AASHTO LRFD Guide Specifications for Seismic Design of Highway Bridges

Roy A. Imbsen

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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

IAI Team (as needed)	T-3 Working Group	Technical Review Panel (to be invited)
Roy Imbsen, IAI Roger Borcherdt, USGS Po Lam, EMI E. V. Leyendecker, USGS Lee Marsh, Berger/Abam Randy Cannon, formerly SCDOT	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	George Lee, MCEER, Chair Rick Land, T-3 Chair Geoff Martin, MCEER Joe Penzien, HSRC, EQ V-team John Kulicki, HSRC Les Youd, BYU Joe Wang, Parsons, EQ V-team Lucero Mesa, SCDOT V-team

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
- Letter Reports for Tasks 1-5 (Ref. NCHRP 20-07/Task 193 Task 6 Report for Updating "Recommended LRFD Guidelines for Seismic Design of Highway Bridges" Imbsen & Associates, Inc., of TRC)

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

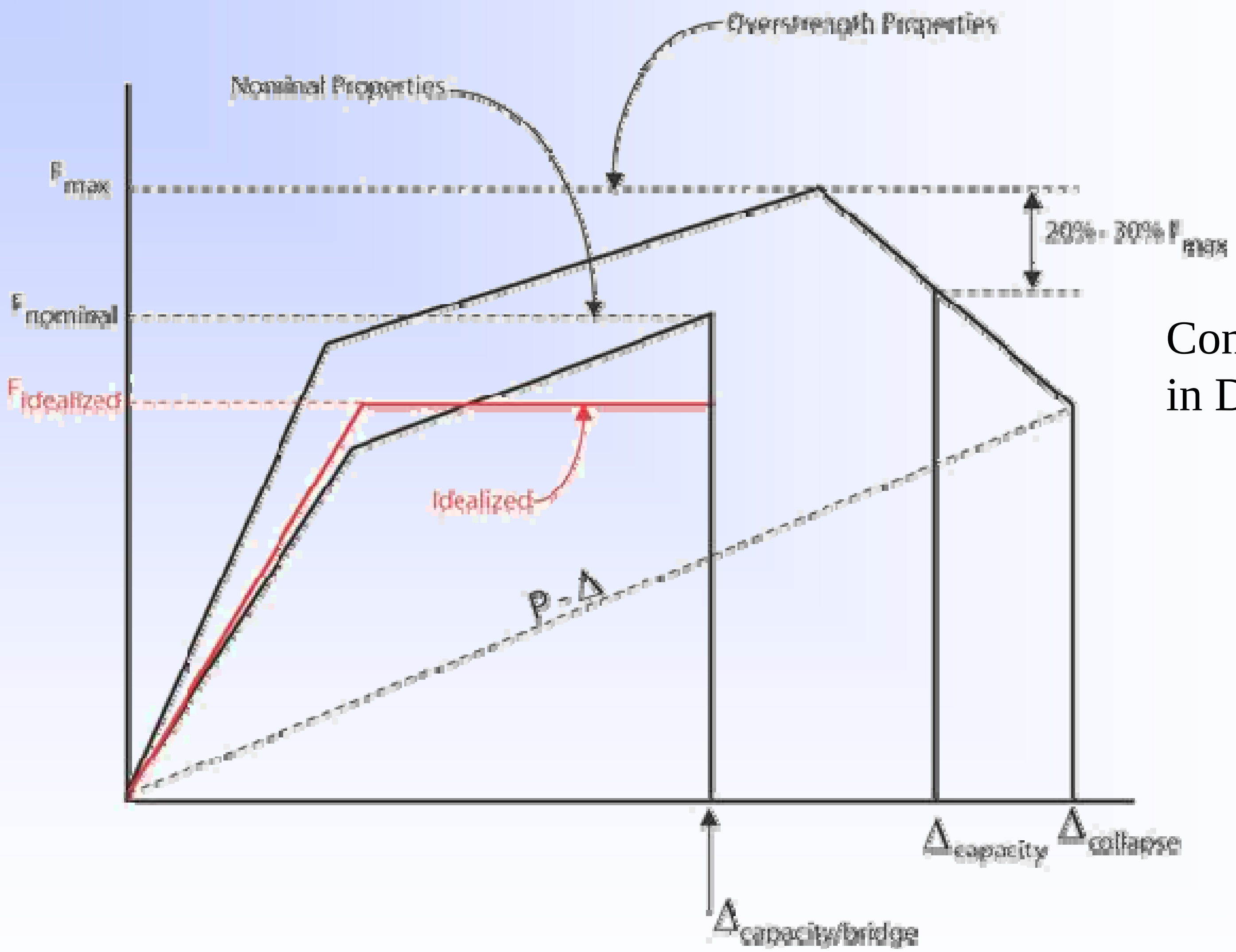
Background Seismic Hazard for Normal Bridges (1.1)

- 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)

Source of Conservatism	Safety Factor
Computational vs. Experimental Displacement Capacity of Components	1.3
Effective Damping	1.2 to 1.5
Dynamic Effect (i.e., strain rate effect)	1.2
Pushover Techniques Governed by First Plastic Hinge to Reach Ultimate Capacity	1.2 to 1.5
Out of Phase Displacement at Hinge Seat	Addressed in Task 3

Idealized Load – Deflection Curve



Considered in Design

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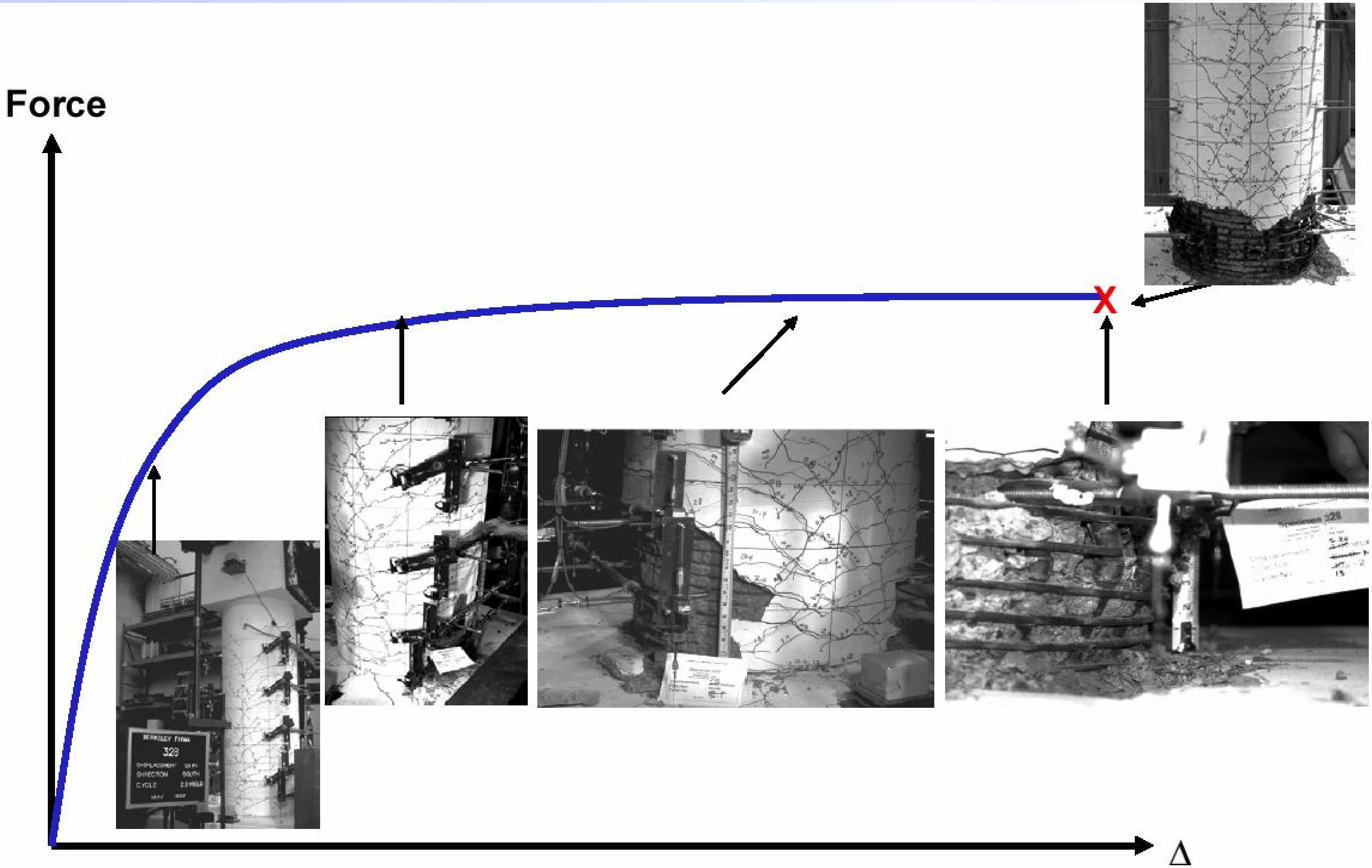
Force
Δ

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 reduce the number of bridges where “Seismic Capacity Analysis” needs to be performed, This objective is accomplished by $P-\Delta$ identifying a threshold where an implicit procedures 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.

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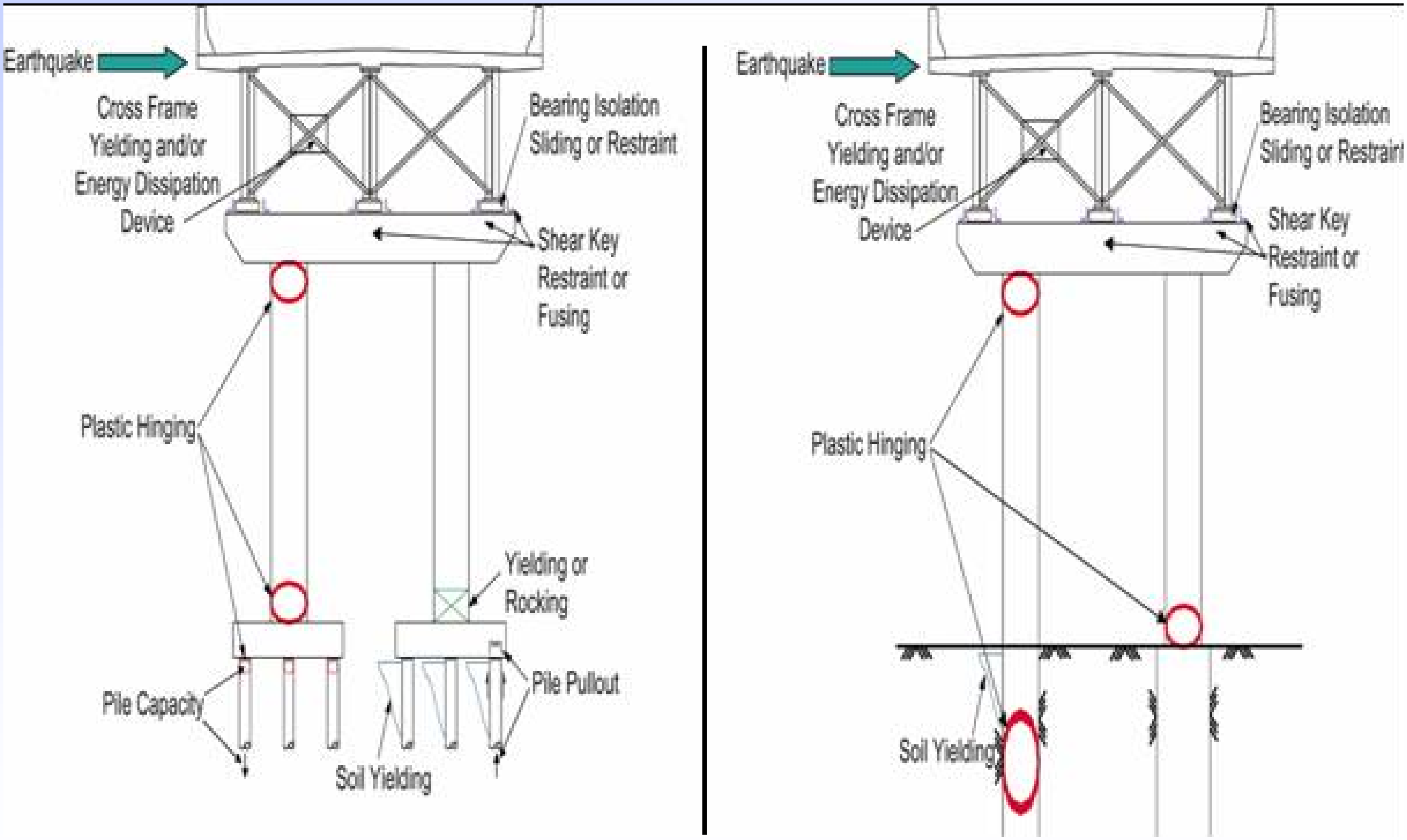
Guidelines-General
Seismic Load Path and Affected Components



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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.

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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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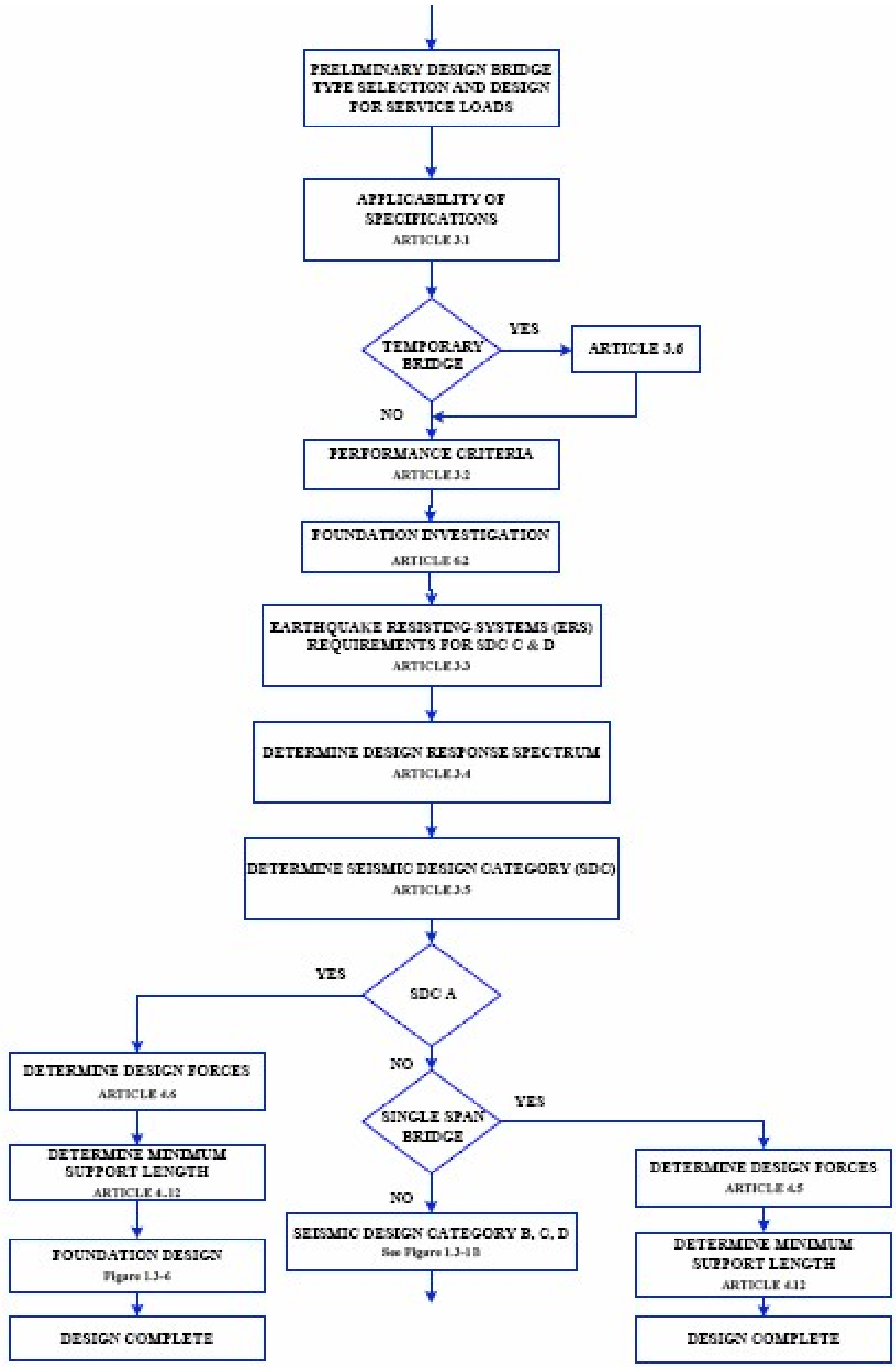
LRFD Flow Chart Fig 1.3-1A



