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Seismic vulnerability assessment criteria for RC ordinary highway bridges in Turkey

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Abstract. One of the most important and challenging steps in seismic vulnerability and performance assessment of highway bridges is the determination of the bridge component damage parameters and their corresponding limit states. These parameters are very essential for defining bridge damage state as well as determining the performance of highway bridges under a seismic event. Therefore, realistic damage limit states are required in the development of reliable fragility curves, which are employed in the seismic risk assessment packages for mitigation purposes. In this article, qualitative damage assessment criteria for ordinary highway bridges are taken into account considering the critical bridge components in terms of proper engineering demand parameters (EDPs). Seismic damage of bridges is strongly related to the deformation of bridge components as well as member internal forces imposed due to seismic actions. A simple approach is proposed for determining the acceptance criteria and damage limit states for use in seismic performance and vulnerability assessment of ordinary highway bridges in Turkey constructed after the 1990s. Physical damage of bridge components is represented by three damage limit states: serviceability, damage control, and collapse prevention. Inelastic deformation and shear force demand of the bent components (column and cap beam), and superstructure displacement are the most common causes for the seismic damage of the highway bridges. Each damage limit state is quantified with respect to the EDPs: i.e. curvature and shear force demand of RC bent components and superstructure relative displacement.

Keywords: vulnerability; seismic damage; bridges; limit states; demand parameters

1. Introduction

Determination of bridge damage parameters and their corresponding limit states is one of the significant steps for the seismic performance assessment of highway bridges. Bridge damage state definitions are one of the main sources of uncertainty engaged in the performance assessment due to the subjectivity involved in defining the limit states. A limit state can be defined as the ultimate point beyond which the bridge structure can no longer satisfy the specified performance level. Each damage limit state corresponds to a functional and operational interpretation. Therefore, realistic damage limit state definitions are necessary for highway bridges to make a reasonable estimate of their seismic performance level. Structural damage is mostly related to the deformation of the bridge

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system and its components. That is why most of the available bridge damage limit state definitions are specified in terms of deformations for local and global response parameters, which are expressed by proper EDPs. Local EDPs are utilized for certain structural components whereas global ones are considered for the estimation of overall structural response. The selected EDPs should have a good correlation with the actual seismic damage of bridges. Because, seismic damage of the bridge components needs to be quantified with the selected EDP, which is used in the calculation of both capacity and seismic demand of the bridge components.

The bridge damage due to seismic actions should be represented with a sufficient number of damage limit states. Although qualitative damage limit state definitions for bridges are available in different codes and studies, widely accepted quantitative damage limit state definitions are not readily available for highway bridges in Turkey. HAZUS (FEMA 2003) defined five qualitative damage limit states, which are none, slight/minor, moderate, extensive and complete damage limit states as explained in Table 1. These limit states were commonly employed in previous studies for defining the corresponding quantitative damage limit states in terms of various EDPs. Hwang *et al.* (2001) used displacement ductility ratios; Liao and Loh (2004) considered ductility and displacement limits; Basoz and Mander (1999) used drift and displacement limits; Banerjee and Shinozuka (2007) considered rotations at plastic hinge regions of bridge columns; Nielson and DesRoches (2007), Padgett and DesRoches (2009) employed column curvature ductility, bearing displacement for fixed and expansion bearings and abutment deformations in active and passive directions. Priestley *et al.* (1996), Kowalsky (2000) utilized different number of damage limit states other than HAZUS (FEMA 2003). Kowalsky (2000) employed material strain limits for defining damage states. Although several quantitative damage limit state definitions are available in literature for bridges, these damage limit state definitions are specific to the interested bridges. In order to perform a reliable seismic vulnerability assessment for Turkish highway bridges, specified seismic damage should be in accordance with the damage likely to occur in highway bridges in Turkey. The main objective of this study is to develop a methodology for defining the seismic damage limit

Table 1 Definitions of damage states by HAZUS (FEMA 2003)

Damage States	Definitions
None (ds_1)	No bridge damage
Slight/Minor (ds_2)	Minor cracking and spalling to the abutment, cracks in shear keys at abutments, minor spalling and cracks at hinges, minor spalling at the column (damage requires no more than cosmetic repair) or minor cracking to the deck
Moderate (ds_3)	Any column experiencing moderate (shear cracks) cracking and spalling (column structurally still sound), moderate movement of the abutment (<2"), extensive cracking and spalling of shear keys, any connection having cracked shear keys or bent bolts, keeper bar failure without unseating, rocker bearing failure or moderate settlement of the approach.
Extensive (ds_4)	Any column degrading without collapse - shear failure - (column structurally unsafe), significant residual movement at connections, or major settlement approach, vertical offset of the abutment, differential settlement at connections, shear key failure at abutments.
Complete (ds_5)	Any column collapsing and connection losing all bearing support, which may lead to imminent deck collapse, tilting of substructure due to foundation failure.

states of the highway bridges in Turkey constructed after the 1990s. These damage limit states are utilized in the seismic performance assessment calculations as well as the risk assessment of the corresponding bridges. A simple approach is introduced to determine the overall bridge damage by evaluating the damage state of each specified bridge component.

The approach developed here focuses on ordinary highway bridges in Turkey. Therefore, typical properties of these bridges are presented first. Based on these features, pertinent damage limit states corresponding to the bridge components that influence the seismic performance of the bridge are discussed. These limit states take into account the most relevant response parameters of the components so the corresponding EDPs were employed.

2. Properties of highway bridges in Turkey

The general understanding of ordinary highway bridges in Turkey constructed after the 1990s in terms of their structural attributes as well as their seismic behavior is essential for defining their damage limit states and assessment of their performance level. A group of 52 highway bridges representing the general characteristics of the ordinary highway bridges constructed after the 1990s in different parts of Turkey are selected (Avşar 2009). Although the selected bridges do not cover all bridge types in the inventory, their structural properties reflect general characteristics of most of the highway bridges in Turkey. C40 concrete class (the characteristic strength is 40 MPa) is used for the prestressed concrete girders and C25 is used for the rest of the reinforced concrete bridge components. S420 is used for the reinforcing bars. Single span bridges are not included in the inventory. Schematic drawings of a sample bridge and its components that constitute the general attributes of the bridges are shown in Fig. 1.

