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Nonlinear Modeling of Bridge Structures in California

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Synopsis: This paper presents a collection of practical and readily implementable recommendations for the modeling of highway bridges and overpasses subjected to earthquake ground motions. The specifications were developed particularly for Ordinary Standard Bridges in California as defined according to the Caltrans Seismic Design Criteria. Bridge components that require special modeling considerations and nonlinear characterization are identified in this paper, establishing specific criteria for the level of sophistication required. To reduce possible errors that arise during modeling and analysis of bridge structures using a particular structural analysis program, a comparison between bridge models using SAP2000 and OpenSees analysis packages was carried out to assess sensitivities and characterize important modeling parameters. Comparisons were made between the two software packages using modal, pushover and nonlinear time history analyses. A total of six typical reinforced concrete bridges in California with box-girder superstructure and different geometries and cross sections were considered. Inconsistencies between the two analysis packages were found for peak displacements obtained through nonlinear time history analysis. Two methods of obtaining response estimate bias factors between the two programs are illustrated for the six bridges analyzed under three seismic hazard levels (50%-, 10%-, and 2%-in-50-year probabilities of exceedance).

Keywords: abutment; bridge nonlinear modeling; column plastic hinge; response bias.

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INTRODUCTION

The seismic demands on a bridge structure subject to a particular ground motion can be estimated through an equivalent analysis of a mathematical model that incorporates the behavior of the main structural components of the bridge, among them are the superstructure, piers, footings, and soil system. To achieve confident results for a variety of earthquake scenarios, the idealized model should properly represent the actual geometry, boundary conditions, gravity load, mass distribution, energy dissipation, and nonlinear properties of these major bridge components. Nonlinear modeling and analysis of a bridge structure is considered a suitable approach for the determination of stresses, strains, deformations, forces and displacements of critical elements, results that are used for the final design of the bridge subsystems or the evaluation of the bridge global capacity and ductility.

This paper presents the main recommendations related to inelastic modeling of bridge systems in California and other high seismicity zones. Additional recommendations for the implementation of nonlinear analysis procedures used primarily for demand estimation on critical bridge components and systems are presented in [1]. In current standard design practice of bridge structures, typical linear and nonlinear analysis methods such as pushover and response spectrum analyses are widely used, particularly for two-dimensional bridge subassemblies. With the advancement in structural analysis tools, more complex global three-dimensional nonlinear dynamic analysis is increasingly used in the design practice to achieve higher accuracy in bridge response estimation. Development of modeling guidelines in this paper intended to assist practicing engineers to perform advanced nonlinear analysis of bridge structures was carried out through a rigorous comparison of nonlinear analysis results. The results were obtained from several bridge models of Ordinary Standard bridges as defined according to Caltrans Seismic Design Criteria (SDC) [2] to obtain a wide range of bridge geometries and cross sections. Different structural analysis programs such as SAP2000 NL by CSI [3], OpenSees by UC Berkeley [4], XTRACT by Imbsen [5], and xSECTION by Caltrans [6], among others, were used to evaluate the accuracy and applicability of the guidelines. An extended literature review of the current engineering practice and code criteria for bridge design, modeling and analysis was carried out concurrently to ensure consistency of the proposed guidelines with the accepted practice.

This paper and the complete comprehensive report [1] present recommendations for linear and nonlinear modeling of bridge structures appropriate for any structural analysis program, as well as specific details on the use of SAP2000 for such procedures. A three-dimensional (3D) model of the structural system is recommended to capture the response of the entire bridge system and the interaction of individual components under specific seismic demand characteristics. Table 1 summarizes the recommended criteria for inelastic modeling of the primary elements comprising an Ordinary Standard Bridge structure. Specific and detailed modeling recommendations for each component are offered. Some general recommendations can be extended to Ordinary Non-standard Bridges and Important Bridges as defined according to SDC [2], where more rigorous and advanced nonlinear analysis is required due to geometric irregularities of the bridge structure including curves and skew, long spans or significant total length, multiple expansion joints, massive substructure components, or unstable soil conditions.

RESEARCH SIGNIFICANCE

The role of this study is to provide the engineering and research communities practical and readily implementable recommendations for accurate and standardized nonlinear modeling of bridge structures in California. This study recognizes the importance of reconciling the differences between different structural analysis platforms, one commonly used in industry and one commonly used in research, in a quantifiable manner. The guidelines were developed following probabilistic performance-based evaluations of different bridge models and an extended literature review of the current state of practice and state of the art in bridge modeling, and are therefore well suited for deployment in a performance-based engineering setting.

