Guidelines for Nonlinear Structural Analysis and Design of Buildings

Curt Haselton, PhD, PE
Greg Deierlein, PhD, PE
and the ATC 114 Team

Presentation Outline

• Scope and background
• Organization of the Guidelines
• Part I: General Modeling Requirements (including approach to modeling cyclic damage)
• Part IIa: Steel Frame Building Modeling
• Part IIb: RC Frame Building Modeling
• Illustrative Example
Scope/Focus for Modeling Guidelines

- Support primarily *nonlinear dynamic analyses* (monotonic backbone curve and then cyclic deterioration).

- Modeling vs. acceptance criteria:
  - ATC-114 focuses on *modeling* guidelines.
  - Rely on parent document (e.g., ASCE 7 Chp. 16) for the needed acceptance criteria checks and associated hazard levels.

- Adopt a reliability framework. Write the approach around the ASCE 7-16 draft Chapter 16, provide guidance on variability numbers.

- Audience: Practicing engineers with ~Master’s education

ATC-114 Project Team

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  - Project Funding & Review
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  - Jay Harris
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  - Kevin Wong

*Plus - project review panel and other contributors listed in reports*
NIST Seismic Design Tech Brief 4: Nonlinear Structural Analysis for Seismic Design
- General guidance on using nonlinear analysis for design
- Focus on high-level goals and objectives
- Overview of key concepts and assumptions
- Summary of modeling capabilities and resources
- Guidance on NL static & dynamic analysis

- General Nonlinear Modeling
- Steel and RC Moment Frame Components
- Shear Walls and Slab-Column Frames
- Podium Diaphragms and Collectors

PEER Tall Building Initiative:
- 2010 (2017) guidelines
- Supporting documents - http://peer.berkeley.edu/tbi/

LA Tall Buildings Structural Design Council:
- 2011 guidelines
- Annual conference
- Special provisions for RC structures

Guideline Documents
- Performance Objectives
- Design Process and Documentation
- Seismic Input and Modeling Criteria
- Service Level Evaluation
- MCE Level Evaluation
- Documentation and Peer Review
Reference Documents

ASCE 7-16 Minimum Design Loads for Buildings
• Chapter 16 – Seismic Response History Procedures
• Emphasis on nonlinear dynamic analysis
• Analyses and checks for MCE levels
• Selection and scaling ground motions (UHS or CMS)
• Risk/probabilistic basis for demand and acceptance criteria:
  - Deformation-controlled components
  - Force-controlled components

ASCE 41-13 (17) Seismic Evaluation & Retrofit of Existing Buildings
• General performance assessment framework (IO, LS, CP)
• Requirements for assessing properties of existing buildings
• Structural component modeling parameters and acceptance criteria
• Nonlinear static (pushover) analysis procedure

ATC 114: How it is Expected to be Used

ATC-114 provides guidelines for nonlinear modeling to support design in accordance with other standards
ATC 114: Organization of Guidelines

Part I: General Guidelines

Part Ia: Guidelines Specific to Steel Moment Frames

Part Iib: Guidelines Specific to RC Moment Frames

Part Ic: Guidelines Specific to RC Shear Walls

Part IId: Guidelines Specific to Steel Braced Frames

Part I: General

1. Introduction and Scope
2. Overview of NL Modeling and Analysis Procedure
3. General Modeling Requirements
4. Nonlinear Static (Pushover)
5. Nonlinear Response-History
6. Performance Assessment and Acceptance Criteria

Appendices
A: Overview of Methods for RHA
B: Consideration of Uncertainties
C: Calibration of Nonlinear Component Models

Part II: System Specific

1. Introduction and Scope
2. Structural Behavior and Failure Modes
3. NL Modeling of Frames & Components
4. Concentrated Hinge Models
5. Fiber-Type Beam-Column Models
6. Continuum FE Component Models

Appendices
A: Non-ductile frames
B: Illustrative Examples
Types of Nonlinear Analysis Models

(a) Plastic Hinge
(b) Nonlinear Spring Hinge
(c) Finite Length Hinge Zone
(d) Fiber Section
(e) Finite Element

Figure 2-7
Range of structural model types (NIST, 2010).

