Contents

Chapter 4 Seismic Design and Retrofit

4.1 General

Seismic design of new bridges and bridge widenings shall conform to LRFD-SGS as modified by Sections 4.2 and 4.3.

Analysis and design of seismic retrofits for existing bridges shall be completed in accordance with Section 4.4. Seismic design of retaining walls shall be in accordance with Section 4.5. For nonconventional bridges, bridges that are deemed critical or Recovery, or bridges that fall outside the scope of the Guide Specifications for any other reasons, project specific design requirements shall be developed and submitted to the State Bridge Design Engineer for approval.

The importance classifications for all highway bridges in Washington State are classified as “Ordinary” except for special major bridges. Special major bridges fitting the classifications of either “Critical” or “Recovery” will be so designated by either the WSDOT Bridge and Structures Engineer or the State Bridge Design Engineer.

Bridges are considered as Critical, Recovery, or Ordinary for their operational classification as described below. Two-level performance criteria are required for design of Recovery, Critical and some Ordinary bridges. Recovery and Critical bridges shall be designated by WSDOT Regions or Local Agencies, in consultation with WSDOT State Bridge and Structures Engineer and State Bridge Design Engineer.

- **Critical Bridges**
  Critical bridges are expected to provide immediate access to emergency and similar life-safety facilities after an earthquake. The Critical designation is typically reserved for high-cost projects where WSDOT intends to protect the investment or for projects that would be especially costly to repair if they were damaged during an earthquake.
- **Recovery Bridges**
  Recovery bridges serve as vital links for rebuilding damaged areas and provide access to the public shortly after an earthquake.
- **Ordinary Bridges**
  All bridges not designated as either Critical or Recovery shall be designated as Ordinary.

4.1.1 Expected Bridge Seismic Performance

The seismic hazard evaluation level for designing bridges shall be in accordance with Table 4.1-1 for Safety Evaluation Earthquake (SEE) and/or Functional Evaluation Earthquake (FEE).

Table 4.1-1 Seismic Hazard Evaluation Levels and Expected Performance

Bridge Operational Importance Category	Seismic Hazard Evaluation Level	Expected Post Earthquake Damage State	Expected Post Earthquake Service Level
"Ordinary Bridges" – Eastern Washington	SEE	Significant	No Service
"Ordinary Bridges" – Western Washington (Not Lifeline)	SEE	Significant	No Service
	FEE	Minimal	Full Service
"Recovery Bridges" (Lifeline)	SEE	Moderate	Limited Service
	FEE	Minimal	Full Service
"Critical Bridges"	SEE	Minimal to Moderate	Limited Service
	FEE	None to Minimal	Full Service

4.1.2 Expected Post-earthquake Service Levels

- **No Service** – Bridge is closed for repair or replacement.
- **Limited Service** – Bridge is open for emergency vehicle traffic: A reduced number of lanes for Ordinary traffic is available within three months of the earthquake; Vehicle weight restriction may be imposed until repairs are completed. It is expected that within three months (Recovery Bridges) or within three days (Critical Bridges) of the earthquake, repair works on a damaged bridge would have reached the stage that would permit Ordinary traffic on at least some portion of the bridge.
- **Full Service** – Full access to Ordinary traffic is available almost immediately after the earthquake. The expected post-earthquake damage states and service levels of Critical bridges are included in Table 4.1-2 to provide an indication of their expected performance relative to Ordinary bridge categories.

Table 4.1-2 Displacement Ductility Demand Values, μ_D

Seismic Critical Member	Displacement Ductility Demand Limits						
	Ordinary Bridges • EW	Ordinary Bridges – WW (Not Lifeline)		Recovery Bridges (Lifeline)		Critical Bridges	
	• SEE	SEE	FEE	SEE	FEE	SEE	FEE
Wall Type Pier in Weak Direction	5.0	5.0	1.5	2.5	1.5	1.5	1.0
Wall Type Pier in Strong Direction	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Single Column Bent	5.0	5.0	1.5	2.5	1.5	1.5	1.0
Multiple Column Bent	6.0	6.0	2.0	3.5	2.0	1.5	1.0
Pile/Shaft-Column with Plastic Hinge at Top of Column	5.0	5.0	2.0	3.5	2.0	1.5	1.0
Pile/Shaft-Column with Plastic Hinge Below Ground	4.0	4.0	1.5	2.5	1.5	1.5	1.0
Superstructure	1.0	1.0	1.0	1.0	1.0	1.0	1.0

4.1.3 Expected Post-earthquake Damage States

- **Significant** – “imminent failure,” i.e., onset of compressive failure of core concrete. Bridge replacement is likely. All plastic hinges within the structure have formed with ductility demand values approaching the limits specified in Table 4.1-2.
- **Moderate** – “extensive cracks and spalling, and visible lateral and/or longitudinal reinforcing bars”. Bridge repair is likely but bridge replacement is unlikely
- **Minimal** – “flexural cracks and minor spalling and possible shear cracks”. Essentially elastic performance
- **None** – No damage

The Design Spectrum for Safety Evaluation Earthquake (SEE) corresponds to the LRFD-SDS seismic design ground motion, and has a targeted risk of incipient collapse of an Ordinary bridge of approximately 1.5 percent in 75 years.

The Design Spectrum for Functional Evaluation Earthquake (FEE) corresponds to a uniform hazard ground motion and shall be taken as a spectrum based on a 30% probability of exceedance in 75 years (or 210-year return period). The Geotechnical Engineer shall provide final design spectrum recommendations.

Ordinary and Recovery bridges subjected to the seismic hazard levels specified in Table 1 shall satisfy the displacement criteria specified in LRFD-SGS as applicable and the maximum displacement ductility demand, μ_D values as specified in Table 4.1-2.

4.2 WSDOT Additions and Modifications to AASHTO *Guide Specifications for LRFD Seismic Bridge Design* (LRFD-SDS)

WSDOT amendments to the LRFD-SGS are as follows:

4.2.1 Definitions

LRFD-SGS Article 2.1 – Add the following definitions:

- **Owner –** Person or agency having jurisdiction over the bridge. For WSDOT projects, regardless of delivery method, the term “Owner” in these Guide Specifications shall be the WSDOT State Bridge Design Engineer or/and the WSDOT State Geotechnical Engineer.

4.2.2 Earthquake Resisting Systems (ERS) Requirements for Seismic Design Categories (SDCs) C and D

LRFD-SGS Article 3.3 – WSDOT Global Seismic Design Strategies:

- **Type 1 –** Ductile Substructure with Essentially Elastic Superstructure. This category is permissible.
- **Type 2 –** Essentially Elastic Substructure with a Ductile Superstructure. This category is not permissible.
- **Type 3 –** Elastic Superstructure and Substructure with a Fusing Mechanism between the two. This category is permissible with WSDOT State Bridge Design Engineer’s approval.

With the approval of the State Bridge Design Engineer, for Type 1 ERS for SDC C or D, if columns or pier walls are considered an integral part of the energy dissipating system but remain elastic at the demand displacement, the forces to use for capacity design of other components are to be a minimum of 1.2 times the elastic forces resulting from the demand displacement in lieu of the forces obtained from overstrength plastic hinging analysis. Because maximum limiting inertial forces provided by yielding elements acting at a plastic mechanism level is not effective in the case of elastic design, the following constraints are imposed. These may be relaxed on a case by case basis with the approval of the State Bridge Design Engineer.

1. Unless an analysis that considers redistribution of internal structure forces due to inelastic action is performed, all substructure units of the frame under consideration and of any adjacent frames that may transfer inertial forces to the frame in question must remain elastic at the design ground motion demand.
2. Effective member section properties must be consistent with the force levels expected within the bridge system. Reinforced concrete columns and pier walls should be analyzed using cracked section properties. For this purpose, in absence of better information or estimated by LRFD-SGS Figure 5.6.2-1, a moment of inertia equal to one half that of the un-cracked section shall be used.
3. Foundation modeling must be established such that uncertainties in modeling will not cause the internal forces of any elements under consideration to increase by more than 10 percent.

4. When site specific ground response analysis is performed, the response spectrum ordinates must be selected such that uncertainties will not cause the internal forces of any elements under consideration to increase by more than 10 percent.
5. Thermal, shrinkage, prestress or other forces that may be present in the structure at the time of an earthquake must be considered to act in a sense that is least favorable to the seismic load combination under investigation.
6. P-Delta effects must be assessed using the resistance of the frame in question at the deflection caused by the design ground motion.
7. Joint shear effects must be assessed with a minimum of the calculated elastic internal forces applied to the joint.
8. Detailing as normally required in either SDC C or D, as appropriate, must be provided.

It is permitted to use expected material strengths for the determination of member strengths except shear for elastic response of members.

The use of elastic design in lieu of overstrength plastic hinging forces for capacity protection described above shall only be considered if designer demonstrates that capacity design of Article 4.11 of the AASHTO *Guide Specifications for LRFD Bridge Seismic Design* is not feasible due to geotechnical or structural reasons.