BRIDGE MODELING

Coordinate system

The coordinate system used for the modeling and analysis of the bridge is shown in Fig. 1. The global *X*-axis is in the direction of the chord connecting the abutments, denoted as the longitudinal direction; the global *Y*-axis is orthogonal to the chord in the horizontal plane, representing the transverse direction; while the global *Z*-axis defines the vertical direction of the bridge. For the analysis and design of elements of the bridge using two-noded elements, a local coordinate system is used, as shown in Fig. 1.

Node and element definition

The bridge model is a three-dimensional spine model of the bridge structure with line elements located at the centroid of the cross section, following the alignment of the bridge. Three-dimensional beam-column elements (line or frame elements) with corresponding cross-sectional properties are used to represent the superstructure and the components of the bents (columns and cap beams). ATC-32 [7] suggests a minimum of three elements per column and four elements per span shall be used in a linear elastic model. However, it is recommended for all analysis cases that the superstructure, cap beam, and column bents be discretized using a minimum of five elements of equal length, except for spans with intermediate hinges or expansion joints. This discretization helps approximate the distributed (translational) mass of the bridge components using lumped masses at the nodes.

Material properties

The expected material strength and stress-strain (σ–ε) relation should be used for unconfined and confined concrete, as well as reinforcing steel. The reinforcement details of the piers and other major bridge components are required. The properties of normal weight Portland Cement Concrete should be applied according to section 3.2.6 of SDC [2], and the Mander model [8] is to be used to represent the uniaxial stress-strain behavior for unconfined and confined concrete. It is recommended that the concrete tensile strength for both confined and unconfined concrete be included since the initial stiffness of RC columns can be significantly altered due to tension-stiffening. The tensile strength is estimated by ACI 318 as $f_t = 7.5\sqrt{f_c'}$ (psi) or $f_t = 0.6\sqrt{f_c'}$ (MPa) for normal weight concrete, defined with an initial modulus of elasticity Ec according to section 3.2.6 of SDC [2]. The properties of the steel reinforcement are according to sections 3.2.2 and 3.2.3 of SDC [2] Guidelines for Steel ASTM A-706 with symmetrical behavior in tension and compression. The material and mass properties for all load cases other than seismic should be selected to comply with the AASHTO LRFD Specifications, 3rd Edition. The definition of the σ–ε relation in SAP2000 must be carried out with a sufficient number of points in the curve to accurately capture the nonlinear behavior of the materials, specifically the yield, ultimate, and failure points, the degradation of strength beyond the maximum point in confined and unconfined concrete, and the variation in the strain-hardening slope in the reinforcing steel.

Translational mass and mass moment of inertia

The weight of normal concrete is specified by SDC [2] section 3.2.6 as ω_c= 144 lb/ft^3 (2300 kg/m^3) and therefore a mass of ρ_c= 4.5 lb-sec^2/ft^4 (230 kg-sec^2/m^4) is to be used when specifying material properties for confined and unconfined concrete. To account for lost formwork in box-girder cells, the unit weight of normal concrete superstructures can be taken as 150 lb/ft^3 (2400 kg/m^3). The weight of other permanent nonstructural accessories and components of the bridge must also be considered in the estimation of the bridge mass. It is desired to approximate all bridge elements with a distributed mass along their length. However, most analysis software packages (such as SAP2000) automatically calculate the translational mass of all longitudinal elements in the 3 global directions of the bridge (longitudinal, transverse, and vertical) and assign them as lumped mass at each node, based on tributary lengths.

Additional assignment of rotational mass (mass moment of inertia) is required, particularly for the superstructure of a spine model of the bridge, since it is not generated automatically in SAP2000 and other structural analysis programs, to represent with greater accuracy the dynamic response and fundamental modes of the bridge associated with the transverse direction. Significant differences in the transverse and global torsion modes were found when neglecting the rotational mass of the superstructure, particularly in long span single-column bent bridges [1]. The mass moment of inertia of the superstructure shall be assigned according to the following:

$$M_{XX} = \frac{Md_w^{\ 2}}{12} = \frac{(m/L)L_{trib}d_w^{\ 2}}{12} \tag{1}$$

where M_{XX} = Rotational mass of superstructure, assigned as lumped mass in axial direction 1-1 or global X-X (R1); M = Total mass of superstructure segment, tributary to the node; m/L = Mass of superstructure per length; L_{trib}, = Tributary length according to node definition; d_w = Superstructure width, which can be taken as average of bottom and top flanges.

