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PROBABILISTIC PERFORMANCE-BASED SEISMIC DESIGN OF BRIDGE COLUMNS

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Abstract. *Because earthquakes are random events it is reasonable to design structures taking into account the non-deterministic features of response. Performance-based design is intended to provide options to designers and bridge owners to manage the target performance of the structure. Probabilistic damage control approach (PDCA) is a new procedure for seismic design of bridges subjected to earthquakes. As part of a study funded by the California Department of Transportation (Caltrans), this innovative design methodology is developed by incorporating "Damage Index" (DI) and different earthquake return periods to provide flexibility to the designer. The damage level of the bridge is established for each earthquake level as a function of the degree of lateral displacement nonlinearity and is related to six apparent damage states (DS) defined in a previous study at the University of Nevada, Reno. The correlation between DI and DS was determined from a statistical analysis of measured data for 21 bridge column models subjected to seismic loads on shake tables. Extensive analytical modeling of seismic response of two-column bents was conducted. A wide range of variables was included in the study to address the effect of aspect ratio, transverse steel ratio, longitudinal steel ratio, material properties, site class, return period, target DI, and the number of columns in bents. Each column bents was analyzed under fifteen near-field and ten far-field ground motions. A statistical analysis of the demand DIs was performed to develop fragility curves and calculate the reliability index for each DS. The calculated reliability index and fragility curves show that the proposed method can be effectively used in seismic design of new bridges.*

1. INTRODUCTION

Based on the lessons learnt from the previous earthquakes, performance-based seismic design of bridges has become of interest to many researchers and structural engineers. Performance based seismic design [PBSD] is based largely on displacement consideration rather than strength/forces [10]. The fundamental philosophy behind PBSD is that the structure should be designed to achieve a specified performance level under design earthquake. However, most of past research on seismic design of bridge columns is based on deterministic approach and therefore, does not take into account the non-deterministic nature of earthquake events, and scatter in column material and geometric properties. Vosooghi and Saiidi [15] developed fragility curves provide a probabilistic approach to account for uncertainties in the seismic response of bridge columns subjected to earthquakes.

In the present study probabilistic damage control approach (PDCA) is introduced in PBSD of bridge columns subjected to earthquakes. PDCA approach considering the possible uncertainties in earthquake demand and bridge response was developed for probabilistic PBSD of bridge columns. This innovative design methodology is developed by incorporating “Damage Index” (DI) and different earthquake return periods. The concept of PDCA approach by incorporating DI was proposed by Tourzani et al [12], for designing and evaluating bridge column based on predefined or expected performance level. However the uncertainties in earthquake demands and bridge responses were not included in their study.

In this article, PDCA approach was applied to many single column and two columns bents (SCB and TCB). However, in the present study, only TCB’s are discussed. TCB’s were design for a predefined performance under design earthquake. To quantify the performance of a bridge column, each performance level was correlated to each possible apparent DS and subsequently, each damage state was correlated to their associated DI. DI was defined as the ratio of plastic displacement demand to plastic displacement capacity (Eq. 1). The correlation between DI and damage state (DS) was determined from a statistical analysis of measured data for 21 bridge column models subjected to seismic loads on shake tables [15].

