Oregon Department of Transportation

SECTION 1.17
Seismic and Tsunami Design

Revisions for the current edition are highlighted yellow. Deleted text is not marked.

Past editions of the Bridge Design Manual are available through our website.

1.17 Seismic and Tsunami Design

1.17.1 Design Philosophy

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1.17.1 Design Philosophy

C1.17.1

Design all new bridges for full seismic loading according to the *3rd edition of AASHTO Guide Specifications for LRFD Seismic Bridge Design* (*AASHTO Guide Specifications*) as modified herein.

Unless specifically modified in this document, each section of the *3rd Edition of AASHTO Guide Specifications* must be used in its entirety.

Notify and consult ODOT Bridge Section for decisions involving deviations to the standard seismic design practices described in this manual. Submit a Design Deviation Request and obtain approval by the State Bridge Engineer for deviations from the following guidelines. Include the approved Design Deviation Request and any related files in the Calculation Book and reference the Design Deviation in the General Notes section of the Contract Plans.

At the end of the design process, complete and submit a copy of the Seismic

Design/Retrofit Data Sheet to ODOT Bridge Section mailbox (ODOTBridgeEngineeringSection@odot.oregon.gov).

NON-CONVENTIONAL BRIDGES

For seismic design of non-conventional bridges or bridges classified as Critical, develop a Project Specific Seismic Design Criteria (PSDC). Obtain approval by the State Bridge Engineer for PSDC no later than Design Verification Package submittals.

C NON-CONVENTIONAL BRIDGES

AASHTO Guide Specifications 3.1 states that the seismic provisions provided in that publication are applicable for conventional bridges classified as Ordinary.

1.17.2 Specification Interpretations and Modifications

C1.17.2

1.17.2.1 Bridge Operational Categories

C 1.17.2.1

For the purpose of seismic design, highway bridges are prioritized in one of the following categories based on the desired post-earthquake operational performance:

- Ordinary bridges.
- Recovery bridges.
- Critical bridges.

ORDINARY BRIDGES

Ordinary bridges are expected to experience major damage and may be closed following Upper Level Motion, but no span is expected to collapse during this motion.

All bridges not designated as Recovery or Critical are designated as Ordinary.

C ORDINARY BRIDGES

RECOVERY BRIDGES

As a minimum, recovery bridges are expected to remain open to emergency vehicles and for security/defense purposes after inspection and clearance of debris following the Ground Motion of a full rupture of Cascadia Subduction Zone Earthquake (CSZE).

Highway bridges carrying the State Seismic Lifeline Routes are designated as Recovery bridges. Refer to *Figure 1.17.2.1-1* for highway routes designated as Seismic Lifeline Routes. An interactive map of Seismic Lifeline Routes is available at ODOT TransGIS.

C RECOVERY BRIDGES

CRITICAL BRIDGES

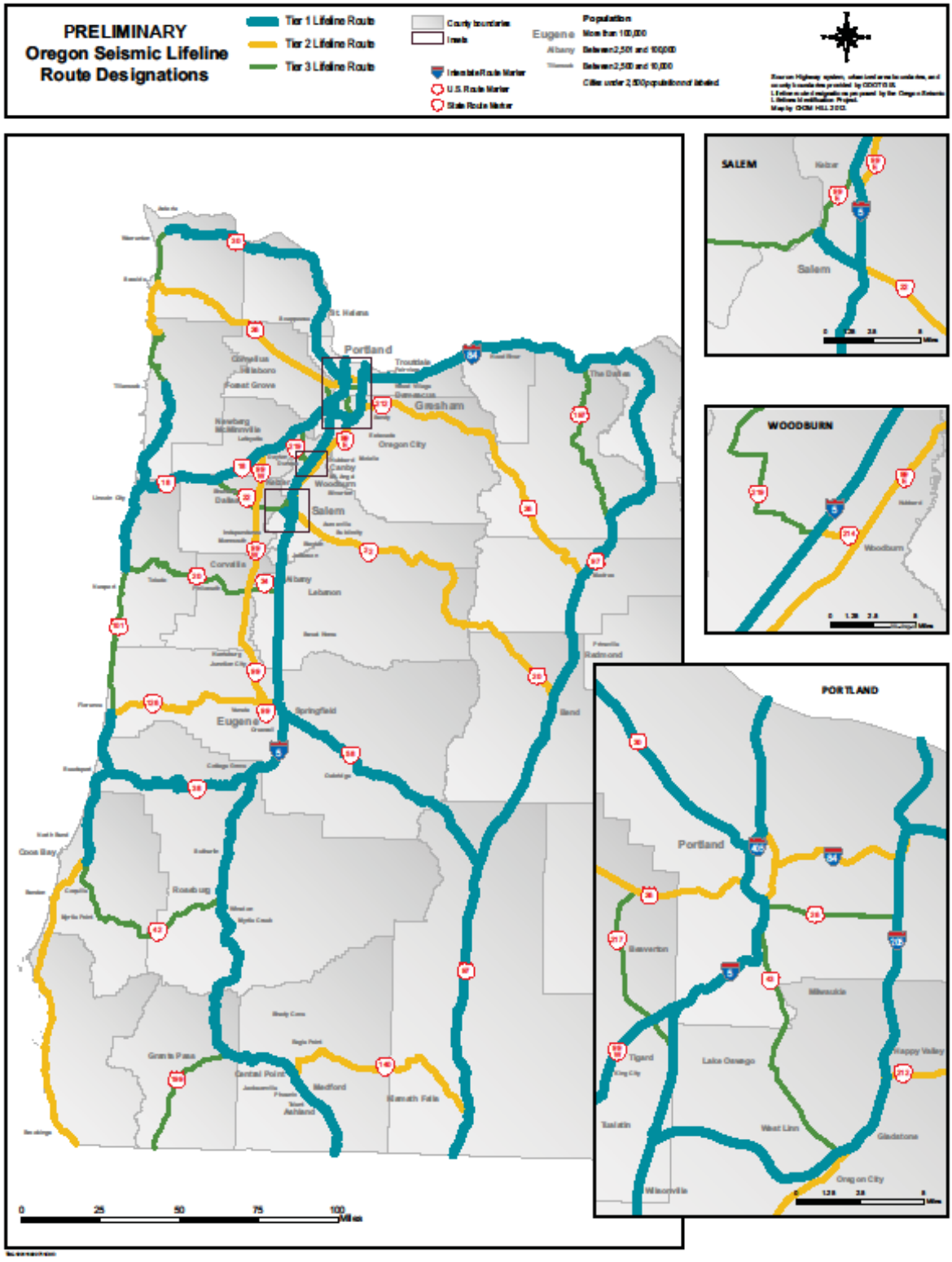
Critical bridges are expected to remain open to all traffic after inspection and clearance of debris following the CSZE and Upper Level Motion. These bridges are expected to be usable by emergency vehicles and for security/defense purposes after an earthquake larger than the Upper Level Motion.

Critical bridges will be designated by the sponsoring ODOT district in consultation with Bridge Engineering Section.

C CRITICAL BRIDGES

By 2022, ODOT has not designated any of the new bridges as Critical.

Figure 1.17.2.1-1 Seismic Lifeline Routes



1.17.2.2 Seismic Performance Levels

C1.17.2.2

Based on Operation Category, seismic design and detailing must provide at least one of the following seismic bridge performances:

- Life Safety.
- Operational.
- Fully Operational.

LIFE SAFETY

Significant damage is sustained during an earthquake and service is significantly disrupted, but life safety is assured. The bridge may need to be replaced after a large earthquake.

OPERATIONAL

Damage sustained is minimal and full service for emergency vehicles should be available after inspection and clearance of debris. Bridge should be reparable with or without restrictions on traffic flow.

FULLY OPERATIONAL

Damage sustained is negligible and full service is available for all vehicles after inspection and clearance of debris. Any damage is repairable without interruption to traffic.

C LIFE SAFETY

C OPERATIONAL

C FULLY OPERATIONAL

1.17.2.3 Seismic Performance Criteria

C 1.17.2.3

Design bridges to meet the performance requirements specified in *Table 1.17.2.3-1*.

Table 1.17.2.3-1 Performance Requirements for each Bridge Operation Category (Ordinary/Recovery/Critical) and Design Ground Motion

Ground Motions	Ordinary	Recovery	Critical
Lower Level (Full rupture CSZE)	N/A[1]	PL2 – Operational	PL3 – Fully Operational
Upper Level (AASHTO 2023 Risk-Targeted Ground Motions)	PL1 – Life Safety	PL1 – Life Safety	PL2 – Operational

[1] – AASHTO 2023 Risk-Targeted Ground Motions of a 1,000 year earthquake are expected to be higher than ground motions of a full rupture of Cascadia Subduction Zone Earthquake anywhere in Oregon. Contact Bridge Engineering Section if encountering an unusual circumstance.