Types of Nonlinear Analysis Models

Concentrated Hinge

Fiber-Type Elements
Basic Requirements

- Expected Properties
  - materials
  - model parameters
  - mass
  - gravity loads (1.0D + 0.5L)

- Geometric Nonlinear (P-\(\Delta\)) Effects

- Structural Behavior and Failure Modes
  - "simulated" vs "non-simulated" effects
  - influence of non-structural components

Modeling Approach for Cyclic Loading

Typical Approach: ASCE 41
type curve that implicitly incorporates cyclic deterioration
Modeling Approach for Cyclic Loading

Nojavan et al., 2014

Modeling Approach for Cyclic Loading

Nojavan et al., 2014
Modeling Approach for Cyclic Loading

Nojavan et al., 2014

Modeling Approach for Cyclic Loading

Nojavan et al., 2014
Modeling Approach for Cyclic Loading

Ingham et al., 2001

Modeling Approach for Cyclic Loading

Suzuki and Lignos, 2015
Modeling Approach for Cyclic Loading

Suzuki and Lignos, 2015

Modeling Approach for Cyclic Loading

(Tremblay et al. 1997, ATC-72-1)
Modeling Approach for Cyclic Loading

- Response is typically between and depends on loading-history.
- First-cycle envelope.
- Monotonic envelope.

Experimental Results

- Model Prediction

- Cyclic (between cycles)
- In-cycle

Test 170 (kN, mm, rad):
- $\theta_{y} = 0.0131$
- $\theta_{stf} = 0.0012$
- $M_y = 3.462725 \times 10^5$ kN-mm
- $\lambda = 81$, $c = 1.00$
- $i/l$ isPDeltaRemoved = 1

ASCE 41

$K_e$
Modeling Approach for Cyclic Loading

Type A – DIRECT SIMULATION:
Cyclic and in-cycle degradation explicitly modeled during analysis; backbone curve hardens/softens as a function of damage (e.g., Ibarra-Krawinkler)

Type B – DEGRADED BACKBONE:
Post-peak capping and cyclic degradation modeled with fixed backbone curve that remains fixed during analysis; backbone curve is defined based on measured (data) or assumed cyclic softening (e.g., ASCE 41)

Type C – ELASTIC-PLASTIC:
Model captures cyclic degradation, but post-peak softening is not modeled; an ultimate limit state is imposed to avoid unconservative analyses in post-peak realm.
### ATC 114: Organization of Guidelines

#### Part I: General

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**Appendices**

- A: Overview of Methods for RHA
- B: Consideration of Uncertainties
- C: Calibration of Nonlinear Component Models

#### Part I: General - Chapter 3

1. General Guidelines
2. Seismic Mass
3. Gravity Loads
4. Geometric Nonlinearities
5. Material Properties
6. Floor Diaphragms and Collectors
7. Equivalent Viscous Damping
8. Torsion
9. Foundation Modeling
10. Soil-Structure Interaction
11. Gravity Framing Systems
12. Non-Modeled Building Stiffness
13. Residual Drifts
Diaphragms, Collectors & Distributors

<table>
<thead>
<tr>
<th>Component</th>
<th>Service Level Earthquake</th>
<th>Design Earthquake</th>
<th>MCEx</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete and Concrete over Metal Deck</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Out-of-Plane [Plate] (%)</td>
<td>35 - 50%</td>
<td>25 - 35%</td>
<td>25 - 35%</td>
</tr>
<tr>
<td>In-Plane [Membrane] (%)</td>
<td>30 - 40%</td>
<td>10 - 30%</td>
<td>5 - 30%</td>
</tr>
<tr>
<td>Untopped Metal Deck</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Out-of-Plane [Plate]</td>
<td>**</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>In-Plane [Membrane]</td>
<td>**</td>
<td>**</td>
<td>**</td>
</tr>
</tbody>
</table>

* Range represents secant stiffness at shear stresses from $f_y$ to $f_y'$

Energy Dissipation and Damping

Is energy dissipation explicitly modeled or approximated by equivalent viscous damping?

<table>
<thead>
<tr>
<th>Phenomena</th>
<th>How is it Usually Represented</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yielding of members in lateral force-resisting system</td>
<td>Explicitly modeled although onset of yielding may not be fully captured</td>
</tr>
<tr>
<td>Yielding of members in secondary gravity system</td>
<td>Explicitly modeled (like lateral force resisting system) or through EVD</td>
</tr>
<tr>
<td>Cracking of floor slab due to beam flexure and differential uplift in tall buildings (with significant column or wall elongation)</td>
<td>EVD or explicitly in the beam or slab models</td>
</tr>
<tr>
<td>Beam-column yielding and floor slab cracking in gravity framing connections</td>
<td>EVD or explicit hinge models</td>
</tr>
<tr>
<td>Beam-column joint panel yielding</td>
<td>EVD or explicit spring models</td>
</tr>
<tr>
<td>Localized yielding and/or bolt slippage or bond slip in connections</td>
<td>EVD or explicit hinge models</td>
</tr>
</tbody>
</table>
Energy Dissipation and Damping

