THE REPUBLIC OF THE PHILIPPINES
DEPARTMENT OF PUBLIC WORKS AND HIGHWAYS (DPWH)

THE PROJECT FOR STUDY ON IMPROVEMENT OF BRIDGES THROUGH DISASTER MITIGATING MEASURES FOR LARGE SCALE EARTHQUAKES IN THE REPUBLIC OF THE PHILIPPINES

FINAL REPORT

APPENDICES

DECEMBER 2013

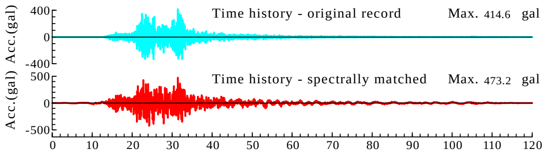
JAPAN INTERNATIONAL COOPERATION AGENCY (JICA)

CTI ENGINEERING INTERNATIONAL CO., LTD
CHODAI CO., LTD.
NIPPON KOEI CO., LTD.

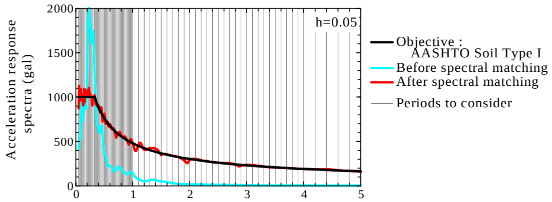
APPENDIX 2-C

ACCELERATION RESPONSE SPECTRA DEVELOPMENT BASED ON AASHTO

(1) Compatible to Target Spectrum (EQ2 – EQ17)



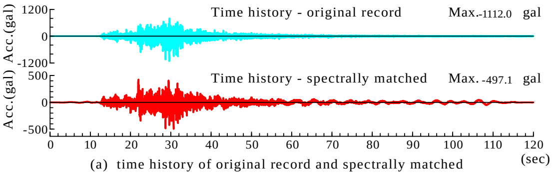
(a) time history of original record and spectrally matched



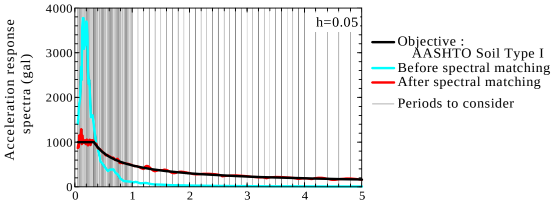
(b) Acceleration response spectra

(2) EQ02 : 2001.03.24 Geiyo Earthquake
K-NET HRS009 YUKI NS

Figure 1-1 EQ2 compatible to target spectrum



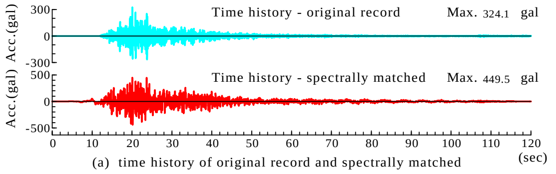
(a) time history of original record and spectrally matched



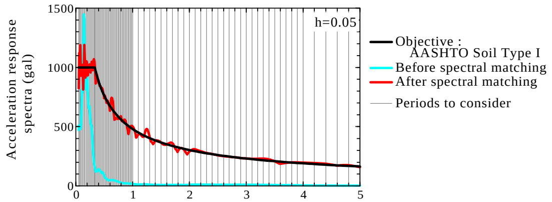
(b) Acceleration response spectra

(1) EQ03 : 2003.05.26 Miyagi-ken Oki Earthquake
K-NET MYG011 OSHIKA EW

Figure 1-2 EQ3 compatible to target spectrum



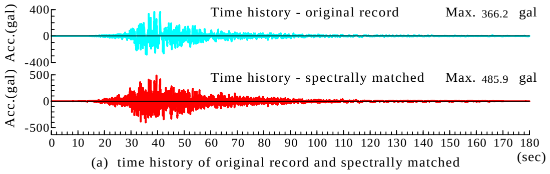
(a) time history of original record and spectrally matched



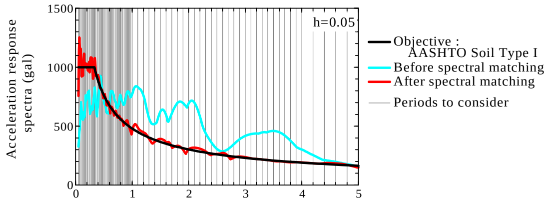
(b) Acceleration response spectra

(2) EQ04 : 2003.07.26 Miyagi-ken Hokubu Earthquake
K-NET MYG011 OSHIKA NS

Figure 1-3 EQ4 compatible to target spectrum



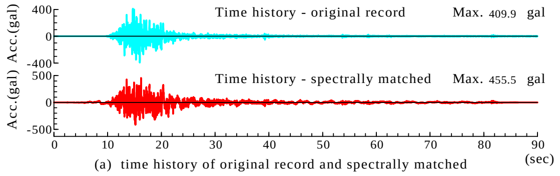
(a) time history of original record and spectrally matched



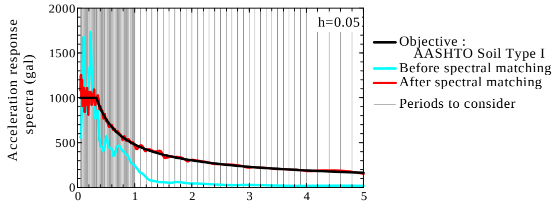
(b) Acceleration response spectra

(1) EQ05 : 2003.09.26 Tokachi Oki Earthquake
K-NET HKD098 TAIKI NS

Figure 1-4 EQ5 compatible to target spectrum



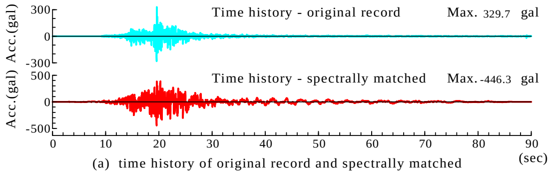
(a) time history of original record and spectrally matched



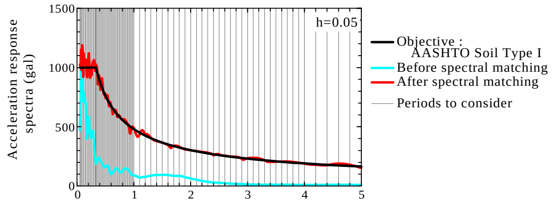
(b) Acceleration response spectra

(2) EQ06 : 2004.10.23 Niigata-ken Chuuetsu Earthquake
KiK-net NIGH12 YUNOTANI NS

Figure 1-5 EQ6 compatible to target spectrum



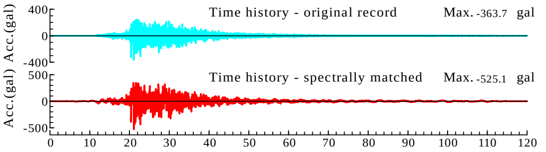
(a) time history of original record and spectrally matched



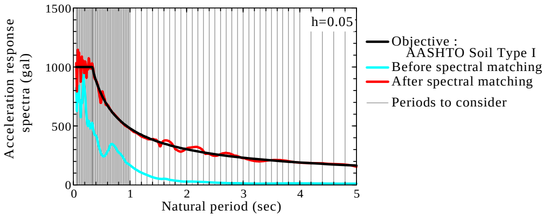
(b) Acceleration response spectra

(1) EQ07 : 2005.03.20 Fukuoka-ken Seiho Oki Earthquake
K-NET SAG001 CHINZEI EW

Figure 1-6 EQ7 compatible to target spectrum



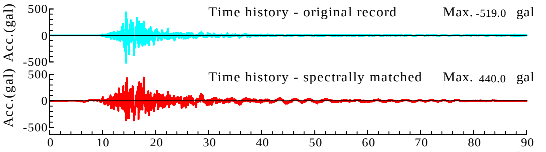
(a) time history of original record and spectrally matched



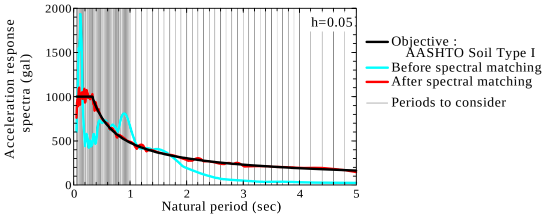
(b) Acceleration response spectra

(2) EQ08 : 2005.08.16 Miyagi-ken Oki Earthquake
K-NET MYG011 OSHIKA EW

Figure 1-7 EQ8 compatible to target spectrum



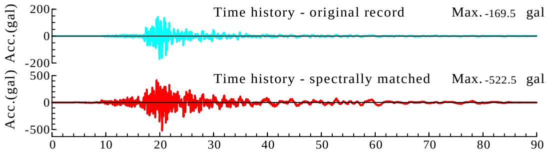
(a) time history of original record and spectrally matched



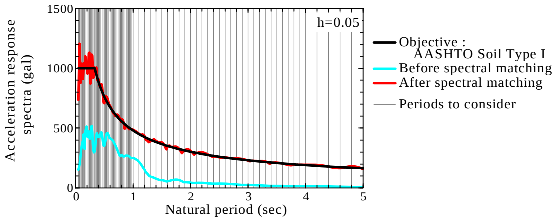
(b) Acceleration response spectra

(1) EQ09 : 2007.03.25 Noto-hanto Earthquake
K-NET ISK003 WAJIMA NS

Figure 1-8 EQ9 compatible to target spectrum



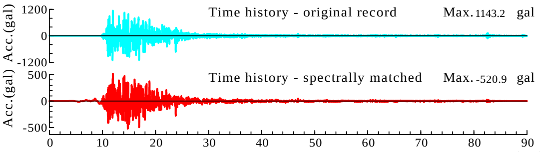
(a) time history of original record and spectrally matched



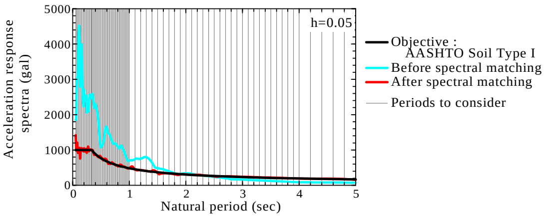
(b) Acceleration response spectra

(2) EQ10 : 2007.07.16 Niigata-ken Chuuetsu Oki Earthquake
KiK-net NIGH13 MAKI EW

Figure 1-9 EQ10 compatible to target spectrum



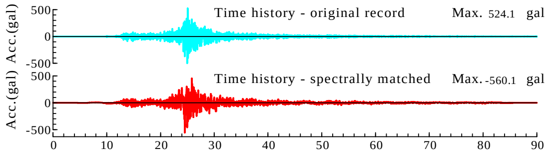
(a) time history of original record and spectrally matched



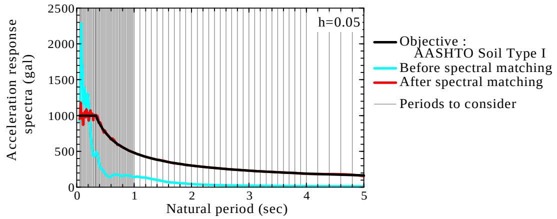
(b) Acceleration response spectra

(1) EQ11 : 2008.06.14 Iwate-Miyagi Nairiku Earthquake
KiK-net IWTH25 ICHINOSEKI-NISHI NS

Figure 1-10 EQ11 compatible to target spectrum



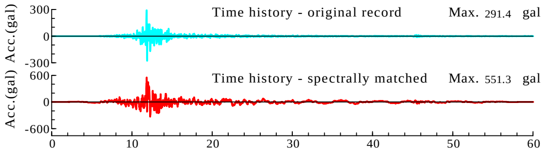
(a) time history of original record and spectrally matched



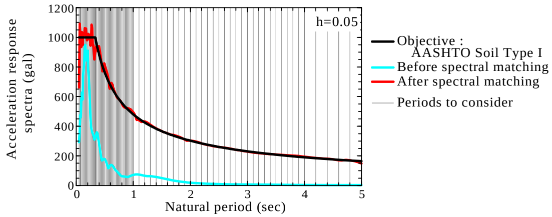
(b) Acceleration response spectra

(2) EQ12 : 2008.07.24 Iwate-ken Engan Hokubu Earthquake
KiK-net IWTH09 KUJI-MINAMI EW

Figure 1-11 EQ12 compatible to target spectrum



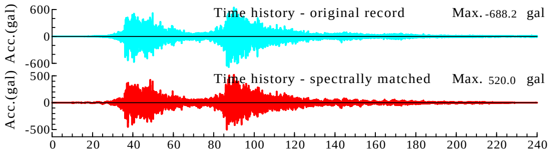
(a) time history of original record and spectrally matched



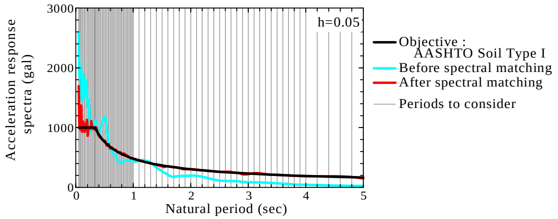
(b) Acceleration response spectra

(1) EQ13 : 2009.08.11 Suruga-wan Earthquake
KiK-net SZOH36 FUJIEDA NS

Figure 1-12 EQ13 compatible to target spectrum



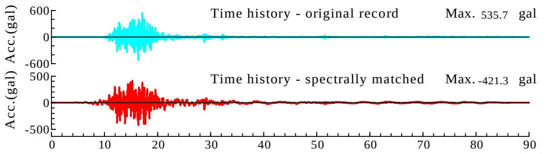
(a) time history of original record and spectrally matched



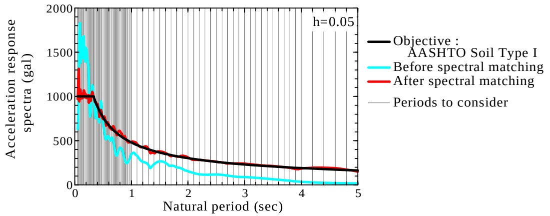
(b) Acceleration response spectra

(2) EQ14 : 2011.03.11 Touhoku-chiho Taiheiyo Oki Earthquake
K-NET MYG011 OSHIKA EW

Figure 1-13 EQ14 compatible to target spectrum



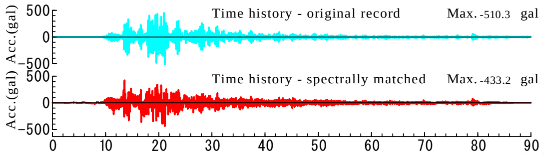
(a) time history of original record and spectrally matched



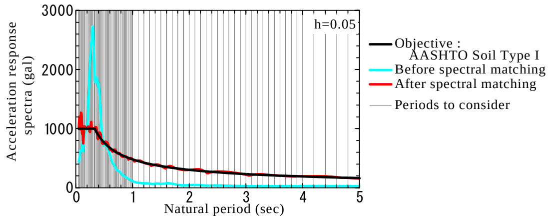
(b) Acceleration response spectra

(1) EQ15 : 2011.03.12 Nagano-Niigata Kenzakai Earthquake
K-NET NIG023 TSUNAN NS

Figure 1-14 EQ15 compatible to target spectrum



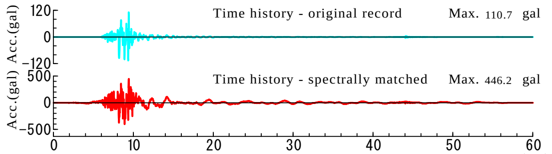
(a) time history of original record and spectrally matched



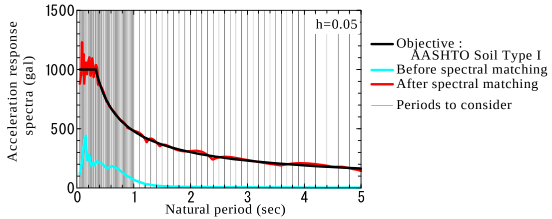
(b) Acceleration response spectra

(1) EQ16 : 2011.04.11 Fukushima-ken Hamadori Earthquake
KiK-net FKSH12 HIRATA NS

Figure 1-15 EQ16 compatible to target spectrum



(a) time history of original record and spectrally matched



(b) Acceleration response spectra

(2) EQ17 : 2011.06.30 Nagano-ken Chuubu Earthquake
K-NET NGN012 MATSUMOTO EW

Figure 1-16 EQ17 compatible to target spectrum

(2) Maximum acceleration, maximum displacement, maximum shear strain and maximum shear stress at different layers

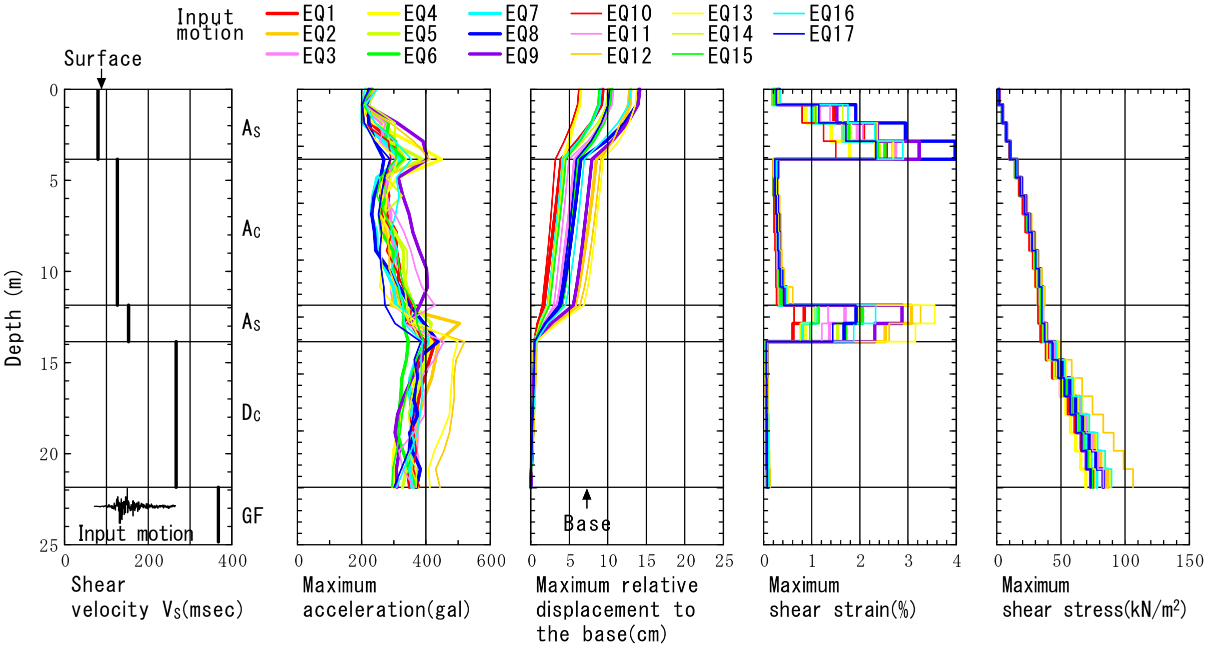


Figure2-1 Maximum acceleration, maximum displacement, maximum shear strain and maximum shear stress at different layers (Soft ground, site No.2)

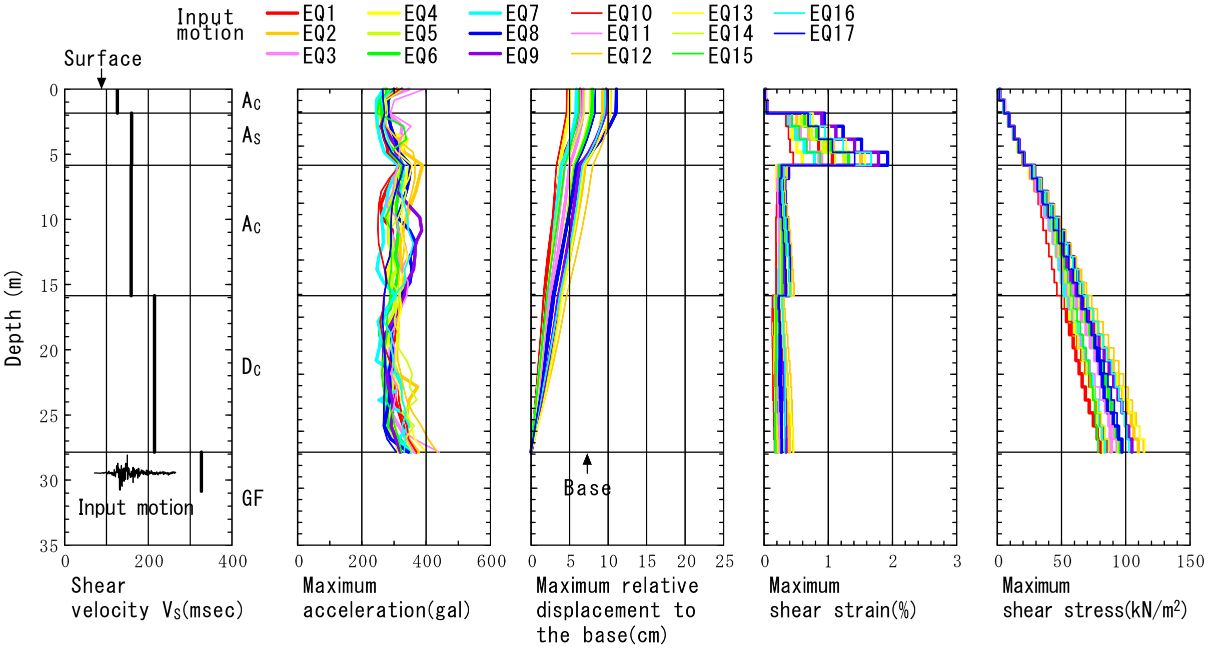


Figure 2-2 Maximum acceleration, maximum displacement, maximum shear strain and maximum shear stress at different layers (Soft ground, site No.3)

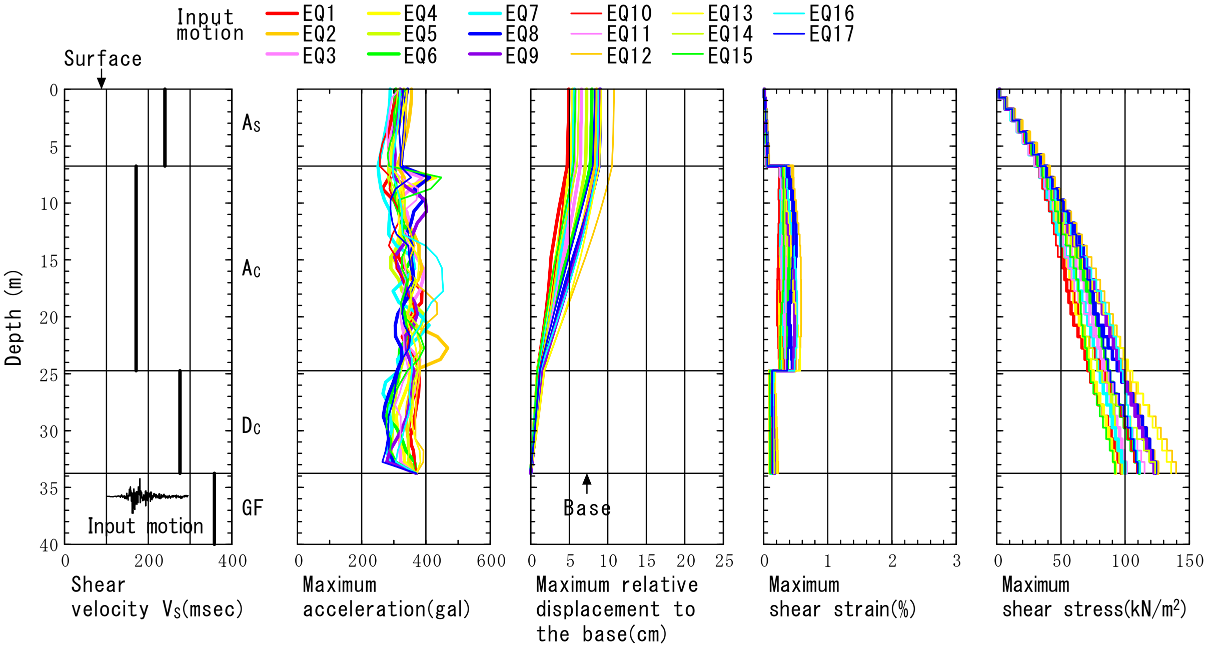


Figure 2-3 **Maximum acceleration, maximum displacement, maximum shear strain and maximum shear stress at different layers (Soft ground, site No.4)**

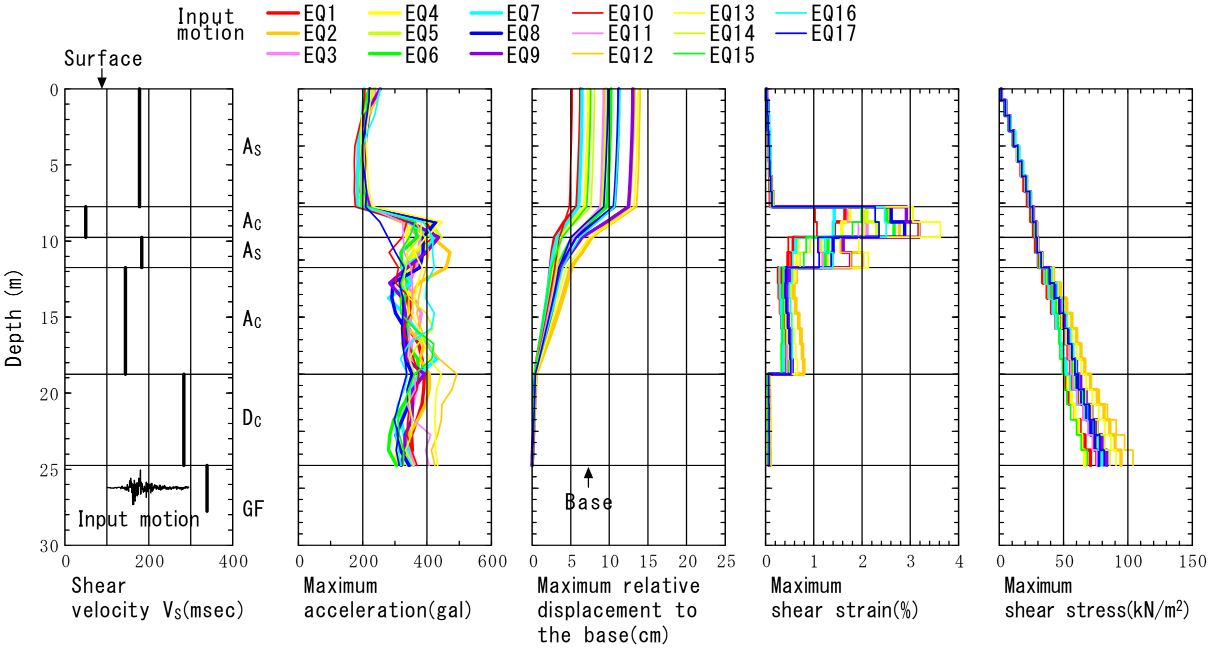


Figure 2-4 **Maximum acceleration, maximum displacement, maximum shear strain and maximum shear stress at different layers (Soft ground, site No.5)**

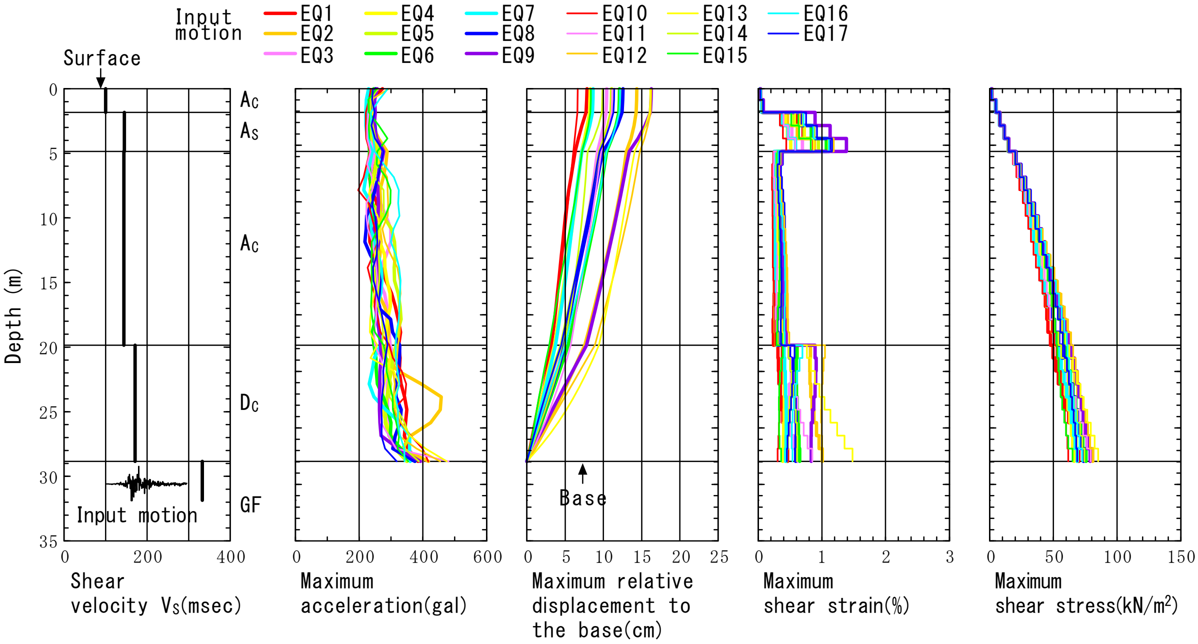


Figure 2-5 Maximum acceleration, maximum displacement, maximum shear strain and maximum shear stress at different layers (Soft ground, site No.6)

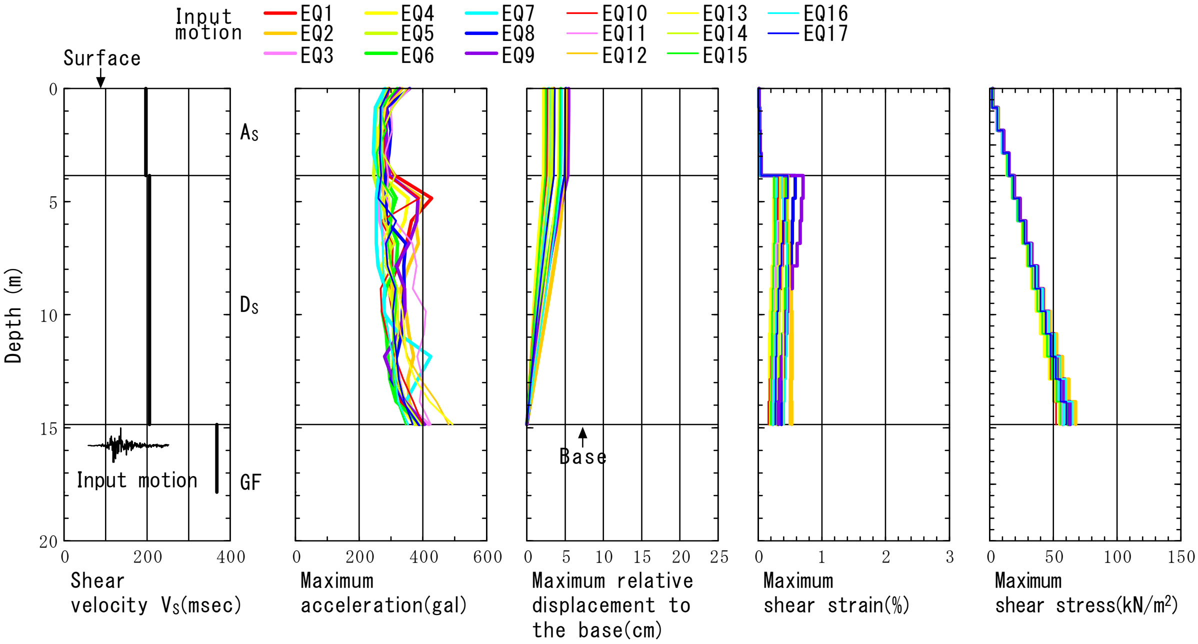


Figure 2-6 Maximum acceleration, maximum displacement, maximum shear strain and maximum shear stress at different layers (Moderate firm ground, site No.2)

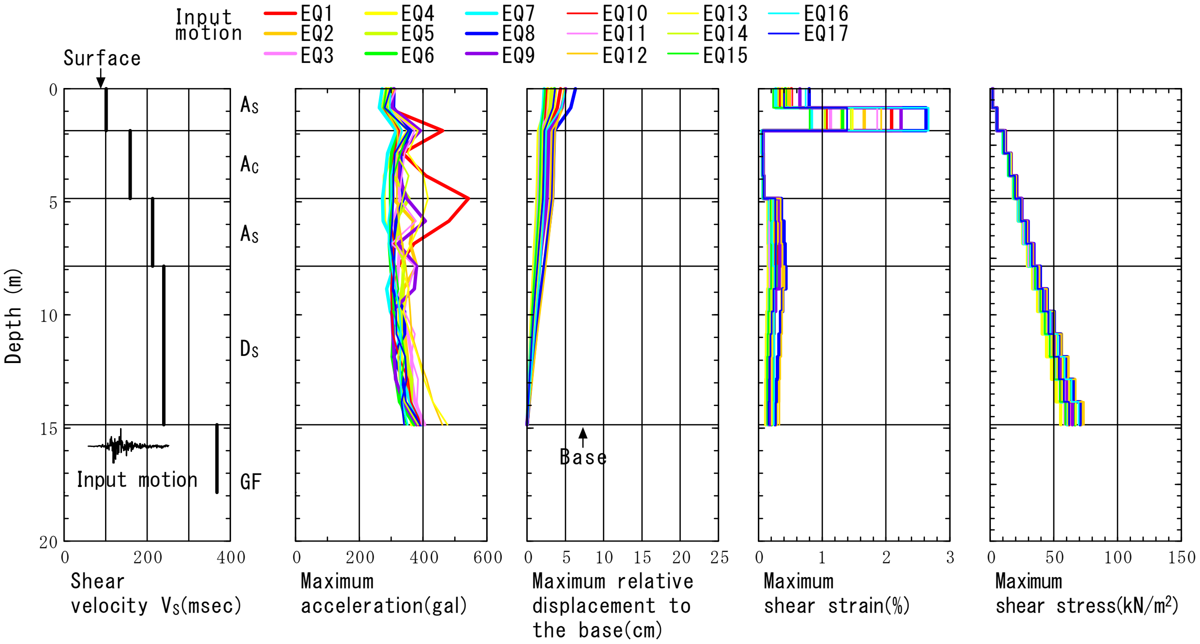


Figure 2-7 Maximum acceleration, maximum displacement, maximum shear strain and maximum shear stress at different layers (Moderate firm ground, site No.3)

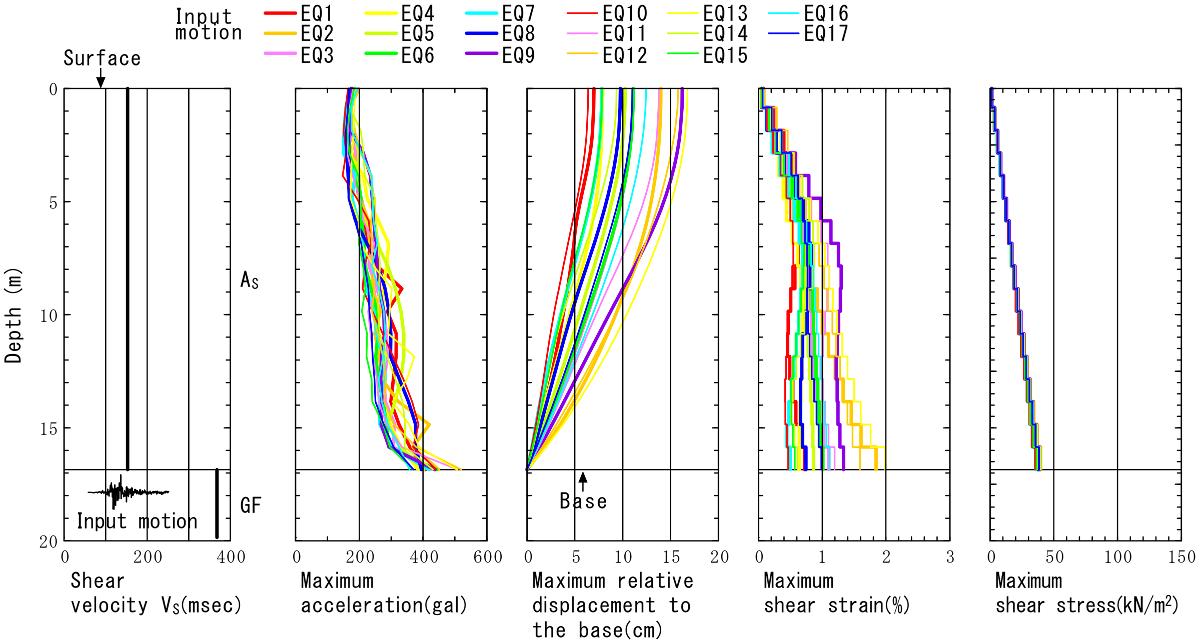


Figure 2-8 Maximum acceleration, maximum displacement, maximum shear strain and maximum shear stress at different layers (Moderate firm ground, site No.4)

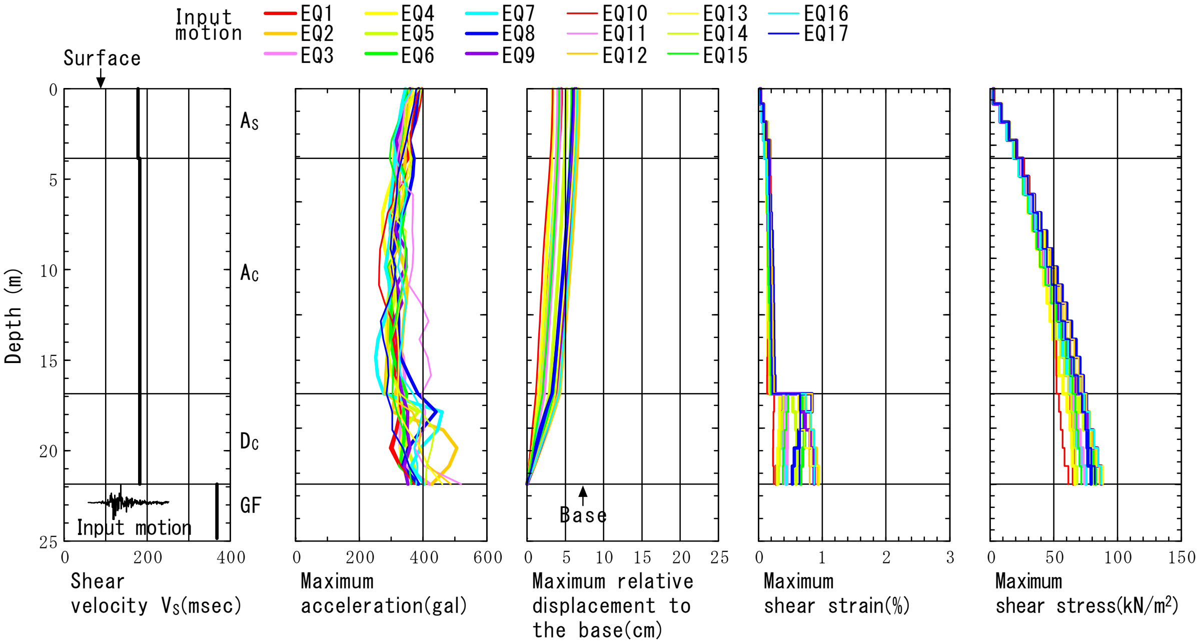


Figure 2-9 Maximum acceleration, maximum displacement, maximum shear strain and maximum shear stress at different layers (Moderate firm ground, site No.5)

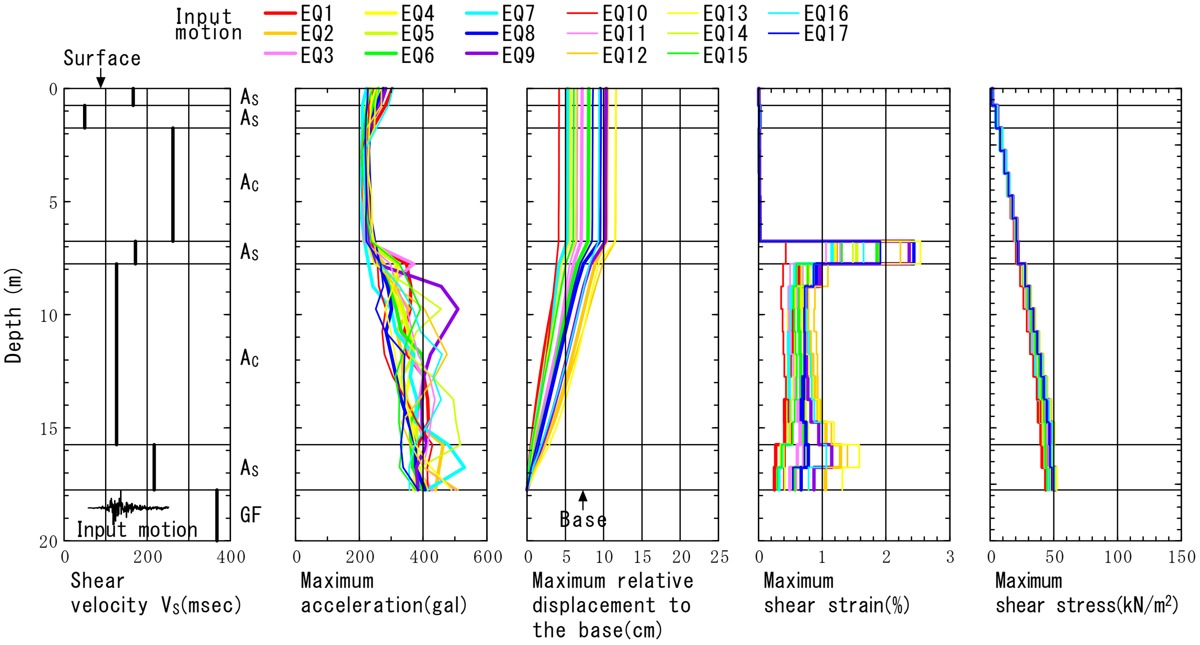


Figure 2-10 Maximum acceleration, maximum displacement, maximum shear strain and maximum shear stress at different layers (Moderate firm ground site No.6)

(3) Spectral amplification factor

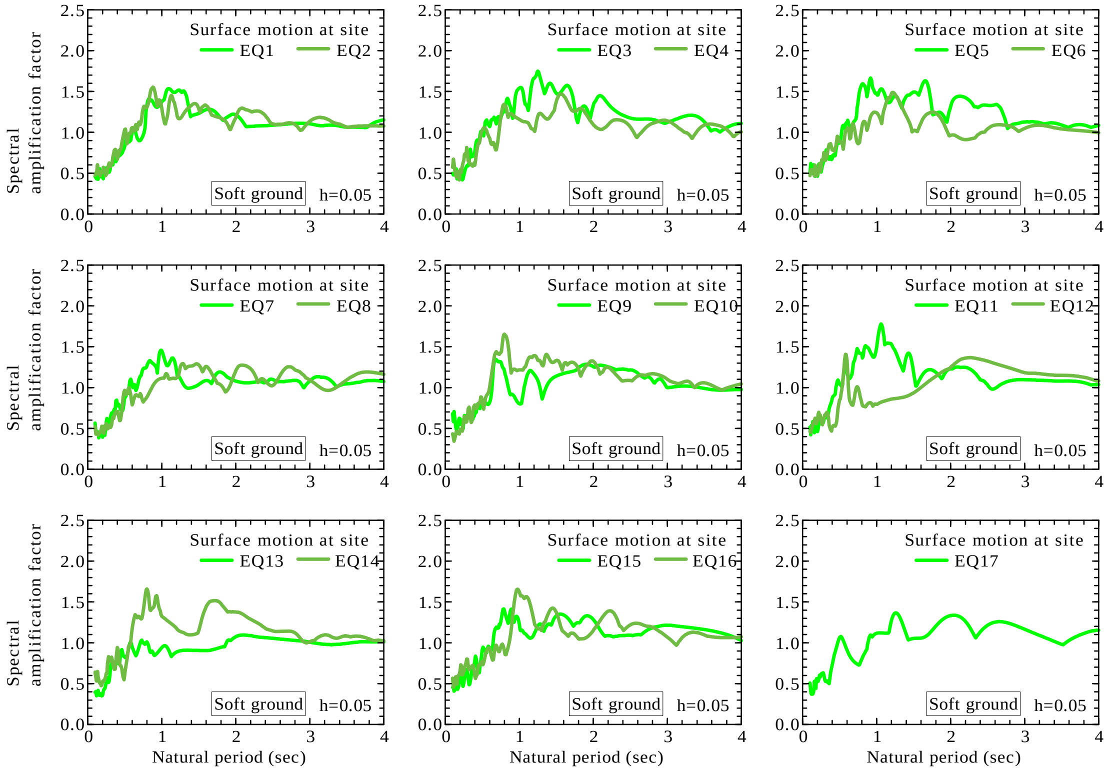


Figure 3-1 Spectral amplification factor (Soft ground: No.1)

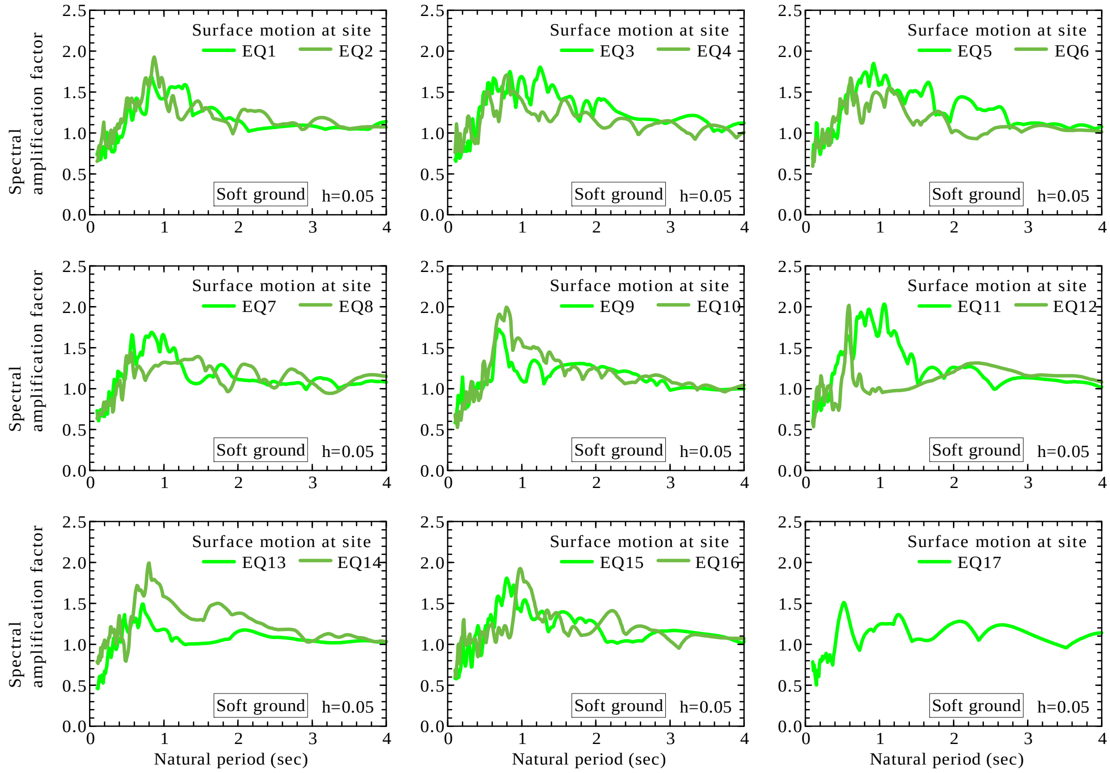


Figure 3-2 Spectral amplification factor (Soft ground: No.2)

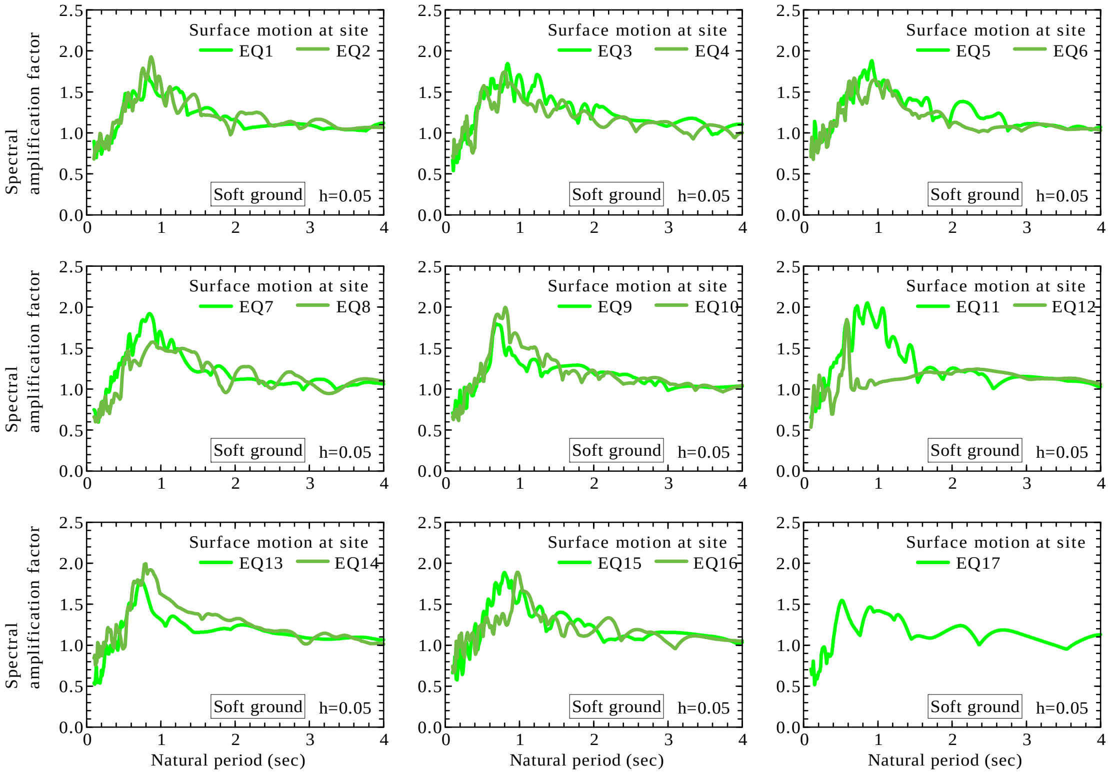


Figure 3-3 Spectral amplification factor (Soft ground: No.3)

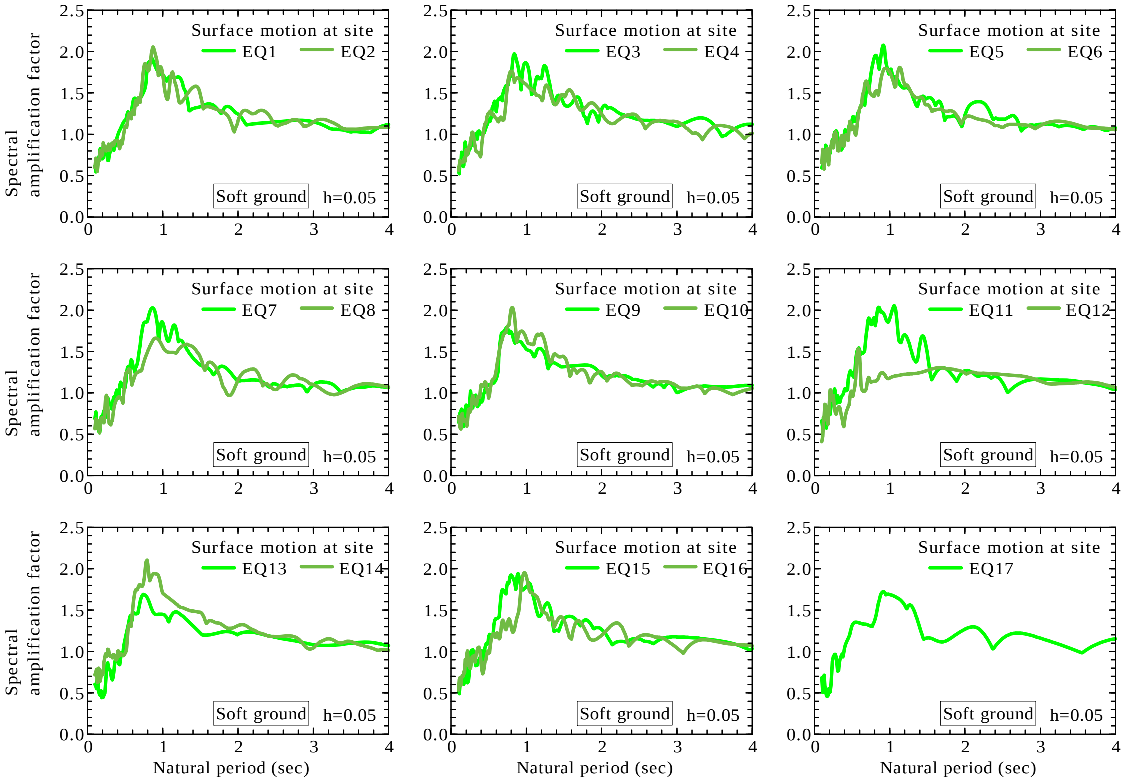


Figure 3-4 Spectral amplification factor (Soft ground: No.4)

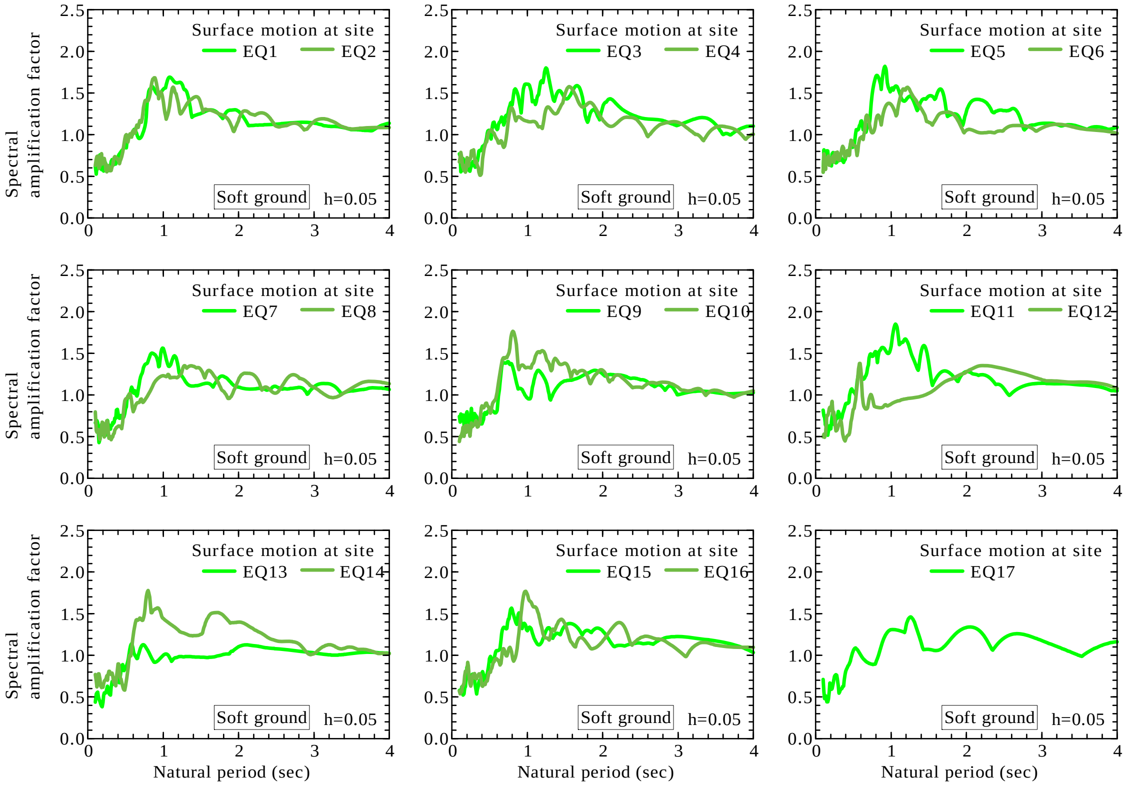


Figure 3-5 Spectral amplification factor (Soft ground: No.5)

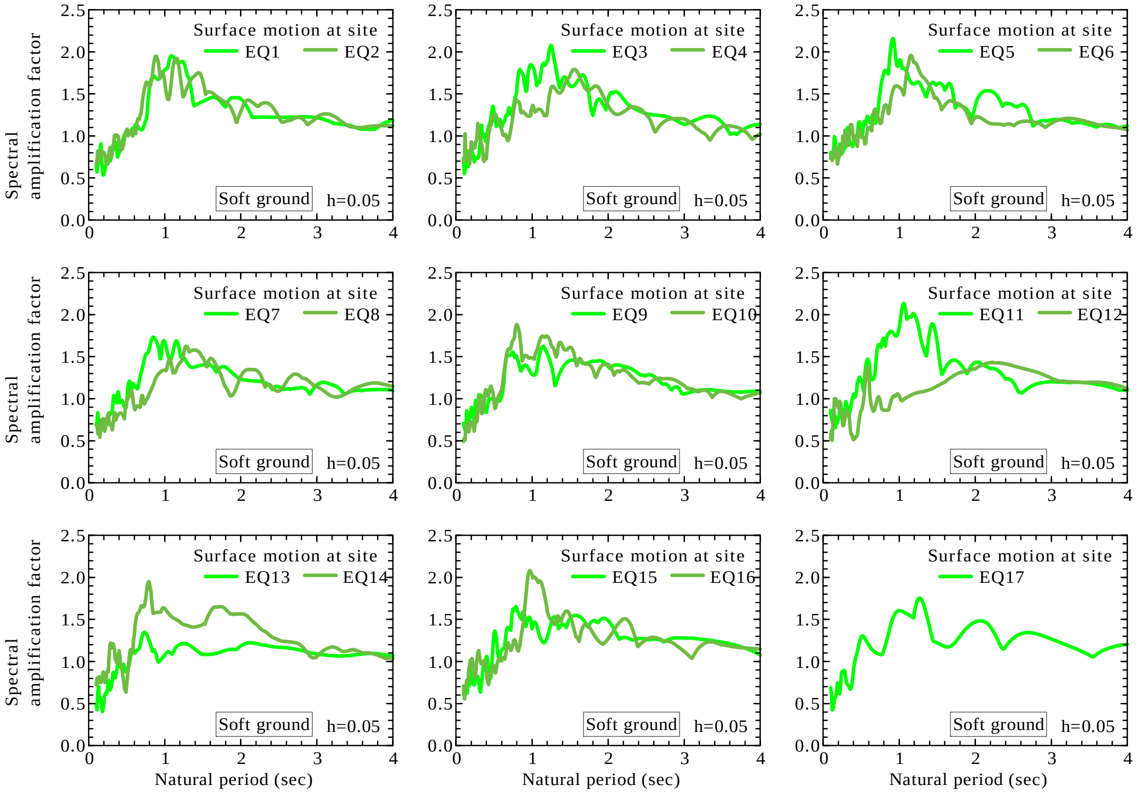


Figure 3-6 Spectral amplification factor (Soft ground: No.6)

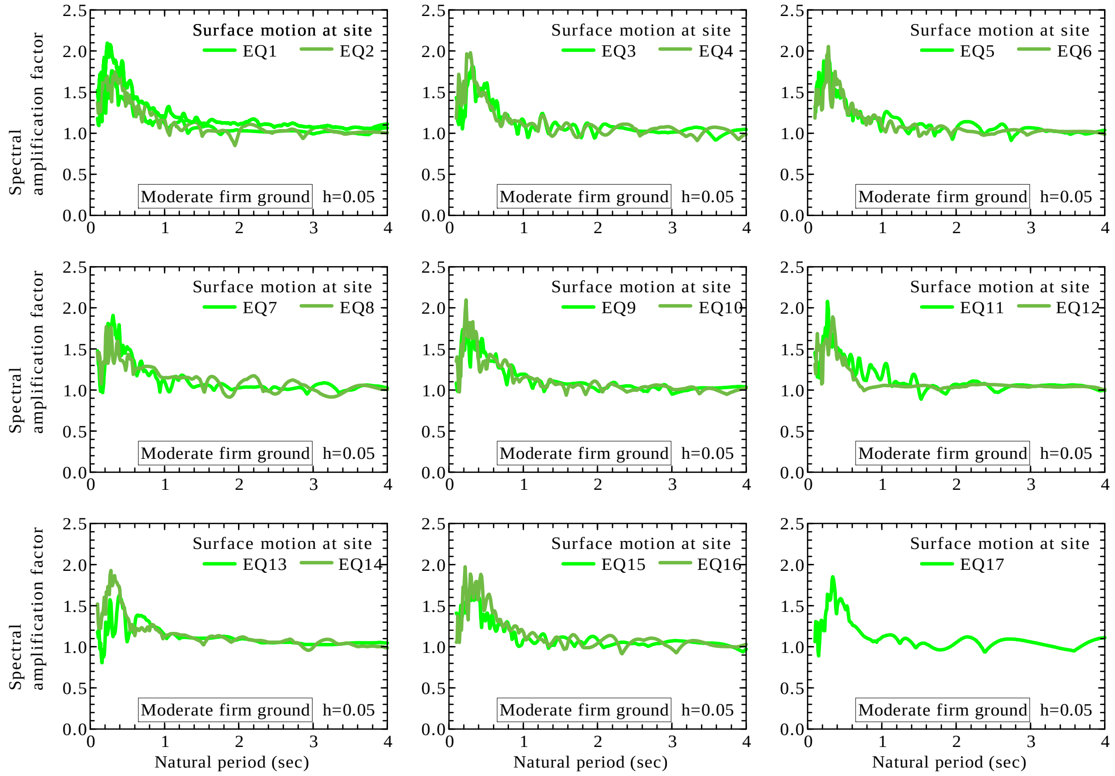


Figure 3-7 Spectral amplification factor (Moderate firm ground: No.1)

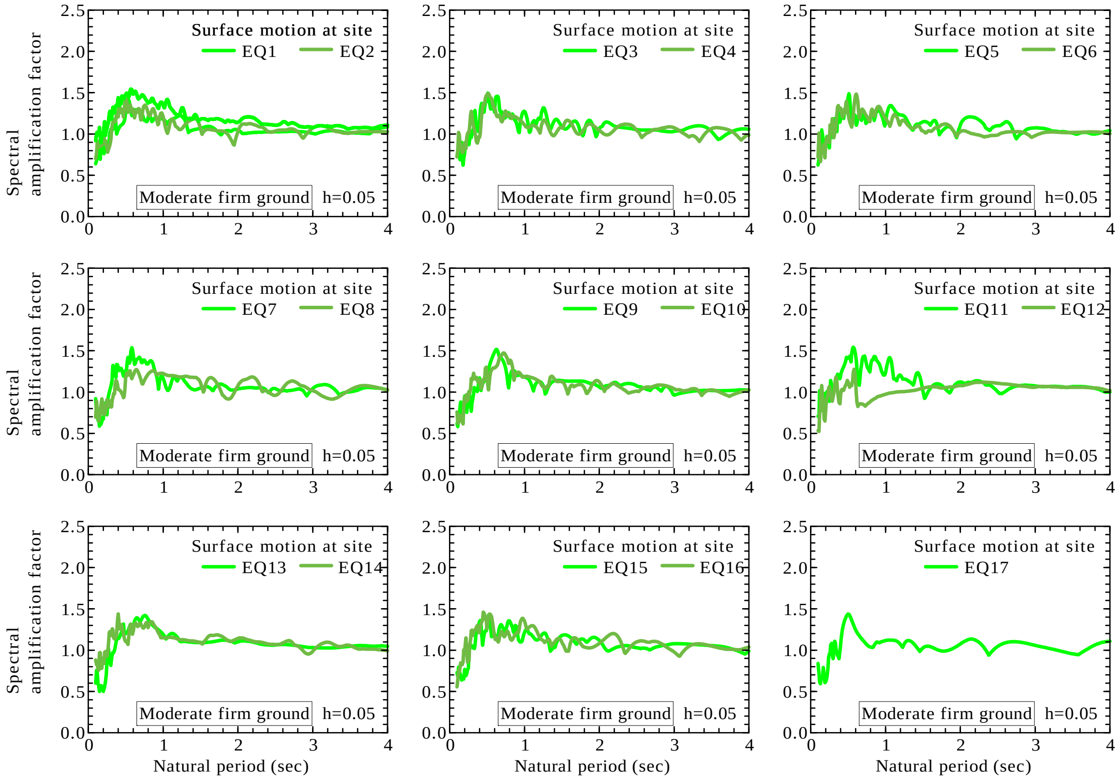


Figure 3-8 Spectral amplification factor (Moderate firm ground: No.2)

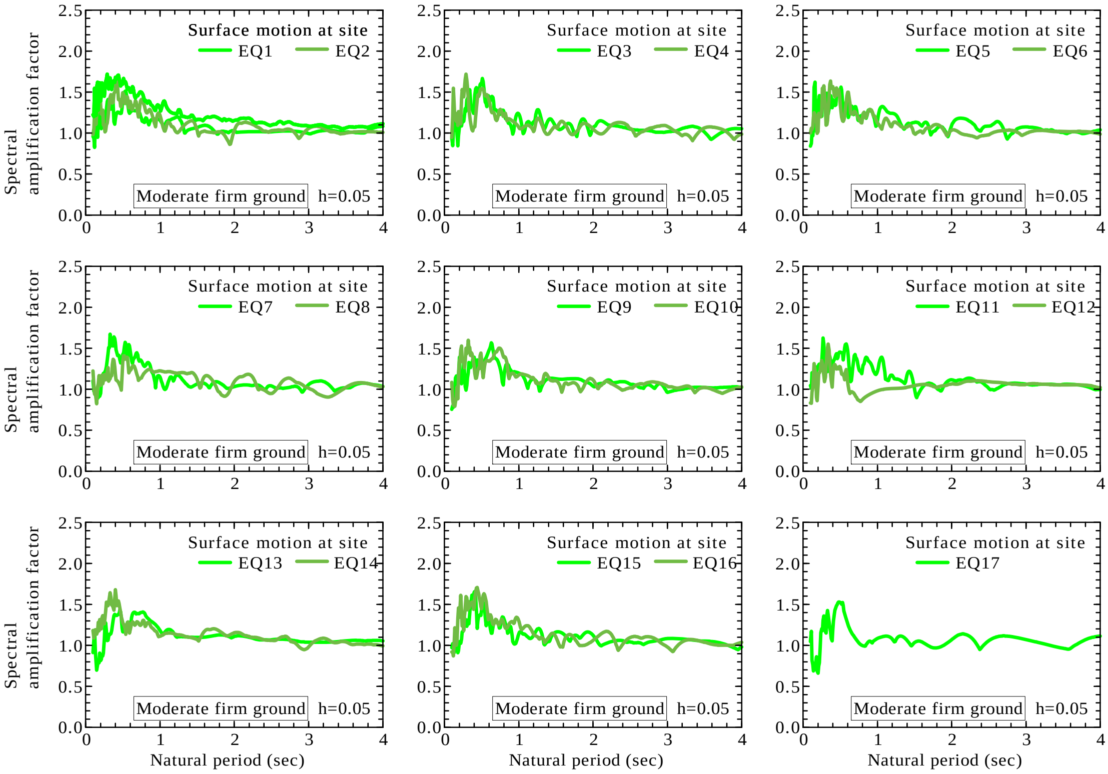


Figure 3-9 Spectral amplification factor (Moderate firm ground: No.3)

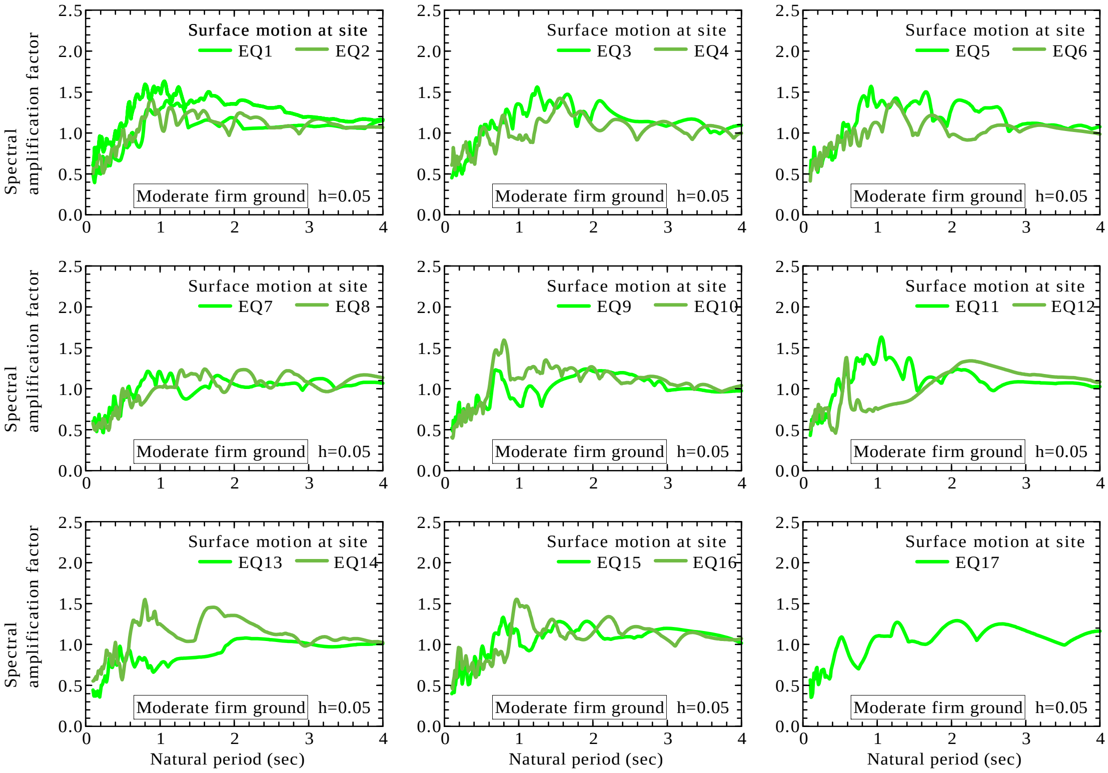


Figure 3-10 Spectral amplification factor (Moderate firm ground: No.4)

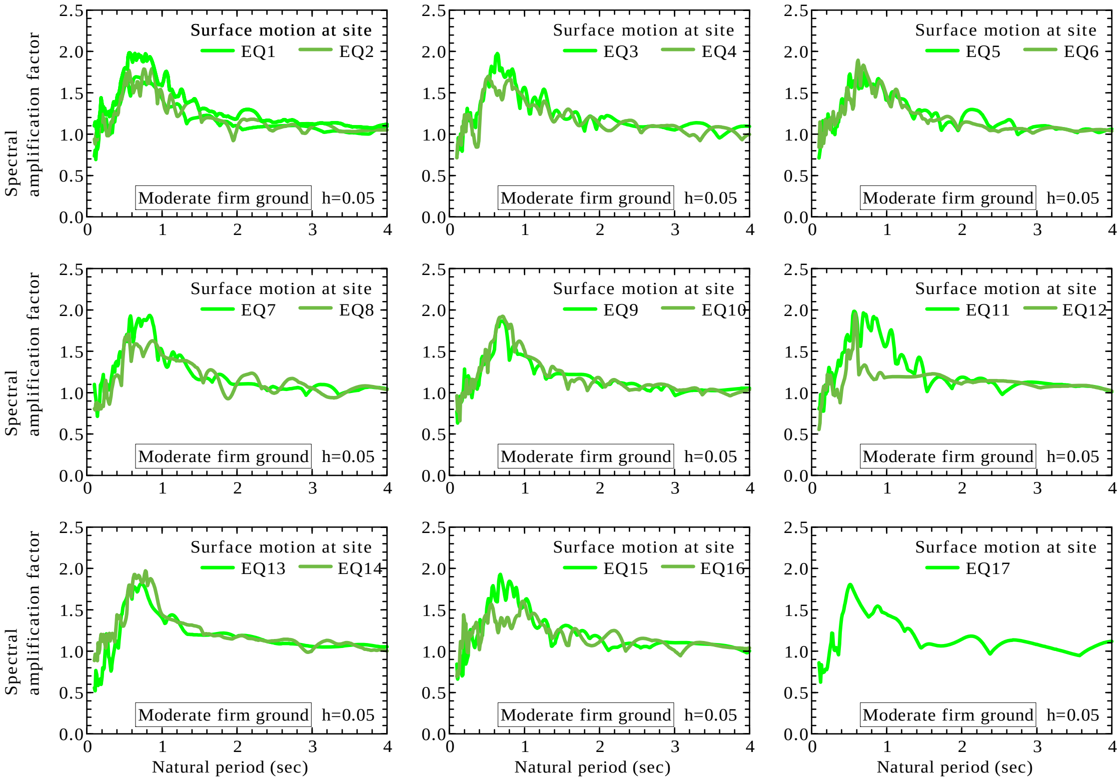


Figure 3-11 Spectral amplification factor (Moderate firm ground: No.5)

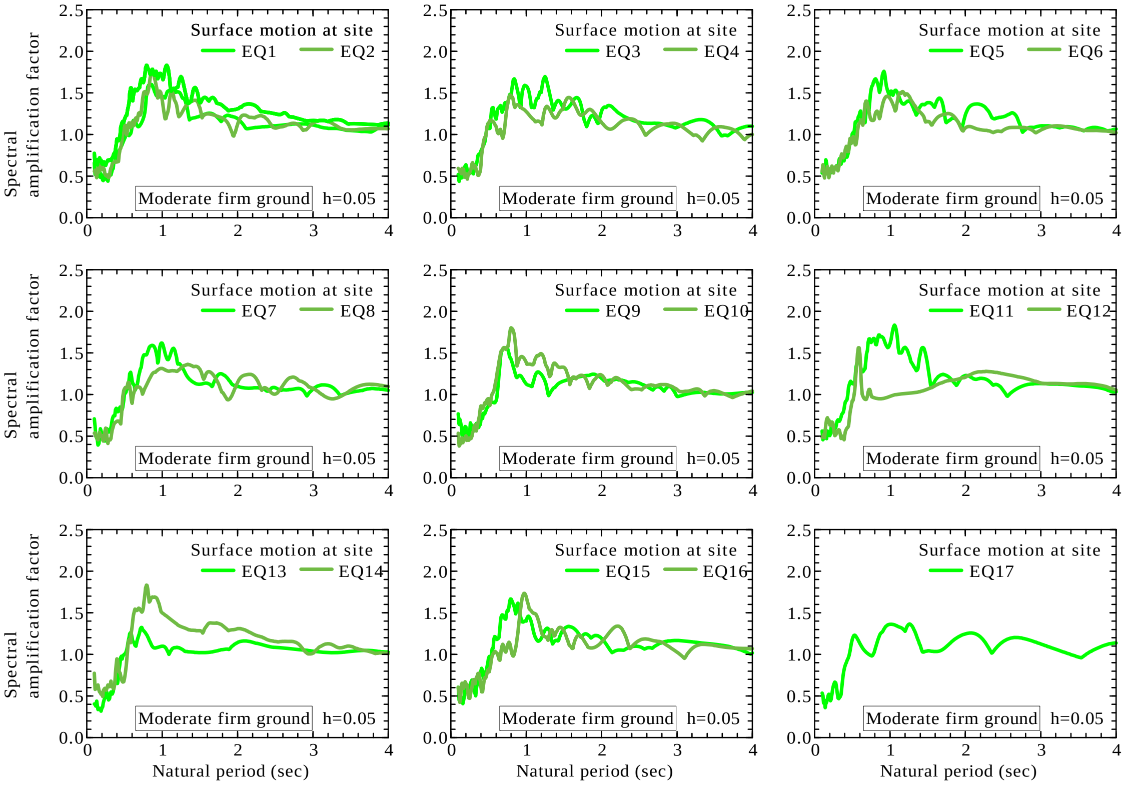


Figure 3-12 Spectral amplification factor (Moderate firm ground: No.6)

(4) Comparison on the Shapes of Acceleration Response Spectra between Analysis Results and AASHTO Specifications

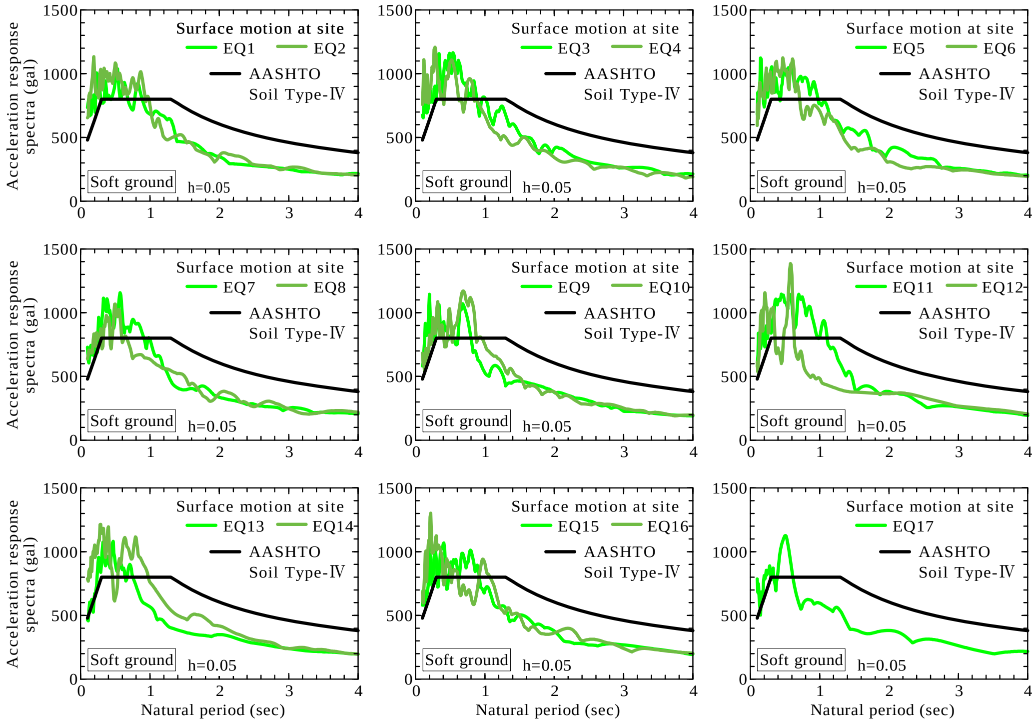


Figure 4-1 Comparison on the Shapes of Acceleration Response Spectra between Analysis Results and AASHTO Specifications (Soft ground: site No.2)

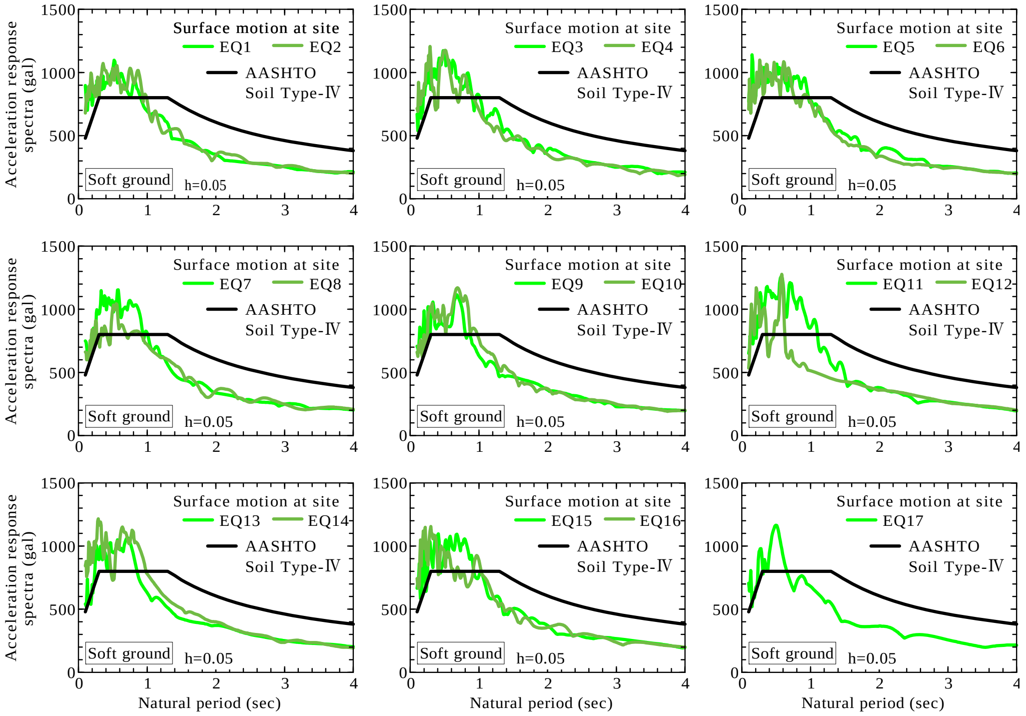


Figure 4-2 Comparison on the Shapes of Acceleration Response Spectra between Analysis Results and AASHTO Specifications (Soft ground: site No.3)

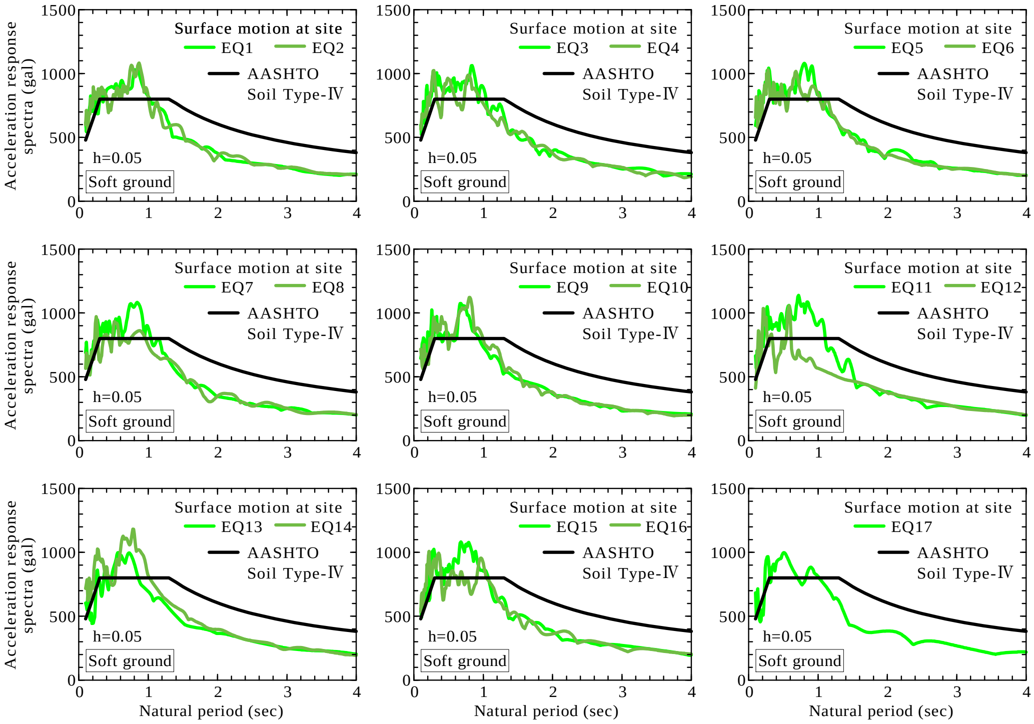


Figure 4-3 Comparison on the Shapes of Acceleration Response Spectra between Analysis Results and AASHTO Specifications (Soft ground: site No.4)

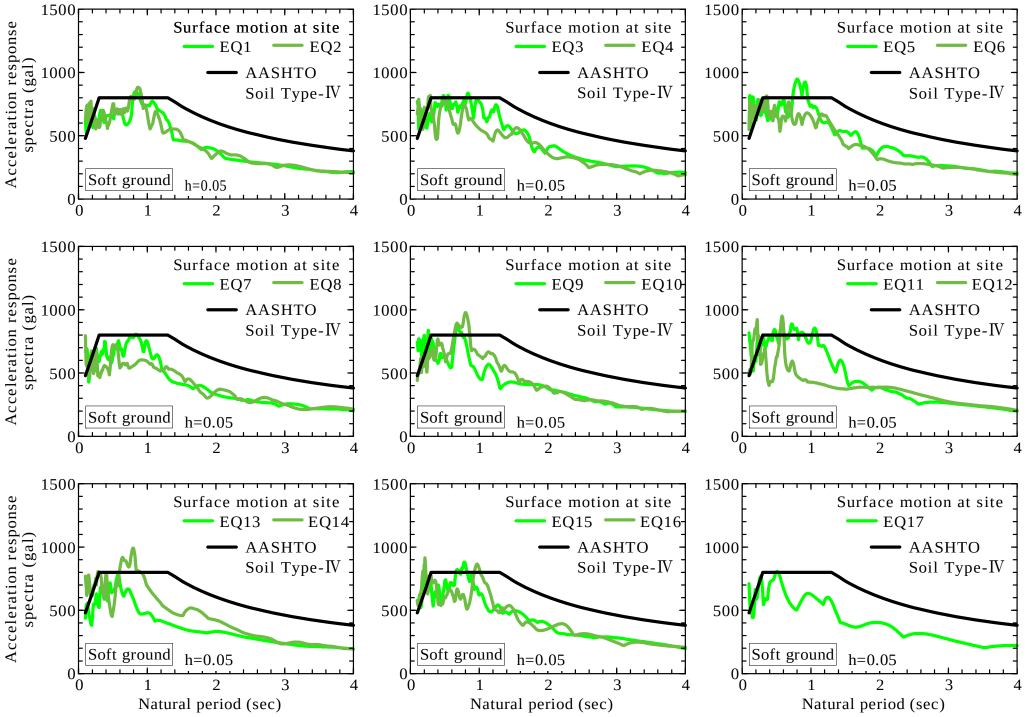


Figure 4-4 Comparison on the Shapes of Acceleration Response Spectra between Analysis Results and AASHTO Specifications (Soft ground: site No.5)

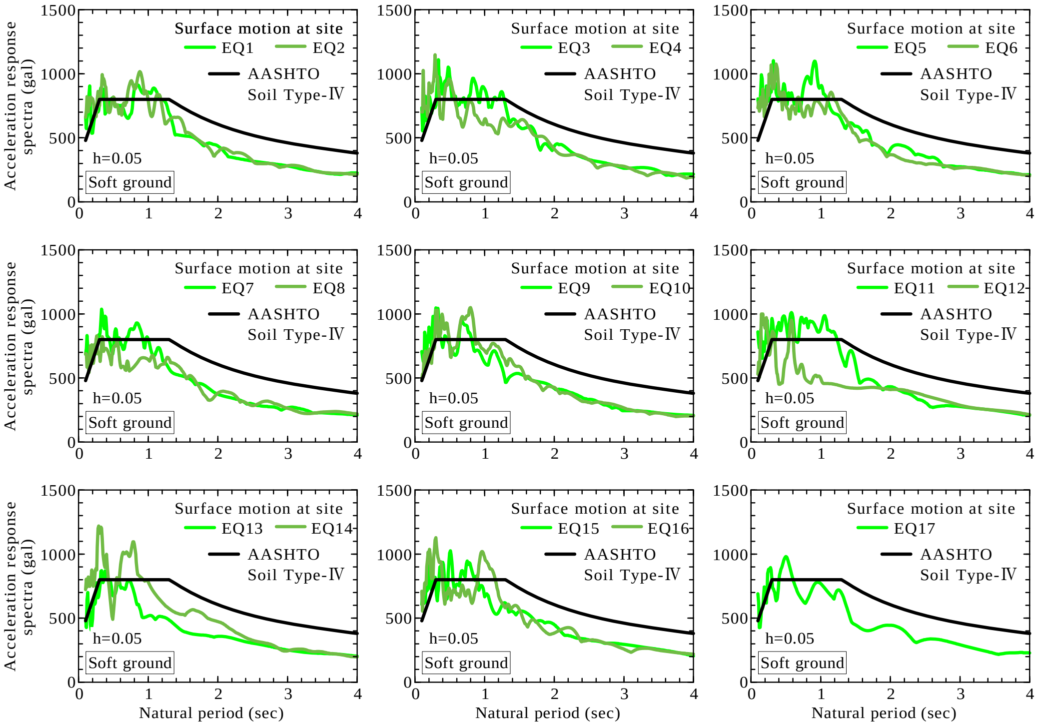


Figure 4-5 Comparison on the Shapes of Acceleration Response Spectra between Analysis Results and AASHTO Specifications (Soft ground: site No.6)

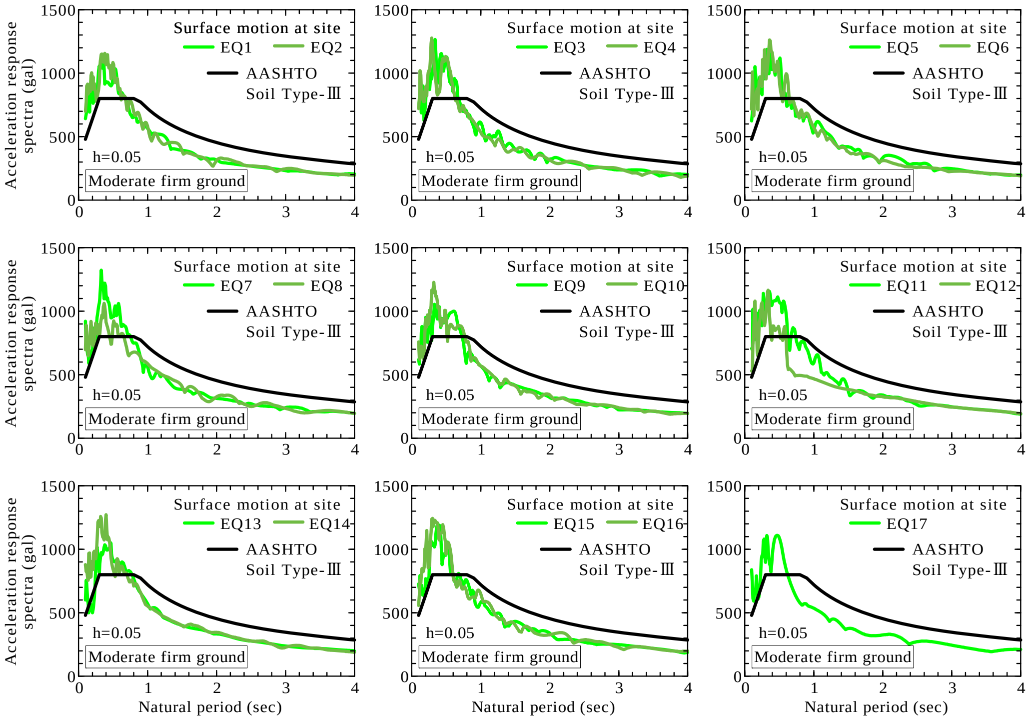


Figure 4-6 Comparison on the Shapes of Acceleration Response Spectra between Analysis Results and AASHTO Specifications (Moderate firm ground: site No.2)

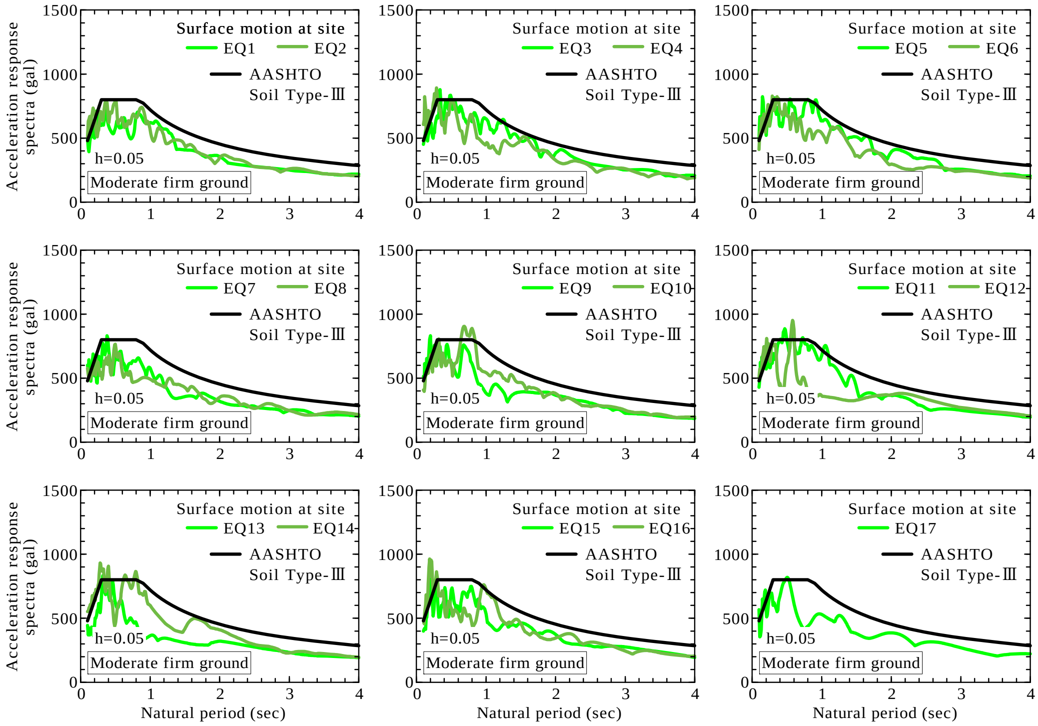


Figure 4-7 Comparison on the Shapes of Acceleration Response Spectra between Analysis Results and AASHTO Specifications (Moderate firm ground: site No.3)

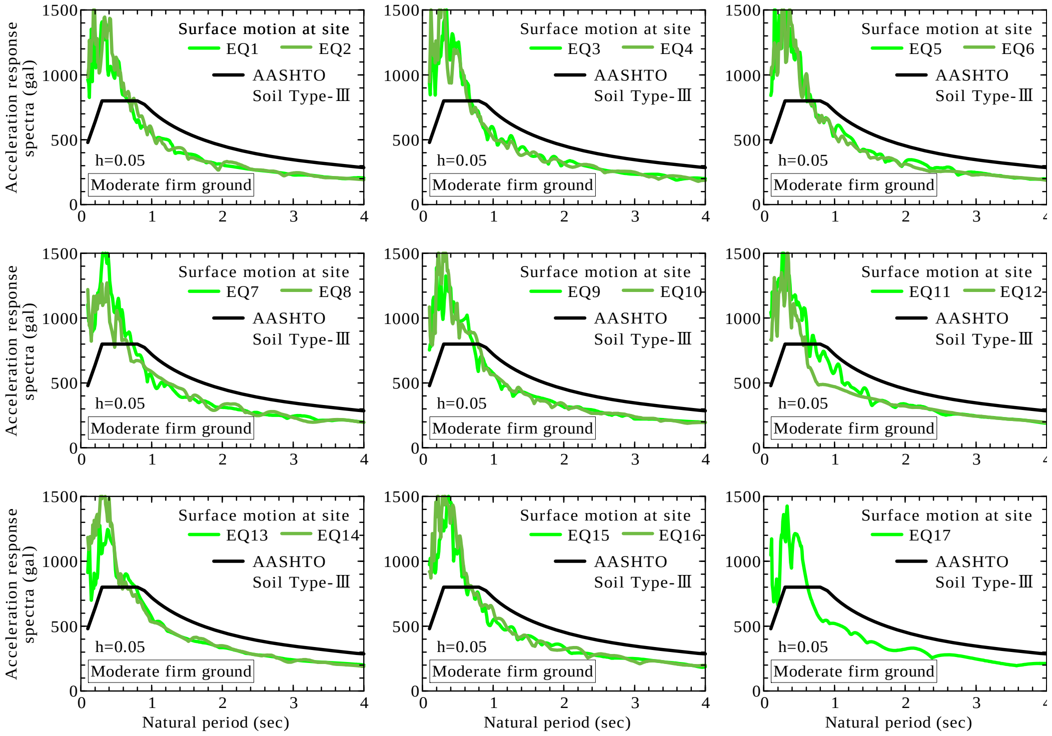


Figure 4-8 Comparison on the Shapes of Acceleration Response Spectra between Analysis Results and AASHTO Specifications (Moderate firm ground: site No.4)

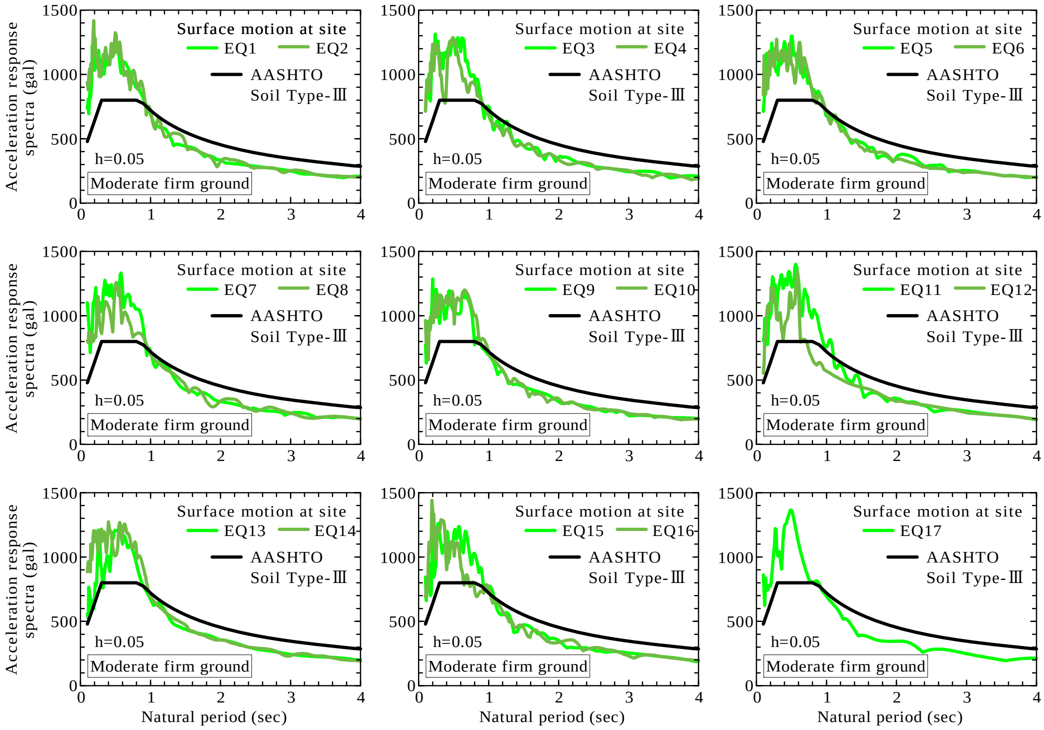


Figure 4-9 Comparison on the Shapes of Acceleration Response Spectra between Analysis Results and AASHTO Specifications (Moderate firm ground: site No.5)

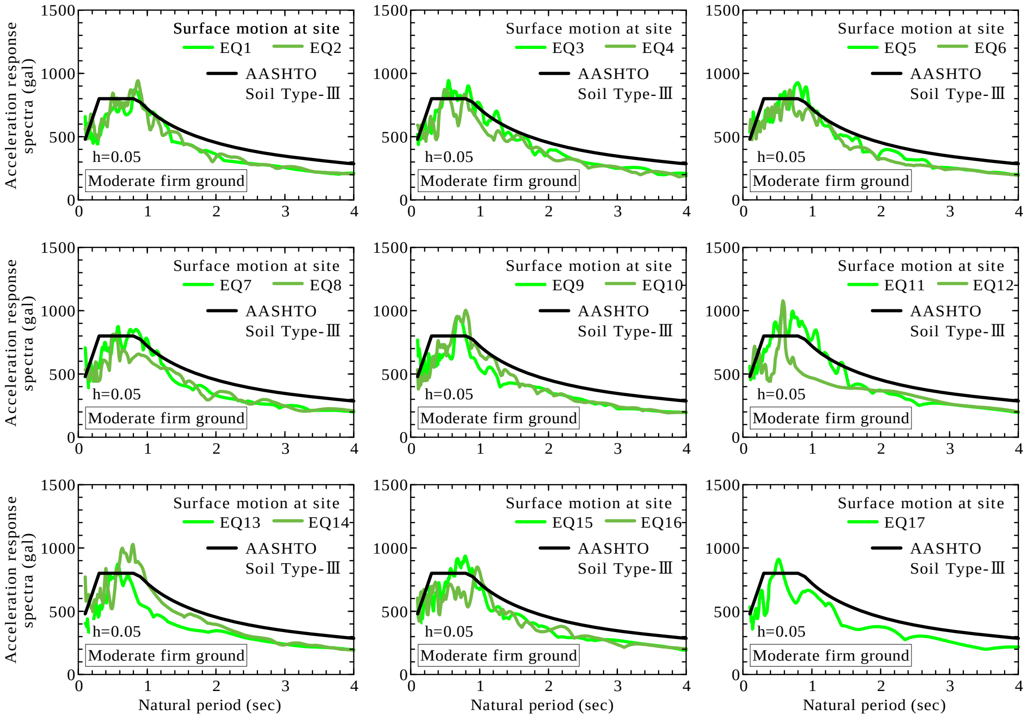


Figure 4-10 Comparison on the Shapes of Acceleration Response Spectra between Analysis Results and AASHTO Specifications (Moderate firm ground: site No.6)

APPENDICES

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1-A PROPOSED DPWH BRIDGES SEISMIC DESIGN SPECIFICATIONS (DPWH-BSDS) (SEPARATE VOLUME)

1-B DESIGN EXAMPLE (NEW BRIDGE) USING DPWH-BSDS (SEPARATE VOLUME)

1-C SEISMIC RETROFIT WORKS EXAMPLE

1-D COMPARISON OF SEISMIC DESIGN SPECIFICATIONS

(1) COMPARISON TABLE OF BRIDGE SEISMIC DESIGN SPECIFICATIONS BETWEEN JRA AND AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS (6TH Ed., 2012)

(2) COMPARISON TABLE OF BRIDGE SEISMIC DESIGN SPECIFICATIONS BETWEEN JRA AND AASHTO GUIDE SPECIFICATIONS LRFD FOR SEISMIC BRIDGE DESIGN (2ND Ed., 2011)

(3) COMPARISON TABLE OF BRIDGE SEISMIC SPECIFICATIONS BETWEEN JRA AND NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005)

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VOLUME 1

SEISMIC DESIGN SPECIFICATIONS

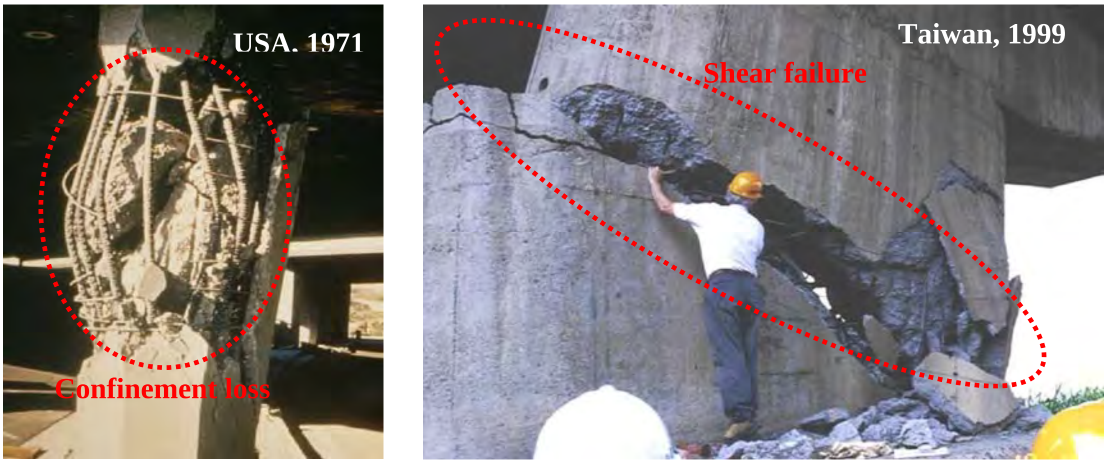
APPENDIX 1-C

SEISMIC RETROFIT WORKS EXAMPLES

CHAPTER 1 SEISMIC LESSONS LEARNED FROM PAST EARTHQUAKES

1.1 Example of Confinement Loss, Shear Failure of Pier Columns/Walls

- Type of damage:
  - Confinement loss
  - Shear failure
- Cause of damage:
  - Lack of transverse hoop reinforcement
  - Lack of shear resistance of columns/wall



(Source: Sixth National Seismic Conference, 2008)

Figure 1.1-1 Confinement Loss, Shear Failure of Columns/Walls

1.2 Example of Flexural Failure of Pier Columns

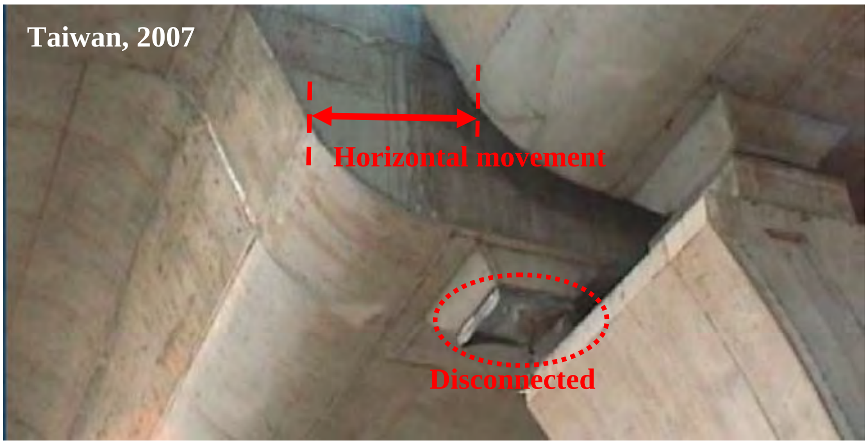
- Type of damage: Flexural failure
- Cause of damage: Lack of flexural resistance of columns/wall

(Source: Sixth National Seismic Conference, 2008)

Figure 1.2-1 Flexural Failure of Columns

1.3 Example of Unseating of Superstructures

- Type of damage: Unseating of superstructure
- Cause of damage:
  - Low seismic capacity of bearings
  - Non-existence of seismic restrainers



(Source: Sixth National Seismic Conference, 2008)

Figure 1.3-1 Flexural Failure of Columns

1.4 Example of Liquefaction-Induced Lateral Spreading

- Type of damage: Liquefaction-induced lateral spreading
- Cause of damage:
  - Low seismic capacity of foundations
  - No countermeasures against liquefaction

(Source: Sixth National Seismic Conference, 2008)

Figure 1.4-1 Flexural Failure of Columns

1.5 Summary of Seismic Vulnerability of Old Bridges & Basic Countermeasures

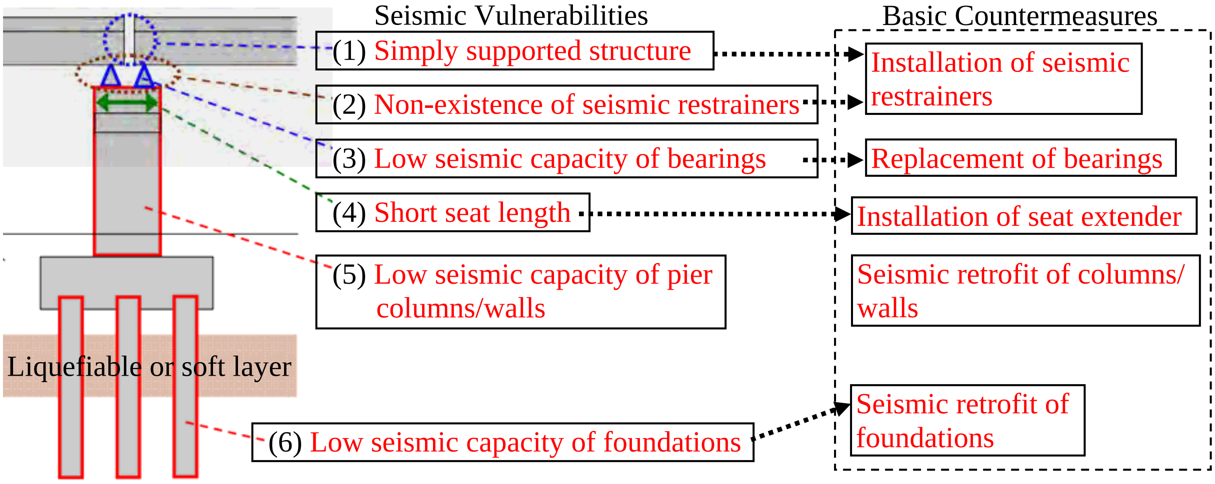
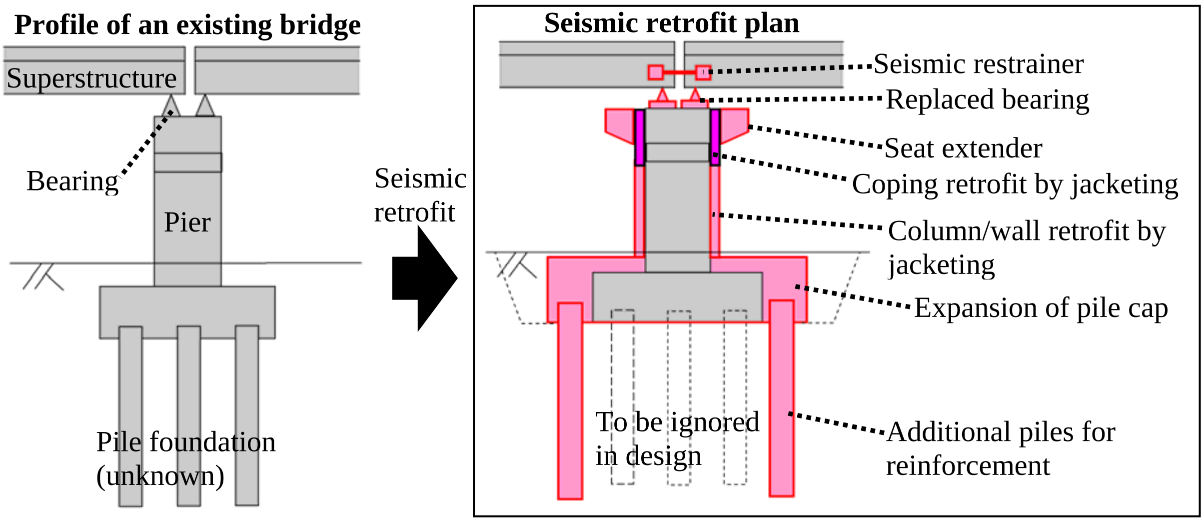


Figure 1.5-1 Summary of Seismic Vulnerability of Old Bridges

CHAPTER 2   Outline of Seismic Retrofit Schemes

2.1   Basic Concept of Seismic Retrofit Planning

Basic Concept of Seismic Retrofit Planning for Piers on Land



Basic Concept of Seismic Retrofit Planning for Piers in Water

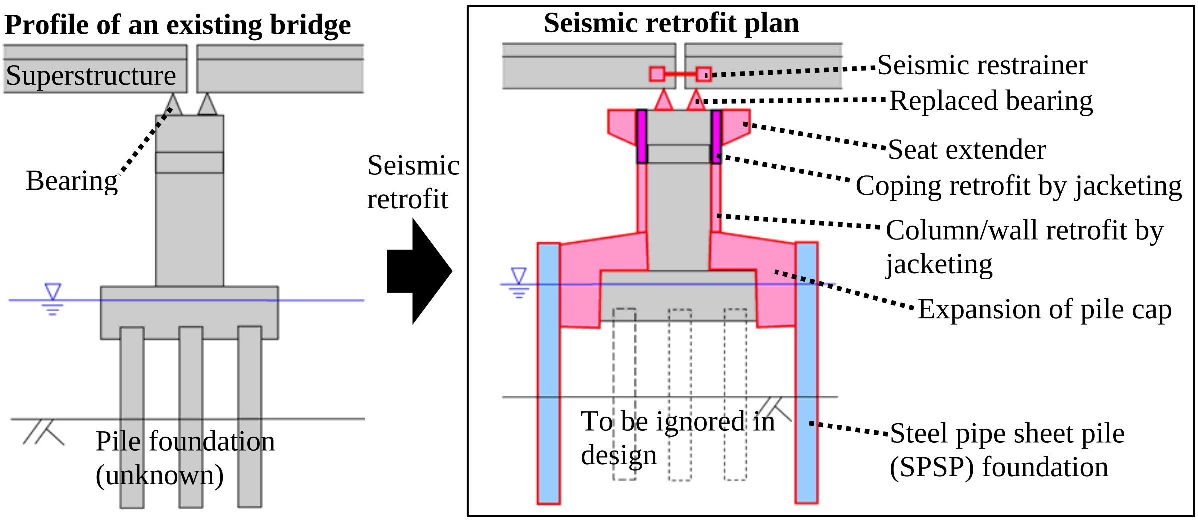


Figure 2.1-1 Basic Concept of Seismic Retrofit Planning

2.2 Additional Options for Seismic Retrofit Planning

2.2.1 Total Replacement of Piers Columns/Walls

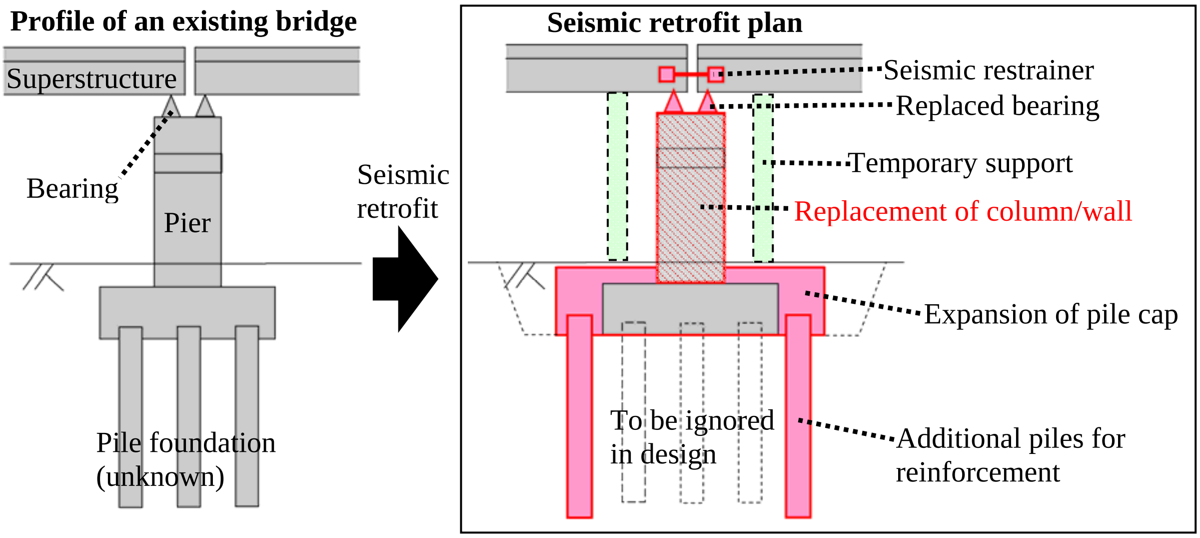


Figure 2.2.1-1 Total Replacement of Piers Columns/Walls

2.2.2 Application of Seismic Devices

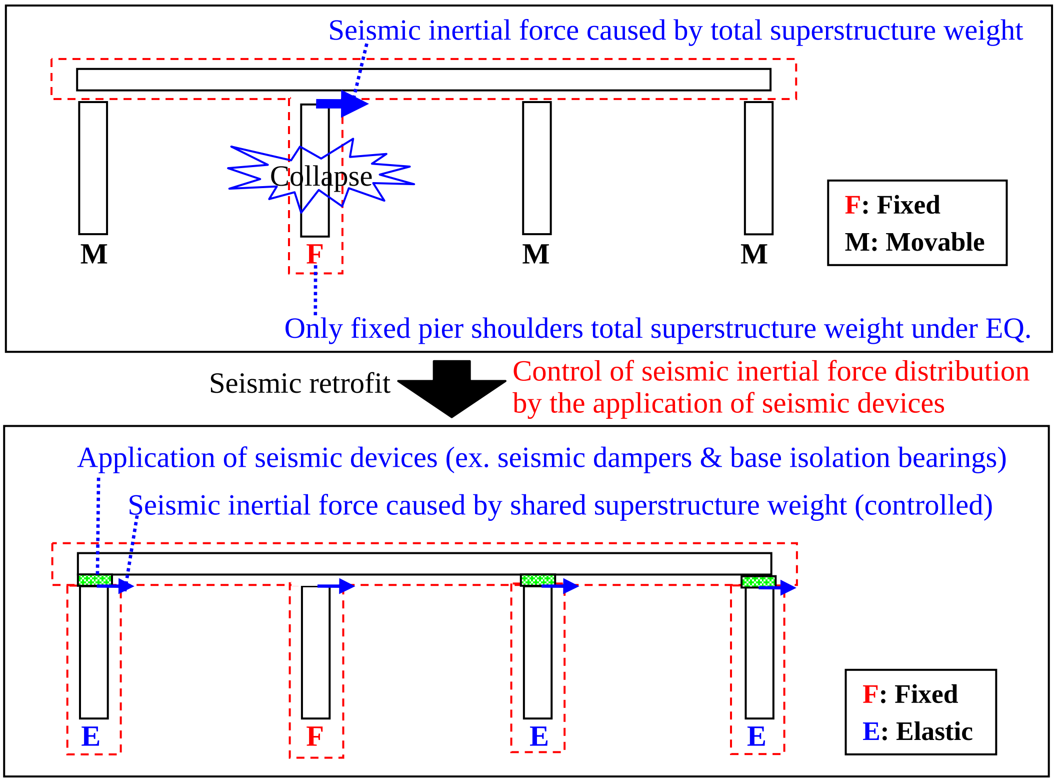
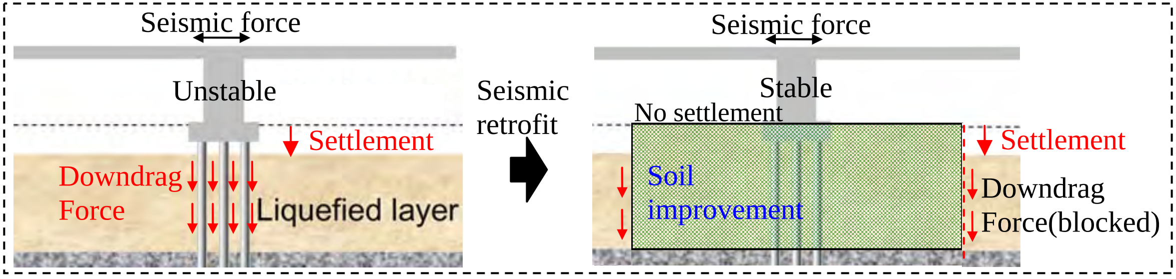


Figure 2.2.2-1 Mechanism of Seismic Device Application

2.2.3 Application of Soil/Ground Improvement

(1) Soil/Ground Improvement for Liquefaction Prevention

[Soil Improvement for Piers]



[Soil Improvement for Abutments]

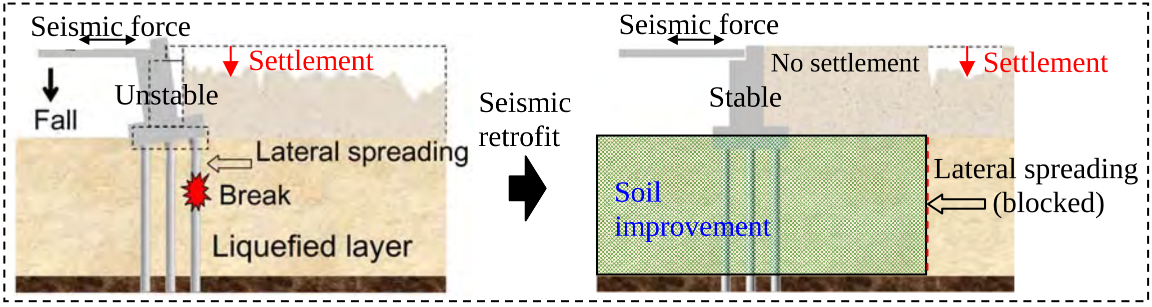


Figure 2.2.3-1 Mechanism of Soil Improvement Application (Liquefaction Prevention)

(2) Soil/Ground Improvement for Earth Pressure Reduction

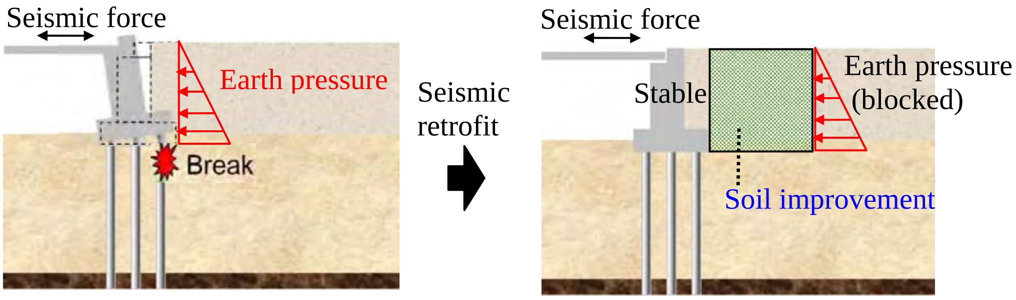


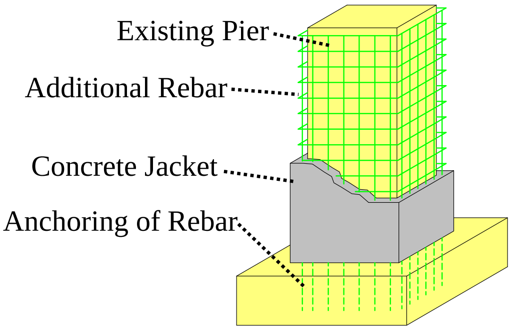
Figure 2.2.3-2 Mechanism of Soil Improvement Application (Earth Pressure Reduction)

CHAPTER 3 SEISMIC RETROFIT OF COLUMNS

3.1 Concrete Jacketing

3.1.1 Outline

- New RC section around piers
- Additional transverse hoop reinforcement for improvement of shear strength and ductility
- Anchoring of additional longitudinal rebar for improvement of flexural strength
- Anchor bolts are to be inserted and grouted with a cement mortar after chipping out of the covering concrete and drilling holes.



(Source: Japan Bridge Association)

Figure 3.1.1-1 Outline of Concrete Jacketing

3.1.2 Construction Step

1. Roughen Surface*
2. Drill Anchor Hole
3. Place Rebar
4. Anchor Rebar
5. Weld Rebar (build up reinforced frame)
6. Set Formwork
7. Pour Concrete
8. Remove Formwork

* includes repair of existing structure s' damages.

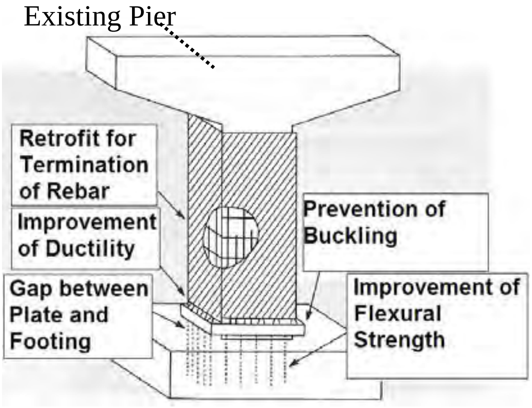
(Source: Japan Bridge Association & Construction Research Institute)

Figure 3.1.2-1 Construction Steps of Concrete Jacketing

3.2 Steel Plate Jacketing

3.2.1 Outline

- Steel plate jacket around piers
- Steel jacket is generally assembled by welding at site.
- The gap is grouted with epoxy resin or pure cement grout
- Improvement of shear strength and ductility depending on a pier shape
- Option for improvement of flexural strength with anchoring of rebar



(Source: Japan Bridge Association)

Figure 3.2.1-1 Outline of Steel Plate Jacketing

3.2.2 Construction Step

1. Measurement
2. Smooth Surface*
3. Fabricate Steel Plates
4. Set Anchors
5. Assemble Plates
6. Weld Plates
7. Inject Mortar between Steel Plate and Existing Structure
8. Anti-corrosion

* includes;
- repair of existing structure s' damages &
- chamfering.

2.

3.

5.

6.

7.

8.

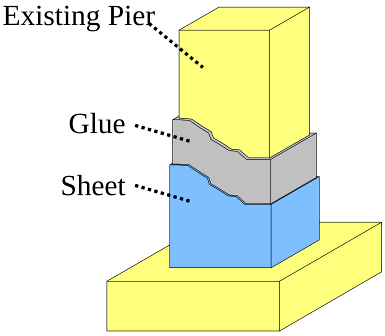
(Source: Japan Bridge Association & Construction Research Institute

Figure 3.2.2-1 Construction Steps of Steel Plate Jacketing

3.3 Material Sheet Jacketing

3.3.1 Outline

- Mostly carbon fiber sheets (carbon strands which are impregnated with resin in the form of sheet)
- Sheets are glued to a pier by resin.
- Enhancing flexural strength needs anchoring of sheets usually combination with RC jacket/lateral steel beams at bottom.



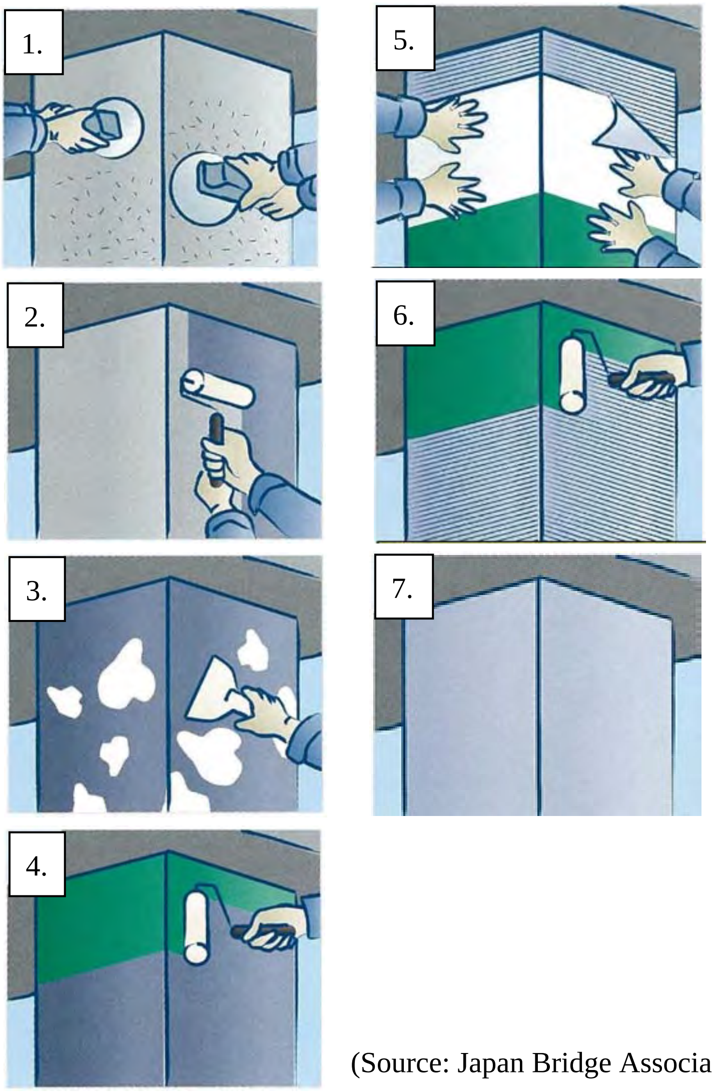
(Source: Japan Bridge Association)

Figure 3.3.1-1 Outline of Material Sheet Jacketing

3.3.2 Construction Step

1. Smooth Surface*
2. Apply Primer
3. Apply Putty
4. Apply Resin
5. Glue Sheets
6. Apply Resin
7. Apply Coating/Mortar

* includes repair of existing structure s' damages.



(Source: Japan Bridge Association)

Figure 3.3.2-1 Construction Steps of Material Sheet Jacketing

3.4 PC Panel Jacketing

3.4.1 Outline

- Encasing of piers in precast concrete panels
- Transversal reinforcement by high-strength PC strands
- Active confinement by spiral post-tensioning strands
- Great improvement of ductility
- Improvement of seismic energy absorption

(Source: Japan Bridge Association)

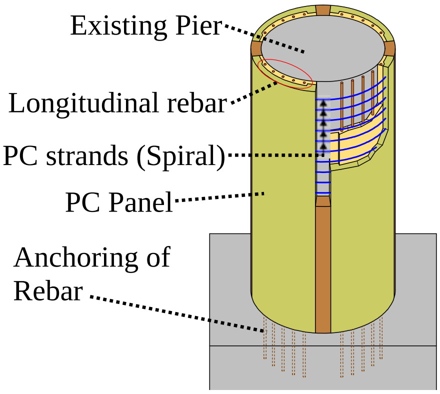


Figure 3.4.1-1 Outline of PC Panel Jacketing

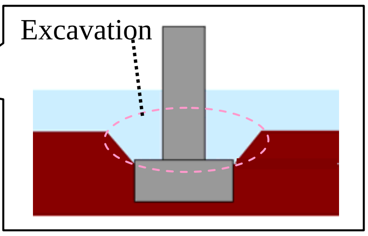
3.4.2 Construction Step (for Piers in Water)

1. Study Riverbed
2. Dig Riverbed
3. Assemble Falsework*
4. Drill holes at Footing
5. Set Longitudinal Rebar
6. Set Precast Panels
7. Pour 1st Concrete

(To be continued)

* includes repair of existing structure s' damages.

2.



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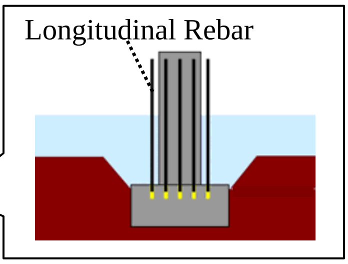

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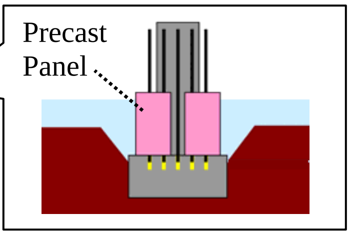

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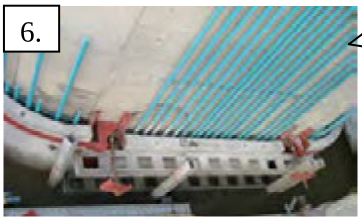

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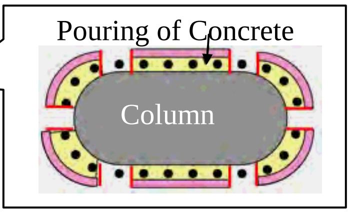
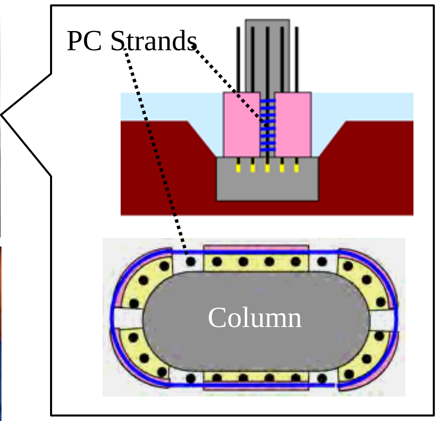


(Source: Japan Bridge Association)

Figure 3.4.2-1 Construction Steps of PC Panel Jacketing (1)

8. Install and tension PC Strands
9. Inject Grout into Panel
10. Pour 2nd Concrete
11. Waterproof Top of Concrete
12. Backfill
13. Finish

8.



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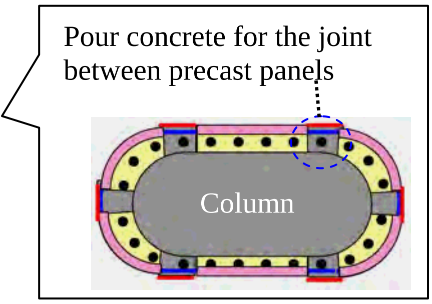

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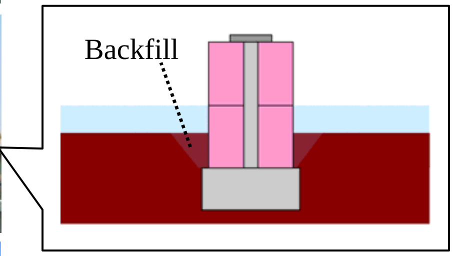


13.

(Source: Japan Bridge Association)

Figure 3.4.2-2 Construction Steps of PC Panel Jacketing (2)

3.5 Comparison of Seismic Retrofit Methods for Columns

3.5.1 Construction on Ground

Table 3.5.1-1 Comparison of Seismic Retrofit Methods for Columns on Ground

		RC Jacket	Steel Plate Jacket	PC Confined	Composite Material
Applicable Shape		Round/Rectangular/Elliptical etc	Round/Rectangular/Elliptical etc	Round/Rectangular/Elliptical etc	Round/Rectangular/Elliptical etc
Effect	Shear	○ Passive confinement	○ Passive confinement	◎ Active confinement	△ Passive confinement
	Ductility				
	Bending		✕ Lateral beam/RC jacket at bottom Required		✕ Lateral beam/RC jacket at bottom Required
on the Ground	Construction	△ Larger space required	○	○	◎ Minimum Space Required
	Construction Period	✕ Longest	△ Long	○ Short	◎ Shortest
	Economical	○	○	△	△
	Durability/ Maintenance	○ Good	△ Need anti-corrosion maintenance	◎ Confinement prevents cracks	△ Resin Deterioaration
	Impact to Environment	○	○	○	○

◎ : Very Good ○ : Good △ : Moderate ✕ : Poor (Source: Japan Bridge Association)

3.5.2 Construction in Water

Table 3.5.2-1 Comparison of Seismic Retrofit Methods for Columns in Water

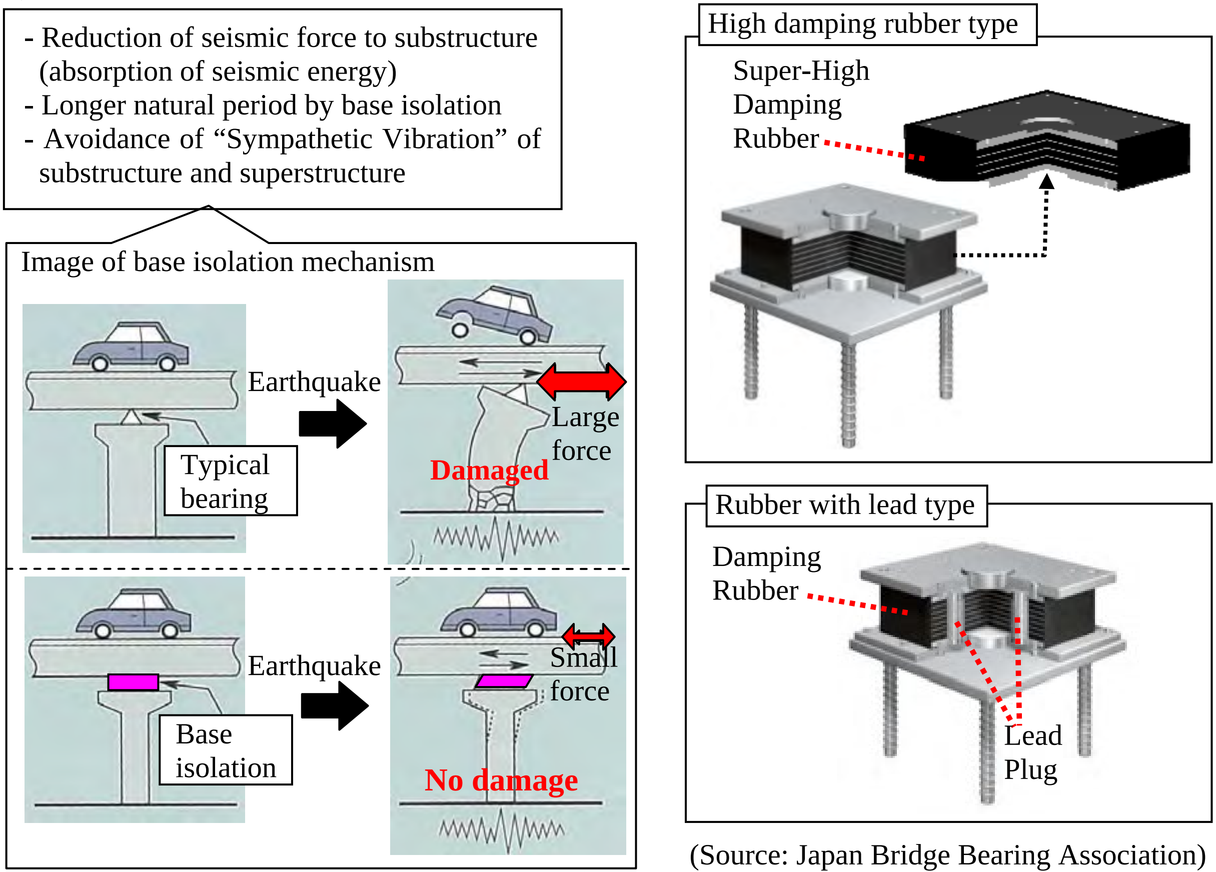
		RC Jacket	Steel Plate Jacket	PC Confined	Composite Material
Applicable Shape		Round/Rectangular/Elliptical etc	Round/Rectangular/Elliptical etc	Round/Rectangular/Elliptical etc	Round/Rectangular/Elliptical etc
Effect	Shear	○ Passive confinement	○ Passive confinement	◎ Active confinement	△ Passive confinement
	Ductility				
	Bending		✕ Lateral beam/RC jacket at bottom Required		✕ Lateral beam/RC jacket at bottom Required
in the Water	Construction	△ Larger space required	○	○	Not Applicable
	Construction Period	✕ Longest	△ Long	◎ Shortest	
	Economical	△ Temporary Cofferdams	△ Temporary Cofferdams	○ Underwater works by divers	
	Durability/ Maintenance	△ Possible Shrinkage Cracks	✕ Anti-corrosion Required	○ High resistance to cracks	
	Impact to Environment	✕ Temporarry Cofferdams /Surface Preparation	△ Temporarry Cofferdams	○ No cofferdams required	

◎ : Very Good ○ : Good △ : Moderate ✕ : Poor (Source: Japan Bridge Association)

CHAPTER 4　SEISMIC DEVICES

4.1　Base Isolation Bearings

4.1.1　Outline

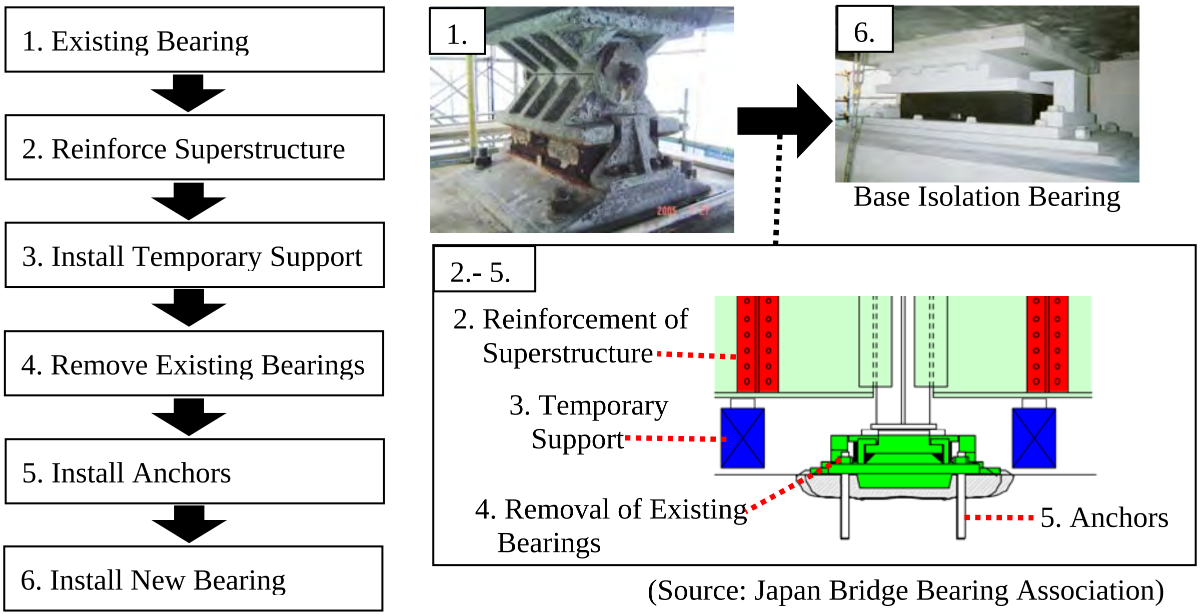


- Reduction of seismic force to substructure (absorption of seismic energy)
- Longer natural period by base isolation
- Avoidance of “Sympathetic Vibration” of substructure and superstructure

(Source: Japan Bridge Bearing Association)

Figure 4.1.1-1 Outline of Base Isolation Bearings

4.1.2　Construction Step (Seismic Retrofit)



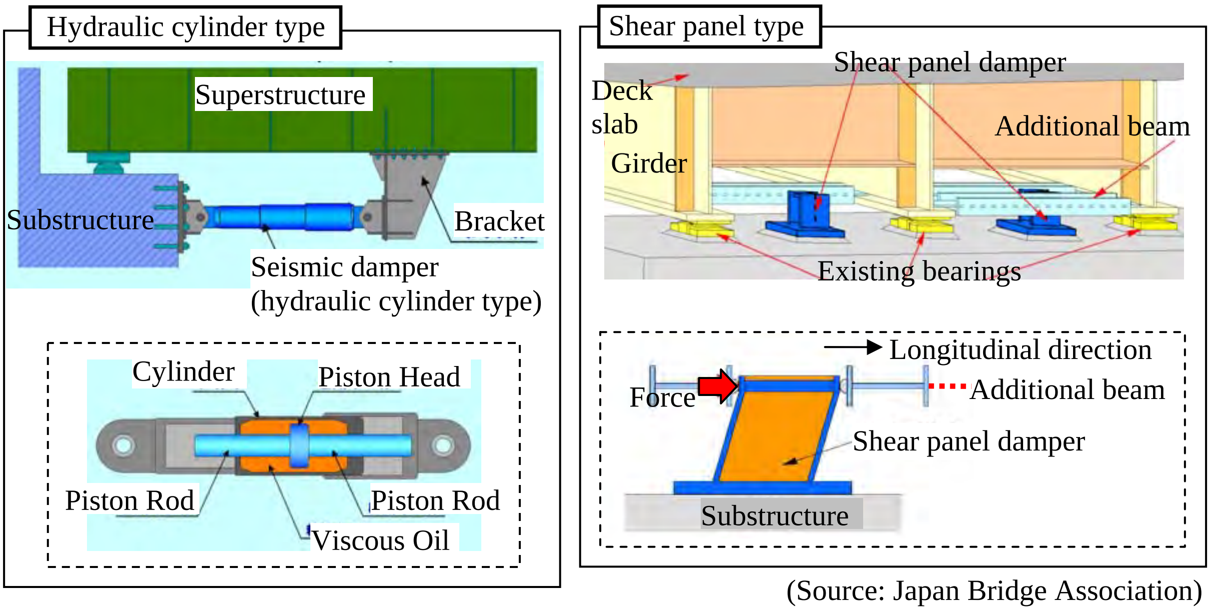
(Source: Japan Bridge Bearing Association)

Figure 4.1.2-1 Construction Steps of Bearing Replacement

4.2 Seismic Dampers

4.2.1 Outline

- Absorb seismic energy and control seismic inertial force on substructures
- Mitigate the seismic force/impact to substructure



(Source: Japan Bridge Association)

Figure 4.2.1-1 Outline of Seismic Dampers

4.2.2 Application Example

[Application Example of Seismic Damper (Hydraulic Cylinder Type)]

[Application Example of Seismic Damper (Shear Panel Type)]

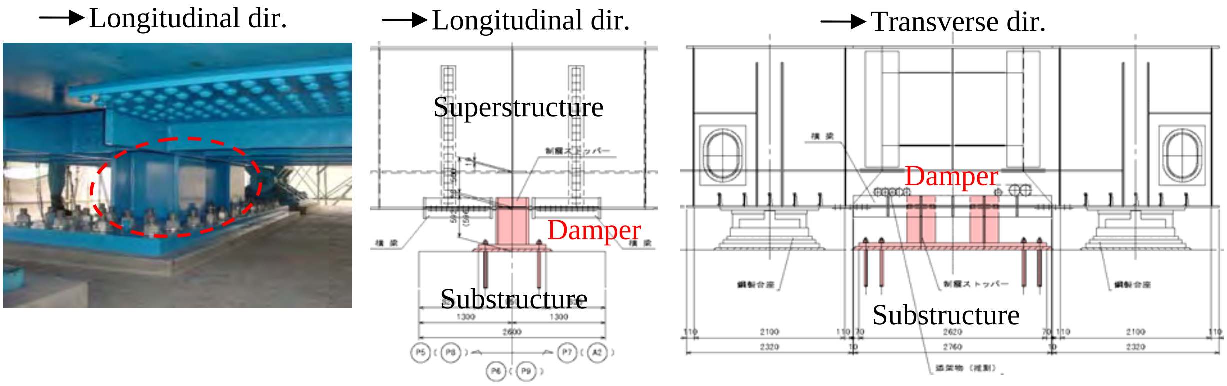


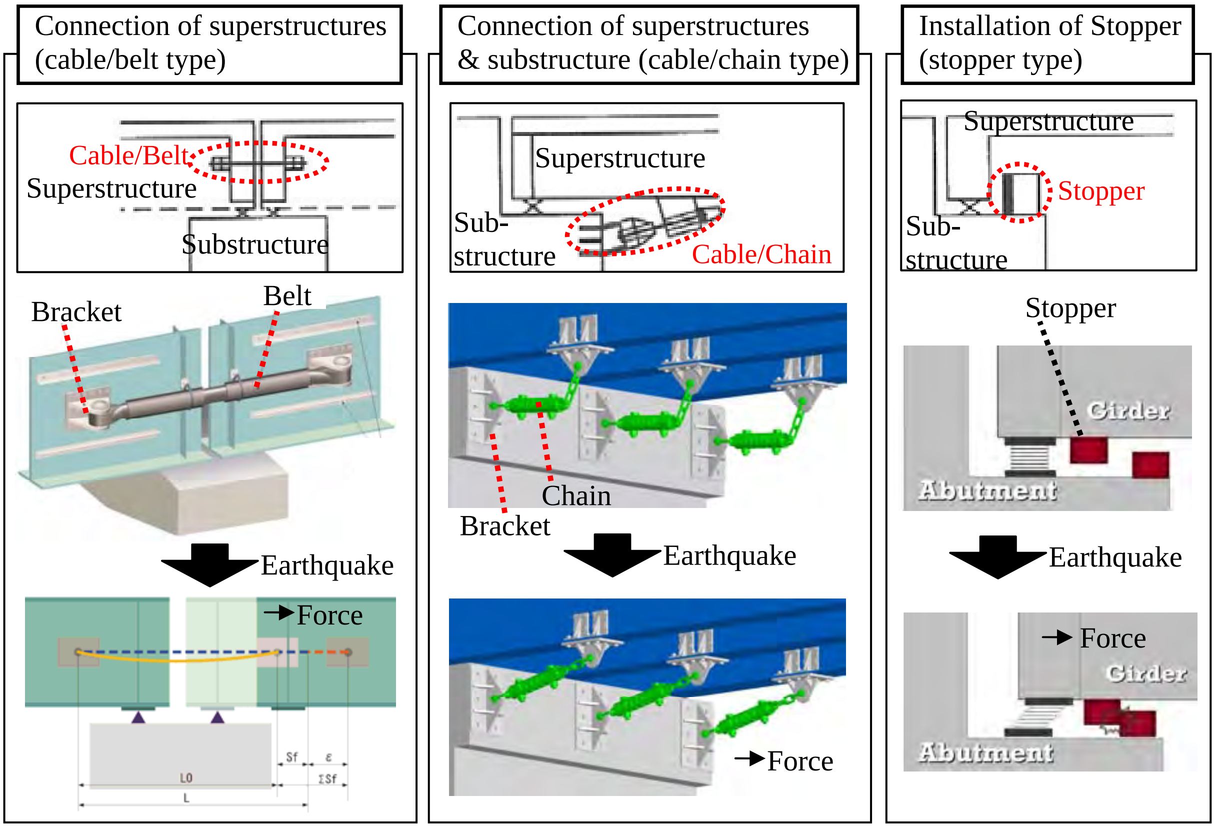
Figure 4.2.2-1 Application Example of Seismic Dampers

CHAPTER 5　UNSEATING PREVENTION DEVICES

5.1 Unseating Prevention Cable/Belt/Chain/Stopper

5.1.1 Outline

- Restrain falling down of superstructures against large-scale earthquakes
- Mitigate the seismic force/impact to substructure



(Source: Japan Bridge Bearing Association & NETIS)

Figure 5.1.1-1 Outline of Unseating Prevention Cable/Belt/Chain/Stopper

5.1.2 Application Example

Belt type　Cable type　Chain type　Stopper type

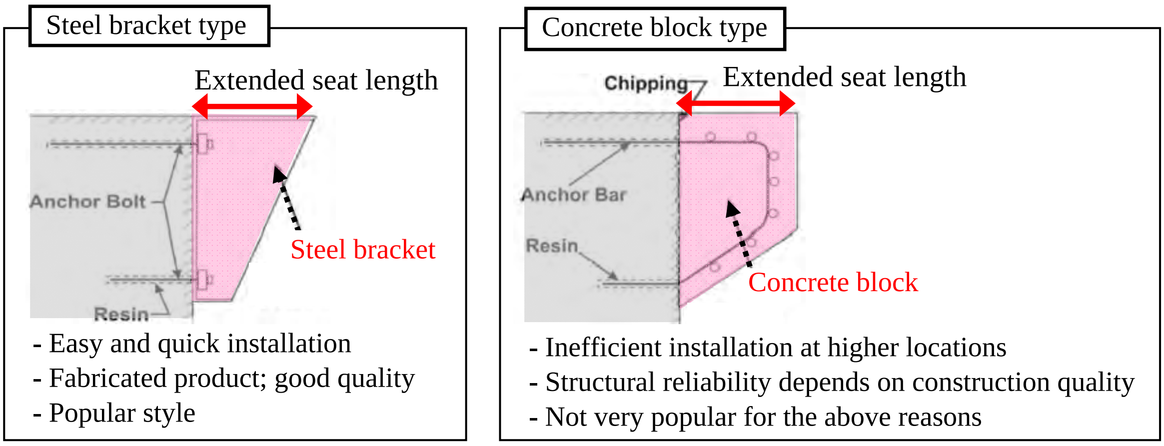
(Source: Japan Bridge Association, NETIS, & Japan Bridge Bearing Association)

Figure 5.1.2-1 Application Example of Unseating Prevention Cable/Belt/Chain/Stopper

5.2 Seat Extender

5.2.1 Outline

- Restrain falling down of superstructures against large-scale earthquakes



- Easy and quick installation
- Fabricated product; good quality
- Popular style

- Inefficient installation at higher locations
- Structural reliability depends on construction quality
- Not very popular for the above reasons

Figure 5.2.1-1 Outline of Seat Extender

5.2.2 Application Example

Typical application of steel bracket

Application of steel bracket with unseating prevention chain

Application of steel bracket with unseating prevention stopper

(Source: Japan Bridge Association & NETIS)

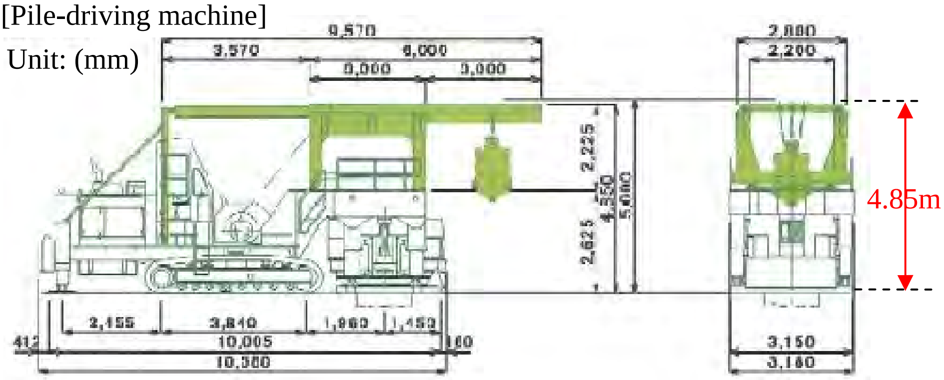
Figure 5.2.2-1 Application Example of Seat Extender

CHAPTER 6 SEISMIC RETROFIT OF FOUNDATIONS

6.1 Additional Pile Foundation (Cast-in-place Concrete Pile; CCP)

6.1.1 Outline

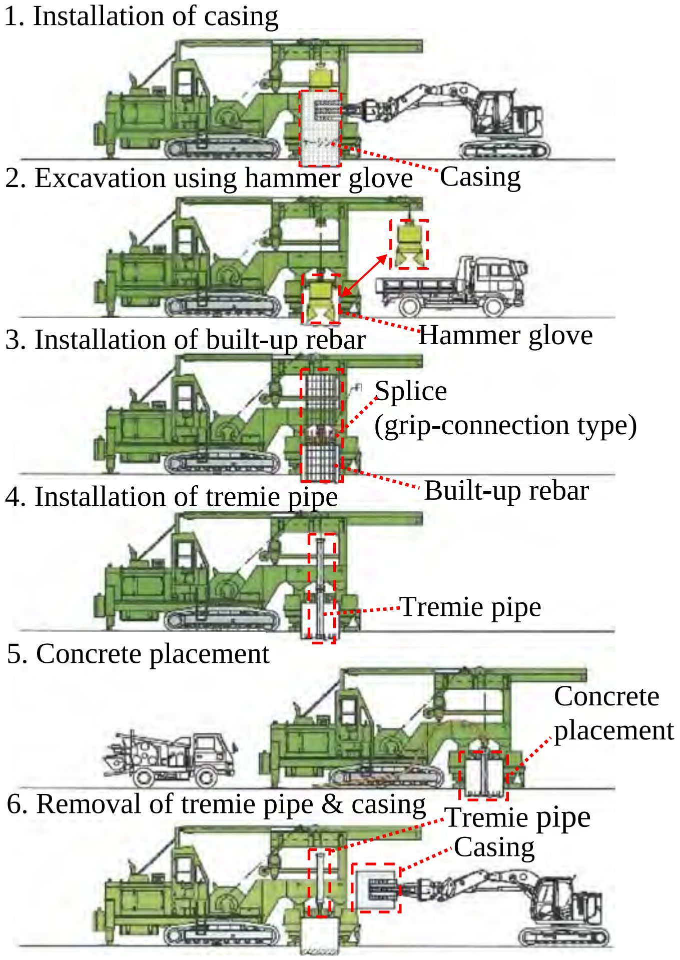
- Improve the foundation stability of existing structures by additional piles
- Application of pile-driving machine under existing superstructure
- Required overhead clearance for construction: about 5.0m



(Source: NETIS)

Figure 6.1.1-1 Outline of Additional Pile Foundation (Cast-in-place Concrete Pile)

6.1.2 Construction Steps



1.

2.

3.

(Source: NETIS)

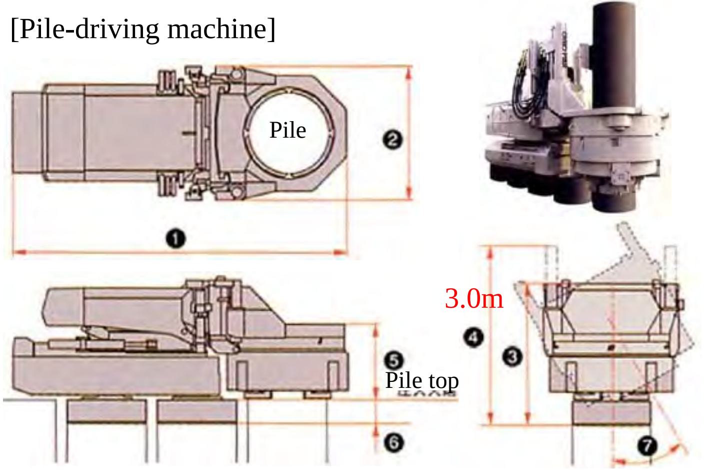
Figure 6.1.2-1 Construction Steps of Additional Pile Foundation (Cast-in-place Concrete Pile)

6.2 Additional Pile Foundation (Steel Pipe Pile; SPP)

6.2.1 Outline

- Improve the foundation stability of existing structures by additional piles
- Application of pile-driving machine under existing superstructure
- Required overhead clearance for construction: about 3.0-4.0m

[Pile-driving machine]



Unit: (mm)

Type/ Power	GRAL1015 (SP6)/ 1500kN (153 ton)	GRAL1520 (SP8)/ 2000kN (205 ton)
Stroke	700mm	800mm
1	4810mm	3620mm
2	2140mm	2280mm
3	2360mm	3170mm
4	3000mm	3620mm
5	1160mm	1560mm
6	300mm	470mm
7	30degrees (side-side)	30degrees (side-side)
EU*	EU200 + EU300	EU500
Weight	28 ton (if φ1000)	42.9 ton (if φ1500)
φ	SPP φ800-1000	SPP φ1300-1500

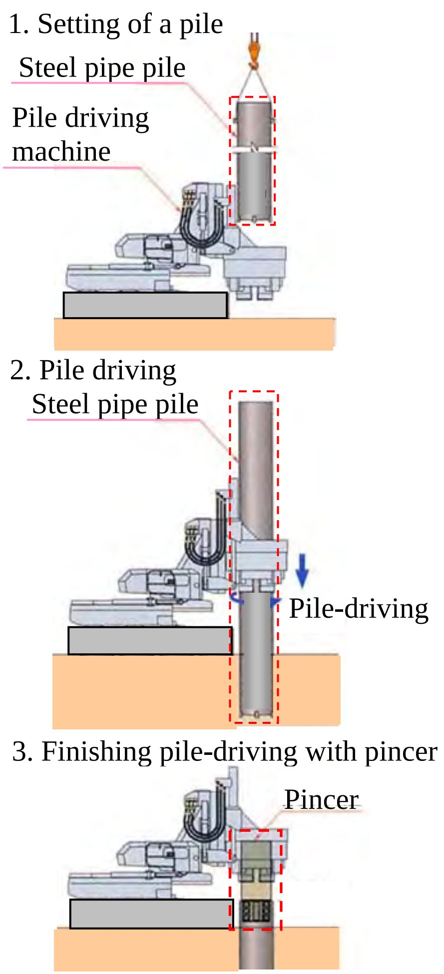
*: Energy Unit

(Source: Japanese Association for Steel Pipe Piles)

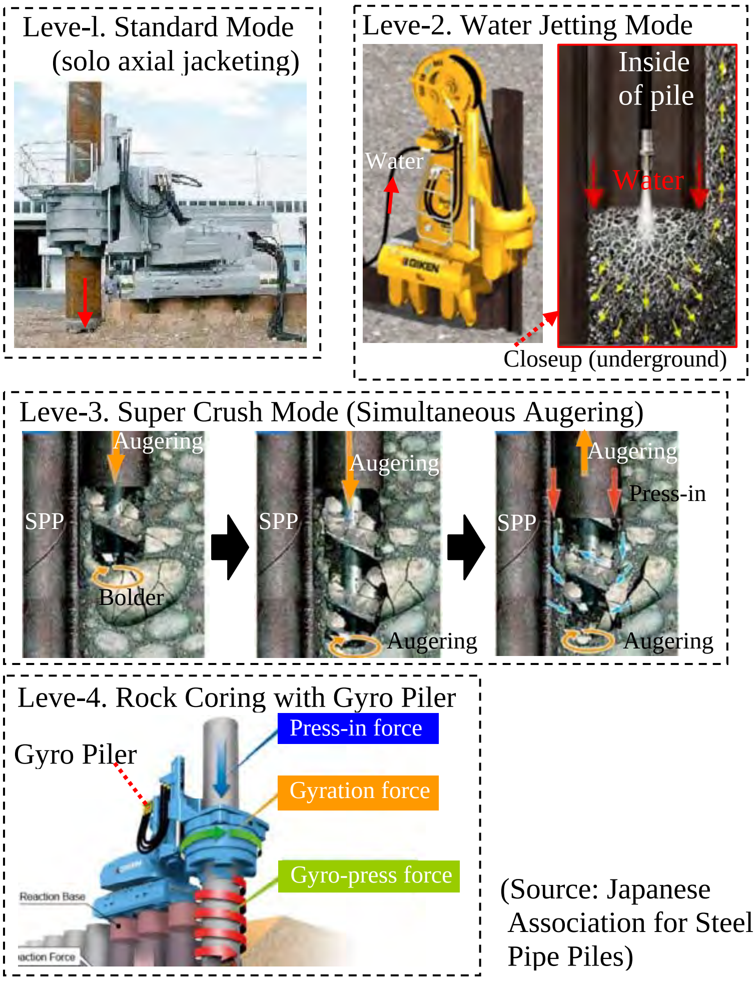
Figure 6.2.1-1 Outline of Additional Pile Foundation (Steel Pipe Pile)

6.2.2 Construction Steps & Penetration Modes

Construction Steps



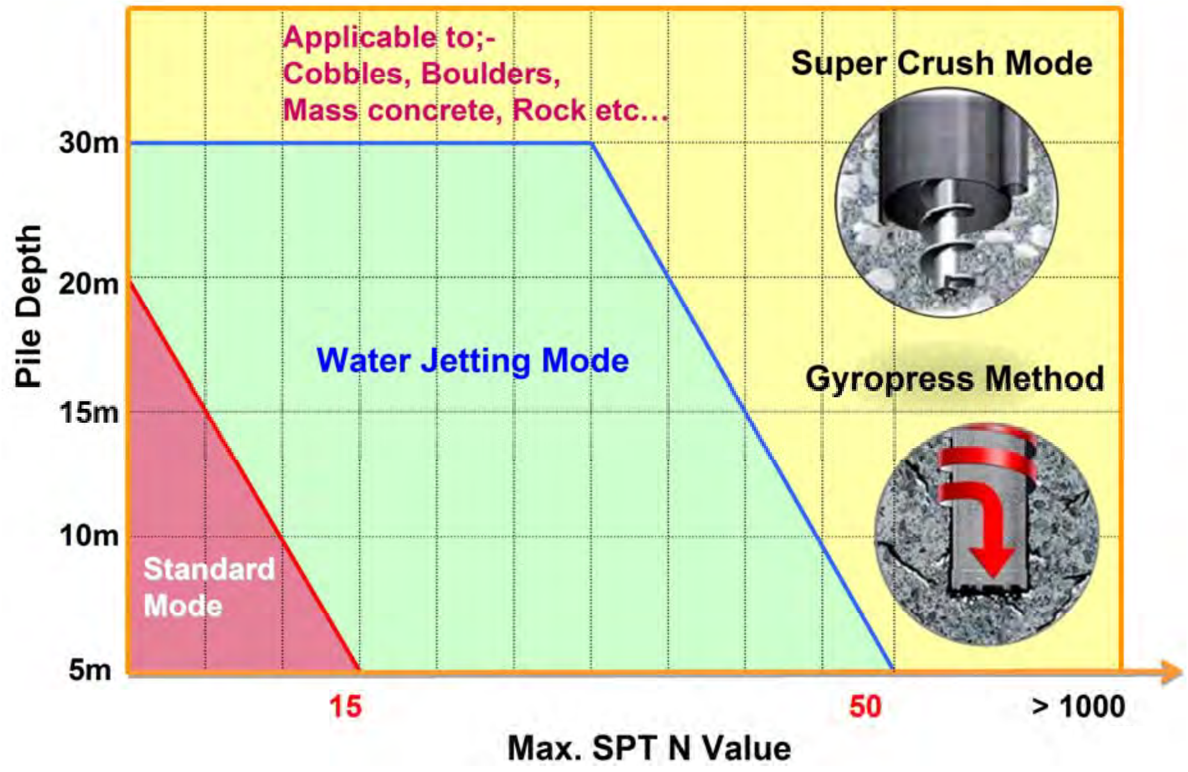
Penetration Modes (Penetration Level 1-4)



(Source: Japanese Association for Steel Pipe Piles)

Figure 6.2.2-1 Construction Steps of Additional Pile Foundation (Steel Pipe Pile)

6.2.3 Comparison of SPP Penetration Modes



(Source: Japanese Association for Steel Pipe Piles)

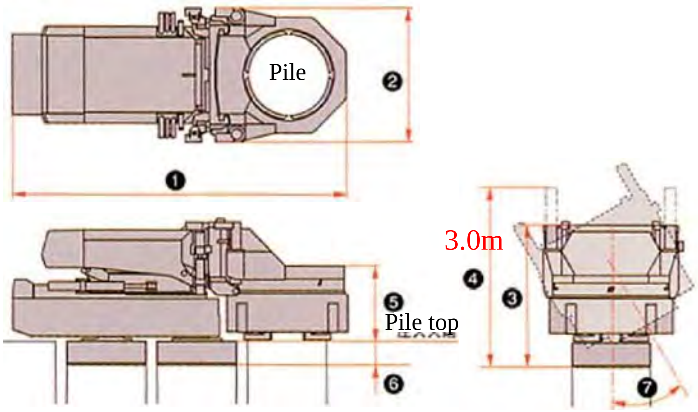
Figure 6.2.3-1 Comparison of SPP Penetration Modes (Pile Depth vs Ground Conditions)

6.3 Foundation Reinforcement for Piers in Water (Steel Pipe Sheet Pile; SPSP)

6.3.1 Outline

- Improve the foundation stability of existing structures by additional piles
- Application of pile-driving machine under existing superstructure without sheet piles
- Required overhead clearance for construction: about 3.0-4.0m

[Pile-driving machine (same as machines for SPP)]



Unit: (mm)

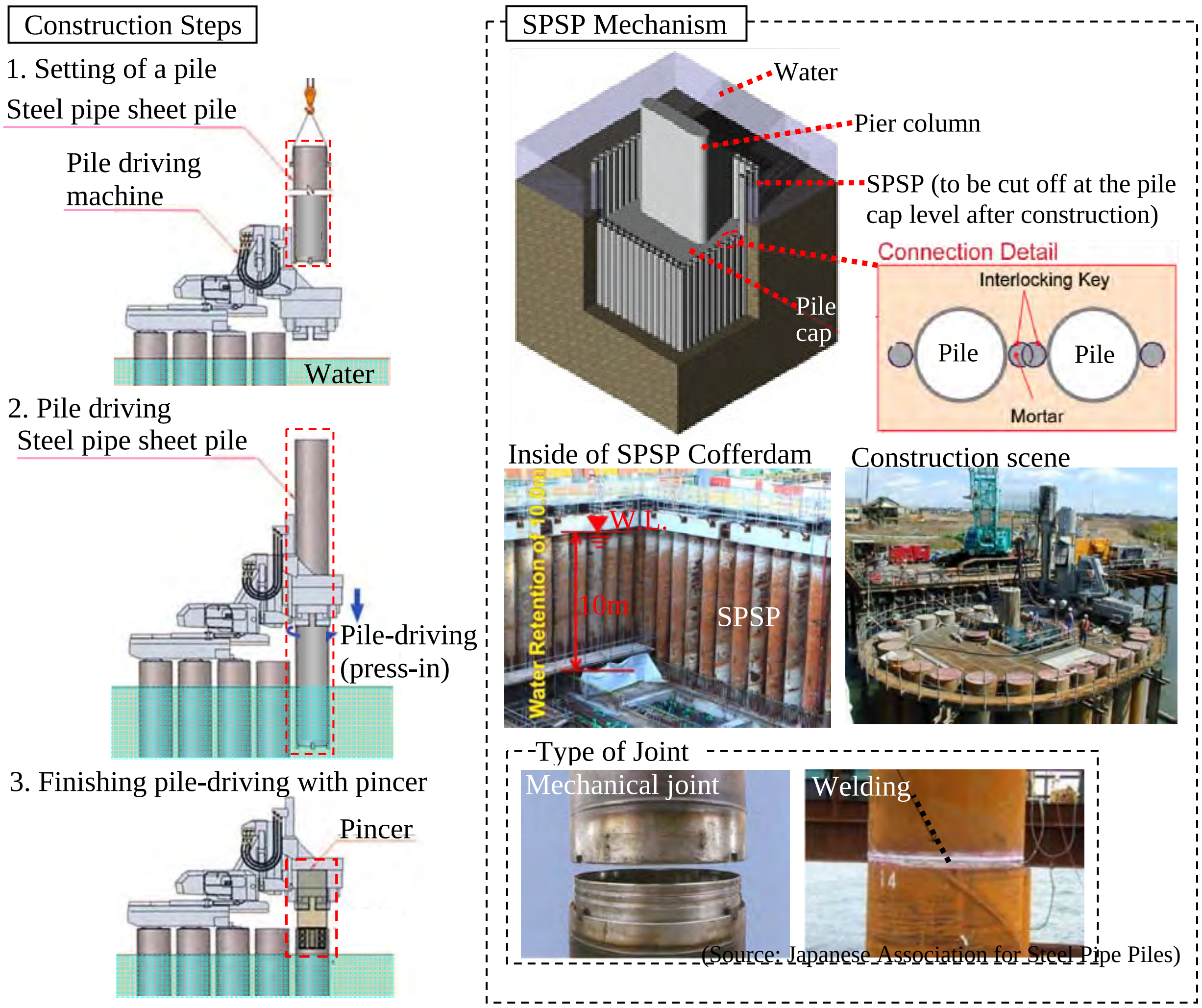
Type/ Power	GRAL1015 (SP6)/ 1500kN (153 ton)	GRAL1520 (SP8)/ 2000kN (205 ton)
Stroke	700mm	800mm
1	4810mm	
2	2140mm	2280mm
3	2360mm	3170mm
4	3000mm	3620mm
5	1160mm	1560mm
6	300mm	470mm
7	30degrees (side-side)	30degrees (side-side)
EU*	EU200 + EU300	EU500
Weight	28 ton (if φ1000)	42.9 ton (if φ1500)
φ	SPP φ800-1000	SPP φ1300-1500

*: Energy Unit

(Source: Japanese Association for Steel Pipe Piles)

Figure 6.3.1-1 Outline of Additional Pile Foundation (Steel Pipe Pile)

6.3.2 Construction Steps & SPSP Mechanism



(Source: Japanese Association for Steel Pipe Piles)

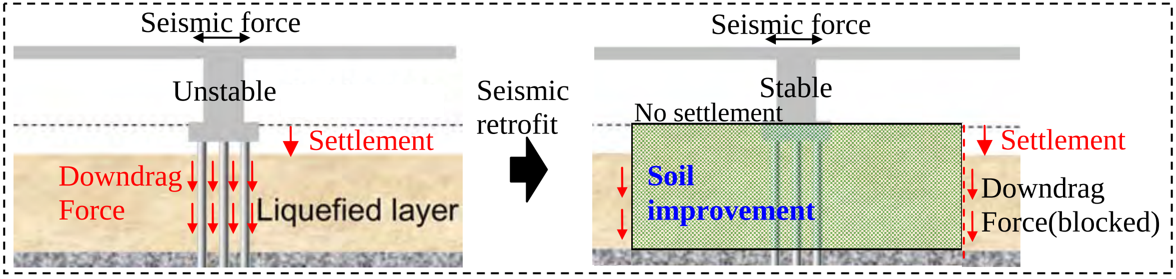
Figure 6.3.2-1 Construction Steps of Additional Pile Foundation (Steel Pipe Pile)

CHAPTER 7 SOIL/GROUND IMPROVEMENT

7.1 Soil/Ground Improvement for Liquefaction Prevention

7.1.1 Outline

[Soil Improvement for Piers]



[Soil Improvement for Abutments]

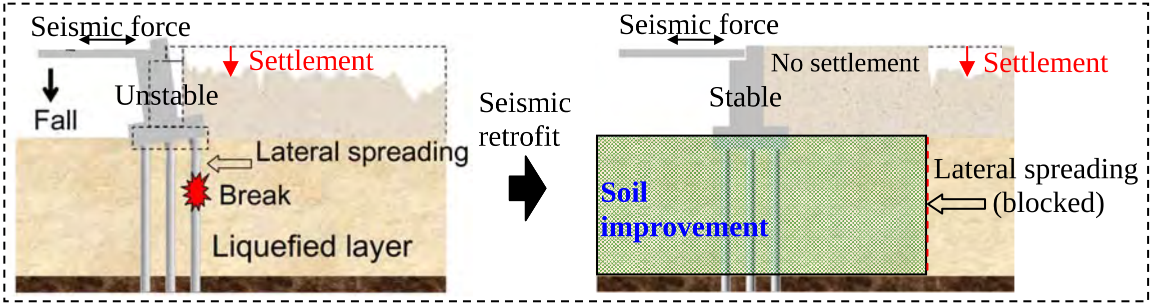
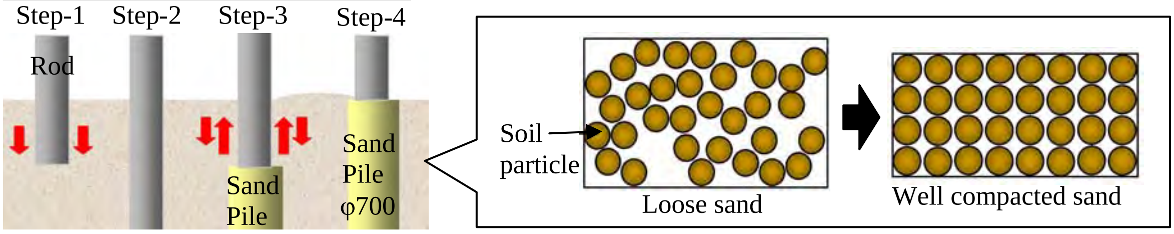


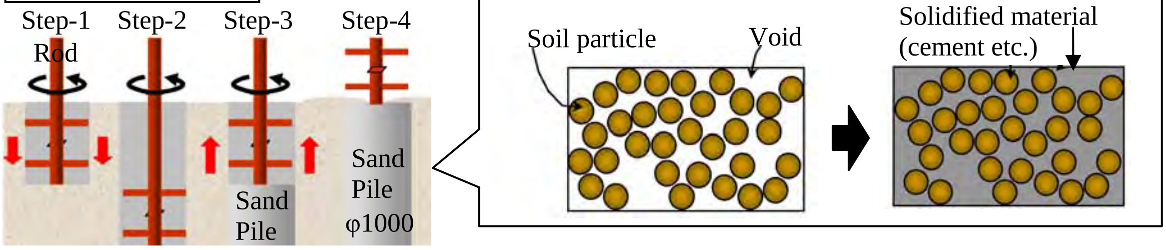
Figure 7.1.1-1 Outline of Soil/Ground Improvement for Liquefaction Prevention

7.1.2 Type of Soil Improvement for Liquefaction Prevention

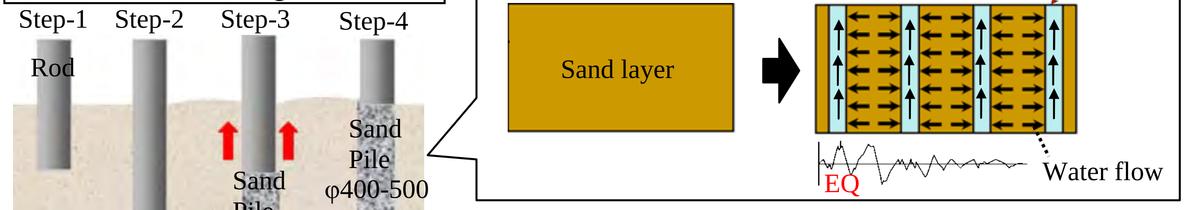
Sand Compaction/Densification



Sand Solidification



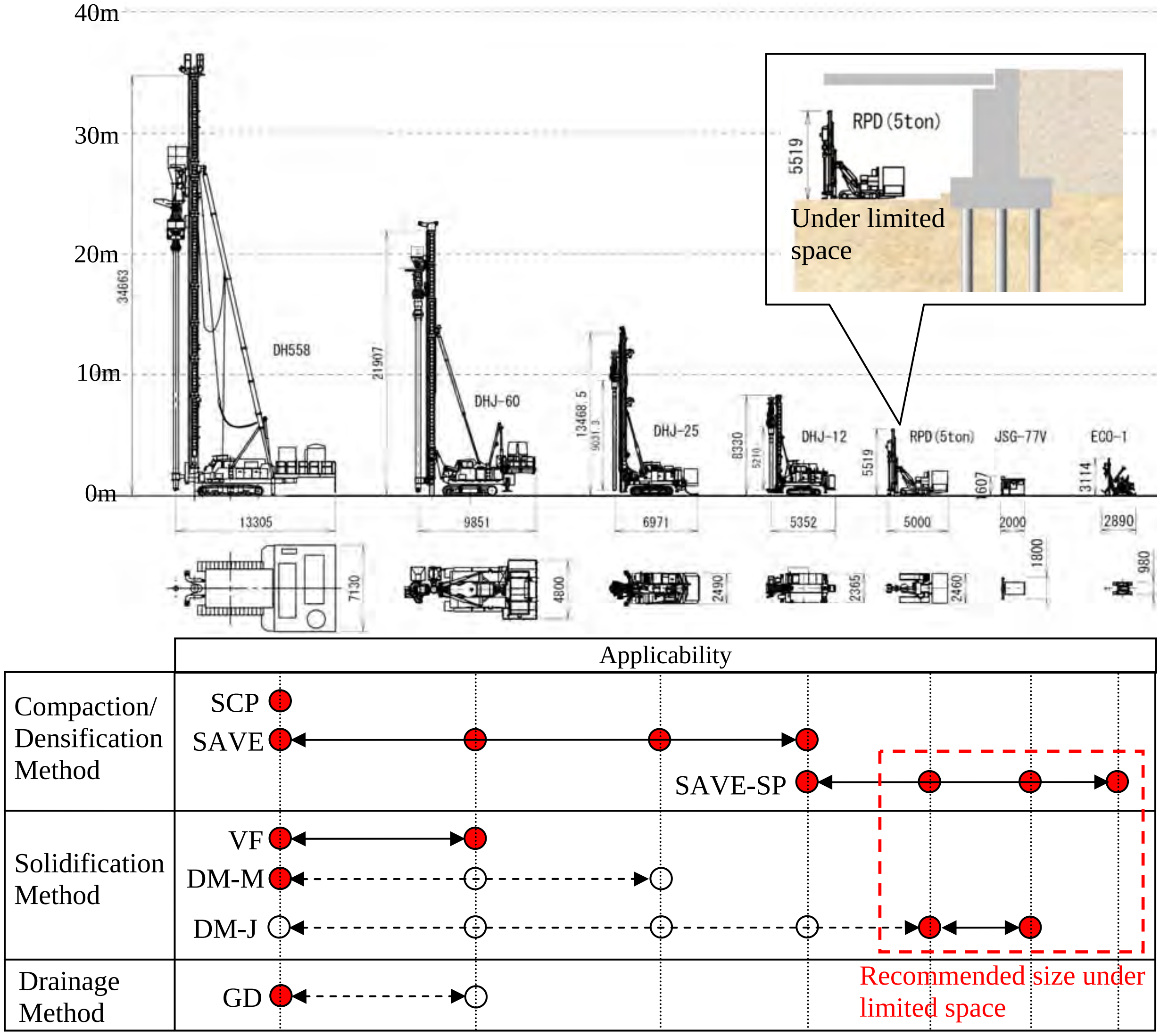
Sand Piles for Drainage Promotion



(Source: OCAJI)

Figure 7.1.2-1 Outline of Soil/Ground Improvement for Liquefaction Prevention

7.1.3 Recommended Machine Size for Construction under Limited Space



● Typical ○ Applicable

SCP: Vibratory Sand Compaction Pile Method
SAVE: SP: Silent, Advanced Vibration-Erasing (Non-vibratory Sand Compaction Pile) Method
SAVE-SP: Silent, Advanced Vibration-Erasing – Sand Press Method
VF: Vibro Floatation Method
DM-M: Mechanical Mixing Type Deep Mixing Method
DM-J: Jetting Type Deep Mixing Method

(Source: Japanese Association for Steel Pipe Piles)

Figure 7.1.3-1 Relationship between Soil Improvement Type and Machine Size

(Source: OCAJI)

Figure 7.1.3-2 Application Example of Soil Improvement Method under Existing Structures

7.2 Soil/Ground Improvement for Earth Pressure Reduction

7.2.1 Outline

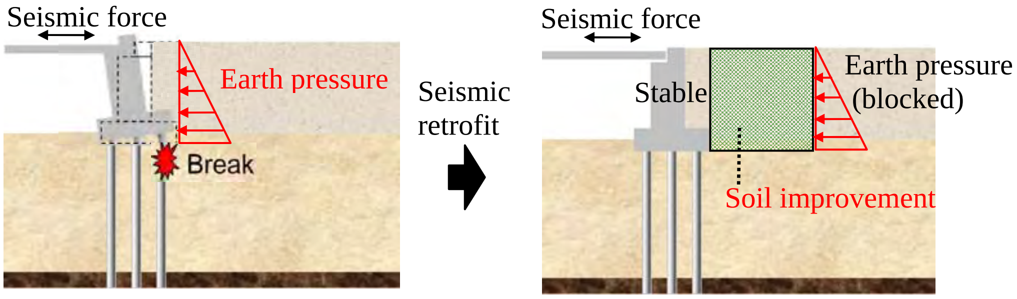
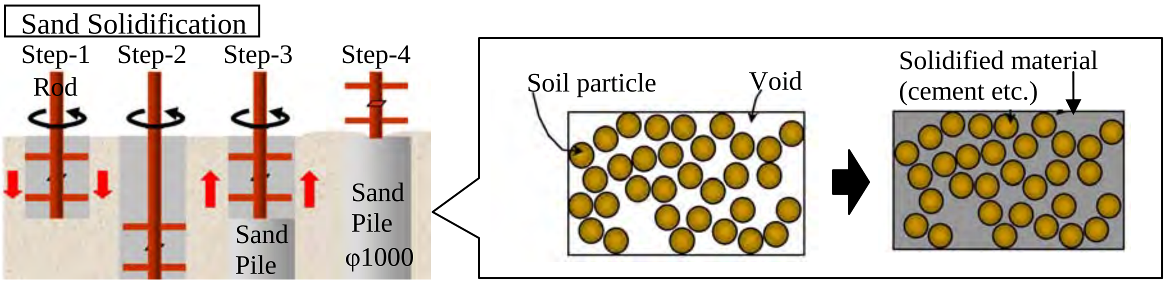


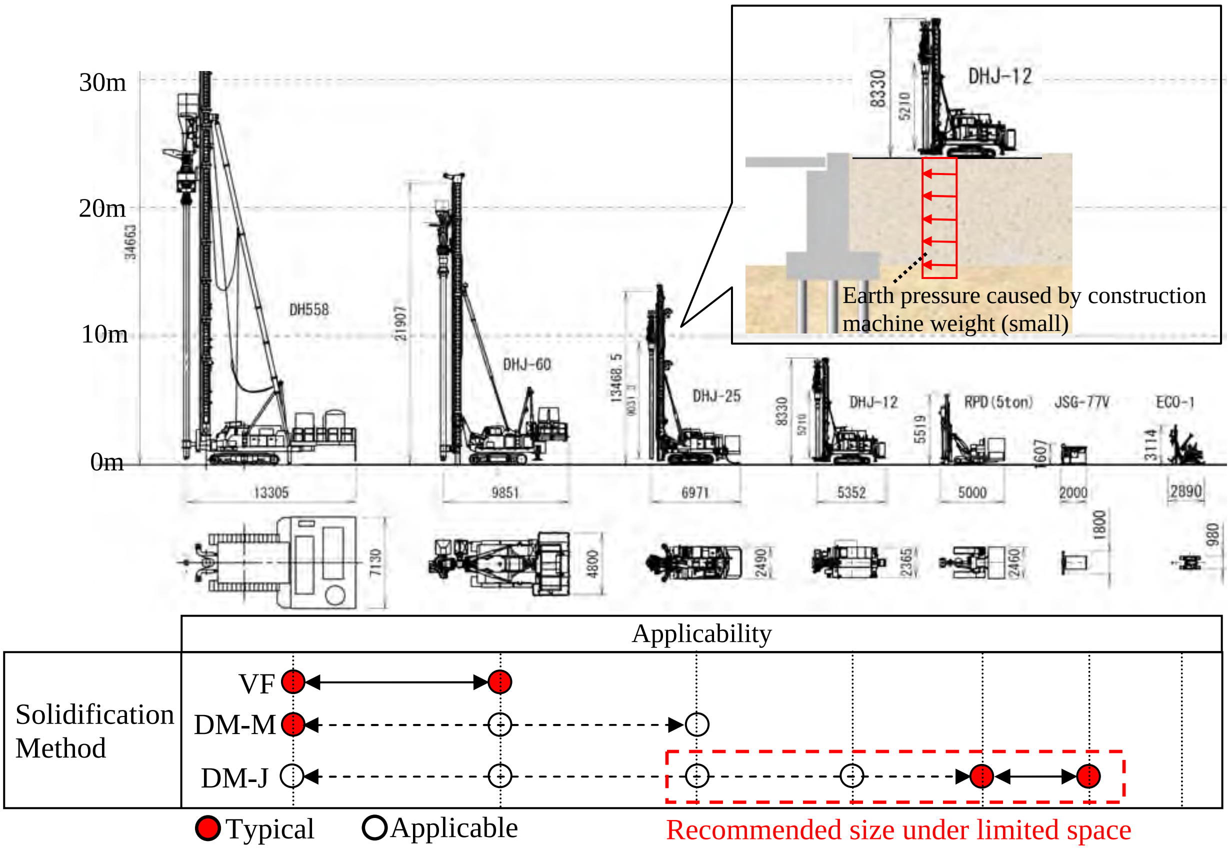
Figure 7.2.1-1 Outline of Soil/Ground Improvement against Liquefaction

7.2.2 Type of Soil Improvement for Earth Pressure Reduction



(Source: OCAJI)

7.2.3 Type of Soil Improvement for Earth Pressure Reduction



(Source: OCAJI)

Figure 7.2.3-1 Relationship between Soil Improvement Type and Machine Size

APPENDIX 1-D

COMPARISON OF SEISMIC DESIGN SPECIFICATIONS (JRA, AASHTO AND NSCP/DPWH)

(1) COMPARISON TABLE OF BRIDGE SEISMIC DESIGN SPECIFICATIONS BETWEEN JRA AND AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS (6TH Ed., 2012)

<table>
<tr><th>Items</th><th>JRA (Part V; English Version, 2002)</th><th>AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)</th></tr>
<tr><td>1. Fundamentals of Seismic Design</td>
<td>(1) It shall be ensured that the seismic performance according to the levels of design earthquake motion and the importance of a bridge.
(2) It is desirable to adopt a multi-span continuous structure, the type of which bearing supports is to be a horizontal force distributed structure.
(3) It is generally better for a bridge with tall piers built in a mountainous region to resist seismic horizontal forces by abutments rather than piers if the ground conditions at the abutments are sufficiently sound.(The seismic performance of the whole bridge should be considered, and proper bearing supports in view of bridge structural conditions and ground bearing properties should be selected.)
(4) On reclaimed land or alluvial ground where ground deformation such as sliding of a soft cohesive clayey layer, liquefaction of sandy layer and liquefaction-induced ground flow may happen, a foundation with high horizontal stiffness should be designed, and a structural system such as multi-fixed-point type and rigid frame type, which has many contact points between the superstructure and substructure, should be selected.
(5) A seismically-isolated bridge should be adopted for a multi-span short-period continuous bridge on stiff ground conditions.
(6) For a strong earthquake motion, a proper structural system shall be designed by clarifying structural members with nonlinear behavior and those basically remaining in elastic states.
(7) A structure greatly affected by geometrical nonlinearity or a structure having extensive eccentricity of dead loads, which have tends to become unstable during a strong earthquake motion, shall not be adopted.
(8) When ground conditions or structural conditions on a pier change remarkably, whether a case of two girder ends or that of a continuous girder is more advantageous is carefully examined.</td>
<td>(1) Bridges shall be designed for specified limit states to achieve the objectives of constructability, safety, and serviceability, with due regard to issues of inspectability, economy, and aesthetics.
(2) Each component and connection shall satisfy Eq. 1-1 for each limit state, unless otherwise specified. For service and extreme event limit states, resistance factors shall be taken as 1.0
$\Sigma\eta_i\gamma_i Q_i \leq \phi R_n = R_r$ (1-1)
where: Q = force effect
η = load modifier/factor relating to ductility, redundancy and operational classification
R_n = minimal resistance
ϕR_n = factored resistance

(3) The extreme event limit state shall be taken to ensure the structural survival of a bridge during a major earthquake.
(4) The structural system of a bridge shall be proportioned and detailed to ensure the development of significant and visible inelastic deformations at the strength and extreme event limit states before failure. Energy dissipating devices may be substituted for conventional ductile earthquake resisting systems.
In order to achieve adequate inelastic behavior, the system should have sufficient number of ductile members and either:
• Joints and connections that are also ductile and can provide energy dissipation without loss of capacity; or
• Joints and connections that have sufficient excess strength so as to assure that the inelastic response occurs at the locations designed to provide ductile, energy absorbing response.
(5) Multiple-load-path and continuous structures should be used unless there are compelling reasons not to use them.
(6) The possibility of partial live load with earthquakes, especially in urban areas, should be considered. Application of Turkstra's rule for combining uncorrelated loads indicates that a factor of 0.5 is reasonable for a wide range of values of average daily truck traffic (ADTT).</td></tr>
<tr><td>2. Principles of Seismic Design</td>
<td>(1) Seismic Performance of Bridges
<table>
<tr><th rowspan="2">Seismic Performance</th><th rowspan="2">Seismic Safety Design</th><th rowspan="2">Seismic Serviceability Design</th><th colspan="2">Seismic Reparability Design</th></tr>
<tr><th>Emergency Reparability</th><th>Permanent Reparability</th></tr>
<tr><td><u>Seismic Performance Level 1</u>
Keeping the sound functions of bridges</td><td>To prevent girders from unseating</td><td>To ensure the normal functions of bridges (within elastic limit states)</td><td>No repair work is needed to recover the functions</td><td>Only easy repair works are needed</td></tr>
<tr><td><u>Seismic Performance Level 2</u>
Limited damages and recovery</td><td>Same as above</td><td>Capable of recovering functions within a short period after the event</td><td>Capable of recovering functions by emergency repair works</td><td>Capable of easily undertaking permanent repair works</td></tr>
<tr><td><u>Seismic Performance Level 3</u>
No critical damages</td><td>Same as above</td><td>- *</td><td>-</td><td>-</td></tr>
</table>
*: "-": Not covered</td>
<td>(1) For seismic design considerations, AASHTO focuses on the "Extreme Event Load Combination I" which is an extreme event limit state that must ensure the structural survival of a bridge during a major earthquake.
(2) <u>Performance Level</u>
There is no specific performance level in AASHTO, however, in terms of earthquake effects "Bridges shall be designed to have a low probability of collapse but may suffer significant damage and disruption to service when subject to earthquake ground motions that have a seven percent probability of exceedance in 75 years. Partial or complete replacement may be required".
<table>
<tr><th>Earthquake Level</th><th>Bridge Types</th><th>Serviceability Performance</th><th>Safety Performance</th></tr>
<tr><td>Small/Moderate</td><td>Conventional and regular bridge types</td><td>• Should resist earthquakes within the elastic range of the structural components</td><td>• No significant damage</td></tr>
<tr><td rowspan="3">Large/Major</td><td>Critical bridges</td><td>• Required to be open to all traffic once inspected after seismic event and usable by emergency vehicles for security, defense, economic or secondary life safety purposes immediately after the seismic event</td><td rowspan="3">• May suffer damage but with low probability of collapse.</td></tr>
<tr><td>Essential bridges</td><td>• Should, as a minimum, be open to emergency vehicles and for security, defense, or economic purposes after the seismic event and open to all traffic within days after that event.</td></tr>
<tr><td>Conventional/ regular bridge and less important bridges</td><td>• May suffer significant damage and disruption to service</td></tr>
</table></td></tr>
</table>

<table>
<tr><th>Items</th><th>JRA (Part V; English Version, 2002)</th><th>AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)</th></tr>
<tr><td></td><td>
(2) Relationship between Design Earthquake Ground Motions and Seismic Performance of Bridges
<table>
<tr><td colspan="2">Levels of Earthquake Ground Motions</td><td>Class A Bridges*</td><td>Class B Bridges*</td></tr>
<tr><td colspan="2">Level 1: Highly probable during the bridge service life</td><td colspan="2">Seismic Performance Level 1 is required</td></tr>
<tr><td rowspan="2">Level 2</td><td>Type I: An Plate Boundary Type Earthquake with a Large Magnitude</td><td rowspan="2">Seismic Performance Level 3 is required</td><td rowspan="2">Seismic Performance Level 2 is required</td></tr>
<tr><td>Type II: An Inland Direct Strike Type Earthquake</td></tr>
</table>
*: Class A Bridges: Standard Importance; Class B Bridges: High Importance (Class A and B are classified according to such importance factors as road class, bridge functions and structural characteristics.)
When bridge importance is classified in view of roles expected in the regional disaster prevention plan and road serviceability, the following should be considered.
(a) To what extent a bridge is necessitated for post-event rescue and recovery activities as emergency transportation routes.
(b) To what extent damages to bridges (such as double-deck bridges and overbridges) affect other structures and facilities.
(c) Present traffic volume of the bridge and availability of substitute in case of the bridges losing pre-event functions.
(d) Difficulty (duration and cost) in recovering bridge function after the event.
</td><td></td></tr>
<tr><td>3. Loads to be considered in Seismic Design</td><td>
(1) Loads and their Combinations
(a) Primary Loads: Dead load (D), Pre-stress force (PS), Effect of creep of concrete (CR), Effect of drying shrinkage of concrete (SH), Earth pressure (E), Hydraulic pressure (HP), Buoyancy or Uplift (U)
(b) Secondary loads: Effects of earthquake (EQ)
(c) Combination of loads: Primary loads + Effects of earthquake (EQ)
(d) Loads and their combinations shall be determined in such manners that they cause the most adverse stress, displacements and effects.
(2) Effects of Earthquake (EQ)
(a) Inertia force, (b) Earth pressure during an earthquake, (c) Hydrodynamic pressure during an earthquake, (d) Effects of liquefaction and liquefaction-induced ground flow, (e) Ground displacement during an earthquake
</td><td>
(1) Loads and Load Combinations
(a) Permanent Loads: Dead load of structural components and non-structural attachments (DC), Dead load of wearing surfaces (DW), Down drag (DD), Horizontal earth pressure load (EH), Earth surcharge (ES), Vertical pressure from dead load of earth fill (EV) Secondary force from post-tensioning (PS), Miscellaneous lock-in force due to construction process (EL), Force effects due to shrinkage (SH), Force due to creep (CR).
(b) Transient Loads: Vehicular live load (LL), Water load and stream pressure (WA), Friction load (FR)
(c) Earthquake Load (EQ)
(d) Combination of Loads: Permanent Loads + Transient Loads + Earthquake Load
(e)The load factors shall be selected to produce the total extreme factored force effects. Both positive and negative extremes shall be investigated. In load combinations where one force effect decreases another effect, the minimum value shall be applied to the load reducing the force effect.
(2) Effects of Earthquake (EQ)
(a) Bridge inertia effect, (b) Earth pressure during and earthquake, (c) Hydrodynamic pressure during an earthquake, (d) Live load inertia effect, (e) Potential for soil liquefaction, (f) Potential for slope movement
</td></tr>
</table>

Items

JRA (Part V; English Version, 2002)

AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)

4. Design Earthquake Ground Motions for Level 1 and Level 2

S = C_Z*C_D***S_0** (S: ARS for Level 1EGM, S0: SARS (Fig.4-1))

SI = C_Z*C_D***S_{I0}** (SI: Type I ARS for Level 2 EGM, SI0:SARS (Fig.4-2))

SII = C_Z*C_D***S_{II0}** (SII: Type II ARS for Level 2 EGM, SII0: SARS(Fig.4-2))

(SARS= **S**tandard **A**cceleration **R**esponse **S**pectra, ARS= **A**cceleration **R**esponse **S**pectra, EGM = **E**arthquake **G**round **M**otion)

C_D: Modification factor for damping ratio (h) of structures (Fig.4-3)

C_Z: Modification factor for zones (Fig.4-4)

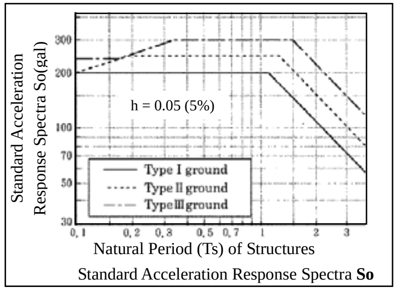
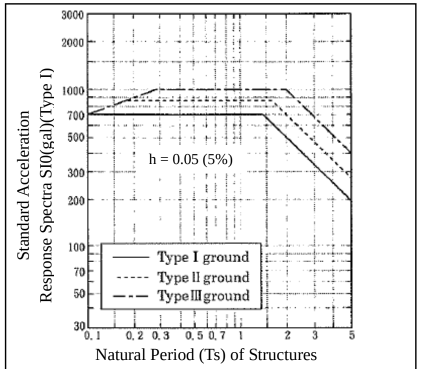
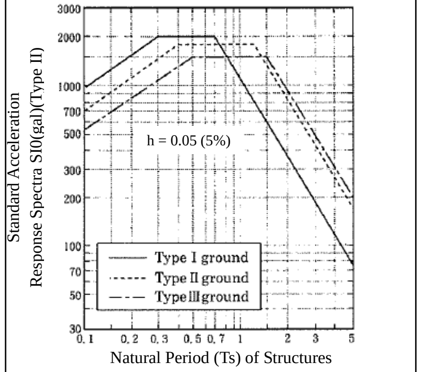


Fig.4-1 Level 1 Earthquake Ground Motion



Standard Acceleration Response Spectra **S_{I0}** (Type I)



Standard Acceleration Response Spectra **S_{II0}** (Type II)

Fig 4-2 Level 2 Earthquake Ground Motions

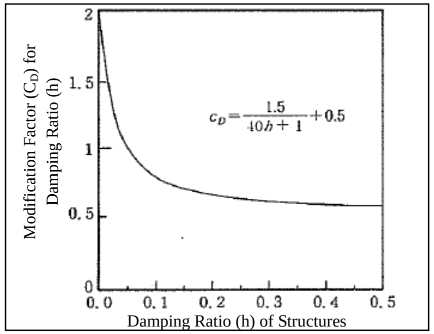
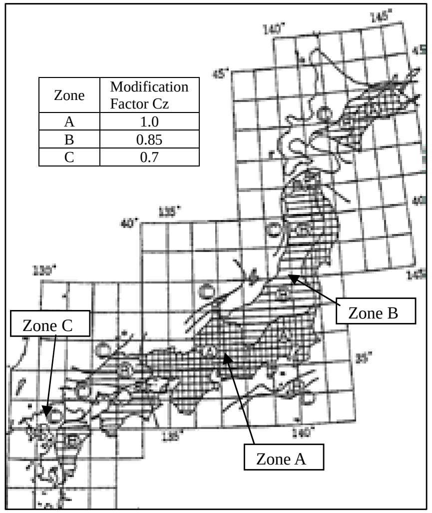


Fig.4-3 Modification Factor (CD) for Damping Ratio (h) of Structures



Zone	Modification Factor Cz
A	1.0
B	0.85
C	0.7

Fig.4-4 Modification Factors for Zones, Cz

(1) The general procedure to develop the design spectrum (Figure 4-1) is to use the peak ground acceleration coefficient (PGA) and the short and long period spectral acceleration coefficients (S_S and S_1) based on the maps prepared in the specifications.

