

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 E_c 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 $\omega_c = 144 \text{ lb/ft}^3$ (2300 kg/m^3) and therefore a mass of $\rho_c = 4.5 \text{ lb-sec}^2/\text{ft}^4$ ($230 \text{ kg-sec}^2/\text{m}^4$) 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/ft³ (2400 kg/m³). 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:

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

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:

$$J_{eff} = J_g \times C \quad (2)$$

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 ($M-\phi$) 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:

$$J_{eff} = 0.2J_g \quad (3)$$

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:

$$A_{v,eff} = 0.8A_{v,g} \quad (4)$$

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

Column moment- curvature ($M-\phi$) analysis

The plastic moment capacity of all ductile concrete members of the bridge, particularly column bents, shall be calculated by the $M-\phi$ analysis based on expected material properties, according to section 3.3.1 of SDC [2]. The $M-\phi$ 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 $M-\phi$ 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 $M-\phi$ analysis:

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- 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 $M-\phi$ 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 ($\sigma-\epsilon$) 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 $M-\phi$ 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 $M-\phi$ 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 ($\sigma-\epsilon$) relationships to individual discretized fibers in the cross section. The $\sigma-\epsilon$ 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 k_v . 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

this paper. Fig. 7 and 8 present the longitudinal and transverse response of an example bridge using the Simplified abutment model, as well as a comparison to the overall bridge response obtained using the Roller and Spring models discussed above.

The comparison between SAP2000 and OpenSees for the six bridge models was carried out using modal, nonlinear pushover and nonlinear time history analysis results, according to the recommendations presented in Table 7 developed in [1]. The epistemic uncertainty related to analytical modeling and analyses of the seismic response of the six bridges was investigated from these analyses results. All bridge structures considered in this study were modeled in both programs using the recommendations presented above. The remaining sources of variability in the structural response assessment were reduced by the use of identical load patterns, a large set of ground motions with different directivity effects and source mechanisms and similar numerical solution methods and parameters in both programs. The effects of the epistemic uncertainty related to modeling were computed through bias factors for the peak displacements obtained from dynamic non-linear analysis of these bridge structures. The OpenSees analysis tool was selected in this study as the benchmark software since it is a reliable research code with an open architecture and widely supported base of users that have used the software in calibration studies. Moreover, since SAP2000 pushover results of the preferred Fiber hinge model and the elastic column element presented an overestimation of the column strength for a 3D analysis (see Fig. 4 and 5), the resulting peak displacements obtained from a 3D dynamic non-linear analysis of the six bridges are considered to be underestimated. Consequently, as will be discussed in the following section, the computed bias factors for SAP2000 dynamic results with respect to the more reliable OpenSees results were consistently larger than 1.0 for all six bridge models analyzed. However, since a comparison of key analysis results to those obtained from more reliable software packages does not solely guarantee accuracy in bridge analysis, ongoing research involving closed-form and benchmark problems is currently carried out to verify the conclusions presented in this paper regarding these two analysis platforms.

Modal analysis results

The elastic periods of the six bridges analyzed, obtained through eigenvalue analyses in OpenSees and SAP2000 assuming effective cross-sectional properties (called OpenSees-Elastic and SAP2000-Elastic models, respectively), matched within 5% for the fundamental mode shapes of the structures. Due to the simple geometric regularity of the different bridges considered in this study, the primary modes of the structures (i.e., transverse and longitudinal translations, global torsion as well as in-plane and out-of-plane superstructure deformations) present a relatively low modal correlation, observed through the participating mass ratio distribution obtained for each mode. Consequently, the dynamic properties and basic results of the bridge models were expected to coincide for a linear-elastic dynamic analysis.

The nonlinear model in OpenSees with an initial uncracked column cross section (called OpenSees-Nonlinear model) yielded another set of modal periods for the elastic range of response. The response of the nonlinear bridge model in OpenSees resulted in a different dynamic response, even for the elastic range of response of the bridge. Table 8 summarizes the modal results for La Veta bridge performed using the SAP2000-Elastic and OpenSees-Elastic models defined with effective cross section properties defined for the column bents, as well as the OpenSees-Nonlinear model with an uncracked initial state of the column bents.

Pushover analysis results

Pushover analyses were carried out for all six bridges in SAP2000 and OpenSees using a force pattern proportional to the translational mass distribution of each structure in the corresponding direction. Pushover analysis combining different mode-shape force patterns was not necessary for the Ordinary bridges analyzed with negligible skew supports and standard column heights. Fig. 13 presents the pushover curves for La Veta bridge computed in the longitudinal (along the deck) and transverse directions of the structure. As previously discussed, the pushover response of the elastic column element, Fiber hinge model and remaining plastic hinge models available in SAP2000 carried out in the direction of the default principal axes of the program did not present any bias in the initial stiffness or ultimate strength of the column model with respect to results obtained from OpenSees and hand calculations. Consequently, no significant bias was expected in the pushover results of the complete bridge models carried out in

the principal axes of the bridge (i.e., transverse and longitudinal directions). The initial stiffness of the bridges computed using the two programs indeed matched within 5% for all bridges, while the computed ultimate base shear differed by only 15% in most cases. The ultimate shear capacity can only be identified directly in Fig 13. for the OpenSees bridge model. The ultimate shear capacity was taken as the maximum shear capacity before failure or degradation of strength. As observed, degradation of strength was not captured by the ductile SAP2000 Fiber hinge model, where the ultimate capacity of the fibers is extrapolated beyond the assigned failure point according to the specific parameter definition of this hinge model. An estimate of the ductility capacity of the bridge column bents was carried out separately according to empirical formulas presented in section 3.1.3 of SDC [2]. The ultimate shear capacity for the SAP2000 bridge models can be estimated at the displacement demand level corresponding to the maximum ductility capacity as per SDC [2]. The bias between SAP2000 and OpenSees pushover results was calculated for all bridge models using this definition.

Time history analysis results

A set of 60 ground motions (GMs) with three components each from the I880n, I880p, and Van Nuys sets [24,25] was used to conduct nonlinear time history seismic response analyses. A uniform scale factor of 2.0 was used for all motions to induce significant nonlinear behavior in the bridge columns. The self-weight of the bridge was included during the time history analysis. The responses of the bridge in the longitudinal and transverse directions were used to estimate the diagonal response of the bridge at an angle with respect to its orthogonal axes by the SRSS combination rule at each time step. The transient analysis was performed using direct integration with Newmark's average acceleration time integration method and assuming 5% Rayleigh damping. The modal superposition solution method for transient analysis was not used in this study since nonlinear behavior was expected to develop in the bridge structures under the scaled ground motion excitations. The transient analysis was initiated in both programs after the application of the gravity (dead) load on the bridge superstructure. Additional time integration methods such as Newmark's central difference, linear acceleration, and modified average acceleration, as well as the Wilson θ -method, Hilber-Hughes-Taylor α -method, Collocation, and Chung and Hulbert methods are available in SAP2000 or OpenSees programs. Important considerations for the proper selection of the integration method for a particular structure including the stability and accuracy of the results, as well as the algorithm noise are discussed in [1].

The assessment of the dynamic results was carried out for all six bridges relating peak displacements of a monitored point in the bridge to an intensity measure (IM) for each record, defined as the spectral displacement at the first mode period of the bridge ($S_{d,T1}$) [26]. The selected monitored point in the bridge model was the intersection point between the superstructure and column top centerline. A natural log fit was used for the median of the data and the results were plotted in linear and logarithmic scales. The parameters of the natural logarithmic regression $\ln(\Delta) = \beta_1 + \beta_2 \ln(S_{d,T1})$ or equivalently $\Delta = e^{\beta_1} S_{d,T1}^{\beta_2}$ were computed for all bridge models.

Three levels of seismic hazard were defined to compute the bias factors between the SAP2000 and OpenSees time history results. The low, moderate and high seismic hazard levels were defined as the 50%-, 10%- and 2%-in-50 year probabilities of exceedance (PE) for a high seismicity zone in California (e.g., Berkeley). The probabilistic uniform hazard curves provided by USGS [27] were used to obtain the elastic spectral displacements ($S_{d,elastic}$) corresponding to the first mode period of each bridge, for each hazard level. The Acceleration Response Spectra (ARS) curves provided in SDC [2] for different hazard levels and soil conditions can alternatively be used to establish the displacement demand on bridge structures.

The dispersion of peak displacements obtained from 60 ground motions and the logarithmic regressions computed in different directions using the OpenSees program for the La Veta bridge are presented in Fig. 14. The dispersion of the SAP2000 results was similar to OpenSees, indicating the aleatory uncertainty of the nonlinear dynamic analysis results of the bridges primarily arises from ground motion variability. Similar plots were generated for the remaining bridges using OpenSees and SAP2000 time history results, in both linear and logarithmic scales [1]. The computation of the bias factors was carried out using two methods, described as follows.

In method 1 (see Fig. 15) the natural logarithmic regression is used for all peak displacements in the seismic intensities in the range of $IM = 0$ to $1.2S_{d,elastic}$ of each bridge, representing a reasonable demand

limit for the lifetime of the structures. The bias factors are then computed by comparing, at each hazard level, the peak displacements estimated using the regression coefficients of SAP2000 and OpenSees results. Fig. 17 presents the regressions for peak displacements obtained from SAP2000 and OpenSees for the La Veta bridge model. The regressions computed for different directions were compared for the low, moderate and high hazard level demand corresponding to each bridge, and the bias factors, representing the ratio of OpenSees over SAP2000 peak displacement values, are recorded in Table 9. The seismic demand at each hazard level was computed for each bridge as the spectral displacement at the first mode period obtained from the USGS hazard curves [27] for a high seismicity zone in California.

Method 2 for bias factor computation is carried out through a pair-wise comparison between OpenSees and SAP2000 nonlinear time history analysis results for each ground motion, in the longitudinal, transverse and diagonal directions of the bridge. A natural logarithmic regression is used relating the bias obtained for every ground motion to the intensity levels, as shown in Fig. 16. The bias factors corresponding to the low, moderate and high hazard levels are computed using the new set of regression coefficients. The regressions for the bias factors in terms of the intensity measure $S_{d,T1}$ were obtained for all six bridges for the longitudinal, transverse and diagonal directions. The resulting bias factors for the low, moderate and high seismic hazard levels are presented in Table 10. Additional details related to the calculation of the SAP2000-OpenSees bias factors can be found in [1].

Table 11 summarizes the results of the bias factor computations for peak displacements obtained from nonlinear time history analyses using OpenSees and SAP2000. The average bias and corresponding standard deviation for the six bridges analyzed is presented, differentiating between the results for short and long bridges. Both methods of computing analysis software bias factors for nonlinear time history displacement results show a clear tendency for the bias to increase with increasing intensity of the ground motions. The average bias factor value greater than 1.0 for all cases indicates that the nonlinear time history analysis displacement response results obtained using OpenSees are consistently larger in magnitude than the corresponding response results obtained using SAP2000 for all the bridge models analyzed.

For the low hazard level, the bridges remain essentially elastic. Since the modal periods computed for SAP2000 and OpenSees were similar for all six bridges analyzed, the nonlinear time history analysis displacement results are comparable as well. The average bias obtained for short and long bridges in the low hazard level range was around 10% using the two methods of bias factor computation, with a standard deviation of around 5%. For the moderate hazard level, the bias factors computed using both methods of computation show a clear tendency for the OpenSees displacement results to exceed the SAP2000 nonlinear time history analysis displacement results. An average bias of 20% and up to 75% was computed for the short and long bridges, respectively, with corresponding standard deviations of 5 and 60%, respectively. At the high hazard level, this tendency becomes more pronounced. Average bias of 25% and 100% were obtained for the short and long bridges, respectively, with significantly larger standard deviations of 10 and 100%, respectively. Despite having low correlation coefficients between the different mode shapes of the bridge structures, the three component ground motions produce a coupled three-dimensional bridge excitation. Therefore, in the case of the SAP2000 bridge models, the interaction between the transverse and longitudinal translations of the bridges results in an overestimation of stiffness and strength of the columns or equivalently an underestimation of the peak displacements computed in each direction. This underestimation of peak displacements and corresponding bias factors increase with increasing ground motion intensity.

The bias factors obtained for the corresponding hazard levels using both methods were similar. These methods are therefore considered adequate for computing bias factors. However, since very few bridges were used in the analysis, the bias factors presented displayed significant dispersion, especially in the case of long bridge structures, and therefore do not represent a sufficiently large and representative data set. Ongoing research is currently carried out using a larger number of bridges with different geometric configurations representing short and long bridges to obtain reliable values of OpenSees-SAP2000 bias factors. The results of this extended analysis also contain an estimate of the dispersion of the dynamic results for different hazard levels.

SUMMARY AND CONCLUSIONS

This paper presented the main recommendations for nonlinear modeling of typical bridge structures

in California, developed based on an analytical and comparative study carried out on several bridge models in the structural analysis programs SAP2000 and OpenSees, as well as an extended literature review of the current knowledge regarding bridge analysis. The recommendations address modeling of the geometry of the structure, definition of elements and materials, and the assignment of mass and boundary conditions. The nonlinear behavior of bridge abutments and foundations was discussed briefly.