Superstructure modeling

The superstructure is protected by a capacity design approach and is expected to remain in the elastic range of response. The superstructure elements will therefore be modeled as linear-elastic beam-column elements with material properties corresponding to cracked reinforced concrete. The elevation (node height) of the superstructure frame elements will be defined at the elevation of the superstructure centroid.

The superstructure frame properties for a box-girder cross section (area, torsional constant, moments of inertia, shear areas and elastic moduli) must be estimated accurately based on elementary solid mechanics theory, assuming a multiply-connected thin-walled section subjected to axial load, bending, shear, and torsion [9,10]. The shear area of the superstructure must also be approximated accurately since elastic shear deformations are included in the stiffness computation of the bridge in SAP2000. The superstructure frame properties for a box-girder or other cross section can be defined in SAP2000 as a General Property or as a Section Designer (SD Section).

The effective moments of inertia I_{eff} and J_{eff} shall be used to obtain realistic values for the structure's period and the seismic demands generated from the analysis. I_{eff} in box-girder superstructures is dependent on the extent of cracking and the effect of the cracking on the element's stiffness. I_{eff} for conventionally reinforced concrete box-girder sections can be estimated between $0.5I_g$ and $0.75I_g$, according to SDC [2], section 5.6.1.2, representing lightly to heavily-reinforced sections. No stiffness reduction is recommended for pre-stressed concrete box-girder sections ($I_{eff} = I_g$), as specified by SDC [2], section 5.6.1.2. Reductions to I_g similar to those specified for box-girders can be used for other superstructure types and cap beams.

A reduction of the torsional moment of inertia is not required for bridge superstructures that meet the Ordinary Bridge requirements in section 1.1 of SDC [2] and do not have a high degree of in-plane curvature. For special bridges, the effective torsional constant, J_{eff} can be taken to be 20% of the gross torsional constant, J_g. The non-reduced properties of the cross section are used to model axial stiffness (A_g) and transverse shear stiffness (A_v). In SAP2000 a reduction to the gross section properties is carried out by specifying Property Modifiers to the desired frame element cross-sectional property.

Cap beam modeling

In the case of multi-column bent bridges, an elastic element representing the concrete cap beam should be modeled as a frame element with a solid rectangular cross section with dimensions according to plans. The material properties used for this element include the Modulus of Elasticity E_c, weight w_c and mass ρ_c of reinforced concrete, as defined by SDC [2], section 3.2.6. The flexibility of the cap beam should be accounted for in the model, instead of joint constraints, if sufficient design details are specified for such an element. Since the concrete superstructure and cap beam are cast simultaneously into a single element, a rigid or moment connection is used between these elements and the torsional stiffness of the cap beam is enhanced due to the superstructure's flexural stiffness. The torsional constant of the cap beam J_g is then modified by an amplification factor C by applying Property Modifiers in SAP2000 to that value, as follows:

$$J_{eff} = J_g \times C \tag{2}$$

where C = torsional constant amplification factor with a minimum value in the order of 102; J_{eff} = effective torsional resistance of the element; J_g = torsional resistance of the element gross cross section.

A minimum value of 102 was determined iteratively for the torsional coefficient C using different multi-column bent bridge models to obtain a minimum 95% reduction in cap beam twist. For bridge systems with separate multiple lanes, the cap beam should be segmented according to the number of decks and stiffened in torsion where the beam is monolithically poured with the superstructure. For other types of bridge superstructures additional research is needed to realistically estimate the rotational stiffness of the cap beam according to the connection details.

Modeling of pier columns

Inelastic three-dimensional beam-column elements are used to model the column and shaft for each of the piers in the bridge. It is recommended to define a separate rigid segment at the column top to represent the portion of the column embedded in the bent cap. In SAP2000 an end offset with a rigid-zone factor of 1.0 can be used for that purpose. Plastic hinges are expected to develop in the top and bottom of the column bent, according to the boundary conditions and foundation details of the bridge. In SAP2000 an elastic beam-column element can be used to represent the column behavior outside the plastic hinge zone with effective cross-sectional properties, while nonlinear behavior is assigned to the plastic hinge zone.

