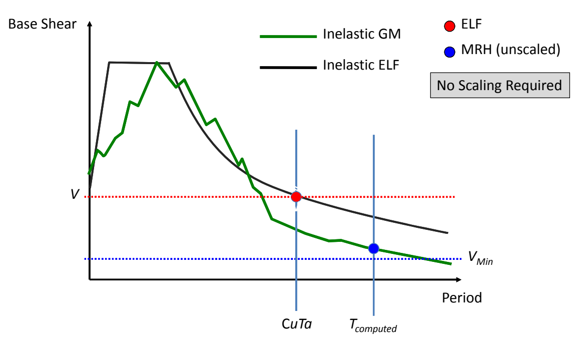
1) Scale all component design forces by the factor (I/R). This is stipulated in Sec. 16.1.4 of ASCE 7-05 and ASCE 7-10.

2) Scale all displacement quantities by the factor (C_d/R). This requirement was inadvertently omitted in ASCE 7-05, but is included in Section 16.1.4 of ASCE 7-10.

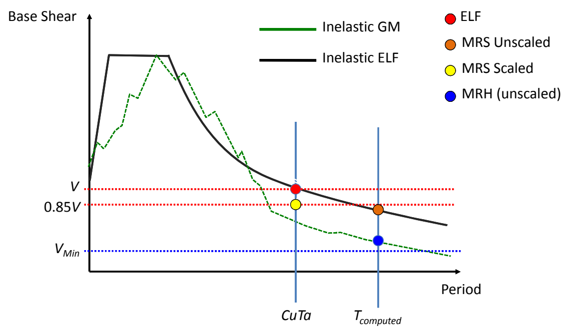
Response Scaling Requirements when MRH Shear is *Less Than* Minimum Base Shear



Response Scaling Requirements when MRH Shear is *Greater Than* Minimum Base Shear



Response Scaling Requirements when MRH Shear is *Greater Than* Minimum Base Shear



12 Individual Response History Analyses Required

1. A00-X: SS Scaled Component A00 applied in X Direction
2. A00-Y: SS Scaled Component A00 applied in Y Direction
3. A90-X: SS Scaled Component A90 applied in X Direction
4. A90-Y: SS Scaled Component A90 applied in Y Direction

5. B00-X: SS Scaled Component B00 applied in X Direction
6. B00-Y: SS Scaled Component B00 applied in Y Direction
7. B90-X: SS Scaled Component B90 applied in X Direction
8. B90-Y: SS Scaled Component B90 applied in Y Direction

9. C00-X: SS Scaled Component C00 applied in X Direction
10. C00-Y: SS Scaled Component C00 applied in Y Direction
11. C90-X: SS Scaled Component C90 applied in X Direction
12. C90-Y: SS Scaled Component C90 applied in Y Direction

Result Maxima from Response History Analysis Using SS Scaled Ground Motions

	Analysis	Maximum base shear (kips)	Time of maximum shear (sec.)	Maximum roof displacement (in.)	Time of maximum displacement (sec.)
	A00-X	3507	11.29	20.28	11.38
	A00-Y	3573	11.27	14.25	11.28
	A90-X	1588	12.22	7.32	12.70
Low >	A90-Y	1392	13.56	5.16	10.80
	B00-X	3009	8.28	12.85	9.39
	B00-Y	3130	9.37	11.20	10.49
	B90-X	2919	8.85	11.99	7.11
	B90-Y	3460	7.06	11.12	8.20
	C00-X	3130	13.5	9.77	13.54
	C00-Y	2407	4.64	6.76	8.58
	C90-X	3229	6.92	15.61	6.98
High >	C90-Y	5075	6.88	14.31	7.80

1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

I/R Scaled Shears and Required 85% Rule Scale Factors

Analysis	(I/R) times maximum base shear from analysis (kips)	Required additional scale factor for $V = 0.85V_{ELF} = 956$ kips
A00-X	438.4	2.18
A00-Y	446.7	2.14
A90-X	198.5	4.81
A90-Y	173.9	5.49
B00-X	376.1	2.54
B00-Y	391.2	2.44
B90-X	364.8	2.62
B90-Y	432.5	2.21
C00-X	391.2	2.44
C00-Y	300.9	3.18
C90-X	403.6	2.37
C90-Y	634.4	1.51

1.0 kip = 4.45 kN

Response History Drifts for all X-Direction Responses

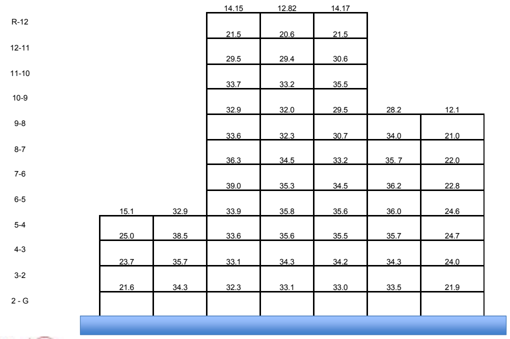
Level	Envelope of drift (in.) for each ground motion						Envelope of drift for all the ground motions	Envelope of drift $\times C_d/R$	Allowable drift (in.)
	A00-X	A90-X	B00-X	B90-X	C00-X	C90-X			
R	1.17	0.49	0.95	0.81	0.91	1.23	1.23	0.85	3.00
12	1.64	0.66	1.22	0.95	1.16	1.27	1.64	1.13	3.00
11	1.97	0.78	1.32	0.99	1.25	1.52	1.97	1.35	3.00
10	2.05	0.86	1.42	1.04	1.20	1.68	2.05	1.41	3.00
9	1.79	0.82	1.26	1.25	0.99	1.41	1.79	1.23	3.00
8	1.83	0.87	1.22	1.42	1.23	1.50	1.83	1.26	3.00
7	1.82	0.83	1.27	1.36	1.21	1.67	1.82	1.25	3.00
6	1.77	0.74	1.36	1.35	1.06	1.94	1.94	1.33	3.00
5	1.50	0.59	1.19	1.21	1.09	1.81	1.81	1.24	3.00
4	1.55	0.62	1.22	1.32	1.23	1.76	1.76	1.21	3.00
3	1.56	0.64	1.24	1.30	1.33	1.60	1.60	1.10	3.00
2	1.97	0.86	1.64	1.58	1.73	1.85	1.97	1.35	4.32

1.0 in. = 25.4 mm.

Load Combinations for Response History Analysis

Load Combination for Response History Analysis					
Earthquake	Load Combination	Loading X Direction		Loading Y Direction	
		Record	Scale Factor	Record	Scale Factor
A	1	A00-X	2.18	A00-Y	5.49
	2	A90-X	-4.81	A90-Y	2.14
	3	A00-X	-2.18	A00-Y	-5.49
	4	A90-X	4.81	A90-Y	-2.14
B	5	B00-X	2.54	B00-Y	2.21
	6	B90-X	-2.62	B90-Y	2.44
	7	B00-X	-2.54	B00-Y	-2.21
	8	B90-X	2.62	B90-Y	-2.44
C	9	C00-X	2.44	C00-y	1.50
	10	C90-X	-2.36	C90-Y	3.18
	11	C00-X	-2.44	C00-Y	-1.50
	12	C90-X	2.36	C90-Y	-3.18

Envelope of Scaled Frame 1 Beam Shears from Response History Analysis



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Overview of Presentation

- Describe Building
- Describe/Perform steps common to all analysis types
- Overview of Equivalent Lateral Force analysis
- Overview of Modal Response Spectrum Analysis
- Overview of Modal Response History Analysis
- **Comparison of Results**
- Summary and Conclusions

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Comparison of Maximum X-Direction Design Story Shears from All Analysis

Level	ELF	Modal response spectrum	Enveloped response history
R	187	180	295
12	341	286	349
11	471	357	462
10	578	418	537
9	765	524	672
8	866	587	741
7	943	639	753
6	999	690	943
5	1,070	784	1,135
4	1,102	840	1,099
3	1,118	895	1,008
2	1,124	956	956

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Comparison of Maximum X-Direction Design Story Drift from All Analysis

Level	X Direction Drift (in.)		
	ELF	Modal response spectrum	Enveloped response history
R	0.99	0.66	0.85
12	1.41	0.89	1.13
11	1.75	1.03	1.35
10	1.92	1.08	1.41
9	1.82	0.98	1.23
8	1.97	1.06	1.26
7	2.01	1.08	1.25
6	1.97	1.08	1.33
5	1.67	0.97	1.24
4	1.69	1.02	1.21
3	1.65	1.05	1.10
2	2.00	1.34	1.35

1.0 in. = 25.4 mm.

Comparison of Maximum Beam Shears from All Analysis

Level	Beam Shear Force in Bay D-E of Frame 1 (kips)		
	ELF	Modal response spectrum	Enveloped response history
R	10.27	8.72	12.82
12	18.91	15.61	20.61
11	28.12	21.61	29.45
10	33.15	24.02	33.22
9	34.69	23.32	32.02
8	35.92	23.47	32.30
7	40.10	26.15	34.53
6	40.58	26.76	35.29
5	36.52	25.29	35.82
4	34.58	24.93	35.65
3	35.08	26.60	34.27
2	35.28	28.25	33.07

1.0 kip = 4.45 kN.

Overview of Presentation

- Describe Building
- Describe/Perform steps common to all analysis types
- Overview of Equivalent Lateral Force analysis
- Overview of Modal Response Spectrum Analysis
- Overview of Modal Response History Analysis
- Comparison of Results
- **Summary and Conclusions**

Required Effort

- The Equivalent Lateral Force method and the Modal Response Spectrum methods require similar levels of effort.
- The Modal Response History Method requires considerably more effort than ELF or MRS. This is primarily due to the need to select and scale the ground motions, and to run so many response history analyses.

Accuracy

It is difficult to say whether one method of analysis is "more accurate" than the others. This is because each of the methods assume linear elastic behavior, and make simple adjustments (using R and C_d) to account for inelastic behavior.

Differences inherent in the results produced by the different methods are reduced when the results are scaled. However, it is likely that the Modal Response Spectrum and Modal Response History methods are generally more accurate than ELF because they more properly account for higher mode response.

Recommendations for Future Considerations

1. Three dimensional analysis should be required for *all* Response Spectrum and Response History analysis.
2. Linear Response History Analysis should be moved from Chapter 16 into Chapter 12 and be made as consistent as possible with the Modal Response Spectrum Method. For example, requirements for the number of modes and for scaling of results should be the same for the two methods.
3. A rational procedure needs to be developed for directly including Accidental Torsion in Response Spectrum and Response History Analysis.
4. A rational method needs to be developed for directly including P-Delta effects in Response Spectrum and Response History Analysis.
5. The current methods of selecting and scaling ground motions for linear response history analysis can be and should be much simpler than required for nonlinear response history analysis. The use of "standardized" motion sets or the use of spectrum matched ground motions should be considered.
6. Drift should always be computed and checked at the corners of the building.

Questions

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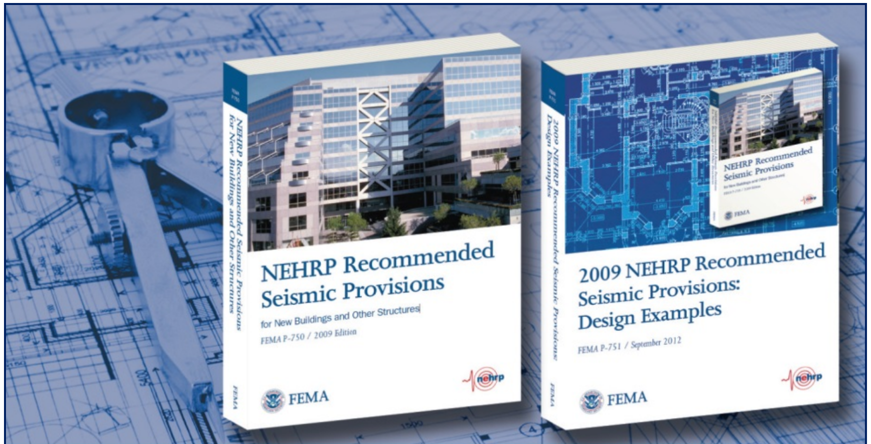


2009 NEHRP Recommended
Seismic Provisions:

Training and Instructional Materials

FEMA P-752 CD / June 2013

FEMA

4

Structural Analysis

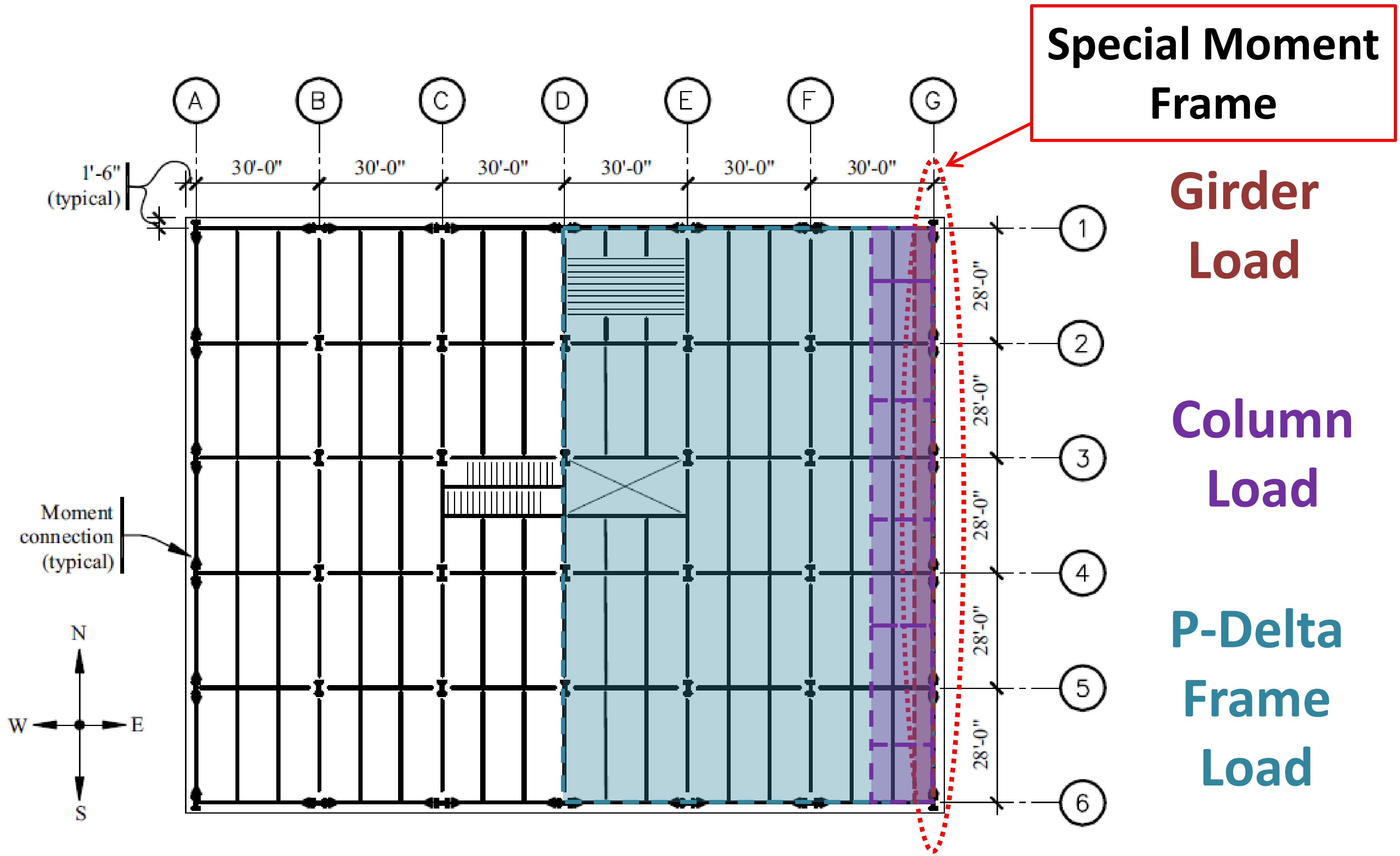
Finley Charney, Adrian Tola Tola, and Ozgur Atlayan

Example 2:
Six-story Moment Resisting Steel Frame

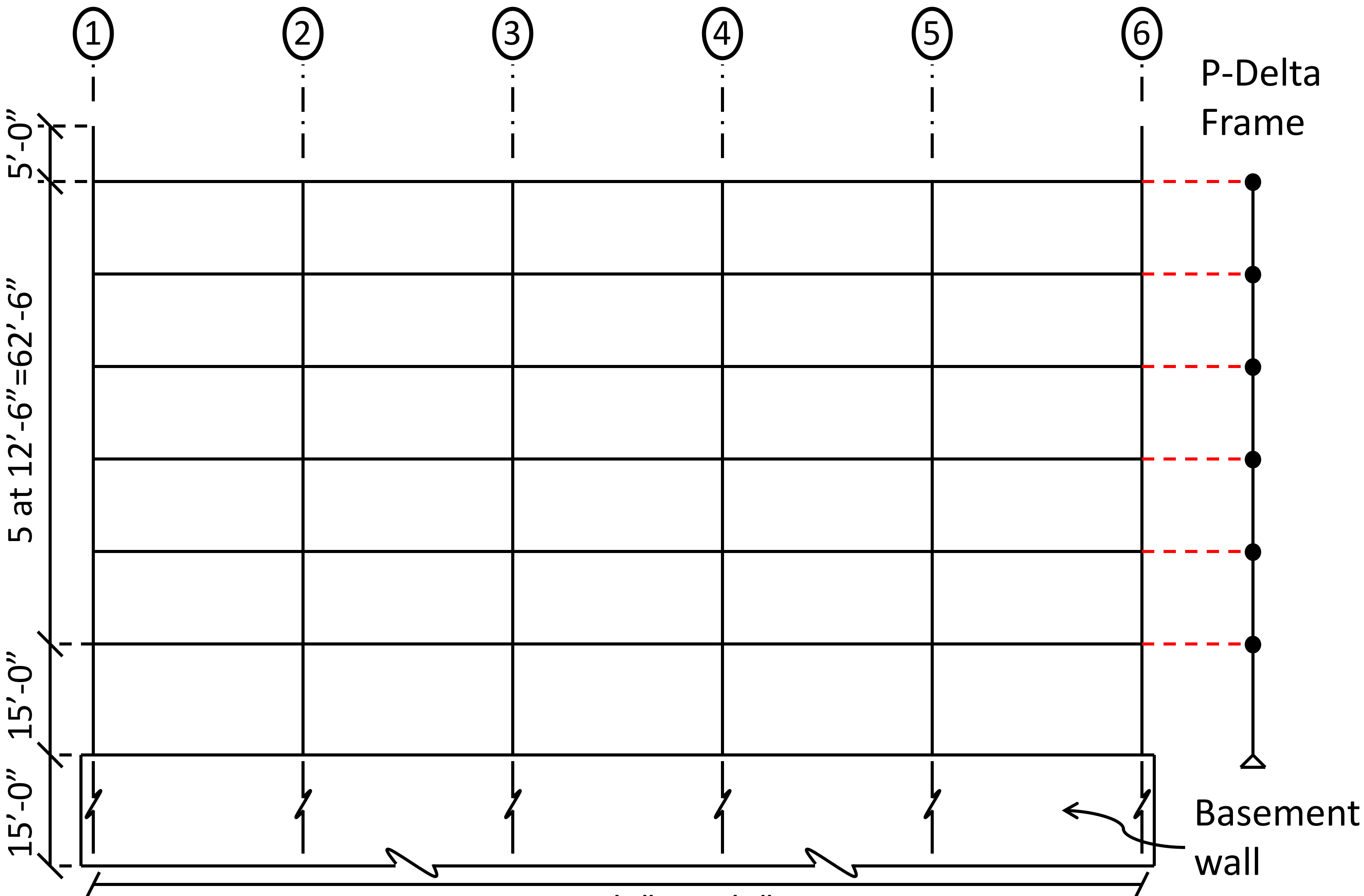
Description of Structure

- 6-story office building in Seattle, Washington
- Occupancy (Risk) Category II
- Importance factor (I) = 1.0
- Site Class = C
- Seismic Design Category D
- Special Moment Frame (SMF), R = 8, C_d = 5.5

Floor Plan and Gravity Loads



Elevation view and P-Delta Column



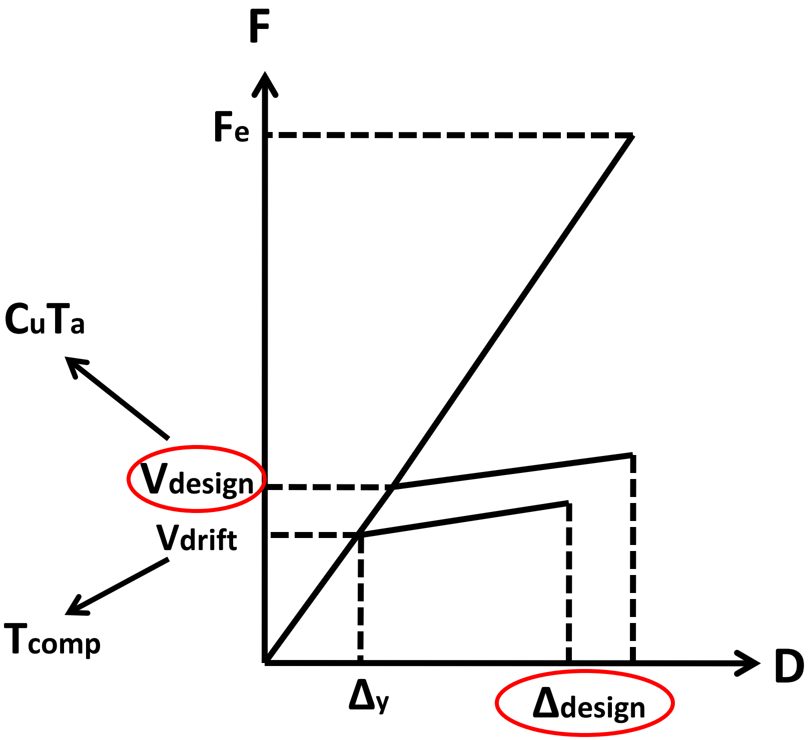
Member Sizes Used in N-S Moment Frames

Member Supporting Level	Column	Girder	Doubler Plate Thickness (in.)
R	W21x122	W24x84	1.00
6	W21x122	W24x84	1.00
5	W21x147	W27x94	1.00
4	W21x147	W27x94	1.00
3	W21x201	W27x94	0.875
2	W21x201	W27x94	0.875

- ✓ Sections meet the width-to-thickness requirements for special moment frames
- ✓ Strong column-weak beam

Equivalent Lateral Force Procedure

$$\frac{Fe}{Vdesign} = R \qquad \frac{\Delta design}{\Delta y} = C_d$$



Approximate Period of Vibration

$$T_a = C_t h_n^x$$

from *Standard* Table 12.8-2

$C_t = 0.028$ and $x = 0.8$

$T_a = 0.028(77.5)^{0.8} = 0.91$ sec/cycle.

$C_u T_a = 1.4(0.91) = 1.27$ sec/cycle.

T_{comp}=2.05 sec (without P-Delta)

T_{comp}=2.13 sec (with P-Delta)

Equivalent Lateral Force Procedure

Vertical Distribution of Forces

$$F_x = C_{vx}V \quad \text{and} \quad C_{vx} = \frac{w_x h^k}{\sum_{i=1}^{n} w_i h_i^k}$$

Equivalent Lateral Forces for Building Responding in N-S Direction

Level x	w_x (kips)	h_x (ft)	$w_x h_x^k$	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
R	2,596	77.5	1,080,327	0.321	243.6	243.6	3,045
6	2,608	65.0	850,539	0.253	191.8	435.4	8,488
5	2,608	52.5	632,564	0.188	142.6	578.0	15,713
4	2,608	40.0	433,888	0.129	97.8	675.9	24,161
3	2,608	27.5	258,095	0.077	58.2	734.1	33,337
2	2,621	15.0	111,909	0.033	25.2	759.3	44,727
Σ	15,650		3,367,323	1.000	759.3		

Computer Programs NONLIN-Pro and DRAIN 2Dx

Shortcomings of DRAIN

- It is not possible to model strength loss when using the ASCE 41-06 (2006) model for girder plastic hinges.
- The DRAIN model for axial-flexural interaction in columns is not particularly accurate.
- Only Two-Dimensional analysis may be performed.

Elements used in Analysis

- Type 1, inelastic bar (truss) element
- Type 2, beam-column element
- Type 4, connection element

Description of Preliminary Model

- Only a single frame (Frame A or G) is modeled.
- Columns are fixed at their base.
- Each beam or column element is modeled using a Type 2 element. For the columns, axial, flexural, and shear deformations are included. For the girders, flexural and shear deformations are included but, because of diaphragm slaving, axial deformation is not included. Composite action in the floor slab is ignored for all analysis.
- All members are modeled using centerline dimensions without rigid end offsets.
- This model does not provide any increase in beam-column joint stiffness due to the presence of doubler plates.
- The stiffness of the girders was decreased by 7% in the preliminary analyses, which should be a reasonable approximate representation of the 35% reduction in the flange sections. Moment rotation properties of the reduced flange sections are used in the detailed analyses.

Results of Preliminary Analysis : Drift

Results of Preliminary Analysis Excluding P-delta Effects

Story	Total Drift (in.)	Story Drift (in.)	Magnified Story Drift (in.)	Drift Limit (in.)	Story Stability Ratio, θ
6	2.08	0.22	1.21	3.00	0.0278
5	1.86	0.32	1.76	3.00	0.0453
4	1.54	0.38	2.09	3.00	0.0608
3	1.16	0.41	2.26	3.00	0.0749
2	0.75	0.41	2.26	3.00	0.0862
1	0.34	0.34	1.87	3.60	0.0691

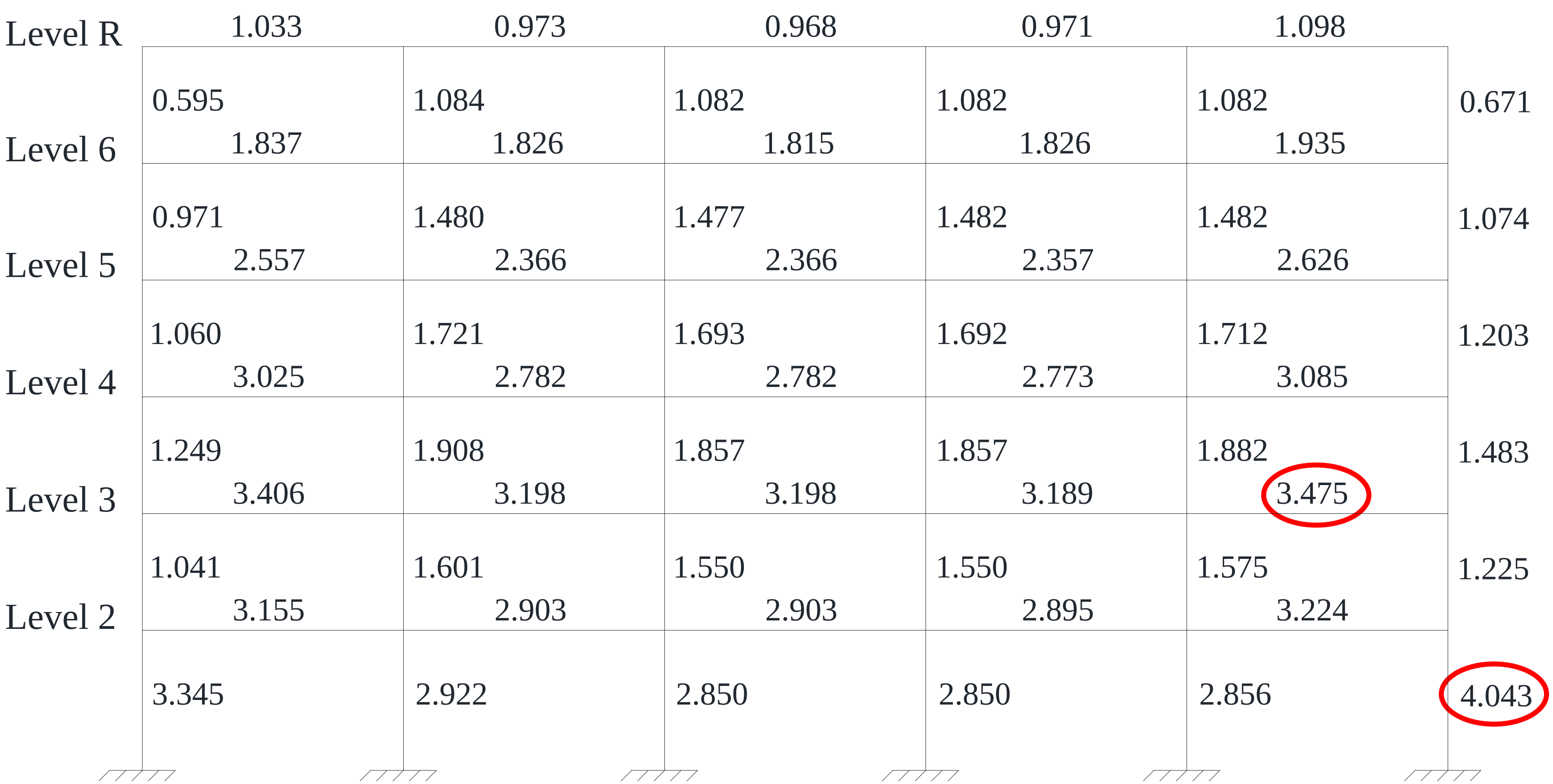
× 5.5 =

Results of Preliminary Analysis Including P-delta Effects

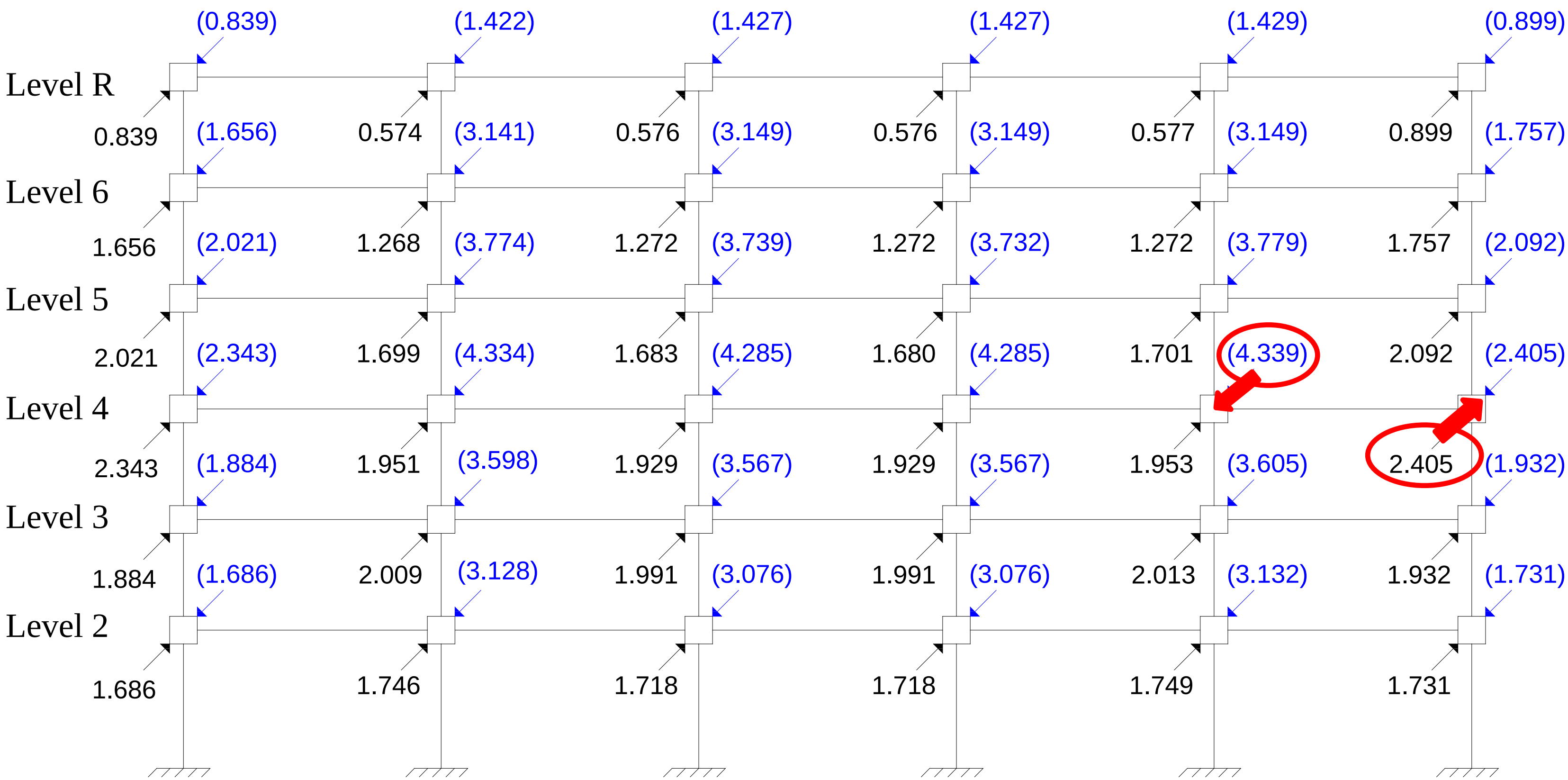
Story	Total Drift (in.)	Story Drift (in.)	Magnified Story Drift (in.)	Drift from θ (in.)	Drift Limit (in.)
6	2.23	0.23	1.27	1.24	3.00
5	2.00	0.34	1.87	1.84	3.00
4	1.66	0.40	2.20	2.23	3.00
3	1.26	0.45	2.48	2.44	3.00
2	0.81	0.45	2.48	2.47	3.00
1	0.36	0.36	1.98	2.01	3.60

× 5.5 =

Results of Preliminary Analysis : Demand Capacity Ratios (Columns-Girders)



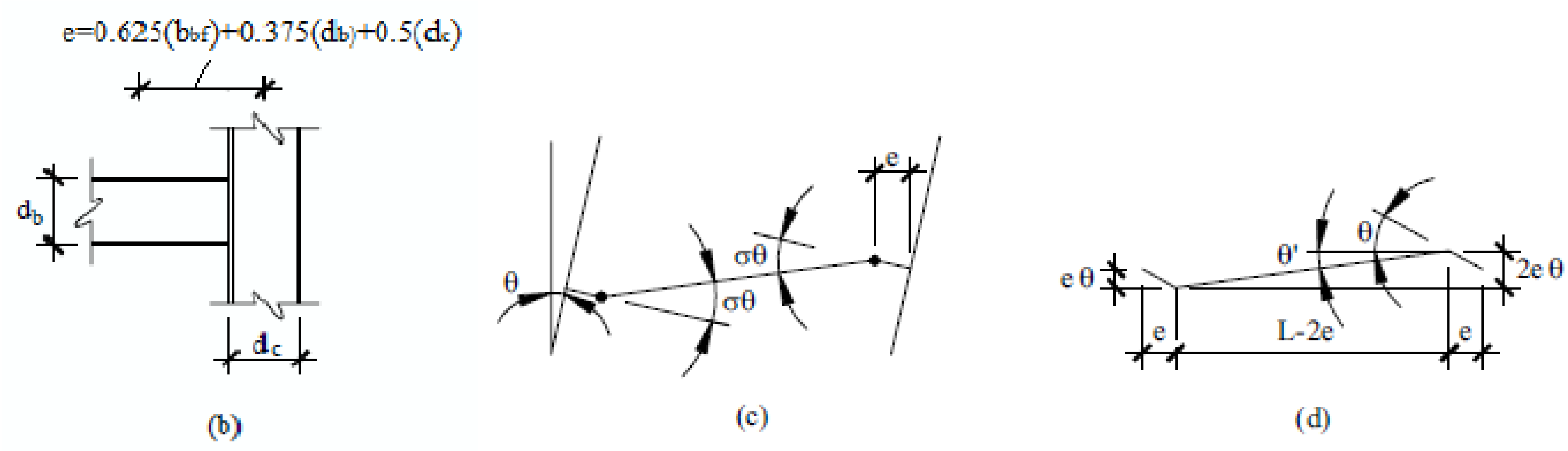
Results of Preliminary Analysis : Demand Capacity Ratios (Panel Zones)



Results of Preliminary Analysis : Demand Capacity Ratios

- The structure has considerable overstrength, particularly at the upper levels.
- The sequence of yielding will progress from the lower level girders to the upper level girders.
- With the possible exception of the first level, the girders should yield before the columns. While not shown in the Figure, it should be noted that the demand-to-capacity ratios for the lower story columns were controlled by the moment at the base of the column. The column on the leeward (right) side of the building will yield first because of the additional axial compressive force arising from the seismic effects.
- The maximum DCR of girders is 3.475, while maximum DCR for panel zones without doubler plates is 4.339. Thus, if doubler plates are not used, the first yield in the structure will be in the panel zones. However, with doubler plates added, the first yield is at the girders as the maximum DCR of the panel zones reduces to 2.405.

Results of Preliminary Analysis: Overall System Strength



Internal Work = External Work

$$\text{Internal Work} = 2[20\sigma\theta M_{PA} + 40\sigma\theta M_{PB} + \theta(M_{PC} + 4M_{PD} + M_{PE})]$$

$$\text{External Work} = V\theta \sum_{i=1}^{nLevels} F_i H_i \quad \text{where} \quad \sum_{i=1}^{nLevels} F_i = 1$$

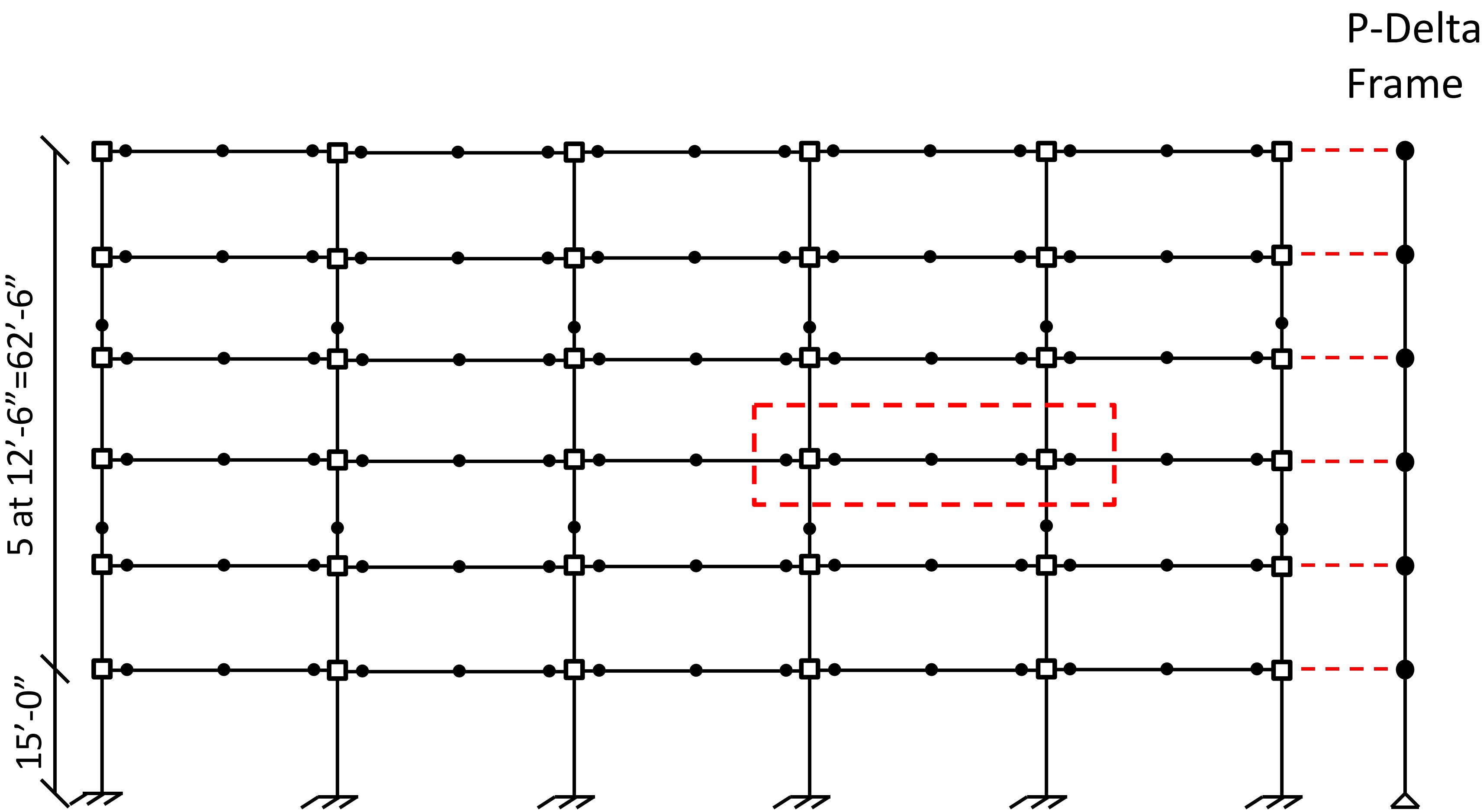
Results of Preliminary Analysis: Overall System Strength

Lateral Strength on Basis of Rigid-Plastic Mechanism

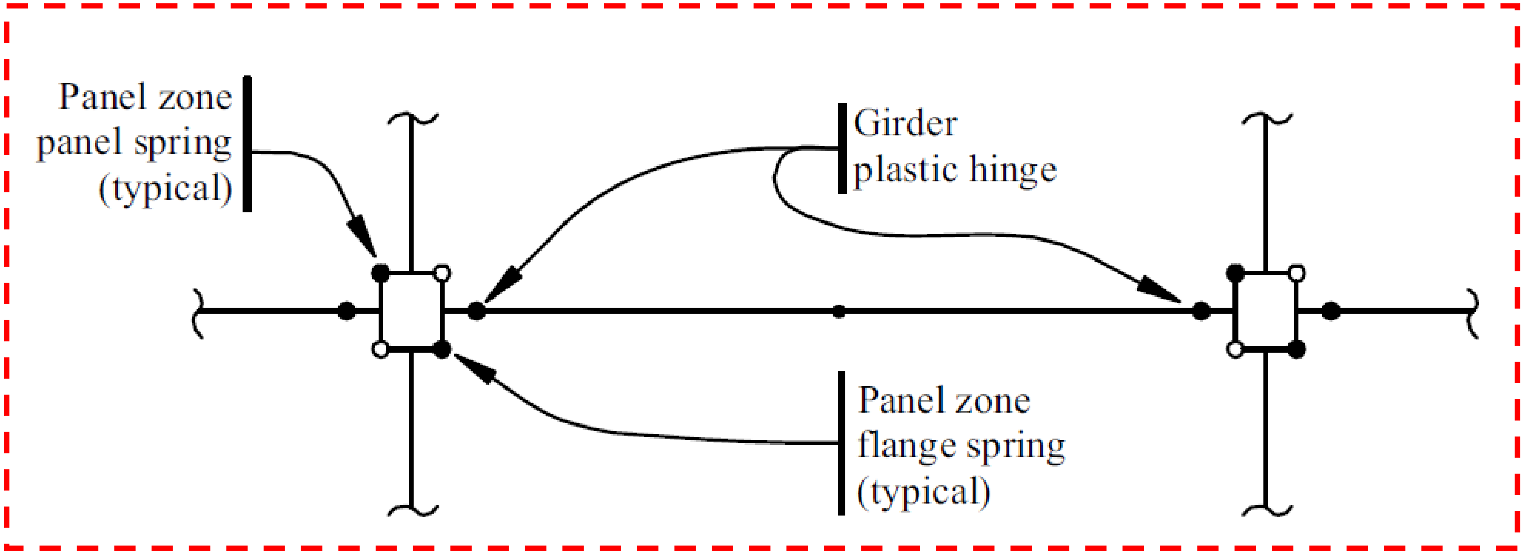
Lateral Load Pattern	Lateral Strength (kips) Entire Structure	Lateral Strength (kips) Single Frame
Uniform	3,332	1,666
Upper Triangular	2,747	1,373
Standard	2,616	1,308

- As expected, the strength under uniform load is significantly greater than under triangular or *Standards* load.
- The closeness of the *Standards* and triangular load strengths is due to the fact that the vertical-load-distributing parameter (k) was 1.385, which is close to 1.0.
- Slightly more than 15 percent of the system strength comes from plastic hinges that form in the columns. If the strength of the column is taken simply as M_p (without the influence of axial force), the "error" in total strength is less than 2 percent.
- The rigid-plastic analysis did not include strain hardening, which is an additional source of overstrength.

Description of Model Used for Detailed Structural Analysis

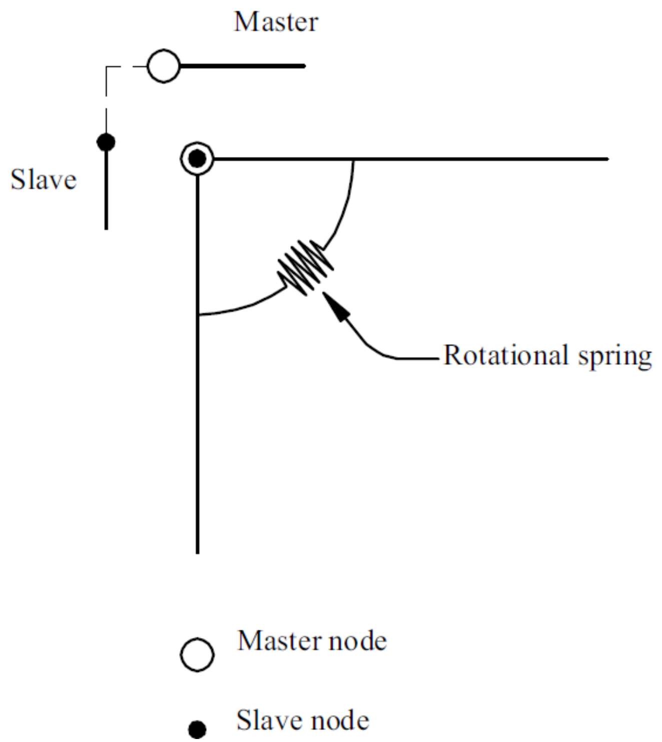


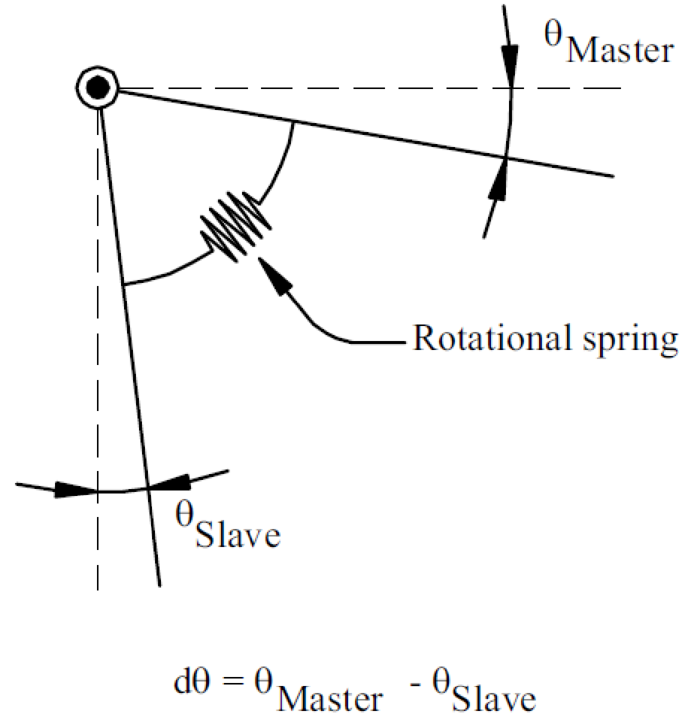
Description of Model Used for Detailed Structural Analysis



- Nonlinear static and nonlinear dynamic analyses require a much more detailed model than was used in the linear analysis.
- The primary reason for the difference is the need to explicitly represent yielding in the girders, columns, and panel zone region of the beam-column joints.

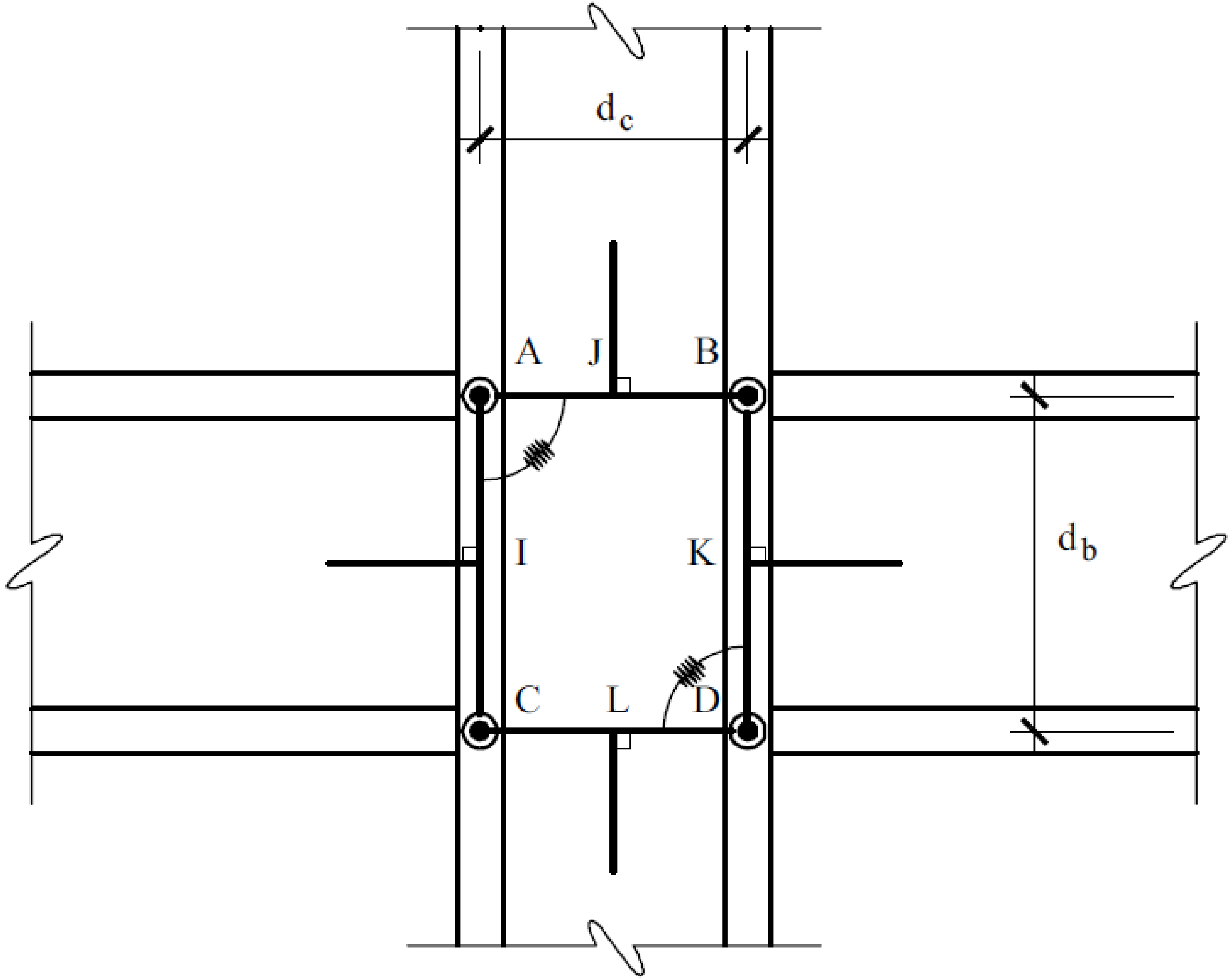
Plastic Hinge Modeling and Compound Nodes





- Compound nodes are used to model plastic hinges in girders and deformations in the panel zone region of beam-column joints
- Typically consist of a pair of single nodes with each node sharing the same point in space. The X and Y degrees of freedom of the first node of the pair (the slave node) are constrained to be equal to the X and Y degrees of freedom of the second node of the pair (the master node), respectively. Hence, the compound node has four degrees of freedom: an X displacement, a Y displacement, and two independent rotations.