The basic requirements in the modeling of the plastic hinge zone in column bents were discussed in this paper. The applicability and limitation of several plastic hinge models in SAP2000 for different analysis types were presented. In summary, the NL-Link and Uncoupled hinge are defined for a unique level of axial load, thus failing to represent the variation of the column capacity and ductility due to fluctuations in the column axial load during dynamic loading of the bridge. Therefore, a plastic hinge model which does not capture the axial-moment interaction properly (e.g., Uncoupled hinge and NL-Link in SAP2000) will result in significant inaccuracy when axial load fluctuations control the capacity. Particularly if the column undergoes tension stresses, the flexural capacity of bridge columns may be considerably overestimated thus resulting in an inaccurate assessment of the force demand and force distribution among the different bridge components. These models are therefore limited to simple preliminary two-dimensional analysis of the bridge. A fiber model (Fiber hinge in SAP2000) as well as an idealized moment-rotation model for different axial load levels (Interaction PMM Hinge in SAP2000) will enable the improved evaluation of the column behavior under an applied curvature and axial strain to the cross section. The use of these models is generally suitable for three-dimensional static and dynamic analysis. However, computational aspects related to convergence and numerical stability during dynamic analysis cause difficulties in the use of the Interaction PMM hinge. Furthermore, the Fiber hinge and the remaining plastic hinge models, as well as the elastic beam-column element available in SAP2000 were found to present an overestimation of stiffness and strength for a static and dynamic 3D analysis. Conversely, the OpenSees distributed plasticity fiber model of the column bent did display coherent 3D pushover analysis results.

A total of six reinforced concrete bridge models with box-girder superstructure were developed and analyzed in OpenSees and SAP2000 to evaluate their applicability and assess the accuracy of the proposed modeling recommendations. Similar modeling assumptions were used for the superstructure, cap beam and abutment system. The column bent model in OpenSees consisted of a distributed plasticity fiber model with nonlinear force formulation and five integration points. Conversely, the column-bent plastic hinge model in SAP2000 consisted of a lumped plasticity fiber model with an elastic beam-column element with effective cross section properties outside the hinge. Eigenvalue and pushover analyses were carried out in both programs: the elastic periods, initial stiffness and ultimate base shear of the bridge models matched in both programs within 5, 5, and 15%, respectively, for all six bridges.

A set of 60 records was used for the nonlinear time history analysis of the bridges including near-field, far-field, fault-normal and fault-parallel ground motions. The integration method and parameters used for nonlinear time history analysis were the same for both programs. Despite similar modal and pushover analysis results for all bridges, the epistemic uncertainty related to column bent modeling in each program introduced significant variations in the dynamic analysis results. Bias factors, representing the ratio of displacement demand computed using OpenSees versus the same displacement demand computed using SAP2000, were presented for a low, moderate and high seismic hazard level in California, differentiating between short and long bridge types. The bias factors were larger than 1.0, indicating that peak displacement demands computed using OpenSees exceeded those computed using SAP2000 for all six bridge models. Furthermore, it was observed that the bias and data dispersion between OpenSees and SAP2000 dynamic analysis results grow with increasing intensity measure of the ground motion. However, since only few bridges were used in the analyses, the presented bias factors do not represent a sufficiently large and representative data set to make general conclusions. Additional analyses are ongoing to compute reliable bias factor estimates and confidence intervals using an extended set of bridges with different geometric configurations.

OpenSees was selected as a reliable research code with an open architecture and widely supported base of users that have used the software in calibration studies. However, a comparison of key analysis results to those obtained from more reliable software packages does not, by itself, guarantee accuracy in bridge analysis. It is recognized that ongoing research involving closed-form and benchmark problems will provide additional insight on how best to achieve analytical results that are consistent between analysis platforms.

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NOTATIONS

A_g, A_{eff}	= gross section and effective area
$A_{vg}, A_{v,eff}$	= gross section and effective shear area
C	= torsional constant amplification factor
C_L	= abutment wing wall effectiveness coefficient
C_W	= abutment wing wall participation coefficient
d_w	= superstructure width, which can be taken as average of bottom and top flanges
E_c, E_s	= modulus of elasticity of concrete and steel, respectively
E_{sh}	= strain-hardening slope of reinforcing steel
f'_c, f_t	= compressive and tensile strength of concrete, respectively
I_g, I_{eff}	= gross section and effective moments of inertia, respectively
$\dot{M}$	= intensity measure of an earthquake event
J_g, J_{eff}	= gross section and effective torsional resistance
K_{abt}	= abutment stiffness
k_v	= bearing pads vertical stiffness
L_{trib}	= tributary length according to node definition
M	= total mass of superstructure segment, tributary to the node
M_p, M_u	= plastic and ultimate moments, respectively
M_y, M_{ne}	= yield and nominal moments, respectively
M_{XX}	= rotational mass of superstructure, assigned as lumped mass in axial direction 1-1
m/L	= mass of superstructure per length
P_{bu}	= abutment ultimate strength
$P_n = f'_c A_g$	= nominal axial capacity of column
$S_{d,T1}$	= spectral displacement at the first mode period of the bridge
V_b	= bridge base shear
Δ	= displacement
ϕ, θ	= curvature and rotation, respectively
ϕ_y, ϕ_p, ϕ_u	= Yield, plastic, and ultimate curvatures, respectively
$\mu_c = \phi_u / \phi_y$	= ductility capacity of reinforced concrete column
σ, ε	= stress and strain, respectively
ω_c, ρ_c	= unitary weight and mass of reinforced concrete, respectively

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Table 1—Component Modeling

Component	Linear-Elastic	Nonlinear
Superstructure	X	
Column- plastic hinge zone		X
Column- outside plastic hinge zone	X	
Cap beam	X	
Abutment- transverse		X
Abutment- longitudinal		X
Abutment- overturning		X
Abutment- gap		X
Expansion joints		X
Foundation springs	X	
Soil-structure interaction	X	

Table 2—Capabilities and Limitations of Nonlinear Models for Column Plastic Hinge in SAP2000

Nonlinear Option	Coupled behavior M2-M3	Axial-moment interaction	Degrading behavior	Ductility estimation	Numerical stability	Low computational effort
Uncoupled Hinge M2,M3			X	X		X
Interaction PMM Hinge	X	X	X	X		X
Fiber PMM Hinge	X	X	X		X	
NL-Link (Plastic Wen)					X	X
NL-Link (Multi-Linear Plastic)			X	X	X	X

Table 3—Nonlinear Models for Column Plastic Hinge in SAP2000

Nonlinear Option	2D Pushover Analysis (L1 or T2)	3D Pushover Analysis (Simultaneous L and T)	Dynamic 2D (THA3 with L,V4 or T,V components)	Dynamic 3D (THA-Simultaneous L,T and V components)
Uncoupled Hinge M2,M3	X			
Interaction PMM Hinge	X	X		
Fiber PMM Hinge	X	X	X	X
NL-Link (Plastic Wen)	X		X	
NL-Link (Multi-Linear Plastic)	X		X	

1L- Longitudinal direction; 2T- Transverse direction; 3THA- Time history analysis; 4V- Vertical direction

Table 4—Summary of Bridge Geometries Used in the Analysis

Bridge	No. of spans	No. of columns	Length, ft (m)	Width, ft (m)	Column diameter, ft (m)	Column height, ft (m)	Super-structure depth, ft (m)	Cap beam dimensions, ft (m)
Route 14	2	2	286 (87.2)	53.7 (16.4)	5.42 (1.65)	37.9 (11.6)	5.74 (1.75)	7.55 x 5.74 (2.30 x 1.75)
La Veta	2	2	299 (91.1)	75.5 (23.0)	5.58 (1.70)	25.4 (7.7)	6.23 (1.90)	7.55 x 6.23 (2.30 x 1.90)
Adobe	2	2	203 (61.9)	41.0 (12.5)	4.00 (1.22)	26.6 (8.1)	4.10 (1.25)	7.00 x 4.10 (2.13 x 1.25)
LADWP	3	4	262 (79.9)	41.6 (12.7)	4.49 (1.37)	25.6 (7.8)	4.27 (1.30)	6.56 x 4.27 (2.00 x 1.30)
MGR	3	2	366 (111.6)	42.3 (12.9)	6.00 (1.83)	39.1 (11.9)	6.23 (1.90)	—
W180-N168	4	3	674 (205.4)	41.2 (12.6)	6.00 (1.83)	26.4 (8.0)	7.74 (2.36)	—

Table 5—Summary of Bridge Column Materials and Reinforcement Details

Bridge	F_y Steel yield strength, ksi (MPa)	f'_c Concrete compressive strength, ksi (MPa)	Cover, in. (cm)	Longitudinal reinforcement	Transverse reinforcement, in. (cm)
Route 14	68 (468.8)	5 (34.5)	2 (5.1)	42#14	#7@5 (12.5)
La Veta	68 (468.8)	5 (34.5)	2 (5.1)	44#11	#6@3.5 (9)
Adobe	68 (468.8)	5 (34.5)	2 (5.1)	20#10	#6@4 (10)
LADWP	68 (468.8)	5 (34.5)	2 (5.1)	34#10	#8@5 (12.5)
MGR	68 (468.8)	5 (34.5)	2 (5.1)	40#11	#8@6 (15)
W180-N168	68 (468.8)	5 (34.5)	2 (5.1)	30#11	#6@4 (10)

Table 6—Summary of Bridge Superstructure Gross Cross-Sectional Properties

Bridge	A Area, in. ² (m ²)	I_x Inertia X, in. ⁴ (m ⁴)	I_y Inertia Y, in. ⁴ (m ⁴)	J Torsional constant, in. ⁴ (m ⁴)	A_{ox} Shear area X, in. ² (m ²)	A_{oy} Shear area Y, in. ² (m ²)
Route 14	12816 (8.27)	448 (3.87)	18,986 (163.87)	28.39e6 (11.82)	3100 (2.00)	7796 (5.03)
La Veta	18595 (11.97)	792 (6.83)	54,083 (466.79)	52.50e6 (21.85)	4778 (3.08)	11328 (7.31)
Adobe	8698 (5.61)	144 (1.25)	7943 (68.56)	9.48e6 (3.95)	2706 (1.75)	5102 (3.29)
LADWP	8381 (5.41)	150 (1.29)	7266 (62.71)	9.54e6 (3.97)	2303 (1.49)	5114 (3.30)
MGR	10936 (7.06)	414 (3.57)	9177 (79.21)	24.37e6 (10.14)	3349 (2.16)	6141 (3.96)
W180-N168	10837 (6.99)	662 (5.72)	8860 (76.47)	36.79e6 (15.31)	3716 (2.40)	5816 (3.75)

Table 7—Analysis Types Applicable to Caltrans Bridges

Bridge Classification	Nonlinear Static		Dynamic		
	Equivalent Static Analysis (ESA)	Incremental Static Analysis (Pushover)	Response Spectrum Analysis (RSA)- Linear	Time History Analysis (THA)- Direct integration	
				Linear	Nonlinear
Ordinary Standard	A	R	A	A	A
Ordinary Non-standard	N	R	A	A	R
Important	N	R	A	A	R

N: Not acceptable analysis type; A: Acceptable analysis type; R: Acceptable and strongly recommended analysis type, not necessarily comprehensive

Table 8—Modal Periods of La Veta Bridge Obtained from SAP2000 and OpenSees

Mode	Periods, sec			
	SAP2000 Elastic model	OpenSees Elastic model	Δ_{OS-SAP} (%)	OpenSees Nonlinear model (Pre-earthquake)
Longitudinal translation	1.183	1.176	0.6	1.091
Transverse translation	0.643	0.641	0.3	0.645
Vertical superstructure deformation (S-shape)	0.561	0.558	0.5	0.542
Global torsion	0.447	0.465	3.9	0.464
Vertical superstructure deformation (W-shape)	0.429	0.426	0.7	0.425

Table 9—Bias Factors Computed for Each Bridge for Different Hazard Levels, According to Regression Results: Longitudinal, Transverse, Combined-SRSS, and Mean Responses

Classification	Bridge	Hazard level		
		Low (50%-in-50yr PE)	Moderate (10%-in-50yr PE)	High (2%-in-50yr PE)
Short (simplified abutment)	Route 14	1.18, 1.18, 1.19, 1.18	1.30, 1.18, 1.15, 1.21	1.37, 1.18, 1.13, 1.23
	Adobe	1.14, 1.12, 1.14, 1.13	1.28, 1.14, 1.11, 1.18	1.36, 1.16, 1.10, 1.21
	LADWP	1.03, 1.00, 1.10, 1.05	1.26, 1.16, 1.18, 1.20	1.40, 1.26, 1.22, 1.29
	La Veta	1.00, 1.17, 1.11, 1.09	1.30, 1.38, 1.21, 1.30	1.49, 1.50, 1.27, 1.42
Long (roller abutment)	MGR	1.09, 1.11, 1.14, 1.12	1.23, 1.17, 1.18, 1.19	1.32, 1.20, 1.19, 1.24
	W180-N168	1.25, 1.18, 1.05, 1.16	2.15, 2.02, 1.85, 2.01	2.87, 2.69, 2.48, 2.68

Table 10—Bias Factors Computed for Six bridges Analyzed by Regressions on Bias Factors Obtained from OpenSees and SAP2000 Pair-Wise Comparison: Longitudinal, Transverse, Combined-SRSS, and Mean Responses