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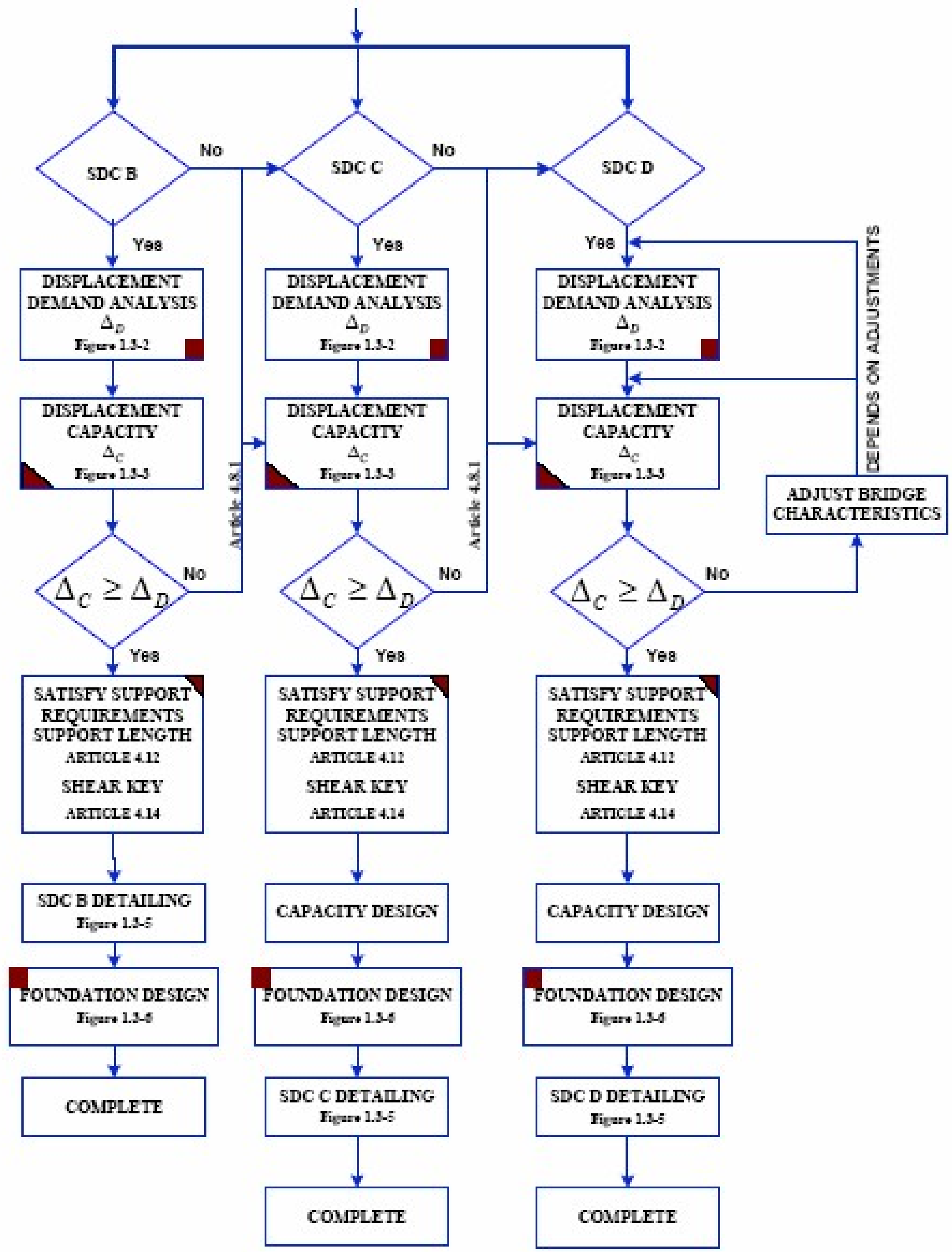
LRFD Flow Chart Fig 1.3-1B

(Continued From Figure 1.3-1A)



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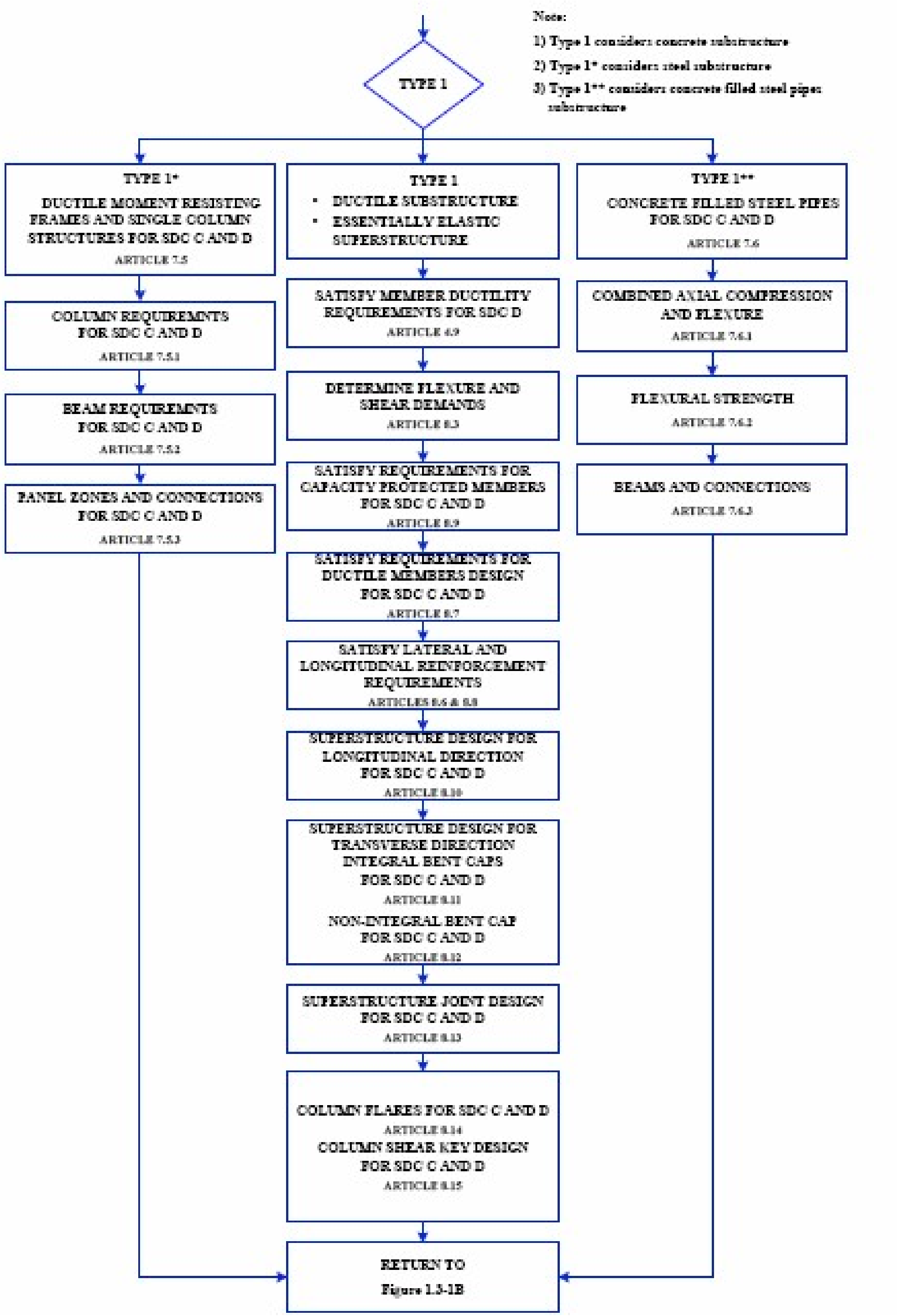
LRFD Flow Chart (Fig 1.3-5A)



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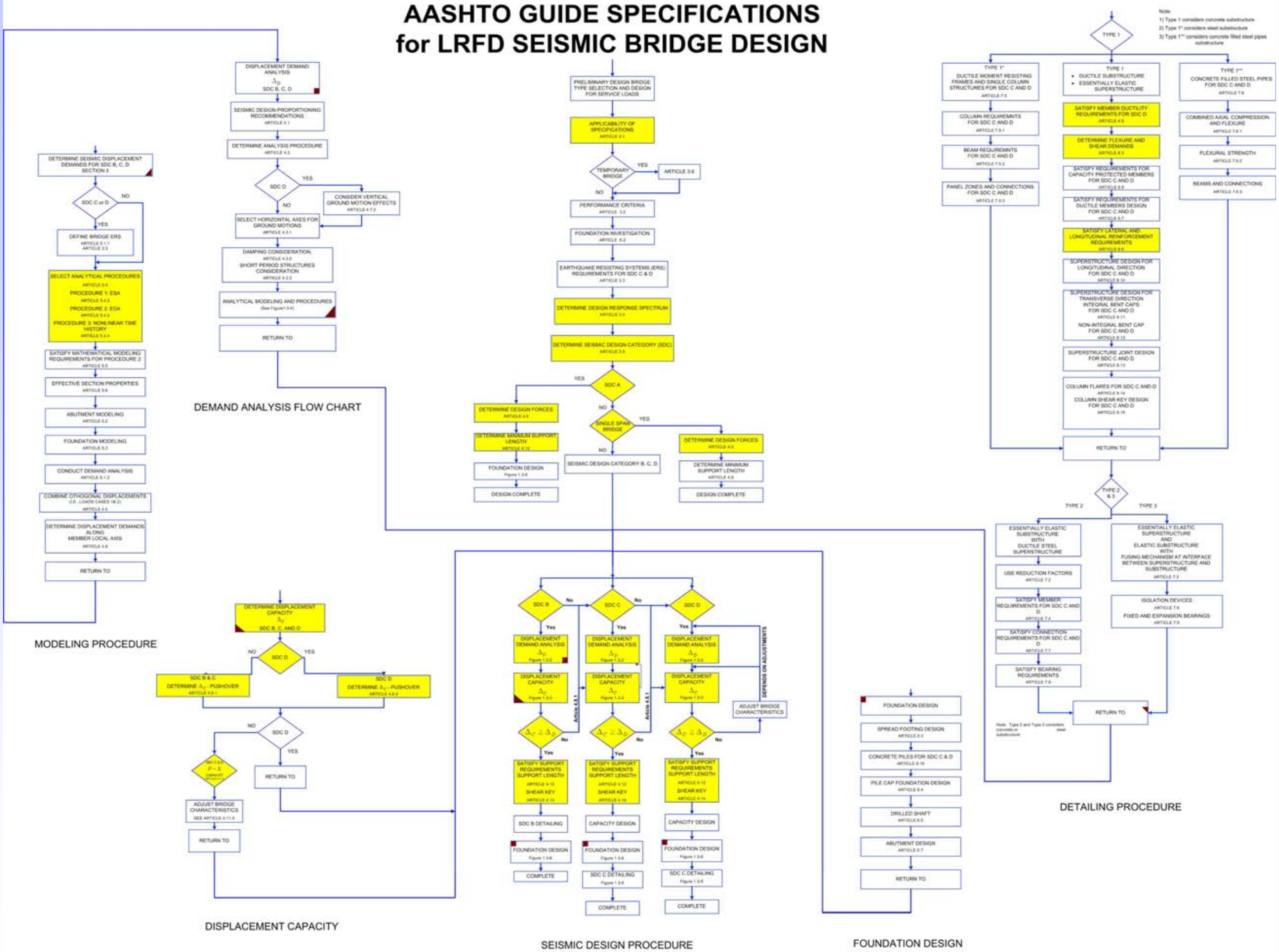
AASHTO GUIDE SPECIFICATIONS for LRFD SEISMIC BRIDGE DESIGN



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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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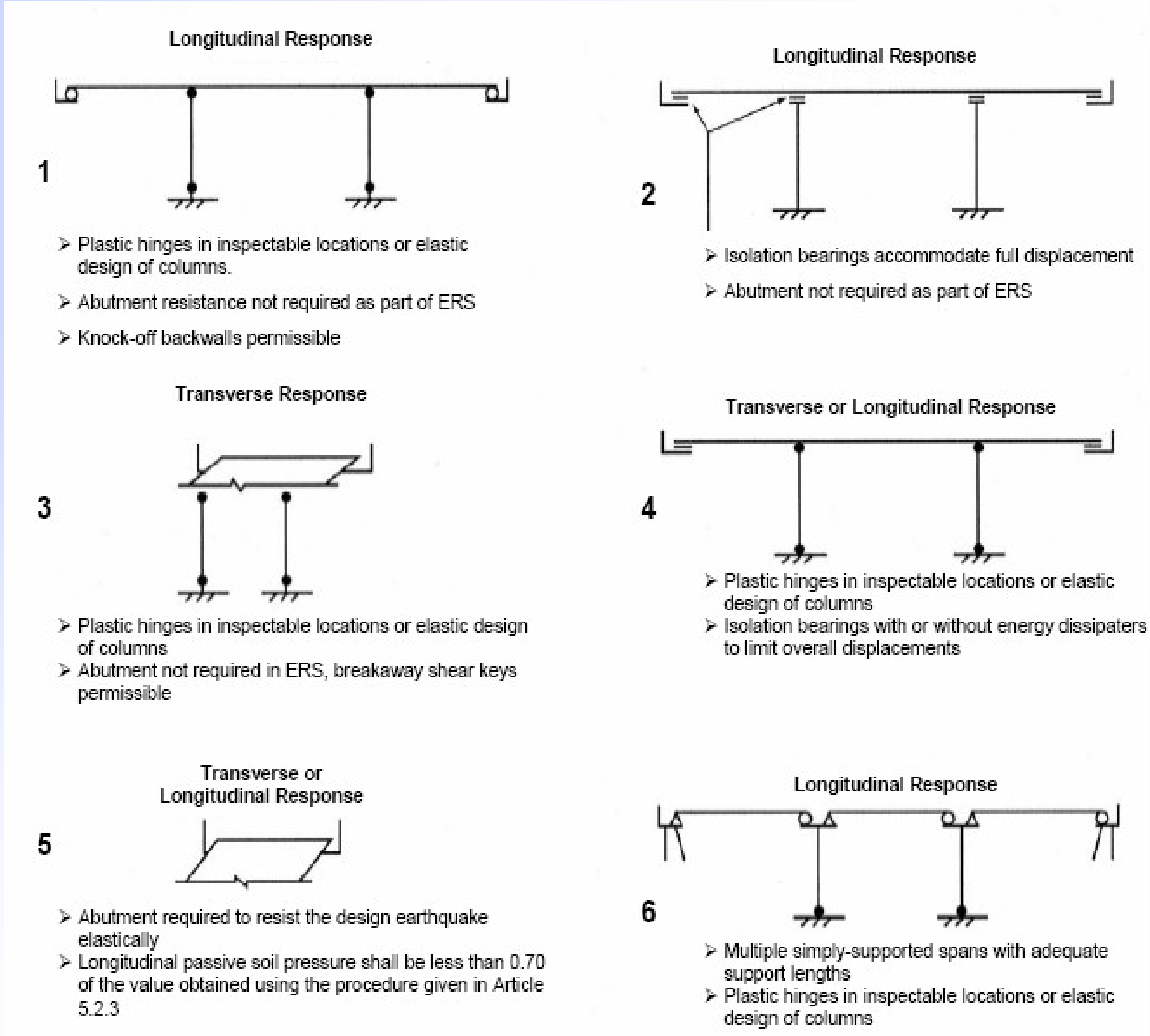
Earthquake Resisting Systems-ERS (3.3)

- 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)