Modeling of Beam-Column Joint Regions



Krawinkler beam-column joint model

Modeling of Beam-Column Joint Regions

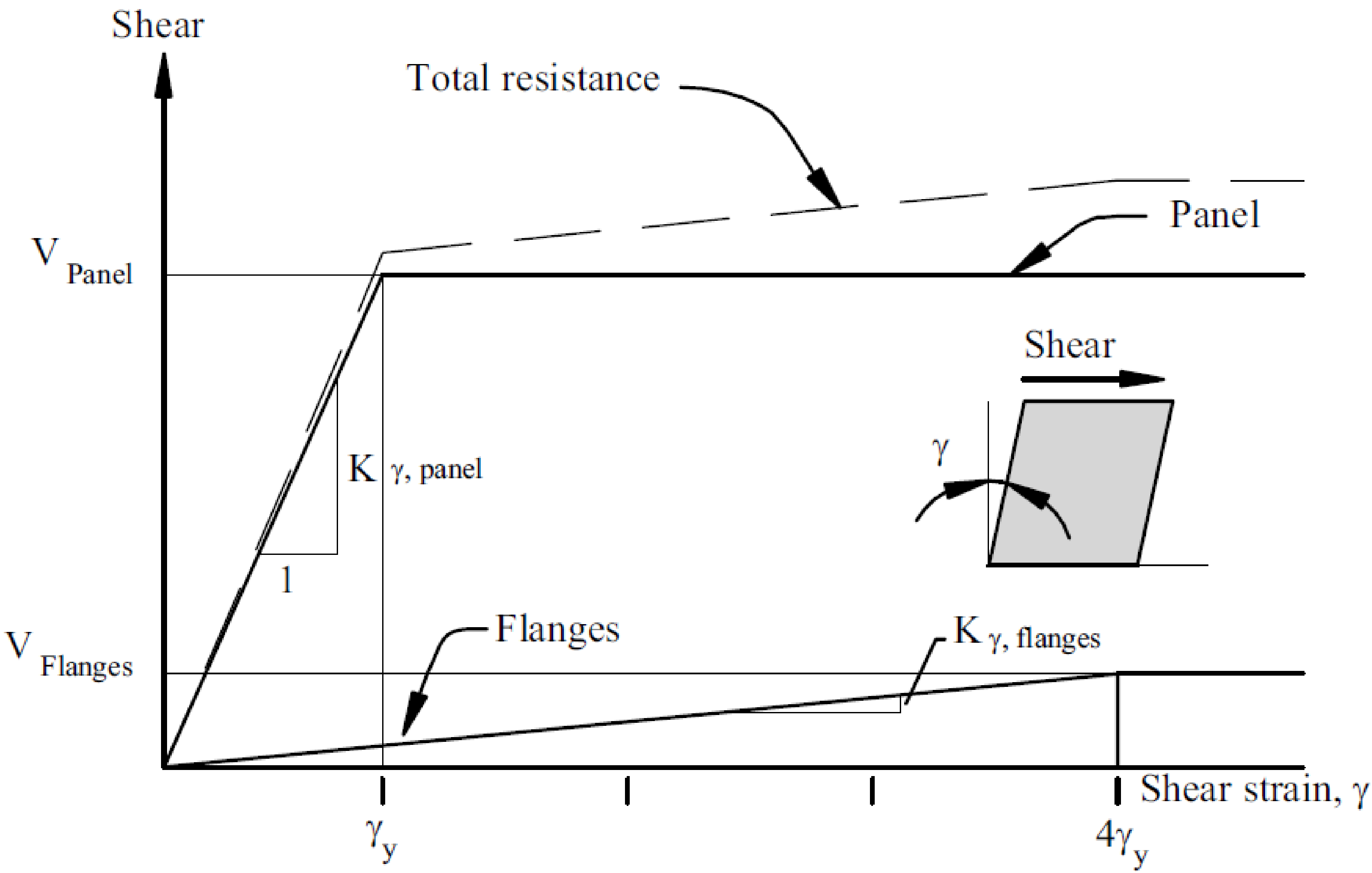
Krawinkler model assumes that the panel zone has two resistance mechanisms acting in parallel:

1. Shear resistance of the web of the column, including doubler plates and
2. Flexural resistance of the flanges of the column.

$$R_v = 0.6F_y d_c t_p + 1.8\frac{F_y b_{cf} t_{cf}^2}{d_b} = V_{Panel} + 1.8V_{Flanges}$$

- F_y = yield strength of the column and the doubler plate,
- d_c = total depth of column,
- t_p = thickness of panel zone region = column web + doubler plate thickness,
- b_{cf} = width of column flange,
- t_{cf} = thickness of column flange, and
- d_b = total depth of girder.

Modeling of Beam-Column Joint Regions

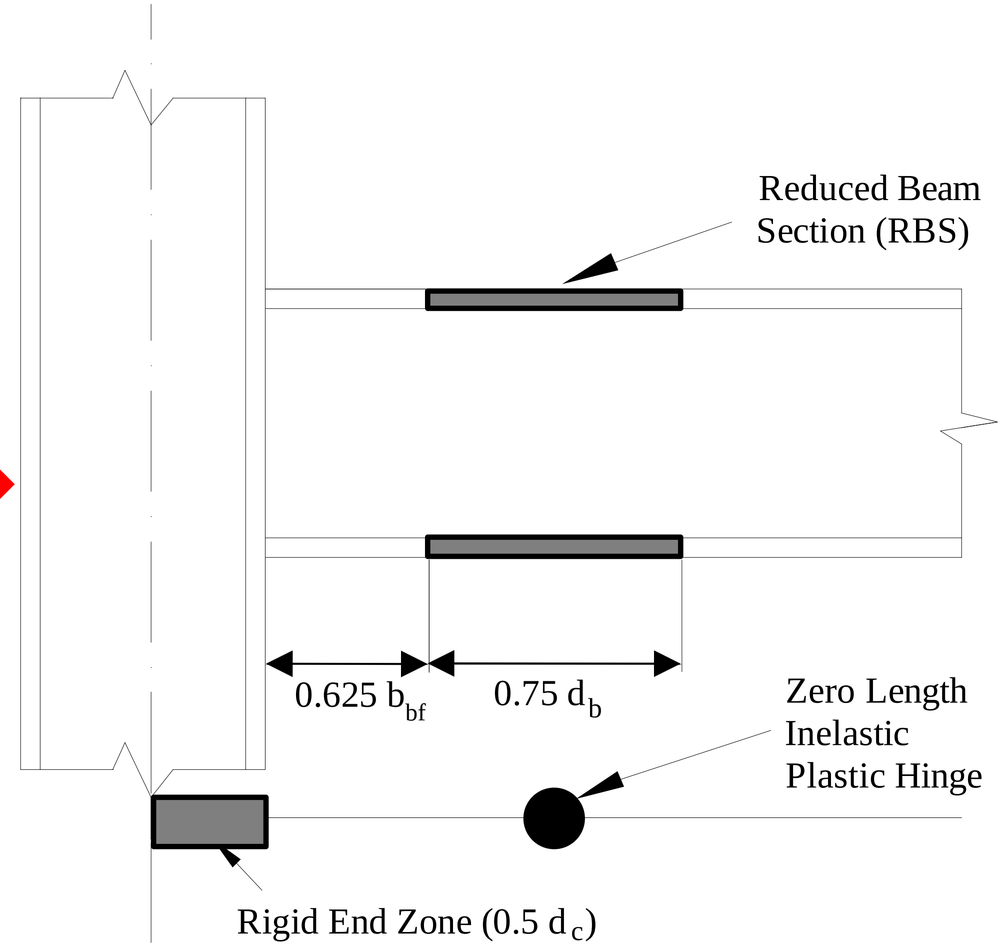


Force-deformation behavior of panel zone region (Krawinkler Model)

Modeling Girders

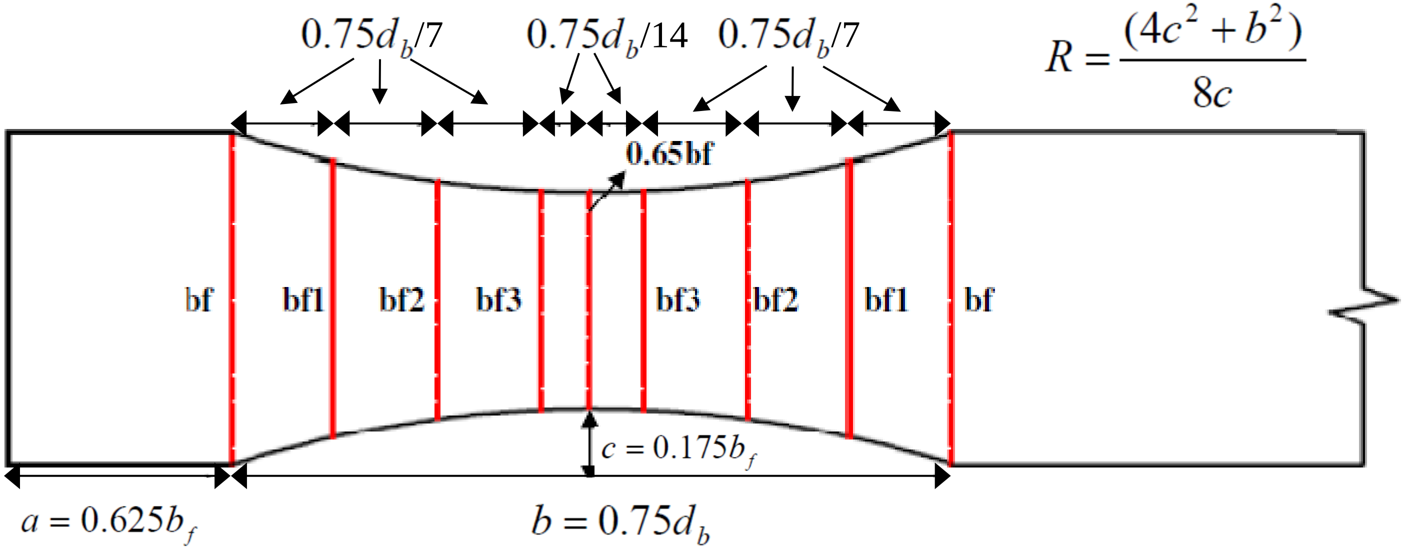
- The AISC Seismic Design Manual (AISC, 2006) recommends design practices to force the plastic hinge forming in the beam away from the column.

1. Reduce the cross sectional properties of the beam at a specific location away from the column
2. Special detailing of the beam-column connection to provide adequate strength and toughness in the connection so that inelasticity will be forced into the beam adjacent to the column face.



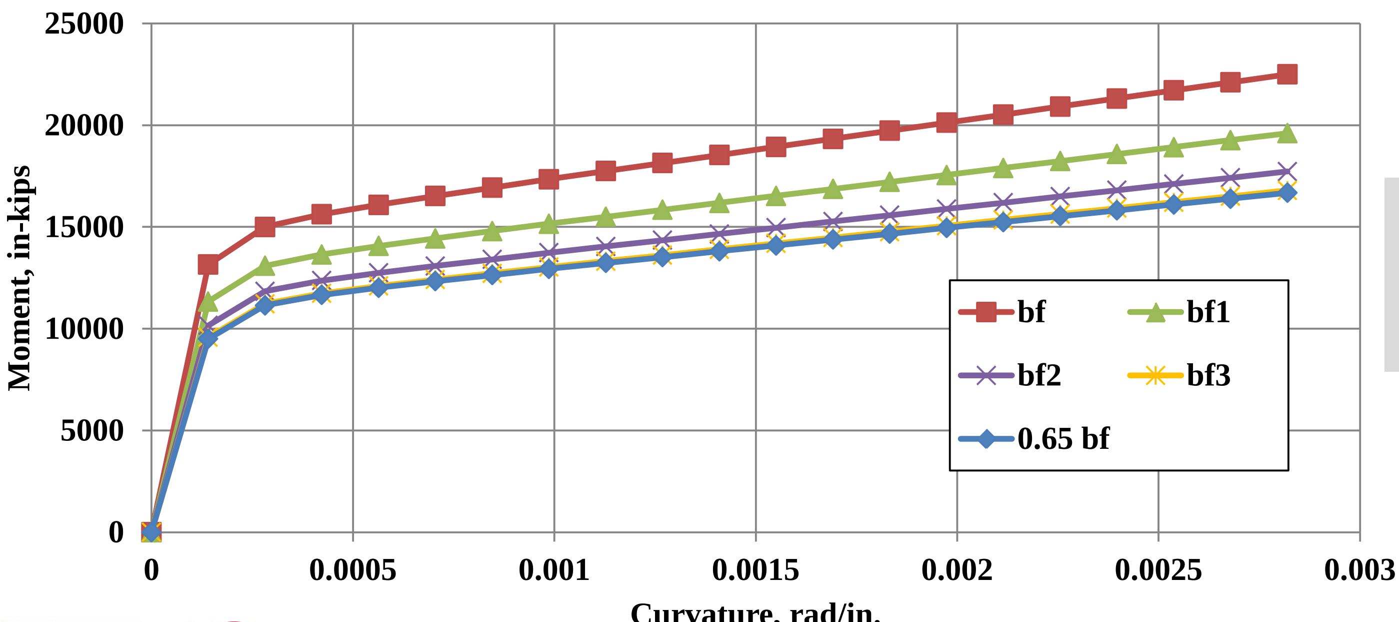
Side view of beam element and beam modeling

Modeling Girders



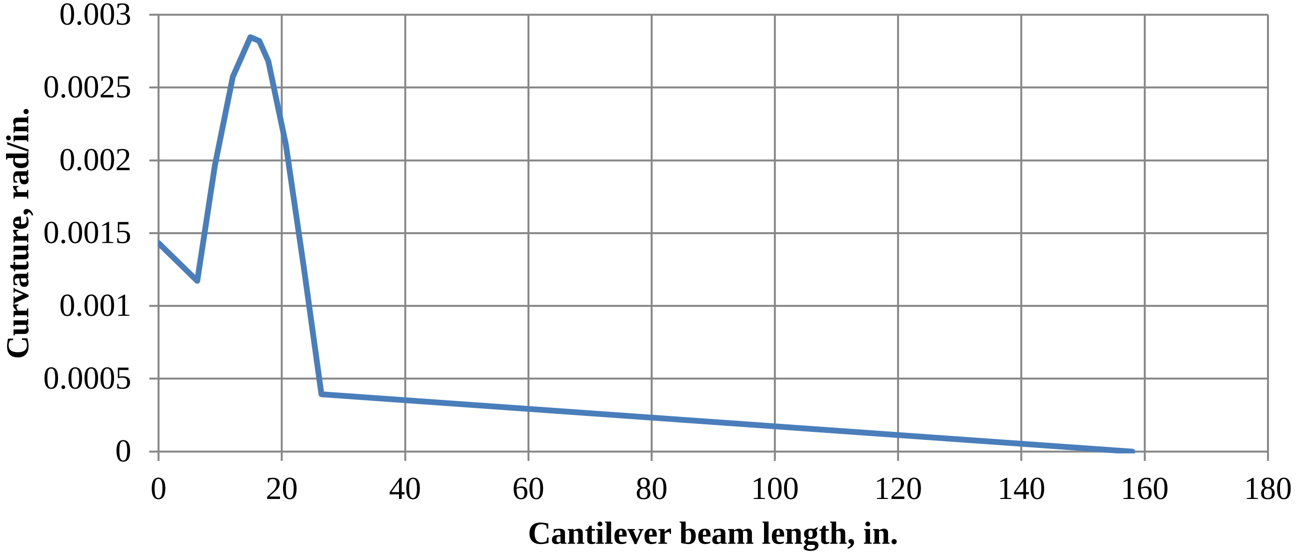
$$R = \frac{(4c^2 + b^2)}{8c}$$

Top view of Reduced Beam Section

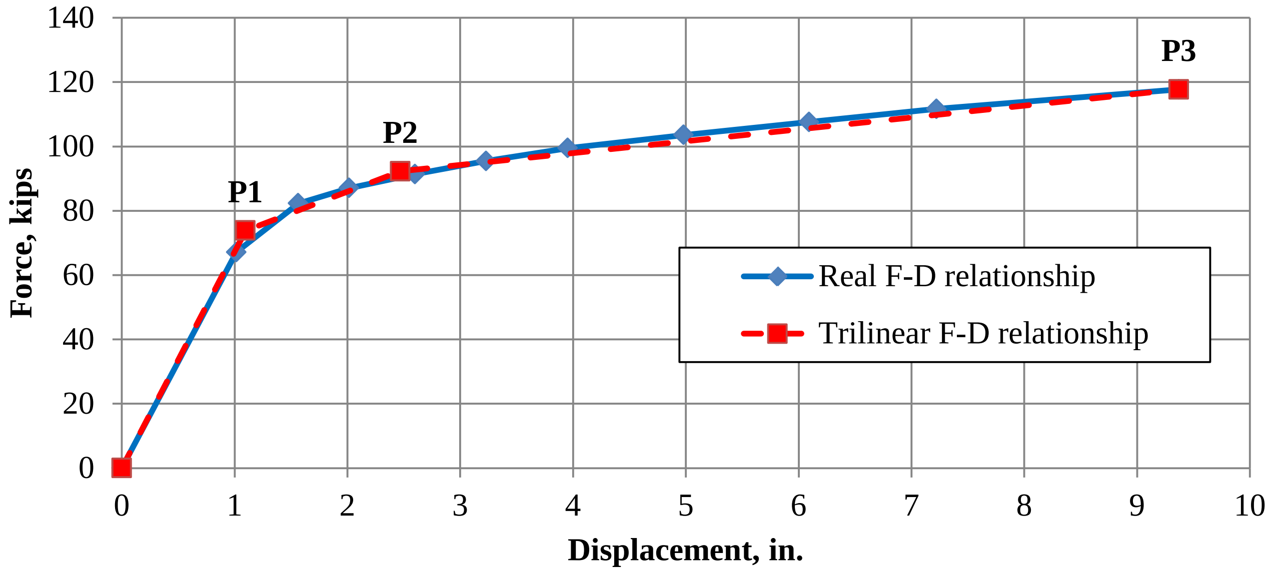


Moment curvature diagram for W27x94 girder

Modeling Girders

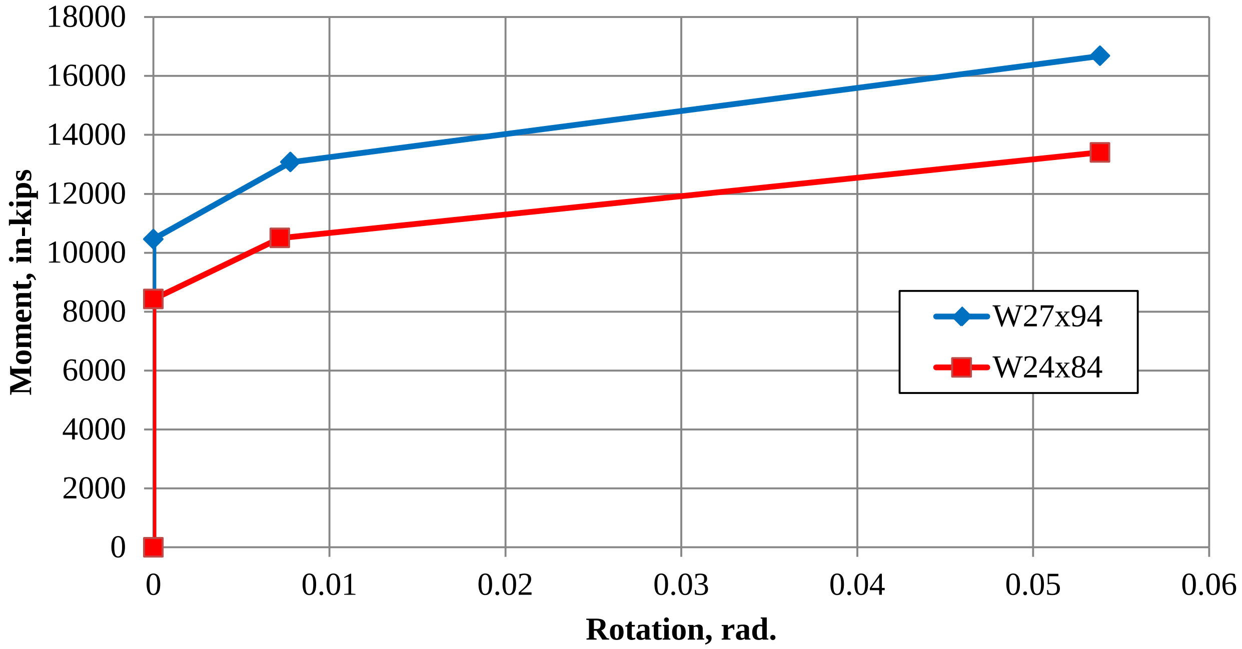


Curvature Diagram for Cantilever Beam with Reduced Beam Section



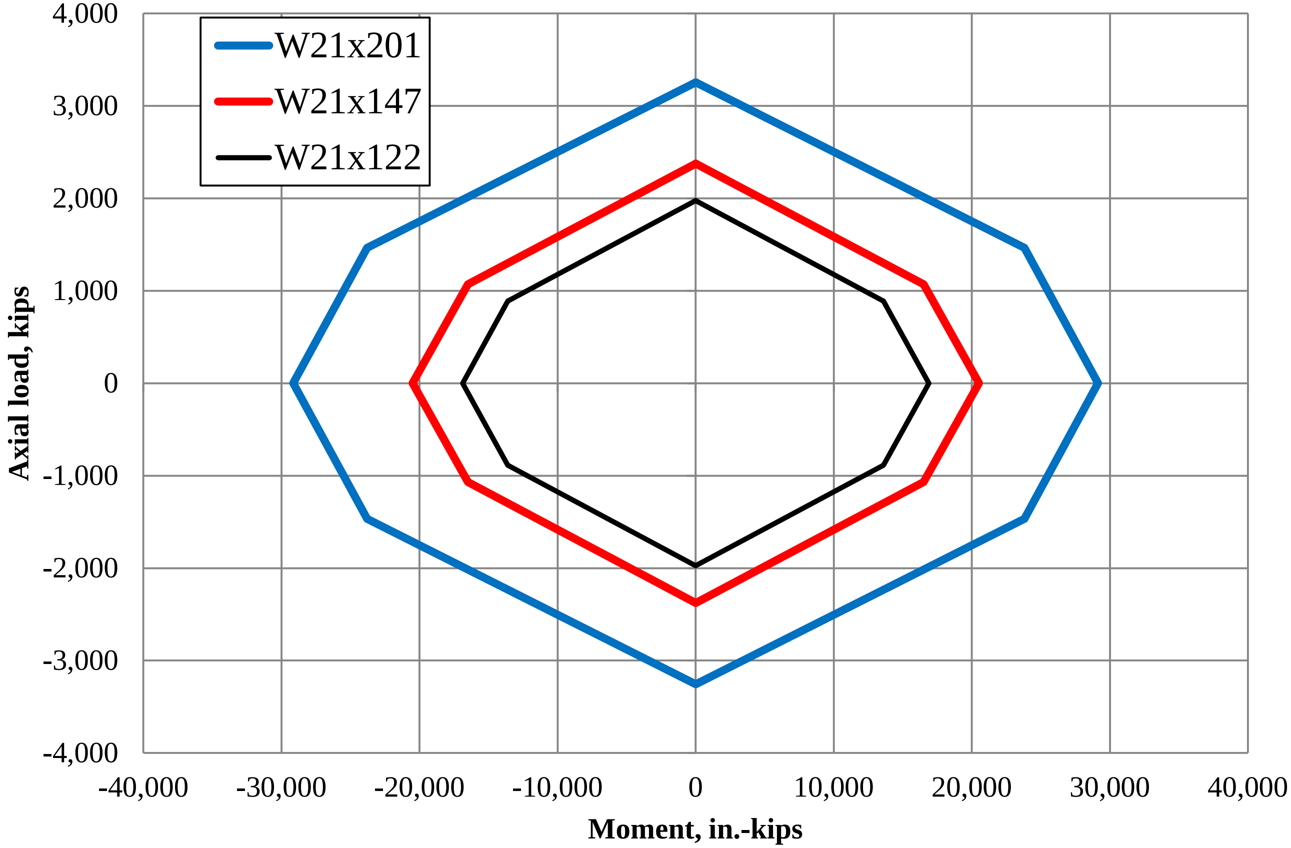
Force Displacement Diagram for W27x94 with RBS

Modeling Girders



Moment-Rotation Diagram for girder hinges with RBS

Modeling Columns



Yield surface used for modeling columns

Results of Detailed Analysis: Period of Vibration

Periods of Vibration From Detailed Analysis (sec/cycle)

Model	Mode	P-delta Excluded	P-delta Included
Strong Panel with doubler plates	1	1.912	1.973
	2	0.627	0.639
	3	0.334	0.339
Weak Panel without doubler plates	1	2.000	2.069
	2	0.654	0.668
	3	0.344	0.349

- P-delta effects increases the period.
- Doubler plates decreases the period as the model becomes stiffer with doubler plates.
- Different period values were obtained from preliminary and detailed analyses.
- Detailed model results in a stiffer structure than the preliminary model especially when doubler plates are added.

Static Pushover Analysis

- Pushover analysis procedure performed in this example follows the recommendations of ASCE/SEI 41-06.
- Pushover analysis should *always* be used as a precursor to nonlinear response history analysis.
- The structure is subjected to the full dead load plus 50 percent of the fully reduced live load, followed by the lateral loads.
- For the entire pushover analyses reported for this example, the structure is pushed to 37.5 in. at the roof level. This value is about two times the total drift limit for the structure where the total drift limit is taken as 2 percent of the total height.
- The effect of lateral load distribution, strong and weak panel zones (doubler plates) and P-delta are investigated separately in this example.

Static Pushover Analysis

Effect of Different Lateral Load Distribution

In this example, three different load patterns were initially considered:

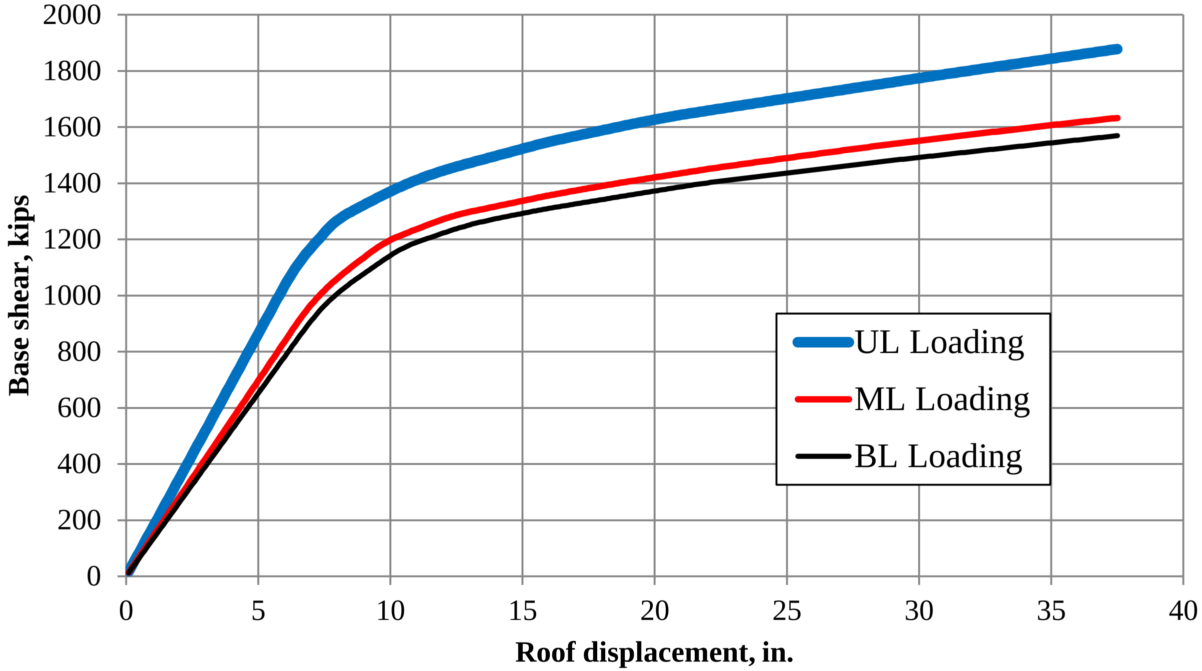
UL = Uniform load (equal force at each level)
ML = Modal load (lateral loads proportional to first mode shape)
BL = *Provisions* load distribution (Equivalent lateral forces used for preliminary analysis)

Lateral Load Patterns Used in Nonlinear Static Pushover Analysis

Level	Uniform Load UL (kips)	Modal Load ML (kips)	*Provisions* Load BL (kips)
R	15.0	85.1	144.8
6	15.0	77.3	114.0
5	15.0	64.8	84.8
4	15.0	49.5	58.2
3	15.0	32.2	34.6
2	15.0	15.0	15.0

Static Pushover Analysis

Effect of Different Lateral Load Distribution



Response of strong panel model to three load patterns, excluding P-delta effects

The *Provisions* states that the lateral load pattern should follow the shape of the first mode. (ML Loading)

Static Pushover Analysis

Static Pushover Curves with P-Delta Effects

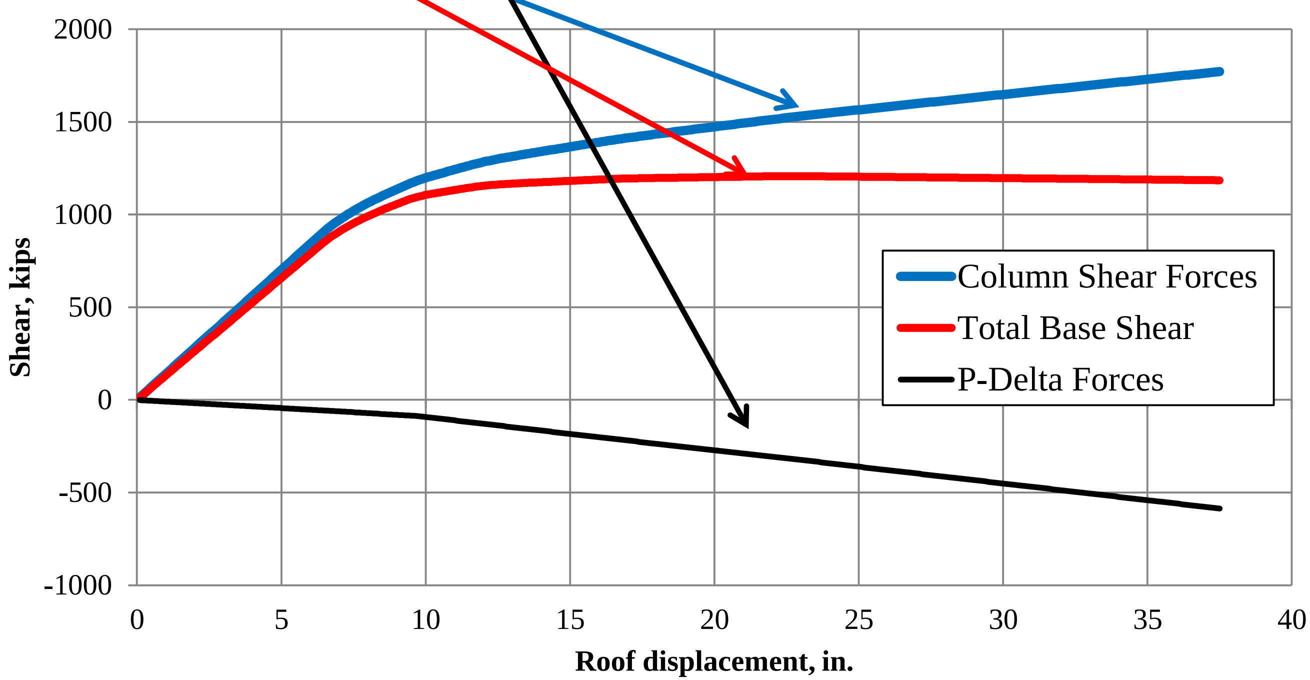
$$\boxed{V} = \boxed{\sum_{i=1}^{n} V_{C,i}} - \boxed{\frac{P_1 \Delta_1}{h_1}}$$

$\sum_{i=1}^{n} V_{C,i}$ = Sum of all column shears in 1st story

P_1 = Total vertical load on P-delta column

Δ_1 = P-delta column 1st story displacement

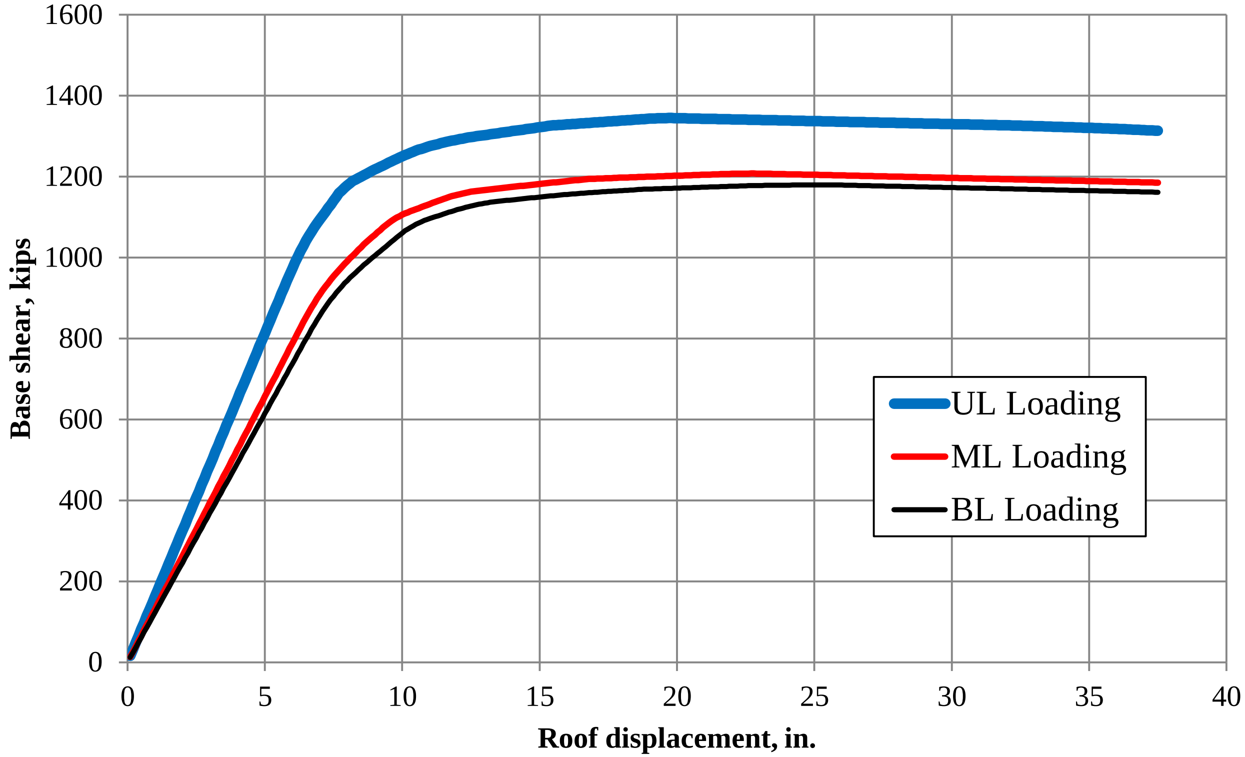
h_1 = 1st story height



Two base shear components of pushover response

Static Pushover Analysis

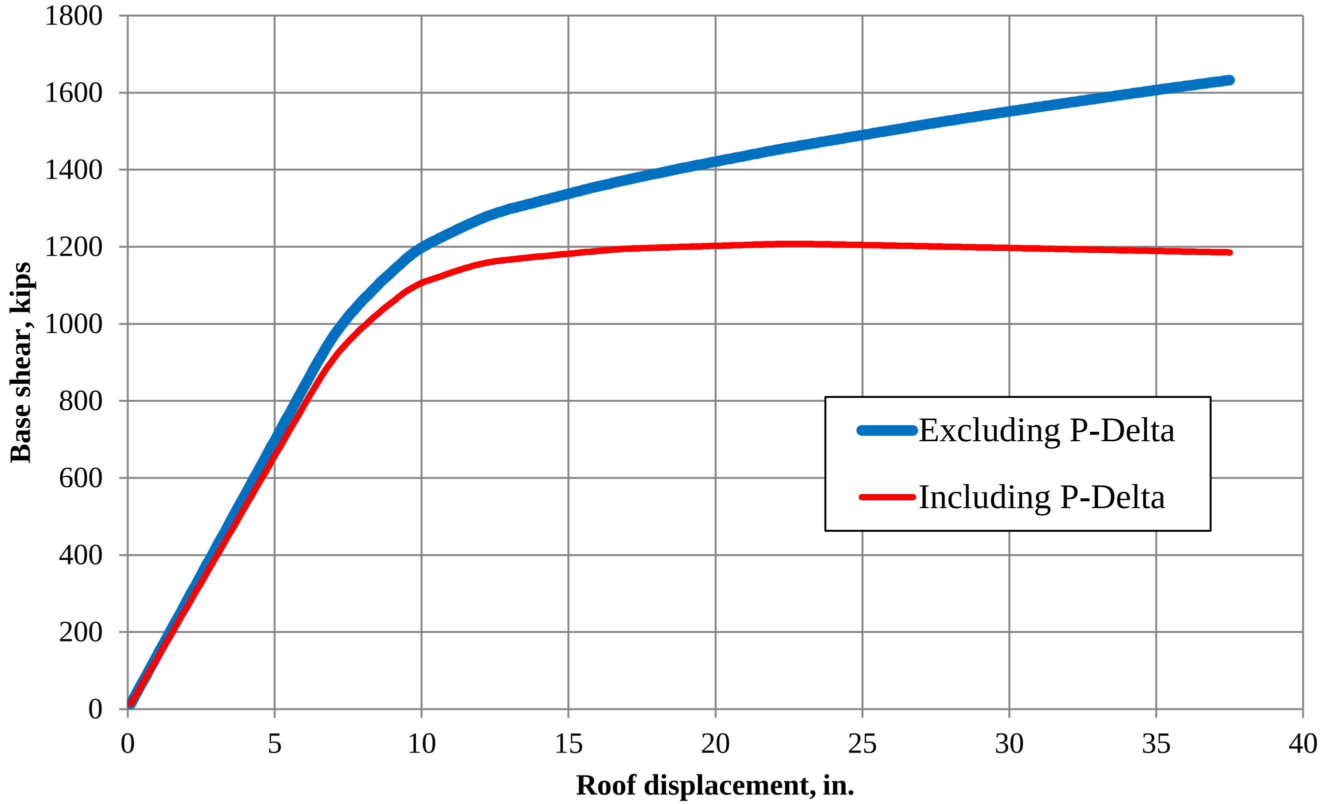
Effect of Different Lateral Load Distribution



Response of strong panel model to three load patterns, including P-delta effects

Static Pushover Analysis

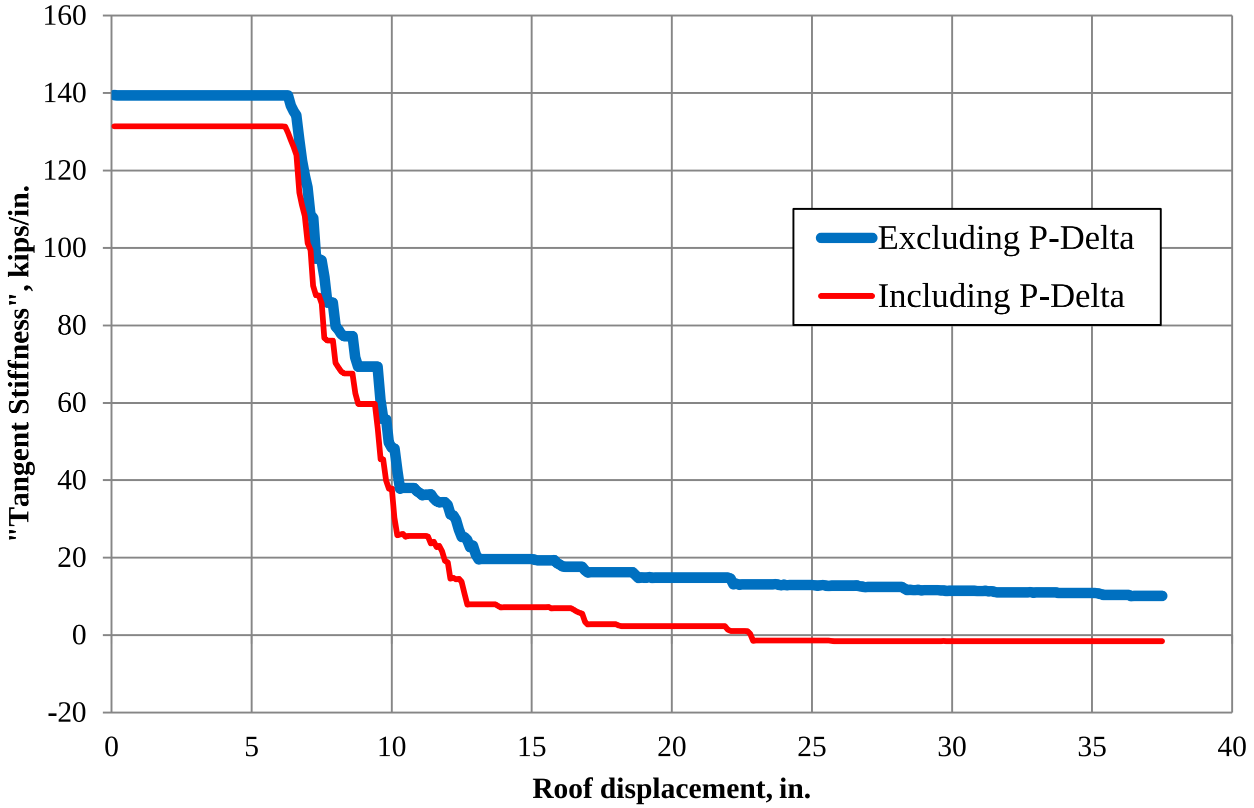
Effect of P-Delta on Pushover Curve



Response of strong panel model to ML loads, with and without P-delta effects

Static Pushover Analysis

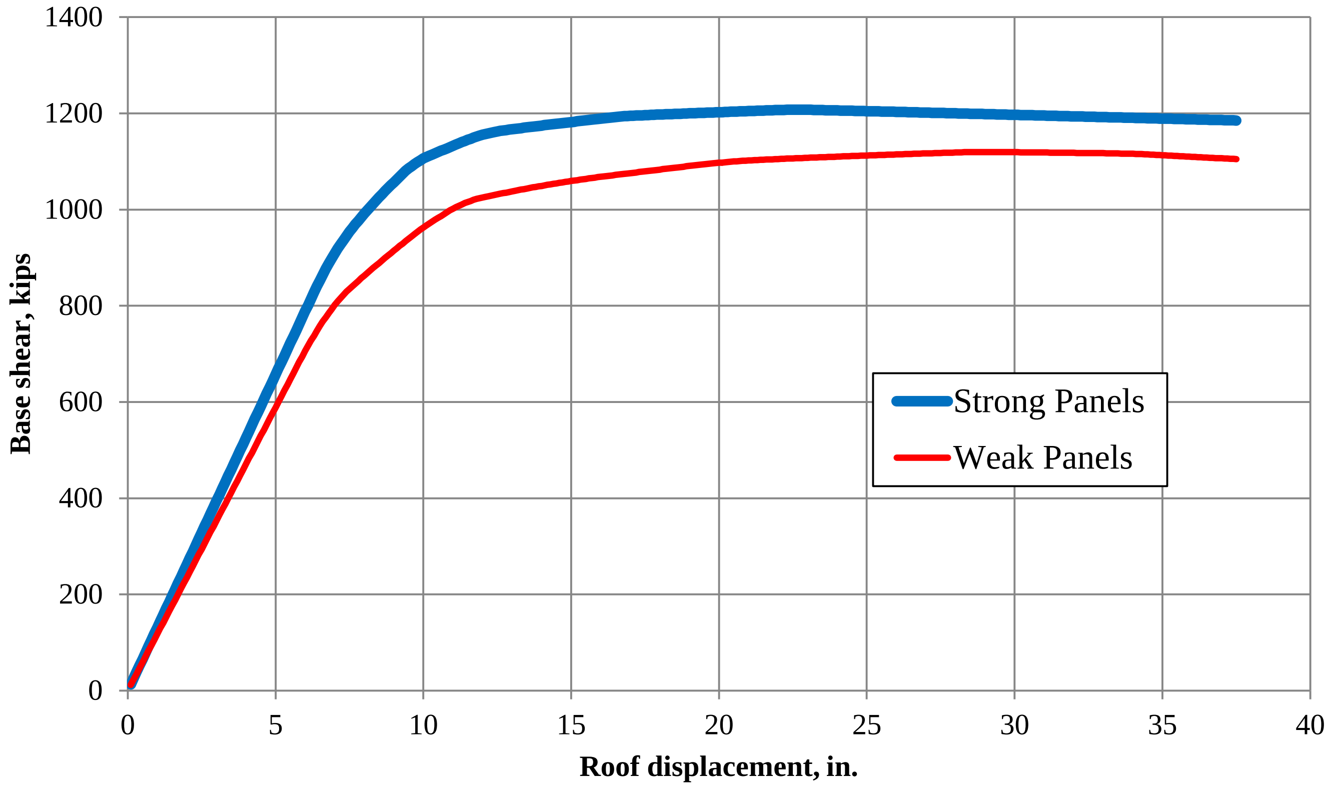
Effect of P-Delta on Pushover Curve



Tangent stiffness history for Strong Panel model under ML loads, with and without P-delta effects

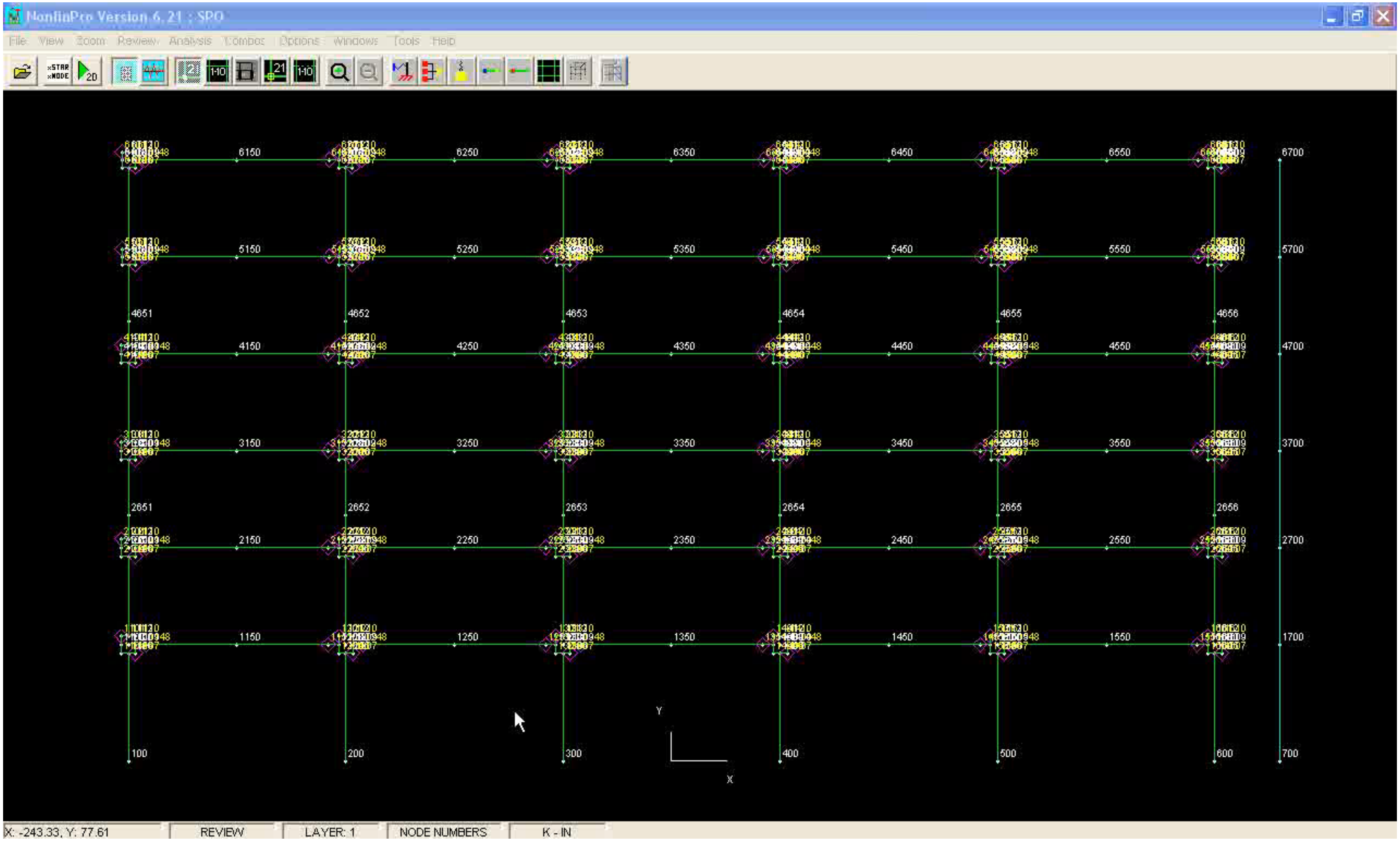
Static Pushover Analysis

Effect of Panel zones (Doubler Plates) on Pushover Curve



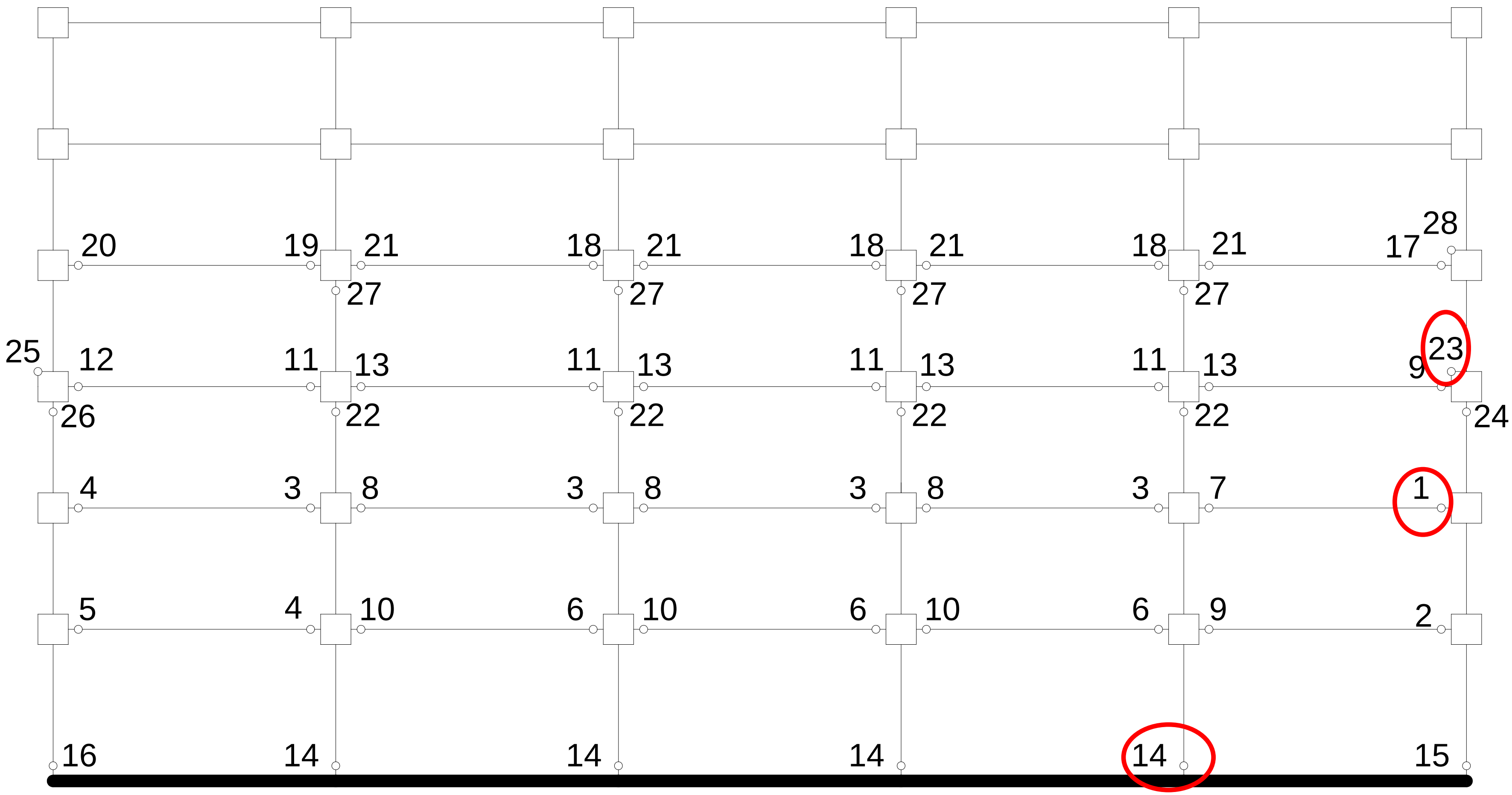
Comparison of weak panel zone model with strong panel zone model, both including P-delta effects