Classification	Bridge	Hazard level		
		Low (50%-in-50yr PE)	Moderate (10%-in-50yr PE)	High (2%-in-50yr PE)
Short (Simplified abutment)	Route 14	1.13, 1.12, 1.20, 1.15	1.20, 1.16, 1.12, 1.16	1.23, 1.18, 1.09, 1.17
	Adobe	1.10, 1.06, 1.13, 1.09	1.20, 1.12, 1.10, 1.14	1.26, 1.16, 1.09, 1.17
	LADWP	0.98, 0.97, 1.10, 1.01	1.25, 1.12, 1.19, 1.19	1.42, 1.21, 1.24, 1.29
	La Veta	1.00, 1.15, 1.13, 1.09	1.36, 1.22, 1.23, 1.27	1.59, 1.25, 1.28, 1.38
Long (Roller abutment)	MGR	1.07, 1.10, 1.09, 1.09	1.35, 1.40, 1.34, 1.36	1.52, 1.58, 1.50, 1.54
	W180-N168	1.29, 1.17, 1.03, 1.16	2.31, 2.11, 2.02, 2.15	3.15, 2.88, 2.89, 2.97

Table 11—Summary of Average Bias Factors Results and Corresponding Standard Deviations

Bridge Type	Method	Hazard level		
		Low (50%-in-50yr PE)	Moderate (10%-in-50yr PE)	High (2%-in-50yr PE)
Short	1-Regression	1.11, 0.06	1.22, 0.05	1.29, 0.09
	2-Pair-wise	1.09, 0.06	1.19, 0.06	1.25, 0.10
Long	1-Regression	1.14, 0.03	1.60, 0.58	1.96, 1.02
	2-Pair-wise	1.13, 0.05	1.75, 0.56	2.25, 1.01

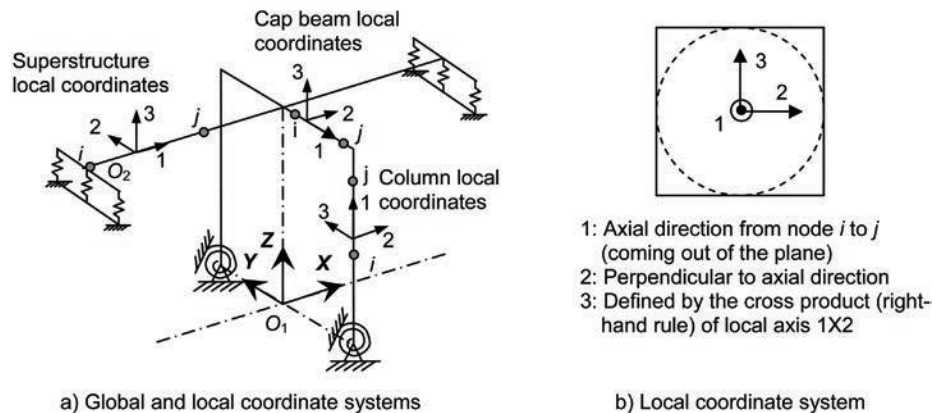


Fig. 1—Coordinate systems and degrees of freedom.

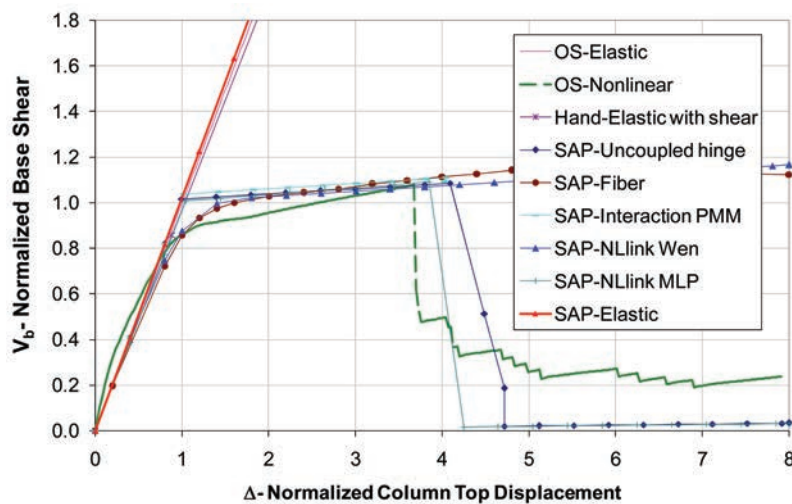
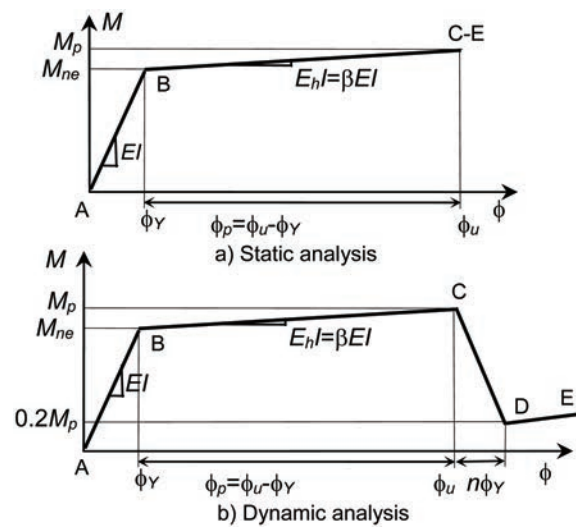
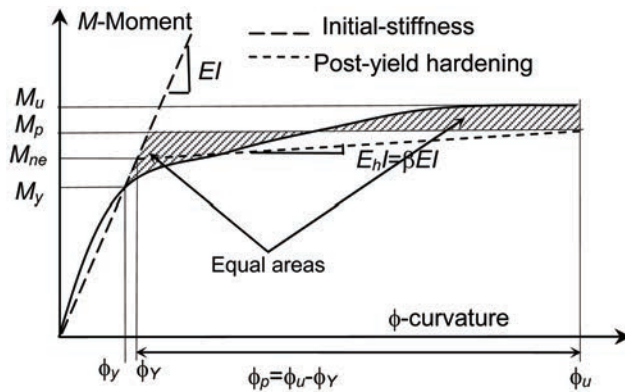


Fig. 4—Pushover curves of SAP2000 and OpenSees column plastic hinge models at principal axes X or Y.

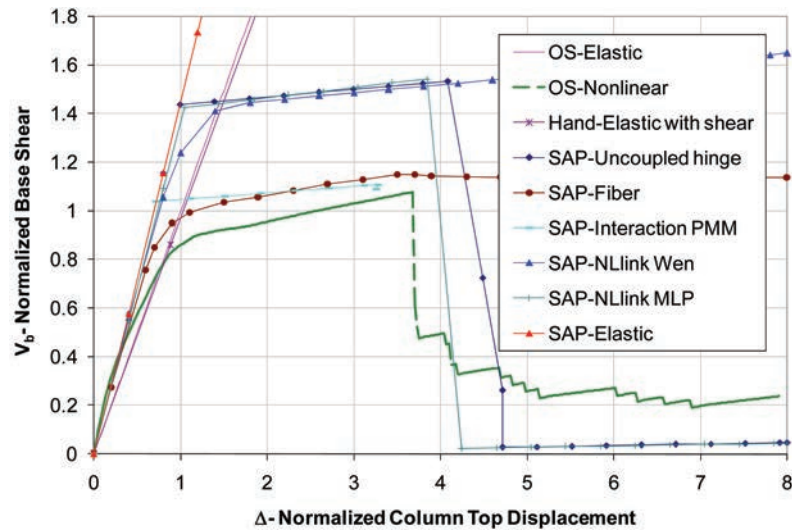


Fig. 5—Pushover curves of SAP2000 and OpenSees column plastic hinge models at a 45 degree angle with respect to principal axes X or Y.

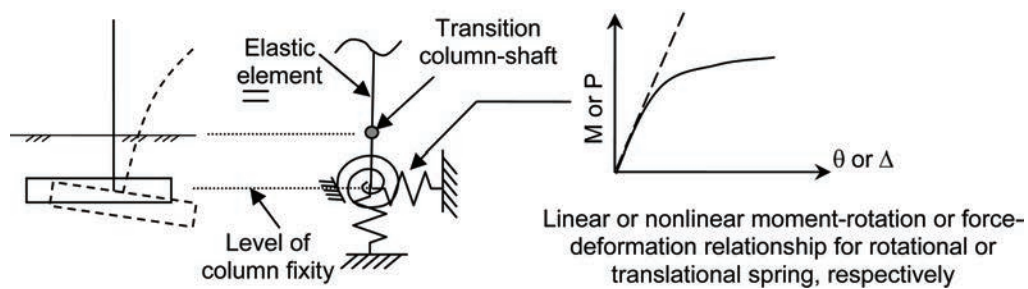


Fig. 6—Modeling scheme of flexible foundations with partial lateral restraint.

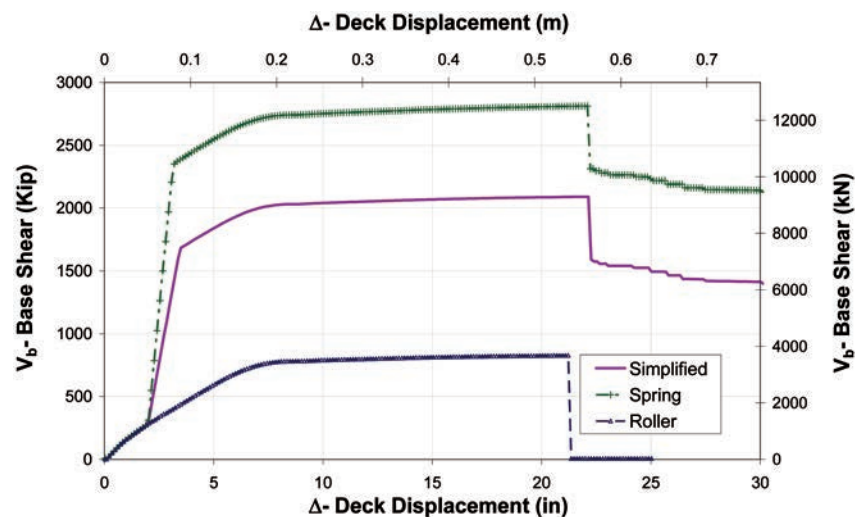


Fig. 7—Longitudinal pushover response of an example bridge using different abutment models.

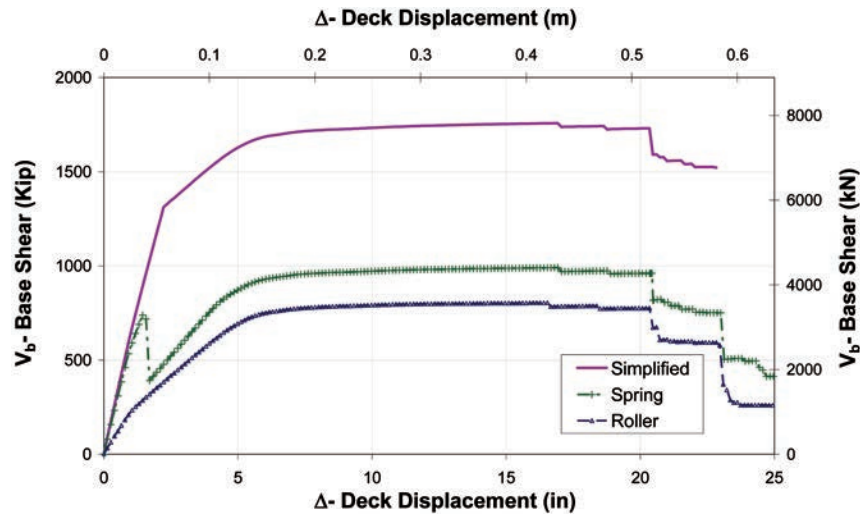


Fig. 8—Transverse pushover response of an example bridge using different abutment models.

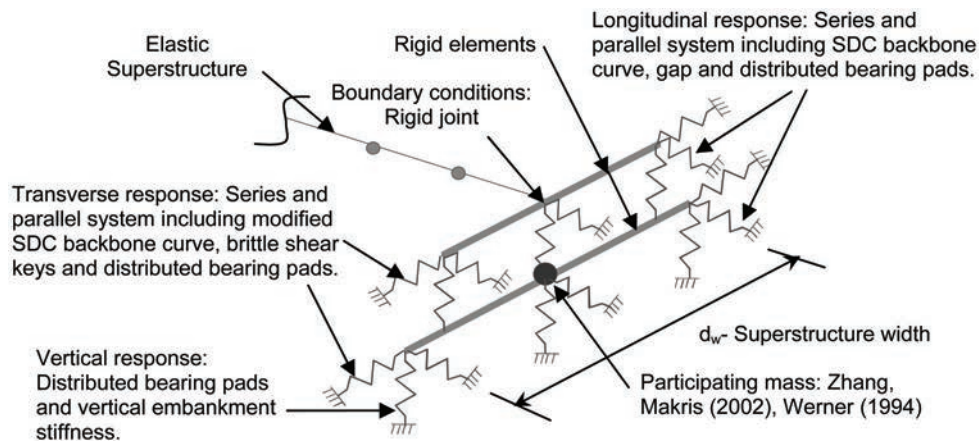


Fig. 9—General scheme of the Spring Abutment model.