The five-percent-damped-design response spectrum shall be taken as specified in Figure 4-1. This spectrum shall be calculated using the mapped peak ground acceleration coefficients and the spectral acceleration coefficients scaled by zero-,short-, and long-period site factors, F_{pga}, F_a and F_v respectively.

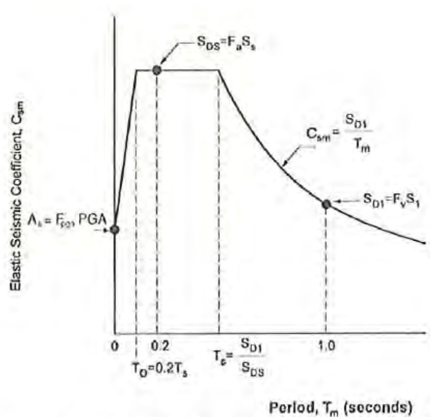


Figure 4-1 AASHTO Design Response Spectrum

where:

A_s = peak seismic ground acceleration coefficient modified by short-period site factor

F_{pga} = site factor at zero-period on acceleration response spectrum

PGA = peak seismic ground acceleration coefficient on rock

S_{DS} = horizontal response spectral acceleration coefficient at 0.2-s period modified by short-period site factor

F_a = site factor for short-period range of acceleration response spectrum

S_s = horizontal response spectral acceleration coefficient at 0.2-s period on rock

C_{sm} = elastic seismic response coefficient for the mth mode of vibration

S_{D1} = horizontal response spectral acceleration coefficient at 1.0-s period modified by long-period site factor

F_v = site factor for long-period range of acceleration response spectrum

S_1 = horizontal response spectral acceleration coefficient at 1.0-s period on rock

- Each bridge is assigned to one of the four seismic zones in accordance with Table 4-1using the value of S_{D1} given by Equation 4-1.

Table 4-1 Seismic Zones

Acceleration Coefficient, S_{D1}	**Seismic Zone**
$S_{D1} \leq 0.15$	1
$0.15 < S_{D1} \leq 0.30$	2
$0.30 < S_{D1} \leq 0.50$	3
$0.50 < S_{D1}$	4

$$S_{D1} = F_v S_1 \qquad (4\text{-}1)$$

where: F_v = site factor for long-period range of acceleration response spectrum

S_1 = horizontal response acceleration coefficient at 1.0s period on rock (Site Class B)

Items

5. Ground Type for Seismic Design

JRA (Part V; English Version, 2002)

Table 5-1 Ground Types in Seismic Design

Ground Type	Characteristic Value of Ground, T_G (s)	Description
Type I	$T_G < 0.2$	Good diluvial ground and rock
Type II	$0.2 \leq T_G < 0.6$	Diluvial and alluvial ground not belonging to Type I and Type II
Type III	$0.6 \leq T_G$	Soft ground of alluvial ground

$T_G = 4 * \sum_{i=1}^{n} Hi / Vsi$ T_G = Characteristic value of ground (s)

Hi = Thickness of the i-th soil layer

Vsi = Average shear wave velocity of the i-th soil layer (m/s). If Vsi is not available, Vsi can be obtained from the following formula.

$Vsi = 100 * Ni^{1/3}$ ($1 \leq Ni \leq 25$): for cohesive soil layer (if N = 0, Vsi = 50 m/s; when N=25, Vsi = 300m/s)

$Vsi = 80 * Ni^{1/3}$ ($1 \leq Ni \leq 50$): for sandy soil layer (if N = 0, Vsi = 50 m/s; when N=50, Vsi = 300m/s)

Ni = Average N value of thei-th soil layer obtained from SPT

i = Number of the i-th layer from the ground surface when the ground is classified into “n” layers up to “the surface of a base ground surface for seismic design”

Note: “The surface of a base ground surface for seismic design” represents upper surface of a fully hard ground layer that exists over a wide area in the construction site, and normally situated below a surface soil layer shaking with a ground motion during an earthquake. Where, the upper surface of a fully hard ground layer might be the upper surface of a highly rigid soil layer with a shear elastic wave velocity of more than 300m/s (an N value of 25 in the cohesive soil layer and of 50 in the sandy soil layer)

If N value is not available, Ground types can be obtained following the flow chart shown in Fig 5-1.

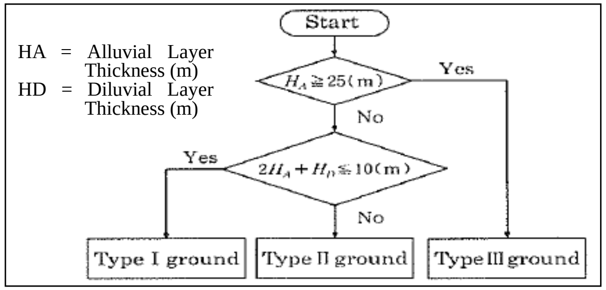

HA = Alluvial Layer Thickness (m)
HD = Diluvial Layer Thickness (m)

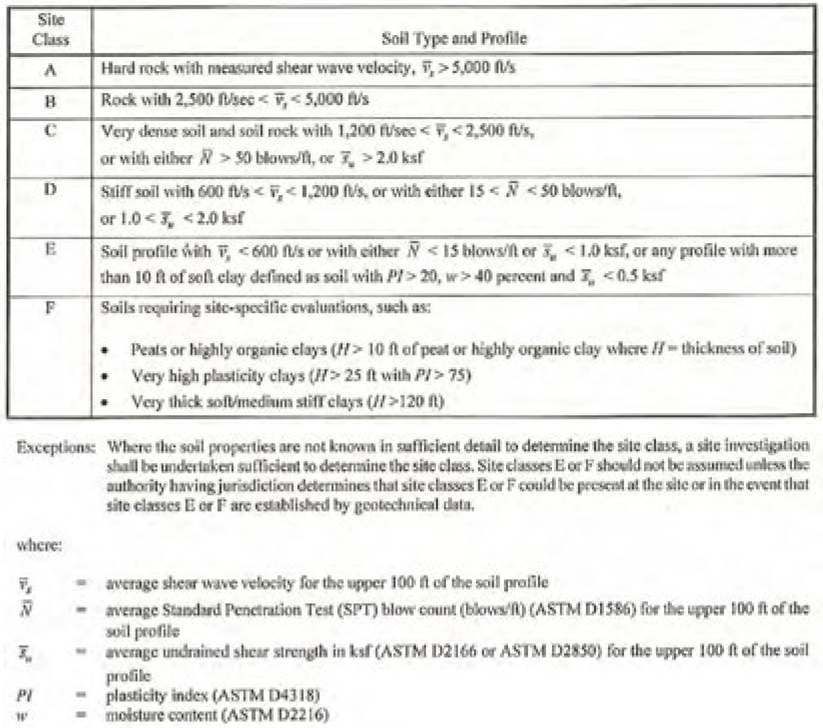
Fig.5-1 Flowchart for Determining Ground Types

AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)

(1) Site Class Definition

A site is classified as A through F in accordance with the site class definitions in Table 5-1.

Table 5-1 Site Class Definitions



Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft/s
B	Rock with 2,500 ft/sec < $\bar{v}_s$ < 5,000 ft/s
C	Very dense soil and soil rock with 1,200 ft/sec < $\bar{v}_s$ < 2,500 ft/s, or with either $\bar{N}$ > 50 blows/ft, or $\bar{s}_u$ > 2.0 ksf
D	Stiff soil with 600 ft/s < $\bar{v}_s$ < 1,200 ft/s, or with either 15 < $\bar{N}$ < 50 blows/ft, or 1.0 < $\bar{s}_u$ < 2.0 ksf
E	Soil profile with $\bar{v}_s$ < 600 ft/s or with either $\bar{N}$ < 15 blows/ft or $\bar{s}_u$ < 1.0 ksf, or any profile with more than 10 ft of soft clay defined as soil with $PI > 20$, $w > 40$ percent and $\bar{s}_u$ < 0.5 ksf
F	Soils requiring site-specific evalutions, such as: • Peats or highly organic clays (H > 10 ft of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays (H > 25 ft with PI > 75) • Very thick soft/medium stiff clays (H >120 ft)

Exceptions: Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site classes E or F should not be assumed unless the authority having jurisdiction determines that site classes E or F could be present at the site or in the event that site classes E or F are established by geotechnical data.

where:

$\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile
$\bar{N}$ = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for the upper 100 ft of the soil profile
$\bar{s}_u$ = average undrained shear strength in ksf (ASTM D2166 or ASTM D2850) for the upper 100 ft of the soil profile
PI = plasticity index (ASTM D4318)
w = moisture content (ASTM D2216)

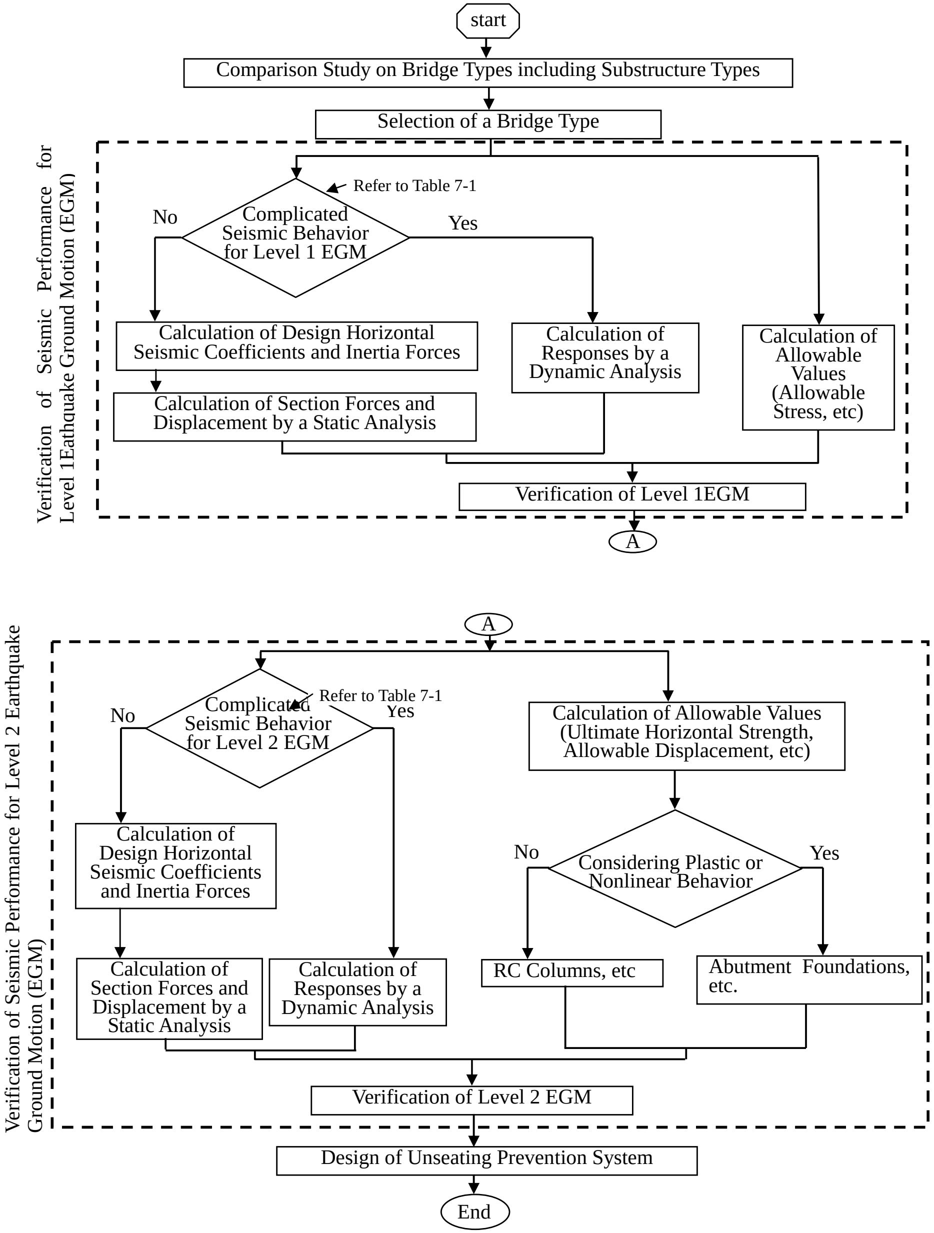
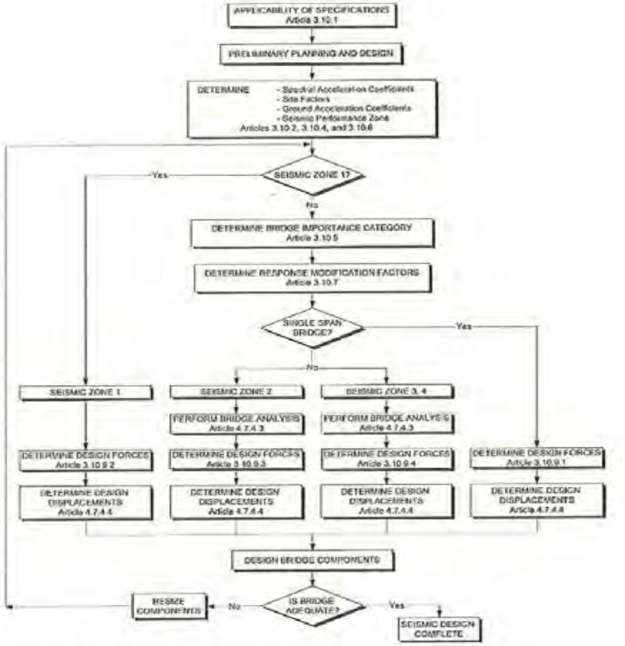
<table>
<tr><th>Items</th><th>JRA (Part V; English Version, 2002)</th><th>AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)</th></tr>
<tr><td>6. Procedure of Seismic Design</td><td>
</td><td>

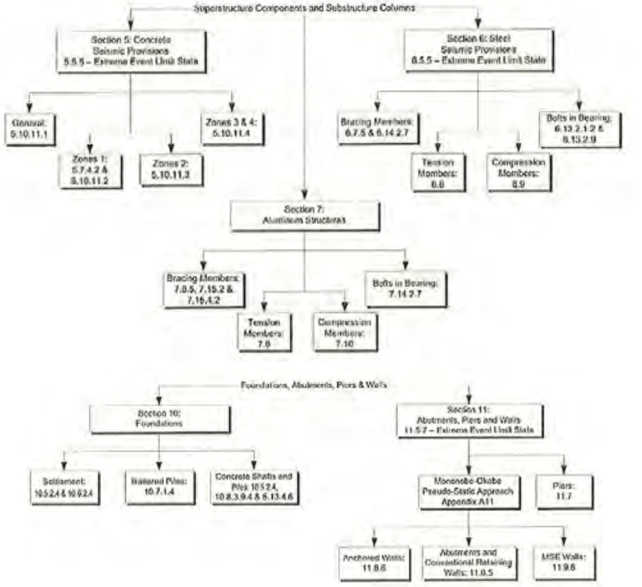
Figure 6-1 Seismic Design Procedure Flow Chart


Figure 6-2 Seismic Detailing and Foundation Design Flow Chart</td></tr>
</table>

Items

7. Limit States of Bridges for each Seismic Performance Level

JRA (Part V; English Version, 2002)

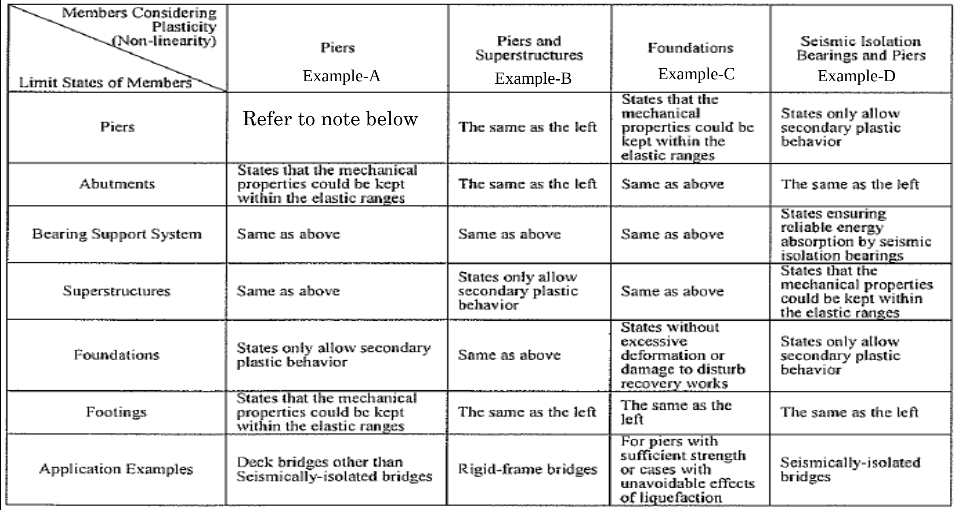
(1) Limit States of Bridges for Seismic Performance Level 1

(a) The mechanical properties of the bridges including expansion joint are maintained within the elastic range.

(b) Stress occurring in concrete of each structural member reaches its allowable stress multiplied by an increase factor of 1.5 for consideration of earthquake effects.

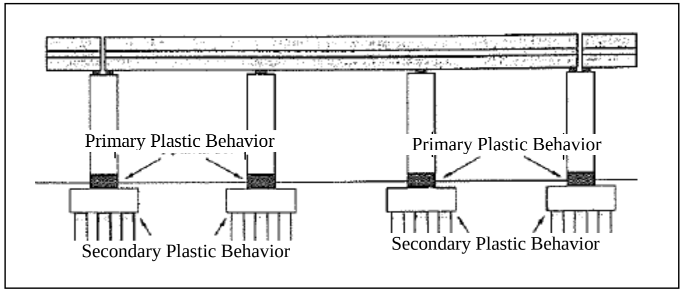
(2) Limit State of Bridges for Seismic Performance Level 2 (Refer to table 7-1 and Fig. 7-1)

Table 7-1 Limit States of Each Member with Applicable Examples for Seismic Performance Level 2 and Level 3 (Refer to Fig. 7-1 about Examples)



Members Considering Plasticity (Non-linearity) / Limit States of Members	Piers Example-A	Piers and Superstructures Example-B	Foundations Example-C	Seismic Isolation Bearings and Piers Example-D
Piers	Refer to note below	The same as the left	States that the mechanical properties could be kept within the elastic ranges	States only allow secondary plastic behavior
Abutments	States that the mechanical properties could be kept within the elastic ranges	The same as the left	Same as above	The same as the left
Bearing Support System	Same as above	Same as above	Same as above	States ensuring reliable energy absorption by seismic isolation bearings
Superstructures	Same as above	States only allow secondary plastic behavior	Same as above	States that the mechanical properties could be kept within the elastic ranges
Foundations	States only allow secondary plastic behavior	Same as above	States without excessive deformation or damage to disturb recovery works	States only allow secondary plastic behavior
Footings	States that the mechanical properties could be kept within the elastic ranges	The same as the left	The same as the left	The same as the left
Application Examples	Deck bridges other than Seismically-isolated bridges	Rigid-frame bridges	For piers with sufficient strength or cases with unavoidable effects of liquefaction	Seismically-isolated bridges

Note: States within a range of easy functional recovering for Seismic Performance Level 2; States that horizontal strength of piers start to get reduced rapidly for Seismic Performance Level 3.



(a) Example A: Single Column Pier with Plastic Behavior (in longitudinal direction)

AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)

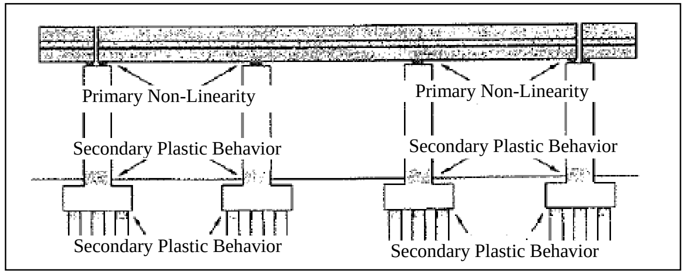
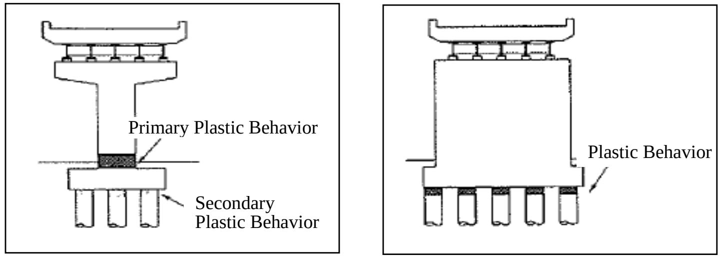
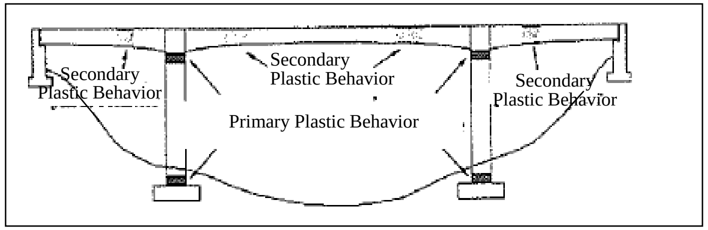
(1) The Guide Specifications are intended to achieve minimal damage to bridges during moderate earthquake ground motions and to prevent collapse during rare earthquakes that results in high levels of ground shaking at the bridge site.

(2) Bridges are designed to have life safety performance objective considering a seismic hazard corresponding to a seven percent probability of exceedance in 75 years. Life safety for the design event shall be taken to imply that the bridge has low probability of collapse but may suffer significant damage and that significant disruption to service is possible.

(3) The approach considers "*ductile substructure with essentially elastic superstructure*". This includes conventional plastic hinging in columns and walls and abutments that limits inertial forces by full mobilization of passive soil resistance.

Table 7-1 Limit States of Members for Performance Level

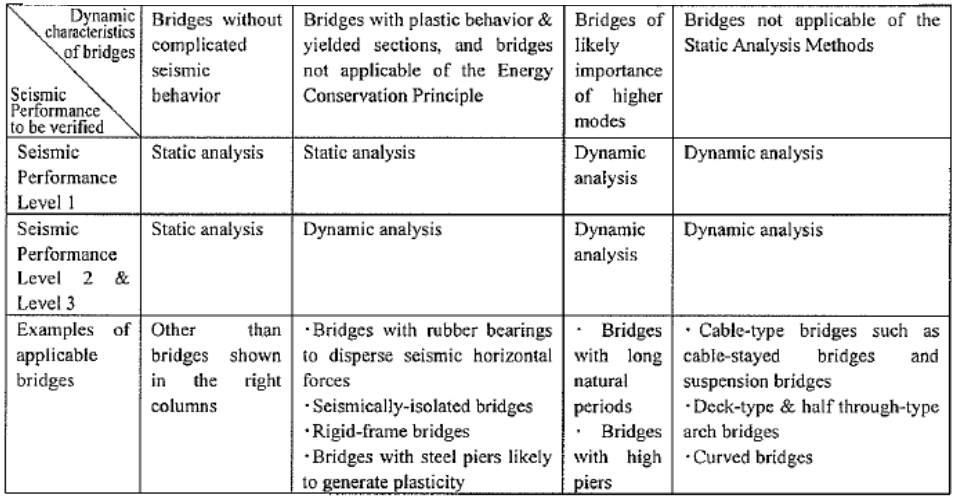
Members	Limit States
Bearings	• At the extreme event limit state, bearings which are designed to act as fuses or sustain irreparable damage may be permitted provided loss of span is prevented. • For rigid bearings and deformable bearings, mechanical properties should be kept at elastic level • For seismic isolation type bearing, reliable energy absorption mechanism should be ensured.
Piers	Formation of plastic hinges are allowed without bridge collapse
Foundation	• Basically kept at the elastic range. • When considering liquefaction induced effects, inelastic deformation is allowed.
Footings	• Basically kept at elastic range.
Abutments	• Basically kept at elastic range.
Superstructure	• Basically kept at elastic range.

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)
	 (b) Example D: Seismic Isolation Bearing with Consideration of Non-Linearity(in longitudinal direction)  (c) Example A: Single Column Pier with Plastic Behavior (in Transverse direction) (d) Example C: Foundations with Plastic Behavior (Pier Wall, in Transverse direction)  (e) Example B: Plasticity in Piers and Superstructures (Rigid-Frame Bridges in Transverse direction) **Fig. 7-1** Limit States of Each Member with Applicable Examples for Seismic Performance Level 2 and Level 3	
8. Design Method Applicable for Seismic Performance Verification	The bridge types and design methods applicable to seismic performance verification is summarized in Table 3. ➢ Although dynamic analysis methods can be applied to bridges without complicated seismic behavior, it is recommended to use static analysis methods because the verification in accordance with static method is generally feasible for these bridges. ➢ Since the seismic behavior of bridges with predominant first mode of vibration and plural plastic behavior or bridges in which investigation on application of Energy Conservation Principle remains unclear may become complicated due to plasticity of members, their Seismic Performance Level 1should be verified by the static analysis methods but Seismic Performance Level 2 or Level 3 be verified by dynamic methods.	(1) It should be demonstrated that a clear, straightforward load path to the substructure exists and that all components and connections are capable of resisting the imposed load effects consistent with the chosen load paths. A viable load path shall be established to transmit lateral loads to the foundation based on the stiffness characteristics of the deck, diaphragms, cross-frames, and lateral bracing. (2) The selection of the method of analysis depends on seismic zone, regularity, and operational classification of the bridge. Minimum analysis requirements for seismic effects are specified in Table 8-1, in which:

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)

JRA (Part V; English Version, 2002)

Table 8-1 Relationship between Complexities of Seismic Behavior and Design Methods Applicable for Seismic Performance Verification



Dynamic characteristics of bridges / Seismic Performance to be verified	Bridges without complicated seismic behavior	Bridges with plastic behavior & yielded sections, and bridges not applicable of the Energy Conservation Principle	Bridges of likely importance of higher modes	Bridges not applicable of the Static Analysis Methods
Seismic Performance Level 1	Static analysis	Static analysis	Dynamic analysis	Dynamic analysis
Seismic Performance Level 2 & Level 3	Static analysis	Dynamic analysis	Dynamic analysis	Dynamic analysis
Examples of applicable bridges	Other than bridges shown in the right columns	･Bridges with rubber bearings to disperse seismic horizontal forces ･Seismically-isolated bridges ･Rigid-frame bridges ･Bridges with steel piers likely to generate plasticity	･ Bridges with long natural periods ･ Bridges with high piers	･ Cable-type bridges such as cable-stayed bridges and suspension bridges ･Deck-type & half through-type arch bridges ･Curved bridges

AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)

Table 8-1 Minimum Analysis Requirements for Seismic Effects

Seismic Zone	Single Span Bridges	Multi-span Bridges					
		Other Bridges		Essential Bridges		Critical Bridges	
		regular	irregular	regular	irregular	regular	irregular
1	No detailed seismic analysis required	*	*	*	*	*	*
2		SM/UL	SM	SM/UL	MM	MM	MM
3		SM/UL	MM	MM	MM	MM	TH
4		SM/UL	MM	MM	MM	TH	TH

* = no seismic analysis required
UL = uniform load elastic method
SM = single-mode elastic method
MM = multi-mode elastic method
TH = time history method

(3) The requirements to satisfy as regular bridges are given in Table 8-2, otherwise it shall be taken as “irregular” bridges.

Table 8-2 Regular Bridge Requirements

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle for a curved bridge	90°	90°	90°	90°	90°
Maximum span length ratio from span to span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span to span, excluding abutment	-	4	4	3	2

Items: 9. Calculation of Natural Period

JRA (Part V; English Version, 2002)

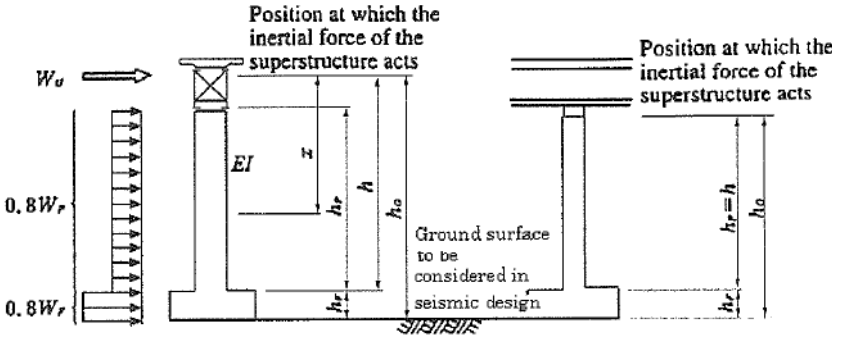
(1) Natural periods shall be appropriately calculated with considering of the effects of deformations of structural members and foundations.

(2) Natural Period of the Design Vibration Unit (s) (T (s))

$$T = 2.01 * \sqrt{\delta} \quad \text{------------------} \ (9\text{-}1)$$

where, δ can be calculated as follows.

(a) in case of a design vibration unit consisting of substructure and its supporting superstructure part as shown in Fig. 9-1



(a) Transverse Direction

(b) Longitudinal Direction

Fig. 9-1 Calculation Model of Natural Period for A Design Vibration Unit Consisting of One Substructure and its Supporting Superstructure Part

AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)

(1) Natural period calculation by **Single Mode Spectral Method**

The single-mode method of spectral analysis shall be based on the fundamental mode of vibration in either the longitudinal or transverse direction. For regular bridges, the fundamental modes of vibration in the horizontal plane coincide with the longitudinal and transverse axes of the bridge structure. This mode shape may be found by applying a uniform horizontal load to the structure and calculating the corresponding deformed shape. The natural period may be calculated by equating the maximum potential and kinetic energies associated with the fundamental mode shape. The amplitude of the displaced shape may be found from the elastic seismic response coefficient, C_{sm}, and the corresponding spectral displacement. This amplitude shall be used to determine force effects.

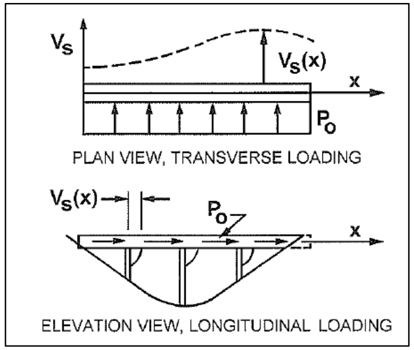


Figure 9-1 Bridge Deck Subjected to Assumed Transverse and Longitudinal Loading

- Calculate the static displacement $v_s(x)$ due to an assumed uniform loading p_o as shown in Figure 9-1.
- Calculate factors a and g as:

$$\alpha = \int v_s(x)dx \qquad (9\text{-}1)$$

$$\gamma = \int w(x)v_s^2(x)dx \qquad (9\text{-}2)$$

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)

$$\delta = \delta_P + \delta_0 + \theta_0 h_0 \quad \cdots\cdots \text{(9-2)}$$

where

δ_P: Bending deformation of substructure body (m)

δ_0: Lateral displacement of foundation (m)

θ_0: Rotation angle of foundation (rad)

h_0: Height from the ground surface to be considered in seismic design to the height of superstructural inertia force (m)

When the body of the substructure has a uniform section, the bending deformation δ_P can be calculated by Eq (9-3).

$$\delta_P = \frac{W_U h^3}{3EI} + \frac{0.8\, W_P h_P{}^3}{8EI} \quad \cdots\cdots \text{(9-3)}$$

where

W_U: Weight of the superstructure portion supported by the substructure body concerned (kN)

W_p : Weight of the substructure body (kN)

EI: Bending stiffness of the substructure body specified in the explanations of (1) above (kN · m^2).

h: Height from the bottom of the substructure body to the height of the superstructural inertia force (m)

h_P: height of the substructure body (m)

Where, δ0 and θ0 are calculated from Eq.(9-4) (Refer to Fig.9-2).

$$\left.\begin{aligned} \delta_0 &= \frac{H_0 A_{rr} - M_0 A_{sr}}{A_{ss} A_{rr} - A_{sr} A_{rs}} \\ \theta_0 &= \frac{-H_0 A_{rs} + M_0 A_{ss}}{A_{ss} A_{rr} - A_{sr} A_{rs}} \end{aligned}\right\} \text{---------(9-4).}$$

$$\left.\begin{aligned} H_0 &= W_U + 0.8(W_P + W_F) \\ M_0 &= W_U h_0 + 0.8 W_P\left(\frac{h_P}{2} + h_F\right) + 0.8 W_F \frac{h_F}{2} \end{aligned}\right\}$$

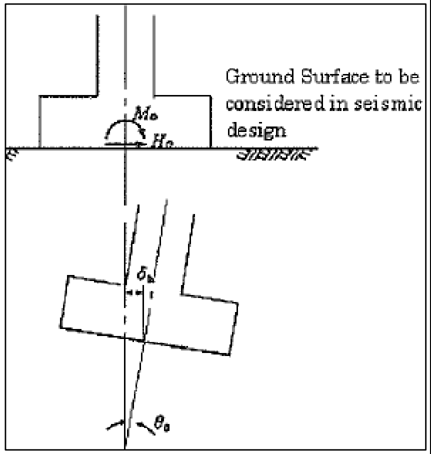


Fig. 9-2 Load and Displacement at Ground Surface for Seismic Design

Where, δ0 and θ0 are calculated from Eq. (9-4) (Refer to Fig.9-2). Arr, Asr, Ars and Ass are spring constants (kN/m, kN/rad, kNm/m, kNm/rad) and calculated from the following formula according to the foundation types.

where:

p_o = a uniform load arbitrarily set equal to 1.0 (N/mm)

$v_s(x)$ = deformation corresponding to p_o (mm)

$w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure (N/mm)

- Calculate the period of the bridge as:

$$T_m = 2\pi \sqrt{\frac{\gamma}{p_o g \alpha}} \qquad \text{(9-3)}$$

where: g = acceleration of gravity (m/sec^2)

(2) Natural period calculation by **Uniform Load Method**

The uniform load method shall be based on the fundamental mode of vibration in either the longitudinal or transverse direction of the base structure. The period of this mode of vibration shall be taken as that of an equivalent single-mass oscillator. The stiffness of this equivalent spring shall be calculated using the maximum displacement that occurs when an arbitrary uniform lateral load is applied to the bridge. The elastic seismic response coefficient, C_{sm}, shall be used to calculate the equivalent uniform seismic load from which force effects are found.

This method is essentially an equivalent static method of analysis that uses the uniform lateral load to approximate the effect of seismic loads. This method is suitable for regular bridges that respond principally in their fundamental mode of vibration. Whereas all displacements and most member forces are calculated with good accuracy, the method is known to overestimate the transverse shears at the abutments by up to 100 percent.

- Calculate the static displacement $v_s(x)$ due to an assumed uniform loading p_o as shown in Figure 9-1. The uniform load p_o is applied over the length of the bridge; it has units of force per unit length and may be arbitrarily set equal to 1.0. The static displacement $v_s(x)$ has unit of length.
- Calculate the bridge lateral stiffness, K, and the total weight, W, from the following expressions:

$$K = \frac{p_o L}{v_{sMAX}} \qquad \text{(9-4)}$$

$$W = \int w(x)dx \qquad \text{(9-5)}$$

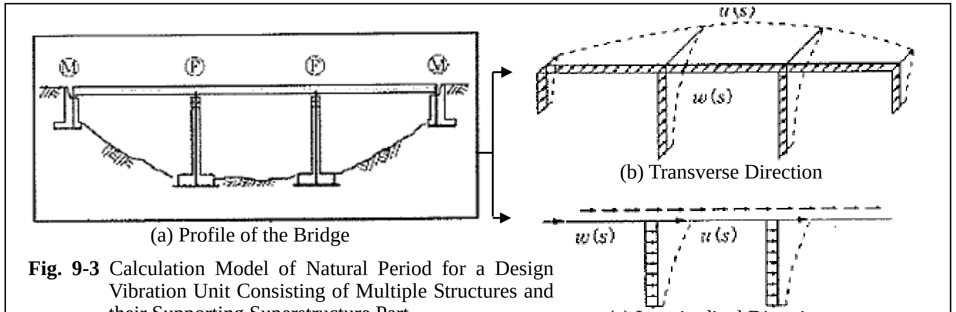
where:

L = total length of the bridge (mm)

v_{sMAX} = maximum value of $v_s(x)$ (mm)

$w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure (N/mm)

The weight should take into account structural elements and other relevant loads including, but not limited to, pier caps, abutments, columns and footings. Other loads, such as live loads may be included.

Items	**JRA (Part V; English Version, 2002)**	**AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)**
	① Spread foundation $\left.\begin{aligned} A_{ss} &= k_{SB}A_B \\ A_{sr} &= A_{rs} = 0 \\ A_{rr} &= k_V I_B \end{aligned}\right\}$ ------ (9-5) ② Pile foundation with only vertical piles arranged symmetrically $\left.\begin{aligned} A_{ss} &= nK_1 \\ A_{sr} &= A_{rs} = -nK_2 \\ A_{rr} &= nK_4 + K_{VP}\sum_{i=1}^{n} y_i^2 \end{aligned}\right\}$ --------- (9-6) where k_{SB} : Coefficient of shear subgrade reaction in horizontal direction at the bottom of foundation (kN/m^3) k_V : Coefficient of subgrade reaction in vertical direction at the bottom of foundation (kN/m^3) n : Total number of piles y_i : y coordinate of the pile head of i-th pile (m) K_1, K_2 K_3 K_4: Spring constants (kN/m, kN/rad, kN·m/m, kN·m/rad) of the piles in the perpendicular direction to the pile axis in case of rigid connection at pile head K_{VP}: Spring constant (kN/m) of the piles in the direction of the pile axis Concrete calculation methods of K1, K2, K3, K4 and Kvp are provided in Part IV. With the coefficients of subgrade reaction for seismic design as shown in Eq. (9-7) and (9-8), K1, K2, K3, K4 and Kvp .can be obtained. $K_{H0} = 1/0.3 * E_D$ ---------- (9-7) $K_{V0} = 1/0.3 * E_D$ ---------- (9-8) where, K_{H0}, K_{V0} = Reference values of the coefficient of subgrade reaction in horizontal direction and in vertical direction, respectively (kN/m^3) (for Level 1 and 2 EGMs) E_D = 2 * (1 + ν_D) * G_D (E_D: Dynamic modulus of deformation of the ground (kN/m^2)) ν_D = Dynamic Poisson's ratio of the ground G_D = γt/g * V^2_{SD} (G_D: Dynamic shear deformation modulus of the ground (kN/m^2)) γt = Unit weight of the ground (kN/m^3) g = Acceleration of gravity (9.8 m/s^2) V_{SD} = Shear elastic wave velocity of the ground (m/s) V_{SDi} = CV * Vsi (V_{SDi}: the average shear elastic wave velocity of the i-th layer) C_V = 0.8 (Vsi < 300 m/s), 1.0 (Vsi ≥ 300 m/s) (CV: Modification factor based on degree of ground strain) Vsi = the average shear elastic wave velocity of thei-th soil layer described in Item 5 (m/s) <u>(b) in case of a design vibration unit consisting of multiple substructures and their supporting superstructure part</u> as shown in Fig. 9-3.  (a) Profile of the Bridge (b) Transverse Direction (c) Longitudinal Direction **Fig. 9-3** Calculation Model of Natural Period for a Design Vibration Unit Consisting of Multiple Structures and their Supporting Superstructure Part	- Calculate the period of the bridge, T_m, using the expression: $T_m = 2\pi\sqrt{\dfrac{W}{gK}}$ (9-6) where: g = acceleration of gravity (m/sec^2) (3) Natural Frequencies For the elastic dynamic response analysis, all relevant damped modes and frequencies shall be considered.

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)

$$\delta = \frac{\int w(s)\, u(s)^2 ds}{\int w(s)\, u(s)\, ds} \quad \cdots\cdots \quad (9\text{-}9)$$

where,

$w(s)$: Weight of the superstructure or the substructure at position s (kN/m)

$u(s)$: Lateral displacement of each structure at position s in the direction of the inertia force when a lateral force corresponding to the weight of the superstructure and that of the substructure above the design ground surface act in the direction of the inertia force (m)

When a bridge is modeled into a discrete skeleton structure, the δ can be obtained from Eq. (9-10).

$$\delta = \frac{\sum_i (W_i u_i^2)}{\sum_i (W_i u_i)} \quad \cdots\cdots \quad (9\text{-}10)$$

where

W_i: Weight of superstructure and substructure at node i (kN)

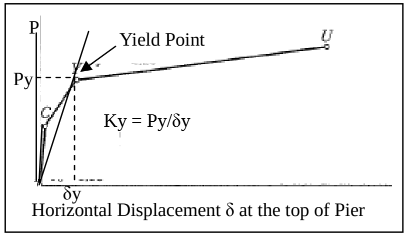
u_i: Displacement of node i occurred in the acting direction of inertia force when a force corresponding to the weight of the superstructure and the substructure above the ground surface to be considered in seismic design is acted in the acting direction of inertia force(m)

∑ represents the sum of all design vibration units.

When calculated with eigenvalue analysis, .the natural period (T) can be obtained directly.

(c) Stiffness Applied to Calculation of Natural Period for Level 2 EGM

During verification of seismic performance for Level 2 EGM, the natural period shall be calculated based on the yield stiffness. The yield stiffness refers to the secant stiffness Ky at yield point due to bending deformation of the pier and is obtained at the ratio of the yield strength Py to the yield displacement δy of the pier (Ky = Py/δy) as shown in Fig. 9-4.



⟹

$Ky = 3*E*Iy / h^3$ ⟹ $Iy = Ky * h^3 / 3E$

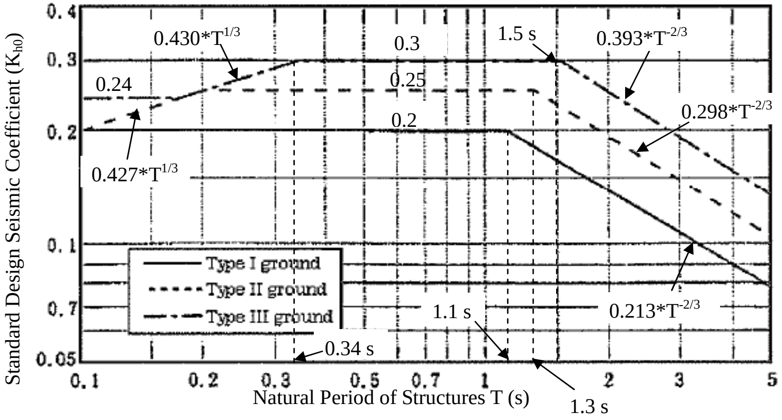
EIy = yield stiffness

E = elastic modulus of pier, Iy = moment of section inertia at the yield point

h = the height of pier

(Iy should be used for every calculation of the natural period for verification of seismic performance for Level 2 EGM)

Fig. 9-4 Stiffness Applied to Calculation of Natural Period for Level 2 EGM

Items	**JRA (Part V; English Version, 2002)**	**AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)**
10. Design Horizontal Seismic Coefficient for Level 1 EGM	(1) Design horizontal coefficient (k_h) for Level 1 EGM shall be calculated by Eq. (10-1). $K_h = C_Z * K_{h0}$ (≥ 0.1) -- (10-1) where, K_{h0} = Standard value of the design horizontal seismic coefficient for Level 1 EGM, which is shown in Fig 10-1.(Fig. 10-1 is obtained from the Figure for Level 1 EGM shown in Item 4 by dividing S_0 by gravity acceleration) C_Z = Modification factor for zones shown in Item 4. (2) Design horizontal coefficient (K_{hg}) at ground level can be obtained from Eq. (10-2), which is used for calculation of inertia force due to soil weight and seismic earth pressure in verifying seismic performance for Level 1 EGM. $K_{hg} = C_Z * K_{hg0}$ -- (10-2) where, K_{hg0} = Standard value of the design horizontal seismic coefficient at ground surface level for Level 1 EGM = 0.16 for Ground Type I, 0.2 for Ground Type II and 0.24 for Ground Type III (3) Though a single value of the design horizontal seismic coefficient shall generally be adopted within the same design vibration unit, different design horizontal seismic coefficient for each pier are given in case that the ground type changes within the same design vibration unit.  **Fig. 10-1**Standard Design Seismic Coefficient for Level 1 EGM	No provision
11. Design Horizontal Seismic Coefficient for Level 2 EGM	(1) Design horizontal seismic coefficients for Level 2 EGM (Type I and II) shall be calculated by Eq. (11-1) and (11-2). $K_{hCI} = C_S * C_Z * K_{hC0I} \geq 0.3 * C_S$ or $0.4 * C_Z$ -- (11-1) $K_{hCII} = C_S * C_Z * K_{hC0II} \geq 0.6 * C_S$ or $0.4 * C_Z$ -- (11-2) K_{hCI} and K_{hCII} = Design horizontal seismic coefficient for Type I and II of Level 2 EGM, respectively. K_{hC0I} and K_{hC0II} = Standard design horizontal seismic coefficients for Type I and Type II of Level 2 EGM, respectively, which are shown in Fig 11-1 and Fig. 11-12. C_S = Force Reduction Factor related to the extent of ductility of a pier, which is specified in Item 12.	(1) Equivalent static earthquake loading by **Single-Mode Spectral Method** The equivalent static earthquake loading $p_e(x)$ is calculated as: $p_e(x) = \frac{\beta C_{sm}}{\gamma} w(x)v_s(x)$ (11-1) $\gamma = \int w(x)v_s^2(x)dx$ (11-2) $\beta = \int w(x)v_s(x)dx$ (11-3)

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)

(2) Design horizontal coefficients at ground surface level can be obtained from Eq. (11-3) and (11-4) for Type I and II of Level 2 EGM

$K_{hgI} = C_Z * K_{hgI0}$ --- (11-3)
$K_{hgII} = C_Z * K_{hgII0}$ --- (11-4)

KhgI and KhgII = Design horizontal seismic coefficients at ground surface for Type I and II of Level 2 EGM, respectively.
K_{hgI0} and K_{hgII0} = Standard horizontal seismic coefficient at ground surface level for Type I and II of Level 2 EGM, respectively.
K_{hgI0} = 0.3 for Ground Type I, 0.35 for Ground Type II and 0.40 for Ground Type III
K_{hgII0} = 0.80 for Ground Type I, 0.70 for Ground Type II and 0.60 for Ground Type III

(3) The highest value of design horizontal seismic coefficient shall generally be used in each design vibration unit.

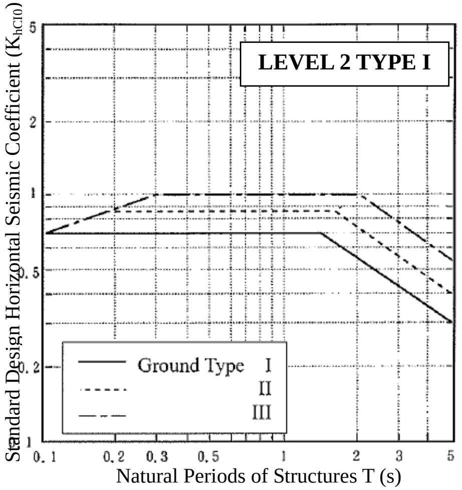


Fig. 11- 1 Relationship between K_{hCI0} and T (s)

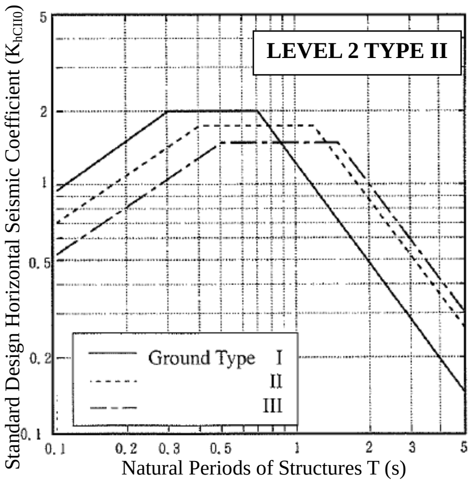
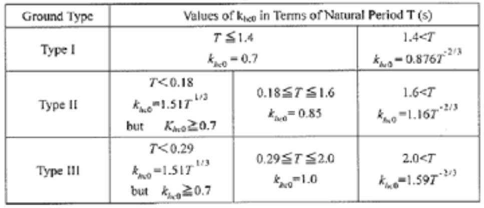


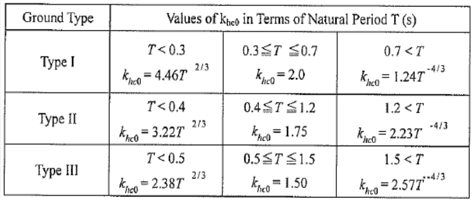
Fig. 11- 2 Relationship between K_{hCII0} and T (s)

Table 11-1 Relationship between K_{hCI0} and T (s)



Ground Type	Values of k_{hc0} in Terms of Natural Period T (s)		
Type I	$T \leqq 1.4$ $k_{hc0} = 0.7$		$1.4<T$ $k_{hc0} = 0.876T^{-2/3}$
Type II	$T<0.18$ $k_{hc0}=1.51T^{1/3}$ but $K_{hc0} \geqq 0.7$	$0.18 \leqq T \leqq 1.6$ $k_{hc0}= 0.85$	$1.6<T$ $k_{hc0} =1.16T^{-2/3}$
Type III	$T<0.29$ $k_{hc0} =1.51T^{1/3}$ but $k_{hc0} \geqq 0.7$	$0.29 \leqq T \leqq 2.0$ $k_{hc0}=1.0$	$2.0<T$ $k_{hc0}=1.59T^{-2/3}$

Table 11-2 Relationship between K_{hCII} and T (s)



Ground Type	Values of k_{hc0} in Terms of Natural Period T (s)		
Type I	$T < 0.3$ $k_{hc0} = 4.46T^{2/3}$	$0.3 \leqq T \leqq 0.7$ $k_{hc0} = 2.0$	$0.7 < T$ $k_{hc0} = 1.24T^{-4/3}$
Type II	$T < 0.4$ $k_{hc0} = 3.22T^{2/3}$	$0.4 \leqq T \leqq 1.2$ $k_{hc0} = 1.75$	$1.2 < T$ $k_{hc0} = 2.23T^{-4/3}$
Type III	$T < 0.5$ $k_{hc0} = 2.38T^{2/3}$	$0.5 \leqq T \leqq 1.5$ $k_{hc0} = 1.50$	$1.5 < T$ $k_{hc0} = 2.57T^{-4/3}$

where:

$p_e(x)$ = the intensity of the equivalent static seismic loading applied to represent the primary mode of vibration (N/mm)
$v_s(x)$ = deformation corresponding to p_o (mm)
$w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure (N/mm)
C_{sm} = the dimensionless elastic seismic response coefficient given by:

$$C_{sm} = A_S + (S_{DS} - A_S)(T_m/T_o) \qquad (11\text{-}4)$$

in which:

$$A_S = F_{pga}PGA \qquad (11\text{-}5)$$

$$S_{DS} = F_aS_S \qquad (11\text{-}6)$$

where:

PGA = peak ground acceleration coefficient on rock (Site Class B)
S_S = horizontal response spectral acceleration coefficient at 0.2-sec period on rock (Site Class B)
T_m = period of vibration of m^{th} mode (sec)
T_o = reference period used to define spectral shape = 0.2 T_S (sec)
T_S = corner period at which spectrum changes from being independent of period to being inversely proportional to period = S_{D1}/S_{DS} (sec)

For periods greater than or equal to T_o and less than or equal to T_S, the elastic seismic response shall be taken as:

$$C_{sm} = S_{DS} \qquad (11\text{-}7)$$

For periods greater than T_S, the elastic seismic response coefficient shall be taken as:

$$C_{sm} = S_{D1}/T_m \qquad (11\text{-}8)$$

in which:

$$S_{D1} = F_vS_1 \qquad (11\text{-}9)$$

where:

S_1 = horizontal response spectral acceleration coefficient at 1.0-sec period on rock (Site Class B)

- Apply loading $p_e(x)$ to the structure and determine the resulting member force effects

<table>
<tr><th>Items</th><th>JRA (Part V; English Version, 2002)</th><th>AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)</th></tr>
<tr><td></td><td></td><td>

(2) Equivalent static earthquake loading by **Uniform Load Method**

The equivalent static earthquake loading p_e is calculated from the expression:

$$p_e = \frac{C_{sm}W}{L} \qquad (11\text{-}10)$$

$$W = \int w(x)dx \qquad (11\text{-}11)$$

where:

C_{sm} = the dimensionless elastic seismic response coefficient (refer to Item 1 above)

p_e = equivalent uniform static seismic loading per unit length of bridge applied to represent the primary mode of vibration (N/mm)

L = total length of the bridge (mm)

$w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure (N/mm)

- Calculate the displacements and member forces for use in design either by applying p_e to the structure and performing a second static analysis or by scaling the results of the first step above the ratio p_e/p_o.

</td></tr>
<tr><td>12. Force Reduction Factor</td><td>

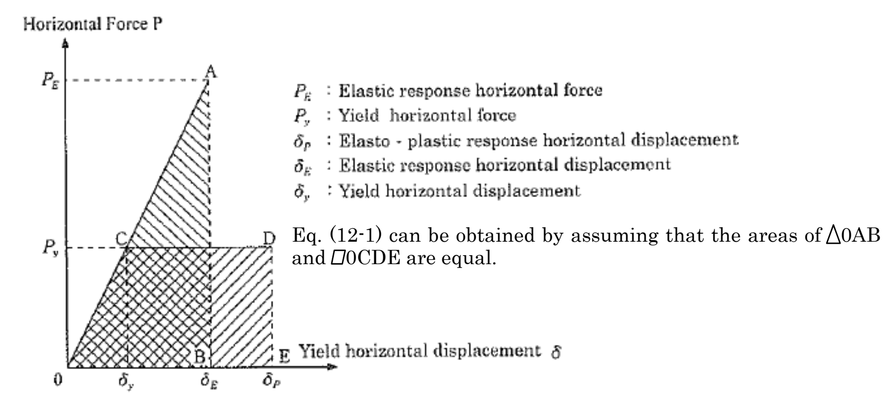
(1) Force reduction factor shall be calculated by Eq. (12-1) for a structural system that can be modeled as a one degree-of freedom vibration system having a plastic force- displacement relation.

$C_S = 1/\sqrt{2*\mu_a - 1}$ -- (12-1)

C_S = Force reduction factor (refer to Fig. 12-1)

μ_a = Allowable ductility ratio. μa can be obtained by Eq. (12-2) for the case of a RC column.

$\mu_a = 1 + (\delta u - \delta y)/\alpha*\delta y$ (refer to Fig. 12-2) ---(12-2)



Eq. (12-1) can be obtained by assuming that the areas of △0AB and □0CDE are equal.

Fig. 12-1 Elasto-Plastic Response Displacement of a Pier

</td><td>

(1) Seismic design force effects for substructures and the connection between parts of structures shall be determined by dividing the force effects resulting from elastic analysis by the appropriate response modification factor, R, as specified in Tables 12-1 and 12-2.

(2) As an alternative to the use of R-factors, specified in Table 13-2 for connections, monolithic joints between structural members and/or structures, such as column-to-footing connection, may be designed to transmit the maximum force effects that can be developed by the inelastic hinging of the column or multicolumn bent the connect.

(3) If an inelastic time history method of analysis is used, the response modification factor, R, shall be taken as 1.0 for all substructure and connections.

Table 12-1 Response Modification Factors – Substructures

<table>
<tr><th rowspan="2">Substructure</th><th colspan="3">Operation Category</th></tr>
<tr><th>Critical</th><th>Essential</th><th>Other</th></tr>
<tr><td>Wall-type piers – larger dimension</td><td>1.5</td><td>1.5</td><td>2.0</td></tr>
<tr><td>Reinforced concrete pile bents
• Vertical piles only
• With batter piles</td><td>
1.5
1.5</td><td>
2.0
1.5</td><td>
3.0
2.0</td></tr>
<tr><td>Single columns</td><td>1.5</td><td>2.0</td><td>3.0</td></tr>
<tr><td>Steel or composite steel and concrete pile bents
• Vertical piles only
• With batter piles</td><td>
1.5
1.5</td><td>
3.5
2.0</td><td>
5.0
3.0</td></tr>
<tr><td>Multiple column bents</td><td>1.5</td><td>3.5</td><td>5.0</td></tr>
</table>

Table 12-2 Response Modification Factors – Connections

<table>
<tr><th>Connection</th><th>All Operational Categories</th></tr>
<tr><td>Superstructure to abutment</td><td>0.8</td></tr>
<tr><td>Expansion joints within a span of the superstructure</td><td>0.8</td></tr>
<tr><td>Columns, piers, or pile bents to cap beam or superstructure</td><td>1.0</td></tr>
<tr><td>Columns or piers to foundation</td><td>1.0</td></tr>
</table>

</td></tr>
</table>

<table>
<tr><th>Items</th><th>JRA (Part V; English Version, 2002)</th><th>AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)</th></tr>
<tr><td></td><td>

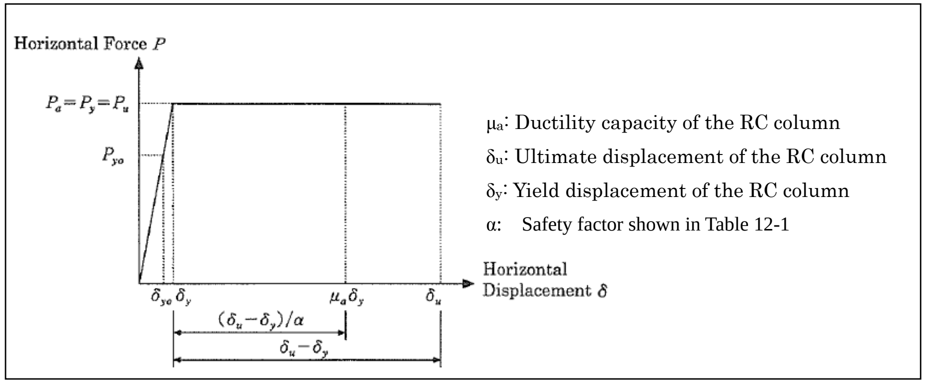


Fig. 12-2 Simplified Relationship between Lateral Strength and Ductility Capacity for Flexural Failure

Table 12-1 Safety Factor of RC Column resulting in Flexural Failure

<table>
<tr><th>Seismic Performance to be Verified</th><th>Safety Factor α in Calculation of Ductility Capacity for Type I Earthquake Ground Motion</th><th>Safety Factor α in Calculation of Ductility Capacity for Type II Earthquake Ground Motion</th></tr>
<tr><td>Seismic Performance Level 2</td><td>3.0</td><td>1.5</td></tr>
<tr><td>Seismic Performance Level 3</td><td>2.4</td><td>1.2</td></tr>
</table>

</td><td></td></tr>
<tr><td>13. Evaluation of Failure Mode of RC Column</td><td>

(1) Failure mode of a RC column shall be evaluated by Eq. (13-1)

$$\left.\begin{array}{lll} P_u \leqq P_s & : & \text{Flexural (or bending) failure} \\ P_s < P_u \leqq P_{s0} & : & \text{Shear failure after flexural yielding} \\ P_{s0} < P_u & : & \text{Shear failure} \end{array}\right\} \cdots\cdots\cdots\cdots \quad (13\text{-}1)$$

Pu = Lateral strength of a RC column
Ps = Shear strength of a RC column
Ps0 = Shear strength of a RC column calculated by the modification factor on the effects of repeated alternative loads is equal to 1.0.

</td><td>

(1) RC column shall satisfy Equation 13-1 for the Extreme Event Load Combination I.

$$\Sigma\eta_i\gamma_iQ_i \quad \leq \quad \phi R_n = R_r \qquad (13\text{-}1)$$

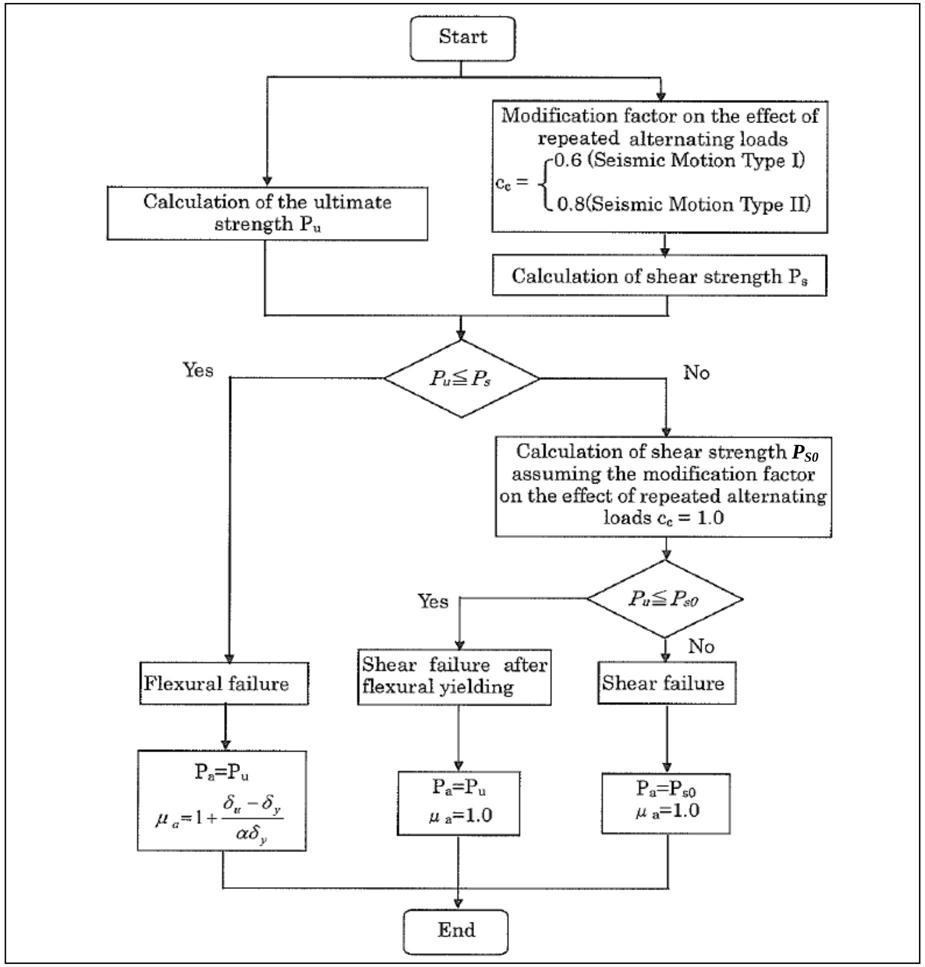
where:

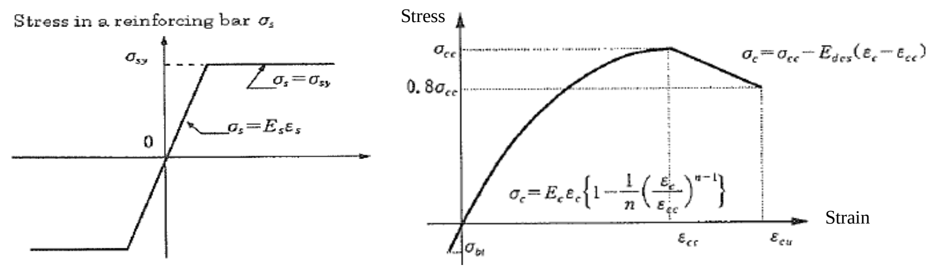
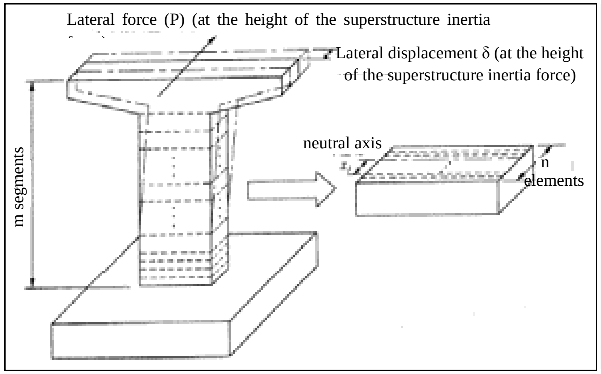
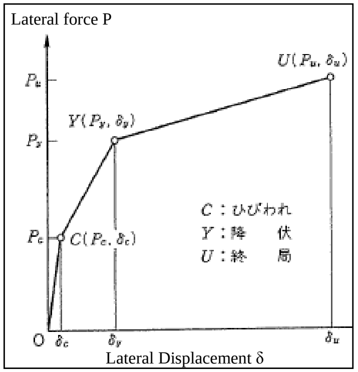
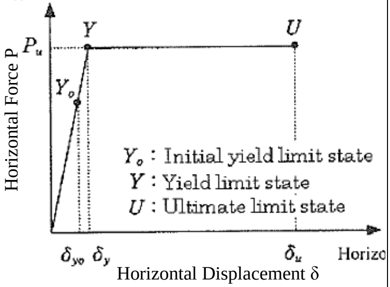
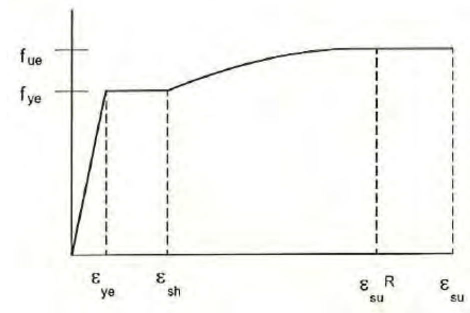
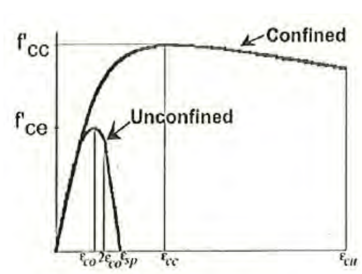
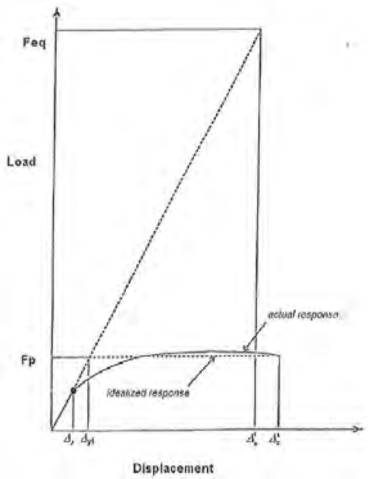
γ_i : load factor; statistically based multiplier applied to force effects
ϕ : resistance factor; statistically based multiplier applied to nominal resistance
η_i : load modifier; a factor relating to ductility, redundancy, and operational classification
Q_i : force effect
R_n : nominal resistance
R_r : factored resistance; ϕR_n

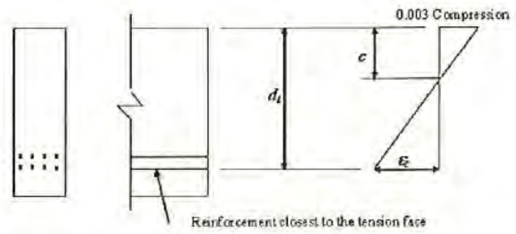
When inelastic hinging is invoked as a basis for seismic design, the force effects resulting from plastic hinging at the top and bottom of the column shall be calculated after the preliminary design of the column has been completed utilizing the modified design forces (using R factors) as seismic loads.

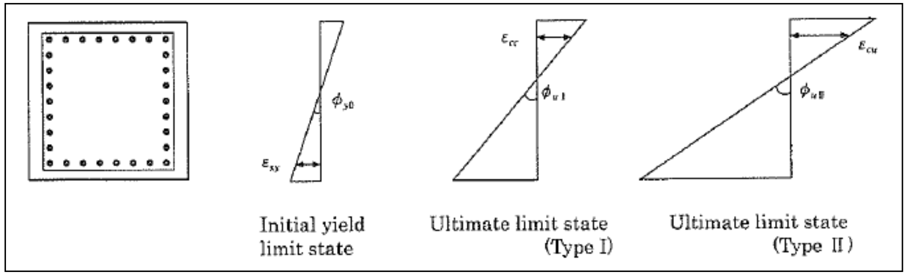
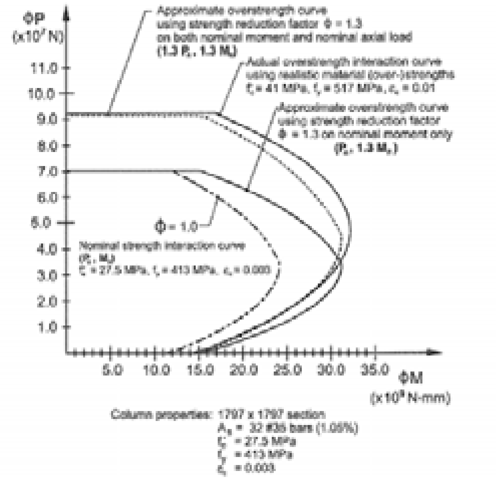
(2) Shear Failure
The shear mode of failure in a column or pile bent will probably result in a partial or total collapse of the bridge; therefore, the design shear force must be calculated conservatively. In calculating the column or pile bent shear force, consideration must be given to the potential locations of plastic hinges – such that the smallest potential column length be used with the plastic moments to calculate the largest potential shear force for design.

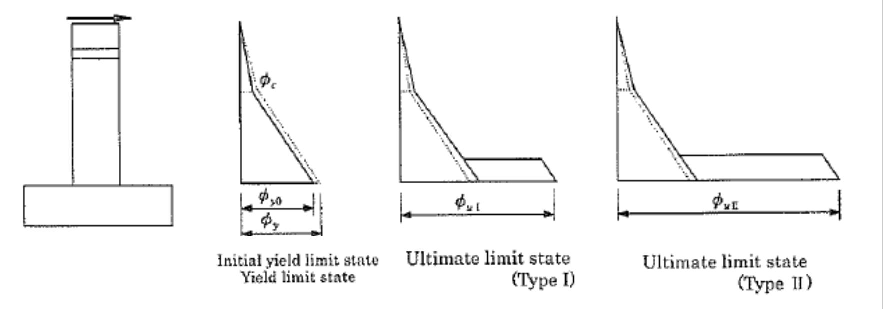
</td></tr>
</table>

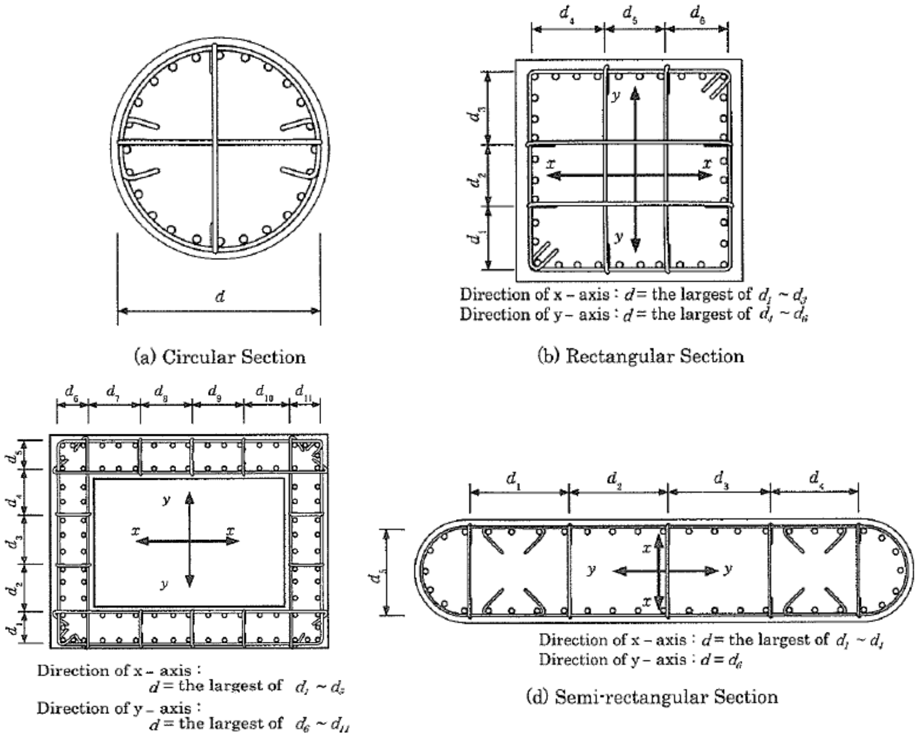
Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)
	(2) Failure mode for a RC column can be judged following the flow shown in Fig. 13-1.  Fig. 13-1 Evaluation of Failure Mode for a RC Column	(3) Lateral Strength The lateral strength of the pier corresponds to the total of inelastic hinging shear force demands at the top and bottom of the pier column/s formed by the column overstrength moment resistance (taken as the plastic moment).
14. Calculation of Lateral Strength and Displacement of a RC Column	(1) Relationships between stress and strain of a reinforcing bar and concrete are shown in Fig. 14-1 (1) and Fig. 14-1 (2), respectively. (2) RC column is divided into m segments along its height and the section of each segment is divided into n elements in the acting direction of the inertia force as shown in Fig. 14-2. With these relationships, Pu, Py, δu and δy at the height of the superstructure inertia force shown in Fig.14-3 can be obtained.	(1) Reinforcing steel is modeled with a stress-strain relationship that exhibits an initial elastic portion, a yield plateau, and a strain-hardening range in which the stress increases with strain as shown in Figure 14-1. On the other hand, the stress-strain model for confined and unconfined concrete is used to determine section response as shown in Figure 14-2.

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)
	 (1) Stress and Strain of Reinforcing Bar (2) Stress and Strain of Concrete **Fig 14-1** Relationships between Stress and Strain of Reinforcing Bar and Concrete   **Fig 14-2** Lateral Force (P) at the Acting Position of the Inertia Force and Displacement (δ), and Division of Column **Fig 14-3** Calculated Relationship between Lateral Force (P) and Displacement (δ) (3) Descriptions P_C = Lateral strength at cracking, P_y = Yielding lateral strength, P_u = Lateral strength, δ_y = Yield displacement, δu = Ultimate displacement (a single -column RC column) (a) Premises ● Fiber strain is proportional to the distance from the neutral axis. ● Skeleton curve between horizontal force and horizontal displacement shall be expressed by an ideal elasto-plastic model shown in Fig. 14-4. (b) Equations $P_c = \frac{W}{h}(\sigma_{bt} + \frac{N}{A})$ ----------------------- (14-1) $P_y = \frac{M_u}{h}$ ----------------------- (14-2) $\delta_y = \frac{M_u}{M_{y0}}\delta_{y0}$ ----------------------- (14-3) $P_u = \frac{M_u}{h}$ ----------------------- (14-4) $\delta_u = \delta_y + (\phi_u - \phi_y) L_p (h - L_p/2)$ -------- (14-5)  **Fig.14-4** Ideal Elasto-Plastic Model	 Figure 14-1 Reinforcing Steel Stress-Strain Model  Figure 14-2 Concrete Stress-Strain Model (2) Figure 14-3 illustrates the design based on the expected behavior of the bridge system which is the ductile substructure with essentially elastic superstructure. This system includes conventional plastic hinging in columns and walls and abutments that limits inertial forces by full mobilization of passive soil resistance. where: F_p : plastic force F_{eq} : elastic force Δ_y : idealized yield displacement Δ_{yi} : idealized yield displacement Δ^L_D : displacement demand Δ^L_C : displacement capacity  Figure 14-3 Design based on ductile substructure with essentially elastic superstructure (3) Assumptions for Strength and Extreme Event Limit States Factored resistance of concrete components shall be based on the conditions of equilibrium and strain compatibility, the resistance factor, and the following assumptions: • If concrete is unconfined, the maximum usable strain at the extreme concrete compression fiber is not greater than 0.003. • If concrete is confined, a maximum usable strain exceeding 0.003 in the confined core may be utilized and verified. Calculation of the factored resistance shall consider that the concrete cover may be lost at strains compatible with those in the confined concrete core. • Except in strut-and-tie model, the stress in the reinforcement is based on a stress-strain curve representative of the steel or an approved mathematical representation. • Tensile strength of concrete is neglected. • The concrete compressive stress-strain distribution is assumed to be rectangular, parabolic, or any other shape that results in a prediction of strength in substantial agreement with test results. • Balanced strain conditions exist at a cross-section when tension reinforcement reaches the strain corresponding to its specified yield strength fy just as the concrete compression reaches its assumed ultimate strain of 0.003.

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)
	(c) Description of Symbols W : Section modulus of a column with consideration of axial reinforcement at the column's bottom section (mm^3) σ_{bt} : Flexural tensile strength of concrete (N/mm^2) to be calculated by Eq. (14-1-1) $\sigma_{bt} = 0.23 \ \sigma_{ck}^{2/3}$ ………………………… (14-1-1) N : Axial force acting on the column's bottom section (N) A : Sectional area of a column, with consideration of axial reinforcement at the column's bottom section (mm^2) h : Height of superstructural inertial force from the bottom of column. (mm) σ_{ck} : Design strength of concrete (N/mm^2) δ_{y0} : Horizontal displacement at the time of yielding of axial tensile reinforcing bars at the outermost edge of the column's bottom section (called "initial yield displacement" hereafter) (mm) M_u : Ultimate bending moment at the column's bottom section (N· mm) M_{y0}: Bending moment at the time of yielding of axial tensile reinforcing bars at the outermost edge of the column's bottom section (N· mm) H : Height of superstructural inertial force from the bottom of column. (mm) L_p: Plastic hinge length (mm) calculated by Eq. (14-5-1) $L_p = 0.2h - 0.1D$ ………………………… (14-5-1) in which $0.1D \leq L_p \leq 0.5D$ D : Sectional depth (mm) (D shall be the diameter of a circular section, or the length of a rectangular section in the analytical direction) ϕ_y: Yield curvature at the column's bottom section (1/mm) ϕ_u : Ultimate curvature at the column's bottom section (1/mm) (d) Detailed Procedure to Obtain Mechanical Values Referring to Fig. 14-2, Fig. 14-5 and Fig. 14-6 $M_c = W_i(\sigma_{bt} + N_i/A_i)$ ………………………… (14-6) $\phi_c = M_c / E_c I_i$ ………………………… (14-7) M_c: Bending moment at cracking (N·mm) ϕ_c: Curvature of cracking (1/mm) W_i: Sectional modulus of pier having considered the axial reinforcement in the i-th section from the height of the superstructural inertia force (mm^3). σ_{bt}: Bending tensile strength of concrete (N/mm^2) to be calculated by Eq. (14-1-1) Ni: Axial force due to the weights of superstructure and substructure acting on the i-th section from the height of the superstructural inertia force (N). A_i: Sectional area of pier having considered the axial reinforcement in the i-th section from the height of the superstructural inertia force (mm^2)	• Sections are compression-controlled when the net tensile strain in the extreme tension steel is equal to or less than the compression-controlled limit at the time the concrete in compression reaches its assumed strain limit of 0.003. The compression-controlled strain limit is the net tensile strain in the reinforcement, at balanced strain conditions. For Grade 60 (476 MPa) reinforcement, and for all Prestressed reinforcement, the compression-controlled strain limit may be set equal to 0.002. • Sections are tension-controlled when the net tensile strain in the extreme tension steel is equal to or greater than 0.005 just as the concrete in compression reaches its assumed strain limit of 0.003. Sections with net tensile strain in the extreme tension steel between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections.  The net tensile strain in the extreme tension steel is determined from a linear strain distribution at nominal strength using similar triangles. Figure 14-4 Strain Distribution and Net Tensile Strain (4) Design Forces • Single Columns Axial Force : determined using Extreme Event Load Combination I with the unreduced maximum and minimum seismic axial load Moment : taken as the column overstrength moment resistance using a resistance factor of ϕ of 1.3 for reinforced concrete column and 1.25 for structural steel columns with the maximum elastic column axial load at Extreme Event Load Combination I. Shear Force : calculated based on the column overstrength moment resistance and the appropriate column height. • Piers with Two or More Columns Axial Force : maximum and minimum axial loads determined using Extreme Event Load Combination I with the axial loads determined by iterating the demand plastic shear from the column overstrength moment and the axial forces developed due tooverturning. Moment : taken as the column overstrength moment resistance, using a resistance factor of ϕ of 1.3 for reinforced concrete column and 1.25 for structural steel columns, corresponding to the maximum compressive axial load above. Shear Force : the shear force corresponding to the column overstrength moment resistances specified above. • Column and Pile Bent Axial Force : maximum and minimum design forces determined using Extreme Event Load Combination I with either the elastic design values determined from the analysis of the two perpendicular directions or the values corresponding to plastic hinging of the column. Moment : modified design moments determined for Extreme Event Limit State Load Combination I. Shear Force : the lesser of either the elastic design value determined for Extreme Event Limit State Load Combination I with the seismic loads combined for the two perpendicular directions and using an R factor of 1 for the column, or the value corresponding to plastic hinging of the column.

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)
	E_c: Young's modulus of concrete (N/mm^2) I_i: Moment of inertia of pier having considered the axial reinforcement in the i-th section from height of the superstructural inertia force (mm^4) σ_{ck} : Design standard strength of concrete (N/mm^2) $M_i = \sum_{j-1}^{n} \sigma_{cj} x_j \varDelta A_{cj} + \sum_{j=1}^{n} \sigma_{sj} x_j \varDelta A_{sj}$ -- (14-8) $\phi_i = \varepsilon_{c0} / x_0$ -- (14-9) σ_{cj}, σ_{sj}: Stress of concrete and reinforcement within the j-th infinitesimal part (N/mm^2) ΔA_{cj}, ΔA_{sj}: Sectional area of concrete and reinforcement within the j-th infinitesimal part (mm^2) M_i: Bending moment acting on the i-th section from the height of the superstructural inertia force (N·mm) ϕ_i: Curvature of the i-th section from the height of the superstructural inertia force (1/mm) x_j: Distance from concrete or reinforcement in the j-th infinitesimal part to the centroid position of section (mm) ε_{c0}: Compressed edge strain of concrete x_0: Distance from the compressed edge of concrete to the neutral axis (mm) $\delta_{y0} = \int \phi y \mathrm{d}y$ $\fallingdotseq \sum_{i=1}^{m} (\phi_i y_i + \phi_{i-1} y_{i-1}) \Delta y_i / 2$ · -- (14-10) $\phi_y = \left(\frac{M_u}{M_{y0}} \right) \phi_{y0}$ -- (14-11)  **Fig 14-5** Strain Distribution within Initial Yielding and Ultimate Limit	 Figure 14-5 Development of Approximate Overstrength Curves from Nominal Strength Curves after Gajer and Wagh (1994)

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)
	 **Fig 14-6** Curvature Distribution in the Direction of Height (e) Stress-Strain Relation of Concrete (refer to Fig. 14-1(2)) The stress-strain curve of concrete shall be determined by Eq. (14-12) based on Fig. 14-1(2)) $\sigma_c = \begin{cases} E_c\varepsilon_c\left\{1-\frac{1}{n}\left(\frac{\varepsilon_c}{\varepsilon_{cc}}\right)^{n-1}\right\} & (0 \leqq \varepsilon_c \leqq \varepsilon_{cc}) \\ \sigma_{cc} - E_{des}(\varepsilon_c - \varepsilon_{cc}) & (\varepsilon_{cc} < \varepsilon_c \leqq \varepsilon_{cu}) \end{cases}$ ………… (14-12) $n = \frac{E_c\varepsilon_{cc}}{E_c\varepsilon_{cc} - \sigma_{cc}}$ ………… (14-13) $\sigma_{cc} = \sigma_{ck} + 3.8\alpha\rho_s\sigma_{sy}$ ………… (14-14) $\varepsilon_{cc} = 0.002 + 0.033\beta\frac{\rho_s\sigma_{sy}}{\sigma_{ck}}$ ………… (14-15) $E_{des} = 11.2\frac{\sigma_{ck}^{\ 2}}{\rho_s\sigma_{sy}}$ ………… (14-16) $\varepsilon_{cu} = \begin{cases} \varepsilon_{cc} & (\text{For Type I Earthguake Ground Motion}) \\ \varepsilon_{cc} + \frac{0.2\sigma_{cc}}{E_{des}} & (\text{For Type II Earthguake Ground Motion}) \end{cases}$ ………… (14-17) $\rho_s = \frac{4A_h}{sd} \leqq 0.018$ ………… (14-18) σ_c : Stress of concrete (N/mm^2) σ_{cc} : Strength of concrete restrained by lateral confining reinforcement (N/mm^2) σ_{ck} : Design strength of concrete (N/mm^2) ε_c : Strain of concrete ε_{cc} : Strain of concrete under the maximum compressive stress ε_{cu} : Ultimate strain of concrete restrained by lateral confining reinforcement	

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)
	E_c : Young's modulus of concrete (N/mm^2) E_{des} : Descending gradient (N/mm^2) ρ_s : Volume ratio of lateral confining reinforcement A_h : Sectional area of each lateral confining reinforcement (mm^2) s : Spacings of lateral confining reinforcement (mm) d : Effective length (mm) of lateral confining reinforcement. It shall be the longest length of core concrete divided and restrained by lateral hoop ties or cross ties (refer to Fig. 14-7) σ_{sy}: Yield point of lateral confining reinforcement (N/mm^2) α,β: Modification factor on section. α=1.0,β=1.0 for a circular section, and α= 0.2, β= 0.4 for rectangular, hollow circular and hollow rectangular sections. n : A constant defined by Eq. (14-13)  (c) Hollow Section **Fig. 14-7** Effective Length of Lateral Confining Reinforcement (in Both Longitudinal and Transverse Direction to the Bridge Axis)	

<table>
<tr><th>Items</th><th>JRA (Part V; English Version, 2002)</th><th>AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)</th></tr>
<tr>
<td>15. Shear Strength (Concrete Structure)</td>
<td>

Shear strength shall be calculated by Eq. (15-1)

$$P_s = S_c + S_s \quad \text{(15-1)}$$

$$S_c = c_c c_e c_{pt} \tau_c b d \quad \text{(15-2)}$$

$$S_s = \frac{A_w \sigma_{sy} d(\sin\theta + \cos\theta)}{1.15a} \quad \text{(15-3)}$$

P_s : Shear strength (N)

S_c : Shear strength resisted by concrete (N)

τ_c : Average shear stress that can be borne by concrete (N/mm^2). Values in Table 15-1 shall be used.

c_c : Modification factor on the effects of alternating cyclic loading. c_c shall be taken as 0.6 for Type I Earthquake Ground Motion and 0.8 for Type II.

c_c : Modification factor in relation to the effective height (d) of a pier section. Values in Table 15-2 shall be used.

(b) Effective Height of a rectangular Section

c_{pt} : Modification factor in relation to the axial tensile reinforcement ratio p_t. Values in Table 15-3 shall be used.

b : Width of a pier section perpendicular to the direction in calculating shear strength (mm)

d : Effective height of a pier section parallel to the direction in calculating shear strength (mm)

p_t : Axial tensile reinforcement ratio. It is the value obtained by dividing the total sectional areas of the main reinforcement on the tension side of the neutral axis by bd (%).

S_s: Shear strength borne by hoop ties (N)

A_w: Sectional area of hoop ties arranged with an interval of α and an angle of θ (mm^2)

σ_{sy}: Yield point of hoop ties (N/mm^2)

θ : Angle formed between hoop ties and the vertical axis (degree)

α : Spacings of hoop ties (mm)

Table 15-1 Average Shear Stress of Concrete τc (N/mm^2)

<table>
<tr><td>Design Compressive Strength of Concreteσ_{ck} (N/mm^2)</td><td>21</td><td>24</td><td>27</td><td>30</td><td>40</td></tr>
<tr><td>Average Shear Stress of Concreteτ_c (N/mm^2)</td><td>0.33</td><td>0.35</td><td>0.36</td><td>0.37</td><td>0.41</td></tr>
</table>

Table 15-2 Modification Factor Ce in Relation to Effective Height of a Pier Section

<table>
<tr><td>Effective Height (mm)</td><td>Below 1000</td><td>3000</td><td>5000</td><td>Above 10000</td></tr>
<tr><td>c_e</td><td>1.0</td><td>0.7</td><td>0.6</td><td>0.5</td></tr>
</table>

</td>
<td>

The column shear strength capacity within the plastic hinge region is calculated on the basis of the nominal material strength properties and satisfies:

$$V_n = V_c + V_s + V_p \quad \text{(15-1)}$$

$$V_n = 0.25 f'_c b_v d_v + V_p \quad \text{(15-2)}$$

in which:

$$V_c = 0.083 \beta \sqrt{f'_c}\, b_v d_v \quad \text{(15-3*)}$$

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} \quad \text{(15-4)}$$

*In the end regions, Vc is taken as specified in Equation 15-3 provided that the minimum factored axial compression exceeds 0.10f'$_c$A$_g$. For compression members less than 0.10f'$_c$ A$_g$, Vc is taken to decrease linearly from the value calculated by the Equation 15-3 to zero at zero compression force.

where:

b_v = effective web width taken as the minimum web width within the depth d_v as determined in (mm)

d_v = effective shear depth as determined in (mm)

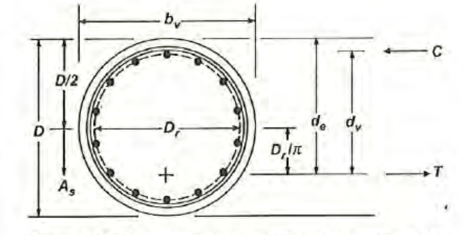
s = spacing of stirrups (mm)

β = factor indicating ability of diagonally cracked concrete to transmit tension

θ = angle of inclination of diagonal compressive stresses (°)

A_v = area of shear reinforcement within a distance s (mm^2)

A simplified procedure would be to take $\beta = 2.0$, $\theta = 45°$, otherwise refer to Section 5.8.3.4.



* dv need not be taken less than the greater of 0.90de or 0.72h

Figure 15-1 bv and dv for a circular section

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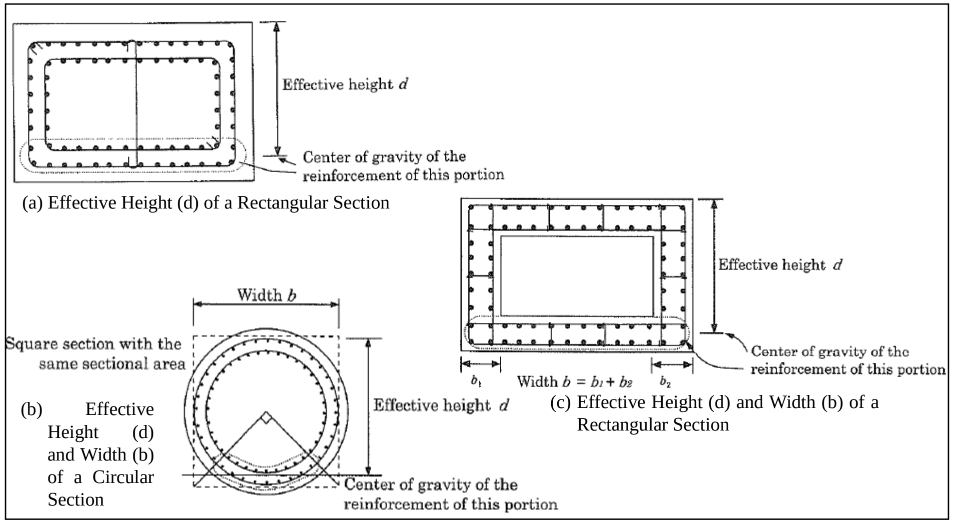
<table>
<tr><th>Items</th><th>JRA (Part V; English Version, 2002)</th><th>AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)</th></tr>
<tr><td></td><td>

Table 15-3 Modification Factor Cpt in Relation to Axial tensile Reinforcement Ratio Pt

<table>
<tr><th>Tensile Reinforcement Ratio (%)</th><th>0.2</th><th>0.3</th><th>0.5</th><th>Above 1.0</th></tr>
<tr><td>c_{pt}</td><td>0.9</td><td>1.0</td><td>1.2</td><td>1.5</td></tr>
</table>

Evaluation method of effective height (d) for each column section shape is shown in Fig. 15-1.



(a) Effective Height (d) of a Rectangular Section

(b) Effective Height (d) and Width (b) of a Circular Section

(c) Effective Height (d) and Width (b) of a Rectangular Section

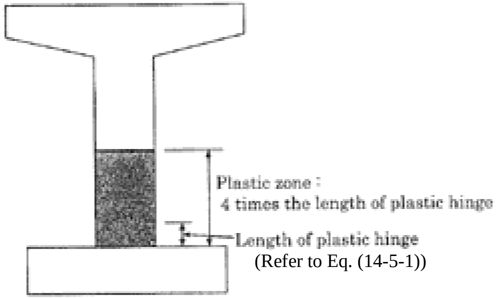
Fig. 15-1 Effective Height (d) and Width (b) of Each Section Shape

</td><td></td></tr>
<tr><td>16. Structural Details for Improving Ductility Performance</td><td>

(1) Incase that generation of plastic deformation of the column is expected, lapping of axial reinforcements shall not generally be placed within the plastic zone (refer to Fig 16-1).

(2) Arrangement of Hoop Ties
(a) To use deformed bars of at least 13mm in diameter, and the intervals shall not generally be greater than 150mm in the plastic zone.
(b) To be arranged so as to enclose the axial reinforcement and be fixed in to the concrete inside a column with the length below.

i) Semi-circular hook = 8Φ or 120mm whichever is the greater.
ii) Acute angle hook = 10Φ
iii) Rectangular angle hook = 12Φ
(Φ: the diameter of the hoop tie)



(Refer to Eq. (14-5-1))

Fig. 16-1 Plastic Zone

(c) Lapping of hoop ties shall be staggered along the column height.
(d) To have a lap length of at least 40Φ in case that hoop ties are lapped at any place other than the corners of a rectangular section (refer to Fig 16-2).

</td><td>

(1) Splices
(a) Lap splices in the longitudinal direction is not allowed.
(b) However, full-welded or full-mechanical connection splices may be used provided that not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section, and the distance between splices of adjacent bars is greater than 600mm.
(c) The spacing of the transverse reinforcement over the length of the splice shall not exceed 100mm or one-quarter of the minimum member dimension.

(2) Transverse Reinforcement for Confining Plastic Hinges
(a) Transverse reinforcement for confinement of plastic hinges shall be:
- provided at the top and bottom of column over a length not less than the greatest of the maximum cross-sectional column dimensions, one-sixth of the clear height of the column or 450mm.
- extended into the top and bottom connections
- provided at top of piles in pile bents over the same length as the columns
- provided within pile bents over a length extending from 3 times the maximum cross-sectional dimension below the calculated point of moment fixity to a distance not less than the maximum cross-sectional dimension or 450mm above the mud line
- spaced not to exceed one-quarter of the minimum member dimension or 100mm center to center.

(b) At the expected plastic hinge region, core of columns and pile bents shall be confined by transverse reinforcement.

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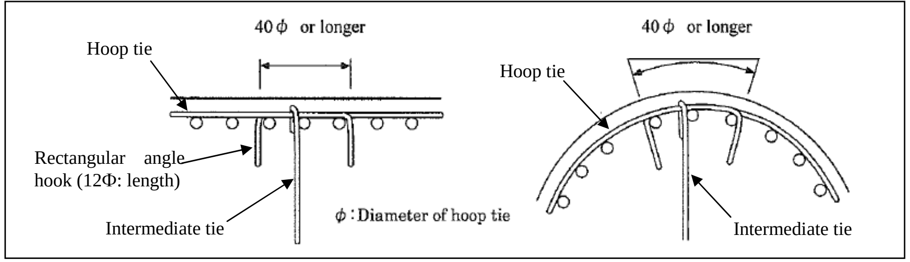
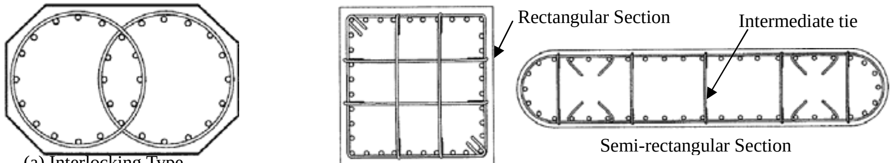
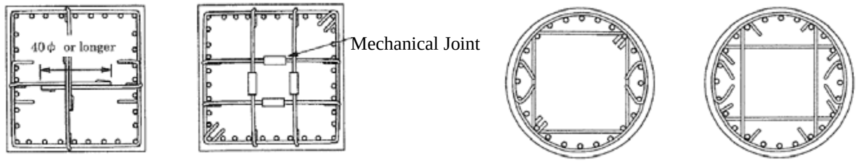
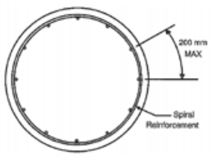
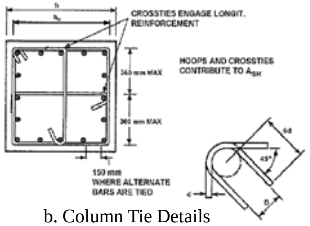
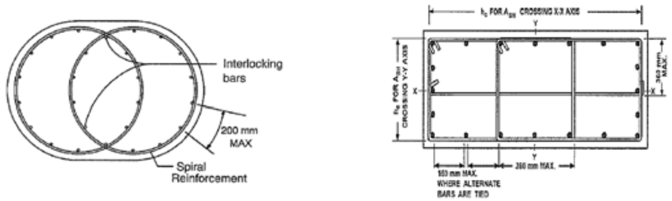
</table>

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)
	 **Fig. 16-2** Anchorage of Hoop Ties with Rectangular Angle Hook (3) Arrangement of Intermediate Ties (a) To be of the same material and the same diameter as the hoop ties. (b) To be arranged in both the directions of the long side and the short side of a column section. (c) Intervals within a column section shall not be greater than one meter. (d) To be arranged in all sections with hoop ties arranged. (e) To be hooked up to the hoop ties arranged in the perimeter directions of the section. (f) To be fixed into the concrete inside a column (refer to Fig. 16-2 and 16-3). (g) To go through a column section, with use of a continuous reinforcing bar or a pair of reinforcing bars with a joint within the column section.  (a) Interlocking Type (b) Hoop Ties and Intermediate Ties in Rectangular Sections  (c) Joint Types of Intermediate Ties (d) Arrangement Types of Intermediate Ties **Fig. 16-3** Arrangement of Hoop Ties and Intermediate Ties According to Column Types	(c) For circular columns, the volumetric ratio of spiral or seismic hoop reinforcement, ρ_s, is given by: $\rho_s \geq 0.12 \frac{f'_c}{f_y}$ (16-1) where: f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (MPa) f_y = yield strength of reinforcing bars (MPa) (c) For rectangular column, the total gross sectional area A_{sh}, of rectangular hoop shall be either: $A_{sh} \geq 0.30\, s h_c \frac{f'_c}{f_y}\left[\frac{A_g}{A_c} - 1\right]$ (16-2) or $A_{sh} \geq 0.12\, s h_c \frac{f'_c}{f_y}$ (16-3) where: s = vertical spacing of hoops, not exceeding 100 mm (mm) A_c = area of column core (mm^2) A_g = gross area of column (mm^2) A_{sh} = total cross-sectional area of tie reinforcement, including supplementary cross-ties having a vertical spacing of s and crossing a section having a core dimension of h_c (mm^2) f_y = yield strength of tie or spiral reinforcement (MPa) h_c = core dimension of tied column in the direction under consideration (mm)  a. Single spiral  b. Column Tie Details

Items	**JRA (Part V; English Version, 2002)**	**AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)**



c. Column Interlocking Spiral Details

d. Column Tie Details

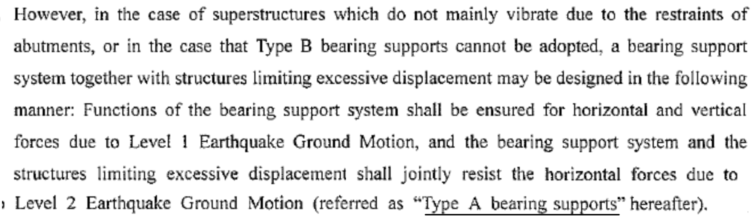
Figure 16-1 Details of spirals, hoops, ties and cross-ties

17. Bearing Support System

JRA (Part V; English Version, 2002)

(1) Bearing support shall be fundamentally designed for horizontal and vertical forces due to Level 1 and Level 2 EGMs (referred as "Type B bearing support").

(2)



However, in the case of superstructures which do not mainly vibrate due to the restraints of abutments, or in the case that Type B bearing supports cannot be adopted, a bearing support system together with structures limiting excessive displacement may be designed in the following manner: Functions of the bearing support system shall be ensured for horizontal and vertical forces due to Level 1 Earthquake Ground Motion, and the bearing support system and the structures limiting excessive displacement shall jointly resist the horizontal forces due to Level 2 Earthquake Ground Motion (referred as "Type A bearing supports" hereafter).

(3) Fig. 17-1 shows the selection flow of bearing support.

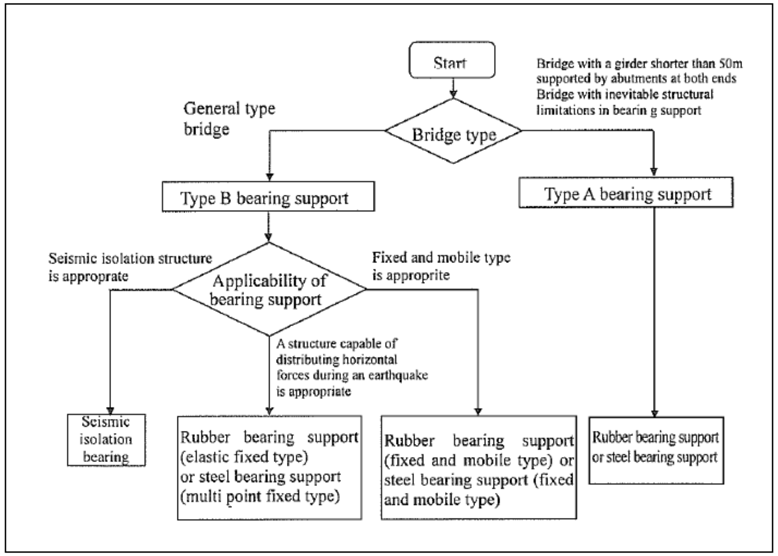


Fig. 17-1 General Consideration in Selection of Bearing Support

AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)

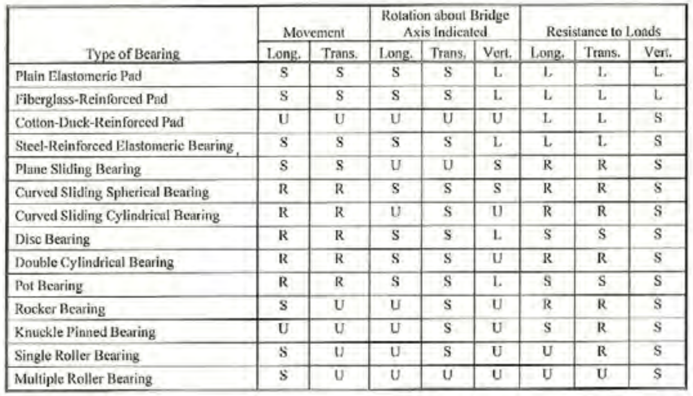
(1) Bearings design shall be consistent with the intended seismic (or other extreme event) response of the whole bridge system. Where rigid-type bearings are used, the seismic (or other horizontal extreme event) forces from the superstructure is assumed to be transmitted through diaphragms and cross-frames and their connections to the bearings and then to the substructure without reduction due to local inelastic action along the load path.

(2) Based on the horizontal stiffness, bearings are divided into four categories:

- Rigid bearings that transmit seismic loads without any movement or deformations,
- Deformable bearings that transmit seismic loads limited by plastic deformations or restricted slippage of bearing components
- Seismic isolation type bearings that transmit reduced seismic loads, limited by energy dissipator
- Structural fuses that are designed to fail at a prescribed load.

(3) The bearing chosen for a particular application shall have appropriate load and movement capabilities. The following table illustrates bearing suitability:

Table 17-1 Bearing Suitability



Type of Bearing	Movement		Rotation about Bridge Axis Indicated			Resistance to Loads		
	Long.	Trans.	Long.	Trans.	Vert.	Long.	Trans.	Vert.
Plain Elastomeric Pad	S	S	S	S	L	L	L	L
Fiberglass-Reinforced Pad	S	S	S	S	L	L	L	L
Cotton-Duck-Reinforced Pad	U	U	U	U	U	L	L	S
Steel-Reinforced Elastomeric Bearing	S	S	S	S	L	L	L	S
Plane Sliding Bearing	S	S	U	U	S	R	R	S
Curved Sliding Spherical Bearing	R	R	S	S	S	R	R	S
Curved Sliding Cylindrical Bearing	R	R	U	S	U	R	R	S
Disc Bearing	R	R	S	S	L	S	S	S
Double Cylindrical Bearing	R	R	S	S	U	R	R	S
Pot Bearing	R	R	S	S	L	S	S	S
Rocker Bearing	S	U	U	S	U	R	R	S
Knuckle Pinned Bearing	U	U	U	S	U	S	R	S
Single Roller Bearing	S	U	U	S	U	U	R	S
Multiple Roller Bearing	S	U	U	U	U	U	U	S

S = Suitable U = Unsuitable L = Suitable for limited applications
R = May be suitable but requires special considerations or additional elements such as sliders or guideways
Long. = Longitudinal axis Trans. = Transverse axis Vert. = Vertical axis

(4) Rockers should be avoided wherever practical and, when used, their movements and tendency to tip under seismic actions shall be considered in the design details.

<table>
<tr><th>Items</th><th>JRA (Part V; English Version, 2002)</th><th>AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)</th></tr>
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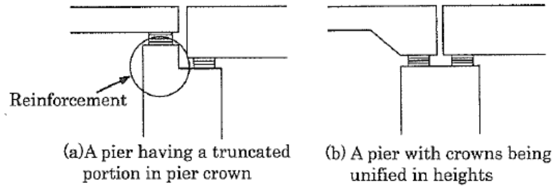

Fig. 17-2 Measures for Dealing with Truncated Portion of a Pier Crown

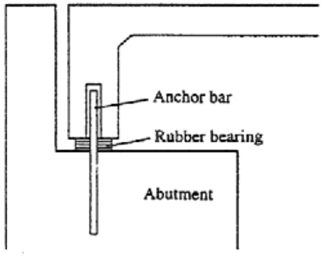

Fig. 17-3 Structure Limiting Excessive Displacement Connecting Superstructure and Substructure

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(5) Elastomeric expansion bearings shall be provided with adequate seismic and other extreme event resistant anchorage to resist the horizontal forces in excess of those accommodated by shear in the pad unless the bearing is intended to act as a fuse of irreparable damage is permitted. Elastomeric fixed bearing shall be provided with horizontal restraint adequate for the full horizontal load.

</td></tr>
<tr><td>18. Unseating Prevention System</td><td>

18.1 Seating Length

(1) Ordinary Bridge

- Eq. (18-1) shows the required seating length of a girder at its support.
- The seat length shall be measured in the direction perpendicular to the front line of the bearing support when the direction of soil pressure acting on the substructure differs from the bridge axis, as in case of askew bridge or a curved bridge.

$S_E = U_R + U_G \geq S_{EM}$ -- (18-1)

$S_{EM} = 0.7 + 0.005 * Ls$ -- (18-2)

$U_G = \varepsilon_G * L$ -- (18-3)

where,

S_E: Refer to Fig. 18-1.

U_R: Maximum relative displacement between the superstructure and the edge of the top of the substructure due to Level 2 EGM (m) (refer to description of U_R below)

U_G: Relative displacement of the ground caused by seismic ground strain (m)

S_{EM}: Minimum seating length of a girder at the support

ε_G: Seismic ground strain

= 0.0025 for Ground Type I, 0.00375 for Ground Type II, 0.005 for Ground Type III

L: Distance between two substructures for determining the seating length (refer to description of L below)

Ls: Length of the effective span (m). When two superstructures with different span length are supported on one bridge pier, the longer one shall be used.

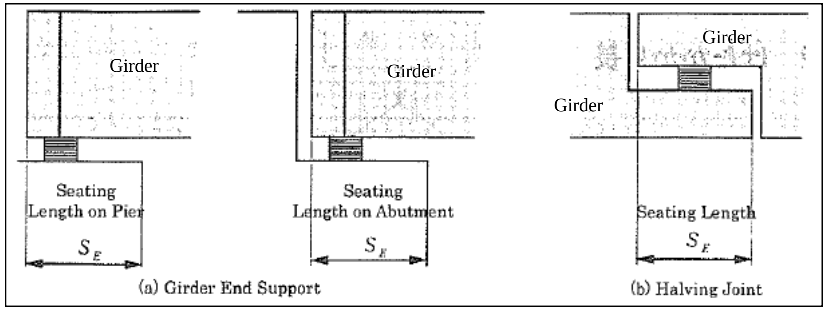


Fig.18-1 Seating Length (S_E) (m)

</td><td>

18.1 Seating Length

(a) Minimum Support Length Requirements (4.7.4.4)

Support lengths at expansion bearings without restrainers, shock transmission unit (STUs), or damper shall either accommodate the greater of the:

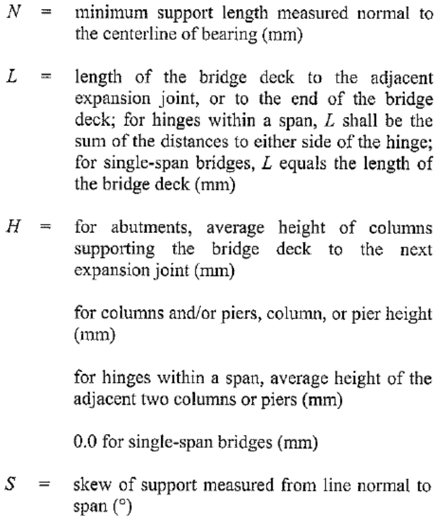
- maximum displacement calculated in the inelastic dynamic response analysis (4.7.4.3)
- or a percentage of the empirical support length , N

Otherwise, longitudinal restrainers shall be provided (3.10.9.5). Bearings restrained for longitudinal movements shall be design in accordance with the calculated seismic design forces (3.10.9)

The empirical support length is shown in Eq. 18-1 while the percentage of N applicable to each seismic zone in given in Table 18-1:

$N = (200 + 0.0017L + 0.0067H)(1 + 0.000125S^2)$ (18-1)

where:



N = minimum support length measured normal to the centerline of bearing (mm)

L = length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck; for hinges within a span, L shall be the sum of the distances to either side of the hinge; for single-span bridges, L equals the length of the bridge deck (mm)

H = for abutments, average height of columns supporting the bridge deck to the next expansion joint (mm)

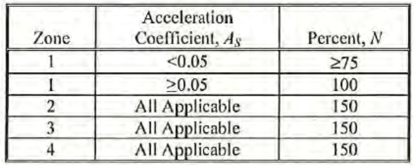
for columns and/or piers, column, or pier height (mm)

for hinges within a span, average height of the adjacent two columns or piers (mm)

0.0 for single-span bridges (mm)

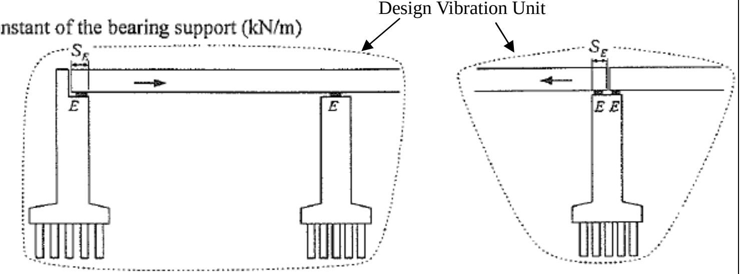
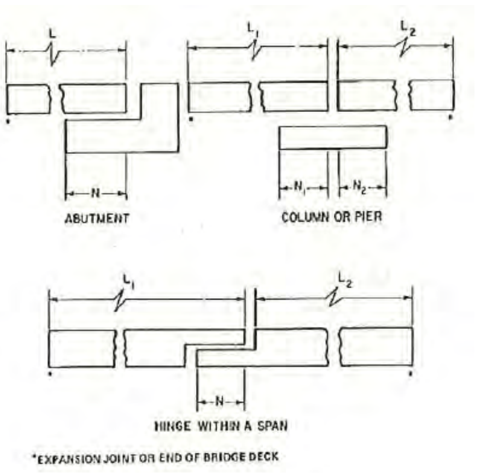
S = skew of support measured from line normal to span (°)

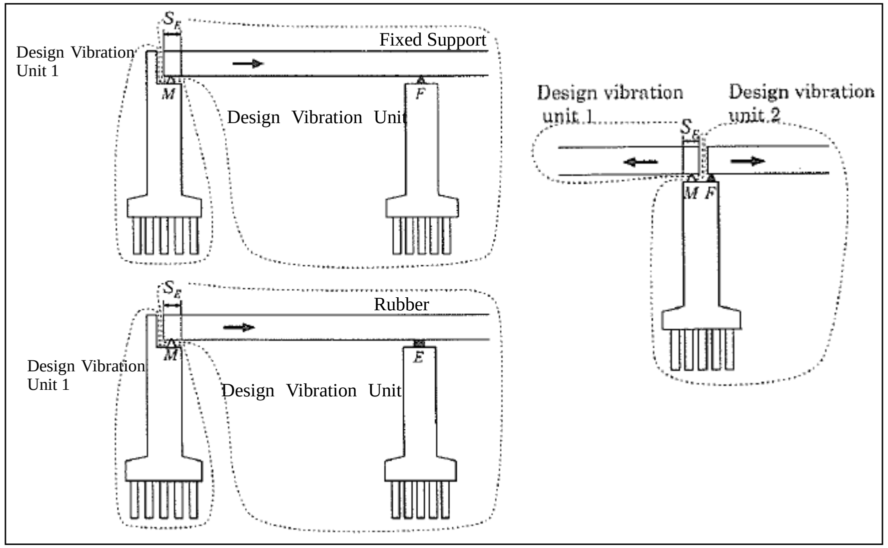
Table 18-1 Percentage N by Zone and Acceleration Coefficient

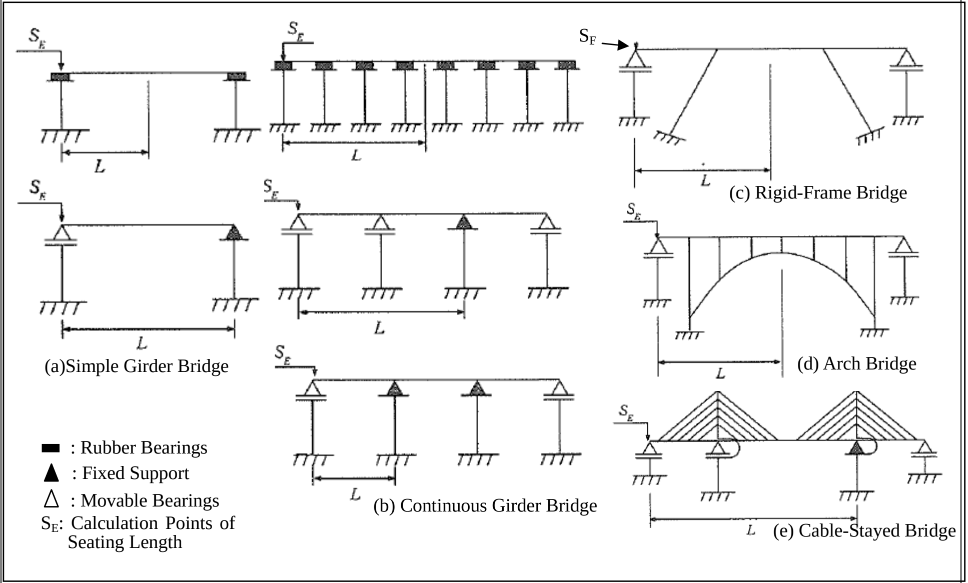


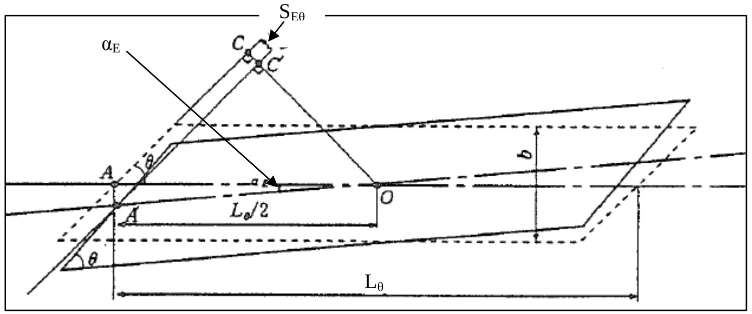
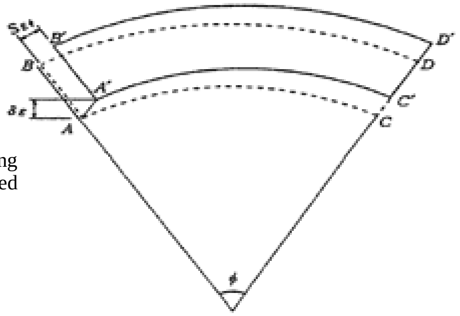
Zone	Acceleration Coefficient, A_S	Percent, N
1	<0.05	≥75
1	≥0.05	100
2	All Applicable	150
3	All Applicable	150
4	All Applicable	150

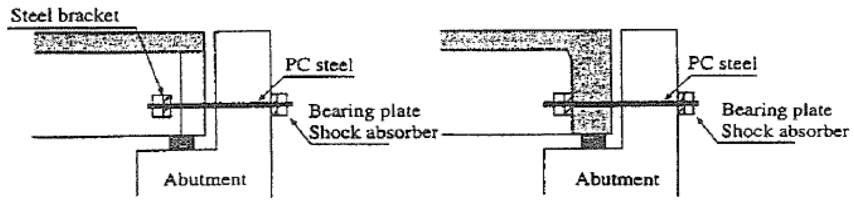
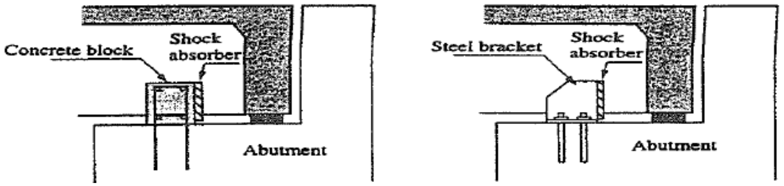
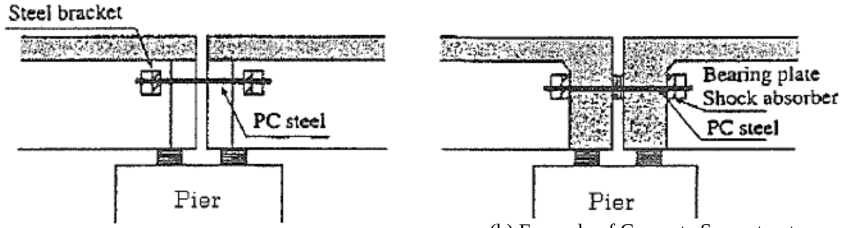
</td></tr>
</table>

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6^{th} Edition, 2012)
	Description of U_R (a) Rubber Bearing (refer to Fig 18-2) $u_R = \frac{c_m P_u}{k_B}$ ································ (18-4) P_u : Horizontal force equivalent to lateral strength of a column when considering plastic behavior of the column; or horizontal force equivalent to maximum response displacement of a foundation when considering plastic behavior of the foundation (kN) c_m : Dynamic modification factor (C_m = 1.2) k_B : Spring constant of the bearing support (kN/m)  **Fig. 18-2** When the girder end is supported by rubber bearings (b) Fixed Bearing When fixed supports of Type B are used at the support concerned, one-half of the width of the bearing support shall be taken as the relative displacement u_R, while the whole width shall be used in case of fixed supports of Type A. This is for preventing unseating of the bridge even though the bearing support system is unexpectedly damaged, although the fixed supports of Type B are designed to be capable of transferring the superstructural inertia force caused either by horizontal force equivalent to lateral strength of a column with plastic behavior, or by horizontal force equivalent to maximum response displacement of a foundation with plastic behavior, and there is no relative displacement generating at the bearing support system under an assumed earthquake ground motion in the design. Providing that the bearing support may be damaged in case of fixed supports of Type A, u_R for Type A is assumed to be greater than that of Type B. In addition, the width of the bearing support shall be equal to width of the lower shoe in the bridge axis in the case of metal bearings, and width of the bearing support in the same direction for rubber bearings. (c) Movable Bearing (refer to Fig 18-3) $U_R = \sqrt{\sum U_{Ri}^2}$ (i = 1,2) -- (18-5) $U_{Ri} = U_{Pi} + U_{Fi} + U_{Bi}$ --- (18-6) $U_{Pi} = \mu_{Ri} * \delta_{yi}$ --- (18-7) $U_{Fi} = \delta_{Fi} + \theta_{Fi} * h_{oi}$ --- (18-8) $U_{Bi} = C_m * P_{ui} / K_{Bi}$ -- (18-9)	 Figure 18-1 Support Length, N 18.2 Longitudinal Restrainers • Restrainers shall be designed for a force calculated as the acceleration coefficient, As, times the permanent load of the lighter of the two adjoining spans or parts of the structure. • Sufficient slack shall be allowed in the restrainer so that the restrainer does not start to act until the design displacement is exceeded. 18.3 Hold Down Device • Hold down devices shall be provided at supports and at hinges in continuous structures for Seismic Zones 2, 3 and 4 where the vertical seismic force due to the longitudinal seismic load opposes and exceeds 50%, but less than 100% of the reaction due to permanent loads. • If the vertical forces result in uplift, the hold down device shall be designed to resist the larger of: o 120% of the difference between the vertical seismic force and the reaction due to permanent loads, or o 10% of the reaction due to permanent loads.

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)
	where, u_{Ri}: Displacement of the i-th design vibration unit shown in Fig. 18-3 u_{Pi}: Response displacement of the column representing the i-th design vibration unit (m) u_{Fi} : Horizontal displacement at the height of superstructural inertia due to displacement of the pier foundation representing the i-th design vibration unit (m). The calculation shall include these effects when liquefaction and lateral spreading of the ground U_{Bi} : Relative displacement at the bearing support system representing the i-th design vibration unit (m) μ_{Ri} : Response ductility factor of the column representing the i-th design vibration unit δ_{yi} : Yielding displacement of the column representing the i-th design vibration unit (m) δ_{Fi} : Response horizontal displacement of the pier foundation representing the i-th design vibration unit (m) θ_{Fi} : Response rotation angle of the pier foundation representing the i-th design vibration unit (rad) h_{0i} : Height from the ground surface of seismic design to the superstructural inertial force in the column representing the i-th design vibration unit (m) P_{ui} : Horizontal force equivalent to lateral strength of a column with plastic behavior, or by horizontal force equivalent to maximum response displacement of a foundation with plastic behavior, representing the i-th design vibration unit (kN) c_m : Dynamic modification factor ($C_m = 1.2$) k_{Bi} : Spring constant of the bearing support for the i-th design vibration unit (kN/m)  **Fig. 18-3** Inertia Forces Used in Calculating Seating Length	

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)
	Description of L The length L between the substructures that may affect the seating length shall be the distance between the substructure supporting the girder at the support where the seating length is to be calculated and one that may primarily affect the vibration of the girder containing the support (refer to Fig. 18-4)  **Fig. 18-4** Measuring Methods of Distance (L) between Substructures as to Bridge Types (2) A Bridge with Complicated Dynamic Structural Behavior by a Dynamic Analysis ➢ The maximum relative displacement (U_R) is to be obtained from the dynamic analysis. (3) A Skew Bridge ➢ The seating length shall be calculated by Eq. (18-10) (refer to Fig. 18-5). $S_{E\theta} \geqq (L_\theta/2)(\sin\theta - \sin(\theta - \alpha_E))$ ……………………………………… (18-10) where $S_{E\theta}$: Seating length for the skew bridge (m) L_θ : Length of a continuous superstructure (m) θ : Skew angle (degree) α_E : Marginal unseating rotation angle (degree). α_E can generally be taken as 5 degrees.	

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)
	 **Fig. 18-5** Seating Length of a Skew Bridge (4) A Curved Bridge - The seating length is to be calculated by Eq. (18-11) (refer to Fig. 18-6) $S_{E\phi} \geqq \delta_E \frac{\sin\phi}{\cos(\phi/2)} + 0.3$ ········ (18-11) $\delta_E = 0.005\,\phi + 0.7$ ········ (18-12) where $S_{E\phi}$: Seating length for the curved bridge (m) δ_E : Displacement of the superstructure toward the outside direction of the curve (m) ϕ : Fan-shaped angle by the two edges of a continuous girder of a curved bridge (degrees)  **Fig. 18-6** Seating Length Corresponding to the Movement of a Curved Bridge 18.2 Unseating Prevention Structure (1) Ultimate Strength of an Unseating Prevention Structure - Ultimate strength of an unseating prevention structure is to be calculated by Eq. (18-13). - The unseating prevention structure is a structure of 1) connecting the superstructure and the substructure (refer to Fig. 18-7), 2) providing protuberance either in superstructure and in the substructure (refer to Fig. 18-8), 3) joining two superstructures together (refer to Fig. 18-9).	

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)
	$H_F = 1.5 R_d$ ········ (18-13) $S_F = c_F S_E$ ········ (18-14) where H_F : Design seismic force of the unseating prevention structure (kN) R_d : Dead load reaction (kN). In case the structure connects two adjacent girders, the larger reaction shall be taken. S_F : Maximum design allowance length of the unseating prevention structure (m) S_E : Seating length specified in Section 16.2 (m). c_F :Design displacement coefficient of the unseating prevention structure. (C_F = 0.75)  (a) Example of steel superstructure (b) Example of concrete superstructure **Fig. 18-7** Unseating Prevention Structures Connecting the Superstructure with the Substructure  (a) Example of Concrete Block (b) Example of Steel Bracket **Fig. 18-8** Unseating Prevention Structures Providing Protuberance on the Superstructure or the Substructure  (a) Example of Steel Superstructure (b) Example of Concrete Superstructure **Fig. 18-9** Unseating Prevention Structures Connecting the Two Adjacent Superstructures	

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)

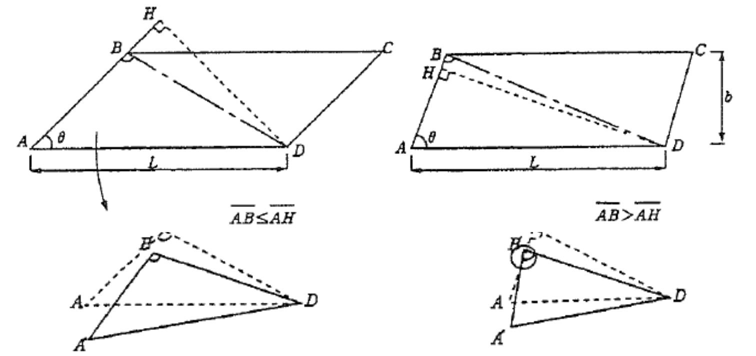
18.3 Structure Limiting Excessive Displacement

(1) For the following bridges, structures limiting excessive displacement working in the direction perpendicular to the bridge axis shall be installed in the terminal support, in addition to the unseating prevention system working in the bridge axis.

(a) Skew bridges with a small skew angle satisfying Eq. (18-15) (refer to Fig. 18-10 and 18-11)

$$\sin 2\theta/2 > b / L \quad \cdots\cdots\cdots\cdots\cdots \quad (18\text{-}15)$$

L: Length of a continuous superstructure (m)
b: Whole width of the superstructure (m)
θ: Skew angle (degree)



(a) ∠ DBA ≥ 90° (a bridge can rotate) (b) ∠ DBA < 90° (a bridge can not rotate)

Fig. 18-10 Conditions in Which a Skew Bridge can Rotate Without Being Affected by Adjoining Girders or Abutment

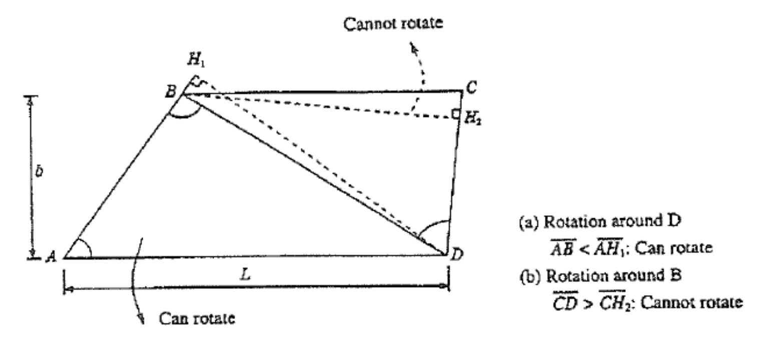
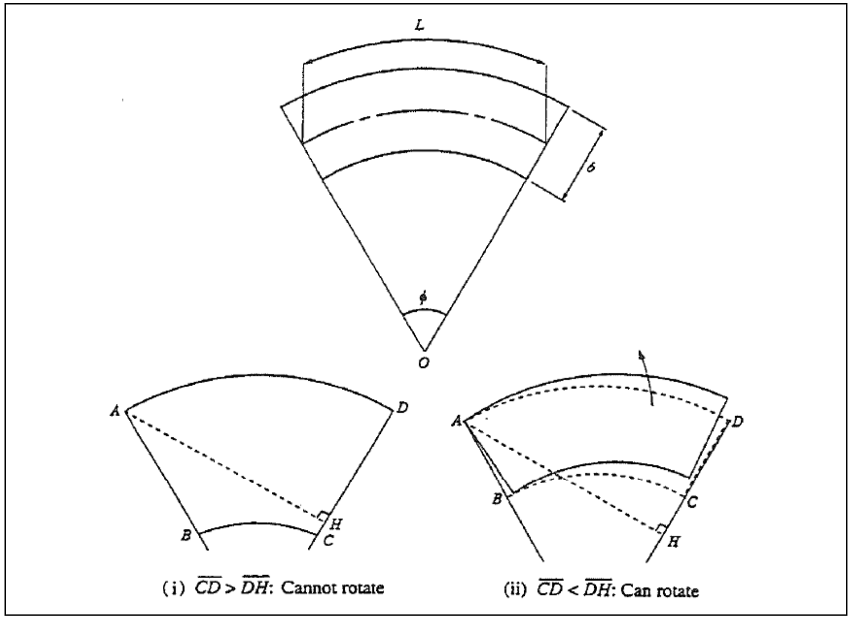
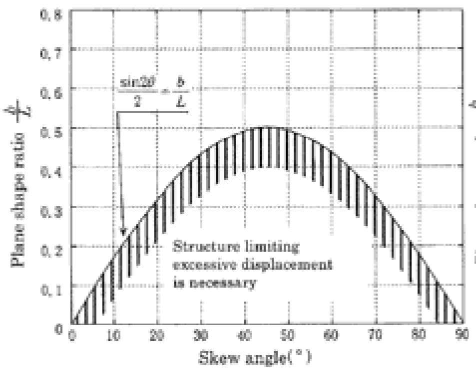
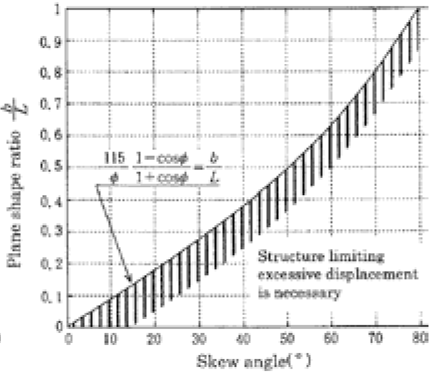


Fig. 18-11 Conditions in Which a Skew Bridge With Unparallel Bearing Lines on Both Edges of the Superstructure can rotate

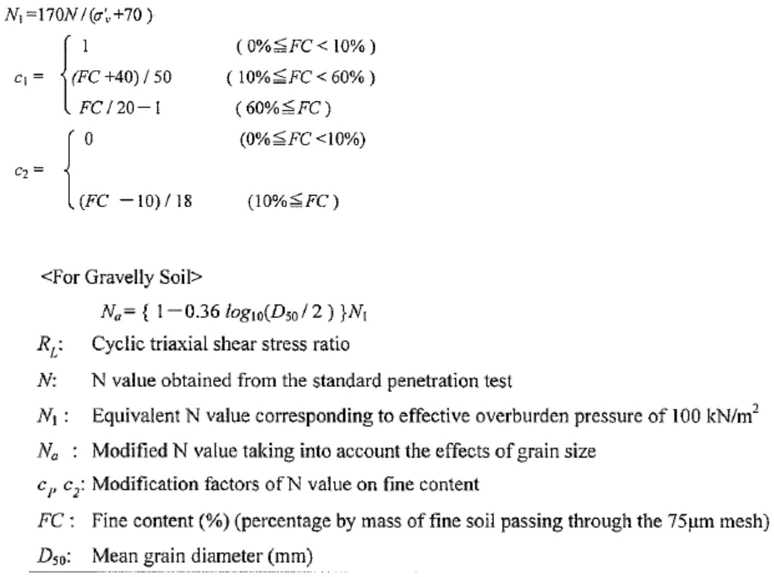
Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6^{th} Edition, 2012)
	(b) Curved bridges satisfying Eq. (18-16) (refer to Fig. 18-12) $\frac{115}{\phi}\frac{1-\cos\phi}{1+\cos\phi} > b/L$ ………………………………………… (18-16) L: Length of a continuous superstructure (m) b: Whole width of the superstructure (m) Φ: Intersection angle (degree)  (i) $\overline{CD} > \overline{DH}$: Cannot rotate (ii) $\overline{CD} < \overline{DH}$: Can rotate **Fig. 18-12** Conditions in Which a Curved Bridge can rotate Without being Affected by Adjoining Girders or Abutment The relation of Eq. (18-15) and Eq. (18-16) is shown in Fig. 18-13 and Fig. 18-14, respectively.	

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)
	 **Fig. 18-13** Conditions in which a Skew Bridge Requires an Structure Limiting Excessive Displacement in the Transverse Direction  **Fig. 18-14** Conditions in which a Curved Bridge Requires an Structure Limiting Excessive Displacement in the Transverse Direction (c) For the following bridges, the structures limiting excessive displacement shall be installed at intermediate supports ➢ Bridges with the superstructure being narrow at the top ➢ Bridges with a small number of bearing supports on one bearing line ➢ Bridges probably to be subject to movement of the bridge piers in the direction perpendicular to the bridge axis as a result of lateral spreading.	
19. Effects of Seismically Unstable Ground	19.1 Assessment of Extremely Soft Clayey Soil Layer in Seismic Design (1) For a clayey layer or a silt layer lying within three meters from the ground surface, and having compressive strength of 20KN/m or less obtained from an unconfined compression test or an in-situ test, the layer shall be regarded as an extremely soft layer in the seismic design. (2) In this case its geological parameters (shear modulus and strength) shall be assumed to be zero in the seismic design. 19.2 Assessment of Soil Liquefaction (1) Sandy Layer Requiring Liquefaction Assessment ➢ Saturated soil layer having ground water level higher than 10m below the ground surface and lying at a depth less than 20m below the ground surface. ➢ Soil layer containing a fine content (FC) of 35% or less, or soil layer having plasticity index Ip less than 15, even if FC is larger than 35%. ➢ Soil layer having a mean particle size (D_{50}) less than 10mm and a particle size at 10% pass (on the grading curve) (D_{10}) is less than 1mm. (2) Assessment of Liquefaction The liquefaction resistance factor FL calculated by Eq. (19-1) turns out to be less than 1.0, the layer shall be regarded as a soil layer having liquefaction potential.	(1) For Seismic Zones 3 and 4, liquefaction assessment shall be conducted when both of the following conditions are present: ▪ *Groundwater Level*. The groundwater level anticipated at the site is within 15.24m (50ft) of the existing ground surface or the final ground surface, whichever is lower. ▪ *Soil Characteristics*. Low plasticity silts and sands within the upper 22.86m (75ft) are characterized by one of the following conditions: (1) the corrected standard penetration test (SPT) blow count, $(N_1)_{60}$, is less than or equal to 25 blows/ft in sand and non-plastic silt layers, (2) the corrected cone penetration test (CPT) tip resistance, q_{ciN}, is less than or equal to 150 in sand and in non-plastic silt layers, (3) the normalized shear wave velocity, V_{s1}, is less than 660fps, or (4) a geologic unit is present at the site that has been observed to liquefy in past earthquakes. (2) For sites that require assessment of liquefaction, the potential effects of liquefaction on soils and foundations shall be evaluated. The assessment shall consider the following effects of liquefaction: ▪ Loss in strength in the liquefied layer or layers, ▪ Liquefaction-induced ground settlement, and ▪ Flow failures, lateral spreading, and slope instability. (3) For sites where liquefaction occurs around bridge foundations, bridges should be analyzed and designed in two configurations as follows:

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6^{th} Edition, 2012)
	$F_L = R / L$ ……………………………………………… (19-1) $R = c_w R_L$ $L = r_d k_{hg} \sigma_v / \sigma'_v$ $R_d = 1.0 - 0.015x$ $\sigma_v = \gamma_{t1} h_w + \gamma_{t2} (x - h_w)$ $\sigma'_v = \gamma_{t1} h_w + \gamma'_{t2} (x - h_w)$ (For Type I Earthquake Ground Motion) $c_w = 1.0$ …………………………… (For Type II Earthquake Ground Motion) $c_w = \begin{cases} 1.0 & (R_L \leqq 0.1) \\ 3.3R_L + 0.67 & (0.1 < R_L \leqq 0.4) \\ 2.0 & (0.4 < R_L) \end{cases}$ F_L : Liquefaction resistance factor R : Dynamic shear strength ratio L : Seismic shear stress ratio c_W : Modification factor on earthquake ground motion R_L : Cyclic triaxial shear stress ratio to be obtained by properties (3) below r_d : Reduction factor of seismic shear stress ratio in terms of depth k_{hg}: Design horizontal seismic coefficient at the ground surface for Level 2 Earthquake Ground Motion specified in Item 11 σ_v: Total overburden pressure (kN/m^2) σ'_v : Effective overburden pressure (kN/m^2) x : Depth from the ground surface (m) γ_{t1}: Unit weight of soil above the ground water level (kN/m^3) γ_{t2}: Unit weight of soil below the ground water level (kN/m^3) γ'_{t2}: Effective unit weight of soil below the ground water level (kN/m^3) h_w: Depth of the ground water level (m) (3) Cyclic Tri-axial Shear Stress Ratio Cyclic tri-axial shear stress ratio R_L shall be calculated by Eq. (19-2). $R_L = \begin{cases} 0.0882\sqrt{N_a/1.7} & (N_a < 14) \\ 0.0882\sqrt{N_a/1.7} + 1.6\times10^{-6}\cdot(N_a - 14)^{4.5} & (14 \leq N_a) \end{cases}$ …… (19-2) <For Sandy Soil> $N_a = c_1 N_1 + c_2$	▪ *Non-liquefied Configuration*. The structure should be analyzed and designed, assuming no liquefaction occurs, using the ground response spectrum appropriate for the site soil conditions in a non-liquefied state. ▪ *Liquefied Configuration*. The structure as designed in non-liquefied configuration above shall be reanalyzed assuming that the layer has liquefied and the liquefied soil provides the appropriate residual resistance for lateral and axial deep foundation response analyses consistent with liquefied soil conditions (i.e., modified *P-y* curves, modulus of subgrade reaction, or *t-z* curves). The design spectrum shall be that used in a non-liquefied configuration. (4) As required by the Owner, a site-specific response spectrum that accounts for the modifications in spectral content from the liquefying soil may be developed. Unless approved otherwise, the reduced response spectrum resulting from the site-specific analyses shall not be less than two-thirds of the spectrum at the ground surface developed using the general procedure modified by the site coefficients. (5) The Designer should provide explicit detailing of plastic hinge zones for both cases mentioned above since it is likely that the locations of plastic hinges for the liquefied configurations are different than the locations of the plastic hinges for the non-liquefied configuration. Design requirements including shear reinforcement should be met for the liquefied and the non-liquefied configuration. Where liquefaction is identified, plastic hinging in he foundation may be permitted with the Owners approval provided that the provisions of earthquake resisting systems are satisfied. (6) The effects of liquefaction-related, permanent lateral ground displacements on bridge and retaining wall performance should be considered separate from the inertial evaluation of the bridge structures. However, if large magnitude earthquakes dominate the seismic hazards, the bridge response evaluation should consider the potential simultaneous occurrence of: ▪ Inertial response of the bridge, and loss in ground response from liquefaction around the bridge foundations, and ▪ Predicted amounts of permanent lateral displacement of the soil. (7) During liquefaction, pore-water pressure build-up occurs, resulting in loss of strength and then settlement as the excess pore-water pressures dissipate after the earthquake. The potential effects of strength loss and settlement includes: ▪ *Slope Failure, Flow Failure, or Lateral Spreading*. The strength loss associated with pore-water pressure build-up can lead to slope instability. Generally, if the factor of safety against liquefaction is less than approximately 1.2 to 1.3, a potential for pore-water pressure build-up will occur, and the effects of this build-up should be assessed. If the soil liquefies, the stability is determined by the residual strength of the soil. The residual strength of liquefied soils can be determined using empirical methods developed by Seed and Harder (1990), Olson and Stark (2002), and others. Loss of lateral resistance can allow abutment soils to move laterally, resulting in bridge substructure distortion and unacceptable deformations and moments in the superstructure. ▪ *Reduced Foundation Bearing Resistance*. Liquefied strength is often a fraction of non-liquefied strength. This loss in strength can result in large displacements or bearing failure. For this reason, spread footing foundations are not recommended where liquefiable soils occur unless the spread footing is located below the maximum depth of liquefaction or soil improvement techniques are used to mitigate the effects of liquefaction. ▪ *Reduced Soil Stiffness and Loss of Lateral Support for Deep Foundation*. This loss in strength can

Items	JRA (Part V; English Version, 2002)	AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)

JRA (Part V; English Version, 2002)



$$N_1 = 170N/(\sigma'_v + 70)$$

$$c_1 = \begin{cases} 1 & (0\% \leqq FC < 10\%) \\ (FC+40)/50 & (10\% \leqq FC < 60\%) \\ FC/20-1 & (60\% \leqq FC) \end{cases}$$

$$c_2 = \begin{cases} 0 & (0\% \leqq FC < 10\%) \\ (FC-10)/18 & (10\% \leqq FC) \end{cases}$$

<For Gravelly Soil>

$$N_a = \{1 - 0.36\, log_{10}(D_{50}/2)\}N_1$$

R_L: Cyclic triaxial shear stress ratio

N: N value obtained from the standard penetration test

N_1 : Equivalent N value corresponding to effective overburden pressure of 100 kN/m^2

N_a : Modified N value taking into account the effects of grain size

c_1, c_2: Modification factors of N value on fine content

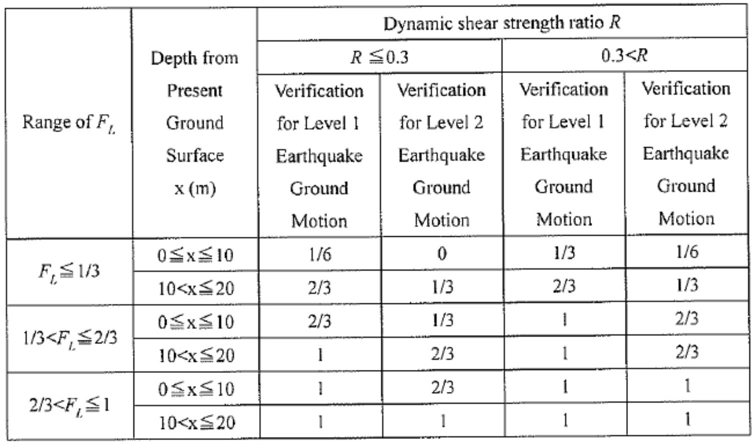
FC : Fine content (%) (percentage by mass of fine soil passing through the 75μm mesh)

D_{50}: Mean grain diameter (mm)

19.3 Reduction Factor (D_E) of Geotechnical Parameters due to Liquefaction

Geotechnical parameters of a sandy layer causing liquefaction affecting a bridge shall be obtained by multiplying geotechnical parameters without liquefaction by reduction factor D_E shown in Table 19-1.

Table 19-1 Reduction Factor DE for Geotechnical Parameters



Range of F_L	Depth from Present Ground Surface x (m)	Dynamic shear strength ratio R			
		$R \leqq 0.3$		$0.3 < R$	
		Verification for Level 1 Earthquake Ground Motion	Verification for Level 2 Earthquake Ground Motion	Verification for Level 1 Earthquake Ground Motion	Verification for Level 2 Earthquake Ground Motion
$F_L \leqq 1/3$	$0 \leqq x \leqq 10$	1/6	0	1/3	1/6
	$10 < x \leqq 20$	2/3	1/3	2/3	1/3
$1/3 < F_L \leqq 2/3$	$0 \leqq x \leqq 10$	2/3	1/3	1	2/3
	$10 < x \leqq 20$	1	2/3	1	2/3
$2/3 < F_L \leqq 1$	$0 \leqq x \leqq 10$	1	2/3	1	1
	$10 < x \leqq 20$	1	1	1	1

AASHTO LRFD Bridge Design Specifications (6th Edition, 2012)

change the lateral response characteristics of piles and shafts under lateral load.

- *Vertical Ground Settlement as Excess Pore-Water Pressures Induced by Liquefaction Dissipate, Resulting in Downdrag Loads on Deep Foundations*. If liquefaction-induced downdrag loads can occur, the downdrag loads should be assessed.

(2) COMPARISON TABLE OF BRIDGE SEISMIC DESIGN SPECIFICATIONS BETWEEN JRA AND AASHTO GUIDE SPECIFICATIONS LRFD FOR SEISMIC BRIDGE DESIGN (2ND Ed., 2011)

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)
1. Fundamentals of Seismic Design	(1) It shall be ensured that the seismic performance according to the levels of design earthquake motion and the importance of a bridge. (2) It is desirable to adopt a multi-span continuous structure, the type of which bearing supports is to be a horizontal force distributed structure. (3) It is generally better for a bridge with tall piers built in a mountainous region to resist seismic horizontal forces by abutments rather than piers if the ground conditions at the abutments are sufficiently sound.(The seismic performance of the whole bridge should be considered, and proper bearing supports in view of bridge structural conditions and ground bearing properties should be selected.) (4) On reclaimed land or alluvial ground where ground deformation such as sliding of a soft cohesive clayey layer, liquefaction of sandy layer and liquefaction-induced ground flow may happen, a foundation with high horizontal stiffness should be designed, and a structural system such as multi-fixed-point type and rigid frame type, which has many contact points between the superstructure and substructure, should be selected. (5) A seismically-isolated bridge should be adopted for a multi-span short-period continuous bridge on stiff ground conditions. (6) For a strong earthquake motion, a proper structural system shall be designed by clarifying structural members with nonlinear behavior and those basically remaining in elastic states. (7) A structure greatly affected by geometrical nonlinearity or a structure having extensive eccentricity of dead loads, which have tends to become unstable during a strong earthquake motion, shall not be adopted. (8) When ground conditions or structural conditions on a pier change remarkably, whether a case of two girder ends or that of a continuous girder is more advantageous is carefully examined.	(1) The Guide Specifications are approved as an alternate to the seismic provisions in the AASHTO LRFD Design Specifications. These Guide Specifications differ from the current procedures in the LRFD Specifications in the use of "***displacement-based***" design procedures, instead of the traditional, "***force-based R-factor***" method. (2) The key features of these Guide Specifications follow: • Adopt the seven percent in 75 year design event for development of a design spectrum. • Adopt the NEHRP Site Classification system and include site factors in determining response spectrum ordinates. • Ensure sufficient conservatism (1.5 safety factor) for minimum support length requirement. This conservatism is needed to accommodate the full capacity of the plastic hinging mechanism of the bridge system. • Establish four Seismic Design Categories (SDCs) A, B, C and D (refer to Table 8-2) • Allow three types of bridge structural system: o Type 1 – Design a ductile substructure with an essentially elastic superstructure. o Type 2 – Design an essentially elastic substructure with a ductile superstructure. o Type 3 – Design an elastic superstructure and substructure with a fusing mechanism at the interface between the superstructure and the substructure. (3) Critical/essential bridges are not specifically addressed in these Guidelines. Classification of critical/essential bridges includes: • Bridges that are required to be open to all traffic once inspected after the design earthquake and usable by emergency vehicles and for security, defense, economic, or secondary life safety purposes immediately after the design earthquake. • Bridges that should, as a minimum, be open to emergency vehicles and for security, defense or economic purposes after the design earthquake and open to all traffic within days after the event. • Bridges that are formally designated as critical for a defined local emergency plan. (4) Seismic effects of box culverts and buried structures need not be considered except where failure of the box culvert or buried structures will affect the bridge. (5) Adjacent bents within a frame or adjacent columns within a bent shall have an effective stiffness ratio equal to or greater than 0.75 while any two bents within a frame or any two columns within a bent shall have an effective stiffness ratio equal to or greater than 0.5. (6) The ratio of the fundamental periods of vibration (less flexible to more flexible) for adjacent frames in the longitudinal and transverse direction shall be equal to or greater than 0.75.

<table>
<tr><th>Items</th><th>JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)</th><th>AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)</th></tr>
<tr>
<td>2. Principles of Seismic Design</td>
<td>
(1) Seismic Performance of Bridges
<table>
<tr><th rowspan="2">Seismic Performance</th><th rowspan="2">Seismic Safety Design</th><th rowspan="2">Seismic Serviceability Design</th><th colspan="2">Seismic Reparability Design</th></tr>
<tr><th>Emergency Reparability</th><th>Permanent Reparability</th></tr>
<tr><td><u>Seismic Performance Level 1</u>
Keeping the sound functions of bridges</td><td>To prevent girders from unseating</td><td>To ensure the normal functions of bridges (within elastic limit states)</td><td>No repair work is needed to recover the functions</td><td>Only easy repair works are needed</td></tr>
<tr><td><u>Seismic Performance Level 2</u>
Limited damages and recovery</td><td>Same as above</td><td>Capable of recovering functions within a short period after the event</td><td>Capable of recovering functions by emergency repair works</td><td>Capable of easily undertaking permanent repair works</td></tr>
<tr><td><u>Seismic Performance Level 3</u>
No critical damages</td><td>Same as above</td><td>- *</td><td>-</td><td>-</td></tr>
</table>
*: “-“: Not covered

(2) Relationship between Design Earthquake Ground Motions and Seismic Performance of Bridges
<table>
<tr><th colspan="2">Levels of Earthquake Ground Motions</th><th>Class A Bridges*</th><th>Class B Bridges*</th></tr>
<tr><td colspan="2">Level 1: Highly probable during the bridge service life</td><td colspan="2">Seismic Performance Level 1 is required</td></tr>
<tr><td rowspan="2">Level 2</td><td>Type I: An Plate Boundary Type Earthquake with a Large Magnitude</td><td rowspan="2">Seismic Performance Level 3 is required</td><td rowspan="2">Seismic Performance Level 2 is required</td></tr>
<tr><td>Type II: An Inland Direct Strike Type Earthquake</td></tr>
</table>
*: Class A Bridges: Standard Importance; Class B Bridges: High Importance (Class A and B are classified according to such importance factors as road class, bridge functions and structural characteristics.)
When bridge importance is classified in view of roles expected in the regional disaster prevention plan and road serviceability, the following should be considered.
(a) To what extent a bridge is necessitated for post-event rescue and recovery activities as emergency transportation routes.
(b) To what extent damages to bridges (such as double-deck bridges and overbridges) affect other structures and facilities.
(c) Present traffic volume of the bridge and availability of substitute in case of the bridges losing pre-event functions.
(d) Difficulty (duration and cost) in recovering bridge function after the event.
</td>
<td>
(1) Bridges shall be designed for the life safety performance objective considering a seismic hazard corresponding to a seven percent probability of exceedance in 75 years.
(2) Life safety for the design event shall be taken to imply that the bridge has a low probability of collapse but may suffer significant damage and that significant disruption to service is possible. Partial or complete replacement may be required.
(3) <u>Performance Level</u>
<table>
<tr><th>Earthquake Level</th><th>Bridge Types</th><th>Serviceability Performance</th><th>Safety Performance</th></tr>
<tr><td>Moderate</td><td>Conventional bridge types</td><td>• Should resist earthquakes within the elastic range of the structural components</td><td>• Minimal damage</td></tr>
<tr><td rowspan="2">Large/Major</td><td>Conventional bridge types</td><td>• Significant disruption to service shall be taken to include limited access (reduced lanes, light emergency traffic) on the bridge.
• May include limited offsets and displacements.</td><td>• May suffer significant damage but with low probability of collapse.
• Significant damage shall be taken to include permanent offsets and damage consisting of:
- Cracking,
- Reinforcement yielding,
- Major spalling of concrete,
- Extensive yielding and local buckling of steel columns,
- Global and local buckling of steel braces, and
- Cracking in the bridge deck slab at shear studs.
• Partial or complete replacement of columns may be required.
• For sites with liquefaction, liquefaction-induced lateral flow, or liquefaction-induced lateral spreading, inelastic deformation may be permitted in piles and shafts. Partial and complete replacement of columns, piles or shafts may be necessary.</td></tr>
<tr><td>Critical bridges/ Essential bridges</td><td>• Required to be open to all traffic once inspected after seismic event and usable by emergency vehicles for security, defense, economic or secondary life safety purposes immediately after the seismic event
• Should, as a minimum, be open to emergency vehicles and for security, defense, or economic purposes after the seismic event and open to all traffic within days after that event.
• Significant disruption to service is possible or closure to repair the damage.</td><td>• Life safety with low probability of collapse but may suffer damage that is readily accessible for repair.</td></tr>
</table>
</td>
</tr>
</table>

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)
3. Loads to be considered in Seismic Design	(1) Loads and their Combinations (a) Primary Loads: Dead load (D), Pre-stress force (PS), Effect of creep of concrete (CR), Effect of drying shrinkage of concrete (SH), Earth pressure (E), Hydraulic pressure (HP), Buoyancy or Uplift (U) (b) Secondary loads: Effects of earthquake (EQ) (c) Combination of loads: Primary loads + Effects of earthquake (EQ) (d) Loads and their combinations shall be determined in such manners that they cause the most adverse stress, displacements and effects. (2) Effects of Earthquake (EQ) (a) Inertia force, (b) Earth pressure during an earthquake, (c) Hydrodynamic pressure during an earthquake, (d) Effects of liquefaction and liquefaction-induced ground flow, (e) Ground displacement during an earthquake	(1) Loads and Load Combinations are similar to *AASHTO LRFD Bridge Design Specifications* (a) Permanent Loads: Dead load of structural components and non-structural attachments (DC), Dead load of wearing surfaces (DW), Down drag (DD), Horizontal earth pressure load (EH), Earth surcharge (ES), Vertical pressure from dead load of earth fill (EV) Secondary force from post-tensioning (PS), Miscellaneous lock-in force due to construction process (EL), Force effects due to shrinkage (SH), Force due to creep (CR). (b) Transient Loads: Vehicular live load (LL), Water load and stream pressure (WA), Friction load (FR) (c) Earthquake Load (EQ) (d) Combination of Loads: Permanent Loads + Transient Loads + Earthquake Load (e)The load factors shall be selected to produce the total extreme factored force effects. Both positive and negative extremes shall be investigated. In load combinations where one force effect decreases another effect, the minimum value shall be applied to the load reducing the force effect. (2) Effects of Earthquake (EQ) (a) Bridge inertia effect, (b) Earth pressure during and earthquake, (c) Hydrodynamic pressure during an earthquake, (d) Live load inertia effect, (e) Potential for soil liquefaction, (f) Potential for slope movement

<table>
<tr><th>Items</th><th>JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)</th><th>AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)</th></tr>
<tr>
<td>4. Design Earthquake Ground Motions for Level 1 and Level 2</td>
<td>

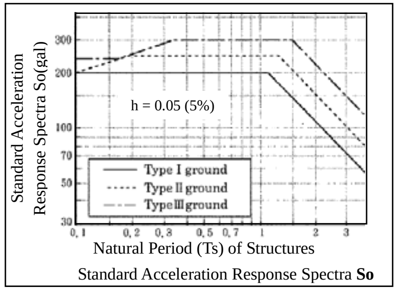
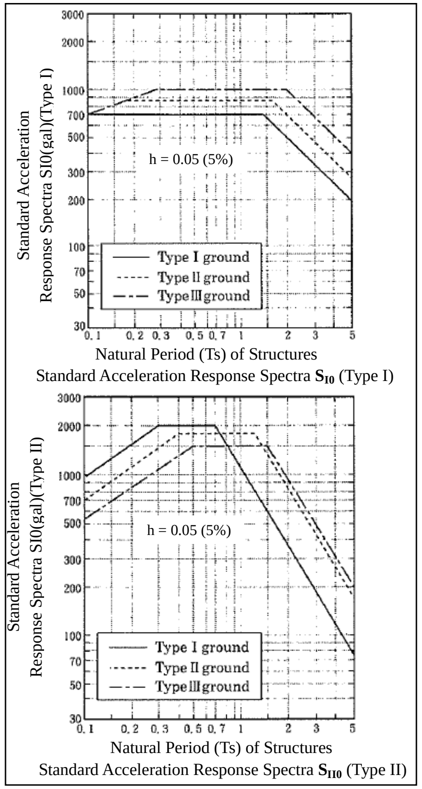


Fig.4-1 Level 1 Earthquake Ground Motion

S = C_Z*C_D***S_0** (S: ARS for Level 1EGM, S0: SARS (Fig.4-1))
SI = C_Z*C_D***S_{I0}** (SI: Type I ARS for Level 2 EGM, SI0:SARS (Fig.4-2))
SII = C_Z*C_D***S_{II0}** (SII: Type II ARS for Level 2 EGM, SII0: SARS(Fig.4-2))

(SARS= **S**tandard **A**cceleration **R**esponse **S**pectra, ARS= **A**cceleration **R**esponse **S**pectra, EGM = **E**arthquake **G**round **M**otion)
C_D: Modification factor for damping ratio (h) of structures (Fig.4-3)
C_Z: Modification factor for zones (Fig.4-4)



Standard Acceleration Response Spectra **S_{I0}** (Type I)

Standard Acceleration Response Spectra **S_{II0}** (Type II)

Fig 4-2 Level 2 Earthquake Ground Motions

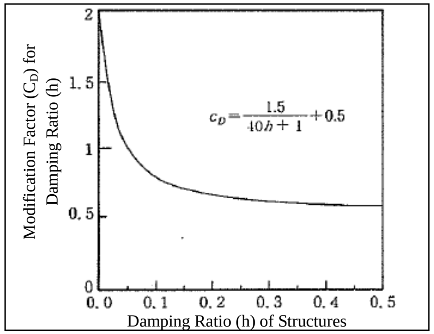
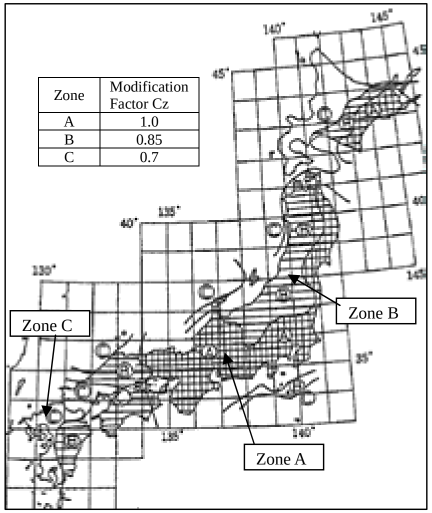


Fig.4-3 Modification Factor (CD) for Damping Ratio (h) of Structures



Zone	Modification Factor Cz
A	1.0
B	0.85
C	0.7

Fig.4-4 Modification Factors for Zones, Cz

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(1) The general procedure to develop the design spectrum (Figure 4-1) is to use the peak ground acceleration coefficient (PGA) and the short and long period response spectral acceleration coefficients (S_S and S_1) based on the maps prepared in the specifications.

The five-percent-damped-design response spectrum shall be taken as specified in Figure 4-1. This spectrum shall be calculated using the mapped peak ground acceleration coefficients and the response spectral acceleration coefficients scaled by zero-,short-, and long-period site factors, F_{pga}, F_a and F_v respectively.

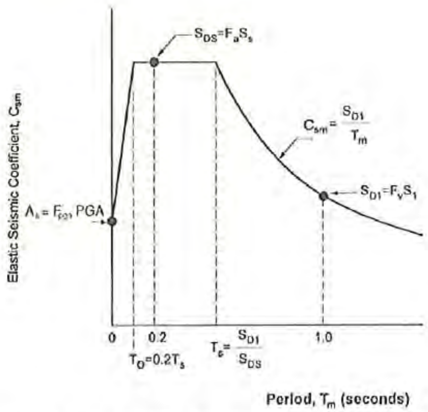


Figure 4-1 AASHTO Design Response Spectrum, Construction Using Three=Point Method

$$A_s = F_{pga}\,PGA \qquad (4\text{-}1)$$
$$S_{DS} = F_a\,S_s \qquad (4\text{-}2)$$
$$S_{D1} = F_v\,S_1 \qquad (4\text{-}3)$$

where:
A_s = peak seismic ground acceleration coefficient modified by short-period site factor
F_{pga} = site factor at zero-period on acceleration response spectrum
PGA = peak seismic ground acceleration coefficient on rock
S_{DS} = horizontal response spectral acceleration coefficient at 0.2-s period modified by short-period site factor
F_a = site factor for short-period range of acceleration response spectrum
S_s = horizontal response spectral acceleration coefficient at 0.2-s period on rock
C_{sm} = elastic seismic response coefficient for the mth mode of vibration
S_{D1} = horizontal response spectral acceleration coefficient at 1.0-s period modified by long-period site factor
F_v = site factor for long-period range of acceleration response spectrum
S_1 = horizontal response spectral acceleration coefficient at 1.0-s period on rock

For periods less than or equal to T_o, the design response spectral acceleration coefficient, S_a is calculated as:

$$S_a = (S_{DS} - A_s)\,T/\,T_o + A_s \qquad (4\text{-}4)$$

For periods greater than of equal to T_o, and less than of equal to T_S, the design response spectral acceleration coefficient, S_a is calculated as:

$$S_a = S_{DS} \qquad (4\text{-}5)$$

For periods greater than T_S, the design response spectral acceleration coefficient, S_a is calculated as:

$$S_a = S_{D1}/T \qquad (4\text{-}6)$$

</td>
</tr>
</table>

Items

5. Ground Type for Seismic Design

JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)

Table 5-1 Ground Types in Seismic Design

Ground Type	Characteristic Value of Ground, T_G (s)	Description
Type I	$T_G < 0.2$	Good diluvial ground and rock
Type II	$0.2 \le T_G < 0.6$	Diluvial and alluvial ground not belonging to Type I and Type II
Type III	$0.6 \le T_G$	Soft ground of alluvial ground

$T_G = 4 * \sum_{i=1}^{n} Hi / Vsi$ T_G = Characteristic value of ground (s)

Hi = Thickness of the i-th soil layer

Vsi = Average shear wave velocity of the i-th soil layer (m/s). If Vsi is not available, Vsi can be obtained from the following formula.

Vsi = 100 * $Ni^{1/3}$ ($1 \le Ni \le 25$): for cohesive soil layer (if N = 0, Vsi = 50 m/s; when N=25, Vsi = 300m/s)

Vsi = 80 * $Ni^{1/3}$ ($1 \le Ni \le 50$): for sandy soil layer (if N = 0, Vsi = 50 m/s; when N=50, Vsi = 300m/s)

Ni = Average N value of thei-th soil layer obtained from SPT

i = Number of the i-th layer from the ground surface when the ground is classified into “n” layers up to “the surface of a base ground surface for seismic design”

Note: “The surface of a base ground surface for seismic design” represents upper surface of a fully hard ground layer that exists over a wide area in the construction site, and normally situated below a surface soil layer shaking with a ground motion during an earthquake. Where, the upper surface of a fully hard ground layer might be the upper surface of a highly rigid soil layer with a shear elastic wave velocity of more than 300m/s (an N value of 25 in the cohesive soil layer and of 50 in the sandy soil layer)

If N value is not available, Ground types can be obtained following the flow chart shown in Fig 5-1.

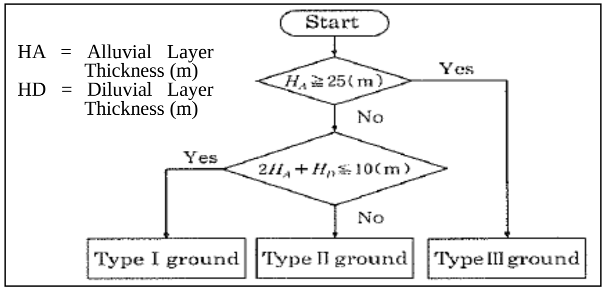


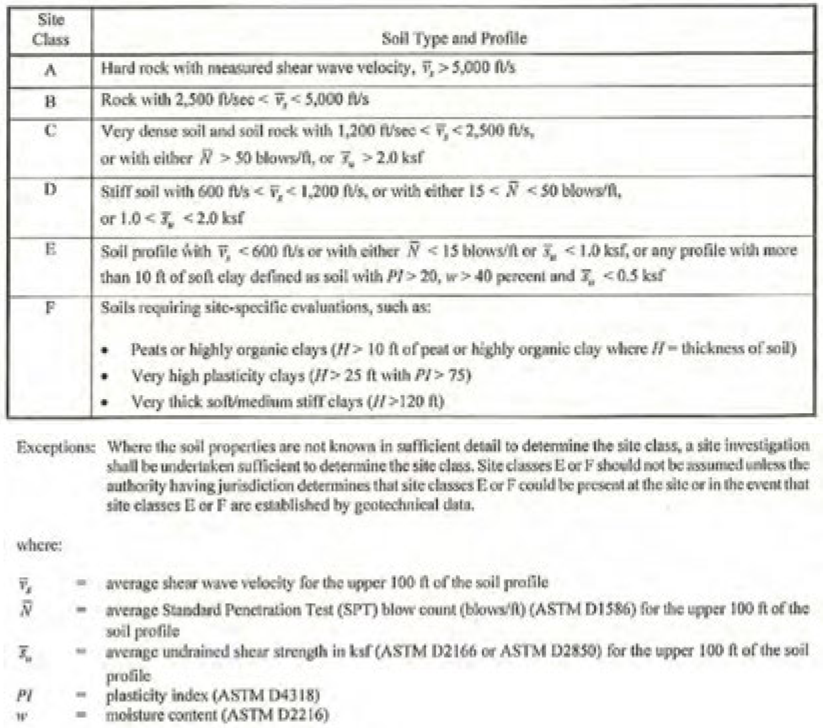
Fig.5-1 Flowchart for Determining Ground Types

AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)

(1) Site Class Definition

A site is classified as A through F in accordance with the site class definitions in Table 5-1.

Table 5-1 Site Class Definitions



Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft/s
B	Rock with 2,500 ft/sec < $\bar{v}_s$ < 5,000 ft/s
C	Very dense soil and soil rock with 1,200 ft/sec < $\bar{v}_s$ < 2,500 ft/s, or with either $\bar{N}$ > 50 blows/ft, or $\bar{s}_u$ > 2.0 ksf
D	Stiff soil with 600 ft/s < $\bar{v}_s$ < 1,200 ft/s, or with either 15 < $\bar{N}$ < 50 blows/ft, or 1.0 < $\bar{s}_u$ < 2.0 ksf
E	Soil profile with $\bar{v}_s$ < 600 ft/s or with either $\bar{N}$ < 15 blows/ft or $\bar{s}_u$ < 1.0 ksf, or any profile with more than 10 ft of soft clay defined as soil with PI > 20, w > 40 percent and $\bar{s}_u$ < 0.5 ksf
F	Soils requiring site-specific evaluations, such as: • Peats or highly organic clays (H > 10 ft of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays (H > 25 ft with PI > 75) • Very thick soft/medium stiff clays (H >120 ft)

Exceptions: Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site classes E or F should not be assumed unless the authority having jurisdiction determines that site classes E or F could be present at the site or in the event that site classes E or F are established by geotechnical data.

where:

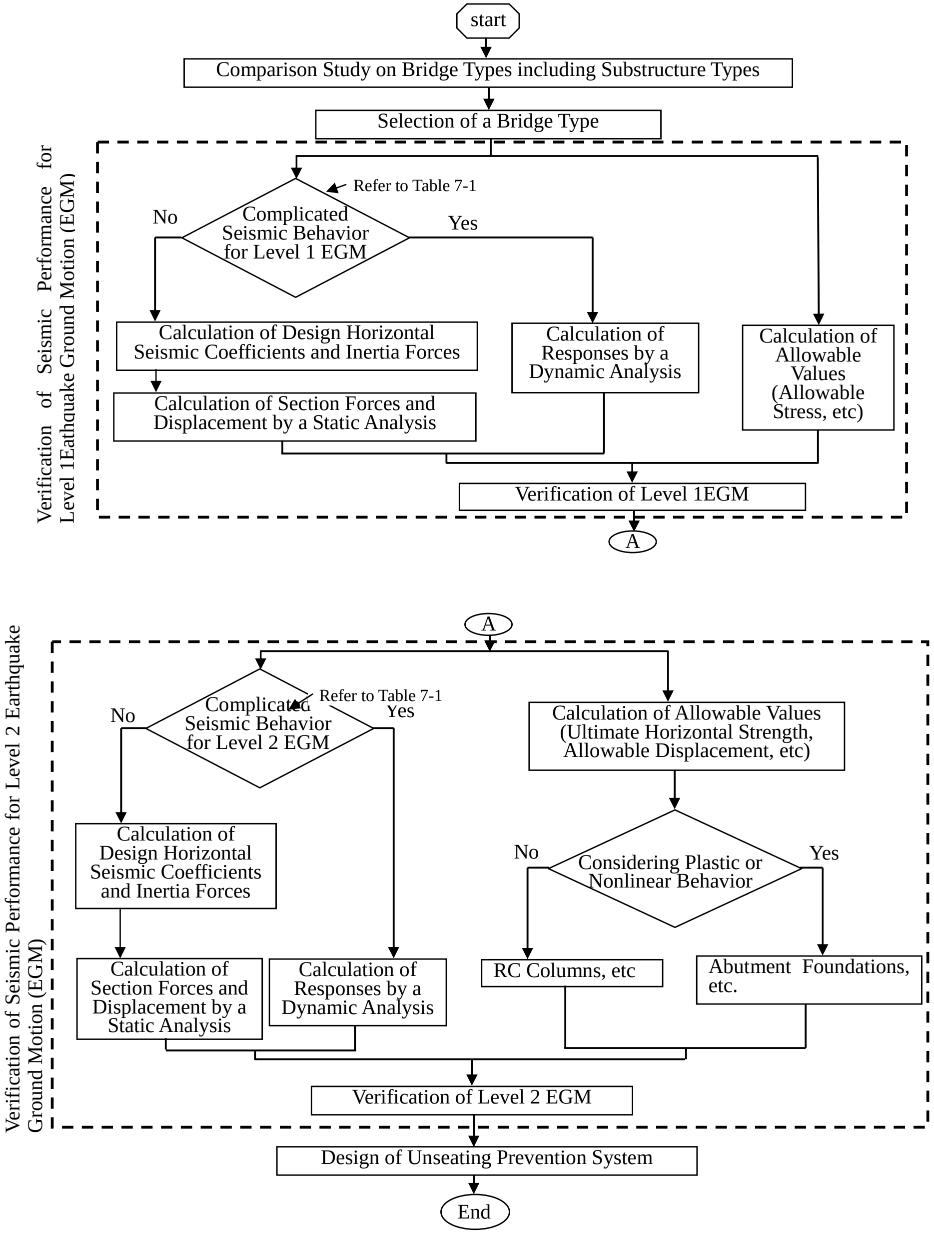
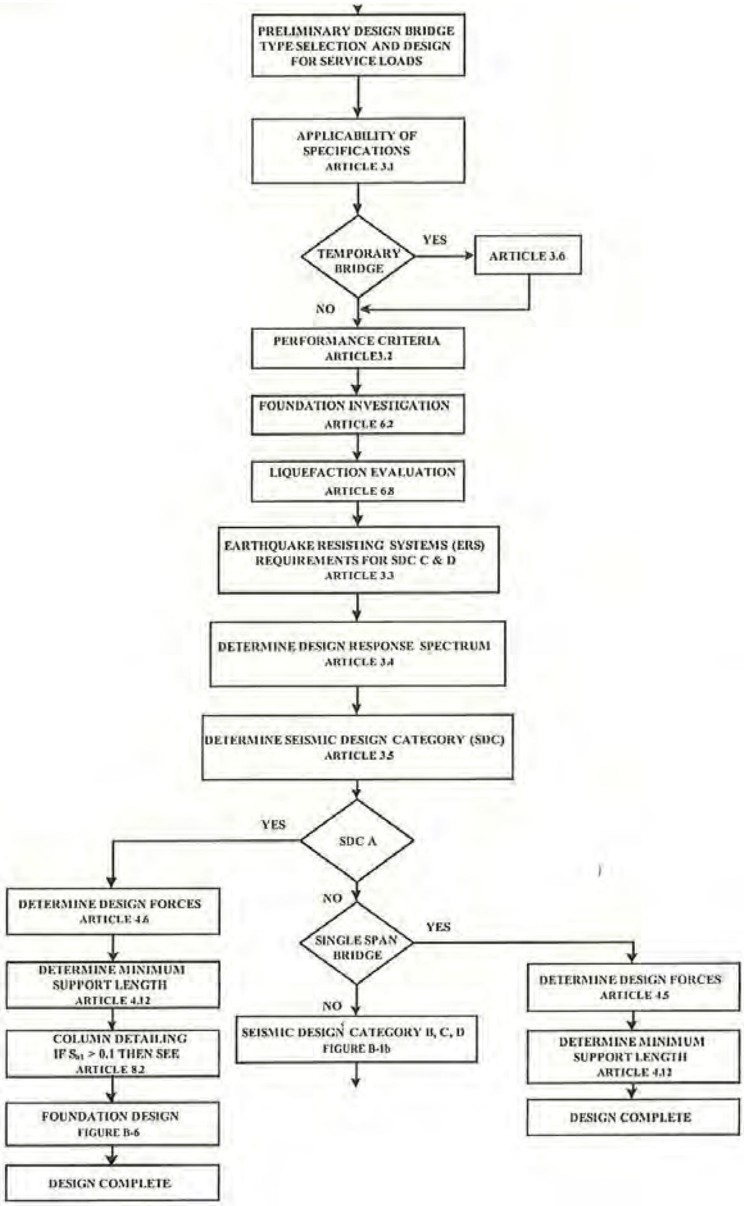
$\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile

$\bar{N}$ = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for the upper 100 ft of the soil profile

$\bar{s}_u$ = average undrained shear strength in ksf (ASTM D2166 or ASTM D2850) for the upper 100 ft of the soil profile

PI = plasticity index (ASTM D4318)

w = moisture content (ASTM D2216)

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)
6. Procedure of Seismic Design		 Figure 6-1a Seismic Design Procedure Flow Chart

<table>
<tr><th>Items</th><th>JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)</th><th>AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)</th></tr>
<tr><td></td><td></td><td>

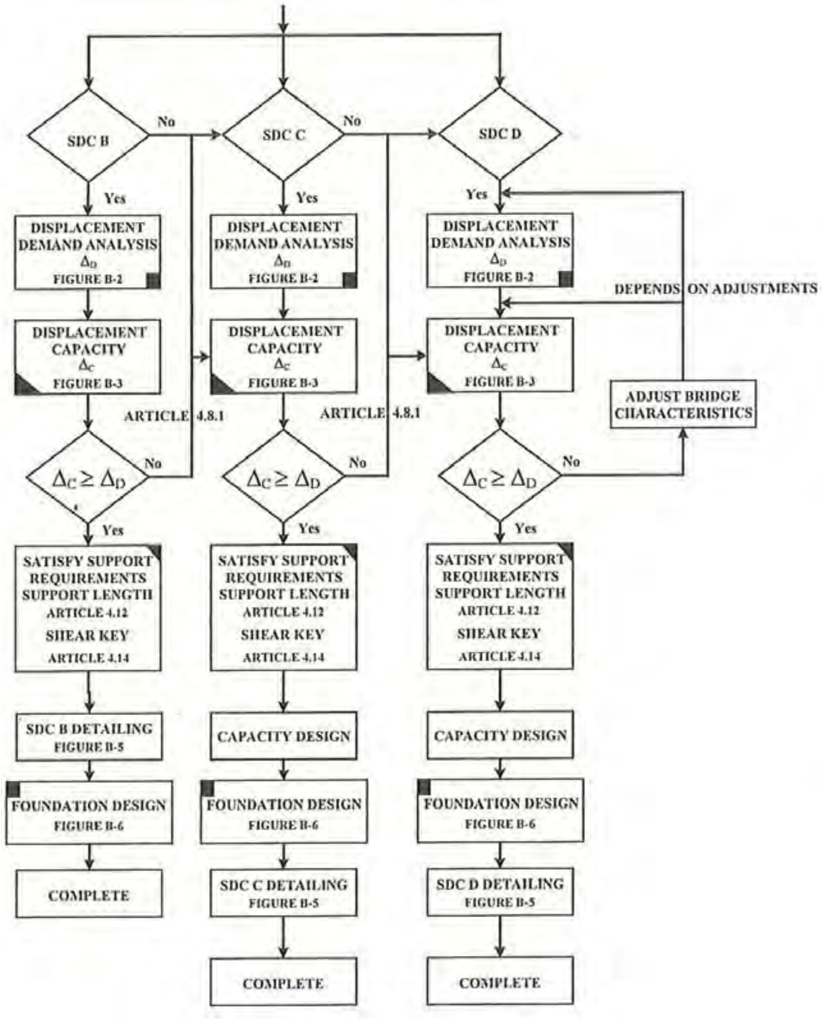


Figure 6-1b Seismic Design Procedure Flow Chart

</td></tr>
</table>

Items

7. Limit States of Bridges for each Seismic Performance Level

JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)

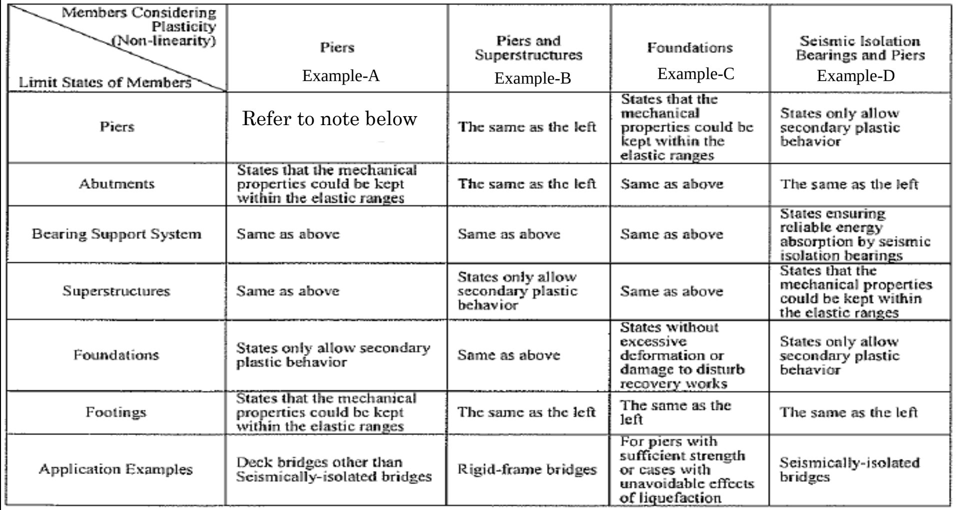
(1) Limit States of Bridges for Seismic Performance Level 1

(a) The mechanical properties of the bridges including expansion joint are maintained within the elastic range.