For column bents designed as ductile members according to SDC [2], the cracked flexural stiffness I_{eff} should be used, estimated from Fig. 5.3 of SDC [2] based on the level of axial load and transverse reinforcement, or from the initial slope of the moment-curvature (M-ϕ curve between the origin and the point designating the first reinforcement bar yield. The torsional stiffness of column concrete members is greatly reduced after the onset of cracking according to section 5.6 of SDC [2] by introducing a Property Modifier in SAP2000 according to the following:

$$J_{eff} = 0.2 J_g \tag{3}$$

According to section 3.6 of SDC [2], a reduction to the gross area of the column due to the combined effects of flexure and axial load is carried out to estimate the expected shear capacity of ductile concrete elements such as column bents. It is therefore recommended to introduce a Property Modifier factor to the shear area of the column elements gross section in SAP2000, according to the following:

$$A_{v,eff} = 0.8 A_{v,g} \tag{4}$$

where $A_{v,eff}$ = Effective shear area of the column; $A_{v,g}$ = Shear area of the column gross cross section.

Column moment- curvature (M-ϕ) analysis

The plastic moment capacity of all ductile concrete members of the bridge, particularly column bents, shall be calculated by the M-ϕ analysis based on expected material properties, according to section 3.3.1 of SDC [2]. The M-ϕ curve can be idealized with an elastic perfectly plastic response to estimate the plastic moment capacity of a member's cross section; however, a bilinear model accounting for strain hardening of steel is preferred, as presented in Fig. 2. The calculation of the M-ϕ curve can be carried out using programs such as xSECTION by Caltrans [6], XTRACT by Imbsen [5], SD-Section by SAP2000 [3], or OpenSees [4]. The resulting bilinear models considered for static and dynamic analysis are presented in Fig. 3. The following considerations are taken into account for the M-ϕ analysis:

- The basic level of axial load for the column will be defined as the dead load on the column, including superstructure tributary and self weights.
- Fluctuations in the column axial load will occur due to vertical excitation and frame action during a seismic event represented by static lateral loads or dynamic analysis caused by ground motions. The maximum range of column axial load is typically estimated by Caltrans engineers between (–)0.05P_n in tension and (+)0.15P_n in compression, where P_n is the column nominal axial capacity [11]. Two additional M-ϕ curves should be obtained for these two levels of column axial load. SAP2000 automatically interpolates between these limiting levels of axial loads in the Interaction PMM hinge.
- The column cross section must be represented in the analysis with a sufficient number of fibers and include the correct dimensions and reinforcement of the cross section.
- The expected material strength and stress-strain (σ–ε) relation for concrete and steel is used according to chapter 3 of SDC [2].
- The failure of the cross section will be defined as fracture of the steel rebar when reaching the ultimate strain or the crushing of confined concrete, defined according to section 3.2.2 of SDC [2] and Fig. 3.
- Plastic capacity M_p is defined by balancing the areas between the actual and the idealized M-ϕ curves beyond the yield point, as defined in section 3.3.1 of SDC [2]. The plastic curvature ϕ_p is defined as the difference between the ultimate and yield curvatures $\phi_p = \phi_u - \phi_Y$ and the ductility capacity of the column given by $\mu_c = \phi_u/\phi_Y$.
- The moment-curvature analysis of column cross sections with biaxial symmetry must be repeated for strong and weak axis bending under gravity axial load and other specified levels of axial load.

Column nonlinear behavior

The nonlinearity and hysteretic behavior in the column can be captured through a distributed plasticity fiber model or idealized through discrete plastic hinge models, assigned to pre-determined locations of the column. Several modeling options can be employed in SAP2000 to represent the behavior of the column plastic hinge using lumped plasticity models, among them the Uncoupled plastic hinge, the Interaction PMM hinge, the Fiber hinge, and the NL-Link. Some of the main capabilities and limitations of these nonlinear models for column plastic hinge are presented in Tables 2 and 3. The primary modeling aspects of the Fiber hinge which represents the more accurate lumped plasticity model available in SAP2000 are presented below. The length of the plastic hinge can be estimated according to section 7.6.2 of SDC [2]. The approximation of a distributed plasticity model of the column in SAP2000 could be achieved through the assignment of multiple hinges along the height of the column.