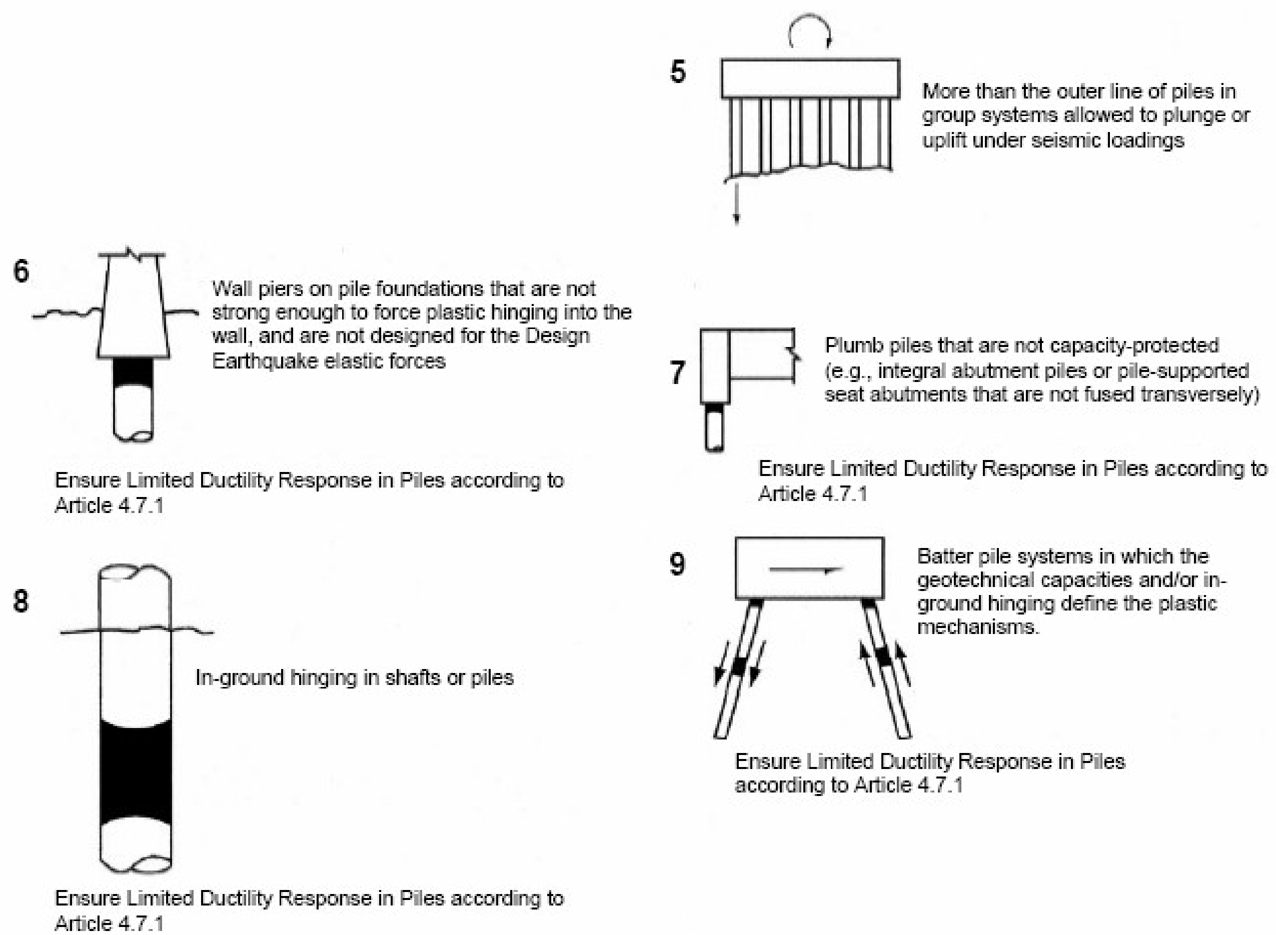
ERS (3.3)

Permissible Earthquake Resisting Systems (ERS)



ERS (3.3)

Permissible Earthquake Resisting Elements that Require Owner's Approval



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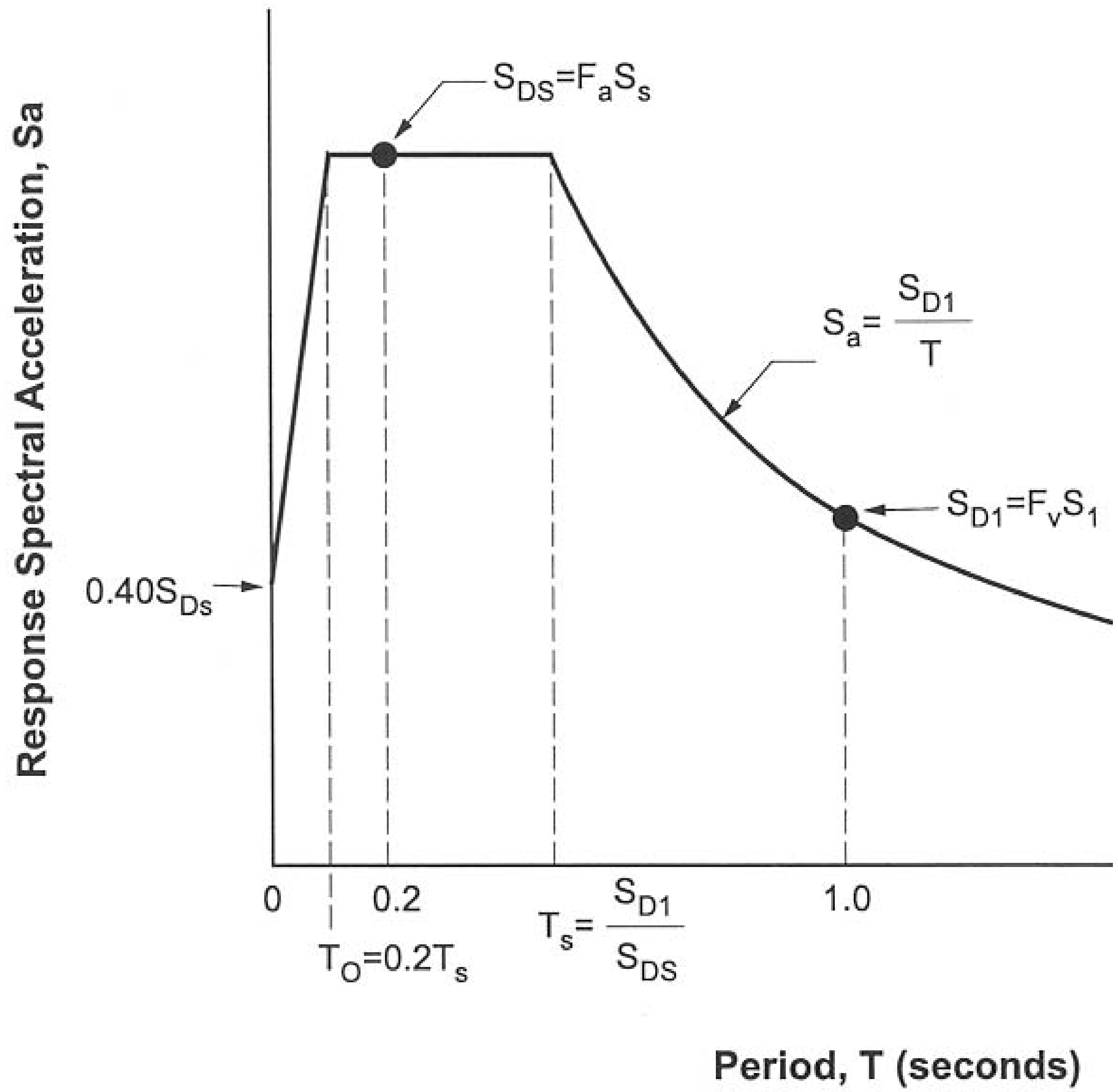
Seismic Hazard (3.4)

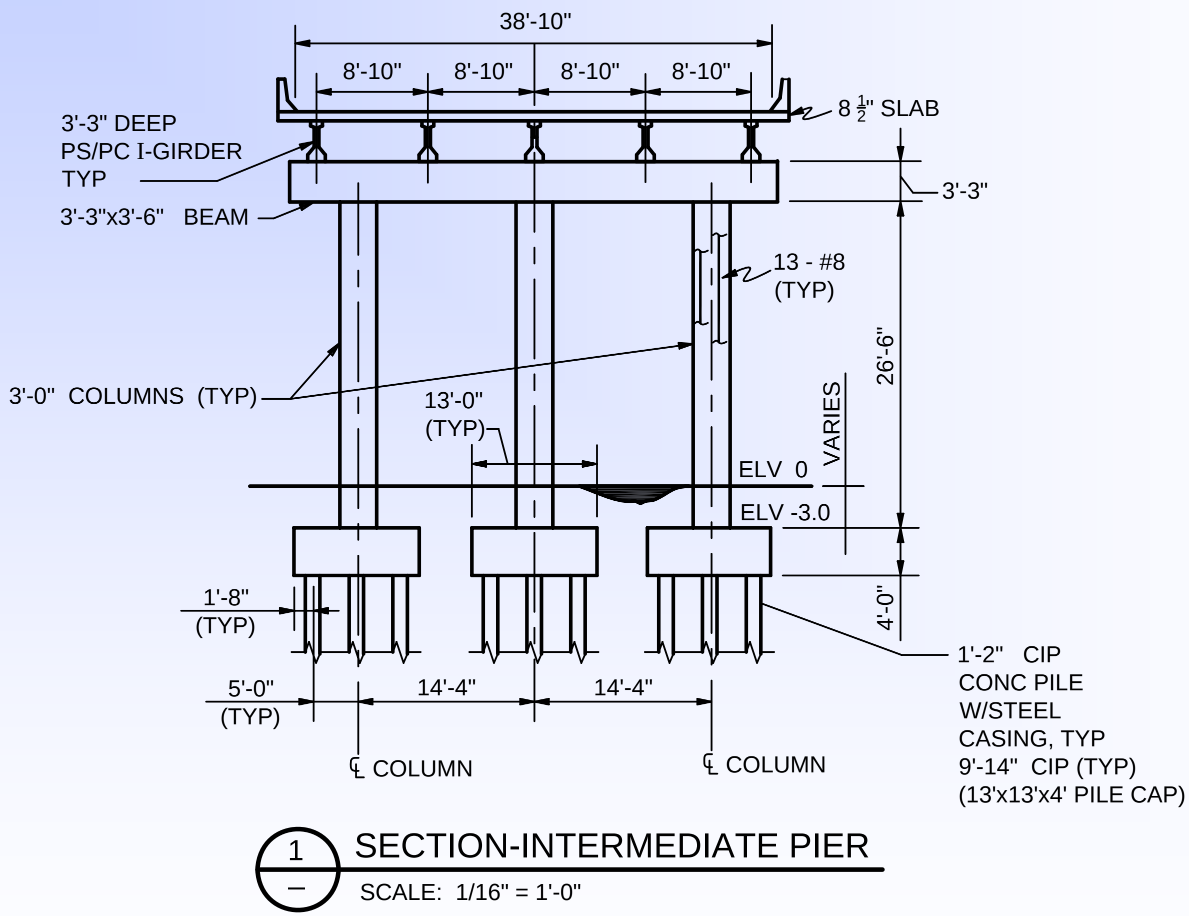
- 7% Probability of Exceedence in 75 Years
- AASHTO-USGS Technical Assistance Agreement to:
  - Provide paper maps
  - Develop ground motion software
- Hazard maps for 50 States and Puerto Rico
  - Conterminous 48 States-USGS 2002 maps
  - Hawaii-USGS 1998 maps
  - Puerto Rico-USGS 2003 maps
  - Alaska-USGS 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

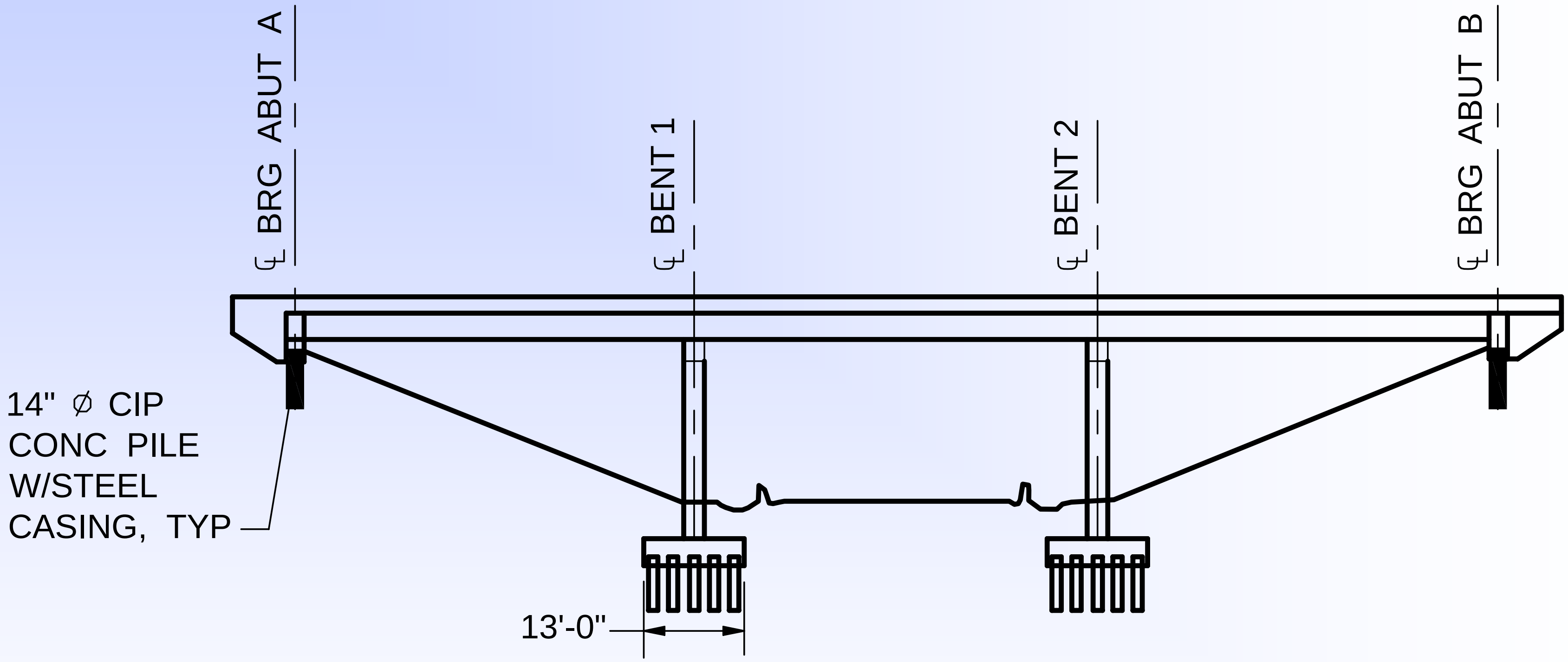


Trial Design MO-2 (3.1)



Elevation of Intermediate Pier

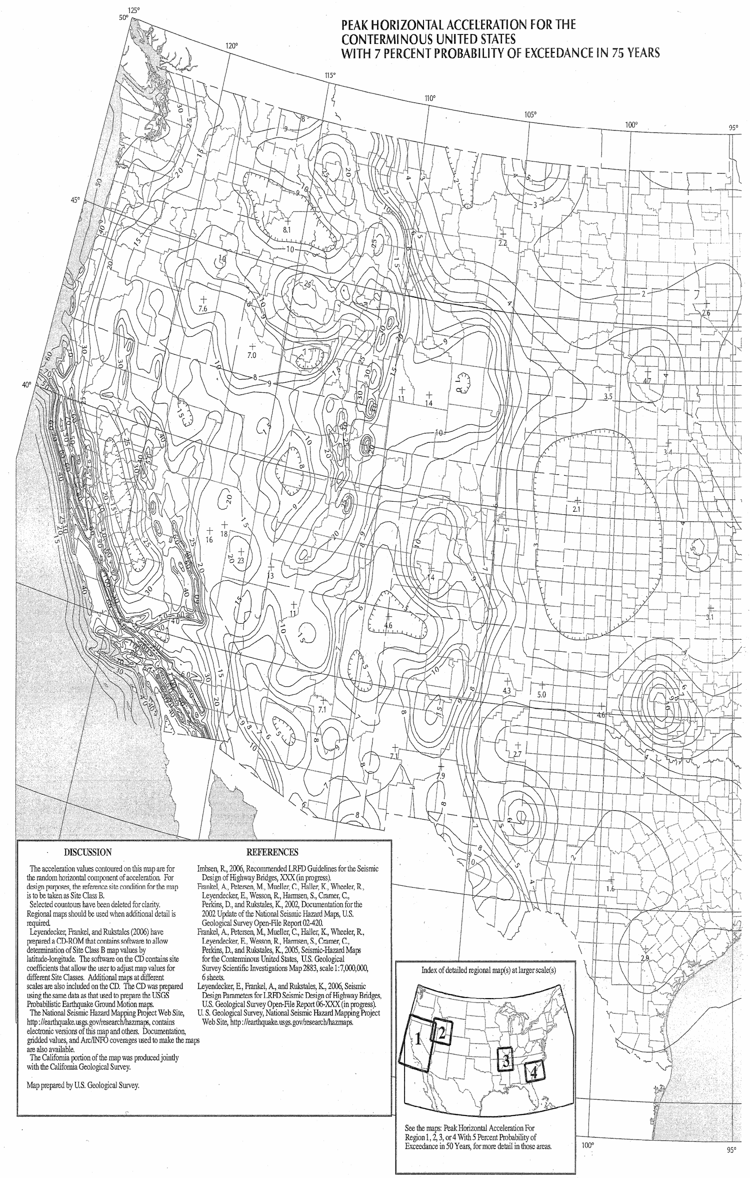
Trial Design MO-2 (3.1)



ELEVATION - MISSOURI SITE
1'=40'

AASHTO/ USGS Maps (3.4.1)

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).



