SM0 55: Seismic Design of Bridges

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1 Design Specifications

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

1. AASHTO LRFD Bridge Design Specifications (AASHTO 2014)
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. 206)

The Guide Specifications for LRFD Seismic Bridge Design (the “Guide Specification” hereafter) require a displacement-based design approach and 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 SMO does not address seismic retrofit. Refer to the FHWA manual for retrofit projects.

1.1 Design Strategies

Three seismic design strategies are available. The seismic design strategy should be determined at the preliminary design phase and the earthquake resisting system (ERS) and elements (ERE) established. The three strategies are defined in Guide Specification Section 3.3.

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. Type 2 and Type 3 design strategies may be effective solutions for very stiff substructures or where it is otherwise advantageous to maintain essentially elastic behavior in the substructures.

Type 1 Bridges in Seismic Zone 1 - Seismic Design Category (SDC) A - may be designed in accordance with either the LRFD Specification or the Guide Specification. Type 1 Bridges in Seismic Zones 2, 3, and 4 - SDC B, C, and D - shall be designed in accordance with the Guide Specification. Type 3 bridges in any Seismic Deign Category shall be designed using the Isolation Specification. Bridges designed using the Type 2 strategy will require the design procedures from UNR Report No. CCEER-13-15 (Itani, et al. 2013) in combination with Guide Specification Sections 7.4.6 and 7.4.7.

Figure 3.3-3 of the Guide Specification identifies features which should 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 Owner's approval and includes Type 2 bridges. Many bridges in Tennessee are designed with integral abutments. According to Figure 3.3-1b, 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, and it is permissible with Owner's approval if 100% of the soil strength is relied upon. Use 70% of the Guide Specification soil strength for integral abutment bridges in which passive resistance is established as an ERE.

1.2 Seismic Loading

The computer program from AASHTO, GM-2.1, shall be used to determine the spectral accelerations (PGA, S_S, S_1, A_S, S_{D1}, S_{DS}) corresponding to a 7% probability of exceedance in 75 years for developing the design response spectrum. Define locations by latitude and longitude, which may be obtained from county maps or Google Earth.

For regular bridges, the earthquake loading is defined in terms of an acceleration response spectrum only. For critical or irregular 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 inelastic response 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 Section 5 of this SMO contain criteria for the selection and modification of ground motion records.

Vertical effects need to be included in the response spectrum analysis of the bridge whenever the site is located within 6 miles of an active fault (Guide Specification Section 4.7.2). Determine the fault distance by a hazard deaggregation for the Site (USGS 2011).

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

1.3 Design Procedures

Displacement-based design in accordance with the Guide Specification includes the following steps:

1. A pushover analysis of the structure is used to determine displacement capacity. Plastic hinges are formed until a collapse mechanism is reached.
2. A dynamic response spectrum analysis of the structure determines the seismic displacement demand.
3. Once a configuration with sufficient displacement capacity is attained, measures are taken to ensure that the cap and footing (and any other non-hinging elements, such as struts) stay elastic and meet the Guide Specification requirements for the plastic shear condition at each pier.
4. Provide member sizes such that joint stresses (cap-column, strut-column, and footing-column joints) are within acceptable limits.
5. Ensure sufficient transverse reinforcement in the form of hoops or spirals is used to satisfy confinement and shear requirements for the columns.
6. Ensure that sufficient seat widths are provided at expansion bearing locations.

Force-based design in accordance with the LRFD Specification includes:

1. A dynamic response spectrum analysis of the structure determines elastic seismic force demands.

2. The elastic demand forces are reduced by appropriate factors to obtain seismic design forces.
3. Design the reinforcement in each element of the structure needed to carry the design forces.
4. Ensure that sufficient transverse reinforcement in the form of hoops or spirals is specified to satisfy confinement and shear requirements.
5. Implicit displacement capacity check for structures in SDC B and higher
6. Ensuring that sufficient seat widths are provided at expansion bearing locations.

The Extreme Event Load Combinations from the LRFD Specification are required for determining structure response regardless of which design method – displacement-based or force-based – is used. Use γ_{EQ} = 0.50 for the Extreme Event Load Combination of the LRFD Specification Table 3.4.1-1.

Shear resistance of anchor rods shall be determined in accordance with the LRFD Specification, Section 6.13.2.12. Anchor rod material specification should typically be ASTM F 1554, Grade 36, 50, or 105 as required. See Section 6.4.3.1 of the LRFD Specification.

A summary of requirements for each Seismic Design Category is provided in the following sections. Refer to the Guide Specification, Section 3.5, as well. Flowcharts are available beginning with Guide Specification Table 1.3-1a, for displacement-based design, and in the LRFD Specification, Appendix A3 - Seismic Design Flowcharts, for force-based design.

Table 1 of this SMO provides a summary of seismic design requirement Specification Sections for both force-based and displacement-based bridge design.

1.3.1 Seismic Design Category A (LRFD Specification Option)

No dynamic analysis is required to determine seismic demands. For bridges with $S_{D1}<0.05$, the design connection force is equal to 15% of the tributary permanent load and that portion of the tributary live load assumed to be active during strong ground shaking (typically, the 0.64 klf HL-93 lane load in ½ of the actual lanes over the entire bridge length).

For bridges with $S_{D1} \geq 0.05$, the design connection force is increased from 15% to 25% of the vertical load. These requirements are from the LRFD Specification, Sections 3.10.9.2 and 4.7.4.3.

The connection design force is to 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.

LRFD Specification Section 4.7.4.4 defines the minimum support length requirements as a function of bridge geometry.

While the LRFD Specification is a force-based approach, displacement estimates under the design force need to 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.

Transverse reinforcement requirements in the top and bottom of columns shall be as required by LRFD Specification Sections 5.10.11.2, 5.10.11.4.1d and 5.10.11.4.1e.

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

1.3.2 Seismic Design Category A (Guide Specification Option)

No dynamic analysis is required to determine seismic demands. For bridges with $S_{D1}<0.05$, the design connection force is equal to 15% of the tributary permanent load and that portion of the tributary live load assumed to be active during strong ground shaking (typically, a 0.64 klf HL-93 lane load in ½ of the actual lanes over the entire bridge length). For bridges with $S_{D1} \geq 0.05$, the design connection force is increased from 15% to 25% of the vertical load. These requirements are from the Guide Specification, Section 4.6.

The connection design force is to 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.

Guide Specification Section 4.12 defines the minimum support length requirements as a function of bridge geometry.

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.

1.3.3 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 an inelastic response history analysis may be required. The elastic, response spectrum displacements are magnified 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. Guide Specification Sections 4.3.2 and 4.3.3 define the required magnifications.

Displacement capacity for each substructure is determined form equations in Guide Specification Section 4.8.1.

Joint shear checks are not required for Seismic Design Category 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 for determining plastic hinging forces.

Required seat lengths at expansion locations are determined from Guide Specification Section 4.12.2 based on the structure geometry.

The AASHTO LRFD Specifications, in Section 6.16, provide design requirements for steel girder bridges in Seismic Design Category B which are to be implemented to complement the requirements from the AASHTO Guide Specification, Section 7.4.7, for these structures. The provisions include requirements for deck shear and shear connectors.

1.3.4 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 an inelastic response history analysis may be required. The elastic, response spectrum displacements are magnified 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. Guide Specification Sections 4.3.2 and 4.3.3 define the required magnifications.

Displacement capacity for each substructure is determined form equations in Guide Specification Section 4.8.1.

Joint shear checks in accordance with Guide Specification Section 8.13 are required for Seismic Design Category 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 for determining plastic hinging forces.

Required seat lengths at expansion locations are determined from Guide Specification Section 4.12.2 based on the structure geometry.

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

The AASHTO LRFD Specifications, in Section 6.16, provide design requirements for steel girder bridges in Seismic Design Category C which are to be implemented to complement the requirements from the AASHTO 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 girder, there is no load path to the foundations for this type of bridge.

1.3.5 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 Seismic Design Category D bridges.

Explicit calculation of strength for passive resistance at abutment back-walls is required for SDC D bridges when the back-walls 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 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 an inelastic response history analysis may be required. The elastic, response spectrum displacements are magnified 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. Guide Specification Sections 4.3.2 and 4.3.3 define the required magnifications.

Displacement capacity is determined from a static pushover analysis described in Guide Specification Section 4.8.2 rather than from implicit equations.

Displacement ductility values must 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 Seismic Design Category 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 for determining plastic hinging forces.

Required seat length requirements at expansion bearings are to be determined using the actual displacement demands form the dynamic analysis as defined in Guide Specification Section 4.12.3.

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

The Guide Specification in Section 8.4.1 requires the use of A706 reinforcing for SDC D structures in areas where hinging is expected. A 706 reinforcing has both a cap on yield strength and a higher ultimate strain than A 615 reinforcing, and usually provides for a subsequent higher displacement capacity when used in hinging members. Permit the use of either of the following for column longitudinal bars:

- A615 reinforcing with a maximum yield strength of 78 ksi, or
- A 706 reinforcing

This means that a lower over-strength factor, λ_{mo}, of 1.2 may be used, but that the reduced ultimate tensile strain, ε^{R}_{su}, corresponding to A 615 steel must also be used in the moment curvature analysis. The overall effect will be a reduced over-strength plastic shear, but without the added displacement capacity possible using A 706 transverse steel. (See Sections 8.4.2 and 8.5 of the Guide Specification).

The AASHTO LRFD Specifications, in Section 6.16, provide design requirements for steel girder bridges in Seismic Design Category D which are to be implemented to complement the requirements from the AASHTO 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 girder, there is no load path to the foundations for this type of bridge.

1.4 Non-traditional Design Options

Among the options available to the engineer for the design of bridges in the New Madrid Seismic Zone when traditional Type 1 design becomes economically disadvantageous are:

- 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 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.

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

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	5.10.11	Section 8
Concrete Piles – Seismic Requirements	5.13.4.6	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	
Elastomeric Bearings - Method B	14.7.5.3.7	
Elastomeric Bearings - Method A	14.7.6.3.8	
Anchorage and Anchor Bolts	14.8.3.2	
Sound Barrier Foundations	15.9.9	

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 30 meters of the foundation profile. The depth to bedrock should also be estimated.

Section 2.1 of this SMO summarizes the method used in averaging shear wave velocities. Maps are available in Section 2.2 of this SMO to assist in estimating site characterization parameters. When the depth to bedrock is significantly greater than 30 meters, 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. See also Section 6 of this SMO.

2.1 Site Class Definition by Average Shear Wave Velocity

Site characterization from the Guide Specification requires some knowledge of the geo-technical 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) among 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.

Table 2. Site Correlations (approximate)

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

When no data is available, it is still possible to estimate the site shear wave velocity, V_{S30}, from latitude and longitude along with the OpenSHA Site data application (http://www.opensha.org/apps).

1. Select “Launch” on the Site Data Viewer/Plotter
2. Select “Global VS30 from Topographic Slope”
3. Change Region Type to “Stable Continent”
4. Enter the Site Latitude (“+” for North latitudes) and Longitude (“-” for West longitudes)
5. Select “View Preferred Data”

The reported shear wave velocity is in units of meters/second.

2.2 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).

Seismic Design Category designations are based on the 1-second period design response spectrum accelerations for the 7% probability of exceedance in 75 years ground motion. Table 3.5.1 of the Guide Specification is reproduced here in Table 3.

Table 3. Seismic Design Category (SDC) Criteria

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

2.3 Embayment Depth

The Mississippi Embayment is unique in that profile depths over 1km exist over intra-plate faults. Profiles characterized as "Lowlands" in the literature have slightly lower shear wave velocities in the upper 80 meters and similar velocity profiles beyond 80 meters compared to "Uplands" profiles. The deep soil sites may deserve special attention to spectrum definition and the code-based response spectrum may be un-conservative 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 here 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).

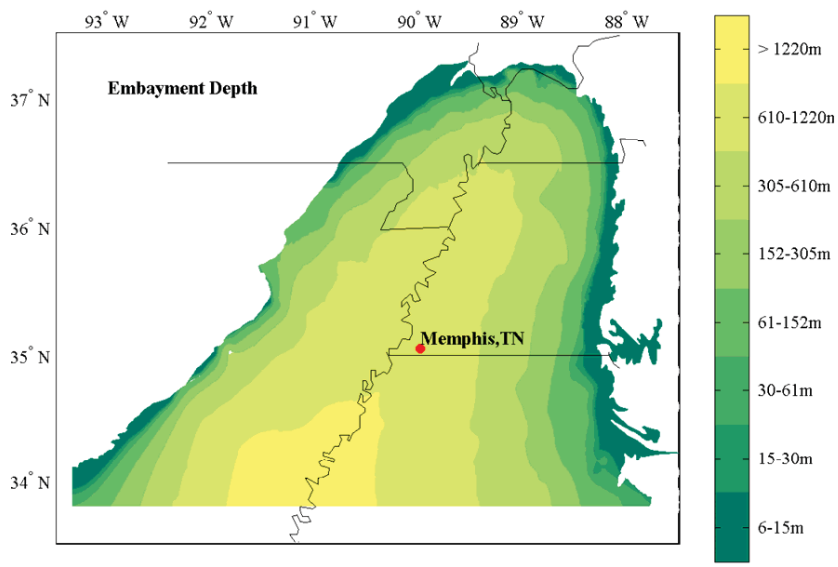

Figure 1. Mississippi Embayment Depth (1)

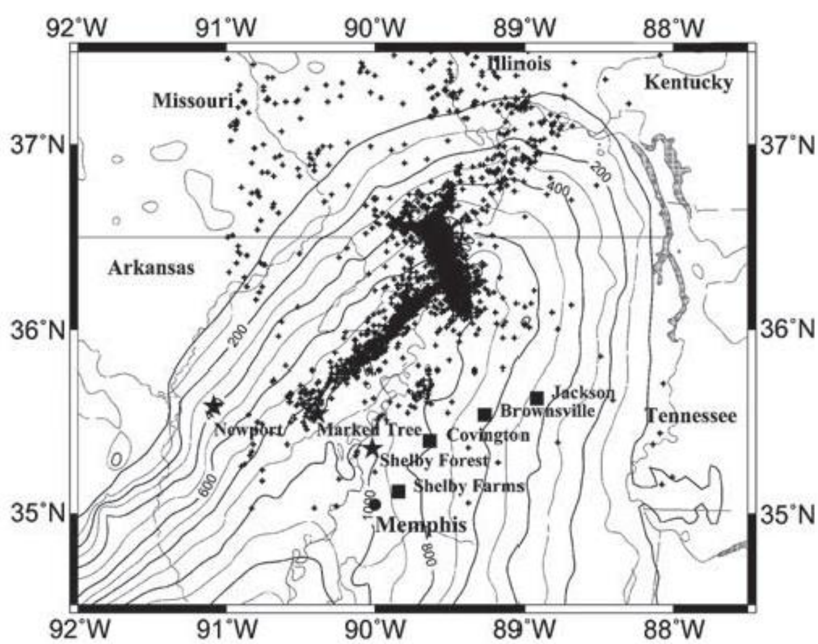

Figure 2. Mississippi Embayment Depth (2)

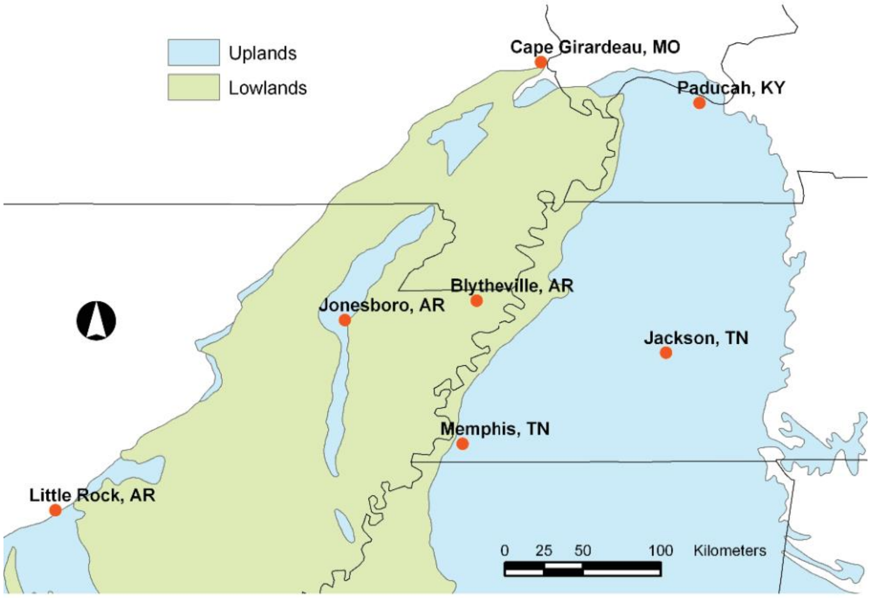

Figure 3. Mississippi Embayment - Uplands and Lowlands

3 Modeling and Analysis

For both force-based design and displacement-based design, determine seismic demand using a dynamic multi-mode response spectrum analysis of the structure. WINSEISAB should be used for the majority of bridges. For very complex structures CSiBridge should be considered. In exceptional cases, response history analysis for a series of ground motion records may be required. Refer to Section 5 of this SMO 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 3.3 of this SMO.

