20.24 NONLINEAR TIME HISTORY ANALYSIS OF BRIDGES

20.24.1 GENERAL

This Bridge Design Memo (BDM) provides guidance for nonlinear time history analysis of bridges and conforms to Section 4.2.3 of the Caltrans Seismic Design Criteria (SDC). For additional requirements not covered herein, refer to the SDC.

20.24.2 DEFINITIONS

Refer to the SDC for definitions.

20.24.3 INPUT GROUND MOTION

For Nonlinear Time History Analysis (NTHA), seismic motion can be applied as either acceleration or displacement time histories at the base of bridge structures. The acceleration time history is the typical input motion. The displacement time history is used as the input motion only for the exceptional cases that require analysis of multi-support excitation.

For site conditions that require site-specific response analysis specified in Appendix B of the SDC, refer to the Geotechnical Manual.

For site conditions that do not need site-specific response analysis specified in Appendix B of the SDC, the procedures to develop acceleration time histories are as follows:

1. Obtain the recommended design acceleration response spectrum (ARS).
2. Identify the parameters for the selection of ground motion time-histories, including:
   - Fault type (strike-slip, normal, or reverse)
   - Shear wave velocity (V_{s30})
   - Deaggregated mean earthquake moment magnitude (M)
   - Mean site-to-fault rupture distance (R) for the 1.0 second period design spectral acceleration.
3. Generate or select input motions.
   a) Generate 50 synthetic ground motion records using the generation tool, which is based on the Dabaghi and Kiureghian (2014) method. The parameters identified in Step 2 should be used to generate the records, and each record should include both the major component and intermediate component.
   b) Select ground motion records from past seismic events based on properties identified in Step 2. The records can be found on the Pacific Earthquake Engineering Research Center (PEER) website.

4. Select seven ground motion records from step 3 that best match the design ARS at the fundamental period or at periods that significantly influence the dynamic characteristics of the bridge.

Vertical acceleration effects should be considered according to the SDC using static loads.

20.24.4 STRUCTURAL MODELING AND ANALYSIS

One of the most critical issues in NTHA is the preparation of an analytical model that captures the bridge nonlinear responses accurately. Model discretization, element types, material properties, assignment of mass, and definition of support conditions can all affect the model response. The model should capture the nonlinear behavior of Earthquake Resisting Elements (EREs) and other potential sources of nonlinearity, including but not limited to in-span hinges, shear keys, and soil-foundation interaction.

Software such as CSiBridge and Midas Civil can be used to perform NTHA. Figure 20.24.4-1 shows a 3D spine model with components defined for instructions in the following sections.