Foundation (Substructure)

- Bearing, rotation, and possible uplift of spread footings or mat foundation: EVD or explicit foundation hysteretic springs and dashpots, or both
- Axial tension/compression deformations and lateral bearing of piles: EVD or explicit foundation hysteretic springs and dashpots, or both
- Lateral bearing and gap opening against footings, mat, pile caps, and/or basement walls: EVD or explicit foundation hysteretic springs and dashpots, or both
- Localized reinforcing steel yielding and concrete cracking in foundation components: EVD (unless explicitly modeled)
- Boundary interactions between the foundation and soil: EVD (unless explicitly modeled)

Nonstructural Components

- Racking and flexure of exterior cladding: EVD
- Racking of full-height partition, stairwell, and elevator walls: EVD
- Racking of partial-height partition walls: EVD
- Racking of stair systems: EVD
- Dynamic interaction with large mechanical equipment: EVD
- Interaction with HVAC, mechanical and electrical risers: EVD

Energy Dissipation and Damping

\[ \zeta_{critical} = \frac{0.36}{\sqrt{H}} \leq 0.05 \]

Ref: Cruz & Miranda, 2016; Bernal et al., 2015
Part I: Acceptance Criteria

- So far, we have talked about doing a lot of detailed nonlinear modeling.
- **Structural responses** do not tell us about performance until to compare with the acceptance criteria.
- The acceptance criteria will depend on what document is being used to govern the design.
Part I: Acceptance Criteria

Will give examples from ASCE7-16 here.

Part I: Acceptance Criteria

- Big Focus of ASCE 7-16 Chapter 16 Revision:
  Develop acceptance criteria more clearly tied to the ASCE7 safety goals.

- Explicit Goal: Acceptable collapse probability.
- Implicit Verification Approach: Use average structural responses (with 11 motions) to show compliance.

<table>
<thead>
<tr>
<th>Risk Category</th>
<th>Tolerable Probability of Collapse</th>
<th>Ground Motion Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>I or II</td>
<td>10%</td>
<td>MCE&lt;sub&gt;r&lt;/sub&gt;</td>
</tr>
<tr>
<td>III</td>
<td>6%</td>
<td>MCE&lt;sub&gt;r&lt;/sub&gt;</td>
</tr>
<tr>
<td>IV</td>
<td>3%</td>
<td>MCE&lt;sub&gt;r&lt;/sub&gt;</td>
</tr>
</tbody>
</table>
**Part I: Acceptance Criteria**

### Uncertainties at Each Step of Assessment Process

- **Ground Motion Uncertainty**
  - Hazard (uncertainty, e.g., S, level)
  - Record-to-record variability (from ground motion isolating)

- **Structural Modeling Uncertainty**
  - Input parameter variability (e.g., element strength)
  - Uncertainty between model prediction and reality

- **Component Damage Uncertainty**
  - (uncertainty in damage for a given level of structural response)

- **Consequence Uncertainty**
  - (uncertainty in consequence, e.g., repair costs, for a given level of damage)

### Commentary on Common Approaches

- Common practice is to use an accelerations spectrum for a specific return period. Ground motions are then scaled to the spectrum without regard to variability.
- Common practice is to use a mean-based structural model and then attempt to handle uncertainties in the acceptance criteria (using demand and capacity factors).
- Common practice is not to look directly at component damage and final consequences (such as repair costs, repair time, etc.).

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**Part I: Acceptance Criteria**

- **Graph** showing the probability density function value over the range of demand and capacity.
  - **Demand**
  - **Capacity**

<table>
<thead>
<tr>
<th>Risk Category</th>
<th>Tolerable Probability of Collapse</th>
<th>Ground Motion Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 or II</td>
<td>10%</td>
<td>MCE_R</td>
</tr>
<tr>
<td>III</td>
<td>6%</td>
<td>MCE_R</td>
</tr>
<tr>
<td>IV</td>
<td>3%</td>
<td>MCE_R</td>
</tr>
</tbody>
</table>

**ASCE 7-16**: Uncertainties considered in specified acceptance criteria, including demand & capacity factors.

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Part I: Acceptance Criteria

- Force-controlled (brittle) components:
  
  2.0 \( I_e F_u \leq F_e \) for "critical" (comparable to PEER-TBI-v1)
  
  1.5 \( I_e F_u \leq F_e \) for "ordinary"
  
  1.0 \( I_e F_u \leq F_e \) for "non-critical" (judgment)

  \( F_u \) = mean demand (from 11 motions)
  
  \( F_e \) = expected strength

  Critical = failure causes immediate global collapse

  Ordinary = failure causes local collapse (one bay)

  Non-critical = failure does not cause collapse

  Contrast: Much more stringent than the average-based approach that could be used in ASCE 41.