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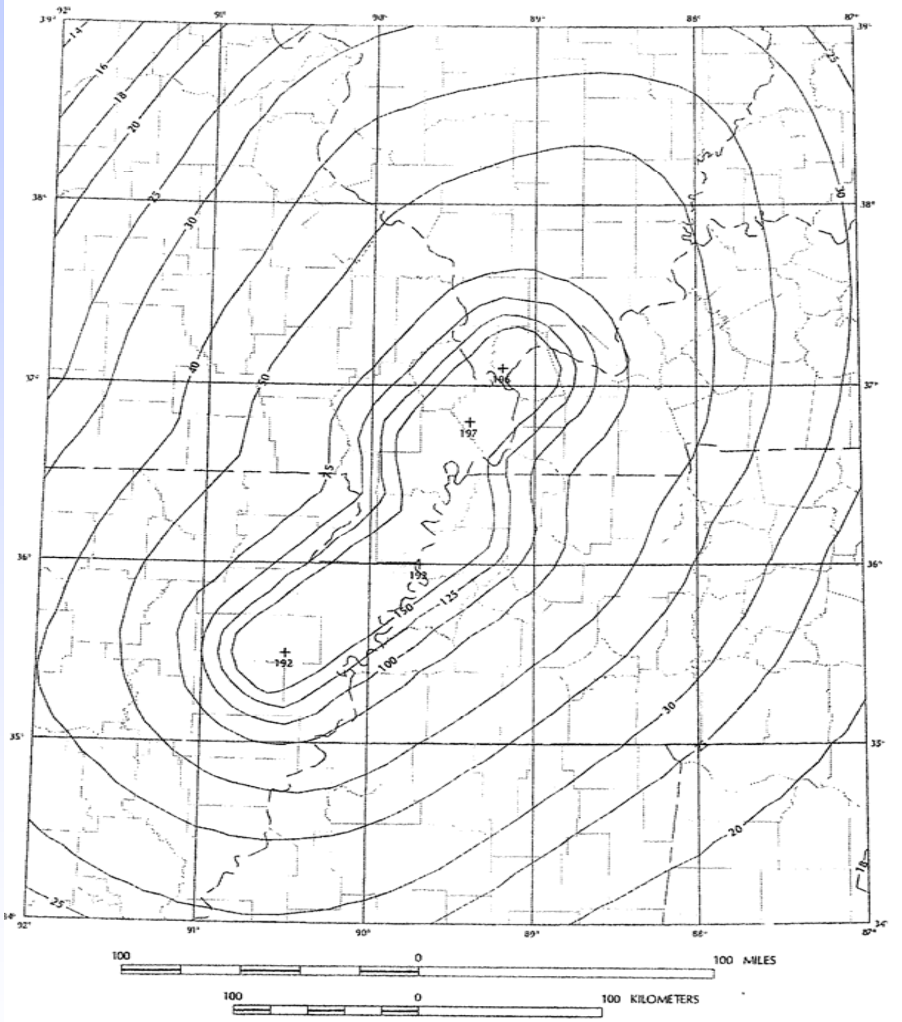
AASHTO/USGS Maps (3.4.1)



HORIZONTAL SPECTRAL RESPONSE ACCELERATION FOR REGION 3 OF 0.2-SECOND PERIOD (5 PERCENT OF CRITICAL DAMPING) WITH 5 PERCENT PROBABILITY OF EXCEEDANCE IN 50 YEARS

EXPLANATION

Contour intervals, % g

300, 200, 150, 125, 100, 75, 50, 40, 30, 25, 20, 18, 16, 14, 12, 10, 8, 6, 4, 2

Note: contours are irregularly spaced

+ 6.2 — Point value of spectral response acceleration expressed as a percent of gravity.

—10— Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.

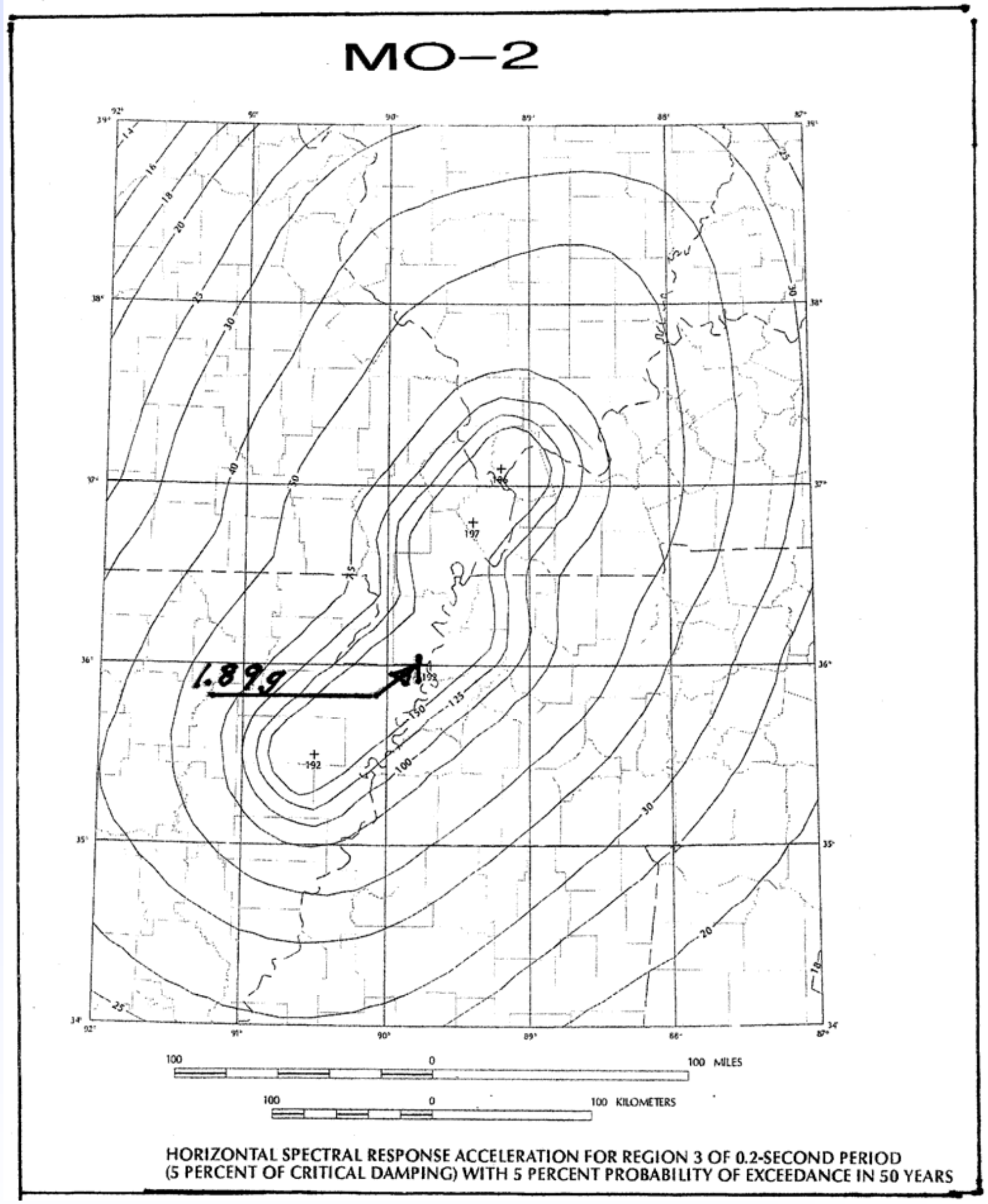
DISCUSSION

Refer to the map of Horizontal Spectral Response Acceleration for the Conterminous United States of 0.2-Second Period (5 Percent of Critical Damping) With 5 Percent Probability of Exceedance in 50 Years for additional discussion and references.

Index map showing location of study area

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)

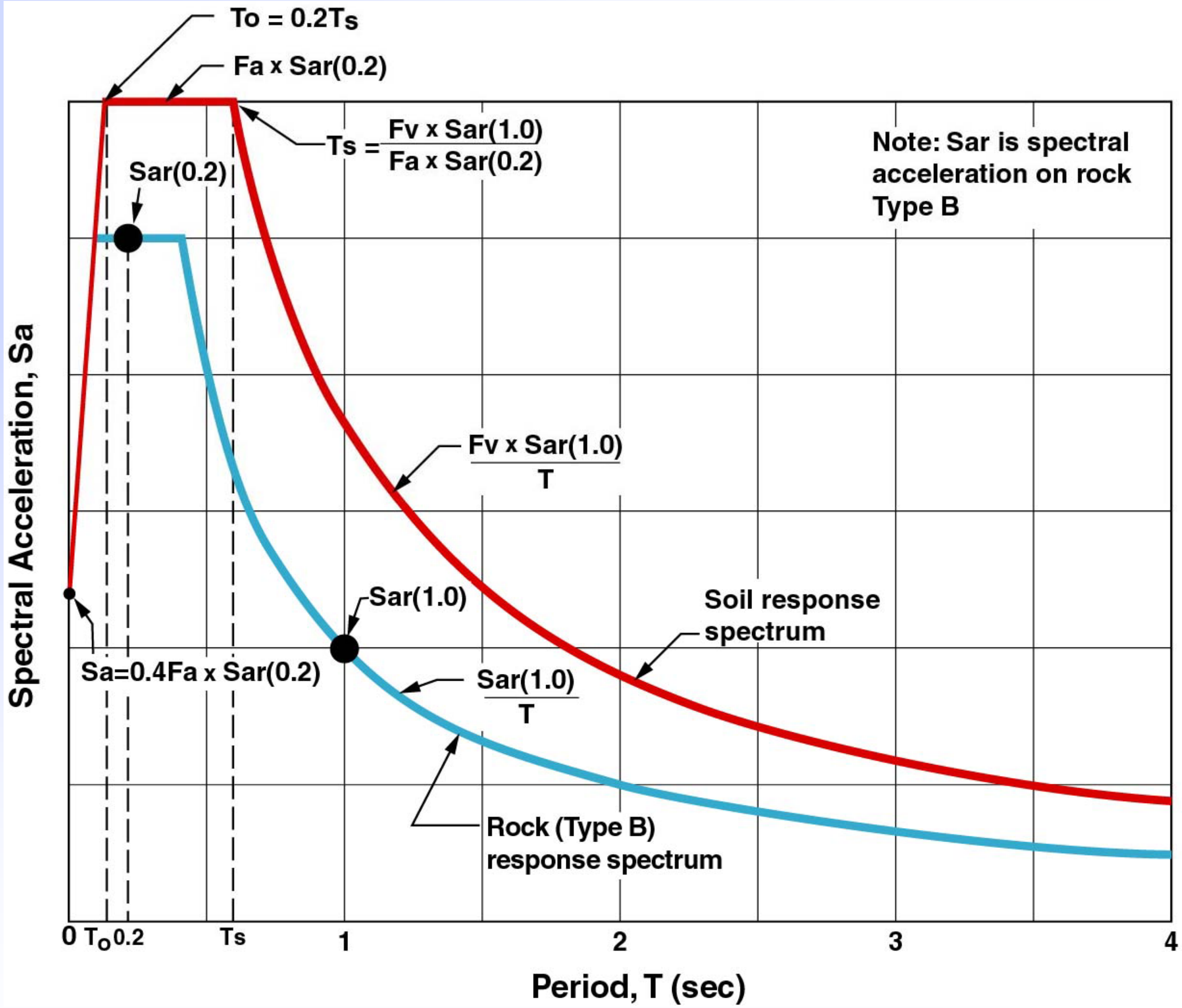
Table 3.4.2.3-2: Values of F_v as a Function of Site Class and Mapped 1 Second Period Spectral Acceleration

Site Class	Mapped Spectral Response Acceleration at 1 Second Periods				
	$S_1 \leq 0.1$ g	$S_1 = 0.2$ g	$S_1 = 0.3$ g	$S_1 = 0.4$ g	$S_1 \geq 0.5$ g
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	a	a	a	a	a

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.

a 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)