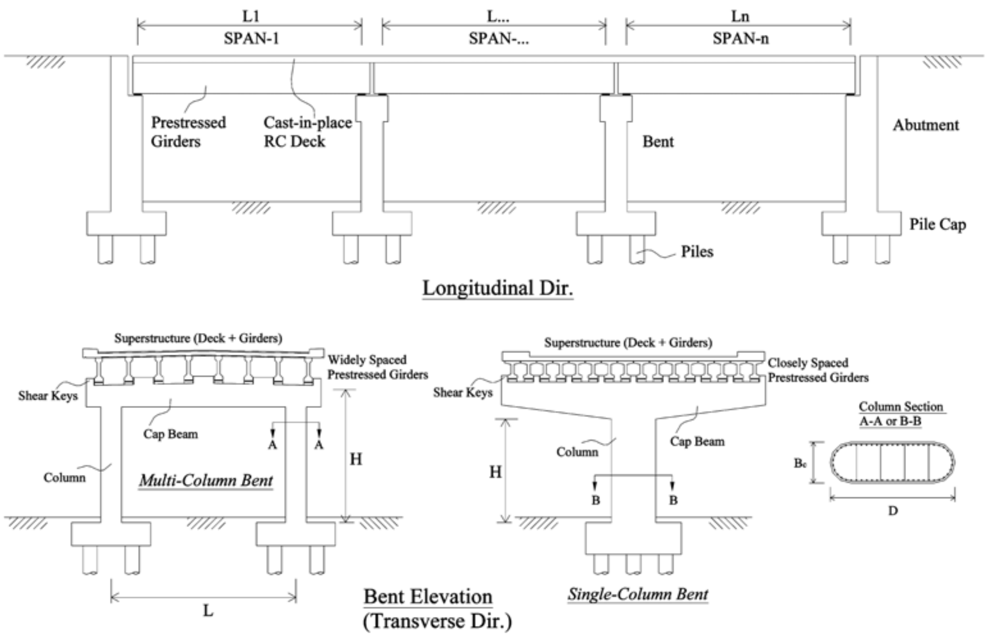


Fig. 1 General characteristics of the ordinary highway bridges in Turkey

The investigated group of bridges can be specified as *Ordinary Standard Bridges* according to California Department of Transportation (CALTRANS 2006). The superstructure is supported by elastomeric bearings, which is placed on the abutments and bent cap beams. There is no connecting device between the elastomeric bearings and the concrete components of superstructure or substructure. Friction between the bearings and the concrete surface is the only resisting force that holds the elastomeric bearing at its place. Several thin metal sheets are provided in the elastomeric bearing to prevent bulging due to the axial loads. In the design stage of elastomeric bearings, they are considered as sacrificial components. Therefore, they are designed such that they can be replaced after failure or occurrence of walk-out problem after an earthquake. Reinforced concrete is the primary structural material used in the ordinary highway bridges constructed after the 1990s in Turkey. Superstructure girders are the only components which are constructed with prestressed concrete other than reinforced concrete. All the bridges in the inventory data are multiple simple-span composite structures that utilize prestressed concrete girders and a continuous cast-in-place reinforced concrete deck. They have a seat type abutment system and multiple- or single-column bents. Column cross

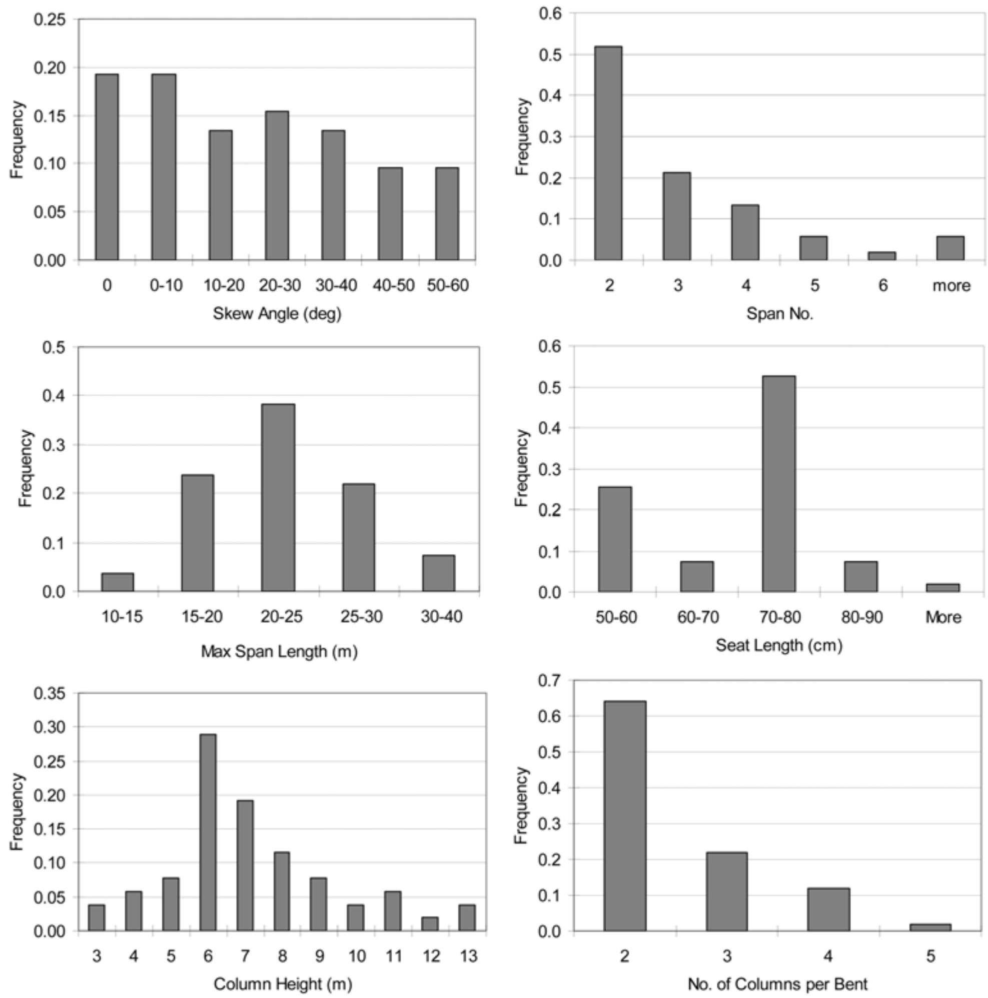


Fig. 2 Statistical distributions of several structural attributes of highway bridges

section is mostly composed of a rectangle section with two half circles at both ends. Longitudinal reinforcement ratio of the columns is around 1%. Shear keys are provided at the abutment and bent to constrain the transverse movement of the superstructure. Therefore pounding can take place between the superstructure and shear keys in the transverse direction. Also, in the longitudinal direction pounding can occur between the superstructure and abutment backwall as well. Spring elements can be employed for the analytical modeling of pounding phenomenon at the expansion joints. These springs monitor the relative displacement of the nodes at the joints connecting the bridge components at which the pounding can take place. When the calculated relative distance in the gap closing direction is greater or equal to the specified gap distance; pounding takes place, and stiffness and capacity of the corresponding bridge components become effective (Wang 2007, Avşar 2009).

The statistical distributions of some of the important structural attributes affecting the seismic response of highway bridges are determined considering the representative 52 highway bridges in the inventory data. Histograms of the investigated structural attributes are obtained and presented in Fig. 2. Some of the investigated structural attributes are; skew angle, span number, maximum span length, seat length, column height and number of columns per bent for multiple-column bent bridges.

