SDG 14:

Seismic Design

Chapter 14

Tennessee Department of Transportation December 15, 2022

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Section 1 Design Specifications

Four specifications with requirements for the seismic design of bridges are:

1. AASHTO LRFD Bridge Design Specifications (AASHTO 2020)
2. AASHTO Guide Specifications for LRFD Seismic Bridge Design (AASHTO 2011)
3. AASHTO Guide Specifications for Seismic Isolation Design (AASHTO 2010)
4. Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges (Buckle, et al. 2006)

The Guide Specifications for LRFD Seismic Bridge Design (the "Guide Specification" hereafter) require a displacement-based design approach. The LRFD Bridge Design Specifications (the "LRFD Specification" hereafter) require a force-based design approach. The Guide Specification for Seismic Isolation Design (the "Isolation Specification" hereafter) is a mixture of force-based and displacement-based provisions for isolation bearings and isolated substructures. This chapter of the Structural Design Guidelines does not address seismic retrofit of bridges. Refer to the FHWA manual for retrofit projects.

14-101.00 Design Strategies

Three seismic design strategies are available. The seismic design strategy shall be determined during the preliminary design phase and includes establishing the earthquake resisting system (ERS) and elements (ERE). The three strategies are defined as follows.

1. Type 1. Ductile Substructures with Essentially Elastic Superstructure
2. Type 2. Essentially Elastic Substructures with a Ductile Superstructure
3. Type 3. Elastic Superstructure and Substructure with a Fusing Mechanism between the Two

The majority of structures are designed using the Type 1 strategy with plastic hinges forming at the base and top of columns. The Type 2 strategy shall not be used for design of new bridges until the theory is further developed. The Type 3 design strategy may be an effective solution for very stiff substructures or where it is otherwise advantageous to maintain essentially elastic behavior in the substructures.

Type 1 Bridges in Seismic Design Category (SDC) A (Seismic Zone 1) may be designed in accordance with either the LRFD Specification or the Guide Specification. Type 1 Bridges in SDC B, C, and D (Seismic Zones 2, 3, and 4) shall be designed in accordance with the Guide Specification. Type 3 bridges in any Seismic Design Category shall be designed using the Isolation Specification.

Acceptable earthquake-resisting systems are as follows. Figure 3.3-3 of the Guide Specification identifies features which shall not be used in the design of new bridges. Permissible earthquake-resisting-systems are listed in Figure 3.3-1a, and permissible earthquake-resisting-elements are listed in Figure 3.3-1b of the Guide Specification. Figure 3.3-2 outlines earthquake-resisting-elements which require the approval of the Director. Most bridges in Tennessee are designed with integral abutments. In accordance with Figure 3.3-1b of the Guide Specification, this is a permissible ERE if the passive soil strength is taken equal to 70% of that designated in Guide Specification Section 5.2.3.

14-102.00 Seismic Loading

Spectral accelerations (PGA, S_S, S_1, A_S, S_{D1}, S_{DS}) and the corresponding design response spectrum can be determined using the following method. Navigate to the following web page: https://earthquake.usgs.gov/ws/designmaps/. From the reference documents link, select "2009 AASHTO Guide Specifications (AASHTO-2009)". Select the URL link under "Example" and Request". Once the URL is open, the latitude, longitude, and site class must be replaced from the example URL with the correct values for the bridge being designed. Below is an example of a modified URL that retrieves data for the Hernando de Soto Bridge in Memphis.

https://earthquake.usgs.gov/ws/designmaps/aashto-2009.json?latitude=35.15&longitude=-90.06&siteClass=D&title=Example

Below is a sample taken from the output of the request shown above.