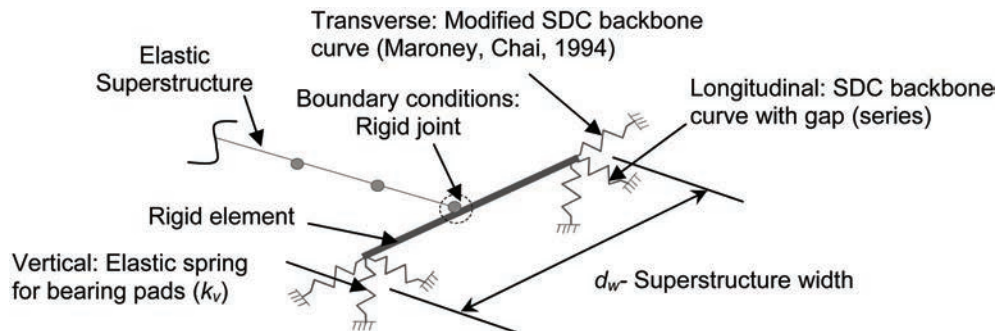


Fig. 10—General scheme of the Simplified abutment model.

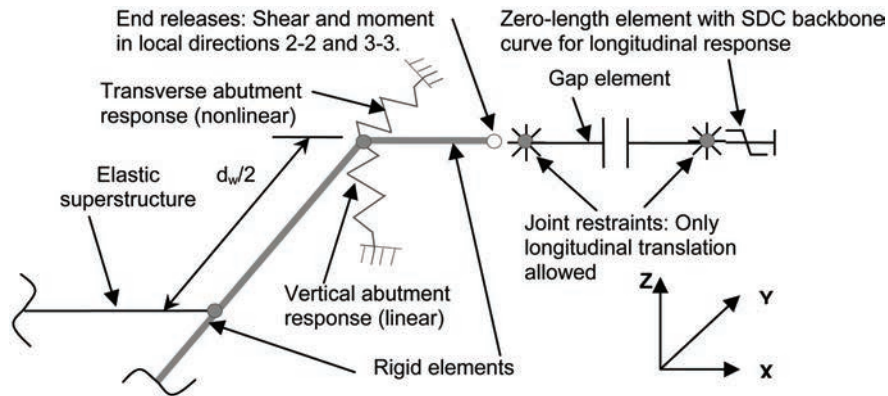


Fig. 11—Series system for the longitudinal simplified abutment response.

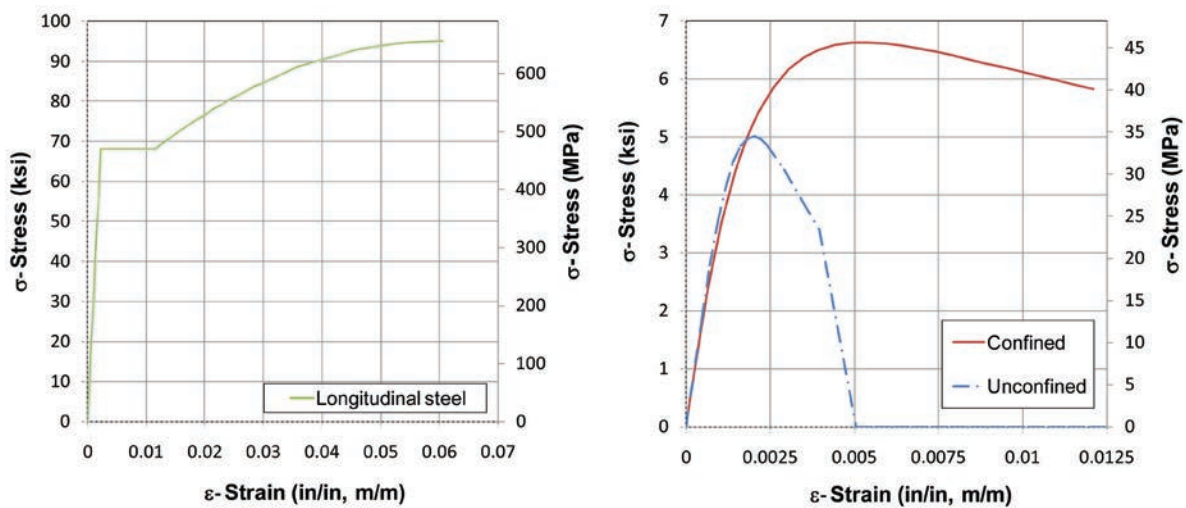


Fig. 12—Stress-strain relationship of steel and concrete used in La Veta bridge models.

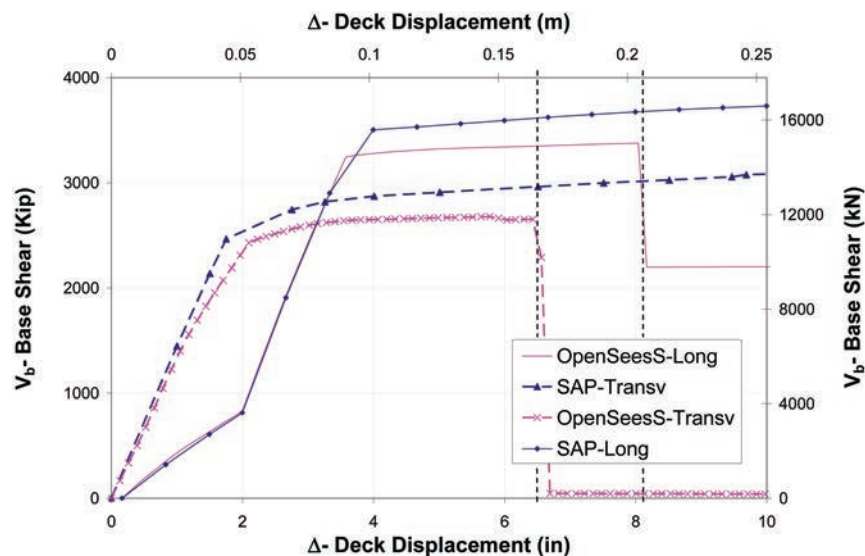


Fig. 13—Pushover curves of La Veta bridge obtained from SAP2000 and OpenSees.

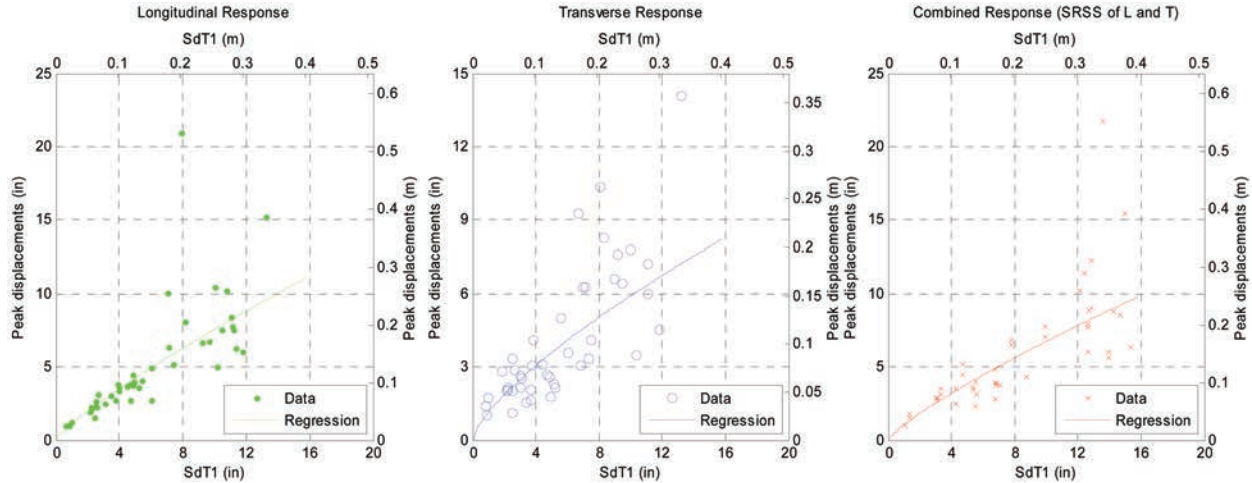
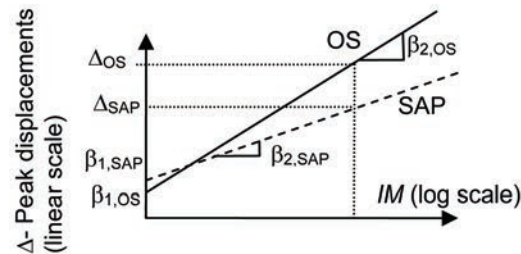


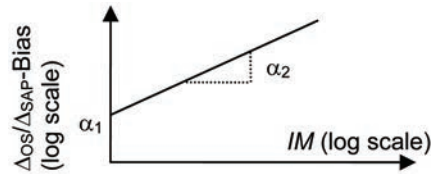
Fig. 14—Nonlinear time history analysis results for OpenSees model of La Veta bridge.



Bias factor (unparallel regression lines):

$$\frac{\Delta_{OS}}{\Delta_{SAP}} = e^{(\beta_{1,OS} - \beta_{1,SAP})} \cdot IM^{(\beta_{2,OS} - \beta_{2,SAP})}$$

Fig. 15—Method 1 for computing OpenSees-SAP2000 bias factors.



$$\frac{\Delta_{OS}}{\Delta_{SAP}} = e^{\alpha_1} \cdot IM^{\alpha_2} = e^{(\beta_{1,OS} - \beta_{1,SAP})} \cdot IM^{(\beta_{2,OS} - \beta_{2,SAP})}$$

Regression of bias factors:

$$\begin{aligned} \ln\left(\frac{\Delta_{OS}}{\Delta_{SAP}}\right) &= \alpha_1 + \alpha_2 \cdot \ln(IM) \\ &= (\beta_{1,OS} - \beta_{1,SAP}) + (\beta_{2,OS} - \beta_{2,SAP}) \cdot \ln(IM) \end{aligned}$$

Fig. 16—Method 2 for computing OpenSees-SAP2000 bias factors.

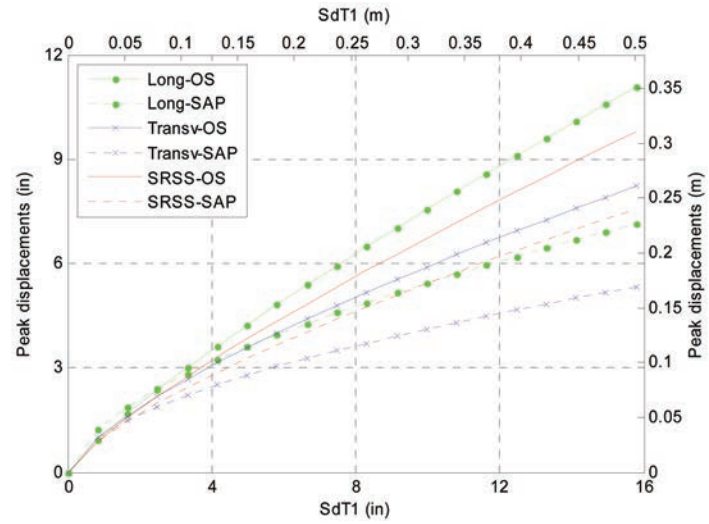


Fig. 17—Comparison SAP2000 and OpenSees regressions for La Veta bridge.

Seismic Damage States and Response Parameters for Bridge Columns

by A. Vosooghi and M. S. Saiidi

Synopsis: Detailed data from 30 bridge column models, mostly tested on shake tables, were evaluated to determine the correlation between the apparent damage and internal and external seismic response parameters. The majority of the columns were of circular cross section. The columns were divided into five categories based on seismic design, shear demand, and ground motion type. Five distinct damage states were proposed based on the apparent damage. Six response parameters were also defined in terms of drift, frequency, strains, yielding, and ultimate displacements. The correlation between damage states and the response parameters were determined for each column category. The results demonstrate a general trend relating damage states to response parameters. Using the observed damage and the data presented in this article it was shown that an estimate of a non-dimensionalized push over curve may be made.

Keywords: frequency ratio; inelasticity index; maximum drift ratio; reinforced concrete column; residual drift ratio; ultimate drift ratio; yielding drift ratio.

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INTRODUCTION

Current seismic design practice in bridge engineering for standard bridges allows for damage to bridge columns during moderate and strong earthquakes. The target response under the maximum credible earthquake is “no-collapse”, realizing that the structure would undergo considerable nonlinearity associated with extensive concrete damage, yielding of bars, or even rupture of some of the bars. For the more frequent earthquakes, the target response is repairable damage that would allow for relatively rapid restoration of the bridge and the highway network.

While numerous past studies have focused on retrofit of undamaged structures, very few investigations have addressed repair strategies (9). To enable engineers to come up with a repair design quickly, it is essential to provide them with tools that inform them about the status of reinforced concrete bridge columns after an earthquake.

It is reasonable to assume that field engineers would have to rely on visual evidence of damage to quickly decide the next course of action that could include bridge closure with important consequence on the highway network operation.