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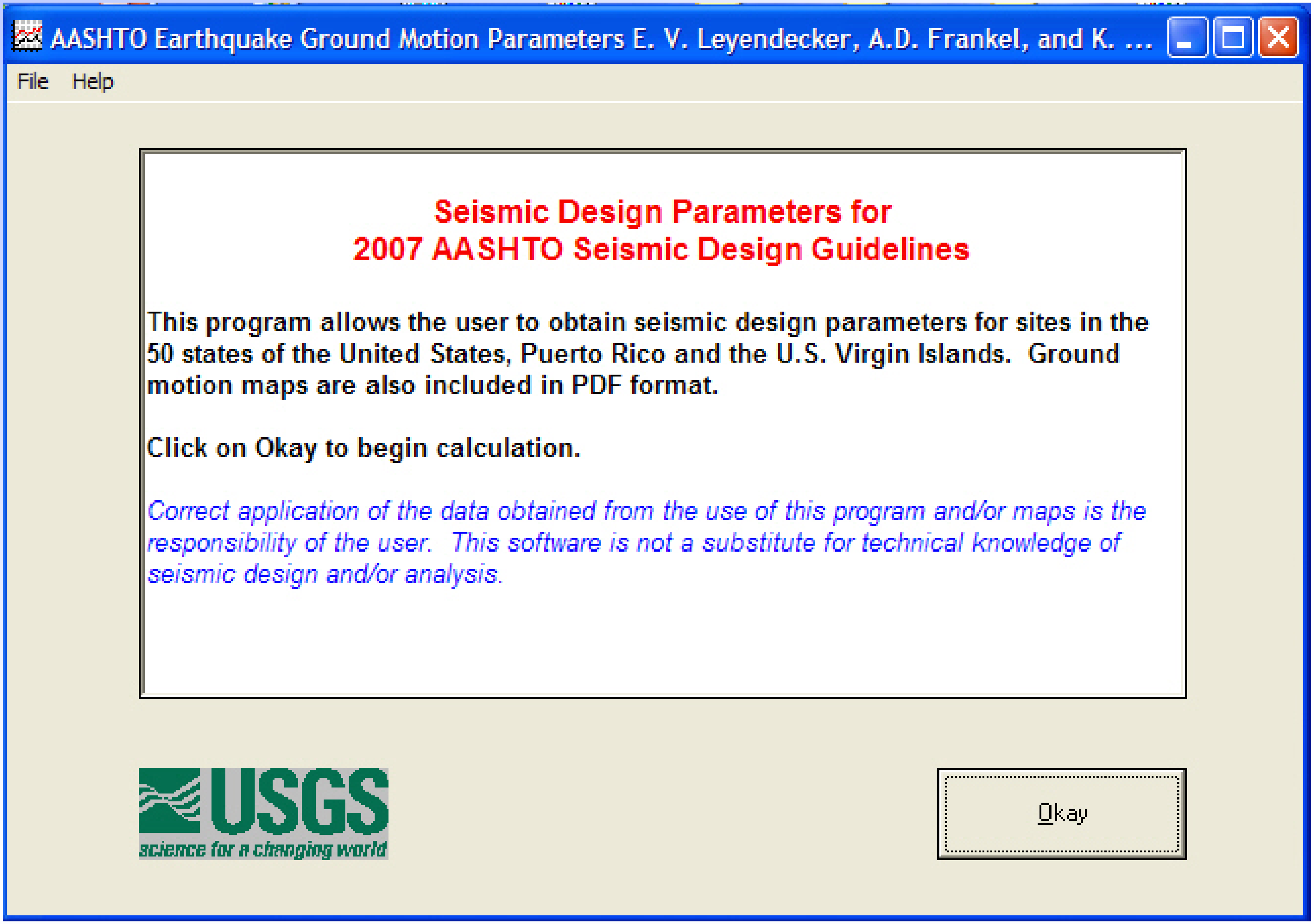
AASHTO Earthquake Ground Motion Parameters E. V. Leyendecker, A.D. Frankel, and K. ...
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Seismic Design Parameters for
2007 AASHTO Seismic Design Guidelines
This program allows the user to obtain seismic design parameters for sites in the 50 states of 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.
USGS
science for a changing world
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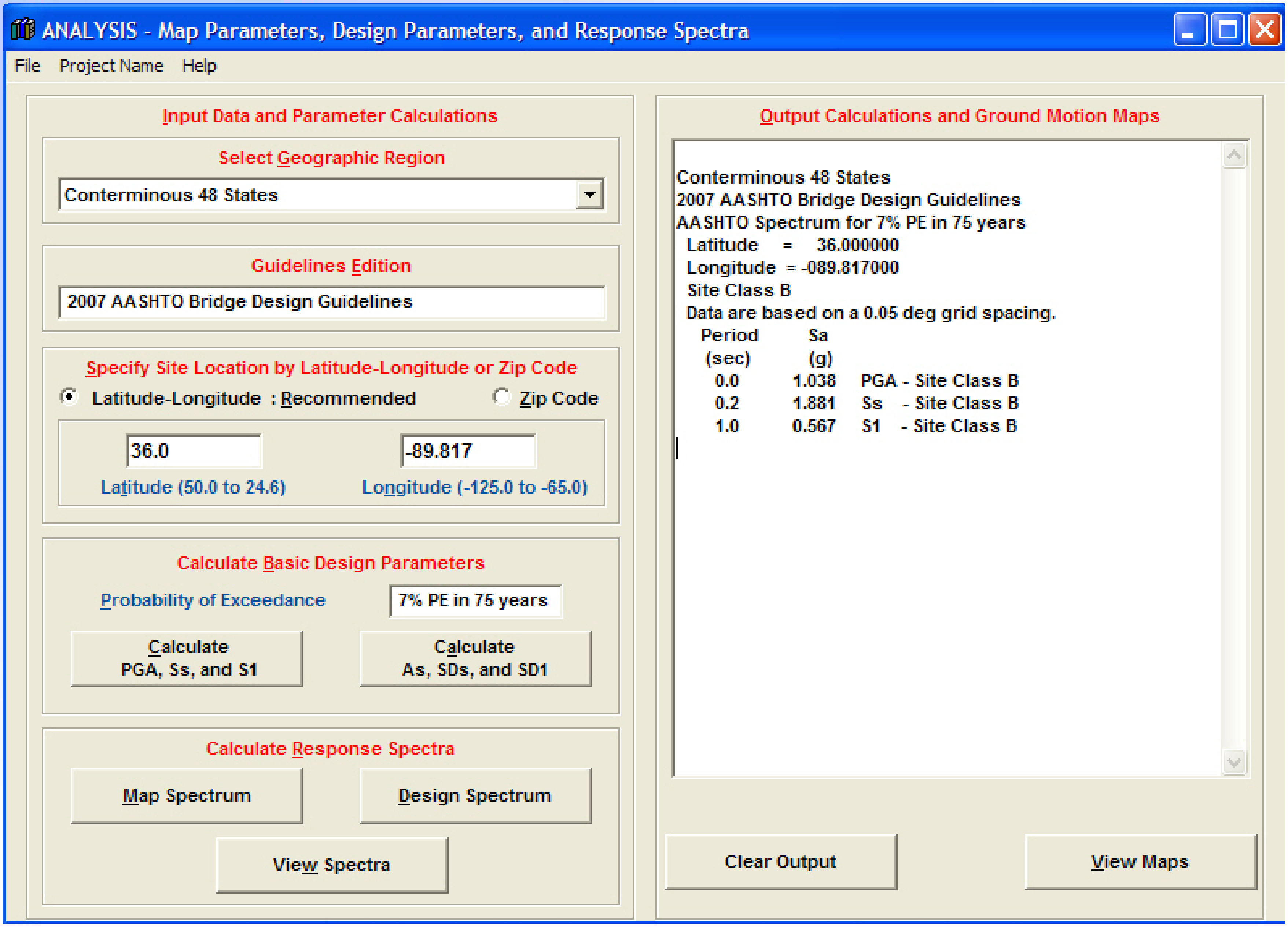
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Input Data and Parameter Calculations
Select Geographic Region
Conterminous 48 States
Guidelines Edition
2007 AASHTO Bridge Design Guidelines
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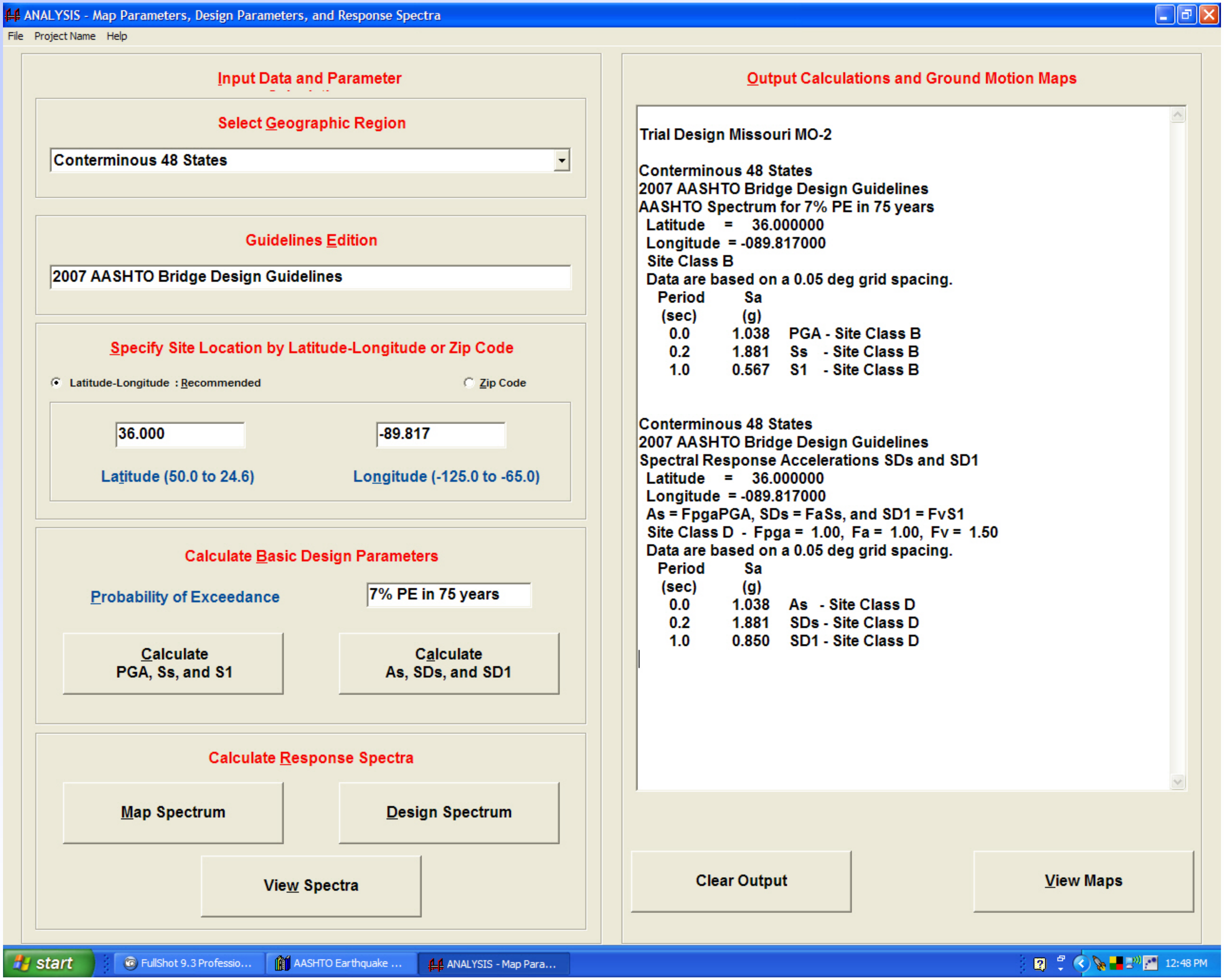
Latitude-Longitude : Recommended
Zip Code
36.0
-89.817
Latitude (50.0 to 24.6)
Longitude (-125.0 to -65.0)
Calculate Basic Design Parameters
Probability of Exceedance
7% PE in 75 years
Calculate PGA, Ss, and S1
Calculate As, SDs, and SD1
Calculate Response Spectra
Map Spectrum
Design Spectrum
View Spectra
Output Calculations and Ground Motion Maps
Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 36.000000
Longitude = -089.817000
Site Class B
Data are based on a 0.05 deg grid spacing.
Period Sa
(sec) (g)
0.0 1.038 PGA - Site Class B
0.2 1.881 Ss - Site Class B
1.0 0.567 S1 - Site Class B
Clear Output
View Maps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Conterminous 48 States
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Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 36.000000
Longitude = -089.817000
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.00, Fa = 1.00, Fv = 1.50
Data are based on a 0.05 deg grid spacing.
Period Sa
(sec) (g)
0.0 1.038 As - Site Class D
0.2 1.881 SDs - Site Class D
1.0 0.850 SD1 - Site Class D
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FullShot 9.3 Professio...
AASHTO Earthquake ...
ANALYSIS - Map Para...
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Site Coefficients

File Help

Calculate Site Coefficient

Accelerations

PGA: 1.038

Ss: 1.881

S1: 0.567

Site Class

- Site Class A
- Site Class B (selected)
- Site Class C
- Site Class D
- Site Class E
- Site Class F

Site Coefficients

Fpga: 1.00

Fa: 1.00

Fv: 1.00

OK

Site Class	Values of Site Factor, Fpga For Zero-Period Range of Acceleration Peak Ground Acceleration Coefficient (PGA)				
	PGA <= 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA >= 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*	*	*	*	*

Site Class	Values of Site Factor, Fa For Short-Period Range of Spectral Spectral Acceleration Coefficient at Period 0.2 sec (Ss)				
	Ss <=0.25	Ss =0.50	Ss =0.75	Ss =1.00	Ss >=1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	*	*	*	*	*

Site Class	Values of Site Factor, Fv For Long-Period Range of Spectral Spectral Acceleration Coefficient at Period 1.0 sec (S1)				
	S1 <=0.10	S1 =0.20	S1 =0.30	S1 =0.40	S1 >=0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	*	*	*	*	*

SITE FACTOR TABLE NOTES

1. Use straight-line interpolation to calculate values of site coefficients for intermediate values of PGA, Ss, and S1.
2. Site-specific geotechnical investigation and dynamic site response analyses should be performed for all sites in Site Class F.

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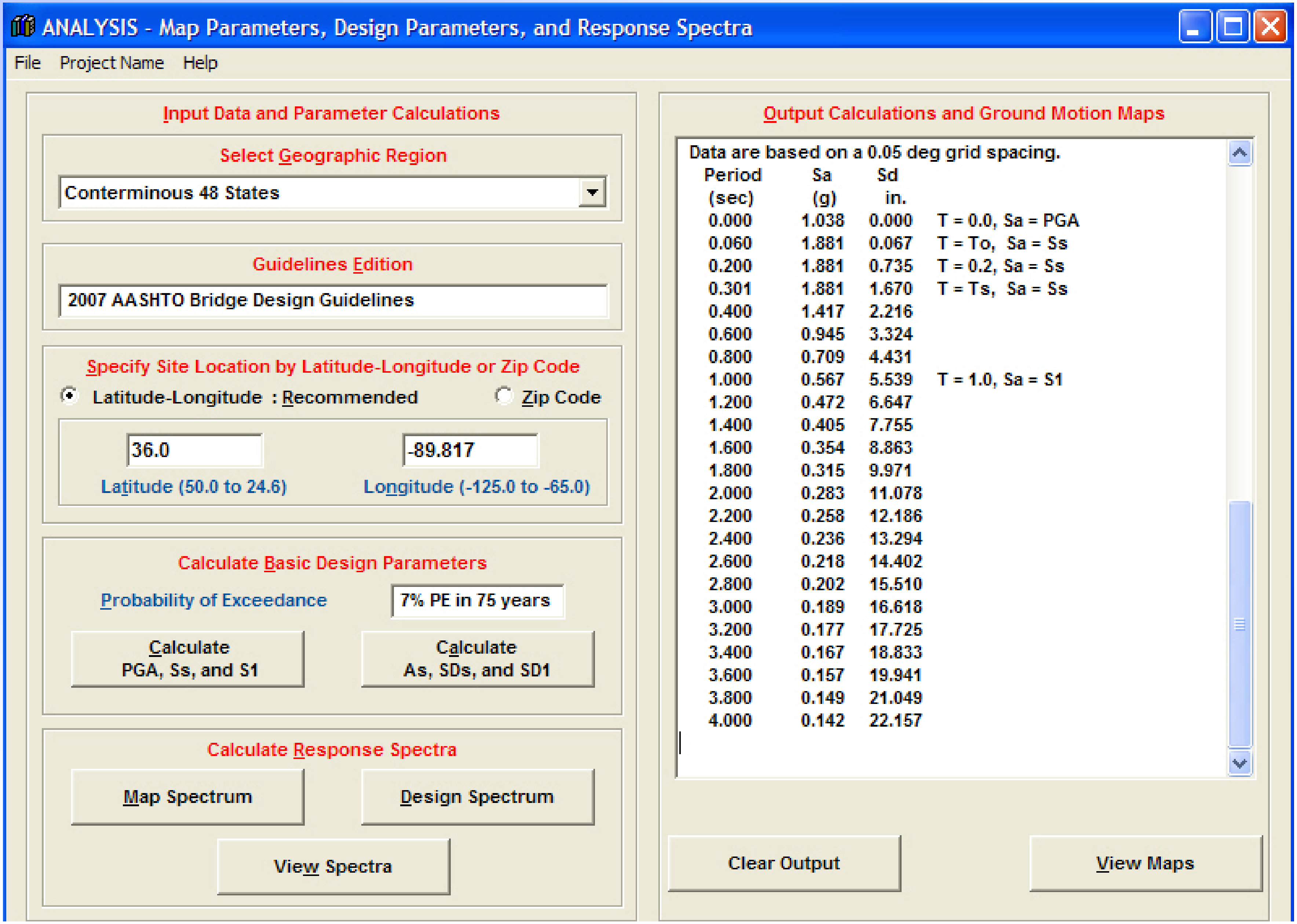
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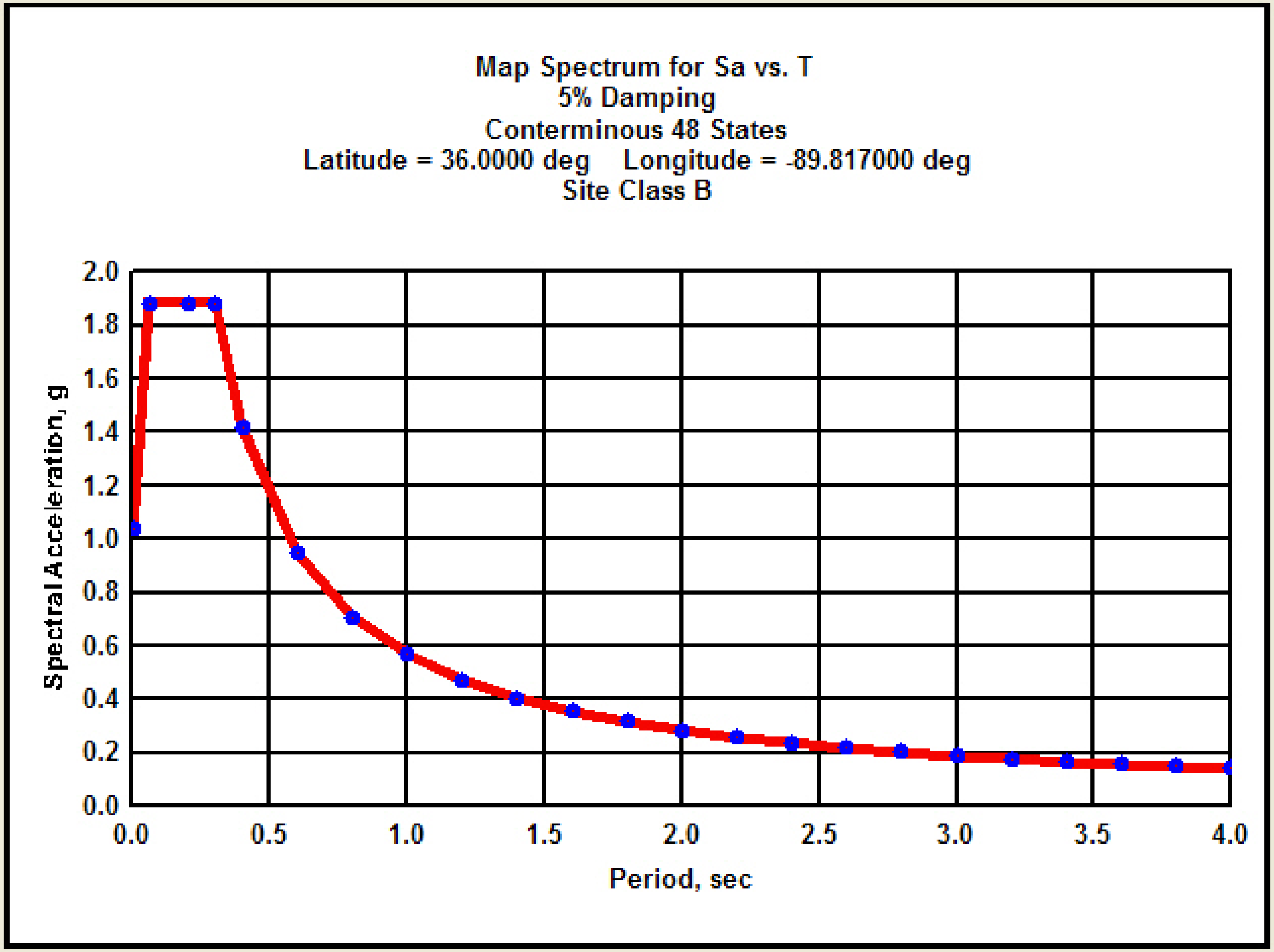
Calculate As, SDs, and SD1
Calculate Response Spectra
Map Spectrum
Design Spectrum
View Spectra
Output Calculations and Ground Motion Maps
Data are based on a 0.05 deg grid spacing.
Period (sec) Sa (g) Sd in.
0.000 1.038 0.000 T = 0.0, Sa = PGA
0.060 1.881 0.067 T = To, Sa = Ss
0.200 1.881 0.735 T = 0.2, Sa = Ss
0.301 1.881 1.670 T = Ts, Sa = Ss
0.400 1.417 2.216
0.600 0.945 3.324
0.800 0.709 4.431
1.000 0.567 5.539 T = 1.0, Sa = S1
1.200 0.472 6.647
1.400 0.405 7.755
1.600 0.354 8.863
1.800 0.315 9.971
2.000 0.283 11.078
2.200 0.258 12.186
2.400 0.236 13.294
2.600 0.218 14.402
2.800 0.202 15.510
3.000 0.189 16.618
3.200 0.177 17.725
3.400 0.167 18.833
3.600 0.157 19.941
3.800 0.149 21.049
4.000 0.142 22.157
Clear Output
View Maps