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"response": {
  "data": {
    "pga": 0.403,
    "fpga": 1.097,
    "as": 0.442,
    "ss": 0.75,
    "fa": 1.2,
    "sds": 0.9,
    "s1": 0.192,
    "fv": 2.032,
    "sd1": 0.39,
    "sdc": "C",
    "ts": 0.434,
    "t0": 0.087,
```

For ordinary bridges, the earthquake loading is defined in terms of an acceleration response spectrum only. For critical or recovery bridges, the earthquake loading is defined both in terms of the acceleration response spectrum and acceleration histories compatible with the design response spectrum and the design ground shaking environment (earthquake magnitude, site-to-source distance, profile depth, etc.). Guide Specification Section 4.2 and LRFD Specification Section 4.7.4 shall be used to determine the type of analysis (none, elastic response spectrum analysis, or nonlinear time history analysis) appropriate for the bridge.

When acceleration histories are required for the analysis of bridges, Section 3.4.4 of the Guide Specification and Appendix A of this document contain criteria for the selection and modification of ground motion records.

Refer to Section 2 of this document for a detailed discussion of site characterization procedures required to develop the design acceleration response spectrum.

14-103.00 Design Procedures

A summary of specific requirements for each SDC is provided in Sections 14-103.01 through 14-103.05. Any additional checks prescribed for each SDC in accordance with the Guide Specification or LRFD Specification shall be required. Refer to Guide Specification Section 3.5 as well. Flowcharts are available beginning with Guide Specification Figure 1.3-1 for displacement-based design and LRFD Specification Appendix A3 - Seismic Design Flowcharts for force-based design.

Table 1 provides a summary of seismic design requirements with specification sections for both force-based and displacement-based bridge design.

Use γ_{EQ} = 0.50 for most bridges for the Extreme Event I Load Combination of the LRFD Specification Table 3.4.1-1. For bridges in urban areas with extremely high traffic volumes, a higher value may be appropriate, and the Director shall be consulted.

Subject	***Force-based LRFD Spec***	***Displ.-based Guide Spec***
Earthquake Effects	3.10	3.4
Seismic Design Flowcharts	APPENDIX A3	1.3
Over-strength Resistance	APPENDIX B3	4.11.2, 8.5
Seismic Lateral Load Distribution	4.6.2.8	4.3, 4.4
Dynamic Analysis – Basic Requirements	4.7.1	4.1
Vertical Ground Motion Effects		4.7.2
Dynamic Analysis for EQ Loads	4.7.4	Section 5
Seat Width Requirements at Expansion Bearings	4.7.4.4	4.12
Design Acceleration Histories	4.7.4.3.4	3.4.4, 5.4.4
Concrete Structures - Extreme Event Limit State	5.5.5	Section 8
Concrete Structures, Reinforcement - Seismic Provisions	5.11	Section 8
Concrete Piles – Seismic Requirements	5.11	4.9, 8.16
Steel Structures - Extreme Event Limit State	6.5.5	Section 7
Steel Structures - Provisions for Seismic Design	6.16	Section 7
Wood Structures - Extreme Event Limit State	8.5.3	
Foundations - Extreme Event Limit State	10.5.4	Section 6
Spread Footings - Extreme Event Limit State	10.6.4	5.3.2, 6.3
Driven Piles - Extreme Event Limit State	10.7.4	5.3.3, 6.4, 6.6
Drilled Shafts - Extreme Event Limit State	10.8.4	5.3.4, 6.5
Micropiles - Extreme Event Limit State	10.9.4	
Seismic Design of Foundations	APPENDIX A10	Section 6
Walls, Abutments, and Piers - Extreme Event Limit State	11.5.8	Section 6
Seismic Design - Abutments and Conventional Walls	11.6.5	6.7
Seismic Design - Non-gravity Cantilever Walls	11.8.6	
Seismic Design - Anchored Walls	11.9.6	
Seismic Design - MSE Walls	11.10.7	
Seismic Provisions for Bearings	14.6.5	
Steel-Reinforced Elastomeric Bearings - Method B	14.7.5.3.7	
Steel-Reinforced Elastomeric Bearings - Method A	14.7.6.3.8	
Anchorage and Anchor Bolts	14.8.3.2	
Sound Barrier Foundations	15.9.9	

Table 1. Force-Based vs. Displacement-Based Provisions

14-103.01 Seismic Design Category A (LRFD Specification Option)

No dynamic analysis is required to determine seismic demands. For all bridges in SDC A, the design connection force is 25% of the tributary permanent load and the portion of the tributary live load assumed to be on the bridge during the earthquake (0.64 klf per lane in half of the design lanes over the entire bridge length.) These requirements are modified from LRFD Specification Section 3.10.9.2. Shear resistance of anchor rods shall be determined in accordance with LRFD Specification Section 6.13.2.12. Anchor rod material specification shall be ASTM F1554, Grade 36, 55, or 105 as required.

The connection design force shall be applied as a static lateral force with the entire substructure and foundations designed to remain essentially elastic under the Extreme Event I Load Combination with resistance (Φ) factors equal to 1.0.

Required seat lengths at expansion joint locations shall be determined from LRFD Specification Section 4.7.4.4.

While the LRFD Specification is a force-based approach, displacement estimates under the design force shall be made to evaluate P-Δ effects specified in Section 4.7.4.5. Base displacement estimates on an effective stiffness factor, $(EI)_{EFF} = 0.30(EI)_{GROSS}$ unless a more detailed analysis is warranted at the discretion of the Design Manager.

Transverse reinforcement requirements in the top and bottom of columns shall be as required by LRFD Specification Sections 5.11.4.1.3, 5.11.4.1.4, and 5.11.4.1.5. Column connections into caps, footings, and drilled shafts shall be as required by LRFD Specification Section 5.11.4.3.

For steel girder bridges, the LRFD Specification has additional requirements given in Section 6.16.3.

14-103.02 Seismic Design Category A (Guide Specification Option)

No dynamic analysis is required to determine seismic demands. For all bridges in SDC A, the design connection force is equal to 25% of the tributary permanent load and the portion of the tributary live load assumed to be on the bridge during the earthquake (0.64 klf per lane in half of the design lanes over the entire bridge length). These requirements are modified from Guide Specification Section 4.6. Shear resistance of anchor rods shall be determined in accordance with LRFD Specification Section 6.13.2.12. Anchor rod material specification shall be ASTM F1554, Grade 36, 55, or 105 as required.

The connection design force shall be applied as a static lateral force with the entire substructure and foundations designed to remain essentially elastic under the Extreme Event I Load Combination with resistance (Φ) factors equal to 1.0.

Required seat lengths at expansion joint locations shall be determined from Guide Specification Section 4.12.2.

P-Δ requirements shall be checked in accordance with Guide Specification Section 4.11.5, even though the bridge is in SDC A.

Transverse reinforcement requirements in the top and bottom of columns shall be as required by Guide Specification Sections 8.2, 8.6.5, 8.8.9, and 4.11.7.

14-103.03 Seismic Design Category B

Seismic displacement demands are determined from an elastic, response spectrum, dynamic analysis of the bridge unless the structure is critical or highly irregular. Guide Specification Section 4.2 provides guidance on situations where a nonlinear time history analysis may be required. The elastic, response spectrum displacements shall be multiplied by R_D when the damping ratio is determined to be other than 5% of critical, and by R_d when the structure is within the short-period classification. See Guide Specification Sections 4.3.2 and 4.3.3 to determine R_D and R_d.

Displacement capacity for each substructure shall be determined from equations in Guide Specification Section 4.8.1 or from a static pushover analysis described in Guide Specification Section 4.8.2.