3. Definition of damage limit states

According to the previous studies, damage states defined by HAZUS are widely accepted among the researchers to be used in the development of fragility curves. Since five damage states were considered, four damage limits should be specified quantitatively to be able to perform seismic vulnerability assessment of highway bridges. Definitions of the first and the last damage limit states, which correspond to the slight and complete damage limit states, have commonly accepted physical meanings. Slight damage limit generally corresponds to the system's yield point beyond which the structure experiences inelastic deformations and residual displacements. Complete damage limit state can be specified as the ultimate capacity of the structure, beyond which the structural system is no longer stable and total or partial collapse may take place. On the other hand, intermediate damage limits that are moderate and extensive damage limit states correspond to bridge damage, which is not commonly defined among the researchers. Defining quantitative measures for the intermediate damage limit states is a subjective task and challenge lies in being able to define these damage limits such that they represent the true physical damage of the bridges. Because of the uncertainties involved in quantifying the intermediate damage limit states, instead of dealing with two intermediate damage limits of moderate and extensive damage limit states, it is more reasonable to consider only one intermediate damage limit state in the calculations. This intermediate limit state represents the extreme level of seismic response after which it would not be economically and technically feasible to repair the bridge according to Federal Highway Administration (FHWA 1995). Consequently, a total of three damage limits need to be quantified with proper EDPs in the seismic vulnerability of highway bridges. Damage limits and damage states employed in this study are in accordance with Turkish Earthquake Code (TEC 2007). These three damage limit states are termed as "serviceability" (LS-1), "damage control" (LS-2) and "collapse prevention" (LS-3). Slight/No, moderate, significant, and collapse states are the four corresponding damage states that the bridges can experience under the effect of an earthquake ground motion. The schematic representation of the three damage limits and their corresponding damage states are shown on a force-deformation curve in Fig. 3.

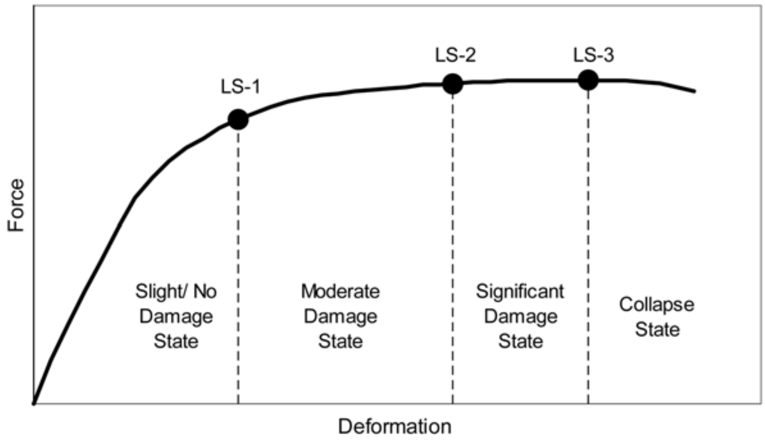


Fig. 3 Damage states and damage limits on a force-deformation curve

4. Engineering demand parameters

Bridge damage criteria are required in order to perform seismic vulnerability assessment. These criteria are used to identify the damage state of the bridge whether it attains the corresponding damage limit state or not. Various quantitative descriptions of bridge damage criteria are employed in the calculations. Quantification of damage limit states can be made through consideration of either local or global measures of bridge damage. Selection of damage measure to quantify the damage limit states has a significant influence on the reliability of the vulnerability assessment results. Considering a single global damage measure can lead to underestimation of local bridge failures. Moreover, considering local damage of individual bridge components has the advantage of investigating the effect of respective bridge component on the overall seismic response of bridge. Several EDPs of various bridge components can be considered for bridge damage assessment.

As a requirement of capacity design principles, no seismic damage is expected in the superstructure (CALTRANS 2006). Because it remains in the elastic range with the help of isolation units placed between the superstructure and substructure components. Past earthquakes revealed that the nonlinear elements of bent columns and cap beams are the most susceptible bridge components to seismic damage. Therefore, curvature and shear capacity of the columns and cap beams should be taken into account in the bridge damage assessment. Also deck unseating is one of the most common seismic damage that bridges can experience during damaging earthquakes. Considering these bridge components, which are likely to experience seismic damage, following EDPs are taken into account to quantify each damage limit state.

- Curvature demands of the RC column and cap beam.
- Shear demand on the RC column and cap beam.
- Superstructure relative displacement.

4.1 Damage limit states for RC column and cap beam curvature

Column and cap beam curvature is employed as an EDP for the quantification of damage limit states of column and cap beam, which are expected to respond in the inelastic range. For this purpose, section analyses are performed to determine the moment-curvature relationship of the column and cap beam RC sections. Material models for the reinforcement steel (bilinear model), unconfined and confined concrete (Kent and Park 1971, Scott *et al*. 1982) are utilized in the section analyses. Nonlinear moment-curvature (M-K) curve is obtained first and then converted to a bilinear representation by the following procedure. Yielding point of longitudinal reinforcement is specified to determine the linear elastic portion and initial slope of the bilinear M-K curve. Ultimate curvature point of the M-K curve is specified when the reinforcement steel or confined concrete extreme fiber has reached its ultimate strain value or when the moment capacity at the M-K curve has decreased to 80 percent of its maximum attained moment capacity (Priestley *et al*. 1996). After obtaining the initial slope and the ultimate point of the M-K curve, bilinear M-K curve is determined by applying trial and error calculations in a way that the area under the nonlinear and bilinear curve is equal to each other, which is termed as the equal energy rule.

Section yield point determined from bilinear M-K curve corresponds to the serviceability limit state for the nonlinear bridge components. A significant change in stiffness of RC members occurs at the onset of section yield (Priestley *et al*. 2007). At this damage limit state, crack widths should be sufficiently small and the member functionality is not impaired. The damage-control limit state is defined as the point at which the concrete cover spalling occurs. According to Priestley *et al*. (1996), the onset of spalling of cover concrete is considered to be the significant damage state at which the negative stiffness as well as sudden strength loss may take place. Beyond this limit state, bridge may experience significant damage, which can be characterized with several damage indicators such as the fracture of transverse reinforcement, buckling of longitudinal reinforcement and the need for replacement of core concrete in the plastic hinge region. Damage-control limit state is quantified with a curvature limit that is calculated when the extreme fiber of the unconfined concrete attains a compressive strain of 0.003, which is assumed to be the strain limit for the spalling of concrete cover.