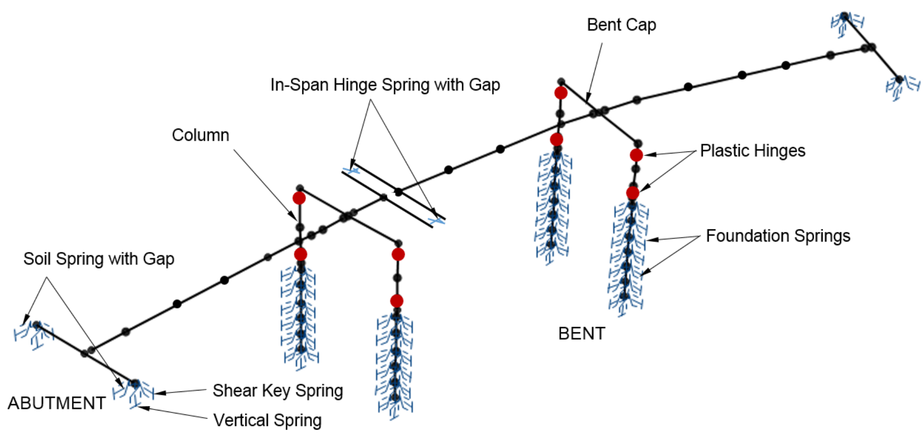


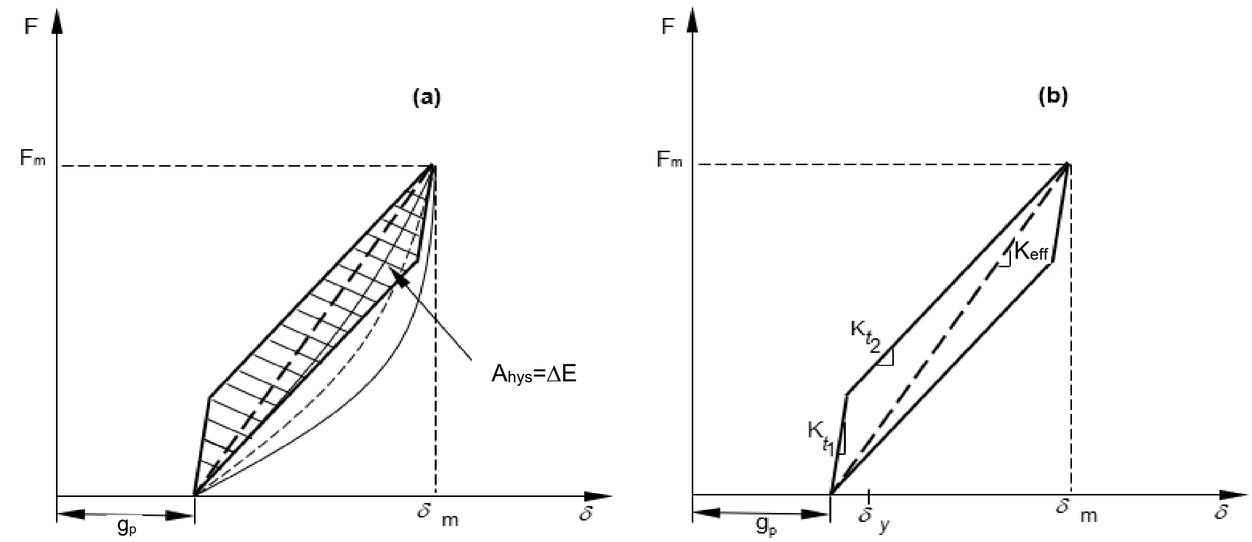
Figure 20.24.4-1 Illustration of a Bridge Model with Detailed Modeling Techniques for Components

20.24.4.1 Superstructure

Superstructure should be designed to remain essentially elastic during seismic events. Therefore, a linear elastic beam-column element is used to model the superstructure. The structure needs to be modeled in three-dimensional space, and the elements should follow the superstructure alignment. Typically, a minimum of four elements per span is used to capture the geometric variations and mass distribution. In a typical bridge, a nodal lumped mass model is sufficient, and rotational mass (mass moment of inertia) may be ignored. For additional information on how to calculate cross-sectional properties for time history analysis, see Aviram et al. (2008).

20.24.4.2 In-Span Hinges

Superstructure in-span hinges provide moment release and axial separation between frames, but they can also close during a seismic event. Therefore, an element that can model a gap and impact on gap-closure is necessary. Relative rotation of the superstructure on the two sides of the hinge causes impact on one edge and opening on the other edge of the deck. One nonlinear element should be placed at each exterior girder (a total of two nonlinear elements per hinge). Accurate modeling of hinge effects is critical in skewed bridges. Impact stiffness should be chosen to provide rigid behavior while avoiding numerical convergence problems. An example of a simplified bilinear truss contact model to represent impact between closely spaced adjacent structures is shown in Figure 20.24.4.2-1.



(a) (b)

Figure 20.24.4.2-1 (a) Inelastic truss contact element for impact simulation, (b) Parameters of the inelastic truss model: the initial stiffness K_{t1}, strain hardening stiffness, K_{t2}, and yield deformation δ_y [Muthukumar, 2003]

20.24.4.3 Bent Cap

In a spine model where the entire superstructure is modeled as a single line of elements, the connection between the superstructure and the cap beam is limited to one node. Therefore, the cap element should be stiff enough to avoid excessive and unrealistic bending in the model. However, the mass of the bent cap should still be included. The cap element is also expected to remain elastic in a seismic event and thus may be modeled as a linear elastic beam element or as a rigid link.

20.24.4.4 Column

Columns should be modeled as inelastic beam-column elements with a minimum of three elements.

Columns are usually designed to undergo considerable nonlinear deformation. Therefore, it is essential to have a relatively accurate model of the nonlinear behavior of each column.