Explicit data on the relationship between the apparent damage and the internal strains and deformations of columns subjected to earthquakes available to engineers to aid in the decision making process is rare. Veletzos (11) et al. (2008) provided tools for post-earthquake inspection and assessment for bridge columns using test data. The column performance was categorized into ductile, strength degrading, and brittle groups. A flow chart was developed to relate column performance to the general column properties. Five damage levels, including failure, were defined. A table was provided to predict the failure mode using field observations. The focus was on the push over response, and internal response parameters were not correlated with the apparent damage.

The bulk of past research on the seismic response of reinforced concrete bridge columns allows study of the correlation between the apparent damage and internal strains or the history of the response that the bridge has experienced. Although the number of past seismic tests of columns is large, comprehensive reports presenting detailed measured internal and external data are scarce. The PEER (<http://nisee.berkeley.edu/spd/>) is one of the largest available databases. It describes 274 tests of rectangular reinforced columns and 160 tests of spiral-reinforced columns, all subjected to cyclic lateral loading. For each test, the database provides a reference, column top force-displacement histories, key material properties, and the description of the column geometry. These data could not be of use in the present study because they lack photographs showing the appearance of columns under different drifts and they do not provide steel reinforcement strain data. However, the large database of numerous columns that have been studied at the University of Nevada, Reno (UNR) are available and present an opportunity to quantify the correlation between response parameters and the observed damage.

The objective of this study was to review past shake table and cyclic load test data on columns to establish correlation between different damage states and important response parameters. The ultimate goal of the project is to utilize these correlations in deciding the repair design for rapid restoration of service of the bridge.

RESEARCH SIGNIFICANCE

The serviceability of bridges is often in question after moderate or strong earthquakes. Based on post-earthquake inspection, engineers have to decide if the bridge is sufficiently safe to be kept open to traffic without repair, it is repairable within a reasonable time frame, or it should be replaced. To assist

engineers in making these decisions, it is necessary to define visual damage states that can be identified quickly in the field. Using past detailed data on the seismic response of bridge column models it is possible to correlate visual damage with important response parameters such as drifts and strains in the reinforcement. This article presents the results of a study showing the correlations with the hope of aiding engineers with making informed yet practical decision about the repair strategy.

Experimental procedure

Test results for thirty columns tested at the University of Nevada, Reno (UNR) were used in this study. The columns were of 1/5 to 1/3 geometric scale and all but one were tested on a shake table. Real concrete and steel reinforcement were used and stresses were not scaled. The input earthquakes consisted of a variety of motions including a few spectrum matched motions. Near-field motions were also included in the testing protocol. The ground motions were generally applied in multiple tests with gradually increasing amplitudes. As expected, multiple motions generally impose more damages to the column compared to a single motion. However, this does not affect the results of this study, because the objective of the current study was to correlate the visual damage to the internal and external response parameters notwithstanding the amplitude of the motion. Both “standard” and “sub-standard” column models were included, although the majority of the columns were in the former category. The columns meeting the recent and current seismic design provisions were placed in the standard category and those designed prior to the 1970s were grouped in the sub-standard category. Column models in the latter category did not meet the current seismic design and detailing requirements. The standard columns exhibited relatively large ductility and failed under flexure or shear/flexure. The sub-standard models failed in a brittle mode under shear or lap-splice debonding. The concrete strength was typically in the range of 4 to 6 ksi (28 to 42 MPa), and the reinforcement yield stress was in the range of 60 to 70 ksi (420 to 490 MPa) for standard columns and 40 to 50 ksi (280 to 350 MPa) for sub-standard columns.

Twenty four standard column models and six sub-standard column models were in the database. The standard columns were divided into two groups based on the type of the earthquake records (Table 1): (1) columns subjected to far-field motions and (2) columns subjected to near-field motions. The former was further divided into two categories of (1.a) low shear columns and (1.b) high-shear columns. Details of the standard columns are listed in Table 2. The sub-standard columns were also placed in two categories (Table 3). Category one consisted of columns in a two-column bent with weak columns and strong cap beam. Category two consisted of columns in two, two-column bents with strong columns and weak cap beams (Table 3, Fig. 6).

1.a. Standard columns with low shear under far-field motions (SLSF)

The standard columns with the shear stress index (SSI) of less than four were placed in this category. The shear stress index is calculated by the following equation:

$$SSI = \frac{V}{A_e \times \sqrt{f'_c}} \text{ (lb, in) or } SSI = \frac{V}{A_e \times \sqrt{f'_c} \times 0.083} \text{ (N, mm)} \quad (1)$$

where V = Shear demand in the column; A_e = Effective shear area, taken as 80% of the gross area; and f'_c = 28-day compressive strength of concrete.

Single columns with circular shape and oval shape with interlocking spirals and columns of two, two-span-bridge models were included in this category. All the single columns were tested as cantilever elements in single curvature. The shake table test setup of the single columns with low shear demand is shown in Fig. 1 as a sample. Due to safety consideration, the inertial mass blocks were supported away from the shake table on a mechanism frame (the mass rig). The mass rig was attached to the column head using a rigid link. Both ends of the link were pinned. During the test, the load cell in the link measured the applied lateral force to the column including the damping and $P-\Delta$ forces caused by weight on the mass rig and its lateral drift, which was the same as the column drift. The axial force was applied using a post tensioning rods attached to the column head. The input ground motion and the details of the columns are listed in Table 2. The cross sections of the oval columns are shown in Fig. 2.

The 1/4 scale two-span-bridge model is shown in Fig. 3. The bridge was composed of three, two-

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column bents with round columns. The results from two, two-span bridge model tests were used in the database. One bridge was subjected to the Northridge ground motion, and the other was subjected to input motions that simulated fault rupture. The “fault rupture” occurred in between Shake [Tables 2](#) and [3](#) which induced the most severe damages to the middle bent. The two other bents remained nearly elastic. All six columns of the first bridge and the columns of the middle bent of the second model were included in the database. Only one column, RSC, was subjected to a quasi-static test.

1.b. Standard columns with high shear under far-field motions (SHSF)

The standard columns with the shear stress index (SSI) greater than six were placed in this category. SSI is calculated using equation (1). All models in this category were single columns. The test setup for a column with high shear is shown in [Fig. 4](#). The setup is similar to low shear column setup except that two, rather than one, rigid links were used to prevent rotation at the top of the column. Using this setup, a column bends in double curvature and a relatively high shear may be applied. One round column and four oval columns with interlocking spirals were tested in this category. The details of the columns are listed in [Table 2](#). The cross sections of the oval columns are shown in [Fig. 5](#).

2. Standard columns with low shear subjected to near-field motions (SLSN)

Six single round columns were tested in this category ([Table 2](#)). All the columns were subjected to a near-field ground motion. “Near-field” or “near-fault” is defined as a region within 10 miles (16 km) of the fault.

The reason for defining a separate category for columns subjected to a near-field ground motion was that previous studies^(1, 6) show that the high velocity pulse in near-field ground motions was found to cause large residual drifts even under moderate motions. All the column models test setups were similar to that of SLSF ([Fig. 1](#)).

Sub-standard columns (SS1 and SS2)

The sub-standard columns do not meet the modern seismic design requirements. They were mostly designed and constructed in the 1960s. These columns had both inadequate and unreliable strength due to insufficient confinement and lap-spliced longitudinal bars. Low confinement reinforcement ratios did not provide the level of ductility needed to survive most earthquake events.

Three sub-standard two-column piers were tested. Based on the relative strength of the column to the cap-beam, the test models were placed in two categories. The first category, SS1, consisted of piers with weak columns and a strong cap-beam. In this category, the columns failed, although the damage in the beam was also significant. One, two-column pier with diamond shape columns (B2DA) was in category SS1. These columns were pinned to the footing and fixed to the cap beam. The second category, SS2, consisted of piers with strong columns and a weak cap-beam. In this category, the cap-beam failed although the column damage was also significant. Two, two-column piers (B2RA1 and B2RA2) with rectangular columns were in category SS2. The columns were fixed at both ends with lap spliced longitudinal bars at the base. Information about the section, reinforcement, etc of models is listed in [Table 3](#) and the cross section details are shown in [Fig. 6](#). The piers were tested in the strong direction.

DAMAGE STATES

Five distinct damage states were proposed based on the review of the shake table test data for thirty reinforced concrete bridge columns ([Fig. 7](#)). The damage states identify five repairable apparent damage levels excluding the failure, and they were mostly arrived at using tests of bridge column models on shake tables. A bridge is inspected after an earthquake when seismic forces are no longer acting on the structure. Accordingly, the damage states for the models were evaluated in between shake table runs or at zero lateral force when using static load test data. The damage states are used as a benchmark to determine the damage level in the reinforced concrete bridge columns after an earthquake.

Deciding the number of damage states is somewhat subjective. The proposed damage states were discussed with a group of leading earthquake engineers in charge of bridge inspection in the California Department of transportation (Caltrans), and the group concluded that the proposed states are reasonable and practical. The proposed damage states are:

1. DS-1: Flexural cracks

2. DS-2: First spalling and shear cracks
3. DS-3: Extensive cracks and spalling
4. DS-4: Visible lateral and longitudinal bars
5. DS-5: Imminent failure

The standard columns were essentially flexural members mostly controlled by flexure and some controlled by flexure/shear. Due to a lack of adequate transverse reinforcement in the sub-standard columns, their response was mostly controlled by shear. Damage state 1 (DS-1) corresponds to flexural cracks and no other damage observed after an earthquake. Note that more cracks beyond those observed are generally developed during the earthquake, but these cracks are closed by the axial load. Damage state 2 (DS-2) consists of minor spalling and might include minor shear cracks. A column in damage state 3 (DS-3) exhibits extensive cracking both in terms of the number and crack width and its unconfined concrete spalling covers a relatively large area. Under damage state 4 (DS-4) one or more spirals and possibly one or more of the longitudinal bars are visible. This indicates the loss of the unconfined concrete. The fifth damage state (DS-5) corresponds to the case in which the column is approaching failure and damage has begun to penetrate the confined core. In addition, there might be signs of slight longitudinal bar bending either in between spiral hoops or the entire column reinforcement cage showing signs of the beginning of buckling. An example of each damage state is shown in Fig. 7.

DAMAGE STATES AND PUSHOVER CURVES

Each column model was tested under multiple shake table runs. A ground motion with gradually increasing amplitudes was applied to the column model to allow capturing all damage states. A pushover curve was developed based on the measured peak lateral forces with the corresponding displacements for all the runs. The failure point for the envelope curve was assumed either by the peak displacement with the corresponding force or 80 percent of the maximum force with the corresponding displacement when the force for the peak displacement dropped more than 20 percent of the maximum force. Fig. 8 shows a sample pushover curve for each column category. The maximum measured displacement for the run leading to a given damage state is marked on the curve. The marked points represent the column status at each damage state. The standard columns experienced all damage states before failure. The sub-standard columns in the bent with weak columns failed immediately after DS-3. As expected the standard columns exhibited considerably larger ductility capacity.

RESPONSE PARAMETERS

To capture the internal and the external response of a bridge column during an earthquake, six response parameters were defined. Under shake table tests, a bridge column underwent a ground motion with gradually increasing amplitudes that allowed distinguishing and recording information for generally all damage states. Except for residual drift ratio (RDR), all the response parameters reflected the data collected during the shaking preceding a given damage state. The response parameters were translated in non-dimensional and are explained in the following sections. The first four parameters are external and the latter two are internal to the column.

Maximum Drift Ratio (MDR)

The maximum lateral displacement of a bridge column during an earthquake can be a measure of the extent of nonlinearity that the column has experienced and may be useful in post-earthquake evaluation. In this paper, the maximum drift ratio (MDR) for each test leading to a given damage state was calculated as follows:

$$MDR = \frac{D_{max}}{H} \quad (2)$$

where D_{max} = the maximum lateral displacement of the column during the test leading to a given damage state; and H = the column height.

Residual Drift Ratio (RDR)

The residual drift is the only response parameter that can be measured after the earthquake directly.

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The residual drift ratio (RDR) was calculated as follows:

$$RDR = \frac{D_{res}}{H} \quad (3)$$

where D_{res} = the residual displacement at the end of the test resulting in a given damage state; and H = the column height.

Frequency Ratio (FR)

The variation of frequency is a measure of the square root of the column stiffness change. The ratio of the effective natural frequency of a column during a test to its uncracked natural frequency was defined as frequency ratio (FR):

$$FR = \frac{F_{eff}}{F_{uncracked}} \quad (4)$$

where F_{eff} = effective natural frequency of the column; and $F_{uncracked}$ = uncracked natural frequency of the column.

The effective natural frequency was calculated using the chord stiffness of the column, which was determined by dividing the lateral force at the peak displacement to the peak displacement during the run leading to different damage states. The uncracked natural frequency was calculated for each column based on the results from quick release or white noise tests applied prior to earthquake simulation. The fast Fourier transformation (FFT) of the measured acceleration of the column top was used to find the natural frequency. In the case of quasi-static test, the uncracked natural frequency of the column was calculated based on the initial stiffness of the column. The frequency ratio (FR) was calculated for each column during the test leading to a given damage state.

Inelasticity Index (II)

The inelasticity index (II) was introduced to locate the column status on the pushover curve. The inelasticity index is calculated as follows:

$$II = \frac{D_{max} - D_Y}{D_u - D_Y} \quad (5)$$

where D_{max} = the maximum lateral displacement of the column during the test leading to a given damage state; D_u = ultimate (failure) displacement of the column; and D_Y = effective yield displacement of the column.