Joint shear checks are not required for SDC B (Guide Specification Section 4.11.1). The plastic hinging forces in the columns of all substructures shall be determined in accordance with Guide Specification Section 4.11.2. All elements not part of the ERS shall be designed to remain essentially elastic when the forces associated with plastic hinging are applied to the structure.

Plastic moment capacities of ductile members shall be determined in accordance with Guide Specification Section 8.5.

Required seat lengths at expansion joint locations shall be determined from Guide Specification Section 4.12.2.

Shear resistance of anchor rods shall be determined in accordance with LRFD Specification Section 6.13.2.12. Anchor rod material specification shall be ASTM F1554, Grade 36, 55, or 105 as required.

P-Δ requirements shall be checked in accordance with Guide Specification Section 4.11.5, even though the bridge is in SDC B.

LRFD Specification Section 6.16 provides design requirements for steel girder bridges in SDC B which shall be implemented to complement the requirements from Guide Specification Section 7.4.7 for these structures. The provisions include requirements for deck shear and shear connectors.

14-103.04 Seismic Design Category C

Seismic displacement demands are determined from an elastic, response spectrum, dynamic analysis of the bridge unless the structure is critical or highly irregular. Guide Specification Section 4.2 provides guidance on situations where a nonlinear time history analysis may be required. The elastic, response spectrum displacements shall be multiplied by R_D when the damping ratio is determined to be other than 5% of critical, and by R_d when the structure is within the short-period classification. See Guide Specification Sections 4.3.2 and 4.3.3 to determine R_D and R_d.

Displacement capacity for each substructure shall be determined from equations in Guide Specification Section 4.8.1 or from a static pushover analysis described in Guide Specification Section 4.8.2.

Joint shear checks in accordance with Guide Specification Section 8.13 are required for SDC C.

The plastic hinging forces in the columns of all substructures shall be determined in accordance with Guide Specification Section 4.11.2. All elements not part of the ERS shall be designed to remain essentially elastic when the forces associated with plastic hinging are applied to the structure.

Plastic moment capacities of ductile members shall be determined in accordance with Guide Specification Section 8.5.

Required seat lengths at expansion joint locations shall be determined from Guide Specification Section 4.12.2.

Shear resistance of anchor rods shall be determined in accordance with the LRFD Specification, Section 6.13.2.12. Anchor rod material specification shall be ASTM F1554, Grade 36, 55, or 105 as required.

P-Δ requirements shall be checked in accordance with Guide Specification Section 4.11.5.

LRFD Specification Section 6.16 provides design requirements for steel girder bridges in SDC C which shall be implemented to complement the requirements from Guide Specification Section 7.4.7 for these structures. The provisions include requirements for deck shear and shear connectors. Without adequate shear connectors to carry inertial loads into the girders, there is no load path to the foundations for this type of bridge.

14-103.05 Seismic Design Category D

Balanced stiffness distribution and balanced frame geometry requirements are given in Guide Specification Sections 4.1.2, 4.1.3, and 4.1.4 for SDC D bridges.

Explicit calculation of strength for passive resistance at abutment endwalls is required for SDC D bridges when the endwalls are designed as part of the ERS. The provisions of Guide Specification Sections 5.2.3 and 5.2.4 shall be met.

Seismic displacement demands shall be determined from an elastic, response spectrum, dynamic analysis of the bridge unless the structure is critical or highly irregular. Guide Specification Section 4.2 provides guidance on situations where a nonlinear time history analysis may be required. The elastic, response spectrum displacements shall be multiplied by R_D when the damping ratio is determined to be other than 5% of critical and by R_d when the structure is within the short-period classification. See Guide Specification Sections 4.3.2 and 4.3.3 to determine R_D and R_d.

Displacement capacity for each substructure shall be determined from a static pushover analysis described in Guide Specification Section 4.8.2.

Displacement ductility values shall be explicitly calculated and limited to the values specified in Guide Specification Section 4.9.

Joint shear checks in accordance with Guide Specification Section 8.13 are required for SDC D.

The plastic hinging forces in the columns of all substructures shall be determined in accordance with Guide Specification Section 4.11.2. All elements not part of the ERS shall be designed to remain essentially elastic when the forces associated with plastic hinging are applied to the structure.

Plastic moment capacities of ductile members shall be determined in accordance with Guide Specification Section 8.5.

Required seat length requirements at expansion joint locations shall be determined using the actual displacement demands from the dynamic analysis as defined in Guide Specification Section 4.12.3.

Shear resistance of anchor rods shall be determined in accordance with LRFD Specification Section 6.13.2.12. Anchor rod material specification shall be ASTM F1554, Grade 36, 55, or 105 as required.

P-Δ requirements shall be checked in accordance with Guide Specification Section 4.11.5.

Guide Specification Section 8.4.1 requires the use of A706 reinforcement for SDC D structures in areas where hinging is expected. A706 reinforcement has both a cap on yield strength and a higher ultimate strain than A615 reinforcement. Also, A706 reinforcement usually provides a higher displacement capacity when used in hinging members.

LRFD Specification Section 6.16 provides design requirements for steel girder bridges in SDC D which shall be implemented to complement the requirements from Guide Specification Section 7.4.7 for these structures. The provisions include requirements for deck shear and shear connectors. Without adequate shear connectors to carry inertial loads into the girders, there is no load path to the foundations for this type of bridge.

14-104.00 Non-Traditional Design Options

The following three options are available for the design of bridges in the New Madrid Seismic Zone when traditional Type 1 design becomes economically disadvantageous:

- Foundation Rocking, Guide Specification Appendix A
- Type 2 Strategy, CCEER Report 13-15
- Type 3 Strategy, Guide Specification for Seismic Isolation Design

These options have historically been ignored, but they should be considered when a more robust structure is possible with little or no economic disadvantage. These alternative options offer the potential for faster and less expensive post-earthquake repair in the form of replacing bearings or cross-frames as opposed to Type 1 Bridges, which could require complete replacement.

Section 2 Site Characterization

To develop the design response spectrum, it is necessary to classify all project sites according to average shear wave velocity in the upper 100 feet of the foundation profile. The depth to bedrock shall also be estimated.

Section 14-201.00 summarizes the method used in averaging shear wave velocities. Maps are available in Section 14-202.00 to assist in estimating site characterization parameters. When the depth to bedrock is significantly greater than 100 feet, alternative means of developing design spectra may need to be considered. Guide Specification Sections 3.4.3 and 3.4.4 discuss site-specific hazard definition requirements.