If the columns or pier walls remain elastic at the demand displacement, shear design of columns or pier walls shall be based on 1.2 times elastic shear force resulting from the demand displacement and normal material strength shall be used for capacities. The minimum detailing according to the bridge seismic design category shall be provided.

Type 3 ERS may be considered only if Type 1 strategy is not suitable and Type 3 strategy has been deemed necessary for accommodating seismic loads. Use of isolation bearings needs the approval of WSDOT State Bridge Design Engineer. Isolation bearings shall be designed per the requirement specified in Section 9.3.

Limitations on the use of ERS and ERE are shown in Figures 3.3-1a, 3.3-1b, 3.3-2, and 3.3-3.

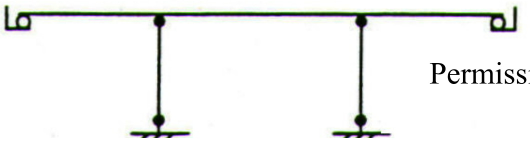
- Figure 3.3-1b Type 6, connection with moment reducing detail should only be used at column base if proved necessary for foundation design. Fixed connection at base of column remains the preferred option for WSDOT bridges.
- The design criteria for column base with moment reducing detail shall consider all applicable loads at service, strength, and extreme event limit states.
- Figure 3.3-2 Types 6 and 8 are not permissible for non-liquefied configuration and permissible with WSDOT State Bridge Design Engineer's approval for liquefied configuration.

For ERSs and EREs requiring approval, the WSDOT State Bridge Design Engineer's approval is required regardless of contracting method (i.e., approval authority is not transferred to other entities).

BDM Figure 4.2.2-1 Figure 3.3-1a Permissible Earthquake-Resisting Systems (ERSs)

1

Longitudinal Response

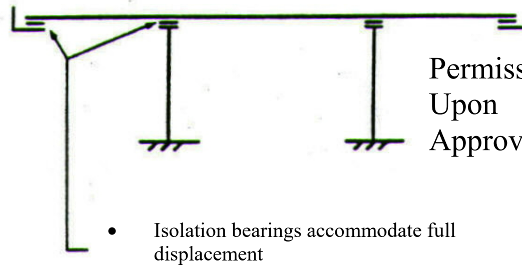


Permissible

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

2

Longitudinal Response

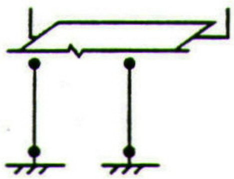


Permissible Upon Approval

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

3

Transverse Response

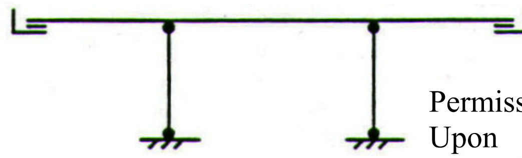


Permissible

- Plastic hinges in inspectable locations.
- Abutment not required in ERS, breakaway shear keys permissible with WSDOT Bridge Design Engineer's Approval

4

Transverse or Longitudinal Response



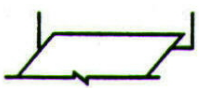
Permissible Upon Approval

- Plastic hinges in inspectable locations
- Isolation bearings with or without energy dissipaters to limit overall displacements

5

Transverse or Longitudinal Response

Permissible



- Abutment required to resist the design earthquake elastically
- Longitudinal passive soil pressure shall be less than 0.70 of the value obtained using the procedure given in BDM Article 4.2.11

