Revised Seismic Design Criteria Report

Multnomah County | Earthquake Ready Burnside Bridge NEPA

Portland, OR

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Earthquake Ready Burnside Bridge Revised Seismic Design Criteria Report

Prepared for

Multnomah County
Transportation Division - Bridges
1403 SE Water Avenue
Portland, OR 97214

Oregon Division
Federal Highway Administration
530 Center Street NE, Suite 420
Salem, OR 97301

Prepared by

HDR
1050 SW Sixth Avenue, Suite 1700
Portland, OR 97204
T 503.423.3700

Parametrix
700 NE Multnomah, Suite 1000
Portland, OR 97232-4110
T 503.233.2400

Exeltech Consulting, Inc.
8729 Commerce Place Drive NE, Suite A
Lacey, WA 98516
T 360.357.8289

CERTIFICATION

The technical material and data contained in this document were prepared under the supervision and direction of the undersigned, whose seal, as a professional engineer licensed to practice as such, is affixed below.

______________________________Signature Reserved for Final Version

Prepared by Mike Lamont (Consultant Bridge Lead)

______________________________Signature Reserved for Final Version

Prepared by Rebecca Bautista (Consultant Bridge Engineer)

______________________________Signature Reserved for Final Version

Checked by Eric Rau (Consultant Bridge Engineer)

______________________________Signature Reserved for Final Version

Checked by Tony Messmer (Consultant Bridge Engineer)

______________________________Signature Reserved for Final Version

Approved by Steve Drahota (Consultant Project Manager)

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Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ARS	acceleration response spectra
BDM	ODOT Bridge Design Manual
BES	City of Portland Bureau of Environmental Services
C/D	Capacity-to-demand
City	City of Portland
Criteria	Seismic Design Criteria
County	Multnomah County, Oregon
CQC	Combined quadratic combination
CSO	Combined Sewer Overflow
CSZE	Cascadia Subduction Zone Earthquake
EQ	earthquake load
EQRB	Earthquake Ready Burnside Bridge Project
FHWA	Federal Highway Administration
FO	Full operation
FODE	Full operation design earthquake
fu	ultimate strength
fy	force to yield
GDM	Geotechnical Design Manual
ksi	kilopound per square inch
L	Longitudinal loading
LO	Limited operation
LODE	Limited operation design earthquake
LRFD	Load and Resistance Factor Design
M-ϕ	Moment curvature
Mne	Expected nominal moment capacity
NEPA	National Environmental Policy Act
ODOT	Oregon Department of Transportation
PL	Performance level
Project	Earthquake Ready Burnside Bridge Project
psi	Pounds per square inch
RSA	Response spectrum analysis
RX	Abutment longitudinal reaction
SDC	Seismic Design Criteria
T	Traverse loading
V	Vertical loading

1 Introduction

Developed for the Earthquake Ready Burnside Bridge Project, this Seismic Design Criteria (SDC) Report identifies the minimum requirements for seismic design for the NEPA Phase design assessment and that are necessary to meet the performance goals defined within this SDC. The Engineer must exercise judgment in the application of these criteria. Situations may arise that warrant detailed attention beyond what is provided in the SDC, including referring to the other design publications for seismic design criteria not explicitly addressed by the SDC.

Based on the NEPA phase seismic assessment results, this SDC can be further updated.

1.1 Bridge Systems

A bridge system consists of superstructure and substructure components. Common components and subcomponents are listed below:

Table 1. Bridge Components

Abutments	**Substructure Support Systems**
Diaphragm	Single Column
Short Seat	Multi Column
High Cantilevered Wall	Pier Walls
Wingwalls	Pile Extensions
Superstructures	**Foundations**
Cast-in-place	Spread Footings
- Reinforced Concrete	Driven Piles
- Pre-tensioned Concrete	Drill Shafts
Precast	Proprietary
- Reinforced Concrete	
- Pre-tensioned Concrete	**Miscellaneous**
- Post-tensioned Concrete	Bearings
Steel	Anchor Bolts
- Plate Girder	Restrainers
- Box Girder	Expansion Joints
- Rolled I-Girder	
- Trusses	

Traditionally, the entire bridge system has been referred to as the global system, whereas an individual bent or column has been referred to as a local system. It is preferable to define these terms as relative and not absolute measures. For example, the analysis of a bridge frame is global relative to the analysis of a column component but is local relative to the analysis of the entire bridge system.

1.2 Bridge Alternatives

The alternatives generally rely on a ductile substructure and essentially elastic superstructure 'Type 1' ERS as defined by the AASHTO Guide Spec, modified for the strain-based performance requirements that limit ductility as defined herein. The following are the bridge alternatives considered in the NEPA Phase seismic assessment:

- Enhanced Seismic Retrofit (Retrofit)
- Replacement Fixed Bridge on Existing Alignment (Fixed Bridge)
- Replacement Alternative with Short-span Approach (Short-span Alternative)
- Replacement Alternative with Long-span Approach (Long-span Alternative)
- Replacement Alternative with Couch Extension (Couch Extension)

2 Applicable Design Specifications

The design codes, specifications and guidelines listed below are applicable to this project. These documents are arranged in order of precedence. The provisions in the below codes and standards may be reconsidered if applicable.

1. This project specific "EQRB Seismic Design Criteria" (Criteria)
2. AASHTO *LRFD Bridge Design Specifications* (AASHTO LRFD)
3. AASHTO *Guide Specifications for LRFD Seismic Bridge Design* (AASHTO Guide Spec)
4. AASHTO *LRFD Movable Highway Bridge Design Specifications* (AASHTO Movable)

Additional design references include, but are not limited to (no order of preference):

- AASHTO Guide Specifications for Seismi*c Isolation Design* (AASHTO GSID)
- ODOT *Bridge Design Manual* (BDM)
- ODOT *Geotechnical Design Manual* (GDM)
- NCHRP Research Report 949 Proposed AASHTO Guidelines of Performance-Based Seismic Bridge Design (NCHRP)
- FHWA-HRT-06-032 *Seismic Retrofitting Manual for Highway Structures: Part 1-Bridges* (FHWA)
- AASHTO *Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements* (AASHTO FRPS)
- AASHTO *LRFD Bridge Construction Specifications* (AASHTO LRFDCONS)
- AASHTO *LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals* (AASHTO LRFDLTS)

The provisions in the above codes and standards may be reconsidered if applicable. <u>However, the provisions of this criteria document have precedence over those in the</u>

other documents. If the provisions in any of the above-listed codes and standards conflict, the order of precedence is according to their rank. Accordingly, document 1 governs over document 2; document 2 governs over document 3, and so on.

3 Functional/Safety Seismic Events and Performance Requirements

3.1 Performance Level Definitions

For the NEPA design evaluations, two performance levels are defined as follows:

Performance Level (FO): Full Operation (full functionality). Damage sustained is negligible. Essentially elastic for all primary structural components, movable spans remain operable to open and close. Only minimal, superficial repairs and maintenance activities will be required post-earthquake without interruption to traffic. All traffic modes are able to use the bridge, including river navigation, immediately after the earthquake.

Performance Level (LO): Limited Operation (limited functionality). Damage sustained is minimal. Limited inelastic behavior to substructure components; the bridge allows for emergency vehicles (after inspection and removal of debris). Movable components may not be operable without repairs. Damage is repairable but may impact traffic. Limited permanent deformation may occur.

The project-specific seismic performance requirements, expressed in terms of allowable damage, are further defined in Section 3.2.1 and 3.2.2.

3.2 EQRB Performance Requirements

EQRB is designated as the only County owned Primary Emergency Transportation Route[1] across the Willamette River in downtown Portland. Correspondingly, the bridge classification is "critical" according to AASHTO Movable, Section 3.3. Seismic assessment is based on two hazard assessment methods and corresponding minimum target performance. There are two levels of performance required, one at the Full Operation Design Earthquake (FODE) ground motion and one at the Limited Operation Design Earthquake (LODE) ground motion. Table 2 summarizes the performance requirements.

Table 2. Performance Requirements

Category	Full Operation Design Earthquake (FODE)	Limited Operation Design Earthquake (LODE)
Designated Performance Level (PL)	Full Operation (FO)	Limited Operation (LO)
Design Level Earthquake[a]	Full rupture of **Cascadia Subduction Zone Earthquake (CSZE)** (Deterministic EQ)	7% probability of exceedance in 75 years (**1,000-year return period**) (Probabilistic EQ)

[1] Reference: https://multco.us/file/64350/download

Table 2. Performance Requirements

Category	Full Operation Design Earthquake (FODE)	Limited Operation Design Earthquake (LODE)
Site-Specific Acceleration Response Spectra (ARS)	See Appendix A	See Appendix A

[a] The FODE and LODE level ground motions shall be characterized by Acceleration Response Spectra (ARS) that correspond to the site subsurface conditions and include near-fault effects as appropriate.

3.2.1 Full Operation Performance Requirement for FODE

For the FODE with full rupture of Cascadia Subduction Zone Earthquake (CSZE), the Performance Level is Full Operation (FO). The FO performance level requires negligible damage. FO requirements shall be defined as follows:

1. Negligible damage includes evidence of movement, and/or minor damage to nonstructural components, but no evidence of inelastic response in structural members or permanent deformations of any kind. (The bridge remains elastic for all main structural components.)
2. Bridge can be open to all traffic modes on the bridge deck immediately.
3. Moveable span is able to be operated as follows:

   (i) If in the closed position during the FODE, the span immediately allows all traffic modes on the bridge deck.

   (ii) If in the open position during the FODE, the span is immediately operable and able to close to allow all traffic modes on the bridge deck.

   (iii) Full open and close operability within 2 weeks following the FODE.

A winch system may be required to operate the bascule span leaves while the bascule span operation system is being repaired immediately after the earthquake. Spare parts for critical machinery and electrical components shall be prefabricated and stored on-site or near the bridge for emergency repair. It is expected that the time window for necessary repairs be up to 2 weeks.

Except for the bridge operating machinery, only non-structural repairs are expected. This may include limited concrete cover spalling on structural elements, small cracks on structural elements, or more significant cracks in non-structural concrete elements.

Quantitative definition for permissible displacements that allow the structure to meet the Full Operation performance requirements at key locations shall be studied further in the Preliminary engineering phase. During the NEPA phase, the following permissible displacements are targeted:

- Relative vertical displacements between the bascule leaf cantilever tips: within allowable limit of the span locks
- Differential settlement between the roadway approaches and the bridge: 1.0 inches

3.2.2 Limited Operation Performance Requirement for LODE

After designing for the FODE, check the bridge for a LODE with an earthquake event of a 1000-year return period. For this check, the bridge is regarded as "essential" (AASHTO Movable) and its specific Performance Level is Limited Operation (LO). The LO performance requirements exceed the "no-collapse" criteria requirements, and damage sustained should be minimal. LO requirements shall be defined as follows:

1. Minimal damage may include minor inelastic response, narrow flexural, and shear cracking in concrete. Permanent deformations are not apparent and repairs can be made under non-emergency conditions with the possible exception of superstructure expansion joints that may need removal and temporary replacement. (FHWA 1.4.1)
2. Certain elements may be permitted to fuse, provided it can be shown that such an occurrence will not reduce the vertical load-carrying capacity of the bridge or lead to superstructure unseating, and that the fusing of these elements will not preclude the structure from meeting the Limited Operation performance requirements. This could include anchor bolts.
3. Limited differential settlement between the bridge and approach roadways and roadway fill may be allowed.
4. Moveable span may not be operable without inspection or repairs to its components.
5. The bridge allows emergency vehicles to pass over the bridge (after inspection and removal of debris).
6. For capacity-protected structural members, no inelastic deformations are allowed. This includes superstructure, bent cap, crossbeam, footings, trunnion tower, and counterweight supporting members.
7. Except for movable operations, the time window for any necessary repairs is up to 2 weeks before opening the bridge to all traffic.
8. For movable operations, the time window for any necessary repairs is up to 2 months before opening the bridge to ship navigation traffic.

Quantitative definition for permissible displacements that allow the structure to meet the Limited Operation performance requirements identified above shall be studied further in the Preliminary engineering phase. During the NEPA phase, the following permissible displacements are targeted:

- Relative vertical displacements at the bascule leaf cantilever tips: within allowable limit of the span locks.
- Differential settlement between the roadway approaches and the bridge: 3.0 inches.