14-201.00 Site Class Definition by Average Shear Wave Velocity

Site characterization from the Guide Specification requires some knowledge of the geotechnical properties of the soil at the bridge location. Table 2 gives approximate relationships from the literature (Priestley, Seible and Calvi, Seismic Design and Retrofit of Bridges 1996) between shear wave velocity, blow count, and unconfined compressive strength and may be helpful in cases for which blow counts at or near the structure are known before the design phase is complete.

Soil	SPT	V_{S30}, fps	s_u, psi
Stiff sand	N > 35	1300	57
Medium sand	15 < N < 35	650	21-57
Loose sand	N < 20	< 650	< 21

Table 2. Site Correlations (approximate)

14-202.00 Seismic Design Category

Once the reference rock accelerations – PGA, S_S, and S_1 - and the site class have been determined, site factors may be determined from Guide Specification Tables 3.4.2.3-1 and 3.4.2.3-2 for the majority of structures. The design response spectra control points – A_S, S_{DS}, and S_{D1} - may then be calculated using Guide Specification equations in Section 3.4.1. For critical structures or non-traditional designs, site-specific site factors - F_{PGA}, F_a, and F_v – may be required. Specific guidance on alternative methods for site factor estimation may be found in the literature (Y. M. Hashash, et al. 2008), (Malekmohammadi and Pezeshk 2014), (Moon, Hashash, & Park, 2016). In particular, the site factors from Moon may need to be considered for bridges with periods longer than T_S.

Seismic Design Category designations are based on the 1-second period design response spectrum accelerations for ground motion with a 7% probability of exceedance in 75 years. Table 3.5-1 of the Guide Specification is reproduced in Table 3.

$S_{D1} = F_v S_1$	Seismic Design Category (SDC)
$S_{D1} < 0.15$	A
$0.15 \le S_{D1} < 0.30$	B
$0.30 \le S_{D1} < 0.50$	C
$S_{D1} \ge 0.50$	D

Table 3. Seismic Design Category (SDC) Criteria

14-203.00 Embayment Depth

The Mississippi Embayment is unique in that profile depths over 0.6 miles exist over intra-plate faults. Profiles characterized as “Lowlands” in the literature have slightly lower shear wave velocities in the upper 250 feet and similar velocity profiles beyond 250 feet compared to “Uplands” profiles. The deep soil sites may deserve special attention to spectrum definition and the code-based response spectrum may be unconservative at periods larger than about 1 second and overly conservative at periods less than about 1 second. Rough estimates of embayment depth and boundaries between uplands and lowlands profiles may be obtained from the maps reproduced in Figures 1 through 3. See the literature for additional discussion on long period-structures and spectral shapes in the Mississippi Embayment (Atkinson and Boore 1995), (Fernandez and Rix 2006), (Hashash, et al. 2008), (Park and Hashash 2005).

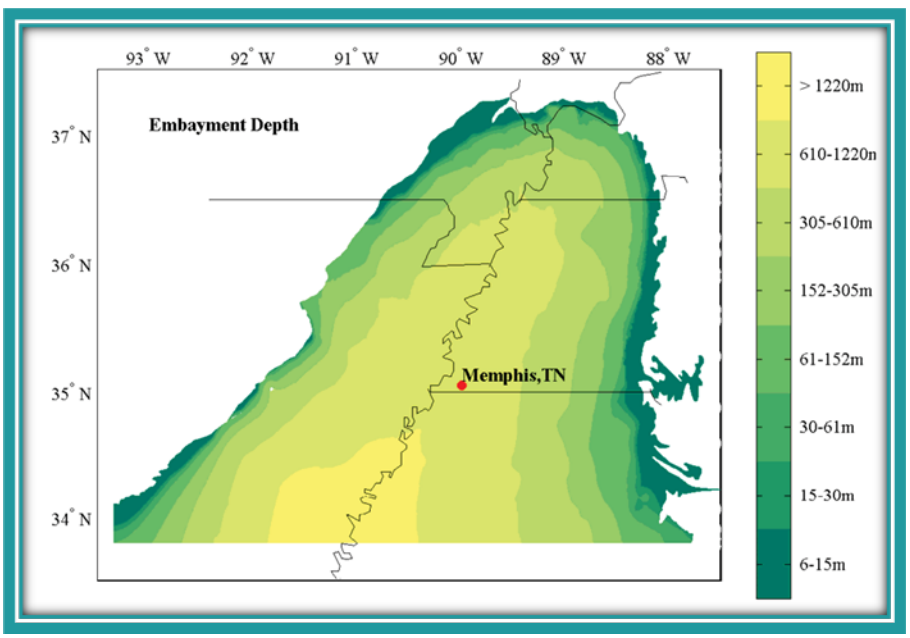

Figure 1. Mississippi Embayment Depth

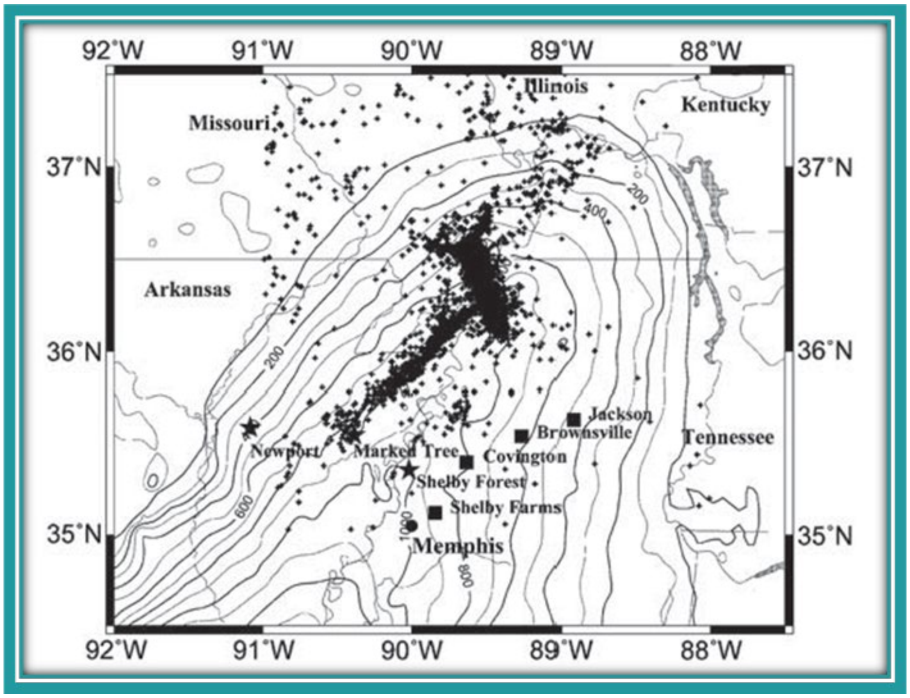

Figure 2. Mississippi Embayment Depth (2)

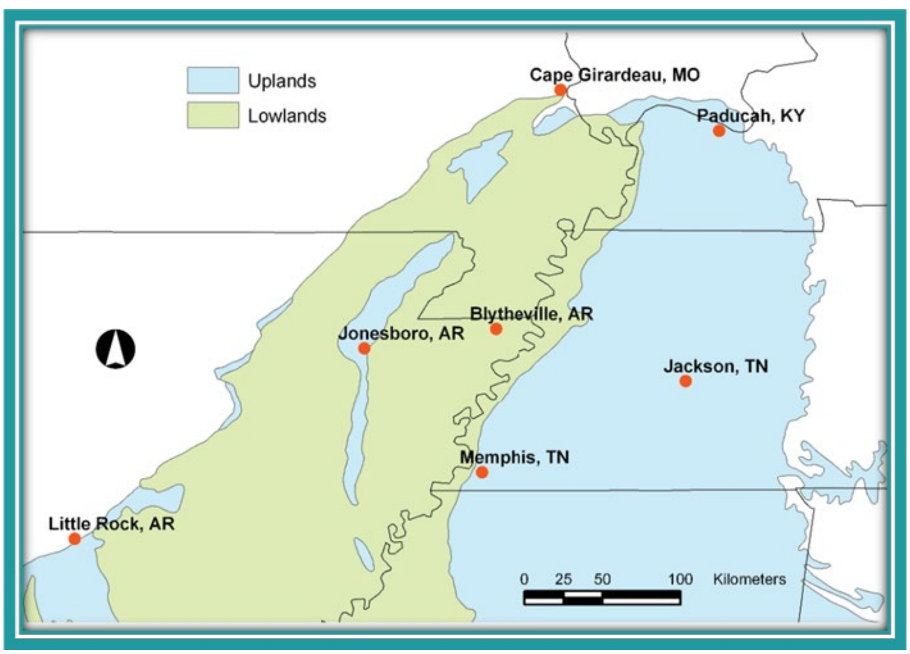

Figure 3. Mississippi Embayment Uplands and Lowlands

Section 3 Modeling and Analysis

Determine the seismic demand using a dynamic multi-mode response spectrum analysis of the structure. Either WinSeisab or CSiBridge shall be used for this analysis unless an alternate program or analysis method is approved by the Design Manager. WinSeisab is sufficient for most bridges, but CSiBridge may be required for very complex bridges at the discretion of the Design Manager. In exceptional cases, nonlinear time history analysis for a series of ground motion records may be required at the discretion of the Design Manager. Refer to Appendix A of this document and Section 4.2.2 of the Guide Specification for guidance on ground motion selection and modification procedures.

Dynamic analysis results are used primarily to obtain displacement demands at all substructures. For SDC B, the dynamic analysis member forces may also be used as the design basis forces if the hinging forces are larger. See Section 14-303.00. Short-period amplification of dynamic analysis displacements shall be applied in accordance with Guide Specification R_d-values.

Dynamic analysis results for a given response spectrum depend primarily upon the mass and stiffness distribution throughout the bridge.

Always ensure that at least 90% of the total mass in both horizontal directions is achieved with the number of modes considered in the dynamic analysis. Increase the default number of modes when necessary. Include the mass of diaphragms, abutments (for integral abutment bridges), parapets, bridge rails, median barriers, sidewalks, and overlay in addition to the self-mass of the other components. Include a 0.64 klf per lane live load over the entire bridge length in half of the design lanes as additional mass for the dynamic analysis. The number of lanes may need to be increased for urban projects with high traffic volumes. Consult the Director for such projects.

Effective member properties are required to perform the dynamic analysis. Guide Specification Section 5.6 requires moment-curvature analysis as the method for determining effective stiffness properties of the concrete columns (or piles in a concrete pile bent substructure).

14-301.00 Moment Curvature Analysis

A moment-curvature (M-Φ) analysis is required for a pushover analysis. In most situations, use CONSEC to do M-Φ analyses. The results of the analysis include:

- Yield curvature
- Ultimate curvature
- Ultimate moment

The curvatures shall be used to estimate displacement and displacement ductility capacities while the moment shall be used in calculating plastic shears and in determining loads to caps, struts, footings, and drilled shafts. Yield and ultimate curvatures may also be estimated using Appendix C.

Expected material properties shall be used for the M-Φ analysis. Section 8.4.4 of the Guide Specification requires that the concrete strength be taken as 1.3f’c. Since 3,000 psi concrete is typically specified for substructures, use 3,900 psi for the expected concrete strength unless the specified 28-day compressive strength for the substructure concrete exceeds 3,000 psi. The use of higher strength substructure concrete will help to meet shear strength and joint shear strength criteria without increasing member sizes and may be used with the approval of the Design Manager.

When Guide Specification displacement-based design is used to estimate displacement capacity, strain-hardening strain and ultimate strain values as well as expected yield and ultimate stress values are needed for the pushover analysis. Use the values given in Guide Specification Table 8.4.2-1. Always use the “Reduced ultimate tensile strain” instead of the “Ultimate tensile strain”.

CONSEC shall be used to model moment-curvature behavior of the cross-section required to hinge unless an alternate program is approved by the Design Manager. The beneficial effects of confining steel on the stress-strain properties of the concrete core shall be included in the analysis.

Cracked section properties shall be used for effective stiffness calculations.

$$EI_{EFF} = EI_{CRACKED} \tag{1}$$

$(EI)_{EFF}$ is simply the pre-yield slope of the bi-linear, idealized moment-curvature diagram. See Figure 4.

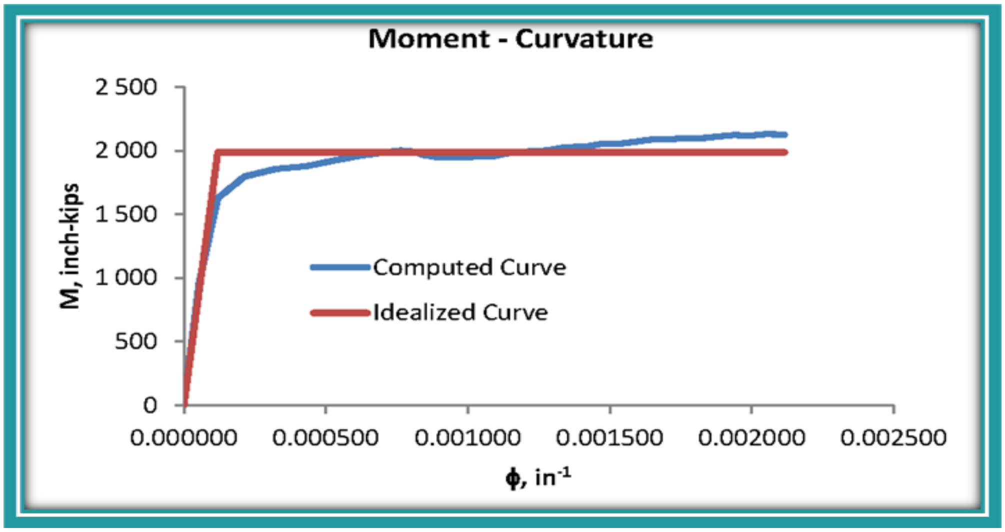

Figure 4. Moment Curvature Plot

14-302.00 2nd Order Effects

For tall slender structures, second-order effects shall be accounted for in the analysis. This may be done by further reducing the flexural stiffness of the columns for input into WinSeisab. For the transverse direction of multi-column bents (or any time rigid-frame behavior is appropriate), the adjustment is given by Equation 2.