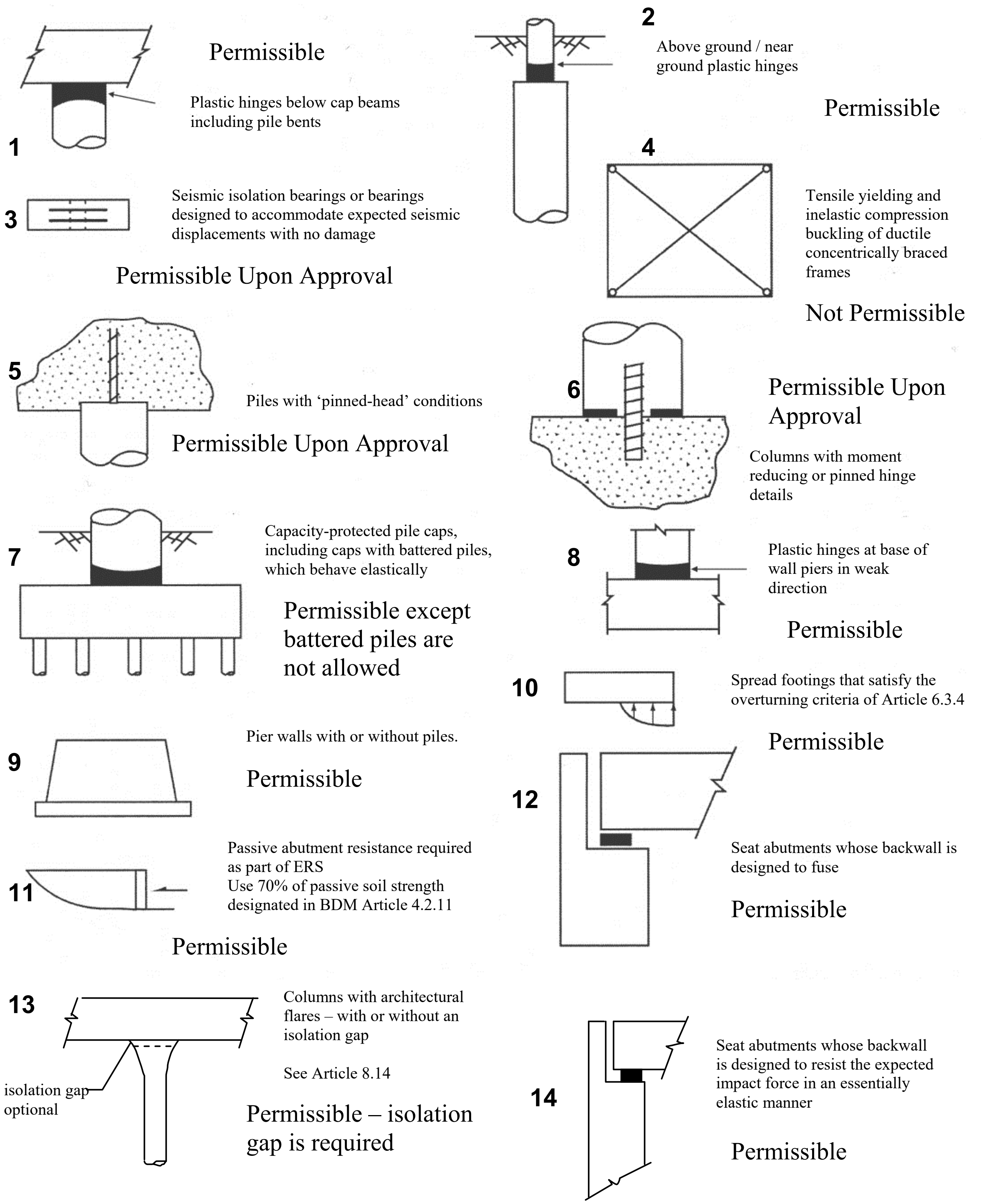
6

Longitudinal Response

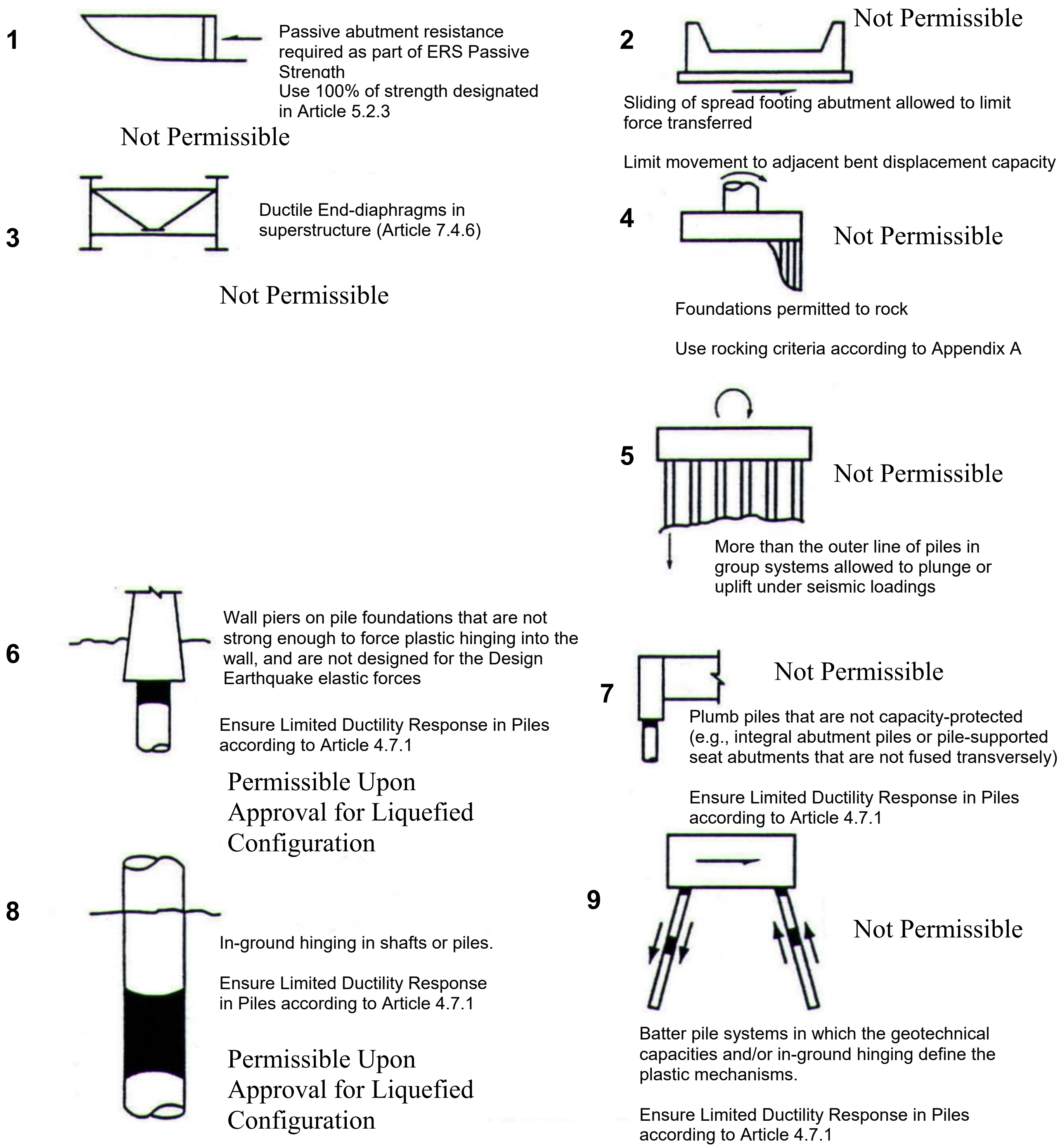
Not Permissible

- Multiple simply-supported spans with adequate support lengths
- Plastic hinges in inspectable locations or elastic design of columns

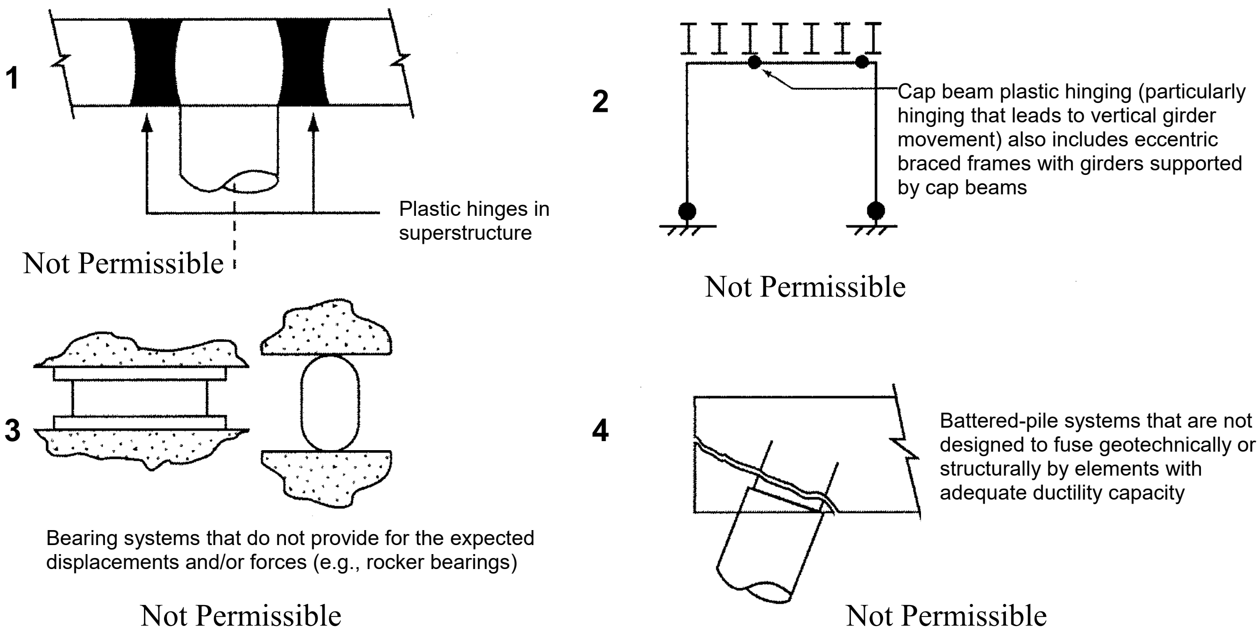
BDM Figure 4.2.2-2 Figure 3.3-1b Permissible Earthquake-Resisting Elements (EREs)



BDM Figure 4.2.2-3 Figure 3.3-2 Permissible Earthquake-Resisting Elements That Require Owner's Approval



BDM Figure 4.2.2-4 Figure 3.3-3 Earthquake-Resisting Elements that Are Not Recommended for New Bridges



4.2.3 Seismic Ground Shaking Hazard

LRFD-SGS Article 3.4 – For bridges that are considered Critical, Recovery or Ordinary bridges with a site Class F, the seismic ground shaking hazard shall be determined based on the WSDOT State Geotechnical Engineer recommendations.

In cases where the site coefficients used to adjust mapped values of design ground motion for local conditions are inappropriate to determine the design spectra in accordance with general procedure of Article 3.4.1 (such as the period at the end of constant design spectral acceleration plateau (T_s) is greater than 1.0 second or the period at the beginning of constant design spectral acceleration plateau (T_o) is less than 0.2 second), a site-specific ground motion response analysis shall be performed.

The Design Spectrum for Safety Evaluation Earthquake (SEE) corresponds to the LRFD-SDS seismic design ground motion, and has a targeted risk of incipient collapse of an Ordinary bridge of approximately 1.5 percent in 75 years.

The Design Spectrum for Functional Evaluation Earthquake (FEE) corresponds to a uniform hazard ground motion and shall be taken as a spectrum based on a 30% probability of exceedance in 75 years (or 210-year return period).

4.2.4 Selection of Seismic Design Category (SDC)

LRFD-SGS Article 3.5 – Pushover analysis shall be used to determine displacement capacity for both SDCs C and D.

4.2.5 Temporary and Staged Construction

LRFD-SGS Article 3.6 – For bridges that are designed for a reduced seismic demand, the contract plans shall either include a statement that clearly indicates that the bridge was designed as temporary using a reduced seismic demand or show the Acceleration Response Spectrum (ARS) used for design. No liquefaction assessment required for temporary bridges. The design response spectra given in Article 3.4 may be reduced by a factor of not more than 2.5 to calculate the component elastic forces and displacements.

4.2.6 Load and Resistance Factors

LRFD-SGS Article 3.7 – Revise as follows:

Use load factors of 1.0 for all permanent loads. The load factor for live load shall be 0.0 when pushover analysis is used to determine the displacement capacity. Use live load factor of 0.5 for all other extreme event cases. Unless otherwise noted, all ϕ factors shall be taken as 1.0.

4.2.7 Balanced Stiffness Requirements and Balanced Frame Geometry Recommendation

LRFD-SGS Articles 4.1.2 and 4.1.3 – Balanced stiffness between bents within a frame and between columns within a bent and balanced frame geometry for adjacent frames are required for bridges in both SDCs C and D. Deviations from balanced stiffness and balanced frame geometry requirements require approval from the State Bridge Design Engineer.

4.2.8 Selection of Analysis Procedure to Determine Seismic Demand

LRFD-SGS Article 4.2 – Analysis Procedures:

- Procedure 1 (Equivalent Static Analysis) shall not be used.
- Procedure 2 (Elastic Dynamic Analysis) shall be used for all "regular" bridges with two through six spans and "not regular" bridges with two or more spans in SDCs B, C, or D.
- Procedure 3 (Nonlinear Time History) shall only be used with State Bridge Design Engineer's approval.

4.2.9 Member Ductility Requirement for SDCs C and D

LRFD-SGS Article 4.9 – In-ground hinging for drilled shaft and pile foundations may be considered for the liquefied configuration with State Bridge Design Engineer approval.

4.2.10 Longitudinal Restrainers

LRFD-SGS Article 4.13.1 – Longitudinal restrainers shall be provided at the expansion joints between superstructure segments. Restrainers shall be designed in accordance with the FHWA Seismic Retrofitting Manual for Highway Structure (FHWA-HRT-06-032) Article 8.4 the Iterative Method. Restrainers shall be detailed in accordance with the requirements of LRFD-SGS Article 4.13.3 and Section 4.4.4. Restrainers may be omitted for SDCs C and D where the available seat width exceeds the calculated support length specified in LRFD-SGS Equation C4.13.1-1.

Omitting restrainers for liquefiable sites shall be approved by the State Bridge Design Engineer.

Longitudinal restrainers shall not be used at the end piers (abutments).

