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Engineering of Structures
and Building Enclosures

CASE STUDY:
40 STORY BRBF BUILDING
LOS ANGELES

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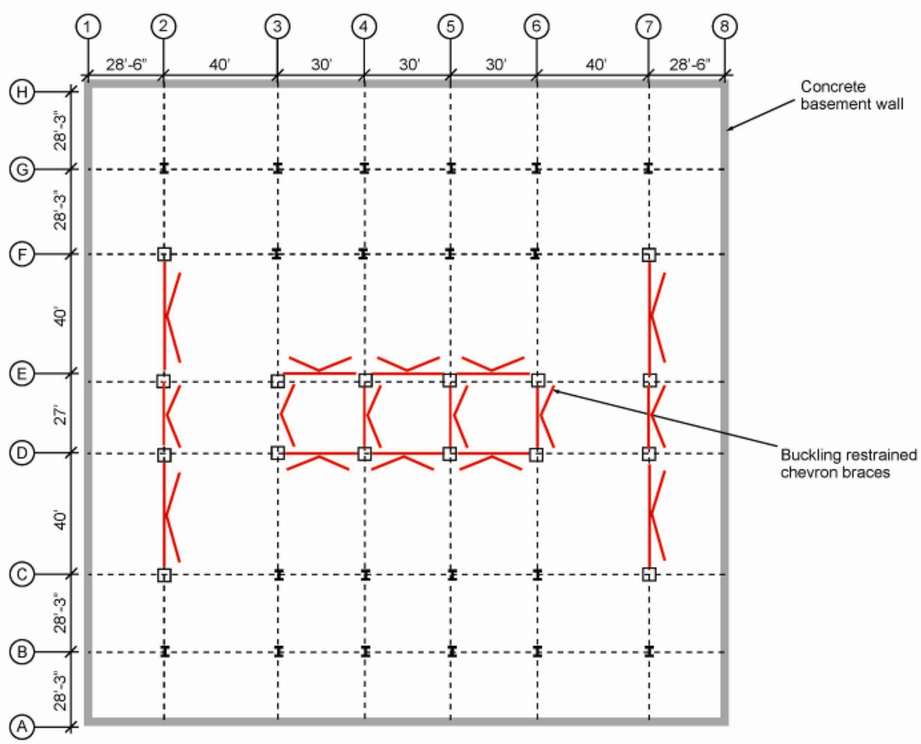
Criteria

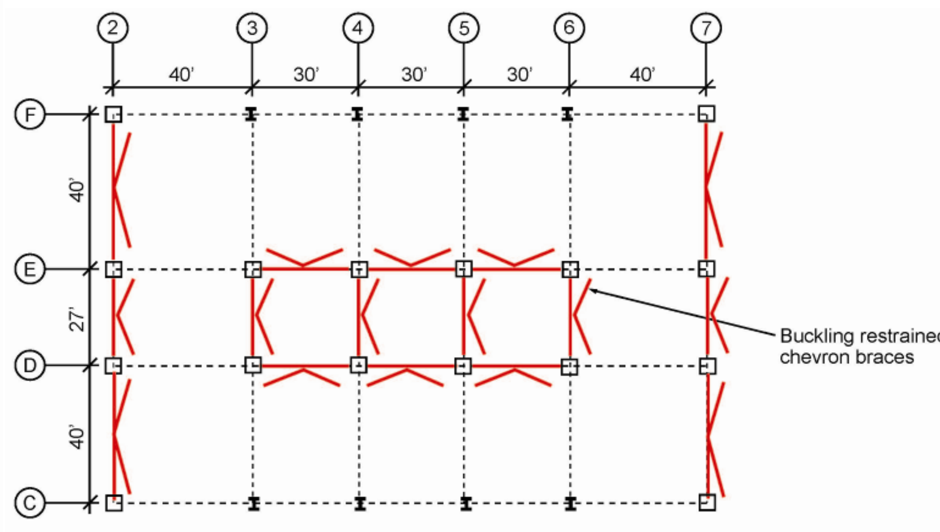
- **Three separate criteria:**
  - **CODE DESIGN**
  - **PERFORMANCE-BASED DESIGN**
    - LATBC criteria
  - **PERFORMANCE +**
    - PEER TBI Guidelines

Building Description

- **Approximate building floor plan**
  - Tower: 170 ft X 107 ft
  - Podium
    - four levels of basement
    - plan dimensions of 227 ft X 220 ft



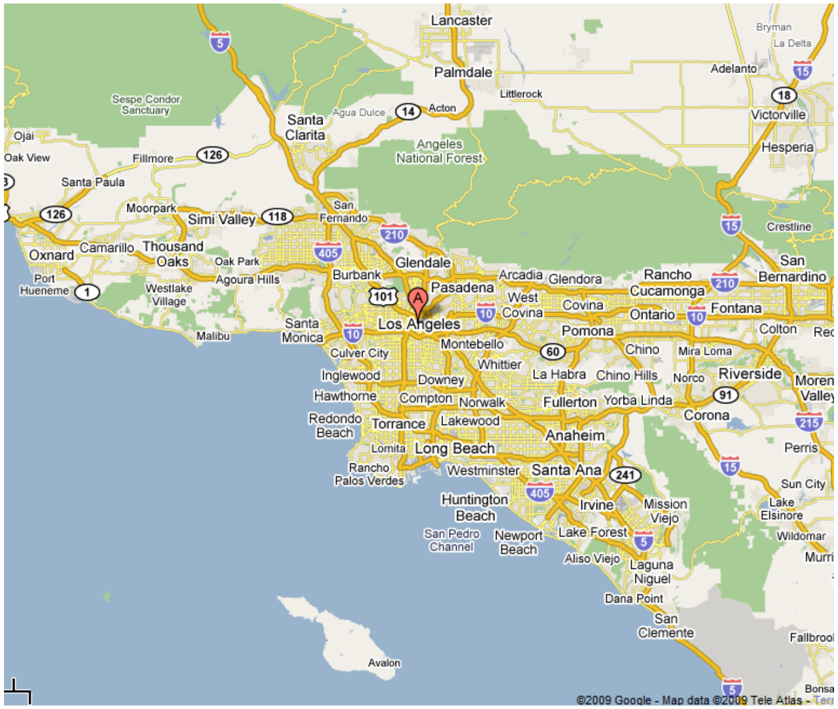
Plan at Ground Floor and Basement



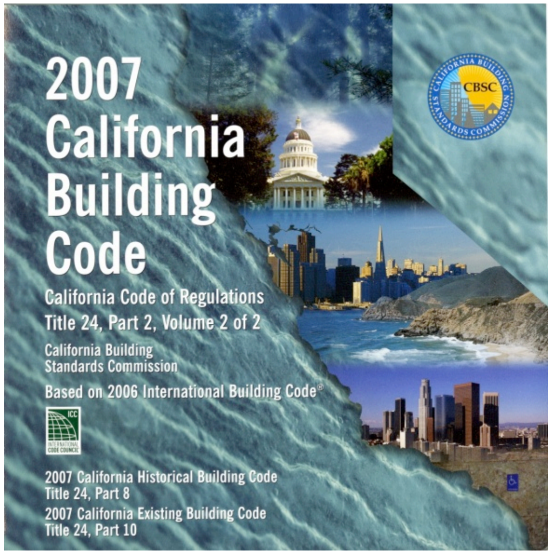
Plan at First through Tenth Floors

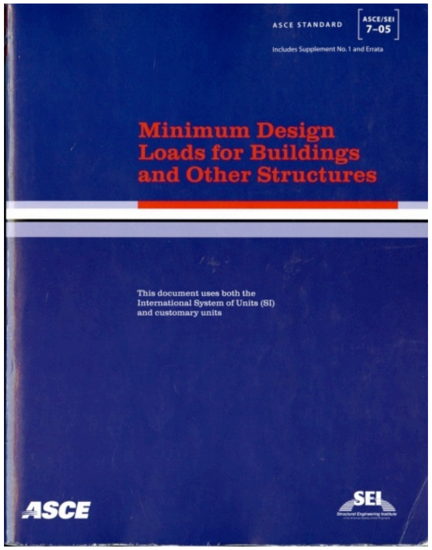
Code Design

- Building located in downtown Los Angeles with S_{DS} = 1.145 and S_{D1} = 0.52



- Design follows all applicable building code and standard provisions





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Code Design – Contd.