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, and overlay in addition to the self-mass of the structural components.

Typically, include a 0.64 klf live lane load over the entire bridge length, in ½ of the actual lanes, as additional mass for the dynamic analysis.

Effective member properties are needed 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).

3.1 Moment-Curvature Analysis

Moment-curvature (M-φ) analysis is required for pushover analysis. In most situations, use CONSEC or CSiBridge to do M-φ analyses. The results of the analysis are:

- Yield curvature
- Ultimate curvature
- Ultimate moment

The curvatures are used to estimate displacement and displacement ductility capacities while the moment is used in calculating plastic shears and in determining loads to caps, struts, and footings. Yield and ultimate curvatures may also be estimated using the ***Appendix*** of this SMO.

Expected material properties shall be used for M-φ analysis. Section 8.4.4 of the Guide Specification requires that the concrete strength be taken as 1.3f'c. Since we usually specify 3,000 psi concrete for our substructures, use 3,900 psi for the expected concrete strength unless the specified 28-day compressive strength for 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 should be considered.

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".

When displacement-based design is used, the appropriate software should be used to model moment-curvature behavior of the cross-section required to hinge. The beneficial effects of confining steel on the stress-strain properties of the concrete core should be included in the analysis so make certain that the software has this capability. CONSEC may be used to model this behavior.

Cracked section properties should 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.

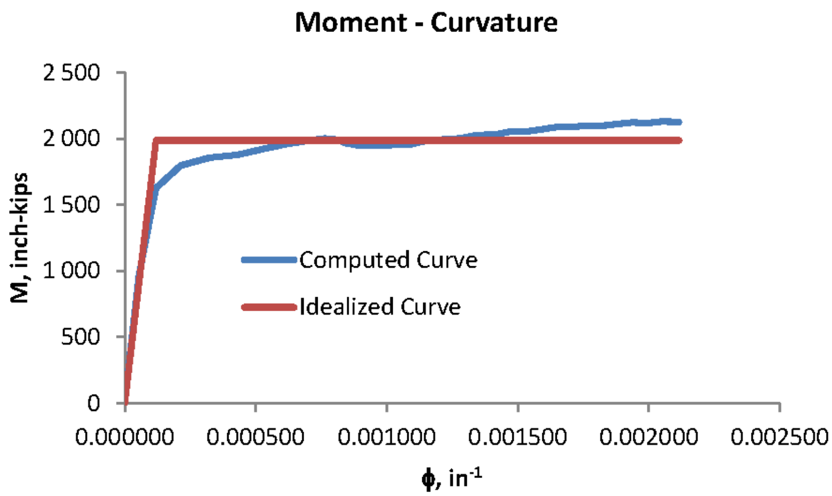

Figure 4. Moment-Curvature Plot

3.2 2nd Order Effects

For tall slender structures, second-order effects should be accounted for. This may be done by further reducing the flexural stiffness of the columns for input into WINSEISAB. For the transverse direction (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 longitudinal direction (or any time cantilever-type behavior is appropriate), the adjustment is according to 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.

Once $(EI)_{EFF}$ is obtained, the ratio of effective to gross $(EI)_{EFF}$ can be used in WINSEISAB. In WINSEISAB, I_{33} is taken about the transverse and I_{22} is taken about the longitudinal axis.

Both CSiBridge and RC-PIER have the capability to directly include 2nd-order effects. CSiBridge nonlinear analysis, RC-PIER P-Delta analysis, and RC-PIER Moment Magnification are all acceptable methods of accounting for 2nd-order effects in the structural analysis. In these cases this adjustment is above is not needed - the effect is included in the results from the software. The need for this adjustment is when WINSEISAB is used to determine earthquake loads to be used for input to CSiBridge or RC-PIER.

3.3 Member Forces

For SDC C and D, Guide Specification Section 8.3 requires that the member forces associated with over-strength plastic hinging be takes 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 un-reduced elastic seismic forces may be takes 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 un-reduced elastic forces.

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

Diaphragm capacities, whether constructed of steel or concrete, should be checked. For concrete diaphragms, this check should include the anchor bolt capacity and a shear check of the diaphragm itself. For steel Z-frame, X-frame, or K-frame diaphragms, the members should be capable of carrying the seismic loads elastically and without buckling. Again, anchors should be designed for elastic seismic loads. Response modification factors are to be taken as 0.8 at Abutments or expansion joint locations and 1.0 elsewhere. See Section 3.10.7.1 of the LRFD Specification.

Anchors should be specified as ASTM F 1554, 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.

4 Design Procedure Guidance

The essence of the seismic design of bridges for the 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.

4.1 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 need to 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 un-conservative.

$$R_d = \left(1 - \frac{1}{\mu_D}\right) \cdot \frac{T^*}{T} + \frac{1}{\mu_D} \geq 1.0 \qquad (4)$$

4.2 Design Displacement Capacity

For SDC B and C, implicit displacement capacity equations from Guide Specification Section 4.8.1 may be used. If the displacement capacity fails using the implicit equations, then a detailed displacement capacity analysis is required. The detailed displacement capacity analysis is always required for SDC D.

The yield and ultimate displacements may be approximated from Equations 5-18 to verify values obtained from computer modeling with CAPP or CSiBridge, for example. 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 computed at the center of gravity of the superstructure, then displacement capacity should be computed at the same location. Likewise, if capacity is calculated at the top of the column, then demand should be taken as the response spectrum displacement at the column top.

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

- Case 1, a pier with cantilever-type behavior is analyzed to find the displacement capacity 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 5 to estimate yield displacement relative to the center of mass. Use $f=f_{tc}$ in Equation 5 to estimate yield displacement at the top of the column.

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

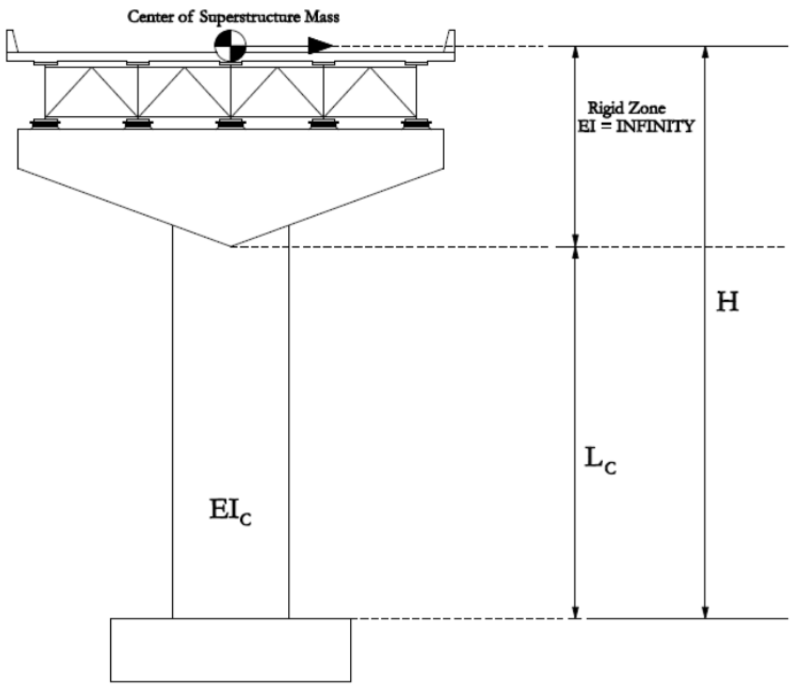

Figure 5. Case 1 - Approximate Equations

Case 1 equations:

$$\Delta_y = \frac{1}{3}\phi_y L_c^2 f \tag{5}$$

$$f_{cm} = \frac{x^2 - 3x + 3}{x} \tag{6}$$

$$f_{tc} = \frac{3 - x}{2} \tag{7}$$

$$x = \frac{L_c}{H} \tag{8}$$

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

$$(\Delta_p)_{tc} = (\phi_u - \phi_y)L_p\left(L_c - \frac{L_p}{2}\right) \tag{10}$$

$$L_p = 0.08L_c + 0.15f_{ye}d_{bl} \geq 0.3f_{ye}d_{bl} \tag{11}$$

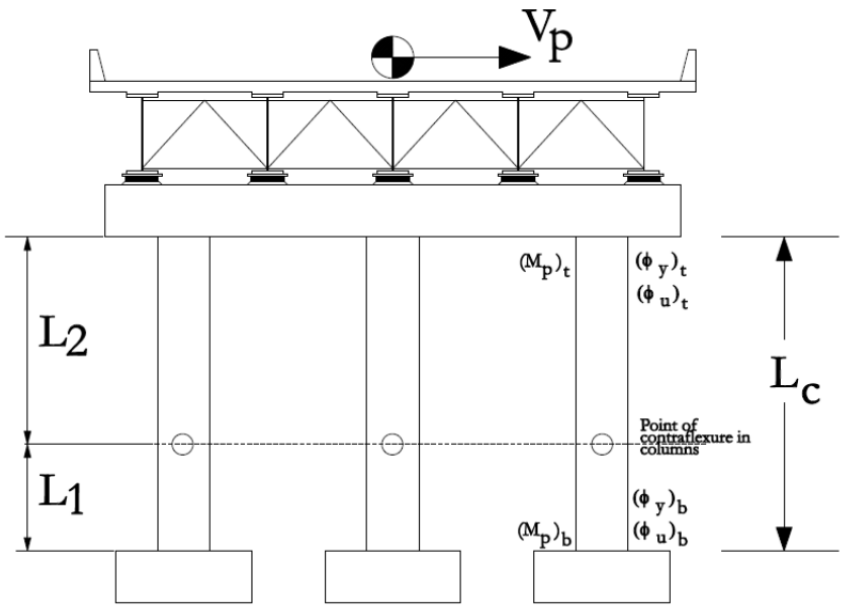

Figure 6. Case 2 - Approximate Equations

Case 2 equations:

$$\Delta_y = \frac{1}{3}\left(\phi_{yb}L_1^2 + \phi_{yt}L_2^2\right) \tag{12}$$

$$L_1 = \frac{M_{pb}}{M_{pb} + M_{pt}} \cdot L_c \tag{13}$$

$$L_2 = L_c - L_1 \tag{14}$$

$$(\Delta_p)_{tc} = (\phi_{ub} - \phi_{yb})L_{pb}\left(L_1 - \frac{L_{pb}}{2}\right) + (\phi_{ut} - \phi_{yt})L_{pb}\left(L_2 - \frac{L_{pt}}{2}\right) \tag{15}$$

$$\Delta_u = \Delta_y + \Delta_p \tag{16}$$

$$L_{pb} = 0.08L_1 + 0.15f_{ye}d_{bl} \geq 0.3f_{ye}d_{bl} \tag{17}$$

$$L_{pt} = 0.08L_2 + 0.15f_{ye}d_{bl} \geq 0.3f_{ye}d_{bl} \tag{18}$$

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 the ***Appendix*** to this SMO.

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 19.

$$\sqrt{\left(\frac{\Delta_l}{\Delta_{ul}}\right)^2 + \left(\frac{\Delta_t}{\Delta_{ut}}\right)^2} \leq 1.00 \qquad (19)$$

- 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:

$$\Delta_{t1} = \Delta_L sin\theta - \Delta_T cos\theta \quad (20)$$

$$\Delta_{l1} = \Delta_L cos\theta + \Delta_T sin\theta \quad (21)$$

$$\Delta_{t2} = \Delta_L sin\theta + \Delta_T cos\theta \quad (22)$$

$$\Delta_{l2} = \Delta_L cos\theta - \Delta_T sin\theta \quad (23)$$

- 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, should 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 Articles 4.3.2 and 4.3.3 of the Guide Specification.

4.3 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 dynamics analysis. The method presented here at least accounts for differing amplification factors in each direction, something which the Guide Specification in Article 4.3.3 does not achieve. Suppose a bridge is located in Seismic Zone 4 (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 upon the ductility demand, which is dependent upon 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.

$$R_d = \left(1 - \frac{1}{\mu_D}\right) \cdot \frac{T^*}{T} + \frac{1}{\mu_D} = \left(1 - \frac{1}{5}\right) \cdot \frac{0.560}{0.414} + \frac{1}{5} = 1.282 \tag{24}$$

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.

$$R_d = \left(1 - \frac{1}{\mu_D}\right) \cdot \frac{T^*}{T} + \frac{1}{\mu_D} = \left(1 - \frac{1}{5}\right) \cdot \frac{0.560}{0.276} + \frac{1}{5} = 1.823 \tag{25}$$

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-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:

$$\mu_{DL} = 1 + \frac{1.753 - 0.510}{0.510} = 3.44 \tag{26}$$

$$\mu_{DT} = 1 + \frac{1.083 - 0.420}{0.420} = 2.58 \tag{27}$$

$$\mu_D = \sqrt{3.44^2 + 2.58^2} = 4.30 \tag{28}$$

Load Case 4:

$$\mu_{DL} = 1 + \frac{0.932 - 0.510}{0.510} = 1.83 \tag{29}$$

$$\mu_{DT} = 1 + \frac{1.949 - 0.420}{0.420} = 4.64 \tag{30}$$

$$\mu_D = \sqrt{1.83^2 + 4.64^2} = 4.99 \tag{31}$$

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.