[2] – These ground motions have a targeted risk of incipient bridge column collapse of 1.5 percent in 75 years using the notional fragility function.

Refer to *Table 1.17.2.3-2* for description of the Performance Levels and earthquake damage.

Table 1.17.2.3-2 Bridge Performance Levels and Associated Damage Descriptors

PL1: Life Safety	**PL2: Operational**	**PL3: Fully Operational**
Significant damage is sustained during an earthquake and service is significantly disrupted, but the potential for collapse is minimized. The bridge may need to be replaced after a large earthquake.	Damage sustained is minimal and access for emergency vehicles is available after inspection and clearance of debris. The bridge should be reparable with or without restrictions on traffic flow.	No damage (or very minor damage that does not require immediate attention) is sustained and full service is available for all vehicles immediately after the earthquake.
Significant damage includes permanent offsets and cracking. Exposed, buckled, and possibly some fractured reinforcing steel. Repair may be possible, but will require invasive measures that may include column replacement. At a minimum, reinforcing bar segments are replaced or plastic hinge relocation techniques employed, if repair is attempted. Beams may be unseated from bearings but no span is expected to collapse. Similarly, foundations are not damaged except in the event of large lateral flows due to liquefaction, in which case inelastic deformation in piles may be evident. Undesirable failure modes such as shear failure in reinforced concrete are avoided.	*Minimal damage* includes minor inelastic response and narrow flexural cracking in concrete. Exposed reinforcing steel, but not visibly buckled. Damage requires repair using minimally invasive techniques that range from simple patching of cover concrete and epoxy injection to grouted steel jackets. Permanent deformations are not apparent and repairs can be made under nonemergency conditions with the possible exception of superstructure expansion joints which may need removal and replacement. Replacement of strip seals and other non-load-carrying elements of modular joints may be necessary, but structural elements, including blockouts, have experienced only minor damage.	*Very minor damage* consists of minor cracking of concrete, possible incipient crushing or flaking of concrete cover.

Refer to *Table 1.17.2.3-3* for Post-Earthquake Residual Displacement Limits.

Table 1.17.2.3-3 Post-Earthquake Residual Displacement Limits[1] for each Bridge Performance Level (PL)

Case No.	Engineering Design Parameters	PL1: Life Safety	PL2: Operational	PL3: Fully Operational
1	Superstructure-to-abutment permanent vertical offset	No Limits	≤ 6 inches	≤ 1inch
2	Superstructure-to-abutment permanent horizontal offset	No Limits	≤ 6 inches	≤ 1 inch
3	Approach fill settlement limit of approach slab (longitudinal differential settlement)	≤ 1/20	≤ 1/50	≤ 1/250
4	Approach fill settlement without an approach slab	≤ 12 inches	≤ 6 inches	≤ 1 inch
5	Lateral soil movement	≤ 36 inches	≤ 12 inches	≤ 6 inches

[1] – Displacement limits provided in this table are driven mainly by geometric parameters required for the expected rideability and accessibility standards for a given performance level which, as a minimum, should provide the life safety of traveling public. Bridge type and size may not always provide the structural capacity to tolerate this magnitude of deformations and should control over these limits.

Illustration photos of engineering design parameters provided in *Table 1.17.2.3-3* are shown on *Figure 1.17.2.3-1.*

Figure 1.17.2.3-1 Illustration Cases of Engineering Design Parameters



Case 1 (Photo courtesy of Mark Yashinsky)



Case 2 (Photo courtesy of Mark Yashinsky)



Case 3 (Photo courtesy of Caltrans)



Case 4 (Photo courtesy of Mark Yashinsky)



Case 5 (Photo courtesy of Mark Yashinsky)

POST-EARTHQUAKE STRUCTURAL CAPACITY CHECK

For PL2 and PL3, when significant permanent displacements are identified by the geotechnical engineer, perform bridge performance evaluation in the post-earthquake condition during the design phase of the project. Ensure the structural capacity of all affected bridge members. Use the following load combinations and load factors:

- PL2: Strength II Load Combination using Oregon Legal Loads as defined in *ODOT LRFR 1.5.1.2* and EV3 with IM = 25 percent.
- PL3: Strength I Load Combination.

ODOT PERFORMANCE CRITERIA

To satisfy the "**Life Safety**" criteria, comply with the following requirements:

Seismic Design Categories (SDC) A, B and C

- Meet all design requirements for SDC A, B and C according to the *AASHTO Guide Specifications*.

Seismic Design Category (SDC) D

- Meet all design requirements for SDC D according to the *AASHTO Guide Specifications*, except as modified below:
    - The maximum concrete strain in confined section of the columns (ε_{cc}) does not exceed 90 percent of the ultimate concrete strain (ε_{cu}), computed by Mander's model.
    - The maximum strain of reinforcing steel does not exceed the ε^{R}_{su} as defined on *Table 8.4.2-1* of the *AASHTO Guide Specs*.

C POST-EARTHQUAKE STRUCTURAL CAPACITY CHECK

C ODOT PERFORMANCE CRITERIA

These guidelines are applicable even for the other Seismic Design Categories, if pushover analysis will be used instead of the implicit equation.

- The maximum strain of prestressing steel does not exceed $\varepsilon^{R}_{ps,u} = 0.03$

To satisfy the "**Operational**" criteria, comply with the following requirements and guidelines:

Seismic Design Categories (SDC) A, B, C and D

- Verify the "Operational" performance for Cascadia Subduction Zone Earthquake when potentially liquefiable soils are present on site.

Seismic Design Categories (SDC) A and B

- No structural analysis is required for "Operational" criteria.

Seismic Design Category (SDC) C

- Satisfy *equation 4.8-1* of the *AASHTO Guide Specifications* ($\Delta^{L}_{D} < \Delta^{L}_{C}$) for each bridge bent, where Δ^{L}_{C} is determined from the *equation 4.8.1.1* of the *AASHTO Guide Specifications* (displacement capacity for SDC B).

Seismic Design Category (SDC) D

- Meet all design requirements for SDC D according to the *AASHTO Guide Specifications*, except as modified below:
  - Ensure the maximum concrete strain in confined section of the columns does not exceed $\varepsilon_{cc} = 0.005$.
  - Ensure the maximum strain of reinforcing steel does not exceed $2*\varepsilon_{sh}$, where ε_{sh} is defined on *Table 8.4.2-1* of the *AASHTO Guide Specifications*.

- Ensure the maximum strain of prestressing steel (for 270 ksi strands) does not exceed $\varepsilon_{ps,EE}$ = 0.0086

1.17.2.4 Seismic Ground Motions

C1.17.2.4

Obtain the seismic hazards, ground motions, soil classification, ground deformations and additional permanent earth loads from the project geotechnical designer.

For preliminary design, determine the spectral response parameters for the Upper Level Ground Motions using the *AASHTO - USGS Seismic Design Ground Motion Database*. These ground motions have a targeted risk of incipient bridge column collapse of 1.5 percent in 75 years using the notional fragility function and can be found at https://doi.org/10.5066/F7NK3C76.

To aid in providing consistency and efficiency, Bridge Section has developed an excel application for constructing the risk-targeted acceleration response spectrum. Latitude, Longitude, and Site Class are the only required input for *Version 2025.1* of this excel application which can be obtained at: https://www.oregon.gov/ODOT/Bridge/Pages/seismic.aspx

Determine the spectral response parameters for the Lower Level Design Earthquake using the Ground Motions of a full rupture of Cascadia Subduction Zone Earthquake (CSZE). The full rupture CSZE hazard maps are available at the ODOT Bridge Section website.

The CSZE is a deterministic event, and a deterministic Design Response Spectrum must be generated.

To allow for consistency and efficiency in design for the CSZE, an application for generating the Design Response Spectra has been developed by Portland State University. Latitude, Longitude, and $V_{s,30}$ are the only required input for running the application which can be accessed at:

https://www.oregon.gov/ODOT/Bridge/Pages/Seismic.aspx

1.17.2.5 Consideration for Other Loads and Hazards

C1.17.2.5

Use the load factor for the Live Load on Extreme Event Load Combination I, γEQ = 0 (*LRFD 3.4.1*), unless the bridge is designated by Bridge Section as non-conventional or Critical.

For bridges subject to liquefaction, perform seismic analyses for both liquefied and non-liquefied conditions. Refer to *Chapter 13 of ODOT GDM* for combination of inertial loads and liquefaction or liquefaction-induced lateral spreading.

For bridges subject to scour, perform seismic analyses for both scoured and non-scoured conditions. Use the scour depths provided in *BDM 1.10.5.8* for the scoured condition.