- **Height limitation ignored**

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

Seismic Force–Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, $\Omega_0{}^g$	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]
22. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
23. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1, 14.1.4.2, and 14.5	7	2½	4½	NL	NL	65	65	65
24. Light-framed walls with shear panels of all other materials	14.1, 14.1.4.2, and 14.5	2½	2½	2½	NL	NL	35	NP	NP
25. Buckling-restrained braced frames, non-moment-resisting beam-column connections	14.1	7	2	5½	NL	NL	160	160	100
26. Buckling-restrained braced frames, moment-resisting beam-column connections	14.1	8	2½	5	NL	NL	160	160	100
27. Special steel plate shear wall	14.1	7	2	6	NL	NL	160	160	100

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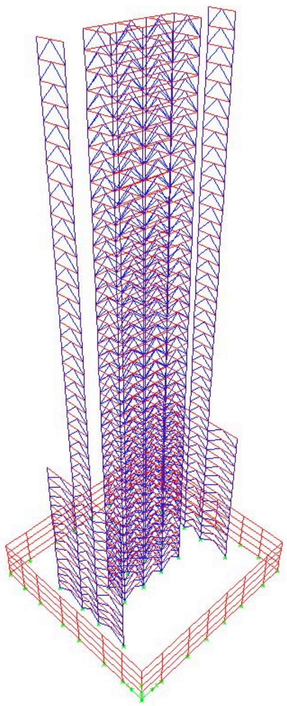
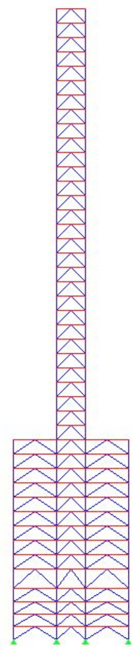
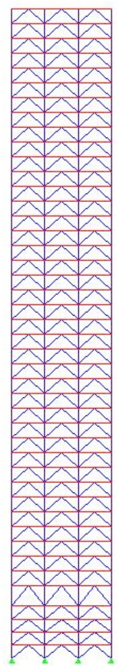
Code Design

- **Gravity framing sized in RAM Structural System**

- **Lateral Analysis and Design performed in ETABS using 3D response spectrum analysis**







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Gravity Loading

Description/Location	Superimposed Dead	Live Load	Reducable
Roof	28 psf	25 psf	Yes
Mechanical, Electrical at Roof	Total of 100 kips	-	-
Residential including Balconies	28 psf	40 psf	Yes
Corridors, Lobbies and Stairs	28 psf	100 psf	No
Retail	110 psf	100 psf	No
Parking Garage, Ramp	3 psf	40 psf[1]	Yes
Construction Loading	3 psf	30 psf	No
Cladding	15 psf	-	-
PEER document showed 50 psf. SGH considered 40 psf in keeping with ASCE 7-05			

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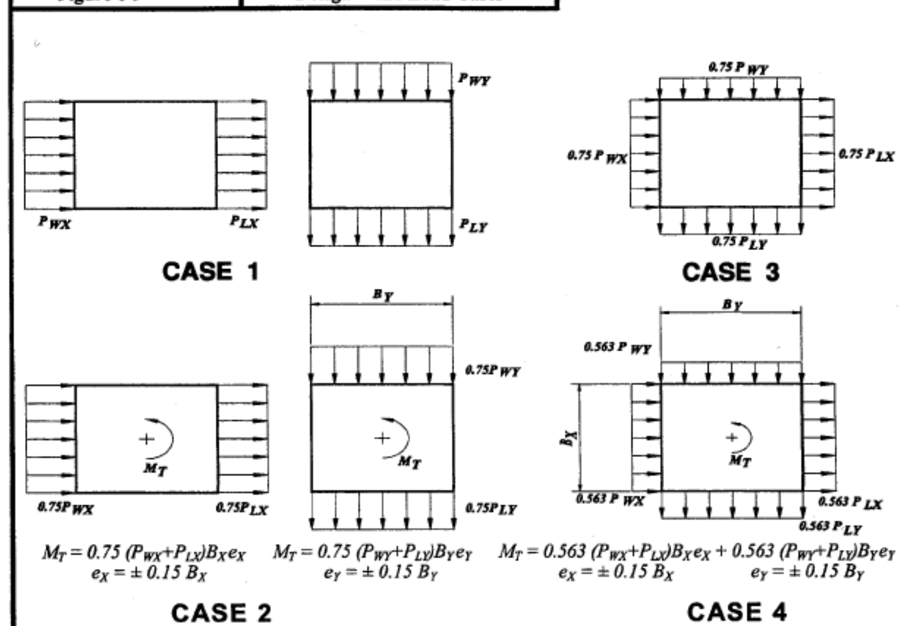
Wind Design

- **ASCE 7-05 Method 2**
- **Application of horizontal X and Y pressures in combination with torsion**
- **Gust factor (G_f) computed using 6.5.8.2 for dynamically sensitive structures with 1% damping**



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

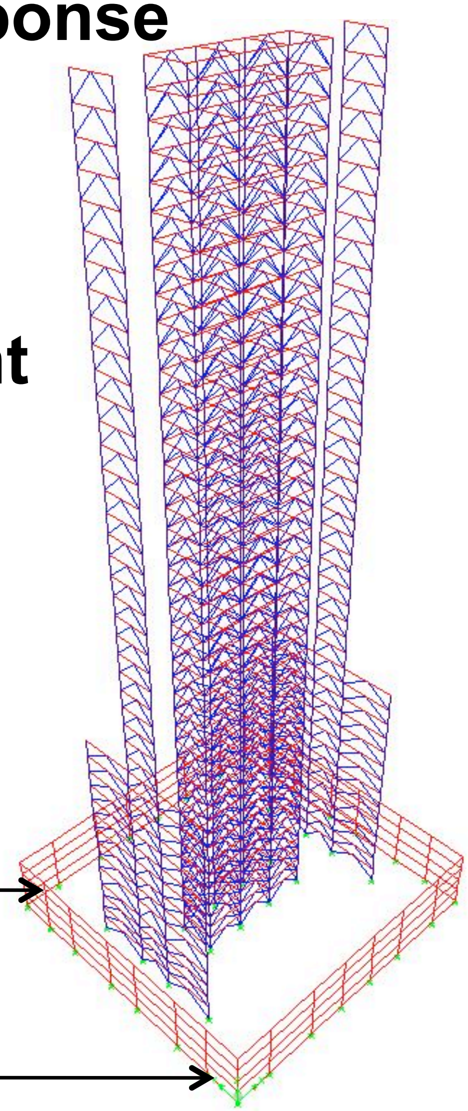
Wind Design

Parameter	Value
Basic Wind Speed, 3 sec. gust (V)	85 mph
Basic Wind Speed, 3 sec gust (V), for serviceability wind demands based on a 10 year mean recurrence interval	67 mph
Exposure	B
Occupancy Category	II
Importance Factor (I_w)	1.0
Topographic Factor (K_{zt})	1.0
Exposure Classification	Enclosed
Internal Pressure Coefficient (GC_{pi})	± 0.18
Mean Roof Height (h)	544’-6”
Wind Base Shear along Two Orthogonal Directions	1436 kips and 2629 kips

- **Wind loads were statically applied in ETABS and brace forces computed**

Seismic Design

- **Seismic analysis performed using the response spectrum provided by PEER**
- **Base Shear scaled to 85% of the static lateral base shear obtained from equivalent static lateral force analysis**
- **Base Shear is the story shear immediately above podium**



Basement walls and floor masses modeled

Restraint provided only at wall base

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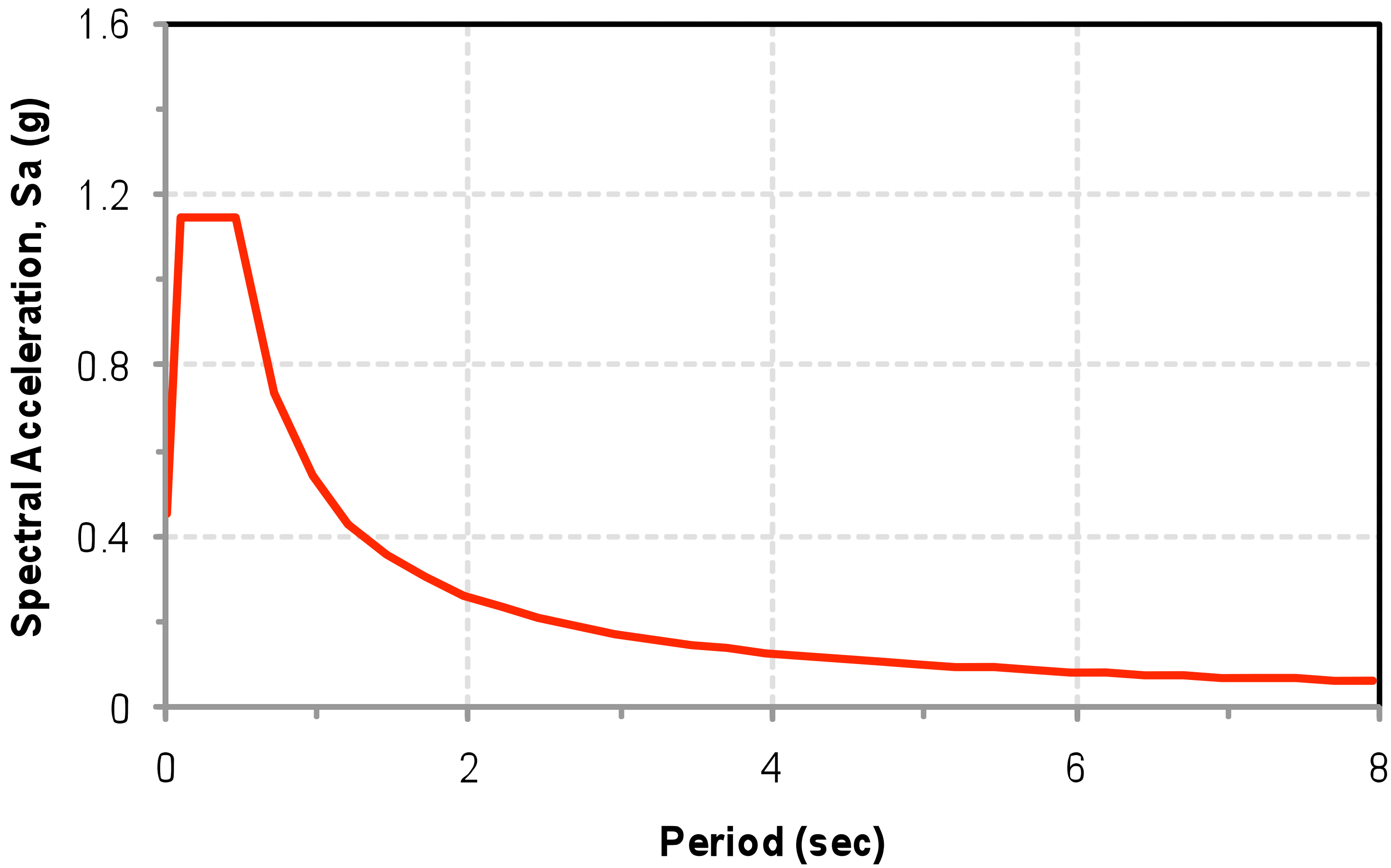
Design Spectrum



