I N T E R O F F I C E M E M O

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DATE: September 28, 2016

TO: Alex Lim, PE
Bridge Design Standards Engineer

FROM: Albert Nako, PE
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SUBJECT: Proposed Revision to Bridge Design & Drafting Manual

RE: BDDM Section 1.17 – Seismic Design

Problem Statement:

Proposal:

Modify Section 1.17 as follows:

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.17 SEISMIC DESIGN

1.17.1 Design Philosophy

1.17.2 Applications of AASHTO LRFD Bridge Design Specifications

1.17.3 Applications of AASHTO Guide Specs for LRFD Seismic Bridge Design

1.17.4 Liquefaction Evaluation and Mitigation Procedures

1.17.5 Costs

1.17.6 Instrumentation

1.17.7 Dynamic Isolators

1.17.8 Seismic Restrainer Design (New Designs And Retrofits)

1.17.1 Design Philosophy

T~~AASHTO has been very active over the past few years on updating the Seismic Design procedures and practices for highway bridges. As a result, t~~he 2008 Interim Revisions to the 4th edition of the *AASHTO LRFD Bridge Design Specifications* were developed in late 2007. Though these revisions still support a "force-based" design philosophy, they represent a significant update to many areas of the seismic design provisions in AASHTO LRFD Bridge Design Specifications. In 2008, AASHTO also adopted the *Guide Specifications for LRFD Seismic Bridge Design*, a standalone document, which represents a "displacement-base" design philosophy.

Design all bridges for full seismic loading according to the 2nd edition of *AASHTO Guide Specifications for LRFD Seismic Bridge Design*. Obtain approval by the ODOT State Bridge Engineer if the use of *AASHTO LRFD Bridge Design Specifications* is to be considered for any unique project on state-owned bridges.

Comply with ODOT's additional requirements and guidelines summarized in *BDDM 1.17.2* if designing seismically according to *AASHTO LRFD Bridge Design Specifications* ~~for projects initiated prior to May 1st, 2009,~~ or *BDDM 1.17.3* if designing seismically according to *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

Notify and consult ODOT Bridge Section for decisions involving deviations to the standard seismic design practices described in this manual. Deviations from the following guidelines should be justified and documented. The documentation should be in the permanent bridge records.

At the end of the design process, fill in and submit to ODOT Bridge HQ a copy of the Seismic Design/Retrofit Data Sheet.

Seismic load effects should be considered for all projects using the following guidelines:

1.17.2 Applications of AASHTO LRFD Bridge Design Specifications

1.17.2.1 General Considerations

New Bridges: Design all bridge components for full seismic loading according to the current edition of *AASHTO LRFD Bridge Design Specifications*, except as modified by *BDDM 1.11.3.5 to 1.11.3.11, and 1.17.1 to 1.17.8*. Consider the load factor for the Live Load on Extreme Event Load Combination I, $\gamma EQ = 0$ (*LRFD 3.4.1*), unless the bridge is designated by Bridge Section as a major, unusual or unique structure.

The Structural Engineer/Designer should rely on the project Geotechnical Engineer to provide the seismic hazards, ground motions, deformations and additional permanent loads.

~~Seismic ground motion values should be based on the 2002 USGS Seismic Hazard Maps. ODOT versions of these maps are available at the ODOT Bridge Standards and Manuals web page. The 2002 USGS Seismic Hazard Maps and other ground motion data may be obtained from the USGS web site. The latitude and longitude of the site is needed to obtain the most precise data.~~

~~A program to develop the response spectra using the general procedure has been developed by the Bridge Section and can be accesses through the following the link:~~

~~http://www.oregon.gov/ODOT/HWY/BRIDGE/standards_manuals.shtml~~

~~ODOT requires all new bridges to be designed for a two-level performance criteria as follows:~~Design new bridges on or West of US97 for a two-level performance criteria; Life Safety and Operational. Design new bridges east of US97 for the Life Safety criteria only.
Seismic Design Criteria for Life Safety and Operational are described below.

~~(1) 1000 year~~ "Life Safety" Criteria: Design all bridges for a 1000-year return period earthquake (7% probability of exceedance in 75 years) ~~under~~ to meet the "Life Safety" criteria using the 2014 USGS Hazard Maps. The probabilistic hazard maps for the 975-year and 475-year return periods are available at ODOT Bridge Section website, but currently not available on USGS website. To satisfy the "Life Safety" criteria, use Response Modification Factors from *LRFD Table 3.10.7.1-1* using an importance category of **"other"**.

To aid in providing consistency and efficiency, Bridge Section has developed an excel application for constructing the probabilistic response spectrum using the general procedure (three-point curve). Latitude, Longitude, and Site Class are the needed input. Version 2014 (v1.0) of this excel application has been released to incorporate the updated Response Modification Factors associated with the 2014 hazard maps and can be obtained at the following link:

http://www.oregon.gov/ODOT/HWY/BRIDGE/Pages/seismic.aspx

Replace *LRFD Tables 3.10.3.2-1, 3.10.3.2-2,* and *3.10.3.2-3* with Tables 1.17.3-1A, 1.17.3-1B, 1.17.3-1C.

~~(2) 500-year~~ "Operational" Criteria: ~~In addition to the 1000-year "Life Safety" criteria, d~~Design all bridges on and West of US97 to remain "Operational" after a ~~500-year return period event (14 percent probability of exceedance in 75 years)~~full rupture of Cascadia Subduction Zone Earthquake (CSZE). The full rupture CSZE hazard maps are available at the ODOT Bridge Section website. To satisfy the "Operational" criteria, use Response Modification Factors from *LRFD Table 3.10.7.1-1* using an importance category of **"essential"**. When requested in writing by a local agency, the "Operational" criteria for local bridges may be waived.