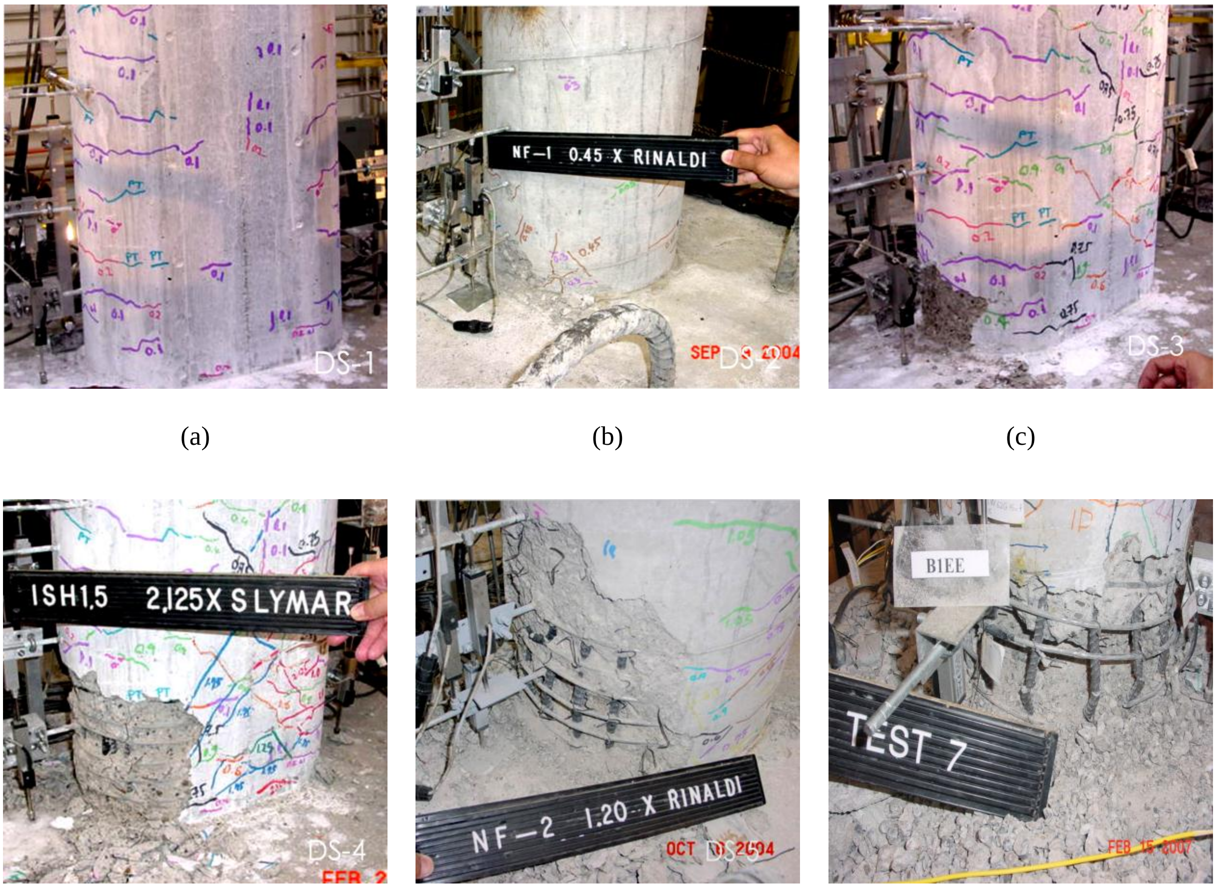
$$\mathrm{DI} = \frac{\mathrm{D_d} - \mathrm{D_Y}}{\mathrm{D_u} - \mathrm{D_Y}} \tag{1}$$

To develop a large database/scatter for damage index demand ($\mathrm{DI_d}$), nearly 400 non-linear dynamic analyses was performed utilizing SAP 2000. To consider uncertainties in $\mathrm{DI_d}$, various column dimensions, support conditions, longitudinal steel ratios, site classes, and distance to active faults were considered in the analyses. A statistical analysis of the demand $\mathrm{DI_d}$ was performed to develop fragility curves and calculate the reliability index. Reliability analysis was conducted to calibrate the design DI to obtain a sufficient reliability against failure.

2. DAMAGE STATES AND DAMAGE INDICES

To develop probabilistic performance based design approach various performance levels for different earthquake return periods were defined. Each performance level was quantified in terms of DS. Each DS was correlated to corresponding DI. Damage states defined previously in the study at University of Nevada Reno [14] was utilized. Six possible apparent damage states defined are as follow [15]; DS1 = flexural cracks, DS2 = minor spalling, DS3 = Extensive spalling, DS4 = visible lateral or longitudinal bars, DS5 = imminent core failure, and DS6 = failure/fractured bars. Six apparent damage states are shown in Fig. 1. Figs. 1(a) to 1 (d) represent the DS’s from DS1 to DS6, respectively.