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Seismic Design

Parameter	Value
Building Latitude/Longitude	Undefined
Occupancy Category	II
Importance Factor (I_e)	1.0
Spectral Response Coefficients	S_{DS} = 1.145; S_{D1} = 0.52
Seismic Design Category	D
Lateral System	Buckling restrained braced frames, non moment resisting beam column connections
Response Modification Factor (R)	7
Deflection Amplification Factor (C_d)	5.5
System Overstrength Factor (Ω_0)	2.0
Building Period (T) using Cl. 12.8.2	3.16 sec[1]
Seismic Response Coefficient C_s (Eq. 12.8-1)	0.051 W (Governed by $C_{s\text{-}min}$ from Eq. 12.8-5)
Scaled Spectral Base Shear	3504 kips (85% of Static Base Shear)
Analysis Procedure	Modal Response Spectral Analysis

1. Actual period from dynamic model: T_Y = 5.05 sec; T_X = 3.62 sec

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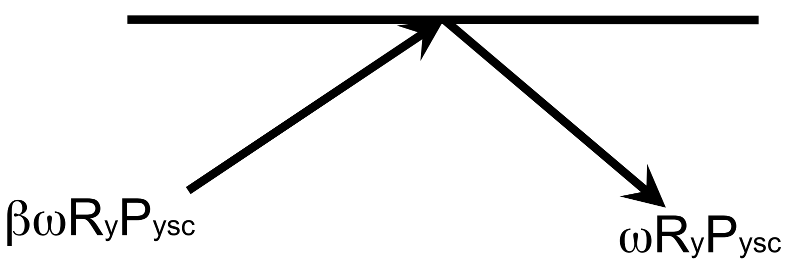
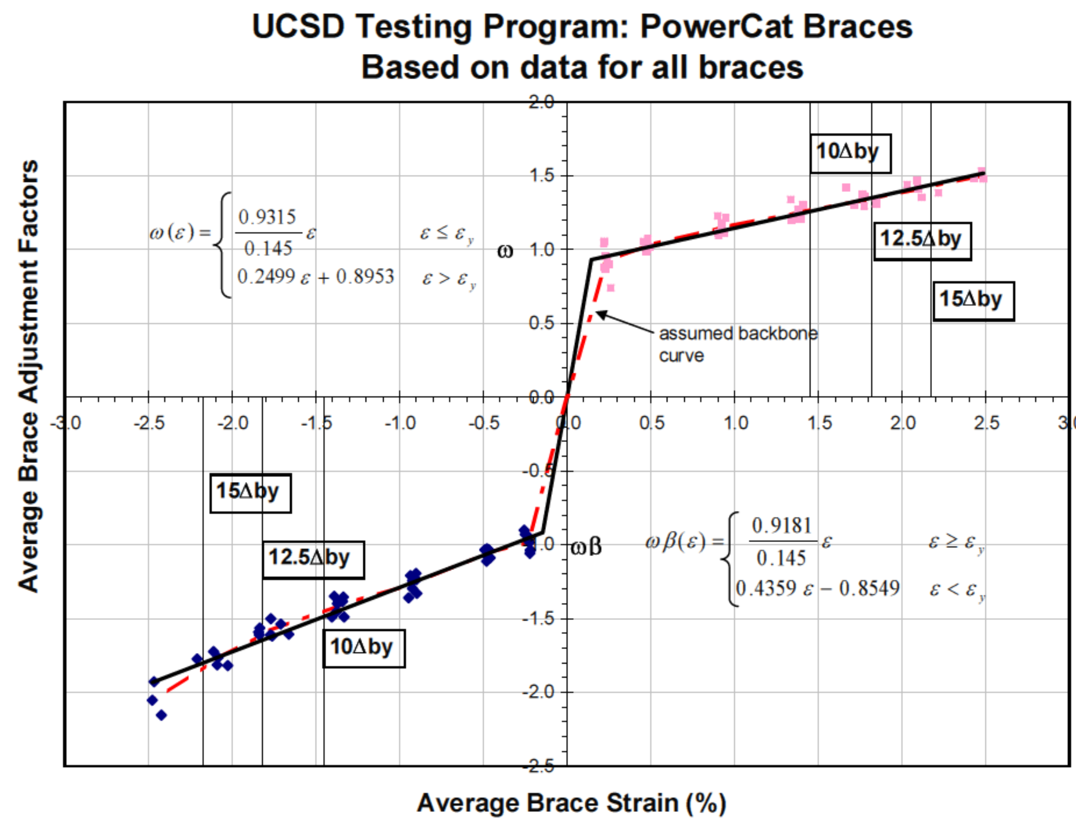
Member Design

- **Member design performed using ANSI/AISC 341-05**

- **Beams designed for unbalanced force corresponding to adjusted brace strength**



Assumed ω = 1.25, β = 1.1
Ry =1.1 and Fy = 38 ksi



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Member Design

- Columns designed for accumulated force (sum of vertical components)corresponding to adjusted brace strengths

- Led to large compression and tension design forces for columns and foundations (Note: Attachment of columns to foundations needs to be designed for same forces used for column design)

8.5a Required Axial Strength

The required axial strength of column bases, including their attachment to the foundation, shall be the summation of the vertical components of the required strengths of the steel elements that are connected to the column base.

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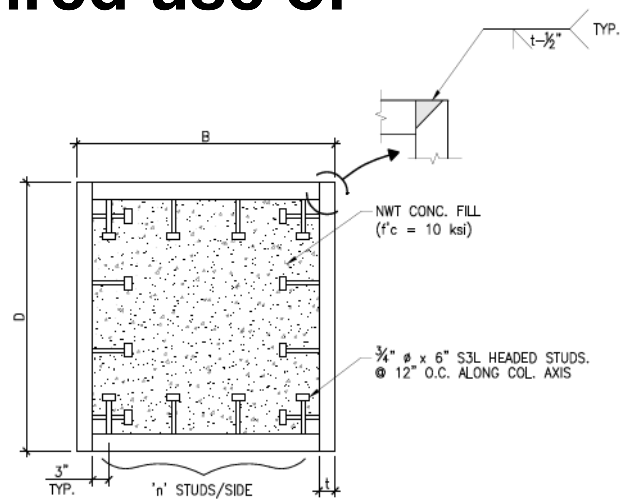
Member Design

- **Accommodation of the large forces required use of steel box sections filled with concrete**

- **Upside: Using Chapter I of AISC 13th Ed. a composite EIeff can be used. This contributed significantly to the lateral stiffness.**

- **Braced frame beams were sized for horizontal adjusted brace forces and unbalanced loading.**