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 needed input for running the application. This application can be accessed at the following link:

http://www.oregon.gov/ODOT/HWY/BRIDGE/Pages/seismic.aspx

~~Long Span~~Non-conventional Bridges: *LRFD 3.10.1* states that the seismic provisions of that manual are applicable for conventional bridges~~ with spans not exceeding 500 ft~~. For seismic design of non-conventional bridges~~ with spans exceeding 500 feet~~, consult with the Seismic Design Standards & Practice Engineer to discuss whether special analysis and design procedures are warranted.

Bridge Widenings: Design selected bridge portions for seismic loading as directed by the flowchart shown in *Figure 1.17.2-1A*. Design by the same criteria as for "New Bridges".

Potential Factors Affecting Seismic Performance of Bridge Widenings – The following considerations refer to the flow chart in *Figure 1.17.2-1A*. The consideration number refers to the corresponding numbered decision box on the flow chart.

Consideration 1

- Widening without adding new columns will make a bridge more vulnerable to seismic loads. Clearances for railroads or highways under structures may prevent adding new columns.

Consideration 2

- Widening on both sides will increase the potential for the new portion to be able to resist seismic loads for the full widened structure.
- Widening on one side only may actually result in a completed structure that is more vulnerable than the original structure.
- If widening is on one side only, is there a possibility another future widening could be placed on the opposite side?
- It will not normally be practical for a widening to resist the total seismic load (existing and widening) when widening on only one side; however, there ~~will~~ could be exceptions~~, however!~~.

Consideration 3

- A formal seismic analysis may be required to answer this question. A “yes” answer to Consideration 3 assumes only minimal work (such as column jacketing) will be needed for the existing structure.
- Although the existing structure may have inadequate capacity, it will have some capacity that can probably be taken advantage of.
- If existing columns are not stressed beyond the elastic range they will probably not need a Phase 2 retrofit.
- The existing structure will have to go through the same deformations as the new portion even though the capacity may not be included in the seismic analysis.

Consideration 4

- Structures which are connected must have compatible deflections at connections.
- We are usually not concerned about the seismic load generated from one structure colliding with an

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adjacent structure; however, there ~~are~~ could be exceptions~~, however!~~.

- Providing a joint between the widening and existing structure will probably increase the potential for the new portion to resist seismic loads. If the widening adds enough width for at least two lanes and the longitudinal joint would not be in a travel lane, a joint should be considered.

Consideration 5

- Base isolation is strongly encouraged, especially when bearing replacement is required anyway.
- When footing strengthening is required, Phase 2 will probably not be practical due to the high cost. If cost is the primary decision factor, a realistic estimate of Phase 2 retrofit cost should be prepared. Don't say it costs too much without knowing how much too much is!
- The closer footings are to the ground surface, the more practical Phase 2 will become.

Consideration 6

- If you can't see the new portion acting separately, do not waste time assuming it will!
- Widenings with only one new column per bent vs. multiple columns on the existing structure probably do not need to be modeled separately.
- When widening with 2 or more columns or with drilled shafts, it is probably reasonable to model the new structure separately.
- Consider the potential for another future widening. Perhaps size the footings larger than necessary.

Consideration 7

- Is it even possible to close the structure to replace it? Can it be replaced in stages? Is it historic?
- A new structure will usually be far superior to a "band-aided" structure.

Consideration 8

- FHWA requirements take effect when the new structure actually has more travel lanes than the existing structure. Widenings that add only shoulder width or median width are not affected. FHWA requirements may assist in convincing Region of including Phase 2 seismic retrofit, but it is not intended to force a Phase 2 retrofit when it really is not practical.
- For projects exempt from FHWA review, the Technical Services Branch Manager will approve exceptions to FHWA policy.

Consideration 9

- Region holds the money. They may have factors/priorities we don't know about.

~~Consideration 10~~

- Refusal by Region to fund the needed retrofit and refusal by FHWA to grant an exception (if federal funding) could lead to cancellation of the project.
- It would be desirable to calculate a cost-benefit ratio. ~~Unfortunately, no guidelines are available to determine the appropriate input values.~~

