30 PROTA Symposium:
New Generation of Seismic Codes and New Technologies in Earthquake Engineering

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Why PBD for Tall Buildings?

• The traditional code-based design everywhere is built on prescriptive rules and linear analysis.
• Using such an approach:
  – We can design building that are generally life-safe during earthquakes
  – They generally have more capacity than indicated by our design analysis
• But how much more capacity they have got?
• What level of excitation is necessary to bring them down?
• The prescriptive code approach is incapable of answering these questions.
Why PBD for Tall Buildings?

- Tall Buildings are a special class of structures with very particular characteristics and requirements:
  - long period
  - multi-mode behavior
  - Significance of P-delta effects
  - large occupancy
  - impact of failure and/or collapse
- We have evolved: from linear static to dynamic to nonlinear; from deterministic to probabilistic.
- Computing advances have made nonlinear analysis practical.
• In fairness to the code-writers, codes are not generally written with tall buildings in mind.

• Let us look at building construction statistics in the United States:

![Pie chart showing the percentage of buildings by story count]

- 6% 1 to 3 Stories
- 1% 4 to 13 Stories
- 1% 14 Stories and Taller

• You can not realistically expect code-writers to dedicate a large portion of their effort to address 1% of the buildings.
Example:

- 2002 Los Angeles Building Code Story Drift Requirement:

  \[
  \text{Story Drift Ratio} \leq \frac{0.020}{T^{1/3}}
  \]

- This provision, which was later retracted, probably did not have a serious effect on design of low-rise buildings but was a huge straightjacket for design of tall buildings with long vibration periods.
Examples of the Need

• Core Wall System
  – The great majority of tall buildings under design or construction in western U.S. use this system.
  – But codes do not allow shear wall alone systems to exceed 240 feet of overall height.
  – How did they get around that limitation?

• How about hybrid systems or new systems not envisioned by the code?
The Mechanism

• Alternative Methods Clause in the Codes
  – Section 104.11 of 2012 IBC:
    • “The provisions of this code are not intended to prevent the installation of any material or to prohibit any design or method of construction not specifically prescribed by this code, provided that any such alternative has been approved. An alternative material, design or method of construction shall be approved where the building official finds that the proposed design is satisfactory and complies with the intent of the provisions of this code, and that the material, method or work offered is, for the purpose intended, at least the equivalent of that prescribed in this code in quality, strength, effectiveness, fire resistance, durability and safety.”
The Mechanism

• Alternative Methods Clause in the Codes
  – Section 12.6 of ASCE 7-10:
    • “The structural analysis required by Chapter 12 shall consist of one of the types permitted in Table 12.6.1, based on the structure's seismic design category, structural system, dynamic properties, and regularity, or with the approval of the authority having jurisdiction, an alternative generally accepted procedure is permitted to be used.....”
    • Section 1.3 of ASCE 7-10 also permits performance-based approaches that use analysis, testing, or a combination thereof, as acceptable alternative means.
There is a problem though!

• How do you establish equivalent or superior performance?

• You need:
  – An acceptable methodology
  – Acceptable seismic hazard evaluation methods
  – Acceptable modeling and analysis techniques
  – Rational acceptance criteria
That is where guidelines come in!
Guidelines

• The two mostly used guidelines are:
  – 2010 PEER-TBI
  – 2014 LATBSDC

• They both refer to:
  – ASCE 41 and ASCE 7 Standards
  – ATC-72-1

• Jurisdictional differences in adaptation:
  – Los Angeles
  – San Diego
  – San Francisco
  – Seattle
1. Introduction
2. Design Performance Objectives
3. Design Process Overview
4. Design Criteria Documentation
5. Seismic Input
6. Preliminary Design
7. Service Level Evaluation
8. MCE Evaluation
9. Presentation of Results
10. Project Review
1. Introduction
2. Intent, Scope, Justification, and Methodology
3. Analysis and Design Procedure
   1. General
   2. Modeling Requirements
   3. Serviceability Requirements
   4. Collapse Prevention Evaluation
   5. Specific Provisions for R/C Structures
4. Peer Review Requirements
5. Seismic Instrumentation
ASCE 41 and the Guidelines

- ASCE41 is officially intended for seismic rehabilitation of existing structures.
- However, its component-based performance limits for NDP are routinely referenced by guidelines for performance based design of tall buildings.
- Engineers who believe ASCE 41 limits are too conservative, or are not applicable to their project, have the opportunity to present and substantiate other appropriate limits.
- Peer review approval is always necessary for any deviation from ASCE 41.
Performance Objectives

- **2010 TBI:**
  - Basic Performance Objective (BPO)
    - Performance equivalent to Code design buildings
    - Two Level Design
  - Enhanced Performance Objective (EPO)
    - Better than BPO
    - Specifics and criteria not laid out

- **2014 LATBSDC:**
  - Serviceable performance under frequent events
  - Repairable following very rare events

- **Hazard Levels Considered:**
  - Serviceability: 43 years mean return period (50% in 30 years)
  - Very Rare Event:
    - PEER-TBI: ASCE 7-05, 7-10 MCE
    - 2014 LATBSDC: MCE$_R$ per ASCE 7-10 with clarifications
Analytical Procedures

• ASCE-41 permits four types of analyses:
  1. Linear elastic static procedure (LSP)
  2. Linear dynamic procedure (LDP) or response spectrum analysis
  3. Non-linear static procedure (NSP) commonly referred to as the push-over analysis, and
  4. Dynamic nonlinear response analysis (NDP).