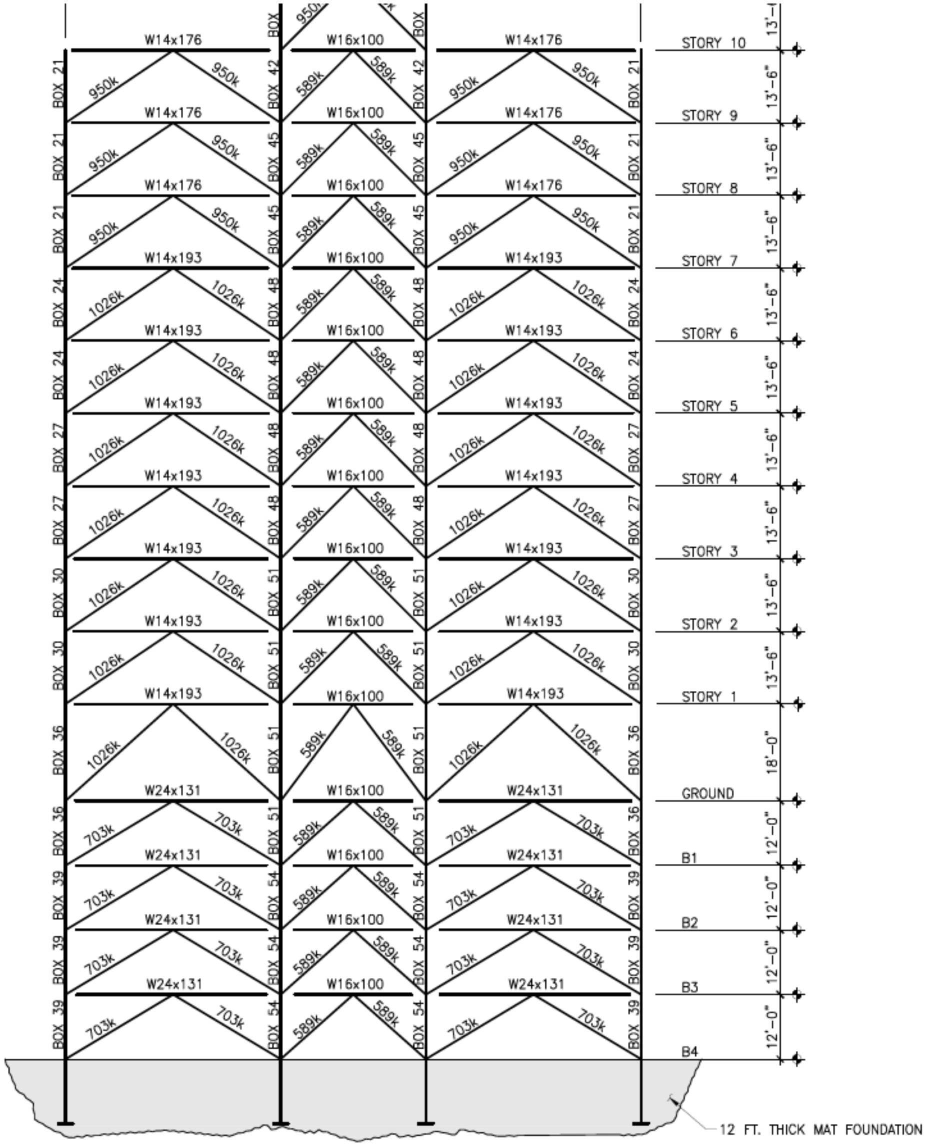


	B (IN)	D (IN)	t (IN)	n
BOX 18	18	18	1½	2
BOX 21	18	18	1½	2
BOX 24	24	24	2	2
BOX 27	27	27	2	3
BOX 30	30	30	2½	3
BOX 33	33	33	2½	3
BOX 36	36	36	2½	3
BOX 39	39	39	3	3
BOX 42	42	42	3	4
BOX 45	45	45	3	4
BOX 48	48	48	3	4
BOX 51	51	51	3	5
BOX 54	54	54	3	5
BOX 57	57	57	3	5
BOX 60	60	60	3	5

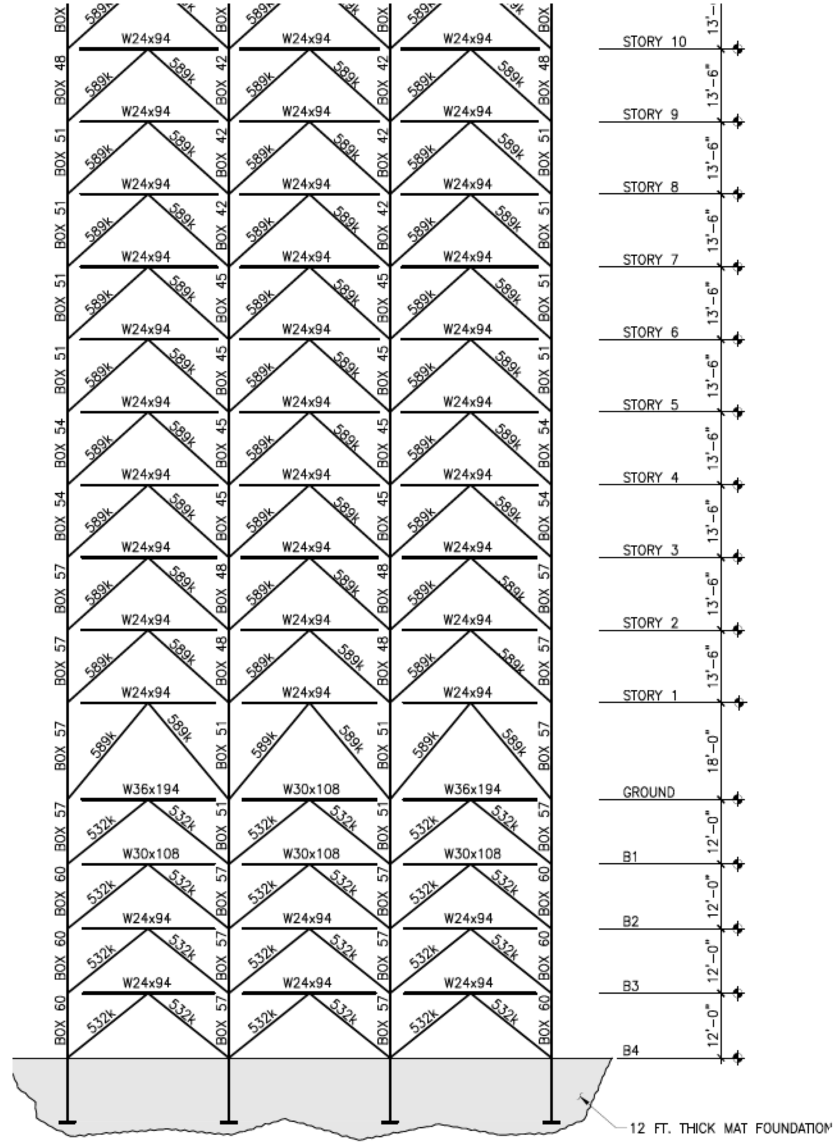
2 BOX COLUMN SCHEDULE

SCALE: NONE

Typical Member Sizes



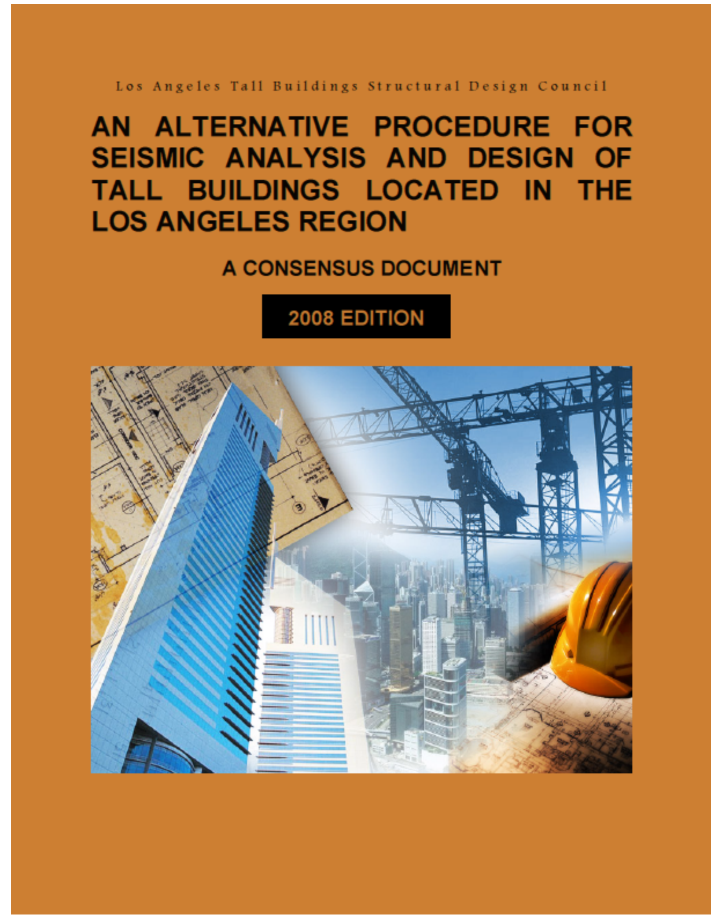
Transverse Frame (Below 10th Floor)



Longitudinal Frame (Below 10th Floor)