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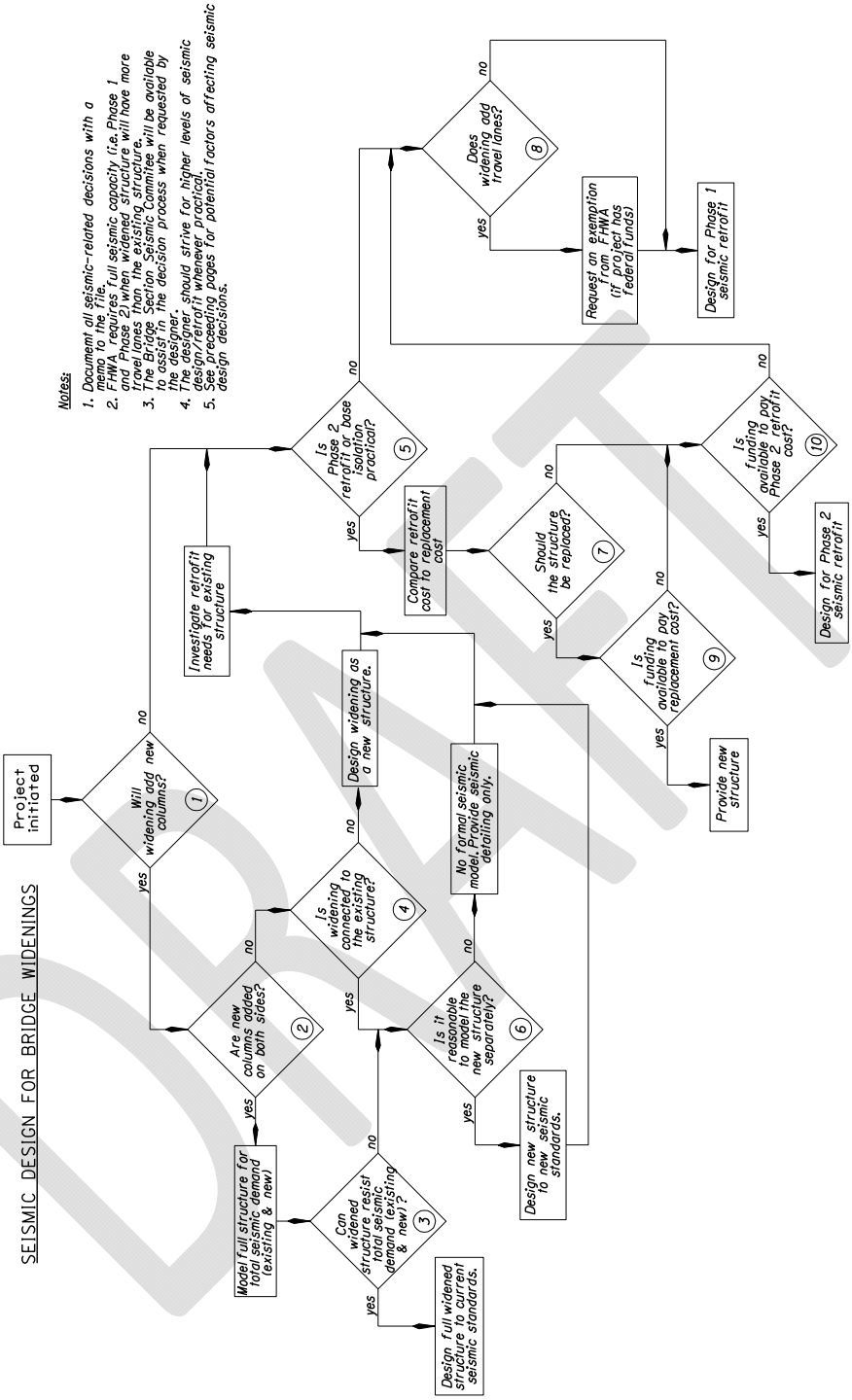

Figure 1.17.2-1A

Seismic Retrofit: There is currently no funding within ODOT solely to upgrade the seismic load resistance of selected structures. However, 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 and Drafting 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 with the following:

Table 1.17.2-1A

Minimum performance levels for retrofitted bridges

EARTHQUAKE GROUND MOTION	BRIDGE IMPORTANCE and SERVICE LIFE CATEGORY					
	Standard			**Essential**		
	ASL 1	**ASL 2**	**ASL 3**	**ASL 1**	**ASL 2**	**ASL 3**
Lower Level Ground Motion ~~50 14 percent probability of exceedance in 75 years;~~ ~~return period is about 100 500 years.~~ Cascadia Subduction Zone Earthquake – Full Rupture	PL0[4]	~~PL3~~PL2	~~PL3~~PL2	PL0[4]	~~PL3~~PL2	PL3
Upper Level Ground Motion 7 percent probability of exceedance in 75 years; return period is about 1,000 years.	PL0[4]	PL1	PL1	PL0[4]	PL1	PL2

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. This is the nature of almost all our retrofit program at this time.

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Incorporate Phase 1 Seismic Retrofit on all bridge 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, include at least Phase 1 Retrofit. Assure 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, and/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 assure 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 "~~Serviceability~~Operational" performance of Oregon bridges under the ~~500-year~~ seismic ~~induced~~ 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 flowchart for seismic design of widenings in *BDDM 1.17.2.1* (*Figure 1.17.2-1A*) can be used as a guide to make the decision. On major, unusual or border bridges, the decision should involve discussion with Bridge Section, since seismic retrofit criteria for these structures are specific to the site.

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".

Temporary Detour Bridges:

Design all temporary detour bridges that are expected to be in service for more than one year according to *LRFD 3.10.10*.

For all bridges that are expected to be in service for one year or less, provide the minimum support length requirements according to *LRFD 4.7.4.4*.

1.17.2.2 Specification Interpretations and Modifications

Nomenclature:

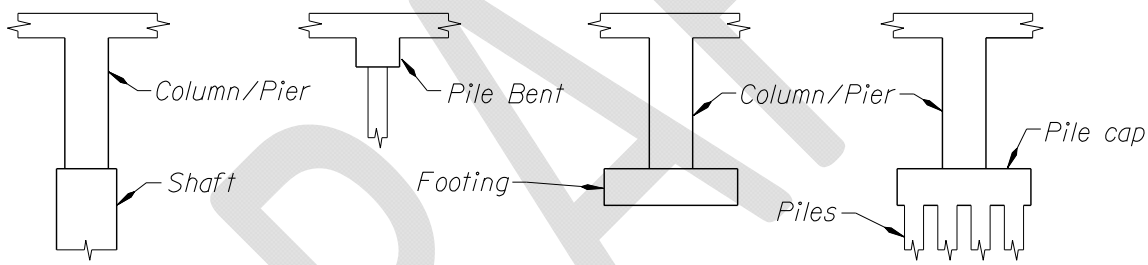

Figure 1.17.2-2A

Response Modification Factors and other Special Items:

<u>All Single Spans:</u>

- No response modification factors -- not applicable.
- Provide for connection force of: “Tributary weight” x "A_s", where $A_s = F_{pga}$*PGA, or provide the specified minimum support length according to *LRFD 4.7.4.4*.
- Free standing abutments (expansion jointed systems) are to be designed for pseudostatic Mononobe-Okabe method lateral earth forces.

<u>Seismic Zone 1:</u>

- No response modification factors -- not applicable.
- Provide for connection force of:
  $0.15*F_v$, when $A_s < 0.05$, or

$0.25*F_v$, when $A_s \geq 0.05$, where F_v is the vertical reaction at connection, or provide the specified minimum support length.