• Tall Building Design Guidelines permit only two:
  1. 3D LDP or NDP for serviceability check
  2. 3D NDP for all checks
More About Performance Objectives

1. Serviceable behavior:
   - building structural and nonstructural components retain their general functionality during and after earthquake
   - Repairs, if necessary, are expected to be minor and could be performed without substantially affecting the normal use and functionality of the building

2. A low probability of collapse under MCE or MCE$_R$ type events.
   - Demands are checked for all structural members (lateral as well as gravity system)
   - Claddings and their connections to the structure must accommodate MCE or MCE$_R$ displacements without failure
Capacity Design

• Both guidelines use capacity design principles:
  – The capacity design criteria shall be described in the project-specific seismic design criteria.
  – The structural system for the building shall be clearly demonstrated to have well defined inelastic behavior where nonlinear action is limited to the clearly identified members and regions.
  – All other members should be stronger than the elements designed to experience nonlinear behavior.
Typical zones and actions commonly designated for nonlinear behavior are identified in the following table. This table is not meant to be conclusive. Other zones may be included into the design based on sufficient justification.

Table C.3.3.2 Zones and actions commonly designated for nonlinear behavior

<table>
<thead>
<tr>
<th>Structural System</th>
<th>Zones and Actions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Special Moment Resisting Frames (steel, concrete, or composite)</td>
<td>- Flexural yielding of Beam ends (except for transfer girders)</td>
</tr>
<tr>
<td></td>
<td>- Shear in Beam-Column Panel Zones</td>
</tr>
<tr>
<td></td>
<td>- P-M-M yielding at the base of columns (top of foundation or basement podiums)</td>
</tr>
<tr>
<td></td>
<td>- Braces (yielding in tension and buckling in compression)</td>
</tr>
<tr>
<td>Special Concentric Braced Frames</td>
<td>- P-M-M yielding at the base of columns (top of foundation or basement podiums)</td>
</tr>
<tr>
<td></td>
<td>- Shear Link portion of the beams (shear yielding preferred but combined shear and flexural yielding permitted)</td>
</tr>
<tr>
<td>Eccentric Braced Frames</td>
<td>- P-M-M yielding at the base of columns (top of foundation or basement podiums)</td>
</tr>
<tr>
<td></td>
<td>- Unbonded brace cores (yielding in tension and compression)</td>
</tr>
<tr>
<td>Unbonded Braced Frames</td>
<td>- P-M-M yielding at the base of columns (top of foundation or basement podiums)</td>
</tr>
<tr>
<td>Special Steel-Plate Shear Walls</td>
<td>- Shear yielding of web plates</td>
</tr>
<tr>
<td></td>
<td>- Flexural yielding of Beam ends</td>
</tr>
<tr>
<td></td>
<td>- P-M-M yielding at the base of the walls (top of foundation or basement podiums) and other clearly defined locations throughout the height of the wall.</td>
</tr>
<tr>
<td>R/C Shear Walls</td>
<td>- Flexural yielding and/or shear yielding of link beams</td>
</tr>
<tr>
<td>Foundations</td>
<td>- Controlled rocking</td>
</tr>
<tr>
<td></td>
<td>- Controlled settlement</td>
</tr>
</tbody>
</table>

* yielding caused by combined axial force and uniaxial or biaxial flexure
Classification of Structural Actions

• All actions must be classified as either
  – Force-Controlled, or
  – Deformation-Controlled

• Force-Controlled actions must further be categorized as either
  – Critical, or
  – Non-Critical
3.5.4.1 Acceptance Criteria at the Component Level

Actions in all lateral load-resisting elements must be categorized as either force-controlled or deformation-controlled and are classified as force-controlled, as either critical, or noncritical actions. Table 5 shows a representative and acceptable classification of such actions.