To find the effective yield displacement of the column, the measured force-displacement envelopes were idealized by elasto-plastic curves. Each envelope was idealized by setting the initial slope to pass through the first yield point and adjusting the plastic portion to preserve the area under the measured force-displacement envelope. The maximum inelasticity index is one and corresponds to failure. A zero inelasticity index indicates that the column displacement and force are at the effective yield point. When the index is negative the column is still in the linear range. The inelasticity index was only calculated for the columns that were tested to failure.

Maximum Longitudinal Steel Strain

The maximum longitudinal steel strain represents the internal flexural response of the column during the test leading to a given damage state. This is an indicator of the status of the bars relative to yielding. In addition, the status of the flexural steel can be identified relative to rupture. To determine the maximum strain in the longitudinal steel for each damage state, the following steps were taken:

- Since the tensile strains are positive and the compressive ones are negative, the absolute measured values for all strain gages were determined.
- In each damage state, the three highest absolute strains were selected. In this step, the unreliable

values were neglected. These values were recognized by comparing the recorded strains at or near the same locations.

- The average of the three quantities during the test leading to a given damage state was assumed as the maximum longitudinal steel strain related to that damage state.

Maximum Transverse Steel Strain

The maximum transverse steel strain represents the internal response of the column section in the transverse direction during the test leading to a given damage state. This parameter may be used to assess the contribution of the transverse reinforcement to shear and confinement.

To determine the maximum strain in the transverse steel for each damage state, the following steps were taken:

- Since the transverse steel acts in tension to carry part of the column shear and to provide confinement, only the measured tensile strains were considered.
- For the earthquake run prior to each damage state, the two highest tensile strains were selected. In this step, unreliable values were neglected. These values were recognized by comparing the recorded strains among the adjacent strain gages. For shear reinforcement, gages on the opposite sides of the reinforcement at the same height were used to identify unreliable data.
- The average of the two quantities during the test leading to a given damage state was assumed as the maximum transverse steel strain related to that damage state.

CORRELATION BETWEEN DAMAGE STATES AND RESPONSE

The bridge column models were subjected to shake table motions (or quasi-static loading) with gradually increasing amplitudes. At the end of each earthquake run, the visual evidence of damages was investigated to determine the damage state. For each damage state, six response parameters were calculated. The parameters were determined during or following the run leading to a given damage state. For each category of the columns, the mean data (μ) were obtained and marked in addition to the mean $\pm$ one standard deviation ($\mu \pm \sigma$). In some categories, only a few column models existed in the data base, and hence the results of this study should be used taking into account the limited number of data.

The combined data for each column category are plotted in Fig. 9 to Fig. 14. Except for frequency ratios (FR), all other response parameters increased as the damage state changed from DS-1 (flexural cracks) to DS-5 (imminent failure). The bars corresponding to the mean plus or minus one standard deviation reflect the amount of scatter in the data. They were shown for the categories containing three or more column models.

Maximum Drift Ratio (MDR)

Fig. 9 shows the variation of the maximum drift ratio (MDR) versus damage states. The significance of these charts is that they provide an estimate of the maximum deformation that the column has undergone based on the observed damage. For example when longitudinal and transverse bars are exposed (DS4), a standard column with low shear is likely to have experienced approximately a maximum drift of 6%. Among the all categories, standard columns subjected to near-field earthquakes experienced the largest lateral displacements. The data for the low and high shear columns were close and might be combined together. The sub-standard pier with weak column and strong cap-beam (SS1) failed immediately after damage state 3 (DS-3). The mode of failure was shear. The shear failure occurred in one of the columns of the pier at a low MDR. The sub-standard pier with strong column and weak cap-beam (SS2) sustained larger drift than that of SS1. The reason was that the cap-beam failed prior to the column and reduced the demand on the columns.

Residual Drift Ratio (RDR)

Fig. 10 shows the variation of the residual drift ratio versus damage states. The residual drift was negligible for the first three damage states. Damage states 4 and 5 were associated with significant residual drift ratios in the columns subjected to near-field earthquakes. For instance, when the longitudinal and transverse steel of the column are visible (DS-4), the average residual drift ratio under near field earthquakes is 5.8%, whereas the average residual drift ratio under a far field motion is only 0.7%.

Frequency Ratio (FR)

The variation of frequency ratio as a function of damage state is shown in Fig. 11. Among standard columns, the frequency ratio is higher in columns with high shear in all damage states. This implies that the stiffness of these columns degrades less rapidly than the columns in other categories as the intensity of the motion is increased. For instance, in DS-3, the frequency ratio of high shear columns is 62% whereas it is 39% for the low shear columns. In other words, in DS-3, the stiffness of the columns with high shear is 38% of the uncracked columns and for those with low shear the stiffness drops to 15% of the uncracked columns. This is attributed to the fact that flexural cracking tends to reduce stiffness more than shear cracking does.

Inelasticity Index (II)

The variation of the inelasticity index (II) as a function of damage state is shown in Fig. 12. In columns with low shear, II is approximately zero for DS-1. This implies that the visual flexural cracks appear under approximately the effective yield displacement of the column. Negative values are observed for those with high shear and sub-standard columns in category SS1, which mean that the visual flexural cracks appear before yielding in these columns. The positive II in near-field columns for DS-1 indicate that yielding begins before flexural cracks are observed.

Because sub-standard columns in category SS2 did not fail, no II was calculated for them. Under DS-3, the II is approximately between 30% and 40% for all the standard columns, while it is 88% for the sub-standard columns. This value is approximately equal to II under DS-5 in standard columns. Consequently, extensive cracks and spalling (DS-3) in sub-standard columns represent imminent failure.

Maximum Longitudinal Steel Strains

Fig. 13 shows the variation of the maximum longitudinal steel strains versus the damage states. The yield strain in the longitudinal bars of the standard columns was 2000 to 2400 microstrains and that of the substandard columns was 1400 to 1700 microstrains. The strains are considerably lower in the sub-standard columns. The low strains in sub-standard columns imply that very small plastic deformations occurred and low ductility was achieved prior to failure.

The maximum longitudinal steel strains for column model SVTN, placed in near-field zone category, was erratic and hence were excluded. The maximum longitudinal steel strain data under damage state 5 (DS-5) for MN (another column in the near-field category) was also excluded because it showed inconsistent trend. Column SETN, placed in near-field zone category, had considerably higher strains in longitudinal reinforcement than those of other columns in the same category. No data were available for SETN under DS-5; therefore, the maximum strain for DS-4 was used for DS-5 as well.

The graphs indicate that all standard columns show similar trends in all damage states. In addition, even under DS-1 in which only flexural cracks are visible, some of the extreme longitudinal bars had yielded.

Maximum Transverse Steel Strains

The variation of the maximum transverse steel strains as a function of damage states is shown in Fig. 14. The average data show that the transverse reinforcement yields under DS-4 and DS-5 in standard columns under high shear or near fault motions. Unless these bars exhibit significant strain hardening, yielding of the transverse steel could reduce their effectiveness in providing confinement.

Transverse reinforcement yielded in sub-standard columns in category SS1 under DS-3 prior to failure. Therefore, yielding of the transverse reinforcement in sub-standard columns could indicate that failure is imminent. In sub-standard columns in category SS2, the transverse steel remained elastic in all damage states.

ESTIMATE OF PUSHOVER CURVE USING RESPONSE PARAMETER

Procedure

Using some of the response parameters that are estimated based on visual damage, it is possible to estimate a non-dimensionalized elasto-plastic pushover curve for the column. The pushover curve enables the engineer to determine the column status after an earthquake and estimates its reserve deformation capacity. The non-dimensionalized elasto-plastic pushover curve, ratio of lateral force to plastic lateral force versus drift ratio, can be constructed using the effective yield drift ratio (YDR) and

ultimate (failure) drift ratio (UDR).

Using equations (2) and (5) leads to following equations to calculate the effective yield and ultimate drift ratios:

$$YDR = \frac{D_y}{H} = \frac{II \times MDR_1 - II_1 \times MDR}{II - II_1} \quad (6)$$

$$UDR = \frac{D_u}{H} = \frac{MDR}{II} - \frac{(1 - II)}{II} \times YDR \quad (7)$$

where YDR = the effective yield drift ratio; MDR_1 = the maximum drift ratio in damage state 1 (DS-1); II_1 = the inelasticity index in damage state 1 (DS-1); and UDR = the ultimate (failure) drift ratio.

The remaining parameters were defined previously.

In addition, the ductility demand (μ_d) and ductility capacity (μ_c) can also be obtained from equations (8) and (9), respectively.

$$\mu_c = \frac{UDR}{YDR} \quad (8)$$

$$\mu_d = \frac{MDR}{YDR} \quad (9)$$

The following steps demonstrate the construction of a non-dimensionalized elasto-plastic pushover curve:

Step 1: Inspect the bridge and its location to determine damage state and column category.

Step 2: Use graphs in **Fig. 9** to determine MDR and MDR_1 .

Step 3: Use graphs in **Fig. 12** to determine II and II_1 .

Step 4: Calculate YDR and UDR using equations (6) and (7).

Step 5: Construct the normalized elasto-plastic pushover curve.

Step 6: Calculate the ductility capacity (μ_c) and ductility demand (μ_d) using equations (8) and (9).

Illustrative example

Step 1: Assume a standard column with low shear is subjected to a far field earthquake and the apparent damage is DS4.

Step 2: Using the upper left graph in **Fig. 9**, $MDR = 5.6\%$ and $MDR_1 = 1.9\%$, based on the average values.

Step 3: Using the upper left graph in **Fig. 12**, $II = 44\%$ and $II_1 = 2\%$, based on the average values.

Step 4: Using equations (6) and (7), and the values from steps 2 and 3, $YDR = 1.7\%$ and $UDR = 10.6\%$.

Step 5: Construct the pushover curve as shown in **Fig. 15**.

Step 6: Calculate ductility capacity (μ_c) and ductility demand (μ_d), using equations (8) and (9):

$$\mu_c = UDR/YDR = 10.6/1.7 = 6.2$$

$$\mu_d = MDR/YDR = 5.6/1.7 = 3.3$$

CONCLUSIONS

Detailed data from 30 bridge column models mostly tested on shake tables were evaluated to determine the correlation between the apparent damage and internal and external seismic response parameters. The following conclusions are drawn:

1. The proposed damage states appear to present a reasonable balance in terms of the number and appearance of columns with distinct apparent damage.
2. Although there was scatter in the data, the response parameters showed a clear trend in terms of correlation with different damage states.

3. Standard columns in the near-field zone experienced the largest drift ratios compared to columns in other categories at all damage states due to the impulsive nature of near-field earthquakes.
4. The variation in frequency ratios showed that stiffness degradation is relatively small in flexure dominated columns with high shear.
5. Flexural cracking always indicated that some yielding of the extreme longitudinal bars has taken place.
6. In high shear columns and those subjected to near-field motions, the transverse steel yields under DS-4 (visible longitudinal and transverse steel) and DS-5 (damaged core, imminent failure), making its contribution to confinement questionable.
7. In sub-standard columns, the transverse steel yields under DS-3 (extensive spalling of concrete with no visible reinforcement). Extensive shear cracks and spalling in substandard columns indicate imminent failure.

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Table 1—Standard columns categories

Earthquake Type	Shear Level	
	Low	High
Far Field	SLSF	SHSF
Near Field	SLSN	—

Table 2—Details of standard columns

Column Model	Ref. No.	Scale	Category	Ground Motion	Ground Motion Type	H, in. (mm)	D, in. (mm)	Long. bars, in. (mm)	Spirals, in. (mm)
SC-CAL	8	0.25	SLSF(3)	ATC-32-D Artificial EQ	Far-field	54 (1372)	12 (305)	16-#4 (Φ12.7)	φ0.192@1.6 (φ4.9@40)
SC-PBD	8	0.25	SLSF	ATC-32-D Artificial EQ	Far-field	54 (1372)	12 (305)	16-#4 (Φ12.7)	φ0.192@1.0 (φ4.9@25)
RSC	5	0.2	SLSF	Quasi-static	Far-field	45 (1143)	10 (254)	8-#4 (Φ12.7)	φ0.177@1.5 (φ4.5@38)
B1E-I, B1W-I(1)	4	0.25	SLSF	North-ridge	Far-field	72 (1829)	12 (305)	16-#3 (Φ9.5)	φ0.192@1.25 (φ4.9@32)
B2E-I, B2W-I(1)	4	0.25	SLSF	North-ridge	Far-field	96 (2438)	12 (305)	16-#3 (Φ9.5)	φ0.192@1.25 (φ4.9@32)
B3E-I, B3W-I(1)	4	0.25	SLSF	North-ridge	Far-field	60 (1524)	12 (305)	16-#3 (Φ9.5)	φ0.192@1.25 (φ4.9@32)
B2E-II, B2W-II(2)	4	0.25	SLSF	Fault Rupture	Far-field	96 (2438)	12 (305)	16-#3 (Φ9.5)	φ0.192@1.25 (φ4.9@32)
ISL1.0	2, 3	0.25	SLSF	Sylmar	Far-field	58 (1473)	Details in Fig. 2		
ISL1.5	2, 3	0.25	SLSF	Sylmar	Far-field	72 (1829)			
NHS1	In progress	0.33	SHSF(4)	Sylmar	Far-field	80 (2032)	16 (406)	20-#5 (Φ15.9)	#3 @ 2.25 (φ9.5@57)
ISH1.0	2, 3	0.2	SHSF	Sylmar	Far-field	58 (1473)	Details in Fig. 5		
ISH1.25	2, 3	0.2	SHSF	Sylmar	Far-field	63 (1600)			
ISH1.50	2, 3	0.2	SHSF	Sylmar	Far-field	69 (1753)			
ISH1.50T	2, 3	0.2	SHSF	Sylmar	Far-field	69 (1753)			
NF-1	6	0.33	SLSN(5)	Rinaldi	Near-field	72 (1829)	16 (406)	20-#4 (Φ12.7)	φ0.25@1.5 (φ6.4@38)
NF-2	6	0.33	SLSN	Rinaldi	Near-field	72 (1829)	16 (406)	22-#4 (Φ12.7)	φ0.25@1.25 (φ6.4@32)
MN	1	0.29	SLSN	Rinaldi	Near-field	63 (1600)	14 (356)	22-#4 (Φ12.7)	φ0.25@1.125 (φ6.4@29)
ETN	1	0.29	SLSN	Rinaldi	Near-field	108.5 (2756)	14 (356)	22-#4 (Φ12.7)	φ0.25@1.0 (φ6.4@25)
SETN	1	0.29	SLSN	Rinaldi/RRS(6)	Near-field	108.5 (2756)	14 (356)	18-#5 (Φ15.9)	φ0.25@0.75 (φ6.4@19)
SVTN	1	0.2	SLSN	Rinaldi/RRS(6)	Near-field	98.5 (2502)	12 (305)	17-#4 (Φ12.7)	φ0.25@1.0 (φ6.4@25)