GRAPHICS - Response Spectra

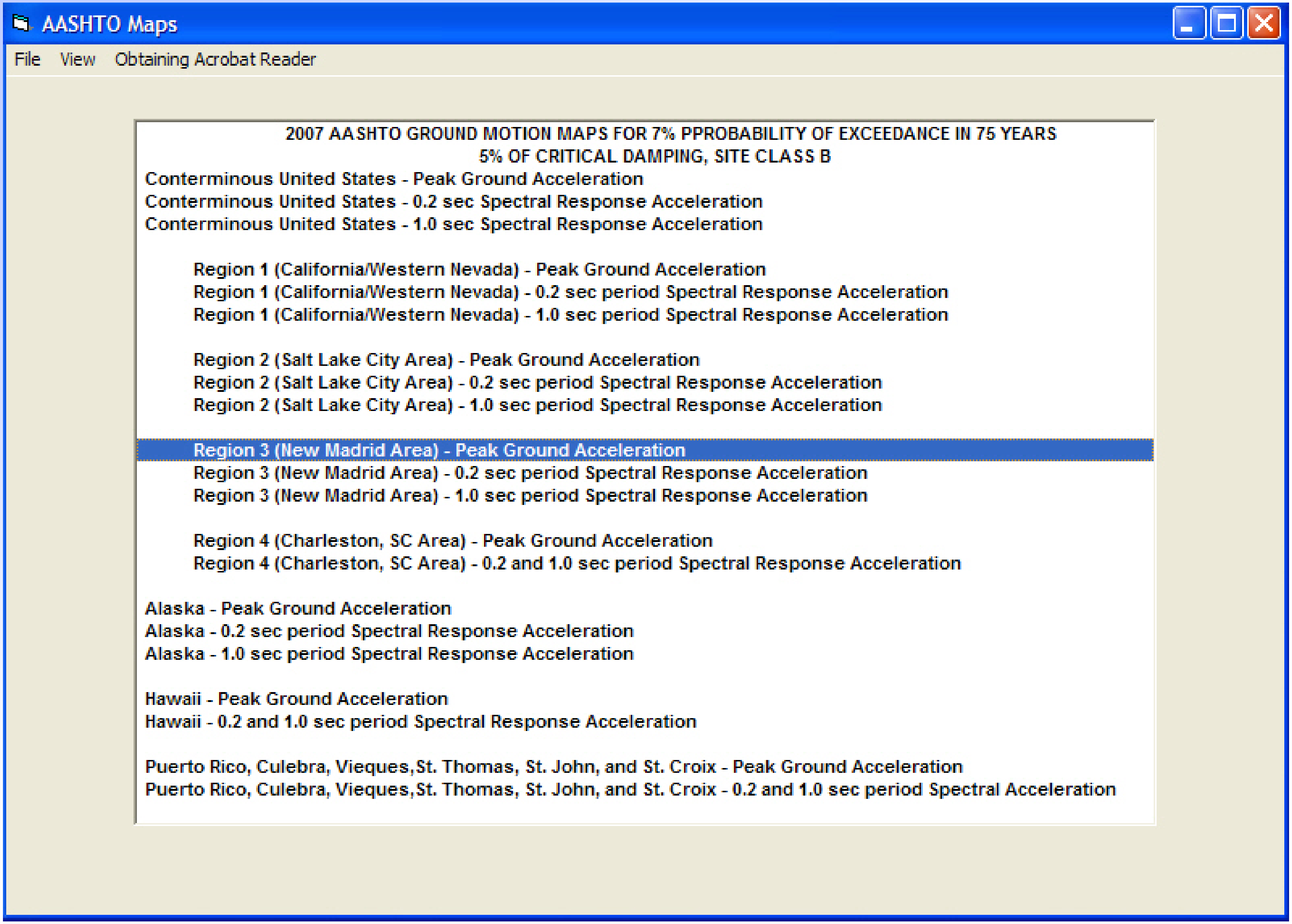
File Edit Graph Scale Help

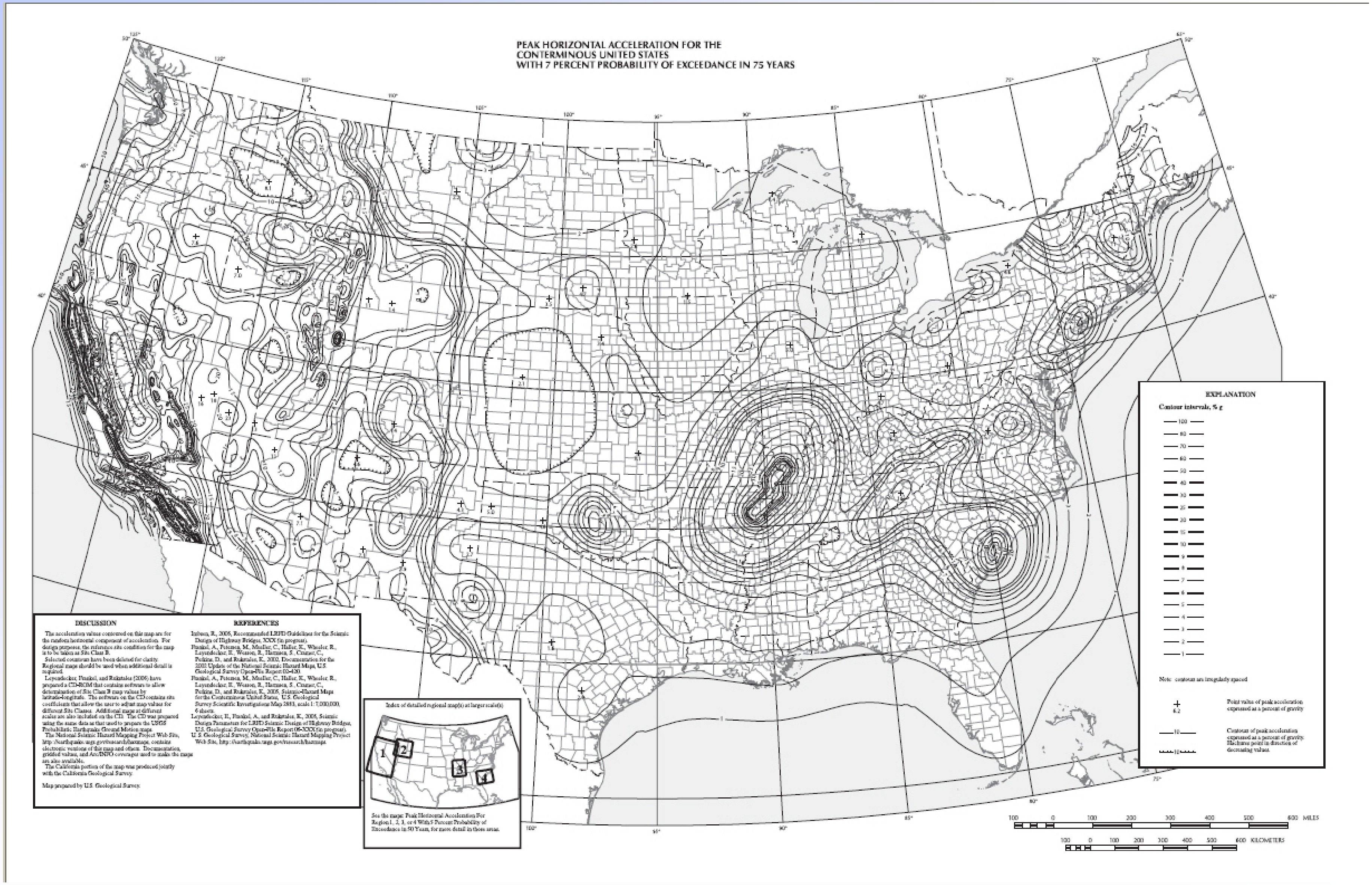
Select Graph: Map Spectrum for Sa vs. T



Graph Data

Period, sec	Sa, g
0.00	1.0381
0.06	1.8811
0.20	1.8811
0.30	1.8811
0.40	1.4174
0.60	0.9449
0.80	0.7087
1.00	0.5670
1.20	0.4725
1.40	0.4050
1.60	0.3543
1.80	0.3150
2.00	0.2835
2.20	0.2577
2.40	0.2362
2.60	0.2181
2.80	0.2025
3.00	0.1890
3.20	0.1772
3.40	0.1668
3.60	0.1575
3.80	0.1492
4.00	0.1417


AASHTO Maps
File View Obtaining Acrobat Reader
2007 AASHTO GROUND MOTION MAPS FOR 7% PPROBABILITY OF EXCEEDANCE IN 75 YEARS
5% OF CRITICAL DAMPING, SITE CLASS B
Conterminous United States - Peak Ground Acceleration
Conterminous United States - 0.2 sec Spectral Response Acceleration
Conterminous United States - 1.0 sec Spectral Response Acceleration
Region 1 (California/Western Nevada) - Peak Ground Acceleration
Region 1 (California/Western Nevada) - 0.2 sec period Spectral Response Acceleration
Region 1 (California/Western Nevada) - 1.0 sec period Spectral Response Acceleration
Region 2 (Salt Lake City Area) - Peak Ground Acceleration
Region 2 (Salt Lake City Area) - 0.2 sec period Spectral Response Acceleration
Region 2 (Salt Lake City Area) - 1.0 sec period Spectral Response Acceleration
Region 3 (New Madrid Area) - Peak Ground Acceleration
Region 3 (New Madrid Area) - 0.2 sec period Spectral Response Acceleration
Region 3 (New Madrid Area) - 1.0 sec period Spectral Response Acceleration
Region 4 (Charleston, SC Area) - Peak Ground Acceleration
Region 4 (Charleston, SC Area) - 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


PEAK HORIZONTAL ACCELERATION FOR THE
CONTERMINOUS UNITED STATES
WITH 7 PERCENT PROBABILITY OF EXCEEDANCE IN 75 YEARS
DISCUSSION
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SDC Range of Applicable Analysis (3.5)

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.

Value of $S_{D1} = F_v S_1$	SDC
$S_{D1} < 0.15$	A
$0.15 \le S_{D1} < 0.30$	B
$0.30 \le S_{D1} < 0.50$	C
$0.50 \le S_{D1}$	D

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

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SDC B (3.5)

2. SDC B

   a. No Identification of ERS according to Article 3.3

   b. Demand Analysis

   c. Implicit Capacity Check Required (displacement, P-Δ, support length)

   d. No Capacity Design Required except for column shear requirement

   e. SDC B Level of Detailing

SDC C (3.5)

3. SDC C

   a. Identification of ERS

   b. Demand Analysis

   c. Implicit Capacity Check Required (displacement, P-Δ, support length)

   d. Capacity Design Required including column shear requirement

   e. SDC C Level of Detailing

SDC D (3.5)

