2020 NEHRP Recommended Seismic Provisions: Design Examples, Training Materials, and Design Flow Charts

FEMA P-2192-V2/November 2021

Volume II: Training Materials
2020 NEHRP (National Earthquake Hazards Reduction Program) Recommended Seismic Provisions: Training Materials

Prepared for

Federal Emergency Management Agency

U.S. Department of Homeland Security

By

Building Seismic Safety Council

National Institute of Building Sciences

Washington, D.C.
The National Institute of Building Sciences (NIBS) brings together members of the building industry, labor and consumer interests, government representatives, and regulatory agencies to identify and resolve problems and potential problems around the built environment. NIBS is a nonprofit, non-governmental organization established by Congress in 1974.

The Building Seismic Safety Council (BSSC) was established in 1979 under the auspices of NIBS as a national platform for dealing with the complex regulatory, technical, social, and economic issues involved in developing and promulgating building earthquake hazard mitigation regulatory provisions that are national in scope. By bringing together in the BSSC all of the needed expertise and all relevant public and private interests, it was believed that issues related to the seismic safety of the built environment could be resolved and jurisdictional problems overcome through authoritative guidance and assistance backed by a broad consensus. BSSC’s mission is to enhance public safety by providing a national forum that fosters coordination of and improvements in seismic planning, design, construction, and regulation in the building community.

This report was prepared under Contract HSFE60-15-D-0022 between the Federal Emergency Management Agency and the National Institute of Building Sciences.

This FEMA resource document can be obtained from the FEMA online library: https://www.fema.gov/emergency-managers/risk-management/building-science/earthquakes.
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Volume II: Training Materials

FEMA P-2192-V2 Volume II Training Materials, as one of the 2020 NEHRP Provisions’ supporting and educational products, contains a set of slides for use with FEMA P-2192-V1, Volume I Design Examples. The training materials are developed to correspond to each chapter in the Volume I Design Examples to provide background information, highlights of relevant code changes, and important application details.

The training materials are prepared to be suitable for use in self-learning, professional training courses, and university classes for teaching related provisions in the 2020 NEHRP Recommended Seismic Provisions for New Buildings and Other Structures (FEMA P-2082) and the ASCE/SEI 7-22 Minimum Design Loads and Associated Criteria for Buildings and Other Structures. This training material presentation is intended to remain complete in its entirety even if used by other presenters. While the training material could be tailored for use in other presentations, we caution users to account for issues of completeness and interpretation if only part of the material is used. We also strongly suggest users give proper credit/citation to this report and related chapter authors.

Images in the Volume II Training Materials presentations taken from the Volume I Design Examples or the NEHRP Provisions are typically not cited in the Volume II Training Materials. The full references for partial citations in the Training Materials presentations are typically given in the Design Examples references.
# Table of Contents

Chapter 1 – Introduction to the 2020 NEHRP Provisions Design Examples, Bret Lizundia

Chapter 2 (Sections 2.1 to 2.6) – Fundamentals, James Harris

Chapter 2 (Section 2.7) – Resilience-Based Design, David Bonowitz

Chapter 3 (Section 3.2 – Part 1) – The 2018 Update of the USGS National Seismic Hazard Model, Sanaz Rezaeian

Chapter 3 (Section 3.2 – Part 2) – Dissection of the Example Changes to the MCE\textsubscript{R} Ground Motion Values, Lucas Luco

Chapter 3 (Section 3.3) – New Multi-Period Response Spectra and Ground Motion Requirements, Charles A. Kircher

Chapter 3 (Section 3.4) – Additional Revisions to Ground-Motion Provisions, C.B. Crouse

Chapter 4 – Reinforced Concrete Ductile Coupled Shear Walls, S.K. Ghosh and Prabuddha Dasgupta

Chapter 5 – Seismic Design of Coupled Composite Plate Shear Walls/Concrete Filled (C-PSW/CF), Soheil Shafaei and Amit H. Varma

Chapter 6 – Cross-Laminated Timber (CLT) Shear Walls, Phillip Line and M. Omar Amini

Chapter 7 – Horizontal Diaphragm Design, Kelly Cobeen

Chapter 8 – Nonstructural Components: Fundamentals and Design Examples, Bret Lizundia
Chapter 1 Introduction to the 2020 NEHRP Provisions Design Examples

2020 NEHRP Provisions Training Materials
Bret Lizundia, S.E., Rutherford + Chekene
Learning Objectives

- Understand the role of the *NEHRP Provisions* in seismic code development
- Gain an awareness of seminal past seismic code changes
- Understand key updates to the 2020 *NEHRP Provisions* and to ASCE/SEI 7-22
- Understand what is contained in the 2020 *Design Examples* and how the document can be used

Acknowledgement: Images are taken from FEMA P-2192-V1 and FEMA P-2192-V2 unless otherwise noted.
Outline of Presentation

- Overview of the 2020 NEHRP Provisions
  - Intent
  - Relationship of the Provisions to ASCE/SEI 7-22
- Summary of notable earthquakes and their impact on seismic design
- History and role of the NEHRP Provisions in advancing seismic design
- Highlights of major updates in the NEHRP Provisions and seismic provisions of ASCE/SEI 7-22
- Introduction to the organization and content in the new Design Examples
Overview of the 2020 NEHRP Provisions
The NEHRP Recommended Seismic Provisions

• The starting point in the U.S. seismic standards development process

• Major ASCE/SEI 7 seismic analysis and design concepts originate in the NEHRP Provisions
Intent of the 2020 NEHRP Provisions

The Provisions are the “minimum recommended requirements for design and construction of new buildings. The objectives of these provisions are to provide reasonable assurance of seismic performance that will:

1. Avoid serious injury and life loss due to
   a. Structural collapse
   b. Failure of nonstructural components or systems
   c. Release of hazardous materials
2. Preserve means of egress
3. Avoid loss of function in critical facilities, and
4. Reduce structural and nonstructural repair costs where practicable.”

From FEMA P-2082-1
From Research to Improved Standards and Seismic Design Practice

Engineering Research
- FEMA, NIST

Post-Earthquake Observations
- FEMA, NIST, NSF, USGS

Basic Engineering Research
- NSF

Field and Research Experience
- Private Sector & Professional Societies

U.S. Seismic Design Maps
- FEMA/BSSC, USGS

NEHRP Recommended Seismic Provisions
- FEMA/BSSC

National Standards and Model Building Codes
- ASCE/SEI 7-22, IBC 2024

Implementation

Slide adapted from Yuan (2021) SEAONC presentation
How US Seismic Codes are Developed

Each provisions cycle starts with a FEMA supported assessment of current research results pertaining to seismic provisions, especially the research funded by the four NEHRP agencies.

- Proposals by Technical Subcommittees
- Proposals by PUC Members
- Significant Technical Proposals by Others
  Including those submitted by the ASCE Seismic Subcommittee

BSSC PUC

FEMA / BSSC NEHRP Provisions

Used and Codified by ASCE / SEI 7

Adoption by IBC / IRC / IEBC

From FEMA P-2156
### Main Committee
- 23 voting members
- 7 non-voting advisors

### Issue Teams
- IT 1 - Seismic Performance Objectives
- IT 2 - Seismic Resisting Systems and Design Coefficients
- IT 3 - Modal Response Spectrum Analysis
- IT 4 - Shear Wall Design
- IT 5 - Nonstructural Components
- IT 6 - Nonbuilding Structures
- IT 7 - Soil Foundation Interaction
- IT 8 - Base Isolation and Energy Dissipation
- IT 9 - Diaphragm Issues
- IT 10 - Seismic Design Maps (Project ‘17)
2020 NEHRP Provisions Organization

To introduce new provisions and modifications to improve current requirements in ASCE/SEI 7-16

Part 2: Commentary
A detailed commentary that corresponds to ASCE/SEI 7 and provides useful explanations and guidance on implementation

Part 3: Resource Papers
Introduce new technologies, procedures, and systems for use by design professionals on a provisional basis

Slide adapted from Bonneville and Yuan (2019) SEAOC presentation
Resources to Support the 2020 NEHRP Provisions and ASCE/SEI 7-22

Design Examples

Training Materials

Design Flow Charts

BSSC NEHRP WEBINAR SERIES

www.nibs.org/events/nehrp-webinar-series

Slide adapted from Yuan (2021)
SEAONC presentation
Evolution of Earthquake Engineering
Recent North American Earthquakes and Subsequent Code Changes

1906 San Francisco EQ
- Good steel frame infill performance

1923 Tokyo and 1925 Santa Barbara EQs
- Seismic recording instruments
- Shake tables
- Committees to create seismic code provisions
- 1927 UBC
Recent North American Earthquakes and Subsequent Code Changes

1933 Long Beach EQ
- Ban on URM
- Field Act for schools

1951:
- Proceedings – Separate No. 66 (ASCE)

1959
- First SEAOC Blue Book

URM Bearing Wall Damage in the 1933 Long Beach EQ (from FEMA P-2156 and Los Angeles County Library)
Recent North American Earthquakes and Subsequent Code Changes

1964 Anchorage EQ
- Reinforced concrete detailing

1971 San Fernando EQ
- Reinforced concrete detailing
- Anchorage of concrete and masonry walls to diaphragms
- ATC-3-06

Excessive Drift at the Soft Ground Story of the Olive View Hospital in the 1971 San Fernando EQ
(from FEMA P-2156 and William Godden, NISEE-PEER)
Recent North American Earthquakes and Subsequent Code Changes

Common Location of Fracture Initiation in Pre-Northridge Steel Moment Frame Beam-to-Column Connections
(from FEMA 350)

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>UBC Edition</th>
<th>Enhancement</th>
</tr>
</thead>
<tbody>
<tr>
<td>1971 San Fernando</td>
<td>1973</td>
<td>• Direct positive anchorage of masonry and concrete walls to diaphragms</td>
</tr>
<tr>
<td>1976</td>
<td></td>
<td>• Seismic Zone 4, with increased base shear requirements</td>
</tr>
<tr>
<td>1979 Imperial Valley</td>
<td>1985</td>
<td>• Occupancy Importance Factor, I, for certain buildings</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Interconnection of individual column foundations</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Special inspection requirements</td>
</tr>
<tr>
<td>1985 Mexico City</td>
<td>1988</td>
<td>• Diaphragm continuity ties</td>
</tr>
<tr>
<td>1987 Whittier Narrows</td>
<td>1991</td>
<td>• Requirements for columns supporting discontinuous walls</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Separation of buildings to avoid pounding</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Design of steel columns for maximum axial forces</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Restrictions for irregular structures</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Ductile detailing of perimeter frames</td>
</tr>
<tr>
<td>1989 Loma Prieta</td>
<td>1991</td>
<td>• Revisions to site coefficients</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Revisions to spectral shape</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Increased wall anchorage forces for flexible diaphragm buildings</td>
</tr>
<tr>
<td>1994 Northridge</td>
<td>1997</td>
<td>• Increased restrictions on chevron-braced frames</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Limitations on b/t ratios for braced frames</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Ductile detailing of piles</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Restrictions on use of battered piles</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Requirements to consider liquefaction</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Near-fault zones and corresponding base shear requirements</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Revised base shear equations using 1/T spectral shape</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Redundancy requirements</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Design of collectors for overstrength</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Increase in wall anchorage requirements</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• More realistic evaluation of design drift</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Steel moment connection verification by test</td>
</tr>
</tbody>
</table>
History and Role of the *NEHRP Provisions*
U.S. Seismic Code Development and Role of the NEHRP Provisions

- **1927 UBC (Uniform Building Code)** included first seismic provisions, with non-mandatory appendix.
- **1933: Field Act and Riley Act**: the first mandatory statewide adoption of seismic requirements.
- **1959 Blue Book**: Developed by SEAOE, incorporated by UBC, adopted by the Western US.
- **1977: Passage of National Earthquake Hazards Reduction Act (NEHRP)**.
- **1978 ATC 3-06 Project**: Funded by NSF and NIST, developed advanced seismic analysis and design methods.

**1900-1980**

- **1906 San Francisco Earthquake**: stimulated research and education efforts in the U.S., but seismic building code regulations were not adopted.
- **1933 Long Beach Earthquake**: extensive damage to schools and other buildings was the impetus for the first statewide seismic code regulations.
- **1971 San Fernando Earthquake**: Damage to modern construction conforming to UBC regulations motivated a fresh look at seismic regulations.

**Seismic Regulation Initiation with a California-Centric Effort**

*From FEMA P-2156*
U.S. Seismic Code Development and Role of the NEHRP Provisions

1985 NEHRP Provisions
1st edition, developed based on lessons learned through a FEMA initiative on a national trial design of ATC-3 methods

Written in code language for direct adoption by regional model codes and national standards.

Written in code language, adopted into International Building Code via ASCE 7. Improved maps by USGS

No code language but resource documents providing recommend changes. Adopted by ASCE 7 then by IBC.

1985 Mexico City and 1989 Loma Prieta Earthquakes: illustrated the importance of soil conditions on amplification of earthquake shaking and vulnerability of soft and weak story buildings.

1994 Northridge Earthquake: Generated world-record ground motions. The high repair cost spurred the movement toward Performance-Based Design.

Advancements with NEHRP Provisions and National in scope

From FEMA P-2156
Evolution of the NEHRP Provisions

From FEMA P-2156
Highlights of Major Changes in the 2020 NEHRP Provisions and in ASCE/SEI 7-22
Highlights of Major Changes to 2020 NEHRP Provisions and ASCE/SEI 7-22

- Updated earthquake design ground motions, site classes, and determination of spectral acceleration parameters
- Addition of three new shear wall seismic force-resisting systems
- Addition of provisions and alternative procedures for diaphragm design
- Relaxed modal response spectrum analysis requirements
- Revisions in configuration irregularity requirements
- Revisions in displacement requirements
- Changes in the nonbuilding structures provisions
- Addition of quantitative reliability targets for individual members and essential facilities
- A Part 3 paper on how to apply the NEHRP Provisions for improved seismic resiliency
- A Part 3 paper on a new approach to seismic lateral earth pressures
- Soil-structure interaction provision definitions for different types of shear wave velocities were clarified
- Significant update of the nonstructural components chapter and the forces used for design
Move from Two-Point Spectra (2PRS) to Multi-Point Spectra (MPRS)

Example MPRS from ASCE/SEI 7-22
Table 21.2-1 (from 2020 NEHRP Training Materials by C. Kircher)

San Mateo, CA for default soil class (from FEMA P-2078, Figure 8.2-2)
Three New Shear Wall Seismic Force-Resisting Systems

Figure 6. The three new seismic force-resisting systems that now have detailed requirements in the 2020 NEHRP Recommended Seismic Provisions: (a) reinforced concrete ductile coupled walls, (source: MKA); (b) steel and concrete coupled composite plate shear walls (Source: MKA); and (c) cross-laminated timber shear wall (Source: Lendlease).

From FEMA P-2156
Updates to Diaphragm Design Provisions

- **ASCE/SEI 7-10**
  - Sections 12.10.1 and 12.10.2 - *Traditional Diaphragm Design Method*

- **ASCE/SEI 7-16 (2015 NEHRP Provisions)**
  - Section 12.10.3 - *Alternative Design Provisions* is added
    - Cast-in-place concrete, precast concrete, and wood structural panel diaphragms

- **ASCE/SEI 7-22 (2020 NEHRP Provisions)**
  - Section 12.10.3 – *Alternative Design Provisions* is expanded
    - Bare steel deck, concrete-filled steel deck diaphragms
  - Section 12.10.4 – *Alternative RWFD Provisions* is added
## Relaxation in Requirement for Response Spectrum Analysis

### 2015 NEHRP Provisions and ASCE/SEI 7-16

### 2020 NEHRP Provisions and ASCE/SEI 7-22

### Table 12.6-1 Permitted Analytical Procedures

<table>
<thead>
<tr>
<th>Seismic Design Category</th>
<th>Structural Characteristics</th>
<th>Equivalent Lateral Force Analysis, Section 12.8?</th>
<th>Modal Response Spectrum Analysis, Section 12.9, and Modal Response History Analysis, Section 12.10?</th>
<th>Seismic Response History Procedures, Chapter 16?</th>
</tr>
</thead>
<tbody>
<tr>
<td>B, C</td>
<td>All structures</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>D, E, F</td>
<td>Risk Category I or II buildings not exceeding 2 stories above the base</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>D, E, F</td>
<td>Structures of light frame construction</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>D, E, F</td>
<td>Structures with no structural irregularities and not exceeding 160 ft. in structural height</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>D, E, F</td>
<td>Structures exceeding 160 ft. in structural height with no structural irregularities and with L &lt; 1.5Tc</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>D, E, F</td>
<td>Structures not exceeding 160 ft. in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12.3-1 or vertical irregularities of Type 4, 5a, or 5b in Table 12.3-2</td>
<td>P</td>
<td>P</td>
<td>P</td>
</tr>
<tr>
<td>D, E, F</td>
<td>All other structures</td>
<td>NP</td>
<td>P</td>
<td>P</td>
</tr>
</tbody>
</table>

*P: Permitted; NP: Not Permitted; T_c = S_{a/d}/S_{o/s}*

### Replace Section 12.6 with the following (delete Table 12.6-1 Permitted Analytical Procedures):

The structural analysis required by Chapter 12 shall be completed in accordance with the requirements of (a) Equivalent Lateral Force Procedure of Section 12.8, (b) Modal Response Spectrum Analysis of Section 12.9.1, (c) Linear Response History Analysis of Section 12.9.2, or (d) with an analysis approved by the authority having jurisdiction. Nonlinear Response History Procedure requirements are given in Chapter 16.
Revisions in Displacement Requirements

- Definitions and graphics developed to include diaphragm deformation in displacements related to deformation compatibility, structural separation, and at members spanning between structures.
- Increase in drift used to check deformation compatibility.
- Part 3 resource paper on issues and available research on whether to amplify drifts by $C_d$ or $R$.

Design Earthquake displacement and design story drift (from FEMA P-2082-1, Figure C12-8.1)
Changes in Nonbuilding Structures Requirements

- New Section 15.2: Addresses coupled systems
- Revised Section 15.3.1: For nonbuilding structures supported by other structures, when the ratio of the nonbuilding structure weight to the nonbuilding structure + supporting structure is below a threshold value, use Chapter 13. When above threshold, use Chapter 15.
  - ASCE/SEI 7-16: Threshold is 25%
  - ASCE/SEI 7-22: Revised to 20%, based on review of research
- New Section 15.7.7.4: Provisions for design of corrugated steel tanks added.
Addition of Quantitative Reliability Targets for Individual Members and Essential Facilities

- Section 1.1.1 of 2020 NEHRP Provisions adds individual member/connection reliability targets to previously available building collapse reliability targets.

- Section 2.1.5 of 2020 NEHRP Provisions states: “A desired target reliability for Risk Category IV buildings and nonbuilding structures is for there to be a 10% probability of loss of essential function given the Design Earthquake ground motion.”

<table>
<thead>
<tr>
<th>Risk Category&lt;sup&gt;1&lt;/sup&gt;</th>
<th>Probability of Collapse</th>
<th>Probability of Failure for Member or Connection</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Given MCE&lt;sub&gt;R&lt;/sub&gt; Shaking</td>
<td>In 50 years&lt;sup&gt;*&lt;/sup&gt;</td>
</tr>
<tr>
<td>I</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>II</td>
<td>10%</td>
<td>1%</td>
</tr>
<tr>
<td>III</td>
<td>5%</td>
<td>less than 1%</td>
</tr>
<tr>
<td>IV</td>
<td>2.5%</td>
<td>less than 1%</td>
</tr>
</tbody>
</table>

*From FEMA P-2082-1*
Part 3 Paper on a New Approach to Seismic Lateral Earth Pressures

- Classical methods (Mononobe-Okabe) assume the seismic earth pressures are related to acceleration.
- They are actually related to relative displacement between the soil and the wall.
- Soil-structure interaction theory and research can be used to relate kinematic interaction and inertial interaction to wall demands.
- The Part 3 paper provides a simplified method and example.

Kinematic interaction

Inertial interaction

Images from FEMA P-2082-2
New Seismic Design Force Equation

ASCE 7-16

$$\frac{F_p}{W_p} = (0.4S_{DS}) \times \left[ 1 + 2 \left( \frac{z}{h} \right) \right] \times \left[ \frac{a_p}{R_p} \right] \times I_p$$

2020 NEHRP Provisions and ASCE 7-22

$$\frac{F_p}{W_p} = (0.4S_{DS}) \times \left[ \frac{Hf}{R_\mu} \right] \times \left[ \frac{C_{AR}}{R_{po}} \right] \times I_p$$

- Ground response
- Peak floor acceleration/PGA
- Resonance and component ductility
- Component strength reserve margin
- Building ductility
Building Modal Periods, $T_{n,bldg}$

**Key Takeaway**
- Longer period means less amplification
- Cantilever systems have more “whipping” action

Effect of period of vibration and lateral system stiffness on PFA/PGA

$\alpha_0 = \text{Lateral stiffness ratio, defined as } \alpha_0 = H/(GA/EI)^{0.5}$

$H = \text{height, }$  \hspace{1cm} $\alpha_0 = 0$ represents a pure flexural model

$GA = \text{shear rigidity of a shear beam}$  \hspace{1cm} $\alpha_0$ approaching infinity represents a pure shear beam

$EI = \text{the flexural stiffness}$  \hspace{1cm} (from Miranda and Taghavi, 2009)

Note: Full reference citations are in NIST GCR 18-917-43.
PFA/PGA ($H_f$) Amplification Factor

$$H_f = 1 + a_1 \left( \frac{z}{h} \right) + a_2 \left( \frac{z}{h} \right)^{10}$$

or:

$$H_f = 1 + 2.5 \left( \frac{z}{h} \right)$$

where:

$$a_1 = \frac{1}{T_a} \leq 2.5$$

$$a_2 = [1 - (0.4/T_a)^2] \geq 0$$

$$T_a = C_t h n$$
Seismic Force-Resisting System

Reinforced Concrete SW

Steel SMRF

Effect of building stiffness on PCA/PGA for instrumental recordings (from NIST GCR 18-917-43, 2018 and Lizundia paper in 2019 SEAOC Convention Proceedings)

Key Takeaway
- Same component responds very differently in different seismic force-resisting systems

Figure Assumptions
- Elastic component assumed with $\beta_{comp} = 5\%$
- Dataset includes 19 recordings with PGA $> 0.15g$
Building Ductility, $R_\mu$

$$R_\mu = (1.1 \frac{R}{(I_e \Omega_0)})^{1/2} \geq 1.3$$

$$\frac{F_p}{W_p} = (0.4S_{DS}) \times \left[ \frac{H_f}{R_\mu} \right] \times \left[ \frac{C_{AR}}{R_{po}} \right] \times I_p$$

where:

- $R$ = Response modification factor for the building or nonbuilding structure
- $I_e$ = Importance Factor for the building or nonbuilding structure
- $\Omega_0$ = Overstrength factor for the building or nonbuilding structure

For components at or below grade, $R_\mu$ shall be taken as 1.0.
Chapter 13: Other Significant Changes from ASCE/SEI 7-16 to ASCE/SEI 7-22

- Three different types of supports defined (Section 11.2) and design of supports depends on their system not components they support (Section 13.5.4.6, 7)

Images from FEMA P-2082-1 (2020) and FEMA E-74
Seismic design force provision using nonlinear response history analysis is updated; other dynamic analysis methods are removed (Section 13.3.1.5).

$\Omega_{0p}$ is required to increase the load effects for anchors in concrete or masonry, instead of $\Omega_0$ (Section 13.4.2).

Architectural component list is expanded, and items account for updated coefficient for seismic design: $C_{AR}$, $R_{po}$, and $\Omega_{0p}$ (Table 13.5-1). Example: Partitions split into short light frame, tall light frame, reinforced masonry and other

Mechanical and electrical component list is expanded, and items account for updated coefficient for seismic design: $C_{AR}$, $R_{po}$, and $\Omega_{0p}$ (Table 13.6-1).
Chapter 13: Other Significant Changes from ASCE/SEI 7-16 to ASCE/SEI 7-22

- Detailed scope of design criteria for nonstructural components (Section 13.1)
- Explicit load combinations for nonstructural components now provided (Section 13.2.2)
- Required analysis for condition where the nonstructural component weight is equal to or greater than 20% the combined effective seismic weight, \( W \) (Section 13.2.9)
- Penthouse and rooftop structure requirements are added (Section 13.5.11).
  - Seismic force-resisting system to conform to one in Table 12.2-1, Table 15.4-1, or new coefficients in Table 13.5-1
Questions?
Overview of Design Example Chapters
Chapter 2 (Section 2.1 to 2.6) - Fundamentals
Chapter 2 - Fundamentals (Harris): Topics

- Fundamental Concepts
- Ground Motions and Their Effects
- Structural Dynamics of Linear SDOF Systems
- Response Spectra
- Structural Dynamics of Simple MDOF Systems
- Inelastic Behavior
- Structural Design

Subduction zone tectonic environment

*Image from E.V. Leyendecker, USGS*
Chapter 2 – Fundamentals: Yield, Ductility, Overstrength

Images from Finley Charney
Section 2.7 – Resilience-Based Design
Section 2.7 - Resilience-Based Design (Bonowitz): Topics

- Development of resilience-based earthquake design
  - 2020 NEHRP Provisions, Resource Paper 1
- Functional Recovery (FR)
  - Its relation to resilience
  - Its relation to current building code provisions
- Hypothetical application to the CLT Design Example
  - CLT Shear Wall Design Example is in Chapter 6
  - Discussion in terms of resilience-based design is in Section 2.7
Section 2.7 - The “Resilience Field”

Technical

About the physical building
- Structure
- Nonstructural systems

Facility

About one building. Typical context for:
- Engineering
- Building code implementation

Community

About the group. Typical context for:
- Planning
- Public policy

Holistic

About more than a building
- Contents → Use, Occupancy
- Purpose

From Meister Consultants Group (2017)
Section 2.7 - Functional Recovery vs. Community Resilience

Functional Recovery
- NEHRP Provisions
- ASCE/SEI 7
- IBC

Community Resilience
- NEHRP Reauthorization
- City resilience plans

Diagram:
- Facility vs. Community
- Technical vs. Holistic
Section 2.7 - FEMA-NIST Definitions* for Functional Recovery

- Functional Recovery (FR) is ...
  - A post-earthquake performance state in which a building is maintained, or restored, to support the basic intended functions associated with the pre-earthquake use or occupancy.

- A Functional Recovery objective is ...
  - FR achieved within an acceptable time following a specified earthquake, where the acceptable time might differ for various building uses and occupancies.

* The FEMA-NIST definitions consider infrastructure systems as well as buildings. These versions are edited to address only buildings.
Section 2.7 - Functional Recovery and Performance-Based Engineering

- A structural safety objective may be written as: $P(\text{collapse}) < X\%$, given $2/3 \times \text{MCE}_R$

- Analogously, a functional recovery objective may be written as:
  
  $$P(T_{FR, \text{expected}} > T_{FR, \text{acceptable}}) < Y\%,$$
  
  given $2/3 \times \text{MCE}_R$ (or other specified hazard)

- Open policy questions for developers of FR codes:
  - What is the acceptable or desirable FR time, $T_{FR, \text{acceptable}}$, for a given occupancy?
  - What is the appropriate confidence level, $Y$?
  - What hazard level should be used for FR?
    - For this example, use $2/3 \times \text{MCE}_R$ (See Resource Paper 1 and Design Example 2.7 for discussion.)
Options for Functional Recovery

- Increase Seismic Importance Factor, $I_e$
- Reduce $R$-factor (but already low)
- Set a lower value for panel connector capacity
- Account for partitions
- Study expected damage and recovery time in more detail

CLT shear wall
(Figure C14.5.2.1 in 2020 NEHRP Provisions)
Chapter 3 – Earthquake Ground Motions
Section 3.2 Part 1 – 2018 Update to the USGS National Seismic Hazard Model (Rezaeian): Topics

- Interplay between the USGS hazard models and the BSSC PUC requirements
- The 2018 USGS National Seismic Hazard Model (NSHM) for Conterminous U.S.
  - Ground motion models in CEUS (e.g. NGA-East)
  - Deep basin effects in WUS
- Outside of the Conterminous U.S. (HI, AK, PRVI, GNMI, AMSAM)

“Design” Ground Motions:
USGS: probabilistic
+ risk targeted
+ site amplifications
+ deterministic caps
+ max direction
→ MCE_R

BSSC PUC:
Section 3.2: USGS NSHMs and BSSC PUC Requirements

Hazard Model (PSHA)  Site-Specific Procedures of Chapter 21

<table>
<thead>
<tr>
<th>USGS NSHM</th>
<th>NEHRP Provisions</th>
<th>ASCE 7 Standards</th>
<th>IBC</th>
</tr>
</thead>
<tbody>
<tr>
<td>2014</td>
<td>2015</td>
<td>2016</td>
<td>2018</td>
</tr>
<tr>
<td>2018</td>
<td>2020</td>
<td>2022*</td>
<td>TBD</td>
</tr>
</tbody>
</table>

PGA, 0.2, 1s 760m/s
22 Periods 8 Vs30s

Hazard Curves+ (RiskTarget, MaxDir, SiteAmpl, DetCaps) → “Design” Ground Motions
Section 3.2 - Updates to 2020 NEHRP Design Ground Motions in Conterminous US

2018 USGS NSHM

1. New ground motion models (GMMs), including **NGA-East**, & amplification factors in the Central & Eastern US (CEUS)
2. Deep **basin effects** in Los Angeles, Seattle, San Francisco, and Salt Lake City regions
3. Minor modifications of GMMs (crustal & subduction) in the Western US (WUS)
4. Updating **background seismicity** to include 2013-2017 earthquakes

BSSC Project ‘17

- No change to risk-targeted calcs
- Using **multi-period multi-Vs30 response spectrum** (**MPRS**) (GMMs applicable for all periods and site classes)
- Modifying **deterministic caps** based on deaggregation of probabilistic hazard
- Updating the **max-direction** factors

**MPRS issue directly influenced the 2018 update of USGS NSHM**
Section 3.2 - Hazard Changes (CEUS)

Ratio Maps (2018/2014):
2% in 50yr uniform hazard, BC site class (760 m/s)

Medians: more significant increases for large M at mid-large distances

Epistemic uncertainty: increased significantly for large M, more around 70-100 km

Aleatory uncertainty: minor changes

Site-effect model: only $F_{760}$ in this figure

Seismicity catalog updates: outside CA, mostly affecting intermountain west region

Figure citation: Petersen et al. (2021). The 2018 update of the US National Seismic Hazard Model: Where, why, and how much probabilistic ground motion maps changed. Earthquake Spectra.
Ratio Maps (2018 local basin depth/2018 default basin depth): 2% in 50-yr uniform hazard, 5 sec, Site Class D (260 m/s)

Disclaimer: This information is preliminary and is subject to revision. It is being provided to meet the need for timely best science. The information is provided on the condition that neither the U.S. Geological Survey nor the U.S. Government shall be held liable for any damages resulting from the authorized or unauthorized use of the information.
Section 3.2 Part 2 – Dissection of Example Changes to the MCE_R Ground Motion Values (Luco): Topics

- Revisions to deterministic caps
- Examples of changes in MCE_R and MCE_G values
- Risk-targeted maximum considered earthquake (MCE_R) spectral response accelerations
- Maximum considered earthquake geometric mean (MCE_G) peak ground accelerations
- Long-period transition maps (T_L)
- USGS seismic design geodatabase and web service
21.2.2 Deterministic (MCE_R) Ground Motions

The deterministic spectral response acceleration at each period shall be calculated as an 84th-percentile 5% damped spectral response acceleration in the direction of maximum horizontal response computed at that period. The largest such acceleration calculated for scenario earthquakes on all known faults within the region shall be used. The scenario earthquakes shall be determined from deaggregation for the probabilistic spectral response acceleration at each period. Scenario earthquakes contributing less than 10% of the largest contributor at each period shall be ignored.
Table C22-3 Comparison of short-period MCE$_R$ spectral response acceleration values from these Provisions, ASCE/SEI 7-16, and ASCE/SEI 7-10. The $S_{MS}$ values are for the default site class.

<table>
<thead>
<tr>
<th>Location Name</th>
<th>ASCE/SEI 7-10</th>
<th>ASCE/SEI 7-16</th>
<th>2020 Provisions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_S$ (g)</td>
<td>$S_{MS}$ (g)</td>
<td>$S_S$ (g)</td>
</tr>
<tr>
<td>Los Angeles, CA</td>
<td>2.40</td>
<td>2.40</td>
<td>1.97</td>
</tr>
<tr>
<td>Century City, CA</td>
<td>2.17</td>
<td>2.17</td>
<td>2.11</td>
</tr>
<tr>
<td>Northridge, CA</td>
<td>1.69</td>
<td>1.69</td>
<td>1.74</td>
</tr>
<tr>
<td>Long Beach, CA</td>
<td>1.64</td>
<td>1.64</td>
<td>1.68</td>
</tr>
<tr>
<td>Irvine, CA</td>
<td>1.55</td>
<td>1.55</td>
<td>1.25</td>
</tr>
<tr>
<td>Riverside, CA</td>
<td>1.50</td>
<td>1.50</td>
<td>1.50</td>
</tr>
<tr>
<td>San Bernardino, CA</td>
<td>2.37</td>
<td>2.37</td>
<td>2.33</td>
</tr>
<tr>
<td>San Luis Obispo, CA</td>
<td>1.12</td>
<td>1.18</td>
<td>1.09</td>
</tr>
<tr>
<td>San Diego, CA</td>
<td>1.25</td>
<td>1.25</td>
<td>1.58</td>
</tr>
<tr>
<td>Santa Barbara, CA</td>
<td>2.83</td>
<td>2.83</td>
<td>2.12</td>
</tr>
<tr>
<td>Ventura, CA</td>
<td>2.38</td>
<td>2.38</td>
<td>2.02</td>
</tr>
</tbody>
</table>
Section 3.2 - Examples of Changes in SDC

From ASCE 7-10 to ASCE 7-16, SDC decreases at 2 of 34 locations, from E to D.

From ASCE 7-16 to 2020 Provisions, SDC increases at 4 of 34 locations, from D to E, mostly due to deterministic capping and basin effects.

Table C22-6 Comparison of seismic design categories from these Provisions, ASCE/SEI 7-16, and ASCE/SEI 7-10, for the default site class and risk categories I, II, or III. The “SDC<sub>S</sub>” categories are determined from Table 11.6-1 (“Seismic Design Category Based on Short-Period Response Acceleration Parameter”) alone, but only where S<sub>r</sub> < 0.75g.

<table>
<thead>
<tr>
<th>Location Name</th>
<th>&quot;SDC&lt;sub&gt;S&lt;/sub&gt;&quot;</th>
<th>SDC</th>
<th>&quot;SDC&lt;sub&gt;S&lt;/sub&gt;&quot;</th>
<th>SDC</th>
<th>&quot;SDC&lt;sub&gt;S&lt;/sub&gt;&quot;</th>
<th>SDC</th>
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<tbody>
<tr>
<td>Los Angeles, CA</td>
<td>N/A</td>
<td>E</td>
<td>N/A</td>
<td>D</td>
<td>N/A</td>
<td>D</td>
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<tr>
<td>Century City, CA</td>
<td>N/A</td>
<td>E</td>
<td>D</td>
<td>D</td>
<td>D</td>
<td>D</td>
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<tr>
<td>San Diego, CA</td>
<td>D</td>
<td>D</td>
<td>D</td>
<td>D</td>
<td>D</td>
<td>D</td>
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<tr>
<td>Santa Barbara, CA</td>
<td>N/A</td>
<td>E</td>
<td>N/A</td>
<td>E</td>
<td>N/A</td>
<td>E</td>
</tr>
<tr>
<td>Ventura, CA</td>
<td>N/A</td>
<td>E</td>
<td>N/A</td>
<td>E</td>
<td>N/A</td>
<td>E</td>
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</tbody>
</table>
Section 3.2 - BSSC Tool for Seismic Design Map Values
https://doi.org/10.5066/F7NK3C76

BSSC Tool For 2020 NEHRP Provisions Seismic Design Map Values

Version: Beta
Section 3.3 – Multi-Period Response Spectra (Kircher): Topics

- Design parameters and response spectra of ASCE/SEI 7-16
- Site-specific requirements of ASCE/SEI 7-16
- New ground motion parameters of ASCE/SEI 7-22 Chapter 11
- New site classes of ASCE/SEI 7-22 Chapter 20
- New site-specific analysis requirements of ASCE/SEI 7-22 Chapter 21
- Example comparisons of design response spectra
Section 3.3 - The “Problem” with ASCE 7-10

- For softer sites, in particular those where seismic hazard is governed by large magnitude earthquakes:
  - Frequency content of ground motions (spectrum shape) is not accurately characterized by of the two-period design response spectrum and site coefficients
  - Design ground motions are significantly underestimated (e.g., by as much as a factor of 2 at longer response periods)
Section 3.3 - Comparison of ASCE/SEI 7-16 Two-Period (ELF) Design Spectrum w/o Spectrum Shape Adjustment with MPRS Design Spectrum

MPRS based on M7.0 earthquake ground motions at 6.8 km – Site Class C

MPRS based on M8.0 earthquake ground motions at 9.9 km – Site Class E
Section 3.3 - Interim Solution of ASCE/SEI 7-16 (2015 NEHRP Provisions)

- Require site-specific analysis to determine design ground motions for softer sites, but provide exceptions to permit design using “conservative” values seismic design parameters
  - Site Class D - Site-specific ground motion procedures are required for structures on Site Class D sites where values of $S_1$ are greater than or equal to 0.2.
    - An exception permits ELF (and MRSA) design using a “conservative” value of the seismic design coefficient based on a 50 percent increase in the value of the seismic parameter $S_{M1}$ ($S_{D1}$), effectively extending the acceleration domain to $1.5T_s$
  - Site Class E - Site-specific ground motion procedures required for structures on Site Class E sites where values of $S_S$ are greater than or equal to 1.0 (or $S_1$ greater than 0.2)
    - An exception permits ELF design using a “conservative” value of the seismic design coefficient based on the seismic parameter $S_{MS}$ ($S_{DS}$) for Site Class C, regardless of the design period, $T$, effectively eliminating the velocity domain.
Define MCE\(_R\) and design ground motions in terms of MPRS (e.g., for MRSA design or as the basis for selecting records for NRHA)

Derive values of seismic design parameters (e.g., \(S_{DS}\) and \(S_{D1}\)) from the MPRS of interest (e.g., for ELF design)

Provide MPRS and associated values of seismic design parameters for User-specified values of:

- Site Location (latitude, longitude)
- Site Class
- From USGS web service at [http://doi.org/10.5066/F7NK3C76](http://doi.org/10.5066/F7NK3C76) (aka USGS Seismic Design Geodatabase for ASCE 7-22) and
- Other user-friendly providers (e.g., WBDG, ASCE 7 Hazard Tool, etc.)
### Section 3.3 - New Site Classes and Associated Values of Shear Wave Velocities

(Table 2.2-1, FEMA P-2078, June 2020)

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Description</th>
<th>Shear Wave Velocity, $V_{s30}$ (fps)</th>
<th>USGS(^2) $V_{s30}$ (mps)</th>
</tr>
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<tr>
<td></td>
<td></td>
<td>Lower Bound(^1)</td>
<td>Upper Bound(^1)</td>
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<tr>
<td>A</td>
<td>Hard rock</td>
<td>5,000</td>
<td>1,500</td>
</tr>
<tr>
<td>B</td>
<td>Medium hard rock</td>
<td>3,000</td>
<td>5,000</td>
</tr>
<tr>
<td>BC</td>
<td>Soft rock</td>
<td>2,100</td>
<td>3,000</td>
</tr>
<tr>
<td>C</td>
<td>Very dense soil or hard clay</td>
<td>1,450</td>
<td>2,100</td>
</tr>
<tr>
<td>CD</td>
<td>Dense sand or very stiff clay</td>
<td>1,000</td>
<td>1,450</td>
</tr>
<tr>
<td>D</td>
<td>Medium dense sand or stiff clay</td>
<td>700</td>
<td>1,000</td>
</tr>
<tr>
<td>DE</td>
<td>Loose sand or medium stiff clay</td>
<td>500</td>
<td>700</td>
</tr>
<tr>
<td>E</td>
<td>Very loose sand or soft clay</td>
<td>500</td>
<td></td>
</tr>
</tbody>
</table>

1. Upper and lower bounds, Table 20.3-1, ASCE/SEI 7-22.
2. Center of range (rounded) values used by USGS to develop MPRS.
### Section 3.3 - MPRS Format

- Values available for conterminous US regions with ground motion models for all combinations of 22 periods and 8 site classes

<table>
<thead>
<tr>
<th>Period T (s)</th>
<th>5%-Damped Response Spectral Acceleration or PGA by Site Class (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
</tr>
<tr>
<td>0.00</td>
<td>0.501</td>
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<tr>
<td>0.010</td>
<td>0.503</td>
</tr>
<tr>
<td>0.020</td>
<td>0.519</td>
</tr>
<tr>
<td>0.030</td>
<td>0.596</td>
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<tr>
<td>0.050</td>
<td>0.811</td>
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<tr>
<td>0.075</td>
<td>1.040</td>
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<tr>
<td>0.10</td>
<td>1.119</td>
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<td>0.15</td>
<td>1.117</td>
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<tr>
<td>0.20</td>
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<tr>
<td>0.25</td>
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<td>5.0</td>
<td>0.100</td>
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<td>7.5</td>
<td>0.063</td>
</tr>
<tr>
<td>10</td>
<td>0.042</td>
</tr>
<tr>
<td><strong>PGA</strong></td>
<td>0.373</td>
</tr>
</tbody>
</table>
Move from Two-Point Spectra (2PRS) to Multi-Point Spectra (MPRS)

Example MPRS from ASCE/SEI 7-22
Table 21.2-1 (from 2020 NEHRP Training Materials by C. Kircher)

San Mateo, CA for default soil class (from FEMA P-2078, Figure 8.2-2)

Velocity domain of the ASCE 7-16 (2PRS) design spectrum includes the 1.5 multiplier of the applicable Section 11.4.8 exception.
Section 3.3 - Design (As Usual) Using New MPRS

- **Design Ground Motions**
  - Ground motion parameters (and MPRS) are available online from a USGS web service [https://doi.org/10.5066/F7NK3C76] for user specified site location (i.e., latitude and longitude) and site conditions (i.e., site class)
  - Site-specific ground motion procedures (Chapter 21) now permit use of MPRS obtained online from the USGS web service (in lieu of a hazard analysis)

- **Design Procedures**
  - ELF procedures (Chapter 12) are not affected by proposed changes (although values of design parameters, \(S_{DS}\) and \(S_{D1}\), would better match the underlying response spectrum of the site of interest)
  - MRSA procedures (Chapter 12) are not affected by proposed changes (although multi-period design spectra would provide a more reliable calculation of dynamic response)
Section 3.4 – Other Changes to Ground Motion Provisions in ASCE/SEI 7-22 (Crouse): Topics

- Maximum Considered Earthquake Geometric Mean (MCE\(_G\)) Peak Ground Acceleration (ASCE/SEI 7-22, Section 21.5)

- Vertical Ground Motion for Seismic Design (ASCE/SEI 7-22, Section 11.9)

- Site Class when Shear Wave Velocity Data Unavailable (ASCE/SEI 7-22, Section 20.3)

Comparison of \(S_{aMv}\) and \(S_{aM}\) for Irvine, CA site and Site Class D
Chapter 4 – Ductile Reinforced Concrete Shear Walls
Chapter 4 – Ductile Coupled RC Shear Walls (Ghosh and Dasgupta): Topics

- Ductile coupled shear wall system
- Research justification
- Design example
  - Overall demands
  - Design of shear wall
  - Design of coupling beams
Chapter 2 – Ductile Coupled RC Shear Wall: Details

Source: http://nees.seas.ucla.edu/pankow

Shear wall plan section
Chapter 5 – Coupled Composite Plate Shear Walls/Concrete Filled (C-PSW/CF)
Chapter 5 – Coupled Composite Plate Shear Walls / Concrete Filled (Shafaei and Varma): Topics

- Introduction to Coupled C-PSW/CFs
- Section detailing, limits, and requirements
- Seismic behavior and capacity design
- Design example
  - Overall demands
  - Coupling beams
  - C-PSW/CF
  - Connection of beams to C-PSW/CF
Chapter 5 – C-PSW/CF: Seismic Design Philosophy

From AISC Design Guide 37 (AISC, 2021)
Chapter 5 – C-PSW/CF: Coupling Beam-to-Wall Connection
Chapter 6 – Cross-Laminated Timber Shear Walls
Chapter 6 - Cross-Laminated Timber (CLT) Shear Wall (Line and Amini): Topics

- This example features the seismic design of cross-laminated timber shear walls used in a three-story, six-unit townhouse cross-laminated timber building of platform construction.

- The CLT shear wall design in this example includes:
  - Check of CLT shear wall shear strength
  - Check of CLT shear wall hold-down size and compression zone length for overturning
  - Check of CLT shear wall deflection for conformance to seismic drift
Chapter 6 – CLT Shear Wall: Construction

Photo credits: Will Pryce
Chapter 6 – CLT: Shear Wall Details

Wall-to-floor intersections

SECTION A-A

SECTION B-B
Chapter 7 – Horizontal Diaphragm Design
Chapter 7 – Horizontal Diaphragm Design (Cobeen): Topics

- All diaphragm seismic design methods
  - Sections 12.10.1 and 12.10.2 - Traditional Diaphragm Design Method (in ASCE/SEI 7-10)
  - Section 12.10.3 - Alternative Design Provisions is added (added in ASCE/SEI 7-16)
    - Cast-in-place concrete, precast concrete, and wood structural panel diaphragms
  - Section 12.10.3 – Alternative Design Provisions is expanded (in ASCE/SEI 7-22)
    - Bare steel deck, concrete-filled steel deck diaphragms
  - Section 12.10.4 – Alternative RWFD Provisions is added (in ASCE/SEI 7-22)

- Design examples
  - Determination of diaphragm design forces
  - One-story wood assembly hall
  - One-story bare steel deck diaphragm building
  - Multi-story steel building with steel deck diaphragms
Chapter 7: Diaphragm Seismic Design Method Comparison

- Advantages of using Section 12.10.3 Alternative Design Provisions:
  - Better reflects vertical distribution of diaphragm forces
  - Better reflects effect of diaphragm ductility and displacement capacity
  - May result in lower seismic demands

- Advantages of using Section 12.10.4 Alternative RWFD Method:
  - Better reflects seismic response of RWFD buildings
  - May result in lower seismic demands
  - Is anticipated to result in better performance

- When will the Section 12.10.1 and 12.10.2 Traditional Method result in lower design forces?
  - Bare steel deck diaphragms not meeting the AISI S400 special seismic detailing provisions
  - Other
Chapter 7: Section 12.10.3 Alternative Design Provisions

Part 1: Vertical distribution of seismic forces for near-elastic diaphragm behavior

Part 2: Parameter $R_s$ modifies near-elastic forces based on diaphragm ductility and deformation capacity

\[ F_{px} = \frac{C_{px}}{R_s} w_{px} \]
Chapter 7: Section 12.10.4 Alternative RWFD Design Method

Design to encourage distributed inelastic behavior for improved seismic performance

Amplified Shear Boundary Zone
Chapter 7: Section 12.10.4 Alternative RWFD Design Method

*Optional* incorporation of actual seismic response of RWFD buildings for vertical elements – 2 stage analysis

Seismic design forces using $M_{\text{diaph}}$ & $T_{\text{diaph}}$

Seismic design forces using $M_{\text{wall}}$ & $T_a$

Seismic Design Forces to Vertical Element

Graph showing Spectral Response Acceleration vs. Period (sec) with lines for Rigid Wall Response and Flexible Diaphragm Response.
Chapter 8 - Nonstructural Components
Chapter 8 - Design Examples for Nonstructural Components (Lizundia): Topics

- Architectural precast concrete
- Egress stairs
- Pressure vessel
- HVAC fan unit
- Plan of piping system
### Example Summary

<table>
<thead>
<tr>
<th>Topic</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Nonstructural component:</strong></td>
<td>Architectural – exterior nonstructural wall elements and connections</td>
</tr>
<tr>
<td><strong>Building seismic force-resisting system:</strong></td>
<td>Steel special moment frames</td>
</tr>
<tr>
<td><strong>Equipment support:</strong></td>
<td>Not applicable</td>
</tr>
<tr>
<td><strong>Occupancy:</strong></td>
<td>Office</td>
</tr>
<tr>
<td><strong>Risk Category:</strong></td>
<td>II</td>
</tr>
<tr>
<td><strong>Component Importance Factor:</strong></td>
<td>$I_p = 1.0$</td>
</tr>
<tr>
<td><strong>Number of stories:</strong></td>
<td>5</td>
</tr>
<tr>
<td>$S_{DS}$</td>
<td>1.487</td>
</tr>
</tbody>
</table>

### Topics Covered

- Providing gravity support and accommodating story drift in cladding
- Spandrel panel
- Column cover
- Prescribed seismic displacements
Chapter 8 - Nonstructural Components Example: Rocking Cladding Mechanism
Chapter 8 - Nonstructural Components Example: Piping System Seismic Design

Topics Covered

- Piping system design
- Pipe supports and bracing
- Prescribed seismic displacements
Chapter 8 - Nonstructural Components Example: Egress Stairs

Topics Covered

- Prescribed seismic forces
  - Egress stairways not part of the building seismic force-resisting system
  - Egress stairs and ramp fasteners and attachments
- Prescribed seismic displacements

Isometric view of egress stairs
Chapter 8 - Nonstructural Components Example: Elevated Vessel

Example Summary

- **Nonstructural components:**
  Mechanical and electrical – pressure vessel not supported on skirts
- **Building seismic force-resisting system:** Special reinforced concrete shear walls
- **Equipment support:** Equipment support structures and platforms – Seismic Force-Resisting Systems with $R > 3$
- **Occupancy:** Storage
- **Risk Category:** II
- **Component Importance Factor:** $I_p = 1.0$
- **Number of stories:** 3
  - $S_{DS} = 1.20$
  - $S_1 = 0.65$
ASCE/SEI 7-16 required the nonstructural components and supporting structure to be designed with the same seismic design forces, $F_p$, regardless of their interaction, and the force was based on the component properties. A platform supporting a pressure vessel would be designed for pressure vessel forces regardless of whether the platform structure was made of concrete, steel braced frames, or steel moment frames.

In ASCE/SEI 7-22, the concept of an equipment support structure or platform has been introduced and defined. Definitions are given in Section 11.2 and properties have been added to Table 13.6-1. Section 13.6.4.6 has been added to ASCE/SEI 7-22 to require that the support structures and platforms be designed in accordance with those properties. This permits a more accurate determination of forces that more realistically reflect the differences in dynamic properties and ductilities between the component and the support structure or platform.
Chapter 8 - Prescribed Seismic Forces: Vessel Support and Attachments

- Vessel and legs weight, \( W_{p,ves} = D_{ves} = 5,000 \text{ lb} \)

- Seismic design force, \( F_p \)

\[
F_p = 0.4S_{DS}l_p W_p \left[ \frac{H_f}{R_\mu} \right] \left[ \frac{C_{AR}}{R_{po}} \right] = 0.4(1.2)(1.0)(W_p) \left[ 2.52 \right] \left[ \frac{1.4}{1.5} \right] = 0.762W_p \quad \text{(controlling equation)}
\]

\[
F_{p,max} = 1.6S_{DS}l_p W_p = 1.6(1.2)(1.0)(W_p) = 1.92W_p
\]

\[
F_{p,min} = 0.3S_{DS}l_p W_p = 0.3(1.2)(1.0)(W_p) = 0.360W_p
\]

\[
F_{p,ves} = 0.762W_p = 0.762(5,000 \text{ lb}) = 3,808 \text{ lb} \quad \text{(controlling seismic design force)}
\]
Chapter 8 - Nonstructural Component Example: HVAC Fan Unit Support

**Topics Covered**

- **Case 1:** Direct attachment to the structure using cast-in-place anchors
- **Case 2:** Support on vibration isolation springs that are attached to the slab post-installed expansion anchors.

HVAC Fan Unit
Organization and Presentation of the Design Example Chapters
Outline of the 2020 Design Examples Chapters

- Chapter 1: Introduction
- Chapter 2: Fundamentals
- Chapter 3: Earthquake Ground Motions
- Chapter 4: Ductile Coupled Reinforced Concrete Shear Walls
- Chapter 5: Coupled Composite Plate Shear Walls/Concrete Filled
- Chapter 6: Three-Story Cross-Laminated Timber (CLT) Shear Wall
- Chapter 7: Horizontal Diaphragm Design
- Chapter 8: Nonstructural Components
How to Use the 2015 and 2020 *Design Examples* Together

- Both the 2015 and 2020 *Design Examples* are intended to be used together.
- The 2020 *Design Examples* cover major changes and new seismic force-resisting systems, but the 2015 *Design Examples* still apply in many situations.
### How to Use the 2015 and 2020 Design Examples Together

<table>
<thead>
<tr>
<th>Topic</th>
<th>2015 Design Examples and ASCE/SEI 7-16</th>
<th>2020 Design Examples and ASCE/SEI 7-22</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic Resilience</td>
<td>Not covered in 2015 Design Examples. Use 2020 Design Examples.</td>
<td>Section 2.7 – Summarizes application of resilience design to the NEHRP Provisions and includes a CLT case study.</td>
</tr>
<tr>
<td>Earthquake Ground Motion</td>
<td>Chapter 3 – Provides basis for Risk Targeted design maps, discusses hazard assessment, site specific spectra, and ground motion selection and scaling. Selection and scaling discussion are still generally applicable with ASCE/SEI 7-22. Use 2020 Design Examples otherwise.</td>
<td>Chapter 3 – Summarizes basis for new design maps, addition of more site classes, major update from two-period spectra to multi-period spectra, and update on vertical ground motion.</td>
</tr>
<tr>
<td>Linear Analysis</td>
<td>Chapter 4 – Design examples with equivalent lateral force procedure, modal response spectrum analysis, and new linear response history analysis. Applicable with ASCE/SEI 7-22.</td>
<td>Not covered in 2020 Design Examples. See Section 1.4 of this Chapter on relaxation of modal response spectrum analysis requirements.</td>
</tr>
<tr>
<td>Diaphragm Analysis</td>
<td>Chapter 6 – Design examples comparing traditional and new alternate methods. Use the 2020 Design Examples.</td>
<td>Chapter 7 – Design examples showing all diaphragm analysis methods including new methods introduced with the 2020 NEHRP Provisions. Diaphragm design for precast diaphragms has been moved out of ASCE/SEI 7-22 to ACI publications, and this is discussed.</td>
</tr>
</tbody>
</table>
How to Use the 2015 and 2020 *Design Examples* Together

<table>
<thead>
<tr>
<th>Topic</th>
<th>2015 <em>Design Examples</em> and ASCE/SEI 7-16</th>
<th>2020 <em>Design Examples</em> and ASCE/SEI 7-22</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil-Structure Interaction (SSI)</td>
<td>Chapter 8 – Design example of a four-story reinforced concrete shear wall building with and without SSI. Applicable with ASCE/SEI 7-22.</td>
<td>No examples in 2020 <em>Design Examples</em>. See Section 1.4 of this Chapter for discussion on changes to SSI provisions in ASCE/SEI 7-22.</td>
</tr>
<tr>
<td>Structural Steel</td>
<td>Chapter 9 – Design examples for a high-bay warehouse with an ordinary concentric braced frame and an intermediate moment frame and for an office building with a special steel moment frame and a special concentric braced frame. Applicable with ASCE/SEI 7-22.</td>
<td>Not covered in 2020 <em>Design Examples</em>.</td>
</tr>
<tr>
<td>Reinforced Concrete</td>
<td>Chapter 10 – Design examples for an intermediate moment frame, a special moment frame, and special concrete shear walls. Applicable with ASCE/SEI 7-22.</td>
<td>Chapter 4 – Design example for a new reinforced concrete ductile coupled wall.</td>
</tr>
<tr>
<td>Composite Steel and Concrete</td>
<td>Chapter 12 – Design example of composite partially restrained moment frame. Applicable with ASCE/SEI 7-22.</td>
<td>Chapter 5 – Design example for a new steel and concrete coupled composite plate shear walls.</td>
</tr>
</tbody>
</table>
# How to Use the 2015 and 2020 Design Examples Together

<table>
<thead>
<tr>
<th>Topic</th>
<th>2015 Design Examples and ASCE/SEI 7-16</th>
<th>2020 Design Examples and ASCE/SEI 7-22</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood</td>
<td>Chapter 14 – Design examples for an apartment, wood roof diaphragm and roof-to-wall anchorage in a masonry building. Use the 2020 Design Examples for wood diaphragms.</td>
<td>Chapter 6 – Design example for new cross-laminated timber shear wall system.</td>
</tr>
<tr>
<td>Nonbuilding Structures</td>
<td>Chapter 17 – Design examples for pipe racks, industrial storage rack, power generating plant, pier, storage tanks, and tall vertical storage vessel. Applicable with ASCE/SEI 7-22.</td>
<td>No examples in 2020 Design Examples. See Section 1.4 for discussion on changes to nonbuilding structures in ASCE/SEI 7-22.</td>
</tr>
<tr>
<td>Nonstructural Components</td>
<td>Chapter 18 – Design examples for precast cladding, egress stair, roof fan anchorage, piping system, and elevated vessel. Use 2020 Design Examples.</td>
<td>Chapter 8 – Background on development of new design equations and other changes, plus design examples for precast cladding, egress stair, roof fan anchorage, piping system, and elevated vessel.</td>
</tr>
</tbody>
</table>
For ease of reader use, the 2020 *Design Examples* typically reference ASCE/SEI 7-22 sections and equations rather than the 2020 *NEHRP Provisions*. However, at the time of completion of writing the 2020 *Design Examples* in the summer of 2021, ASCE/SEI 7-22 had not been finalized or published. Publication was expected in December 2022. The June 17, 2021, draft of ASCE/SEI 7-22 issued for public comment was used as the reference document for ASCE/SEI 7-22. At that time, all major proposals from the ASCE committee responsible for the standard had been incorporated, but public review remained. This may lead to changes in the final published version of ASCE/SEI 7-22. As such, when that published version is available, the reader of this 2020 *Design Examples* should look at the sections in the published version where revisions from ASCE/SEI 7-16 are indicated to determine whether there are meaningful differences.
Presentation Techniques in the 2020 Design Examples

- Free-body diagrams are used.
- A worked-out example of the calculations is typically shown in detail only once. Summary tables then show the results for the other similar components.
- The focus is on key selected items in each example to keep the document size manageable. Not all necessary items that would need to be checked or designed are shown. In many cases, a list of these additional items is provided.
- Changes between the NEHRP provisions and ASCE/SEI 7-22 are flagged:

**Changes Between the NEHRP Provisions and ASCE/SEI 7-22**

Equation 13.3-6 in the 2020 NEHRP Provisions was modified for ASCE/SEI 7-22, by adding $I_v$ into the denominator to better estimate the structure ductility.
Introduction to the 2020 NEHRP Provisions Design Examples: Bret Lizundia and Mai Tong

Fundamentals of Earthquake Engineering: James Harris

Diaphragm Seismic Design Part 1 and Part 2: Kelly Cobeen

Ductile Coupled Reinforced Concrete Shear Walls: S.K. Ghosh

Nonstructural Components Part 1 and Part 2: Bret Lizundia

Fundamentals and Evolution of U.S. Seismic Design Values and the 2018 Update of the USGS National Seismic Hazard Model: Sanaz Rezaeian and Ronald Hamburger

Multi-Period Response Spectra Provisions, Other Changes to Ground Motion Provisions, and Dissection of Example Changes to the Ground Motion Values: Charles Kircher, C.B. Crouse, and Nicolas Luco

Cross-Laminated Timber Shear Wall Design and Resilience-Based Design: M. Omar Amini, David Bonowitz, and Philip Line

Coupled Composite Plate Shear Walls / Concrete Filled: Soheil Shafaei and Amit Varma
Questions?
DISCLAIMER

 NOTICE: Any opinions, findings, conclusions, or recommendations expressed in this publication do not necessarily reflect the views of the Federal Emergency Management Agency. Additionally, neither FEMA nor any of its employees make any warranty, expressed or implied, nor assume any legal liability or responsibility for the accuracy, completeness, or usefulness of any information, product or process included in this publication.