Note: φ stands for deformed rebar; f stands for plain wire; (1) All columns for Two-span-bridge-I; (2) Middle bent columns for Two-span-bridge-II; (3) SLSF: Standard, low shear, far-field; (4) SHSF: Standard, high shear, far-field; (5) SLSN: Standard, low shear, near-field; (6) Synthetic near-field motion.

Table 3—Details of sub-standard columns

Column Model	Ref. No.	Scale	Category	Ground Motion	H, in (mm)	D, in (mm)	Long. Bars, in (mm)	Spirals, in (mm)
B2DA-E, B2DA-W	10	0.25	SS1(1)	Sylmar	60 (1524)	Details in Fig. 6		
B2RA1-E, B2RA1-W, B2RA2-E, B2RA2-W	7	0.25	SS2(2)	Sylmar	48 (1219)			

Notes: (1) Sub-Standard with weak column and strong cap-beam; (2) Sub-Standard with strong column and weak cap-beam.

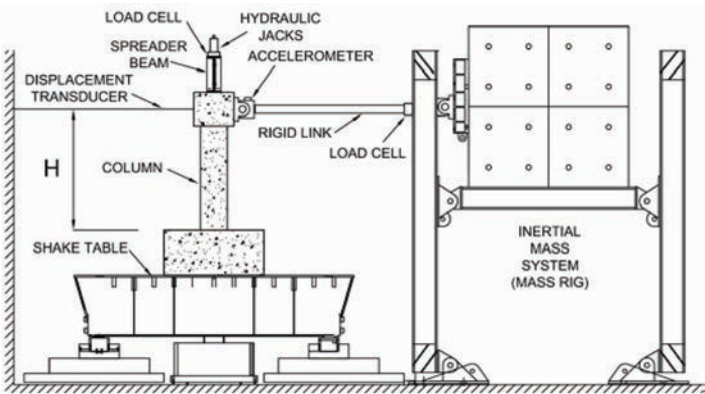


Fig. 1—Test setup for single columns with low shear and sub-standard column bents.

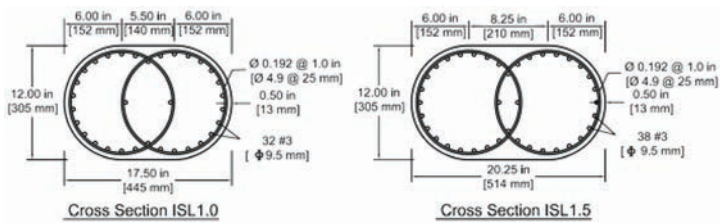


Fig. 2—Cross sections of the oval columns with low shear.

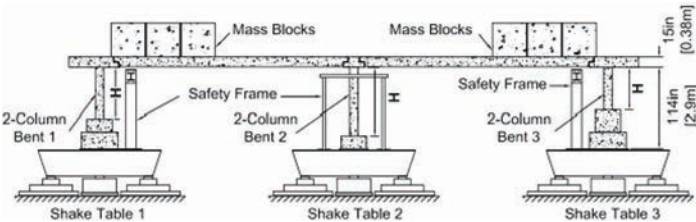


Fig. 3—Two-span-bridge model and test setup.

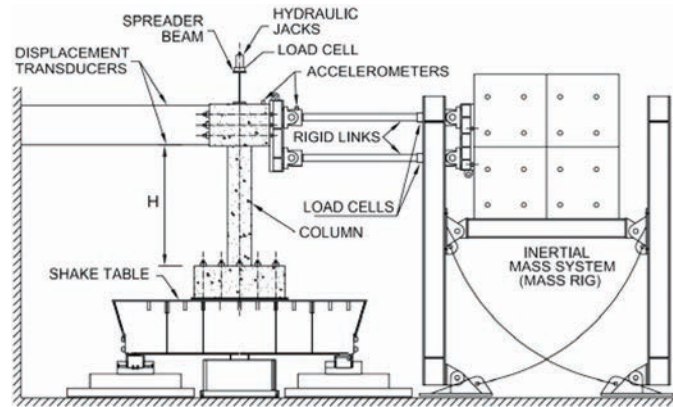


Fig. 4—Test setup for single columns with high shear.

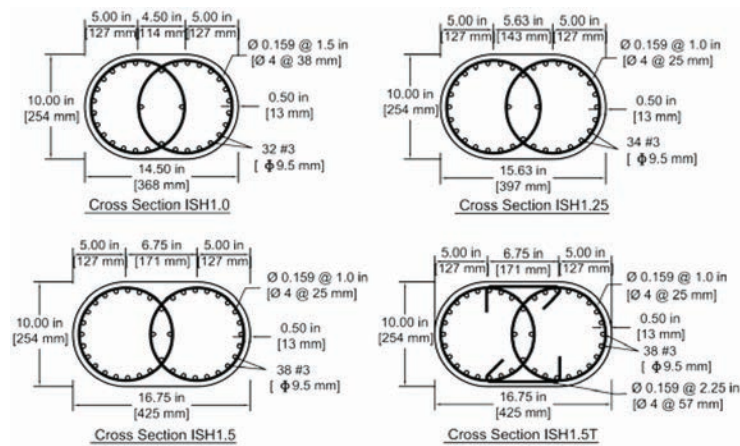


Fig. 5—Cross sections of the oval columns with high shear.

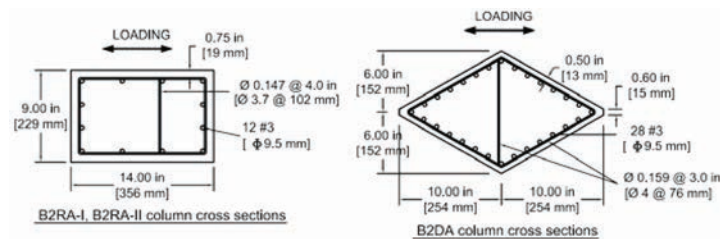
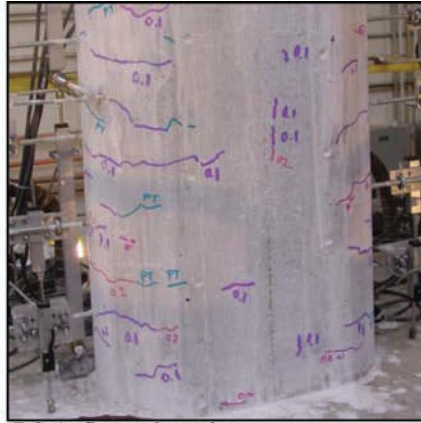
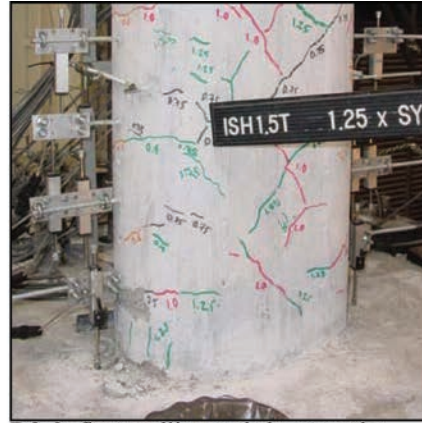


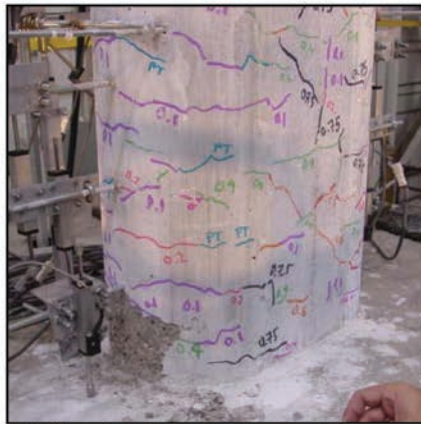
Fig. 6—Cross sections of the sub-standard columns.



DS-1: flexural cracks



DS-2: first spalling and shear cracks



DS-3: extensive cracks and spalling



DS-4: lateral and longitudinal bars visible



DS-5: imminent failure

Fig. 7—Five distinct damage states in the RC bridge columns.

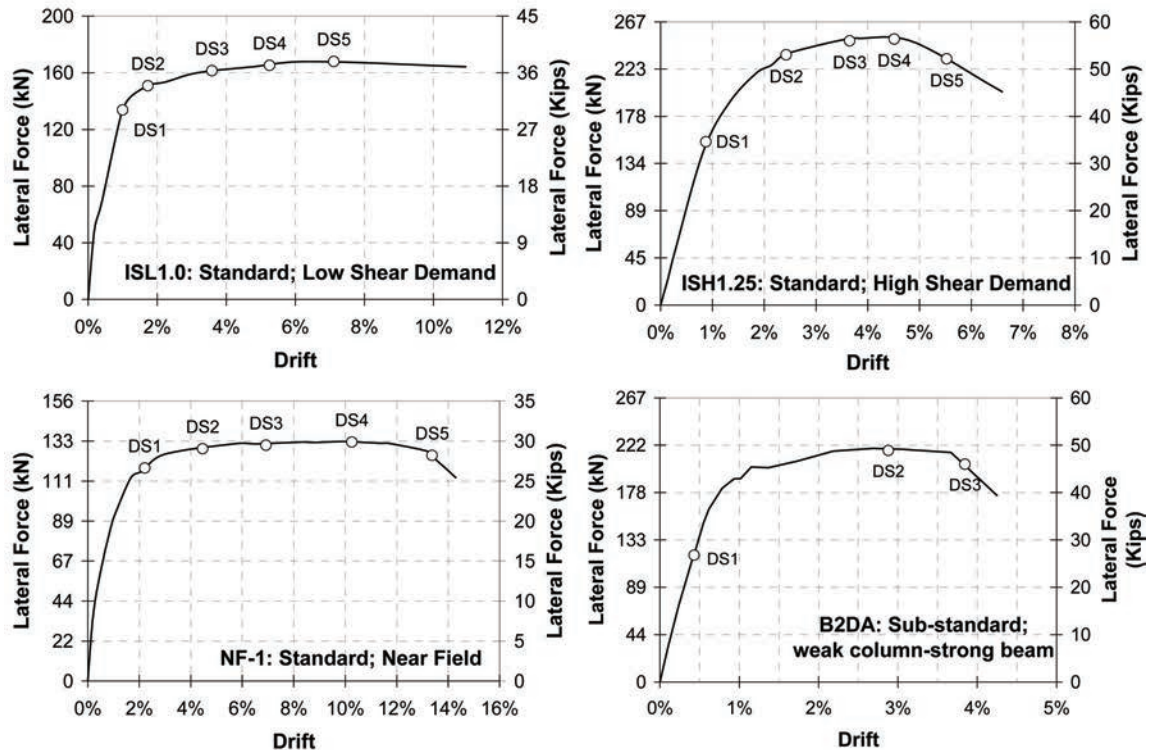


Fig. 8—Sample pushover curves and damage states for standard and sub-standard columns.