Seismic Zone 2:

- Design and detail Zone 2 structures by Zone 3 and 4 criteria except for the following design provisions:
- When determining the capacity for compression-controlled sections for extreme event limit state use Resistance Factors of Φ=0.75 as specified for Zone 2 in *LRFD 5.5.4.2.1*.
- When designing the reinforcement for compression members, design in accordance with *LRFD 5.7.4.2* "Limits of Reinforcement" for Seismic Zone 2.

Zones 3 and 4:

- Columns and Piers:
  - Moment: R = 2 to 5 (*LRFD Table 3.10.7.1-1*, right column)
  - Shear: R = 1
  - Axial: R = 1

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.

Foundations:

- Pile Bent: Treat as columns and piers (R=5). Splices shall be designed to at least the lesser of $1.3(M_{plastic})$ for the portion above or below the splice. This splicing requirement shall not apply to full penetration welded splices.

Footing - pile cap - piles:

- Moment, shear, & axial: R = 1 (elastic analysis forces) or,
- Moment: Plastic moment capacity of the selected column.
- Shear and Axial: Value accompanying the plastic moment capacity of the column (see "Columns" above).

Other Special Items:

- Confining Reinforcement (plastic hinge zones)
  - ~~The~~ Provide transverse reinforcement for confinement ~~required~~ in plastic hinging zone ~~shall~~ to satisfy *LRFD equations 5.7.4.6-1, 5.10.11.4.1d-1, 5.10.11.4.1d-2, and 5.10.11.4.1d-3*.
  - Plastic zone limits ~~shall be~~are defined as the greatest of maximum column dimension, (column height)/6, or 18 inches.
  - Extend confining reinforcement into footing or crossbeam by the greatest of (maximum column dimension)/2, or 15 inches.
  - Maximum confining reinforcement spacing is the lesser of (the least member dimension)/4, or

4 inches.

- o Shear reinforcing meeting the detailing requirements of confining reinforcement may be considered as part of the required confining reinforcing.

Column Moment Strength Reduction Factor (Φ factor)

- Use Φ = 0.9 on checking the P-Δ Requirements as per *LRFD 4.7.4.5*.

Column Shear Strength Modifications (end regions)

- End region limits ~~shall be~~are defined as the greatest of maximum column dimension, (column height)/6 or 18 inches.
- If axial stress > 0.1f´c use Vc as specified in *LRFD 5.8.3*. Vary Vc linearly from normal value to 0 for axial stress between 0.1f´c and 0.

Longitudinal Reinforcement Development

- Provide anchorage development for steel stress ≥ 1.25fy.

1.17.2.3 Detailing

(1) Columns:

- For column design and reinforcement practices, see BDDM 1.11.3.

(2) Footings:

- All footings must have a top mat of bars whether or not uplift is calculated. Extend spirals at least 2 inches into top of the footing. Place the footing top mat immediately below the spiral termination. Place additional spirals below the mat (use a 6 inch spiral gap) as needed to meet the confining reinforcement layout of BDDM 1.11.3.11. Use the same spiral pitch at all locations. See the optional detail for alternate containment reinforcing in the column to footing connection in BDDM 1.11.3.10.
- Note the allowable reduction in reinforcement development length for bars enclosed within a spiral (*LRFD 5.11.2.1.3*).

(3) Crossbeams:

- For ~~C~~column to crossbeam connections where plastic moment capacity is required, ~~shall have~~provide spirals extending into the crossbeam in the same general manner as described above for the column-to-footing connection.

1.17.2.4 Structure Modeling

(1) 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.

(2) Footing Springs:

- See *BDDM 1.10.4*.

(3) 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.

- ~~M-STRUDL, a PC program, has been~~MIDAS and GTStrudl are ODOT's primary in-house static and dynamic analysis programs, ~~tool. GT STRUDL and MIDAS can also solve dynamic problems~~ and are 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.

(4) Sample Problems:

- Sample problems are shown in the Bridge Example Design notebook, and can be downloaded under Seismic Design Examples.

1.17.2.5 Footing/Pile Cap Design

(1) Piling:

- Nominal pile resistances should be used with the seismic load case (*LRFD Table 3.4.1-1*, Extreme Event-I) to determine pile requirements. Uplift resistance may be used for friction piles if the piles are properly anchored. Consult with the Geotechnical~~Foundation~~ designer for site specific values. Piles under tension that are not capable of resisting uplift should be neglected during analysis for seismic loadings. The remaining piles must provide sufficient support and stability.

(2) Reinforcing Steel:

- Control of cracking requirements of *LRFD 5.7.3.4* do not apply to seismic load cases.

- Pile supported footings should normally have the bottom mat reinforcing above the pile tops. Footings with this scheme are preferable to thinner footings with the bottom mat detailed lower (between the piling). This is for constructability.

1.17.3 Applications of AASHTO Guide Specs for LRFD Seismic Bridge Design

1.17.3.1 General Considerations

As of 2009, ODOT has fully adopted the use of *AASHTO Guide Specifications for LRFD Seismic Bridge Design* for designing Oregon bridges subjected to earthquake loading. The following summarizes ODOT's additional requirements and deviations from the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

Design all bridge components for full seismic loading according to the 2nd edition of *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, except as modified by *BDDM 1.11.3.5 to 1.11.3.11, and 1.17.1 to 1.17.8*. Consider the load factor for the Live Load on Extreme Event Load Combination I, $\gamma EQ = 0$, unless the bridge is designated by Bridge Section as a major, unusual or unique structure. ~~Seismic ground motion values should be based on the 2002 USGS Seismic Hazard Maps. ODOT versions of these maps are available at the ODOT Bridge Standards and Manuals web page. The 2002 USGS Seismic Hazard Maps and other ground motion data may be obtained from the USGS web site. The latitude and longitude of the site is needed to obtain the most precise data.~~

The Structural Engineer/Designer should rely on the project Geotechnical Engineer to provide the seismic hazards, ground motions, deformations and additional permanent loads.