4.2.11 Abutments

LRFD-SGS Article 5.2 – Diaphragm Abutment type shown in Figure 5.2.3.2-1 shall not be used for WSDOT bridges.

LRFD-SGS Article 5.2 – Abutments to be revised as follows:

4.2.11.1 - General

The participation of abutment walls in providing resistance to seismically induced inertial loads may be considered in the seismic design of bridges either to reduce column sizes or reduce the ductility demand on the columns. Damage to backwalls and wingwalls during earthquakes may be considered acceptable when considering no collapse criteria, provided that unseating or other damage to the superstructure does not occur. Abutment participation in the overall dynamic response of the bridge system shall reflect the structural configuration, the load transfer mechanism from the bridge to the abutment system, the effective stiffness and force capacity of the wall-soil system, and the level of acceptable abutment damage. The capacity of the abutments to resist the bridge inertial loads shall be compatible with the soil resistance that can be reliably mobilized, the structural design of the abutment wall, and whether the wall is permitted to be damaged by the design earthquake. The lateral load capacity of walls shall be evaluated on the basis of a rational passive earth-pressure theory.

The participation of the bridge approach slab in the overall dynamic response of bridge systems to earthquake loading and in providing resistance to seismically induced inertial loads may be considered permissible upon approval from both the State Bridge Design Engineer and the WSDOT Geotechnical Engineer.

The participation of the abutment in the ERS should be carefully evaluated with the Geotechnical Engineer and the Owner when the presence of the abutment backfill may be uncertain, as in the case of slumping or settlement due to liquefaction below or near the abutment.

4.2.11.2 - Longitudinal Direction

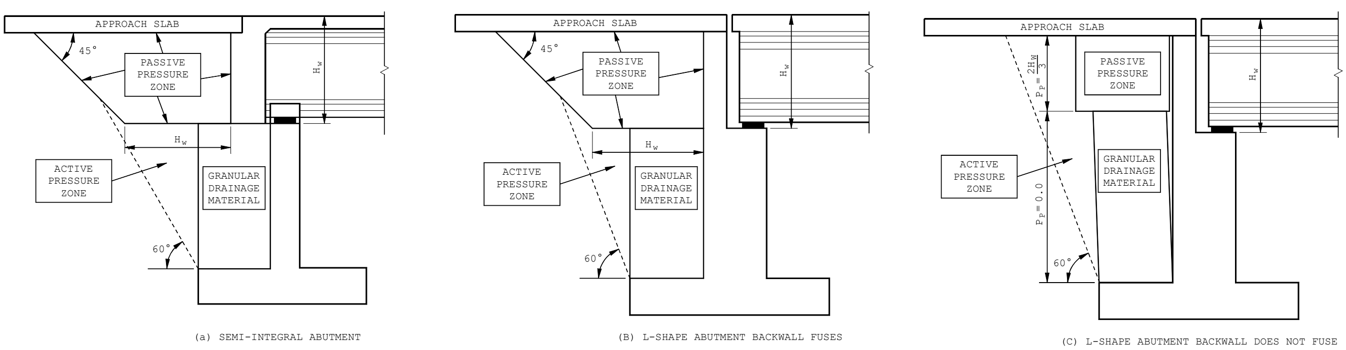
Under earthquake loading, the earth pressure action on abutment walls changes from a static condition to one of two possible conditions:

- The dynamic active pressure condition as the wall moves away from the backfill, or
- The passive pressure condition as the inertial load of the bridge pushes the wall into the backfill.

The governing earth pressure condition depends on the magnitude of seismically induced movement of the abutment walls, the bridge superstructure, and the bridge/abutment configuration.

For semi-integral (Figure 4.2.11-1a), L-shape abutment with backwall fuse (Figure 4.2.11-1b), or without backwall fuse (Figure 4.2.11-1c), for which the expansion joint is sufficiently large to accommodate both the cyclic movement between the abutment wall and the bridge superstructure (i.e., superstructure does not push against abutment wall), the seismically induced earth pressure on the abutment wall shall be considered to be the dynamic active pressure condition. However, when the gap at the expansion joint is not sufficient to accommodate the cyclic wall/bridge seismic movements, a transfer of forces will occur from the superstructure to the abutment wall. As a result, the active earth pressure condition will not be valid and the earth pressure approaches a much larger passive pressure load condition behind the backwall. This larger load condition is the main cause for abutment damage, as demonstrated in past earthquakes. For semi-integral or L-shape abutments, the abutment stiffness and capacity under passive pressure loading are primary design concerns.

Figure 4.2.11-1 Abutment Stiffness and Passive Pressure Estimate



Where the passive pressure resistance of soils behind semi-integral or L-shape abutments will be mobilized through large longitudinal superstructure displacements, the bridge may be designed with the abutments as key elements of the longitudinal ERS. Abutments shall be designed to sustain the design earthquake displacements. When abutment stiffness and capacity are included in the design, it should be recognized that the passive pressure zone mobilized by abutment displacement extends beyond the active pressure zone normally used for static service load design. This is illustrated schematically in Figures 4.2.11-1a and 4.2.11-1b. Dynamic active earth pressure acting on the abutment need not be considered in the dynamic analysis of the bridge. The passive abutment resistance shall be limited to 70 percent of the value obtained using the procedure given in Article 4.2.11.2.1.

4.2.11.2.1 - Abutment Stiffness and Passive Pressure Estimate

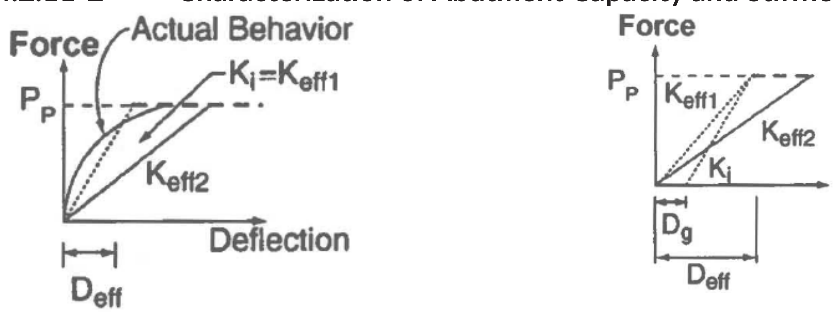
Abutment stiffness, K_{eff} in kip/ft, and passive capacity, P_p in kips, should be characterized by a bilinear or other higher order nonlinear relationship as shown in Figure 4.2.11-2. When the motion of the back wall is primarily translation, passive pressures may be assumed uniformly distributed over the height (H_w) of the backwall or end diaphragm. The total passive force may be determined as:

$$P_p = {}_{pp} H_w W_w \quad (4.2.11.2.1\text{-}1)$$

Where:

p_p	=	passive lateral earth pressure behind backwall or diaphragm (ksf)
H_w	=	height of back wall or end diaphragm exposed to passive earth pressure (feet)
W_w	=	width of back wall or diaphragm (feet)

Figure 4.2.11-2 Characterization of Abutment Capacity and Stiffness



(a) Semi-integral Abutment (b) L-shape Abutment

4.2.11.2.2 - Calculation of Best Estimate Passive Pressure P_p

If the strength characteristics of compacted or natural soils in the "passive pressure zone" are known, then the passive force for a given height, H_w, may be calculated using accepted analysis procedures. These procedures should account for the interface friction between the wall and the soil. The properties used shall be those indicative of the entire "passive pressure zone" as indicated in Figure 4.2.11-1. Therefore, the properties of backfill present immediately adjacent to the wall in the active pressure zone may not be appropriate as a weaker failure surface can develop elsewhere in the embankment.

For L-shape abutments where the backwall is not designed to fuse, H_w shall conservatively be taken as the depth of the superstructure, unless a more rational soil-structure interaction analysis is performed.

If presumptive passive pressures are to be used for design, then the following criteria shall apply:

- Soil in the "passive pressure zone" shall be compacted in accordance with *Standard Specifications* Section 3-03.3(14)I, which requires compaction to 95 percent maximum density for all “Bridge Approach Embankments”.
- For cohesionless, nonplastic backfill (fines content less than 30 percent), the passive pressure P_p may be assumed equal to $2H^w/3$ ksf per foot of wall length.

For other cases, including abutments constructed in cuts, the passive pressures shall be developed by a geotechnical engineer.

4.2.11.2.3 - Calculation of Passive Soil Stiffness

Equivalent linear secant stiffness, K_{eff} in kip/ft, is required for analyses. For semi-integral or L-shape abutments initial secant stiffness may be determined as follows:

$$K_{eff1} = \frac{P_p}{(F_w H_w)} \quad (4.2.11.2.3\text{-}1)$$

Where:

P_p = passive lateral earth pressure capacity (kip)
H_w = height of back wall (feet)
F_w = the value of Fw to use for a particular bridge may be found in Table C3.11.1-1 of the AASHTO LRFD.

For L-shape abutments, the expansion gap should be included in the initial estimate of the secant stiffness as specified in:

$$K_{eff1} = \frac{P_p}{(F_w H_w + D_g)} \quad (4.2.11.2.3\text{-}2)$$

Where:

D_g = width of gap between backwall and superstructure (feet)

For SDCs C and D, where pushover analyses are conducted, values of P_p and the initial estimate of K_{eff1} should be used to define a bilinear load-displacement behavior of the abutment for the capacity assessment.