Defining the collapse prevention limit state by the ultimate curvature determined from the section M-K curve does not represent the true damage state of the bridge columns. The reliability of ultimate curvature is directly influenced by the material models, which involve several assumptions and approximations, where stiffness degradation and stiffness reduction are not considered in moment-curvature analyses. Moreover, during the section analyses, perfect bond between concrete and reinforcement is assumed and bond slip is not taken into account in the calculation of ultimate curvature. Therefore, it is more realistic to use the results of experimental data for determining the ultimate curvature that the nonlinear bridge component can experience without occurrence of complete failure. Erduran and Yakut (2004) proposed an empirical equation for the column displacement ductility capacity based on the results of previous column experiments. The given equation for the column displacement ductility has the parameters of ρ_s and N/N_o, which are the amount of transverse reinforcement and axial load level of the columns, respectively. Column displacement ductility of a highway bridge is determined using Eq. (1). Although axial load level of the columns change during the dynamic analyses of the bridges, an average value is assumed for the column axial load level, which is calculated from gravity analyses.

$$\mu_u = 0.6\ln\left[\left(\frac{\rho_s}{N/N_o}\right)^2\right] + 7.5 \tag{1}$$

Curvature ductility (μ_Φ) of the columns can be calculated using the column displacement ductility

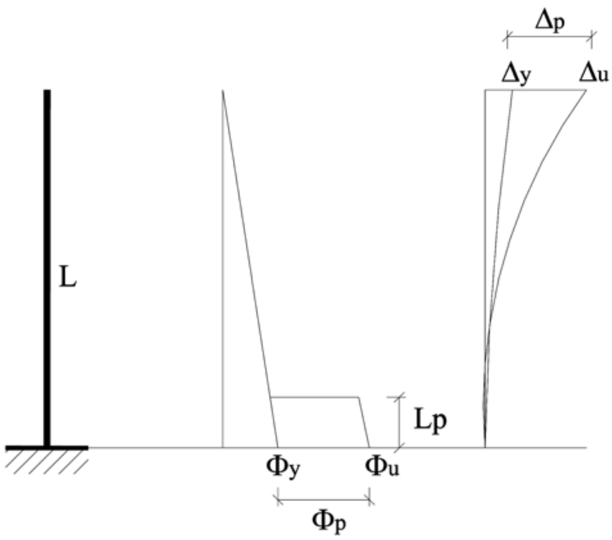


Fig. 4 Distribution of a cantilever column curvature and displacement

(μ_Δ), the column length, and the plastic hinge length (L_p) of column using the formulations derived through Eqs. (2) to (8). The parameters employed in Eqs. (2), (3) and (4) are schematically shown in Fig. 4 for a cantilever column. Plastic hinge length formulation proposed by Priestley *et al.* (1996) is employed in the calculations as shown in Eq. (9). Where, d_{bl} is the diameter of the longitudinal reinforcement, f_{ye} is the design yield strength for longitudinal reinforcement, and L is the distance from the critical section of the plastic hinge to the point of contraflexure. L is taken as the total column height for the single column bent. Since the column has a cantilever structural system, development of plastic hinge takes place only at the bottom of the column. This is also valid for the columns in the longitudinal direction of the multi column bents. However, in the transverse direction of multi column bents, cap beams and columns form a frame system. In this system, plastic hinges can develop both at the bottom and top of the column members. Due to the flexibility of the cap beams, plastic hinges will be developed at the bottom of the column first and point of contraflexure occurs closer to the column top joint. For simplicity, point of contraflexure is assumed to occur at the mid height of the column in the transverse direction and hence distance L to be used in Eq. (9) is calculated as half of the column clear height.

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} = \frac{\Delta_p + \Delta_y}{\Delta_y} = 1 + \frac{\Delta_p}{\Delta_y} \tag{2}$$

$$\Delta_y = \frac{\Phi_y \cdot L^2}{3} \tag{3}$$

$$\Delta_p = (\Phi_u - \Phi_y) \cdot L_p \cdot L - (0.5 \cdot L_p) \tag{4}$$

$$\mu_\Delta = 1 + \frac{(\Phi_u - \Phi_y) \cdot L_p \cdot (L - 0.5 \cdot L_p)}{\frac{\Phi_y \cdot L^2}{3}} \tag{5}$$

$$\mu_{\Delta}-1 = \left(\frac{\Phi_u-\Phi_y}{\Phi_y}\right)\cdot\frac{3L_p}{L}\cdot\left(1-0.5\cdot\frac{L_p}{L}\right) \quad (6)$$

$$\mu_{\Delta}-1 = (\mu_{\Phi}-1)\cdot\frac{3L_p}{L}\cdot\left(1-0.5\cdot\frac{L_p}{L}\right) \quad (7)$$

$$\mu_{\Phi} = 1+\frac{\mu_{\Delta}-1}{\frac{3L_p}{L}\cdot\left(1-0.5\cdot\frac{L_p}{L}\right)} \quad (8)$$

$$L_p = 0.08L+0.022f_{ye}d_{bl}\geq 0.044f_{ye}d_{bl} \quad (f_{ye}[\mathrm{MPa}]) \quad (9)$$

Due to the lack of experimental data on the calculation of ultimate curvature ductility or displacement ductility of the cap beams, the ultimate curvature obtained from M-K is considered as the collapse prevention limit state for the cap beams.

Fig. 5 shows the schematic representation of the three damage limits and their corresponding damage states on a moment-curvature diagram. In some cases, curvatures calculated for damage-control and collapse prevention limit can be very close to each other especially for the single column bent columns in longitudinal direction. This implies very narrow interval for the significant damage state. In this case, ultimate curvature calculated using empirical equations for the collapse prevention limit state is modified by considering the curvature specified for the damage-control limit state, which is a commonly accepted damage limit for the concrete cover spalling. According to Eurocode 8 - Part3 (2005), the chord rotation capacity corresponding to significant damage may be assumed to be 3/4 of the ultimate chord rotation. In a similar way, ultimate curvature capacity for the collapse prevention limit state is updated with a factor of 4/3 of the calculated curvature for the damage-control limit state. Therefore, curvature capacity for the collapse prevention limit state is obtained by taking the maximum value of the curvature ductility calculated with Eq. (8) employing the empirical formulation for displacement ductility and the curvature corresponding to the 4/3 of the limiting curvature for damage-control limit state, which is calculated for the compressive strain of 0.003 at the unconfined concrete.

