AASHTO
LRFD Guide Specifications
for
Seismic Design of Highway Bridges

Roy A. Imbsen
Presentation Topics

♦ Background-AASHTO LRFD Guide Specifications
♦ Excerpts selected from the Guide Specifications
♦ AASHTO T-3 Committee recent activities supporting adoption as a Guide Specification
♦ Current status
♦ Planned activities post-adoption
♦ Conclusions
AASHTO T-3 Working Group that defined the objectives and directed the project

- Rick Land, CA (Past chair)
- Harry Capers, NJ (Past Co-chair)
- Richard Pratt, AK (Current chair)
- Kevin Thompson, CA (Current Co-chair)
- Ralph Anderson, IL
- Jugesh Kapur, WA
- Ed Wasserman, TN
- Paul Liles, GA
Project Phases

- 2002 AASHTO T-3 Committee Meeting
- 2003 MCEER/FHWA
  - Task F3-4 Road Map
  - Task F3-5 Suggested Approach
- 2004 NCHRP 20-07/Task 193 *AASHTO Guide Specifications for LRFD Seismic Bridge Design*
- AASHTO T-3 Committee and Volunteer States
  - 2006 Trial Designs
  - 2007 Technical Review
- 2007 AASHTO Adoption as a Guide Specification with the continuous support and guidance of the T-3 Committee
Overall T-3 Project Objectives

♦ Assist T-3 Committee in developing a LRFD Seismic Design Specification using available specifications and current research findings
♦ Develop a specification that is user friendly and implemental into production design
♦ Complete six tasks specifically defined by the AASHTO T-3 Committee, which were based on the NCHRP 12-49 review comments
## Stakeholders Table

<table>
<thead>
<tr>
<th>IAI Team (as needed)</th>
<th>T-3 Working Group</th>
<th>Technical Review Panel (to be invited)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roy Imbsen, IAI</td>
<td>Rick Land, CA (Past chair)</td>
<td>George Lee, MCEER, Chair</td>
</tr>
<tr>
<td>Roger Borcherdt, USGS</td>
<td>Harry Capers, NJ (Past Co-chair)</td>
<td>Rick Land, T-3 Chair</td>
</tr>
<tr>
<td>Po Lam, EMI</td>
<td>Richard Pratt, AK (Current chair)</td>
<td>Geoff Martin, MCEER</td>
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<tr>
<td>E. V. Leyendecker, USGS</td>
<td>Kevin Thompson, CA (Current Co-chair)</td>
<td>Joe Penzien, HSRC, EQ V-team</td>
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<tr>
<td>Lee Marsh, Berger/Abam</td>
<td>Ralph Anderson, IL</td>
<td>John Kulicki, HSRC</td>
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<tr>
<td>Randy Cannon, formerly SCDOT</td>
<td>Jugesh Kapur, WA</td>
<td>Les Youd, BYU</td>
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<td></td>
<td>Ed Wasserman, TN</td>
<td>Joe Wang, Parsons, EQ V-team</td>
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<tr>
<td></td>
<td>Paul Liles, GA</td>
<td>Lucero Mesa, SCDOT V-team</td>
</tr>
</tbody>
</table>
THE MEMBERS OF THE TECHNICAL REVIEW TEAM

- MARK MAHAN, CA DOT (TEAM LEADER)
- ROY A. IMBSEN, IMBSEN CONSULTING
- ELMER MARX, AK DOT & PF
- JAY QUIOGUE, CA DOT
- CHRIS UNANWA, CA DOT
- FADEL ALAMEDDINE, CA DOT
- CHYUAN-SHEN LEE, WA STATE DOT
- STEPHANIE BRANDENBERGER, MT DOT
- DANIEL TOBIAS, IL DOT
- DERRELL MANCEAUX, FHWA
- LEE MARSH, BERGER/ABAM
THE STATES WHO PERFORMED THE TRIAL DESIGNS

♦ ALASKA
♦ ARKANSAS
♦ CALIFORNIA
♦ ILLINOIS
♦ INDIANA
♦ MISSOURI
♦ MONTANA
♦ NEVADA
♦ OREGON
♦ TENNESSEE
♦ WASHINGTON STATE
Support