4.2.11.2.4 - Modeling Passive Pressure Stiffness in the Longitudinal Direction

In the longitudinal direction, when the bridge is moving toward the soil, the full passive resistance of the soil may be mobilized, but when the bridge moves away from the soil no soil resistance is mobilized. Since passive pressure acts at only one abutment at a time, linear elastic dynamic models and frame pushover models should only include a passive pressure spring at one abutment in any given model. Secant stiffness values for passive pressure shall be developed independently for each abutment.

As an alternative, for straight or with horizontal curves up to 30 degrees single frame bridges, and compression models in straight multi-frame bridges where the passive pressure stiffness is similar between abutments, a spring may be used at each abutment concurrently. In this case, the assigned spring values at each end need to be reduced by half because they act in simultaneously, whereas the actual backfill passive resistance acts only in one direction and at one time. Correspondingly, the actual peak passive resistance force at either abutment will be equal to the sum of the peak forces developed in two springs. In this case, secant stiffness values for passive pressure shall be developed based on the sum of peak forces developed in each spring. If computed abutment forces exceed the soil capacity, the stiffness should be softened iteratively until abutment displacements are consistent (within 30 percent) with the assumed stiffness.

4.2.11.3 - Transverse Direction

Transverse stiffness of abutments may be considered in the overall dynamic response of bridge systems on a case by case basis upon State Bridge Design Engineer approval.

Upon approval, the transverse abutment stiffness used in the elastic demand models may be taken as 50 percent of the elastic transverse stiffness of the adjacent bent.

Girder stops are typically designed to transmit the lateral shear forces generated by small to moderate earthquakes and service loads and are expected to fuse at the design event earthquake level of acceleration to limit the demand and control the damage in the abutments and supporting piles/shafts. Linear elastic analysis cannot capture the inelastic response of the girder stops, wingwalls or piles/shafts. Therefore, the forces generated with elastic demand assessment models should not be used to size the abutment girder stops. Girder stops for abutments supported on a spread footing shall be designed to sustain the lesser of the acceleration coefficient, A_s, times the superstructure dead load reaction at the abutment plus the weight of abutment and its footing or sliding friction forces of spread footings. Girder stops for pile/shaft supported foundations shall be designed to sustain the sum of 75 percent total lateral capacity of the piles/shafts and shear capacity of one wingwall.

The elastic resistance may be taken to include the use of bearings designed to accommodate the design displacements, soil frictional resistance acting against the base of a spread footing supported abutment, or pile resistance provided by piles acting in their elastic range.

The stiffness of fusing or breakaway abutment elements such as wingwalls (yielding or non-yielding), elastomeric bearings, and sliding footings shall not be relied upon to reduce displacement demands at intermediate piers.

Unless fixed bearings are used, girder stops shall be provided between all girders regardless of the elastic seismic demand. The design of girder stops should consider that unequal forces that may develop in each stop.

When fusing girder stops, transverse shear keys, or other elements that potentially release the restraint of the superstructure are used, then adequate support length meeting the requirements of Article 4.12 of the LRFD-SGS must be provided in the transverse direction as well as the longitudinal direction. Additionally, the expected redistribution of internal forces in the superstructure and other bridge system element must be considered. Bounding analyses considering incremental release of transverse restraint at each end of the bridge should also be considered.

4.2.11.4 - Curved and Skewed Bridges

Passive earth pressure at abutments may be considered as a key element of the ERS of straight and curved bridges with abutment skews up to 20 degrees. For larger skews, due to a combination of longitudinal and transverse response, the span has a tendency to rotate in the direction of decreasing skew. Such motion will tend to cause binding in the obtuse corner and generate uneven passive earth pressure forces on the abutment, exceeding the passive pressure near one end of the backwall, and providing little or no resistance at other end. This requires a more refined analysis to determine the amount of expected movement. The passive pressure resistance in soils behind semi-integral or L-shape abutments shall be based on the projected width of the abutment wall normal to the centerline of the bridge. Abutment springs shall be included in the local coordinate system of the abutment wall.

4.2.12 Foundation – General

LRFD-SGS Article 5.3.1 – The required foundation modeling method (FMM) and the requirements for estimation of foundation springs for spread footings, pile foundations, and drilled shafts shall be based on the WSDOT State Geotechnical Engineer's recommendations.

4.2.13 Foundation – Spread Footing

LRFD-SGS Article C5.3.2 – Foundation springs for spread footings shall be determined in accordance with Section 7.2.7, *Geotechnical Design Manual* Section 6.5.1.1 and the WSDOT State Geotechnical Engineer's recommendations.

4.2.14 Procedure 3: Nonlinear Time History Method

LRFD-SGS Article 5.4.4 – The time histories of input acceleration used to describe the earthquake loads shall be selected in consultation with the WSDOT Geotechnical Engineer and the WSDOT State Bridge Design Engineer.

4.2.15 I_{eff} for Box Girder Superstructure

LRFD-SGS Article 5.6.3 – Gross moment of inertia shall be used for box girder superstructure modeling.

4.2.16 Foundation Rocking

LRFD-SGS Article 6.3.9 – Foundation rocking shall not be used for the design of WSDOT bridges.

4.2.17 Drilled Shafts

LRFD-SGS Article C6.5 – For WSDOT bridges, the scale factor for p-y curves or subgrade modulus for large diameter shafts shall not be used unless approved by the WSDOT State Geotechnical Engineer and WSDOT State Bridge Design Engineer.

4.2.18 Longitudinal Direction Requirements

LRFD-SGS Article 6.7.1 – Case 2: Earthquake Resisting System (ERS) with abutment contribution may be used provided that the mobilized longitudinal passive pressure is not greater than 70 percent of the value obtained using procedure given in Article 5.2.3.3.

4.2.19 Liquefaction Design Requirements

LRFD-SGS Article 6.8 – Soil liquefaction assessment shall be based on the WSDOT State Geotechnical Engineer's recommendation and *Geotechnical Design Manual* Section 6.4.2.6.

4.2.20 Reinforcing Steel

LRFD-SGS Article 8.4.1 – Longitudinal reinforcement for ductile members in SDC's B, C & D, including foundations where in-ground-hinging is considered as part of the ERS, shall conform to ASTM A706 Grade 60. ASTM A706 Grade 80 for longitudinal reinforcement for ductile members in SDC's B, C & D, including foundations where in-ground-hinging is considered as part of the ERS may be used on a case-by-case basis with the WSDOT State Bridge Design Engineer's approval. See Section 5.1.2 for other requirements.

For SDCs B, C, and D, the moment-curvature analyses based on strain compatibility and nonlinear stress strain relations shall be used to determine the plastic moment capacities of all ductile concrete members. The properties of reinforcing steel, as specified in Table 8-4.2-1, shall be used.

Deformed welded wire fabric may be used with the WSDOT State Bridge Design Engineer's approval.

Table 8.4.2-1 Properties for Reinforcing Steel Bars

Property	Notation	Bar Size	ASTM A706 Grade 60	ASTM A706 Grade 80	ASTM A615 Grade 60
Specified minimum yield strength (ksi)	f_y	#3– #18	60	80	60
Expected yield strength (ksi)	f_{ye}	#3– #18	68	85	68
Expected tensile strength (ksi)	f_{ue}	#3– #18	95	112	95
Expected yield strain	ε_{ye}	#3– #18	0.0023	0.0033	0.0023
Tensile strain at the onset of strain hardening	ε_{sh}	#3– #8	0.0150	0.0074	0.0150
		#9	0.0125		0.0125
		#10 & #11	0.0115		0.0115
		#14	0.0075		0.0075
		#18	0.0050		0.0050
Reduced ultimate tensile strain	ε^R_{su}	#4– #10	0.090	0.060	0.060
		#11– #18	0.060		0.040
Ultimate tensile strain	ε_{su}	#4– #10	0.120	0.095	0.090
		#11– #18	0.090		0.060

4.2.21 *Concrete Modeling*

LRFD-SGS Article 8.4.4- Revise the last paragraph as follows:

Where in-ground plastic hinging approved by the WSDOT State Bridge Design Engineer is part of the ERS, the confined concrete core shall be limited to a maximum compressive strain as specified in AASHTO Guidelines for Performance-Based Seismic Table 3.2-4, and the member ductility demand shall be limited to 4 maximum. The clear spacing between the longitudinal reinforcements and between spirals and hoops in drilled shafts shall not be less than 6 inches or more than 8 inches when tremie placement of concrete is anticipated.

4.2.22 *Interlocking Bar Size*

LRFD-SGS Article 8.6.7 – The longitudinal reinforcing bar inside the interlocking portion of column (interlocking bars) shall be the same size of bars used outside the interlocking portion.