4. SDC D

   a. Identification of ERS

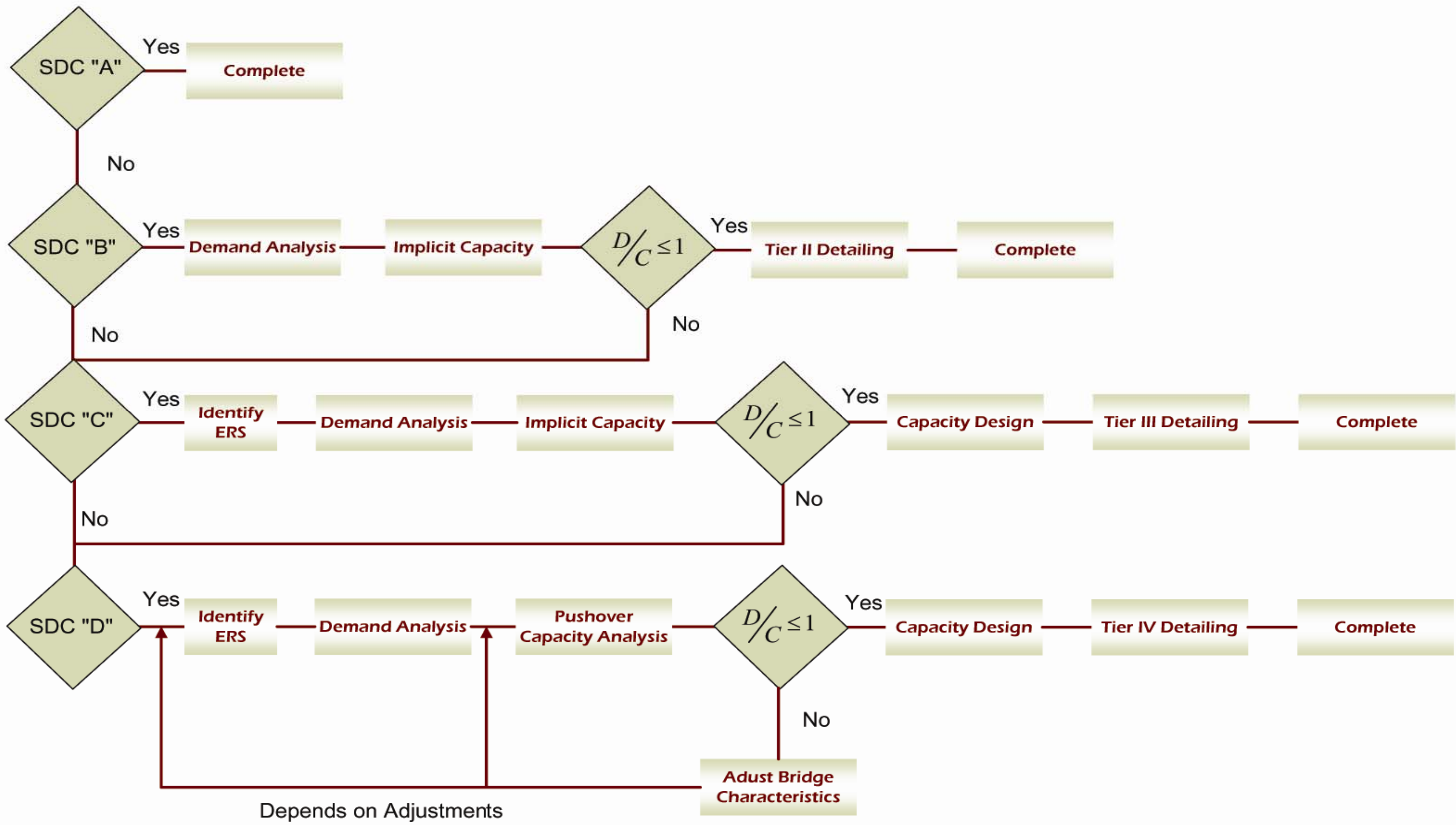
   b. Demand Analysis

   c. Displacement Capacity Required using Pushover Analysis (check P-Δ 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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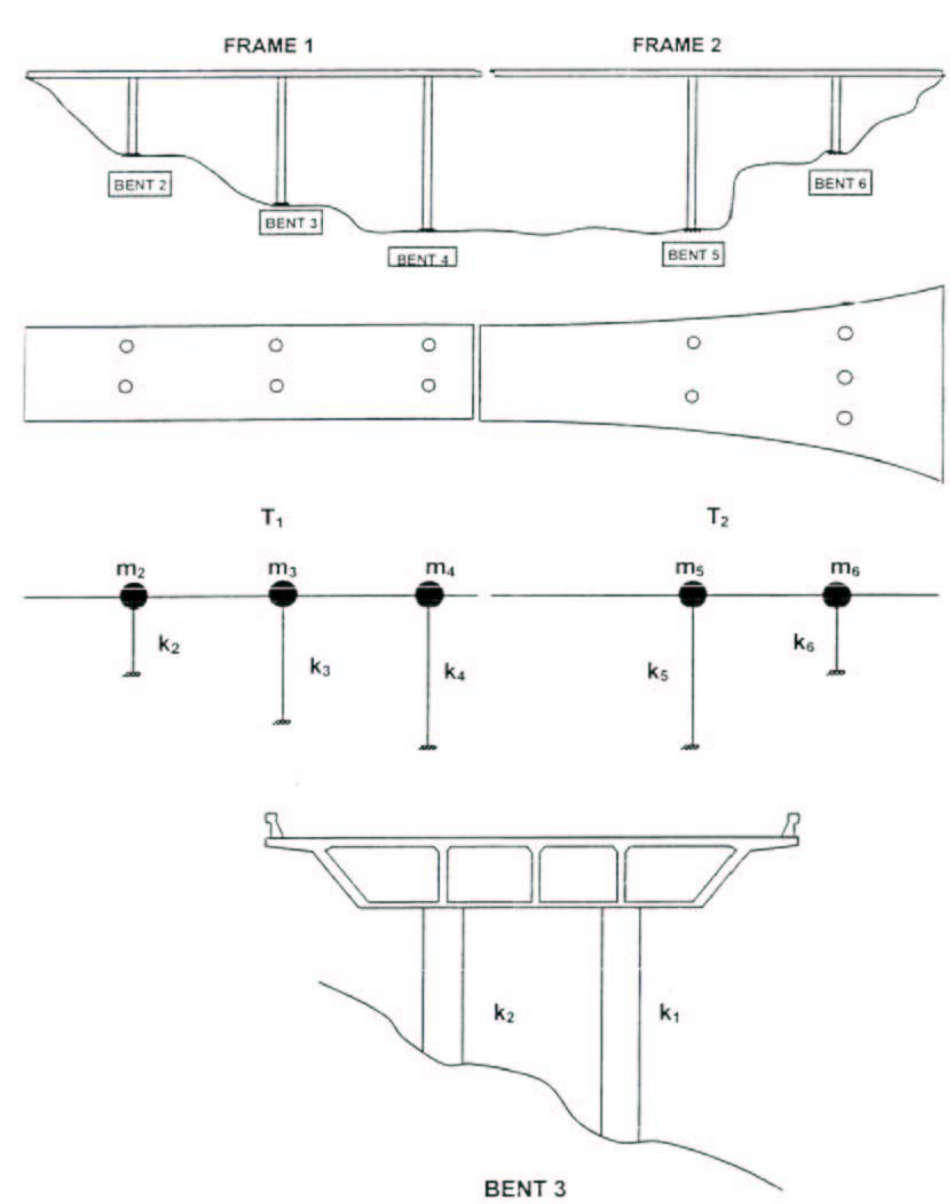
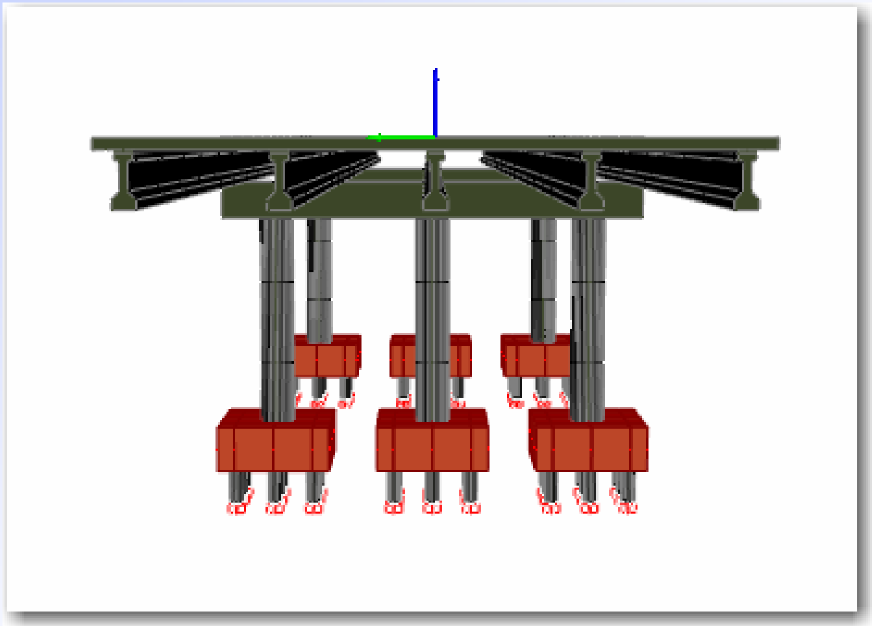
Balanced Stiffness Recommendation (4.1)



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:

$$\frac{k_i^e}{k_j^e} \geq 0.5 \qquad (4.1.2\text{-}1)$$

Variable Width Frames:

$$\frac{k_i^e m_j}{k_j^e m_i} \geq 0.5 \qquad (4.1.2\text{-}2)$$

Balanced Bent (4.1.2)

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

Constant Width Frames:

$$\frac{k_i^e}{k_j^e} \geq 0.75 \qquad (4.1.2\text{-}3)$$

Variable Width Frames:

$$\frac{k_i^e m_j}{k_j^e m_i} \geq 0.75 \qquad (4.1.2\text{-}4)$$

Analysis Procedure (4.2)

Seismic Design Category	Regular Bridges with 2 through 6 Spans	Not Regular Bridges with 2 or more Spans
A	Not required	Not required
B, C, or D	Use Procedure 1 or 2	Use Procedure 2

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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:

$$\Delta_C^L = 0.12H_o\left(-1.27\ln(x) - 0.32\right) \geq 0.12H_o \qquad (4.8.1\text{-}1)$$

For SDC C:

$$\Delta_C^L = 0.12H_o\left(-2.32\ln(x) - 1.22\right) \geq 0.12H_o \qquad (4.8.1\text{-}2)$$

in which:

$$x = \frac{\Lambda B_o}{H_o} \qquad (4.8.1\text{-}3)$$

Displacement Capacity for SDC D (4.8.2)

- Inelastic Quasi-Static Pushover analysis (IQPA) is required to determine realistic displacement capacities as it reaches it's limit states
- IQPA is an incremental linear analysis which captures the overall nonlinear behavior of the structure and it's 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:

$$\mu_D \leq 5 \quad (4.9\text{-}1)$$

For multiple column bents:

$$\mu_D \leq 6 \quad (4.9\text{-}2)$$

For pier walls in the weak direction:

$$\mu_D \leq 5 \quad (4.9\text{-}3)$$

For pier walls in the strong direction:

$$\mu_D \leq 1 \quad (4.9\text{-}4)$$

Member Ductility Requirement for SDC D (4.9)

$$\mu_D = 1 + \frac{\Delta_{pd}}{\Delta_{yi}} \qquad (4.9\text{-}5)$$

Where:

Δ_{pd} = plastic displacement demand (in.)

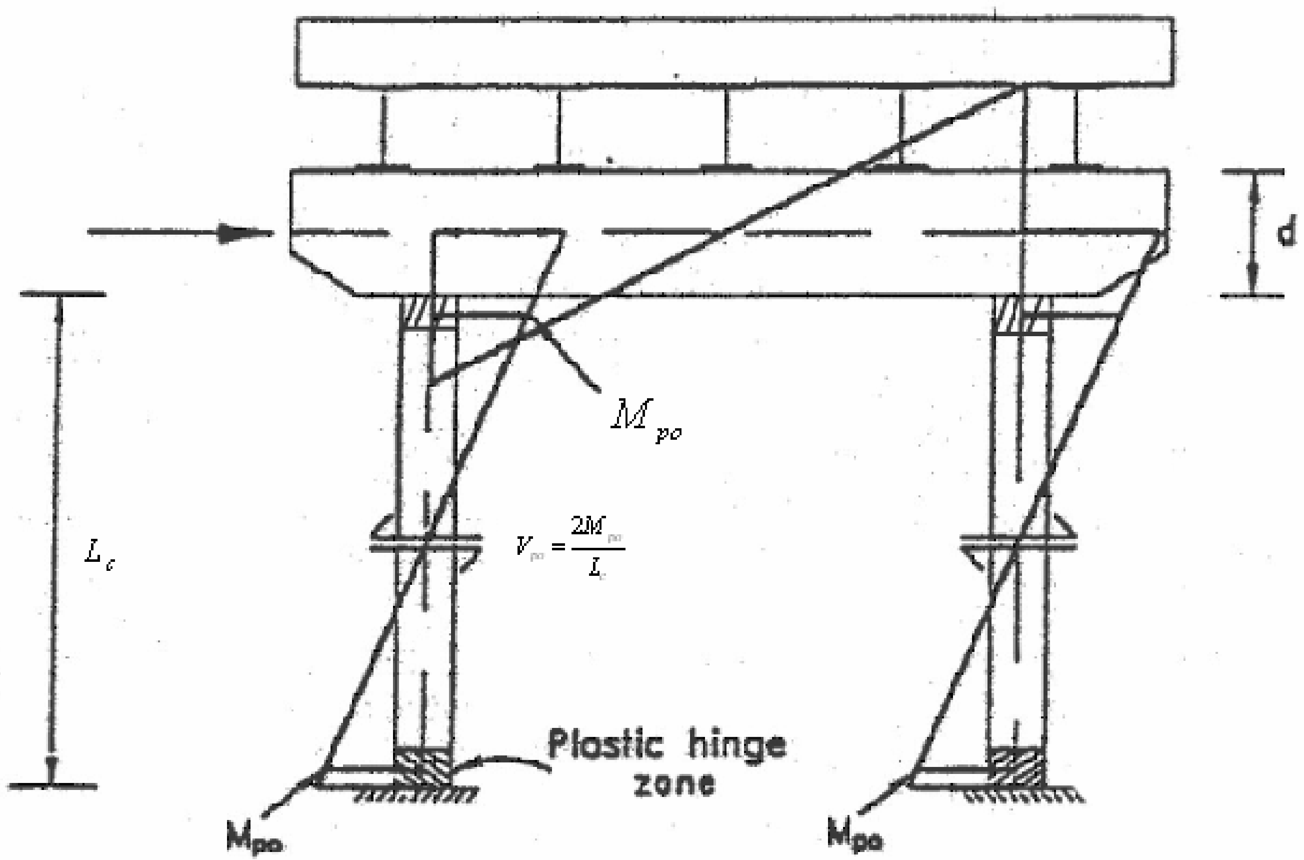
Δ_{yi} = idealized yield displacement corresponding to the idealized yield curvature, ϕ_{yi} , shown in figure 8.5-1 (in.)

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)

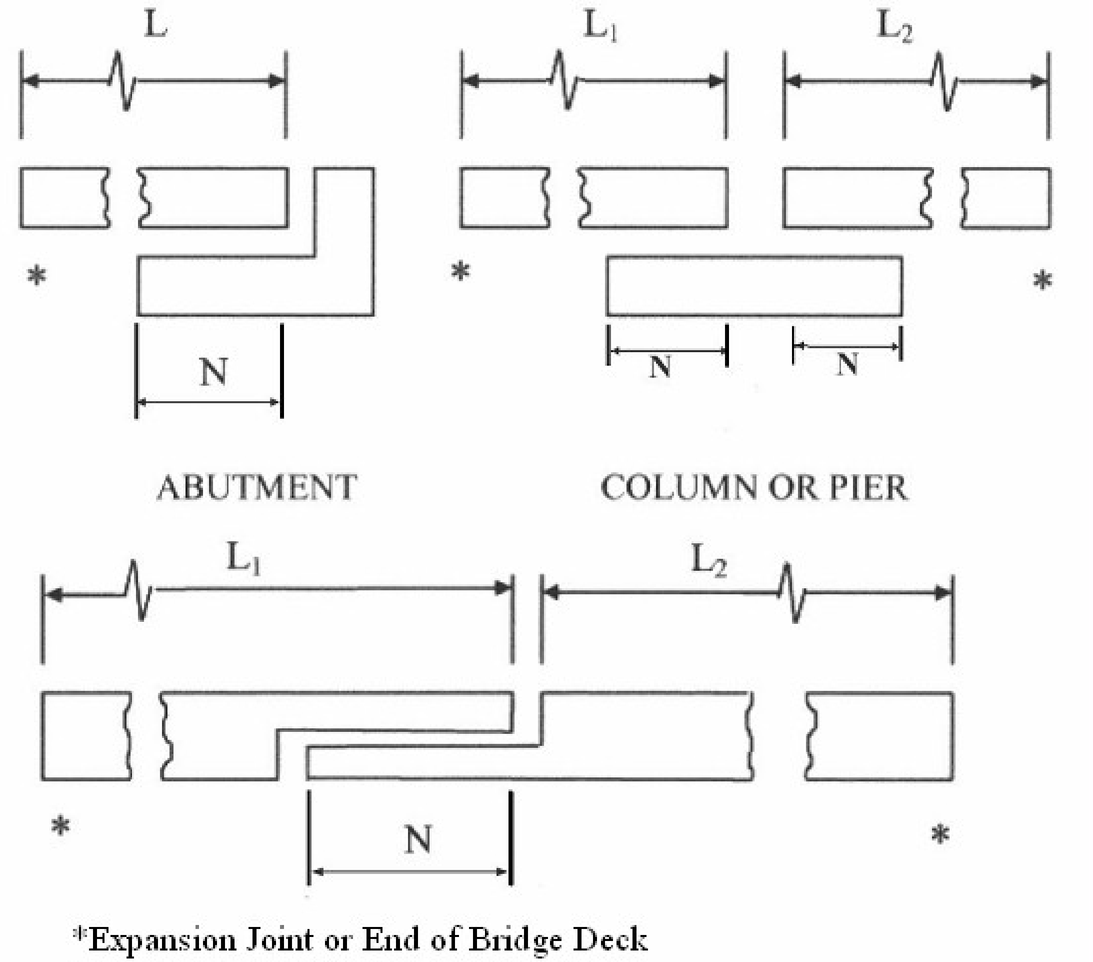


Minimum Support Length Requirements (4.12)

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



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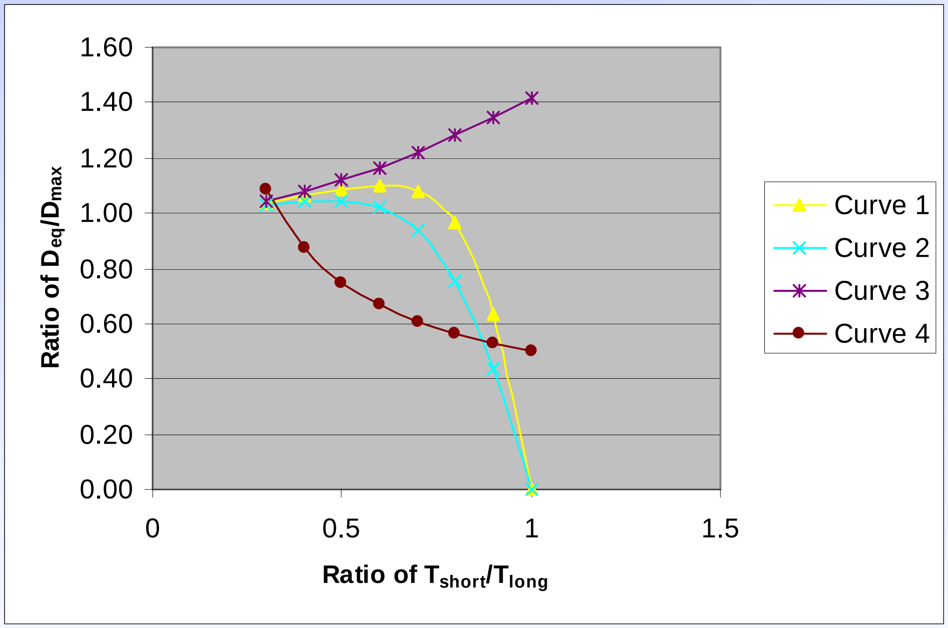
Minimum Support Length (4.12) SDC A,B, C

$$N = (8 + 0.02L + 0.08H)(1 + 0.000125S^2) \qquad (4.12.2\text{-}1)$$

Table 4.12.2-1 Percentage N by SDC and effective peak ground acceleration, A_s

SDC	Effective peak ground acceleration, A_s	Percent N
A	< 0.05	≥ 75
A	≥ 0.05	100
B	All applicable	150
C	All applicable	150

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

$$N = (4 + 1.65\Delta_{eq})(1 + 0.00025\ S^2) \geq 24 \qquad (4.12.3\text{-}1)$$

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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