Static Pushover Analysis: Sequence and Pattern of Plastic Hinging with NonlinPro



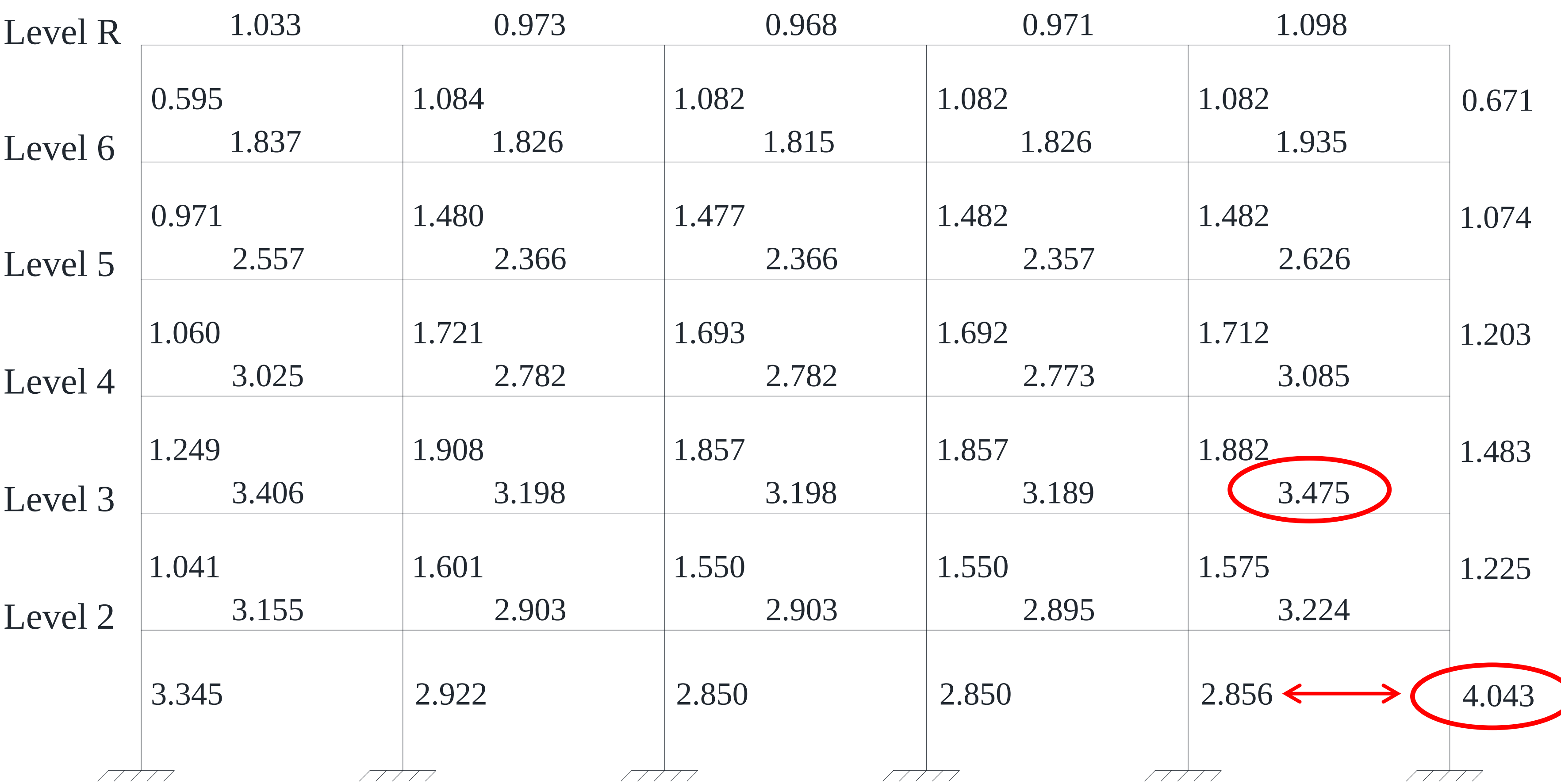
Static Pushover Analysis

Sequence and Pattern of Plastic Hinging for Strong Panel Model



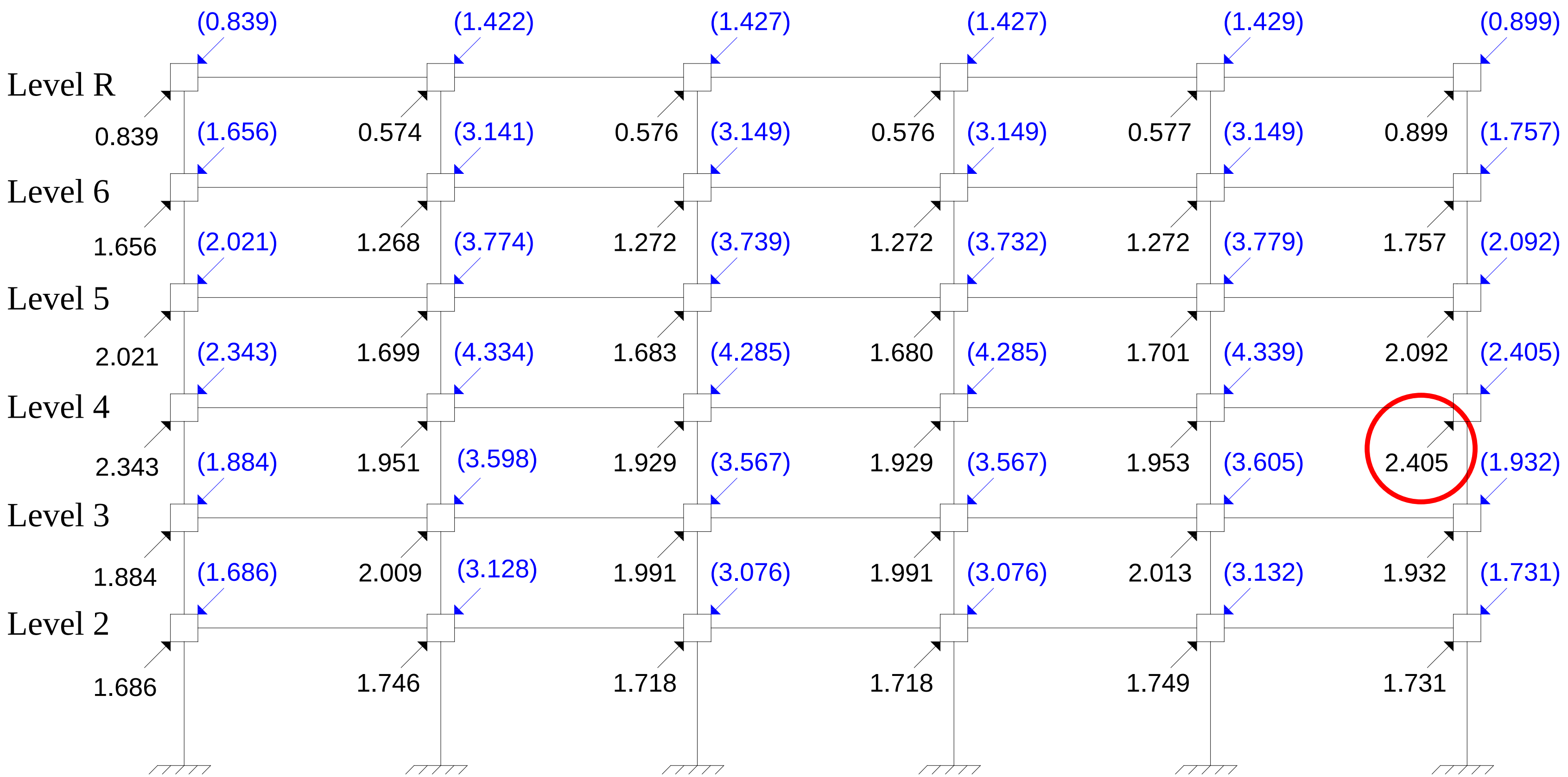
Static Pushover Analysis

DCR – Plastic Hinge Sequence Comparison for Girders and Columns



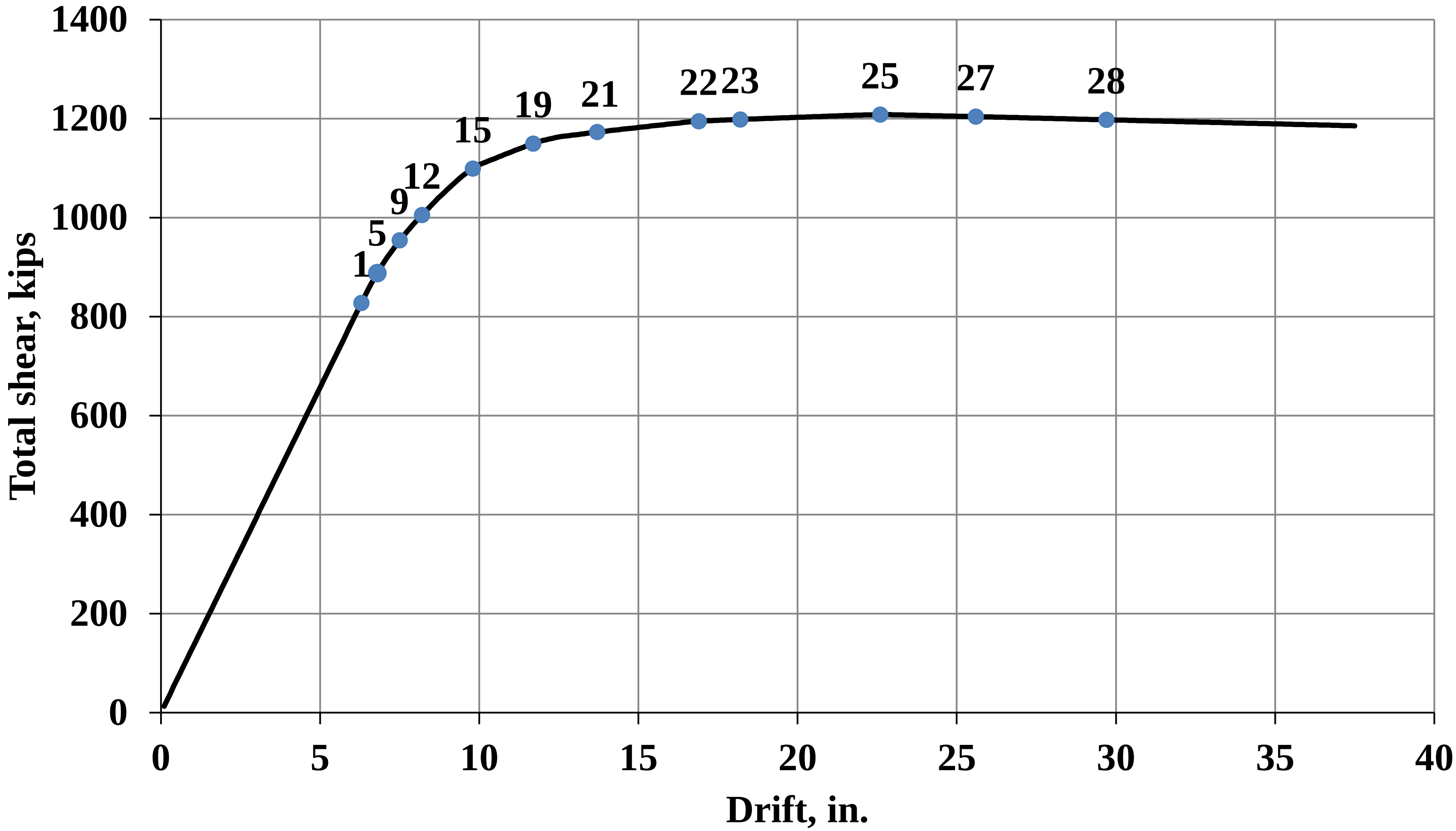
Static Pushover Analysis

DCR – Plastic Hinge Sequence Comparison for Panel Zones



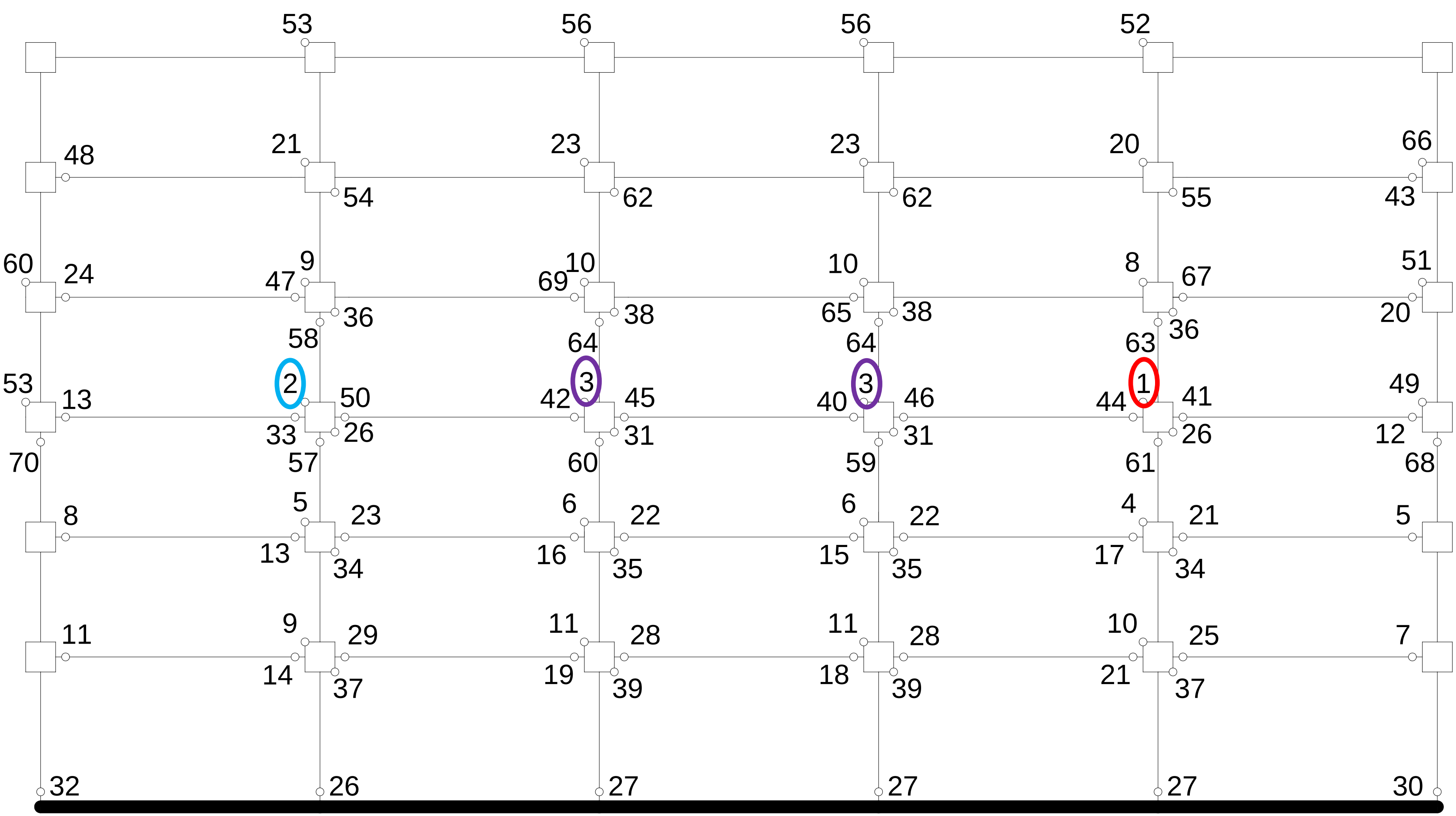
Static Pushover Analysis

Sequence and Pattern of Plastic Hinging for Strong Panel Model



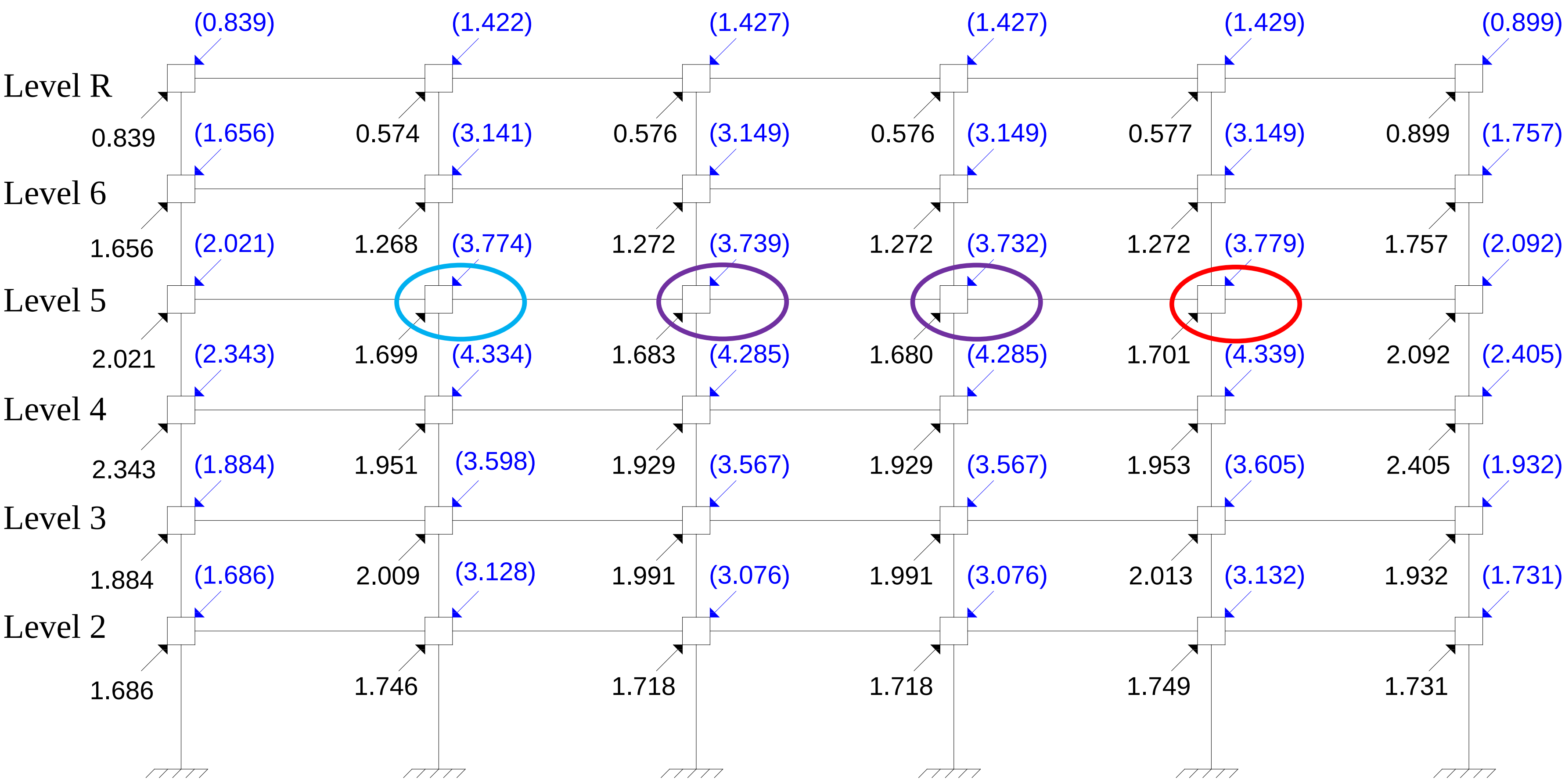
Static Pushover Analysis

Sequence and Pattern of Plastic Hinging for Weak Panel Model



Static Pushover Analysis

DCR – Plastic Hinge Sequence Comparison for Panel Zones



Static Pushover Analysis

Target Displacement

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$$

$C_0 = \phi_{1,r}\Gamma_1 =$ modification factor to relate spectral displacement of an equal single degree of freedom system to the roof displacement of the building multi-degree of freedom system.

$\phi_{1,r} =$ the ordinate of mode shape 1 at the roof (control node)

$\Gamma_1 =$ the first mode participation factor

$C_1 =$ modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

$C_2 =$ modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation and strength deterioration on maximum displacement response.

$S_a =$ response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration.

$T_e = T_i\sqrt{\frac{K_i}{K_e}} =$ effective fundamental period of the building in the direction under consideration

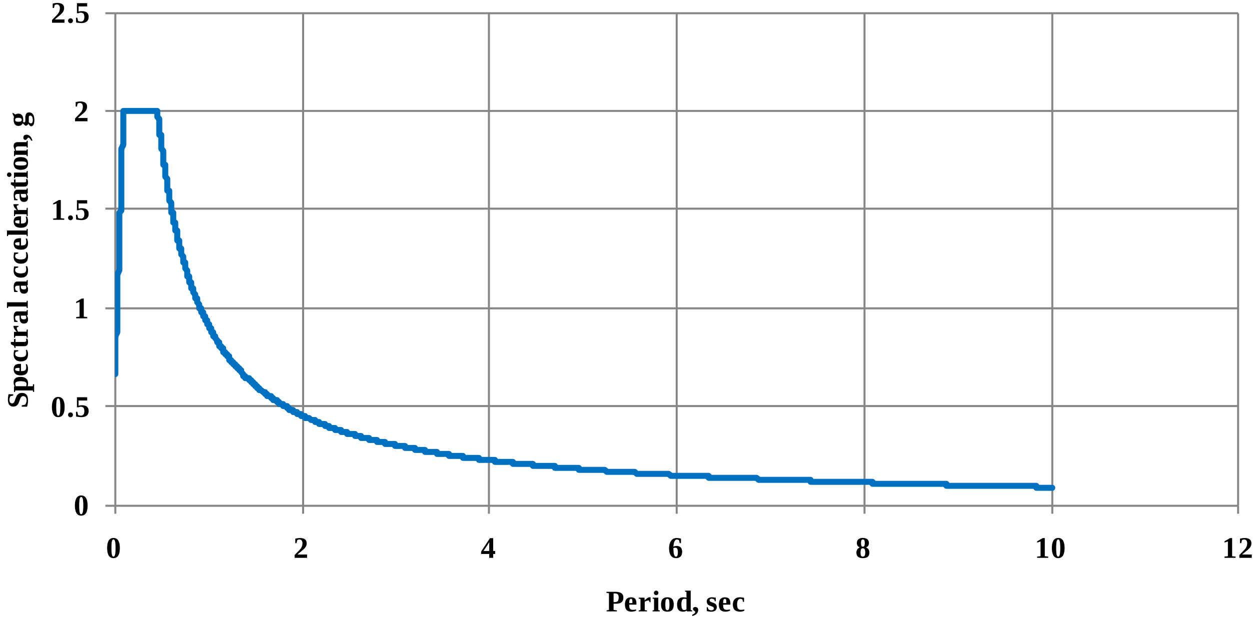
$T_i =$ elastic fundamental period in the direction under consideration calculated by elastic dynamic analysis.

$K_i, K_e =$ elastic, and effective lateral stiffness of the building in the direction under consideration.

$g =$ acceleration of gravity

Static Pushover Analysis

Target Displacement



2% damped horizontal response spectrum from ASCE 41-06

This spectrum is for BSE-2 (Basic Safety Earthquake 2) hazard level which has a 2% probability of exceedence in 50 years.

Static Pushover Analysis

Target Displacement

- Nonlinear force-displacement relationship between base shear and displacement of control node shall be replaced with an idealized force-displacement curve. The effective lateral stiffness and the effective period depend on the idealized force-displacement curve.

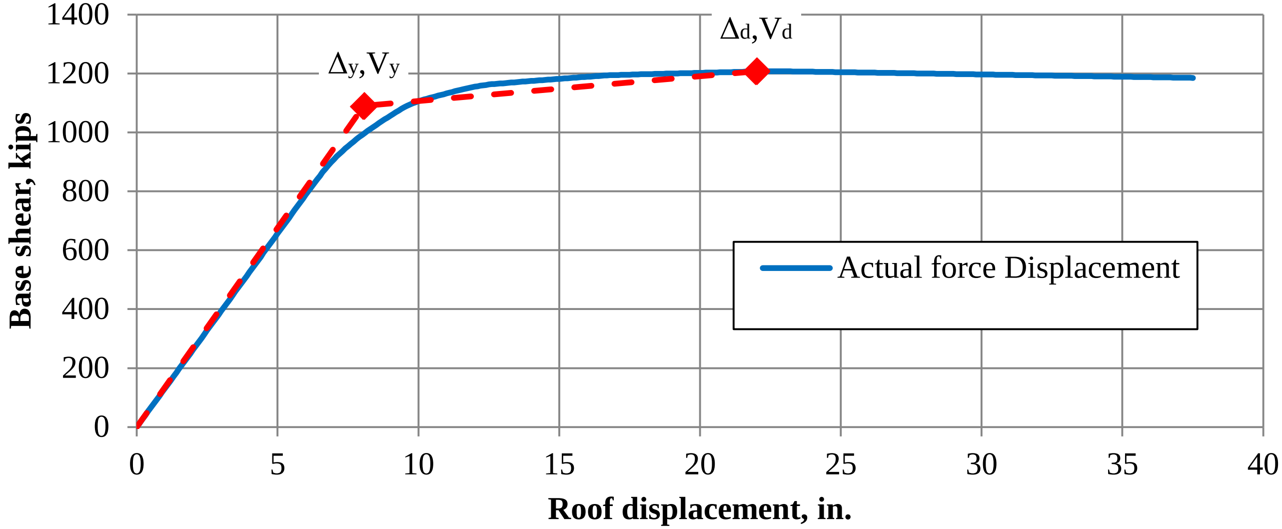
- The idealized force-displacement curve is developed by using an iterative graphical procedure where the areas below the actual and idealized curves are approximately balanced up to a displacement value of Δ_d. Δ_d is the displacement at the end of second line segment of the idealized curve and V_d is the base shear at the same displacement.

- (Δ_d, V_d) should be a point on the actual force displacement curve at either the calculated target displacement, or at the displacement corresponding to the maximum base shear, whichever is the least.

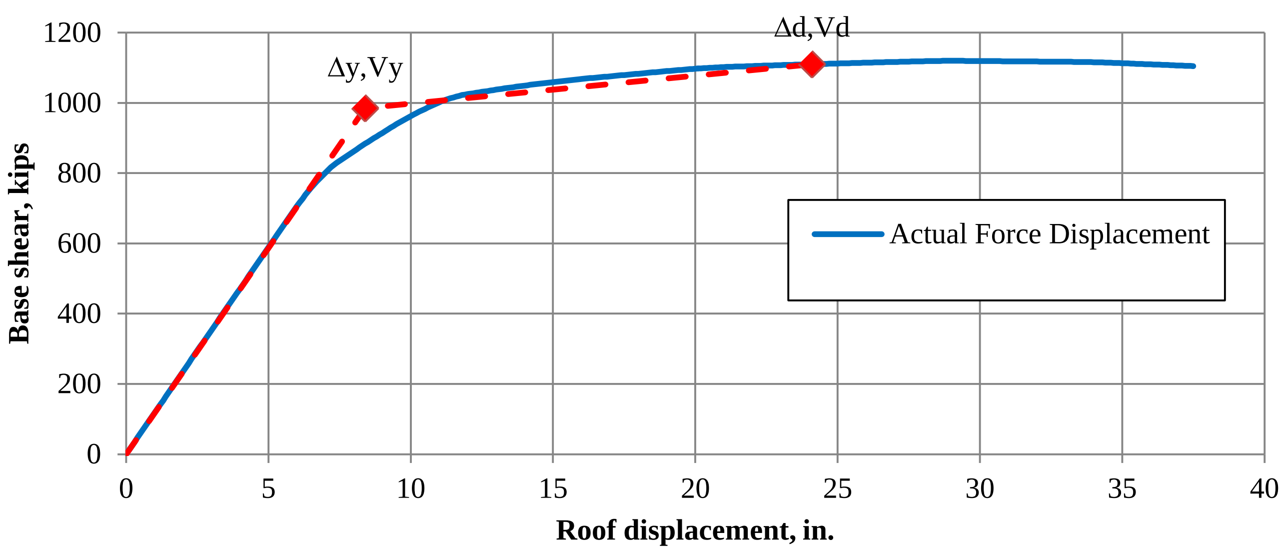
- The first line segment of the idealized force-displacement curve should begin at the origin and finish at (Δ_y, V_y), where V_y is the effective yield strength and Δ_y is the yield displacement of idealized curve.

- The slope of the 1st line segment is equal to the effective lateral stiffness K_e which should be taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength of the structure.

Static Pushover Analysis



Actual and idealized force displacement curves for STRONG panel model, under ML load, with P-delta effects



Actual and idealized force displacement curves for WEAK panel model, under ML load, with P-delta effects

Static Pushover Analysis

Target displacement for strong and weak panel models

	Strong Panel	Weak Panel
C_0	1.303	1.310
C_1	1.000	1.000
C_2	1.000	1.000
S_a (g)	0.461	0.439
T_e (sec)	1.973	2.069
δ_t (in.) at Roof Level	**22.9**	**24.1**
Drift R-6 (in.)	0.96	1.46
Drift 6-5 (in.)	1.76	2.59
Drift 5-4 (in.)	2.87	3.73
Drift 4-3 (in.)	4.84	4.84
Drift 3-2 (in.)	5.74	5.35
Drift 2-1 (in.)	6.73	6.12

- Story drifts are also shown at the load level of target displacement.
- Negative stiffness starts after target displacements for both models.

Response History Analysis

Modeling and Analysis Procedure

- Response response history analysis method is used to estimate the inelastic deformation demands for the detailed structure.
- Three ground motions were used. (Seven or more ground motions is generally preferable.)
- The analysis considered a number of parameters, as follows:
  - Scaling of ground motions to the DBE and MCE level
  - With and without P-delta effects
  - Two percent and five percent inherent damping
  - Added linear viscous damping
- Identical structural model used in Nonlinear Pushover Analyses and 2^{nd} order effects were included through the use of leaning column.
- All of the model analyzed had “Strong Panels” (wherein doubler plated were included in the interior beam-column joints).

Response History Analysis

Rayleigh Damping

- Rayleigh proportional damping was used to represent viscous energy dissipation in the structure.
- The mass and stiffness proportional damping factors were initially set to produce 2.0 percent damping in the first and third modes.
- It is generally recognized that this level of damping (in lieu of the 5 percent damping that is traditionally used in elastic analysis) is appropriate for nonlinear response history analysis.

$$C = \alpha M + \beta K$$

$$\begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = \frac{2\xi}{w_1 + w_3} \begin{Bmatrix} w_1 w_3 \\ 1 \end{Bmatrix}$$

Structural frequencies and damping factors used in response history analysis.
(Damping factors that produce 2 percent damping in modes 1 and 3)

Model/Damping Parameters	ω_1 (rad/sec)	ω_3 (rad/sec)	α	β
Strong Panel with P-delta	3.184	18.55	0.109	0.00184
Strong Panel without P-delta	3.285	18.81	0.112	0.00181

Response History Analysis

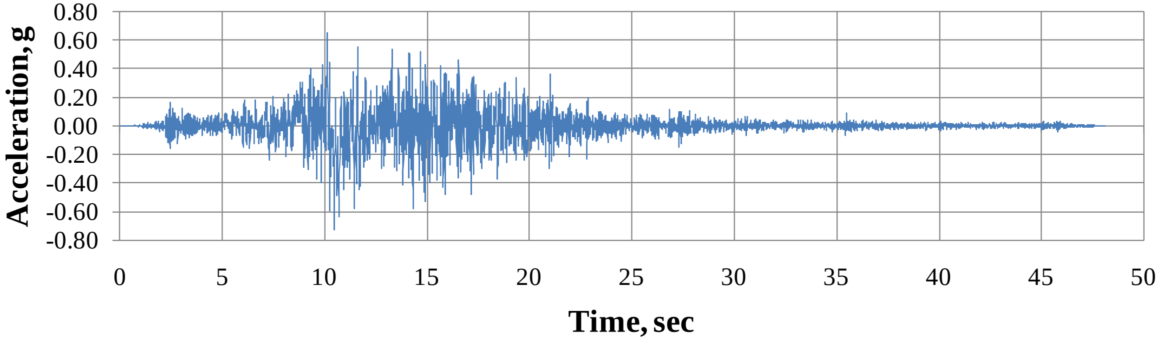
Development of Ground Motion Records

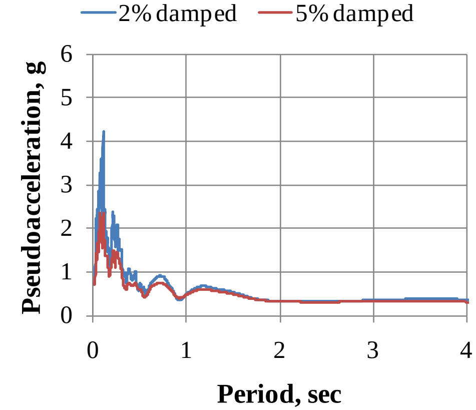
- Because only a two-dimensional analysis of the structure is performed using DRAIN, only a single component of ground motion is applied at one time.
- For the analyses reported herein, the component that produced the larger spectral acceleration at the structure's fundamental period was used.
- A complete analysis would require consideration of both components of ground motions, and possibly of a rotated set of components.

NGA Record Number	Magnitude, [Epicenter Distance (km)]	Site Class	Number of Points and Time step	Integration Time Step used in analyses	Component Source Motion	PGA (g)	Record Name
0879	7.28 , [44]	C	9625 @ 0.005 sec	0.0005 sec	Landers / LCN260	0.727	A00
0725	6.54 , [11.2]	D	2230 @ 0.01 sec	0.001 sec	SUPERST/ B-POE360	0.300	B90
0139	7.35 , [21]	C	1192 @ 0.02 sec	0.001 sec	TABAS/ DAY-TR	0.406	C90

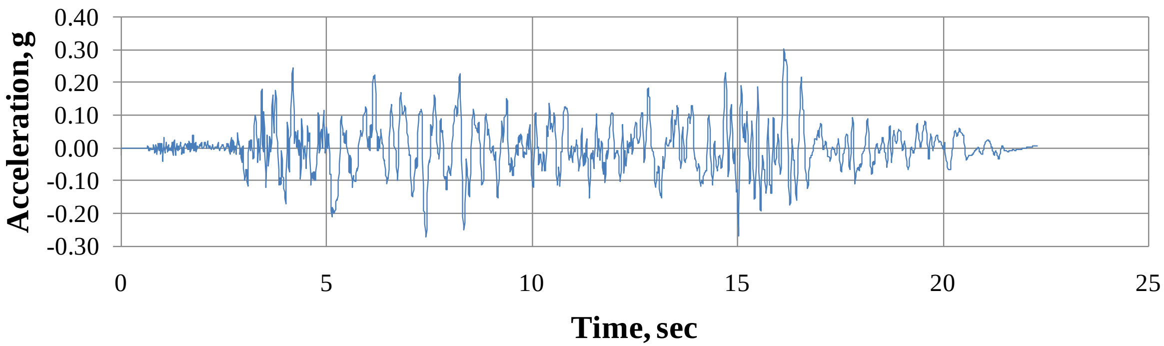
Response History Analysis

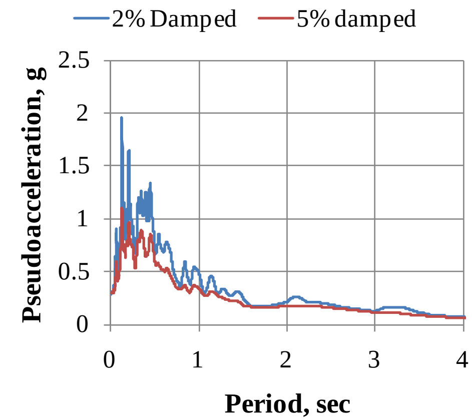
A00



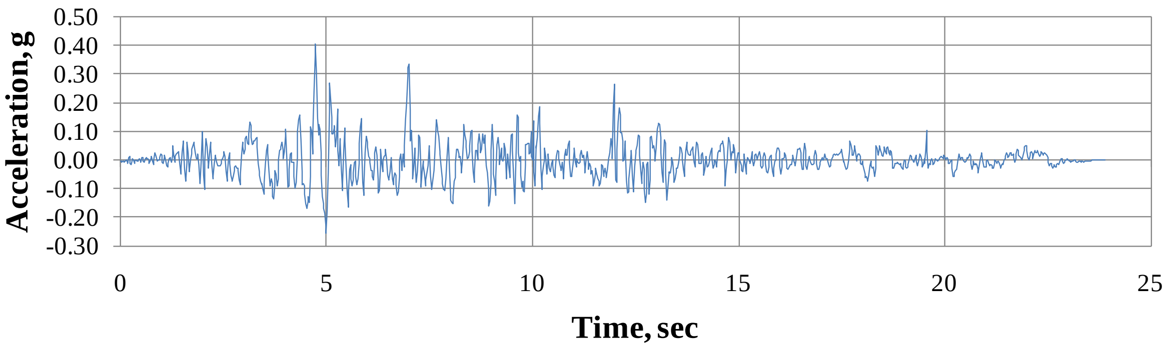


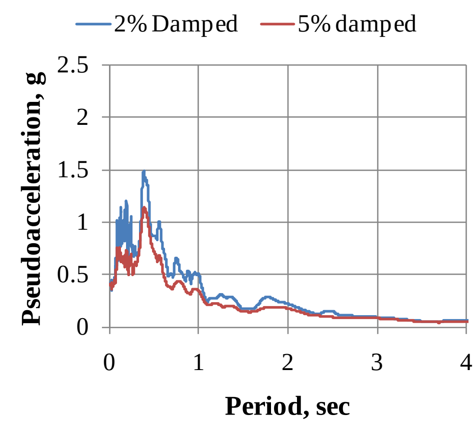
B90





C90



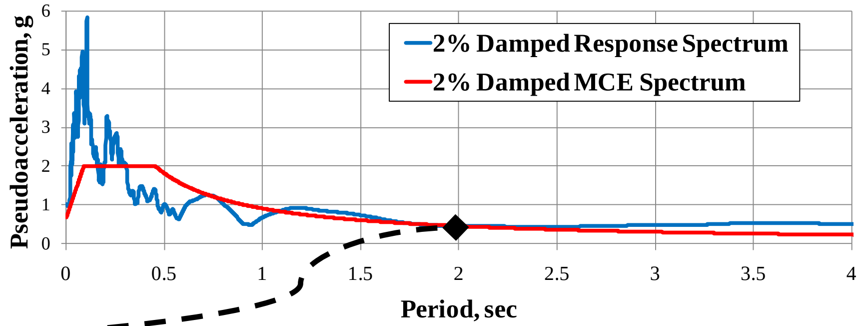


Response History Analysis

Ground Motion Scaling Procedure

1. Each spectrum is initially scaled to match the target spectrum at the structure's fundamental period.

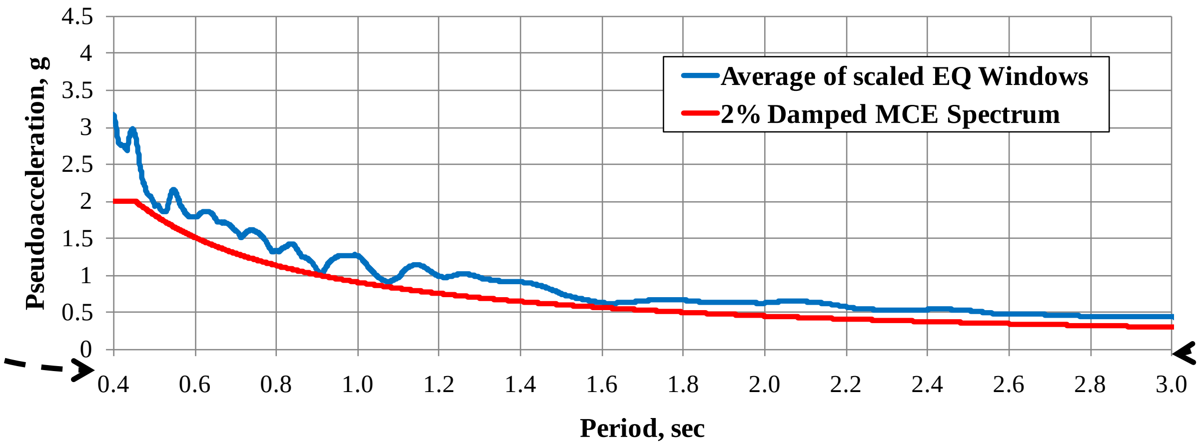
T_1=1.973 sec.



2. The average of the scaled spectra are re-scaled such that no ordinate of the scaled average spectrum falls below the target spectrum in the range of periods between 0.2 and 1.5*T*.

T_1=1.973 sec.

0.2*T_1



1.5*T_1

3. The final scale factor for each motion consists of the product of the initial scale factor (different for each ground motion), and the second scale factor (which is the same for each ground motion).

Results of Response History Analysis

<u>DBE</u> Results for 2% Damped Strong Panel Model with P-Δ Excluded / P-Δ Included

(a) Maximum Base Shear (kips)

	Motion A00	Motion B90	Motion C90
Column Forces	1780 / 1467	1649 / 1458	1543 / 1417
Inertial Forces	1848 / 1558	1650 / 1481	1540 / 1419

(b) Maximum Story Drifts (in.)

Level	Motion A00	Motion B90	Motion C90	Limit*
Total Roof	26.80 / 32.65	14.57 /14.50	13.55 / 14.75	NA
R-6	1.85 / 1.86	1.92 / 1.82	1.71 / 1.70	3.00 (3.75)
6-5	2.51 / 2.64	2.60 / 2.50	2.33 / 2.41	3.00 (3.75)
5-4	**3.75** / **4.08**	3.08 / 2.81	3.03 / 3.19	3.00 (3.75)
4-3	**5.62** / **6.87**	2.98 / 3.21	3.03 / 3.33	3.00 (3.75)
3-2	**6.61** / **8.19**	3.58 / 3.40	2.82 / 2.90	3.00 (3.75)
2-G	**8.09** / **10.40**	**4.68 / 4.69**	3.29 / 3.44	3.60 (4.50)

* Values in ( ) reflect increased drift limits provided by Sec. 16.2.4.3 of the *Standard*

Results of Response History Analysis

MCE Results for 2% Damped Strong Panel Model with P-Δ Excluded / P-Δ Included

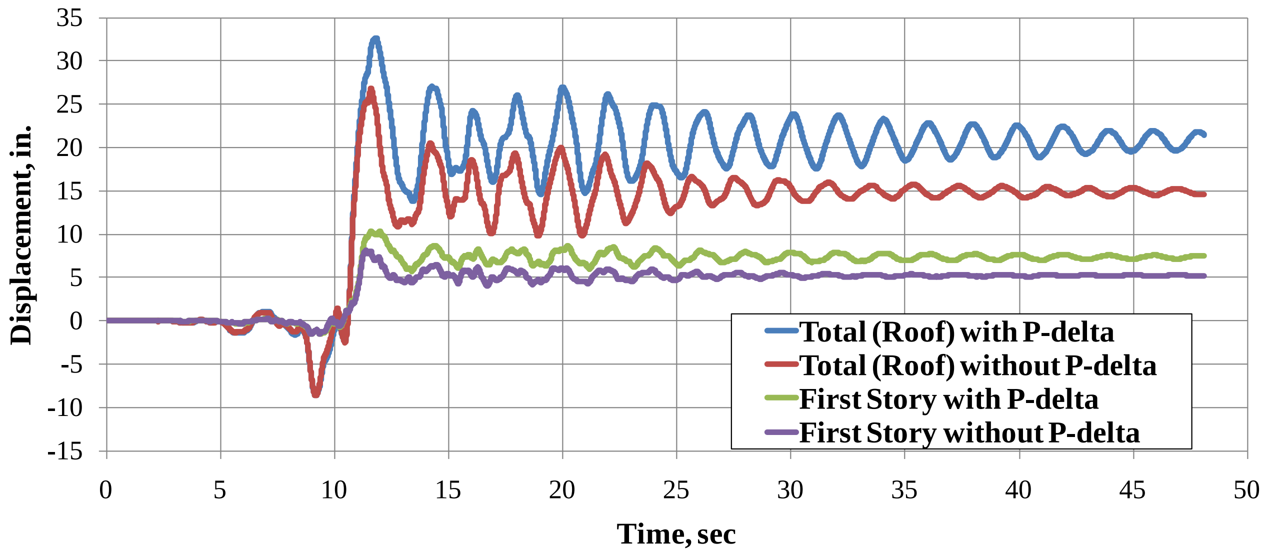
(a) Maximum Base Shear (kips)

	Motion A00	Motion B90	Motion C90
Column Forces	2181 / 1675	1851 / 1584	1723 / 1507
Inertial Forces	2261 / 1854	1893 / 1633	1725 / 1515

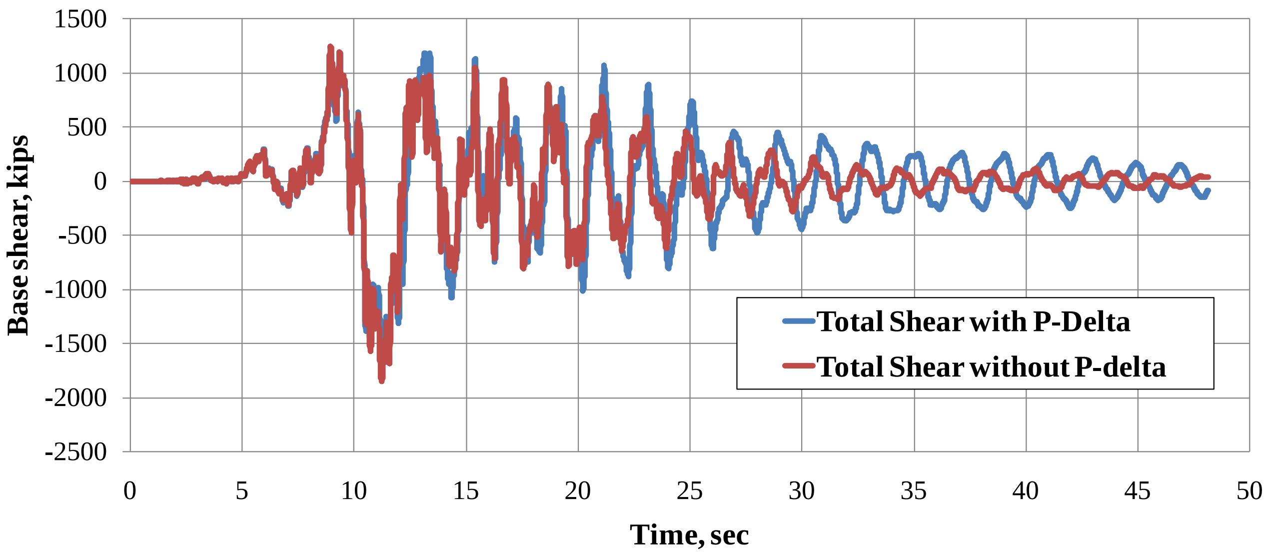
(b) Maximum Story Drifts (in.)