(b) Stress occurring in concrete of each structural member reaches its allowable stress multiplied by an increase factor of 1.5 for consideration of earthquake effects.

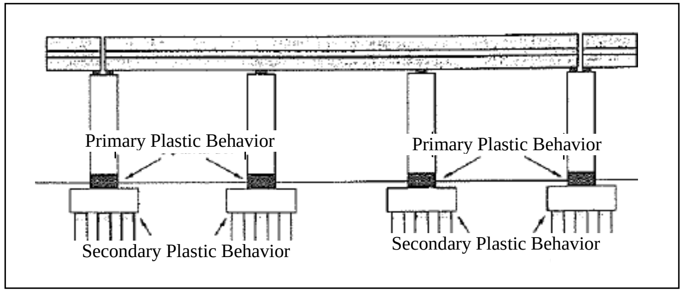
(2) Limit State of Bridges for Seismic Performance Level 2 (Refer to table 7-1 and Fig. 7-1)

Table 7-1 Limit States of Each Member with Applicable Examples for Seismic Performance Level 2 and Level 3 (Refer to Fig. 7-1 about Examples)



Members Considering Plasticity (Non-linearity) / Limit States of Members	Piers Example-A	Piers and Superstructures Example-B	Foundations Example-C	Seismic Isolation Bearings and Piers Example-D
Piers	Refer to note below	The same as the left	States that the mechanical properties could be kept within the elastic ranges	States only allow secondary plastic behavior
Abutments	States that the mechanical properties could be kept within the elastic ranges	The same as the left	Same as above	The same as the left
Bearing Support System	Same as above	Same as above	Same as above	States ensuring reliable energy absorption by seismic isolation bearings
Superstructures	Same as above	States only allow secondary plastic behavior	Same as above	States that the mechanical properties could be kept within the elastic ranges
Foundations	States only allow secondary plastic behavior	Same as above	States without excessive deformation or damage to disturb recovery works	States only allow secondary plastic behavior
Footings	States that the mechanical properties could be kept within the elastic ranges	The same as the left	The same as the left	The same as the left
Application Examples	Deck bridges other than Seismically-isolated bridges	Rigid-frame bridges	For piers with sufficient strength or cases with unavoidable effects of liquefaction	Seismically-isolated bridges

Note: States within a range of easy functional recovering for Seismic Performance Level 2; States that horizontal strength of piers start to get reduced rapidly for Seismic Performance Level 3.



(a) Example A: Single Column Pier with Plastic Behavior (in longitudinal direction)

AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)

(1) For SDC C or D, all bridges and their foundations shall have a clearly identifiable earthquake-resisting system (ERS) selected to achieve the life safety criteria.

(2) The ERS shall provide a reliable and uninterrupted lad path for transmitting seismically induced forces into the surrounding soil and sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements. All structural and foundation elements of the bridge shall be capable of achieving anticipated displacements consistent with the requirements of the chosen design strategy of seismic resistance and other structural requirements.

(3) Design shall be based on the following three Global Seismic Design Strategies based on the expected behavior characteristics of the bridge system (Table 7-1).

Table 7-1 Global Seismic Design Strategies

Design Strategy Type	Description
Type 1	• **Ductile Substructure with Essentially Elastic Superstructure.** This category includes conventional plastic hinging in columns and walls and abutments that limits inertial forces by full mobilization of passive soil resistance. Also included are foundations hat may limit inertial forces by in-ground hinging, such as pile bents and integral abutments on piles.
Type 2	• **Essentially Elastic Substructure with a Ductile Superstructure.** This category applies only to steel superstructures, and ductility is achieved by ductile elements in the pier cross-frames.
Type 3	• **Elastic Superstructure and Substructure with a Fusing Mechanism between the Two.** This category includes seismically isolated structures and structures in which supplemental energy-dissipation devices, such as dampers, are used to control inertial forces transferred between the superstructure and substructure.

(4) Earthquake-resisting elements (EREs) are categorized as: (a) Permissible (Figure 7-1 and 7-2), (b) Permissible with Owner's approval (Figure 7-3), and (c) Not recommended for new bridges (Figure 7-4).

(5) Permissible systems and elements have the following characteristics:

- All significant inelastic action shall be ductile and occur in locations with adequate access for inspection and repair. Piles subject to lateral movement from lateral flow resulting from liquefaction are permitted to hinge below the ground line provided the Owner is informed and does not require any higher performance criteria for a specific objective. If all structural elements of a bridge are designed elastically, then no inelastic deformation is anticipated and elastic elements are permissible, but minimum detailing is required according to the bridge seismic design category.
- Inelastic action of a structural member does not jeopardize the gravity load support capability of the structure (e.g., cap beam and superstructure hinging).

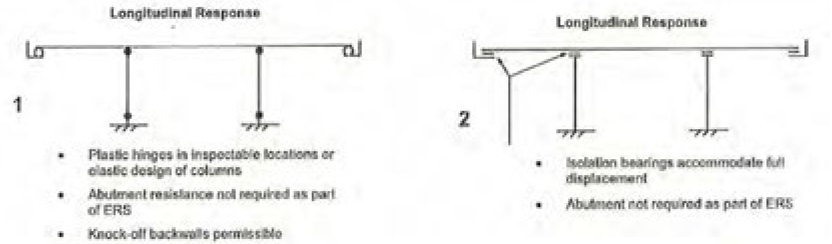


Figure 7-1a Permissible Earthquake-Resisting Systems (ERSs)

<table>
<tr><th>Items</th><th>JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)</th><th>AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)</th></tr>
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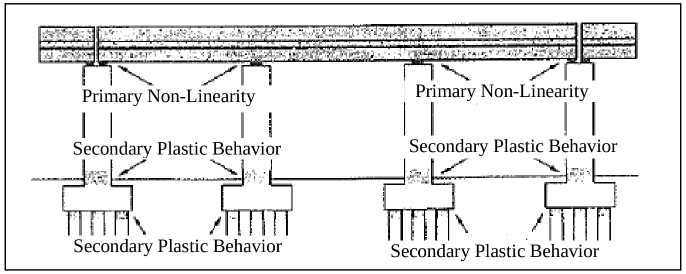
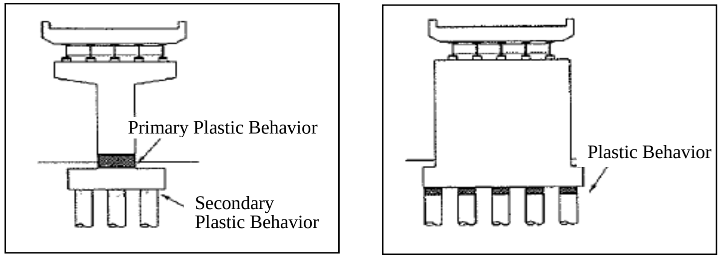
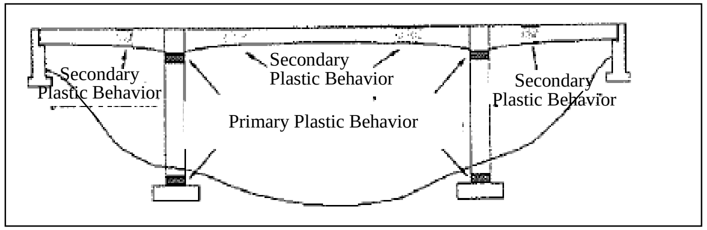


(b) Example D: Seismic Isolation Bearing with Consideration of Non-Linearity(in longitudinal direction)



(c) Example A: Single Column Pier with Plastic Behavior (in Transverse direction)

(d) Example C: Foundations with Plastic Behavior (Pier Wall, in Transverse direction)



(e) Example B: Plasticity in Piers and Superstructures (Rigid-Frame Bridges in Transverse direction)

Fig. 7-1 Limit States of Each Member with Applicable Examples for Seismic Performance Level 2 and Level 3

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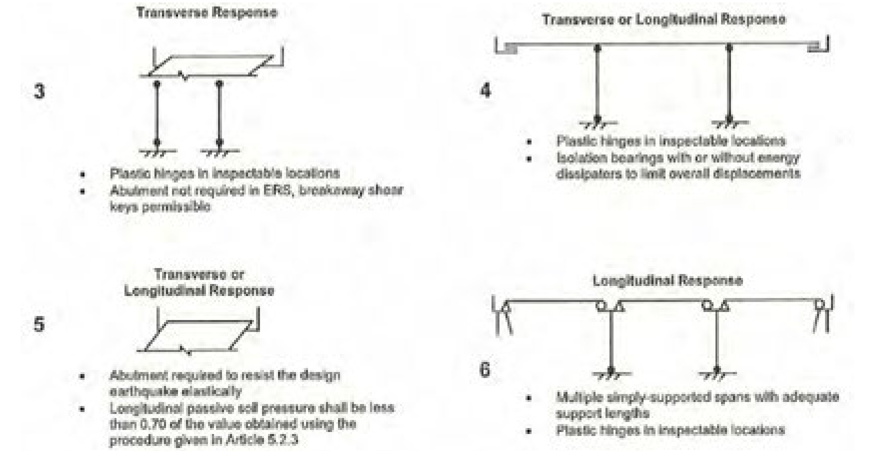

Figure 7-1b Permissible Earthquake-Resisting Systems (ERSs)

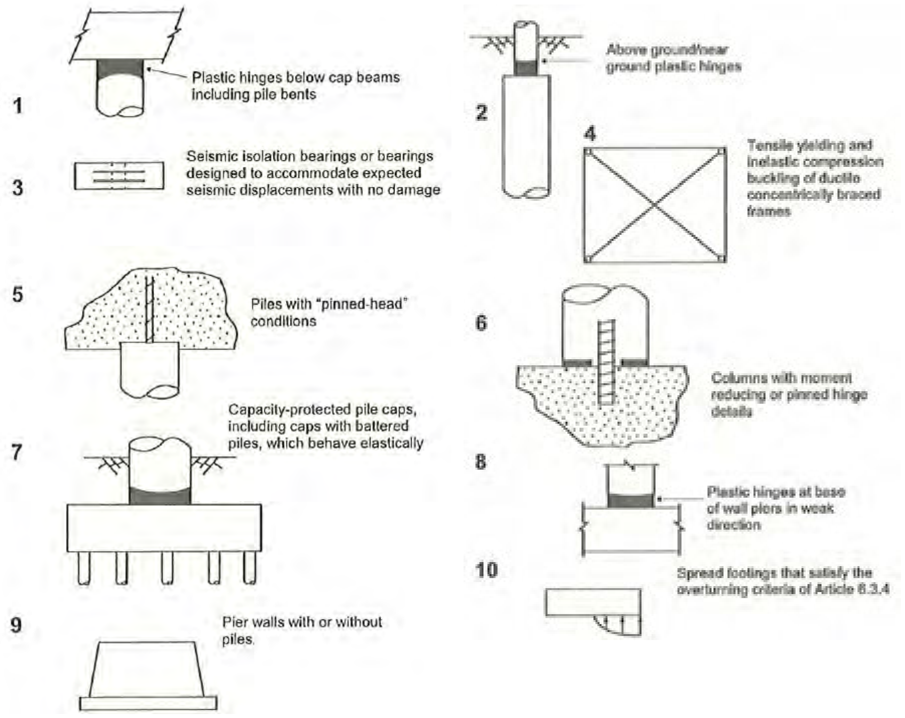

Figure 7-2a Permissible Earthquake-Resisting Elements (EREs)

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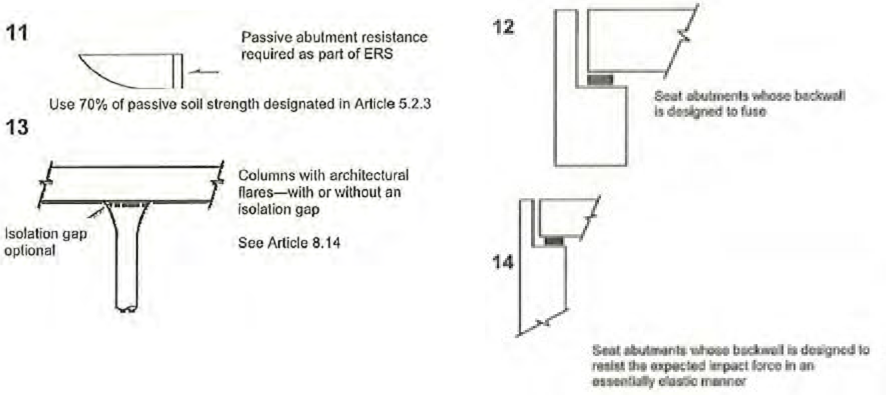
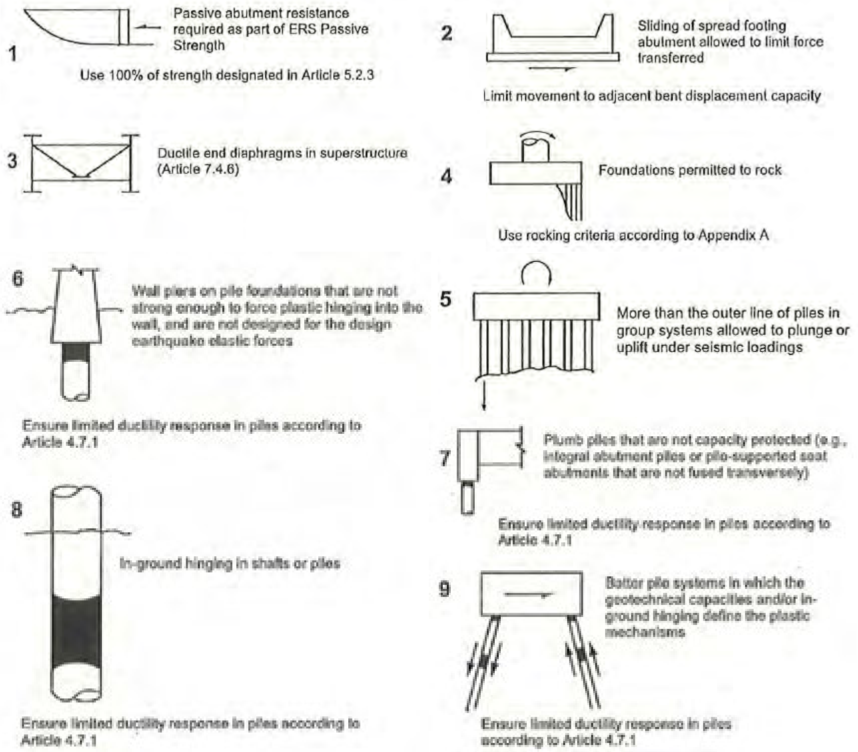
</table>

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)
		 Figure 7-2b Permissible Earthquake-Resisting Elements (EREs)  Figure 7-3 Permissible Earthquake-Resisting Elements Requiring Owner's Approval

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)

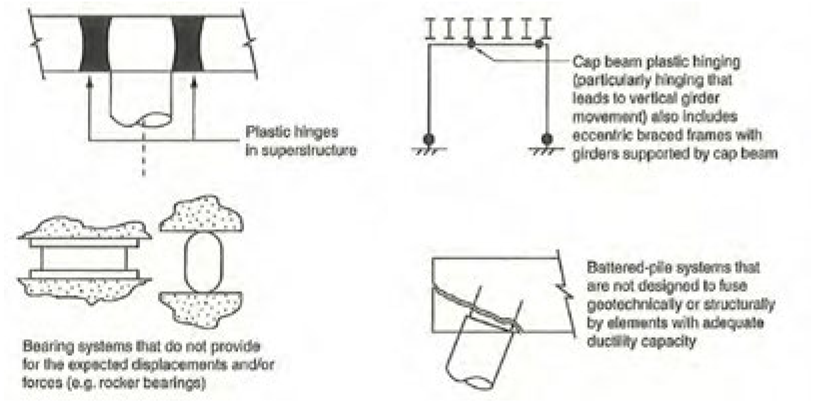


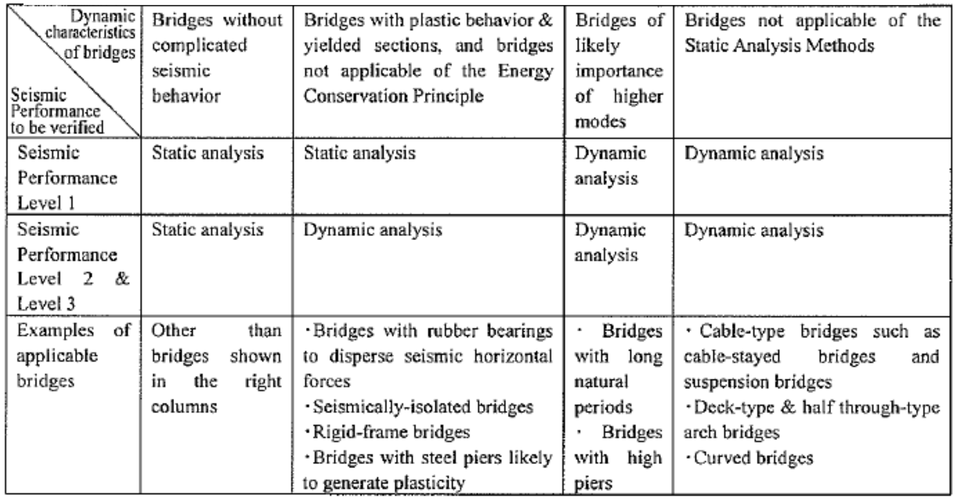
Figure 7-3 Earthquake-Resisting Elements that are Not Recommended for New Bridges

8. Design Method Applicable for Seismic Performance Verification

The bridge types and design methods applicable to seismic performance verification is summarized in Table 3.

- Although dynamic analysis methods can be applied to bridges without complicated seismic behavior, it is recommended to use static analysis methods because the verification in accordance with static method is generally feasible for these bridges.
- Since the seismic behavior of bridges with predominant first mode of vibration and plural plastic behavior or bridges in which investigation on application of Energy Conservation Principle remains unclear may become complicated due to plasticity of members, their Seismic Performance Level 1should be verified by the static analysis methods but Seismic Performance Level 2 or Level 3 be verified by dynamic methods.

Table 8-1 Relationship between Complexities of Seismic Behavior and Design Methods Applicable for Seismic Performance Verification



Dynamic characteristics of bridges / Seismic Performance to be verified	Bridges without complicated seismic behavior	Bridges with plastic behavior & yielded sections, and bridges not applicable of the Energy Conservation Principle	Bridges of likely importance of higher modes	Bridges not applicable of the Static Analysis Methods
Seismic Performance Level 1	Static analysis	Static analysis	Dynamic analysis	Dynamic analysis
Seismic Performance Level 2 & Level 3	Static analysis	Dynamic analysis	Dynamic analysis	Dynamic analysis
Examples of applicable bridges	Other than bridges shown in the right columns	･Bridges with rubber bearings to disperse seismic horizontal forces ･Seismically-isolated bridges ･Rigid-frame bridges ･Bridges with steel piers likely to generate plasticity	･ Bridges with long natural periods ･ Bridges with high piers	･Cable-type bridges such as cable-stayed bridges and suspension bridges ･Deck-type & half through-type arch bridges ･Curved bridges

(1) Each bridge shall be assigned to one of four seismic design categories (SDCs), A through D, based on 1-sec period design spectral acceleration for the design earthquake (S_{D1}) (refer to Table 8-1).

(2) If liquefaction-induced lateral spreading or slope failure that may impact the stability of the bridge could occur, the bridge should be designed with SDC D, regardless of the magnitude S_{D1}.

Table 8-1 Seismic Design Categories

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

(3) The requirements for each of the proposed SDCs shall be taken as shown in Figure 8-1 and described below.

Table 8-2 SDC Requirements

Design Requirements	**Seismic Design Category (SDC)**			
	A	**B**	**C**	**D**
1. Identification of Earthquake Resisting System (ERS)	Not required	To be considered	Required	Required
2. Demand Analysis	Not required	Required	Required	Required
3. Capacity Check	Implicit capacity check not required	Implicit capacity check required (displacement, P-Δ, support length)	Implicit capacity check required (displacement, P-Δ, support length)	Required (displacement by pushover analysis, P-Δ, support length)
4. Capacity design	Not required	To be considered for column shear; considered to avoid weak links in the ERS	Required including column shear requirement	Required
5. Detailing Level	Minimum detailing for support length, superstructure/ substructure	SDC B level	SDC C level	SDC D level

<table>
<tr><th>Items</th><th>JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)</th><th>AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)</th></tr>
<tr><td></td><td></td><td>
<table>
<tr><td></td><td>connection design force, column and transverse steel</td><td></td><td></td><td></td></tr>
<tr><td>6. Liquefaction Evaluation</td><td>Not required</td><td>To be considered for certain conditions</td><td>Required</td><td>Required</td></tr>
</table>

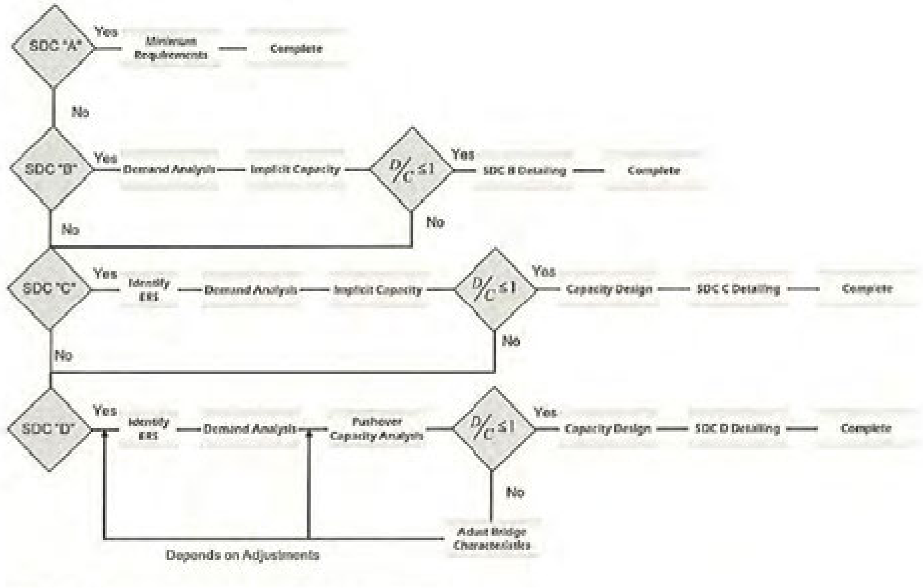


Figure 8-1 Seismic Design Category (SDC) Core Flowchart

Table 8-3 Analysis Procedures

<table>
<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>
<tr><td>A</td><td>Not required</td><td>Not required</td></tr>
<tr><td>B, C or D</td><td>Equivalent Static or Elastic Dynamic Analysis</td><td>Elastic Dynamic Analysis</td></tr>
</table>

(4) Nonlinear time history procedure is generally not required unless:

- P-D effects are too large to be neglected,
- Damping provided by a base isolation system is large, and
- Requested by Owner.
</td></tr>
<tr><td>9. Calculation of Natural Period</td><td>
(1) Natural periods shall be appropriately calculated with considering of the effects of deformations of structural members and foundations.
(2) Natural Period of the Design Vibration Unit (s) (T (s))

$T = 2.01 * \sqrt{\delta}$ ------------------ (9-1)

where, δ can be calculated as follows.
(a) in case of a design vibration unit consisting of substructure and its supporting superstructure part as shown in Fig. 9-1
</td><td>
Procedure 1:

The **Equivalent Static Analysis (ESA)** may be used to estimate displacement demands for structures or individual frames with well-balanced spans and uniformly distributed stiffness where the response can be captured by a predominant translational mode. Both uniform load method and the single-mode spectral analysis method shall be considered acceptable equivalent static analysis procedures.

(1) Natural period calculation by **Single Mode Spectral Method**

The single-mode method of spectral analysis shall be based on the fundamental mode of vibration in either the longitudinal or transverse direction. For regular bridges, the fundamental modes of vibration in the horizontal
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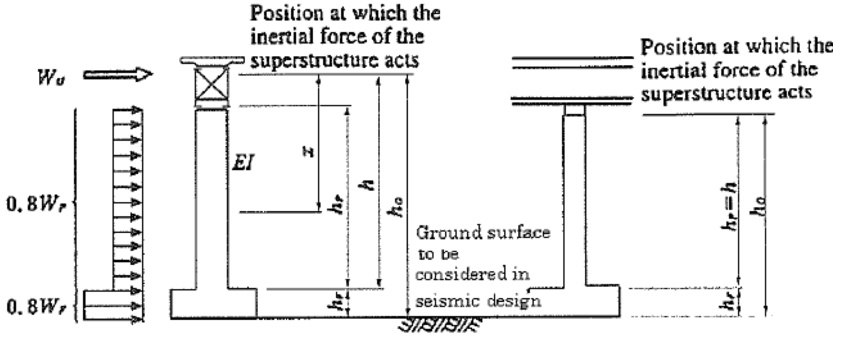
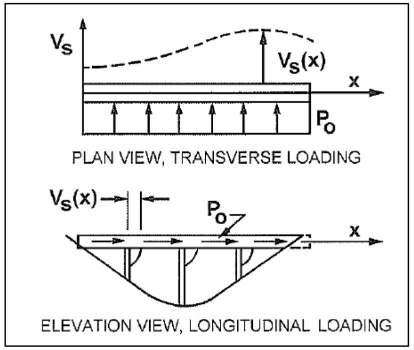
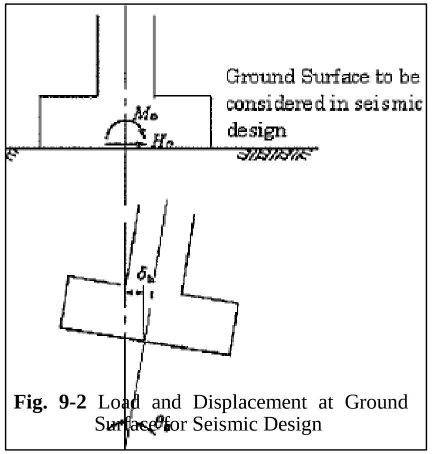
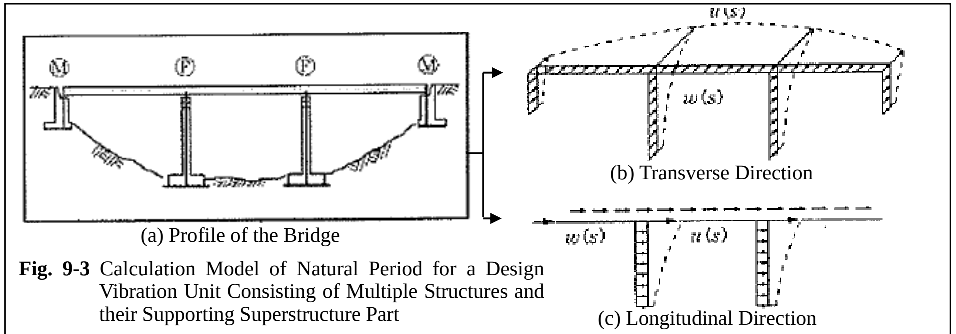
Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)
	 (a) Transverse Direction (b) Longitudinal Direction **Fig. 9-1** Calculation Model of Natural Period for A Design Vibration Unit Consisting of One Substructure and its Supporting Superstructure Part $\delta = \delta_P + \delta_0 + \theta_0 h_0$ ·········· (9-2) where δ_P: Bending deformation of substructure body (m) δ_0: Lateral displacement of foundation (m) θ_0: Rotation angle of foundation (rad) h_0: Height from the ground surface to be considered in seismic design to the height of superstructural inertia force (m) When the body of the substructure has a uniform section, the bending deformation δ_P can be calculated by Eq (9-3). $\delta_P = \frac{W_U h^3}{3EI} + \frac{0.8\ W_P h_P{}^3}{8EI}$ ·········· (9-3) where W_U: Weight of the superstructure portion supported by the substructure body concerned (kN) W_p : Weight of the substructure body (kN) EI: Bending stiffness of the substructure body specified in the explanations of (1) above (kN · m^2). h: Height from the bottom of the substructure body to the height of the superstructural inertia force (m) h_P: height of the substructure body (m) Where, δ0 and θ0 are calculated from Eq.(9-4) (Refer to Fig.9-2).	plane coincide with the longitudinal and transverse axes of the bridge structure. This mode shape may be found by applying a uniform horizontal load to the structure and calculating the corresponding deformed shape. The natural period may be calculated by equating the maximum potential and kinetic energies associated with the fundamental mode shape. The amplitude of the displaced shape may be found from the elastic seismic response coefficient, C_{sm}, and the corresponding spectral displacement. This amplitude shall be used to determine force effects.  Figure 9-1 Bridge Deck Subjected to Assumed Transverse and Longitudinal Loading • Calculate the static displacement $v_s(x)$ due to an assumed uniform loading p_o as shown in Figure 9-1. • Calculate factors a and g as: $\alpha = \int v_s(x)dx$ (9-1) $\gamma = \int w(x)v_s^2(x)dx$ (9-2) where: p_o = a uniform load arbitrarily set equal to 1.0 (N/mm) $v_s(x)$ = deformation corresponding to p_o (mm) $w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure (N/mm) • Calculate the period of the bridge as: $T_m = 2\pi \sqrt{\frac{\gamma}{p_o g \alpha}}$ (9-3) where: g = acceleration of gravity (m/sec^2) (2) Natural period calculation by **Uniform Load Method** The uniform load method shall be based on the fundamental mode of vibration in either the longitudinal or transverse direction of the base structure. The period of this mode of vibration shall be taken as that of an equivalent single-mass oscillator. The stiffness of this equivalent spring shall be calculated using the maximum displacement that occurs when an arbitrary uniform lateral load is applied to the bridge. The elastic seismic response coefficient, C_{sm}, shall be used to calculate the equivalent uniform seismic load from which force effects are found. This method is essentially an equivalent static method of analysis that uses the uniform lateral load to approximate the effect of seismic loads. This method is suitable for regular bridges that respond principally in their fundamental mode of vibration. Whereas all displacements and most member forces are calculated with good accuracy, the method is known to overestimate the transverse shears at the abutments by up to 100 percent. • Calculate the static displacement $v_s(x)$ due to an assumed uniform loading p_o as shown in Figure 9-1. The uniform load p_o is applied over the length of the bridge; it has units of force per unit length and may be arbitrarily set equal to 1.0. The static displacement $v_s(x)$ has unit of length.

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)
	$\left.\begin{aligned}\delta_0 &= \frac{H_0A_{rr}-M_0A_{sr}}{A_{ss}A_{rr}-A_{sr}A_{rs}}\\ \theta_0 &= \frac{-H_0A_{rs}+M_0A_{ss}}{A_{ss}A_{rr}-A_{sr}A_{rs}}\end{aligned}\right\}$ ---------- (9-4). $\left.\begin{aligned}H_0 &= W_U+0.8(W_P+W_F)\\ M_0 &= W_Uh_0+0.8W_P\left(\frac{h_P}{2}+h_F\right)+0.8W_F\frac{h_F}{2}\end{aligned}\right\}$  **Fig. 9-2** Load and Displacement at Ground Surface for Seismic Design Where, δ0 and θ0 are calculated from Eq. (9-4) (Refer to Fig.9-2). Arr, Asr, Ars and Ass are spring constants (kN/m, kN/rad, kNm/m, kNm/rad) and calculated from the following formula according to the foundation types. ① Spread foundation $\left.\begin{aligned}A_{ss} &= k_{SB}A_B\\ A_{sr} &= A_{rs} = 0\\ A_{rr} &= k_VI_B\end{aligned}\right\}$ ------ (9-5) ② Pile foundation with only vertical piles arranged symmetrically $\left.\begin{aligned}A_{ss} &= nK_1\\ A_{sr} &= A_{rs} = -nK_2\\ A_{rr} &= nK_4+K_{VP}\sum_{i=1}^{n}y_i^2\end{aligned}\right\}$ --------- (9-6) where k_{SB}: Coefficient of shear subgrade reaction in horizontal direction at the bottom of foundation (kN/m^3) k_V: Coefficient of subgrade reaction in vertical direction at the bottom of foundation (kN/m^3) n : Total number of piles y_i : y coordinate of the pile head of i-th pile (m) K_1, K_2 K_3 K_4: Spring constants (kN/m, kN/rad, kN·m/m, kN·m/rad) of the piles in the perpendicular direction to the pile axis in case of rigid connection at pile head K_{VP}: Spring constant (kN/m) of the piles in the direction of the pile axis Concrete calculation methods of K1, K2, K3, K4 and Kvp are provided in Part IV. With the coefficients of subgrade reaction for seismic design as shown in Eq. (9-7) and (9-8), K1, K2, K3, K4 and Kvp .can be obtained. $K_{H0} = 1/0.3 * E_D$ ---------- (9-7) $K_{V0} = 1/0.3 * E_D$ ---------- (9-8) where, K_{H0}, K_{V0} = Reference values of the coefficient of subgrade reaction in horizontal direction and in vertical direction, respectively (kN/m^3) (for Level 1 and 2 EGMs) E_D = 2 * (1 + ν_D) * G_D (E_D: Dynamic modulus of deformation of the ground (kN/m^2)) ν_D = Dynamic Poisson's ratio of the ground G_D = γt/g * V^2_{SD} (G_D: Dynamic shear deformation modulus of the ground (kN/m^2)) γt = Unit weight of the ground (kN/m^3) g = Acceleration of gravity (9.8 m/s^2) V_{SD} = Shear elastic wave velocity of the ground (m/s) V_{SDi} = CV * Vsi (V_{SDi}: the average shear elastic wave velocity of the i-th layer) C_V = 0.8 (Vsi < 300 m/s), 1.0 (Vsi ≥ 300 m/s) (CV: Modification factor based on degree of ground strain) Vsi = the average shear elastic wave velocity of thei-th soil layer described in Item 5 (m/s)	• Calculate the bridge lateral stiffness, *K*, and the total weight, *W*, from the following expressions: $K = \frac{p_oL}{v_{sMAX}}$ (9-4) $W = \int w(x)dx$ (9-5) where: L = total length of the bridge (mm) v_{sMAX} = maximum value of $v_s(x)$ (mm) $w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure (N/mm) The weight should take into account structural elements and other relevant loads including, but not limited to, pier caps, abutments, columns and footings. Other loads, such as live loads may be included. • Calculate the period of the bridge, T_m, using the expression: $T_m = 2\pi\sqrt{\frac{W}{gK}}$ (9-6) where: g = acceleration of gravity (m/sec^2) **Procedure 2: Elastic Dynamic Analysis (EDA)** For the elastic dynamic analysis, all relevant damped (5%) modes and frequencies shall be considered.

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)
	(b) in case of a design vibration unit consisting of multiple substructures and their supporting superstructure part as shown in Fig. 9-3.  (a) Profile of the Bridge (b) Transverse Direction (c) Longitudinal Direction **Fig. 9-3** Calculation Model of Natural Period for a Design Vibration Unit Consisting of Multiple Structures and their Supporting Superstructure Part $\delta = \frac{\int w(s)\, u(s)^2 ds}{\int w(s)\, u(s)\, ds}$ ……………………………………………… (9-9) where, $w(s)$:Weight of the superstructure or the substructure at position s (kN/m) $u(s)$: Lateral displacement of each structure at position s in the direction of the inertia force when a lateral force corresponding to the weight of the superstructure and that of the substructure above the design ground surface act in the direction of the inertia force (m) When a bridge is modeled into a discrete skeleton structure, the δ can be obtained from Eq. (9-10). $\delta = \frac{\sum_i (W_i u_i^2)}{\sum_i (W_i u_i)}$ ……………………………………………… (9-10) where W_i: Weight of superstructure and substructure at node i (kN) u_i: Displacement of node i occurred in the acting direction of inertia force when a force corresponding to the weight of the superstructure and the substructure above the ground surface to be considered in seismic design is acted in the acting direction of inertia force(m) ∑ represents the sum of all design vibration units. When calculated with eigenvalue analysis, .the natural period (T) can be obtained directly. (c) Stiffness Applied to Calculation of Natural Period for Level 2 EGM During verification of seismic performance for Level 2 EGM, the natural period shall be calculated based on the yield stiffness. The yield stiffness refers to the secant stiffness Ky at yield point due to bending deformation of the pier and is obtained at the ratio of the yield strength Py to the yield displacement δy of the pier (Ky = Py/δy) as shown in Fig. 9-4.	

<table>
<tr><th>Items</th><th>JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)</th><th>AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)</th></tr>
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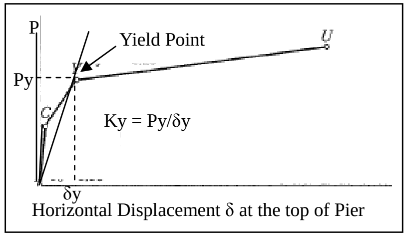

$Ky = 3*E*Iy / h^3$ ⟹ $Iy = Ky * h^3 / 3E$

EIy = yield stiffness

E = elastic modulus of pier, Iy = moment of section inertia at the yield point

h = the height of pier

(Iy should be used for every calculation of the natural period for verification of seismic performance for Level 2 EGM)

Fig. 9-4 Stiffness Applied to Calculation of Natural Period for Level 2 EGM

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<tr><td>10. Design Horizontal Seismic Coefficient for Level 1 EGM</td><td>

(1) Design horizontal coefficient (k_h) for Level 1 EGM shall be calculated by Eq. (10-1).

$K_h = C_Z * K_{h0}$ (≥ 0.1) -- (10-1)

where,

K_{h0} = Standard value of the design horizontal seismic coefficient for Level 1 EGM, which is shown in Fig 10-1.(Fig. 10-1 is obtained from the Figure for Level 1 EGM shown in Item 4 by dividing S_0 by gravity acceleration)

C_Z = Modification factor for zones shown in Item 4.

(2) Design horizontal coefficient (K_{hg}) at ground level can be obtained from Eq. (10-2), which is used for calculation of inertia force due to soil weight and seismic earth pressure in verifying seismic performance for Level 1 EGM.

$K_{hg} = C_Z * K_{hg0}$ -- (10-2)

where,

K_{hg0} = Standard value of the design horizontal seismic coefficient at ground surface level for Level 1 EGM

= 0.16 for Ground Type I, 0.2 for Ground Type II and 0.24 for Ground Type III

(3) Though a single value of the design horizontal seismic coefficient shall generally be adopted within the same design vibration unit, different design horizontal seismic coefficient for each pier are given in case that the ground type changes within the same design vibration unit.

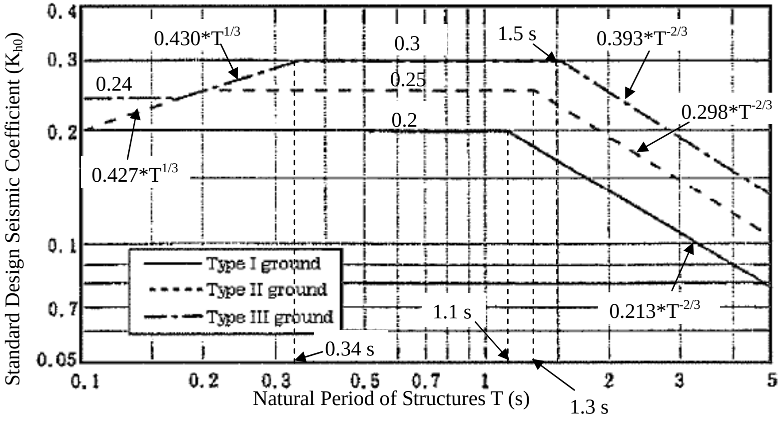

Fig. 10-1 Standard Design Seismic Coefficient for Level 1 EGM

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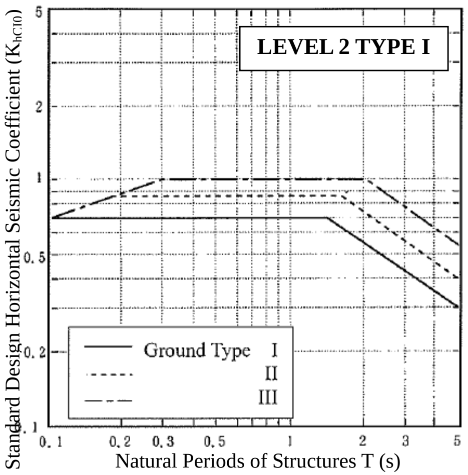
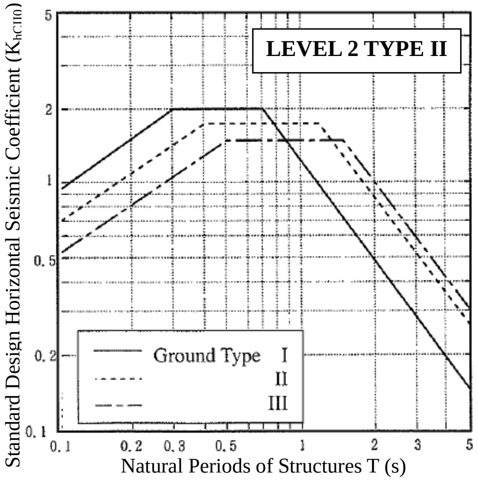
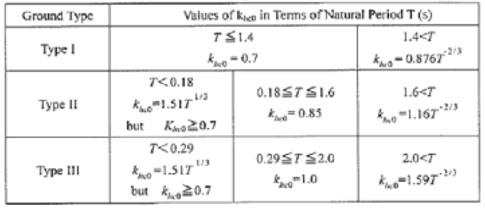
</table>

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)
11. Design Horizontal Seismic Coefficient for Level 2 EGM	(1) Design horizontal seismic coefficients for Level 2 EGM (Type I and II) shall be calculated by Eq. (11-1) and (11-2). $K_{hCI} = C_S * C_Z * K_{hC0I} \geq 0.3 * C_S$ or $0.4 * C_Z$ -- (11-1) $K_{hCII} = C_S * C_Z * K_{hC0II} \geq 0.6 * C_S$ or $0.4 * C_Z$ -- (11-2) K_{hCI} and K_{hCII} = Design horizontal seismic coefficient for Type I and II of Level 2 EGM, respectively. K_{hC0I} and K_{hC0II} = Standard design horizontal seismic coefficients for Type I and Type II of Level 2 EGM, respectively, which are shown in Fig 11-1 and Fig. 11-12. C_S = Force Reduction Factor related to the extent of ductility of a pier, which is specified in Item 12. (2) Design horizontal coefficients at ground surface level can be obtained from Eq. (11-3) and (11-4) for Type I and II of Level 2 EGM $K_{hgI} = C_Z * K_{hgI0}$ -- (11-3) $K_{hgII} = C_Z * K_{hgII0}$ -- (11-4) KhgI and KhgII = Design horizontal seismic coefficients at ground surface for Type I and II of Level 2 EGM, respectively. K_{hgI0} and K_{hgII0} = Standard horizontal seismic coefficient at ground surface level for Type I and II of Level 2 EGM, respectively. K_{hgI0} = 0.3 for Ground Type I, 0.35 for Ground Type II and 0.40 for Ground Type III K_{hgII0} = 0.80 for Ground Type I, 0.70 for Ground Type II and 0.60 for Ground Type III (3) The highest value of design horizontal seismic coefficient shall generally be used in each design vibration unit.  Fig. 11- 1 Relationship between K_{hCI0} and T (s)  Fig. 11- 2 Relationship between K_{hCII0} and T (s)	(1) For **regular bridges with 2 to 6 spans**, an **Equivalent Static Analysis (ESA)** is allowed when the S**eismic Design Category (SDC)** is classified as **B, C or D**. (2) The **Uniform Load Method** can be used as an ESA to approximate the effect of seismic load. This method shall be based on the fundamental mode of vibration in either the longitudinal or transverse direction. The period of this mode of vibration shall be taken as that of an equivalent single mass-spring oscillator. The stiffness of this equivalent spring shall be calculated using the maximum displacement that occurs when an arbitrary uniform lateral load is applied to the bridge. (3) The steps in the uniform load method are as follows: 1. Calculate the static displacement $v_s(x)$ due to an assumed uniform load p_o (applied over the length of the bridge set equal to 1.0) 2. Calculate the bridge lateral stiffness, K, and the toal weight W, following the expressions: $K = \frac{p_o L}{v_{sMAX}}$ (11-1) $W = \int w(x)dx$ (11-2) 3. Calculate the period of the bridge T_m, using the expression: $T_m = 2\pi \sqrt{\frac{W}{Kg}}$ (11-3) 4. Calculate the equivalent static earthquake loading p_e from the equation: $p_e = \frac{S_a W}{L}$ (11-4) 5. Calculate the displacement and member forces for use in the design either by applying p_e to the structure and performing a second static analysis or by scaling the results of the first step above by the ratio p_e / p_o where: T_m = period of vibration of m^{th} mode (sec) S_a = the design response spectral acceleration coefficient determined for $T = T_m$ p_e = equivalent uniform static lateral seismic load per unit length of bridge applied to represent the primary mode of vibration (N/mm) W = total weight of structure (N) L = total length of the bridge (mm) $w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure (N/mm) (4) However, the **Elastic Displacement Analysis (EDA)** is used to estimate the displacement demands for structures where ESA does not provide an adequate level of sophistication to estimate the dynamic behavior. A linear elastic multimodal spectral analysis using the appropriate response spectrum (5% damping) shall be performed. The number of degrees of freedom and the number of modes considered in the analysis shall be sufficient to capture at least 90 percent mass participation in both the longitudinal and transverse directions. A minimum of three elements per flexible column and four elements per span shall be

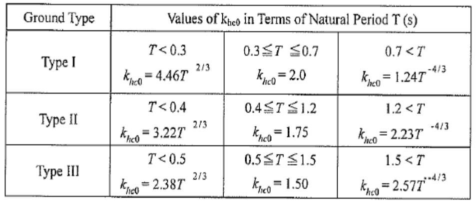
<table>
<tr><th>Items</th><th>JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)</th><th>AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)</th></tr>
<tr><td></td><td>

Table 11-1 Relationship between K_{hCI0} and T (s)



Ground Type	Values of k_{hc0} in Terms of Natural Period T (s)		
Type I	$T \leqq 1.4$ $k_{hc0} = 0.7$		$1.4 < T$ $k_{hc0} = 0.876T^{-2/3}$
Type II	$T < 0.18$ $k_{hc0} = 1.51T^{1/3}$ but $K_{hc0} \geqq 0.7$	$0.18 \leqq T \leqq 1.6$ $k_{hc0} = 0.85$	$1.6 < T$ $k_{hc0} = 1.16T^{-2/3}$
Type III	$T < 0.29$ $k_{hc0} = 1.51T^{1/3}$ but $k_{hc0} \geqq 0.7$	$0.29 \leqq T \leqq 2.0$ $k_{hc0} = 1.0$	$2.0 < T$ $k_{hc0} = 1.59T^{-2/3}$

Table 11-2 Relationship between K_{hCII} and T (s)



Ground Type	Values of k_{hc0} in Terms of Natural Period T (s)		
Type I	$T < 0.3$ $k_{hc0} = 4.46T^{2/3}$	$0.3 \leqq T \leqq 0.7$ $k_{hc0} = 2.0$	$0.7 < T$ $k_{hc0} = 1.24T^{-4/3}$
Type II	$T < 0.4$ $k_{hc0} = 3.22T^{2/3}$	$0.4 \leqq T \leqq 1.2$ $k_{hc0} = 1.75$	$1.2 < T$ $k_{hc0} = 2.23T^{-4/3}$
Type III	$T < 0.5$ $k_{hc0} = 2.38T^{2/3}$	$0.5 \leqq T \leqq 1.5$ $k_{hc0} = 1.50$	$1.5 < T$ $k_{hc0} = 2.57T^{-4/3}$

</td><td>used in the linear elastic model.</td></tr>
<tr><td>12. Force Reduction Factor</td><td>

(1) Force reduction factor shall be calculated by Eq. (12-1) for a structural system that can be modeled as a one degree-of freedom vibration system having a plastic force- displacement relation.

$C_S = 1/\sqrt{2*\mu_a - 1}$ -- (12-1)

C_S = Force reduction factor (refer to Fig. 12-1)

μ_a = Allowable ductility ratio. μa can be obtained by Eq. (12-2) for the case of a RC column.

$\mu_a = 1 + (\delta u - \delta y)/\alpha*\delta y$ (refer to Fig. 12-2) ---(12-2)

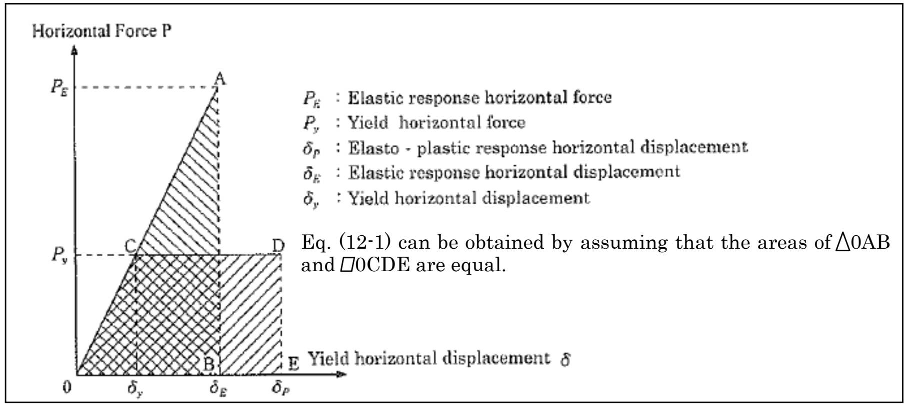
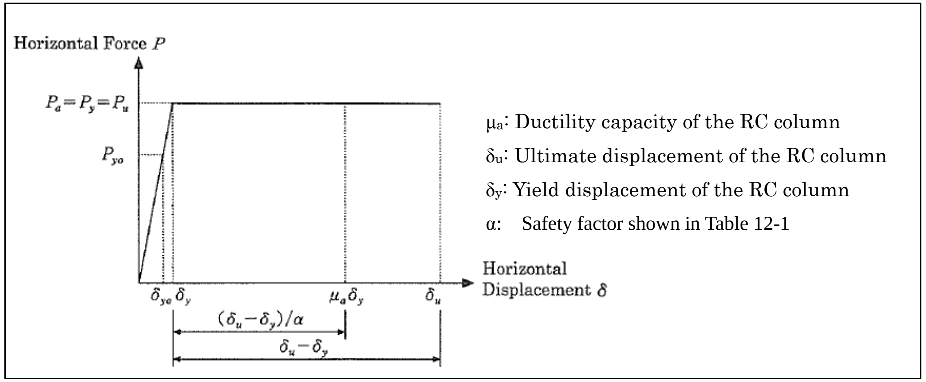


Fig. 12-1 Elasto-Plastic Response Displacement of a Pier



</td><td>

(1) The force reduction factor is not explicitly given in the AASHTO Guide Specifications for LRFD Seismic Bridge Design, unlike the JRA Specifications which gives the force reduction factor based on the member allowable ductility ratio.

(2) The first principles in the AASHTO Guide would be to satisfy the relationship:

$\Delta_D^L \leq \Delta_C^L$ (12-1)

where: Δ_D^L = displacement demand taken along the local principal axis of the ductile member. The displacement demand may be conservatively taken as the bent displacement inclusive of flexibility contribution from the foundations, superstructure, or both.

Δ_C^L = displacement capacity taken along the local principal axis corresponding to of the ductile member as determined in accordance with SDC B, C and D.

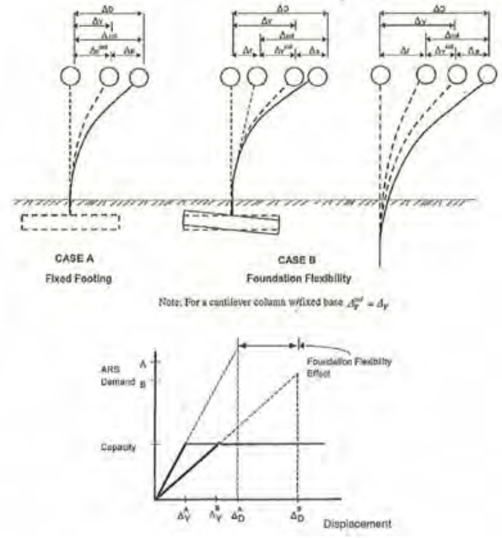


Figure 12-1 Force-Deflection Relation for a Single Column Bent

</td></tr>
</table>

Items

JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)

Fig. 12-2 Simplified Relationship between Lateral Strength and Ductility Capacity for Flexural Failure

Table 12-1 Safety Factor of RC Column resulting in Flexural Failure

Seismic Performance to be Verified	Safety Factor α in Calculation of Ductility Capacity for Type I Earthquake Ground Motion	Safety Factor α in Calculation of Ductility Capacity for Type II Earthquake Ground Motion
Seismic Performance Level 2	3.0	1.5
Seismic Performance Level 3	2.4	1.2

AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)

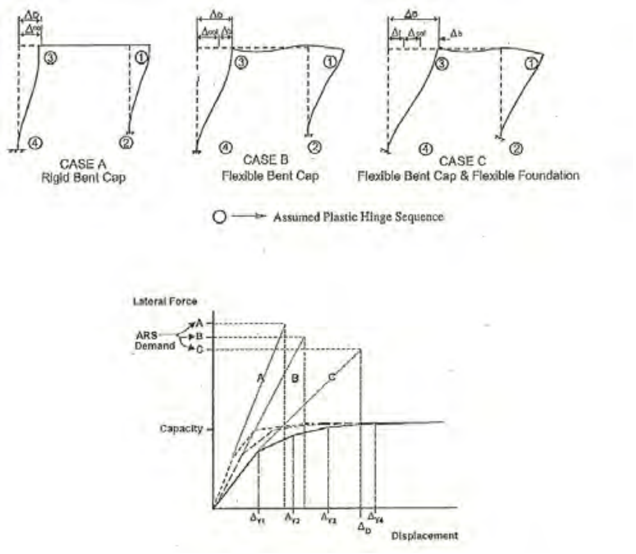


Figure 12-2 Force-Deflection Relation for a Bent Frame

(3) For Type 1 structures, comprising reinforced concrete columns in SDCs B and C, the displacement capacity may be approximated by:

For SDC B: $\Delta_C^L = 0.12H_o\ (-1.27\ln(x)-0.32) \geq 0.12\ H_o$, in. (12-2)

For SDC C: $\Delta_C^L = 0.12H_o\ (-2.32\ln(x)-1.22) \geq 0.12\ H_{o,}$ in. (12-2)

In which: $x = \Lambda B_o/H_o$ (12-3)

where: H_o = clear height of column (ft)
B_o = column diameter or width measured parallel to the direction of displacement under consideration (ft.)
Λ = factor for column end restraint condition
= 1 for fixed-free (pinned on one end)
= 2 for fixed top and bottom

(4) For bridge bent or frames that do not satisfy Eq. (12-1), or are not Type 1 reinforced concrete structures, the designer may either:

- Increase the allowable displacement capacity , by meeting detailing requirements of a higher SDC, or
- Adjust the dynamic characteristics of the bridge.

<table>
<tr><th>Items</th><th>JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)</th><th>AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)</th></tr>
<tr><td></td><td></td><td>(5) Member ductility demand shall satisfy:

Table 12-1 Ductility Demand
<table>
<tr><th>Member</th><th>μ_D</th></tr>
<tr><td>Single column bent</td><td>≤ 5</td></tr>
<tr><td>Multiple column bent</td><td>≤ 6</td></tr>
<tr><td>Pier walls in weak direction</td><td>≤ 5</td></tr>
<tr><td>Pier walls in strong direction</td><td>≤ 1</td></tr>
</table>
In which: $\mu_D = 1 + \Delta_{pd}/\Delta_{yi}$
where: Δ_{pd} = plastic displacement demand (in)
Δ_{yi} = idealized yield displacement corresponding to the idealized yield curvature, φ_{yi}

*Reinforced concrete members such as drilled shafts, cast-in-place piles, and prestressed piles subject to inground hinging shall have ductility limit satisfying $\mu_D \leq 4$.

The member ductility demand may be determined using the M-ϕ analysis as illustrated in Figure 12-3

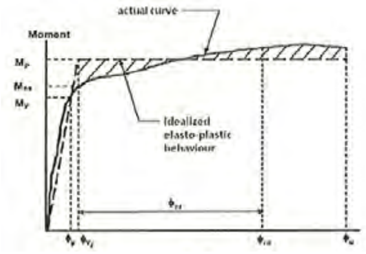
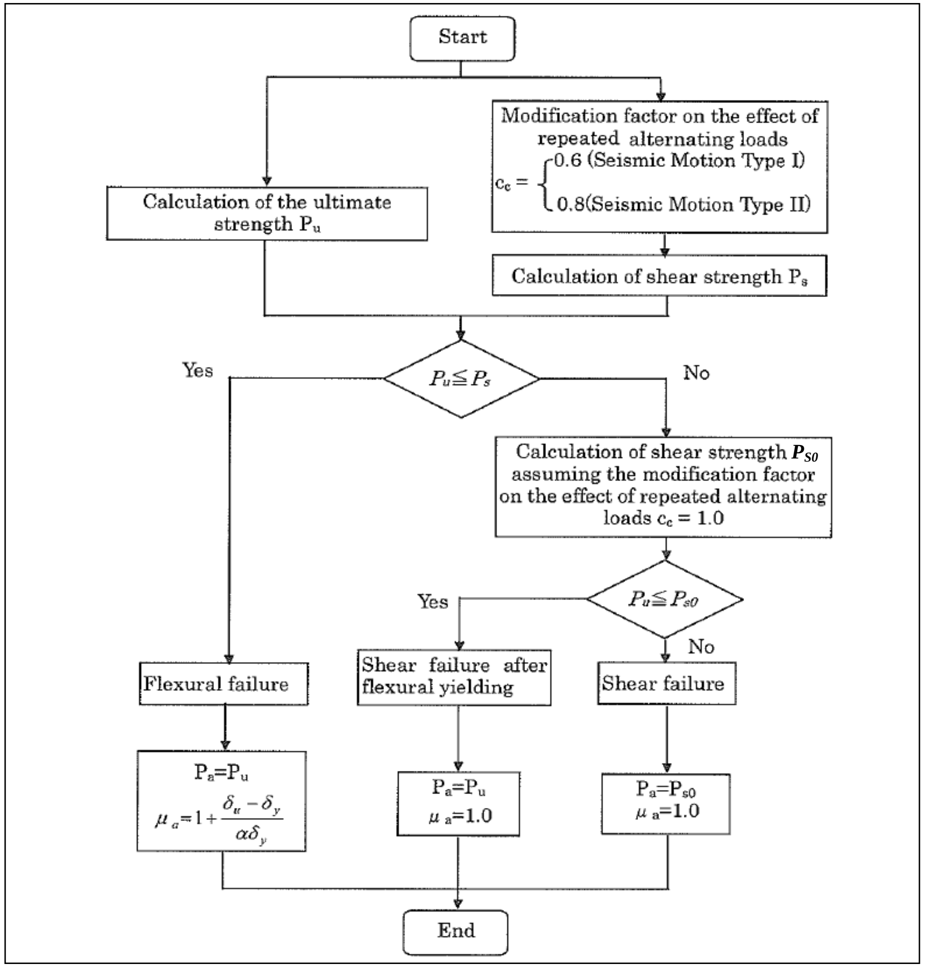


Figure 12-3 Moment-Curvature Diagram</td></tr>
<tr><td>13. Evaluation of Failure Mode of RC Column</td><td>(1) Failure mode of a RC column shall be evaluated by Eq. (13-1)

$$\left.\begin{array}{lll} P_u \leqq P_s & : & \text{Flexural (or bending) failure} \\ P_s < P_u \leqq P_{s0} & : & \text{Shear failure after flexural yielding} \\ P_{s0} < P_u & : & \text{Shear failure} \end{array}\right\} \cdots\cdots\cdots (13\text{-}1)$$

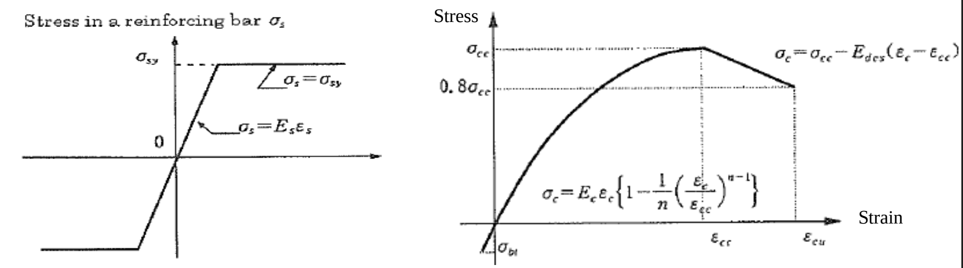
Pu = Lateral strength of a RC column
Ps = Shear strength of a RC column
Ps0 = Shear strength of a RC column calculated by the modification factor on the effects of repeated alternative loads is equal to 1.0.</td><td>(1) There is no specific and explicit AASHTO provision for column failure mode, but for design purposes, each structure shall be categorized according to its intended structural seismic response in terms of damage level (i.e. ductility demand, μ_D as specified in Item 12).

(2) For SDC B, C and D, the following design methods are further defined:
• **Conventional Ductile Response (i.e. Full-Ductility Structures):**
For horizontal loading, a plastic mechanism is intended to develop. The plastic mechanism shall be defined clearly as part of the design strategy. Yielding may occur in areas that are not readily accessible for inspection depending on the Owner’s approval. Inelastic action is intended to be restricted to flexural plastic hinges in columns an pier walls and inelastic soil deformation behind abutment walls and wingwalls. Details and member proportions shall ensure large ductility capacity, μ_C, under load reversals without significant strength loss with ductility demands ($4.0 \leq \mu_D \leq 6.0$). This response is anticipated for a bridge in SDC D designed for the life safety criteria.
• **Limited Ductility Response:**
For horizontal loading, a plastic mechanism as described above for full-ductility structures is intended to develop, but in this case for limited-ductility response, ductility demands are reduced ($\mu_D \leq 4.0$). Intended yielding shall be restricted to locations that are readily accessible for inspection following a design earthquake unless prohibited by structural configuration. Inelastic action is intended to be restricted to flexural plastic hinges in columns and pier walls and inelastic soil deformation behind abutment walls and wingwalls. Detailing and proportioning requirements are</td></tr>
</table>

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)
	(2) Failure mode for a RC column can be judged following the flow shown in Fig. 13-1.  Fig. 13-1 Evaluation of Failure Mode for a RC Column	less than those required for full ductility structures. This response is anticipated for a bridge in SDC B or C. - **Limited-Ductility Response in Concert with Added Protective Systems:** In this case, a structure has limited ductility with the additional seismic isolation, passive energy-dissipating devices and/or other mechanical devices to control seismic response. Using this strategy, a plastic mechanism may or may not form. The occurrence of a plastic mechanism shall be verified by analysis. This response may be used for a bridge in SDC C or D designed for an enhanced performance. Nonlinear time history analysis may be required for this design strategy. (3) The shear demand and capacity design for ductile concrete members is intended to avoid column shear failure by using the principles of "***capacity protection***". For SDCs C and D, the design shear force is specified as a result of the overstrength plastic moment capacity, regardless of the elastic earthquake design forces. This requirement is necessary because of the potential for superstructure collapse if a column fails in shear.
14. Calculation of Lateral Strength and Displacement of a RC Column	(1) Relationships between stress and strain of a reinforcing bar and concrete are shown in Fig. 14-1 (1) and Fig. 14-1 (2), respectively. (2) RC column is divided into m segments along its height and the section of each segment is divided into n elements in the acting direction of the inertia force as shown in Fig. 14-2. With these relationships, Pu, Py, δu and δy at the height of the superstructure inertia force shown in Fig.14-3 can be obtained.	(1) Reinforcing steel is modeled with a stress-strain relationship that exhibits an initial elastic portion, a yield plateau, and a strain-hardening range in which the stress increases with strain as shown in Figure 14-1. On the other hand, the stress-strain model for confined and unconfined concrete is used to determine section response as shown in Figure 14-2.

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)

JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)



(1) Stress and Strain of Reinforcing Bar (2) Stress and Strain of Concrete

Fig 14-1 Relationships between Stress and Strain of Reinforcing Bar and Concrete

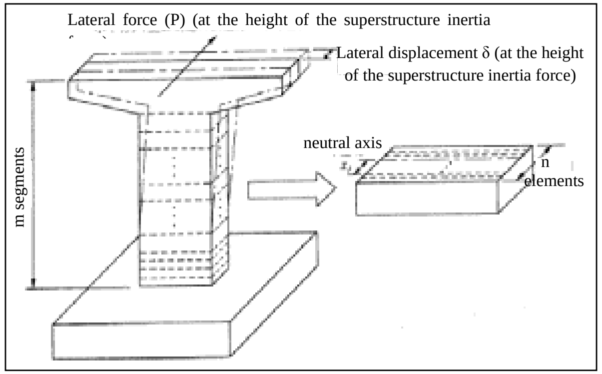


Fig 14-2 Lateral Force (P) at the Acting Position of the Inertia Force and Displacement (δ), and Division of Column

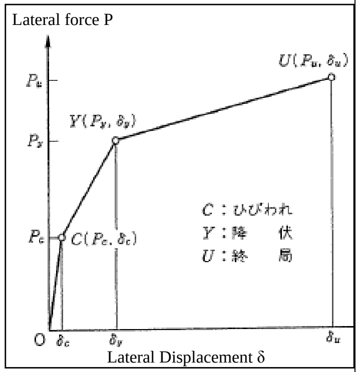


Fig 14-3 Calculated Relationship between Lateral Force (P) and Displacement (δ)

(3) Descriptions

P_C = Lateral strength at cracking, P_y = Yielding lateral strength, P_u = Lateral strength, δ_y = Yield displacement, δu = Ultimate displacement (a single -column RC column)

(a) Premises

- Fiber strain is proportional to the distance from the neutral axis.
- Skeleton curve between horizontal force and horizontal displacement shall be expressed by an ideal elasto-plastic model shown in Fig. 14-4.

(b) Equations

$$P_c = \frac{W}{h}(\sigma_{bt} + \frac{N}{A}) \quad \text{------------------------} (14\text{-}1)$$

$$P_y = \frac{M_u}{h} \quad \text{------------------------} (14\text{-}2)$$

$$\delta_y = \frac{M_u}{M_{y0}}\delta_{y0} \quad \text{------------------------} (14\text{-}3)$$

$$P_u = \frac{M_u}{h} \quad \text{------------------------} (14\text{-}4)$$

$$\delta_u = \delta_y + (\phi_u - \phi_y) L_p (h - L_p/2) \quad \text{--------} (14\text{-}5)$$

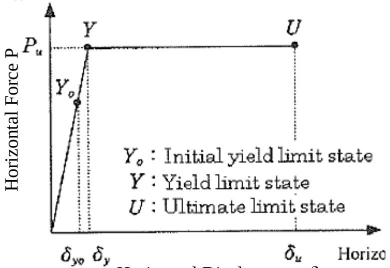


Fig.14-4 Ideal Elasto-Plastic Model

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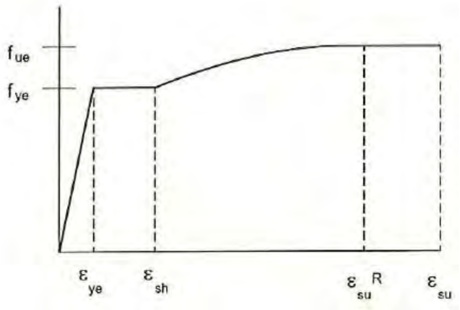


Figure 14-1 Reinforcing Steel Stress-Strain Model

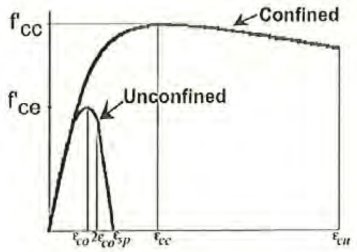


Figure 14-2 Concrete Stress-Strain Model

(2) The plastic moment capacity of all ductile concrete members shall be calculated by moment-curvature (M-ϕ) analysis on the basis of the expected material properties. The moment-curvature analysis shall include axial forces due to dead load together with the axial forces due to overturning.

(3) The M-ϕ curve should be idealized with an elastic perfectly plastic response to estimate the plastic moment capacity of a member's cross-section. The elastic portion of the idealized curve shall pass thru the point marking the first reinforcing bar yield. The idealized plastic moment capacity shall be obtained by equating the areas between the actual and the idealized M-ϕ curves beyond the first reinforcing bar yield point.

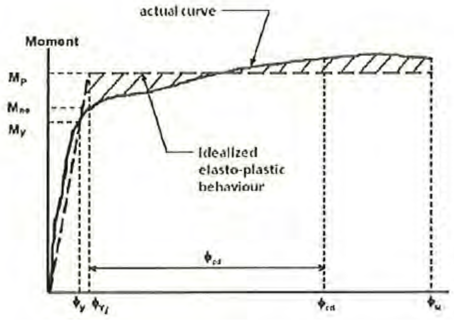


Figure 14-3 Moment-Curvature Model

(4) The member ductility demand may be determined using the M-ϕ which is based on the following assumptions:

- The plastic rotation, θ_p, is concentrated at the center of the plastic hinge,
- The distribution of elastic curvature is linear along the column, and
- The plastic curvature is constant over the equivalent analytical plastic hinge length, L_p.

$\theta_{pd} = (\phi_{pd}) L_p$ (14-1)

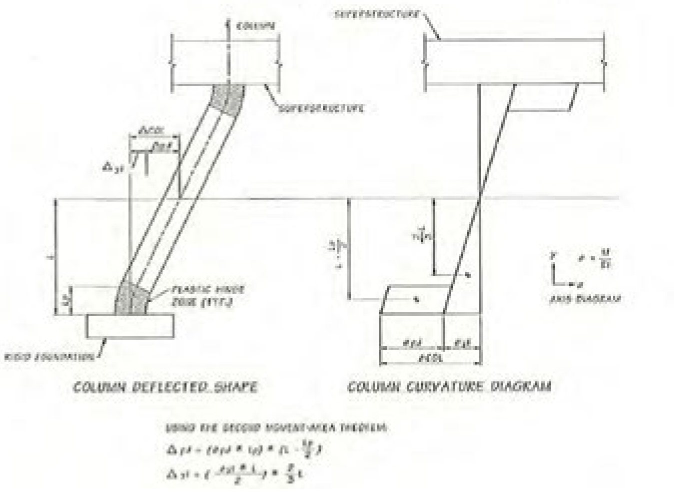
$\phi_{pd} = (\phi_{col} - \phi_{yi})$ (14-2)

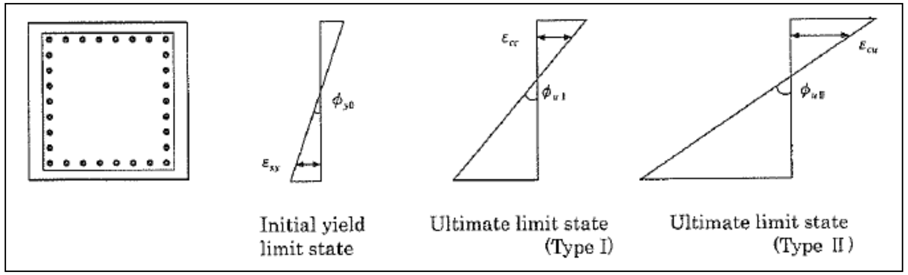
$\Delta_{yi} = (\phi_{yi} L^2)/3$ (14-3)

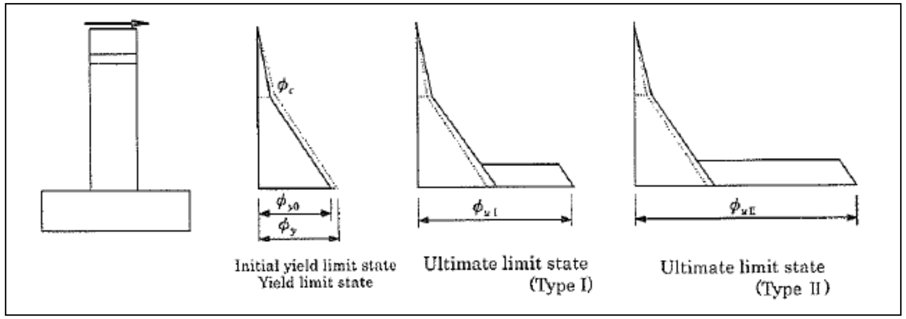
$\Delta_{pd} = \theta_{pd} (L - L_p/2)$ (14-4)

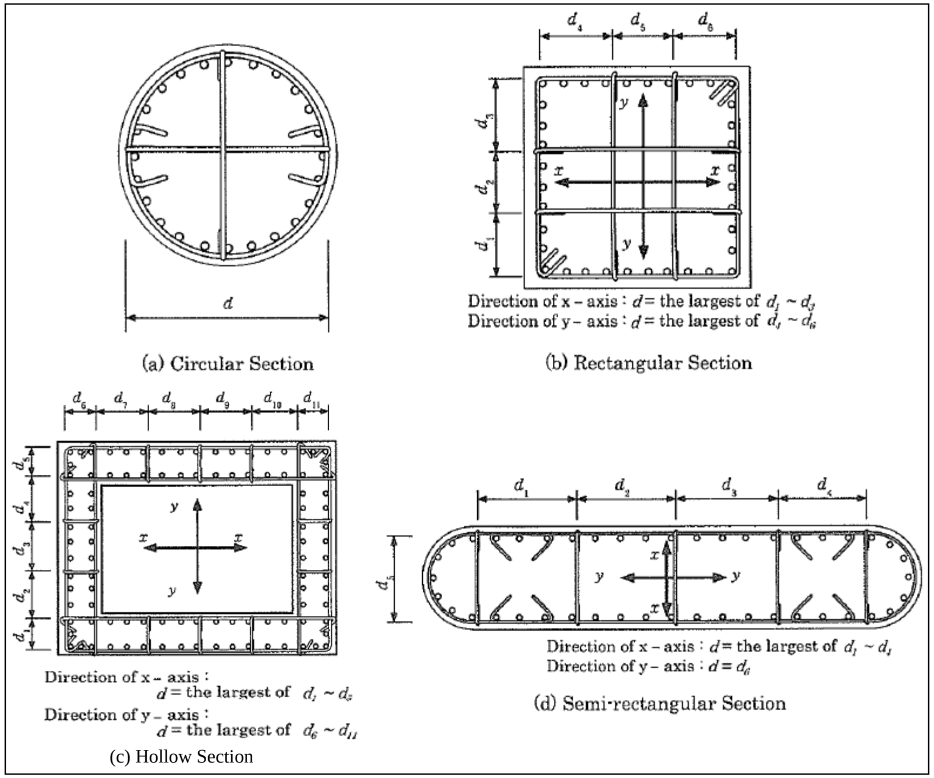
$\mu_D = 1 + \Delta_{pd} / \Delta_{yi}$ (14-5)

$\mu_D = 1 + 3(\phi_{col}/\phi_{yi}-1)(L_p/L)(1-0.5L_p/L)$ (14-6)

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)
	(c) Description of Symbols W : Section modulus of a column with consideration of axial reinforcement at the column's bottom section (mm^3) σ_{bt} : Flexural tensile strength of concrete (N/mm^2) to be calculated by Eq. (14-1-1) $\sigma_{bt} = 0.23\ \sigma_{ck}^{2/3}$ ……… (14-1-1) N : Axial force acting on the column's bottom section (N) A : Sectional area of a column, with consideration of axial reinforcement at the column's bottom section (mm^2) h : Height of superstructural inertial force from the bottom of column. (mm) σ_{ck} : Design strength of concrete (N/mm^2) δ_{y0} : Horizontal displacement at the time of yielding of axial tensile reinforcing bars at the outermost edge of the column's bottom section (called "initial yield displacement" hereafter) (mm) M_u : Ultimate bending moment at the column's bottom section (N· mm) M_{y0}: Bending moment at the time of yielding of axial tensile reinforcing bars at the outermost edge of the column's bottom section (N· mm) H : Height of superstructural inertial force from the bottom of column. (mm) L_p: Plastic hinge length (mm) calculated by Eq. (14-5-1) $L_p = 0.2h - 0.1\,D$ ……… (14-5-1) in which $0.1D \leq L_p \leq 0.5D$ D : Sectional depth (mm) (D shall be the diameter of a circular section, or the length of a rectangular section in the analytical direction) ϕ_y: Yield curvature at the column's bottom section (1/mm) ϕ_u : Ultimate curvature at the column's bottom section (1/mm) (d) Detailed Procedure to Obtain Mechanical Values Referring to Fig. 14-2, Fig. 14-5 and Fig. 14-6 $M_c = W_i(\sigma_{bt} + N_i/A_i)$ ……… (14-6) $\phi_c = M_c / E_c I_i$ ……… (14-7) M_c: Bending moment at cracking (N·mm) ϕ_c: Curvature of cracking (1/mm) W_i: Sectional modulus of pier having considered the axial reinforcement in the i-th section from the height of the superstructural inertia force (mm^3). σ_{bt}: Bending tensile strength of concrete (N/mm^2) to be calculated by Eq. (14-1-1) Ni: Axial force due to the weights of superstructure and substructure acting on the i-th section from the height of the superstructural inertia force (N). A_i: Sectional area of pier having considered the axial reinforcement in the i-th section from the height of the superstructural inertia force (mm^2)	where: ϕ_{col} = column curvature at maximum displacement demand (computed from push over analysis) (1/in) ϕ_{yi} = idealized yield curvature (1/in) ϕ_{pd} = column plastic curvature demand (1/in) L_p = analytical plastic hinge length (in) L = length of column from point of maximum moment to the point of moment contraflexure (in)  Figure 14-4 Pier Deflected Shape and Curvature Diagram (5) The expected nominal moment capacity, M_{ne}, shall be based on the expected concrete and reinforcing steel strengths when either the concrete strain reaches a magnitude of 0.003 or the reinforcing steel reaches the reduced ultimate tensile strain. For SDC B, the expected nominal moment capacity, M_{ne}, may be used as M_p in lieu of the development of a moment-curvature analysis. (6) Requirements for Ductile Member • Minimum Lateral Strength The minimum lateral flexural capacity of each column is taken as: $M_{ne} \geq 0.1 P_{trib}\,[(H_h + 0.5D_s)/\Lambda]$ where: M_{ne} = nominal moment capacity of the column based on expected material properties (kip-ft) P_{trib} = greater of the dead load per column or force associated with the tributary seismic mass at the bent (kips)

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)
	E_c: Young's modulus of concrete (N/mm^2) I_i: Moment of inertia of pier having considered the axial reinforcement in the i-th section from height of the superstructural inertia force (mm^4) σ_{ck} : Design standard strength of concrete (N/mm^2) $M_i=\sum_{j-1}^{n}\sigma_{cj}x_j\Delta A_{cj}+\sum_{j=1}^{n}\sigma_{sj}x_j\Delta A_{sj}$ ---------- (14-8) $\phi_i=\varepsilon_{c0}/x_0$ ---------- (14-9) σ_{cj}, σ_{sj}: Stress of concrete and reinforcement within the j-th infinitesimal part (N/mm^2) ΔA_{cj}, ΔA_{sj}: Sectional area of concrete and reinforcement within the j-th infinitesimal part (mm^2) M_i: Bending moment acting on the i-th section from the height of the superstructural inertia force (N·mm) ϕ_i: Curvature of the i-th section from the height of the superstructural inertia force (1/mm) x_j: Distance from concrete or reinforcement in the j-th infinitesimal part to the centroid position of section (mm) ε_{c0}: Compressed edge strain of concrete x_0: Distance from the compressed edge of concrete to the neutral axis (mm) $\delta_{y0}=\int\phi y\,dy \fallingdotseq \sum_{i=1}^{m}(\phi_i y_i+\phi_{i-1}y_{i-1})\Delta y_i/2$ ---------- (14-10) $\phi_y=\left(\frac{M_u}{M_{y0}}\right)\phi_{y0}$ ---------- (14-11)  **Fig 14-5** Strain Distribution within Initial Yielding and Ultimate Limit	H_h = the height from the top of the footing to the top of the column or the equivalent column height for a pile extension column (ft) D_s = depth of superstructure (ft) Λ = fixity factor for the column

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)
	 **Fig 14-6** Curvature Distribution in the Direction of Height (e) Stress-Strain Relation of Concrete (refer to Fig. 14-1(2)) The stress-strain curve of concrete shall be determined by Eq. (14-12) based on Fig.14-1(2)) $\sigma_c = \begin{cases} E_c \varepsilon_c \left\{ 1 - \frac{1}{n} \left(\frac{\varepsilon_c}{\varepsilon_{cc}} \right)^{n-1} \right\} & (0 \leqq \varepsilon_c \leqq \varepsilon_{cc}) \\ \sigma_{cc} - E_{des} (\varepsilon_c - \varepsilon_{cc}) & (\varepsilon_{cc} < \varepsilon_c \leqq \varepsilon_{cu}) \end{cases}$ .... (14-12) $n = \frac{E_c \varepsilon_{cc}}{E_c \varepsilon_{cc} - \sigma_{cc}}$ .... (14-13) $\sigma_{cc} = \sigma_{ck} + 3.8 \alpha \rho_s \sigma_{sy}$ .... (14-14) $\varepsilon_{cc} = 0.002 + 0.033 \beta \frac{\rho_s \sigma_{sy}}{\sigma_{ck}}$ .... (14-15) $E_{des} = 11.2 \frac{\sigma_{ck}{}^2}{\rho_s \sigma_{sy}}$ .... (14-16) $\varepsilon_{cu} = \begin{cases} \varepsilon_{cc} & \text{(For Type I Earthguake Ground Motion)} \\ \varepsilon_{cc} + \frac{0.2\sigma_{cc}}{E_{des}} & \text{(For Type II Earthguake Ground Motion)} \end{cases}$ .... (14-17) $\rho_s = \frac{4A_h}{sd} \leqq 0.018$ .... (14-18) σ_c : Stress of concrete (N/mm^4) σ_{cc} : Strength of concrete restrained by lateral confining reinforcement (N/mm^2) σ_{ck} : Design strength of concrete (N/mm^2) ε_c : Strain of concrete ε_{cc} : Strain of concrete under the maximum compressive stress ε_{cu} : Ultimate strain of concrete restrained by lateral confining reinforcement	

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)
	E_c : Young's modulus of concrete (N/mm^2) E_{des} : Descending gradient (N/mm^2) ρ_s : Volume ratio of lateral confining reinforcement A_h : Sectional area of each lateral confining reinforcement (mm^2) s : Spacings of lateral confining reinforcement (mm) d : Effective length (mm) of lateral confining reinforcement. It shall be the longest length of core concrete divided and restrained by lateral hoop ties or cross ties (refer to Fig. 14-7) σ_{sy}: Yield point of lateral confining reinforcement (N/mm^2) α,β: Modification factor on section. α=1.0,β=1.0 for a circular section, and α= 0.2, β= 0.4 for rectangular, hollow circular and hollow rectangular sections. n : A constant defined by Eq. (14-13)  (a) Circular Section Direction of x – axis : d = the largest of $d_1 \sim d_3$ Direction of y – axis : d = the largest of $d_4 \sim d_6$ (b) Rectangular Section Direction of x – axis : d = the largest of $d_1 \sim d_5$ Direction of y – axis : d = the largest of $d_6 \sim d_{11}$ (c) Hollow Section Direction of x – axis : d = the largest of $d_1 \sim d_4$ Direction of y – axis : $d = d_6$ (d) Semi-rectangular Section **Fig. 14-7** Effective Length of Lateral Confining Reinforcement (in Both Longitudinal and Transverse Direction to the Bridge Axis)	

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)
15. Shear Strength (Concrete Structure)		

Shear strength shall be calculated by Eq. (15-1)

$$P_s = S_c + S_s \quad \text{(15-1)}$$

$$S_c = c_c c_e c_{pt} \tau_c bd \quad \text{(15-2)}$$

$$S_s = \frac{A_w \sigma_{sy} d(\sin\theta + \cos\theta)}{1.15a} \quad \text{(15-3)}$$

P_s : Shear strength (N)

S_c : Shear strength resisted by concrete (N)

τ_c : Average shear stress that can be borne by concrete (N/mm^2). Values in Table 15-1 shall be used.

c_c : Modification factor on the effects of alternating cyclic loading. c_c shall be taken as 0.6 for Type I Earthquake Ground Motion and 0.8 for Type II.

c_e : Modification factor in relation to the effective height (d) of a pier section. Values in Table 15-2 shall be used.

(b) Effective Height of a rectangular Section

c_{pt} : Modification factor in relation to the axial tensile reinforcement ratio p_t. Values in Table 15-3 shall be used.

b : Width of a pier section perpendicular to the direction in calculating shear strength (mm)

d : Effective height of a pier section parallel to the direction in calculating shear strength (mm)

p_t : Axial tensile reinforcement ratio. It is the value obtained by dividing the total sectional areas of the main reinforcement on the tension side of the neutral axis by bd (%).

S_s: Shear strength borne by hoop ties (N)

A_w: Sectional area of hoop ties arranged with an interval of α and an angle of θ (mm^2)

σ_{sy}: Yield point of hoop ties (N/mm^2)

θ : Angle formed between hoop ties and the vertical axis (degree)

α : Spacings of hoop ties (mm)

Table 15-1 Average Shear Stress of Concrete τc (N/mm^2)

Design Compressive Strength of Concreteσ_{ck} (N/mm^2)	21	24	27	30	40
Average Shear Stress of Concreteτ_c (N/mm^2)	0.33	0.35	0.36	0.37	0.41

Table 15-2 Modification Factor Ce in Relation to Effective Height of a Pier Section

Effective Height (mm)	Below 1000	3000	5000	Above 10000
c_e	1.0	0.7	0.6	0.5

(1) The shear strength capacity within the plastic hinge region is calculated on the basis of nominal material strength properties and satisfies:

$$\phi_s V_n \geq V_u \quad \text{(15-1)}$$

in which:

$$V_n = V_c + V_s \quad \text{(15-2)}$$

where:

ϕ_s = 0.90 for shear in reinforced concrete

V_n = nominal shear capacity of member (kips)

V_c = concrete contribution to shear capacity (kips)

V_s = reinforcing steel contribution to shear capacity (kips)

V_u = shear demand of a column, wall or pile shaft

(a) Concrete Shear Capacity

$$V_c = v_c A_e \quad \text{(15-3)}$$

$$A_e = 0.8A_g \quad \text{(15-4)}$$

If P_u is compressive,

$$v_c = 0.032\alpha'\left(1 + \frac{P_u}{2A_g}\right)\sqrt{f_c'} \leq \min\begin{cases} 0.11\sqrt{f_c'} \\ 0.047\alpha'\sqrt{f_c'} \end{cases} \quad \text{(15-5)}$$

otherwise:

$$v_c = 0 \quad \text{(15-6)}$$

for circular columns with spiral hoop or hoop reinforcing:

$$\alpha' = \frac{f_s}{0.15} + 3.67 - \mu_D \quad \text{(15-7)}$$

$$f_s = \rho_s f_{yh} \leq 0.35 \quad \text{(15-8)}$$

$$\rho_s = \frac{4A_{sp}}{sD'} \quad \text{(15-9)}$$

for rectangular columns with ties:

$$\alpha' = \frac{f_w}{0.15} + 3.67 - \mu_D \quad \text{(15-10)}$$

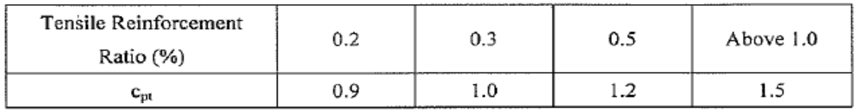
$$f_w = 2\rho_w f_{yh} \leq 0.35 \quad \text{(15-11)}$$

$$\rho_w = \frac{A_v}{bs} \quad \text{(15-12)}$$

The concrete shear stress adjustment factor, α′, shall not be taken as greater than 3 and need not be taken as less than 0.30.

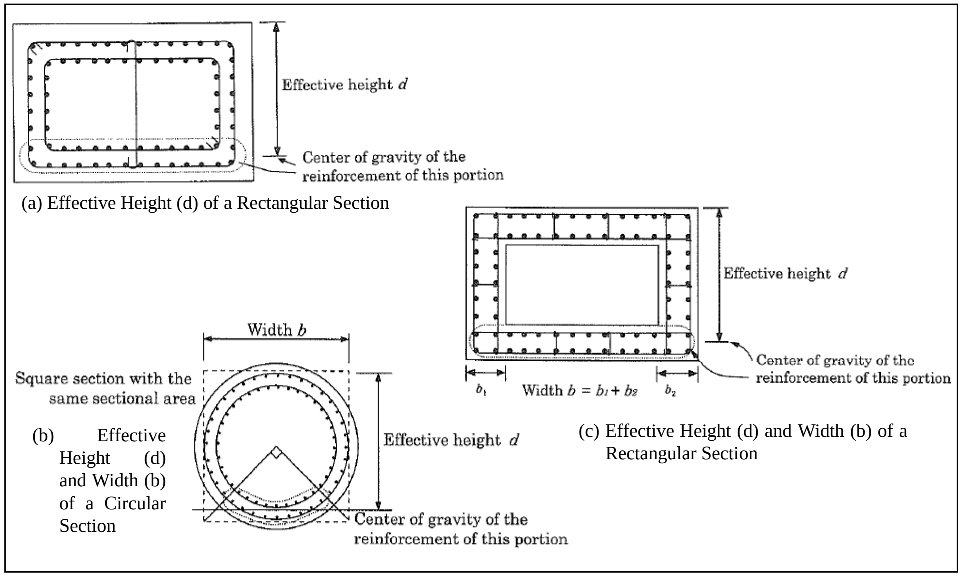
Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)

Table 15-3 Modification Factor Cpt in Relation to Axial tensile Reinforcement Ratio Pt



Tensile Reinforcement Ratio (%)	0.2	0.3	0.5	Above 1.0
c_{pt}	0.9	1.0	1.2	1.5

Evaluation method of effective height (d) for each column section shape is shown in Fig. 15-1.



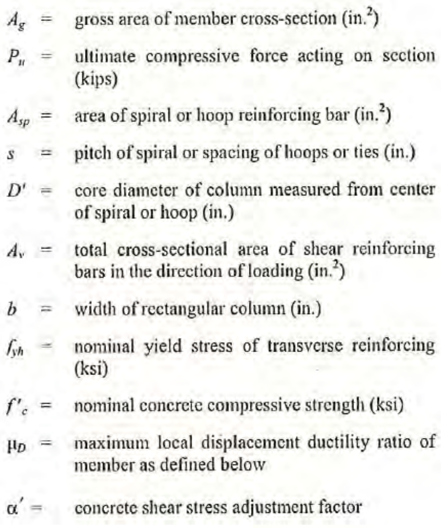
(a) Effective Height (d) of a Rectangular Section

(b) Effective Height (d) and Width (b) of a Circular Section

(c) Effective Height (d) and Width (b) of a Rectangular Section

Fig. 15-1 Effective Height (d) and Width (b) of Each Section Shape

where:



A_g = gross area of member cross-section (in.^{2})

P_u = ultimate compressive force acting on section (kips)

A_{sp} = area of spiral or hoop reinforcing bar (in.^{2})

s = pitch of spiral or spacing of hoops or ties (in.)

D' = core diameter of column measured from center of spiral or hoop (in.)

A_v = total cross-sectional area of shear reinforcing bars in the direction of loading (in.^{2})

b = width of rectangular column (in.)

f_{yh} = nominal yield stress of transverse reinforcing (ksi)

f'_c = nominal concrete compressive strength (ksi)

μ_D = maximum local displacement ductility ratio of member as defined below

α' = concrete shear stress adjustment factor

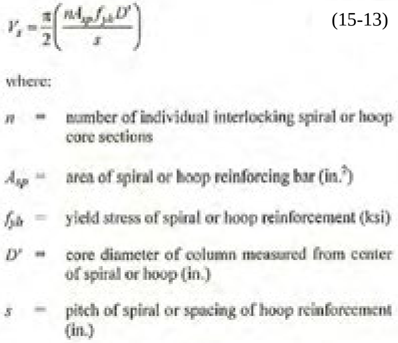
The concrete shear capacity, V_c, at plastic hinge section shall be determined using member ductility demand, μ_D as follows:

SDC	μ_D
B	2
C	3
D	$\mu_D = 1 + \Delta_{pd}/\Delta_{yi}$

(b) Shear Reinforcement Capacity

The nominal shear reinforcement strength, V_s, is taken as:

For members with circular hoops, spirals, or interlocking hoops or spirals,



$$V_s = \frac{\pi}{2}\left(\frac{nA_{sp}f_{yh}D'}{s}\right) \quad (15\text{-}13)$$

where:

n = number of individual interlocking spiral or hoop core sections

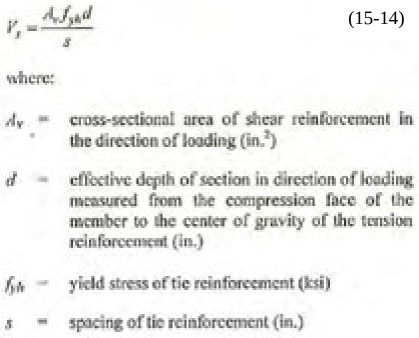
A_{sp} = area of spiral or hoop reinforcing bar (in.^{2})

f_{yh} = yield stress of spiral or hoop reinforcement (ksi)

D' = core diameter of column measured from center of spiral or hoop (in.)

s = pitch of spiral or spacing of hoop reinforcement (in.)

For members with rectangular ties or stirrups,



$$V_s = \frac{A_v f_{yh} d}{s} \quad (15\text{-}14)$$

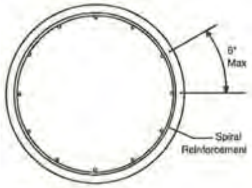
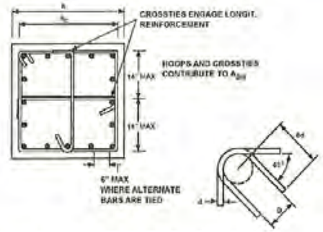
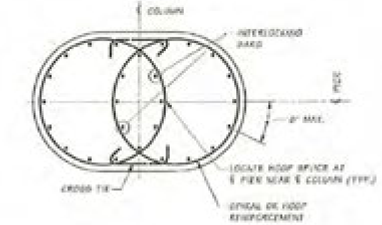
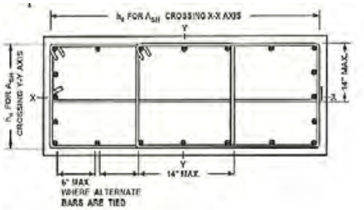
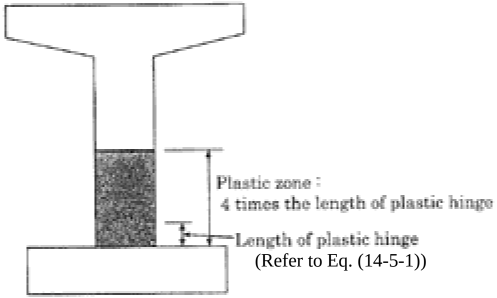
where:

A_v = cross-sectional area of shear reinforcement in the direction of loading (in.^{2})

d = effective depth of section in direction of loading measured from the compression face of the member to the center of gravity of the tension reinforcement (in.)

f_{yh} = yield stress of tie reinforcement (ksi)

s = spacing of tie reinforcement (in.)

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)
		 Figure 15-1 Single Spiral  Figure 15-2 Column Tie Details  Figure 15-3 Column Interlocking Spiral and Hoop Details  Figure 15-4 Column Tie Details
16. Structural Details for Improving Ductility Performance	(1) Incase that generation of plastic deformation of the column is expected, lapping of axial reinforcements shall not generally be placed within the plastic zone (refer to Fig 16-1). (2) Arrangement of Hoop Ties (a) To use deformed bars of at least 13mm in diameter, and the intervals shall not generally be greater than 150mm in the plastic zone. (b) To be arranged so as to enclose the axial reinforcement and be fixed in to the concrete inside a column with the length below. i) Semi-circular hook = 8Φ or 120mm whichever is the greater. ii) Acute angle hook = 10Φ iii) Rectangular angle hook = 12Φ (Φ: the diameter of the hoop tie)  (Refer to Eq. (14-5-1)) **Fig. 16-1** Plastic Zone (c) Lapping of hoop ties shall be staggered along the column height. (d) To have a lap length of at least 40Φ in case that hoop ties are lapped at any place other than the corners of a rectangular section (refer to Fig 16-2).	(1) Splices (a) Splicing of longitudinal column reinforcement in SDC C or D shall be outside the plastic hinging region. (b) For pile or shaft where liquefaction is anticipated and where splicing in the potential plastic hinge zone cannot be avoided, mechanical couplers that are capable of developing the expected tensile strength of bars shall be specified. (c) Lapping longitudinal reinforcement with dowels at the column base is not allowed. (2) Lateral Reinforcement (a) Lateral Reinforcement Inside Plastic Hinge Region for SDCs C and D - the volume of lateral reinforcement, ρ_s (spiral or circular) or ρ_w (web), provided inside plastic hinge region shall be sufficient to ensure that the column or pier wall has adequate shear capacity and confinement level to achieve the required ductility capacity, - for columns designed to achieve a displacement ductility demand greater than 4, the lateral reinforcement shall be either butt-welded hoops or spirals, - combination of hoops and spirals are not permitted in footing or bent cap, - at spiral or hoop-to-spiral discontinuities, the spiral shall determine with one extra turn plus a tail equal to the cage diameter. (b) Lateral Reinforcement for SDCs B, C, and D - all longitudinal bars in compression members shall be enclosed by lateral reinforcement, - transverse hoop reinforcement may be provided by single or overlapping hoops, - each end of cross-tie shall engage a peripheral longitudinal bar with cross-ties having seismic hook; seismic hook shall consist of 135° bend, plus an extension of not less than the larger of 6 bar dia or 75mm.

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)

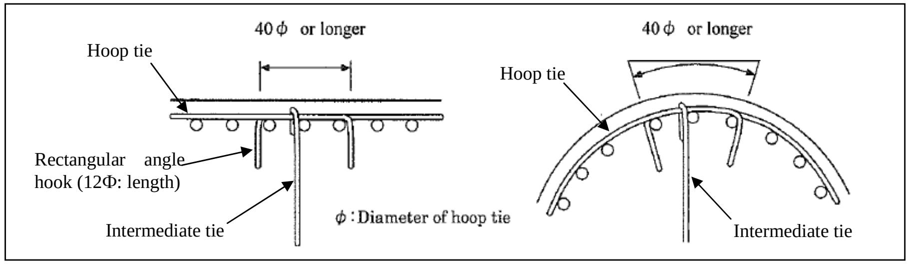


Fig. 16-2 Anchorage of Hoop Ties with Rectangular Angle Hook

(3) Arrangement of Intermediate Ties

(a) To be of the same material and the same diameter as the hoop ties.

(b) To be arranged in both the directions of the long side and the short side of a column section.

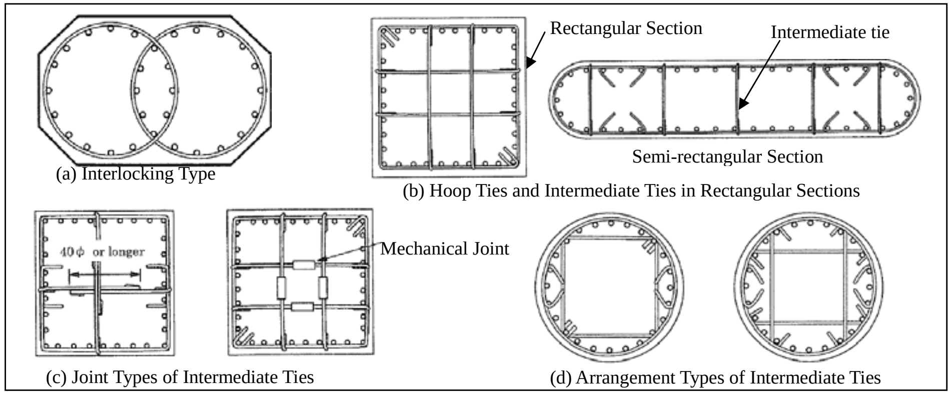
(c) Intervals within a column section shall not be greater than one meter.

(d) To be arranged in all sections with hoop ties arranged.

(e) To be hooked up to the hoop ties arranged in the perimeter directions of the section.

(f) To be fixed into the concrete inside a column (refer to Fig. 16-2 and 16-3).

(g) To go through a column section, with use of a continuous reinforcing bar or a pair of reinforcing bars with a joint within the column section.



(a) Interlocking Type

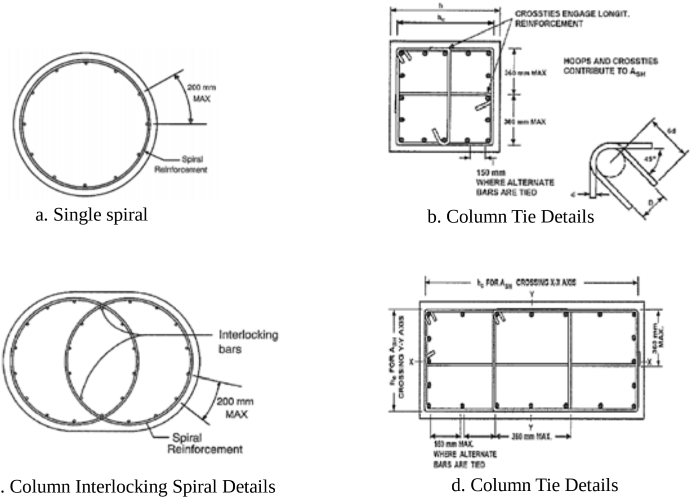
(b) Hoop Ties and Intermediate Ties in Rectangular Sections

(c) Joint Types of Intermediate Ties

(d) Arrangement Types of Intermediate Ties

Fig. 16-3 Arrangement of Hoop Ties and Intermediate Ties According to Column Types

- for members reinforced with single circular hoops, the hoop weld splices shall be staggered around the column a minimum distance of one-third of the hoop circumference,
- for members reinforced with interlocking hoops, the hoop weld splices shall be placed within the interlocking area of the column section,
- the maximum spacing for lateral reinforcement in the plastic hinge regions shall not exceed the smallest of:
  - o one-fifth of the least dimension of the cross-section for columns and one-half of the least cross-section dimension of piers,
  - o six times the nominal diameter of the longitudinal reinforcement,
  - o 150mm (6 in.) for single hoop or spiral reinforcement,
  - o 200mm (8 in.) for bundled hoop reinforcement.



a. Single spiral

b. Column Tie Details

c. Column Interlocking Spiral Details

d. Column Tie Details

Figure 16-1 Details of spirals, hoops, ties and cross-ties

(3) Joint Design for SDCs C and D

(a) Joint Performance

Moment-resisting connections shall be designed to transmit the maximum force produced when the column has reached its overstrength capacity, M_{po}.

(b) Joint Proportioning

Moment-resisting joints shall be proportioned so that the principal stresses satisfy the requirements of:

- for principal compression, p_c:

$$p_c \leq 0.25f'_c \qquad (16\text{-}1)$$

- for principal tension, p_t:

$$p_t \leq 0.38\sqrt{f'_c} \qquad (16\text{-}2)$$

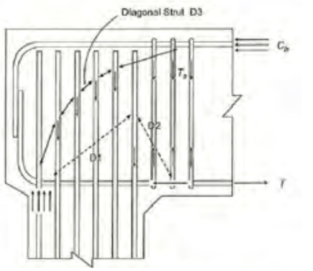
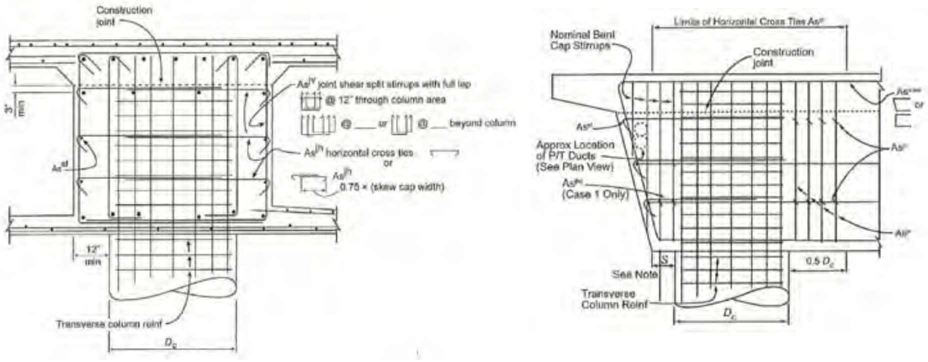
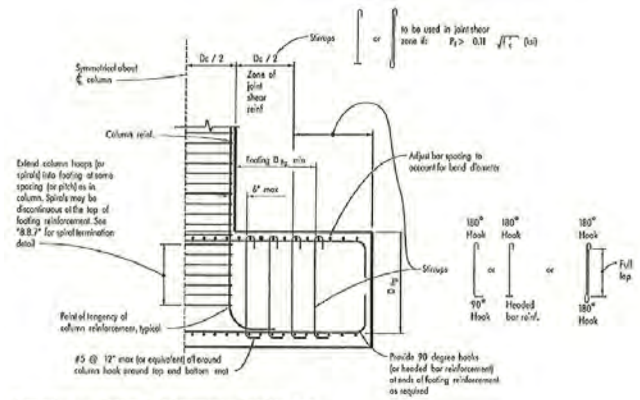


Figure 16-2 External Vertical Joint Reinforcement for Joint Force Transfer

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)
		 (a) T Joint Shear Reinforcement Details (b) Knee Joint Shear Reinforcement Details Figure 16-3 Typical Example of Integral Bent Cap Joint Details  Figure 16-4 Footing Joint Shear Reinforcement – Fixed Column
17. Bearing Support System	(1) Bearing support shall be fundamentally designed for horizontal and vertical forces due to Level 1 and Level 2 EGMs (referred as "Type B bearing support"). (2) However, in the case of superstructures which do not mainly vibrate due to the restraints of abutments, or in the case that Type B bearing supports cannot be adopted, a bearing support system together with structures limiting excessive displacement may be designed in the following manner: Functions of the bearing support system shall be ensured for horizontal and vertical forces due to Level 1 Earthquake Ground Motion, and the bearing support system and the structures limiting excessive displacement shall jointly resist the horizontal forces due to Level 2 Earthquake Ground Motion (referred as "Type A bearing supports" hereafter). (3) Fig. 17-1 shows the selection flow of bearing support.	(1) There is no specific Chapter on Bearings under this Guide Specifications except for the Article 7.8-Isolation Devices which just refer to the provisions of "*AASHTO Guide Specifications for Seismic Isolation Design*" and Article 7.9 - Fixed and Expansion Bearing. (2) Bearings design shall be consistent with the intended seismic (or other extreme event) response of the whole bridge system. Where rigid-type bearings are used, the seismic (or other horizontal extreme event) forces from the superstructure is assumed to be transmitted through diaphragms and cross-frames and their connections to the bearings and then to the substructure without reduction due to local inelastic action along the load path. (3) Based on the horizontal stiffness, bearings are divided into four categories: • Rigid bearings that transmit seismic loads without any movement or deformations, • Deformable bearings that transmit seismic loads limited by plastic deformations or restricted slippage of bearing components, • Seismic isolation type bearings that transmit reduced seismic loads, limited by energy dissipator, • Structural fuses that are designed to fail at a prescribed load.

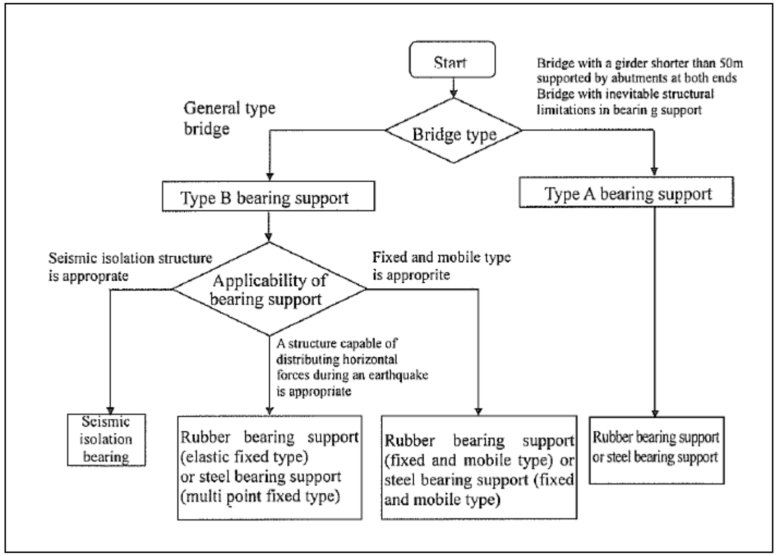
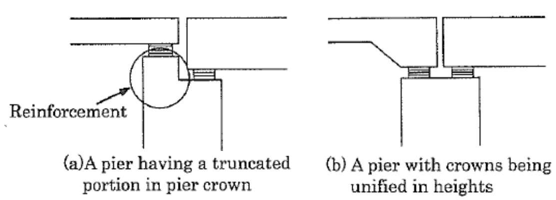
<table>
<tr><th>Items</th><th>JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)</th><th>AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)</th></tr>
<tr><td></td><td>


Fig. 17-1 General Consideration in Selection of Bearing Support



(a)A pier having a truncated portion in pier crown

(b) A pier with crowns being unified in heights

Fig. 17-2 Measures for Dealing with Truncated Portion of a Pier Crown

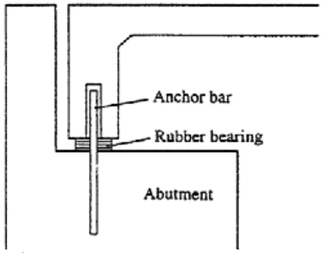
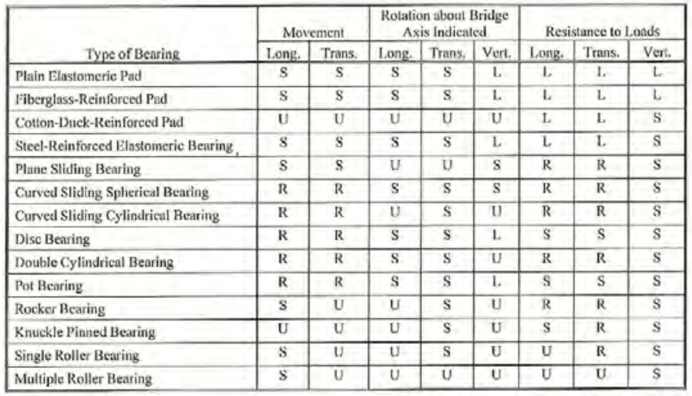


Fig. 17-3 Structure Limiting Excessive Displacement Connecting Superstructure and Substructure
</td><td>
(4) Roller bearings or rocker bearings shall not be used in new bridge construction.
(5) Expansion bearings and their supports shall be designed in such a manner that the structure can undergo movements in the unrestrained direction not less than the seismic displacements determined from analysis without collapse.
(6) The frictional resistance of the bearing interface sliding surfaces shall be neglected when it contributes to resisting seismic loads. Conversely, the frictional resistance shall be conservatively calculated (i.e overestimated) when the friction resistance results I the application of greater force effects to the structural components.
(7) Elastomeric expansion bearings shall be provided with anchorage to adequately resist the seismically induced horizontal forces in excess of those accommodated by shear in the pad. Elastomeric fixed bearings, on the other hand, shall be provided with horizontal restraint adequate for the full horizontal load.
(8) Pot and disc bearings should not be used for seismic applications where significant vertical acceleration is present. Where the use of pot and disc bearings is unavoidable, they shall be provided with an independent seismically resistant anchorage system.
(9) Sufficient reinforcement shall be provided around anchor bolts to develop the horizontal forces and anchor them into the mass of the substructure unit.
(10) The selection of bearings should also relate to the strength and stiffness characteristics of both the superstructure and the substructure. Following the "*AASHTO LRFD Bridge Design Specifications*", the bearing chosen for a particular application shall have appropriate load and movement capabilities.

The following table illustrates bearing suitability:

Table 17-1 Bearing Suitability


<table>
<tr><th rowspan="2">Type of Bearing</th><th colspan="2">Movement</th><th colspan="3">Rotation about Bridge Axis Indicated</th><th colspan="3">Resistance to Loads</th></tr>
<tr><th>Long.</th><th>Trans.</th><th>Long.</th><th>Trans.</th><th>Vert.</th><th>Long.</th><th>Trans.</th><th>Vert.</th></tr>
<tr><td>Plain Elastomeric Pad</td><td>S</td><td>S</td><td>S</td><td>S</td><td>L</td><td>L</td><td>L</td><td>L</td></tr>
<tr><td>Fiberglass-Reinforced Pad</td><td>S</td><td>S</td><td>S</td><td>S</td><td>L</td><td>L</td><td>L</td><td>L</td></tr>
<tr><td>Cotton-Duck-Reinforced Pad</td><td>U</td><td>U</td><td>U</td><td>U</td><td>U</td><td>L</td><td>L</td><td>S</td></tr>
<tr><td>Steel-Reinforced Elastomeric Bearing</td><td>S</td><td>S</td><td>S</td><td>S</td><td>L</td><td>L</td><td>L</td><td>S</td></tr>
<tr><td>Plane Sliding Bearing</td><td>S</td><td>S</td><td>U</td><td>U</td><td>S</td><td>R</td><td>R</td><td>S</td></tr>
<tr><td>Curved Sliding Spherical Bearing</td><td>R</td><td>R</td><td>S</td><td>S</td><td>S</td><td>R</td><td>R</td><td>S</td></tr>
<tr><td>Curved Sliding Cylindrical Bearing</td><td>R</td><td>R</td><td>U</td><td>S</td><td>U</td><td>R</td><td>R</td><td>S</td></tr>
<tr><td>Disc Bearing</td><td>R</td><td>R</td><td>S</td><td>S</td><td>L</td><td>S</td><td>S</td><td>S</td></tr>
<tr><td>Double Cylindrical Bearing</td><td>R</td><td>R</td><td>S</td><td>S</td><td>U</td><td>R</td><td>R</td><td>S</td></tr>
<tr><td>Pot Bearing</td><td>R</td><td>R</td><td>S</td><td>S</td><td>L</td><td>S</td><td>S</td><td>S</td></tr>
<tr><td>Rocker Bearing</td><td>S</td><td>U</td><td>U</td><td>S</td><td>U</td><td>R</td><td>R</td><td>S</td></tr>
<tr><td>Knuckle Pinned Bearing</td><td>U</td><td>U</td><td>U</td><td>S</td><td>U</td><td>S</td><td>R</td><td>S</td></tr>
<tr><td>Single Roller Bearing</td><td>S</td><td>U</td><td>U</td><td>S</td><td>U</td><td>U</td><td>R</td><td>S</td></tr>
<tr><td>Multiple Roller Bearing</td><td>S</td><td>U</td><td>U</td><td>U</td><td>U</td><td>U</td><td>U</td><td>S</td></tr>
</table>

S = Suitable *U = Unsuitable* *L = Suitable for limited applications*
R = May be suitable but requires special considerations or additional elements such as sliders or guideways
Long. = Longitudinal axis *Trans. = Transverse axis* *Vert. = Vertical axis*
</td></tr>
<tr><td>18. Unseating Prevention System</td><td>
18.1 Seating Length
(1) Ordinary Bridge
➢ Eq. (18-1) shows the required seating length of a girder at its support.
➢ The seat length shall be measured in the direction perpendicular to the front line of the bearing support when
</td><td>
18.1 Seating Length (Minimum Support Length Requirements)
(a) <u>Seismic Design Categories A, B, and C </u>(4.12)
Support lengths at expansion bearings without restrainers, shock transmission unit (STUs), or damper shall
</td></tr>
</table>

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)

the direction of soil pressure acting on the substructure differs from the bridge axis, as in case of askew bridge or a curved bridge.

$S_E = U_R + U_G \geq S_{EM}$ -- (18-1)

$S_{EM} = 0.7 + 0.005 * Ls$ -- (18-2)

$U_G = \varepsilon_G * L$ -- (18-3)

where,

S_E: Refer to Fig. 18-1.

U_R: Maximum relative displacement between the superstructure and the edge of the top of the substructure due to Level 2 EGM (m) (refer to description of U_R below)

U_G: Relative displacement of the ground caused by seismic ground strain (m)

S_{EM}: Minimum seating length of a girder at the support

ε_G: Seismic ground strain

= 0.0025 for Ground Type I, 0.00375 for Ground Type II, 0.005 for Ground Type III

L: Distance between two substructures for determining the seating length (refer to description of L below)

Ls: Length of the effective span (m). When two superstructures with different span length are supported on one bridge pier, the longer one shall be used.

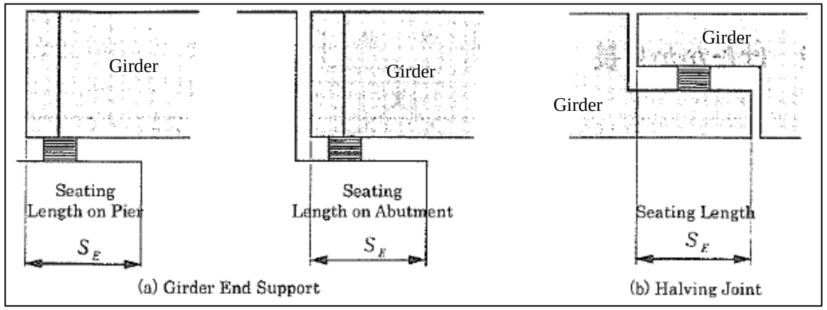


Fig.18-1 Seating Length (S_E) (m)

<u>Description of U_R</u>

(a) Rubber Bearing (refer to Fig 18-2)

$$u_R = \frac{c_m P_u}{k_B} \quad \cdots\cdots\cdots\cdots\cdots\cdots \quad (18\text{-}4)$$

P_u : Horizontal force equivalent to lateral strength of a column when considering plastic behavior of the column; or horizontal force equivalent to maximum response displacement of a foundation when considering plastic behavior of the foundation (kN)

c_m : Dynamic modification factor ($C_m = 1.2$)

k_B : Spring constant of the bearing support (kN/m)

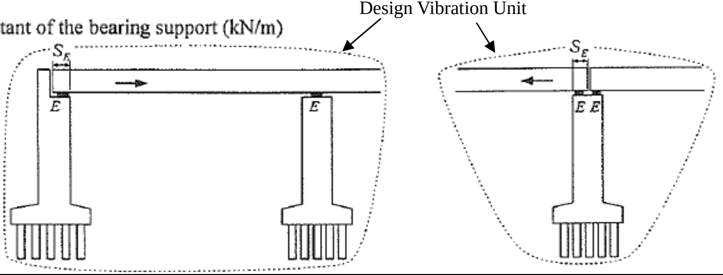


Fig. 18-2
When the girder end is supported by rubber bearings

either accommodate the greater of the:

- maximum displacement calculated in the inelastic dynamic response analysis
- or a percentage of the empirical support length , N

The empirical support length is shown in Eq. 18-1 while the percentage of N applicable to each seismic zone in given in Table 18-1:

$$N = (200 + 0.0017L + 0.0067H)(1 + 0.000125S^2) \qquad (18\text{-}1)$$

where:

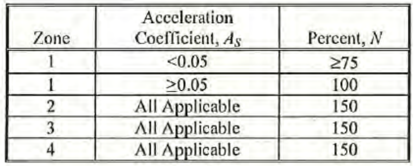
N = minimum support length measured normal to the centerline of bearing (mm),

L = length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck; for hinges within the span, L shall be the sum of the distances to either side of the hinge; for single span bridges, L equals to the length of the bridge deck (mm),

H = for abutment, average height of columns supporting the bridge deck from the abutments to the next expansion joint (mm),

S = angle of skew of support measured from a line normal to span (°).

Table 18-1 Percentage N by Zone and Acceleration Coefficient



Zone	Acceleration Coefficient, A_S	Percent, N
1	<0.05	≥75
1	≥0.05	100
2	All Applicable	150
3	All Applicable	150
4	All Applicable	150

(b) <u>Seismic Design Category D</u> (4.12)

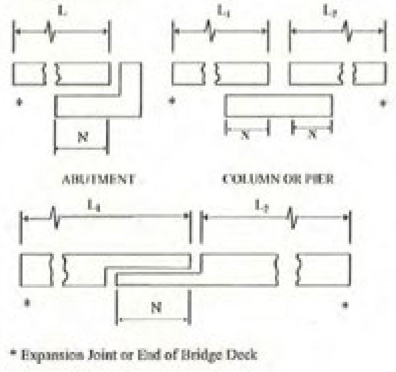
For SDC D, hinge seat or support length, N, shall be available to accommodate the relative longitudinal earthquake displacement demand at the support or at the hinge within a span between two frames and shall be determined as:

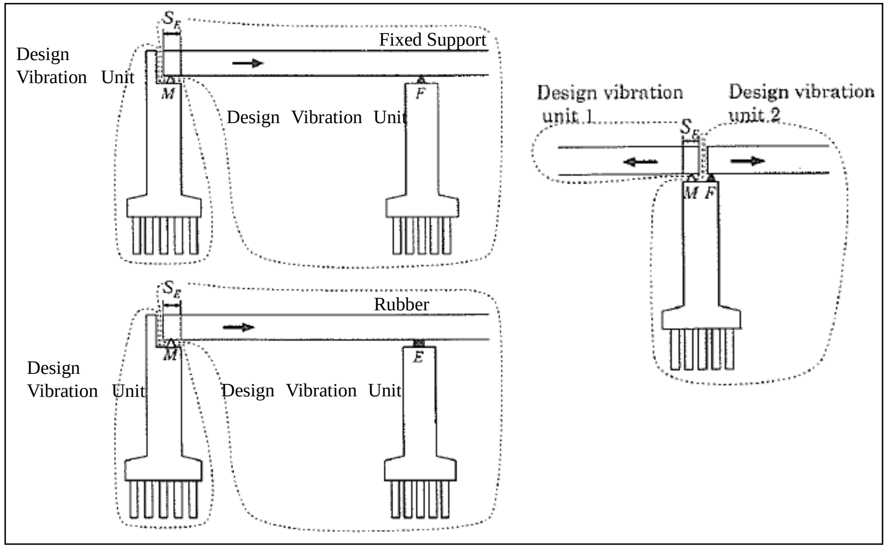
$$N = (4 + 1.65\Delta_{eq})(1 + 0.00025S^2) \geq 24 \qquad (18\text{-}2)$$

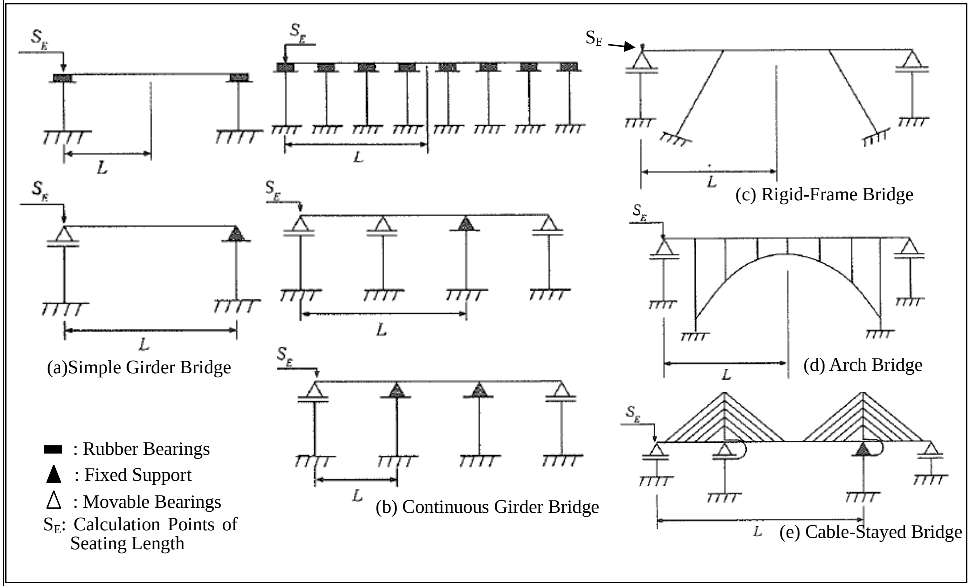
where:

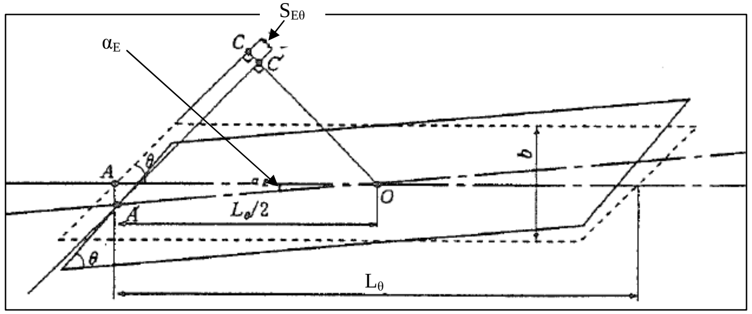
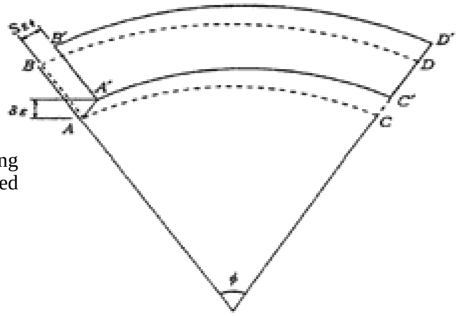
Δ_{eq} = seismic displacement demand of the long period frame on one side of the expansion joint (in),

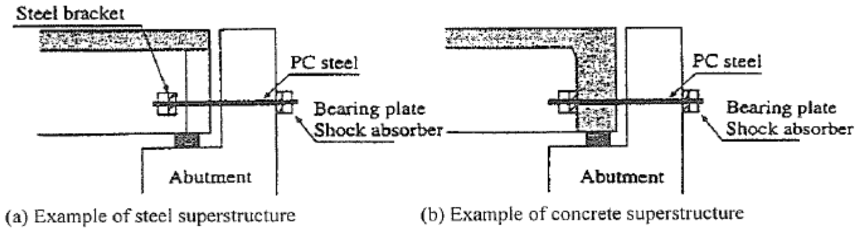
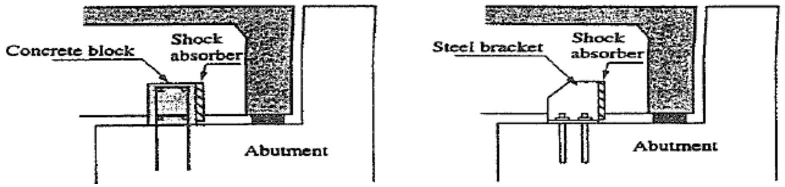
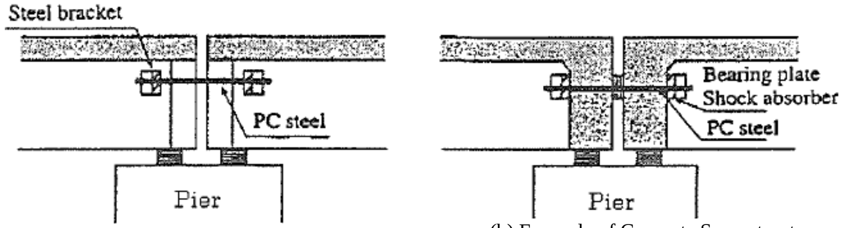
S = angle of skew of support measured from a line normal to span (°).

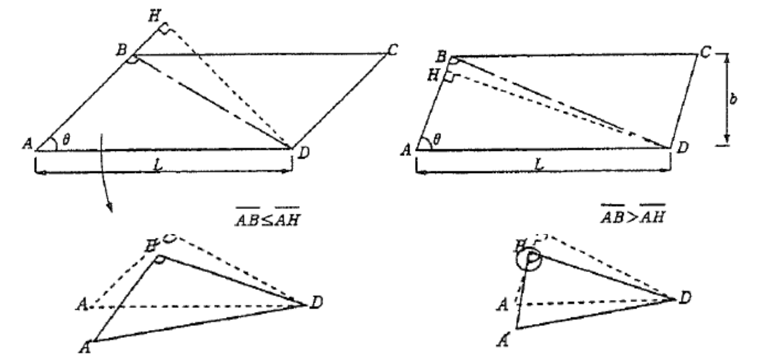
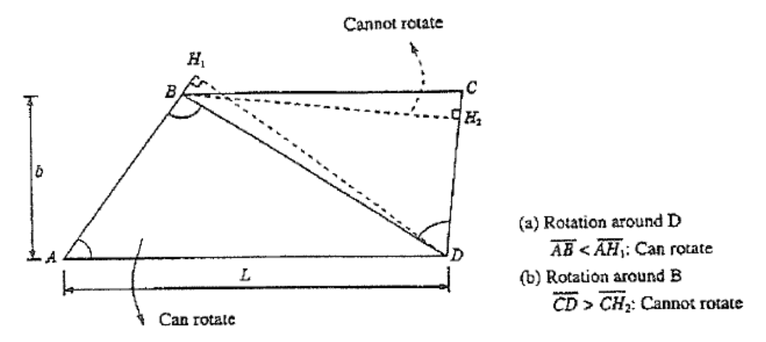
Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)
	(b) Fixed Bearing When fixed supports of Type B are used at the support concerned, one-half of the width of the bearing support shall be taken as the relative displacement u_R, while the whole width shall be used in case of fixed supports of Type A. This is for preventing unseating of the bridge even though the bearing support system is unexpectedly damaged, although the fixed supports of Type B are designed to be capable of transferring the superstructural inertia force caused either by horizontal force equivalent to lateral strength of a column with plastic behavior, or by horizontal force equivalent to maximum response displacement of a foundation with plastic behavior, and there is no relative displacement generating at the bearing support system under an assumed earthquake ground motion in the design. Providing that the bearing support may be damaged in case of fixed supports of Type A, u_R for Type A is assumed to be greater than that of Type B. In addition, the width of the bearing support shall be equal to width of the lower shoe in the bridge axis in the case of metal bearings, and width of the bearing support in the same direction for rubber bearings. (c) Movable Bearing (refer to Fig 18-3) $U_R = \sqrt{\sum U_{Ri}^2}$ (i = 1,2) -- (18-5) $U_{Ri} = U_{Pi} + U_{Fi} + U_{Bi}$ -- (18-6) $U_{Pi} = \mu_{Ri} * \delta_{yi}$ --- (18-7) $U_{Fi} = \delta_{Fi} + \theta_{Fi} * h_{oi}$ -- (18-8) $U_{Bi} = C_m * P_{ui} /K_{Bi}$ --- (18-9) where, u_{Ri}: Displacement of the i-th design vibration unit shown in Fig. 18-3 u_{Pi}: Response displacement of the column representing the i-th design vibration unit (m) u_{Fi} : Horizontal displacement at the height of superstructural inertia due to displacement of the pier foundation representing the i-th design vibration unit (m). The calculation shall include these effects when liquefaction and lateral spreading of the ground U_{Bi} : Relative displacement at the bearing support system representing the i-th design	 Figure 18-1 Support Length, N 18.2 Longitudinal Restrainers Support restraints, used to achieve an enhanced performance of the expansion joint, may be provided for longitudinal linkage at expansion joints within the space and at adjacent sections of simply supported superstructures. • Friction shall not be considered to be an effective restrainer. • Restrainers shall be detailed to allow for easy inspection and replacement. • Restrainer layout shall be symmetrical about the centerline of the superstructure. • Restrainer systems shall incorporate an adequate gap for service conditions. • Yield indicators may be used on cable restrainers to facilitate post-earthquake investigations. 18.3 Superstructure Shear Keys • For slender bents, shear keys on top of the bent cap may function elastically at the design hazard level. • In lieu of experimental test data, the overstrength shear key capacity, Vok, is taken as: $V_{ok} = 1.5V_n$ (18-3) where: V_{ok} = overstrength shear key capacity used in assessing the load path to adjacent capacity-protected members (kip) V_n = nominal interface shear capacity of shear key using nominal material properties and interface surface conditions (kip) • For shear keys at intermediate hinges within a span, the designer shall assess the possibility of shear key fusing mechanism, which is highly dependent on out-of-phase frame movements.

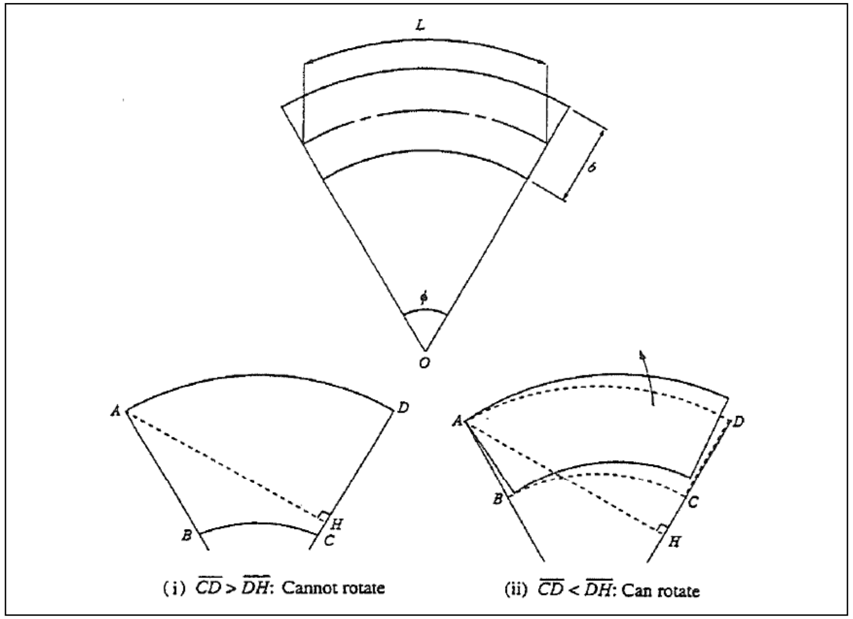
Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)
	vibration unit (m) μ_{Ri} : Response ductility factor of the column representing the i-th design vibration unit δ_{yi} : Yielding displacement of the column representing the i-th design vibration unit (m) δ_{Fi} : Response horizontal displacement of the pier foundation representing the i-th design vibration unit (m) θ_{Fi} : Response rotation angle of the pier foundation representing the i-th design vibration unit (rad) h_{0i} : Height from the ground surface of seismic design to the superstructural inertial force in the column representing the i-th design vibration unit (m) P_{ui} : Horizontal force equivalent to lateral strength of a column with plastic behavior, or by horizontal force equivalent to maximum response displacement of a foundation with plastic behavior, representing the i-th design vibration unit (kN) c_m : Dynamic modification factor (C_m = 1.2) k_{Bi} : Spring constant of the bearing support for the i-th design vibration unit (kN/m)  **Fig. 18-3** Inertia Forces Used in Calculating Seating Length <u>Description of L</u> The length L between the substructures that may affect the seating length shall be the distance between the substructure supporting the girder at the support where the seating length is to be calculated and one that may primarily affect the vibration of the girder containing the support (refer to Fig. 18-4)	

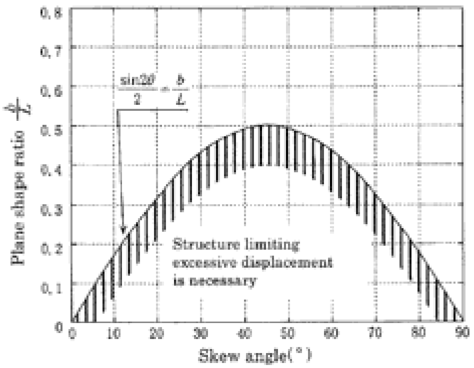
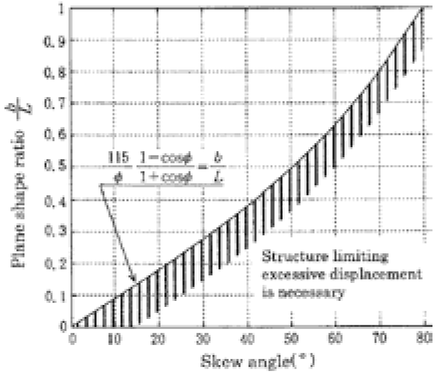
Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)
	 **Fig. 18-4** Measuring Methods of Distance (L) between Substructures as to Bridge Types (2) A Bridge with Complicated Dynamic Structural Behavior by a Dynamic Analysis ➢ The maximum relative displacement (U_R) is to be obtained from the dynamic analysis. (3) A Skew Bridge ➢ The seating length shall be calculated by Eq. (18-10) (refer to Fig. 18-5). $S_{E\theta} \geqq (L_\theta/2)(\sin\theta - \sin(\theta - \alpha_E))$ ·· (18-10) where $S_{E\theta}$: Seating length for the skew bridge (m) L_θ : Length of a continuous superstructure (m) θ : Skew angle (degree) α_E : Marginal unseating rotation angle (degree). α_E can generally be taken as 5 degrees.	

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)
	 **Fig. 18-5** Seating Length of a Skew Bridge (4) A Curved Bridge - The seating length is to be calculated by Eq. (18-11) (refer to Fig. 18-6) $S_{E\phi} \geqq \delta_E \frac{\sin\phi}{\cos(\phi/2)} + 0.3$ ………… (18-11) $\delta_E = 0.005\,\phi + 0.7$ ………… (18-12) where $S_{E\phi}$: Seating length for the curved bridge (m) δ_E : Displacement of the superstructure toward the outside direction of the curve (m) ϕ : Fan-shaped angle by the two edges of a continuous girder of a curved bridge (degrees) **Fig. 18-6** Seating Length Corresponding to the Movement of a Curved Bridge  18.2 Unseating Prevention Structure (1) Ultimate Strength of an Unseating Prevention Structure - Ultimate strength of an unseating prevention structure is to be calculated by Eq. (18-13). - The unseating prevention structure is a structure of 1) connecting the superstructure and the substructure (refer to Fig. 18-7), 2) providing protuberance either in superstructure and in the substructure (refer to Fig. 18-8), 3) joining two superstructures together (refer to Fig. 18-9).	

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)
	$H_F = 1.5\,R_d$ ········ (18-13) $S_F = c_F S_E$ ········ (18-14) where H_F : Design seismic force of the unseating prevention structure (kN) R_d : Dead load reaction (kN). In case the structure connects two adjacent girders, the larger reaction shall be taken. S_F : Maximum design allowance length of the unseating prevention structure (m) S_E : Seating length specified in Section 16.2 (m). c_F :Design displacement coefficient of the unseating prevention structure. (C_F = 0.75)  **Fig. 18-7** Unseating Prevention Structures Connecting the Superstructure with the Substructure  (a) Example of Concrete Block (b) Example of Steel Bracket **Fig. 18-8** Unseating Prevention Structures Providing Protuberance on the Superstructure or the Substructure  (a) Example of Steel Superstructure (b) Example of Concrete Superstructure **Fig. 18-9** Unseating Prevention Structures Connecting the Two Adjacent Superstructures	

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)
	18.3 Structure Limiting Excessive Displacement (1) For the following bridges, structures limiting excessive displacement working in the direction perpendicular to the bridge axis shall be installed in the terminal support, in addition to the unseating prevention system working in the bridge axis. (a) Skew bridges with a small skew angle satisfying Eq. (18-15) (refer to Fig. 18-10 and 18-11) $\sin 2\theta/2 > b / L$ .... (18-15) L: Length of a continuous superstructure (m) b: Whole width of the superstructure (m) θ: Skew angle (degree)  (a) ∠ DBA ≥ 90° (a bridge can rotate) (b) ∠ DBA < 90° (a bridge can not rotate) **Fig. 18-10** Conditions in Which a Skew Bridge can Rotate Without Being Affected by Adjoining Girders or Abutment  **Fig. 18-11** Conditions in Which a Skew Bridge With Unparallel Bearing Lines on Both Edges of the Superstructure can rotate	

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)
	(b) Curved bridges satisfying Eq. (18-16) (refer to Fig. 18-12) $\frac{115}{\phi}\frac{1-\cos\phi}{1+\cos\phi} > b/L$ ………………………… (18-16) L: Length of a continuous superstructure (m) b: Whole width of the superstructure (m) Φ: Intersection angle (degree)  (i) $\overline{CD} > \overline{DH}$: Cannot rotate (ii) $\overline{CD} < \overline{DH}$: Can rotate **Fig. 18-12** Conditions in Which a Curved Bridge can rotate Without being Affected by Adjoining Girders or Abutment The relation of Eq. (18-15) and Eq. (18-16) is shown in Fig. 18-13 and Fig. 18-14, respectively.	

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2nd Edition, 2011)
	 **Fig. 18-13** Conditions in which a Skew Bridge Requires an Structure Limiting Excessive Displacement in the Transverse Direction  **Fig. 18-14** Conditions in which a Curved Bridge Requires an Structure Limiting Excessive Displacement in the Transverse Direction (c) For the following bridges, the structures limiting excessive displacement shall be installed at intermediate supports ➢ Bridges with the superstructure being narrow at the top ➢ Bridges with a small number of bearing supports on one bearing line ➢ Bridges probably to be subject to movement of the bridge piers in the direction perpendicular to the bridge axis as a result of lateral spreading.	
19. Effects of Seismically Unstable Ground	19.1 Assessment of Extremely Soft Clayey Soil Layer in Seismic Design (1) For a clayey layer or a silt layer lying within three meters from the ground surface, and having compressive strength of 20KN/m or less obtained from an unconfined compression test or an in-situ test, the layer shall be regarded as an extremely soft layer in the seismic design. (2) In this case its geological parameters (shear modulus and strength) shall be assumed to be zero in the seismic design. 19.2 Assessment of Soil Liquefaction (1) Sandy Layer Requiring Liquefaction Assessment ➢ Saturated soil layer having ground water level higher than 10m below the ground surface and lying at a depth less than 20m below the ground surface. ➢ Soil layer containing a fine content (FC) of 35% or less, or soil layer having plasticity index Ip less than 15, even if FC is larger than 35%. ➢ Soil layer having a mean particle size (D_{50}) less than 10mm and a particle size at 10% pass (on the grading curve) (D_{10}) is less than 1mm. (2) Assessment of Liquefaction The liquefaction resistance factor FL calculated by Eq. (19-1) turns out to be less than 1.0, the layer shall be regarded as a soil layer having liquefaction potential.	(1) For SDC C and D, liquefaction assessment shall be conducted when both of the following conditions are present: ▪ *Groundwater Level*. The groundwater level anticipated at the site is within 15.24m (50ft) of the existing ground surface or the final ground surface, whichever is lower. ▪ *Soil Characteristics*. Low plasticity silts and sands within the upper 22.86m (75ft) are characterized by one of the following conditions: (1) the corrected standard penetration test (SPT) blow count, $(N_1)_{60}$, is less than or equal to 25 blows/ft in sand and non-plastic silt layers, (2) the corrected cone penetration test (CPT) tip resistance, q_{ciN}, is less than or equal to 150 in sand and in non-plastic silt layers, (3) the normalized shear wave velocity, V_{s1}, is less than 660fps, or (4) a geologic unit is present at the site that has been observed to liquefy in past earthquakes. (2) For sites that require assessment of liquefaction, the potential effects of liquefaction on soils and foundations shall be evaluated. The assessment shall consider the following effects of liquefaction: ▪ Loss in strength in the liquefied layer or layers, ▪ Liquefaction-induced ground settlement, and ▪ Flow failures, lateral spreading, and slope instability. (3) For sites where liquefaction occurs around bridge foundations, bridges should be analyzed and designed in two configurations as follows:

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)
	$F_L = R / L$ ……………………………………………………………… (19-1) $R = c_w R_L$ $L = r_d k_{hg} \sigma_v / \sigma'_v$ $R_d = 1.0 - 0.015x$ $\sigma_v = \gamma_{t1} h_w + \gamma_{t2} (x - h_w)$ $\sigma'_v = \gamma_{t1} h_w + \gamma'_{t2} (x - h_w)$ (For Type I Earthquake Ground Motion) $c_w = 1.0$ ……………………………… (For Type II Earthquake Ground Motion) $c_w = \begin{cases} 1.0 & (R_L \leqq 0.1) \\ 3.3R_L + 0.67 & (0.1 < R_L \leqq 0.4) \\ 2.0 & (0.4 < R_L) \end{cases}$ F_L : Liquefaction resistance factor R : Dynamic shear strength ratio L : Seismic shear stress ratio c_W : Modification factor on earthquake ground motion R_L : Cyclic triaxial shear stress ratio to be obtained by properties (3) below r_d : Reduction factor of seismic shear stress ratio in terms of depth k_{hg}: Design horizontal seismic coefficient at the ground surface for Level 2 Earthquake Ground Motion specified in Item 11 σ_v: Total overburden pressure (kN/m^2) σ'_v : Effective overburden pressure (kN/m^2) x : Depth from the ground surface (m) γ_{t1}: Unit weight of soil above the ground water level (kN/m^3) γ_{t2}: Unit weight of soil below the ground water level (kN/m^3) γ'_{t2}: Effective unit weight of soil below the ground water level (kN/m^3) h_w: Depth of the ground water level (m) (3) Cyclic Tri-axial Shear Stress Ratio Cyclic tri-axial shear stress ratio R_L shall be calculated by Eq. (19-2). $R_L = \begin{cases} 0.0882\sqrt{N_a / 1.7} & (N_a < 14) \\ 0.0882\sqrt{N_a / 1.7} + 1.6 \times 10^{-6} \cdot (N_a - 14)^{4.5} & (14 \leq N_a) \end{cases}$ …… (19-2) <For Sandy Soil> $N_a = c_1 N_1 + c_2$	▪ *Non-liquefied Configuration*. The structure should be analyzed and designed, assuming no liquefaction occurs, using the ground response spectrum appropriate for the site soil conditions in a non-liquefied state. ▪ *Liquefied Configuration*. The structure as designed in non-liquefied configuration above shall be reanalyzed assuming that the layer has liquefied and the liquefied soil provides the appropriate residual resistance for lateral and axial deep foundation response analyses consistent with liquefied soil conditions (i.e., modified *P-y* curves, modulus of subgrade reaction, or *t-z* curves). The design spectrum shall be that used in a non-liquefied configuration. (4) As required by the Owner, a site-specific response spectrum that accounts for the modifications in spectral content from the liquefying soil may be developed. Unless approved otherwise, the reduced response spectrum resulting from the site-specific analyses shall not be less than two-thirds of the spectrum at the ground surface developed using the general procedure modified by the site coefficients. (5) The Designer should provide explicit detailing of plastic hinge zones for both cases mentioned above since it is likely that the locations of plastic hinges for the liquefied configurations are different than the locations of the plastic hinges for the non-liquefied configuration. Design requirements including shear reinforcement should be met for the liquefied and the non-liquefied configuration. Where liquefaction is identified, plastic hinging in he foundation may be permitted with the Owners approval provided that the provisions of earthquake resisting systems are satisfied. (6) The effects of liquefaction-related, permanent lateral ground displacements on bridge and retaining wall performance should be considered separate from the inertial evaluation of the bridge structures. However, if large magnitude earthquakes dominate the seismic hazards, the bridge response evaluation should consider the potential simultaneous occurrence of: ▪ Inertial response of the bridge, and loss in ground response from liquefaction around the bridge foundations, and ▪ Predicted amounts of permanent lateral displacement of the soil. (7) During liquefaction, pore-water pressure build-up occurs, resulting in loss of strength and then settlement as the excess pore-water pressures dissipate after the earthquake. The potential effects of strength loss and settlement includes: ▪ *Slope Failure, Flow Failure, or Lateral Spreading*. The strength loss associated with pore-water pressure build-up can lead to slope instability. Generally, if the factor of safety against liquefaction is less than approximately 1.2 to 1.3, a potential for pore-water pressure build-up will occur, and the effects of this build-up should be assessed. If the soil liquefies, the stability is determined by the residual strength of the soil. The residual strength of liquefied soils can be determined using empirical methods developed by Seed and Harder (1990), Olson and Stark (2002), and others. Loss of lateral resistance can allow abutment soils to move laterally, resulting in bridge substructure distortion and unacceptable deformations and moments in the superstructure. ▪ *Reduced Foundation Bearing Resistance*. Liquefied strength is often a fraction of non-liquefied strength. This loss in strength can result in large displacements or bearing failure. For this reason, spread footing foundations are not recommended where liquefiable soils occur unless the spread footing is located below the maximum depth of liquefaction or soil improvement techniques are used to mitigate the effects of liquefaction. ▪ *Reduced Soil Stiffness and Loss of Lateral Support for Deep Foundation*. This loss in strength can

Items	JRA Specifications for Highway Bridges, Part V-Seismic Design (English Version, 2002)	AASHTO Guide Specifications for LRFD Seismic Bridge Design (2^{nd} Edition, 2011)

$$N_1 = 170N/(\sigma'_v + 70)$$

$$c_1 = \begin{cases} 1 & (0\% \leqq FC < 10\%) \\ (FC+40)/50 & (10\% \leqq FC < 60\%) \\ FC/20-1 & (60\% \leqq FC) \end{cases}$$

$$c_2 = \begin{cases} 0 & (0\% \leqq FC < 10\%) \\ (FC-10)/18 & (10\% \leqq FC) \end{cases}$$

<For Gravelly Soil>

$$N_a = \{1 - 0.36\, log_{10}(D_{50}/2)\} N_1$$

R_L: Cyclic triaxial shear stress ratio

N: N value obtained from the standard penetration test

N_1 : Equivalent N value corresponding to effective overburden pressure of 100 kN/m^2

N_a : Modified N value taking into account the effects of grain size

c_1, c_2: Modification factors of N value on fine content

FC : Fine content (%) (percentage by mass of fine soil passing through the 75μm mesh)

D_{50}: Mean grain diameter (mm)

19.3 Reduction Factor (D_E) of Geotechnical Parameters due to Liquefaction

Geotechnical parameters of a sandy layer causing liquefaction affecting a bridge shall be obtained by multiplying geotechnical parameters without liquefaction by reduction factor D_E shown in Table 19-1.

Table 19-1 Reduction Factor DE for Geotechnical Parameters

Range of F_L	Depth from Present Ground Surface x (m)	Dynamic shear strength ratio R			
		$R \leqq 0.3$		$0.3<R$	
		Verification for Level 1 Earthquake Ground Motion	Verification for Level 2 Earthquake Ground Motion	Verification for Level 1 Earthquake Ground Motion	Verification for Level 2 Earthquake Ground Motion
$F_L \leqq 1/3$	$0 \leqq x \leqq 10$	1/6	0	1/3	1/6
	$10<x \leqq 20$	2/3	1/3	2/3	1/3
$1/3<F_L \leqq 2/3$	$0 \leqq x \leqq 10$	2/3	1/3	1	2/3
	$10<x \leqq 20$	1	2/3	1	2/3
$2/3<F_L \leqq 1$	$0 \leqq x \leqq 10$	1	2/3	1	1
	$10<x \leqq 20$	1	1	1	1

change the lateral response characteristics of piles and shafts under lateral load.

- *Vertical Ground Settlement as Excess Pore-Water Pressures Induced by Liquefaction Dissipate, Resulting in Downdrag Loads on Deep Foundations*. If liquefaction-induced downdrag loads can occur, the downdrag loads should be assessed.

(3) COMPARISON TABLE OF BRIDGE SEISMIC SPECIFICATIONS BETWEEN JRA AND NSCP Vol. II Bridges ASD (Allowable Stress Design),

2nd Ed., 1997 (Reprint Ed. 2005)

<table>
<tr><th>Items</th><th>JRA (Part V; English Version, 2002)</th><th>NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP</th></tr>
<tr><td>1. Fundamentals of Seismic Design</td><td>(1) It shall be ensured that the seismic performance according to the levels of design earthquake motion and the importance of a bridge.
(2) It is desirable to adopt a multi-span continuous structure, the type of which bearing supports is to be a horizontal force distributed structure.
(3) It is generally better for a bridge with tall piers built in a mountainous region to resist seismic horizontal forces by abutments rather than piers if the ground conditions at the abutments are sufficiently sound.(The seismic performance of the whole bridge should be considered, and proper bearing supports in view of bridge structural conditions and ground bearing properties should be selected.)
(4) On reclaimed land or alluvial ground where ground deformation such as sliding of a soft cohesive clayey layer, liquefaction of sandy layer and liquefaction-induced ground flow may happen, a foundation with high horizontal stiffness should be designed, and a structural system such as multi-fixed-point type and rigid frame type, which has many contact points between the superstructure and substructure, should be selected.
(5) A seismically-isolated bridge should be adopted for a multi-span short-period continuous bridge on stiff ground conditions.
(6) For a strong earthquake motion, a proper structural system shall be designed by clarifying structural members with nonlinear behavior and those basically remaining in elastic states.
(7) A structure greatly affected by geometrical nonlinearity or a structure having extensive eccentricity of dead loads, which have tends to become unstable during a strong earthquake motion, shall not be adopted.
(8) When ground conditions or structural conditions on a pier change remarkably, whether a case of two girder ends or that of a continuous girder is more advantageous is carefully examined.</td><td>(1) The design earthquake motions and forces specified in these provisions in the provisions are based on low probability of their being exceeded during the normal life expectancy of the bridge (probability of the elastic forces not being exceeded in 50 years in the range of 80 to 95%).
(2) Bridges and their components that are designed to resist earthquake forces and that are constructed in accordance with the design details contained in the provisions may suffer damage, but should have low probability of collapse due to seismically induced ground shaking. Where possible, damage that does occur should be readily detectable and accessible for inspection and repair.
(3) Development of the Standards has been predicated on the following basic concepts:
• Hazard to life be minimized,
• Bridges may suffer damage but have low probability of collapse due to earthquake motions,
• Function of essential bridges be maintained,
• Design ground motions have low probability of being exceeded during normal life of bridge,
• Provision be applicable to all parts of the Philippines, and
• Ingenuity of design not be restricted.
(4) The Standards are for the design and construction of new bridges to resist the effect of earthquake motions. The provisions apply to bridges of conventional steel and concrete girder and box girder construction with spans not exceeding 150m. Suspension bridges, cable-stayed bridges, arch type and movable bridges are not covered by these Standards.
(5) The Department of Public Works and Highways (DPWH) issued a Department Order No. 75 (D.O.75), July 17, 1992 – “<i>DPWH Advisory for Seismic Design of Bridges</i>” which requires the following design concept to be adopted:
• Continuous bridges with monolithic multi-column bents have a high degree of redundancy and are preferred type of bridge structure to resist shaking. Deck discontinuities such as expansion joints and hinges should be kept to an absolute minimum. Suspended spans, brackets, rockers, etc. are not recommended.
• Where multi-span simple span bridges are justified, decks should be continuous.
• Restrainers (horizontal linkage device between adjacent spans) are required at all joints in accordance with AASHTO provisions and generous seat widths at piers and abutments should be provided to prevent loss-of-span type of failures.
• Transverse reinforcement in the zones of yielding is essential to the successful performance of reinforced concrete columns during earthquakes. Transverse reinforcement serves to confine the main longitudinal reinforcement and the concrete within the core of the column, thus presenting buckling of the main reinforcement.
• Plastic hinging should be forced to occur in ductile column regions of the pier rather than in the foundation unit. A scheme to protect the abutment piles from failure is often accomplished by designing the backwall to shear off when subjected to the design seismic lateral force that would otherwise fail the abutment piles.
• The stiffness of the bridge as a whole should be considered in the analysis. In regular structures, as defined previously, it is particularly important to include the soil-structure interaction.</td></tr>
<tr><td>2. Principles of Seismic Design</td><td>(1) Seismic Performance of Bridges
<table>
<tr><th rowspan="2">Seismic Performance</th><th rowspan="2">Seismic Safety Design</th><th rowspan="2">Seismic Serviceability Design</th><th colspan="2">Seismic Reparability Design</th></tr>
<tr><th>Emergency Reparability</th><th>Permanent Reparability</th></tr>
<tr><td><u>Seismic Performance</u></td><td>To prevent</td><td>To ensure the normal</td><td>No repair work is</td><td>Only easy repair</td></tr>
</table></td><td>(1) <u>Performance Level</u>
The performance levels for bridges after the occurrence of the design earthquake event is stated in the Standards as summarized below:</td></tr>
</table>

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP

Level 1 Keeping the sound functions of bridges	girders from unseating	functions of bridges (within elastic limit states)	needed to recover the functions	works are needed
Seismic Performance Level 2 Limited damages and recovery	Same as above	Capable of recovering functions within a short period after the event	Capable of recovering functions by emergency repair works	Capable of easily undertaking permanent repair works
Seismic Performance Level 3 No critical damages	Same as above	- *	-	-

*: "-": Not covered

(2) Relationship between Design Earthquake Ground Motions and Seismic Performance of Bridges

Levels of Earthquake Ground Motions		Class A Bridges*	Class B Bridges*
Level 1: Highly probable during the bridge service life		**Seismic Performance Level 1** is required	
Level 2	Type I: An Plate Boundary Type Earthquake with a Large Magnitude	**Seismic Performance Level 3** is required	**Seismic Performance Level 2** is required
	Type II: An Inland Direct Strike Type Earthquake		

*: Class A Bridges: Standard Importance; Class B Bridges: High Importance (Class A and B are classified according to such importance factors as road class, bridge functions and structural characteristics.)

When bridge importance is classified in view of roles expected in the regional disaster prevention plan and road serviceability, the following should be considered.

(a) To what extent a bridge is necessitated for post-event rescue and recovery activities as emergency transportation routes.

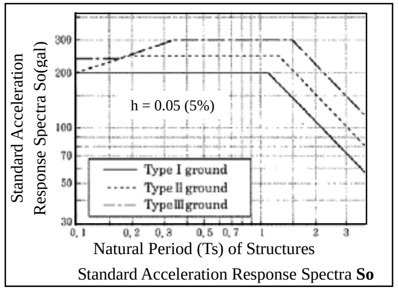
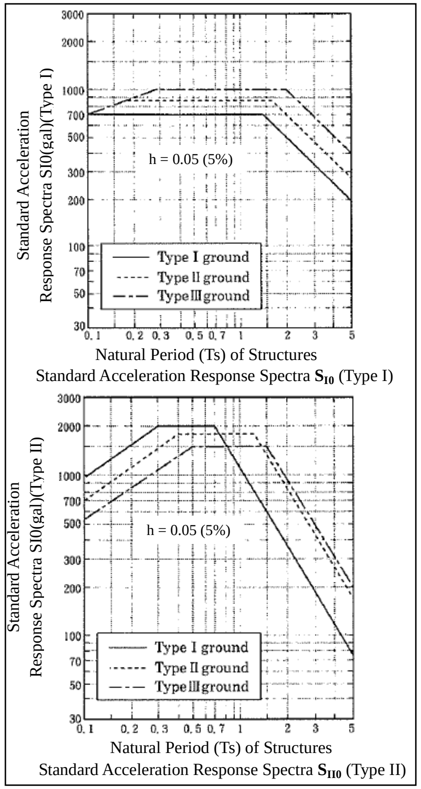
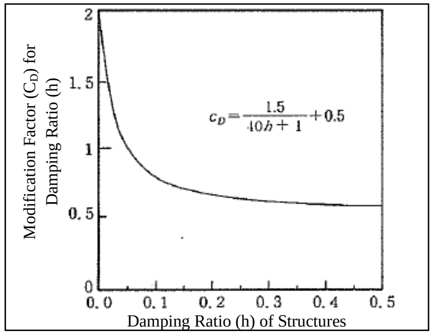
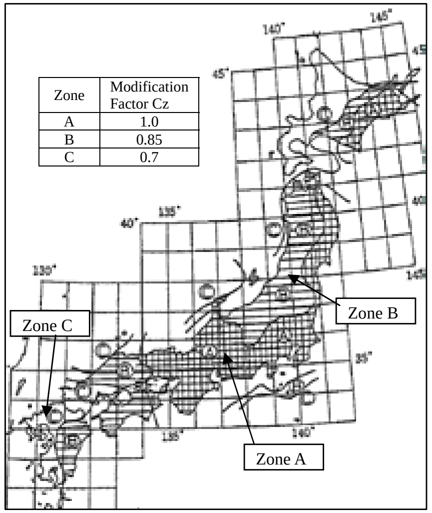
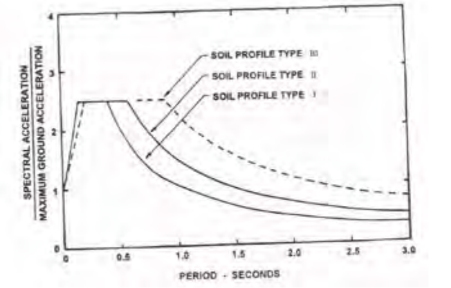
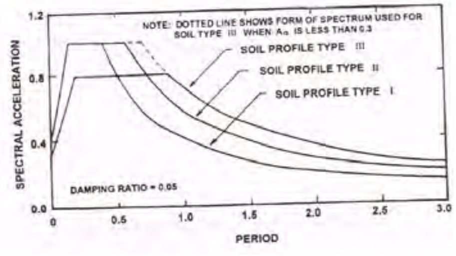
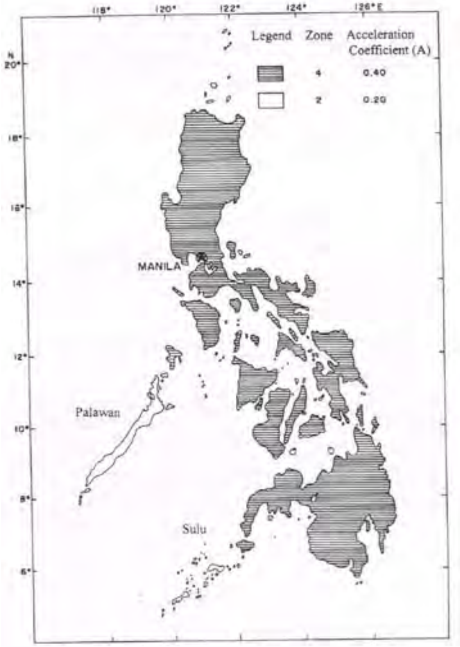
(b) To what extent damages to bridges (such as double-deck bridges and overbridges) affect other structures and facilities.

(c) Present traffic volume of the bridge and availability of substitute in case of the bridges losing pre-event functions.

(d) Difficulty (duration and cost) in recovering bridge function after the event.

Earthquake Level	Bridge Types	Serviceability Performance	Safety Performance
Small/Moderate	Conventional and regular bridge types	• Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage.	• No significant damage to members
Large/Major	Critical bridges/ Essential bridges/ Conventional and regular bridges	• No explicit performance criteria but since collapse is not allowed and damages can be repaired, bridges are expected to function after the design earthquake event.	• May suffer damage but should not cause collapse of all or any of its parts. • Damage should be readily detectable and accessible for inspection and repair.

Items	**JRA (Part V; English Version, 2002)**	**NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP**
3. Loads to be considered in Seismic Design	(1) Loads and their Combinations (a) Primary Loads: Dead load (D), Pre-stress force (PS), Effect of creep of concrete (CR), Effect of drying shrinkage of concrete (SH), Earth pressure (E), Hydraulic pressure (HP), Buoyancy or Uplift (U) (b) Secondary loads: Effects of earthquake (EQ) (c) Combination of loads: Primary loads + Effects of earthquake (EQ) (d) Loads and their combinations shall be determined in such manners that they cause the most adverse stress, displacements and effects. (2) Effects of Earthquake (EQ) (a) Inertia force, (b) Earth pressure during an earthquake, (c) Hydrodynamic pressure during an earthquake, (d) Effects of liquefaction and liquefaction-induced ground flow, (e) Ground displacement during an earthquake	(1) Design Forces Seismic design forces shall apply to: (a) the superstructure, its expansion joints and the connection between the superstructure and the supporting substructure, (b) the supporting substructure down to the base of the columns and piers but not including the footing, pile cap or piles, and (c) components connecting the superstructure to the abutment. (2) Group Load Combination The maximum loading for each component is calculated as: Group Load = 1.0 (D + B + SF + E + EQM) (3-1) where: D = dead load B = buoyancy SF = stream-flow pressure E = earth pressure EQM = elastic seismic force for Load Case 1 or Load Case 2 divided by the appropriate R-Factor. Note that seismic forces are reversible (positive and negative). Maximum and minimum axial forces for columns, shall be calculated for each load case by taking the seismic axial force as positive and negative. (3) Combination of Orthogonal Seismic Forces Load Case 1: Combination of 100% of the absolute value of member elastic forces and moments resulting from analysis in the first perpendicular direction (longitudinal) with 30% of the absolute value of member elastic forces and moments resulting from analysis in the second perpendicular direction (transverse). Load Case 2: Combination of 100% of the absolute value of member elastic forces and moments resulting from analysis in the second perpendicular direction (transverse) with 30% of the absolute value of member elastic forces and moments resulting from analysis in the first perpendicular direction (longitudinal).

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2^{nd} Ed., 1997 (Reprint Ed. 2005) - ASEP
4. Design Earthquake Ground Motions for Level 1 and Level 2	**S** = $C_Z*C_D*\mathbf{S_0}$ (S: ARS for Level 1EGM, S0: SARS (Fig.4-1)) **SI** = $C_Z*C_D*\mathbf{S_{I0}}$ (SI: Type I ARS for Level 2 EGM, SI0:SARS (Fig.4-2)) **SII** = $C_Z*C_D*\mathbf{S_{II0}}$ (SII: Type II ARS for Level 2 EGM, SII0: SARS(Fig.4-2)) (SARS= **S**tandard **A**cceleration **R**esponse **S**pectra, ARS= **A**cceleration **R**esponse **S**pectra, EGM = **E**arthquake **G**round **M**otion) C_D: Modification factor for damping ratio (h) of structures (Fig.4-3) C_Z: Modification factor for zones (Fig.4-4)  **Fig.4-1** Level 1 Earthquake Ground Motion  **Fig 4-2** Level 2 Earthquake Ground Motions  **Fig.4-3** Modification Factor (CD) for Damping Ratio (h) of Structures  **Fig.4-4** Modification Factors for Zones, Cz	(1) The Standards assigns two (2) seismic zones in the Philippines as shown in Figure 4-1. Basically, the whole of the country is in zone 4, except Palawan which is in zone 2. (2) The design ground motion spectra for 5% damping is developed for 3 soil type conditions, as shown in Figure 4-2a (normalized) and Figure 4-2b with the effective peak acceleration (EPA) A=0.40.  (a) normalized  (b) for A=0.4 Figure 4-2 Response Spectra  Figure 4-1 Seismic Zone Map in the Philippines

Items

5. Ground Type for Seismic Design

JRA (Part V; English Version, 2002)

Table 5-1 Ground Types in Seismic Design

Ground Type	Characteristic Value of Ground, T_G (s)	Description
Type I	$T_G < 0.2$	Good diluvial ground and rock
Type II	$0.2 \leq T_G < 0.6$	Diluvial and alluvial ground not belonging to Type I and Type II
Type III	$0.6 \leq T_G$	Soft ground of alluvial ground

$T_G = 4 * \sum_{i=1}^{n} Hi / Vsi$ T_G = Characteristic value of ground (s)

Hi = Thickness of the i-th soil layer

Vsi = Average shear wave velocity of the i-th soil layer (m/s). If Vsi is not available, Vsi can be obtained from the following formula.

Vsi = 100 * $Ni^{1/3}$ ($1 \leq Ni \leq 25$): for cohesive soil layer (if N = 0, Vsi = 50 m/s; when N=25, Vsi = 300m/s)

Vsi = 80 * $Ni^{1/3}$ ($1 \leq Ni \leq 50$): for sandy soil layer (if N = 0, Vsi = 50 m/s; when N=50, Vsi = 300m/s)

Ni = Average N value of thei-th soil layer obtained from SPT

i = Number of the i-th layer from the ground surface when the ground is classified into “n” layers up to “the surface of a base ground surface for seismic design”

Note: “The surface of a base ground surface for seismic design” represents upper surface of a fully hard ground layer that exists over a wide area in the construction site, and normally situated below a surface soil layer shaking with a ground motion during an earthquake. Where, the upper surface of a fully hard ground layer might be the upper surface of a highly rigid soil layer with a shear elastic wave velocity of more than 300m/s (an N value of 25 in the cohesive soil layer and of 50 in the sandy soil layer)

If N value is not available, Ground types can be obtained following the flow chart shown in Fig 5-1.

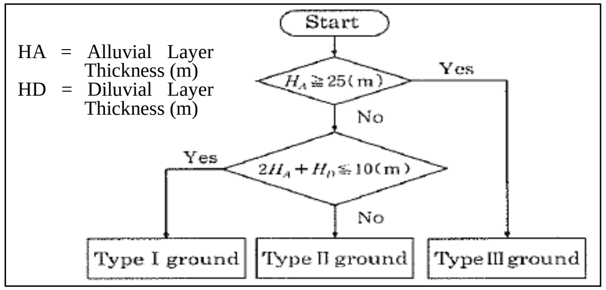


Fig.5-1 Flowchart for Determining Ground Types

NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP

(1) Site Effects

The effects of site condition on bridge response shall be determined from site coefficient (S) based on soil profile types defined as:

SOIL PROFILE TYPE I is a soil profile with either:

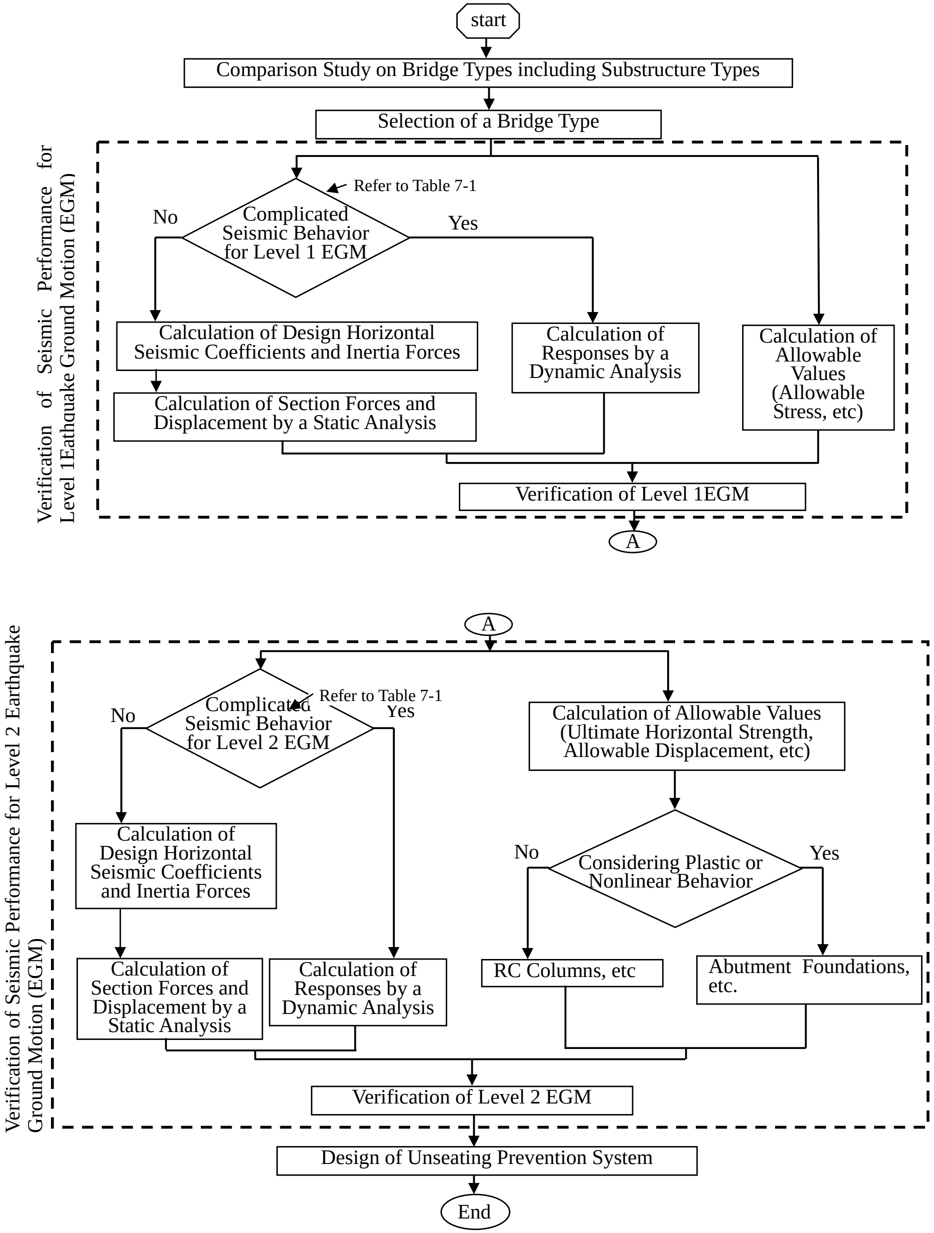
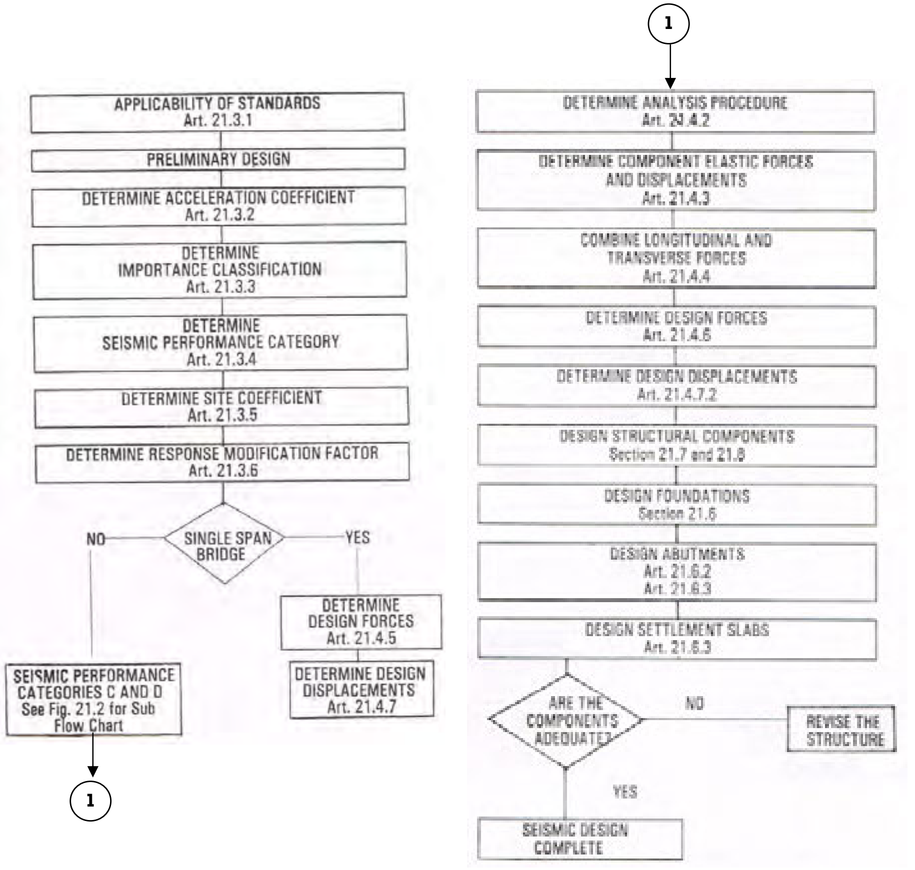
1. Rock of any characteristics, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 760m/s, or by other appropriate means of classification); or
2. Stiff soil conditions where the soil depth is less than 60m and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

SOIL PROFILE TYPE II is a profile with stiff clay or deep cohesionless conditions where the soil depth exceeds 60m and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

SOIL PROFILE TYPE III is a profile with soft to medium-stiff clays and sands, characterized by 10m or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils.

Table 5-1 Site Coefficient (S)

Coeff.	Soil Profile Type		
	I	II	III
S	1.0	1.2	1.5

<table>
<tr><th>Items</th><th>JRA (Part V; English Version, 2002)</th><th>NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP</th></tr>
<tr><td>6. Procedure of Seismic Design</td><td>
</td><td>

(a) Design Flow Chart
(b) Sub-flow chart for Seismic Performance Categories C and D
Figure 6-1 Seismic Design Procedure Flow Chart</td></tr>
</table>

Items

7. Limit States of Bridges for each Seismic Performance Level

JRA (Part V; English Version, 2002)

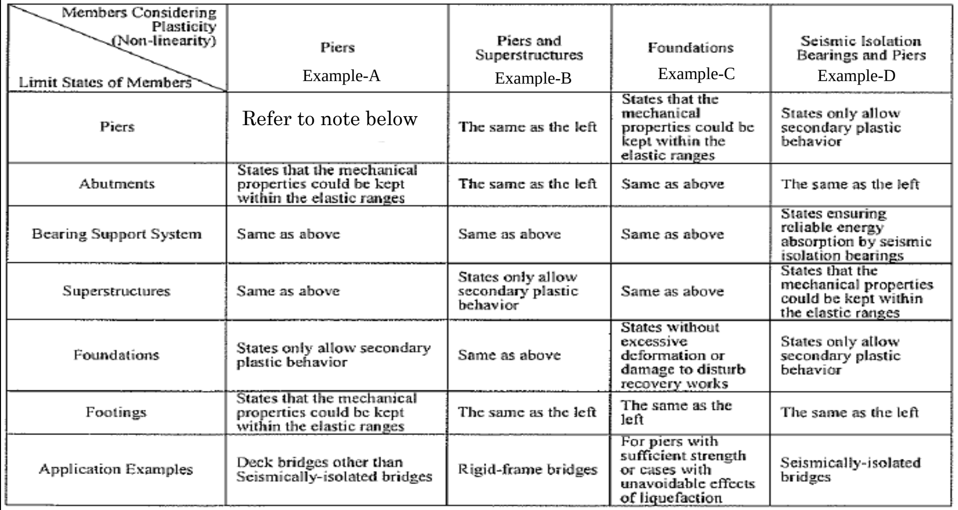
(1) Limit States of Bridges for Seismic Performance Level 1

(a) The mechanical properties of the bridges including expansion joint are maintained within the elastic range.

(b) Stress occurring in concrete of each structural member reaches its allowable stress multiplied by an increase factor of 1.5 for consideration of earthquake effects.

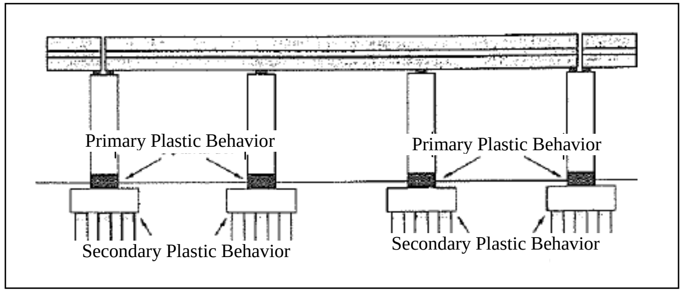
(2) Limit State of Bridges for Seismic Performance Level 2 (Refer to table 7-1 and Fig. 7-1)

Table 7-1 Limit States of Each Member with Applicable Examples for Seismic Performance Level 2 and Level 3 (Refer to Fig. 7-1 about Examples)



Members Considering Plasticity (Non-linearity) / Limit States of Members	Piers Example-A	Piers and Superstructures Example-B	Foundations Example-C	Seismic Isolation Bearings and Piers Example-D
Piers	Refer to note below	The same as the left	States that the mechanical properties could be kept within the elastic ranges	States only allow secondary plastic behavior
Abutments	States that the mechanical properties could be kept within the elastic ranges	The same as the left	Same as above	The same as the left
Bearing Support System	Same as above	Same as above	Same as above	States ensuring reliable energy absorption by seismic isolation bearings
Superstructures	Same as above	States only allow secondary plastic behavior	Same as above	States that the mechanical properties could be kept within the elastic ranges
Foundations	States only allow secondary plastic behavior	Same as above	States without excessive deformation or damage to disturb recovery works	States only allow secondary plastic behavior
Footings	States that the mechanical properties could be kept within the elastic ranges	The same as the left	The same as the left	The same as the left
Application Examples	Deck bridges other than Seismically-isolated bridges	Rigid-frame bridges	For piers with sufficient strength or cases with unavoidable effects of liquefaction	Seismically-isolated bridges

Note: States within a range of easy functional recovering for Seismic Performance Level 2; States that horizontal strength of piers start to get reduced rapidly for Seismic Performance Level 3.



(a) Example A: Single Column Pier with Plastic Behavior (in longitudinal direction)

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(1) Bridges and their components that are designed to resist these forces and that are constructed in accordance with the design details contained in the provisions may suffer damage, but should have low probability of collapse due to seismically induced ground shaking. - *NSCP*

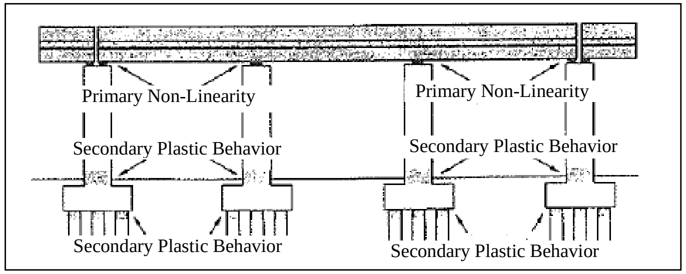
(2) In case of large earthquakes, a bridge may suffer damage but this should not cause collapse of all or any of its parts and such damage should readily be detectable and accessible for inspection and repair. – *DPWH D.O.75*

(3) The design concept considers "*ductile substructure with essentially elastic superstructure and foundation*". This includes conventional plastic hinging in columns.