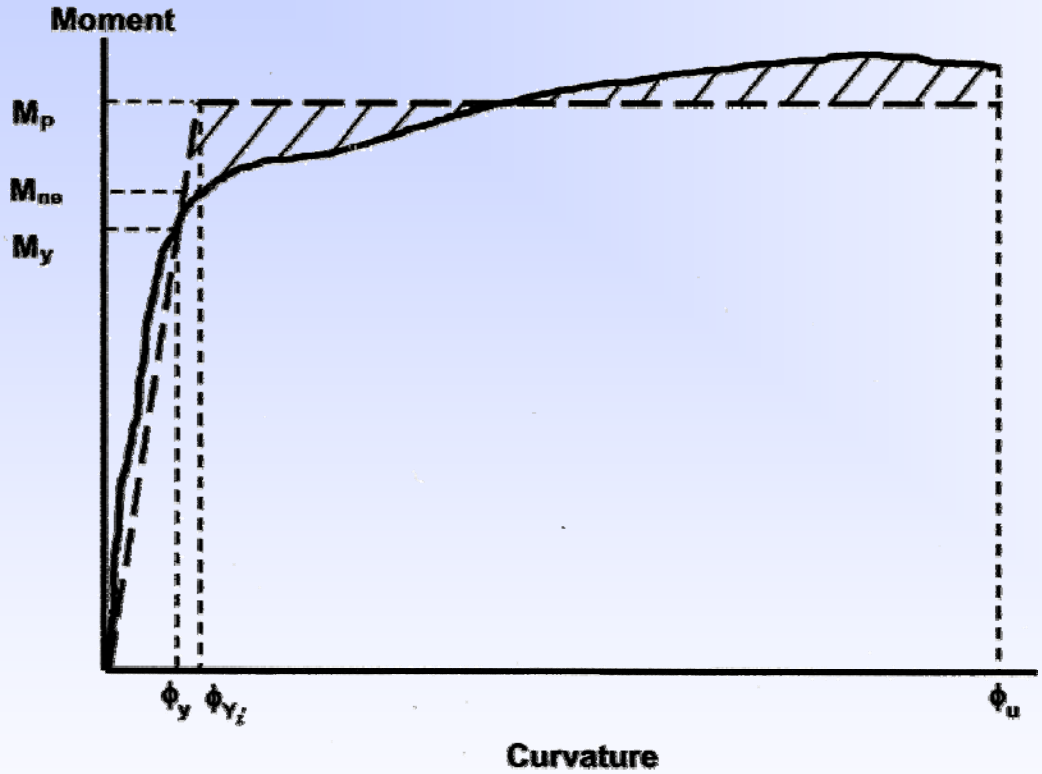


Figure 8.5-1 Moment-Curvature Model

Force Demands on Capacity Protected Members

$$M_{po} = \lambda_{mo} M_{p} \qquad (8.5\text{-}1)$$

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)

λ_{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_μ is the shear demand force, with the overstrength moment M_{po} and corresponding plastic shear

Shear Demand & Capacity (8.6.1) con't

- Shear strength capacity within the plastic hinge is based on nominal motion strength properties

$$\phi_s V_n \geq V_u \qquad (8.6.1\text{-}1)$$

in which

$$V_n = V_e + V_g \qquad (8.6.1\text{-}2)$$

ϕ_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)

$$V_c = v_c A_e \quad (8.6.2\text{-}1)$$

$$A_e = 0.8 A_g \quad (8.6.2\text{-}2)$$

If Pc is compressive then

$$v_c = 0.032\alpha' \left\{ \{1 + \frac{P_u}{2A_g}\} \right\} \sqrt{f_c'} \le \left\{ \begin{array}{l} 0.11\sqrt{f_c'} \\ 0.047\alpha\sqrt{f_c'} \end{array} \right\} \quad (8.6.2\text{-}3)$$

Otherwise (i.e., not compression)

$$v_c = 0 \quad (8.6.2\text{-}4)$$

Concrete Shear Capacity SDC B, C & D (8.6.2)

For circular columns in compression with spiral or hoop reinforcing:

$$0.3 \leq \alpha' \frac{f_s}{0.15} + 3.67 - \mu_o \leq 3 \qquad (8.6.2\text{-}5)$$

$$f_s = e_s f_{yh} \leq 0.35 \qquad (8.6.2\text{-}6)$$

$$e_s = \frac{4A_{sp}}{sD} \qquad (8.6.2\text{-}7)$$

Concrete Shear Capacity SDC B, C & D (8.6.2)

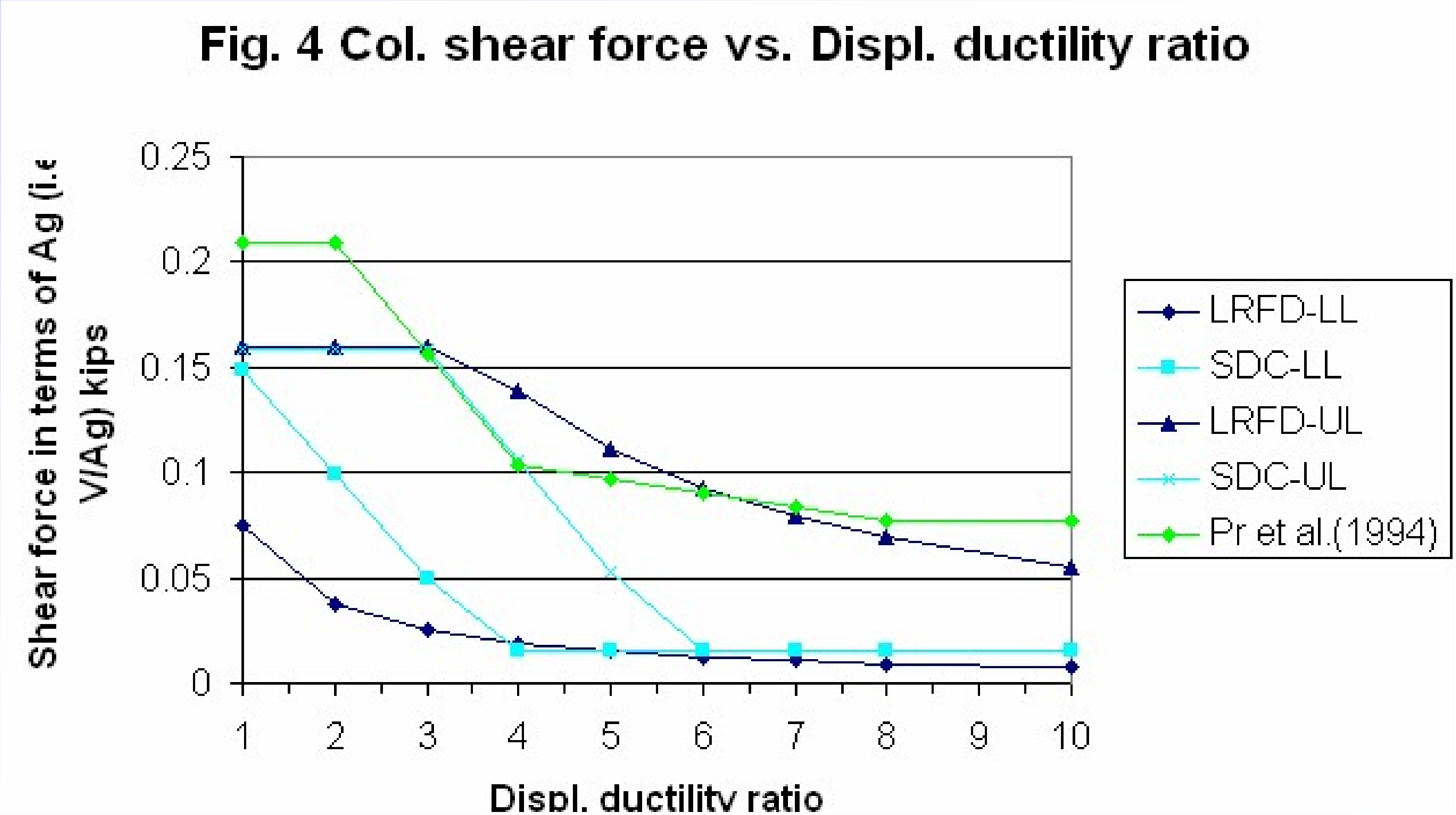
For rectangular columns in compression with ties:

$$0.3 \le \alpha' \frac{f_w}{0.15} + 3.67 - \mu_D \le 3 \qquad (8.6.2\text{-}8)$$

$$f_w = 2 e_w f_{yh} \le 0.35 \qquad (8.6.2\text{-}9)$$

$$e_w = \frac{A_v}{bs} \qquad (8.6.2\text{-}10)$$

Column Shear Requirement (8.10) SDC D



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Integral Joint Shear Requirement (8.13) SDC C,and D

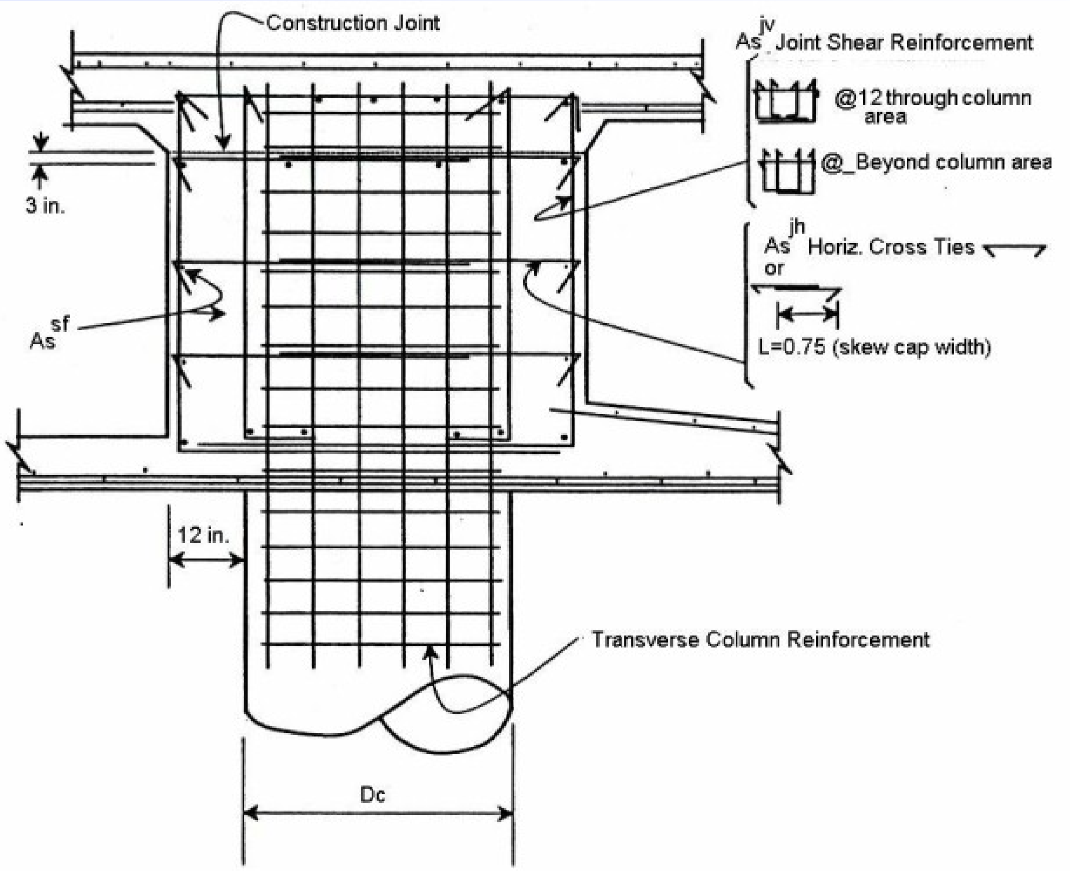
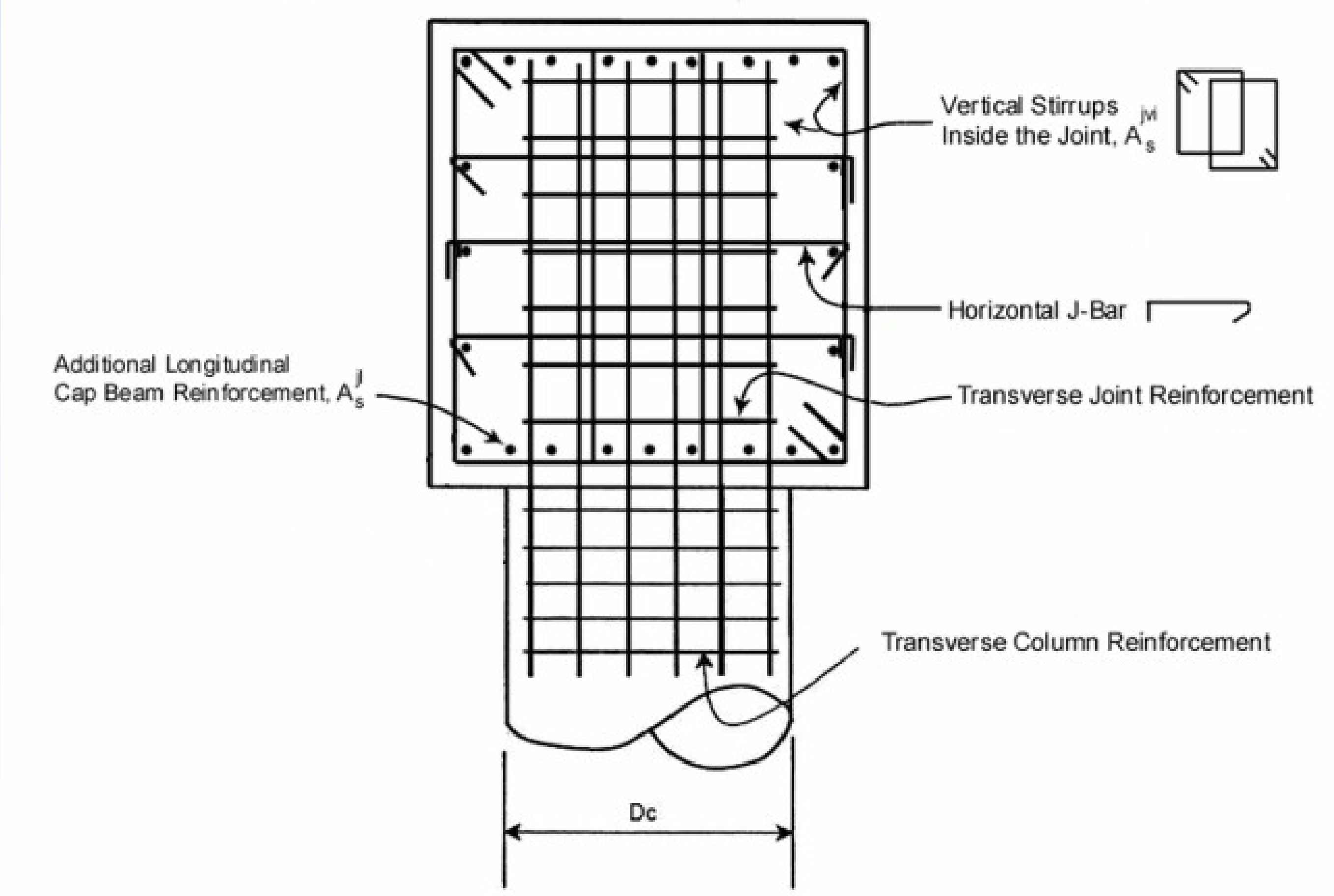


Figure 8.13.4.2.1-2 Joint Shear Reinforcement Details.

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

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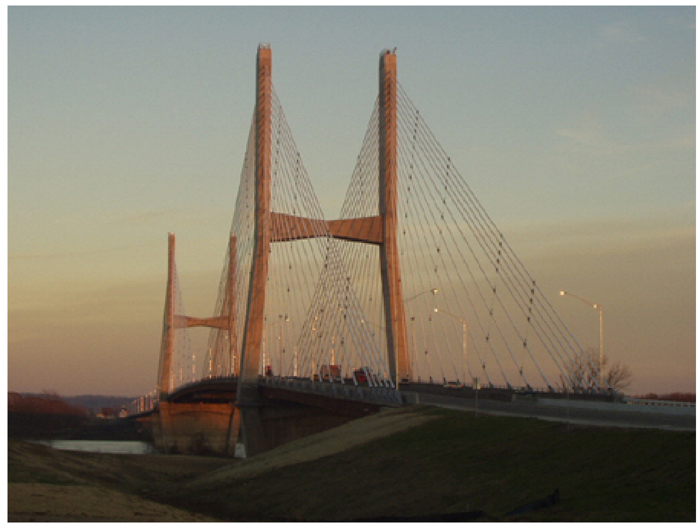

This page will serve as the posting site for questions and answers concerning the LRFD Guidelines for the Seismic Design of Highway Bridges.

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

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Presentation Topics

- Background-AASHTO LRFD Guide Specifications
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- 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

Module	**Description**	**Duration**
1	Introduction	45
2	Description of Story Line Bridge [SLB]	15
3	Structural Dynamics	45
4	Bridge Modeling & Analysis [SLB]	30
5	Seismic Hazard [SLB]	30
6	LRFD Guide Specifications-Introduction [SLB]	45
7	LRFD – Guide Specifications-Demand Analysis [SLB]	45
8	LRFD Guide Spec.-Concrete Substructure Type 1A [SLB]	60
9	LRFD Guide Spec.-Concrete Substructure Type 1A [SLB]	30
10	LRFD Guide Spec.-Reinforced Concrete Components	45
	Wrap-up and Summary	15

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 !!!!!!!!