<table>
<thead>
<tr>
<th>Component</th>
<th>Seismic Action</th>
<th>Classification</th>
<th>Criticality</th>
</tr>
</thead>
<tbody>
<tr>
<td>Below grade perimeter walls</td>
<td>Flexure</td>
<td>Force Controlled</td>
<td>Non-Critical</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Force Controlled</td>
<td>Critical</td>
</tr>
<tr>
<td>Parking ramp walls</td>
<td>Flexure</td>
<td>Deformation Controlled</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Force Controlled</td>
<td>Critical</td>
</tr>
<tr>
<td>Podium walls</td>
<td>Flexure</td>
<td>Deformation Controlled</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Force Controlled</td>
<td>Critical</td>
</tr>
<tr>
<td>Tower core walls (over their entire height)</td>
<td>Flexure</td>
<td>Deformation Controlled</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Force Controlled</td>
<td>Critical</td>
</tr>
<tr>
<td>Core wall coupling beams</td>
<td>Shear / Flexure</td>
<td>Deformation Controlled</td>
<td>N/A</td>
</tr>
<tr>
<td>Floor slabs</td>
<td>Out of plane flexure around supports</td>
<td>Deformation Controlled</td>
<td>N/A</td>
</tr>
<tr>
<td>Diaphragms with major shear transfer</td>
<td>Flexure</td>
<td>Force Controlled</td>
<td>Non-Critical</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Force Controlled</td>
<td>Critical</td>
</tr>
<tr>
<td>Gravity columns</td>
<td>Axial</td>
<td>Force Controlled</td>
<td>Critical</td>
</tr>
<tr>
<td>Foundations</td>
<td>Flexure</td>
<td>Force Controlled</td>
<td>Non-Critical</td>
</tr>
<tr>
<td></td>
<td>Shear</td>
<td>Force Controlled</td>
<td>Critical</td>
</tr>
</tbody>
</table>

Source: 2014 LATBSDC
Evaluation Procedures

• Both guidelines require a three-dimensional detailed mathematical model of the physical structure

• Realistic estimates of stiffness, strength and damping

• Strength:
  – 2014 LATBSDC uses expected material properties and reduction factor of 1.0.
  – 2010 PEER-TBI uses the same for MCE but specified properties and code reduction factors for serviceability evaluation.
Table 2. Expected Material Strengths

<table>
<thead>
<tr>
<th>Material</th>
<th>Expected Strength</th>
<th>Strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Yield Strength for Structural steel</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hot-rolled structural shapes and bars</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ASTM A36/A36M</td>
<td>1.5$F_y$</td>
<td></td>
</tr>
<tr>
<td>ASTM A572/A572M Grade 42 (290)</td>
<td>1.3$F_y$</td>
<td></td>
</tr>
<tr>
<td>ASTM A992/A992M</td>
<td>1.1$F_y$</td>
<td></td>
</tr>
<tr>
<td>All other grades</td>
<td>1.1$F_y$</td>
<td></td>
</tr>
<tr>
<td><strong>Hollow Structural Sections</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ASTM A500, A501, A618 and A847</td>
<td>1.3$F_y$</td>
<td></td>
</tr>
<tr>
<td><strong>Steel Pipe</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ASTM A53/A53M</td>
<td>1.4$F_y$</td>
<td></td>
</tr>
<tr>
<td><strong>Plates</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>All other products</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Yield Strength for Reinforcing steel</strong></td>
<td></td>
<td>1.17 times specified $f_y$</td>
</tr>
<tr>
<td><strong>Ultimate Strength for Concrete</strong></td>
<td></td>
<td>1.3 times specified $f'_{c}$</td>
</tr>
</tbody>
</table>

Table 3. Reinforced Concrete Stiffness Properties

<table>
<thead>
<tr>
<th>Element</th>
<th>Serviceability and Wind</th>
<th>MCE-Level Nonlinear Models</th>
</tr>
</thead>
<tbody>
<tr>
<td>February 2016</td>
<td>Flexural – 0.75 Ig</td>
<td>Flexural – 1.0 Ec **</td>
</tr>
<tr>
<td></td>
<td>Shear – 1.0 Ag</td>
<td>Shear – 0.5 Ag</td>
</tr>
</tbody>
</table>
Reinforcing Steel

Concrete

* fy is used to designate specified yield strength of steel materials in this Guideline. It is equivalent to Fy used in AISC standards.

### Table 7.2 Effective component stiffness values.

<table>
<thead>
<tr>
<th>Component</th>
<th>Flexural Rigidity</th>
<th>Shear Rigidity</th>
<th>Axial Rigidity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural steel Beams, Columns and Braces</td>
<td>$E_s I$</td>
<td>$G_s A$</td>
<td>$E_s A$</td>
</tr>
<tr>
<td>Composite Concrete Metal Deck Floors</td>
<td>$0.5E_c I_g$</td>
<td>$G_c A_g$</td>
<td>$E_c A_g$</td>
</tr>
<tr>
<td>R/C Beams – nonprestressed</td>
<td>$0.5E_c I_g$</td>
<td>$G_c A_g$</td>
<td>$E_c A_g$</td>
</tr>
<tr>
<td>R/C Beams – prestressed</td>
<td>$E_c I_g$</td>
<td>$G_c A_g$</td>
<td>$E_c A_g$</td>
</tr>
<tr>
<td>R/C Columns</td>
<td>$0.5E_c I_g$</td>
<td>$G_c A_g$</td>
<td>$E_c A_g$</td>
</tr>
<tr>
<td>R/C Walls</td>
<td>$0.75E_c I_g$</td>
<td>$G_c A_g$</td>
<td>$E_c A_g$</td>
</tr>
<tr>
<td>R/C Slabs and Flat Plates</td>
<td>$0.5E_c I_g$</td>
<td>$G_c A_g$</td>
<td>$E_c A_g$</td>
</tr>
</tbody>
</table>

Notes:

$E_c$ shall be computed per ACI 318, using expected material strength per Table 7.1.