 The opinions expressed herein regarding the requirements of the NEHRP Recommended Seismic Provisions, the referenced standards, and the building codes are not to be used for design purposes. Rather the user should consult the jurisdiction’s building official who has the authority to render interpretation of the code.

 This training material presentation is intended to remain complete in its entirety even if used by other presenters. While the training material could be tailored for use in other presentations, we caution users to account for issues of completeness and interpretation if only part of the material is used. We also strongly suggest users give proper credit/citation to this presentation and its author.
Chapter 2 (Sections 2.1 to 2.6) Fundamentals

2020 NEHRP Provisions Training Materials
James Harris, J. R. Harris & Company
Overview

- **Fundamental Concepts**
- Ground Motions and Their Effects
- Structural Dynamics of Linear SDOF Systems
- Response Spectra
- Structural Dynamics of Simple MDOF Systems
- Inelastic Behavior
- Structural Design

The consists of text outlining the key topics in this presentation.

This slide provides an overview of the topics presented in this slide set.
**Fundamental Concepts (1)**

- Ordinarily, a large earthquake produces the most severe loading that a building is expected to survive. The probability that failure will occur is very real and is greater than for other loading phenomena. Also, in the case of earthquakes, the definition of failure is altered to permit certain types of behavior and damage that are considered unacceptable in relation to the effects of other phenomena.

- The levels of uncertainty are much greater than those encountered in the design of structures to resist other phenomena. The high uncertainty applies both to knowledge of the loading function and to the resistance properties of the materials, members, and systems.

- The details of construction are very important because flaws of no apparent consequence often will cause systematic and unacceptable damage simply because the earthquake loading is so severe and an extended range of behavior is permitted.

Slide contains text taken from Chapter 2 of FEMA P-1051 discussing probabilities and uncertainties of failure due to earthquakes.

These bullet items are taken directly from the text of Chapter 2 of FEMA P-1051.
Fundamental Concepts (2)

- During an earthquake the ground shakes violently in all directions. Buildings respond to the shaking by vibration, and the movements caused by the vibration and the ground motion induce inertial forces throughout the structure.

- In most parts of the country the inertial forces are so large that it is not economical to design a building to resist the forces elastically. Thus, inelastic behavior is necessary, and structures must be detailed to survive several cycles of inelastic behavior during an earthquake.

- The structural analysis that is required to exactly account for the dynamic loading and the inelastic response is quite complex and is too cumbersome for most projects. The NEHRP Provisions and ASCE 7 provide simplified approximate analysis approaches that overcome these difficulties.

- Rules for detailing structures for seismic resistance are provided by standards such as ACI 318 and the AISC Specification and the AISC Seismic Provisions

This slide contains text with additional text discussing fundamental concepts of seismic design.

Fundamental concepts, continued.
Overview

- Fundamental Concepts
- Ground Motions and Their Effects
- Structural Dynamics of Linear SDOF Systems
- Response Spectra
- Structural Dynamics of Simple MDOF Systems
- Inelastic Behavior
- Structural Design

Slide contains text of an overview of the topics presented in this slide set.

This slide provides an overview of the topics presented in this slide set.
This slide contains an image where earthquakes have occurred historically in the world.

This slide shows a series of dots where earthquakes have occurred historically in the world. As may be seen, the dots are not randomly oriented, but instead for distinct patterns. It has been determined that the dot patterns outline the boundaries of tectonic “plates” that form the crust of the earth. The Sand Andreas fault in western California lies along one of the plate boundaries. Note also the heavy concentration of dots along the west coast of South America, and along the north-western Pacific coast, and the Aleutian Island Chain. Most of the historic “great earthquakes” have occurred in these locations.

Note the scarcity of dots in the central and eastern U.S. Earthquakes are rare here, but large earthquakes have occurred in central Missouri and in South Carolina.
This figure shows an image of the tectonic plates and the plate names on earth.

This slide shows the tectonic plates and the plate names. Heavy black lines mark the main plate boundaries.
An image of a section of the earth’s crust and the driving mechanism for plate movement in ocean rift valleys.

This slide, from Bolt, shows a section of the Earth’s crust, and the driving mechanism for plate movement in ocean rift valleys.
This slide contains an image showing the subduction type mechanism.

If the plates continue to diverge at the ridges, the surface of the earth would have to grow (or buckle) unless there were some mechanism to return some cool rock into the asthenosphere. This slide, also from Bolt, shows how the plate submerges (or subducts) under the continental shelf at the plate boundary. The sudden release of frictional forces that develop at this interface are a major source of earthquakes. Volcanic activity is also a source of earthquakes, but the resulting ground motions are usually not as severe. A note about the buckling: this does occur where two continental plates converge, such as the boundary between India and Asia; the result is the highest mountain chain on earth.
This slide contains an image highlighting fault and their features.

Zones of relative weakness in the Earth’s crust are called faults. After the stresses build up in the rock, one particular area will rupture with relative movement. Define a few key terms for description of faults.
This slide contains an image highlighting fault plane, hypocenter, rupture surface, and epicenter of an earthquake.

When an earthquake occurs the rupture spreads over a portion of the fault. The point where the rupture begins is called the focus or hypocenter. The rupture will then propagate at a very high speed, forming a fault plane. The vertical projection of the focus to the surface is the epicenter. For shallow earthquakes, the fault plate may intersect the surface, causing a visible fault rupture and possible escarpment. For some deeper earthquakes, the fault may not be seen at the surface. These are called blind faults.
Types of Faults

Contains images of different fault types.

These are the various types of faults representing either lateral or vertical movement. The fault plane may be vertical or skewed as shown. For strike slip faults, the type designation comes from the movement of one block relative to an observer. If the observer is standing on one of the blocks looking across the fault and the far block moves to the observer’s right, it is a right lateral fault. For a normal fault, the two blocks move away from each other (extensional). For a thrust fault, the blocks are moving towards each other (compressional). The visible wall formed from the movement is called an escarpment. Of course, the fault may be a combined strike-slip or normal/reverse fault.
Seismic Wave Forms (Body Waves)

This slide contains two images, one illustrating a compression wave (p wave), and the other illustrating a shear wave (s wave).

The energy released during an earthquake propagates in waves. The two types of waves are body waves and surface waves. The principal body waves are the Compression (P) wave and the Shear (S) wave. Compression and shear waves move on a spherical front. Sometimes the compression waves are called “push-pull” as they work like an accordion. Compression waves travel the fastest of all waves (4.8 km/second in granite), and they travel through both solids and liquids. Shear waves move from side to side. Because fluids (e.g. water and magma) have no shear stiffness, shear waves do not pass through. Shear waves are the second wave type to arrive, moving at about 3.0 km/second.
This slide contains two images, one showing a love wave, the other showing a Rayleigh wave.

The next waves to arrive are the surface waves. The two main types are Love waves and Rayleigh waves. These waves have a somewhat longer period than P or S waves.
This slide contains an image of an earthquake recording, showing the sequential arrival of P, S, and Love waves.

This recording shows the sequential arrival of P, S, and Love waves. With travel speeds of the various waves known, this type of diagram can be used to estimate the distance to the wave source.
Effects of Earthquakes

- Ground Failure
  - Rupture
  - Landslide
  - Liquefaction
  - Lateral Spreading
- Tsunami
- Seiche
- Ground Shaking

This slide contains text of hazards related to earthquakes

There are numerous hazards related to earthquakes. While ground shaking is emphasized in this topic, it is not necessarily the greatest hazard. Tsunamis in December 2004 and March 2011 were incredibly destructive. Ground shaking is responsible for several of the other effects: landslide, liquefaction, lateral spreading and seiche, which is oscillation of a body of water with effects similar to tsunami, but a completely different cause.
Slide contains images of recorded ground motions.

In most seismically active areas of the world seismologists have laid out vast instrument arrays that can capture the ground motion in terms of a recording of ground acceleration vs time. Usually, each instrument can record two horizontal components and one vertical component at the same station.

This slide shows some of the horizontal component recordings for a variety of earthquakes worldwide. All of the recordings are of the same horizontal (time) and vertical (acceleration) scale, with the maximum acceleration approximately 1.0 g. The character of the ground motion recording depends on many factors, including the soil type, the distance from the epicenter, and the direction of travel of seismic waves. The 1984 Mexico City Earthquake, shown at the bottom, is noted for its long duration and low frequency. This is characteristic of recordings on very soft soil, taken at some distance from the epicenter.
Slide contains images of records of the 1971 San Fernando Valley earthquake at a Holiday Inn.

Holiday Inn ground and building roof motion during the M6.4 1971 San Fernando earthquake: (a) north-south ground acceleration, velocity, and displacement and (b) north-south roof acceleration, velocity, and displacement (Housner and Jennings, 1982). The Holiday Inn, a 7-story, reinforced concrete frame building, was approximately 5 miles from the closest portion of the causative fault. The recorded building motions enabled an analysis to be made of the stresses and strains in the structure during the earthquake.
Overview

- Fundamental Concepts
- Ground Motions and Their Effects
- **Structural Dynamics of Linear SDOF Systems**
- **Response Spectra**
- Structural Dynamics of Simple MDOF Systems
- Inelastic Behavior
- Structural Design

Slide contains text providing an overview of the topics presented in this slide set.

This slide provides an overview of the topics presented in this slide set.
NEHRP (2009) Seismic Hazard Maps

- Probabilistic / Deterministic (Separate Maps)
- Uniform Risk (Separate Maps)
- Spectral Contours (PGA, 0.1, 0.2 sec)
- 5 % Damping
- Site Class B/C Boundary
- Maximum Direction Values

From 2009 NEHRP Provisions

Slide contains two images of maps of the USA, showing the spectral accelerations for 0.2 second structures and 1.0 second structures.

In the 2009 NEHRP Provisions and in ASCE 7-05 and -10, ground motion is represented by a quantity called Spectral Acceleration, which represents the total expected acceleration that a mass of a Single Degree of Freedom Structure (SDOF) would feel at a given location in the country. Chapter 3 of P-751 and Topic 3 of this slide series discusses the maps in some detail. The point made here is that the spectral accelerations are Response Spectrum ordinates, and that some grasp of structural dynamics is needed to understand what a response spectrum is.

The 2020 NEHRP Provisions establish spectral accelerations at 22 periods of vibrations, which will be shown following the introduction of the response spectrum concept.
This slide shows an image of an idealized and simple structure to demonstrate a dynamic dashpot system. An equation underneath shows the equation of dynamic equilibrium.

This slide shows a simple, highly idealized structure. In this structure the columns are flexible, and the beam is rigid. All of the mass of the structure is assumed to reside in the beam. A (fictitious) dashpot is shown for the purpose of providing some damping in the system. The properties of the system are the stiffness, $k$, the damping constant, $c$, and the mass, $m$. A time-varying horizontal load $F(t)$ is applied to the mass, and the displacement history $u(t)$ is to be obtained. The solution is obtained by solving the equation of dynamic equilibrium shown at the bottom of the slide, which represents a time-wise balance of inertial, damping, elastic, and applied forces.
This slide is focused on the mass of the dynamic system, and it contains text overviewing its parameters. An image on the left points out the mass in the simplified and idealizes structure. The graph on right shows the relationship between acceleration and force of mass.

The mass, $m$, of the system represents all of the weight of the structure and its fixed attachments. It may include some live load if it can be assumed that the live load (e.g. storage loads) will move in phase with the structure when it vibrates. Note that the units of force/acceleration.
In absence of dampers, is called inherent damping
- Usually represented by linear viscous dashpot
- Has units of force/velocity

Experience shows that a system set in motion and allowed to vibrate freely will eventually come to rest. This is due to damping in the system, which is a means of converting energy into heat, which is then irrecoverably dissipated. In real structures, damping occurs due to a variety of reasons, ranging from material damping, to friction in connections, and friction in nonstructural components and contents. In structural dynamics, it is convenient to represent the damping as a linear viscous “dashpot” for which the resistance is proportional to the deformational velocity in the dashpot (which is the same as the velocity of the mass relative to the base of the structure. The damping constant, \( c \), is impossible to specify directly, and instead, a damping ratio is used in computations (as shown later).
This slide is focused on the damping of the dynamic system, and it shows how energy is dissipated. An image on the left points out the damping in the simplified and idealizes structure. The graph on right illustrates the relationship between energy dissipated and the area of the force displacement plot.

When the damping force is plotted vs displacement, an elliptical hysteresis occurs (when the system is under steady state vibration). The area within the hysteresis curve represent the energy dissipated per cycle.

Image from Finley Charney
This slide is focused on the stiffness of the dynamic system, it and contains text overviewing its parameters. An image on the left points out the stiffness in the simplified and idealizes structure. The graph on right shows the relationship between displacement and force of stiffness.

The stiffness, $k$, of the structure is described in this slide. For real structures it is difficulty to obtain an exact value of $k$ because of a variety of uncertainties. For example, in concrete structures, cracking has a large influence on stiffness, but the amount of cracking due to environmental and service loads can not be predicted with any precision. Similarly, in steel structures, it is difficult to quantify the effect of connection stiffness, or the influence of partially composite slabs.
Inelastic Behavior

- Is almost always nonlinear in real seismic response
- Nonlinearity is implicitly handled by codes
- Explicit modelling of nonlinear effects is possible (but very difficult)

![Image from Finley Charney]

This slide is focused on the stiffness of the dynamic system, and it shows how energy is dissipated. An image on the left points out the stiffness in the simplified and idealizes structure. The graph on right illustrates the relationship between energy dissipated and the area of the force displacement plot.

A mentioned earlier, we can generally not afford to design structures to remain elastic during major earthquakes. Thus, we must allow the structure to deform inelastically in a controlled manner. Like damping, inelastic behavior produces an irrecoverable energy dissipation, shown here as the area enclosed within the cyclic force-deformation plot.

While it is possible to perform a nonlinear analysis, this is quite difficult, and is done only in special circumstances. The vast majority of analysis is performed using linear procedures which implicitly account for the inelastic behavior in the structure.
Undamped Free Vibration

Equation of motion: \[ m\ddot{u}(t) + k u(t) = 0 \]

Initial conditions: \[ \dot{u}_0 \quad u_0 \]

Assume: \[ u(t) = A \sin(\omega t) + B \cos(\omega t) \]

Solution: \[ A = \frac{\dot{u}_0}{\omega} \quad B = u_0 \quad \omega = \sqrt{\frac{k}{m}} \]

\[ u(t) = \frac{\dot{u}_0}{\omega} \sin(\omega t) + u_0 \cos(\omega t) \]

Slide contains equations of undamped free vibration of a structure.

It is beyond the scope of this topic to present the details of computing the dynamic response. However, the computed responses of a few simple loadings will be presented to provide some needed nomenclature.

The first such loading is undamped free vibration, where the damping is assumed to be absent, and the structure is set in motion by an initial displacement and/or velocity. The response is in the form of a sine wave. The frequency of vibration, (omega) is called the circular or angular frequency, and it has units of radians/second. As can be seen, this frequency is in effect a structural property as it involves only mass and stiffness.
Undamped Free Vibration (2)

Circular Frequency (radians/sec) \[ \omega = \sqrt{\frac{k}{m}} \]

Cyclic Frequency (cycles/sec, Hertz) \[ f = \frac{\omega}{2\pi} \]

Period of Vibration (sec/cycle) \[ T = \frac{1}{f} = \frac{2\pi}{\omega} \]

Slide contains an image illustrating undamped free vibration. Equations are displayed to find the circular frequency, cyclic frequency and period of vibration.

This slide shows the dynamic response of a system in undamped free vibration. In structural engineering, it is common to use the period of vibration, \( T \), instead of circular frequency (omega) or cyclic frequency (Hz).

The proper units of period of vibration is seconds/cycle, although the word “cycle” is almost universally omitted.
### Periods of Vibration of Common Structures

<table>
<thead>
<tr>
<th>Structure</th>
<th>Period (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20-story moment resisting frame</td>
<td>T = 2.4 sec</td>
</tr>
<tr>
<td>10-story moment resisting frame</td>
<td>T = 1.3 sec</td>
</tr>
<tr>
<td>1-story moment resisting frame</td>
<td>T = 0.2 sec</td>
</tr>
<tr>
<td>20-story steel braced frame</td>
<td>T = 1.6 sec</td>
</tr>
<tr>
<td>10-story steel braced frame</td>
<td>T = 0.9 sec</td>
</tr>
<tr>
<td>1-story steel braced frame</td>
<td>T = 0.1 sec</td>
</tr>
<tr>
<td>Gravity dam</td>
<td>T = 0.2 sec</td>
</tr>
<tr>
<td>Suspension bridge</td>
<td>T = 20 sec</td>
</tr>
</tbody>
</table>

Slide shows typical periods of vibrations of common structures.

This slide shows some periods of vibration of common structures. The *Provisions* provides approximate formulas for computing period, based on material, structural system, and height.
Damped Free Vibration

Equation of motion: \[ m \ddot{u}(t) + c \dot{u}(t) + k u(t) = 0 \]

Initial conditions: \( u_0, \dot{u}_0 \)

Assume: \( u(t) = e^{st} \)

Solution:

\[
 u(t) = e^{-\xi \omega_D t} \left[ u_0 \cos(\omega_D t) + \frac{\dot{u}_0 + \xi \omega u_0}{\omega_D} \sin(\omega_D t) \right]
\]

\[ \xi = \frac{c}{2m \omega} = \frac{c}{c_c} \]

\[ \omega_D = \omega \sqrt{1 - \xi^2} \]

Slide contains equations of damped free vibration of a structure.

In this slide the equation for motion is shown for the system under damped free vibration is shown. Two important quantities result from the solution to the equation, the damping ratio (Greek letter xi), and damped frequency of vibration omega sub D. In earthquake engineering, xi is almost universally taken as 0.05, and c is then back-calculated using this value, m, and omega. For xi=0.05, there is practically no difference between the damped and undamped frequency, so the undamped value is used for convenience.
Damped Free Vibration (2)

\[ \xi = \frac{c}{2m\omega} = \frac{c}{c_c} \]

\( c_c \) is the critical damping constant.

\( \xi \) is expressed as a ratio \((0.0 < \xi < 1.0)\) in computations.

Sometimes \( \xi \) is expressed as a\% \((0 < \xi < 100\%)\).

Slide contains equations regarding damping, and a graph of a displacement vs time of a critically damped system.

The important point on this slide is the use of \( \xi \) as a ratio versus a percentage. Some computer programs require this number as input, and it is important to get the value entered correctly.
True damping in structures is NOT viscous. However, for low damping values, viscous damping allows for linear equations and vastly simplifies the solution.

This slide contains a graph showing the effect of different percentages of damping on a displacement vs time plot.

In free vibration, damping reduces the response over time, and the greater the damping, the more rapid the decay in vibration amplitude.
## Damping in Structures

<table>
<thead>
<tr>
<th>Structure</th>
<th>$\zeta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Welded steel frame</td>
<td>0.010</td>
</tr>
<tr>
<td>Bolted steel frame</td>
<td>0.020</td>
</tr>
<tr>
<td>Uncracked prestressed concrete</td>
<td>0.015</td>
</tr>
<tr>
<td>Uncracked reinforced concrete</td>
<td>0.020</td>
</tr>
<tr>
<td>Cracked reinforced concrete</td>
<td>0.035</td>
</tr>
<tr>
<td>Glued plywood shear wall</td>
<td>0.100</td>
</tr>
<tr>
<td>Nailed plywood shear wall</td>
<td>0.150</td>
</tr>
<tr>
<td>Damaged steel structure</td>
<td>0.050</td>
</tr>
<tr>
<td>Damaged concrete structure</td>
<td>0.075</td>
</tr>
<tr>
<td>Structure with added damping</td>
<td>0.250</td>
</tr>
</tbody>
</table>

This slide contains typical damping percentages of different structures.

This slide lists some reasonable damping ratios in structures, and as can be seen the values can vary considerably. In the *Provisions*, values of damping other than 5% critical are (very approximately) accounted for in the response modification coefficient, $R$. Note that the seismic hazard maps are based on 5% critical damping.
Under Harmonic Loading and Resonance

\[ m \ddot{u}(t) + k u(t) = p_0 \sin(\omega t) \]

This slide contains an image of undamped harmonic loading and response.

When an undamped system is loaded harmonically, the response will grow without bound when the loading frequency (omega bar) is equal to the structure’s own natural frequency. Such a phenomena is known as “resonance.” An unbounded response is, of course, only mathematical because any system will become damaged and fail at some point of deformation if the system is brittle, or it will yield (changing the structure’s frequency and adding damping in the form of hysteretic energy dissipation) and fall out of resonance. Nevertheless, resonance is undesirable. Unfortunately, all ground motions will contain at least one frequency which is in phase with the structure’s own natural frequency, so all earthquake response is resonant. In most cases the structure will not fall out of resonance when the system yields because there is some other frequency component of the ground motion which will be in resonance with the new structural frequency.
This slide contains an image showing damped harmonic loading and response.

When the damped system is loaded at resonance the response will initially build up rapidly, but it will eventually reach a steady state displacement as shown in the plot. The maximum displacement achieved (assuming no yielding) is \( \frac{1}{2\zeta} \) times the static displacement (e.g. the displacement under \( p_0 \) applied as a static load).
This slide contains an image of a resonant response curve.

This plot is a set of resonance response curves, with one curve plotted for each of four damping values. The vertical axis (amplifier) is the ratio of maximum dynamic response to static response, and the horizontal axis is the ratio of loading frequency to structural frequency. As may be seen, when beta=1 the system is in resonance, and damping is extremely effective in reducing response. Since earthquake loading is a resonant phenomena, damping is always very important, and the more the better.
General Dynamic Loading

- Fourier transform
- Duhamel integration
- Piecewise exact
- Newmark techniques

All techniques are carried out numerically.

This slide contains text showing the different ways in which it is possible to solve dynamic response.

For most types of dynamic loading there is not a closed-form solution. In such cases it is necessary to solve for the response using numerical procedures. This slide lists some of the more common methods. The last two methods are most commonly used. The piecewise exact method is restricted to linear systems, and it produces a mathematically exact response of the loading consists of linear load between discreet time increments. The Newmark method is close to exact for liner systems, and it may be used for nonlinear systems as well.
This slide contains two images, one of the simplified and idealized structure, the other of a ground acceleration response history. An equation underneath is of dynamic equilibrium.

Seismic load in the form of a ground acceleration history is converted to an equivalent seismic load as shown in this slide. Note that the response quantities (displacement, velocity, acceleration) are relative to the base of the structure, hence the subscript \( r \). Given the digital nature of the loading, the piecewise linear method is most suitable for solution when the response is linear.
Simplified SDOF Equation of Motion

\[ \ddot{u}_r(t) + c \dot{u}_r(t) + ku_r(t) = -m\ddot{g}(t) \]

Divide through by m:

\[ \ddot{u}_r(t) + \frac{c}{m} \dot{u}_r(t) + \frac{k}{m} u_r(t) = -\ddot{g}(t) \]

Make substitutions:

\[ \frac{c}{m} = 2\xi\omega \quad \frac{k}{m} = \omega^2 \]

Simplified form:

\[ \ddot{u}_r(t) + 2\xi\omega \dot{u}_r(t) + \omega^2 u_r(t) = -\ddot{g}(t) \]

This slide contains equations of a simplified single degree of freedom system, or SDOF.

In this slide a revised equation of motion is shown where all terms have been divided through by mass (never zero). Now the response to a given ground motion can be computed “generically” for a system with a given damping ratio and frequency of vibration (actual mass and stiffness need not be specified). This form of the equation of motion is useful for computing response spectra.
Use of Simplified Equation of Motion

For a given ground motion, the response history $u_r(t)$ is function of the structure’s frequency $\omega$ and damping ratio $\xi$.

$$\ddot{u}_r(t) + 2\xi \omega \dot{u}_r(t) + \omega^2 u_r(t) = -\ddot{u}_g(t)$$

This slide clarifies different parts of the SDOF equation.

This slide repeats some of the points made on the previous slide, but in a more clear fashion.
Use of Simplified Equation

Change in ground motion or structural parameters $\zeta$ and $\omega$ requires re-calculation of structural response.

This image illustrates the use of a “Solver”, such as the Piecewise Exact Method, to compute the response history. There are two graphs of the input and output of the solver.

This slide illustrates the use of a “Solver”, such as the Piecewise Exact Method, to compute the response history. For the purpose of computing response spectra, the key parameter of interest in the solution is the absolute value of maximum or “Peak” displacement, regardless of the sign or time of occurrence.
Creating an Elastic Response Spectrum

An elastic displacement response spectrum is a plot of the peak computed relative displacement, $u_r$, for an elastic structure with a constant damping $\zeta$, a varying fundamental frequency $\omega$ (or period $T = 2\pi/\omega$), responding to a given ground motion.

Image from Finley Charney

The graph is a spectral displacement plot.

An elastic displacement response spectrum is computed by repeatedly solving a system with a given ground motion, given damping, and varying period of vibration. Note the jagged appearance of the curve, and mention that no two earthquakes will produce the same response spectrum. Note also that the displacement quantity being computed is the relative displacement.
This slide contains a plot of a pseudoacceleration spectrum, and an equation showing how it is derived.

A very important curve, derived from the displacement spectrum, is the pseudoacceleration spectrum. It is obtained by dividing each displacement ordinate by the square of the circular frequency at a given period. The mapped values of acceleration given by the Provisions are, essentially, pseudoacceleration ordinates at periods of 0.2 and 1.0 seconds.
The pseudoacceleration response spectrum represents the total acceleration of the system, not the relative acceleration. It is nearly identical to the true total acceleration response spectrum for lightly damped structures.

This slide shows a plot of pseudoacceleration, and it highlights that pseudoacceleration is total acceleration.

It is very important to recognize that pseudoacceleration is total acceleration, even though it is derived from relative displacement. Thus, for $T=0$, pseudoacceleration is equal to the peak ground acceleration. At very large period (greater than about 10 seconds) pseudoacceleration will approach zero.
Using Pseudoacceleration to Compute Seismic Force

Example Structure

\[ K = 500 \text{ k/in} \]
\[ W = 2,000 \text{ k} \]
\[ M = \frac{2000}{386.4} = 5.18 \text{ k-sec}^2/\text{in} \]
\[ \omega = (K/M)^{0.5} = 9.82 \text{ rad/sec} \]
\[ T = 2\pi/\omega = 0.64 \text{ sec} \]
5% critical damping

At \( T = 0.64 \text{ sec} \), pseudoacceleration = 301 in./sec\(^2\)
Base shear = \( M \times \text{PSA} = 5.18(301) = 1559 \text{ kips} \)

This slide shows the same graph of pseudoacceleration and demonstrates how to use the graph to compute seismic force.

Given the pseudoacceleration spectrum, it is easy to compute the peak base shear developed in a SDOF system. Assuming 5% damping and the earthquake used to generate the spectrum (El Centro), the shear can be computed as shown. Once the shear is known the relative displacement can be found by dividing by \( k \) (thus the displacement spectrum is not needed)
Response Spectra for 1971 San Fernando Valley EQ (Holiday Inn)

This slide contains a plot of spectral acceleration of different damping ratios for the 1971 San Fernando valley earthquake.

This slide shows several pseudoacceleration spectra for the 1971 San Fernando Valley, 1971 EQ. Here, the different spectra are for different damping values, ranging from zero to 20% critical. The bold red line is for 5% damping. Note the tremendous influence of damping on amplitude and shape, particularly at the lower periods.
This slide contains a plot comparing the average spectrum compared to a smoothed code spectrum.

This chart shows the mean spectrum among seven ground motion spectra, the mean plus one standard deviation, and the “smoothed” code version of the spectrum (red line). Note the close fit between the average spectrum and the code spectrum (such fits are not always possible). The *Provisions*, as well as ASCE 7 use the smoothed spectrum in lieu of true ground motion spectra because of the ease of use and uniformity.
This slide contains a graph of the NEHRP/ASCE7 basic design spectrum.

This slide shows the basic design spectrum, which is Figure 11.4-1 from ASCE 7-10. This spectrum would be used in Modal Response Spectrum Analysis as described in Section 12.9 of ASCE 7-10. The shape of this spectrum can be traced back to work done by Newmark in the 1960s. The Constant Velocity label indicates that if the spectrum were to be converted to a velocity spectrum, this segment of the spectrum would be constant. Similar for the Constant Displacement branch. Note the unit inconsistency of the values; for example, the division by \( T \) in the constant velocity region produces units inconsistency if the units of seconds are attached to \( T \).

See the slide set on “Ground Motions” for more detail.
This slide shows the new multi-period spectrum (22 points) that is the basis of the 2020 NEHRP Provisions and ASCE 7-22, and how the conventional two period spectrum is derived from it. The $S_{DS}$ value is 90% of the maximum $S_a$ in short periods, and the $S_{D1}$ value is set so the curve does not fall below 90% of the $S_a$ in the velocity domain.
Overview

- Fundamental Concepts
- Ground Motions and Their Effects
- Structural Dynamics of Linear SDOF Systems
- Response Spectra
- **Structural Dynamics of Simple MDOF Systems**
- Inelastic Behavior
- Structural Design

This slide contains text that provides an overview of the topics presented in this slide set.

This slide provides an overview of the topics presented in this slide set.
This slide contains an image of a simplified multiple degree of freedom system, or MDOF.
Analysis of Linear MDOF Systems

- MDOF Systems may either be solved step by step through time by using the full set of equations in the original coordinate system, or by transforming to the “Modal” coordinate system, analyzing all modes as SDOF systems, and then converting back to the original system. In such a case the solutions obtained are mathematically exact, and identical. This analysis is referred to as either Direct (no transformation) or Modal (with transformation) Linear Response History Analysis. This procedure is covered in Chapter 16 of ASCE 7.

- Alternately, the system may be transformed to modal coordinates, and only a subset (first several modes) of equations be solved step by step through time before transforming back to the original coordinates. Such a solution is approximate. This analysis is referred to as Modal Linear Response History Analysis. This procedure is not directly addressed in ASCE 7 (although in principle, Ch. 16 could be used)

This slide contains text explaining MDOFs, and how to solve them.

The points in this slide are self-explanatory.
Another alternate is to convert to the modal coordinates, and instead of solving step-by-step, solve a subset (the first several modes) of SDOF systems system using a response spectrum. Such a solution is an approximation of an approximation. This analysis is referred to as Modal Response Spectrum Analysis. This procedure is described in Chapter 12 of ASCE 7.

Finally, the equivalent lateral force method may be used, which in essence, is a one-mode (with higher mode correction) Modal Response Spectrum Analysis. This is an approximation of an approximation of an approximation (but is generally considered to be “good enough for design”). The Provisions and ASCE 7 do place some restrictions on the use of this method.

This slide contains additional text explaining MDOFs, and how to solve them.

The points in this slide are self-explanatory.
Overview

- Fundamental Concepts
- Ground Motions and Their Effects
- Structural Dynamics of Linear SDOF Systems
- Response Spectra
- Structural Dynamics of Simple MDOF Systems
- Inelastic Behavior
- Structural Design

This slide contains text that provides an overview of the topics presented in this slide set.

This slide provides an overview of the topics presented in this slide set.
Basic Base Shear Equations in NEHRP and ASCE 7

\[ V = C_s W \]

\[ C_s = \frac{S_{DS}}{R/I_e} \]

\[ C_s = \frac{S_{D1}}{T(R/I_e)} \]

\( S_{DS} \) and \( S_{D1} \) are short and one second (T=0.2 s and 1.0 s) Design Basis Spectral Accelerations, including Site Effects

\( I_e \) is the Importance Factor

\( R \) is a Response Modification Factor, representing Inelastic Behavior (Ductility, Over-strength, and a few other minor ingredients).

This slide contains equations covering how to compute base shear based on the NEHRP Provisions and ASCE 7

These formulas from Chapter 12 of ASCE 7 have the value \( R \) in the denominator. The next several slides provide the background on \( R \).
Building Designed for Wind or Seismic Load

Building properties:
- Moment resisting frames
- Density $\rho = 8$ pcf
- Period $T = 1.0$ sec
- Damping $\xi = 5\%$
- Soil Site Class “B”

Total wind force on 120’ face = 406 kips
Total wind force on 90’ face = 304 kips
Total **ELASTIC** earthquake force (in each direction) = 2,592 kips

This slide contains an image of a simplified structure and its properties, and design wind and seismic loads.

This slide shows (without detailed computations) the total factored wind forces acting on a building at some location, and the ELASTIC seismic forces for the same location. This could represent, for example, a building bear Charleston, S.C.
Comparison of EQ vs Wind

- ELASTIC earthquake forces 6 to 9 times wind!
- Virtually impossible to obtain economical design

\[
\frac{V_{EQ}}{V_{W\text{120}}} = \frac{2592}{406} = 6.4
\]
\[
\frac{V_{EQ}}{V_{W\text{90}}} = \frac{2592}{304} = 8.5
\]

This slide compares the difference between design elastic seismic forces and design wind loads.

The computation shows that the seismic loads are 6.4 to 8.5 times the wind forces. When designing for wind it is assumed that the structure remains elastic up to the factored wind loads. It would seem economically prohibitive to design the building to remain elastic for the earthquake loads.
How to Deal with Huge EQ Force?

- Pay the premium for remaining elastic
- Isolate structure from ground (seismic isolation)
- Increase damping (passive energy dissipation)
- **Allow controlled inelastic response**

Historically, building codes use inelastic response procedure. Inelastic response occurs through structural damage (yielding). We must control the damage for the method to be successful.

This slide contains text discussing different means to design for large earthquake forces.

There are a variety of strategies for dealing with the large earthquake forces. The first choice should be avoided except in very special circumstances, such as the design of nuclear power plants. The second two approaches are viable (and supported by the Provisions and ASCE 7 with separate chapters) but are not commonly used due to extra cost and required expertise. The vast majority of buildings are designed using the inelastic response method. This is true even in the highest seismic risk areas.
This slide contains an image demonstrating a nonlinear static pushover on a frame, and below shows a plot of its resulting force displacement relationship.

In this slide, we will assume the building with a 2,592 k earthquake load is designed for a strength of about 500 kips, well less that the seismic demand. Assuming the structure has sufficient ductility, it is mathematically “Pushed Over” to form the solid blue line force-displacement shown. Note that the building has considerable reserve strength (called over-strength) beyond first yield. We will make a rather large simplification and assume that the entire building can then be represented dynamically as a SDOF system with the bilinear force-deformation curve shown in red, perform a nonlinear response history analysis to assess the expected behavior. Note that ductility is the ability to deform beyond first yield without excessive loss of strength.

Note that actual ductility above 10 (as shown in the slide) is not realistic for real buildings, and even when R=8 a significant portion of that value is related to entities other than ductility (as explained later in this set).
This slide shows the simplified mathematical model and the ground motion used for response history analysis.
Results of Nonlinear Analysis

This slide shows plots of the displacement, spring force, and number yield events of a nonlinear analysis.

The analysis results, produced by the NONLIN Program, is show on this slide. The maximum displacement achieved is 4.79 inches and the maximum shear is 542 kips. There are 15 yield events, seven in one direction and eight in the other.
Yield displacement $= \frac{500}{550} = 0.91$ inch

\[
Ductility\ Demand = \frac{\text{Maximum}\ Displacement}{\text{Yield}\ Displacement} = \frac{4.79}{0.91} = 5.26
\]

This image demonstrates the concept of ductility demand in graphical form. The first panel is member force only, the second panel is member shear only, and the third panel is member shear plus damping, or total base shear.

This slide gives the basic definition for ductility demand. If this much ductility is not actually supplied by the structure, collapse may occur. Note the difference in the second and third force-displacement panels of the slide. The second panel is member shear only and the third panel is member shear plus damping, or total base shear.

Write basic design equation:

Ductility Demand $<$ Ductility Supply
Interim Conclusion (the Good News)

The frame, designed for a wind force which is 15% of the ELASTIC earthquake force, can survive the earthquake if:

- It has the capability to undergo numerous cycles of INELASTIC deformation
- It has the capability to deform at least 5 to 6 times the yield deformation
- It suffers no appreciable loss of strength

REQUIRES ADEQUATE DETAILING

This slide has text going over conclusions of the analysis.

This slide is the “Good News.” It appears that the building can survive the earthquake, but ONLY if the above conditions are met.
Interim Conclusion (The Bad News)

As a result of the large displacements associated with the inelastic deformations, the structure will suffer considerable structural and nonstructural damage.

- This damage must be controlled by adequate detailing and by limiting structural deformations (drift).

This slide has text going over conclusions of the analysis.

The bad news is that the building will probably suffer considerable damage to the structural and nonstructural system. It may be repairable after the earthquake, but there is no guarantee. Also, the Provisions and ASCE 7 are based on the recognition that there is a small (about 1% in 50 years) but real probability of collapse. Neither the NEHRP Provisions or ASCE 7 provides any explicit protection from damage during moderate earthquakes, and thus they are both considered as “Life Safety” provisions. New concepts in earthquake engineering, called Performance Based Design are being developed to alleviate this issue, but it will be several years before these concepts are brought into the code.
This slide contains text that goes over the development of the equal displacement concept.

The equal displacement concept is the basis for dividing the “Elastic” force demands by the factor $R$. This is one of the most important concepts in earthquake engineering. The basis for the equal displacement concept is illustrated in the following slides.
The Equal Displacement Concept

“The displacement of an inelastic system, with stiffness $K$ and strength $F_y$, subjected to a particular ground motion, is approximately equal to the displacement of the same system responding elastically.”

(The displacement of a system is independent of the yield strength of the system.)

This slide contains text explaining the equal displacement concept.

The Equal Displacement Concept in words.
Repeated Analysis for Various Yield Strengths (and constant stiffness)

This slide contains plots from NONLIN analyses showing the effects of yield strength on response.

This slide is based on a series of NONLIN analyses wherein all parameters were kept the same as in the original model except for the yield strength, which was systematically increased in 500 kip increments to a maximum of 3,500 kips. The structure with a 3,500 kip strength remains elastic during the earthquake. Note that the displacement appears to be somewhat independent of yield strength, but the ductility demand is much higher for relatively lower strengths.
This slide contains two plots highlighting the difference between actual and idealized inelastic response of equal displacement.

This slide shows simplified force-displacement envelopes from the different analyses. An apparently conservative assumption (with regard to displacements) is shown on the right. The basic assumption is that the displacement demand is relatively insensitive to system yield strength. This is often referred to as the “equal displacement” concept of seismic-resistant design.
Equal Displacement Idealization of Inelastic Response

- For design purposes, it may be assumed that inelastic displacements are equal to the displacements that would occur during an elastic response.
- The required force levels under inelastic response are much less than the force levels required for elastic response.

This slide contains text summarizing last slide.
This slide contains a plot showing the concept of equal displacement in a force displacement plot.

The equal displacement concept allows us to use elastic analysis to predict inelastic displacements. For the example system, the predicted elastic displacement (red line) is 5.77 inches, and it is assumed that the inelastic response (blue line) displacement is the same.
This slide contains the same plot as last slide, but it highlights the ductility demand of the inelastic system.

For this simple bilinear system, the ductility demand can now be computed. The system must be detailed to have this level of ductility. For design purposes, we typically reverse the process. We assume some ductility supply (based on the level of detailing provided) and, using this, we can estimate the strength requirements.
Application in Principle

Using response spectra, estimate **elastic** force demand $F_E$

Estimate ductility supply, $m$, and determine **inelastic** force demand $F_I = F_E / m$. Design structure for $F_I$.

Compute reduced displacement, $d_R$, and multiply by $m$ to obtain true inelastic displacement, $d_I$. Check Drift using $d_I$.

This slide highlights the procedure to apply inelastic force demand in practice. There is an image of a force displacement plot, and it highlights the elastic and inelastic forces one would design for.

The procedure mentioned in the previous slide is explained in more detail here.

Building codes allow for an elastic structural analysis based on applied forces reduced to account for the presumed ductility supplied by the structure. For elastic analysis, use of the reduced forces will result in a significant underestimate of displacement demands. Therefore, the displacements from the reduced-force elastic analysis must be multiplied by the ductility factor to produce the true “inelastic” displacements.
Use basic elastic spectrum but, for strength, divide all pseudoacceleration values by $R$, a response modification factor that accounts for:

- Anticipated ductility supply
- Overstrength
- Damping (if different than 5% of critical)
- Past performance of similar systems
- Redundancy

This slide contains text explaining the response modification factor $R$, and its implications.

The approach to using the equal displacement concept is discussed in the next several slides. One of the key aspects of the method is the use of the response modification factor, $R$. This term includes a variety of “ingredients,” the most important of which are ductility and overstrength.

Note that overstrength did not enter into the previous discussion because we were working with idealized systems. Real structures are usually much stronger than required by design. This extra strength, when recognized, can be used to reduce the ductility demands. (If the overstrength was so large that the response was elastic, the ductility demand would be less than 1.0.)
This slide contains an image of a simplified frame undergoing a pushover analysis. Underneath shows a force displacement plot, highlighting first significant yield.

In this slide and several that follow as structure is being subjected to a pushover analysis. The structure remains essentially elastic until the first full plastic hinge forms. The formation of this “first significant yield” occurs at a level of load referred to as the design strength of the system.

If the hinging region has adequate ductility, it can sustain increased plastic rotations without loss of strength. At the same time, the other potential hinging regions of the structure will attract additional moment until they begin to yield.
First Significant Yield and Design Strength

First Significant Yield is the level of force that causes complete plastification of at least the most critical region of the structure (e.g., formation of the first plastic hinge).

The design strength of a structure is equal to the resistance at first significant yield.

This slide contains text explaining first significant yield.

These definitions come from the commentary to the NEHRP Provisions and the commentary to ASCE 7.
Overstrength

This slide shows the same simplified frame undergoing pushover and the sequential formation of plastic hinges. Underneath, the force displacement curve also shows the plastic hinges.

This slide show the sequential formation of plastic hinges in the structure. With sufficient ductility, the apparent strength can be considerably greater than the design strength. The reserve capacity is called Overstrength.
Sources of Overstrength

- Sequential yielding of critical regions
- Material overstrength (actual vs specified yield)
- Strain hardening
- Capacity reduction ($\phi$) factors
- Member selection
- Structures where the proportioning is controlled by the seismic drift limits

This slide contains text listing the sources of overstrength.

This slide lists most of the sources of overstrength. It is not uncommon for the true strength of a structure, including overstrength, to be two to three times the design strength.
The image is a force displacement plot showing the definition of overstrength.

This slide defines overstrength in both an equation, and in graphical format in a force displacement plot.

The apparent strength divided by the design strength is called the “overstrength factor.” Note that the symbol $\Omega$ used for the overstrength factor is similar to the term $\Omega_0$ in ASCE 7. As implemented in ASCE 7, $\Omega_0$ is intended to be a high estimate of true overstrength (although not an upper bound).
The image is a force displacement plot showing the definition of ductility reduction factor $R_d$.

This slide defines ductility reduction factor $R_d$ in both equation and in graphical format in a force displacement plot.

This is the definition of that part of $R$ due to ductility.
This slide contains equations of overstrength and ductility in equation format, and $R$, which is the product of overstrength and ductility reduction.

The NEHRP/ASCE 7 response modification factor, $R$, is equal to the ductility reduction factor times the overstrength factor.

Caution the student that the required $R_d$ is not equal to $R$ divided by the tabulated $\Omega_0$, because the tabulated values are definitely not lower bound values.
This figure is a graphical version of the information presented in the previous slide. The response modification factor, $R$, is used to reduce the expected elastic strength demand to the DESIGN level strength demand.

This slide defines the response modification coefficient, $R$, in graphical format in a force displacement plot.

On the basis of the equal displacement theory the inelastic displacement demand is the same as the elastic displacement demand. For design purposes, however, the reduced design strength is applied to the structure to determine the member forces. The analysis domain represents the response of the linear elastic system as analyzed with the reduced forces. Clearly the displacement predicted by this analysis is too low. The *Provisions* (ASCE 7) compensates through the use of the $C_d$ factor.
This slide defines the deflection amplification factor, $C_d$, in graphical format in a force displacement plot.

To correct for the too-low displacement predicted by the reduced force elastic analysis, the “computed design displacement” is multiplied by the factor $C_d$. This factor is always less than the $R$ factor because $R$ contains ingredients other than pure ductility. In theory the primary factor reducing displacement demand is damping that is higher than the standard 5% damping used to develop the design spectra in the Provisions, but some of the $C_d$ factors are not consistent with this explanation. Overstrength does not contribute to reduction of displacement demand.
### Example of Design Factors for Reinforced Concrete Structures

<table>
<thead>
<tr>
<th>System</th>
<th>$R$</th>
<th>$\Omega_0$</th>
<th>$C_d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Special Moment Frame</td>
<td>8</td>
<td>3</td>
<td>5.5</td>
</tr>
<tr>
<td>Intermediate Moment Frame</td>
<td>5</td>
<td>3</td>
<td>4.5</td>
</tr>
<tr>
<td>Ordinary Moment Frame</td>
<td>3</td>
<td>3</td>
<td>2.5</td>
</tr>
<tr>
<td>Special Reinforced Shear Wall</td>
<td>5</td>
<td>2.5</td>
<td>5.0</td>
</tr>
<tr>
<td>Ordinary Reinforced Shear Wall</td>
<td>4</td>
<td>2.5</td>
<td>4.0</td>
</tr>
<tr>
<td>Detailed Plain Concrete Wall</td>
<td>2</td>
<td>2.5</td>
<td>2.0</td>
</tr>
<tr>
<td>Ordinary Plain Concrete Wall</td>
<td>1.5</td>
<td>2.5</td>
<td>1.5</td>
</tr>
</tbody>
</table>

This slide contains example design factors for typical structure types.

These are the design coefficients for a few selected concrete systems. Note the very low $R$ values for the plain walls. These plain wall systems are allowed only in SDC A and B buildings. ACI 318 only permits plain concrete where it is continuously supported.

Note that the values $\Omega_0$ are not exactly the same as the overstrength factor Omega, and might be considered as reasonable upper bounds on Omega. They are used in certain load combinations that require extra capacity in key elements and components of a structure.
This slide contains a graph of design spectra adjusted for inelastic behavior.

In the previous slides, the concept of the reduction factor, \( R \), was presented, and several values were illustrated. Here, the effect of the \( R \) value of the design response spectrum is illustrated for \( R = 1 \) (elastic) through 6. The value for \( R = 1 \) has been normalized to produce a peak short period acceleration of 1.0g.
This slide contains a graph of pseudoacceleration spectra with different $R$ factors.

This slide simply shows how the design base shear is determined for a system with $T = 0.8$ seconds and $R = 4$. The pseudoacceleration spectrum is used.
Using the Inelastic Spectrum and $C_d$ to Determine the Inelastic Displacement Demand

$$\Delta_{INELASTIC} = C_d \times \Delta_{REDUCED ELASTIC}$$

This slide conveys how to compute the inelastic displacement demand through the use of $C_d$ and the inelastic displacement spectrum.

At the period of 0.8 the displacement is read off the red line, which includes $C_d$. In practice, one would not generally use a displacement spectrum. Instead, the displacements would be determined from the elastic model with the reduced $(1/R)$ loads, and these would be multiplied by $C_d$. 

$C_d = 3.5$

$\Delta_{INELASTIC} \approx 3.65 \text{ in}$
Overview

- Fundamental Concepts
- Ground Motions and Their Effects
- Structural Dynamics of Linear SDOF Systems
- Response Spectra
- Structural Dynamics of Simple MDOF Systems
- Inelastic Behavior
- **Structural Design**

This slide contains text that provides an overview of the topics presented in this slide set.

This slide provides an overview of the topics presented in this slide set.
Design and Detailing Requirements

This slide contains images of the cover of ACI and AISC.

While the Provisions and ASCE 7 provide the basic configuration and loading requirements, the Detailing that is necessary to support a given $R$ value is provided in materials standards such as ACI 318 and AISC 341. In some cases, the International Building Code or local jurisdictions will have additional requirements.
Questions
DISCLAIMER

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- The opinions expressed herein regarding the requirements of the NEHRP Recommended Seismic Provisions, the referenced standards, and the building codes are not to be used for design purposes. Rather the user should consult the jurisdiction’s building official who has the authority to render interpretation of the code.

- This training material presentation is intended to remain complete in its entirety even if used by other presenters. While the training material could be tailored for use in other presentations, we caution users to account for issues of completeness and interpretation if only part of the material is used. We also strongly suggest users give proper credit/citation to this presentation and its author.
Chapter 2 (Section 2.7)
Resilience-Based Design

2020 NEHRP Provisions Training Materials
David Bonowitz, S.E.
Content

- Development of resilience-based earthquake design
  - 2020 NEHRP Provisions, Resource Paper 1
- Functional Recovery (FR)
  - Its relation to resilience
  - Its relation to current building code provisions
- Hypothetical application to the CLT Design Example
  - CLT Shear Wall Design Example is in Chapter 6
  - Discussion in terms of resilience-based design is in Section 2.7
Consensus

NIST, 2016
ICC, 2019
EERI, 2019
FEMA, 2020
FEMA-NIST, 2021

Oregon, 2013
White House, 2016
San Francisco, 2016
Los Angeles, 2018
Public Law, 2018
California, 2021
Consensus understanding of *resilience*

- An attribute of organizations or social units, not buildings
  - Congress’ 2018 NEHRP Reauthorization focuses on resilience at the *community* scale
- Emphasis on recovery, not just safety
- Measured in terms of time, not immediate damage
- Relative to a natural hazard event

**Q:** If resilience is not about individual buildings, what does it mean to design individual buildings for resilience?

**A:** Design to achieve a “functional recovery objective.”
The “Resilience Field”

Technical

About the physical building
• Structure
• Nonstructural systems

Facility

About one building. Typical context for:
• Engineering
• Building code implementation

Holistic

About more than a building
• Contents → Use, Occupancy
• Purpose

Community

About the group. Typical context for:
• Planning
• Public policy

Meister Consultants Group, 2017
The “Resilience Field”

Design
• NEHRP Provisions
• ASCE 7
• IBC

Thinking
• NEHRP Reauthorization
• City resilience plans
FR : Building : CR : Community

- Functional Recovery
- Technical
- Holistic
- Community Resilience

Facility

Community
“Resilience-Based Design and the NEHRP Provisions”

- Federal policy now prioritizes earthquake resilience
- Improve resilience by designing for functional recovery
- Current “code and standard” model is promising for development of functional recovery design provisions
- NEHRP Provisions can support a functional recovery design standard
New definitions: Functional Recovery

NIST, 2016

ICC, 2019

EERI, 2019

FEMA, 2020

FEMA-NIST, 2021

Oregon, 2013

White House, 2016

San Francisco, 2016

Los Angeles, 2018

Public Law, 2018

California, 2021
FEMA-NIST definitions*

- Functional Recovery (FR) is ...
  - A post-earthquake performance state in which a building is maintained, or restored, to support the basic intended functions associated with the pre-earthquake use or occupancy.

- A Functional Recovery objective is ...
  - FR achieved within an acceptable time following a specified earthquake, where the acceptable time might differ for various building uses and occupancies.

* The FEMA-NIST definitions consider infrastructure systems as well as buildings. These versions are edited to address only buildings.
Functional recovery and performance-based engineering

- A structural safety objective may be written as: \( P(\text{collapse}) < X\% \), given \( 2/3 \cdot \text{MCE}_{R} \)

- Analogously, a functional recovery objective may be written as:
  \[
P(\text{T}_{\text{FR, expected}} > \text{T}_{\text{FR, acceptable}}) < Y\% , \text{ given } 2/3 \cdot \text{MCE}_{R} \text{ (or other specified hazard)}
  \]

- Open policy questions for developers of FR codes:
  - What is the acceptable or desirable FR time, \( \text{T}_{\text{FR, acceptable}} \), for a given occupancy?
  - What is the appropriate confidence level, \( Y \)?
  - What hazard level should be used for FR?
    - For this example, use \( 2/3 \cdot \text{MCE}_{R} \) (See Resource Paper 1 and Design Example 2.7 for discussion.)
The technical question

\[ P(T_{FR, \text{expected}} > T_{FR, \text{acceptable}}) < Y\%, \text{ given } 2/3 \times MCE_R \text{ (or other specified hazard)} \]

- For a given building, what is the expected functional recovery time, \( T_{FR, \text{expected}} \)?
  - The subject of ongoing research, using analysis and testing
  - Also answerable by judgment, experience (reconnaissance research), and, in the interim, with our established consensus procedures for developing codes and standards.

**Q: Can’t we ask a similar question about safety? How do we know a given design is “safe”?**

**A: Yes. Current design provisions in the NEHRP Provisions, ASCE standards, and building codes reflect engineering consensus applied to collected research, judgment, and experience. The same approach can be used to develop provisions for functional recovery design.**
Functional recovery and the current building code

- Risk Category IV (IBC Table 1604.5)
  - Used for “essential facilities” to preserve functionality after a design earthquake (NEHRP Provisions Section 1.1.5)
  - Could be used as interim FR criteria
- Two differences between FR and RC IV
  - RC IV presumes immediate performance; some FR objectives would allow time for repairs
  - FR provisions might cover externalities the current building code ignores

<table>
<thead>
<tr>
<th>Risk Category</th>
<th>Nature of Occupancy</th>
</tr>
</thead>
</table>
| I             | 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. |
| II            | Buildings and other structures except those listed in Risk Categories I, III and IV. |
| III           | Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300.  
- Buildings and other structures containing Group I-2 occupancies with an occupant load greater than 250.  
- Buildings and other structures containing educational occupancies for students above the 12th grade with an occupant load greater than 500.  
- Group I-2 occupancies with an occupant load of 50 or more resident care recipients but not having surgery or emergency treatment facilities.  
- Group I-3 occupancies.  
- Any other occupant with an occupant load greater than 5,000.²  
- Power-generating stations, water treatment facilities for public water, wastewater treatment facilities and other public utility facilities not included in Risk Category IV.  
- Buildings and other structures containing quantities of toxic or explosive materials that exceed the maximum allowable quantities per control area as given in Table 307.1(1) or 307.1(2) or per outdoor control area in accordance with the International Fire Code, and are sufficient to pose a threat to the public if released.²³ |
| IV            | Buildings and other structures designated as essential facilities, including but not limited to:  
- Group I-2 occupancies having surgery or emergency treatment facilities.  
- Fire, police, ambulance and police stations and emergency vehicle garages.  
- Designated earthquake, hurricane or other emergency shelters.  
- Designated emergency preparedness, communications and operations centers and other facilities required for emergency response.  
- Power-generating stations and other public utility facilities required as emergency backup facilities for Risk Category IV structures.  
- Buildings and other structures containing quantities of highly toxic materials that exceed the maximum allowable quantities per control area as given in Table 307.1(2) or per outdoor control area in accordance with the International Fire Code, and are sufficient to pose a threat to the public if released.²³  
- Aviation control towers, air traffic control centers and emergency aircraft hangars.  
- Buildings and other structures having critical national defense functions.  
- Water storage facilities and pump structures required to maintain water pressure for fire suppression. |
CLT Shear Wall Design Example (Chapter 6)

- 6-unit townhouse
  - Multi-family residential (R-2) occupancy
    - Occupant load less than 50
  - Risk Category II
  - Seismic Design Category D
- Similar structure could be used for:
  - Assisted living, or housing for other vulnerable tenants
  - Office suites
  - Mixed use or live/work units
Expected structural damage

- Low $R$-factor
- Inelasticity limited to replaceable ductile steel connectors
- CLT tests showed no hard-to-repair damage

Expected nonstructural damage

- Residential systems typically lighter, less fragile than in office or other occupancies
- For RC II buildings, current code exempts most components from anchorage because they pose no safety hazards
Functional recovery objective

- Presumed hazard: $2/3 \times \text{MCE}_R$
- What is an acceptable FR time ($T_{\text{FR, acceptable}}$)?
  - Or $desired$ FR time, absent code requirements
- What is the actual expected FR time ($T_{\text{FR, expected}}$)?
Policy precedents for acceptable FR time?
Policy precedents for acceptable FR time?

- NIST CRPG: 1 to 12 weeks for most housing, 3 days for vulnerable tenants
- SPUR, cited by San Francisco: Usable within a day of M7.2 event
- FEMA-NIST: "Days to weeks"
- ASCE 7: RC IV (immediate FR) should be considered where damage would cause “substantial economic impact” or “mass disruption” of normal community functions.
  - Does this apply to housing?
  - Consider pandemic lessons: is housing essential?
Functional recovery objective

- Presumed hazard: \( 2/3 \times MCE_R \)
- What is an acceptable FR time \((T_{FR, \text{acceptable}})\)?
  - A few weeks, at most a month?

- What is the actual expected FR time \((T_{FR, \text{expected}})\)?
Expected FR time: What does current research say?

- FEMA P-58 (2018)
  - 5 concrete and steel systems; nonstructural for office occupancy
  - 5- to 13-story model buildings
  - High seismicity sites
- Repair time after $2/3 \times MCE_R$ event
  - 15 – 81 days
  - Does not include time for permitting, financing, mobilization, etc.
  - 12 – 33 days even if designed as RC IV
- FR time can be less than repair time
Expected FR time: What does current research say?

- Haselton et al. (2021)
  - 5 woodframe residential model buildings (not CLT); nonstructural typical for woodframe residential occupancy
  - High seismicity site
- FR time after $2/3 \times MCE_R$ event
  - 3-story building: 1 – 6 months
  - Includes time for permitting, financing, mobilization, etc.

<table>
<thead>
<tr>
<th>Building Type</th>
<th>Los Angeles Site (High-Seismicity)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Reoccup.</td>
</tr>
<tr>
<td>1-story (single-family)</td>
<td>0-1 wks.</td>
</tr>
<tr>
<td>2-story (single-family)</td>
<td>1-4 wks.</td>
</tr>
<tr>
<td>3-story (12-plex)</td>
<td>1-4 wks.</td>
</tr>
<tr>
<td>4-story (apartments)</td>
<td>1-4 wks.</td>
</tr>
<tr>
<td>5-story (apartments)</td>
<td>1-4 wks.</td>
</tr>
<tr>
<td>5-story on podium (apt.)</td>
<td>1-4 wks.</td>
</tr>
</tbody>
</table>
Expected FR time: What does current research say?

- Furley et al. (2021)
  - 2-story office building
  - Post-tensioned CLT rocking walls w/ UFP hysteretic dissipators; nonstructural systems for office occupancy
- FR time after $2/3 \times \text{MCE}_R$ event ($S_{DS} = 1.0g$)
  - ~135 days, w/ 10% probability of exceedance
    - But also ~95 days for just reoccupancy
  - Includes time for permitting, financing, mobilization, etc.
  - FR time driven by nonstructural damage.
Expected FR time: What does current research say?

- **Summary:** Expected FR time after $2/3\times MCE_R$ event
  - At least a few weeks, perhaps a few months
  - FR time for different seismic force-resisting systems varies widely
  - Effect of nonstructural damage on FR time can be substantial
  - Using RC IV criteria helps, but to an unknown degree

- **Work to establish reliable predictive tool continues**
  - Academia (PEER, CRCRP, etc.)
  - Government agencies (FEMA, NIST, etc., w/ ATC, etc.)
  - Professional associations (SEAOC, etc.)
  - Private sector
Functional recovery objective

- Presumed hazard: $2/3 \times MCE_R$
- What is an acceptable FR time ($T_{FR, \text{acceptable}}$)?
  - A few weeks, at most a month?
- What is the actual expected FR time ($T_{FR, \text{expected}}$)?
  - Several months?