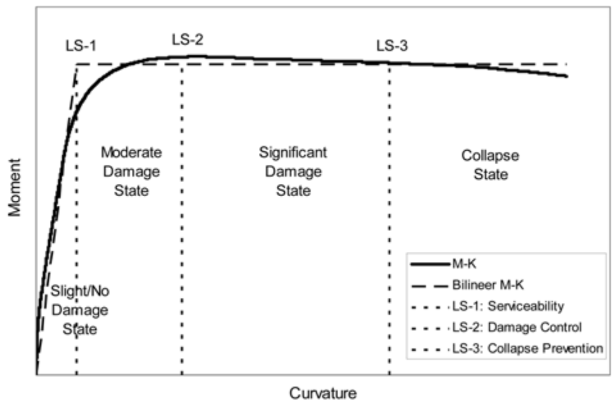


Fig. 5 Damage limits defined for column and cap beam curvature

4.2 Damage limit states for RC column and cap beam shear

Column and cap beam shear capacity is considered to be another EDP for the quantification of damage limit states. Shear failure is a brittle type of failure mode resulting in a sudden collapse of RC members and this type of failure mode has been observed in the past EQs (Kawashima 2002). There is no distinction between the damage limit states for brittle type of failure mode and hence an identical capacity level is considered for all damage limit states. Since total collapse occurs when the shear capacity is exceeded by the seismic shear demand, only collapse prevention limit state is defined for the shear capacity of columns and cap beams in both principal axes. Shear strength of RC members is calculated using the equation proposed by Priestley *et al*. (1996), which is presented in Eq. (10).

$$V_{total} = V_c + V_s + V_p \tag{10}$$

where V_c is the shear carried by concrete shear resisting mechanism, V_s is the shear carried by transverse reinforcement shear resisting mechanism, and V_p is the shear strength provided by axial force in columns.

The shear strength provided by concrete, V_c is calculated as

$$V_c = k\sqrt{f_c}A_e \tag{11}$$

where A_e is the effective shear area of cross section that is equal to $0.8A_{gross}$. f_c is compressive strength of unconfined concrete. "k" is expressed as a factor defining the relationship between ductility and strength of concrete shear resisting mechanism (Priestley *et al*. 1996). For the initial shear strength of the RC members, a constant value of 0.29 MPa can be assumed in the calculations.

Shear strength contribution of transverse reinforcement for rectangular RC sections is determined with Eq. (12)

$$V_s = \frac{A_{sw}f_yD'}{s}\cot\theta \tag{12}$$

where A_{sw} is the area of transverse reinforcement in the direction of applied shear force, f_y is the yield strength of transverse reinforcement, D' is the core dimension in the direction of applied shear force, "s" is the spacing of transverse reinforcement, is the angle of the critical inclined flexure shear cracking to the member axis, which is taken as 30° (Priestley *et al*. 1996).

Shear strength contribution provided by axial force in columns is calculated by Eq. (13), where P is the axial force and is the angle formed between the column axis and the strut from the point of load application to the center of the flexural compression zone at the column plastic hinge critical section.

$$V_p = P\tan\alpha \tag{13}$$

Shear capacity of the RC sections is compared with the shear demand obtained from nonlinear response history analyses to decide whether the member attains the collapse prevention limit state or not.

4.3 Damage limit states for superstructure displacement

Superstructure displacement in both orthogonal axes of the bridge is considered to be the last EDP for the quantification of damage limit states. Due to the movement of the superstructure, different levels of seismic damage can occur. FHWA (1995) described qualitative damage states due to the displacement of bearings and superstructure. It is mentioned that settlement and vertical misalignment of a span due to an overturned bearing may be a minor problem, resulting in only a temporary loss of access which can be restored. Collapse may occur due to loss of support resulting from large relative transverse or longitudinal movement at the support in vulnerable structures. Moreover, it is stated that "walk out" phenomenon may occur under severe shaking due to inadequate fastening of the bearings.

In the Turkish highway bridges constructed after the 1990s, there is no fastener or connecting device between the elastomeric bearings and the superstructure and substructure components. Therefore, friction force developed between the concrete surfaces and the bearings is the only resisting force that holds the elastomeric bearing at its place. When the seismic demand for the superstructure displacement exceeds the friction force, which depends on the axial load level of the bearings and the friction coefficient, bearings will be no longer stable and superstructure starts to make permanent displacement leading to minor problems at the bridge. Walk-out of the elastomer bearings has been observed during Kocaeli earthquake causing minor/no damage (Bruneau *et al.* 1999). Displacement capacity of the bearings, beyond which the friction force is exceeded by the seismic forces, is accepted as the ultimate bearing displacement for defining the serviceability limit state.

The displacement capacity of the elastomeric bearings for serviceability limit state is calculated using Eq. (14), which is a function of bearing lateral stiffness and friction force developed between elastomeric pad and concrete surface. Friction force is determined by Eq. (15) for each elastomeric bearing at the bridge based on the level of axial load on the elastomeric bearings and the dynamic coefficient of friction between the concrete surface and bearings, which is specified as $\eta = 0.40$ by CALTRANS (2006). The lateral stiffness of the elastomeric bearing is calculated using Eq. (16), in which required parameters are shown for a typical elastomer bearing in Fig. 6. Where G, A and h_{rt} are the shear modulus, area and the total rubber height of the elastomeric bearings, respectively. The shear modulus of elastomeric bearings is specified according to their hardness as per AASHTO (2010). In general, nominal hardness of the elastomeric bearings is 60 on the Shore A scale for the

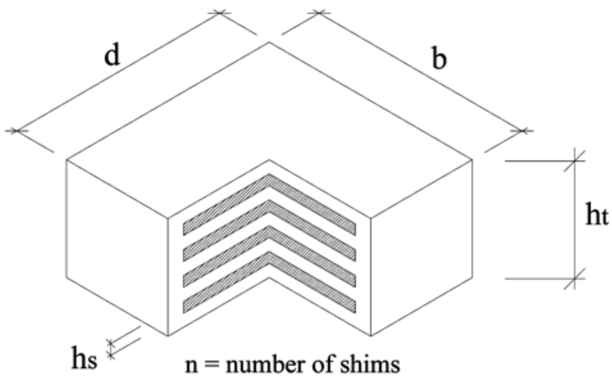


Fig. 6 Typical elastomeric bearing of a highway bridge

inspected highway bridges. The shear modulus, G of the elastomeric bearings is calculated as 1.1 MPa, which is the average value of the recommended range by AASHTO (2010).

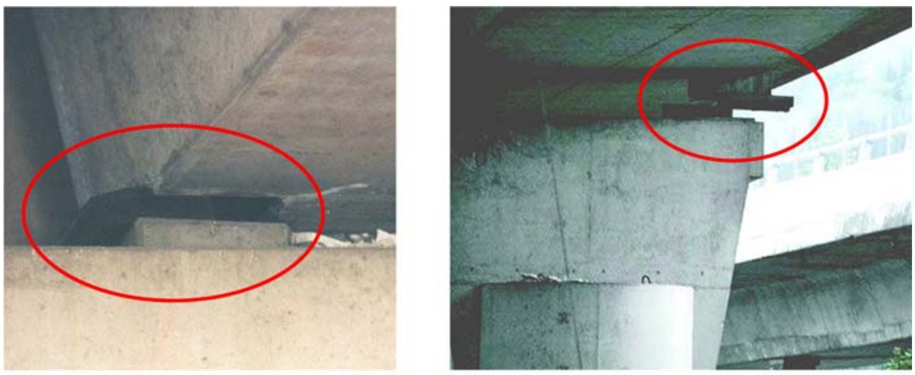
$$\delta_{friction} = \frac{F_{friction}}{K_{bearing}} \tag{14}$$

$$F_{friction} = N_{bearing} \times \eta \tag{15}$$

$$K_{bearing} = \frac{G \times A}{h_{rt}};\ A = d \times b;\ h_{rt} = h_t - n \times h_s \tag{16}$$

For ordinary highway bridges, bearings are considered sacrificial components and they need to be inspected for damage and replaced after a damaging earthquake. Especially, due to lack of connecting devices between the bearing and concrete surface, "walk-out" phenomenon can be observed after severe earthquakes when the friction force is exceeded. Examples of dislodgment of bearing systems at Sakarya Viaduct during 1999 Kocaeli Earthquake (Pamuk *et al*. 2005) and at A24 Expressway during L'aquile Earthquake (Aydan *et al*. 2009) are presented in Fig. 7(a) and Fig. 7(b), respectively.