LATBC-Performance Based Design

- **Wind and Gravity Design per code.**
- **Seismic Design**
  - Service level design
    - 2.5%-damped 25-year event
    - Essentially elastic behavior
    - Maximum drift of 0.005
  - MCE Verification
    - Nonlinear response history analysis used to verify adequacy for "collapse prevention" performance



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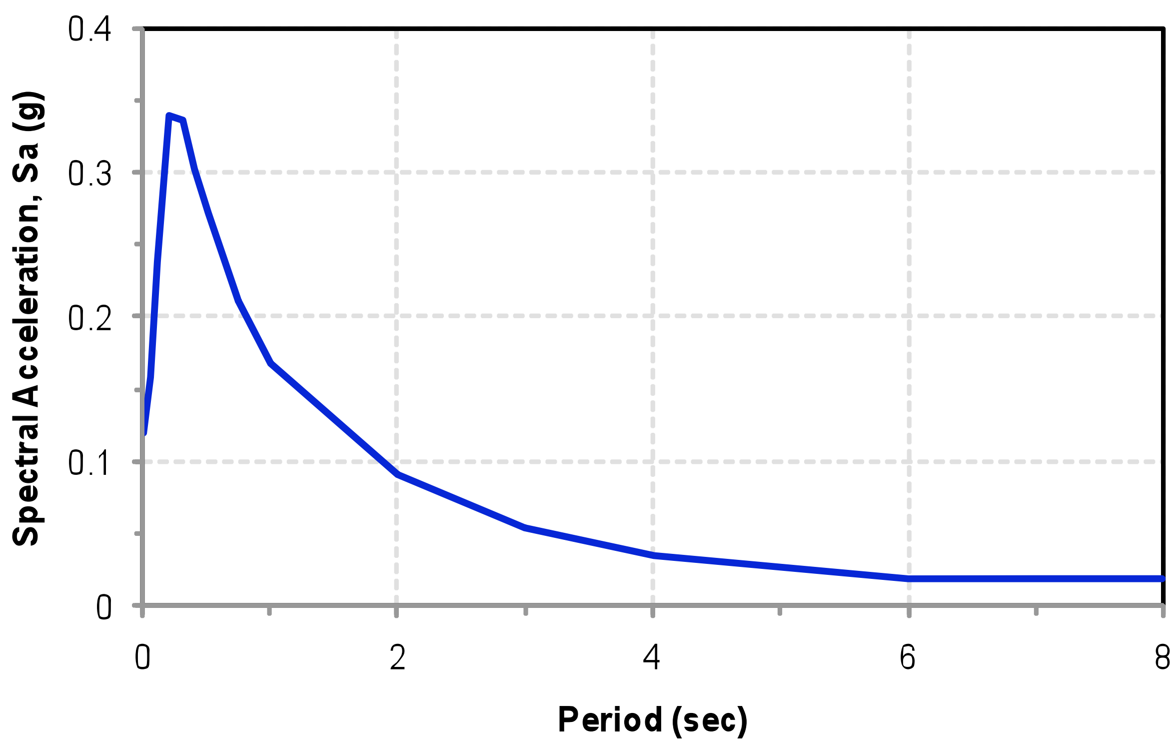
LATBC Design – Service Level

- **Used linear response spectrum analysis in ETABS. Max drift was 0.34%(<0.5%)**



- **<u>Brace sizes governed by wind design</u>**

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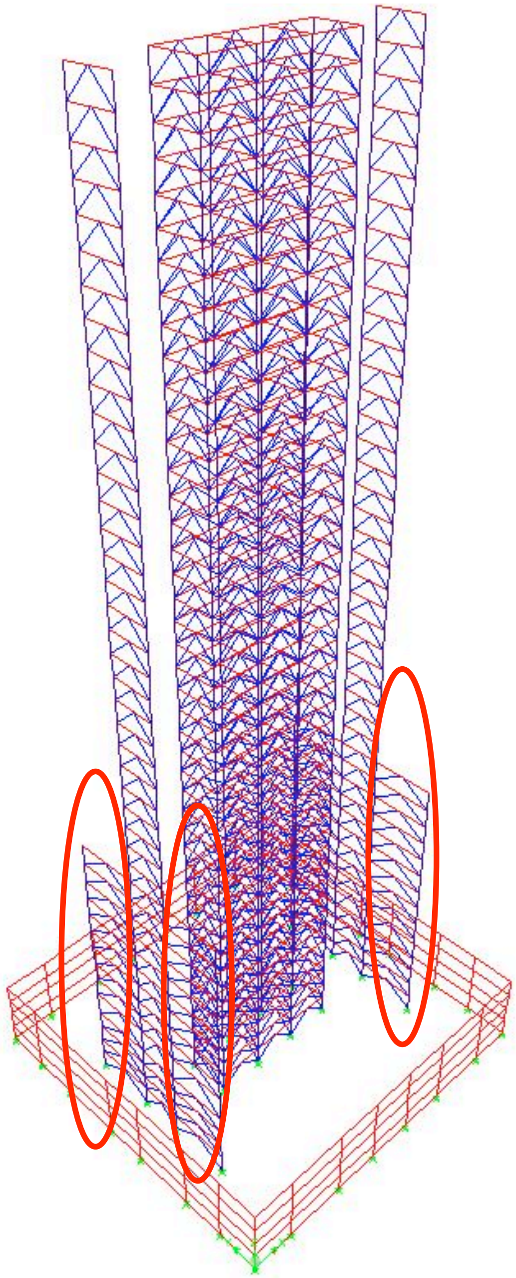
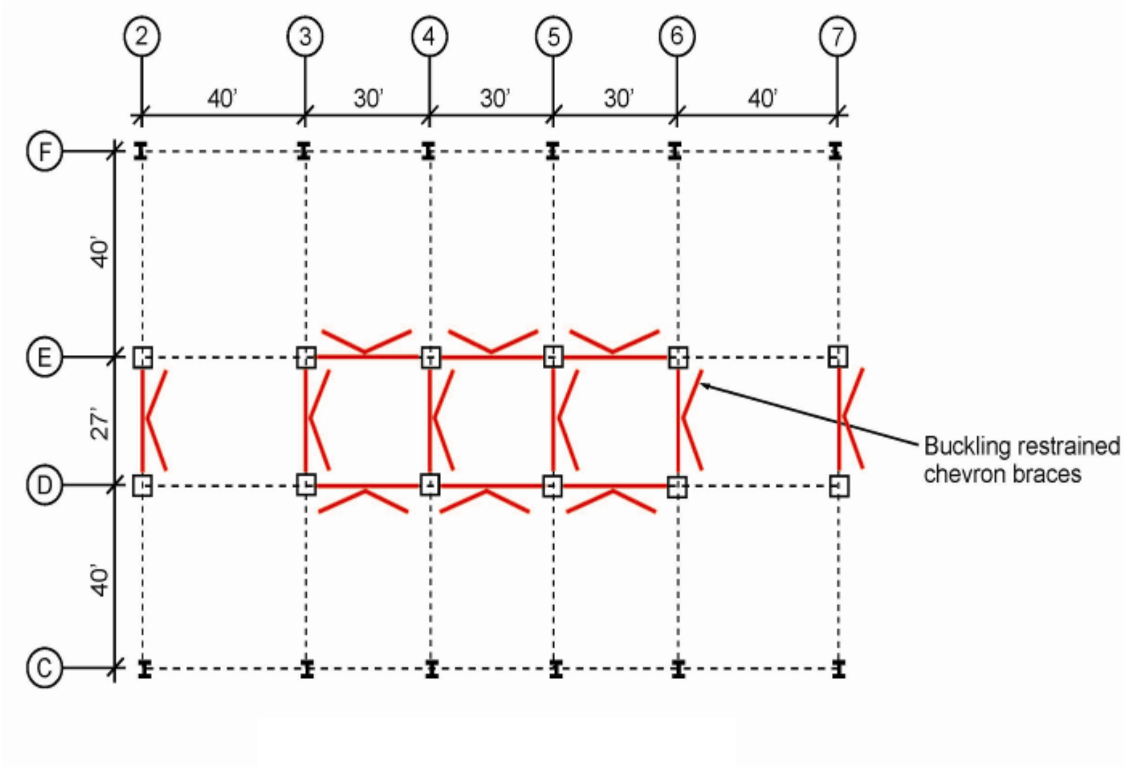
LATBC Design - Findings

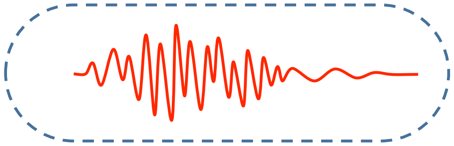


- **Member sizes more economical.**
- **Additional bays required in the transverse direction below 10th floor eliminated.**



LATBC Design- MCE Analysis

- Non linear response history analysis performed using CSI Perform (Tx = 6.5s, Ty = 4.5s)
- 7 ground motion pairs provided by PEER