4.2.23 *Splicing of Longitudinal Reinforcement in Columns Subject to Ductility Demands for SDCs C and D*

LRFD-SGS Article 8.8.3 – The splicing of longitudinal column reinforcement outside the plastic hinging region shall be accomplished using mechanical couplers that are capable of developing the tensile strength of the spliced bar. Splices shall be staggered at least 2 feet. Lap splices shall not be used. The design engineer shall clearly identify the locations where splices in longitudinal column reinforcement are permitted on the plans. In general where the length of the rebar cage is less than 60 ft (72 ft for No. 14 and No. 18 bars), no splice in the longitudinal reinforcement shall be allowed.

4.2.24 Development Length for Column Bars Extended into Oversized Pile Shafts f or SDCs C and D

LRFD-SGS Article 8.8.10 – Extending column bars into oversized shaft shall be per Section 7.3.5.D.3, based on TRAC Report WA-RD 417.1 "Non-Contact Lap Splice in Bridge Column-Shaft Connections."

4.2.25 Lateral Confinement for Oversized Pile Shaft for SDCs C and D

LRFD-SGS Article 8.8.12 – The requirement of this article for shaft lateral reinforcement in the column-shaft splice zone may be replaced with Section 7.8.2.11.

4.2.26 Lateral Confinement for Non-Oversized Strengthened Pile Shaft for SDCs C and D

LRFD-SGS Article 8.8.13 – Non oversized column shaft (the cross section of the confined core is the same for both the column and the pile shaft) is not permissible unless approved by the WSDOT State Bridge Design Engineer.

4.2.27 Requirements for Capacity Protected Members

LRFD-SGS Article 8.9 – Add the following paragraphs:

For SDCs C and D where liquefaction is identified, with the WSDOT State Bridge Design Engineer's approval, pile and drilled shaft in-ground hinging may be considered as an ERE. Where in-ground hinging is part of ERS, the confined concrete core should be limited to a maximum compressive strain of 0.008 and the member ductility demand shall be limited to 4.

Bridges shall be analyzed and designed for the non-liquefied condition and the liquefied condition in accordance with Article 6.8. The capacity protected members shall be designed in accordance with the requirements of Article 4.11. To ensure the formation of plastic hinges in columns, oversized pile shafts shall be designed for an expected nominal moment capacity, M_{ne}, at any location along the shaft, that is, equal to 1.25 times moment demand generated by the overstrength column plastic hinge moment and associated shear force at the base of the column. The safety factor of 1.25 may be reduced to 1.0 depending on the soil properties and upon the WSDOT State Bridge Design Engineer's approval.

The design moments below ground for extended pile shaft may be determined using the nonlinear static procedure (pushover analysis) by pushing them laterally to the displacement demand obtained from an elastic response spectrum analysis. The point of maximum moment shall be identified based on the moment diagram. The expected plastic hinge zone shall extend 3D above and below the point of maximum moment. The plastic hinge zone shall be designated as the "no splice" zone and the transverse steel for shear and confinement shall be provided accordingly.

4.2.28 Superstructure Capacity Design for Transverse Direction (Integral Bent Cap) for SDCs C and D

LRFD-SGS Article 8.11 – Revise the last paragraph as follows:

For SDCs C and D, the longitudinal flexural bent cap beam reinforcement shall be continuous. Splicing of cap beam longitudinal flexural reinforcement shall be accomplished using mechanical couplers that are capable of developing the tensile strength of the spliced bar. Splices shall be staggered at least 2 feet. Lap splices shall not be used.

4.2.29 Superstructure Design for Non Integral Bent Caps for SDCs B, C, and D

LRFD-SGS Article 8.12 – Non integral bent caps shall not be used for continuous concrete bridges in SDC B, C, and D except at the expansion joints between superstructure segments.

4.2.30 Joint Proportioning

LRFD-SGS Article 8.13.4.1.1 – Revise the last bullet as follows:

Exterior column joints for box girder superstructure and other superstructures if the cap beam extends the joint far enough to develop the longitudinal cap reinforcement.

4.2.31 Cast-in-Place and Precast Concrete Piles

LRFD-SGS Article 8.16.2 – Minimum longitudinal reinforcement of 0.75 percent of A_g shall be provided for CIP piles in SDCs B, C, and D. Longitudinal reinforcement shall be provided for the full length of pile unless approved by the State Bridge Design Engineer.

4.2.32 Seismic Resiliency using Innovative Materials and Construction

Innovative materials and bridge construction are ideas that encourage engineers to consider principles that will enhance bridge performance, speed up construction, or add any other benefit to the industry. BDM Section 14.4 describes the self-centering columns that are designed restore much of their original shape after a seismic event. They're intended to improve the serviceability of a bridge after an earthquake. Self-centering columns are constructed with a precast concrete column segment with a duct running through it longitudinally. They rest on footings with post-tensioning (PT) strand developed into them. Once the precast column piece is set on the footing, the PT strand threads through the duct and gets anchored into the crossbeam above the column. The PT strand is unbonded to the column segment. As a column experiences a lateral load, the PT strand elastically stretches to absorb the seismic energy and returns to its original tension load after the seismic event. The expectation is the column would rotate as a rigid body and the PT strand would almost spring the column back to its original orientation.

Self-centering in bridge columns can be achieved using Shape Memory Alloy (SMA) and Engineered Cementitious Composite (ECC). These products are introduced into bridge design as a means to improve ductility, seismic resiliency, and serviceability of a bridge after an earthquake. SMA is a class of alloys that are manufactured from either a combination of nickel and titanium or copper, magnesium and aluminum. The alloy is shaped into round bars in sizes similar to conventional steel reinforcement. When stressed, the SMA can undergo large deformations and return to original shape.

4.3 Seismic Design Requirements for Bridge Modifications and Widening Projects

4.3.1 General

A bridge modification or widening is defined as where substructure bents are modified and new columns or piers are added, or an increase of bridge deck width or widenings to the sidewalk or barrier rails of an existing bridge resulting in significant mass increase or structural changes.

Bridge widenings in Washington State shall be designed in accordance with the requirements of the current edition of the LRFD-BDS. The seismic design of Ordinary, Recovery and Critical bridges shall be in accordance with the requirements of the LRFD-SGS, and WSDOT BDM.

The spectral response parameters shall be as defined in Section 4.2.3. The widening portion (new structure) shall be designed to meet current WSDOT standards for new Ordinary, Recovery and Critical bridges. Seismic analysis is required in accordance with Section 4.3.3 and is not required for single span bridges and bridges in SDC A. However, existing elements of single span bridges shall meet the requirements of LRFD-SGS as applicable.

4.3.2 Bridge Widening Project Classification

Bridge widening projects are classified according to the scope of work as either minor or major widening projects.

4.3.2.A Minor Modification and Widening Projects

A bridge widening project is classified as a minor widening project if all of the following conditions are met:

- Substructure bents are not modified and no new columns or piers are added, while abutments may be widened to accommodate the increase of bridge deck width.
- The net superstructure mass increase is equal or less than 10 percent of the original superstructure mass.
- Fixity conditions of the foundations are unchanged.
- There are no major changes of the seismicity of the bridge site that can increase seismic hazard levels or reduce seismic performance of the structure since the initial screening or most recent seismic retrofit.
- No additional girder lines are added.

4.3.2.B Major Modifications and Widening Projects

A bridge widening project is classified as a major widening project if any of the following conditions are met:

- Substructure bents are modified and new columns or piers are added, excepting abutments, which may be widened to accommodate the increase of bridge deck width.
- The net superstructure mass increase is more than 20 percent of the original superstructure mass.
- Fixity conditions of the foundations are changed.
- There are major changes of the seismicity of the bridge site that can increase seismic hazard levels or reduce seismic performance of the structure since the initial screening or most recent seismic retrofit.
- Girder lines are added.

Major changes in seismicity include, but are not limited to, the following: near fault effect, significant liquefaction potential, or lateral spreading. If there are concerns about changes to the Seismic Design Response Spectrum at the bridge site, about a previous retrofit to the existing bridge, or an unusual imbalance of mass distribution resulting from the structure widening, the designer should consult the WSDOT Bridge and Structures Office.

4.3.3 Seismic Design Requirements Bridge Widening Projects

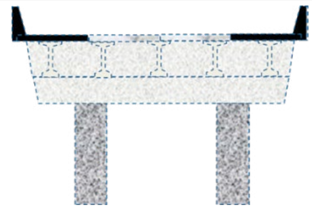
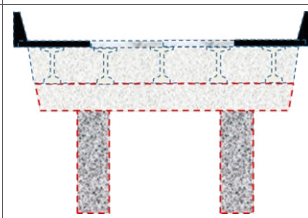
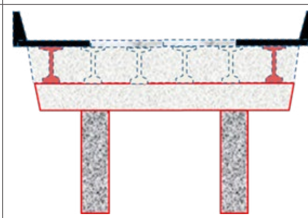
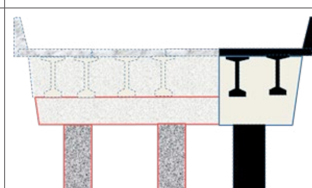
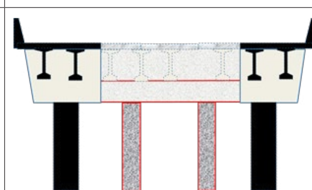
The Seismic Design requirements for Bridge Modifications and Widening are as follows and as illustrated in BDM Figure 4.3-1:

1. Ordinary bridge modification or widening projects classified as Minor Modification or Widening do not require either a seismic evaluation or a retrofit of the structure. If the conditions for Minor Modification or Widening project are met, it is anticipated that the modified or widened structure will not draw enough additional seismic demand to significantly affect the existing sub-structure elements.