♦ MCEER/FHWA “Seismic Vulnerability of the Highway System” Task F3-4 AASHTO T-3 Support
♦ NCHRP 20-07/Task 193 Updating “Recommended LRFD Guidelines for Seismic Design of Highway Bridges”
♦ AASHTO T-3 Committee
Background-NCHRP 20-07 Task 6 Report (1.1)

- Review Reference Documents
- Finalize Seismic Hazard Level
- Expand the Extent of the No-Analysis Zone
- Select the Most Appropriate Design Procedure for Steel Bridges
- Recommend Liquefaction Design Procedure
Proposed

AASHTO Guide Specifications for LRFD Seismic Bridge Design

Subcommittee for Seismic Effects on Bridges T-3

Prepared by:
Roy A. Imbsen
Imbsen Consulting

March 2007
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Background Task 2 - Seismic Hazard Level (1.1)

Recommended approach to addressing the seismic hazard:

♦ Design against the Effects Ground Shaking Hazard
♦ Selection of a Return Period for Design less than 2500 Years
♦ Inclusion of the USGS 2002 Update of the National Seismic Hazard Maps
♦ Effects of Near Field and Fault Rupture to be addressed in a following Task
♦ Displacement Based Approach with both Design Spectral Acceleration and corresponding Displacement Spectra provided
♦ Hazard Map under the control of AASHTO with each State having the option to Modify or Update their own State Hazard using the most recent Seismological Studies
Background Task 2-Seismic Hazard (1.1)

Seismic Hazard Practice can be best illustrated in looking at the following sources:

♦ NEHRP 1997 Seismic Hazard Practice
♦ Caltrans Seismic Hazard Practice
♦ NYCDOT and NYSDOT Seismic Hazard Practice
♦ NCHRP 12-49 Seismic Hazard Practice
♦ SCDOT Seismic Hazard Practice
♦ Site-Specific Hazard Analyses Conducted for Critical Bridges
Background Seismic Hazard for Normal Bridges (1.1)

♦ Selection of a lower return period for Design is made such that Collapse Prevention is not compromised when considering large historical earthquakes.

♦ A reduction can be achieved by taking advantage of sources of conservatism not explicitly taken into account in current design procedures.

♦ The sources of conservatism are becoming more obvious based on recent findings from both observations of earthquake damage and experimental data.
## Background Task 2-Sources of Conservatism (1.1)

<table>
<thead>
<tr>
<th>Source of Conservatism</th>
<th>Safety Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Computational vs. Experimental Displacement Capacity of Components</td>
<td>1.3</td>
</tr>
<tr>
<td>Effective Damping</td>
<td>1.2 to 1.5</td>
</tr>
<tr>
<td>Dynamic Effect (i.e., strain rate effect)</td>
<td>1.2</td>
</tr>
<tr>
<td>Pushover Techniques Governed by First Plastic Hinge to Reach Ultimate Capacity</td>
<td>1.2 to 1.5</td>
</tr>
<tr>
<td>Out of Phase Displacement at Hinge Seat</td>
<td>Addressed in Task 3</td>
</tr>
</tbody>
</table>
Idealized Load – Deflection Curve

Considered in Design
Design Approaches

-Force-

- Division 1A and Current LRFD Specification
- Complete w/ service load requirements
- Elastic demand forces w/ applied prescribed ductility “R”
- Ductile response is assumed to be adequate w/o verification

-Displacement-

- New 2007 Guide Specification
- Complete w/ service load requirements
- Displacements demands w/ displacement capacity checks for deformability
- Ductile response is assured with limitations prescribed for each SDC
Background Seismic Hazard Normal Bridges (1.1)

Two distinctly different aspects of the design process need to be provided:

♦ An appropriate method to design adequate seat width(s) considering out of phase motion.
♦ An appropriate method to design the ductile substructure components without undue conservatism

These two aspects are embedded with different levels of conservatism that need to be calibrated against the single level of hazard considered in the design process.
Background Task 3
Expand the No-Analysis Zone (1.1)

♦ At a minimum, maintain the number of bridges under the “Seismic Demand Analysis” by comparing Proposed Guidelines to AASHTO Division I-A.

♦ Develop implicit procedures that can be used to reduce the number of bridges where “Seismic Capacity Analysis” needs to be performed. This objective is accomplished by identifying a threshold where an implicit procedure can be used (Drift Criteria, Column Shear Criteria).