The columns were tested under gradually increasing loads in the original studies to cover all possible damage states (DS1 to DS6). The peak displacement was measured at each DS and subsequently, DI was calculated. DI varies between zero and one corresponds to no damage and failure, respectively. For each DS, large scatter in the calculated DI was observed. To take into account the scatter in DI, fragility analysis for each DS was performed utilizing cumulative lognormal distribution function [15]. Fragility curves were justified utilizing Smirnov-Kolmogorov goodness of fit test [5] with 10% level of significance [6]. Fragility curves for each DS are shown in Fig. 2. DS6 is corresponds to failure with DI equal to one, therefore, even though in some cases where DI was greater than one, it was considered equal to one and consequently, no scatter in the data was considered and no fragility curve was developed for DS6. Also for reliability analysis mean and standard deviation of DS6 for resistance model was considered one and zero, respectively. Fragility curves shown in Fig. 2 are the resistance model for the reliability analysis.



(a) (b) (c)

(d) (e) (f)

Figure 1. Possible apparent damage states of bridge columns

3. PERFORMANCE OBJECTIVES

Apparent damage states were used to define the performance objectives of bridge columns under design earthquake of 1000 year return period. Earthquake of 1000 year return period is corresponds to 7% probability of exceedance in 75 years [1]. Based on the discussion with Caltrans engineers, columns were designed for DS3 under design earthquake. To design columns for DS3, initially tentative target design DI of 0.35 was selected from the fragility curve shown in Fig. 2. Target design DI of 0.35 is corresponds to the 50% probability of exceedance of DS3. To calibrate the target design DI, reliability analysis was performed. The choice of target reliability index for any structure is bit subjective. The AASHTO LRFD recommends target reliability index of 3.5 for gravity loads. In the present study, it was decided to use β of 3 against failure because earthquake loading is non-deterministic and not always present on a structure unlike gravity loads.

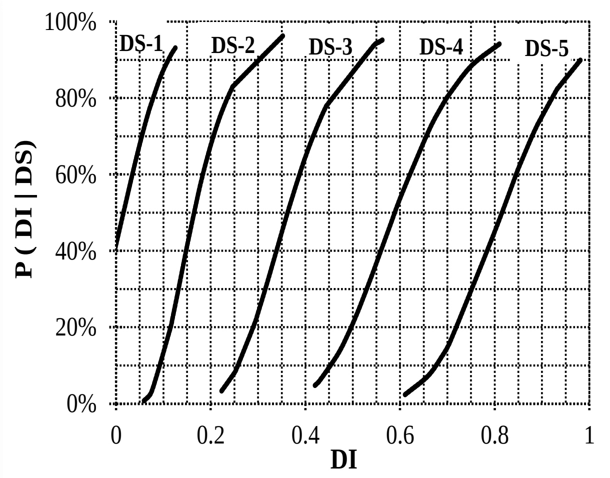


Figure 2. Fragility curves of damage index for various damage states (Vosooghi and Saiidi, 2010)

4 SEISMIC DEMAND ANALYSIS

To determine DI_d non-linear dynamic analysis was performed on various TCB models utilizing SAP 2000. All bents were analyzed under ten far-field and fifteen near-field ground motions selected from PEER strong ground motion data base [9]. Peak displacement of bent for each ground motion was recorded and associated DI_d to peak displacement was calculated. To consider uncertainties in seismic demand, various parameters were considered in the analyses. These parameters are divided mainly into the following categories:

4.1. Bridge site class

Bridge site class was divided into two categories; site B/C and site D. Site B, C, and D is correspond to the rock, soft rock, and stiff soil, respectively [1]. Because site B and C both represents rock, therefore, for sake of simplicity they are lumped together in one site class as B/C. Caltrans ARS online [3] and USGS de-aggregation beta website [13] were utilized to determine design spectrum for site class B/C and D, respectively. For design spectrum shear wave velocity (V_{S30}) of 760 m/s and 270 m/s was used for site B/C and site D, respectively. 760 m/s represents the median of V_{S30} of site class B and C, whereas, 270 m/s represents the average V_{S30} of site class D.