Part I: Acceptance Criteria

- Deformation-controlled (ductile) components:

  1) Limits applied to mean demands:
  
  - ASCE 41 Limit: \( CP / I_e \)
  
  - Loss in Vertical Load Carrying Capacity:
    \( \phi_s \Delta_{LVCC} \) where \( \phi_s \) is equal to
    
    \( 0.3 / I_e \) critical
    \( 0.5 / I_e \) ordinary

  2) Limits of analysis model for peak demands from individual ground motions
Part I: Acceptance Criteria

- Drift limits:
  - Mean drift $\leq 2.0 \times$ (normal limit)
  - The factor of two comes from:
    - $1.5 = \text{MCE} / \text{DBE}$
    - $1.25 = \text{Approx. ratio of } R / C_d$
    - $1.1 = \text{A little extra because we trust NL RHA more}$

Part I: Acceptance Criteria

- Treatment of “collapses” and “unacceptable responses”:
  - Past Treatment in ASCE7-10: Nothing but silence….
  - ASCE7-16 Criteria:
    - Basic Case: Allow up to 1/11 “collapses” but not 2/11.
    - With Spectral Matching: Require 0/11 collapses.
    - For Risk Categories III-IV: Require 0/11 collapses.
  - “Collapses” are more generally called “unacceptable responses” and include:
    - True dynamic instability,
    - Analytical solution fails to converge,
    - Deformation-controlled demands exceed valid modeling range,
    - Critical/ordinary force-controlled demands exceed capacity,
    - Predicted deformation demands on elements not modeled exceed the deformations at gravity load failure.
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Part IIa – Steel SMF

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Appendices
A: Pre-Northridge Steel SMF
B: Illustrative Example

Photo: D. Lignos
Part IIa: Steel

Expected Behavior:
- Deterioration modes
- Likelihood of occurrence

### Table 2-1 Nonlinear Behavioral Effects to Consider in Nonlinear Analysis

<table>
<thead>
<tr>
<th>Component</th>
<th>Nonlinear Behavior</th>
<th>Moment Frame Types</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>Yielding followed by gradual deterioration due to local buckling and/or lateral-torsional buckling and ductile tearing</td>
<td>SFM</td>
</tr>
<tr>
<td></td>
<td>SAME AS ABOVE – except with rapid deterioration</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Yielding followed by rapid deterioration due to sudden connection fracture (ductile and/or brittle fracture)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>No yielding or limited yielding followed by rapid deterioration due to local buckling and/or lateral-torsional buckling and ductile tearing</td>
<td></td>
</tr>
<tr>
<td></td>
<td>No yielding or limited yielding followed by rapid deterioration due to sudden connection fracture (ductile and/or brittle fracture)</td>
<td></td>
</tr>
<tr>
<td>Column</td>
<td>Yielding followed by gradual deterioration due to local buckling and/or limited lateral-torsional buckling with possible ductile fracture (fracture)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SAME AS ABOVE – except with rapid deterioration</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Yielding followed by rapid deterioration due to sudden column splice fracture (ductile and/or brittle fracture)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>No yielding or limited yielding followed by rapid deterioration due to local buckling and/or lateral-torsional buckling and ductile fracture</td>
<td></td>
</tr>
<tr>
<td></td>
<td>No yielding or limited yielding followed by rapid deterioration due to sudden column splice fracture (ductile and/or brittle fracture)</td>
<td></td>
</tr>
<tr>
<td>Panel Zone</td>
<td>Limited yielding and shear hardening after beams have begun to yield</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Significant yielding prior to yielding in adjacent beams or columns</td>
<td></td>
</tr>
<tr>
<td>Connection</td>
<td>Significant rotation capacity prior to gradual loss of flexural and/or shear capacity</td>
<td></td>
</tr>
<tr>
<td>Column Base</td>
<td>Limited rotation capacity prior to sudden loss of flexural and/or shear capacity</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Full fixity with limited yielding and deformation</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Partial fixity with gradual yielding and deformation and significant rotation-capacity</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Partial or full fixity with sudden deterioration due to fracture or rupture in the plate</td>
<td></td>
</tr>
</tbody>
</table>

**Figure:**
- **Concentrated Hinge**
- **Continuum FEM**
- **Fiber-Type Elements**
Connection Panel Zone

Panel Zone Shear Demand

Panel Zone Shear Response

ANSI/AISC 360 Section J10.6

\[ R_n = 0.60 F_y d_c t_w \left( 1 + \frac{3 b_c f_y^2}{d_b d_c t_w} \right) \]

\[ R_n = 0.60 F_y d_c t_w \]