Design new bridges on or West of US97 for a two-level performance criteria; Life Safety and Operational. Design new bridges east of US97 for the Life Safety criteria only.
Seismic Design Criteria for Life Safety and Operational are described below.

~~ODOT requires all new bridges to be designed for a two-level performance criteria as follows:~~

~~(1) 1000-year~~ "Life Safety" Criteria: Design all bridges for a 1000-year return period earthquake (7 percent probability of exceedance in 75 years) ~~under~~ to meet the "Life Safety" criteria using the 2014 USGS Hazard Maps. The probabilistic hazard maps for the 975-year and 475-year return periods are available at ODOT Bridge Section website, but currently not available on USGS website.

To aid in providing consistency and efficiency, Bridge Section has developed an excel application for constructing the probabilistic response spectrum using the general procedure (three-point curve). Latitude, Longitude, and Site Class are the needed input. Version 2014 (v1.0) of this excel application has been released to incorporate the updated Response Modification Factors associated with the 2014 hazard maps and can be obtained at the following link:

http://www.oregon.gov/ODOT/HWY/BRIDGE/Pages/seismic.aspx

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

Seismic Design Categories (SDC) A, B and C

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

Seismic Design Category (SDC) D

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

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

The above guidelines are applicable even for the other Seismic Design Categories, if Pushover Analysis will be used instead of the implicit equation.

Replace *AASHTO Guide Spec Table 3.4.2.3-1* with two following tables:

Table 1.17.3-1A

Values of Site Factor, F_{pga}, at Zero-Period on Acceleration Spectrum

Site Class	Mapped Peak Ground Acceleration Coefficient (PGA)[1]					
	PGA ≤ 0.1	PGA = 0.2	PGA = 0.3	PGA = 0.4	PGA = 0.5	PGA ≥ 0.6
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.2	1.2	1.2	1.2	1.2
D	1.6	1.4	1.3	1.2	1.1	1.1
E	2.4	1.9	1.6	1.4	1.2	1.1
F[2]	*	*	*	*	*	*

Table 1.17.3-1B

Values of Site Factor, F_a, for Short-Period Range of Acceleration Spectrum

Site Class	Mapped Spectral Acceleration Coefficient at Period 0.2 sec (S_S)[1]					
	S_S ≤ 0.25	S_S = 0.5	S_S = 0.75	S_S = 1.0	S_S = 1.25	S_S ≥ 1.5
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E[3]	2.4	1.7	1.3	*	*	*
F[2]	*	*	*	*	*	*

Replace *AASHTO Guide Spec Table 3.4.2.3-2* with following table:

Table 1.17.3-1C

Values of Site Factor, F_v, for Long-Period Range of Acceleration Spectrum

Site Class	Mapped Spectral Response Acceleration Coefficient at Period 1.0 sec (S_1)[1]					
	S_1 ≤ 0.1	S_1 = 0.2	S_1 = 0.3	S_1 = 0.4	S_1 = 0.5	S_1 ≥ 0.6
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8

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C	1.5	1.5	1.5	1.5	1.5	1.4
D[4]	2.4	2.2[4]	2.0[4]	1.9[4]	1.8[4]	1.7[4]
E[4]	4.2	3.3[4]	2.8[4]	2.4[4]	2.2[4]	2.0[4]
F[2]	*	*	*	*	*	*

Notes:

[1] – Use straight-line interpolation for intermediate values of PGA, S_S, or S_1.

[2] – Perform a site-specific geotechnical investigation and dynamic site response analysis for all sites in Site Class F.

[3] – Perform a ground motion hazard analysis for structures on Site Class E sites with S_S greater than or equal to 1.0.

[4] – Perform a ground motion hazard analysis for structures on Site Class D and E sites with S_1 greater than or equal to 0.2.

~~(2) 500-year~~ "Operational" Criteria: ~~In addition to the 1000-year "Life Safety" criteria, d~~Design all bridges on and West of US97 to remain "Operational" after a ~~500-year return period event (14 percent probability of exceedance in 75 years)~~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 needed input for running the application. This application can be accessed at the following link:

http://www.oregon.gov/ODOT/HWY/BRIDGE/Pages/seismic.aspx

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 ~~a 500-year return event~~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

- ~~Each bridge bent shall s~~Satisfy ~~the~~ *equation 4.8-1* of the AASHTO Guide Spec ($\Delta^L_D < \Delta^L_C$) for each bridge bent, where Δ^L_C is determined from the *equation 4.8.1.1* of the AASHTO Guide Spec (displacement capacity for SDC B).

Seismic Design Category (SDC) D

- Meet all design requirements for SDC D according to the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, except as modified below:
  - Ensure ~~T~~the maximum concrete strain in confined section of the columns ~~shall~~ does not exceed $\varepsilon_{cc} = 0.005$

- Ensure ~~T~~the maximum strain of reinforcing steel ~~shall~~ does not exceed $2*\varepsilon_{sh}$, where ε_{sh} is defined on *Table 8.4.2-1* of the AASHTO Guide Spec.
- Ensure ~~T~~the maximum strain of prestressing steel (for 270 ksi strands) shall not exceed $\varepsilon_{ps,EE} = 0.0086$

~~Long Span~~Non-conventional Bridges: *Article 3.1* of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* states that the seismic provisions of this Manual are applicable for conventional bridges ~~with spans not exceeding 500 ft~~. For seismic design of non-conventional bridges ~~with spans exceeding 500 feet~~, consult with the Seismic Design Standards & Practice Engineer to discuss whether special analysis and design procedures are warranted.

Pedestrian Bridges: *Article 3.6* of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* states that pedestrian bridges over roads carrying vehicular traffic shall satisfy the performance criteria defined for other highway bridges. Design new pedestrian bridges over roads carrying vehicular traffic per the requirements of this section. However, pedestrian bridges that do not cross roads carrying vehicular traffic do not need be designed for the ~~500 year~~ "Operational" Criteria.