♦ Identify threshold where “Capacity Design” shall be used. This objective is achieved in conjunction with the “Seismic Capacity Analysis” requirements.
Guidelines-General
Seismic Load Path and Affected Components
Guidelines
Performance Criteria

♦ Type 1 – Design a ductile substructure with an essentially elastic superstructure.
♦ Type 2 – Design an essentially elastic substructure with a ductile superstructure.
♦ Type 3 – Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure.
Guidelines
Performance Criteria

♦ For Type 3 choice, the designer shall assess the overstrength capacity for the fusing interface including shear keys and bearings, then design for an essentially elastic superstructure and substructure.

♦ The minimum overstrength lateral design force shall be calculated using an acceleration of 0.4 g or the elastic seismic force whichever is smaller.

♦ If isolation devices are used, the superstructure shall be designed as essentially elastic.
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LRFD Flow Chart
Fig 1.3-1A
LRFD Flow Chart
Fig 1.3-1B
LRFD Flow Chart
(Fig 1.3-5A)
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♦ Appendix A – Rocking Foundation Rocking Analysis
Applicability (3.1)

♦ Design and Construction of New Bridges
♦ Bridges having Superstructures Consisting of:
  – Slab
  – Beam
  – Girder
  – Box Girder
♦ Spans less than 500 feet
Performance Criteria (3.2)

♦ One design level for life safety
♦ Seismic hazard level for 7% probability of exceedance in 75 years (i.e., 1000 year return period)
♦ Low probability of collapse
♦ May have significant damage and disruption to service
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  3.3 Earthquake Resisting Systems
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Earthquake Resisting Systems-ERS (3.3)

♦ Required for SDC C and D
♦ Must be identifiable within the bridge system
♦ Shall provide a reliable and uninterrupted load path
♦ Shall have energy dissipation and/or restraint to control seismically induced displacements
♦ Composed of acceptable Earthquake Resisting Elements (ERE)
ERS (3.3)

Permissible Earthquake Resisting Systems (ERS)

1. Plastic hinges in inspectable locations or elastic design of columns.
   - Abutment resistance not required as part of ERS
   - Knock-off backwalls permissible

2. Isolation bearings accommodate full displacement
   - Abutment not required as part of ERS

3. Plastic hinges in inspectable locations or elastic design of columns
   - Abutment not required in ERS, breakaway shear keys permissible

4. Plastic hinges in inspectable locations or elastic design of columns
   - Isolation bearings with or without energy dissipaters to limit overall displacements

5. Abutment required to resist the design earthquake elastically
   - Longitudinal passive soil pressure shall be less than 0.70 of the value obtained using the procedure given in Article 5.2.3

6. Multiple simply-supported spans with adequate support lengths
   - Plastic hinges in inspectable locations or elastic design of columns
ERS (3.3)

Permissible Earthquake Resisting Elements that Require Owner’s Approval

- Wall piers or pile foundations that are not strong enough to force plastic hinging into the wall, and are not designed for the Design Earthquake elastic forces
- More than the outer line of piles in group systems allowed to plunge or uplift under seismic loadings
- Plumb piles that are not capacity-protected (e.g., integral abutment piles or pile-supported seat abutments that are not fused transversely)
- In-ground hinging in shafts or piles
- Batter pile systems in which the geotechnical capacities and/or in-ground hinging define the plastic mechanisms

Ensure Limited Ductility Response in Piles according to Article 4.7.1
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Seismic Hazard (3.4)

♦ 7% Probability of Exceedence in 75 Years

♦ AASHTO-USGS Technical Assistance Agreement to:
  – Provide paper maps
  – Develop ground motion software

♦ Hazard maps for 50 States and Puerto Rico
  – Conterminous 48 States-USGS 2002 maps
  – Hawaii-USGS 1998 maps
  – Puerto Rico-USGS 2003 maps
  – Alaska-USGS 2006 maps

♦ Maps for Spectral Accelerations Site Class B
  – Short period (0.2 sec.)
  – Long period (1.0 sec.)
  – Peak (PGA 0.0 sec.)
Seismic Hazard 2-Point Method for Design Spectrum Construction (3.4)

Design Spectrum, Figure 3.4.1-1

Western Bridge Engineers’ Seminar
September 24-26, 2007
Trial Design MO-2 (3.1)