4.4 Foundations

Footings and piles caps should be initially designed for Strength and Service limit states. This design is then adjusted (by changing dimensions, reinforcement, adding piles, etc. to meet the Extreme Event limit state requirements.

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. RC-PIER has the capability of modeling the struts. If struts are used, they must be 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 Limit State, spread footings and pile cap footings should 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_f 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 4.5 of this SMO 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, bending the column bars inward instead of outward in combination with hoops inside the joint is an effective means of providing adequate strength. When column bars are bent outward in the footing, reinforcement external to the joint in addition to column hoops within the joint may be required. See pages 408-412 of (Priestley, Seible and Calvi, Seismic Design and Retrofit of Bridges 1996) for a further discussion on joint shear in footings.

Refer to SMO 31 for detailed guidance on foundation design.

4.5 Joint Shear Design

For Bridges in SDC A and B, joint design consists of satisfying provisions for extending column transverse reinforcement into cap beams and footings found in the LRFD Specification, Section 5.10.11.4.3.

For bridges in SDC C and D, the Guide Specification provisions for joint shear design shall be used. Expected concrete strength may be used in determining permissible principal stress levels. Joints are classified as either

- knee joints or
- T-joints or
- 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 upon the criteria used. Each of the following documents present 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 tee-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, say 4,000 psi or higher. In itself or along with member dimensional changes, this can be an effective means of providing sufficient joint strength.

Specific procedures for Tee-joints and Knee-joints are covered in Sections 4.5.1 and 4.5.2, respectively.

4.5.1 Bent Cap Tee Joints

For bent cap T-joints – Figure 12 - on bridges in SDC C and D, joints shall be sized so that Guide Specification Section 8.13.2 is satisfied (principal compression = p_c; principal tension = p_t).

$$p_c \leq .25f_c' \tag{32}$$

$$p_t \leq 0.38\sqrt{f_c'} \tag{33}$$

If either of these criteria is not met, then member size must 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 re-calculating 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 should be applied to T-joint design. These equations are applicable for ASTM A 615 reinforcing steel. If ASTM A 706 steel is specified, the required values may be multiplied by the ratio (1.2/1.4).

Vertical stirrups outside the joint region should be distributed over a distance equal to ½ the beam height, h_b, from the column face. The added bottom reinforcement should 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.

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

$$\sum F_y = 0: V_{bl} - V_{br} = P_c - P_{ss} \tag{35}$$

$$\sum M_o = 0: M_{br} + M_{bl} = M_c + M_{ss} + (V_c + V_{ss})\frac{h_b}{2} - (V_{br} + V_{bl})\frac{h_c}{2} \tag{36}$$

From Equation 35:

$$V_{br} + V_{bl} = 2V_{br} + P_c - P_{ss} \tag{37}$$

Substitute this into Equation 36:

$$M_{br} + M_{bl} = M_c + M_{ss} + (V_c + V_{ss})\frac{h_b}{2} - (2V_{br} + P_c - P_{ss})\frac{h_c}{2} \quad (38)$$

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 must be added in the top, the bottom, or both. The column moment, shear, and axial load should be taken as the over-strength values.

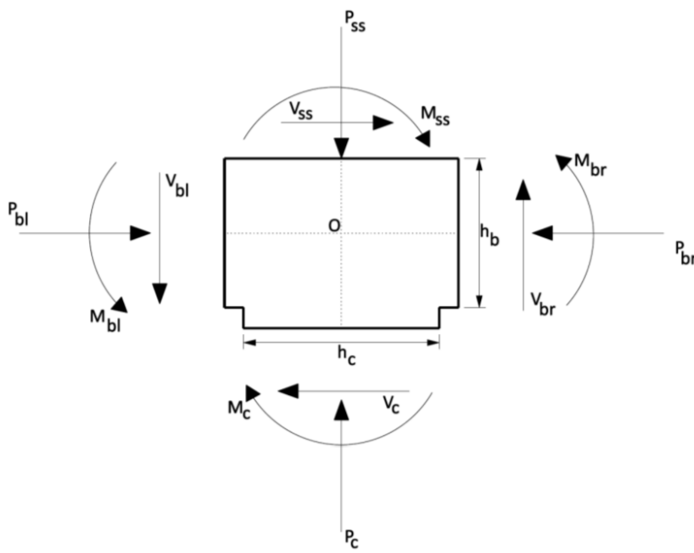

Figure 8. Tee-joint Equilibrium

4.5.2 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 39-41) and "closing" (Figure 10 with Equations 42-44) conditions for bridges in SDC C and D. Principal stress values are limited as for Tee-Joints. Both "opening" and "closing" conditions need to be examined.

$$\sum F_x = 0: P_b = V_c - V_{ss} \quad (39)$$

$$\sum F_y = 0: V_b = P_c - P_{ss} \tag{40}$$

$$\sum M_o = 0: M_b = M_c + M_{ss} + (V_c + V_{ss})\frac{h_b}{2} - (V_b)\frac{h_c}{2} \tag{41}$$

$$\sum F_x = 0: P_b = V_c - V_{ss} \tag{42}$$

$$\sum F_y = 0: V_b = P_{ss} - P_c \tag{43}$$

$$\sum M_o = 0: M_b = M_c + M_{ss} + (V_c + V_{ss})\frac{h_b}{2} - (V_b)\frac{h_c}{2} \tag{44}$$

The column moment, shear, and axial load should be taken as the over-strength values. The superstructure moment, shear, and axial load should be un-factored 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 reinforcement and stirrups are required in the cap. The calculated values will also be used to find the joint stresses.

The column moment, shear, and axial load should be taken as the over-strength values. The superstructure moment, shear, and axial load should be un-factored 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 bottom reinforcement and stirrups are required in the cap. The calculated values will also be used to find the joint stresses.

4.5.3 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 tee-joint use Equations 45-51.

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

- the cap width, b_b, and $\sqrt{2}\times D$, 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.

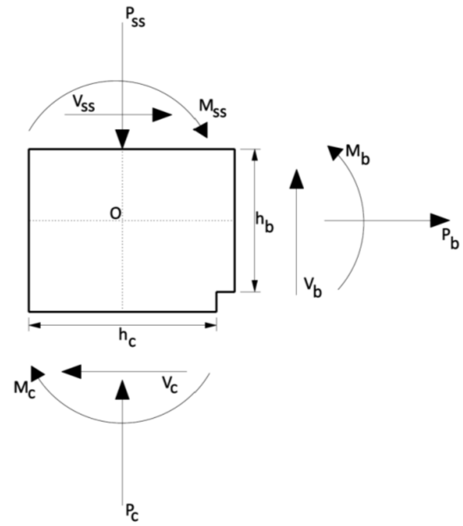

Figure 9. Knee Joint - Opening Condition

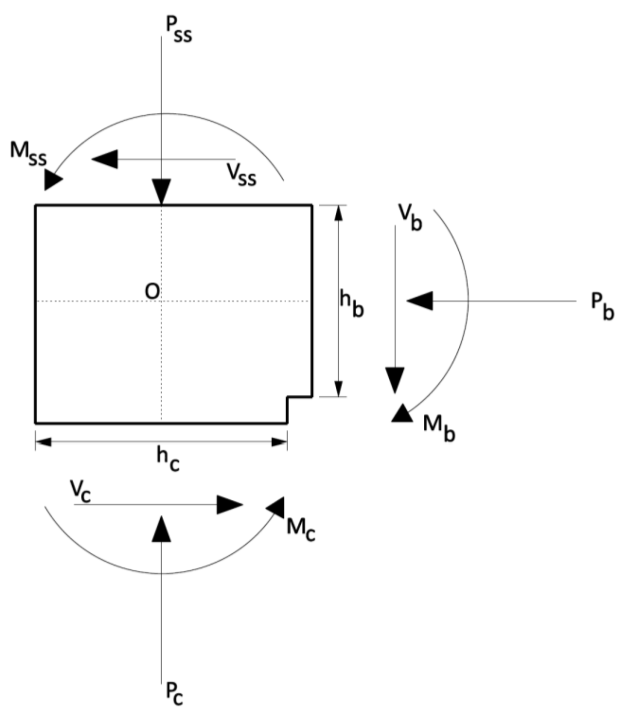

Figure 10. Knee Joint - Closing Condition

$$v_{jh} = v_{jv} = \frac{M_{po}}{h_c h_b b_{je}} \tag{45}$$

$$f_v = \frac{P_c}{A_{jh}} \tag{46}$$

$$f_h = \frac{P_b}{b_b h_b} \tag{47}$$

$$A_{jh} = \left(h_c + \frac{h_b}{2}\right) b_b - Knee\ joints \tag{48}$$

$$A_{jh} = (h_c + h_b) b_b - Tee\ joints \tag{49}$$

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

$$p_c = \frac{f_h + f_v}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{jh}^2} \tag{51}$$

4.6 Concrete Column Shear Capacity

In some cases, concrete shear capacity of columns can vary widely depending on the method used to predict behavior. Occasionally, shear strength provisions may actually indicate a tighter spacing of transverse reinforcement outside the hinge zone than is required inside the hinge zone, which is counter-intuitive. In such cases, the use of an alternate model – the Modified UCSD Model – should be used to design concrete columns for seismic shear.

The Modified UCSD Model has been shown to predict seismic shear behavior more accurately than other models currently in use. References for the model and comparison with other models are found in the literature (Priestley, Calvi and Kowalsky 2007).

5 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. This example problem provides a step-by-step procedure for selecting and modifying ground motions for final design of such structures.

A proposed 5-span bridge on State Route 14 near Memphis in Shelby County Tennessee is to be constructed on pile bents. Severe scour potential at the site creates the need for large (20" diameter) steel pipe piles at the bents to meet stability requirements in the scoured condition. This makes seismic design in the un-scoured condition problematic in that large diameter, short pile bents will be very stiff and the un-scoured structure will have a short period and high first-mode spectral acceleration. Seismic isolation will be explored as a design alternative. The bridge consists of 5 spans, each 50 feet for a total bridge length of 250 feet. The cross-section is made up of 5 Type-II AASHTO pre-stressed girders composite with a 8-1/4" cast-in-place concrete deck. The total superstructure weight is 9.38 kips/ft including parapets and wearing surface. Bent heights are 15' for Bents 1 through 4, measured from the top of the pile to the point of fixity in the ground. A CSiBridge Model of the structure is shown in Figure 11.

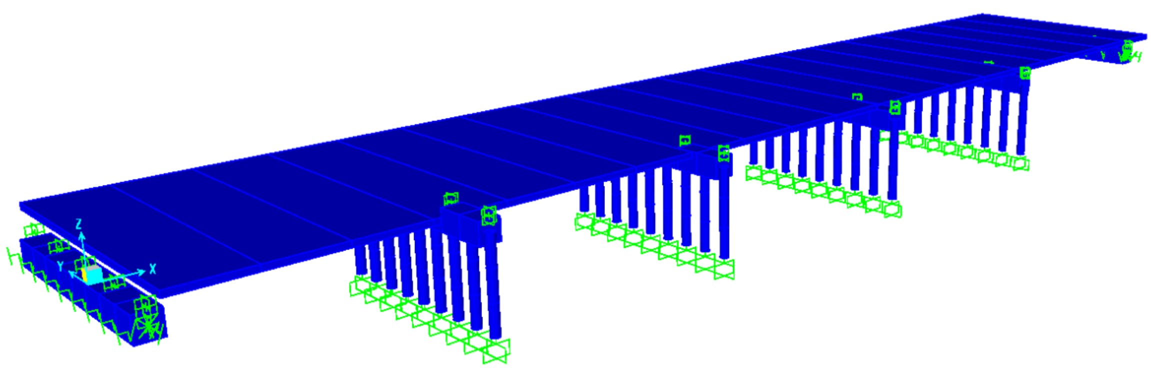

Figure 11. CSiBridge Model

5.1 Design Ground Motion

The site will be defined in terms of three parameters:

1. V_{S30}, average shear wave velocity in the upper 30 meters of the subsurface profile
2. the corresponding AASHTO Site Classification
3. the embayment depth

The bridge coordinates are:

- Latitude = 35°19’26” N
- Longitude = 89°48’25” W

Two borings at the site indicate standard penetration test (SPT) blow counts of 11 blows per foot. Without tests to explicitly determine shear wave velocity, there are two methods used to estimate V_{S30} values:

1. N-V_{S30} correlations from AASHTO and
2. the OpenSHA Site Data application – www.opensha.org

From Table 3.4.2.1-1 of the AASHTO Guide Specification, blow counts less than 15 blows per foot correspond to shear wave velocities less than 180 meters per second. An inferred V_{S30} value of 180 m/sec is obtained for the site using OpenSHA with the site latitude, longitude, the “Global VS30 from Topographic Slope” option, and the “stable continent” option (as opposed to the “active tectonic” option in OpenSHA). The shear wave velocity breakpoint between “D” and “E” site classes is 180 m/sec. The AASHTO site classification is taken as “E”. Maps from Fernandez (Fernández 2007) and Toro & Silva (Toro and Silva 2001) are used to establish an approximate depth of soil profile equal to 750 meters for the site, which is about 30 kilometers (18 miles) northeast of downtown Memphis.

Uniform hazard design response spectra are defined using USGS 2008 data. Ground shaking with a 7% probability of exceedance in 75 years corresponds to a return period of about 1,000 years. This is the Design Basis Earthquake (DBE) in current bridge design practice. Ground shaking with a 3% probability of exceedance in 75 years corresponds to a return period of about 2,500 years – the Maximum Credible (or Considered) Earthquake (MCE). Critical bridges and specialized designs such as seismic isolation may require design criteria at the MCE hazard level. The control points of the uniform hazard acceleration response spectra developed using code-based site amplification factors are summarized in Table 4.