Buried Structures: According to *Article 3.1* of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, buried structures, generally, do not need be designed for seismic loads. However, for all buried structures supported on piling or drilled shafts type foundations, design the structure for seismic loading as required by this section.

1.17.3.2 Specification Interpretations and Modifications

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 Spec *Article 3.3*. However, in case of a steel substructure, design the bridge ~~shall be designed~~ according to the latest edition of the *AASHTO LRFD Bridge Design Specifications*.

  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 ~~upon the approval of the ODOT~~if approved by the State Bridge ~~Section~~Engineer. ~~The designer shall i~~Include a clear description of the selected Seismic Design Strategy in the appropriate Calculation Book for the structure.

- The following types of Earthquake Resisting Systems (ERS) or Earthquake Resisting Elements (ERE) provided in AASHTO Guide Spec *Article 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, 12 and 14 on *Figure 3.3-1b*

- Obtain ~~Agency~~ approval from the State Bridge Engineer before considering the application of the following types of Earthquake Resisting Systems (ERS) or Earthquake Resisting Elements (ERE) provided in AASHTO Guide Spec *Article 3.3*:

  - Type 6 on *Figure 3.3-1a*
  - Types 4, 5, 6 and 11 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 Spec *Article 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 Spec *Article 3.5*) when~~:~~
- $0.25 \leq S_{D1} < 0.30$.

- 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 Spec *Article 3.5*. However, in such a case provide SDC D Level of Detailing, regardless of what SDC the structure is designed for.

- Satisfy the balanced stiffness and balanced frame geometry requirements for all bridges in SDC C and D (AASHTO Guide Spec *Article 4.1.2 and 4.1.3*).

- Use Procedure Number 2 (Elastic Dynamic Analysis) for designing all bridges with two or more spans under seismic loading (AASHTO Guide Spec *Article 4.2*).
  Use Procedure Number 3 (Nonlinear Time History), where applicable, with ~~Agency's~~ approval from the State Bridge Engineer.

- Use a Damping Ratio of 5 percent (AASHTO Guide Spec *Article 4.3.2*) on all new bridges for seismic loading. The application of the reduction factor, R_D, is not allowed without ~~Agency's~~ 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 Spec *Article 4.7.1*) only upon ~~Agency's~~ approval from the State Bridge Engineer.

- Design Longitudinal Restrainers (AASHTO Guide Spec *Article 4.13.1*) in accordance with *BDDM 1.17.8*.

- Participation of the end panel, wingwalls, and backwalls in the overall dynamic response of bridge systems may be considered in seismic design of bridges. The provisions of *Article 5.2* of AASHTO Guide Spec may be used to determine the stiffness of abutment and wingwall backfill material in lieu of *BDDM 1.10.4.2(1)*.

- Select the Foundation Modeling Method (FMM) (AASHTO Guide Spec *Article 5.3.1*) according to *BDDM 1.10.4*. For spread footing foundations, multiply the initial stiffness (spring constant) as defined in *BDDM 1.10.4* by 2.
  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 *Article 7.2* of AASHTO Guide Spec in conjunction with the forced-based seismic design procedure utilized in the *AASHTO LRFD Bridge Design Specification* and requirements of this section of the BDDM.

- Provide minimum shear reinforcement for bridges in SDC A, when $0.10 \leq SD1 \leq 0.15$, according to the requirements of *Article 8.6.5* for SDC B, in addition of satisfying the requirements of AASHTO Guide Spec *Article 8.2*.

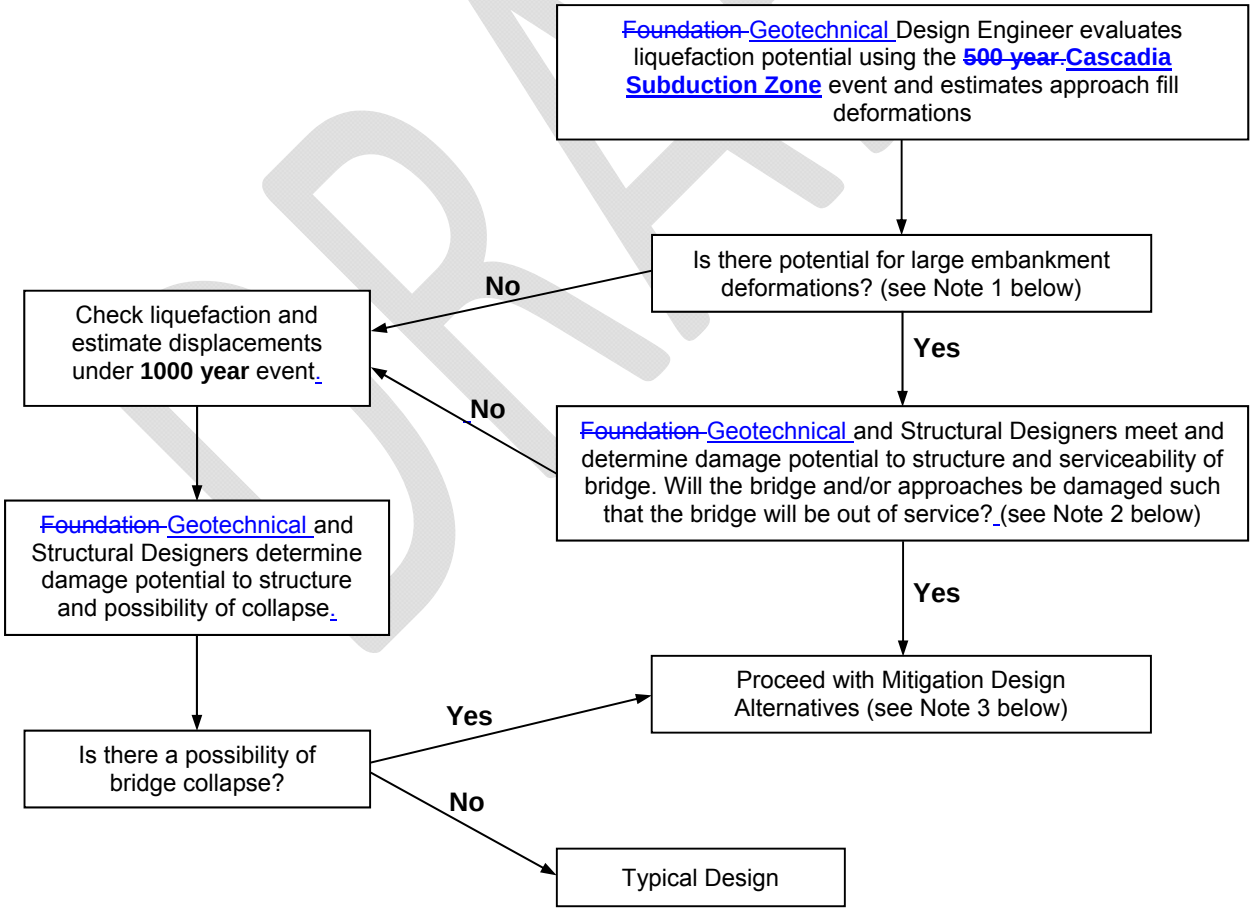
- Do not use wire rope or strands for spirals, and high strength bars with yield strength exceeding 75 ksi.