Elevation of Intermediate Pier

Western Bridge Engineers’ Seminar
September 24-26, 2007
Trial Design MO-2 (3.1)

ELEVATION - MISSOURI SITE

14" φ CIP CONC PILE W/STEEL CASING, TYP

13'-0"

1'=40'
Figure 3.4.1-2a Peak Horizontal Ground Acceleration for the Conterminous United States (Western) With 7 Percent Probability of Exceedance in 75 Years (Approx. 1000 Year Return Period).
AASHTO/USGS Maps (3.4.1)
LRFD – Horizontal Spectral Response Acceleration (3.4.1)

AASHTO/USGS Maps
Region 3
0.2 second period
Longitude 89.817° West
Latitude 36.000° North
Acceleration=1.89g
### Site Effects $F_v$ (3.4.2)

**Table 3.4.2.3-2: Values of $F_v$ as a Function of Site Class and Mapped 1 Second Period Spectral Acceleration**

<table>
<thead>
<tr>
<th>Site Class</th>
<th>$S_1 \leq 0.1 \ g$</th>
<th>$S_1 = 0.2 \ g$</th>
<th>$S_1 = 0.3 \ g$</th>
<th>$S_1 = 0.4 \ g$</th>
<th>$S_1 \geq 0.5 \ g$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.7</td>
<td>1.6</td>
<td>1.5</td>
<td>1.4</td>
<td>1.3</td>
</tr>
<tr>
<td>D</td>
<td>2.4</td>
<td>2.0</td>
<td>1.8</td>
<td>1.6</td>
<td>1.5</td>
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<tr>
<td>E</td>
<td>3.5</td>
<td>3.2</td>
<td>2.8</td>
<td>2.4</td>
<td>2.4</td>
</tr>
<tr>
<td>F</td>
<td>$a$</td>
<td>$a$</td>
<td>$a$</td>
<td>$a$</td>
<td>$a$</td>
</tr>
</tbody>
</table>

*Table notes: Use straight line interpolation for intermediate values of $S_1$, where $S_1$ is the spectral acceleration at 1.0 second obtained from the ground motion maps.*

*Site-specific geotechnical investigation and dynamic site response analyses shall be performed (Article 3.4.3).*
Seismic Hazard 2-Point Method for Design Spectrum Construction (3.4)
Seismic Design Parameters for 2007 AASHTO Seismic Design Guidelines

This program allows the user to obtain seismic design parameters for sites in the 50 states of the United States, Puerto Rico and the U.S. Virgin Islands. Ground motion maps are also included in PDF format.

Click on Okay to begin calculation.

Correct application of the data obtained from the use of this program and/or maps is the responsibility of the user. This software is not a substitute for technical knowledge of seismic design and/or analysis.
Western Bridge Engineers’ Seminar
September 24-26, 2007
Western Bridge Engineers’ Seminar
September 24-26, 2007
Map Spectrum for Sa vs. T
5% Damping
Continuous 48 States
Latitude = 36.0000 deg  Longitude = -89.817000 deg
Site Class B

<table>
<thead>
<tr>
<th>Period, sec</th>
<th>Sa, g</th>
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<tbody>
<tr>
<td>0.00</td>
<td>1.0381</td>
</tr>
<tr>
<td>0.06</td>
<td>1.8811</td>
</tr>
<tr>
<td>0.20</td>
<td>1.8811</td>
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<tr>
<td>0.30</td>
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<tr>
<td>0.40</td>
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<td>0.60</td>
<td>0.9449</td>
</tr>
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<td>0.80</td>
<td>0.7087</td>
</tr>
<tr>
<td>1.00</td>
<td>0.5670</td>
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<tr>
<td>1.20</td>
<td>0.4725</td>
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<td>1.40</td>
<td>0.4050</td>
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<td>2.00</td>
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<td>2.20</td>
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<tr>
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<tr>
<td>2.80</td>
<td>0.2025</td>
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<td>3.00</td>
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<td>3.20</td>
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<tr>
<td>3.60</td>
<td>0.1575</td>
</tr>
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<td>3.80</td>
<td>0.1492</td>
</tr>
<tr>
<td>4.00</td>
<td>0.1417</td>
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</table>
### 2007 AASHTO Ground Motion Maps for 7% Probability of Exceedance in 75 Years