3.3 Ground Motions

3.3.1 Horizontal Ground Motion

For this bridge, site-specific seismic design acceleration response spectra (ARS) were developed by Shannon & Wilson, geotechnical engineer for this project, according to design specifications listed in Section 2, and the tools, source information and procedures specified in those specifications. See Appendix A Site-Specific Acceleration Response Spectra.

The complex geotechnical profile changes significantly along this bridge. Potential variations in the soil profile (including depth to rock) and dynamic soil properties (including shear wave velocity profiles) will be considered at each pier. The site-specific ARS developed by Shannon & Wilson are divided into three zones: Zone 1 from existing Bent 1 to Bent 18 (west approach), Zone 2 from Bent 19 to Bent 27 (river spans and east approach to 2nd Avenue), and Zone 3 from Bent 28 to Bent 35 (east approach from 2nd Avenue to abutment). Subsequently the zones were reduced to two zones (Bent 1 to Bent 27 and Bent 28 to Bent 35). An envelope and geometric mean ARS curve was developed for each of these two zones, for both CSZ spectra and 1000-year spectra (Appendix A).

Acceleration response spectra used in the NEPA phase seismic assessment will be based on individual spectra for each zone.

3.3.2 Vertical Ground Motion

In lieu of detailed analysis, the vertical acceleration response spectrum may be derived by using two-thirds (67 percent) of the horizontal response spectrum. A two-thirds ratio is slightly conservative for periods of vibration above 0.2 seconds, which covers the vast majority of bridge seismic response.

3.3.3 Time History Ground Motions

A nonlinear time history analysis is not required for the NEPA Phase seismic assessment.

4 Geological Hazard Considerations

4.1 Scour, Liquefaction, and Lateral Spreading Considerations

Liquefaction and lateral spreading potential of foundation soils will be evaluated and determined by the project geotechnical engineer. If the foundation soils are predicted to liquefy, the effects of liquefaction on design and performance evaluation should be according to ODOT Bridge Design Manual (BDM) Section 1.10.5 and 1.17.4.

Because the performance requirements for this bridge are more stringent than the required performance level stated in BDM, the acceptable lateral deformations of the abutment approach fills described in BDM Section 1.17.4, Note 1 and Note 2 are modified, targeted maximum acceptable lateral deformations of the foundation soil are:

- Full Operation Design Earthquake (FODE), excluding movable span: 6 inches
- Limited Operation Design Earthquake (LODE), excluding movable span: 12 inches

4.1.1 Effects on Performance Evaluation

The NEPA Phase assessment will consider the following effects of liquefaction:

- Complete loss in strength in the liquefied layer or layers
- Liquefaction-induced ground settlement and down drag
- Flow failures, lateral spreading, and slope instability
- Impact potential due to adjacent structures resulting from liquefaction-induced failures

4.1.2 Liquefaction Conditions

Liquefaction impacts on vertical and lateral loads on piles include vertical loading due to down-drag resulting from liquefaction-induced soil settlement and lateral loading resulting from liquefaction-induced lateral spreading. Liquefaction-induced down-drag may be accounted for by applying the side resistance of the soils above the lowest liquefiable layer in the soil profile as a negative (downward) load on the pile. Liquefaction-induced lateral spreading loads may be considered by treating the lateral displacement as a kinematic load and applying the calculated lateral displacement in each soil layer as a displacement to the base of the p-y spring representing that layer. The p-y resistance of the soil in the liquefied layers should be reduced in such an analysis.

The increased loads are generally decoupled from the seismic inertia loads, i.e., the liquefaction-induced soil settlement and lateral spreading are assumed to occur after the peak inertial loading. Note that the calculated lateral displacement of the pile cap or footing in such an analysis should be applied as a kinematic load to the bridge structure itself, but the displacement is usually assumed to occur after the peak inertial loading, and so this analysis is usually decoupled from the analysis for the inertial loads on the bridge. For purposes of NEPA Phase assessment, seismic inertial loads and liquefaction induced lateral spreading are combined as described below.

Liquefaction and lateral spreading affect the seismic response of bridge structures. In general, for bridges potentially subject to scour and/or liquefaction/lateral spreading, the effects of these conditions will be considered in performing lateral analyses of the bridges. The lateral analyses in the NEPA Phase assessment will be based on the probable maximum and minimum effects at the bridge site considering the following conditions:

1. To establish the critical condition for shear design: Perform lateral analysis assuming the soil is not susceptible to liquefaction and/or scour, using non-liquefied soil springs.

2. If a liquefiable soil layer exists at or near the ground surface: Perform lateral analysis using liquefied soil springs for the liquefiable layer and assume reduced or no soil springs for the soil above it, if appropriate.
3. Lateral spread effects will be evaluated by applying a lateral displacement as a kinematic load with reduced soil spring stiffness. The lateral spread effects shall be determined by the following combined load cases:
   - 100% Seismic Inertial + 0% liquefaction-induced soil / Lateral Spreading
   - 50% Seismic Inertial + 100% liquefaction-induced soil / Lateral Spreading

   Seismic Inertial = Earthquake Load as defined in Section 5.4 and 5.5.

5 Loads and Load Combinations

5.1 Load Factors and Combination

New bridge components will be designed for the applicable load combinations in accordance with the requirements of AASHTO LRFD.

The load effect will be obtained by:

$$\text{Load Effect} = \sum \eta_i \gamma_i Q_i$$

Where:

Q_i = force effect

η_i = a factor relating to ductility, redundancy, and operational importance

γ_i = load factor corresponding to Q_i

The load modifiers shall be according to AASHTO LRFD, Section 1.3.3, 1.3.4, and 1.3.5

The load factors shall be according to AASHTO LRFD, Section 3.4

5.2 Dead Load Consideration

Dead load will include the weight of all components of the structure, railing, sidewalk, appurtenances and utilities attached thereto, earth cover, future wearing surface, attached end panels and planned widening (if applicable).

- Load factors for all permanent loads γ_p shall be according to AASHTO LRFD, Tables 3.4.1-1, 3.4.1-2, and 3.4.1-3, for Extreme Event I load combination (seismic analysis) $\gamma_p = 1.0$

5.3 Live Load Consideration

The presence of live load will be considered in assessing the performance of the bridge elements during a seismic event.

- Load factor for live load in Extreme Event I load combination for the FODE shall be $\gamma_{EQ} = 0.5$
- Load factor for live load in Extreme Event I load combination for the LODE shall be $\gamma_{EQ} = 0.5$

Further research has supported the increase of live load to 0.5 from previously lower values recommended in the Feasibility Phase. Commonly used live load factors equal to 0.50 in combination with earthquake effects lead to conservative results (source: NCHRP Rep. 489). Additional research has concluded that for wide ranges of ADTT and congested roads, 0.5 is a reasonable factor.

For the NEPA Phase assessment, the presence of live load will be considered in assessing the performance of the bridge elements during a seismic event. Live loading will consist of vertical gravity loads only.

The magnitude of the load will be based on the AASHTO LRFD lane loading of 640 pounds per linear foot of lane. Application of the live loading in combination with other loads should be considered only when the live loads increase the demands on individual structural elements, and not applied when the live load decreases the demands due to seismic loads.

EQRB will carry more than three lanes; therefore, AASHTO LRFD live load multiple presence factors may be used as applicable.

The weight or equivalent mass due to live loads on the structure shall NOT be included in the inertial mass.

5.4 Earthquake Load

5.4.1 Fixed Spans

The earthquake load – ground motions and response spectra shall be considered for the FODE and LODE ground motions.

- Load factor for earthquake loads (EQ), shall be $\gamma_{EQ} = 1.0$

5.4.2 Movable Span in Closed Position

The same earthquake load as for the fixed span applies. In addition, for Extreme Event I load combination (AASHTO LRFD, Table 3.4.1-1), a combined vertical seismic acceleration and horizontal seismic acceleration analysis is required for both LODE and FODE ground motions (AASHTO Movable, 3.4.1).

- For LODE and FODE ground motion, load factor for EQ $\gamma_{\gamma_{EQ}} = 1.0$

5.4.3 Movable Span in Open Position

The same earthquake load as for the fixed span applies. In addition, for Extreme Event I load combination (AASHTO LRFD, Table 3.4.1-1), a combined vertical seismic acceleration and horizontal seismic acceleration analysis is required for both LODE and FODE ground motions (AASHTO Movable, 3.4.1).

- For LODE ground motion, load factor for EQ $\gamma_{EQ} = 0.5$
- For FODE ground motion, load factor for EQ $\gamma_{\gamma_{EQ}} = 1.0$

5.5 Earthquake Load in Orthogonal Directions

The acceleration response spectrum can be applied in three orthogonal directions along a set of global axes (Guide Spec 4.4), transverse (T), longitudinal (L), and vertical (V) loading. The longitudinal axis is typically represented by a horizontal chord connecting the two abutments and the transverse axis is perpendicular to the longitudinal axis.

Movable Bridge Spans - Earthquake effects from analysis in combined orthogonal directions shall be determined by the following cases:

Case I: 100 percent T + 30 percent L + 30 percent V

Case II: 30 percent T +100 percent L + 30 percent V

Case III: 30 percent T + 30 percent L + 100 percent V

Fixed Approach Spans - Earthquake effects from analysis in combined orthogonal directions shall be determined by the following cases:

Case I: 100 percent T + 30 percent L

Case II: 30 percent T +100 percent L

Vertical acceleration will not be applied to the fixed spans in the NEPA phase analysis, but rather applied later in the design phase when designing the superstructure.

5.6 Bascule Span Operating System

Design in accordance with AASHTO Movable, Section 3.4.3.

6 Structural Materials

6.1 Concrete

New concrete shall have a minimum specified 28-day compressive strength f´c of 4 ksi. For normal weight Portland cement concrete, the properties are calculated using the equations below.

Modulus of Elasticity, $E_c = 33 \times w^{1.5} \times \sqrt{f'_{ce}}\ (psi)$

Where w = unit weight of concrete in lb/ft3. For w = 145 lb/ft3

Shear Modulus, $G_c = \frac{E_c}{2\times(1+v_c)}$

Poisson's Ratio, $v_c = 0.2$

The analytical expected 28-day strength f'_{ce} shall be taken as 1.3 x f´c. For additional concrete modeling properties, such as limits on unconfined concrete compression strain and ultimate compressive strain for confined concrete using Mander's model, see Section 7.3.2.

6.2 Reinforcing Steel

New reinforcing steel properties shall be as provided in Table 3. These material properties are used in conjunction with strain values as defined in Section 8.1.3 for FODE and 8.2.3 for LODE which limit inelastic response (i.e. essentially elastic behavior) to achieve seismic performance objectives.

Table 3. Reinforcing Steel Properties

Property	Notation	Bar Size	ASTM A706 Grade 60	ASTM A706 Grade 80	ASTM A615 Grade 60	ASTM A432 HS Grade	ASTM A15, A408 Grade 40, A615 Grade 40
Modulus of elasticity (ksi)	E_s	No. 3 – No. 18	29,000	29,000	29,000	29,000	29,000
Specified minimum yield stress (ksi)	f_y	No. 3 – No. 18	60	80	60	60	40
Expected yield stress (ksi)	f_{ye}	No. 3 – No. 18	68	85	68	68	48
Specified tensile strength (ksi)	f_u	No. 3 – No. 18	80	98	90	90	70
Expected tensile strength (ksi)	f_{ue}	No. 3 – No. 18	95	112	95	95	81
Nominal yield strain	ε_y	No. 3 – No. 18	0.0021	0.0028	0.0021	0.0021	0.0014
Expected yield strain	ε_{ye}	No. 3 – No. 18	0.0023	0.0033	0.0023	0.0023	0.0017
Onset of strain hardening	ε_{sh}	No. 3 – No. 8	0.0150	0.0074	0.0150	0.0150	0.0193
		No. 9	0.0125		0.0125	0.0125	
		No. 10 & No. 11	0.0115		0.0115	0.0115	
		No. 14	00075		0.0075	0.0075	
		No. 18	0.0050		0.0050	0.0050	
Reduced ultimate tensile strain	ε_{su}^R	No.4 – No. 10	0.090	0.060	0.060	0.060	0.090
		No. 11 – No. 18	0.060		0.040	0.040	0.060
Ultimate tensile strain	ε_{su}	No.4 – No. 10	0.120	0.095	0.090	0.090	0.120
		No. 11 – No. 18	0.090		0.060	0.060	0.090

Notes:
Ksi (kips per square inch)
ASTM A305 prescribes requirements for bar deformations, not a material strength.
ASTM A408 material is similar to A15 but applied to #14 and #18 bars.
ASTM A432 HS Grade was the 60 ksi yield reinforcing steel used on some projects in the years immediately prior to ASTM A615 becoming the standard for bar reinforcing.
Source: Guide Spec 8.4.2

6.3 Prestressing Steel

Prestressing steel will be modeled with an idealized nonlinear stress strain model. Figure 1 is an idealized stress-strain model for 7-wire low-relaxation prestressing strand. The curves in the figure can be approximated by the equations below (Guide Spec 8.4.3).