1.17.2.6 Modifications to AASHTO Guide Specifications for LRFD Seismic Bridge Design

The following items summarize ODOT's additional requirements and deviations from *AASHTO Guide Specifications for LRFD Seismic Bridge Design*:

- Design all bridges to satisfy the Type-1 Global Seismic Design Strategy (ductile substructure with essentially elastic superstructure), *AASHTO Guide Specifications 3.3*. However, in the case of a steel substructure, design the bridge according to the latest edition of the *AASHTO LRFD Bridge Design Specifications*.

C1.17.2.6

Type-2 Global Seismic Design Strategy (essentially elastic substructure with ductile superstructure) is not permitted by ODOT. Type-3 Global Seismic Design Strategy (elastic superstructure and substructure with a fusing mechanism between the two) can be considered if approved by the State Bridge Engineer. Include a clear description of the selected Seismic Design Strategy in the calculation book for the structure.

- The following types of Earthquake Resisting Systems (ERS) or Earthquake Resisting Elements (ERE) provided in *AASHTO Guide Specifications 3.3* are permissible ERS or ERE for ODOT bridges:
  - Type 1, 2, 3, 4 and 5 on *Figure 3.3-1a.*
  - Types 1, 2*, 3, 7, 8, 9, 10, 11**, 12 and 14 on *Figure 3.3-1b.*

    * Near ground plastic hinge is limited to less than 4 feet below ground surface.

    ** To use this Earthquake Restraining Element the following must be applied:

    1. Liquefaction induced-lateral spread and slope instability are deemed unlikely to occur under the design earthquake.

2. For the Life Safety criteria a maximum of 70 percent of the passive abutment resistance can be used in seismic analyses in accordance with *AASHTO Guide Specifications*.
3. For the Operational criteria a maximum of 30 percent of the passive abutment resistance can be used in seismic analyses.

- Obtain approval from the State Bridge Engineer before utilizing any of the following types of Earthquake Resisting Systems (ERS) or Earthquake Resisting Elements (ERE) provided in *AASHTO Guide Specifications 3.3*:
  - Type 6 on *Figure 3.3-1a.*
  - Types 4, 5, and 6 on *Figure 3.3-1b.*
  - Types 1 and 2 on *Figure 3.3-2.*
- The following types of Earthquake Resisting Systems (ERS) or Earthquake Resisting Elements (ERE) provided in *AASHTO Guide Specifications 3.3* are not permissible ERS or ERE for ODOT bridges:
  - Type 13 on *Figure 3.3-1b.*
  - Types 3, 4, 5, 6, 7, 8 and 9 on *Figure 3.3-2.*
  - Types 1, 2, 3 and 4 on *Figure 3.3-3.*
- Identify the ERS for bridges in SDC B (*AASHTO Guide Specifications 3.5*) when $0.25 \leq S_{D1} < 0.30$.

- When pushover analysis is performed, provide SDC D Level of Detailing regardless of the design SDC.

Pushover analysis can be used instead of the implicit equations to determine the Displacement Capacity for SDC B and C as prescribed on *AASHTO Guide Specifications 3.5*.

- Satisfy the balanced stiffness and balanced frame geometry requirements for all bridges in SDC C and D (*AASHTO Guide Specifications 4.1.2* and *4.1.3*).
- For bridges with two or more spans in SDC B, C, or D, use analysis Procedure Number 2 (Elastic Dynamic Analysis) to determine seismic demand (*AASHTO Guide Specifications 4.2*).

- Use Procedure 3, (Nonlinear Time History Analysis) if the geotechnical engineer is performing a site-specific hazard motion analysis and any of the following exists:
  - The maximum bridge span length is more than 300 feet, or the total bridge length is more than 1,800 feet.
  - Bridge geometry does not allow for the balanced stiffness or balanced frame geometry requirements be satisfied.
  - Special bearing and damping devices (isolation bearing, shock transmission units, etc.) and non-conventional expansion joints are expected to be installed.

Nonlinear Time History Analysis is the most expensive seismic analysis procedure; however, the extra design cost is often offset by construction cost savings and can be effectively used to manage risk.

- Consult the project geotechnical engineer to determine if there are geotechnical factors that may lead to pursuing the development of ground response and non-linear time history analyses. Geotechnical factors may include:
  - Bridge is within 6 miles of an active fault.
  - Soils at bridge site are defined as Site Class "E" or "F" soils.
  - Soil profile supporting the bridge varies significantly among bridge bents.
- Use a Damping Ratio of 5 percent (*AASHTO Guide Specifications 4.3.2*) on all new bridges for seismic loading. The application of the reduction factor, R_D, is not allowed without approval from the State Bridge Engineer.
- Use Design Method 3 (Limited-Ductility Response in Concert with Added Protective Systems) for designing the lateral seismic displacement demand (*AASHTO Guide Specifications 4.7.1*) only upon approval from the State Bridge Engineer.
- 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 *(AASHTO Guide Specifications 4.2)*, design and detail the capacity protected members to resist the lesser of:
  - 2.0 times the elastic forces resulting from the demand displacement.

  - Column/pier overstrength plastic moment and shear.
- Design Longitudinal Restrainers (*AASHTO Guide Specifications 4.13.1*) in accordance with *BDM 1.17.8*.

Participation of the approach slab, wingwalls, and backwalls in the overall dynamic response of bridge systems may be considered in seismic design of bridges using *BDM 1.10.4.2*.

- Select the Foundation Modeling Method (FMM) (*AASHTO Guide Specifications 5.3.1*) according to *BDM 1.10.4*.
- Do not allow uplift or rocking of spread footings in all SDCs.
- Perform Liquefaction Assessment for all bridge sites according to *Chapter 6* of the ODOT Geotechnical Design Manual.
- Use the provisions in *AASHTO Guide Specifications 7.2* in conjunction with the forced-based seismic design procedure utilized in the *AASHTO LRFD Bridge Design Specification* and requirements of this section of the *BDM*.
- Provide minimum shear reinforcement for bridges in SDC A, when $0.10 \leq SD1 \leq 0.15$, according to the requirements of *AASHTO Guide Specifications 8.6.5* for SDC B, in addition of satisfying the requirements of *AASHTO Guide Specifications 8.2*.
- Do not use wire rope or strands for spirals in bridge columns containing potential plastic hinge regions. Deformed welded wire

reinforcement (*AASHTO Guide Specifications 8.4.1*) require a design deviation.

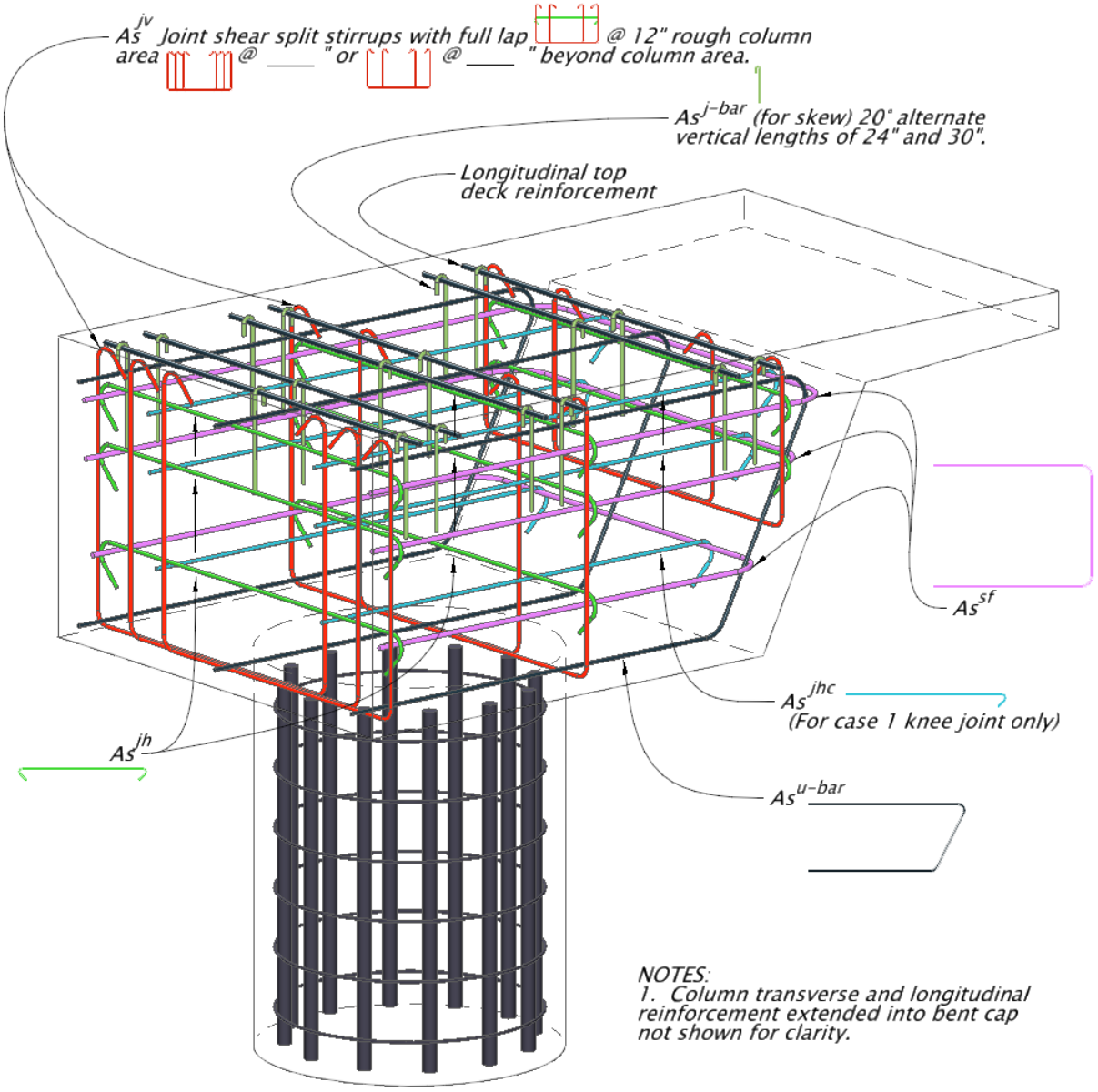
- Do not use high strength reinforcing bars with yield strength exceeding 60 ksi for spirals, hoops, or longitudinal column reinforcement where plastic hinging is expected.
- The same size vertical bars may be used inside and outside of interlocking spirals (*AASHTO Guide Specifications 8.6.7*).
- Provide minimum longitudinal reinforcement (*AASHTO Guide Specifications 8.8.2*) of 1 percent for columns in SDC B, C and D.
- Extend the vertical column bars into oversized drilled shafts according to *BDM 1.10.5.5*, in lieu of *AASHTO Guide Specifications 8.8.10*.
- Do not splice spirals in potential plastic hinge regions. When spiral splice is required outside potential plastic hinge regions, use lap-welded splice or mechanical splice and specify the location and details on the plan.