Initial Yield Strength

Panel Zone Shear Response
### Connection Panel Zone

**Panel Zone – NL Models**

- **Equation:**
  
  \[ V_{x,y} = \frac{M_{A1} + M_{A2}}{d_y - t_y} \leq V_y \]

- **Equation:**
  
  \[ V_y = R_n = 0.60F_y d_y t_w \]

**Panel Zone – Elastic Models**

### Beam Hinge Model

**Idealized Envelope Curve Parameters**

- **Monotonic vs. Cyclic Backbone**

**Type A – Direct Simulation**

**Type B – Degraded Backbone**
**Deteriorating Beam Hinge Model**

*Semi-Empirical* -- calibrated from tests, fiber analyses, and mechanics:
- Secant Stiffness ($E_{\text{int}}$)
- Yield Strength ($M_y$)
- Hardening Stiffness

*Empirical* - calibrated from tests:
- Capping (peak) point
- Post-peak unloading (strain softening) stiffness
- Hysteretic stiffness/strength degradation

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**Deteriorating Beam Hinge Model**

Monotonic Backbone
- measured from monotonic tests
- inferred from cyclic data
  (e.g., calibration with Ibarra-Krawinkler model in OpenSees)

First-Cycle Envelope
- measured from symmetric cyclic tests
- tri-linear envelope parameters based on $M_{\text{max}}$
Deteriorating Beam Hinge Model

Strength Parameters

\[ M_y^* = 1.1 \cdot R_y \cdot Z_{RBS} \cdot F_{y,u} \]

\[ M_u^* = 1.15 M_y \]  \textit{COV}_S = 0.1

\[ M_r^* = 0.3 M_y \]

Deformation Parameters

\[ \theta_{p}^* = 0.55 \left( \frac{h}{l_w} \right)^{-0.25} \left( \frac{b_f}{2 l_f} \right)^{-0.7} \left( \frac{L_b}{r_y} \right)^{0.5} \left( \frac{L}{d} \right)^{0.8} \]

\[ \theta_{pc}^* = 20.0 \left( \frac{h}{r_w} \right)^{0.8} \left( \frac{b_f}{2 l_f} \right)^{0.1} \left( \frac{L_b}{r_y} \right)^{-0.6} \]

\[ \theta_{u}^* = 0.08 \]  \textit{COV} = 0.4

\[ \theta_{cap,pl}^* = 0.1 \]  \textit{COV} = 0.3

\[ \theta_{pc}^* = 0.1 \]  \textit{COV} = 0.3

(Suzuki and Lignos 2015)

Deteriorating Beam Hinge Model

Pre-Peak Plastic Rotation, \( \theta_p \)

\[ \theta_p^* = 0.55 \left( \frac{h}{l_w} \right)^{-0.25} \left( \frac{b_f}{2 l_f} \right)^{-0.7} \left( \frac{L_b}{r_y} \right)^{0.5} \left( \frac{L}{d} \right)^{0.8} \]

Statistically fit to \(~50\) tests

Measured vs Fitted

Histogram of residuals

Residuals vs fitted
Deteriorating Beam Hinge Model

Illustrative Examples of New Model Calibrations

Composite Beam Effects

(Suzuki and Lignos 2015)
Composite Beam Effects

Composite Beam Action

- detailing considerations (studs, bearing strength, slab reinforcement)
- increased strength
- reduced deterioration

Steel Columns: Test Data

- Elkady and Lignos, 2015
- Ozkula and Uang, 2015
Columns: Effects of Loading Protocol

Chord Rotation $\theta [\text{rad}]$

Column End Moment [kN-m]

Monotonic Curve
First-Cycle Envelope

(Suzuki and Lignos 2015)

Steel Columns: FEM Analyses

Meshing
Residual Stresses
Imperfections

Source: Elkady and Lignos (2015)
Steel Columns: FEM Analyses

Source: Elkady and Lignos (2015)