5% of Critical Damping, Site Class B

- **Conterminous United States** - Peak Ground Acceleration
- **Conterminous United States** - 0.2 sec period Spectral Response Acceleration
- **Conterminous United States** - 1.0 sec period Spectral Response Acceleration

#### Region 1 (California/Western Nevada)
- Peak Ground Acceleration
- 0.2 sec period Spectral Response Acceleration
- 1.0 sec period Spectral Response Acceleration

#### Region 2 (Salt Lake City Area)
- Peak Ground Acceleration
- 0.2 sec period Spectral Response Acceleration
- 1.0 sec period Spectral Response Acceleration

#### Region 3 (New Madrid Area)
- Peak Ground Acceleration
- 0.2 sec period Spectral Response Acceleration
- 1.0 sec period Spectral Response Acceleration

#### Region 4 (Charleston, SC Area)
- Peak Ground Acceleration
- 0.2 and 1.0 sec period Spectral Response Acceleration

- **Alaska** - Peak Ground Acceleration
- **Alaska** - 0.2 sec period Spectral Response Acceleration
- **Alaska** - 1.0 sec period Spectral Response Acceleration

- **Hawaii** - Peak Ground Acceleration
- **Hawaii** - 0.2 and 1.0 sec period Spectral Response Acceleration

- **Puerto Rico, Culebra, Vieques, St. Thomas, St. John, and St. Croix** - Peak Ground Acceleration
- **Puerto Rico, Culebra, Vieques, St. Thomas, St. John, and St. Croix** - 0.2 and 1.0 sec period Spectral Acceleration
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Four Seismic Design Categories (SDC) A, B, C and D encompassing requirements for:

- Seismic Demand Analysis requirement
- Seismic Capacity Analysis requirement
- Capacity Design requirement
- Level of seismic detailing requirement including four tiers corresponding to SDC A, B, C and D
- Earthquake Resistant System
### SDC (3.5)

**Table 3.5-1 Partitions for Seismic Design Categories A, B, C and D.**

<table>
<thead>
<tr>
<th>Value of $S_{D1} = F_v S_1$</th>
<th>SDC</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_{D1} &lt; 0.15$</td>
<td>A</td>
</tr>
<tr>
<td>$0.15 \leq S_{D1} &lt; 0.30$</td>
<td>B</td>
</tr>
<tr>
<td>$0.30 \leq S_{D1} &lt; 0.50$</td>
<td>C</td>
</tr>
<tr>
<td>$0.50 \leq S_{D1}$</td>
<td>D</td>
</tr>
</tbody>
</table>
SDC A (3.5)

1. SDC A

   a. No identification of ERS according to Article 3.3

   b. No Demand Analysis

   c. No Implicit Capacity Check Needed

   d. No Capacity Design Required

   e. Minimum Detailing requirements for support length and superstructure/substructure connection design force
2. SDC B

a. No Identification of ERS according to Article 3.3

b. Demand Analysis

c. Implicit Capacity Check Required (displacement, $P-\Delta$, support length)

d. No Capacity Design Required except for column shear requirement

e. SDC B Level of Detailing
3. SDC C

a. Identification of ERS

b. Demand Analysis

c. Implicit Capacity Check Required (displacement, $P-\Delta$, support length)

d. Capacity Design Required including column shear requirement

e. SDC C Level of Detailing
4. SDC D

a. Identification of ERS

b. Demand Analysis

c. Displacement Capacity Required using Pushover Analysis (check $P-\Delta$ and support length)

d. Capacity Design Required including column shear requirement

e. SDC D Level of Detailing
SDC Core Flowchart (3.5)
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Balanced Stiffness Recommendation (4.1)
Seismic Analysis Using SAP2000 Bridge Modeler

Missouri Design Example
3-Span P/S I-girder bridge
Balanced Frame SDC D (4.1.2)

- Any Two Bents Within a Frame or Any Two Columns Within a Bent

Constant Width Frames:
\[ \frac{k_i^e}{k_j^e} \geq 0.5 \quad (4.1.2-1) \]

Variable Width Frames:
\[ \frac{k_i^e m_j}{k_j^e m_i} \geq 0.5 \quad (4.1.2-2) \]
Balanced Bent (4.1.2)

- Adjacent Bents Within a Frame or Adjacent Columns Within a Bent

Constant Width Frames:
\[ \frac{k_i^e}{k_j^e} \geq 0.75 \quad (4.1.2-3) \]