Essentially elastic prestress steel strain, $\varepsilon_{ps,EE} = \begin{cases} 0.0076 \ for \ f_u = 250 \ ksi \\ 0.0086 \ for \ f_u = 270 \ ksi \end{cases}$

Reduced ultimate prestress steel strain, $\varepsilon^{R}_{ps,u}$ = 0.03

250 ksi Strand

$$\varepsilon_{ps} \leq 0.0076\text{: } f_{ps} = 28{,}500 \times \varepsilon_{ps} \ \ (ksi)$$

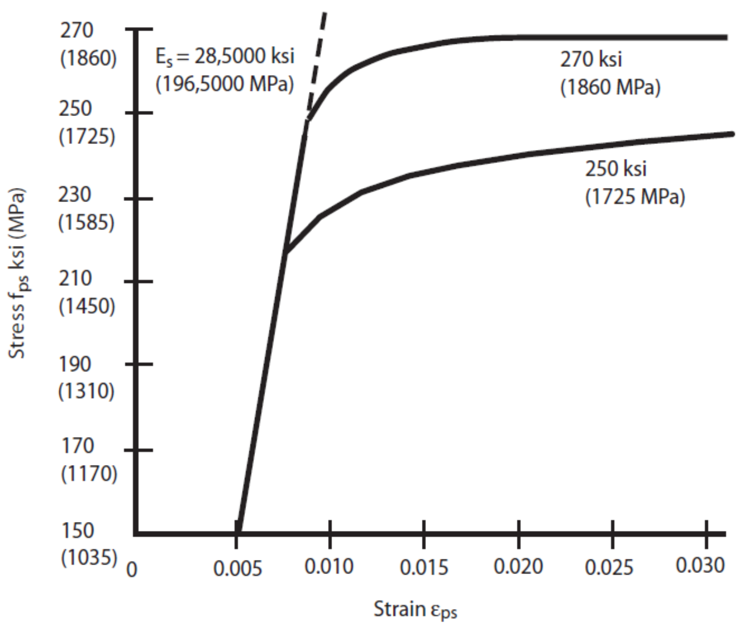
$$\varepsilon_{ps} \geq 0.0076\text{: } f_{ps} = 250 - \frac{0.25}{\varepsilon_{ps}} \ \ (ksi)$$

270 ksi Strand

$$\varepsilon_{ps} \leq 0.0086\text{: } f_{ps} = 28{,}500 \times \varepsilon_{ps} \ \ (ksi)$$

$$\varepsilon_{ps} \geq 0.0086\text{: } f_{ps} = 270 - \frac{0.04}{\varepsilon_{ps} - 0.007} \ \ (ksi)$$

Figure 1. Idealized Stress Strain Model for 7 wire Low-relaxation Prestressing Strand



6.4 Structural Steel

For new structural steel elements that are expected to remain elastic under the FODE and LODE, events, the following properties shall be used:

- Structural steel conforming to ASTM A709, Grade 50 or 50W shall be evaluated based upon a nominal yield strength Fy of 50 ksi and a nominal tensile strength Fu of 65 ksi.

- Structural steel conforming to ASTM A709, Grade 36 shall be evaluated based upon a nominal yield strength Fy of 36 ksi and a nominal tensile strength Fu of 58 ksi.
- Structural HSS shapes shall conform to ASTM A500, Grade B and shall be evaluated based upon a nominal yield strength Fy of 46 ksi for shaped tubes and 42 ksi for round tubes, and a nominal tensile strength Fu of 58 ksi, regardless of cross-sectional shape.

For new structural steel elements that are permitted to behave in a ductile manner under the LODE event, expected yield strengths shall be used to determine connection and other capacity-protected member force demand. Expected yield strengths shall be calculated by factoring the nominal yield strengths denoted above in accordance with Guide Spec 7.3.

6.5 Existing Materials

The original structure was built during 1924 – 1925, and the material properties were defined in Working Stresses in the as-built plans:

Concrete:

• Floor Slabs, Cross Girders, Cantilevers, Girders, Etc.	650 psi
• Beams continuous over Supports	815 psi
• Arch Rings Case 1: Not including Temperature and Wind	600 psi
• Arch Rings Case 2: Including Temperature and Wind	800 psi
• Bond for Steel in Concrete	100 psi
• Flexural Stress for all conditions not Including Wind	650 psi
• Flexural Stress for all conditions Including Wind	800 psi
• Columns Direct Compression	450 psi

Structural Steel:

• Tension, Net Section	16,000 psi
• Compression in Compression Members Fixed Ends	16,000 – 70L/r psi

The Main (River) Spans were rehabilitated in 2005 and the deck was replaced. The various material properties of the replaced structural components are specified in the plans of “Burnside Bridge Main Span Rehabilitation (#00511)” General Notes, Drawing No. 70380, dated July 2005.

During the Painting and Rehabilitation Project in 2017, some of the structural components were replaced or added. The material properties of those replaced or added structural components are specified in the plans of “Burnside St: Willamette River Bridge Painting and Rehabilitation Project”, General Notes, Drawing No. 98058, dated January 2017.

7 Determination of Demands and Capacities

7.1 Analysis Objective

The objective of seismic analysis is to assess the force and deformation demands and capacities on the structural system and its individual components. In the NEPA Phase assessment, the following will apply:

- A linear elastic dynamic analysis through Response Spectrum Analysis (RSA) is the appropriate analytical tool for estimating the force and displacement demands. The RSA models developed will be as described in Section 7.4.
- Simplified analysis described in Section 7.5.1 and as stated in Guide Spec Section 4.8 will be used to establish the displacement capacities for the approach bridges.
- Inelastic static pushover analysis is the appropriate analytical tool to establishing the displacement capacities for the piers where ductility design is applicable. A stand-alone local analysis will be used as described in Section 7.5.2.

7.2 Demands

For the NEPA phase assessment, estimate the earthquake demands by using the elastic RSA method, as described in Section 7.4.

When evaluating the existing structure: Use the lesser of elastic demands or overstrength demands.

When evaluating the replacement structure for the FODE analysis: Use the elastic demands with the strain limits of Section 8.

When evaluating the replacement structure for the LODE analysis: Use the elastic demands with the strain limits of Section 8.

For determination of force demands in capacity-protected elements, expected material properties are to be used. Overstrength factor is required as specified in Section 7.3.5. Strain limits of Section 8 are not applicable for overstrength evaluation of capacity-protected members.

7.3 Capacities

The capacities of the bridge globally and locally are generally independent of the ground motion. One exception to this is that column flexural strength is dependent on the axial load in the column, which varies with the lateral loads induced by ground motions. The sections below list some common seismic capacity evaluations. For capacity to demand acceptance criteria and other performance acceptance criteria, see Section 8.

7.3.1 Expected Versus Nominal Material Properties

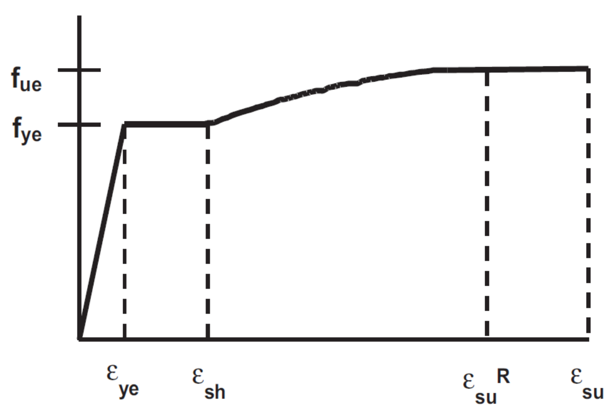
The capacity of concrete components to resist all seismic demands shall be based on the most probable (expected) material properties to provide a more realistic estimate for design strength. An expected concrete compressive strength, f'_{ce}, recognizes the typically conservative nature of concrete batch design, and the expected strength gain with age. The yield stress f_y for ASTM A706 steel can range between 60 ksi and 78 ksi. An expected reinforcement yield stress, f_{ye}, is a "characteristic" strength and better represents the actual strength than the specified minimum of 60 ksi. The possibility that the yield stress may be less than f_{ye} in ductile components will result in a reduced ratio of actual plastic moment strength to design strength, thus conservatively impacting capacity-protected components. Expected material properties shall only be used to assess capacity for earthquake loads.

7.3.2 Nonlinear Concrete Models for Ductile Concrete Members

Reinforcing steel shall be modeled with a stress-strain relationship that exhibits an initial linear elastic portion, a yield plateau, and a strain hardening range in which the stress increases with strain.

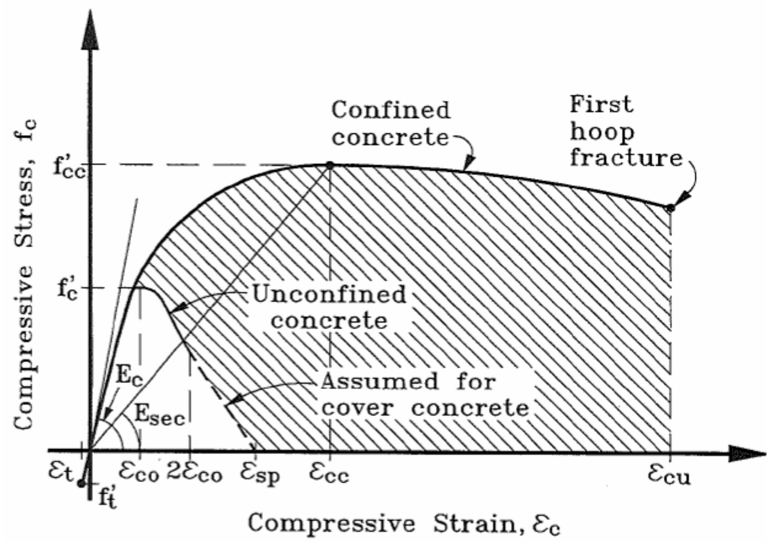
The yield point should be defined by the expected yield stress of the steel, f_{ye}. The length of the yield plateau shall be a function of the steel strength and bar size. The strain-hardening curve can be modeled as a parabola or other non-linear relationship and should terminate at the ultimate tensile strain, ε_{su}. The ultimate strain should be set at the point where the stress begins to drop with increased strain as the bar approaches fracture. It is common practice to reduce the allowable ultimate strain by up to thirty-three percent to decrease the probability of fracture of the reinforcement. The commonly used steel model is shown in below.

Figure 2. Steel Model



A stress-strain model for confined and unconfined concrete shall be used in the analysis to determine the local capacity of ductile concrete members. The initial ascending curve may be represented by the same equation for both the confined and unconfined model since the confining steel has no effect in this range of strains. As the curve approaches the compressive strength of the unconfined concrete, the unconfined stress begins to fall to an unconfined strain level before rapidly degrading to zero at the ultimate compressive strain of unconfined concrete (spalling strain), ε_{sp}. typically $\varepsilon_{sp} \approx 0.005$. The confined concrete model should continue to ascend until the confined compressive strength f'_{cc} is reached. This segment should be followed by a descending curve that is dependent on the parameters of the confining steel. The ultimate strain for confined concrete, ε_{cu}, should be the point where strain energy equilibrium is reached between the concrete and the confinement steel. A commonly used model is Mander's stress-strain model for confined concrete, shown in the figures below.

Figure 3. Mander's Stress-strain Model for Confined Concrete



For modeling purposes, the unconfined concrete compressive strain at the maximum compressive stress shall be taken as $\varepsilon_{co} = 0.002$ (Guide Spec 8.4.4). The ultimate unconfined compression strain based on spalling shall be taken as $\varepsilon_{sp} = 0.005$ (Guide Spec 8.4.4).