Level	Motion A00	Motion B90	Motion C90	Limit*
Total Roof	62.40 / 101.69	22.45 / 26.10	20.41 / 20.50	NA
R-6	1.98 / 1.95	2.30 / 2.32	3.05 / 2.93	4.50
6-5	3.57 / 2.97	2.77 / 2.60	3.69 / 3.49	4.50
5-4	**7.36** / **6.41**	3.33 / 3.62	4.43 / 4.32	4.50
4-3	**14.61** / **20.69**	**4.61** / **5.61**	4.45 / **4.63**	4.50
3-2	**16.29** / **31.65**	**5.21** / **6.32**	3.97 / 4.18	4.50
2-G	**19.76** / **40.13**	**6.60 / 7.03**	5.11 / 5.11	5.40

Results of Response History Analysis

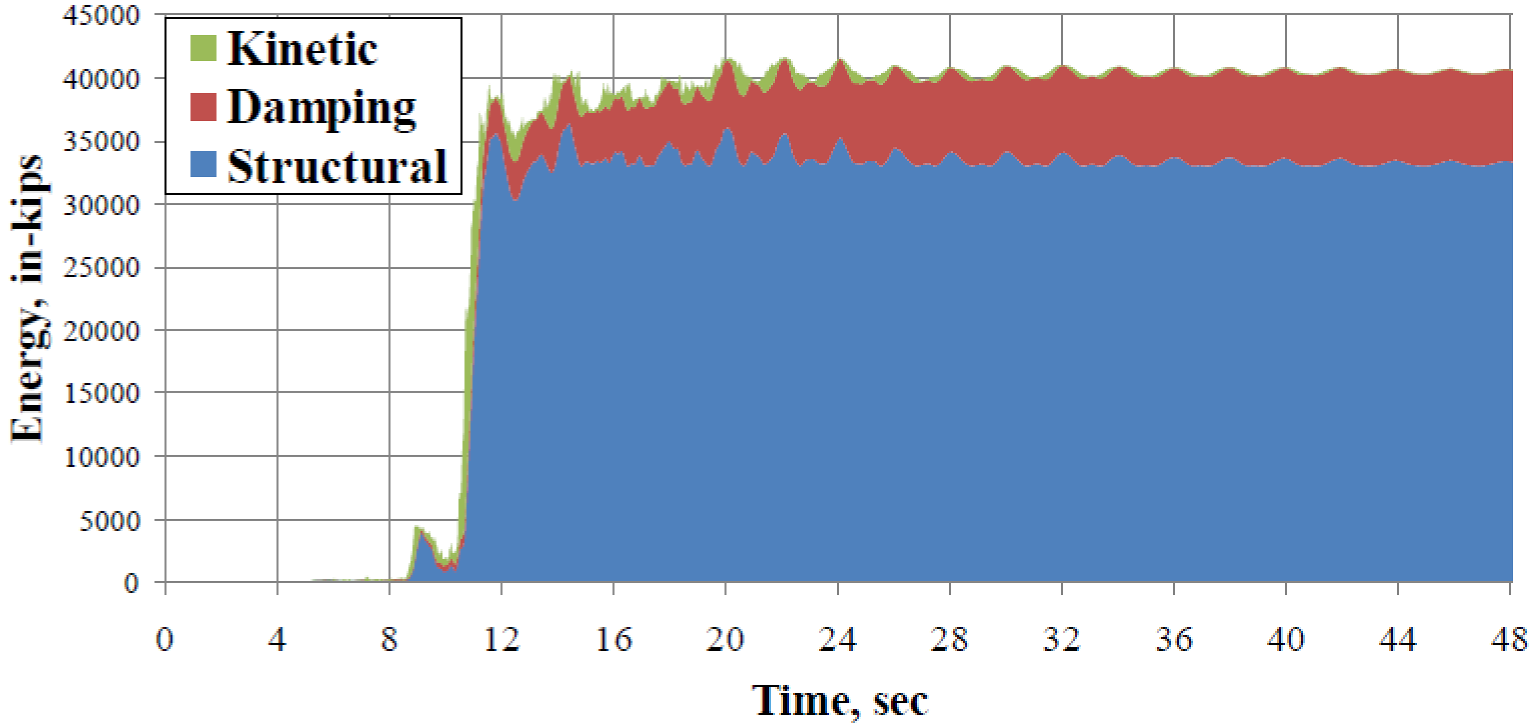


Response Histories of Roof and First-story Displacement, Ground Motion A00 (DBE)



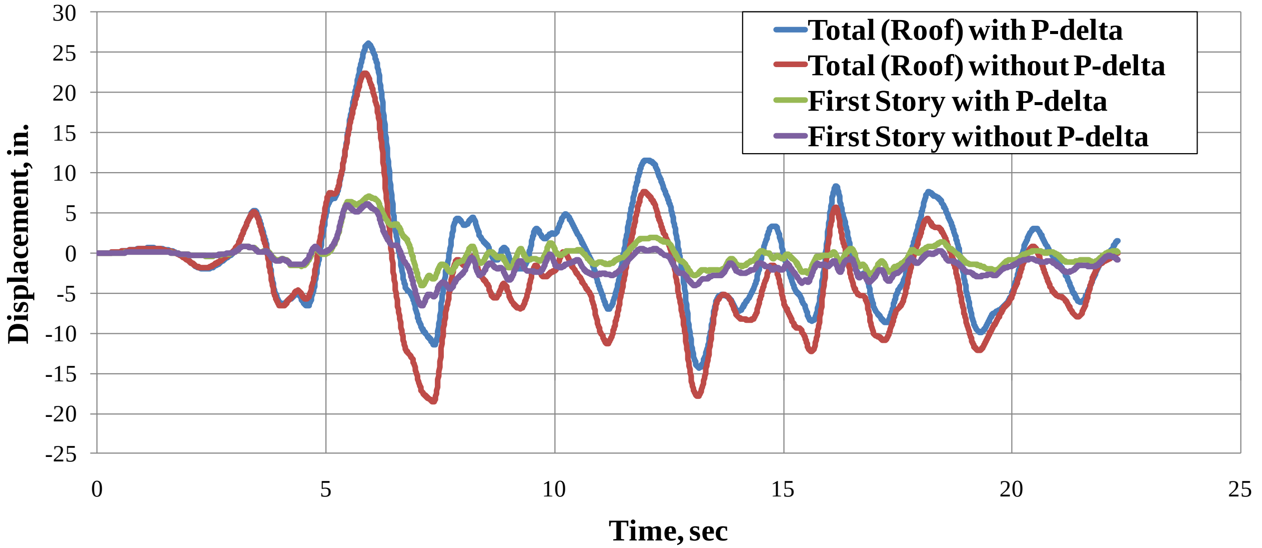
Response History of Total Base Shear, Ground Motion A00 (DBE)

Results of Response History Analysis

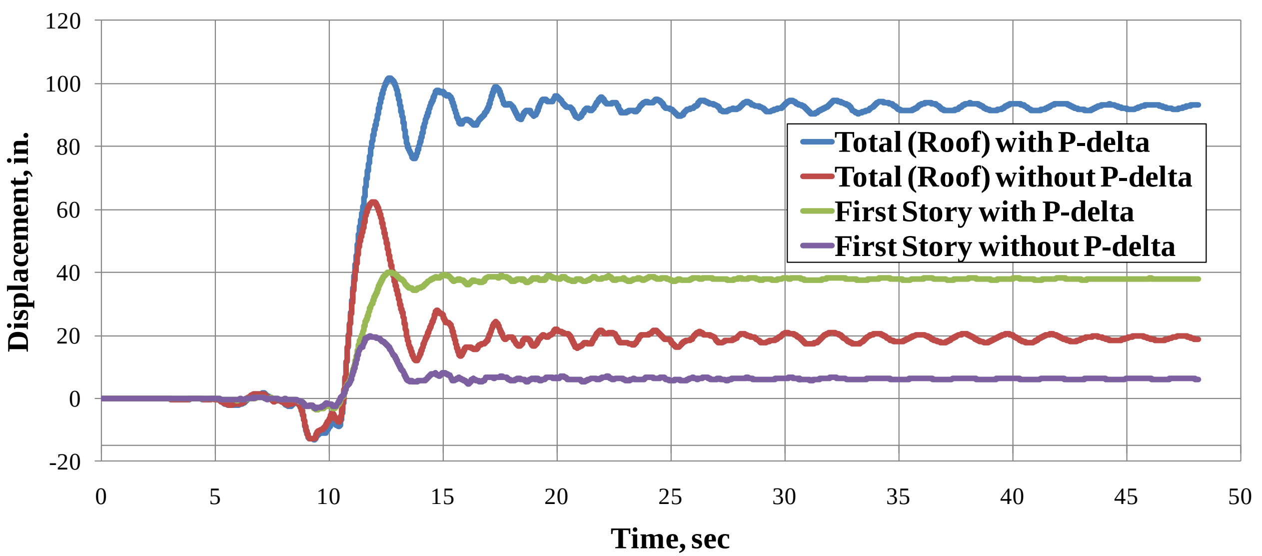


Energy Response History, Ground Motion A00 (DBE), including P-delta effects

Results of Response History Analysis



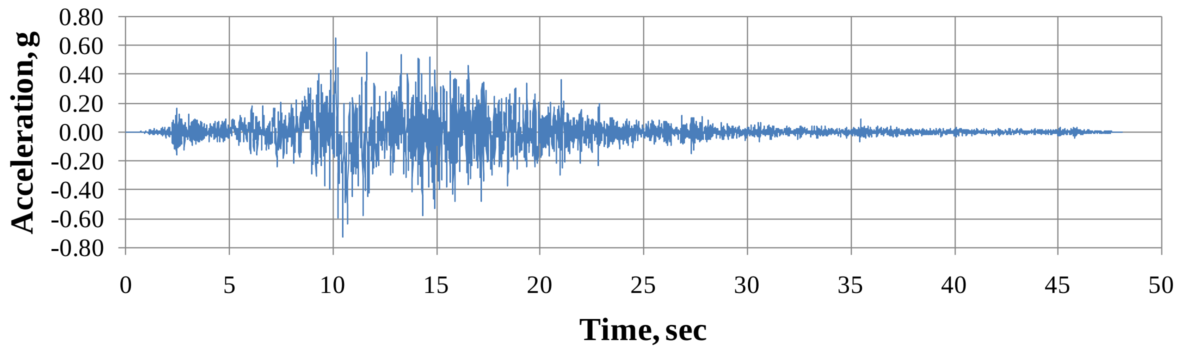
Response Histories of Roof and First-story Displacement, Ground Motion B90 (MCE)

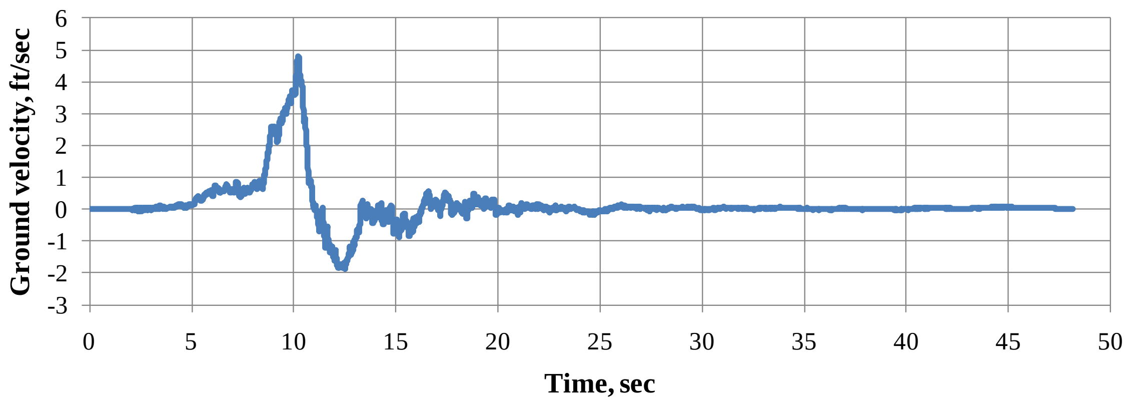


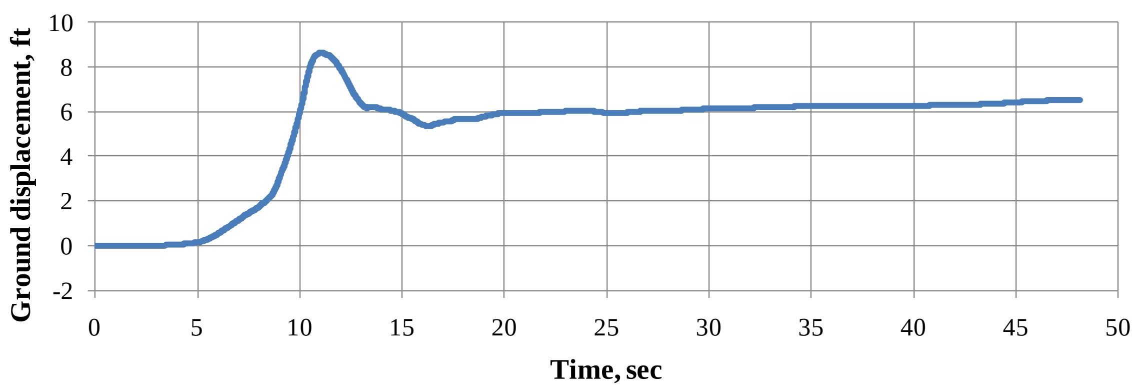
Response History of Roof and First-story Displacement, Ground Motion A00 (MCE)

Response History Analysis

A00 Motion Ground Acceleration, Velocity and Displacement

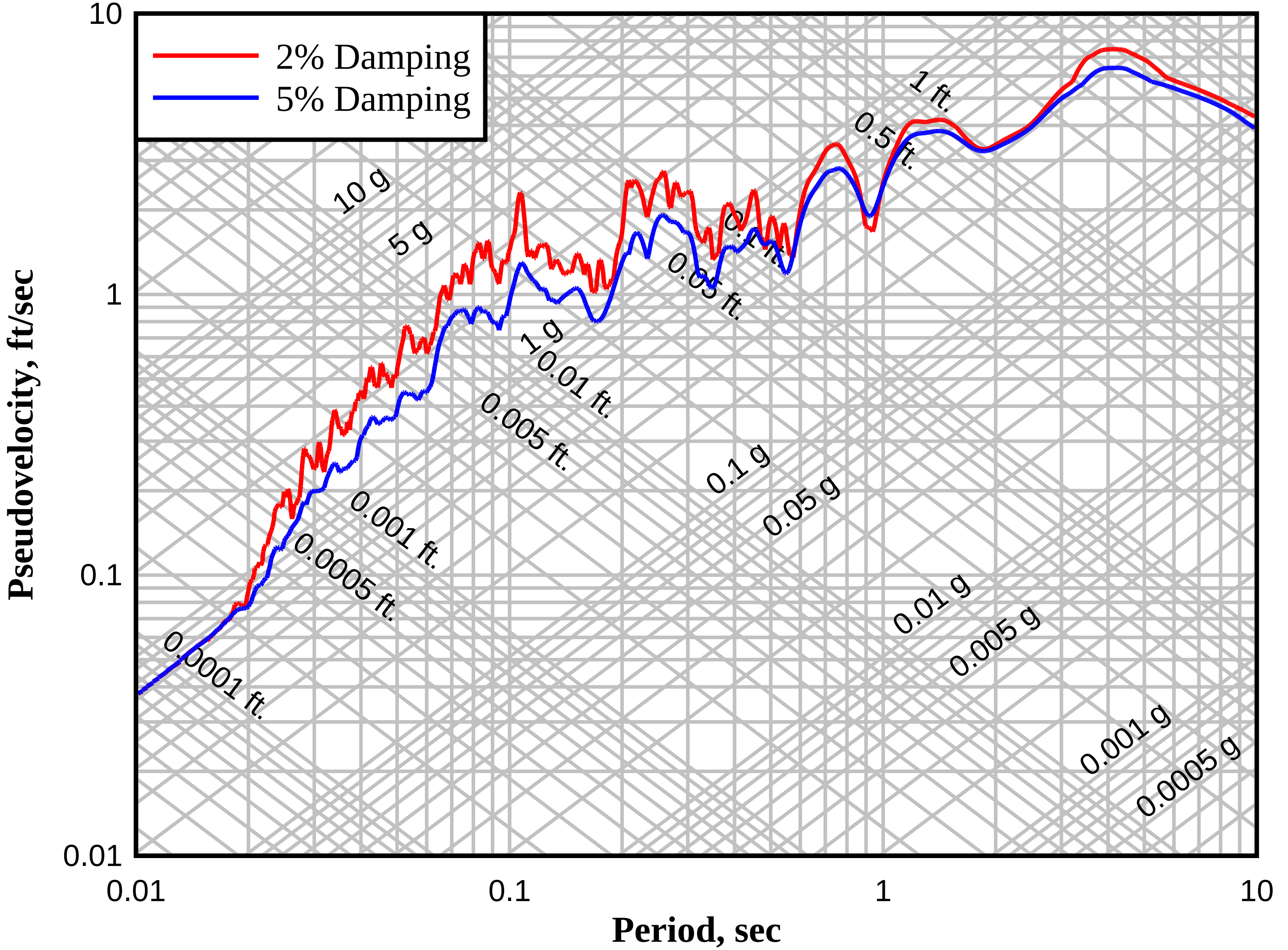




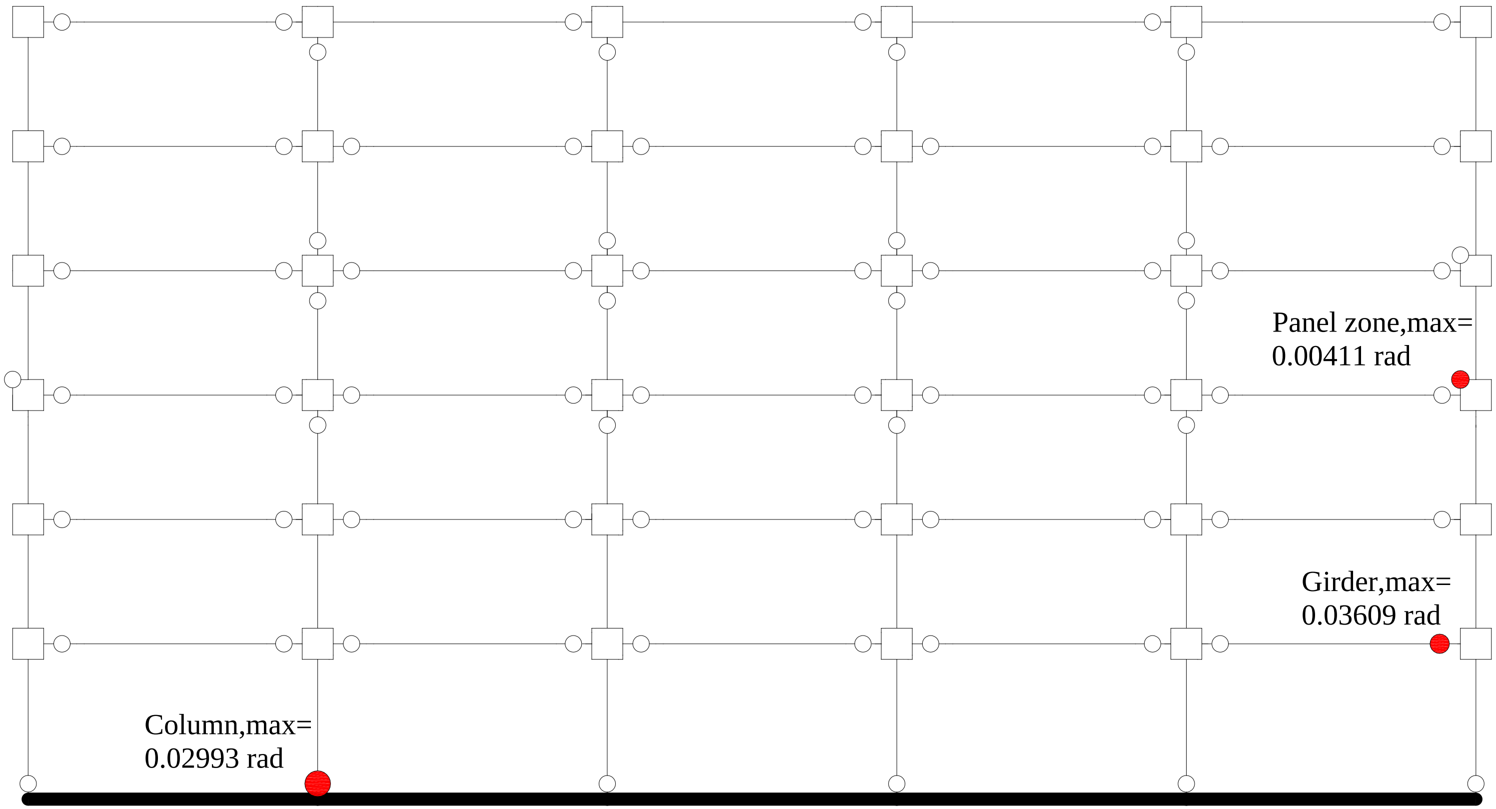


Response History Analysis

A00 Motion tripartite Spectrum



Results of Response History Analysis



Yielding locations for structure with strong panels subjected to MCE scaled B90 motion, including P-delta effects

Results of Response History Analysis

Comparison with Results from Other Analyses

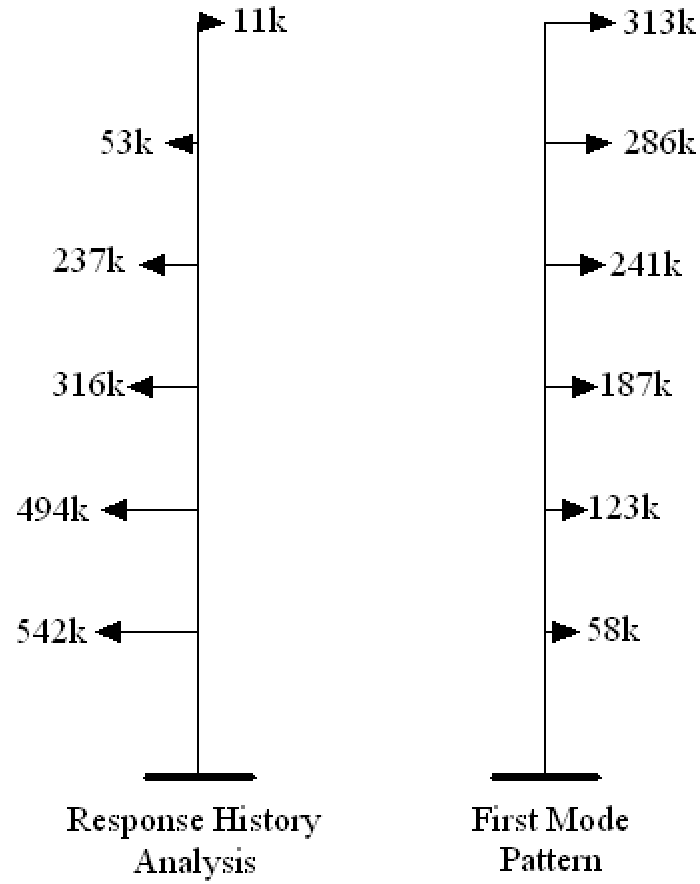
Response Quantity	Analysis Method: Equivalent Lateral Forces	Analysis Method: Nonlinear Static Pushover	Analysis Method: Nonlinear Dynamic
Base Shear (kips)	569	1208	1633
Roof Disp. (in.)	18.4	22.9	26.1
Drift R-6 (in.)	1.86	0.96	2.32
Drift 6-5 (in.)	2.78	1.76	2.60
Drift 5-4 (in.)	3.34	2.87	3.62
Drift 4-3 (in.)	3.73	4.84	5.61
Drift 3-2 (in.)	3.67	5.74	6.32
Drift 2-1 (in.)	2.98	6.73	7.03
Girder Hinge Rot. (rad)	NA	0.03304	0.03609
Column Hinge Rot. (rad)	NA	0.02875	0.02993
Panel Hinge Rot. (rad)	NA	0.00335	0.00411
Panel Plastic Shear Strain	NA	0.00335	0.00411

Note: Shears are for half of total structure.

Results of Response History Analysis

<u>Reasons of the differences between Pushover and Response History Analyses</u>

- Scale factor of 1.367 was used for the 2^{nd} part of the scaling procedure.
- The use of the first-mode lateral loading pattern in the nonlinear static pushover response.
- The higher mode effects shown in the Figure are the likely cause of the different hinging patterns and are certainly the reason for the very high base shear developed in the response history analysis.



Comparison of inertial force patterns

Results of Response History Analysis

Effect of Increased Damping on Response

- Excessive drifts occur in the bottom three stories.
- Additional strength and/or stiffness should be provided at these stories.
- Considered next, Added damping is also a viable approach.
- Four different damper configurations were used.
- Dampers were added to the Strong Panel frame with 2% inherent damping.
- The structure was subjected to the DBE scaled A00 and B90 ground motions.
- P-delta effects were included in the analyses.

Modeling Added Dampers

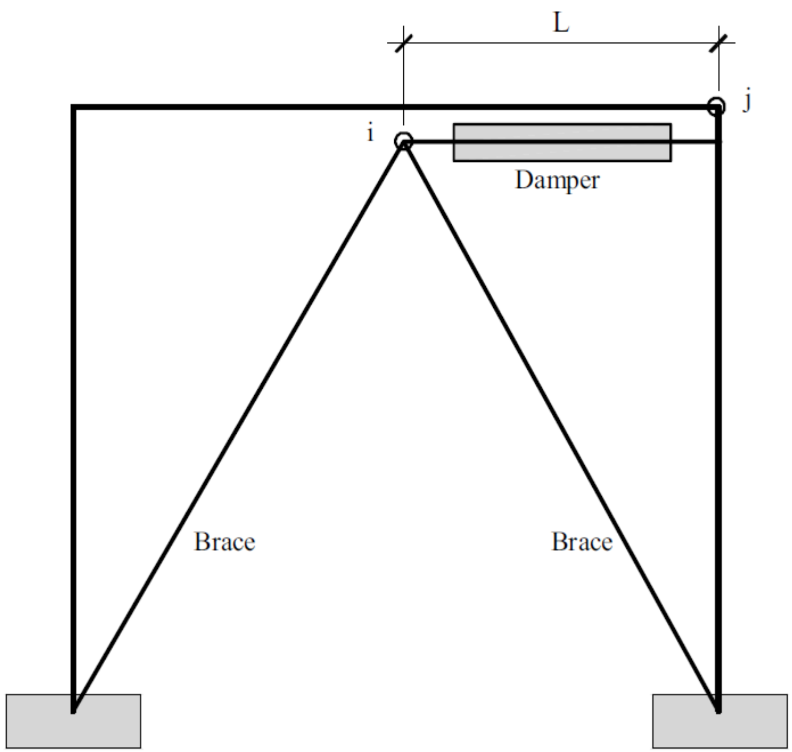
- Added damping is easily accomplished in DRAIN by use of the stiffness proportional component of Rayleigh damping.
- Linear viscous fluid damping device can be modeled through use of a Type-1 (truss bar) element.

$$k_{device} = \frac{A_{device} E_{device}}{L_{device}}$$

$$C_{device} = \beta_{device} k_{device}$$

- Set damper elastic stiffness to negligible value. k_{device} = 0.001 kips/in.

$$\beta_{device} = \frac{C_{device}}{0.001} = 1000 C_{device}$$



Modeling a simple damper

- It is convenient to set E_{device} = 0.001 and A_{device} = Damper length L_{device}

Results of Response History Analysis

Effect of Increased Damping on Response

Effect of different added damper configurations when SP model is subjected to DBE scaled A00 motion, including P-delta effects

Level	No Damper	1st combo		2nd combo		3rd combo		4th combo		
	Drift, In.	Damper Coeff, kip-sec/in.	Drift, in.	Damper Coeff, kip-sec/in.	Drift, in.	Damper Coeff, kip-sec/in.	Drift, in.	Damper Coeff, kip-sec/in.	Drift, in.	Drift Limit in.
R-6	1.86	10.5	1.10	60	1.03	-	1.82	-	1.47	3.75
6-5	2.64	33.7	1.90	60	1.84	-	3.56	-	2.41	3.75
5-4	**4.08**	38.4	2.99	70	2.88	-	**4.86**	56.25	3.46	3.75
4-3	**6.87**	32.1	**5.46**	70	**4.42**	-	**5.24**	56.25	**4.47**	3.75
3-2	**8.19**	36.5	**6.69**	80	**5.15**	160	**4.64**	112.5	**4.76**	3.75
2-G	**10.40**	25.6	**8.39**	80	**5.87**	160	4.40	112.5	**4.96**	4.50
Column Base Shear,kips	1467	1629		2170		2134		2267		
Inertial Base Shear,kips	1558	1728		2268		2215		2350		
Total Damping,%	2	10.1		20.4		20.2		20.4		

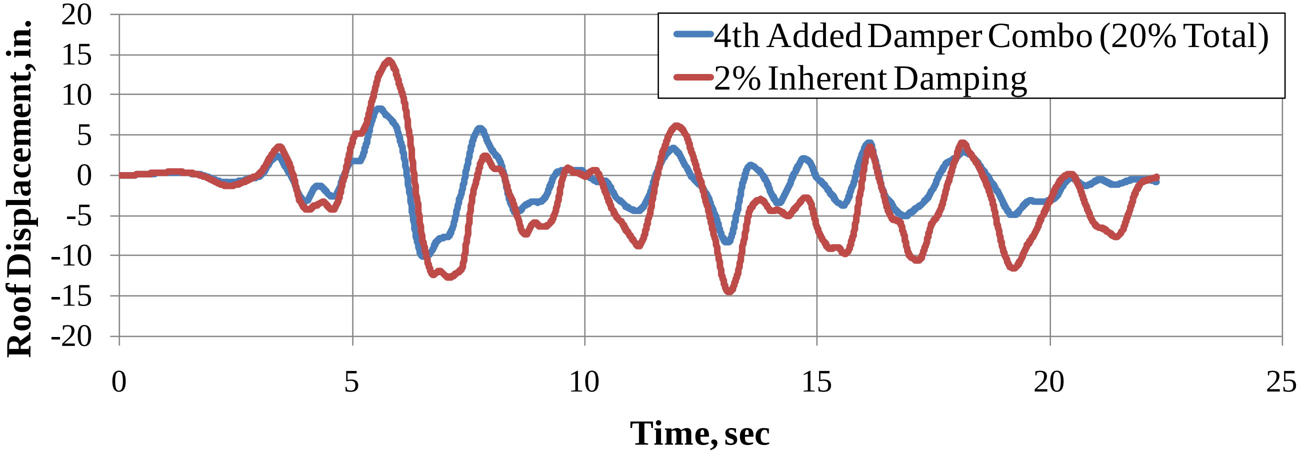
Results of Response History Analysis

Effect of Increased Damping on Response

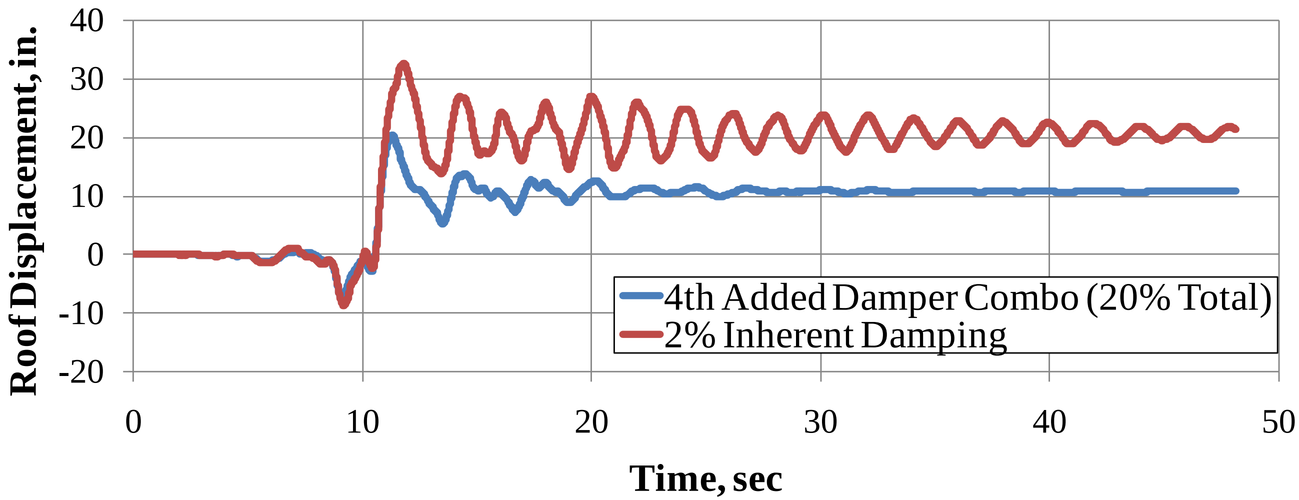
Effect of different added damper configurations when SP model is subjected to DBE scaled B90 motion, including P-delta effects

	No Damper	1st combo		2nd combo		3rd combo		4th combo		
Level	Drift, In.	Damper Coeff, kip-sec/in.	Drift, in.	Damper Coeff, kip-sec/in.	Drift, in.	Damper Coeff, kip-sec/in.	Drift, in.	Damper Coeff, kip-sec/in.	Drift, in.	Drift Limit in.
R-6	1.82	10.5	1.11	60	0.86	-	1.53	-	1.31	3.75
6-5	2.50	33.7	1.76	60	1.35	-	2.11	-	1.83	3.75
5-4	2.81	38.4	2.33	70	1.75	-	2.51	56.25	2.07	3.75
4-3	3.21	32.1	2.67	70	2.11	-	2.37	56.25	2.16	3.75
3-2	3.40	36.5	2.99	80	2.25	160	2.09	112.5	2.13	3.75
2-G	**4.69**	25.6	3.49	80	1.96	160	1.87	112.5	1.82	4.50
Column Base Shear,kips	1458	1481		1485		1697		1637		
Inertial Base Shear,kips	1481	1531		1527		1739		1680		
Total Damping,%	2	10.1		20.4		20.2		20.4		

Results of Response History Analysis : Roof Displacements

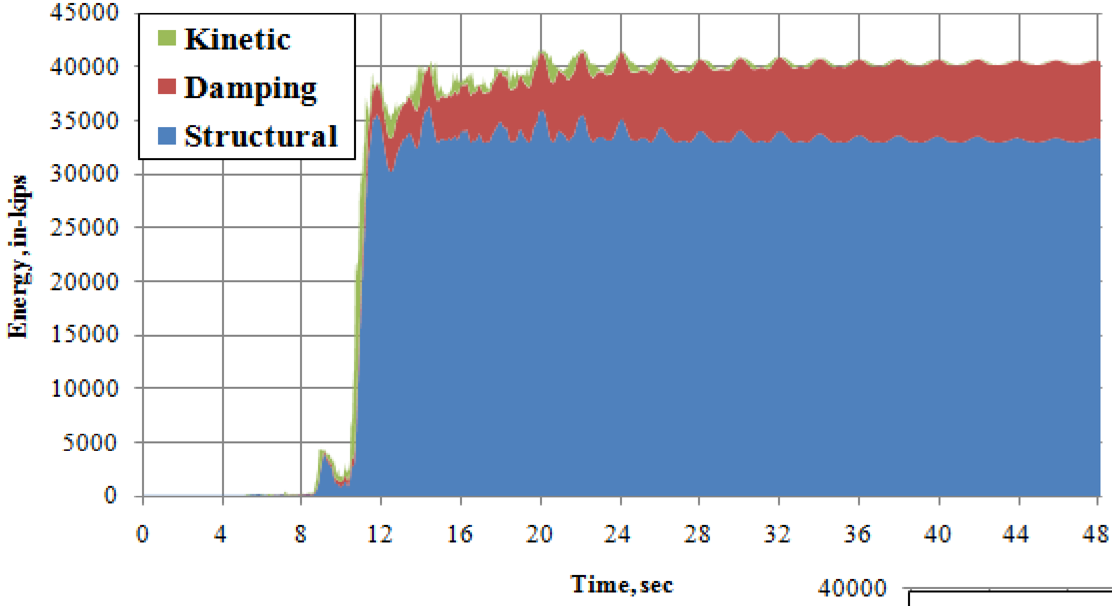


Roof Displacement Response Histories with added damping (20% total) and inherent damping (2%) for B90 motion



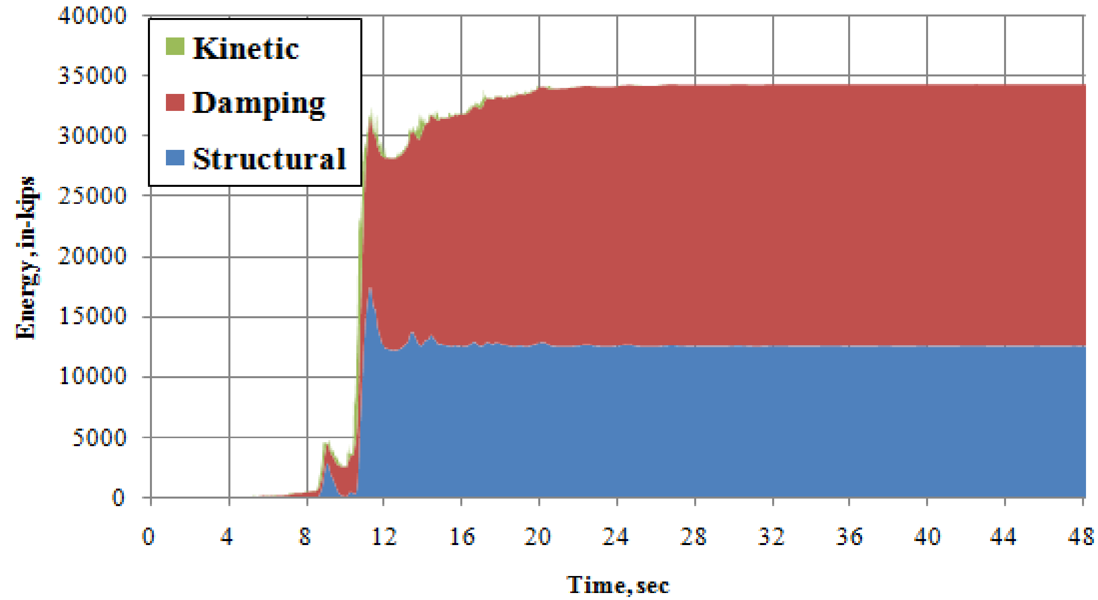
Roof Displacement Response Histories with added damping (20% total) and inherent damping (2%) for A00 motion

Results of Response History Analysis: Energy Plots

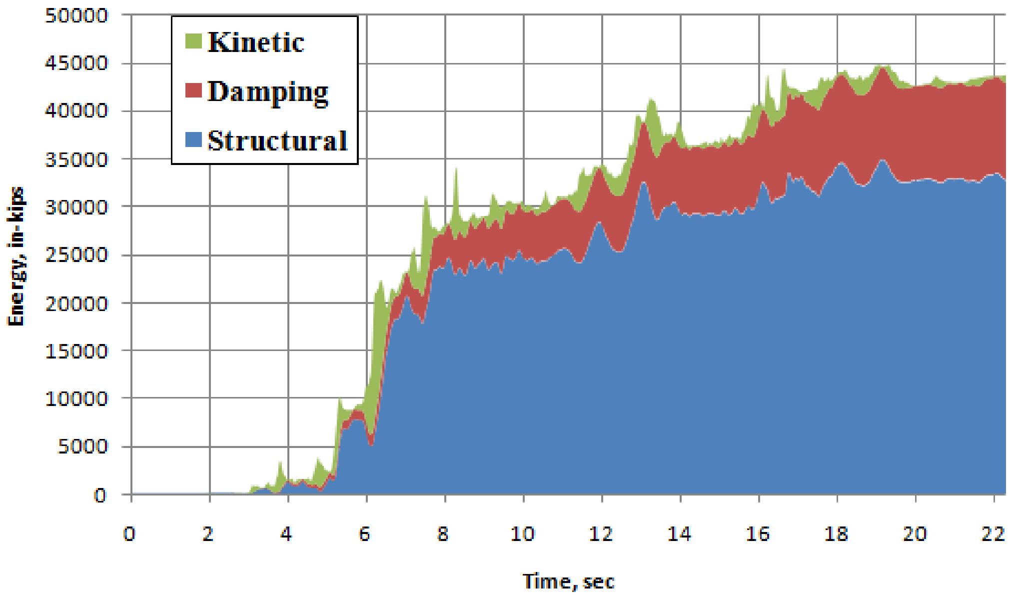


Energy Response History with inherent damping (2% total damping) for A00 motion

Energy Response History with added damping of 4th combination (20% total damping) for A00 motion

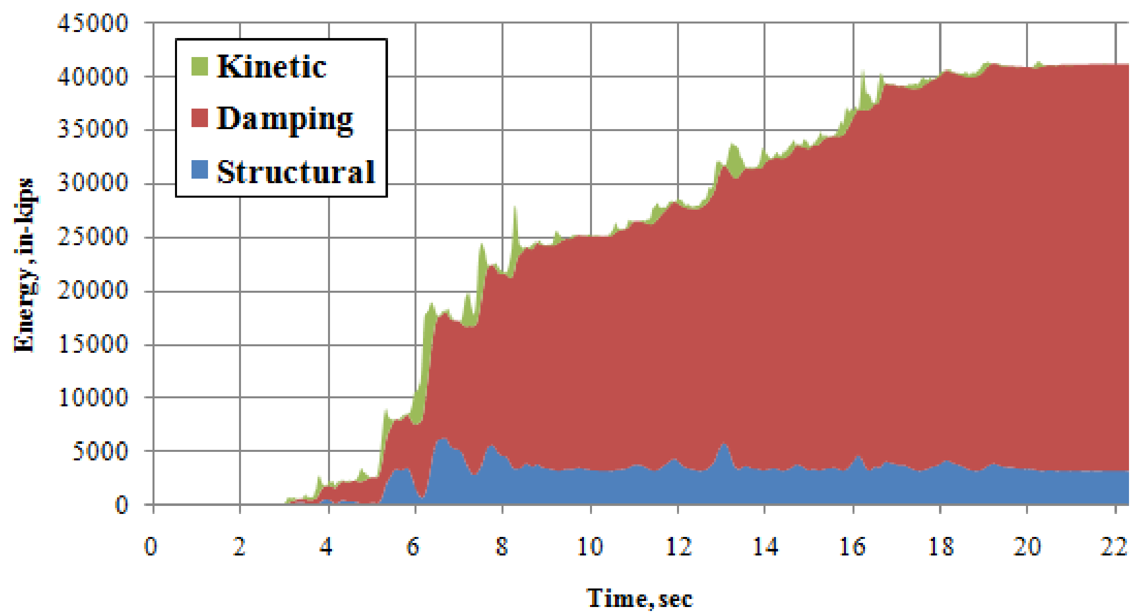


Results of Response History Analysis: Energy Plots

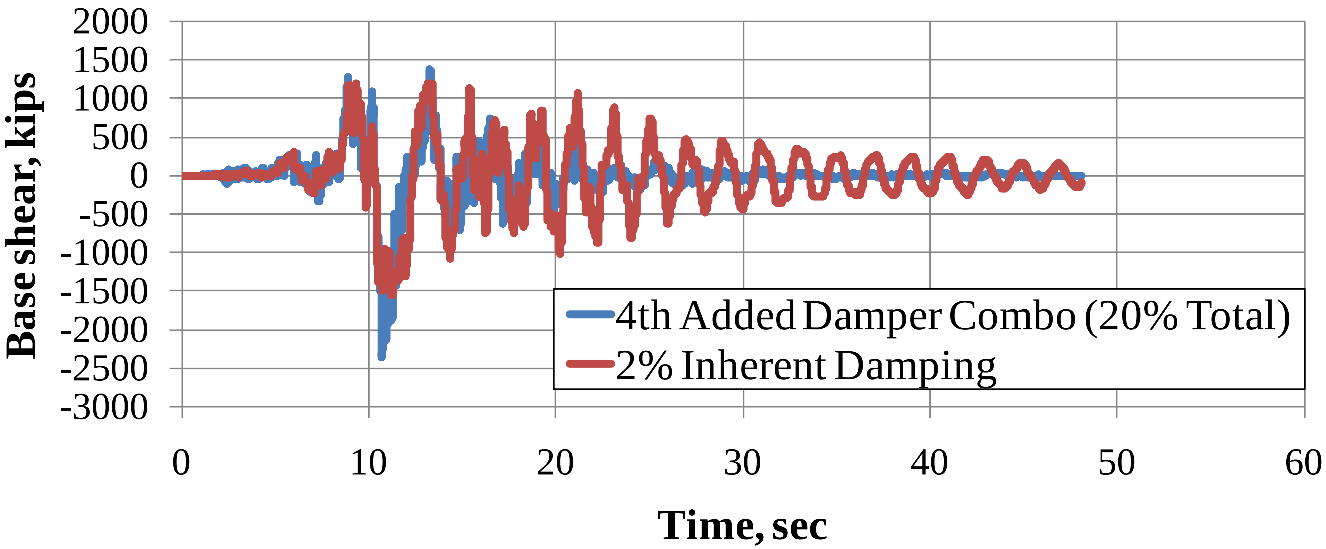


Energy Response History with inherent damping (2% total damping) for B90 motion

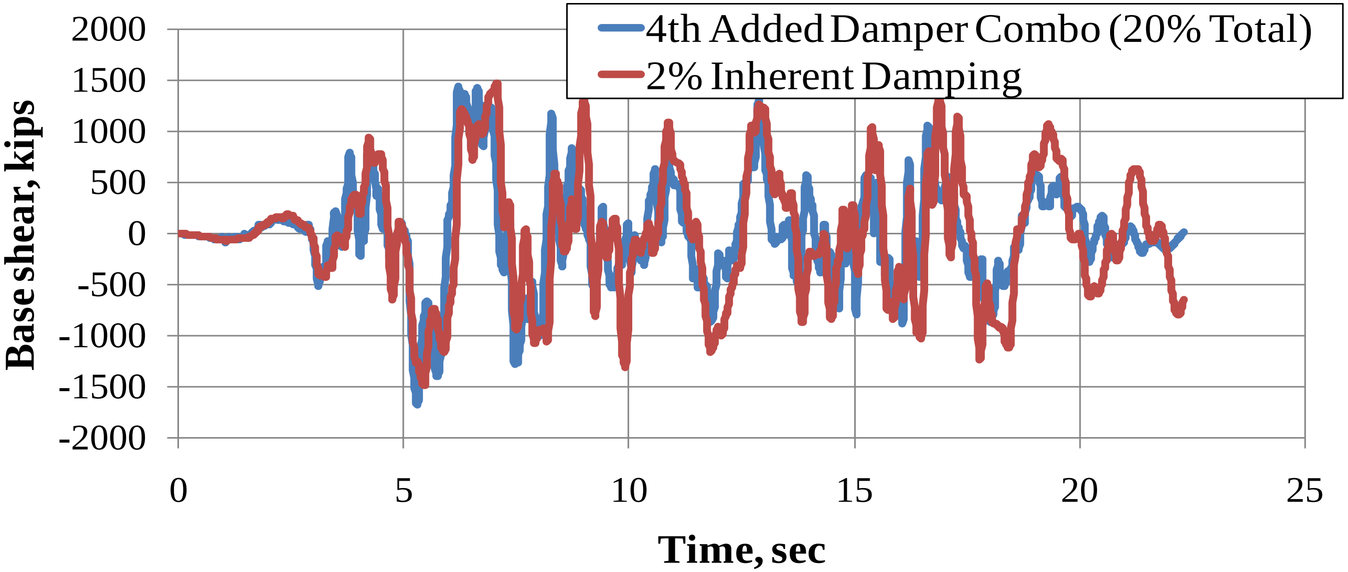
Energy Response History with added damping of 4th combination (20% total damping) for B90 motion



Results of Response History Analysis: Base Shear

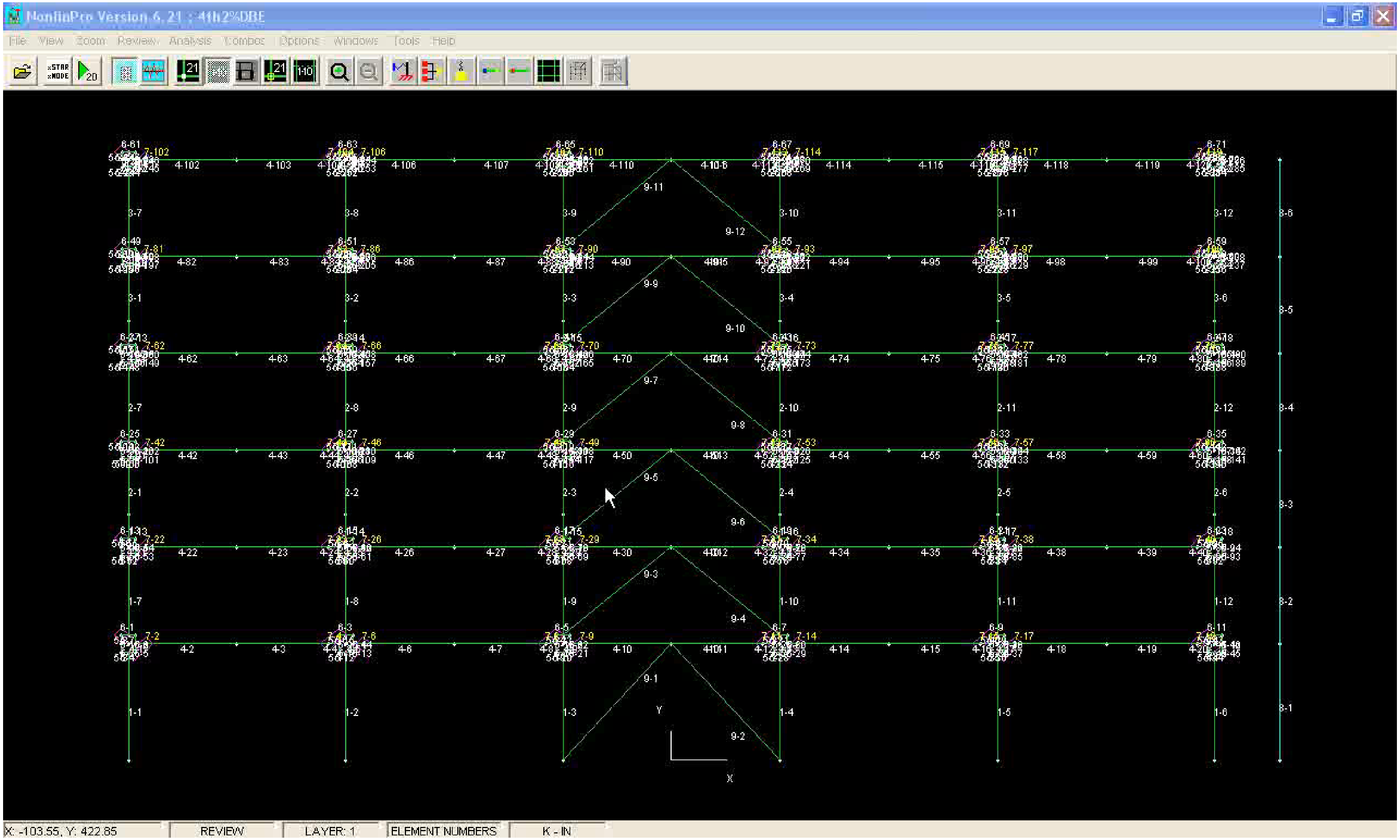


Inertial Base Shear Response Histories with added damping (20% total) and inherent damping (2%) for A00 motion



Inertial Base Shear Response Histories with added damping (20% total) and inherent damping (2%) for B90 motion

Results of Response History Analysis: Deflected Shape of by NonlinPro for Added Damper Frame (4th combination) During B90 Motion



Summary and Conclusions

- Five different analytical approaches were used to estimate the deformation demands in a simple unbraced steel frame structure:
    1. Linear static analysis (the equivalent lateral force method)
    2. Plastic strength analysis (using virtual work)
    3. Nonlinear static pushover analysis
    4. Linear dynamic analysis
    5. Nonlinear dynamic response history analysis
- Approaches 1, 3, and 5 were carried to a point that allowed comparison of results. The results obtained from the three different analytical approaches were quite dissimilar.
- Because of the influence of the higher mode effects on the response, pushover analysis, where used alone, is inadequate.
- Except for preliminary design, the ELF approach should not be used in explicit performance evaluation as it has no mechanism for determining location and extent of yielding in the structure.
- Response history analysis as the most viable approach. However, significant shortcomings, limitations, and uncertainties in response history analysis still exist.
- In modeling the structure, particular attention was paid to representing possible inelastic behavior in the panel-zone regions of the beam-column joints.