Q: How to address this discrepancy?
A: Review CLT design criteria.
CLT Shear Wall structural design criteria

- Generally low damage already expected, so limited opportunities for improvement
- System selection is important; “Low damage design” beneficial for fast FR

Earthquake Protection Systems

Restrepo, in Filiatrault, 2004

Hogg, 2013
CLT Shear Wall structural design criteria

- **Seismic importance factor, $I_e$**
  - A tool provided by the code, usually taken as 1.0 for residential occupancy
  - Higher value can be used, but full RC IV performance requires more than $I_e > 1.0$

- **Height limit for CLT shear wall systems**
  - Design example $H = 30'$, already well under 65-ft code limit

- **Response modification coefficient, $R$**
  - Already limited to $R = 3$, a relatively low value that already limits expected inelasticity

- **CLT material grade**
  - Not likely to affect performance, since design is controlled by strength of connectors
CLT Shear Wall structural design criteria

- CLT partition classification
  - Unintended partition strength and stiffness beneficial, but difficult to codify
  - More effective to focus on intended shear walls
- Steel connector capacity
  - Lower presumed (or prescribed) capacity would reduce damage
  - But artificially low presumed capacity could interfere with test-validated design procedure
- Drift limit
  - Unlikely to affect performance, since predicted drift is already well under current limit
CLT Shear Wall structural design criteria

- Hold-down deformation limit
  - Unlikely to affect performance, since predicted elongation is already well under current limit

- Hold-down design force
  - Unlikely to affect performance, since design force is set only to ensure yielding in steel connectors

- CLT panel aspect ratio
  - Unlikely to affect performance, since current design already uses low $R$ value
  - For other designs, prohibiting higher $R$ value for high aspect ratio panels could reduce inelasticity demands and damage
Townhouse nonstructural design criteria

- Not addressed in CLT Design Example
  - But expected to have significant effect on building FR time
  - Even more significant for a low-damage structural system like CLT shear walls
- Current code for RC II buildings
  - Functionality considered for life safety systems (alarms, exit lighting, sprinklers)
  - Other components exempt from protection because they pose no safety hazards

<table>
<thead>
<tr>
<th>TABLE 1004.5</th>
<th>RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>RISK CATEGORY</strong></td>
<td><strong>NATURE OF OCCUPANCY</strong></td>
</tr>
<tr>
<td>I</td>
<td>Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to:</td>
</tr>
<tr>
<td></td>
<td>Agricultural facilities.</td>
</tr>
<tr>
<td></td>
<td>Certain temporary facilities.</td>
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<td></td>
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<td>Water storage facilities and pump structures required to maintain water pressure for fire suppression.</td>
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Townhouse nonstructural design criteria

- Current code for RC IV buildings
  - Broader scope of bracing, anchorage
  - Importance factor $I_p = 1.5$
  - Backup utility service
- 2020 *NEHRP Provisions* Section 1.1.5
  - Eight characteristics of expected RC IV performance
  - Focus on “essential functions”
  - Useful reference for voluntary FR improvement

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</table>
Characteristics of RC IV functionality (*NEHRP Provisions* Section 1.1.5)

- **Immediate reoccupancy**
  - Largely a function of structural performance; see above.
  - Also a function of fire safety and hazmat protection, already provided by code for RC II
- **Functional equipment for “essential functions”**
  - For residential building, housing code habitability standards are useful reference
    - Light, ventilation, power, potable water, heat in winter, sanitation, cooking and food storage
    - For some tenants, elevators and communications systems
  - Difference between FR and RC IV: FR might not require these to be available immediately
    - Some of these are even waived after large events to facilitate basic reoccupancy
Characteristics of RC IV functionality (*NEHRP Provisions* Section 1.1.5)

- Limited damage to contents for “essential functions”
  - Contents are not usually considered by building code, but can be important for FR
  - For residential building, could include kitchen appliances, etc.
- For non-essential equipment and contents, no damage affecting “essential functions”
  - Might include extensive damage to architectural components (glass, ceilings)
  - Often repairable (or removable) within desired FR time
- Building envelope “maintains integrity ... To preserve essential functions.”
  - Mostly already covered by code for RC II (glazing, cladding, roofing)
  - Repair can often be done from exterior with limited effect on functional recovery
Characteristics of RC IV functionality (*NEHRP Provisions* Section 1.1.5)

- Only “minor leakage” in “piping carrying nontoxic substances”
  - Intent seems clear, but subject to broad interpretation
- “Controlled” release of toxic substances
  - Intent seems clear, but subject to broad interpretation
  - Unlikely to be an issue in new residential buildings
- Egress “maintained”
  - Needed for basic reoccupancy as well
  - Mostly covered by code for RC II (drift limits, protection from falling hazards, lighting)
  - Could apply to special accessibility features or equipment
Voluntary FR and emerging best practices

- Basic strategies for improving FR time
  - SFRS selection: Low-damage design
  - Drift limits
  - Nonstructural & contents scope
  - Quality assurance
  - Planning strategies

- Needed tools
  - Consensus design criteria
  - Design and analysis software
## Voluntary FR and emerging best practices

<table>
<thead>
<tr>
<th>Project</th>
<th>Building Use</th>
<th>Functional Recovery Objective or Expectation</th>
<th>Recovery-focused Design Features or Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>181 Fremont (Almufti et al., 2016)</td>
<td>Office high-rise</td>
<td>Within weeks after design EQ</td>
<td>Reinforced concrete core, designed w/ ARUP’s REDi criteria</td>
</tr>
<tr>
<td>Beaverton, OR schools (SEFT, 2015)</td>
<td>Public schools</td>
<td>RC IV performance, to suit services as post-EQ shelter</td>
<td>RC IV criteria, backup generator</td>
</tr>
<tr>
<td>UCSF Mission Hall (Bade, 2014)</td>
<td>University offices</td>
<td>Operational performance after 84th percentile Hayward event</td>
<td>Enhanced RC II criteria, concrete shear walls</td>
</tr>
<tr>
<td>Casa Adelante (Mar, 2021)</td>
<td>Senior housing</td>
<td>Within 1 day after 475-year event, no tenant relocation</td>
<td>Rocking walls, dampers</td>
</tr>
<tr>
<td>85 Bluxome (Moore, 2021)</td>
<td>Office mid-rise</td>
<td>Within “days to weeks” after “major EQ”</td>
<td>Tight drift limits (zero lot lines), SidePlate moment-resisting frame</td>
</tr>
</tbody>
</table>
## Voluntary FR and emerging best practices

<table>
<thead>
<tr>
<th>Project</th>
<th>Building Use</th>
<th>Functional Recovery Objective or Expectation</th>
<th>Recovery-focused Design Features or Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>UCSF Center for Vision Neuroscience</td>
<td>University research</td>
<td>Within 60 days after M7 San Andreas event</td>
<td>1.25 importance factor, 1.5% allowable drift</td>
</tr>
<tr>
<td>(Berkowitz, 2021)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oregon Treasury</td>
<td>Gov’t offices</td>
<td>Within 0 days after MCE$_R$</td>
<td>Base isolation, minimized nonstructural components</td>
</tr>
<tr>
<td>(Zimmerman, 2021)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stanford Biomedical Innovations</td>
<td>University research</td>
<td>Within 26 days after 475-year event</td>
<td>Modified RC III criteria, element-specific $R$ values, 1.5 importance factor</td>
</tr>
<tr>
<td>(Lizundia, 2021)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Allenby Building</td>
<td>Gov’t offices</td>
<td>Within 0 days after 475-year event</td>
<td>Reduced drift limits, amplified demand, post-EQ recovery plan</td>
</tr>
<tr>
<td>(Westermeyer, 2021)</td>
<td></td>
<td></td>
<td></td>
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</tbody>
</table>
References


References


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Chapter 3 (Section 3.2 - Part 1)
The 2018 Update of the USGS National Seismic Hazard Model

2020 NEHRP Provisions Training Materials
Sanaz Rezaeian, Ph.D., USGS
Outline

1. Interplay between the USGS hazard models and the BSSC PUC requirements

2. The 2018 USGS National Seismic Hazard Model (NSHM) for Conterminous U.S.
   - Ground motion models in CEUS (e.g. NGA-East)
   - Deep basin effects in WUS

3. Outside of the Conterminous U.S. (HI, AK, PRVI, GNMI, AMSAM)

“Design” Ground Motions:
- USGS: probabilistic
- BSSC: + risk targeted
  - + site amplifications
- PUC: + deterministic caps
  - + max direction

\( \text{MCE}_R \)
USGS NSHMs & BSSC PUC Requirements

Hazard Model (PSHA)  Site-specific Procedures of Ch21

<table>
<thead>
<tr>
<th>USGS NSHM</th>
<th>NEHRP Provisions</th>
<th>ASCE 7 Standards</th>
<th>IBC</th>
</tr>
</thead>
<tbody>
<tr>
<td>2014</td>
<td>2015</td>
<td>2016</td>
<td>2018</td>
</tr>
<tr>
<td>2018</td>
<td>2020</td>
<td>2022*</td>
<td>TBD</td>
</tr>
</tbody>
</table>

PGA, 0.2, 1s 760m/s
22 Periods 8 Vs30s

Hazard Curves: (RiskTarget, MaxDir, SiteAmpl, DetCaps) → “Design” Ground Motions
Updates to 2020 NEHRP Design Ground Motions in Conterminous US

2018 USGS NSHM

Updated hazard model (eqk sources, GMMs)

Updated site-specific procedures of Ch21

BSSC Project ‘17


Updated site-specific procedures of Ch21
Updates to 2020 NEHRP Design Ground Motions in Conterminous US

2018 USGS NSHM

- The 2018 update of the US National Seismic Hazard Model: Overview of model and implications
- Multi-period multi-Vs30 response spectrum (MPRS)
- Modifying deterministic caps based on deaggregation of probabilistic hazard
- Updating the max-direction factors

BSSC Project ‘17

- No change to risk-targeted calcs
- Using multi-period multi-Vs30 response spectrum (MPRS)
- Modifying deterministic caps based on deaggregation of probabilistic hazard
- Updating the max-direction factors

MPRS issue directly influenced the 2018 update of USGS NSHM (GMMs applicable for all periods and site classes)
Updates to 2020 NEHRP Design Ground Motions in Conterminous US

2018 USGS NSHM

1. New ground motion models (GMMs), including **NGA-East**, & amplification factors in the Central & Eastern US (CEUS)
2. Deep **basin effects** in Los Angeles, Seattle, San Francisco, and Salt Lake City regions
3. Minor modifications of GMMs (crustal & subduction) in the Western US (WUS)
4. Updating **background seismicity** to include 2013-2017 earthquakes

BSSC Project ‘17

No change to risk-targeted calcs

1. Using **multi-period multi-Vs30 response spectrum** (MPRS)
2. Modifying **deterministic caps** based on deaggregation of probabilistic hazard
3. Updating the **max-direction** factors

**MPRS issue directly influenced the 2018 update of USGS NSHM** (GMMs applicable for all periods and site classes)
Old CEUS Ground Motion Models

Table from Rezaeian et al. (2021):

<table>
<thead>
<tr>
<th>2014 CEUS GMMs:</th>
<th>Period Range</th>
<th>Site Classes</th>
</tr>
</thead>
<tbody>
<tr>
<td>AB06’</td>
<td>PGA to 5 s</td>
<td>A, BC (A to E)</td>
</tr>
<tr>
<td>A08’</td>
<td>PGA to 5 s</td>
<td>A, BC (A to E)</td>
</tr>
<tr>
<td>C03</td>
<td>PGA to 2 s (4 s)</td>
<td>A, BC*</td>
</tr>
<tr>
<td>F96</td>
<td>PGA to 2 s</td>
<td>A, BC</td>
</tr>
<tr>
<td>P11</td>
<td>PGA to 5 s (10 s)</td>
<td>A, BC*</td>
</tr>
<tr>
<td>S02</td>
<td>PGA to 5 s (10 s)</td>
<td>A, BC*</td>
</tr>
<tr>
<td>S01</td>
<td>PGA to 2 s (4 s)</td>
<td>A, BC*</td>
</tr>
<tr>
<td>TP05</td>
<td>PGA to 4 s</td>
<td>A, BC*</td>
</tr>
<tr>
<td>T02</td>
<td>PGA to 2 s</td>
<td>A, BC*</td>
</tr>
</tbody>
</table>

Parentheses indicate the published range when a different range is supported in the USGS codes.
*Through conversion factors.

New CEUS Ground Motion Models

14 Updated Seed GMMs
from 19 published plus 2 new
varying weights based on
geometric spreading & model type
(1/3 weight)

17 NGA-East GMMs
Sammon’s Mapping
varying weights based on
frequency & magnitude
(2/3 weight)

31 CEUS GMMs

Changes made to:
1. Median ground motions
   (increases for large M, middle to large distances)
2. Epistemic uncertainty (increased)
3. Aleatory uncertainty (minor)
New CEUS Ground Motion Models

14 Updated Seed GMMs:

- R-1 Geometric Spreading (Point-Source, Empirical-Factors)
- R-1.3 Geometric Spreading (Hybrid, Stochastic-Equivalent Point-Source)
- Other Geometric Spreading (Simulation-based, Reference-Empirical)

17 NGA-East GMMs:

New CEUS Site-Effects Models

CEUS has very different spectral shapes compared to WUS, as expected!

This is the first time that site-effects specific to the CEUS have been implemented in the NSHMs (prior NEHRP coefficients were based on WUS)

Hazard Changes (CEUS)

Ratio Maps (2018/2014):
2% in 50yr uniform hazard, BC site class (760 m/s)

- **Medians:** more significant increases for large M at mid-large distances
- **Epistemic uncertainty:** increased significantly for large M, more around 70-100 km
- **Aleatory uncertainty:** minor changes
- **Site-effect model:** only $F_{760}$ in this figure
- **Seismicity catalog updates:** outside CA, mostly affecting intermountain west region

Deep Basin Effects

Categorized by:
basin depth terms $Z_{1.0}$ & $Z_{2.5}$

Within basins:
measurements only in deep portions of basins are used, “default” values are used in shallow depths

Outside basins:
“default” values are used

Deep Basin Effects

Minor modifications made to crustal and subduction models. Basin effects fully applied at periods above 1 sec:

Implementation of Crustal Earthquake GMMs:

Modifications to Subduction Earthquake GMMs:

Hazard Changes (WUS)

Ratio Maps (2018 local basin depth/2018 default basin depth):
2% in 50yr uniform hazard, 5 sec, Site Class D (260 m/s)

Disclaimer: This information is preliminary and is subject to revision. It is being provided to meet the need for timely best science. The information is provided on the condition that neither the U.S. Geological Survey nor the U.S. Government shall be held liable for any damages resulting from the authorized or unauthorized use of the information.
Developed Generic Spectral Shapes:
FEMA/ATC report, approved by BSSC PUC.
Shapes developed based on WUS data, function($S_s$, $S_s/S_1$, $T_L$)

Figure B-17. Plots of probabilistic response spectrum shape parameters (RSSPs) by site class for Table B-17. GTL12S3R2.

Figure citation: Kircher C, Rezaeian S, Luco N - FEMA P-2078 (2020), Procedures for Developing Multi-Period Response Spectra of Non-Conterminous United States Sites, FEMA P-2078, prepared by ATC for FEMA, Washington, D.C.
Outside of Conterminous US (OCONUS)

Solid Lines: Predicted values from $S_s$ & $S_1$

Dashed Lines: Exact values calculated for 2020 NEHRP

Irvine, CA (validation)

Honolulu, HI (prediction) matching to updated 1998 $S_s$ & $S_1$

Figure citation: Kircher C, Rezaeian S, Luco N – FEMA P-2078 (2020), Procedures for Developing Multi-Period Response Spectra of Non-Conterminous United States Sites, FEMA P-2078, prepared by ATC for FEMA, Washington, D.C.
Summary

- The Multi-Period-Response-Spectra requirement of the BSSC PUC influenced the 2018 update of USGS NSHM because GMMs needed to be applicable for 22 periods and 8 site classes.

- The 2018 USGS NSHM updates included: (1) new GMMs in CEUS (14 updated seeds + 17 NGA-East + new site-effects model), (2) incorporation of deep basin effects in WUS, (3) removal of one crustal and one subduction GMM and minor modifications in WUS, and (4) update of seismicity catalog.

1. Petersen et al. (Feb 2020), Earthquake Spectra (Overview)
2. Petersen et al. (2021), Earthquake Spectra (sensitivity analysis)
3. Shumway et al. (2021), Earthquake Spectra (data paper on added Ts and Vs30s)
4. Rezaeian et al. (2021), Earthquake Spectra (CEUS GMM details)
5. Powers et al. (2021), Earthquake Spectra (WUS GMM and basin effect details)

- Generic spectral shapes used for OCONUS locations in 2020 NEHRP Provisions (FEMA P-2078 / ATC 136)
Questions
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Chapter 3 (Section 3.2 - Part 2) 
Dissection of Example Changes to the $MCE_R$ Ground Motion Values 

2020 NEHRP Provisions Training Materials 
Nicolas Luco, Ph.D., U.S. Geological Survey
Commentary to Chapter 22

- Modifications to $\text{MCE}_R$ and $\text{MCE}_G$ ground motions from Project '17 recommendations
- Modifications to $\text{MCE}_R$ and $\text{MCE}_G$ ground motions from 2018 USGS NSHM update
- Examples of changes in $\text{MCE}_R$ and $\text{MCE}_G$ values
- RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE ($\text{MCE}_R$) SPECTRAL RESPONSE ACCELERATIONS
- MAXIMUM CONSIDERED EARTHQUAKE GEOMETRIC MEAN ($\text{MCE}_G$) PEAK GROUND ACCELERATIONS
- LONG-PERIOD TRANSITION MAPS
- USGS SEISMIC DESIGN GEODATABASE AND WEB SERVICE
USGS 2018 National Seismic Hazard Model (NSHM) Updates

Incorporation of ...

1) the NGA-East ground-motion models
2) deep sedimentary basin effects in the Los Angeles, Seattle, San Francisco, and Salt Lake City regions
3) earthquakes that occurred in 2013 through 2017
4) updated weighting of the western U.S. ground-motion models
BSSC Project ‘17 Recommendations

Modifications to ...

1) site-class effects
2) spectral periods that define the $S_{MS}$ & $S_{M1}$ ground-motion parameters
3) deterministic caps on the otherwise probabilistic ground motions
4) maximum-direction scale factors
Maximum-Direction Scale Factors

RESOURCE PAPER 4
UPDATED MAXIMUM-RESPONSE SCALE FACTORS

RP4-1 UPDATED MAXIMUM-RESPONSE SCALE FACTORS

The proposed changes below update the “maximum-response scale factors” specified in the site-specific ground motion procedures (Chapter 21) of ASCE/SEI 7-10. These factors increase spectral response accelerations that represent the geometric mean (or a similar metric) of two horizontal ground motion components, such that they represent the maximum response in the horizontal plane. Recall that ASCE/SEI 7-10, via both Chapter 21 and the MCE_R ground motion maps, specifies maximum-response spectral response accelerations. Typical ground motion attenuation relations, including those applied by the USGS in preparing the MCE_R ground motion maps, provide geometric-mean spectral response accelerations.
Maximum-Direction Scale Factors

21.2.2 Deterministic (MCE$_R$) Ground Motions

The deterministic spectral response acceleration at each period shall be calculated as an 84th-percentile 5% damped spectral response acceleration in the direction of maximum horizontal response computed at that period. The largest such acceleration calculated for scenario earthquakes on all known faults within the region shall be used. The scenario earthquakes shall be determined from deaggregation for the probabilistic spectral response acceleration at each period. Scenario earthquakes contributing less than 10% of the largest contributor at each period shall be ignored.
Table C21.2.2-1 Examples of scenario earthquake from hazard deaggregations at a site in San Jose, California

<table>
<thead>
<tr>
<th>Period $T$ (s)</th>
<th>Scenario Earthquake</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hayward</td>
</tr>
<tr>
<td></td>
<td>$M$</td>
</tr>
<tr>
<td>0.20</td>
<td>7.0</td>
</tr>
<tr>
<td>0.25</td>
<td>7.0</td>
</tr>
<tr>
<td>0.30</td>
<td>7.0</td>
</tr>
<tr>
<td>0.40</td>
<td>7.0</td>
</tr>
<tr>
<td>0.50</td>
<td>7.0</td>
</tr>
<tr>
<td>0.75</td>
<td>7.1</td>
</tr>
<tr>
<td>1.0</td>
<td>7.1</td>
</tr>
</tbody>
</table>
Commentary to Chapter 22

- Modifications to $\text{MCE}_R$ and $\text{MCE}_G$ ground motions from Project ’17 recommendations
- Modifications to $\text{MCE}_R$ and $\text{MCE}_G$ ground motions from 2018 USGS NSHM update
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- MAXIMUM CONSIDERED EARTHQUAKE GEOMETRIC MEAN ($\text{MCE}_G$) PEAK GROUND ACCELERATIONS
- LONG-PERIOD TRANSITION MAPS
- USGS SEISMIC DESIGN GEODATABASE AND WEB SERVICE
Examples of Changes in MCE\textsubscript{R} Values

Table C11.4-1 Thirty-Four Cities, Site Locations (Latitude and Longitude), and Associated Counties and Populations At Risk for Which Values of Ground Motions Are Provided

<table>
<thead>
<tr>
<th>Region</th>
<th>City and Location of Site</th>
<th>Latitude</th>
<th>Longitude</th>
<th>County or Metropolitan Statistical Area</th>
<th>Name</th>
<th>Population</th>
</tr>
</thead>
<tbody>
<tr>
<td>Southern California</td>
<td>Los Angeles</td>
<td>34.05</td>
<td>-118.25</td>
<td>Los Angeles</td>
<td></td>
<td>9,948,081</td>
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<tr>
<td></td>
<td>Century City</td>
<td>34.05</td>
<td>-118.40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Northridge</td>
<td>34.20</td>
<td>-118.55</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Long Beach</td>
<td>33.80</td>
<td>-118.20</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Irvine</td>
<td>33.65</td>
<td>-117.80</td>
<td>Orange</td>
<td></td>
<td>3,002,048</td>
</tr>
<tr>
<td></td>
<td>Riverside</td>
<td>33.95</td>
<td>-117.40</td>
<td>Riverside</td>
<td></td>
<td>2,026,803</td>
</tr>
<tr>
<td></td>
<td>San Bernardino</td>
<td>34.10</td>
<td>-117.30</td>
<td>San Bernardino</td>
<td></td>
<td>1,999,332</td>
</tr>
<tr>
<td></td>
<td>San Luis Obispo</td>
<td>35.30</td>
<td>-120.65</td>
<td>San Luis Obispo</td>
<td></td>
<td>257,005</td>
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<td></td>
<td>San Diego</td>
<td>32.70</td>
<td>-117.15</td>
<td>San Diego</td>
<td></td>
<td>2,941,454</td>
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<td></td>
<td>Santa Barbara</td>
<td>34.45</td>
<td>-119.70</td>
<td>Santa Barbara</td>
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<td>400,335</td>
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<td>Ventura</td>
<td>34.30</td>
<td>-119.30</td>
<td>Ventura</td>
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<td>799,720</td>
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<td></td>
<td><strong>Total Population - S. California</strong></td>
<td><strong>22,349,098</strong></td>
<td><strong>Population - 8 Counties</strong></td>
<td><strong>21,374,778</strong></td>
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</table>

<table>
<thead>
<tr>
<th>Region</th>
<th>City and Location of Site</th>
<th>Latitude</th>
<th>Longitude</th>
<th>County or Metropolitan Statistical Area</th>
<th>Name</th>
<th>Population</th>
</tr>
</thead>
<tbody>
<tr>
<td>Northern California</td>
<td>Oakland</td>
<td>37.80</td>
<td>-122.25</td>
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<td></td>
<td>Concord</td>
<td>37.95</td>
<td>-122.00</td>
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<td></td>
<td>Monterey</td>
<td>36.60</td>
<td>-121.90</td>
<td></td>
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<tr>
<td></td>
<td>Sacramento</td>
<td>38.60</td>
<td>-121.50</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>San Francisco</td>
<td>37.75</td>
<td>-122.40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>San Mateo</td>
<td>37.56</td>
<td>-122.30</td>
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</tr>
<tr>
<td></td>
<td>San Jose</td>
<td>37.35</td>
<td>-121.90</td>
<td></td>
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</tr>
<tr>
<td></td>
<td>Santa Cruz</td>
<td>36.95</td>
<td>-122.05</td>
<td></td>
<td></td>
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<td></td>
<td>Vallejo</td>
<td>38.10</td>
<td>-122.25</td>
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<td></td>
<td>Santa Rosa</td>
<td>36.45</td>
<td>-122.70</td>
<td></td>
<td></td>
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<tr>
<td>Pacific Northwest</td>
<td>Seattle</td>
<td>47.60</td>
<td>-122.30</td>
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<td></td>
<td>Tacoma</td>
<td>47.25</td>
<td>-122.45</td>
<td></td>
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<tr>
<td></td>
<td>Everett</td>
<td>48.00</td>
<td>-122.20</td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>Portland</td>
<td>45.50</td>
<td>-122.65</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td><strong>Total Population - OR and WA</strong></td>
<td><strong>10,096,569</strong></td>
<td><strong>Population - 8 Counties</strong></td>
<td><strong>10,096,569</strong></td>
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<tr>
<td>Other WUS</td>
<td>Salt Lake City</td>
<td>40.75</td>
<td>-111.90</td>
<td></td>
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<tr>
<td></td>
<td>Boise</td>
<td>43.60</td>
<td>-116.20</td>
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<tr>
<td></td>
<td>Reno</td>
<td>39.55</td>
<td>-119.80</td>
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<tr>
<td></td>
<td>Las Vegas</td>
<td>36.20</td>
<td>-115.15</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>Total Population - ID,UT,NV</strong></td>
<td><strong>6,512,057</strong></td>
<td><strong>Population - 8 Counties</strong></td>
<td><strong>6,512,057</strong></td>
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<tr>
<td>CEUS</td>
<td>St. Louis</td>
<td>38.60</td>
<td>-90.20</td>
<td></td>
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<tr>
<td></td>
<td>Memphis</td>
<td>35.15</td>
<td>-90.05</td>
<td></td>
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<tr>
<td></td>
<td>Charleston</td>
<td>32.80</td>
<td>-79.95</td>
<td></td>
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<td></td>
<td>Chicago</td>
<td>41.85</td>
<td>-87.65</td>
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<td><strong>Total Population - MO,TN,SC,IL,NY</strong></td>
<td><strong>48,340,919</strong></td>
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Examples of Changes in $MCE_R$ Values

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<th>$S_S$ (g)</th>
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<td>2.42</td>
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</table>
Examples of Changes in $MCE_R$ Values

Changes >15% at 20 of 34 locations.

Vallejo: +34% mostly due to deterministic caps

Sacramento: +28% mostly due to site class effects

New York: -33% mostly due to NGA-East & site class effects

Examples of Changes in $MCE_R$ Values

With 1.5 multiplier of Section 11.4.8 exception, changes >15% at 31 of 34 locations.

San Mateo & San Bernardino: mostly due to spectral periods that define $S_{M1}$

Vallejo: mostly due to deterministic & basin effects

8 locations: mostly due to site class effects

Examples of Changes in SDC

From ASCE 7-10 to ASCE 7-16, SDC decreases at 2 of 34 locations, from E to D.

From ASCE 7-16 to 2020 Provisions, SDC increases at 4 of 34 locations, from D to E, mostly due to deterministic capping and basin effects.
Examples of Changes in SDC

Disclaimer: This information is preliminary and is subject to revision. It is being provided to meet the need for timely best science. The information is provided on the condition that neither the U.S. Geological Survey nor the U.S. Government shall be held liable for any damages resulting from the authorized or unauthorized use of the information.
Summary of Changes in $MCE_R$ Values

For default site conditions ...

- $S_{MS}$ changes by less than 15% at 31 of the 34 locations;
- $S_{M1}$ changes by less than 15% at 23 of the 34 locations;
- SDC changes at 4 of the 34 locations, from SDC D to E;
- Most of these changes are due to the Project ’17 modifications to site-class effects or deterministic caps, but some are caused by the other Project ’17 and 2018 NSHM updates, particularly the 2018 NSHM incorporation of basin effects.

Changes for other site classes at other locations can be probed using the USGS Seismic Design Web Services and BSSC Tool for Seismic Design Map Values.
Commentary to Chapter 22

- Modifications to $MCE_R$ and $MCE_G$ ground motions from Project ’17 recommendations
- Modifications to $MCE_R$ and $MCE_G$ ground motions from 2018 USGS NSHM update
- Examples of changes in $MCE_R$ and $MCE_G$ values
- RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE ($MCE_R$) SPECTRAL RESPONSE ACCELERATIONS
- MAXIMUM CONSIDERED EARTHQUAKE GEOMETRIC MEAN ($MCE_G$) PEAK GROUND ACCELERATIONS
- LONG-PERIOD TRANSITION MAPS
- USGS SEISMIC DESIGN GEODATABASE AND WEB SERVICE
USGS Seismic Design Geodatabase

Gridded earthquake ground motions for the 2020 NEHRP Recommended Seismic Provisions and 2022 ASCE/SEI 7 Standard
### Conterminous United States

**Attached Files**

Click on title to download individual files attached to this item or **download all** files listed below as a compressed file.

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<td>2020-05-11</td>
<td>14.91 KB</td>
<td><a href="mailto:rukstales@usgs.gov">rukstales@usgs.gov</a></td>
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</table>
NEHRP-2020 Web Service Documentation

**latitude**
Latitude of site of interest, in decimal degrees
Example: 34.05

**longitude**
Longitude of site of interest, in decimal degrees
Example: -118.25

**siteClass**
Site Class, as defined in Chapter 20
Options: Default, A, B, BC, C, CD, D, DE, E
USGS Seismic Design Web Service

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BSSC Tool for Seismic Design Map Values

BSSC Tool For 2020 NEHRP Provisions Seismic Design Map Values

Version: Beta
BSSC Tool for Seismic Design Map Values

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**Multi-Period Design Spectrum**
USGS Earthquake Hazards

2020 NEHRP Provisions (NEHRP-2020)

**Web Interface:** BSSC Tool for 2020 NEHRP Provisions Seismic Design Maps Values

**Web Service (source of data for Web Interface):** "USGS Seismic Design Web Service" for NEHRP-2020

**Maps (in document):** See 2020 NEHRP Recommended Seismic Provisions for New Buildings and Other Structures

**Maps (online only):** USGS Online-only maps referenced by the 2020 NEHRP Recommended Seismic Provisions and 2022 ASCE/SEI 7 Standard ([preview one example](https://sites.google.com/site/seismicdesignmaps/))

**Data:** "USGS Seismic Design Geodatabase" for NEHRP-2020 (currently requires [sign up](https://doi.org/10.5066/F7NK3C76))
DISCLAIMER

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Design (As Usual) Using New MPRS

- **Design Procedures**
  - ELF procedures (Chapter 12) are not affected by proposed changes (although values of design parameters, $S_{DS}$ and $S_{D1}$, would better match the underlying response spectrum of the site of interest)
  - MRSA procedures (Chapter 12) are not affected by proposed changes (although multi-period design spectra would provide a more reliable calculation of dynamic response)

- **Design Ground Motions**
  - Ground motion parameters (and MPRS) are available online from a USGS web service [https://doi.org/10.5066/F7NK3C76] for user specified site location (i.e., latitude and longitude) and site conditions (i.e., site class)
  - Site-specific ground motion procedures (Chapter 21) now permit use of MPRS obtained online from the USGS web service (in lieu of a hazard analysis)
New Multi-Period Response Spectra (MPRS)

- Collectively improve the accuracy of the frequency content of earthquake design ground motions
- Enhance the reliability of the seismic design parameters derived from these ground motions
- Make better use of the available earth science (including the 2018 update of the USGS NSHM) which has, in general, sufficiently advanced to accurately define spectral response for different site conditions over a broad range of periods
- Eliminate the need for site-specific hazard analysis required by ASCE/SEI 7-16 (2015 NEHRP Provisions) for certain (soft soil) sites where the site coefficients are either undefined or inadequate
- Do no change the ELF (MRSA) design procedures commonly used by most design engineers and projects
Summary of MPRS and Related Changes (to ASCE/SEI 7-16)

- Chapter 11 – Seismic Ground Motion Values
  - Added new “site-specific” multi-period design spectra and related values of seismic design parameters (e.g., $S_{MS}$, $S_{M1}$ and $PGA_M$) of the “USGS Seismic Design Geodatabase”, available online from a USGS web service for user-defined site location and site conditions (i.e., site class)
  - Deleted site coefficient tables (i.e., site factors are no longer required)
  - Removed the site-specific (interim solution) ground motion procedures of ASCE/SEI 7-16

- Chapter 20 – Site Classification Procedure for Seismic Design
  - Added three new site classes (Site Classes BC, CD and DE) to Table 20.3-1
  - Added new site class shear wave velocity-based requirements
Summary of MPRS and Related Changes (to ASCE/SEI 7-16)

- Chapter 21 – Site-specific Ground Motion Procedures for Seismic Design
  - Added new deterministic $M_{CE_R}$ “scenario” earthquake requirements (based on de-aggregation)
  - Revised determination of $S_{D1}$ from site-specific design spectrum (Section 21.4)

- Chapter 22 – Seismic Ground Motion and Long-Period Period Maps
  - Incorporated USGS update of $M_{CE_R}$ ground motions based on 2018 update of the USGS NSHM
  - Updated to provide new maps of $S_{MS}$ and $S_{M1}$ (and $PGA_M$) for “default” site conditions
Two-Period Design Response Spectrum (Multi-Period Design Spectrum)
(Figure 11.4-1, ASCE/SEI 7-05, ASCE/SEI 7-10 and ASCE/SEI 7-16 with annotation)

- $S_{DS} = \frac{2}{3} \times S_{MS} = \frac{2}{3} \times F_a \times S_s$
- $S_{D1} = \frac{2}{3} \times S_{M1} = \frac{2}{3} \times F_v \times S_1$

Site-Specific Multi-Period Response Spectrum

- $C_s = \frac{S_{DS}}{(R/I_e)}$
  - $T \leq T_s$

- $C_s = \frac{S_{D1}}{T(R/I_e)}$
  - $T_s < T \leq T_L$

- $T_s = \frac{S_{D1}}{S_{DS}}$
The “Problem” with ASCE/SEI 7-10

- For softer sites, in particular those where seismic hazard is governed by large magnitude earthquakes:
  - Frequency content of ground motions (spectrum shape) is not accurately characterized by of the two-period design response spectrum and site coefficients
  - Design ground motions are significantly underestimated (e.g., by as much as a factor of 2 at longer response periods)
Comparison of ASCE/SEI 7-16 Two-Period (ELF) Design Spectrum w/o Spectrum Shape Adjustment and Multi-Period Response Spectra based on M7.0 earthquake ground motions at $R_x = 6.8$ km – Site Class C

ELF Design Spectrum

$$S_s = 1.5$$
$$F_a = 1.2$$
$$S_{MS} = F_a \times S_s = 1.8$$
$$S_{DS} = 2/3 \times S_{MS} = 1.2$$

$$S_1 = 0.6$$
$$F_v = 1.4$$
$$S_{M1} = F_v \times S_1 = 0.84$$
$$S_{D1} = 2/3 \times S_{M1} = 0.56$$
Comparison of ASCE/SEI 7-16 Two-Period (ELF) Design Spectrum w/o Spectrum Shape Adjustment and Multi-Period Response Spectra based on M7.0 earthquake ground motions at $R_x = 6.8$ km) – Site Class D
Comparison of ASCE/SEI 7-16 Two-Period (ELF) Design Spectrum w/o Spectrum Shape Adjustment and Multi-Period Response Spectra based on M7.0 earthquake ground motions at $R_x = 6.8$ km – Site Class E
Comparison of ASCE/SEI 7-16 Two-Period (ELF) Design Spectrum w/o Spectrum Shape Adjustment and Multi-Period Response Spectra based on M8.0 earthquake ground motions at $R_X = 9.9$ km) – Site Class E

- MCEr Multi-Period Response Spectrum - Site Class BC
- MCEr Multi-Period Response Spectrum - Site Class E
- Design Multi-Period Response Spectrum - Site Class E

### ELF Design Spectrum (Cs x R/Ie) - ASCE 7-16 w/o SSAF

- $S_s = 1.5$
- $F_a = 0.8$
- $S_{MS} = F_a \times S_s = 1.2$
- $S_{DS} = 2/3 \times S_{MS} = 0.8$
- $S_1 = 0.72$
- $F_v = 2.0$
- $S_{M1} = F_v \times S_1 = 1.44$
- $S_{D1} = 2/3 \times S_{M1} = 0.96$

---

**Comparison of ASCE/SEI 7-16 Two-Period (ELF) Design Spectrum w/o Spectrum Shape Adjustment and Multi-Period Response Spectra based on M8.0 earthquake ground motions at $R_X = 9.9$ km) – Site Class E**
Interim Solution of ASCE/SEI 7-16 (2015 NEHRP Provisions)

- Require site-specific analysis to determine design ground motions for softer sites, but
- Provide exceptions to permit design using “conservative” values seismic design parameters
Site-Specific Requirements of Section 11.4.7 of ASCE/SEI 7-16 (2015 NEHRP Provisions)

- Site Class D - Site-specific ground motion procedures are required for structures on Site Class D sites where values of $S_1$ are greater than or equal to 0.2.
  - An exception permits ELF (and MRSA) design using a “conservative” value of the seismic design coefficient based on a 50 percent increase in the value of the seismic parameter $S_{M1} (S_{D1})$, effectively extending the acceleration domain to $1.5 T_s$. 
Site-Specific Requirements of Section 11.4.7 of ASCE/SEI 7-16 (2015 NEHRP Provisions)

- Site Class E - Site-specific ground motion procedures required for structures on Site Class E sites where values of $S_s$ are greater than or equal to 1.0 (or $S_1$ greater than 0.2)
  - An exception permits ELF design using a “conservative” value of the seismic design coefficient based on the seismic parameter $S_{MS}$ ($S_{DS}$) for Site Class C, regardless of the design period, $T$, effectively eliminating the velocity domain
Conterminous United States Regions with $S_1 \geq 0.2g$ (ASCE/SEI 7-16)

Orange Shaded Regions ($S_1 \geq 0.2g$)
10 percent of the area
90 percent of the risk
(AEL, FEMA 366)

Image source: USGS

- Define $MCE_R$ and design ground motions in terms of MPRS (e.g., for MRSA design or as the basis for selecting records for NRHA)

- Derive values of seismic design parameters (e.g., $S_{DS}$ and $S_{D1}$) from the MPRS of interest (e.g., for ELF design)

- Provide MPRS and associated values of seismic design parameters for User-specified values of:
  - Site Location (latitude, longitude)
  - Site Class
  - From USGS web service at [http://doi.org/10.5066/F7NK3C76](http://doi.org/10.5066/F7NK3C76) (aka USGS Seismic Design Geodatabase for ASCE/SEI 7-22) and
  - Other User-friendly providers (e.g., WBDG, ASCE 7 Hazard Tool, etc.)
MCE_R Ground Motions (Section 21.2)  
(Site-specific requirements of the 2020 NEHRP Provisions and ASCE/SEI 7-22)

- Probabilistic MCE_R Ground Motions (Section 21.2.1):
  - Risk-Targeted – 1% probability of collapse in 50 years
  - Collapse Fragility – 10% probability of collapse given MCE_R ground motions assuming lognormal standard deviation of 0.6 (Risk Category II)

- (New) Deterministic MCE_R Ground Motions (Section 21.2.2):
  - Scenario-Based – 84th percentile ground motions of the governing source (ignoring sources that contribute less than 10% to site hazard)
  - Derived from probabilistic ground motion hazard
  - Not less than deterministic lower-limit MCE_R Ground Motions

- MCE_R Ground Motions (Section 21.2.3):
  - Lesser of probabilistic MCE_R and deterministic MCE_R ground motions
Approach for Developing Multi-Period Response Spectra for United States Regions of Interest (CONUS and OCONUS sites)

- **CONUS Sites (WUS and CEUS):**
  - Science - 2018 Update of the USGS National Seismic Hazard Model (NSHM)
  - MCE\(_R\) Ground Motions – Site-specific requirements of Section 21.2 of the 2020 NEHRP Provisions and ASCE 7-22

- **OCONUS Sites (Alaska, Hawaii, etc.):**
  - Science – Most current values of \(S_S\) and \(S_1\) (and \(T_L\))
  - MCE\(_R\) Ground Motions – Site-specific requirements of Section 21.2 of the 2020 NEHRP Provisions and ASCE/SEI 7-22 and the MPRS procedures of FEMA P-2018

- **FEMA P-2078 (FEMA-funded ATC-136-1 Project)**
Multi-Period Response Spectra Format
(example matrix showing the combinations of twenty-two response periods, plus PGA_G, and eight hypothetical site classes of the standard format of multi-period response spectra)

- CONUS regions with ground motion models for all 22 x 8 combinations of site class and period (USGS 2018 NSHM):
  - WUS
  - CEUS

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Multi-Period Response Spectra Format
(example matrix showing the combinations of twenty-two response periods, plus PGA_g, and eight hypothetical site classes of the standard format of multi-period response spectra)

- CONUS regions with ground motion models for all 22 x 8 combinations of site class and period (USGS 2018 NSHM):
  - WUS
  - CEUS
- OCONUS regions with only two ground motion response parameters (S_S and S_I) and PGA (2018 USGS NSHM):
  - Alaska
  - Hawaii
  - Puerto Rico and the Virgin Islands
  - Guam and American Samoa

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<td>0.10</td>
<td>1.119</td>
</tr>
<tr>
<td>0.15</td>
<td>1.117</td>
</tr>
<tr>
<td>0.20</td>
<td>1.012</td>
</tr>
<tr>
<td>0.25</td>
<td>0.897</td>
</tr>
<tr>
<td>0.30</td>
<td>0.810</td>
</tr>
<tr>
<td>0.40</td>
<td>0.699</td>
</tr>
<tr>
<td>0.50</td>
<td>0.598</td>
</tr>
<tr>
<td>0.75</td>
<td>0.460</td>
</tr>
<tr>
<td>1.0</td>
<td>0.368</td>
</tr>
<tr>
<td>1.5</td>
<td>0.261</td>
</tr>
<tr>
<td>2.0</td>
<td>0.207</td>
</tr>
<tr>
<td>3.0</td>
<td>0.152</td>
</tr>
<tr>
<td>4.0</td>
<td>0.120</td>
</tr>
<tr>
<td>5.0</td>
<td>0.100</td>
</tr>
<tr>
<td>7.5</td>
<td>0.063</td>
</tr>
<tr>
<td>10</td>
<td>0.042</td>
</tr>
<tr>
<td>PGA_g</td>
<td>0.373</td>
</tr>
</tbody>
</table>
Example Multi-Period Response Spectra (MPRS)
(showing the new deterministic MCE\textsubscript{R} Lower Limit, Table 21.2-1, 2020 NEHRP Provisions and ASCE/SEI 7-22, which are anchored to $S_S = S_{SD} = 1.5$ g, $S_1 = S_{1D} = 0.6$ g)
Conterminous United States Regions Governed Solely by Probabilistic MCE_\text{R} Ground Motions for Default Site Conditions

Non-Orange Shaded Regions (Deterministic MCE_\text{R})

- > 90 percent of the area
- ≈ 10 percent of the risk (AEL, FEMA 366)

*Image source: USGS*
New Site Classes and Associated Values of Shear Wave Velocities
(Table 2.2-1, FEMA P-2078, June 2020)

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Shear Wave Velocity, $V_{s30}$ (fps)</th>
<th>USGS $V_{s30}$ (mps)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name</td>
<td>Description</td>
<td>Lower Bound¹</td>
</tr>
<tr>
<td>A</td>
<td>Hard rock</td>
<td>5,000</td>
</tr>
<tr>
<td>B</td>
<td>Medium hard rock</td>
<td>3,000</td>
</tr>
<tr>
<td>BC</td>
<td>Soft rock</td>
<td>2,100</td>
</tr>
<tr>
<td>C</td>
<td>Very dense soil or hard clay</td>
<td>1,450</td>
</tr>
<tr>
<td>CD</td>
<td>Dense sand or very stiff clay</td>
<td>1,000</td>
</tr>
<tr>
<td>D</td>
<td>Medium dense sand or stiff clay</td>
<td>700</td>
</tr>
<tr>
<td>DE</td>
<td>Loose sand or medium stiff clay</td>
<td>500</td>
</tr>
<tr>
<td>E</td>
<td>Very loose sand or soft clay</td>
<td>500</td>
</tr>
</tbody>
</table>

1. Upper and lower bounds, Table 20.3-1, ASCE/SEI 7-22.
2. Center of range (rounded) values used by USGS to develop MPRS.
Distribution of 9,050 of Census Tracts of Densely Populated Areas of California, Oregon and Washington by Site Class (90% of Population)

- BC, 1.9%
- C, 15%
- CD, 45%
- D, 37%
- DE, 1.6%

from Table A.2-1, FEMA P-2078, June 2020
Improved Values of Seismic Design Parameters

- Derive values of seismic design parameters ($S_{DS}$ and $S_{D1}$) from “best fit” of the 2-period spectrum to the multi-period design spectrum of the site of interest

- “Best Fit” based on site-specific requirements of Section 21.4:
  - $S_{DS}$ based on 90% of peak short-period response (acceleration domain)
  - $S_{D1}$ Based on 90% of peak response in the velocity domain (not less than 100% of 1-second response)
Example Derivation of $S_{DS}$ and $S_{D1}$ from a Multi-Period Design Spectrum

$S_{DS} = \text{Max}(0.9 \times S_a[0.2s \leq T \leq 5s])$

$S_{D1}/T = \text{max}(S_{a1}, T \times 0.9 \times S_a[1s \leq T \leq 2s])/T \quad v_{S30} > 1,200 \text{ fps}$

$\text{max}(S_{a1}, T \times 0.9 \times S_a[1s \leq T \leq 5s])/T \quad v_{S30} \leq 1,200 \text{ fps}$
Comparison of ASCE/SEI 7-16 Two-Period (ELF) Design Spectrum w/o Spectrum Shape Adjustment and Multi-Period Response Spectra based on M8.0 earthquake ground motions at $R_X = 9.9$ km) – Site Class E

ELF Design Spectrum

$S_s = 1.5$
$F_a = 0.8$
$S_{MS} = F_a \times S_s = 1.2$
$S_{DS} = 2/3 \times S_{MS} = 0.8$
$S_1 = 0.72$
$F_v = 2.0$
$S_{M1} = F_v \times S_1 = 1.44$
$S_{DL} = 2/3 \times S_{M1} = 0.96$
Multi-Period Design Spectrum
(Figure 11.4-1, 2020 NEHRP Provisions and ASCE/SEI 7-22 with annotation)

Site-Specific Multi-Period Design Spectrum

\[ S_{DS} = 2/3 \times S_{MS} \]

\[ C_s = S_{DS}/(R/I_e) \]

\[ T \leq T_s \]

\[ T_s = S_{D1}/S_{DS} \]

Acceleration Domain

\[ C_s = S_{D1}/T(R/I_e) \]

\[ T_s < T \leq T_L \]

Velocity Domain

\[ S_{D1} = 2/3 \times S_{M1} \]

Displacement Domain

\[ S_{DS} = 2/3 \times S_{MS} \]
Example Comparisons of Design Spectra (default site conditions)

- By Seismic Code Vintage
  - ASCE/SEI 7-10 - Two-period design spectrum
  - ASCE/SEI 7-16 - Two-period design spectrum
  - 2020 NEHRP Provisions (ASCE/SEI 7-22) - Multi-period design spectrum
  - 2020 NEHRP Provisions (ASCE/SEI 7-22) - Two-period design spectrum (for comparison with two-period spectra of ASCE/SEI 7-10 and ASCE/SEI 7-16)

- By Location
  - Irvine – WUS “probabilistic” site (magnitude M7.0 – M7.5)
  - San Mateo – WUS “deterministic” site (magnitude M7.5 - M8.0)
  - Anchorage – OCONUS “deterministic” site (magnitude M8.0 – M9.0)
  - Memphis – CEUS “probabilistic/deterministic” site (magnitude M7.5 – M8.0)
Comparison of Design Response Spectra – Irvine
(assuming default site conditions, Figure 8.2-1, FEMA P-2078, June 2020)

Velocity domain of the ASCE/SEI 7-16 (2PRS) design spectrum includes the 1.5 multiplier of the applicable Section 11.4.8 exception.
Comparison of Design Response Spectra – San Mateo
(assuming default site conditions, Figure 8.2-2, FEMA P-2078, June 2020)

Velocity domain of the ASCE/SEI 7-16 (2PRS) design spectrum includes the 1.5 multiplier of the applicable Section 11.4.8 exception.
Comparison of Design Response Spectra – Anchorage
(assuming default site conditions, Figure 8.2-4, FEMA P-2078, June 2020)

Velocity domain of the ASCE/SEI 7-16 (2PRS) design spectrum includes the 1.5 multiplier of the applicable Section 11.4.8 exception.

Derived MPRS based on:

- $S_S = 1.50 \text{ g}$ (deterministic MCE$_R$ floor)
- $S_r = 0.65 \text{ g}$ (deterministic MCE$_R$)
- $T_r = 16 \text{ s} (M = 8.0 \text{ – } 8.5)$
Comparison of Design Response Spectra – Memphis
(assuming default site conditions, Figure 8.2-4, FEMA P-2078, June 2020)

Velocity domain of the ASCE/SEI 7-16 (2PRS) design spectrum includes the 1.5 multiplier of the applicable Section 11.4.8 exception.
Design (As Usual) Using New MPRS

- **Design Ground Motions**
  - Ground motion parameters (and MPRS) are available online from a USGS web service [https://doi.org/10.5066/F7NK3C76] for user specified site location (i.e., latitude and longitude) and site conditions (i.e., site class).
  - Site-specific ground motion procedures (Chapter 21) now permit use of MPRS obtained online from the USGS web service (in lieu of a hazard analysis).

- **Design Procedures**
  - ELF procedures (Chapter 12) are not affected by proposed changes (although values of design parameters, $S_{DS}$ and $S_{D1}$, would better match the underlying response spectrum of the site of interest).
  - MRSA procedures (Chapter 12) are not affected by proposed changes (although multi-period design spectra would provide a more reliable calculation of dynamic response).
Questions
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Chapter 3 (Section 3.4)
Additional Revisions to Ground-Motion Provisions

2020 NEHRP Provisions Training Materials
C.B. Crouse, PhD, PE, AECOM
Presentation

- Maximum Considered Earthquake Geometric Mean ($MCE_G$) Peak Ground Acceleration (ASCE/SEI 7-22, Section 21.5)

- Vertical Ground Motion for Seismic Design (ASCE/SEI 7-22, Section 11.9)

- Site Class when Shear Wave Velocity Data Unavailable (ASCE/SEI 7-22, Section 20.3)
Background: In ASCE/SEI 7-16, Section 11.8.3, MCE\textsubscript{G} Peak Ground Acceleration (PGA\textsubscript{M}) was 

\[\text{PGA}_{M} = F_{PGA} \text{ PGA} \text{ (Equation 11.8-1)}\]

where \(F_{PGA}\) = site coefficient; and, \(PGA\) = Mapped MCE\textsubscript{G} Peak Ground Acceleration

In Section 21.5.2 Deterministic MCE\textsubscript{G} Peak Ground Acceleration, lower limit set at 

\[0.5 \times F_{PGA}\]
MCE_d Peak Ground Acceleration (ASCE/SEI 7-22, Section 21.5)

- **Update:** In ASCE/SEI 7-22, site coefficients eliminated because of MPRS
  - $\text{PGA}_M$ in Section 11.8.3 obtained from USGS Seismic Design Geodatabase for the applicable site class.
  - Deterministic lower limit value listed in bottom row of Table 21.2-1, ASCE/SEI 7-22

**Table 21.2-1 (Bottom Row) Deterministic Lower Limit Values of $\text{PGA}_d (g)$**

<table>
<thead>
<tr>
<th>Period $T$ (s)</th>
<th>A</th>
<th>B</th>
<th>BC</th>
<th>C</th>
<th>CD</th>
<th>D</th>
<th>DE</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\text{PGA}_d$</td>
<td>0.37</td>
<td>0.43</td>
<td>0.50</td>
<td>0.55</td>
<td>0.56</td>
<td>0.53</td>
<td>0.46</td>
<td>0.42</td>
</tr>
</tbody>
</table>
Additional Revisions (ASCE/SEI 7-22, Section 21.5)

- **Section 21.5.2 Deterministic $MCE_G$ Peak Ground Acceleration:**
  - Replaces “characteristic earthquakes” with “scenario earthquakes”
    - Determined from disaggregation of probabilistic $MCE_G$ peak ground acceleration.
    - From PSHA output, obtain mean magnitude, $M$, and mean rupture distance, $R_{rup}$, for each fault and its % contribution to probabilistic $MCE_G$ peak ground acceleration. The $M$ and $R_{rup}$ define the “scenario earthquake”
    - scenario earthquakes contributing < 10% of the largest contributor shall be ignored.

Example: Fault X has 75% contribution (largest)
- Fault Y has 20% contribution (included)
- Fault Z has 6% contribution (ignored)
Additional Revisions (ASCE/SEI 7-22, Section 21.5)

- **Section 21.5.3 Site-Specific MCE\textsubscript{G} Peak Ground Acceleration:**
- Determination of MCE\textsubscript{G} PGA similar in ASCE/SEI 7-16 & 7-22, i.e.,
  - Take lower of probabilistic & deterministic MCE\textsubscript{G} PGA
  - Resulting MCE\textsubscript{G} PGA must be $\geq 80\%$ of MCE\textsubscript{G} PGA from USGS Seismic Design Geodatabase
Vertical Ground Motion (ASCE/SEI 7-22, Section 11.9)

- **Background:** First introduced in ASCE/SEI 7-16 as $S_{aMV}$
  - Provision optional
  - $S_{aMV}$ given by Equations (11.9-1 through 11.9-4) for four specific $T_v$ ranges
  - $S_{aMV}$ derived from vertical/horizontal ($V/H$) component ratios applied to $MCE_R S_a(T)$
  - **Limitation:** No $S_{aMV}$ equation for $T_v > 2$ sec; site-specific determination required
  - **Oversight:** H component in $V/H$ ratio was geomean; $MCE_R S_a(T)$ was for direction of maximum shaking
**Vertical Ground Motion (ASCE/SEI 7-22, Section 11.9)**

- **Update:** Limitation & Oversight corrected in ASCE/SEI 7-22 $S_{aMv}$
  - Introduced $S_{aMv}$ Equation (11.9-5) for $T_v > 2$ sec
  - Corrected oversight by dividing MCE$_R$ $S_{aM}$, by $F_{md}$ to covert max direction $S_a$ to geomean $S_a$
  - $F_{md}$ given by Equations (11.9-6 through 11.9-8) for three specific $T_v$ ranges
  - $F_{md}$ based on Shahi & Baker (2014)

- Vertical coefficient, $C_v$, also revised to accommodate the nine site classes
  - New $C_v$ values in Table 11.9-1
  - $C_v$ depend on $S_{MS}$ (not $S_S$ in Table 11.9-1 of ASCE/SEI 7-16)
Vertical Ground Motion (ASCE/SEI 7-22, Section 11.9)

- **Example:** Comparison of $S_{aMv}$ and $S_{aM}$ for Irvine, CA site and Site Class D
Site Class when Shear Wave Velocity Data Unavailable (ASCE/SEI 7-22, Section 20.3)

- **Background:** In Section 20.3 of ASCE/SEI 7-16, site class determined from either
  - $\bar{v}_s$ - average shear-wave velocity in upper 100 ft (30 m)
  - $N$ - average STP in upper 100 ft (30 m)
  - $s_u$ - average undrained shear strength in upper 100 ft (30 m)
  - Ranges of these parameters for each site class provided in Table 20.3-1

- **Update:** In Section 20.2 of ASCE/SEI 7-22, Table 20.2-1 only includes $\bar{v}_s$; $N$ and $S_u$ have been eliminated
Site Class when Shear Wave Velocity Data Unavailable
(ASCE/SEI 7-22, Section 20.3)

- **Reasons for Revisions to Table 20.3-1:**
  - \( \bar{v}_s \) is better indicator of site response effects
  - \( N \) and \( s_u \) ranges were outdated and had no solid technical basis
  - Encourage the use of shear-wave velocity (Vs) measurements

- **Provision when Vs not Measured:**
  - Use applicable correlations between Vs & \( N \), or Vs & CPT, etc. to obtain Vs profile
  - Compute \( \bar{v}_s \) from Vs profile
  - Determine site classes from \( \bar{v}_s \), 1.3 \( \bar{v}_s \), and \( \bar{v}_s / 1.3 \)
  - Select most critical site class at each \( T \), i.e., one resulting in largest \( MCE_R S_a \)
Site Class when Shear Wave Velocity Data Unavailable

- **Hypothetical Example:** Irvine, CA site
  - Vs profile constructed from correlation with another soil parameter
  - $\bar{v}_s$ computed as 850 ft/sec (Site Class D)
  - $1.3 \bar{v}_s = 1,105$ ft/sec (Site Class CD)
  - $\bar{v}_s/1.3 = 654$ ft/sec (Site Class DE)
Hypothetical Example: Irvine, CA site. Envelope of three $S_{aM}$ must be taken.
Questions
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Coupled Walls

COUPLING BEAM, TYPICAL
WALL PIER, TYPICAL

LATERAL LOAD

$M_1$, $V_1$, $M_2$, $V_2$

2
Coupled Walls

Courtesy: Cary Kopczynski & Company, Bellevue, WA
Coupled Walls

Courtesy: Cary Kopczynski & Company, Bellevue, WA
Coupled Walls
Coupled Walls

Coupled shear wall systems are recognized as distinct from isolated shear wall systems in Canadian and New Zealand codes; they are also accorded higher response modification factors in view of their superior seismic performance. ASCE/SEI 7, prior to its 2022 edition, made no such distinction.
Ductile Coupled Shear Walls

Bertero wrote in 1977: “Use of coupled walls in seismic-resistant design seems to have great potential. To realize this potential it would be necessary to prove that it is possible to design and construct “ductile coupling girders” and “ductile walls” that can SUPPLY the required strength, stiffness, and stability and dissipate significant amounts of energy through stable hysteretic behavior of their critical regions.”

Thus, discussion needs to focus not on just coupled walls, but ductile coupled walls consisting of ductile shear walls and ductile coupling beams.
Energy Dissipation in Coupling Beams

MKA Study:
Non-linear response history analyses were conducted using spectrally matched ground motion records on a variety of coupled shear wall archetypes. Archetypes ranged from 5 to 50 stories in height and contained a range of longitudinal reinforcement ratios in the coupling beams as well as the shear walls.
Energy Dissipation in Coupling Beams

![Graph showing energy dissipation in coupling beams](image)

Courtesy: Magnusson Klemencic Associates
ACI 318-19 18.10.9 Ductile Coupled Walls

18.10.9.1 Ductile coupled walls shall satisfy the requirements of this section.

18.10.9.2 Individual walls shall satisfy \( \frac{h_{wcs}}{\ell_w} \geq 2 \) and the applicable provisions of 18.10 for special structural walls.

18.10.9.3 Coupling beams shall satisfy 18.10.7 [Coupling beams] and (a) through (c) in the direction considered.

(a) Coupling beams shall have \( \frac{\ell_n}{h} \geq 2 \) at all levels of the building.

(b) All coupling beams at a floor level shall have \( \frac{\ell_n}{h} \leq 5 \) in at least 90 percent of the levels of the building.