The column plastic hinge can be modeled with greater accuracy using the Fiber hinge option in SAP2000. The Fiber hinge computes an M-ϕ relation in any bending direction and for varying levels of axial load. This interaction between biaxial moment and axial force, and the distribution of inelastic action throughout the section is obtained automatically by assigning particular stress-strain (σ–ε) relationships to individual discretized fibers in the cross section. The σ–ε relationships are assigned to fibers of unconfined concrete, confined concrete, and longitudinal steel reinforcement. The definition of each fiber in the cross section of the pier columns and shafts includes the area, centroid coordinates and material type, for which a stress-strain relation was defined previously. A sufficient number of fibers is required to represent the cross section configuration with enough accuracy and obtain values for the hinge area and moment of inertias within 5% of the column gross section properties. The recommendations for column cross section fiber discretization provided by [12] can be used for this purpose.

The Fiber hinge model is a lumped plasticity model with a characteristic length L_p, assigned to an elastic element at a location corresponding to the middle of the plastic hinge zone. The fiber hinge model can represent the loss of stiffness caused by concrete cracking, yielding of reinforcing steel due to flexural yielding, and strain hardening. It is successful in representing degradation and softening after yielding, however pinching and bond slip are not included in the present model for simplicity. Shear and torsion behaviors of the cross section are represented elastically. The bridges are not being modeled for collapse simulation, only to capture the initial elastic and flexural yielding portions of the response. As these are relatively newer, non-shear-critical bridges, the shear capacity is larger than the flexural capacity and therefore, it is only necessary to capture the added elastic shear and torsional flexibility, not the nonlinear flexural-shear interaction.

In SAP2000 the Fiber hinge is assigned to an elastic beam-column segment in the region of the plastic hinge zone. Since both the nonlinear Fiber hinge model and the elastic column have finite stiffness, the resulting column stiffness is calculated in SAP2000 as a series stiffness of the two components, resulting in an overestimation of the flexibility of the column (the flexibilities of the two components are added). To correct this overestimation, Property Modifiers or gross section scale factors can be determined iteratively until achieving the target elastic period and assigned to the elastic column segment in the plastic hinge zone. A uniform factor in the order of 1.0 to 3.0, applied to the gross area A_g, shear area A_v, and gross inertia I_g of the column, is found to approximate well the correct first mode period [1].

The use of a lumped plasticity model defined in series with an elastic element will nevertheless produce an overestimation of the strength of the column. This overestimation is produced since the capacity of the elastic segment is automatically added to the nonlinear behavior defined for the hinge. One method found to be successful in correcting this capacity overestimation involved scaling down the stress values in the σ–ε relationship defined for each fiber to approximate the desired behavior of the Fiber hinge-elastic element series system. Further details on lumped plasticity models and the accurate representation of both the elastic stiffness and yield force can be found in [13].

The steeply descending branch of the σ–ε curve defined in the material models could produce problems converging to an equilibrium solution. Since the Fiber hinge model is a displacement control (ductile) model, an extrapolation of stress values after the assigned failure point is computed without strength degradation, which results in infinite ductility of the column. A separate estimation of the ductility capacity can be carried out following section 3.1.3 of SDC [2].

The Fiber hinge model can be used for modal analysis, nonlinear static analysis (pushover), and nonlinear time history analysis using the direct integration method. It is important to note that the strength and stiffness of the elastic column outside the plastic hinge zone is still overestimated in SAP2000 for a 3D analysis (see Fig. 4 and 5).

The nonlinear behavior of the column can be effectively captured in OpenSees using a distributed plasticity fiber model and a sufficient number of fibers and integration points along the height of the column. The pushover response of the different plastic hinge models available in SAP2000 is presented in Fig. 4 and 5 for a pushover analysis of a typical circular cantilever column in the transverse axes of the column *X* or *Y* and for a 45 degree angle with respect to the default principal axes. The closed-form solution for a cantilever beam with shear deformations was calculated and compared to all of the OpenSees and SAP2000 models in these figures. The accuracy of the hinge models in capturing the complete nonlinear response of the column was determined based on this comparison and used for parameters calibration. These figures clearly illustrate the overestimation of the stiffness and strength of the different plastic hinge models and the elastic column in SAP2000 for a 3D analysis, due to the observations discussed above. This overestimation of the column capacity in SAP2000 of up to 40% is due to the incorrect mathematical formulation of the resultant of the cross section capacities in each orthogonal direction corresponding to the default primary axes in the program (see Fig. 4 and 5). The response of these models is compared to the OpenSees distributed plasticity fiber model of the cantilever column. As expected, the pushover response of the OpenSees model of a circular cantilever column is symmetrical with respect to any specified angle for both the elastic and nonlinear models of the column. The force and displacement values were equally normalized in SAP2000 and OpenSees programs with respect to the ultimate base shear and the yield displacement, respectively. Additional nonlinear modeling options available in OpenSees to represent the column plastic hinge behavior can be found in [4].