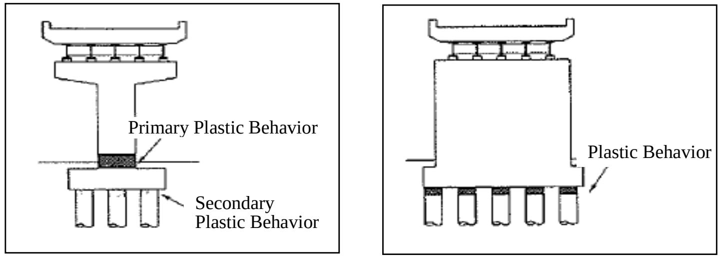
Table 7-1 Limit States of Members for Performance Level

Members	Limit States
Bearings	• To resist the greater of the designated horizontal design load effects or 10% of the vertical load.
Piers	• Formation of plastic hinges are allowed without bridge collapse
Foundation	• Basically kept at the elastic range.
Footings	• Basically kept at elastic range.
Abutments	• Basically kept at elastic range. • To protect the abutment piles from failure, the backwall is designed to shear off when subjected to the design seismic lateral force.
Superstructure	• Basically kept at elastic range.

<table>
<tr><th>Items</th><th>JRA (Part V; English Version, 2002)</th><th>NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP</th></tr>
<tr><td></td><td>

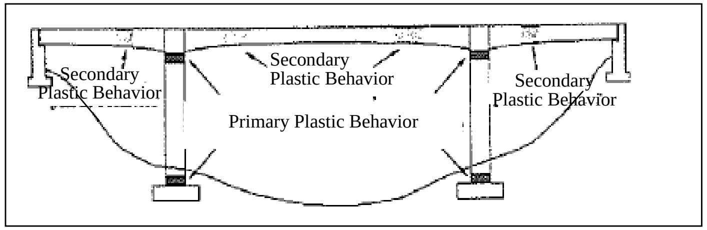


(b) Example D: Seismic Isolation Bearing with Consideration of Non-Linearity(in longitudinal direction)



(c) Example A: Single Column Pier with Plastic Behavior (in Transverse direction)

(d) Example C: Foundations with Plastic Behavior (Pier Wall, in Transverse direction)



(e) Example B: Plasticity in Piers and Superstructures (Rigid-Frame Bridges in Transverse direction)

Fig. 7-1 Limit States of Each Member with Applicable Examples for Seismic Performance Level 2 and Level 3

</td><td></td></tr>
<tr><td>8. Design Method Applicable for Seismic Performance Verification</td><td>

The bridge types and design methods applicable to seismic performance verification is summarized in Table 3.

- Although dynamic analysis methods can be applied to bridges without complicated seismic behavior, it is recommended to use static analysis methods because the verification in accordance with static method is generally feasible for these bridges.
- Since the seismic behavior of bridges with predominant first mode of vibration and plural plastic behavior or bridges in which investigation on application of Energy Conservation Principle remains unclear may become complicated due to plasticity of members, their Seismic Performance Level 1should be verified by the static analysis methods but Seismic Performance Level 2 or Level 3 be verified by dynamic methods.

</td><td>

(1) Two analysis procedures are recommended:

Procedure 1. Single-Mode Spectral Method
Procedure 2. Multimode Spectral Method

(2) In both methods, all fixed column, pier or abutment supports are assumed to have the same ground motion at the same instant of time. At movable supports, displacements determined from the analysis which exceeded the minimum requirements shall be used in the design without reduction.

(3) <u>Mathematical Model</u>. The bridge should be modeled as a three-dimensional space frame with joints and

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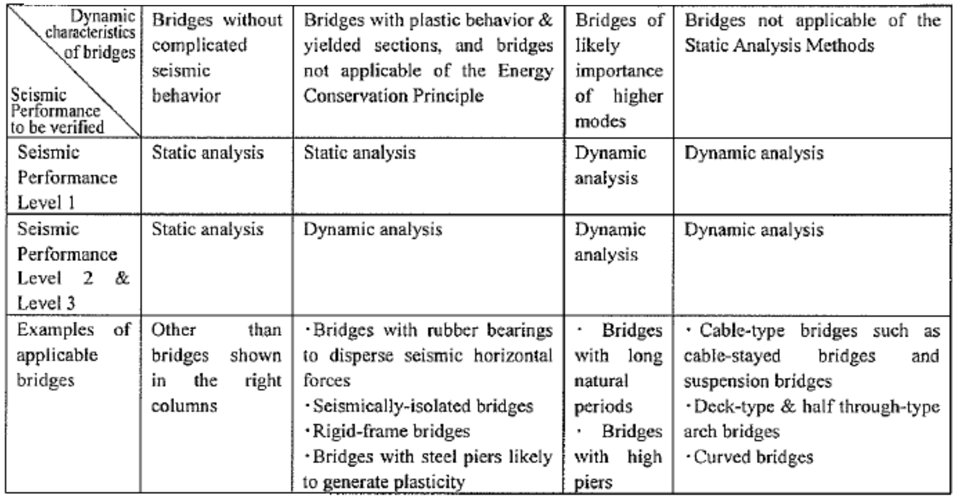
<table>
<tr><th>Items</th><th>JRA (Part V; English Version, 2002)</th><th>NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP</th></tr>
<tr><td></td><td>

Table 8-1 Relationship between Complexities of Seismic Behavior and Design Methods Applicable for Seismic Performance Verification



<table>
<tr><th>Dynamic characteristics of bridges
Seismic Performance to be verified</th><th>Bridges without complicated seismic behavior</th><th>Bridges with plastic behavior & yielded sections, and bridges not applicable of the Energy Conservation Principle</th><th>Bridges of likely importance of higher modes</th><th>Bridges not applicable of the Static Analysis Methods</th></tr>
<tr><td>Seismic Performance Level 1</td><td>Static analysis</td><td>Static analysis</td><td>Dynamic analysis</td><td>Dynamic analysis</td></tr>
<tr><td>Seismic Performance Level 2 & Level 3</td><td>Static analysis</td><td>Dynamic analysis</td><td>Dynamic analysis</td><td>Dynamic analysis</td></tr>
<tr><td>Examples of applicable bridges</td><td>Other than bridges shown in the right columns</td><td>・Bridges with rubber bearings to disperse seismic horizontal forces
・Seismically-isolated bridges
・Rigid-frame bridges
・Bridges with steel piers likely to generate plasticity</td><td>・ Bridges with long natural periods
・ Bridges with high piers</td><td>・Cable-type bridges such as cable-stayed bridges and suspension bridges
・Deck-type & half through-type arch bridges
・Curved bridges</td></tr>
</table>

</td><td>

nodes selected to realistically model the stiffness and inertia effects of the structure. The mass should take into account structural elements and other relevant loads including, but not limited to, pier caps, abutments, columns and footings. Other loads such as live loads may be included (Generally, the inertia effects of live loads are not included in the analysis; however, the design of bridges having high live to dead load ratios located in metropolitan areas where traffic congestion is likely to occur should consider the probability of large live load being on the bridge during an earthquake).

(4) Superstructure. The superstructure should, as a minimum, be modeled as a series of space frame members with nodes at such points as the span quarter points in addition to the joints at the ends of each span. The effects of earthquake restrainers at expansion joints may be approximated by superimposing one or more linearly elastic members having the stiffness properties of the engaged restrainer units.

(5) Substructure. The intermediate columns or piers should also be modeled as space frame members. The model should consider the eccentricity of the columns with respect to superstructure. Foundation conditions at the base of the columns and at the abutments may be modeled using equivalent linear spring coefficients.

(6) Mode Shapes and Periods. The required periods and mode shapes of the bridge in the direction under consideration shall be calculated by established methods for the fixed base condition using the mass and elastic stiffness of the entire seismic resisting system. The response should, as a minimum, include the effects of a number of modes equivalent to three times the number of spans up to a maximum of 25 modes.

(7) The member forces and displacement can be estimated by combining the respective response quantities (e.g. force, displacement or relative displacement) from the individual modes by the Square Root of the Sum of the Squares (SRSS) method. For bridges with closely spaced modes (within 10%), other more appropriate methods of combining or weighting the individual contributions should be considered to obtain the total final response.

</td></tr>
<tr><td>9. Calculation of Natural Period</td><td>

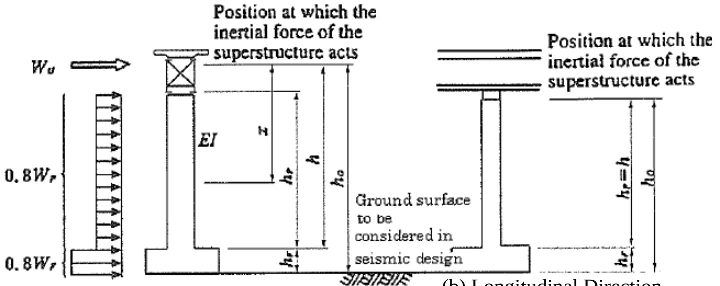
(1) Natural periods shall be appropriately calculated with considering of the effects of deformations of structural members and foundations.

(2) Natural Period of the Design Vibration Unit (s) (T (s))

$T = 2.01 * \sqrt{\delta}$ ------------------ (9-1)

where, δ can be calculated as follows.

(a) in case of a design vibration unit consisting of substructure and its supporting superstructure part as shown in Fig. 9-1



(a) Transverse Direction (b) Longitudinal Direction

Fig. 9-1 Calculation Model of Natural Period for A Design Vibration Unit Consisting of One Substructure and its Supporting Superstructure Part

</td><td>

(1) Natural period calculation by **Single Mode Spectral Method**

The single-mode method of spectral analysis shall be based on the fundamental mode of vibration in either the longitudinal or transverse direction. For regular bridges, the fundamental modes of vibration in the horizontal plane coincide with the longitudinal and transverse axes of the bridge structure. This mode shape may be found by applying a uniform horizontal load to the structure and calculating the corresponding deformed shape. The natural period may be calculated by equating the maximum potential and kinetic energies associated with the fundamental mode shape.

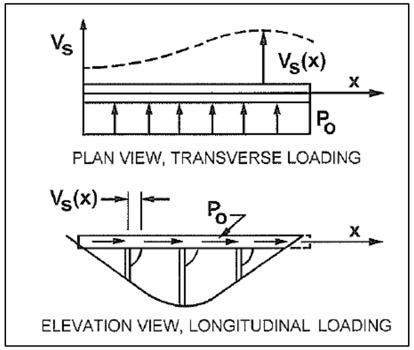


Figure 9-1 Bridge Deck Subjected to Assumed Transverse and Longitudinal Loading

- Calculate the static displacement $v_s(x)$ due to an assumed uniform loading p_o as shown in Figure 9-1.
- Calculate factors a and g as:

</td></tr>
</table>

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP

JRA (Part V; English Version, 2002)

$$\delta = \delta_P + \delta_0 + \theta_0 h_0 \quad \text{(9-2)}$$

where

δ_P: Bending deformation of substructure body (m)

δ_0: Lateral displacement of foundation (m)

θ_0: Rotation angle of foundation (rad)

h_0: Height from the ground surface to be considered in seismic design to the height of superstructural inertia force (m)

When the body of the substructure has a uniform section, the bending deformation δ_P can be calculated by Eq (9-3).

$$\delta_P = \frac{W_U h^3}{3EI} + \frac{0.8\ W_P h_P{}^3}{8EI} \quad \text{(9-3)}$$

where

W_U: Weight of the superstructure portion supported by the substructure body concerned (kN)

W_p : Weight of the substructure body (kN)

EI: Bending stiffness of the substructure body specified in the explanations of (1) above ($kN \cdot m^2$).

h: Height from the bottom of the substructure body to the height of the superstructural inertia force (m)

h_P: height of the substructure body (m)

Where, δ0 and θ0 are calculated from Eq.(9-4) (Refer to Fig.9-2).

$$\left.\begin{aligned} \delta_0 &= \frac{H_0 A_{rr} - M_0 A_{sr}}{A_{ss} A_{rr} - A_{sr} A_{rs}} \\ \theta_0 &= \frac{-H_0 A_{rs} + M_0 A_{ss}}{A_{ss} A_{rr} - A_{sr} A_{rs}} \end{aligned}\right\} \text{---------(9-4).}$$

$$\left.\begin{aligned} H_0 &= W_U + 0.8(W_P + W_F) \\ M_0 &= W_U h_0 + 0.8 W_P\left(\frac{h_P}{2} + h_F\right) + 0.8 W_F \frac{h_F}{2} \end{aligned}\right\}$$

Where, δ0 and θ0 are calculated from Eq. (9-4) (Refer to Fig.9-2). Arr, Asr, Ars and Ass are spring constants (kN/m, kN/rad, kNm/m, kNm/rad) and calculated from the following formula according to the foundation types.

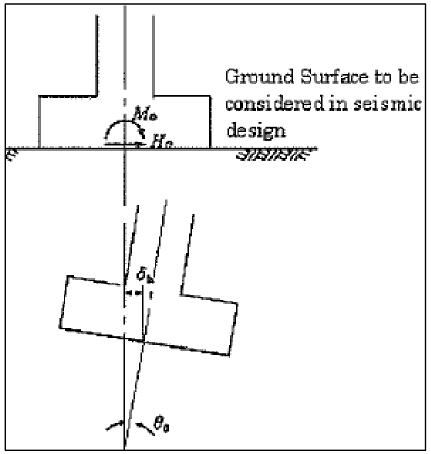


Fig. 9-2 Load and Displacement at Ground Surface for Seismic Design

NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP

$$\alpha = \int v_s(x)dx \quad \text{(9-1)}$$

$$\gamma = \int w(x)v_s^2(x)dx \quad \text{(9-2)}$$

where:

p_o = a uniform load arbitrarily set equal to 1.0 (N/mm)

$v_s(x)$ = deformation corresponding to p_o (mm)

$w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure (N/mm)

- Calculate the period of the bridge as:

$$T_m = 2\pi \sqrt{\frac{\gamma}{p_o g \alpha}} \quad \text{(9-3)}$$

where: g = acceleration of gravity (m/sec^2)

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP

① Spread foundation

$$\left.\begin{aligned} A_{ss} &= k_{SB} A_B \\ A_{sr} &= A_{rs} = 0 \\ A_{rr} &= k_V I_B \end{aligned}\right\} \text{------} \quad (9\text{-}5)$$

② Pile foundation with only vertical piles arranged symmetrically

$$\left.\begin{aligned} A_{ss} &= nK_1 \\ A_{sr} &= A_{rs} = -nK_2 \\ A_{rr} &= nK_4 + K_{VP} \sum_{i=1}^{n} y_i^2 \end{aligned}\right\} \text{---------} \quad (9\text{-}6)$$

where

k_{SB} : Coefficient of shear subgrade reaction in horizontal direction at the bottom of foundation (kN/m^3)

k_V : Coefficient of subgrade reaction in vertical direction at the bottom of foundation (kN/m^3)

n : Total number of piles

y_i : y coordinate of the pile head of i-th pile (m)

K_1, K_2 K_3 K_4: Spring constants (kN/m, kN/rad, kN·m/m, kN·m/rad) of the piles in the perpendicular direction to the pile axis in case of rigid connection at pile head

K_{VP}: Spring constant (kN/m) of the piles in the direction of the pile axis

Concrete calculation methods of K1, K2, K3, K4 and Kvp are provided in Part IV. With the coefficients of subgrade reaction for seismic design as shown in Eq. (9-7) and (9-8), K1, K2, K3, K4 and Kvp .can be obtained.

$K_{H0} = 1/0.3 * E_D$ ---------- (9-7) $K_{V0} = 1/0.3 * E_D$ ---------- (9-8)

where,

K_{H0}, K_{V0} = Reference values of the coefficient of subgrade reaction in horizontal direction and in vertical direction, respectively (kN/m^3) (for Level 1 and 2 EGMs)

E_D = 2 * (1 + ν_D) * G_D (E_D: Dynamic modulus of deformation of the ground (kN/m^2))

ν_D = Dynamic Poisson’s ratio of the ground

G_D = $\gamma t/g * V^2_{SD}$ (G_D: Dynamic shear deformation modulus of the ground (kN/m^2))

γt = Unit weight of the ground (kN/m^3)

g = Acceleration of gravity (9.8 m/s^2)

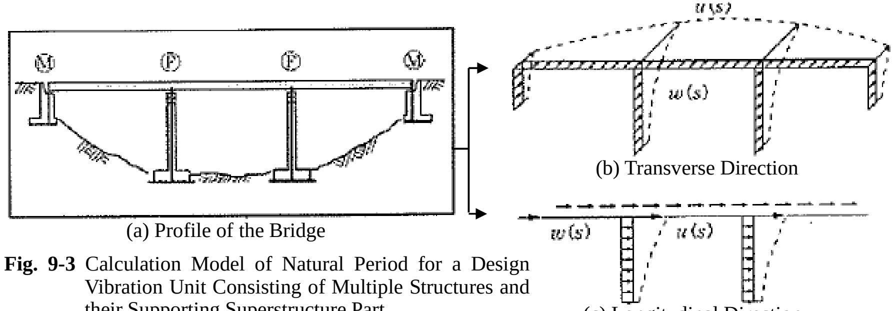
V_{SD} = Shear elastic wave velocity of the ground (m/s)

V_{SDi} = CV * Vsi (V_{SDi}: the average shear elastic wave velocity of the i-th layer)

C_V = 0.8 (Vsi < 300 m/s), 1.0 (Vsi ≥ 300 m/s) (CV: Modification factor based on degree of ground strain)

Vsi = the average shear elastic wave velocity of thei-th soil layer described in Item 5 (m/s)

<u>(b) in case of a design vibration unit consisting of multiple substructures and their supporting superstructure part</u> as shown in Fig. 9-3.



(b) Transverse Direction

(a) Profile of the Bridge

(c) Longitudinal Direction

Fig. 9-3 Calculation Model of Natural Period for a Design Vibration Unit Consisting of Multiple Structures and their Supporting Superstructure Part

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP

$$\delta = \frac{\int w(s)\ u(s)^2\, ds}{\int w(s)\ u(s)\, ds} \quad \cdots\cdots\cdots\cdots \quad (9\text{-}9)$$

where,

$w(s)$: Weight of the superstructure or the substructure at position s (kN/m)

$u(s)$: Lateral displacement of each structure at position s in the direction of the inertia force when a lateral force corresponding to the weight of the superstructure and that of the substructure above the design ground surface act in the direction of the inertia force (m)

When a bridge is modeled into a discrete skeleton structure, the δ can be obtained from Eq. (9-10).

$$\delta = \frac{\sum_i (W_i u_i^2)}{\sum_i (W_i u_i)} \quad \cdots\cdots\cdots\cdots \quad (9\text{-}10)$$

where

W_i: Weight of superstructure and substructure at node i (kN)

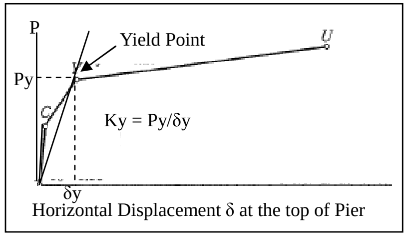
u_i: Displacement of node i occurred in the acting direction of inertia force when a force corresponding to the weight of the superstructure and the substructure above the ground surface to be considered in seismic design is acted in the acting direction of inertia force(m)

∑ represents the sum of all design vibration units.

When calculated with eigenvalue analysis, .the natural period (T) can be obtained directly.

(c) Stiffness Applied to Calculation of Natural Period for Level 2 EGM

During verification of seismic performance for Level 2 EGM, the natural period shall be calculated based on the yield stiffness. The yield stiffness refers to the secant stiffness Ky at yield point due to bending deformation of the pier and is obtained at the ratio of the yield strength Py to the yield displacement δy of the pier (Ky = Py/δy) as shown in Fig. 9-4.



⟹

Ky = 3*E*Iy / h^3 ⟹ Iy = Ky * h^3 / 3E

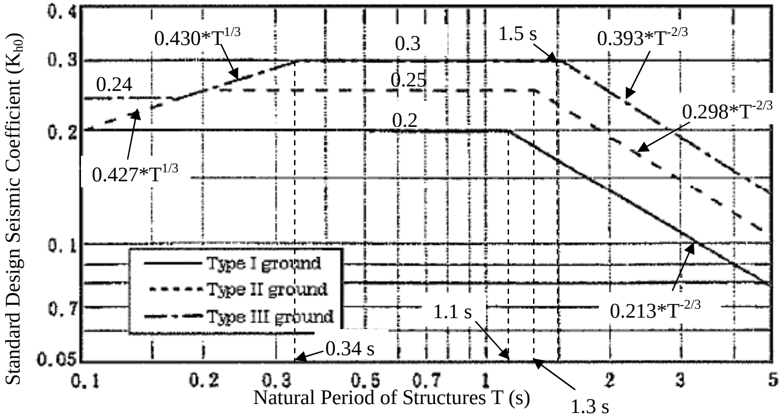
EIy = yield stiffness

E = elastic modulus of pier, Iy = moment of section inertia at the yield point

h = the height of pier

(Iy should be used for every calculation of the natural period for verification of seismic performance for Level 2 EGM)

Fig. 9-4 Stiffness Applied to Calculation of Natural Period for Level 2 EGM

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP
10. Design Horizontal Seismic Coefficient for Level 1 EGM	(1) Design horizontal coefficient (k_h) for Level 1 EGM shall be calculated by Eq. (10-1). $K_h = C_Z * K_{h0}$ (≥ 0.1) -- (10-1) where, K_{h0} = Standard value of the design horizontal seismic coefficient for Level 1 EGM, which is shown in Fig 10-1.(Fig. 10-1 is obtained from the Figure for Level 1 EGM shown in Item 4 by dividing S_0 by gravity acceleration) C_Z = Modification factor for zones shown in Item 4. (2) Design horizontal coefficient (K_{hg}) at ground level can be obtained from Eq. (10-2), which is used for calculation of inertia force due to soil weight and seismic earth pressure in verifying seismic performance for Level 1 EGM. $K_{hg} = C_Z * K_{hg0}$ -- (10-2) where, K_{hg0} = Standard value of the design horizontal seismic coefficient at ground surface level for Level 1 EGM = 0.16 for Ground Type I, 0.2 for Ground Type II and 0.24 for Ground Type III (3) Though a single value of the design horizontal seismic coefficient shall generally be adopted within the same design vibration unit, different design horizontal seismic coefficient for each pier are given in case that the ground type changes within the same design vibration unit.  **Fig. 10-1**Standard Design Seismic Coefficient for Level 1 EGM	No provision
11. Design Horizontal Seismic Coefficient for Level 2 EGM	(1) Design horizontal seismic coefficients for Level 2 EGM (Type I and II) shall be calculated by Eq. (11-1) and (11-2). $K_{hCI} = C_S * C_Z * K_{hC0I} \geq 0.3 * C_S$ or $0.4 * C_Z$ -- (11-1) $K_{hCII} = C_S * C_Z * K_{hC0II} \geq 0.6 * C_S$ or $0.4 * C_Z$ -- (11-2) K_{hCI} and K_{hCII} = Design horizontal seismic coefficient for Type I and II of Level 2 EGM, respectively. K_{hC0I} and K_{hC0II} = Standard design horizontal seismic coefficients for Type I and Type II of Level 2 EGM, respectively, which are shown in Fig 11-1 and Fig. 11-12. C_S = Force Reduction Factor related to the extent of ductility of a pier, which is specified in Item 12. (2) Design horizontal coefficients at ground surface level can be obtained from Eq. (11-3) and (11-4) for Type I and II of Level 2 EGM $K_{hgI} = C_Z * K_{hgI0}$ -- (11-3)	(1) Equivalent static earthquake loading by **Single-Mode Spectral Method** The equivalent static earthquake loading $p_e(x)$ is calculated as: $p_e(x) = \frac{\beta C_s}{\gamma} w(x)v_s(x)$ (11-1) $\gamma = \int w(x)v_s^2(x)dx$ (11-2) $\beta = \int w(x)v_s(x)dx$ (11-3) where: $p_e(x)$ = the intensity of the equivalent static seismic loading applied to represent the primary mode of

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP

$K_{hgII} = C_Z * K_{hgII0}$ -- (11-4)

KhgI and KhgII = Design horizontal seismic coefficients at ground surface for Type I and II of Level 2 EGM, respectively.

K_{hgI0} and K_{hgII0} = Standard horizontal seismic coefficient at ground surface level for Type I and II of Level 2 EGM, respectively.

K_{hgI0} = 0.3 for Ground Type I, 0.35 for Ground Type II and 0.40 for Ground Type III

K_{hgII0} = 0.80 for Ground Type I, 0.70 for Ground Type II and 0.60 for Ground Type III

(3) The highest value of design horizontal seismic coefficient shall generally be used in each design vibration unit.

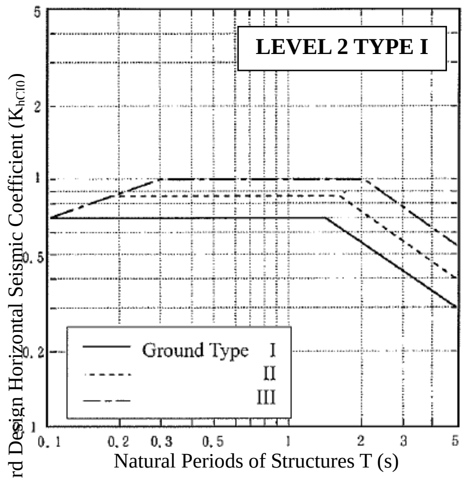


Fig. 11- 1 Relationship between K_{hCI0} and T (s)

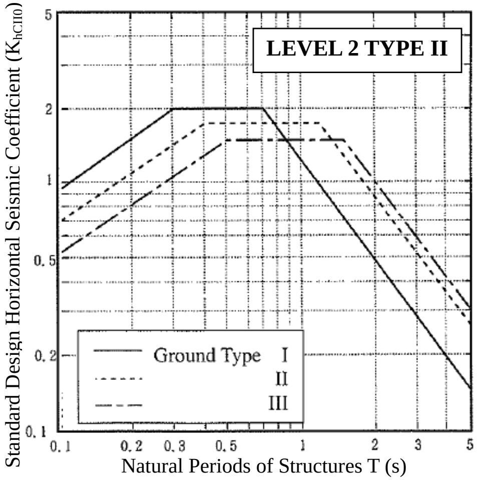


Fig. 11- 2 Relationship between K_{hCII0} and T (s)

Table 11-1 Relationship between K_{hCI0} and T (s)

Ground Type	Values of k_{hc0} in Terms of Natural Period T (s)		
Type I	$T \leqq 1.4$ $k_{hc0} = 0.7$		$1.4<T$ $k_{hc0} = 0.876T^{-2/3}$
Type II	$T<0.18$ $k_{hc0}=1.51T^{1/3}$ but $K_{hc0} \geqq 0.7$	$0.18 \leqq T \leqq 1.6$ $k_{hc0} = 0.85$	$1.6<T$ $k_{hc0}=1.16T^{-2/3}$
Type III	$T<0.29$ $k_{hc0}=1.51T^{1/3}$ but $k_{hc0} \geqq 0.7$	$0.29 \leqq T \leqq 2.0$ $k_{hc0}=1.0$	$2.0<T$ $k_{hc0}=1.59T^{-2/3}$

Table 11-2 Relationship between K_{hCII} and T (s)

Ground Type	Values of k_{hc0} in Terms of Natural Period T (s)		
Type I	$T < 0.3$ $k_{hc0} = 4.46T^{2/3}$	$0.3 \leqq T \leqq 0.7$ $k_{hc0} = 2.0$	$0.7 < T$ $k_{hc0} = 1.24T^{-4/3}$
Type II	$T < 0.4$ $k_{hc0} = 3.22T^{2/3}$	$0.4 \leqq T \leqq 1.2$ $k_{hc0} = 1.75$	$1.2 < T$ $k_{hc0} = 2.23T^{-4/3}$
Type III	$T < 0.5$ $k_{hc0} = 2.38T^{2/3}$	$0.5 \leqq T \leqq 1.5$ $k_{hc0} = 1.50$	$1.5 < T$ $k_{hc0} = 2.57T^{-4/3}$

vibration (N/mm)

$v_s(x)$ = deformation corresponding to p_o (mm)

$w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure (N/mm)

C_s = the dimensionless elastic seismic response coefficient.

(2) Elastic Seismic Response Coefficient:

Procedure 1

The elastic seismic coefficient C_s used to determine the design forces is given by the dimensionless formula:

$$C_s = 1.2AS/T^{2/3} \qquad (11\text{-}4)$$

where:

A = the Acceleration Coefficient based on the Seismic Zone Map

S = the dimensionless coefficient for the profile characteristics of the site given in Table 5-1.

T = period of the bridge

The value C_s need not exceed 2.5A. For Soil Profile Type III soils in areas where A ≥ 0.30, C_s need not exceed 2.0A.

(3) Elastic Seismic Response Spectrum:

Procedure 2

The elastic seismic coefficient for mode “m”, C_{sm}, shall be determined in accordance with the following formula:

$$C_{sm} = 1.2AS/T_m^{2/3} \qquad (11\text{-}5)$$

where:

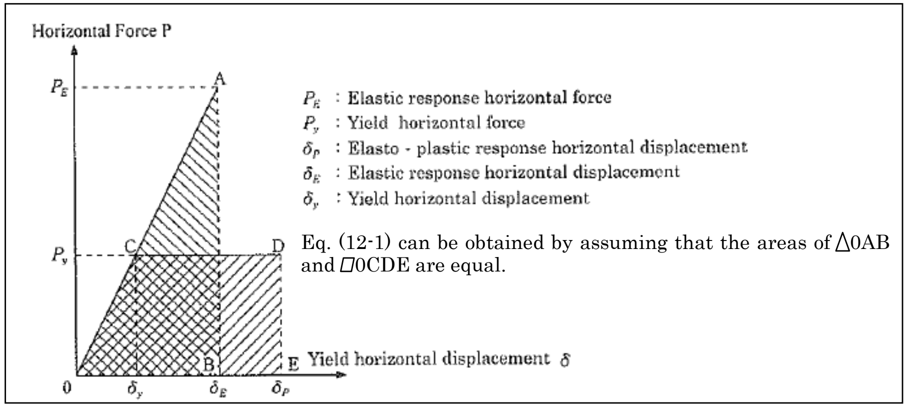
T_m = period of the mth mode of vibration

The value C_{sm} need not exceed 2.5A. For Soil Profile Type III soils in areas where A ≥ 0.30, C_{sm} need not exceed 2.0A.

EXCEPTIONS:

1. For Soil Profile Type III C_{sm} for modes other than the fundamental mode, which have periods less than 0.3 sec. may be determined in accordance with the following formula:

$$C_{sm} = A(0.8+4.0T_m) \qquad (11\text{-}6)$$

<table>
<tr><th>Items</th><th>JRA (Part V; English Version, 2002)</th><th>NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP</th></tr>
<tr><td></td><td></td><td>2. For structures in which any T_m exceeds 4.0 sec., the value of C_{sm} for that mode may be determined in accordance with the following formula:
$C_{sm} = 3AS/T_m^{4/3}$ (11-6)</td></tr>
<tr><td>12. Force Reduction Factor</td><td>(1) Force reduction factor shall be calculated by Eq. (12-1) for a structural system that can be modeled as a one degree-of freedom vibration system having a plastic force- displacement relation.
$C_S = 1/\sqrt{2*\mu_a - 1}$ --- (12-1)
C_S = Force reduction factor (refer to Fig. 12-1)
μ_a = Allowable ductility ratio. μa can be obtained by Eq. (12-2) for the case of a RC column.
$\mu_a = 1 + (\delta u - \delta y)/\alpha*\delta y$ (refer to Fig. 12-2) --(12-2)


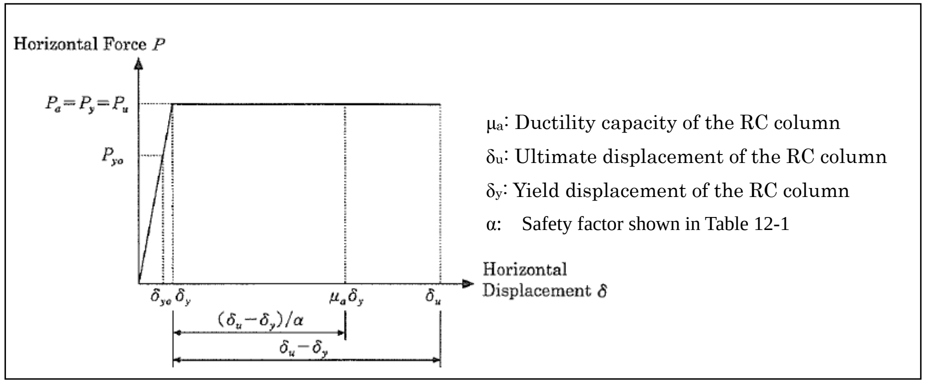
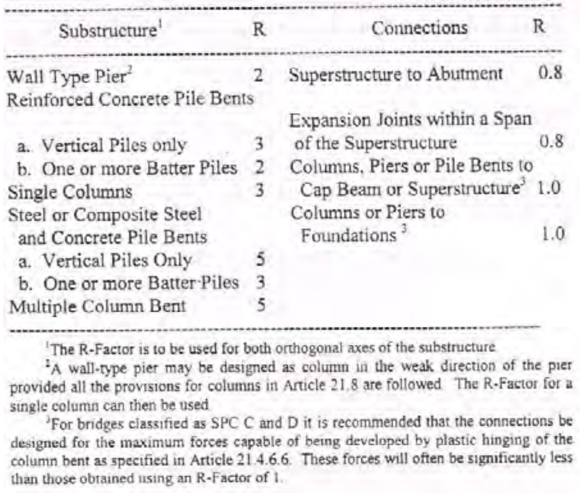
Eq. (12-1) can be obtained by assuming that the areas of △0AB and □0CDE are equal.
Fig. 12-1 Elasto-Plastic Response Displacement of a Pier


Fig. 12-2 Simplified Relationship between Lateral Strength and Ductility Capacity for Flexural Failure</td><td>(1) Seismic design forces for individual members and connections of bridges classified as SPC C and D are determined by dividing the elastic forces by the appropriate Response Modification Factor (R).
(2) The Response Modification Factors for various components are given in Table 12-1.
Table 12-1 Response Modification Factor (R)

<table><tr><th>Substructure[1]</th><th>R</th><th>Connections</th><th>R</th></tr><tr><td>Wall Type Pier[2]</td><td>2</td><td>Superstructure to Abutment</td><td>0.8</td></tr><tr><td>Reinforced Concrete Pile Bents</td><td></td><td>Expansion Joints within a Span</td><td></td></tr><tr><td>a. Vertical Piles only</td><td>3</td><td>of the Superstructure</td><td>0.8</td></tr><tr><td>b. One or more Batter Piles</td><td>2</td><td>Columns, Piers or Pile Bents to</td><td></td></tr><tr><td>Single Columns</td><td>3</td><td>Cap Beam or Superstructure[3]</td><td>1.0</td></tr><tr><td>Steel or Composite Steel</td><td></td><td>Columns or Piers to</td><td></td></tr><tr><td>and Concrete Pile Bents</td><td></td><td>Foundations [3]</td><td>1.0</td></tr><tr><td>a. Vertical Piles Only</td><td>5</td><td></td><td></td></tr><tr><td>b. One or more Batter Piles</td><td>3</td><td></td><td></td></tr><tr><td>Multiple Column Bent</td><td>5</td><td></td><td></td></tr></table>
[1] The R-Factor is to be used for both orthogonal axes of the substructure
[2] A wall-type pier may be designed as column in the weak direction of the pier provided all the provisions for columns in Article 21.8 are followed The R-Factor for a single column can then be used
[3] For bridges classified as SPC C and D it is recommended that the connections be designed for the maximum forces capable of being developed by plastic hinging of the column bent as specified in Article 21.4.6.6. These forces will often be significantly less than those obtained using an R-Factor of 1.
(3) The force resulting from plastic hinging at the top and/or bottom of the column shall be calculated after the preliminary design of the column is complete. The forces resulting from plastic hinging are recommended for determining design forces for most components.
(4) The recommended connection design forces between the superstructure and columns, columns and cap beams, and columns and spread footings or pile caps are the forces developed at the top and bottom of the columns due to plastic hinging.
(5) The design forces for foundations including footings, pile caps and piles may be either those forces determined from elastic seismic forces divided by the corresponding response modification factors or the forces at the bottom of the columns corresponding to column hinging, whichever values are smaller.</td></tr>
</table>

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2^{nd} Ed., 1997 (Reprint Ed. 2005) - ASEP

JRA (Part V; English Version, 2002)

Table 12-1 Safety Factor of RC Column resulting in Flexural Failure

Seismic Performance to be Verified	Safety Factor α in Calculation of Ductility Capacity for Type I Earthquake Ground Motion	Safety Factor α in Calculation of Ductility Capacity for Type II Earthquake Ground Motion
Seismic Performance Level 2	3.0	1.5
Seismic Performance Level 3	2.4	1.2

Items

13. Evaluation of Failure Mode of RC Column

JRA (Part V; English Version, 2002)

(1) Failure mode of a RC column shall be evaluated by Eq. (13-1)

$$\left.\begin{array}{lll} P_u \leqq P_s & : & \text{Flexural (or bending) failure} \\ P_s < P_u \leqq P_{s0} & : & \text{Shear failure after flexural yielding} \\ P_{s0} < P_u & : & \text{Shear failure} \end{array}\right\} \cdots\cdots\cdots\cdots \quad (13\text{-}1)$$

Pu = Lateral strength of a RC column
Ps = Shear strength of a RC column
Ps0 = Shear strength of a RC column calculated by the modification factor on the effects of repeated alternative loads is equal to 1.0.

(2) Failure mode for a RC column can be judged following the flow shown in Fig. 13-1.

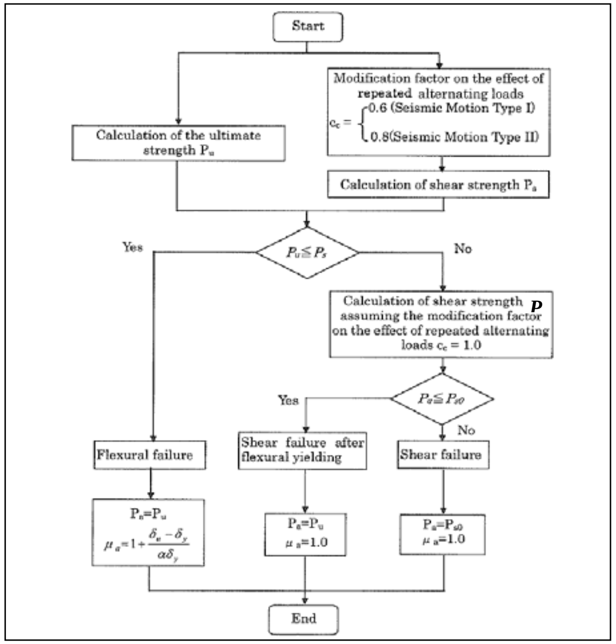


Fig. 13-1 Evaluation of Failure Mode for a RC Column

NSCP Vol. II Bridges ASD (Allowable Stress Design), 2^{nd} Ed., 1997 (Reprint Ed. 2005) - ASEP

(1) Since the Standards utilize the force-base approach method, the column failure mode is not investigated but rather for the preliminary design, the initial column seismic design forces are determined by the elastic seismic design forces divided by the response modification factor. Appropriate column sections and reinforcements are then decided based on the demand forces.

(2) Once the preliminary design of the column is complete, the forces resulting from plastic hinging are calculated and used to determine the design forces for most components.

(3) Shear Failure

The shear mode of failure in a column or pile bent will probably result in a partial or total collapse of the bridge; therefore, the design shear force must be calculated conservatively. In calculating the column or pile bent shear force, consideration must be given to the potential locations of plastic hinges – such that the smallest potential column length be used with the plastic moments to calculate the largest potential shear force for design.

(4) Lateral Strength

The lateral strength of the pier corresponds to the total of inelastic hinging shear force demands at the top and bottom of the pier column/s formed by the column overstrength moment resistance (taken as the plastic moment).

(5) The lateral force demand from the overstrength plastic moments is calculated and corresponds to the column shear force. The procedure for column shear demand force is calculated for two types of piers – (a) single columns and piers and (b) bents with two or more columns. Table 13-1 presents the design forces corresponding to both pier types.

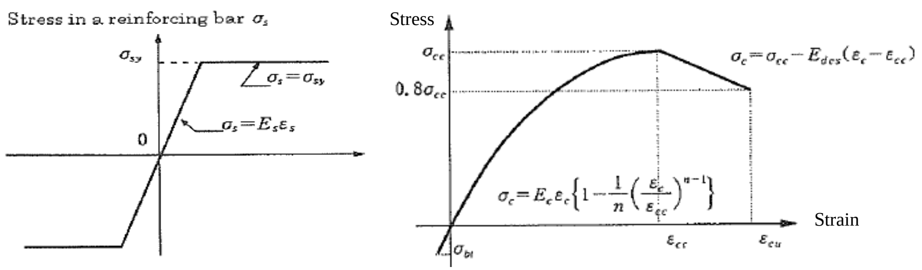
Table 13-1 Design Forces

Design Forces	Single Columns and Piers	Bents with Two or More Columns
Axial	• Unreduced maximum and minimum seismic axial loads plus dead load	• The maximum and minimum axial load is the dead load, plus, or minus, the axial load determined from the final iteration due to overturning when applying the calculated bent shear force to the top of the bent (center of mass of the superstructure) • The column overstrength plastic moment is determined based on the calculated column axial forces.
Moments	• Column overstrength plastic moment capacity using strength reduction factor (ϕ) of 1.3 for concrete and 1.25 for steel	• The column overstrength plastic moments corresponding to the maximum compressive axial load calculated above, multiplied by the strength reduction

<table>
<tr><th>Items</th><th>JRA (Part V; English Version, 2002)</th><th>NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP</th></tr>
<tr><td></td><td></td><td>
<table>
<tr><td></td><td>and the maximum elastic column axial load.</td><td>factor (ϕ) of 1.3 for concrete and 1.25 for steel.</td></tr>
<tr><td>Shear</td><td>• Corresponding shear force using the column overstrength plastic moments and the appropriate column height.</td><td>• The shear force corresponding to the column overstrength plastic moments above.</td></tr>
</table>
</td></tr>
<tr><td>14. Calculation of Lateral Strength and Displacement of a RC Column</td><td>

(1) Relationships between stress and strain of a reinforcing bar and concrete are shown in Fig. 14-1 (1) and Fig. 14-1 (2), respectively.

(2) RC column is divided into m segments along its height and the section of each segment is divided into n elements in the acting direction of the inertia force as shown in Fig. 14-2. With these relationships, Pu, Py, δu and δy at the height of the superstructure inertia force shown in Fig.14-3 can be obtained.



(1) Stress and Strain of Reinforcing Bar (2) Stress and Strain of Concrete

Fig 14-1 Relationships between Stress and Strain of Reinforcing Bar and Concrete

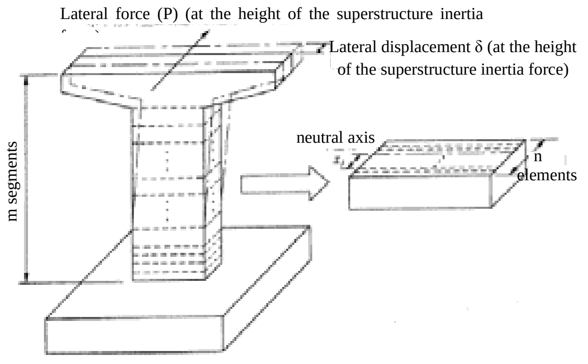


Fig 14-2 Lateral Force (P) at the Acting Position of the Inertia Force and Displacement (δ), and Division of Column

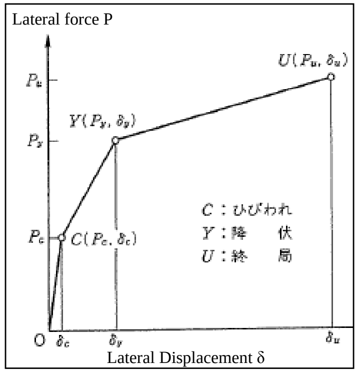


Fig 14-3 Calculated Relationship between Lateral Force (P) and Displacement (δ)

(3) Descriptions

P_C = Lateral strength at cracking, P_y = Yielding lateral strength, P_u = Lateral strength, δ_y = Yield displacement, δu = Ultimate displacement (a single -column RC column)

</td><td>

(1) Reinforcing steel is modeled with a stress-strain relationship that exhibits an initial elastic portion, a yield plateau, and a strain-hardening range in which the stress increases with strain as shown in Figure 14-1. On the other hand, the stress-strain model for confined and unconfined concrete is used to determine section response as shown in Figure 14-2.

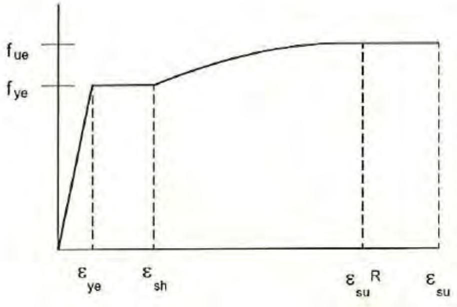


Figure 14-1 Reinforcing Steel Stress-Strain Model

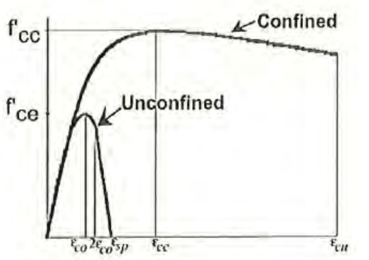
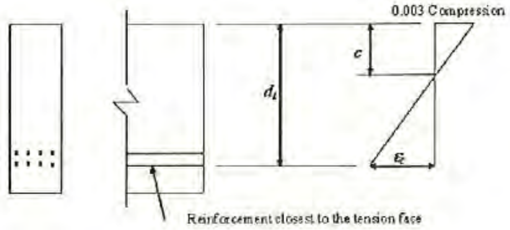


Figure 14-2 Concrete Stress-Strain Model



The net tensile strain in the extreme tension steel is determined from a linear strain distribution at nominal strength using similar triangles.

Figure 14-3 Strain Distribution and Net Tensile Strain

(2) The design based on the expected behavior of the bridge system which is the ductile substructure (columns) with essentially elastic superstructure and foundation. This system includes conventional plastic hinging in columns and walls and abutments that limits inertial forces by full mobilization of passive soil resistance.

(3) Lateral Strength

The lateral strength of the pier corresponds to the total of inelastic hinging shear force demands at the top and bottom of the pier column/s formed by the column overstrength moment resistance (taken as the plastic moment, Figure 14-4).

</td></tr>
</table>

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP

(a) Premises

- Fiber strain is proportional to the distance from the neutral axis.
- Skeleton curve between horizontal force and horizontal displacement shall be expressed by an ideal elasto-plastic model shown in Fig. 14-4.

(b) Equations

$$P_c = \frac{W}{h}(\sigma_{bt} + \frac{N}{A}) \quad \text{------------------------} \ (14\text{-}1)$$

$$P_y = \frac{M_u}{h} \quad \text{------------------------} \ (14\text{-}2)$$

$$\delta_y = \frac{M_u}{M_{y0}}\delta_{y0} \quad \text{------------------------} \ (14\text{-}3)$$

$$P_u = \frac{M_u}{h} \quad \text{------------------------} \ (14\text{-}4)$$

$$\delta_u = \delta_y + (\phi_u - \phi_y) L_p (h - L_p / 2) \quad \text{--------} \ (14\text{-}5)$$

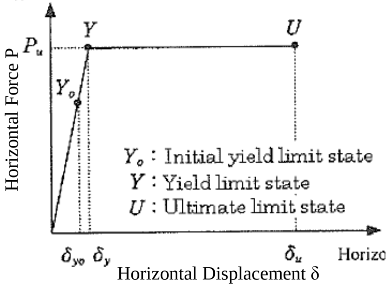


Fig.14-4 Ideal Elasto-Plastic Model

(c) Description of Symbols

W : Section modulus of a column with consideration of axial reinforcement at the column's bottom section (mm^3)

σ_{bt} : Flexural tensile strength of concrete (N/mm^2) to be calculated by Eq. (14-1-1)

$$\sigma_{bt} = 0.23 \ \sigma_{ck}^{2/3} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \ (14\text{-}1\text{-}1)$$

N : Axial force acting on the column's bottom section (N)

A : Sectional area of a column, with consideration of axial reinforcement at the column's bottom section (mm^2)

h : Height of superstructural inertial force from the bottom of column. (mm)

σ_{ck} : Design strength of concrete (N/mm^2)

δ_{y0} : Horizontal displacement at the time of yielding of axial tensile reinforcing bars at the outermost edge of the column's bottom section (called "initial yield displacement" hereafter) (mm)

M_u : Ultimate bending moment at the column's bottom section (N· mm)

M_{y0}: Bending moment at the time of yielding of axial tensile reinforcing bars at the outermost edge of the column's bottom section (N· mm)

H : Height of superstructural inertial force from the bottom of column. (mm)

L_p: Plastic hinge length (mm) calculated by Eq. (14-5-1)

$$L_p = 0.2h - 0.1\ D \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \ (14\text{-}5\text{-}1)$$

in which $0.1D \leq L_p \leq 0.5D$

D : Sectional depth (mm) (D shall be the diameter of a circular section, or the length of a rectangular section in the analytical direction)

ϕ_y: Yield curvature at the column's bottom section (1/mm)

ϕ_u : Ultimate curvature at the column's bottom section (1/mm)

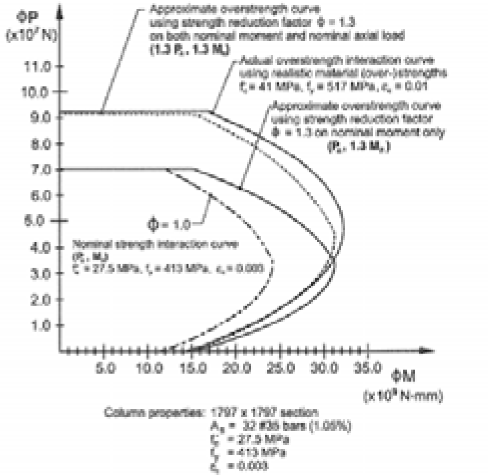


Figure 14-4 Development of Approximate Overstrength Curves from Nominal Strength Curves after Gajer and Wagh (1994)

(6) Column and Pier Flexural Strength Requirements

In the Standards, a vertical support is considered to be a column if the ration of the clear height to maximum plan dimensions of the supports is equal to or greater than 2.5. Note that the maximum plan dimension is taken at the maximum section of the flare for a flared column. For supports with a ratio less than 2.5, the provisions of the piers will apply. Note that a pier may be designed as a pier in the strong direction and column in the weak direction.

(a) Column Requirements

The biaxial strength of columns shall not be less than that required for the design moments determined in Section 13. The design of column shall be checked for both minimum and maximum axial loads. The strength reduction factors shall be replaced for both spirally and tied reinforced columns by the value of 0.50 when the stress due to maximum axial load for the column exceed $0.20f'_c$. The value ϕ may be increased linearly from 0.5 to the value for flexure (0.90) when the stress due to the maximum axial load is between $0.20f'_c$ and 0. Moment magnification and slenderness effects shall be considered in the design of columns.

(b) Pier Requirements

The minimum reinforcement ratio both horizontally, ρ_h, and vertically, ρ_n, in any pier shall not be less than 0.0025. Reinforcement (horizontally and vertically) shall be distributed uniformly.

ρ_h = the ratio of horizontal shear reinforcement area to gross concrete area of a vertical section.

ρ_n = the ratio of vertical shear reinforcement area to the gross concrete area of a horizontal section.

Items	**JRA (Part V; English Version, 2002)**	**NSCP Vol. II Bridges ASD (Allowable Stress Design), 2^{nd} Ed., 1997 (Reprint Ed. 2005) - ASEP**
	(d) Detailed Procedure to Obtain Mechanical Values Referring to Fig. 14-2, Fig. 14-5 and Fig. 14-6 $M_c = W_i(\sigma_{bt} + N_i/A_i)$ ·· (14-6) $\phi_c = M_c/E_c I_i$ ·· (14-7) M_c: Bending moment at cracking (N·mm) ϕ_c: Curvature of cracking (1/mm) W_i: Sectional modulus of pier having considered the axial reinforcement in the i-th section from the height of the superstructural inertia force (mm^3). σ_{bt}: Bending tensile strength of concrete (N/mm^2) to be calculated by Eq. (14-1-1) Ni: Axial force due to the weights of superstructure and substructure acting on the i-th section from the height of the superstructural inertia force (N). A_i: Sectional area of pier having considered the axial reinforcement in the i-th section from the height of the superstructural inertia force (mm^2) E_c: Young's modulus of concrete (N/mm^2) I_i: Moment of inertia of pier having considered the axial reinforcement in the i-th section from height of the superstructural inertia force (mm^4) σ_{ck} : Design standard strength of concrete (N/mm^2) $M_i = \sum_{j=1}^{n} \sigma_{cj} x_j \Delta A_{cj} + \sum_{j=1}^{n} \sigma_{sj} x_j \Delta A_{sj}$ --- (14-8) $\phi_i = \varepsilon_{c0}/x_0$ --- (14-9) σ_{cj}, σ_{sj}: Stress of concrete and reinforcement within the j-th infinitesimal part (N/mm^2) ΔA_{cj}, ΔA_{sj}: Sectional area of concrete and reinforcement within the j-th infinitesimal part (mm^2) M_i: Bending moment acting on the i-th section from the height of the superstructural inertia force (N·mm) ϕ_i: Curvature of the i-th section from the height of the superstructural inertia force (1/mm) x_j: Distance from concrete or reinforcement in the j-th infinitesimal part to the centroid position of section (mm) ε_{c0}: Compressed edge strain of concrete x_0: Distance from the compressed edge of concrete to the neutral axis (mm) $\delta_{y0} = \int \phi y \, dy$ $\fallingdotseq \sum_{i=1}^{m} (\phi_i y_i + \phi_{i-1} y_{i-1}) \Delta y_i / 2$ --- (14-10) $\phi_y = \left(\frac{M_u}{M_{y0}}\right) \phi_{y0}$ --- (14-11)	

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP

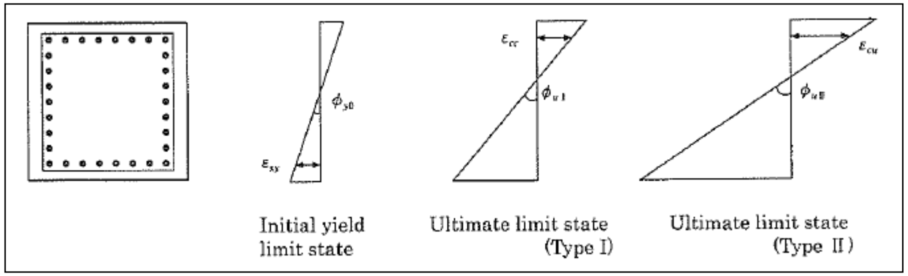


Fig 14-5 Strain Distribution within Initial Yielding and Ultimate Limit

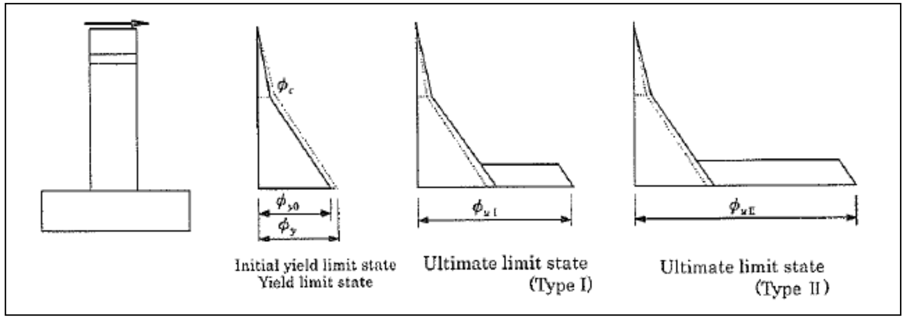


Fig 14-6 Curvature Distribution in the Direction of Height

(e) Stress-Strain Relation of Concrete (refer to Fig. 14-1(2))

The stress-strain curve of concrete shall be determined by Eq. (14-12) based on Fig. 14-1(2)

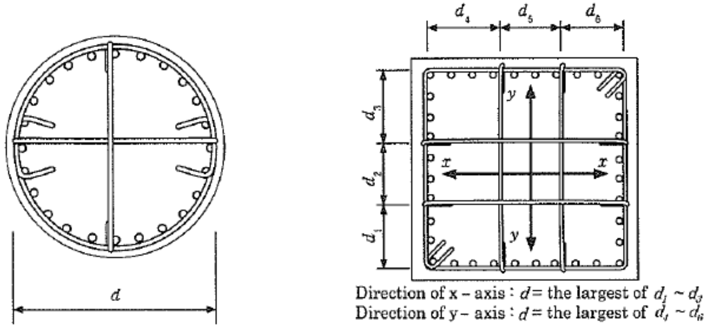
$$\sigma_c = \begin{cases} E_c\varepsilon_c\left\{1-\dfrac{1}{n}\left(\dfrac{\varepsilon_c}{\varepsilon_{cc}}\right)^{n-1}\right\} & (0 \leqq \varepsilon_c \leqq \varepsilon_{cc}) \\ \sigma_{cc} - E_{des}(\varepsilon_c - \varepsilon_{cc}) & (\varepsilon_{cc} < \varepsilon_c \leqq \varepsilon_{cu}) \end{cases} \qquad \text{(14-12)}$$

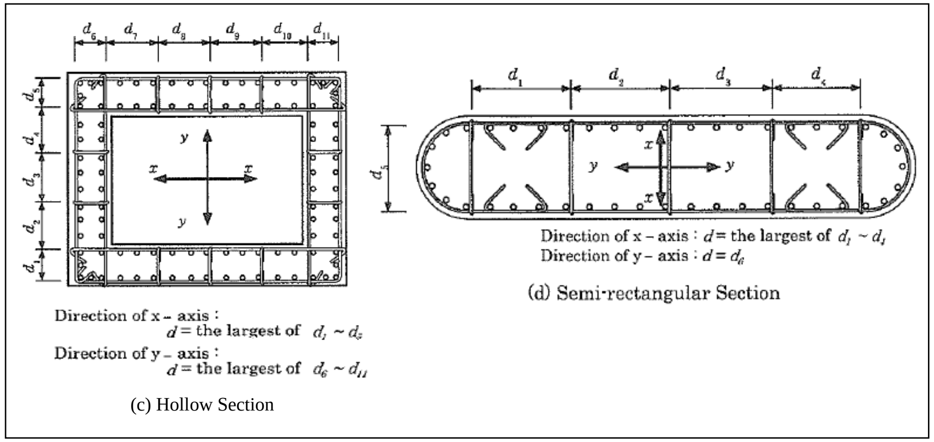
$$n = \frac{E_c\varepsilon_{cc}}{E_c\varepsilon_{cc} - \sigma_{cc}} \qquad \text{(14-13)}$$

$$\sigma_{cc} = \sigma_{ck} + 3.8\alpha\rho_s\sigma_{sy} \qquad \text{(14-14)}$$

$$\varepsilon_{cc} = 0.002 + 0.033\beta\frac{\rho_s\sigma_{sy}}{\sigma_{ck}} \qquad \text{(14-15)}$$

$$E_{des} = 11.2\frac{\sigma_{ck}^{\ 2}}{\rho_s\sigma_{sy}} \qquad \text{(14-16)}$$

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2^{nd} Ed., 1997 (Reprint Ed. 2005) - ASEP
	$\varepsilon_{cu} = \begin{cases} \varepsilon_{cc} & \text{(For Type I Earthguake Ground Motion)} \\ \varepsilon_{cc} + \dfrac{0.2\sigma_{cc}}{E_{des}} & \text{(For Type II Earthguake Ground Motion)} \end{cases}$ ....(14-17) $\rho_s = \dfrac{4A_h}{sd} \leqq 0.018$ .... (14-18) σ_c : Stress of concrete (N/mm^4) σ_{cc} : Strength of concrete restrained by lateral confining reinforcement (N/mm^2) σ_{ck} : Design strength of concrete (N/mm^2) ε_c : Strain of concrete ε_{cc} : Strain of concrete under the maximum compressive stress ε_{cu} : Ultimate strain of concrete restrained by lateral confining reinforcement E_c : Young's modulus of concrete (N/mm^2) E_{des} : Descending gradient (N/mm^2) ρ_s : Volume ratio of lateral confining reinforcement A_h : Sectional area of each lateral confining reinforcement (mm^2) s : Spacings of lateral confining reinforcement (mm) d : Effective length (mm) of lateral confining reinforcement. It shall be the longest length of core concrete divided and restrained by lateral hoop ties or cross ties (refer to Fig. 14-7) σ_{sy}: Yield point of lateral confining reinforcement (N/mm^2) α,β: Modification factor on section. α=1.0,β=1.0 for a circular section, and α= 0.2, β= 0.4 for rectangular, hollow circular and hollow rectangular sections. n : A constant defined by Eq. (14-13)  (a) Circular Section (b) Rectangular Section	

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP
	 **Fig. 14-7** Effective Length of Lateral Confining Reinforcement (in Both Longitudinal and Transverse Direction to the Bridge Axis)	
15. Shear Strength (Concrete Structure)	Shear strength shall be calculated by Eq. (15-1) $P_s = S_c + S_s$ ……… (15-1) $S_c = c_c c_e c_{pt} \tau_c bd$ ……… (15-2) $S_s = \dfrac{A_w \sigma_{sy} d(\sin\theta + \cos\theta)}{1.15a}$ ……… (15-3) P_s : Shear strength (N) S_c : Shear strength resisted by concrete (N) τ_c : Average shear stress that can be borne by concrete (N/mm^2). Values in Table 15-1 shall be used. c_c : Modification factor on the effects of alternating cyclic loading. c_c shall be taken as 0.6 for Type I Earthquake Ground Motion and 0.8 for Type II. c_e : Modification factor in relation to the effective height (d) of a pier section. Values in Table 15-2 shall be used. (b) Effective Height of a rectangular Section c_{pt} : Modification factor in relation to the axial tensile reinforcement ratio p_t. Values in Table 15-3 shall be used. b : Width of a pier section perpendicular to the direction in calculating shear strength (mm) d : Effective height of a pier section parallel to the direction in calculating shear strength (mm) p_t : Axial tensile reinforcement ratio. It is the value obtained by dividing the total sectional areas of the main reinforcement on the tension side of the neutral axis by bd (%).	(1) The design of cross-sections subject to shear is based on: $V_u \leq \phi V_n$ (15-1) where: V_u = factored shear force at the section considered (N) V_n = nominal shear strength (N) ϕ = strength reduction factor (0.85) The nominal shear strength is determined by: $V_n = V_c + V_s$ (15-2) where: V_c = nominal shear strength provided by concrete (N) V_s = nominal shear strength provided by shear reinforcement (N) (2) In the end regions the quantity of shear stress taken by the concrete V_c is assumed zero unless the minimum design axial compression force produces an average stress in excess of 0.10f'$_c$ of gross concrete area. (3) When the average compression stress in the member exceeds 0.10f'$_c$ the value V_c is compute as:

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP

S_s: Shear strength borne by hoop ties (N)

A_w: Sectional area of hoop ties arranged with an interval of α and an angle of θ (mm^2)

σ_{sy}: Yield point of hoop ties (N/mm^2)

θ : Angle formed between hoop ties and the vertical axis (degree)

α : Spacings of hoop ties (mm)

Table 15-1 Average Shear Stress of Concrete τc (N/mm^2)

Design Compressive Strength of Concreteσ_{ck} (N/mm^2)	21	24	27	30	40
Average Shear Stress of Concreteτ_c (N/mm^2)	0.33	0.35	0.36	0.37	0.41

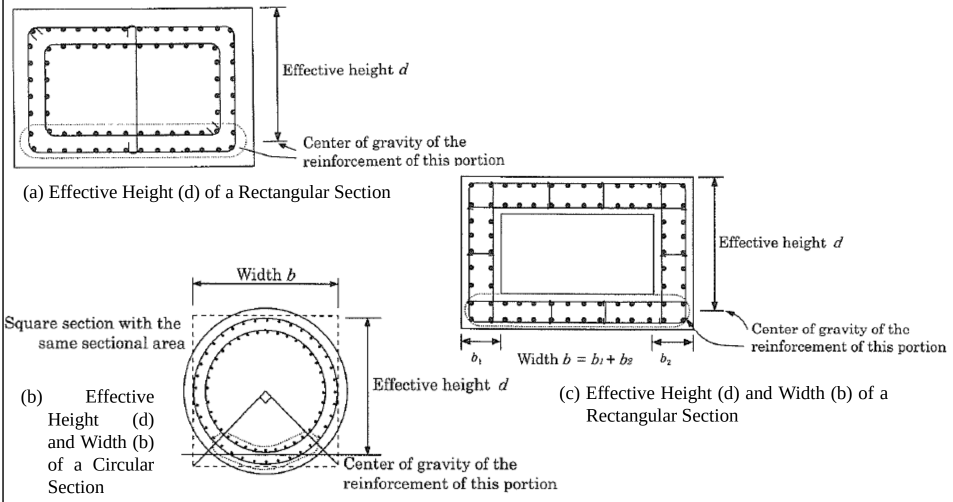
Table 15-2 Modification Factor Ce in Relation to Effective Height of a Pier Section

Effective Height (mm)	Below 1000	3000	5000	Above 10000
c_e	1.0	0.7	0.6	0.5

Table 15-3 Modification Factor Cpt in Relation to Axial tensile Reinforcement Ratio Pt

Tensile Reinforcement Ratio (%)	0.2	0.3	0.5	Above 1.0
c_{pt}	0.9	1.0	1.2	1.5

Evaluation method of effective height (d) for each column section shape is shown in Fig. 15-1.



(a) Effective Height (d) of a Rectangular Section

(b) Effective Height (d) and Width (b) of a Circular Section

(c) Effective Height (d) and Width (b) of a Rectangular Section

Fig. 15-1 Effective Height (d) and Width (b) of Each Section Shape

For members subject to axial compression:

$$V_c = 1+\frac{0.07N_u}{A_g}\left(\frac{\sqrt{f'_c}}{6}\right)b_w d \qquad (15\text{-}3)$$

or

$$V_c = (1/6)\ \sqrt{f'_c}\ b_w d \qquad (15\text{-}4)$$

where:

N_u = factored axial load normal to the cross-section occurring simultaneously with V_u to be taken positive for compression and negative for tension (N)

A_g = gross area of section (mm^2)

M_u = factored moment (N-m)

d = distance from the extreme compression fiber to the centroid of the longitudinal reinforcement (mm)

b_w = width of web (mm)

f'_c = specified compressive strength of concrete (MPa)

The quantity N_u/A_g shall be expressed in MPa.

For members subject to axial tension, shear reinforcement shall be designed to carry total shear, unless a more detailed calculation is made using:

$$v_c = \left(1+\frac{0.3N_u}{A_g}\right)\left(\frac{\sqrt{f'_c}}{6}\right)b_w d \qquad (15\text{-}5)$$

Note:

(a) N_u is negative for tension

(b) The quantity N_u/A_g shall be expressed in MPa.

(4) The allowable shear stress, v_u, in the pier shall be determined in accordance with the following equation:

$$v_u = 2\sqrt{f'_c} + \rho_h f_y \qquad (15\text{-}6)$$

where:

ρ_h = the ratio of horizontal shear reinforcement area to gross concrete area of a vertical section.

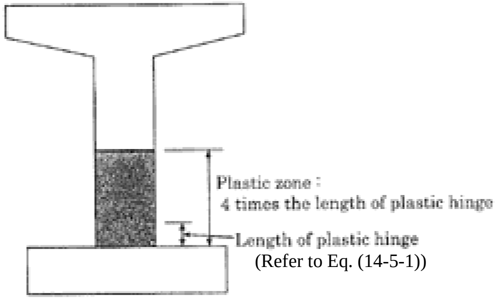
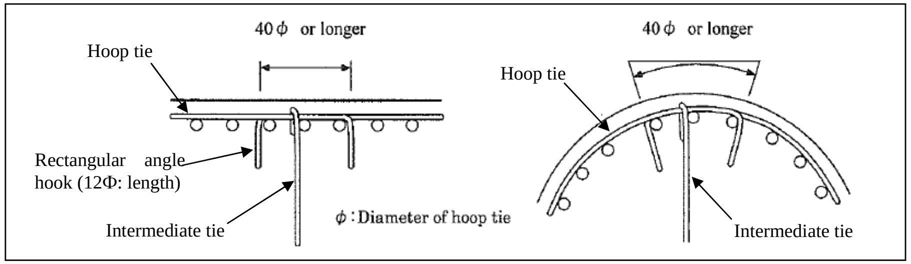
f_y = specified yield strength of reinforcement (MPa)

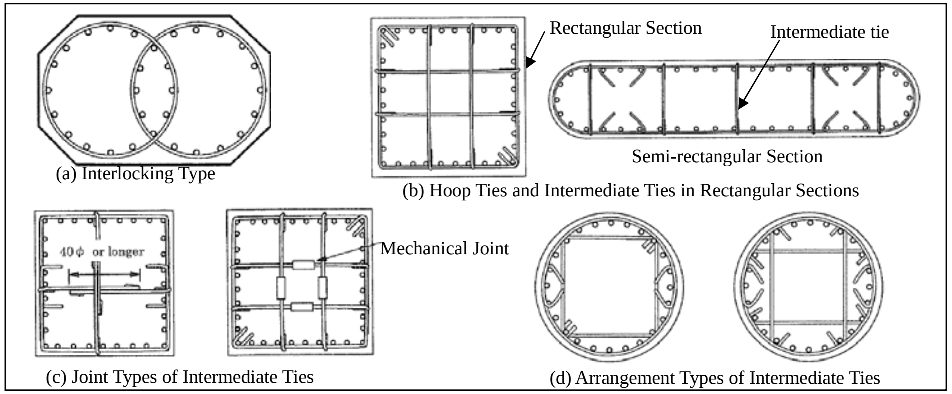
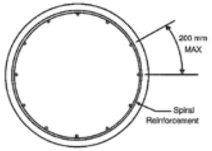
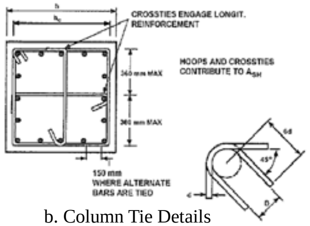
The allowable shear stress shall not exceed $8\sqrt{f'_c}$.

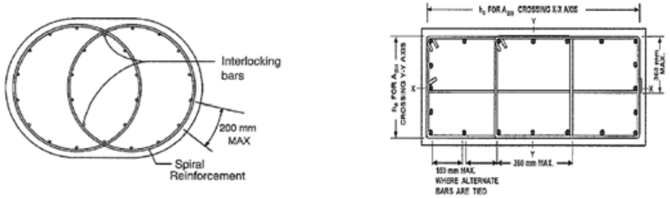
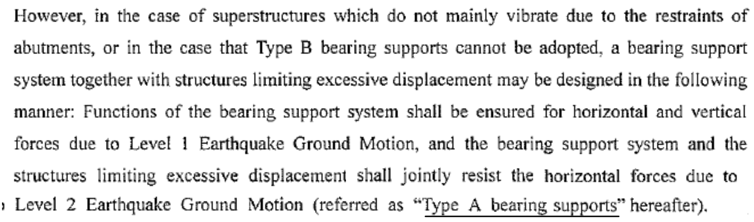
(5) When shear reinforcement is perpendicular to the axis of the member,

$$V_s = A_v f_y d / s \qquad (15\text{-}7)$$

Where A_v is the area of shear reinforcement within the distance *s*.

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP
16. Structural Details for Improving Ductility Performance	(1) Incase that generation of plastic deformation of the column is expected, lapping of axial reinforcements shall not generally be placed within the plastic zone (refer to Fig 16-1). (2) Arrangement of Hoop Ties (a) To use deformed bars of at least 13mm in diameter, and the intervals shall not generally be greater than 150mm in the plastic zone. (b) To be arranged so as to enclose the axial reinforcement and be fixed in to the concrete inside a column with the length below. i) Semi-circular hook = 8Φ or 120mm whichever is the greater. ii) Acute angle hook = 10Φ iii) Rectangular angle hook = 12Φ (Φ: the diameter of the hoop tie)  **Fig. 16-1** Plastic Zone (c) Lapping of hoop ties shall be staggered along the column height. (d) To have a lap length of at least 40Φ in case that hoop ties are lapped at any place other than the corners of a rectangular section (refer to Fig 16-2).  **Fig. 16-2** Anchorage of Hoop Ties with Rectangular Angle Hook (3) Arrangement of Intermediate Ties (a) To be of the same material and the same diameter as the hoop ties. (b) To be arranged in both the directions of the long side and the short side of a column section. (c) Intervals within a column section shall not be greater than one meter. (d) To be arranged in all sections with hoop ties arranged. (e) To be hooked up to the hoop ties arranged in the perimeter directions of the section. (f) To be fixed into the concrete inside a column (refer to Fig. 16-2 and 16-3). (g) To go through a column section, with use of a continuous reinforcing bar or a pair of reinforcing bars with a joint within the column section.	(1) Splices (a) Lap splices is permitted only within the center half of the column height, and the splice length shall not be less than 400mm or 60 bar diameters whichever is greater. (b) The maximum spacing of the transverse reinforcement over the length of the splice shall not exceed the smaller of 100mm or one-quarter of the minimum number of dimension. (c) Welded splices may be used for splicing provided not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section and the distance between splices of adjacent bars is greater than 610mm as measured along the longitudinal axis of the column. (2) Transverse Reinforcement for Confining Plastic Hinges (a) Transverse reinforcement for confinement of plastic hinges shall be: - provided at the top and bottom of column over a length equal to the maximum cross-sectional column dimensions, one-sixth of the clear height of the column or 450mm, - extended into the top and bottom connections, - provided at top of piles in pile bents over the same length as the columns, - provided within pile bents over a length extending from 3 pile diameters below the calculated point of moment fixity to one pile diameter but not less than 450mm above the mud line, - spaced not exceed the smaller of one-quarter of the minimum member dimension or 100mm. (b) Lapping of the spiral reinforcement in the specified transverse confinement regions is not permitted. Connections of spiral reinforcement in this region must be full strength lap welds. (c) The end region shall be assumed to extend from the soffit of girders or cap beams at the top of the columns, or the top of the foundations at the bottom of the columns, a distance not less than (a) maximum cross-sectional dimension of the column, (b) one sixth of the clear height of the column, (c) 450mm. (d) The end region of a pile bent shall be the same as specified for columns at the top of the pile bent, and three pile diameters below the calculated point of moment fixity to one pile diameter but not less than 450mm above the mud line at the bottom of the pile bent. (e) The cores of columns and pile bents shall be confined by transverse reinforcement in the expected plastic hinge regions, generally located at the top and bottom of columns and pile bents wit the largest confinement governed by the provisions given above. (f) The transverse reinforcement for confinement shall have yield strength not more than that of the longitudinal reinforcement. (g) The volumetric ratio of spiral reinforcement, ρ_s, for a circular column shall be: $\rho_s = 0.45[A_g/A_c - 1](f'_c/f_{yh})$ (16-1) or $\rho_s = 0.12(f'_c/f_{yh})$ (16-2) For rectangular column, the total gross sectional area A_{sh}, of rectangular hoop shall be either:

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP
	 **Fig. 16-3** Arrangement of Hoop Ties and Intermediate Ties According to Column Types	$A_{sh} = 0.30ah_c\,[A_g/A_c - 1](f'_c/f_{yh})$ (16-3) or $A_{sh} = 0.12ah_c\,(f'_c/f_{yh})$ (16-4) where: a = vertical spacing of hoops with a maximum of 100 (mm) A_c = area of column core (mm^2) A_g = gross area of column (mm^2) A_{sh} = total cross-sectional area of hoop reinforcement including supplementary cross ties having a vertical spacing in mm and crossing a section having a core dimension of h_c mm (mm^2) f'_c = specified compressive strength of concrete (MPa) f_{yh} = specified yield strength of hoop or spiral reinforcement (MPa) h_c = core dimension of tied column in the direction under consideration (mm) ρ_s = ratio of volume of spiral reinforcement to total volume of concrete core (out-to-out of spirals) (h) Transverse hoop reinforcement may be provided by single or overlapping hoops. Cross-ties having the same bar size as the hoop may be used. Each end of the cross-tie shall engage a peripheral longitudinal reinforcing bar. All ties shall have 135 deg hooks with extensions not less than the larger of ten tie diameter or 150mm. (3) Column Connection Column connection is referred to as the vertical extension of the column area into the adjoining member. (a) The design force for the connection between the column and the cap beam superstructure, pile cap or spread footing shall be the group load combination in Section 3 or the forces developed due to column hinging. (b) The development length for all longitudinal steel shall be that required for steel stress of $1.25\sqrt{f'_c}$ for normal-weight aggregate concrete or $9\sqrt{f'_c}$ for light-weight aggregate concrete. (c) The required column transverse reinforcement shall be continued for a distance equal to one-half the maximum column dimension but not less than 380mm from the face of the column connection into the adjoining member. (d) The shear stress in the joint of a frame or bent, in the direction under consideration, shall not exceed 12  a. Single spiral  b. Column Tie Details

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP
		 c. Column Interlocking Spiral Details d. Column Tie Details **Figure 16-1** Details of spirals, hoops, ties and cross-ties (4) Construction Joints in Piers and Columns (a) Where shear is resisted at a construction joint solely by dowel action and friction on a roughened concrete surface, the total shear force across the joint shall not exceed V_j determined from the following formula: $V_j = \phi (A_{vf} f_y + 0.75P_n)$ (16-5) where: A_{vf} = total area of reinforcement, including flexural reinforcement (mm^2) P_n = minimum axial load (N)
17. Bearing Support System	(1) Bearing support shall be fundamentally designed for horizontal and vertical forces due to Level 1 and Level 2 EGMs (referred as "<u>Type B bearing support</u>"). (2)  However, in the case of superstructures which do not mainly vibrate due to the restraints of abutments, or in the case that Type B bearing supports cannot be adopted, a bearing support system together with structures limiting excessive displacement may be designed in the following manner: Functions of the bearing support system shall be ensured for horizontal and vertical forces due to Level 1 Earthquake Ground Motion, and the bearing support system and the structures limiting excessive displacement shall jointly resist the horizontal forces due to Level 2 Earthquake Ground Motion (referred as "<u>Type A bearing supports</u>" hereafter).	(1) Although joint connection design forces are stipulated as either the elastic forces divided by the response modification factor or the force effects resulting from the plastic hinging moments of the columns, there is no seismic design provisions for bearings in the NSCP Standards (1997/2005). (2) However, in order for the bearings to transmit the forces of the superstructure to the substructure, it is taken that the bearings shall behave elastically to transmit the seismic forces from the superstructure. (3) The NSCP Standards (1997/2005) recommends four types of bearings, although no seismic provision is explicitly stated, as follows: - Elastomeric Bearing - TFE Bearing Surface - Pot Bearings - Disc Bearings

<table>
<tr><th>Items</th><th>JRA (Part V; English Version, 2002)</th><th>NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP</th></tr>
<tr><td></td><td>

(3) Fig. 17-1 shows the selection flow of bearing support.

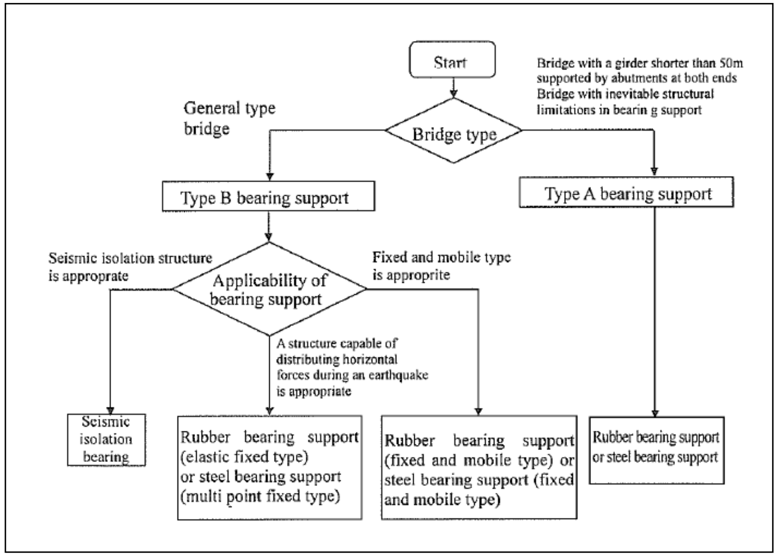
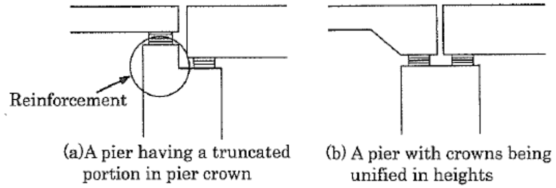


Fig. 17-1 General Consideration in Selection of Bearing Support



(a)A pier having a truncated portion in pier crown

(b) A pier with crowns being unified in heights

Fig. 17-2 Measures for Dealing with Truncated Portion of a Pier Crown

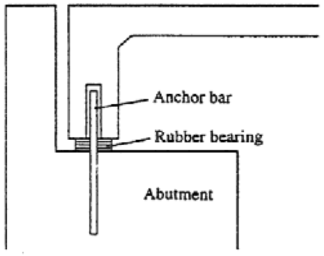


Fig. 17-3 Structure Limiting Excessive Displacement Connecting Superstructure and Substructure

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<tr><td>18. Unseating Prevention System</td><td>

18.1 Seating Length

(1) Ordinary Bridge

- Eq. (18-1) shows the required seating length of a girder at its support.
- The seat length shall be measured in the direction perpendicular to the front line of the bearing support when the direction of soil pressure acting on the substructure differs from the bridge axis, as in case of askew bridge or a curved bridge.

</td><td>

18.1 Seating Length

(a) Minimum Support Length Requirements

Bearing seats supporting the expansion end of girders shall be designed to provide a minimum support length N (mm) not less than:

$$N = 305 + 2.5L + 10H \qquad (18\text{-}1)$$

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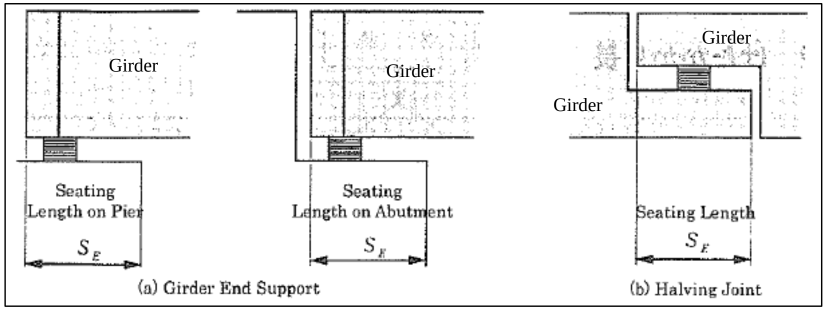
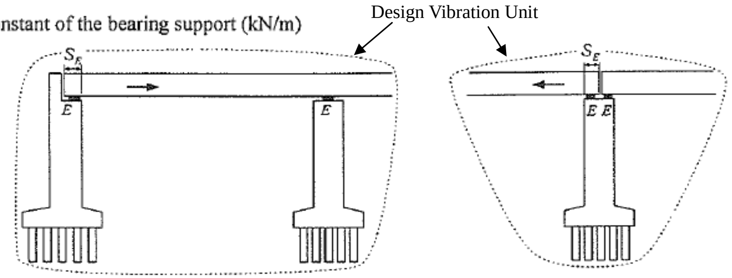
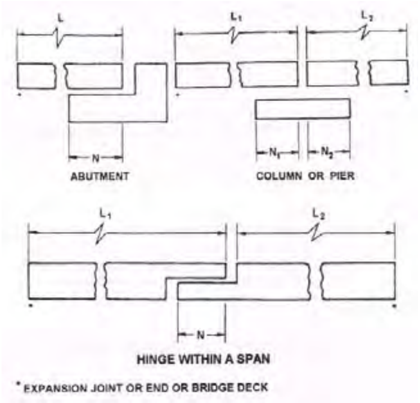
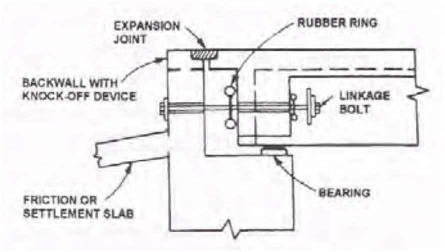
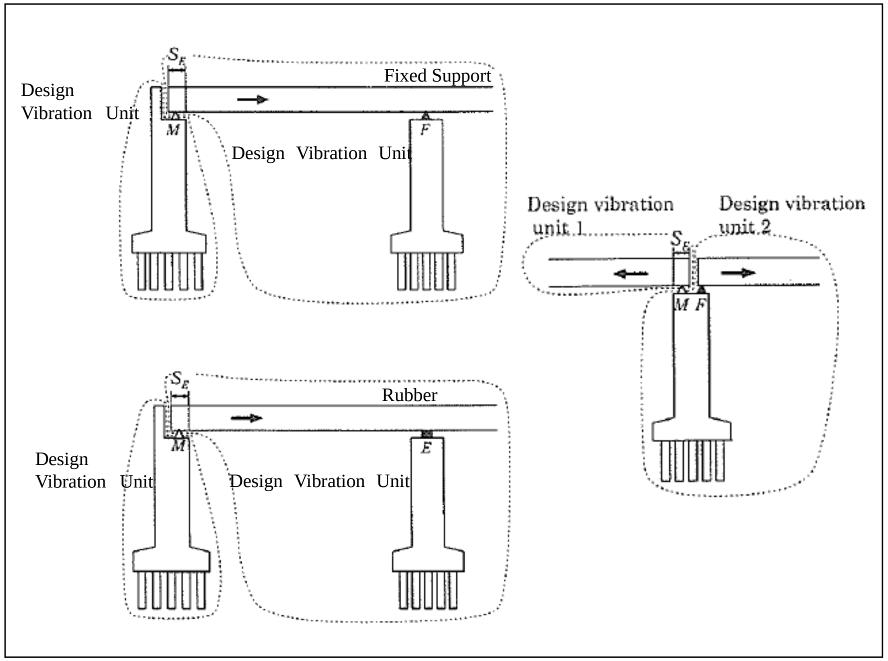
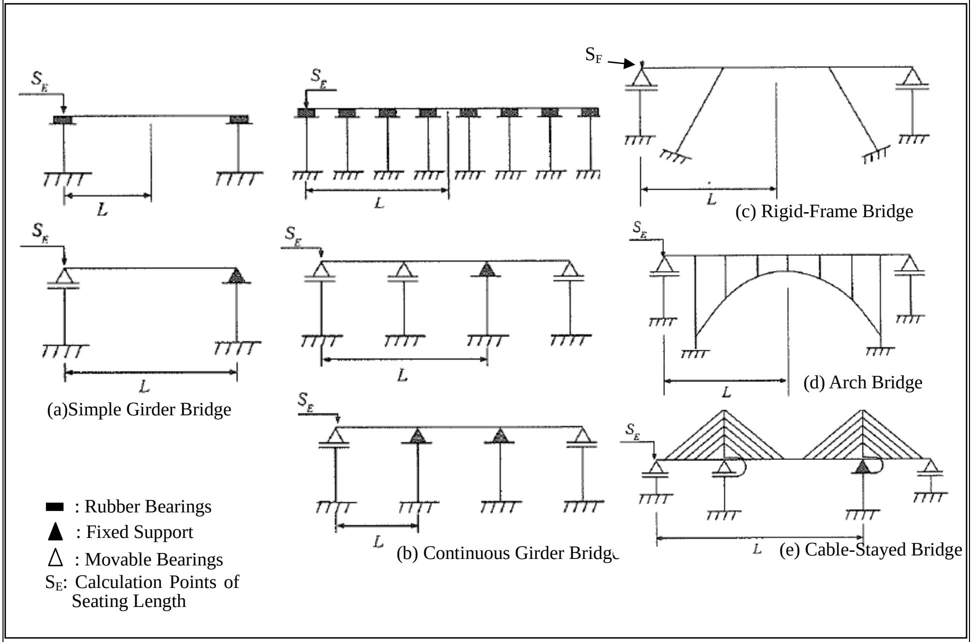
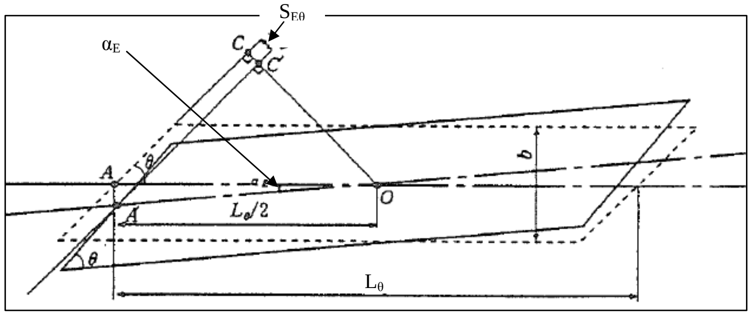
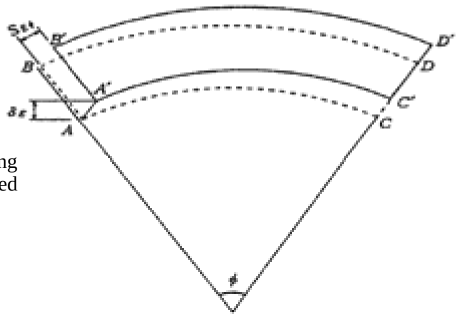
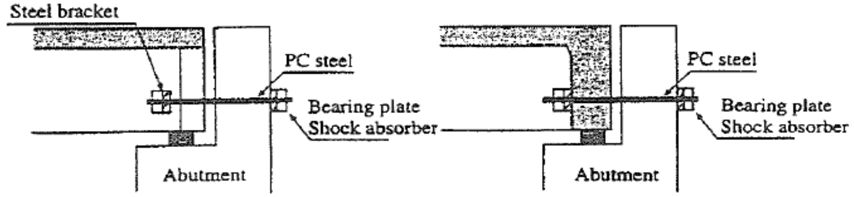
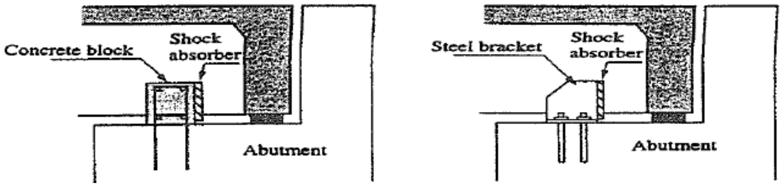
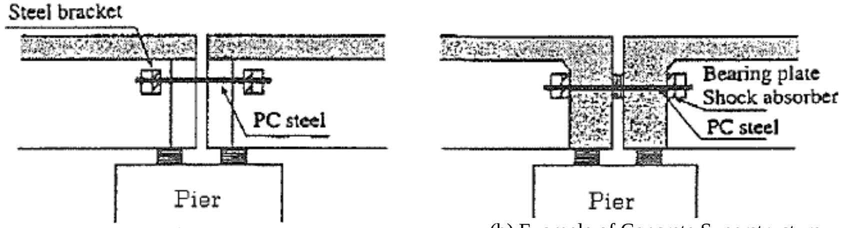
Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2^{nd} Ed., 1997 (Reprint Ed. 2005) - ASEP
	$S_E = U_R + U_G \geq S_{EM}$ --- (18-1) $S_{EM} = 0.7 + 0.005 * Ls$ --- (18-2) $U_G = \varepsilon_G * L$ -- (18-3) where, S_E: Refer to Fig. 18-1. U_R: Maximum relative displacement between the superstructure and the edge of the top of the substructure due to Level 2 EGM (m) (refer to description of U_R below) U_G: Relative displacement of the ground caused by seismic ground strain (m) S_{EM}: Minimum seating length of a girder at the support ε_G: Seismic ground strain = 0.0025 for Ground Type I, 0.00375 for Ground Type II, 0.005 for Ground Type III L: Distance between two substructures for determining the seating length (refer to description of L below) Ls: Length of the effective span (m). When two superstructures with different span length are supported on one bridge pier, the longer one shall be used.  **Fig.18-1** Seating Length (S_E) (m) Description of U_R (a) Rubber Bearing (refer to Fig 18-2) $u_R = \frac{c_m P_u}{k_B}$ .... (18-4) P_u : Horizontal force equivalent to lateral strength of a column when considering plastic behavior of the column; or horizontal force equivalent to maximum response displacement of a foundation when considering plastic behavior of the foundation (kN) c_m : Dynamic modification factor (C_m = 1.2) k_B : Spring constant of the bearing support (kN/m)  **Fig. 18-2** When the girder end is supported by rubber bearings	where: *L* = length of the bridge deck to the adjacent expansion joint, or to the end of the bridge deck. For hinges within a span, *L* shall be the sum of L_1 and L_2, the distances to either side of the hinge (m) *H* = average height of columns supporting the bridge deck to the next expansion joint (m) - For columns and Piers: H = column or pier height (m) - For hinges within a span: H = average height of the adjacent two columns or piers (m)  Figure 18-1 Dimension for minimum support length 18.2 Horizontal Linkage/Longitudinal Restrainers • Positive horizontal linkage shall be provided between adjacent sections of the superstructure at supports and expansion joints within a span. The linkage shall be designed for a minimum force of the Acceleration Coefficient times the weight of the lighter of the two adjoining spans or parts of the structure. • If the linkage is at a point where relative displacement of the sections of superstructure is designed to occur during seismic motion, sufficient slack must be allowed in the linkage so that the linkage force does not start to act until the design displacement is exceeded. • Where linkage is to be provided at columns or piers, the linkage of each span may be attached to the column or pier rather than between adjacent spans. • Positive linkage shall be provided by ties, cables, dampers or equivalent mechanism. Friction shall not be considered a positive linkage. 18.3 Hold Down Device • Hold down devices shall be provided at supports and at hinges in continuous structures, where the vertical seismic force due to the longitudinal horizontal seismic load opposes and exceeds 50%, but is less than 100% of the dead load reaction. • If the vertical forces result in uplift, the hold down device shall be designed to resist the larger of the following net upward force: o 120% of the difference between the vertical seismic force (Q) due to longitudinal horizontal seismic load and the dead load reaction (DR), or o 10% of the dead load downward force that would be exerted if the span were simply supported.

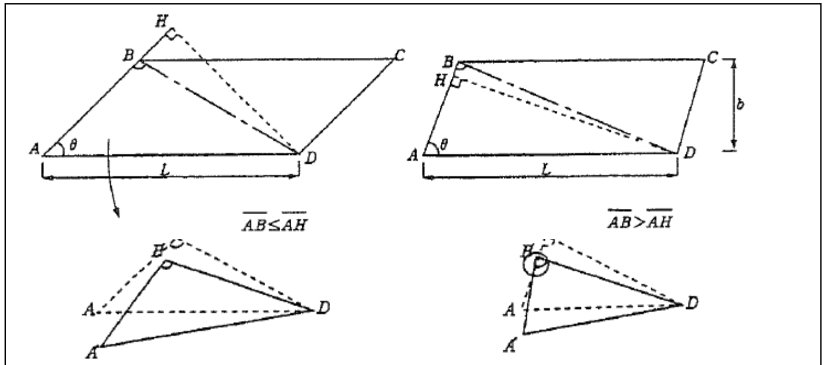
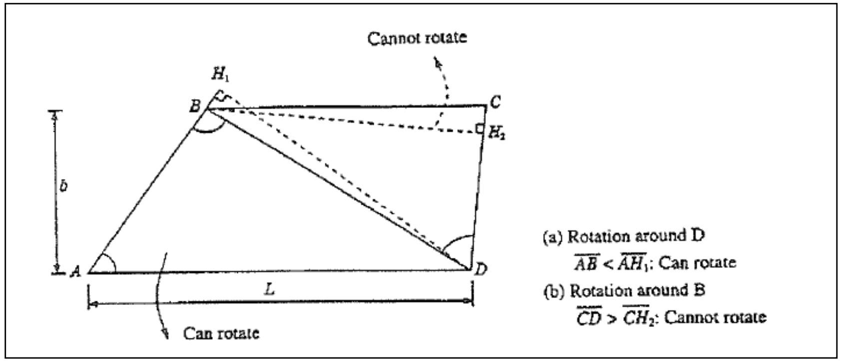
Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP
	(b) Fixed Bearing When fixed supports of Type B are used at the support concerned, one-half of the width of the bearing support shall be taken as the relative displacement u_R, while the whole width shall be used in case of fixed supports of Type A. This is for preventing unseating of the bridge even though the bearing support system is unexpectedly damaged, although the fixed supports of Type B are designed to be capable of transferring the superstructural inertia force caused either by horizontal force equivalent to lateral strength of a column with plastic behavior, or by horizontal force equivalent to maximum response displacement of a foundation with plastic behavior, and there is no relative displacement generating at the bearing support system under an assumed earthquake ground motion in the design. Providing that the bearing support may be damaged in case of fixed supports of Type A, u_R for Type A is assumed to be greater than that of Type B. In addition, the width of the bearing support shall be equal to width of the lower shoe in the bridge axis in the case of metal bearings, and width of the bearing support in the same direction for rubber bearings. (c) Movable Bearing (refer to Fig 18-3) $U_R = \sqrt{\sum U_{Ri}^2}$ (i = 1,2) -- (18-5) $U_{Ri} = U_{Pi} + U_{Fi} + U_{Bi}$ -- (18-6) $U_{Pi} = \mu_{Ri} * \delta_{yi}$ --- (18-7) $U_{Fi} = \delta_{Fi} + \theta_{Fi} * h_{oi}$ -- (18-8) $U_{Bi} = C_m * P_{ui} / K_{Bi}$ -- (18-9) where, u_{Ri}: Displacement of the i-th design vibration unit shown in Fig. 18-3 u_{Pi}: Response displacement of the column representing the i-th design vibration unit (m) u_{Fi} : Horizontal displacement at the height of superstructural inertia due to displacement of the pier foundation representing the i-th design vibration unit (m). The calculation shall include these effects when liquefaction and lateral spreading of the ground U_{Bi} : Relative displacement at the bearing support system representing the i-th design vibration unit (m) μ_{Ri} : Response ductility factor of the column representing the i-th design vibration unit δ_{yi} : Yielding displacement of the column representing the i-th design vibration unit (m) δ_{Fi} : Response horizontal displacement of the pier foundation representing the i-th design vibration unit (m) θ_{Fi} : Response rotation angle of the pier foundation representing the i-th design vibration unit (rad) h_{0i} : Height from the ground surface of seismic design to the superstructural inertial force in the column representing the i-th design vibration unit (m) P_{ui} : Horizontal force equivalent to lateral strength of a column with plastic behavior, or by	 Figure 18-2 Horizontal Linkage

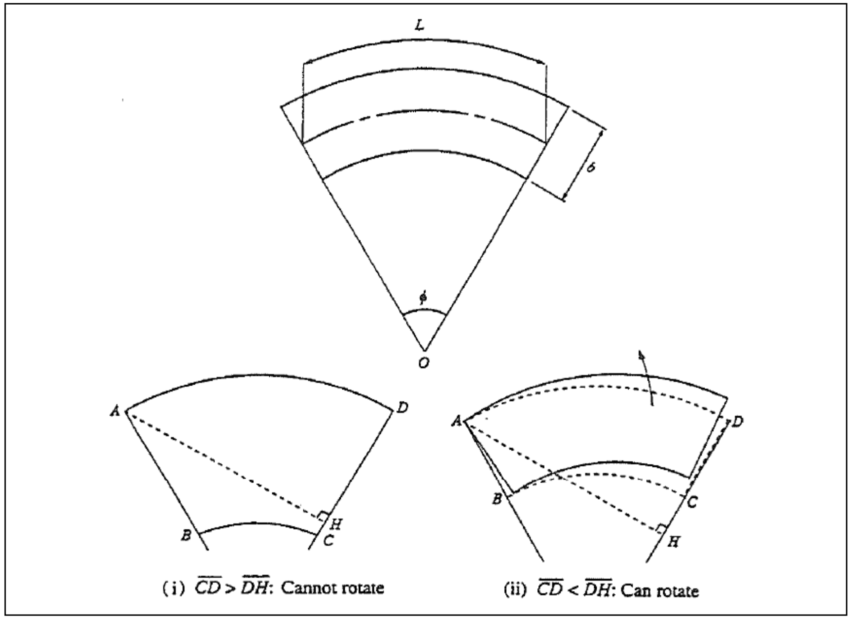
Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP
	horizontal force equivalent to maximum response displacement of a foundation with plastic behavior, representing the i-th design vibration unit (kN) c_m : Dynamic modification factor (C_m = 1.2) k_{Bi} : Spring constant of the bearing support for the i-th design vibration unit (kN/m)  **Fig. 18-3** Inertia Forces Used in Calculating Seating Length Description of L The length L between the substructures that may affect the seating length shall be the distance between the substructure supporting the girder at the support where the seating length is to be calculated and one that may primarily affect the vibration of the girder containing the support (refer to Fig. 18-4)	

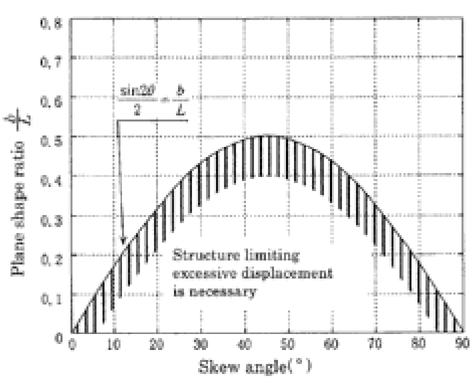
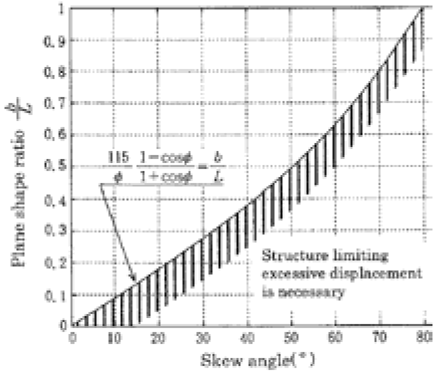
Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP
	 **Fig. 18-4** Measuring Methods of Distance (L) between Substructures as to Bridge Types (2) A Bridge with Complicated Dynamic Structural Behavior by a Dynamic Analysis ➢ The maximum relative displacement (U_R) is to be obtained from the dynamic analysis. (3) A Skew Bridge ➢ The seating length shall be calculated by Eq. (18-10) (refer to Fig. 18-5). $S_{E\theta} \geqq (L_\theta/2)(\sin\theta - \sin(\theta - \alpha_E))$ ……………………………………………… (18-10) where $S_{E\theta}$: Seating length for the skew bridge (m) L_θ : Length of a continuous superstructure (m) θ : Skew angle (degree) α_E : Marginal unseating rotation angle (degree). α_E can generally be taken as 5 degrees.	

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP
	 **Fig. 18-5** Seating Length of a Skew Bridge (4) A Curved Bridge ➢ The seating length is to be calculated by Eq. (18-11) (refer to Fig. 18-6) $S_{E\phi} \geqq \delta_E \dfrac{\sin\phi}{\cos(\phi/2)} + 0.3$ ······ (18-11) $\delta_E = 0.005\,\phi + 0.7$ ······ (18-12) where $S_{E\phi}$: Seating length for the curved bridge (m) δ_E : Displacement of the superstructure toward the outside direction of the curve (m) ϕ : Fan-shaped angle by the two edges of a continuous girder of a curved bridge (degrees)  **Fig. 18-6** Seating Length Corresponding to the Movement of a Curved Bridge 18.2 Unseating Prevention Structure (1) Ultimate Strength of an Unseating Prevention Structure ➢ Ultimate strength of an unseating prevention structure is to be calculated by Eq. (18-13). ➢ The unseating prevention structure is a structure of 1) connecting the superstructure and the substructure (refer to Fig. 18-7), 2) providing protuberance either in superstructure and in the substructure (refer to Fig. 18-8), 3) joining two superstructures together (refer to Fig. 18-9).	

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP
	$H_F = 1.5\,R_d$ ······ (18-13) $S_F = c_F S_E$ ······ (18-14) where H_F : Design seismic force of the unseating prevention structure (kN) R_d : Dead load reaction (kN). In case the structure connects two adjacent girders, the larger reaction shall be taken. S_F : Maximum design allowance length of the unseating prevention structure (m) S_E : Seating length specified in Section 16.2 (m). c_F :Design displacement coefficient of the unseating prevention structure. ($C_F = 0.75$)  (a) Example of steel superstructure (b) Example of concrete superstructure **Fig. 18-7** Unseating Prevention Structures Connecting the Superstructure with the Substructure  (a) Example of Concrete Block (b) Example of Steel Bracket **Fig. 18-8** Unseating Prevention Structures Providing Protuberance on the Superstructure or the Substructure  (a) Example of Steel Superstructure (b) Example of Concrete Superstructure **Fig. 18-9** Unseating Prevention Structures Connecting the Two Adjacent Superstructures	

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP
	18.3 Structure Limiting Excessive Displacement (1) For the following bridges, structures limiting excessive displacement working in the direction perpendicular to the bridge axis shall be installed in the terminal support, in addition to the unseating prevention system working in the bridge axis. (a) Skew bridges with a small skew angle satisfying Eq. (18-15) (refer to Fig. 18-10 and 18-11) $\sin 2\theta/2 > b / L$ ··· (18-15) L: Length of a continuous superstructure (m) b: Whole width of the superstructure (m) θ: Skew angle (degree)  (a) ∠ DBA ≥ 90° (a bridge can rotate) (b) ∠ DBA < 90° (a bridge can not rotate) **Fig. 18-10** Conditions in Which a Skew Bridge can Rotate Without Being Affected by Adjoining Girders or Abutment  **Fig. 18-11** Conditions in Which a Skew Bridge With Unparallel Bearing Lines on Both Edges of the Superstructure can rotate	

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP
	(b) Curved bridges satisfying Eq. (18-16) (refer to Fig. 18-12) $\frac{115}{\phi}\frac{1-\cos\phi}{1+\cos\phi} > b/L$ ··· (18-16) L: Length of a continuous superstructure (m) b: Whole width of the superstructure (m) Φ: Intersection angle (degree)  (i) $\overline{CD} > \overline{DH}$: Cannot rotate (ii) $\overline{CD} < \overline{DH}$: Can rotate **Fig. 18-12** Conditions in Which a Curved Bridge can rotate Without being Affected by Adjoining Girders or Abutment The relation of Eq. (18-15) and Eq. (18-16) is shown in Fig. 18-13 and Fig. 18-14, respectively.	

Items	**JRA (Part V; English Version, 2002)**	**NSCP Vol. II Bridges ASD (Allowable Stress Design), 2^{nd} Ed., 1997 (Reprint Ed. 2005) - ASEP**
	 **Fig. 18-13** Conditions in which a Skew Bridge Requires an Structure Limiting Excessive Displacement in the Transverse Direction  **Fig. 18-14** Conditions in which a Curved Bridge Requires an Structure Limiting Excessive Displacement in the Transverse Direction (c) For the following bridges, the structures limiting excessive displacement shall be installed at intermediate supports ➢ Bridges with the superstructure being narrow at the top ➢ Bridges with a small number of bearing supports on one bearing line ➢ Bridges probably to be subject to movement of the bridge piers in the direction perpendicular to the bridge axis as a result of lateral spreading.	
19. Effects of Seismically Unstable Ground	19.1 Assessment of Extremely Soft Clayey Soil Layer in Seismic Design (1) For a clayey layer or a silt layer lying within three meters from the ground surface, and having compressive strength of 20KN/m or less obtained from an unconfined compression test or an in-situ test, the layer shall be regarded as an extremely soft layer in the seismic design. (2) In this case its geological parameters (shear modulus and strength) shall be assumed to be zero in the seismic design. 19.2 Assessment of Soil Liquefaction (1) Sandy Layer Requiring Liquefaction Assessment ➢ Saturated soil layer having ground water level higher than 10m below the ground surface and lying at a depth less than 20m below the ground surface. ➢ Soil layer containing a fine content (FC) of 35% or less, or soil layer having plasticity index Ip less than 15, even if FC is larger than 35%. ➢ Soil layer having a mean particle size (D_{50}) less than 10mm and a particle size at 10% pass (on the grading curve) (D_{10}) is less than 1mm. (2) Assessment of Liquefaction The liquefaction resistance factor FL calculated by Eq. (19-1) turns out to be less than 1.0, the layer shall be regarded as a soil layer having liquefaction potential.	(1) Two basic approaches to evaluate the cyclic liquefaction potential of a deposit of saturated sand subject to earthquake shaking includes: ▪ Empirical methods based on field observations of the performance of sand deposits in previous earthquakes, and correlations between sites which have and have not liquefied and Relative Density of Standard Penetration Test (SPT) blow counts. ▪ Analytical methods based on laboratory determination of liquefaction strength characteristics of undisturbed samples and the use of dynamic site response analysis to determine the magnitude of earthquake-induced shearing stresses. (2) For conventional evaluations using a "total stress" approach the two methods are similar, but differ only in the manner in which the field liquefaction strength is determined. In the "total stress" approach, liquefaction strength are normally expressed as the ratio of an equivalent uniform or average cyclic shearing stress τ_h acting on horizontal surfaces of the sand to the initial vertical effective stress σ'. As the first approximation, the cyclic stress ratio developed in the field because of earthquake ground shaking may be computed from an equation given by Seed and Idriss, namely: $(\tau_h)_{av}/\sigma'_o = 0.65r_d(a_{max}/g)(\sigma_o/\sigma'_o)$ (19-1) where: a_{max} = maximum or effective peak ground acceleration at the ground surface σ_o = total overburden pressure on sand layer under consideration σ'_o = initial effective overburden pressure on sand layer under consideration

<table>
<tr><th>Items</th><th>JRA (Part V; English Version, 2002)</th><th>NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP</th></tr>
<tr><td></td><td>
$F_L = R / L$ ········ (19-1)
$R = c_w R_L$
$L = r_d k_{hg} \sigma_v / \sigma'_v$
$R_d = 1.0 - 0.015x$

$\sigma_v = \gamma_{t1} h_w + \gamma_{t2} (x - h_w)$
$\sigma'_v = \gamma_{t1} h_w + \gamma'_{t2} (x - h_w)$

(For Type I Earthquake Ground Motion)
$c_w = 1.0$ ········
(For Type II Earthquake Ground Motion)
$c_w = \begin{cases} 1.0 & (R_L \leqq 0.1) \\ 3.3R_L + 0.67 & (0.1 < R_L \leqq 0.4) \\ 2.0 & (0.4 < R_L) \end{cases}$

F_L : Liquefaction resistance factor
R : Dynamic shear strength ratio
L : Seismic shear stress ratio
c_W : Modification factor on earthquake ground motion
R_L : Cyclic triaxial shear stress ratio to be obtained by properties (3) below
r_d : Reduction factor of seismic shear stress ratio in terms of depth
k_{hg}: Design horizontal seismic coefficient at the ground surface for Level 2 Earthquake Ground Motion specified in Item 11
σ_v: Total overburden pressure (kN/m^2)
σ'_v : Effective overburden pressure (kN/m^2)
x : Depth from the ground surface (m)
γ_{t1}: Unit weight of soil above the ground water level (kN/m^3)
γ_{t2}: Unit weight of soil below the ground water level (kN/m^3)
γ'_{t2}: Effective unit weight of soil below the ground water level (kN/m^3)
h_w: Depth of the ground water level (m)

(3) Cyclic Tri-axial Shear Stress Ratio
Cyclic tri-axial shear stress ratio R_L shall be calculated by Eq. (19-2).

$R_L = \begin{cases} 0.0882\sqrt{N_a / 1.7} & (N_a < 14) \\ 0.0882\sqrt{N_a / 1.7} + 1.6 \times 10^{-6} \cdot (N_a - 14)^{4.5} & (14 \leq N_a) \end{cases}$ ······ (19-2)

<For Sandy Soil>
$N_a = c_1 N_1 + c_2$
</td><td>
r_d = stress reduction factor varying from a value of 1 at the ground surface to 0.9 at a depth of about 9.1m

(3) *Empirical Method*
Values of cyclic stress ratio defined by Equation 19-1 have been correlated with parameters such as relative density based on SPT. The latest form of this correlation is shown in Figures 19-1 and 19-2. N_1 is the measured standard penetration resistance of the sand corrected to an effective overburden pressure of 95.8 kN/m^2 using the relationship:

$N_1 = NC_N$ (19-2)

Thus for a given site and a given maximum ground surface acceleration, the average stress ratio developed during the earthquake, $(\tau_h)_{av}/\sigma'_o$, at which liquefaction may be expected to occur, is expressed by the empirical correlations shown in Figure 19-1. It is suggested that a factor of safety of 1.5 is desirable to establish a reasonable measure of safety against liquefaction in the case of important bridge sites.

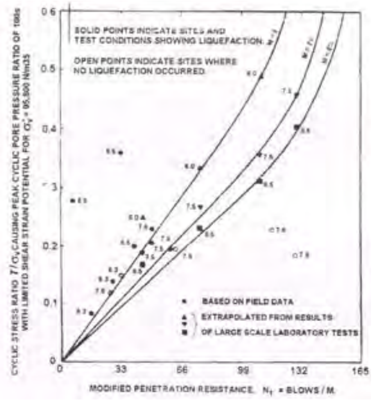


Figure 19-1 Correlation Between Field Liquefaction Behavior and Penetration Resistance

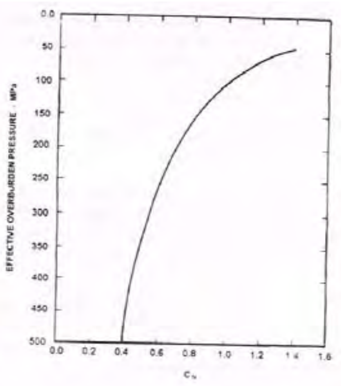


Figure 19-2 Relationship Between C_N and Effective Overburden Pressure

(4) Analytical Method
The analytical approach for evaluating liquefaction potential is based on a comparison between field liquefaction strengths established from cyclic laboratory tests on undisturbed samples and earthquake-induced shearing stresses. In this approach, liquefaction strength curve from laboratory test results requires data adjustment to account for factors such as correct cyclic stress simulation, sample disturbance, aging effects, field cyclic stress history, and magnitude of in situ lateral stresses.

Once liquefaction strength curve has been established, if a total stress analysis is used, liquefaction potential is evaluated from comparisons with earthquake-induced shear stresses (see Figure 19-3).

An improved representation of the progressive development of liquefaction is provided by the use of an effective stress approach where pore water pressure increases are coupled to nonlinear dynamic response solutions and the influence of potential pore water pressure dissipation during an earthquake, which
</td></tr>
</table>

Items	JRA (Part V; English Version, 2002)	NSCP Vol. II Bridges ASD (Allowable Stress Design), 2nd Ed., 1997 (Reprint Ed. 2005) - ASEP

$$N_1 = 170N / (\sigma'_v + 70)$$

$$c_1 = \begin{cases} 1 & (0\% \leqq FC < 10\%) \\ (FC + 40) / 50 & (10\% \leqq FC < 60\%) \\ FC / 20 - 1 & (60\% \leqq FC) \end{cases}$$

$$c_2 = \begin{cases} 0 & (0\% \leqq FC < 10\%) \\ \\ (FC - 10) / 18 & (10\% \leqq FC) \end{cases}$$

<For Gravelly Soil>

$$N_a = \{ 1 - 0.36 \log_{10}(D_{50} / 2) \} N_1$$

R_L: Cyclic triaxial shear stress ratio

N: N value obtained from the standard penetration test

N_1 : Equivalent N value corresponding to effective overburden pressure of 100 kN/m^2

N_a : Modified N value taking into account the effects of grain size

c_1, c_2: Modification factors of N value on fine content

FC : Fine content (%) (percentage by mass of fine soil passing through the 75μm mesh)

D_{50}: Mean grain diameter (mm)

19.3 Reduction Factor (D_E) of Geotechnical Parameters due to Liquefaction

Geotechnical parameters of a sandy layer causing liquefaction affecting a bridge shall be obtained by multiplying geotechnical parameters without liquefaction by reduction factor D_E shown in Table 19-1.

Table 19-1 Reduction Factor DE for Geotechnical Parameters

Range of F_L	Depth from Present Ground Surface x (m)	Dynamic shear strength ratio R			
		$R \leqq 0.3$		$0.3 < R$	
		Verification for Level 1 Earthquake Ground Motion	Verification for Level 2 Earthquake Ground Motion	Verification for Level 1 Earthquake Ground Motion	Verification for Level 2 Earthquake Ground Motion
$F_L \leqq 1/3$	$0 \leqq x \leqq 10$	1/6	0	1/3	1/6
	$10 < x \leqq 20$	2/3	1/3	2/3	1/3
$1/3 < F_L \leqq 2/3$	$0 \leqq x \leqq 10$	2/3	1/3	1	2/3
	$10 < x \leqq 20$	1	2/3	1	2/3
$2/3 < F_L \leqq 1$	$0 \leqq x \leqq 10$	1	2/3	1	1
	$10 < x \leqq 20$	1	1	1	1

provides data on the time history of pore water pressure increase.

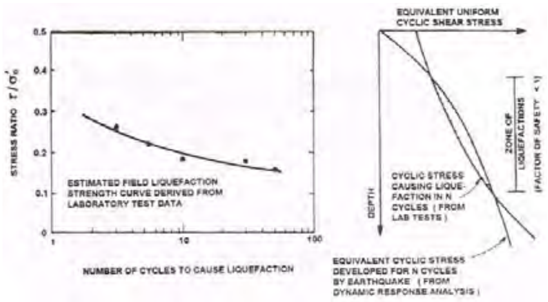


Figure 19-3 Principles of Analytical Approach (Total Stress) to Liquefaction Potential Evaluation

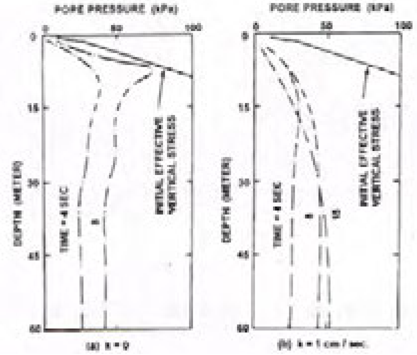


Figure 19-4 Effective Stress Approach to Liquefaction Evaluation Showing Effect of Permeability

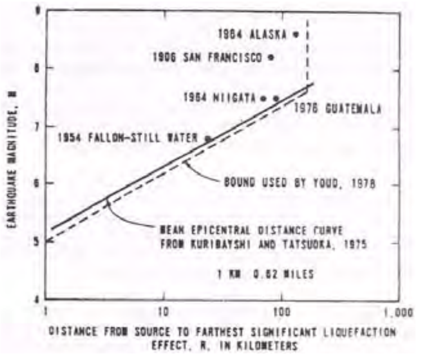


Figure 19-5 Maximum Distance to Significant Liquefaction as a Function of Earthquake Magnitude

It is of interest to note that a rough indication of the potential for liquefaction may be obtained by making use of empirical correlations established between earthquake magnitude and the epicentral distance to the most distant field manifestation of liquefaction.

VOLUME 2

DEVELOPMENT OF ACCELERATION RESPONSE SPECTRA

APPENDIX 2-B

DETERMINATION OF SITE SPECIFIC DESIGN SEISMIC RESPONSE SPECTRA FOR SEVEN (7) BRIDGES

Site-specific design spectra are obtained for 7 bridge sites (2 bridges in Package B and 5 bridges in Package C) as shown in Figure 2B-3 and 2B-4. The results shall be used in the outline design of the selected bridges.

Site-specific design spectra at a location are obtained using the procedure shown in Figure 2B-1 and Figure 2B-2. It basically consists of conducting a probabilistic seismic hazard analysis and a dynamic site response analysis.

Active faults as presently identified by Phivolcs (Philippine Institute of Volcanology and Seismology) are shown plotted in Figure 2B-3. Also shown plotted are instrumentally recorded earthquake events from 1907 to 2012 with magnitude greater than 4 and focal depth of less than 100 kms. which are compiled (consisting of about 26,000+ events) from Phivolcs and ISC (International Seismological Centre) websites into an earthquake catalog. The magnitude scale is homogenized in a common moment magnitude scale, moment magnitude scale in this study for the reasons that moment magnitude does not suffer from saturation during large earthquakes and it is now most commonly adopted in most ground motion predictive models that are presently being proposed. Declustering algorithm based on Gardner and Knopoff (1974) is applied to retain only independent main shocks (into 7000+ events as shown plotted in Figure 2B-4), removing aftershocks and foreshocks. Completeness analysis based on the method of Stepp (1972) is applied to the catalog to remove possible biases towards bigger events in subsequent regression analysis for temporal characterization of earthquake occurrences for each defined seismic source model since it is known that lower magnitude earthquake events had been under-reported in the early part of the instrumental era; and become less so with progressively improved instruments.

Seismic source modeling consisting of fault models and background seismicity models are shown in Figure 2B-4. Background seismicity modeling is used to model seismic occurrences into areal zones where the observed seismicity exhibits a more or less diffused pattern that cannot be clearly identified with a specific fault. This may include earthquake occurrences in the future that could be attributable to blind thrusts or faults with no previous ground surface fault manifestations. Each earthquake event in the declustered set is identified to be associated with one of the fault models or background seismicity seismogenic areal zones. Bigger events are preferably made to be associated with the fault models.

For source-to-site distance uncertainty modeling, earthquakes in this study are assumed to be uniformly distributed within a particular source zone (i.e., earthquakes are considered equally likely to occur at any location within a source). Rupture may occur with equal likelihood anywhere in the fault plane in the fault zone and anywhere in the seismogenic areal zone. The spatial (source-tosite distance) uncertainty can be described by a probability density function P(R) which may be approximated by a normalized frequency distribution histogram.

For fault models characterizing crustal earthquakes, maximum potential earthquake size capable to be produced within the source is computed using the empirical method of Wells and Coppersmith (1994). On the other hand, the method of Papazachos et al (2004) is used to compute maximum potential earthquake size for sources due to trenches. For seismogenic areal zones modeling background seismicity, the highest recorded or documented magnitude plus 0.5 is used. List of historically documented earthquakes from 1589 to 1895 is based on the study by Bautista and Oike (2000).

Two types of earthquake recurrence models are used in this study: bounded Gutenberg-Richter recurrence model and characteristic earthquake recurrence model. The more commonly used bounded Gutenberg-Richter recurrence model in most PSHA implementation is expressed as:

$$\lambda_m = \nu \frac{\exp[-\beta(m-m_0)] - \exp[-\beta(m_{\text{max}}-m_0)]}{1-\exp[-\beta(m_{\text{max}}-m_0)]} \qquad \text{for } m_0 \leq m \leq m_{\text{max}}$$

where m_0 is the lowest magnitude considered to be of engineering significance and m_{max} is the maximum magnitude based on seismological and geological considerations as discussed earlier. Characteristic earthquake recurrence model using data based on paleoseimological observation is preferred (but limited in use in this study due to the scarcity of data) due to the short history of instrumental recording in the world relative to geological period over which earthquakes recurred.

Probabilistic seismic hazard analysis (PSHA) provides a framework in which uncertainties in the size, location, and rate of recurrence of earthquakes and in the variation of ground motion characteristics with earthquake size and location can be identified, quantified, and combined in a rational manner (Thenhaus and Campbell, 2003).

The probability that an observed ground motion parameter X (spectral acceleration, in this study) will be greater than or equal to the value x in the next t years (the exposure period) given the annual exceedance rate $\lambda\,[X \geq x]$ is computed as:

$$P\,[X \geq x] = 1 - \exp\left(-t\,\lambda\,[X \geq x]\right)$$

$$\lambda\,[X \geq x] \approx \sum_{\text{sources } i} v_i \int_{m_0}^{m_{\text{max}}} \int_{R|M} P\,[X \geq x|M,R]\, f_M(m)\, f_{R|M}(r|m)\, dr\, dm$$

where	$\lambda\,[X \geq x]$	the annual frequency that ground motion at a site exceeds the chosen level $X = x$;
	v_i	the annual rate of occurrence of earthquakes on seismic source i having magnitudes between m_0 and m_{max};
	m_0	the minimum magnitude of engineering significance (taken to be 5.0 in this study);
	m_{max}	the maximum magnitude assumed to occur on the source;
	$P\,[X \geq x\|M,R]$	the conditional probability that the chosen ground motion level is exceeded for a given magnitude M and distance R;
	$f_M(m)$	probability density function of earthquake magnitude;
	$f_{R\|M}(r\|m)$	probability density function of distance from the earthquake source to the site of interest.

Ground motion estimation models used in this study are based on Boore-Atkinson NGA (2007) applied to crustal earthquake sources, Young et al model (1997) applied to subduction sources; and Zhao et al (2006) applied to both crustal and subduction sources.

Four locations at Guadalupe Bridge (see Figure 2B-5) are considered. For Guadalupe bridge site, probability distribution functions for source-to-site distances and magnitudes for four example sources are shown in Figure 2B-6. Computed total seismic hazard curves at several key periods from 0. to 3. seconds are shown in Figure 2B-7. Also shown in Figures 2B-9 to 2B-12 are the major contributing sources to the total hazard at four shown periods (0., 0.2, 1., 3. sec) at Guadalupe Bridge site. Exceedance rates corresponding to 50-year, 100-year, 500-year and 1000-year return periods at each key periods used in the computation are calculated (shown in Figure 2B-12 for 0., 0.2, 1., 3. sec. and in Figure 2B-13 for 0. to 3. sec.). Uniform hazard spectral curves for the basement rock at Guadalupe Bridge corresponding to return periods of 1000 years, 500 years, 100 years, and 50 years are shown in Figure 2B-14.

The seismic hazard curve for 50-year return period is deaggregated as shown for your key periods (0., 0.2, 1., 3. sec) in Figures 2B-15 to 2B-18. Based on this information, seven seed records obtained from earthquake recording database available are selected considering similar tectonic regimes as close as possible. Spectral matching based on Rspmatch (Bommer 2007) are performed on the 7 pairs of seed records to match the uniform hazard curve at 50-year return period as shown in Figure 2B-19. The mean of the response spectra of the 7 × 2 spectrally matched earthquake time histories is shown in Figure 2B-20 to match the uniform hazard curve at 50-year return period.

Site-specific design spectra are generated for four locations at the Guadalupe Bridge site in which 7 pairs of spectrally matched earthquake ground motion time histories are applied at the basement-rock level (assumed at 30 to 45 meters deep) and propagated up to the ground surface level by nonlinear site response analysis procedure as shown in Figures 2B-21 to 2B-35. Site-specific design spectrum for Guadalupe Bridge at location A1 for 50-year return period is constructed based on the mean of the nonlinear site response analyses due to 7 × 2 earthquake time histories as shown in Figure 2B-36 and in standard code form in Figure 2B-37.

Similar procedure (deaggregation, selection of 7 seed records, spectral matching, nonlinear site response analyses) is done to obtained site-specific design spectra at 100-year (Figure 2B-43), 500-year (Figure 2B-46), 1,000-year (Figure 2B-49) return periods for Guadalupe Bridge A1 site.

Nonlinear site response analyses are conducted for C2 (Figures 2B-51 to 2B-58), B2 (Figures 2B-60 to 2B-67), and D2 (Figures 2B-69 to 2B-76).

Site-specific design spectra at each location corresponding to the four return periods are shown in Figure 2B-50 (for A1), Figure 2B-59 (for C2), Figure 2B-68 (for B2), and Figure 2B-77 (for D2). Further, site-specific design spectra at the 4 locations are compared at each return period, as shown in Figure 2B-78 (for 50-year return period), Figure 2B-79 (for 100-year return period), Figure 2B-80 (for 500-year return period), and Figure 2B-81 (for 1000-year return period).

Uniform hazard spectral curves for the basement rock at Lambingan Bridge (see Figures 2B-82 and 2B-83) corresponding to return periods of 1000 years, 500 years, 100 years, and 50 years are shown in Figure 2B-84. Site-specific design spectra are generated for two locations at the Lambingan Bridge site in which 7 pairs of spectrally matched earthquake ground motion time histories are applied at the basement-rock level and propagated up to the ground surface level by nonlinear site response analysis procedure. Site-specific design spectra at each location corresponding to the four return periods are shown in Figure 2B-97 (for A1), and Figure 2B-106 (for B2).

Uniform hazard spectral curves for the basement rock at Palanit Bridge (see Figure 2B-107) corresponding to return periods of 1000 years, 500 years, 100 years, and 50 years are shown in Figure 2B-108. Site-specific design spectra are generated for location A1 at the Palanit Bridge site in which 7 pairs of spectrally matched earthquake ground motion time histories are applied at the basement-rock level and propagated up to the ground surface level by nonlinear site response analysis procedure. Site-specific design spectra at each location corresponding to the four return periods are shown in Figure 2B-121.

Uniform hazard spectral curves for the basement rock at Mawo Bridge (see Figure 2B-122) corresponding to return periods of 1000 years, 500 years, 100 years, and 50 years are shown in Figure 2B-123. Site-specific design spectra are generated for two locations at the Mawo Bridge site in which 7 pairs of spectrally matched earthquake ground motion time histories are applied at the basement-rock level and propagated up to the ground surface level by nonlinear site response analysis procedure. Site-specific design spectra at each location corresponding to the four return periods are shown in Figure 2B-136 (for A1), and Figure 2B-145 (for B2).

Uniform hazard spectral curves for the basement rock at Liloan Bridge (see Figure 2B-146) corresponding to return periods of 1000 years, 500 years, 100 years, and 50 years are shown in Figure 2B-147. Site-specific design spectra are generated for A1 at Liloan Bridge site in which 7 pairs of spectrally matched earthquake ground motion time histories are applied at the basement-rock level and propagated up to the ground surface level by nonlinear site response analysis procedure. Site-specific design spectra at each location corresponding to the four return periods are shown in Figure 2B-160.

Uniform hazard spectral curves for the basement rock at 1st Mactan-Mandaue Bridge (see Figure 2B-161) corresponding to return periods of 1000 years, 500 years, 100 years, and 50 years are shown in Figure 2B-162. Site-specific design spectra are generated for two locations at the 1st Mactan-Mandaue Bridge site in which 7 pairs of spectrally matched earthquake ground motion time histories are applied at the basement-rock level and propagated up to the ground surface level by nonlinear site response analysis procedure. Site-specific design spectra at each location corresponding to the four return periods are shown in Figure 2B-173 (for A1), and Figure 2B-182 (for B2).

Last but not the least, uniform hazard spectral curves for the basement rock at Wawa Bridge (see Figure 2B-183) corresponding to return periods of 1000 years, 500 years, 100 years, and 50 years are shown in Figure 2B-184. Site-specific design spectra are generated for A1 at Wawa Bridge site in which 7 pairs of spectrally matched earthquake ground motion time histories are applied at the basement-rock level and propagated up to the ground surface level by nonlinear site response analysis procedure. Site-specific design spectra at each location corresponding to the four return periods are shown in Figure 2B-197.

References:

D. M. Boore and G. M. Atkinson, "Boore-Atkinson NGA ground motion relations for the geometric mean horizontal component of peak and spectral ground motion parameters," Technical Report PEER 2007/01, Pacific Earthquake Engineering Research Center, 2007.

Maria Leonila P. Bautista and Kazuo Oike, "Estimation of the magnitudes and epicenters of Philippine historical earthquakes," *Tectonophysics,* 317(12):137-169, 2000.

Maria Leonila P. Bautista and Bartolome C. Bautista, "The Philippine historical earthquake catalog: its development, current state and future directions," *Annals of Geophysics,* 47(2/3), April/June 2004.

J. K. Gardner and L. Knopoff, "Is the sequence of earthquakes in Southern California, with aftershocks removed, poissonian?" *Bulletin of the Seismological Society of America,* 64(5), 1974.

Thomas H. Heaton, Fumiko Tajima, and Ann Wildenstein Mori, "Estimating ground motions using recorded accelerograms," *Surveys in Geophysics,* 8:2583, 1986.

A. R. Nelson, S. F. Personius, R. E. Rimando, R. S. Punongbayan, N. Tuñgol, H. Mirabueno, and A. Rasdas. "Multiple large earthquakes in the past 1500 years on a fault in Metropolitan Manila, the Philippines."*Bulletin of Seismological Society of America,* 90:7385, 2000.

B. C. Papazachos, E. M. Scordilis, D. G. Panagiotopoulos, C. B. Papazachos, and G. F. Karakaisis, "Global relations between seismic fault parameters and moment magnitude of earthquakes," *Bulletin of the Geological Society of Greece,* XXXVI, 2004.

L. Reiter. *Earthquake Hazard Analysis: Issues and Insights.* Columbia University Press, 1990.

Carl J. Stepp, "Analysis of completeness of the earthquake sample in the Puget Sound area and its effects on statistical estimates of earthquake hazard," In *Proceedings of the International Conference on Microzonation for Safer Construction Research and Application,* Seattle, Oct 30 to Nov 3, 1972, vol. 2, 1972.

P. C. Thenhaus and K. W. Campbell. "Seismic Hazard Analysis," in *Earthquake Engineering Handbook,* edited by W.-F. Chen and C. Scawthorn, CRC Press, 2003.

D. L. Wells and K. J. Coppersmith, "New empirical relationships among magnitude, rupture length, rupture width, rupture area, and surface displacement," *Bulletin of the Seismological Society of America,* 84(4):974–1002, August 1994.

R. R. Youngs, S.-J. Chiou, W. J. Silva, and J. R. Humphrey, "Strong ground motion attenuation relationships for subduction zone earthquakes," *Seismological Research Letters,* 68(1), January/February 1997.

J. X. Zhao, J. Zhang, A. Asano, Y. Ohno, T. Oouchi, T. Takahashi, H. Ogawa, K. Irikura, H. K. Thio, P. G. Somerville, Y. Fukushima, and Y. Fukushima, "Attenuation relations of strong ground motion in Japan using site classification based on predominant period," *Bulletin of the Seismological Society of America,* 96(3):898913, June 2006.

Table 2B-1 Seed earthquake records selected as base rock motion of 50-year return period

	Description of Event	**Magnitude**	**Distance to Station (km)**	**Fault Mechanism**	**Recorded PGA (g)**	
					x	**y**
RP0050_eq1	IMPERIAL VALLEY 1940 p Stn: USGS Station 0117	6.9	12.2	Strike-Slip	0.35	0.21
RP0050_eq2	KOCAELI, TURKEY 1999 p Stn: Arcelik	7.5	10.6	Strike-Slip	0.22	0.15
RP0050_eq3	CHICHI 1999 c Stn: Taichung	7.6	60.9	Reverse Oblique	0.11	0.11
RP0050_eq4	TABAS, IRAN 1978 p Stn: Dayhook	7.4	13.9	Reverse	0.41	0.38
RP0050_eq5	CAPE MENDOCINO 1992 p Stn: CDMG Station 89509	7	42	Reverse	0.09	0.18
RP0050_eq6	KOBE, JAPAN 1995 p Stn: Takatori	6.9	1.5	Strike-Slip	0.61	0.62
RP0050_eq7	VALPARAISO 1985 c Stn: DGG Station 4407	7.8	129.2	Reverse	0.17	0.17

p – As given in the Pacific Earthquake Engineering Research Center (PEER) Ground Motion Database

c – As given in the Consortium of Organizations for Strong Motion Observation Systems (COSMOS) Virtual Data Center

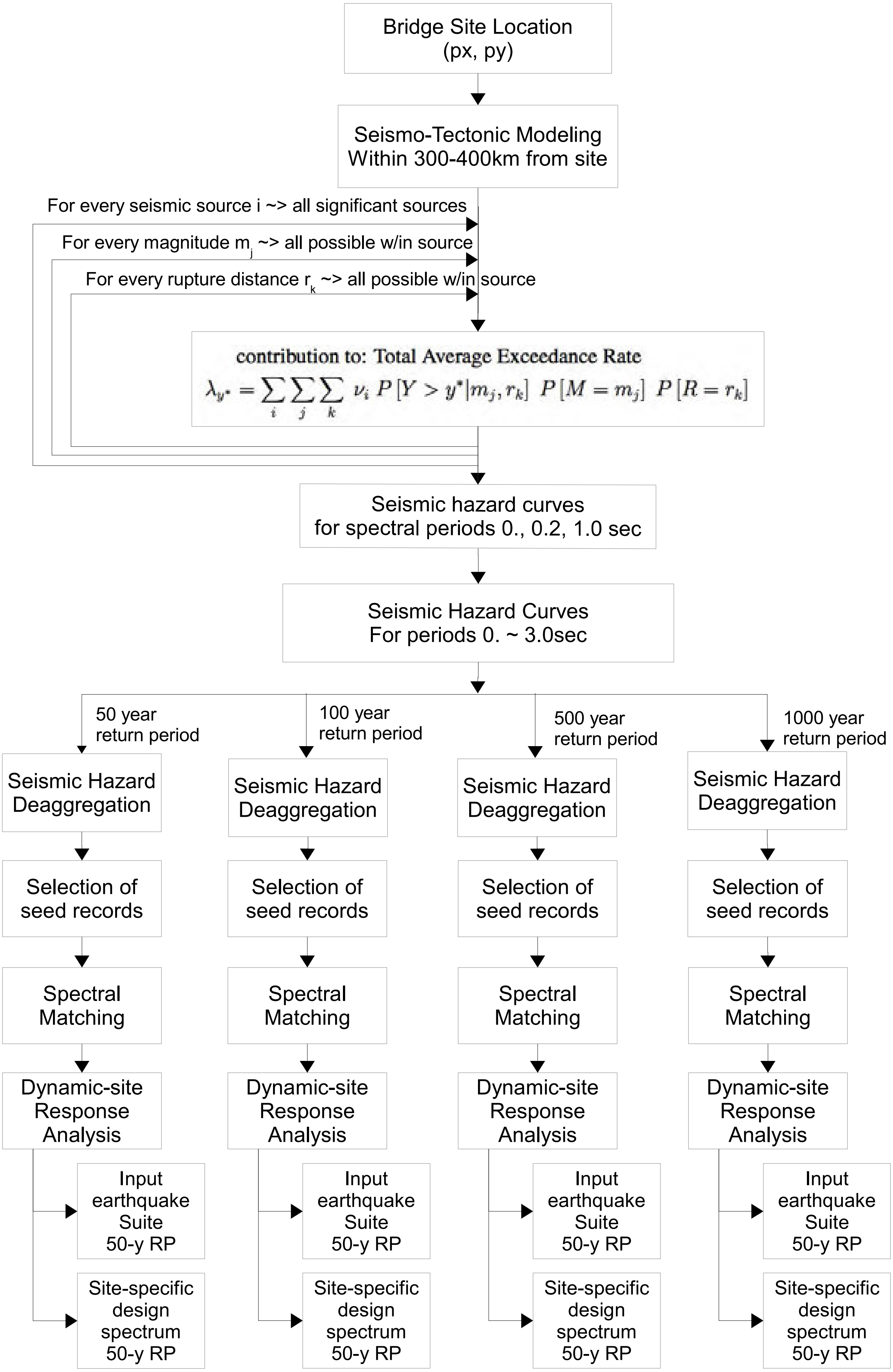


Figure 2B-1 Procedure of PSHA study for site-specific design spectra corresponding to return periods of 50, 100, 500, and 1000 years

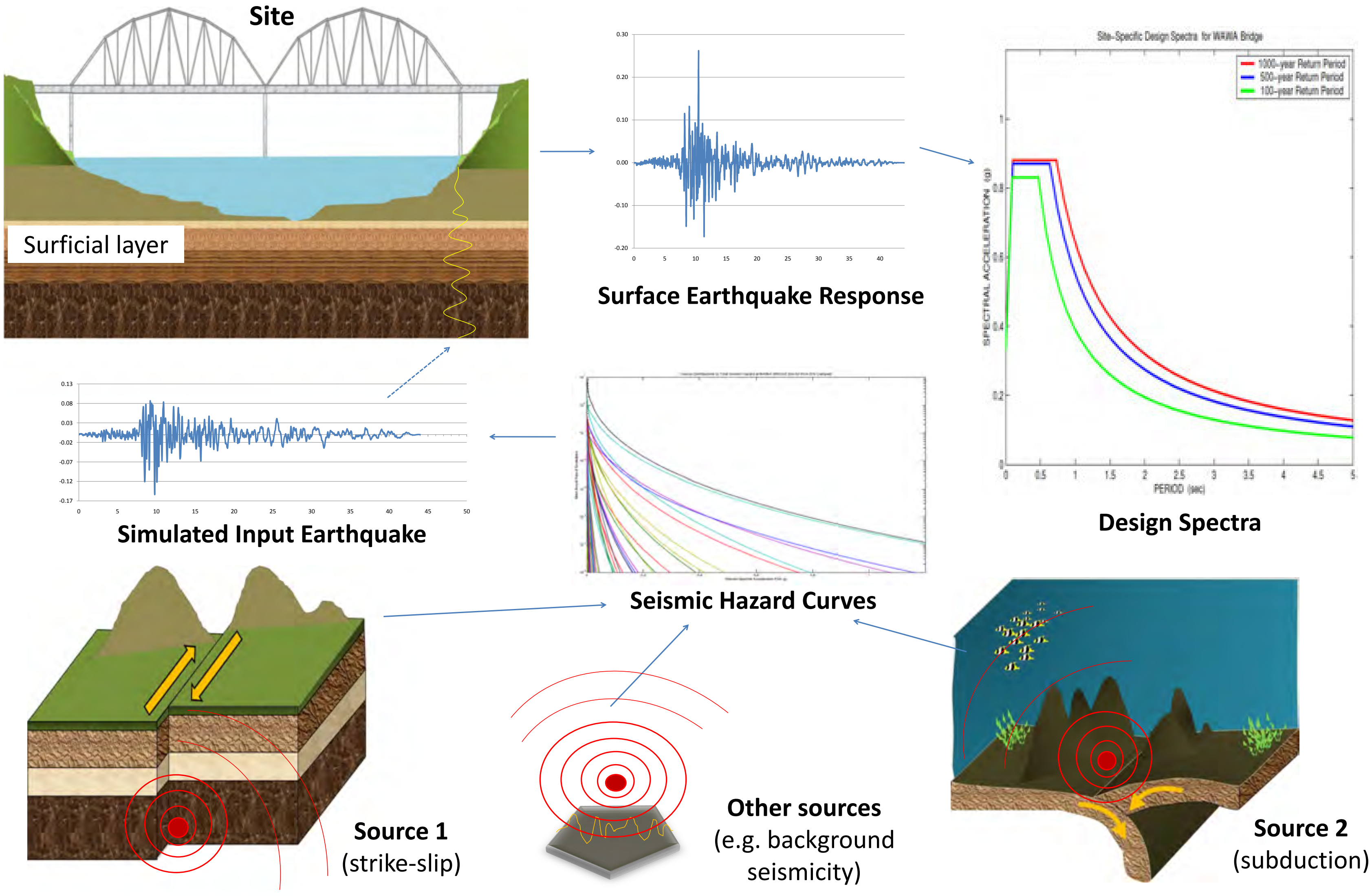


Figure 2B-2 Schematic analysis flow for generating site-specific design spectrum

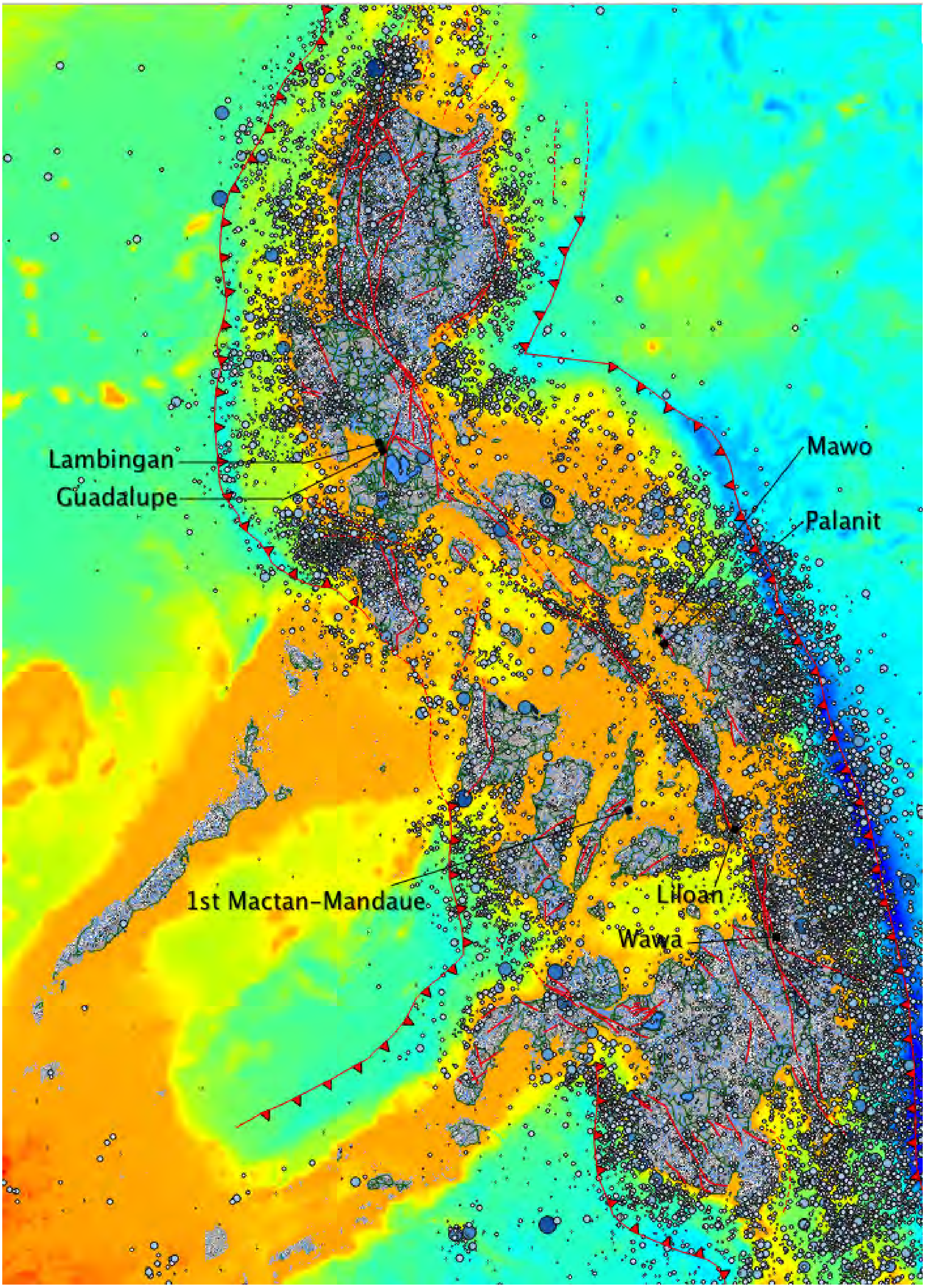


Figure 2B-3 Philippine seismological and tectonic setting (earthquakes 1907–2012 with depth < 100 kms; fault traces after PHIVOLCS)

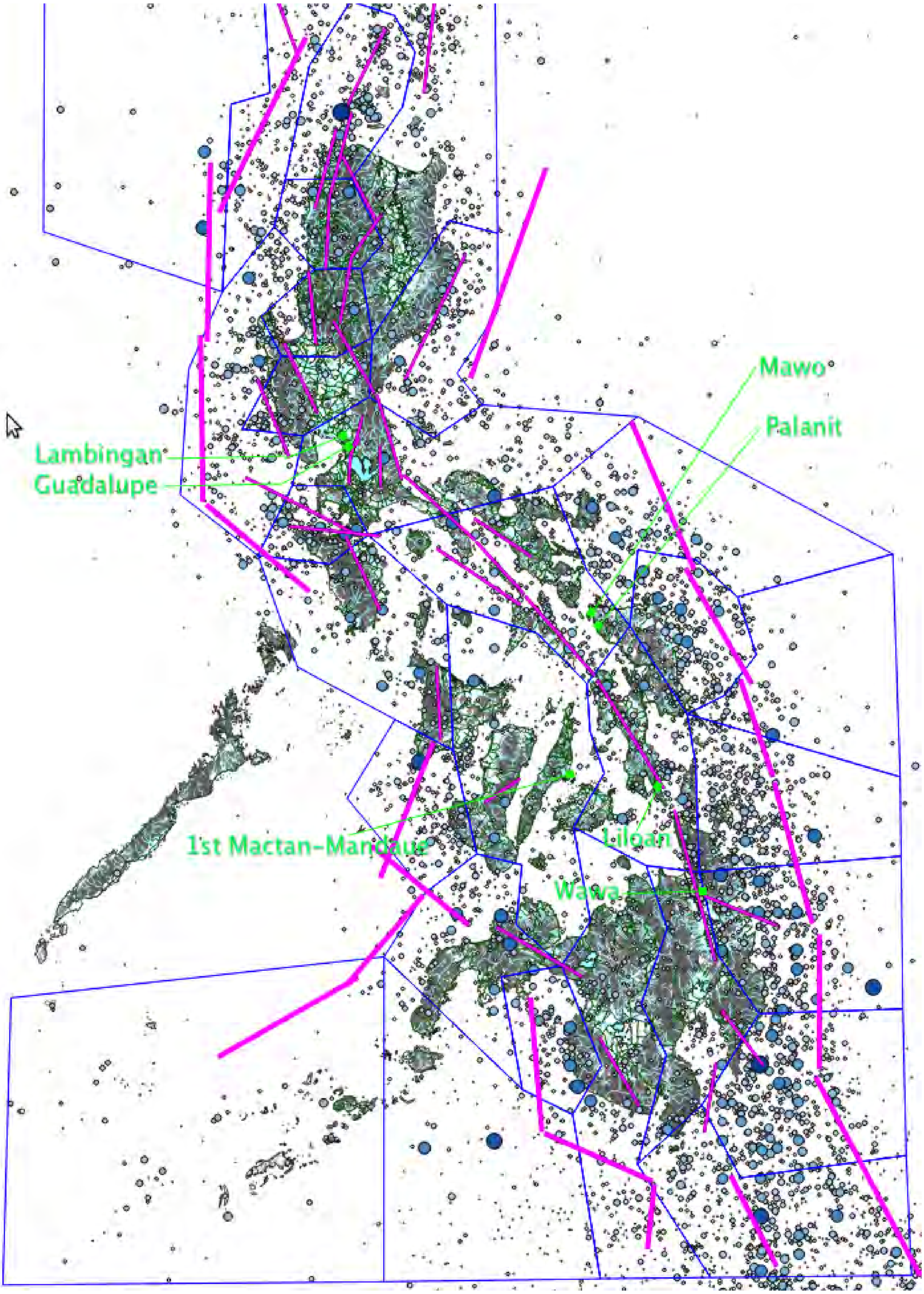
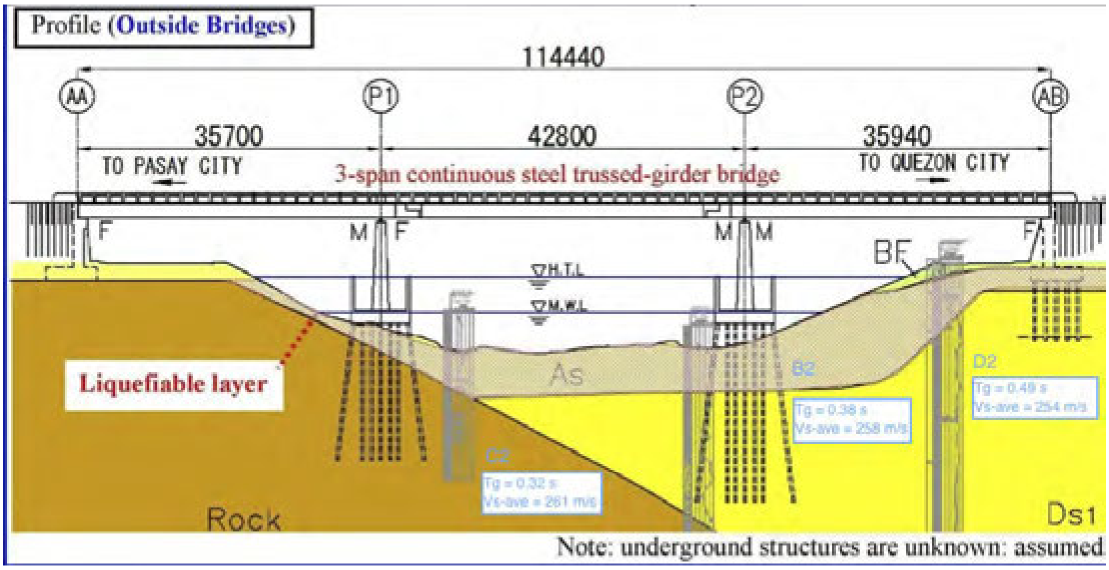


Figure 2B-4 Seismic source modeling used in this study
— fault models and background seismicity models
(also plotted are declustered earthquakes from 1907 to 2012)

(a) site plan



(b) site profile

Figure 2B-5 Guadalupe Bridge site location and data

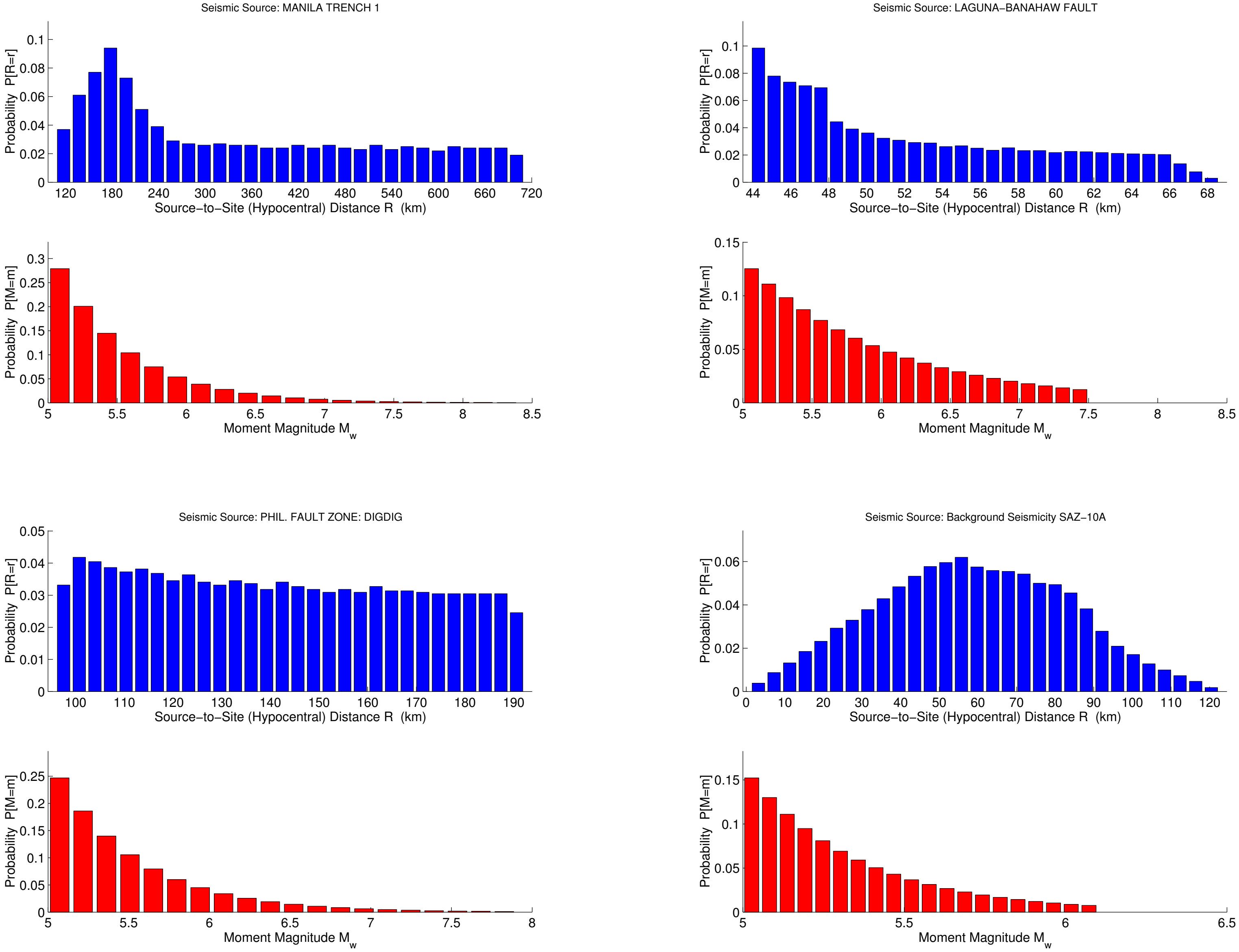


Figure 2B-6 Probability distribution functions for source-to-site distances and magnitudes of major seismic sources that potentially could produce significant ground shaking at Guadalupe Bridge site

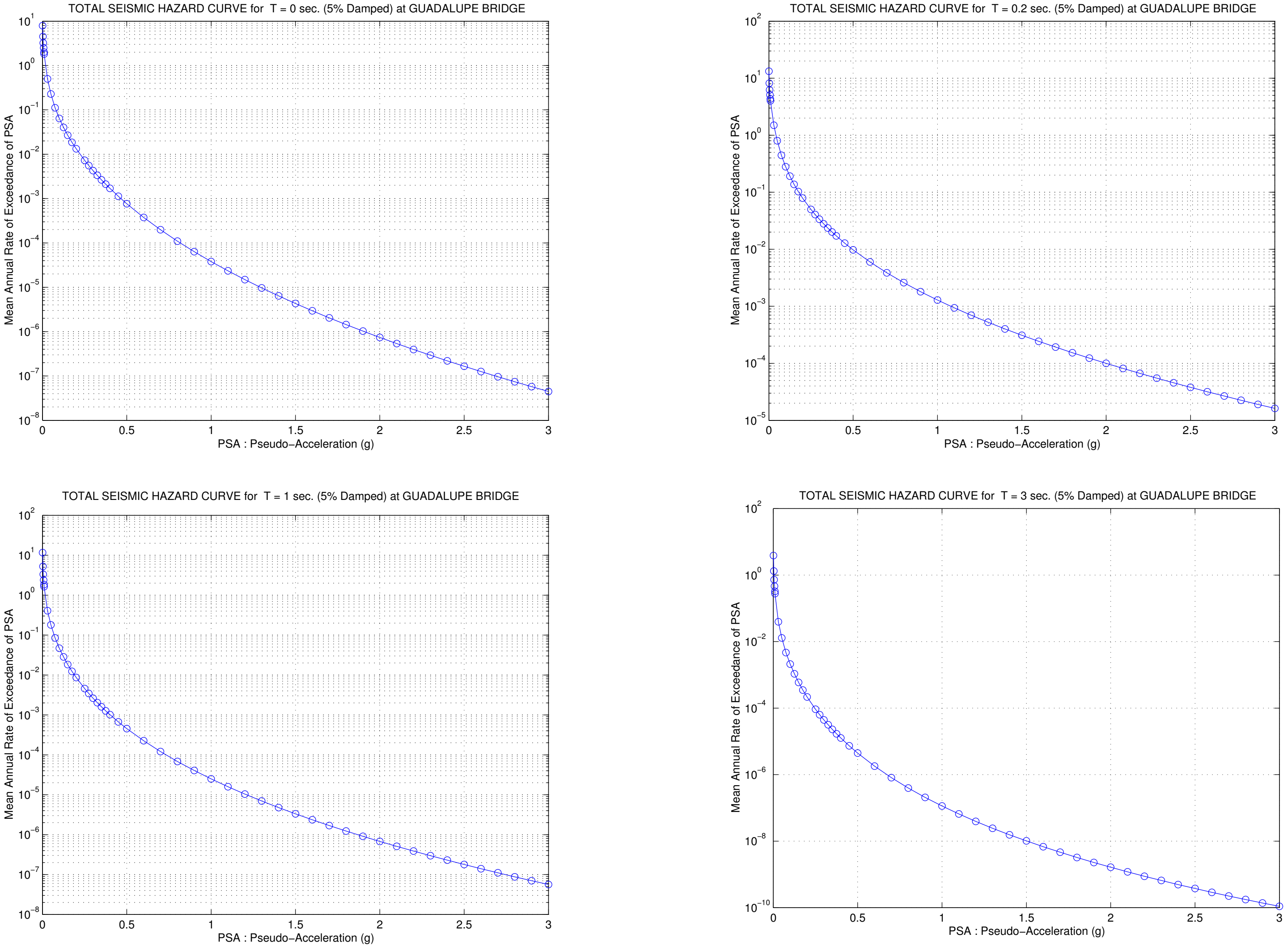


Figure 2B-7 Computations of total seismic hazard curves at several key periods (PGA, 0.02, 1., 3. sec) for Guadalupe Bridge site

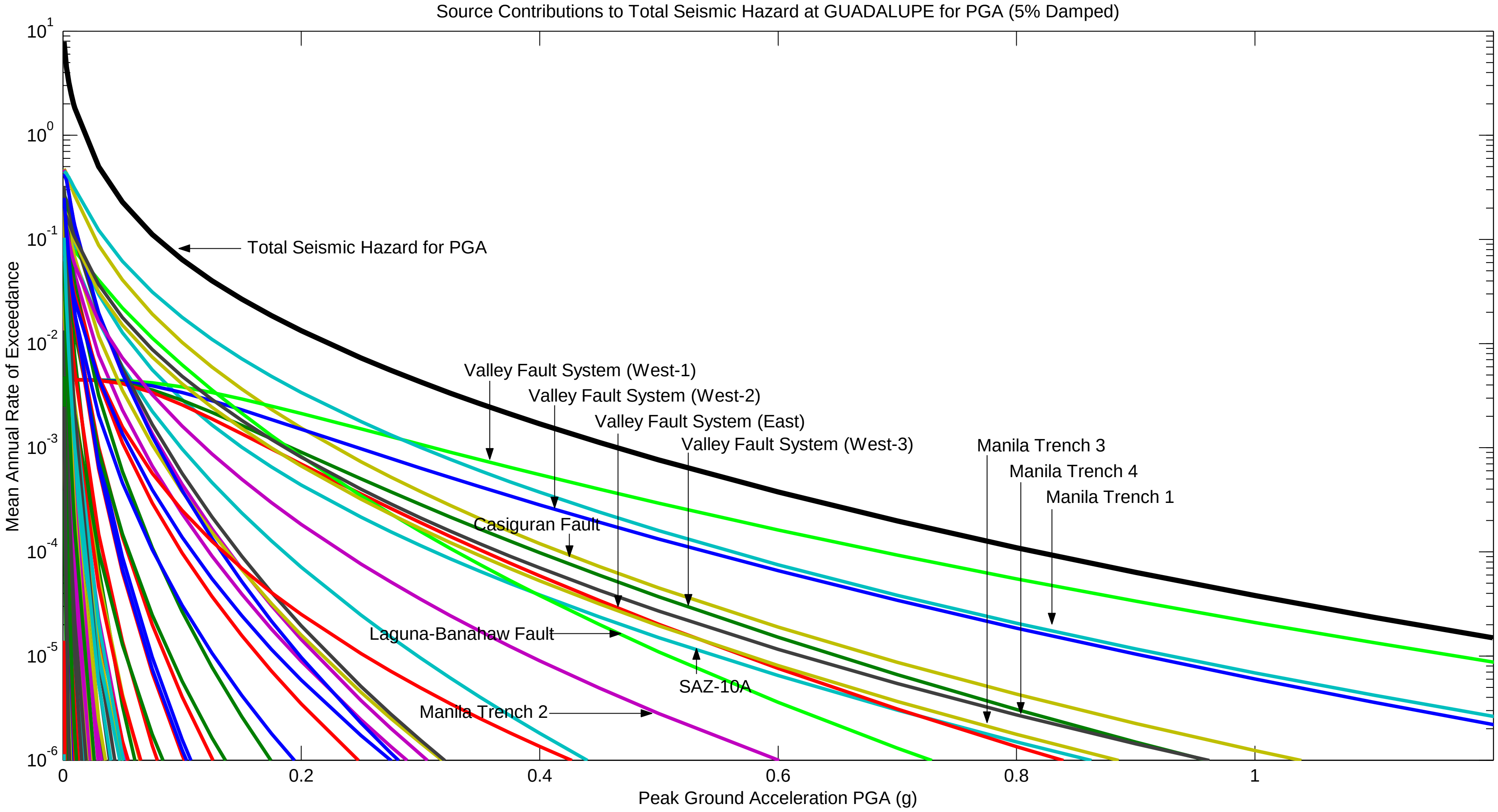


Figure 2B-8 Total seismic hazard curve for PGA showing major contributing sources

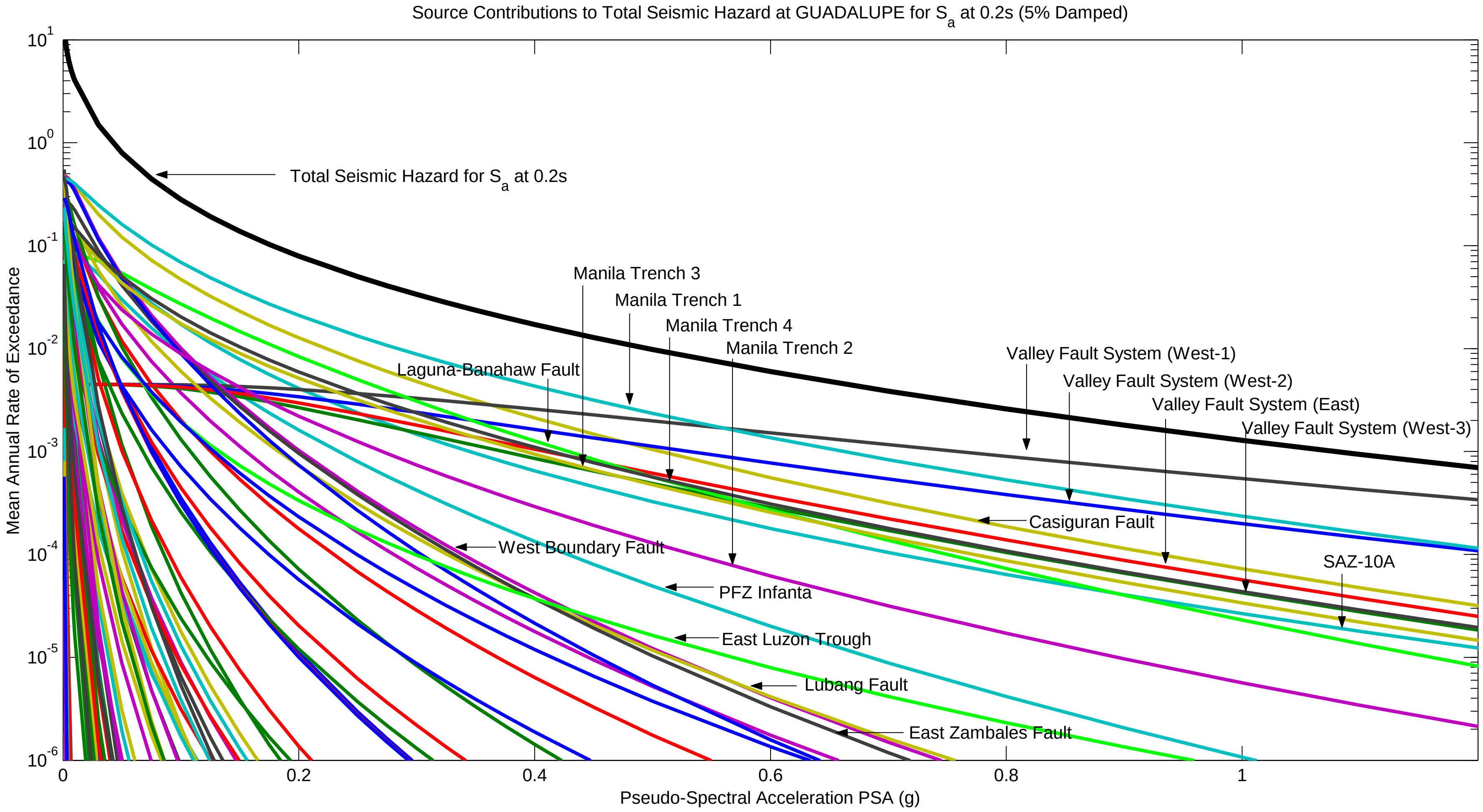


Figure 2B-9 Total seismic hazard curve for S_a at 0.2 sec showing major contributing sources

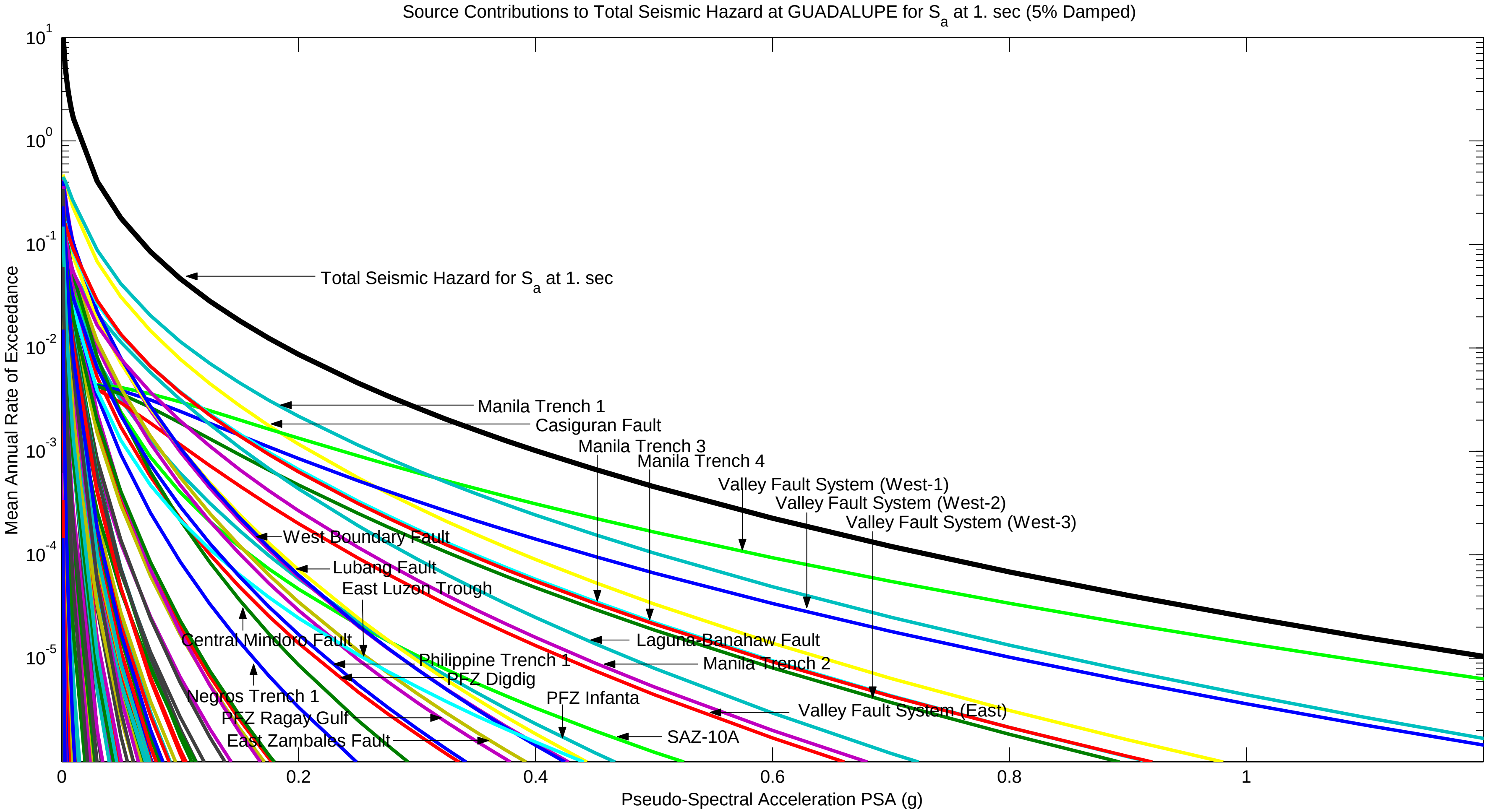


Figure 2B-10 Total seismic hazard curve for S_a at 1. sec showing major contributing sources

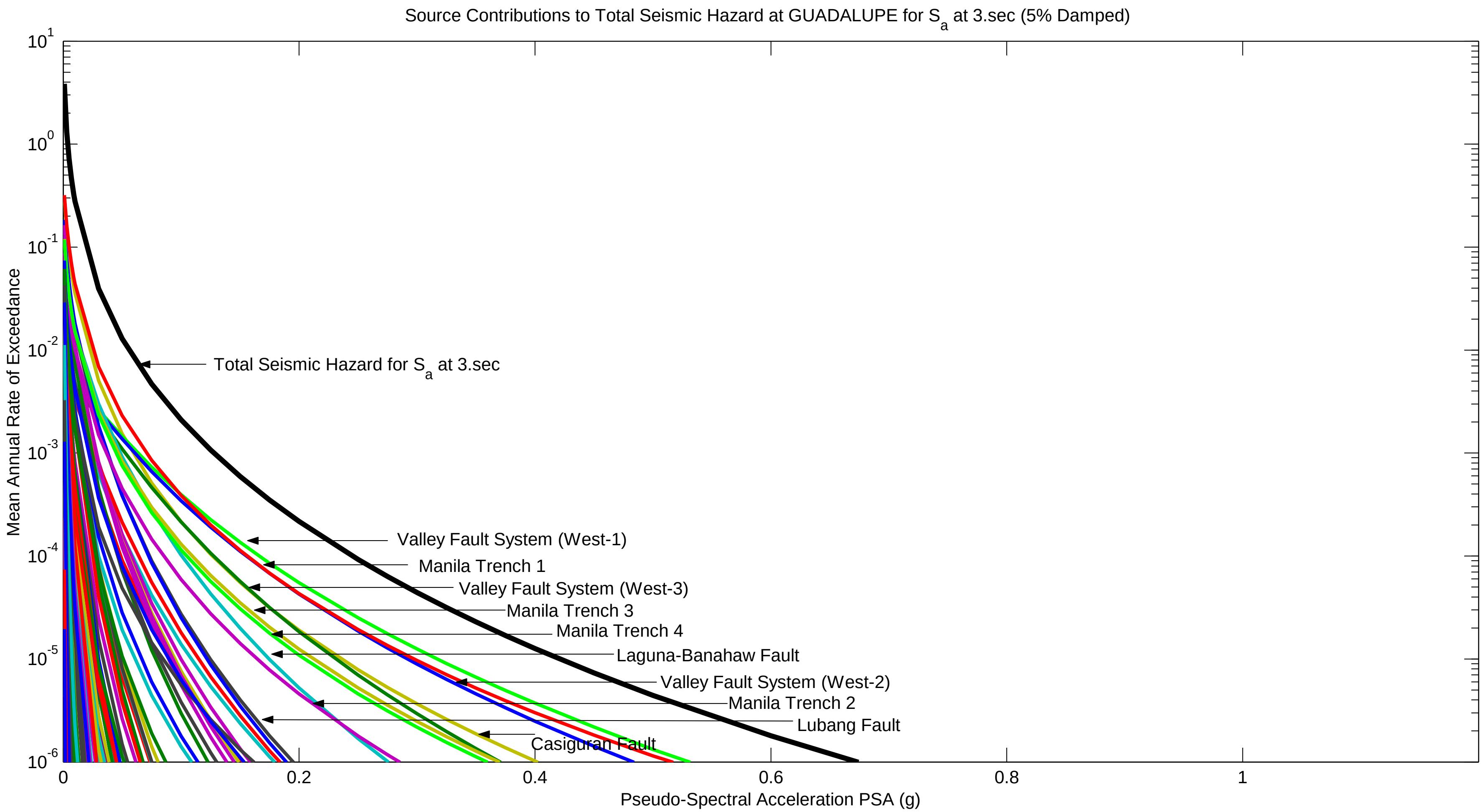
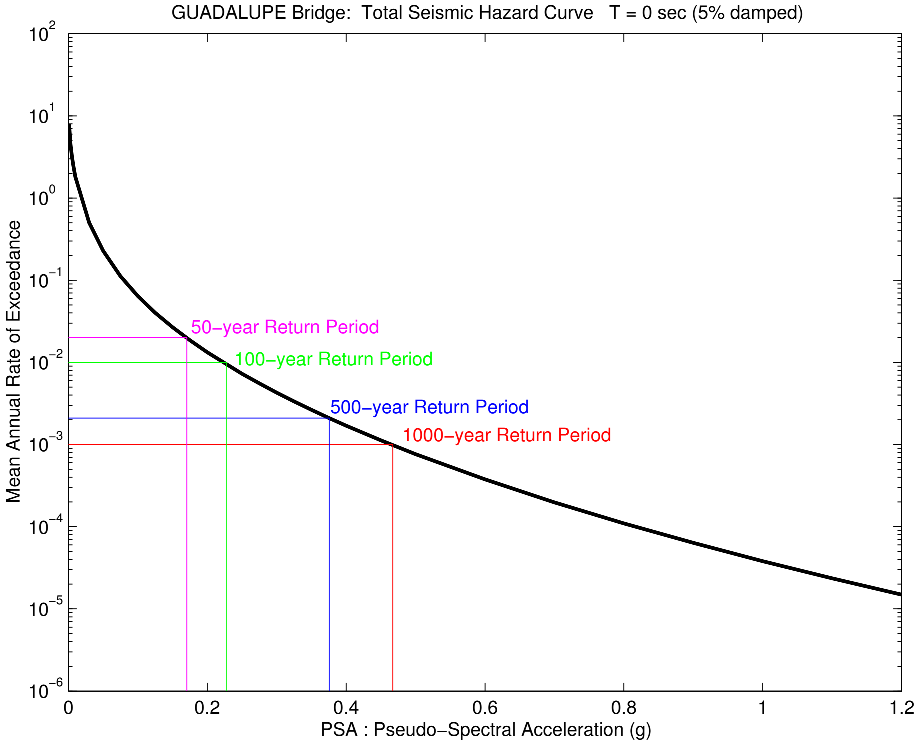
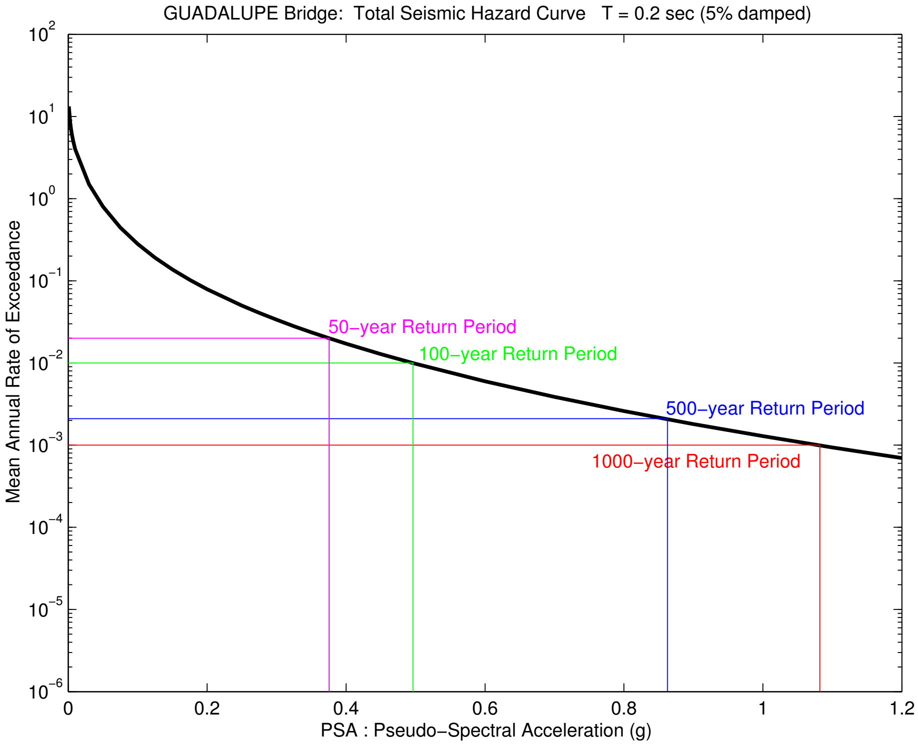


Figure 2B-11 Total seismic hazard curve for S_a at 3. sec showing major contributing sources

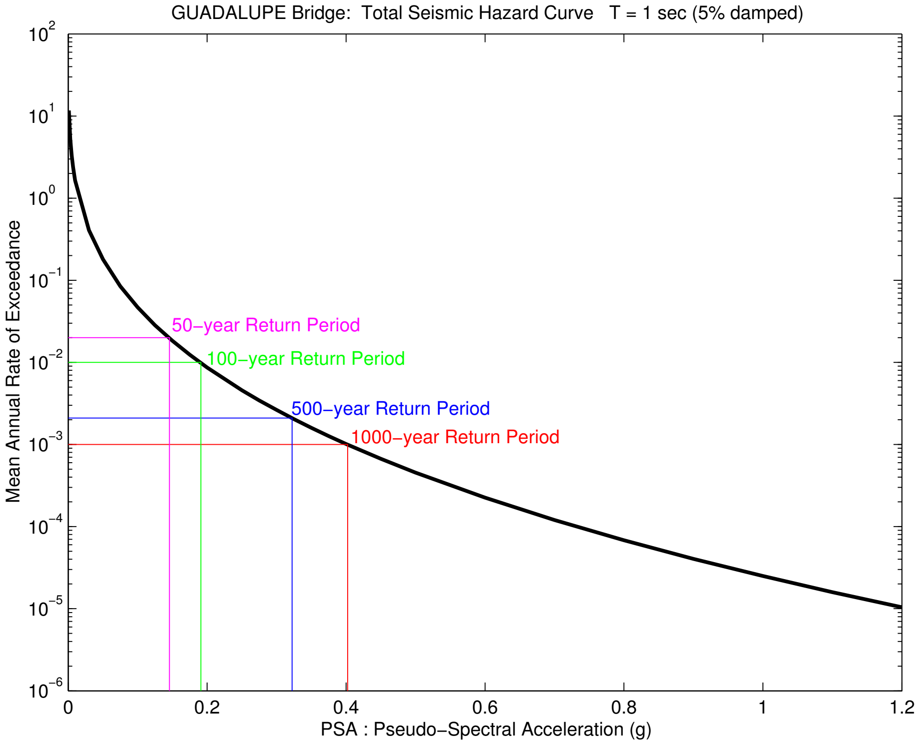


(a) PGA

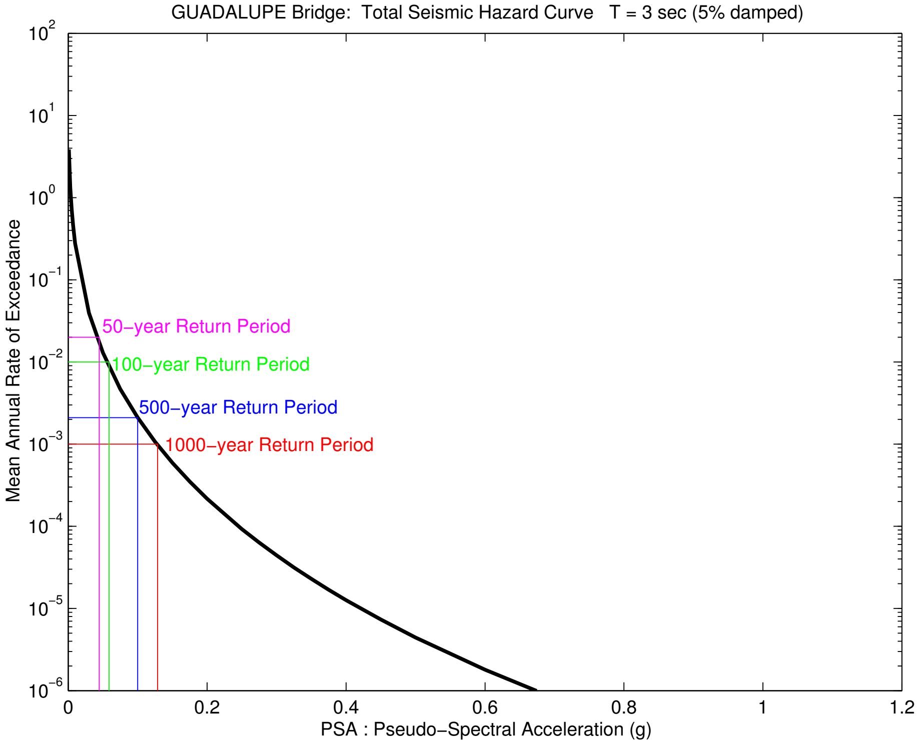


(b) S_a at 0.2 sec.

Figure 2B-12 (1/2) Exceedance rates at 50-year, 100-year, 500-year, and 1000-year return periods



(a) S_a at 1. sec.



(b) S_a at 3. sec.