(c) The requirements of 18.10.2.5 shall be satisfied at both ends of coupling beams [reinforcement developed for 1.25\(f_y\)].
Special Shear Walls

(a) Wall with $h_w/\ell_w \geq 2.0$ and a single critical section controlled by flexure and axial load designed using 18.10.6.2, 18.10.6.4, and 18.10.6.5.
Ductile Coupling Beams

- Aspect ratio $\frac{l_n}{h} \geq 4$
  - Satisfy requirements of 18.6
- Aspect ratio $\frac{l_n}{h} < 4$
  - Permitted to be reinforced with two intersecting groups of diagonal bars
- Aspect ratio $\frac{l_n}{h} < 2$ and $V_u > 4\sqrt{f_c}A_{cw}$
  - Must be reinforced with two intersecting groups of diagonal bars
Ductile Coupling Beams

Source: http://nees.seas.ucla.edu/pankow
Ductile Coupling Beams

ACI 318-08 Alternate Detail

Source: http://nees.seas.ucla.edu/pankow
2020 NEHRP Provisions
2020 NEHRP Provisions

- Part 1: Modifications to ASCE/SEI 7-16
- Part 2: Commentary to the Modifications
- Part 3: Resource Papers
Additional ACI 318-19 Changes in Special Shear Wall Design

There have been four significant ACI 318-19 code changes, all adopted in our FEMA P695 study, to address the flexural-compression wall failure issue.

(1) 18.10.3.1 (shear amplification) - would typically require design shear (required shear strength) $V_u$ to be amplified by a factor of up to 3 (similar to New Zealand, Canada).

(2) 18.10.6.4 - requires improved wall boundary and wall web detailing, i.e, overlapping hoops if the boundary zone dimensions exceed 2:1, crossties with 135-135 hooks on both ends, and 135-135 crossties on web vertical bars.
Additional ACI 318-19 Changes in Special Shear Wall Design

(3) **18.10.6.2(b)** (Wall drift or deformation capacity check) - requires a low probability of lateral strength loss at MCE level hazard (you can think of it as requiring a minimum wall compression zone thickness), and

(4) **18.10.2.4** - Minimum wall boundary longitudinal reinforcement, to limit the potential of brittle tension failures for walls that are lightly-reinforced.
Shear Amplification: Concrete Shear Walls

18.10.3.1 The design shear force $V_e$ shall be calculated by:

$$V_e = \Omega_v \omega_v V_u \leq 3 V_u$$  \hspace{1cm} (18.10.3.1)

where $V_u$, $\Omega_v$, and $\omega_v$ are defined in 18.10.3.1.1, 18.10.3.1.2, and 18.10.3.1.3, respectively.

18.10.3.1.1 $V_u$ is the shear force obtained from code lateral load analysis with factored load combinations.
Shear Amplification: Concrete Shear Walls

18.10.3.1.2 $\Omega_v$ shall be in accordance with Table 18.10.3.1.2.

**Table 18.10.3.1.2—Overstrength factor $\Omega_v$ at critical section**

<table>
<thead>
<tr>
<th>Condition</th>
<th>$\Omega_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h_{wc}/\ell_w &gt; 1.5$</td>
<td>Greater of</td>
</tr>
<tr>
<td></td>
<td>$M_{pr}/M_{tu}[1]$</td>
</tr>
<tr>
<td></td>
<td>1.5[2]</td>
</tr>
<tr>
<td>$h_{wc}/\ell_w \leq 1.5$</td>
<td>1.0</td>
</tr>
</tbody>
</table>

[1] For the load combination producing the largest value of $\Omega_v$.

[2] Unless a more detailed analysis demonstrated a smaller value, but not less than 1.0.
Shear Amplification: Concrete Shear Walls

18.10.3.1.3 For walls with $h_{wcs}/L_w < 2.0$, $\omega_v$ shall be taken as 1.0. Otherwise, $\omega_v$ shall be calculated as:

\[
\omega_v = 0.9 + \frac{n_s}{10} \quad n_s \leq 6
\]

(18.10.3.1.3)

\[
\omega_v = 1.3 + \frac{n_s}{30} \leq 1.8 \quad n_s > 6
\]

where $n_s$ shall not be taken less than the quantity $0.007h_{wcs}$. 
Earthquake Force-Resisting Structural Systems of Concrete — ASCE/SEI 7-22

<table>
<thead>
<tr>
<th>Basic Seismic Force-resisting System</th>
<th>Detailing Reference Section</th>
<th>$R$</th>
<th>$\Omega_0$</th>
<th>$C_d$</th>
<th>System Limitations And Building Height Limitations (Ft) By Seismic Design Category</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>B</td>
</tr>
<tr>
<td>A. Bearing Wall System</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Special reinforced concrete shear walls</td>
<td>14.2</td>
<td>5</td>
<td>$2^{1/2}$</td>
<td>5</td>
<td>NL</td>
</tr>
<tr>
<td>2. Ductile Coupled reinforced concrete shear walls</td>
<td>14.2</td>
<td>8</td>
<td>$2^{1/2}$</td>
<td>8</td>
<td>NL</td>
</tr>
<tr>
<td>3. Ordinary reinforced concrete shear walls</td>
<td>14.2</td>
<td>4</td>
<td>$2^{1/2}$</td>
<td>4</td>
<td>NL</td>
</tr>
</tbody>
</table>

$q$ Structural height, $h_n$, shall not be less than 60 ft (18.3 m).

Minimum height is intended to ensure adequate degree of coupling and significant energy dissipation provided by the coupling beams.
# Earthquake Force-Resisting Structural Systems of Concrete — ASCE/SEI 7-22

<table>
<thead>
<tr>
<th>Basic Seismic Force-resisting System</th>
<th>Detailing Reference Section</th>
<th>( R )</th>
<th>( \Omega_0 )</th>
<th>( C_d )</th>
<th>System Limitations And Building Height Limitations (Ft) By Seismic Design Category</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>( B )</td>
</tr>
<tr>
<td><strong>B. Building Frame System</strong></td>
<td></td>
<td></td>
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</table>

\(^q\) Structural height, \( h_n \), shall not be less than 60 ft (18.3 m).
### Earthquake Force-Resisting Structural Systems of Concrete — ASCE/SEI 7-22

<table>
<thead>
<tr>
<th>Basic Seismic Force-resisting System</th>
<th>Detailing Reference Section</th>
<th>R</th>
<th>$\Omega_0$</th>
<th>$C_d$</th>
<th>System Limitations And Building Height Limitations (Ft) By Seismic Design Category</th>
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<td>B</td>
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<td>D. Dual Systems with Special Moment Frames</td>
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<sup>q</sup> Structural height, $h_n$, shall not be less than 60 ft (18.3 m).
Example Problem

Design of a Special Reinforced Concrete Ductile Coupled Wall
A 22-story reinforced concrete residential building is designed following the requirements of ASCE/SEI 7-22, and ACI 318-19. The building consists of a flat plate-column gravity system with a central core, formed by four reinforced concrete coupled structural walls, which acts as the seismic force-resisting system. The structural walls are designed as Ductile Coupled Reinforced Concrete Shear (Structural) Walls.

A computer rendering of the building framing is shown on the next two slides. The plan view of the building changes from one floor to another. A plan view of the second floor of the building is shown.
Example Building Configuration
Example Building Configuration

Second Floor Plan View
Design Criteria

- **Member Sizes:**
  - Shear walls: 26 in. thick
  - Slabs (2nd and 3rd floors): 8 in. thick
    - (4th floor and higher): 7.5 in. thick
  - Gravity columns: Various sizes

- **Material properties:**
  - Concrete (used in structural walls and columns): $f'_c = 8000$ psi (all floors)
  - Concrete (used in slabs): $f'_c = 6000$ psi (floors)
  - All members are constructed of normal weight concrete ($w_c = 150$ pcf)
  - Reinforcement (used in all structural members): $f_y = 60,000$ psi
Design Criteria

- **Service Loads:**
  - Superimposed dead load: 25 psf (includes SDL on the floor plus the weight of cladding distributed over the floor slab.)
  - Live load: Based on the 40 psf live load prescribed in ASCE/SEI 7-22 Table 4.3-1 for residential buildings (private rooms and corridors serving them), a reduced live load of 20 psf is used in the example.
  - Reduced roof Live load: 20 psf

- **Seismic Design Data:**
  - Risk Category: II
  - Seismic importance factor, $I_e = 1.0$
  - Site Class: D
Design Criteria

- Seismic Design Data (contd.):
  - The maximum considered earthquake spectral response acceleration:
    At short periods, $S_S = 1.65g$, and
    At 1-sec period, $S_1 = 0.65g$.
  - The maximum considered earthquake spectral response acceleration (site modified):
    At short periods, $S_{MS} = 1.65g$, and
    At 1-sec period, $S_{M1} = 0.98g$.
  - Design Spectral Response Acceleration Parameters (at 5% damping):
    At short periods: $S_{DS} = 2/3 \frac{S_{MS}}{g} = 2/3 \times 1.65 = 1.10$
    At 1-sec period: $S_{D1} = 2/3 \frac{S_{M1}}{g} = 2/3 \times 0.98 = 0.65$
Design Criteria

- Seismic Design Data (contd.):
  - Long-period transition period, $T_L = 8$ sec
  - Ductile Coupled Reinforced Concrete Structural Walls ... $R = 8$; $C_d = 8.0$, $\Omega_0 = 2.5$ (ASCE/SEI 7-22 Table 12.2-1)
  - Seismic Design Category: Based on both $S_{DS}$ (ASCE/SEI 7-22 Table 11.6-1) and $S_{D1}$ (ASCE/SEI 7-22 Table 11.6-2), the Seismic Design Category (SDC) for the example building is D.
Design Procedure

Although ASCE/SEI 7-22 permits the Equivalent Lateral Force procedure to be used in all situations, the modal response spectrum analysis (MRSA) procedure (ASCE/SEI 7-22 Section 12.9.1) is used in this example. However, as part of the MRSA procedure, base shear is also determined using Equivalent Lateral Force (ELF) procedure. This is because ASCE/SEI 7-22 requires that the base shear obtained from MRSA be scaled up to match the ELF base shear.

The building was modeled in ETABS 2016, and the total seismic weight was obtained from the program as **43,099 kips**.
Analysis by Equivalent Lateral Force Procedure

- Structural period calculation
  - Coefficient, $C_t = 0.02$ [ASCE/SEI 7-22 Table 12.8-2]
  - Coefficient, $x = 0.75$ [ASCE/SEI 7-22 Table 12.8-2]
  - Structure height above base, $h_n = 234.25$ ft
  - Approximate period, $T_a = 0.02h_n^x = 0.02 \times 234.25^{0.75} = 1.2$ sec.
  - Fundamental period from modal analysis by ETABS, $T = 2.58$ sec (along x-axis)
  - Fundamental period from modal analysis by ETABS, $T = 2.26$ sec (along y-axis)
  - Calculated period is larger than the approximate period. However, the fundamental period cannot exceed $C_uT_a$.
    - For $S_{D1} = 0.65$, $C_u = 1.4 \rightarrow C_uT_a = 1.4 \times 1.2 = 1.68$ sec $\rightarrow T$ used in design $= 1.68$ sec $< T_L$ (= 8 sec)
Analysis by Equivalent Lateral Force Procedure

- **Base shear calculation**
  - \( C_S = \frac{S_{DSle}}{R} = \frac{1.101 \times 1.0}{8} = 0.138 \)  
    (ASCE/SEI 7-22 Eq. 12.8-2)
  - \( C_S \leq \frac{S_{DSle}}{RT} = \frac{0.65 \times 1.0}{(8 \times 1.68)} = 0.048 \)  
    (for \( T \leq T_L \))  
    (ASCE/SEI 7-22 Eq. 12.8-3)
  - \( C_S \geq 0.044 S_{DSle} = 0.044 \times 0.65 \times 1.0 = 0.029 \)  
    (ASCE/SEI 7-22 Eq. 12.8-5)
  - \( C_S \geq 0.01 \)  
    (ASCE/SEI 7-22 Eq. 12.8-5)
  - \( C_S \leq \frac{0.5 S_{1le}}{R} = \frac{0.5 \times 0.65 \times 1.0}{8} = 0.041 \)  
    (for \( S_1 \leq 0.6g \))  
    (ASCE/SEI 7-22 Eq. 12.8-6)
  - Governing \( C_s = 0.048 \)
  - Base shear, \( V = C_s \times W = 0.048 \times 43,099 = 2090 \text{ kips} \)
Modal Response Spectrum Analysis

A 3-D modal response spectrum analysis (MRSA) is performed using ETABS (v2016).

- Semi-rigid diaphragms are assigned at each level.
- The effective cracked member stiffnesses used in the analyses are as follows:
  - Columns and shear walls, $I_{\text{eff}} = 0.7I_g$
  - Coupling beams, $I_{\text{eff}} = 0.25I_g$
  - Gravity columns, $I_{\text{eff}} = 0.1I_g$ (with pinned connections at the base)
- Adequate number of modes are considered in the modal analysis to incorporate 100% of the modal mass in each of x- and y-directions. Also, appropriate scale factors are applied to the base shears calculated in the x- and y-directions to amplify them to those calculated in the ELF procedure.
## Floor Forces from MRSA

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## Story Drifts from MRSA (X-Direction)

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## Story Drifts from MRSA (Y-Direction)

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Story Drift Limitation

According to ASCE/SEI 7-22 Section 12.12.1, the calculated relative story drift at any story must not exceed 2% (ASCE/SEI 7-22 Table 12.12-1 for all other buildings in Risk Category I and II). As can be seen from the previous slide, this is satisfied in all stories.
Design of Shear Wall

- The design of one of the shear walls at the base of the structure is illustrated in this example in accordance with the provisions of ACI 318-19.
- One L-shaped segment of the shear wall core is designed as two flanged walls.
- Orthogonal combination of seismic forces is NOT required as axial loads on the wall from seismic forces are less than 20% of the design axial strength.
Seismic forces acting along x-axis are considered in this design example. The design calculations for the seismic forces acting along the y-axis are similar and are not shown. However, the final wall configuration will incorporate effects of seismic forces in both directions.

<table>
<thead>
<tr>
<th>Load Combinations</th>
<th>Axial Force, $P_u$ (kips)</th>
<th>Shear Force, $V_u$ (kips)</th>
<th>Bending Moment, $M_u$ (ft-kips)</th>
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<tr>
<td>2</td>
<td>$1.2D + 1.6L + 0.5L_r$</td>
<td>6071</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>$(1.2+0.2S_{DS})D + \rho Q_E + 0.5L$</td>
<td>10,015</td>
<td>576</td>
</tr>
<tr>
<td>4</td>
<td>$(0.9D - 0.2S_{DS}D) + \rho Q_E$</td>
<td>6460</td>
<td>573</td>
</tr>
<tr>
<td>5</td>
<td>$(0.9D - 0.2S_{DS}D) - \rho Q_E$</td>
<td>-378</td>
<td>573</td>
</tr>
</tbody>
</table>
Design of Shear Wall – Design for Shear

- Height of the shear wall, $h_{wcs} = 2811$ in. (234.25 ft)
- Length of the shear wall, $\ell_w = 164$ in. (13.67 ft)
- $h_{wcs}/\ell_w = 2811/164 = 17.1$

**ACI 318-19 (hereafter ACI 318) Section 18.10.2.2**

At least two curtains of reinforcement shall be used if $V_u > 2A_{cv}\lambda \sqrt{f'_c}$ or $h_{wcs}/\ell_w \geq 2.0$. In this case, $h_{wcs}/\ell_w = 17.1 > 2.0$.

So, at least two curtains of reinforcement are required.
Design of Shear Wall – Design for Shear

**ACI 318 Section 18.10.3.1**

Design shear force, \( V_e = \Omega_v \omega_v V_u \leq 3V_u \)

- For walls with \( \frac{h_{wcs}}{\ell_w} > 1.5 \), \( \Omega_v \) is the greater of \( \frac{M_{pr}}{M_u} \) and 1.5. The probable moment strength \( M_{pr} \) is unknown at this stage. So, it is assumed that \( \Omega_v = 1.5 \). This may very well prove to be unconservative. Once the flexural reinforcement has been provided, this will be verified or corrected, if necessary

- For walls with \( \frac{h_{wcs}}{\ell_w} \geq 2.0 \) and the number of stories above critical section, \( n_s > 6 \), \( \omega_v = 1.3 + \frac{n_s}{30} \leq 1.8 \)
  - In this example, \( n_s = 22 \). \( n_s \) cannot be taken less than the quantity \( 0.007h_{wcs} (= 19.68) \), which is satisfied.
  - \( \omega_v = 1.3 + 22/30 = 2.03 \rightarrow \omega_v = 1.8 \)
Design of Shear Wall – Design for Shear

**ACI 318 Section 18.10.3.1**

Design shear force, $V_e = \Omega \omega_v V_u \leq 3V_u$

- $V_e = 1.5 \times 1.8 \times 576 = 1555$ kips (governs)
- $V_e = 3V_u = 3 \times 576 = 1728$ kips

**ACI 318 Section 18.10.4.4.**

The maximum nominal shear strength, $V_n$, allowed for a wall section is

$$10A_{cv}\sqrt{f'_c} = 10 \times 4264 \times \sqrt{f'_c} /1000 = 3813$$ kips

So, $\varphi V_n = 0.75 \times 3813 = 2860$ kips $> V_e \rightarrow$ The provided wall section size is acceptable.
Design of Shear Wall – Design for Shear

**ACI 318 Section 18.10.4.1**

\[ V_n = (\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y) A_{cv} \geq \frac{V_e}{\phi} \]  

(ACI 318 Eq. 18.10.4.1)

- For \( h_w/\ell_w = 17.1 \geq 2.0 \), \( \alpha_c = 2 \)
- For normal-weight concrete, \( \lambda = 1 \)

Required horizontal shear reinforcement ratio:

\[ \rho_t = \left[ \frac{V_e}{\phi A_{cv}} - \alpha_c \lambda \sqrt{f'_c} \right] / f_y = 0.0051 \]
ACI 318 Section 18.10.4.1

- Two curtains of #7 horizontal shear reinforcement at a vertical spacing of 7 in. is adequate to resist this shear force. However, the 7” spacing is reduced to 5” in order to maintain uniformity with the reinforcement provided in the other leg of the shear wall to resist shear in y direction, leading to a provided $\rho_t = 0.009$. The 5” spacing also matches the vertical spacing of the transverse reinforcement provided in the special boundary element of the wall (shown later), which helps with construction efficiency.

- Per ACI 318 Section 18.10.2.1, the minimum $\rho_t = 0.0025$ and maximum reinforcement spacing = 18 in., both of which are satisfied.
Design of Shear Wall – Design for Shear

**ACI 318 Section 18.10.4.3**

\[ \frac{h_w}{\ell_w} > 2.0. \] Therefore, \( \rho_{\ell} \) need not be larger than or equal to \( \rho_t \).

**ACI 318 Section 18.10.2.1**

Longitudinal reinforcement ratio:

- \( \rho_{\ell} \geq 0.0025 \) with a maximum spacing of 18 in.
- Provided two curtains of \#8 vertical reinforcement at 14 in. spacing (\( \rho_{\ell} = 0.004 \)). This will need to be increased in the end regions of the wall.
ACI 318 Section 18.10.2.4

In walls with $h_w/\ell_w \geq 2.0$ that are effectively continuous from the base of the structure to the top of wall and are designed to have a single critical section for flexure and axial loads:

- longitudinal reinforcement ratio within $0.15\ell_w$ of the ends of the wall needs to be at least $6\sqrt{f'_c/f_y}$. 
ACI 318 Section 18.10.2.4

- 9 #8 bars are provided in a 3×3 pattern in the wall intersection area.
  \[ \rho = 0.01 > 6\sqrt{f'_c / f_y} = \frac{0.009}{6} \]  
  \( = 0.009 \) ..........OK

- This needs to be satisfied at the other ends of the two legs of the wall. However, reinforcement provided there would be governed by special boundary element requirements, which is shown next.
Boundary Elements of Special RC Shear Walls

**ACI 318 Section 18.10.6.1**

The need for special boundary elements at the edges of shear walls is to be evaluated in accordance with ACI 318 Section 18.10.6.2 (displacement-based approach) or ACI 318 Section 18.10.6.3 (stress-based approach). In this example, the displacement-based approach is used as the wall satisfies the three required conditions:

- 1) \( \frac{h_{wcs}}{\ell_w} \geq 2.0 \),
- 2) The wall is continuous from the base of the structure to the top of the wall, and
- 3) The wall has a single critical section for bending and axial loads.
**ACI 318 Section 18.10.6.2(a): Displacement-based Approach**

Compression zones are to be reinforced with special confinement reinforcement where:

\[
\frac{1.5\delta_u}{h_{WCS}} \geq \frac{\ell}{600c}
\]

(ACI 318 Eq. 18.10.6.2a)

- \(\delta_u\) is the design displacement. For seismic forces along the x-axis of the structure, \(\delta_u\) was determined from the ETABS analysis as 26.84 in.

- \(c\) is the largest neutral axis depth of the wall cross-section calculated for the factored axial force and nominal moment strength consistent with the direction of the design displacement. This was determined using *spColumn* v7.00 as 95 in.
Boundary Elements of Special RC Shear Walls

ACI 318 Section 18.10.6.2(a): Displacement-based Approach

- The check was satisfied when the non-flanged end of the wall is in compression. Thus, a special boundary element is required at the non-flanged end.

- The same check was performed for the flanged end of the wall as well. However, when the flanged end of the wall is under compression, the neutral axis depth is small due to the presence of the flange, and as a result, the check is not satisfied. So, a special boundary element is not required for the flanged end of the wall.
**Boundary Elements of Special RC Shear Walls**

**ACI 318 Section 18.10.6.2(b)(i): Height of special boundary element**

The special boundary element reinforcement is to extend vertically from the critical section a distance not less than the larger of $\ell_w$ and $M_u/4V_u$.

- $\ell_w = 164$ (13.67 ft) ... Governs
- $M_u/4V_u = 10.84$ ft

**ACI 318 Section 18.10.6.2(b)(ii): Width of special boundary element**

The width of boundary element, $b$ (=26 in.) $\geq \sqrt{0.025c\ell_w} = \sqrt{0.025\times95\times164} = 19.3$ in.

...............OK
ACI 318 Section 18.10.6.4(a): Length of boundary element

Confined boundary element to extend horizontally from the extreme compression fiber a distance not less than the larger of \( c - 0.1\ell_w \) and \( c/2 \).

- \( c - 0.1\ell_w = 95 - 0.1 \times 164 = 78.6 \text{ in} \approx 80 \text{ in} \ldots \) governs
- \( c/2 = 95/2 = 47.5 \text{ in} \).

ACI 318 Section 18.10.6.4(b): Stability check for wall compression zone

Minimum width of the compression zone, \( b = 26 \text{ in.} \) is required to be at least \( h_u/16 \), where \( h_u \) is the laterally unsupported height of the wall (18.10.6.4(b)).

- \( h_u = \text{Story height} - \text{depth of coupling beam} = 158 \text{ in.} \)
- \( h_u/16 = 9.875 \text{ in.} < 26 \text{ in.} \ldots \) OK
Boundary Elements of Special RC Shear Walls

ACI 318 Section 18.10.6.4(c): Stability check for wall compression zone

For this wall,

- $h_{wcs}/l_w = 17.1 > 2.0$
- It is effectively continuous from the base of the structure to top of the wall
- It is designed to have a single critical section for flexure and axial loads.
- $c/l_w = 95/164 = 0.58 > 3/8$.

As a result, Section 18.10.6.4(c) requires the width of the flexural compression zone $b$ over the length of 80 in. (calculated before) to be greater than or equal to 12 in. This is satisfied as the width of the wall is 26 in.
Boundary Elements of Special RC Shear Walls

**ACI 318 Section 18.10.6.4(g): Minimum area of transverse reinforcement**

\[
A_{sh}/s_{bc} = \text{Greater of } \begin{cases} 
0.3 \left( \frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_yt} \\
0.09 \frac{f'_c}{f_yt}
\end{cases} 
\]

(ACI 318 Table 18.10.6.4(g))

**Confinement perpendicular to the length of the wall:**

\[
A_{sh}/s_{bc} = \text{Greater of } \begin{cases} 
0.3 \left( \frac{2080}{1771} - 1 \right) \frac{8}{60} \\
0.09 \frac{8}{60}
\end{cases} = 0.012
\]

Provided 16 #5 bars in the form of hoops and cross-ties with a vertical spacing of 5 in.
Boundary Elements of Special RC Shear Walls

ACI 318 Section 18.10.6.4(g): Minimum area of transverse reinforcement

\[ A_{sh}/sb_c = \text{Greater of} \begin{cases} 
0.3 \left( \frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}} \\
0.09 \frac{f'_c}{f_{yt}} 
\end{cases} \]  

(ACI 318 Table 18.10.6.4(g))

Confinement parallel to the length of the wall:

\[ A_{sh}/sb_c = \text{Greater of} \begin{cases} 
0.3 \left( \frac{2080}{1771} - 1 \right) \frac{8}{60} \\
0.09 \frac{8}{60} 
\end{cases} \]

= 0.012

Provided 5 #5 bars in the form of hoops and cross-ties with a vertical spacing of 5 in.
ACI 318 Section 18.10.6.4(e): Vertical spacing of transverse reinforcement

According to ACI 318 Section 18.7.5.3, as revised by ACI 318 Section 18.10.6.4(e), the transverse reinforcement is to be vertically spaced at a distance not exceeding

- One-third of the least dimension of the boundary element = 23/3 = 7.67 in.
- Six times the diameter of the smallest longitudinal reinforcement = 6 × 1.0 = 6.0 in.

..... Governs

- \( s_o \), as defined by ACI 318 Eq. (18.7.5.3).

\[
4 \text{ in.} \leq s_o = 4 + \frac{14 - h_x}{3} \leq 6 \text{ in.} \implies 4 \text{ in.} \leq s_o = 4 + \frac{14 - 5.18}{3} \leq 6 \text{ in.} \implies s_o = 6.0 \text{ in.}
\]
ACI 318 Section 18.10.6.4(e): Vertical spacing of transverse reinforcement

The vertical spacing also cannot exceed the maximum value given in ACI 318 Table 18.10.6.5(b). For Grade 60 reinforcement within the height of the special boundary element, it is the lesser of

- Six times the diameter of the smallest longitudinal reinforcement = 6 × 1.0 = 6 in.
- 6 in.

The provided spacing of 5 in. satisfies both these limits.
Boundary Elements of Special RC Shear Walls

**ACI 318 Section 18.10.6.4(h): Concrete in floor system**

Concrete within the thickness of the floor system at the special boundary element location is required to have a specified compressive strength of at least $0.7f'_c$. With the slab concrete strength of 6000 psi, this is satisfied.

**ACI 318 Section 18.10.6.4(i and j)**

The special boundary element confinement is to be provided at the non-flanged end of the wall at the base of the shear walls. The confinement needs to extend vertically by at least 12.67 ft above the base (ACI 318 Section 18.10.6.2(b)). Below the base, the boundary element transverse reinforcement needs to extend at least 12 in.
ACI 318 Section 18.10.6.5: Boundary confinement where special boundary element is not required

- In this example, \( V_u \geq \lambda \sqrt{f'_c} A_{cv} \). So, the end of the horizontal web reinforcement that terminates at the edges of wall flange is required to have a standard hook engaging the edge reinforcement. Alternatively, the edge reinforcement at the flange is required to be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement. The first option is utilized for this example.

- In the intersection region of the wall, 9 #8 longitudinal bars are provided within an area of \( 26 \times 26 = 676 \) in.\(^2\)

\[
\rho_\ell = 9 \times 0.79 / 676 = 0.0105 > 400/f_y = 400/60,000 = 0.0067
\]

A vertical transverse reinforcement spacing of 5 in. is provided to match the spacing of the other transverse reinforcement for construction efficiency.
Design of Shear Wall (Grade 60 Reinforcement)
Check Strength Under Flexure and Axial Loads
Design of Shear Wall (Grade 80 Reinforcement)
Design of Shear Wall (Grade 80 Reinforcement)

- Use of Grade 80 steel leads to a considerable reduction in the amount of reinforcement in the wall. In addition to the smaller bar sizes, lesser congestion in the special boundary elements is especially noticeable. However, the vertical spacing of the transverse hoops and cross-ties in the special boundary elements remained 5 in. as that in the Grade 60 design. This is because the maximum value of that spacing is limited to 6 times the diameter of the smallest longitudinal bar. So, smaller bar sizes achieved by higher strength reinforcement ironically led to a tighter spacing compared to what would be necessary for confinement alone. The vertical spacing of the horizontal shear reinforcement is also smaller than what is required for resisting shear so that it matches the spacing of transverse reinforcement in the boundary elements for construction efficiency. Thus, some of the gains achieved by using Grade 80 reinforcement are negated by various other other considerations.
Design of Coupling Beam

A coupling beam oriented along the y-axis of the building at the second floor level is selected for this example. The dimensions of the beam are given below:

- Clear span of the beam, $\ell_n = 76$ in. (6.33 ft)
- Height of the beam, $h = 28$ in. (2.33 ft)
- Width of the beam, $b_w = 26$ in. (2.17 ft)
- $\ell_n/h = 76/28 = 2.7$

Since $2 < \ell_n/h < 4$, per ACI 318 Section 18.10.7.3, this beam can be designed as a deep coupling beam using two intersecting groups of diagonally placed bars, or as a special moment frame flexural member in accordance with the ACI 318 Sections 18.6.3 through 18.6.5. The second option is adopted for this example.
Design of Coupling Beam – Design Loads

The forces on this beam due to gravity loads are minimal. So, the design shear and moment are determined from the seismic forces alone. The governing forces on this beam come when the seismic forces are acting along the y-axis of the building. Those forces are shown below.

- $V_u = \pm 154$ kips
- $M_u = \pm 488$ ft-kips
Design of Coupling Beam – Design for Flexure

**ACI 318 Section 18.6.3.1: Limits on flexural reinforcement**

Assuming a 1.5 in. clear cover, No. 8 bars (1 in. dia.) as longitudinal reinforcement and No. 4 bars (0.5 in. dia.) as transverse reinforcement:

Effective depth, \( d = 28 - 1.5 - 0.5 - 0.5 = 25.5 \) in.

\[
A_{s,min} \geq \frac{3\sqrt{f_c}}{f_y} b_w d = \frac{3\sqrt{8000}}{60,000} \times 26 \times 25.5 = 2.97 \text{ in.}^2 \quad \text{...(ACI 318 Section 9.6.1.2(a))}
\]

\[
\geq \frac{200}{f_y} b_w d = \frac{200}{60,000} \times 26 \times 25.5 = 2.21 \text{ in.}^2 \quad \text{...(ACI 318 Section 9.6.1.2(b))}
\]
ACI 318 Section 18.6.3.1: Limits on flexural reinforcement

For Grade 60 steel, the maximum area of flexural reinforcement for both top and bottom faces of the beam is

\[ A_{s,\text{max}} = 0.025b_w d = 0.025 \times 26 \times 25.5 = 16.58 \text{ in.}^2 \]  

(ACI 318 Section 18.6.3.1)

Also, at least two bars should be continuous at both top and bottom (ACI 318 Section 18.6.3.1).

Provided flexural reinforcement and flexural strength

The following reinforcement is provided:

- 6-No.8 bars at the bottom => \( A_s = 4.74 \text{ in.}^2 \)
- 6-No.8 bars at the top => \( A_s = 4.74 \text{ in.}^2 \)
Design of Coupling Beam – Design for Flexure

**Flexural Strength Check**

Using *spColumn* software, the positive and negative design moment strengths (i.e., $\phi M_n^+$ and $\phi M_n^-$) at all locations of the beam were found to be

$$\phi M_n = 526 \text{ ft-kips} > M_u (= 488 \text{ ft-kips}) \ldots \text{ O.K.}$$

The same reinforcement is continued through the length of the beam. For a beam with a length of 6.33 ft, it is not worth cutting off some of the bars near midspan.
Design of Coupling Beam – Design for Flexure

**ACI 318 Section 18.6.3.2**

- At the joint face, the positive moment strength must be at least half the negative moment strength. Since the top and bottom reinforcement are the same, this is automatically satisfied.

- Additionally, both the negative and the positive moment strength at any section along member length must be at least one-fourth the maximum moment strength provided at face of either joint. Since no bar is being cut-off near midspan, this requirement is also satisfied.
Design of Coupling Beam – Design for Flexure

ACI 318 Section 18.10.9.3 – Development Length

In a ductile coupled wall, the longitudinal reinforcement needs to be developed at both ends of the beam in accordance with ACI 318 Section 18.10.2.5. Item (a) of that section requires that for coupling beams reinforced like a special moment frame beam, the development length of longitudinal reinforcement must be 1.25 times the values calculated for $f_y$ in tension.

Using the provisions of ACI 318 Section 25.4.2.3, the development length of No. 8 bars is calculated as 55 in.

Thus all longitudinal reinforcing bars need to be extended into the wall web by a distance of 55 in.
Design of Coupling Beam – Minimum Transverse Requirements

ACI 318 Section 18.6.4.1

- Confinement reinforcement is required to be provided over a length of $2h = 2 \times 28 = 56$ in. from both support faces.,
- The first hoop is to be placed no more than 2 in. from support.
- The hoop spacing must not exceed:
  - $d/4 = 25.5/4 = 6.375$ in.
  - 6 in.
  - For Grade 60 reinforcement - six times the diameter of the smallest primary flexural reinforcing bar = $6 \times 1.0 = 6$ in.
- Since this hoop spacing needs to be provided within 56 in. from both supports, and the total length of the coupling beam is 76 in., #4 confinement hoops are provided at 6 in. spacing over the whole length of the beam starting at 2 in. from each wall face.
Design of Coupling Beam – Design for Shear

**ACI 318 Section 18.6.5.1**

\[
V_e = \frac{M^-_{pr} + M^+_{pr}}{\ell_n} = 229 \text{ kips}
\]

where the probable flexural strength, \(M_{pr}\), at the joint faces is calculated by taking the tensile stress in steel as \(1.25f_y\) and the strength reduction factor \(\phi\) as 1.0. Additionally, gravity load effects are small on this beam and are neglected for simplicity.

- With 6-No.8 bars at top and bottom: \(M^+_{pr}\) and \(M^-_{pr}\) = 724 ft-kips

**ACI 318 Section 18.10.4.5**

\[
V_{n,\text{max}} = 10A_{cv}\sqrt{f'_c} = 10 \times 26 \times 28 \times \sqrt{8000}/1000 = 651 \text{ kips} > V_e/\phi \quad \text{.... OK}
\]
Design of Coupling Beam – Design for Shear

ACI 318 Section 18.6.5.2 – Shear Reinforcement

\[ V_s = V_e/\phi = 229/0.75 = 305 \text{ kips} \]

Required spacing of six-legged No. 4 stirrups,

\[ s = \frac{A_v f_{yt} d}{V_s} = \frac{6 \times 0.2 \times 60 \times 25.50}{305} = 6.0 \text{ in.} \]

\[ V_{s,max} = 8\sqrt{f_c'} b_w d = 8 \times \sqrt{8000} / 26 \times 25.5 / 1000 = 474 \text{ kips} > V_s \quad \text{OK} \]
Design of Coupling Beam – Design for Shear

- #8 long. top reinf.
- Six-legged #4 stirrup @ 6 in.
- #8 long. bottom reinf.

Dimensions:
- 25 in.
- 28 in.
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Topics Covered

- Introduction to Coupled C-PSW/CFs (SpeedCore System)
- Section Detailing, Limits, Requirements
- Seismic Behavior & Capacity Design
- Design Example
Introduction to Coupled C-PSW/CFs (SpeedCore System)
C-PSW/CF (SpeedCore System)

Composite Plate Shear Walls – Concrete Filled (C-PSW/CF)

- Steel plates
- Concrete infill
- Tie bars
- Shear studs
- No rebars or formwork

- Shear walls and/or elevation core walls

The SpeedCore system has evolved organically from this past with steel-concrete composite construction. C-PSW/CF or composite walls consist of steel modules that are filled with concrete! The steel modules consist of steel plates, tie bars and maybe shear studs. There are no rebars at all and no need for formwork or falsework for construction. These composite walls can be used as shear walls or elevator core walls in lieu of conventional reinforced concrete walls.
A New Chapter in Composite Construction

Rainier Square, Seattle

- Client
- Architect
- Structural & Civil
- GC/GM
The contributions, early engagement, and detailed discussions with the steel fabricator and erector were key for the success of the project.
The main reason for considering SpeedCore for this project was the **SPEED OF CONSTRUCTION**. This structure was constructed in 10 months, providing savings of about 8 months as compared to conventional RC construction. This structure and its conceive are so iconic that they both ended up on the cover of the ENR.
From an engineering perspective, the coupled composite core walls are highlighted within the structural floor plans along the structure height here. They are replacing the convention RC elevator core wall structures.
A New Chapter in Composite Construction

200 Park Avenue, San Jose, CA

- High seismic region
- 937,000 square foot
- 19 stories
- Under construction

The first application of SpeedCore in a high seismic region is 200 Park Avenue in San Jose. This 19-story structure is under construction.
Section Detailing, Limits, Requirements
Key Components of C-PSW/CF (SpeedCore System)

- Steel plates
- Concrete infill
- Tie bars
- Shear studs

These are the key components of SpeedCore composite walls. Just a reminder / reference picture. It's quite simple. Steel plates or module, tie bars / shear studs, and concrete infill.
### Steel Plates

- **Reinforcement ratio limits:**
  - Minimum = 1%
  - Maximum = 10%

- Two steel plates must be connected to each other using ties

- Ties can consist of bars, steel shapes, or built-up shapes

- Steel plates must be anchored to concrete infill using stud anchors or ties or combination of ties and studs

---

For the steel plates, the minimum and maximum reinforcement ratio limits are 1% and 10%. These are based on the range of parameters considered in research. The steel plates have to be connected to each other using ties. These ties can consist of steel bars, rods, or shapes. To prevent local buckling and develop composite action, the steel plates must be anchored to the concrete infill using studs, ties, or a combination of both.
Local Buckling, Plate Slenderness, Axial Compression

Under compressive stresses, local buckling can occur in the steel plates in between the anchor points provided by studs or ties. The plate slenderness ratio is defined as the anchor spacing $s$ divided by plate thickness $t_p$. Years of experimental and numerical investigations conducted around the world are summarized in this plot between the normalized plate slenderness ratio $\lambda = \frac{s}{t_p \sqrt{E}}$ and the normalized local buckling strain ($e_{cr}/e_Y$). The rectangular shaded region is of interest for design, because there are no data point in it indicating the slenderness limit.
Local Buckling, Plate Slenderness, Axial Compression

Seismic Design:

\[ \frac{b}{t_p} \leq 1.05 \sqrt{\frac{E_s}{R_y F_y}} \]

\[ F_{cr} \geq F_y \]

\[ P_{no} = A_s F_y + 0.85 f'_c A_c \]

Using these experimental and numerical results, plate slenderness requirements were developed to ensure compactness, i.e., yielding in compression occurs before local buckling. Equations were also developed for slender plates, but the code requires nonslender or compact plates for the seismic design. These requirements are slightly tighter for seismic design as they should be.
Local Buckling, Plate Slenderness, Axial Compression

- In accordance with AISC 341-22 Section H7.5s, steel plate slenderness ratio at the base of C-PSW/CF (protected zones) should be limited as follows:

\[
\frac{S}{t_p} < 1.05 \frac{E_s}{\sqrt{R_y F_y}}
\]

- Steel plate slenderness ratio at regions, which are protected zones should be limited as follows:

\[
\frac{S}{t_p} < 1.2 \frac{E_s}{F_y}
\]
The design of tie bars is often governed by the design requirements for empty steel modules before concrete placement. Experimental investigations and numerical studies show that the flexibility of empty steel modules is dominated by the “effective shear stiffness” associated with Vierendeel truss / frame action. This is useful when considering behavior during transportation, handling, and erection activities.
The stability of empty modules during erection, construction, and concrete placement is an important design concern, which ends us governing the tie bar size and spacing / detailing. We looked at the global buckling of a unit width of the empty module, which is conservative but representative. Unit width is defined by the tie bar spacing in the horizontal direction.
Tie Bar Size, Spacing, and Stability of Empty Modules

- Minimum \((GA)_{\text{eff}}\) of empty module for transportation, erection, and stability during construction, concrete casting
- Refined calculations can be made using theory
- Recommendations for tie bar size

\[
\frac{S}{t_p} < 1.0 \sqrt{\frac{E_s}{\alpha + 1}}
\]

Where, \(\alpha = 1.7 \left(\frac{t_{tie}}{t_p} - 2\right) \left(\frac{t_p}{d_{tie}}\right)^4\)

- \(\alpha\) is the ratio of plate flexural stiffness to tie flexural stiffness
- \(\alpha\) governs the value of \((GA)_{\text{eff}}\), and thus the tie spacing \(S/t_p\) requirement
- Still need to meet plate slenderness req.

Using all these experimental and analytical results, we developed minimum requirements for the stiffness of empty modules and to simplify design checks. The minimum tie size and spacing requirements are summarized. Alpha is the ratio of steel plate flexural stiffness to tie bar flexural stiffness. It governs the value of \((GA)_{\text{eff}}\), and thus the \(S/t_p\), tie spacing to plate thickness requirement. This requirement is to be used along with the local buckling slenderness requirement. Together they govern the section detailing of composite walls.
Recommendations for Stiffness

In-Plane Flexural Stiffness

- Account for concrete cracking corresponding to the required strength level
- Section moment-curvature response $\rightarrow$ secant stiffness corresponding to 60% of moment capacity
- Extent of concrete cracking, if drift governs or walls are overdesigned

\[
EI_{eff} = E_s I_s + 0.35 E_c I_c
\]

Effective flexural stiffnesses (AISC Design Guide 37, 2021)

\[
EA_{eff} = E_s A_s + 0.45 E_c A_c
\]

Effective axial stiffnesses (AISC Design Guide 37, 2021)

\[
GA_{v,eff} = G_s A_{s,wall} + G_c A_c
\]

Effective shear stiffnesses (AISC Design Guide 37, 2021)

These are the recommendations for the effective stiffness values when modeling SpeedCore walls for analysis for calculating design demands. The stiffness should account for concrete cracking corresponding to the design scenario. The engineer can perform a section moment-curvature analysis to estimate stiffness, and account for the extent of concrete cracking (which may be quite limited) if drift governs for lateral wind loading. In the absence of better information, we have provided some simple stiffness recommendations for the engineer to consider / use.
The in-plane flexural strength of SC walls can be calculated using plastic stress distribution over the composite cross-section. It’s quite simple, with the steel limited to yield stress $f_y$ in compression and tension, and concrete to $0.85 f'_c$ in compression. Equations have been developed and provided in the Design Guide (I will mention at the end of my talk).
Recommendations for Shear Strength

- In accordance with AISC 360-22 Section I4.4, nominal in-plane shear strength of L-shaped C-PSW/CFs is determined considering the steel section and infill concrete contributions as follows:

\[
V_{n,\text{wall}} = \frac{K_s + K_{sc}}{\sqrt{3 K_s^2 + K_{sc}^2}} A_{s,\text{wall}} F_y
\]

where, 
\[K_s = G_s A_{s,\text{wall}}\]  

where, 
\[K_{sc} = \frac{0.7 (E_c A_c) (E_s A_{s,\text{wall}})}{(4 E_s A_{s,\text{wall}}) (E_c A_c)}\]

(AISC Design Guide 37, 2021)
Seismic Design of Coupled Composite Plate Shear Walls / Concrete Filled (Capacity Design)
Seismic Design of Coupled C-PSW/CF


Seismic design can be performed according to FEMA P-2082 NEHRP (2020), ASCE/SEI 7-22 Standard, ANSI/AISC 341-22 Seismic Provisions, and AISC Design Guide 37 (2021). Performance-based design can also be conducted, but that is beyond the scope of this presentation.
Seismic Design of Coupled C-PSW/CF

The 2020 Edition of the NEHRP Recommended Seismic Provisions:

- Response modification factor $R = 8$
- Over-strength factor $\Omega_0 = 2.5$
- Deflection amplification factor $C_d = 5.5$
Seismic Design Philosophy for Coupled C-PSW/CF

- Coupling beams form plastic hinges and distributed plasticity along structure height
- Walls sized to develop plastic hinges along entire wall height

\[ \gamma_2 = \frac{\sum 1.2 M_{R,EB} \cdot \omega_{CR,EB}}{\sum M_{R,EB}} \]

\[ V_e = \gamma_1 \gamma_2 \]

\[ \Omega_\theta = \gamma_1 \gamma_2 \]

(AISC Design Guide 37, 2021)
Seismic Design Philosophy

2D Finite Element Model (Pushover Response)

This is showing the same system level behavior but now using the 2D finite element model. In these figures black indicates beyond yielding. Again, the system shows desired and designed behavior. Between “Point A” and “Point B” all coupling beams yield along height of structure. At “Point C” wall forms plastic hinge at base. Coupling beams fracture at “Point D”.

(Shafrail et al., 2022)
Design Example
Building Description

- Coupled L-shaped Composite Plate Shear Walls / Concrete Filled (C-PSW/CFs) are used to resist seismic loads.

- Steel gravity frames are placed around the coupled C-PSW/CFs, and elevators and stairs are located inside the core walls.

Floor plan of the office building with 120 ft length and 100 ft width (a total of 12,000 square feet of area)
Building Description

- 18-story office building
- First story height = 17 ft
- Typical story height = 13 ft
- Total height = 238 ft.
Material Properties

Steel:
- ASTM A572 Grade 50 steel (steel plates) & ASTM A992 Grade 50 steel (wide flange sections)
- $F_y = 50$ ksi
- $F_u = 65$ ksi
- $E_s = 29,000$ ksi
- $G_s = 11,500$ ksi
- $R_y = 1.1$ (ANSI/AISC 341-22 Table A3.1)

Concrete:
- Self-compacting concrete (SCC)
- $f'c = 6$ ksi
- $E_c = 4,500$ ksi
- $G_c = 1,770$ ksi
- $R_c = 1.5$ (ANSI/AISC 341-16 H5-5)

Floor plan of the office building with 120 ft length and 100 ft width (a total of 12,000 square feet of area)
**Loads & Load Combinations**

**Loads:**
- Self-weight of structure (gravity frames and core walls) (dead load)
- Floor live load = 50 psf (Redactable)
- Partition = 15 psf
- Superimposed dead load (ceiling and floor finish) = 15 psf
- Curtain wall = 15 psf (wall surface area)

**Load Combinations:**
- Load combination provided in Chapter 2 of ASCE/SEI 7-16 are considered.
  - $1.4D$
  - $1.2D + 1.6L$
  - $1.2D + 0.5L \pm 1.0E$
  - $0.9D \pm 1.0E$

Floor plan of the office building with 120 ft length and 100 ft width (a total of 12,000 square feet of area)
Building Description

- 3D computer model of the building was developed using a commercial software program for the design of steel gravity frames.

- Based on the preliminary design of gravity frames, the self-weight of structure is calculated.
Seismic Forces

**Building Seismic Weight:**
- First Story = 1,555 kips
- Typical Story = 1,440 kips
- Roof = 1,263 kips

**Seismic Design Parameters:**
- $S_{DS} = 1.101g$
- $S_{D1} = 0.650g$
- Site Class D
- Risk Category II
- Seismic Design Category D

**Period of the structure**
- $T_u = C_t \frac{h^x}{n} = (0.020)(238 \text{ ft})^{0.75} = 1.21$ seconds
- $C_u = 1.4$ (ASCE/SEI 7 Table 12.8-1)
- $T = C_u T_u = (1.4)(1.21) = 1.70$ seconds
- $T = 1.87$ (3D ETABS model)
- The period of structure is considered to be the upper limit, $C_u T_u = 1.70$

Period of the structure is calculated according to Section 12.8.2 of ASCE/SEI 7-16 standard. The approximate period is generally lower a detailed computational model; therefore, an upper limit on the period recommended by ASCE/SEI 7-16 Standard is considered. The period of structure is also estimated using a detailed 3D computer model developed in ETABS software program. The computed period of 3D ETABS model is 1.87, which is higher than the upper limit. Therefore, the period of structure is considered to be the upper limit.
Design Base Shear

Equivalent Lateral Forces (ELF) procedure was used to calculate the seismic loads:

- \( V = C_s W \)
- \( C_s = \frac{S_{DS}}{R/I_e} = \frac{1.101}{8/1} = 0.138 \) (ASCE/SEI 7 12.8-2)
- \( C_{s,\text{Max}} = \frac{S_{DS}}{R(I_e) / 1.7} = \frac{1.101}{8/1} = 0.048 \) (ASCE/SEI 7 12.8-3)
- \( C_{s,\text{Min}} = 0.44 S_{DS} I_e = (0.44)(1.101)(1) = 0.048 \) (ASCE/SEI 7 12.8-5)
- \( C_s = \frac{0.5 S_1}{(R/I_e)} = \frac{(0.5)(0.65)(8/1)}{1.7} = 0.041 \) (ASCE/SEI 7 12.8-6)
- \( V = C_s W = (0.048)(25844) = 1,238 \text{ kips} \)
- \( OTM = \sum_{i=1}^{n} F_i h_i = 217,217 \text{ kip-ft} \)

Seismic response coefficient is selected 0.048 and the design base shear is calculated 1238 kips.

The overturning moment (OTM) of the building is computed 217,217 kip-ft.
C-PSW/CFs and Coupling Beam Dimensions

**C-PSW/CF:**
- \( L_w = 12 \text{ ft} \)
- \( t_{sc} = 16 \text{ in.} \)
- \( t_p = \frac{1}{2} \text{ in.} \)

**Coupling beams:**
- \( L_{CB} = 10 \text{ ft} \)
- \( b_{CB} = 16 \text{ in.} \)
- \( h_{CB} = 24 \text{ in.} \)
- \( t_{CB,f} = \frac{1}{2} \text{ in.} \)
- \( t_{CB,w} = \frac{3}{8} \text{ in.} \)
- \( L_{CB}/h_{CB} = 5 \)

L-shape C-PSW/CFs have length \((L_w)\) of 12 feet, wall thicknesses \((t_{sc})\) of 16 in., and steel plate thicknesses \((t_p)\) of \(\frac{1}{2}\) in. Composite coupling beam width \((b_{CB})\) and height \((h_{CB})\) are 16 and 24 in., respectively. Coupling beam flange \((t_{CB,f})\) and web \((t_{CB,w})\) plate thicknesses are \(\frac{1}{2}\) and \(\frac{3}{8}\) in., respectively.
In seismic design, 2D computer model of coupled C-PSW/CFs was developed using a commercial software (SAP2000) to determine the inter-story drift and shear force demands in coupling beams. Coupling beams and L-shape C-PSW/CFs were modeled using beam elements. When the coupled C-PSW/CFs are subjected to lateral seismic loads, two walls are in tension and the other two walls in compression due to the coupling action. In this example, the two compression or tension walls are considered one wall (beam element) in computer modeling and the limit state checks. Additionally, in 2D computer model, flexural, axial, and shear stiffnesses of coupling beam are doubled to model the two beams.
Inter-story Drift Limit

- Deformation shape, lateral displacement, and inter-story drift.
- Amplified displacement is calculated by multiplying story displacement value by the deflection amplification factor. Inter-story drift is calculated using the amplified displacement.
- Maximum inter-story is 1.65%.

Linear elastic analysis was performed to determine the lateral deflection and coupling beam shear force demands. Amplified displacement is calculated by multiplying story displacement value by the deflection amplification factor ($C_d$). Inter-story drift is calculated using the amplified displacement. In this design example, the maximum design inter-story drift is limited to 2% in accordance with the ASCE/SEI 7 Standard. From the structural analysis of 2D model, maximum inter-story of the structure is 1.65%, which is lower than the maximum design inter-story drift limit. Figure shows deformation shape, lateral displacement, and inter-story drift of coupled C-PSW/CFs.
Linear Elastic Analysis

- $V_{r.CB} = 167$ kips (average)
- $V_{Max.CB} = 223.5$ kips (maximum)
- $M_{U.CB} = \frac{V_{r.CB} L_{CB}}{2} = 835$ kip-ft
- $M_{Max.CB} = \frac{V_{Max.CB} L_{CB}}{2} = 1,117$ kip-ft

<table>
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<tr>
<th>(#)</th>
<th>Story Elevation (ft.)</th>
<th>Disp. (in.)</th>
<th>Amplified Disp. (in.)</th>
<th>Interstory Drift (%)</th>
<th>CB Shear Force (kips)</th>
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From the structural analysis of 2D model, the average and maximum required shear strengths ($V_{r.CB}$ and $V_{max.CB}$) for coupling beams are calculated. The average required shear strength is used to size the coupling beams. Structural designers can choose to use the average or maximum required shear strengths. Since a portion of the OTM will be resisted by the coupling action and the remainder by the individual walls, the result of this choice is the relative proportioning of wall and coupling beam elements. Since the system is designed to ensure plasticity spreads along the height of the structure, either method is acceptable. The average and maximum required shear strengths for coupling beams are 167 and 223.5 kips, respectively. The average and maximum required flexural strengths ($M_{U.CB}$ and $M_{Max.CB}$) for coupling beams are calculated 835 and 1,117 kip-ft, respectively.
Design Of Coupling Beams

Flexure-Critical Coupling Beams:

\[ V_{n, \text{exp.CB}} \geq \frac{2.4 \cdot M_{p, \text{exp.CB}}}{L_{CB}} \]  
(AISC Design Guide 37, 2021)

Expected Flexural Capacity \( M_{p, \text{exp.CB}} \):

\[ M_{p, \text{exp.CB}} = 1,582.6 \text{ kip-ft} \]

Minimum Area of Steel:

\[ A_{s, \text{CB.min}} = 0.01 \cdot h_{CB} \cdot b_{CB} = (0.01)(24)(16) = 3.8 \text{ in.}^2 \]  
(AISC Spec. I2.2a)

\[ A_{s, \text{CB}} = 33.25 > A_{s, \text{CB.min}} = 3.8 \text{ in.}^2 \]

Coupling beams of coupled C-PSW/CF systems are designed to be a flexure critical member in accordance with ANSI/AISC 341-22 Section H8.5c.

The expected flexural capacity \( M_{p, \text{exp,CB}} \) of coupling beam is calculated assuming the steel plate reaches a yield stress of \( R_y F_y \) (in both compression and tension) and infill concrete reaches a yield stress of \( R_c f'_c \) (in compression).

In accordance AISC 360-22 Section I2.2a, steel plates should comprise at least 1% of the total cross section area of composite coupling beam.
## Design Of Coupling Beams

### Steel Plate Slenderness Requirement for Coupling Beams:

- \( \frac{b_{c, CB}}{t_{CB,f}} = 30.5 < 2.37 \sqrt{\frac{E_s}{R_y F_y}} = 2.37 \sqrt{\frac{29000}{(1.1)(50)}} = 54.4 \) (AISC 360–22 Table I1.1b)
- \( \frac{h_{c, CB}}{t_{CB,w}} = 61.3 \geq 2.66 \sqrt{\frac{E_s}{R_y F_y}} = 2.66 \sqrt{\frac{29000}{(1.1)(50)}} = 61.1 \) (AISC 360–22 Table I1.1b)

### Flexural Strength \( (M_{p, CB}) \):

- \( M_{n, CB} = M_{p, CB} = 1,407 \text{ kip–ft} \)  
  **(AISC Design Guide 37, 2021)**
- \( \phi_B M_{n, CB} = 1,266 \text{ kip–ft} \) > \( M_{U, CB} = 835 \text{ kip–ft} \)
- \( \frac{M_{T, CB}}{\phi_B M_{n, CB}} = 0.66 \)  
  \( \frac{M_{U, CB, Max}}{\phi_B M_{n, CB}} = 0.88 \)

Although the slenderness ratio of web plate is slightly higher than the requirement for, in this design example, it is assumed the web plates of coupling beams meet the requirement.

In seismic design of coupled C-PSW/CFs, composite coupling beams are designed to be compact sections. The slenderness requirements of flange and web plates are checked in accordance with AISC 360-22 Section I1.4.

Plastic stress distribution method is used to calculate flexural capacity \( (M_{p, CB}) \). The flexural capacity \( (M_{p, CB}) \) of coupling beam is calculated assuming the steel plate reaches a yield stress of \( F_y \) (in both compression and tension) and infill concrete reaches a stress of \( 0.85f'_c \) (in compression).
Design Of Coupling Beams

Nominal Shear Strength ($V_{n, CB}$):

- $V_{n, CB} = 0.6 F_y A_{w, CB} + 0.06 K_c \sqrt{f'_c A_{c, CB}} = 592 \text{ kips (AISC Design Guide 37, 2021)}$
- $\phi_v V_{n, CB} = 532 \text{ kips} > V_{U, CB} = 167 \text{ kips}$

\[
\frac{V_{r, CB}}{\phi_v V_{n, CB}} = \frac{167 \text{ kips}}{532 \text{ kips}} = 0.31 \\
\frac{V_{U, CB}}{\phi_v V_{n, CB}} = \frac{223.5 \text{ kips}}{532 \text{ kips}} = 0.42
\]

Flexure-Critical Coupling Beams (revisited):

- $V_{n, exp, CB} = 0.6 R_y F_y A_{w, CB} + 0.06 K_c \sqrt{R_c f'_c A_{c, CB}} = 657 \text{ kips}$
- $V_{n, exp, CB} = 657 \text{ kips} > \frac{2.4 M_{P, exp, CB}}{L_{CB}} = 380 \text{ kips} \quad (\text{AISC Design Guide 37, 2021})$

Nominal shear strength, $V_{n, CB}$, of composite coupling beam is calculated in accordance with AISC 360-22 Section I4.2. The nominal shear strength is the summation of shear strengths of steel web plates ($V_s$) and infill concrete ($V_c$). The selected composite coupling beams are flexure critical members.
Design Of C-PSW/CFs

Minimum and Maximum Area of Steel:

- \( A_{\text{gross.wall}} = (2)[(L_w \cdot t_{sc}) + (L_w - t_{sc})t_{sc}] = 8,704 \text{ in.}^2 \)
- \( A_{s, \text{min}} = 0.01 A_{\text{gross.wall}} = (0.01)(8,704) = 87 \text{ in.}^2 \) (ANSI/AISC 360-22 I2.2a)
- \( A_{s, \text{max}} = 0.1 A_{\text{gross.wall}} = (0.1)(8,704) = 870 \text{ in.}^2 \) (ANSI/AISC 360-22 I2.2a)
- \( A_s = (t_p)[8L_w + 4t_{sc} - 16t_p] = 604 \text{ in.}^2 \)
- \( A_{s, \text{min}} = 87 \text{ in.}^2 < A_s = 604 \text{ in.}^2 < A_{s, \text{max}} = 870 \text{ in.}^2 \)

In accordance with ANSI/AISC 360-22 Section I1.6, the steel plates in C-PSW/CF should comprise at least 1% but no more than 10% of the total composite cross-section area.
Design Of C-PSW/CFs

Slenderness Requirements:

- In accordance with ANSI/AISC 341-22 Section H8.4b, steel plate slenderness ratio, $b/t$, at the base of C-PSW/CF (protected zones) should be limited as follows:

  $S_{tie} = 12 \text{ in. (the bottom two stories)}$

  $\frac{S_{tie}}{t_p} = 24 < 1.05 \frac{E_s}{R_y F_y} = 1.05 \frac{29,000}{(1.1)(50)} = 24.1$  
  (ANSI/AISC 341-22 H8.4b)

- Steel plate slenderness ratio, $b/t$, at regions which are not protected zones:

  $S_{tie.top} = 14 \text{ in.}$

  $\frac{S_{tie.top}}{t_p} = 28 < 1.2 \frac{E_s}{F_y} = 1.2 \frac{29,000}{(50)} = 28.9$  
  (ANSI/AISC 360-22)

In this design example, steel tie bars are only used in L-shaped C-PSW/CFs (on shear studs); therefore, the largest unsupported length between tie bars is considered for slenderness requirements check. Tie bar spacings are selected 12 and 14 in. for the bottom (the bottom two stories) and top (remaining stories) of L-shaped C-PSW/CFs.
Design Of C-PSW/CFs

Tie spacing requirements:

- In accordance with ANSI/AISC 360-22 Section I1.6b, the tie bar spacing to plate thickness ratio, \( S/t_p \), should be limited as follows:

  \[ d_{tie} = \frac{3}{4} \text{ in.} \]

  \[ \alpha = 1.7 \left( \frac{t_{sc}}{t_p} - 2 \right) \left( \frac{t_p}{d_{tie}} \right)^4 = 1.7 \left( \frac{16}{0.5} - 2 \right) \left( \frac{0.5}{0.75} \right)^4 = 10.07 \]  
  (AISC Design Guide 37, 2021)

  \[ \frac{S_{tie\_bottom}}{t_p} = 24 < 1.0 \sqrt{\frac{E_s}{2\alpha + 1}} = 1.0 \sqrt{\frac{29,000}{2(10.07)+1}} = 37.0 \]  
  (AISC Design Guide 37, 2021)

  \[ \frac{S_{tie\_top}}{t_p} = 32 < 1.0 \sqrt{\frac{E_s}{2\alpha + 1}} = 1.0 \sqrt{\frac{29,000}{2(10.07)+1}} = 37.0 \]  
  (AISC Design Guide 37, 2021)

The stability of empty steel module of C-PSW/CF during the construction and concrete casting depends on tie bar spacing to plate thickness ratio.
Design Of C-PSW/CFs

Required Wall Shear Strength:

- A shear amplification factor of 4 is used to amplify the base shear.

\[ V_{\text{Amplified}} = 4,952 \text{ kips} \]
\[ V_{r\text{.wall}} = \frac{4,952}{2} = 2,476 \text{ kips} \]

(AISC Design Guide 37, 2021)
Design Of C-PSW/CFs

Required Flexural Strength of Coupled C-PSW/CFs

- A shear amplification factor of 4 is used to amplify the base shear.