Columns: Hinge Model Calibration

Elkady and Lignos (2015)
Ozkula and Uang (2015)
**Columns: Hinge Model Calibration**

\[ M'_y = 1.15 Z R_y F_y \left( 1 - \frac{P_y}{P_{yw}} \right) \quad \frac{P_y}{P_{yw}} \leq 0.20 \]

\[ M'_y = 1.15 Z R_y F_y \left[ \frac{9}{8} \left( 1 - \frac{P_y}{P_{yw}} \right) \right] \quad \frac{P_y}{P_{yw}} > 0.20 \]

\[ M_u' = a'M_y \]

\[ a' = 9.5 \left( \frac{h}{I_y} \right)^{0.24} \left( \frac{I_y}{r_y} \right)^{0.18} \left( 1 - \frac{P_y}{P_{yw}} \right)^{0.23} \geq 1.0 \text{ and } < 1.3 \]

\[ M_e' = \left( 0.4 - 0.4 \frac{P_e}{P_{yw}} \right) M_y \]

\[ \theta'_{e,15} = 15 \left( \frac{h}{I_y} \right)^{-2.7} \left( \frac{I_y}{r_y} \right)^{-2.3} \left( 1 - \frac{P_y}{P_{yw}} \right)^{2.0} \leq 0.10 \]

\[ \theta'_{e,14} = 14 \left( \frac{h}{I_y} \right)^{-3.9} \left( \frac{I_y}{r_y} \right)^{-3.5} \left( 1 - \frac{P_y}{P_{yw}} \right)^{-2.0} \leq 0.10 \]

\[ \theta_{ei} = 0.08 \left( 1 - 0.6 \frac{P_e}{P_{yw}} \right) \]

Elkady and Lignos (2015); calibrated to 50 tests & 400 FE analyses

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**Gravity Beam (Shear Tab) Connections**

Gravity Beam (Shear Tab) Connections


Steel: Appendix on PN Connections

Fracture Mechanics Based Stress Limits for Fiber Cross Section Model

\[ \sigma_{cr} = \frac{K_{IC}}{F(a_0)} \]

\[ F(a_0) = 1.2 + 2a_0 \quad \text{(bottom)} \]

\[ F(a_0) = 0.5 + 2a_0 \quad \text{(top)} \]

\[ K_{IC,\text{dynamic}} \approx \sqrt{\alpha CVN} \cdot E \]
Steel: Appendix on PN Connections

Strain limit for fracture critical (pre-Northridge) connections that pass the stress check:

\[ \varepsilon_{cr} = \theta_{plastic}(d_b/l_p) \]

\[ \theta_{plastic} = \alpha r e^{(3.6 + 0.04d_b)} \]

- \( \alpha r = 1 \) (bottom flange); 2 (top flange)
- \( d_b = \) beam depth

Calibration of fiber hinge model to test data

Data from Ramirez, Lignos, Miranda, Kolios 2012

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- Scope and background
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- Part I: General Modeling Requirements (including approach to modeling cyclic damage)
- Part IIa: Steel Frame Building Modeling
- Part IIb: RC Frame Building Modeling
- Illustrative Example
Part IIb: Concrete

Concentrated Hinge

Fiber-Type Elements

RC Beam-Column Hinge Models
RC Beam-Column Hinge Models

Key Response Parameters:
- strength
- initial stiffness
- post-yield stiffness
- post-rotation (capping) capacity
- post-capping slope
- cyclic deterioration rate

Calibration Process:
- 250+ columns (PEER database)
- flexure & flexure-shear dominant
- calibrated to expected values

![Graph showing key response parameters](image1)

![Graph showing calibration process](image2)
RC Beam-Column Hinge Models

\[ \theta_p = 0.12(1 + 0.55\rho_{sh})(0.16)\sqrt{(0.02 + 40\rho_{sh})^0.43} \left(0.54\right)^{0.012\rho_{sh}f_c}(0.66)^{0.12} \left(2.27\right)^{0.04} \]

Key Design/Detailing Variables:
- \(\rho_{sh}\) – amount of steel stirrups
- \(\nu\) – axial load ratio \((P/Af'c)\)
- \(s_n\) – tie spacing
- \(\alpha_{sl}\) – joint bond slip

Dispersion:
- \(\sigma_{sh} = 0.54\)

---

\[ \theta_{pc} = (0.76)(0.031)\sqrt{(0.02 + 40\rho_{sh})^1.02} \leq 0.10 \]

Table 3-3: Empirical Plastic Rotation Values, \(\theta_p\) and \(\theta_{pc}\), for a Representative Column Section (Haselton et al., 2008)

<table>
<thead>
<tr>
<th>(v = P/Af'c)</th>
<th>(\theta_p)</th>
<th>(\theta_{pc})</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>0.002</td>
<td>0.031</td>
</tr>
<tr>
<td></td>
<td>0.006</td>
<td>0.047</td>
</tr>
<tr>
<td></td>
<td>0.020</td>
<td>0.077</td>
</tr>
<tr>
<td>0.6</td>
<td>0.002</td>
<td>0.012</td>
</tr>
<tr>
<td></td>
<td>0.006</td>
<td>0.019</td>
</tr>
<tr>
<td></td>
<td>0.020</td>
<td>0.031</td>
</tr>
</tbody>
</table>
Comparison of $\theta_{p,cyclic}$ and "a"