2. Seismic analysis is required for all Major Modifications and Widening projects at project scoping level in accordance with Section 4.1. A complete seismic analysis is required for Ordinary bridges in Seismic Design Category (SDC) B, C, and D for major modifications and widening projects as described below. A project geotechnical report (including any unstable soil or liquefaction issues) shall be available to the structural engineer for seismic analysis. Seismic analysis shall be performed for both existing and widened structures. Capacity/Demand (C/D) ratios are required for existing bridge elements including foundation. The “Do No Harm” policy requires the designer to compare the C/D ratios of the existing foundation elements (footings, pile caps, piles, and shafts) in the before widening condition to those of the after widening condition. If the C/D ratios are not decreased, the widening can be designed and constructed without retrofitting existing deficient bridge foundations. Foundation elements with C/D ratios made worse by the widening/modification work shall be retrofitted to restore their C/D ratios to before-widening values. However, seismic improvement of the existing foundation elements could be deferred to the Bridge Seismic Retrofit Program.

3. The widening portion of the structure shall be designed for liquefiable soils condition in accordance to the LRFD-SGS, and WSDOT BDM, unless soils improvement is provided to eliminate liquefaction.
4. Procedure for Ordinary Bridges: Seismic improvement of existing columns and crossbeams to C/D > 1.0 is required. The cost of seismic improvement shall be paid for with widening project funding (not from the Retrofit Program). The seismic retrofit of the existing Ordinary structure shall conform to the BDM, while the newly widened portions of the bridge shall comply with the LRFD-SGS, except for balanced stiffness criteria, which may be difficult to meet due to the existing bridge configuration. However, the designer should strive for the best balanced frame stiffness for the entire widened structure that is attainable in a cost effective manner. Major Modification and Widening Projects require the designer to determine the seismic C/D ratios of the existing bridge elements in the final widened condition. If the C/D ratios of columns and crossbeam of existing structure are less than 1.0, the improvement of seismically deficient elements is mandatory and the widening project shall include the improvement of existing seismically deficient bridge elements to C/D ratio of above 1.0. The C/D ratio of 1.0 is required to prevent the collapse of the bridge during the seismic event as required for life safety. Seismic improvement of the existing foundation elements (footings, pile caps, piles, and shafts to C/D ratios > 1.0) could be deferred to the Bridge Seismic Retrofit Program.
5. Procedure for Recovery/Critical Bridges: The initial goal is to conduct the seismic design effort so the composite structure (existing bridge and widening) meet requirements of the two-level seismic design (FEE and SEE) described in BDM Section 4.1. This includes the superstructure, substructure and foundation elements of the composite structure. Retrofitting or strengthening of the existing structure may be necessary to achieve this. Depending on the year the bridge was constructed, type of foundation and capacity of the soils during a seismic event, it may become expensive to meet this goal. If the Engineer determines it is cost prohibitive to meet the two-level design criteria, the State Bridge Design Engineer may approve deviations. Examples of potential deviations include:
   A. Meeting two-level design criteria for the widened portion, but only achieving Ordinary bridge criteria for the existing bridge.
   B. Meeting two-level design criteria for the above-ground portions of the composite structure, but not achieving this for the below-ground portions (foundations).
   C. Performing a two-level design, but requiring deviations from the displacement ductility demand limits identified in BDM Section 4.1.
   D. Only achieving Ordinary (no collapse) criteria for the composite structure.

Figure 4.3-1 Seismic Design Criteria for Bridge Modifications and Widening

Modifications or Widening	Alterations	Illustration	Seismic Design Guidance
Minor Modifications • Deck Rehabilitations • Traffic Barrier Replacements • sidewalk addition/ rehabilitation • No added girder lines	• Superstructure mass increase is less than 10% • Fixity conditions are not changed		• Do not Require seismic evaluation • Do not require retrofit of the structure
Major Modifications Minor Modifications PLUS • Replacing/adding girder and slab	• Superstructure mass increase up to 20% and/or • Fixity conditions are changed		• Seismic evaluation of the structure is required. • C/D ratio of equal or greater than 1.0 is required for substructure. • Do-No-Harm could be used for foundation.
Major Widening – Case 1 Minor Modifications PLUS • Superstructure or Bent Widening	• Superstructure mass increase is more than > 20% and/or • Substructure/bents modified and/or • Fixity conditions are changed		• Seismic evaluation of the structure is required. • C/D ratio of equal or greater than 1.0 is required for substructure. • Do-No-Harm could be used for Foundation.
Major Widening – Case 2 • widening on one side	• Substructure or bents are modified. Columns are added on one side.		• Seismic evaluation of the structure is required. • C/D ratio of equal or greater than 1.0 is required for substructure. • Do-No-Harm could be used for Foundation.
Major Widening – Case 3 • widening on both sides	• Substructure or bents are modified. Columns are added on both sides.		• Seismic evaluation of the structure is required. • C/D ratio of equal or greater than 1.0 is required for substructure. • Do-No-Harm could be used for Foundation.

4.3.4 Scoping for Bridge Widening and Liquefaction Mitigation

The Region project manager should contact the Bridge Office for bridge widening and retaining wall scoping assistance before project funding commitments are made to the legislature and the public. The WSDOT Bridge and Structures office will work with the WSDOT Geotechnical Office to assess the potential for liquefaction or other seismic hazards that could affect the cost of the proposed structures. The initial evaluation design time and associated costs for the WSDOT Geotechnical and WSDOT Bridge and Structures offices shall be considered at the scoping phase.

4.3.5 Design and Detailing Considerations

Support Length – The support length at existing abutments, piers, in-span hinges, and pavement seats shall be checked. If there is a need for longitudinal restrainers, transverse restrainers, or additional support length on the existing structure, they shall be included in the widening design.

Connections Between Existing and New Elements – Connections between the new elements and existing elements should be designed for maximum over-strength forces. Where yielding is expected in the crossbeam connection at the extreme event limit state, the new structure shall be designed to carry live loads independently at the Strength I limit state. In cases where large differential settlement and/or a liquefaction-induced loss of bearing strength are expected, the connections may be designed to deflect or hinge in order to isolate the two parts of the structure. Elements subject to inelastic behavior shall be designed and detailed to sustain the expected deformations.

Longitudinal joints between the existing and new structure are not permitted.

Differential Settlement – The geotechnical designer should evaluate the potential for differential settlement between the existing structure and widening structure. Additional geotechnical measures may be required to limit differential settlements to tolerable levels for both static and seismic conditions. The bridge designer shall evaluate, design, and detail all elements of new and existing portions of the widened structure for the differential settlement warranted by the WSDOT State Geotechnical Engineer. Angular distortions between adjacent foundations greater than 0.008 (RAD) in simple spans and 0.004 (RAD) in continuous spans should not be permitted in settlement criteria.

The horizontal displacement of pile and shaft foundations shall be estimated using procedures that consider soil-structure interaction (see *Geotechnical Design Manual* Section 8.12.2.3). Horizontal movement criteria should be established at the top of the foundation based on the tolerance of the structure to lateral movement with consideration of the column length and stiffness. Tolerance of the superstructure to lateral movement will depend on bridge seat widths, bearing type(s), structure type, and load distribution effects.

Foundation Types – The foundation type of the new structure should match that of the existing structure. However, a different type of foundation may be used for the new structure due to geotechnical recommendations or the limited space available between existing and new structures. For example, a shaft foundation may be used in lieu of spread footing.

Existing Strutted Columns – The horizontal strut between existing columns may be removed. The existing columns shall then be analyzed with the new unbraced length and retrofitted if necessary.

Non Structural Element Stiffness – Median barrier and other potentially stiffening elements shall be isolated from the columns to avoid any additional stiffness to the system.

Deformation capacities of existing bridge members that do not meet current detailing standards shall be determined using the provisions of Section 7.8 of the *Retrofitting Manual for Highway Structures: Part 1 – Bridges*, FHWA-HRT-06-032. Deformation capacities of existing bridge members that meet current detailing standards shall be determined using the latest edition of the LRFD-SGS.

Joint shear capacities of existing structures shall be checked using *Caltrans Bridge Design Aid*, 14-4 Joint Shear Modeling Guidelines for Existing Structures.

In lieu of specific data or using the parameters given in Table 3.2-4 of AASHTO Guidelines for Performance-Based Seismic Design of Highway Bridges, the reinforcement properties provided in Table 4.3.5-1 shall be used.