- \( M_{p,exp.CB} = 1,583 \text{ kip-ft} \) (Expected flexural capacity of CB)
- \( V_{n,Mp,exp.CB} = \frac{2.4 M_{p,exp.CB}}{L_{CB}} = 380 \text{ kips} \) (Expected shear strength of CB)
- \( \gamma_1 = \frac{\sum n 1.2 M_{p,exp.CB}}{\sum n M_{U,CB}} = \frac{(18)(1.2)(1583)}{(18)(835)} = 2.27 \) (Overstrength amplification factor)
- \( P_{CB} = 2 \sum n V_{n,Mp,exp.CB} = 13,673 \text{ kips} \) (Axial force due to coupling action)
- \( M_{r,wall} = \gamma_1 OTM - P_{CB} L_{eff} = 125,077 \text{ kip-ft} \) (Required amplified OTM)
- \( P = -2 \sum n V_{n,Mp,exp.CB} - (1.2 \sum n F_{Tri,DL}) - (0.5 \sum n F_{Tri,LL}) = -20,644 \text{ kips} \) (axial compression force)
- \( T = 2 \sum n V_{n,Mp,exp.CB} - (0.9 \sum n F_{Tri,DL}) = 9,219 \text{ kips} \) (axial tension force)
Design Of C-PSW/CFs

Wall Tensile Strength:

▪ $P_{n,T} = A_s F_y = (604)(50) = 30,200$ kips
▪ $\phi_t P_{n,T} = 27,180$ kips > $T = 9,219$ kips
▪ $\frac{P_{n,T}}{T} = 0.35$

Wall Compression Strength:

▪ A simplified unite width method is considered to calculate nominal compression strength.

A simplified unite width method is considered to calculate nominal compression strength. This is a conservative approach to calculate the nominal compression strength, as the effect of end plates on the compression capacity is not considered. However, this simplified unit width method can be used for C-PSW/CFs with different configurations, for example, L-shaped, C-shaped, I-shaped walls. Selected unit width cross-section of L-shaped C-PSW/CF is shown. The nominal compression strength of L-shaped C-PSW/CFs is calculated.
Design Of C-PSW/CFs

Wall compression Strength:

- $S_{tie} = 12\ \text{in} = 1\ \text{ft}$ (Length of selected unit width)
- $L_{wall\_total} = 48\ \text{ft}$ (Total length of two C-PSW/CFs)
- $P_{no} = 2t_p S_{tie} F_y + 0.85 f'_c (t_{sc} - 2t_p) S_{tie} = 1,518\ \text{kips}$ (ANSI/AISC 360-22)
- $P_e = \frac{\pi^2 E I_{eff\_min}}{L_{cr}^2} = 1797\ \text{kips}$
- $\frac{P_{no}}{P_e} = 0.84 < 2.25$ (ANSI/AISC 360-22)
- $P_{n.c} = P_{no} \left(0.685 \frac{P_{no}}{P_e}\right) = 1,066\ \text{kips}$
- $P_{n.c\_total} = P_{n.c} n_{unit\_width} = (1,066\ \text{kips})(48) = 51,168\ \text{kips}$
- $\phi C P_{n.c\_total} = (0.9)(51,168\ \text{kips}) = 46,051\ \text{kips} > P = 20,644\ \text{kips}$
- $\phi C P_{n.c\_total} = 0.45$

A simplified unite width method is considered to calculate nominal compression strength. This is a conservative approach to calculate the nominal compression strength, as the effect of end plates on the compression capacity is not considered. However, this simplified unit width method can be used for C-PSW/CFs with different configurations, for example, L-shaped, C-shaped, I-shaped walls. Selected unit width cross-section of L-shaped C-PSW/CF is shown. The nominal compression strength of L-shaped C-PSW/CFs is calculated.
Plastic Stress Distribution:

\[ M_{P,T \text{wall}} = M_{n,T \text{wall}} = 1,598,236 \text{ kip-in.} \]

\[ M_{P,C \text{wall}} = M_{n,T \text{wall}} = 1,761,166 \text{ kip-in.} \]

When the core system is subjected to lateral seismic forces, two L-shaped C-PSW-CFs are subjected in tension force and the other two L-shaped C-PSW-CFs are in compression. The flexural capacities of tension and compression L-shaped C-PSW-CFs are calculated using plastic stress distribution method, when they are subjected to -18365 kips compression and 6940 kips tension forces.
Design Of C-PSW/CFs (Flexural Strength)

The effective flexural stiffnesses of tension and compression ($EI_{T,wall}$ and $EI_{C,wall}$) L-shaped C-PSW/CFs are used to calculated required flexural strengths of tension and compression walls.

\[
M_{U,T,wall} = \left[ \frac{EI_{T,wall}}{(EI_{C,wall} + EI_{T,wall})} \right] M_{r,wall} = 652833 \text{ kip-in.} = 54403 \text{ kip-ft}
\]

\[
M_{U,C,wall} = \left[ \frac{EI_{C,wall}}{(EI_{C,wall} + EI_{T,wall})} \right] M_{r,wall} = 848094 \text{ kip-in.} = 70675 \text{ kip-ft}
\]

**Ratio of demand to capacity:**

\[
\frac{M_{U,T,wall}}{\phi_t M_{n,T,wall}} = 0.45
\]

\[
\frac{M_{U,C,wall}}{\phi_t M_{n,C,wall}} = 0.54
\]

The effective flexural stiffnesses of tension and compression ($EI_{T,wall}$ and $EI_{C,wall}$) L-shaped C-PSW/CFs are used to calculated required flexural strengths of tension and compression walls.
Alternatively, P-M interaction diagrams of tension and compression L-shaped C-PSW/CF can be developed and compared with required flexural and axial strengths. The figure shows P-M interaction diagrams of tension and compression L-shaped C-PSW/CFs. As shown in the figure, L-shaped C-PSW/CFs can clearly resist the required axial (tension or compression) and flexural loads.
Design Of C-PSW/CFs (Shear Strength)

Wall Shear Strength:

- \( A_{s, wall} = 4 (L_W t_p) + 2(t_{sc} t_p) = (4)(144)(0.5) + (2)(16)(0.5) = 304 \text{ in.}^2 \)
- \( K_s = G_s A_{s, wall} = (11200)(304) = 3.39 \times 10^6 \text{ kips} \)
- \( K_{sc} = \frac{(0.7 (E_s A_s) (E_s A_{s, wall}))}{(4E_s A_{s, wall}) + (E_c A_c)} = 3.14 \times 10^6 \text{ kips} \)
- \( V_{n, wall} = \frac{K_s + K_{sc}}{\sqrt{3K_s^2 + K_{sc}^2}} A_{s, wall} F_Y = 14906 \text{ kips} \)
- \( \phi_v V_{n, wall} = 13416 \text{ kips} \) \( > V_{u, wall} = 2476 \text{ kips} \)
- \( \frac{V_{u, wall}}{\phi_v V_{n, wall}} = 0.19 \)
As shown in the figure, there are slots in the C-PSW/CF web plates and coupling beam flange plates are inserted into the slots.

Coupling beam flange plates are 1 in. wider than wall cross section from each side to provide adequate clearance for CJP welding.

The slots of C-PSW/CF web plates are beveled and welded to the coupling beam flange plates using complete joint penetration (CJP) welding.

The coupling beam web plates are overlapped the C-PSW/CF web plates and C-shaped fillet welding was done.

The depth of coupling beam web plate is reduced 1 in. from top and bottom at the connection region to provide adequate clearance for fillet welding.
As shown in the figure, there are slots in the C-PSW/CF web plates and coupling beam flange plates are inserted into the slots. Coupling beam flange plates are 1 in. wider than wall cross section from each side to provide adequate clearance for welding. The slots of C-PSW/CF web plates are beveled and welded to the coupling beam flange plates using complete joint penetration (CJP) welding. The depth of coupling beam web plate is reduced 1 in. from top and bottom at the connection region to provide adequate clearance for fillet welding. The coupling beam web plates are lapped over the C-PSW/CF web plates and “C” shape fillet welding was done.
As shown in the figure, there are slots in the C-PSW/CF web plates and coupling beam flange plates are inserted into the slots. Coupling beam flange plates are 1 in. wider than wall cross section from each side to provide adequate clearance for welding. The slots of C-PSW/CF web plates are beveled and welded to the coupling beam flange plates using complete joint penetration (CJP) welding. The depth of coupling beam web plate is reduced 1 in. from top and bottom at the connection region to provide adequate clearance for fillet welding. The coupling beam web plates are lapped over the C-PSW/CF web plates and “C” shape fillet welding was done.
Flange Plate Connection Demand:

- \( T_{\text{flange}} = \min\left(1.2R_yF_yA_{CB,f}, R_tF_uA_{CB,f}\right) = 594 \text{ kips} \)
- \( \frac{T_{\text{flange}}}{2} = 297 \text{ kips} \)

Required Length of CJP Welding:

- \( \frac{T_{\text{flange}}}{2} \leq \phi_d \ 0.6 \ F_y t_{CB,f} L_{\text{req.}} \)
  \( \phi_d = 1.0 \)
  \( \phi_n = 0.9 \)
- \( L_{\text{req.}} \geq \frac{T_{\text{flange}}}{2(\phi_d 0.6 F_y t_{CB,f})} = \frac{594}{2(1.0)(0.6)(50)(0.5)} = 19.8 \text{ in.} \)
- \( L_{\text{weld,fc}} = 20 \text{ in.} \)
Check Shear Strength of Coupling Beam Flange Plate

Shear yielding of coupling beam flange plate:

\[ A_{f,SY} = t_{CB,f} L_{weld,f} = (0.5)(20) = 10 \text{ in.}^2 \]
\[ \phi d 0.6 F_y A_{f,SY} = 300 \text{ kips} \geq \frac{T_{flange}}{2} = 297 \text{ kips} \]

Shear rupture of coupling beam flange plate:

\[ A_{f,SR} = t_{CB,f} L_{weld,f} = (0.5)(20) = 10 \text{ in.}^2 \]
\[ \phi_n 0.6 F_u A_{f,SR} = 351 \text{ kips} > \frac{T_{flange}}{2} = 297 \text{ kips} \]

Shear yielding and shear rupture of coupling beam flange plate are calculated and compared with required strength of flange plate connection.
Check Shear Strength of Wall Web Plates

Shear yielding of wall web plates:

- \( A_{wy} = 2 \, t_p \, L_{weld, f} = 2(0.5)(20) = 20 \text{ in.}^2 \)
- \( \phi_d \, 0.6 \, F_y \, A_{wy} = 600 \text{ kips} > \frac{T_{flange}}{2} = 297 \text{ kips} \)

Shear rupture of wall web plates:

- \( A_{wr} = 2 \, t_p \, L_{weld, f} = 2(0.5)(20) = 20 \text{ in.}^2 \)
- \( \phi_n \, 0.6 \, F_y \, A_{wr} = 702 \text{ kips} > \frac{T_{flange}}{2} = 297 \text{ kips} \)

Shear yielding and shear rupture of C-PSW/CF web plate are calculated and compared with required strength of flange plate connection.
Check Ductile Behavior of Flange Plates

In coupling beam flange plate to C-PSW/CF connection design, the available tensile rupture strength should be higher than the available tensile yield strength.

- \( A_{CB,f,g} = (b_{CB} + 2\text{ in.}) t_{CB,f} = (16 + 2)(0.5) = 9 \text{ in.}^2 \) (Gross area)
- \( A_{CB,f,n} = b_{CB} t_{CB,f} = (16)(0.5) = 8 \text{ in.}^2 \) (Net area)
- \( R_y F_y A_{CB,f,g} = (1.1)(50)(9) = 495 \text{ kips} \) (Available tension yielding capacity)
- \( R_t F_u A_{CB,f,n} = (1.2)(65)(8) = 624 \text{ kips} \) (Available tension rupture capacity)
- \( R_t F_u A_{CB,f,n} = 624 \text{ kips} > R_y F_y A_{CB,f,g} = 495 \text{ kips} \)

Shear yielding and shear rupture of C-PSW/CF web plate are calculated and compared with required strength of flange plate connection.
In coupling beam web plate to C-PSW/CF connection design, axial tension, shear force, and moment of coupling beam web plates are calculated.
Calculate Force Demand on C-Shaped Weld

\[ T_{C\text{.weld}} = \frac{T_{\text{web}}}{2} = 333 \text{ kips} \]

\[ M_{C\text{.weld}} = \frac{M_{\text{web}}}{2} = 203 \text{ kip} \cdot \text{ft} \]

\[ V_{C\text{.weld}} = \frac{V_{\text{web}}}{2} = 190 \text{ kips} \]

\[ D_{\text{min}} = \frac{3}{16} \text{ in.} \]
\[ D_{\text{max}} = \frac{5}{16} \text{ in.} \]
\[ D = \frac{5}{16} \text{ in.} \]
\[ D_{\text{min}} \leq D \leq D_{\text{max}} \]

Required forces for the design of “C” shape fillet weld are .....
Calculate Capacity of C-Shaped Weld

\[ Eccentricity = \frac{M_{C\text{web}}}{V_{C\text{web}}} = 12.85 \text{ in.} \]

\[ c. \ g. = \frac{L_{H,\text{weld.w}}^2}{2L_{H,\text{weld.w}} + L_{V,\text{weld.w}}} = \frac{30^2}{2(36) + (22)} = 10.98 \text{ in.} \]

\[ e_x = Eccentricity + (L_{H,\text{weld.w}} - c. g.) = 31.88 \text{ in.} \]

\[ k = \frac{L_{H,\text{weld.w}}}{L_{V,\text{weld.w}}} = \frac{30}{22} = 1.36 \]

\[ a = \frac{e_x}{L_{V,\text{weld.w}}} = \frac{11}{22} = 1.45 \]

\[ P_{V,\text{weld}} = \phi_n C_{8.8} C_{1-8.3} (16D)L_{V,\text{weld.w}} \]

\[ P_{V,\text{weld}} = 334 \text{ kips} > V_{C,\text{weld}} = 190 \text{ kips} \]

\[ \frac{V_{C,\text{weld}}}{P_{V,\text{weld}}} = 0.62 \]
Calculate Capacity of C-Shaped Weld

\[ P_{T,\text{weld}} = \phi_n 0.6 F_{xx} 2 L_{H,\text{weld, w}} 0.7071 D = (0.9)(0.6)(70)(2)(30)(0.7071)(5/16) \]
\[ P_{T,\text{weld}} = 501 \text{ kips} \quad > \quad T_{C,\text{weld}} = 333 \text{ kips} \]
\[ \frac{T_{C,\text{weld}}}{P_{T,\text{weld}}} = 0.67 \]

Capacity = \[ \sqrt{\left(\frac{V_{C,\text{weld}}}{P_{V,\text{weld}}}\right)^2 \left(\frac{T_{C,\text{weld}}}{P_{T,\text{weld}}}\right)^2} = \sqrt{(0.64)^2 (0.67)^2} = 0.91 \leq 1 \]
This slide is intended to initiate questions from participants.
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Chapter 6 Cross-Laminated Timber (CLT) Shear Walls

2020 NEHRP Provisions Training Materials
Philip Line, PE, American Wood Council
M. Omar Amini, PhD, Simpson Strong-Tie
6.1 Overview - Cross-Laminated Timber (CLT) Shear Wall Example

- This example features the seismic design of cross-laminated timber shear walls used in a three-story, six-unit townhouse cross-laminated timber building of platform construction.

- The CLT shear wall design in this example includes:
  - Check of CLT shear wall shear strength
  - Check of CLT shear wall hold-down size and compression zone length for overturning
  - Check of CLT shear wall deflection for conformance to seismic drift
6.1 Overview - Useful Design Aid Resources

The following documents are used in this example:

6.2 Background

- NEHRP (2020a) proposed additions for ASCE/SEI 7-22 Table 12.2-1 featuring cross-laminated timber (CLT) shear walls

<table>
<thead>
<tr>
<th>Seismic Force-Resisting System</th>
<th>Detailing Requirements, ASCE/SEI 7-22 Section</th>
<th>$R$</th>
<th>$\Omega_0$</th>
<th>$C_d$</th>
<th>Structural System Limitations Including Structural Height, $h_n$ (ft) Limits$^a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. BEARING WALL SYSTEMS</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Seismic Design Category</td>
</tr>
<tr>
<td>Cross laminated timber shear</td>
<td></td>
<td>14.5</td>
<td>3</td>
<td>3</td>
<td>65</td>
</tr>
<tr>
<td>walls</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>C</td>
</tr>
<tr>
<td>Cross laminated timber shear</td>
<td></td>
<td>14.5</td>
<td>4</td>
<td>3</td>
<td>65</td>
</tr>
<tr>
<td>walls with shear resistance</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>D</td>
</tr>
<tr>
<td>provided by high aspect ratio</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>E</td>
</tr>
<tr>
<td>panels only</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>F</td>
</tr>
</tbody>
</table>
6.2 Background

- **Cross-laminated timber (CLT)**
  - Usually 3, 5 or 7 layers of dimension lumber stacked in alternating directions and bonded together with adhesive
  - Research and development for CLT began in the early 1990s in Europe
  - The first production facilities established in 1994 in Austria, Germany and Switzerland
  - The term coined in 2000 at the COST E5 conference in Italy
6.2 Background

- Cross-laminated timber (CLT)
  - Stadthaus, London, 2009
  - Residential
  - 9 stories
  - 9 weeks of CLT construction
  - 4 laborers
  - 1 supervisor

Photo credit: Will Pryce
6.2 Background

Photo credit: Will Pryce
6.2 Background

Photo credit: Will Pryce
6.2 Background

Ft. Drum, NY (4-story), 2017; Courtesy Jeff Morrow, Lendlease
6.2 Background

- FEMA P695, *Quantification of Building Seismic Performance Factors*
  - Peer review throughout
  - Archetypes
  - Design methodology
  - Nonlinear time history analysis
  - Performance evaluation (CMR & ACMR)

6.2 Background

Note: Scaled results
6.2 Background
6.2 Background
6.2 Background
6.3 Cross-Laminated Timber Shear Wall Example Description

- A three-story, six-unit townhouse cross-laminated timber building of platform construction
- The CLT shear wall design in this example includes:
  - Check of CLT shear wall shear strength
  - Check of CLT shear wall hold-down size and compression zone length for overturning
  - Check of CLT shear wall deflection for conformance to seismic drift
### 6.3 Cross-Laminated Timber Shear Wall Example Description

Table 6-1: Weights of Roof/Ceiling, Floors, and Walls

<table>
<thead>
<tr>
<th>Item</th>
<th>Description</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof/Ceiling</td>
<td>Light-frame roof, gypsum board ceiling, roofing, insulation</td>
<td>25 psf</td>
</tr>
<tr>
<td>Floor</td>
<td>5-layer CLT (6.875 in. thick), gypsum board ceiling, flooring. Includes 8 psf of floor area for wall partitions</td>
<td>35 psf</td>
</tr>
<tr>
<td>Interior Walls</td>
<td>3-layer CLT (4.125 in. thick), light-frame wall, gypsum board finish, sound insulation</td>
<td>20 psf</td>
</tr>
<tr>
<td>Exterior Walls</td>
<td>3-layer CLT (4.125 in. thick), light-frame wall, gypsum board interior finish, stucco exterior, insulation</td>
<td>30 psf</td>
</tr>
</tbody>
</table>
# 6.3 Cross-Laminated Timber Shear Wall Example Description

Table 6-3: Design Coefficients and Factors for CLT Seismic Force-Resisting Systems (ASCE/SEI 7-22)

<table>
<thead>
<tr>
<th>Seismic Force-Resisting System</th>
<th>Detailing Requirements, ASCE/SEI 7-22 Section</th>
<th>$R$</th>
<th>$\Omega_0$</th>
<th>$C_\alpha$</th>
<th>Structural Height, $h_n$, Limit Seismic Design Category B, C, D, E &amp; F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross-laminated timber shear walls</td>
<td></td>
<td>14.5</td>
<td>3</td>
<td>3</td>
<td>65 feet</td>
</tr>
<tr>
<td>Cross-laminated timber shear walls with shear resistance provided by high aspect ratio panels only</td>
<td></td>
<td>14.5</td>
<td>4</td>
<td>3</td>
<td>4</td>
</tr>
</tbody>
</table>
6.4 Seismic Forces

- Seismic base shear calculation assumptions:
  - $S_{DS} = 1.0$
  - $I_e = 1.0$
  - $R = 3$ (for CLT shear walls)

- Seismic base shear, $V$, per ASCE/SEI 7-22 Equation 12.8-2 (for short-period structures):
  \[
  V = C_s W = \frac{S_{DS}}{(R/I)} W = \frac{1.0}{(3.0/1.0)} W = 0.333 W \text{ kips}
  \]

- The portion of base shear tributary to the CLT shear walls of interest is:
  \[
  V_{(\text{Line 4})} = 42.3 \text{ kips}
  \]
6.4 Seismic Forces

Table 6-4: Summary of Cumulative Lateral Seismic Force and Unit Shear Force per Story (Along Line 4)

<table>
<thead>
<tr>
<th>Story</th>
<th>Lateral force, $V_x$ (kips)</th>
<th>Unit Shear Force per Foot of Shear Wall Length (plf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>15.9</td>
<td>477</td>
</tr>
<tr>
<td>2</td>
<td>33.5</td>
<td>1,009</td>
</tr>
<tr>
<td>1</td>
<td>42.3</td>
<td>1,273</td>
</tr>
</tbody>
</table>

Figure 6-4. Vertical Distribution of Seismic Force and Dead Load Tributary to the CLT Shear Walls Located Along Line 4

$V_{(Line\ 4)} = 42.3 \text{ kips}$
6.5.1 Shear Capacity of Prescribed Connectors

- LRFD design unit shear capacity for seismic:

\[ v_{s\text{(seismic)}} = \phi(n) \left( \frac{2605}{b_s} \right) C_G \]  

(SDPWS-21 Eq. B.5)

where:

- \( n \) = number of angle connectors along bottom of panel face
- 2,605 = connector nominal shear capacity (lb)
- \( b_s \) = individual CLT panel length (ft)
- \( C_G \) = CLT panel specific gravity factor which equals 1.0 for \( G \geq 0.42 \) specific gravity panels used in this example, and
- \( \phi \) = resistance factor equal to 0.5 for seismic design

From SDPWS Figure C-B.1
### 6.5.1 Shear Capacity of Prescribed Connectors

Table 6-5: CLT Shear Wall Connectors and LRFD Design Unit Shear Capacity

<table>
<thead>
<tr>
<th>Story</th>
<th>Panel thickness</th>
<th>Panel length, $b_s$</th>
<th>Panel height, $h$</th>
<th>Number of connectors per panel at top and bottom panel edge</th>
<th>Number of connectors at each adjoining vertical panel edge</th>
<th>$V_{r,n}$ Nominal unit shear capacity, $(n \cdot 2605)/b_s$</th>
<th>$V_{s(seismic)}$, LRFD design unit shear capacity, $(\phi = 0.5)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>4.125 (in.)</td>
<td>4.75 (ft)</td>
<td>9.5 (ft)</td>
<td>2</td>
<td>4</td>
<td>1,096 (plf)</td>
<td>548 (plf)</td>
</tr>
<tr>
<td>2</td>
<td>4.125 (in.)</td>
<td>4.75 (ft)</td>
<td>9.5 (ft)</td>
<td>4</td>
<td>8</td>
<td>2,193 (plf)</td>
<td>1,097 (plf)</td>
</tr>
<tr>
<td>1</td>
<td>4.125 (in.)</td>
<td>4.75 (ft)</td>
<td>9.5 (ft)</td>
<td>5</td>
<td>10</td>
<td>2,742 (plf)</td>
<td>1,371 (plf)</td>
</tr>
</tbody>
</table>
6.5.1 Shear Capacity of Prescribed Connectors

Figure 6-6. Wall-floor Intersections

Figure 6-5. CLT Shear Walls at 1st, 2nd, and 3rd Story with Connector And Hold-down Location
6.5.2 Shear Capacity of CLT Panel

- For this 3-layer E1 grade CLT panel, the allowable stress design (ASD) in-plane shear unit shear capacity is converted to LRFD using NDS-2018 Table 10.3.1:

\[ v'_r = \phi \lambda K_F F_v(t_v) = 0.75(1.0)(2.88)(9,700) = 20,849 \text{ plf} \]

where:

- \( F_v(t_v) = 9,700 \text{ plf} \) (ASD value from CLT panel manufacturer’s evaluation report)

- CLT panel in-plane unit shear capacity, \( v'_r = 20,849 \text{ plf} \) is greater than the largest unit shear force story demand of 1,273 plf (from Table 6-4)

\[ 20,849 \text{ plf} \gg 1,273 \text{ plf} \]

- In-plane unit shear capacity value does not account for holes, cuts or other modifications
6.6.1 CLT Shear Wall Hold-down Design

Figure 6-1. Illustration of Rocking Behavior of Seven Individual Panels in A Multi-panel CLT Shear Wall Designed in Accordance with SDPWS-21 Appendix B

Figure 6-7. Free-body Diagram for the Tension End Panel of the CLT Multi-panel Shear Wall
6.6.1 CLT Shear Wall Hold-down Design

- \( \sum M_o = 0 \)

- \( T \left( b_{eff} \right) - \nu b_s h + \omega b_s \left( \frac{b_s}{2} \right) - T_T \left( b_{eff} \right) = 0 \) \hspace{1cm} \text{(SDPWS-21 Eq. C-B.1)}

- \( T = \frac{\nu b_s h - \omega b_s \left( \frac{b_s}{2} \right)}{b_{eff}} + T_T \) \hspace{1cm} \text{(SDPWS-21 Eq. C-B.2)}

Table 6-6: Solution for Tension Force, T, for Hold-down Strength Requirement

<table>
<thead>
<tr>
<th>Story</th>
<th>Unit shear force per foot of shear wall length</th>
<th>( v_s(\text{seismic}) ) LRFD design unit shear capacity, ( \phi = 0.5 )</th>
<th>2 x ( v_s(\text{seismic}) )</th>
<th>( T_T ) from story above</th>
<th>T for 2 x ( v_s(\text{seismic}) ) requirement for load combination 1.0E - 0.7D</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>(plf)</td>
<td>(plf)</td>
<td>(plf)</td>
<td>(lb)</td>
<td>(lb)</td>
</tr>
<tr>
<td></td>
<td>477</td>
<td>548</td>
<td>1,097</td>
<td>0</td>
<td>11,293</td>
</tr>
<tr>
<td>2</td>
<td>1,009</td>
<td>1,097</td>
<td>2,194</td>
<td>11,293</td>
<td>34,540</td>
</tr>
<tr>
<td>1</td>
<td>1,273</td>
<td>1,371</td>
<td>2,742</td>
<td>34,540</td>
<td>63,968</td>
</tr>
</tbody>
</table>
6.6.1 CLT Shear Wall Hold-down Design

- The same screw attached hold-down is used for all locations with each having an LRFD design tension capacity of 17,678 lb and associated deflection of 0.253 in.
  - 1st story walls to foundation, four hold-downs
    - $4 \times 17,687 \text{ lb} = 70,748 \text{ lb} > 63,968 \text{ lbs}$
  - 2nd story to top of 1st story, four hold-downs
    - $4 \times 17,687 \text{ lb} = 70,748 \text{ lb} > 34,540 \text{ lbs}$
  - 3rd story to top of 2nd story, two hold-downs
    - $2 \times 17,687 \text{ lb} = 37,374 \text{ lb} > 11,293 \text{ lbs}$
- Check CLT panel for tension, row and group tear out
6.6.1 CLT Shear Wall Hold-down Design

- From SDPWS-21 Section B.3.4, hold-down device deformation for each story shall not exceed 0.185 in. for T forces from strength design load combinations (see Table 6-7).

<table>
<thead>
<tr>
<th>Story</th>
<th>Unit shear force per foot of shear wall length (plf)</th>
<th>$T$ for load combination $1.0E - 0.7D$ (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>477</td>
<td>4,714</td>
</tr>
<tr>
<td>2</td>
<td>1,009</td>
<td>14,604</td>
</tr>
<tr>
<td>1</td>
<td>1,273</td>
<td>27,472</td>
</tr>
</tbody>
</table>

- Deflection of most highly loaded hold-down is less than 0.185 in. The SDPWS-21 deflection limit is satisfied.

$$\Delta_{hold-down} = \frac{27,472 \text{ lb}}{4(17,678 \text{ lb})} (0.253 \text{ in.}) = 0.098 \text{ in.} < 0.185 \text{ in.}$$
6.6.2 CLT Shear Wall Compression Zone

- Compression force, $C$, and length of compression zone, $x$, from compression end panel moment equilibrium

\[ \sum M_o = 0 \]
\[ C \left( b_s - \frac{x}{2} \right) - v b_s h - w b_s \left( \frac{b_s}{2} \right) - C_T \left( b_s - \frac{x_T}{2} \right) = 0 \]  
(SDPWS-21 Eq. C-B.3)

\[ C = F_{C\perp} \left( t \right) \left( x \right) \left( \frac{12 \text{in}}{\text{ft}} \right) \]  
(SDPWS-21 Eq. C-B.4)

\[ C = Fc' \left( t_{\text{parallel}} \right) \left( x \right) \left( \frac{12 \text{in}}{\text{ft}} \right) \]  
(SDPWS-21 Eq. C-B.5)

- $C$ and $x$ summarized in Table 6-8

Figure 6-8. Free-body Diagram for the Compression End Panel of the CLT Multi-panel Shear Wall
### 6.6.2 CLT Shear Wall Compression Zone

Table 6-8: Solution for Compression Zone Length, $x$, and Force $C$

<table>
<thead>
<tr>
<th>Story</th>
<th>Unit shear force per foot of shear wall length</th>
<th>Dead load, $w_{DL}$ (plf)</th>
<th>Live load, $w_{LL}$ (plf)</th>
<th>$C_T$, Compression from top (lb)</th>
<th>Compression zone length, $x$, (in.)</th>
<th>$C$, for load combination $1.0E + 1.4D + 0.5L$, (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>(plf)</td>
<td>477</td>
<td>190</td>
<td>0</td>
<td>2.00</td>
<td>5,257</td>
</tr>
<tr>
<td>2</td>
<td>1,009</td>
<td>793</td>
<td>690</td>
<td>5,257</td>
<td>7.64</td>
<td>20,144</td>
</tr>
<tr>
<td>1</td>
<td>1,273</td>
<td>793</td>
<td>690</td>
<td>20,144</td>
<td>4.56</td>
<td>36,545</td>
</tr>
</tbody>
</table>

- Check of CLT wall panel axial capacity is required
### 6.7 CLT Shear Wall Deflection

- \( \delta_{SW} = \frac{576vb_s h^3}{E_{eff(in-plane)}} + \frac{vh}{G_{A eff(in-plane)}} + 3 \Delta_{nail slip,h} + 2 \Delta_{nail slip,v} \left( \frac{h}{b_s} \right) + \Delta a \frac{h}{\sum b_s} \)

---

<table>
<thead>
<tr>
<th>Total shear wall = deflection, ( \delta_{SW} )</th>
<th>Panel bending and shear +</th>
<th>Sliding +</th>
<th>Panel rotation +</th>
<th>Rigid body rotation</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1.png" alt="Diagram" /></td>
<td><img src="image2.png" alt="Diagram" /></td>
<td><img src="image3.png" alt="Diagram" /></td>
<td><img src="image4.png" alt="Diagram" /></td>
<td><img src="image5.png" alt="Diagram" /></td>
</tr>
</tbody>
</table>
6.7 CLT Shear Wall Deflection

Table 6-9: CLT Shear Wall Deflection Components and Total Shear Wall Deflection, $\delta_{SW}$

<table>
<thead>
<tr>
<th>Story</th>
<th>$\frac{576vb_s h^3}{EI_{eff,(in–plane)}}$ (in.)</th>
<th>$\frac{vh}{G A_{eff,(in–plane)}}$ (in.)</th>
<th>$3\Delta_{nail , slip, h} + 2\Delta_{nail , slip, v} \left(\frac{h}{b_s}\right)$ (in.)</th>
<th>$\Delta_a \frac{h}{\sum b_s}$ (in.)</th>
<th>$\delta_{SW}$, shear wall deflection (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>0.02</td>
<td>0.04</td>
<td>0.15</td>
<td>0.04</td>
<td>0.24</td>
</tr>
<tr>
<td>2</td>
<td>0.04</td>
<td>0.09</td>
<td>0.16</td>
<td>0.04</td>
<td>0.33</td>
</tr>
<tr>
<td>1</td>
<td>0.05</td>
<td>0.11</td>
<td>0.16</td>
<td>0.03</td>
<td>0.35</td>
</tr>
</tbody>
</table>

- For allowable story drift limit is 2.5%$h$ from ASCE/SEI 7-22 Table 12.12-1, corresponding allowable deflection calculated using, $C_d$, equal to 3 for cross-laminated timber shear walls:

$$\delta_e = \frac{0.025(h)}{C_d} = \frac{0.025(114 \, \text{in.})}{3.0} = 0.95 \, \text{in.} > \delta_{SW}$$
6.8 References

- See the “Useful Design Aid Resources” in Section 6.1 for additional references.
Questions
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Welcome to 2020 NEHRP Provisions training, Chapter 7 addressing Horizontal diaphragm design. This training is presented in two parts.

Part 1 provides an introduction and presents an example problem with a multi-story steel building with steel deck diaphragms. Part 1 is anticipated to take approximately 90 minutes to present including question and answer sessions.

Part 2 presents an example problem with a single-story large-box building with tilt-up walls and a flexible diaphragm. Part 2 is anticipated to take approximately 60 minutes to present, including a question-and-answer session.

Both examples incorporate the 2020 NEHRP Previsions as incorporated into ASCE/SEI 7-22.
What's New in Diaphragm Design Provisions

- **ASCE/SEI 7-10**
  - Sections 12.10.1 and 12.10.2 - *Traditional Diaphragm Design Method*

- **ASCE/SEI 7-16 (2015 NEHRP Provisions)**
  - Section 12.10.3 - *Alternative Design Provisions* is added
    - Cast-in-place concrete, precast concrete, and wood structural panel diaphragms

- **ASCE/SEI 7-22 (2020 NEHRP Provisions)**
  - Section 12.10.3 – *Alternative Design Provisions* is expanded
    - Bare steel deck, concrete-filled steel deck diaphragms
  - Section 12.10.4 – *Alternative RWFD Provisions* is added

This training on seismic design of horizontal diaphragms is important because there have been significant changes to diaphragm design provisions between ASCE 7-10 and ASCE 7-16 and again between ASCE 7-16 and ASCE 7-22.

Very significant changes include the addition and expansion of diaphragm design methods as follows:

ASCE 7-16 included the addition of the diaphragm alternative design provisions.

ASCE 7-22 included the expansion of the diaphragm alternative design provisions to address bare steel deck diaphragms and concrete-filled steel deck diaphragms.

ASCE 7-22 also included the addition of a diaphragm design method for one-story rigid wall flexible diaphragm structures.
What’s New in Diaphragm Design Provisions

- **ASCE/SEI 7-16 (2015 NEHRP Provisions)**
  - Definition of diaphragm transfer forces
  - Amplification of transfer forces by $\Omega_0$ for horizontal structural irregularity type 4

- **ASCE/SEI 7-22 (2020 NEHRP Provisions)**
  - Introduction of special seismic detailing provisions for bare steel deck diaphragms
  - Differentiation of design provisions for diaphragms meeting or not meeting the special seismic detailing provisions

From a more detailed perspective,

ASCE 7-16 added the definition for diaphragm transfer forces and designated some circumstances in which transfer forces are to be amplified by a $W_0$ factor.

ASCE 7-22, in concert with AISI S400 have added special seismic detailing provisions for bare steel deck diaphragms.
Why Are Diaphragm Design Provisions Changing?

- Driven by research including both testing and numerical studies
- To better reflect diaphragm dynamic response
- To better reflect diaphragm deformation capacity
- Thought to provide better diaphragm performance at the same or potentially lower cost
- More detail later...

It is reasonable to ask why diaphragm design provisions have been significantly changed. The reasons include the following:
Diaphragm Design Presentation Outline – Part 1

- What's new in 2020 NEHRP Provisions and ASCE/SEI 7-22
- Overview of horizontal diaphragm design
- Diaphragm seismic design methods
- Example multi-story steel building with steel deck diaphragms
  - Section 12.10.1 and 12.10.2 Traditional Design Method
  - Section 12.10.3 Alternative Design Method
  - Comparison of results

Part 1 of this presentation will cover the following...
Diaphragm Design Presentation Outline – Part 2

- Example one-story RWFD building with steel deck diaphragm
  - Section 12.10.1 and 12.102 *Traditional Design Method*
  - Section 12.10.4 *Alternative RWFD Design Method*
  - Comparison of results

Part 2 of this presentation will cover the following...
Overview of Diaphragm Design

We will start with a high-level reminder of the process of diaphragm design.
Overview of Diaphragm Design

This diagram illustrates a simple box building subject to seismic ground motion. Inertial forces are generated by the weight of the structure. The horizontal diaphragm acts like a beam to transmit the horizontal inertial forces generated by its weight to the top of the walls or other vertical elements of the SFRS.
Overview of Diaphragm Design

1. Determine base shear, $V$, and vertical distribution of $F_x$ forces
2. Categorize diaphragm for purposes of design: Idealized as flexible, Idealized as rigid. Calculated as flexible, Modeled as semi-rigid (or semi-flexible)
3. Apply $F_x$ forces to model and evaluate inherent and accidental torsion (rigid and semi-rigid diaphragms) and transfer forces (all diaphragms)

At a high level, the design of diaphragms starts with the following”
1. Determining the base shear, $V$, and distributing the base shear vertically to determine $F_x$ forces at each level.
2. Categorizing the diaphragm for purposes of seismic force distribution
3. Apply....
Overview of Diaphragm Design

4. Determine diaphragm $F_{px}$ forces at each story and adjust shear, chord and collector forces from $F_x$ force to $F_{px}$ force level.

Following the vertical distribution of $F_x$ forces used to design the vertical elements of the SFRS, a second vertical distribution is used to determine the $F_{px}$ forces used for design of diaphragms.
Once the $F_{px}$ forces are determined, these are used to:
5. Design the diaphragm for shear and flexure, and
6. Design the chords, collectors and collector connections to vertical elements.

Finally applicable deflection or drift provisions need to be checked. These generally look at the story drift or deflection of the full structure, including both vertical and horizontal elements.

The figure on the right illustrates the determination of shear, flexure and chord forces for a flexible diaphragm modeled as a simple-span beam. Where diaphragms are not idealized as flexible, shear flexure and other seismic demand information is extracted from the analysis model.
Overview of Diaphragm Design – Transfer Forces

ASCE/SEI 7-22 Section 11.2:

- Transfer Forces, Diaphragm:
  Forces that occur in a diaphragm caused by transfer of seismic forces from the vertical seismic force-resisting elements above the diaphragm to other vertical seismic force-resisting elements below the diaphragm because of offsets in the placement of the vertical elements or changes in the relative stiffness of the vertical elements.

In one of the introductory slides, it was noted that provisions regarding diaphragm transfer forces were added in ASCE 7-16. On the left the ASCE 7 definition of transfer forces is shown. On the right are two common ways in which diaphragm transfer forces are generated.

- The left figure shows a horizontal offset between shear walls in the first and second stories. The transfer forces occur because forces at the base of the second-story wall need to move through the floor diaphragm to get to first-story shear walls; these forces are diaphragm transfer forces.
- The right figure shows a shear wall of significantly reduced width at the lowest floor. With a rigid or semi-rigid diaphragm assumption, forces from the second floor above will be distributed between the first-story shear walls. Transfer forces are created by the seismic forces that travel through the diaphragm to the left hand first-story wall.
The construction of diaphragms and their design for shear, flexure, chords and collectors can vary substantially between diaphragm systems, as seen in the NEHRP Tech Brief photos above. These detailed aspects of design will not be addressed in this training, but the derivation of the seismic forces used for design will be, along with detailing provisions for some diaphragm systems.
Overview of Diaphragm Design - NEHRP Diaphragm Tech Briefs

- NIST, 2011. NEHRP Seismic Design Technical Brief No. 5, Seismic Design of Composite Steel Deck and Concrete-filled Diaphragms (NIST GRC 11-917-10), National Institute of Standards and Technology, Gaithersburg, MD.
- NIST, 2016a. NEHRP Seismic Design Technical Brief No. 12, Seismic Design of Cold-Formed Steel Lateral Load-Resisting Systems (NIST GRC 16-917-38), National Institute of Standards and Technology, Gaithersburg, MD.
- NIST, 2016b. NEHRP Seismic Design Technical Brief No. 3, Seismic Design of Cast-in-Place Concrete Diaphragms, Chords and Collectors, Second Edition (NIST GRC 16-917-42), National Institute of Standards and Technology, Gaithersburg, MD.
- NIST, 2017. NEHRP Seismic Design Technical Brief No. 12, Seismic Design of Precast Concrete Diaphragms (NIST GRC 17-917-47), National Institute of Standards and Technology, Gaithersburg, MD.

The above NEHRP Tech Notes are recommended for viewers interested in the details of diaphragm design on a system-by-system basis. While these do not incorporate all of the changes from ASCE 7-16 and 7-22, they provide valuable information on the details of diaphragm design.
Diaphragm Seismic Design Methods
ASCE/SEI 7-22
Diaphragm Seismic Design Methods

1. Section 12.10.1 and 12.10.2 Traditional Design Method
2. Section 12.10.3 Alternative Design Method
3. Section 12.10.4 Alternative “RWFD” Design Method:
   - Alternative Diaphragm Design Provisions for One-Story Structures with Flexible Diaphragms and Rigid Vertical Elements

- Scope: Diaphragms, Chords and Collectors
  - Design forces
  - In some instances, detailing

As mentioned in the introduction, as of ASCE 7-22, there are now three methods by which seismic design of diaphragms can be provided. These three methods are ....

It is important to note that the Section 12.10.1 and 12.10.2 traditional diaphragm design provisions still remain and are still permitted with the exception of precast concrete diaphragms in SDC C through F. The two added methods will give the design a choice of methods.
## Diaphragm Seismic Design Methods

<table>
<thead>
<tr>
<th>Method and ASCE/SEI 7 22 Section</th>
<th>Number of Stories Permitted</th>
<th>Diaphragm Systems Included</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traditional Sections 12.10.1 and 12.10.2</td>
<td>Any</td>
<td>All</td>
<td>▪ Not permitted for precast concrete diaphragms in SDC C through F</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>▪ Diaphragm design forces are determined using seismic design parameters ($R$, $\Omega_0$, and $C_d$) for the vertical SFRS</td>
</tr>
</tbody>
</table>

The above is excerpted from a table in Chapter 7 of the example problems publication and gives an overview of each of the three diaphragm design methods, with the intent of helping the designer decide which method to use.

The first row addresses the Sections 112.10.1 and 12.10.2 Traditional Provisions as follows:

......
### Diaphragm Seismic Design Methods

<table>
<thead>
<tr>
<th>Method and ASCE/SEI 7 22 Section</th>
<th>Number of Stories Permitted</th>
<th>Diaphragm Systems Included</th>
<th>Comments</th>
</tr>
</thead>
</table>
| Alternative Section 12.10.3      | Any                        | ▪ Cast-in-place concrete  
▪ Precast concrete  
▪ Wood structural panel  
▪ Bare steel deck  
▪ Concrete-filled metal deck | ▪ Required for precast concrete diaphragms in SDC C through F, providing improved seismic performance  
▪ Optional for other diaphragm types  
▪ Better reflects vertical distribution of diaphragm forces  
▪ $R_s$ diaphragm design force reduction factor better reflects effect of diaphragm ductility and displacement capacity on diaphragm seismic forces  
▪ Forces in collectors and their connections to vertical elements are amplified by 1.5 in place of $\Omega_0$ |

The second row addresses the Section 12.10.3 Alternative Provisions as follows:

......
Diaphragm Seismic Design Methods

<table>
<thead>
<tr>
<th>Method and ASCE/SEI 7-22 Section</th>
<th>Number of Stories Permitted</th>
<th>Diaphragm Systems Included</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alternative RWFD Section 12.10.4</td>
<td>One Story</td>
<td>▪ Wood structural panel</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>▪ Bare steel deck</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>▪ Diaphragm must meet</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>scoping limitations of</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>ASCE/SEI 7-22 Section 12.10.4.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>▪ Primarily intended for</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>buildings with</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>diaphragm spans of 100 feet</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>or greater</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>▪ New $T_{\text{diaph}}$, $R_{\text{diaph}}$, $\Omega_{\text{diaph}}$, and $C_{\text{d-diaph}}$, better reflect response of</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>RWFD building type</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>▪ Provides better performance with</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>the same or reduced construction cost</td>
<td></td>
</tr>
</tbody>
</table>

The third row addresses the Section 12.10.4 Alternative RWFD Provisions as follows:

......
Diaphragm Seismic Design Methods

- Advantages of using Section 12.10.3 Alternative Design Provisions:
  - Better reflects vertical distribution of diaphragm forces
  - Better reflects effect of diaphragm ductility and displacement capacity
  - May result in lower seismic demands

- Advantages of using Section 12.10.4 Alternative RWFD Method:
  - Better reflects seismic response of RWFD buildings
  - May result in lower seismic demands
  - Is anticipated to result in better performance

- When will the Section 12.10.1 and 12.10.2 Traditional Method result in lower design forces?
  - Bare steel deck diaphragms not meeting the AISI S400 special seismic detailing provisions
  - Other

It is worthwhile to consider some of the advantages of design in accordance with the newer diaphragm design provisions ....
The next few slides will provide a brief introduction to the Section 12.10.3 Alternative Design Provisions.

There are two primary parts to the changes incorporated into the Alternative Design Provisions:
- The first part, seen on the left, involved derivation and vertical distribution of diaphragm seismic forces based on an assumption of near-elastic diaphragm response. This was driven by a collection of a significant body of diaphragm force information from testing and numerical studies.
- The second part, seen on the right, involved the derivation of Rs diaphragm design force reduction factors, serving to reduce the near-elastic diaphragm forces based on ductility and deformation capacity of the diaphragm system. These also were derived from testing and numerical studies.

\[
F_{px} = \frac{C_{px}}{R_s} \cdot w_{px}
\]
Introduction to Section 12.10.3 Alternative Design Provisions

Studies Behind Alternative Provisions Diaphragm Forces and Tabulated $R_s$ factors:

- Precast concrete diaphragms
  - 2020 NEHRP Provisions Commentary
- Concrete diaphragms - 2020 NEHRP Provisions Commentary
- Wood structural panel diaphragms – 2020 NEHRP Provisions Commentary

The above notes some of the studies behind these changes in the diaphragm design provisions.
Introduction to Section 12.10.3 Alternative Design Provisions

Studies Behind Tabulated $R_s$ factors:

- **Bare steel deck diaphragms and concrete filled metal deck diaphragms**

The above notes more of the studies behind these changes in the diaphragm design provisions.
This slide shows some of the data that was considered in the derivation of the alternative seismic design forces and their vertical distribution. See the commentary to the 2020 NEHRP Provisions for a detailed discussion.
### Introduction to Section 12.10.3 Alternative Design Provisions – Part 2

<table>
<thead>
<tr>
<th>Diaphragm System</th>
<th>$R_s$ Shear Controlled*</th>
<th>$R_s$ Flexure Controlled*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cast-in-place concrete designed in accordance with ACI 318</td>
<td>-</td>
<td>1.5</td>
</tr>
<tr>
<td>Precast concrete designed in accordance with ACI 318</td>
<td>Elastic design option</td>
<td>0.7</td>
</tr>
<tr>
<td></td>
<td>Basic design option</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Reduced design option</td>
<td>1.4</td>
</tr>
<tr>
<td>Wood sheathed designed in accordance with ASCE/SEI 7-22 Section 14.5 and AWC Special Design Provisions for Wind and Seismic</td>
<td>-</td>
<td>3.0</td>
</tr>
<tr>
<td>Bare steel deck designed in accordance with ASCE/SEI 7-22 Section 14.1.5</td>
<td>With special seismic detailing</td>
<td>2.5</td>
</tr>
<tr>
<td></td>
<td>Other</td>
<td>1.0</td>
</tr>
<tr>
<td>Concrete-filled metal deck designed in accordance with ASCE/SEI 7-22 Section 14.1.6</td>
<td>-</td>
<td>2.0</td>
</tr>
</tbody>
</table>

The above table shows the diaphragm seismic force reduction factors, $R_s$, as incorporated into ASCE 7-22. The top three diaphragm systems were included in ASCE 7-16. The bottom two diaphragm systems were added in ASCE 7-22, addressing base steel deck diaphragms and concrete-filled steel deck diaphragms.
The next few slides will provide a brief introduction to the Section 12.10.4 Alternative RWFD Design Provisions.

It has long been recognized that large footprint buildings with rigid vertical elements and flexible diaphragms have seismic response driven primarily by the flexible vertical diaphragm rather than the rigid vertical elements.

The traditional diaphragm design method ignores this behavior, calculating the diaphragm seismic forces as a function of the system used for the vertical elements.

The Alternative RWFD design method incorporates this known behavior into diaphragm design.
Introduction to Section 12.10.4 Alternative RWFD Design Method

Studies Behind Alternative RWFD Design Method:


As was true in the Section 12.10.3 Alternative Design Provisions, the Section 12.101.4 Alternative RWFD Design Provisions also have significant research serving as the basis, as seen in the references shown above.
In addition to developing more realistic seismic design forces, the Alternative RWFD provisions incorporate a method of achieving better seismic performance. This is accomplished by designing a boundary zone for amplified shears. This reinforces the diaphragm zones alongside the vertical elements that see the highest forces and would otherwise experience very high inelastic demands. As a result of this, numerical studies have demonstrated that inelastic behavior will be better distributed through the interior portions of the diaphragm.
A second, optional portion of the Alternative RWFD provisions allow the designer to use a two-stage analysis method for the vertical elements of the seismic force resisting system. This method sums for design of the wall the forces from the diaphragm based on flexible diaphragm response with the forces from mass tributary to the wall considering rigid wall response, as shown in the diagram above.
Example Multi-Story Steel Building with Steel Deck Diaphragms

Next, we will go through a design example using a multi-story steel building with steel deck diaphragms. The slides will follow in detail the design example found in Section 7.5 of the 2020 NEHRP example problems publication. The diaphragm seismic design forces will first be calculated using the Section 12.10.1 and 12.10.2 Traditional Method. The diaphragm seismic design forces will then be calculated using the Section 12.10.3 Alternative Method.

This example focuses on an ELF seismic design. Some variations would occur in a modal response spectrum design were to be used.
This slide shows the plan and an elevation of the six-story building that we will use. The plan shows columns and shows steel concentric braced frames on all four exterior walls. Elevation AA shows the elevation of the right-hand side exterior framing bay. Included are super-X braced frames in the center two framing bays.
Example Multi-Story Steel Building with Steel Deck Diaphragms

Building Configuration
- Six stories
- Risk Category II, $I_e = 1.0$
- Mean roof height = 72 feet - six stories at 12 feet each
- Length = 150 feet
- Width = 120 feet
- $S_{d_2} = 1.2, S_{d_1} = 0.70$ (determined using ASCE/SEI 7-22 Section 11.4.4)
- Floor Diaphragm: Concrete-filled metal deck
- Roof Diaphragm: Bare steel deck
- Steel special concentrically braced frame system - $R = 6, \Omega_0 = 2$
- $\rho_v, \rho = 1.0$ for both vertical elements and diaphragm
- All seismic forces are at strength level

This slide identifies some of the configuration and seismic design parameters for this building.
Example Multi-Story Steel Building with Steel Deck Diaphragms

Building Analytical Modeling

- The step-by-step descriptions in this presentation focus on use of the ASCE/SEI 7-22 equivalent lateral force (ELF) procedure; some modifications are needed when using linear dynamic analysis procedures.
- This step-by-step description also focuses primarily on diaphragm inertial forces due to the mass tributary to each diaphragm level. Where diaphragm transfer forces as defined in ASCE/SEI Section 11.2 occur, they are required to be addressed in accordance with ASCE/SEI 7-22 Section 12.10.1.1 or 12.10.3.3, as applicable.

Before discussing details of diaphragm design forces, some discussion of analytical modeling is appropriate.
In order to perform seismic analysis of the SFRS and diaphragms, it is necessary to define the diaphragm flexibility in accordance with ASCE/SEI 7-22 Section 12.3. This section sets criteria by which diaphragms can be idealized as flexible, idealized as rigid, or calculated as flexible. Where these do not apply, the diaphragm is required to be modeled as semi-rigid.

Where diaphragms are designated as rigid or semi-rigid for modeling and design, the process of seismic design will start with overall modeling of the building and then proceed to diaphragm design. Regardless of diaphragm designation, the seismic design of the diaphragm and vertical elements usually proceed in parallel.

Further comments on analytical modeling include...
Example Multi-Story Steel Building with Steel Deck Diaphragms

**Step 1 - Weight for Seismic Analysis**

- Roof + ceiling = 40 psf
- Floor + ceiling = 80 psf
- Exterior wall = 20 psf
- Interior partitions are included as 10 psf in floor + ceiling weight of 80 psf

The unit weights for roof, ceiling and wall are provided as...

Note that seismic mass from interior partitions is included in the floor plus ceiling weights shown, as required by ASCE/SEI 7-22
Example Multi-Story Steel Building with Steel Deck Diaphragms

Step 1 - Seismic weight at roof

Roof: 40 psf (150 ft)(120 ft) = 720 kips
Longitudinal exterior walls: 20 psf (150 ft)(12/2 + 4 ft)(2 sides) = 60 kips
Transverse exterior walls: 20 psf (120 ft)(12/2 + 4 ft)(2 sides) = 48 kips

TOTAL = 720 + 60 + 48
= 828 kips acting at roof

The total mass acting at the roof is summed to be....

Resulting in a total seismic mass of 828 kips acting at the roof.
Example Multi-Story Steel Building with Steel Deck Diaphragms

Step 1 - Seismic weight at 2nd through 6th floors

Floor: 80 psf (150 ft)(120 ft) = 1440 kips
Longitudinal exterior wall: 20 psf (150 ft)(12 ft)(2 sides) = 72 kips
Transverse exterior wall: 20 psf (120 ft)(12 ft)(2 sides) = 58 kips

TOTAL = 1440 + 72 + 58
= 1,570 kips acting at each floor

Seismic weight TOTAL = 828 + 5 (1,570) = 8,678 kips

The seismic weight acting at each floor level is summed to be ....

Resulting in a seismic mass of 1570 kips acting at each floor.

With this information the full seismic weight for the building is calculated to be 8678 kips.
Example Multi-Story Steel Building with Steel Deck Diaphragms

Step 1 - Diaphragm seismic weight, \( w_{px} \), at the roof:
= 828 kips (transverse and longitudinal directions)

Step 1 - Diaphragm seismic weight, \( w_{px} \), at the 2nd through 6th floors:
= 1,570 kips (transverse and longitudinal direction)

Diaphragm seismic weights with exterior wall weight parallel to the direction of seismic forces neglected are between 4 and 8 percent lower than total seismic weight. These forces are not carried by the diaphragm but instead act directly at the vertical elements. For simplicity, however, use total seismic weights of 828 and 1,570 kips to determine diaphragm design forces.

From Step 1 we have \( w_{px} \) seismic weight acting at the roof and typical floor of 828 and 1,570 kips, respectively.

In some instances, the designer may want to subtract the weights of the exterior wall from the \( w_{px} \) forces. This can be done because this weight is not traveling through the diaphragm to the exterior wall where the braces are located but originates at the exterior wall and therefore does not load the diaphragm. While this is permitted, the weight of the exterior wall may not be large enough to justify the introduction of this level of complexity in the seismic calculations. For this reason, the weight of the exterior walls is included in the \( w_{px} \) used for diaphragm design. For this building this will result in diaphragm design forces that are conservative by 4 to 8 percent.
Example Multi-Story Steel Building with Steel Deck Diaphragms

Step 2 - ASCE 7 Base Shear

\[ T_a = C_t h_n^\gamma = 0.020(72)^{0.75} = 0.49 \text{ sec} \quad \text{(ASCE/SEI 7-22 Eq. 12.8-7)} \]

\[ C_s = \frac{S_{ps}}{R_{1.2}} = \frac{1.20}{6} = 0.200 \text{ (governs)} \quad \text{(ASCE/SEI 7-22 Eq. 12.8-2)} \]

\[ C_s \text{ need not exceed:} \]

\[ C_s = \frac{S_{p1}}{T(\frac{T_e}{T})} = \frac{0.70}{0.49(\frac{72}{4})} = 0.238 \quad \text{(ASCE/SEI 7-22 Eq. 12.8-3)} \]

\[ V = C_s W = 0.20 \times 8,678 = 1,736 \text{ kips} \quad \text{(ASCE/SEI 7-22 Eq. 12.8-1)} \]

FEMA  
Building Seismic Safety Council  

Fire Resistance Protection
Example Multi-Story Steel Building with Steel Deck Diaphragms

**Step 3 - Vertical distribution of seismic base shear:**

The lateral seismic force at any level is determined as

$$F_x = C_{vx}V$$

(ASCE/SEI 7-22 Eq. 12.8-11)

Where:

$$C_{vx} = \frac{w_xh_x^k}{\sum_{i=1}^{n} w_ih_i^k}$$

(ASCE/SEI 7-22 Eq. 12.8-12)

For $T \leq 0.5$ sec., $k = 1.0$
Example Multi-Story Steel Building with Steel Deck Diaphragms

### Table 7.5-1: Vertical Distribution of Base Shear

<table>
<thead>
<tr>
<th>Level X</th>
<th>( w_x ) (kips)</th>
<th>( h_x ) (ft)</th>
<th>( w_x h_x^k ) (ft kips)</th>
<th>( C_{xx} )</th>
<th>( F_x ) (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>828</td>
<td>72</td>
<td>59,616</td>
<td>0.174</td>
<td>302</td>
</tr>
<tr>
<td>6</td>
<td>1,570</td>
<td>60</td>
<td>94,200</td>
<td>0.275</td>
<td>478</td>
</tr>
<tr>
<td>5</td>
<td>1,570</td>
<td>48</td>
<td>75,360</td>
<td>0.220</td>
<td>382</td>
</tr>
<tr>
<td>4</td>
<td>1,570</td>
<td>36</td>
<td>56,520</td>
<td>0.165</td>
<td>287</td>
</tr>
<tr>
<td>3</td>
<td>1,570</td>
<td>24</td>
<td>37,680</td>
<td>0.110</td>
<td>191</td>
</tr>
<tr>
<td>2</td>
<td>1,570</td>
<td>12</td>
<td>18,840</td>
<td>0.055</td>
<td>96</td>
</tr>
<tr>
<td>Sum</td>
<td>8,678</td>
<td>342,216</td>
<td></td>
<td>0.999</td>
<td>1,736</td>
</tr>
</tbody>
</table>

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m, 1.0 ft-kip = 1.36 kN-m
Example Multi-Story Steel Building with Steel Deck Diaphragms

Traditional Design Method (12.10.1 & 12.10.2)

From this point on, the diaphragm seismic design provisions will start to diverge between the Section 12.10.1 and 12.10.2 Traditional Design Method and the Section 12.10.3 Alternative Design Method. We will first look at the Traditional Design Method.
Step 4 - Strength level diaphragm design force, $F_{px}$:

Diaphragm design force is given by the larger of $F_x$ determined previously and $F_{px}$

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_{px}}$$

(ASCE/SEI 7-22 Eq. 12.10-1)

Note that for purposes of diaphragm forces $\rho$ is set to 1.0.