Questions?

In this example, the behavior of a simple, six-story structural steel moment-resisting frame is investigated using a variety of analytical techniques. The structure was initially proportioned using a preliminary analysis, and it is this preliminary design that is investigated. The analysis will show that the structure falls short of several performance expectations. In an attempt to improve performance, viscous fluid dampers are considered for use in the structural system.

Complete details for the analysis are provided in the written example, and the example should be used as the "Instructors Guide" when presenting this slide set. Many, but not all of the slides in this set have "Speakers Notes", and these are intentionally kept very brief.

Description of Structure

- 6-story office building in Seattle, Washington
- Occupancy (Risk) Category II
- Importance factor (I) = 1.0
- Site Class = C
- Seismic Design Category D
- Special Moment Frame (SMF), R = 8, C_d = 5.5

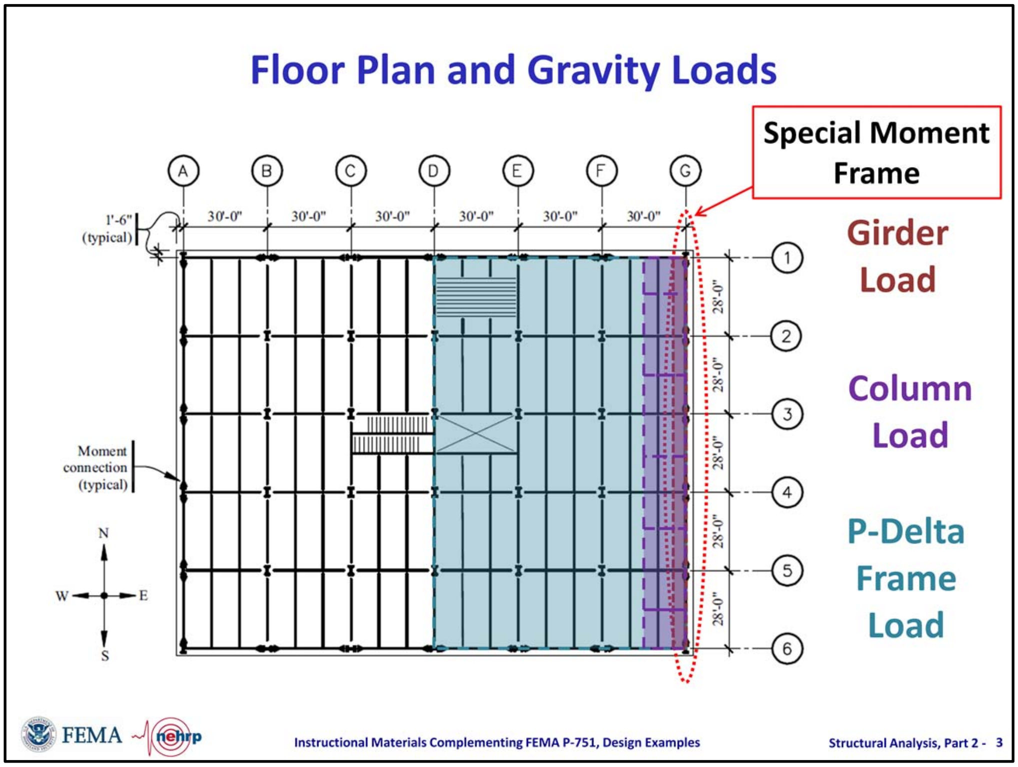
According to the descriptions in ASCE 7-05 Table 1-1, the building is assigned to Occupancy Category II. This is similar to Risk Category II in ASCE 7-10 Table 1.5-1.

From ASCE 7-05 Table 11.5-1, the importance factor (I) is 1.0. Importance factor is provided in Table 1.5-2 in ASCE 7-10. I_e (seismic importance factor) is 1.0 for Risk Category II.

Site classification is provided in *Standard* Table 20.3-1.

Seismic design category is provided in Tables 11.6-1 and 11.6-2 in *Standard.*

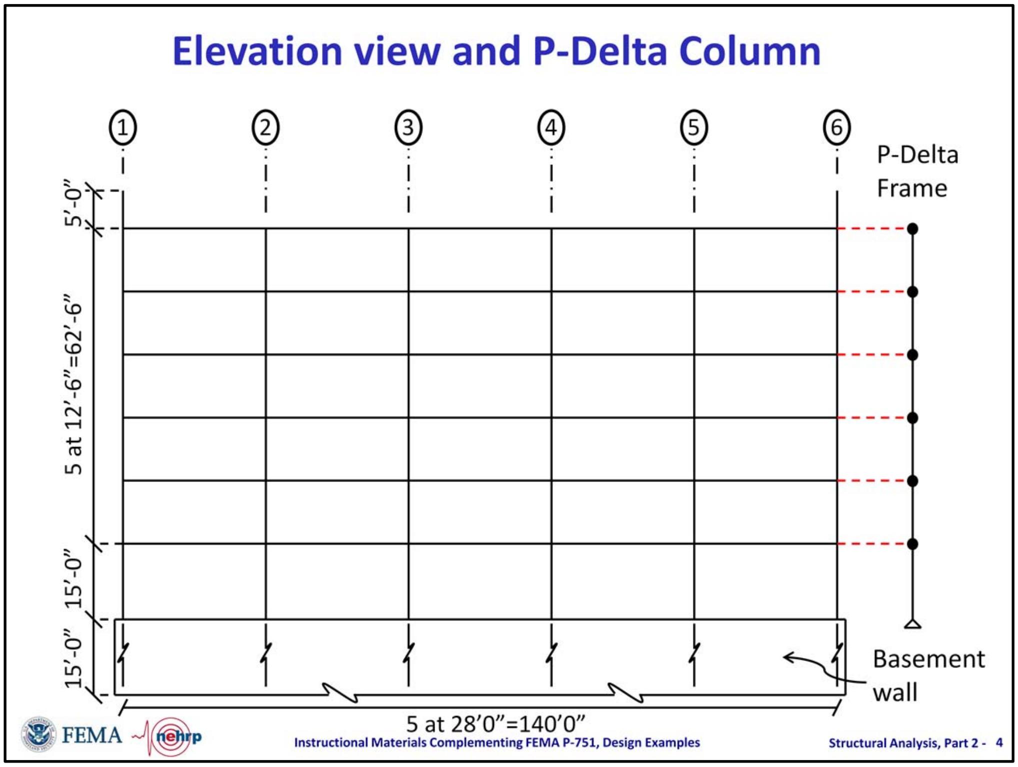
Response modification coefficient (R), overstrength factor (Ω_o), and deflection amplification factor (C_d) for seismic force-resisting systems are provided in Table 12.2-1 in *Standard.*



The lateral-load-resisting system consists of steel moment-resisting frames on the perimeter of the building. There are five bays at 28 ft on center in the N-S direction and six bays at 30 ft on center in the EW direction. The lateral load-resisting system consists of steel moment-resisting frames on the perimeter of the building.

For the moment-resisting frames in the N-S direction (Frames A and G), all of the columns bend about their strong axes, and the girders are attached with fully welded moment-resisting connections. The expected plastic hinge regions of the girders have reduced flange sections, detailed in accordance with the AISC 341-05 *Seismic Provisions for Structural Steel Buildings* (AISC, 2005a).

For the frames in the E-W direction (Frames 1 and 6), moment-resisting connections are used only at the interior columns. At the exterior bays, the E-W girders are connected to the weak axis of the exterior (corner) columns using non-moment-resisting connections. All interior columns are gravity columns and are not intended to resist lateral loads. A few of these columns, however, would be engaged as part of the added damping system described in the last part of this example. With minor exceptions, all of the analyses in this example will be for lateral loads acting in the N-S direction. Analysis for lateral loads acting in the E-W direction would be performed in a similar manner.



The typical story height is 12 ft-6 in. with the exception of the first story, which has a height of 15 ft. There is a 5-ft-tall perimeter parapet at the roof and one basement level that extends 15 ft below grade. For this example, it is assumed that the columns of the moment resisting frames are embedded into pilasters formed into the basement wall.

P-Delta effects are modeled using the leaner "ghost" column shown in Figure at the right of the main frame. This column is modeled with an axially rigid truss element. P-Delta effects are activated for this column only (P-Delta effects are turned off for the columns of the main frame). The lateral degree of freedom at each level of the P-Delta column is slaved to the floor diaphragm at the matching elevation. Where P-Delta effects are included in the analysis, a special initial load case was created and executed. This special load case consists of a vertical force equal to one-half of the total story weight (dead load plus 50 percent of the fully reduced live load) applied to the appropriate node of the P-Delta column.

Member Sizes Used in N-S Moment Frames

Member Supporting Level	Column	Girder	Doubler Plate Thickness (in.)
R	W21x122	W24x84	1.00
6	W21x122	W24x84	1.00
5	W21x147	W27x94	1.00
4	W21x147	W27x94	1.00
3	W21x201	W27x94	0.875
2	W21x201	W27x94	0.875

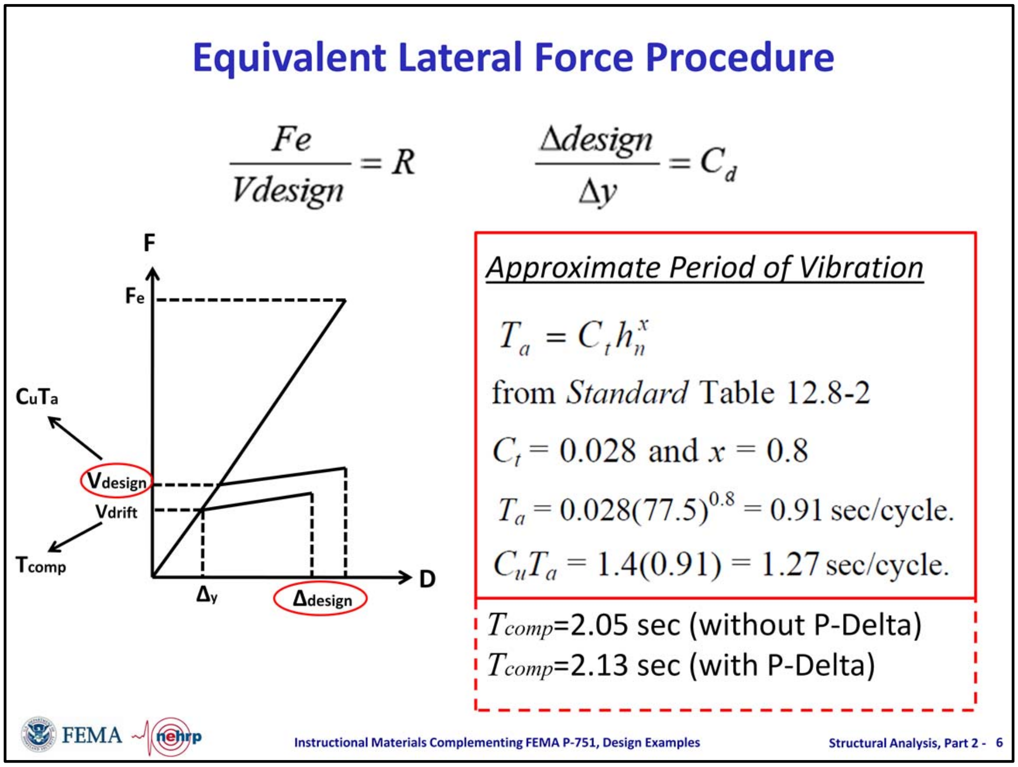
✓ Sections meet the width-to-thickness requirements for special moment frames

✓ Strong column-weak beam

Prior to analyzing the structure, a preliminary design was performed in accordance with the *AISC Seismic Provisions*. All members, including miscellaneous plates, were designed using steel with a nominal yield stress of 50 ksi and expected yield strength of 55 ksi**.** Detailed calculations for the design are beyond the scope of this example.

The sections shown in Table meet the width-to-thickness requirements for special moment frames, and the size of the column relative to the girders should ensure that plastic hinges will form in the girders. Due to strain hardening, plastic hinges will eventually form in the columns.

However, these form under lateral displacements that are in excess of those allowed under the Design Basis Earthquake (DBE). Doubler plates of 0.875 in. thick are used at each of the interior columns at Levels 2 and 3, and 1.00 in. thick plates are used at the interior columns at Levels 4, 5, 6, and R. Doubler plates were not used in the exterior columns.



Although the main analysis in this example is nonlinear, equivalent static forces are computed in accordance with the Section 12.8 of the *Standard.* These forces are used in a preliminary static analysis to determine whether the structure, as designed, conforms to the drift requirement limitations imposed by Section 12.12 of the *Standard.*

For the purpose of analysis, it is assumed that the structure complies with the requirements for a special moment frame, which, according to *Standard Table 12.2-1,* has the following design values:

R = 8

C_d = 5.5

Ω_o = 3.0

Note that the overstrength factor Ω_0 is not needed for the analysis presented herein.

In *Standard* section 12.8.6.2, it is permitted to determine the elastic drifts using seismic design forces based on the computed fundamental period of the structure without the upper limit on calculated approximate period ($C_u T_a$). Thus, a new set of lateral forces (V_{drift} in Figure) were calculated and elastic drifts were found using these forces. Drift limitations of *Standard* Section 12.12 were satisfied with the amplified drifts (Δ_{drift} in Figure) found with these new set of lateral forces.

Equivalent Lateral Force Procedure

Vertical Distribution of Forces

$$F_x = C_{vx}V \text{ and } C_{vx} = \frac{w_x h^k}{\sum_{i=1}^{n} w_i h_i^k}$$

Equivalent Lateral Forces for Building Responding in N-S Direction

Level x	w_x (kips)	h_x (ft)	$w_x h_x^k$	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
R	2,596	77.5	1,080,327	0.321	243.6	243.6	3,045
6	2,608	65.0	850,539	0.253	191.8	435.4	8,488
5	2,608	52.5	632,564	0.188	142.6	578.0	15,713
4	2,608	40.0	433,888	0.129	97.8	675.9	24,161
3	2,608	27.5	258,095	0.077	58.2	734.1	33,337
2	2,621	15.0	111,909	0.033	25.2	759.3	44,727
Σ	15,650		3,367,323	1.000	759.3		

Vertical distribution of lateral forces were calculated in accordance with Standard Section 12.8.3.

The lateral forces acting at each level (F_x) and the story shears (V_x) at the bottom of the story below the indicated level are summarized in the table. Note that these are the forces acting on the whole building. Thus, for analysis of a single frame, one-half of the tabulated values are used.

Computer Programs NONLIN-Pro and DRAIN 2Dx

<u>Shortcomings of DRAIN</u>

- It is not possible to model strength loss when using the ASCE 41-06 (2006) model for girder plastic hinges.
- The DRAIN model for axial-flexural interaction in columns is not particularly accurate.
- Only Two-Dimensional analysis may be performed.

<u>Elements used in Analysis</u>

- Type 1, inelastic bar (truss) element
- Type 2, beam-column element
- Type 4, connection element

Loss of strength generally occurs at plastic hinge rotations well beyond the rotational demands produced under the DBE ground motions. Maximum plastic rotation angles of plastic hinges were checked with the values in Table 5-6 of ASCE 41-06.

The rules employed by DRAIN to model column yielding are adequate for event-to-event nonlinear static pushover analysis, but leave much to be desired where dynamic analysis is performed. The greatest difficulty in the dynamic analysis is adequate treatment of the column when unloading and reloading.

Two dimensional analysis is reasonable for the structure considered in this example because of its regular shape and because full moment connections are provided only in the N-S direction for the corner columns.

Description of Preliminary Model

- Only a single frame (Frame A or G) is modeled.
- Columns are fixed at their base.
- Each beam or column element is modeled using a Type 2 element. For the columns, axial, flexural, and shear deformations are included. For the girders, flexural and shear deformations are included but, because of diaphragm slaving, axial deformation is not included. Composite action in the floor slab is ignored for all analysis.
- All members are modeled using centerline dimensions without rigid end offsets.
- This model does not provide any increase in beam-column joint stiffness due to the presence of doubler plates.
- The stiffness of the girders was decreased by 7% in the preliminary analyses, which should be a reasonable approximate representation of the 35% reduction in the flange sections. Moment rotation properties of the reduced flange sections are used in the detailed analyses.

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P-delta effects are modeled using the leaner “ghost” column shown which is laterally constrained to the main frame, as explained before.

Results of Preliminary Analysis : Drift

Results of Preliminary Analysis Excluding P-delta Effects

Story	Total Drift (in.)	Story Drift (in.)	Magnified Story Drift (in.)	Drift Limit (in.)	Story Stability Ratio, θ
6	2.08	0.22	1.21	3.00	0.0278
5	1.86	0.32	1.76	3.00	0.0453
4	1.54	0.38	2.09	3.00	0.0608
3	1.16	0.41	2.26	3.00	0.0749
2	0.75	0.41	2.26	3.00	0.0862
1	0.34	0.34	1.87	3.60	0.0691

× 5.5 =

Results of Preliminary Analysis Including P-delta Effects

Story	Total Drift (in.)	Story Drift (in.)	Magnified Story Drift (in.)	Drift from θ (in.)	Drift Limit (in.)
6	2.23	0.23	1.27	1.24	3.00
5	2.00	0.34	1.87	1.84	3.00
4	1.66	0.40	2.20	2.23	3.00
3	1.26	0.45	2.48	2.44	3.00
2	0.81	0.45	2.48	2.47	3.00
1	0.36	0.36	1.98	2.01	3.60

× 5.5 =

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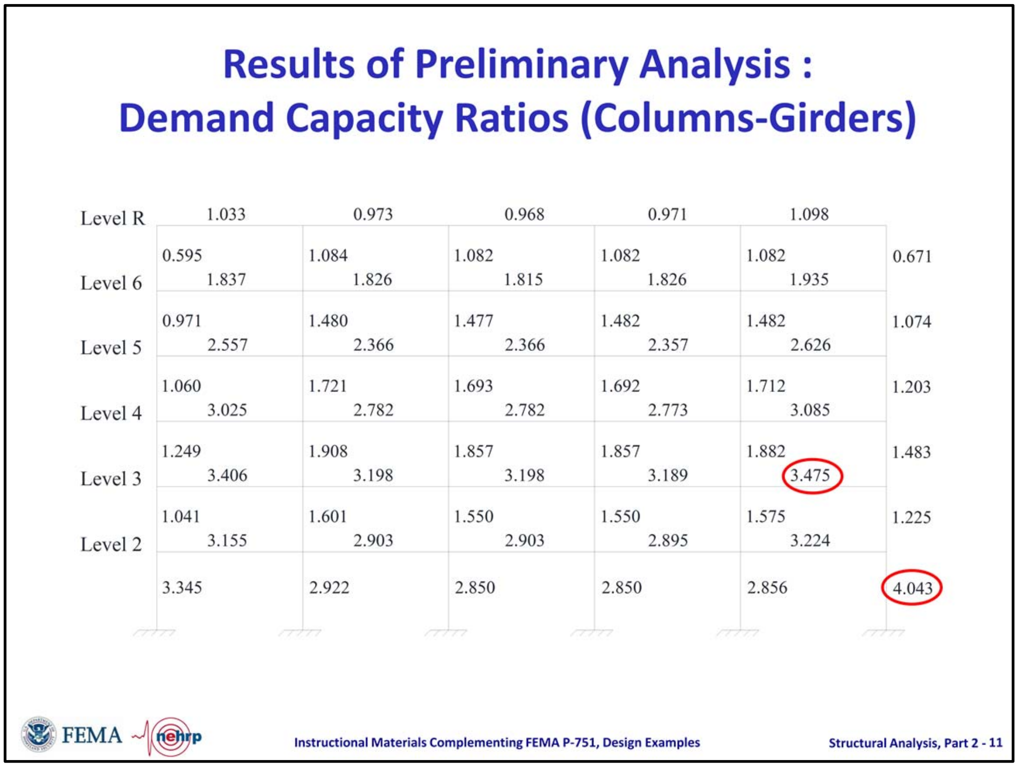
The results of the preliminary analysis for drift are shown in Tables for the computations excluding and including P-delta effects, respectively. In each table, the deflection amplification factor (C_d) equals 5.5, and the acceptable story drift (story drift limit) is taken as 2% of the story height which is the limit provided by *Standard* Table 12.12-1. As explained before, a new set of lateral loads based on the computed period of the actual structure were found and applied to the structure to calculate the elastic drifts.

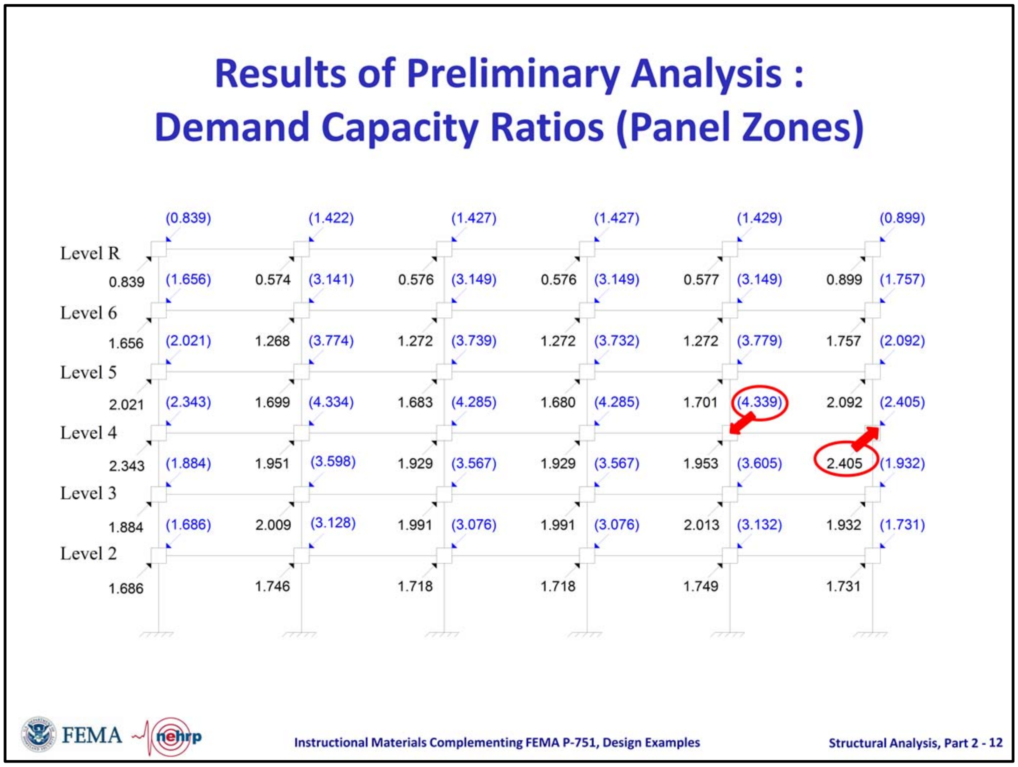
When P-delta effects are included, the drifts can also be estimated as the drifts without P-delta times the quantity 1/(1-θ), where θ is the stability coefficient for the story. As can be seen in bottom Table, back calculated drift values from θ are fairly consistent with the real results obtained by running the analyses with P-delta effects. The difference is always less than 2%.



For DCR analysis, the structure is subjected to full dead load plus 0.5 times the fully reduced live load, followed by equivalent lateral forces found without R factor. Equivalent lateral forces are applied towards right in the analyses. P-delta effects are included.

Since the DCRs in the Figure are found from preliminary analyses, in which the centerline model is used, doubler plates are not added into the model.

For girders, the DCR is simply the maximum moment in the member divided by the member's plastic moment capacity where the plastic capacity is Z_eF_{ye}. Z_e is the plastic section modulus at center of reduced beam section and F_{ye} is the expected yield strength. For columns, the ratio is similar except that the plastic flexural capacity is estimated to be $Z_{col}(F_{ye}\text{-}P_u/A_{col})$ where P_u is the total axial force in the column. The ratios are computed at the center of the reduced section for beams and at the face of the girder for columns.



The values in parentheses (in blue) represent the DCRs without doubler plates. The maximum DCR values with and doubler plates added are highlighted in the Figure.

Since the DCRs in Figure are found from preliminary analyses, in which the centerline model is used, doubler plates aren't added into the model. Thus, the demand values shown in the Figure are the same with and without doubler plates. However, since the capacity of the panel zone increases with added doubler plates, the DCRs decrease at the interior beam column joints as the doubler plates are used only at the interior joints. As may be seen in Figure, the DCR at the exterior joints are the same with and without doubler plates added.

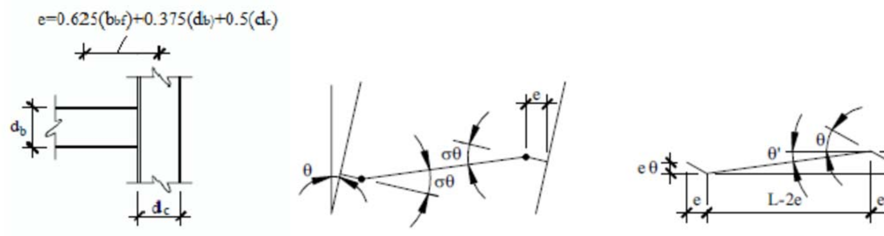
To find the shear demand at the panel zones, the total moment in the girders (at the left and right sides of the joint) is divided by the effective beam depth to produce the panel shear due to beam flange forces. Then the column shear at above or below the panel zone joint was subtracted from the beam flange shears, and the panel zone shear force is obtained. This force is divided by the shear strength capacity to determine the DCR of the panel zones.

Results of Preliminary Analysis : Demand Capacity Ratios

- The structure has considerable overstrength, particularly at the upper levels.
- The sequence of yielding will progress from the lower level girders to the upper level girders.
- With the possible exception of the first level, the girders should yield before the columns. While not shown in the Figure, it should be noted that the demand-to-capacity ratios for the lower story columns were controlled by the moment at the base of the column. The column on the leeward (right) side of the building will yield first because of the additional axial compressive force arising from the seismic effects.
- The maximum DCR of girders is 3.475, while maximum DCR for panel zones without doubler plates is 4.339. Thus, if doubler plates are not used, the first yield in the structure will be in the panel zones. However, with doubler plates added, the first yield is at the girders as the maximum DCR of the panel zones reduces to 2.405.

Note that although the maximum DCR for the columns (4.043) is greater than the maximum DCR for the beams (3.475), it is likely that the beam will yield earlier than the column. Column DCR gets bigger here because of the huge additional axial compressive force arising from the seismic load which was applied without R factor.

Results of Preliminary Analysis: Overall System Strength



(b) (c) (d)

Internal Work = External Work

$$\text{Internal Work} = 2[20\sigma\theta M_{PA} + 40\sigma\theta M_{PB} + \theta(M_{PC} + 4M_{PD} + M_{PE})]$$

$$\text{External Work} = V\theta \sum_{i=1}^{nLevels} F_i H_i \quad \text{where} \quad \sum_{i=1}^{nLevels} F_i = 1$$

The total lateral strength of the frame is calculated using virtual work.

In the analysis, it is assumed that plastic hinges are perfectly plastic. Girders hinge at a value Z_eF_{ye}, and the hinges form at the center of the reduced section (approximately 15 inches from the face of the column). Columns hinge only at the base, and the plastic moment capacity is assumed to be $Z_{col}(F_{ye}-P_u/A_{col})$.

Results of Preliminary Analysis: Overall System Strength

Lateral Strength on Basis of Rigid-Plastic Mechanism

Lateral Load Pattern	Lateral Strength (kips) Entire Structure	Lateral Strength (kips) Single Frame
Uniform	3,332	1,666
Upper Triangular	2,747	1,373
Standard	2,616	1,308

- As expected, the strength under uniform load is significantly greater than under triangular or *Standards* load.
- The closeness of the *Standards* and triangular load strengths is due to the fact that the vertical-load-distributing parameter (k) was 1.385, which is close to 1.0.
- Slightly more than 15 percent of the system strength comes from plastic hinges that form in the columns. If the strength of the column is taken simply as M_p (without the influence of axial force), the "error" in total strength is less than 2 percent.
- The rigid-plastic analysis did not include strain hardening, which is an additional source of overstrength.

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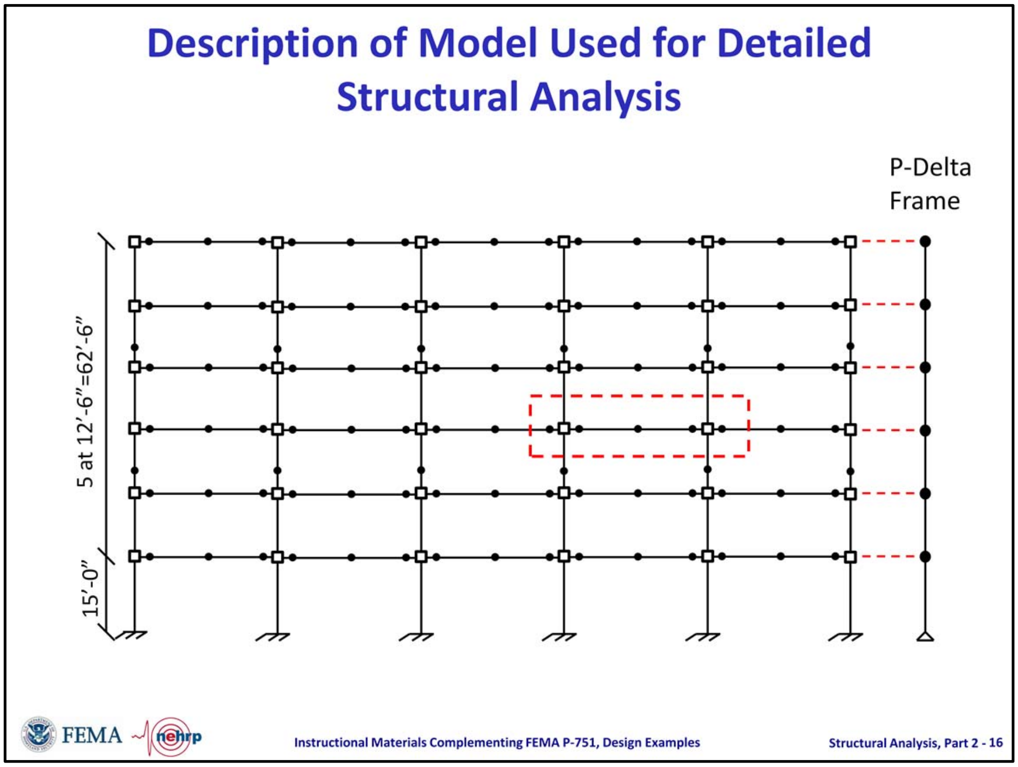
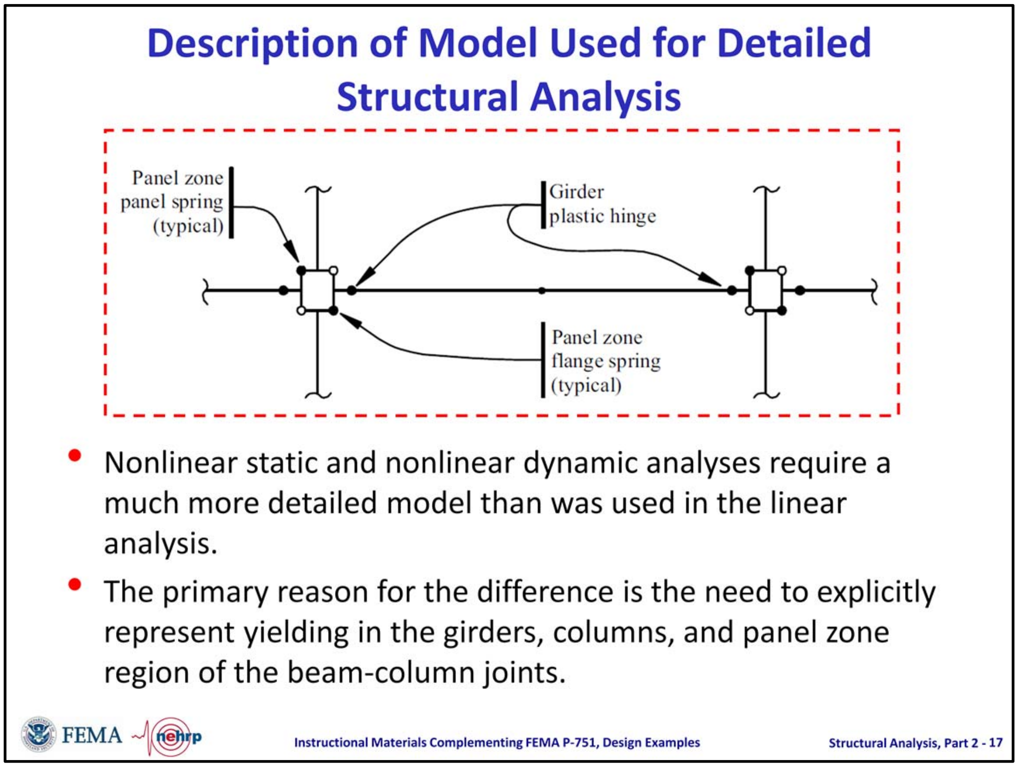
Three lateral force patterns are used: uniform, upper triangular, and *Standard* (where the *Standard* pattern is consistent with the vertical force distribution provided in Slide 7).

The rigid-plastic analysis does not consider the true behavior of the panel zone region of the beam-column joint. Yielding in this area can have a significant effect on system strength.

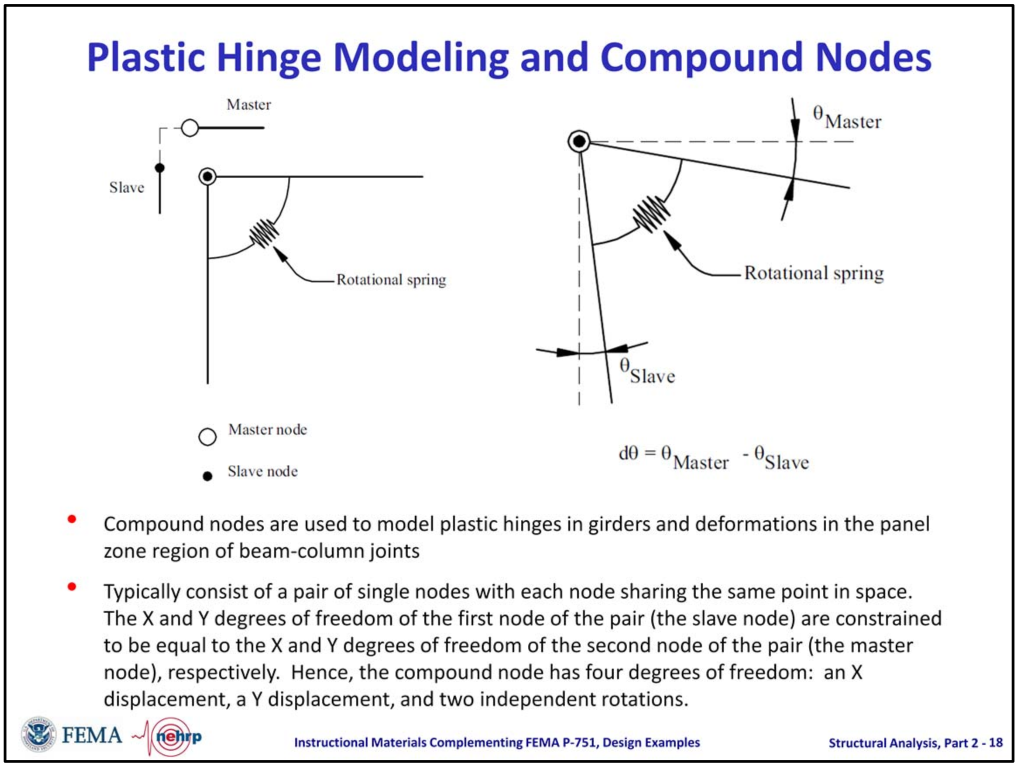


The DRAIN model used for the nonlinear analysis is shown in the Figure.

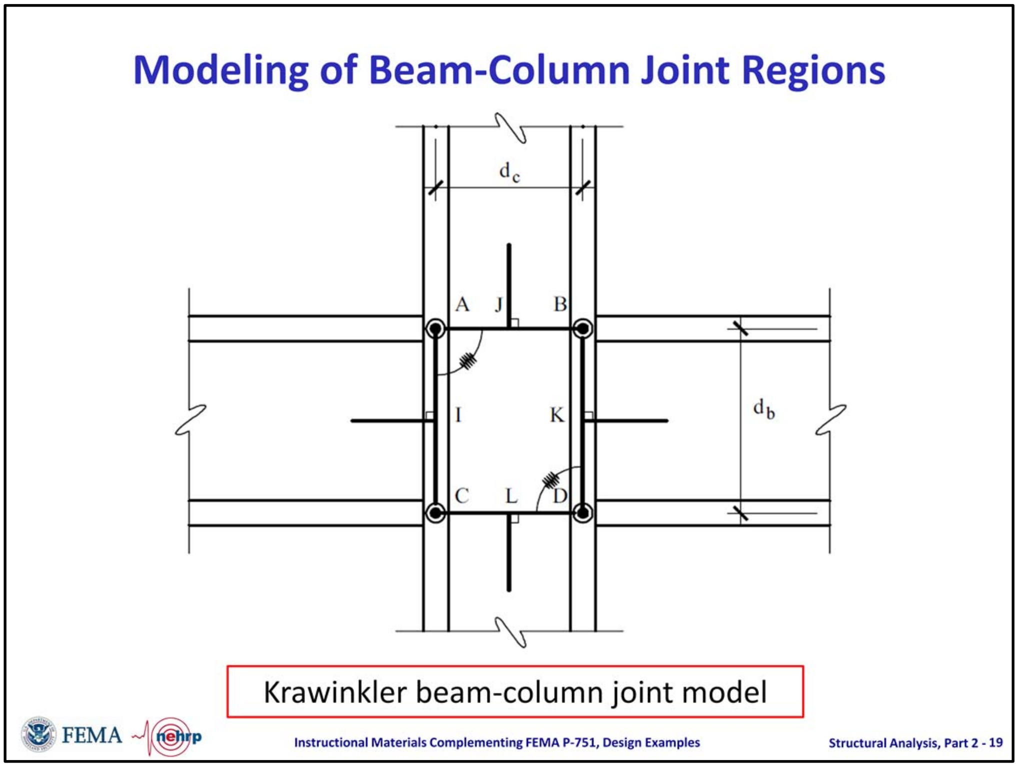
In detailed model, Krawinkler type panel zones are added to the model. Plastic hinges are assigned at the reduced flange sections. P-Delta effects are included by use of a linear column similar to preliminary model.



The detail illustrates the two main features of the model: an explicit representation of the panel zone region and the use of concentrated plastic hinges in the girders. Connection elements (Type 4) are used for both girder plastic hinges and panel zone panel and flange springs.



In most cases, one or more rotational spring connection elements (DRAIN element Type 4) are placed between the two single nodes of the compound node, and these springs develop bending moment in resistance to the relative rotation between the two single nodes. If no spring elements are placed between the two single nodes, the compound node acts as a moment-free hinge.



Krawinkler model represents the panel zone stiffness and strength by an assemblage of four rigid links and two rotational springs. The links form the boundary of the panel, and the springs are used to provide the desired inelastic behavior.

Modeling of Beam-Column Joint Regions

Krawinkler model assumes that the panel zone has two resistance mechanisms acting in parallel:

1. Shear resistance of the web of the column, including doubler plates and
2. Flexural resistance of the flanges of the column.

$$R_v = 0.6F_y d_c t_p + 1.8\frac{F_y b_{cf} t_{cf}^2}{d_b} = V_{Panel} + 1.8V_{Flanges}$$

- F_y = yield strength of the column and the doubler plate,
- d_c = total depth of column,
- t_p = thickness of panel zone region = column web + doubler plate thickness,
- b_{cf} = width of column flange,
- t_{cf} = thickness of column flange, and
- d_b = total depth of girder.

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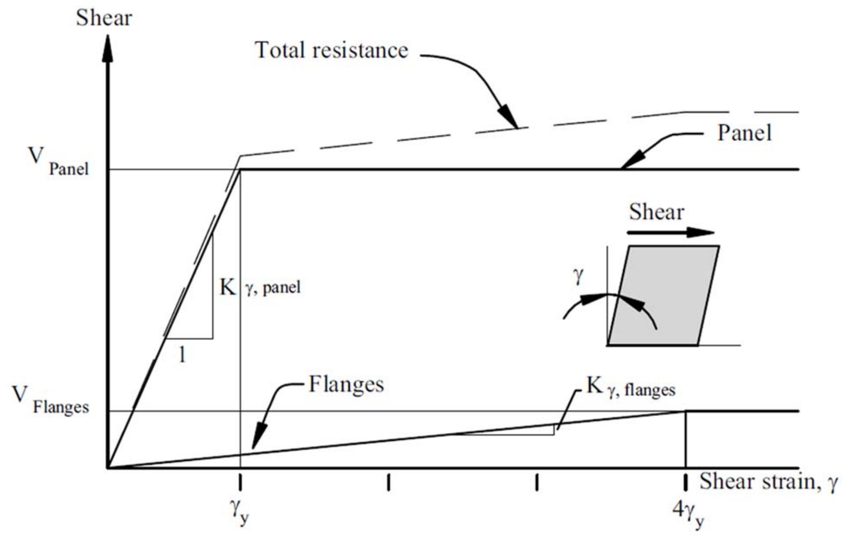
The Krawinkler model assumes that the panel zone has two resistance mechanisms acting in parallel:

1. Shear resistance of the web of the column, including doubler plates

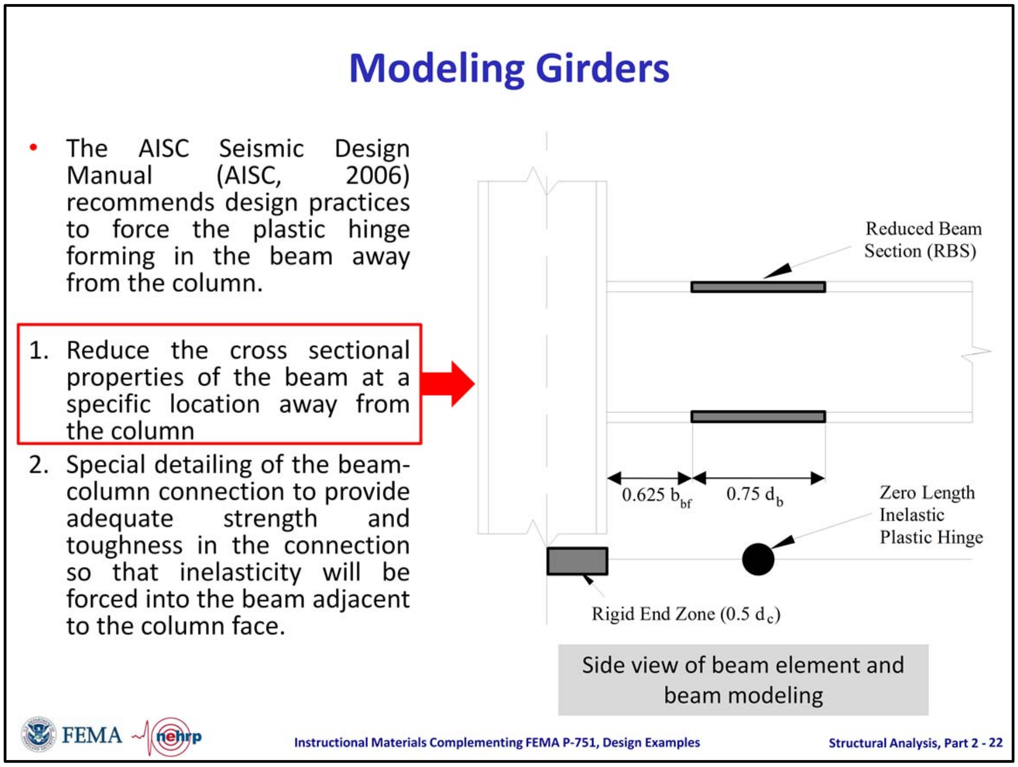
2. Flexural resistance of the flanges of the column

Modeling of Beam-Column Joint Regions

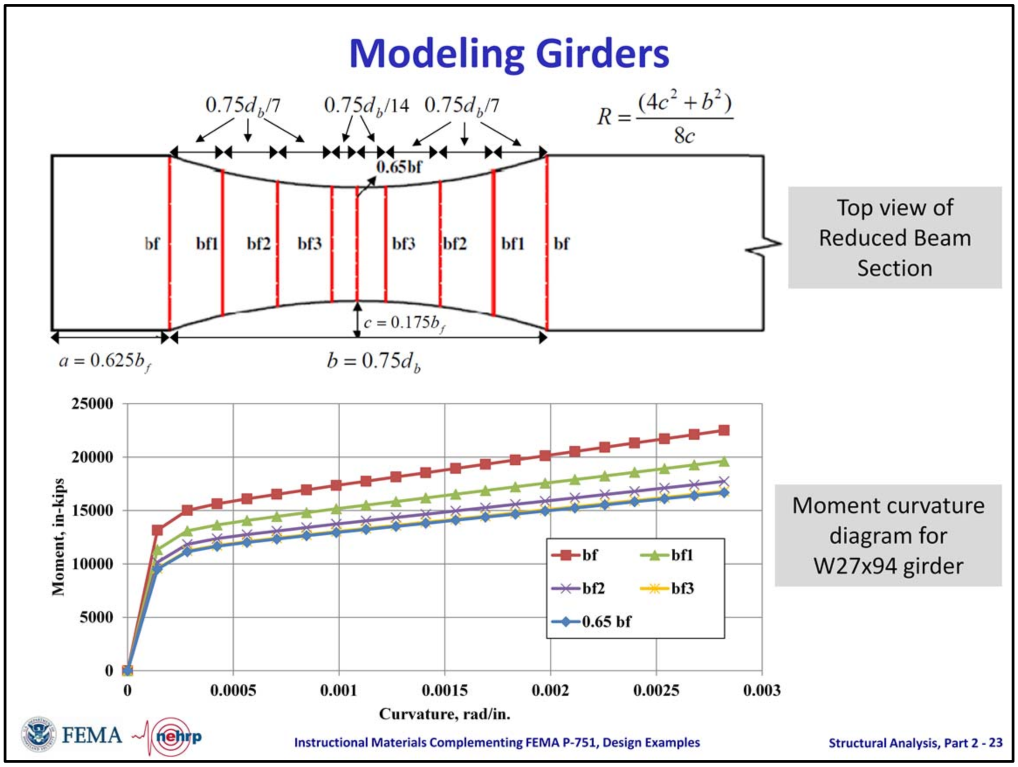


Force-deformation behavior of panel zone region (Krawinkler Model)

The complete resistance mechanism, in terms of rotational spring properties, is shown in Figure. This trilinear behavior is represented by two elastic-perfectly plastic springs at the opposing corners of the joint assemblage.



A side view of the reduced beam sections is shown in Figure. The distance between the column face and the edge of the reduced beam section was chosen as $a = 0.625b_{bf}$ and the reduced section length was assumed as $b = 0.75d_b$. Both of these values are just at the middle of the limits stated in AISC 358. Plastic hinges of the beams are modeled at the center of the reduced section length.



To determine the plastic hinge capacities, a moment-curvature analysis of the cross section, which is dependent on the stress-strain curve of the steel used in girders, was implemented.

Figure demonstrates the moment-curvature graph for the W27x94 girder. As may be seen in the figure, the moment-curvature relationship is different at each section of the reduced length. The locations of the different reduced beam sections used in Figure 1, named as “bf1”, “bf2”, and “bf3”, can be seen in Figure 2. Note that because of closely adjacent locations chosen for “0.65bf” and “bf3” (See Figure 1), their moment-curvature plots are nearly indistinguishable from other in Figure 2.