Figure 2B-12 (2/2) Exceedance rates at 50-year, 100-year, 500-year, and 1000-year return periods

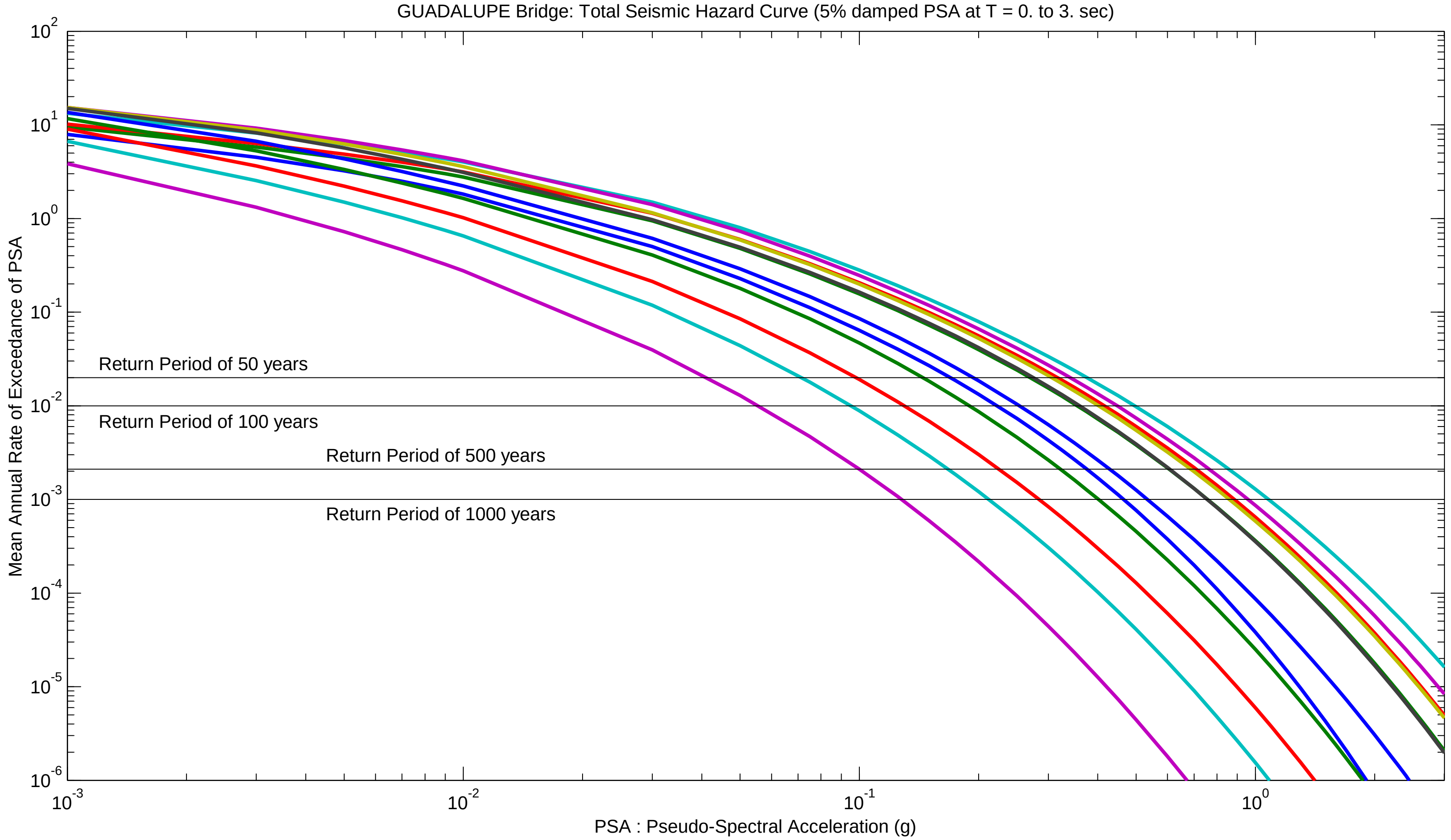


Figure 2B-13 Generating uniform-hazard (at return periods of 50, 100, 500, and 1000 years) spectral values for Guadalupe site

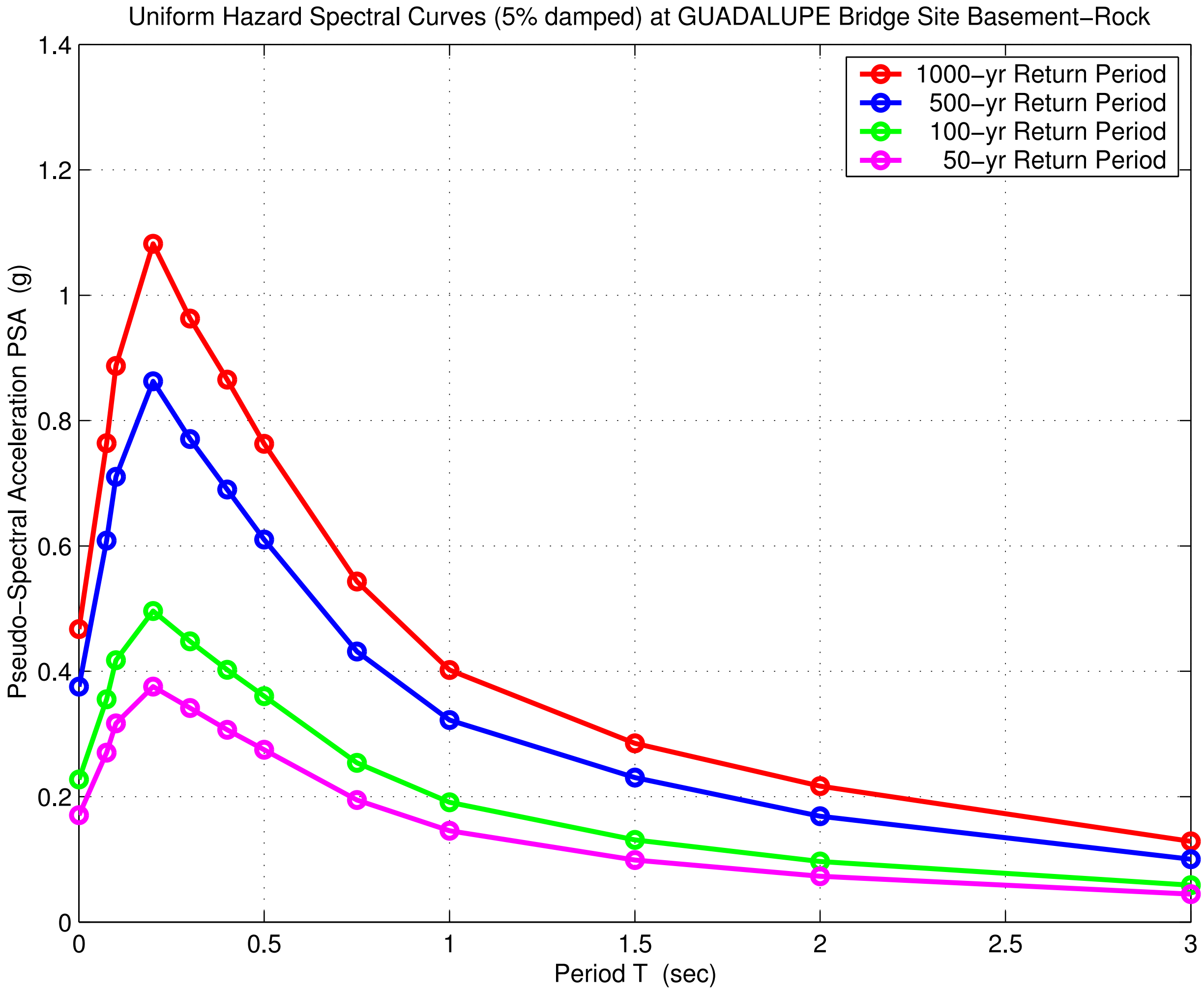


Figure 2B-14 Uniform hazard spectral curve for Guadalupe Bridge site at basement-rock level

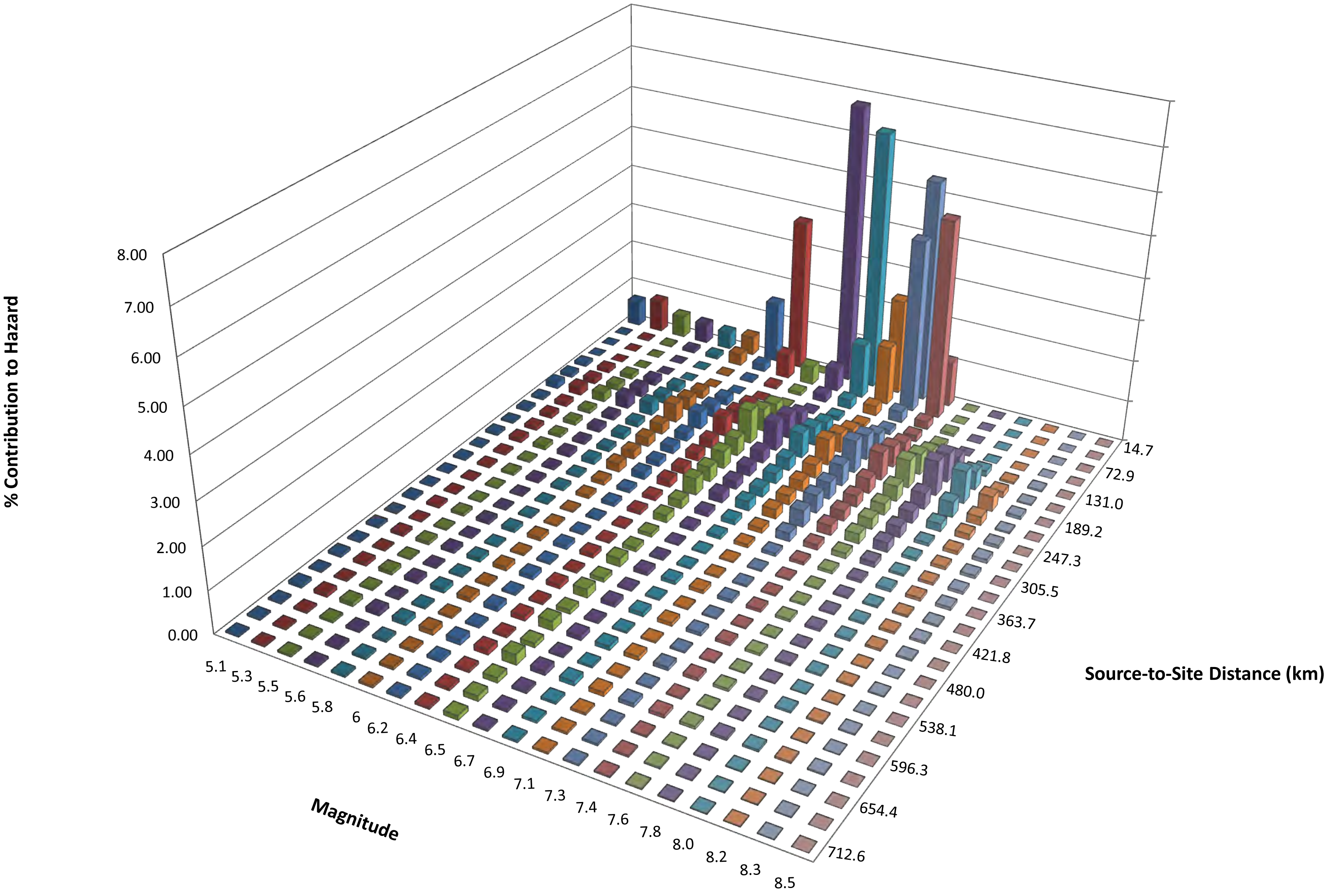


Figure 2B-15 Seismic hazard deaggregation for PGA (50-year return period): Guadalupe Bridge site

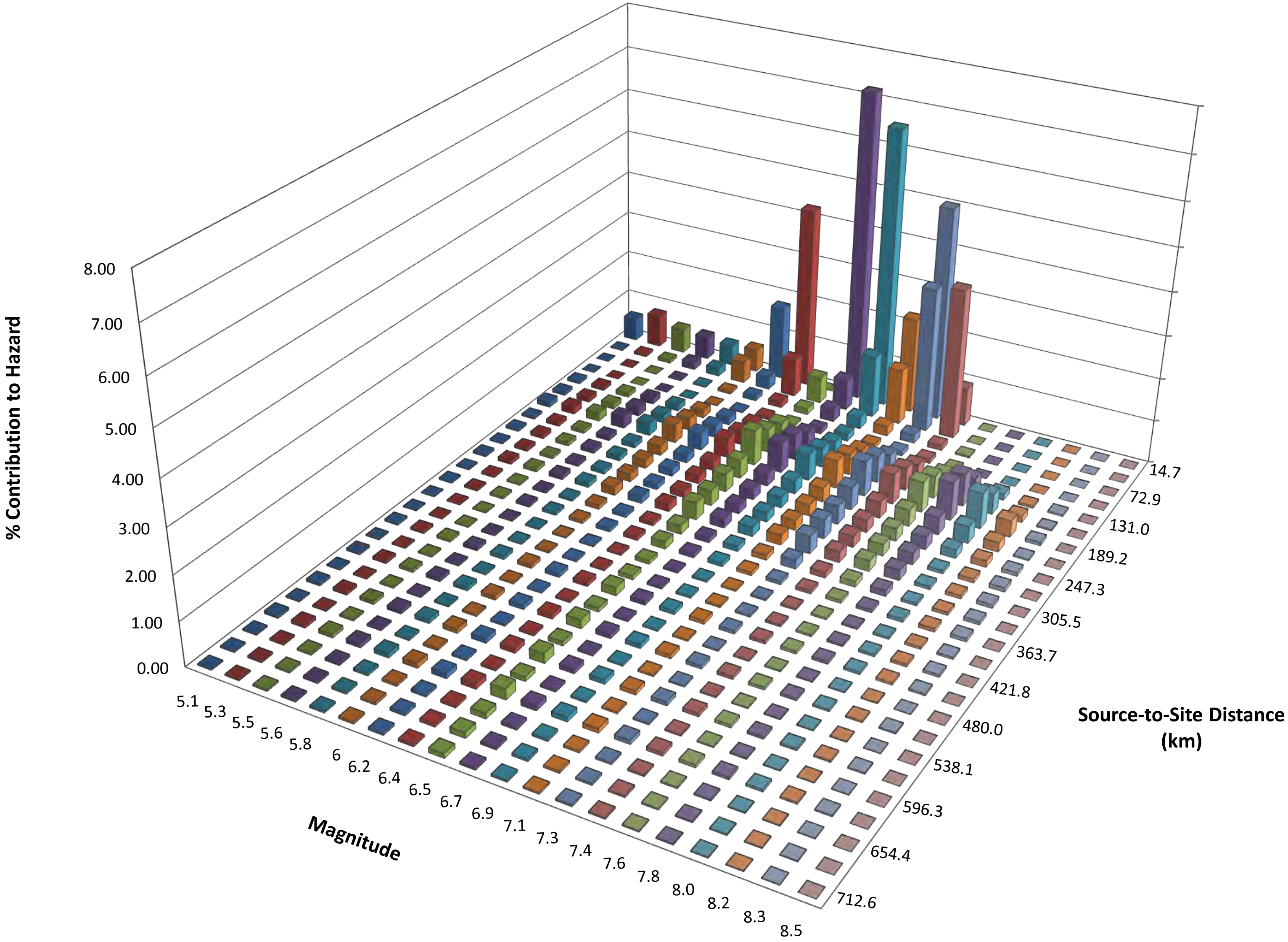


Figure 2B-16 Seismic hazard deaggregation for S_a at 0.2 sec (50-year return period): Guadalupe Bridge site

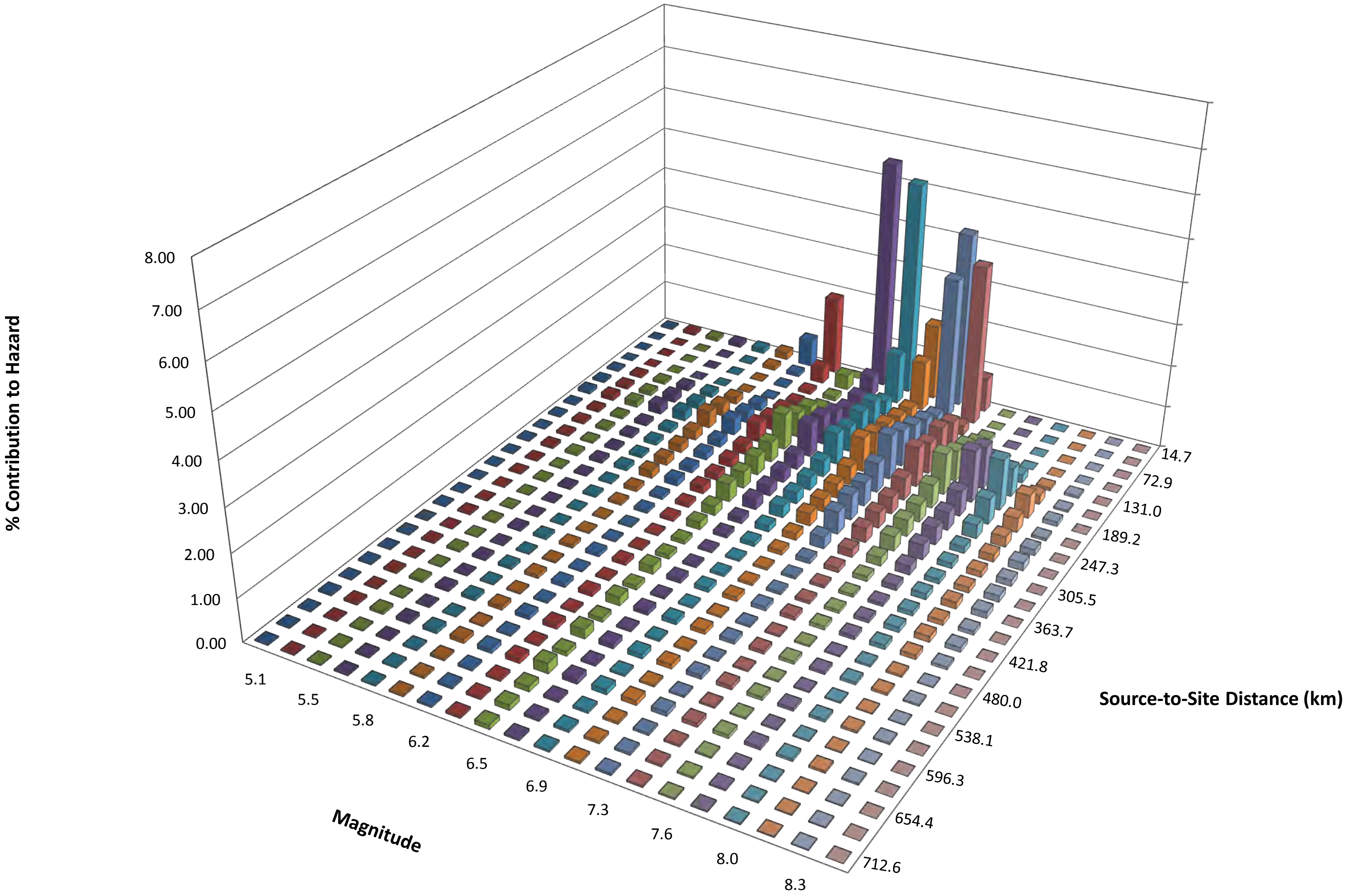


Figure 2B-17 Seismic hazard deaggregation for S_a at 1. sec (50-year return period): Guadalupe Bridge site

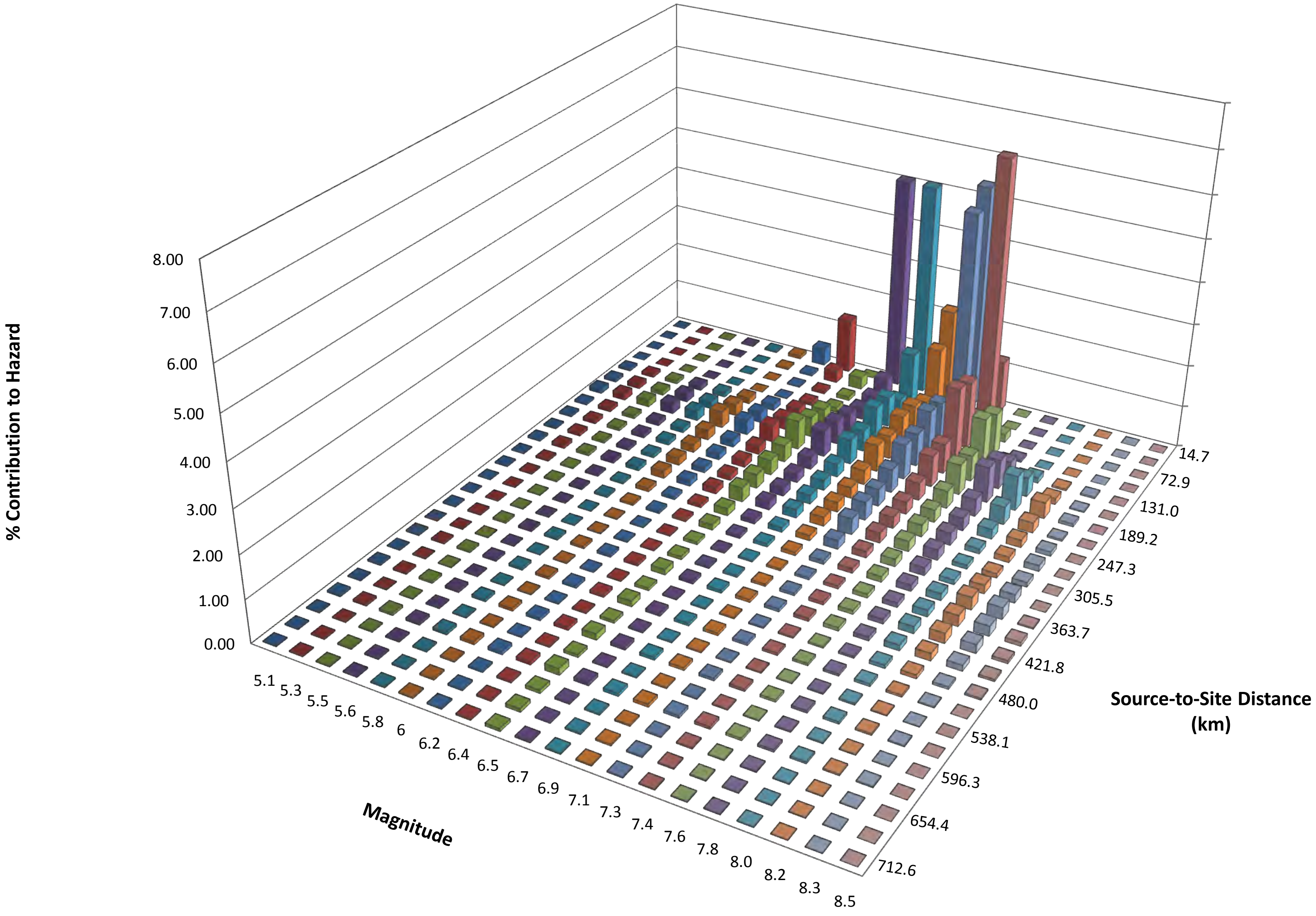
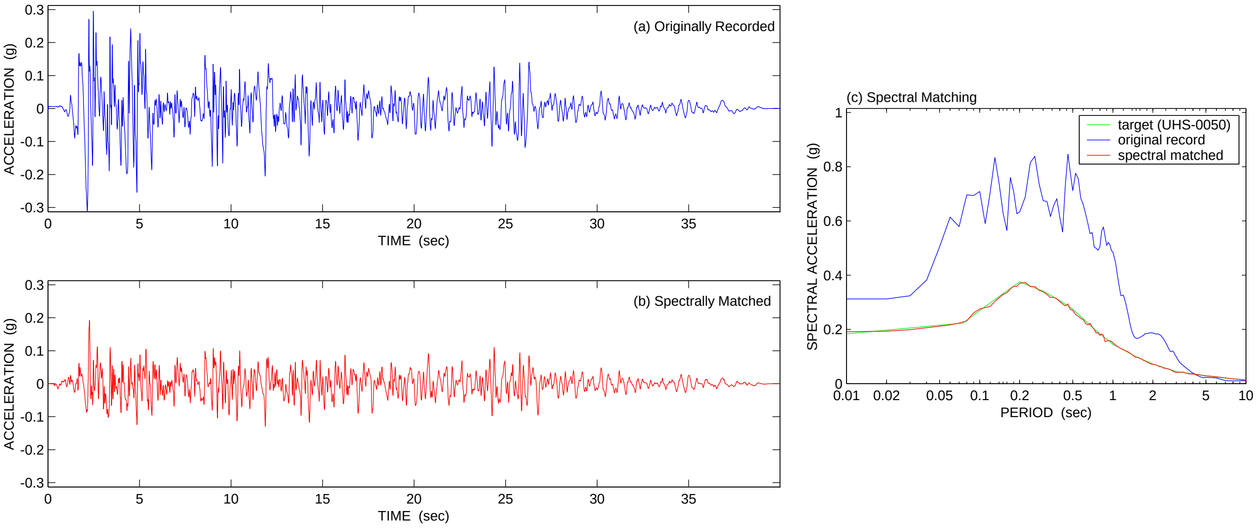
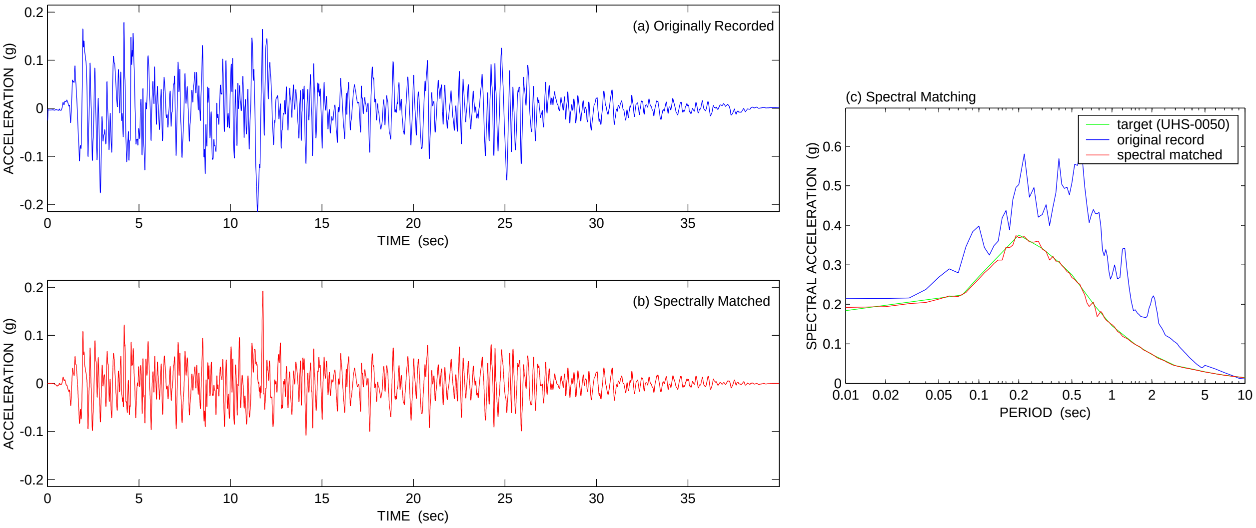


Figure 2B-18 Seismic hazard deaggregation for S_a at 3. sec (50-year return period): Guadalupe Bridge site

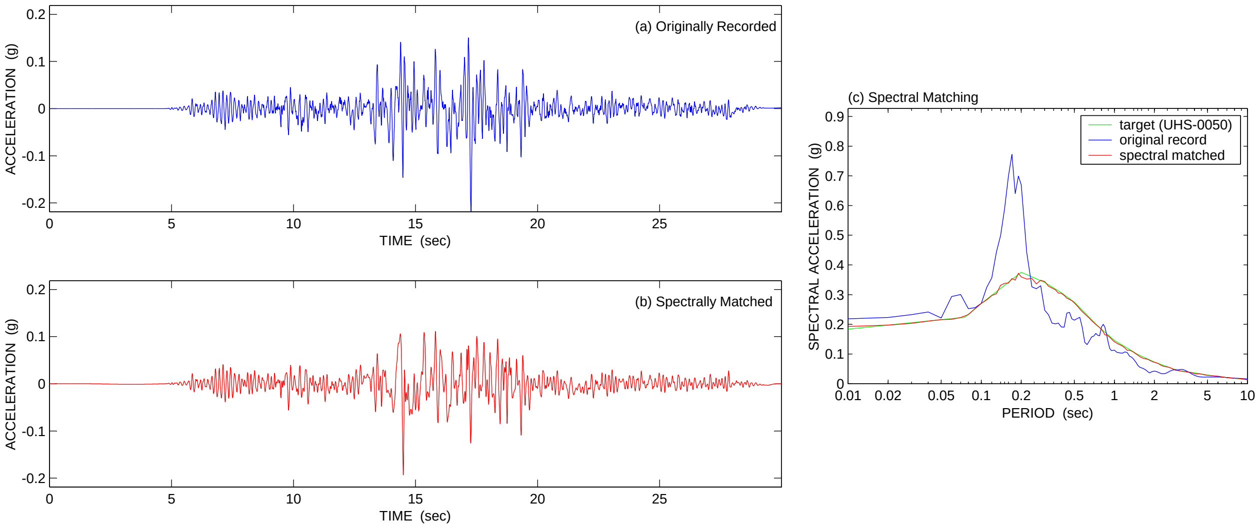


(a) RP0050-Eq1X

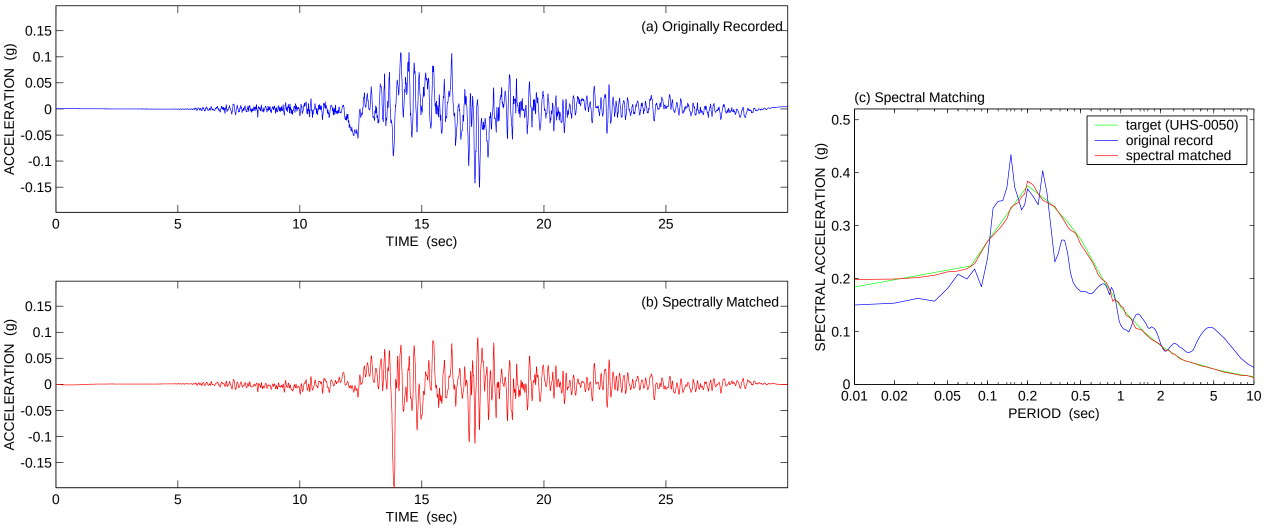


(b) RP0050-Eq1Y

Figure 2B-19 (1/7) Selection and spectral matching of earthquakes for 50-year return period

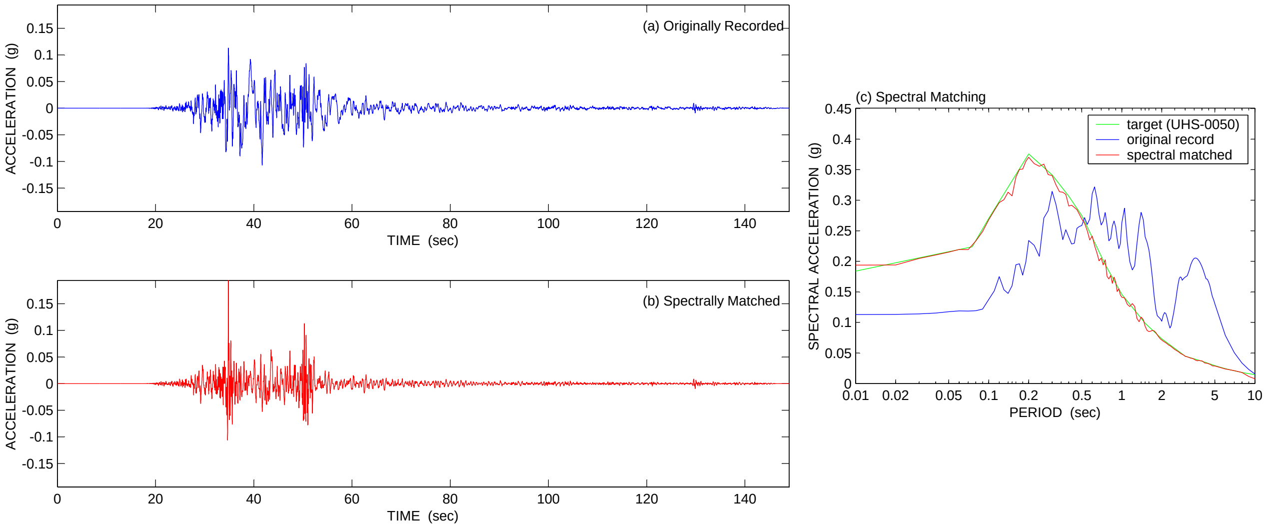


(a) RP0050-Eq2X

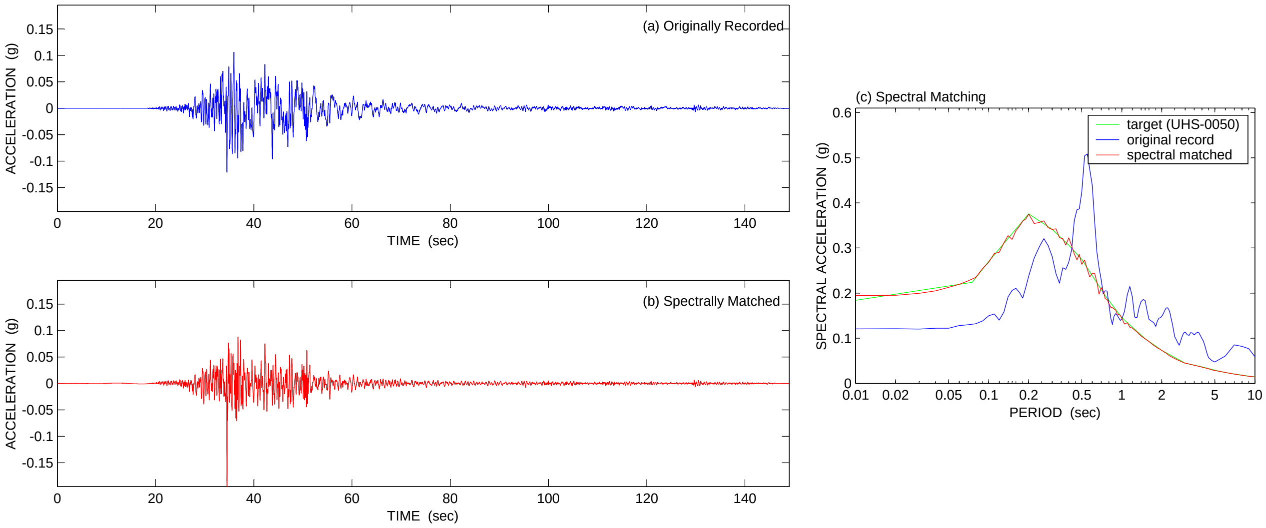


(b) RP0050-Eq2Y

Figure 2B-19 (2/7) Selection and spectral matching of earthquakes for 50-year return period

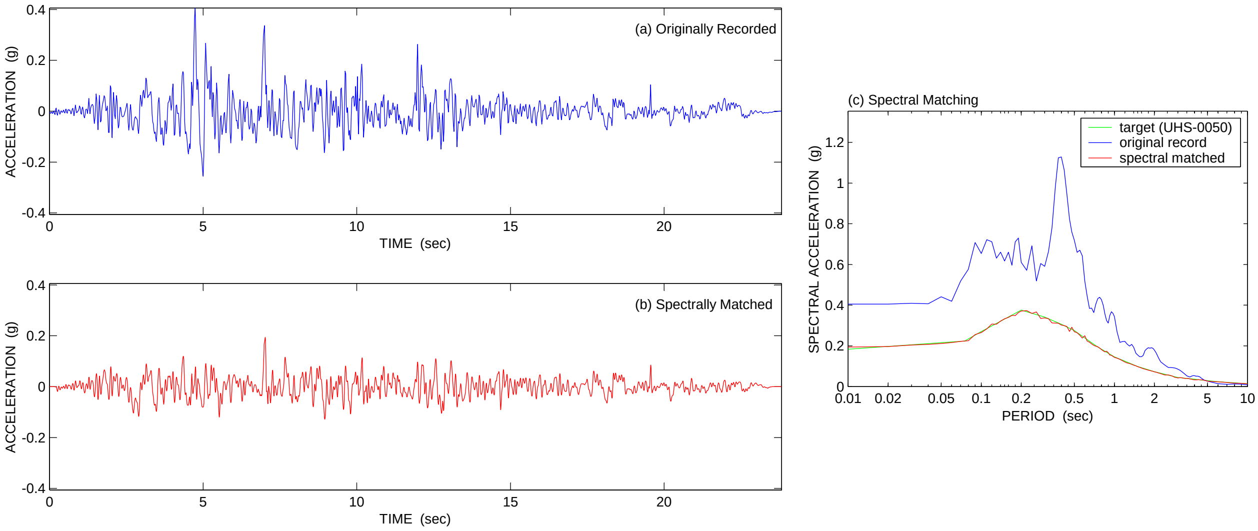


(a) RP0050-Eq3X

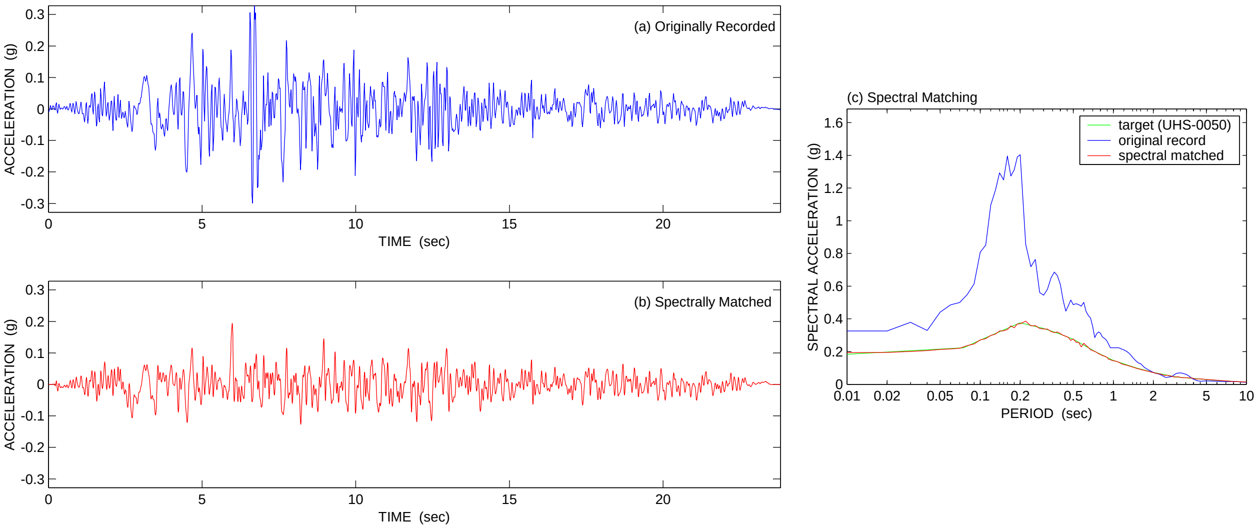


(b) RP0050-Eq3Y

Figure 2B-19 (3/7) Selection and spectral matching of earthquakes for 50-year return period

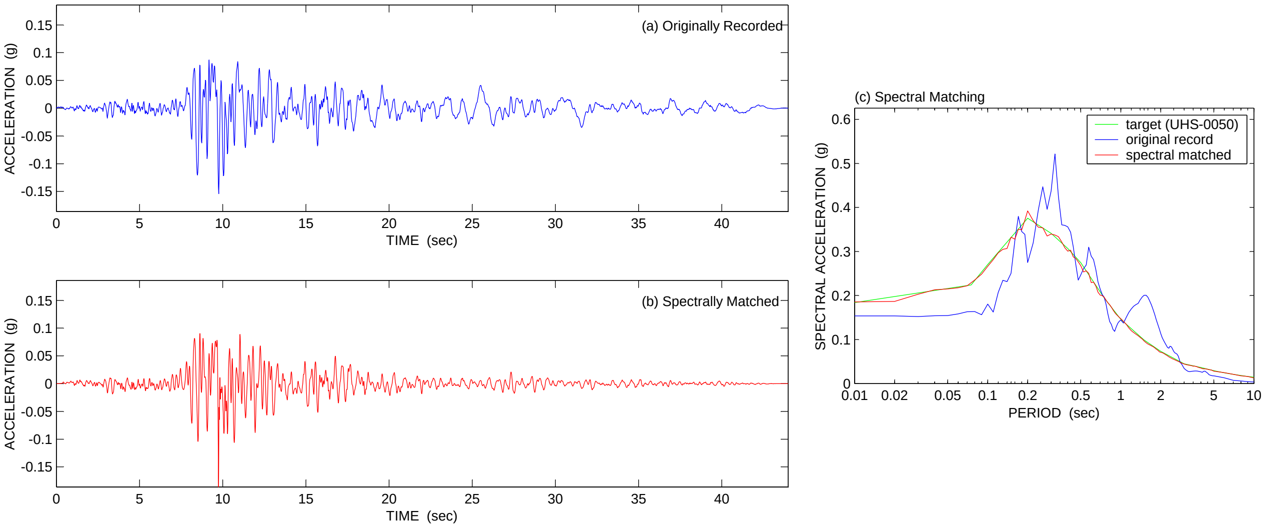


(a) RP0050-Eq4X

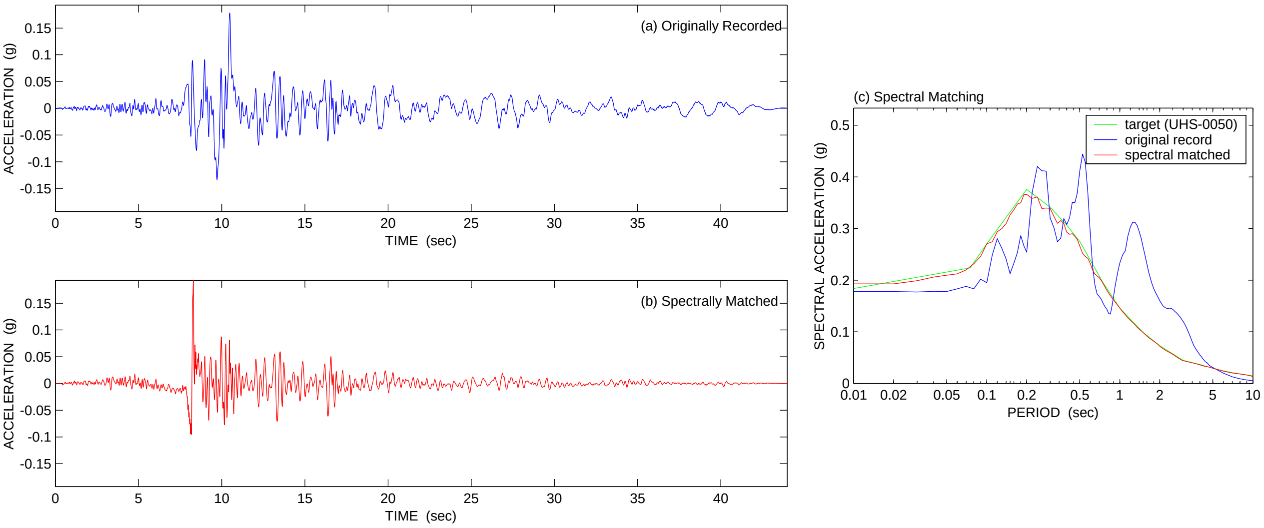


(b) RP0050-Eq4Y

Figure 2B-19 (4/7) Selection and spectral matching of earthquakes for 50-year return period

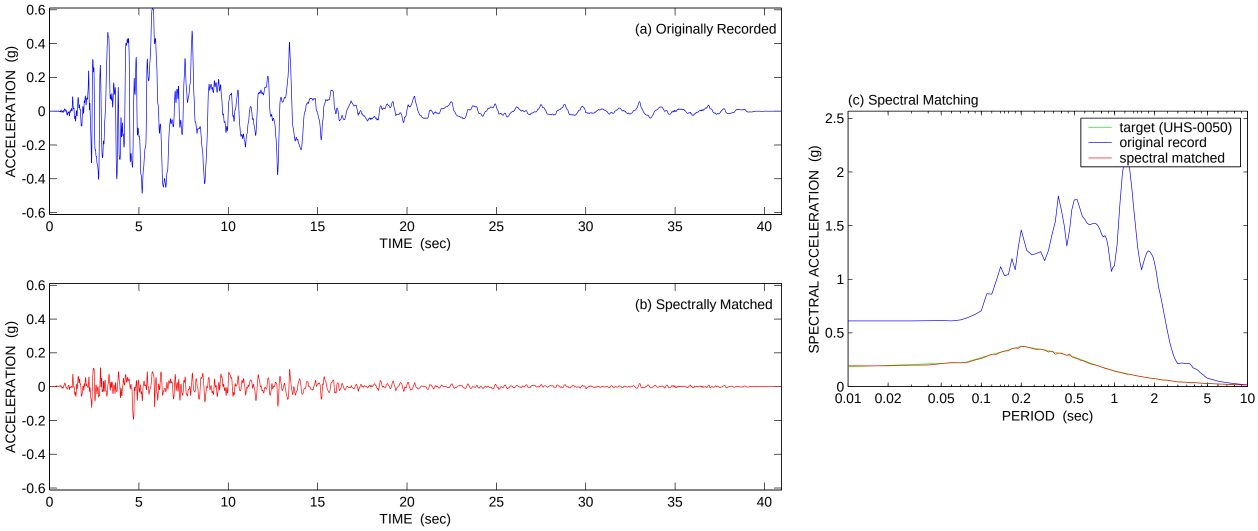


(a) RP0050-Eq5X

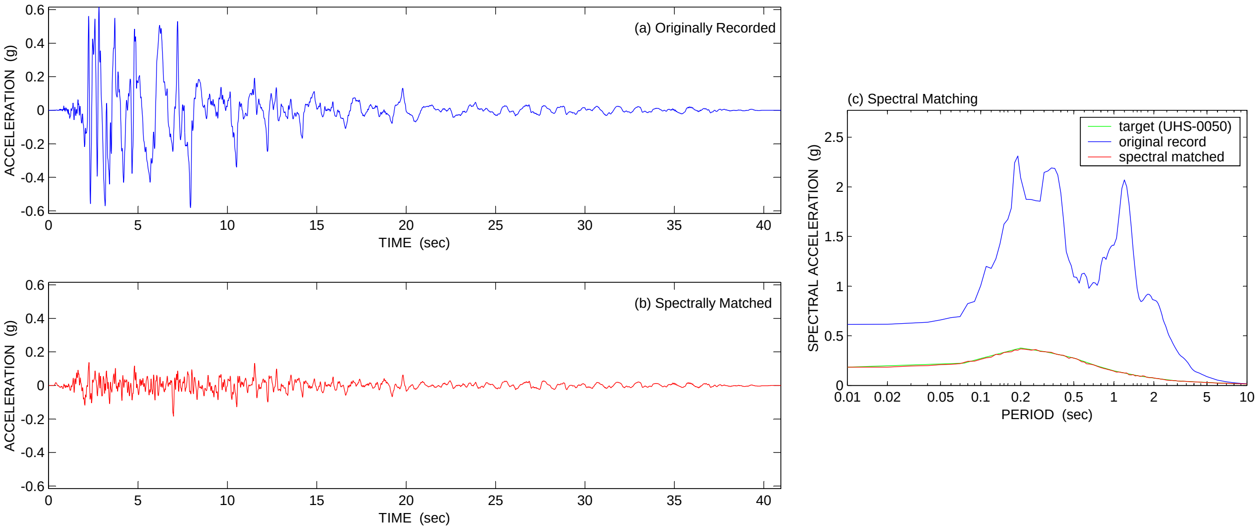


(b) RP0050-Eq5Y

Figure 2B-19 (5/7) Selection and spectral matching of earthquakes for 50-year return period

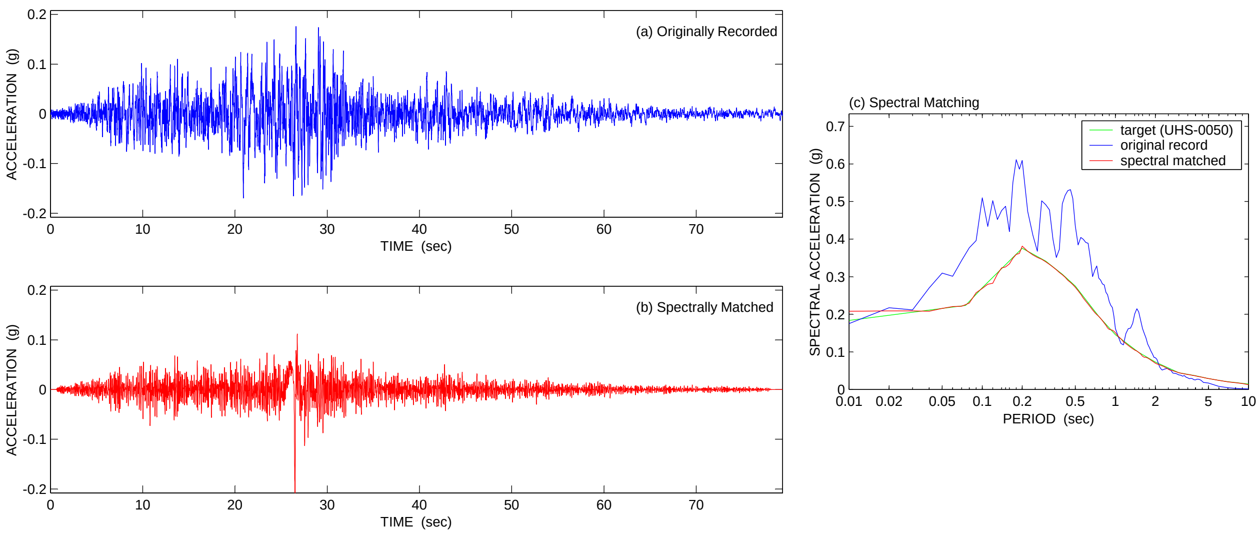


(a) RP0050-Eq6X

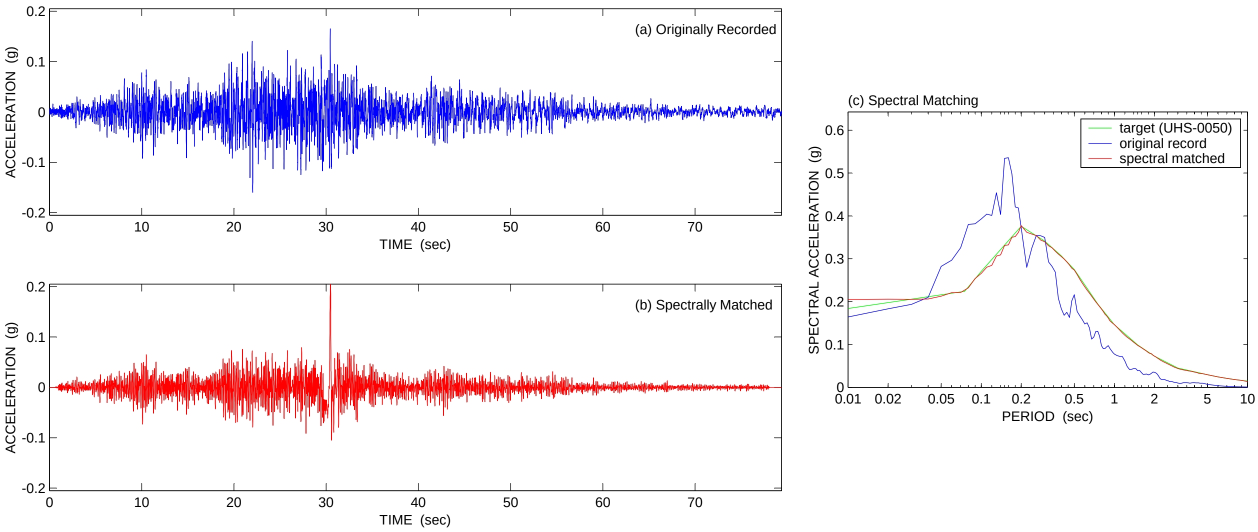


(b) RP0050-Eq6Y

Figure 2B-19 (6/7) Selection and spectral matching of earthquakes for 50-year return period



(a) RP0050-Eq7X



(b) RP0050-Eq7Y

Figure 2B-19 (7/7) Selection and spectral matching of earthquakes for 50-year return period

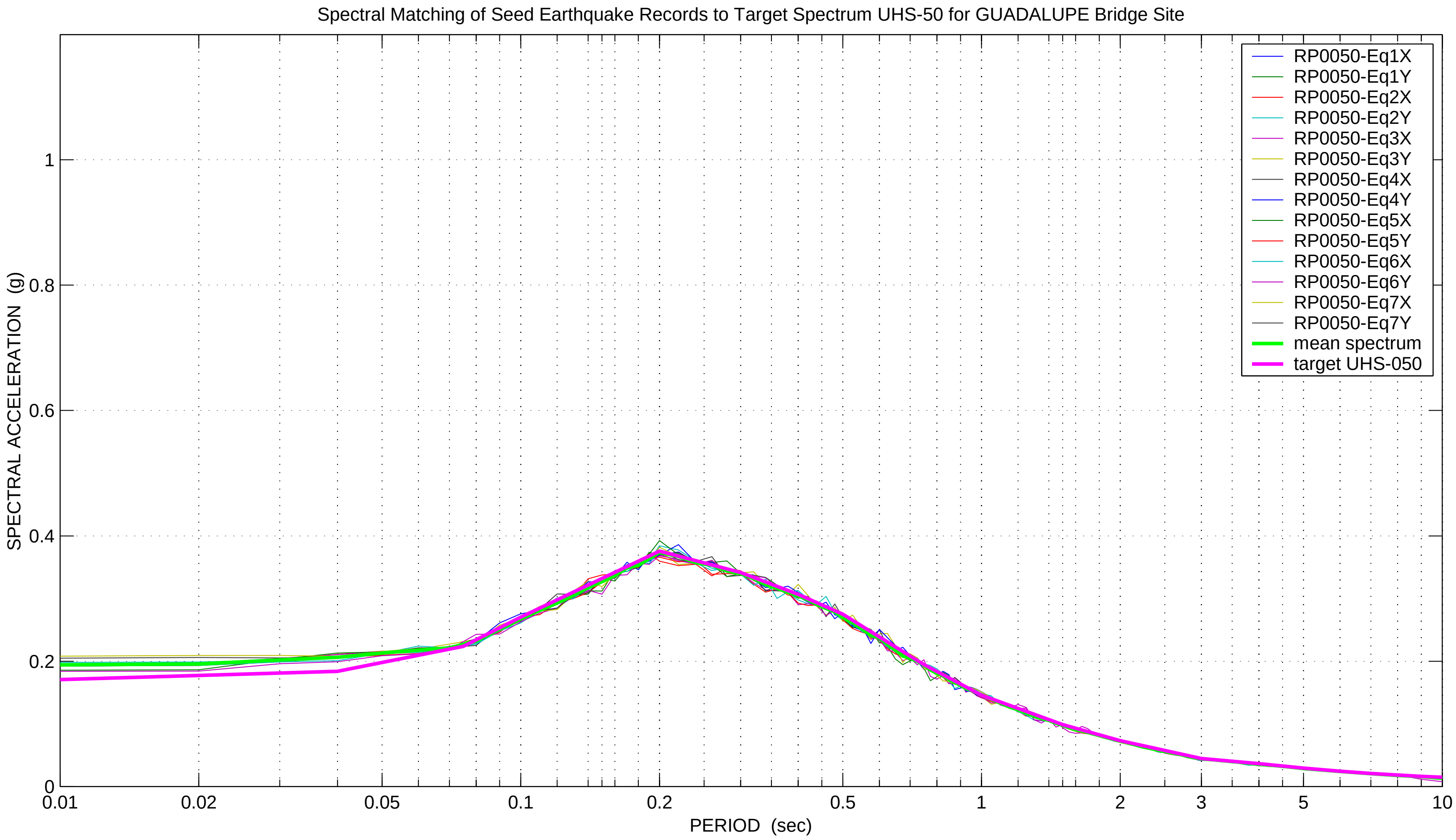


Figure 2B-20 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 50-year return period as compared to the target spectrum UHS-50

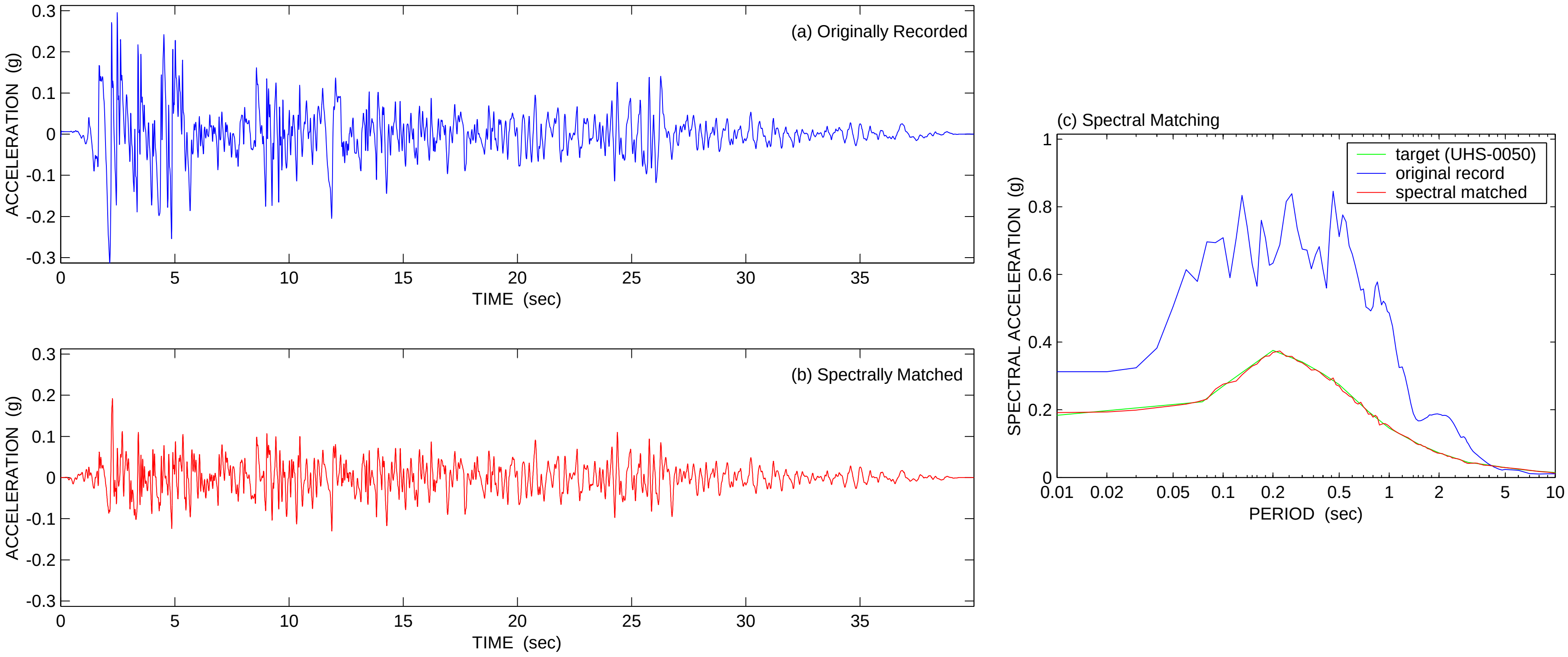


Figure 2B-21 Spectral matching ↝ input earthquake ground acceleration time-history RP0050-Eq1X
Earthquake Ground Motion Suite (50-year Return Period)
(a) seed motion (originally recorded ground acceleration time history)
(b) spectrally matched ground acceleration time history
(c) comparison of response spectra (target, seed, spectrally matched)

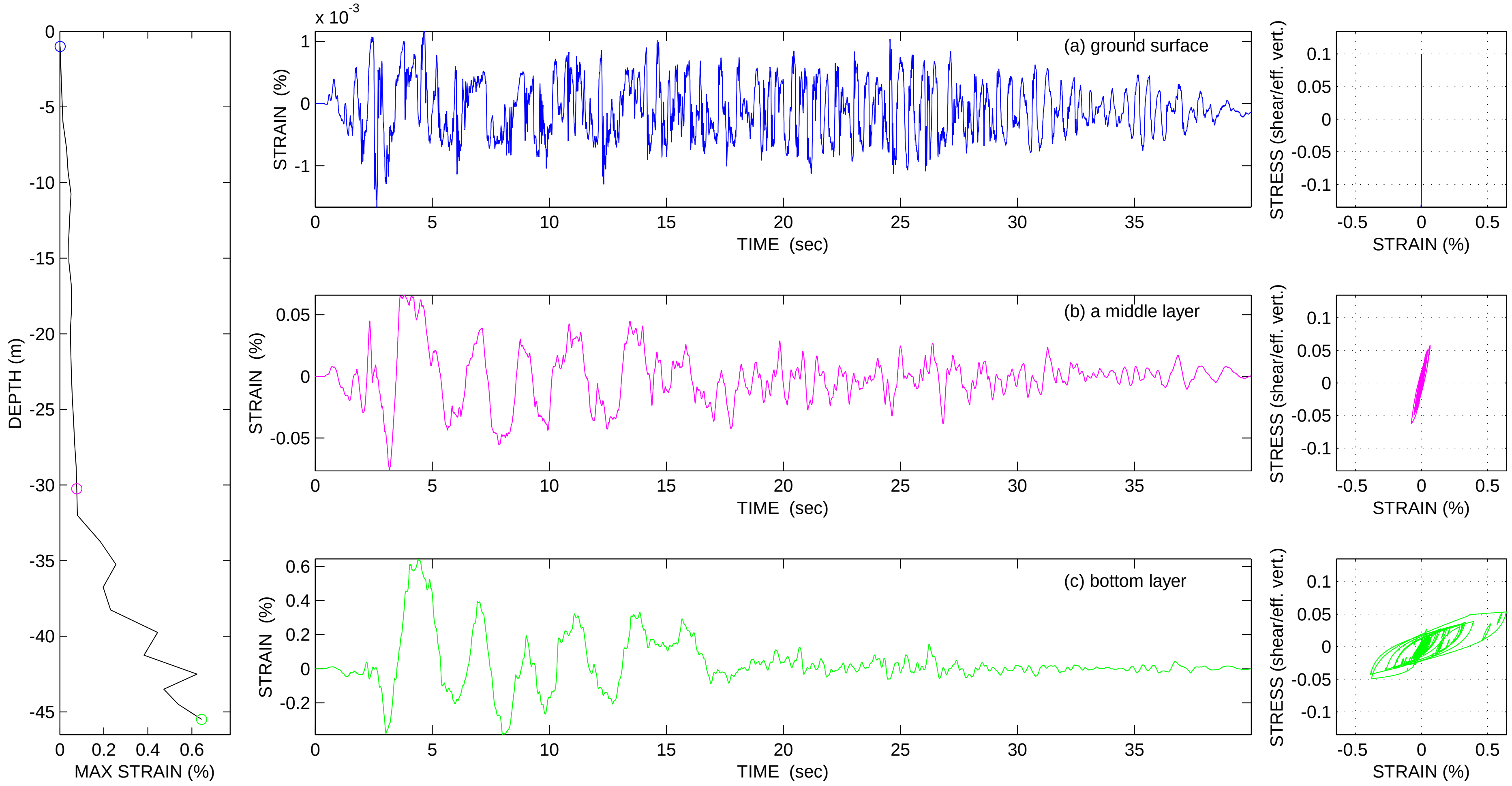


Figure 2B-22 (1/2) Nonlinear site response analysis of Guadalupe bridge site subjected to input base motion RP0050-Eq1X — maximum strain profile; strain time-histories; stress-strain hystereses — (a) at ground surface; (b) at about 30m depth; (c) at the layer atop base-rock

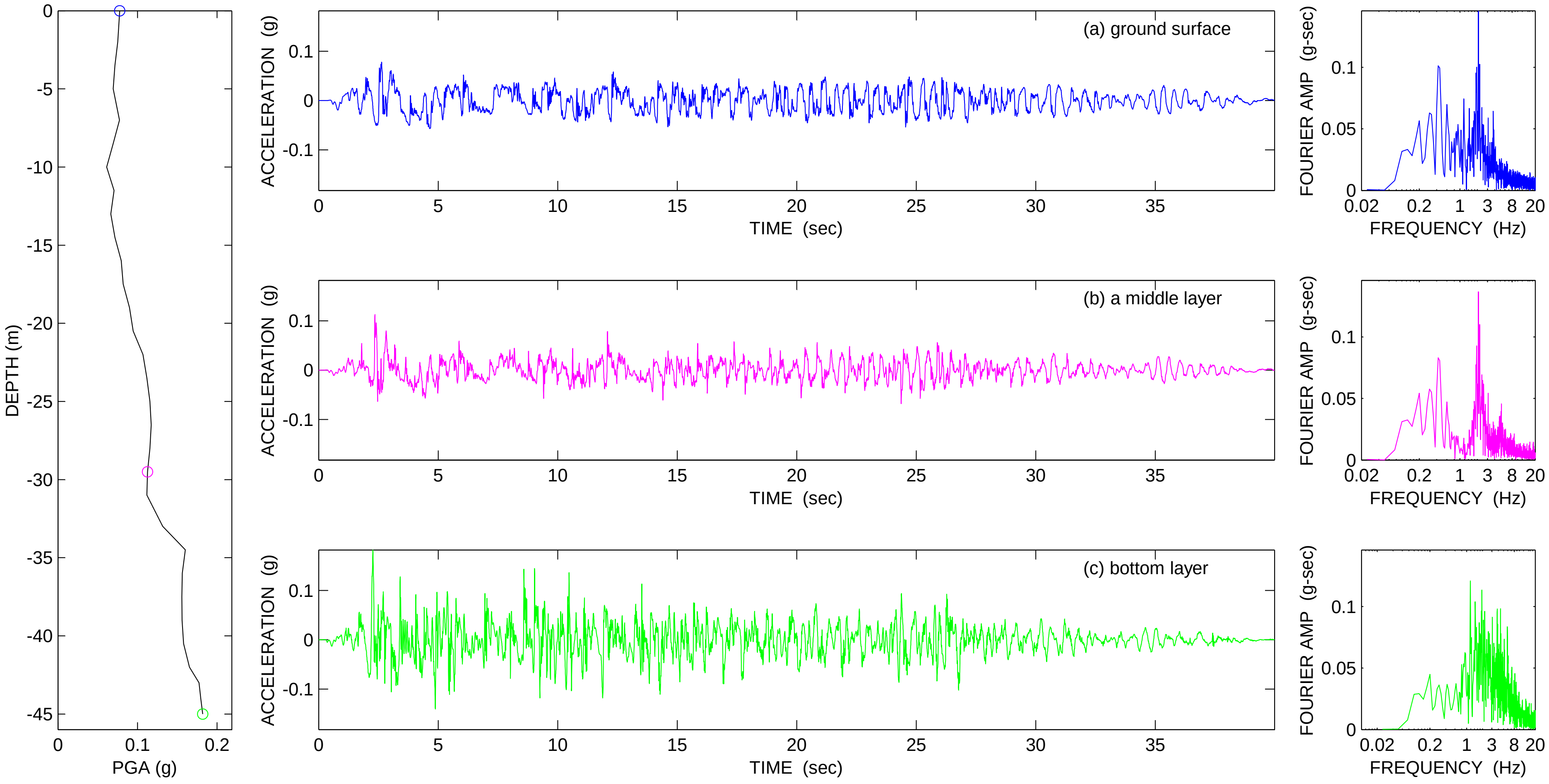


Figure 2B-22 (2/2) Nonlinear site response analysis of Guadalupe bridge site subjected to input base motion RP0050-Eq1X — PGA profile; ground acceleration time-histories; Fourier amplitude spectra — (a) at ground surface; (b) at about 30m depth; (c) at the layer atop base-rock

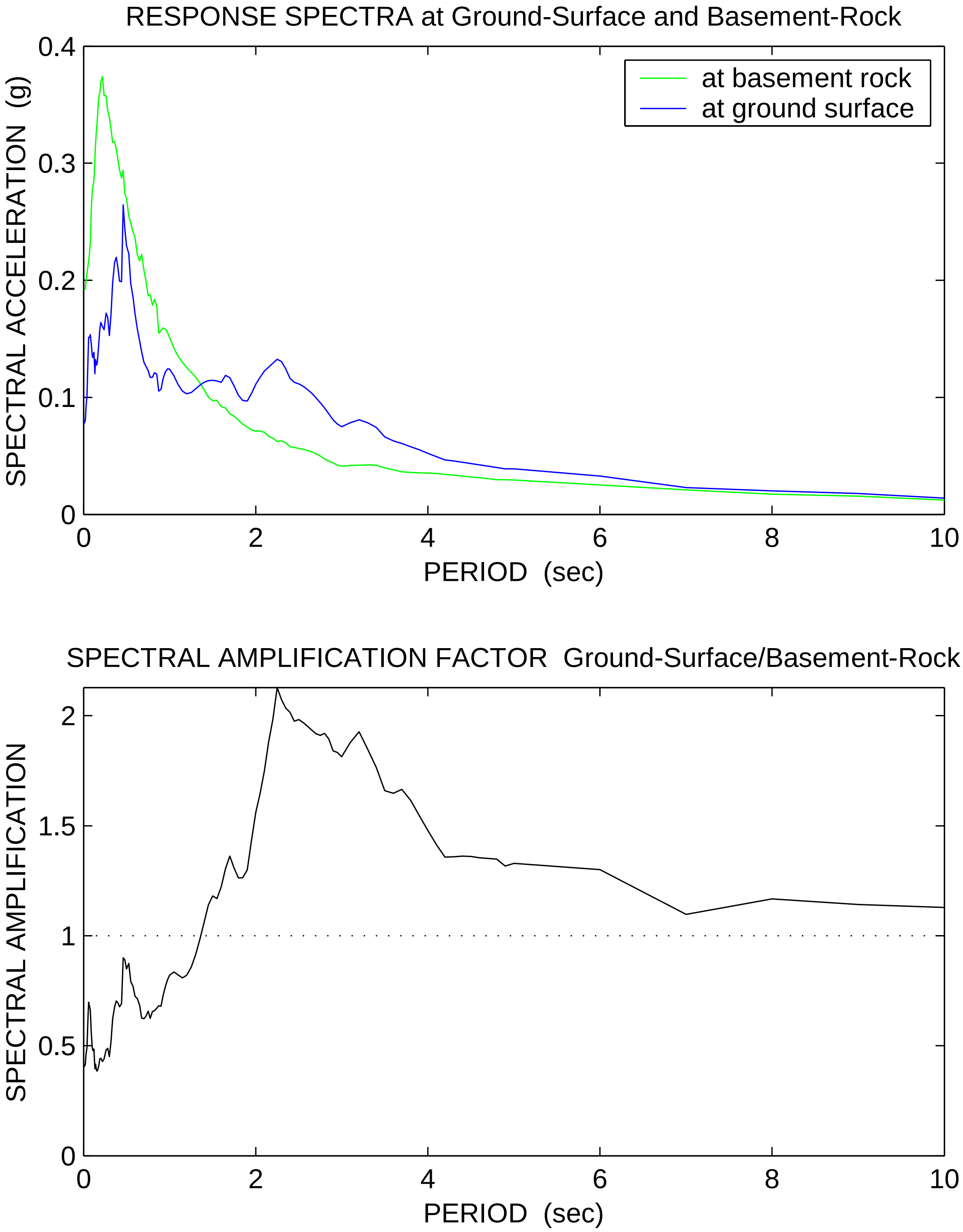


Figure 2B-23 Response spectra at ground surface and base-rock and spectral amplification factor: Guadalupe bridge site subjected to RP0050-Eq1X

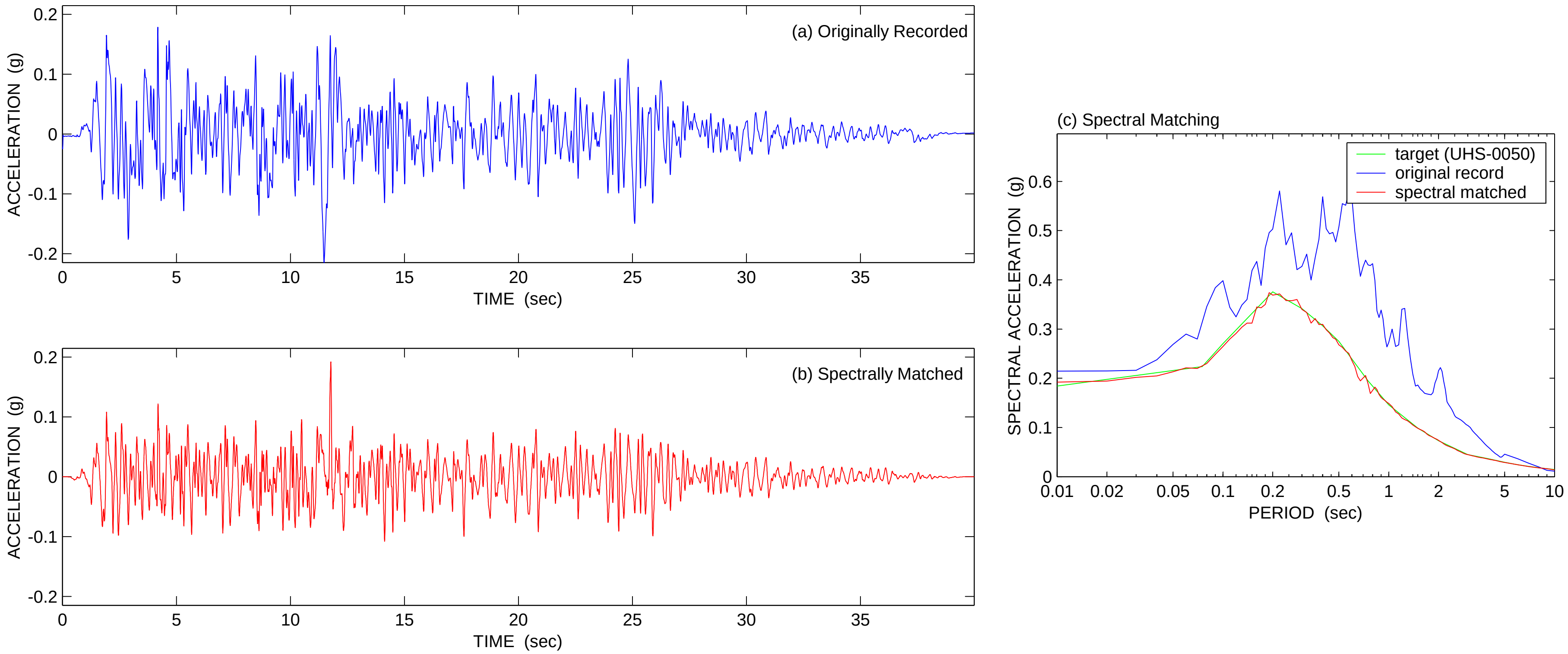


Figure 2B-24 Spectral matching ↝ input earthquake ground acceleration time-history RP0050-Eq1Y Earthquake Ground Motion Suite (50-year Return Period)

(a) seed motion (originally recorded ground acceleration time history)
(b) spectrally matched ground acceleration time history
(c) comparison of response spectra (target, seed, spectrally matched)

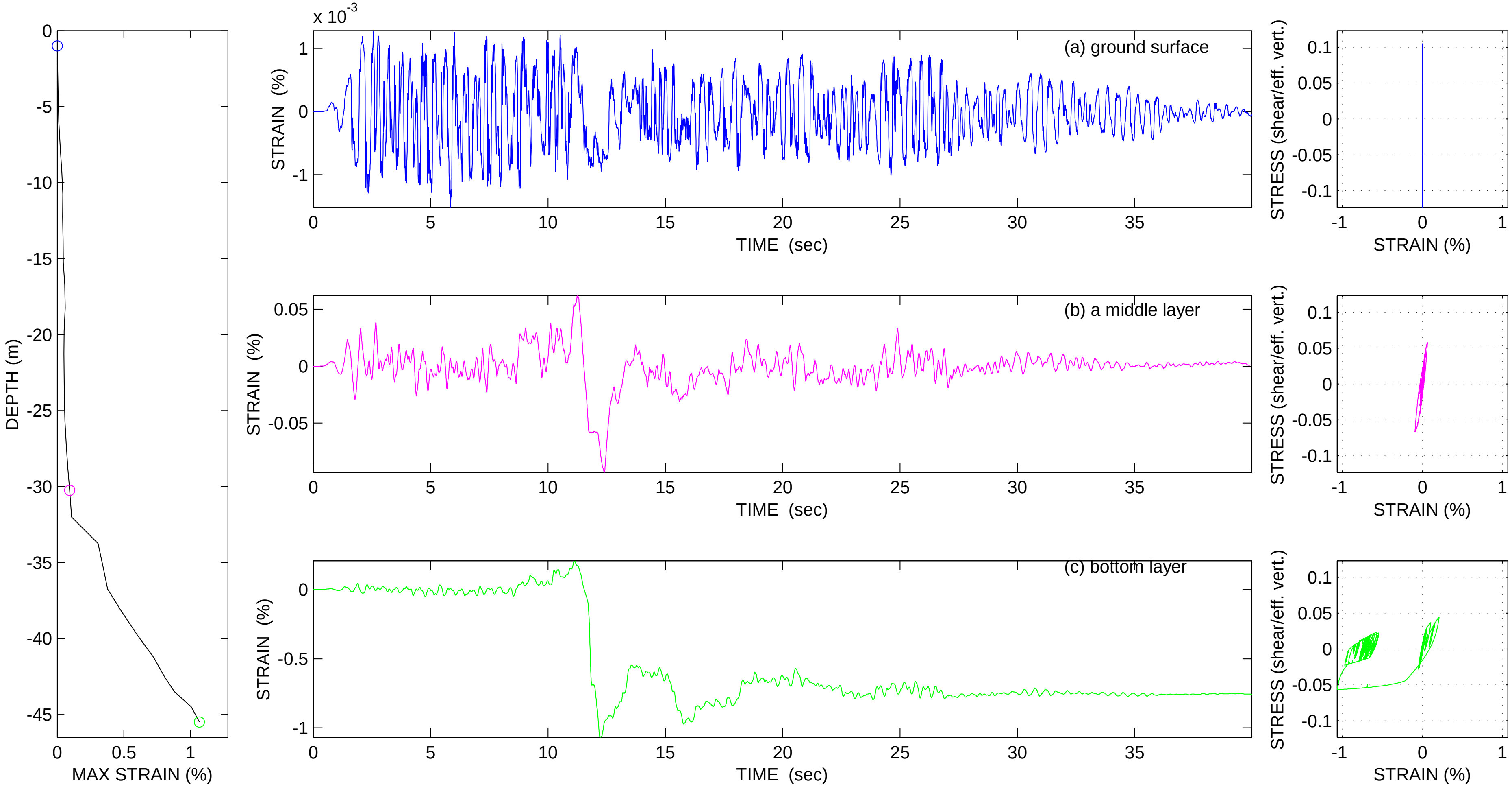


Figure 2B-25 Nonlinear site response analysis of Guadalupe bridge site subjected to input base motion RP0050-Eq1Y — maximum strain profile; strain time-histories; stress-strain hystereses — (a) at ground surface; (b) at about 30m depth; (c) at the layer atop base-rock

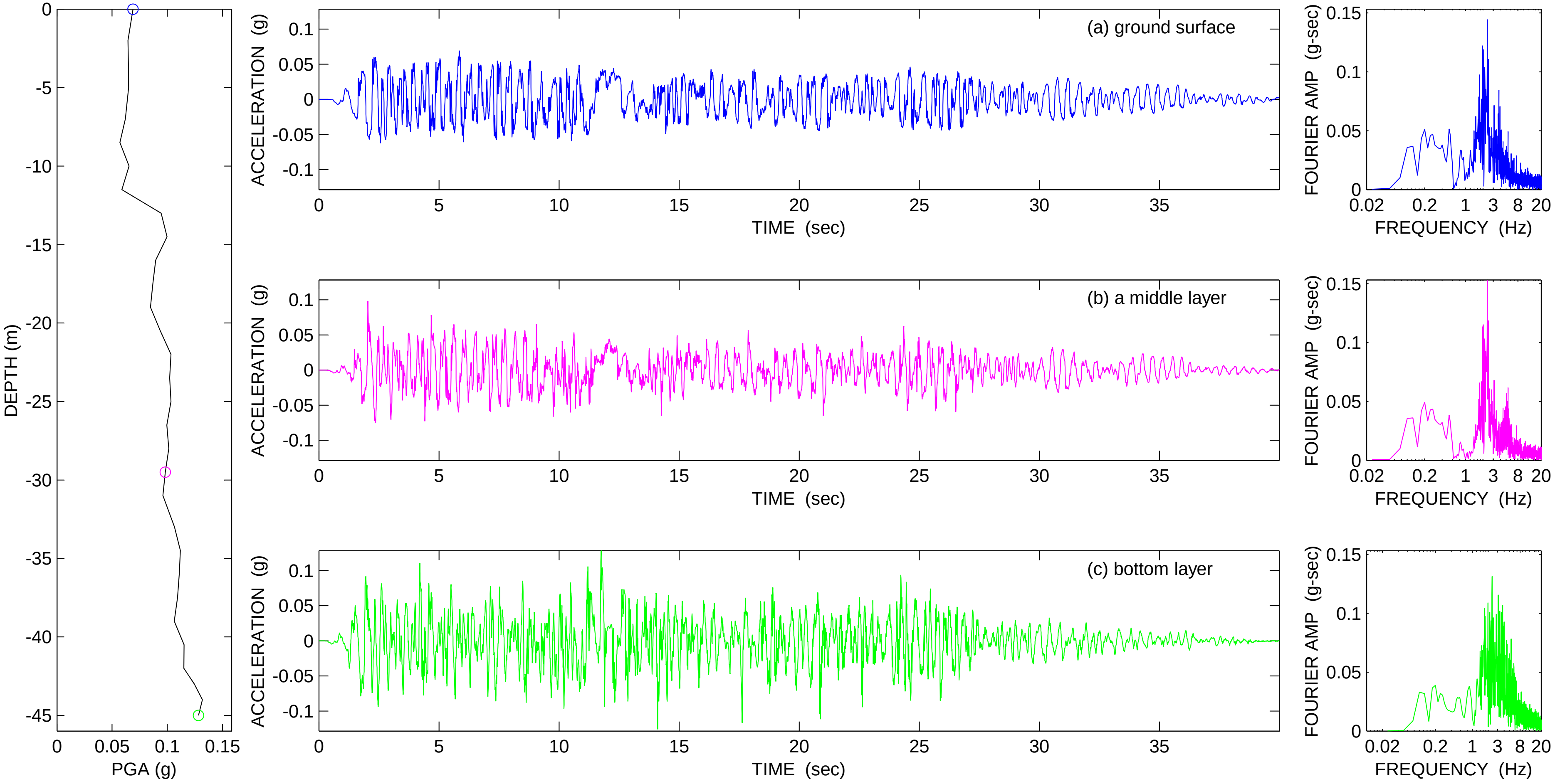


Figure 2B-26 Nonlinear site response analysis of Guadalupe bridge site subjected to input base motion RP0050-Eq1Y — PGA profile; ground acceleration time-histories; Fourier amplitude spectra — (a) at ground surface; (b) at about 30m depth; (c) at the layer atop base-rock

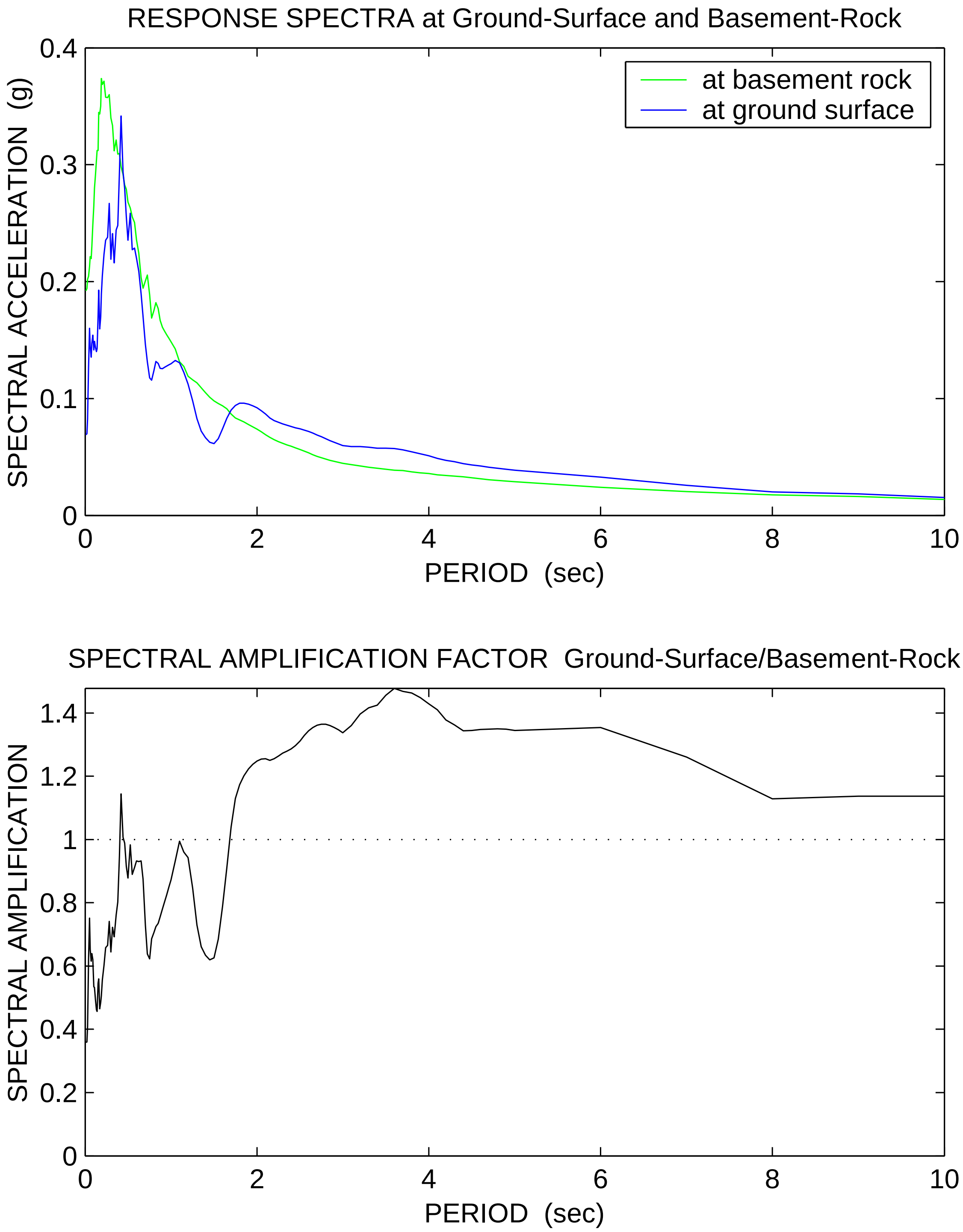


Figure 2B-27 Response spectra at ground surface and base-rock and spectral amplification factor: Guadalupe bridge site subjected to RP0050-Eq1Y

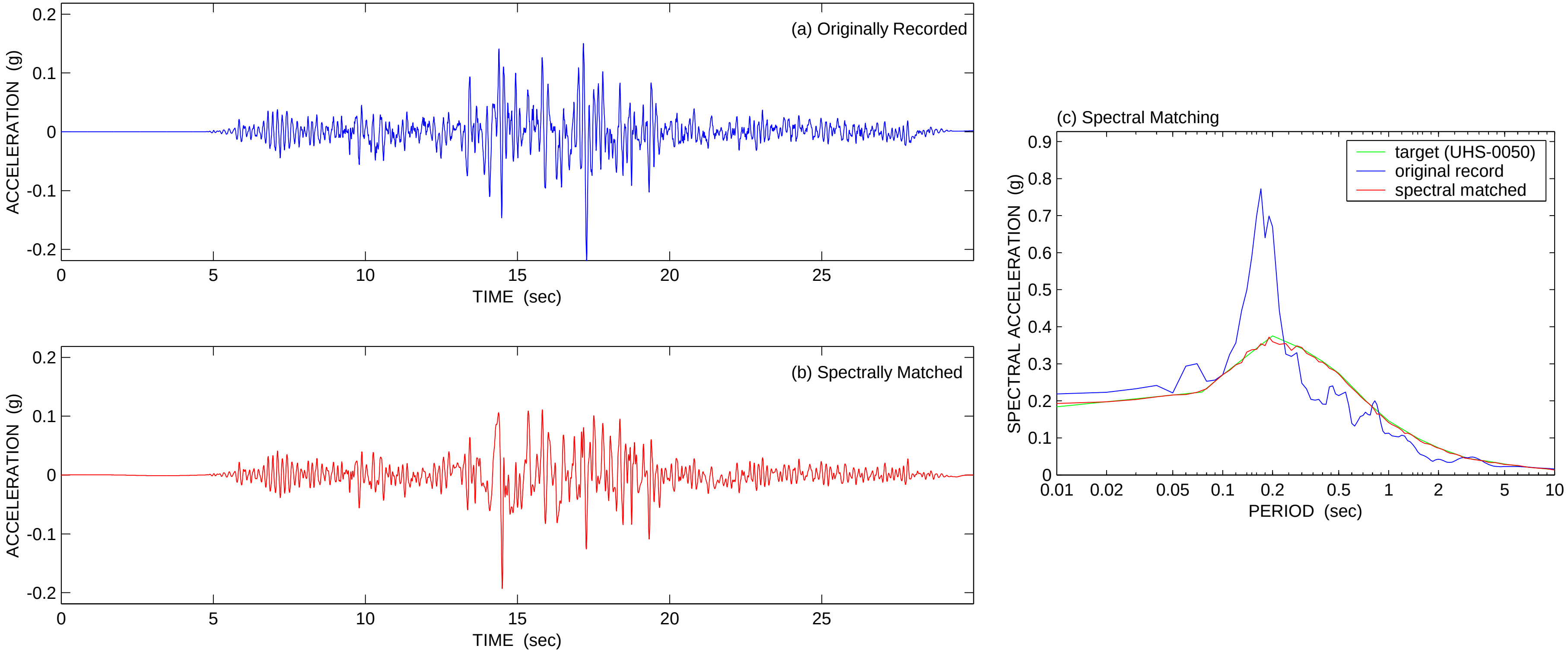


Figure 2B-28 Spectral matching ↝ input earthquake ground acceleration time-history RP0050-Eq2X
Earthquake Ground Motion Suite (50-year Return Period)
(a) seed motion (originally recorded ground acceleration time history)
(b) spectrally matched ground acceleration time history
(c) comparison of response spectra (target, seed, spectrally matched)

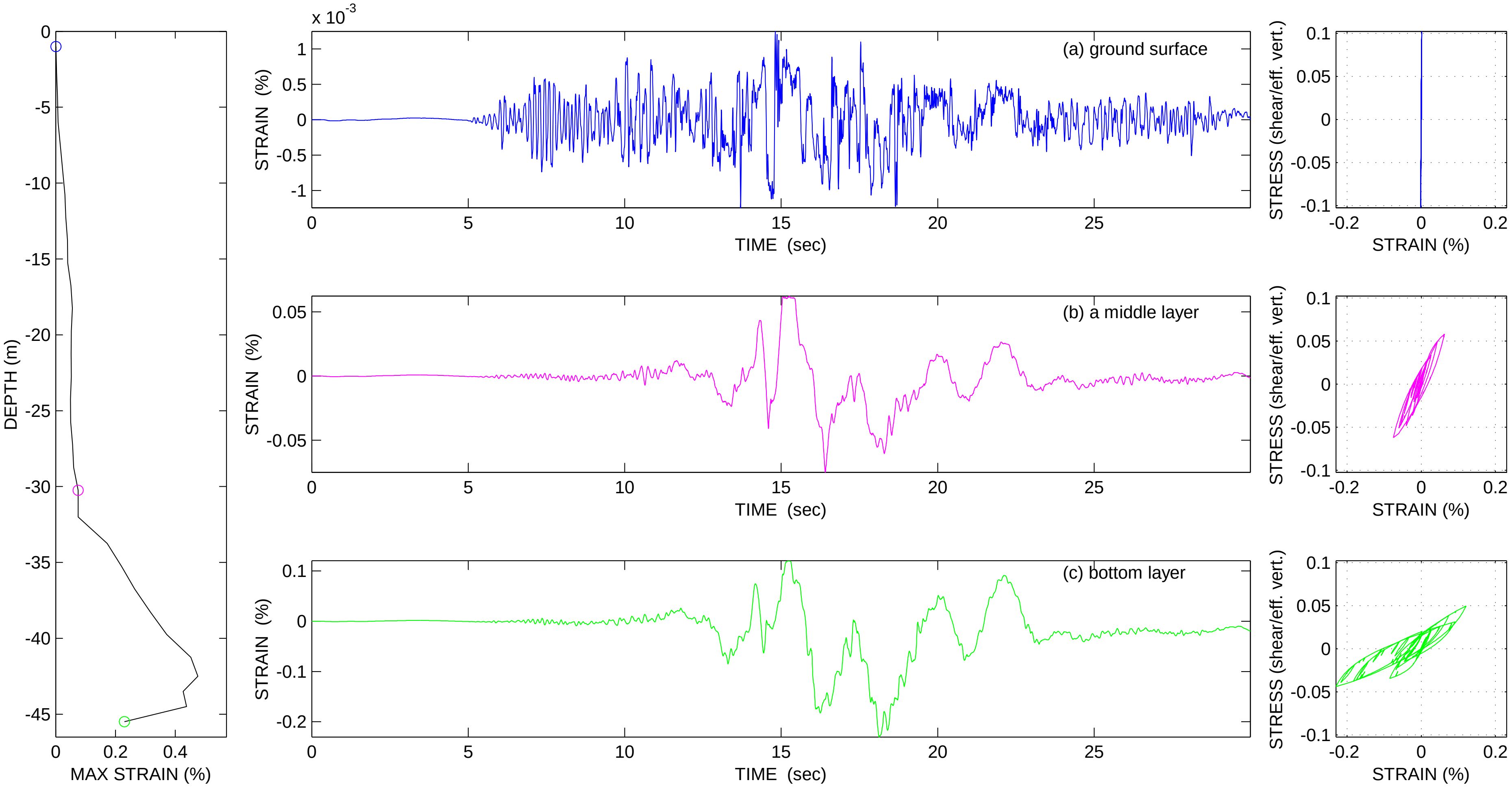


Figure 2B-29 Nonlinear site response analysis of Guadalupe bridge site subjected to input base motion RP0050-Eq2X — maximum strain profile; strain time-histories; stress-strain hystereses — (a) at ground surface; (b) at about 30m depth; (c) at the layer atop base-rock

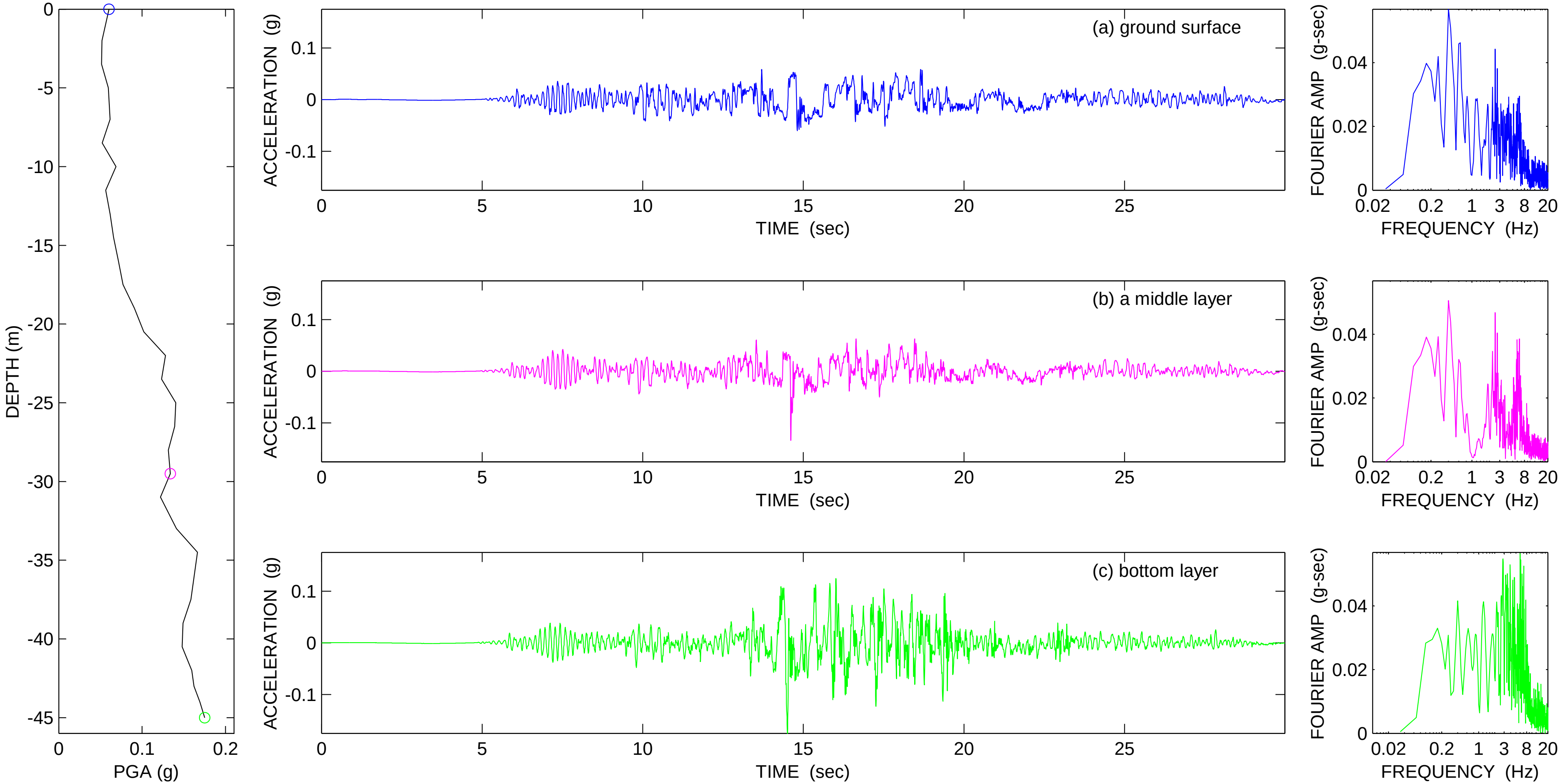


Figure 2B-30 Nonlinear site response analysis of Guadalupe bridge site subjected to input base motion RP0050-Eq2X — PGA profile; ground acceleration time-histories; Fourier amplitude spectra — (a) at ground surface; (b) at about 30m depth; (c) at the layer atop base-rock

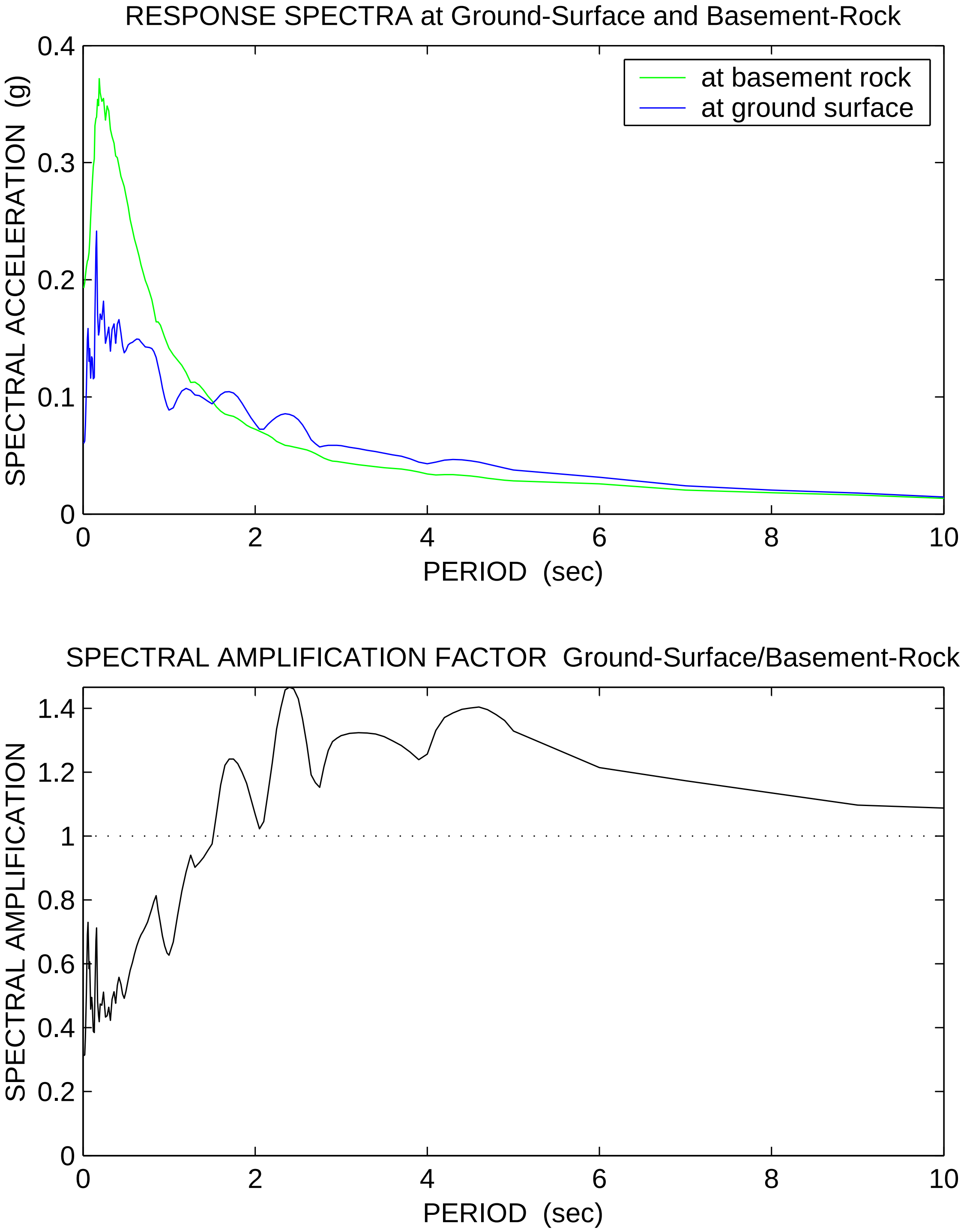


Figure 2B-31 Response spectra at ground surface and base-rock and spectral amplification factor: Guadalupe bridge site subjected to RP0050-Eq2X

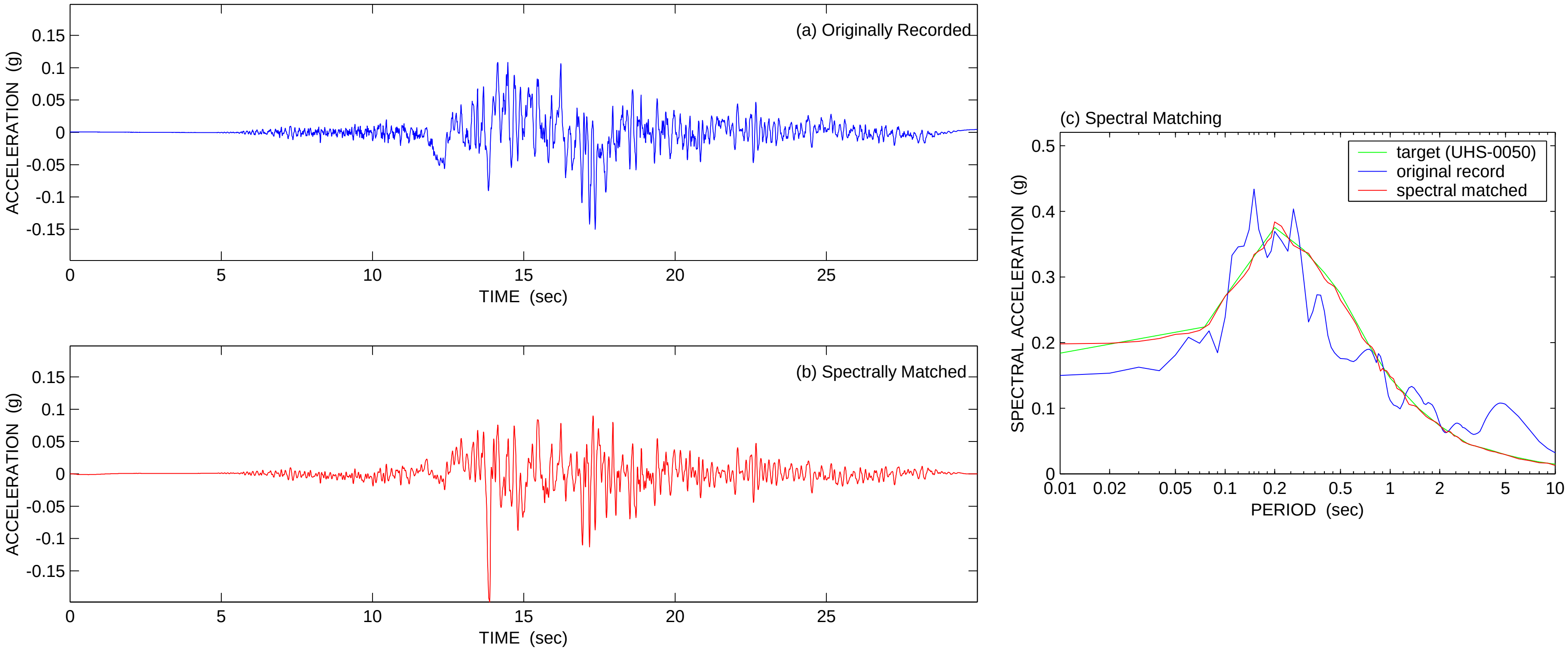


Figure 2B-32 Spectral matching ↝ input earthquake ground acceleration time-history RP0050-Eq2Y Earthquake Ground Motion Suite (50-year Return Period)

(a) seed motion (originally recorded ground acceleration time history)

(b) spectrally matched ground acceleration time history

(c) comparison of response spectra (target, seed, spectrally matched)

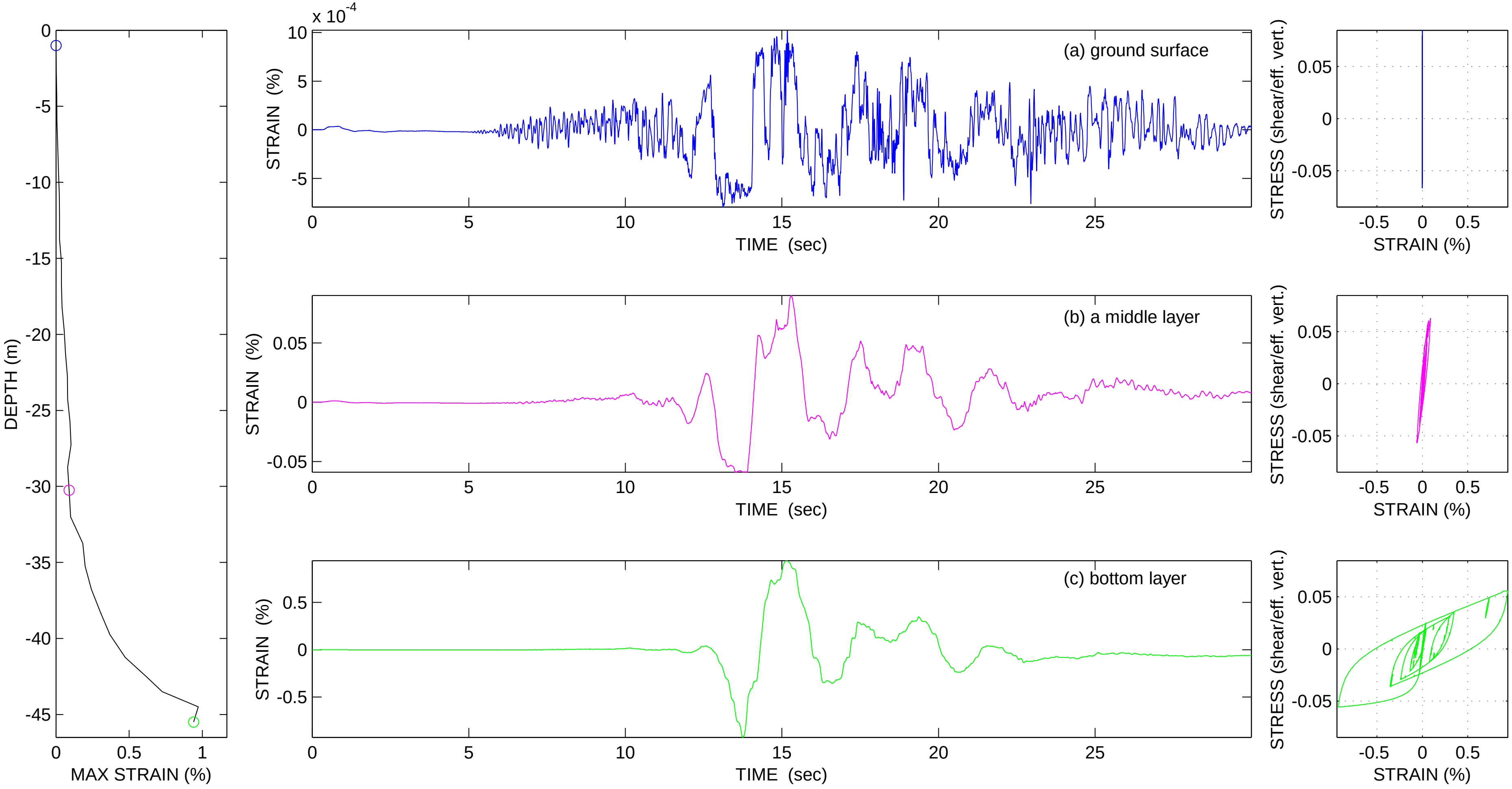


Figure 2B-33 Nonlinear site response analysis of Guadalupe bridge site subjected to input base motion RP0050-Eq2Y — maximum strain profile; strain time-histories; stress-strain hystereses — (a) at ground surface; (b) at about 30m depth; (c) at the layer atop base-rock

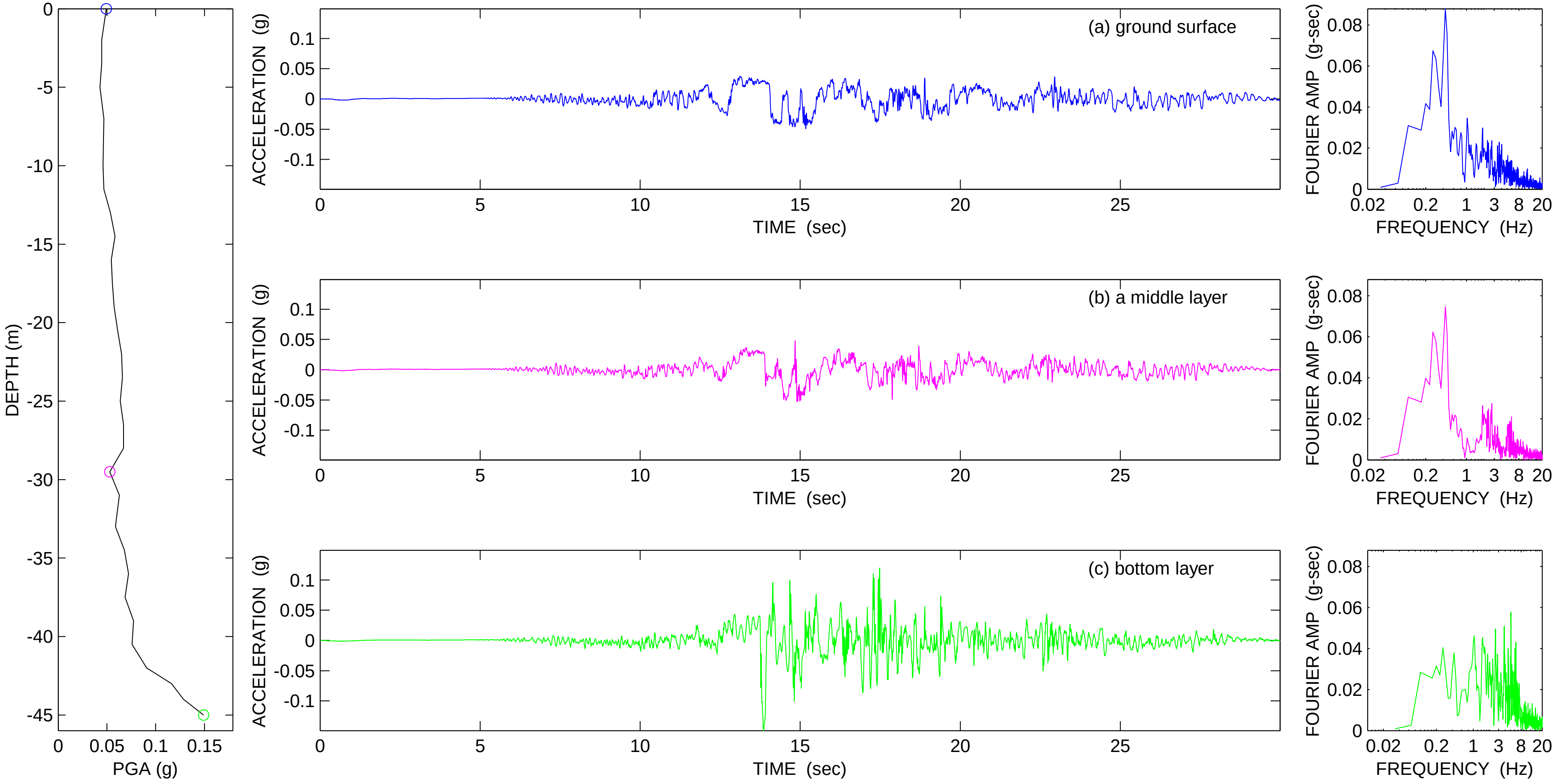


Figure 2B-34 Nonlinear site response analysis of Guadalupe bridge site subjected to input base motion RP0050-Eq2Y — PGA profile; ground acceleration time-histories; Fourier amplitude spectra — (a) at ground surface; (b) at about 30m depth; (c) at the layer atop base-rock

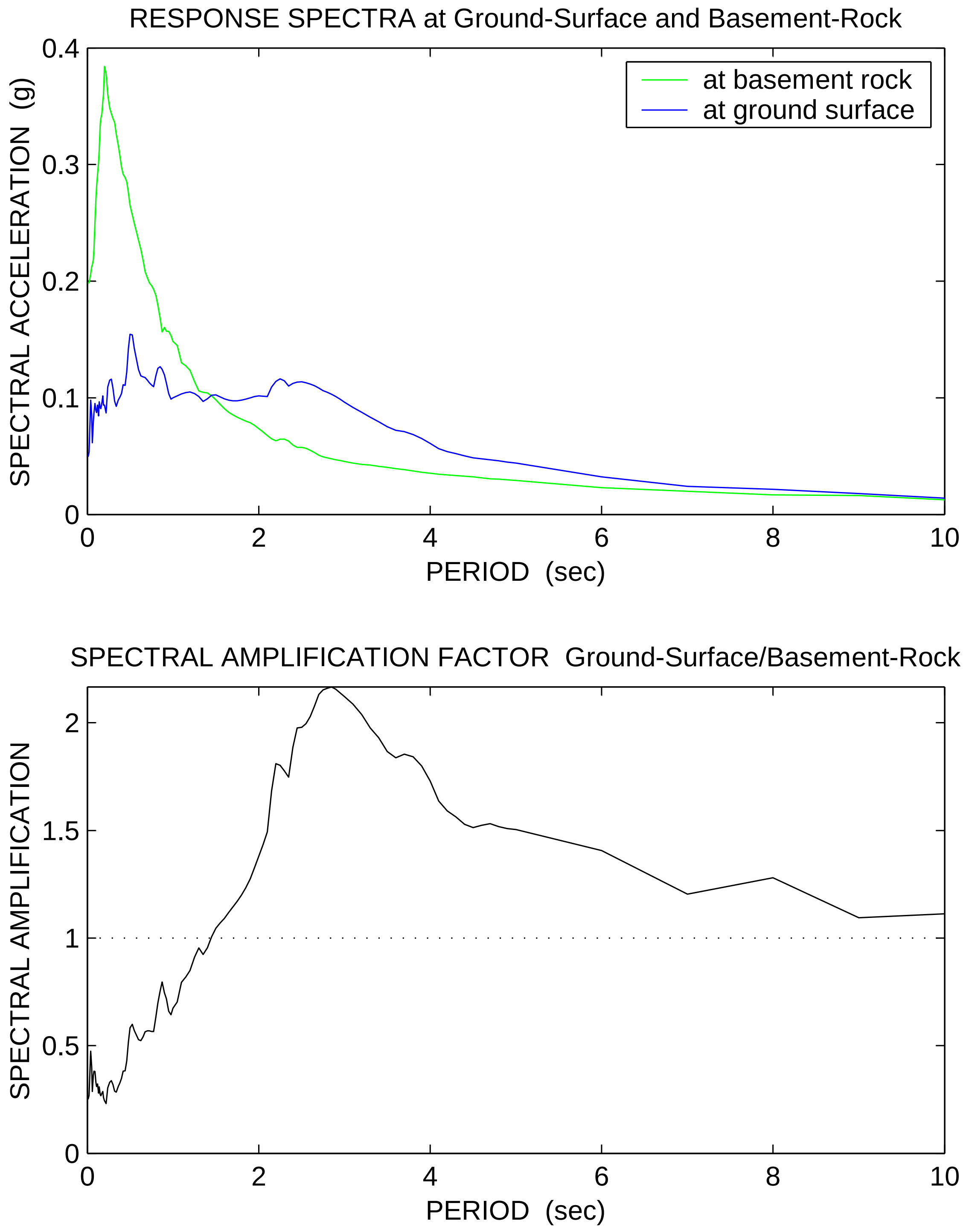


Figure 2B-35 Response spectra at ground surface and base-rock and spectral amplification factor: Guadalupe bridge site subjected to RP0050-Eq2Y

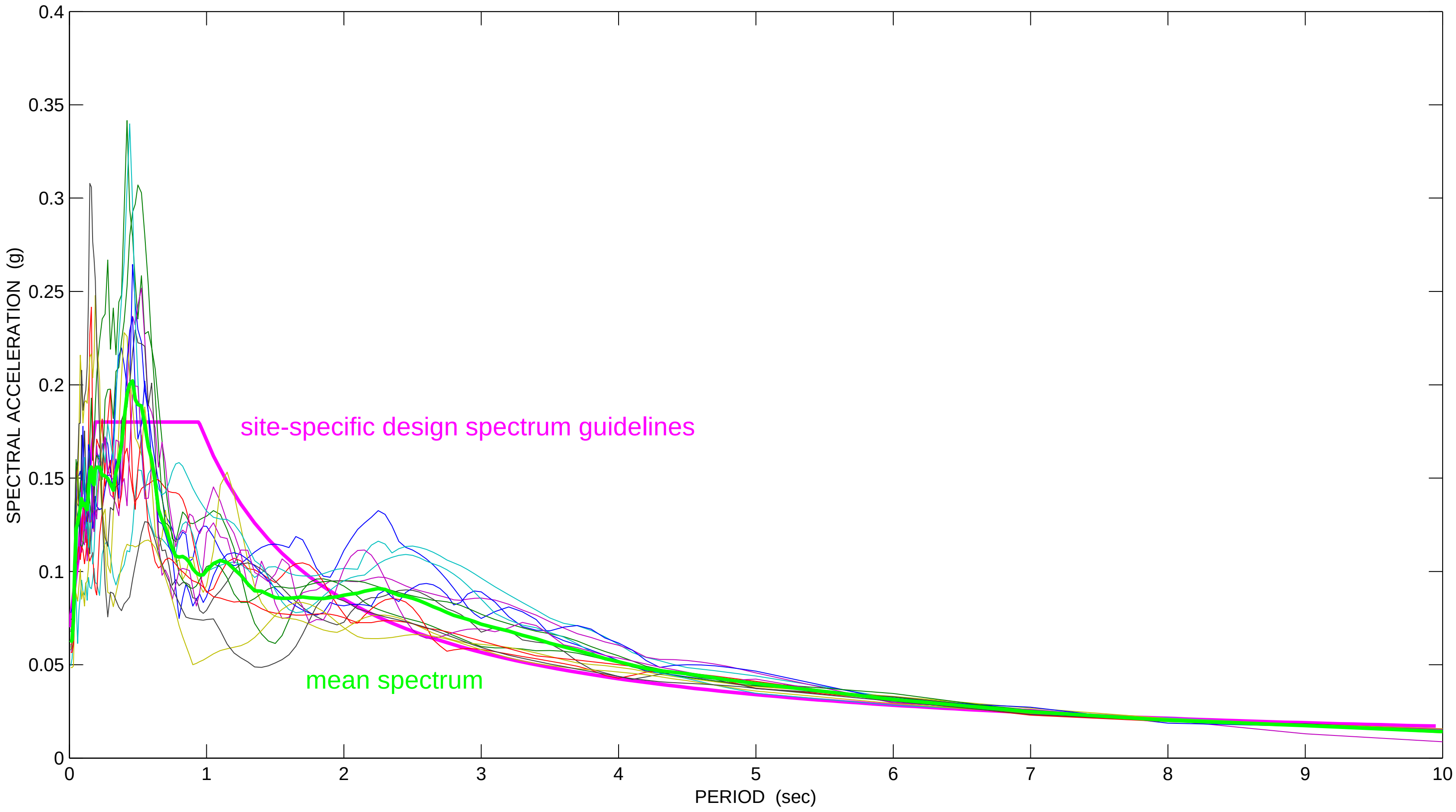


Figure 2B-36 Construction of site-specific design spectrum at 50-year return period for Guadalupe Bridge A1 based on mean of 14 response spectra from nonlinear site response analysis

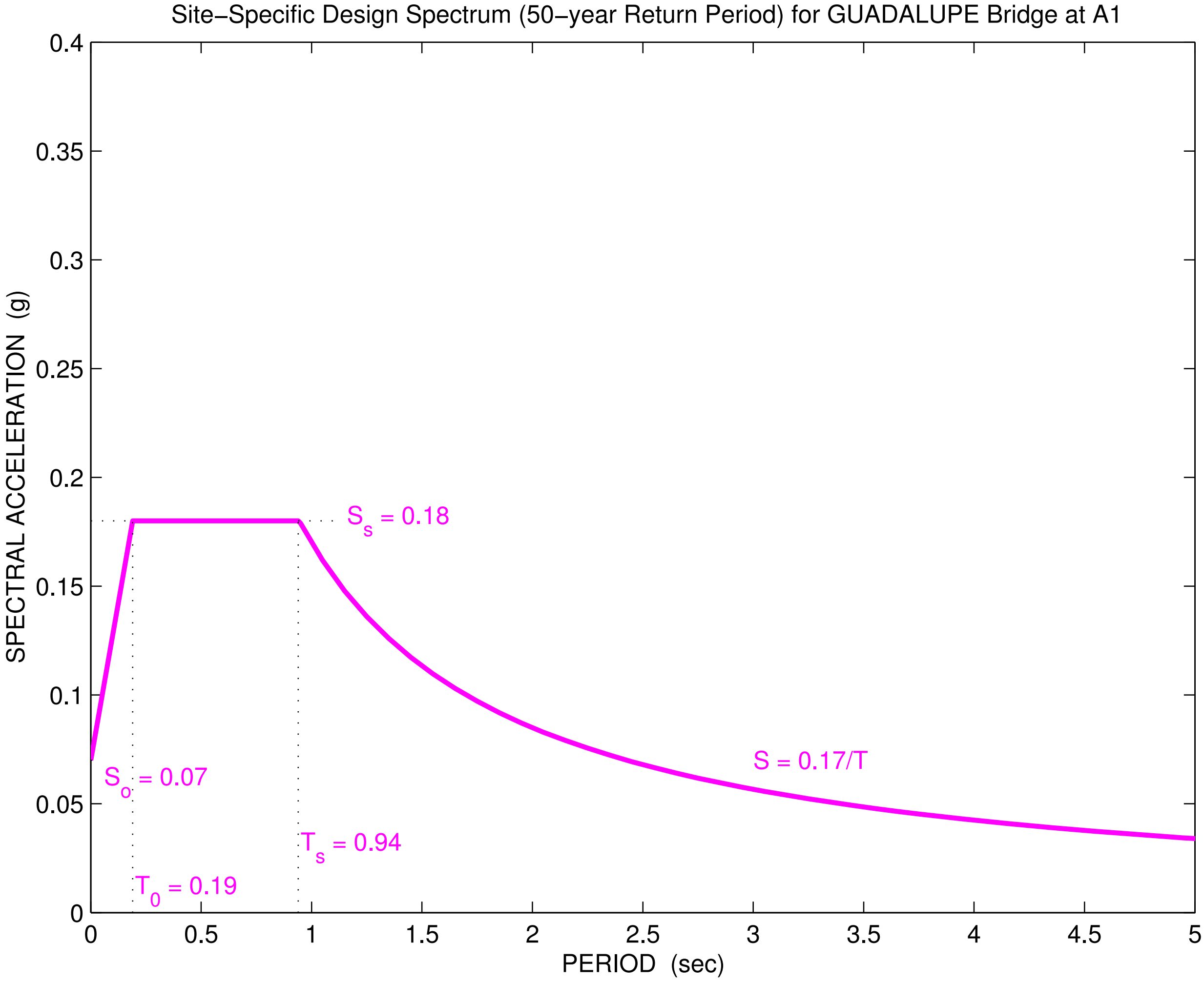


Figure 2B-37 Site-specific design spectrum at 50-year return period for Guadalupe Bridge at A1 site

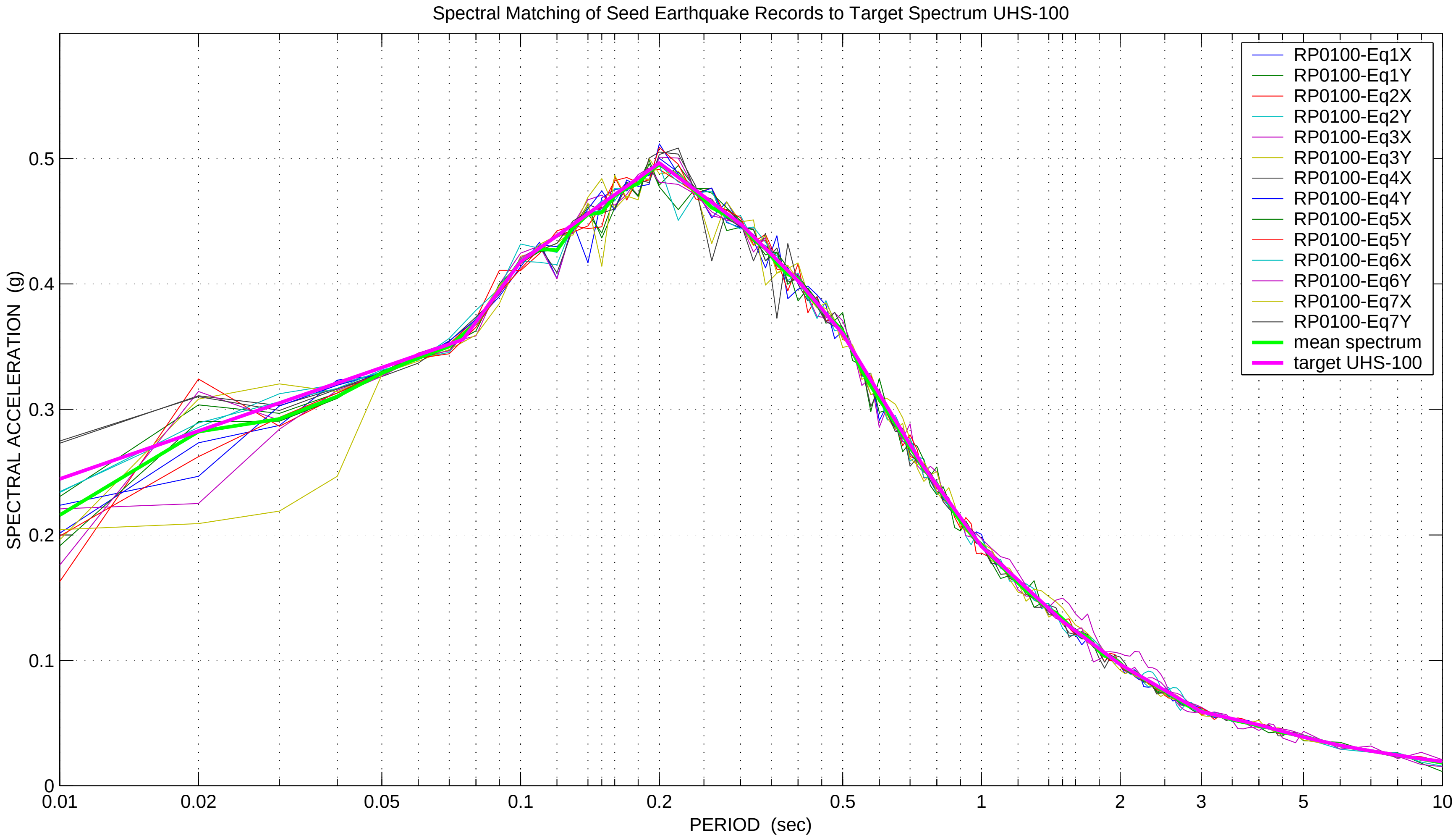


Figure 2B-38 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 100-year return period as compared to the target spectrum UHS-100

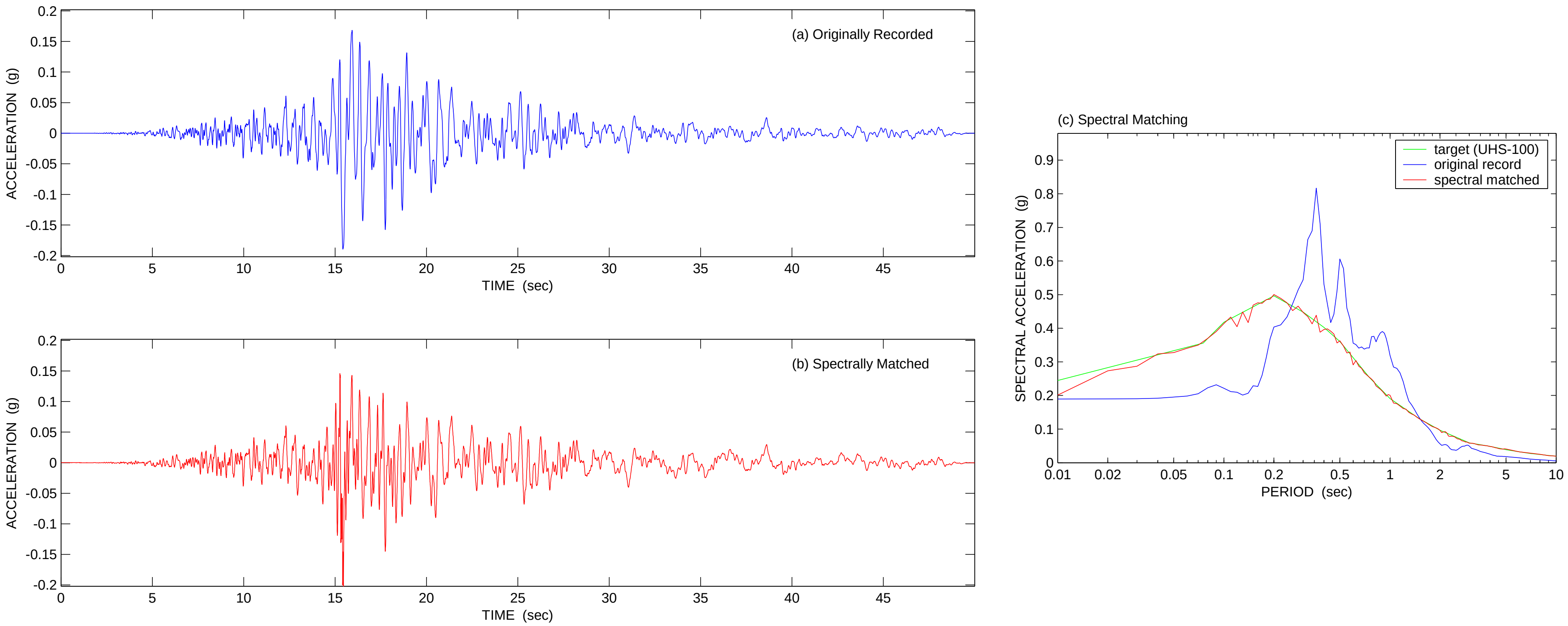


Figure 2B-39 Spectral matching ⇝ input earthquake ground acceleration time-history RP0100-Eq1X Earthquake Ground Motion Suite (50-year Return Period)
(a) seed motion (originally recorded ground acceleration time history)
(b) spectrally matched ground acceleration time history
(c) comparison of response spectra (target, seed, spectrally matched)

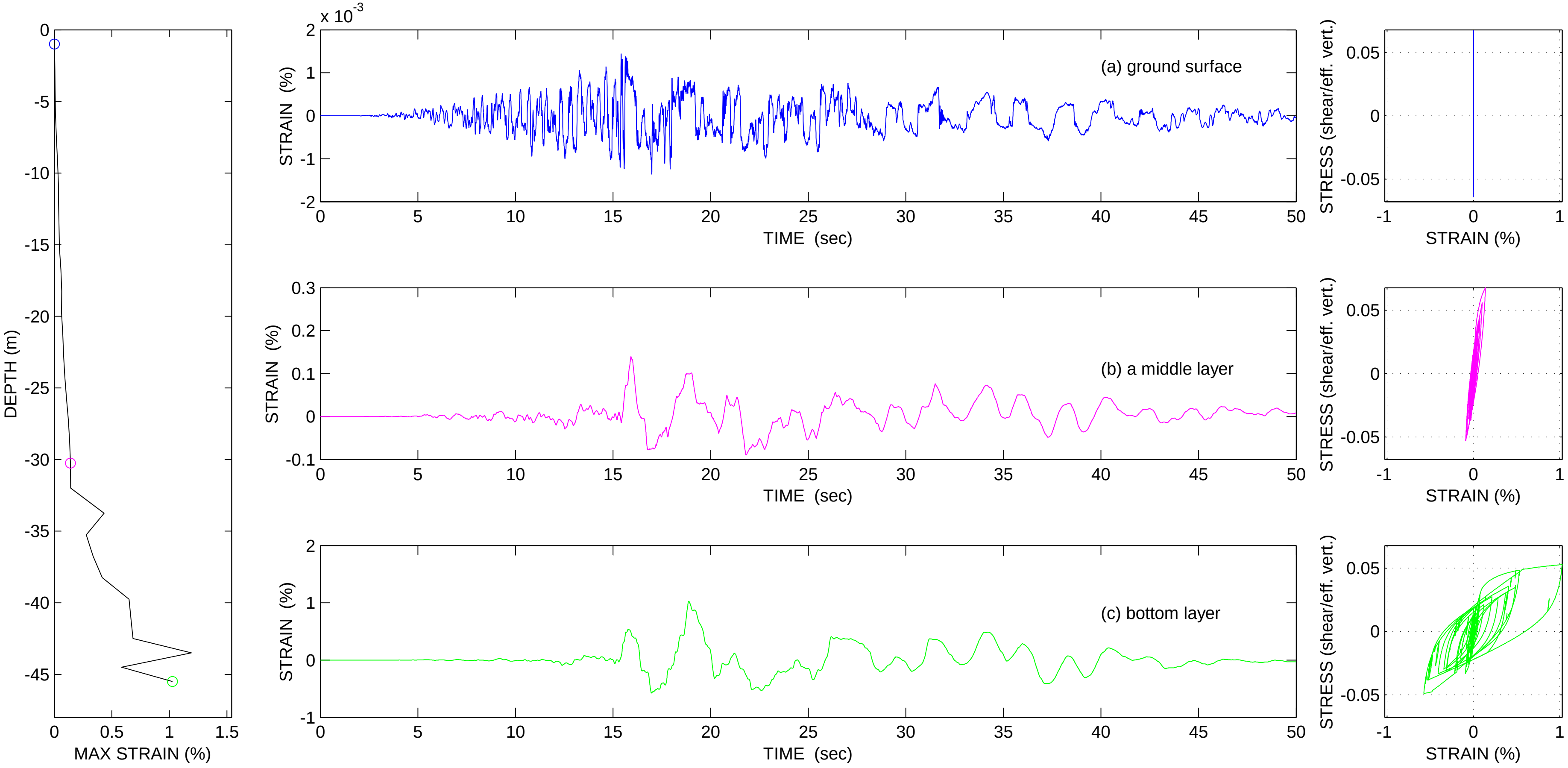


Figure 2B-40 (1/2) Nonlinear site response analysis of Guadalupe bridge site subjected to input base motion RP0100-Eq1X — maximum strain profile; strain time-histories; stress-strain hystereses — (a) at ground surface; (b) at about 30m depth; (c) at the layer atop base-rock

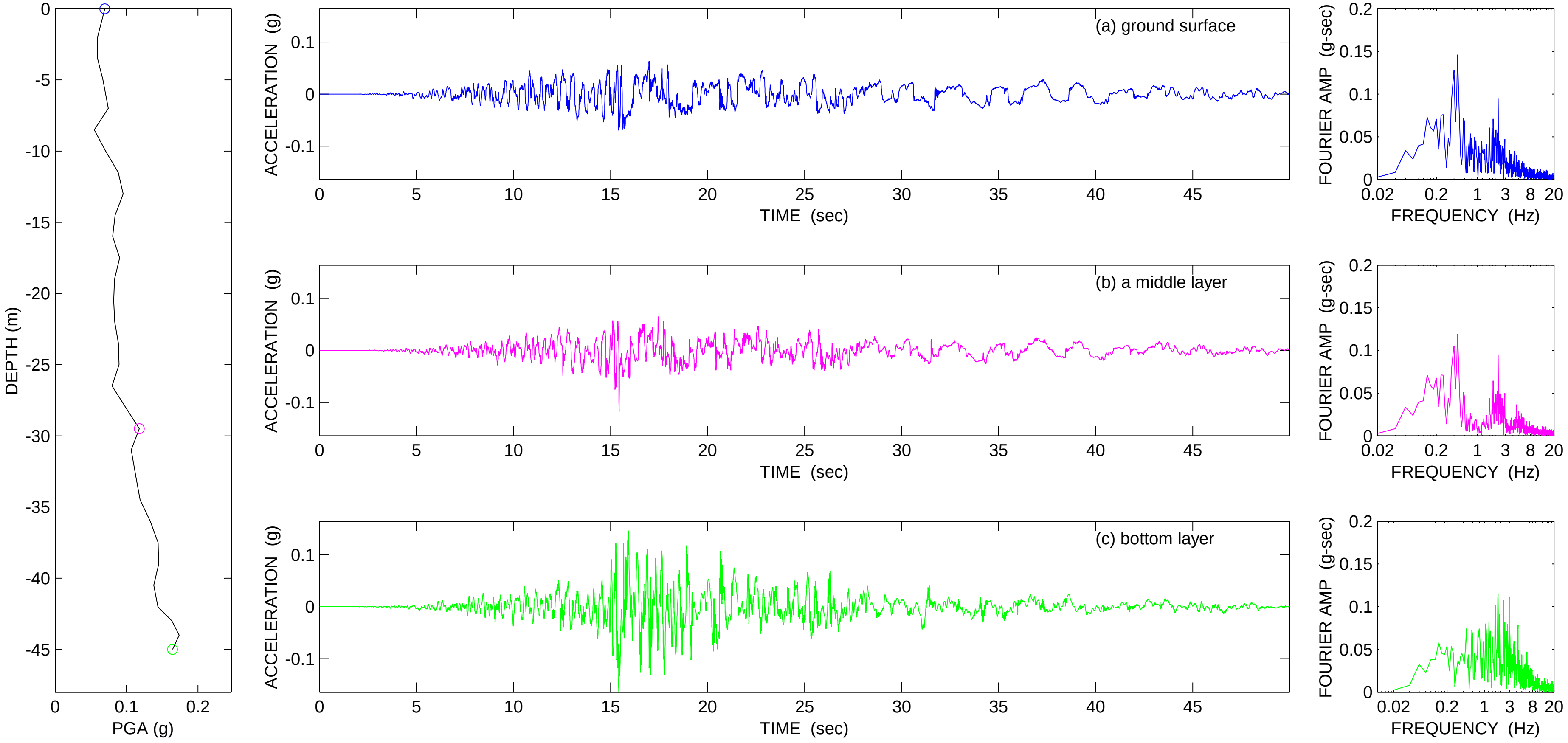


Figure 2B-40 (2/2) Nonlinear site response analysis of Guadalupe bridge site subjected to input base motion RP0100-Eq1X — PGA profile; ground acceleration time-histories; Fourier amplitude spectra — (a) at ground surface; (b) at about 30m depth; (c) at the layer atop base-rock

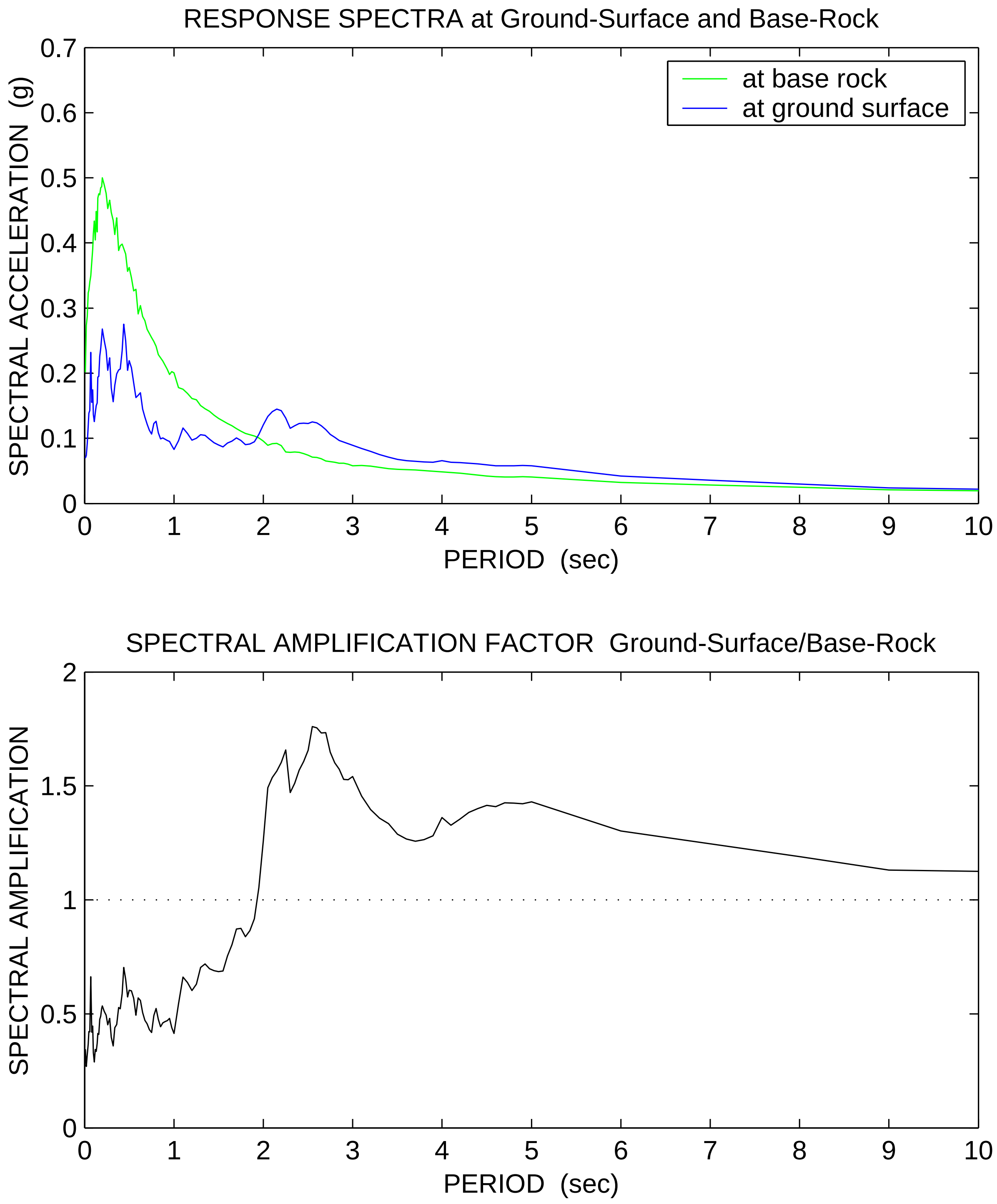


Figure 2B-41 Response spectra at ground surface and base-rock and spectral amplification factor: Guadalupe bridge site subjected to RP0100-Eq1X

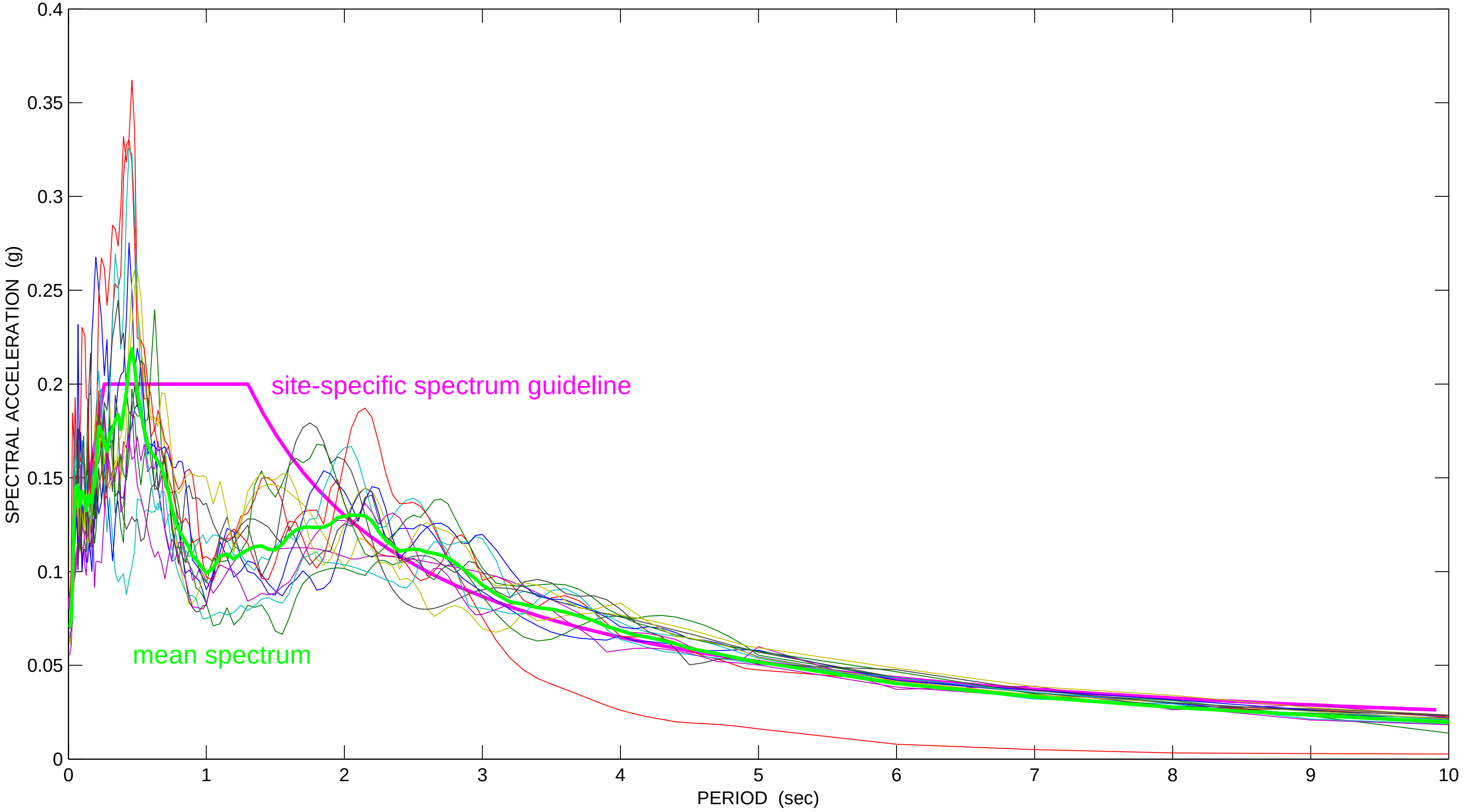


Figure 2B-42 Construction of site-specific design spectrum at 100-year return period for Guadalupe Bridge A1 based on mean of 14 response spectra from nonlinear site response analysis

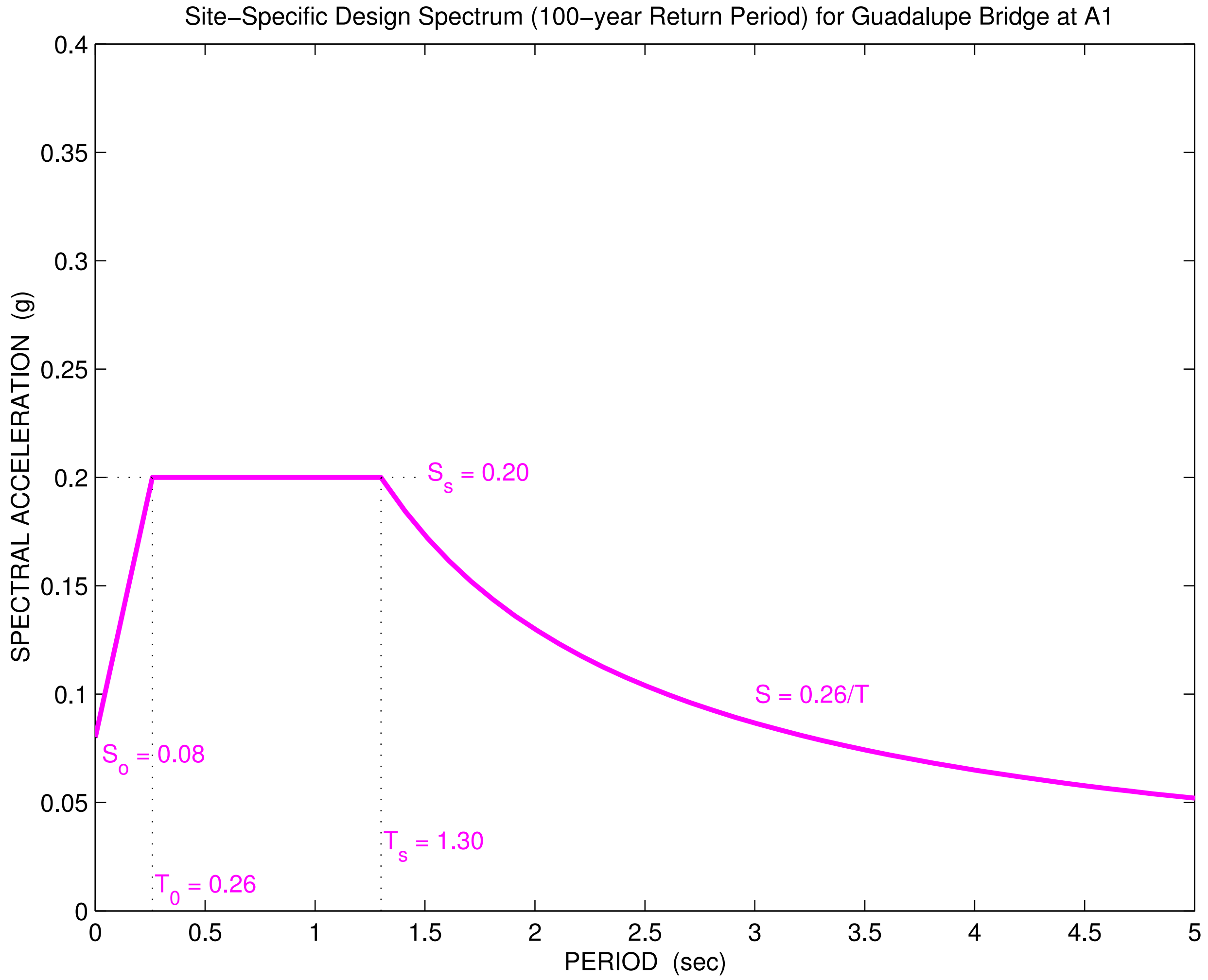


Figure 2B-43 Site-specific design spectrum at 100-year return period for Guadalupe Bridge at A1 site

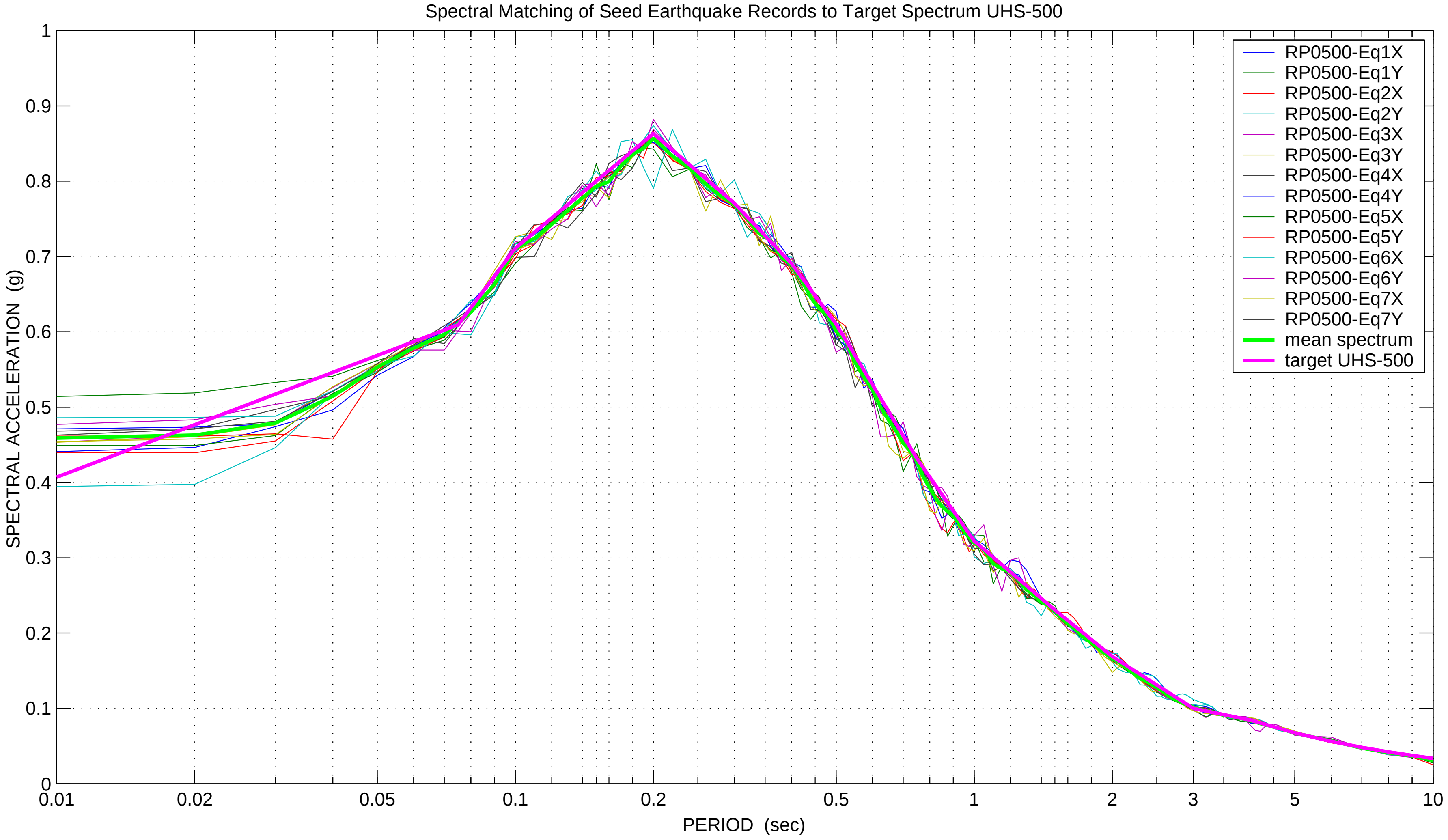


Figure 2B-44 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 500-year return period as compared to the target spectrum UHS-500

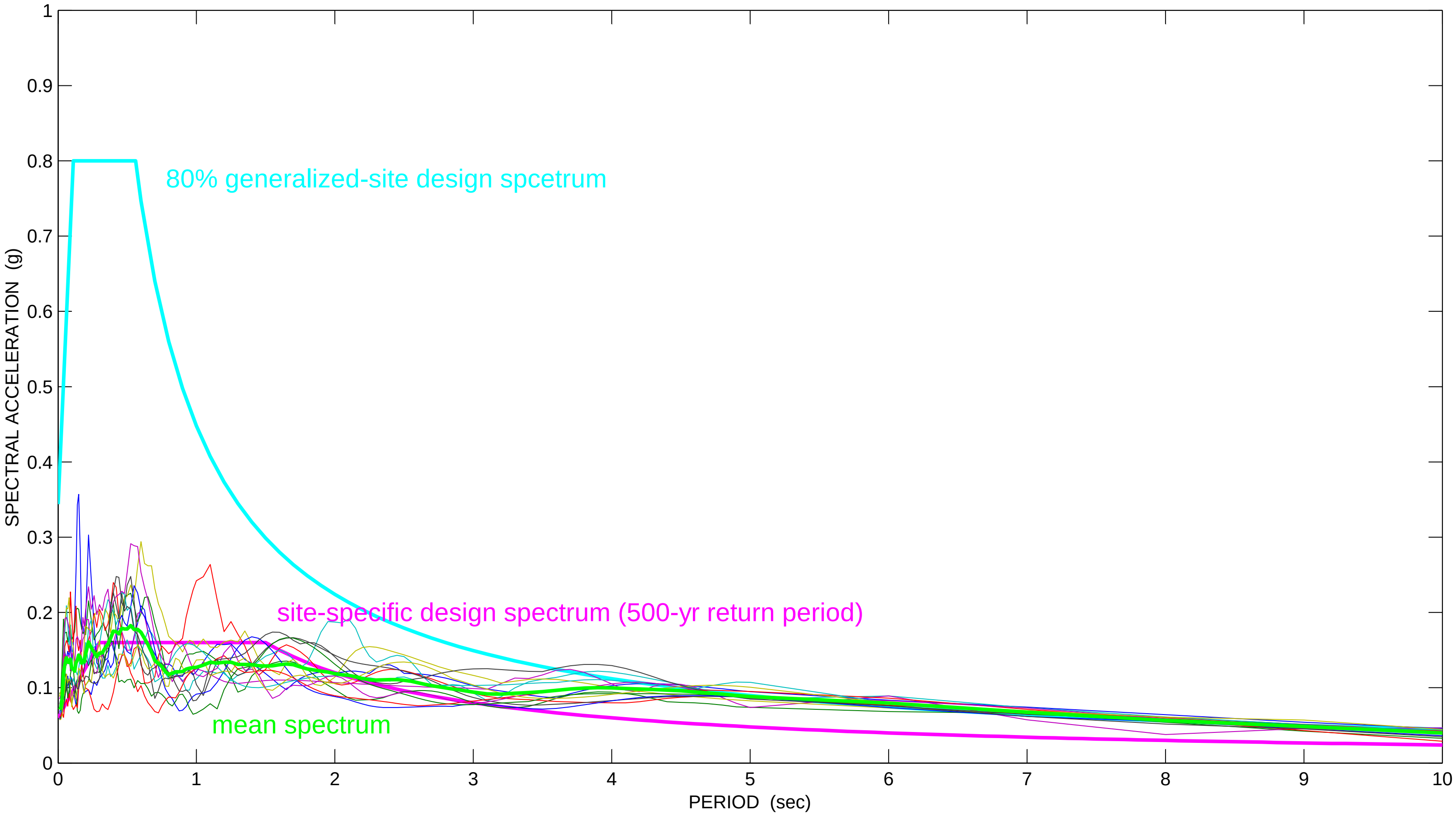


Figure 2B-45 Construction of site-specific design spectrum at 500-year return period for Guadalupe Bridge A1 based on mean of 14 response spectra from nonlinear site response analysis

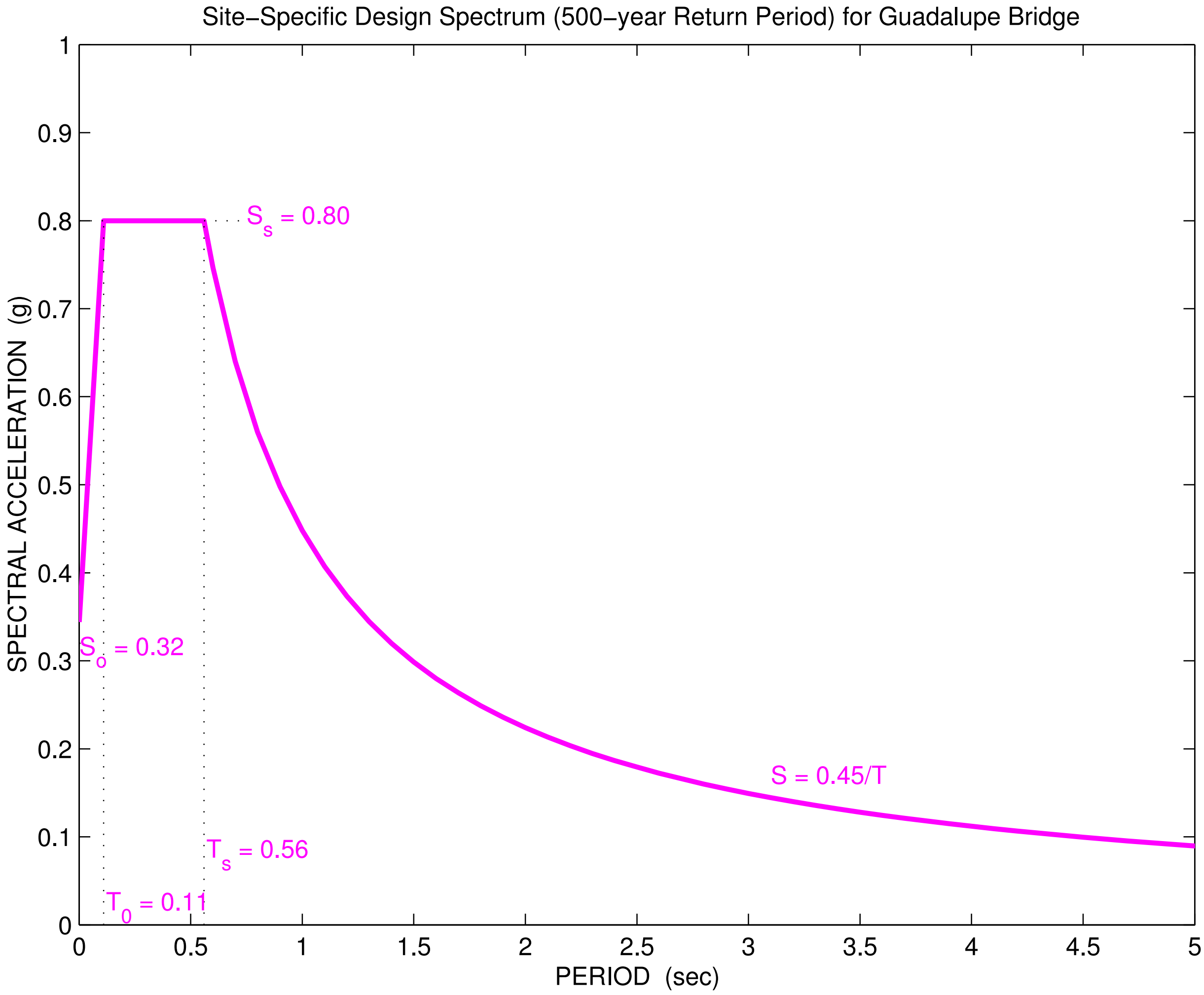


Figure 2B-46 Site-specific design spectrum at 500-year return period for Guadalupe Bridge at A1 site

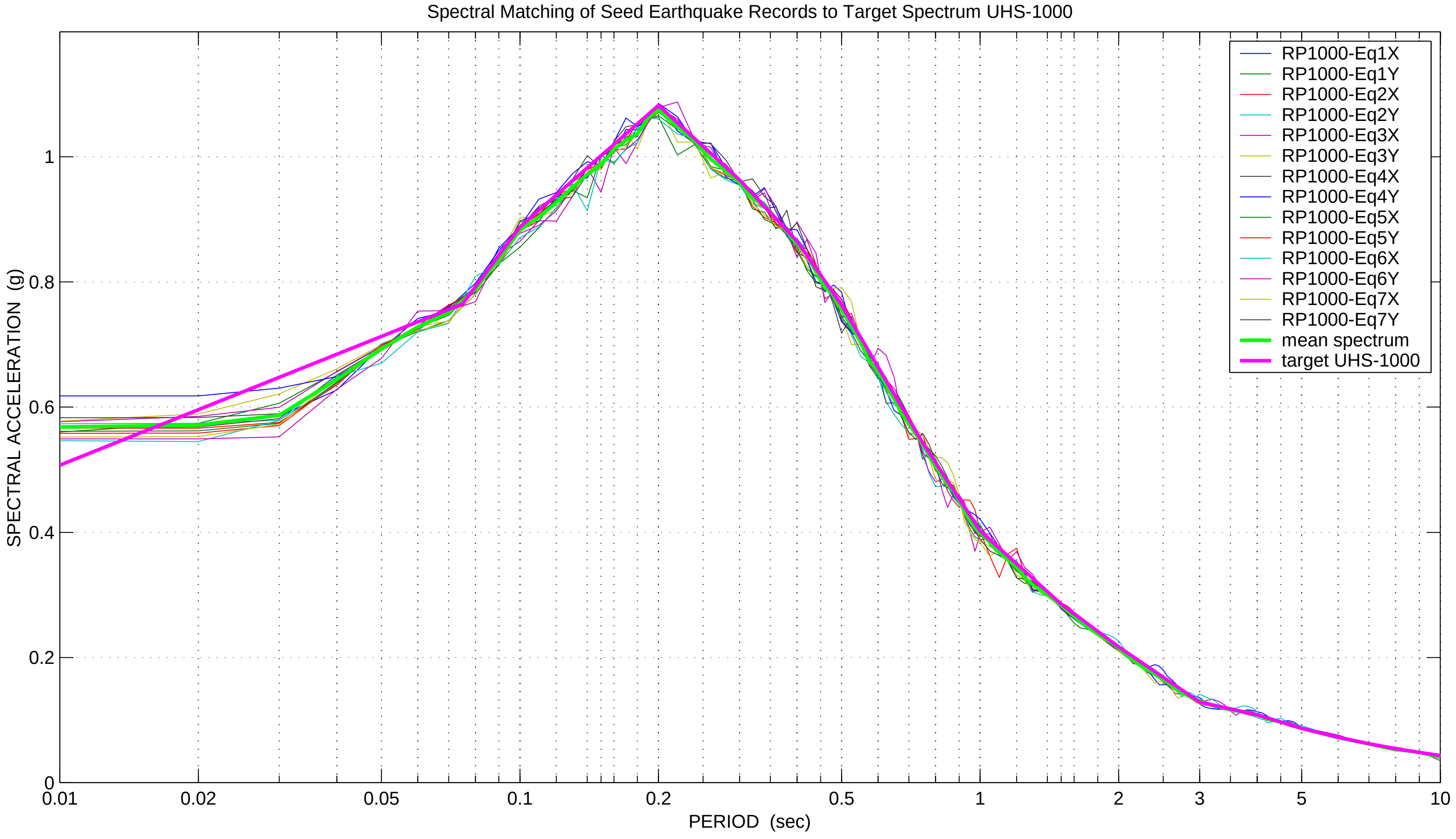


Figure 2B-47 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 1000-year return period as compared to the target spectrum UHS-1000

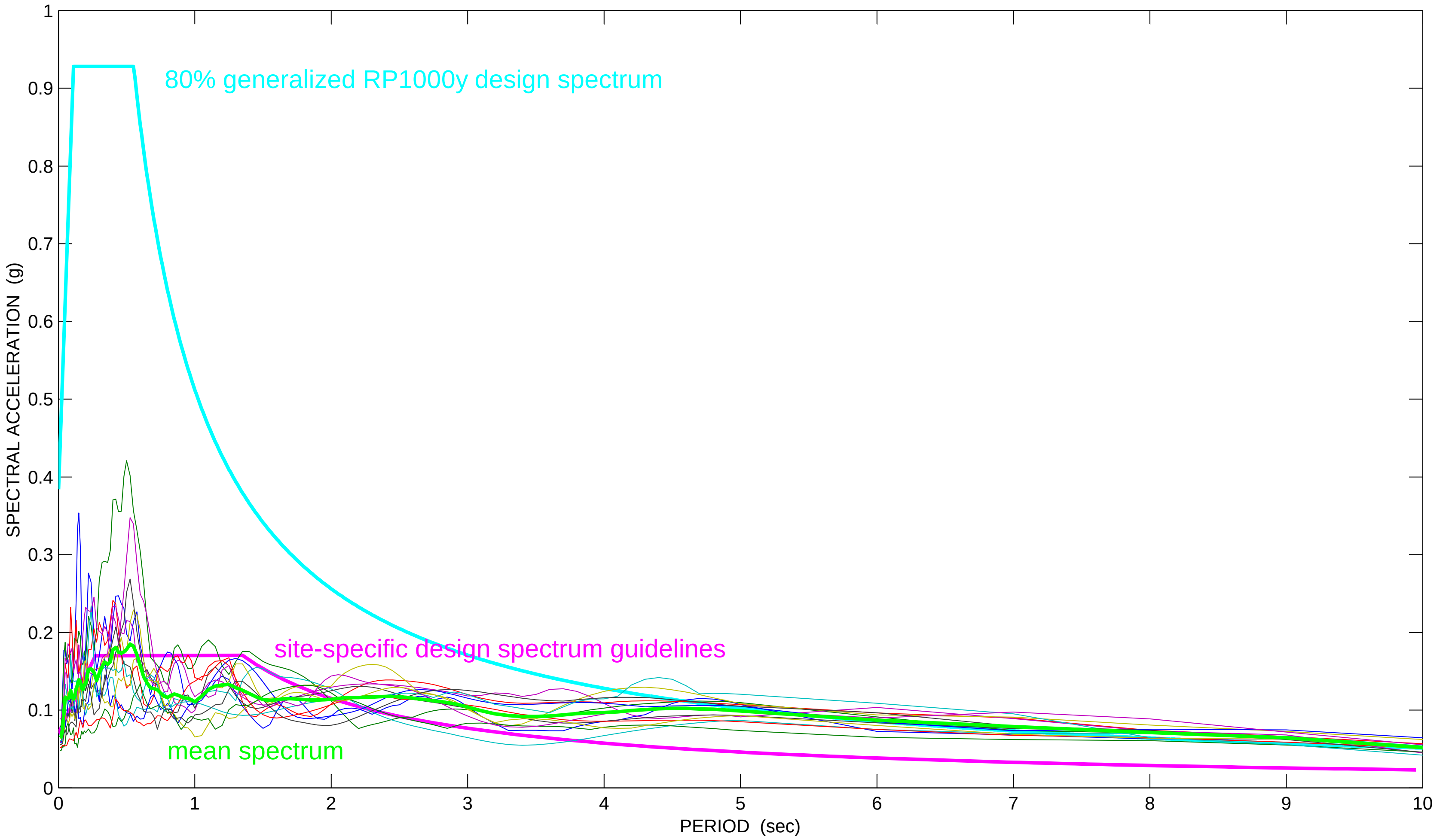


Figure 2B-48 Construction of site-specific design spectrum at 1000-year return period for Guadalupe Bridge A1 based on mean of 14 response spectra from nonlinear site response analysis

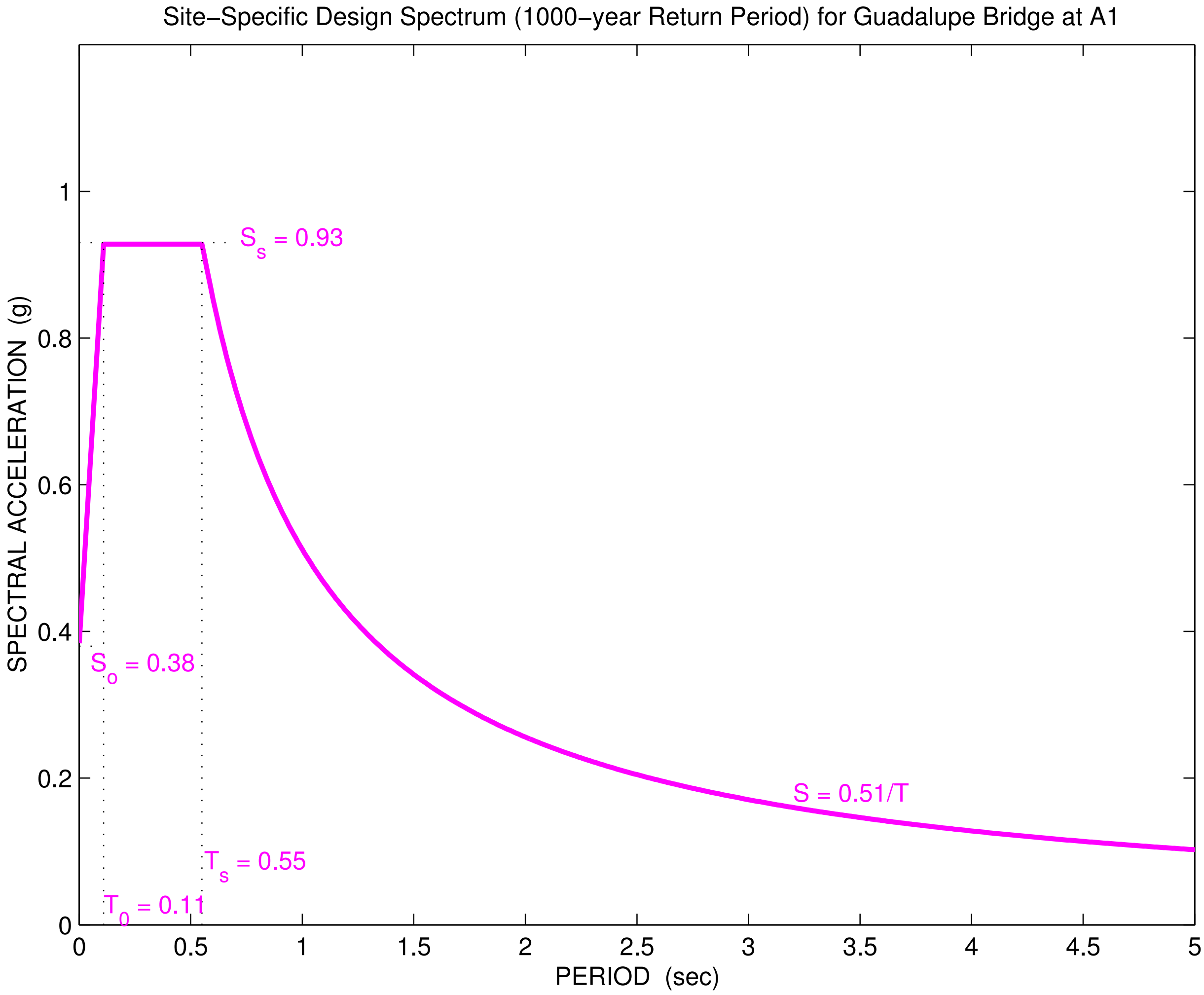


Figure 2B-49 Site-specific design spectrum at 1000-year return period for Guadalupe Bridge at A1 site

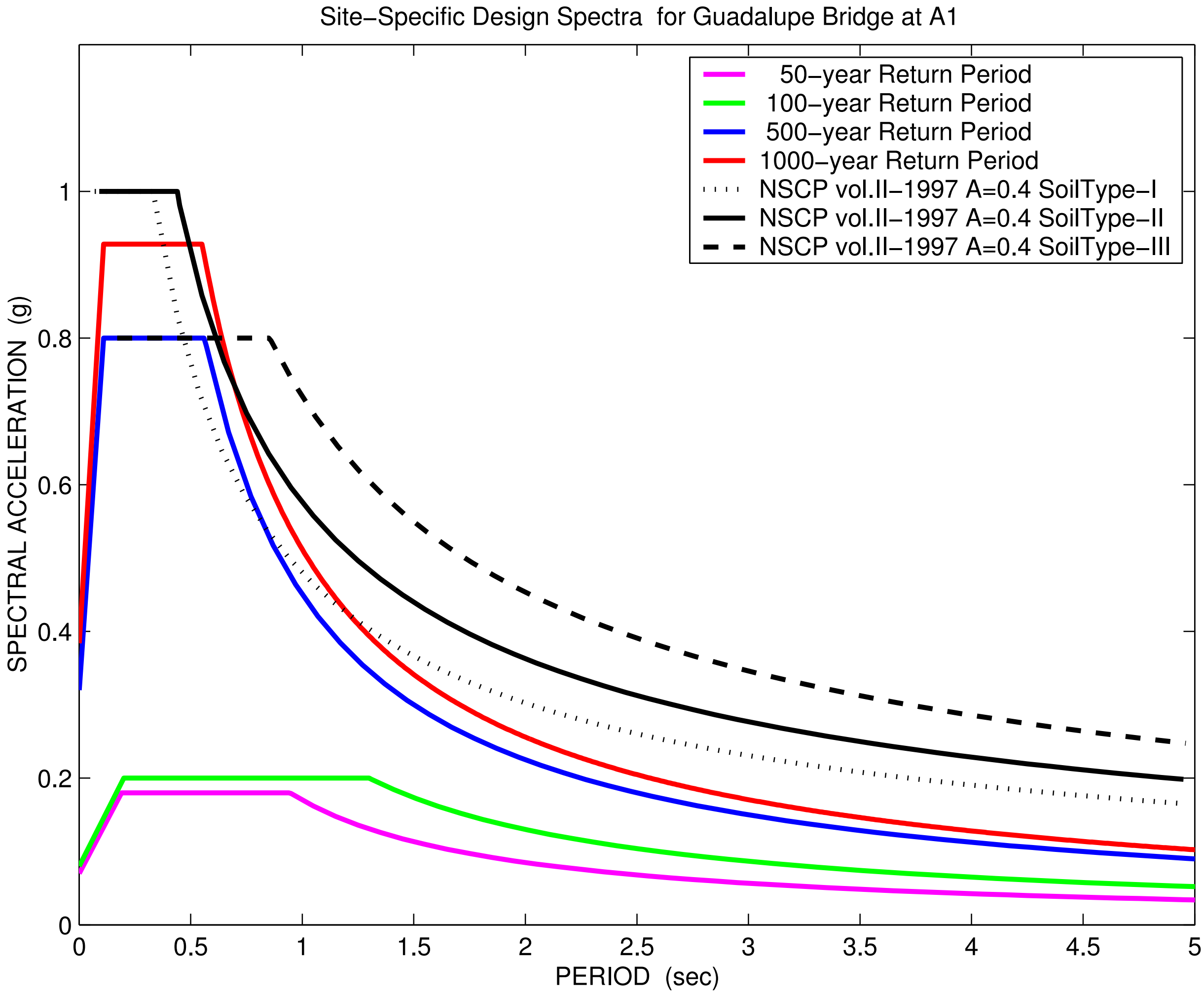


Figure 2B-50 Site-specific design spectra at 50-, 100-, 500-, 1000-year return periods for Guadalupe Bridge A1 site vs. NSCP-Bridges-1997

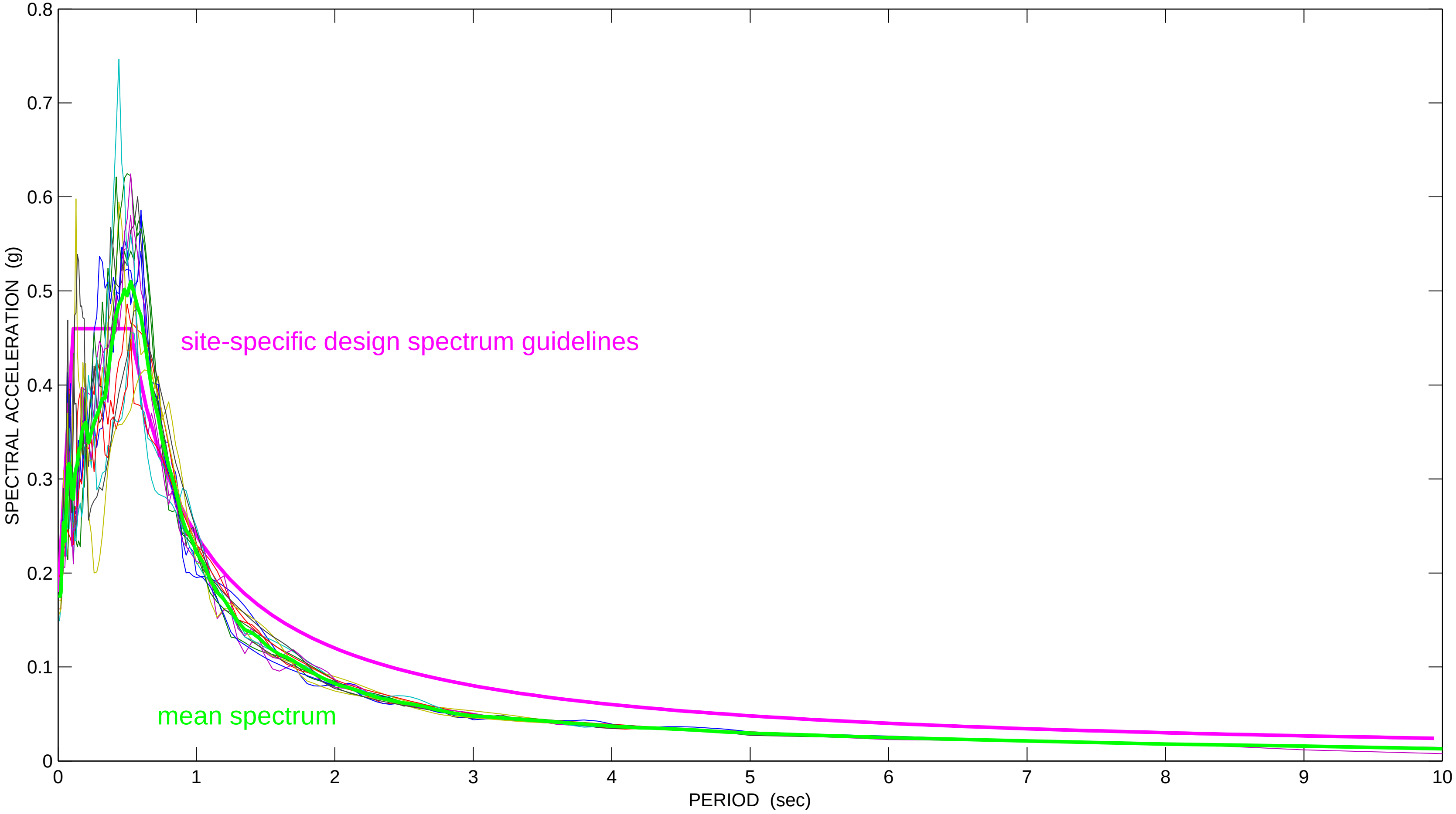


Figure 2B-51 Construction of site-specific design spectrum at 50-year return period for Guadalupe Bridge C2 based on mean of 14 response spectra from nonlinear site response analysis

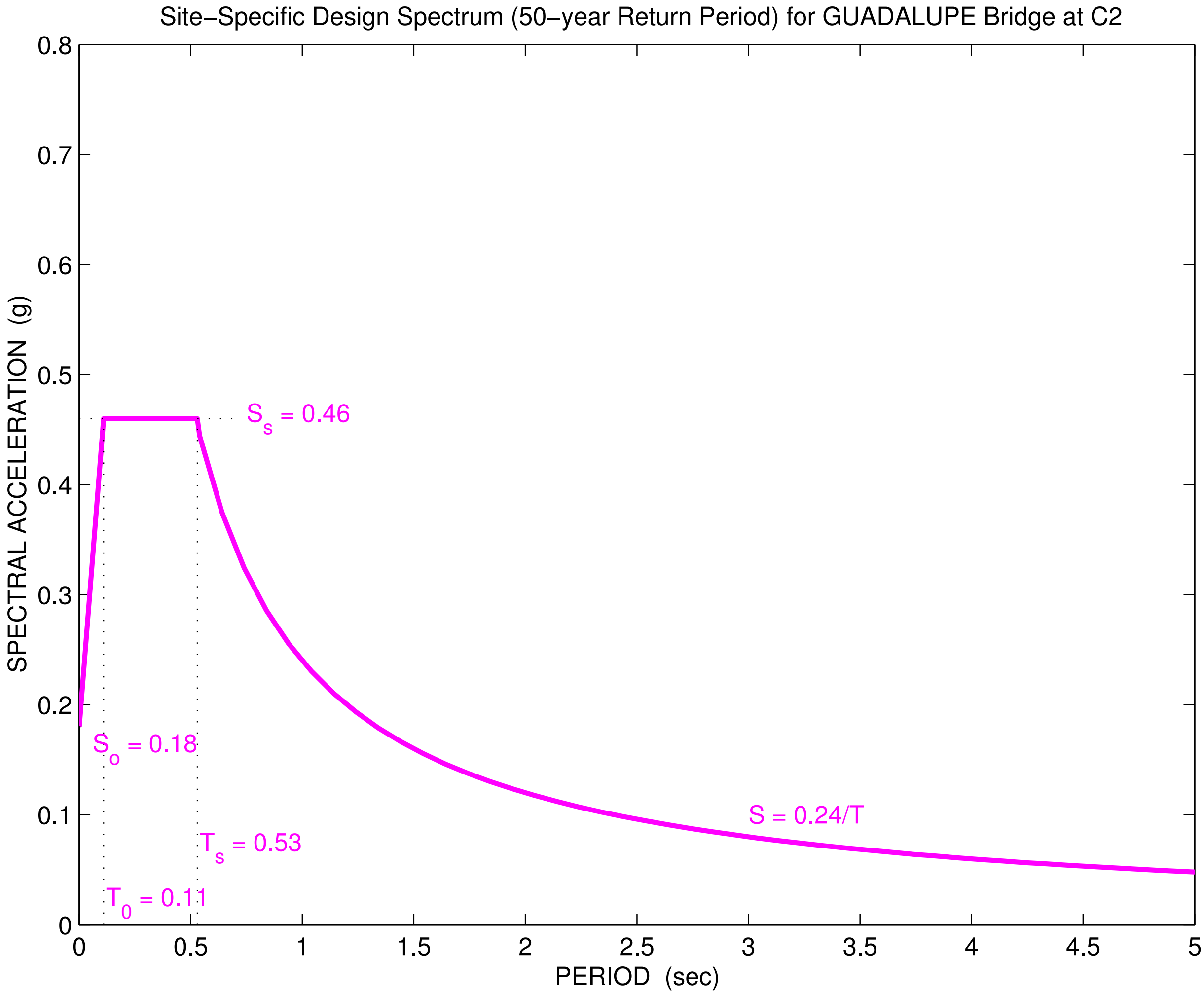


Figure 2B-52 Site-specific design spectrum at 50-year return period for Guadalupe Bridge at C2 site