Designers should be familiar with this equation for the vertical distribution of diaphragm design forces. Note that roe is set to 1.0 for purposes of calculating diaphragm design forces.
### Traditional Design Method

#### Table 7.5-2: Diaphragm Seismic Forces, $F_{px}$

<table>
<thead>
<tr>
<th>Level</th>
<th>$w_i$ (kips)</th>
<th>$\sum_{i=1}^{n} w_i$ (kips)</th>
<th>$F_i$ (kips)</th>
<th>$\sum_{i=1}^{n} F_i = V_i$ (kips)</th>
<th>$w_{px} = \frac{\sum_{i=1}^{n} F_i}{\sum_{i=1}^{n} w_i}$ (kips)</th>
<th>$F_{px}$ = $w_{px}$ (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>828</td>
<td>828</td>
<td>302</td>
<td>302</td>
<td>828</td>
<td>302</td>
</tr>
<tr>
<td>6</td>
<td>1,570</td>
<td>2,398</td>
<td>478</td>
<td>780</td>
<td>1,570</td>
<td>510</td>
</tr>
<tr>
<td>5</td>
<td>1,570</td>
<td>3,968</td>
<td>382</td>
<td>1,162</td>
<td>1,570</td>
<td>460</td>
</tr>
<tr>
<td>4</td>
<td>1,570</td>
<td>5,538</td>
<td>287</td>
<td>1,449</td>
<td>1,570</td>
<td>411</td>
</tr>
<tr>
<td>3</td>
<td>1,570</td>
<td>7,108</td>
<td>191</td>
<td>1,640</td>
<td>1,570</td>
<td>362</td>
</tr>
<tr>
<td>2</td>
<td>1,570</td>
<td>8,678</td>
<td>96</td>
<td>1,736</td>
<td>1,570</td>
<td>314</td>
</tr>
<tr>
<td>Sum</td>
<td>8,678</td>
<td>1,736</td>
<td>8,678</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1.0 kip = 4.45 kN

Using information from the previous steps, this table shows the calculation of $F_{px}$ forces over the height of the structure.
Traditional Design Method

\[ F_{px} \text{ at the roof cannot be less than:} \]
\[ F_{pr} = 0.2S_{DS}l_{e}w_{pr} \quad \text{(ASCE/SEI 7-22 Eq. 12.10-2)} \]
\[ = 0.2(1.2)(1.0)(828) = 199 \text{ kips} \]

\[ F_{px} \text{ at the floor levels cannot be less than:} \]
\[ F_{px} = 0.2S_{DS}l_{e}w_{px} \quad \text{(ASCE/SEI 7-22 Eq. 12.10-2)} \]
\[ = 0.2(1.2)(1.0)(1,570) = 377 \text{ kips} \]

The lower limits for the \( F_{px} \) forces are shown here....
Traditional Design Method

\[ F_{px} \text{ at the roof need not exceed:} \]
\[ F_{pr} = 0.4S_{DS}\ell w_{pr} \quad \text{(ASCE/SEI 7-22 Eq. 12.10-3)} \]
\[ = 0.4(1.2)(1.0)(828) = 397 \text{ kips} \]

\[ F_{px} \text{ at the floor levels need not exceed:} \]
\[ F_{px} = 0.4S_{DS}\ell w_{px} \quad \text{(ASCE/SEI 7-22 Eq. 12.10-3)} \]
\[ = 0.4(1.2)(1.0)(1,570) = 754 \text{ kips} \]

And the upper limits for the \( F_{px} \) forces are calculated here...
Traditional Design Method

Table 7.5-3: Summary of Diaphragm Design Forces

<table>
<thead>
<tr>
<th>Level</th>
<th>$F_{px}$ From Vertical Distribution (kips)</th>
<th>$F_{px}$ Minimum (kips)</th>
<th>$F_{px}$ Maximum (kips)</th>
<th>$F_{px}$ Design (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>302</td>
<td>199</td>
<td>397</td>
<td>302</td>
</tr>
<tr>
<td>6</td>
<td>510</td>
<td>377</td>
<td>754</td>
<td>510</td>
</tr>
<tr>
<td>5</td>
<td>460</td>
<td>377</td>
<td>754</td>
<td>460</td>
</tr>
<tr>
<td>4</td>
<td>411</td>
<td>377</td>
<td>754</td>
<td>411</td>
</tr>
<tr>
<td>3</td>
<td>362</td>
<td>377</td>
<td>754</td>
<td>377</td>
</tr>
<tr>
<td>2</td>
<td>314</td>
<td>377</td>
<td>754</td>
<td>377</td>
</tr>
</tbody>
</table>

1.0 kip = 4.45 kN

This table summarizes the $F_{px}$ forces calculated using at each story, the lower-limit forces, the upper-limit forces, and the $F_{px}$ forces assigned for diaphragm design (right-hand column).
Step 5 addresses diaphragm transfer forces. This topic was discussed in the introduction portion of this presentation....
Traditional Design Method

Step 6 – Design for Shear and Flexure

▪ Diaphragms at each level are designed for shear and flexure using the tabulated $F_{px}$ design forces. Should diaphragm transfer forces be applicable these would also be included and be amplified where required.

▪ Where a computer analysis model is used, this can involve taking the shear and flexure forces at the $F_x$ level from the model and amplifying them to the $F_{px}$ level.

▪ For diaphragms idealized as rigid or semi-rigid, inherent torsion, accidental torsion and transfer forces are addressed in the building model such that the extracted shear and flexure forces include these effects.

Step 6 addresses the design of diaphragms for shear and flexure based on the derived diaphragm design forces.
Traditional Design Method

Step 7 - Collector Seismic Design Forces

Collectors in the example building are, per ASCE/SEI 7-22 Section 12.10.2.1, required to be designed for seismic loads effect including overstrength. This involves the seismic load effect with overstrength provisions of ASCE/SEI 7-22 Section 12.4.3, used in the appropriate load combinations from ASCE/SEI 7-22 Chapter 2. The following demonstrates the calculation of the collector seismic design force due to horizontal seismic forces. This will need to the combined with applicable gravity loads and vertical seismic forces.

Step 7 addresses collector seismic design forces....
To illustrate the calculation of collector forces the exterior framing line shown above will be used. We will calculate the collector forces at the 5th floor and roof levels.
Traditional Design Method

Step 7 - Diaphragm Transverse Force Reactions and Units Shears

The roof diaphragm is idealized to be flexible. As a result, the diaphragm reaction to the exterior wall line can be based on tributary seismic weight or a simple-span beam idealization. Based on this assumption:

Roof Diaphragm

\[ V = \frac{302 \text{ kips}}{2} = 151 \text{ kips} \]

\[ v = \frac{151 \text{ kips}}{120 \text{ ft}} = 1.26 \text{ klf} \]
Traditional Design Method

Step 7 - Diaphragm Transverse Force Reactions and Units Shears

For this example, the floor diaphragms are idealized as rigid. As a result, inherent and accidental torsion are applied to the model seismic forces in accordance with Sec. 12.8.4. In this example it is assumes a 10% increase of the floor diaphragm shear due to torsion and any transfer forces. Based on this assumption:

5th Floor diaphragm  \( V = 460 \text{ kips} \times \frac{1.1}{2} = 253 \text{ kips} \)

\( v = \frac{253 \text{ kips}}{120 \text{ ft}} = 2.11 \text{ klf} \)
Traditional Design Method

Step 7 - Collector Force at Location shown in Figure 7.5-2, amplified by $W_0 = 2.0$

Roof Diaphragm
$T/C = 1.26 \text{ klf (30 ft)} (2.0) = 76 \text{ kips}$

5th Floor Diaphragm
$T/C = 2.11 \text{ klf (30 ft)} (2.0) = 127 \text{ kips}$
Traditional Design Method

Step 8 – Deflection and Drift Requirements

For ELF design, this step incorporates the revised displacement and drift determination provisions of ASCE/SEI 7-22 Section 12.8.6 and the drift and deformation provisions of Section 12.12.

The structural separation provisions of Section 12.12.2, structural separation requirements of Section 12.12.3, and deformation compatibility provisions of 12.12.4 each require that diaphragm deflection be considered in addition to the deflection of the vertical elements.
Steps 4 through 8 will now be repeated using the ASCE/SEI 7-22 Alternative Diaphragm Design Method. This was first added in ASCE 7-22 and expanded to include steel deck diaphragms in ASCE/SEI 7-16.
Part 1: Vertical distribution of seismic forces for near-elastic diaphragm behavior

Part 2: Parameter $R_s$ modifies near-elastic forces based on diaphragm ductility and deformation capacity

$$ F_{px} = \frac{C_{px}}{R_s} W_{px} $$

As a quick reminder, the two major differences incorporated in the Alternative Design Procedure are...
Alternative Design Method (Section 12.10.3) - Introduction

Advantages of using Section 12.10.3 Alternative Design Provisions:
- Better reflects vertical distribution of diaphragm forces
- Better reflects affect of diaphragm ductility and displacement capacity
- May result in lower seismic demands

It is advantageous to use the Alternative Design Procedure because...
Example Multi-Story Steel Building with Steel Deck Diaphragms

Table 7.5-1: Vertical Distribution of Base Shear

<table>
<thead>
<tr>
<th>Level X</th>
<th>$w_x$ (kips)</th>
<th>$h_x$ (ft)</th>
<th>$w_xh_x^k$ (ft kips)</th>
<th>$C_{xx}$</th>
<th>$F_x$ (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>828</td>
<td>72</td>
<td>59,616</td>
<td>0.174</td>
<td>302</td>
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<tr>
<td>6</td>
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<td>60</td>
<td>94,200</td>
<td>0.275</td>
<td>478</td>
</tr>
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<td>287</td>
</tr>
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<td>24</td>
<td>37,680</td>
<td>0.110</td>
<td>191</td>
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<td>1,570</td>
<td>12</td>
<td>18,840</td>
<td>0.055</td>
<td>96</td>
</tr>
<tr>
<td>Sum</td>
<td>8,678</td>
<td>342</td>
<td>342,216</td>
<td>0.999</td>
<td>1,736</td>
</tr>
</tbody>
</table>

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m, 1.0 ft-kip = 1.36 kN-m

At the end of Step 3 the vertical distribution of the $F_x$ forces had been calculated, as tabulated above. We will move forward from this point using the Alternative Diaphragm Design Method of Section 12.10.3.
Step 4 addresses vertical distribution of diaphragm $F_{px}$ forces. When using the Alternative Design Procedure for buildings with three or more stories, the figure shown dictates the $F_{px}$ forces at each story level.
Alternative Design Method (Section 12.10.3)

\[ N = 6 \]

\[ z_s = 1.0 \] (all other SFRS, ASCE/SEI 7-22 Section 12.10.3.2.1)

\[ R_s = 2.0 \] for concrete-filled metal deck floor diaphragm (ASCE/SEI 7-22 Table 12.10-1)

\[ R_s = 1.0 \] bare steel deck roof diaphragm with welded connections not meeting special seismic detailing provisions (ASCE/SEI 7-22 Table 12.10-1)

\[ C_s = 0.200 \] (Slide 39)

The background for this pattern of vertical distribution and supporting research was already discussed in the introduction to this presentation.

In order to move forward with calculation of \( F_{px} \) forces, a series of variables need to be defined.

\( \text{N=} \text{number of stories} = 6 \)
Alternative Design Method (Section 12.10.3)

<table>
<thead>
<tr>
<th>Diaphragm System</th>
<th>Shear Controlled</th>
<th>Flexure Controlled</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cast in place concrete designed in accordance with ACI 318</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Precast concrete designed in accordance with ACI 318</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wood sheathed designed in accordance with ASCE/SEI 7 22 Section 14.5 and AWC Special Design Provisions for Wind and Seismic</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bare steel deck designed in accordance with ASCE/SEI 7 22 Section 14.1.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete filled steel deck designed in accordance with ASCE/SEI 7 22 Section 14.1.6</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$R_s$ factors are found in ASCE/SEI 7-22 Table 12.10-1, as shown above. The $R_s$ factor serves to reduce the diaphragm design forces from a near-elastic level to a force level that reflects the diaphragm ductility and displacement capacity, as determined using testing and analytical studies.

$R_s$ for the concrete-filled steel deck floor diaphragm can
Alternative Design Method (Section 12.10.3)

Modal Contribution Coefficient Modifier, $z_s$ (ASCE/SEI 7-22 Section 12.10.3.2.1)

<table>
<thead>
<tr>
<th>Description</th>
<th>$z_s$ value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buildings designed with Buckling Restrained Braced Frame systems defined in ASCE/SEI 7-22 Table 12.2 1</td>
<td>0.30</td>
</tr>
<tr>
<td>Buildings designed with Moment Resisting Frame systems defined in ASCE/SEI 7-22 Table 12.2 1</td>
<td>0.70</td>
</tr>
<tr>
<td>Buildings designed with Dual Systems defined in ASCE/SEI 7-22 Table 12.2 1 with Special or Intermediate Moment Frames capable of resisting at least 25% of the prescribed seismic forces</td>
<td>0.85</td>
</tr>
<tr>
<td>Buildings designed with all other seismic force resisting systems</td>
<td>1.00</td>
</tr>
</tbody>
</table>

$z_s$ factors are found in ASCE/SEI 7-22 Section 12.10.3.2.1 and are provided in tabular form above for ease of use. These are again derived from research. A larger value of $z_s$ will result in a larger contribution of higher mode behavior to $F_{px}$ forces.

$z_s$ for this steel braced frame building...
Using this information and the calculated value of $C_s$ from slide 39, we move forward with defining the $F_{px}$ forces.
Alternative Design Method (Section 12.10.3)

First Mode Contribution Factor
\[ \Gamma_{m1} = 1 + 0.5z_s \left( 1 - \frac{1}{N} \right) \]  
\[ = 1 + 0.5 \times 1.00 \times \left( 1 - \frac{1}{6} \right) = 1.42 \]  
(Eq. 12.10-13)

Higher Mode Contribution Factor
\[ \Gamma_{m2} = 0.9z_s \left( 1 - \frac{1}{N} \right)^2 \]  
\[ = 0.9 \times 1.00 \times \left( 1 - \frac{1}{6} \right)^2 = .625 \]  
(Eq. 12.10 - 14)

From the variables just defined, we can calculate the first mode contribution factor and the higher mode contribution factor. Rather than just addressing the second mode, the higher-mode factor is intended to address all higher modes that provide a significant contribution.
Higher Mode Response Coefficient $C_{s2}$ is taken as the lesser of the following:

- $C_{s2} = (0.15N + 0.25)I_eS_{DS}$
- $= (0.15 \times 6 + 0.25) \times 1.0 \times 1.2 = 1.38$  \hspace{1cm} (Eq. 12.10–10)
- $C_{s2} = I_eS_{DS} = 1.0 \times 1.2 = 1.2$  \hspace{1cm} (Eq. 12.10–11)
- $C_{s2} = \frac{I_eS_{D1}}{0.03(N-1)} = \frac{1.0 \times 0.7}{0.03 (6-1)} = 4.7$  \hspace{1cm} (Eq. 12.10–12a)
- Use $C_{s2} = 1.2$

The higher mode response coefficient is then calculated as...
Next, we calculate the three diaphragm design acceleration coefficients, $C_{p0}$, $C_{pi}$ and $C_{pn}$ that define the vertical distribution.

$C_{p0}$, at the base of the structure is calculated as:

$$C_{p0} = 0.4S_{DSIe} = 0.4(1.2)(1.00) = 0.48$$  \text{ (Eq 12.10-6)}
Alternative Design Method (Section 12.10.3)

Diaphragm Design Acceleration Coefficient at 80% of the Structure Height

C_{pi} is taken as the greater of the following:

\[ C_{pi} = C_{p0} = 0.48 \]  \hspace{1cm} (Eq. 12.10-8)

\[ C_{pi} = 0.9 \Gamma_m \Omega_0 C_s \]  \hspace{1cm} (Eq. 12.10-9)

\[ = 0.9(1.42)(2.0)(0.200) = 0.51 \]

Use \( C_{pi} = 0.51 \)

\( C_{pi} \) at the bend in the vertical distribution is taken as...
Alternative Design Method (Section 12.10.3)

- Diaphragm Design Acceleration Coefficient at the Structure Height, $h_n$

\[
C_{pn} = \sqrt{(f_{m1} \Omega_0 C_s)^2 + (f_{m2} C_s)^2}
\]

\[
C_{pn} = \sqrt{(1.42 \times 2 \times 0.200)^2 + (0.625 \times 1.2)^2}
\]

= 0.94 \hspace{1cm} \text{(Eq. 12.10-7)}

Finally, $C_{pn}$ at the top of the structure is calculated...
Using the calculated values, the vertical distribution of diaphragm design forces is defined by...

A line is drawn showing the 6th floor relative to the vertical distribution, which falls between $C_{pi}$ and $C_{pn}$. Each level can be located and the applicable diaphragm design acceleration coefficient at that level, $C_{px}$, can be calculated.
**Alternative Design Method** (Section 12.10.3)

**Diaphragm Design Acceleration Coefficient at 6th Floor**

\[ h_6 = 5 \text{ (12)} = 60 \text{ ft} \]

\[ C_{p6} = 0.51 + (0.94-0.51) (60-57.6)/12 = 0.60 \quad \text{(linear interpolation)} \]

\[ F_{p6} = \frac{C_{p6}}{R_s} \times \omega_{p6} \]

\[ = \frac{0.60}{2.0} \times 1,570 = 471 \text{ kips} \]

But not less than:

\[ F_{p6} = 0.2S_{DS} e \times \omega_{p6} \quad \text{(ASCE/SEI 7-22 Eq. 12.10-5)} \]

\[ = 0.2(1.2)(1.0)(1,570) = 377 \text{ kips} \quad \text{(floor)} \]

Putting this information together for the 6th floor concrete-filled steel deck diaphragm, we are able to calculated \( C_{p6} \) from the vertical distribution and using the previously identified \( \omega_{p6} \) of 1570 kips and \( R_s \) of 2.0, \( F_{p6} \) is calculated in accordance with ASCE/SEI 7-22 equation 12.10-4.

In addition, a lower bound for the \( F_{p6} \) force is provided per ASCE/SEI 7-22 equation 12.10-5.
### Alternative Design Method (Section 12.10.3)

Table 7.5-4: Summary of Section 12.10.3 Alternative Diaphragm Design Forces, $F_{px}$, (kips)

<table>
<thead>
<tr>
<th>Level</th>
<th>$C_{pn}$</th>
<th>$F_{px}$ Eq. 12.10 4 Force (kips)</th>
<th>$F_{px}$ Minimum (kips)</th>
<th>$F_{px}$ Design (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>0.94</td>
<td>778</td>
<td>199</td>
<td>778</td>
</tr>
<tr>
<td>6</td>
<td>0.60</td>
<td>471</td>
<td>377</td>
<td>471</td>
</tr>
<tr>
<td>5</td>
<td>0.51</td>
<td>400</td>
<td>377</td>
<td>400</td>
</tr>
<tr>
<td>4</td>
<td>0.50</td>
<td>392</td>
<td>377</td>
<td>392</td>
</tr>
<tr>
<td>3</td>
<td>0.49</td>
<td>385</td>
<td>377</td>
<td>385</td>
</tr>
<tr>
<td>2</td>
<td>0.49</td>
<td>385</td>
<td>377</td>
<td>385</td>
</tr>
</tbody>
</table>

1.0 kip = 4.45 kN

Applying this method for the second floor through the roof, the $F_{px}$ forces can be defined for each, as tabulated here.
Alternative Design Method (Section 12.10.3)

Step 5 – Diaphragm Transfer Forces

- Diaphragm transfer forces, as defined in ASCE/SEI 7-22 Section 11.2, occur where vertical elements of the SFRS are offset or discontinued at lower levels; they also occur due to changes in the stiffness of the SFRS vertical elements between levels. The occurrence of diaphragm transfer forces is determined by examining the distribution of forces from the analysis model.

- For simplicity, the building in this example building is assumed to not have diaphragm transfer forces.

Step 5 for the Alternative Design Provisions is the same as for the Traditional Method.
Alternative Design Method (Section 12.10.3)

Step 6 – Design for Shear and Flexure

- Diaphragms at each level are designed for shear and flexure using the tabulated $F_{px}$ design forces. Should diaphragm transfer forces be applicable these would also be included and be amplified where required (ASCE/SEI 7-22 Section 12.10.3.3).

- Where a computer analysis model is used, this can involve taking the shear and flexure forces at the $F_x$ level from the model and amplifying them to the $F_{px}$ level.

- For diaphragms idealized as rigid or semi-rigid, inherent torsion, accidental torsion and transfer forces are addressed in the building model such that the extracted shear and flexure forces include these effects.

Step 6 for the Alternative Design Provisions is the same as for the Traditional Method.

For the alternative Design Procedure, the requirement for amplification of transfer forces at Horizontal Irregularity Type 4 is found in ASCE/SEI Section 12.10.3.3.
Collectors in the example building are, per ASCE/SEI 7-22 Section 12.10.3.4, required to be designed for amplified seismic forces. In lieu of the overstrength requirements of ASCE/SEI 7-22 Section 12.10.2.1, the collectors are required to be amplified by a factor of 1.5. Just like the seismic load effect with overstrength provisions of ASCE/SEI 7-22 Section 12.4.3, the amplified forces are required to be used in the appropriate load combinations from ASCE/SEI 7-22 Chapter 2. The following demonstrates the calculation of the collector seismic design forces due to horizontal seismic loads. This will need to be combined with applicable gravity loads and vertical seismic forces.

The overstrength force level for collectors and their connections to vertical elements applies when using the Alternative Design Procedure. The overstrength factor, however, is defined in ASCE/SEI 7-22 Section 12.10.3.4, rather than being defined by the vertical elements of the SFRS and Table 12.2-1.
Diaphragm Transverse Force Reactions and Units Shears

Roof Diaphragm
\[ V = \frac{778 \text{ kips}}{2} = 389 \text{ kips} \]
\[ v = \frac{389 \text{ kips}}{120 \text{ ft}} = 3.24 \text{ klf} \]

5th Flr diaphragm
\[ V = \frac{400 (1.1) \text{ips}}{2} = 220 \text{ kips} \]
\[ v = \frac{220 \text{ kips}}{120 \text{ ft}} = 1.83 \text{ klf} \]

Using the same assumptions as for the Traditional Design Method and the \( F_{px} \) forces calculated using the Alternative Design Procedure, the unit shears at the edges of the 5th floor and roof diaphragms are calculated as....
Alternative Design Method (Section 12.10.3)

- Collector Force at Location shown in Figure, amplified by 1.5 (in lieu of $\Omega_0$)
- Roof Diaphragm $T/C = 3.24$ klf (30 ft) (1.5) = 146 kips
- 5th Floor Diaphragm $T/C = 1.83$ klf (30 ft) (1.5) = 82 kips

Using the procedures of ASCE/SEI 7-22 Section 12.10.3.4, the collector forces are calculated as....
Alternative Design Method (Section 12.10.3)

Step 8 – Deflection and Drift Requirements

For ELF design, this step incorporates the revised displacement and drift determination provisions of ASCE/SEI 7-22 Section 12.8.6 and the drift and deformation provisions of Section 12.12.

The structural separation provisions of Section 12.12.2, structural separation requirements of Section 12.12.3, and deformation compatibility provisions of 12.12.4 each require that diaphragm deflection be considered in addition to the deflection of the vertical elements.

Step 8 for the Alternative Design Provisions is the same as for the Traditional Method.
Example Multi-Story Steel Building with Steel Deck Diaphragms
Comparison of Methods
### Table 7.5-5: Comparison of Traditional and Alternative $F_{px}$ Diaphragm Design Forces (kips)

<table>
<thead>
<tr>
<th>Level</th>
<th>$F_{px}$ Traditional ASCE/SEI 7 22 Section 12.10.1 and 12.10.2 (kips)</th>
<th>$F_{px}$ Alternative ASCE/SEI 7 22 Section 12.10.3 (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>302</td>
<td>778 ($R_s = 1.0$)</td>
</tr>
<tr>
<td>6</td>
<td>510</td>
<td>471</td>
</tr>
<tr>
<td>5</td>
<td>460</td>
<td>400</td>
</tr>
<tr>
<td>4</td>
<td>411</td>
<td>392</td>
</tr>
<tr>
<td>3</td>
<td>377</td>
<td>385</td>
</tr>
<tr>
<td>2</td>
<td>377</td>
<td>385</td>
</tr>
</tbody>
</table>

1.0 kip = 4.45 kN

This table compares the diaphragm design forces from this example.
Comparison of Design Methods

For this structure and the diaphragm systems used, the alternative method force is higher than the traditional method at some diaphragm levels (particularly at the roof), and lower at others. The much higher diaphragm design force at the roof comes from the combination of using the alternative method, and the very low values of $R_s = 1.0$ for the welded bare steel deck diaphragm that is recognized in the ASCE/SEI 7-22 to have low ductility. If the roof diaphragm were instead changed to conform to the special seismic detailing requirements, the roof diaphragm design forces would essentially match the traditional method forces.
### Table 7.5-6: Comparison of Traditional and Alternative Diaphragm Collector Forces

<table>
<thead>
<tr>
<th>Level</th>
<th>Traditional ASCE/SEI 7 22 Section 12.10.1 and 12.10.2 (kips)</th>
<th>Alternative ASCE/SEI 7 22 Section 12.10.3 (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>76</td>
<td>146 ($R_s = 1.0$)</td>
</tr>
<tr>
<td>5</td>
<td>127</td>
<td>82</td>
</tr>
</tbody>
</table>

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m, 1.0 ft-kip = 1.36 kN-m

This table compares the collector forces. This outcome is similar.
Part 1 Closing Comments
This slide is intended to initiate questions from participants.
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Welcome to 2020 NEHRP Provisions training, Chapter 7 addressing Horizontal diaphragm design. This training is presented in two parts.

Part 1 provides an introduction and presents an example problem with a multi-story steel building with steel deck diaphragms. Part 1 is anticipated to take approximately 90 minutes to present including question and answer sessions.

Part 2 presents an example problem with a single-story large-box building with tilt-up walls and a flexible diaphragm. Part 2 is anticipated to take approximately 60 minutes to present, including a question-and-answer session.

Both examples incorporate the 2020 NEHRP Previsions as incorporated into ASCE/SEI 7-22.
Part 2 presents an example one story rigid wall flexible diaphragm building with bare steel deck.
Diaphragm Design Presentation Outline – Part 2

- Example one-story RWFD building with steel deck diaphragm
  - Section 12.10.1 and 12.10.2 Traditional Design Method
  - Section 12.10.4 Alternative RWFD Design Method
  - Comparison of results

This presentation will cover determination of diaphragm design forces using both the Traditional Design Methods of Sections 12.10.1 and 12.10.2 and the Alternative RWFD Method of Section 12.10.4. The ASCE/SEI 7-22 title for this section is: “Alternative Diaphragm Design Provisions for One-Story Structures with Flexible Diaphragms and Rigid Vertical Elements.”
This design example will show a very common occurrence of this building type – a large footprint single-story building with tilt-up concrete perimeter walls and a flexible diaphragm. For the example, the diaphragm is of bare steel deck. The simple rectangular plan and a typical section are shown in the figures above.
Example One-Story RWFD Building with Steel Deck Diaphragm

Building Configuration
- One story
- $I_e = 1.0$
- Mean roof height = 30 feet
- Length = 600 feet
- Width = 360 feet
- $S_{DS} = 1.0$, $S_{D1} = 0.50$ (ASCE/SEI 7-22 Section 11.4.4)
- Bare steel deck diaphragm
- Intermediate precast concrete shear walls - $R=4$, $W_0=2.5$, $C_d=4$

Particulars of the building configuration and seismic design parameters are....
Example One-Story RWFD Building with Steel Deck Diaphragm

The system includes a bare steel deck diaphragm supported on open-web steel joists and girders. The perimeter walls are 9-1/4-inch-thick tilt-up concrete walls, with a mean roof height of 30 feet, and a parapet above the roof of 3 feet.

For purposes of design, the diaphragm will be categorized as flexible:

- When using Section 12.10.1 and 12.10.2 provisions, Section 12.3.1.1 permits the combination of bare steel deck and concrete walls to be idealized as flexible.
- When using Section 12.10.4, diaphragms meeting the applicable limitations of Section 12.10.4.1 are automatically considered flexible and able to use the flexible diaphragm-based provisions of Section 12.10.4.
Example One-Story RWFD Building with Steel Deck Diaphragm

**Step 1 - Weight for Seismic Analysis**

- Roof D = 20 psf
- Wall D = 116 psf

Wall seismic weight tributary to roof:

\[ w = 116 \times \frac{(33)(33/2)}{30} = 2,105 \text{ plf} \]

Seismic weight – Roof: 0.02 ksf (600 ft) (360 ft) = 4,320 kips

Longitudinal walls: (2.105 klf)(600 ft)(2 sides) = 2,526 kips

Transverse walls: (2.105 klf)(360 ft)(2 sides) = 1,516 kips

TOTAL = 8,362 kips acting at roof

Step 1 takes the information about the building configuration and identifies seismic weights for use in design...
Step 1 - Diaphragm Weight, $w_{px}$ at the Roof

$w_{px} = \text{Total seismic weight} - \text{weight of the walls resisting seismic forces}$

- $= 8,362 - 1,516 = 6,846 \text{ kips (for seismic forces in transverse direction)}$
- $= 8,362 - 2,526 = 5,836 \text{ kips (for seismic forces in longitudinal direction)}$

When designing this type of low-rise building with heavy exterior walls it is common practice to subtract the weight of the walls parallel to the seismic design direction from the seismic weight $w_{px}$. This is because the wall weight is understood to not be carried by the diaphragm, it originates in the walls and is resisted by the walls.
Example One-Story RWFD Building with Steel Deck Diaphragm

Step 2 - Base Shear

\[ T_a = C_t h_n^x = 0.020(30)^{0.75} = 0.26 \text{ sec} \]  \hspace{1cm} (ASCE/SEI 7-22 Eq. 12.8-7)

\[ C_s = \frac{S_{DS}}{R/I_e} = \frac{1.0}{4/1.0} = 0.250 \]  \hspace{1cm} (ASCE/SEI 7-22 Eq. 12.8-2)

\[ C_s \text{ need not exceed:} \]

\[ C_s = \frac{S_{D1}}{T(R)/I_e} = \frac{0.50}{0.26(4)/1.0} = 0.481 \]  \hspace{1cm} (ASCE/SEI 7-22 Eq. 12.8-3)

\[ \text{Base Shear } V = C_s W = (0.250)(8,362) = 2,090 \text{ kips} \]  \hspace{1cm} (ASCE/SEI 7-22 Eq. 12.8-1)

Using the equations of ASCE/SEI 7-22 Section 12.8, the seismic base shear can be calculated...
From this point forward, the Traditional Design Method and the Alternative RWFD Provisions start to diverge. We will complete the traditional design method first and follow up with the Alternative RWFD Provisions.
Traditional Design Method

Step 4 - Strength Level diaphragm design force:

\[
F_{px} = \frac{\sum_{i=1}^{n} F_i}{\sum_{i=1}^{n} w_i}
\]  
(ASCE/SEI 7-22 Eq. 12.10-1)

For a single-story building, \( F_{px} = C_s (w_{px}) \)

\[
F_{px} = C_s (w_{px})
\]

\[
= 0.25 \times (6,846) = 1,712 \text{ kips (transverse direction)}
\]

\[
= 0.25 \times (5,836) = 1,459 \text{ kips (longitudinal direction)}
\]

Step 3 in the diaphragm design process addresses vertical distribution of \( F_x \) forces. This is not applicable for our one-story building.

Moving on with the Section 12.10.1 and 12.10.2 traditional design method, Step 4 calculated the diaphragm \( F_{px} \) forces. Again, vertical distribution is not applicable. The diaphragm design forces are determined as \( C_s \times w_{px} \). Note that the \( w_{px} \) value that deducts the weight of walls parallel to the direction of seismic load is carried through this calculation, resulting in different design forces in the longitudinal and transverse directions.
Traditional Design Method

The minimum value of $F_{px}$ is:

$$F_{px} = 0.2S_{DSf_p}w_{px} \quad \text{(ASCE/SEI 7-22 Eq. 12.10-2)}$$

- $= 0.2(1.0)(1.0)(6,846) = 1,369 \text{ kips (transverse direction)}$
- $= 0.2(1.0)(1.0)(5,836) = 1,167 \text{ kips (longitudinal direction)}$

The maximum value of $F_{px}$ is:

$$F_{px} = 0.4S_{DSf_p}w_{px} \quad \text{(ASCE/SEI 7-22 Eq. 12.10-3)}$$

- $= 0.4(1.0)(1.0)(6,846) = 2,738 \text{ kips (transverse direction)}$
- $= 0.4(1.0)(1.0)(5,836) = 2,334 \text{ kips (longitudinal direction)}$

It is necessary to also check the upper and lower bounds of the diaphragm design forces using ASCE/SEI 7-22 equations 12.10-2 and 12.10-3. These are calculated as....
Traditional Design Method

Step 4 - Governing diaphragm design force

\[ F_{px} = 1,712 \text{ kips (transverse direction)} \]
\[ F_{px} = 1,459 \text{ kips (longitudinal direction)} \]

The result of Step 4 is the following \( F_{px} \) forces to be used for diaphragm design...
Traditional Design Method

Step 6 - Diaphragm Design for Shear

The diaphragm is design for shear using \( F_{px} \) forces. The following illustrates shear calculations for the transverse direction.

For transverse roof diaphragm forces:

\[
\begin{align*}
w &= \frac{1,712 \text{ kips}}{600 \text{ ft}} = 2.85 \text{ klf} \\
V &= 2.85 \text{ klf} \times \frac{600 \text{ ft}}{2} = 856 \text{ kips} \\
v &= \frac{856 \text{ kips}}{360 \text{ ft}} = 2.37 \text{ klf maximum at end of diaphragm span}
\end{align*}
\]

Step 5 in the process addresses transfer forces. Transfer forces are not applicable to one-story buildings and so are skipped in this example.

Step 6 involves diaphragm design for shear and flexure. For design for forces in the transverse direction, the \( F_{px} \) forces can be turned into a uniformly distributed load as shown in the figure at the right. From this the diaphragm shear reactions can be calculated and translated into unit shear forces in plf.

This is repeated in the longitudinal direction.
Step 6 - Diaphragm Design for Flexure

For transverse roof diaphragm forces:

\[ w = \frac{1,712 \text{ kips}}{600 \text{ ft}} = 2.85 \text{ klf} \]

\[ M = \frac{2.85 \text{ klf} (600 \text{ ft})^2}{8} = 128,250 \text{ kip-ft} \]

Chord \( T/C = \frac{128,250 \text{ kip-ft}}{360 \text{ ft}} = 356 \text{ kips} \) maximum at diaphragm mid-span

Design for flexure uses the same uniformly distributed loads to calculate moment and mid-span of the diaphragm. It is assumed that this is resisted by a tension chord at one edge of the diaphragm and a compression chord at the other, as shown in the graphic. The tension and compression sides reverse with reversing earthquake loading. The chord tension and compression forces can be calculated as....

For a steel deck diaphragm, a structural steel member will often be provided to serve as the diaphragm chord. In this case a member with a fairly substantial section area will be required.
Step 7 addresses calculation of design forces for collectors and their connections to vertical element.

For this problem it is assumed that for a length of approximately 90 feet at one corner as shown above, the concrete tilt-up panels are replaced with a storefront glazing system. At this location a collector will need to be provided to pull the unit shear forces from the edge of the diaphragm into the tilt-up walls beyond. The collector force can be calculated as...

The overstrength factor in this cases is taken from Table 12.2-1 based on intermediate precast concrete wall vertical system.
Traditional Design Method

Step 8 - Deflection and Drift Limitations.

▪ All applicable ASCE/SEI 7-22 deflection and drift checks are to be completed. It is important that this include a check that the gravity system can accommodate the mid-span deflection of the roof diaphragm, and the P-D stability of the tilt-up wall panels when subject to the diaphragm deflection.

▪ See the commentary to the ASCE/SEI 7-22 Section 12.10.4 provisions for further discussion of these checks.
We will now go through diaphragm design for the same building using the Section 12.10.4 Alternative RWFD Provisions.

As noted in the subtitle, we will first look at design using a bare steel deck diaphragm system that conforms to the AISI S400 provisions for special seismic details. We will follow up with a second design for a diaphragm design that does not. The specifics of the special seismic detailing will be discussed in more detail in a few slides.
Diaphragm Seismic Design Methods

- Advantages of using Section 12.10.3 Alternative Design Provisions:
  - Better reflects vertical distribution of diaphragm forces
  - Better reflects affect of diaphragm ductility and displacement capacity
  - May result in lower seismic demands

- Advantages of using Section 12.10.4 Alternative RWFD Method:
  - Better reflects seismic response of RWFD buildings
  - May result in lower seismic demands
  - Is anticipated to result in better performance

- When will the Section 12.10.1 and 12.10.2 Traditional Method result in lower design forces?
  - Bare steel deck diaphragms not meeting the AISI S400 special seismic detailing provisions
  - Other

As a reminder from the presentation introduction, some of the advantages of design in accordance with the Section 12.10.4 diaphragm design provisions include....
Step 1 - Check ASCE/SEI 7-22 Section 12.10.4.1 Scoping Limitations

The following are the scoping limitations that must be checked. If the building conforms to all scoping limitations, it is eligible to use the ASCE/SEI 7-22 Section 12.10.4 procedure.

1. All portions of the diaphragm shall be designed using the provisions of this section in both orthogonal directions.
2. The diaphragm shall consist of either a) a wood structural panel diaphragm designed in accordance with AWC SDPWS and fastened to wood framing members or wood nailers with sheathing nailing in accordance with the AWC SDPWS Section 4.2 nominal shear capacity tables, or b) a bare (untopped) steel deck diaphragm meeting the requirements of AISI S400 and AISI S310.
3. Toppings of concrete or similar materials that affect diaphragm strength or stiffness shall not be placed over the wood structural panel or bare steel deck diaphragm.

The steps used to design the diaphragm using Section 12.10.4 are distinctly different from Sections 12.10.1 and 12.10.2 Traditional Method, and as a result are numbered differently.

Before starting to implement the Alternative RWFD design provisions of Section 12.10.4, it is necessary to check the limitations of Section 12.10.4.1 to make sure that the structure qualifies for use of this method. Section 12.10.4.1 includes a list of 7 limitations. They are...
4. The diaphragm shall not contain horizontal structural irregularities, as specified in ASCE/SEI 7-22 Table 12.3-1, except that Horizontal Structural Irregularity Type 2 (reentrant corner irregularity) is permitted.

5. The diaphragm shall be rectangular in shape or shall be divisible into rectangular segments for purpose of seismic design, with vertical elements of the seismic force-resisting system or collectors provided at each end of each rectangular segment span.

For Item 4...
For Item 5 – the diagram below shows a non-rectangular building being divided into multiple rectangles for purposes of diaphragm design. Each edge of each segment must be supported by vertical elements or by collectors that extend to vertical elements, as seen in the left figure.

The right figure shows a plan configuration though to be somewhat prevalent in the eastern states. A large plan building is constructed using two segment separated by a seismic joint. Each segment only has walls or other vertical elements on three sides and no vertical element is provided to support the diaphragm at the line of the seismic joint. The result is a c-shaped configuration in plan, and a very long span diaphragm cantilever. This does not meet the criteria of Item 5 and this configuration cannot be designed using the Alternative RWFD provisions.

The ASCE/SEI 7-22 commentary provides examples of plan configurations for which this design method is not applicable.
6. The vertical elements of the seismic force-resisting system shall be limited to one or more of the following: concrete shear walls, precast concrete shear walls, masonry shear walls, steel concentrically braced frames, steel and concrete composite braced frames, or steel and concrete composite shear walls.

7. The vertical elements of the seismic force-resisting system shall be designed in accordance with ASCE/SEI 7-22 Section 12.8, except that they shall be permitted to be designed using the two-stage analysis procedure of ASCE/SEI 7-22 Section 12.2.3.2.2, where applicable.

The example building conforms to all of these limitations and can be designed in accordance with ASCE/SEI 7-22 Section 12.10.4.
Alternative RWFD Design Method (Meeting Special Seismic Detailing Requirements, 12.10.4)

Step 2 - Break roof diaphragm into a series of rectangular segments for purposes of design with each segment spanning to vertical elements or a collector.

- Because the example building is rectangular in plan and shear walls are located at the building perimeter, a single rectangular segment extending for the full building plan (600 ft by 360 ft) will be used.

If the building is not rectangular in plan, see the ASCE/SEI 7-22 commentary for discussion of how it might be broken into rectangular elements for the purposes of design. Buildings with walls that are angled or curved in plan are not intended to be designed using this method.
Alternative RWFD Design Method (Meeting Special Seismic Detailing Requirements, 12.10.4)

Step 3 - Determine $W_{px}$.

$W_{px}$ was determined in previous slides to be:

- 6,846 kips (transverse forces)
- 5,836 kips (longitudinal forces)

Step 3 involves determining $W_{px}$ forces. These are the same as was determined for the Section 12.10.1 and 12.10.2 Traditional Design Method.
Step 4 involves the determination of the response modification coefficient for design of diaphragms, $R_{\text{diaph}}$. This is found in ASCE/SEI 7-22 Section 12.10.4.2.1. This is a new variable that applies explicitly to the design of the diaphragm. It is derived from research including analytical studies to confirm resulting seismic performance can be anticipated to be in line with the intent of ASCE 7 and the building code.

The $R_{\text{diaph}}$ factor is set as 4.5 for wood structural panel diaphragms. For bare steel deck diaphragms, however, it is necessary to determine whether the diaphragm meets the AISI S400 requirements for special seismic detailing in order to assign the $R_{\text{diaph}}$. 

Step 4 - Determine $R_{\text{diaph}}$

Section 12.10.4.2.1:

$R_{\text{diaph}} = 4.5$ for bare steel deck diaphragms that meet the special seismic detailing requirements of AISI S400 Section F3.5.1.
### Alternative RWFD Design Method (Meeting Special Seismic Detailing Requirements AISI S400 Section F3.5.1)

<table>
<thead>
<tr>
<th>Item</th>
<th>Prescriptive Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>The steel deck panel type shall be 36 in. (914 mm) wide, 1.5 in. (38.1 mm) deep wide rib, 6 in. (152 mm) pitch (WR) deck.</td>
</tr>
<tr>
<td>2</td>
<td>The steel deck base steel thickness shall be greater than or equal to 0.0295 in. (0.749 mm) and less than or equal to 0.0598 in. (1.52 mm).</td>
</tr>
<tr>
<td>3</td>
<td>The steel deck material shall conform to Section A.3.1.1 of AISI S100 [CSA S136].</td>
</tr>
<tr>
<td>4</td>
<td>The structural connection between the steel deck and the supporting steel member (with minimum thickness of 1/8 in. (3.18 mm)) shall be limited to mechanical connectors qualified in accordance with AISI S400 Section F3.5.1.1.</td>
</tr>
</tbody>
</table>

The easiest method to meet the special seismic detailing provisions is by using the prescriptive provisions for bare steel deck with mechanical fasteners. There are 8 requirements that need to be met, shown in the table above and the following slides.

These are...
## Alternative RWFD Design Method (Meeting Special Seismic Detailing Requirements AISI S400 Section F3.5.1)

<table>
<thead>
<tr>
<th>Item</th>
<th>Prescriptive Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>The structural connection perpendicular to the steel deck ribs shall be no less than a 36/4 pattern (12 in. (305 mm) on center) and no more than a 36/9 pattern (6 in. (152mm) on center) with double fasteners in the last panel rib.</td>
</tr>
<tr>
<td>6</td>
<td>The structural connection parallel to the steel deck ribs shall be no less than 3 in. (76.2 mm) and no more than 24 in. (610 mm) and shall not be greater than the sidelap connection spacing.</td>
</tr>
<tr>
<td>7</td>
<td>The sidelap connection between steel deck shall be limited to #10, #12, or #14 screws sized such that shear in the screws is not the controlling limit state, or connectors qualified in accordance with AISI S400 Section F3.5.1.2.</td>
</tr>
</tbody>
</table>

These are...
Alternative RWFD Design Method (Meeting Special Seismic Detailing Requirements AISI S400 Section F3.5.1)

<table>
<thead>
<tr>
<th>Item</th>
<th>Prescriptive Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>The sidelap connection shall be spaced no less than 6 in. (152 mm) and no more than 24 in. (610 mm).</td>
</tr>
</tbody>
</table>

Impact of Prescriptive Requirements:
- Requires mechanical fasteners
- Welded connections not permitted under prescriptive requirements

Other AISI methods:
- Qualification by testing – AISI S400 Section F3.5.2.1
- Principles of mechanics – AISI S400 Section F3.5.2.2

Finally, Item 8 is...

Welded connections do not satisfy these prescriptive special seismic detailing provisions. Other methods are available by which welded connections might be able to be qualified in the future, including qualification by testing and principles of mechanics.

Again, for this example we will used a mechanical connection system that meets the special seismic detailing requirements, and so qualify for $R_{dilap} = 4.5$. 
Step 5 involves the calculation of the diaphragm period. Fundamental to the alternative method is recognition that the seismic response of this building type is governed by the long period response of the diaphragm. $T_{\text{diaph}}$ is used to explicitly incorporate this into the design process.

The formula provided comes from research and is a function of the diaphragm span. Because the diaphragm span is different in the transverse and longitudinal directions, we get a separate period for each direction.
Alternative RWFD Design Method (Meeting Special Seismic Detailing Requirements, 12.10.4)

Step 6 - Determine $C_{s\text{-diaph}}$

For transverse forces:

$$C_{s\text{-diaph}} = \frac{S_{DS}}{R_{diaph}/I_e} = \frac{1.0}{4.5/1.0} = 0.222$$

(ASCE/SEI 7-22 Eq. 12.10-16a)

But need not exceed

$$C_{s\text{-diaph}} = \frac{S_{DS}/(R_{diaph}/I_e)}{0.60(4.5)/1.0} = 0.185$$

(ASCE/SEI 7-22 Eq. 12.10-16b)

Use $C_{s\text{-diaph}} = 0.185$ transverse

With $R_{diaph}$ and $T_{diaph}$ determined, the seismic design coefficient $C_{s\text{-diaph}}$ can be calculated. This is again a new variable that is explicitly for use in diaphragm design and is at the heart of the Alternative RWFD Provision methodology. First $C_{s\text{-diaph}}$ is determined for the transverse direction seismic loads.

In seeing equation 12.10-16b provide the controlling value for $C_{s\text{-diaph}}$ in the transverse direction we are directly seeing the impact of the Alternative RWFD provisions and recognition of the long-period (off plateau) response of the structure.
For longitudinal forces:

\[ C_{s-diaph} = \frac{S_{DS}}{R_{diaph} / I_e} = \frac{1.0}{4.5 / 1.0} = 0.222 \]  

(ASCE/SEI 7-22 Eq. 12.10-16a)

But need not exceed:

\[ C_{s-diaph} = \frac{S_{DI}}{T_{diaph} (R_{diaph}) / I_e} = \frac{0.50}{0.36(4.5) / 1.0} = 0.309 \]  

(ASCE/SEI 7-22 Eq. 12.10-16b)

Use \( C_{s-diaph} = 0.222 \) longitudinal

Next, the same is determined for the transverse loads. In this instance, because of the shorter diaphragm span in the longitudinal direction, \( T_{diaph} \) is smaller and as a result equation 12.10-16a (plateau) winds up controlling the value of \( C_{s-diaph} \).
Alternative RWFD Design Method *(Meeting Special Seismic Detailing Requirements, 12.10.4)*

Step 7 - Determine diaphragm design force, $F_{px}$

- $F_{px} = C_{s-diaph} \times (w_{px})$  
  (ASCE/SEI 7-22 Eq.12.10-15)
- $F_{px} = 0.185 \times (6,846) = 1,266$ kips transverse
- $F_{px} = 0.222 \times (5,836) = 1,296$ kips longitudinal

Note that unlike the ASCE/SEI 7-22 Section 12.10.1 and 12.10.2 traditional method and the Section 12.10.3 alternative method, for the Section 12.10.4 alternative RWFD method there is no lower bound for diaphragm seismic design forces.

In step 7 the $F_{px}$ forces can now be calculated...
Step 8 - Determine amplified shear and extent of amplified shear boundary zone

- Because the diaphragm span in both directions is greater than 100 ft., an amplified shear zone will be located at each end of the diaphragm span and extend for ten percent of the diaphragm span. The extent of the amplified shear zones are:
  - 0.10 (600) = 60 ft each end for transverse forces
  - 0.10 (360) = 36 ft each end for longitudinal forces.

Where diaphragm spans are over 100 feet, as occurs in both the transverse and longitudinal direction in this example, amplified shear boundary zones are provided at each end of the diaphragm span over a length of 10% of the diaphragm span. This is true for loading in both orthogonal directions. The upper diagram shows the amplified shear boundary zone for loading in the longitudinal direction, while the lower diagram shows the transverse direction.
Alternative RWFD Design Method *(Meeting Special Seismic Detailing Requirements, 12.10.4)*

**Step 9 - Diaphragm Design for Shear**
- The diaphragm is designed for shear using $F_{px}$ forces. The following illustrates shear calculations for the transverse direction.
- For transverse roof diaphragm forces:
  - $w = 1,266$ kips / 600 ft = 2.11 klf
  - $V = R = 2.11$ klf (600 ft / 2) = 633 kips
  - $v = 633$ kips / 360 ft = 1.76 klf maximum at end of diaphragm span WITHOUT shear amplification
  - $v = 1.76$ klf (1.5) = 2.64 klf maximum at end of diaphragm span WITH shear amplification
- Using $R$ (presumably for reaction) is a bit confusing since there already is an $R$ for the Response Modification Coefficient. Typical.

Next, the diaphragm is designed for shear....
The resulting shear diagram for seismic forces in the transverse direction is shown in this figure. The step in shear at the amplified shear zones can be seen at either end of the span. With the shear diagram determined, the specifics of the diaphragm decking and fastening can be determined.
Step 9 addresses the diaphragm design for flexure. The moment at diaphragm mid-span is first calculated and then resolved into a tension and compression couple acting at the diaphragm chord members. The diaphragm design forces used to calculate the moment do NOT include the amplified shear zone forces and do not include any consideration of overstrength.
**Alternative RWFD Design Method (Meeting Special Seismic Detailing Requirements, 12.10.4)**

Step 10 - Determine collector forces in accordance with ASCE/SEI 7-22 Section 12.10.4.2.4. Collector forces are to be based on $F_{p_{0}}$ forces WITHOUT the 1.5 amplification factor, multiplied by $\Omega_{0-dihp}$:

- Per ASCE/SEI 7-22 Section 12.10.4.2.4, the collector force is calculated based on the maximum transverse diaphragm shear WITHOUT amplification, multiplied by $\Omega_{0-dihp}$:

  $$T/C = 1.76 \text{ klf (90 ft) (2.0)} = 317 \text{ kips}$$

As with the Traditional Design Method, collector force calculation will be illustrated at the location shown in this figure. Two points are important when calculating the collector forces:

1. The collector force is determined using the diaphragm design forces WITHOUT shear amplification, and
2. Instead of using the overstrength factor $\Omega_{0}$ based on the vertical system and taken from Table 12.2-1, Section 12.10.4.2.4 specifies that $\Omega_{0-dihp} = 2.0$. This value was derived from the research use to develop the Alternative RWFD provisions.
Alternative RWFD Design Method (Meeting Special Seismic Detailing Requirements, 12.10.4)

Step 11 - Check applicable ASCE/SEI 7-22 deflection and drift limitations.

- Where required by ASCE/SEI 7-22, determine $C_{rd,\text{diaph}}$ and diaphragm deflections in accordance with ASCE/SEI 7-22 Section 12.10.4.2.5. All applicable ASCE/SEI 7-22 deflection and drift checks are to be completed. It is important that this includes a check that the gravity system can accommodate the mid-span deflection of the roof diaphragm, and the P-D stability of the tilt-up wall panels when subject to the diaphragm deflection.

The Step 11 check of deflection and displacement limits is the same under the Alternative RWFD Provisions as the Traditional Method.
Example One-Story RWFD Building with Steel Deck Diaphragm

Alternative RWFD Design Method (12.10.4)

NOT Meeting AISI S400 Special Seismic Detailing Requirements

Next, we will repeat the same Alternative RWFD Provisions design process with a steel deck diaphragm system that does NOT meet the special seismic detailing provisions.
Alternative RWFD Design Method (NOT Meeting Special Seismic Detailing Requirements, 12.10.4)

- This example building has a bare steel deck diaphragm that is welded instead of using mechanical fasteners. Because of this, the diaphragm does not meet the special seismic detailing requirements of AISI S400 Section F3.5.1.
Step 1 - Check ASCE/SEI 7-22 Section 12.10.1.1 Scoping Limitations

- The following are the scoping limitations that must be checked. If the building conforms with all scoping limitations, it is eligible to use the ASCE/SEI 7-22 Section 12.10.4 procedure.

1. All portions of the diaphragm shall be designed using the provisions of this section in both orthogonal directions.

2. The diaphragm shall consist of either a) a wood structural panel diaphragm designed in accordance with AWC SDPWS and fastened to wood framing members or wood nailers with sheathing nailing in accordance with the AWC SDPWS Section 4.2 nominal shear capacity tables, or b) a bare (untopped) steel deck diaphragm meeting the requirements of AISI S400 and AISI S310.

Once again, the design starts with a check of the Section 12.10.4 scoping provisions.
**Alternative RWFD Design Method (NOT Meeting Special Seismic Detailing Requirements, 12.10.4)**

3. Toppings of concrete or similar materials that affect diaphragm strength or stiffness shall not be placed over the wood structural panel or bare steel deck diaphragm.

4. The diaphragm shall not contain horizontal structural irregularities, as specified in ASCE/SEI 7-22 Table 12.3-1, except that Horizontal Structural Irregularity Type 2 (reentrant corner irregularity) is permitted.

5. The diaphragm shall be rectangular in shape or shall be divisible into rectangular segments for purpose of seismic design, with vertical elements of the seismic force-resisting system or collectors provided at each end of each rectangular segment span.
6. The vertical elements of the seismic force-resisting system shall be limited to one or more of the following: concrete shear walls, precast concrete shear walls, masonry shear walls, steel concentrically braced frames, steel and concrete composite braced frames, or steel and concrete composite shear walls.

7. The vertical elements of the seismic force-resisting system shall be designed in accordance with ASCE/SEI 7-22 Section 12.8, except that they shall be permitted to be designed using the two-stage analysis procedure of ASCE/SEI 7-22 Section 12.2.3.2.2, where applicable.

As in the previous example, the structure meets all of the scoping limitations, allowing use of the Alternative RWFD Provisions.
Alternative RWFD Design Method (NOT Meeting Special Seismic Detailing Requirements, 12.10.4)

Step 2 - Break roof diaphragm into a series of rectangular segments for purposes of design with each segment spanning to vertical elements or a collector.

- Because the example building is rectangular in plan and shear walls are located at the building perimeter, one rectangular segment extending for the full building plan will be used.

Step 3 - Determine $w_{px}$

- $w_{px}$ was determined in Example Section 7.7.1 to be:
  - 6,846 kips (transverse forces)
  - 5,836 kips (longitudinal forces)

Steps 2 and 3 are the same as the previous example...
Alternative RWFD Design Method (NOT Meeting Special Seismic Detailing Requirements, 12.10.4)

Step 4 - Determine $R_{\text{diaph}}$
- $R_{\text{diaph}} = 1.5$ for bare steel deck diaphragms NOT meeting the special seismic detailing requirements for AISI S400

Step 5 - Determine $T_{\text{diaph}}$
- $T_{\text{diaph}} = 0.001L_{\text{diaph}}$ for bare steel deck diaphragms (ASCE/SEI 7-22 Section 12.10.4.2.1)
- $T_{\text{diaph}} = 0.001$ s/ft (600 ft) = 0.60 s (transverse forces)
- $T_{\text{diaph}} = 0.001$ s/ft (360 ft) = 0.36 s (longitudinal forces)

Step 4 is distinctly different from the previous example. Because the diaphragm system does NOT meet the special seismic detailing requirements of AISI S400, $R_{\text{diaph}}$ is required to be taken as 1.5 instead of 4.5. This will effectively triple the diaphragm design forces.

Step 5 is the same as the previous example. The diaphragm period is taken as a function of the diaphragm span and is independent of the diaphragm system.
Step 6 - Determine $C_{s\text{-diaph}}$

For transverse forces:

$$C_{s\text{-diaph}} = \frac{S_{ds}}{R_{\text{diaph}}} = \frac{1.0}{1.5} = 0.667$$  

(ASCE/SEI 7-22 Eq. 12.10-16a)

But need not exceed

$$C_{s\text{-diaph}} = \frac{S_{r1}}{T_{\text{diaph}}(R_{\text{diaph}})} = \frac{0.50}{0.60(1.5)} = 0.555$$  

(ASCE/SEI 7-22 Eq. 12.10-16b)

- Use $C_{s\text{-diaph}} = 0.555$ transverse

Because Rdiaph is taken as 1.5 instead of 4.5, the values calculated for $C_{s\text{-diaph}}$ are three times higher than the previous example. This is true for the transverse forces shown here.
For longitudinal forces:

\[ C_{s-diaph} = \frac{S_{diaph}}{R_{diaph}} = \frac{1.0}{1.5/1.0} = 0.667 \]  
\[ \text{But need not exceed:} \]

\[ C_{s-diaph} = \frac{S_{diaph}}{T_{diaph}(R_{diaph})} = \frac{0.50}{0.36(1.5)/1.0} = 0.926 \]  

\[ \text{Use } C_{s-diaph} = 0.667 \text{ longitudinal} \]

The \( C_{s-diaph} \) forces are also similarly increased for the longitudinal forces shown here.
Step 7 - Determine diaphragm design force, $F_{px}$

- $F_{px} = C_{s-diaph} \cdot (w_{px})$  
  (ASCE/SEI 7-22 Eq. 12.10-15)
- $F_{px} = 0.555 \cdot (6,846) = 3,800$ kips transverse
- $F_{px} = 0.667 \cdot (5,836) = 3,893$ kips longitudinal

Step 7 involves calculation of the $F_{px}$ forces, which are similarly increased by a factor of three.
Step 8 involves the determination of the amplified shear zone. The extent of this zone remains the same as the previous example.
Step 9 involves the diaphragm design for shear. The process is the same as the previous example, but the shear forces are three times higher.
Matching the previous example, the resulting shear diagram for seismic forces in the transverse direction is shown in this figure. The step in shear at the amplified shear zones can be seen at either end of the span. With the shear diagram determined, the specifics of the diaphragm decking and fastening can be determined.
Alternative RWFD Design Method (NOT Meeting Special Seismic Detailing Requirements, 12.10.4)

Step 9 - Diaphragm Design for Flexure

- For transverse roof diaphragm forces the chord force is calculated using $F_{px}$ forces without amplification:
  - $w = \frac{3,800 \text{ kips}}{600 \text{ ft}} = 6.33 \text{ klf}$
  - $M = \frac{6.33 \text{ klf} \times (600 \text{ ft})^2}{8} = 284,850 \text{ kip-ft}$
  - Chord $T/C = \frac{284,850 \text{ kip-ft}}{360 \text{ ft}} = 791 \text{ kips maximum at diaphragm mid-span}$

Like the design for shear, the process of design for flexure is the same as the previous example, but the forces remain three times higher.
Alternative RWFD Design Method (NOT Meeting Special Seismic Detailing Requirements, 12.10.4)

Step 10 - Determine collector forces in accordance with ASCE/SEI 7-22 Section 12.10.4.2.4. Collector forces are to be based on $F_{px}$ forces WITHOUT the 1.5 amplification factor, multiplied by $W_{0-diaph}$, however $W_{0-diaph}$ need not be taken as greater than $R_{diaph}$.