Deformed welded wire fabric (AASHTO Guide Spec *Article 8.4.1*) may be used with ~~Agency's~~ approval from the State Bridge Engineer.

- The same size vertical bars may be used inside and outside of interlocking spirals (AASHTO Guide Spec *Article 8.6.7*).
- Provide minimum longitudinal reinforcement (AASHTO Guide Spec *Article 8.8.2*) of 1 percent for columns in SDC B, C and D.
- Extend the vertical column bars into oversized drilled shaft according to *BDDM 1.10.5.5*, in lieu of AASHTO Guide Spec *Article 8.8.10*.

1.17.4 Liquefaction Evaluation and Mitigation Procedures

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



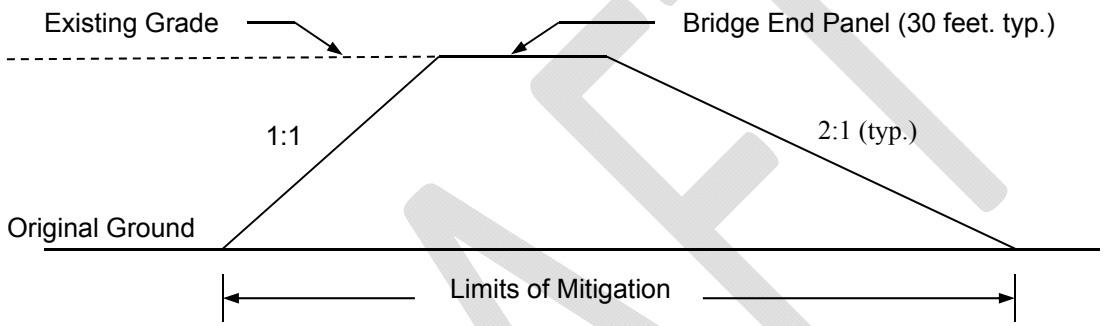
Note 1: For meeting the performance requirements of the ~~500 year return~~Cascadia Subduction Zone event

(~~serviceability~~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. Larger lateral deformations and settlements may be acceptable under the 1000 year event as long as the "~~no-collapse~~Life Safety" criteria are met.

Note 2: The bridge should be open to emergency vehicles after the Cascadia Subduction Zone~~500-year design~~ 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: Submit all liquefaction mitigation designs and cost estimates (either ground improvement or structural) to the Bridge HQ for review and approval. Refer to ODOT research report SPR Project 361: "Assessment and Mitigation of Liquefaction Hazards to Bridge Approach Embankments in Oregon", Nov. 2002 and FHWA Demonstration Project 116; "Ground Improvement Technical Summaries, Volumes I & II", (Pub. No. FHWA-SA-98-086) for mitigation alternatives and design procedures.

As a general guideline, the foundation mitigation should extend from the toe of the end slope to a point that is located at the base of a 1:1 slope which starts at the end of the bridge end panel:



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1.17.5 Costs

(1) Construction costs:- Apply the following factors to TS&L (preliminary) structure cost estimates to approximate the additional cost of seismic criteria (excluding liquefaction):

- Single Spans: 1.00
- Multiple Spans: 1.30 Irregular (widely varying columns lengths or support materials; unusual geometry or curvature)
- 1.10 Other

(2) Design costs:- Apply the following factors to TS&L (preliminary) design cost estimates to approximate the additional cost of seismic design criteria (excluding liquefaction):

- Single Spans: 1.00
- Multiple Spans: 1.20 Trestles
  1.50 Irregular (widely varying columns lengths or support materials; unusual geometry or curvature)
  1.35 Other

1.17.6 Instrumentation

Consider ~~P~~placement of accelerometers on the ground and on structure portions ~~should be considered~~ for large or unusual structures. ~~The Designer and Supervisor should c~~Consult with the ~~Seismic Design Standards and Practice~~State Bridge Engineer to determine~~cide~~ if this is appropriate and fits with the ODOT ~~Seismic Instrumentation and Monitoring~~Strong Motion Program.

1.17.7 Dynamic Isolators

Isolators may be useful for either new construction or retrofit work. Isolators change structure response by lengthening the periods of primary vibration. This tunes the structure response away from the typical earthquake's maximum response frequencies. This effect, along with added damping, works to reduce the system response. The result is reduced substructure forces.