Variable Width Frames:
\[ \frac{k_i^e m_j}{k_j^e m_i} \geq 0.75 \quad (4.1.2-4) \]
## Analysis Procedure (4.2)

<table>
<thead>
<tr>
<th>Seismic Design Category</th>
<th>Regular Bridges with 2 through 6 Spans</th>
<th>Not Regular Bridges with 2 or more Spans</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Not required</td>
<td>Not required</td>
</tr>
<tr>
<td>B, C, or D</td>
<td>Use Procedure 1 or 2</td>
<td>Use Procedure 2</td>
</tr>
</tbody>
</table>
Displacement Demands (4.3)

- Horizontal ground motions for SDC B, C, & D determined independently along two axes and combined
- Displacement modification for other than 5% damped bridges having energy dissipation at abutments
- Displacement magnification for short period short period structures
Combination of Seismic Displacement Demands (4.4)

♦ LOAD CASE 1: 100% Longitudinal Displacement Demands (absolute value), Combined with 30% Transverse Displacement Demands (absolute value)

♦ LOAD CASE 2: 100% Transverse Displacement Demands (absolute value), Combined with 30% Longitudinal Displacement Demands (absolute value)
Design for SDC B, C, & D (4.7)

- Conventional – Full ductility structures with a plastic mechanism having $4.0 < u_D < 6.0$ for a bridge in SDC D
- Limited ductility – For structures with a Plastic mechanism readily accessible for inspection having $u_D < 4.0$ for a bridge in SDC B or C
- Limited Ductility – For structures having a plastic mechanism working in concert with a protective system. The plastic hinge may or may not form. This strategy is intended for SDC C or D
Displacement Capacity for SDC B and C (4.8.1)

For SDC B:

\[ \Delta_C^L = 0.12H_o \left( -1.27 \ln(x) - 0.32 \right) \geq 0.12H_o \quad (4.8.1-1) \]

For SDC C:

\[ \Delta_C^L = 0.12H_o \left( -2.32 \ln(x) - 1.22 \right) \geq 0.12H_o \quad (4.8.1-2) \]

in which:

\[ x = \frac{\Lambda B_o}{H_o} \quad (4.8.1-3) \]
Inelastic Quasi-Static Pushover analysis (IQPA) is required to determine realistic displacement capacities as it reaches its limit states.

IQPA is an incremental linear analysis which captures the overall nonlinear behavior of the structure and its elements through each limit state.

The IQPA model includes the redistribution of forces as each limit state is reached.

Foundation effects may also be included in the model.
Member Ductility Requirement for SDC D (4.9)

For single column bents:

\[ \mu_D \leq 5 \]  \hspace{1cm} (4.9-1)

For multiple column bents:

\[ \mu_D \leq 6 \]  \hspace{1cm} (4.9-2)

For pier walls in the weak direction:

\[ \mu_D \leq 5 \]  \hspace{1cm} (4.9-3)

For pier walls in the strong direction:

\[ \mu_D \leq 1 \]  \hspace{1cm} (4.9-4)
Member Ductility Requirement for SDC D (4.9)

\[ \mu_D = 1 + \frac{\Delta_{pd}}{\Delta_{yi}} \] (4.9-5)

Where:
\[ \Delta_{pd} = \text{plastic displacement demand (in.)} \]
\[ \Delta_{yi} = \text{idealized yield displacement corresponding to the idealized yield curvature, } \phi_{yi}, \text{ shown in figure 8.5-1 (in.)} \]

Pile shafts should be treated similar to columns.
Capacity Design Requirement for SDC C & D

Capacity protection is required for all members that are not participating as part of the energy dissipating system.