Use of hoops with butt welded splice is an option.

- When plastic moment capacity is required between column-to-crossbeam or column-to-footing connections, extend the transverse reinforcing into the crossbeam or footing. Design and detail spirals or hoops in crossbeams and footings according to *AASHTO Guide Specifications Sections 8.9 through 8.13* with requirements of *BDM 1.11.3.7*.

- Revise the third bullet of *AASHTO Guide Specifications 8.13.4.1.1* as follows:
  - Exterior column joints for box girder superstructures and other superstructure types if the cap beam extends beyond the joint far enough to develop the longitudinal cap reinforcement.

Figure 1.17.2.6-1 Three-Dimensional Representation of Knee Joint Reinforcing



1.17.2.7 Bridge Widenings and Modifications

Refer to *BDM 1.2.6.1.4* for general requirements regarding bridge widening and modification.

For the purpose of seismic design, Bridge Widening and Modifications are classified as Minor or Major Modification projects.

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

- Widening can accommodate at least one additional travel lane.
- Superstructure to substructure connection type is modified.
- Foundation fixity is modified.
- The increase in superstructure mass supported by the existing substructure exceeds the following thresholds compared to the existing superstructure mass:
  - 10 percent for bridges built before 2004.
  - 20 percent for bridges built after 2004.
- The base shear of the modified structure is increased by more than 20% relative to the original structure.

All bridge widening and modification projects not meeting the definition of a Major Modification are classified as Minor Modification.

No seismic evaluation or retrofit is required for Minor Modification or single-span bridge widening projects that match the existing substructure type and fixity condition.

C.1.17.2.7

Examples of Minor Modifications include bridge rail or safety curb modifications, deck rehabilitation, modification or installation of new signs, luminaires, traffic signals, etc.

Provide seismic detailing of all new bridge components for Minor Modification projects. Seismic detailing consists of providing the reinforcement of the Earthquake Resisting Elements within the limits recommended in the design specifications, including:

- Maximum and minimum shear reinforcement (per *AASHTO Guide Specifications 8.6.4 & 8.6.5*).
- Maximum and minimum longitudinal reinforcement (per *AASHTO Guide Specifications 8.8.1 & 8.8.2*).
- Maximum spacing of longitudinal reinforcement (6 or 8 inch, depending on the shape of the column).
- Minimum size of lateral reinforcement (per *AASHTO Guide Specifications 8.8.9*).
- Maximum spacing of lateral reinforcement in plastic region (4 inch), where the plastic hinge region is determined according to the *AASHTO Guide Specifications 4.11.7*.
- Joint proportioning (per *AASHTO Guide Specifications 8.13.2*) and minimum joint shear reinforcing (per *AASHTO Guide Specifications 8.13.3*).

1.17.2.7.1 Seismic Design Requirements for Major Modification Projects

A geotechnical report, containing similar information as for new bridge design, is required for Major Modification Projects.

ORDINARY BRIDGES

Determine the seismic demand based on analysis of the combined (new plus existing) structure. Refer to *Table 1.17.2.3-1* for ground motions for Ordinary Bridges.

Perform seismic capacity analysis based on a structural model that reflects the displacement compatibility and interaction between the widening and the existing structure.

Design strengthening of existing crossbeams, columns and footings with Capacity over Demand (C/D) ratios of less than 1.0. If strengthening of the existing footing results uneconomical, verify that "Do No Harm" criteria have been met. Provide a statement in the general notes indicating that C/D ratios for existing footings are below 1.0 and do not get worse after widening.

Design and detail the widening according to design requirements for new bridges.

C1.17.2.7.1

C ORDINARY BRIDGES

To avoid imposing additional loads on the existing structure, consider the following details for the widening structure:

- Use the same structure frame on the widened portion as on the existing bridge.
- Match the stiffness of the new and existing superstructures and substructures.
- Match the existing bearing types in terms of fixity.

Construction of longitudinal joint between the new and existing structure is not a preferred alternative for addressing stiffness dissimilarities. Unless the longitudinal joint is located under a permanent traffic barrier, it could become hazardous to traveling public.

The "Do No Harm" criteria refer to comparison of C/D ratios of the existing foundation elements (spread footings, drilled shafts, piles, and pile caps) before and after widening and ensuring that C/D ratios are not decreased by the widening.

RECOVERY BRIDGES

Determine the seismic demand based on analysis of the combined (new plus existing) structure. Refer to *Table 1.17.2.3-1* for ground motions for Recovery Bridges.

Perform seismic capacity analysis based on a structural model that reflects the displacement compatibility and interaction between the widening and the existing structure for both Lower-Level and Upper-Level ground motions.

If required, design retrofit of the existing bridge according to *BDM 1.17.2.8 – Phase 2* requirements.

Design and detail the widening according to design requirements for new bridges.

C RECOVERY BRIDGES

1.17.2.8 Seismic Retrofit

The *ODOT's Seismic Implementation: Policies and Design Guidelines* provides guidance for planning and executing ODOT's Seismic Program. It also provides recommendations regarding the design criteria and other considerations that need be evaluated during the early design phase of the project. When the seismic retrofit design is included in a project, use a phased approach for establishing a practical and economical retrofit strategy. The publication "Seismic Retrofitting Manual for Highway Structures" (*FHWA-HRT-06-032*) is recommended as a reference source to supplement the *Bridge Design Manual* and *Bridge CAD Manual*.

The following steps are provided to help designers initiating the design process:

Most Oregon bridges fall under importance category of "**standard**", based on the Bridge Importance Category definitions provided on *FHWA-HRT-06-032*. Contact Bridge HQ when this category becomes questionable for a given structure.

Contact Bridge HQ for information on the Anticipated Service Life (ASL) of the bridge.

Revise the top-half of the *Table 1-2* of *FHWA-HRT-06-032* as shown in *Table 1.17.2.8-1*.

Table 1.17.2.8-1 Minimum Performance Levels for Retrofitted Bridges for each Earthquake Ground Motion Based on Bridge Operation Category and Anticipated Service Life (ASL)

Bridge Operation and Service Life Category	**Lower-Level Ground Motion** ***(Cascadia Subduction Zone Earthquake – Full Rupture)***	**Upper-Level Ground Motion** ***(7 percent probability of exceedance in 75 years; return period ~ 1,000 years.)***
Standard – ASL 1	PL0[4]: No Minimum Performance	PL0[4]: No Minimum Performance
Standard – ASL 2	PL2: Operational	PL1: Life safety
Standard – ASL 3	PL2: Operational	PL1: Life safety
Essential – ASL 1	PL0[4]: No Minimum Performance	PL0[4]: No Minimum Performance
Essential – ASL 2	PL2: Operational	PL1: Life safety
Essential – ASL 3	PL3: Fully Operational	PL2: Operational

Table Notes:

1 – Anticipated Service Life categories are:

- ASL 1: 0 – 15 years
- ASL 2: 16 – 50 years
- ASL 3: > 50 years

2 – Refer to *Table 1.17.2.3-2* for Performance Levels PL1 through PL3. PL0 means that no minimum level of performance is recommended.