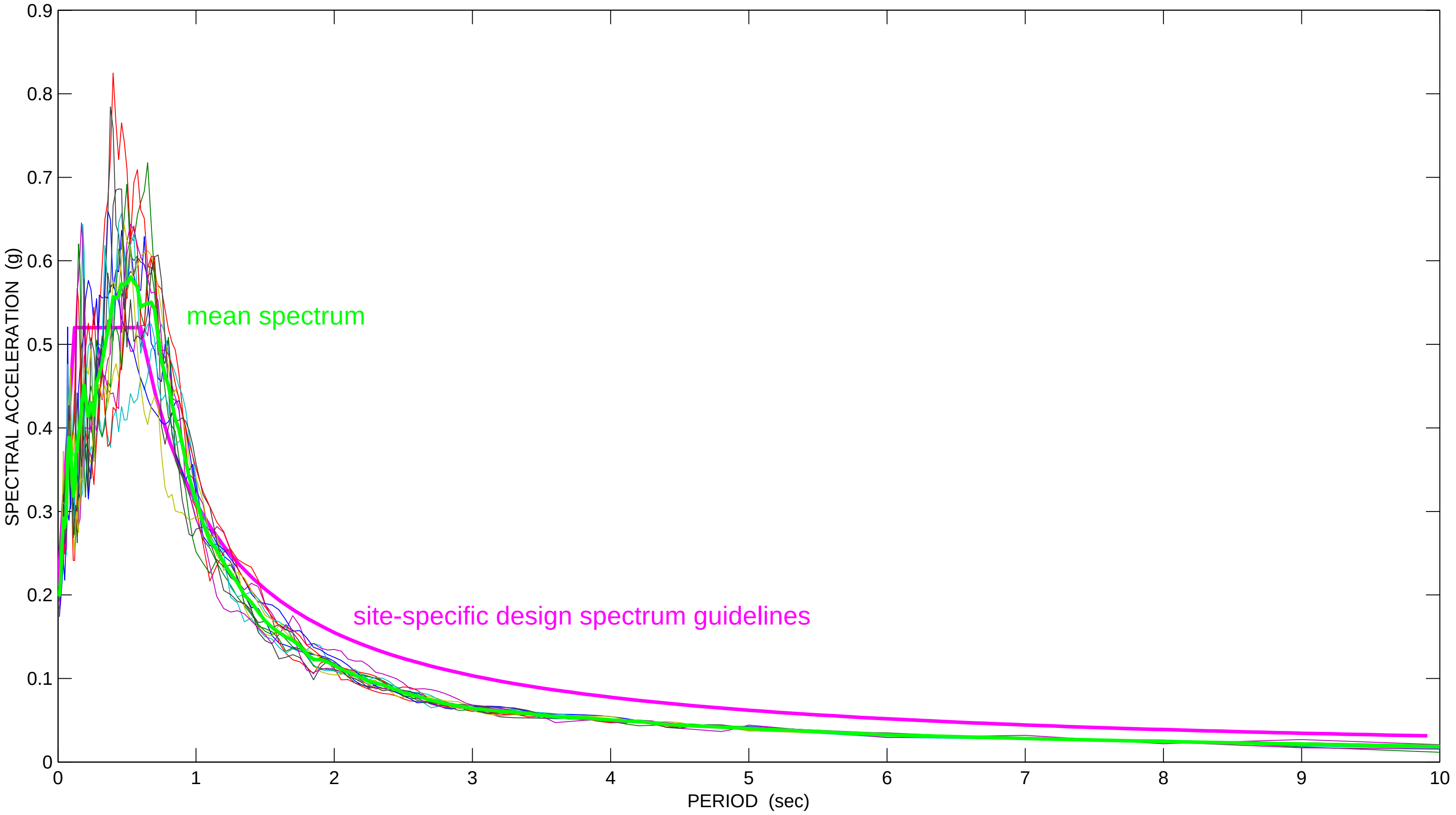


Figure 2B-53 Construction of site-specific design spectrum at 100-year return period for Guadalupe Bridge C2 based on mean of 14 response spectra from nonlinear site response analysis

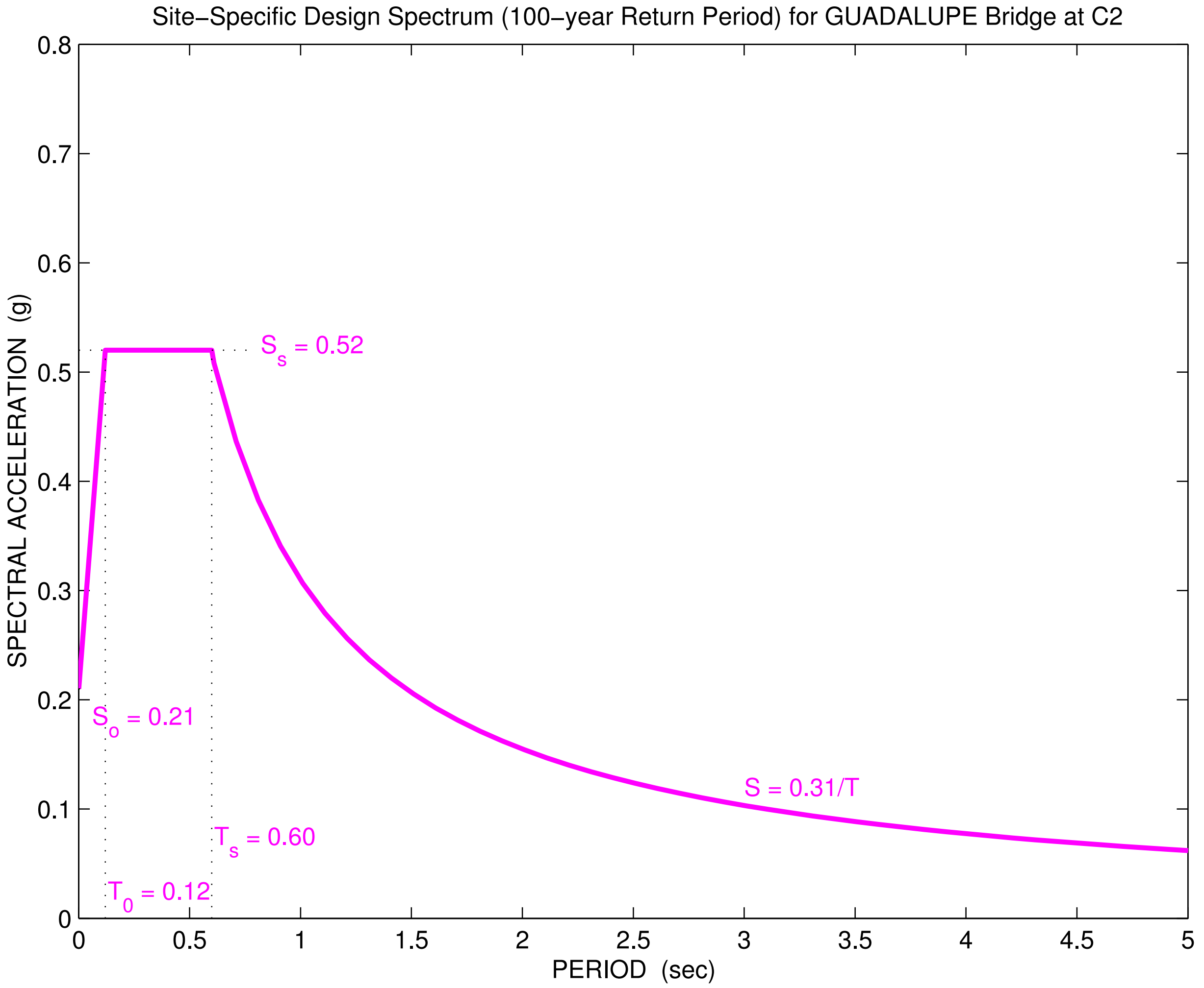


Figure 2B-54 Site-specific design spectrum at 100-year return period for Guadalupe Bridge at C2 site

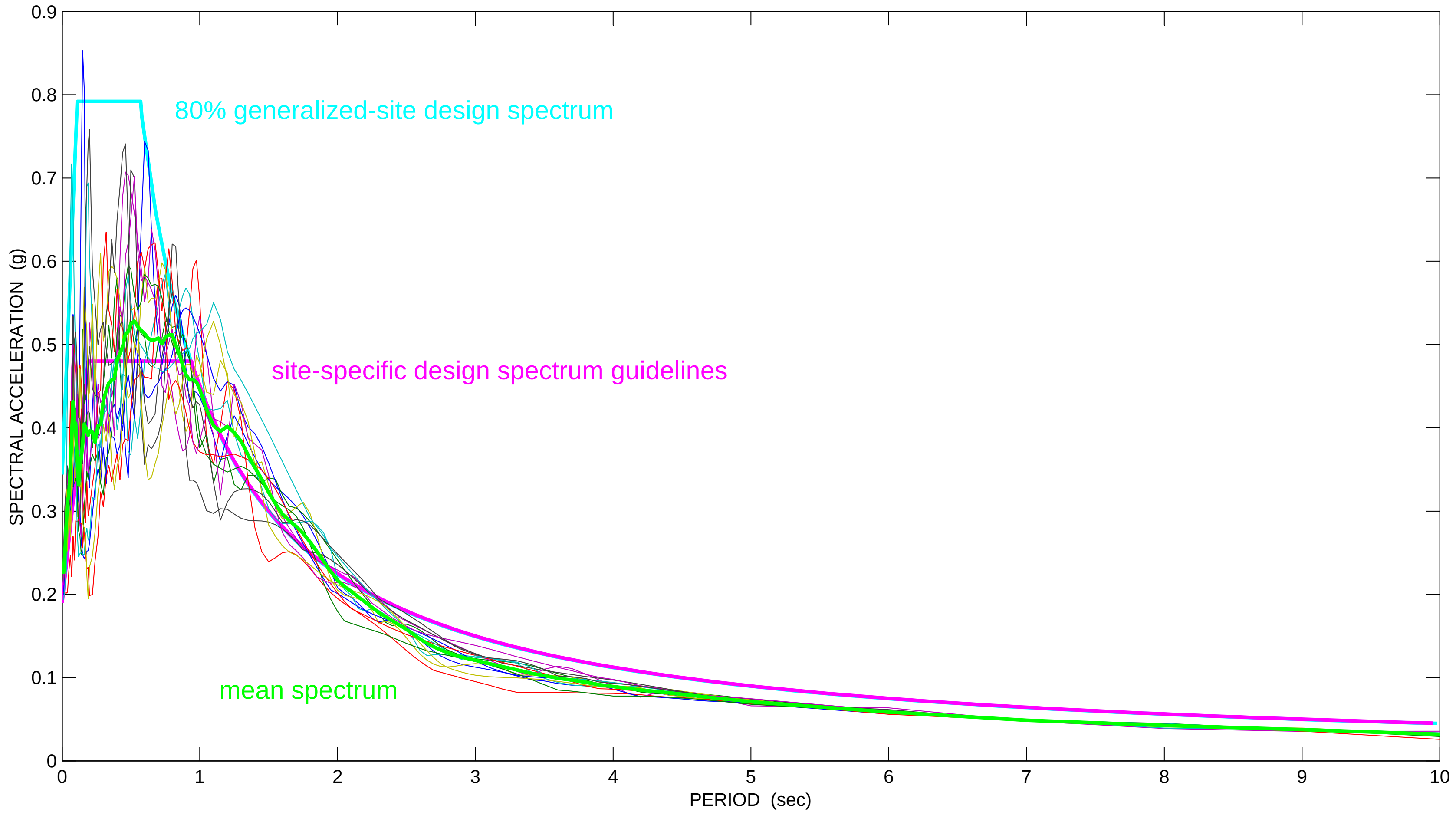


Figure 2B-55 Construction of site-specific design spectrum at 500-year return period for Guadalupe Bridge C2 based on mean of 14 response spectra from nonlinear site response analysis

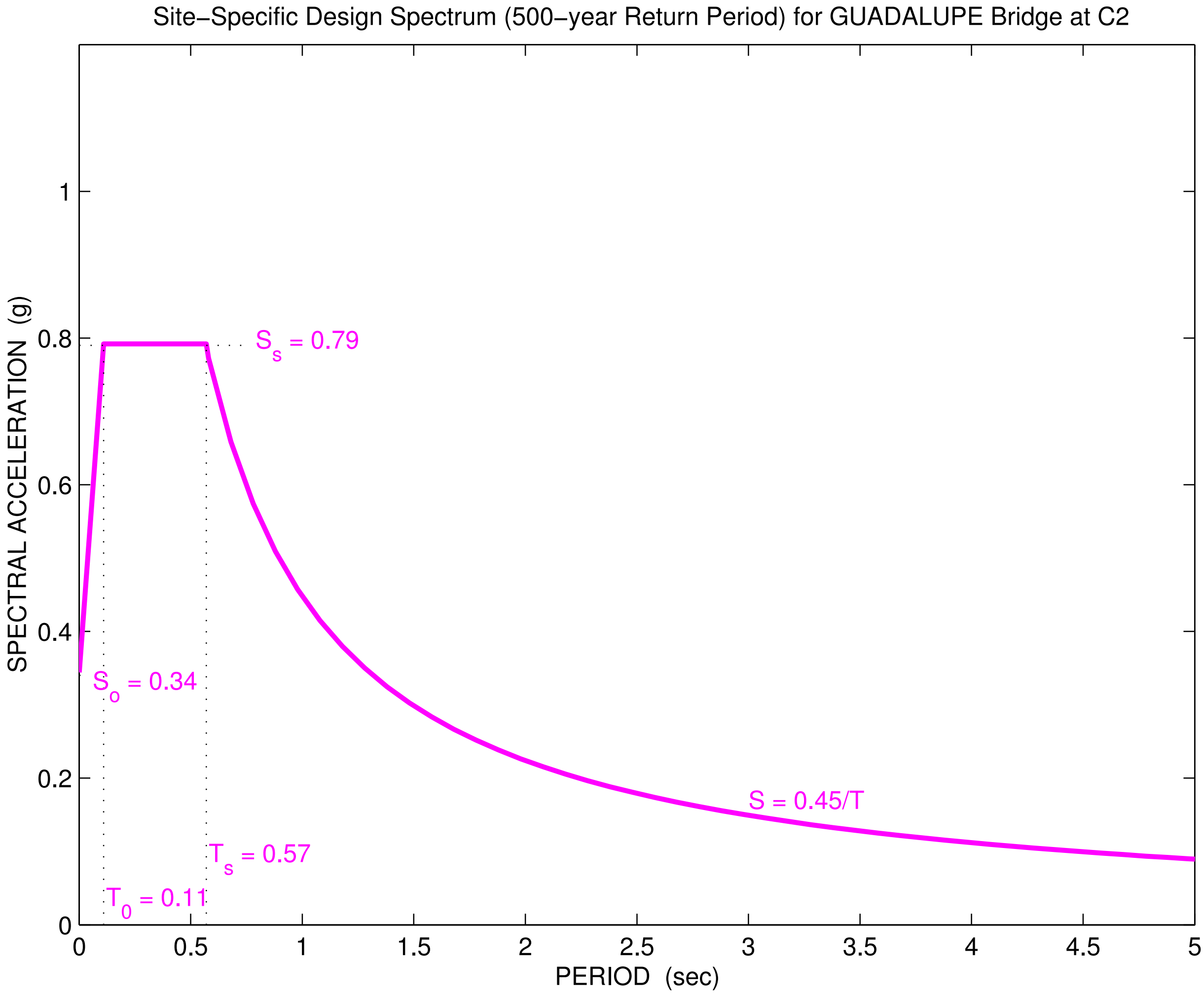


Figure 2B-56 Site-specific design spectrum at 500-year return period for Guadalupe Bridge at C2 site

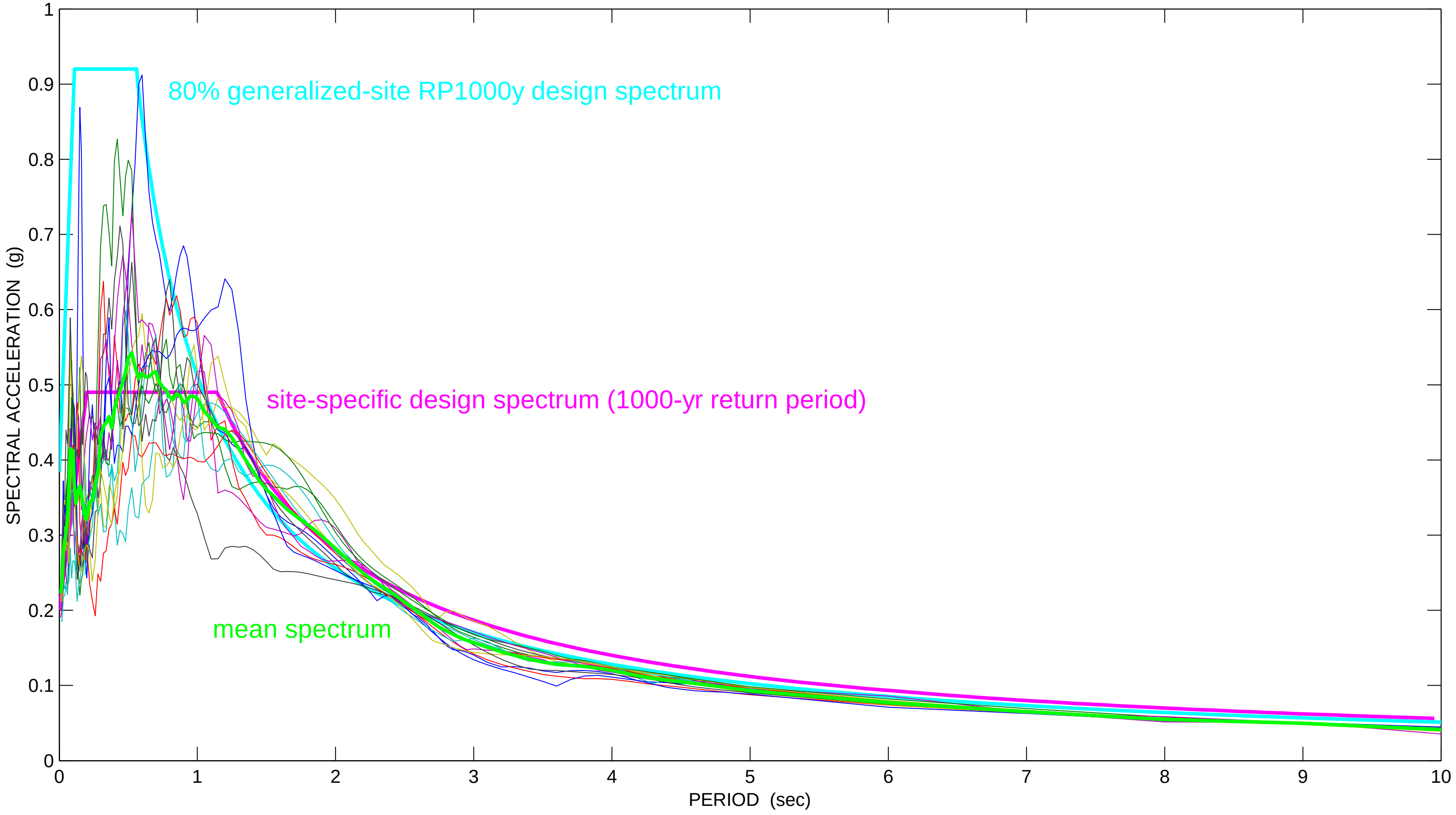


Figure 2B-57 Construction of site-specific design spectrum at 1000-year return period for Guadalupe Bridge C2 based on mean of 14 response spectra from nonlinear site response analysis

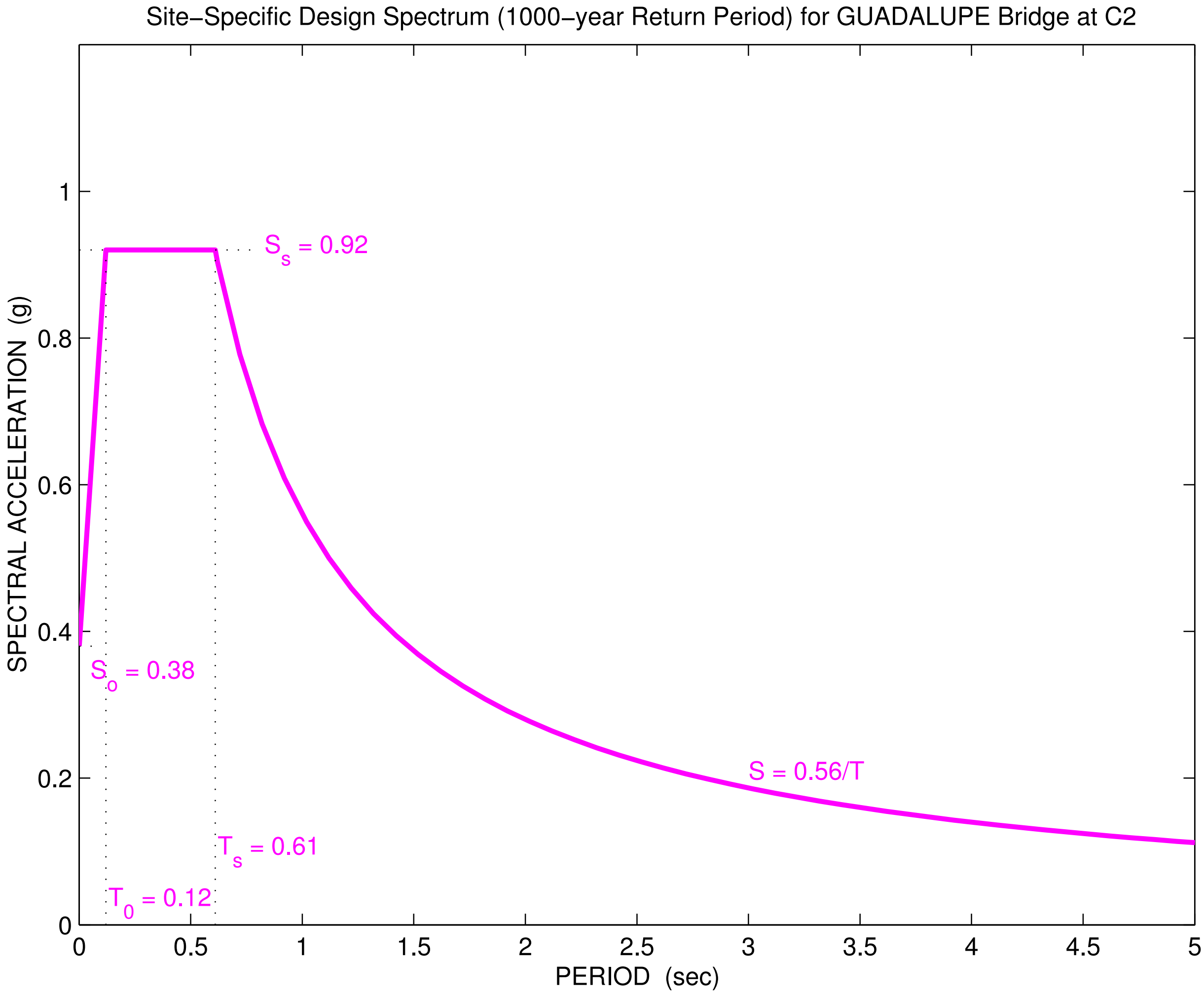


Figure 2B-58 Site-specific design spectrum at 1000-year return period for Guadalupe Bridge at C2 site

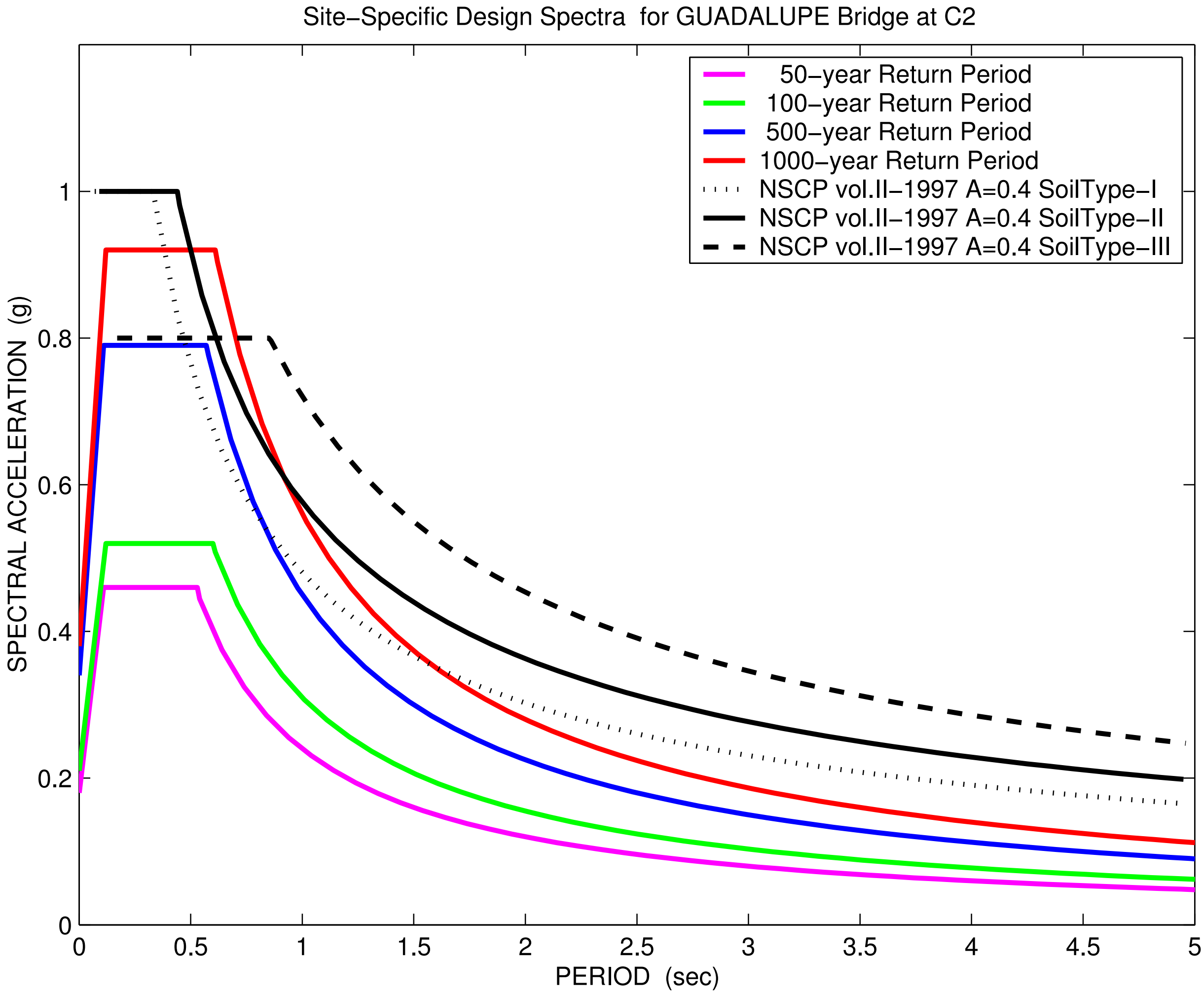


Figure 2B-59 Site-specific design spectra at 50-, 100-, 500-, 1000-year return periods for Guadalupe Bridge C2 site vs. NSCP-Bridges-1997

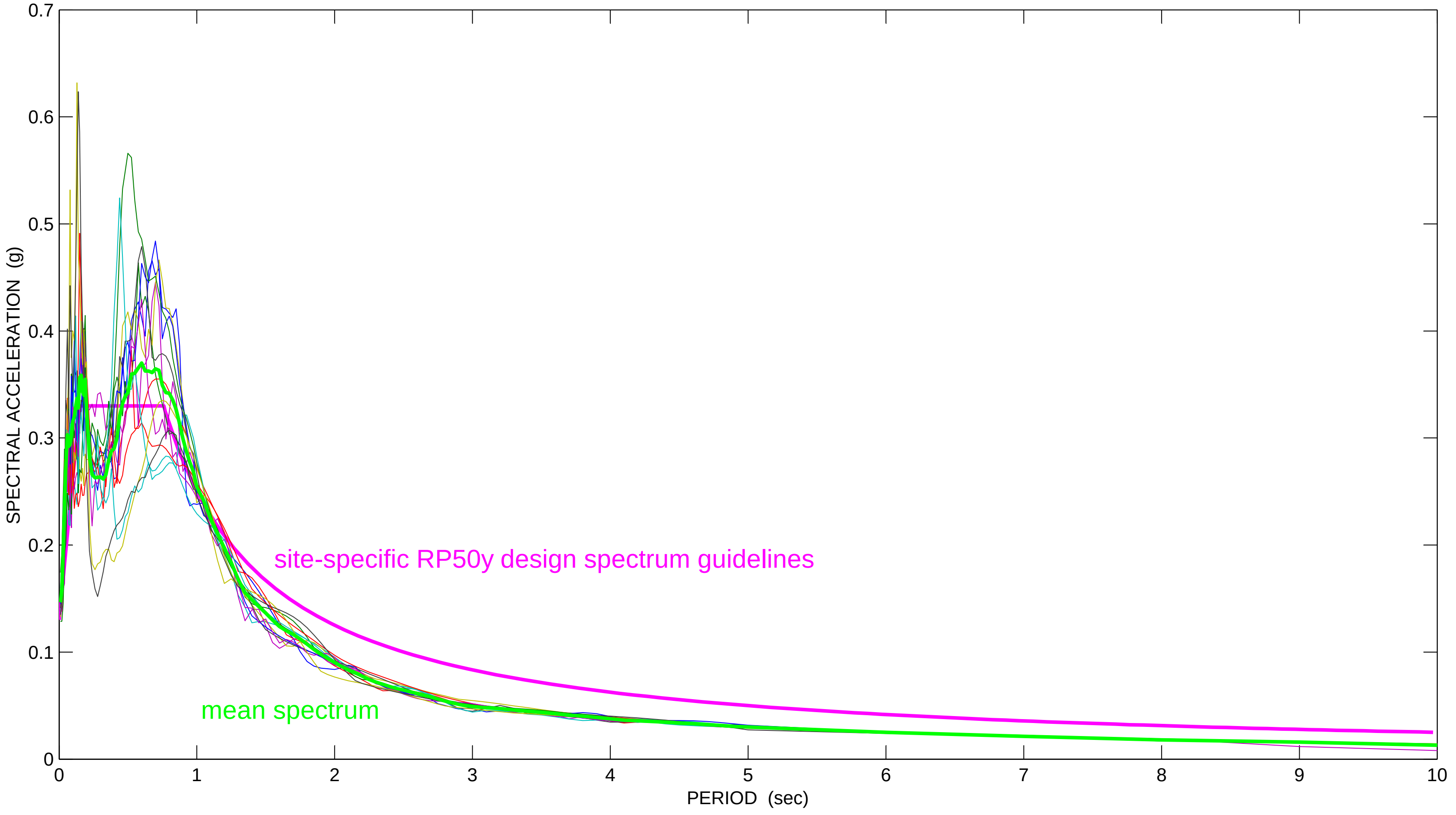


Figure 2B-60 Construction of site-specific design spectrum at 50-year return period for Guadalupe Bridge B2 based on mean of 14 response spectra from nonlinear site response analysis

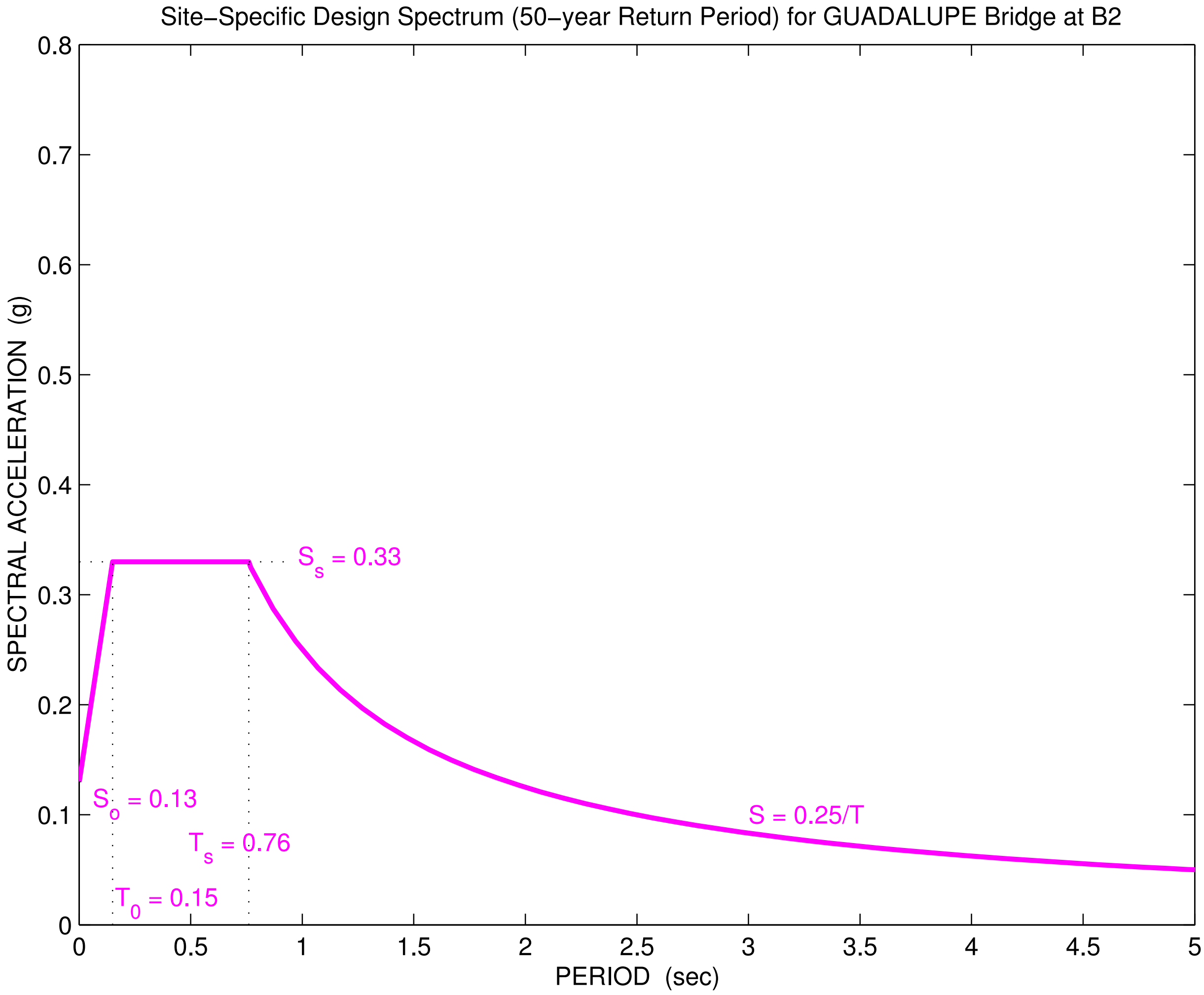


Figure 2B-61 Site-specific design spectrum at 50-year return period for Guadalupe Bridge at B2 site

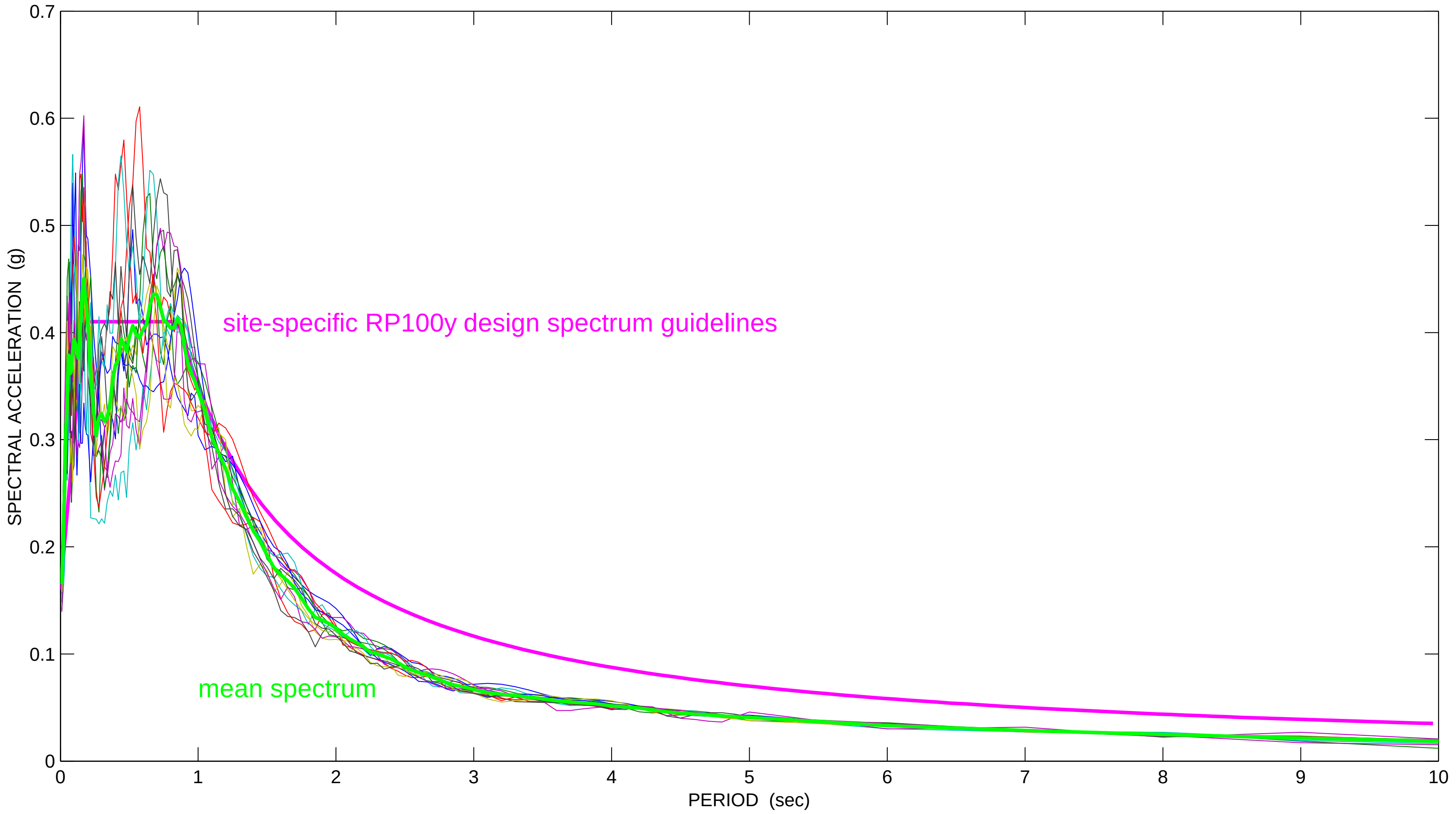


Figure 2B-62 Construction of site-specific design spectrum at 100-year return period for Guadalupe Bridge B2 based on mean of 14 response spectra from nonlinear site response analysis

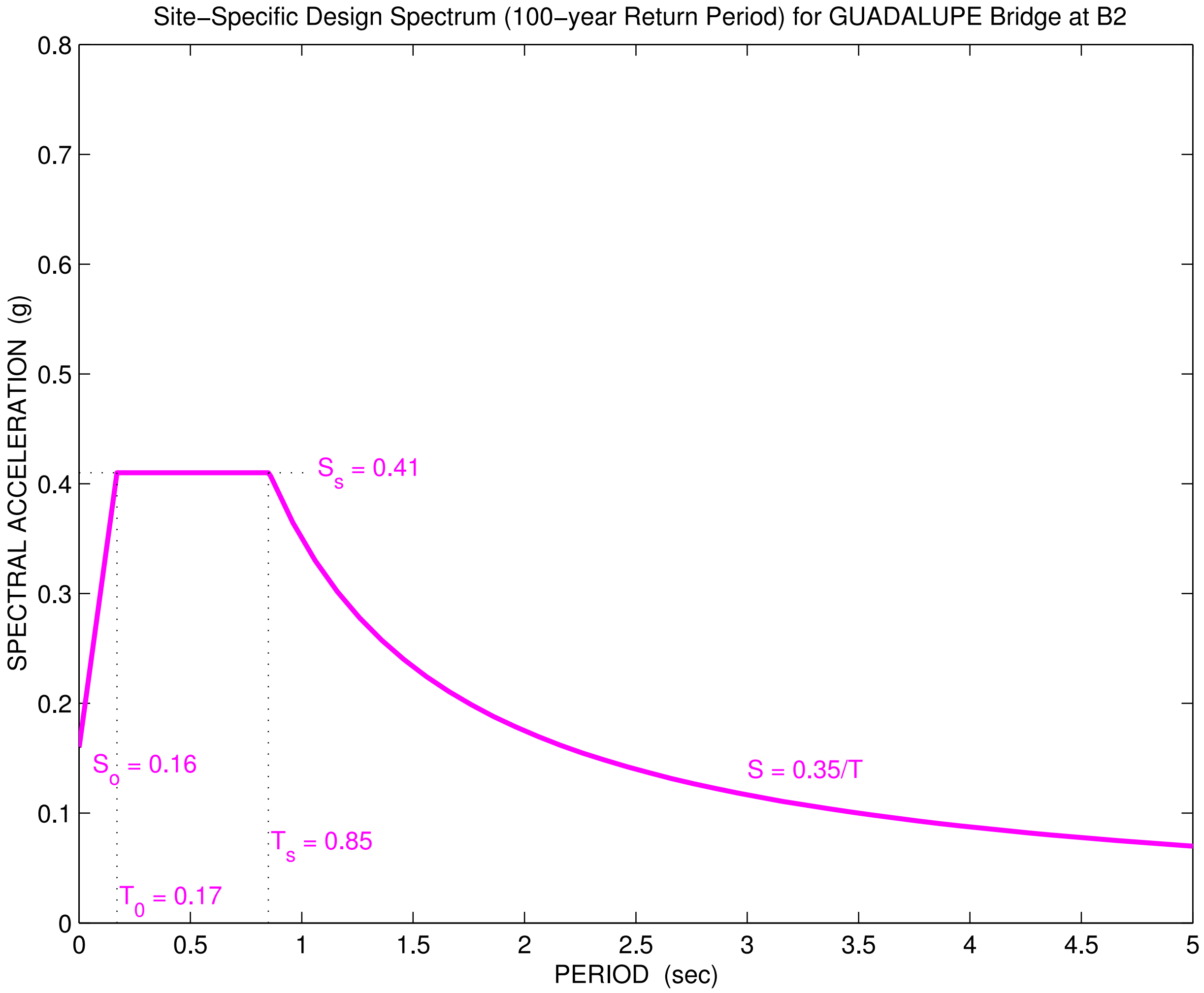


Figure 2B-63 Site-specific design spectrum at 100-year return period for Guadalupe Bridge at B2 site

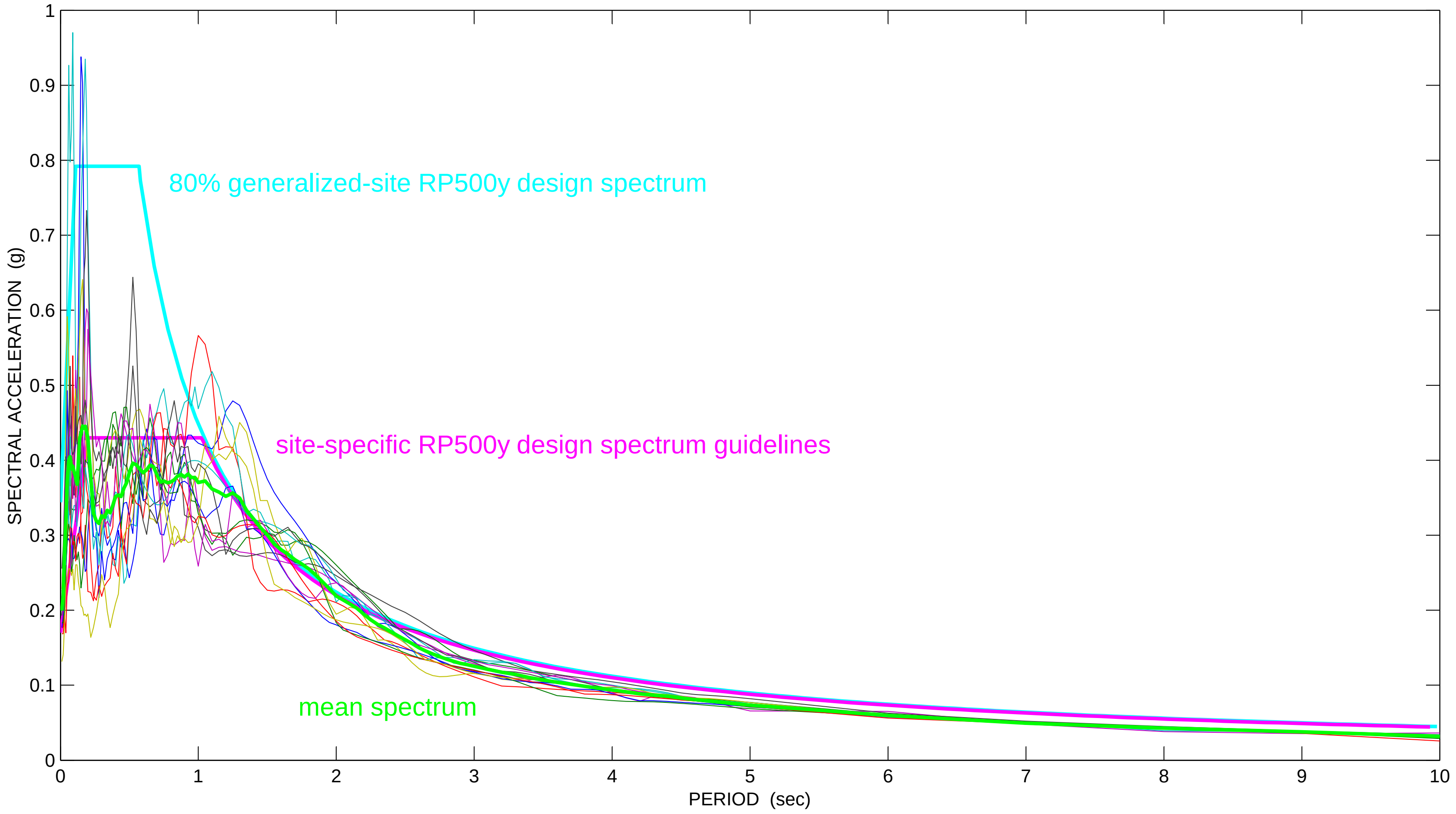


Figure 2B-64 Construction of site-specific design spectrum at 500-year return period for Guadalupe Bridge B2 based on mean of 14 response spectra from nonlinear site response analysis

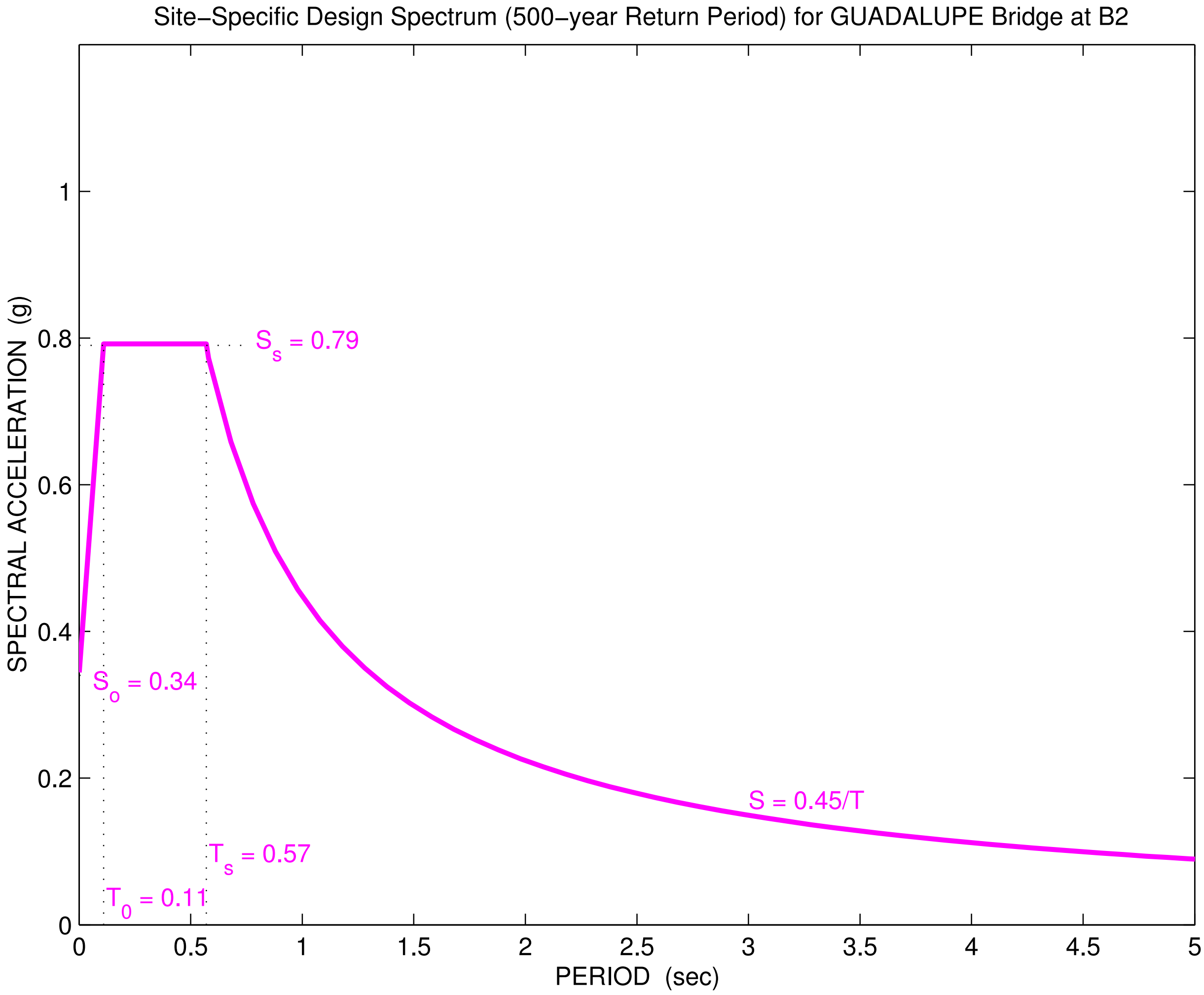


Figure 2B-65 Site-specific design spectrum at 500-year return period for Guadalupe Bridge at B2 site

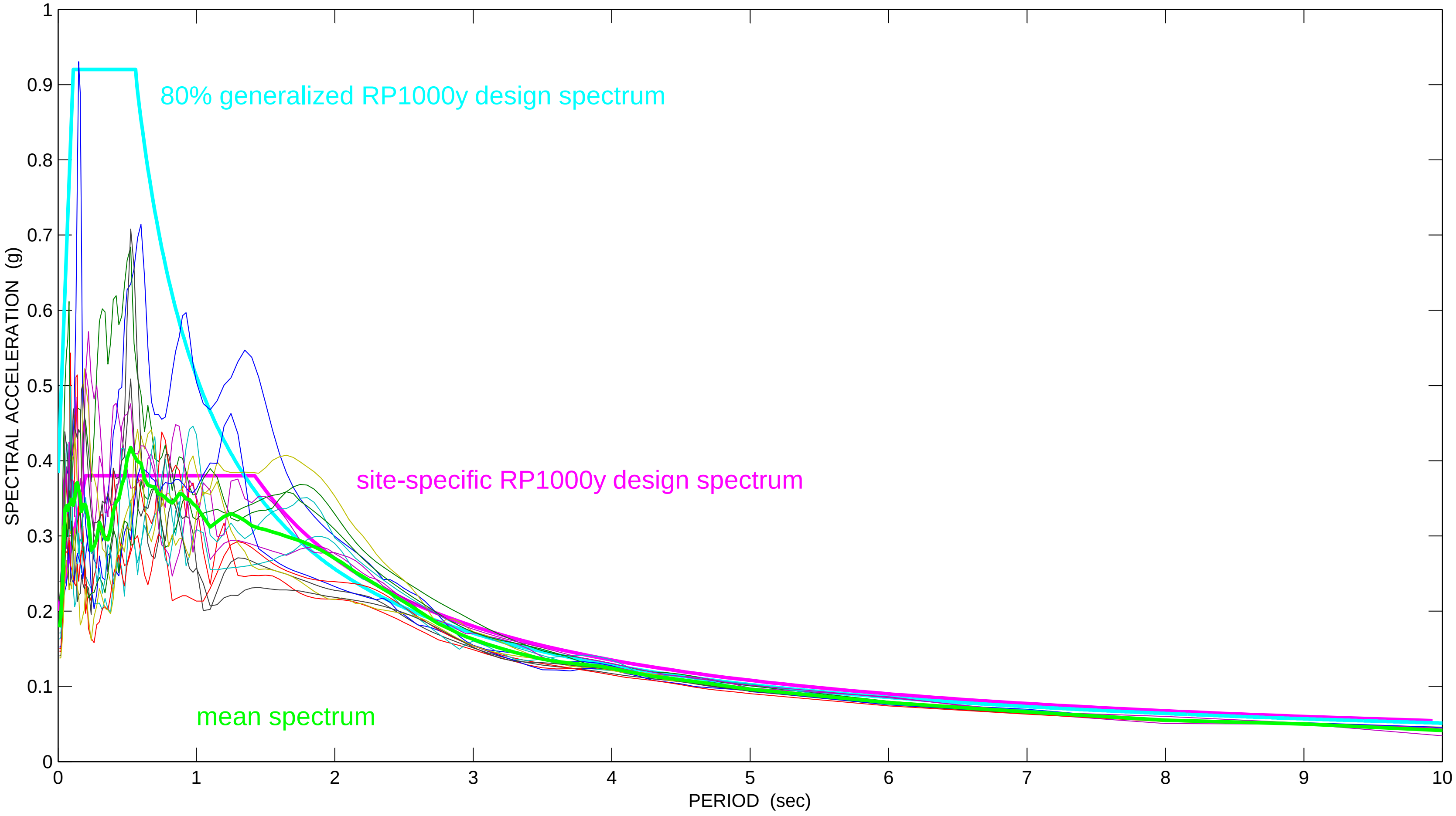


Figure 2B-66 Construction of site-specific design spectrum at 1000-year return period for Guadalupe Bridge B2 based on mean of 14 response spectra from nonlinear site response analysis

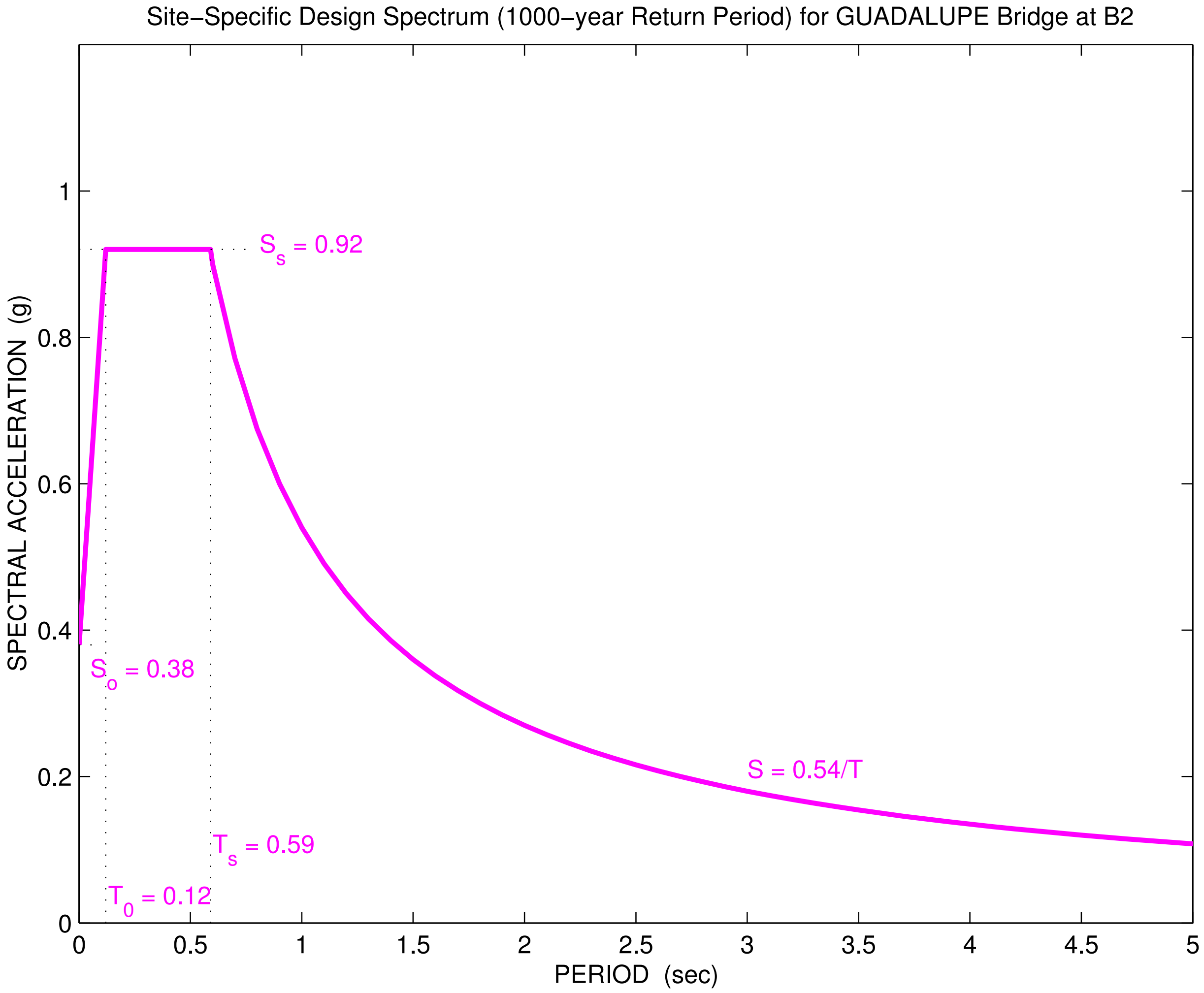


Figure 2B-67 Site-specific design spectrum at 1000-year return period for Guadalupe Bridge at B2 site

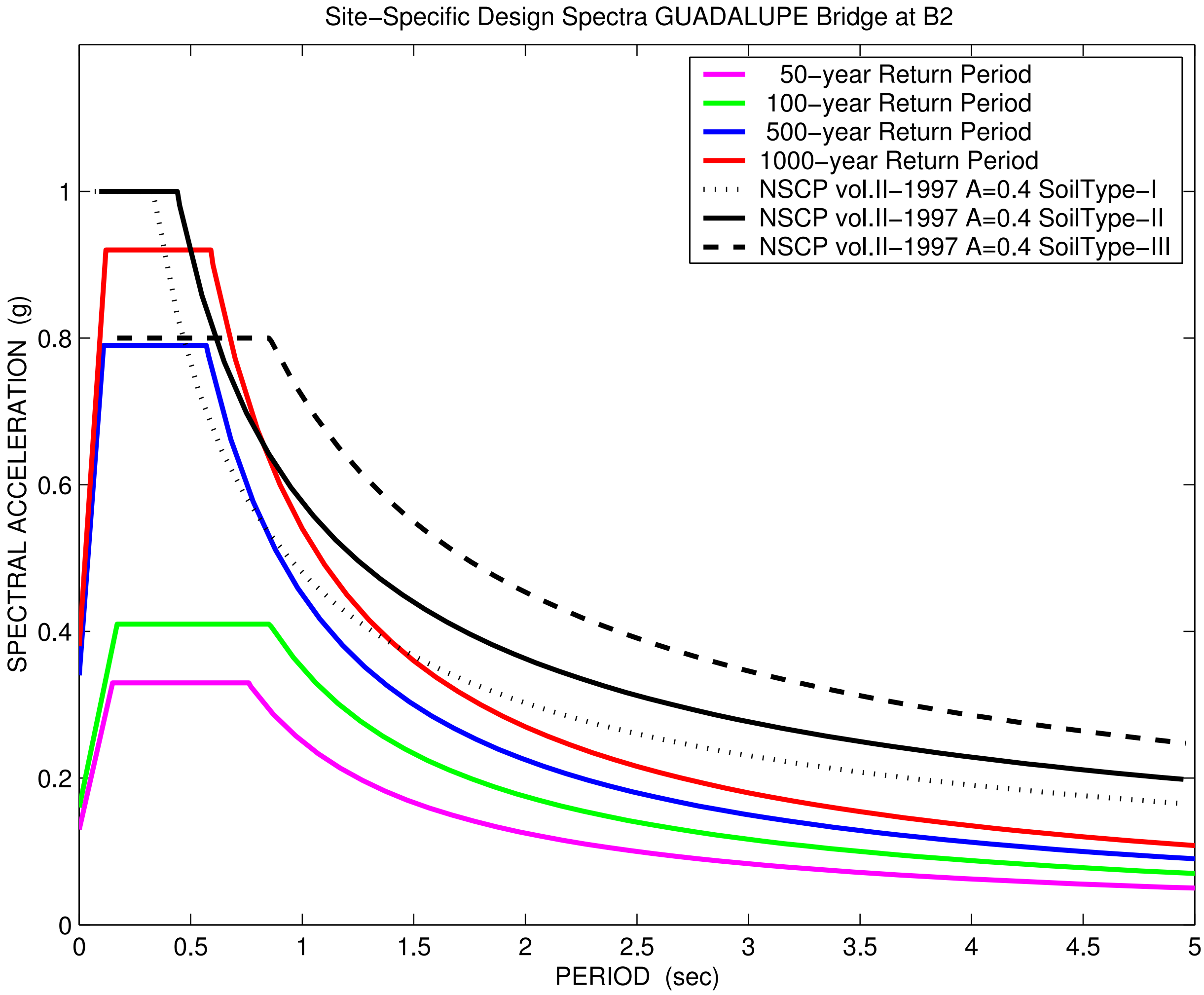


Figure 2B-68 Site-specific design spectra at 50-, 100-, 500-, 1000-year return periods for Guadalupe Bridge B2 site vs. NSCP-Bridges-1997

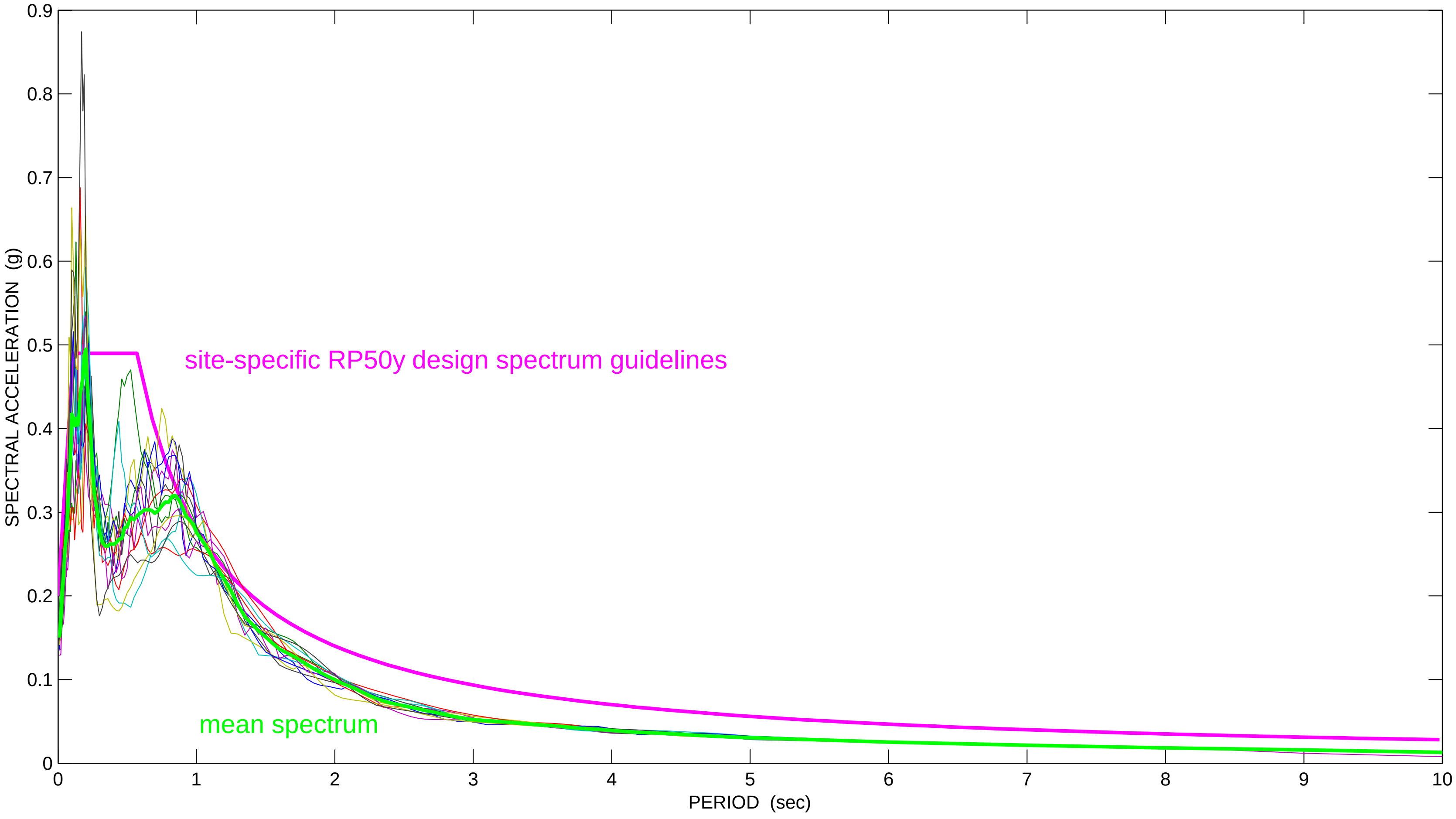


Figure 2B-69 Construction of site-specific design spectrum at 50-year return period for Guadalupe Bridge D2 based on mean of 14 response spectra from nonlinear site response analysis

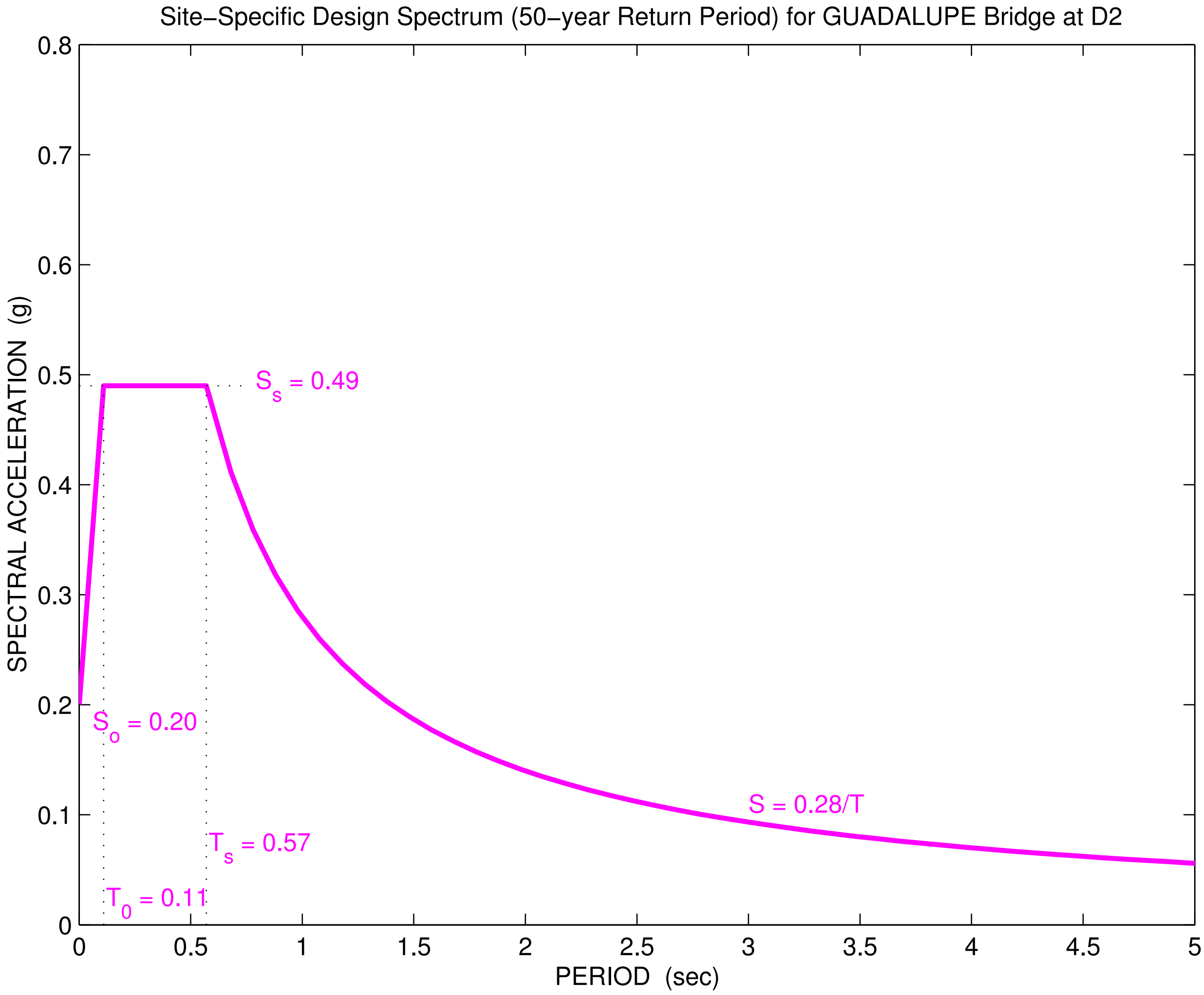


Figure 2B-70 Site-specific design spectrum at 50-year return period for Guadalupe Bridge at D2 site

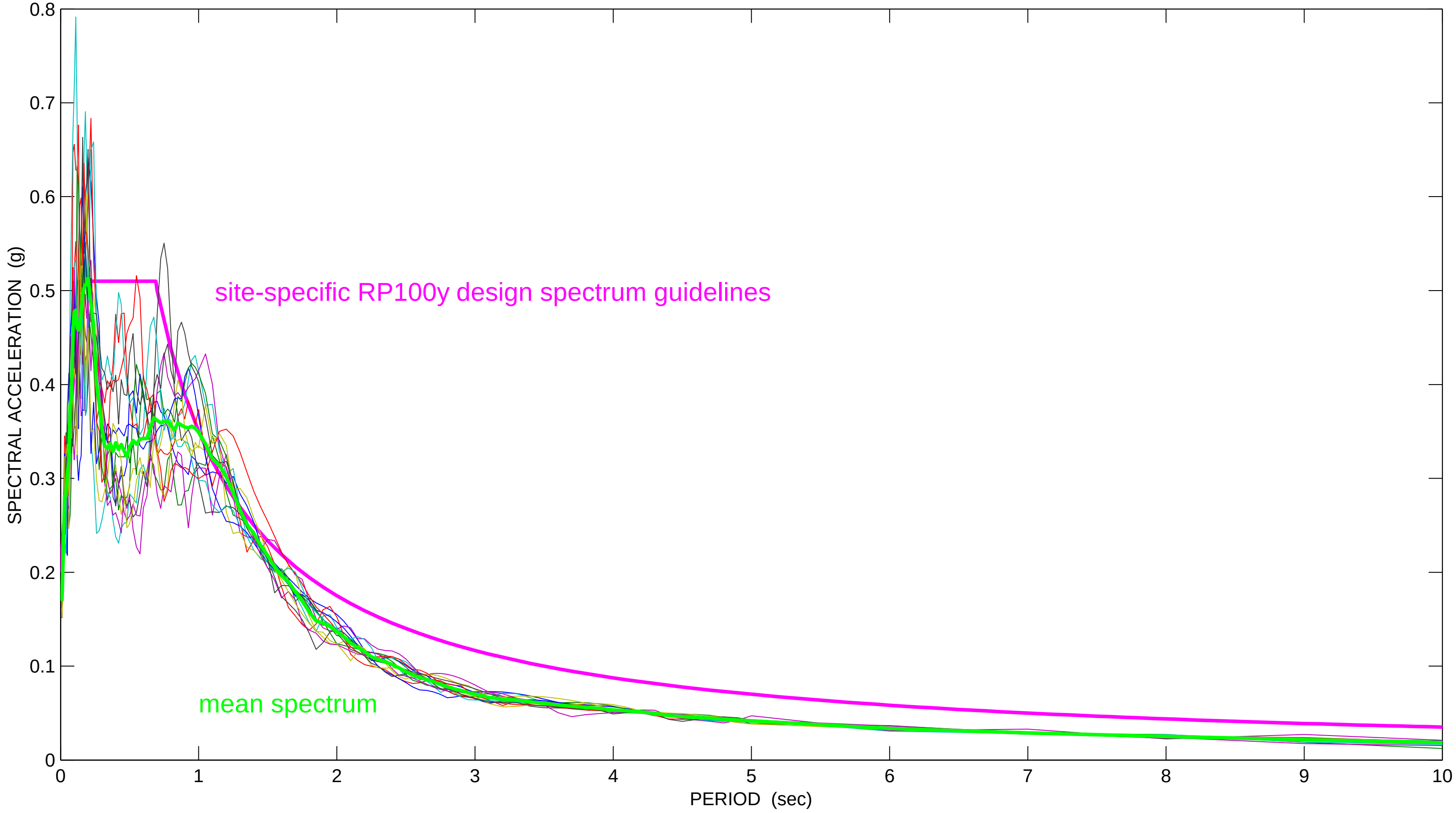


Figure 2B-71 Construction of site-specific design spectrum at 100-year return period for Guadalupe Bridge D2 based on mean of 14 response spectra from nonlinear site response analysis

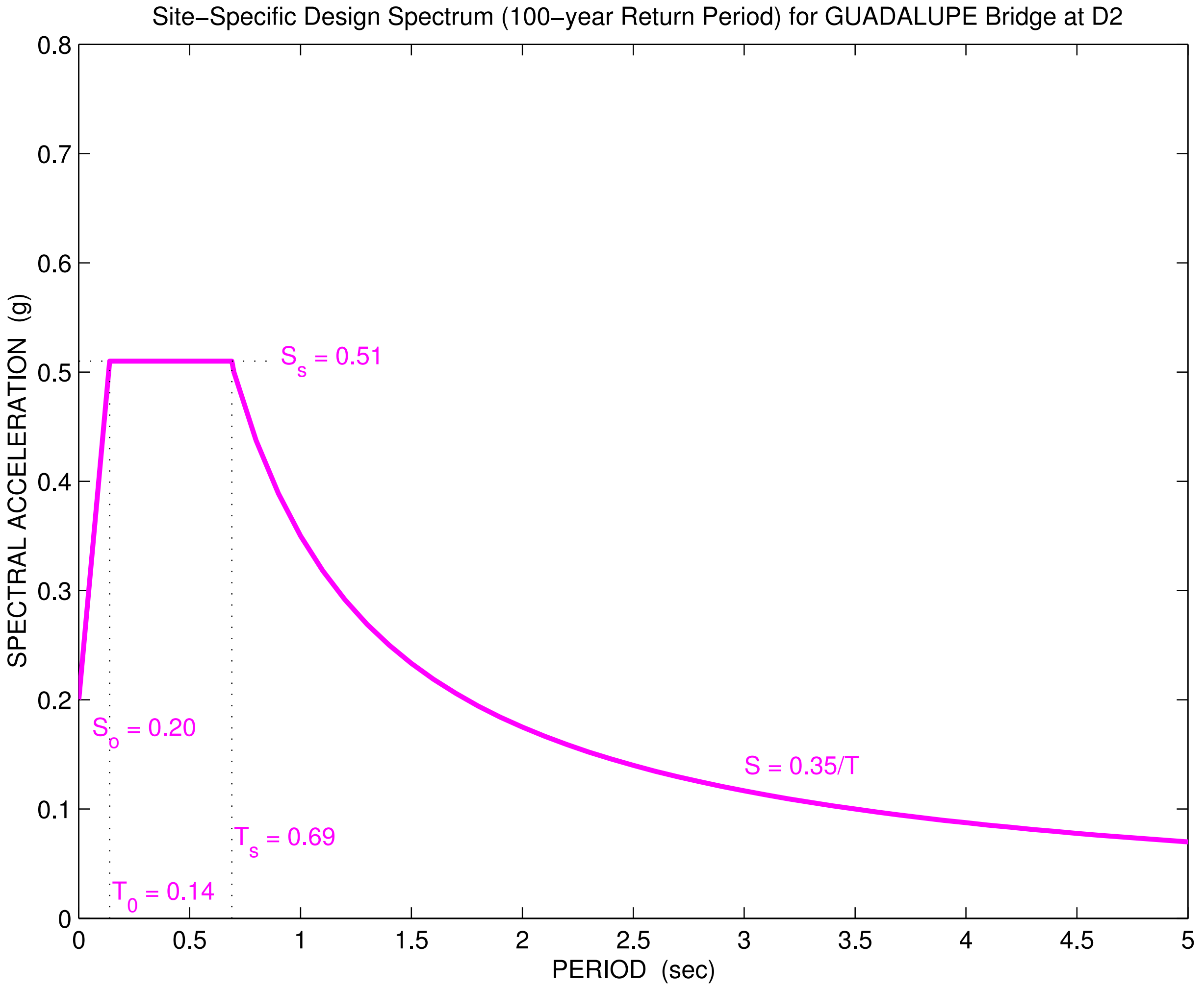


Figure 2B-72 Site-specific design spectrum at 100-year return period for Guadalupe Bridge at D2 site

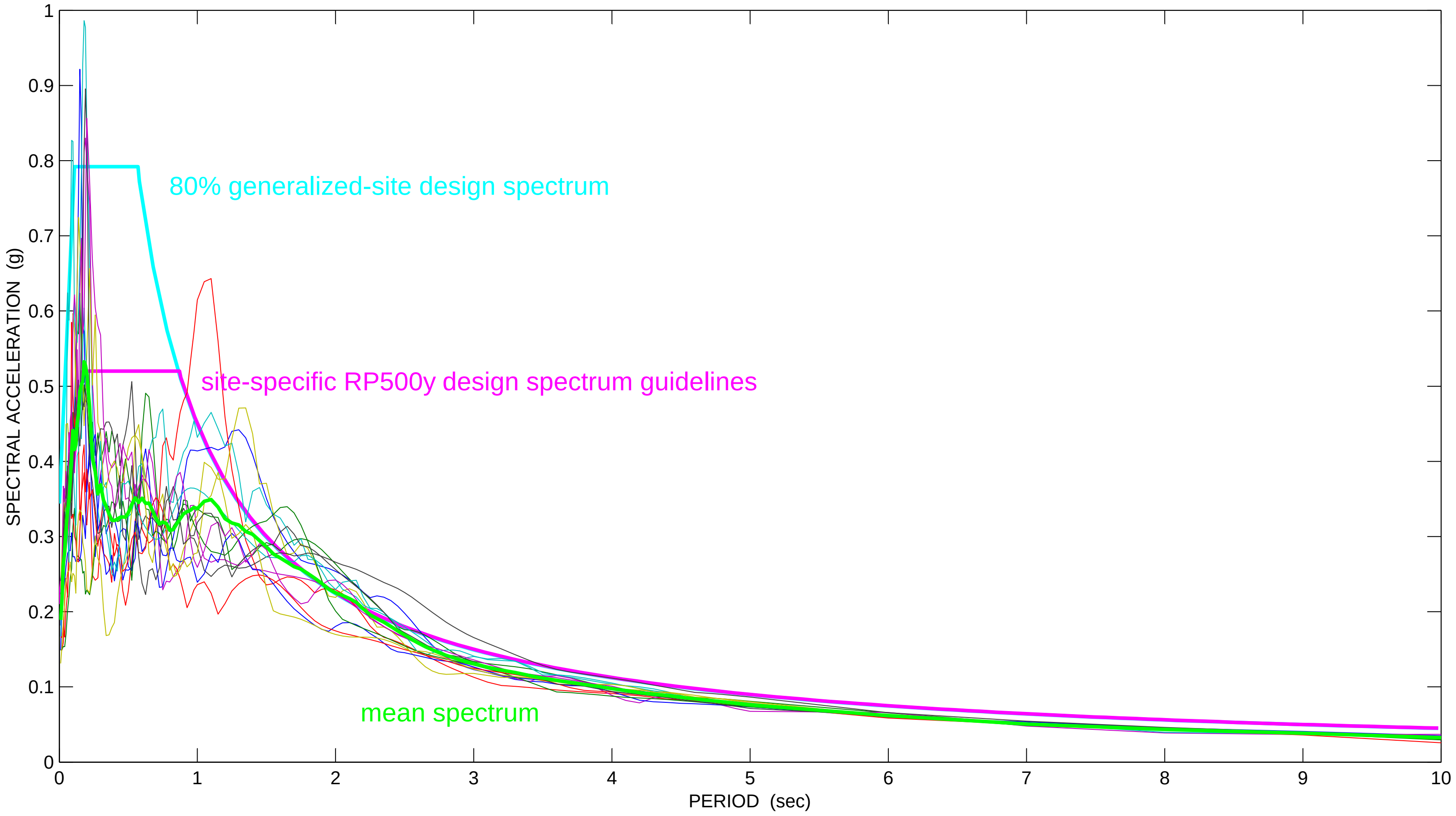


Figure 2B-73 Construction of site-specific design spectrum at 500-year return period for Guadalupe Bridge D2 based on mean of 14 response spectra from nonlinear site response analysis

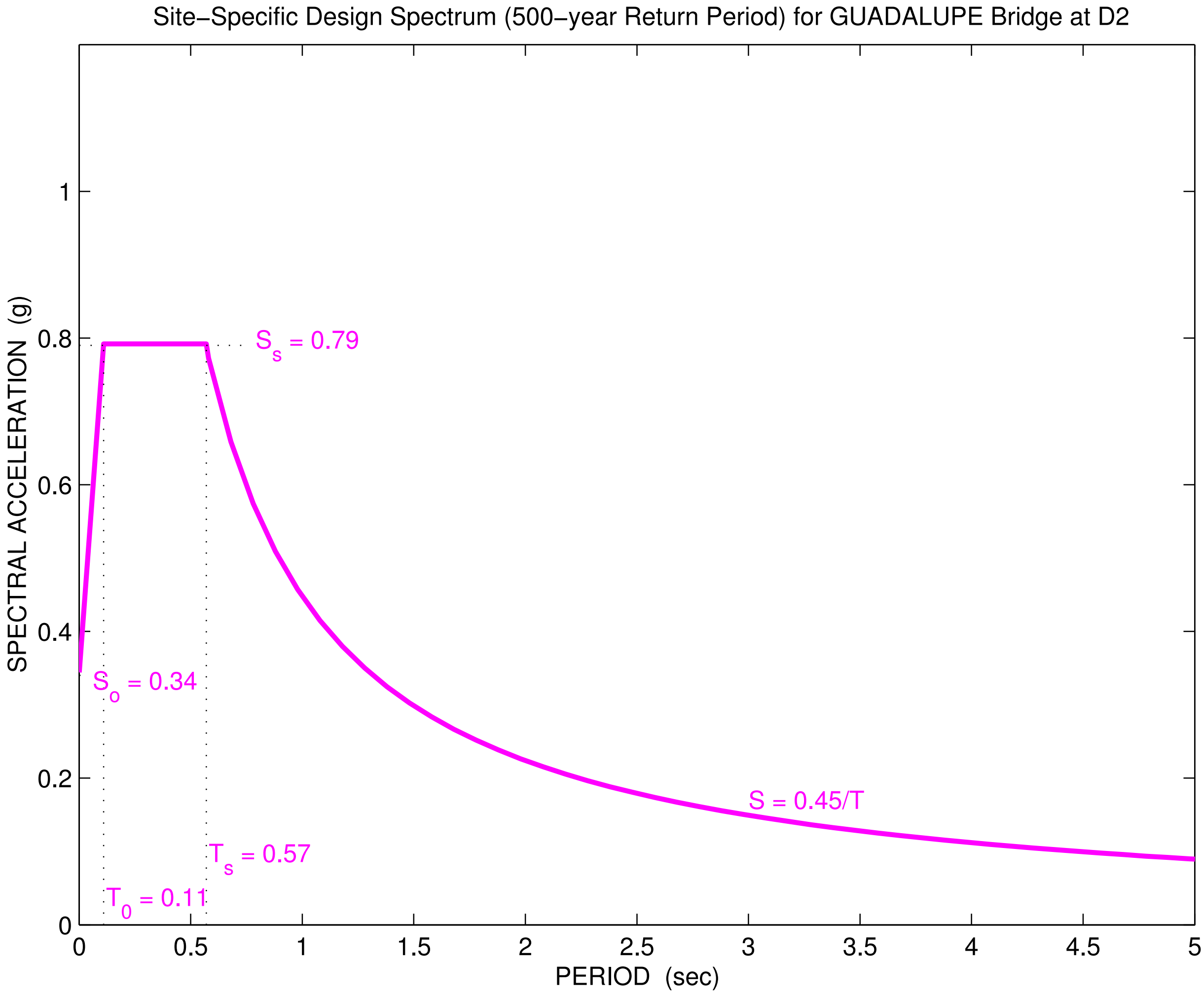


Figure 2B-74 Site-specific design spectrum at 500-year return period for Guadalupe Bridge at D2 site

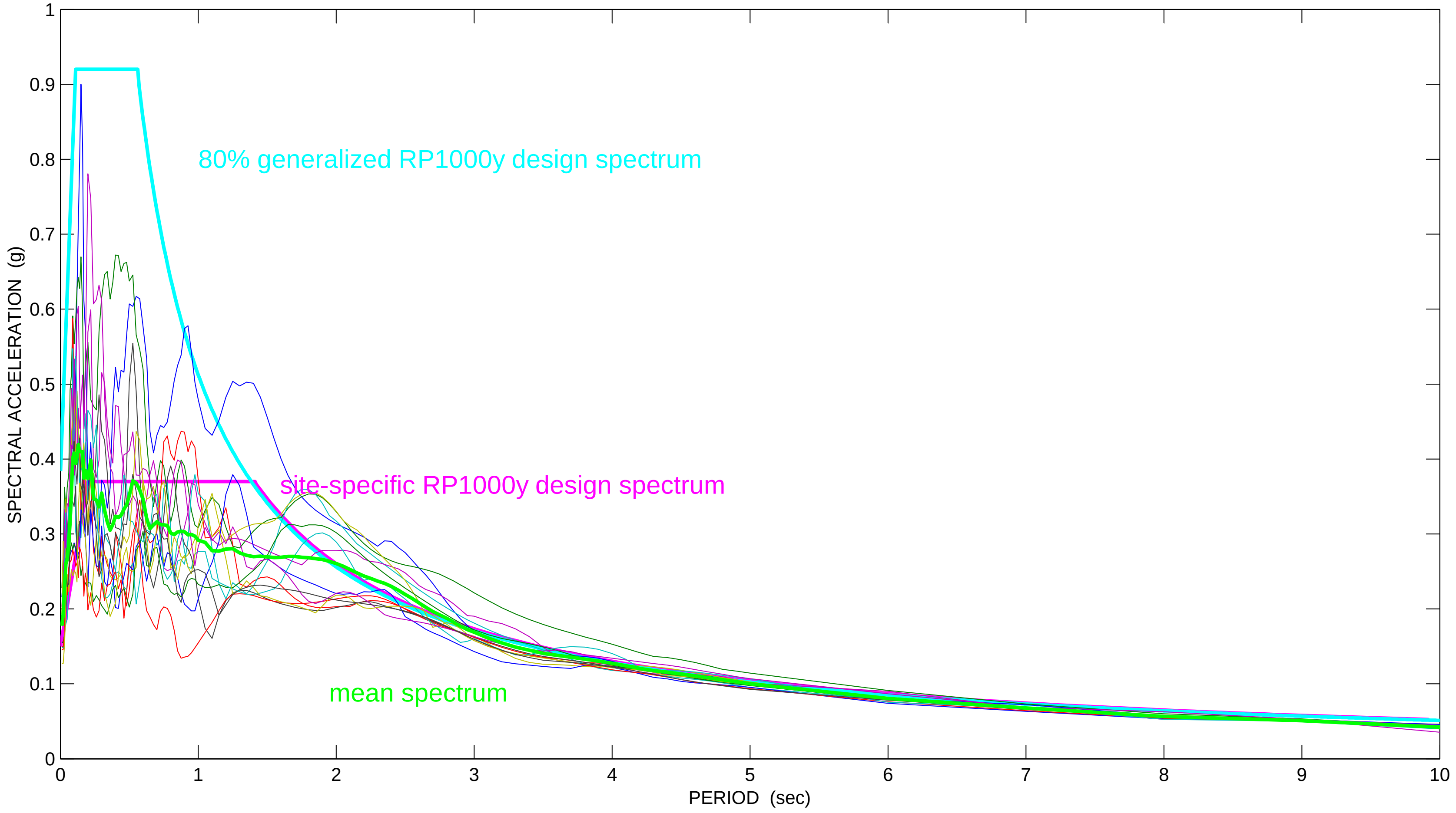


Figure 2B-75 Construction of site-specific design spectrum at 1000-year return period for Guadalupe Bridge D2 based on mean of 14 response spectra from nonlinear site response analysis

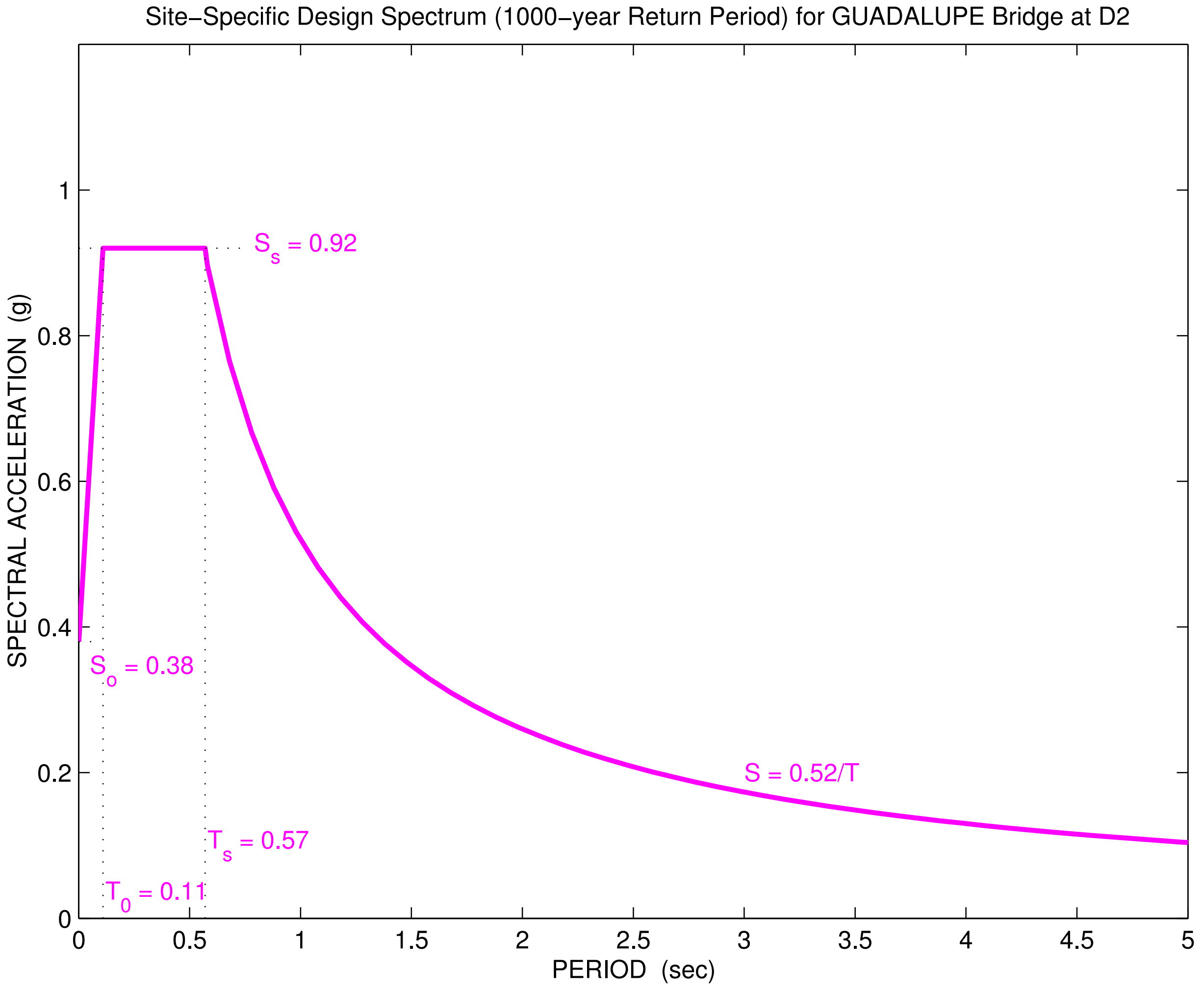


Figure 2B-76 Site-specific design spectrum at 1000-year return period for Guadalupe Bridge at D2 site

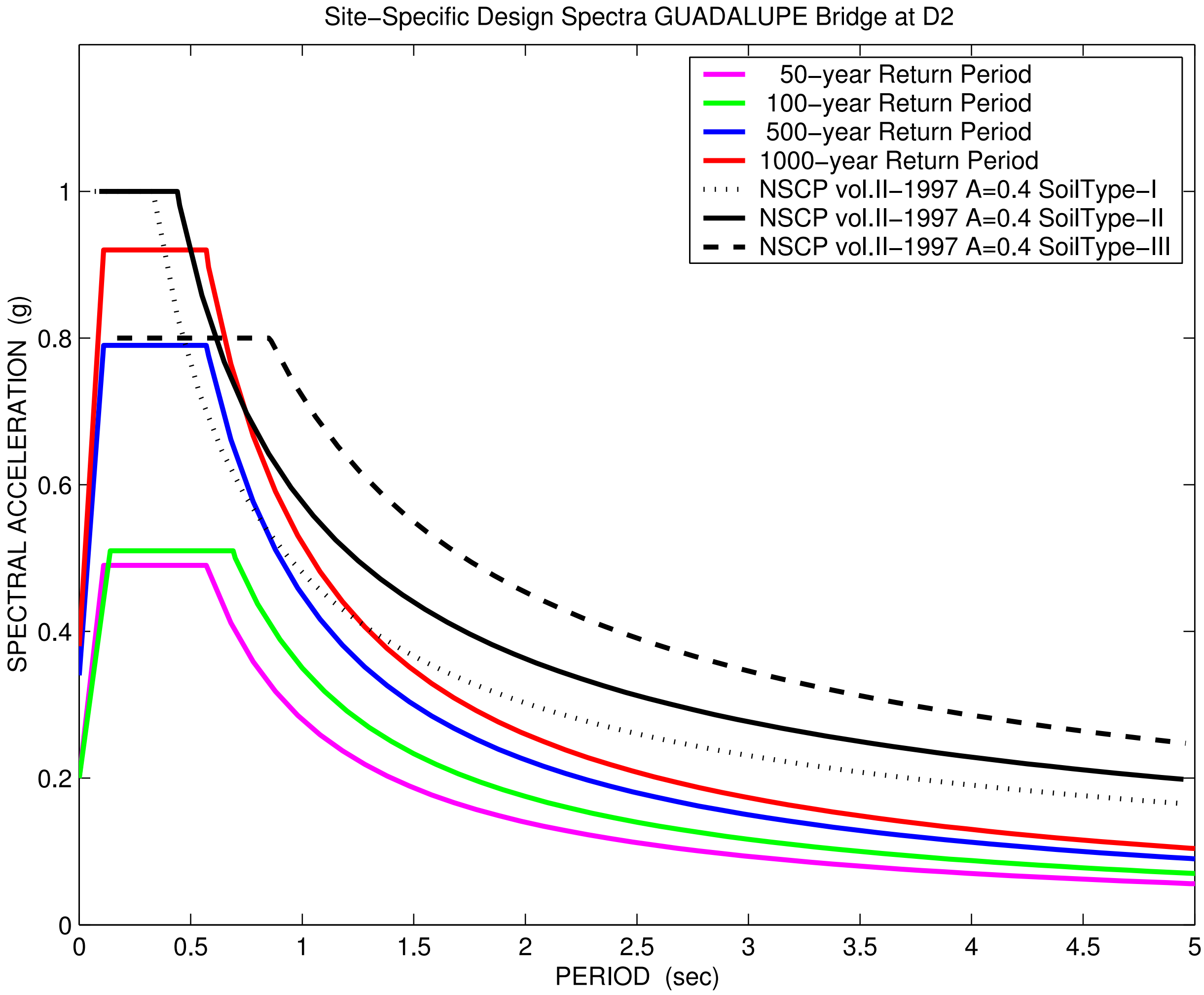


Figure 2B-77 Site-specific design spectra at 50-, 100-, 500-, 1000-year return periods for Guadalupe Bridge D2 site vs. NSCP-Bridges-1997

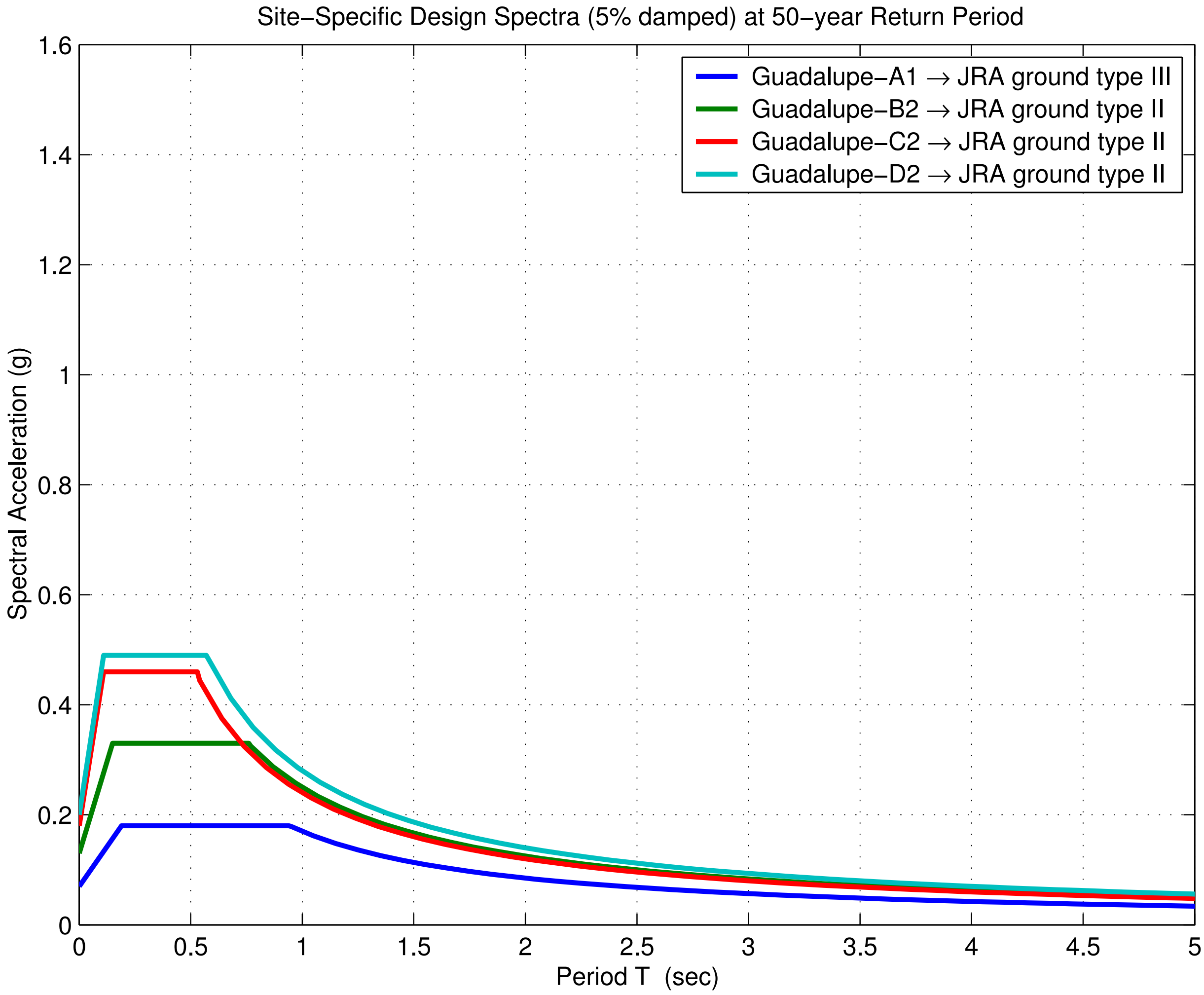


Figure 2B-78 Site-specific design spectra at 50-year return period for Guadalupe Bridge site

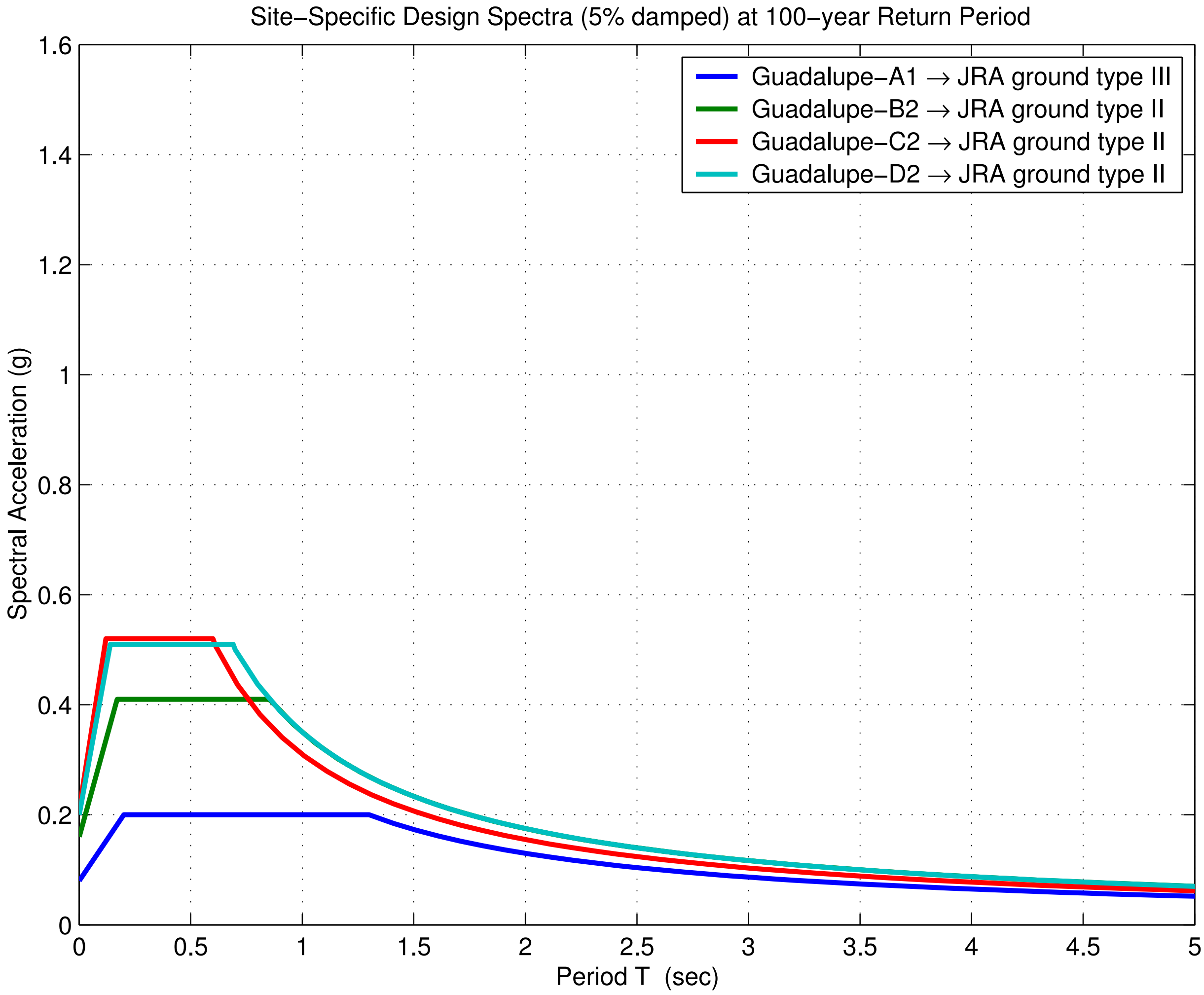


Figure 2B-79 Site-specific design spectra at 100-year return period for Guadalupe Bridge site

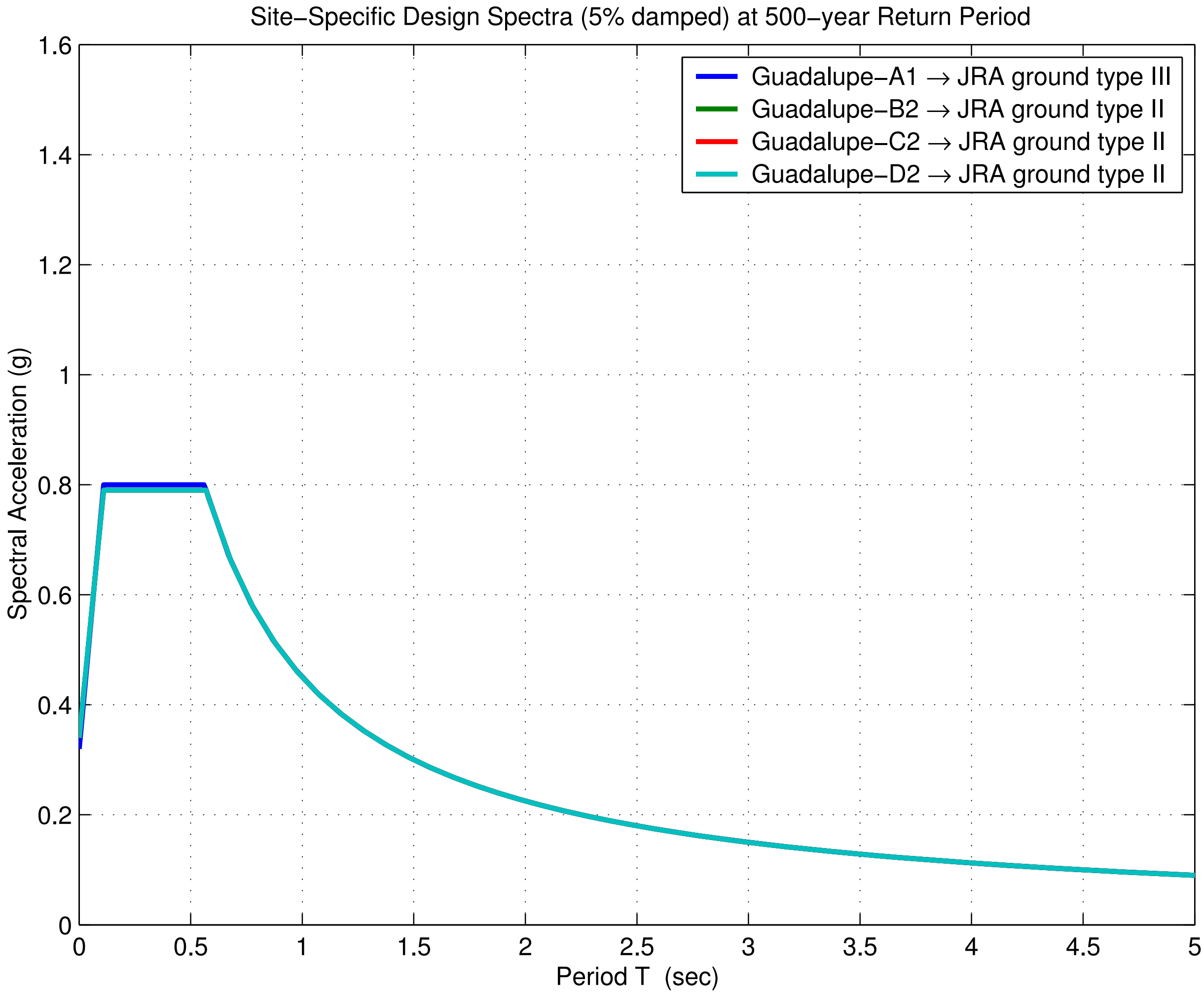


Figure 2B-80 Site-specific design spectra at 500-year return period for Guadalupe Bridge site

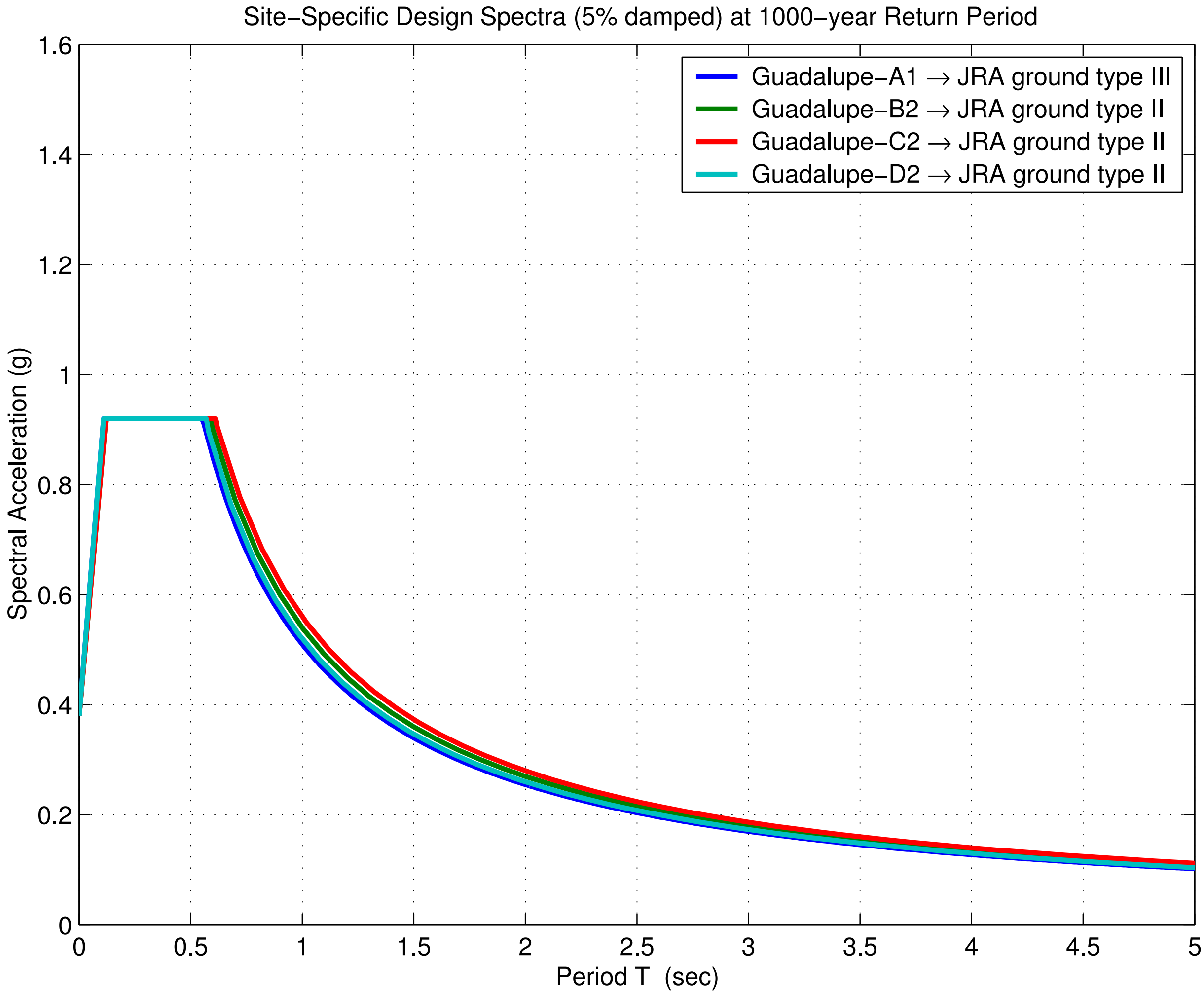
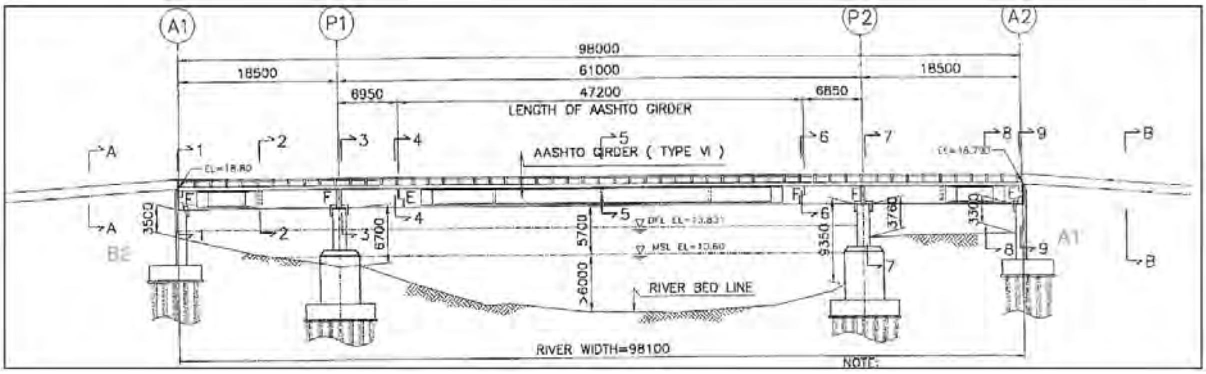


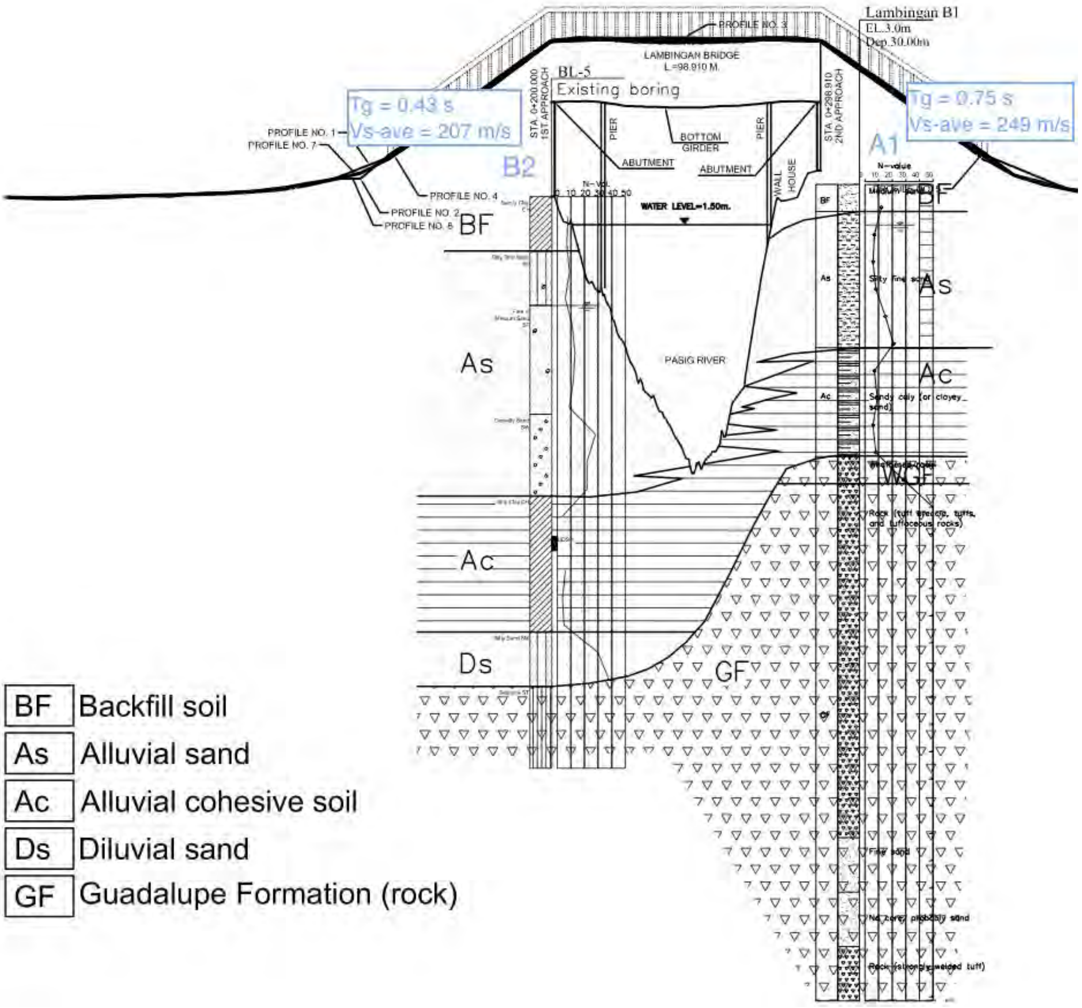
Figure 2B-81 Site-specific design spectra at 1000-year return period for Guadalupe Bridge site

(a) site plan



(b) bridge profile

Figure 2B-82 Lambingan Bridge site location and data



(c) soil profile

Figure 2B-83 Lambingan Bridge site location and data

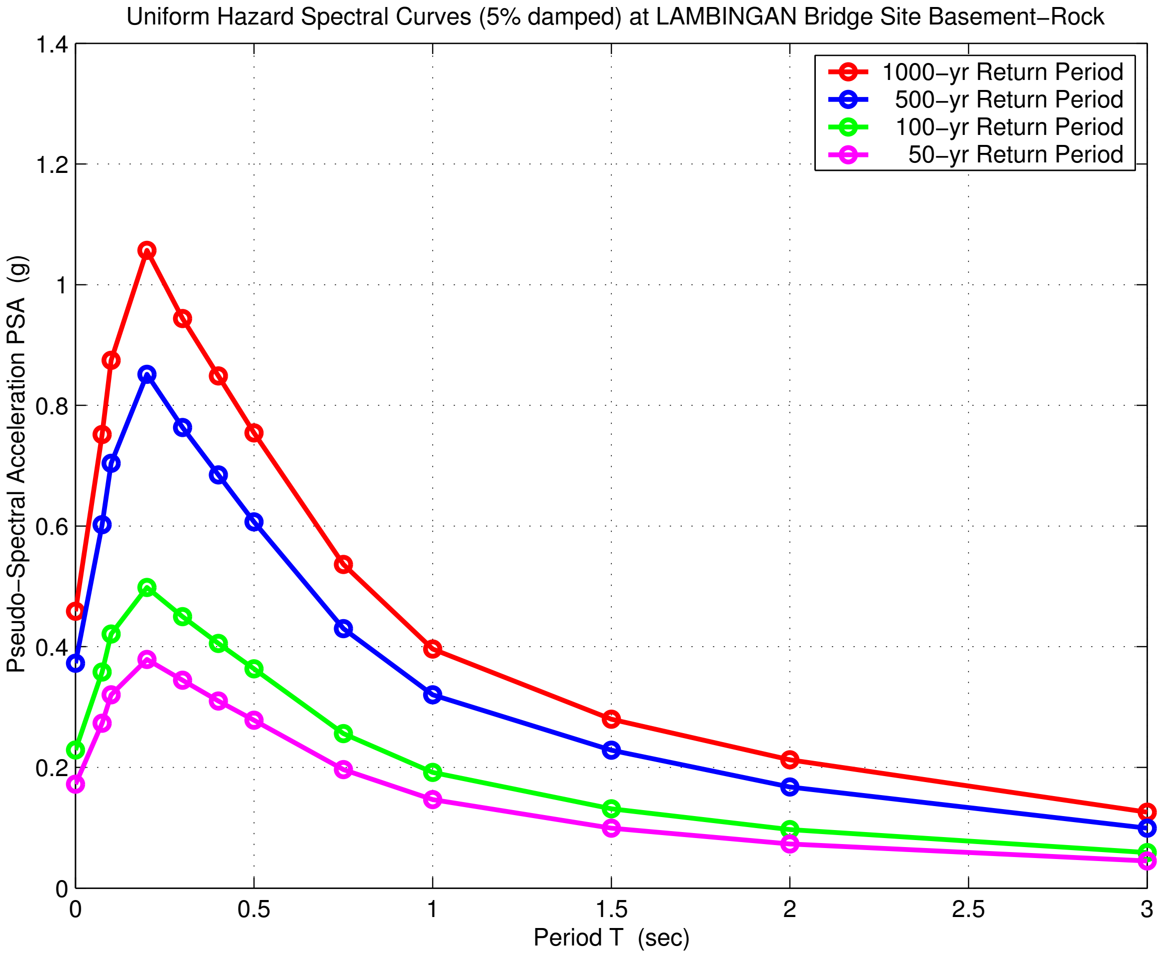


Figure 2B-84 Uniform hazard spectral curve for Lambingan Bridge site at basement-rock

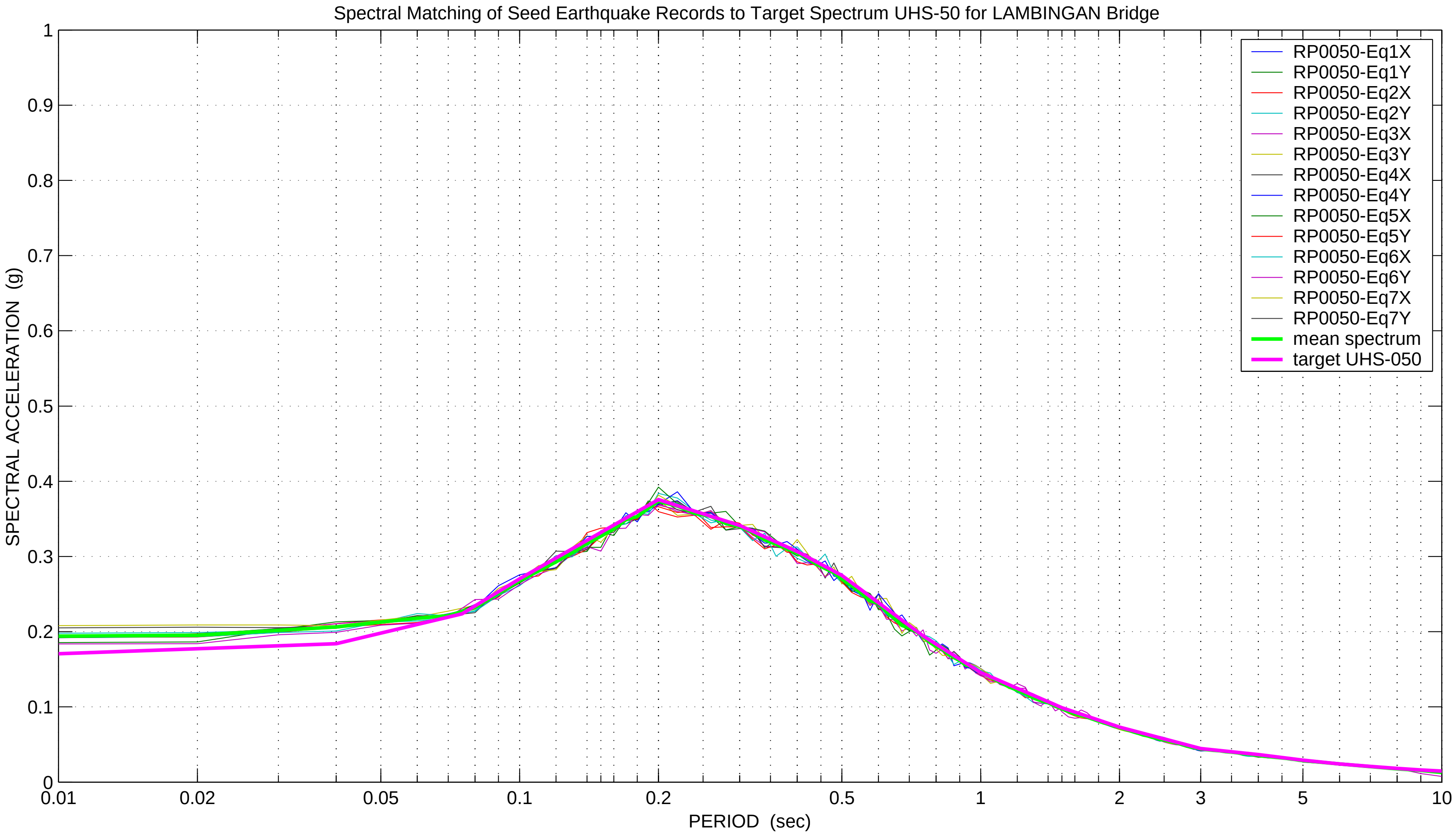


Figure 2B-85 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 50-year return period as compared to the target spectrum UHS-50 at Lambingan Bridge

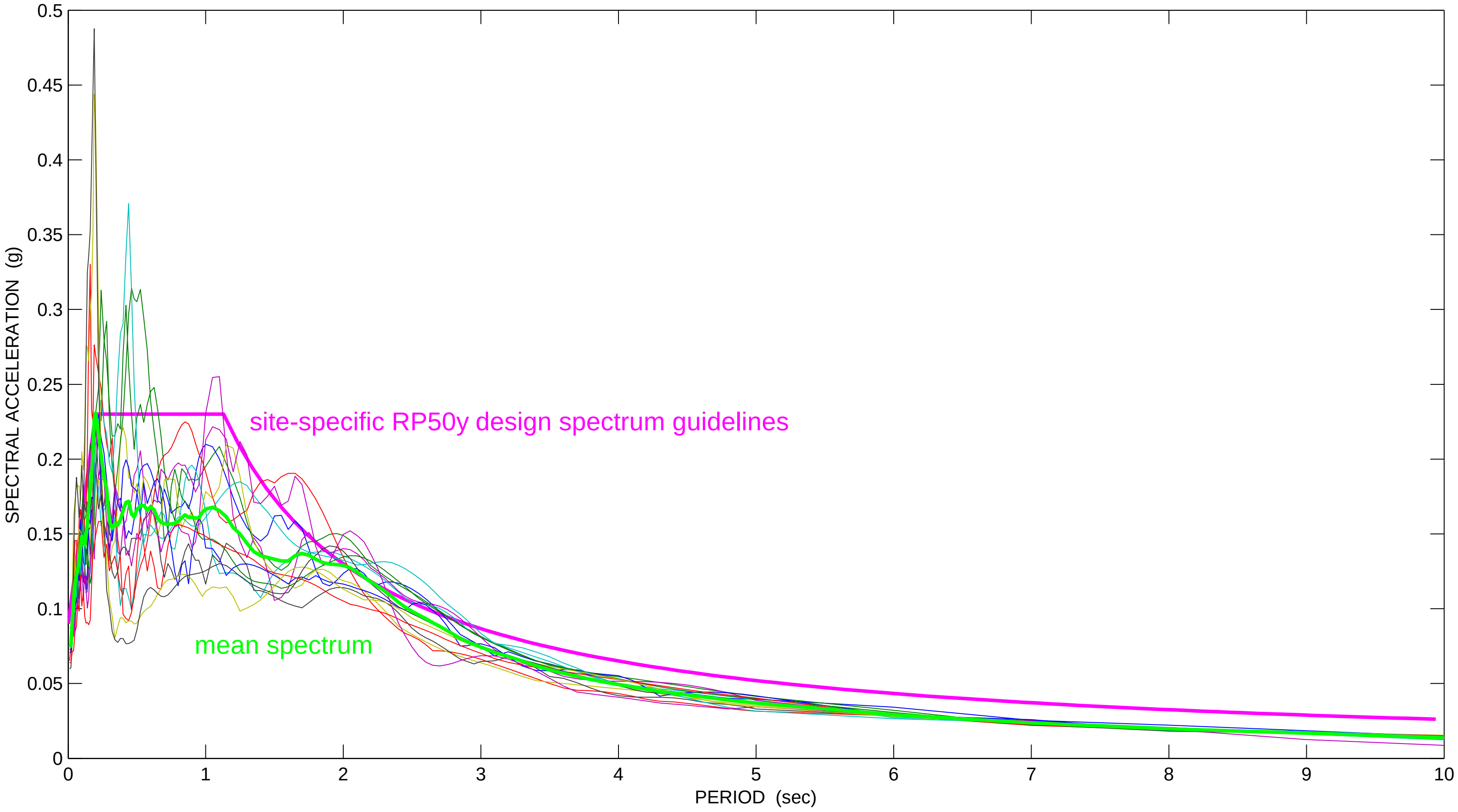


Figure 2B-86 Construction of site-specific design spectrum at 50-year return period for Lambingan A1 based on mean of 14 response spectra from nonlinear site response analysis

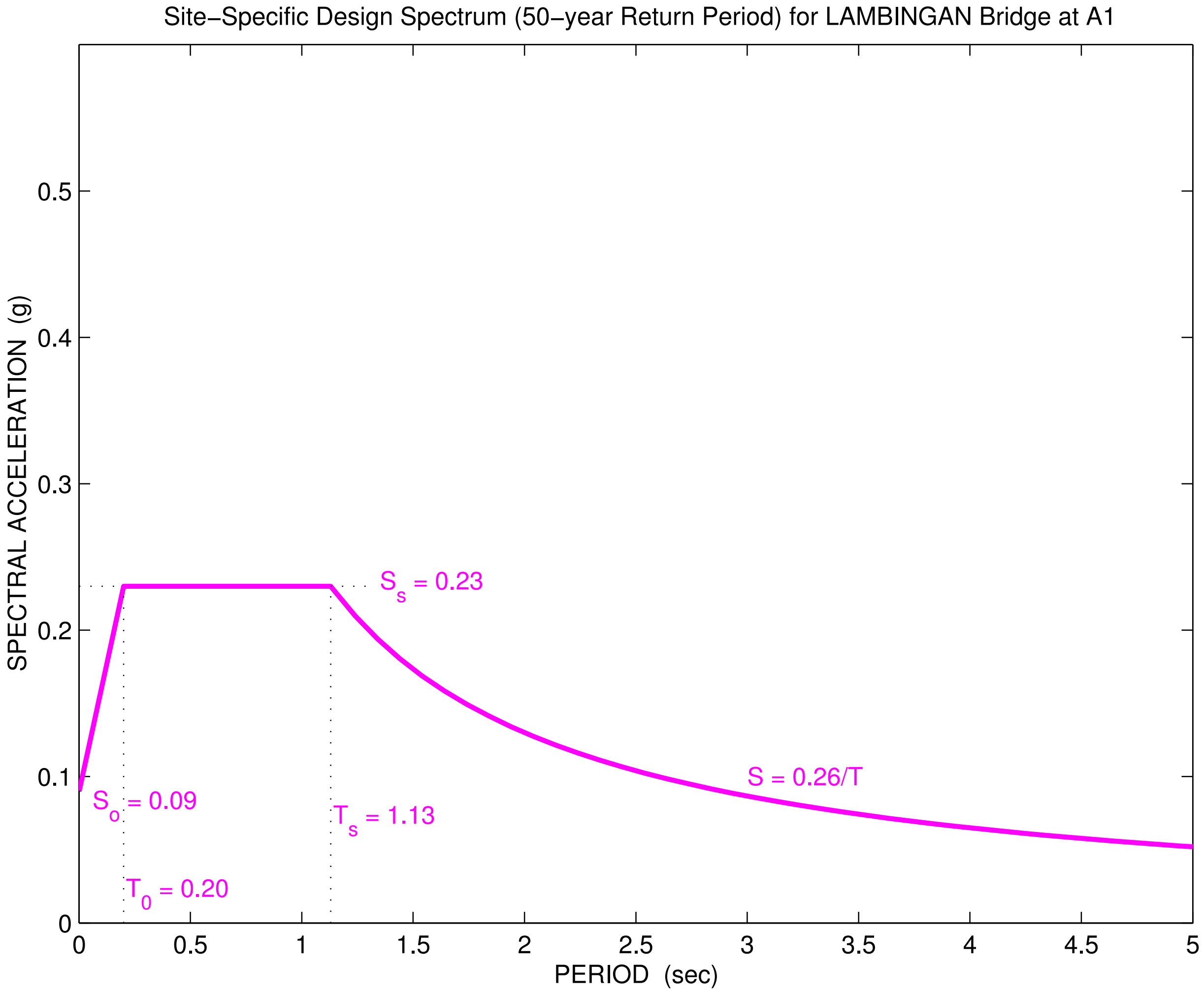
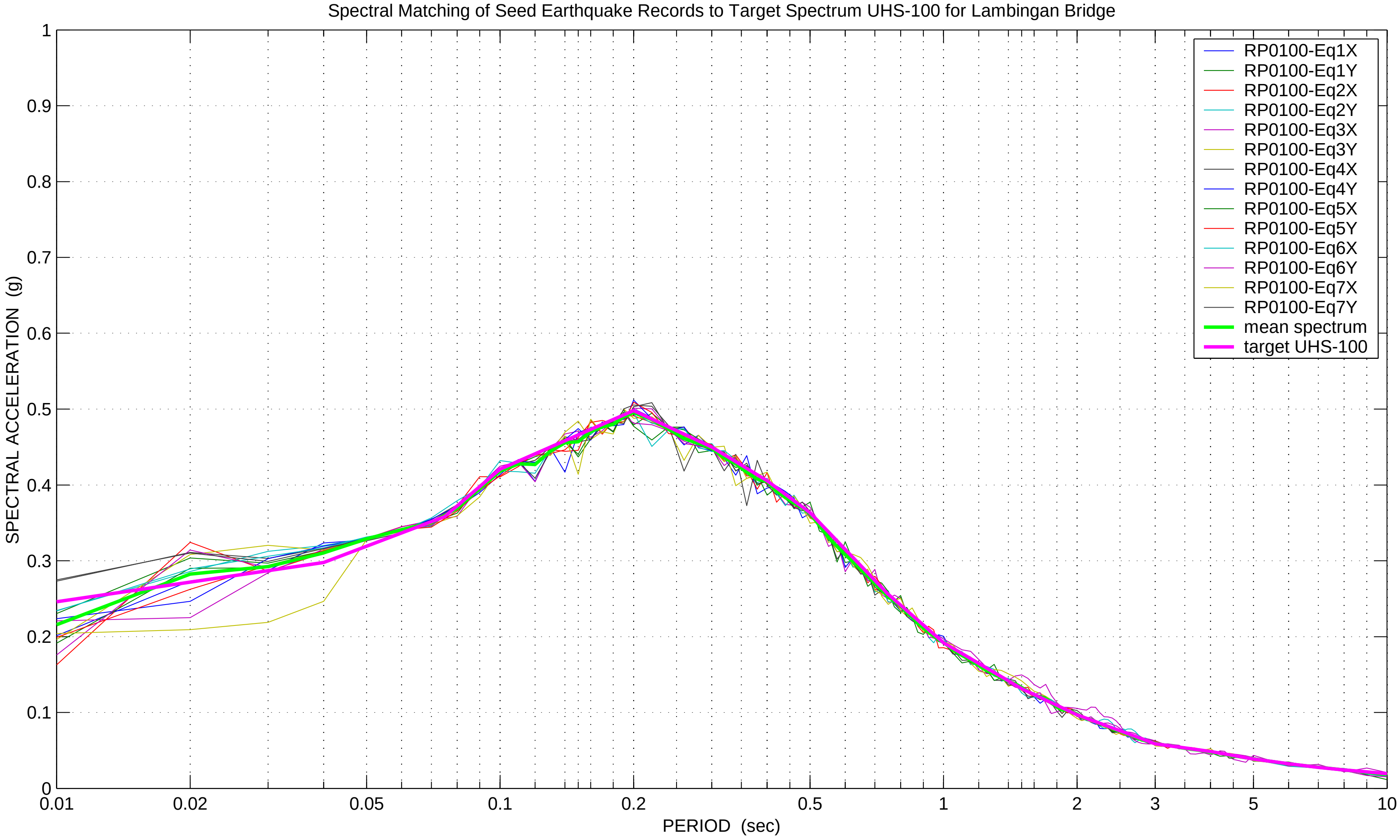


Figure 2B-87 Site-specific design spectrum at 50-year return period for Lambingan Bridge at A1 site



Spectral Matching of Seed Earthquake Records to Target Spectrum UHS-100 for Lambingan Bridge

Figure 2B-88 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 100-year return period as compared to the target spectrum UHS-100 at Lambingan Bridge

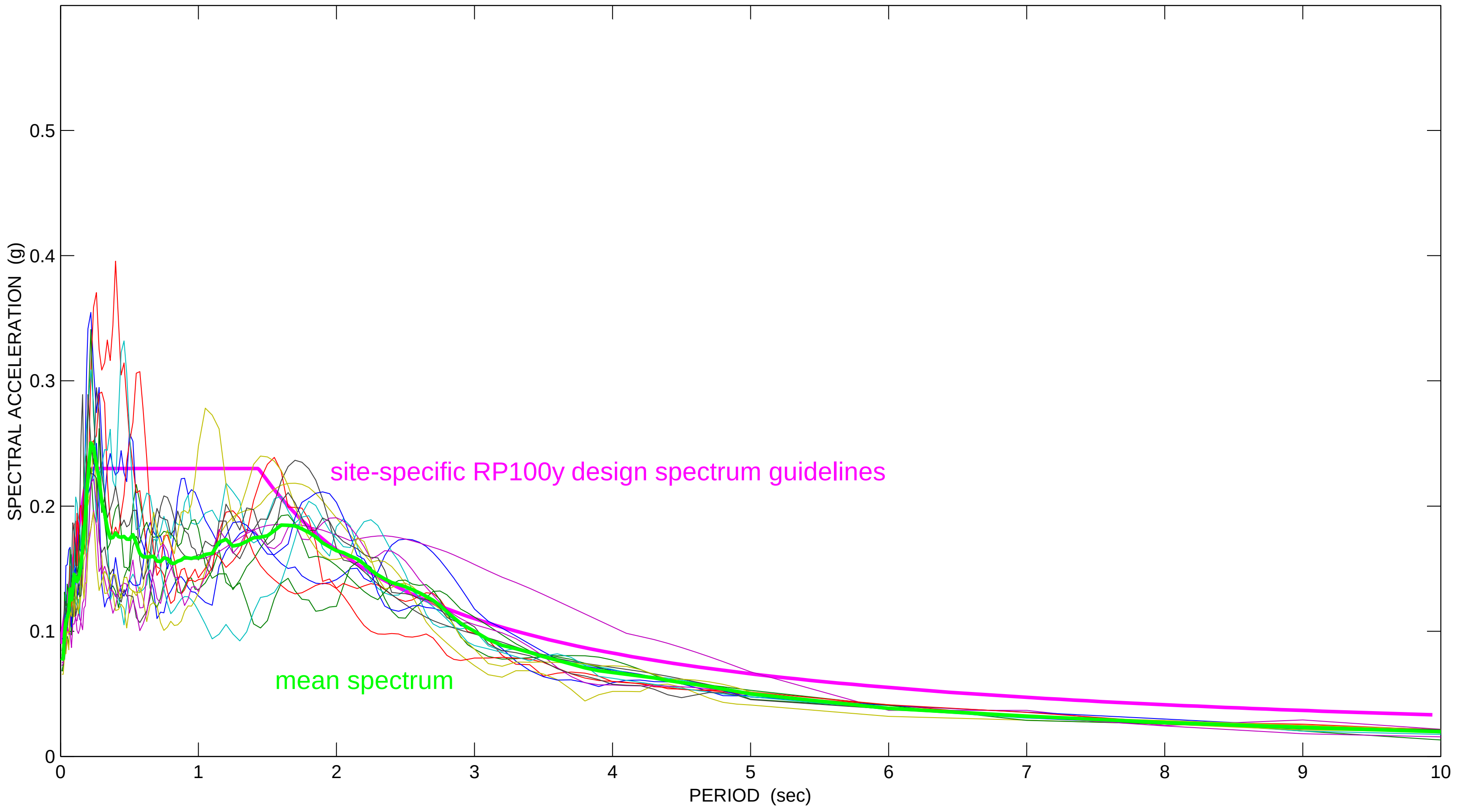


Figure 2B-89 Construction of site-specific design spectrum at 100-year return period for Lambingan A1 based on mean of 14 response spectra from nonlinear site response analysis

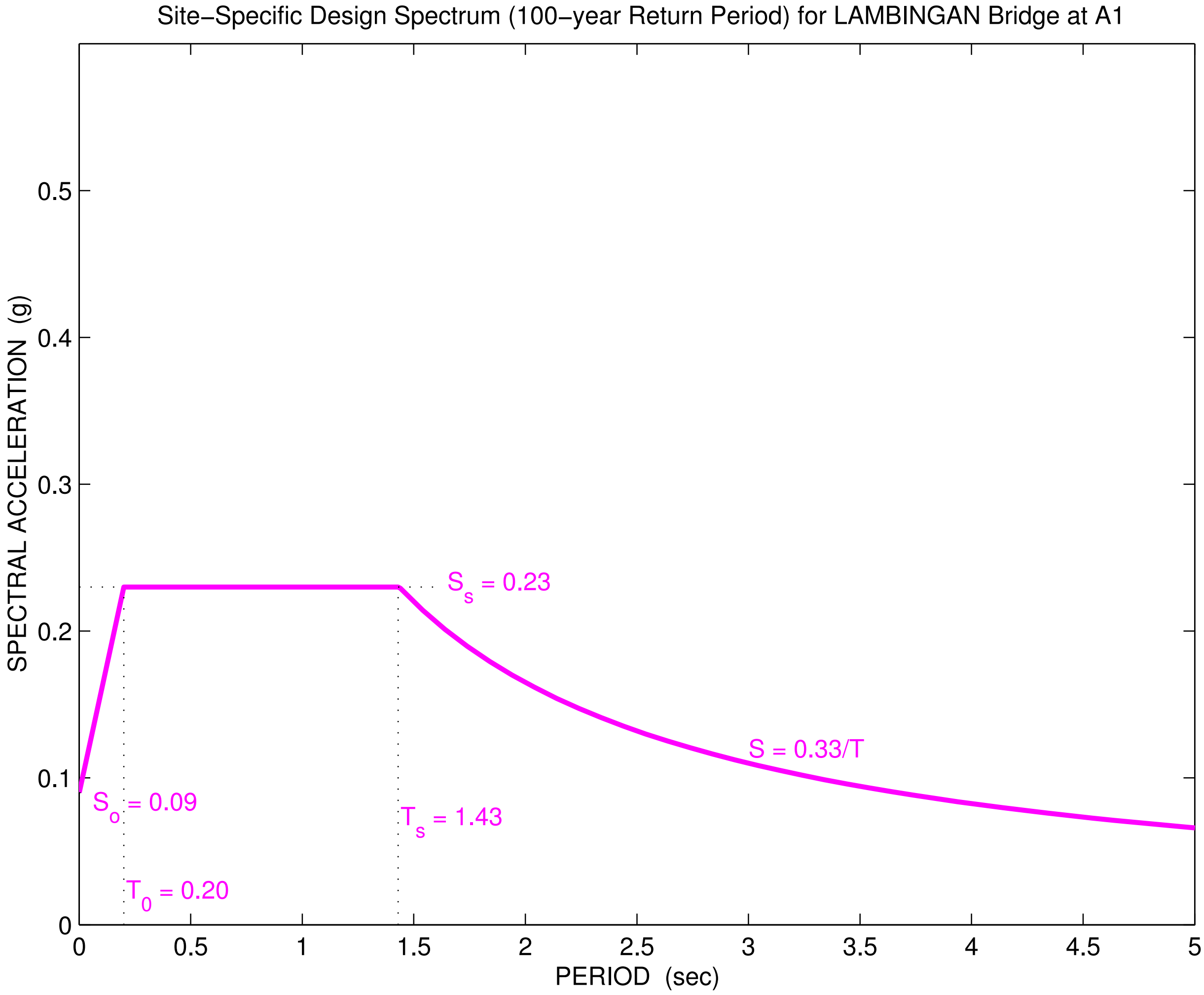


Figure 2B-90 Site-specific design spectrum at 100-year return period for Lambingan Bridge at A1 site

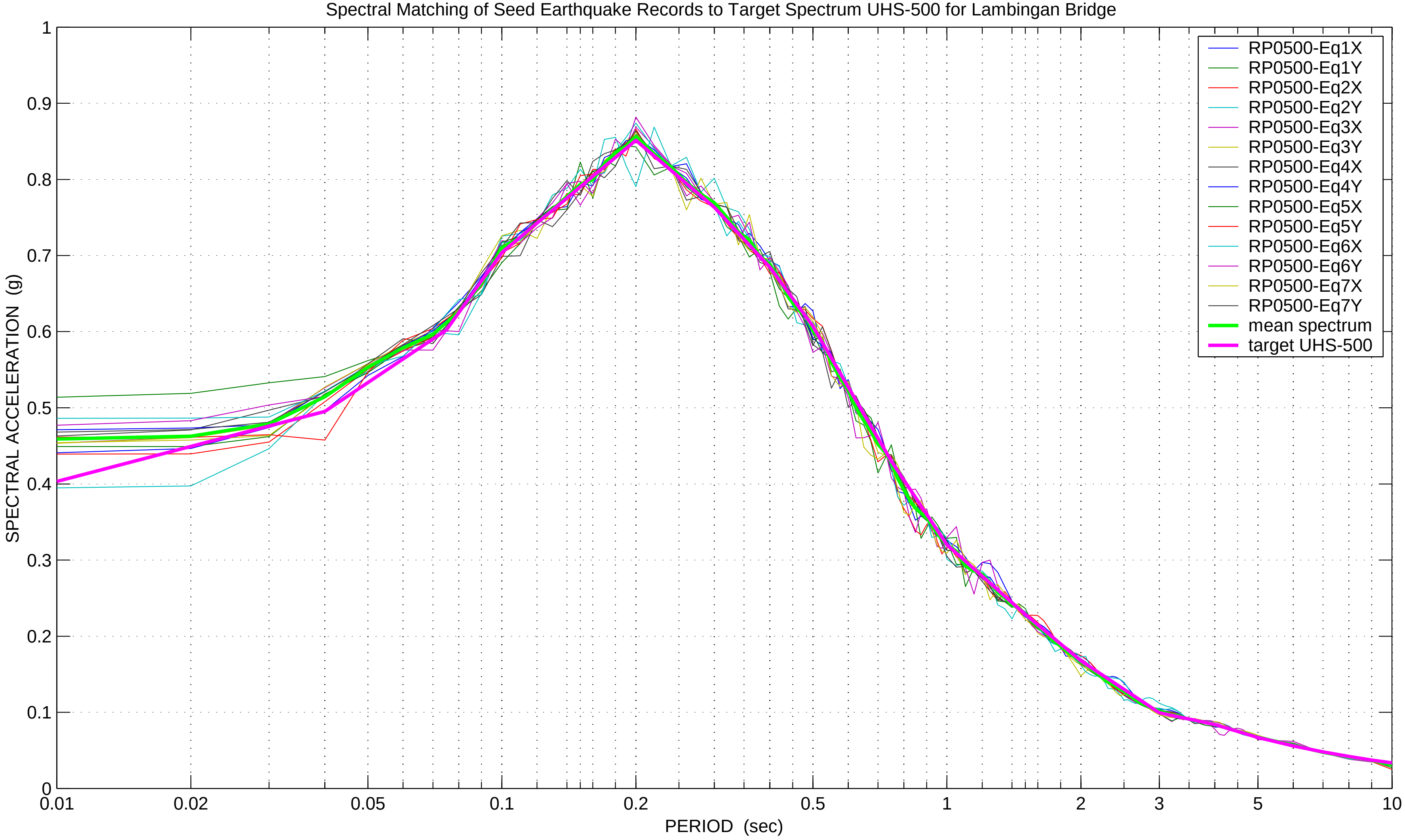


Figure 2B-91 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 500-year return period as compared to the target spectrum UHS-500 at Lambingan Bridge

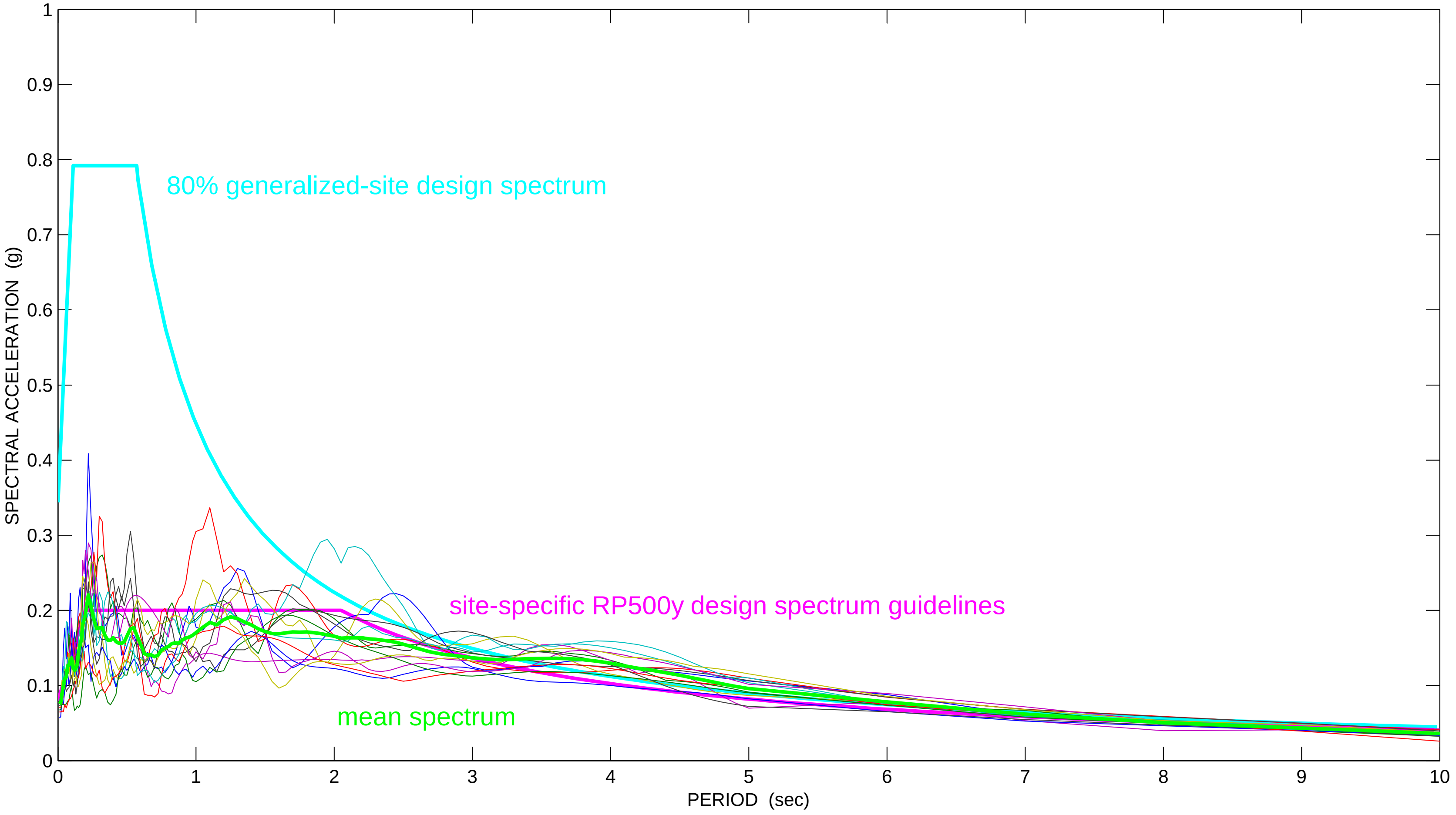


Figure 2B-92 Construction of site-specific design spectrum at 500-year return period for Lambingan A1 based on mean of 14 response spectra from nonlinear site response analysis

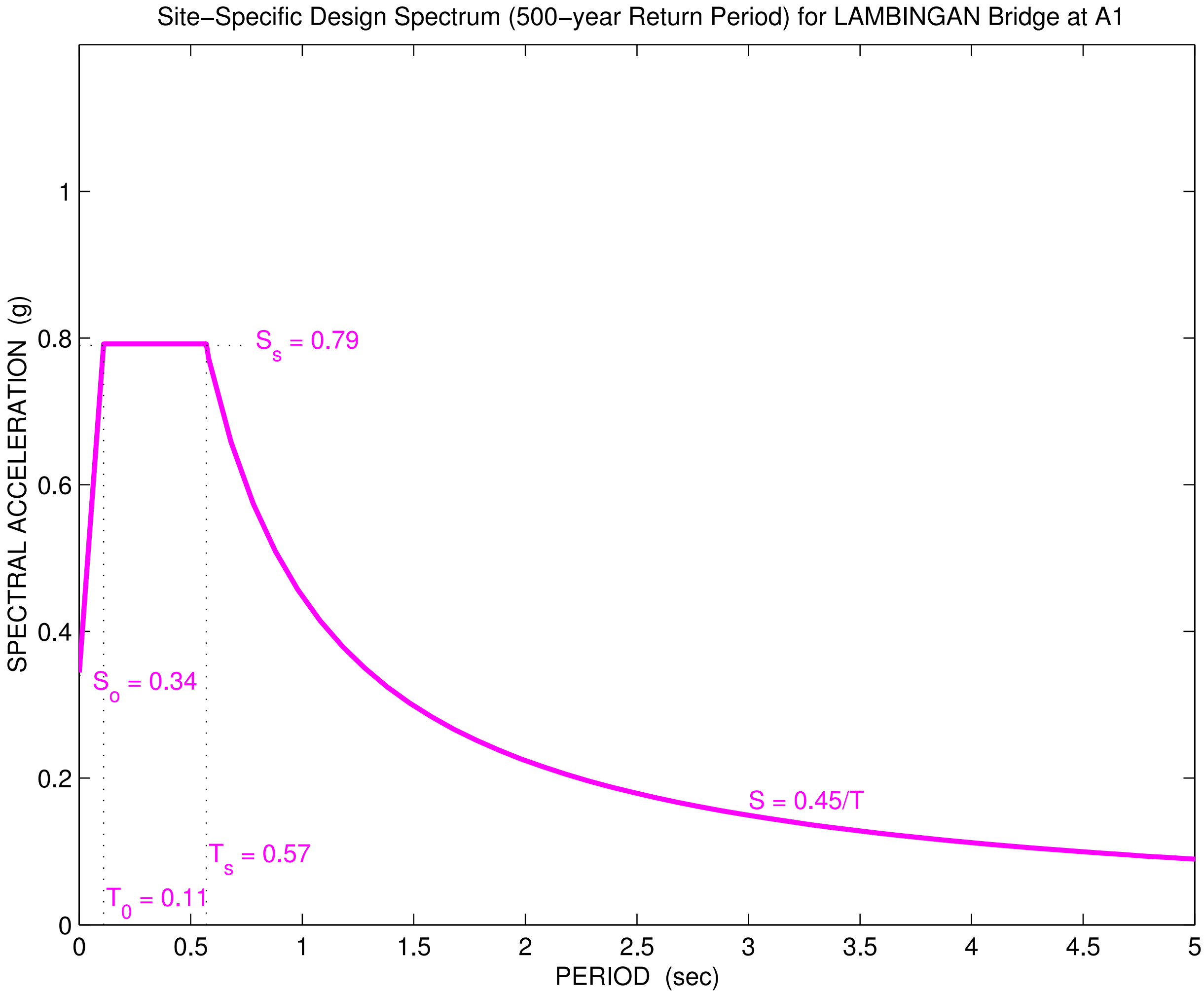


Figure 2B-93 Site-specific design spectrum at 500-year return period for Lambingan Bridge at A1 site

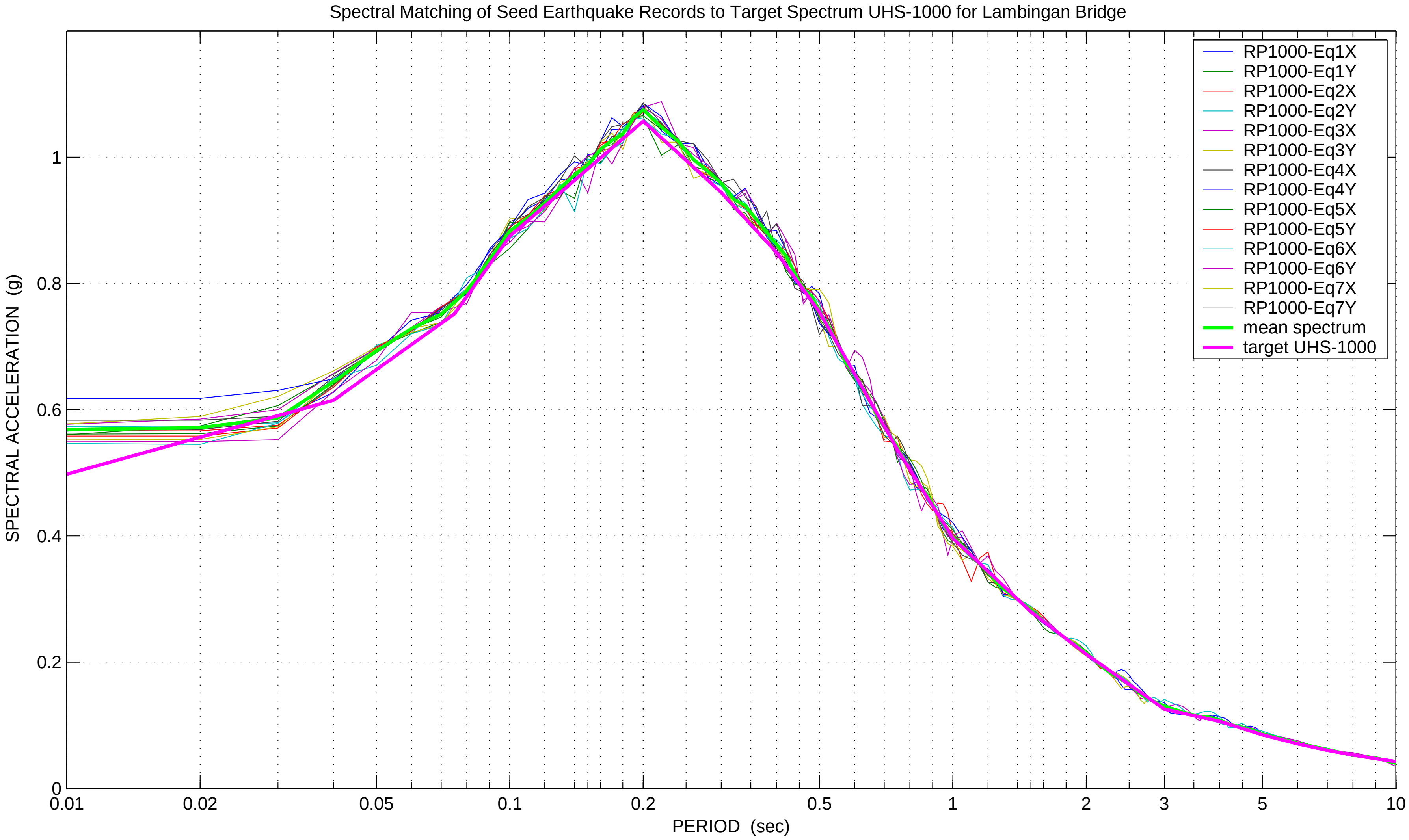


Figure 2B-94 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 1000-year return period as compared to the target spectrum UHS-1000 at Lambingan Bridge

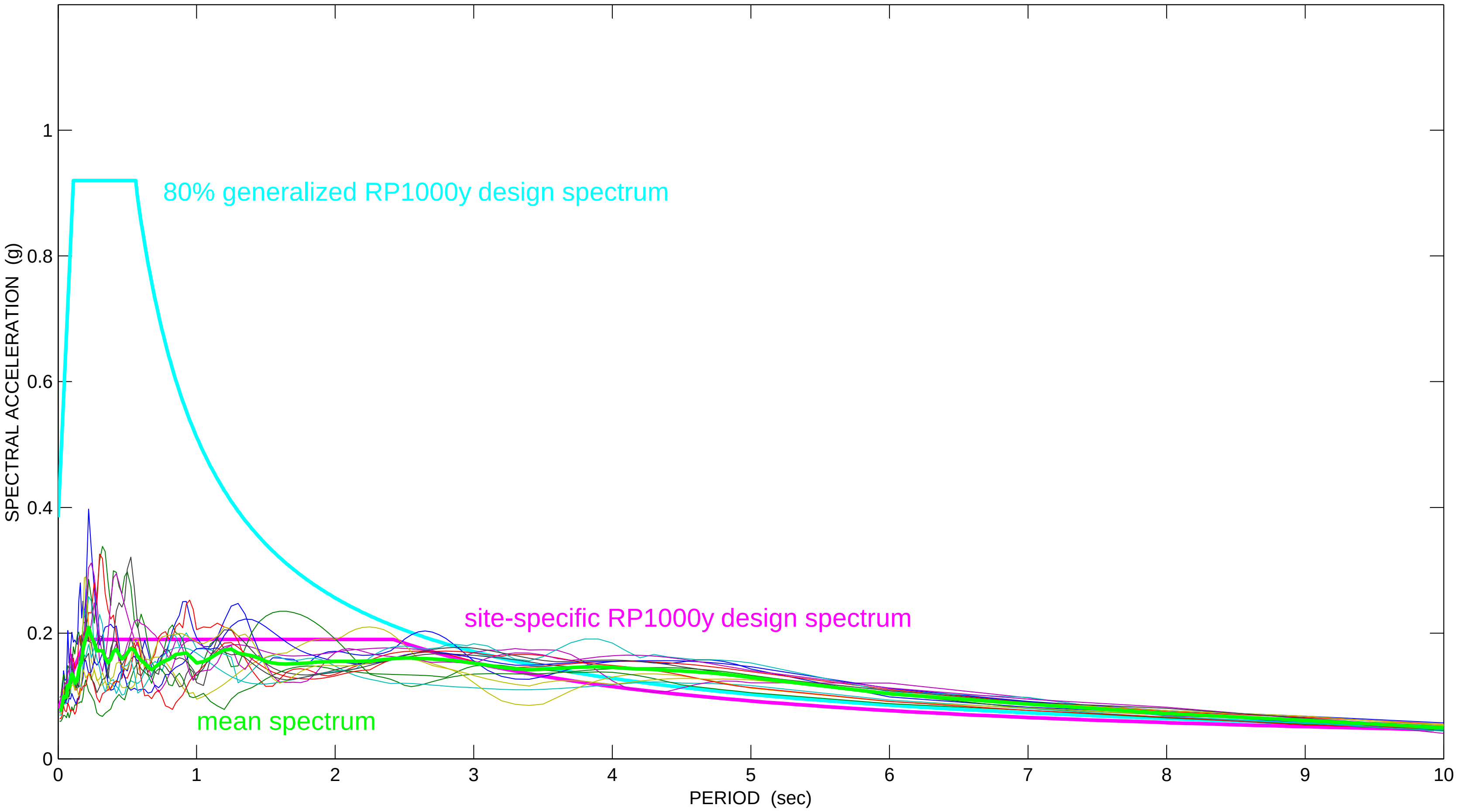


Figure 2B-95 Construction of site-specific design spectrum at 1000-year return period for Lambingan A1 based on mean of 14 response spectra from nonlinear site response analysis

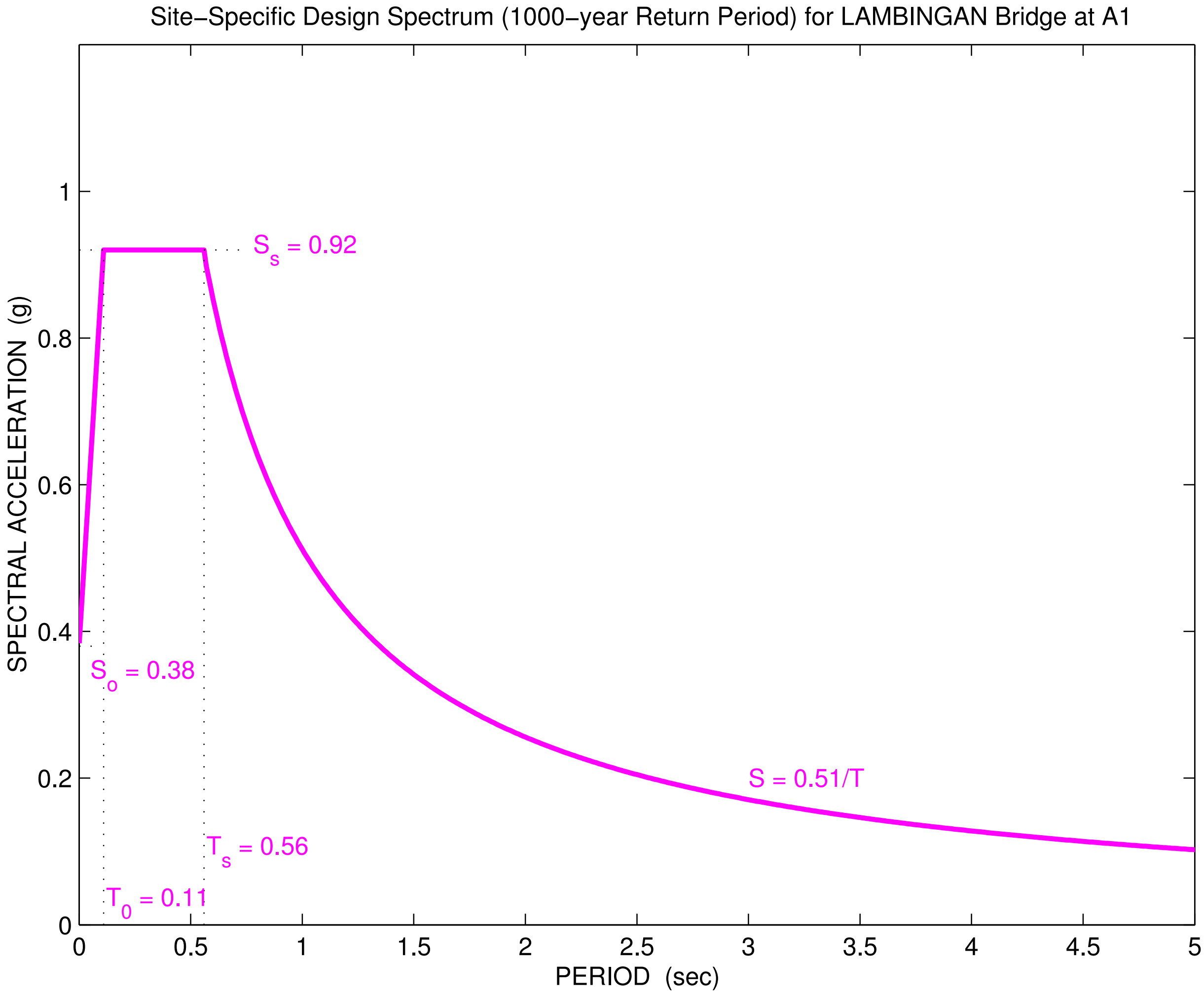


Figure 2B-96 Site-specific design spectrum at 1000-year return period for Lambingan Bridge at A1 site

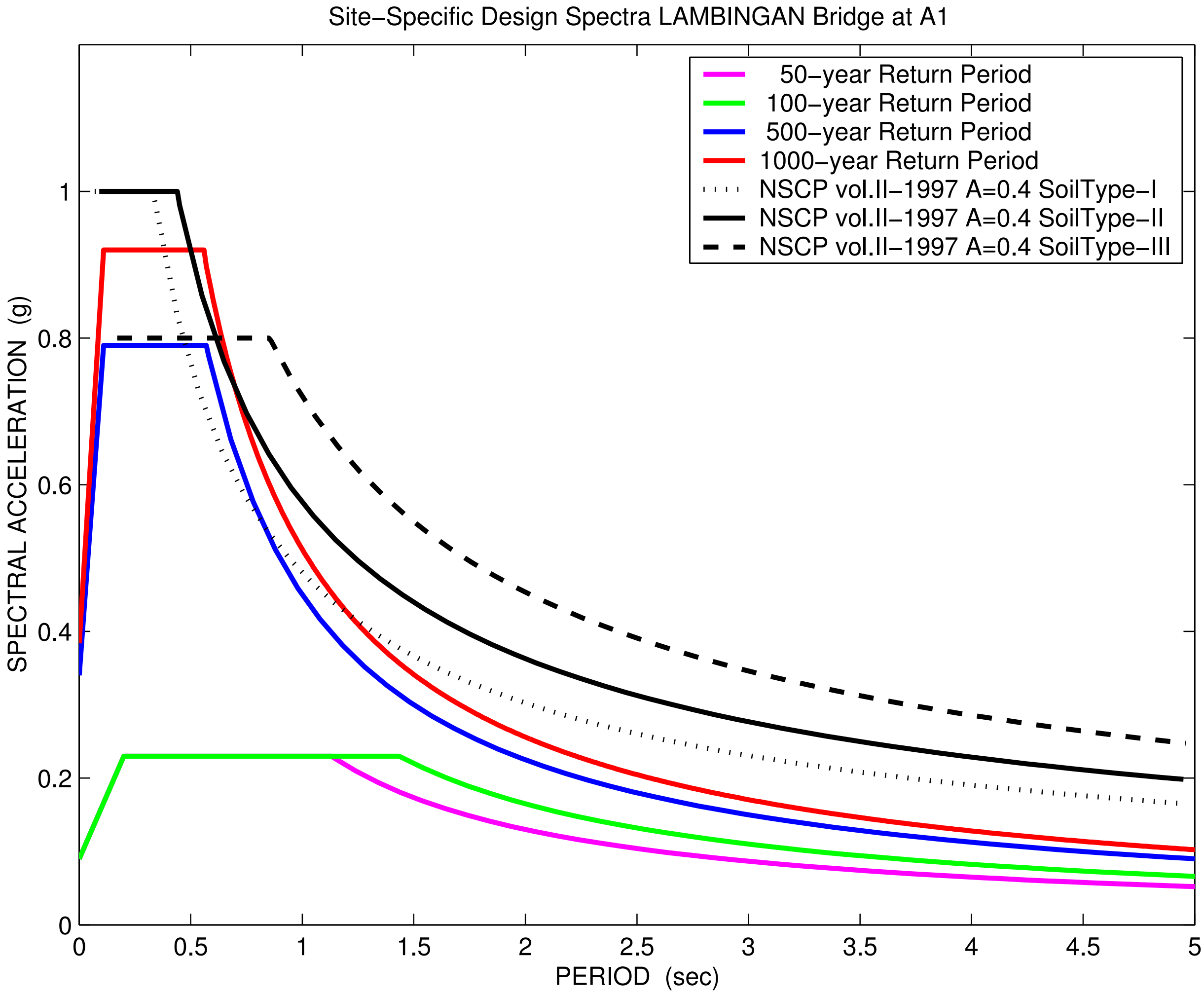


Figure 2B-97 Site-specific design spectra at 50-, 100-, 500-, 1000-year return periods for Lambingan Bridge A1 site vs. NSCP-Bridges-1997

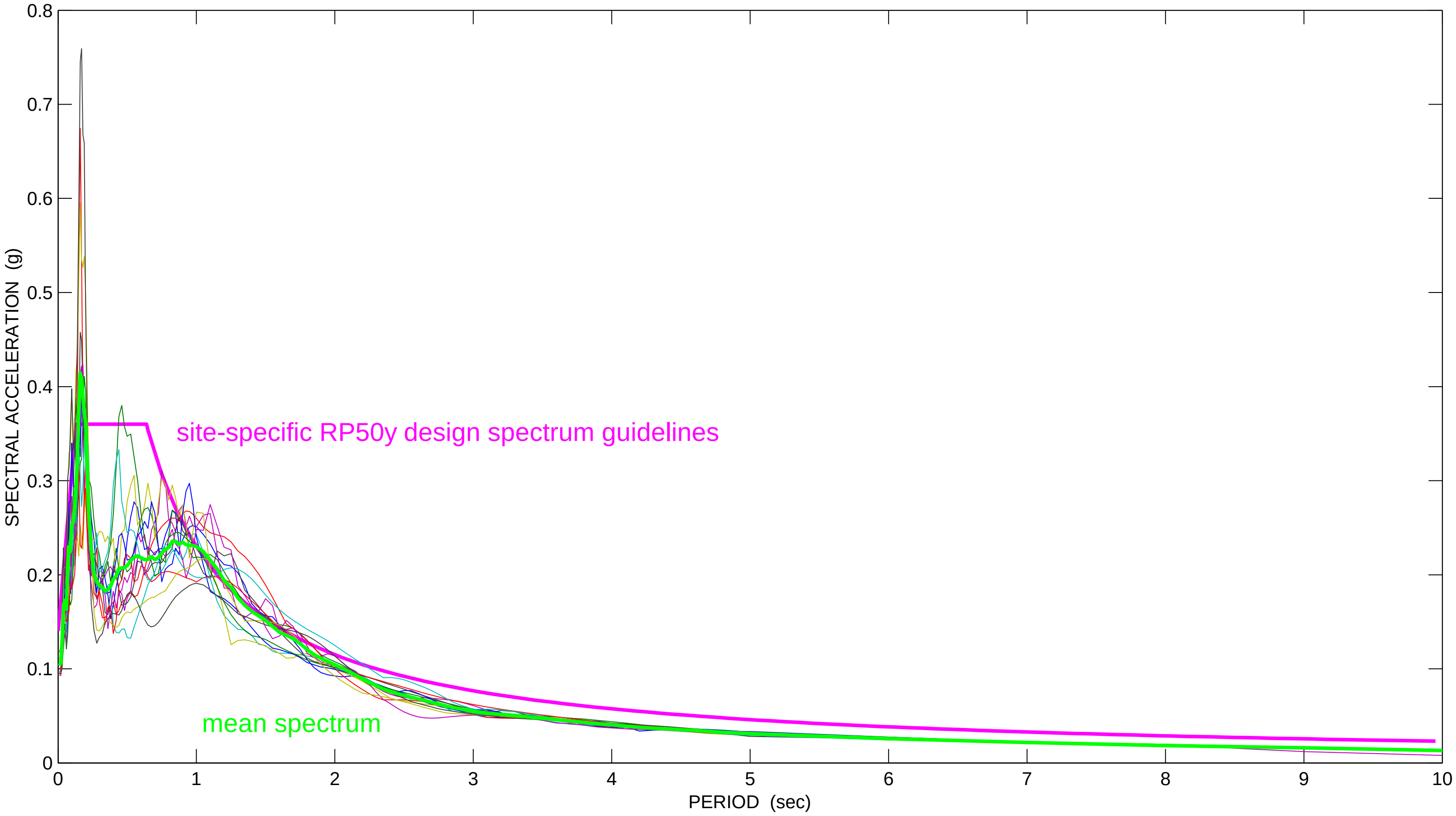


Figure 2B-98 Construction of site-specific design spectrum at 50-year return period for Lambingan B2 based on mean of 14 response spectra from nonlinear site response analysis

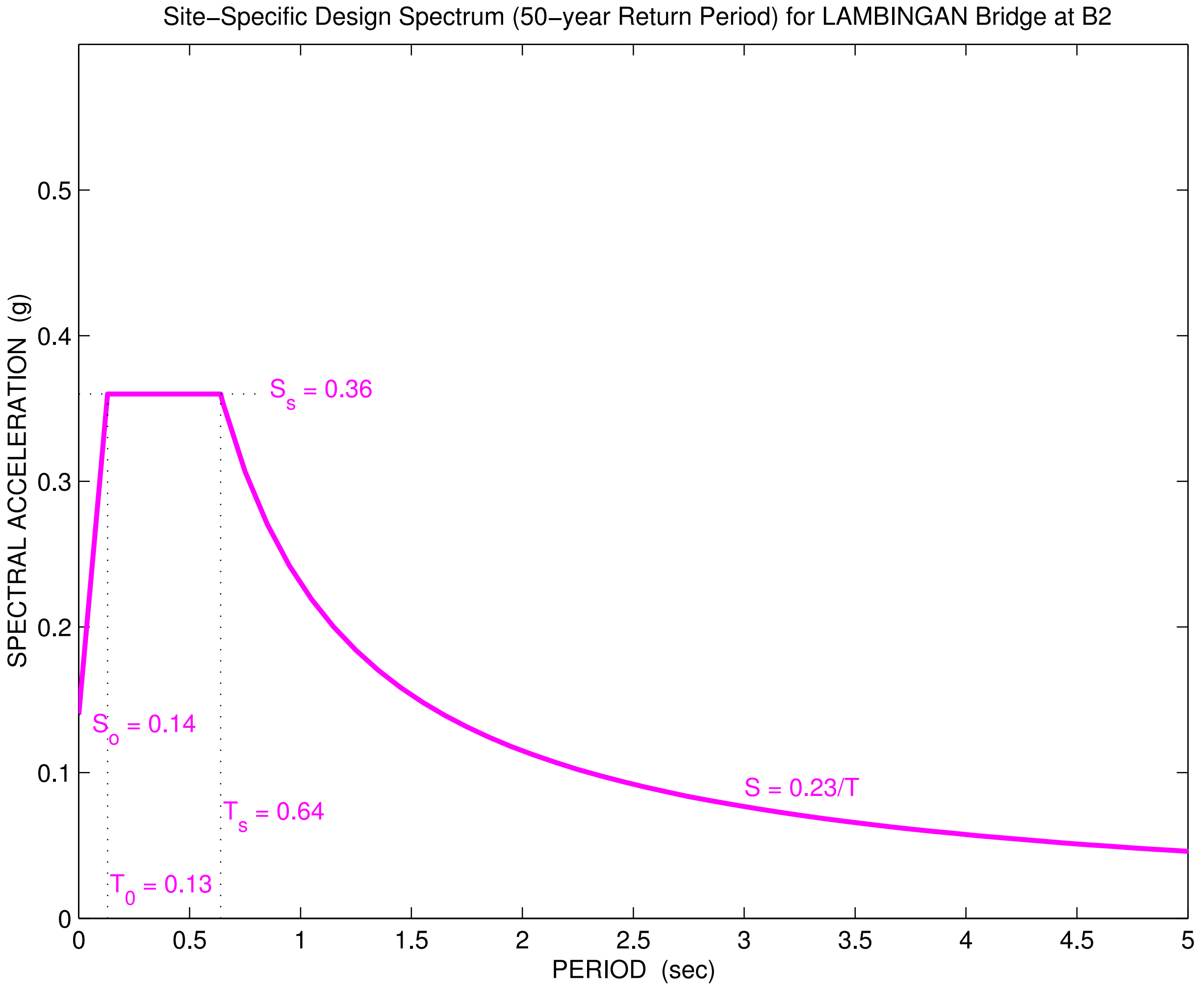


Figure 2B-99 Site-specific design spectrum at 50-year return period for Lambingan Bridge at B2 site

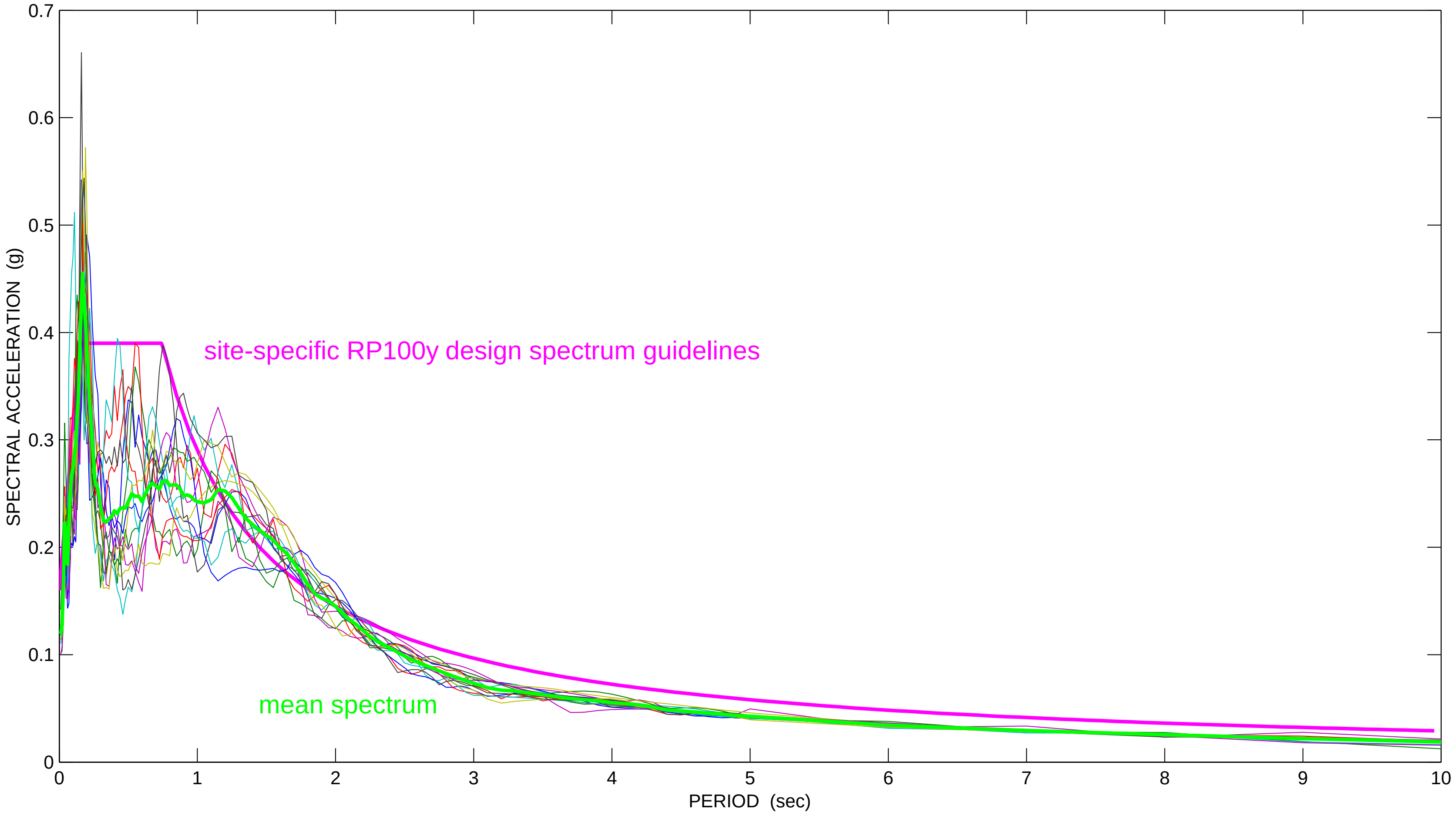


Figure 2B-100 Construction of site-specific design spectrum at 100-year return period for Lambingan B2 based on mean of 14 response spectra from nonlinear site response analysis

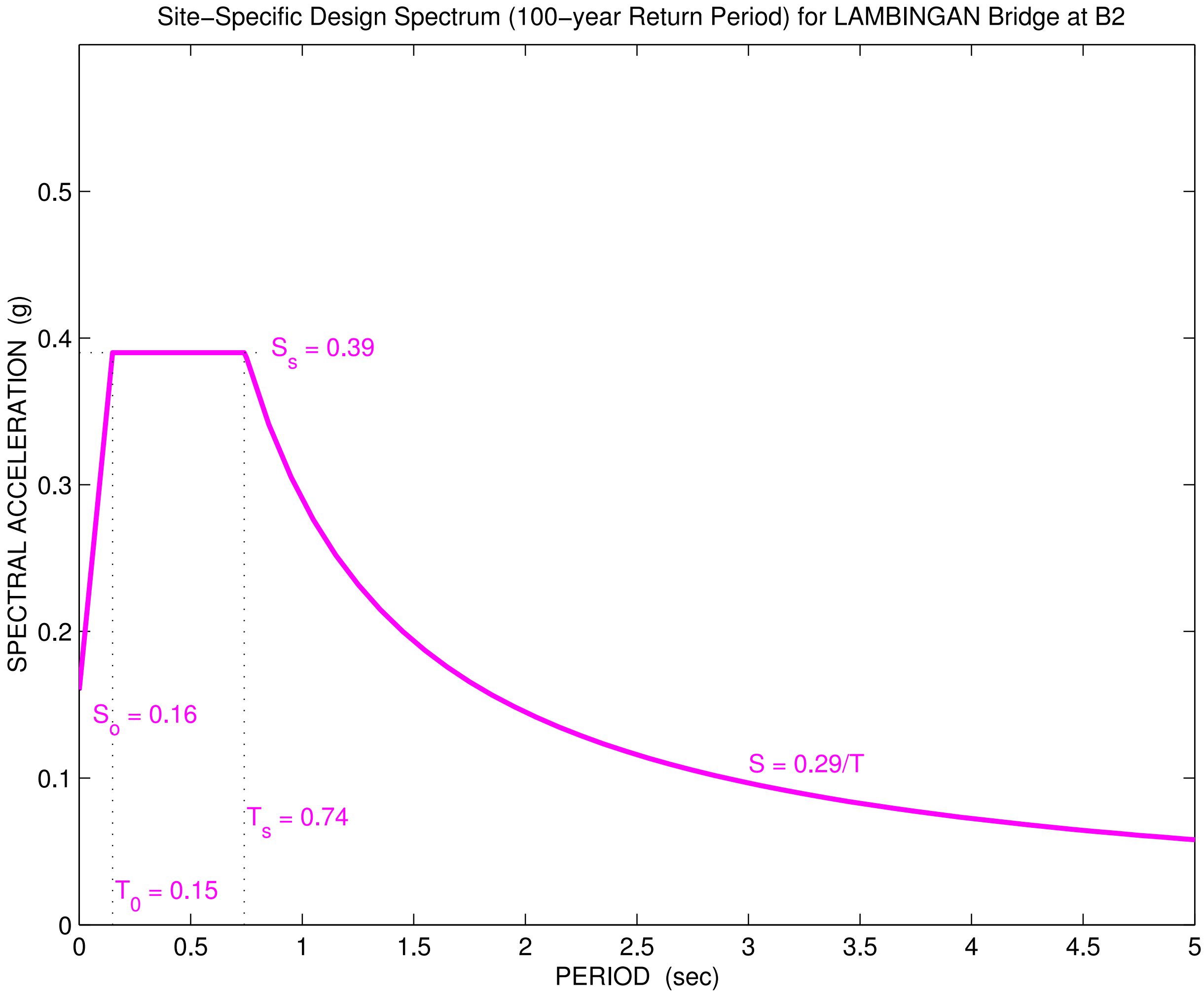


Figure 2B-101 Site-specific design spectrum at 100-year return period for Lambingan Bridge at B2 site