$G_c$ shall be computed as $E_c / \left(2 \left(1 + n \right)\right)$, where $n$ shall be taken as 0.2.
### Table 3. Reinforced Concrete Stiffness Properties

<table>
<thead>
<tr>
<th>Element</th>
<th>Serviceability and Wind</th>
<th>MCE-Level Nonlinear Models</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural Walls</td>
<td>Flexural – 0.75 Ig</td>
<td>Flexural – 1.0 Ec **</td>
</tr>
<tr>
<td></td>
<td>Shear – 1.0 Ag</td>
<td>Shear – 0.5 Ag</td>
</tr>
<tr>
<td>Basement Walls</td>
<td>Flexural – 1.0 Ig</td>
<td>Flexural – 0.8 Ig</td>
</tr>
<tr>
<td></td>
<td>Shear – 1.0 Ag</td>
<td>Shear – 0.5 Ag</td>
</tr>
<tr>
<td>Coupling Beams</td>
<td>Flexural – 0.3 Ig</td>
<td>Flexural – 0.2 Ig</td>
</tr>
<tr>
<td></td>
<td>Shear – 1.0 Ag</td>
<td>Shear – 1.0 Ag</td>
</tr>
<tr>
<td>Diaphragms (in-plane only)</td>
<td>Flexural – 0.5 Ig</td>
<td>Flexural – 0.25 Ig</td>
</tr>
<tr>
<td></td>
<td>Shear – 0.8 Ag</td>
<td>Shear – 0.25 Ag</td>
</tr>
<tr>
<td>Moment Frame Beams</td>
<td>Flexural – 0.7 Ig</td>
<td>Flexural – 0.35 Ig</td>
</tr>
<tr>
<td></td>
<td>Shear – 1.0 Ag</td>
<td>Shear – 1.0 Ag</td>
</tr>
<tr>
<td>Moment Frame Columns</td>
<td>Flexural – 0.9 Ig</td>
<td>Flexural – 0.7 Ig</td>
</tr>
<tr>
<td></td>
<td>Shear – 1.0 Ag</td>
<td>Shear – 1.0 Ag</td>
</tr>
</tbody>
</table>

* Modulus of elasticity is based on the following equations:

\[
E_c = 57000 \sqrt{f_c} \quad \text{for } f_c \leq 6000 \text{ psi}
\]

\[
E_c = 40000 \sqrt{f_c} + 1 \times 10^6 \quad \text{for } f_c > 6000 \text{ psi} \quad \text{(per ACI 363R-92)}
\]

** Nonlinear fiber elements automatically account for cracking of concrete because the concrete fibers have zero tension stiffness.
PEER-TBI & LATBSDC Provisions

1. Use 2.5% damping instead of 5% damping but permit DCR = 1.5 for deformation controlled members for serviceability.

2. 2014 LATBSDC limits DCR to 0.70 for force controlled members in serviceability check.

3. 2010 PEER requirements for collapse prevention are more elaborate and detailed than 2011 LATBSDC

4. No minimum base shear capacity requirement in either document
Analysis Methods

• Serviceability:
  – Can use either
    1. Linear Response Spectrum Analyses
       – CQC mode combination
       – 90% mass participation
    2. Nonlinear Dynamic Response Analyses

• For MCE evaluation:
  – Must use
    • Nonlinear Dynamic Response Analyses

• Inherent torsional properties of the structural system should always be considered.
Accidental Eccentricity (AE)

• 2014 LATBSDC
  – Consider implications during serviceability evaluation
  – Address if significant during MCE evaluation

• 2010 PEER TBI
  – Do not need to consider

• Consideration of AE in nonlinear analyses may require multiple evaluations.
Floor Diaphragms

- Floor diaphragms shall be included in the mathematical model using realistic stiffness properties.
- Regardless of the relative rigidity or flexibility of floor diaphragms, flexibility of diaphragms with significant force transfer (for example podium levels and other setback levels) shall be explicitly included in the mathematical model.
- Diaphragm chord and drag forces shall be established in a manner consistent with the floor characteristics, geometry, and well-established principles of structural mechanics.
- Both shear and bending stresses in diaphragms must be considered.
- At diaphragm discontinuities, such as openings and re-entrant corners, the dissipation or transfer of edge (chord) forces combined with other forces in the diaphragm shall be evaluated.
Elastic Response Spectrum Analyses

At least 90 percent of the participating mass of the structure shall be included in the calculation of response for each principal horizontal direction. Modal responses shall be combined using the

**Load Combinations**

- **Response Spectrum Analysis**
  
  \[ 1.0D + L_{\text{exp}} + 1.0E_x + 0.3E_y \]
  
  \[ 1.0D + L_{\text{exp}} + 1.0E_y + 0.3E_x \]

- **Nonlinear Dynamic Response Analysis**
  
  \[ 1.0D + L_{\text{exp}} + 1.0E \]

where \( D \) is the service dead load and \( L_{\text{exp}} \) is the expected service live load. \( L_{\text{exp}} \) may be taken as 25% of the unreduced live load unless otherwise substantiated and shall be included in all gravity calculations and P-D analyses.