Concrete pedestals are constructed over the cap beams and abutments with varying height to position the vertical alignment of the superstructure girders as shown in Fig. 8. Under extreme seismic events, superstructure girders may experience large horizontal displacements and fall over the concrete pedestal and rests on the cap beam or abutment directly. This could cause excessive damage on the asphalt disturbing the traffic flow and affecting the functionality of the bridge. Damage control limit state is specified for the displacement when the superstructure falls over concrete pedestal on the cap beam or abutment as depicted by LS-2 in Fig. 8. Finally, when the superstructure displacement exceeds the available seat length provided by the cap beam and abutment, it will fall over the substructure and total collapse occurs. Typical seat length distribution of the investigated representative 52 highway bridges in Turkey is given in Fig. 2. Therefore, this damage limit state is defined as the collapse prevention limit state (LS-3 in Fig. 8), beyond which the bridge is no longer stable. Depending on the position of the concrete pedestal on the cap beams and abutments as well as the available seat length, displacement limits for the limit states of LS-2 and LS-3 can be very variable among the existing highway bridges in Turkey. Due to insufficient seat length of existing



(a) Sakarya Viaduct (Kocaeli EQ) (b) A24 Expressway (L'aquila EQ)

Fig. 7 Dislodgment of bearing systems during earthquakes

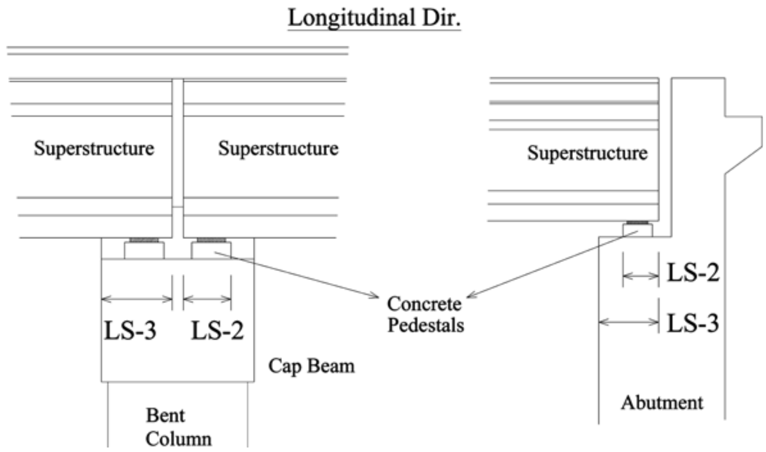


Fig. 8 Superstructure seat length at the bent and abutment

highway bridges, examples of deck fall off have been observed after severe earthquakes like Kocaeli (1999), Chi-Chi (1999), Wenchuan (2008), (Kawashima 2002, Lin *et al*. 2010).

5. Seismic performance assessment of highway bridges

Seismic damage state of the bridge components as well as the overall bridge system under the effect of an earthquake ground motion can be determined by the following procedure:

1. Threshold values for the EDPs of the investigated bridge are determined for each damage limit state by employing the previously explained methods.
2. Analytical model of the bridge is developed to carry out nonlinear response history analysis (NRHA).
3. Maximum seismic response of bridge components are obtained from the NRHA results for the selected ground motion.
4. Seismic demand of each bridge component is compared with the calculated threshold values of the corresponding EDPs to determine the performance level of the component.
5. Performance level of the bridge system is evaluated for the given ground motion record by considering the most critical bridge component. The damage state of the bridge is considered to be the damage state of the critical bridge component that has the most severe of all three damage states.

Seismic damage state of a bridge as a whole cannot be determined directly by identifying the damage state of its components. Since there does not exist any specific method that relates the overall bridge damage with the damage state of its components, an assumption is made for identifying the bridge damage state. If any of the bridge components attains or exceeds a damage limit state, bridge system as a whole is assumed to be in the same damage state regardless of the damage states of the rest of the bridge components. In this method, a series system is assumed for the bridge. Therefore, a conservative approach is assumed and overall bridge damage is determined by considering the weakest component of the bridge without considering the correlation among the damage state of the bridge components.

6. Application of the proposed procedure

A group of seven bridge samples are selected to illustrate the seismic performance assessment procedure under the bidirectional effect of Duzce, Bingol and Kobe Earthquakes. In selecting the sample bridges, Fig. 2 is considered while choosing the most common bridge samples in terms of its structural attributes. The general properties of the selected bridge samples and the important features of the earthquake records are given in Table 2 and Table 3, respectively.

Seismic demand values for each EDP of the selected bridges have been determined through NRHA with the 3D analytical model developed in the OpenSees (2009) platform. Comprehensive analytical models for each of the structural component of the sample bridges have been developed as shown in Fig. 9 schematically. The details of the analytical model of the bridges can be found in Avşar *et al*. (2011), Avşar (2009). Superstructure was modeled using standard prismatic elastic beam elements (Olmos and Roesset 2010). Substructure components of column and cap beam members were modeled with nonlinear elements. Nonlinear modeling of column and cap beam was made using fiber-based nonlinear elements to represent the distributed plasticity along the member length at certain control points. For reinforcement steel, confined and unconfined concrete fibers, relevant material models were employed to consider the stiffness degradation and strength reduction of the reinforced concrete members under reversed cyclic loading in nonlinear time history analysis. Rigid elements were employed at the superstructure ends and at the rigid zones of column and cap beam connections (Tubaldi *et al.* 2010). As shown in Fig. 9, analytical modeling of embankment fill, piles, elastomer bearings and pounding elements were made using spring elements with proper parameters, which were discussed in detail in Avşar *et al*. (2011), Avşar (2009).