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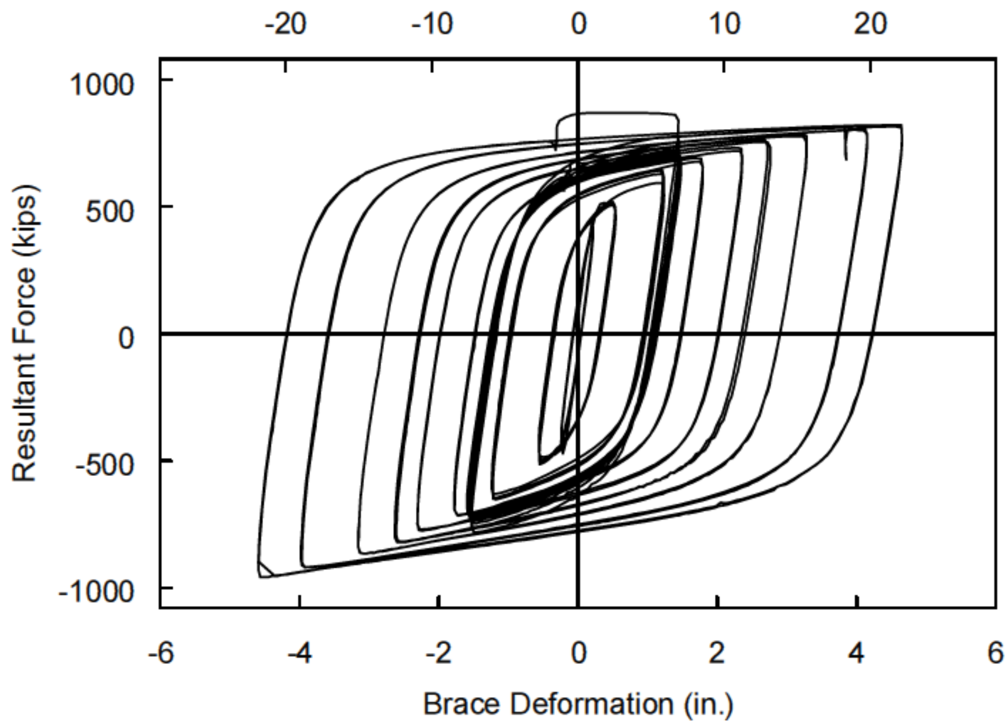
LATBC Design – MCE Acceptance Criterion

- **Acceptance based on mean demands from 7 analyses**
  - 3% maximum interstory drift
  - BRBs limited strain to 10 times yield (~0.013) based on observance of data from a large number of tests.



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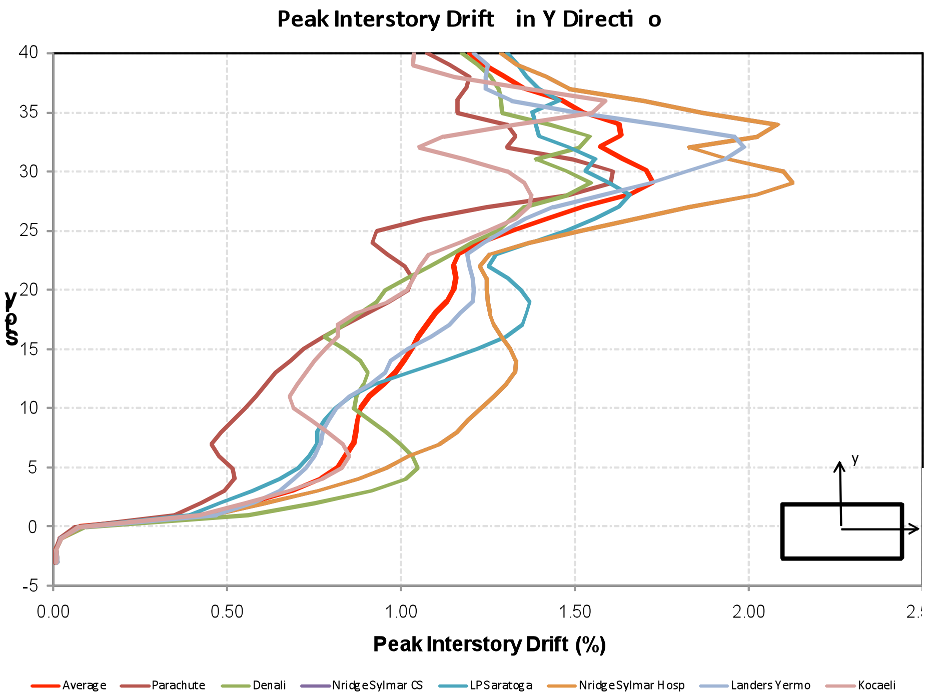
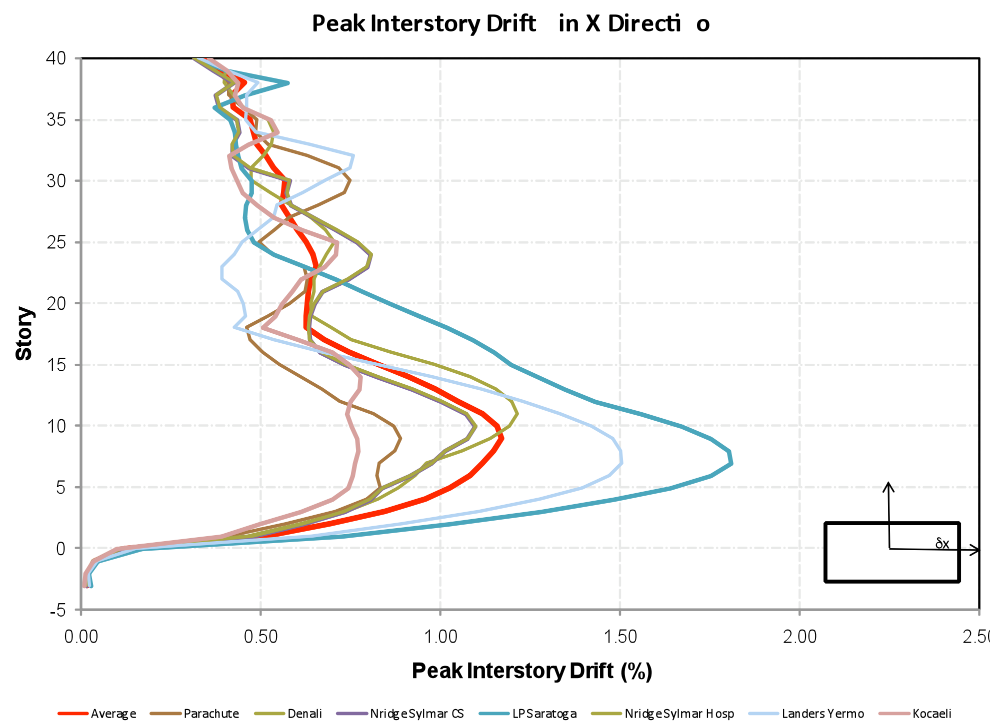
LATBC Design – MCE Story Drift





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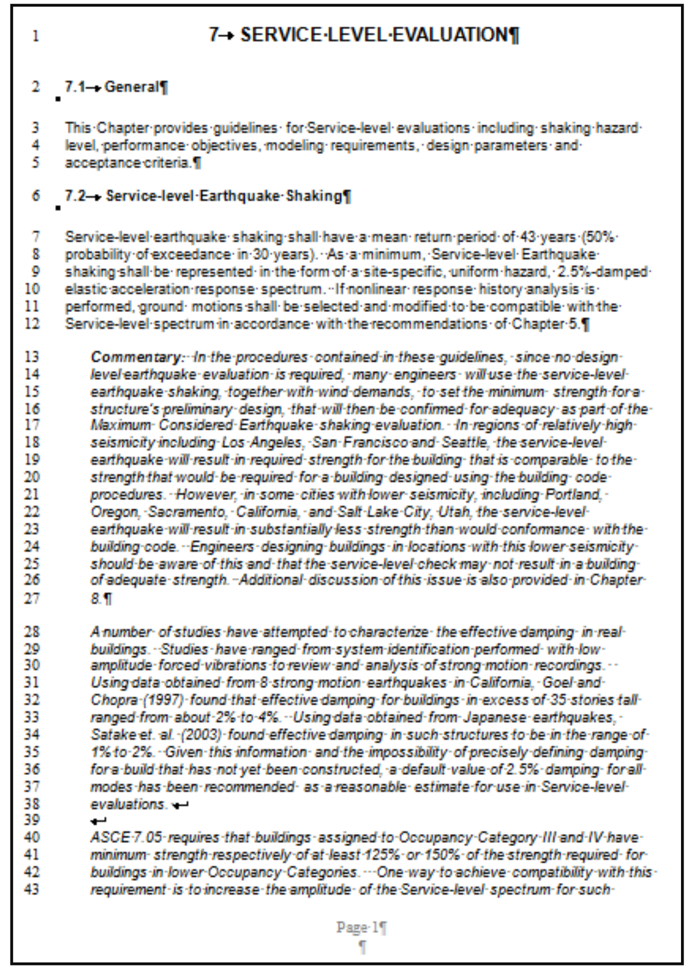
PEER TBI- Performance "+"

- **Wind and Gravity Design follow code.**
- **Seismic Design**
  - Service level – 2.5% damped 43-year spectrum
    - Essentially elastic performance
    - Drift limited to 0.005
  - MCE level
    - Max transient drift <0.03 average 0.045 any single run
    - Max residual drift < 0.01 average 0.015 any single run
    - BRB's respond within range of acceptable modeling



7 SERVICE LEVEL EVALUATION

7.1 General

This Chapter provides guidelines for Service-level evaluations including shaking hazard level, performance objectives, modeling requirements, design parameters and acceptance criteria.