**C.3.4.3.** Building Code response modification factors do not apply (that is, \( R, W_0, r, \) and \( C_d \), are all taken as unity). \( L_{\text{exp}} \) need not be included in the mass calculations.

**3.4.3.1. Elastic Response Spectrum Analyses**

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At least 90 percent of the participating mass of the structure shall be included in the calculation of response for each principal horizontal direction. Modal responses shall be combined using the
P-Δ Inclusion

- P-Δ effects must be included in all analyses
Modeling Nonlinear Behavior

Figure courtesy of Prof. Greg Deierlein
Modeling Nonlinear Behavior

- Concentrated plasticity model for beams and columns and fiber elements for walls are most common.
- All other elements and components that in combination significantly contribute to or affect the total or local stiffness of the building should be included in the mathematical model.
- Axial deformation of gravity columns in a core-wall system is one example of effects that should be considered in the structural model of the building.

Figure courtesy of MKA
Modeling Strength / Stiffness Degradation

• 2010 PEER TBI
  – Provides detailed guidelines on four approved methods for modeling degradation

• 2014 LATBSDC
  – Adopts the first two of the detailed procedures contained in 2010 PEER.
2010 PEER-TBI Degradation Modeling Options

(a) Option 1 – with cyclic deterioration

(b) Option 2 – modified backbone curve
   = envelope curve

(c) Option 3 - modified backbone curve
   = factored monotonic backbone curve

(d) Option 4 – no strength deterioration

Figure courtesy of Prof. Helmut Krawinkler

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February 2015
Soil-Foundation-Structure-Interaction (SFSI)

• Naeim & Stewart (2008) demonstrated the difficulties of realistic modeling of SFSI in a design environment.

• 2010 PEER-TBI has two recommended modeling techniques

• 2014 LATBSDC recommends a single approach for this.
2010 PEER TBI Suggested Modeling Techniques for SFSI
Foundations

• Modeling, Rocking and Uplift:
  – Foundation components that have significant flexibility or will experience significant inelastic behavior shall be modeled following the same approach outlined for components of the superstructure.
Response Modification Devices

- Seismic isolation and energy dissipation devices shall be modeled based on data from laboratory tests representing conditions anticipated during MCE / MCE$_R$ shaking.

- If the properties of these devices vary significantly, lower and upper bound properties shall be modeled.

- The consequences of attaining device limits must be demonstrated to be tolerable to the structure.
Table 4 contains recommendations for UB and LB Stiffness parameters for backstay sensitivity analyses. The sensitivity analyses, where applicable, shall be performed in addition to the analyses performed using stiffness properties provided in Table 3.

C.3.2.3. Any lower part of a tall building structure that is larger in floor plate, and contains substantially increased seismic-force resistance in comparison to the tower above, can be considered a podium. Backstay effects are the transfer of lateral forces from the seismic-force resisting elements in the tower into additional elements that exist within the podium, typically through one or more floor diaphragms. The lateral force resistance in the podium levels, and force transfer through floor diaphragms at these levels, helps a tall building resist seismic overturning forces.

This component of overturning resistance is referred to as the backstay effect, based on its similarity to the backspan of a cantilever beam. It is also sometimes called “shear reversal” because the shear in the seismic-force resisting elements can change direction within the podium levels. Since the stiffness properties of the elements, particularly diaphragms, are both influential on the seismic design and uncertain, a sensitivity analysis is required. The UB analysis provides an upper-bound estimate of forces in the backstay load path and a lower bound estimate of forces in the foundation below the tower. This case will govern the forces in the backstay below the tower.

<table>
<thead>
<tr>
<th>Stiffness Parameters</th>
<th>UB</th>
<th>LB</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diaphragms at the podium and below</td>
<td>$\frac{\overline{E}<em>{p,0}}{\overline{I}</em>{p,0}}$</td>
<td>$0.5$</td>
</tr>
</tbody>
</table>
Damping

• A particularly thorny issue
  – In nonlinear analyses most of the damping is represented by hysteretic behavior of the elements
  – Some small additional viscous damping may be justified for:
    • Energy dissipation provided by components and systems not explicitly modeled
    • As necessary to avoid numerical instability

• 2014 LATB SDC
  – Limits viscous damping to 2.5% for both serviceability and MCE.