Comparison of $(\theta_p + \theta_{pc})_{cyclic}$ and "b"
RC Fiber Models

Confined concrete

First hoop fracture

Unconfined concrete

Assumed for cover concrete

Effectively confined core

Compressive Stress $\sigma_c$

Compressive Strain $\varepsilon_c$
RC Fiber Models

Adapted from Ghannoum and Moehle, 2012
Presentation Outline

- Scope and background
- Organization of the Guidelines
- Part I: General Modeling Requirements (including approach to modeling cyclic damage)
- Part IIa: Steel Frame Building Modeling
- Part IIb: RC Frame Building Modeling
- Illustrative Example

Illustrative Example - Steel SMF

5-Story Steel Frame Building
- SMF with RBS connections
- Seismic Design Category D
  \[ S_{M1} = 1g; \ T_1 = 1.1s \ (1.6s) \]
  \[ R=8; \ Cs = 0.07 \]
  MRSA design (ASCE 7-10)
  \[ SDR_{max} = 1.6\% \]
- Irregularity
  setback at 4-th floor
  vertical irregularity in mass, stiffness & strength
  1-bay offset of SMF
  1 discontinuous column
- NLRHA with PERFORM 3D
Flowchart of Analysis Approach

PERFORM 3D Model

SMF Framing Lines

5% is used where V exceeds 0.5Vn
Illustrative Example - Steel SMF

Input Ground Motions
- ASCE 7-16
- MCE, (UHS) Target
- 11 pairs of GM, representing characteristic earthquakes (causal features)
- Period Range 0.2T1 to 2T1 (T1 = 1.6s)

Acceptance Criteria – Deformation Controlled

Table B-12 Demand Parameters and Acceptance Criteria for ASCE/SEI 7-16 MCE Analysis – Inelastic Behavior

<table>
<thead>
<tr>
<th>Item</th>
<th>Action</th>
<th>Type</th>
<th>Acceptance Criteria</th>
<th>Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>RBS Hinges</td>
<td>Deformation- Controlled</td>
<td>Ordinary</td>
<td>$\theta_e \leq \theta_F$</td>
<td>ASCE/SEI 41-17 Table 9-6.2 and NIST GCR 17-917-45 Sec 4.5</td>
</tr>
<tr>
<td>Non-RBS Hinges at Transfer</td>
<td>Deformation- Controlled</td>
<td>Critical</td>
<td>$\theta_e \leq \theta_F$</td>
<td>ASCE/SEI 41-17 Table 9-6.2 and NIST GCR 17-917-45 Sec 4.5</td>
</tr>
<tr>
<td>Column with $P_d / P_{cr} &gt; 0.6$</td>
<td>Deformation- Controlled</td>
<td>Critical</td>
<td>$\theta_e \leq \theta_F$</td>
<td>ASCE/SEI 41-17 Table 9-6.1 and NIST GCR 17-917-45 Sec 4.5</td>
</tr>
<tr>
<td>Column Hinges at Base</td>
<td>Deformation- Controlled</td>
<td>Ordinary</td>
<td>$\theta_e \leq \theta_F$</td>
<td>NIST GCR 17-917-45 Sec 4.5</td>
</tr>
<tr>
<td>Panel Zone</td>
<td>Deformation- Controlled</td>
<td>Ordinary</td>
<td>$\gamma \leq \gamma_F$ and $\gamma_F \leq \gamma_{Pleq}$</td>
<td>ASCE/SEI 41-17 Table 9-6.2 and ASCE/SEI 41-17 Eq. 9-18 and NIST GCR 17-917-45 Sec 4.5</td>
</tr>
<tr>
<td>Gravity Connection</td>
<td>Deformation- Controlled</td>
<td>Ordinary</td>
<td>$\theta_e \leq \theta_F$</td>
<td>ASCE/SEI 41-17 Table 9-6.2 and NIST GCR 17-917-45 Sec 4.5</td>
</tr>
</tbody>
</table>
### Acceptance Criteria – Force Controlled