7.2 Service-level Earthquake Shaking

Service-level earthquake shaking shall have a mean return period of 43 years (50% probability of exceedance in 30 years). As a minimum, Service-level Earthquake shaking shall be represented in the form of a site-specific, uniform hazard, 2.5%-damped elastic acceleration response spectrum. If nonlinear response history analysis is performed, ground motions shall be selected and modified to be compatible with the Service-level spectrum in accordance with the recommendations of Chapter 5.

Commentary: *In the procedures contained in these guidelines, since no design level earthquake evaluation is required, many engineers will use the service-level earthquake shaking, together with wind demands, to set the minimum strength for a structure's preliminary design, that will then be confimed for adequacy as part of the Maximum Considered Earthquake shaking evaluation. In regions of relatively high seismicity including Los Angeles, San Francisco and Seattle, the service-level earthquake will result in required strength for the building that is comparable to the strength that would be required for a building designed using the building code procedures. However, in some cities with lower seismicity, including Portland, Oregon, Sacramento, California, and Salt Lake City, Utah, the service-level earthquake will result in substantially less strength than would conformance with the building code. Engineers designing buildings in locations with this lower seismicity should be aware of this and that the service-level check may not result in a building of adequate strength. Additional discussion of this issue is also provided in Chapter 8.*

A number of studies have attempted to characterize the effective damping in real buildings. Studies have ranged from system identification performed with low amplitude forced vibrations to review and analysis of strong motion recordings. Using data obtained from 8 strong motion earthquakes in California, Goel and Chopra (1997) found that effective damping for buildings in excess of 35 stories tall ranged from about 2% to 4%. Using data obtained from Japanese earthquakes, Satake et. al. (2003) found effective damping in such structures to be in the range of 1% to 2%. Given this information and the impossibility of precisely defining damping for a build that has not yet been constructed, a default value of 2.5% damping for all modes has been recommended as a reasonable estimate for use in Service-level evaluations.

ASCE 7.05 requires that buildings assigned to Occupancy Category III and IV have minimum strength respectively of at least 125% or 150% of the strength required for buildings in lower Occupancy Categories. One way to achieve compatibility with this requirement is to increase the amplitude of the Service-level spectrum for such

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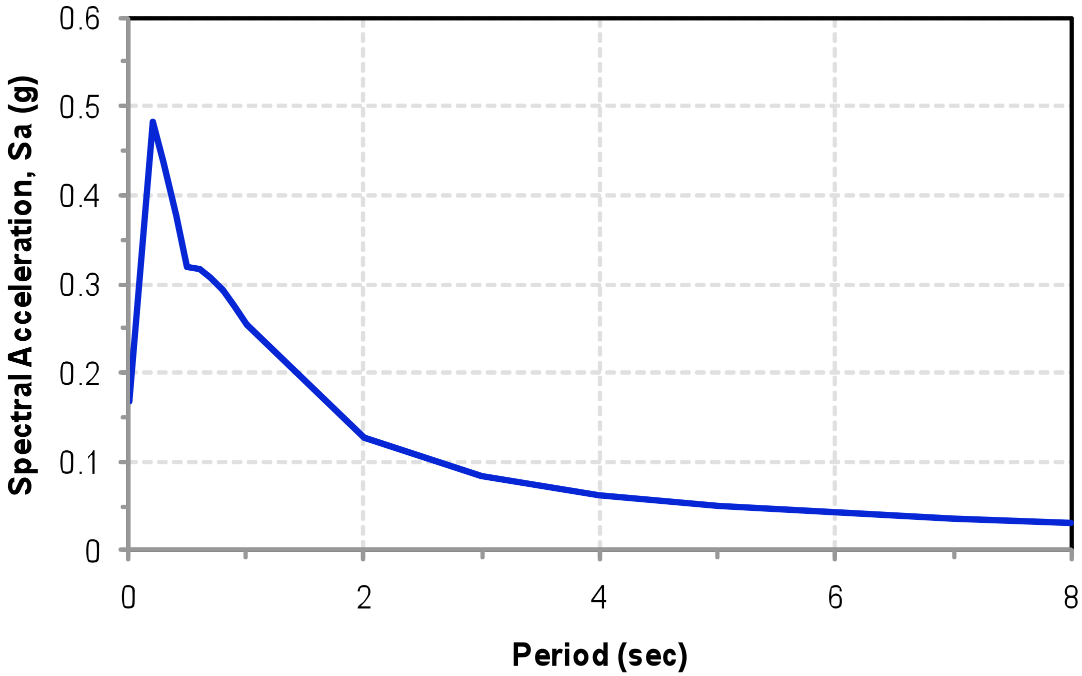
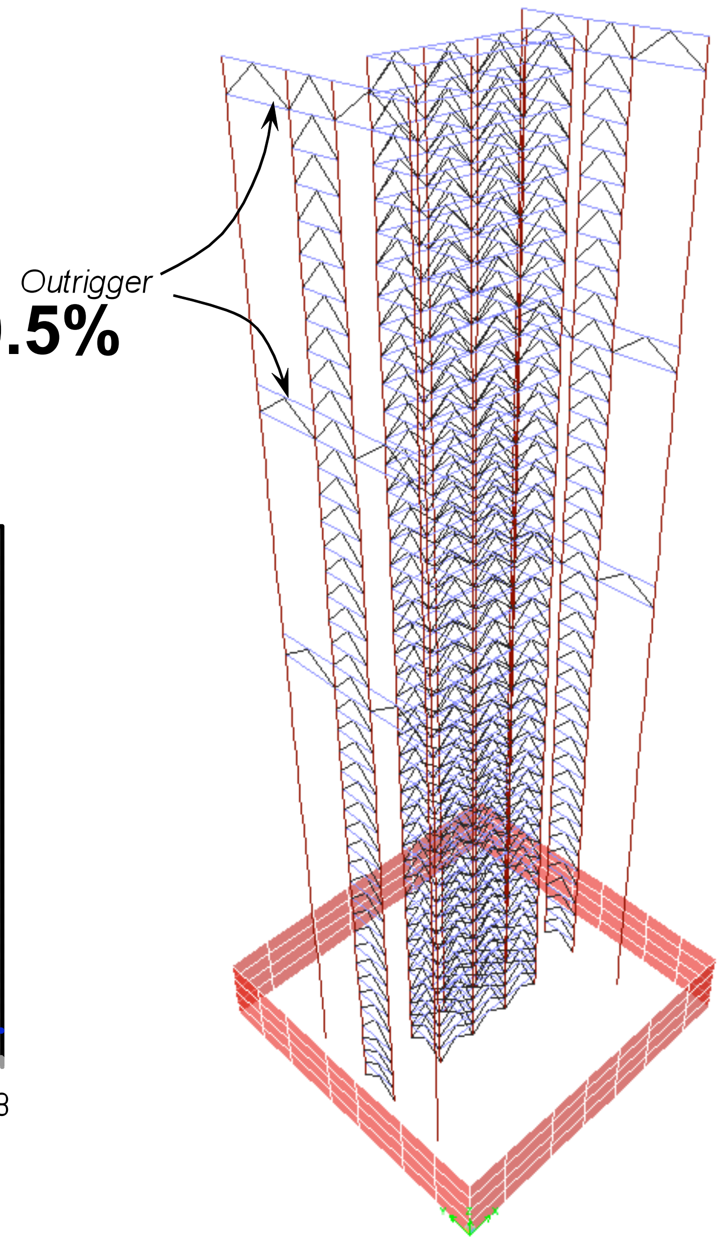
PEER TBI Design