For representation purposes, threshold values for the EDPs of the first bridge sample (BR1)

Table 2 General properties of the selected sample bridges

Bridge Id	Max Span Length (m)	Number of Spans	Column Height (m)	No. of Columns per Bent	Skew Angle (°)
BR1	20	2	6.7	3	20
BR2	35	4	8.7	1	25
BR3	25	3	7.2	2	5
BR4	25	5	9.6	1	15
BR5	30	4	7.8	3	50
BR6	30	2	4.3	3	0
BR7	20	2	5.6	2	40

Table 3 Important features of earthquake records used in the analyses

Earthquake	Date	Station	Mw	D (km)	PGA (g)	PGV (cm/s)	PGA/PGV (1/s)
Duzce	12.11.1999	375 Lamont 375	7.1	8.2	0.706	27.15	25.51
Bingol	01.05.2003	Bingol Dir. of Public Works and Settlement	6.1	4.9	0.396	28.37	13.67
Kobe	16.01.1995	0 KJMA	6.9	0.6	0.701	77.72	8.85

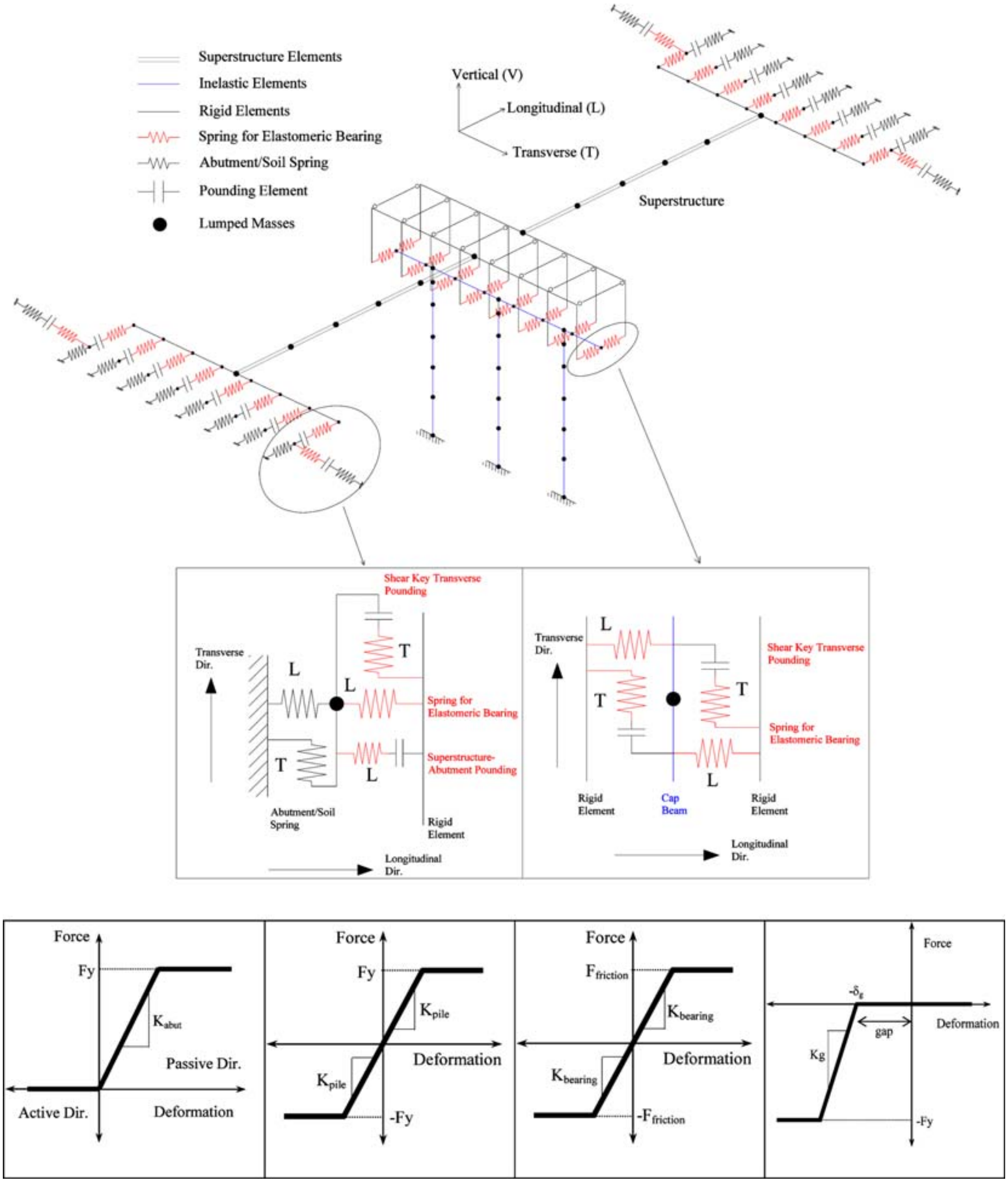


Fig. 9 3D analytical model of bridges and spring models for nonlinear bridge components: embankment fill, piles, elastomeric bearings, and pounding elements

components are determined for each damage limit state and shown in Fig. 10 both for positive and negative values. These limit values are compared with the maximum seismic response of bridge to determine the damage states of bridge components under Düzce Earthquake. Threshold values for the EDPs of column curvature 3-3 (section strong axis), column curvature 2-2 (section weak axis), and superstructure relative displacement are compared with their seismic demand represented by