Capacity protected members include:

- Superstructures
- Joints and cap beams
- Spread footings
- Pile caps
- Foundations
Over-strength Capacity Design Concepts for SDC C & D Trans. (4.11)
Minimum Support Length Requirements (4.12)

The calculation for a hinge seat width involves four components:

- Minimum edge distance
- Other movement attributed to prestress shortening, creep, shrinkage, and thermal expansion or contraction
- Skew effect
- Relative hinge displacement
Minimum Support Length (4.12)
SDC A, B, C & D

*Expansion Joint or End of Bridge Deck
Minimum Support Length (4.12)
SDC A, B, C

\[ N = (8 + 0.02L + 0.08H)(1 + 0.000125S^2) \]  
(4.12.2-1)

<table>
<thead>
<tr>
<th>SDC</th>
<th>Effective peak ground acceleration, ( A_s )</th>
<th>Percent N</th>
</tr>
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<tr>
<td>A</td>
<td>&lt; 0.05</td>
<td>≥ 75</td>
</tr>
<tr>
<td>A</td>
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<tr>
<td>B</td>
<td>All applicable</td>
<td>150</td>
</tr>
<tr>
<td>C</td>
<td>All applicable</td>
<td>150</td>
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</table>
LRFD - Relative Seismic Displacement vs. Period Ratio For SDC D (4.12)

- Deq for a target ductility of 2 shown as Curve 1
- Deq for a target ductility of 4 shown as Curve 2
- Caltrans SDC shown as Curve 3
- Relative hinge displacement based on (Trocholak is et. Al. 1997) shown as Curve 4

\[ N = \left(4 + 1.65 \Delta_{eq}\right) \left(1 + 0.00025 S^2\right) \geq 24 \quad (4.12.3-1) \]
Table of Contents

♦ 1. Introduction
♦ 2. Symbols and Definitions
♦ 3. General Requirements
♦ 4. Analysis and Design Requirements
♦ 5. Analytical Models and Procedures
♦ 6. Foundation and Abutment Design Requirements
♦ 7. Structural Steel Components
♦ 8. Reinforced Concrete Components
♦ Appendix A – Rocking Foundation Rocking Analysis
Ductility Demand on a Column or Pier is a Function of

♦ Earthquake characteristics, including duration, frequency content and near-field (or pulse) effects.
♦ Design force level
♦ Periods of vibration of the bridge
♦ Shape of the inelastic hysteresis loop of the columns, and hence effective hysteretic damping
♦ Elastic damping coefficient
♦ Contribution of foundation and soil conditions to structural flexibility
♦ Spread of plasticity (plastic hinge length) in the column
Plastic Moment Capacity SDC B, C & D (8.5)

- Moment-Curvature Analyses $M - \phi$
- Expected Material Properties
- Axial Dead Load Forces with Overturning
- $M - \phi$ Curve Idealized as Elastic Perfectly Plastic
- Elastic Portion of the Curve Pass through the point of marking the first reinforcing bar yield
- Plastic moment capacity determined from equal areas of idealized and actual
Figure 8.5-1 Moment-Curvature Model
Force Demands on Capacity Protected Members

\[ M_{po} = \lambda_{mo} M_p \]  \hspace{1cm} (8.5-1)

where:

- \( M_p \) = idealized plastic moment capacity of reinforced concrete member based upon expected material properties (kip-ft)
- \( M_{po} \) = overstrength plastic moment capacity (kip-ft)
- \( \lambda_{mo} \) = overstrength magnifier
  - 1.2 for ASTM A 706 reinforcement
  - 1.4 for ASTM A 615 Grade 60 reinforcement
Shear Demand & Capacity (8.6.1)

♦ SDC B $V_u$ is the lesser of:
  – Force obtained from linear elastic seismic analysis
  – Force, $V_{po}$, corresponding to plastic hinging with overstrength

♦ SDC C and D, $V_\mu$ is the shear demand force, with the overstrength moment $M_{po}$ and corresponding plastic shear
Shear strength capacity within the plastic hinge is based on nominal motion strength properties

\[ \phi_s V_n \geq V_u \]  

in which

\[ V_n = V_e + V_g \]  

\( \phi_s = 0.85 \) for shear in reinforced concrete

\( V_n = \) nominal shear capacity of member (kip)

\( V_c = \) concrete contribution to shear capacity

\( V_s = \) reinforcing steel contribution to shear capacity
Concrete Shear Capacity SDC B, C & D (8.6.2)

\[ V_c = \nu_c A_e \]  \hspace{1cm} (8.6.2-1)

\[ A_e = 0.8 A_g \]  \hspace{1cm} (8.6.2-2)

If \( P_c \) is compressive then

\[ \nu_c = 0.032 \alpha' \left\{ 1 + \frac{P_u}{2A_g} \right\} \sqrt{f_c'} \leq \begin{cases} 0.11 \sqrt{f_c'} \\ 0.047 \alpha \sqrt{f_c'} \end{cases} \]  \hspace{1cm} (8.6.2-3)