$$EI_{EFF} = \frac{12EI_{CRACKED} - H^2 \sum P_{Bent}}{12} \tag{2}$$

For the transverse direction of single column bents and the longitudinal direction of single and multi-column bents (or any time cantilever-type behavior is appropriate), the adjustment is given by Equation 3.

$$EI_{EFF} = \frac{3EI_{CRACKED} - H^2 \sum P_{Bent}}{3} \tag{3}$$

ΣP_{Bent} is the sum of all vertical loads on the bent for the loading condition under consideration.

Both Leap Bridge Concrete Substructure and CSiBridge have the capability to directly include 2nd-order effects. Leap Bridge Concrete Substructure Moment Magnification and CSiBridge nonlinear analysis are both acceptable methods of accounting for 2nd-order effects in the structural analysis. In these cases, the adjustment above is not needed - the effect is included in the results from the software.

14-303.00 Member Forces

For SDC C and D, Guide Specification Section 8.3 requires that the member forces associated with over-strength plastic hinging be taken as the design basis for caps, footings, and all other capacity-protected elements expected to remain elastic during strong ground shaking. For SDC B, the smaller of (a) the over-strength plastic hinging forces and (b) the unreduced elastic seismic forces may be taken as the basis for the design of capacity-protected elements. It is good practice to use the over-strength plastic hinging forces, even when larger than the unreduced elastic seismic forces.

Bent diaphragms shall be capable of carrying the seismic forces to the substructure elastically. While the Guide Specification does permit the use of ductile superstructures, Tennessee bridges shall be designed with essentially elastic superstructures and ductile substructures unless the Director specifies otherwise.

Diaphragm capacities, whether constructed of steel or concrete, shall be checked. For concrete diaphragms, this check shall include the anchor capacity and a shear check of the diaphragm itself. For steel diaphragms, the members shall be capable of carrying the seismic forces elastically and without buckling. Again, anchors shall be designed to resist the seismic forces. Anchors shall be specified as ASTM F1554, Grade 36 (F_{ub} = 58 to 80 ksi), Grade 55 (F_{ub} = 75 to 95 ksi), or Grade 105 (F_{ub} = 125 to 150 ksi) as required by design. See Section 6.13.2.12 of the LRFD Specification for anchor shear capacity.

See Sections 6.8 and 6.9 of the LRFD Specification for determining tensile and compressive axial load capacities of steel diaphragm members.

The design seismic forces are applied to the substructure, and the caps, footings, piles, etc. are designed to remain elastic (with resistance factors, Φ, taken equal to 1.0 as specified in Guide Specification Section 3.7) under these design forces.

Section 4 Design Procedure Guidance

The essence of the seismic design of bridges for typical conditions includes ensuring each of the following.

- The displacement capacity exceeds the displacement demand from ground shaking,
- The inelastic behavior is limited to the intended elements (typically, columns), and
- All other elements are capacity-protected, i.e., elements not intended to behave in an inelastic manner do, in fact, remain elastic during ground shaking.

14-401.00 Design Displacement Demand

Bridge substructures are typically designed to displace laterally beyond yield. Historically, the 'equal-displacement-rule', which asserts that the displacements of yielding and non-yielding structures of equal period are the same, was applied to seismic design. Elastic response spectrum analysis was used to estimate inelastic displacements. For structures with short natural periods, the assumption that a yielding system will displace the same amount as a non-yielding system of the same initial stiffness is now known to be invalid. It is necessary to magnify the displacement demands from an elastic response spectrum analysis to determine the inelastic displacement demands. This is in the form of the R_d factor in Guide Specification Section 4.3.3. Since R_d is a function of structure period, there are different R_d values for each mode of vibration. Dynamic analysis displacements shall be amplified by R_d for all modes in which the period, T, is less than $T^* = 1.25(S_{D1} / S_{DS})$. Since each mode of vibration has a unique period, it may be most accurate to amplify the entire spectrum for all periods up to T*. Using the first mode R_d-factor for all modes is unconservative.

14-402.00 Design Displacement Capacity

For SDC B and C, displacement capacity for each substructure shall be determined from equations in Guide Specification Section 4.8.1 or from a static pushover analysis described in Guide Specification Section 4.8.2. For SDC D, displacement capacity for each substructure shall be determined from a static pushover analysis described in Guide Specification Section 4.8.2.

The yield and ultimate displacements may be approximated from Equations 4-17 to verify values obtained from computer modeling with CAPP, CSiBridge, or an alternate program approved by the Design Manager. Section C4.9 of the Guide Specifications contains similar equations, which may be useful.

It is important to compare the appropriate displacement demand and capacity values. If displacement demand is calculated at the center of gravity of the superstructure, then displacement capacity shall be calculated at the same location. Likewise, if displacement demand is calculated at the top of the column, then displacement capacity shall be calculated at the same location.

The following cases illustrate the use of various displacement capacity formulas.

- Case 1: A single hammerhead pier displacement capacity is calculated at the center of gravity of the superstructure. See Figure 5.

- Case 2: A multi-post bent displacement capacity is calculated at the top of the column. See Figure 6.

Subscripts “cm” and “tc” refer to “center of mass” and “top of column” respectively. Use $f=f_{cm}$ in Equation 4 to estimate yield displacement relative to the center of mass. Use $f=f_{tc}$ in Equation 4 to estimate yield displacement at the top of the column.

- d_{bl} is the longitudinal bar diameter, inches
- Φ_y is the yield curvature, in^{-1}
- Φ_u is the ultimate curvature, in^{-1}

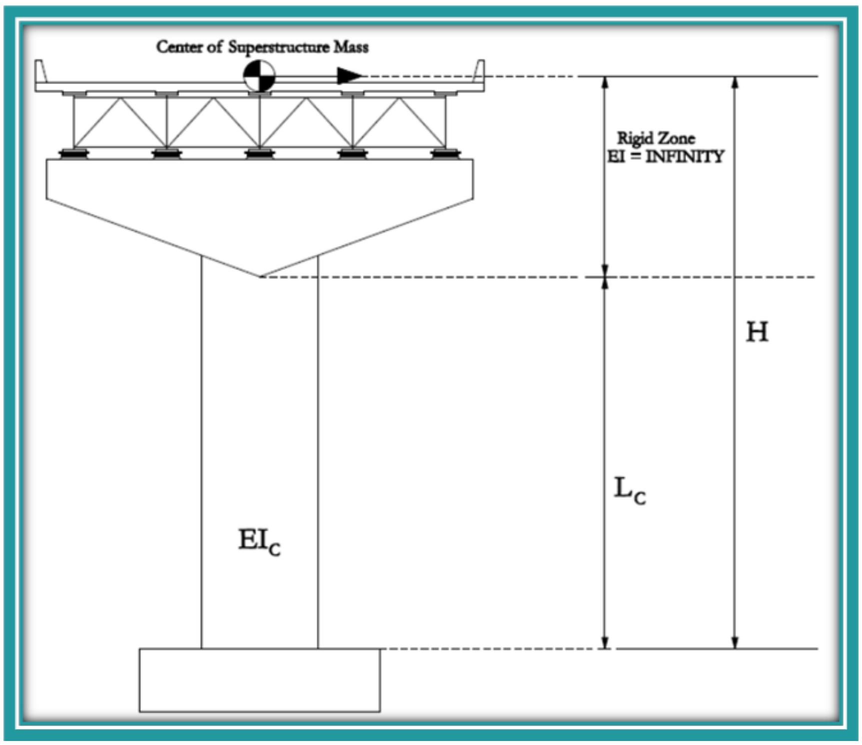

Figure 5. Case 1 Approximate Equations

Case 1 equations:

$$\Delta_y = \frac{1}{3}\phi_y L_c^2 f \tag{4}$$

$$f_{cm} = \frac{x^2 - 3x + 3}{x} \tag{5}$$

$$f_{tc} = \frac{3 - x}{2} \tag{6}$$

$$x = \frac{L_c}{H} \tag{7}$$

$$(\Delta_p)_{cm} = (\phi_u - \phi_y)L_p\left(H - \frac{L_p}{2}\right) \tag{8}$$

$$(\Delta_p)_{tc} = (\phi_u - \phi_y)L_p\left(L_c - \frac{L_p}{2}\right) \tag{9}$$

$$L_p = 0.08L_c + 0.15f_{ye}d_{bl} \geq 0.3f_{ye}d_{bl} \tag{10}$$

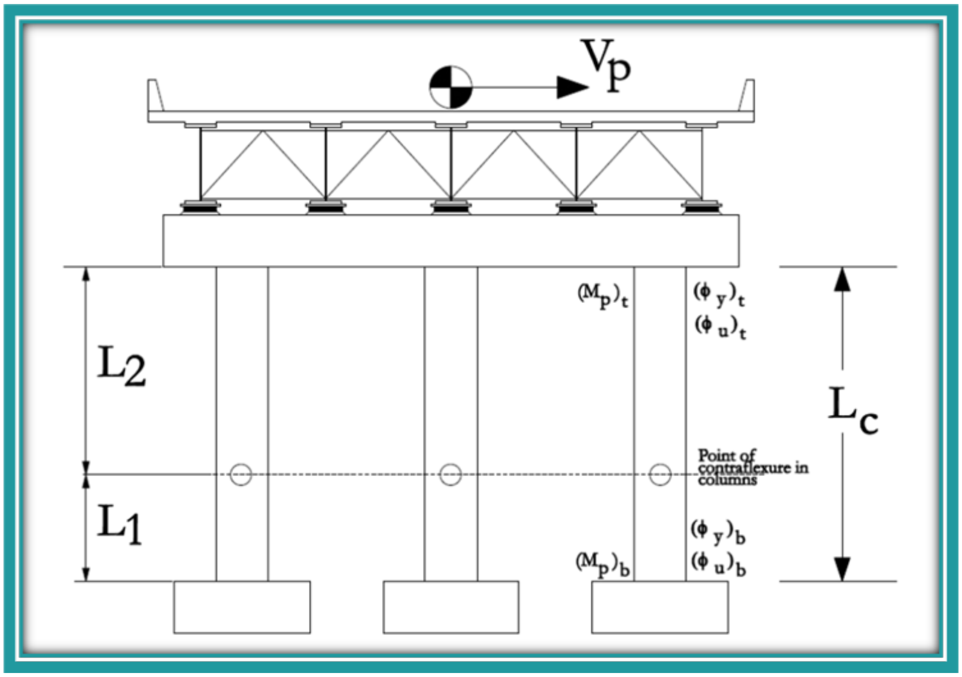

Figure 6. Case 2 Approximate Equations

Case 2 equations:

$$\Delta_y = \frac{1}{3}(\phi_{yb}L_1^2 + \phi_{yt}L_2^2) \tag{11}$$

$$L_1 = \frac{M_{pb}}{M_{pb} + M_{pt}} \cdot L_c \tag{12}$$

$$L_2 = L_c - L_1 \tag{13}$$

$$(\Delta_p)_{tc} = (\phi_{ub} - \phi_{yb})L_{pb}\left(L_1 - \frac{L_{pb}}{2}\right) + (\phi_{ut} - \phi_{yt})L_{pb}\left(L_2 - \frac{L_{pt}}{2}\right) \tag{14}$$

$$\Delta_u = \Delta_y + \Delta_p \tag{15}$$

$$L_{pb} = 0.08L_1 + 0.15f_{ye}d_{bl} \geq 0.3f_{ye}d_{bl} \tag{16}$$

$$L_{pt} = 0.08L_2 + 0.15f_{ye}d_{bl} \geq 0.3f_{ye}d_{bl} \qquad (17)$$

The difficulty is coming up with values for Φ_u and Φ_y consistent with the level of axial loading present, which may require an iterative approach. A suggested approach is outlined here. Additional guidance on yield and ultimate curvatures, Φ_y and Φ_u, is provided in Appendix C.

1. Use the average Extreme Event Limit State axial load in the columns and run a moment-curvature (M-Φ) analysis using CONSEC.