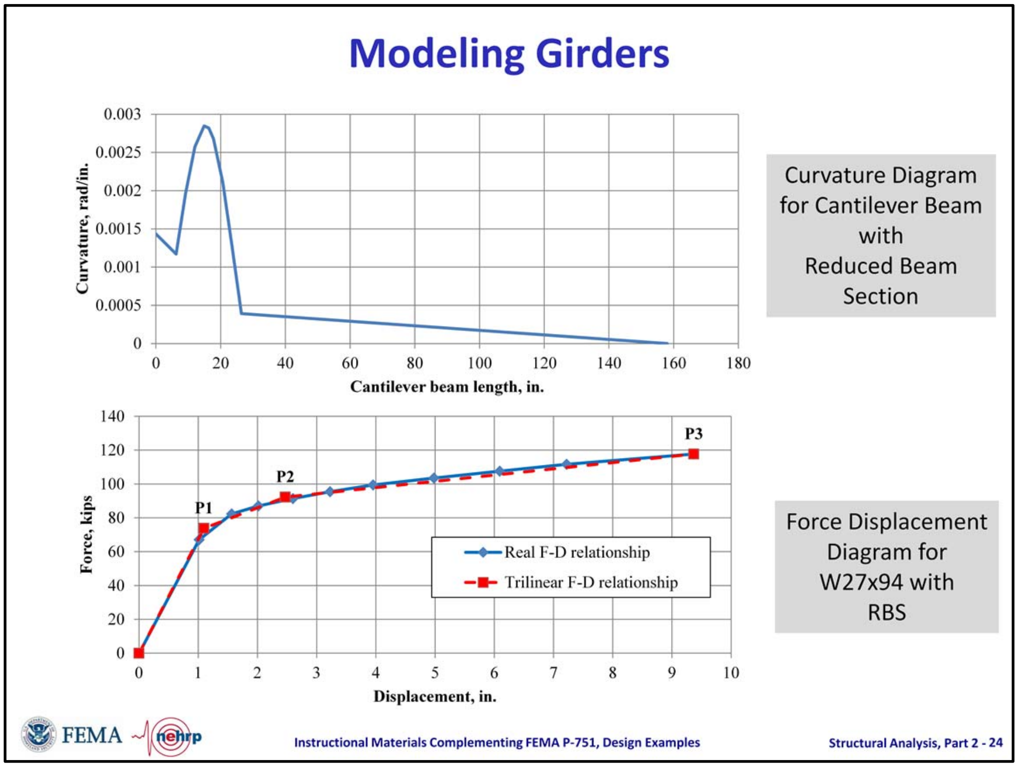
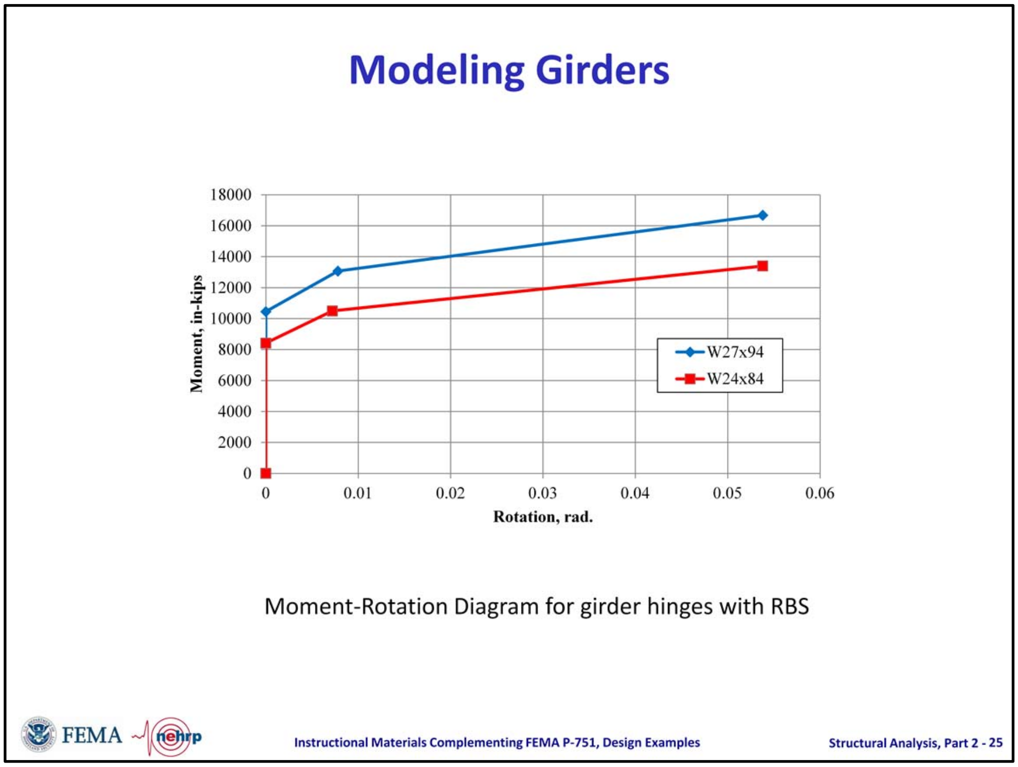


Figure 1 shows the curvature diagram when the curvature ductility reaches 20. The curvature difference (bump at the center of RBS in Figure) section is less prominent when the ductility is smaller.

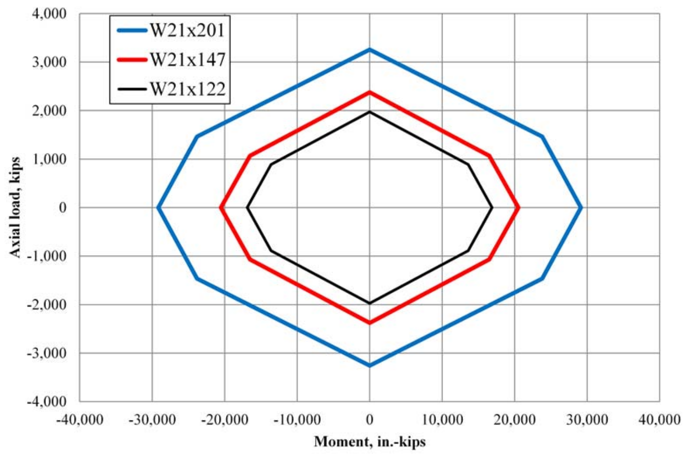
Given the curvature distribution along cantilever beam length, the deflections at the point of load (tip deflections) can be found by using the moment area method. Figure 2 illustrates the force- displacement relationship at the end of the ½ span cantilever for the W27x94 with the reduced flange section.



To convert the force-tip displacement diagram into moment-rotation of the plastic hinge, the following procedure is followed.

1. Using the trilinear force displacement relationship shown in previous slide (Figure 2) , find the moment at the plastic hinge for P1, P2 and P3 load levels and call them as M1, M2 and M3. To find the moments, the tip forces (P1, P2 and P3) were multiplied with the difference of the ½ span cantilever length and the plastic hinge distance from the column face.
2. Calculate the change in moment for each added load (For ex: dM1= M2-M1).
3. Find the flexural rigidity (EI) of the beam given tip displacement of 1 in. under the 1st load (P1 in Figure 2 of previous slide).
4. Calculate the required rotational stiffnesses of the hinge between M1 and M2, and then M2 and M3.
5. Calculate the change in rotation from M1 to M2, and from M2 to M3 by dividing the change in moment found at Step 2 by the required rotational stiffness values calculated at Step 4.
6. Find the specific rotations at M1, M2 and M3 using the change in rotation values found in step 5. Note that the rotation is zero at M1.
7. Plot moment-rotation diagram of the plastic hinge using the values calculated at Step1 and Step6.

Modeling Columns



Yield surface used for modeling columns

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All columns in the analysis were modeled in DRAIN with Type-2 elements.

Preliminary analysis indicated that columns should not yield, except at the base of the first story. Subsequent analysis showed that the columns will yield in the upper portion of the structure as well. For this reason, column yielding had to be activated in all of the Type-2 column elements. The columns were modeled using the built-in yielding functionality of the DRAIN program, wherein the yield moment is a function of the axial force in the column. The yield surfaces used by DRAIN for all the columns in the model are shown in Figure.

Results of Detailed Analysis: Period of Vibration

Periods of Vibration From Detailed Analysis (sec/cycle)

Model	Mode	P-delta Excluded	P-delta Included
Strong Panel	1	1.912	1.973
with	2	0.627	0.639
doubler plates	3	0.334	0.339
Weak Panel	1	2.000	2.069
without	2	0.654	0.668
doubler plates	3	0.344	0.349

- P-delta effects increases the period.
- Doubler plates decreases the period as the model becomes stiffer with doubler plates.
- Different period values were obtained from preliminary and detailed analyses.
- Detailed model results in a stiffer structure than the preliminary model especially when doubler plates are added.

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Slide shows vibration of periods of vibration using different analysis assumptions.

Static Pushover Analysis

- Pushover analysis procedure performed in this example follows the recommendations of ASCE/SEI 41-06.
- Pushover analysis should *always* be used as a precursor to nonlinear response history analysis.
- The structure is subjected to the full dead load plus 50 percent of the fully reduced live load, followed by the lateral loads.
- For the entire pushover analyses reported for this example, the structure is pushed to 37.5 in. at the roof level. This value is about two times the total drift limit for the structure where the total drift limit is taken as 2 percent of the total height.
- The effect of lateral load distribution, strong and weak panel zones (doubler plates) and P-delta are investigated separately in this example.

Slide is self-explanatory. Describes procedure for nonlinear static pushover analysis.

Static Pushover Analysis

Effect of Different Lateral Load Distribution

In this example, three different load patterns were initially considered:

UL = Uniform load (equal force at each level)
ML = Modal load (lateral loads proportional to first mode shape)
BL = *Provisions* load distribution (Equivalent lateral forces used for preliminary analysis)

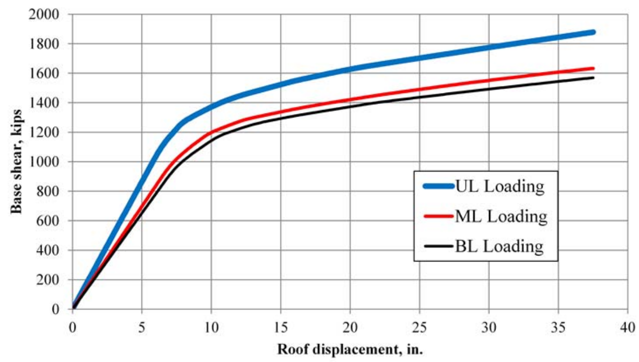
Lateral Load Patterns Used in Nonlinear Static Pushover Analysis

Level	Uniform Load UL (kips)	Modal Load ML (kips)	*Provisions* Load BL (kips)
R	15.0	85.1	144.8
6	15.0	77.3	114.0
5	15.0	64.8	84.8
4	15.0	49.5	58.2
3	15.0	32.2	34.6
2	15.0	15.0	15.0

Relative values of these load patterns are summarized in Table. The loads have been normalized to a value of 15 kips at Level 2.

Static Pushover Analysis

Effect of Different Lateral Load Distribution



Response of strong panel model to three load patterns, excluding P-delta effects

The *Provisions* states that the lateral load pattern should follow the shape of the first mode. (ML Loading)

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Figure shows the pushover response of the SP structure to all three lateral load patterns where P-delta effects are excluded. In each case, gravity loads are applied first and then the lateral loads are applied using the displacement control algorithm.

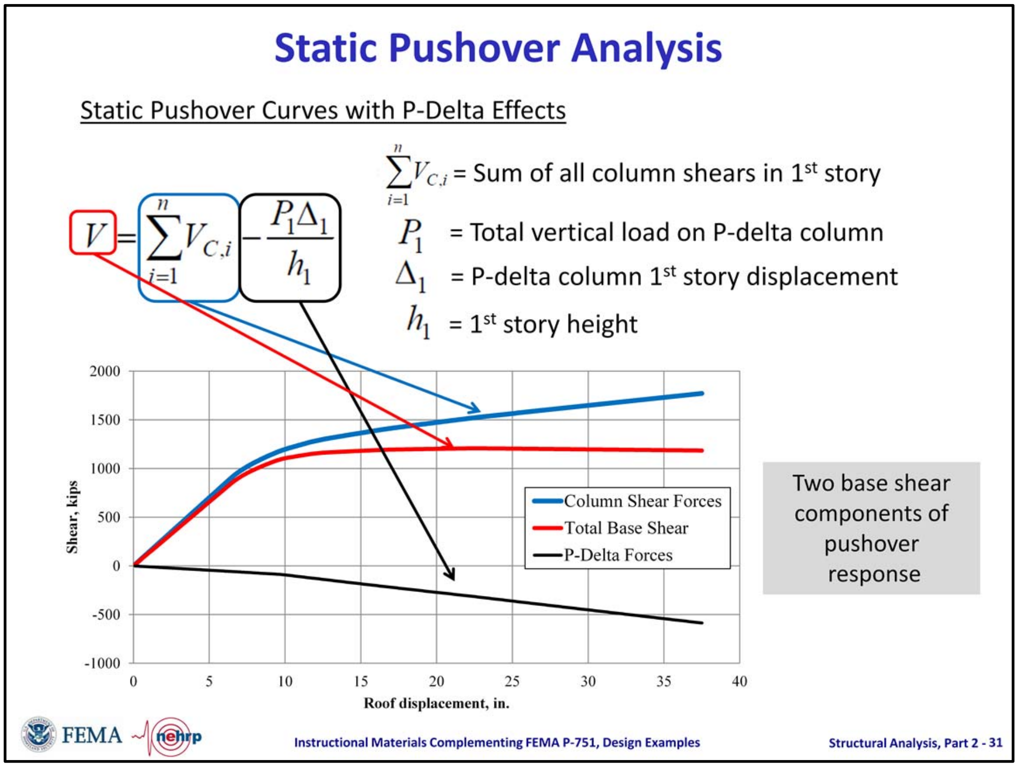
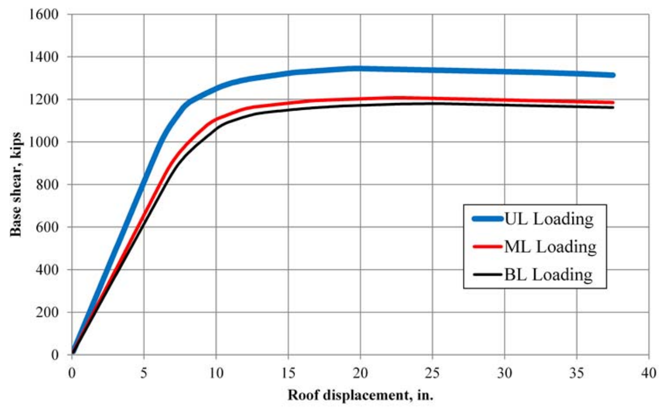


Figure plots two base shear components of the pushover response for the SP structure subjected to the ML loading.

The kink in the line representing P-delta forces occurs because these forces are based on first-story displacement, which, for an inelastic system, generally will not be proportional to the roof displacement.

Static Pushover Analysis

Effect of Different Lateral Load Distribution



Response of strong panel model to three load patterns, including P-delta effects

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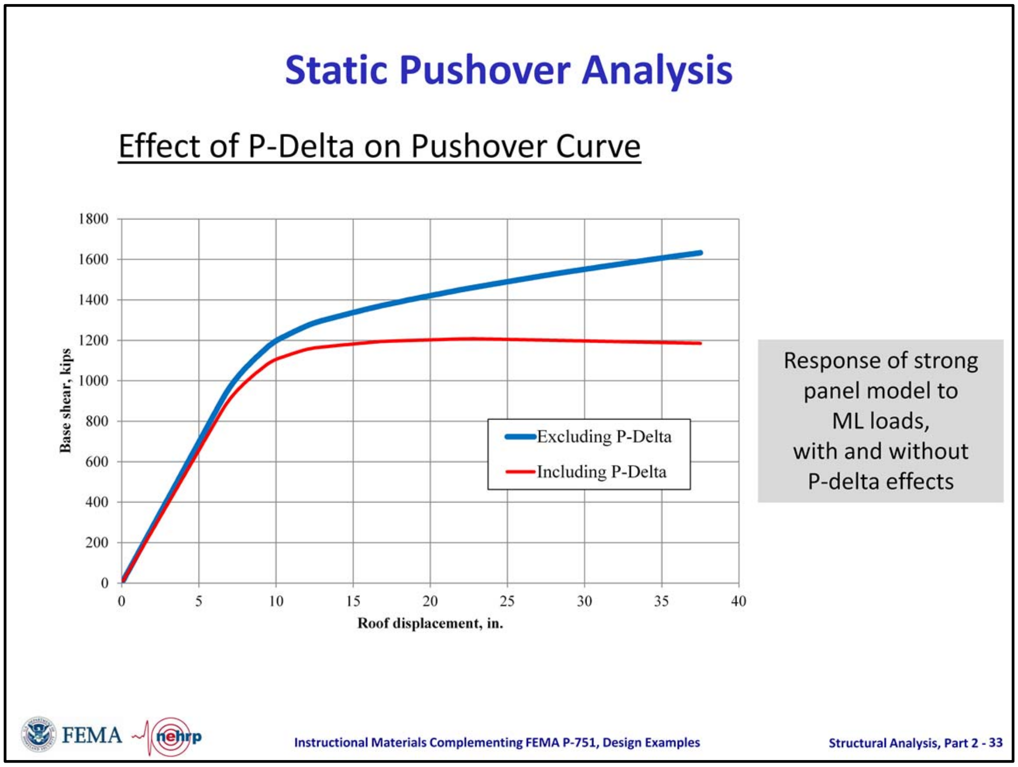
Figure shows the pushover response of the SP structure to all three lateral load patterns where P-delta effects are included.

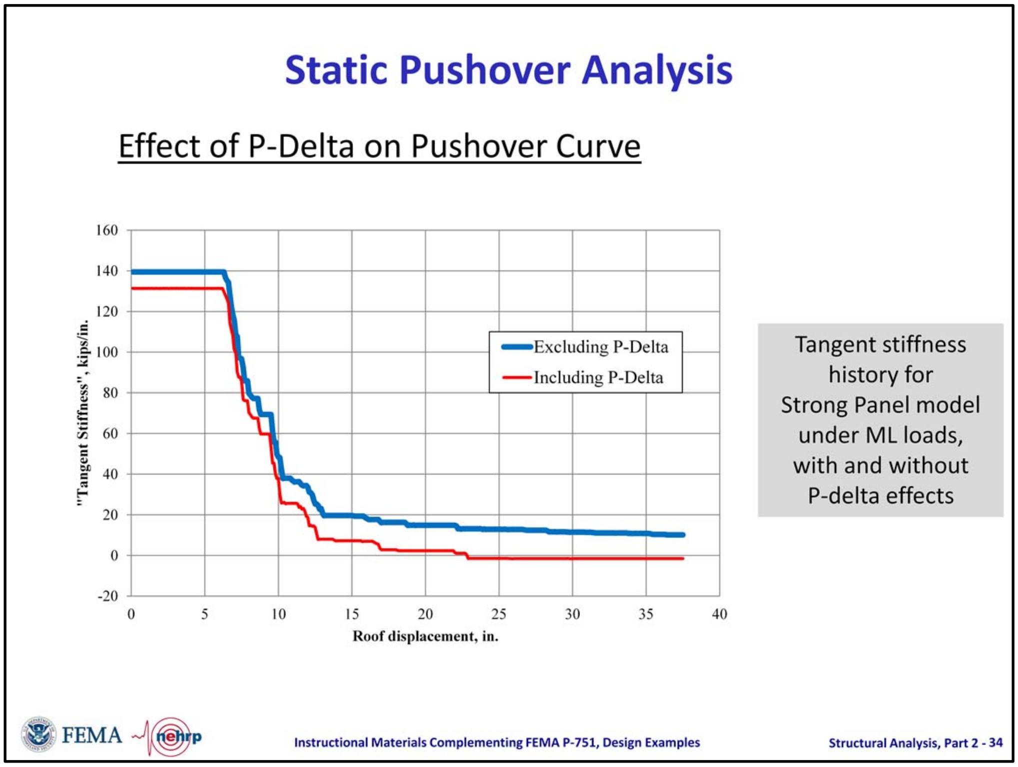


The response of the structure under ML loading with and without P-delta effects is illustrated in Figure.

Clearly, P-delta effects are an extremely important aspect of the response of this structure, and the influence grows in significance after yielding. This is particularly interesting in the light of the *Standard*, which ignores P-delta effects in elastic analysis if the maximum stability ratio is less than 0.10 (see Sec. 12.8-7). For this structure, the maximum computed stability ratio is 0.0862 (see Slide 10), which is less than 0.10 and is also less than the upper limit of 0.0909. The upper limit is computed according to *Standard* Equation 12.8-17 and is based on the very conservative assumption that $\beta = 1.0$.

While the *Standard* allows the analyst to exclude P-delta effects in an elastic analysis, this clearly should not be done in the pushover analysis (or in response history analysis).

In the *Provisions* the upper limit for the stability ratio is eliminated. Where the calculated θ is greater than 0.10, a pushover analysis must be performed in accordance with ASCE 41, and it must be shown that that the slope of the pushover curve is positive up to the target displacement. The pushover analysis must be based on the MCE spectral acceleration and must include P-delta effects [and loss of strength, as appropriate]. If the slope of the pushover curve is negative at displacements less than the target displacement, the structure must be redesigned such that θ is less than 0.10 or the pushover slope is positive up to the target displacement.



The first significant yield occurs at a roof displacement of approximately 6.5 inches and that most of the structure's original stiffness is exhausted by the time the roof displacement reaches 13 inches.

For the case with P-delta effects excluded, the final stiffness shown in Figure is approximately 10.2 kips/in., compared to an original value of 139 kips/in. Hence, the strain-hardening stiffness of the structure is 0.073 times the initial stiffness. This is somewhat greater than the 0.03 (3.0 percent) strain hardening ratio used in the development of the model because the entire structure does not yield simultaneously.

Where P-delta effects are included, the final stiffness is -1.6 kips per in. The structure attains this negative residual stiffness at a displacement of approximately 23 in.

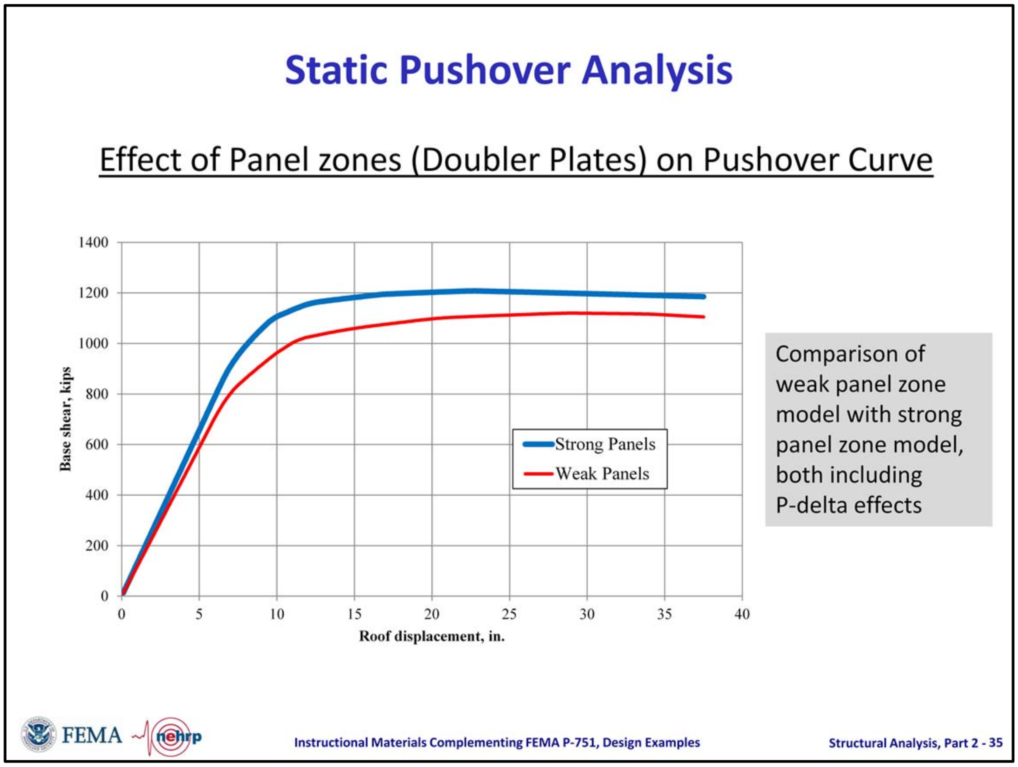
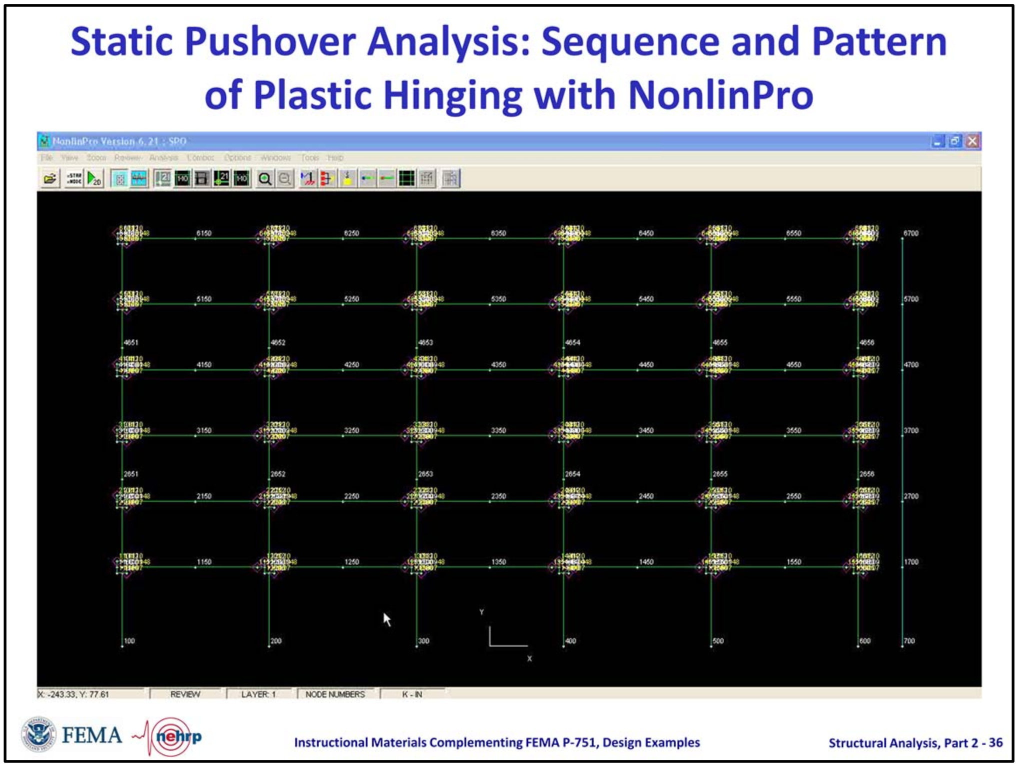
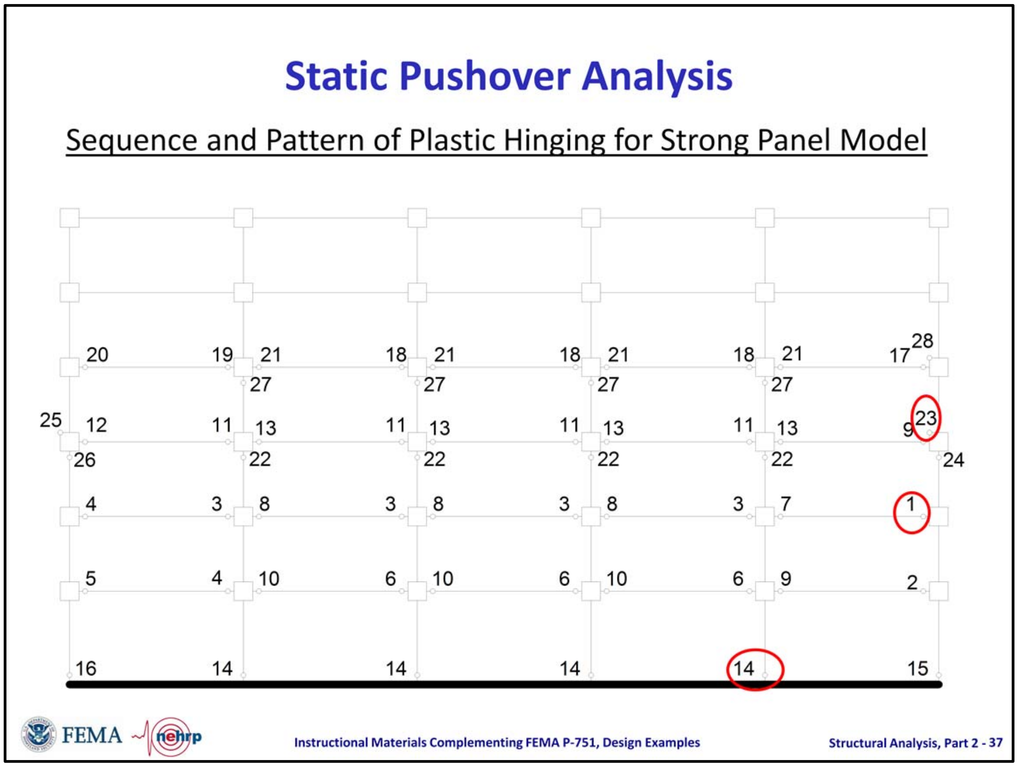


Figure shows that the doubler plates, which represent approximately 2.0 percent of the volume of the structure, increase the strength and initial stiffness by approximately 10 percent.



This slide shows a movie which is obtained using the snapshot tool of NonlinPro. Yielded displaced shape showing sequence and pattern of plastic hinging is displayed.

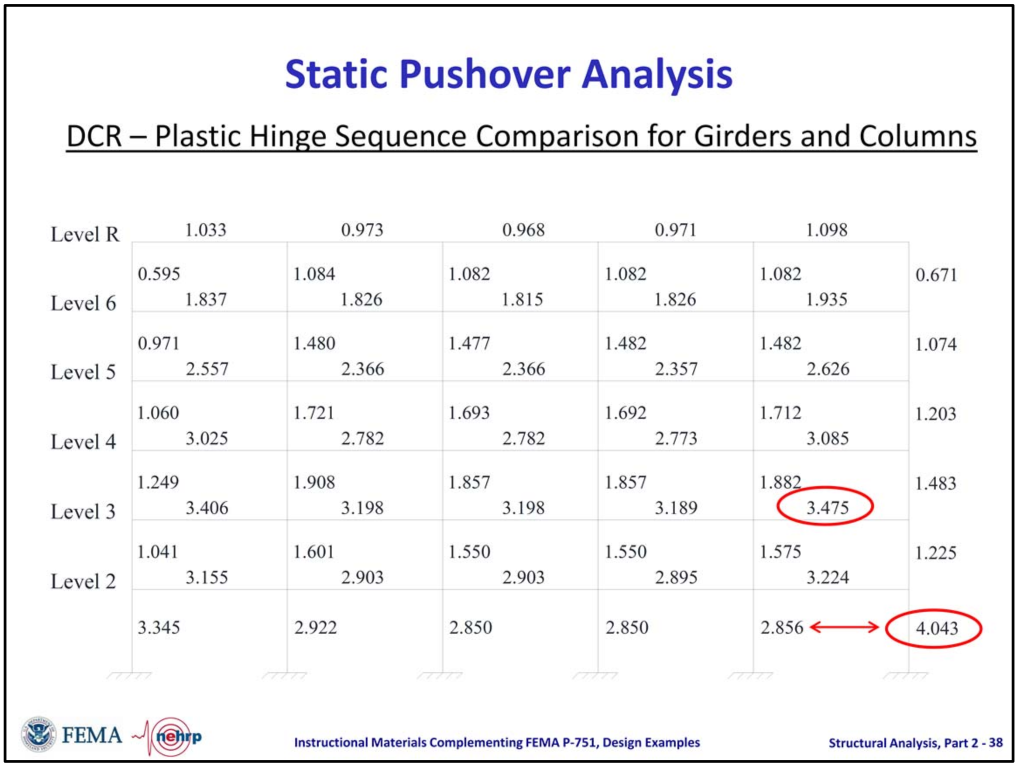


It appears that the structure is somewhat weak in the middle two stories and is relatively strong at the upper stories. The doubler plates added to the interior columns prevented panel zone yielding.

Figure shows the first yielding locations of the girder, column and panel zones.

<u>Some observations:</u>

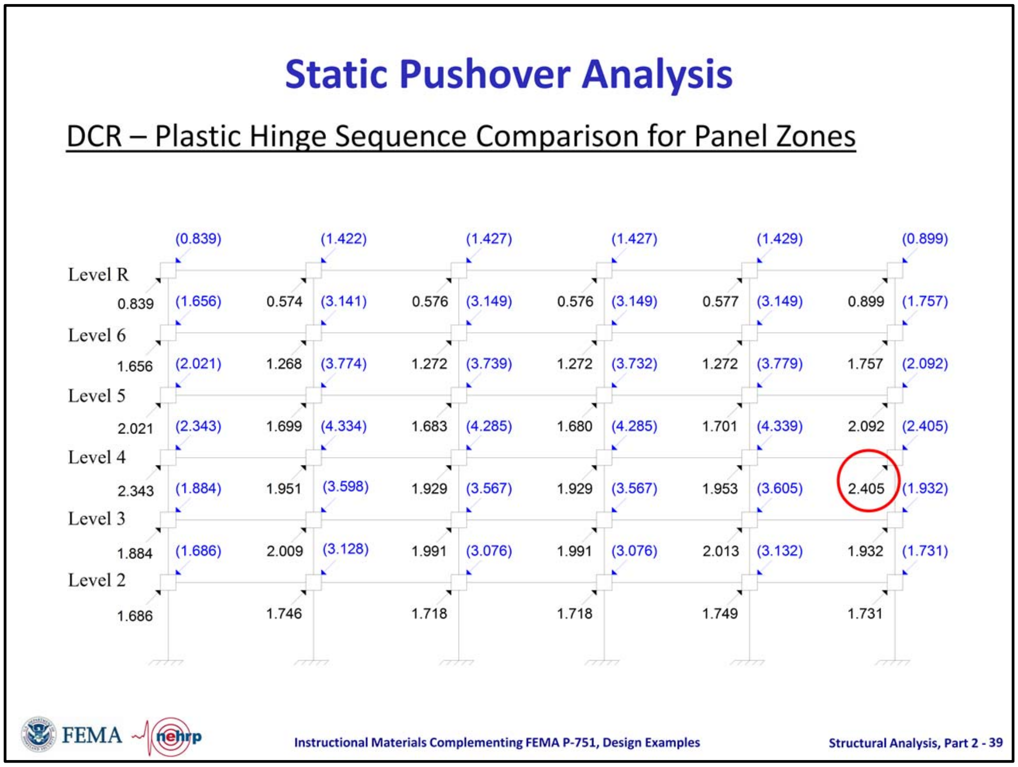
- There is no hinging in Levels 6 and R.
- There is panel zone hinging only at the exterior columns at Levels 4 and 5. Panel zone hinges do not form at the interior joints where doubler plates are used.
- Hinges form at the base of all the Level 1 columns.
- Plastic hinges form in all columns on Level 3 and all the interior columns on Level 4.



The demand capacity ratios match the plastic hinge formation sequence, i.e. first plastic hinges form at the maximum DCR's for columns, girders and panel zones.

The highest DCR was observed at the girders of 3rd level beginning from the bays at the leeward (right) side. As may be seen, first plastic hinges form at the same locations of the building.

As may be seen in the previous slide the first column hinge forms at the base of the fifth column. However, the DCR of the sixth column (leeward side) is the maximum. This is due to huge axial compressive forces that reduce the capacity of the leeward side column when DCR is calculated. Note that if R=8 is used for the lateral load of DCR analysis, the base of the fifth column results in the maximum DCR which would match better with the hinging sequence of the pushover analysis. In addition, as seen in the Figure of Slide 37, base column hinges form almost simultaneously.



First panel zone hinge forms at the beam column joint of the sixth column at the fourth level (see Slide 37), and this is where the highest DCR values were obtained for the panel zones in preliminary DCR analyses.

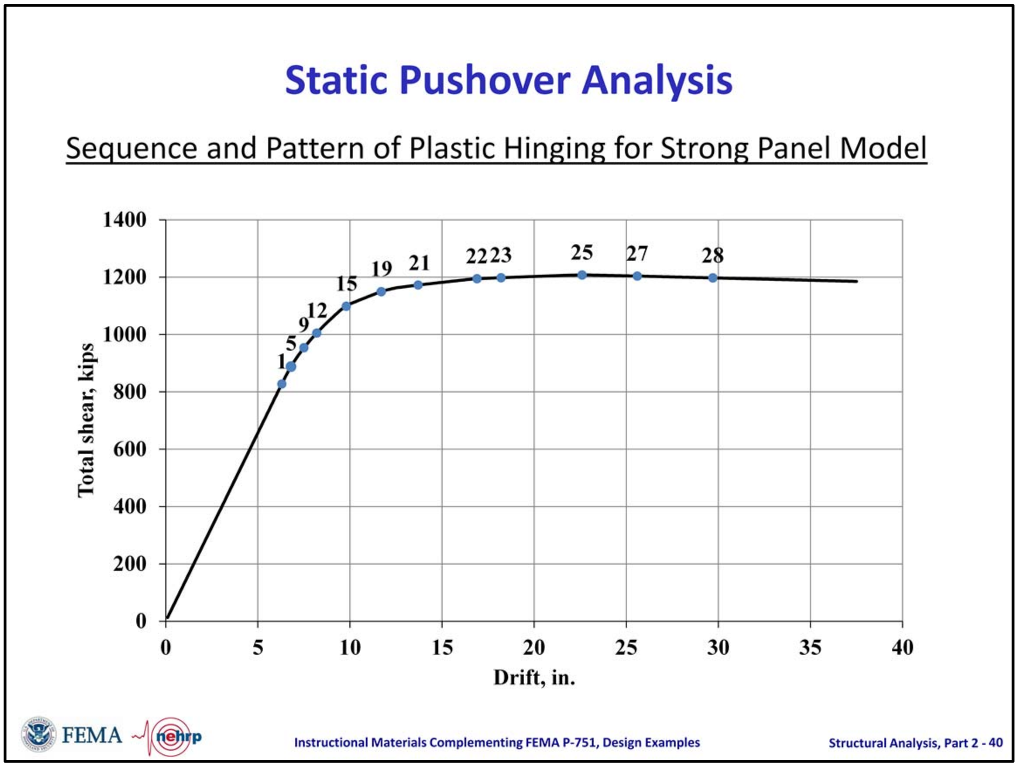
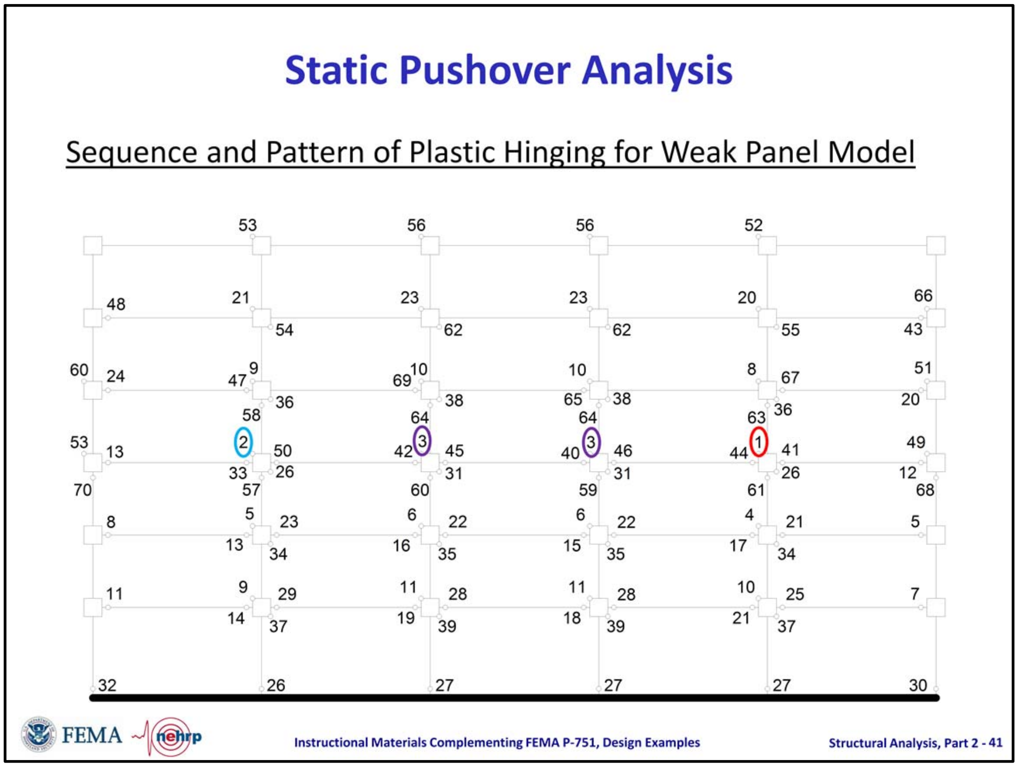


Diagram shows sequencing of plastic hinge formation on a pushover curve.

Figure shows the sequence of the hinging on the pushover curve. These events correspond to numbers shown in Figure of Slide 37. The pushover curve only shows selected events because an illustration showing all events would be difficult to read.



As may be seen in Figure, first yielding occurs in the panel zones when doubler plates are not used. Panel hinges of Level 4 form first.

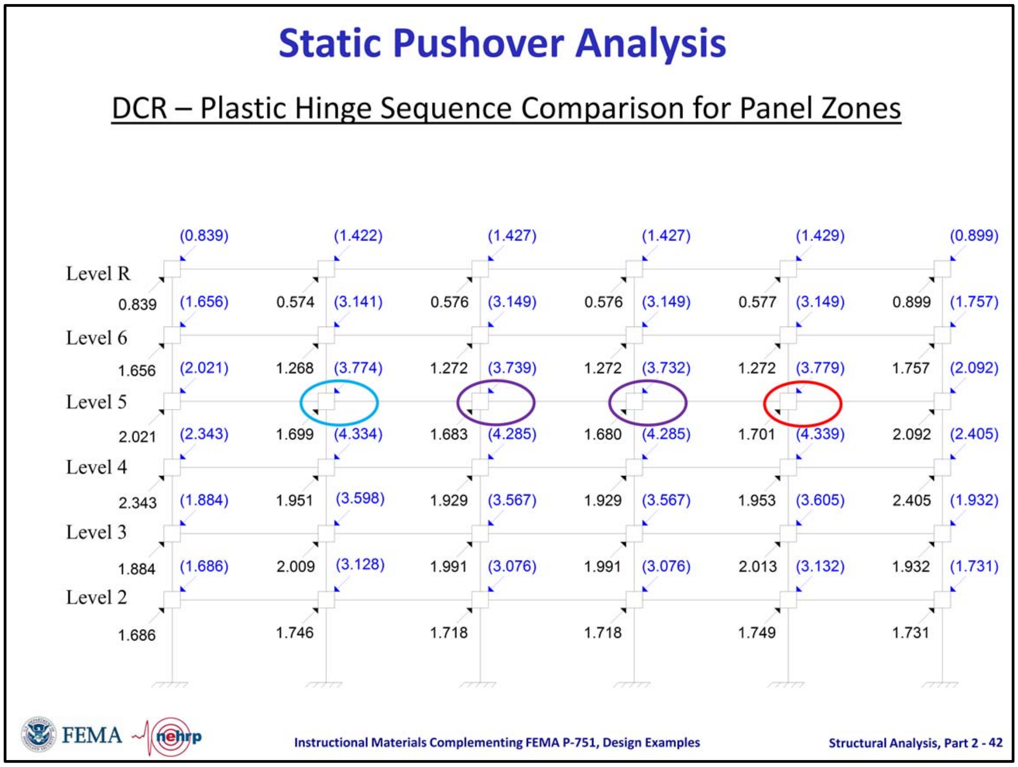


Figure shows the same plot displayed in Slide 12 (DCR of panel zones by preliminary analysis). The values in parentheses (in blue) represent the DCRs without doubler plates.

As may be seen in Figure, the hinges of the panels, where highest DCR are obtained from preliminary analyses, form first (Compare Figure with the Figure in the previous slide).

Static Pushover Analysis

Target Displacement

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$$

$C_0 = \phi_{1,r}\Gamma_1 =$ modification factor to relate spectral displacement of an equal single degree of freedom system to the roof displacement of the building multi-degree of freedom system.

$\phi_{1,r} =$ the ordinate of mode shape 1 at the roof (control node)

$\Gamma_1 =$ the first mode participation factor

$C_1 =$ modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

$C_2 =$ modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation and strength deterioration on maximum displacement response.

$S_a =$ response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration.

$T_e = T_i\sqrt{\frac{K_i}{K_e}} =$ effective fundamental period of the building in the direction under consideration

$T_i =$ elastic fundamental period in the direction under consideration calculated by elastic dynamic analysis.

$K_i, K_e =$ elastic, and effective lateral stiffness of the building in the direction under consideration.

$g =$ acceleration of gravity

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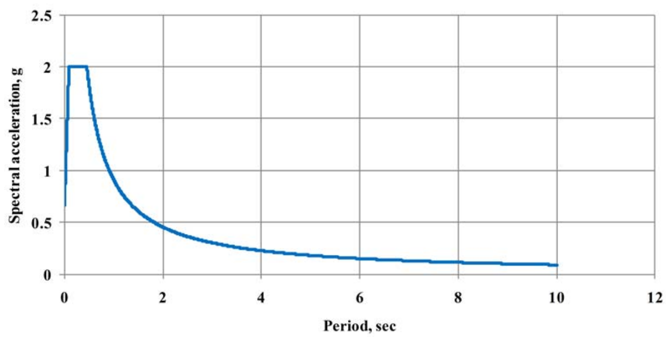
The formula is from section 3.3.3.3.2 of ASCE 41 which uses the coefficient method for calculating target displacement.

Static Pushover Analysis

Target Displacement



2% damped horizontal response spectrum from ASCE 41-06

This spectrum is for BSE-2 (Basic Safety Earthquake 2) hazard level which has a 2% probability of exceedence in 50 years.

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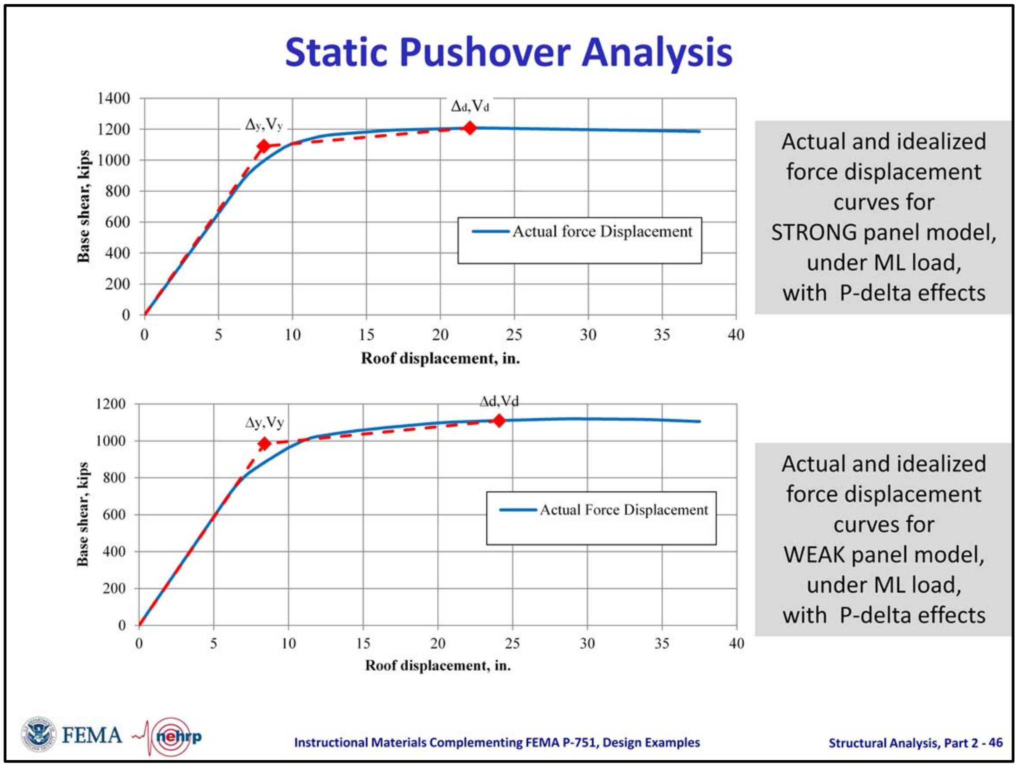
Spectral acceleration at the fundamental period of the structure was found from the 2% damped horizontal response spectrum as described in Section 1.6.1.5 of
ASCE 41-06.

Static Pushover Analysis

Target Displacement

- Nonlinear force-displacement relationship between base shear and displacement of control node shall be replaced with an idealized force-displacement curve. The effective lateral stiffness and the effective period depend on the idealized force-displacement curve.
- The idealized force-displacement curve is developed by using an iterative graphical procedure where the areas below the actual and idealized curves are approximately balanced up to a displacement value of Δ_d. Δ_d is the displacement at the end of second line segment of the idealized curve and V_d is the base shear at the same displacement.
- (Δ_d, V_d) should be a point on the actual force displacement curve at either the calculated target displacement, or at the displacement corresponding to the maximum base shear, whichever is the least.
- The first line segment of the idealized force-displacement curve should begin at the origin and finish at (Δ_y, V_y), where V_y is the effective yield strength and Δ_y is the yield displacement of idealized curve.
- The slope of the 1st line segment is equal to the effective lateral stiffness K_e which should be taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength of the structure.

Slide explains static pushover analysis.



Target displacement is 22.9 in. for Strong Panel model and 24.1 in. for Weak Panel model.

Negative tangent stiffness starts at 22.9 inches and 29.3 inches for strong and weak panel models, respectively. Thus negative tangent stiffness starts after target displacements for both models.

Static Pushover Analysis

Target displacement for strong and weak panel models

	Strong Panel	Weak Panel
C_0	1.303	1.310
C_1	1.000	1.000
C_2	1.000	1.000
S_a (g)	0.461	0.439
T_e (sec)	1.973	2.069
δ_t (in.) at Roof Level	**22.9**	**24.1**
Drift R-6 (in.)	0.96	1.46
Drift 6-5 (in.)	1.76	2.59
Drift 5-4 (in.)	2.87	3.73
Drift 4-3 (in.)	4.84	4.84
Drift 3-2 (in.)	5.74	5.35
Drift 2-1 (in.)	6.73	6.12

- Story drifts are also shown at the load level of target displacement.
- Negative stiffness starts after target displacements for both models.

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Slide describes Target Displacements.

Response History Analysis

Modeling and Analysis Procedure

- Response response history analysis method is used to estimate the inelastic deformation demands for the detailed structure.
- Three ground motions were used. (Seven or more ground motions is generally preferable.)
- The analysis considered a number of parameters, as follows:
  - Scaling of ground motions to the DBE and MCE level
  - With and without P-delta effects
  - Two percent and five percent inherent damping
  - Added linear viscous damping
- Identical structural model used in Nonlinear Pushover Analyses and 2nd order effects were included through the use of leaning column.
- All of the model analyzed had “Strong Panels” (wherein doubler plated were included in the interior beam-column joints).

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The structure is subjected to dead load and half of the fully reduced live load, followed by ground acceleration. The incremental differential equations of motion are solved in a step-by-step manner using the Newmark constant average acceleration approach. Time steps and other integration parameters are carefully controlled to minimize errors. The minimum time step used for analysis is as small as 0.0005 second for the first earthquake and 0.001 second for the second and third earthquakes. A smaller integration time step is required for the first earthquake because of its impulsive nature.