Otherwise (i.e., not compression)

\[ \nu_c = 0 \]  \hspace{1cm} (8.6.2-4)
Concrete Shear Capacity SDC B, C & D (8.6.2)

For circular columns in compression with spiral or hoop reinforcing:

\[ 0.3 \leq \alpha' \frac{f_s}{0.15} + 3.67 - \mu_o \leq 3 \]  \hspace{2cm} (8.6.2-5)

\[ f_s = e_s f_{yh} \leq 0.35 \]  \hspace{2cm} (8.6.2-6)

\[ e_s = \frac{4A_{sp}}{sD} \]  \hspace{2cm} (8.6.2-7)
Concrete Shear Capacity SDC B, C & D (8.6.2)

For rectangular columns in compression with ties:

\[ 0.3 \leq \alpha' \frac{f_w}{0.15} + 3.67 - \mu_D \leq 3 \]  \hspace{1cm} (8.6.2-8)

\[ f_w = 2e_w f_{yh} \leq 0.35 \]  \hspace{1cm} (8.6.2-9)

\[ e_w = \frac{A_v}{bs} \]  \hspace{1cm} (8.6.2-10)
Column Shear Requirement (8.10) SDC D

Fig. 4 Col. shear force vs. Displ. ductility ratio

Shear force in terms of $\frac{Ag}{V}$ kips

Displ. ductility ratio

- LRFD-LL
- SDC-LL
- LRFD-UL
- SDC-UL
- Pr et al. (1994)
Integral Joint Shear Requirement (8.13) SDC C, and D
Non-Integral Joint Shear Requirement

(8.13) SDC C, and D
Presentation Topics

♦ Background-AASHTO LRFD Guide Specifications
♦ Excerpts selected from the Guide Specifications
♦ AASHTO T-3 Committee recent activities supporting adoption as a Guide Specification
♦ Current status
♦ Planned activities post-adoption
♦ Conclusions
AASHTO Website

This page will serve as the posting site for questions and answers concerning the LRFD Guidelines for the Seismic Design of Highway Bridges.

The information contained on this website deals with NCHRP Project 20-07, Task 193, National Cooperative Highway Research Program.
Presentation Topics

♦ Background-AASHTO LRFD Guide Specifications
♦ Excerpts selected from the Guide Specifications
♦ AASHTO T-3 Committee recent activities supporting adoption as a Guide Specification
♦ Current status
♦ Planned activities post-adoption
♦ Conclusions
Current Status

♦ Completed in accordance with the AASHTO T-3 Committee Recommendations
♦ Reviewed by a Technical Group and modified to meet their state requirements
♦ Formatted to AASHTO specifications
♦ Scheduled five one-day FHWA introduction and overview course
♦ Reviewer comments and recommendations were tabulated, reviewed and implemented or placed on a priority list (“parking lot”) for future consideration
## Outline for FHWA One-Day Overview of AASHTO-2007 LRFD Guide Specifications

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<th>Duration</th>
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<td>LRFD Guide Spec.-Reinforced Concrete Components</td>
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<td>Wrap-up and Summary</td>
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Scheduled One-Day Seminars

♦ Montana DOT…………….9/20/07
♦ Washington DOT……….10/26/07
♦ Oregon DOT……………11/14/07
♦ Tennessee DOT…………1/10/08
♦ Idaho DOT……………..1/31/08
Planned Activities-Post Adoption

♦ Development of an FHWA funded training manual and course geared toward practicing engineer
♦ Review of the geotechnical issues addressed in the comments and recommendations
♦ Address tabulated comments and recommendations placed in a “parking lot” as funding becomes available
Conclusions

♦ Adopted as a Guide Specifications
♦ Developed a specification that is user friendly and implemental into production design
♦ Logical progression from the current AASHTO force-based seismic design criteria to a displacement-based criteria
♦ Technical reviewers were focused on making adjustments to bridge the gap between the seismic design approaches to ease the implementation of the displacement-based approach
♦ Computer software is available to assist the designer, Computers & Structures Inc. (CSI) is enhancing SAP 2000 to be used with the new 2007 Guide Specifications
♦ Lets do it !!!!!!!!