2. Use M_p from Step 1 to determine the plastic shear on the bent. Unless a hinged or semi-hinged column base is used, assume points of contra-flexure in the columns to be at mid-height.
3. Use the plastic shear in Step 2 to determine new axial loads in the columns.
4. Use the new axial loads in the columns to construct new M-Φ curves for the range of axial loads encountered.
5. Each column will now have its own value for M_p, so a new plastic shear can be computed.
6. If the two most recent plastic shears are not within 5% of one another, return to Step 3. Otherwise, you have determined the plastic shear for design. Take the smallest values for Φ_y (which will be the "windward" column) and Φ_u (which will be the "leeward" column) for use in the equations.

Once the displacement capacities in the bent transverse ("t") and bent longitudinal ("l") directions are known, they need be evaluated against the displacement demands, which are generally known with respect to the bridge transverse ("T") and bridge longitudinal ("L") directions. The assumed interaction failure surface may be taken as an ellipse and the check made using Equation 18.

$$\sqrt{\left(\frac{\Delta_l}{\Delta_{ul}}\right)^2 + \left(\frac{\Delta_t}{\Delta_{ut}}\right)^2} \leq 1.00 \qquad (18)$$

- Δ_l = the displacement demand, amplified by R_d, in the bent longitudinal direction
- Δ_{ul} = the displacement capacity in the bent longitudinal direction
- Δ_t = the displacement demand, amplified by R_d, in the bent transverse direction

- Δ_{ut} = the displacement capacity in the bent transverse direction

For each reported set of displacement demands (from WinSeisab, for example) two cases need to be considered:

$$\Delta_{t1} = \Delta_L sin\theta - \Delta_T cos\theta \quad (19)$$

$$\Delta_{l1} = \Delta_L cos\theta + \Delta_T sin\theta \quad (20)$$

$$\Delta_{t2} = \Delta_L sin\theta + \Delta_T cos\theta \quad (21)$$

$$\Delta_{l2} = \Delta_L cos\theta - \Delta_T sin\theta \quad (22)$$

- Δ_L = the displacement demand in the bridge longitudinal direction
- Δ_T = the displacement demand in the bridge transverse direction
- θ = the angle between the bent centerline and a line perpendicular to the bridge centerline

There will typically be at least 2 sets of reported demands: one for 100% Longitudinal + 30% Transverse and one for 30% Longitudinal + 100% Transverse. This means the displacement interaction equation needs to be evaluated 4 times.

The displacement demands, Δ_L and Δ_T, shall be modified according to the Guide Specification for structures (a) with other than 5% damping or (b) with short natural periods. These modifications are given in Sections 4.3.2 and 4.3.3 of the Guide Specification.

14-403.00 Displacement Capacity Check Example

The following example presents one rational method of magnifying the elastic response spectrum displacements. The more accurate method is to apply the period-dependent amplification factor, R_d, to the input response spectrum for the dynamic analysis. The method presented here at least accounts for differing amplification factors in each direction, something which Guide Specification Section 4.3.3 does not achieve. Suppose a bridge is located in SDC D with the following design spectrum data:

- As = 0.674 g
- S_{DS} = 1.213 g
- S_{D1} = 0.543 g

- $T_S = S_{D1}/S_{DS}$ = 0.448 seconds
- $T^* = 1.25\ T_S$ = 0.560 seconds

A response spectrum analysis using WinSeisab gives the vibration modes and elastic displacements shown in Figure 7.

VIBRATION CHARACTERISTICS

			PARTICIPATION FACTORS			% OF TOTAL MASS		
MODE	PERIOD	CS	Long	Vert	Tran	Long	Vert	Tran
1	0.562	0.97	0.000	0.000	0.000	0.000	0.000	0.000
2	0.427	1.21	0.329	0.000	0.027	0.079	0.000	0.001
3	0.414	1.21	1.922	0.000	-11.420	2.761	0.000	94.731
4	0.277	1.21	0.000	9.111	0.000	2.761	68.315	94.731
5	0.276	1.21	-11.476	0.000	-2.006	98.416	68.315	97.654
6	0.132	1.21	0.858	0.000	-1.243	98.952	68.315	98.776
7	0.108	1.21	-0.095	0.000	0.007	98.958	68.315	98.776
8	0.089	1.21	0.000	-1.720	0.000	98.958	70.748	98.776
9	0.068	1.11	0.000	0.000	0.000	98.958	70.748	98.776
10	0.056	1.03	0.152	0.000	-1.090	98.975	70.748	99.639
11	0.055	1.03	0.000	0.000	0.000	98.975	70.748	99.639
12	0.051	1.00	0.000	0.000	0.000	98.975	70.748	99.639
13	0.050	0.99	0.056	0.000	0.168	98.977	70.748	99.660
14	0.047	0.98	0.000	4.176	0.000	98.977	85.096	99.660
15	0.033	0.88	0.246	0.000	0.071	99.021	85.096	99.664
16	0.031	0.87	0.000	-1.346	0.000	99.021	86.587	99.664
17	0.028	0.85	-0.117	0.000	-0.011	99.031	86.587	99.664
18	0.025	0.83	-0.007	0.000	-0.015	99.031	86.587	99.664
19	0.023	0.82	-0.991	0.000	-0.136	99.744	86.587	99.677
20	0.022	0.81	0.000	1.422	0.000	99.744	88.251	99.677

BENT CQC DISPLACEMENTS

		....LEFT FACE....		....RGHT FACE....		...OPNNG/CLSNG...	
ITEM	LC	LNGTUDNL	TRNSVRSE	LNGTUDNL	TRNSVRSE	LNGTUDNL	TRNSVRSE
BNT 2	1	0.074	0.025	0.074	0.025	0.000	0.000
	2	0.029	0.116	0.029	0.116	0.000	0.000
	3	0.083	0.059	0.083	0.059	0.000	0.000
	4	0.051	0.123	0.051	0.123	0.000	0.000

Figure 7. WinSeisab Output

A pushover analysis produces the yield displacements in the transverse and longitudinal directions as follows:

- Δ_{yT} = 0.42 inches
- Δ_{yL} = 0.51 inches

Determine the appropriate displacement magnification factor, R_d in each direction:

Assume μ_D = 5. The process will be iterative since the magnifier is dependent on the ductility demand, which is dependent on the magnifier.

Calculate the transverse value for R_d. The fundamental period in the transverse direction is T = 0.414 seconds, with 94.7% of the mass participating. Since this is less than T*, magnification is required. Guide Specification equation 4.3.3-1 is used.

$$R_d = \left(1 - \frac{1}{\mu_D}\right) \cdot \frac{T^*}{T} + \frac{1}{\mu_D} = \left(1 - \frac{1}{5}\right) \cdot \frac{0.560}{0.414} + \frac{1}{5} = 1.282 \qquad (23)$$

Calculate the longitudinal value for R_d. The fundamental period in the longitudinal direction is 0.276 seconds with 98.4% of the mass participating. Again, since T is less than T*, magnification is required.

$$R_d = \left(1 - \frac{1}{\mu_D}\right) \cdot \frac{T^*}{T} + \frac{1}{\mu_D} = \left(1 - \frac{1}{5}\right) \cdot \frac{0.560}{0.276} + \frac{1}{5} = 1.823 \qquad (24)$$

Multiply the longitudinal earthquake displacements (Load Case 1 in WinSeisab) by 1.823 and multiply the transverse earthquake displacements (Load Case 2 in WinSeisab) by 1.282. Use these magnified displacements to determine Load Cases 3 (100% L + 30% T) and 4 (30% L + 100% T).

Load Case 1:

- Δ_L = 0.074 X 1.823 = 0.135 feet = 1.619 inches
- Δ_T = 0.025 X 1.823 = 0.046 feet = 0.547 inches

Load Case 2:

- Δ_L = 0.029 X 1.282 = 0.037 feet = 0.446 inches
- Δ_T = 0.116 X 1.282 = 0.149 feet = 1.785 inches

Load Case 3:

- Δ_L = 1.619 + 0.3(0.446) = 1.753 inches
- Δ_T = 0.547 + 0.3(1.785) = 1.083 inches

Load Case 4:

- Δ_L = 0.3(1.619) + 0.446 = 0.932 inches

- Δ_T = 0.3(0.547) + 1.785 = 1.949 inches

Determine the new value for μ_D. Use Guide Specification equation 4.9-5 in each direction independently and then combine using the square-root-of-the-sum-of-the-squares (SRSS). Load Case 3 is clearly more severe than Load Case 1, and Load Case 4 is clearly more severe than Load Case 2, so Load Cases 1 and 2 are omitted.

Load Case 3:

$$\mu_{DL} = 1 + \frac{1.753 - 0.510}{0.510} = 3.44 \tag{25}$$

$$\mu_{DT} = 1 + \frac{1.083 - 0.420}{0.420} = 2.58 \tag{26}$$

$$\mu_D = \sqrt{3.44^2 + 2.58^2} = 4.30 \tag{27}$$

Load Case 4:

$$\mu_{DL} = 1 + \frac{0.932 - 0.510}{0.510} = 1.83 \tag{28}$$

$$\mu_{DT} = 1 + \frac{1.949 - 0.420}{0.420} = 4.64 \tag{29}$$

$$\mu_D = \sqrt{1.83^2 + 4.64^2} = 4.99 \tag{30}$$