The concrete compressive strain at maximum compressive stress of confined concrete, ε_{cc}, and the ultimate compressive strain for confined concrete, ε_{cu}, should be computed using Mander's model. (Guide Spec 8.4.4)

$$f_c(x) = \frac{(f'_{cc})(x)(r)}{r - 1 + x^r}$$

$$f'_{cc} = f'_c \left[-1.254 + 2.254 \sqrt{1 + \frac{7.94 f'_l}{f'_c}} - 2\frac{f'_l}{f'_c} \right]$$

$$x = \frac{\varepsilon_c}{\varepsilon_{cc}}$$

$$\varepsilon_{cc} = \varepsilon_{co}\left[1 + 5\left(\frac{f'_{cc}}{f'_c} - 1\right)\right] = (0.002)\left[1 + 5\left(\frac{f'_{cc}}{f'_c} - 1\right)\right]$$

$$\mathrm{r} = \frac{\mathrm{E_c}}{\mathrm{E_c} - \mathrm{E_{sec}}}$$

$$\mathrm{E_c} = 60{,}000\sqrt{f_c'}\ \text{(psi)}$$

$$E_{sec} = \frac{f_{cc}'}{\varepsilon_{cc}}$$

$$\varepsilon_{cu} = 0.004 + \left[\frac{1.4 \times \rho_s \times f_{yh} \times \varepsilon_{su}}{f_{cc}'}\right]$$

Where:

$f_c(x)$ = function for predicting concrete stress at strain condition x

f_{cc}' = confined concrete compressive strength

x = ratio of concrete compressive strain at a given state to concrete compressive strain at maximum compressive stress

ε_c = concrete compressive strain at a given compressive stress

ε_{cc} = confined concrete compressive strain at maximum compressive stress

ε_{c0} = unconfined concrete compressive strain at maximum compressive stress

r = term representing the difference between the concrete modulus of elasticity and the secant modulus of elasticity for confined concrete

f_c' = nominal concrete strength (expected concrete strength, f_{ce}', is substituted for this term for evaluation of seismic performance)

f_l' = effective lateral confining stress (defined in the discussion that follows)

E_c = modulus of elasticity for concrete; Mander's formulation shown; for this project, however, *AASHTO LRFD* Equation 5.4.2.4-1 will be used, except that f_{ce}' (defined in Section 4), shall be substituted for f_c'

E_{sec} = secant modulus of elasticity for confined concrete

ρ_s = transverse (confinement) reinforcing area ratio

f_{yh} = yield strength of transverse reinforcing; expected yield strength, f_{ye} (defined in Section 4) is substituted for this term for evaluation of seismic performance

ε_{su} = ultimate tensile strain of transverse reinforcing; reduced ultimate tensile strain ε_{su}^R (defined in Section 5.2) is substituted for this term for evaluation of seismic performance

When $f_l' = 0$, the value of f_{cc}' will be equal to f_c' and the equations above produce results that are appropriate for unconfined concrete.

For circular sections, the effective lateral confining stress, f_l', is related to the average confining stress by the following expressions:

$$f_l^{'} = K_e f_l$$

$$f_l = \frac{2 \times f_{yh} \times A_{sp}}{s \times D^{'}}$$

Where:

A_{sp} = the cross-sectional area of typical transverse confinement reinforcing bar

s = the spacing of the transverse confinement reinforcing bars

$D^{'}$ = the diameter of the confined core, measured at the hoop or spiral centerline

For rectangular sections, with different transverse reinforcement area ratios, ρ_x and ρ_y, in the principal directions, different confining stresses are developed in accordance with the following relationships:

$$f_{lx}^{'} = K_e \times \rho_x \times f_{yh}$$

$$f_{ly}^{'} = K_e \times \rho_y \times f_{yh}$$

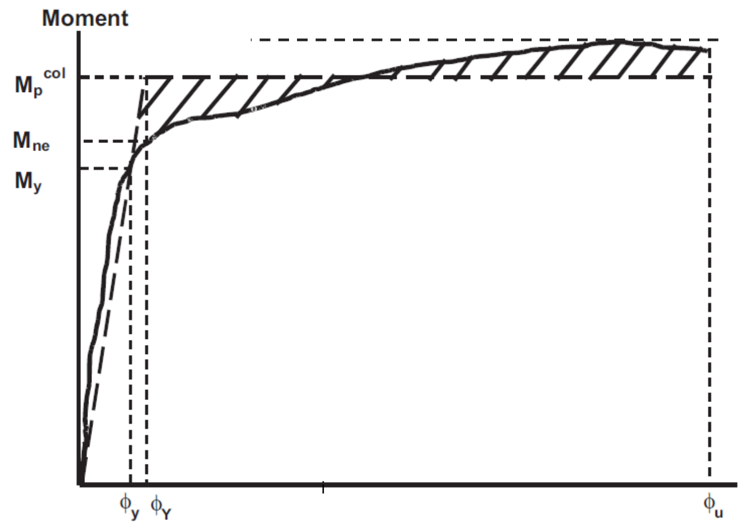
In the equations above, K_e is a confinement effectiveness coefficient. The typical values of K_e are 0.95 for circular sections, 0.75 for rectangular sections, and 0.6 for rectangular wall sections.

7.3.3 Moment Curvature Analysis

The plastic moment capacity of all ductile concrete members shall be calculated by moment-curvature analysis on the basis of the expected material properties. Moment curvature (M-ϕ) analysis derives the curvatures associated with a range of moments for a cross section based on the principles of strain compatibility and equilibrium of forces. The M-ϕ analysis shall include the axial forces due to dead load together with axial forces due to overturning. (Guide Spec 8.5)

The M-ϕ curves shall 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 through the point marking the first reinforcing bar yield. The idealized plastic moment capacity, Mp, shall be obtained by equating the area between the actual and the idealized M-ϕ curve beyond the first reinforcing bar yield point, as shown below. (Guide Spec 8.5)

Figure 4. Moment Curvature Curve



The ultimate curvature, ϕ_u, is determined as the smaller of:

- The ultimate compressive strain, ε_{cu}, of the confined concrete divided by the distance from the plastic neutral axis to the extreme fiber of the confined concrete core, or
- The reduced ultimate tensile strain, ε_{su}^{R}, of the reinforcing steel divided by the distance from the plastic natural axis to the extreme tension fiber of the longitudinal column reinforcement (Guide Spec 8.5).

7.3.4 Seismic Shear for Ductile Concrete Members

The Seismic Shear capacity analysis for seismic retrofit design shall follow AASHTO Guide Spec. This methodology is also consistent with other publications such as Priestly et al., ATC 32, MCEER/ATC 49, and Caltrans SDC.

- Explicit Shear Capacity for Ductile Concrete Members (Guide Spec 8.6)
  - For the capacity determination, see Section 8.2.1 for columns and Section 8.2.2 for pier walls.

7.3.5 Capacity-Protected Concrete Members

Capacity-protected concrete flexural components such as footings, pile shafts, crossbeams, joints and superstructure shall be designed to remain elastic when the ductile columns reach its overstrength moment demand. Excluding minimal inelastic components identified in Sections 8.1.3 and 8.2.3. See below for determination of overstrength factor.

The expected nominal moment capacity, M_{ne}, for capacity-protected concrete components determined by either M-ϕ or strength design, is the minimum requirement for essentially elastic behavior. Ductile behavior (hinging) is not permitted in capacity-protected members. Due to cost consideration a factor of safety is not required (i.e., Resistance factor ϕ = 1.0 for flexure). Expected material properties shall only be used to assess flexural component capacity for resisting earthquake loads. The material properties used for assessing all other load cases shall comply with the ODOT BDM.

Expected Nominal Moment Capacity

The expected nominal moment capacity, M_{ne}, is defined as the flexural strength of a reinforced concrete section when the extreme compression fiber of the section reaches a strain of 0.003 or the reinforcing steel strain reaches the reduced ultimate tensile strain, ε_{su}^R.

Overstrength Factor

The overstrength factor shall be based on the following methodology for assessment and design of new bridge elements.

The standard practice is to use the expected material properties to determine idealized plastic moment M_p^{col} (obtained during the moment-curvature analysis in Section 7.3.3) as the moment demand applied by the ductile column. When this method is used to determine the force demand on a capacity-protected member, an overstrength factor of 1.2 shall be used.

$$M_o^{col} = 1.2 \times M_p^{col}$$

7.3.6 Superstructure/Crossbeam

The nominal capacity of the superstructure longitudinally and of the crossbeam transversely must be sufficient to ensure that columns have the ability to become fully plastic prior to the superstructure or crossbeam reaching its expected nominal strength Mne, for seismic assessment. Longitudinally, the superstructure capacity shall be greater than the demand distributed to the superstructure on each side of the column by the largest combination of dead load moment, secondary prestress moment, and column earthquake moment. Crossbeams shall meet similar requirements.

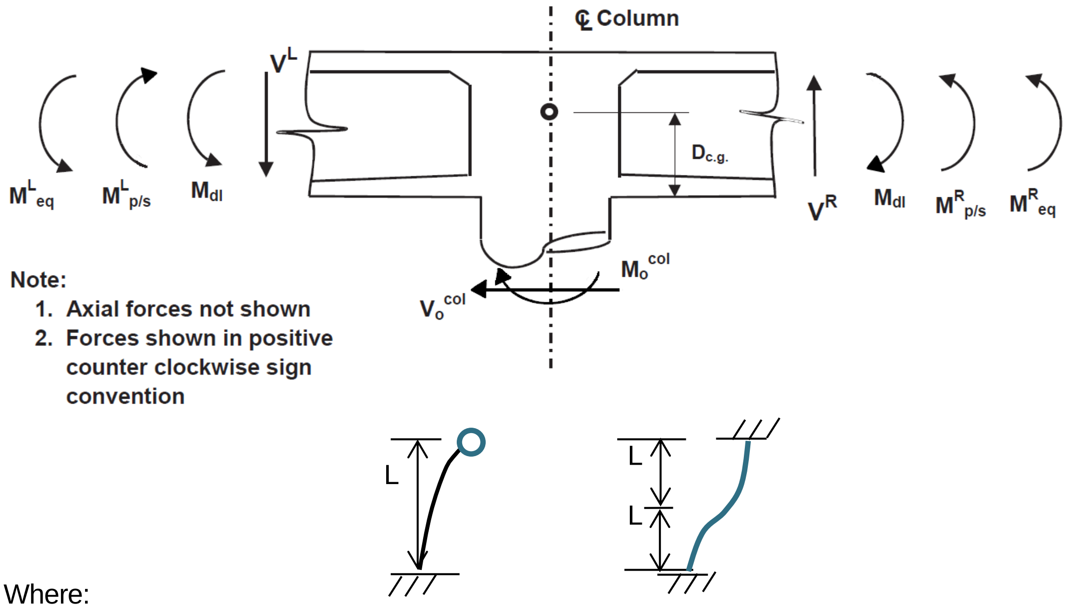
For span containing a hinge, the resisting moment on the hinge span side of the column shall not exceed the moment of the cantilever self-weight coupled with the reaction on the hinge times the distant to the hinge (the strength of the superstructure shall not be effective).

Any moment demand caused by dead load or secondary prestress effects shall be distributed to the entire frame. The distribution factors shall be based on cracked sectional properties of the superstructure crossbeam. The column earthquake moment represents the amount of moment induced by an earthquake, when coupled with the existing column dead load moment and column secondary prestress moment or the column's overstrength capacity, whichever is smaller. Subsequently, the column earthquake moment is distributed to the adjacent superstructure spans.

$$M_{ne}^{\sup\,(L)} \geq \Sigma\, M_{dl}^L + M_{p/s}^L + M_{eq}^L \qquad M_{ne}^{\sup\,(R)} \geq \Sigma\, M_{dl}^R + M_{p/s}^R + M_{eq}^R$$

$$M_o^{col} = M_{dl}^{col} + M_{p/s}^{col} + M_{eq}^{col} \qquad V_0 = \frac{M_o}{L}$$

$$M_{eq}^R + M_{eq}^L + M_{eq}^{col} + \left(V_o^{col} \times D_{c\,g}\right) = 0$$



Where:

$M_{ne}^{\text{sup}R,L}$ = Expected nominal moment capacity of the adjacent left or right superstructure span.

M_{dl} = Dead load plus added dead load moment (unfactored).

$M_{p/s}$ = Secondary effective prestress moment (after losses have occurred).

M_{eq}^{col} = The column earthquake moment when coupled with the existing column dead load moment and column secondary prestress moment, <u>*or*</u> the column's overstrength capacity, whichever is smaller.