• 2010 PEER-TBI
  – 2.5% for linear serviceability evaluation
  – Refers to ATC-72 for nonlinear evaluation
Ground Motion Selection and Scaling

• Three General Methods:
  – **Code Scaling** in time-domain so that the average of spectra of records stays above design spectrum over a range of periods
  – **Spectral Matching** by modifying the frequency content of the ground motion
  – **Conditional Mean Spectrum** (CMS) by scaling at a particular period or periods

• All three methods permitted by both guidelines
Code Scaling

Pseudoacceleration, g

Design spectrum
Avg. of scaled suite spectra

Source: FEMA 451B
Spectral Matching

Source: Ninyo and More

30 PROTA Symposium: New Generation of Seismic Codes and New Technologies in Earthquake Engineering
February 2015
CMS Scaling

Ground Motion Selection and Scaling

• A minimum of 7 pairs is usually required
• If CMS is used, 2014 LATBSDC requires at least two suites of 7 pairs.
• Attention should be paid to higher modes being addressed by scaling (0.2T to 1.5T)
• Most practicing engineers prefer matching
  – One must be careful as, matched motion contains less record to record dispersion
  – This is one reason 2014 LATBSDC uses 1.5 x mean rather than Mean + x% of SD
2014 LATBSDC R/C Specific Requirements

• Beams in Special R/C Moment Frames
  – In regions where post-yield rotations are expected, the member shall be detailed in the vertical direction as required by ACI 318-11 Eq. (21-5).
The following concrete compressive strength limits as indicated in Table 7 shall be attained. In
addition, the strain attained at the point of maximum strength shall not be less than 0.002 and the
strain attained past the point of maximum strength at stress level equal to half of the maximum
strength value shall not be less than 0.004 (see Figure 3.6.2.1).

Table 7. Intermediate and final strength values for high-strength concrete

<table>
<thead>
<tr>
<th>Specified Strength (psi)</th>
<th>Intermediate and final strength values (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6,000 ≤ $f'_c$ &lt; 8000</td>
<td>- 6,000 at 28 days</td>
</tr>
<tr>
<td></td>
<td>- 1.0 $f'_c$ at 90 days</td>
</tr>
<tr>
<td>8,000 ≤ $f'_c$ ≤ 12,000</td>
<td>- 6,000 at 28 days</td>
</tr>
<tr>
<td></td>
<td>- 0.75 $f'_c$ at 90 days</td>
</tr>
<tr>
<td></td>
<td>- 1.0 $f'_c$ at 365 days</td>
</tr>
<tr>
<td>$f'_c$ &gt; 12,000</td>
<td>- 0.50 $f'_c$ at 28 days</td>
</tr>
<tr>
<td></td>
<td>- 0.75 $f'_c$ at 90 days</td>
</tr>
<tr>
<td></td>
<td>- 1.0 $f'_c$ at 365 days</td>
</tr>
</tbody>
</table>

Figure 3.6.2.1. Strain capacity requirements
Peer Review Requirements

• Both documents have extensive peer review requirements which will be discussed in a separate presentation.
Instrumentation Requirements

- **2010 PEER TBI**
  - No requirements

- **2011 LATBSDC**
  - Detailed requirements
  - Consistent with CGS / CSMIP

<table>
<thead>
<tr>
<th>Minimum tall building instrumentation levels</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Number of Stories Above Ground</strong></td>
<td><strong>Minimum Number of Sensors</strong></td>
</tr>
<tr>
<td>10 – 20</td>
<td>15</td>
</tr>
<tr>
<td>20 – 30</td>
<td>21</td>
</tr>
<tr>
<td>30 – 50</td>
<td>24</td>
</tr>
<tr>
<td>&gt; 50</td>
<td>30</td>
</tr>
</tbody>
</table>
LATBSDC Instrumentation Requirements

• Objective:
  – Improve safety and reliability of building systems by providing data to improve computer modeling and enable damage detection for post-event condition assessment