- Per ASCE/SEI 7-22 Section 12.10.4.2.4, the collector force is calculated based on the maximum transverse diaphragm shear WITHOUT amplification, multiplied by $W_{0-diaph}$, however $W_{0-diaph}$ need not be taken as greater than $R_{diaph}$.

- $T/C = 5.28$ klf (90 ft) (1.5) = 713 kips

Again, the collector force calculation will be illustrated at the location shown in this figure.

The same two points are important when calculating the collector forces:

1. The collector force is determined using the diaphragm design forces WITHOUT shear amplification, and

2. Instead of using the overstrength factor $\Omega_0$ based on the vertical system and taken from Table 12.2-1, Section 12.10.4.2.4 specifies that $\Omega_{0-diaph} = 2.0$. This value was derived from the research use to develop the Alternative RWFD provisions.
Step 11 - Check applicable ASCE/SEI 7-22 deflection and drift limitations.

- Where required by ASCE/SEI 7-22, determine $C_{d,\text{diaph}}$ and diaphragm deflections in accordance with ASCE/SEI 7-22 Section 12.10.4.2.5. All applicable ASCE/SEI 7-22 deflection and drift checks are to be completed. It is important that this include a check that the gravity system can accommodate the mid-span deflection of the roof diaphragm, and the P-D stability of the tilt-up wall panels when subject to the diaphragm deflection.

- This would be good to illustrate.

The Step 11 check of deflection and displacement limits is the same under the previous example and the Traditional Method.
Example One-Story RWFD Building with Steel Deck Diaphragm

Comparison of Methods

Finally, we will take a side-by-side look at the diaphragm design forces resulting from these design examples.
## Comparison of Design Methods

### Table 7.7-1: Comparison of Traditional and Alternative RWFD Design Forces

<table>
<thead>
<tr>
<th>Diaphragm Design Method</th>
<th>Special Seismic Detailing met?</th>
<th>Transverse</th>
<th>Longitudinal</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$F_{px}$ (kips)</td>
<td>$v_{px}$ (plf)</td>
</tr>
<tr>
<td>Traditional ASCE/SEI 7 22 Section 12.10.1 and 12.10.2</td>
<td>NA</td>
<td>1,712</td>
<td>2,370</td>
</tr>
<tr>
<td>Alternative RWFD ASCE/SEI 7 22 Section 12.10.4</td>
<td>Yes</td>
<td>1,266</td>
<td>1,760</td>
</tr>
<tr>
<td>Alternative RWFD ASCE/SEI 7 22 Section 12.10.4</td>
<td>No</td>
<td>3,800</td>
<td>5,280</td>
</tr>
</tbody>
</table>

Comparison of Traditional and Alternative RWFD Design Forces
## Comparison of Design Methods

### Table 7.7-2: Comparison of Traditional and Alternative RWFD Collector Forces

<table>
<thead>
<tr>
<th>Diaphragm Design Method</th>
<th>Special Seismic Detailing met?</th>
<th>Chord Force T/C (kips)</th>
<th>Collector Force T/C (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traditional ASCE/SEI 7 22 Section 12.10.1 and 12.10.2</td>
<td>NA</td>
<td>356</td>
<td>533</td>
</tr>
<tr>
<td>Alternative RWFD ASCE/SEI 7 22 Section 12.10.4</td>
<td>Yes</td>
<td>264</td>
<td>317</td>
</tr>
<tr>
<td>Alternative RWFD ASCE/SEI 7 22 Section 12.10.4</td>
<td>No</td>
<td>791</td>
<td>713</td>
</tr>
</tbody>
</table>

1.0 kip = 4.45 kN, 1.0 ft = 0.3048 m, 1.0 ft-kip = 1.36 kN-m

A similar pattern of increases and decreases in collector forces can be seen.
Part 2 - Closing Comments

The Alternative RWFD Provisions were developed through research that included both testing and numerical studies. The designs resulting from these provisions were demonstrated by numerical studies to provide improved seismic performance over the Traditional Design Method.

For the diaphragm illustrated in these examples, the use of the Alternative RWFD Provisions also resulted in significantly reduced diaphragm design forces in the diaphragm where AISI S400 special seismic detailing requirements were met. This reduction in diaphragm design forces should result in reduced cost of construction in addition to improved performance.

It is hoped that the improved performance and potential reduced cost will help inspire design engineers to implement the Alternative RWFD Provisions.
This slide is intended to initiate questions from participants.
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Learning Objectives

- Understand the parameters influencing nonstructural response
- Understand key changes coming in ASCE/SEI 7-22, including
  - The new seismic design force equation
  - How equipment support structures and platforms are handled
  - How distribution system supports are handled
- Understand how to use the 2020 NEHRP Provision Design Examples as a resource for nonstructural component design
Outline of Presentation

**Fundamentals**
- Overview and code development process
- Parameters influencing nonstructural response
- Seismic design force equation
- Equipment support structures and platforms and distribution system supports
- Accommodation of seismic relative displacements
- Code change summary

**Design Examples**
- Architectural concrete wall panel
- Seismic analysis of egress stairs
- HVAC fan unit support
- Piping system seismic design
- Elevated vessel seismic design

Note: Images without references are taken from FEMA P-2192-V1. Full references for partial citations are given in FEMA P-2192-V1.
Fundamentals

Nonstructural Components
Nonstructural Components

- Defined as “a part of an architectural, mechanical, or electrical system within or without a building or nonbuilding structure” (ASCE/SEI 7-22 Section 11.2)
- These items make up the majority of the replacement value of most buildings.
- Design based on two fundamentally different demands:
  - Resistance to inertial forces (seismic forces)
  - Accommodation of imposed displacements

Suspended Acoustic Tile Ceilings
Restrained by Compression Struts and Diagonal Splayed Wires
Relative Costs

The largest capital investment in most commercial buildings is in the nonstructural systems and contents.

Anticipated Behavior of Noncritical Nonstructural Components
From ASCE/SEI 7-22 Sections C13.1 and C13.1.3

**Minor Earthquake**
- Minimal damage, not likely to affect functionality

**Moderate Earthquake**
- Some damage that might affect functionality

**Design Earthquake**
- Major damage but significant falling hazards are avoided; likely loss of functionality

**MCE_R Earthquake**
- No implicit performance goals
# ASCE/SEI 7-22 Chapter 13: Seismic Design Requirements for Nonstructural Components

## Section 13.1
- Information on applicability of nonstructural design provisions

## Section 13.2
- Importance of the component or system
- Adequacy of component for seismic forces and certification requirements

## Section 13.3
- Acceleration and displacement demands for nonstructural components

## Section 13.4
- Design considerations for attachments of nonstructural components to the structure

## Section 13.5
- Design considerations for architectural components

## Section 13.6
- Design considerations for mechanical and electrical components
Code Development Process for Recent Revisions to Nonstructural Provisions

- General Research (through 2018)
  - Synthesize past research
  - Project research and analyses
  - Recommended equation
- BSSC PUC (2018-2020)
  - Issue Team 5 develops code proposal
  - Main PUC reviews, makes recommendations, ballots proposal
  - BSSC member orgs. ballot proposal
  - 2020 NEHRP Provisions published

- ASCE 7 Code Committee (2020-2021)
  - Nonstructural Issue Team refines proposal
  - ASCE 7 Seismic Subcommittee ballots proposal
  - Main committee ballots proposal
  - Public comment
  - Final approval for ASCE/SEI 7-22
  - ASCE/SEI 7-22 published
Key Terminology

- PCA: Peak Component Acceleration
- PFA: Peak Floor Acceleration
- PGA: Peak Ground Acceleration
Parameters Influencing Nonstructural Response

- Ground shaking intensity, PGA
- Building
  - Seismic force-resisting system
  - Building modal period, \( T_{n,bldg} \)
  - Building ductility, \( \mu_{bldg} \)
  - Building damping, \( \beta_{bldg} \)
  - Building configuration (such as plan and vertical irregularities)
  - Floor diaphragm rigidity
- Height of component within the building, \( z/h \)
- Component
  - Component period, \( T_{comp} \)
  - Component and/or anchorage ductility, \( \mu_{comp} \)
  - Component damping, \( \beta_{comp} \)
  - Component reserve strength margin, \( R_{po,comp} \)
Seismic Force-Resisting System

Reinforced Concrete SW

Steel SMRF

Key Takeaway

- Same component responds very differently in different seismic force-resisting systems

Figure Assumptions

- Elastic component assumed with $\beta_{comp}=5\%$
- Dataset includes 19 recordings with PGA>0.15g

Effect of building stiffness on PCA/PGA for instrumental recordings (from NIST GCR 18-917-43, 2018 and Lizundia paper in 2019 SEAOC Convention Proceedings)
Building Modal Periods, $T_{n,bldg}$

### Key Takeaway
- Longer period means less amplification
- Cantilever systems have more “whipping” action

**Effect of period of vibration and lateral system stiffness on PFA/PGA**

$$\alpha_0 = \text{Lateral stiffness ratio, defined as } \alpha_0 = \frac{H}{(GA/EI)^{0.5}}$$

- $H = \text{height,}$
- $GA = \text{shear rigidity of a shear beam}$
- $EI = \text{the flexural stiffness}$

- $\alpha_0 = 0$ represents a pure flexural model
- $\alpha_0$ approaching infinity represents a pure shear beam

*(from Miranda and Taghavi, 2009)*
Component Period, $T_{comp}$, and Building Period Resonance

Key Takeaway
- Normalized x-axis is helpful to understand influence of building in component response

Figure Assumptions
- Elastic component with $\beta_{comp}=5\%$
- Dataset includes eight recordings with PCA>0.9g

Relationship between PCA/PFA comparing spectra without normalization (left) and with normalization (right) by $T_{bldg}$ (from Kazantzi et al., 2019)
Sources of Component and/or Anchorage Ductility

1. Component
2. Connection of component to anchor
3. Anchor
Component/Anchorage Ductility, $\mu_{\text{comp}}$

$\mu_{\text{comp}} = 1.25$ (low)  
$\mu_{\text{comp}} = 1.5$ (moderate)  
$\mu_{\text{comp}} = 2$ (high)

**Key Takeaway**
- Ductility substantially reduces component response, particularly at resonance

**Figure Assumptions**
- Elastic component assumed with $\beta_{\text{comp}}=5\%$
- Dataset includes 86 recordings with PCA>0.9g

Mean response of PCA/PFA versus $T_{\text{comp}}/T_{\text{bldg}}$ for different levels of component ductility (*from NIST, 2018 and Lizundia, 2019*)
ATC-120 Proposed Seismic Design Force Equation

\[
\frac{F_p}{W_p} = PGA \times \left[ \left( \frac{PFA}{PGA} \right) / R_{\mu bldg} \right] \times \left[ \left( \frac{PCA}{PFA} \right) / R_{po, bldg} \right] \times I_p
\]

Reduction factor for building ductility

Reduction factor for component reserve strength
Evolution of Seismic Design Force Equation

**ASCE 7-16**

\[
\frac{F_p}{W_p} = (0.4S_{DS}) \times \left[ 1 + 2 \left( \frac{z}{h} \right) \right] \times \left( \frac{a_p}{R_p} \right) \times I_p
\]

**NIST GCR 18-917-43 (ATC-120)**

\[
\frac{F_p}{W_p} = \frac{PGA}{R_{\mu,bldg}} \times \left( \frac{PFA}{PGA} \right) \times \left( \frac{PCA}{PFA} \right) \times I_p
\]

**2020 NEHRP Provisions and ASCE 7-22**

\[
\frac{F_p}{W_p} = (0.4S_{DS}) \times \left( \frac{H_f}{R_{\mu}} \right) \times \left( \frac{C_{AR}}{R_{po}} \right) \times I_p
\]
PFA/PGA ($H_f$) Amplification Factor

$$H_f = 1 + a_1 \left( \frac{z}{h} \right) + a_2 \left( \frac{z}{h} \right)^{10}$$

or:

$$H_f = 1 + 2.5 \left( \frac{z}{h} \right)$$

where:

$$a_1 = \frac{1}{T_a} \leq 2.5$$

$$a_2 = [1 - (0.4/T_a)^2] \geq 0$$

$$T_a = C_t h_n^x$$

$$\frac{F_p}{W_p} = (0.4S_{DS}) \times \left[ \frac{H_f}{R_\mu} \right] \times \left[ \frac{C_{AR}}{R_{po}} \right] \times I_p$$
Building Ductility, $R_\mu$

$$R_\mu = (1.1 \frac{R}{(I_e \Omega_0)})^{1/2} \geq 1.3$$

$$\frac{F_p}{W_p} = (0.4S_{DS}) \times \left[ \frac{H_f}{R_\mu} \right] \times \left[ \frac{C_{AR}}{R_{po}} \right] \times I_p$$

where:

- $R$ = Response modification factor for the building or nonbuilding structure
- $I_e$ = Importance Factor for the building or nonbuilding structure
- $\Omega_0$ = Overstrength factor for the building or nonbuilding structure

For components at or below grade, $R_\mu$ shall be taken as 1.0.
**PCA/PFA** $(C_{AR})$

\[
\frac{F_p}{W_p} = (0.4S_{DS}) \times \left[ \frac{H_f}{R_\mu} \times \left[ \frac{C_{AR}}{R_{po}} \right] \times I_p \right]
\]

### Component Ductility

<table>
<thead>
<tr>
<th>Location of Component</th>
<th>Resonance</th>
<th>Category</th>
<th>Assumed Ductility</th>
<th>$(\frac{PCA}{PFA})$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground</td>
<td>More Likely</td>
<td>Elastic</td>
<td>$\mu_{comp}=1$</td>
<td>2.5</td>
</tr>
<tr>
<td></td>
<td>Less Likely</td>
<td>Any</td>
<td>-</td>
<td>1.0</td>
</tr>
<tr>
<td>Roof or Elevated Floor</td>
<td>More Likely</td>
<td>Elastic</td>
<td>$\mu_{comp}=1$</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td>Low</td>
<td>$\mu_{comp}=1.25$</td>
<td>2.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Moderate</td>
<td>$\mu_{comp}=1.5$</td>
<td>2.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>High</td>
<td>$\mu_{comp} \geq 2$</td>
<td>1.4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Less Likely</td>
<td>Any</td>
<td>-</td>
<td>1.0</td>
</tr>
</tbody>
</table>
Unlikely to be in resonance:

- \( \frac{T_{comp}}{T_{bldg}} < 0.5 \)
- \( \frac{T_{comp}}{T_{bldg}} > 1.5 \)
- \( T_{comp} \leq 0.06 \)
Component Resonance Ductility Factor, $C_{AR}$, and Component Strength, $R_{po}$

- Architectural components shall be assigned a factor per ASCE/SEI 7-22 Table 13.5-1

- Mechanical and electrical equipment shall be assigned a factor per ASCE/SEI 7-22 Table 13.6-1

\[
\frac{F_p}{W_p} = (0.4S_{DS}) \times \left[ \frac{H_f}{R_\mu} \right] \times \frac{C_{AR}}{R_{po}} \times I_p
\]
Alternative Procedure for Nonlinear Response History Analysis

\[
\frac{F_p}{W_p} = \frac{0.4 S_{DS}}{C} \times a_i \times \left[ \frac{C_{AR}}{R_{po}} \right] \times I_p
\]

where:

\( a_i \) = Maximum acceleration at Level \( i \) obtained from nonlinear response history analysis at the design earthquake ground motion.

- \( a_i \) replaces the ratio \( \frac{H_f}{R_\mu} \) in main seismic design force equation

- The nonlinear analysis can account for the following parameters:
  - \( H_f \), variation of acceleration up the height of the structure specific to its dynamic properties
  - \( R_\mu \), reduction of PFA due to the structure’s ductility
Three different types of supports:

- **Nonstructural component with integral equipment supports**
- **Equipment support platform supporting two mechanical components**
- **Distribution system support for piping**

*Images from FEMA P-2082-1 (2020) and FEMA E-74 (2012)*
Accommodation of Seismic Relative Displacements

\[ D_{pI} = D_{pIe} \]

- **Displacement within Structure A between Level x and Level y**

\[ D_p = \delta_{xA} - \delta_{yA} \]

but not greater than:

\[ D_p = \frac{(h_x-h_y)\Delta_{aA}}{h_{sx}} \]

- **Displacement between Structures A and B between Level x and Level y**

\[ D_p = |\delta_{xA}| + |\delta_{yB}| \]

but not greater than:

\[ D_p = \frac{h_x\Delta_{aA}}{h_{sx}} + \frac{h_y\Delta_{aB}}{h_{sx}} \]
The revisions for the nonstructural seismic design force equations in ASCE/SEI 7-22 are based on the following publications:

- NIST GCR 18-917-43: Recommendations for Improved Seismic Performance of Nonstructural Components (2018), produced by the Applied Technology Council ATC-120 project
- 2020 NEHRP Recommended Seismic Provisions for New Buildings and Other Structures, used to develop code proposals for ASCE/SEI 7-22
Proposed Equations in NIST GCR 18-917-43

- Key features in the proposed equations:
  - Refinement to relate PFA to PGA at different heights in the building and incorporate the building period, $T$
  - Inclusion of building ductility. Increased building ductility generally reduces nonstructural component response.
  - Refinement of relationship between PCA and PFA to account for resonance due to ratio of component period-to-building period and component ductility

Key features in the proposed equations (continued):

- Differentiation between ground-supported and superstructure-supported components for amplification factors
- Inclusion of components and attachments inherent overstrength, $R_{po}$
- Revision to architectural component categories in ASCE/SEI 7-16 Table 13.5
- Distinction addressing different parameters for the component and the equipment support structure
Revisions in the 2020 NEHRP Provisions

The NIST GCR 18-917-43 recommendations were used to develop code proposals for the 2020 NEHRP Provisions, which include the following key issues:

- Terminology revision: PGA $\rightarrow$ 0.4S_{DS}, PFA/PGA $\rightarrow$ H_f, PCA/PFA $\rightarrow$ C_{AR}, R_{\mu bldg} $\rightarrow$ R_{\mu}, R_{pocomp} $\rightarrow$ R_{po}
- R_{\mu} clarification, R_{\mu} = 1.0 for ground supported components
- Assignment of C_{AR}, R_{po}, and \Omega_{op} values for different components based on ductility and likelihood of resonance
- Separation of elevators and escalators in Table 13.6-1 into two different categories
Revisions in the 2020 NEHRP Provisions

- The NIST GCR 18-917-43 recommendations were used to develop code proposals for the 2020 NEHRP Provisions, which include the following key issues:
  - $R_{po}$ refinement for reasonable value for most components
  - Maximum (cap) value consensus of $1.6S_{DS}I_p W_p$ to be compatible with analytical results
  - Overstrength factor in concrete and masonry, $\Omega_{op}$ compatible with the $F_p$ equation
  - Addition of detailed provisions for different types of supports
  - Clarification and improved requirements for seismic design of penthouses and rooftop structures
Revisions for ASCE/SEI 7-22

- The 2020 NEHRP Provisions were used to develop code proposals for ASCE/SEI 7-22, which include the following key issues:
  - Seismic Importance Factor, $I_e$, added to the denominator of the $R_\mu$ equation
  - Revision to specify direction of loading using $F_p$
  - Increase of $\Omega_{op}$ values for selected architectural components in Table 13.5-1 and mechanical and electrical components in Table 13.6-1
  - Increase of $R_{op}$ values for selected piping systems and duct systems for mechanical and electrical components in Table 13.6-1
Significant Changes from ASCE/SEI 7-16 to ASCE/SEI 7-22

- Definition of equipment supports (Section 11.2)
- Detailed scope of design criteria for nonstructural components (Section 13.1)
- Explicit load combinations for nonstructural components (Section 13.2.2)
- Required analysis for condition where the nonstructural component weight is equal to or greater than 20% the combined effective seismic weight, \( W \) (Section 13.2.9)
- Updated horizontal seismic design forces, \( F_p \) (Section 13.3.1)
  - Equation and coefficients are more rigorously based in instrumental records and analytical findings and better account for key parameters that affect response.
- Seismic design force provision using nonlinear response history analysis is updated; other dynamic analysis methods are removed (Section 13.3.1.5).
Significant Changes from ASCE/SEI 7-16 to ASCE/SEI 7-22 (cont.)

- $\Omega_{op}$ is required to increase the load effects for anchors in concrete or masonry, instead of $\Omega_0$ (Section 13.4.2).

- Architectural component list is expanded, and items account for updated coefficient for seismic design: $C_{AR}$, $R_{po}$, and $\Omega_{op}$ (Table 13.5-1). Example: Partitions split into short light frame, tall light frame, reinforced masonry and other

- Penthouse and rooftop structure requirements are added (Section 13.5.11).

- Mechanical and electrical component list is expanded, and items account for updated coefficient for seismic design: $C_{AR}$, $R_{po}$, and $\Omega_{op}$ (Table 13.6-1).

- Equipment support structures and platforms are required to be designed (Section 13.6.4.6).

- Distribution system supports are required to be designed (Section 13.6.4.7).
Minor Changes from ASCE/SEI 7-16 to ASCE/SEI 7-22

- Seismic Design Category applicability is extended (Section 13.1.2).
- Component Importance Factor, $I_p$ (Section 13.1.3)
- Nonstructural components exempt from seismic requirements are summarized in a table (Section 13.1.4).
- Most of general design requirements remain the same, with the exceptions noted previously (Section 13.2).
- Specific requirements for architectural components are mostly unchanged (Section 13.5).
- Except for the support conditions, specific requirements for mechanical and electrical components are mostly unchanged (Section 13.6).
Unchanged in ASCE/SEI 7-22 (same as ASCE/SEI 7-16)

- Horizontal seismic forces maximum and minimum values remain the same (Section 13.3.1).
  \[
  \frac{F_p}{W_p} \leq 1.6S_{DS}I_p \quad \text{(Equation 13.3-2)}
  \]
  \[
  \frac{F_p}{W_p} \geq 0.3S_{DS}I_p \quad \text{(Equation 13.3-3)}
  \]
- Seismic relative displacements equations are unchanged (Section 13.3.2).
- Component fundamental period is determined with the same equation (Section 13.3.3).
Questions?
Design Examples

Nonstructural Components
Design Examples for Architectural Components

- Architectural concrete wall panel
  - Providing gravity support and accommodating story drift in cladding
  - Spandrel panel
  - Column cover
  - Prescribed seismic displacements

- Seismic analysis of egress stairs
  - Prescribed seismic forces
  - Prescribed seismic displacements

- HVAC fan unit support
  - Case 1: Direct attachment to structure
  - Case 2: Support on vibration isolation springs

- Piping system seismic design
  - Piping system design
  - Pipe supports and bracing
  - Prescribed seismic displacements

- Elevated vessel seismic design
  - Vessel support and attachments
  - Supporting frame
Architectural Concrete Wall Panel

- Providing gravity support and accommodating story drift in cladding
- Spandrel panel
- Column cover
- Prescribed seismic displacements

Public domain image from piqsels.com
Architectural Concrete Wall Panel Description

Example Summary

- **Nonstructural component**: Architectural – exterior nonstructural wall elements and connections
- **Building seismic force-resisting system**: Steel special moment frames
- **Equipment support**: Not applicable
- **Occupancy**: Office
- **Risk Category**: II
- **Component Importance Factor**: $I_p = 1.0$
- **Number of stories**: 5
- $S_{DS} = 1.487$

- Architectural components are a 4.5-inch-thick NWC spandrel panel and column cover
- Spandrel panel supports glass windows weighing 10 psf
- The girders at Level 3 support the spandrel panel under consideration.
- The column cover under consideration is between Level 3 and Level 4.
Architectural Concrete Wall Panel Description

Five-story building showing panels location

Detailed building elevation

EL. 67'-6"

EL. 40'-6"
Providing Gravity Support and Accommodating Story Drift in Cladding

- Understanding the cladding system components is the first step for the concrete precast panel seismic design.
- Two crucial items should be determined:
  - Gravity support approach for the precast panel components
  - Mechanism to accommodate story drift
- Approaches to accommodate interstory drift
  - Rocking
  - Sliding

Rocking and sliding mechanisms in panels
Rocking Cladding Connection System
Rocking Cladding Connection System
Window Framing System Racking Mechanism

Initial position

Deflected position
**Coefficients for Architectural Components (Table 13.5-1)**

- Exterior nonstructural wall elements and connections – wall element, and body of wall panel connections:
  - $C_{AR} = 1.0$
  - $R_{po} = 1.5$
  - $\Omega_{0p} = 2.0$

- Exterior nonstructural wall elements and connections – fasteners of the connecting system:
  - $C_{AR} = 2.8$
  - $R_{po} = 1.5$
  - $\Omega_{0p} = 1.0$
ASCE/SEI 7-22 Parameters and Coefficients

- Design coefficients and factors for seismic force-resisting system (Table 12.2-1)
  - Steel SMRF: $R = 8.0$ and $\Omega_0 = 3.0$
- Short period design spectral acceleration, $S_{DS} = 1.487$
- Seismic Importance Factor, $I_e = 1.0$
- Component Importance Factor, $I_p = 1.0$
- Redundancy factor, $\rho = 1.0$
- Height of attachment at Level 3, $z = 40.5$ ft
- Average roof height, $h = 67.5$ ft
- Story height, $h_{sx} = 13.5$ ft
Spandrel panel weight

\[ W_p = (150 \text{ lb/ft}^3)(24 \text{ ft long })(6.5 \text{ ft high})(4.5 \text{ in. thick}/12 \text{ ft/in. }) = 8,775 \text{ lb} \]

Glass weight,

\[ W_p = (10 \text{ lb/ft}^2)(21 \text{ ft long})(7 \text{ ft high}) = 1,470 \text{ lb} \]

Column cover weight,

\[ W_p = (150 \text{ lb/ft}^3)(3 \text{ ft wide})(7 \text{ ft high})(4.5 \text{ in. thick}/12 \text{ ft/in. }) = 1,181 \text{ lb} \]

Approximate fundamental period of the supporting structure, \( T_a \) (Section 12.8.2.1)

Steel SMRF: \[ T_a = C_t h_n^x = (0.028)(67.5 \text{ ft})^{0.8} = 0.81 \text{ s} \]
Force amplification factor as a function of height in the structure, $H_f$

\[ a_1 = \frac{1}{T_a} = \frac{1}{0.81 \text{ s}} = 1.23 \leq 2.5 \]

\[ a_2 = [1 - (0.4/T_a)^2] = [1 - (0.4/0.81 \text{ s})^2] = 0.76 \geq 0 \]

\[ H_f = 1 + a_1 \left( \frac{z}{h} \right) + a_2 \left( \frac{z}{h} \right)^{10} = 1 + 1.23 \left( \frac{40.5 \text{ ft}}{67.5 \text{ ft}} \right) + 0.76 \left( \frac{40.5 \text{ ft}}{67.5 \text{ ft}} \right)^{10} = 1.74 \]

Compare $H_f$ value using alternative equation that does not require $T_a$:

\[ H_f = 1 + 2.5 \left( \frac{z}{h} \right) = 1 + 2.5 \left( \frac{40.5 \text{ ft}}{67.5 \text{ ft}} \right) = 2.50 \quad (44\% \text{ increase}) \]

Structure ductility reduction factor, $R_\mu$

\[ R_\mu = (1.1 \frac{R}{(I_e \Omega_0)})^{1/2} = (1.1(8/((1)(3)))^{1/2} = 1.71 \geq 1.3 \]
Applicable Requirements

- Component failure shall not cause failure of an essential architectural, mechanical, or electrical component (Section 13.2.4).
- Seismic attachments shall be bolted, welded, or otherwise positively fastened without considering the frictional resistance produced by the effects of gravity (Section 13.4).
- $F_p$ shall be applied at the component’s center of gravity and distributed relative to the component’s mass distribution (Section 13.3.1).
- Effects of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate (Section 13.3.2).
- Exterior nonstructural wall panels or elements that are attached to or enclose the structure shall be designed to accommodate the seismic relative displacements, and movements caused by temperature changes (Section 13.5.3).
Spandrel Panel Layout

Spandrel panel layout connection from interior

Spandrel panel section at midspan

Note: alternative layout is upper angle to be welded to top flange of steel beam.
Prescribed Seismic Forces: Wall Element and Body of Wall Panel Connections

- Spandrel panel and glass weight, \( W_p = D = 8,775 \text{ lb} + 1,470 \text{ lb} = 10,245 \text{ lb} \)
- Seismic design force, \( F_p \)

\[
F_p = 0.4S_{DS}I_p W_p \left[ \frac{H_f}{R_\mu} \right] \left[ \frac{C_{AR}}{R_{po}} \right] = 0.4(1.487)(1.0)(W_p) \left[ \frac{1.74}{1.71} \right] \left[ \frac{1.0}{1.5} \right] = 0.403W_p
\]

\[
F_{p,max} = 1.6S_{DS}I_p W_p = 1.6(1.487)(1.0)(W_p) = 2.379W_p
\]

\[
F_{p,min} = 0.3S_{DS}I_p W_p = 0.3(1.487)(1.0)(W_p) = 0.446W_p \quad \text{(controlling equation)}
\]

\[
F_p = 0.446W_p = 0.446(10,245 \text{ lb}) = 4,570 \text{ lb} \quad \text{(controlling seismic design force)}
\]
Prescribed Seismic Forces: Wall Element and Body of Wall Panel Connections

- Horizontal seismic load effect, \( E_h \)
  \[ Q_E = F_p = 4,570 \text{ lb} \]
  \[ E_h = \rho Q_E = (1.0)(4,570 \text{ lb}) = 4,570 \text{ lb} \]

- Vertical seismic load effect, \( E_v \)
  \[ E_v = 0.2 S_{DS} D = (0.2)(1.487g)(10,245 \text{ lb}) = 3,047 \text{ lb} \]

- Basic Load Combinations for Strength Design to determine the design member and connection forces to be used in conjunction with seismic and gravity loads:
  \[ 1.2D + E_v + E_h + L + 0.2S \]  (Load Combination 6)
  \[ 0.9D - E_v + E_h \]  (Load Combination 7)

For nonstructural components, the terms \( L \) and \( S \) are typically zero.
The vertical forces, $V_u$, horizontal forces, $H_u$, and moments, $M_u$, are calculated using the applicable strength load combinations.

Spandrel panel bending moments
Proportioning and Design: Wall Element and Body of Wall Panel Connections

- **Basic Load Combination 1: 1.4D**
  \[ V_u = 1.4D = 1.4(10,245 \text{ lb}) = 14,343 \text{ lb} \]  
  \[ M_{ux} = \frac{V_u L}{8} = \frac{(14,343 \text{ lb})(24 \text{ ft})}{8} = 43,029 \text{ lb-ft} \]  
  (vertical downward force)  
  (strong axis bending moment)

- **Basic Load Combination 6: 1.2D + E_v + E_h + L + 0.2S**
  \[ V_{u,max} = 1.2D + E_v = 1.2(10,245 \text{ lb}) + 3,047 \text{ lb} = 15,341 \text{ lb} \]  
  \[ H_u = E_h = 4,570 \text{ lb} \]  
  \[ H_u = E_h = 4,570 \text{ lb} \]  
  \[ M_{ux,max} = \frac{V_{u,max} L}{8} = \frac{(15,341 \text{ lb})(24 \text{ ft})}{8} = 46,023 \text{ lb-ft} \]  
  (vertical downward force)  
  (hor. load parallel to panel)  
  (hor. load perpendicular to panel)  
  (strong axis bending moment)

\[ M_{uy} = \frac{H_u L}{32} = \frac{(4,570 \text{ lb})(24 \text{ ft})}{32} = 3,428 \text{ lb-ft} \]  
(weak axis bending moment)
Proportioning and Design: Wall Element and Body of Wall Panel Connections

- Basic Load Combination 7: $0.9D - E_v + E_h$
  
  $V_{u,\text{min}} = 0.9D - E_v = 0.9(10,245 \text{ lb}) - 3,047 \text{ lb} = 6,174 \text{ lb}$  
  (vertical downward force)

  $\parallel H_u = E_h = 4,570 \text{ lb}$  
  (horizontal load parallel to panel)

  $\perp H_u = E_h = 4,570 \text{ lb}$  
  (horizontal load perp. to panel)

  $M_{ux,min} = \frac{V_{u,\text{min}}L}{8} = \frac{(6,174 \text{ lb})(24 \text{ ft})}{8} = 18,521 \text{ lb-ft}$  
  (strong axis bending moment)

  $M_{uy} = \frac{H_uL}{32} = \frac{(4,570 \text{ lb})(24 \text{ ft})}{32} = 3,428 \text{ lb-ft}$  
  (weak axis bending moment)
Prescribed Seismic Forces: Fasteners of the Connecting System

- The “fasteners of the connecting system” category is intended to apply to the connections with limited ductility that can have a brittle failure mechanism.

- Spandrel panel and glass weight, \( W_p = D = 8,775 \text{ lb} + 1,470 \text{ lb} = 10,245 \text{ lb} \)

- Seismic design force, \( F_p \)
  \[
  F_p = 0.4S_{DS}I_pW_p \left[ \frac{H_f}{R_R} \right] \frac{C_{AR}}{R_{po}} = 0.4(1.487)(1.0)(W_p) \left[ \frac{1.74}{1.71} \right] \frac{2.8}{1.5} = 1.129W_p \text{ (controlling equation)}
  \]

  \[
  F_{p,max} = 1.6S_{DS}I_pW_p = 1.6(1.487)(1.0)(W_p) = 2.379W_p
  \]

  \[
  F_{p,min} = 0.3S_{DS}I_pW_p = 0.3(1.487)(1.0)(W_p) = 0.446W_p
  \]

  \[
  F_p = 1.1291W_p = 1.129(10,245 \text{ lb}) = 11,568 \text{ lb} \text{ (controlling seismic design force)}
  \]

For this example, \( F_p \) almost triples when compared to the spandrel panel wall element calculations.
Prescribed Seismic Forces: Fasteners of the Connecting System

- Horizontal seismic load effect, $E_h$
  
  \[ Q_E = F_p = 11,568 \text{ lb} \]
  \[ E_h = \rho Q_E = (1.0)(11,568 \text{ lb}) = 11,568 \text{ lb} \]

- Vertical seismic load effect, $E_v$
  
  \[ E_v = 0.2S_{DS}D = (0.2)(1.487g)(10,245 \text{ lb}) = 3,047 \text{ lb} \]

- Basic Load Combinations for Strength Design to determine the design member and connection forces to be used in conjunction with seismic and gravity loads:
  
  1. $1.2D + E_v + E_h + L + 0.2S$ \quad \text{(Load Combination 6)}
  2. $0.9D - E_v + E_h$ \quad \text{(Load Combination 7)}

For nonstructural components, the terms $L$ and $S$ are typically zero.
Proportioning and Design: Fasteners of the Connecting System

- Proportioning and design is determined in a similar manner as the “wall element and body of wall panel connections”
- Refer to NEHRP Design Examples. The vertical forces, $V_u$, horizontal forces, $H_u$, and moments, $M_u$, are calculated using the applicable strength load combinations.

Spandrel panel connection and design forces
Concrete Cover Layout and Seismic Forces

Column cover layout connection

Column cover connection forces
Prescribed Seismic Displacements

- Calculations based on allowable story drift requirements.
- Since this is a five-story building, does not use masonry in the primary seismic force-resisting system, and it is in Risk Category II, the allowable story drift is $0.020h_{sx}$.

Story height, $h_{sx} = 13.5$ ft

Height of upper and lower support attachment for column cover, $h_x = 47.75$ ft and $h_y = 41.75$ ft

Seismic relative displacements, $D_{pl}$

$$D_p = \frac{(h_x-h_y)\Delta aA}{h_{sx}} = \frac{(47.75\text{ft}-41.75\text{ft})(12\text{ in./ft})(0.020h_{sx})}{h_{sx}} = 1.44 \text{ in.}$$

$D_{pl} = D_p I_e = D_p I_e = (1.44 \text{ in.})(1.0) = 1.44 \text{ in.}$

- The joints at the top and bottom of the column cover must be designed to accommodate an in-plane relative displacement of 1.44 inches.
Prescribed Seismic Displacements: Accommodating Drift in Glazing

- Drift requirements for glazing are in ASCE/SEI 7-22 Section 13.5.9.

- Clearance shall be large enough so that the glass panel will not fall out of the frame as required by $\Delta_{\text{fallout}} \geq \text{maximum} (1.25D_{pl}, 0.5")$

- This can be satisfied in several ways:
  - By test or engineering analysis. The test is AAMA 501.6, *Recommended Dynamic Test Method for determining the Seismic Drift Causing Glass Fallout from a Wall System* (AAMA, 2018).
  - Use fully tempered monolithic glass in Risk Category I, II, or III and less than 10 feet above a walking surface.
  - Use annealed or heat-strengthened laminated glass in single thickness with interlayer no less than 0.030 in. that is captured mechanically in a wall system glazing pocket, and whose perimeter is secured to the frame by a wet glazed gunable curing elastomeric sealant perimeter bead of $\frac{1}{2}$ in. (13mm) minimum glass contact width, or other approved anchorage system.
Prescribed Seismic Displacements: Accommodating Drift in Glazing

- Or the prescriptive formula of Section 13.5.9.1, Exception 1 can be used:

\[ D_{\text{clear}} = 2c_1 \left(1 + \frac{h_p c_2}{b_p c_1}\right) \geq 1.25 D_{pl} = \text{maximum (1.25 } D_p I_e \text{ and 0.5")} \]

Scenario 1: Glass does not move

Scenario 2: Glass translates but does not rock
Prescribed Seismic Displacements: Accommodating Drift in Glazing

- Combining translating and rocking gives

\[ D_{clear} = 2c_1 \left( 1 + \frac{h_pc_2}{b_pc_1} \right) \]

- This assumes the toe of the glass can rotate downward. Limitations on rotation due to location and flexibility of setting blocks under the glass need to be addressed.

Scenario 3: Glass translates and rocks with vertical movement at both sill and head
Questions?
Seismic Analysis of Egress Stairs

- Prescribed seismic forces
- Prescribed seismic displacements

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### Example Summary

<table>
<thead>
<tr>
<th><strong>Nonstructural components</strong></th>
<th>Architectural – egress stairways not part of the building seismic force-resisting system</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Architectural – egress stair and ramp fasteners and attachments</td>
</tr>
<tr>
<td><strong>Building seismic force-resisting systems</strong></td>
<td>East–west direction: steel special concentrically braced frames</td>
</tr>
<tr>
<td></td>
<td>North–south direction: steel special moment frames</td>
</tr>
<tr>
<td><strong>Equipment support</strong></td>
<td>Not applicable</td>
</tr>
<tr>
<td><strong>Occupancy</strong></td>
<td>Emergency medical facility</td>
</tr>
<tr>
<td><strong>Risk Category</strong></td>
<td>IV</td>
</tr>
<tr>
<td><strong>Component Importance Factor</strong>: $I_p$</td>
<td>$= 1.5$</td>
</tr>
<tr>
<td><strong>Number of stories</strong></td>
<td>5</td>
</tr>
<tr>
<td><strong>$S_{DS}$</strong></td>
<td>$= 1.00$</td>
</tr>
</tbody>
</table>

- Calculations for flight of stairs and landing between Level 3 and Level 4
- Effective dead load: 25 psf
- Design live load: 100 psf
Egress Stairs Description

Plan of egress stairs

EL. 28'-0"
EL. 42'-0"
EL. 70'-0"

Elevation of egress stairs
ASCE/SEI 7-22 Parameters and Coefficients

Coefficients for Architectural Components (Table 13.5-1)

- Egress stairways not part of the building seismic force-resisting system:
  - \( C_{AR} = 1.0 \)
  - \( R_{po} = 1.5 \)
  - \( \Omega_{0p} = 2.0 \)

- Egress stairs and ramp fasteners and attachments:
  - \( C_{AR} = 2.2 \)
  - \( R_{po} = 1.5 \)
  - \( \Omega_{0p} = 1.75 \)
ASCE/SEI 7-22 Parameters and Coefficients

- Design coefficients and factors for seismic force-resisting systems (Table 12.2-1)
  - E-W direction – steel SCBF: $R = 6.0$ and $\Omega_0 = 2.0$
  - N-S direction – steel SMRF: $R = 8.0$ and $\Omega_0 = 3.0$
- Short period design spectral acceleration, $S_{DS} = 1.00$
- Seismic Importance Factor, $I_e = 1.5$
- Component Importance Factor, $I_p = 1.5$
- Redundancy factor, $\rho = 1.0$
- Average height of attachments, $z = 35$ ft
- Average roof height, $h = 70$ ft
- Story height, $h_{sx} = 14$ ft
Stair flight weight, \( W_p = (20 \text{ lb/ft}^2)(10.083 \text{ ft long})(3.5 \text{ ft wide}) = 706 \text{ lb} \)

Stair landing weight, \( W_p = (20 \text{ lb/ft}^2)(7.333 \text{ ft long})(3.5 \text{ ft wide}) = 513 \text{ lb} \)

Approximate fundamental period of the supporting structure, \( T_a \) (Section 12.8.2.1)

- E-W direction – steel SCBF: \( T_a = C_t h_n x = (0.02)(70 \text{ ft})^{0.75} = 0.484 \text{ s} \)
- N-S direction – steel SMRF: \( T_a = C_t h_n x = (0.028)(70 \text{ ft})^{0.8} = 0.838 \text{ s} \)

Per Section 13.3.1.1, for structures with combinations of seismic force-resisting systems, the lowest value of \( T_a \) shall be used. For this example, \( T_a = 0.484 \text{ s} \) controls.
Force amplification factor as a function of height in the structure, $H_f$

$$a_1 = \frac{1}{T_a} = \frac{1}{0.484 s} = 2.07 \leq 2.5$$

$$a_2 = [1 - (0.4/T_a)^2] = [1 - (0.4/0.484 s)^2] = 0.32 \geq 0$$

$$H_f = 1 + a_1 \left(\frac{z}{h}\right) + a_2 \left(\frac{z}{h}\right)^{10} = 1 + 2.07 \left(\frac{35 \text{ ft}}{70 \text{ ft}}\right) + 0.32 \left(\frac{35 \text{ ft}}{70 \text{ ft}}\right)^{10} = 2.03$$

Structure ductility reduction factor, $R_\mu$

$$R_\mu = (1.1 R/(I_e \Omega_0))^{1/2} \geq 1.3$$

- E-W direction – steel SCBF: $R_\mu = (1.1(6/((1.5)(2)))^{1/2} = 1.48 \geq 1.3$
- N-S direction – steel SMRF: $R_\mu = (1.1(8/((1.5)(3)))^{1/2} = 1.40 \geq 1.3$

Per Section 13.3.1.2, if the structure contains a combination of seismic force-resisting systems in different directions, the lowest value of $R_\mu$ shall be used. For this example, $R_\mu = 1.40$ controls.
Applicable Requirements

- Supports, attachments, and the egress stairs themselves shall be designed to meet the seismic requirements of Chapter 13 (Section 13.2.1).
- Component failure shall not cause failure of an essential architectural, mechanical, or electrical component (Section 13.2.4).
- Seismic attachments shall be bolted, welded, or otherwise positively fastened without considering the frictional resistance produced by the effects of gravity (Section 13.4).
- $F_p$ shall be applied at the component’s center of gravity and distributed relative to the component’s mass distribution (Section 13.3.1).
- Effects of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate (Section 13.3.2).
The net relative displacement shall be assumed to occur in any horizontal direction, and it shall be accommodated through slotted or sliding connections, or metal supports designed with rotation capacity to accommodate $D_{pl}$ (Section 13.5.10).

Sliding connections with slotted or oversize holes, sliding bearing supports with restraints that engage after the displacement, $D_{pl}$, is exceeded, and connections that permit movement by deformation of metal attachments, shall accommodate a displacement $D_{pl}$, but not less than 0.5 in. (13 mm), without loss of vertical support or inducement of displacement-related compression forces in the stair (Section 13.5.10).

The strength of the supports shall not be limited by bolt shear, weld fracture, or other limit states with lesser ductility (Section 13.5.10).
Prescribed Seismic Forces:
Egress Stairways not Part of the Building Seismic Force-Resisting System

Flight of Stairs

- Component weight, \( W_p = D = 706 \text{ lb} \)
- Seismic design force, \( F_p \)

\[
F_p = 0.4S_{DS}I_pW_p \left[ \frac{H_f}{R_\mu} \right] \left[ \frac{C_{AR}}{R_{po}} \right] = 0.4(1.0)(1.5)(W_p) \left[ \frac{2.03}{1.40} \right] \left[ \frac{1.0}{1.5} \right] = 0.582W_p \quad \text{(controlling equation)}
\]

\[
F_{p,\text{max}} = 1.6S_{DS}I_pW_p = 1.6(1.0)(1.5)(W_p) = 2.4W_p
\]

\[
F_{p,\text{min}} = 0.3S_{DS}I_pW_p = 0.3(1.0)(1.5)(W_p) = 0.45W_p
\]

\[
F_p = 0.582W_p = 0.582(706 \text{ lb}) = 410 \text{ lb} \quad \text{(controlling seismic design force)}
\]
Prescribed Seismic Forces:
Egress Stairways not Part of the Building Seismic Force-Resisting System

Flight of Stairs (Continued)

- Horizontal seismic load effect, \( E_h \)
  \[ Q_E = F_p = 410 \text{ lb} \]
  \[ E_h = \rho Q_E = (1.0)(335 \text{ lb}) = 410 \text{ lb} \]

- Vertical seismic load effect, \( E_v \)
  \[ E_v = 0.2S_{DS}D = (0.2)(1.0g)(706 \text{ lb}) = 141 \text{ lb} \]

- Basic Load Combinations for Strength Design to determine the design member and connection forces to be used in conjunction with seismic and gravity loads:
  \[ 1.2D + E_v + E_h + L + 0.2S \quad \text{(Load Combination 6)} \]
  \[ 0.9D - E_v + E_h \quad \text{(Load Combination 7)} \]
Prescribed Seismic Forces:
Egress Stairways not Part of the Building Seismic Force-Resisting System

Landing

- Component weight, \( W_p = D = 513 \text{ lb} \)
- Seismic design force, \( F_p \)

\[
F_p = 0.4 S_{DS} I_p W_p \left[ \frac{H_f}{R_\mu} \right] \left[ \frac{C_{AR}}{R_{po}} \right] = 0.4 (1.0)(1.5)(W_p) \left[ \frac{2.03}{1.40} \right] \left[ \frac{1.0}{1.5} \right] = 0.582 W_p \quad \text{(controlling equation)}
\]

\[
F_{p,max} = 1.6 S_{DS} I_p W_p = 1.6(1.0)(1.5)(W_p) = 2.4 W_p
\]

\[
F_{p,min} = 0.3 S_{DS} I_p W_p = 0.3(1.0)(1.5)(W_p) = 0.45 W_p
\]

\[
F_p = 0.582 W_p = 0.582(513 \text{ lb}) = 298 \text{ lb} \quad \text{(controlling seismic design force)}
\]
Prescribed Seismic Forces:
Egress Stairways not Part of the Building Seismic Force-Resisting System

Landing (Continued)

- Horizontal seismic load effect, $E_h$
  \[ Q_E = F_p = 298 \text{ lb} \]
  \[ E_h = \rho Q_E = (1.0)(298 \text{ lb}) = 298 \text{ lb} \]

- Vertical seismic load effect, $E_v$
  \[ E_v = 0.2S_{DS}D = (0.2)(1.0g)(513 \text{ lb}) = 103 \text{ lb} \]

- Basic Load Combinations for Strength Design to determine the design member and connection forces to be used in conjunction with seismic and gravity loads:
  
  \[
  1.2D + E_v + E_h + L + 0.2S \quad \text{(Load Combination 6)}
  \]
  
  \[
  0.9D - E_v + E_h \quad \text{(Load Combination 7)}
  \]
Increased Seismic Forces for Fasteners and Attachments

“Egress stair and ramp fasteners and attachments” definition in ASCE/SEI 7-22 Table 13.5-1 only applies at these connections.

- The attachment to the primary structure is not well delimited to the rest of the egress stairway connections.
- It is recommended to apply the increased design forces in attachments with a nonductile failure mechanism.
Prescribed Seismic Forces:
Egress Stairs and Ramp Fasteners and Attachments

Flight of Stairs

- Component weight, $W_p = D = 706$ lb
- Seismic design force, $F_p$
  \[
  F_p = 0.4 S_{DS} I_p W_p \left[ \frac{H_f}{R_{\mu}} \right] \left[ \frac{C_{AR}}{R_{po}} \right] = 0.4(1.0)(1.5)(W_p) \left[ \frac{2.03}{1.40} \right] \left[ \frac{2.2}{1.5} \right] = 1.280W_p \quad \text{(controlling equation)}
  \]
  \[
  F_{p,\text{max}} = 1.6 S_{DS} I_p W_p = 1.6(1.0)(1.5)(W_p) = 2.4W_p
  \]
  \[
  F_{p,\text{min}} = 0.3 S_{DS} I_p W_p = 0.3(1.0)(1.5)(W_p) = 0.45W_p
  \]
- $F_p = 1.280W_p = 1.280(706 \text{ lb}) = 903 \text{ lb} \quad \text{(controlling seismic design force)}$
- Compare increased design force, $F_p = 903$ lb for fasteners and attachments, against $F_p = 410$ lb for design of rest of egress stairs.
Prescribed Seismic Forces:
Egress Stairs and Ramp Fasteners and Attachments

Landing

- Component weight, \( W_p = D = 513 \text{ lb} \)
- Seismic design force, \( F_p \)

\[
F_p = 0.4 S_{DS} I_p W_p \left[ \frac{H_f}{R_\mu} \right] \left[ \frac{C_{AR}}{R_{po}} \right] = 0.4(1.0)(1.5)(W_p) \left[ \frac{2.03}{1.40} \right] \left[ \frac{2.2}{1.5} \right] = 1.280W_p \quad \text{(controlling equation)}
\]

\[
F_{p,max} = 1.6 S_{DS} I_p W_p = 1.6(1.0)(1.5)(W_p) = 2.4W_p
\]

\[
F_{p,min} = 0.3 S_{DS} I_p W_p = 0.3(1.0)(1.5)(W_p) = 0.45W_p
\]

\[
F_p = 1.280W_p = 1.280(513 \text{ lb}) = 657 \text{ lb} \quad \text{(controlling seismic design force)}
\]

- Compare increased design force, \( F_p = 657 \text{ lb} \) for fasteners and attachments, against \( F_p = 298 \text{ lb} \) for design of rest of egress stairs.
Prescribed Seismic Displacements

- Calculations based on allowable story drift requirements.
- Since this is a five-story building, does not use masonry in the primary seismic force-resisting system, and it is in Risk Category IV, the allowable story drift is \(0.010h_{sx}\).

  Story height, \(h_{sx} = 14\) ft

  Height of upper and lower support attachment, \(h_x = 42\) ft and \(h_y = 28\) ft

  Seismic relative displacements, \(D_{pl}\)

  \[ D_p = \frac{(h_x-h_y)\Delta_d}{h_{sx}} = \frac{(42\text{ ft} - 28\text{ ft})(12\text{ in./ft})(0.010h_{sx})}{h_{sx}} = 1.68\text{ in.} \]

  \[ D_{pl} = D_p I_e = (1.68\text{ in.})(1.5) = 2.52\text{ in.} \]

- The displacement can act in any direction, so the connection must be able to accommodate a total range of movement of two times \(D_{pl}\), or \(2 \times D_{pl} = 5.04\) in. in all directions.
Stairway Design Load Combinations

- The egress stairway and connections should be designed for the linear combination:
  \[
  \text{Design Load Combination} = \text{Inertial Force Demand} + \text{Displacement-Induced Demand}
  \]

- For this example, the following load combinations would be required in the analysis:
  \[
  EQX = \pm F_{px}
  \]
  \[
  EQY = \pm F_{py} \pm EQY_{drift}
  \]

- The unrestrained connection in the X-direction (longitudinal direction) and the induced demand at the fixed connection in the Y-direction (transverse direction) at Level 4 shall be able to accommodate the story drift, and the seismic relative displacements.
Questions?
HVAC Fan Unit Support

- Case 1: Direct attachment to structure
- Case 2: Support on vibration isolation springs

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Example Summary

- **Nonstructural components**
  - Case 1: Mechanical and electrical – HVAC fan unit
  - Case 2: Mechanical and electrical – spring-isolated component

- **Building seismic force-resisting system:** Ordinary reinforced masonry shear wall (bearing wall system)

- **Equipment support:** Integral

- **Occupancy:** Office

- **Risk Category:** II

- **Component Importance Factor:** \( I_p = 1.0 \)

- **Number of stories:** 3

- \( S_{DS} = 0.474 \)

- 4-foot high, 5-foot-wide, 8-foot-long, 3,000-pound HVAC fan unit that is supported on two long sides near each corner

- 4,000 psi normalweight concrete roof slab

- Three-story office building. All stories are 12-feet tall.
HVAC Fan Unit Support Description

- **Case 1:** Direct attachment to the structure using 36 ksi, carbon steel, cast-in-place anchors

- **Case 2:** Support on vibration isolation springs that are attached to the slab with 36 ksi, carbon steel, post-installed expansion anchors. The nominal gap between the vibration spring seismic restraints and the base frame of the fan unit is presumed to be greater than 0.25 in.
Coefficients for Mechanical and Electrical Components (Table 13.6-1)

- Air-side HVACR, fans, air handlers, air conditioning units, cabinet heaters, air distribution boxes, and other mechanical components constructed of sheet metal framing:
  - $C_{AR} = 1.4$
  - $R_{po} = 2.0$
  - $\Omega_{0p} = 2.0$

- Spring-isolated components and systems and vibration-isolated floors closely restrained using built-in or separate elastomeric snubbing devices or resilient perimeter stops:
  - $C_{AR} = 2.2$
  - $R_{po} = 1.3$
  - $\Omega_{0p} = 1.75$
ASCE/SEI 7-22 Parameters and Coefficients

- Design coefficients and factors for seismic force-resisting system (Table 12.2-1)
  - Bearing wall system – ordinary reinforced masonry shear wall: \( R = 2.0 \) and \( \Omega_0 = 2.5 \)
- Short period design spectral acceleration, \( S_{DS} = 0.474 \)
- Seismic Importance Factor, \( I_e = 1.0 \)
- Component Importance Factor, \( I_p = 1.0 \)
- Redundancy factor, \( \rho = 1.0 \)
- Height of attachment at roof, \( z = 36 \) ft
- Average roof height, \( h = 36 \) ft
ASCE/SEI 7-22 Parameters and Coefficients

- HVAC fan unit weight, $W_p = 3,000$ lb

- Approximate fundamental period of the supporting structure, $T_a$ (Section 12.8.2.1)
  Ordinary reinforced masonry shear walls (all other structural systems, per Table 12.8-2):
  
  \[ h_n = h = 36 \text{ ft} \]
  \[ C_t = 0.02 \]
  \[ x = 0.75 \]
  \[ T_a = C_t h_n^x = (0.02)(36 \text{ ft})^{0.75} = 0.29 \text{ s} \]
ASCE/SEI 7-22 Parameters and Coefficients

- Force amplification factor as a function of height in the structure, $H_f$
  
  $$a_1 = \frac{1}{T_a} = \frac{1}{0.29 \text{s}} = 3.45 > 2.5, \text{ use } a_1 = 2.5$$
  
  $$a_2 = [1 - (0.4/T_a)^2] = [1 - (0.4/0.29 \text{s})^2] = -0.90 < 0, \text{ use } a_2 = 0$$
  
  $$H_f = 1 + a_1 \left(\frac{z}{h}\right) + a_2 \left(\frac{z}{h}\right)^{10} = 1 + 2.5 \left(\frac{36 \text{ ft}}{36 \text{ ft}}\right) + 0 \left(\frac{36 \text{ ft}}{36 \text{ ft}}\right)^{10} = 3.5$$
  
  For supporting structures with $T_a \leq 0.4 \text{ s}$, the parameters $a_1$ and $a_2$ are controlled by their limits, i.e., $a_1 = 2.5$ and $a_2 = 0$.

- Structure ductility reduction factor, $R_\mu$
  
  $$R_\mu = (1.1 R/(I_e \Omega_0))^{1/2} = (1.1(2/((1.0)(2.5)))^{1/2} = 0.94 < 1.3, \text{ use } R_\mu = 1.3$$
Applicable Requirements

- Component failure shall not cause failure of an essential architectural, mechanical, or electrical component (Section 13.2.4).
- Seismic attachments shall be bolted, welded, or otherwise positively fastened without considering the frictional resistance produced by the effects of gravity (Section 13.4).
- $F_p$ shall be applied at the component’s center of gravity and distributed relative to the component’s mass distribution (Section 13.3.1).
- Attachments to concrete or masonry shall be designed to resist the seismic load effects including overstrength, $\Omega_0$ shall be taken as $\Omega_{0p}$ (Section 13.4.2).
- Exterior nonstructural wall panels or elements that are attached to or enclose the structure shall be designed to accommodate the seismic relative displacements, and movements caused by temperature changes (Section 13.5.3).
Applicable Requirements (Continued)

- Attachments and supports transferring seismic loads shall be constructed of materials suitable for the application and must be designed and constructed in accordance with a nationally recognized structural standard (Section 13.6.4.4).

- Components mounted on vibration isolation systems shall have a bumper restraint or snubber in each horizontal direction. Vertical restraints must be provided where required to resist overturning. Isolator housings and restraints must also be constructed of ductile materials. A viscoelastic pad, or similar material of appropriate thickness, must be used between the bumper and equipment item to limit the impact load. **Such components also must resist doubled seismic design forces if the nominal clearance (air gap) between the equipment support frame and restraints is greater than 0.25 in.** (Section 13.6.4.5 and Table 13.6-1, Footnote a).
Prescribed Seismic Forces:  
Case 1: Direct Attachment to Structure

- HVAC fan unit weight, $W_p = D = 3,000$ lb
- Seismic design force, $F_p$

\[
F_p = 0.4S_{DS}I_pW_p \left\{ \frac{H_f}{R_\mu} \right\} \left[ \frac{C_{AR}}{R_{po}} \right] = 0.4(0.474)(1.0)(W_p) \left[ \frac{3.5}{1.3} \right] \left[ \frac{1.4}{2.0} \right] = 0.357W_p \text{ (controlling equation)}
\]

\[
F_{p,\text{max}} = 1.6S_{DS}I_pW_p = 1.6(0.474)(1.0)(W_p) = 0.758W_p
\]

\[
F_{p,\text{min}} = 0.3S_{DS}I_pW_p = 0.3(0.474)(1.0)(W_p) = 0.142W_p
\]

\[
F_p = 0.357W_p = 0.357(3,000 \text{ lb}) = 1,072 \text{ lb} \quad \text{(controlling seismic design force)}
\]
Prescribed Seismic Forces:
Case 1: Direct Attachment to Structure

- Horizontal seismic load effect, $E_h$
  \[ Q_E = F_p = 1,072 \text{ lb} \]
  \[ E_h = \rho Q_E = (1.0)(1,072 \text{ lb}) = 1,072 \text{ lb} \]

- Vertical seismic load effect, $E_v$
  \[ E_v = 0.2 S_{DS} D = (0.2)(0.474g)(3,000 \text{ lb}) = 284 \text{ lb} \]

- Basic Load Combinations for Strength Design to determine the design member and connection forces to be used in conjunction with seismic loads:
  \[ 1.2D + E_v + E_h + L + 0.2S \]  \hspace{1cm} (Load Combination 6)
  \[ 0.9D - E_v + E_h \]  \hspace{1cm} (Load Combination 7)

For nonstructural components, the terms $L$ and $S$ are typically zero.
Seismic load effects are used to determine bolt shear, $V_u$, and tension, $T_u$ (where a negative value indicates tension).