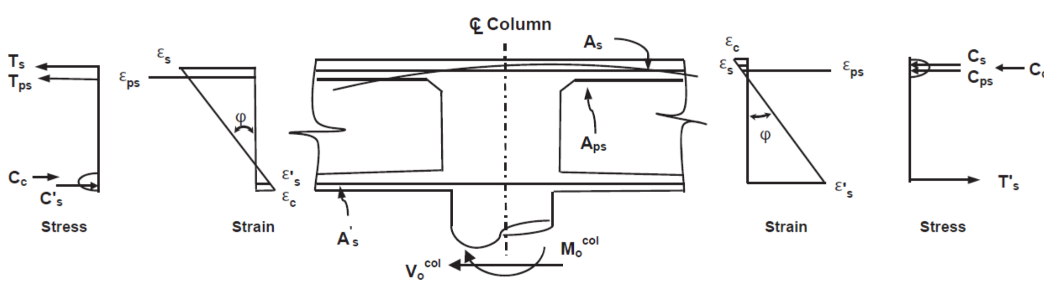
$M_{eq}^{R,L}$ = The portion of $M_{eq}^{col}$and $V_o^{col}{\times}D_{\,c\,g}$ (moment induced by the overstrength shear) distributed to the left or right adjacent superstructure span.

L = Member length from point of maximum moment to point of contra-flexure.

7.3.7 Longitudinal Superstructure Capacity

Reinforcement included in the deck, A_s and/or soffit A'_s contributes to the moment capacity of the superstructure, see the following figure. The effective width of the superstructure increases and the moment demand decreases with distance from the crossbeam.

Figure 5. International Resultant Force Couple



The superstructure shall be designed as a capacity-protected member. Any moment demand caused by dead load or secondary prestress effects shall be distributed to the entire width of the superstructure. The column overstrength moment M_o in addition to the moment induced due to the eccentricity between the plastic hinge location and the center of gravity of the superstructure shall be distributed to the spans framing into the bent on the basis of their stiffness distribution factors. This moment demand shall be considered within the effective width of the superstructure. The effective width of superstructure resisting longitudinal seismic moments B_{eff} shall be determined by the equations below. (Guide Spec 8.10)

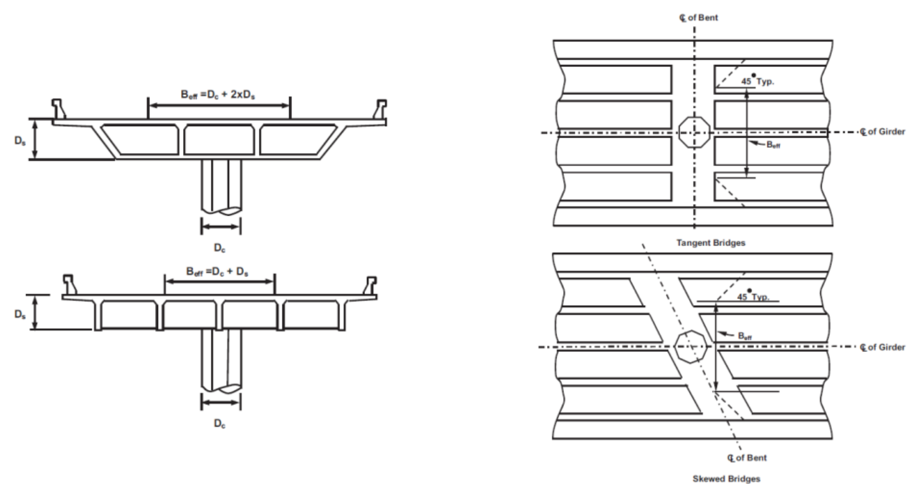
For box girders and solid superstructure: $B_{eff} = D_c + 2D_s$

For open soffit, girder-deck superstructures: $B_{eff} = D_c + D_s$

Where, D_c = diameter of column (in.)

D_s = depth of superstructure (in.)

Figure 6. Effective Superstructure Width



(Guide Spec Figure C8.10-1)

7.3.8 Crossbeam Capacity

Crossbeam reinforcement required for overstrength must be developed beyond the column cap joint. Crossbeams are considered integral if they terminate at the outside of the exterior girder and respond monolithically with the girder system during dynamic excitation. The crossbeam shall be designed as an essentially elastic member. Any moment demand caused by dead load or secondary prestress effects shall be distributed to the effective width of the crossbeam B_{eff} as shown in figure below.

The column overstrength moment M_o and the moment induced due to the eccentricity between the plastic hinge location and the center of gravity of the crossbeam shall be distributed on the basis of the effective stiffness characteristics of the frame. The moment shall be considered within the effective width of the crossbeam. The effective width, B_{eff} shall be determined by the equation below. (Guide Spec 8.11)

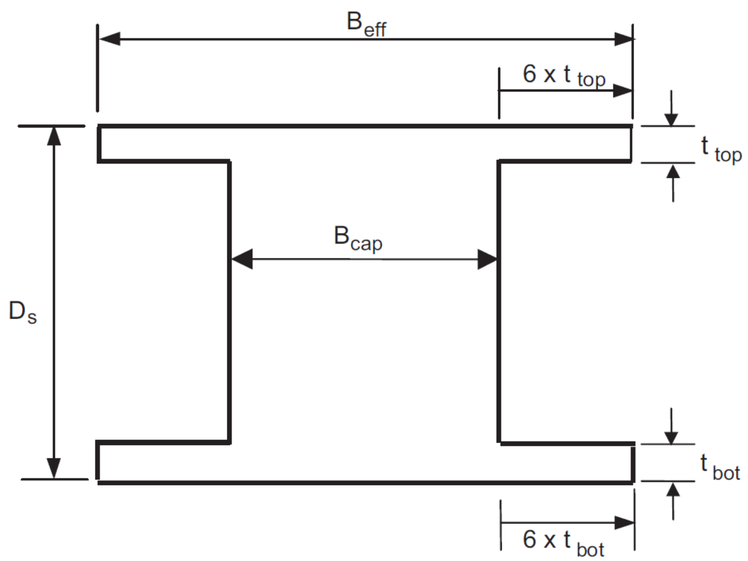
$B_{eff} = B_{cap} + 12t$

Where,

t = thickness of the top or bottom slab (in.)

B_{cap} = thickness of the crossbeam/bent cap (in.)

Figure 7. Effective Bent Cap Width



(Guide Spec Figure 8.11-1)

7.3.9 Footing or Drilled Shaft Capacity

The foundation must have sufficient strength to ensure the column has moved well beyond its elastic capacity prior to the foundation reaching its expected nominal capacity.

7.4 Response Spectrum Analysis

RSA will be used for global model analysis to determine mode shapes, structure periods and estimated seismic force and displacement demands. The RSA is also known as the linear elastic multimode spectral analysis (FHWA 5.4.2.2, AASHTO LRFD 4.7.4.3.3), dynamic response spectrum analysis, or elastic dynamic analysis (Guide Spec 5.4.3).

ARS curves with 5 percent damping will be used. Modal responses will be combined using the complete quadratic combination (CQC) method (AASHTO LRFD 4.7.4.3.3, Guide Spec 5.4.3, and Guide Spec C4.4).

Models included in the NEPA Phase assessment:

Model I: A global RSA will be performed on the West Approach spans independent of the rest of the structure.

Model II: A global RSA will be performed on the East Approach spans independent of the rest of the structure.

Model III: An independent localized RSA will be performed on the trunnion pier for the movable span, independent of the approach spans.

7.4.1 Model Orientation

The Engineer is responsible for selecting the orientation of the two orthogonal axes that will represent longitudinal and transverse directions of seismic motion for the RSA. In general, the selection will be made from one of the following:

1. Orientation described in Section 5.5 above.
2. For a given frame, the longitudinal axis will be oriented along a line connecting the centerline of bridge at the first bent in the frame and the centerline of bridge at the last bent in the frame. The transverse axis will be perpendicular to the longitudinal axis.
3. For skewed structures, the orientation of the longitudinal and transverse motion may be rotated to be parallel to weak and strong axes, respectively, of the intermediate supports.

7.4.2 Modeling Requirements

The RSA model(s) will contain sufficient detail to assess the anticipated behavior of the structure in a seismic event. Accordingly, the model(s) will contain a sufficient number of degrees of freedom, nodes, and number of modes to capture at least 90 percent mass participation in the longitudinal and transverse directions (Guide Spec 5.4.3). The number of modes included in the analysis should be at least three times the number of spans in the model (AASHTO LRFD 4.7.4.3.3). For most bridges, an RSA model that is assigned 4 segments per column and 10 segments per span for superstructure is sufficient to meet this criterion.

7.4.3 I_{eff} for Ductile Members

The RSA based on design spectral accelerations will likely produce stress in some elements that exceed their elastic limit. The presence of such stresses indicates nonlinear behavior. The Engineer should recognize that forces generated by linear elastic analysis could vary considerably from the actual force demands on the structure.

For the FODE analysis: The column flexural and torsional stiffness properties may be reduced down to no less than 50% of the gross section properties (to reflect some cracking) if deemed appropriate by the Engineer. (AASHTO LRFD C4.7.1.3).

For the LODE analysis: Column sections shall be modeled using equivalent cracked section properties, as the structure is expected to behave inelastically during those analyses. In lieu of moment curvature analysis, for this phase, cracked section properties in the plastic hinge zones shall be estimated by applying a 35% modification factor on the gross section properties. Between plastic hinge zones, 50 percent of the column flexural and torsional stiffness properties will be used (FHWA 7.3.2.1).

7.4.4 I_{eff} for Superstructures

I_{eff} in box girder superstructures is dependent on the extent of cracking and the effect of the cracking on the element's stiffness. I_{eff} for reinforced concrete box girder sections may be estimated between 0.5 I_g – 0.75 I_g, if deemed appropriate by the Engineer. The lower bound represents lightly reinforced sections and the upper bound represents heavily reinforced sections. (FHWA 7.3.2.1, Guide Spec 5.6.3)

For prestressed concrete members, the location of the prestressing steel's centroid and the direction of bending have a significant impact on how cracking affects the stiffness. Multi-modal elastic analysis is incapable of capturing the variations in stiffness caused by moment reversal. Therefore, no stiffness reduction is recommended for prestressed concrete box girder sections (FHWA 7.3.2.1, Guide Spec 5.6.3,).

For the FODE analysis: Superstructure sections shall be modeled using gross section properties, as the structure is expected to behave essentially elastically under the FODE response spectrum analysis.

For the LODE analysis: Reductions to I_g similar to those specified for box girders can be used for other superstructure types and cap beams. A more refined estimate of I_{eff} based on moment curvature analysis will not be conducted in this phase.

7.4.5 Effective Torsional Moment of Inertia

A reduction of the torsional moment of inertia is not required for the bridge superstructure in this phase.

Because the torsional stiffness of concrete members can be greatly reduced after the onset of cracking, the torsional moment of inertia for columns may be reduced by the equation below (Guide Spec 5.6.5) for both the FODE and LODE analysis.

$$J_{eff} = 0.2 \times J_g$$

Where:

J_{eff} = effective torsional (polar) moment of inertia of reinforced concrete (in^4)

J_g = gross torsional (polar) moment of inertia of reinforced concrete (in^4)

7.4.6 Boundary Conditions

Boundary conditions will be included in the model to represent the behavior of the structure supports and interconnection of member elements. Where a component or boundary condition may behave in a nonlinear manner, an iterative solution is required as prescribed below.

7.4.7 Abutments

Longitudinal Abutments Response

The backfill passive pressure force resisting movement at the abutment varies nonlinearly with longitudinal abutment displacement and is dependent upon the material properties of the backfill. Abutment spring stiffness is estimated through abutment longitudinal response analysis using a bilinear approximation of the force-deformation relationship. The bilinear demand model shall include an effective abutment stiffness that accounts for expansion gaps and incorporates a realistic value for the embankment fill response. The geotechnical professional shall be responsible to provide recommendation for the initial stiffness K_i. In case the geotechnical recommendation is not available, based on passive earth pressure tests and the force deflection results from large-scale abutment testing, the initial stiffness K_i may be estimated between 10 kip/in/ft to 50 kip/in/ft for soils ranging from loose sand to dense sand. A reasonable starting point for the initial stiffness may be taken at 50 kip/in/ft.