The fiber-hinge model can be used for an accurate and stable model, which is also computationally efficient enough to be used in practice (Aviram et al., 2008, and Omrani et al., 2015). A fiber-hinge is defined based on the cross-sectional geometry and typically includes three types of materials, which are cover (unconfined concrete), core (confined concrete), and reinforcing steel for longitudinal rebar.

The cover concrete should be designed as a nonlinear material that can reach its ultimate capacity and spall. The core concrete should also be a nonlinear material that can capture the compressive stress-strain curve of the material with sufficient accuracy. The reinforcing steel plays a crucial role in the hinge capacity. It is recommended to use a material model that can capture the strain hardening behavior of steel. While an elastic-perfectly plastic model (bilinear) model is accurate at lower strain levels, it may underestimate forces in steel at high strains. Therefore, if a strain hardening material model is available, then it should be used. Otherwise, an elastic-perfectly plastic model can be employed as a conservative alternative.

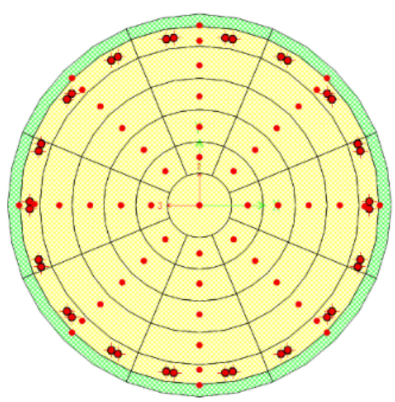


Figure 20.24.4.4-1 Fiber Hinge Modeled by CSiBridge Section Designer

A fiber-hinge model should have enough concrete fibers to provide acceptable accuracy. A minimum of eight fibers in both radial and circumferential directions for circular columns and in longitudinal and transverse directions for rectangular columns is recommended.

If a lumped plasticity is selected, the plastic hinge element should be located at the center of the plastic hinge length at both the base and the top of the column to be compatible with the SDC. The elastic properties of the column element are based on the gross cross-section, except for the moment of inertia, which is based on effective section properties associated with dead load axial loads.

A bidirectional moment-rotation model (P-M-M) may also be used for more efficient computational speed; however, it is considered less accurate than a fiber-hinge model.

20.24.4.5 Abutment

Abutments may be modeled as simple linear elastic beam-column elements, with stiffness models discussed in SDC Section 4.3.1. The soil stiffness should be estimated based on the SDC, and for seat abutments, the gap value is based on the designed expansion joint opening. The abutment torsion may be modeled as fixed, as it is expected to be rigid enough to limit the superstructure torsion. However, for a skewed support, the abutment is to be modeled as two sets of springs at the edges of the superstructure.

20.24.4.6 Support Conditions at Base of Columns and Piles/shafts

Column bases may be modeled as pinned, fixed, or linear springs unless considerable soil/foundation nonlinearities are expected. In that case, a set of multilinear plastic springs should be used to model the foundation.

Soil-foundation-structure interaction (direct modeling of the shafts and pile groups using nonlinear p-y's, t-z's, q-z's) between the shaft/pile and the surrounding soil should be modeled using nonlinear springs.

Horizontal springs (p-y) are included as lumped springs spaced at maximum intervals equal to the pile diameter along the length of the pile in two orthogonal directions and may have different stiffness or capacity in each direction. This model will result in higher stiffness in directions other than the two spring directions, but the approximation is usually acceptable. Some software can model coupled horizontal springs and result in a uniform response in all directions. Use this option if it is available. Vertical springs (t-z) should be modeled as nodal springs along the length of the pile. A single spring at the base of the shaft/pile (q-z) may be used to model the end bearing and act in compression only.

Typically, it is acceptable to use the uniform acceleration time history along the length of the pile. However, different input motions may be necessary, depending on the soil properties and the pile length. The analysis should then be performed with the multi-support excitation model using displacement time history records.

20.24.4.7 Isolation Bearing

Seismic isolation bearings are EREs with energy dissipation that limit the transferred inertial forces between the superstructure and substructure. The concept of isolation bearings is to increase the fundamental period by increasing the flexibility of the structure, thereby reducing seismic-induced forces.