3 – Refer to *1.17.2.4* for Lower-Level and Upper-Level Seismic Ground Motions.

4 – Bridges assigned a Performance Level of PL0 have 15 years, or less, anticipated service life (ASL) and are candidates for replacement or rehabilitation.

Replace *Equation 7-49* of *FHWA-HRT-06-032* with the following equation:

$$\phi_p = \left[2 - 5\left(\frac{V_m - V_i}{V_i - V_f}\right)\right]\phi_y$$

Replace *Equation 7-51* of *FHWA-HRT-06-032* with the following equation:

$$\phi_p = \left[2 - 4\left(\frac{V_{jh} - V_{ji}}{V_{ji} - V_{jf}}\right)\right]\phi_y$$

The equations above will produce results that are consistent with the narrative in *Article 7.8.2.7(b)* and *7.8.2.8(b)* of *FHWA-HRT-06-032*, given the following definition of curvature ductility, μ_ϕ:

$$\mu_\phi = \frac{\phi_y + \phi_p}{\phi_y}$$

For assessing the seismic performance of existing bridges use concrete strain limits provided in *Table 1.17.2.8-2* for existing bridge columns with poor confinement detailing.

Table 1.17.2.8-2 Concrete Strain Limits for Existing Bridge Columns with Inadequate Hoops and Hoop Spacing[1]

Design Criteria	Inadequate Lap Splice[2]	Adequate Lap Splice
Operational	0.002	0.002
Life Safety	0.003	0.004

Table notes:
1. As adequate hoops are qualified those that meet the definition of "seismic hooks" in *Article 8.8.9* of *AASHTO Guide Specifications for Seismic Bridge Design* and are spaced no more than 6 inches apart.

2. As adequate lap splices are qualified those that meet the requirements of *Article 5.10.8.4.3a* of *AASHTO LRFD Bridge Design Specifications* for Class B splice.

PHASE 1

The Phase 1 Seismic Retrofit is considered to achieve "Life Safety" performance of Oregon bridges under seismic induced loading. Work during this phase is intended to prevent superstructure pull-off and bearing failure. Phase 1 Retrofit is not required on bridge painting projects or any projects where the scope of work does not extend below the deck.

Incorporate Phase 1 Seismic Retrofit on bridge rehabilitation projects when bridges are located in Seismic Zone 3 or 4. Bridges located in Seismic Zone 2 may be considered for Seismic Retrofit if situated between bridges (on the same route) that have received or are receiving Seismic Retrofit, or between new bridges built to current seismic design standards.

As a minimum, for a Phase 1 Retrofit ensure that the girders will not pull off longitudinally or slide off laterally from the bents. This will normally involve addition of cable restraints, shear blocks, or beam seat lengthening and widening.

Identify a seismic design concept which will accomplish the intent to preclude span pull off or collapse of the superstructure. Depending on the concept selected, some strengthening of the superstructure may be required to ensure loads generated at the restraints or shear blocks can be transmitted without exceeding design stresses in the superstructure. For steel truss bridges, ensure all truss elements and connections provide sufficient resistance to failure or plastic

deformation under seismic induced loading. Short pedestals or secondary columns above the main bent cap level must also be investigated for seismic induced loading and strengthened or braced, if necessary.

Upgrade existing bearings to elastomeric bearings, if needed to assure the designer's concept will work. Upgrading bearings to elastomeric should, also, be considered to improve seismic performance when existing bearings are known to have poor seismic performance, such as steel rocker bearings. Analysis for Phase 1 Retrofit will normally consist of a single degree of freedom model, which may be sufficient for normal bridges. However, a higher level analysis may be required, if needed to fully develop the designer's concept, or for bridges with irregular column lengths of multi-column bents or if the bents have significantly different stiffness. Use full column sections (uncracked) for this level of analysis to develop connection design loads. This is the minimum level of work that must be included. A cracked section analysis may be used to investigate the maximum anticipated movements.

PHASE 2

Work during this phase involves substructure (columns, footings and foundations) ductility enhancement and strengthening. Any additional or deferred Phase 1 Retrofit work would also be included. The end product is a retrofitted bridge with as much seismic loading resistance as a new bridge would have for the site. The Phase 2 Seismic Retrofit is considered to achieve the "Operational" performance of Oregon bridges under the seismic loading induced by the full rupture of Cascadia Subduction Zone Earthquake.

Evaluate the structure to investigate the level of effort and scope of work needed to do Phase 2 Retrofit. Phase 2 involves a complete seismic analysis of the widened or rehabilitated bridge for full seismic loading, including consideration of strengthening or restraints to the superstructure, substructure and foundations. The work may involve column and footing strengthening or enlargement, or the use of isolation bearings, and soil improvement, if there is potential for liquefaction. The decision about whether to actually do Phase 2 Retrofit in the project will be made after developing a retrofit concept, rough cost estimate and evaluation of the relative importance of the bridge to the transportation network, in comparison to the estimated cost and available funding for the project. The remaining service life, existing condition, and retrofit cost versus replacement cost are also important factors and must be evaluated. Consult with the Bridge Section before proceeding with any Phase 2 seismic retrofit.

A seismic retrofit analysis typically requires the use of a "Site Factor" to develop the response spectrum used in the analysis. Site factors are based on the soil conditions at the site, (categorized as Site Classes A - F) as described in the *FHWA Seismic Retrofitting Manual for Highway Structures, Table 1-3*. For most normal bridges requiring Phase 1 retrofit work the site class can be determined using either existing soils data or a general knowledge of the site geology and soil conditions. If limited knowledge is available the default designation of Site Class D is acceptable. However, for Phase 2 level retrofit analysis more detailed soils information is required to better determine the design response spectrum and also to adequately characterize and model the foundations in the analysis. Additional exploration work may be required to obtain this information. This additional

work is justified due to the increased cost of Phase 2 retrofit work and the need for a more refined analysis.

RAIL UPGRADE, DECK OVERLAYS, PRESERVATIONS, REPAIR, STRENGTHENING, AND OTHERS:

These projects should include seismic retrofit as described previously for "Seismic Retrofit".

1.17.2.9 Diversion Bridges and Pedestrian Bridges

DIVERSION BRIDGES

Design all diversion bridges meeting at least one of the following criteria according to *AASHTO Guide Specifications 3.6*:

- Bridge is over roads carrying vehicular traffic.
- Bridge is expected to be in service for more than one year and ADT > 10,000.
- Bridge is expected to be in service for more than two years.

For all other diversion bridges, provide the minimum support length requirement according to *AASHTO Guide Specifications 4.12* and minimum connection equal to A_s times tributary permanent loads.

When designing according to *AASHTO Guide Specifications 3.6*, use the design response spectra for the permanent structure reduced by a factor of 2.5.

Liquefaction assessment for diversion bridges is not required.

PEDESTRIAN BRIDGES

Design new pedestrian bridges that cross over roads carrying vehicular traffic according to requirements for Ordinary bridges, as specified in *BDM 1.17.2.3*.

1.17.2.10 Structure Modeling

STRUCTURE MODELING, GENERAL

- Use a "first cut" analysis with fixed supports. These results will be easier to interpret than a spring supported model and will give a baseline for comparison with additional analyses. With these results, make a rough substructure design. Now a new analysis can be performed with footing springs and the substructure design checked and refined. Additional cycles of redesign, analysis, and force comparison to previous analyses could be used in some cases but generally would not be required or warranted.
- A reasonable target for a seismic design check is 20 percent. Designer and checker should resolve differences greater than 20 percent, but it is impractical to try to refine the design beyond that.

FOOTING SPRINGS

See *BDM 1.10.4*.