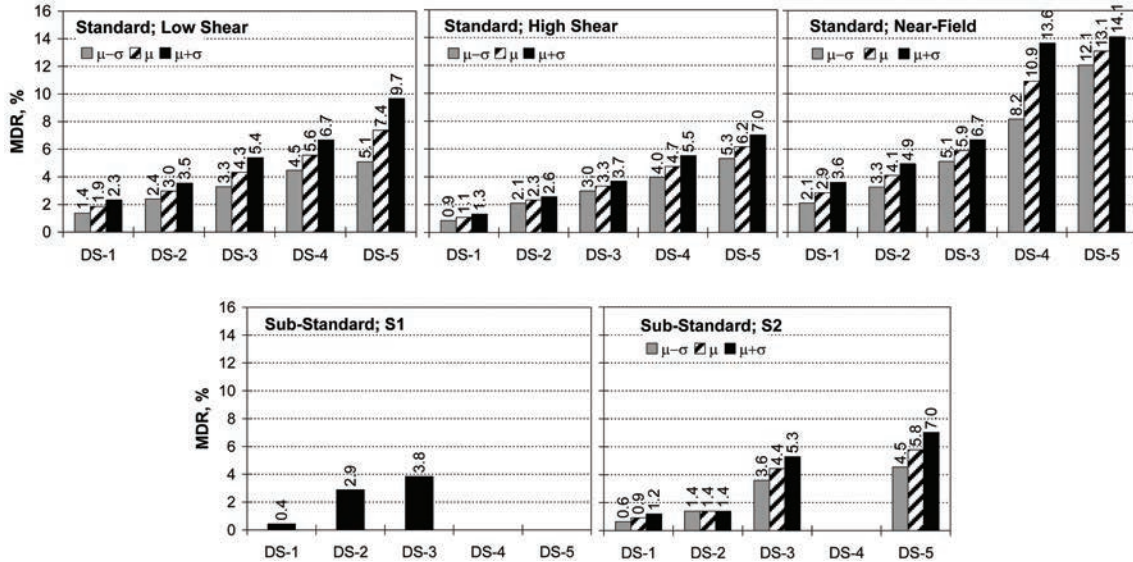


Fig. 9—Maximum drift ratio (MDR) versus damage state.

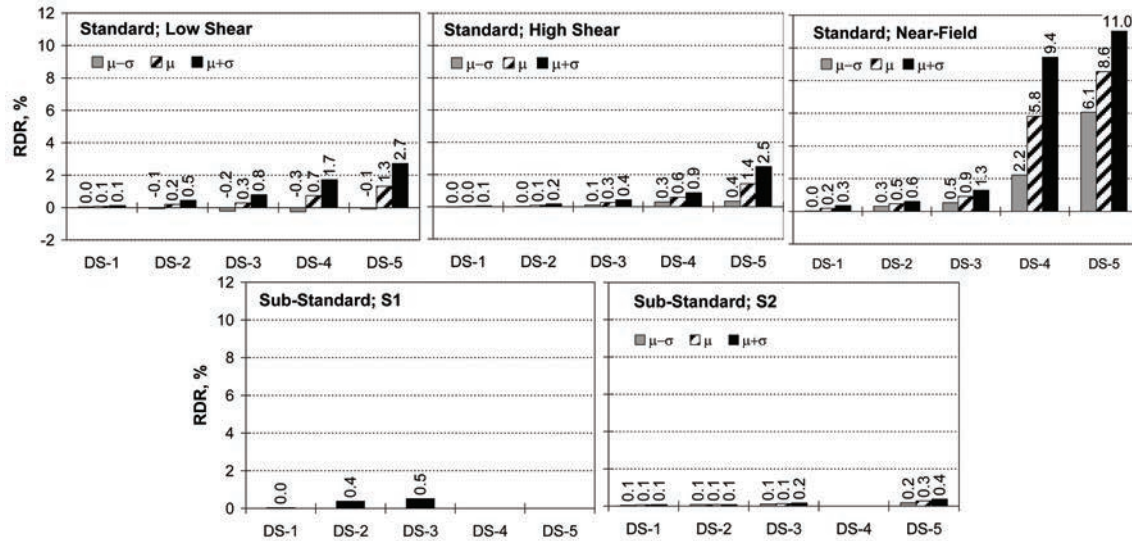


Fig. 10—Residual drift versus damage state.

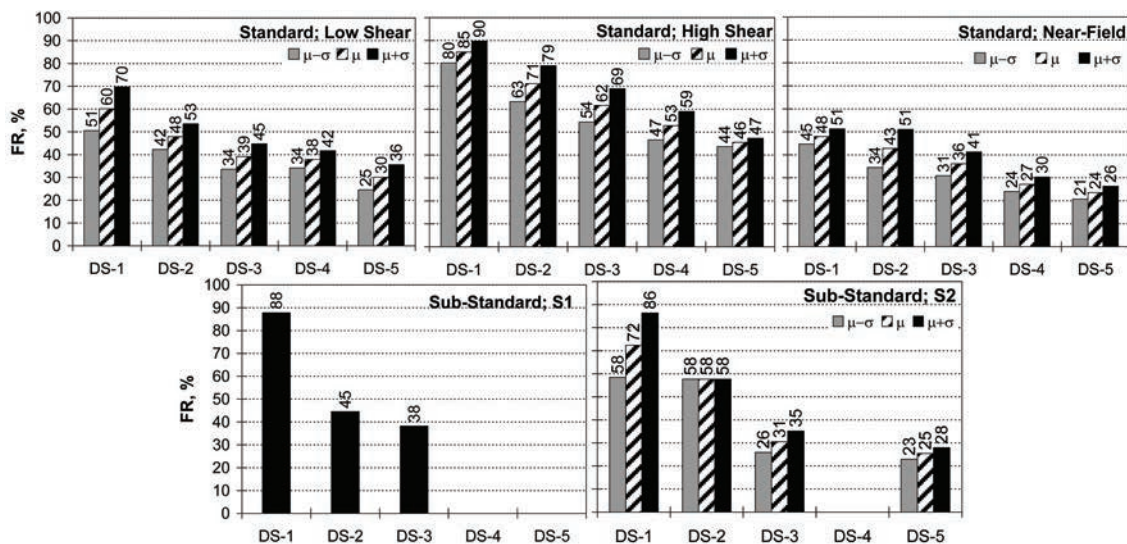


Fig. 11—Frequency ratio versus damage state.

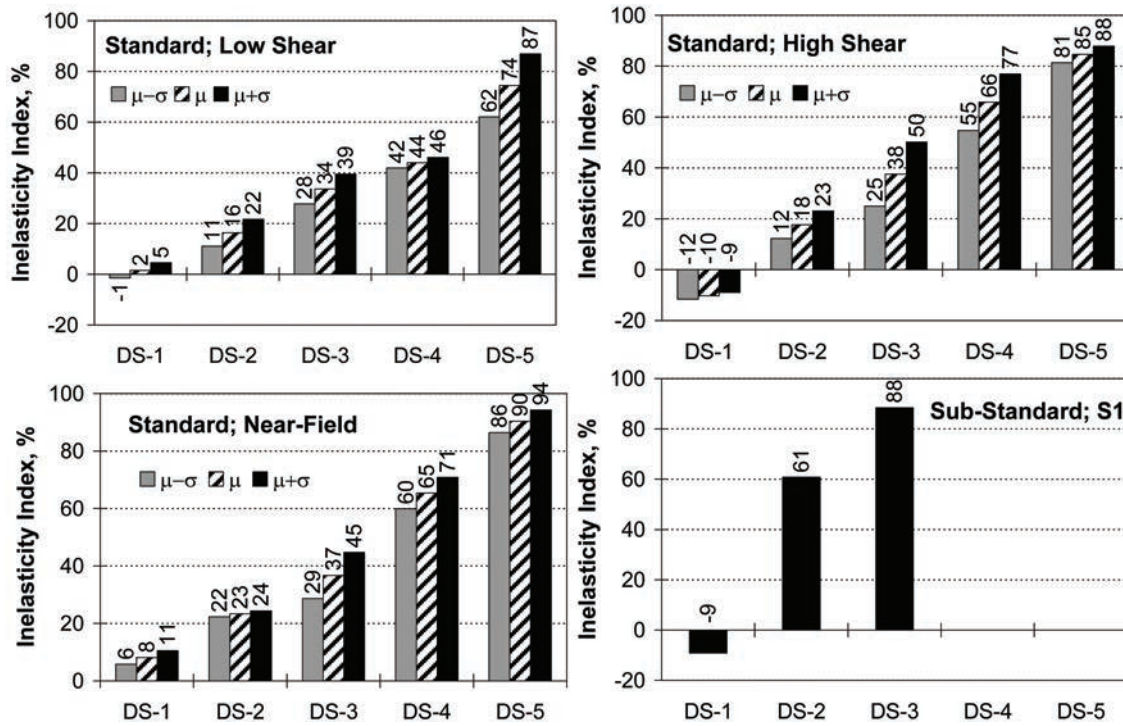


Fig. 12—Inelastic index versus damage state.

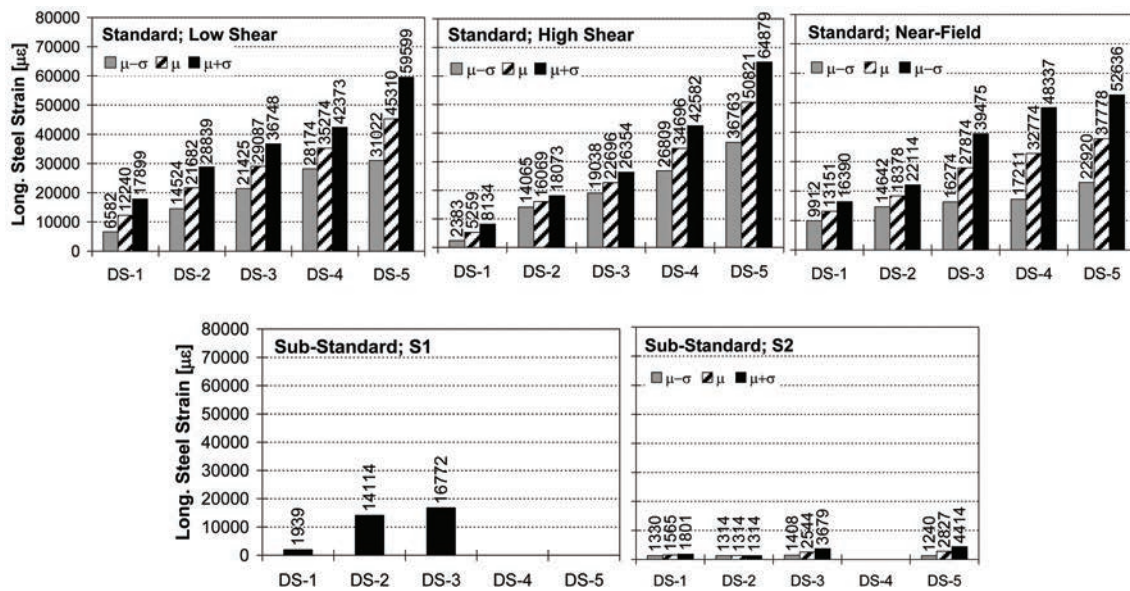


Fig. 13—Longitudinal steel strain versus damage state.

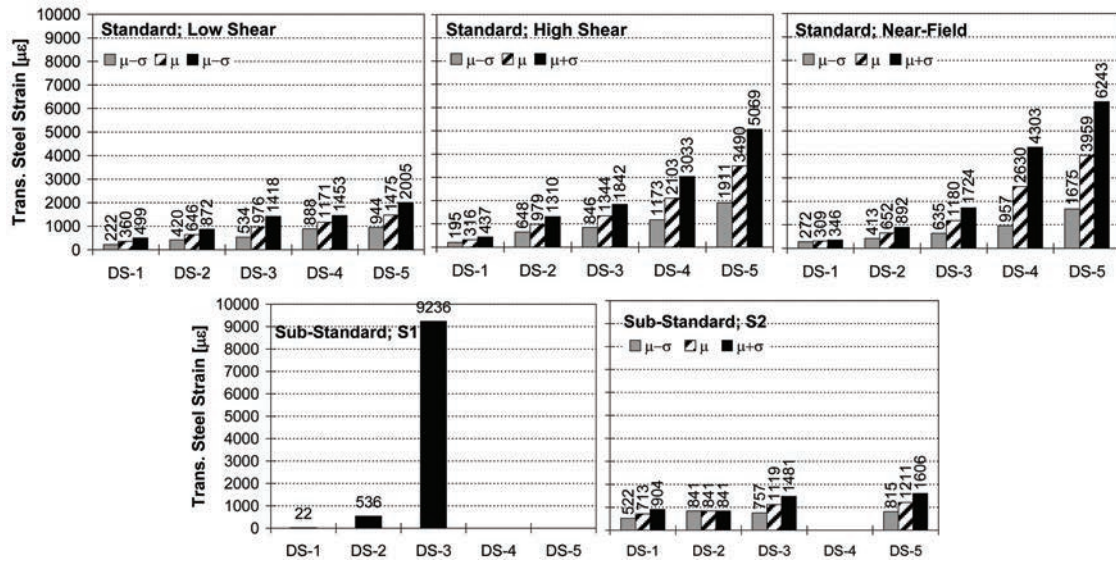


Fig. 14—Transverse steel strain versus damage state.

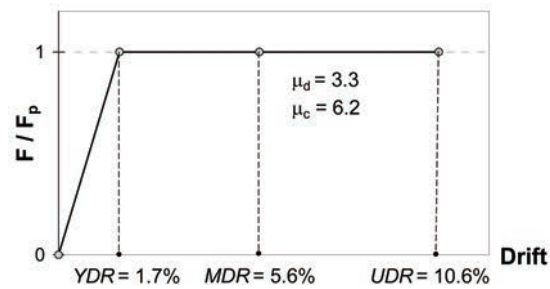


Fig. 15—Estimated pushover curve using response parameters.