Boundary conditions

Although it is impractical to include all the effects of the soil and foundation on the earthquake response of a bridge, the design engineer should recognize that soil-structure interaction introduces flexibility and energy dissipation into the system compared with the assumption of a rigid or pinned support, especially in the case of very rigid systems or irregular geometries. There is evidence that for pile footings the rotational stiffness is of greater significance than lateral stiffness on overall bridge response [14]. The total vertical and rotational stiffnesses for a pile-supported footing or abutment are largely related to the pile axial stiffness in compression and uplift [14]. For oversized bridge foundations resulting from the capacity design approach in section 7.7 of Caltrans SDC [2] and stable soil conditions, soil-structure interaction can therefore be modeled through a system of elastic springs. For large kinematic soil demands (liquefaction hazard, lateral spreading, intermediate soft soil layers, or marginal soil) a more complex nonlinear soil-structure interaction model should be used for the column foundations. Section 4.2.2 of ATC 32 [7] and section 17 of BDS [15] provide general guidelines for the consideration of soil-structure interaction effects in the modeling of bridge structures.

Even when soil-structure interaction is neglected according to geotechnical specification, the column foundation may still be considered to have semi-rigid behavior through the assignment of rotational and translational springs. Depending on the details of the foundations, a pinned, semi-rigid, or fixed connection should be specified at the column base. In the case of flexible foundations with appropriate lateral restraint, a pinned connection is specified at the column base through joints restraints at the translational degrees of freedom, while the linear or nonlinear behavior of the foundations is introduced at the rotational degrees of freedom. This spring model can be achieved in SAP2000 by using either the Uncoupled Hinge or the zero-length NL-Link.

The effective height of the column should also be adjusted to the idealized location of column fixity. If a flexible foundation response is expected in the longitudinal, transverse, or vertical directions, the column base can be modeled as a semi-rigid connection using elastic or nonlinear springs (see Fig. 6), according to the geotechnical specifications for the site. The geometrical properties of the column cross section at the transition point between the foundation footing or piles and the column bent are also considered in the model. Section 5.3 of ATC 32 [7] provides additional recommendations for foundation modeling.

Abutment modeling

Abutment behavior, soil-structure interaction, and embankment flexibility have been found by post-earthquake reconnaissance reports to significantly influence the response of an entire bridge system under moderate to strong intensity ground motions. Specifically for bridge structures with short spans and relatively high superstructure stiffness, the embankment mobilization and the inelastic behavior of the soil material under high shear deformation levels dominate the response of the bridge and the intermediate column bents [16].

A realistic abutment model should represent all major resistance mechanisms and components, including an accurate estimation of their mass, stiffness, and nonlinear hysteretic behavior. Values of embankment critical length and participating mass were suggested by many research studies in order to quantify the embankment mobilization [16-18]. However, additional research is needed to standardize the modeling recommendations for highway bridges, including soil-structure interaction.

Chapter 5 of ATC 32 [7] presents several aspects of the modeling and design of different foundation types including bridge abutments, as well as pile footings, spread footings, cast-in-place column shafts, and cast-in-place pile shafts. Section 7.8 of SDC [2] provides backbone curves for seat and diaphragm abutment types for the longitudinal direction and discusses modeling limitations for the transverse direction. Three abutment models, namely Roller, Simplified and Spring, were implemented to investigate the sensitivity of the global seismic response of the bridge to abutment modeling [19]. The resulting force-displacement response of a bridge example using these abutment models in the longitudinal and transverse directions is presented in Fig. 7 and 8, respectively.

The Roller abutment model consists of a simple boundary condition module that applies single point constraints against displacement in the vertical direction (vertical support in SAP2000). This model can be used to provide a lower-bound estimate of the longitudinal and transverse resistance of the bridge, captured through a pushover analysis. The response of this simple bridge model is dominated by the formation of plastic hinges and the ductility capacity of the column bents.