Response History Analysis

Rayleigh Damping

- Rayleigh proportional damping was used to represent viscous energy dissipation in the structure.
- The mass and stiffness proportional damping factors were initially set to produce 2.0 percent damping in the first and third modes.
- It is generally recognized that this level of damping (in lieu of the 5 percent damping that is traditionally used in elastic analysis) is appropriate for nonlinear response history analysis.

$$C = \alpha M + \beta K \qquad \begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = \frac{2\xi}{w_1 + w_3} \begin{Bmatrix} w_1 w_3 \\ 1 \end{Bmatrix}$$

Structural frequencies and damping factors used in response history analysis. (Damping factors that produce 2 percent damping in modes 1 and 3)

Model/Damping Parameters	ω_1 (rad/sec)	ω_3 (rad/sec)	α	β
Strong Panel with P-delta	3.184	18.55	0.109	0.00184
Strong Panel without P-delta	3.285	18.81	0.112	0.00181

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Note that $\alpha\lambda\pi\eta\alpha$ and $\beta\varepsilon\tau\alpha$ are directly proportional to ξ. To increase the target damping from 2 percent to 5 percent of critical, all that is required is a multiplying factor of 2.5 on $\alpha\lambda\pi\eta\alpha$ and $\beta\varepsilon\tau\alpha$.

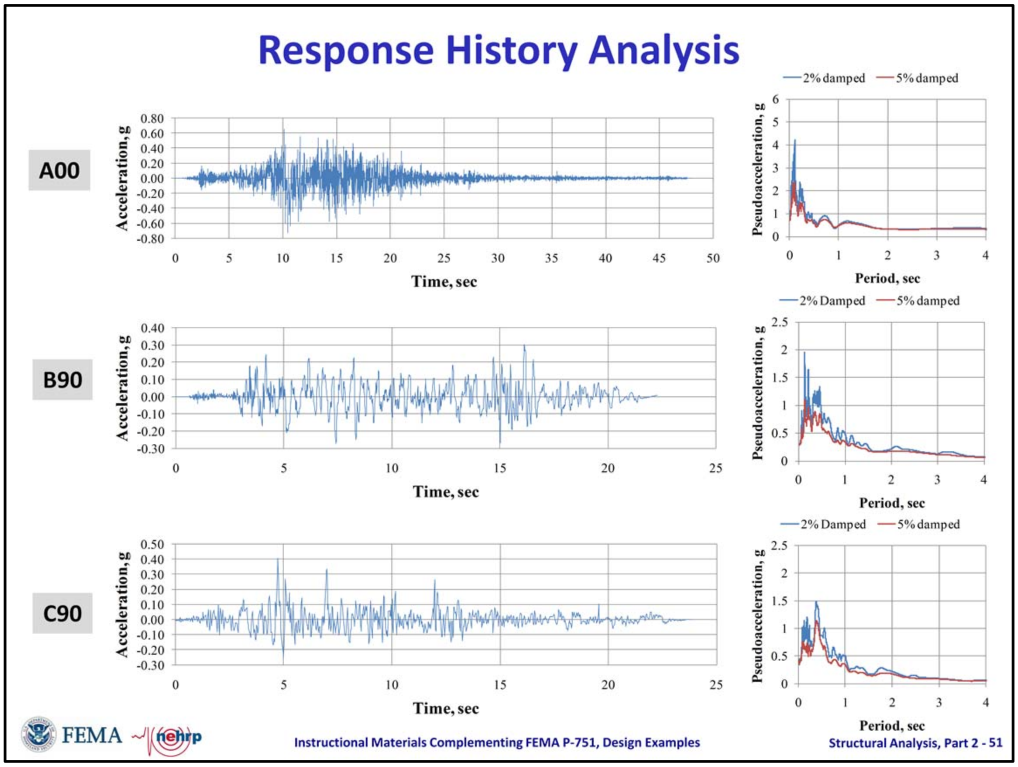
Response History Analysis

Development of Ground Motion Records

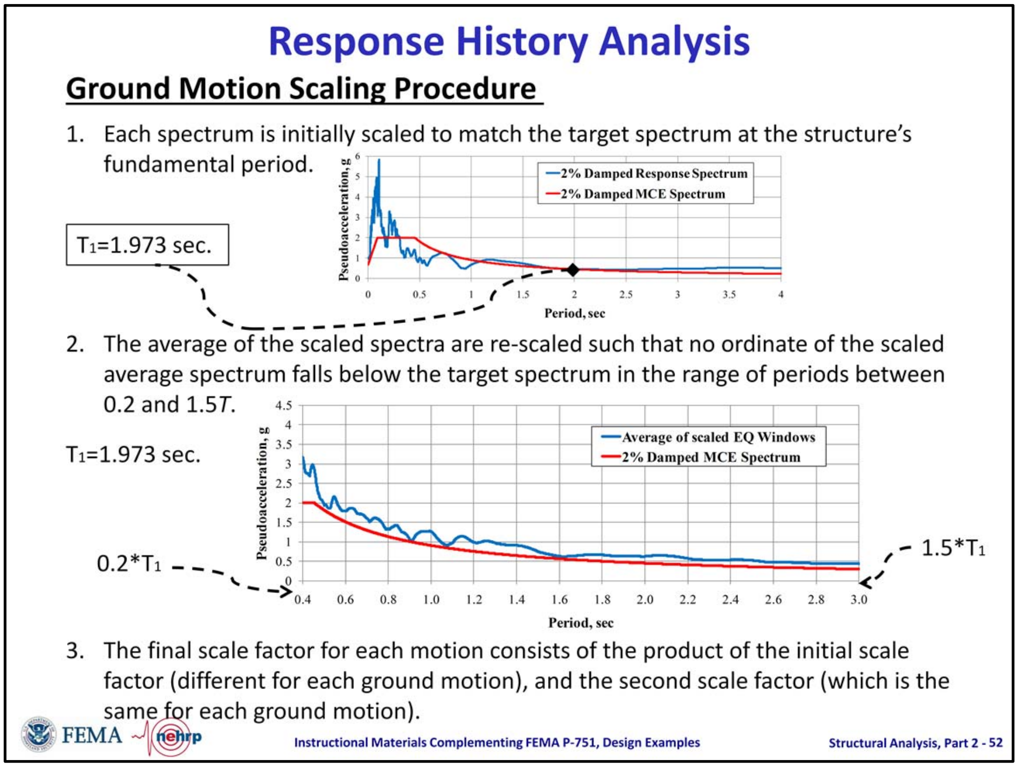
- Because only a two-dimensional analysis of the structure is performed using DRAIN, only a single component of ground motion is applied at one time.
- For the analyses reported herein, the component that produced the larger spectral acceleration at the structure's fundamental period was used.
- A complete analysis would require consideration of both components of ground motions, and possibly of a rotated set of components.

NGA Record Number	Magnitude, [Epicenter Distance (km)]	Site Class	Number of Points and Time step	Integration Time Step used in analyses	Component Source Motion	PGA (g)	Record Name
0879	7.28 , [44]	C	9625 @ 0.005 sec	0.0005 sec	Landers / LCN260	0.727	A00
0725	6.54 , [11.2]	D	2230 @ 0.01 sec	0.001 sec	SUPERST/ B-POE360	0.300	B90
0139	7.35 , [21]	C	1192 @ 0.02 sec	0.001 sec	TABAS/ DAY-TR	0.406	C90

Slide describes development of ground motion records for Response History Analysis.



Slide shows the acceleration time histories and response spectra of the selected motions.



When analyzing structures in two dimensions, Section 16.1.3.1 of the *Standard* (as well as ASCE 7-10) gives the following instructions for scaling:

"The ground motions shall be scaled such that the average value of the 5 percent damped response spectra for the suite of motions is not less than the design response spectrum for the site for periods ranging from 0.2*T* to 1.5*T* where *T* is the natural period of the structure in the fundamental mode for the direction of response being analyzed."

The scaling requirements in *Provisions* Part 3 Resource Paper 3are similar, except that the target spectrum for scaling is the MCE_R spectrum. In this example, the only adjustment is made for scaling when the inherent damping is taken as 2 percent of critical. In this case, the ground motion spectra are based on 2 percent damping, and the DBE or MCE spectrum is adjusted from 5 percent damping to 2 percent damping using the modification factors given in ASCE 41.

The scaling procedure described above has a "degree of freedom" in that there are an infinite number of scaling factors that can fit the criterion. To avoid this, a two-step scaling process is used wherein each spectrum is initially scaled to match the target spectrum at the structure's fundamental period, and then the average of the scaled spectra are re-scaled such that no ordinate of the scaled average spectrum falls below the target spectrum in the range of periods between 0.2*T* and 1.5*T*. The final scale factor for each motion consists of the product of the initial scale factor and the second scale factor.

Results of Response History Analysis

<u>DBE</u> Results for 2% Damped Strong Panel Model with P-Δ Excluded / P-Δ Included

(a) Maximum Base Shear (kips)

	Motion A00	Motion B90	Motion C90
Column Forces	1780 / 1467	1649 / 1458	1543 / 1417
Inertial Forces	1848 / 1558	1650 / 1481	1540 / 1419

(b) Maximum Story Drifts (in.)

Level	Motion A00	Motion B90	Motion C90	Limit*
Total Roof	26.80 / 32.65	14.57 /14.50	13.55 / 14.75	NA
R-6	1.85 / 1.86	1.92 / 1.82	1.71 / 1.70	3.00 (3.75)
6-5	2.51 / 2.64	2.60 / 2.50	2.33 / 2.41	3.00 (3.75)
5-4	**3.75 / 4.08**	3.08 / 2.81	3.03 / 3.19	3.00 (3.75)
4-3	**5.62 / 6.87**	2.98 / 3.21	3.03 / 3.33	3.00 (3.75)
3-2	**6.61 / 8.19**	3.58 / 3.40	2.82 / 2.90	3.00 (3.75)
2-G	**8.09 / 10.40**	**4.68 / 4.69**	3.29 / 3.44	3.60 (4.50)

* Values in ( ) reflect increased drift limits provided by Sec. 16.2.4.3 of the *Standard*

Part (a) of each table provides the maximum base shears, computed either as the sum of column forces (including P-delta effects as applicable), or as the sum of the products of the total acceleration and mass at each level. In each case, the shears computed using the two methods are similar, which serves as a check on the accuracy of the analysis. Had the analysis been run without damping, the shears computed by the two methods should be identical. As expected base shears decrease when P-delta effects are included.

The drift limits in the table, equal to 2 percent of the story height, are the same as provided in *Standard* Table 12.12-1. *Standard* Section 16.2.4.3 provides for the allowable drift to be increased by25 percent where nonlinear response history analysis is used; these limits are shown in the tables in parentheses. *Provisions* Part 2 states that the increase in drift limit is attributed to “the more accurate analysis, and the fact that drifts are computed explicitly.” Drifts that exceed the increased limits are shown in bold text in the tables.

It is interesting that P-Delta effects more or less reduces the drifts for B90 motion. These values are the maximum values though i.e. they don’t necessarily occur at the same time.

Results of Response History Analysis

MCE Results for 2% Damped Strong Panel Model with P-Δ Excluded / P-Δ Included

(a) Maximum Base Shear (kips)

	Motion A00	Motion B90	Motion C90
Column Forces	2181 / 1675	1851 / 1584	1723 / 1507
Inertial Forces	2261 / 1854	1893 / 1633	1725 / 1515

(b) Maximum Story Drifts (in.)

Level	Motion A00	Motion B90	Motion C90	Limit*
Total Roof	62.40 / 101.69	22.45 / 26.10	20.41 / 20.50	NA
R-6	1.98 / 1.95	2.30 / 2.32	3.05 / 2.93	4.50
6-5	3.57 / 2.97	2.77 / 2.60	3.69 / 3.49	4.50
5-4	**7.36** / **6.41**	3.33 / 3.62	4.43 / 4.32	4.50
4-3	**14.61** / **20.69**	**4.61** / **5.61**	4.45 / **4.63**	4.50
3-2	**16.29** / **31.65**	**5.21** / **6.32**	3.97 / 4.18	4.50
2-G	**19.76** / **40.13**	**6.60** / **7.03**	5.11 / 5.11	5.40

The limits are 1.5 times those allowed by *Standard* Section 12.2.1. The 50 percent increase in drift limits is consistent with the increase in ground motion intensity when moving from DBE to MCE ground motions.

Earthquake A00 results in 62.40-inch displacement at the roof level and approximately between 15- to 20-inch drifts at the first three stories of the structure. These story drifts are well above the limits. When P-delta effects are included with the same level of motion, roof displacement increases to 101.69 inches with approximately 20- to 40-inch displacement at the first three stories.

It is clear from Part (b) of Tables that Ground Motion A00 is much more demanding with respect to drift than are the other two motions. The drifts produced by Ground Motion A00 are particularly large at the lower levels, with the more liberal drift limits being exceeded in the lower four stories of the building. When P-delta effects are included, the drifts produced by Ground Motion A00 increase significantly; drifts produced by Ground Motions B90 and C90 change only slightly.

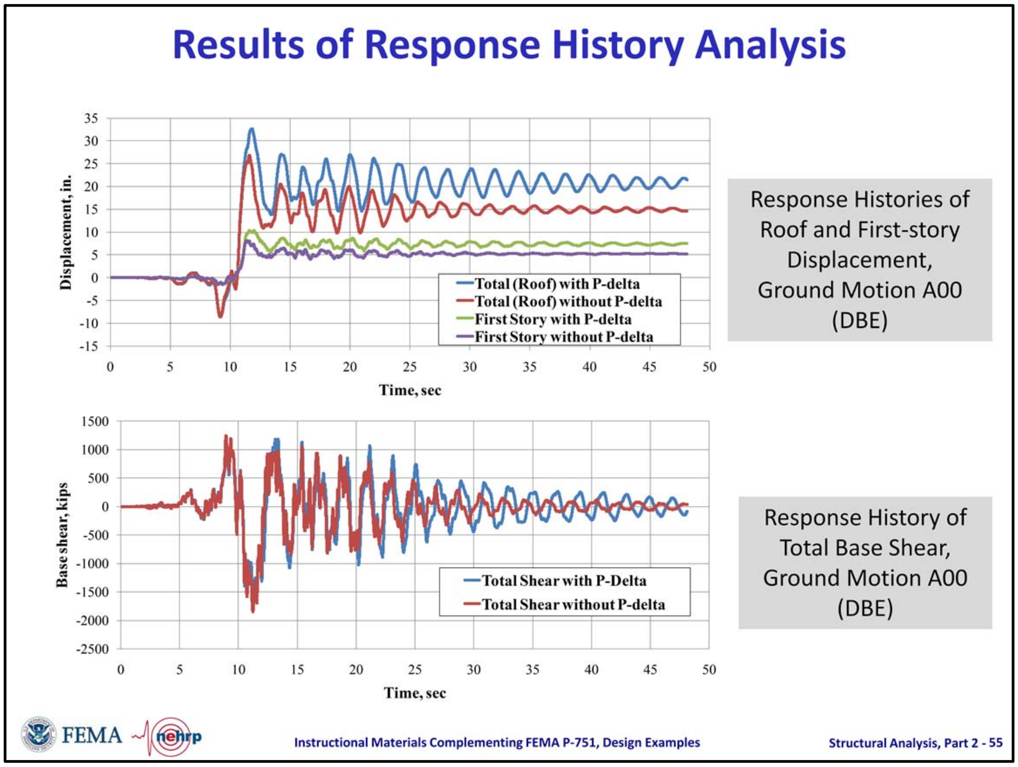
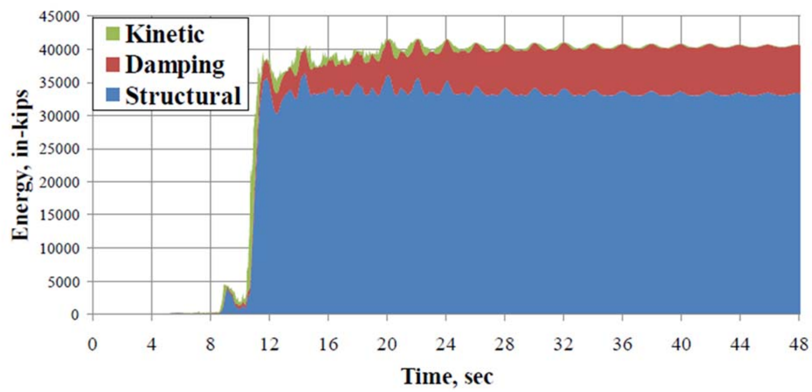


Figure 1 shows response histories of roof displacement and first story drift for the 2 percent damped SP model subjected to the DBE-scaled A00 ground motion. Two trends are readily apparent. First, the vast majority of the roof displacement is due to residual deformation in the first story. Second, the P-delta effect increases residual deformations by about 50 percent. Such extreme differences in behavior do not appear in plots of base shear, as provided in Figure 2.

The residual deformations shown in Figure 1 may be real (due to actual system behavior) or may reflect accumulated numerical errors in the analysis. Numerical errors are unlikely because the shears computed from member forces and from inertial forces are similar.

Results of Response History Analysis



Energy Response History, Ground Motion A00 (DBE), including P-delta effects

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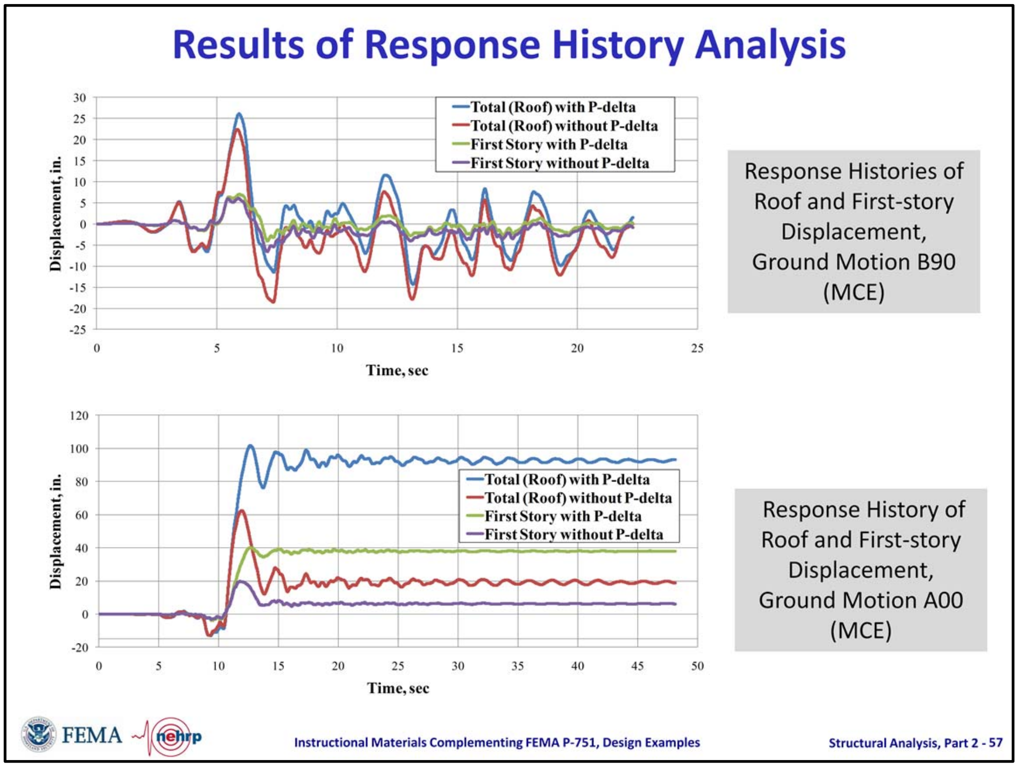
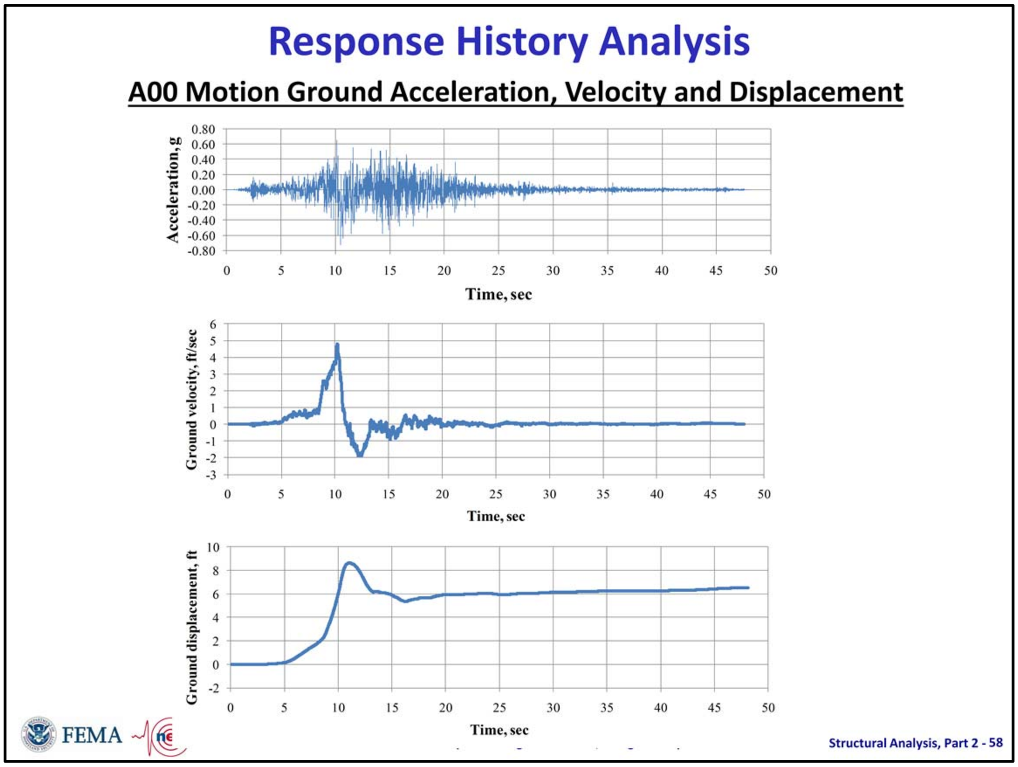
If the analysis is accurate, the input energy will coincide with the total energy (sum of kinetic, damping, and structural energy). DRAIN 2D produces individual energy values as well as the input energy. As seen in Figure, the total and input energy curves coincide, so the analysis is numerically accurate. Where this accuracy is in doubt, the analysis should be re-run using a smaller integration time step.



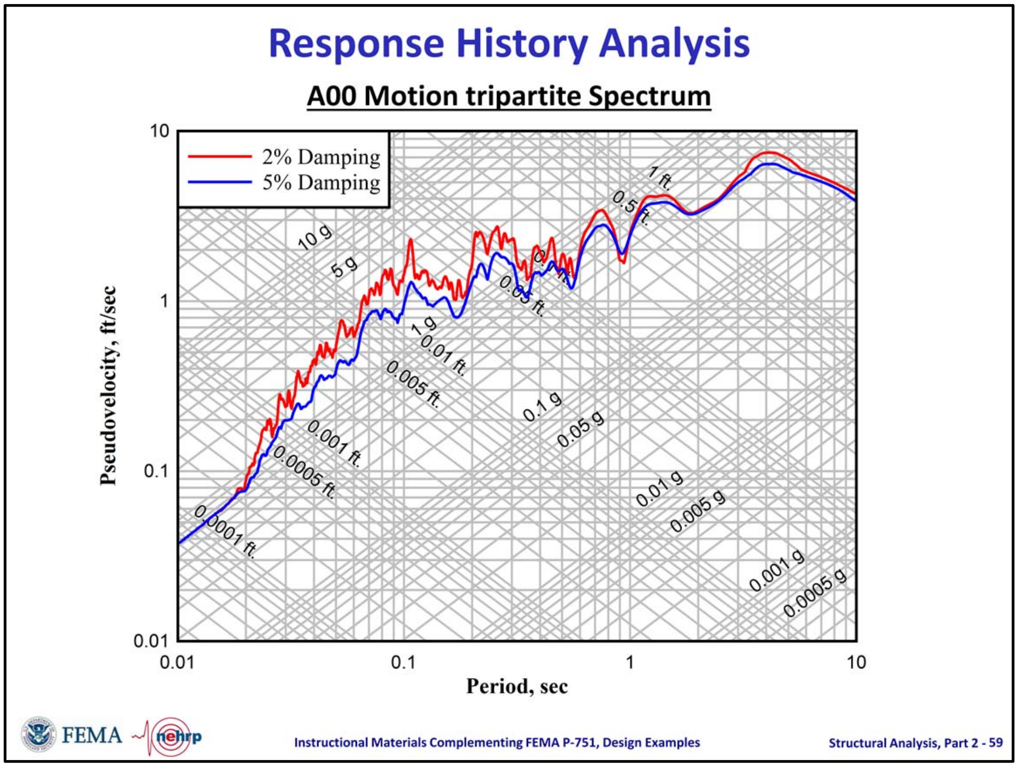
It is interesting to compare the response computed for Ground Motion B90 with that obtained for ground motion A00. While there is some small residual deformation in Figure 1 (B90 motion), it is not extreme, and it appears that the structure is not in danger of collapse. (The corresponding plastic rotations are less than those that would be associated with significant strength loss.)

As may be seen in Figure 2, when MCE type A00 motion is used, residual deformations again dominate (as the DBE case), and in this case the total residual roof displacement with P-delta effects included is five times that without P-delta effects. This behavior indicates dynamic instability and eventual collapse.

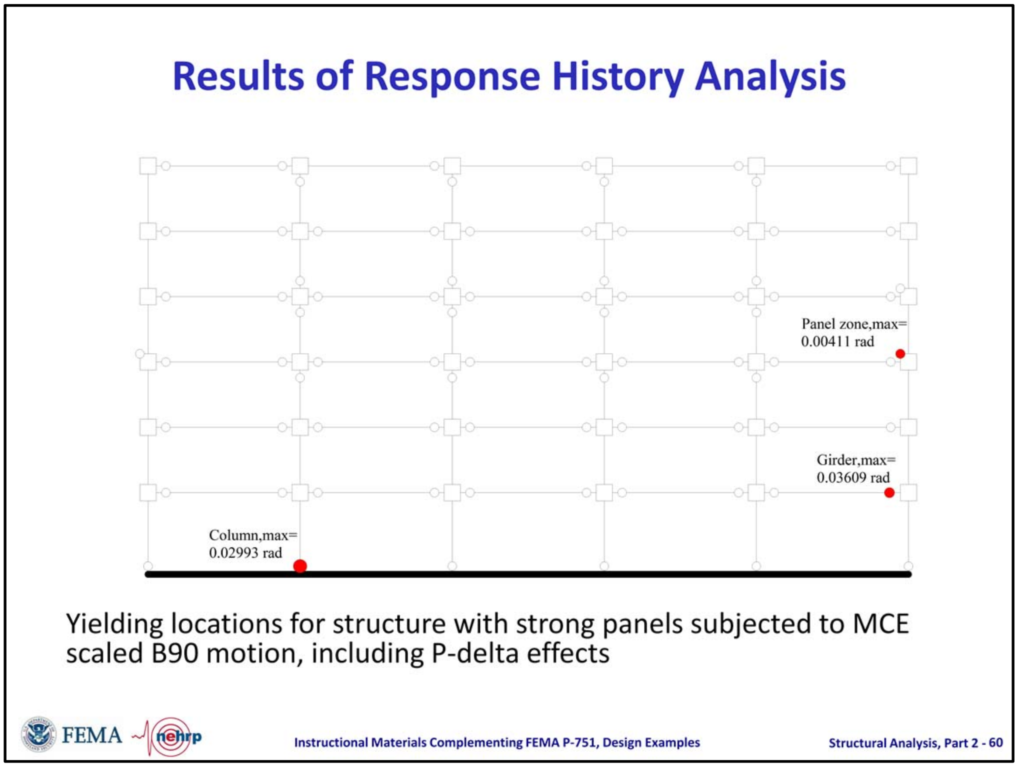


The characteristic of the ground motion (A00) that produces the residual deformations is not evident from the ground acceleration history or from the acceleration response spectrum. The source of the behavior is quite obvious from plots of the ground velocity and ground displacement histories.

The ground velocity history shows that a very large velocity pulse occurs approximately 10 seconds into the earthquake. This leads to a surge in ground displacement, also occurring approximately 10 seconds into the response. The surge in ground displacement is more than *8 feet*, which is somewhat unusual.



The unusual characteristics of Ground Motion A00 may be seen in Figure which is a tripartite spectrum.



The circles on the figure represent yielding at any time during the response; consequently, yielding does not necessarily occur at all locations simultaneously. The circles shown at the upper left corner of the beam-column joint region indicate yielding in the rotational spring, which represents the web component of panel zone behavior. There is no yielding in the flange component of the panel zones, as seen in Figure.

Yielding patterns for the other ground motions and for analyses run with and without P-delta effects are similar but are not shown here. As expected, there is more yielding in the columns when the structure is subjected to the A00 ground motion.

The maximum plastic hinge rotations are shown where they occur for the columns, girders, and panel zones.

Results of Response History Analysis

Comparison with Results from Other Analyses

Response Quantity	Analysis Method		
	Equivalent Lateral Forces	Nonlinear Static Pushover	Nonlinear Dynamic
Base Shear (kips)	569	1208	1633
Roof Disp. (in.)	18.4	22.9	26.1
Drift R-6 (in.)	1.86	0.96	2.32
Drift 6-5 (in.)	2.78	1.76	2.60
Drift 5-4 (in.)	3.34	2.87	3.62
Drift 4-3 (in.)	3.73	4.84	5.61
Drift 3-2 (in.)	3.67	5.74	6.32
Drift 2-1 (in.)	2.98	6.73	7.03
Girder Hinge Rot. (rad)	NA	0.03304	0.03609
Column Hinge Rot. (rad)	NA	0.02875	0.02993
Panel Hinge Rot. (rad)	NA	0.00335	0.00411
Panel Plastic Shear Strain	NA	0.00335	0.00411

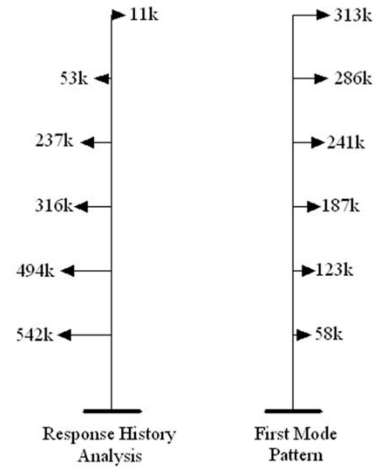
Note: Shears are for half of total structure.

Table compares the results obtained from the response history analysis with those obtained from the ELF and the nonlinear static pushover analyses. Recall that the base shears in the table represent half of the total shear in the building. As it was discussed before, 2% damped MCE based spectrum was used for the pushover analysis. To be consistent, the results of 2% damped MCE scaled B90 motion was used for the nonlinear dynamic analysis part of the table. In addition, the lateral forces used to find the ELF drifts in Slide 7 were multiplied by 1.5 to make them consistent with the MCE level of shaking. The ELF analysis drift values include the deflection amplification factor of 5.5. The results tabulated as results of pushover analysis are obtained at the load level of target displacement.

Results of Response History Analysis

<u>Reasons of the differences between Pushover and Response History Analyses</u>

- Scale factor of 1.367 was used for the 2nd part of the scaling procedure.
- The use of the first-mode lateral loading pattern in the nonlinear static pushover response.
- The higher mode effects shown in the Figure are the likely cause of the different hinging patterns and are certainly the reason for the very high base shear developed in the response history analysis.



Comparison of inertial force patterns

Figure shows the inertial forces from the nonlinear response history analyses at the time of peak base shear and the loads applied to the nonlinear static analysis model at the target displacement.

Results of Response History Analysis

Effect of Increased Damping on Response

- Excessive drifts occur in the bottom three stories.
- Additional strength and/or stiffness should be provided at these stories.
- Considered next, Added damping is also a viable approach.
- Four different damper configurations were used.
- Dampers were added to the Strong Panel frame with 2% inherent damping.
- The structure was subjected to the DBE scaled A00 and B90 ground motions.
- P-delta effects were included in the analyses.

Slide summarizes results of response history analysis.

Modeling Added Dampers

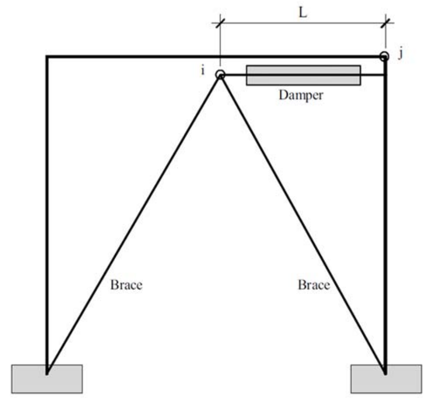
- Added damping is easily accomplished in DRAIN by use of the stiffness proportional component of Rayleigh damping.
- Linear viscous fluid damping device can be modeled through use of a Type-1 (truss bar) element.

$$k_{device} = \frac{A_{device}E_{device}}{L_{device}}$$

$$C_{device} = \beta_{device}k_{device}$$

- Set damper elastic stiffness to negligible value. k_{device} = 0.001 kips/in.

$$\beta_{device} = \frac{C_{device}}{0.001} = 1000C_{device}$$



Modeling a simple damper

- It is convenient to set E_{device} = 0.001 and A_{device} = Damper length L_{device}

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Base shear increases with added damping, so in practice added damping systems usually employ nonlinear viscous fluid devices with a “softening” relationship between the deformational velocity in the device and the force in the device, to limit base shears when deformational velocities become large.

This value of β_{device} is for the added damper element *only*. Different dampers may require different values. Also, a different (global) value of β is required to model the stiffness proportional component of damping in the remaining nondamper elements.

Modeling the dynamic response using Type 1 elements is exact within the typical limitations of finite element analysis. Using the modal strain energy approach, DRAIN reports a damping value in each mode. These modal damping values are approximate and may be poor estimates of actual modal damping, particularly where there is excessive flexibility in the mechanism that connects the damper to the structure.

Results of Response History Analysis

Effect of Increased Damping on Response

Effect of different added damper configurations when SP model is subjected to DBE scaled A00 motion, including P-delta effects

Level	No Damper Drift, In.	1st combo Damper Coeff, kip-sec/in.	1st combo Drift, in.	2nd combo Damper Coeff, kip-sec/in.	2nd combo Drift, in.	3rd combo Damper Coeff, kip-sec/in.	3rd combo Drift, in.	4th combo Damper Coeff, kip-sec/in.	4th combo Drift, in.	Drift Limit in.
R-6	1.86	10.5	1.10	60	1.03	-	1.82	-	1.47	3.75
6-5	2.64	33.7	1.90	60	1.84	-	3.56	-	2.41	3.75
5-4	**4.08**	38.4	2.99	70	2.88	-	**4.86**	56.25	3.46	3.75
4-3	**6.87**	32.1	**5.46**	70	**4.42**	-	**5.24**	56.25	**4.47**	3.75
3-2	**8.19**	36.5	**6.69**	80	**5.15**	160	**4.64**	112.5	**4.76**	3.75
2-G	**10.40**	25.6	**8.39**	80	**5.87**	160	4.40	112.5	**4.96**	4.50
Column Base Shear,kips	1467	1629		2170		2134		2267		
Inertial Base Shear,kips	1558	1728		2268		2215		2350		
Total Damping,%	2	10.1		20.4		20.2		20.4		

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Four different added damper configurations are used to asses their effect on story drifts and base shear. These configurations increase total damping of the structure from 2 percent (inherent) to 10 and 20 percent. In the first configuration added dampers are distributed proportionally to approximate story stiffnesses. In the second configuration, dampers are added at all six stories, with larger dampers in lower stories. Since the structure seems to be weak at the bottom stories (where it exceeds drift limits), dampers are concentrated at the bottom stories in the last two configurations. Added dampers are used only at the first and second stories in the third configuration and at the bottom four stories in the fourth configuration.

Based on this supplemental damper study, it appears to be impossible to decrease the story drifts for the A00 ground motion below the limits. This is because of the incremental velocity of Ground Motion A00 causes such significant structural damage. The drift limits could be satisfied if the total damping ratio is increased to 33.5 percent, but since that is impractical the results are not reported here. The third configuration of added dampers reduces the first-story drift from 10.40 inches to 4.40 inches.

Results of Response History Analysis

Effect of Increased Damping on Response

Effect of different added damper configurations when SP model is subjected to DBE scaled B90 motion, including P-delta effects

	No Damper	1st combo		2nd combo		3rd combo		4th combo		
Level	Drift, In.	Damper Coeff, kip-sec/in.	Drift, in.	Damper Coeff, kip-sec/in.	Drift, in.	Damper Coeff, kip-sec/in.	Drift, in.	Damper Coeff, kip-sec/in.	Drift, in.	Drift Limit in.
R-6	1.82	10.5	1.11	60	0.86	-	1.53	-	1.31	3.75
6-5	2.50	33.7	1.76	60	1.35	-	2.11	-	1.83	3.75
5-4	2.81	38.4	2.33	70	1.75	-	2.51	56.25	2.07	3.75
4-3	3.21	32.1	2.67	70	2.11	-	2.37	56.25	2.16	3.75
3-2	3.40	36.5	2.99	80	2.25	160	2.09	112.5	2.13	3.75
2-G	**4.69**	25.6	3.49	80	1.96	160	1.87	112.5	1.82	4.50
Column Base Shear,kips	1458	1481		1485		1697		1637		
Inertial Base Shear,kips	1481	1531		1527		1739		1680		
Total Damping,%	2	10.1		20.4		20.2		20.4		

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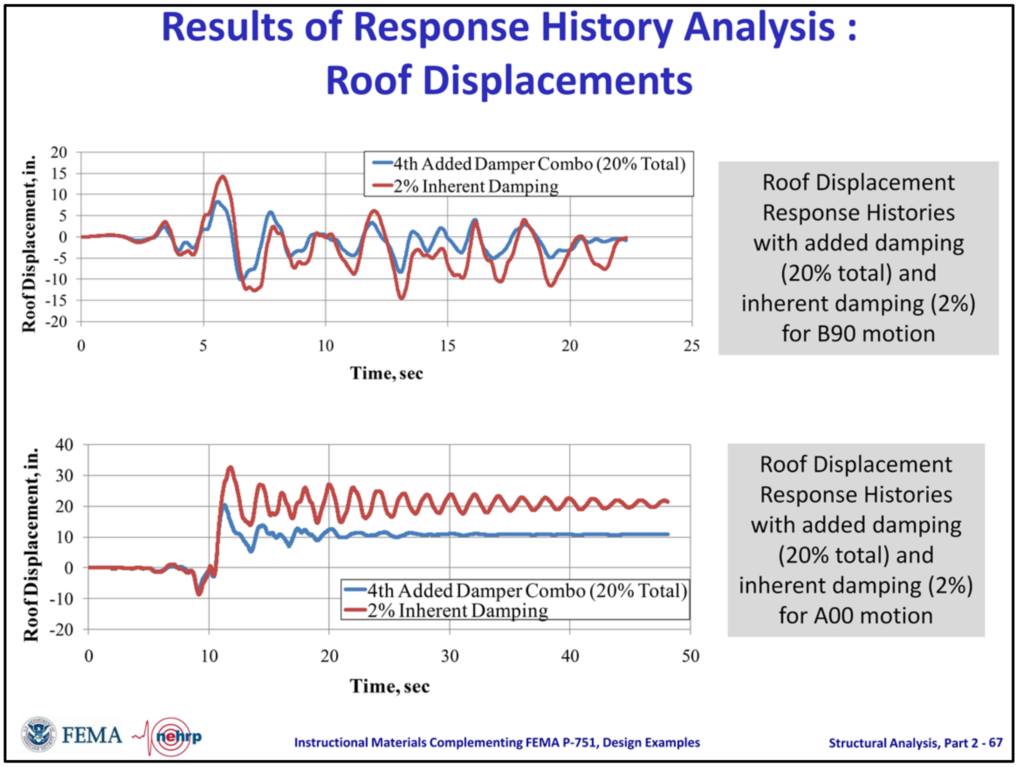
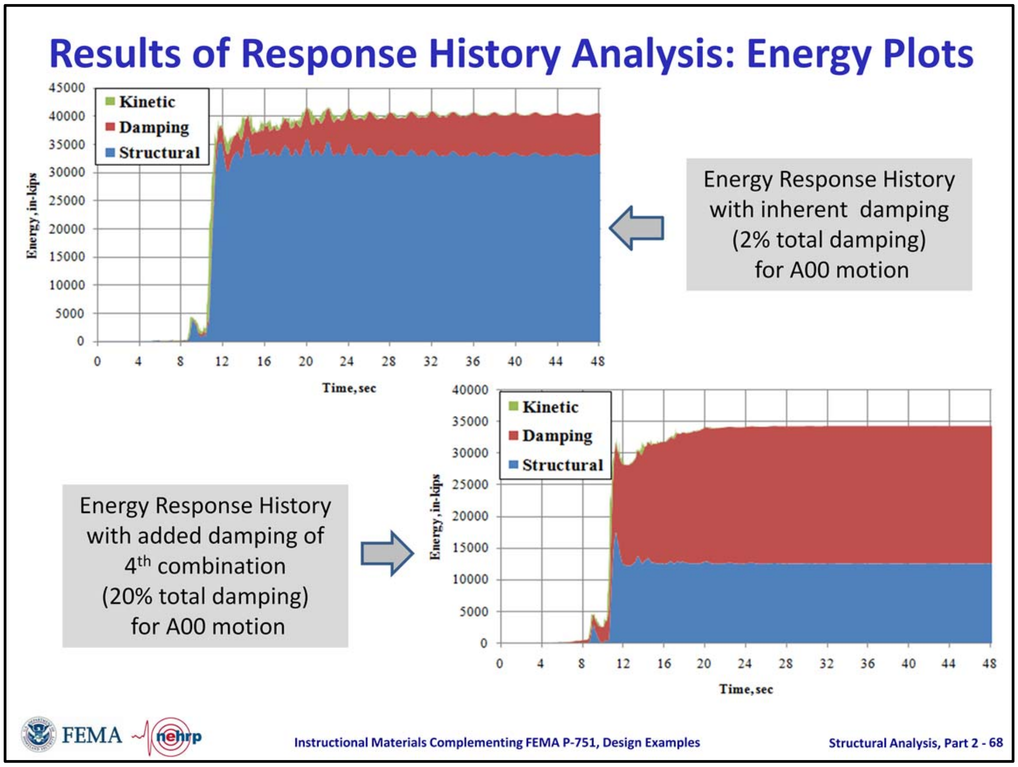
All of the configurations easily satisfy drift limits for the B90 ground motion. While the system with 10 percent total damping is sufficient for drift limits, systems with 20 percent damping further improve performance. Although configurations 3 and 4 have the same amount of total damping as configuration 2, story drifts are higher at the top stories since dampers are added only at lower stories.

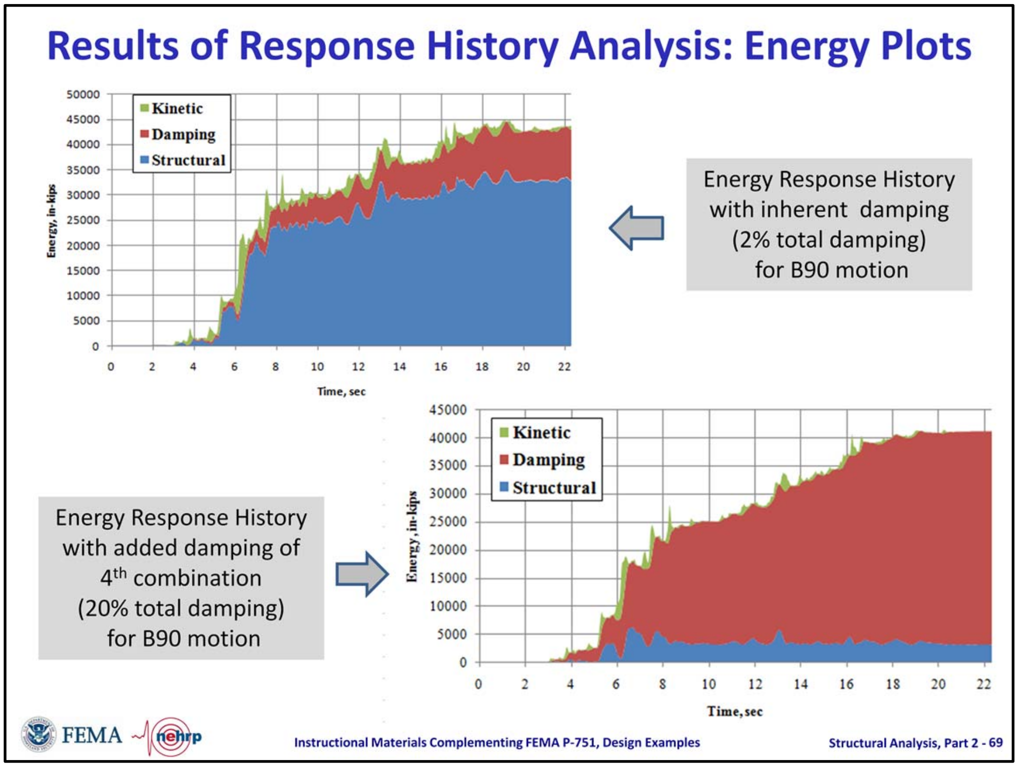


Added dampers reduce the roof displacement for both A00 and B90 ground motions.

As Figure 2 shows added dampers reduce roof displacement significantly but do not prevent residual displacement for the A00 ground motion.

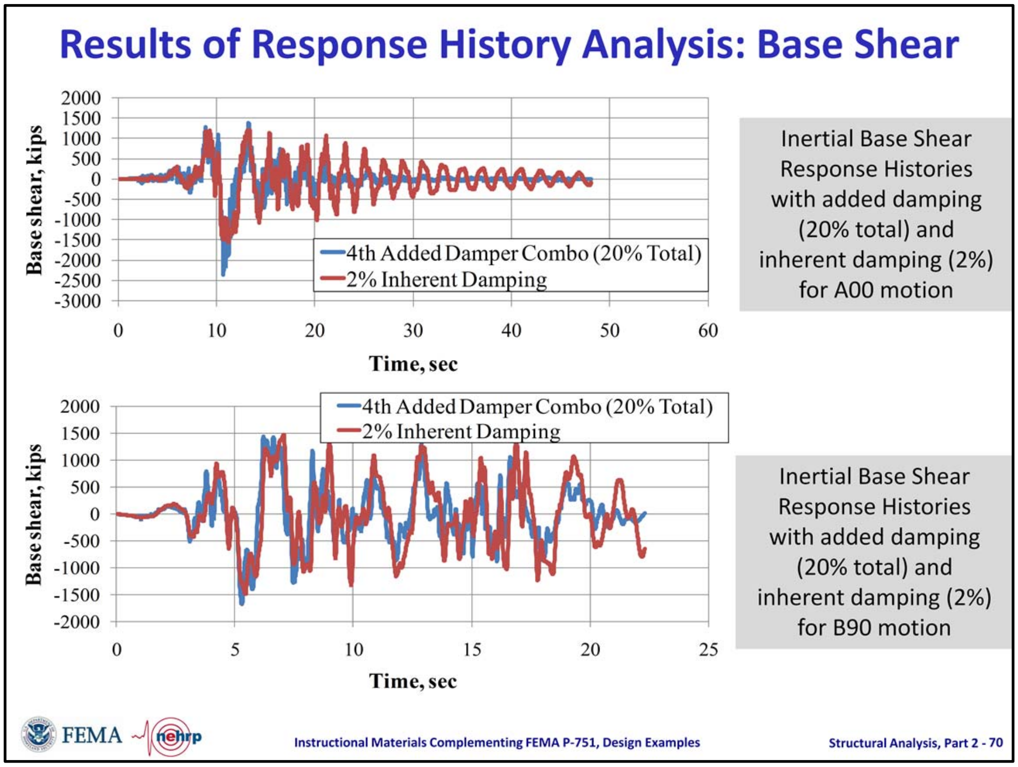


As should be expected, adding discrete damping reduces the hysteretic energy demand in the structure (designated as structural energy in Figures). A reduction in hysteretic energy demand for the system with added damping corresponds to a reduction in structural damage.



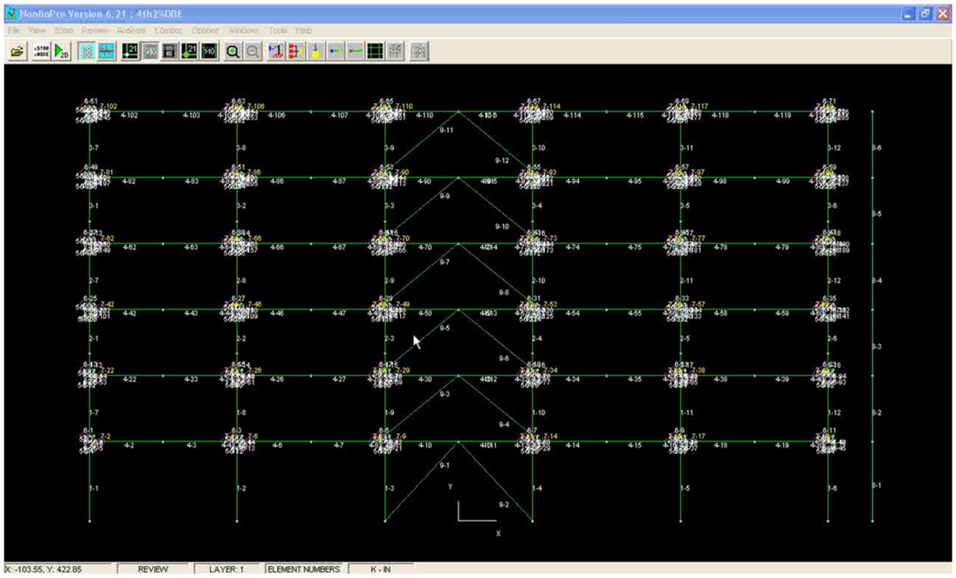
Again, adding discrete damping reduces the hysteretic energy demand, which results in a reduction in structural damage for B90 motion.

As may be seen, added dampers are more efficient in terms of energy dissipation for B90 motion than A00 motion (See previous slide).



Figures show how added damping increases base shear. Especially, for A00 motion, the maximum base shear increases more than 50%.

Results of Response History Analysis: Deflected Shape of by NonlinPro for Added Damper Frame (4th combination) During B90 Motion



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This slide shows a movie which is obtained using the snapshot tool of NonlinPro. Displaced shape of the 4th combination added damper frame under B90 motion is displayed.

Summary and Conclusions

- Five different analytical approaches were used to estimate the deformation demands in a simple unbraced steel frame structure:
    1. Linear static analysis (the equivalent lateral force method)
    2. Plastic strength analysis (using virtual work)
    3. Nonlinear static pushover analysis
    4. Linear dynamic analysis
    5. Nonlinear dynamic response history analysis
- Approaches 1, 3, and 5 were carried to a point that allowed comparison of results. The results obtained from the three different analytical approaches were quite dissimilar.
- Because of the influence of the higher mode effects on the response, pushover analysis, where used alone, is inadequate.
- Except for preliminary design, the ELF approach should not be used in explicit performance evaluation as it has no mechanism for determining location and extent of yielding in the structure.
- Response history analysis as the most viable approach. However, significant shortcomings, limitations, and uncertainties in response history analysis still exist.
- In modeling the structure, particular attention was paid to representing possible inelastic behavior in the panel-zone regions of the beam-column joints.