PROGRAMS

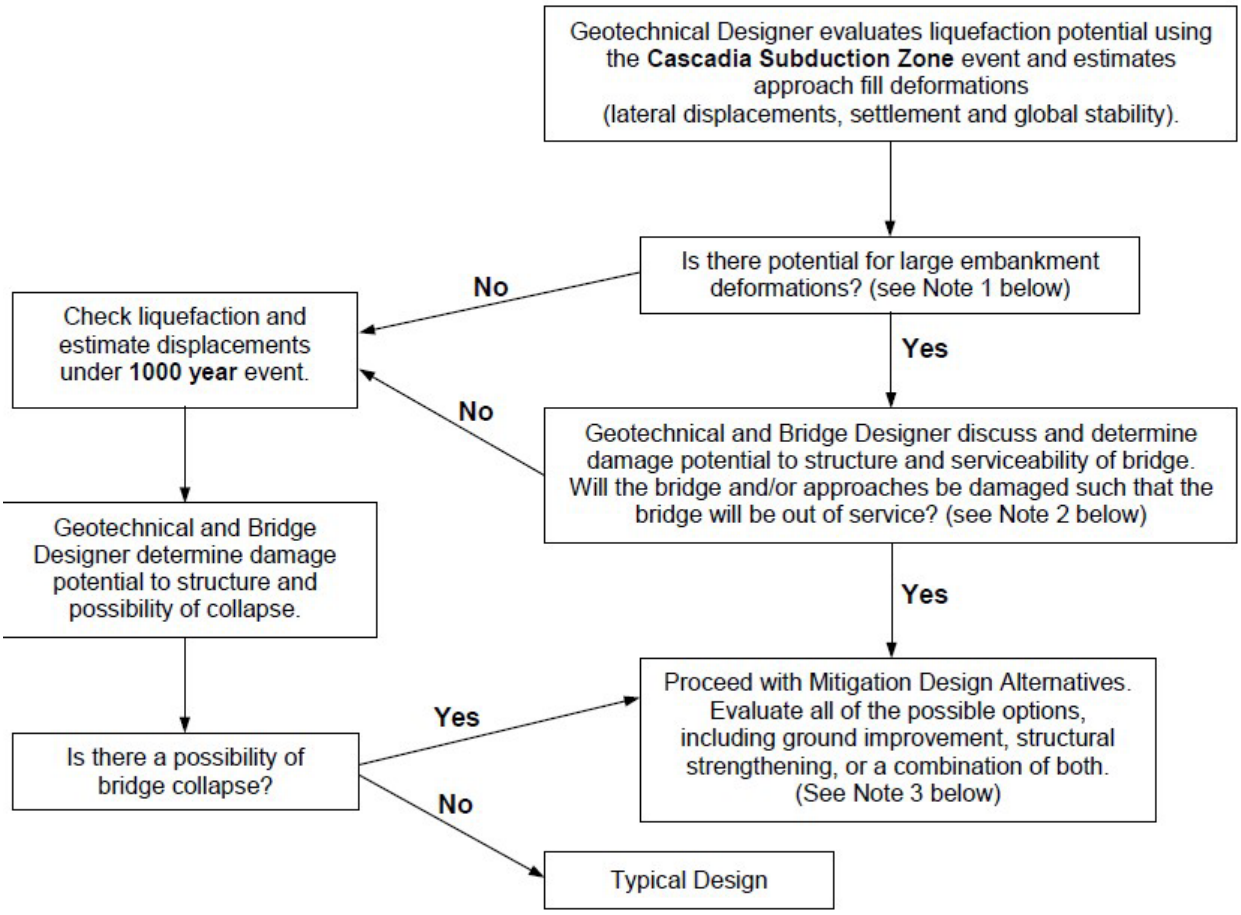
- The Uniform Load and single mode dynamic analysis methods are acceptable for many structures (see the code limitations) but multi-mode dynamic analysis by computer may be easier. The result of any analysis method must be judged for correctness. Is the result reasonable? Reviewing the calculated periods, modal participation factors and mode shapes can greatly aid this judgment. A high level of engineering judgment will be required at all times.
- MIDAS is ODOT's primary in-house static and dynamic analysis programs, and is available for bridge designers working at Bridge HQ or Region Tech Centers. Many design firms have adopted the use of SAP2000 or STAAD for seismic design of bridges. Other programs are also acceptable, provided the programs satisfy the analysis requirements and have been previously verified.

1.17.3 Reserved

1.17.4 Liquefaction Evaluation and Mitigation Procedures

The liquefaction potential of foundation soils will be determined by the geotechnical designer. If foundation soils are predicted to liquefy, the effects of liquefaction on foundation design and performance will be provided as described in *BDM 1.10.5*. The need for liquefaction mitigation will be in accordance with the following ODOT Liquefaction Mitigation Policy.

Figure 1.17.4-1 ODOT Liquefaction Mitigation Policy



Note 1: For meeting the performance requirements of the Cascadia Subduction Zone event (Operational), lateral deformation of approach fills of up to 12 inches are generally considered acceptable under most circumstances pending an evaluation of this amount of lateral deformation on abutment piling and abutment/cap. Larger structural lateral deformations and settlements may be acceptable under the 1000 year event as long as the "Life Safety" criteria are met.

Note 2: The bridge should be open to emergency vehicles after the Cascadia Subduction Zone event, following a thorough inspection. If the estimated embankment deformations (vertical or horizontal or both) are sufficient enough to cause concerns regarding the serviceability of the bridge, mitigation is recommended.

Note 3: Geotechnical and bridge designer to submit all liquefaction mitigation designs and cost estimates to Bridge Standards for review and approval.

A continuous discussion between the Geotechnical and bridge designers is to be expected for determining the most cost-effective mitigation alternative. The iteration for both geotechnical and structural analyses start with the geotechnical designer providing the bridge designer the following information:

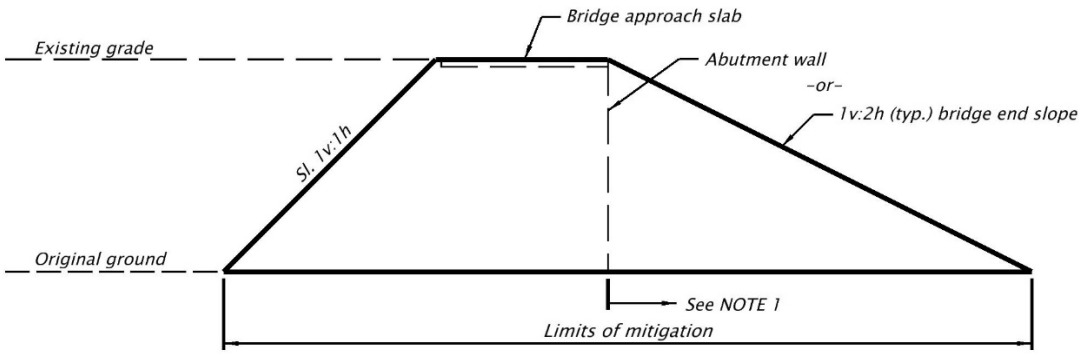
- lateral soil displacement.
- vertical soil displacements.
- vertical and horizontal loading on piling and end bent due to the soil displacement.

Bridge designer determines the lateral and vertical deformation demand and capacity of the bridge foundation and follows up with the geotechnical designer on the need for soil mitigation. Evaluation of the mitigation alternatives should consider both structural improvements and soil mitigation by maintaining a cost balance and equal risk between these two forms of mitigation. Final cost estimates should reflect this evaluation.

Refer to *ODOT research report SPR Project 361: "Assessment and Mitigation of Liquefaction Hazards to Bridge Approach Embankments in Oregon", Nov. 2002* and *"Ground Modification Methods Reference Manual, Volume I & Volume II", (Pub. No. FHWA-NHI-16-027 & NHI-16-028)* for mitigation alternatives and design procedures.

As a general guideline, the foundation mitigation should extend from the toe of the bridge end slope (or face of abutment wall) to a point that is located at the base of a 1:1 slope which starts at the end of the bridge approach slab, as shown in *Figure 1.17.4-2*.

Figure 1.17.4-2 Foundation Mitigation



NOTE 1: Extend ground improvement beyond the abutment face as needed for design.

1.17.5 Reserved

1.17.6 Instrumentation

Consider placement of accelerometers on the ground and on structure portions for large or unusual structures. Consult with the State Bridge Engineer to determine if this is appropriate and fits with the ODOT Strong Motion Program.

C 1.17.6

1.17.7 Seismic Isolation Bearings

Bridges with isolation bearings fall under Global Seismic Design Strategy Type 3 (Elastic Superstructure and Substructure with a Fusing Mechanism between the two). Submit a Design Deviation request to use seismic isolation devices. As a minimum, provide the following information in design deviation request:

- Challenges with designing a Type 1 bridge (Ductile Substructure with Essentially Elastic Superstructure)
- Type of isolation bearings to be considered
- Specific location on the bridge (e.g., bent 2 and 3, under each girder line)
- Cost/benefit analysis.

For new bridges, it is very likely that isolation bearings will need to be replaced during the service life of the structure. Provide in bridge plans structural details that would allow for bearing replacement while the bridge remains in service.

C1.17.7

The primary benefit of using isolation bearings is the reduction of earthquake-induced forces on the bridge due to the increase of fundamental period of vibration of the structure. However, the increase of fundamental period of vibration leads to a much larger displacement demand, which may require special expansions joints (e.g., modular joints) or innovative detailing of the Earthquake Resisting Systems or Elements (e.g., construction of sacrificial elements).

Furthermore, seismic design of bridges with isolation bearing can be more complex and may require more design time. Inspection and maintenance may require specialized personnel and can be costly.

Combination of isolation bearings and other bearing types within the same bent is not allowed.