The Spring Abutment model is a more complex abutment model [20] that includes sophisticated longitudinal, transverse, and vertical nonlinear abutment response, as well as a participating mass corresponding to the concrete abutment and mobilized embankment soil. A general scheme of this abutment module, denoted as Spring abutment model, is presented in Fig. 9. More details are available in Reference [1].

The Simplified abutment model is a simplification of the Spring abutment model described below. The general scheme of the Simplified model is presented in Fig. 10. The Simplified abutment model consists of a rigid element of length d_w (superstructure width), connected through a rigid joint to the superstructure centerline, with defined longitudinal, transverse and vertical nonlinear response at each end. The longitudinal response defined for the Simplified abutment model accounts only for the gap and the embankment fill response, where passive pressures are produced by the abutment back wall. The shear resistance of the bearing pads is ignored. In the longitudinal direction, a series system is defined, consisting of a rigid element with shear and moment releases, a gap element with boundary conditions at each end allowing only longitudinal translation, and a zero-length element (see Fig. 11). The zero-length element is assigned an elastic-perfectly-plastic (EPP) backbone curve with abutment stiffness (K_{abt}) and ultimate strength (P_{bw}) obtained from section 7.8.1 of SDC [2].

In the transverse direction, a zero-length element is defined at each end of the rigid link with an assigned elastic-perfectly-plastic (EPP) backbone curve representing the back fill, wing wall and pile system response. The abutment stiffness (K_{abt}) and back wall strength (P_{bw}) obtained for the longitudinal direction from section 7.8 of SDC [2] are modified using factors corresponding to wall effectiveness (C_L) of 2/3 and participation coefficients (C_W) of 4/3 [21]. The wing wall length can be assumed to be 1/2-1/3 of the back wall length. The resistance of the brittle shear keys and distributed bearing pads is ignored in this model.

In the vertical direction, an elastic spring is defined at each end of the rigid link, with a stiffness corresponding to the bearing pads stiffness kv. Assuming rigid soil conditions, the vertical embankment stiffness is neglected in the model, and the horizontal distribution of bearing pad stiffnesses is lumped into a single vertical spring.

Additional modeling considerations

Damping — The recommended damping values for pre-stressed bridges with only minor cracking, and reinforced concrete bridges with considerable cracking, undergoing small deformations or subjected to low intensity ground motions is estimated at 2-3% and 3-5% of critical, respectively [22]. For bridges with a pre-stressed superstructure (without complete loss in pre-stress), the estimated damping coefficient is increased to about 5-7% of critical [22] when demands are near the yield point. In yielding bridge structures with stable hysteretic behavior and structural damage occurring only in ductile components due to severe seismic conditions (pre-stressed and normal reinforced concrete), the damping coefficient is estimated at 7-10% of critical [22]. Typically, a 5% damping ratio is used in design codes; however, higher damping ratios up to 10% of critical may be anticipated and justified by the design engineer for bridges with characteristics according to section 2.1.5 of SDC [2]. It is recommended to use a viscous damping ratio of 8 to 20% for abutment fills with cohesionless soils in which the maximum shear strain ranges between 0.05 to 5%, respectively [23].

Material damping coefficients can be specified in SAP2000 when defining material properties, and used in dynamic analyses. Modal damping ratios are required for response-spectrum analysis and viscous damping is required for direct-integration time-history analyses, defined through mass- and stiffness-proportional components.

P-Delta effects — The consideration of P-Delta or second order effects helps identify structural instability hazard of a bridge by capturing the degradation of strength and amplification of the seismic demand on the column bents, caused by the relative displacement between the column top and bottom. In SAP2000 two types of geometric nonlinearities are available for nonlinear static analysis and direct-integration time history analysis. These nonlinearities are the P-Delta and large displacements effects. For typical bridges, the P-Delta option is adequate, particularly when material nonlinearity dominates the nonlinear behavior. Section 4.2 of SDC [2] provides a conservative limit for Ordinary Standard bridges meeting the specified ductility requirements to ignore P-Delta effects in a static analysis. The large displacement option is used for structures undergoing significant deformation and for buckling analysis, therefore it is not recommended for typical bridge analysis.