The initial stiffness shall be adjusted proportional to the backwall/diaphragm height.

$$K_{abut} = K_i \times w \times \left(\frac{h}{5.5\ ft}\right)$$

For seat-type abutments, the effective abutment wall stiffness shall account for the expansion hinge gaps as shown in the figures below. Based on a bilinear idealization of the force-deformation relationship, the passive pressure force resisting the movement at the abutment (P_{bw} or P_{dia}) is calculated with the following equation. The maximum passive pressure of 5.0 ksf is based on the ultimate static force developed in the full-scale abutment testing. The height proportionality factor, $\frac{h}{5.5\ ft}$ is based on the height of the tested abutment walls.

$$P_{bw}\ or\ P_{dia} = A_e \times 5.0\ ksf \times \left(\frac{h_{bw}\ or\ h_{dia}}{5.5}\right)$$

Where:

$A_e = h_{bw} \times w_{bw}$ for Seat Abutment, or

$A_e = h_{dia} \times w_{dia}$ for Diaphragm Abutment

$h_{dia} = h_{dia}^{*}$ Effective height if the diaphragm is not design for full soil pressure as shown in figure below

$h_{dia} = h_{dia}^{**}$ Effective height if the diaphragm is design for full soil pressure as shown in figure below

$w_{bw}, w_{dia}, w_{abut} =$ Effective abutment width

Figure 8. Effective Abutment Stiffness

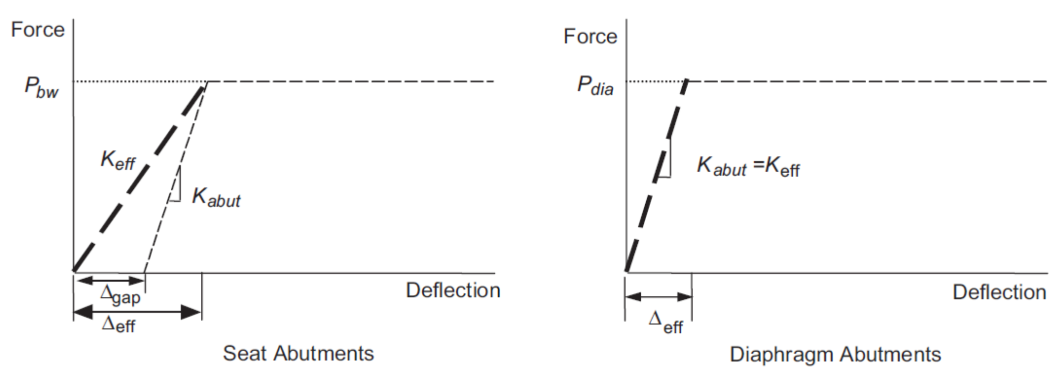


Figure 9. Effective Abutment Area

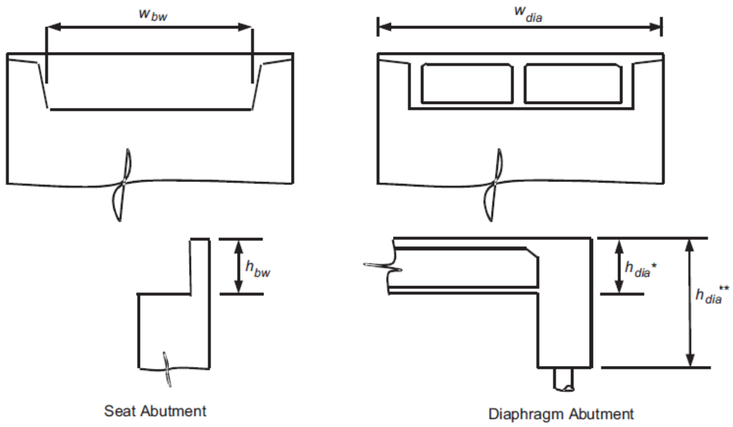
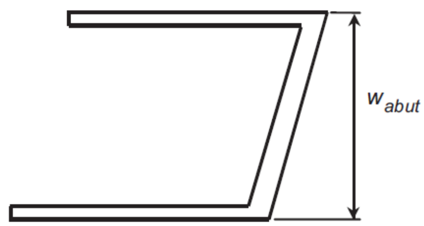


Figure 7.14B Effective Abutment Area



The longitudinal abutment spring magnitude shall be iterated for force convergence if computed abutment forces exceed the soil capacity. The stiffness should be softened iteratively (K_{eff1} to K_{eff2}) until the abutment displacement are consistent (within 30 percent) with the assumed stiffness (Guide Spec 5.2.3.3.2). The suggested spring iteration procedure is as following:

Step 1. The longitudinal abutment springs shall be modeled with two separate springs, one at each end of the bridge. Each with the stiffness magnitude equal to $\frac{K_{eff1}}{2}$, where the K_{eff1} is the initial spring stiffness described previously in the equation above.

Step 2. Run the Response Spectrum Analysis (RSA), and check the abutment longitudinal reaction, RX, against the abutment passive pressure resisting capacity, P_{bw} or P_{dia}, as describe previously in the equation above.

Step 3. If the abutment longitudinal reaction (RX) is smaller than the abutment capacity (P_{bw} or P_{dia}), iteration is not required. The abutment stiffness magnitude can be used as modeled. Each spring will have a longitudinal spring stiffness of K_{eff1}.

Step 4. However, if the abutment longitudinal reaction (RX) is greater than the abutment capacity (P_{bw} or P_{dia}), iteration is required. Reduce the longitudinal abutment springs stiffness magnitude and re-run the Response Spectrum Analysis.

Step 5. Re-check the new abutment longitudinal reaction (RX) against the abutment resisting capacity, to see if the reaction demand is similar in magnitude as the resisting capacity (within approximately 10%).

Step 6. Iterate the spring stiffness either up or down until the abutment longitudinal reaction (RX) and the abutment resisting capacity reaches convergence.

For bridges with unusual geometry or differing connectivity at each abutment, it may be necessary to produce multiple RSA models, each with an appropriate full-stiffness spring at only one abutment, in order to capture directionally-dependent differences in behavior.

Longitudinal springs shall be orientated perpendicular to the abutment backwall.

Transverse Abutment Response

Abutments are designed to resist transverse service load and moderate levels of ground motion elastically. Linear elastic analysis cannot capture the inelastic response of the shear keys, wingwalls, or piles that may occur during higher level ground motion. The transverse capacity of an abutment foundation should be considered effective for the design seismic hazards and should include force-deflection characteristics and stiffness for each element that contributes to the transverse resistance.

7.4.8 Bents and Piers

In the NEPA phase assessment, the RSA model(s) will not use soil foundation springs at intermediate bents, but rather assume a depth to fixity beyond the liquefied zone. Initially, four pile diameters will be assumed to achieve a fixed depth in the pile/shaft. The Engineer shall coordinate with the Geotechnical Engineer to calibrate this depth.

The preliminary engineering phase will require a more detailed level of modelling and will require foundation springs at the intermediate bents (unless not required by the BDM). The Engineer shall coordinate with Geotechnical Engineer during that phase.

The combined load case (1.0L and 0.3T) shall be assumed for the design of structural members only, and not applied when determining foundation response. For the simple case of a regular bridge with no skew, the longitudinal shear and moment are the result of the seismic longitudinal load, and the transverse components are ignored. This is somewhat inexact for highly skewed piers or curved structures with rotated springs, but the principle remains the same.

7.4.9 Tension and Compression Models

Global dynamic analyses are required to capture the assumed nonlinear response of a bridge because it possesses different characteristics in tension versus compression. When hinges or other superstructure structural discontinuities are present in a multi-span bridge, both compression and tension models are necessary to capture the maximum seismic force and displacement effects. (Guide Spec 5.1.2).

A compression model is a continuous model in which the hinges are considered closed/deactivated and restrained. The superstructure elements are locked longitudinally to capture structural response modes where the joints close up, and the abutments are mobilized.

A tension model frees a number of degrees of freedom at the joint location(s) to produce greater relative displacement at hinge/support locations. This is modeled to capture the effects of an open hinge or restrainers.

7.4.10 Equal Displacement Rule

The equal displacement rule is a common approximation used for the analysis of bridges that states that the peak displacement amplitude for a structure responding inelastically is equal to the peak displacement amplitude calculated for the same structure responding elastically. The equal displacement rule is not theoretically based; instead, it is an observation made from experimental and analytical studies.

7.5 Pushover Analysis

The pushover method is also known as the Nonlinear Static Procedure. A nonlinear inelastic static pushover analysis, with considerations for geometric nonlinearity (second-order effects, P-Δ effects), should be used for the determination of the seismic displacement capacity, Δ_C, for LODE and ground motion. (FHWA 5.6)

The pushover analysis may be a stand-alone local analysis of a bent or the pushover analysis may be performed on a global model of an entire bridge.

A pushover analysis without geometric nonlinearity (P-Δ effects) is acceptable if the column exhibits sufficient levels of base shear and provided the equation below is satisfied:

$P_{dl} \times \Delta_r \leq 0.20 \times M_p^{col}$ (Guide Spec 4.11.5)

Where:

P_{dl} = the dead load on top of the pushover column

Δ_r = the relative lateral offset between the point of contra-flexure and the base of the plastic hinge

M_p^{col} = the plastic moment strength of the column

The column plastic moment capacity, M_p^{col}, shall be obtained using the idealized plastic moment capacity determination process through the moment-curvature process described in Section 7.3.3. The moment-curvature results shall utilize the moment-rotation, and “equivalent cracked” moment of inertia properties of column and crossbeams for pushover analysis.

The pushover analysis model may consist of an individual local pier/bent model to determine transverse displacement capacities of individual bents. A longitudinal model may also be required to determine displacement capacities of columns in the longitudinal direction of the bridge. Moment-rotation information shall be incorporated to capture the moment-curvature behavior of the ductile members. The component demands due to dead load shall be applied at the initial stage of the pushover model. The pushover analysis shall include sufficient finite step-increments to capture formation of the first plastic hinge and shall proceed until the first hinge reaches its ultimate capacity, which will define the displacement capacity.

7.5.1 Simplified Analysis

For simple piers and bents, a hand calculation can be performed to verify the pushover analysis local displacement capacity result (Guide Spec C5.4.3). The following equations illustrate the definition of moment-curvature properties and the relationship used to calculate global displacement capacity:

Cantilever column with fixed base:

$$\Delta_c = \Delta_Y^{col} + \Delta_p$$

$$\Delta_Y^{col} = \frac{L^2}{3} \times \phi_Y$$

$$\Delta_p = \theta_p \times \left(L - \frac{L_p}{2}\right)$$

$$\theta_p = L_p \times \phi_p$$

$$\phi_p = \phi_u - \phi_y$$

Framed column (fix-fix condition)

$$\Delta_{c1} = \Delta_{Y1}^{col} + \Delta_{p1} \qquad \Delta_{c2} = \Delta_{Y2}^{col} + \Delta_{p2}$$

$$\Delta_{Y1}^{col} = \frac{(L_1)^2}{3} \times \phi_{Y1} \qquad \Delta_{Y2}^{col} = \frac{(L_2)^2}{3} \times \phi_{Y2}$$

$$\Delta_{p1} = \theta_{p1} \times \left(L_1 - \frac{L_{p1}}{2}\right) \qquad \Delta_{p2} = \theta_{p2} \times \left(L_2 - \frac{L_{p2}}{2}\right)$$

$$\theta_{p1} = L_{p1} \times \phi_{p1} \qquad \theta_{p2} = L_{p2} \times \phi_{p2}$$

$$\phi_{p1} = \phi_{u1} - \phi_{y1} \qquad \phi_{p2} = \phi_{u2} - \phi_{y2}$$

Where:

$L =$ distance from the point of maximum moment to the point of contra-flexure

$L_p =$ equivalent analytical plastic hinge length as defined below

$\Delta_p =$ idealized plastic displacement capacity due to rotation of the plastic hinge

$\Delta_y^{col} =$ idealized yield displacement of the column at the formation of the plastic hinge

$\phi_y =$ idealized yield curvature defined by an elastic-perfectly-plastic representation of the cross section's M-ϕ curve

$\phi_p =$ idealized plastic curvature capacity (assumed constant over L_p)

$\phi_u =$ curvature capacity at the Failure Limit State, defined as the concrete strain reaching ε_{cu} or the confinement reinforcing steel reaching the reduced ultimate strain ε_{su}^R

$\theta_p =$ plastic rotation capacity

The analytical plastic hinge length, L_p, is taken as the equivalent length of column over which the plastic curvature is assumed constant for estimating plastic rotation. (Guide Spec 4.11.6)

For columns and pile shafts:

$L_p = 0.08L + 0.15 f_{ye} d_{bl}$, where:

$L =$ distance from the point of maximum moment to the point of contra-flexure (in)

$f_{ye} =$ expected yield strength of longitudinal column reinforcing steel bars (ksi)

$d_{bl} =$ nominal diameter of longitudinal column reinforcing steel bar (in)

For non-cased Pile extensions:

$L_p = 0.1H' + D^* \leq 1.5D^*$, where:

$D^* =$ diameter of circular shafts or cross-section dimension in direction under consideration for oblong shafts (in)

$H' =$ length of shaft from the ground surface to point of contraflexure above ground (in)

For horizontally isolated flared columns:

$L_p = G + 0.3 f_{ye} d_{bl}$, where:

$G =$ The gap between the isolated flare and the soffit of the crossbeam (in)

$f_{ye} =$ expected yield strength of longitudinal column reinforcing steel bars (ksi)

$d_{bl} =$ nominal diameter of longitudinal column reinforcing steel bar (in)

For concrete filled pipe pile extensions:

$L_p = 0.1H' + 1.25D \quad \leq 2D$, where:

$D \quad =$ diameter of concrete filled pipe (in)

$H' =$ length of shaft from the ground surface to point of contraflexure above ground (in)

Figure 10. Local Displacement Capacity – Cantilever Column with Fixed Base

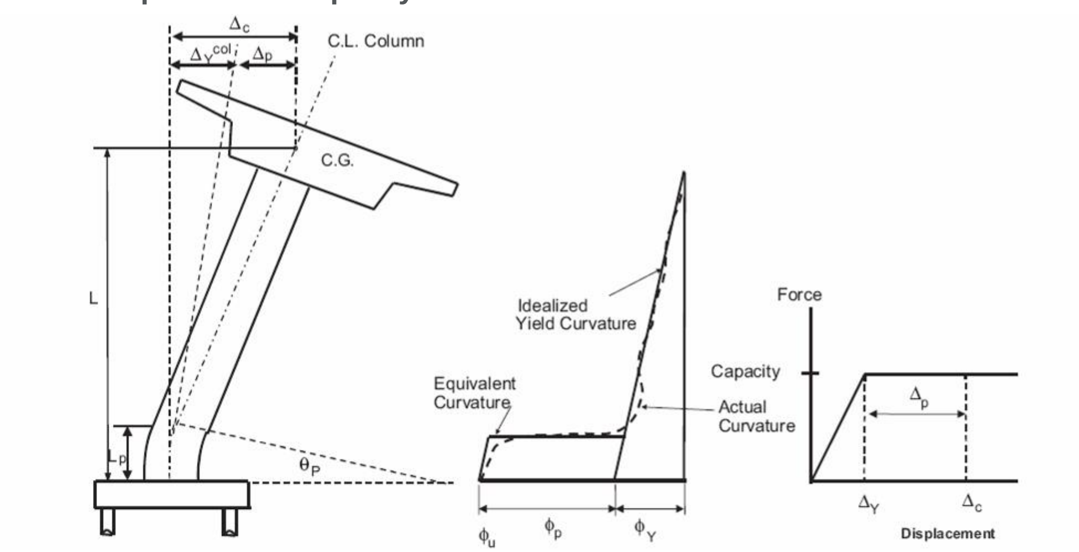
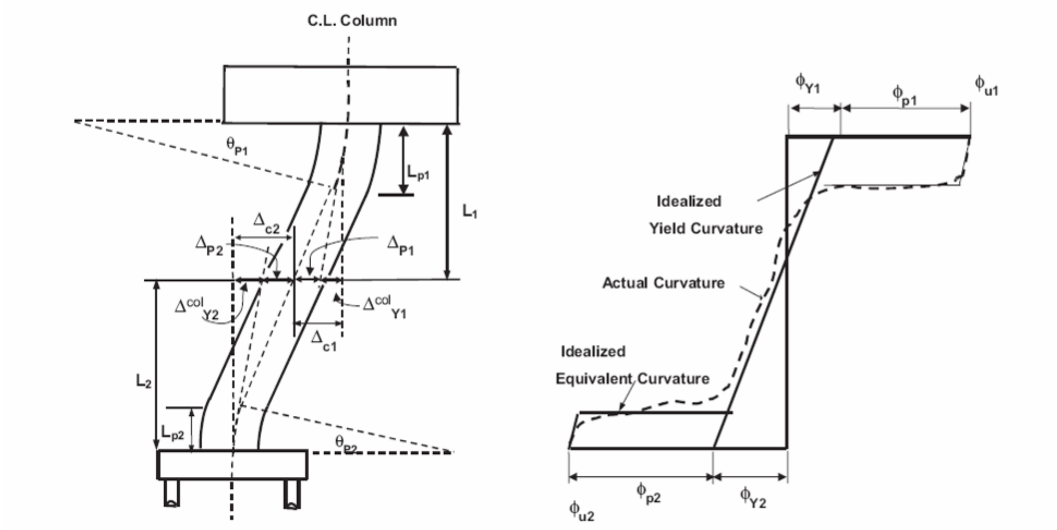


Figure 11. Local Displacement Capacity – Framed Column, Assumed as Fixed-Fixed



7.5.2 Stand-Alone Local Analysis

Stand-alone analysis quantifies the strength and ductility capacity of an individual frame, bent, or column. Stand-alone analysis may be performed in both the transverse and longitudinal directions.

The two-dimensional plane frame Pushover Analysis of a bent or frame can further be simplified to a column model (fixed-fixed or fixed-pinned) if it does not cause a significant loss in accuracy in estimating the displacement demands or the displacement capacities. The effect of overturning on the column axial load and associated member capacities must be considered in the simplified model.

8 Performance Acceptance Criteria

8.1 Full Operation Design Earthquake Ground Motion Acceptance Criteria

The performance level for the FODE ground motion is FO – Full Operation, as stipulated in Section 3. Under the FODE event, the bridge should be repairable without restriction to traffic flow. (FHWA 1.4.1)

Minimal damage may include minor inelastic response and narrow flexural or shear cracks in concrete. Permanent deformations are not apparent and repairs can be made under non-emergency conditions with possible exception of superstructure expansion joints which may need removal and temporary replacement. (FHWA 1.4)

Note the differentiation between Limited Operation (LO) performance level and Fully Operation (FO) performance level. The fully operational criteria require that any damage sustained is negligible and traffic service is available for all vehicles. Except for joint seals, damage is minor such that it can be repaired without interruption to traffic (FHWA 1.4.1).

Example of acceptable level of damage:

- ✓ Damage to bearing at the local level that results in a fractions of inches of vertical displacement while maintaining vertical stability.
- ✓ Bearing is damaged and requires replacement after the seismic event.
- ✓ Bearing replacement requires the bridge superstructure to be temporary supported and the bearing repaired after the seismic event.
- ✓ Dowels in pin connections that fuse without resulting in a reduction in vertical load-carrying capacity of the bridge or superstructure unseating, and the loss of which will not preclude the structure from meeting the LODE performance requirements.

Example of unacceptable level of damage:

- × Damages that require extensive time for repairing the bridge before it can be opened for emergency vehicles.

8.1.1 FODE Force Criteria

Bridge component Capacity-to-Demand (C/D) ratios shall be evaluated for all relevant failure modes, including but not limited to: Girders, In-Span Hinges, Bearings, Expansion Joints, Crossbeams, Outriggers, Columns/Piers, Footing/Pile Caps, Column-to-Crossbeam Connections, Column-to-Footing/Pile Cap Connections, Piles, and Pile Connections.

When evaluating the existing structure, use the overstrength demands for capacity protected elements.

Results indicating that C/D ≥ 1.0 are considered acceptable.

Lateral loads should not fracture any abutment back wall, pier cap, bearing connection, or pile connections that could prohibit traffic flow following an FODE event.

Force and moment reactions in rectangular or oblong columns shall only be evaluated about each principal axis of the column individually, without consideration for biaxial effects (FHWA 7.4.2).

8.1.2 FODE Displacement Criteria

Local displacement capacities, such as at hinges and bearing seats, shall be calculated. Both global and local displacement demands shall comply with the “Full Operation” level of performance following an FODE seismic event.

Abutment or pier bearing displacements should be minimal. Any permanent bearings displacements due to FODE ground motions should be sufficiently small that they will not impede vehicle traffic after the event. The abutment or pier bearing displacement capacity should be 6” more than the abutment or pier displacement demand.

$$\Delta_c \geq \Delta_d + 6''$$

Where,

Δ_c = relative local displacement capacity

Δ_d = relative local displacement demand

Seat width requirements defined in FHWA 5.2.1 and displacement limitations at abutments defined in FHWA Appendix D.6 need not be met.

8.1.3 FODE Stress/Strain Criteria

To achieve the seismic performance objectives, the demands in the various structural components shall be limited to the values listed in Table 4 below.

Table 4. FODE Strain Criteria

Element	Full Operation Design Earthquake (FODE)
Moderate Inelastic Components (Concrete Columns)	εc = 0.004 (confined) εc = 0.002 (confined, locations with lap splices in tension; Priestley 7.4.5) εs = 0.010
Minimal Inelastic Components (Drilled Shafts, Cable Stay Tower, Moveable Substructure)	εc = 0.004 (confined) εc = 0.002 (confined, locations with lap splices in tension; Priestley 7.4.5) εs = 0.010

s = steel, c = concrete

8.1.4 FODE Foundation Behavior

The geotechnical capacity of the foundation shall be established based upon the nominal, or ultimate, strength of the soil. Strengths shall be determined by geotechnical analysis or recommendation by the Geotechnical Engineer. Nominal strengths shall take into consideration liquefaction, other earthquake-induced soil strength reduction, existing scour, or other deleterious subsurface effects that may be present or are likely to occur under seismic loading conditions.

Force and moment reactions for evaluation of spread footings shall only be applied about each principal axis of the footing individually, without consideration for off-axis resultants (Guide Spec 6.3.4).

8.1.5 FODE Shallow Foundation (Spread Footing)

Bearing

$q_R = \phi_b \times q_n$ (*AASHTO LRFD* 10.6.3.1.1-1)

Where:

q_R = factored bearing resistance

ϕ_b = bearing resistance factor = 1.0

q_n = nominal bearing resistance

Sliding

$R_R = \phi \times R_n = \phi_\tau \times R_\tau + \phi_{ep} \times R_{ep}$ (*AASHTO LRFD* 10.6.4.3-1)

Where:

R_R = factored sliding resistance

R_n = nominal sliding resistance

ϕ_τ = resistance factor for shear resistance between soil and foundation = 1.0

R_τ = nominal sliding resistance between soil and foundation

ϕ_{ep} = resistance factor for passive resistance = 1.0

R_{ep} = nominal passive resistance of soil

Overturning

In general, the resultant of the reaction forces shall be within the middle two-thirds of the footing (AASHTO LRFD 11.6.5.1). If this condition cannot be achieved, limited unloading of the footing may be allowed so long as the ultimate bearing capacity is not exceeded. Footings experiencing reduced bearing across the footing surface shall be modeled with a bi-linear stress curve, where the maximum stress plateau shall equal the foundation soil bearing resistance. The force resultant shall remain within the footing.

8.1.6 Deep Foundations

Pile Axial Resistance

$R_R = \phi \times R_n$ (*AASHTO LRFD* 10.7.3.8.6a-1)

Where:

R_R = factored nominal axial resistance

ϕ = axial resistance factor = 1.0

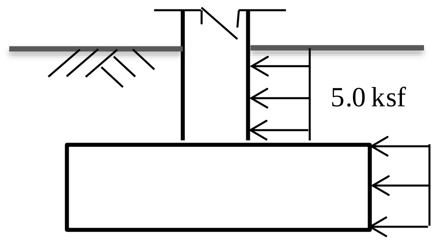
R_n = nominal axial resistance

Pile and Footing Lateral Resistance

Pile lateral resistance or capacity shall be determined by a lateral analysis using GROUP. LPILE, or other similar pile analysis software

Footing passive pressure of 5.0 ksf could be utilized (adjusted for depth of soil according to Section 7.4.7.1) for the initial analysis until the geotechnical report is available. Similarly, the column passive pressure of 5.0 ksf may also be utilized for initial analysis as shown in figure below.