Isolation bearing devices are modeled as nonlinear link elements at the locations of bearings between the superstructure and substructure or between the abutment seat and superstructure. Isolation bearing spring initial, post elastic, and effective stiffnesses are defined from the performance curve of the device and should be used to define the isolation bearing spring link element.

For additional requirements on isolation bearings, refer to BDM 20.33 *Seismic Design of Bridges with Isolation Bearings*.

20.24.4.8 Damping

NTHA captures some energy dissipation through the hysteretic behavior of plastic hinges and nonlinear springs. The model may also include specific damper elements to account for damping devices. In addition, Rayleigh damping, also known as mass and stiffness proportional damping, is commonly used to model general damping. Most software programs offer users the option to specify damping ratios at two periods to calculate the mass and stiffness proportional damping coefficients. A common range of damping ratio for Rayleigh damping is 2% to 4%; therefore, in the absence of better information, an average ratio of 3% is recommended for standard bridges per SDC. The two periods are chosen to cover the range of periods that contribute to the response of the structure. The two periods should be obtained from the modal analysis of a linear version of the model. One period is the first modal period of the structure (T1), and the other period (T2) is associated with the mode shape that achieves 80% mass modal participation in horizontal directions, as shown in Figure 20.24.4.8-1.

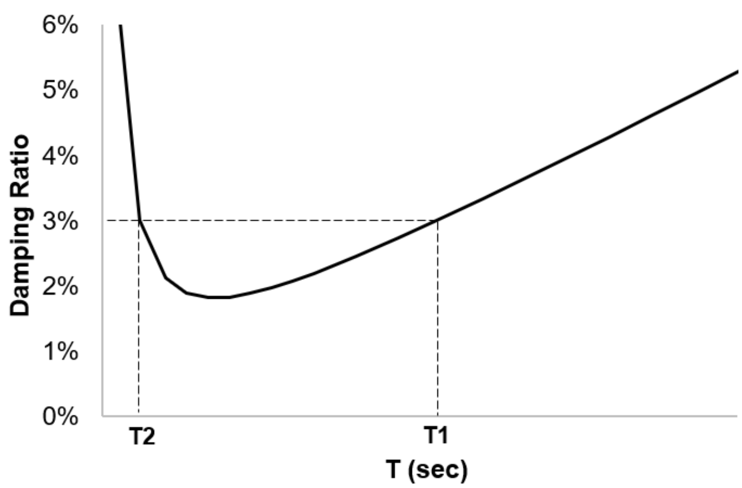


Figure 20.24.4.8-1 Example of Rayleigh Damping Curve

20.24.5 REFERENCES

1. AASHTO. (2017). *AASHTO LRFD Bridge Design Specifications*, 8th Edition. American Association of State Highway and Transportation Officials. Washington, DC.
2. Aviram, A., Mackie, K. R. & Stojadinovic, B. (2008). "*Guidelines for Nonlinear Analysis of Bridge Structures in California*", Pacific Earthquake Engineering Research Center Research Report No. PEER 2008/03, University of California, Berkeley, CA.
3. Caltrans. (2019). *California Amendments to AASHTO LRFD Bridge Design Specifications, 8th Edition*, California Department of Transportation, Sacramento, CA.

4. Caltrans. (2025). *Caltrans Seismic Design Criteria,* Version 2.1, California Department of Transportation, Sacramento, CA.
5. Caltrans. (2025). Bridge Design Memo 20.33 *Seismic Design of Bridges with Isolation Bearings*, California Department of Transportation, Sacramento, CA.
6. Dabaghi, M. and Kiureghian, A. D. (2014). *Stochastic Modeling and Simulation of Near-Field Ground Motions for Performance-Based Earthquake Engineering*, Pacific Earthquake Engineering Research Center Research Report No. PEER 2014/20, University of California, Berkeley, CA.
7. Muthukumar, S. (2003). *A Contact Element Approach with Hysteresis Damping for the Analysis and Design of Pounding in Bridges*. Ph.D. Dissertation, Department of Civil and Environmental Engineering, Georgia Institute of Technology, Georgia.
8. Omrani, R., Mobasher, B., Liang, X., Gunay, S., Mosalam, K. M., Zareian, F., and Taciroglu, E. (2015). *Guidelines for Nonlinear Seismic Analysis of Ordinary Bridges*, UCLA/CA15-65A0454, University of California, Los Angeles, CA.