Signs of $E_v$ and $E_h$ are selected to result in the largest value of $T_u$.
Proportioning and Design:  Case 1: Direct Attachment to Structure

- **Basic Load Combination 6:** $1.2D + E_v + E_h + L + 0.2S$
  
  $$V_u = \frac{E_h}{4 \text{ bolts}} = \frac{1,072 \text{ lb}}{4 \text{ bolts}} = 268 \text{ lb/bolt}$$

  $$T_u = \frac{(1.2D-E_v)(5.5/2 \text{ ft})-(E_h)(2 \text{ ft})}{(5.5 \text{ ft})(2 \text{ bolts})} = \frac{(1.2(3,000 \text{ lb})-284 \text{ lb})(5.5/2 \text{ ft})-(1,072 \text{ lb})(2 \text{ ft})}{(5.5 \text{ ft})(2 \text{ bolts})} = 634 \text{ lb/bolt (no tension)}$$

- **Basic Load Combination 6:** $0.9D - E_v + E_h$
  
  $$V_u = \frac{E_h}{4 \text{ bolts}} = \frac{1,072 \text{ lb}}{4 \text{ bolts}} = 268 \text{ lb/bolt}$$

  $$T_u = \frac{(0.9D-E_v)(5.5/2 \text{ ft})-(E_h)(2 \text{ ft})}{(5.5 \text{ ft})(2 \text{ bolts})} = \frac{(0.9(3,000 \text{ lb})-284 \text{ lb})(5.5/2 \text{ ft})-(1,072 \text{ lb})(2 \text{ ft})}{(5.5 \text{ ft})(2 \text{ bolts})} = 409 \text{ lb/bolt (no tension)}$$

- Anchors with design capacities exceeding the calculated demands would be selected using ACI 318 Chapter 17.
Prescribed Seismic Forces:
Case 2: Support on Vibration Isolation Springs

- HVAC fan unit weight, $W_p = D = 3,000$ lb
- Seismic design force, $F_p$

\[
F_p = 0.4S_{DS}I_pW_p \left[ \frac{H_f}{R_\mu} \right] \left[ \frac{CA_R}{R_{po}} \right] = 0.4(0.474)(1.0)(W_p) \left[ \frac{3.5}{1.3} \right] \left[ \frac{2.2}{1.3} \right] = 0.864W_p
\]

\[
F_{p,\text{max}} = 1.6S_{DS}I_pW_p = 1.6(0.474)(1.0)(W_p) = 0.758W_p \quad \text{(controlling equation)}
\]

\[
F_{p,\text{min}} = 0.3S_{DS}I_pW_p = 0.3(0.474)(1.0)(W_p) = 0.142W_p
\]

\[
F_p = 0.758W_p = 0.758(3,000 \text{ lb}) = 2,275 \text{ lb} \quad \text{(controlling seismic design force)}
\]
Prescribed Seismic Forces:
Case 2: Support on Vibration Isolation Springs

- Per Table 13.6-1 Footnote a, the design force should be taken as $2F_p$ if nominal clearance (air gap) between equipment and seismic restraint is greater than 0.25 in.

- Horizontal seismic load effect, $E_h$
  \[ Q_E = 2F_p = 2(2,275 \text{ lb}) = 4,550 \text{ lb} \]
  \[ E_h = \rho Q_E = (1.0)(4,550 \text{ lb}) = 4,550 \text{ lb} \]

- Vertical seismic load effect, $E_v$
  \[ E_v = 0.2S_{DS}D = (0.2)(0.474g)(3,000 \text{ lb}) = 284 \text{ lb} \]

- Basic Load Combinations for Strength Design to determine the design member and connection forces to be used in conjunction with seismic loads:
  \[ 1.2D + E_v + E_h + L + 0.2S \]  (Load Combination 6)
  \[ 0.9D - E_v + E_h \]  (Load Combination 7)
The seismic load effects are used to determine the bolt shear, $V_u$, and tension, $T_u$ (negative value indicates tension).

Design forces are determined by an analysis of earthquake forces applied in a diagonal horizontal direction.

ASHRAE A56 equations are used to estimate these demands.

Angle of diagonal bending, $\theta = \tan^{-1}\left(\frac{b}{a}\right)$

For this example, $\theta = \tan^{-1}\left(\frac{5.5\text{ ft}}{7.0\text{ ft}}\right) = 38.16^\circ$

Tension per isolator, $T_u = \frac{W_p - F_{pv}}{4} - \frac{F_{ph}}{2}\left(\frac{\cos \theta}{b} + \frac{\sin \theta}{a}\right)$

Compression per isolator, $C_u = \frac{W_p + F_{pv}}{4} + \frac{F_{ph}}{2}\left(\frac{\cos \theta}{b} + \frac{\sin \theta}{a}\right)$

Shear per isolator, $V_u = \frac{F_p}{4}$
Proportioning and Design:  
Case 2: Support on Vibration Isolation Springs

- Basic Load Combination 6: \(1.2D + E_v + E_h + L + 0.2S\)

\[
T_u = \frac{1.2D - E_v}{4} - \frac{E_h}{2} \left( \frac{\cos \theta}{b} + \frac{\sin \theta}{a} \right) = \frac{1.2(3,000 \text{ lb}) - 284 \text{ lb}}{4} - \frac{(4,550 \text{ lb})(2 \text{ ft})}{2} \left( \frac{\cos(38.16^\circ)}{5.5 \text{ ft}} + \frac{\sin(38.16^\circ)}{7 \text{ ft}} \right) = -223 \text{ lb}
\]

\[
C_u = \frac{1.2D + E_v}{4} + \frac{E_h}{2} \left( \frac{\cos \theta}{b} + \frac{\sin \theta}{a} \right) = \frac{1.2(3,000 \text{ lb}) + 284 \text{ lb}}{4} + \frac{(4,550 \text{ lb})(2 \text{ ft})}{2} \left( \frac{\cos(38.16^\circ)}{5.5 \text{ ft}} + \frac{\sin(38.16^\circ)}{7 \text{ ft}} \right) = 2,023 \text{ lb}
\]

\[
V_u = \frac{E_h}{4} = \frac{4,550 \text{ lb}}{4} = 1,138 \text{ lb}
\]

- Basic Load Combination 6: \(0.9D + E_v + E_h + L + 0.2S\)

\[
T_u = \frac{0.9D - E_v}{4} - \frac{E_h}{2} \left( \frac{\cos \theta}{b} + \frac{\sin \theta}{a} \right) = \frac{0.9(3,000 \text{ lb}) - 284 \text{ lb}}{4} - \frac{(4,550 \text{ lb})(2 \text{ ft})}{2} \left( \frac{\cos(38.16^\circ)}{5.5 \text{ ft}} + \frac{\sin(38.16^\circ)}{7 \text{ ft}} \right) = -448 \text{ lb}
\]

\[
C_u = \frac{0.9D + E_v}{4} + \frac{E_h}{2} \left( \frac{\cos \theta}{b} + \frac{\sin \theta}{a} \right) = \frac{0.9(3,000 \text{ lb}) + 284 \text{ lb}}{4} + \frac{(4,550 \text{ lb})(2 \text{ ft})}{2} \left( \frac{\cos(38.16^\circ)}{5.5 \text{ ft}} + \frac{\sin(38.16^\circ)}{7 \text{ ft}} \right) = 1,798 \text{ lb}
\]

\[
V_u = \frac{E_h}{4} = \frac{4,550 \text{ lb}}{4} = 1,138 \text{ lb}
\]

- The vibration isolator would be designed to resist these forces.
There is no component or a support that undergoes ductile yielding at a load level less than the design strength of the corresponding anchor.

The Basic Load Combination 7 including $\Omega_{0p}$ is applied to obtain the controlling vertical design tension force.

**Tension per isolator:**

\[
T_u = \frac{0.9D-E_v}{4} - \frac{\Omega_{0p}E_h h}{2} \left( \frac{\cos \theta}{b} + \frac{\sin \theta}{a} \right) = \\
T_u = \frac{0.9(3,000 \text{ lb})-284 \text{ lb}}{4} - \frac{(1.75)(4,550 \text{ lb})(2 \text{ ft})}{2} \left( \frac{\cos(38.16^\circ)}{5.5 \text{ ft}} + \frac{\sin(38.16^\circ)}{7 \text{ ft}} \right) = -1,237 \text{ lb}
\]

Acting concurrently with tension, the horizontal design shear force is:

\[
V_u = \frac{\Omega_{0p}E_h}{4} = \frac{(1.75)(4,550 \text{ lb})}{4} = 1,991 \text{ lb}
\]
Proportioning and Design: Case 2: Support on Vibration Isolation Springs

- Horizontal shear force applied at the top of the isolator generates a moment that induces prying action, which will increase the tension on the anchor.
- Each isolator is attached to the concrete slab with two anchors.
- Tension force per anchor including prying effects, $T_b$:
  \[ T_b = \frac{T_u}{2\text{ anchors}} - \left(\frac{5\text{ in.}}{2\text{ in.}}\right)\left(\frac{V_u}{2\text{ anchors}}\right) \]
  \[ T_b = -\frac{1,237\text{ lb}}{2\text{ anchors}} - \left(\frac{5\text{ in.}}{2\text{ in.}}\right)\left(\frac{1,991\text{ lb}}{2\text{ anchors}}\right) = -3,107\text{ lb} \]
- Design shear force per bolt, $V_b$:
  \[ V_b = \frac{V_u}{2\text{ anchors}} = \frac{1,991\text{ lb}}{2\text{ anchors}} = 995\text{ lb} \]
Questions?
Piping System Seismic Design

- Piping system design
- Pipe supports and bracing
- Prescribed seismic displacements

*Image from FEMA E-74 (2012) Figure 6.4.3.1-5 (courtesy of Mason Industries)*
Example Summary

- **Nonstructural components**: Mechanical and electrical – piping not in accordance with ASME B31 with threaded joints
- **Building seismic force-resisting system**: Steel buckling-restrained braced frames
- **Equipment support**: Distribution system supports – distribution system supports using hot-rolled steel bracing
- **Occupancy**: Acute care hospital
- **Risk Category**: IV
- **Component Importance Factor**: $I_p = 1.5$
- **Number of stories**: 2
- $S_{DS} = 1.00$

- Three piping runs of a chilled water piping system supported from the roof.
- The system is not intended to meet the ASME B31 requirements.
- One run of the piping system crosses a seismic separation joint to enter an adjacent structure.
Piping System Description

Plan of piping system
Piping System Description

Piping system near column line ‘A’
Piping System Description: Bracing

Typical trapeze-type support assembly with transverse bracing

Typical trapeze-type support assembly with longitudinal bracing
Piping System Description: System Configuration

**Piping Run “A”:** a 4-inch-diameter pipe, which connects to a large mechanical unit at Line 1 supported at the second level. It crosses a seismic separation between adjacent structures at Line 3.
Piping System Description: System Configuration

**Piping Run “B”:** A 6-inch-diameter pipe, which has a vertical riser to the second level at Line 3.
Piping Run “C”: a 4-inch-diameter pipe, which turns 90 degrees to parallel Line 3 at Column Line A-3.
Coefficients for Mechanical and Electrical components (Table 13.6-1)

- Distribution systems – Piping and tubing not in accordance with ASME B31, including in-line components, constructed of high- or limited-deformability materials, with joints made by threading, bonding, compression couplings, or grooved couplings:
  - $C_{AR} = 2.2$
  - $R_{po} = 2.0$
  - $\Omega_{0p} = 1.75$

- Distribution system supports – hot-rolled steel bracing:
  - $C_{AR} = 1.0$
  - $R_{po} = 1.5$
  - $\Omega_{0p} = 2.0$
ASCE/SEI 7-22 Parameters and Coefficients

- Design coefficients and factors for seismic force-resisting system (Table 12.2-1)
  - Bearing frame system – steel BRBF: \( R = 8.0 \) and \( \Omega_0 = 2.5 \)
- Short period design spectral acceleration, \( S_{DS} = 1.00 \)
- Seismic Importance Factor, \( I_e = 1.5 \)
- Component Importance Factor, \( I_p = 1.5 \)
- Redundancy factor, \( \rho = 1.0 \)
- Height of attachment at roof, \( z = 30 \) ft
- Average roof height, \( h = 30 \) ft
- Story height, \( h_{sx} = 15 \) ft
Piping and Braces Parameters

- Gravity (non-seismic) support spacing, $L_{grav\ sup} = 10$ ft
- Lateral brace spacing, $L_{lat\ brace} = 40$ ft
- Longitudinal brace spacing, $L_{long\ brace} = 80$ ft
- Length from Support 1 to mechanical unit, $L_{1M} = 9$ ft
- ASTM A53 pipe with threaded connections, $F_y = 35,000$ psi
- System working pressure, $P = 200$ psi
- 4-inch diameter water-filled pipe weight, $D = W_p = 16.4$ plf
- 6-inch diameter water-filled pipe weight, $D = W_p = 31.7$ plf
ASCE/SEI 7-22 Parameters and Coefficients

- Approximate fundamental period of the supporting structure, $T_a$ (Section 12.8.2.1)
  
  $$T_a = C_t h_n^x = (0.03)(30 \text{ ft})^{0.75} = 0.38 \text{ s}$$

- Force amplification factor as a function of height in the structure, $H_f$

  $$a_1 = \frac{1}{T_a} = \frac{1}{0.38 \text{ s}} = 2.63 > 2.5, \text{ use } a_1 = 2.5$$

  $$a_2 = [1 - (0.4/T_a)^2] = [1 - (0.4/0.38 \text{ s})^2] = -0.11 < 0, \text{ use } a_2 = 0$$

  $$H_f = 1 + a_1 \left(\frac{z}{h}\right) + a_2 \left(\frac{z}{h}\right)^{10} = 1 + 2.5 \left(\frac{30 \text{ ft}}{30 \text{ ft}}\right) + 0 \left(\frac{30 \text{ ft}}{30 \text{ ft}}\right)^{10} = 3.5$$

  For supporting structures with $T_a \leq 0.4 \text{ s}$, $a_1 = 2.5$ and $a_2 = 0$.

- Structure ductility reduction factor, $R_\mu$

  $$R_\mu = (1.1 R/((I_e \Omega_0))^{1/2} = (1.1(8/((1.5)(2.5))))^{1/2} = 1.53 \geq 1.3$$
Applicable Requirements

- Component failure shall not cause failure of an essential architectural, mechanical, or electrical component (Section 13.2.4).
- Seismic attachments shall be bolted, welded, or otherwise positively fastened without considering the frictional resistance produced by the effects of gravity (Section 13.4).
- $F_p$ shall be applied at the component’s center of gravity and distributed relative to the component’s mass distribution (Section 13.3.1).
- Effects of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate (Section 13.3.2).
- Piping system shall be designed for the seismic forces and $D_{pl}$ (Section 13.6.7).
- Distribution system supports shall be designed for seismic forces and $D_{pl}$. Distribution systems braced to resist vertical, transverse, and long seismic loads (Section 13.6.7).
Prescribed Seismic Forces: Piping System Design

- Seismic design force, \( F_p \)
  \[
  F_p = 0.4 S_{DS} I_p W_p \left[ \frac{H_f}{R_u} \right] \left[ \frac{CAR}{R_{po}} \right] = 0.4(1.0)(1.5)(W_p) \left[ \frac{3.5}{1.53} \right] \left[ \frac{2.2}{2.0} \right] = 1.508 W_p \quad \text{(controlling equation)}
  \]
  \[
  F_{p,max} = 1.6 S_{DS} I_p W_p = 1.6(1.0)(1.5)(W_p) = 2.40 W_p
  \]
  \[
  F_{p,min} = 0.3 S_{DS} I_p W_p = 3(1.0)(1.5)(W_p) = 0.45 W_p
  \]

- Horizontal seismic load effect, \( E_h \)
  \[
  E_h = \rho Q_E = \rho F_p = (1.0)(1,072 \text{ lb}) = 1,072 \text{ lb}
  \]

- Vertical seismic load effect, \( E_v \)
  \[
  E_v = 0.2 S_{DS} D = (0.2)(0.474g)(3,000 \text{ lb}) = 284 \text{ lb}
  \]

- Basic Load Combinations for Strength Design:
  \[1.2D + E_v + E_h + L + 0.2S \text{ (Load Combination 6)} \] and \[0.9D - E_v + E_h \text{ (Load Combination 7)} \]
Proportioning and Design: Piping System Design

4-in. diameter pipe (Pipe Runs “A” and “C”)
- Inner diameter, \( d_1 = 4.026 \text{ in.} \)
- Outer diameter, \( d = 4.5 \text{ in.} \)
- Wall thickness, \( t = 0.237 \text{ in.} \)
- Plastic modulus, \( Z = 4.31 \text{ in.}^3 \)
- Moment of inertia, \( I = 7.23 \text{ in.}^4 \)

6-in. diameter pipe (Pipe Run “B”)
- Inner diameter, \( d_1 = 6.065 \text{ in.} \)
- Outer diameter, \( d = 6.625 \text{ in.} \)
- Wall thickness, \( t = 0.28 \text{ in.} \)
- Plastic modulus, \( Z = 11.28 \text{ in.}^3 \)
- Moment of inertia, \( I = 28.14 \text{ in.}^4 \)
Proportioning and Design: Piping System Design

Gravity and Pressure Loads

- Longitudinal stresses in piping due to pressure and weight

\[ f_L = \frac{P_d}{4t} + \frac{M_g}{Z} \text{, where } M_g = \frac{(D)(t_{grav sup})^2}{8} \text{ is the resultant moment due to forces in gravity direction} \]

For 4-inch-diameter pipe

\[ M_g = \frac{(16.4 \text{ plf})(10 \text{ ft})^2}{8} = 2,460 \text{ lb-in.} \]

\[ f_{L,Dead} = \frac{M_g}{Z} = \frac{2,460 \text{ lb-in.}}{4.31 \text{ in.}^3} = 571 \text{ psi} \]

\[ f_{L,Pressure} = \frac{P_d}{4t} = \frac{(200 \text{ psi})(4.5 \text{ in.})}{4(0.237 \text{ in.})} = 949 \text{ psi} \]

For 6-inch-diameter pipe

\[ M_g = \frac{(31.7 \text{ plf})(10 \text{ ft})^2}{8} = 4,755 \text{ lb-in.} \]

\[ f_{L,Dead} = \frac{M_g}{Z} = \frac{4,755 \text{ lb-in.}}{11.28 \text{ in.}^3} = 422 \text{ psi} \]

\[ f_{L,Pressure} = \frac{P_d}{4t} = \frac{(200 \text{ psi})(6.625 \text{ in.})}{4(0.28 \text{ in.})} = 1,183 \text{ psi} \]
Proportioning and Design: Piping System Design

Seismic Loads on Piping Runs “A” and “C” – 4-in-diameter pipe

- Horizontal seismic load effect, $E_h$, and vertical seismic load effect, $E_v$

  \[ F_p = 1.508W_p = 1.508(16.4 \text{ plf}) = 24.7 \text{ lb/ft} \]

  \[ E_h = \rho Q_E = \rho F_p = (1.0)(24.7 \text{ lb/ft}) = 24.7 \text{ lb/ft} \]

  \[ E_v = 0.2S_{DS}D = 0.2(1.0)(16.4 \text{ plf}) = 3.28 \text{ lb/ft} \]

- Maximum moment due to horizontal seismic load, $M_{Eh}$, and associated flexural stress

  \[ M_{Eh} = \frac{(E_h)(L_{\text{lat brace}})^2}{8} = \frac{(24.7 \text{ lb/ft})(40 \text{ ft})^2}{8} = 4,946 \text{ lb-ft} = 59,353 \text{ lb-in.} \]

  \[ f_{bh} = \frac{M_{Eh}}{Z} = \frac{59,353 \text{ lb-in.}}{4.31 \text{ in.}^3} = 13,766 \text{ psi} \]
Seismic Loads on Piping Runs “A” and “C” – 4-in-diameter pipe (Continued)

- Maximum moment due to vertical seismic load, $M_{Ev}$, and associated flexural stress
  \[
  M_{Ev} = \frac{(E_v) (L_{grav sup})^2}{8} = \frac{(3.28 \text{ plf})(10 \text{ ft})^2}{8} = 41 \text{ lb-ft} = 492 \text{ lb-in.}
  \]
  \[
  f_{bv} = \frac{M_{Ev}}{Z} = \frac{492\text{lb-in.}}{4.31\text{in.}^3} = 114 \text{ psi}
  \]

- Design stress in the pipe, $f_u$, using Load Combination 6: $1.2D + E_v + E_h + L + 0.2S$
  \[
  f_u = 1.2(f_{L,Dead} + f_{L,Pressure}) + f_{bv} + f_{bh}
  \]
  \[
  f_u = 1.2(571 \text{ psi} + 949 \text{ psi}) + 114 \text{ psi} + 13,766 \text{ psi} = 15,704 \text{ psi}
  \]

- Permissible stress check, $f_u < 0.7F_y$, where $0.7F_y = 0.7 \times 35,000 \text{ psi} = 24,500 \text{ psi}$
  \[
  15,704 \text{ psi} < 24,500 \text{ psi} \quad \text{OK}
  \]
Seismic Loads on Piping Runs “B” – 6-in-diameter pipe

- Horizontal seismic load effect, $E_h$, and vertical seismic load effect, $E_v$
  \[
  F_p = 1.508W_p = 1.508(31.7 \text{ plf}) = 47.8 \text{ lb/ft}
  \]
  \[
  E_h = \rho Q_E = \rho F_p = (1)(47.8 \text{ lb/ft}) = 47.8 \text{ lb/ft}
  \]
  \[
  E_v = 0.2S_{DS}D = 0.2(1.0)(31.7 \text{ plf}) = 6.34 \text{ lb/ft}
  \]

- Maximum moment due to horizontal seismic load, $M_{Eh}$, and associated flexural stress
  \[
  M_{Eh} = \frac{(E_h)(L_{lat brace})^2}{8} = \frac{(47.8 \text{ lb/ft})(40 \text{ ft})^2}{8} = 9,560 \text{ lb-ft} = 114,725 \text{ lb-in.}
  \]
  \[
  f_{bh} = \frac{M_{Eh}}{Z} = \frac{114,725 \text{ lb-in.}}{11.28 \text{ in.}^3} = 10,171 \text{ psi}
  \]
Seismic Loads on Piping Runs “B” – 6-in-diameter pipe (Continued)

- Maximum moment due to vertical seismic load, $M_{Ev}$, and associated flexural stress

$$M_{Ev} = \frac{(E_v)(L_{grav\ sup})^2}{8} = \frac{(6.34 \text{ plf})(10 \text{ ft})^2}{8} = 79 \text{ lb-ft} = 951 \text{ lb-in.}$$

$$f_{bv} = \frac{M_{Ev}}{Z} = \frac{951 \text{ lb-in.}}{11.28 \text{ in.}^3} = 84 \text{ psi}$$

- Design stress in the pipe, $f_u$, using Load Combination 6: $1.2D + E_v + E_h + L + 0.2S$

$$f_u = 1.2(f_{L,\ Dead} + f_{L,\ Pressure}) + f_{bv} + f_{bh}$$

$$f_u = 1.2(421 \text{ psi} + 1,183 \text{ psi}) + 84 \text{ psi} + 10,171 \text{ psi} = 12,181 \text{ psi}$$

- Permissible stress check, $f_u < 0.7F_y$, where $0.7F_y = 0.7 \times 35,000 \text{ psi} = 24,500 \text{ psi}$

12,181 psi < 24,500 psi  OK
Design demands are calculated for vertical supports, lateral supports, and anchorage at Support 1.

The following elements should be designed:

- Beam f-g
- Hangers f-b and g-d
- Transvers brace a-f
- Longitudinal braces f-c and g-e
- Connections at a, b, c, d, and e
Prescribed Seismic Forces: Pipe Supports and Bracing

- Seismic design force, $F_p$
  
  \[
  F_p = 0.4 S_{DS} I_p W_p \left[ \frac{H_f}{R_u} \right] \left[ \frac{C_{AR}}{R_{po}} \right] = 0.4(1.0)(1.5)(W_p) \left[ \frac{3.5}{1.53} \right] \left[ \frac{1.0}{1.5} \right] = 0.914W_p \]  
  (controlling equation)

  \[
  F_{p,max} = 1.6 S_{DS} I_p W_p = 1.6(1.0)(1.5)(W_p) = 2.40W_p \]

  \[
  F_{p,min} = 0.3 S_{DS} I_p W_p = 3(1.0)(1.5)(W_p) = 0.45W_p \]

- Horizontal seismic load effect, $E_h$
  
  \[
  E_h = \rho Q_E = \rho F_p = (1.0)(0.914W_p) = 0.914W_p \]

- Vertical seismic load effect, $E_v$
  
  \[
  E_v = 0.2 S_{DS} D \]

- Basic Load Combinations for Strength Design:
  
  1. $1.2D + E_v + E_h + L + 0.2S$ (Load Combination 6) and
  2. $0.9D - E_v + E_h$ (Load Combination 7)
Proportioning and Design: Pipe Supports and Bracing

Vertical Loads

- For 4-inch diameter pipes
  
  Dead load, $P_{v4} = (D)(L_{grav sup}) = (16.4 \text{ plf})(10 \text{ ft}) = 164 \text{ lb}$

  Vertical seismic load, $P_{Ev4} = 0.2S_{DS}D(L_{grav sup}) = 0.2(1.0)(16.4 \text{ plf})(10 \text{ ft}) = 33 \text{ lb}$

- For 4-inch diameter pipes
  
  Dead load, $P_{v6} = (D)(L_{grav sup}) = (31.7 \text{ plf})(10 \text{ ft}) = 317 \text{ lb}$

  Vertical seismic load, $P_{Ev6} = 0.2S_{DS}D(L_{grav sup}) = 0.2(1.0)(31.7 \text{ plf})(10 \text{ ft}) = 63 \text{ lb}$
Longitudinal Lateral Loads

- For Piping Run “A”, the total length of pipe tributary to Support 1 is 40 feet (half the distance between longitudinal braces at Supports 1 and 3) plus 9 feet (length of pipe from Support 1 to Support M, the mechanical unit), or 49 feet:

\[ P_{X1A} = \rho F_p = \rho (0.914 W_p) = (1)(0.914)(16.4 \text{ lb/ft})(49 \text{ ft}) = 734 \text{ lb} \]

- For Piping Runs “B” and “C”, the total length of pipe tributary to Support 1 is approximately 80 feet:

\[ P_{X1B} = \rho F_p = \rho (0.914 W_p) = (1)(0.914)(31.7 \text{ lb/ft})(80 \text{ ft}) = 2,318 \text{ lb} \]

\[ P_{X1C} = \rho F_p = \rho (0.914 W_p) = (1)(0.914)(16.4 \text{ lb/ft})(80 \text{ ft}) = 1,199 \text{ lb} \]
Proportioning and Design: Pipe Supports and Bracing

Transverse Lateral Loads

- Pipes are idealized as continuous beams spanning between pinned connections, representing the transverse braces. Reaction at the beam’s midspan is calculated as:

\[ P_Z = \frac{5}{8} W (L_{left} + L_{right}) \]

- For Piping Run “A”, we analyze the transverse Support 1, which is adjacent to the mechanical unit. The total length between the support and the unit is \( L_{1M} = 9 \text{ ft} \)

\[ P_{Z1A} = \left( \frac{5}{8} \right) W (L_{left} + L_{right}) = \left( \frac{5}{8} \right) (\rho F_p) (L_{1M} + L_{lat brace}) \]

\[ P_{Z1A} = \left( \frac{5}{8} \right) (1)(0.914) \left( 16.4 \frac{\text{lb}}{\text{ft}} \right) (9 \text{ ft} + 40 \text{ ft}) = 459 \text{ lb} \]
Transverse Lateral Loads (Continued)

- For Piping Runs “B” and “C”, we assume that 5/8 of the total length of pipe on each side of Support 1 is laterally braced at Support 1.

\[
P_{Z1B} = \left(\frac{5}{8}\right) W(L_{left} + L_{right}) = \left(\frac{5}{8}\right) (\rho F_p)((2)(L_{lat \ brace}))
\]

\[
P_{Z1B} = \left(\frac{5}{8}\right) (1)(0.914) \left(\frac{31.7 \text{ lb}}{\text{ft}}\right) ((2)(40 \text{ ft})) = 1,449 \text{ lb}
\]

\[
P_{Z1C} = \left(\frac{5}{8}\right) W(L_{left} + L_{right}) = \left(\frac{5}{8}\right) (\rho F_p)((2)(L_{lat \ brace}))
\]

\[
P_{Z1C} = \left(\frac{5}{8}\right) (1)(0.914) \left(\frac{16.4 \text{ lb}}{\text{ft}}\right) ((2)(40 \text{ ft})) = 749 \text{ lb}
\]
Prescribed Seismic Displacements

Design for Displacements within Structures

- The building is designed for a maximum allowable story drift of 1.5% per floor.

  Allowable story drift, $\Delta_a = 0.015h_{sx} = 0.015(15 \text{ ft})(12 \text{ in./ft}) = 2.7 \text{ in.}$

- Seismic relative displacements, $D_{pl}$

  $D_{pl} = D_p I_e = (2.7 \text{ in. })(1.5) = 4.05 \text{ in.}$

- Drift can be accommodated by providing a flexible coupling or through bending in the pipe.

- Piping Run “A” connects to a large mechanical unit at Line 1 supported at Level 2. The entire story drift must be accommodated in the 5 feet piping drop.
Prescribed Seismic Displacements

Design for Displacements within Structures (Continued)

- For a 4-inch-diameter pipe, assuming the pipe is fixed against rotation at both ends, the shear and moments required to deflect the pipe 4.05 in. are:

\[
V = \frac{12EI D_{pl}}{L^3} = \frac{12(29,000,000 \text{ psi})(7.23 \text{ in.}^4)(4.05 \text{ in.})}{((5 \text{ ft})(12 \text{ in./ft}))^3} = 47,193 \text{ lb}
\]

\[
M = VL = (47,176 \text{ lb})(5 \text{ ft})(12 \text{ in./ft}) = 2,831,563 \text{ lb \cdot in.}
\]

- The stress in the pipe displaced \( D_{pl} \) is:

\[
f_b = \frac{M}{Z} = \frac{2,831,563 \text{ lb \cdot in.}}{4.31 \text{ in.}^3} = 656,750 \text{ psi}
\]

- The permissible stress is \( 0.7F_y = 0.7(35,000 \text{ psi}) = 24,500 \text{ psi} \).

- The demand far exceeds the capacity of the pipe. Therefore, either a flexible coupling or a loop piping layout is required to accommodate the story drift.
Prescribed Seismic Displacements

Design for Displacements within Structures (Continued)

- Piping Run “B” drops from the roof to Level 2 at Line 3. The drift is the same and it can be accommodated over the full story height of 15 feet.

\[
V = \frac{12EI D_p I}{L^3} = \frac{12(29,000,000 \text{ psi})(28.14 \text{ in.}^4)(4.05 \text{ in.})}{((15 \text{ ft})(12 \text{ in./ft}))^3} = 6,801 \text{ lb}
\]

\[
M = VL = (6,800 \text{ lb})(15 \text{ ft})(12 \text{ in./ft})/2 = 612,092 \text{ lb-in.}
\]

- The stress in the pipe displaced \(D_p I\) is:

\[
f_b = \frac{M}{Z} = \frac{612,092 \text{ lb-in.}}{11.28 \text{ in.}^3} = 54,264 \text{ psi}
\]

- The permissible stress is \(0.7F_y = 0.7(35,000 \text{ psi}) = 24,500 \text{ psi.}\)

- The demand exceeds the permissible stress in the pipe, but not by a wide margin.
Prescribed Seismic Displacements

Design for Displacements Between Structures

- At the roof, Piping Run “A” crosses a seismic separation between adjacent two-story structures. The story heights are 15 ft and buildings are designed for $\Delta_a = 0.015h_{sx}$.

  Deflections of the buildings:
  
  \[
  \delta_{XA} = \delta_{XB} = (2)0.015h_{sx} = (2)(0.015)(15 \text{ ft})(12 \text{ in./ft}) = 5.4 \text{ in.}
  \]

  Displacement demand:
  
  \[
  D_{pmax} = |\delta_{XA}| + |\delta_{XB}| = |5.4 \text{ in.}| + |5.4 \text{ in.}| = 10.8 \text{ in.}
  \]

  Seismic relative displacements:
  
  \[
  D_{pl} = D_pI_e = (10.8 \text{ in.})(1.5) = 16.2 \text{ in.}
  \]

- The seismic separation joint must accommodate movement perpendicular and parallel to the pipe. Assuming an 18-inch joint, the joint could vary from 8.1 in. to 32.4 in.
Questions?
Elevated Vessel Seismic Design

- Vessel support and attachments
- Supporting frame
Elevated Vessel Description

Example Summary

- **Nonstructural components:**
  Mechanical and electrical – pressure vessel not supported on skirts
- **Building seismic force-resisting system:** Special reinforced concrete shear walls
- **Equipment support:** Equipment support structures and platforms – Seismic Force-Resisting Systems with $R > 3$
- **Occupancy:** Storage
- **Risk Category:** II
- **Component Importance Factor:** $I_p = 1.0$
- **Number of stories:** 3
- $S_{DS} = 1.20$
- $S_1 = 0.65$

- Vessel supported by an OCBF platform with tension-only rods as braces.
- The vessel contains a non-hazardous compressed non-flammable gas.
- The weight of the vessel is less than 5% the total weight of the building structure, which is below the 20% threshold where ASCE/SEI 7-22 requires it be designed as a nonbuilding structure per Chapter 15.
Elevated Vessel Description

Elevated vessel – section

Elevated vessel – Level 3 plan
ASCE/SEI 7-22 Parameters and Coefficients

Changes in ASCE/SEI 7-22

ASCE/SEI 7-16 required the nonstructural components and supporting structure to be designed with the same seismic design forces, $F_p$, regardless of their interaction, and the force was based on the component properties. A platform supporting a pressure vessel would be designed for pressure vessel forces regardless of whether the platform structure was made of concrete, steel braced frames, or steel moment frames.

In ASCE/SEI 7-22, the concept of an equipment support structure or platform has been introduced and defined. Definitions are given in Section 11.2 and properties have been added to Table 13.6-1. Section 13.6.4.6 has been added to ASCE/SEI 7-22 to require that the support structures and platforms be designed in accordance with those properties. This permits a more accurate determination of forces that more realistically reflect the differences in dynamic properties and ductilities between the component and the support structure or platform.
Section 13.6.4.6 requires the engineers to select the seismic force-resisting system listed in Chapter 12 or Chapter 15 for equipment support structures and platforms.

$C_{AR}$ for the supported component cannot be less than that for the equipment support structure.

The reactions applied by the component to the support structure can either stay the same or are effectively scaled down using a two-stage analysis approach.
ASCE/SEI 7-22 Parameters and Coefficients

Coefficients for Mechanical and Electrical Components (Table 13.6-1)

- Mechanical and electrical components – Engines, turbines, pumps, compressors, and pressure vessels not supported on skirts and not within the scope of Chapter 15:
  - $C_{AR} = 1.0 \quad 1.4$
  - $R_{po} = 1.5$
  - $\Omega_{0p} = 2.0$

- Equipment support structures and platforms – Seismic Force-Resisting Systems with $R > 3$ (Building frame system – steel ordinary concentrically braced frame, $R = 6$ per Table 12.2-1):
  - $C_{AR} = 1.4$
  - $R_{po} = 1.5$
  - $\Omega_{0p} = 2.0$

New concept: Per Section 13.3.1.3, $C_{AR}$ for the vessel shall not be less than $C_{AR}$ used for equipment support structure or platform itself.
ASCE/SEI 7-22 Parameters and Coefficients

- Design coefficients and factors for seismic force-resisting system (Table 12.2-1)
  - Bearing wall system – special reinforced concrete shear walls: $R = 5.0$ and $\Omega_0 = 2.5$
- Short period design spectral acceleration, $S_{DS} = 1.20$
- Seismic Importance Factor, $I_e = 1.0$
- Component Importance Factor, $I_p = 1.0$
- Redundancy factor, $\rho = 1.0$
- Height of attachment at roof, $z = 28$ ft
- Average roof height, $h = 46$ ft
Vessel and legs weight, $D_{ves} = W_{p,ves} = 5,000$ lb
Supporting frame weight, $D_{sup} = W_{p,sup} = 1,000$ lb
Vessel leg length, $L_{leg} = 18$ in.

Steel material properties
- HSS sections: ASTM A500 Grade B, $F_y = 46,000$ psi, $F_u = 58,000$ psi
- Bars and Plates: ASTM A36, $F_y = 36,000$ psi, $F_u = 58,000$ psi
- Pipes: ASTM A53 Grade B, $F_y = 35,000$ psi, $F_u = 60,000$ psi
- Bolts and threaded rods: ASTM A307
Approximate fundamental period of the supporting structure, $T_a$ (Section 12.8.2.1)

$$T_a = C_t h_n^x = (0.02)(46 \text{ ft})^{0.75} = 0.353 \text{ s}$$

Force amplification factor as a function of height in the structure, $H_f$

$$a_1 = \frac{1}{T_a} = \frac{1}{0.353 \text{ s}} = 2.83 > 2.5, \text{ use } a_1 = 2.5$$

$$a_2 = [1 - (0.4/T_a)^2] = [1 - (0.4/0.353 \text{ s})^2] = -0.28 < 0, \text{ use } a_2 = 0$$

$$H_f = 1 + a_1 \left(\frac{z}{h}\right) + a_2 \left(\frac{z}{h}\right)^{10} = 1 + 2.5 \left(\frac{28 \text{ ft}}{46 \text{ ft}}\right) + 0 \left(\frac{28 \text{ ft}}{46 \text{ ft}}\right)^{10} = 2.52$$

For supporting structures with $T_a \leq 0.4 \text{ s}$, $a_1 = 2.5$ and $a_2 = 0$.

Structure ductility reduction factor, $R_\mu$

$$R_\mu = (1.1 R/(I_E \Omega_0))^{1/2} = (1.1(5/((1)(2.5))))^{1/2} = 1.48 \geq 1.3$$
Applicable Requirements

- Component failure shall not cause failure of an essential architectural, mechanical, or electrical component (Section 13.2.4).

- Seismic attachments shall be bolted, welded, or otherwise positively fastened without considering the frictional resistance produced by the effects of gravity (Section 13.4).

- $F_p$ shall be applied at the component’s center of gravity and distributed relative to the component’s mass distribution (Section 13.3.1).

- Effects of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate (Section 13.3.2).

- Local elements of the structure, including connections, shall be designed and constructed for the component forces where they control the design of the elements or their connections (Section 13.4).
Applicable Requirements (Continued)

- Attachments to concrete or masonry shall be designed to resist the seismic load effects including overstrength; $\Omega_0$ shall be taken as $\Omega_{0p}$ (Section 13.4.2).

- The equipment support structures and platforms shall be designed for $F_p$. The seismic force-resisting system for the equipment support structures and platforms shall conform to one of the types indicated in Table 12.2-1 or Table 15.4-1 and abide by the system limitations noted in the tables (Section 13.6.4.6).
Prescribed Seismic Forces: Vessel Support and Attachments

- Vessel and legs weight, $W_{p,ves} = D_{ves} = 5,000$ lb
- Seismic design force, $F_p$

$$F_p = 0.4 S_{DS} I_p W_p \left[ \frac{H_f}{R_\mu} \right] \left[ \frac{C_{AR}}{R_{po}} \right] = 0.4(1.2)(1.0)(W_p) \left[ \frac{2.52}{1.48} \right] \left[ \frac{1.4}{1.5} \right] = 0.762 W_p \text{ (controlling equation)}$$

$$F_{p,max} = 1.6 S_{DS} I_p W_p = 1.6(1.2)(1.0)(W_p) = 1.92 W_p$$

$$F_{p,min} = 0.3 S_{DS} I_p W_p = 0.3(1.2)(1.0)(W_p) = 0.360 W_p$$

$$F_{p,ves} = 0.762 W_p = 0.762(5,000 \text{ lb}) = 3,808 \text{ lb} \quad \text{(controlling seismic design force)}$$
Prescribed Seismic Forces: Vessel Support and Attachments

- **Horizontal seismic load effect,** $E_h$
  
  $$Q_E = F_p = 3,808 \text{ lb}$$

  $$E_{h,ves} = \rho Q_E = (1.0)(3,808 \text{ lb}) = 3,808 \text{ lb}$$

- **Vertical seismic load effect,** $E_v$
  
  $$E_{v,ves} = 0.2S_{DS}D = (0.2)(1.2g)(5,000 \text{ lb}) = 1,200 \text{ lb}$$

- **Basic Load Combinations for Strength Design** to determine the design member and connection forces to be used in conjunction with seismic loads:
  
  1. $1.2D + E_v + E_h + L + 0.2S$  \(\text{(Load Combination 6)}\)
  2. $0.9D - E_v + E_h$  \(\text{(Load Combination 7)}\)

For nonstructural components, the terms $L$ and $S$ are typically zero.
Proportioning and Design: Vessel Support and Attachments

- Components to be designed
  - Legs supporting the vessel
  - Connection between the legs and vessel shell
  - Base plates and welds attaching them to legs
  - Bolts connecting base plates to supporting frame

Free body diagram for vessel support and attachments design
Proportioning and Design: Vessel Support and Attachments

- Vessel vertical load in each leg due to dead load, $P_{g,ves}$
  \[ P_{g,ves} = \frac{D_{ves}}{4\text{ legs}} = \frac{5,000\text{ lb}}{4\text{ legs}} = 1,250\text{ lb/leg} \]

- Vessel vertical load in each leg due to vertical seismic load effect, $P_{Ev,ves}$
  \[ P_{Ev,ves} = \frac{E_{v,ves}}{4\text{ legs}} = \frac{1,200\text{ lb}}{4\text{ legs}} = 300\text{ lb/leg} \]

- Vessel shear force in each leg due to the horizontal seismic load effect, $V_{ves}$
  \[ V_{ves} = \frac{E_{h,ves}}{4\text{ legs}} = \frac{3,808\text{ lb}}{4\text{ legs}} = 952\text{ lb/leg} \]

- Overturning moment at the bottom of leg base plates, height of 5.5 feet
  \[ M = (5.5\text{ ft})(E_{h,ves}) = (5.5\text{ ft})(3,808\text{ lb}) = 20,946\text{ lb–ft} \]
Proportioning and Design: Vessel Support and Attachments

- $F_p$ shall be applied independently in at least two orthogonal horizontal directions.
- For vertically cantilevered systems, the lateral force also shall be assumed to act in any horizontal direction.
- In this example, the layout of the vessel legs is symmetric, and there are two horizontal directions of interest, separated by 45 degrees.
Load Case 1 – Overturning moment about y-y axis

- Overturning moment is resisted by two legs along the x-x axis (one in tension and other in compression). The vessel rotates about the legs on the y-y axis.
- Maximum tension and compression loads in each leg, where the distance between Legs A and C is \( d = 6 \text{ ft} \):

\[
P_{Eh_{y-y}} = \frac{M}{d} = \frac{20,946 \text{ lb-ft}}{6 \text{ ft}} = 3,491 \text{ lb}
\]
Proportioning and Design: Vessel Support and Attachments

Load Case 2 – Overturning moment about x’-x’ axis

- Overturning moment is resisted by four legs (two in tension and two in compression). The vessel rotates about the legs on the x’-x’ axis.
- Maximum tension and compression loads in each leg, where the distance between Legs A and C is $d/\sqrt{2}=4.24$ ft:
  \[ P_{Eh_{x’-x’}} = \frac{M}{2(d/\sqrt{2})} = \frac{20,946 \text{ lb-ft}}{2(4.24 \text{ ft})} = 2,469 \text{ lb} \]

Load Case 1 governs the vessel leg design

\[ P_{Eh,ves} = P_{Eh_{y-y}} = 3,491 \text{ lb} \]
Proportioning and Design: Vessel Support and Attachments

- The design compression loads on the vessel legs is controlled by Load Combination 6:
  \[ 1.2D + E_v + E_h + L + 0.2S \]
  \[ C_u = 1.2(P_{g,ves}) + P_{Ev,ves} + P_{Eh,ves} = 1.2(1,250 \text{ lb}) + 300 \text{ lb} + 3,491 \text{ lb} = 5,291 \text{ lb} \]

- The design tension load on the vessel legs is controlled by Load Combination 7:
  \[ 0.9D - E_v + E_h \]
  \[ T_u = 0.9(P_{g,ves}) - P_{Ev,ves} + P_{Eh,ves} = 0.9(1,250 \text{ lb}) - 300 \text{ lb} - 3,491 \text{ lb} = -2,666 \text{ lb (tension)} \]

- The vessel legs shall be designed for the following shear force:
  \[ V_u = V_{ves} = 952 \text{ lb} \]
Proportioning and Design: Vessel Support and Attachments

Vessel Leg Design

- Section properties of the vessel leg: $A = 1.02 \text{ in.}^2$ and $Z = 0.713 \text{ in.}^3$

- Maximum axial compressive stress in the leg:
  
  $$f_a = \frac{C_u}{A} = \frac{5,291 \text{ lb}}{1.02 \text{ in.}^2} = 5,291 \text{ psi}$$

- Moment and bending stress in the leg, assuming pinned-fixed condition at connections:
  
  $$M_u = (V_u)(L_{leg}) = (952 \text{ lb})(18 \text{ in.}) = 17,138 \text{ lb-in.}$$

  $$f_b = \frac{M_u}{Z} = \frac{17,138 \text{ lb-in.}}{0.713 \text{ in.}^3} = 24,036 \text{ psi}$$

- Permissible compressive strength, and bending strength: $F_a = F_{bw} = 31,500 \text{ psi}$

- Combined loading:
  
  $$\left| \frac{f_a}{F_a} + \frac{f_b}{F_{bw}} \right| = \left| \frac{5,291 \text{ psi}}{31,500 \text{ psi}} + \frac{24,036 \text{ psi}}{31,500 \text{ psi}} \right| = 0.93 \leq 1.0 \rightarrow \text{OK}$$
Connections of the Vessel Leg

- The design of this connection involves checking the
  - Weld between the pipe leg and the base plate
  - Base plate
  - Bolts to the supporting frame
- Maximum compression and tension:
  \[ C_u = 5,291 \text{ lb} \text{ and } T_u = -2,666 \text{ lb} \text{ (tension)} \]
- Design shear in each leg:
  \[ V_u = 952 \text{ lb} \]
Connections of the Vessel Leg – Bolts

- Each vessel leg connection has two bolts, thus, the connection demand is divided by two.

- Maximum tension in each bolt:
  \[ T_{u,bolt} = \frac{T_u}{2 \text{ bolts}} = \frac{-2,666 \text{ lb}}{2 \text{ bolts}} = -1,333 \text{ lb/bolt} \]

- Maximum shear per bolt:
  \[ V_{u,bolt} = \frac{V_u}{2 \text{ bolts}} = \frac{952 \text{ lb}}{2 \text{ bolts}} = 476 \text{ lb/bolt} \]

- Available tensile and shear strengths of the of 5/8-inch-diameter ASTM A307 bolts:
  \[ \phi r_n = 10,400 \text{ lb (tension) and } \phi v r_n = 5,520 \text{ lb (shear)} \]

- Bolts are adequate, \[ \phi r_n > T_{u,bolt} \text{ and } \phi v r_n > V_{u,bolt} \] OK
Proportioning and Design: Vessel Support and Attachments

Connections of the Vessel Leg – Connection Plates

- Connection plates are 3/8 inch thick and 3 inches wide. The plastic section modulus is:
  \[ Z = \frac{bd^2}{4} = \frac{(3 \text{ in.})(0.375 \text{ in.})^2}{4} = 0.1055 \text{ in.}^3 \]

- Maximum moment in the plate based on the 1.5 in. edge distance to the bolt centerline:
  \[ M_{u,\text{plate}} = T_{u,\text{bolt}}(1.5 \text{ in.}) = (1,333 \text{ lb/bolt})(1.5 \text{ in.}) = 1,999 \text{ lb-in.} \]

- Bending stress in the plate:
  \[ f_b = \frac{M_u}{Z} = \frac{1,999 \text{ lb-in.}}{0.1055 \text{ in.}^3} = 18,958 \text{ psi} \]

- Bending stress capacity of the ASTM A36 plate:
  \[ F_b = \Phi F_y = 0.9(36,000 \text{ psi}) = 32,400 \text{ psi} \]

- Steel plate is adequate, \( F_b > f_b \) \( \rightarrow \text{OK} \)
Connections of the Vessel Leg – Connection Plates (Continued)

- ANSI/AISC 360 Equation 9-20 permits prying action to be neglected if plates meets minimum thickness requirement:

\[ t_{\text{min}} = \sqrt{\frac{4.44Tb'}{pF_u}}, \text{ where } p = 3 \text{ in. is the tributary length per pair of bolts.} \]

\[ b' = (b - db/2) = (1.5 \text{ in.} - 0.625 \text{ in.}/2) = 1.188 \text{ in.} \]

\[ t_{\text{min}} = \sqrt{\frac{4.44Tb'}{pF_u}} = \sqrt{\frac{(4.44)(1,333 \text{ lb/bolt})(1.188 \text{ in.})}{(3 \text{ in.})(58,000 \text{ psi})}} = 0.201 \text{ in.} \]

- \( t_{\text{min}} = 0.201 \text{ in.} \) is less than the 0.375-inch thickness provided for the connection plates. Thus, prying action need not be considered further.
Connections of the Vessel Leg – Welds

- The vessel legs have two welds at each end: the welds to the vessel body, and the welds to the upper connection plate.
- The outer diameter of the vessel leg is \( d = 2.38 \text{ in.} \). The weld properties are simplified by assuming a weld of unit thickness.

\[
Z_w = \frac{\pi d^2}{4} = \frac{\pi (2.38 \text{ in.})^2}{4} = 4.45 \text{ in.}^2
\]

\[
A = \pi d = \pi (2.38 \text{ in.}) = 7.48 \text{ in.}
\]

- Shear force in the weld of unit thickness:

\[
v = \frac{V_u}{A} = \frac{952 \text{ lb}}{7.48 \text{ in.}} = 127 \text{ lb/in.}
\]
Connections of the Vessel Leg – Welds (Continued)

- Tension force due to axial load in a weld of unit thickness:
  \[ T_a = \frac{T_u}{a} = \frac{2,666 \text{ lb}}{7.48 \text{ in.}} = 356 \text{ lb/in.} \]

- Tension force due to bending in a weld of unit thickness (at connection to the vessel):
  \[ T_b = \frac{M}{Z_w} = \frac{17,138 \text{ lb-in.}}{4.45 \text{ in.}^2} = 3,852 \text{ lb/in.} \]

- For a unit length, a 3/16-inch fillet weld has a capacity of:
  \[ \phi R_n = 1.392 DL = 1.392 \times 3 = 4.18 \text{ kip/in.} \]

- Thus, the 3/16-inch fillet weld is adequate.
Prescribed Seismic Forces: Supporting Frame

- Supporting frame weight, \( W_{p,\text{sup}} = D_{\text{sup}} = 1,000 \text{ lb} \)
- Seismic design force, \( F_p \)

\[
F_p = 0.4 S_{DS} I_p W_p \left[ \frac{H_f}{R_\mu} \right] \left[ \frac{C_{AR}}{R_{po}} \right] = 0.4(1.2)(1.0)(W_p) \left[ \frac{2.52}{1.48} \right] \left[ \frac{1.4}{1.5} \right] = 0.762W_p \quad \text{(controlling equation)}
\]

\[
F_{p,\text{max}} = 1.6 S_{DS} I_p W_p = 1.6(1.2)(1.0)(W_p) = 1.92W_p
\]

\[
F_{p,\text{min}} = 0.3 S_{DS} I_p W_p = 0.3(1.2)(1.0)(W_p) = 0.360W_p
\]

\[
F_{p,\text{sup}} = 0.762W_p = 0.762(1,000 \text{ lb}) = 762 \text{ lb} \quad \text{(controlling seismic design force)}
\]
Prescribed Seismic Forces: Supporting Frame

- Horizontal seismic load effect, $E_h$
  
  $Q_E = F_p = 762$ lb

  $E_{h,\text{sup}} = \rho Q_E = (1.0)(762 \text{ lb}) = 762$ lb

- Vertical seismic load effect, $E_v$

  $E_{v,\text{sup}} = 0.2 S_D S D = (0.2)(1.2g)(1,000 \text{ lb}) = 240$ lb

- Basic Load Combinations for Strength Design to determine the design member and connection forces to be used in conjunction with seismic loads:

  1. $1.2D + E_v + E_h + L + 0.2S$  \hspace{1cm} (Load Combination 6)

  2. $0.9D - E_v + E_h$  \hspace{1cm} (Load Combination 7)

For nonstructural components, the terms $L$ and $S$ are typically zero.
Proportioning and Design: Supporting Frame

- Components to be designed
  - Beams supporting the vessel legs
  - Braces
  - Columns supporting the platform and vessel
  - Base plates and anchor bolts

Elevated vessel supporting frame

Free body diagram for supporting frame system
Support Frame Beams

- Beam vertical load at midspan due to dead load, $P_{g,\text{sup}}$
  
  $$P_{g,\text{sup}} = \frac{D_{\text{ves}}+D_{\text{sup}}}{4 \text{ supports}} = \frac{5,000 \text{ lb}+1,000 \text{ lb}}{4 \text{ supports}} = 1,500 \text{ lb/support}$$

- Beam vertical load at midspan due to vertical seismic load effect, $P_{E_{v,sup}}$
  
  $$P_{E_{v,sup}} = \frac{E_{v,\text{ves}}+E_{v,\text{sup}}}{4 \text{ supports}} = \frac{1,200 \text{ lb}+240 \text{ lb}}{4 \text{ supports}} = 360 \text{ lb/support}$$

- Beam lateral load of the combined vessel and supporting frames, $V_{\text{sup}}$
  
  $$V_{\text{sup}} = \frac{E_{h,\text{ves}}+E_{h,\text{sup}}}{4 \text{ supports}} = \frac{3,808 \text{ lb}+762 \text{ lb}}{4 \text{ supports}} = 1,143 \text{ lb/support}$$

- Beam vertical load at midspan due to horizontal seismic load effect (Case 1), $P_{E_{h,\text{beam}}}$
  
  $$P_{E_{h,\text{beam}}} = P_{E_{h,\text{ves}}} = 3,491 \text{ lb/support}$$
Proportioning and Design: Supporting Frame

Support Frame Beams (Continued)

- The HSS6x2x1/4 frame beams have the following geometric and material properties:
  \[ Z_{x-x} = 5.84 \text{ in.}^3, Z_{y-y} = 2.61 \text{ in.}^3, F_b = \Phi F_y = 0.9(46,000 \text{ psi}) = 41,400 \text{ psi} \]

- Moment and bending stress about the x-x axis in the beams, where \( L = 6 \text{ ft} \)
  \[ M_{x-x} = \frac{C_u L}{4} = \frac{(5,651 \text{ lb})(6 \text{ ft})(12 \text{ in./ft})}{4} = 101,718 \text{ lb-in.} \quad f_{bx} = \frac{M_{x-x}}{Z_{x-x}} = \frac{101,718 \text{ lb-in.}}{5.84 \text{ in.}^3} = 17,417 \text{ psi} \]

- Moment and bending stress about the y-y axis in the beams, where \( L = 6 \text{ ft} \)
  \[ M_{y-y} = \frac{V_u L}{4} = \frac{(1,143 \text{ lb})(6 \text{ ft})(12 \text{ in./ft})}{4} = 20,565 \text{ lb-in.} \quad f_{by} = \frac{M_{x-x}}{Z_{y-y}} = \frac{20,565 \text{ lb-in.}}{2.61 \text{ in.}^3} = 7,879 \text{ psi} \]

- Interaction of bending demand in the strong and weak axis
  \[
  \left| \frac{f_{bx}}{F_b} + \frac{f_{by}}{F_b} \right| = \left| \frac{17,417 \text{ psi}}{41,400 \text{ psi}} + \frac{7,879 \text{ psi}}{41,400 \text{ psi}} \right| = 0.611 \leq 1.0 \quad \text{OK}
  \]
Proportioning and Design: Supporting Frame

Support Frame Braces

- Maximum brace force occurs when loads are resisted by two braces.
- Horizontal force:
  \[ V_{brace} = \frac{E_{h,ves} + E_{h,sup}}{2 \text{ braces}} = \frac{3,808 \text{ lb} + 762 \text{ lb}}{2 \text{ braces}} = 2,285 \text{ lb/brace} \]
- Length of the brace: \[ L = \sqrt{(5 \text{ ft})^2 + (6 \text{ ft})^2} = 7.81 \text{ ft} \]
- Tension force in the brace:
  \[ T_u = \left(\frac{7.81 \text{ ft}}{6 \text{ ft}}\right)(2,285 \text{ lb}) = 2,974 \text{ lb} \quad \text{(tension)} \]
- Nominal tensile capacity of 5/8-inch-diameter ASTM A307 threaded rods: \( \phi r_n = 10,400 \text{ lb} \)
- Threaded rods are adequate, \( \phi r_n > T_u, 10,400 \text{ lb} > 2,974 \text{ lb} \quad \rightarrow \text{OK} \)
Support Frame Columns

- HSS2x2x1/4 columns support vertical loads from vessel and frame. Column length, \( L = 5 \text{ ft} \).
- Overturning moment:
  \[
  M = (10.5 \text{ ft})(E_{h,ves}) + (5.0 \text{ ft})(E_{h,\text{sup}}) = (10.5 \text{ ft})(3,808 \text{ lb}) + (5.0 \text{ ft})(762 \text{ lb}) = 43,796 \text{ lb-ft}
  \]
- Maximum T-C loads in the columns due to overturning, where \( d = (6 \text{ ft})\sqrt{2} = 8.48 \text{ ft} \)
  \[
  P_{Eh,\text{col}} = \frac{M}{d} = \frac{43,796 \text{ lb-ft}}{8.48 \text{ ft}} = 5,161 \text{ lb}
  \]
- The vertical gravity load in each leg is \( P_{g,\text{sup}} = 1,500 \text{ lb/support} \) and \( P_{Ev,\text{sup}} = 360 \text{ lb/support} \).
- The compression load on the columns is: \( C_u = 1.2(P_{g,\text{sup}}) + P_{Ev,\text{sup}} + P_{Eh,\text{col}} = 7,321 \text{ lb} \)
- The tension load on the columns is: \( C_u = 0.9(P_{g,\text{sup}}) - P_{Ev,\text{sup}} - P_{Eh,\text{col}} = -4,171 \text{ lb} \)
- The capacity of the HSS2x2x1/4 column is 38,300 lb. Therefore, it is adequate.
Proportioning and Design: Supporting Frame

Anchor Bolts

- The combination that results in net tension on the anchors will govern. Thus, the Load Combination 7 including overstrength is applied: $0.9D - E_v + E_{mh}$ where $E_{mh} = \Omega_{0p}Q_E$

- Vertical design tension force:
  
  $$T_u = 0.9\left(P_{g,\text{sup}}\right) - P_{E_v,\text{sup}} - \Omega_{0p}P_{E_h,\text{col}}$$

  $$T_u = 0.9(1,500 \text{ lb}) - 360 \text{ lb} - (2.0)(5,161 \text{ lb}) = -9,333 \text{ lb}$$

- Horizontal design shear force:
  
  $$V_u = \Omega_{0p}V_{E_h,\text{col}} = (2.0)(1,143 \text{ lb}) = 2,285 \text{ lb}$$

- When comparing the support frame column forces to the connection to the floor slab forces, the tension force increases by 124%, and the shear force increases by 100%.
Questions?
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