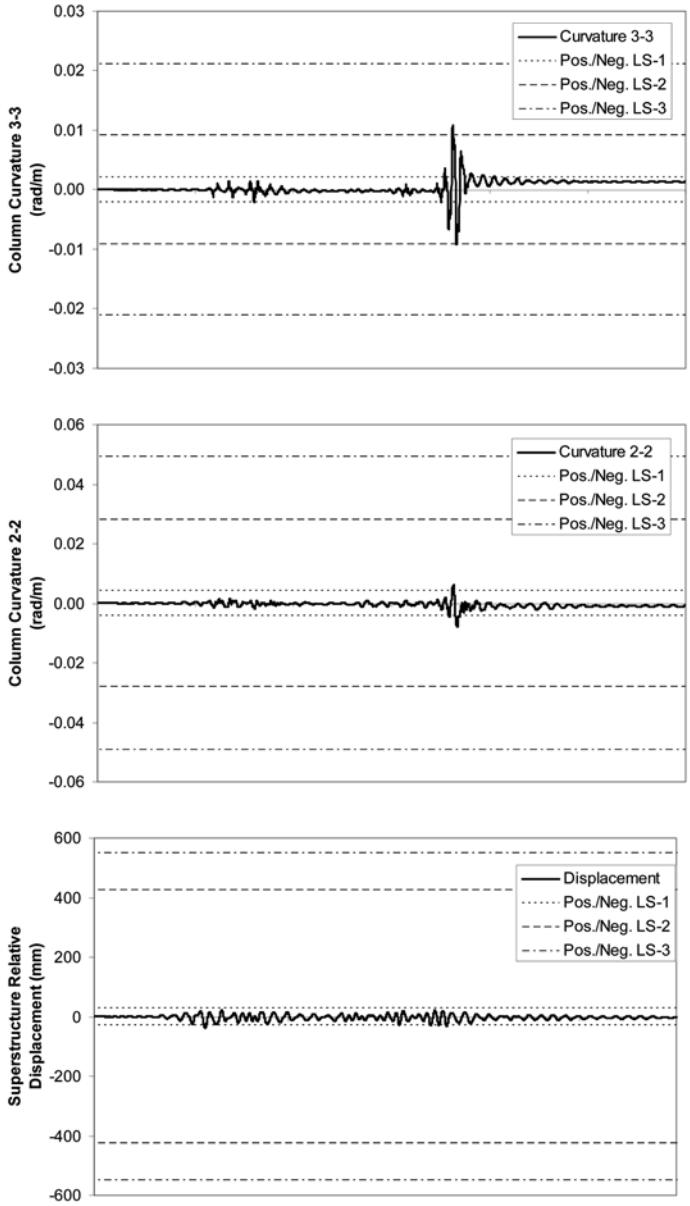


Fig. 10 Maximum seismic response of different damage parameters of BR1 under Duzce EQ

time series as shown in Fig. 10. Maximum seismic response for column curvature 3-3 exceeds the limiting value of the Damage Control Limit State (LS-2). Whereas, maximum seismic response for column curvature 2-2 and superstructure relative displacement exceed the limiting value of the Serviceability Limit State (LS-1). This indicates that the column is in the Significant Damage State. Since Column curvature 3-3 is the most critical EDP, it dictates the damage state of the bridge.

Seismic damage assessment of the sample bridges is tabulated in Table 4 for the three earthquake records. Damage state of the bridge samples is calculated for different damage limit states by

Table 4 Determination of the damage state of the selected bridge samples

Serviceability Limit State (LS-1)

Bridge Sample	Column Curvature Long. Dir.			Column Curvature Trans. Dir.			Cap Beam Curvature			Superstructure Disp.			OverAll		
	Duzce	Bingol	Kobe	Duzce	Bingol	Kobe	Duzce	Bingol	Kobe	Duzce	Bingol	Kobe	Duzce	Bingol	Kobe
BR1	1	1	1	1	1	1	0	0	1	1	1	1	1	1	1
BR2	0	1	1	0	0	1	0	0	0	0	1	1	0	1	1
BR3	1	1	1	0	1	1	1	0	1	1	1	1	1	1	1
BR4	0	0	1	0	0	1	0	0	0	1	1	1	1	1	1
BR5	0	1	1	0	1	1	0	0	1	1	1	1	1	1	1
BR6	1	1	1	1	1	1	0	0	1	1	1	1	1	1	1
BR7	0	0	1	0	1	1	1	0	0	1	1	1	1	1	1

Damage Control Limit State (LS-2)

Bridge Sample	Column Curvature Long. Dir.			Column Curvature Trans. Dir.			Cap Beam Curvature			Superstructure Disp.			OverAll		
	Duzce	Bingol	Kobe	Duzce	Bingol	Kobe	Duzce	Bingol	Kobe	Duzce	Bingol	Kobe	Duzce	Bingol	Kobe
BR1	1	0	0	0	0	1	0	0	1	0	0	0	1	0	1
BR2	0	0	1	0	0	1	0	0	0	0	0	0	0	0	1
BR3	0	0	1	0	0	1	0	0	1	0	0	0	0	0	1
BR4	0	0	1	0	0	1	0	0	0	0	0	0	0	0	1
BR5	0	0	1	0	0	1	0	0	1	0	0	0	0	0	1
BR6	1	1	1	0	0	1	0	0	0	0	0	0	1	1	1
BR7	0	0	1	0	0	1	0	0	0	0	0	0	0	0	1

Collapse Prevention Limit State (LS-3)

Bridge Sample	Column Curvature Long. Dir.			Column Curvature Trans. Dir.			Cap Beam Curvature			Col. Shear Long. Dir.			Col. Shear Trans. Dir.			Cap Beam Shear			Superstructure Disp.			OverAll		
	Duzce	Bingol	Kobe	Duzce	Bingol	Kobe	Duzce	Bingol	Kobe	Duzce	Bingol	Kobe	Duzce	Bingol	Kobe	Duzce	Bingol	Kobe	Duzce	Bingol	Kobe	Duzce	Bingol	Kobe
BR1	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
BR2	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
BR3	0	0	1	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
BR4	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
BR5	0	0	0	0	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1
BR6	1	0	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	1
BR7	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1

0 = NOT Attained the Specified Damage Limit State
1 = Attained the Specified Damage Limit State

inspecting the component level damage states. If the bridge component has reached or exceeded a certain damage limit state, then the score of the corresponding bridge component for that limit state is assumed to be 1, otherwise 0. According to the assumption made in identifying the overall bridge damage state, if any of the bridge component has the score of 1, then the bridge is assumed to be in that damage state with the score of 1. In this simple approach, if any of the bridge components attains or exceeds a damage limit state, bridge system as a whole is assumed to be in the same damage state regardless of the damage states of the rest of the bridge components. In other words, seismic performance of the bridge as a whole is dominated by the weakest component having the worst seismic performance.

In the first two damage limit states (Serviceability and Damage Control) only 4 damage parameters are taken into account in the seismic performance assessment of the sample bridges. Whereas in the Collapse Prevention limit state, in addition to the previous damage parameters column and cap beam shear demands are also included in the assessment. As mentioned in the previous section, since shear failure is a brittle type of failure resulting in a sudden collapse of the RC members, only collapse prevention limit state is defined for the shear capacity of cap beams and columns in both principal axes. Seismic shear demand values of the RC components under three earthquakes have been computed to be less than the section capacity determined for the collapse prevention limit state of all bridge samples. According to Table 4, collapse prevention limit state of the selected bridge samples is dominated by the EDP of column and cap beam curvature.

7. Conclusions

The most significant contribution of this study is the development of seismic vulnerability assessment criteria for the RC highway bridges in Turkey constructed after the 1990s. These criteria are one of key components in the seismic performance and risk assessment of these bridges. Thus, the approach and criteria presented here is proposed to be used for seismic performance assessment of ordinary bridges in Turkey as well as for the development of their fragility curves. Specific damage limit states are described for highway bridge components. These are the "Serviceability", "Damage Control" and "Collapse Prevention" damage limit states. Slight/No, moderate, significant, and collapse are the four corresponding damage states that the bridges can experience under the effect of an earthquake ground motion. Both qualitative and quantitative descriptions are given for the three damage limit states. Several EDPs of various bridge components are considered for the quantitative definitions of each damage limit state. Curvature and shear capacities of the RC column and cap beam members in both principal axes and the superstructure relative displacement are the EDPs in defining the damage limit states of the highway bridges. The overall bridge damage state can be determined by inspecting the damage state of each specified bridge component. The most critical bridge component dominates the seismic performance of the bridge system.

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