• Plan and Review:
  – An instrumentation plan shall be prepared by the EOR and submitted to SPRP and Building Official for review and approval.
  – SPRP Approved instrumentation plans shall be marked accordingly on the structural drawings.
  – If the building is intended to be included in the inventory of buildings monitored by the California Geologic Survey (CGS) then the recorders and accelerometers must be of a type approved by CGS.
<table>
<thead>
<tr>
<th><strong>Property</strong></th>
<th><strong>Detail</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Latitude</strong></td>
<td>37.7858 N</td>
</tr>
<tr>
<td><strong>Longitude</strong></td>
<td>122.3921 W</td>
</tr>
<tr>
<td><strong>Elevation (m)</strong></td>
<td>—</td>
</tr>
<tr>
<td><strong>Site Geology</strong></td>
<td>—</td>
</tr>
<tr>
<td><strong>Vs30 (m/sec)</strong></td>
<td>—</td>
</tr>
<tr>
<td><strong>Site Class</strong></td>
<td>—</td>
</tr>
<tr>
<td><strong>Remarks</strong></td>
<td>—</td>
</tr>
<tr>
<td><strong>No. of Stories above/below ground</strong></td>
<td>62/0</td>
</tr>
<tr>
<td><strong>Plan Shape</strong></td>
<td>Rectangular</td>
</tr>
<tr>
<td><strong>Base Dimensions</strong></td>
<td>137 ft x 113 ft</td>
</tr>
<tr>
<td><strong>Typical Floor Dimensions</strong></td>
<td>102 ft x 113 ft</td>
</tr>
<tr>
<td><strong>Design Date</strong></td>
<td>2000</td>
</tr>
<tr>
<td><strong>Instrumentation</strong></td>
<td>2012, 72 accelerometers on 20 levels.</td>
</tr>
<tr>
<td><strong>Vertical Load Carrying System</strong></td>
<td>Post-tensioned concrete flat slabs supported by concrete columns and core shear walls</td>
</tr>
<tr>
<td><strong>Lateral Force Resisting System</strong></td>
<td>Concrete core shear walls with supplemental tall outrigger columns of steel reconfined concrete in the transverse direction. The core is attached to outrigger columns by steel buckling restrained braces. Water tanks at the top of the building are part of the &quot;tuned liquid mass damper&quot; system to reduce wind induced motions.</td>
</tr>
<tr>
<td><strong>Foundation Type</strong></td>
<td>Concrete mat foundation (12 feet beneath the tower) bearing on bedrock</td>
</tr>
<tr>
<td><strong>Remarks</strong></td>
<td>This building was instrumented jointly by the California Geological Survey and the U.S. Geological Survey. The building structural design was based on performance-based seismic design.</td>
</tr>
</tbody>
</table>
San Francisco - 62-story Residential Bldg
(CSMIP Station No. 58389)
(NSMP Station No. 8389)

SENSOR LOCATIONS

Elevation A-A' (N/S)  Elevation B-B' (E/W)  Level 1 (P4) Plan

February 2015
San Francisco - 62-story Residential Bldg
(CSMIP Station No. 58389)
(NSMP Station No. 8389)

SENSOR LOCATIONS

CSMIP Sensors (1-36)
NSMP Sensors (37-72)

Level 7 (Mezzanine) Plan
Level 8 Plan
Level 12 Plan
Level 13 Plan
Level 18 Plan
Level 19 Plan
Level 20 Plan
Level 24 Plan

Structure Reference
Orientation: N_{ref} = 315°

February 2015
San Francisco - 62-story Residential Bldg
(CSMIP Station No. 58389)
(NSMP Station No. 8389)

SENSOR LOCATIONS

CSMIP Sensors (1-36)
NSMP Sensors (37-72)

Level 51 Plan

Level 53 Plan

Level 55 Plan

Level 56 Plan

Level 61 Plan

Level 62 Plan

Level 64 (Roof) Plan

February 2015

Structure Reference Orientation: $N_{ref} = 315^\circ$

Diagram: 7/11/2013

Installed: 5/18/2012
American Canyon Earthquake of 24 Aug 2014

6.0ML, 03:20:44 AM PDT, 38.22N 122.31W Depth 11.3 km

Interactive Map
ShakeMap

Last Update: 2014-08-29 09:02 (Pacific)

<table>
<thead>
<tr>
<th>Station</th>
<th>Code/ID</th>
<th>Network</th>
<th>Distance (km)</th>
<th>Horiz Apk (g)</th>
<th>View</th>
</tr>
</thead>
<tbody>
<tr>
<td>San Francisco - 62-story Resid. Bldg</td>
<td>58389</td>
<td>CGS</td>
<td>48.1</td>
<td>0.005</td>
<td>☐</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.021</td>
<td>☑</td>
</tr>
</tbody>
</table>
Acceptance Criteria

• Key differences to be aware of:
  – Reduction Factors:
    • In 2014 LATBSDC, $\phi = 1.0$, always.
    • In 2010 PEER-TBI, $\phi = 1.0$ for serviceability; 
      $\phi = \text{code values for MCE}$
  – Risk Categories:
    • 2014 LATBSDC considers various categories
    • 2010 PEER-TBI assumes Risk Category to be II
  – Modeling Dispersion:
    • 2014 LATBSDC uses $1.5 \times \text{mean}$
    • 2010 PEER-TBI uses $1.5 \times \text{mean}$, or 
      $\text{mean} + 1.3 \times \text{SD} > 1.2 \times \text{mean}$, depending on the situation.
## 2014 LATBSDC

### Table 6. Risk Category Reduction Factor

<table>
<thead>
<tr>
<th>Risk Category from ASCE 7-10 Table 1.5-1</th>
<th>Risk Reduction factor, $\kappa_i$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>1.00</td>
</tr>
<tr>
<td>II</td>
<td>1.00</td>
</tr>
<tr>
<td>III</td>
<td>0.80</td>
</tr>
<tr>
<td>IV</td>
<td>Value to be established by SPRP (see Section 4)</td>
</tr>
</tbody>
</table>
Acceptance Criteria -- Maximum Drift

• Absolute Maximum Transient Drift Angle Limits
  – Serviceability:
    
    0.005 overall

  – MCE or MCE_R:
    
    0.030 \times K_j \text{ max average at any story}

    0.045 \times K_j \text{ max. interstory drift at any story under any record}
Acceptance Criteria -- Residual Drift