<table>
<thead>
<tr>
<th>Item</th>
<th>Action</th>
<th>Type</th>
<th>Acceptance Criteria</th>
<th>Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-Transfer</td>
<td>Force-Controlled</td>
<td>Ordinary</td>
<td>$V_{re} \geq 1.5(V_s - V_{so}) + V_{so}$</td>
<td>ACI 318</td>
</tr>
<tr>
<td>Diaphragms</td>
<td>Force-Controlled</td>
<td>Critical</td>
<td>$V_{re} \geq 2.0(V_s - V_{so}) + V_{so}$</td>
<td>ACI 318</td>
</tr>
<tr>
<td>Transfer Diaphragms</td>
<td>Force-Controlled</td>
<td>Critical</td>
<td>$P_{re} \geq 2.0(P_s - P_{so}) + P_{so}$</td>
<td>AISC 341 and 360</td>
</tr>
<tr>
<td>Column with $P_o/P_{cr} &gt; 0.6$</td>
<td>Force-Controlled</td>
<td>Critical</td>
<td>$M_{re} \geq 2.0(M_s - M_{so}) + M_{so}$</td>
<td>NIST GCR 17.917-45 Sec 4.5</td>
</tr>
<tr>
<td>Column Splice</td>
<td>Force-Controlled</td>
<td>Critical</td>
<td>$P_{re} \geq 2.0(P_s - P_{so}) + P_{so}$</td>
<td>Geotechnical Report and</td>
</tr>
<tr>
<td>(non-CJP)</td>
<td></td>
<td></td>
<td>$M_{re} \geq 2.0(M_s - M_{so}) + M_{so}$</td>
<td>ACI 318</td>
</tr>
<tr>
<td>Mat Foundation</td>
<td>Force-Controlled</td>
<td>Ordinary</td>
<td>$q_{re} \geq 1.5(q_s - q_{so}) + q_{so}$</td>
<td>Geotechnical Report and</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ASCE/SEI 41-17 Chapter 8</td>
</tr>
</tbody>
</table>

### Story Drifts - MCEr

#### GL 0 Story Drift (North-South)

- Level 5: 3.3%
- Level 4: 2.8%
- Level 3: 2.4%
- Level 2: 2.1%
- Level 1: 1.5%

#### GL 1 Story Drift (East-West)

- Level 5: 1.8%
- Level 4: 2.0%
- Level 3: 2.2%
- Level 2: 2.2%
- Level 1: 1.8%
**Story Drifts – Influence of Gravity Framing**

- PR Gravity Connections
- Pin-Connected Connections

**Hinge Rotations - MCEr**

- Beam Hinge, $\theta$
- Column Base Hinge, $\theta$
Panel Zones

- Design intentionally violated AISC 341 PZ strength requirements
- 10% of non-conforming PZ exceeded the fracture control limit of $\gamma_{p,z} < 0.015$ rad
- Conforming PZ all had minimal deformations

Frame Overstrength

Shear in Frames

Overturning Moment in Frames
Influence of Diaphragm Stiffness

Frame Layout

- Diaphragm at floor 4 considered critical, so force demand is multiplied by 2. Load factor of 1.5 used at other floors.
- Transfer level: $V_u = 7.4$ and $7.6$ klf (for 5% and 30% stiffness)
- Other levels $V_u = 7.5$ klf to 2.6 klf
- Even rigid diaphragm did not make much difference in this case.

Shear in Frames

Illustrative Example - Summary

1) Prescriptive SMF Design (R=8) evaluated by NLRHA
2) Story Drift: mean 3.3%<4%; peak 6.2% okay
3) Deformation Limits
   - RBS hinge okay
   - PZ – exceed limits, but would be okay for AISC conforming design
4) Other Limit States
   - Gravity Connection Rotations ($0.037 < 0.10$ to $0.09$)
   - Force checks not presented, but should be confirmed, especially given the high overstrength
5) ASCE 7-16 deformation limits (with f-factors) are more conservative of ASCE 41 CP limits
6) Results are not overly sensitive to diaphragm stiffness, gravity framing, composite beam action
Concluding Remarks

1) Nonlinear response history analysis is an effective tool to inform design, but it should not replace good design
   • reliable load paths and details
   • capacity design
   • well-behaved response
2) Develop clear objectives for the analysis (basis of design)
   • acceptance criteria
   • demand parameters
3) Quality assurance
   • utilization of elastic and nonlinear static analyses
   • selective plots of response quantities
   • sensitivity analyses
   • selected validation with test data
4) Effective presentation of results

More to come ...

Part I: General Guidelines

Part IIa: Guidelines Specific to Steel Moment Frames

Part IIb: Guidelines Specific to RC Moment Frames

Part IIc: Guidelines Specific to RC Shear Walls

Part IIId: Guidelines Specific to Steel Braced Frames

ATC-114: Guidelines for Nonlinear Structural Analysis and Design of Buildings