Table 4. UHRS Control Points - SR14 Site

Parameter	DBE	MCE
PGA	0.330	0.591
S_1	0.168	0.324
S_S	0.629	1.136
F_{PGA}	1.109	0.900
F_a	1.443	0.900

F_v	3.295	2.703
A_S	0.366	0.532
S_{DS}	0.907	1.023
S_{D1}	0.555	0.877
T_S	0.612	0.857
T_O	0.122	0.172

A deaggregation of the seismic hazard is needed to select ground motions for the site. The 2008 USGS Interactive Deaggregations (Beta) online tool is used for this purpose (geohazards.usgs.gov/deaggint/2002/). Peak ground acceleration and spectral accelerations at 0.2 seconds, 1.0 seconds, and 2.0 seconds (the highest period available from the USGS in the NMSZ) are included in the deaggregation. For the NMSZ these deaggregations are for rock sites (AASHTO Site Class B/C boundary) only and no site-effects are included in the deaggregations for NMSZ locations. As described in the documentation for the USGS national seismic hazard maps (Petersen, et al. 2008), the PSHA for the NMSZ includes ground motion models from 7 sources:

1. (Atkinson and Boore 2006)
2. (Campbell 2003)
3. (Frankel, et al. 1996)
4. (Silva, Gregor and Darragh 2003)
5. (Somerville, et al. 2001)
6. (Tavakoli and Pezeshk 2005)
7. (Toro, Abrahamson and and Schneider 1997)

Campbell used a hybrid empirical approach and is explicit in stating that the developed ground motion model corresponds to the geometric mean of two horizontal components. Toro used the stochastic ground motion method to develop a model for spectral acceleration and compared results to Eastern North America (ENA) ground motion data from previous work (Electric Power Research Institute 1993). The EPRI report used the geometric mean of spectral ordinates for 66 horizontal recordings from earthquakes. While the other referenced works are not explicit in identifying the geometric mean as the basis of the ground motion model, take the basis of the USGS data to be the geometric mean of two horizontal components as opposed to an arbitrary component or a maximum horizontal component. This is consistent with previous work by others on relationships between various measures of ground motion intensity (Watson-Lamprey and Boore October 2007). The deaggregations for 1-second spectral acceleration at the B/C boundary for DBE and MCE hazard levels are given in Figures 12 and 13, respectively. The modal event - the one most likely to produce ground motion exceeding the design value - is important in selecting records for nonlinear analysis (Bazzurro and Cornell 1999). For each of the spectral accelerations at both hazard levels, the modal event is Mw7.7 with R = 59.5 kilometers.

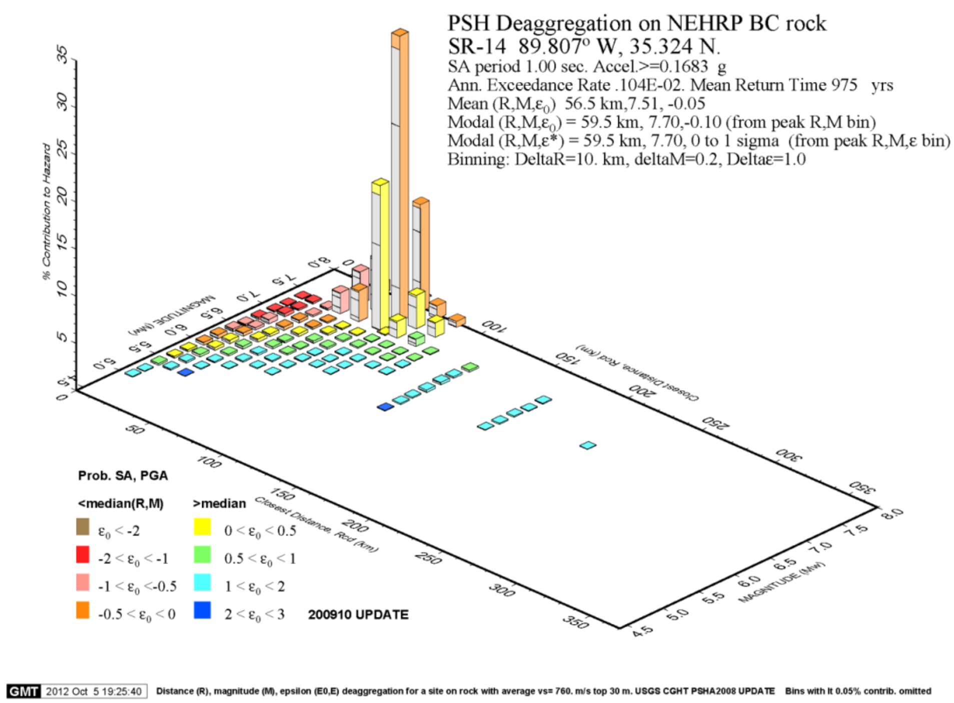

Figure 12. SR14 1-second Spectral Acceleration Deaggregation - DBE

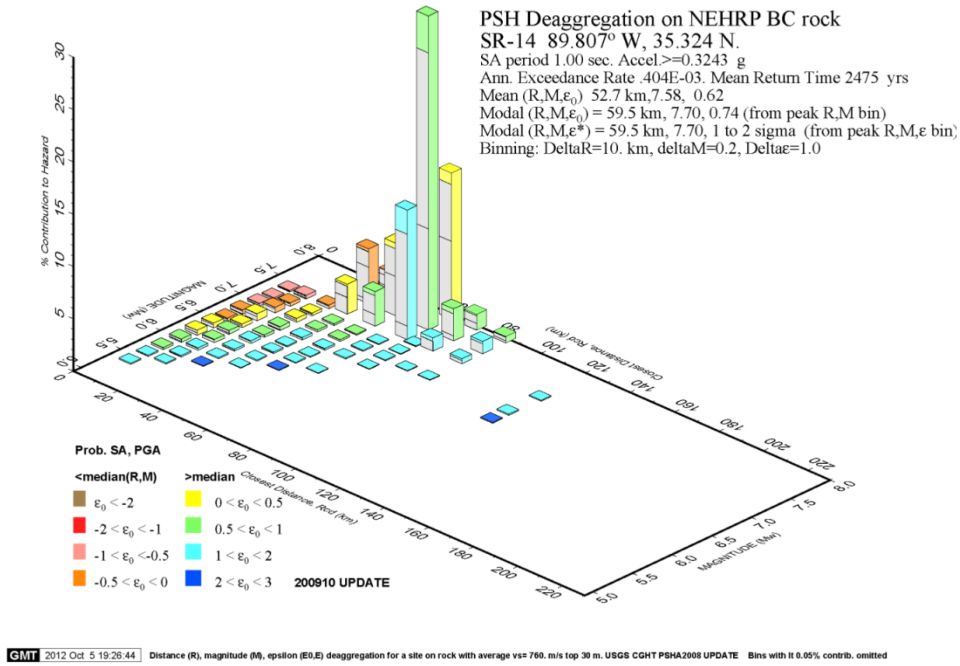

Figure 13. SR14 1-second Spectral Acceleration Deaggregation - MCE

Much of the Mississippi Embayment (ME) consists of deep soils which alter the character of bedrock motions. Nonlinear site response studies (Hashash, et al. 2008) of the ME have shown that code-based site factors may be too high at low periods and too low at long periods when the embayment depth is greater than 30 meters. This will be particularly important for isolated bridges since isolated effective periods will most likely be longer than about 1.5 seconds. For this reason, focus on the UHRS spectral acceleration at a period of 1-second. Implicit in code spectra is a constant site factor for periods equal to 1 second and longer. While deaggregation of the seismic hazard at deep sites in the ME provides a 2-second spectral acceleration at the B/C boundary, avoid the use of code site factors at periods beyond 1 second for this class “E” site, which has as estimated embayment depth of 750 meters. This choice will be reflected in the target spectra definitions.

On pages 195-196 of a PEER Report (Stewart, et al. 2001) the authors recommend consideration of permitting spectral shapes of selected ground motions to “deviate from the design spectrum” at longer periods when basin effects are not included in the design spectrum at sites where basin effects are possible. The PEER Report also recommends scaling of selected records to match the design spectrum even in cases for which magnitude, distance, and site condition criteria are met.

In a Georgia Tech study (Romero and Rix 2005), site effects in the Mississippi Embayment, described as a basin, including depth of the soil profile were studied and the observation made that “current” code-based site provisions “may significantly underestimate ground motions at periods longer than 1 second.” See page 397 of the referenced study.

The choice is made to define dual target spectra. The first target spectrum is taken as the DBE-level uniform hazard spectrum with code-based site amplification factors. For the second target, results from Fernandez and Rix (Fernandez and Rix 2006) are used to develop a NMSZ-specific response spectrum for a profile depth in the 610-1220 meter range. Fernandez and Rix used stochastic point-source simulations based on three source models, three stress drops, two soil profiles, and six ranges of profile depth. A ground motion prediction model (attenuation relationship) was developed and the results made available online at: http://geosystems.ce.gatech.edu/soil_dynamics/research/soilattenuations/.

For the second target, the average of nine spectra developed from the ground motion prediction model of Fernandez and Rix is scaled to the DBE-level 1-second spectral acceleration of 0.555 g. The nine spectra represent the low, medium, and high stress drop values for each of the three source models generated for a magnitude 7.7 earthquake and an epicentral distance of 59.5 kilometers, corresponding to the deaggregated modal (M,R) pair. The DBE-level uniform hazard spectrum (target 1) and the NMSZ-specific spectrum (target 2) at the site are shown in Figures 14 (spectral acceleration) and 15 (spectral velocity). Target 1, with greater high frequency content, is expected to control the design of substructures for the isolated bridge while target 2, richer in long period content, is expected to control the design of the isolation systems. For reference, the MCE-level uniform hazard spectra are included in the plots as well.

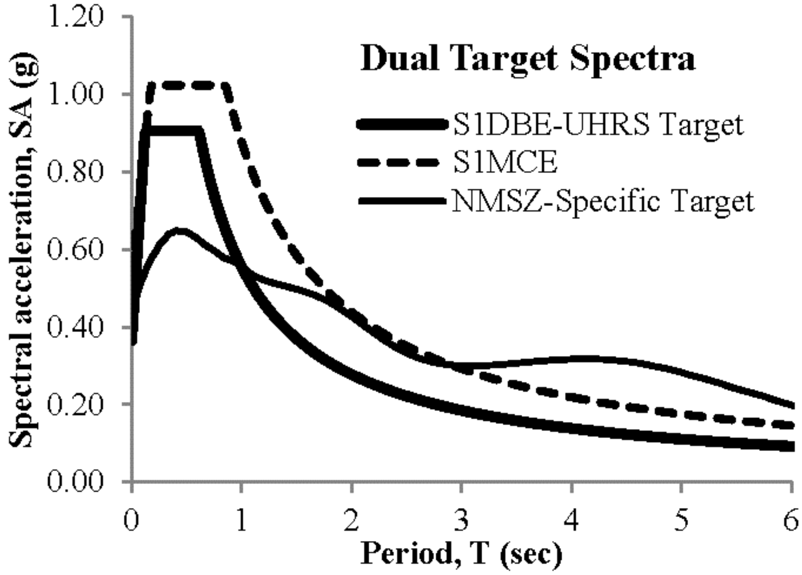

Figure 14. SR14 Site - Dual Target Spectra (Acceleration)

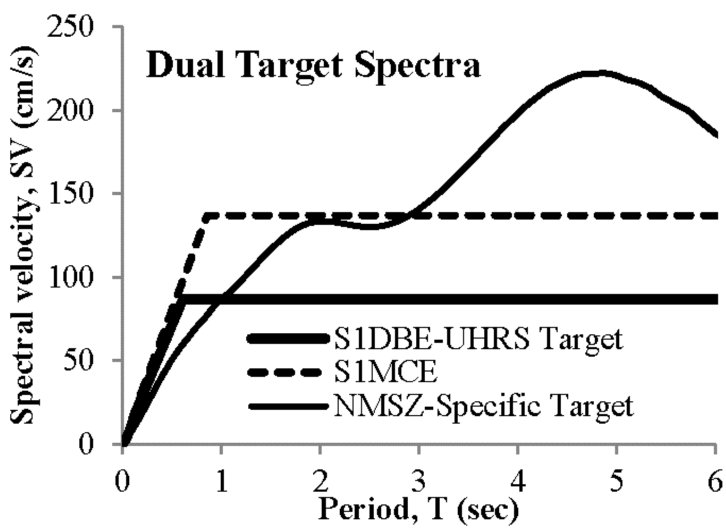

Figure 15. SR14 Site - Dual Target Spectra (Velocity)

5.2 Non-isolated Design

The scoured pile length from top of pile to point of fixity in the ground (5 diameters below the scoured surface) is 39 feet. The piles would be expected to be loaded into the inelastic range during ground shaking at the design basis level, so Guide Specification requirements for "Ductile Elements" will have to be met for the non-isolated bridge. The piles can be expected to be "flexural moment dominant" since the axial stress for a steel pile is typically 25% of yield (it will be even less in the case considered here). Assume F_y = 35 ksi for the piles initially and adjust as needed throughout the design. From Section 7.4.1 of the Guide Specification:

$$\lambda_b = \frac{L}{r_y} = \frac{39 \times 12}{r_y} \leq \frac{0.086E}{F_y} \tag{52}$$

$$r_y \geq \frac{39 \times 12 \times 35}{0.086 \times 29{,}000} = 6.57\ inches \tag{53}$$

$$\frac{KL}{r_{min}} \leq 120 \;\Rightarrow r_{min} \geq \frac{0.85 \times 39 \times 12}{120} = 3.32\ inches \tag{54}$$

$$\frac{D}{t} \leq \frac{0.044E}{F_y} = \frac{0.044 \times 29{,}000}{35} = 36.5 \tag{55}$$

Try 18” diameter piles:

$$t \geq \frac{18}{36.5} = 0.493\ inches \tag{56}$$

Try 18” x ½” pipe piles. The pile properties are thus:

$$A_p = \frac{\pi}{4}(18^2 - 17^2) = 27.49\ in^2 \tag{57}$$

$$I_p = \frac{\pi}{64}(18^4 - 17^4) = 1{,}053.2\ in^4 \tag{58}$$

$$r = \sqrt{\frac{1{,}053.2}{27.49}} = 6.19\ inches\ < 6.57\ inches\ \Rightarrow No\ Good \tag{59}$$

Try 20” x 9/16” pipe piles. The properties are:

$$\frac{D}{t} = \frac{20}{9/16} = 35.6\ < 36.5 \Rightarrow\ OK \tag{60}$$

$$A_p = \frac{\pi}{4}(20^2 - 18.875^2) = 34.35\ in^2 \tag{61}$$

$$I_p = \frac{\pi}{64}(20^4 - 18.875^4) = 1{,}623.6\ in^4 \tag{62}$$

$$r = \sqrt{\frac{1{,}623.6}{34.35}} = 6.87\ inches\ > 6.57\ inches\ \Longrightarrow OK \qquad (63)$$

So, 20” x 9/16” piles are required to meet stability requirements for ductile members in the scoured condition. The potential advantage of isolation is that the substructures may be designed to remain elastic in some cases for isolated structures. If the piles can be designed to respond well below yield during ground shaking, then the L/r and D/t ratios may be relaxed considerably (see Guide Specification Sections 7.4.1 and 7.4.2) resulting in material savings in piles which would help offset the cost of expensive isolation bearings and the expansion joints required for an isolated bridge. If thin-walled piles could be kept well below yield, then requirements for “essentially elastic” elements could be adopted for the piles.

5.3 Ground Motion Records

Few ground motion records from intra-plate earthquakes on deep soil sites are available. In such a case it is advisable to explore the use of synthetic records to complement records from actual earthquakes of appropriate magnitude in different tectonic settings.