Table 4.3.5-1 Stress Properties of Reinforcing Steel Bars

Property	Notation	Bar Size	ASTM A706	ASTM A615 Grade 60	ASTM A615 Grade 40*
Specified minimum yield stress (ksi)	f_y	No. 3 - No. 18	60	60	40
Expected yield stress (ksi)	f_{ye}	No. 3 - No. 18	68	68	48
Expected tensile strength (ksi)	f_{ue}	No. 3 - No. 18	95	95	68
Expected yield strain	ε_{ye}	No. 3 - No. 18	0.0023	0.0023	0.00166
Onset of strain hardening	ε_{sh}	No. 3 - No. 8	0.0150	0.0150	0.0193
		No. 9	0.0125	0.0125	
		No. 10 & No. 11	0.0115	0.0115	
		No. 14	0.0075	0.0075	
		No. 18	0.0050	0.0050	
Reduced ultimate tensile strain	ε_{su}^{R}	No. 4 - No. 10	0.090	0.060	0.090
		No. 11 - No. 18	0.060	0.040	0.060
Ultimate tensile strain	ε_{su}	No. 4 - No. 10	0.120	0.090	0.120
		No. 11 - No. 18	0.090	0.060	0.090

* ASTM A615 Grade 40 is for existing bridges in widening projects.

Isolation Bearings – Isolation bearings may be used for bridge widening projects to reduce the seismic demand through modification of the dynamic properties of the bridge. These bearings are a viable alternative to strengthening weak elements or non-ductile bridge substructure members of the existing bridge. Use of isolation bearings needs the approval of State Bridge Design Engineer. Isolation bearings shall be designed per the requirements specified in Section 9.3.

4.4 Seismic Retrofitting of Existing Ordinary Bridges

Seismic retrofitting of existing ordinary bridges shall be performed in accordance with the FHWA publication FHWA-HRT-06-032, *Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges* and WSDOT amendments as follows:

- Article 1.5.3 The spectral response parameters shall be determined as defined in Section 4.2.3.
- Article 7.4.2 Seismic Loading in Two or Three Orthogonal Directions

Revise the first paragraph as follows:

When combining the response of two or three orthogonal directions the design value of any quantity of interest (displacement, bending moment, shear or axial force) shall be obtained by the 100-30 percent combination rule as described in LRFD-SGS Article 4.4.

- Delete Eq. 7.44 and replace with the following:

$$L_p = \text{the maximum of } [(8800\varepsilon_y d_b) \text{ or } (0.08L + 4400\varepsilon_y d_b)] \tag{7-44}$$

- Delete Eq. 7.49 and replace with the following:

$$\phi_p = \left(5\left(\frac{V_i - V_m}{V_i - V_f}\right) + 2\right)\phi_y \tag{7.49}$$

- Delete Eq. 7.51 and replace with the following:

$$\phi_p = \left(4\left(\frac{V_{ji} - V_{jh}}{V_{ji} - V_{jf}}\right) + 2\right)\phi_y \tag{7.51}$$

The seismic retrofit of Recovery and Critical bridges shall be in accordance with the requirements of the WSDOT BDM with consultation of Bridge Design Engineer and Geotechnical with regard to practicability and cost.

4.4.1 Seismic Analysis Requirements

The seismic retrofit of Ordinary, Recovery and Critical bridges shall be in accordance with the requirements of the Seismic Retrofitting Manual, and WSDOT BDM. For Ordinary bridges, the seismic analysis need only be performed for the SEE as defined in Section 4.1.1 with a life safety seismic performance level. For Recovery and Critical Bridges, the seismic design required for Ordinary bridges shall be performed and adequacy of the existing foundation for lower level seismic demand shall be investigated. The lower level earthquake has a return period of about 210 years (FEE defined in Section 4.1.1). A summary of C/D ratios for all elements shall be provided. With the approval of the WSDOT State Bridge and Structures, State Bridge Design and State Geotechnical Engineers the retrofit of foundation elements with seismic deficiencies could be deferred to the Seismic Retrofit Program.

The first step in retrofitting a bridge is to analyze the existing structure to identify seismically deficient elements. The initial analysis consists of generating capacity/demand ratios for all relevant bridge components. Seismic displacement and force demands shall be determined using the multi-mode spectral analysis of Section 5.4.2.2 (at a minimum). Prescriptive requirements, such as support length, shall be considered a demand and shall be included in the analysis. Seismic capacities shall be determined in accordance with the requirements of the *Seismic Retrofitting Manual*. Displacement capacities shall be determined by the Method D2 – Structure Capacity/Demand (Pushover) Method of Section 5.6.

4.4.2 Seismic Retrofit Design

Once seismically deficient bridge elements have been identified, appropriate retrofit measures shall be selected and designed. Table 1-11, Chapters 8, 9, 10, 11, and Appendices D thru F of the *Seismic Retrofitting Manual* shall be used in selecting and designing the seismic retrofit measures. The WSDOT Bridge and Structure Office Seismic Specialist will be consulted in the selection and design of the retrofit measures.

4.4.3 Computer Analysis Verification

The computer results will be verified to ensure accuracy and correctness. The designer should use the following procedures for model verification:

- Using graphics to check the orientation of all nodes, members, supports, joint, and member releases. Make sure that all the structural components and connections correctly model the actual structure.
- Check dead load reactions with hand calculations. The difference should be less than 5 percent.
- Calculate fundamental and subsequent modes by hand and compare results with computer results.
- Check the mode shapes and verify that structure movements are reasonable.
- Increase the number of modes to obtain 90 percent or more mass participation in each direction. GTSTRUDL/CSiBridge directly calculates the percentage of mass participation.
- Check the distribution of lateral forces. Are they consistent with column stiffness? Do small changes in stiffness of certain columns give predictable results?

4.4.4 Earthquake Restrainers

Longitudinal restrainers shall be high strength steel rods conform to ASTM F 1554 Grade 105, including Supplement Requirements S2, S3 and S5. Nuts, and couplers if required, shall conform to ASTM A 563 Grade DH. Washers shall conform to AASHTO M 293. High strength steel rods and associated couplers, nuts and washers shall be galvanized after fabrication in accordance with AASHTO M 232 or epoxy coated. The length of longitudinal restrainers shall be less than 24 feet.

4.4.5 Isolation Bearings

Isolation bearings may be used for seismic retrofit projects to reduce the demands through modification of the dynamic properties of the bridge as a viable alternative to strengthening weak elements of non-ductile bridge substructure members of existing bridge. Use of isolation bearings needs the approval of WSDOT State Bridge Design Engineer. Isolation bearings shall be designed per the requirements specified in Section 9.3.

4.5 Seismic Design Requirements for Retaining Walls and Buried Structure

4.5.1 Seismic Design of Retaining Walls

Seismic design of all retaining walls shall conform to LRFD-SGS as modified by this section.

The definition for Ordinary, Critical, and Recovery retaining wall structures is provided in Section 4.1 General. References to the terminology "bridges" utilized in Section 4.1 General shall be replaced by "retaining walls".

Table 4.5-1 Retaining Wall Seismic Hazard Evaluation Levels

Retaining Wall Operational Importance Category	Seismic Hazard Evaluation Level
"Ordinary Retaining Walls" Eastern Washington	SEE
"Ordinary Retaining Walls" Western Washington (Not Lifeline)	SEE
	FEE
"Recovery Retaining Walls" (Lifeline)	SEE
	FEE
"Critical Retaining Walls"	SEE
	FEE

The risk-targeted ground surface acceleration (A_S) for the Safety Evaluation Earthquake (SEE) shall be determined in accordance with LRFD-SGS Article 3.4.1 and 3.4.2. The AASHTO-USGS Seismic Design Maps, AASHTO-2023 Data Sets (https://earthquake.usgs.gov/ws/designmaps/aashto-2023/), or a site-specific procedure, shall be utilized to determine A_S.

The risk-targeted ground surface acceleration (A_S) for the Functional Evaluation Earthquake (FEE) shall be determined in accordance with LRFD-BDS Article 3.10.4.1, except that uniform hazard acceleration coefficients shall be obtained from the USGS Earthquake Hazard Toolbox (https://earthquake.usgs.gov/nshmp/), or with a site specific procedure.

When obtaining the design acceleration coefficients for the FEE,

- The calculated Site Class shall be utilized, rather than the calculated Vs30 value (note that shear wave velocities are reported in meters per second on the above website)
- A return period of 210 years shall be utilized.
- Interpolate between data points in log-log space.

4.5.2 Seismic Design of Buried Structure

Buried Structures shall be designed for seismic effects in accordance with the requirements in Section 8.3.3.H.

4.99 References

1. See Section 1.99.
2. AASHTO Guidelines for Performance-Based Seismic Design of Highway Bridges, 1st Edition, 2023
3. Caltrans *Bridge Design Aids* 14 4 Joint Shear Modeling Guidelines for Existing Structures, California Department of Transportation, August 2008
4. FHWA *Seismic Retrofitting Manual for Highway Structures: Part 1 Bridges*, Publication No. FHWA-HRT-06-032, January 2006
5. McLean, D.I. and Smith, C.L., *Noncontact Lap Splices in Bridge Column-Shaft Connections*, Report Number WA-RD 417.1, Washington State University

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