Expansion joints and restrainers — The opening and closing of expansion joints between segments of a bridge's superstructure introduce nonlinearities and discontinuities that affect the load path and hence the dynamic response of bridges. Section 4.2.2 of ATC 32 [7] provides general guidelines for the modeling of bridges with expansion joints and restrainers, as well as recommendations for the selection of input ground motions to be used in the analysis. The expansion joints and restrainers can be modeled in SAP2000 using the Gap, Hook, or Multi-Linear Plastic special Link/Support to model the nonlinear spring elements.

COMPARISON OF SAP2000 AND OPENSEES MODELS

A total of six existing reinforced concrete bridge structures with box-girder superstructure and different geometries and column cross sections were modeled and analyzed using the structural analysis programs SAP2000 and OpenSees. The models were created based on existing construction drawings, supplied by Caltrans. This comparison served to evaluate the applicability and examine the accuracy of the proposed modeling guidelines. The selected bridge structures represent Ordinary bridges in California with box-girder superstructure, typical column bent details, zero skew, and simple geometric regularity. The principal characteristics of the bridges are presented in Tables 4-6. The first four bridges of Tables 4-6 consist of Ordinary Standard Bridges and the last two are Ordinary Nonstandard bridges with simple geometric regularity. Specific cross section details and the complete analytical models of the six bridges analyzed in this study can be found in [1].

The bridge superstructures and cap beams were modeled in both programs as elastic beam-column elements divided into five discrete segments per span with translational and rotational tributary mass lumped at each node, according to the recommendations presented above. Effective cross-sectional properties were defined in both programs for the elastic bridge superstructure following the recommendations above, according to the deck geometry and pre-stress reinforcement. The cap beam was assigned a rigid torsional stiffness to account for the monolithic construction of the superstructure and cap beam into a single element. Expected material strength properties were used for all steel and concrete elements and fibers, rather than nominal properties. Elastic shear deformation was included for all beam and column elements in both programs to properly approximate the stiffness of the superstructure and mode shapes of the bridge. Shear deformation was incorporated into the OpenSees column and deck models, while in SAP2000 it is automatically included in the stiffness calculation. In OpenSees the elastic shear deformations were added to the column flexibility matrix by aggregating shear force-shear strain relationships at each of the integration points. P-Delta effects were considered in the static and dynamic analysis.

The plastic hinge zone of the column bents was modeled in SAP2000 as a lumped plasticity Fiber hinge model, while the column outside the plastic hinge zone was modeled as an elastic beam-column element divided into five discrete segments with tributary mass and effective cross-sectional properties. For this purpose, Property Modifiers were applied in SAP2000 to the gross section inertia, torsional resistance, and shear area of the elastic beam-column element, according to the level of axial load on the column and the corresponding recommendations presented above. The OpenSees model of the column bent consisted of a single segment with distributed plasticity fiber model, nonlinear force formulation and five integration points. The stiffness of the column element in OpenSees was therefore determined according to the development of nonlinear behavior and crack propagation. The concrete constitutive model used in OpenSees was Concrete02 which has Kent-Scott-Park behavior and includes tensile strength. The steel fibers utilized Steel02 which has Menegotto-Pinto behavior with ultimate strains specified according to the SDC [2]. Similar stress-strain relationships were defined in SAP2000 Fiber hinge model for the longitudinal and transverse steel reinforcement, as well as the concrete cover and core. The stress values in the different σ–ε relationships used for the Fiber hinge in SAP2000 were scaled down to adjust for the overestimation in strength obtained from the automatic series system resulting. The unscaled concrete and longitudinal steel stress-strain relations used for La Veta bridge Fiber hinge model are presented in Fig. 12. The stress values in these σ–ε relationships were scaled down to offset the capacity overestimation resulting from the Fiber hinge-elastic element series system in SAP2000. The discretization of the cross section into fibers was carried out similarly in both programs according to [12]. Rigid offsets were defined in both programs at the top of the column element to account for the column-superstructure moment connection.

The column foundations were modeled as fixed and pinned boundary conditions, for single and multi-column bent bridges, respectively. In the case of the longer MGR and W180-N168 bridges exceeding 300 ft (91.44 m), the superstructure ends were assigned a roller support, since nonlinear abutment behavior does not control the response of the structures. A more elaborate abutment model defined as the Simplified model [19] was similarly defined in both programs for the remaining four shorter bridges. The Simplified model accounts for gap closure in the longitudinal direction, vertical stiffness of the elastomeric bearing pads, and soil embankment elastic-perfectly-plastic resistance in the longitudinal and transverse directions. The details of the Simplified abutment model were previously described in