A search for real records is made of various sources for strong ground motion recordings at Site Class “D” and “E” stations from earthquakes in the MW6.9-8.0 range. The final record set for a site in the New Madrid Seismic Zone is likely to include record triplets from at least two or three of the following earthquakes:

- August 17, 1999 M_W7.51 Kocaeli, Turkey (strike-slip)
- November 3, 2002 Denali, Alaska M_W7.90 (strike-slip)
- September 21, 1999 Chi-Chi, Taiwan M_W7.62 (strike-slip)
- June 28, 1992 Landers M_W7.28 (strike-slip)
- September 19, 1985 Michoacan, Mexico M_W8.0 (subduction)
- October 17, 1989 Loma Prieta M_W6.93 (reverse-oblique)
- October 16, 1999 Hector Mine M_W7.13 (strike-slip)
- November 12, 1999 Duzce, Turkey M_W 7.14 (strike-slip)
- September 3, 2010 Darfield, New Zealand (Canterbury), M_W7.1 (strike-slip)
- April 4, 2010 Sierra el Mayor, M_W7.2 (strike-slip)

For this example, Kocaeli, Denali, Chi-Chi, Loma Prieta, Hector Mine, Duzce, Sierra el Mayor, and Landers records were obtained from the PEER Ground Motion Database (Pacific Earthquake Engineering Research Center 2014).

Michoacan records were obtained from the University of Nevada at Reno with the assistance of Dr. Ian Buckle. Darfield records were obtained from the Center for Engineering Strong Motion Data (USGS, CGS, ANSS 2014).

To save space, included here are only those records and sources which are part of the final set. A total of 12 sources produced over 800 record triplets from 30 earthquakes for initial analysis.

Two sources were used for synthetic records:

1. Atkinson and Beresnev (Atkinson and Beresnev 2002)

2. Fernandez (Fernández 2007)

Atkinson used a finite-fault simulation and studied a wide range of input variables - the most important of which were found to be magnitude, hypocenter location, and maximum slip velocity - to generate ground motion for the cities of St. Louis, Missouri and Memphis, Tennessee for magnitude 7.5 and magnitude 8.0 events. Six ground motions were produced for rock, linear soil, and nonlinear soil conditions at each magnitude and each city. A 1,000 meter deep generic soil profile for Memphis was used.

Fernandez and Rix took McGuire’s (McGuire, Silva and Costantino 2001) records and wavelet-matched them to independently generated spectra from a probabilistic seismic hazard analysis for various sites in the NMSZ including: Memphis and Jackson in Tennessee; Jonesboro, Blytheville, and Little Rock in Arkansas; Paducah, Kentucky and Cape Girardeau, Missouri. Further distinction was made between “uplands” and “lowlands” for the Memphis data. Mean annual return periods of 475 years, 975 years, and 2,475 years were included in the subsequently generated ground motions. Ten records were generated for each city, return period and profile. The profile depths at Memphis and Blytheville were taken to be 900 meters.

For bi-directional nonlinear analysis, record pairs are needed. Each of these synthetic motion sources provides single component motions. To remedy this, synthetic motions may be paired. This cannot be done arbitrarily. The records should come from the same source and the same magnitude or return period. And the record should have similar durations. Studies have determined that the correlation coefficient of orthogonal components of real records lies within the range -0.50 to +0.50 (Hadjian 1981). NIST GCR 11-917-15 (NEHRP Consultants Joint Venture 2011) summarizes requirements for ground motion selection and modification from several codes and notes the requirement that the coefficient be no more than 0.30. For this example, require that the correlation coefficient between components of a record pair be less than 0.30 in order for that record to be used for structural analysis.

Table 5 lists a subset of the synthetic ground motions considered for this study. Atkinson-Beresnev records are identified by magnitude, condition, and number. For example, “M7.5L-06” is the 6th simulated motion for a magnitude 7.5 earthquake based on linear soil assumptions. The three conditions are L (linear soil), N (nonlinear soil), and R (rock). Fernandez records are identified by return period, profile condition, and number. For example, “2478-08” in the “Uplands” portion of the table refers to the 8th simulation using an uplands (as opposed to lowlands) profile and a 2,475-year return period. Two cases are shown - records nos. 32 and 33 - to demonstrate the need to check correlation between components rather than assuming statistical independence. Spectral acceleration values at a period of 1-second are included to emphasize the importance of maintaining in some fashion the relative magnitudes of two horizontal components - either by scaling both components by the same factor or spectrally matching each component to the appropriate multiple of the target spectrum.

Table 5. Synthetic Record Pairs

No.	H1	H2	Corr.	Dur.	SAH1(1)	SAH2(1)	Source
1	7.5L-03	7.5L-01	0.0690	40.3	0.226	0.315	A & B
2	7.5L-02	7.5L-06	0.1418	40.3	0.261	0.244	A & B
3	7.5L-04	7.5L-05	0.0286	40.3	0.171	0.153	A & B
10	8.0N-02	8.0N-06	0.1346	64.7	0.490	0.326	A & B
11	8.0N-01	8.0N-04	0.0624	64.7	0.282	0.431	A & B
12	8.0N-03	8.0N-05	0.0485	64.7	0.398	0.550	A & B
13	0975-07	0975-08	0.1867	59.7	0.201	0.238	F & R (Lowlands)
14	0975-01	0975-02	0.0668	55.3	0.469	0.488	F & R (Lowlands)
16	0975-05	0975-06	0.0358	71.8	0.215	0.377	F & R (Lowlands)
17	2475-09	2475-10	0.1350	43.6	0.605	0.554	F & R (Lowlands)
18	2475-03	2475-07	0.0456	59.5	0.584	0.635	F & R (Lowlands)

19	2475-05	2475-06	0.0472	73.3	0.648	0.730	F & R (Lowlands)
21	0975-10	0975-09	0.0009	44.8	0.191	0.251	F & R (Uplands)
22	0975-05	0975-06	0.0071	72.7	0.256	0.238	F & R (Uplands)
23	2475-08	2475-07	0.1837	58.8	0.512	0.920	F & R (Uplands)
24	2475-05	2475-06	0.0499	73.5	0.432	0.653	F & R (Uplands)
27	0975-05	0975-02	0.1301	83.0	0.497	0.568	F & R (Blythville)
28	0975-03	0975-06	0.1911	52.8	0.689	0.608	F & R (Blythville)
30	2475-04	2475-10	0.0650	75.7	1.127	1.073	F & R (Blythville)
32	2475-08	2475-09	***0.5315***	43.2	1.224	0.973	F & R (Blythville)
33	2475-06	2475-03	***0.3832***	52.6	1.185	1.143	F & R (Blythville)

5.4 Selection and Modification Procedure

Three sources of guidance for record selection on highway bridge projects are available.

1. The AASHTO Guide Specifications for LRFD Seismic Bridge Design
2. The AASHTO Guide Specifications for Seismic Isolation Design
3. The FHWA Seismic Retrofitting Manual for Highway Structures

The Guide Specifications for LRFD Seismic Bridge Design (AASHTO, Guide Specifications for LRFD Seismic Bridge Design 2011) require a minimum of seven time histories for nonlinear analysis of bridge structures if the mean response is to be taken as the design value. Three time histories are required if the maximum response is to be used. The Commentary to Section 3.4.4 identifies tectonic environment, earthquake magnitude, type of faulting, source-to-site distance, local site conditions, and expected ground motion characteristics as the parameters deserving careful attention in record selection. The Commentary also notes that “compromises are usually required”, that magnitude and distance are particularly important, and that large scale factors are to be avoided. Section 3.4.4 further requires that time histories be scaled to the design spectrum in the “period range of interest” with preference towards a single scaling factor for all components of a record pair or triplet.

The Guide Specifications for Seismic Isolation Design (AASHTO, Guide Specification for Seismic Isolation Design 2010) contain the same requirements as do the Guide Specifications for LRFD Seismic Bridge Design for number of records and refer to Section 3.4.4 of the Guide Specifications for LRFD Seismic Bridge Design as the basis for record selection.

The FHWA Seismic Retrofitting Manual (Buckle, et al. 206) is not directly applicable to new designs but does contain valuable information on the subject of record selection for the structural analysis of bridges. Section 2.8.2 permits the use of either scaled records or spectrally-matched records. Section 2.8.3 requires the calculated mean spectrum for the scaled records to be no more than 15 percent lower than the design spectrum at any period over the range of periods of "structural significance" and that the average mean-to-design spectrum ratio be no less than 1.0 over the same period range. Requirements for number of records are identical to those in AASHTO.

A partial summary of observations from a National Institute of Standards and Technology report (NEHRP Consultants Joint Venture 2011) are important enough to mention here because the report recommendations will be incorporated into our process of selection and modification. Spectral shape over the period range of interest is identified as the single most important variable in selecting records to match a target spectrum at far-field sites (R>10 kilometers). Earthquake magnitude, source-to-site distance, and local site conditions are listed among the secondary factors. For near-field sites, spectral shape and the presence of pulse-type motions are the two most important factors. Spectral matching is identified as a useful tool when mean response is the design parameter of interest. When the dispersion of design parameters is needed, spectral matching cannot be relied upon. ASCE SRSS-based scaling procedures (American Society of Civil Engineers 2005) are shown to "have no technical basis" and do "not recover the geomean spectrum". The USACE drafted guidelines (EC 1110-2-6000) are identified as the "most comprehensive of those used in design practice at the time of this writing". The USACE report places no limits on scaling factors provided the "time history sets after scaling have characteristics that correspond with those developed for the design ground motion". The use of uniform hazard target spectra is considered "appropriate if conservative estimates of building response are acceptable". With the AASHTO and the NIST guidelines in mind, the choice is made to:

- permit the use of records requiring high scale factors as long as close matches to spectral shape are maintained
- include spectral matching as an option to estimate mean response

- include synthetic and artificial records paired such that the correlation between components is no more than 0.3
- avoid SRSS-based scaling since the target spectra used for this work are geometric mean in nature

A paper presented at the 14th World Conference on Earthquake Engineering (Malaga-Chuquitaype, et al. 2008) has shown that the number of records required to obtain accurate estimates of mean response in nonlinear structures is highly dependent upon the method used to select and modify records. Scaling to initial PGA was shown to require more records (13) compared to peak ground velocity based scaling (7) for the same level of confidence. While the number of records required in the referenced study is for a specific case and cannot be used as a rule for any structure, a dependence on the dispersion of the nonlinear analysis results is evident. Fourteen records will be included in the selection for this site even though only seven records would be required by the design specifications.

Amplitude scaling to minimize the mean-square error between the scaled, geometric mean spectrum of the record pair and the design (target) spectrum is one option frequently used for ground motion modification. The nuclear industry has used this method with five discrete periods: 0.3, 0.6, 1.0, 2.0, and 4.0 seconds. The Pacific Earthquake Engineering Research (PEER) Program proposes this method using a set of 301 logarithmically spaced periods between 0.01 and 10.0 seconds. Scale factors (f) and mean-square-errors (MSE) are determined from equations 64-66. Preference may be given to certain periods of interest through the assignment of weighting factors (w(Ti)).

Match to spectral shape may be measured by the post-scaled MSE or by the factor given in Equation 67 (Katsanos, Sextos and Manolis 2010) and may be important in ground motion selection procedures.

$$SA_{GM}(T_i) = \sqrt{SA_{H1}(T_i) \cdot SA_{H2}(T_i)} \tag{64}$$

$$\ln f = \frac{\sum \left[w(T_i) \cdot \ln \frac{SA_{TARGET}(T_i)}{SA_{GM}(T_i)} \right]}{\sum w(T_i)} \tag{65}$$

$$MSE = \frac{\sum w(T_i)\{\ln[SA_{TARGET}(T_i)] - \ln[f \cdot SA_{GM}(T_i)]\}^2}{\sum w(T_i)} \tag{66}$$

$$D_{rms} = \frac{1}{N}\sqrt{\sum_{i=1}^{N}\left(\frac{SA_{GM}(T_i)}{PGA_{GM}} - \frac{SA_{TARGET}(T_i)}{PGA_{TARGET}}\right)^2} \tag{67}$$

Wavelet algorithms may be used to modify earthquake records such that the response spectrum of the modified record matches the design (target) spectrum within some desired tolerance of a specified range of periods. Within spectral matching procedures there are various wavelet options available. Among these are impulse functions, tapered cosine waves, sinusoidal displacement-compatible wavelets, and polynomial functions. The target spectra - the DBE uniform hazard spectrum developed from the AASHTO specifications and the NMSZ-specific spectrum - are taken as geometric mean in nature. The issue of geometric mean versus arbitrary or maximum component spectra is an important one (Baker and Cornell 2006). For each selected real record pair, the spectrum of each horizontal component is first computed using SeismoSpect (SeismoSoft 2011). The scale factor for the record is then calculated to minimize the mean-square-error (MSE) for a period range of:

- 0.2-6.0 seconds for scaling to the uniform hazard spectrum with code-based site amplification factors (target 1)
- 1.0-6.0 seconds for scaling to the NMSZ-specific spectrum (target 2)

Records are then ranked by MSE in ascending order and the records with the lowest values are included in the design set. No more than 6 records from a single event are taken before moving on to the next event with its lowest post-scaled MSE values. While AASHTO currently places no limitation of records from a single event, an often used criterion is to select no more than 3 records from a given event when 7 records are used to make up the total set. Spectral matching will be also used in this example for ground motion modification so a strategy is developed to take care of the relative magnitudes of component spectra. The spectral accelerations of the two components at a period of 1 second will be determined as will the geometric mean of the two at this same period. The ratio of each component to the geometric mean will be maintained in the matching process. So for example, suppose that $SA_{H1}(1) = 0.475$ and $SA_{H2}(1) = 0.396$. Then $SA_{GM}(1) = \sqrt{0.475 \cdot 0.396} = 0.434$. In this example, the H1 component would be matched to 1.094 (0.475/0.434) times the target spectrum and H2 would be matched to 0.912 (0.396/0.434) times the target spectrum. For amplitude scaling, the relative magnitudes of the components is automatically preserved since both components are multiplied by the same scale factor.

Three sets of ground motion are developed:

1. For Set No. 1, records are amplitude-scaled to match target no. 1
2. For Set No. 2, records are amplitude-scaled to match target no. 2
3. For Set No. 3, records are first scaled to target no. 2 and then spectrum-matched with SeismoMatch (SeismoSoft 2013) to a composite spectrum which envelopes target nos. 1 and 2.

The minimum and average record mean-to-target ratios for the sets are as follows:

- Set No. 1: Minimum = 0.85, Average = 1.02 (0.2-6 second range)
- Set No. 2: Minimum = 0.78, Average = 0.95 (1-6 second range)
- Set No. 3: Minimum = 0.99, Average = 1.01 (0.2-6.0 second range)

Yet another method of adjusting record sets was used for Set No. 2. The retrofit manual limits the minimum mean-to-target ratio to no less than 0.85 and the average ratio to no less than 1.0. So, for Set No. 2, the final scale factors were increased by the larger of (a) 0.85/0.78 = 1.05 and (b) 1.00/0.95 = 1.09. Final scale factors for Set No. 2 were amplified by a factor of 1.09 and these are the values shown in the tables.

Tables 6 through 8 along with Figures 16 through 18 summarize the three record sets to be used in nonlinear analyses of the isolated bridge. It may be possible to use Record Set No.3 - spectrum matched to the composite target - alone in performing the final design. Guidance from the Director of Structures is required in establishing final ground motion sets for design.