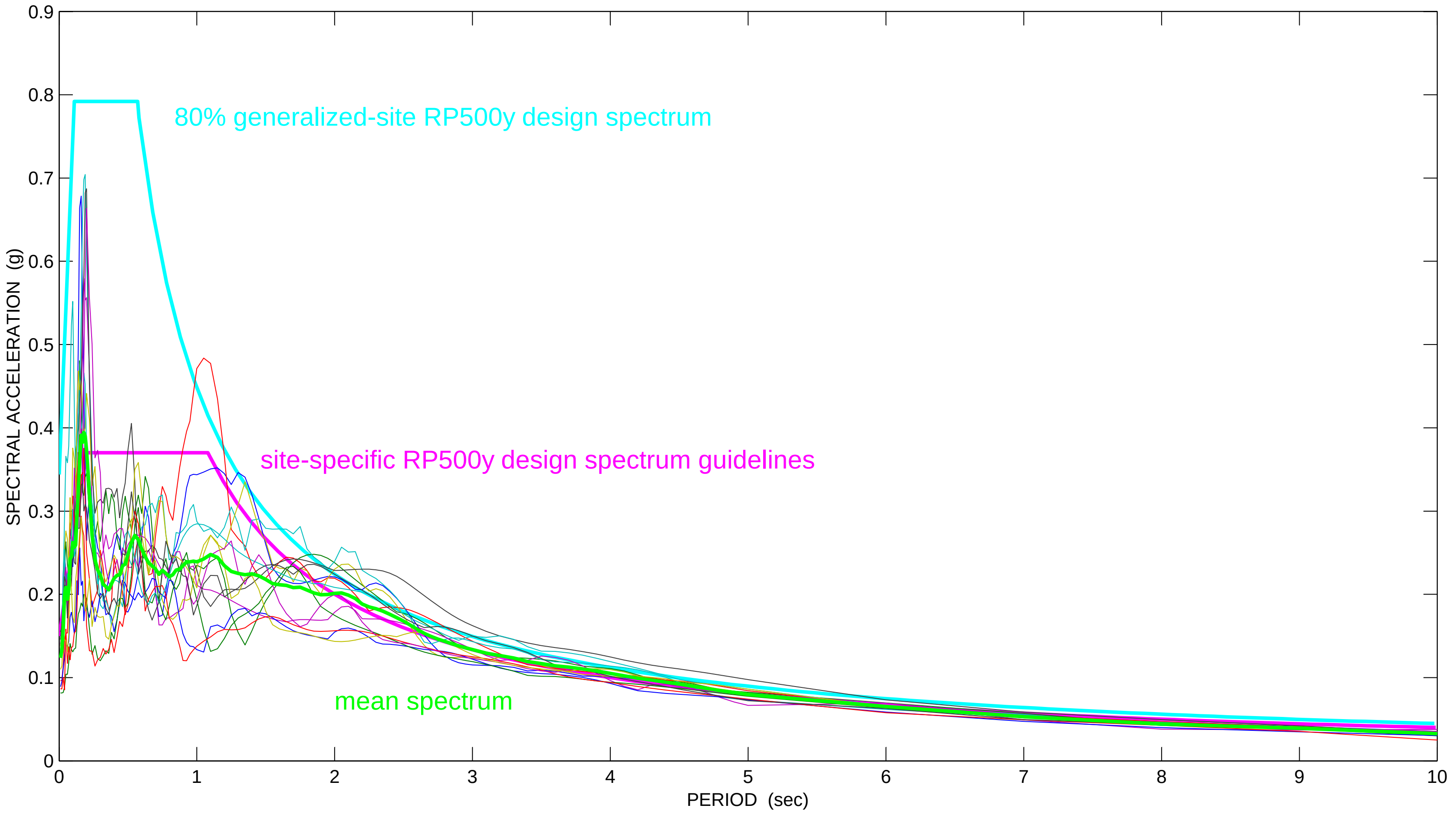


Figure 2B-102 Construction of site-specific design spectrum at 500-year return period for Lambingan B2 based on mean of 14 response spectra from nonlinear site response analysis

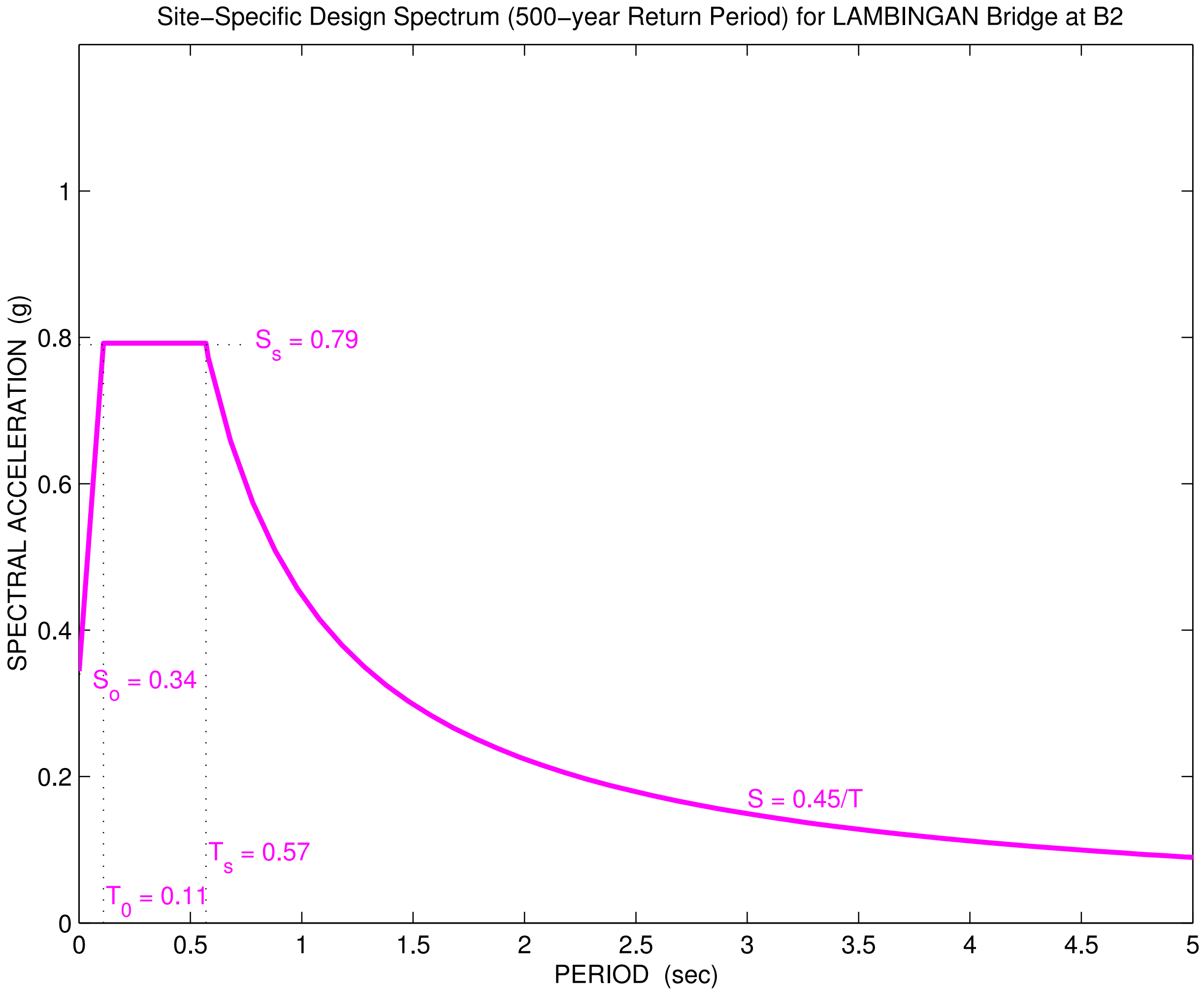


Figure 2B-103 Site-specific design spectrum at 500-year return period for Lambingan Bridge at B2 site

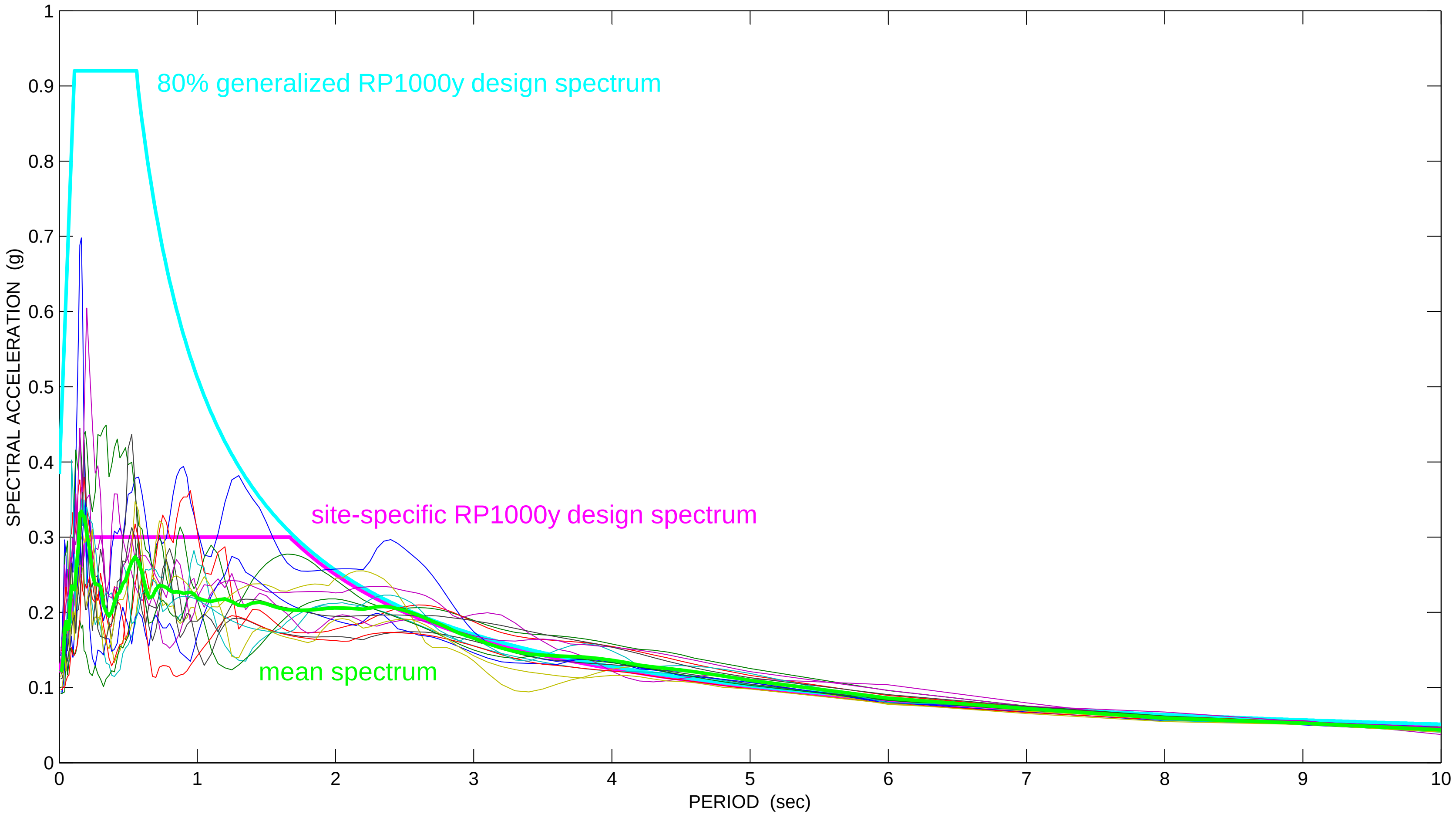


Figure 2B-104 Construction of site-specific design spectrum at 1000-year return period for Lambingan B2 based on mean of 14 response spectra from nonlinear site response analysis

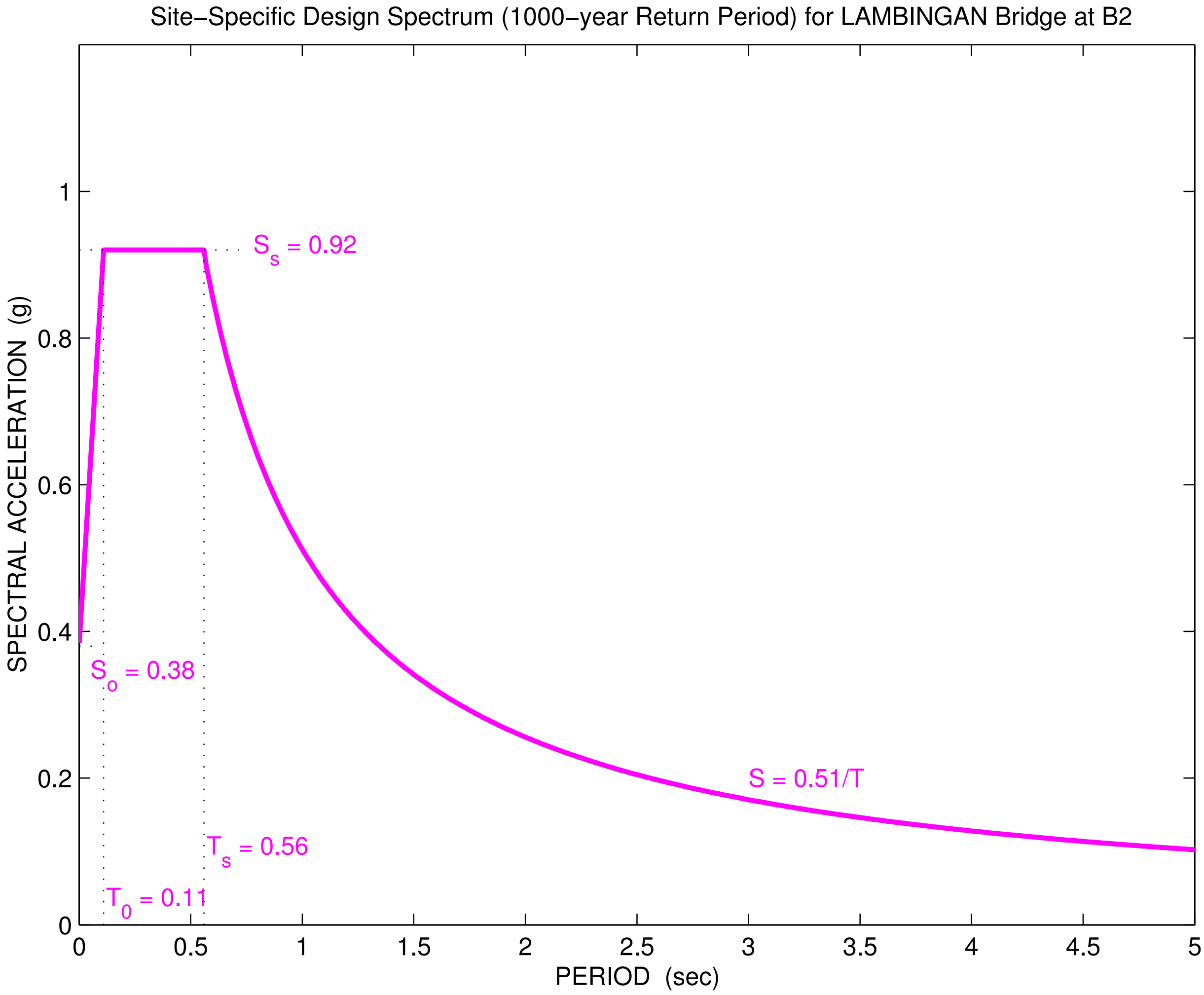


Figure 2B-105 Site-specific design spectrum at 1000-year return period for Lambingan Bridge at B2 site

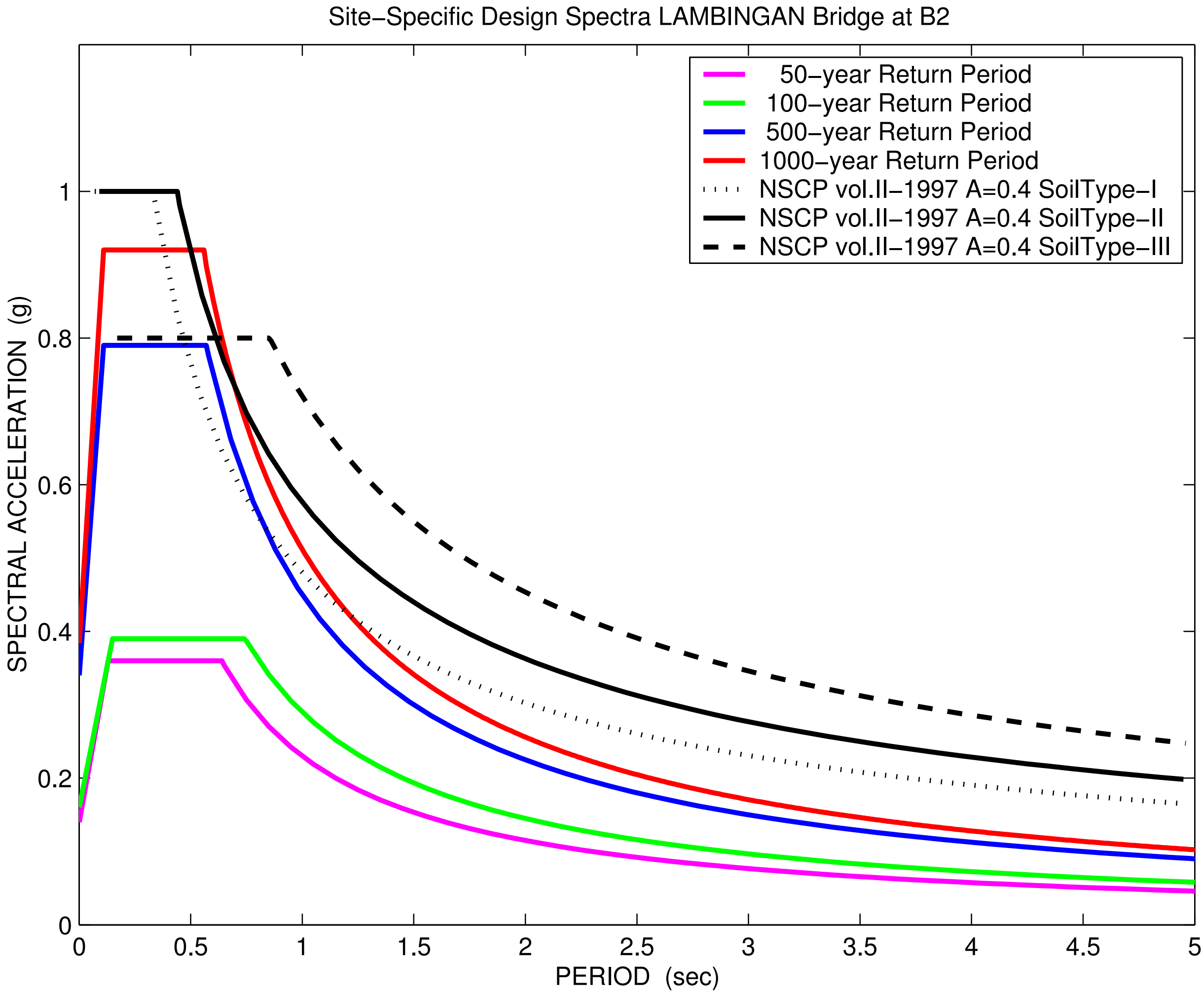
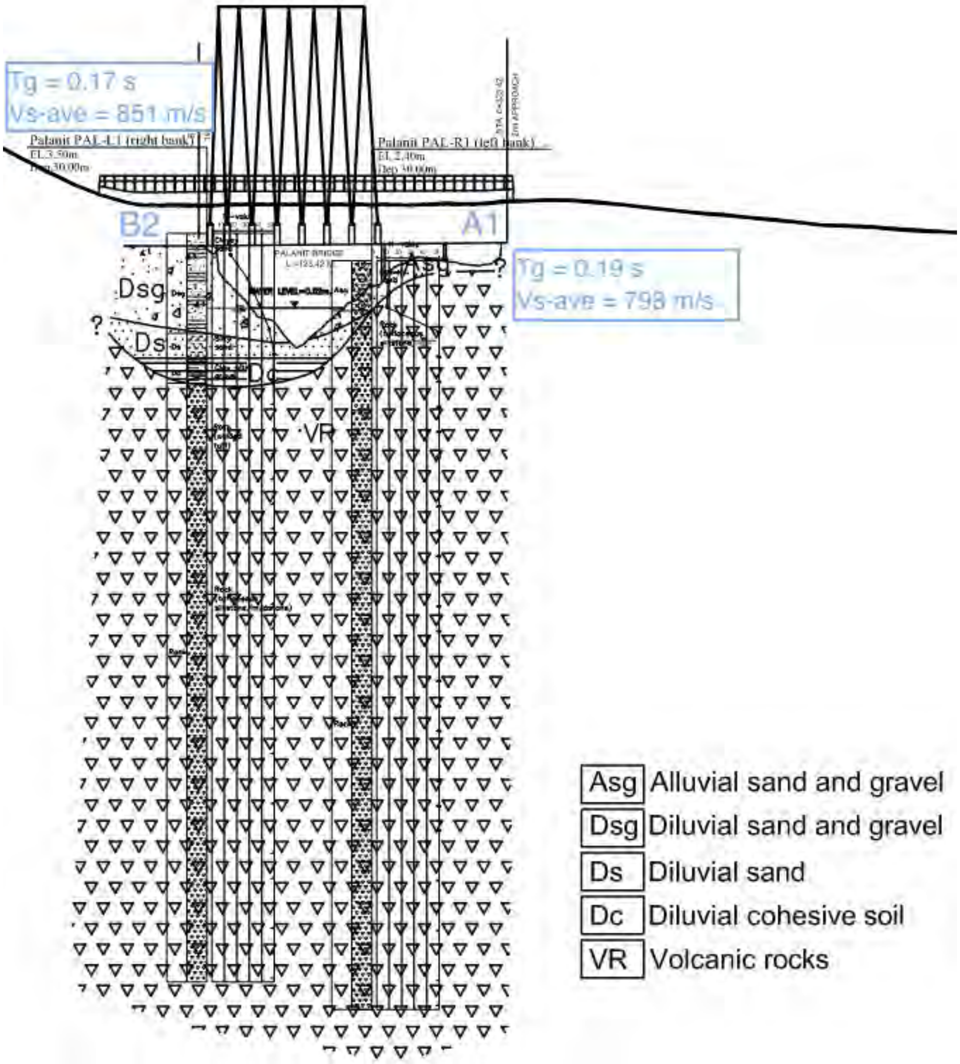


Figure 2B-106 Site-specific design spectra at 50-, 100-, 500-, 1000-year return periods for Lambingan Bridge B2 site vs. NSCP-Bridges-1997



(c) soil profile

Figure 2B-107 Palanit Bridge site data

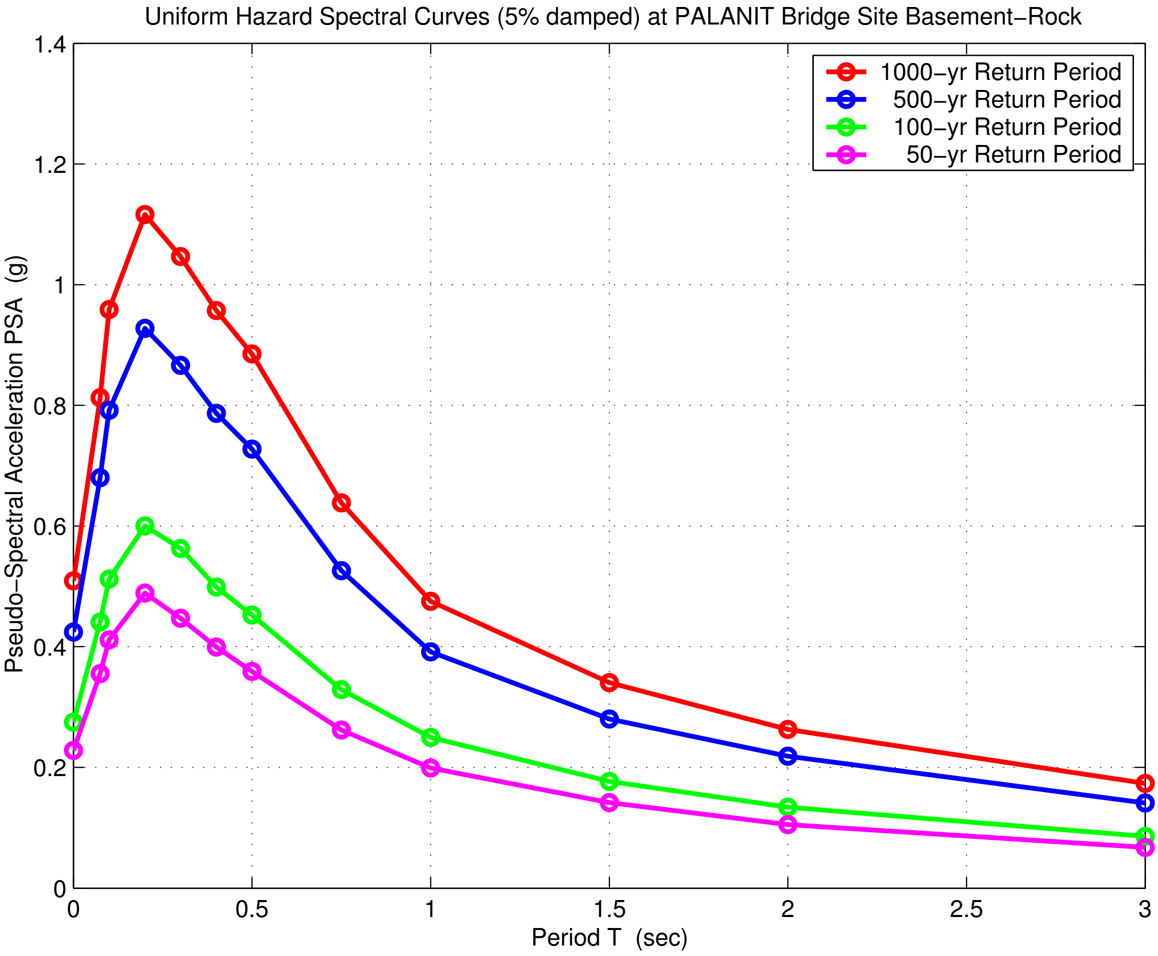


Figure 2B-108 Uniform hazard spectral curve for Palanit Bridge site at basement-rock

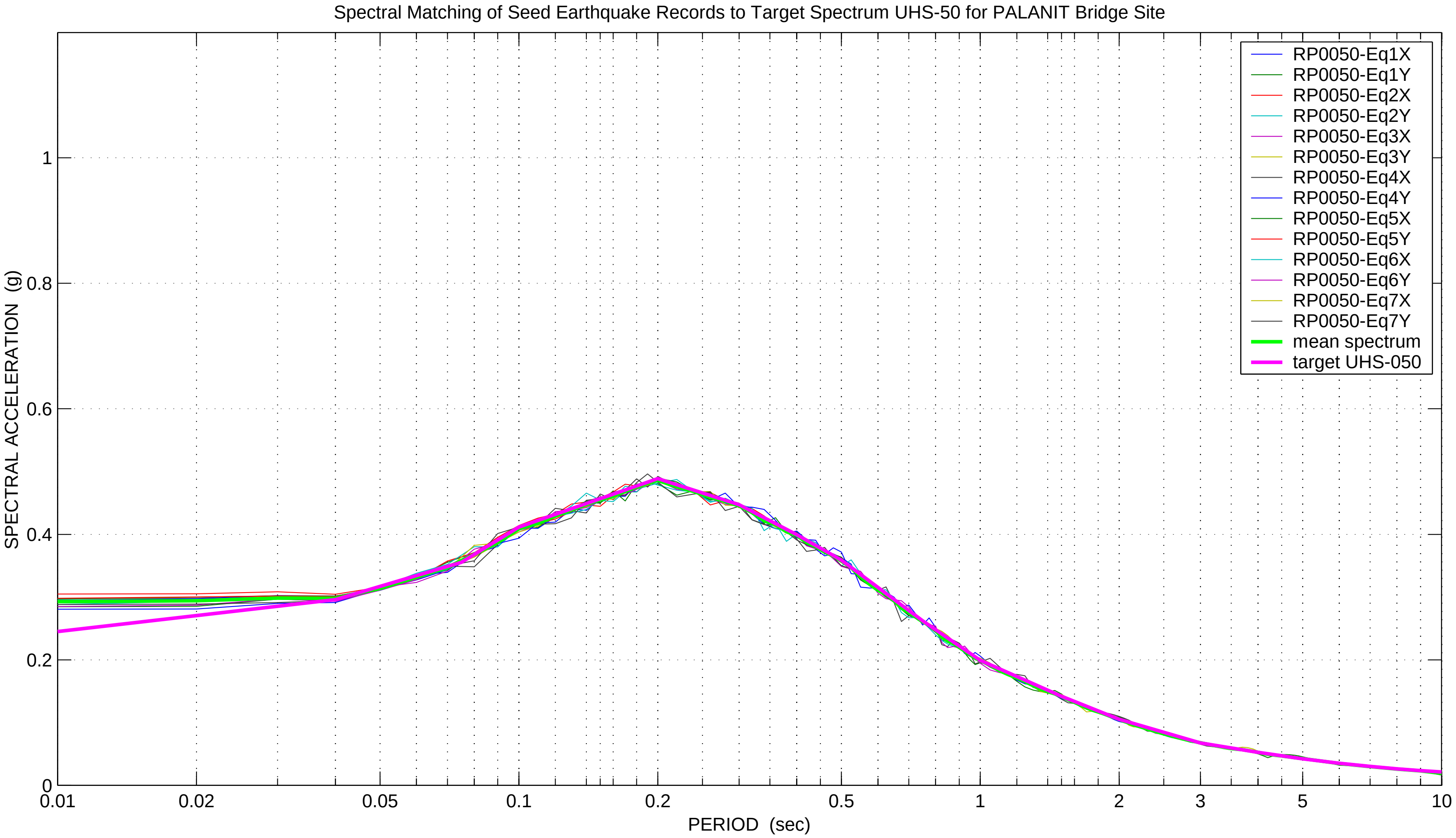


Figure 2B-109 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 50-year return period as compared to the target spectrum UHS-50 at Palanit Bridge

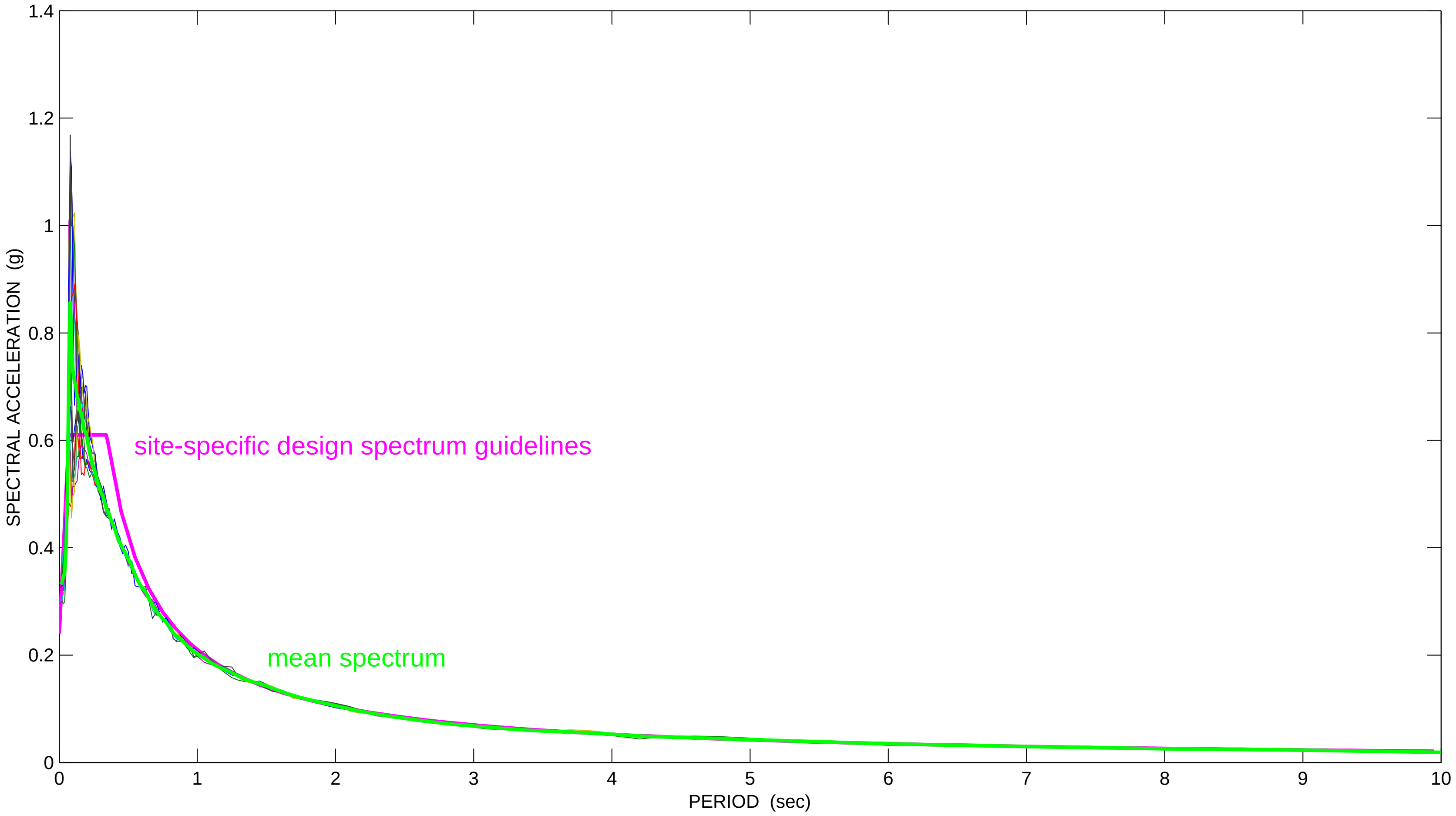


Figure 2B-110 Construction of site-specific design spectrum at 50-year return period for Palanit A1 based on mean of 14 response spectra from nonlinear site response analysis

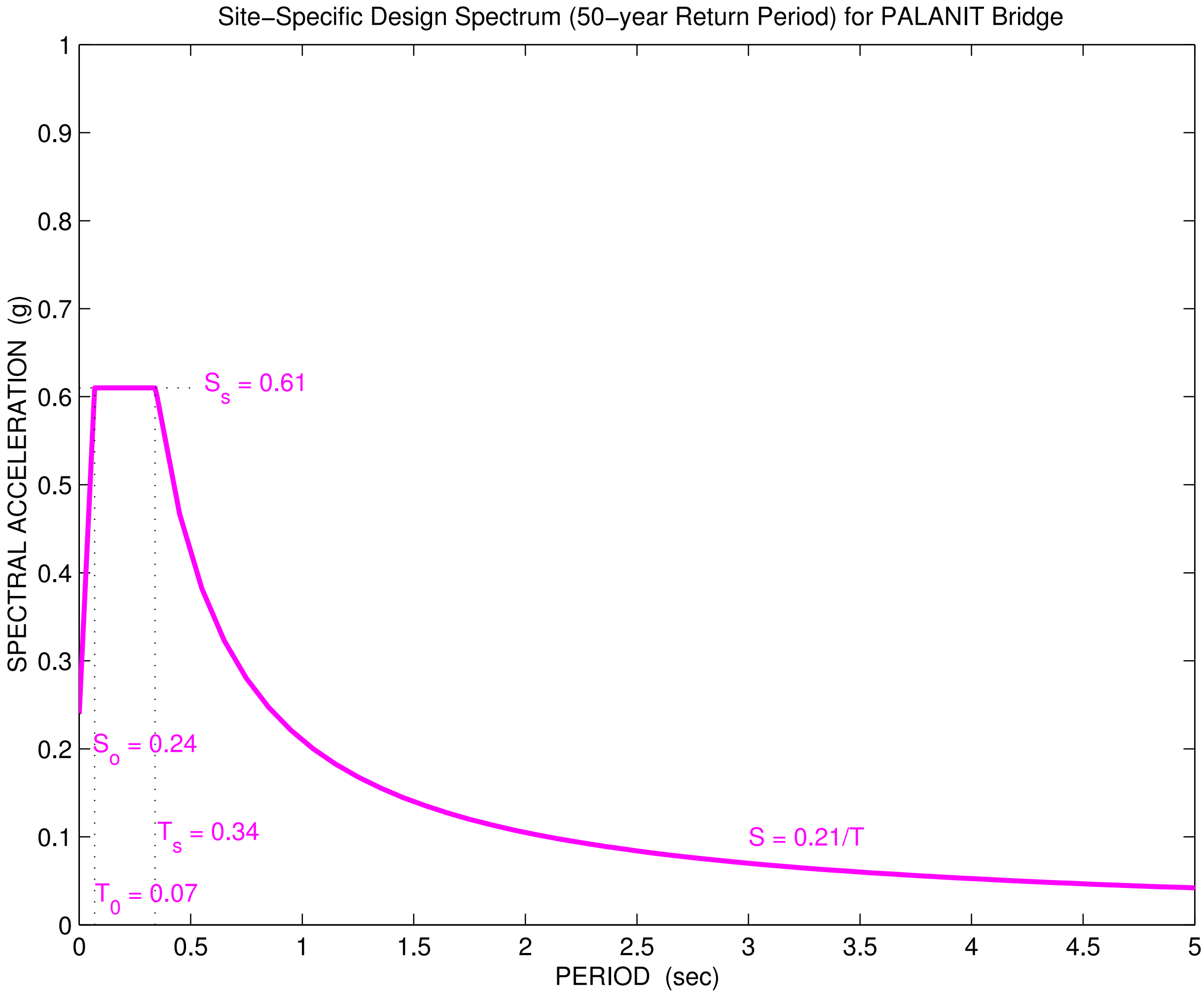


Figure 2B-111 Site-specific design spectrum at 50-year return period for Palanit Bridge at A1 site

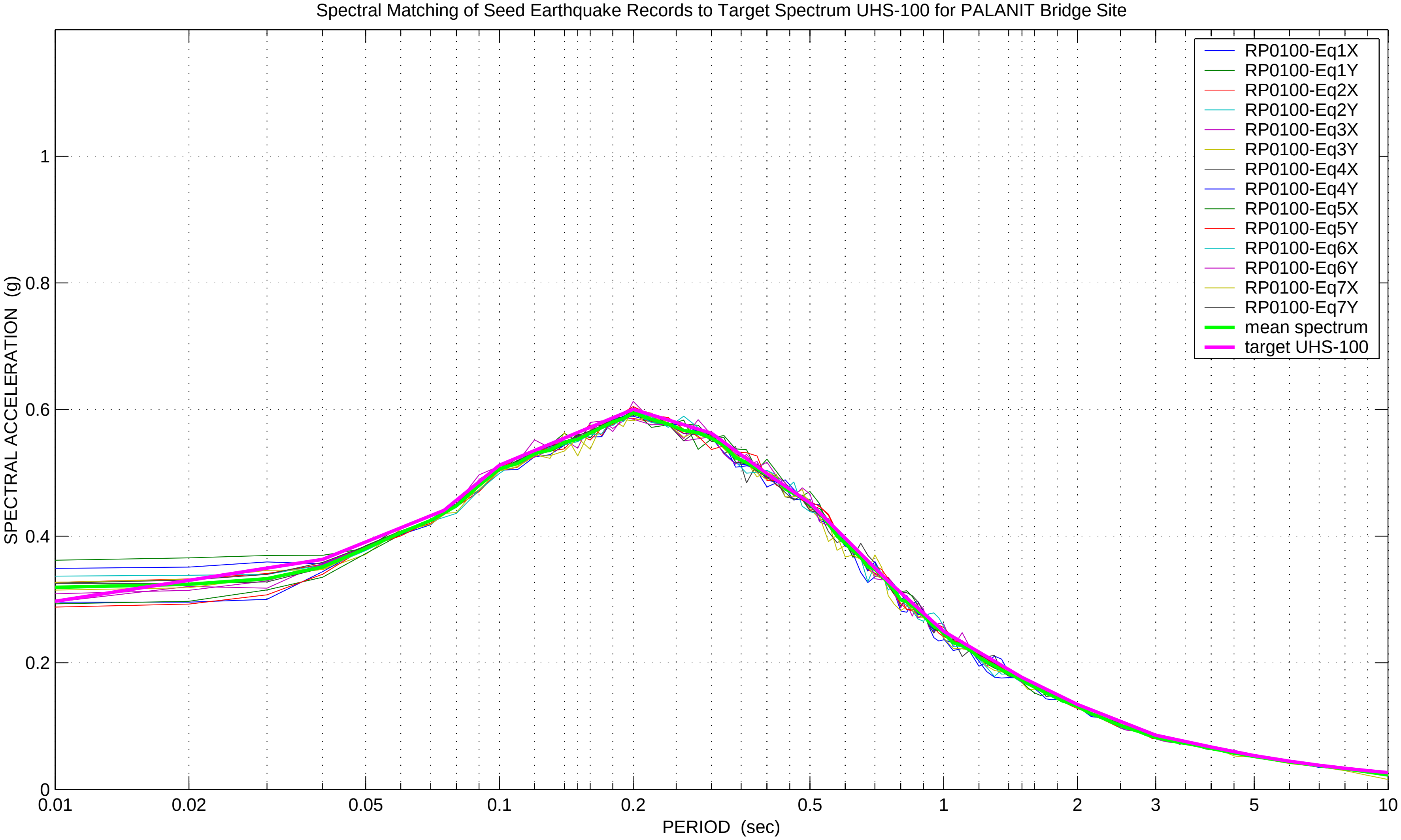


Figure 2B-112 Mean of $7\times 2 = 14$ spectrally matched earthquake time histories for 100-year return period as compared to the target spectrum UHS-100 at Palanit Bridge

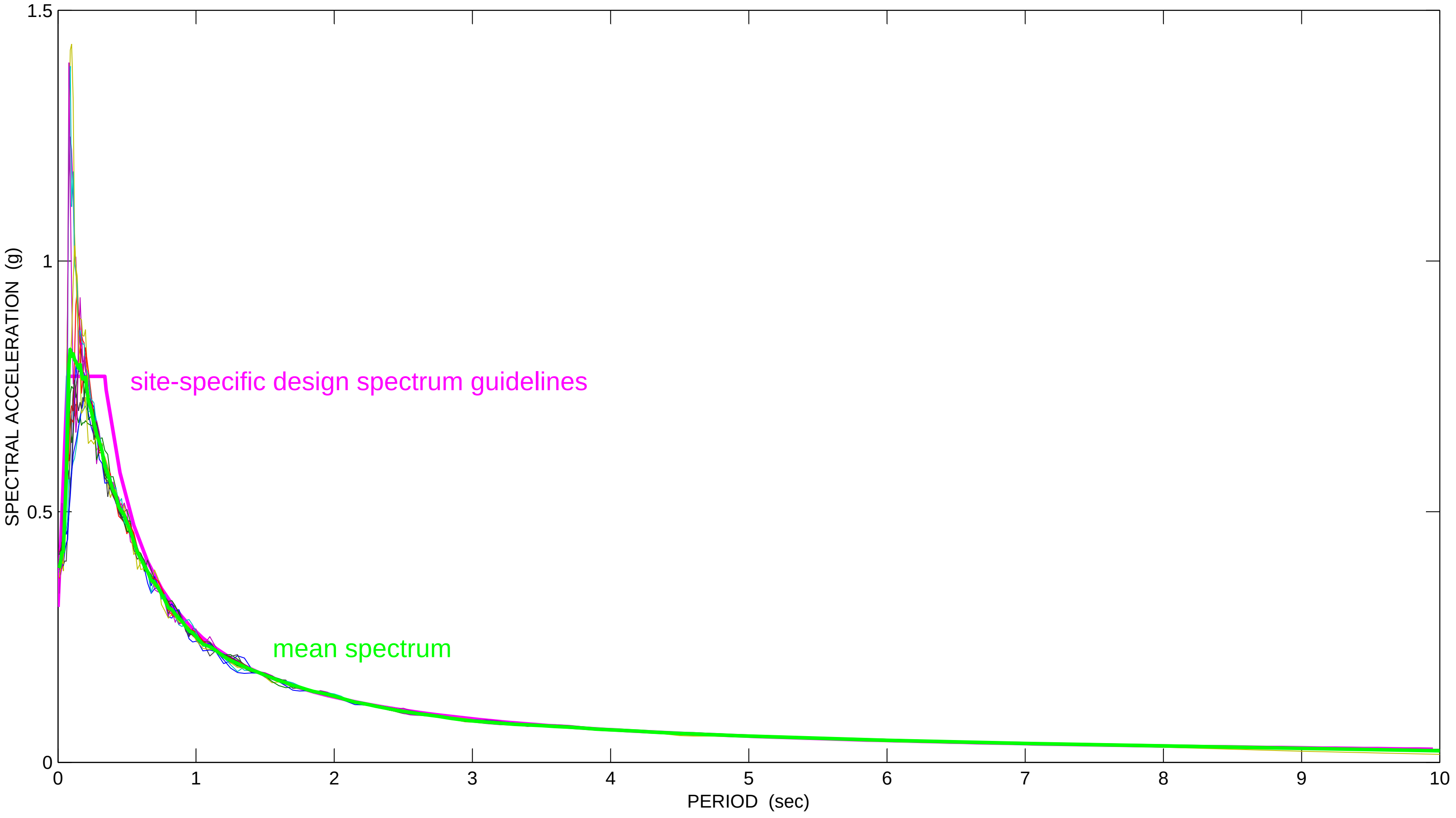


Figure 2B-113 Construction of site-specific design spectrum at 100-year return period for Palanit A1 based on mean of 14 response spectra from nonlinear site response analysis

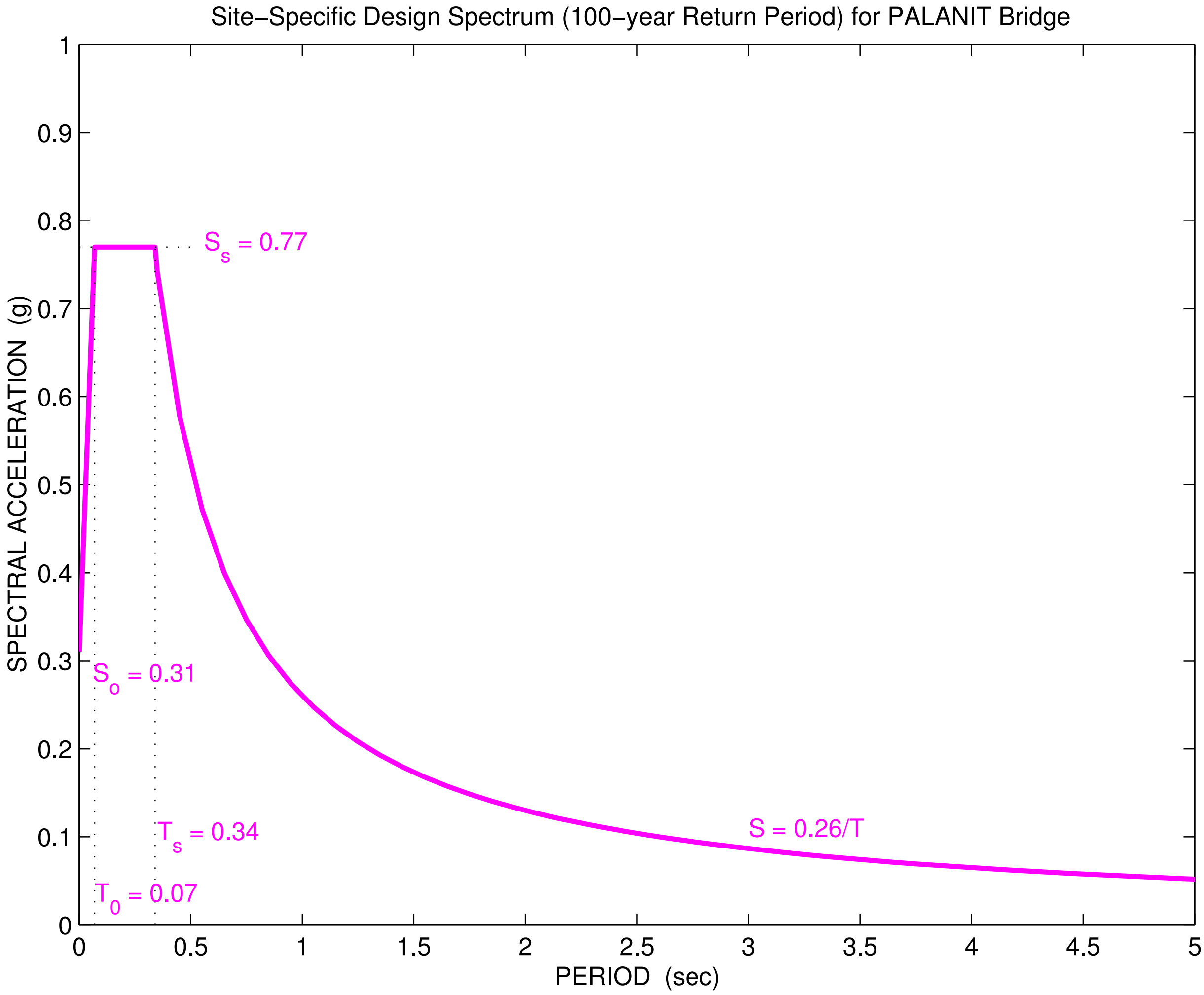


Figure 2B-114 Site-specific design spectrum at 100-year return period for Palanit Bridge at A1 site

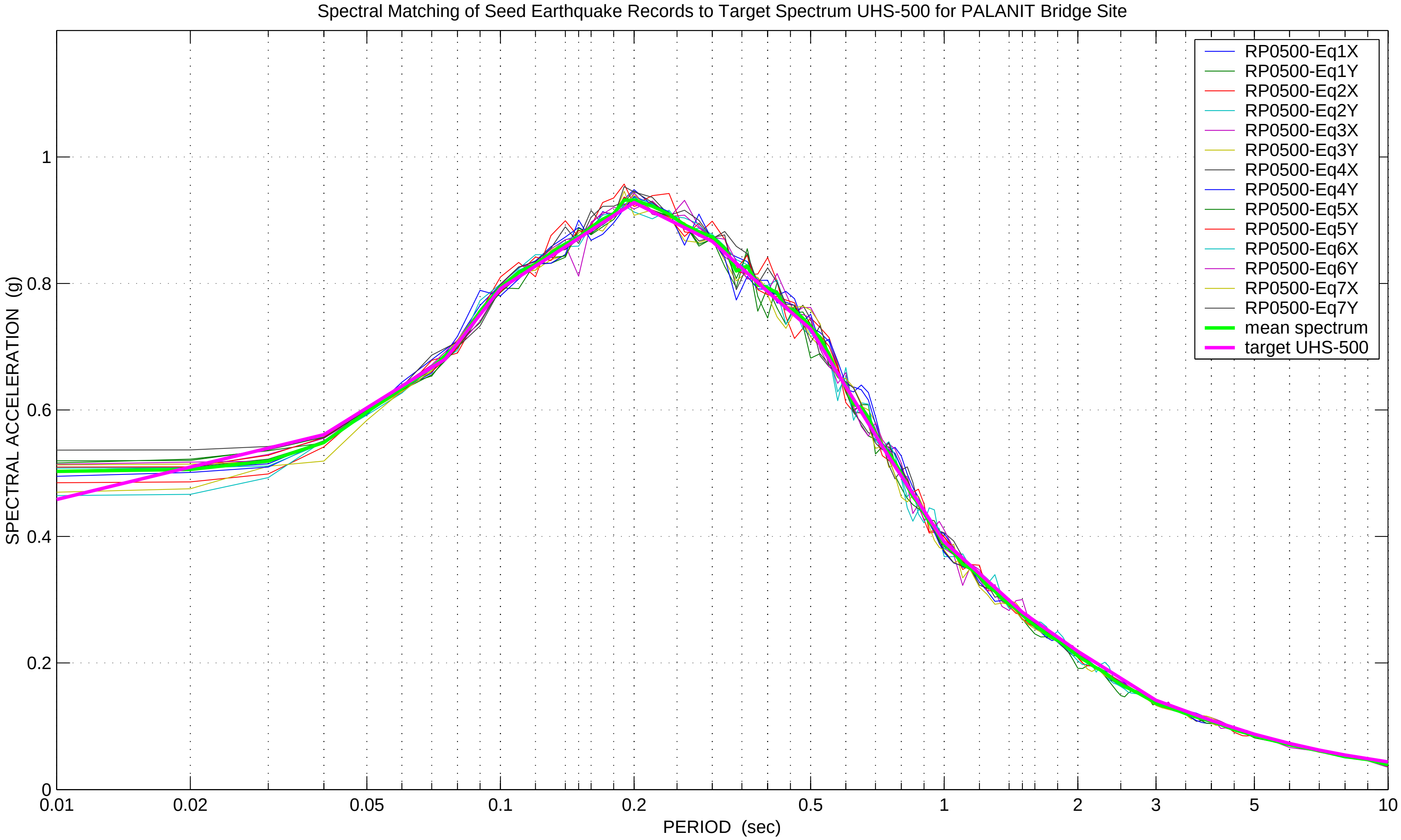


Figure 2B-115 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 500-year return period as compared to the target spectrum UHS-500 at Palanit Bridge

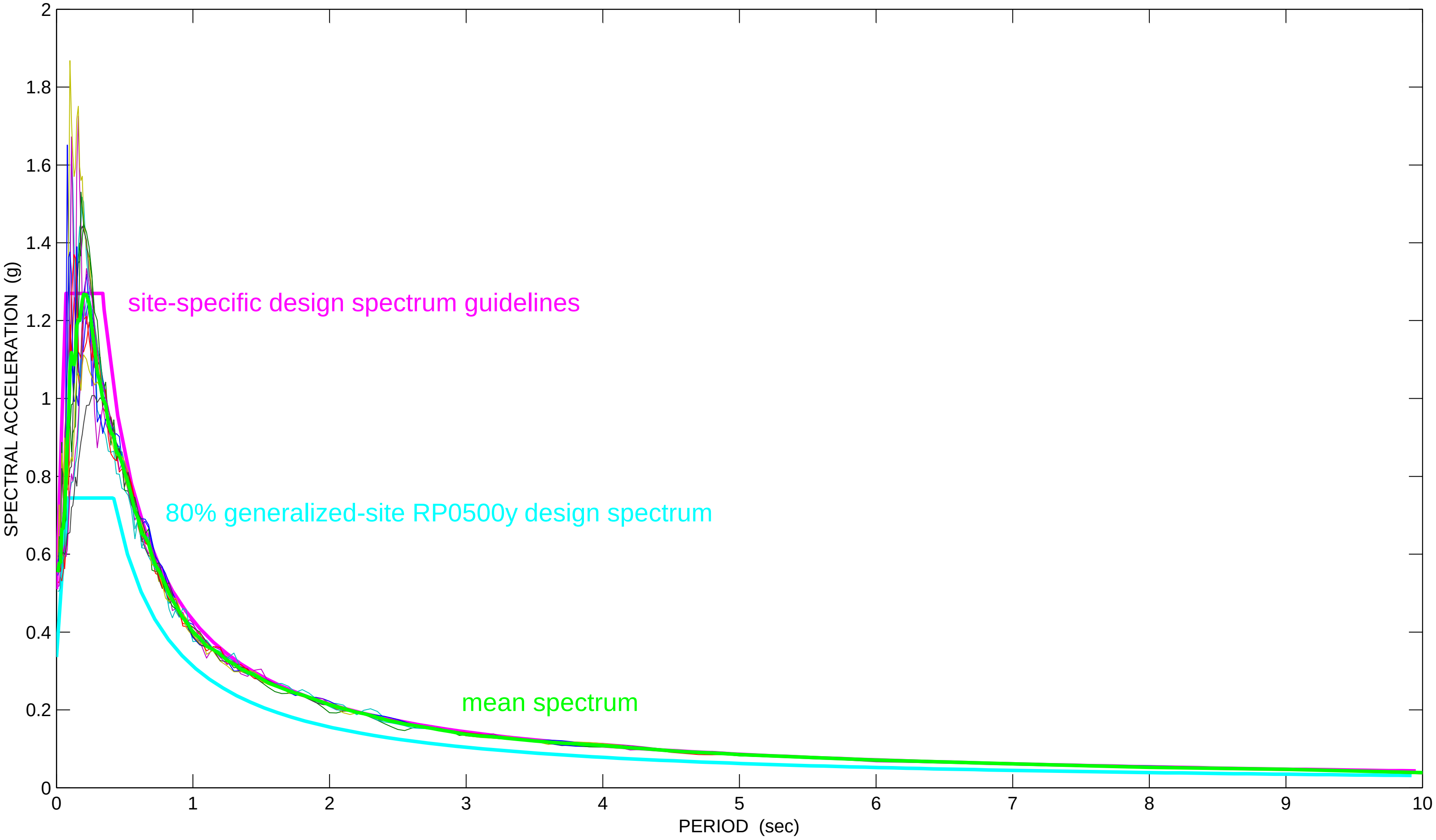


Figure 2B-116 Construction of site-specific design spectrum at 500-year return period for Palanit A1 based on mean of 14 response spectra from nonlinear site response analysis

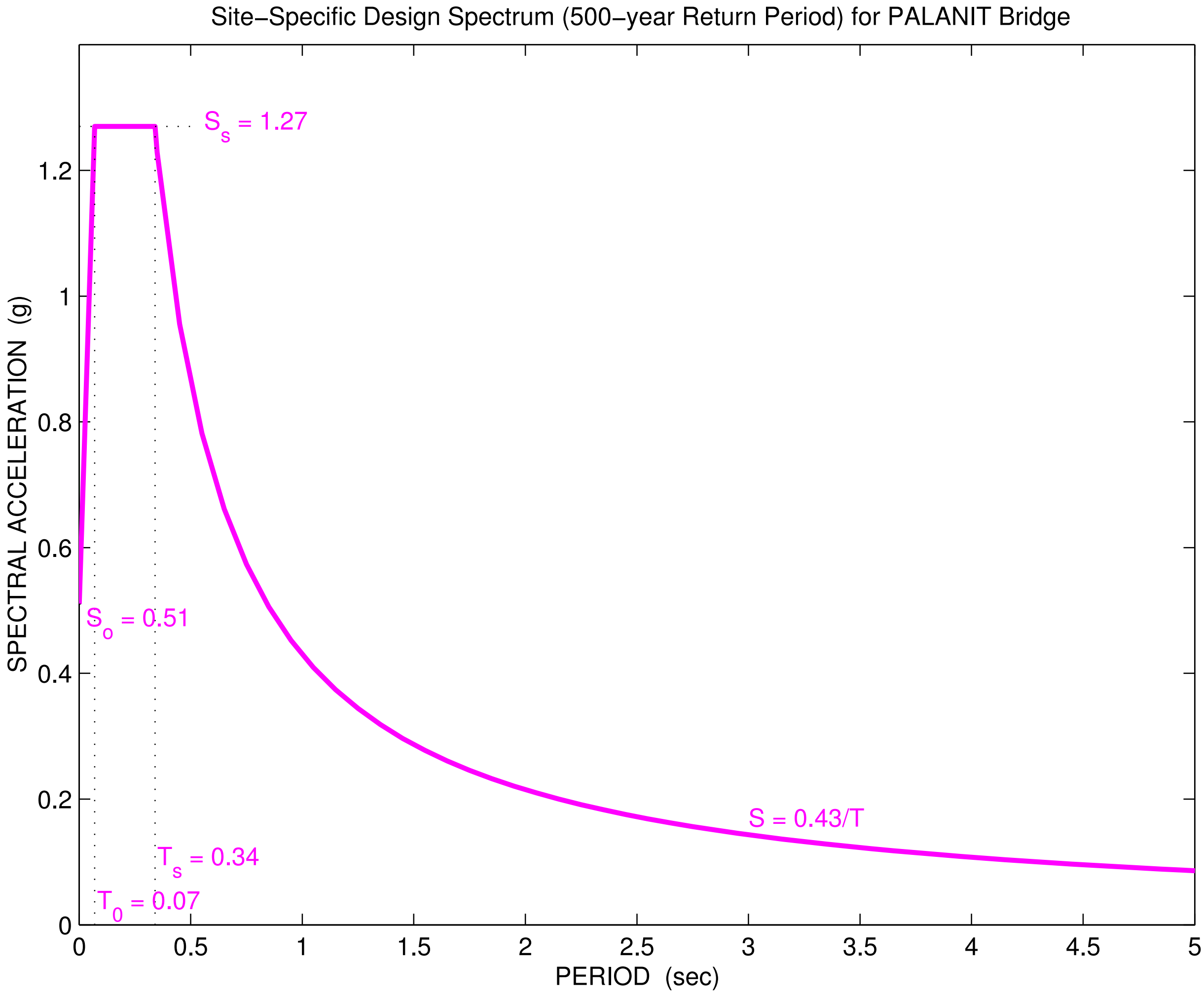


Figure 2B-117 Site-specific design spectrum at 500-year return period for Palanit Bridge at A1 site

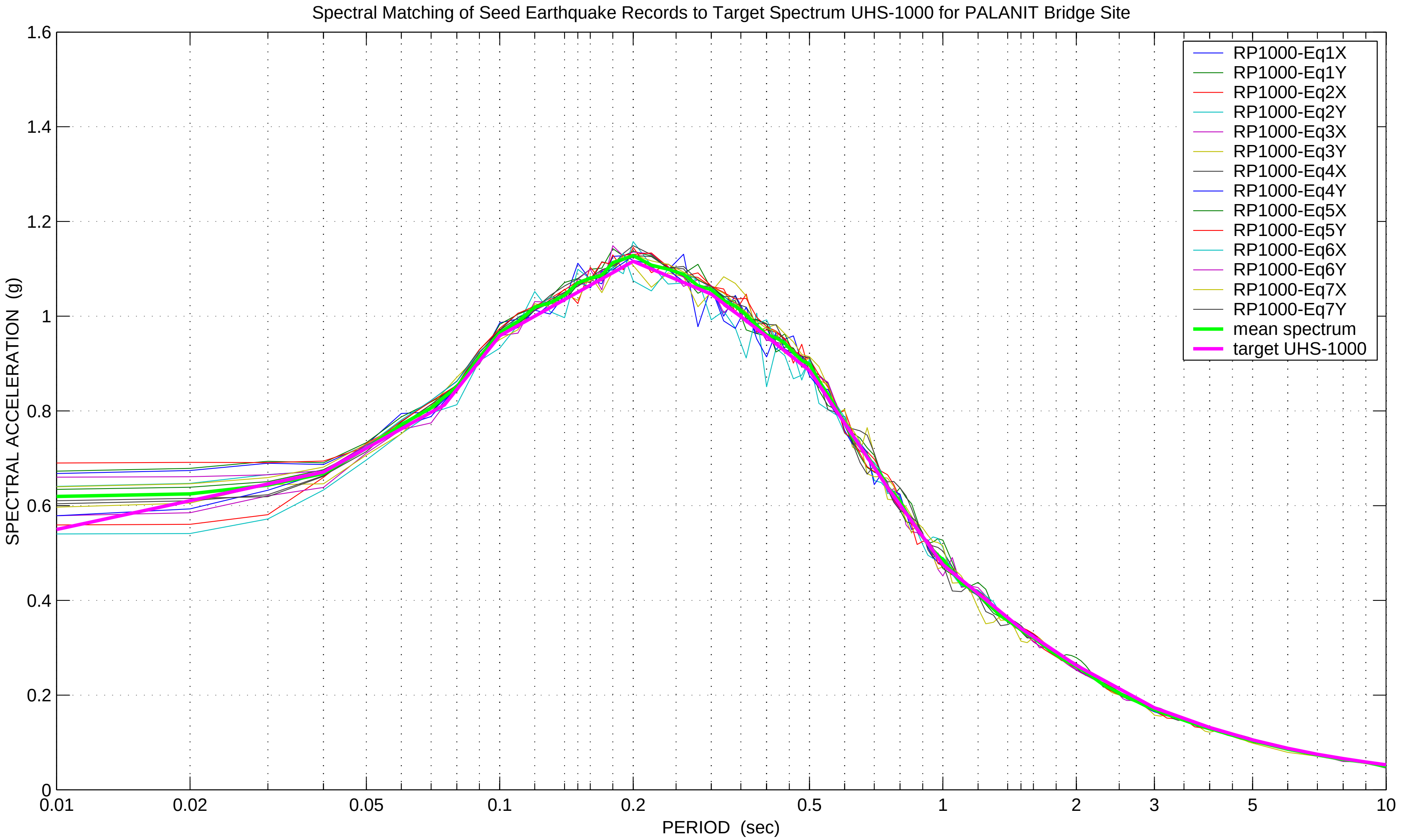


Figure 2B-118 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 1000-year return period as compared to the target spectrum UHS-1000 at Palanit Bridge

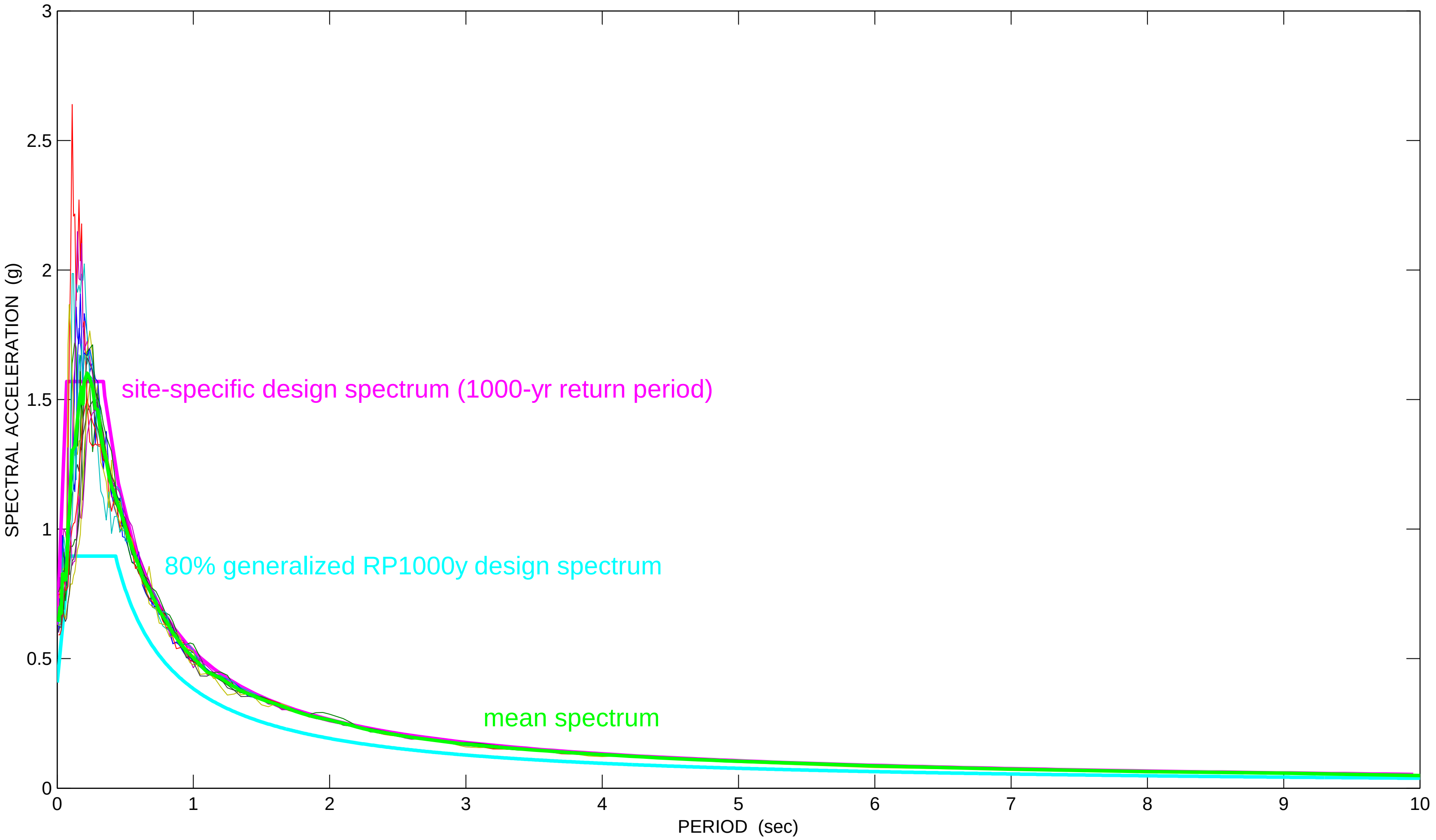


Figure 2B-119 Construction of site-specific design spectrum at 1000-year return period for Palanit A1 based on mean of 14 response spectra from nonlinear site response analysis

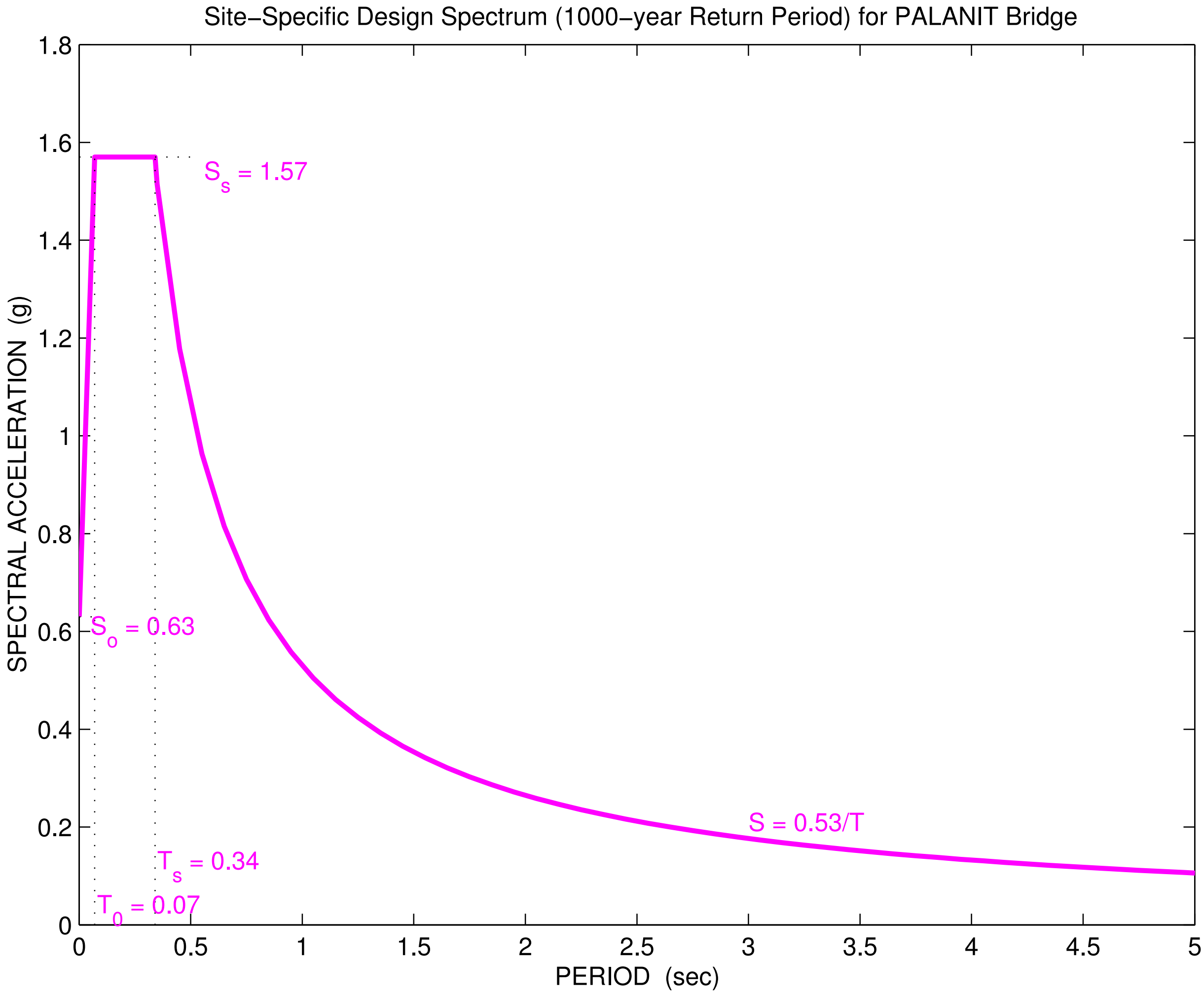


Figure 2B-120 Site-specific design spectrum at 1000-year return period for Palanit Bridge at A1 site

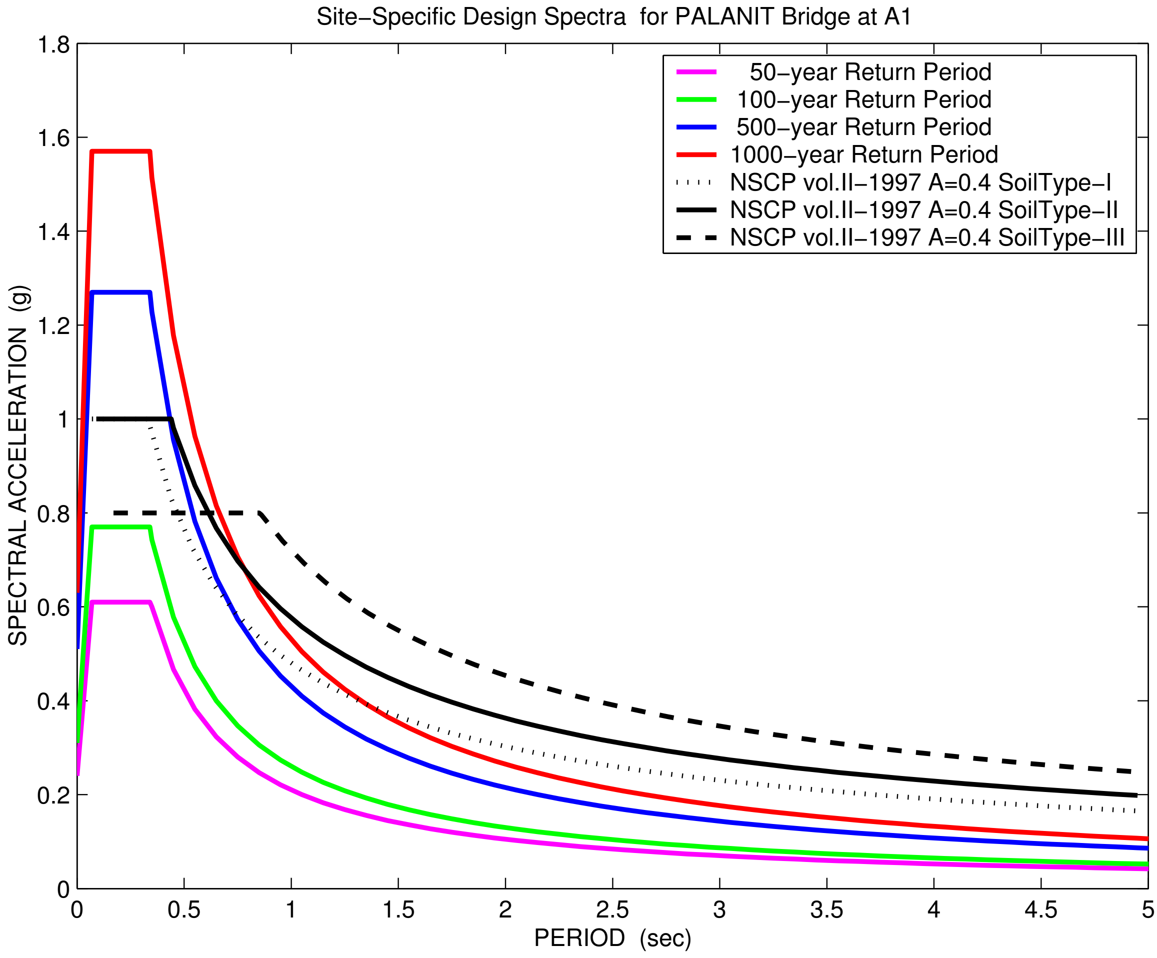
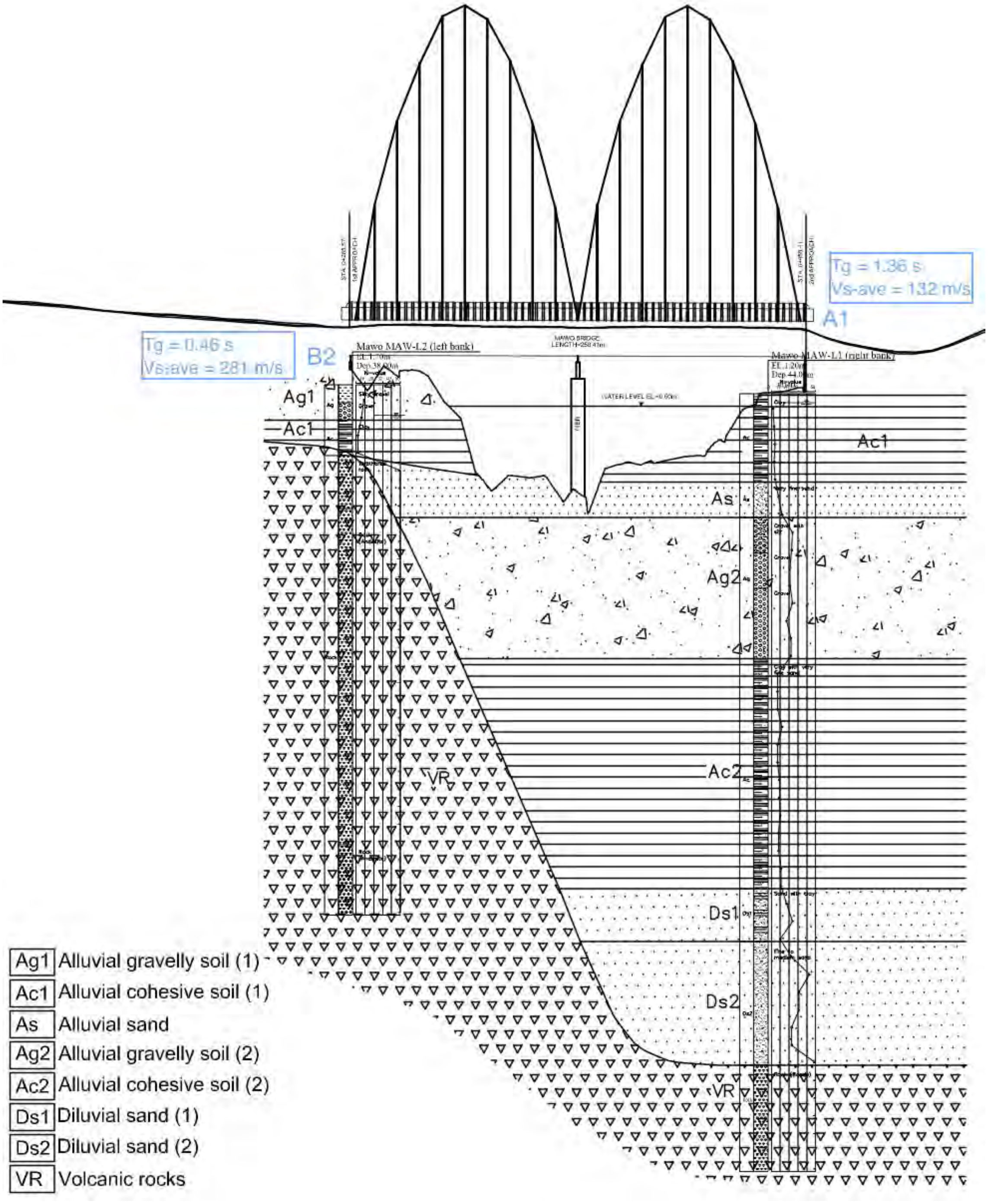


Figure 2B-121 Site-specific design spectra for Palanit Bridge at A1



(c) soil profile

Figure 2B-122 Mawo Bridge site data

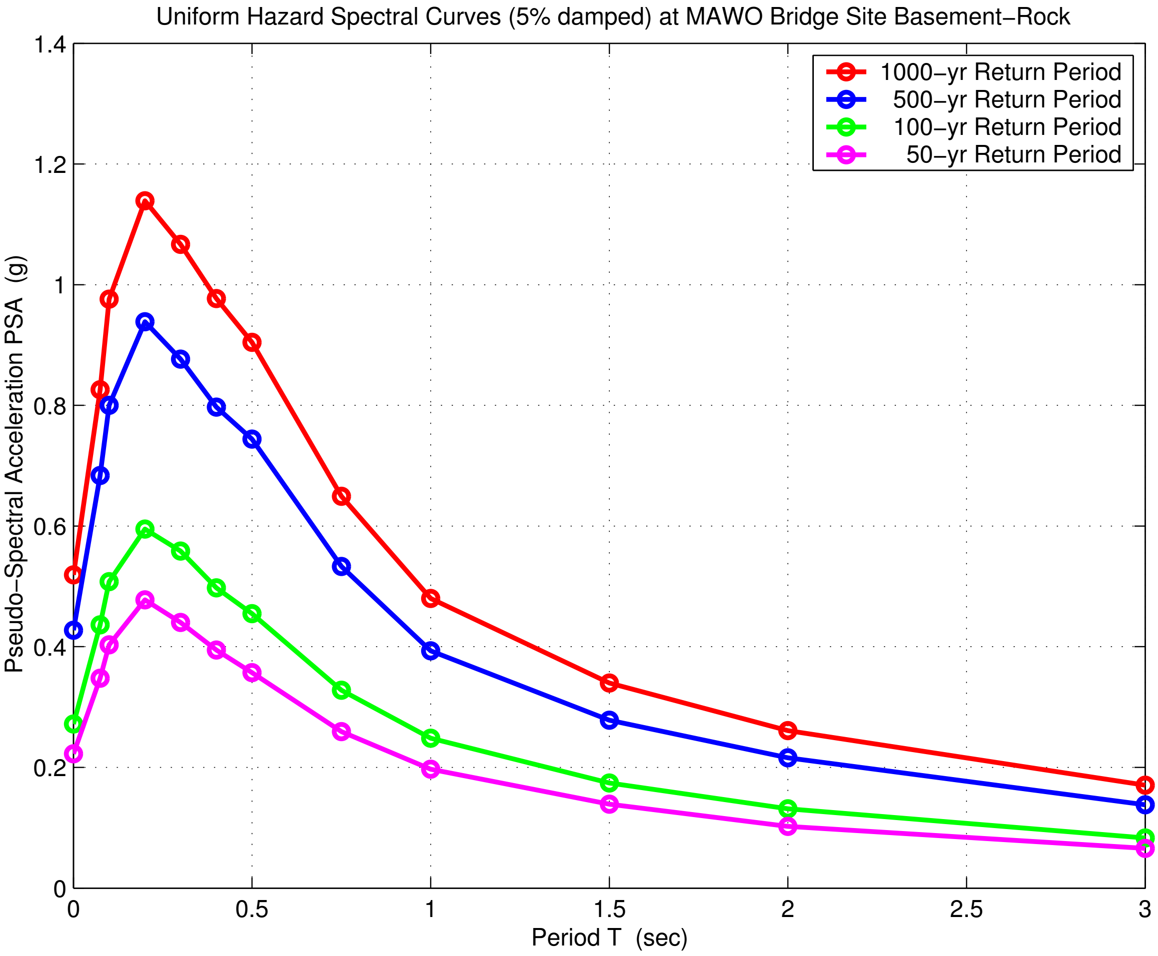


Figure 2B-123 Uniform hazard spectral curve for Mawo Bridge site at basement-rock

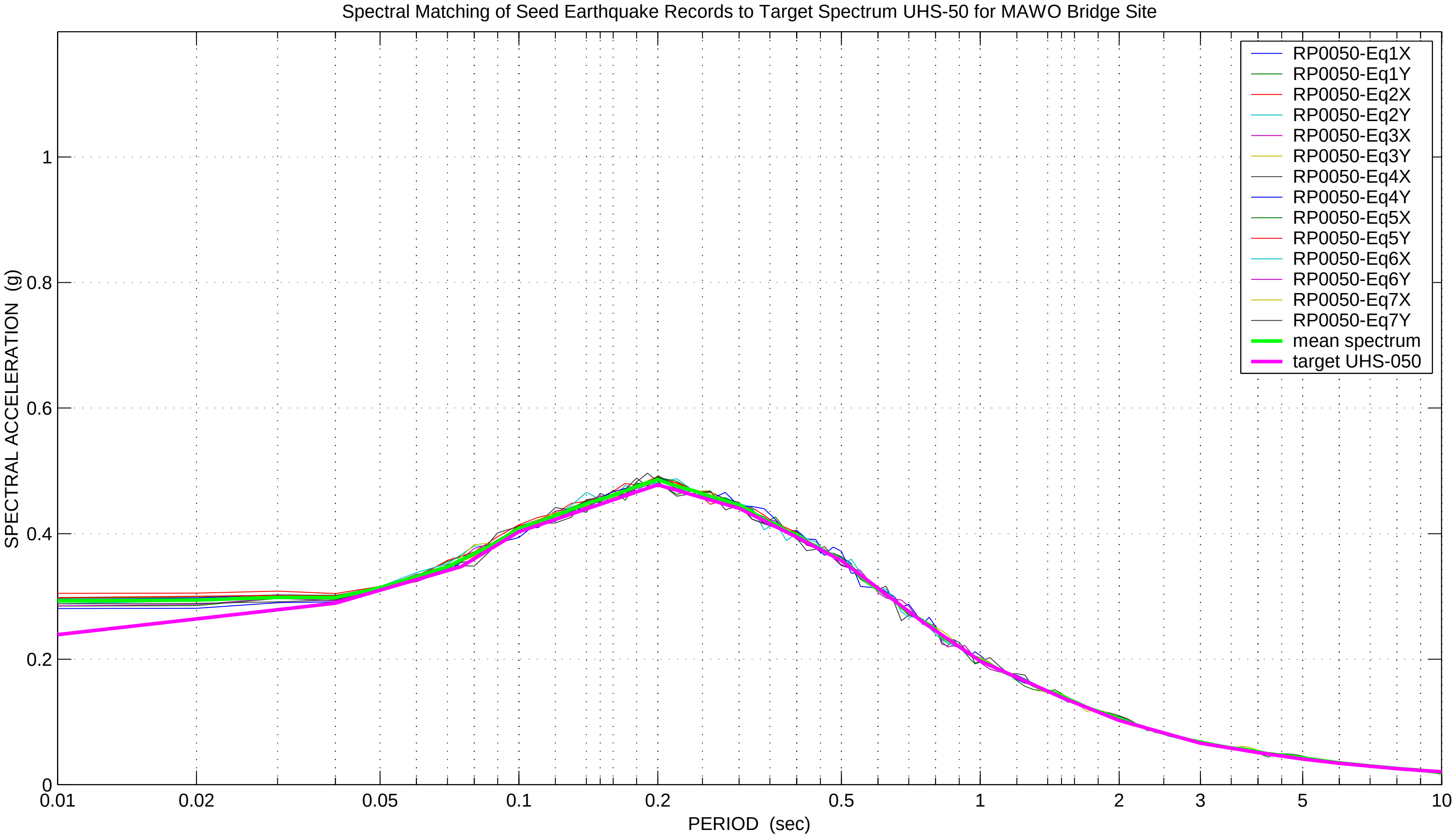


Figure 2B-124 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 50-year return period as compared to the target spectrum UHS-50 at Mawo Bridge

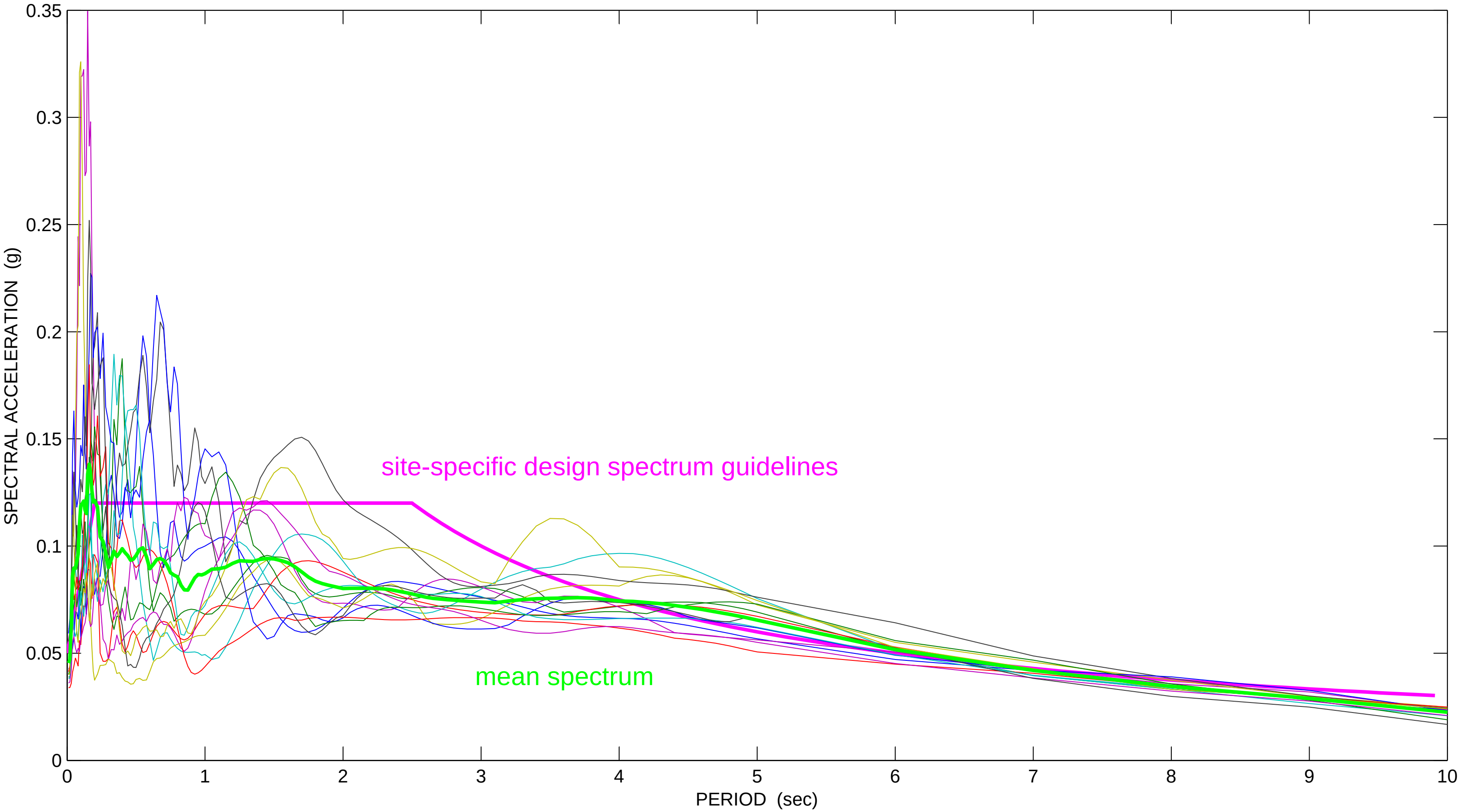


Figure 2B-125 Construction of site-specific design spectrum at 50-year return period for Mawo A1 based on mean of 14 response spectra from nonlinear site response analysis

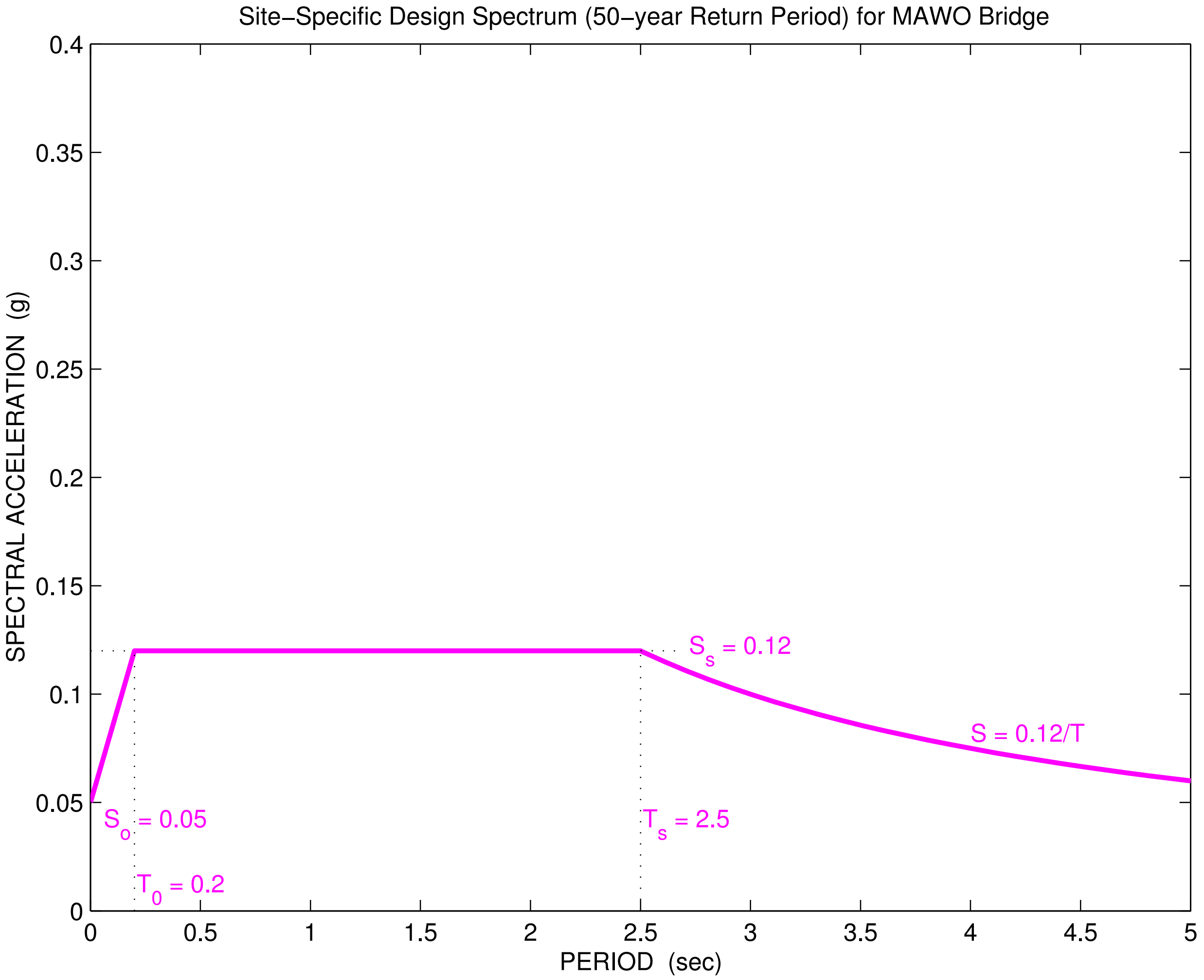


Figure 2B-126 Site-specific design spectrum at 50-year return period for Mawo Bridge at A1 site

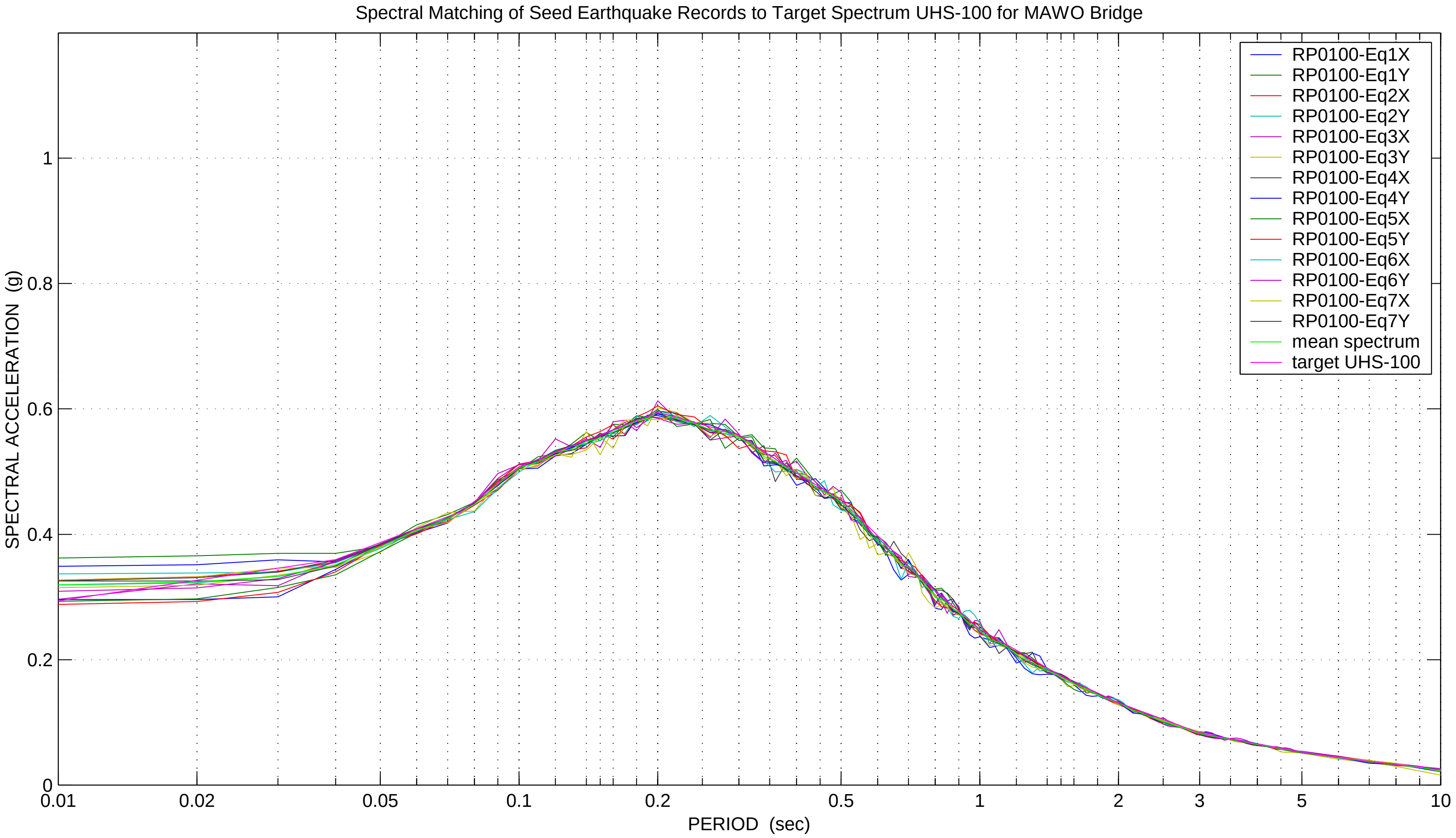


Figure 2B-127 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 100-year return period as compared to the target spectrum UHS-100 at Mawo Bridge

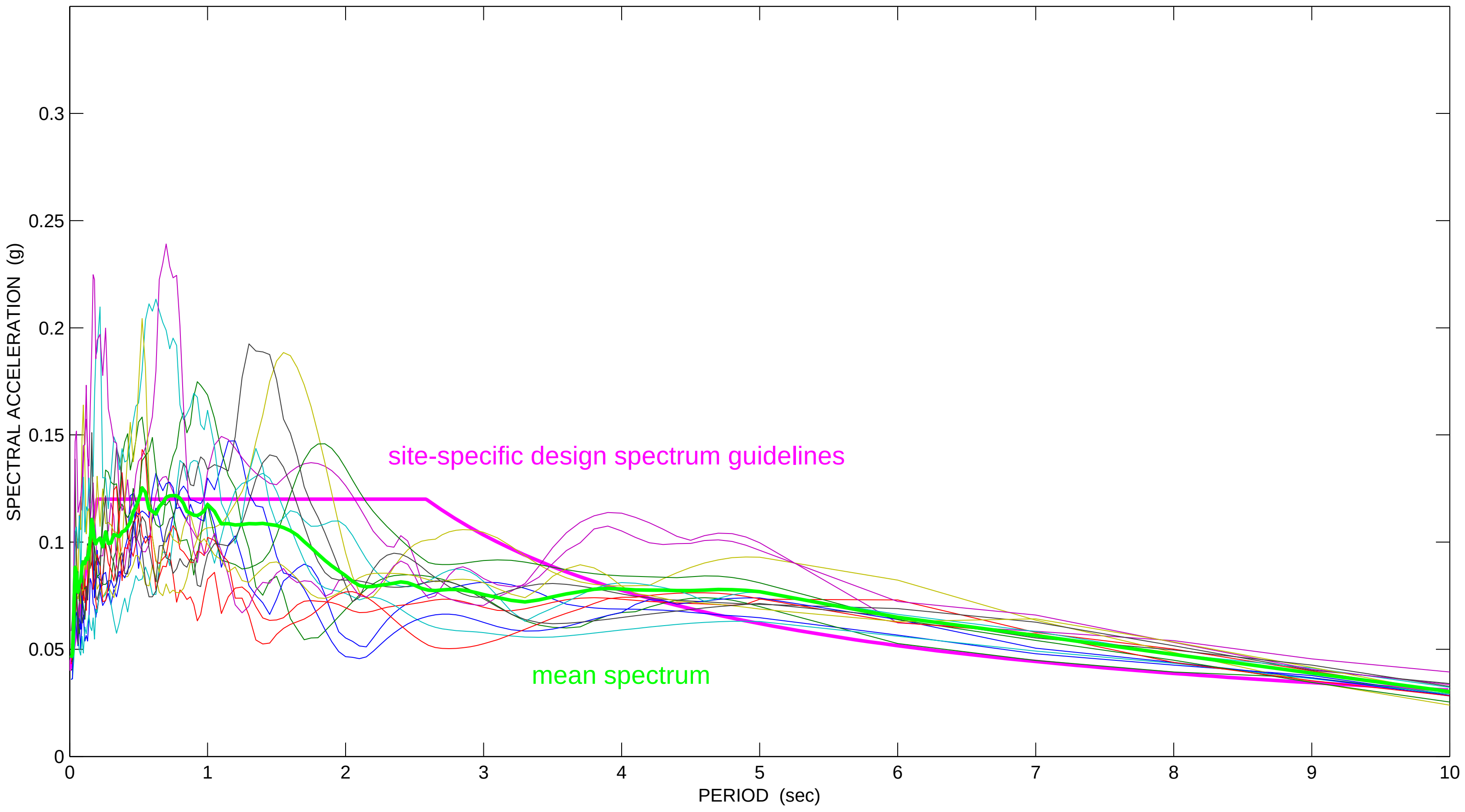


Figure 2B-128 Construction of site-specific design spectrum at 100-year return period for Mawo A1 based on mean of 14 response spectra from nonlinear site response analysis

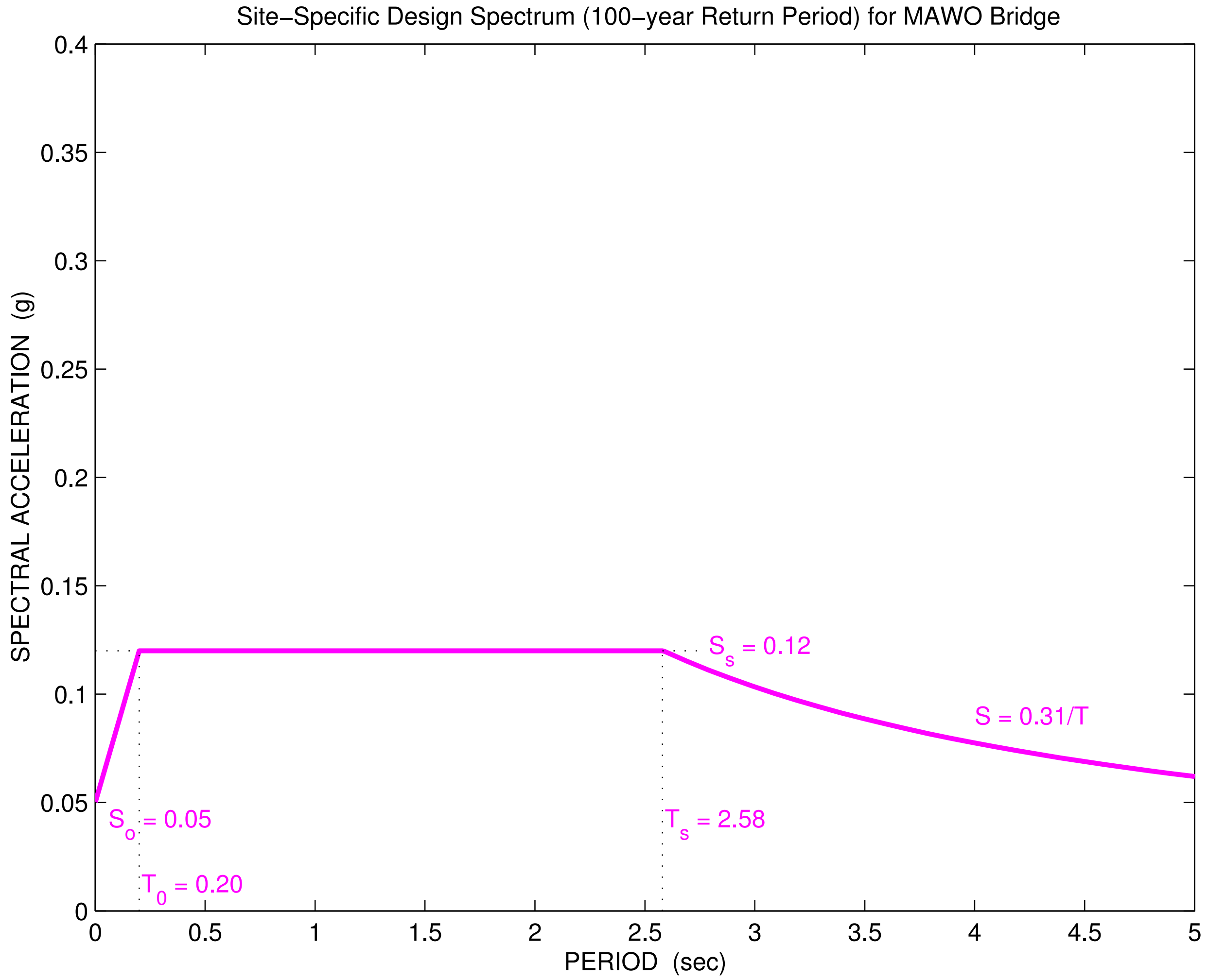


Figure 2B-129 Site-specific design spectrum at 100-year return period for Mawo Bridge at A1 site

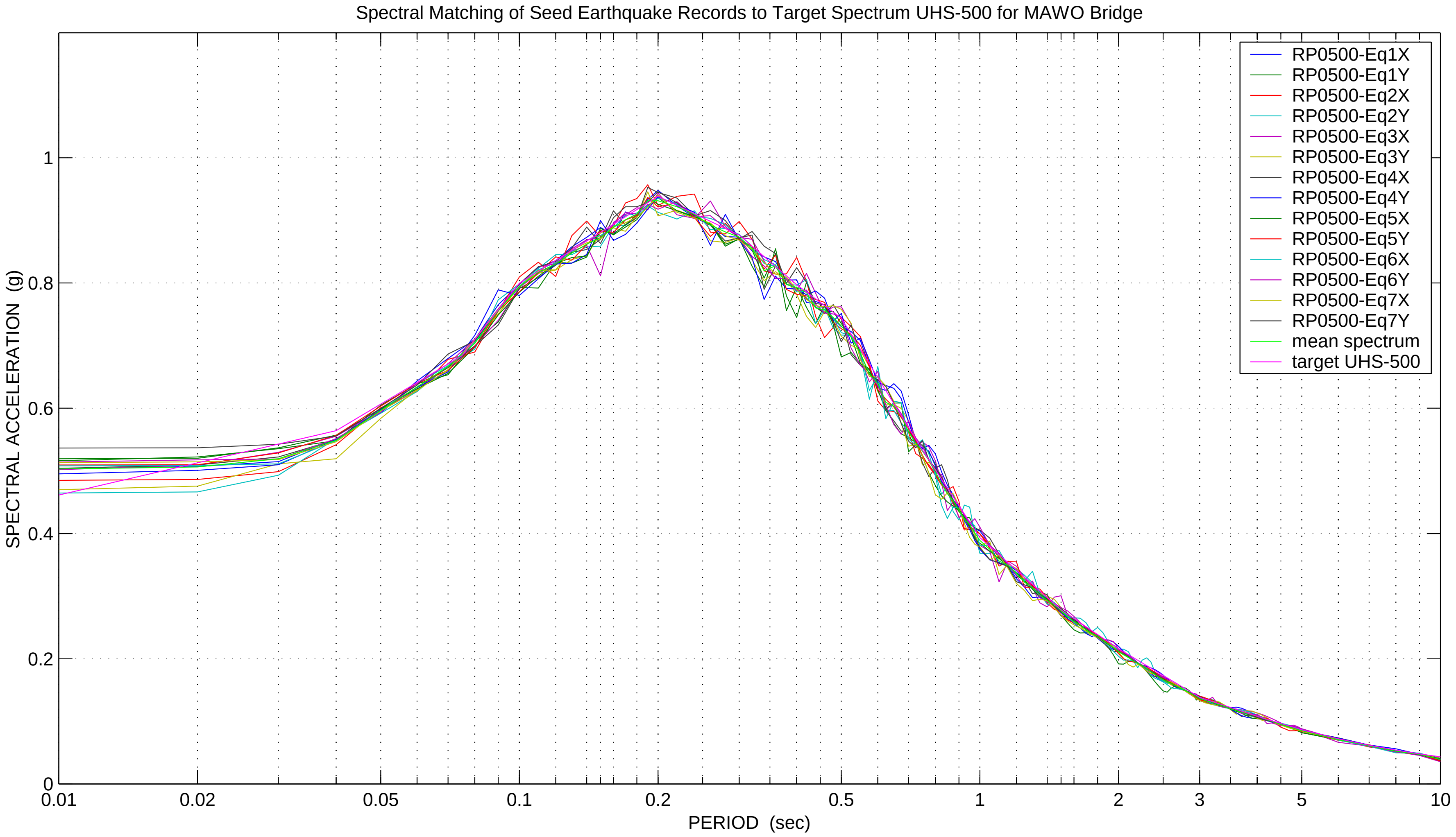


Figure 2B-130 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 500-year return period as compared to the target spectrum UHS-500 at Mawo Bridge

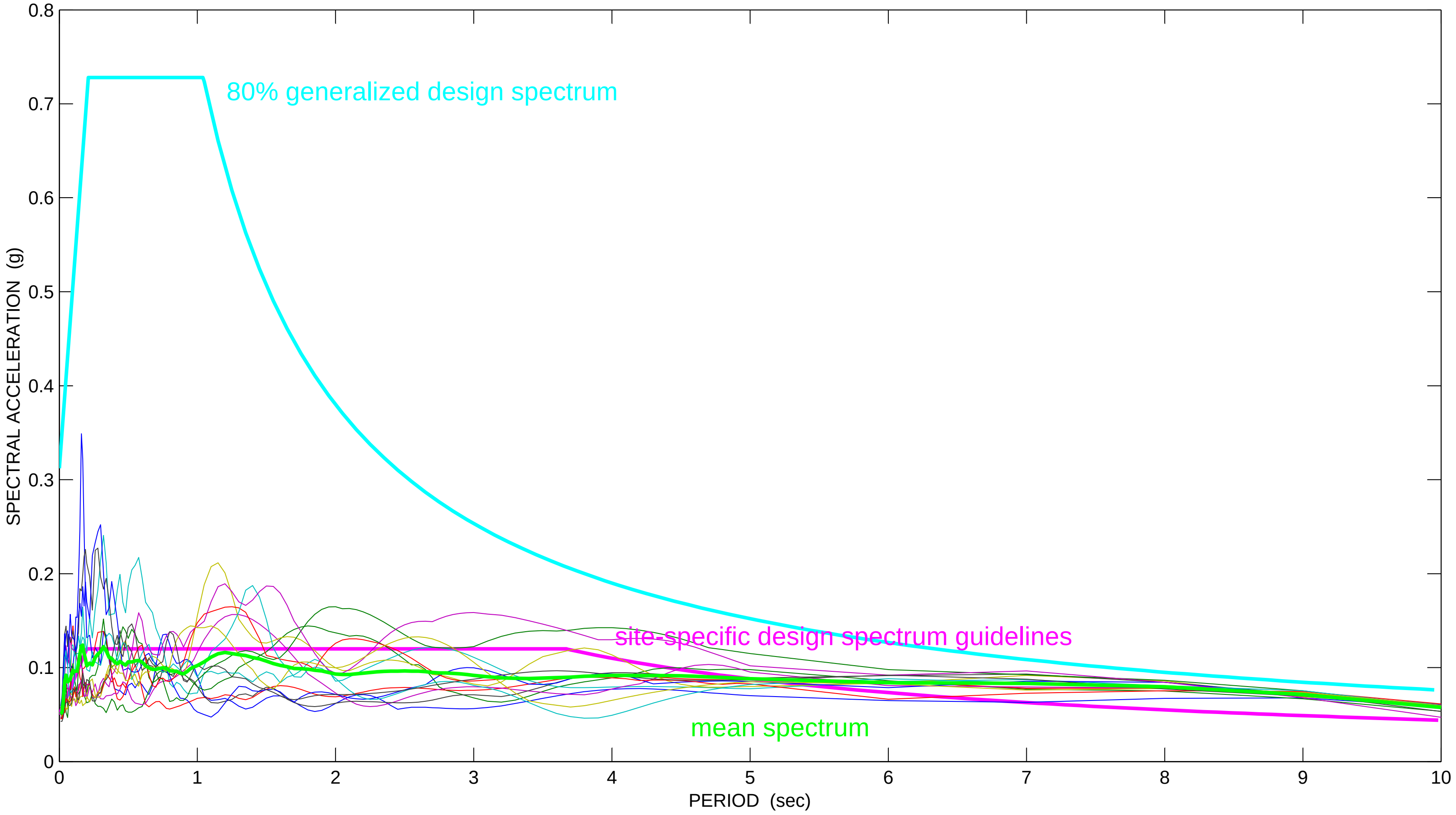


Figure 2B-131 Construction of site-specific design spectrum at 500-year return period for Mawo A1 based on mean of 14 response spectra from nonlinear site response analysis

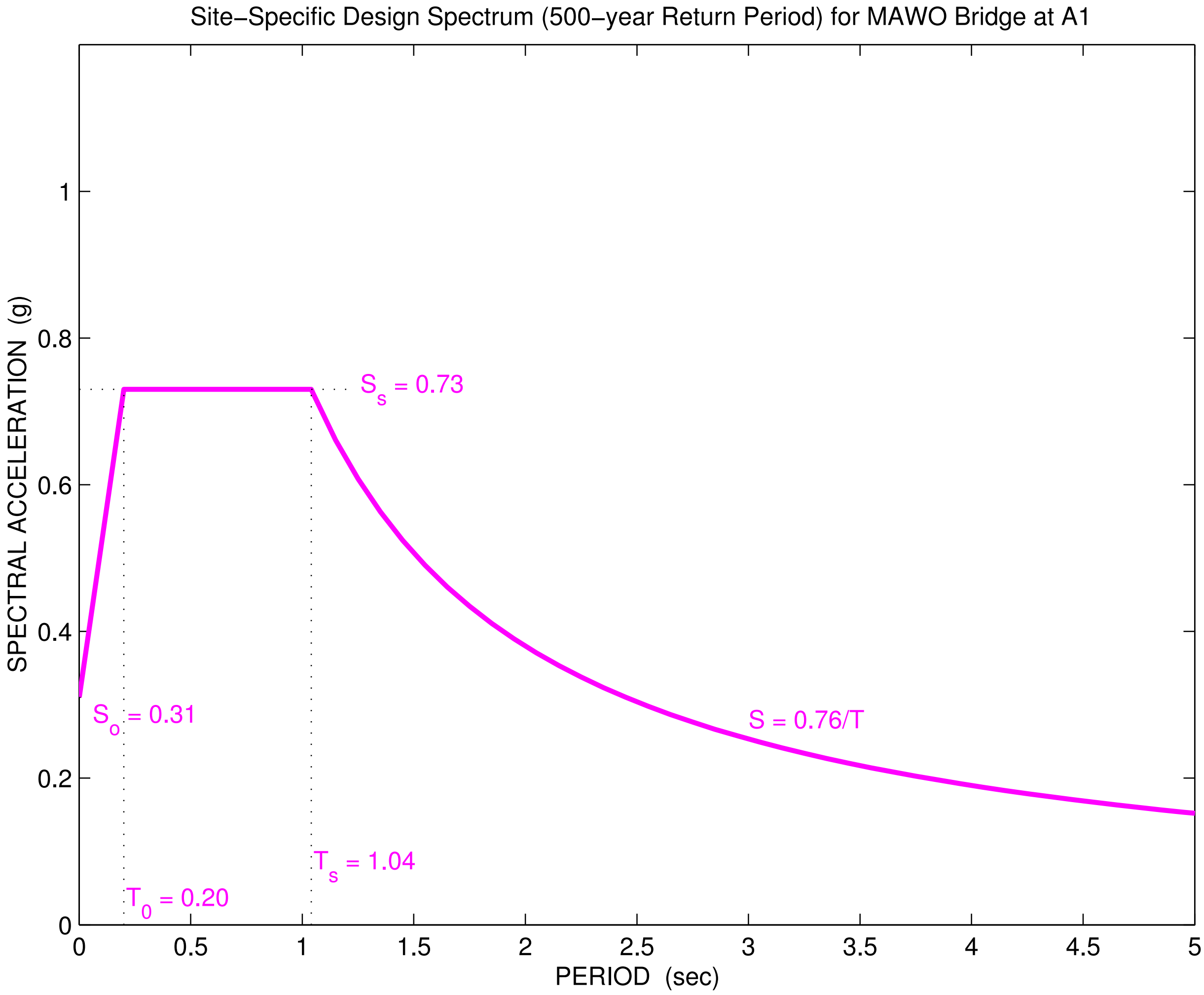


Figure 2B-132 Site-specific design spectrum at 500-year return period for Mawo Bridge at A1 site

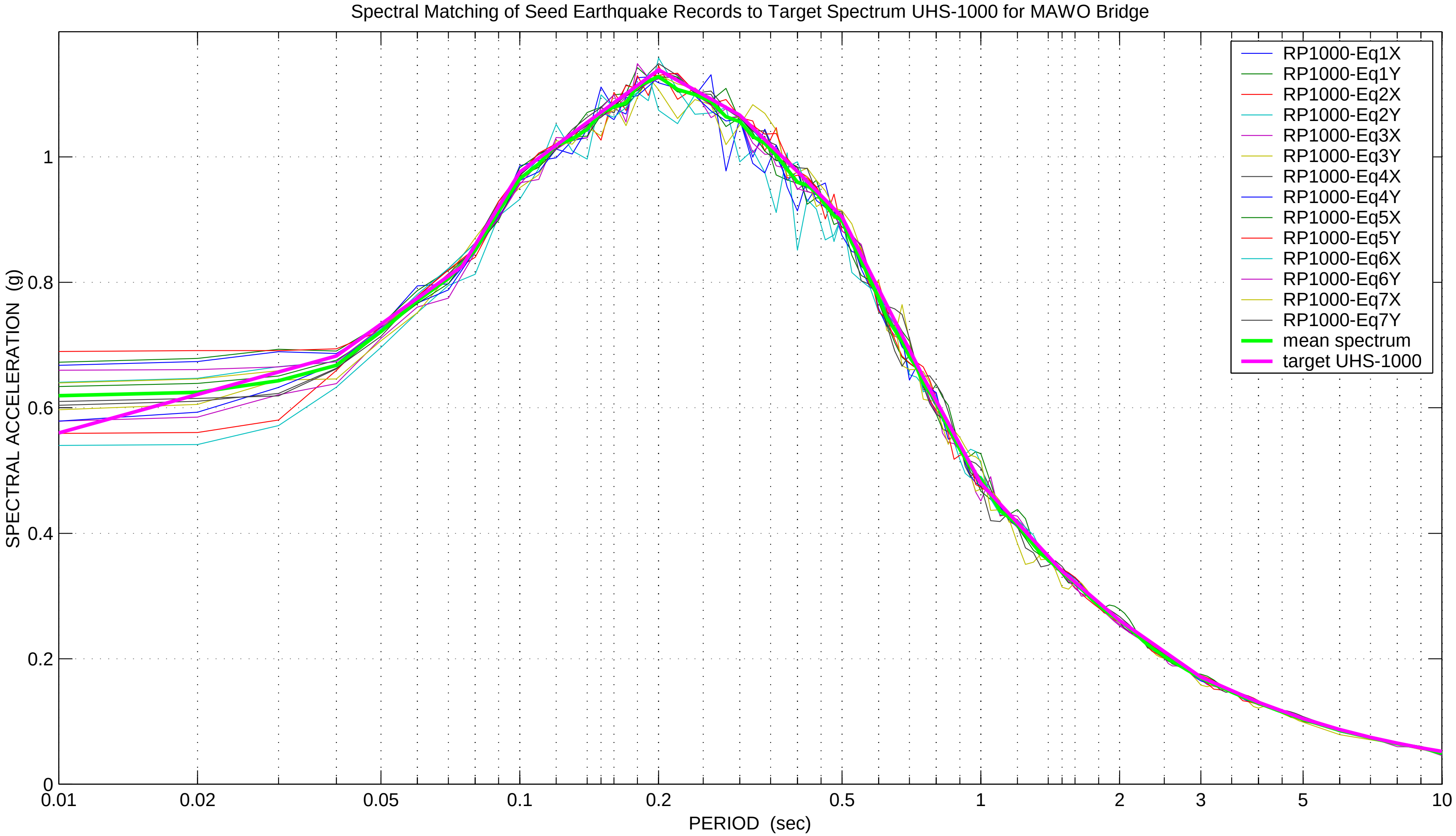


Figure 2B-133 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 1000-year return period as compared to the target spectrum UHS-1000 at Mawo Bridge

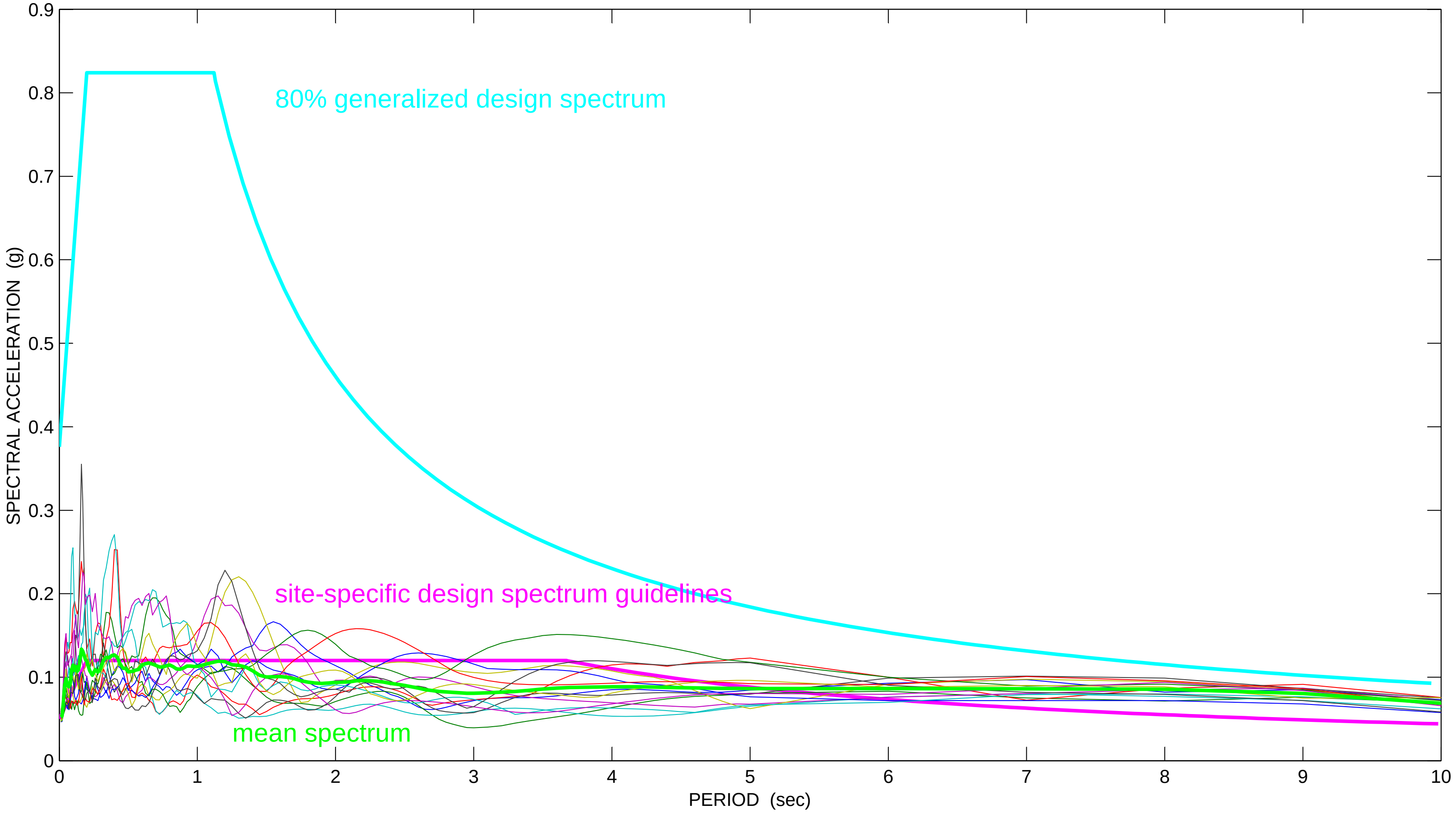


Figure 2B-134 Construction of site-specific design spectrum at 1000-year return period for Mawo A1 based on mean of 14 response spectra from nonlinear site response analysis

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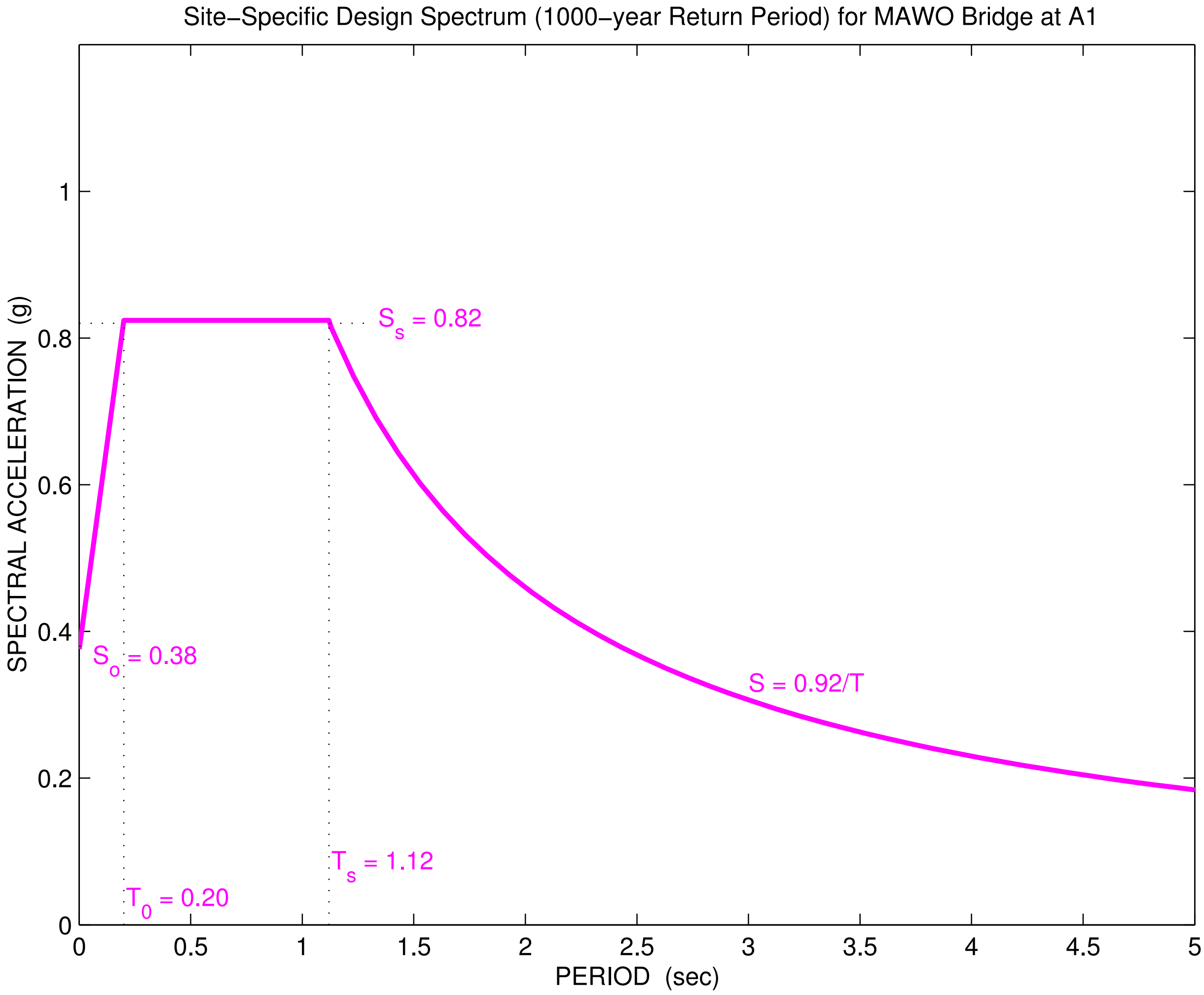


Figure 2B-135 Site-specific design spectrum at 1000-year return period for Mawo Bridge at A1 site

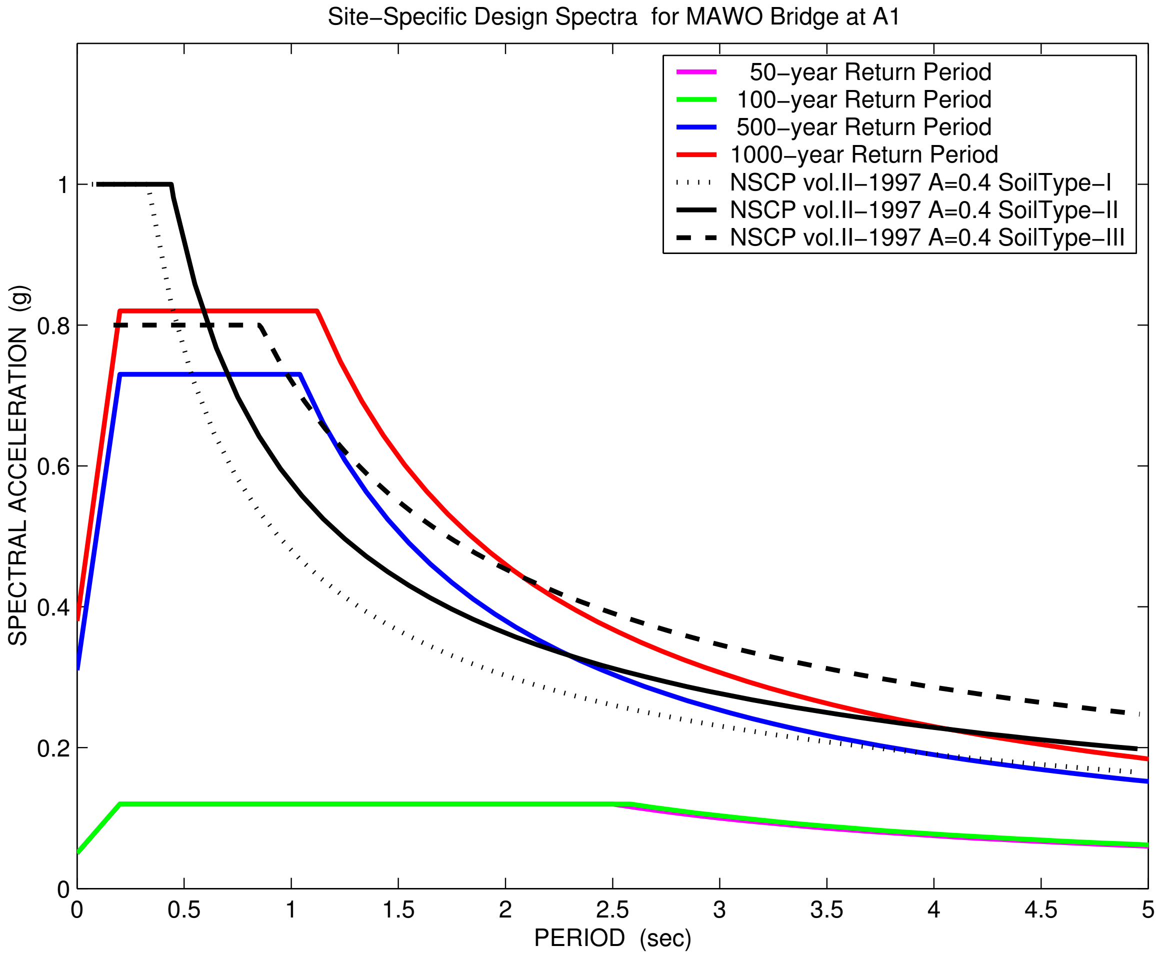


Figure 2B-136 Site-specific design spectra for Mawo Bridge at A1

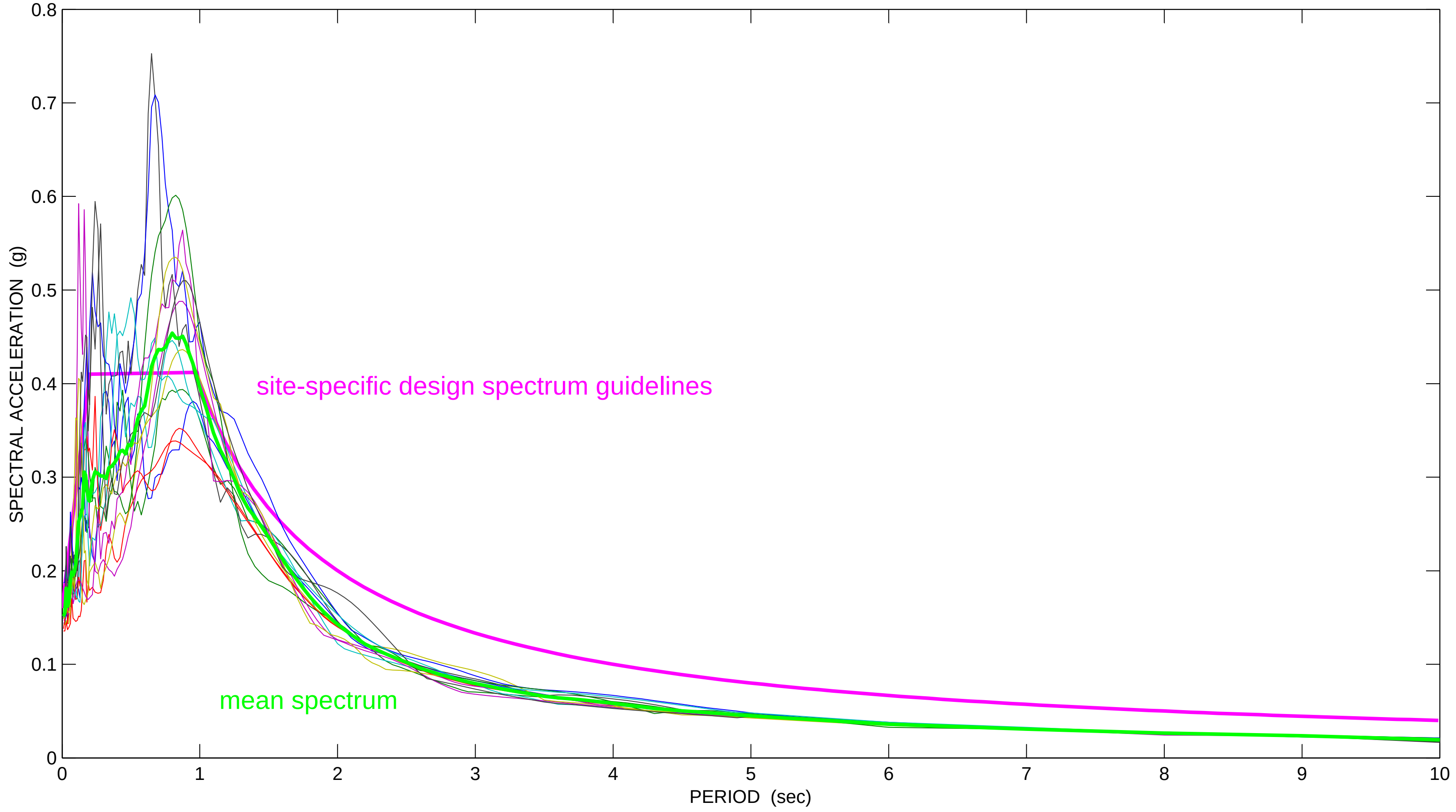


Figure 2B-137 Construction of site-specific design spectrum at 50-year return period for Mawo B2 based on mean of 14 response spectra from nonlinear site response analysis

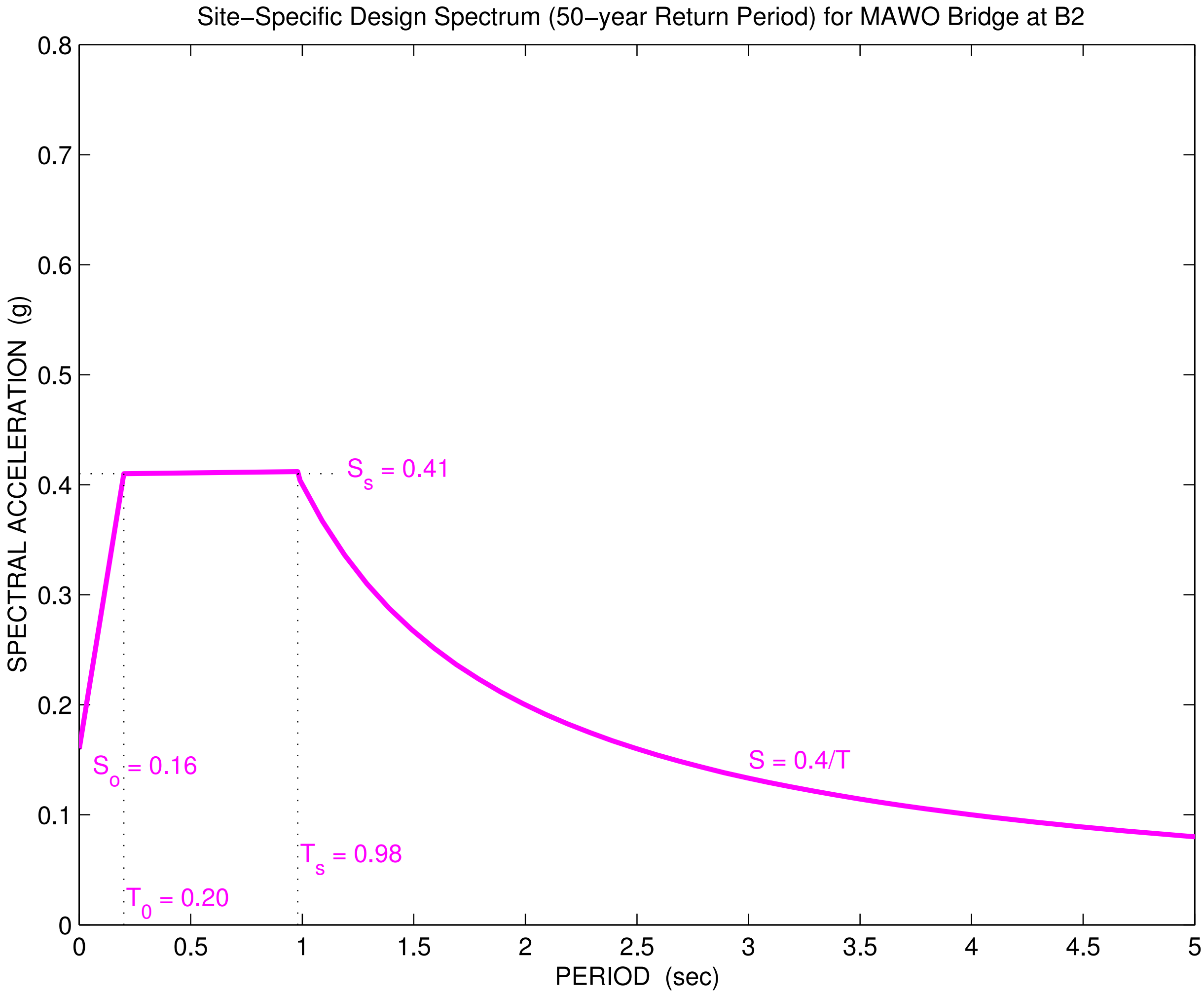


Figure 2B-138 Site-specific design spectrum at 50-year return period for Mawo Bridge at B2 site

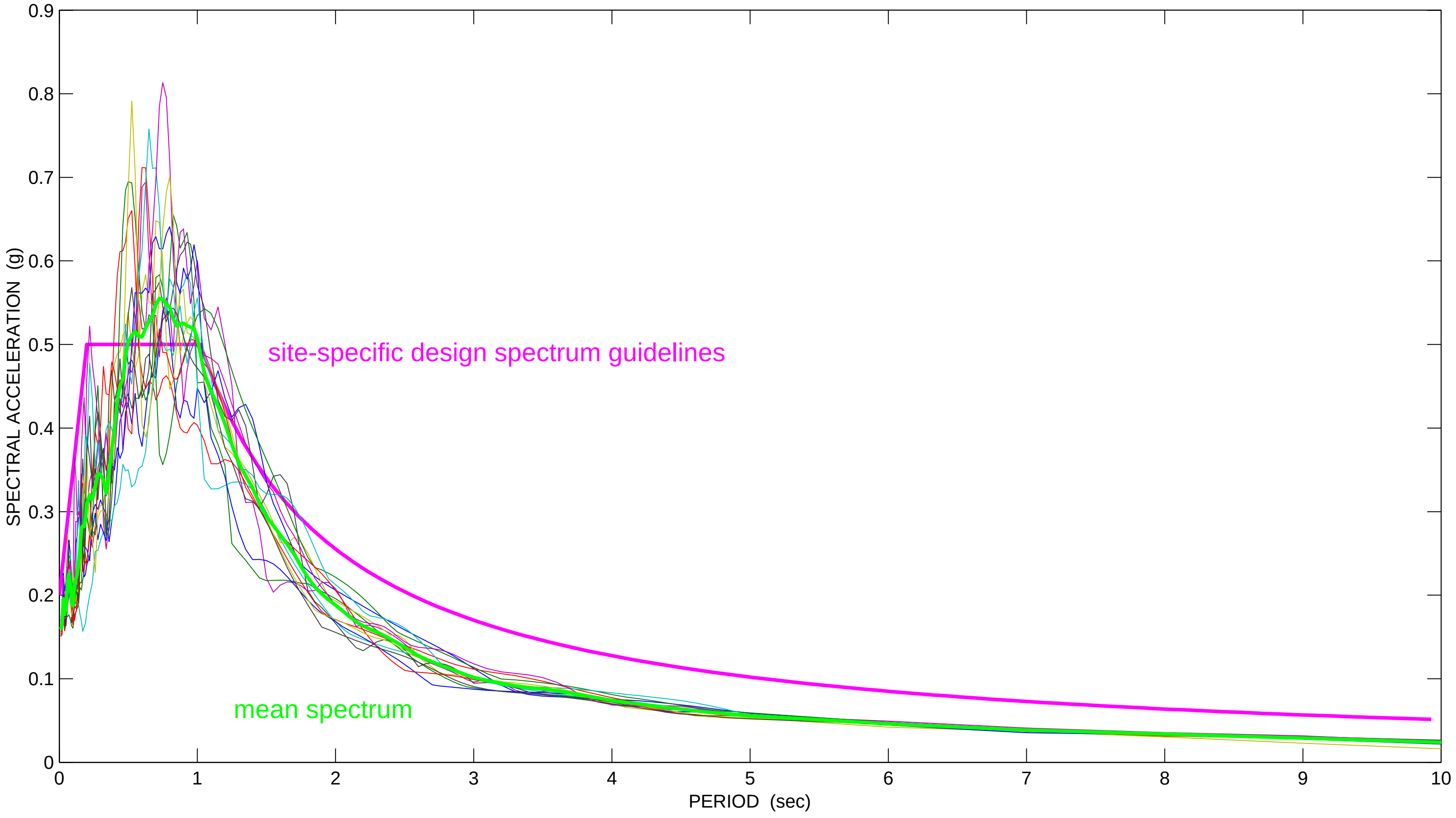


Figure 2B-139 Construction of site-specific design spectrum at 100-year return period for Mawo B2 based on mean of 14 response spectra from nonlinear site response analysis

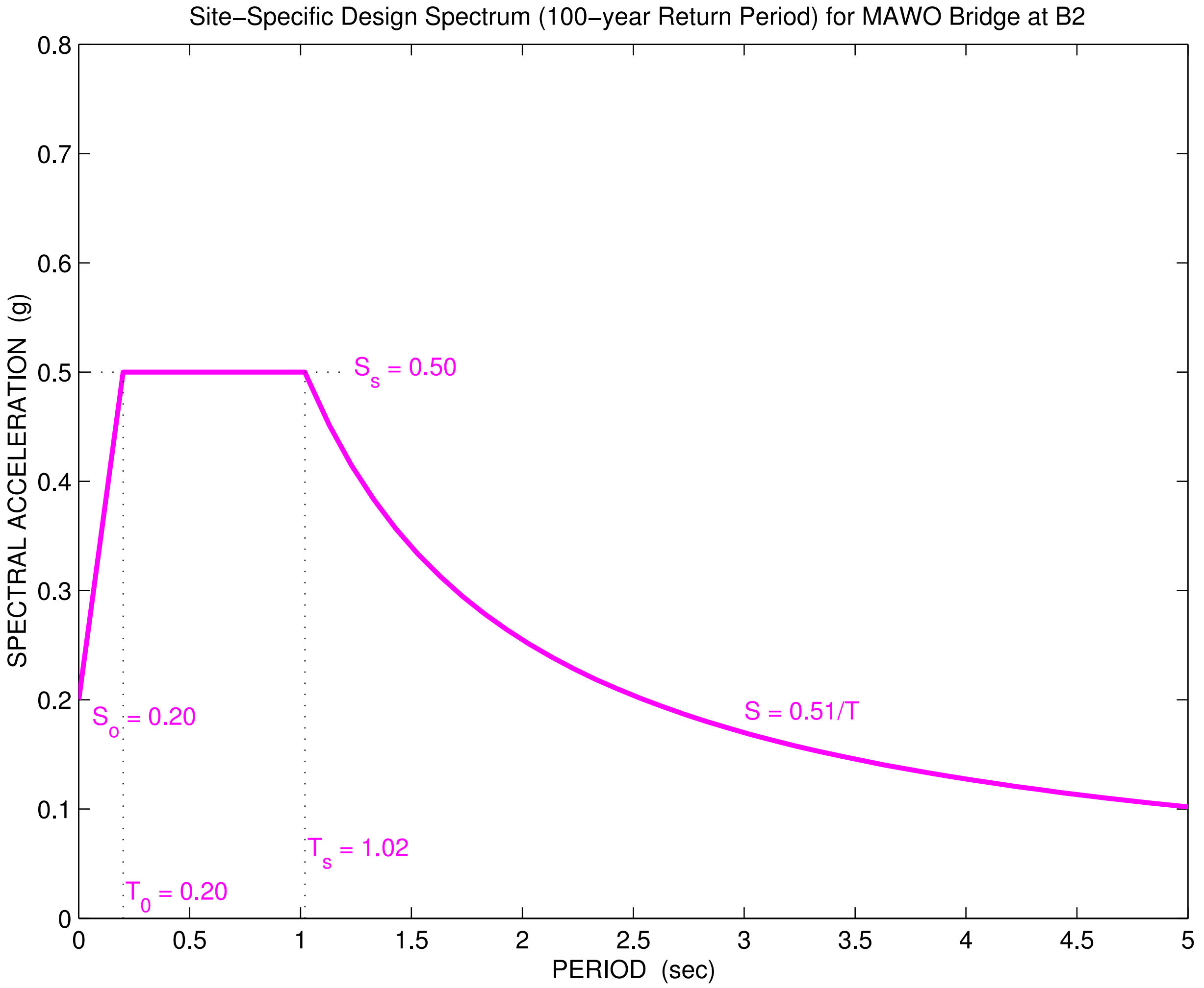


Figure 2B-140 Site-specific design spectrum at 100-year return period for Mawo Bridge at B2 site

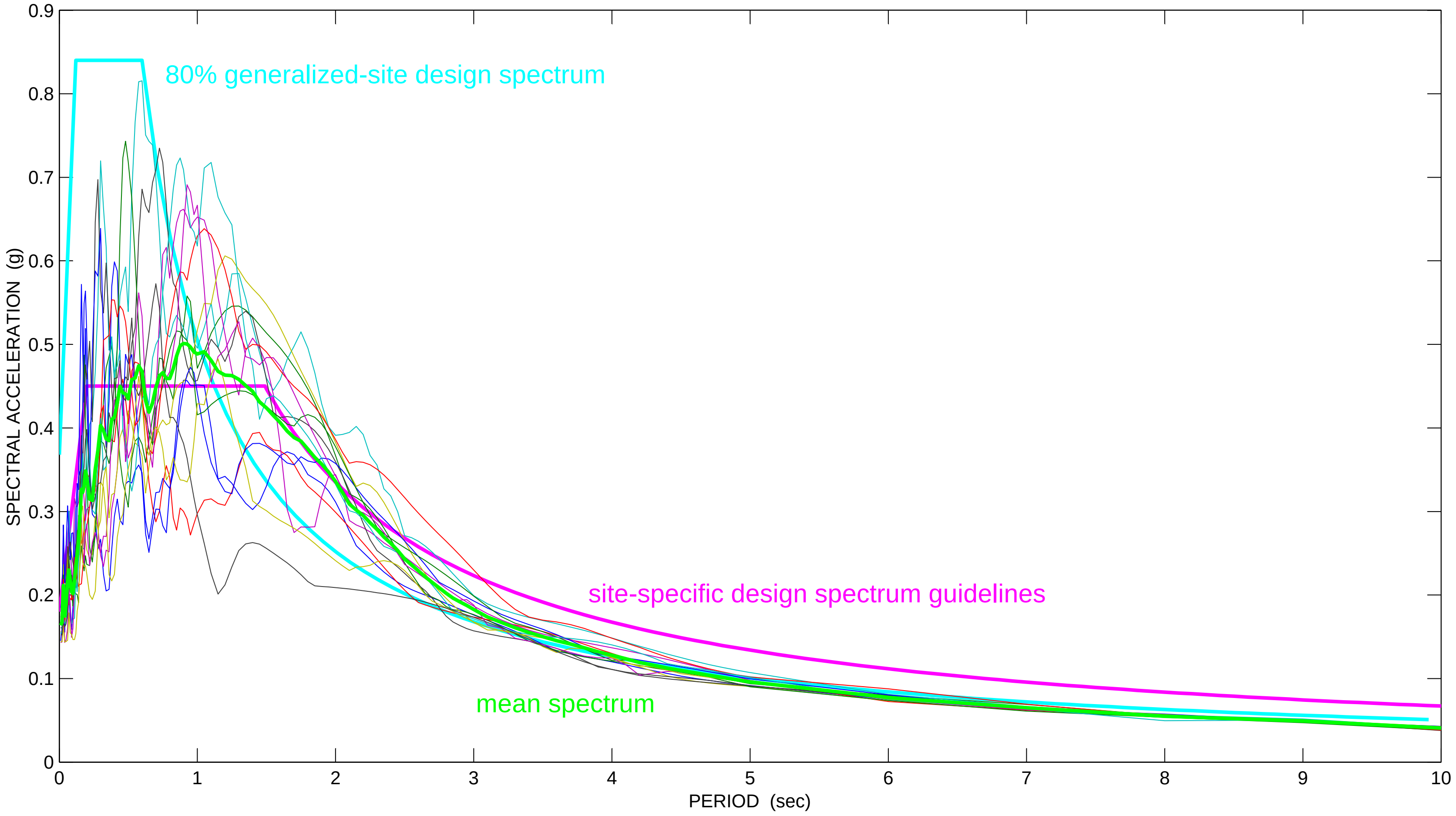


Figure 2B-141 Construction of site-specific design spectrum at 500-year return period for Mawo B2 based on mean of 14 response spectra from nonlinear site response analysis

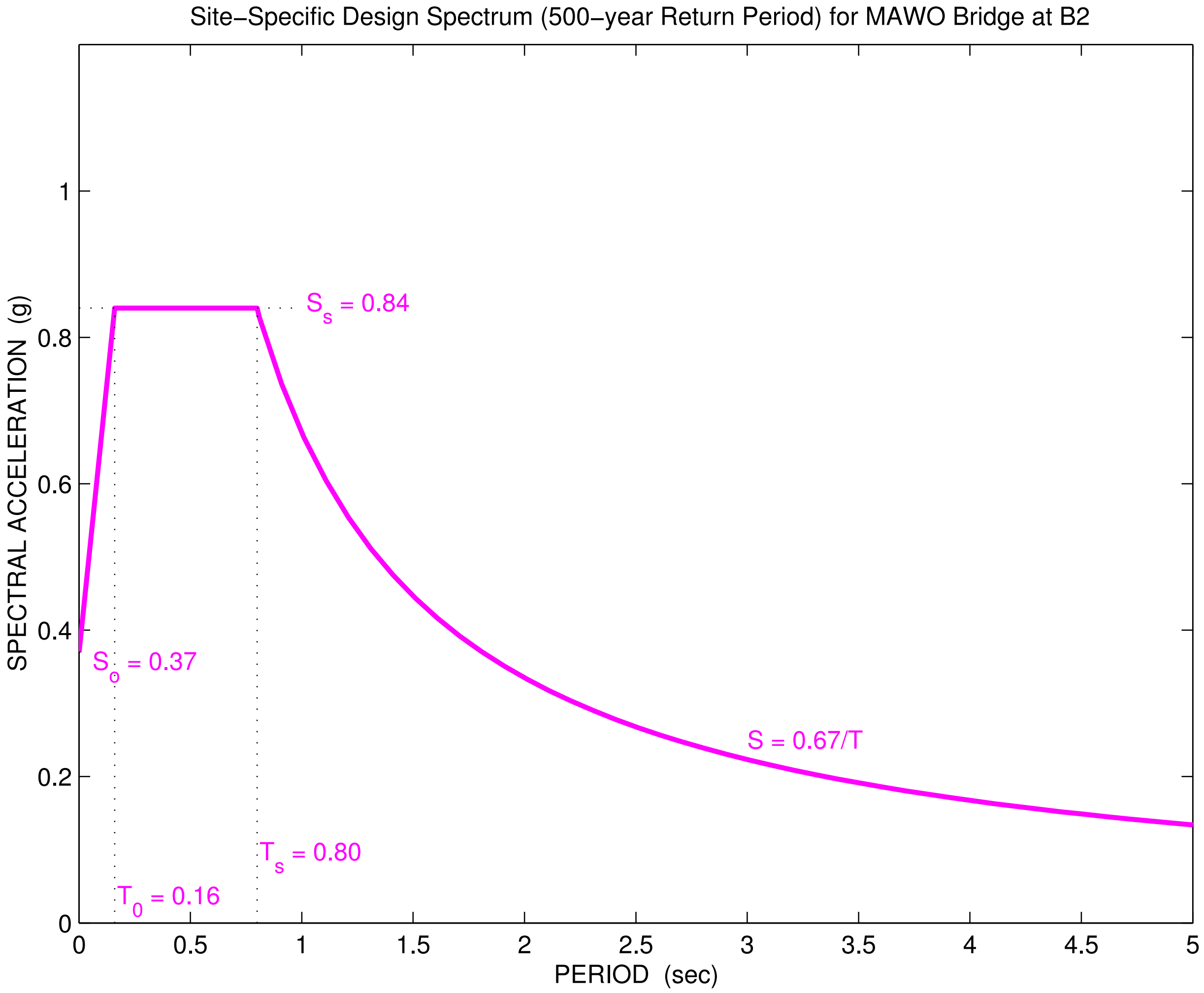


Figure 2B-142 Site-specific design spectrum at 500-year return period for Mawo Bridge at B2 site

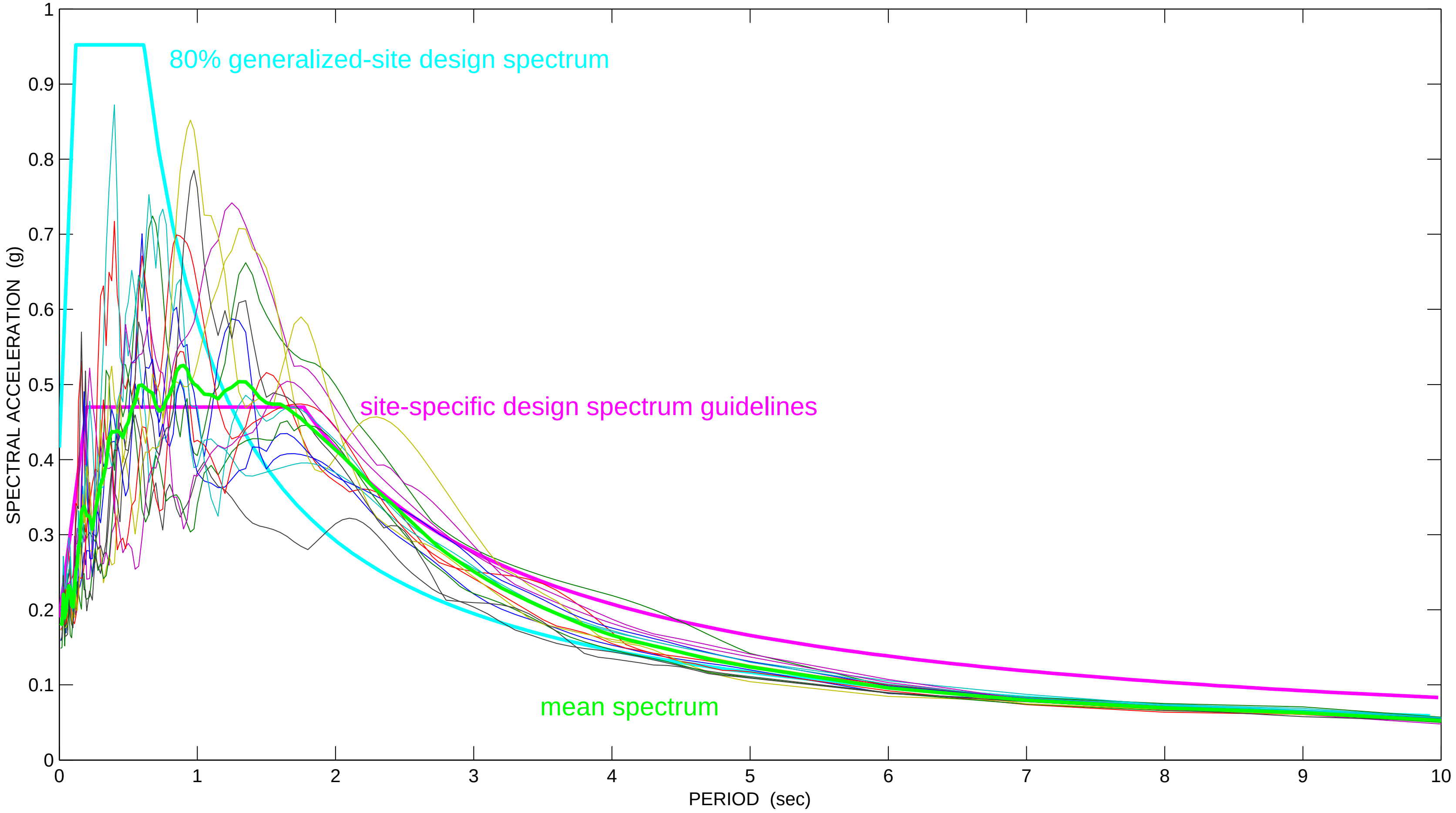


Figure 2B-143 Construction of site-specific design spectrum at 1000-year return period for Mawo B2 based on mean of 14 response spectra from nonlinear site response analysis

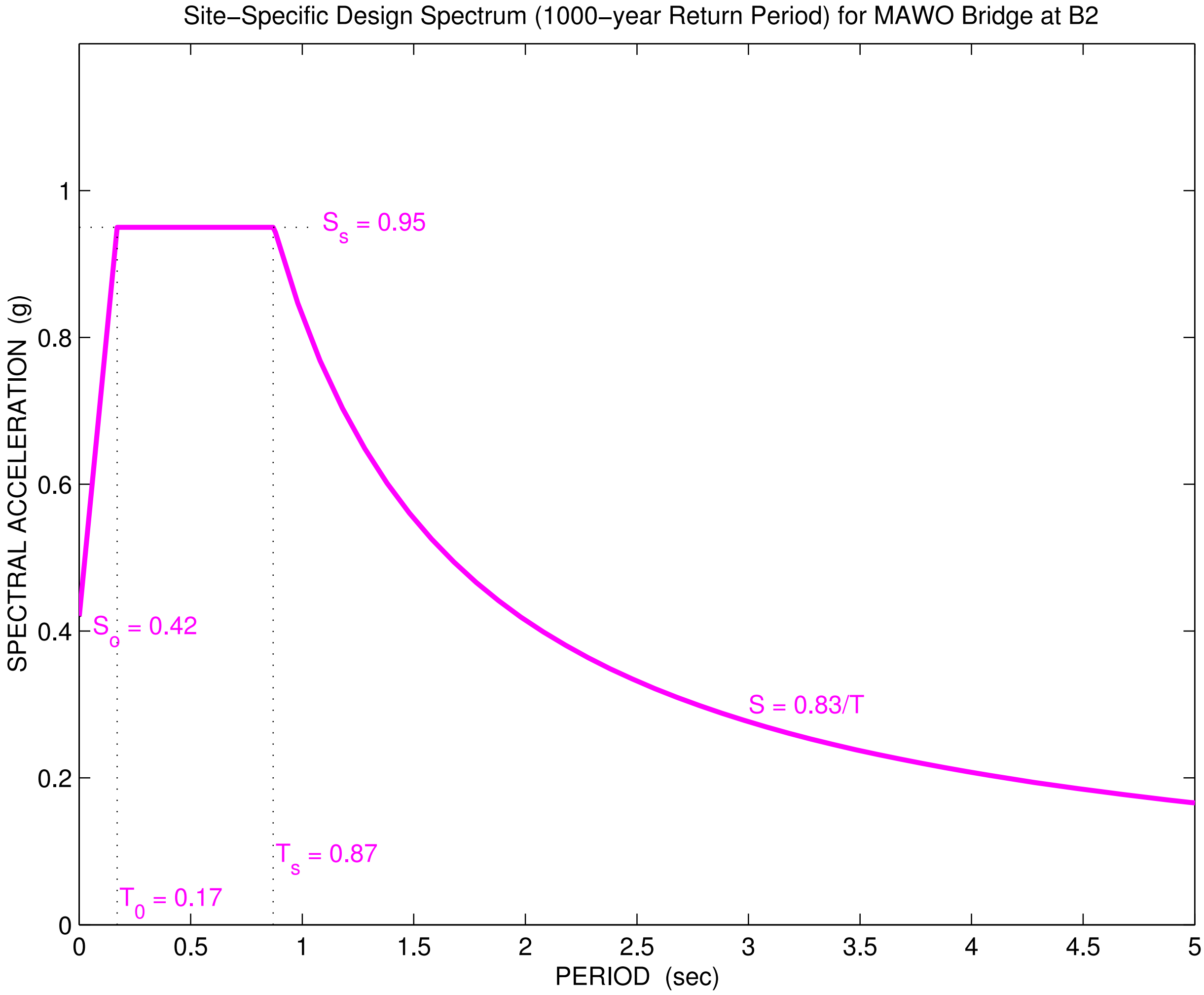


Figure 2B-144 Site-specific design spectrum at 1000-year return period for Mawo Bridge at B2 site

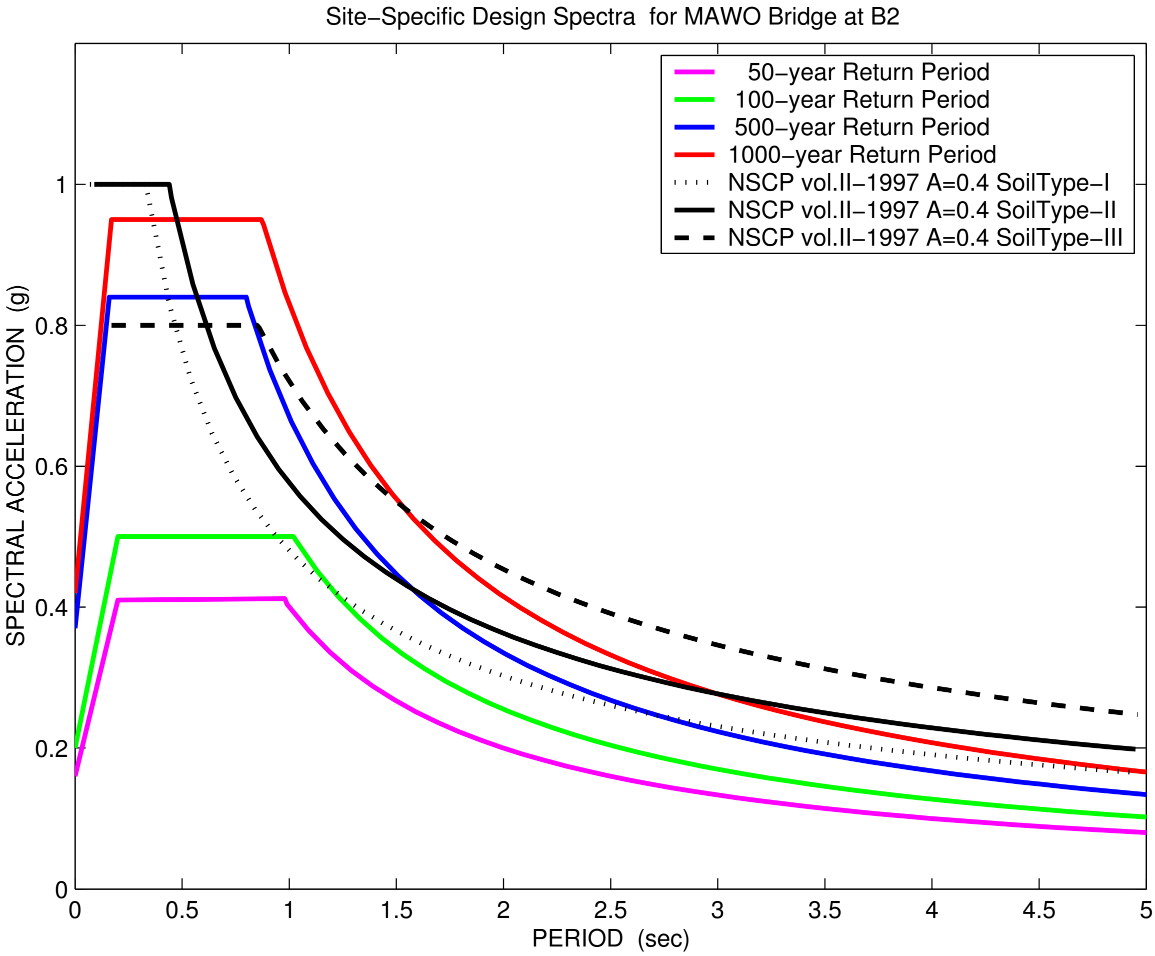
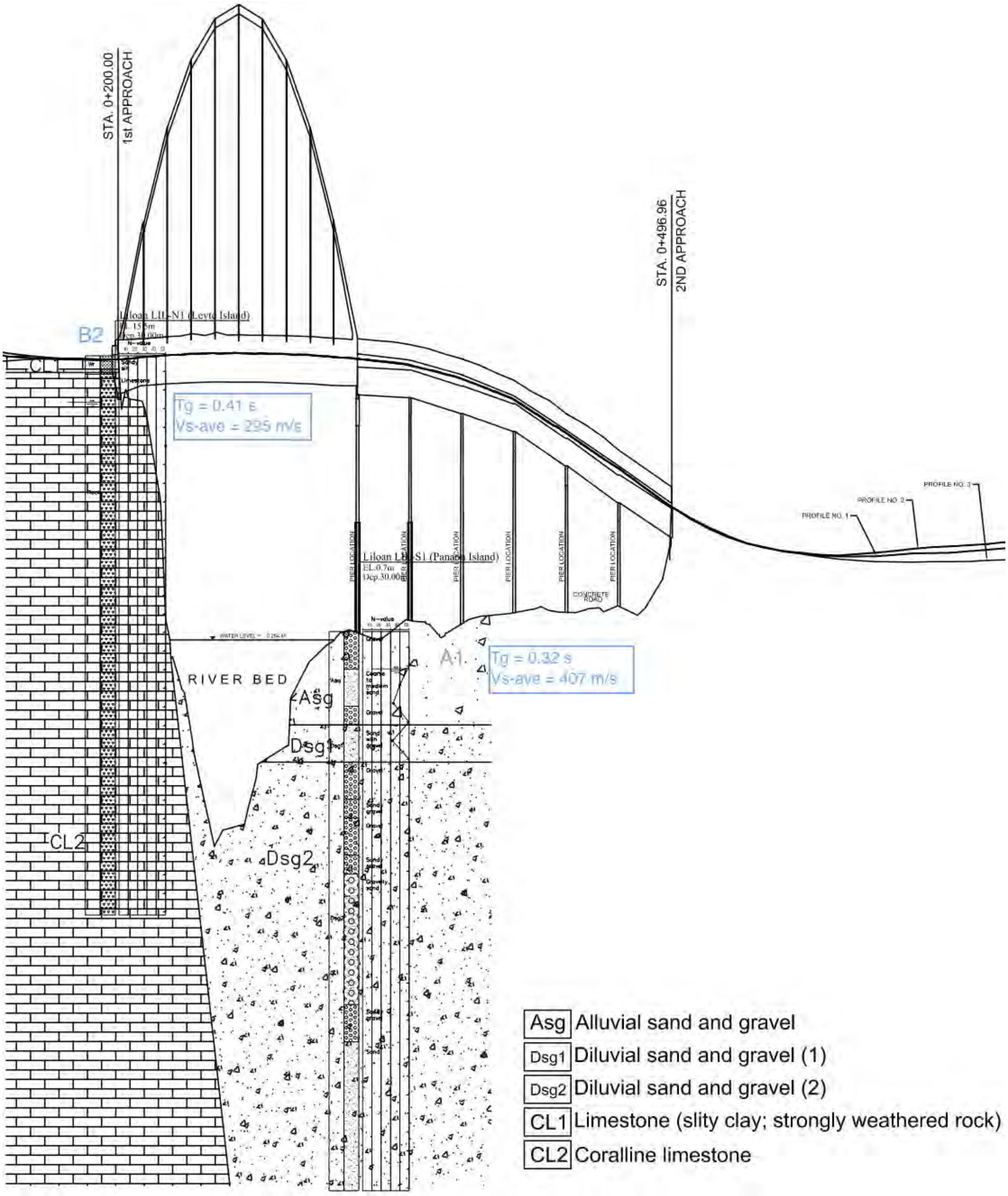


Figure 2B-145 Site-specific design spectra for Mawo Bridge at B2



(c) soil profile

Figure 2B-146 Liloan Bridge site data

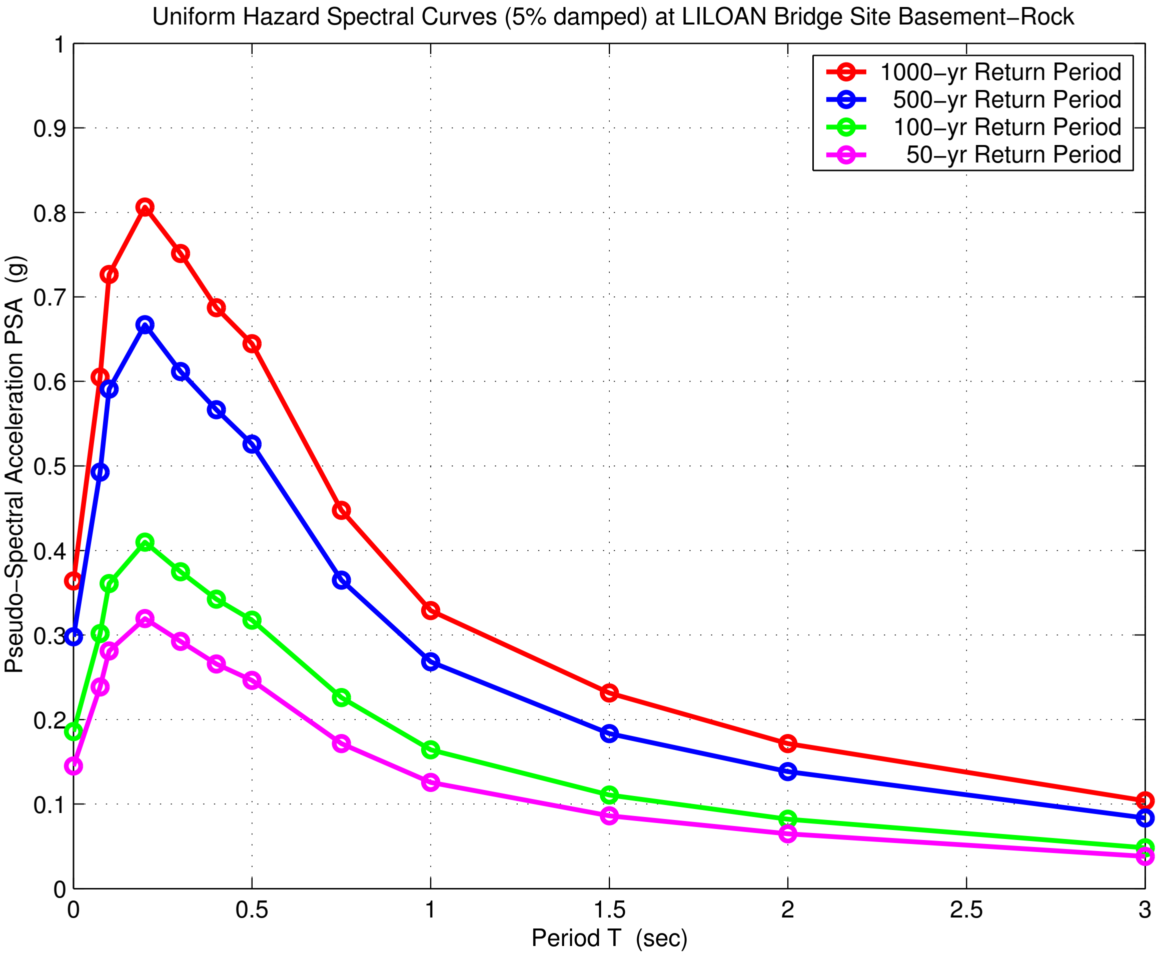


Figure 2B-147 Uniform hazard spectral curve for Liloan Bridge site at basement-rock

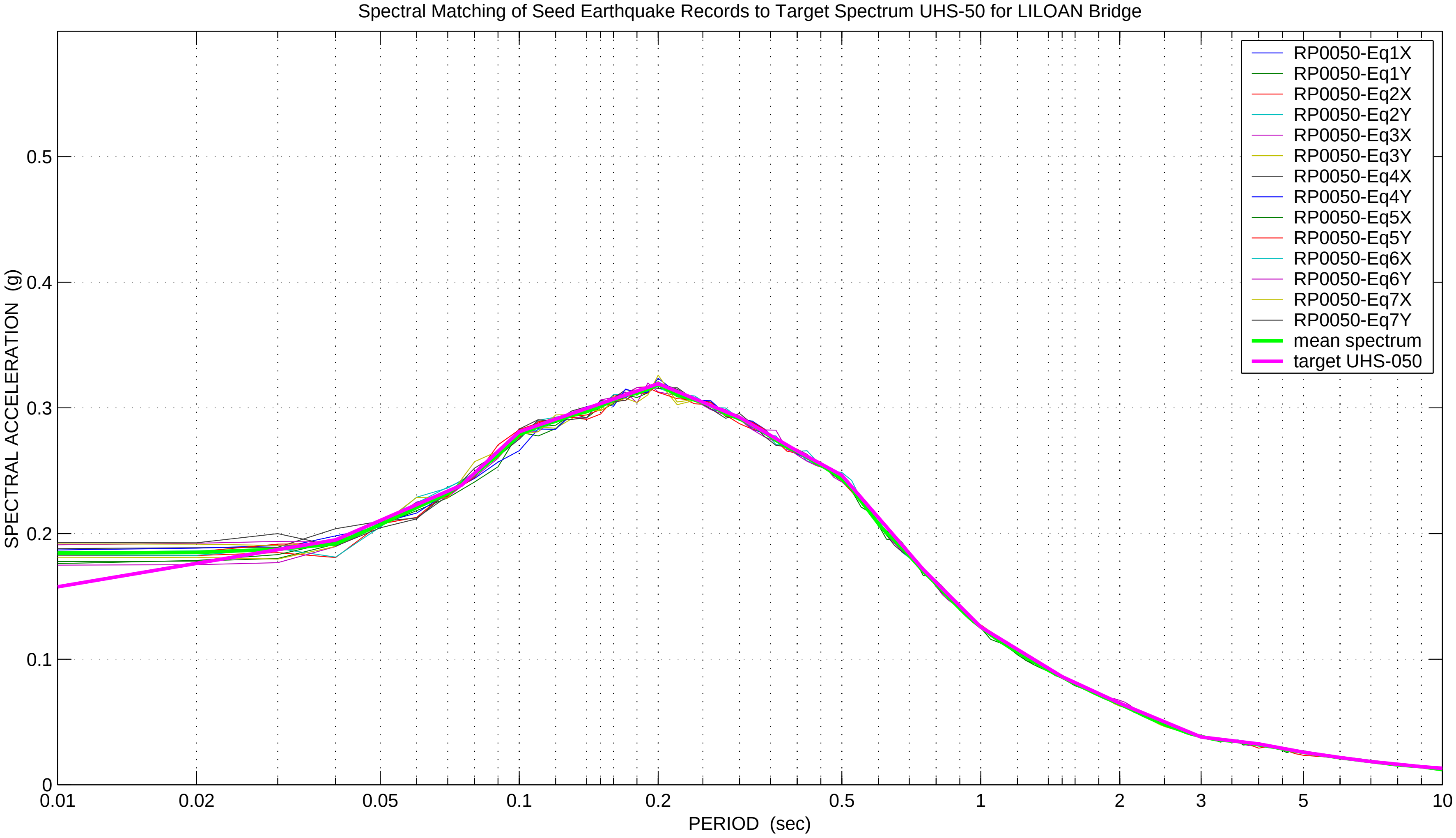


Figure 2B-148 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 50-year return period as compared to the target spectrum UHS-50 at Liloan Bridge

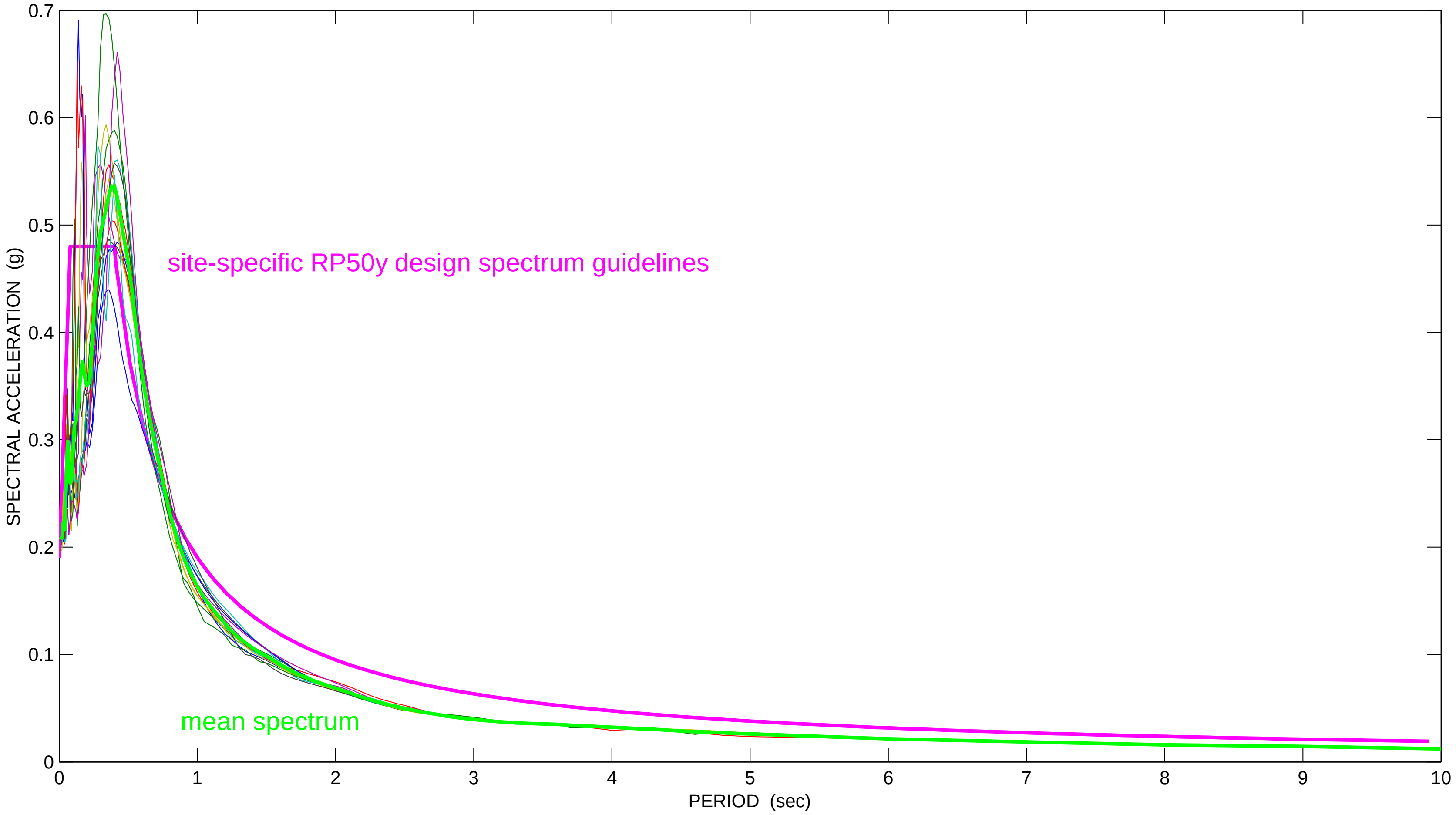


Figure 2B-149 Construction of site-specific design spectrum at 50-year return period for Liloan A1 based on mean of 14 response spectra from nonlinear site response analysis

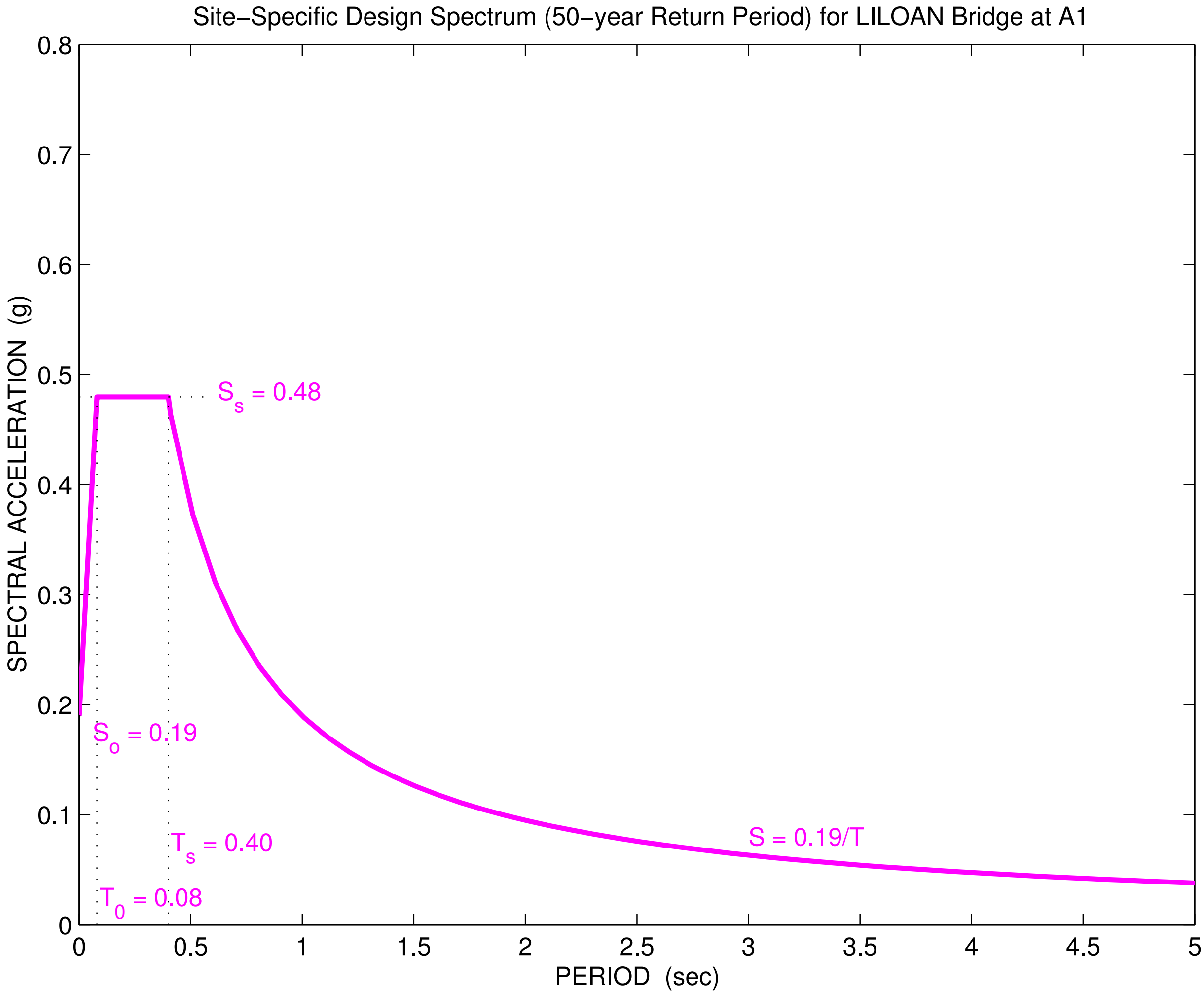


Figure 2B-150 Site-specific design spectrum at 50-year return period for Liloan Bridge at A1 site