Design Seismic Isolation Bearings according to *AASHTO Guide Specifications for Seismic Isolation Design, 4th Edition* (*AASHTO Seismic Isolation*), including *2023 Interim Revisions* in conjunction with *AASHTO Guide Specifications for LRFD Seismic Bridge Design, 3rd Edition*.

AASHTO Seismic Isolation are applicable for both force-based and displacement-based seismic design philosophies. Therefore, some definitions or notations may require to be translated into corresponding terms used in the *AASHTO Guide Specifications* (e.g., Seismic Zones to Seismic Design Categories), whereas some requirements may not be applicable (e.g., Response Modification Factor (R), *AASHTO Seismic Isolation 6*).

1.17.8 Seismic Restrainer Design (New Designs and Retrofits)

1.17.8.1 Seismic Restrainer Design, General

The intent is to prevent superstructure pull-off and bearing failure. Work restrainers only in the elastic range. Design the restrainer connection for 125 percent of the restrainer design force.

Note that *LRFD 3.10.9.5* requires "sufficient slack" so that the restrainer does not start to act until the design displacement is exceeded.

Restrainers may be omitted where the available seat width meets or exceeds "N" of the Design Specifications and 4 times the calculated design earthquake elastic deflection. Seat widths meeting these criteria are presumed to accommodate the large elasto-plastic movements of a real structure under seismic loading.

Design restrainers for a minimum force equal to the peak site bedrock acceleration coefficient "A" times the weight of the lighter portion being connected.

In all instances it is necessary to design or check the transfer mechanism for force transfer from superstructure to substructure (bearings, diaphragms).

1.17.8.2 Information for Restrainer Design

CONCRETE

Concrete bearing strength based on $0.85f''_c$ ($\Phi = 1.0$).

Maximum increase for supporting surface wider than loaded area = 2.0.

Multiply by 0.75 when loaded area is subject to high edge stresses.

For concrete shear lugs, use *LRFD equation 5.8.4.1-1* for shear friction as outlined in *LRFD 5.8.4*.

STRUCTURAL STEEL

Design structural steel members using the *AASHTO LRFD Bridge Design Specifications*.

FASTENERS

Table 1.17.8.2.3-1 Steel to Steel Restrainer Design Parameters

Diameter (inch)	Nominal Area (in^2)	A307 – Tension (0.76 x 60 ksi)	A307 – Shear (0.38 x 60 ksi)	A325 – Tension (0.76 x 120 ksi)	A325 – Shear (0.38 x 120 ksi)
0.75	0.4418	20.1 k	10.1 k	40.3 k	20.1 k
0.875	0.6013	27.4 k	13.7 k	54.8 k	27.4 k
1.0	0.7854	35.8 k	17.9 k	71.6 k	35.8 k

Table Notes:

Tension loads are based on *LRFD equation 6.13.2.10.2-1*.

Shear loads are based on *LRFD equation 6.13.2.7-2* assuming one shear plane per bolt and with threads included in the shear plane.

Shear loads may be increased 25 percent if the threads are excluded from the shear plane.

STEEL RODS

Table 1.17.8.2.4-1 Steel Rods Restrainer Design Parameters

Diameter (Inch)	Stress Area (in^2)	A307 – Fu = 58 ksi Tension (kips) Ft=Fy=36 ksi	A449 – Fu varies Tension (kips) Ft = Fy	A449 – Fu varies Fy (ksi)
0.750	0.334	12.0	30.7	
0.875	0.462	16.6	42.5	92
1.00	0.606	21.8	55.8	
1.125	0.763	27.5	61.8	
1.250	0.969	34.9	78.5	81
1.375	1.155	41.6	93.9	
1.500	1.405	50.6	114.0	
1.750	1.900	68.4	110.0	58
2.250	2.500	90.0	145.0	

Table notes:

Tensioning of A 449 steel rods must be specified, if required by the design.
Tensioning requirements are not part of the specification as they are with A 325.
Use nominal area for elongation calculations.

WIRE ROPE

See *BDM 1.17.8.6* for a complete discussion of Structural Wire Rope, Wire Rope Connections & Turnbuckles.

F_t = (0.95)(176.1 ksi)(area) = 0.95(minimum breaking strength).

Note: Yield strength is approximately equal to minimum breaking strength.

Table 1.17.8.2.5-1 Wire Rope Restrainer Design Parameters

Diameter (in)	Area (in^2)	Minimum Breaking Strength (kips)	Design Load (kips)
1/2	0.119	23.9	22.7
3/4	0.268	52.9	50.2
7/8	0.361	71.6	68.0
1	0.471	93.0	88.3
1 3/8	0.906	173.0	164.0

The area values in *Table 1.17.8.2.5-1* are based on ASTM A603. The minimum breaking strength above is based on ASTM A1023. The design load in *Table 1.17.8.2.5-1* is based on 0.95 x the minimum breaking strength. For sizes other than 7/8 inch diameter, ASTM A1023 is likely to be used.

E for wire rope = 10,000 ksi

f_y for wire rope = 176.1 ksi

ASTM A603 lists the E for structural wire rope as 20,000 ksi for "prestretched" wire rope. Wire rope used for seismic applications will not be prestretched, however, so an E of 10,000 ksi should be used.

RESIN BONDED ANCHORS

See *BDM 1.5.11.2*, "Post-Installed Anchors".

CONCRETE INSERTS

See *BDM 1.5.11.3.2 Coil Inserts*.

1.17.8.3 Longitudinal Restrainer Design

(1) IN-SPAN HINGES

Use the following general procedure (a modified CALTRANS method):

- Estimate restrainers to use (with elongation) and gapping desired/allowed.
- Determine joint openings (including approximate temperature movement (fall) and creep and shrinkage if appropriate).
- Determine frame stiffness and capacity.
- Determine adjacent frame stiffness and capacity.
- Plot force/deflection relationship considering component stiffnesses, joint openings (including temperature, creep, and shrinkage openings), and restrainer gapping.
- Assume a final force and deflection under single-mode response to get equivalent stiffness.
- Calculate period and resulting response coefficient.
- Calculate dynamic force and locate on the force/deflection curve.
- Review that the force capacity of the system is not exceeded, the assumed/acceptable deflection is not exceeded, and the equivalent stiffness and period are approximately as before.
- If checks are not okay modify system and recycle through as needed.

BENTS WITH SUPERSTRUCTURE CONTINUOUS OVER THE BENT

Connect superstructure to substructure with capacity to form plastic hinging in the column(s).

BENTS WITH ONLY THE DECK CONTINUOUS OVER THE BENT

Connect each span to substructure to form plastic hinging in the column(s).

BENTS WITH NO SUPERSTRUCTURE CONTINUITY OVER THE BENT

- With frames each side of bent:

  Connect each span to substructure to form plastic hinging in the column(s). Also connect adjacent superstructure portions by the same techniques as "in-span hinges." The adjacent super-structure portions may be connected by span to substructure connections of adequate capacity to function for both portions.

- With simple spans each side of bent:

  Connect each span to the substructure to form plastic hinging in the column(s).

Note: The plastic hinging capacity should be determined from column interaction curves with axial and moment Φ values of 1.0. Enter the curve with the unfactored dead load axial force (plus any redundancy induced axial force due to lateral seismic loading), determine the accompanying moment capacity and multiply this value by 1.3. This is the plastic moment capacity.

1.17.8.4 Transverse Restrainer Design

IN-SPAN HINGES

Design for force transfer of 2.5(A)(supported dead load).

BENTS WITH SUPERSTRUCTURE CONTINUOUS OVER THE BENT

Connect supported spans with force to form a failure mechanism (plastic hinging at the top of frame (column or crossbeam) and plastic hinging at bottom of column.

BENTS WITH ONLY THE DECK CONTINUOUS OVER THE BENT

Connect supported spans with force to form a failure mechanism (plastic hinging at the top of frame (column or crossbeam) and plastic hinging at bottom of column.

Prorate design force to ahead and back side of bent by dead load ratio.

BENTS WITH NO SUPERSTRUCTURE CONTINUITY OVER THE BENT

Connect supported spans with a force equal to 2.5(A)(supported dead load).

Note: The plastic hinging capacity should be determined from column interaction curves with axial and moment Φ values of 1.0. Enter the curve with the unfactored dead load axial force (plus any redundancy induced axial force due to lateral seismic loading), determine the accompanying moment capacity and multiply this value by 1.3. This is the plastic moment capacity.

1.17.8.5 Hold-downs

Hold-downs or bearing replacement may be needed at vulnerable bearings such as fixed or rocker type steel bearings.

1.17.8.6 Structural Wire Rope (Cables) and Turnbuckles

Structural wire rope (cable) may be used in seismic retrofit and safety cable applications. For these applications, structural wire rope must have zinc coating for corrosion protection. *ASTM A603* structural wire rope with a Class C coating is the preferred wire rope specification. This wire rope has large wires and significant zinc coating. However, *A603* wire rope is only available by special order at a minimum of 10,000 feet.