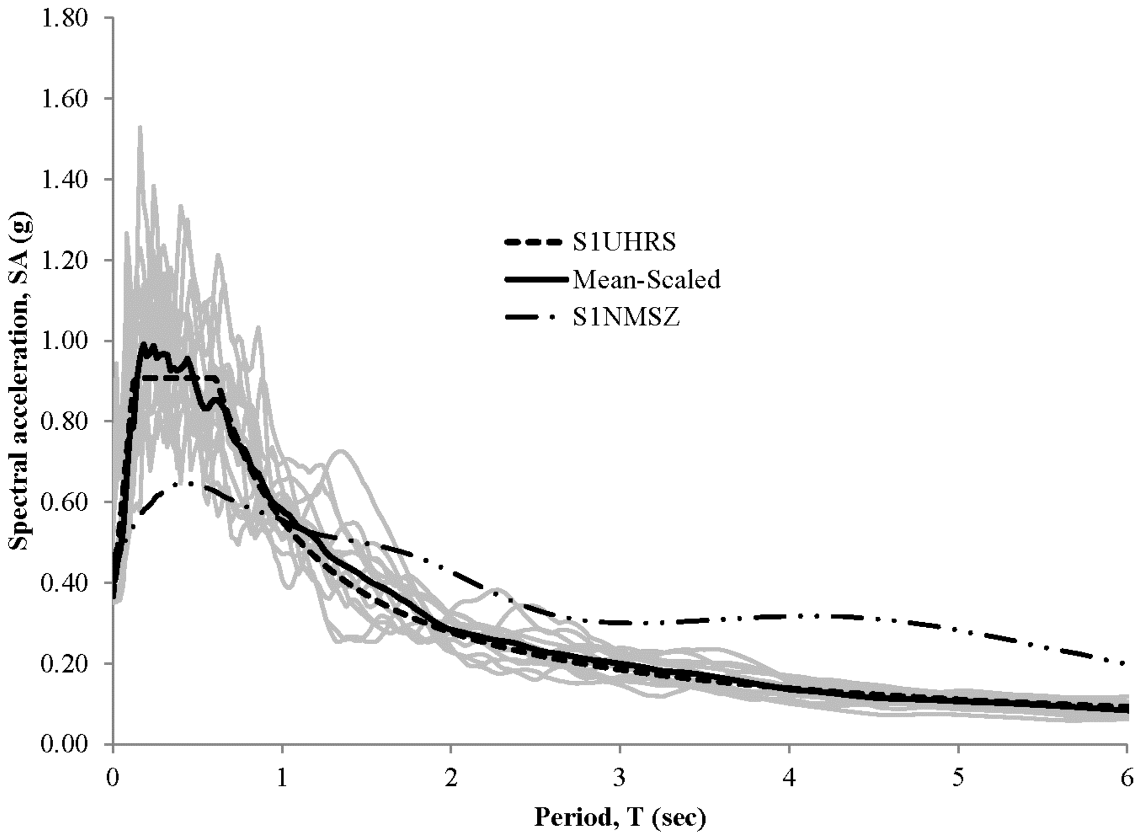

Figure 16. Acceleration Spectra - Record Set No. 1

Table 6. Record Set No. 1 - Amplitude Scaled to Code UHRS

Event	Station	M_W	R, km	V_{S30}, m/s	f
Chi-Chi	CHY057	7.62	57	411	8.37
Chi-Chi	CHY109	7.62	50	478	7.66
Chi-Chi	CHY102	7.62	36	680	8.80
Loma Prieta	LGPC	6.93	0	478	0.69
Loma Prieta	Saratoga	6.93	8	371	1.30
Loma Prieta	Bear Valley	6.93	53	391	5.79
Hector Mine	North Pair	7.13	62	345	5.63
Hector Mine	San Bernadino	7.13	102	271	7.33
Hector Mine	Indio	7.13	73	345	4.65
Landers	Amboy	7.28	69	271	2.77
Landers	MC Fault	7.28	27	345	4.39
Landers	Yermo	7.28	24	354	1.89
Kocaeli	Zeytinburr	7.51	52	274	4.09
Kocaeli	Botas	7.51	126	274	5.63

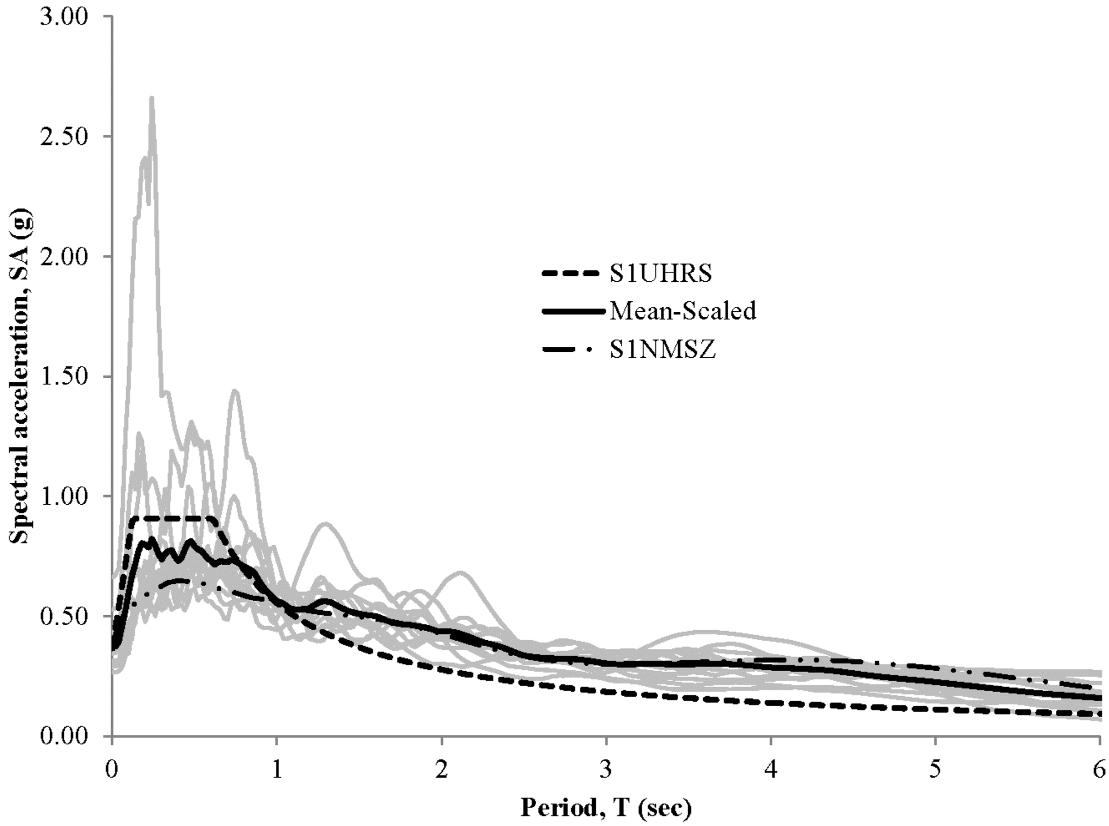

Figure 17. Acceleration Spectra - Record Set No. 2

Table 7. Record Set No. 2- Amplitude Scaled to NMSZ-specific Spectrum

Event	Station	M_W	R, km	V_{S30}, m/s	f
Chi-Chi	TCU100	7.62	11	474	2.72
Chi-Chi	TCU-057	7.62	12	474	2.84
Chi-Chi	CHY107	7.62	59	191	5.37
Chi-Chi	CHY025	7.62	19	278	2.03
Hector Mine	Pico	7.13	187	270	20.89
Hector Mine	Lake Street	7.13	184	371	11.30
Hector Mine	Keys View	7.13	50	685	15.30
Denali	PS#11	7.90	126	376	10.44
Duzce	Duzce	7.14	0	276	1.39
Synthetic	SYN27	-	-	-	1.05
Synthetic	SYN21	-	-	-	2.71
Synthetic	SYN24	-	-	-	1.26
Synthetic	SYN17	-	-	-	1.34
Synthetic	SYN22	-	-	-	2.98

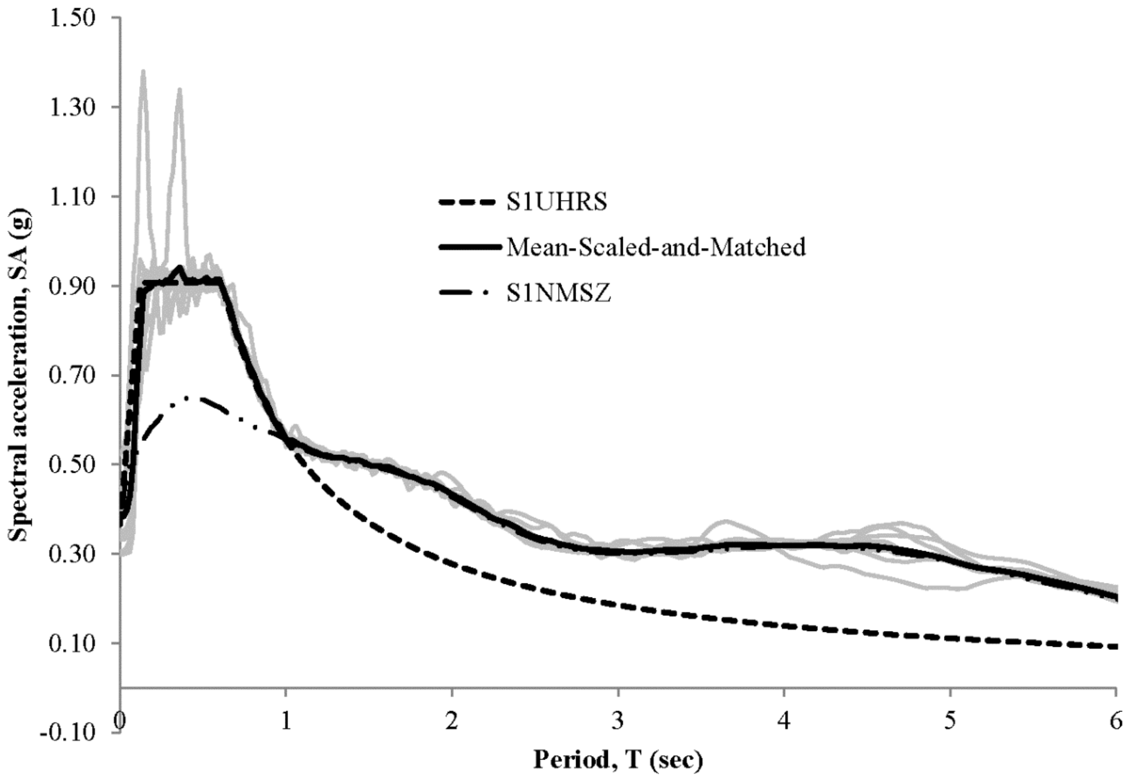

Figure 18. Acceleration Spectra - Record Set No. 3

Table 8. Record Set No. 3 - Spectrum Matched to Composite Spectrum

Event	Station	M_W	R, km	V_{S30}, m/s	f
Kocaeli	Yarmica	7.51	1	297	1.30
Kocaeli	Usak	7.51	227	274	31.52
Kocaeli	Balikesir	7.51	180	339	21.95
Denali	Ester FS	7.90	139	274	25.16
Denali	NOAA WF	7.90	275	274	18.71
Denali	DOI	7.90	273	279	17.91
Michoacan	CALE	8.00	38	180	3.27
Landers	DCMB	7.28	157	272	6.62
Chi-Chi	CHY017	7.62	59	191	1.02
Chi-Chi	CHY025	7.62	19	277	2.92
Chi-Chi	CHY060	7.62	69	229	7.65
Chi-Chi	CHY099	7.62	65	229	1.90
Chi-Chi	ILA004	7.62	87	124	4.26
Chi-Chi	ILA005	7.62	85	239	2.99

5.5 Isolated Bridge Modeling

Nonlinear modal response history analysis - Fast Nonlinear Analysis (FNA) - methods will be used. These procedures are extremely efficient and accurate for problems in which nonlinear behavior is confined to link-type elements. CSiBridge (Computers and Structures, Inc., 2011) is used for the analysis of the bridge. Nonlinear link elements in the CSiBridge library include biaxial hysteretic elements with coupled plasticity for the two shear deformations. The model incorporated into CSiBridge is that proposed by Wen (Wen 1976) and recommended for isolators by Nagarajaiah (Nagarajaiah, Reinhorn and Constantinou 1991). The isolators have been modeled with non-linear properties for both shear directions. Ritz vectors are used instead of the usual eigenvector modal analysis. Ritz vectors are generally preferred for this type of analysis unless every possible mode is included in the eigenvector analysis. Elastic damping is taken as zero for the response history analyses. This is due to problems which can frequently occur with viscous damping models in nonlinear analyses. It has been shown (Priestley and Grant, Viscous Damping in Seismic Design and Analysis 2005) that the appropriate elastic damping for nonlinear analyses is significantly less than the frequently assumed value of 5%.

Final design of isolators may in some cases be done using simplified methods from AASHTO. When the conditions required in order for these simplified procedures are not met, the steps in this example may be used as a guide in developing loads for design and structural models of the bridge.

6 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 30 meters of the soil profile. The Mississippi Emabyment consists of profiles 1,000 meters and more in thickness and research has shown that the generic site factors may be un-conservative 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 alternative means of developing site factors for critical structures are given here.

1. Work at the University of Memphis (Malekmohammadi and Pezeshk 2014) proposed a site amplification model of the form given by Equations 68a-68e. The coefficients are reproduced here in Figure 19.

$$ln(F) = f_{base} + f_{depth} + f_{geology} \tag{68a}$$

$$f_{base} = a_1 ln\left(\frac{V_{S30}}{a_2}\right) - a_3 ln(PGA_{rock} + a_4) + a_3 ln\left[PGA_{rock} + a_4\left(\frac{V_{S30}}{a_2}\right)^{a_5}\right] \tag{68b}$$

$$f_{depth} = a_6 ln\left[\frac{Z_{3000} + a_7}{exp\left[a_8 - ln\left(\frac{V_{S30}}{a_9}\right)\right]}\right] + [a_{10} ln(PGA_{rock}) + a_{12}V_{S30}] \times |lnZ_{3000} - a_{11}| \tag{68c}$$

$$f_{depth} = \frac{a_{14} \times ln(PGA_{rock}) + a_{13}}{lnZ_{3000}}\ , Uplands\ profiles \tag{68d}$$

$$f_{depth} = 0\ , Lowlands\ profiles \tag{68e}$$

2. Figure 20 is reproduced from work at the University of Illinois at Urbana-Champaign (Y. M. Hashash, et al. 2008).
3. The QWL (quarter-wave-length) method as described in (Atkinson and Beresnev 2002), (Boore and Joyner 1997), may be useful in developing site factors.