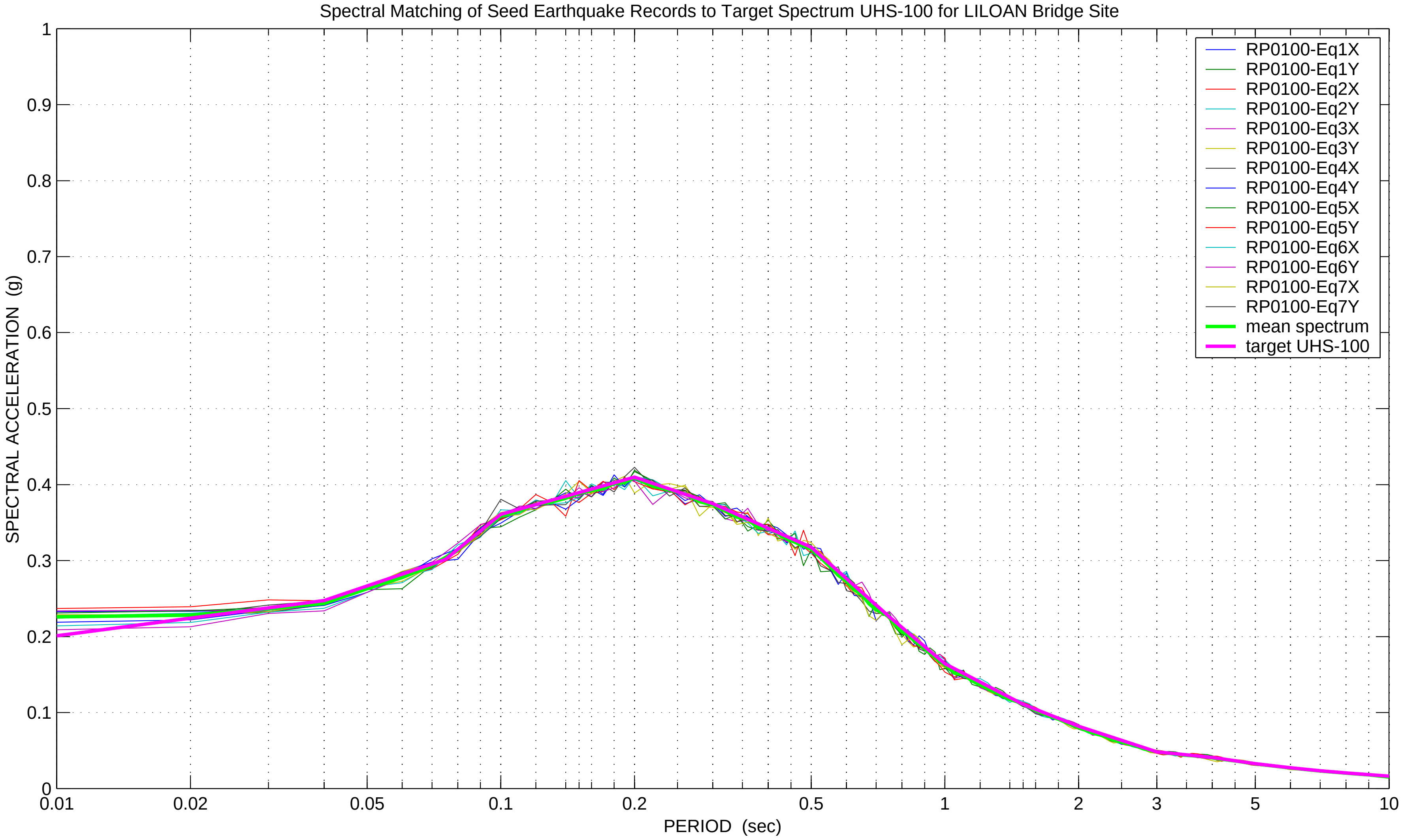


Figure 2B-151 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 100-year return period as compared to the target spectrum UHS-100 at Liloan Bridge

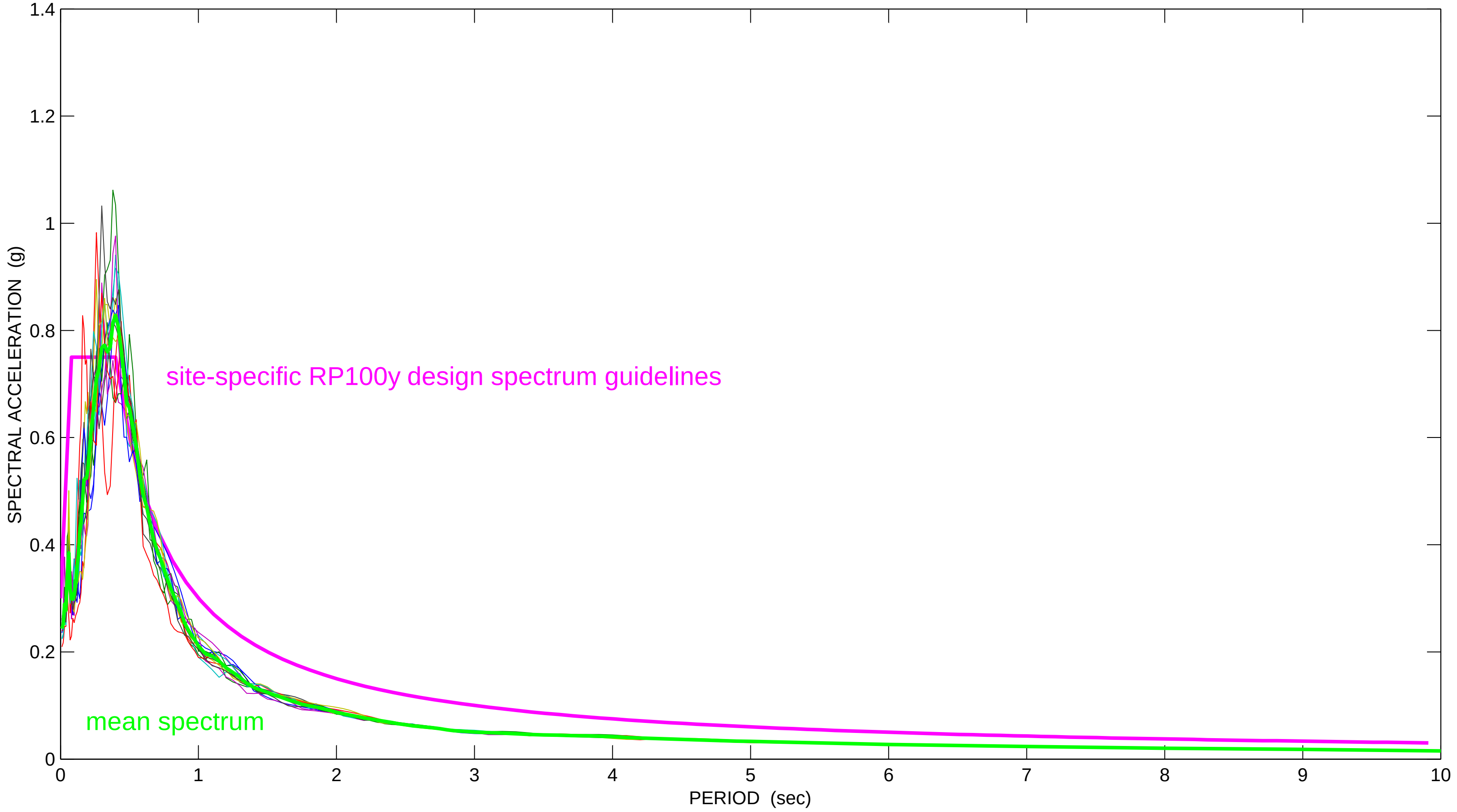


Figure 2B-152 Construction of site-specific design spectrum at 100-year return period for Liloan A1 based on mean of 14 response spectra from nonlinear site response analysis

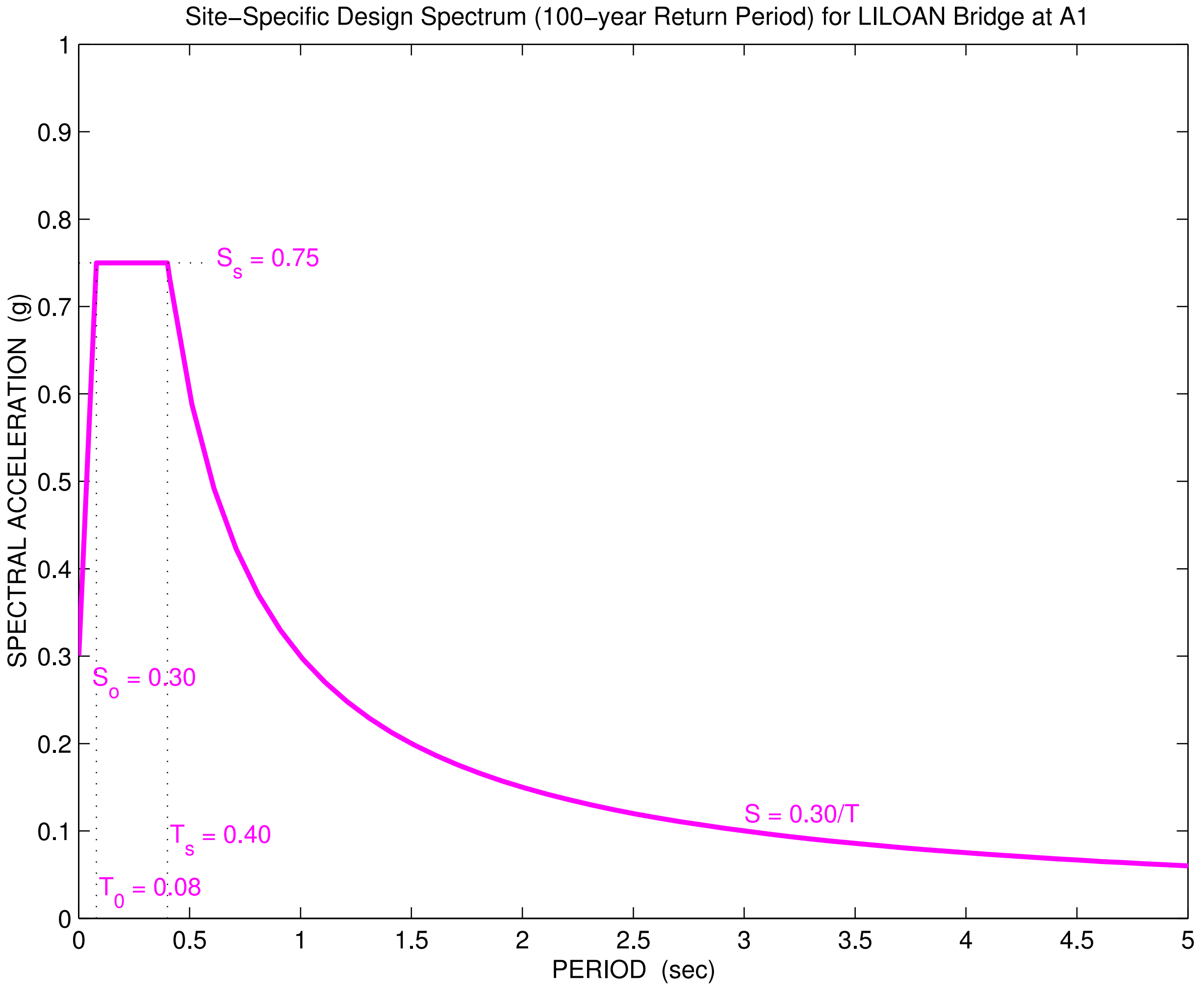


Figure 2B-153 Site-specific design spectrum at 100-year return period for Liloan Bridge at A1 site

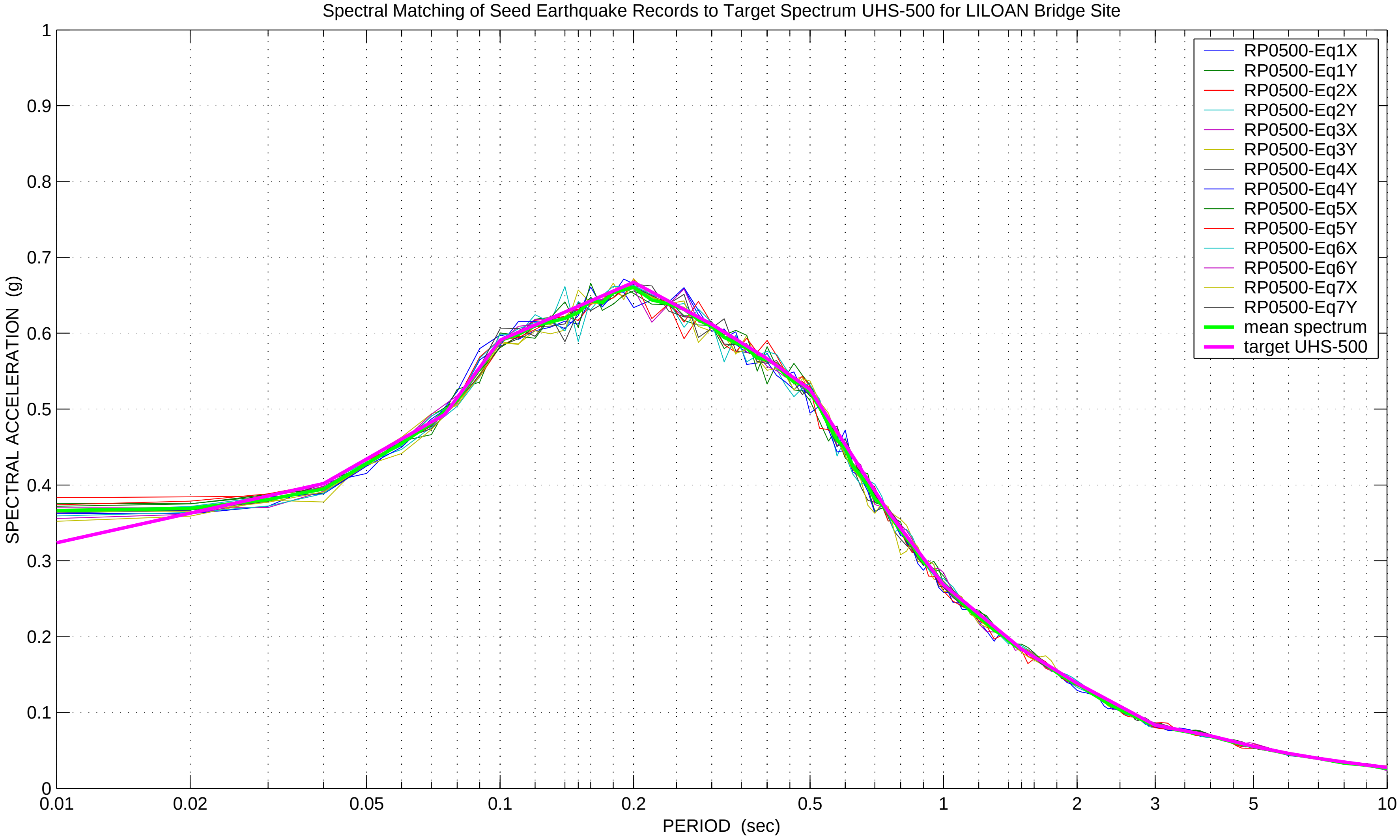


Figure 2B-154 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 500-year return period as compared to the target spectrum UHS-500 at Liloan Bridge

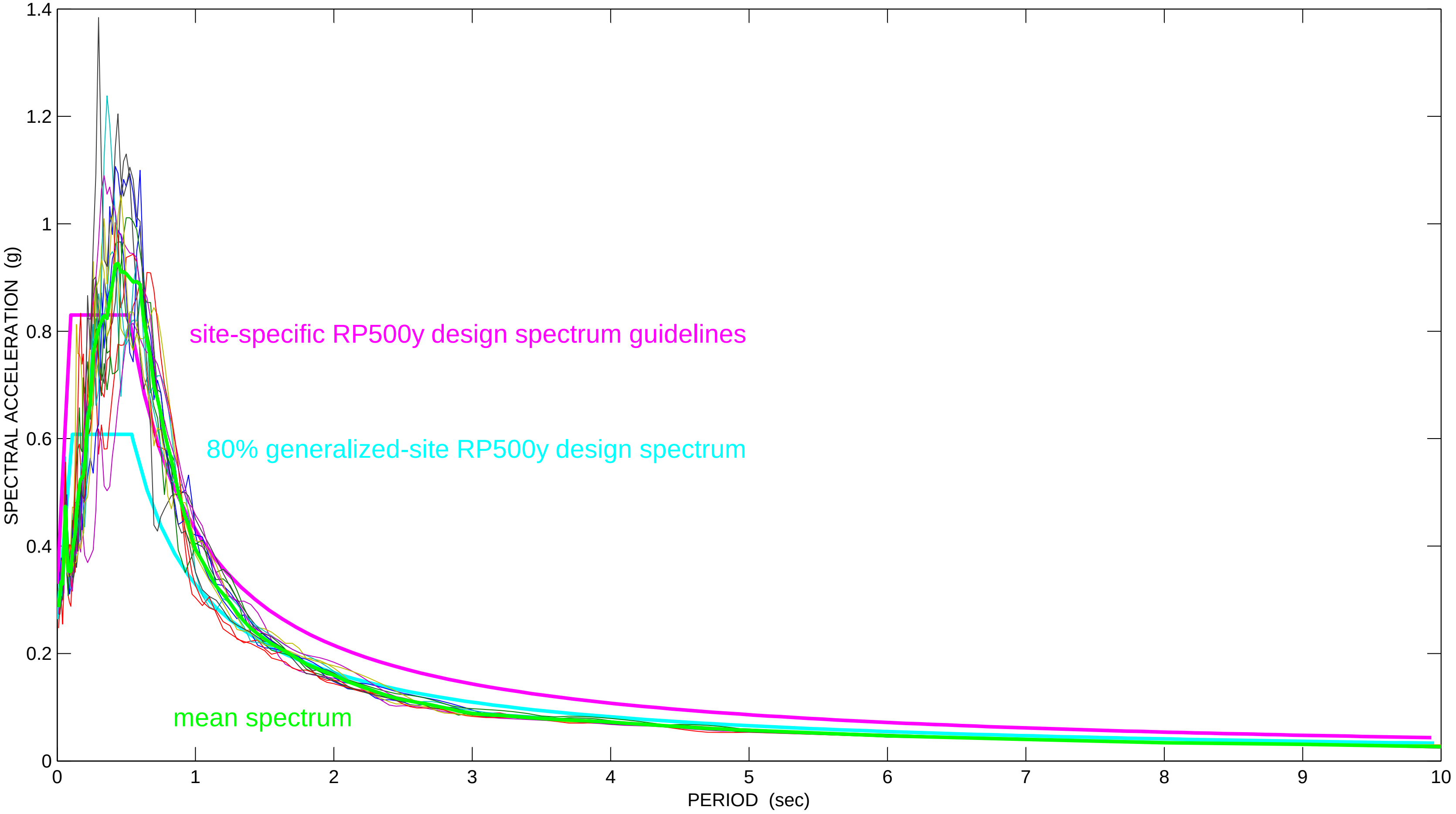


Figure 2B-155 Construction of site-specific design spectrum at 500-year return period for Liloan A1 based on mean of 14 response spectra from nonlinear site response analysis

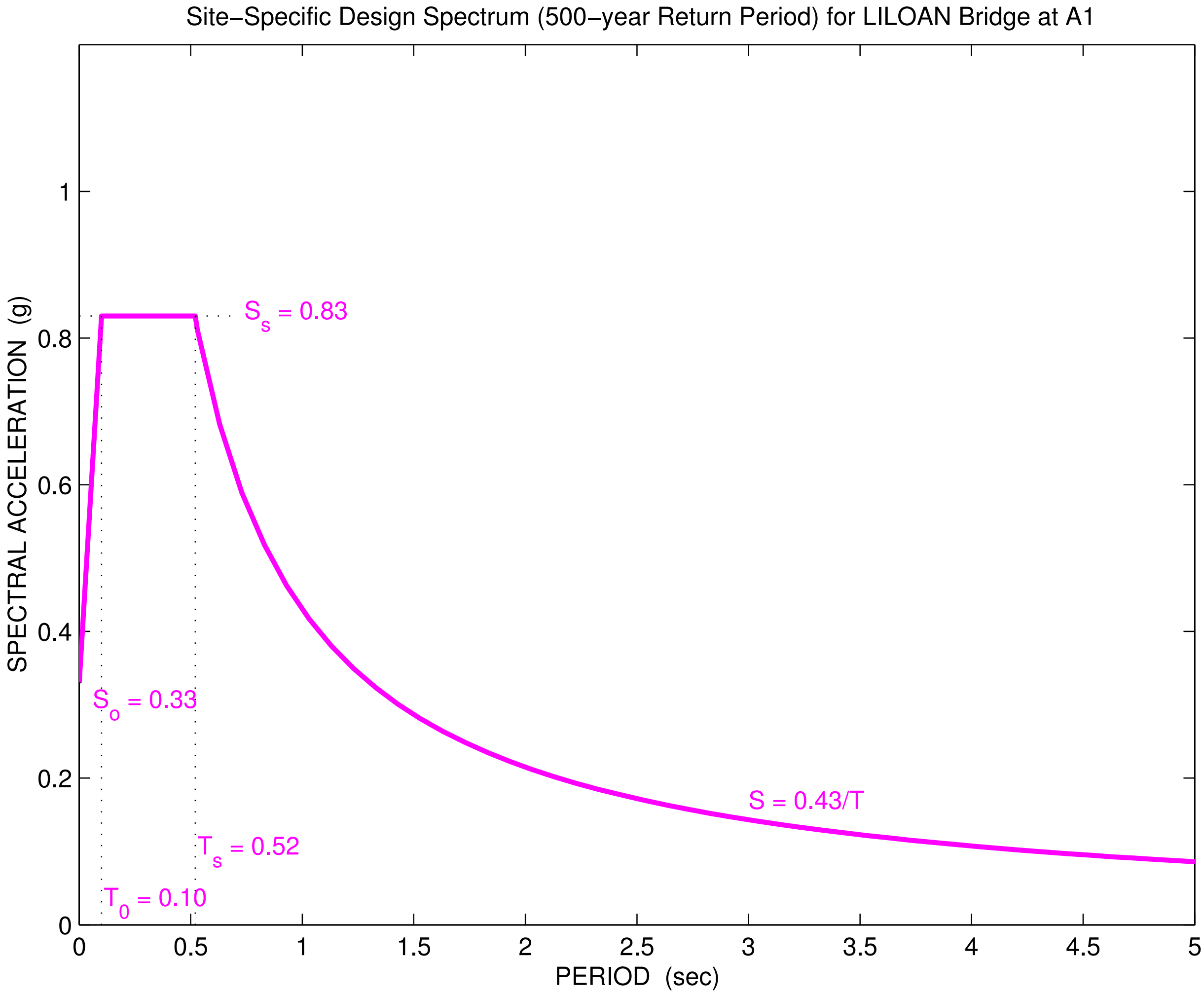


Figure 2B-156 Site-specific design spectrum at 500-year return period for Liloan Bridge at A1 site

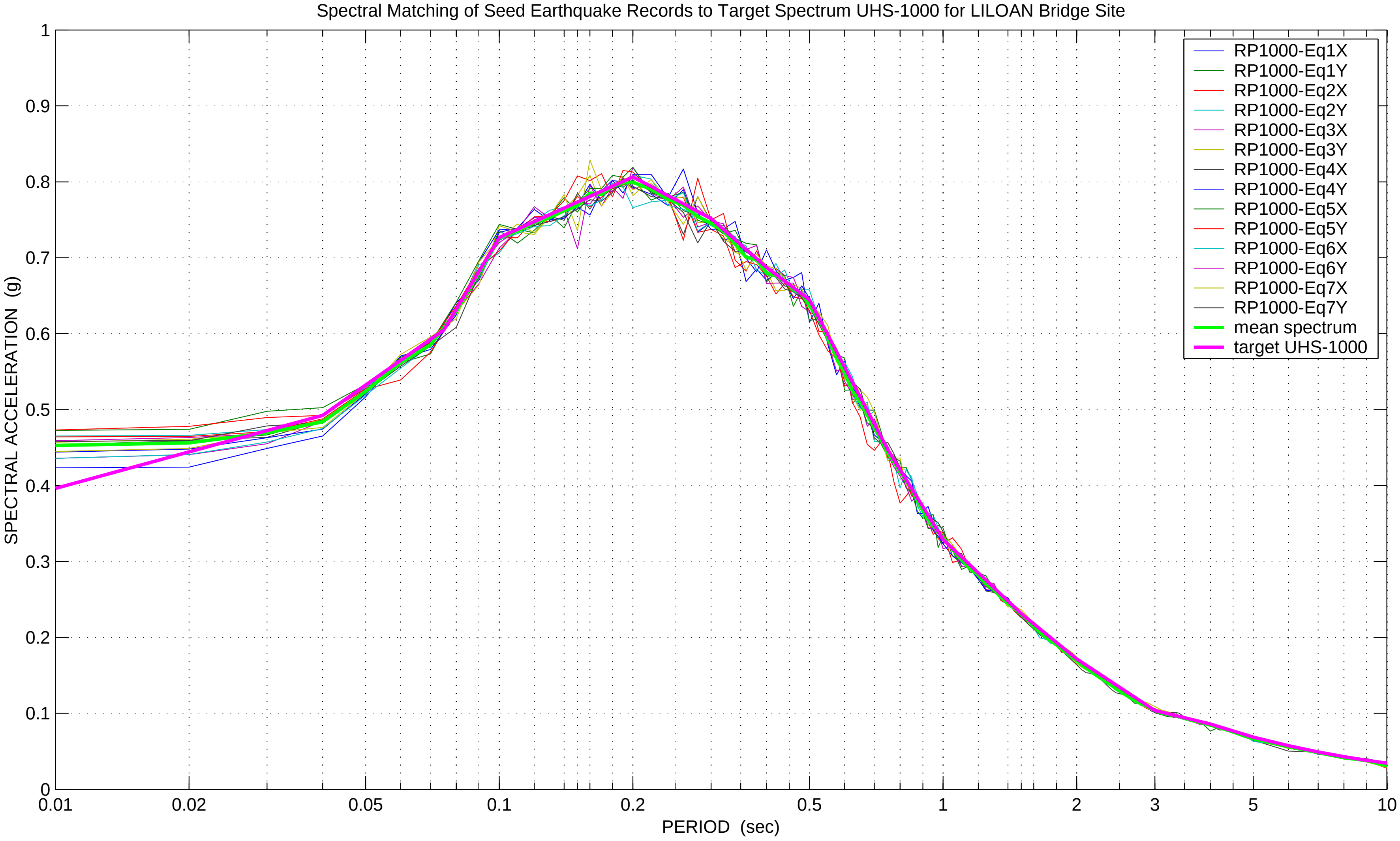


Figure 2B-157 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 1000-year return period as compared to the target spectrum UHS-1000 at Liloan Bridge

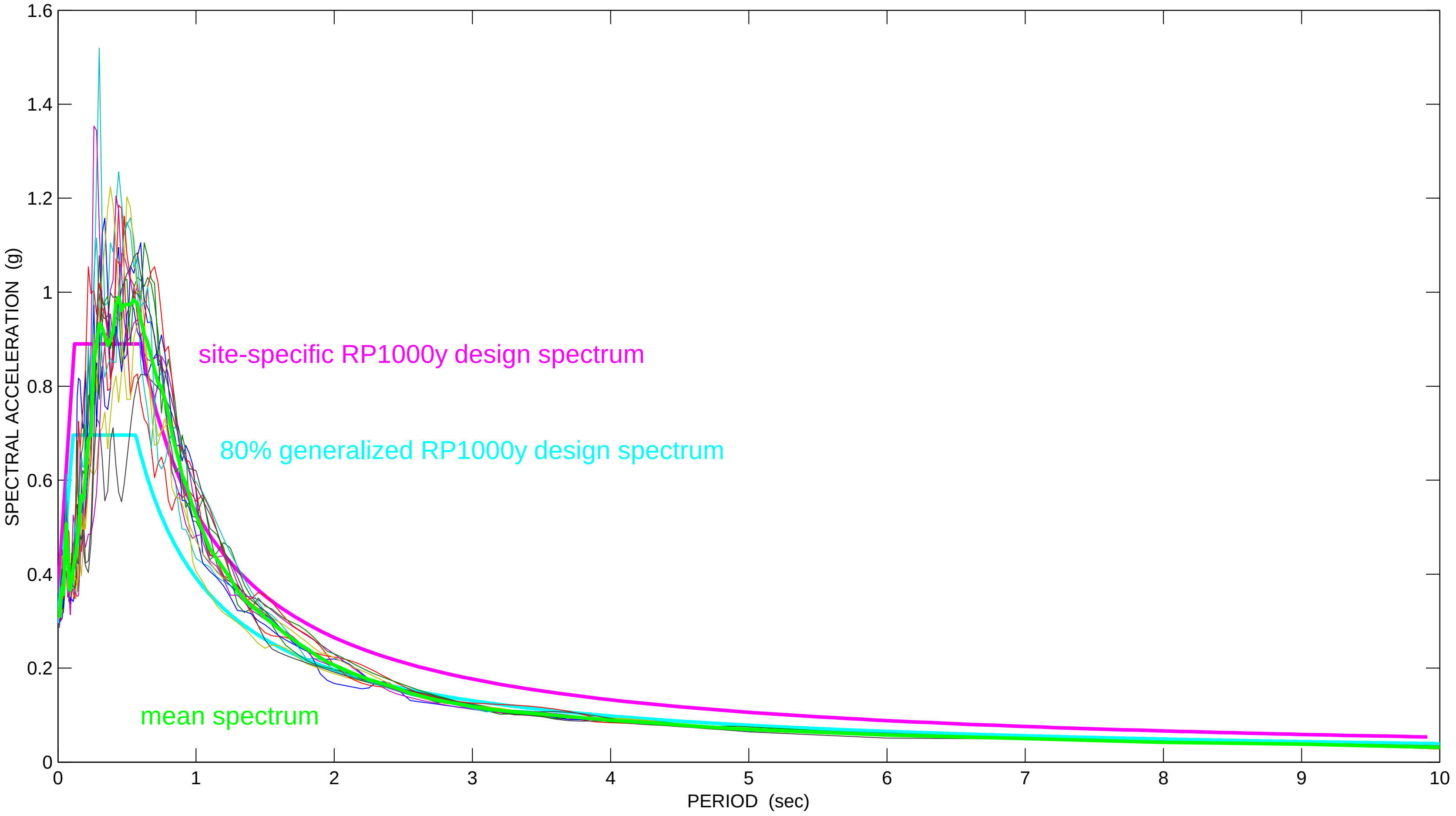


Figure 2B-158 Construction of site-specific design spectrum at 1000-year return period for Liloan A1 based on mean of 14 response spectra from nonlinear site response analysis

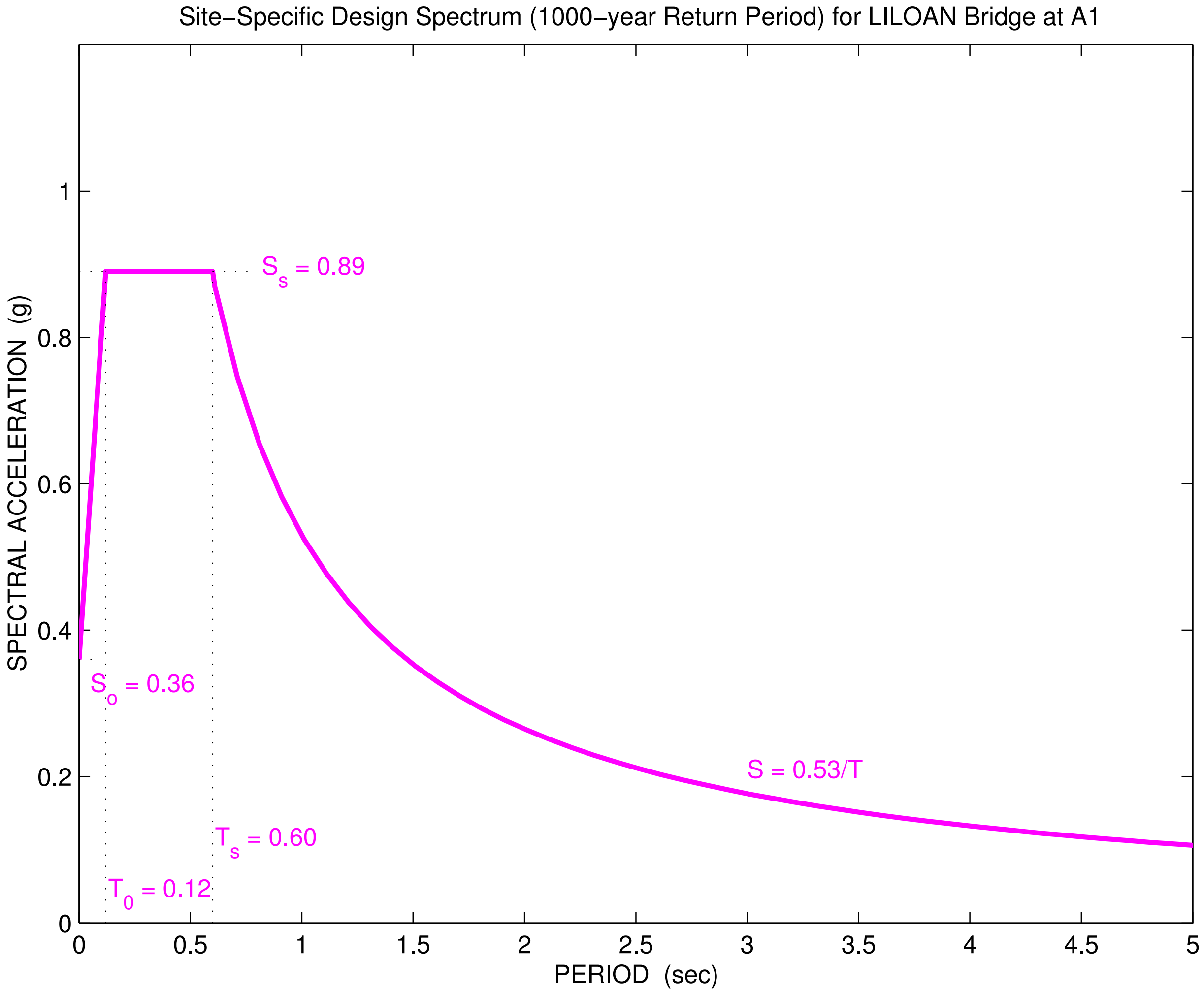


Figure 2B-159 Site-specific design spectrum at 1000-year return period for Liloan Bridge at A1 site

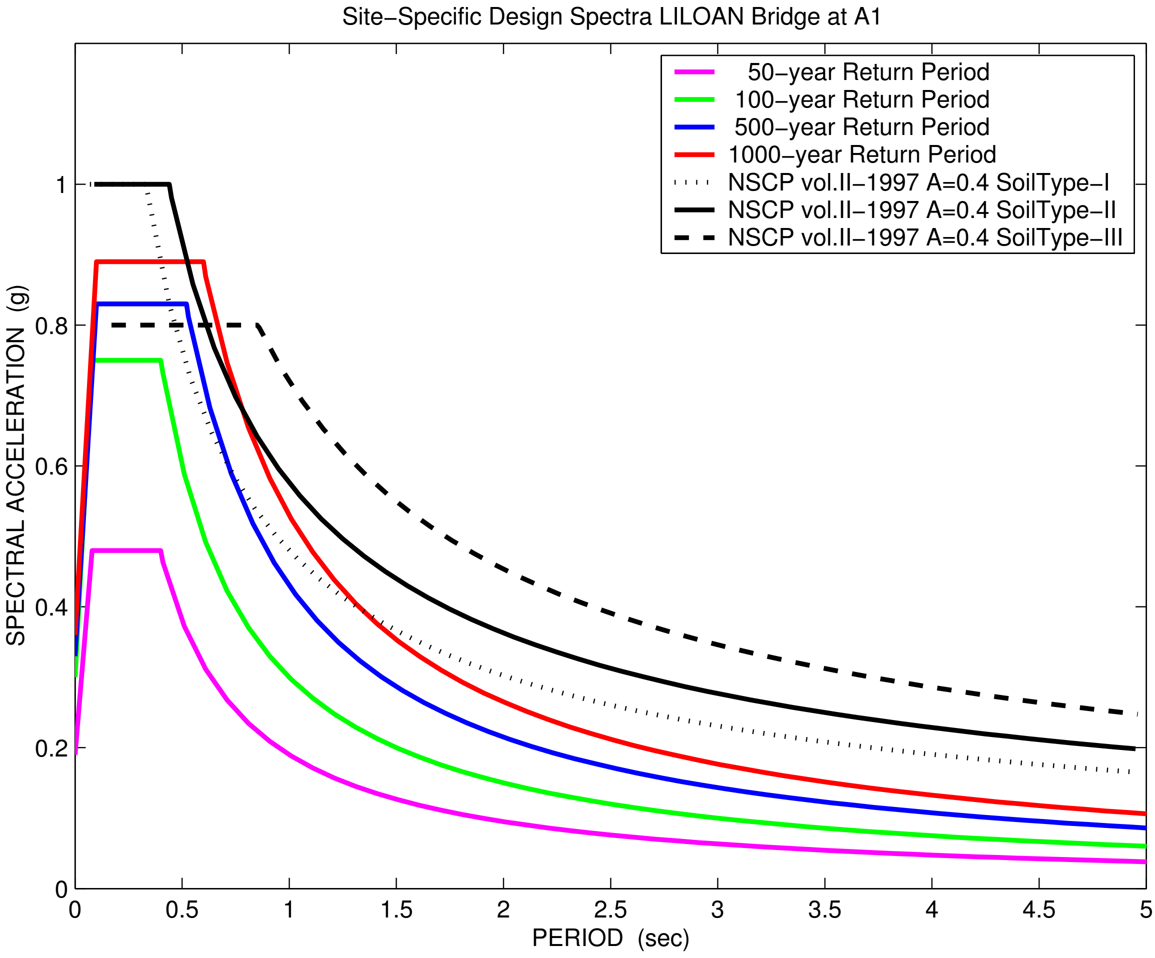
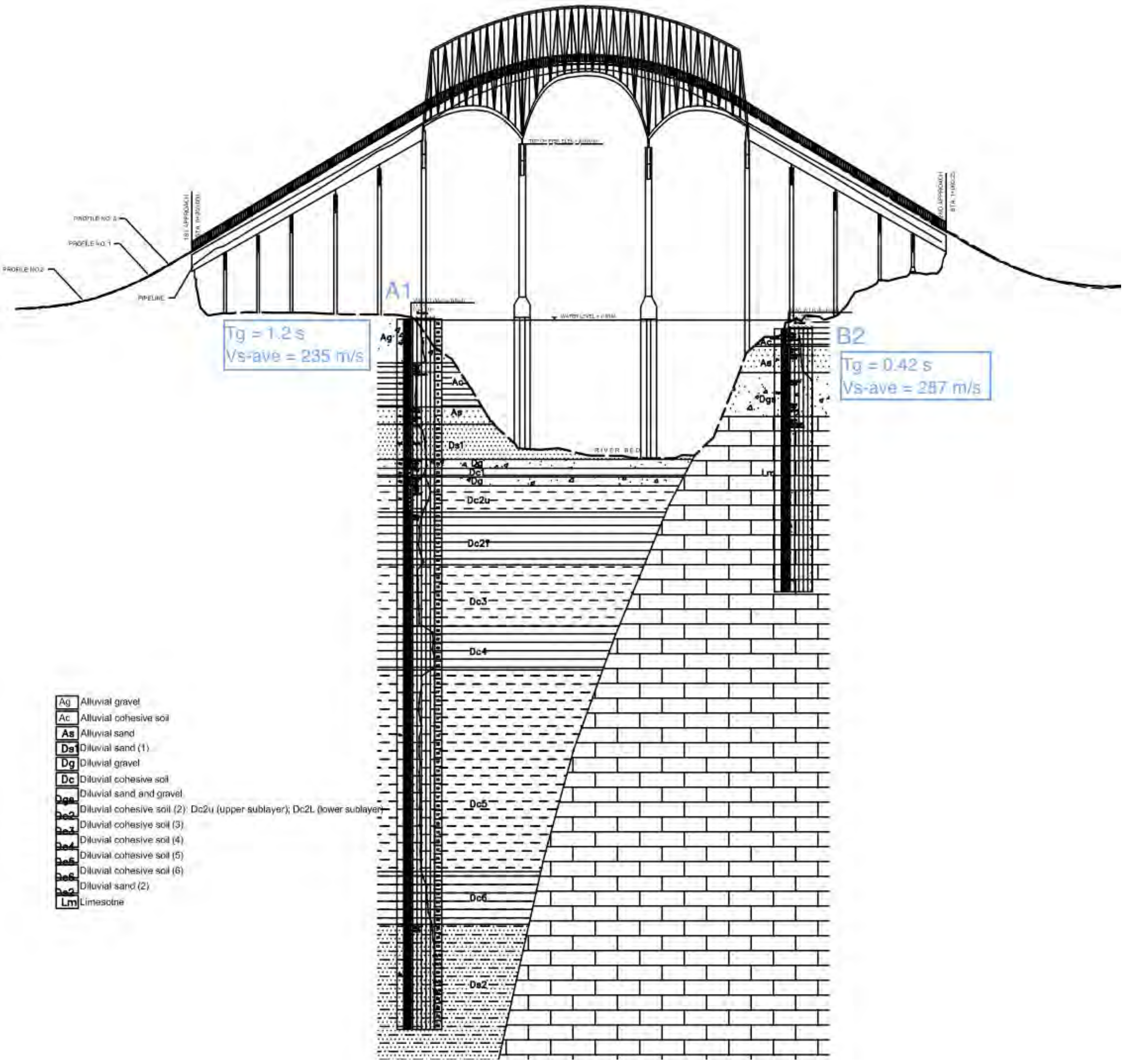


Figure 2B-160 Site-specific design spectra for Liloan Bridge at A1



(c) soil profile

Figure 2B-161 1st Mactan-Mandaue Bridge site data

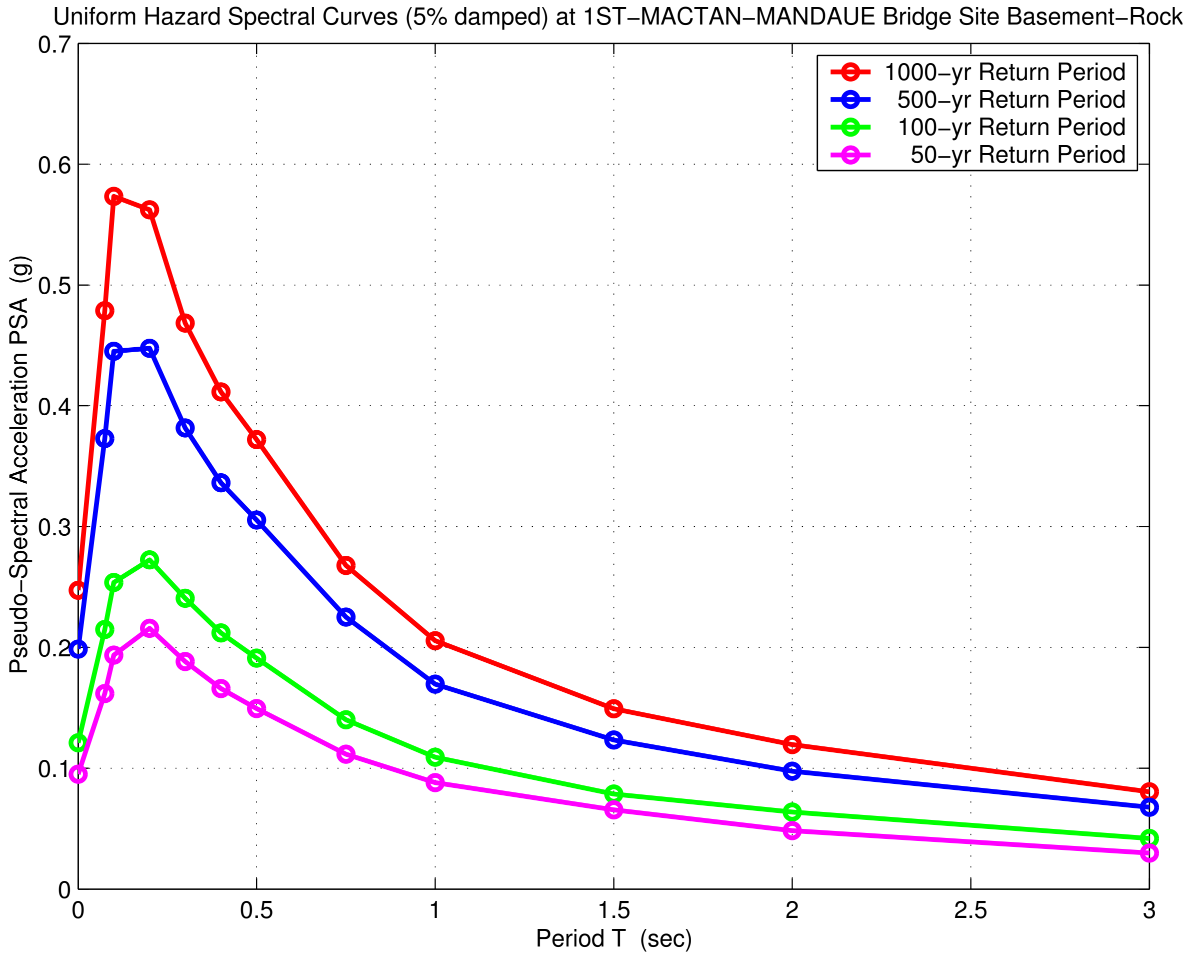


Figure 2B-162 Uniform hazard spectral curve for 1st Mactan-Mandaue Bridge site at basement-rock

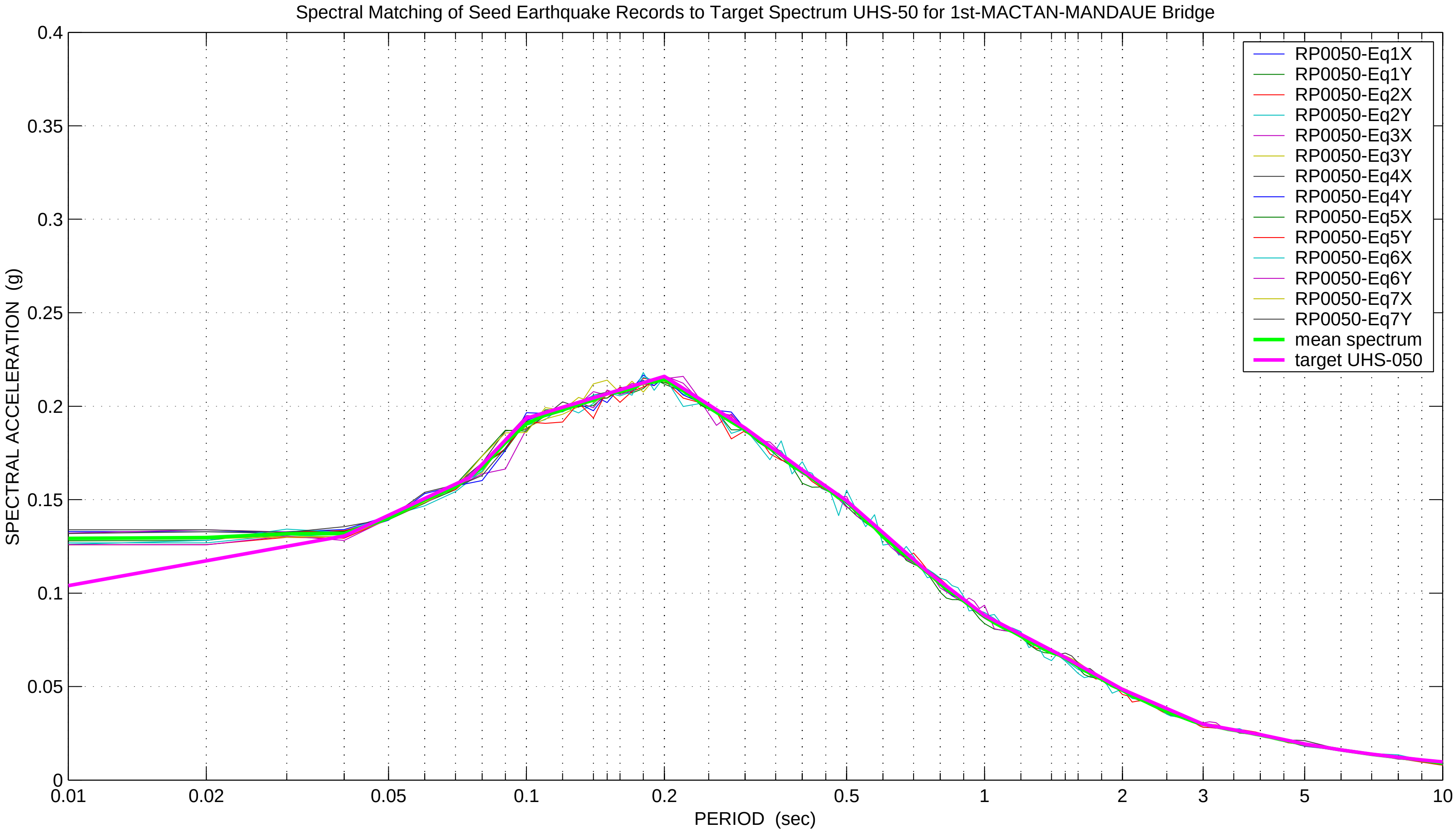


Figure 2B-163 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 50-year return period as compared to the target spectrum UHS-50 at 1st Mactan-Mandaue Bridge

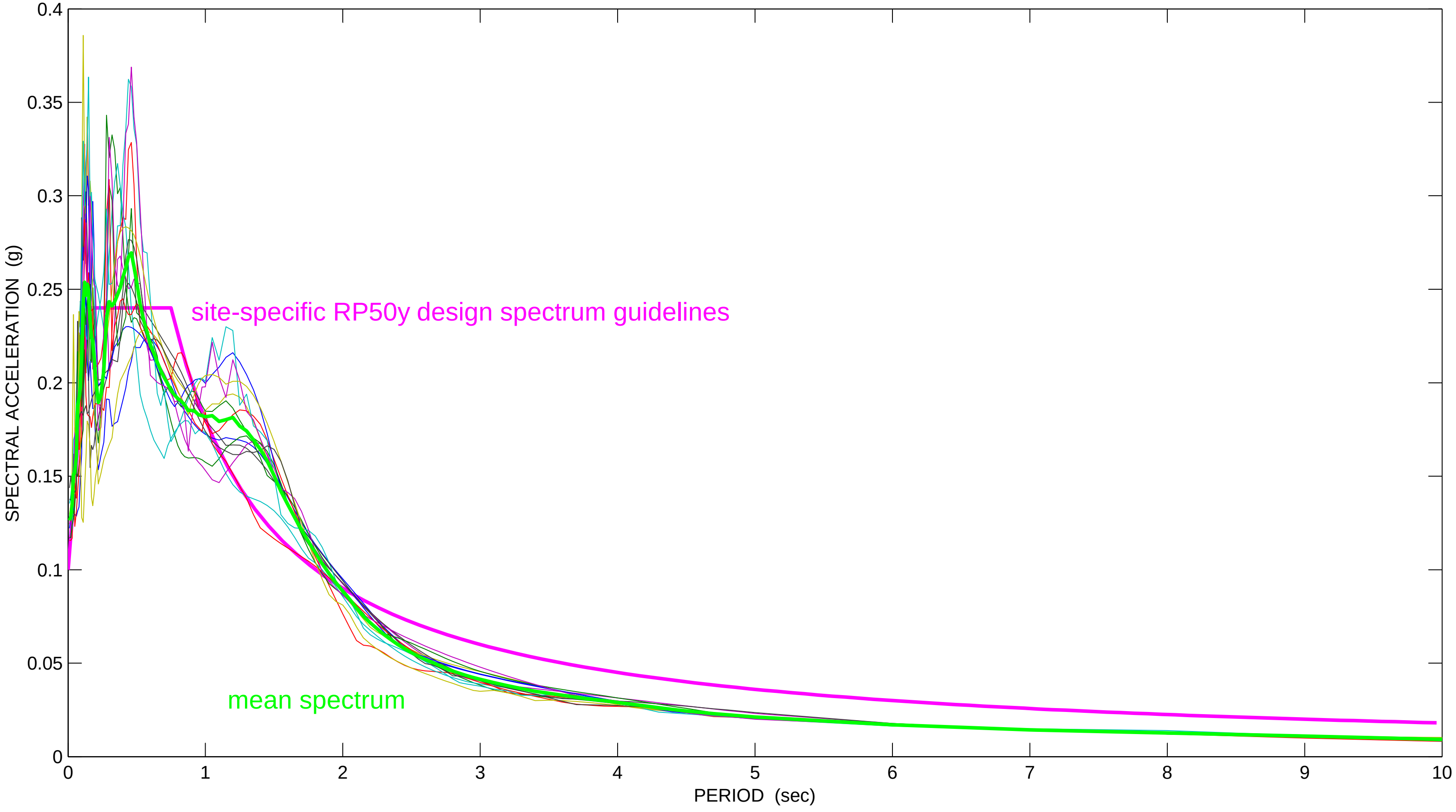


Figure 2B-164 Construction of site-specific design spectrum at 50-year return period for 1st Mactan-Mandaue A1 based on mean of 14 response spectra from nonlinear site response analysis

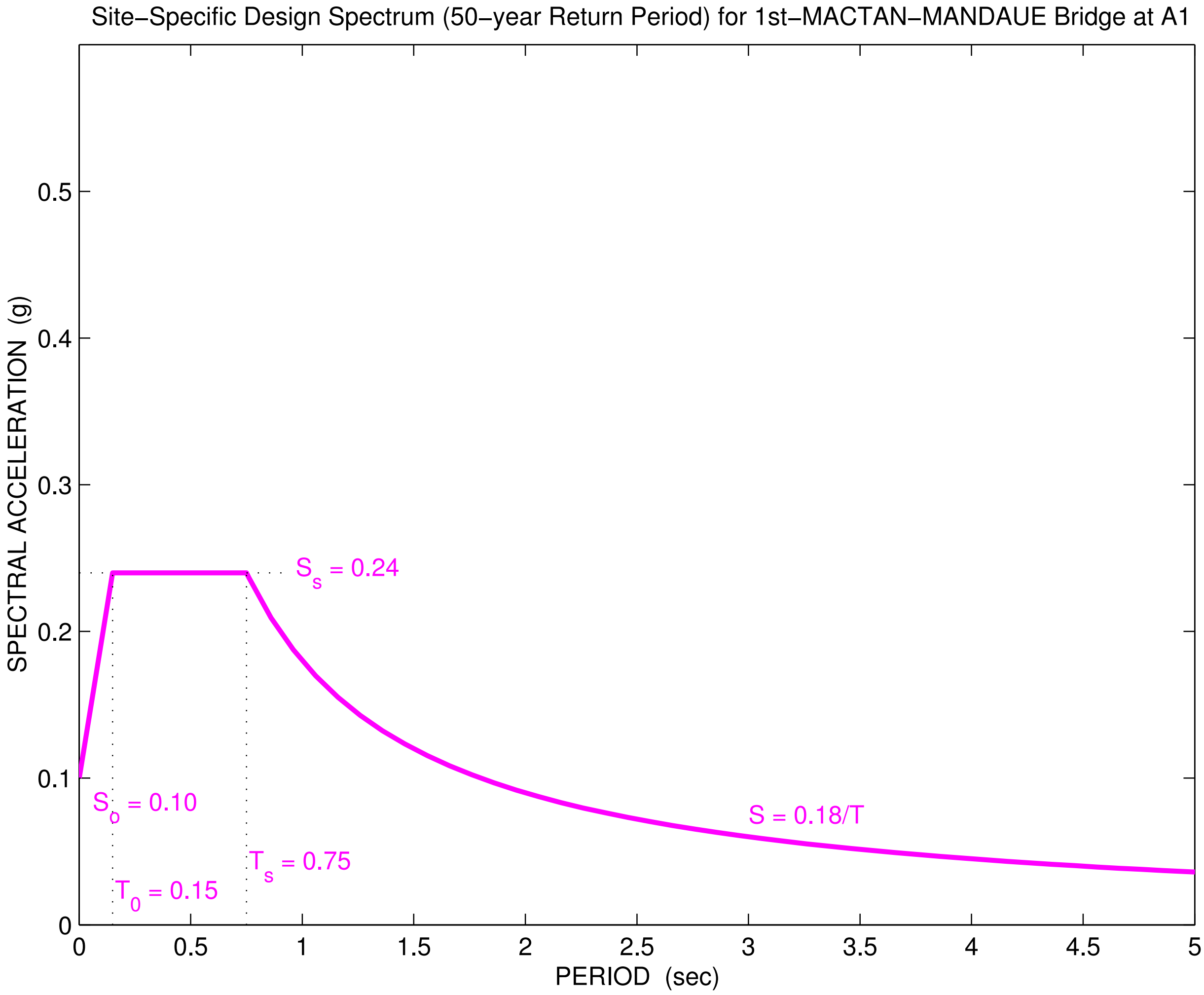


Figure 2B-165 Site-specific design spectrum at 50-year return period for 1st Mactan-Mandaue Bridge at A1 site

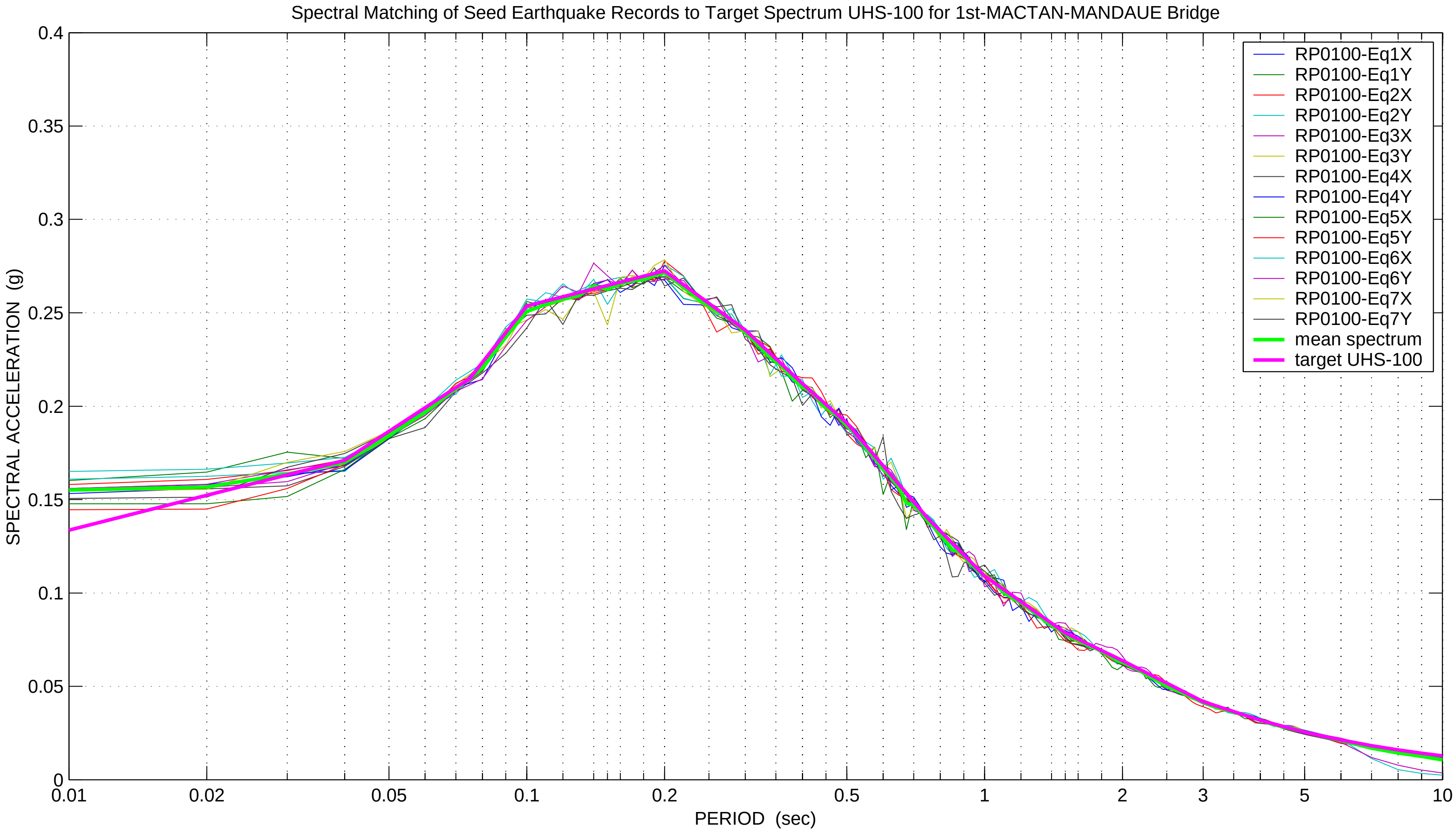


Figure 2B-166 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 100-year return period as compared to the target spectrum UHS-100 at 1st Mactan-Mandaue Bridge

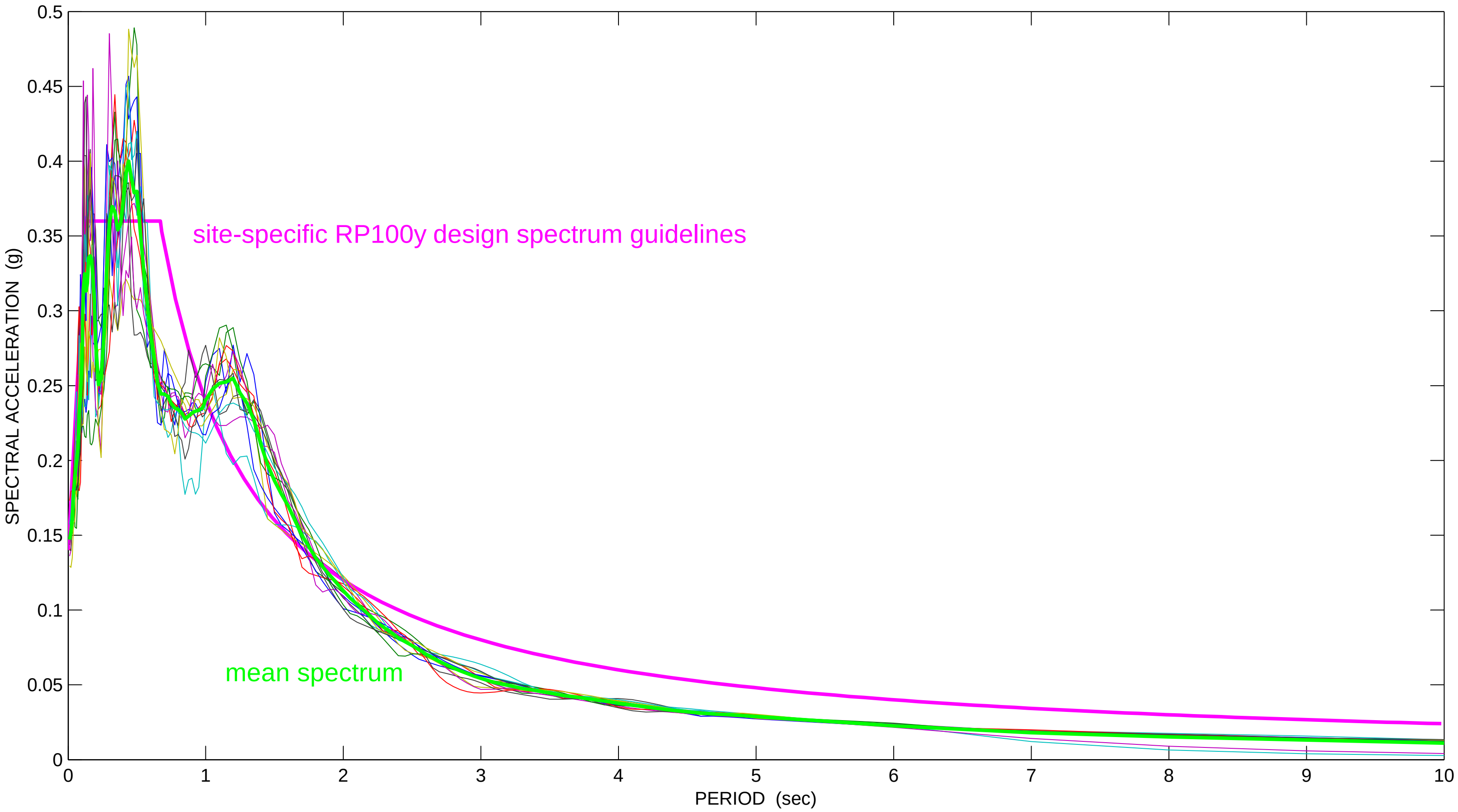


Figure 2B-165 Construction of site-specific design spectrum at 100-year return period for 1st Mactan-Mandaue A1 based on mean of 14 response spectra from nonlinear site response analysis

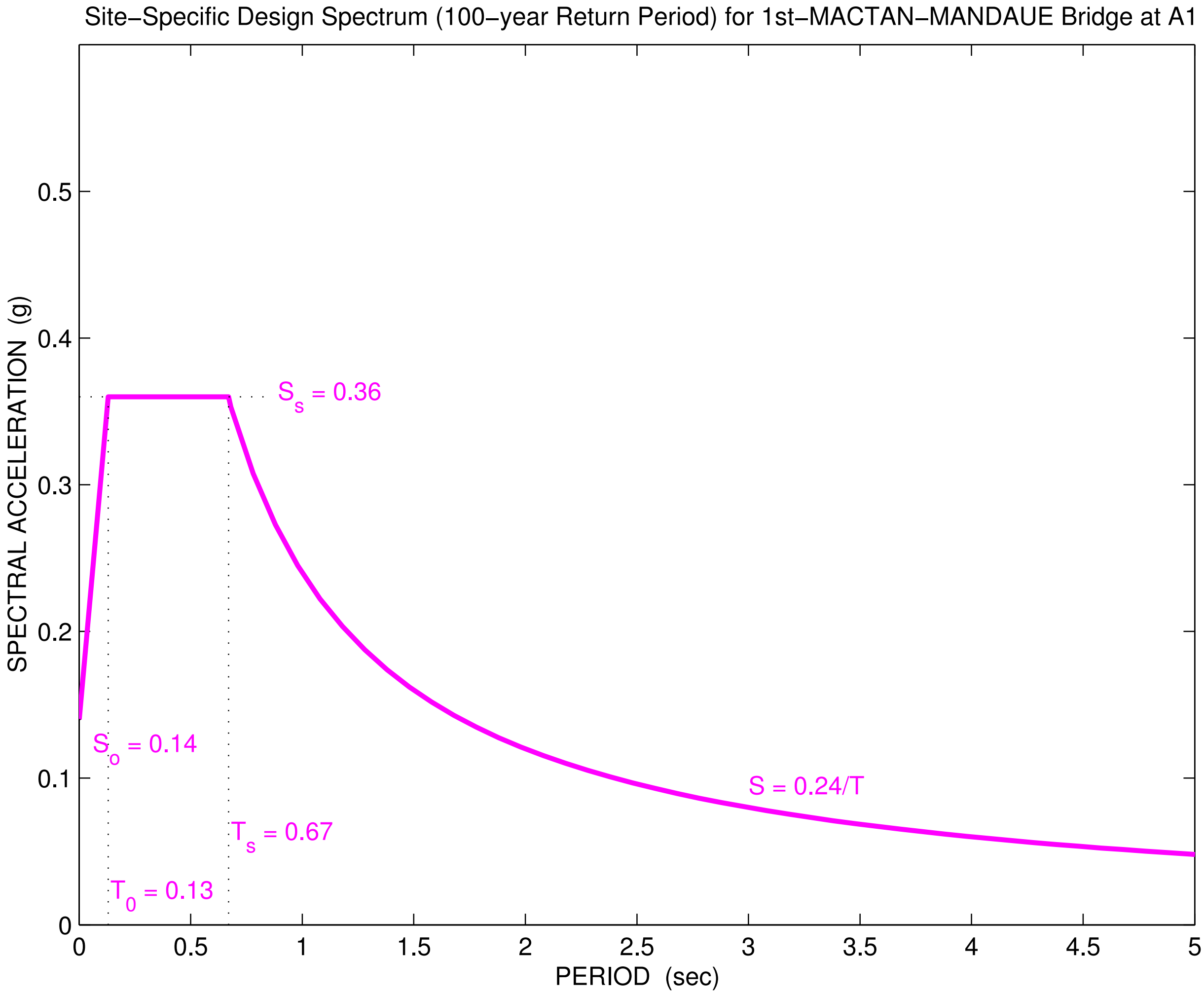


Figure 2B-166 Site-specific design spectrum at 100-year return period for 1st Mactan-Mandaue Bridge at A1 site

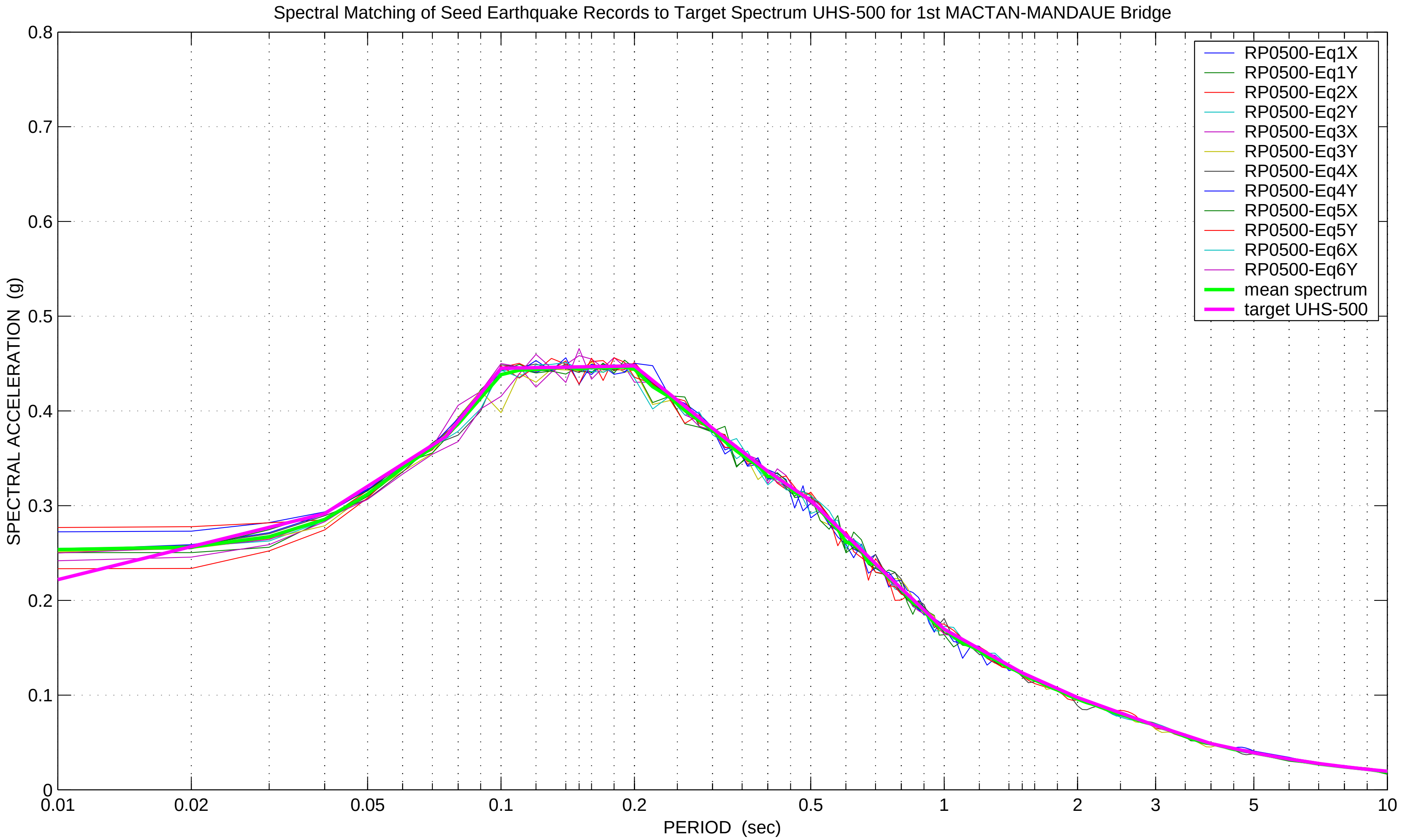


Figure 2B-167 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 500-year return period as compared to the target spectrum UHS-500 at 1st Mactan-Mandaue Bridge

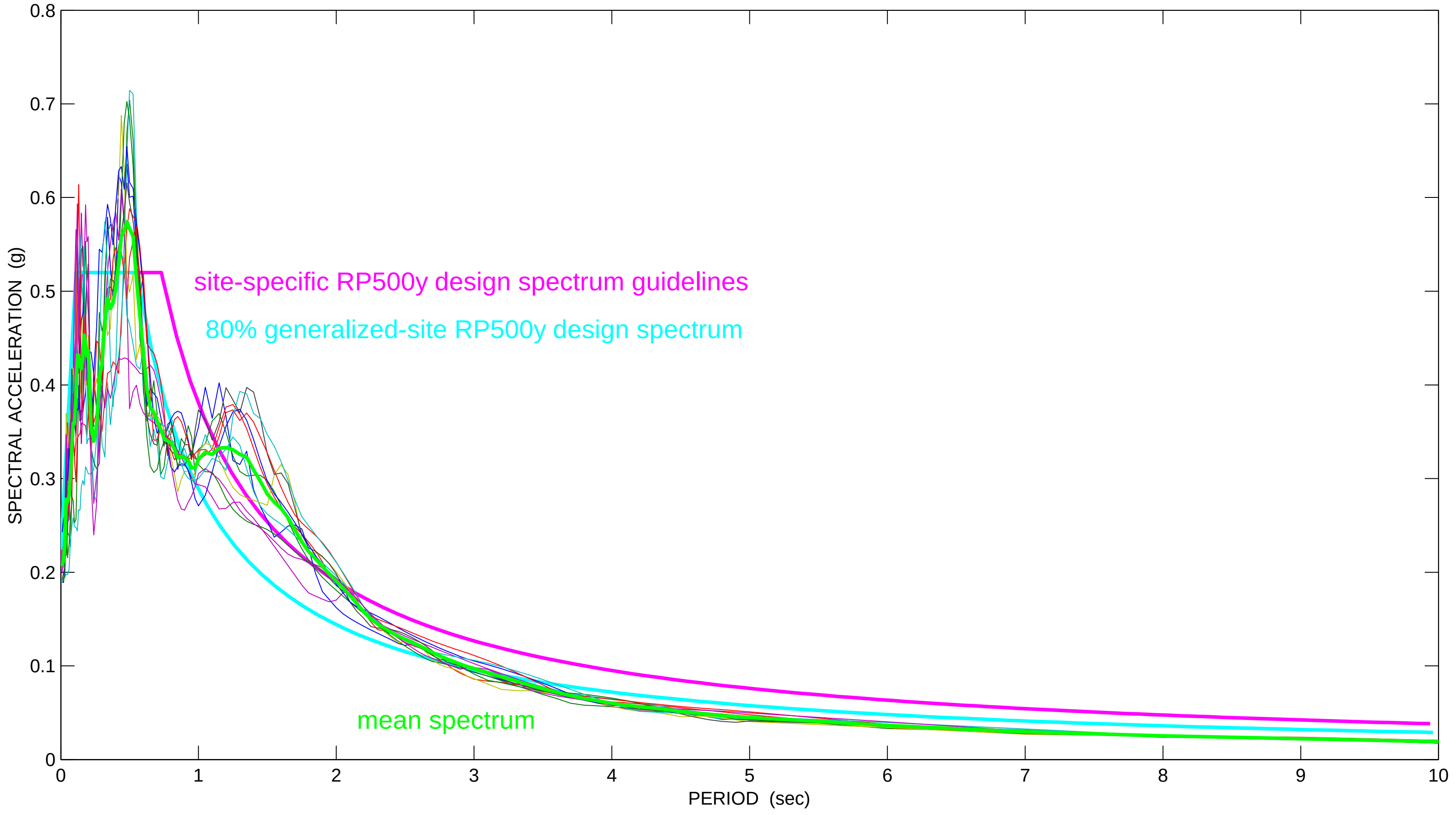


Figure 2B-168 Construction of site-specific design spectrum at 500-year return period for 1st Mactan-Mandaue A1 based on mean of 14 response spectra from nonlinear site response analysis

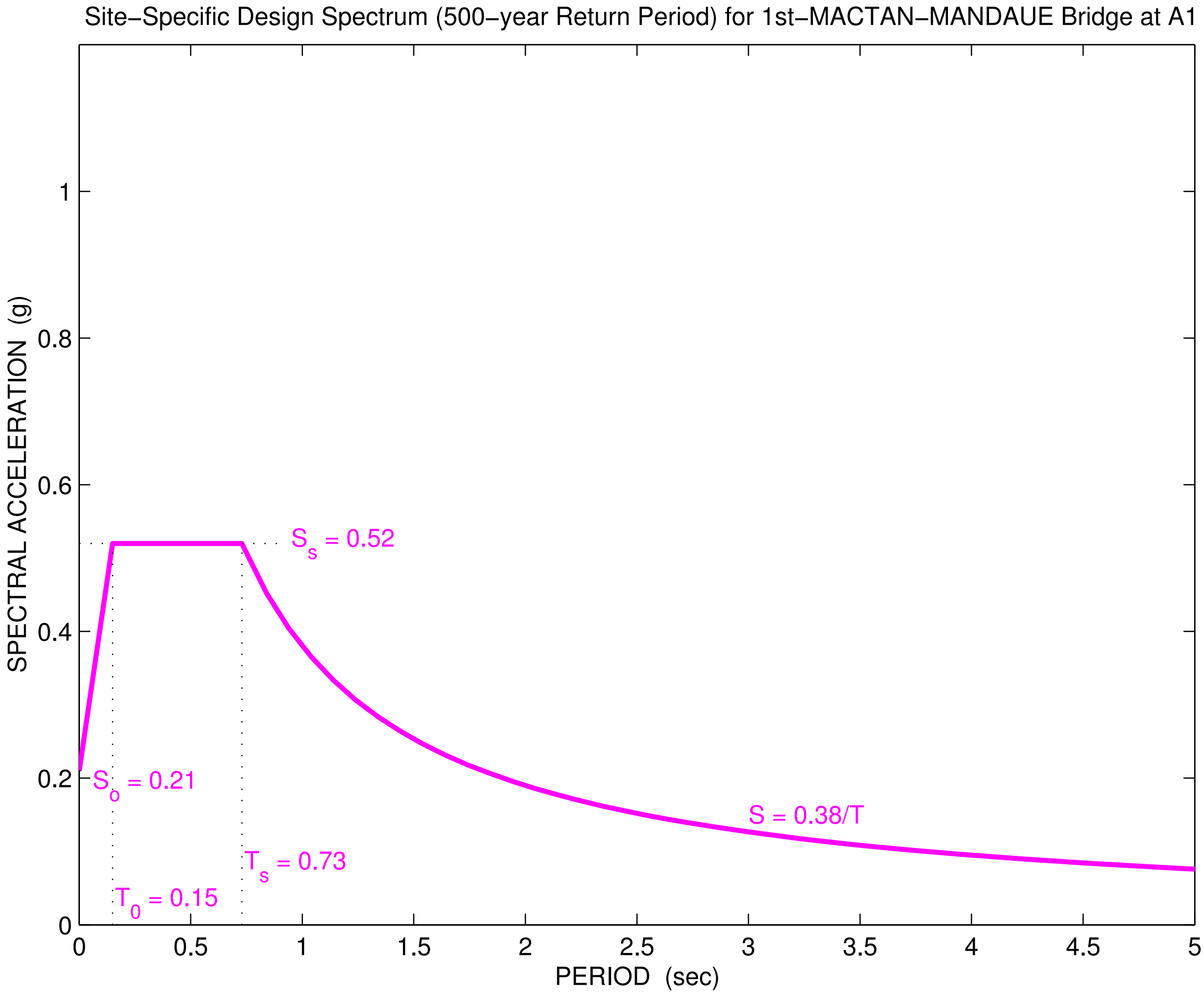


Figure 2B-169 Site-specific design spectrum at 500-year return period for 1st Mactan-Mandaue Bridge at A1 site

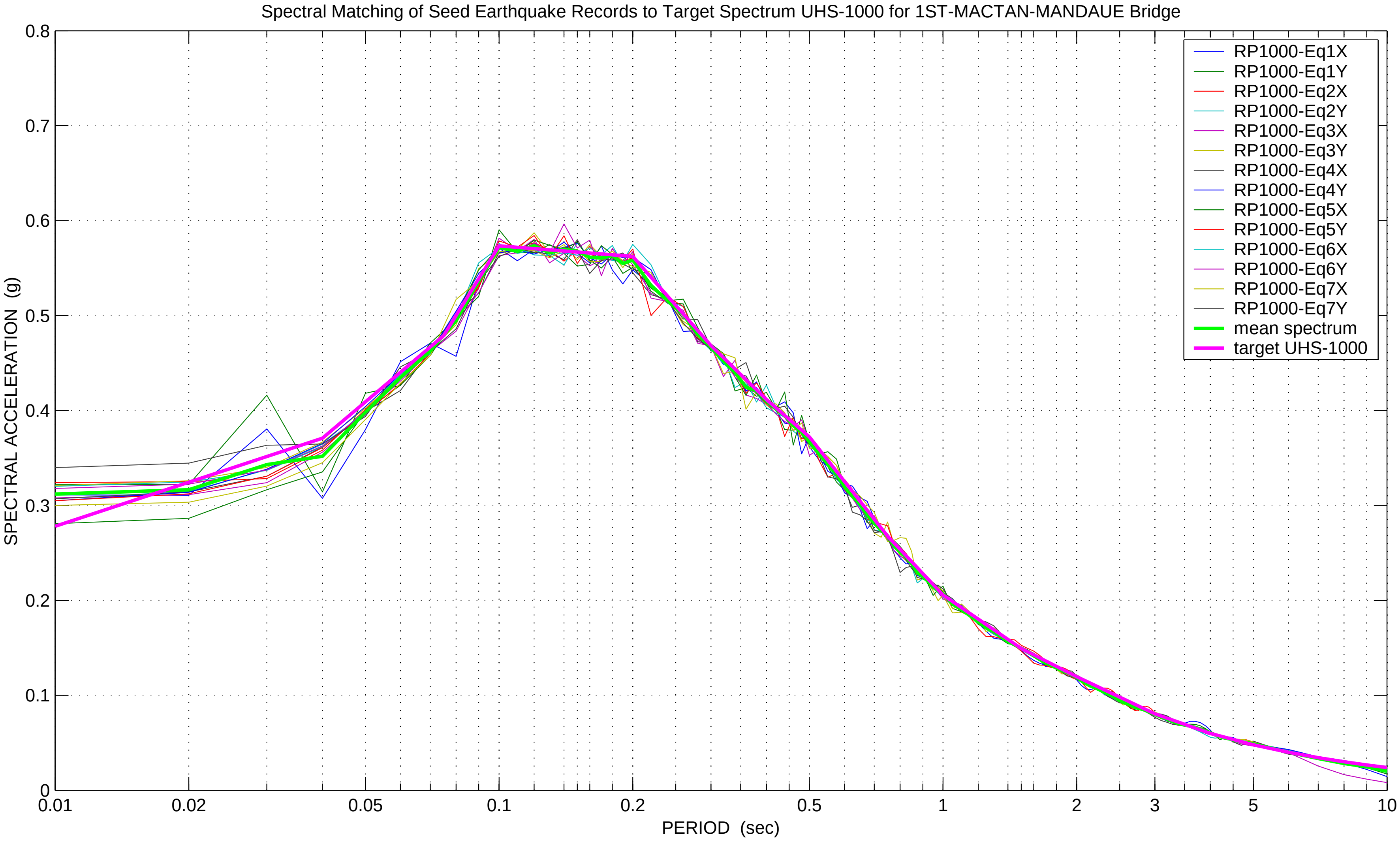


Figure 2B-170 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 1000-year return period as compared to the target spectrum UHS-1000 at 1st Mactan-Mandaue Bridge

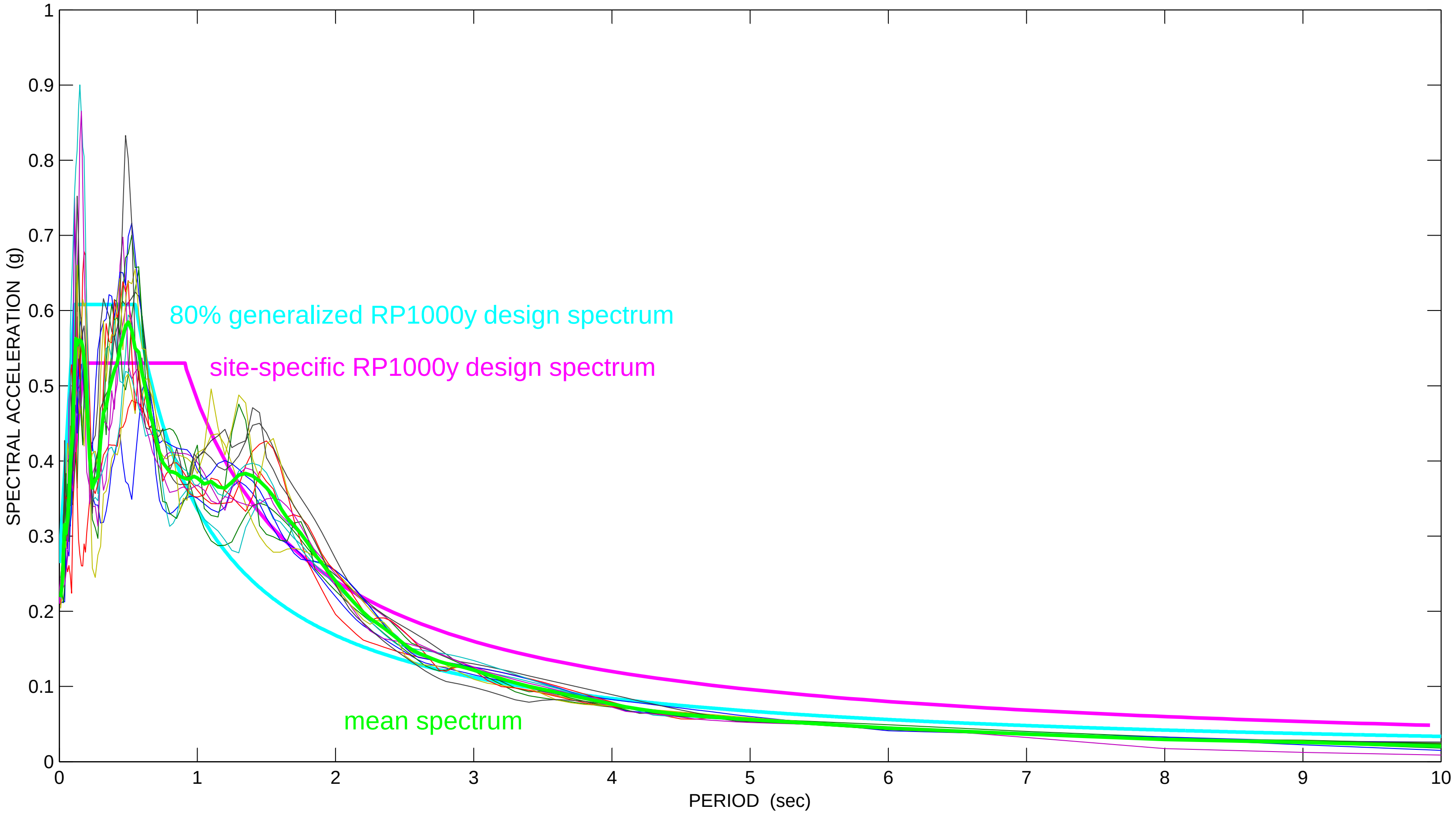


Figure 2B-171 Construction of site-specific design spectrum at 1000-year return period for 1st Mactan-Mandaue A1 based on mean of 14 response spectra from nonlinear site response analysis

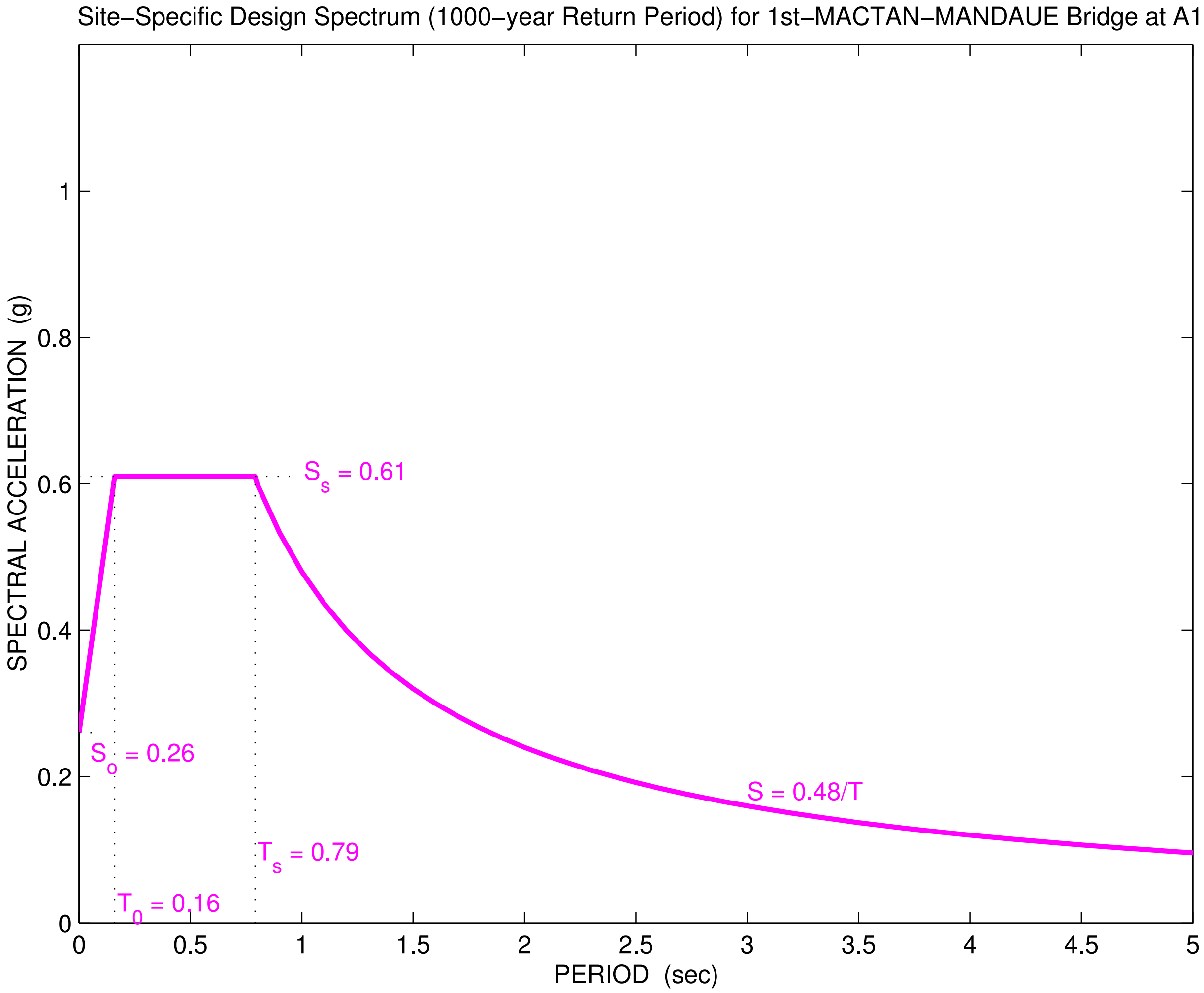


Figure 2B-172 Site-specific design spectrum at 1000-year return period for 1st Mactan-Mandaue Bridge at A1 site

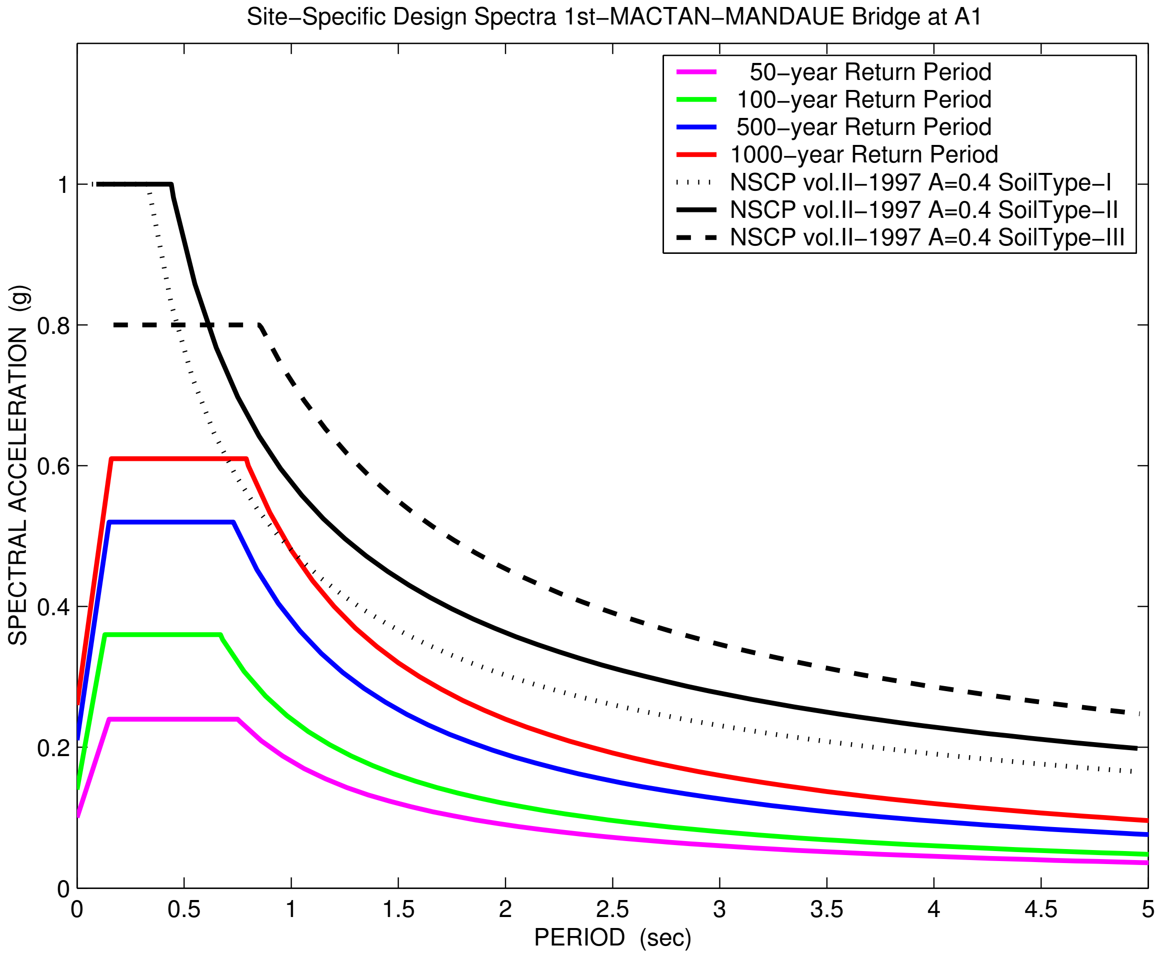


Figure 2B-173 Site-specific design spectra for 1st Mactan-Mandaue Bridge at A1

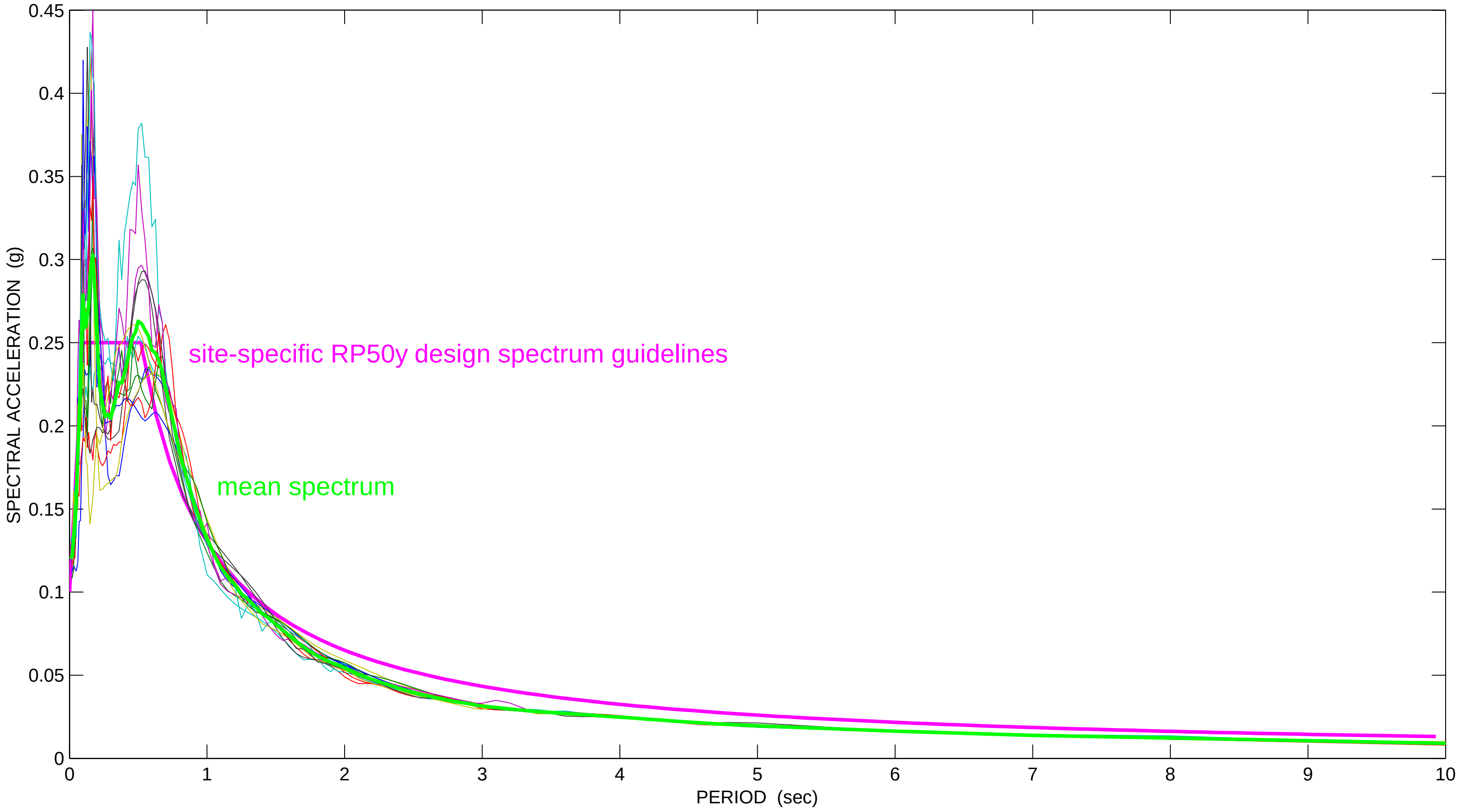


Figure 2B-174 Construction of site-specific design spectrum at 50-year return period for 1st Mactan-Mandaue B2 based on mean of 14 response spectra from nonlinear site response analysis

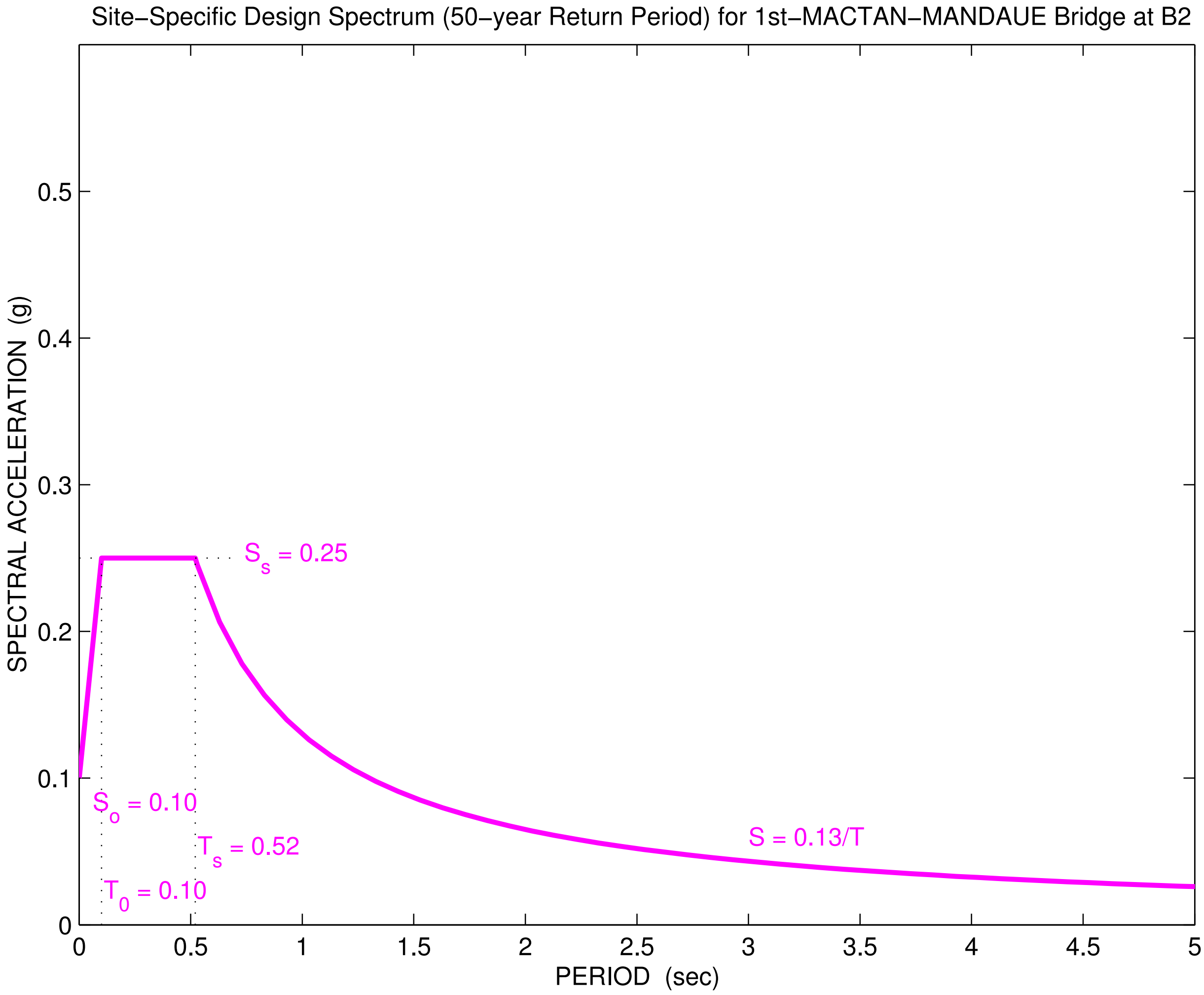


Figure 2B-175 Site-specific design spectrum at 50-year return period for 1st Mactan-Mandaue Bridge at B2 site

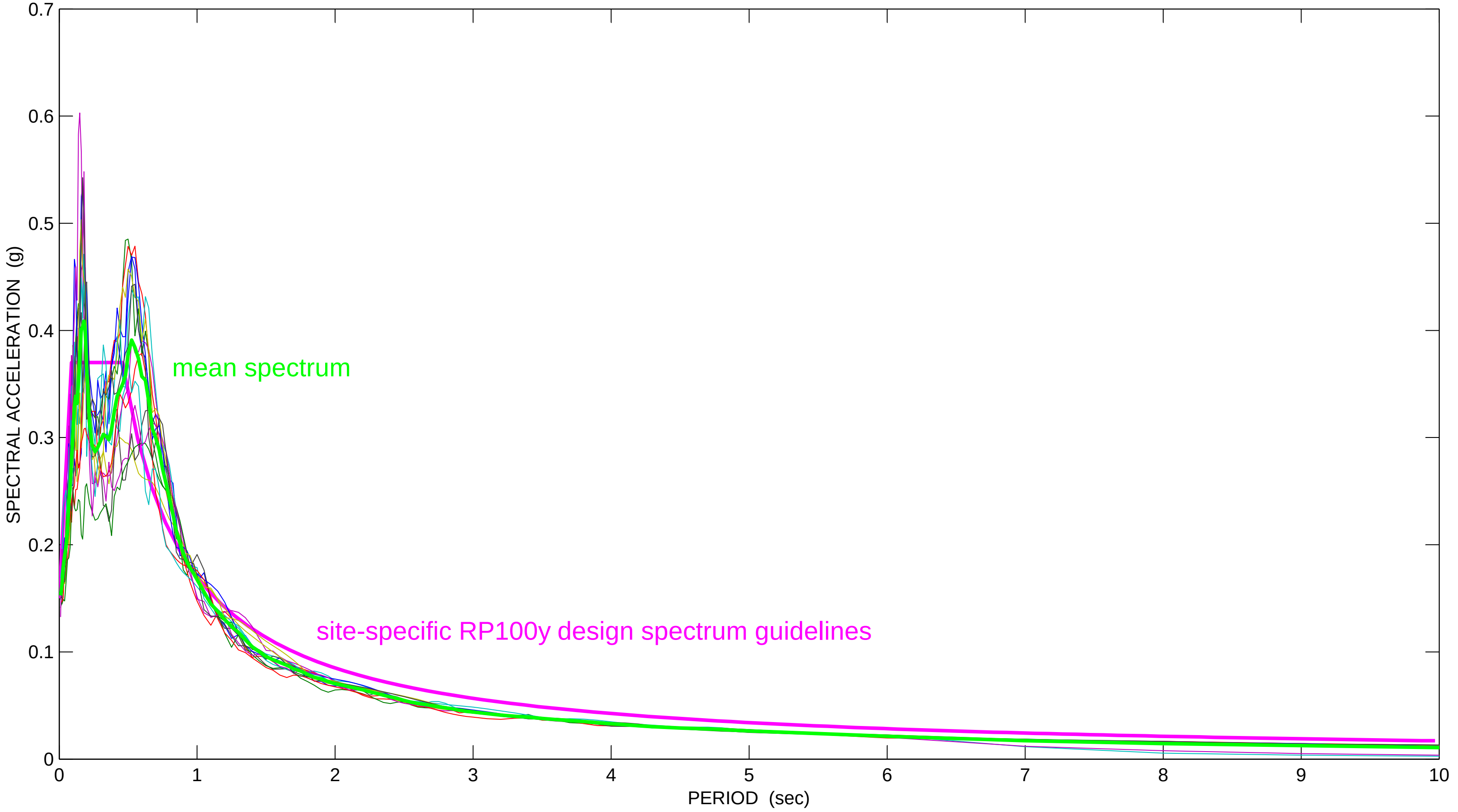


Figure 2B-176 Construction of site-specific design spectrum at 100-year return period for 1st Mactan-Mandaue B2 based on mean of 14 response spectra from nonlinear site response analysis

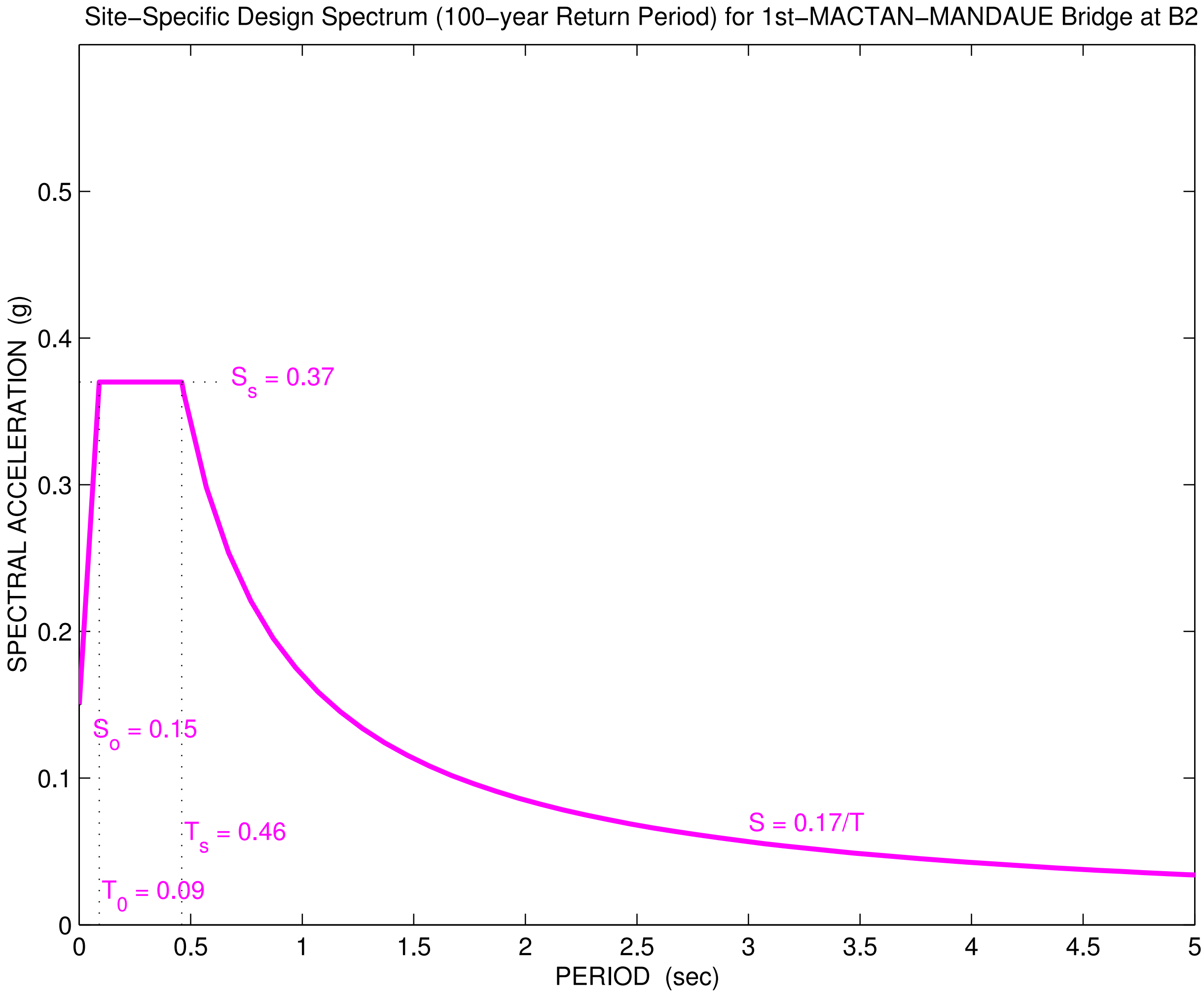


Figure 2B-177 Site-specific design spectrum at 100-year return period for 1st Mactan-Mandaue Bridge at B2 site

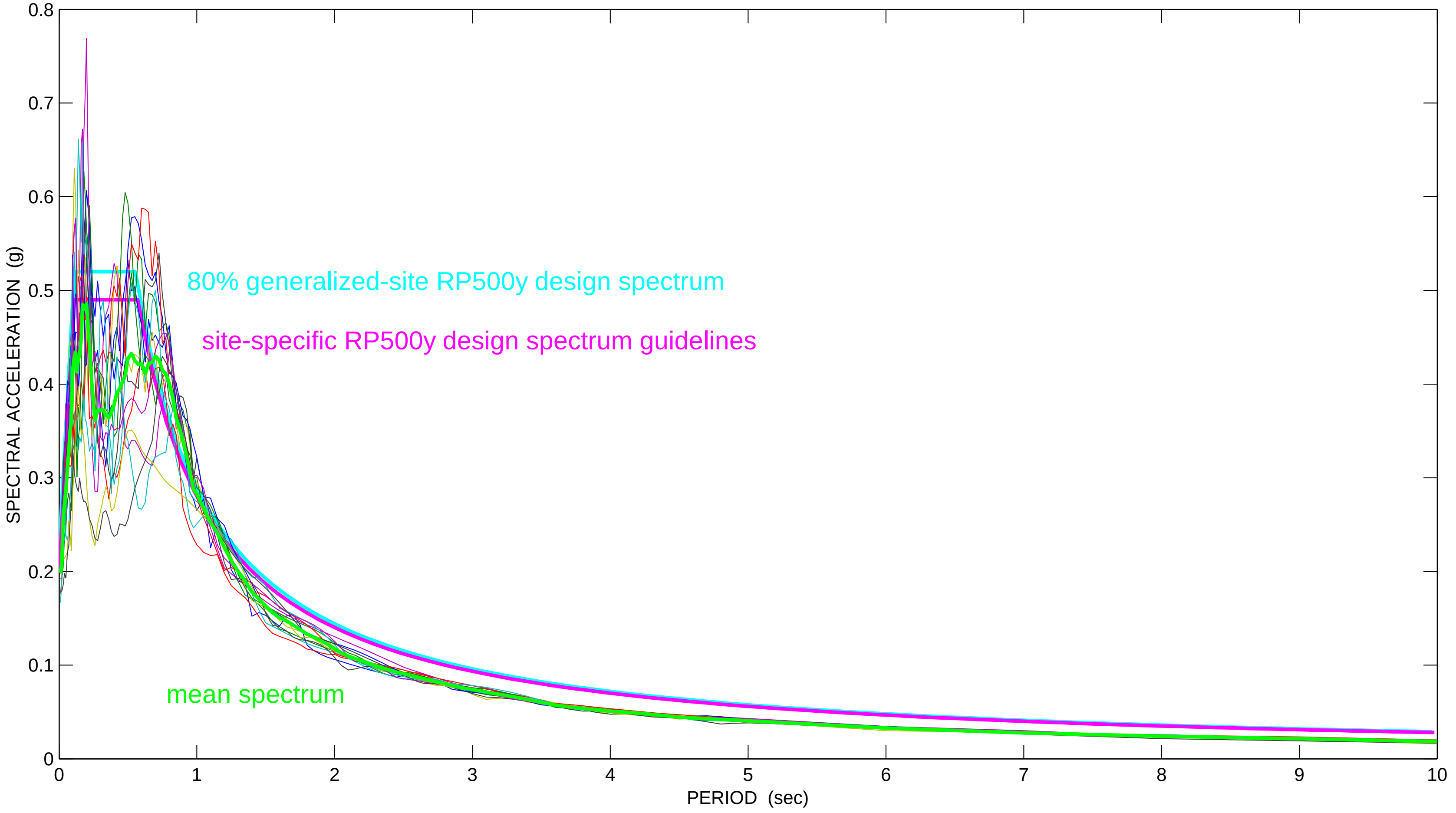


Figure 2B-178 Construction of site-specific design spectrum at 500-year return period for 1st Mactan-Mandaue B2 based on mean of 14 response spectra from nonlinear site response analysis

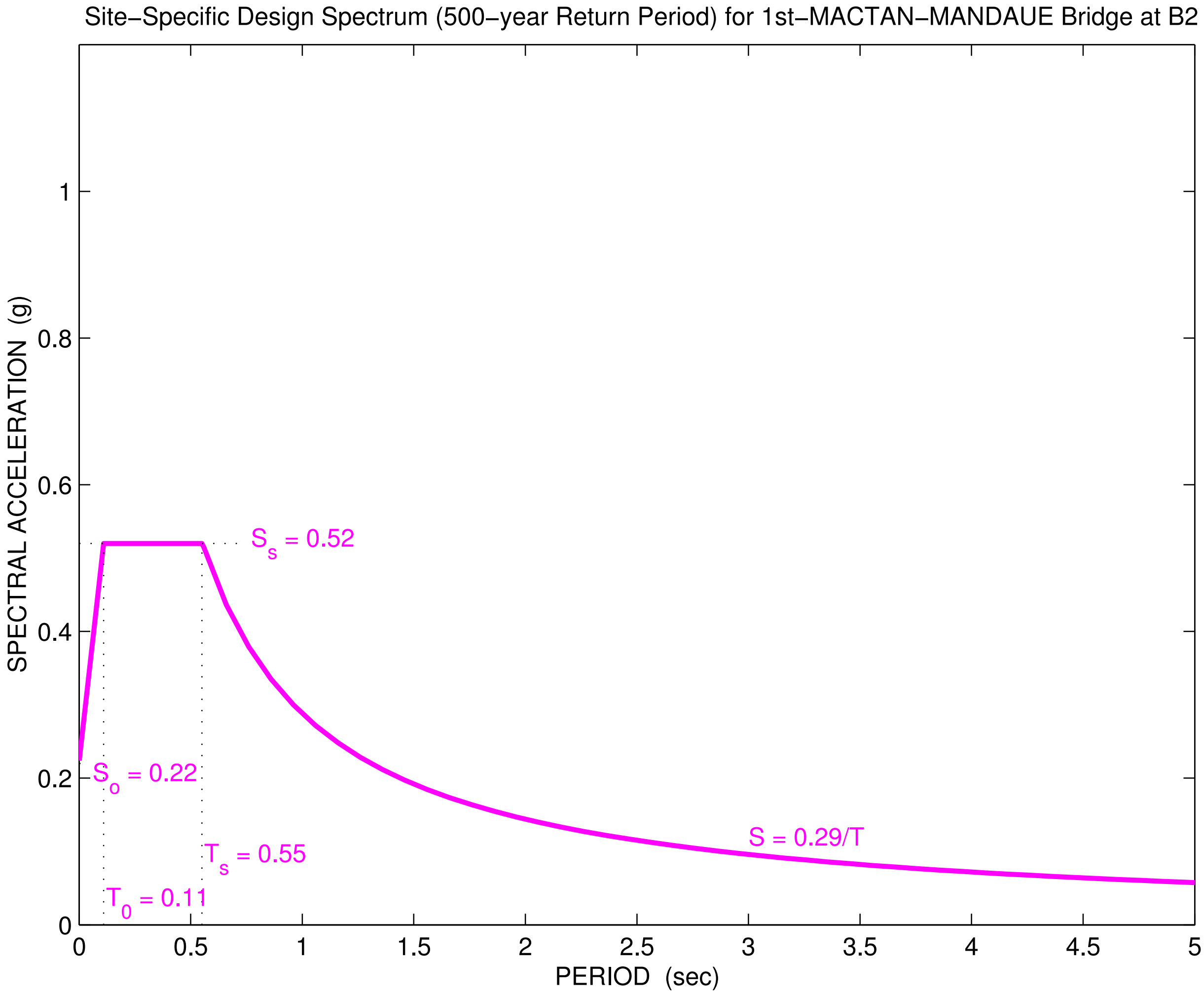


Figure 2B-179 Site-specific design spectrum at 500-year return period for 1st Mactan-Mandaue Bridge at B2 site

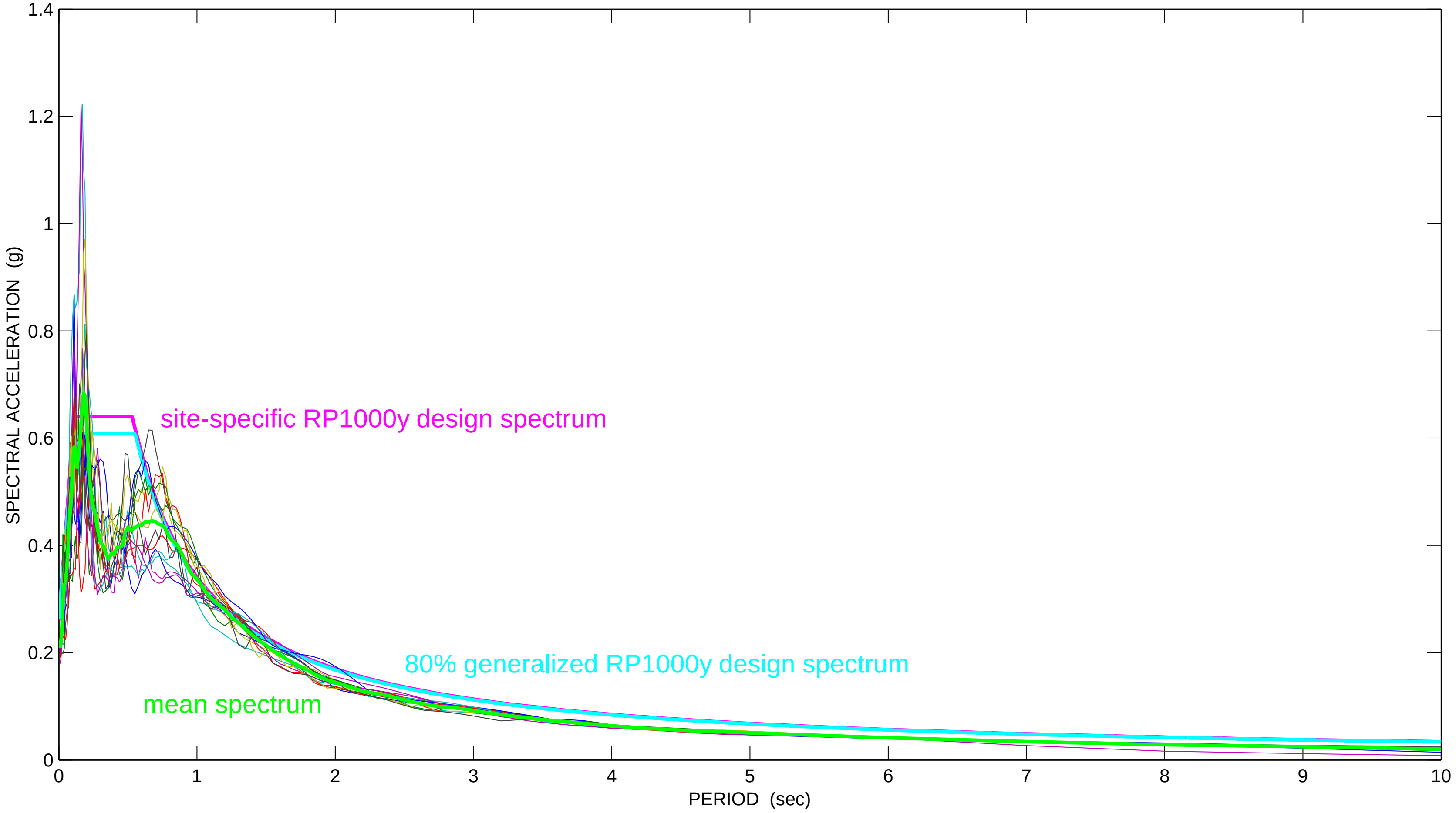


Figure 2B-180 Construction of site-specific design spectrum at 1000-year return period for 1st Mactan-Mandaue B2 based on mean of 14 response spectra from nonlinear site response analysis

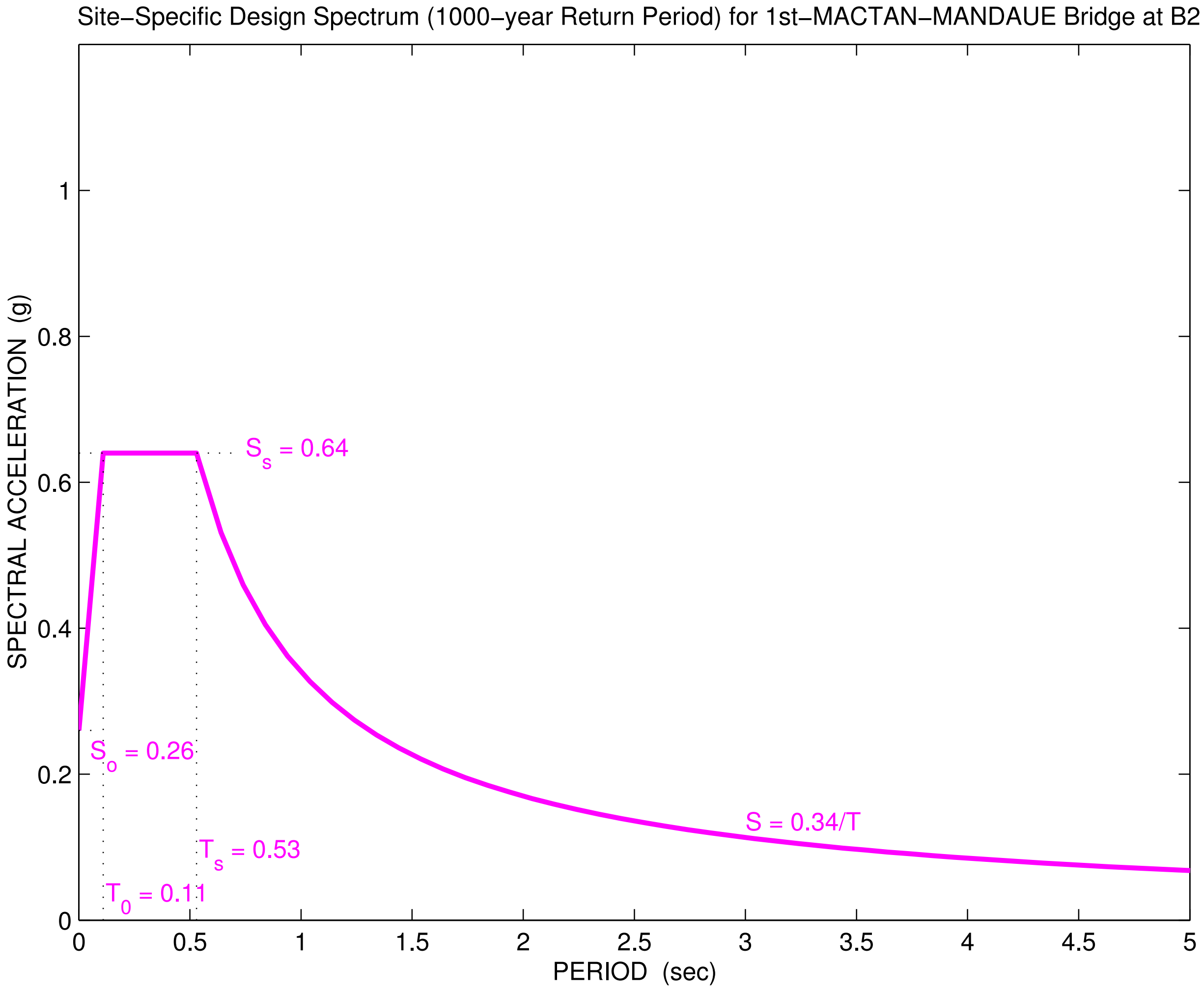


Figure 2B-181 Site-specific design spectrum at 1000-year return period for 1st Mactan-Mandaue Bridge at B2 site

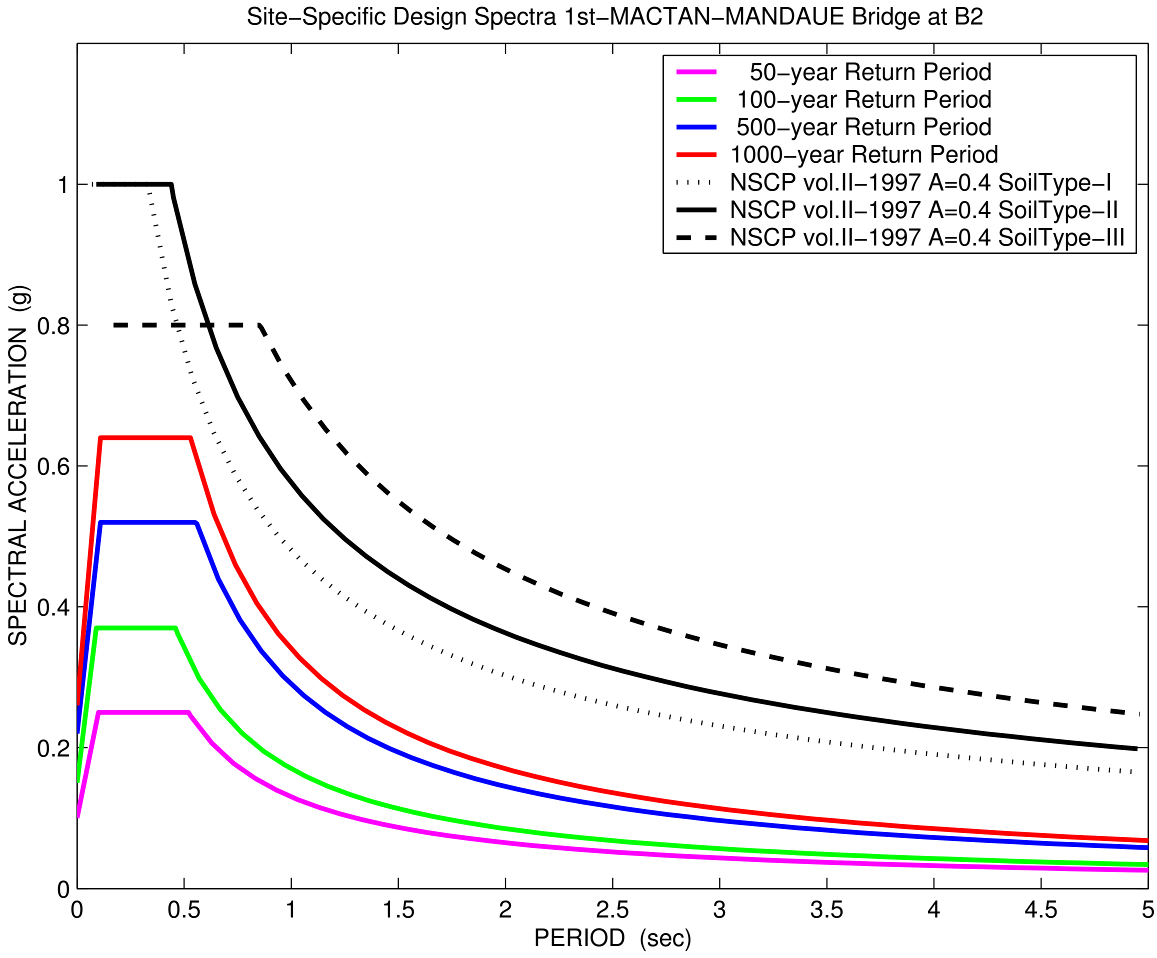
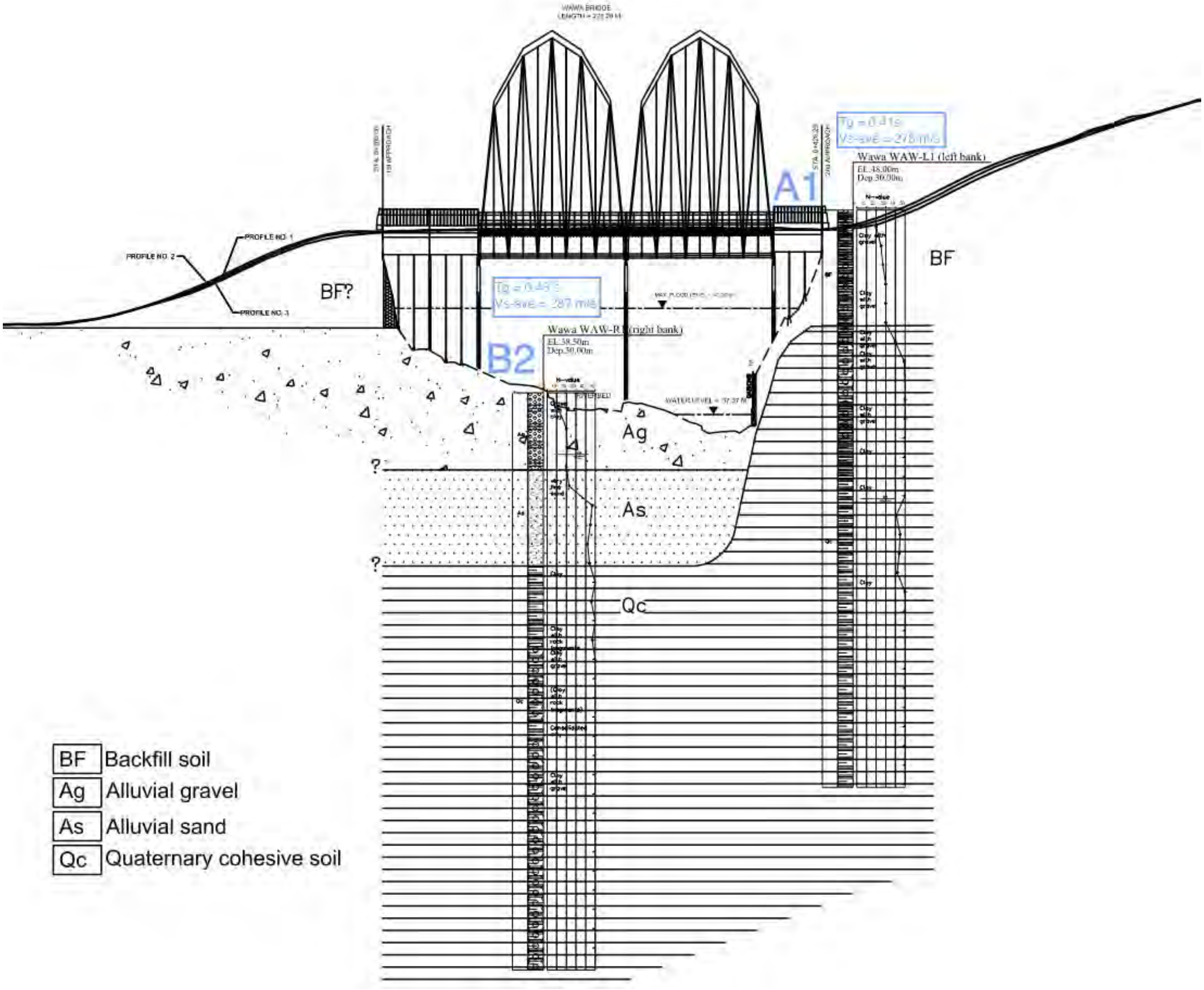


Figure 2B-182 Site-specific design spectra for 1st Mactan-Mandaue Bridge at B2



(c) soil profile

Figure 2B-183 Wawa Bridge site data

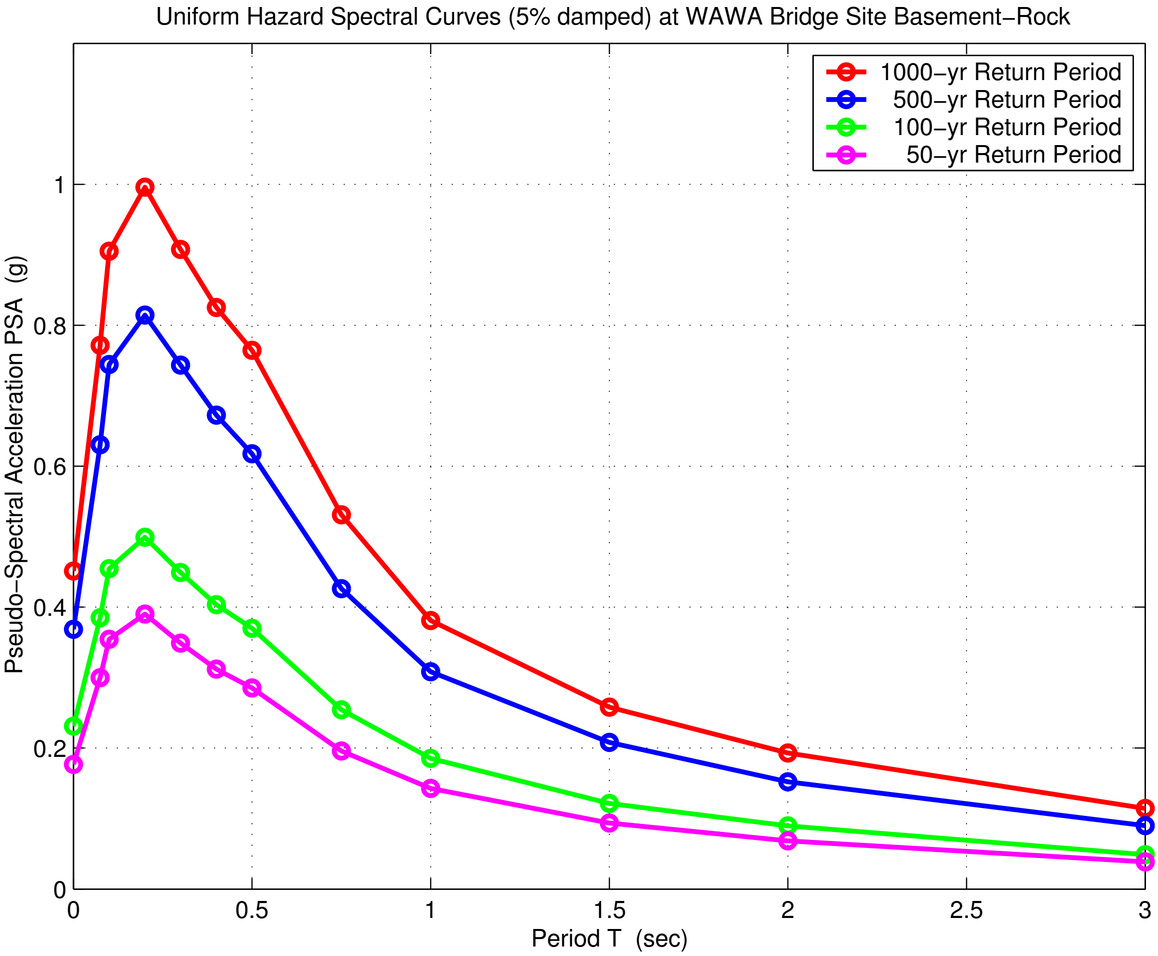


Figure 2B-184 Uniform hazard spectral curve for Wawa Bridge site at basement-rock

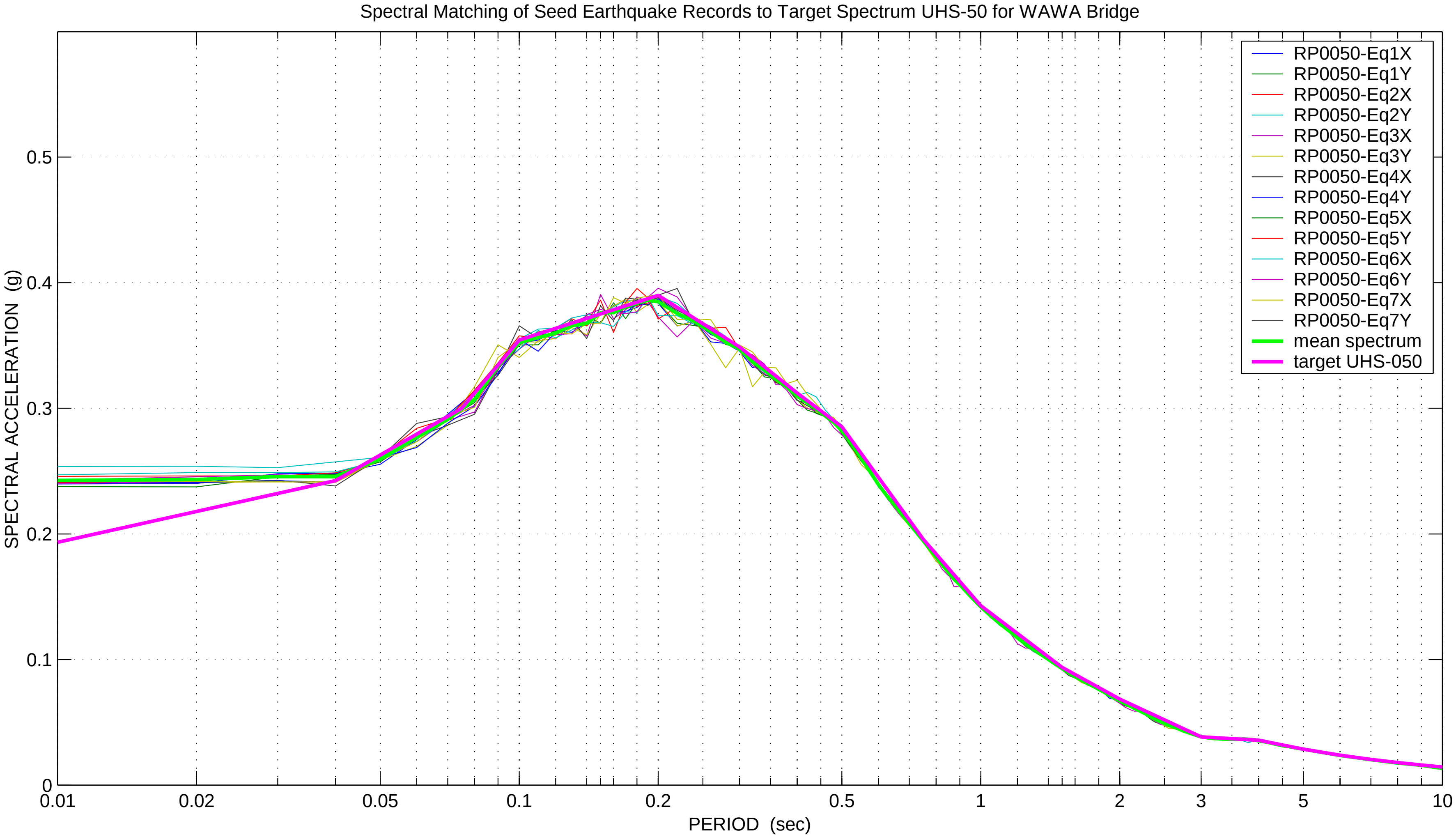


Figure 2B-185 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 50-year return period as compared to the target spectrum UHS-50 at Wawa Bridge

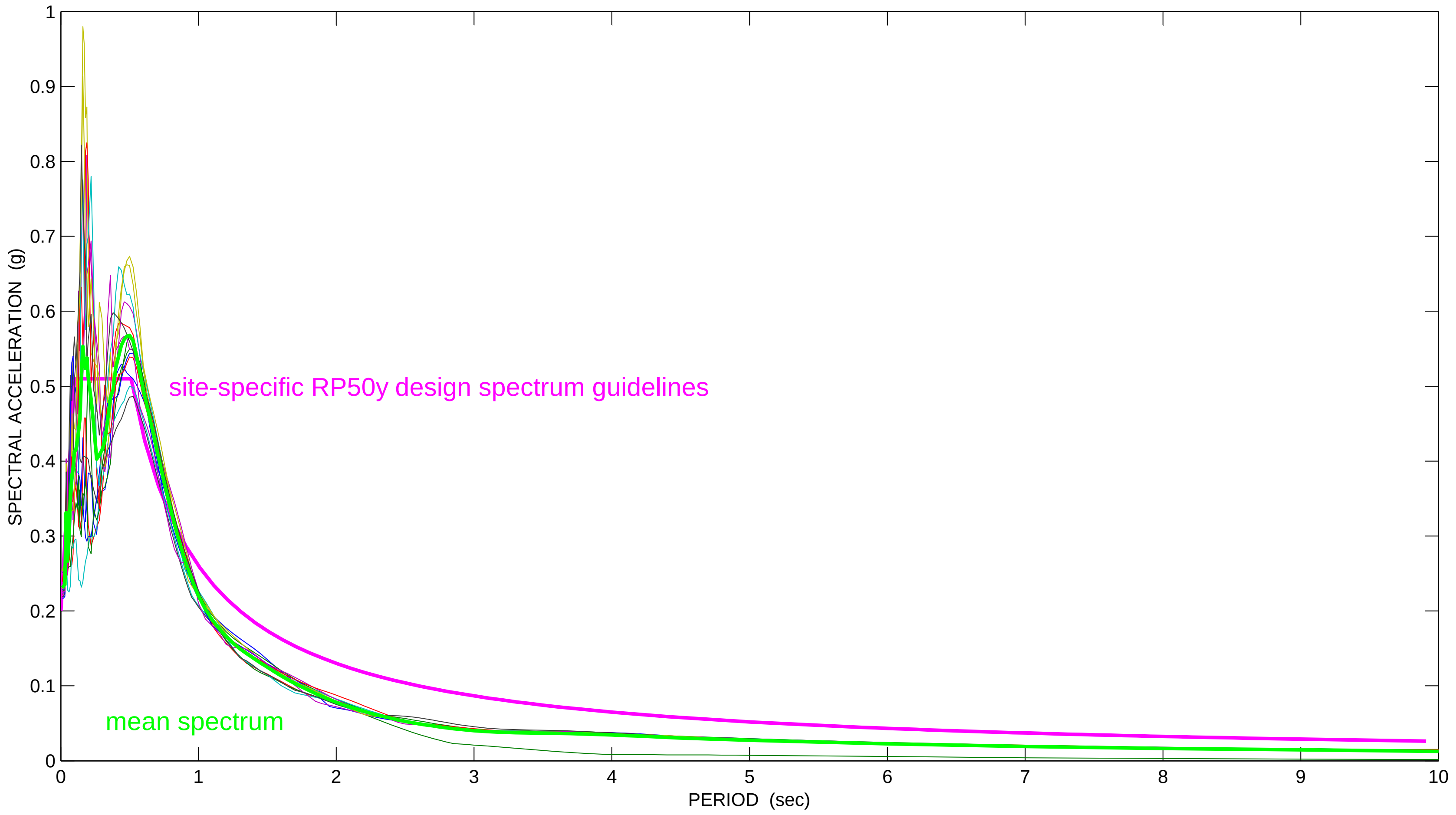


Figure 2B-186 Construction of site-specific design spectrum at 50-year return period for Wawa A1 based on mean of 14 response spectra from nonlinear site response analysis

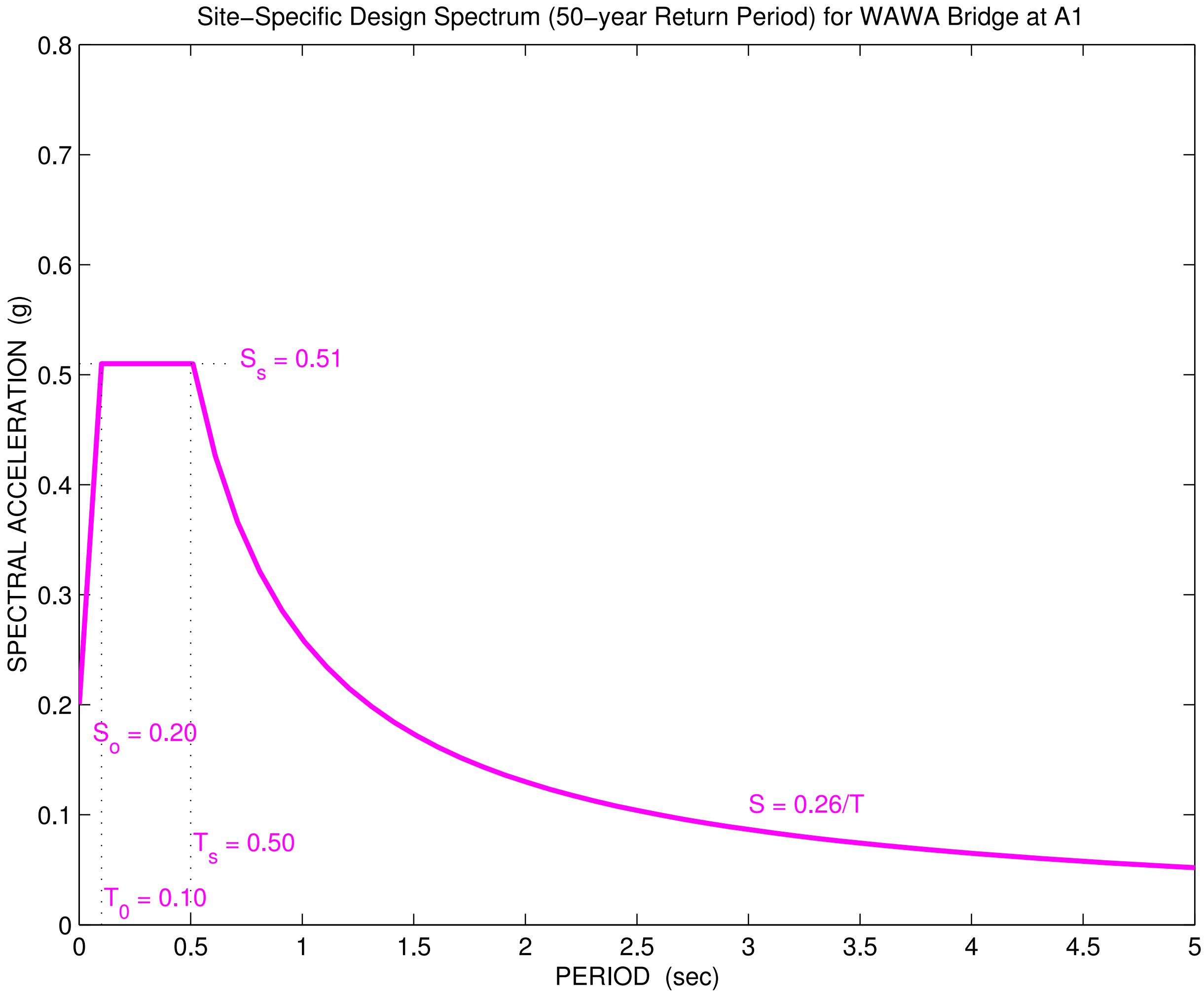


Figure 2B-187 Site-specific design spectrum at 50-year return period for Wawa Bridge at A1 site

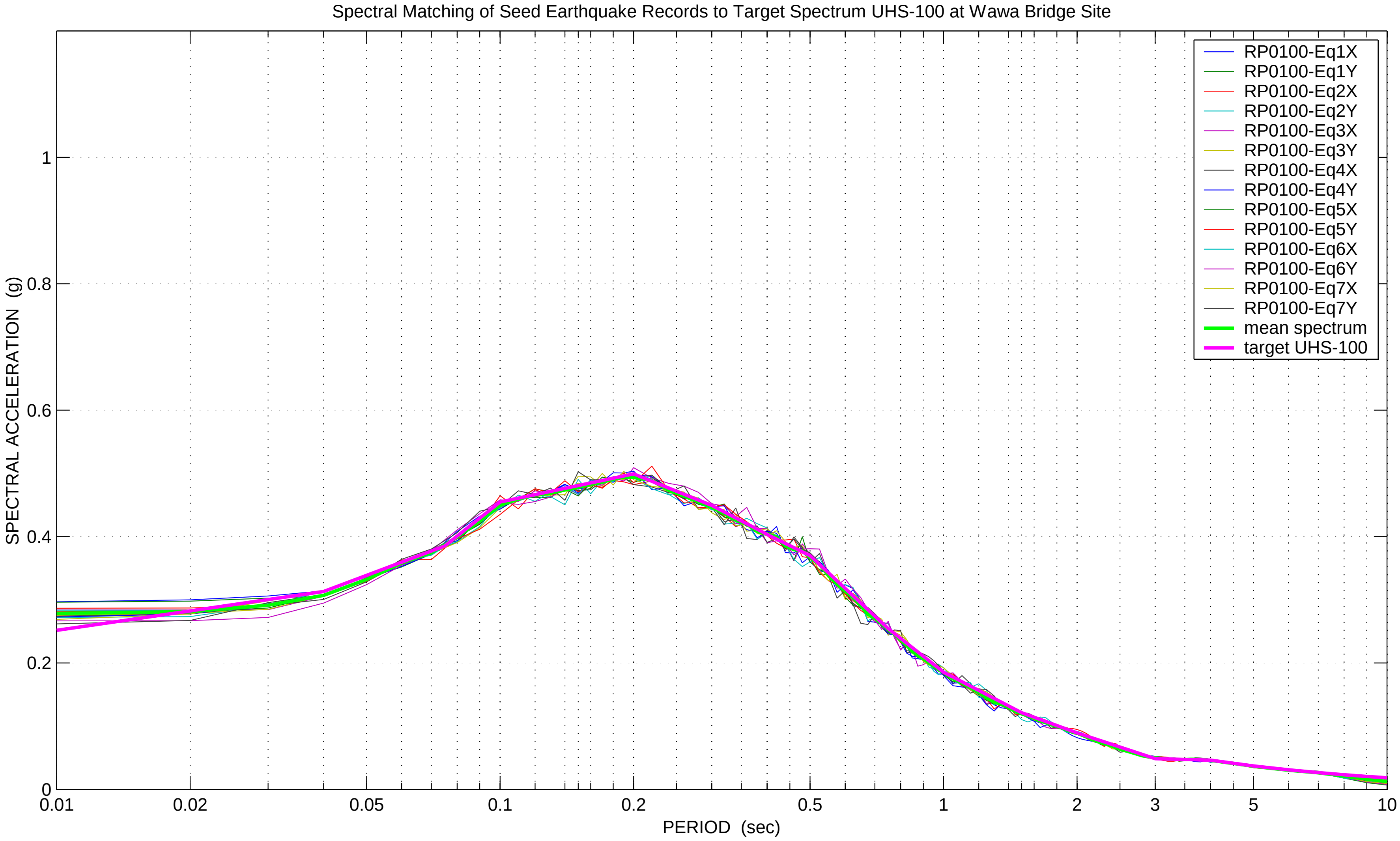


Figure 2B-188 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 100-year return period as compared to the target spectrum UHS-100 at Wawa Bridge

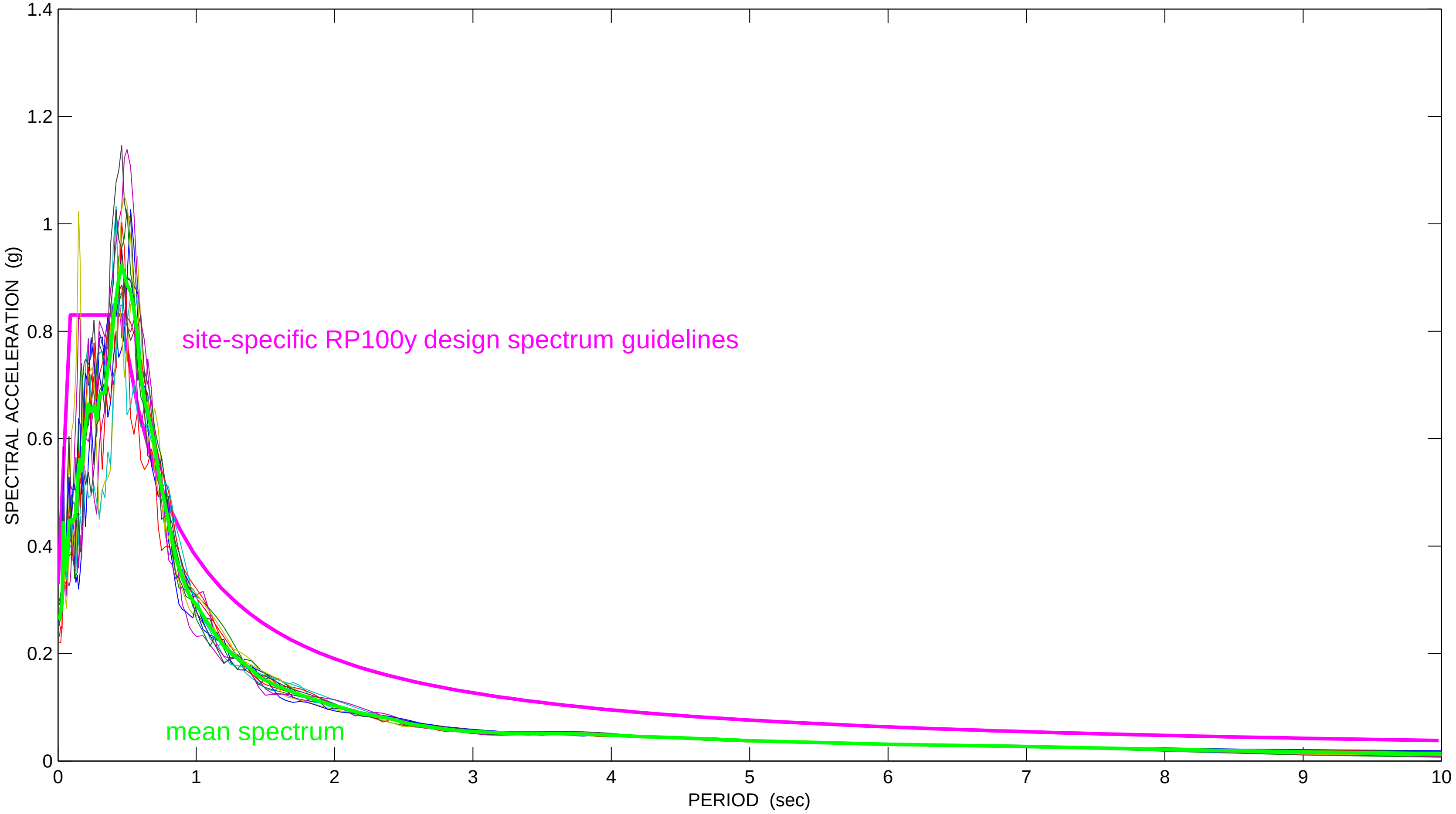


Figure 2B-189 Construction of site-specific design spectrum at 100-year return period for Wawa A1 based on mean of 14 response spectra from nonlinear site response analysis

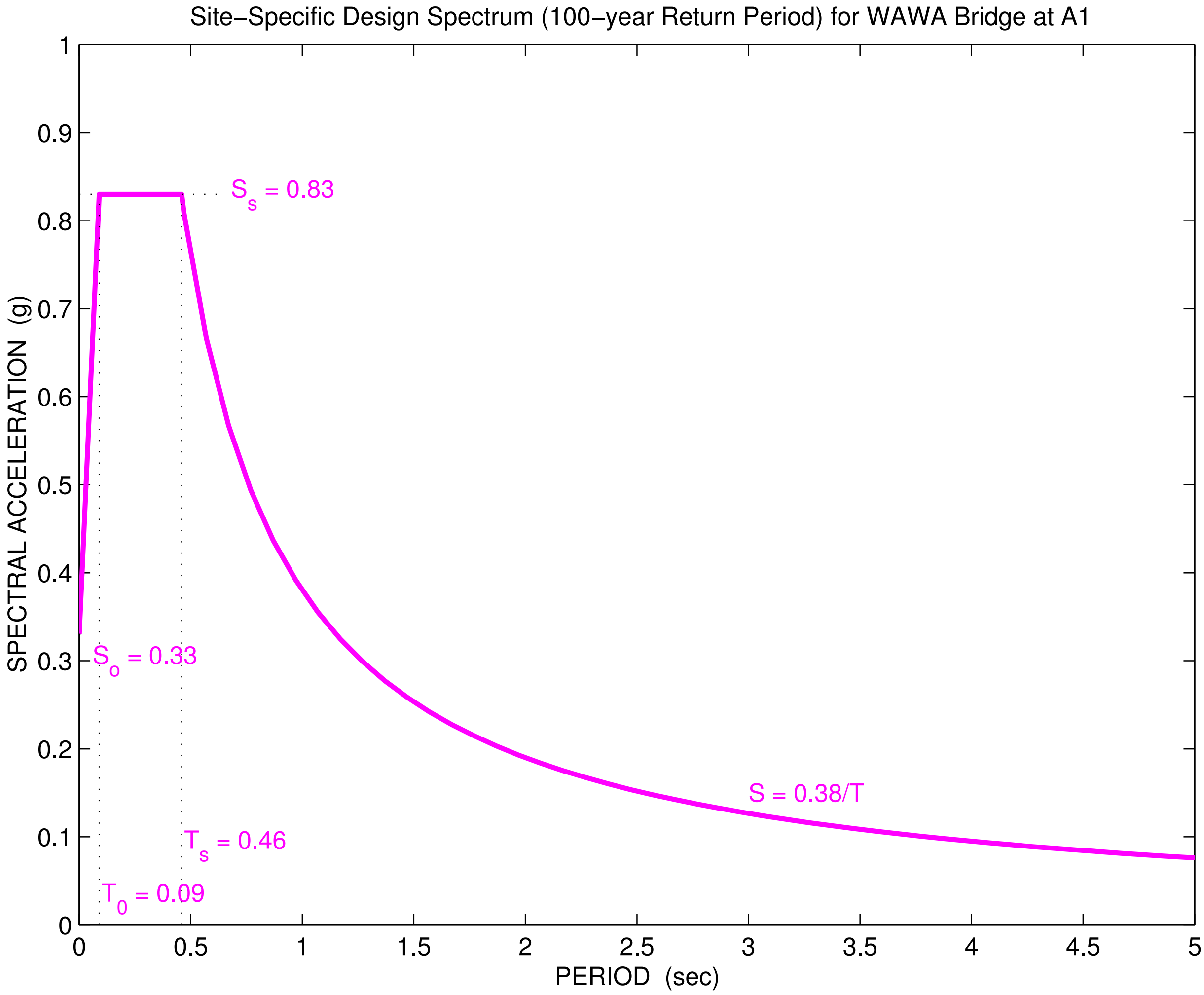


Figure 2B-190 Site-specific design spectrum at 100-year return period for Wawa Bridge at A1 site

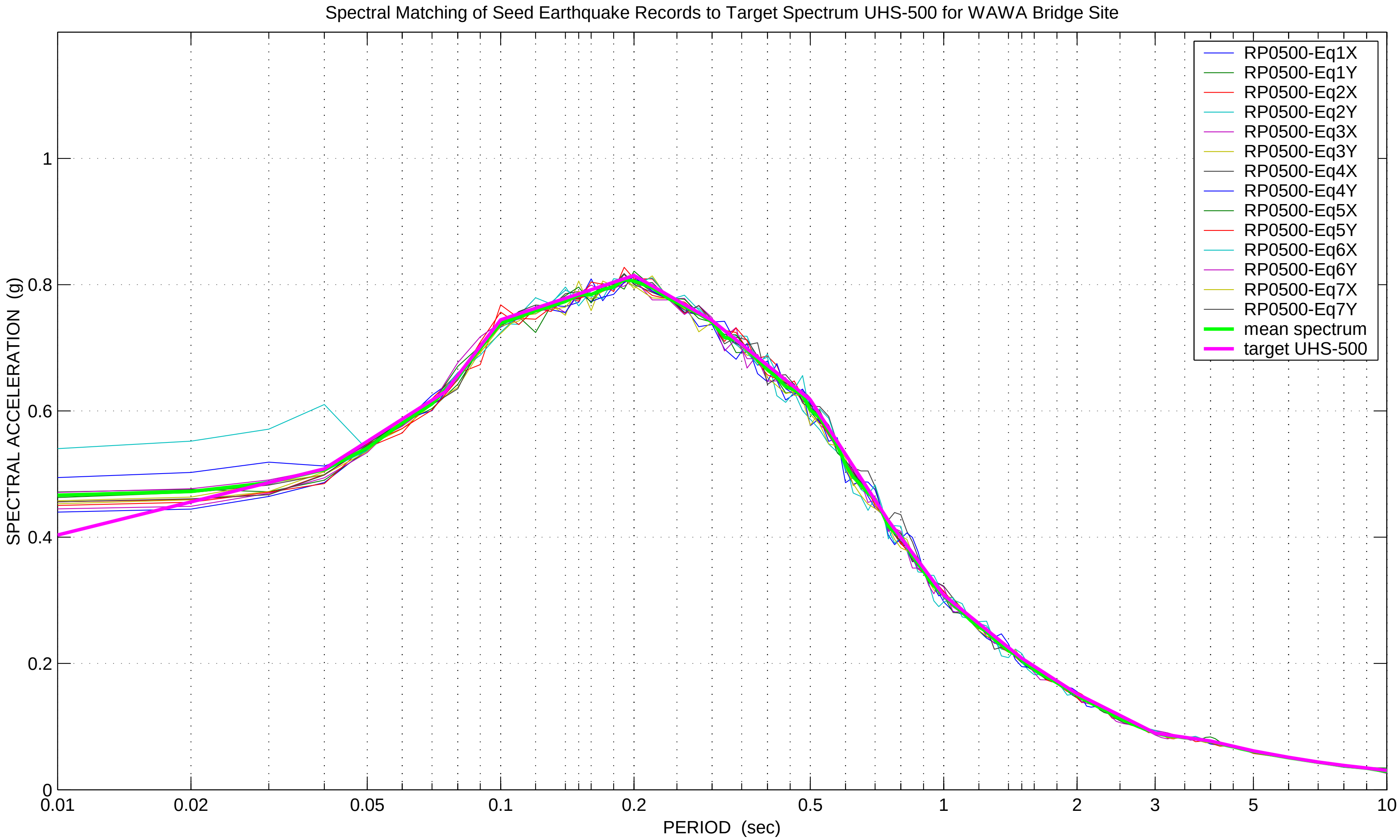


Figure 2B-191 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 500-year return period as compared to the target spectrum UHS-500 at Wawa Bridge

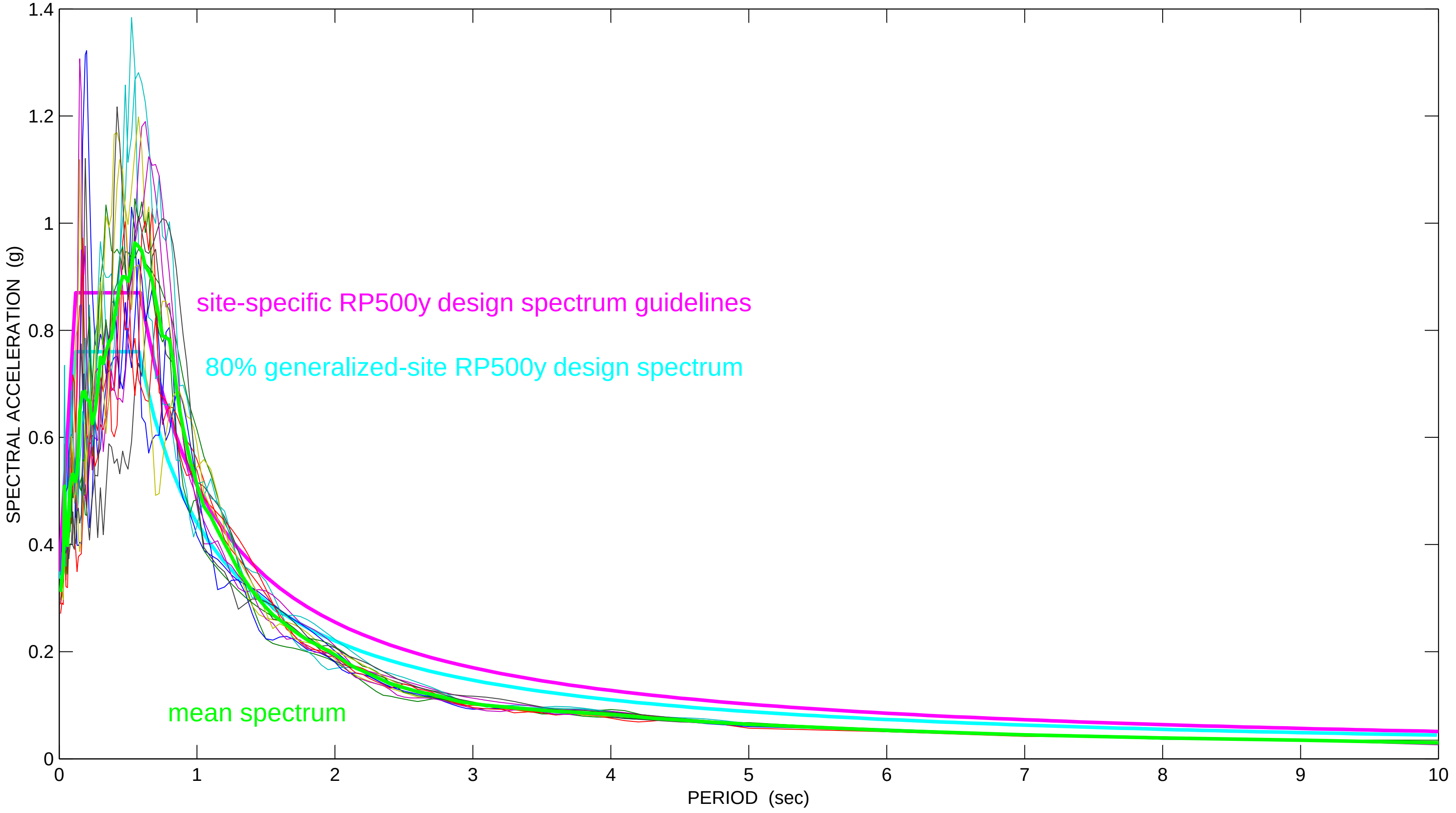


Figure 2B-192 Construction of site-specific design spectrum at 500-year return period for Wawa A1 based on mean of 14 response spectra from nonlinear site response analysis

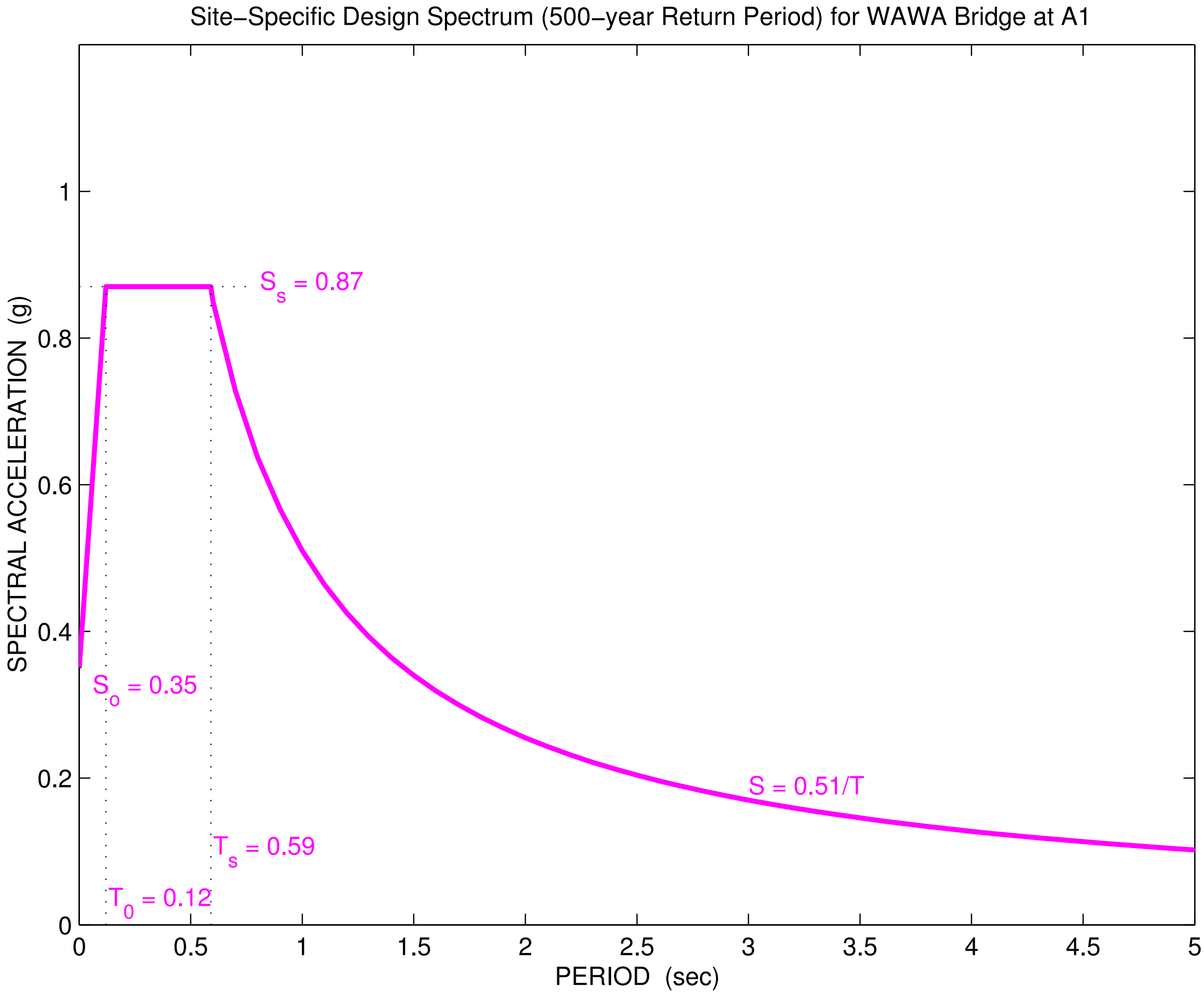


Figure 2B-193 Site-specific design spectrum at 500-year return period for Wawa Bridge at A1 site

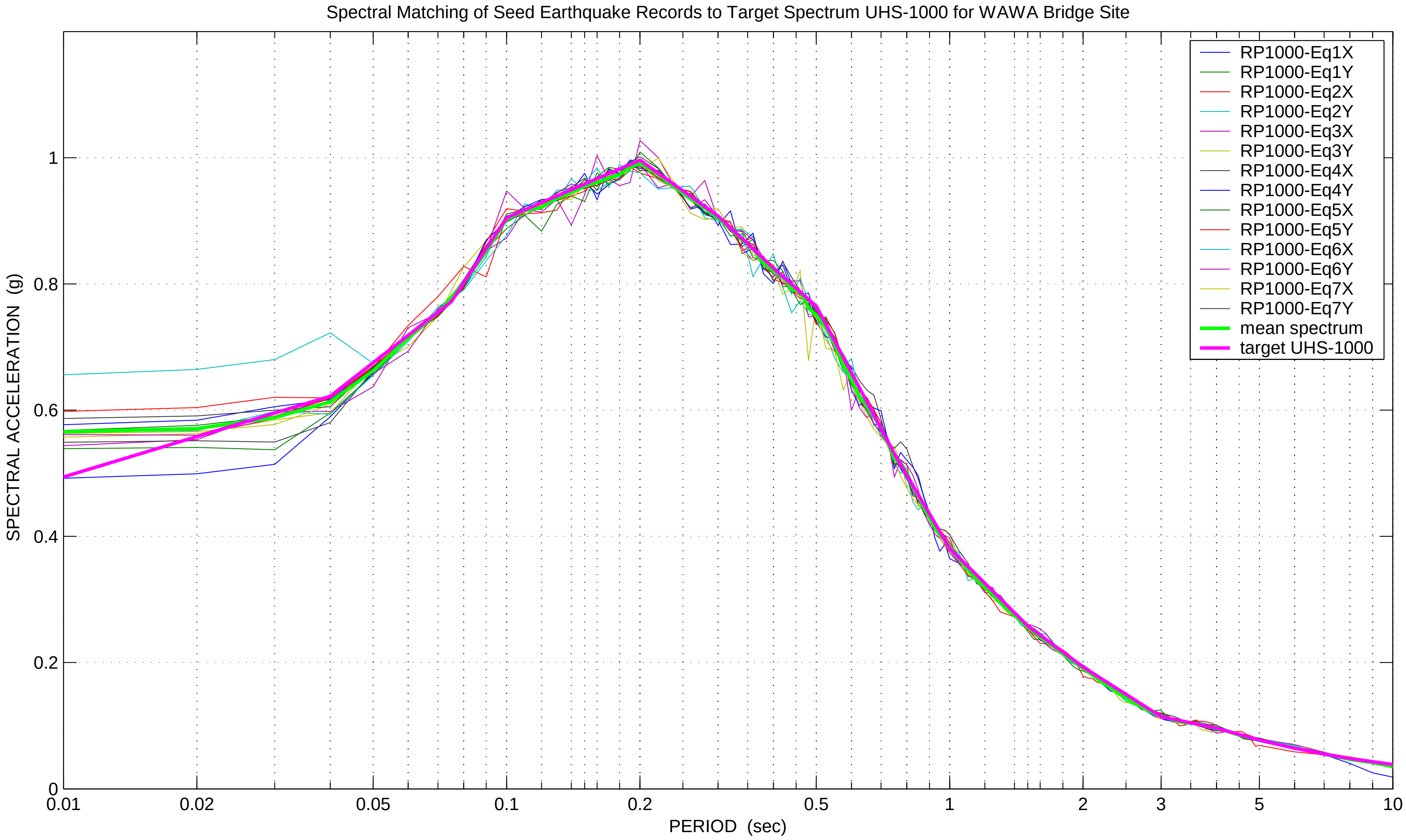


Figure 2B-194 Mean of $7 \times 2 = 14$ spectrally matched earthquake time histories for 1000-year return period as compared to the target spectrum UHS-1000 at Wawa Bridge

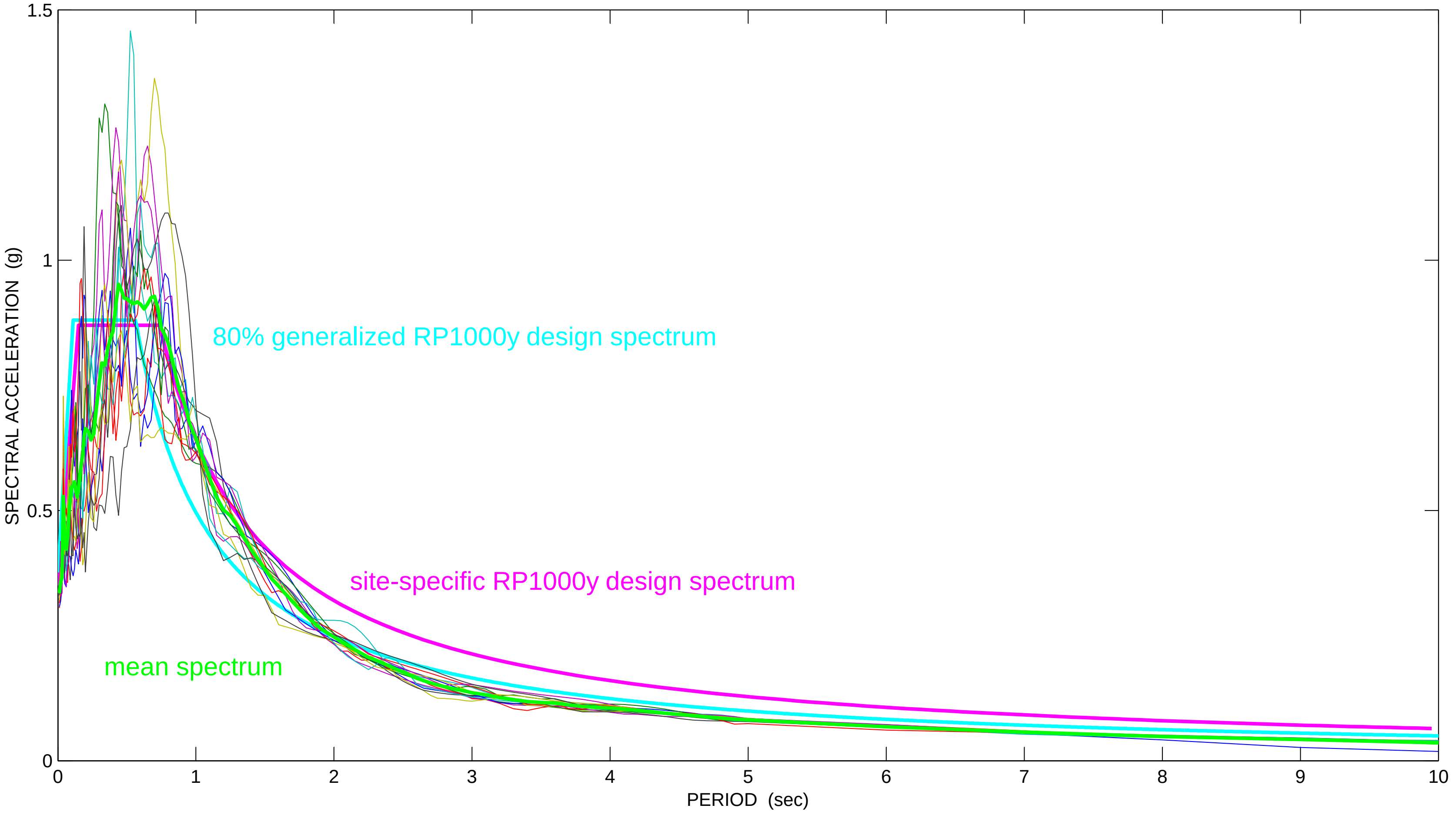


Figure 2B-195 Construction of site-specific design spectrum at 1000-year return period for Wawa A1 based on mean of 14 response spectra from nonlinear site response analysis

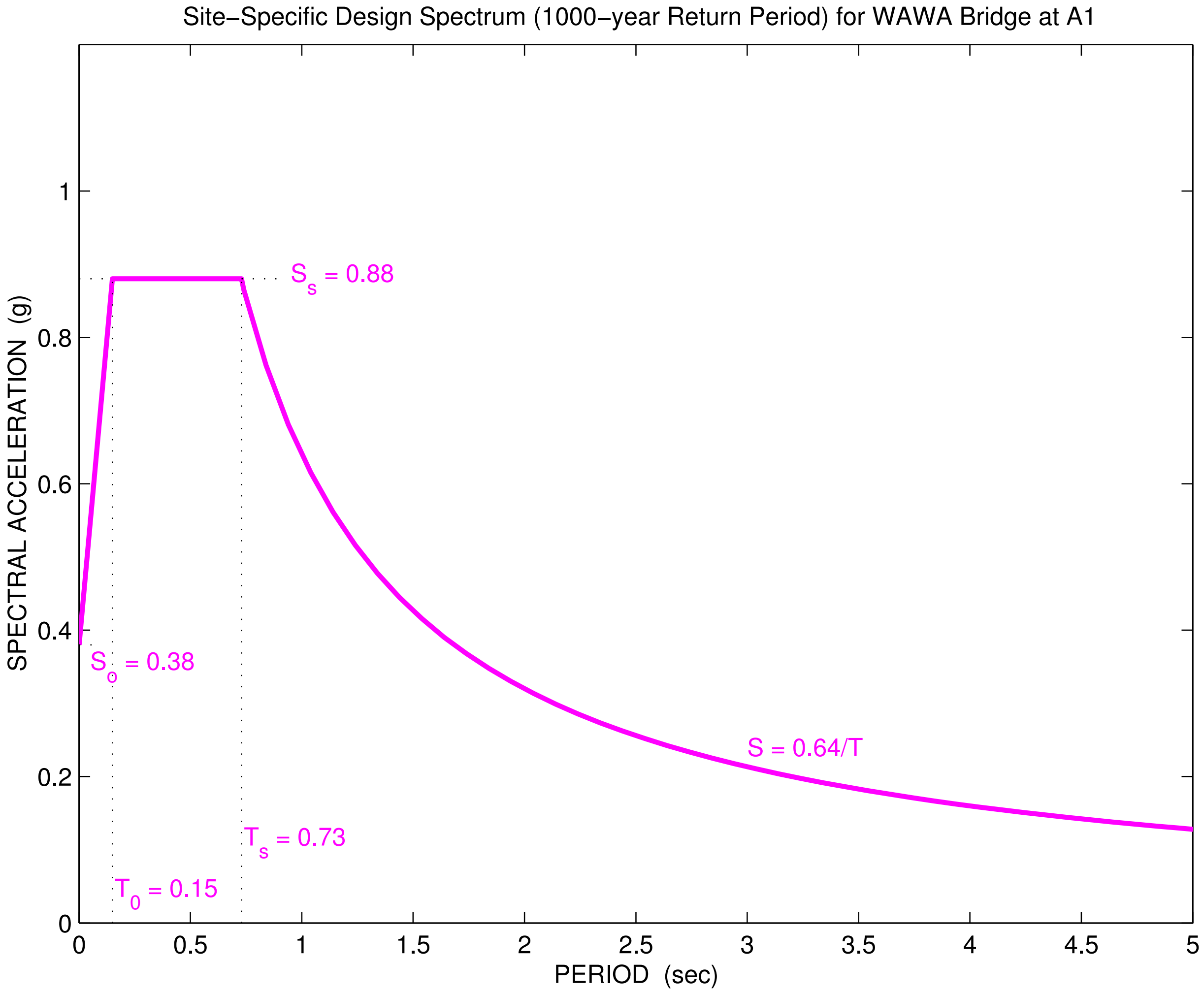


Figure 2B-196 Site-specific design spectrum at 1000-year return period for Wawa Bridge at A1 site

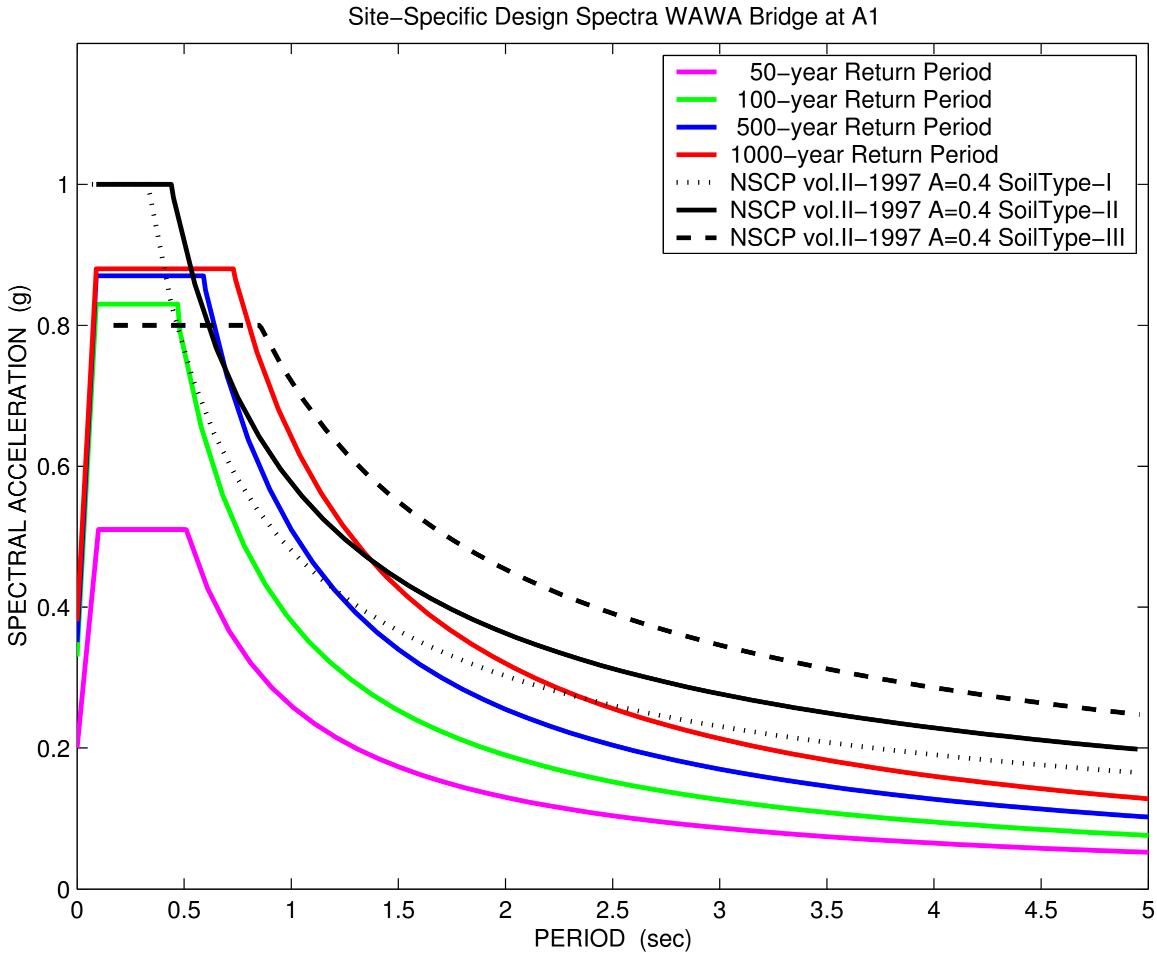


Figure 2B-197 Site-specific design spectra for Wawa Bridge at A1