Summary and Conclusions.

Slide prompts participants to ask questions.

4

Structural Analysis

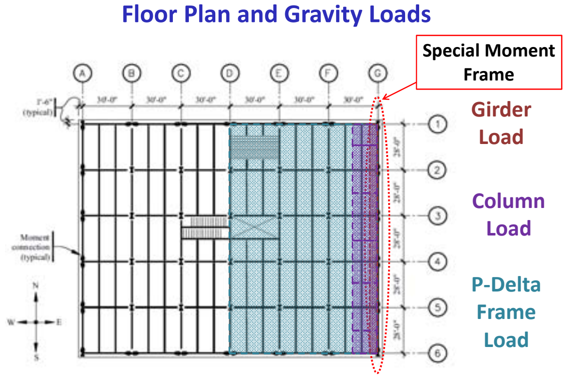
Finley Charney, Adrian Tola Tola, and Ozgur Atlayan

Example 2:
Six-story Moment Resisting Steel Frame

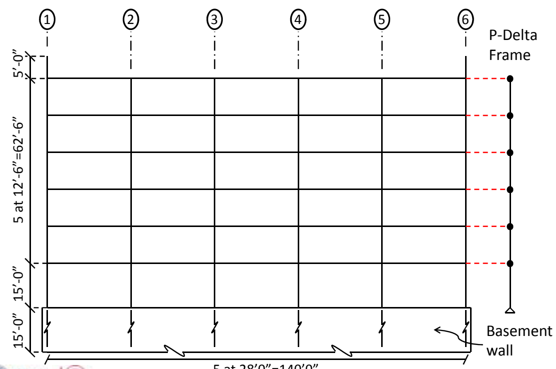
Description of Structure

- 6-story office building in Seattle, Washington
- Occupancy (Risk) Category II
- Importance factor (I) = 1.0
- Site Class = C
- Seismic Design Category D
- Special Moment Frame (SMF), R = 8, C_d = 5.5

Floor Plan and Gravity Loads



Elevation view and P-Delta Column



Member Sizes Used in N-S Moment Frames

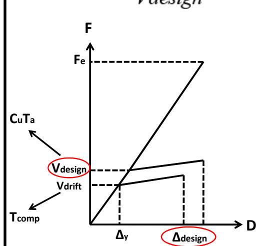
Member Supporting Level	Column	Girder	Doubler Plate Thickness (in.)
R	W21x122	W24x84	1.00
6	W21x122	W24x84	1.00
5	W21x147	W27x94	1.00
4	W21x147	W27x94	1.00
3	W21x201	W27x94	0.875
2	W21x201	W27x94	0.875

✓ Sections meet the width-to-thickness requirements for special moment frames

✓ Strong column-weak beam

Equivalent Lateral Force Procedure

$$\frac{Fe}{Vdesign} = R \qquad \frac{\Delta design}{\Delta y} = C_d$$



Approximate Period of Vibration

$T_a = C_t h_n^x$

from *Standard* Table 12.8-2

$C_t = 0.028$ and $x = 0.8$

$T_a = 0.028(77.5)^{0.8} = 0.91$ sec/cycle.

$C_u T_a = 1.4(0.91) = 1.27$ sec/cycle.

T_{comp}=2.05 sec (without P-Delta)

T_{comp}=2.13 sec (with P-Delta)

Equivalent Lateral Force Procedure

Vertical Distribution of Forces

$$F_x = C_{vx}V \text{ and } C_{vx} = \frac{w_x h^k}{\sum_{i=1}^{n} w_i h_i^k}$$

Equivalent Lateral Forces for Building Responding in N-S Direction

Level x	w_x (kips)	h_x (ft)	$w_x h_x^k$	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
R	2,596	77.5	1,080,327	0.321	243.6	243.6	3,045
6	2,608	65.0	850,539	0.253	191.8	435.4	8,488
5	2,608	52.5	632,564	0.188	142.6	578.0	15,713
4	2,608	40.0	433,888	0.129	97.8	675.9	24,161
3	2,608	27.5	258,095	0.077	58.2	734.1	33,337
2	2,621	15.0	111,909	0.033	25.2	759.3	44,727
Σ	15,650		3,367,323	1.000	759.3		

Computer Programs NONLIN-Pro and DRAIN 2Dx

Shortcomings of DRAIN

- It is not possible to model strength loss when using the ASCE 41-06 (2006) model for girder plastic hinges.
- The DRAIN model for axial-flexural interaction in columns is not particularly accurate.
- Only Two-Dimensional analysis may be performed.

Elements used in Analysis

- Type 1, inelastic bar (truss) element
- Type 2, beam-column element
- Type 4, connection element

Description of Preliminary Model

- Only a single frame (Frame A or G) is modeled.
- Columns are fixed at their base.
- Each beam or column element is modeled using a Type 2 element. For the columns, axial, flexural, and shear deformations are included. For the girders, flexural and shear deformations are included but, because of diaphragm slaving, axial deformation is not included. Composite action in the floor slab is ignored for all analysis.
- All members are modeled using centerline dimensions without rigid end offsets.
- This model does not provide any increase in beam-column joint stiffness due to the presence of doubler plates.
- The stiffness of the girders was decreased by 7% in the preliminary analyses, which should be a reasonable approximate representation of the 35% reduction in the flange sections. Moment rotation properties of the reduced flange sections are used in the detailed analyses.

Results of Preliminary Analysis : Drift

Results of Preliminary Analysis Excluding P-delta Effects

Story	Total Drift (in.)	Story Drift (in.)		Magnified Story Drift (in.)	Drift Limit (in.)	Story Stability Ratio, θ
6	2.08	0.22		1.21	3.00	0.0278
5	1.86	0.32		1.76	3.00	0.0453
4	1.54	0.38	× 5.5 =	2.09	3.00	0.0608
3	1.16	0.41		2.26	3.00	0.0749
2	0.75	0.41		2.26	3.00	0.0862
1	0.34	0.34		1.87	3.60	0.0691

Results of Preliminary Analysis Including P-delta Effects

Story	Total Drift (in.)	Story Drift (in.)		Magnified Story Drift (in.)		Drift from θ (in.)	Drift Limit (in.)
6	2.23	0.23		1.27	↔	1.24	3.00
5	2.00	0.34		1.87	↔	1.84	3.00
4	1.66	0.40	× 5.5 =	2.20	↔	2.23	3.00
3	1.26	0.45		2.48	↔	2.44	3.00
2	0.81	0.45		2.48	↔	2.47	3.00
1	0.36	0.36		1.98	↔	2.01	3.60

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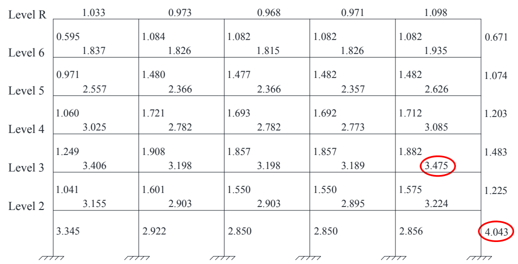
Results of Preliminary Analysis : Demand Capacity Ratios (Columns-Girders)



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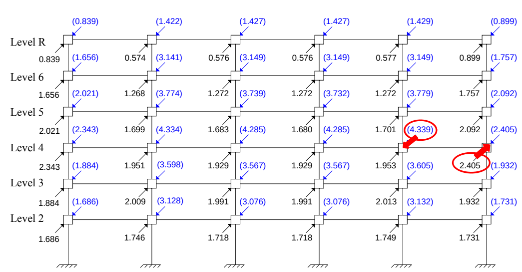
Results of Preliminary Analysis : Demand Capacity Ratios (Panel Zones)



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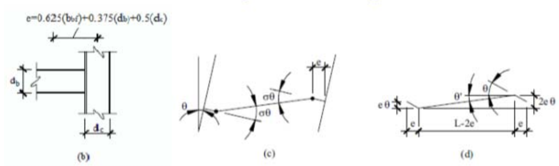
Results of Preliminary Analysis : Demand Capacity Ratios

- The structure has considerable overstrength, particularly at the upper levels.
- The sequence of yielding will progress from the lower level girders to the upper level girders.
- With the possible exception of the first level, the girders should yield before the columns. While not shown in the Figure, it should be noted that the demand-to-capacity ratios for the lower story columns were controlled by the moment at the base of the column. The column on the leeward (right) side of the building will yield first because of the additional axial compressive force arising from the seismic effects.
- The maximum DCR of girders is 3.475, while maximum DCR for panel zones without doubler plates is 4.339. Thus, if doubler plates are not used, the first yield in the structure will be in the panel zones. However, with doubler plates added, the first yield is at the girders as the maximum DCR of the panel zones reduces to 2.405.

Results of Preliminary Analysis: Overall System Strength



Internal Work = External Work

$$\text{Internal Work} = 2[20\sigma\theta M_{PA} + 40\sigma\theta M_{PB} + \theta(M_{PC} + 4M_{PD} + M_{PE})]$$

$$\text{External Work} = V\theta \sum_{i=1}^{nLevels} F_i H_i \quad \text{where} \quad \sum_{i=1}^{nLevels} F_i = 1$$

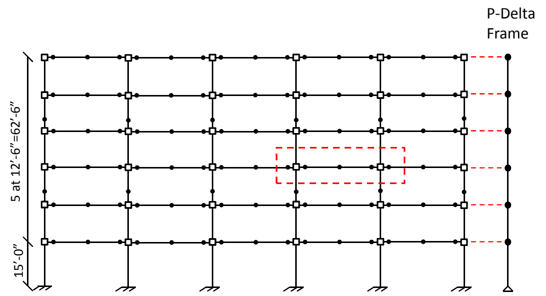
Results of Preliminary Analysis: Overall System Strength

Lateral Strength on Basis of Rigid-Plastic Mechanism

Lateral Load Pattern	Lateral Strength (kips) Entire Structure	Lateral Strength (kips) Single Frame
Uniform	3,332	1,666
Upper Triangular	2,747	1,373
Standard	2,616	1,308

- As expected, the strength under uniform load is significantly greater than under triangular or *Standards* load.
- The closeness of the *Standards* and triangular load strengths is due to the fact that the vertical-load-distributing parameter (k) was 1.385, which is close to 1.0.
- Slightly more than 15 percent of the system strength comes from plastic hinges that form in the columns. If the strength of the column is taken simply as M_p (without the influence of axial force), the "error" in total strength is less than 2 percent.
- The rigid-plastic analysis did not include strain hardening, which is an additional source of overstrength.

Description of Model Used for Detailed Structural Analysis



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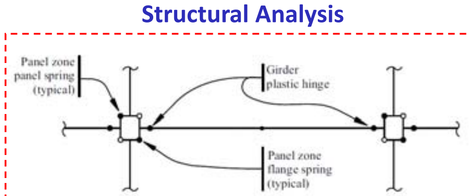
Structural Analysis, Part 2 - 16

Description of Model Used for Detailed Structural Analysis



- Nonlinear static and nonlinear dynamic analyses require a much more detailed model than was used in the linear analysis.
- The primary reason for the difference is the need to explicitly represent yielding in the girders, columns, and panel zone region of the beam-column joints.

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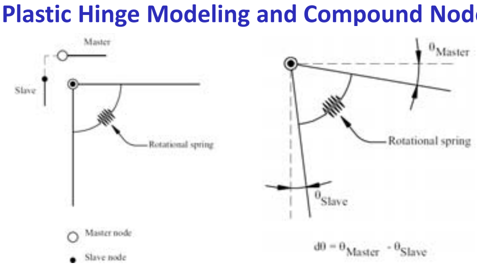
Structural Analysis, Part 2 - 17

Plastic Hinge Modeling and Compound Nodes



- Compound nodes are used to model plastic hinges in girders and deformations in the panel zone region of beam-column joints
- Typically consist of a pair of single nodes with each node sharing the same point in space. The X and Y degrees of freedom of the first node of the pair (the slave node) are constrained to be equal to the X and Y degrees of freedom of the second node of the pair (the master node), respectively. Hence, the compound node has four degrees of freedom: an X displacement, a Y displacement, and two independent rotations.

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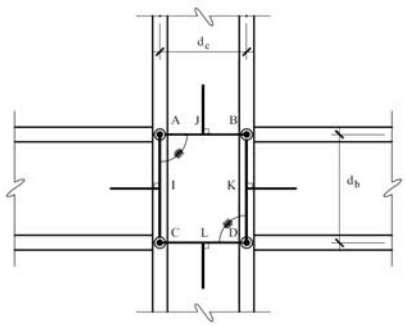
Structural Analysis, Part 2 - 18

Modeling of Beam-Column Joint Regions



Krawinkler beam-column joint model

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Modeling of Beam-Column Joint Regions

Krawinkler model assumes that the panel zone has two resistance mechanisms acting in parallel:

1. Shear resistance of the web of the column, including doubler plates and
2. Flexural resistance of the flanges of the column.

$$R_v = 0.6F_y d_c t_p + 1.8\frac{F_y b_{cf} t_{cf}^2}{d_b} = V_{Panel} + 1.8V_{Flanges}$$

- F_y = yield strength of the column and the doubler plate,
- d_c = total depth of column,
- t_p = thickness of panel zone region = column web + doubler plate thickness,
- b_{cf} = width of column flange,
- t_{cf} = thickness of column flange, and
- d_b = total depth of girder.

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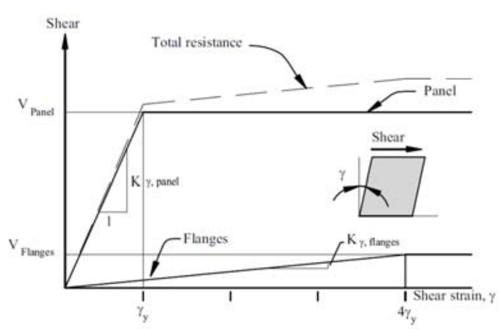
Modeling of Beam-Column Joint Regions



Force-deformation behavior of panel zone region (Krawinkler Model)

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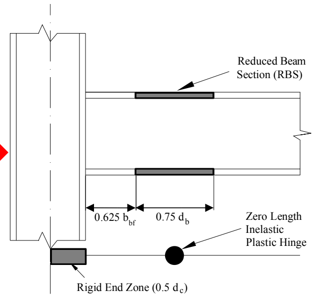
Modeling Girders

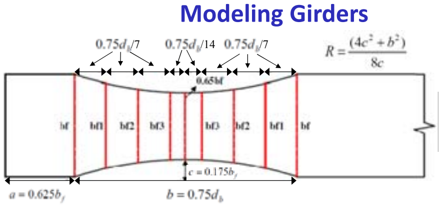
- The AISC Seismic Design Manual (AISC, 2006) recommends design practices to force the plastic hinge forming in the beam away from the column.

1. Reduce the cross sectional properties of the beam at a specific location away from the column
2. Special detailing of the beam-column connection to provide adequate strength and toughness in the connection so that inelasticity will be forced into the beam adjacent to the column face.

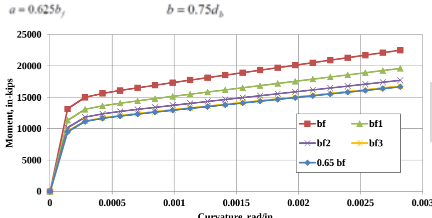


Side view of beam element and beam modeling

Modeling Girders

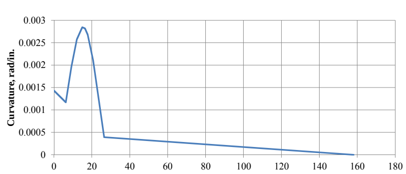


Top view of Reduced Beam Section

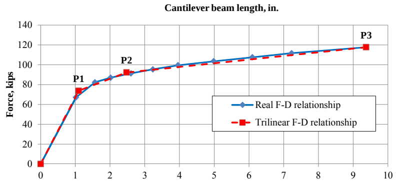


Moment curvature diagram for W27x94 girder

Modeling Girders

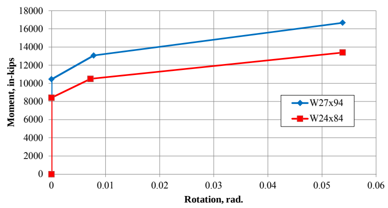


Curvature Diagram for Cantilever Beam with Reduced Beam Section



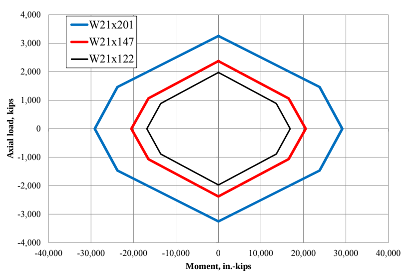
Force Displacement Diagram for W27x94 with RBS

Modeling Girders



Moment-Rotation Diagram for girder hinges with RBS

Modeling Columns



Yield surface used for modeling columns

Results of Detailed Analysis: Period of Vibration

Periods of Vibration From Detailed Analysis (sec/cycle)

Model	Mode	P-delta Excluded	P-delta Included
Strong Panel	1	1.912	1.973
with	2	0.627	0.639
doubler plates	3	0.334	0.339
Weak Panel	1	2.000	2.069
without	2	0.654	0.668
doubler plates	3	0.344	0.349

- P-delta effects increases the period.
- Doubler plates decreases the period as the model becomes stiffer with doubler plates.
- Different period values were obtained from preliminary and detailed analyses.
- Detailed model results in a stiffer structure than the preliminary model especially when doubler plates are added.

Static Pushover Analysis

- Pushover analysis procedure performed in this example follows the recommendations of ASCE/SEI 41-06.
- Pushover analysis should *always* be used as a precursor to nonlinear response history analysis.
- The structure is subjected to the full dead load plus 50 percent of the fully reduced live load, followed by the lateral loads.
- For the entire pushover analyses reported for this example, the structure is pushed to 37.5 in. at the roof level. This value is about two times the total drift limit for the structure where the total drift limit is taken as 2 percent of the total height.
- The effect of lateral load distribution, strong and weak panel zones (doubler plates) and P-delta are investigated separately in this example.

Static Pushover Analysis

Effect of Different Lateral Load Distribution

In this example, three different load patterns were initially considered:

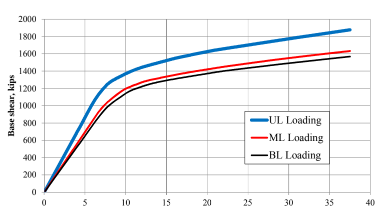
UL = Uniform load (equal force at each level)
ML = Modal load (lateral loads proportional to first mode shape)
BL = *Provisions* load distribution (Equivalent lateral forces used for preliminary analysis)

Lateral Load Patterns Used in Nonlinear Static Pushover Analysis

Level	Uniform Load UL (kips)	Modal Load ML (kips)	*Provisions* Load BL (kips)
R	15.0	85.1	144.8
6	15.0	77.3	114.0
5	15.0	64.8	84.8
4	15.0	49.5	58.2
3	15.0	32.2	34.6
2	15.0	15.0	15.0

Static Pushover Analysis

Effect of Different Lateral Load Distribution



Response of strong panel model to three load patterns, excluding P-delta effects

The *Provisions* states that the lateral load pattern should follow the shape of the first mode. (ML Loading)

Static Pushover Analysis

Static Pushover Curves with P-Delta Effects

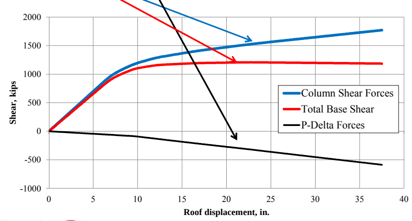
$$V = \sum_{i=1}^{n} V_{C,i} - \frac{P_1 \Delta_1}{h_1}$$

$\sum_{i=1}^{n} V_{C,i}$ = Sum of all column shears in 1st story

P_1 = Total vertical load on P-delta column

Δ_1 = P-delta column 1st story displacement

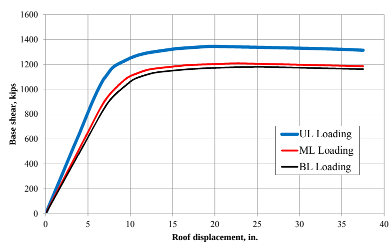
h_1 = 1st story height



Two base shear components of pushover response

Static Pushover Analysis

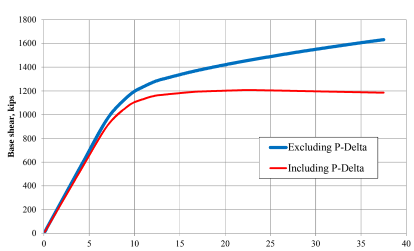
Effect of Different Lateral Load Distribution



Response of strong panel model to three load patterns, including P-delta effects

Static Pushover Analysis

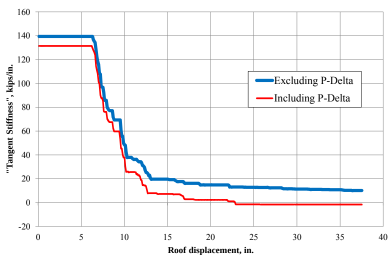
Effect of P-Delta on Pushover Curve



Response of strong panel model to ML loads, with and without P-delta effects

Static Pushover Analysis

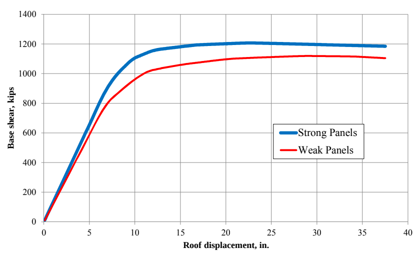
Effect of P-Delta on Pushover Curve



Tangent stiffness history for Strong Panel model under ML loads, with and without P-delta effects

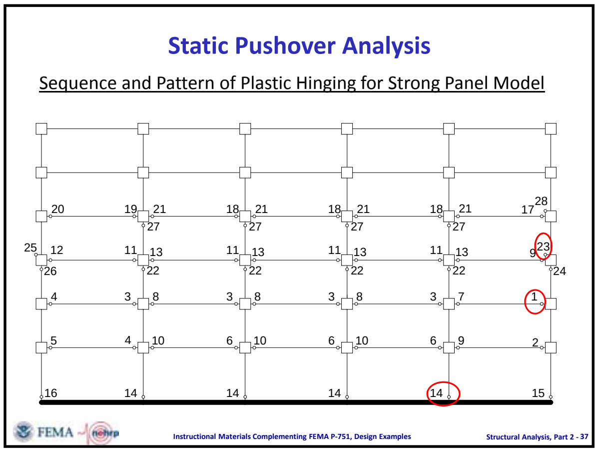
Static Pushover Analysis

Effect of Panel zones (Doubler Plates) on Pushover Curve



Comparison of weak panel zone model with strong panel zone model, both including P-delta effects

Static Pushover Analysis: Sequence and Pattern of Plastic Hinging with NonlinPro


Static Pushover Analysis
Sequence and Pattern of Plastic Hinging for Strong Panel Model
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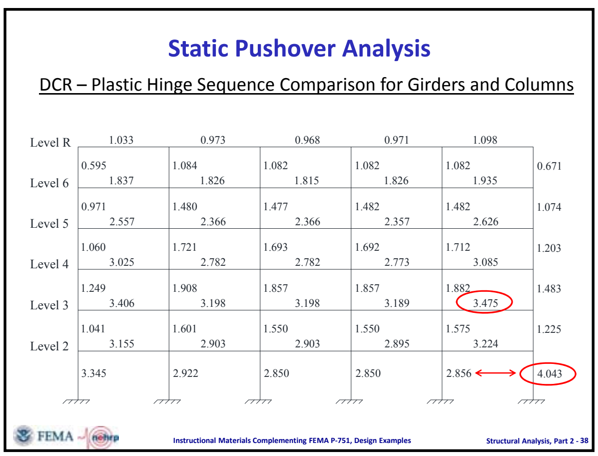
Structural Analysis, Part 2 - 37


Static Pushover Analysis
DCR – Plastic Hinge Sequence Comparison for Girders and Columns
Level R
Level 6
Level 5
Level 4
Level 3
Level 2
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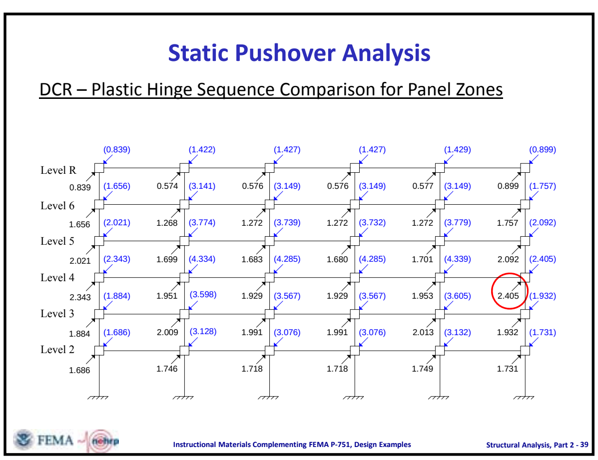
Static Pushover Analysis
DCR – Plastic Hinge Sequence Comparison for Panel Zones
Level R
Level 6
Level 5
Level 4
Level 3
Level 2
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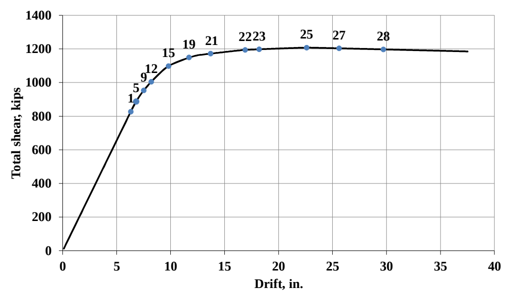
Static Pushover Analysis

Sequence and Pattern of Plastic Hinging for Strong Panel Model



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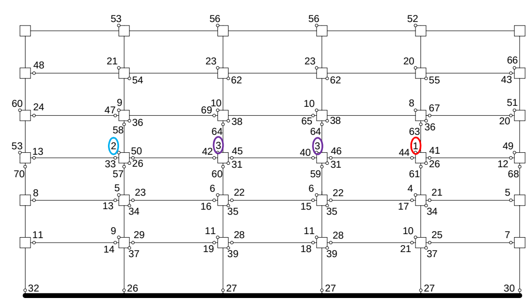
Static Pushover Analysis

Sequence and Pattern of Plastic Hinging for Weak Panel Model



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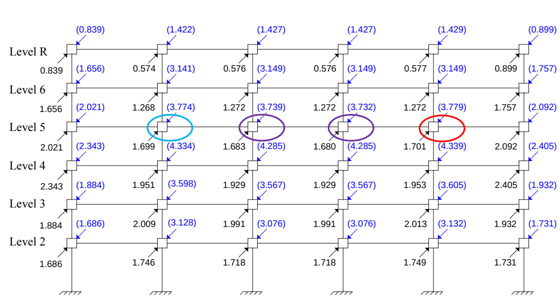
Static Pushover Analysis

DCR – Plastic Hinge Sequence Comparison for Panel Zones



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Static Pushover Analysis

Target Displacement

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$$

$C_0 = \phi_{1,r}\Gamma_1 =$ modification factor to relate spectral displacement of an equal single degree of freedom system to the roof displacement of the building multi-degree of freedom system.

$\phi_{1,r} =$ the ordinate of mode shape 1 at the roof (control node)

$\Gamma_1 =$ the first mode participation factor

$C_1 =$ modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

$C_2 =$ modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation and strength deterioration on maximum displacement response.

$S_a =$ response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration.

$T_e = T_i\sqrt{\frac{K_i}{K_e}} =$ effective fundamental period of the building in the direction under consideration

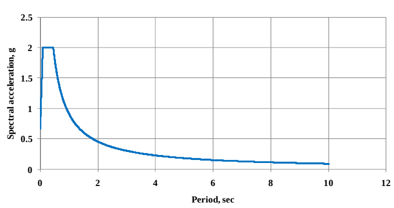
$T_i =$ elastic fundamental period in the direction under consideration calculated by elastic dynamic analysis.

$K_i, K_e =$ elastic, and effective lateral stiffness of the building in the direction under consideration.

$g =$ acceleration of gravity

Static Pushover Analysis

Target Displacement



2% damped horizontal response spectrum from ASCE 41-06

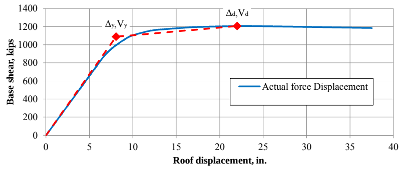
This spectrum is for BSE-2 (Basic Safety Earthquake 2) hazard level which has a 2% probability of exceedence in 50 years.

Static Pushover Analysis

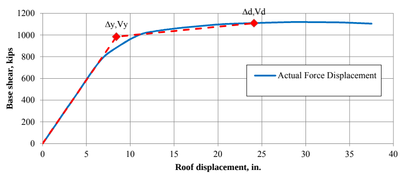
Target Displacement

- Nonlinear force-displacement relationship between base shear and displacement of control node shall be replaced with an idealized force-displacement curve. The effective lateral stiffness and the effective period depend on the idealized force-displacement curve.
- The idealized force-displacement curve is developed by using an iterative graphical procedure where the areas below the actual and idealized curves are approximately balanced up to a displacement value of Δ_d. Δ_d is the displacement at the end of second line segment of the idealized curve and V_d is the base shear at the same displacement.
- (Δ_d, V_d) should be a point on the actual force displacement curve at either the calculated target displacement, or at the displacement corresponding to the maximum base shear, whichever is the least.
- The first line segment of the idealized force-displacement curve should begin at the origin and finish at (Δ_y, V_y), where V_y is the effective yield strength and Δ_y is the yield displacement of idealized curve.
- The slope of the 1st line segment is equal to the effective lateral stiffness K_e which should be taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength of the structure.

Static Pushover Analysis



Actual and idealized force displacement curves for STRONG panel model, under ML load, with P-delta effects



Actual and idealized force displacement curves for WEAK panel model, under ML load, with P-delta effects

Static Pushover Analysis

Target displacement for strong and weak panel models

	Strong Panel	Weak Panel
C_0	1.303	1.310
C_1	1.000	1.000
C_2	1.000	1.000
S_a (g)	0.461	0.439
T_e (sec)	1.973	2.069
δ_t (in.) at Roof Level	22.9	24.1
Drift R-6 (in.)	0.96	1.46
Drift 6-5 (in.)	1.76	2.59
Drift 5-4 (in.)	2.87	3.73
Drift 4-3 (in.)	4.84	4.84
Drift 3-2 (in.)	5.74	5.35
Drift 2-1 (in.)	6.73	6.12

- Story drifts are also shown at the load level of target displacement.
- Negative stiffness starts after target displacements for both models.

Response History Analysis

Modeling and Analysis Procedure

- Response response history analysis method is used to estimate the inelastic deformation demands for the detailed structure.
- Three ground motions were used. (Seven or more ground motions is generally preferable.)
- The analysis considered a number of parameters, as follows:
  - Scaling of ground motions to the DBE and MCE level
  - With and without P-delta effects
  - Two percent and five percent inherent damping
  - Added linear viscous damping
- Identical structural model used in Nonlinear Pushover Analyses and 2nd order effects were included through the use of leaning column.
- All of the model analyzed had "Strong Panels" (wherein doubler plated were included in the interior beam-column joints).

Response History Analysis

Rayleigh Damping

- Rayleigh proportional damping was used to represent viscous energy dissipation in the structure.
- The mass and stiffness proportional damping factors were initially set to produce 2.0 percent damping in the first and third modes.
- It is generally recognized that this level of damping (in lieu of the 5 percent damping that is traditionally used in elastic analysis) is appropriate for nonlinear response history analysis.

$$C = \alpha M + \beta K \qquad \begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = \frac{2\xi}{w_1 + w_3} \begin{Bmatrix} w_1 w_3 \\ 1 \end{Bmatrix}$$

Structural frequencies and damping factors used in response history analysis.
(Damping factors that produce 2 percent damping in modes 1 and 3)

Model/Damping Parameters	ω_1 (rad/sec)	ω_3 (rad/sec)	α	β
Strong Panel with P-delta	3.184	18.55	0.109	0.00184
Strong Panel without P-delta	3.285	18.81	0.112	0.00181

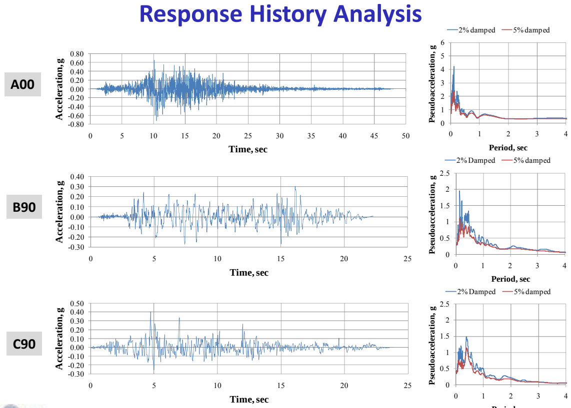
Response History Analysis

Development of Ground Motion Records

- Because only a two-dimensional analysis of the structure is performed using DRAIN, only a single component of ground motion is applied at one time.
- For the analyses reported herein, the component that produced the larger spectral acceleration at the structure's fundamental period was used.
- A complete analysis would require consideration of both components of ground motions, and possibly of a rotated set of components.

NGA Record Number	Magnitude, [Epicenter Distance (km)]	Site Class	Number of Points and Time step	Integration Time Step used in analyses	Component Source Motion	PGA (g)	Record Name
0879	7.28 , [44]	C	9625 @ 0.005 sec	0.0005 sec	Landers / LCN260	0.727	A00
0725	6.54 , [11.2]	D	2230 @ 0.01 sec	0.001 sec	SUPERST/ B-POE360	0.300	B90
0139	7.35 , [21]	C	1192 @ 0.02 sec	0.001 sec	TABAS/ DAY-TR	0.406	C90

Response History Analysis

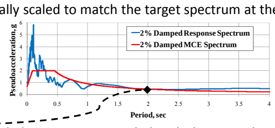


Response History Analysis

Ground Motion Scaling Procedure

1. Each spectrum is initially scaled to match the target spectrum at the structure's fundamental period.

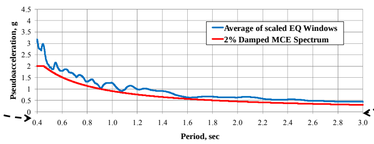
T₁=1.973 sec.



2. The average of the scaled spectra are re-scaled such that no ordinate of the scaled average spectrum falls below the target spectrum in the range of periods between 0.2 and 1.5*T*.

T_1=1.973 sec.

0.2*T_1



1.5*T_1

3. The final scale factor for each motion consists of the product of the initial scale factor (different for each ground motion), and the second scale factor (which is the same for each ground motion).

Results of Response History Analysis

DBE Results for 2% Damped Strong Panel Model with P-Δ Excluded / P-Δ Included

(a) Maximum Base Shear (kips)

	Motion A00	Motion B90	Motion C90
Column Forces	1780 / 1467	1649 / 1458	1543 / 1417
Inertial Forces	1848 / 1558	1650 / 1481	1540 / 1419

(b) Maximum Story Drifts (in.)

Level	Motion A00	Motion B90	Motion C90	Limit*
Total Roof	26.80 / 32.65	14.57 /14.50	13.55 / 14.75	NA
R-6	1.85 / 1.86	1.92 / 1.82	1.71 / 1.70	3.00 (3.75)
6-5	2.51 / 2.64	2.60 / 2.50	2.33 / 2.41	3.00 (3.75)
5-4	**3.75** / **4.08**	3.08 / 2.81	3.03 / 3.19	3.00 (3.75)
4-3	**5.62** / **6.87**	2.98 / 3.21	3.03 / 3.33	3.00 (3.75)
3-2	**6.61** / **8.19**	3.58 / 3.40	2.82 / 2.90	3.00 (3.75)
2-G	**8.09** / **10.40**	**4.68** / **4.69**	3.29 / 3.44	3.60 (4.50)

* Values in ( ) reflect increased drift limits provided by Sec. 16.2.4.3 of the *Standard*

Results of Response History Analysis

MCE Results for 2% Damped Strong Panel Model with P-Δ Excluded / P-Δ Included

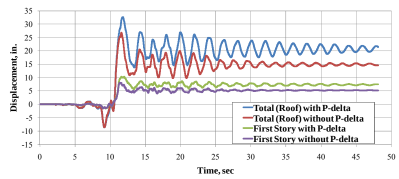
(a) Maximum Base Shear (kips)

	Motion A00	Motion B90	Motion C90
Column Forces	2181 / 1675	1851 / 1584	1723 / 1507
Inertial Forces	2261 / 1854	1893 / 1633	1725 / 1515

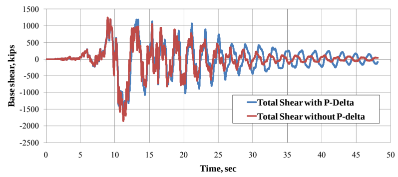
(b) Maximum Story Drifts (in.)

Level	Motion A00	Motion B90	Motion C90	Limit*
Total Roof	62.40 / 101.69	22.45 / 26.10	20.41 / 20.50	NA
R-6	1.98 / 1.95	2.30 / 2.32	3.05 / 2.93	4.50
6-5	3.57 / 2.97	2.77 / 2.60	3.69 / 3.49	4.50
5-4	**7.36** / **6.41**	3.33 / 3.62	4.43 / 4.32	4.50
4-3	**14.61** / **20.69**	**4.61** / **5.61**	4.45 / **4.63**	4.50
3-2	**16.29** / **31.65**	**5.21** / **6.32**	3.97 / 4.18	4.50
2-G	**19.76** / **40.13**	**6.60** / **7.03**	5.11 / 5.11	5.40

Results of Response History Analysis



Response Histories of Roof and First-story Displacement, Ground Motion A00 (DBE)



Response History of Total Base Shear, Ground Motion A00 (DBE)

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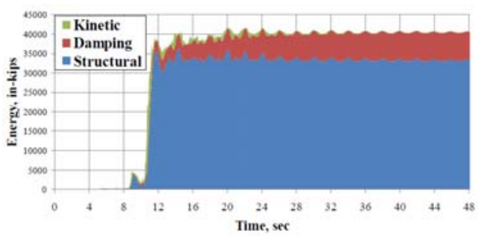
Results of Response History Analysis



Energy Response History, Ground Motion A00 (DBE), including P-delta effects

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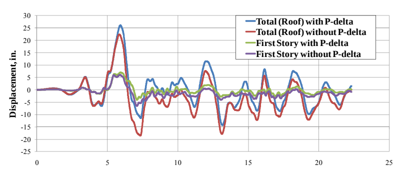
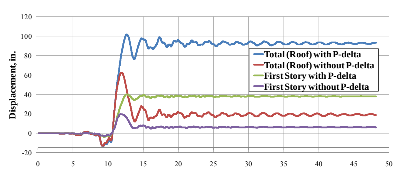
Results of Response History Analysis



Response Histories of Roof and First-story Displacement, Ground Motion B90 (MCE)



Response History of Roof and First-story Displacement, Ground Motion A00 (MCE)

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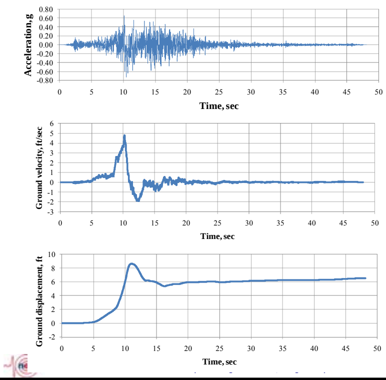
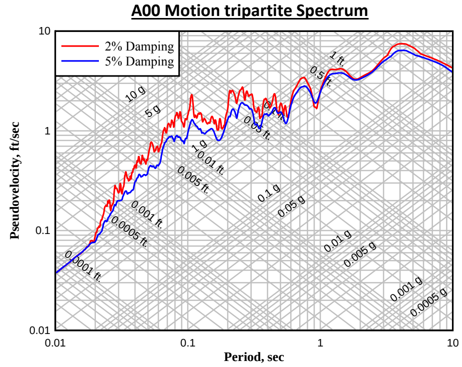
Response History Analysis

A00 Motion Ground Acceleration, Velocity and Displacement

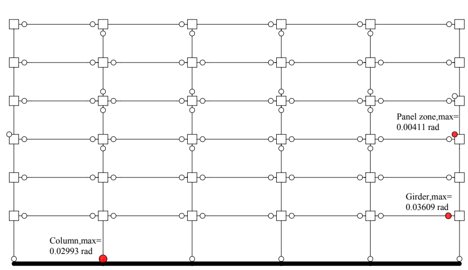


Response History Analysis

A00 Motion tripartite Spectrum



Results of Response History Analysis



Yielding locations for structure with strong panels subjected to MCE scaled B90 motion, including P-delta effects

Results of Response History Analysis

Comparison with Results from Other Analyses

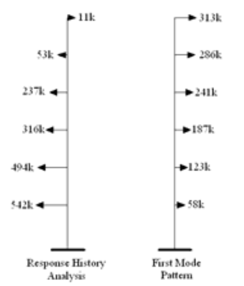
Response Quantity	Analysis Method: Equivalent Lateral Forces	Nonlinear Static Pushover	Nonlinear Dynamic
Base Shear (kips)	569	1208	1633
Roof Disp. (in.)	18.4	22.9	26.1
Drift R-6 (in.)	1.86	0.96	2.32
Drift 6-5 (in.)	2.78	1.76	2.60
Drift 5-4 (in.)	3.34	2.87	3.62
Drift 4-3 (in.)	3.73	4.84	5.61
Drift 3-2 (in.)	3.67	5.74	6.32
Drift 2-1 (in.)	2.98	6.73	7.03
Girder Hinge Rot. (rad)	NA	0.03304	0.03609
Column Hinge Rot. (rad)	NA	0.02875	0.02993
Panel Hinge Rot. (rad)	NA	0.00335	0.00411
Panel Plastic Shear Strain	NA	0.00335	0.00411

Note: Shears are for half of total structure.

Results of Response History Analysis

Reasons of the differences between Pushover and Response History Analyses

- Scale factor of 1.367 was used for the 2nd part of the scaling procedure.
- The use of the first-mode lateral loading pattern in the nonlinear static pushover response.
- The higher mode effects shown in the Figure are the likely cause of the different hinging patterns and are certainly the reason for the very high base shear developed in the response history analysis.



Comparison of inertial force patterns

Results of Response History Analysis

Effect of Increased Damping on Response

- Excessive drifts occur in the bottom three stories.
- Additional strength and/or stiffness should be provided at these stories.
- Considered next, Added damping is also a viable approach.
- Four different damper configurations were used.
- Dampers were added to the Strong Panel frame with 2% inherent damping.
- The structure was subjected to the DBE scaled A00 and B90 ground motions.
- P-delta effects were included in the analyses.

Modeling Added Dampers

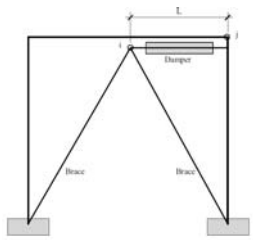
- Added damping is easily accomplished in DRAIN by use of the stiffness proportional component of Rayleigh damping.
- Linear viscous fluid damping device can be modeled through use of a Type-1 (truss bar) element.

$$k_{device} = \frac{A_{device} E_{device}}{L_{device}}$$

$$C_{device} = \beta_{device} k_{device}$$

- Set damper elastic stiffness to negligible value. k_{device} = 0.001 kips/in.

$$\beta_{device} = \frac{C_{device}}{0.001} = 1000 C_{device}$$



Modeling a simple damper

- It is convenient to set E_{device} = 0.001 and A_{device} = Damper length L_{device}

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Results of Response History Analysis

Effect of Increased Damping on Response

Effect of different added damper configurations when SP model is subjected to DBE scaled A00 motion, including P-delta effects

	No Damper	1st combo		2nd combo		3rd combo		4th combo		
Level	Drift, In.	Damper Coeff, kip-sec/in.	Drift, in.	Damper Coeff, kip-sec/in.	Drift, in.	Damper Coeff, kip-sec/in.	Drift, in.	Damper Coeff, kip-sec/in.	Drift, in.	Drift Limit in.
R-6	1.86	10.5	1.10	60	1.03	-	1.82	-	1.47	3.75
6-5	2.64	33.7	1.90	60	1.84	-	3.56	-	2.41	3.75
5-4	**4.08**	38.4	2.99	70	2.88	-	**4.86**	56.25	3.46	3.75
4-3	**6.87**	32.1	**5.46**	70	**4.42**	-	**5.24**	56.25	**4.47**	3.75
3-2	**8.19**	36.5	**6.69**	80	**5.15**	160	**4.64**	112.5	**4.76**	3.75
2-G	**10.40**	25.6	**8.39**	80	**5.87**	160	4.40	112.5	**4.96**	4.50
Column Base Shear,kips	1467	1629		2170		2134		2267		
Inertial Base Shear,kips	1558	1728		2268		2215		2350		
Total Damping,%	2	10.1		20.4		20.2		20.4		

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Results of Response History Analysis

Effect of Increased Damping on Response

Effect of different added damper configurations when SP model is subjected to DBE scaled B90 motion, including P-delta effects

	No Damper	1st combo		2nd combo		3rd combo		4th combo		
Level	Drift, In.	Damper Coeff, kip-sec/in.	Drift, in.	Damper Coeff, kip-sec/in.	Drift, in.	Damper Coeff, kip-sec/in.	Drift, in.	Damper Coeff, kip-sec/in.	Drift, in.	Drift Limit in.
R-6	1.82	10.5	1.11	60	0.86	-	1.53	-	1.31	3.75
6-5	2.50	33.7	1.76	60	1.35	-	2.11	-	1.83	3.75
5-4	2.81	38.4	2.33	70	1.75	-	2.51	56.25	2.07	3.75
4-3	3.21	32.1	2.67	70	2.11	-	2.37	56.25	2.16	3.75
3-2	3.40	36.5	2.99	80	2.25	160	2.09	112.5	2.13	3.75
2-G	**4.69**	25.6	3.49	80	1.96	160	1.87	112.5	1.82	4.50
Column Base Shear,kips	1458	1481		1485		1697		1637		
Inertial Base Shear,kips	1481	1531		1527		1739		1680		
Total Damping,%	2	10.1		20.4		20.2		20.4		

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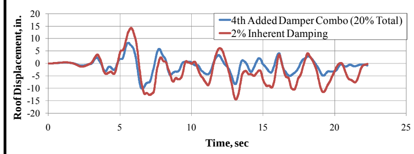
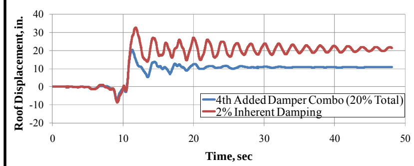
Results of Response History Analysis : Roof Displacements

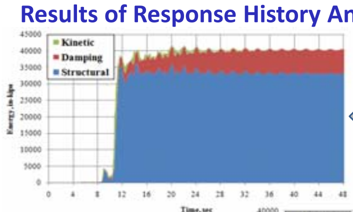


Roof Displacement Response Histories with added damping (20% total) and inherent damping (2%) for B90 motion

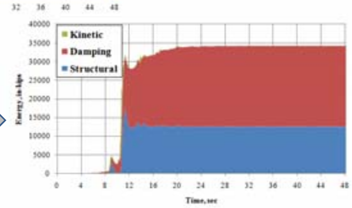


Roof Displacement Response Histories with added damping (20% total) and inherent damping (2%) for A00 motion

Results of Response History Analysis: Energy Plots

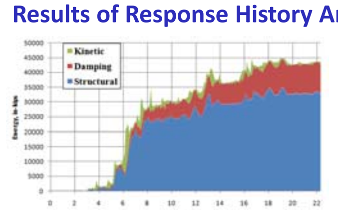


Energy Response History with inherent damping (2% total damping) for A00 motion

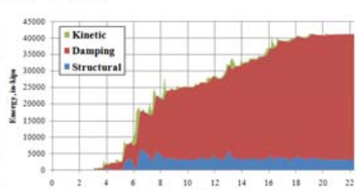


Energy Response History with added damping of 4th combination (20% total damping) for A00 motion

Results of Response History Analysis: Energy Plots

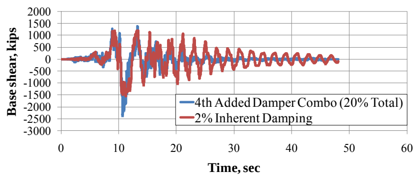


Energy Response History with inherent damping (2% total damping) for B90 motion

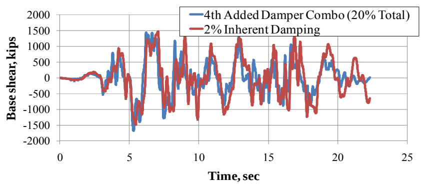


Energy Response History with added damping of 4th combination (20% total damping) for B90 motion

Results of Response History Analysis: Base Shear

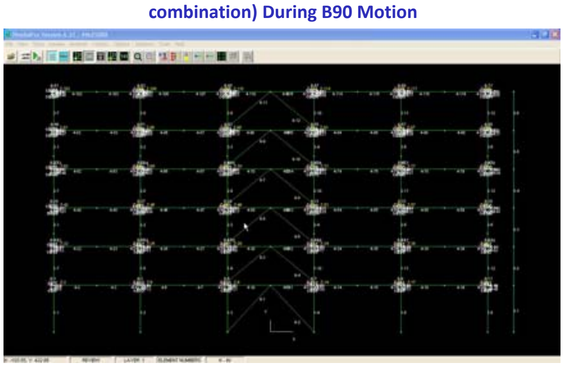


Inertial Base Shear Response Histories with added damping (20% total) and inherent damping (2%) for A00 motion



Inertial Base Shear Response Histories with added damping (20% total) and inherent damping (2%) for B90 motion

Results of Response History Analysis: Deflected Shape of by NonlinPro for Added Damper Frame (4th combination) During B90 Motion



Summary and Conclusions

- Five different analytical approaches were used to estimate the deformation demands in a simple unbraced steel frame structure:
  1. Linear static analysis (the equivalent lateral force method)
  2. Plastic strength analysis (using virtual work)
  3. Nonlinear static pushover analysis
  4. Linear dynamic analysis
  5. Nonlinear dynamic response history analysis
- Approaches 1, 3, and 5 were carried to a point that allowed comparison of results. The results obtained from the three different analytical approaches were quite dissimilar.
- Because of the influence of the higher mode effects on the response, pushover analysis, where used alone, is inadequate.
- Except for preliminary design, the ELF approach should not be used in explicit performance evaluation as it has no mechanism for determining location and extent of yielding in the structure.
- Response history analysis as the most viable approach. However, significant shortcomings, limitations, and uncertainties in response history analysis still exist.
- In modeling the structure, particular attention was paid to representing possible inelastic behavior in the panel-zone regions of the beam-column joints.

Questions?