Since both μ_D values are less than the assumed value of 5, the displacement demands may be taken as those listed in Step 4 for Load Cases 3 and 4. Guide Specification Section 4.9 limitations on μ_D must be checked. A pushover analysis is still required to ensure that the ductility demands are reachable with the planned reinforcement details. If the calculated values of μ_D in step 5 are higher than the value assumed in Step 1, then iteration with a higher assumed value for μ_D is required. Also, if the calculated values are significantly lower than the assumed value, iteration will reduce the estimated demands.

14-404.00 Foundations

Spread footings and pile footings shall be initially designed for Strength Limit States. This design shall then adjusted (by changing dimensions, reinforcement, adding piles, etc.) to meet the Extreme Event I Limit State requirements.

The maximum pile displacement at abutments shall be checked.

In certain cases, it may be desirable to reduce loads into specific footings. This can be accomplished in at least two ways:

1. Selectively hinging certain column bases. For example, it may be desirable to hinge the bases of the center pier in a 4-span structure to minimize foundation sizes at that pier. The longitudinal and transverse forces must still be appropriately distributed to the other piers and the abutments.

2. The use of struts between columns at mid-height or lower. Leap Bridge Concrete Substructure has the capability to model and design struts. If struts are used, they shall be designed and detailed to meet the same requirements as the cap regarding the application of plastic hinging forces - they must be capacity protected.

At the Extreme Event I Limit State, spread footings and pile footings shall be evaluated for both shear and moment at the face of the column. At the Strength Limit State, the critical section for shear is typically taken at d_v from the face of the column. See references (Priestley, Seible and Calvi 1996) for a discussion of this requirement.

Joint shear requirements are applicable to bridges in SDC B, C, and D. See Section 14-405.00 for detailed joint shear requirements and Section 6.4.5 of the Guide Specification for footings in particular.

From the standpoint of joint shear stresses within a column-footing joint, turning the hooks of the column bars inward instead of outward in combination with hoops inside the joint is an effective means of providing adequate strength. When the hooks of the column bars are turned outward in the footing, reinforcement external to the joint in addition to column hoops within the joint may be required. See *Seismic Design and Retrofit of Bridges* (Priestley, Seible, and Calvi, 1996, p. 408-412) for a further discussion on joint shear in footings.

Refer to SDG 10 for further guidance on foundation design.

14-405.00 Joint Shear Design

For bridges in SDC A and B, joint design consists of satisfying provisions for extending column transverse reinforcement into caps, footings, and drilled shafts found in the LRFD Specification Section 5.11.4.3.

For bridges in SDC C and D, the Guide Specification provisions for joint shear design shall be used. Expected concrete strength shall be used to determine permissible principal stress levels. Joints shall be classified as one of the following:

- T joints
- knee joints
- Footing joints

Guide Specification joint shear design involves limiting the principal compressive and tensile stresses acting on the joint.

The science of properly reinforcing joints to provide adequate strength is somewhat inconsistent. Different reinforcing patterns will be established depending on the criteria used. Each of the following documents presents a discussion of joint shear design principles:

- AASHTO Guide Specification for LRFD Seismic Bridge Design.
- CALTRANS Seismic Design Criteria, version 1.4, June, 2006.
- South Carolina Seismic Design Specifications for Highway Bridges, October, 2002.

Provisions in each of these are for T joints only. Trying to apply the procedures to knee joints may produce invalid results.

Additional guidance on both T joint and knee joint design may be found in the literature (Priestley, Seible and Calvi 1996), (S. Sritharan 2005).

While the Guide Specification, in Section C8.13.5, references the report by Sritharan as the basis for joint shear design in non-integral bent caps, the Guide Specification procedures outlined are quite different than those presented by Sritharan.

In certain cases for which joint shear reinforcing is required and the joint becomes congested, it may be worthwhile to consider specifying higher strength concrete for the cap beam, such as

4,000 psi or higher. This change alone or along with member dimensional changes can be an effective means of providing sufficient joint strength.

Specific procedures for T joints and knee joints are covered in Sections 14-405.01 and 14-405.02, respectively.

14-405.01 Bent Cap T Joints

A bent cap T joint is shown in Figure 8. For bridges in SDC C and D, T joints shall be sized so that Guide Specification Section 8.13.2 is satisfied (principal compression = p_c; principal tension = p_t).

$$p_c \leq 0.25 f_c' \tag{31}$$

$$p_t \leq 0.38\sqrt{f_c'} \tag{32}$$

If either of these criteria is not met, then the member size shall be increased – either the cap or the column or both – until the limits are met. Typically, it is preferable to increase only the cap dimensions since changing the column dimensions would require recalculating the plastic shear and the displacement capacity.

Whether or not special joint reinforcing is required depends on the principal tension stress level. Likewise, the area of confining steel within the joint depends on the principal tension. Provisions of Guide Specification Section 8.13.5.1 shall be applied to T joint design. These equations are applicable for ASTM A615 reinforcing steel. If ASTM A706 steel is specified, the required values may be multiplied by the ratio 1.2/1.4.

Vertical stirrups outside the joint region shall be distributed over a distance equal to half the beam height, h_b, from the column face. The added bottom reinforcement shall be capable of developing yield at h_b / 2 from the column faces.

Joint equilibrium equations (see Figure 8) are used to estimate principal stresses.

$$\sum F_x = 0: P_{bl} - P_{br} = V_c - V_{ss} \tag{33}$$

$$\sum F_y = 0: V_{bl} - V_{br} = P_c - P_{ss} \tag{34}$$

$$\sum M_o = 0: M_{br} + M_{bl} = M_c + M_{ss} + (V_c + V_{ss})\frac{h_b}{2} - (V_{br} + V_{bl})\frac{h_c}{2} \tag{35}$$

From Equation 34:

$$V_{br} + V_{bl} = 2V_{br} + P_c - P_{ss} \tag{36}$$

Substitute this into Equation 35:

$$M_{br} + M_{bl} = M_c + M_{ss} + (V_c + V_{ss})\frac{h_b}{2} - (2V_{br} + P_c - P_{ss})\frac{h_c}{2} \tag{37}$$

Under gravity loads, V_{br} is actually downward instead of upward as shown. Once lateral loads are applied, the downward V_{br} becomes smaller and smaller and may go upward as shown if the loads are large enough. Since the effect of an upward V_{br} is to reduce the moment sum M_{bl} + M_{br}, take $V_{br} = 0$ to estimate the joint shear.

The sum of the positive and negative moment capacities in the cap must be greater than the value from the above equation. Otherwise, additional longitudinal cap reinforcement shall be added in the top, bottom, or both. The column moment, shear, and axial load shall be taken as the over-strength values.

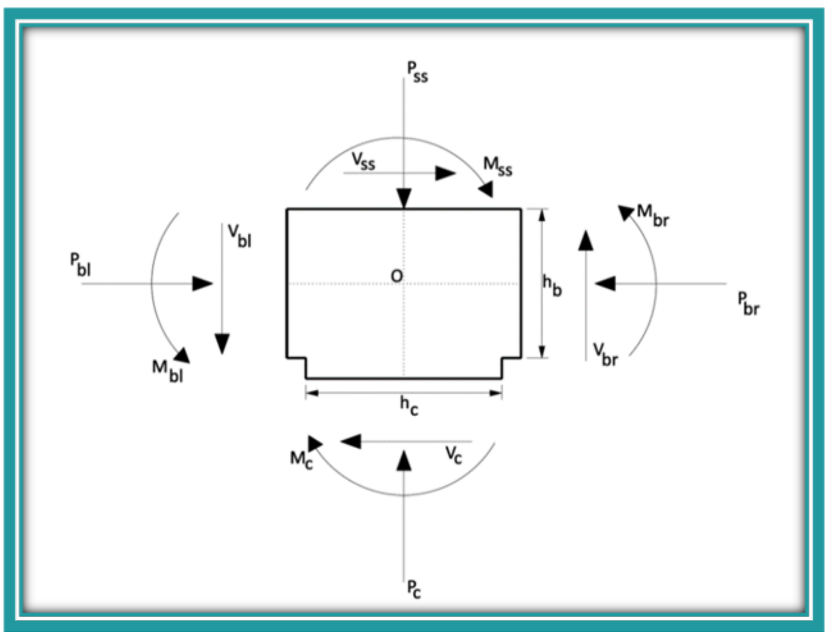

Figure 8. T Joint Equilibrium

14-405.02 Bent Cap Knee Joints

A knee joint has a beam framing into the column on one side only and must be designed for both "opening" (Figure 9 with Equations 38-40) and "closing" (Figure 10 with Equations 41-43) conditions for bridges in SDC C and D. Principal stress values are limited as for T joints. Both "opening" and "closing" conditions shall be examined.

$$\sum F_x = 0: P_b = V_c - V_{ss} \tag{38}$$

$$\sum F_y = 0: V_b = P_c - P_{ss} \tag{39}$$

$$\sum M_o = 0: M_b = M_c + M_{ss} + (V_c + V_{ss})\frac{h_b}{2} - (V_b)\frac{h_c}{2} \tag{40}$$