Period (sec)	Base Model					Depth Model							Geology Model	
	a_1	a_2	a_3	a_4	a_5	a_6	a_7	a_8	a_9	a_{10}	a_{11}	a_{12}	a_{13}	a_{14}
PGA	1.382	1897	-0.686	1.901	1.6176	-0.652	517	8.78	96.1	-0.03585	0.014	0.00005	-1.05298	-0.18068
0.2	2.629	1651	-2.035	3.500	0.8387	-1.271	900	10.09	100.4	-0.06325	3.341	-0.00004	-1.55272	-0.24581
1	1.804	893	-2.607	1.221	0.8176	0.200	878	-0.79	281.1	0.00275	4.933	-0.00058	-0.95441	-0.38725
5	0.970	632	-2.821	1.893	0.5614	0.751	162	5.81	83.8	-0.00019	2.118	-0.00027	0.10866	0.00435

Figure 19. UM Alternative Site Factor Coefficients (Malekmohammadi and Pezeshk 2014)

Table 8

Recommended Site Coefficient for the UME, F_a as a Function of Soil Profile Thickness and Mapped 0.2 sec Period Spectral Acceleration from 2002 USGS Hazard Maps

Thickness (m)	$S_s = 0.25g$			$S_s = 0.50g$			$S_s = 0.75g$			$S_s = 1.0g$			$S_s \geq 1.25g$		
	NEHRP	Upland	Lowland	NEHRP	Upland	Lowland	NEHRP	Upland	Lowland	NEHRP	Upland	Lowland	NEHRP	Upland	Lowland
30	1.6	1.46	1.41	1.4	1.32	1.27	1.2	1.18	1.13	1.1	1.11	1.06	1.0	1.06	1.01
100	1.6	1.41	1.31	1.4	1.27	1.17	1.2	1.13	1.03	1.1	1.06	0.96	1.0	1.01	0.91
200	1.6	1.36	1.21	1.4	1.22	1.07	1.2	1.08	0.93	1.1	1.01	0.86	1.0	0.96	0.81
300	1.6	1.31	1.11	1.4	1.17	0.97	1.2	1.03	0.83	1.1	0.96	0.76	1.0	0.91	0.71
500	1.6	1.27	1.06	1.4	1.13	0.92	1.2	0.99	0.78	1.1	0.92	0.71	1.0	0.87	0.66
1000	1.6	1.23	1.04	1.4	1.09	0.90	1.2	0.95	0.76	1.1	0.88	0.70	1.0	0.83	0.64

Table 9

Recommended Site Coefficient for the UME, F_v as a Function of Soil Profile Thickness and Mapped 1.0 sec Period Spectral Acceleration from 2002 USGS Hazard Maps

Thickness (m)	$S_l = 0.10g$			$S_l = 0.20g$			$S_l = 0.30g$			$S_l = 0.40g$			$S_l \geq 0.50g$		
	NEHRP	Upland	Lowland	NEHRP	Upland	Lowland	NEHRP	Upland	Lowland	NEHRP	Upland	Lowland	NEHRP	Upland	Lowland
30	2.4	2.4	2.4	2.0	2.00	2.00	1.8	1.80	1.80	1.6	1.60	1.60	1.5	1.50	1.50
100	2.4	2.7	2.55	2.0	2.3	2.15	1.8	2.10	1.95	1.6	1.95	1.75	1.5	1.80	1.65
200	2.4	2.85	2.67	2.0	2.45	2.27	1.8	2.25	2.07	1.6	2.08	1.87	1.5	1.91	1.77
300	2.4	2.95	2.77	2.0	2.55	2.37	1.8	2.37	2.17	1.6	2.18	1.97	1.5	2.01	1.87
500	2.4	3	2.82	2.0	2.60	2.42	1.8	2.42	2.22	1.6	2.23	2.02	1.5	2.06	1.92
1000	2.4	3.05	2.87	2.0	2.65	2.47	1.8	2.47	2.27	1.6	2.28	2.07	1.5	2.08	1.97

Figure 20. UIUC Alternative Site Factors for Deep Soil Sites (Y. M. Hashash, et al. 2008)

Certain critical projects may require site-response analysis rather than simple site factors, code-based or otherwise. Software which may be useful for site response analyses includes Strata (Kottke and Rathje 2008) and DeepSoil (Hashash, et al. 2012) among others. Site response analyses may be either time-domain based, requiring several ground motion acceleration histories as the loading input, or frequency-domain random vibration theory (RVT) based, requiring only a response spectrum – either Fourier amplitude or spectral acceleration – as the loading input. For time-domain site response analysis, the input records should be at the base bedrock. SeismoArtif (SeismoSoft 2013) may be useful in developing a series of bedrock input motions. For frequency-domain site response analysis, the input acceleration response spectrum should be the PGA, S_S, and S_1 accelerations from AASHTO/SGS with no site factors applied. Site response analysis will require the specification of each of the following parameters.

1. Shear modulus reduction model
2. Small strain damping ratios
3. Shear wave velocity profile
4. Density profile
5. Bedrock shear wave and density values.

The literature on site response analysis in the Mississippi Embayment has become much more extensive in recent years. Each of the following cited works should be considered in performing site response analysis.

- (Fernandez and Rix 2006)
- (Y. M. Hashash, et al. 2008)
- (Hashash and Park 2001)
- (Hashash, et al. 2014)
- (Park 2004)
- (Park and Hashash 2005)
- (Park and Hashash 2004)
- (Hashash and Pezeshk 2004)
- (Saikia, Pitarka and Ichinose 2006)

Two options for small strain damping values are found in (Park and Hashash 2005), from which Figure 21 is taken. The back-calculated curve is specific to the Mississippi

Emabyment based on actual recorded ground motions for small magnitude events in the NMSZ. Representative Lowlands, Uplands, and deep profiles are taken from (Park and Hashash 2005), (Fernández 2007) and shown in Figures 22 through 25. Values for bedrock shear wave velocity and density should be taken as 3,000 m/s (9,843 fps) and 2.6 g/cm^3 (162 pcf), respectively. These values are in agreement with those found in the cited literature. Based on data found in (Fernández 2007), soil density may be assume to vary from 1.9 g/cm^3 (119 pcf) at the surface to 2.25 g/cm^3 (140 pcf) at a depth of 300 meters, at which point the density may be assumed approximately constant to the base of the soil profile (See Figure 26). Shear modulus reduction curves may be taken from (Darandeli 2001) or (Park and Hashash 2005). As noted by Park and Hashash, (Hashash and Park 2001), (Park and Hashash 2005), the hyperbolic modulus reduction model given by Equation 69 and a pressure-dependent alternative small strain damping model as given by Equation 72 may be applicable in the Mississippi Embayment. The constants used in the study are as follows:

$$\beta = 1.4$$

$$s = 0.8$$

$$\sigma_{ref} = 0.18 \text{ MPa}$$

$$a = 0.163$$

$$b = 0.630$$

$$c = 1.50$$

$$d = 0.30$$

$$\frac{G}{G_{max}} = \frac{1}{1 + \beta \left(\frac{\gamma}{\gamma_r}\right)^s} \qquad (69)$$

$$G_{max} = \rho v_s^2 \qquad (70)$$

$$\gamma_r = a \left(\frac{\sigma'}{\sigma_{ref}}\right)^b \qquad (71)$$

$$\xi = \frac{c}{(\sigma')^d} \qquad (72)$$

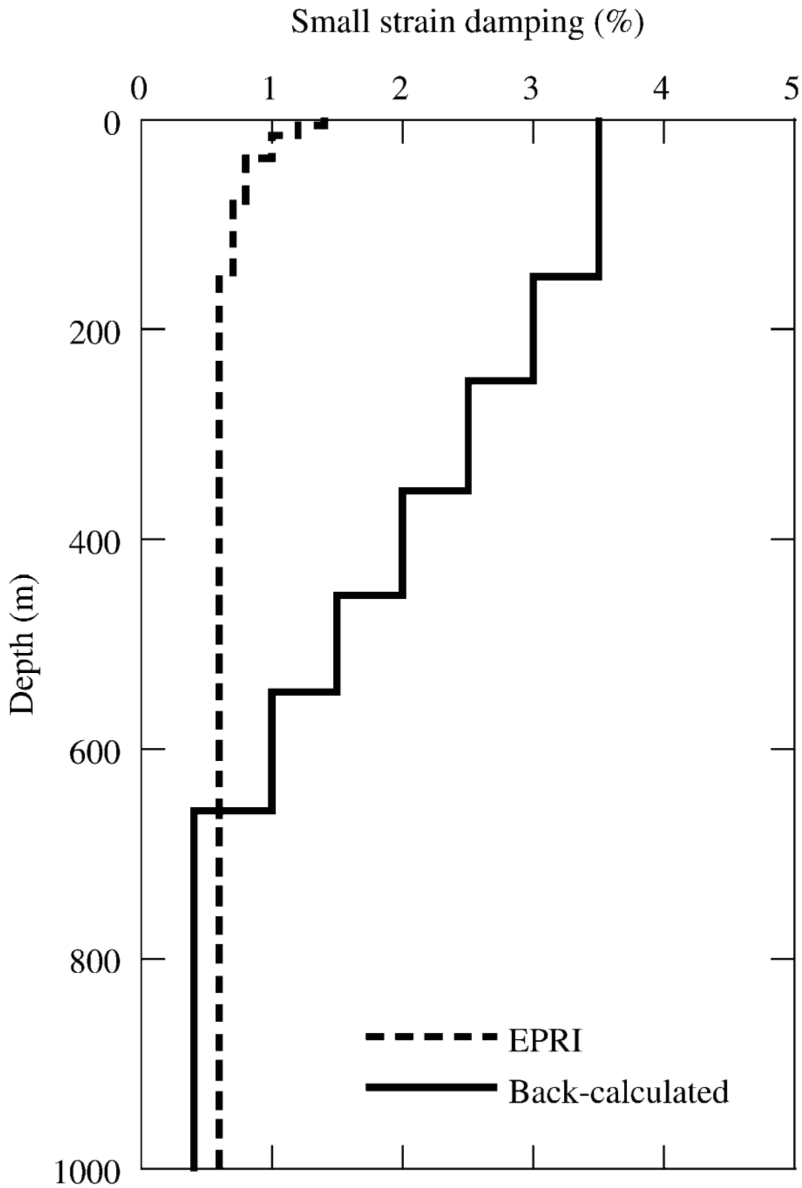

Figure 21. Small Strain Damping Ratio (Park and Hashash 2005)

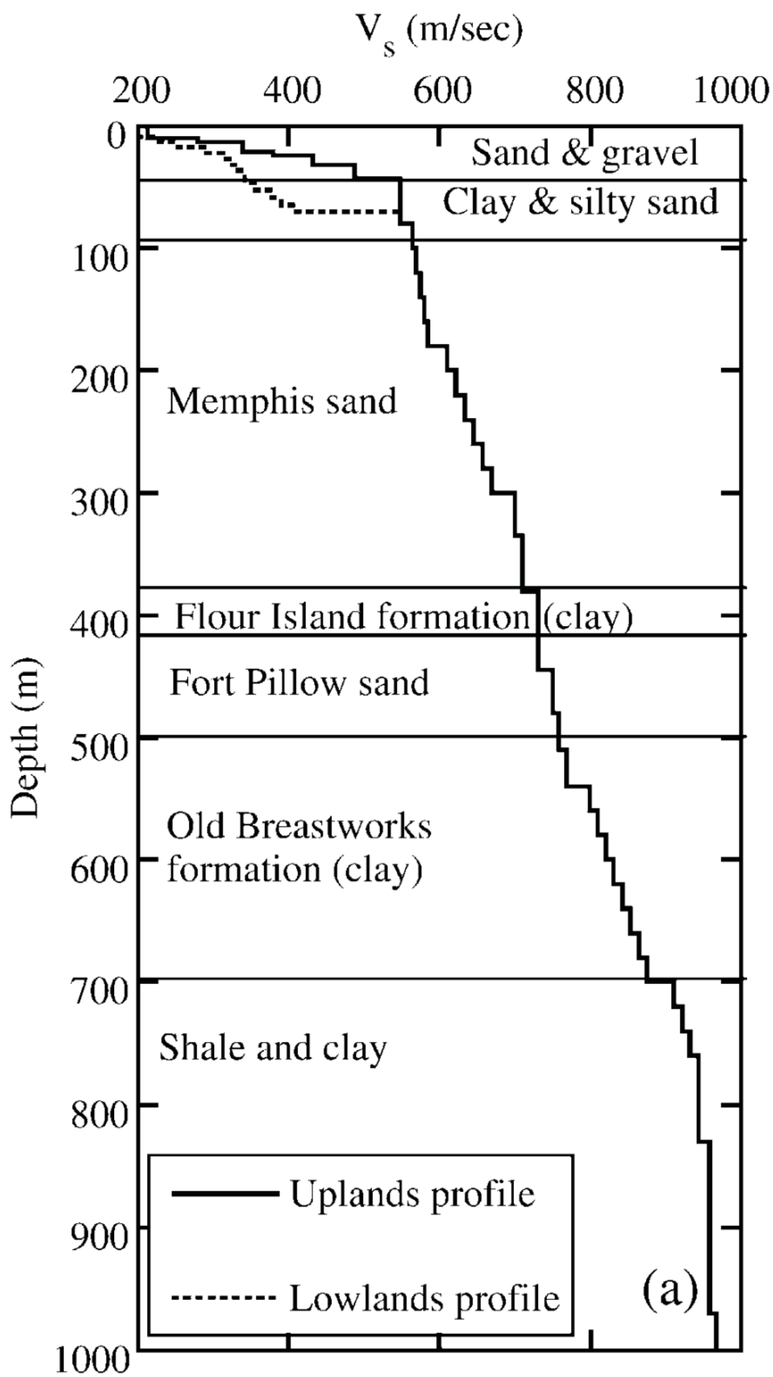

Figure 22. Shear Wave Velocity - Full Profiles in the ME (Park and Hashash 2005)