Typical steps to model an isolated structure include:

1. Compute service loads (D, L, LF, CF, W, WL, R, S and T) for the worst single girder.

2. Use these loads, and the applicable seismic loading, in the Dynamic Isolation System, Inc. (DIS) program PC-LEADER to get a preliminary isolator size and its properties. DIS has given us permission to use the program even though we will not specify only their bearing.

3. Develop a full ~~M-STRUDL~~structural model (superstructure, substructure, and bearings/isolators). Normally this will be done on a per girder basis so the substructure should be proportioned to fit this basis. The model can often be a two dimensional model.

4. In the ~~M-STRUDL~~structural model use the equivalent isolator stiffness (Keff). This stiffness should be further modified to fit modeling assumptions of a bearing cantilevered from the substructure at interior supports.

5. Load the ~~M-STRUDL~~structural model with dynamic loading through a modified response spectrum.

The response spectrum can be taken from the PC-LEADER output or developed from the Guide Specification for Seismic Isolation Design.

6. Develop another full ~~M-STRUDL~~structural model to represent the "as-is" structure. Dynamically load this model with a normal response spectrum. This gives a basis to evaluate the isolation effectiveness.

7. It may be necessary or desirable to adjust the relative isolator stiffness to better distribute the dynamic forces. It is important the final isolator properties function adequately for service loads. The isolator characteristics must also be realistic and achievable.

An example isolator modeling is given in the Bridge Example Design notebook.

Other computer programs are acceptable, provided the programs satisfy the analysis requirements and have been previously verified.

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

(1) Concrete:

Concrete bearing strength based on $0.85f'_c$ (Φ = 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*.

(2) Structural Steel:

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

(3) Fasteners:

(Steel to Steel)

		A 307		A 325	
Diameter	Nominal Area (in^2)	Tension (0.76 x 60 ksi)	Shear (0.38 x 60 ksi)	Tension (0.76 x 120 ksi)	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

Note: 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% if the threads are excluded from the shear plane.

(4) Steel Rods:

		A307 Fu = 58 ksi	A449 Fu varies	
Diameter	Stress Area (in^2)	Tension (kips) Ft=Fy=36 ksi	Tension (kips) Ft = Fy	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	

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.

(5) Wire Rope:

See *BDDM 1.21* 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.

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

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The area values above are based on ASTM A603. The minimum breaking strength above is based on ASTM A1023. The design load above 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. ~~For 7/8 inch A603, the current ODOT stockpile of 7/8 inch diameter wire rope (purchased in September 2000) has been certified to 80,000 pounds. Therefore, the A1023 loads can be used for all seismic retrofit applications. If a newer stockpile of A603 wire rope is procured by ODOT, the actual breaking strength will need to be verified and the design load may require minor adjustment.~~

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.

(6) Resin Bonded Anchors:

See *BDDM 1.20.2*, "Drilled Concrete Anchors"

(7) Concrete Inserts:

Use hot-dip galvanized expanded coil concrete inserts with closed-back ferrule threaded to receive UNC threaded bolts.

Inserts are readily available in 1/4 inch sizes. Other sizes are only available in very large quantities. Therefore, only the standard sizes listed below are recommended.

Diameter (in)	Tension (kips) A307 or A325	Shear (kips)	
		A307	A325
3/4	12.6	7.4	12.6
1	19.3	13.4	19.3
1-1/4	34.4	21.4	34.4
1-1/2	54.3	31.0	54.3

Tension and shear capacity for concrete failure is based on equation 6.5.2 from the *PCI Design Handbook* (3rd Edition) with $\Phi = 1.0$ and with a factor of safety of 1.5. Equation 6.5.2 controls both shear and tension for shallow embedment depths. See the *PCI Design Handbook* for group effects, edge distance effects and combined tension and shear.

Tension capacity of the insert cannot exceed the tension capacity of the bolt. Shear capacity of the insert cannot exceed the shear capacity of the bolt or the insert tension capacity.

Tension capacity of the bolt = $0.76A_bf_{ub}$, where A_b = bolt stress area (*LRFD equation 6.13.2.10.2-1*).
Shear capacity of the bolt = $0.38A_bf_{ub}$ (*LRFD equation 6.13.2.7-2*).

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.

(2) Bents with superstructure continuous over the bent:

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

(3) Bents with only the deck continuous over the bent:

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

(4) 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

(1) In-span hinges:

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

(2) 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.

(3) 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.

(4) 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 Use of State Stockpile Wire Rope (Cable) for Seismic Retrofit

~~To achieve economy and supply stability, Bridge Engineering Section has purchased a quantity of structural wire rope (cable) to be used on future seismic retrofit projects. The wire rope is stockpiled in Portland. Before using stockpile wire rope, contact the Bridge Operations & Standards Managing Engineer, Bridge Engineering Headquarters to verify availability. See *BDDM 1.21* for general notes and special provisions to be used with the stockpile wire rope.~~

~~For projects requiring quantities beyond the available stockpile, contact the Bridge Operations & Standards Managing Engineer to discuss whether an additional quantity of wire rope can be purchased.~~The Agency is no longer stockpiling wire rope.

Analysis / Research / Other Supporting Data:

☒ None
☐ Attached:

Bridge Engineering Section Response:

☒ Accepted for consideration as submitted ☐ Accepted for consideration as noted ☐ Proposal tabled, see Remarks ☐ Proposal not accepted, see Remarks		
Remarks:		
Craig Shike Bridge Design Standards Reviewer		Date
Alex Lim Digitally signed by Alex Lim DN: cn=Alex Lim, o=ODOT, ou=BDDM Standards Engineer, email=Alex.K.LIM@odot.state.or.us, c=US Date: 2016.10.03 17:32:42 -07'00'		
Alex Lim Bridge Design Standards Engineer		Date 10/3/2016

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