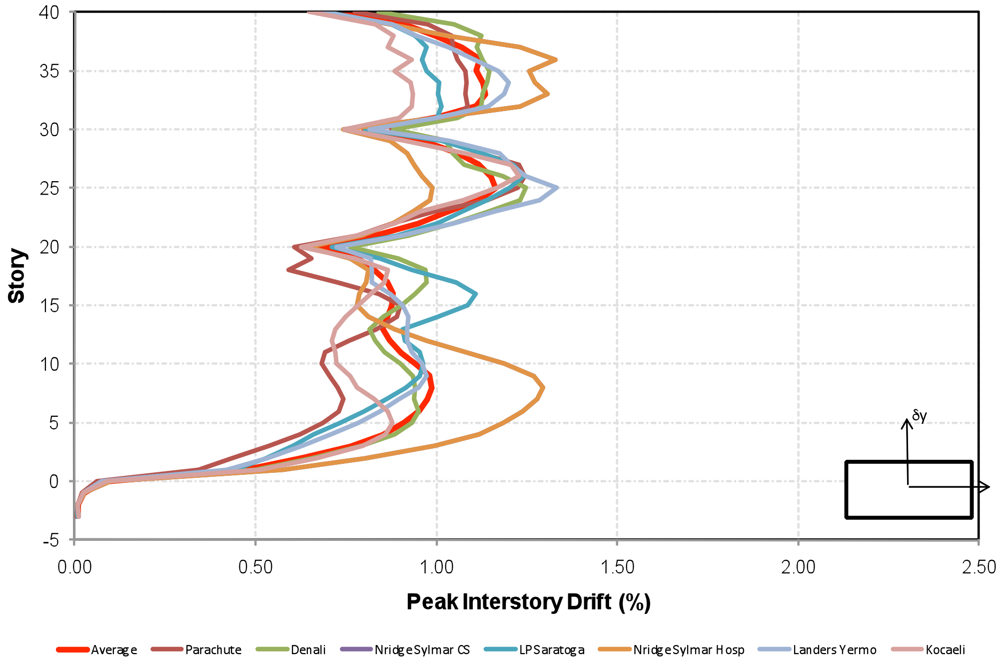
- **Started with LATBC design**
- **Drift not satisfied above 30th floor**
- **Addition of outriggers at 40th, 30th and 20th floors to control drift to <0.5%**
- **Upsize some columns & braces**



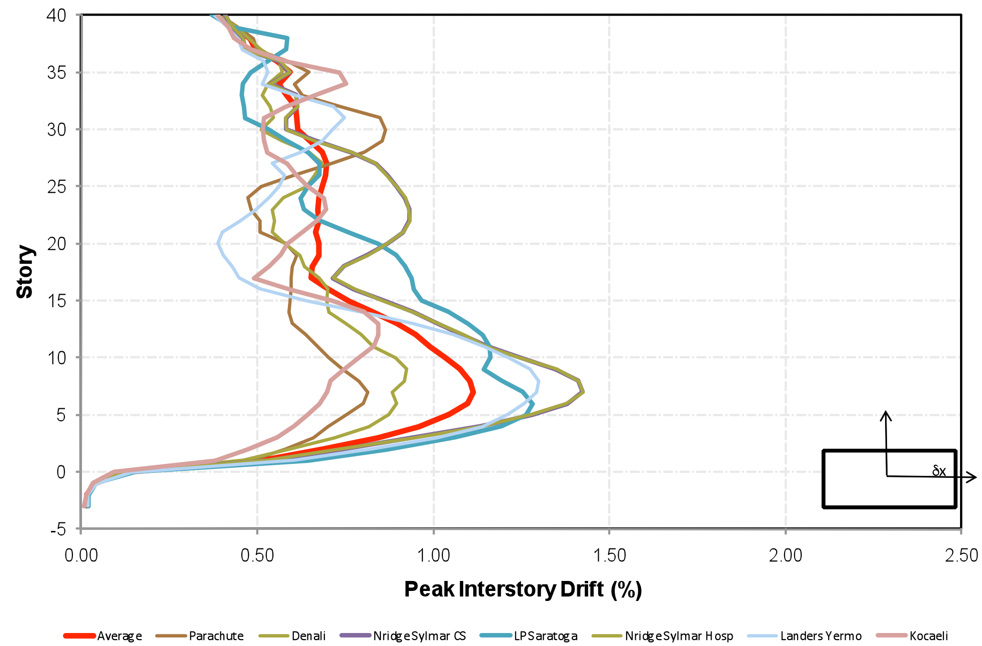


PEER TBI Design - MCE Story Drifts

Peak Interstory Drift in Y Directi o

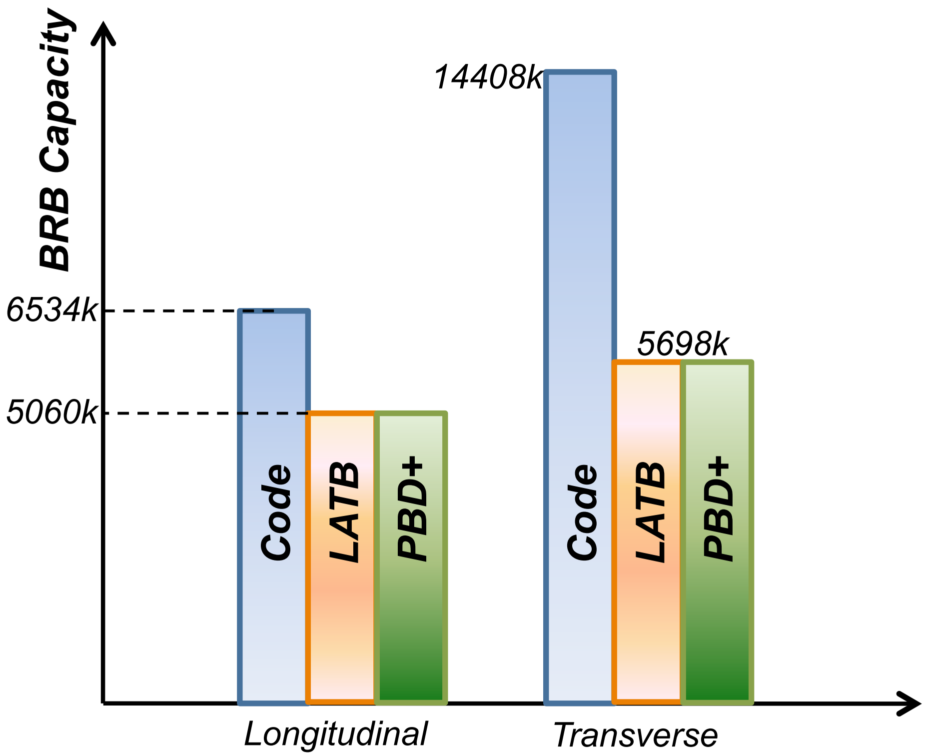


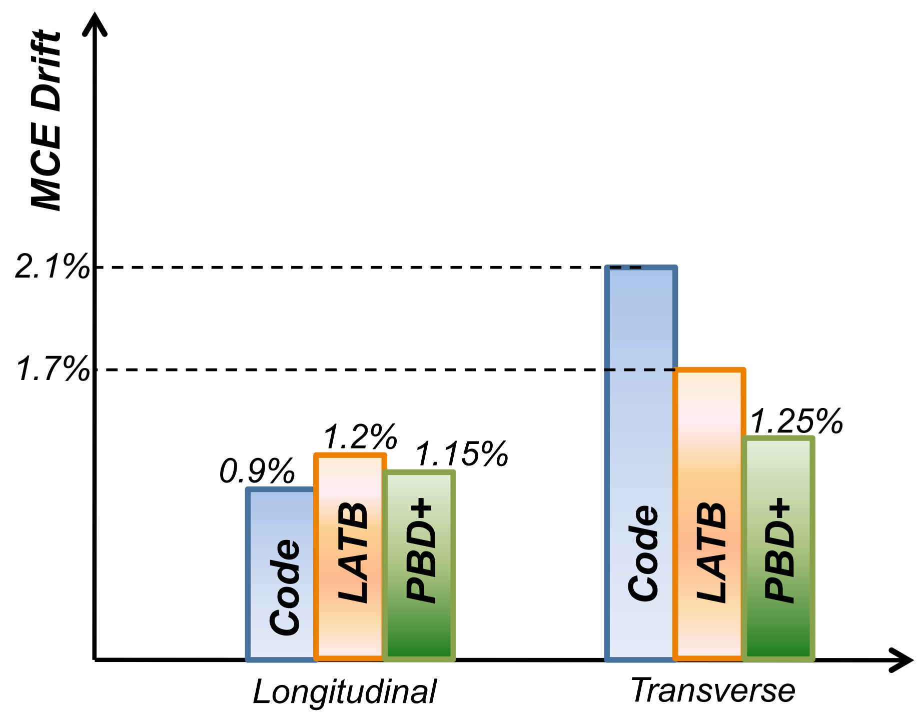
Peak Interstory Drift in X Directi o



Summary & Conclusions

- **Three prototype designs developed**
  - Code Design(without height limit)
  - LATBSDC-Performance Based Design
  - PEER TBI Performance Based Plus Design





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Summary & Conclusions

- **Performance-based Design resulted in more economical member sizes and more practical column base connection**

- **Building code for BRBs seems to be overly conservative for high rise structures**
  - Assumption that all braces yield simultaneously incorrect

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