$$\sum F_x = 0: P_b = V_c - V_{ss} \tag{41}$$

$$\sum F_y = 0: V_b = P_{ss} - P_c \tag{42}$$

$$\sum M_o = 0: M_b = M_c + M_{ss} + (V_c + V_{ss})\frac{h_b}{2} - (V_b)\frac{h_c}{2} \tag{43}$$

The column moment, shear, and axial load shall be taken as the over-strength values. The superstructure moment, shear, and axial load shall be taken as the unfactored values. If there are no bearings within the joint area, the superstructure values are all zero. If the beam shear and moment under this condition are greater than the Strength Limit State values, additional top and bottom longitudinal reinforcement and stirrups are required in the cap. The calculated values shall also be used to find the joint stresses.

14-405.03 Principal Stress Equations

To calculate the joint shear stresses (f_v and f_h) and the principal stresses (p_c and p_t) for a knee joint or a T joint, use Equations 44-50.

The effective joint width, b_{je} is the smaller of:

- the cap width, b_b, and $\sqrt{2}$ x D, for circular columns of diameter D, or
- the cap width, b_b, and (h_c + b_c) for rectangular columns with dimensions h_c x b_c

The effective joint area equation above permits spread of the joint at 45^o up to mid-depth of the cap. For knee joints with a cantilever on one side, this same spread is permitted, and the equation may be adjusted to account for the higher effective area.

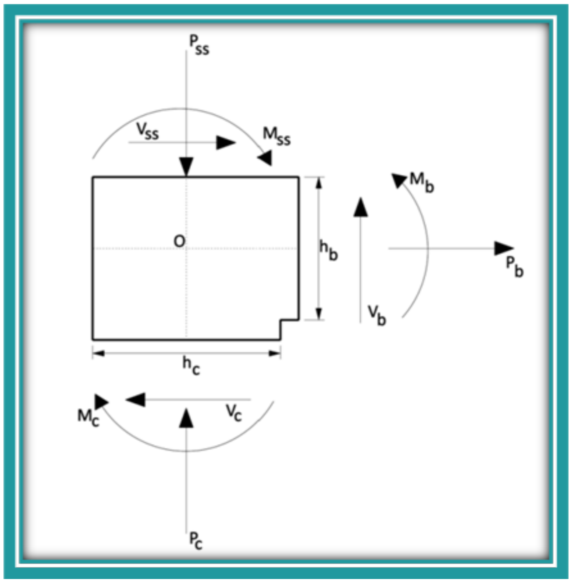

Figure 9. Knee Joint Opening Condition

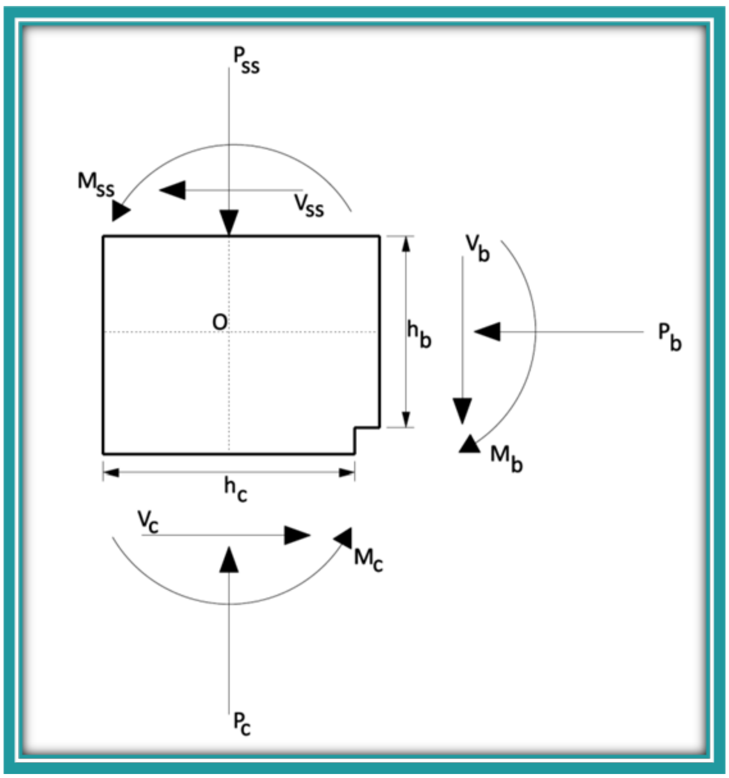

Figure 10. Knee Joint Closing Condition

$$v_{jh} = v_{jv} = \frac{M_{po}}{h_c h_b b_{je}} \tag{44}$$

$$f_v = \frac{P_c}{A_{jh}} \tag{45}$$

$$f_h = \frac{P_b}{b_b h_b} \tag{46}$$

$$A_{jh} = \left(h_c + \frac{h_b}{2}\right) b_b - Knee\ joints \tag{47}$$

$$A_{jh} = (h_c + h_b) b_b - T\ joints \tag{48}$$

$$p_t = \frac{f_h + f_v}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jh}^2} \tag{49}$$

$$p_c = \frac{f_h + f_v}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jh}^2} \tag{50}$$

Appendix A: Ground Motion Selection and Modification

In some cases, it will be necessary to use nonlinear time history analyses to finalize bridge designs which either (a) are critical structures or (b) incorporate non-traditional mechanisms such as seismic isolation.

Ground motion selection shall be accomplished using the following criteria:

1. No fewer than 11 record pairs (two perpendicular horizontal components) shall be used.

2. The target response spectrum shall correspond to uniform hazard, geometric-mean-based, 1000-year mean recurrence interval ground shaking appropriate for the site class for the project.

3. The target log-based standard deviation shall be 0.60 across all periods of interest.

4. The period range of interest shall be defined by T_{LOWER} and T_{UPPER}. T_{LOWER} is the smaller of (a) the period required to obtain 90% mass participation and (b) 0.2 times the smaller of the two fundamental periods (in the two horizontal directions). T_{UPPER} is 2.5 times the larger of the two fundamental periods.

5. Only ground motions reasonably consistent with the site class and characteristic magnitudes at the project site shall be included in the suite of 11 record pairs.

6. Ground motion pairs shall be amplitude-scaled such that the average suite spectrum at all periods in the period range of interest satisfies: (a) the suite average does not fall below the target by more than 10% at any period in the range, (b) the suite average does not exceed the target by more than 30% at any period in the range, (c) the average, over the entire range of periods, for the suite mean-to-target ratio shall exceed 1.0 (preferably only slightly), and (d) the log-based variability of the suite reasonably matches the target log-based variability.

7. Seismic hazard at the site shall be performed to identify the modal, mean, and other significant earthquake magnitudes and corresponding distances contributing to the site hazard. USGS web applications shall be used for the deaggregation.

Appendix B: Alternate Site Factors for Deep Soil Sites

AASHTO site factors, F_{PGA}, F_a, and F_v, used to develop design response spectra are based on average shear wave velocity in the upper 100 feet of the soil profile. The Mississippi Embayment consists of profiles 3,300 feet and more in thickness, and research has shown that the generic site factors may be unconservative at periods of 1-second and larger. Higher amplification at long periods and lower amplification at short periods can be expected based on research on the Mississippi Embayment. Three sources for alternative site factors for critical structures are given here. Source 3 is the most recent.

1. Hashash, Y. M., Tsai, C.-C., Phillips, C., & Park, D. (2008). Soil-Column Depth-Dependent Seismic Site Coefficients and Hazard Maps for the Upper Mississippi Embayment. Bulletin of the Seismological Society of America, 98(4), 2004-2021.
2. Malekmohammadi, M., & Pezeshk, S. (2014). Nonlinear Site Amplification Factors for Sites Located within the Mississippi Embayment with Consideration for Deep Soil Deposit. Earthquake Spectra.
3. Moon, Sung-woo; Hashash, Youssef M. A.; Park, Duhee. (2016). USGS Hazard Map Compatible Depth-Dependent Seismic Site Coefficients for the Upper Mississippi Embayment, Korean Society of Civil Engineers, KSCE Journal of Civil Engineering, pp. 1-12.

Appendix C: Yield and Ultimate Curvature of Reinforced Concrete

For a given column size and transverse reinforcement, the yield and ultimate curvatures for displacement capacity of the column may be estimated using the following procedure (Priestley, Calvi and Kowalsky 2007).

A_g: gross concrete area of a column

A_v: area of transverse hoop or rectangular stirrup

c: depth from extreme compression fiber to neutral axis

C_e: confinement effectiveness coefficient, 1.0 for circular, 0.75-0.85 for rectangular

d: depth from extreme tensile reinforcement to the extreme compression fiber

d_{bl}: diameter of longitudinal reinforcing bars

D: column diameter

D': column core diameter measured to the centerlines of transverse bars

f_l: lateral confining stress on concrete core from lateral bars

f_{ye}: expected yield strength of longitudinal reinforcement

f_{yh}: yield strength of transverse reinforcing bars

f'_{cc}: compressive strength of confined concrete core

f'_{ce}: expected unconfined concrete strength, usually $1.3f'_c$

h_c: core dimension perpendicular to a plane, measured to centerline of transverse bars

L_P: plastic hinge length

L_{SP}: strain penetration length

n: number of ties crossing a shear plane

P_u: axial load on a column

e_{SU}: ultimate tensile strain of transverse bars

$e_{dc,c}$: damage-control compression strain in concrete

$e_{dc,s}$: damage-control tension strain in reinforcing, typically e_{SU} for current bridge design

e_{y}: yield strain of longitudinal reinforcing bars

f_{y}: yield curvature

f_{u}: ultimate curvature; the smaller of $f_{dc,c}$ and $f_{dc,s}$

$$L_{SP} = 0.15 f_{ye} d_{bl} \tag{51}$$

$$k = 0.2\left(\frac{f_u}{f_y} - 1\right) \leq 0.08 \tag{52}$$

$$L_P = kL_C + L_{SP} \geq 2L_{SP} \tag{53}$$

$$\rho_v = \frac{4A_v}{D's}\ , Circular\ hoops\ and\ spirals \tag{54}$$

$$\rho_v = \frac{n_x A_v}{h_{cy} s} + \frac{n_y A_v}{h_{cx} s}\ , Rectangular\ ties \tag{55}$$

$$f_l = 0.5 C_e \rho_v f_{yh} \tag{56}$$

$$f'_{cc} = f'_c\left(2.254\sqrt{1 + \frac{7.94 f_l}{f'_c}} - 2\frac{f_l}{f'_c} - 1.254\right) \tag{57}$$

$$\varepsilon_{dc,c} = 0.004 + 1.4\frac{\rho_v f_{yh} \varepsilon_{su}}{f'_{cc}} \tag{58}$$

$$\frac{c}{D} = 0.20 + 0.65\frac{P_u}{f'_{ce} A_g} \tag{59}$$

$$\phi_{dc,c} = \frac{\varepsilon_{dc,c}}{c} \tag{60}$$

$$\phi_{dc,s} = \frac{\varepsilon_{dc,s}}{d - c} \tag{61}$$

$$\phi_y = 2.25\frac{\varepsilon_y}{D} \tag{62}$$

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