Use A603 wire rope for all coastal area seismic retrofit applications. When there is not sufficient quantity of wire rope available in the stockpile, a new order of 10,000 feet should be purchased using project funds. Such a purchase will require preapproval from FHWA since the excess wire rope will be stockpiled for use on future projects.

For non-coastal applications, A603 wire rope is still preferred. However, *ASTM A1023* wire rope can be used where less corrosion protection is considered acceptable. A1023 wire rope uses smaller wires and has approximately one-third the zinc coating compared to A603. However,

A1023 wire rope is readily available on the market and so does not need to be stockpiled. Optional sizes of A1023 wire rope are also readily available. Those sizes are listed in *BDM 1.17.8.6*.

A603 and A1023 are the only wire rope specifications recommended for seismic retrofit applications. Other types of wire rope investigated are *ASTM A586* and *ASTM A741*. A586 wire rope is used for high-strength structural tension members, but is not readily available on the market. A741 wire rope is used for safety barrier applications (such as I-5 median between Portland and Salem). A741 has less strength compared to A603 and A1023, is difficult to make swaged connections, and is also not readily available.

7/8 inch diameter wire rope is recommended for most seismic retrofit applications. 1/2 inch diameter wire rope is recommended for safety cable applications and seismic retrofit applications where the wire rope must be wrapped around tight corners. Bending radius for A603 and A1023 wire rope are shown in *Table 1.21.1-1 and Table 1.21.1-2*.

Table 1.17.8.6-1 A603 Wire Rope Bending Radius

Wire Rope Diameter (inch)	Suggested (inch)	Minimum (inch)
1/2	18	11
7/8	32	18

Table 1.17.8.6-2 ASTM A1023 Wire Rope Bending Radius

Wire Rope Diameter (inch)	Suggested (inch)	Minimum (inch)
1/2	13	9
3/4	19	13
7/8	23	15
1	26	17
1-3/8	35	24

The bending radius values above are based on a *1997 Bethlehem Wire Rope product catalog* from Williamsport Wirerope Works, Inc.

1.17.8.6.1 General Notes for Structural Wire Rope, Turnbuckles and Connections

Use the following general notes on the plans for structural wire rope in seismic retrofit applications using the 7/8 inch diameter wire rope from the ODOT stockpile:

- Zinc-coated 7/8 inch diameter structural wire rope for seismic restraint devices will be provided by the Agency.

Use the following general notes on the plans for structural wire rope in seismic retrofit and safety cable applications using ASTM A1023 wire rope:

- Provide zinc-coated X″ (1/2″, 3/4″, 7/8″, 1″ or 1 3/8″) structural wire rope for seismic restraint devices (and safety cables) according to ASTM A1023.

Use the following general notes on the plans for turnbuckles and wire rope connections in seismic retrofit and safety cable applications:

- Provide hot-dip galvanized turnbuckles according to ASTM F1145.
- Provide hot-dip galvanized socket connections. Ensure socket connections can develop the minimum breaking strength of the connecting wire rope.

1.17.8.6.2 Special Provisions for Wire Rope

Under the heading **Structural Wire Rope for Seismic Restraints & Safety Cables** use the following:

[When using 7/8 inch wire rope from the ODOT stockpile for seismic retrofit:]

- Zinc-coated 7/8″ diameter structural wire rope for seismic restraint devices will be provided by the Agency. Agency provided wire rope was manufactured according to ASTM A603 with Class C coating. Wire rope construction is 6 x 7 with a Wire Strand Core (WSC). Agency provided wire rope has been previously certified to meet a minimum breaking strength of 71,600 pounds. Wire rope is stored on spools with up to 2500 feet on each spool.
- Agency provided wire rope is stored at the following location:

  c/o District 2B Manager

  Oregon Department of Transportation

  9200 SE Lawnfield Rd

  Clackamas, OR 97015

  Phone: 971-673-6200

- Notify Bridge Engineering Headquarters of the quantity of wire rope removed within 24 hours. Follow up this notification with a written memo documenting the time of removal, quantity removed (to the nearest foot), and the project for which it will be used. Send the memo to:

  State Bridge Operations Engineer

  Bridge Engineering Headquarters

  4040 Fairview Industrial Drive SE, MS #4

  Salem, OR 97302-1142

  Phone: 503-986-3323

  FAX: 503-986-3407

- The quantity of wire rope included for use in this project, including both testing and installation, is (____) linear feet. This quantity of wire rope will be provided at no cost to the contractor. Additional wire rope required by the contractor due to fabrication errors or waste must be purchased from the Department at the Department's cost as established by the Engineer.

[When using ASTM A1023 wire rope for seismic retrofit:]

- Provide zinc-coated X" (1/2", 3/4", 7/8", 1" or 1 3/8") diameter wire rope for seismic restraint devices according to ASTM A1023. Provide 6 x 19 wire rope construction with a steel core. Manufacture wire rope from extra improved plow steel. Ensure a minimum breaking strength of XX,XXX pounds (insert appropriate strength from design properties in *BDM 1.17.8.6*).

[When using 1/2 inch wire rope for safety cable:]

- Provide zinc-coated 1/2" diameter structural wire rope for safety cable according to ASTM A1023. Provide 6 x 19 wire rope construction with a steel core. Manufacture wire rope from extra improved plow steel. Ensure a minimum breaking strength of 23,900 pounds.

1.17.8.6.3 Special Provisions for Turnbuckles and Socket Connections

Use the following special provisions for turnbuckles and socket connections in seismic retrofit and safety cable applications:

Provide Type 1 hot-dip galvanized turnbuckles according to *ASTM F1145*.

- Ensure turnbuckles develop the minimum breaking strength of the connecting wire rope.
- Provide turnbuckles with a 24 inch take-up unless shown otherwise.
- Test turnbuckles according to the requirements outlined in ASTM A1023.
- For seismic restraint devices, provide either a jam nut or lock wire at each end of each turnbuckle. For safety cables, provide lock wire at each end of each turnbuckle. Provide 14 gage or heavier lock wire that is either hot-dip galvanized or plastic coated.

Testing for Socket Connections – Select an independent laboratory to test three sets of wire rope assemblies. Provide approximately 3 foot segments of wire rope with galvanized stud attachments at each end. Provide stud attachments of similar size and material as to be used on the project. Test each wire rope assembly to failure in tension. Ensure the tested wire rope assembly develops the minimum breaking strength of the wire rope and ensure that failure does not occur in the connecting parts. Ensure all three wire rope segments meet the minimum breaking strength requirement. However, when the wire rope breaks at a load less than the minimum breaking strength of the wire rope **and** at a location at least 6 inches from a connection, that test will be disregarded. When any wire rope assembly fails to meet these requirements, except as noted above, revise the connection details and prepare and test three new wire rope assemblies.

1.17.8.6.4 Design Properties

Modulus of elasticity for wire rope (non-prestretched) = 10,000 ksi.

Table 1.17.8.6.4-1 Approximate Gross Metallic Area and Minimum Breaking Strength for Wire Rope

Wire Rope Diameter (inch)	Area (in^2)	Strength (lb)	Weight (lb/ft)
1/2	0.119	23,900	0.46
3/4	0.268	52,900	1.04
7/8	0.361	71,600	1.41
1	0.471	93,000	1.85
1-3/8	0.906	173,000	3.49

Area values above are approximate and are based on *ASTM A603*. Minimum breaking strength and weight values above are based on *ASTM A1023*. Note that A1023 does not provide area values. Weight values for A603 are slightly smaller.

The sizes of *ASTM A1023* zinc-coated wire rope shown above are readily available from northwest suppliers.

The Agency is no longer stockpiling wire rope.

1.17.9 Tsunami Design

Design bridges within the tsunami inundation zone for tsunami loads according to the *AASHTO Guide Specifications for Bridges Subject to Tsunami Effects*. The Statewide Geohazards Viewer can be used for the initial evaluation of the bridge being located within the tsunami inundation zone.

Wave parameters for estimating tsunami loads may be obtained from the Tsunami Inundation Portal by entering the latitude and longitude of the bridge site.

The *AASHTO Guide Specifications for Bridges Subject to Tsunami Effects* are intended for the design and construction of new conventional bridges to resist the effects of tsunami waves. For tsunami design of non-conventional bridges, develop a Project Specific Tsunami Design Criteria (PSDC). Obtain approval by the State Bridge Engineer for PSDC no later than Design Verification Package submittals.

Tsunami loads need not be combined with seismic loads.

Obtain approval by the State Bridge Engineer for incorporating provisions outlined in *Section 7* of the *AASHTO Guide Specifications for Bridges Subject to Tsunami Effects* for reduction of tsunami loading, such as venting and sacrificial elements.