• Collapse Prevention:

\[ 0.010 \times K_i \text{ average max. of time histories} \]

\[ 0.015 \times K_i \text{ maximum from any} \]
Acceptance Criteria

• Strength Loss:
  – In any nonlinear dynamic response analysis, deformation imposed at any story shall not result in a loss of total story strength that exceeds 20% of the initial strength.
Acceptance Criteria -- Serviceability

• 2014 LATBSDC
  – Force-Controlled Actions:
    Strength Demand ≤ 0.7 x Capacity
  – Deformation-Controlled Actions:
    • Response Spectrum Analysis
      ▪ Strength Demand ≤ 1.50 x Capacity, (Risk Category I, II)
      ▪ Strength Demand ≤ 1.20 x Capacity, (Risk Category III)
      ▪ Talk to SPRP for Risk Category > III
    • Nonlinear Analysis
      ▪ Can use up to IO limit of ASCE 41
Acceptance Criteria -- MCE

• 2010 PEER
  – Force-Controlled Actions:
    • Two Groups:
      – Critical Actions
        » failure mode pose severe consequences to structural stability under gravity and/or lateral loads
        » Design for mean + 1.3 to 1.5 times SD
        » Use code reduction factors, $\phi = 0.75$ for shear
      – Noncritical Actions
        » Design for mean demand values and $\phi = 1.0$.

• 2011 LATBSDC
  – Essentially the same, except uses 1.5 times mean, $\phi = 1.0$, and a factor for risk category.
3.5.4.1(a) Force-Controlled Actions

(a) Critical Actions

Force-controlled critical actions are those force-controlled actions that pose severe consequences to structural stability under gravity and/or lateral loads. Force-controlled critical actions shall satisfy:

\[ F_{uc} \leq k_i f F_{n,e} \]

where

- \( F_{uc} = 1.5 \) times the mean value of demand.
- \( F_{n,e} = \) nominal strength as computed from applicable material codes but based on expected material properties.
- \( f = 1.0 \).
- \( k_i = \) Risk reduction factor given in Table 6.

**EXCEPTION:** For buildings located in the Los Angeles region if the serviceability acceptance criteria are satisfied per requirements of Section 3.4.5.1, then \( k_i \) may be taken as 1.0.

C.3.5.4.1.1(a) Use of the mean value would imply a significant probability of failure with associated consequences. The use of mean plus one standard deviation is more appropriate. However, when fewer than 20 ground motion pairs are used in nonlinear dynamic response analysis, little confidence can be placed in the computed value of the standard deviation or the mean. A factor of 1.5 is utilized to represent a simple yet reasonable means to reduce
Acceptance Criteria -- MCE

Non-Critical Actions

Noncritical actions are those force-controlled actions for which failure does not result in structural instability or potentially threaten damage such as diaphragm shear and axial forces in diaphragm chords and drag members as well as foundation forces.

Force-controlled noncritical actions shall satisfy:

\[ F_u \leq k_i \xi F_{n,e} \]

where

\[ F_u = \text{the mean demand obtained from the suite of analyses,} \]

\[ F_{n,e} = \text{nominal strength as computed from applicable material codes but based on expected material properties.} \]

\[ \xi = 1.0. \]

\[ k_i = \text{Risk reduction factor given in Table 6.} \]

EXCEPTION: For buildings located in the Los Angeles region if the serviceability acceptance criteria are satisfied per requirements of Section 3.4.5.1, then \( k_i \) may be taken as 1.0.
Upper Limit on Column Axial Forces

• Large axial forces reduce available R/C column ductility

• 2014 LATBSDC
  – MCE: \( P \leq 0.4f'_cA_g \)

• 2010 PEER-TBI
  – MCE: \( P < \) balanced load
    \( \leq 0.3f'_cA_g \)
Acceptance Criteria -- MCE

- Deformation-Controlled Actions:
  - \( \leq K_i \times \) Primary CP limits in ASCE-41
  - Larger values may be used only if substantiated by appropriate laboratory tests.
  - If exceeded, strength degradation, stiffness degradation and hysteretic pinching shall be considered and base shear capacity of the structure shall not fall below 90% of the base shear capacity at deformations corresponding to ASCE 41 Primary CP limits
  - 2010 PEER-TBI is not as rigid and refers to ASCE 41 as a source of information.
APPLICATION EXAMPLES
Applications

• Many tall buildings have been designed using these guidelines in Los Angeles, San Francisco, San Diego, and elsewhere

• Here are some examples
  – Los Angeles:
    – 888 Olive
    – 1133 Olive
    – 1212 Flower Towers
    – Wilshire & Grand
    – Metropolis Tower
  – San Diego
    – 7th & Ash
  – San Francisco
    – Transbay Tower
888 Olive Street in downtown Los Angeles

- 34 stories
- Core wall construction
- Podium
- Subterranean levels
- Basement walls
- Flat plates
- Gravity columns
Thank you!