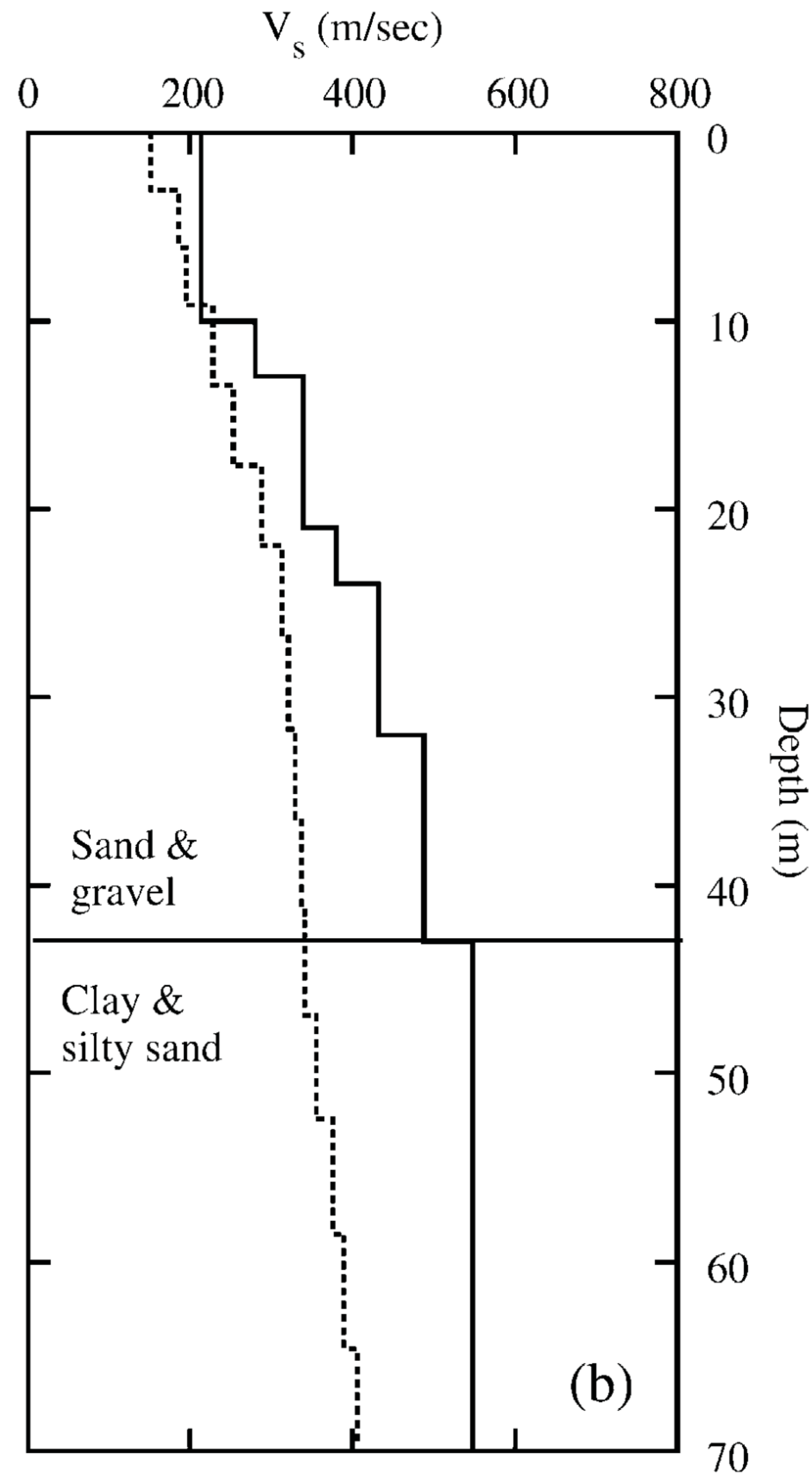

Figure 23. Shear Wave Velocity Profile - Upper 70 meters (Park and Hashash 2005)

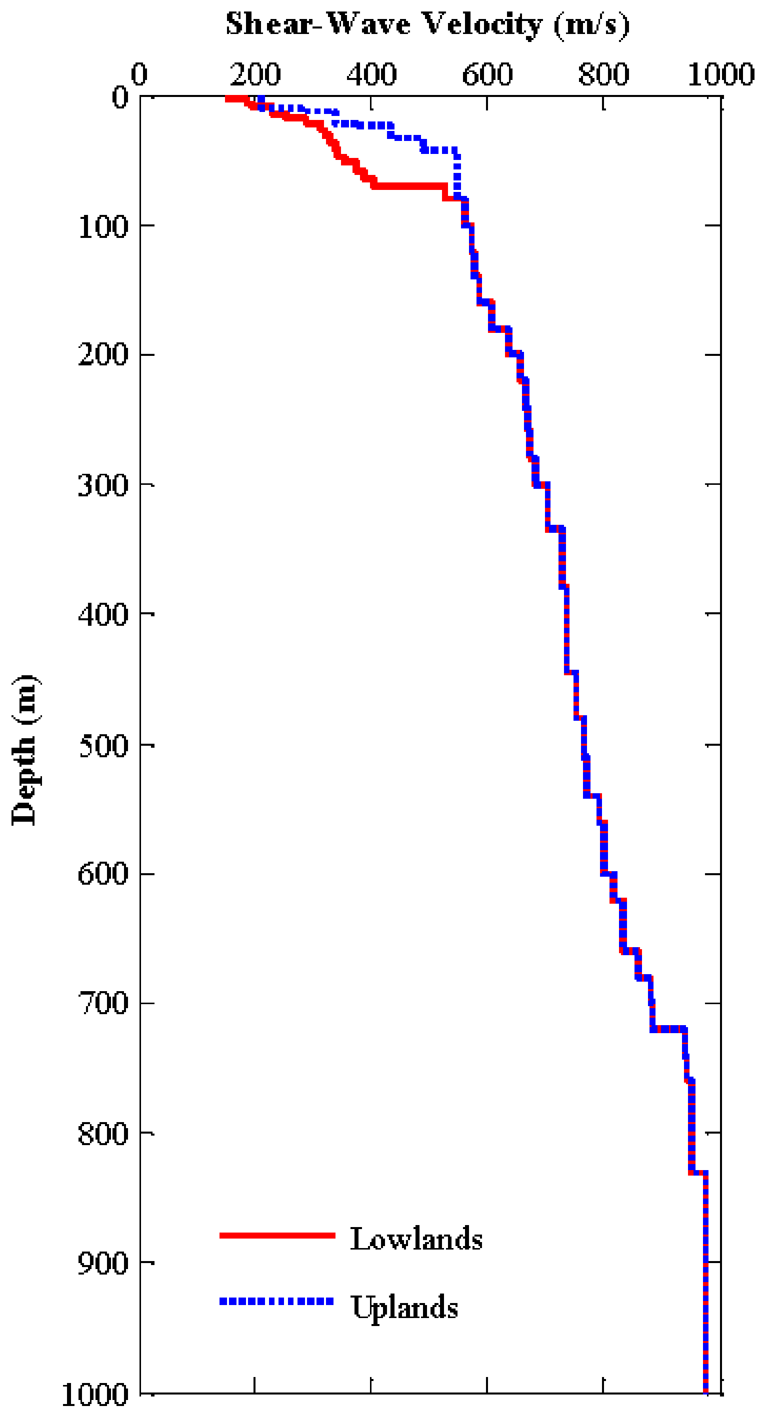

Figure 24. Shear Wave Velocity - Full Profile – ME (Fernández 2007)

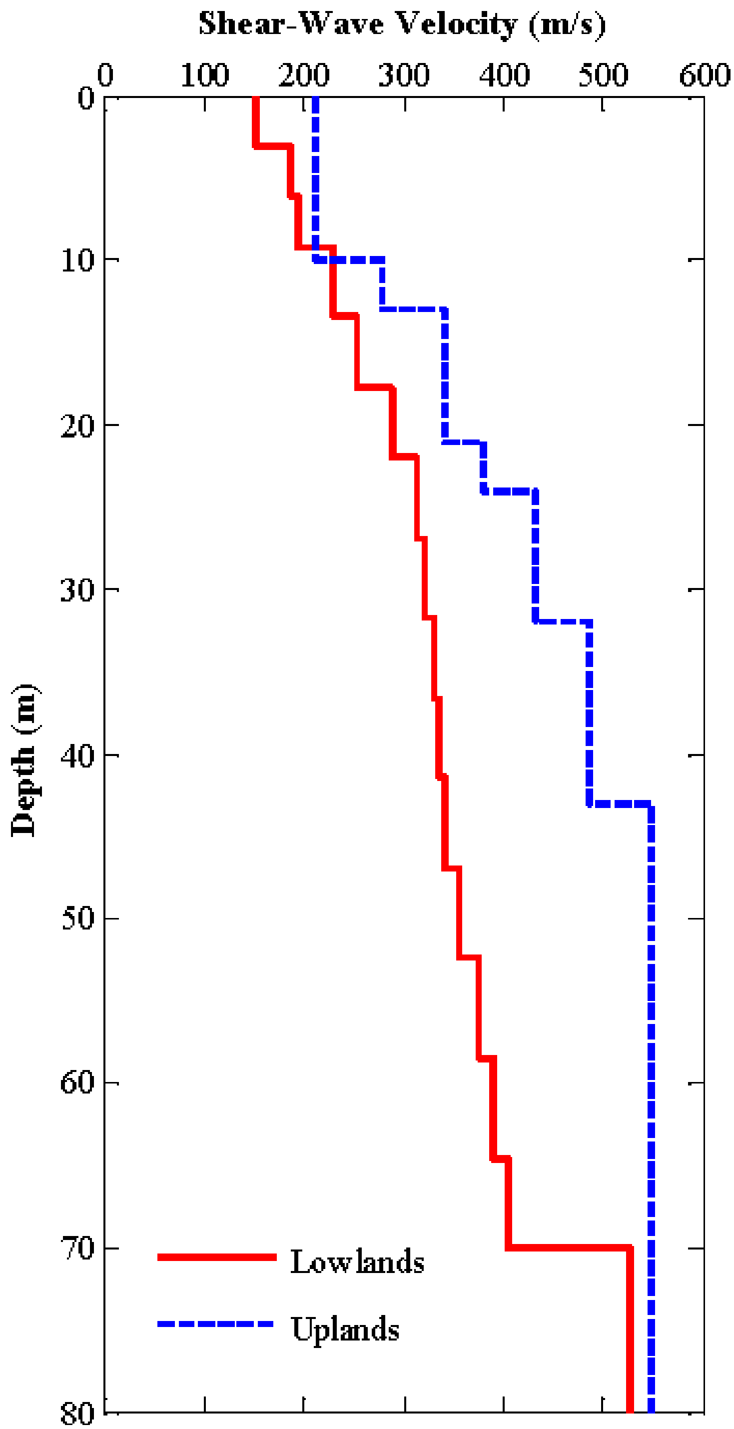

Figure 25. Shear Wave Velocity - Upper 70 meters (Fernández 2007)

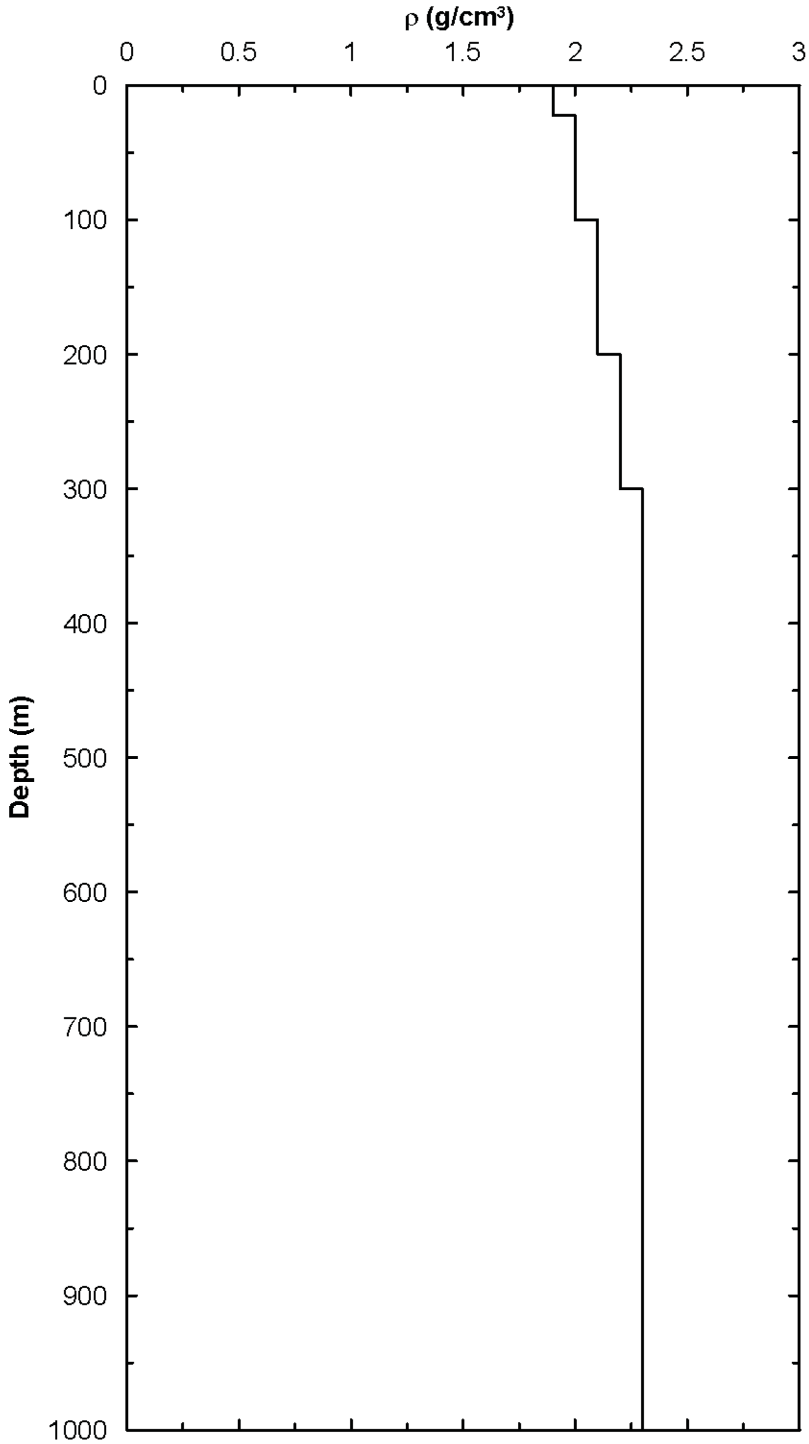

Figure 26. Density Profile (Romero and Rix 2005)

Appendix: 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 spiral

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

ε_{SU}: ultimate tensile strain of transverse bars

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

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

ε_{y}: yield strain of longitudinal reinforcing bars

ϕ_y: yield curvature

ϕ_u: ultimate curvature; the smaller of $\phi_{dc,c}$ and $\phi_{dc,s}$

$$L_{SP} = 0.15 f_{ye} d_{bl} \tag{73}$$

$$k = 0.2\left(\frac{f_u}{f_y} - 1\right) \leq 0.08 \tag{74}$$

$$L_P = kL_C + L_{SP} \geq 2L_{SP} \tag{75}$$

$$\rho_v = \frac{4A_v}{D's}\ , Circular\ hoops\ and\ spirals \tag{76}$$

$$\rho_v = \frac{n_x A_v}{h_{cy} s} + \frac{n_y A_v}{h_{cx} s}\ , Rectangular\ ties \tag{77}$$

$$f_l = 0.5 C_e \rho_v f_{yh} \tag{78}$$

$$f'_{cc} = f'_c\left(2.254\sqrt{1 + \frac{7.94 f_l}{f'_c}} - 2\frac{f_l}{f'_c} - 1.254\right) \tag{79}$$

$$\varepsilon_{dc,c} = 0.004 + 1.4\frac{\rho_v f_{yh} \varepsilon_{su}}{f'_{cc}} \tag{80}$$

$$\frac{c}{D} = 0.20 + 0.65\frac{P_u}{f'_{ce} A_g} \tag{81}$$

$$\phi_{dc,c} = \frac{\varepsilon_{dc,c}}{c} \tag{82}$$

$$\phi_{dc,s} = \frac{\varepsilon_{dc,s}}{d - c} \tag{83}$$

$$\phi_y = 2.25\frac{\varepsilon_y}{D} \tag{84}$$

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