Figure 12. Footing Passive Pressure



8.1.7 Pile Structural Behavior

Steel Piles

Combined Axial Compression and Flexure

$$\frac{P_u}{P_r} + \frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \leq 1.0 \qquad \text{(}\textit{AASHTO LRFD}\text{ 6.9.4.2.1-6)}$$

Where:

P_u = axial compressive load

P_r = factored compressive resistance

M_{ux} = factored flexural moment about the strong axis

M_{uy} = factored flexural moment about the weak axis

M_{rx} = factored flexural resistance about the strong axis = $F_{ye} \times S$

M_{ry} = factored flexural resistance about the weak axis = $F_{ye} \times Z_y$

Z_y = section properties represented by yielding of 50% of flange area.

Precast Concrete Piles

Due to historically poor pile reinforcing details in typical ODOT precast prestressed concrete piles, pile structural capacity shall be the nominal capacity of the pile. Pile flexural capacity shall be determined in accordance with strain limits defined in Section 8.1.3.

8.2 Limited Operation Design Earthquake Ground Motion Acceptance Criteria

The performance level for LODE ground motion is LO – Limited Operation, as stipulated in Section 3. This criterion is to ensure that during the 1,000-year return probabilistic ground shaking considered feasible for the site, the bridge will enable emergency service and heavy haul vehicles to cross the bridge. Access for first responders and escape for downtown populations are the primary concerns and is the focus of the overall retrofit design philosophy.

In addition to above, the performance objective as described in Section 3 shall be met.

Note: LO performance level is required for the 1,000-year return period Design Earthquake.

Example of acceptable level of damage (In addition to the damage described in Section 8.1):

- ✓ Pier column cracks that require repairs
- ✓ Reduced traffic lanes to limit the total live load on the bridge before the repairs are completed.
- ✓ Posted speed limit to reduce the impact loads on the bridge before the repairs are completed.
- ✓ Misalignment of the bascule leafs that restrict the bascule operation before repairs are completed.

Example of unacceptable level of damage:

- × A vertical displacement large enough (more than three inches) to prevent emergency vehicles from crossing the bridge.
- × Superstructure element falling off the abutment seat, hinge seat, or crossbeam.

8.2.1 LODE Force Criteria

Bridge component Capacity-to-Demand (C/D) ratios shall be evaluated for all relevant failure modes, including but not limited to: Girders, In-Span Hinges, Bearings, Expansion Joints, Crossbeams, Outriggers, Columns/Piers, Footing/Pile Caps, Column-to-Crossbeam Connections, Column-to-Footing/Pile Cap Connections, Piles, and Pile Connections.

When evaluating the existing structure, use the overstrength demands for capacity protected elements.

Results indicating that C/D ≥ 1.0 are considered acceptable using limited ductility displacement capacities.

Lateral loads should not fracture any abutment back wall, pier cap, bearing connection, or pile connections that requires extensive repair and prohibit traffic flow following a LODE event.

8.2.2 LODE Displacement Criteria

Abutment or pier bearing displacements should be minimal. Any permanent bearings displacements due to LODE ground motions should be sufficiently small that they will not require extensive repair thus impede emergency vehicle traffic after the event.

8.2.3 LODE Stress/Strain Criteria

To achieve the seismic performance objectives, the demands in the various structural components shall be limited to the values listed in Table 5 below.

Table 5. LODE Strain Criteria

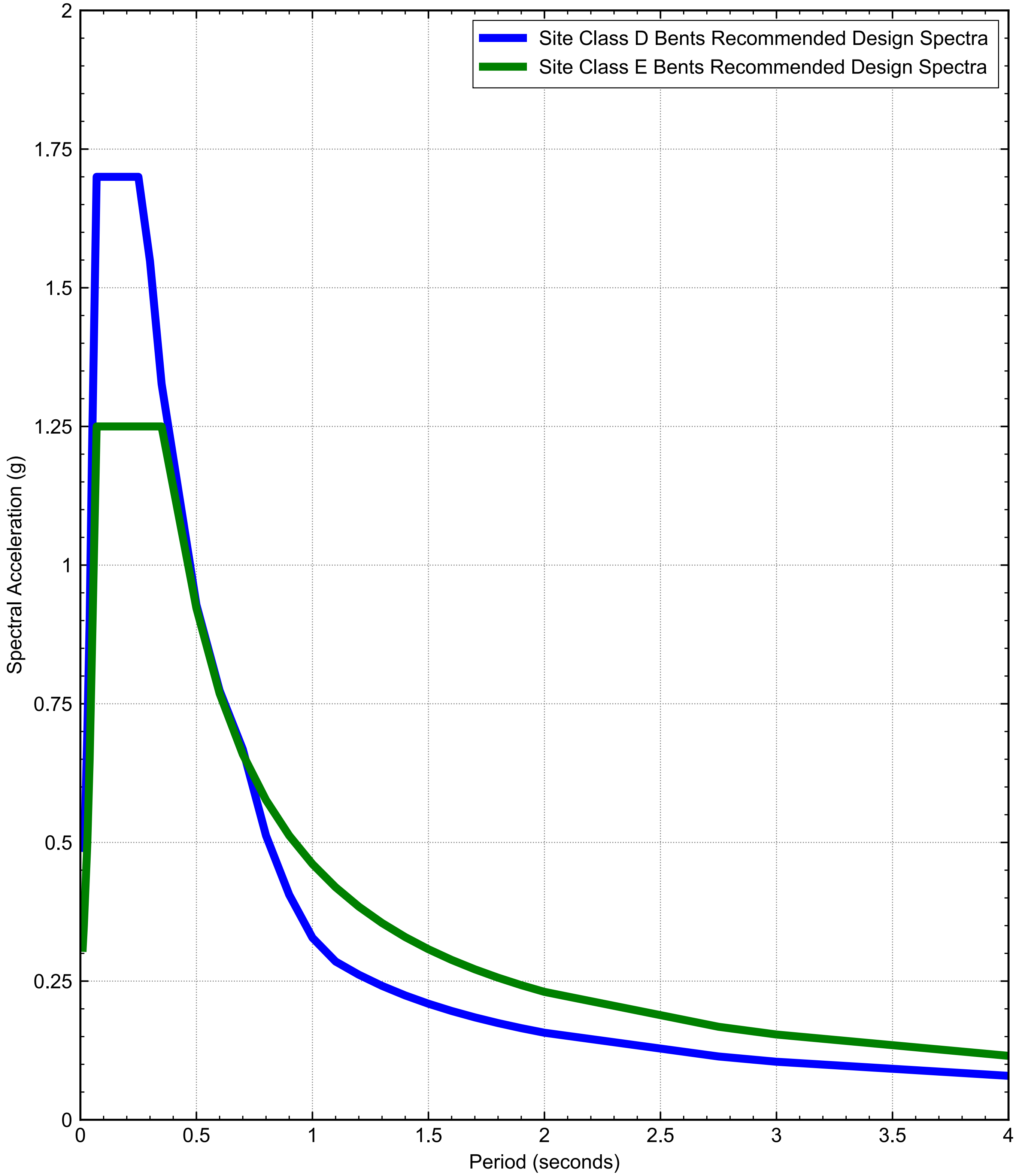
Element	Limited Operation Design Earthquake (LODE)
Moderate Inelastic Components (Concrete Columns)	$\varepsilon c = \varepsilon cu$ (confined) $\varepsilon s = 0.80 \times \varepsilon_{\text{bar buckling}}$
Minimal Inelastic Components (Drilled Shafts, Cable Stay Tower, Moveable Substructure)	$\varepsilon c = \varepsilon cu$ (confined) $\varepsilon s = 0.015$

s = steel, c = concrete

$$\varepsilon_{bar\ buckling} = 0.032 + \left[\frac{790 \times \rho_s \times f_{yhe}}{\mathrm{Es}}\right] - \left[\frac{0.14 \times P}{\mathrm{Ag\ x}\ f'_{ce}}\right]$$

εs = 0.015 is in accordance with Serviceability Limit Tension Strain discussed by Priestly, Calvi, Kowalsky in "Displacement-Based Seismic Design of Structures"

Appendix A. Site-Specific Acceleration Response Spectra



Note:
1. Our recommended design spectrum were evaluated based on AASHTO and the GDM and the results of our site response analysis.

Earthquake Ready Burnside Bridge
Portland, Oregon

RECOMMENDED DESIGN SPECTRUM 1,000-YEAR PROBABILISTIC HAZARD LEVEL

January 2022 102636-009

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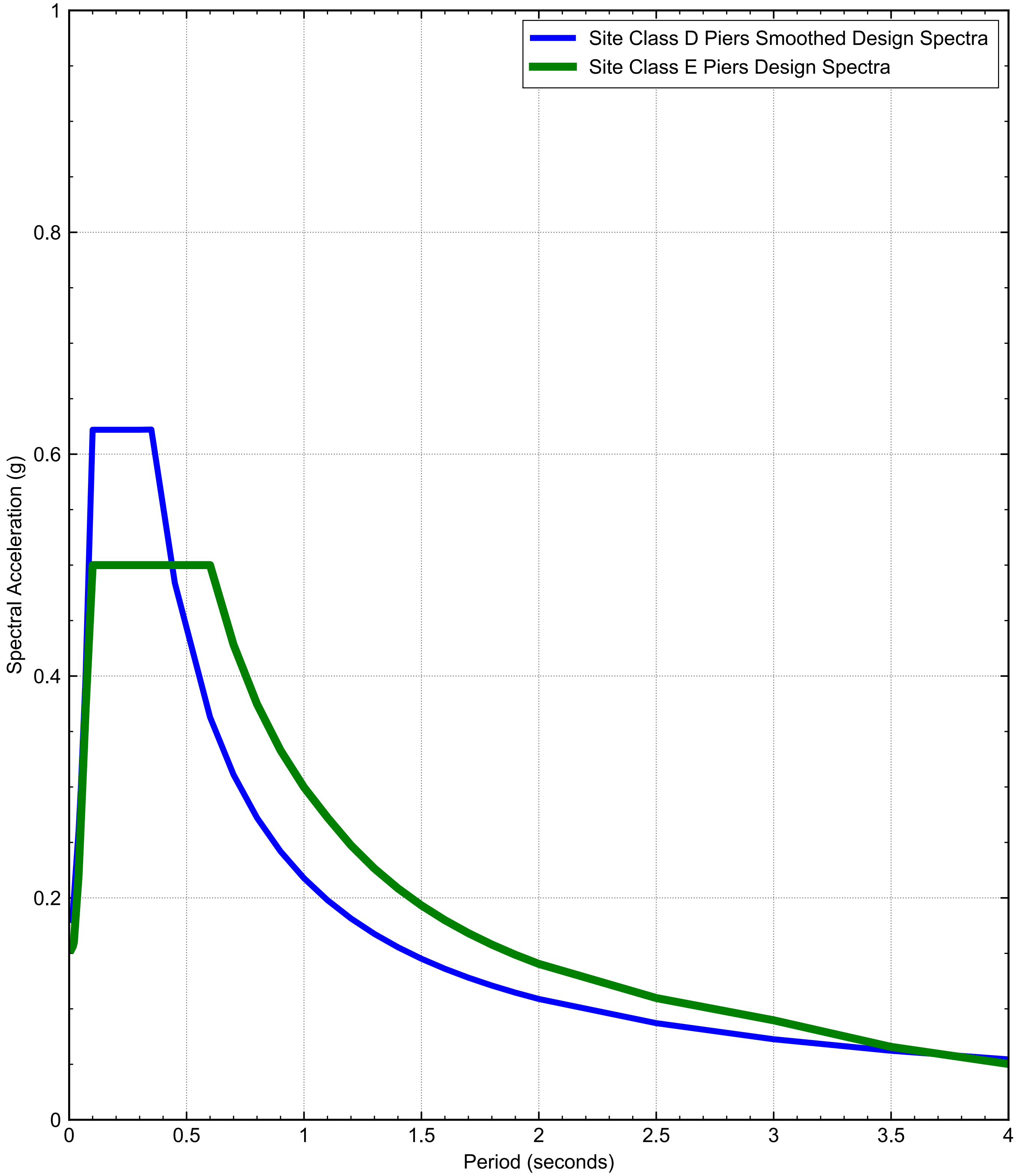
FIG. 8-1



Note:
1. Our recommended design spectrum were evaluated based on AASHTO and the GDM and the results of our site response analysis.

Earthquake Ready Burnside Bridge
Portland, Oregon

RECOMMENDED DESIGN SPECTRUM DETERMINISTIC CSZ HAZARD LEVEL

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FIG. 8-2