4.2. Ground motion selection

Un-scaled ground motions were selected from PEER strong ground motion database [9]. To consider the uncertainties in the ground motion, various site parameters were considered. The selection criteria of ground motions were based on the V_{S30}, distance to the fault (R_{jb}), and scaling factors (SF). V_{S30} between 500 to 1500 m/s and 200 to 360 m/s were considered to select ground motions for site class B/D and D, respectively. R_{jb} between 0 to 15 km and 15 to 30 km was selected for near-field and far-field ground motions, respectively. Ground motions were selected so that the scale factor calculated based on spectral acceleration associated with period of one second is not greater than 3. Based on these parameters, fifteen near-field and ten far-field ground motions were selected for each site class.

4.3. Column bent properties

To consider uncertainties in column bent properties, the affect column height to diameter ratio (H/D), longitudinal steel ratio (ρ_l), and column support conditions were considered in the analyses. All bents had circular section with 6 feet diameter. Square cap beam section of seven feet was used in all TCB's. Various TCB models used in the analyses are presented in Table 1. All bents were designed for axial load index of 10%. The expected material properties as specified in SDC Caltrans 2010 were used to design column bents. . The specified concrete strength of 3.6 ksi and steel Gr. 60 were used in the design. The columns were not designed for shear because the intention of this research was to study the scatter on seismic demand of the columns designed for a given DI.

Site class	Config.	H/D	Long. steel ratio	Period	Yield Displacement	Demand Displacement	Ultimate Displacement	Damage Index
			%	secs	in	in	in	in/in
B	Fix-Pin	5	1	1.29	3.80	6.70	12.20	0.34
			2	1.15	4.70	6.10	9.50	0.29
			3	1.05	5.10	5.70	8.60	0.16
D	Fix-Pin	5	1	1.30	4.10	10.70	21.30	0.39
			2	1.16	5.10	9.50	16.20	0.39
			3	1.06	5.70	8.70	14.70	0.34
B	Fix-Fix	5	1	0.69	2.20	3.50	5.60	0.37
D	Fix-Fix	5	1	0.70	2.40	5.10	8.60	0.43
			2	0.64	3.00	4.40	6.70	0.38
			3	0.58	3.20	3.80	5.40	0.27
D	Fix-Pin	10	1	3.62	14.9	26	47.3	0.34
			2	3.13	17.4	23.3	38	0.29
			3	2.81	18.5	21.5	32.8	0.61
D	Fix-Fix	10	1	1.87	8.2	15.3	25.9	0.4
			2	1.63	9.7	13.5	21.2	0.33
			3	1.47	10.2	12	17.7	0.24

Table 1. Two column bent properties and design parameters

5 Analyses results

Non-linear dynamic analysis were conducted using fifteen near-field and ten far-field ground motions scaled at the spectral acceleration associated with the fundamental period of the bent. Nearly 400 analyses were conducted. The peak relative displacement for each analysis was determined and subsequently, DI_d associated to peak displacement was calculated. A large scatter was observed in the calculated DI_d. To take into account the scatter in DI_d, cumulative normal distribution functions were utilized to perform fragility analysis. Fragility analysis was justified utilizing Smirnov-Kolmogorov goodness of fit test [5] with 10% level of significance [6]. The fragility of DI_d is shown in Fig. 3. This DI_d fragility curve represent the load model in the reliability analysis.

Fig. 3 show that 18% of the calculated DI_d data is falling outside the limit curves. It was observed that fragility curve for TCB's using normal distribution function is very sensitive to number of failures (DI = 1) occurred in the analyses. In TCB's six failures were occurred which cause this data to fall outside the limit curves. Excluding these six data points, only 5% of the total data falls outside the limit curves. Moreover non-cumulative normal frequency distribution of DI_d shows that, calculated DI_d data follow the normal distribution, as shown in Fig. 4. Due to these reasons normal distribution function for TCB's was considered reasonable.

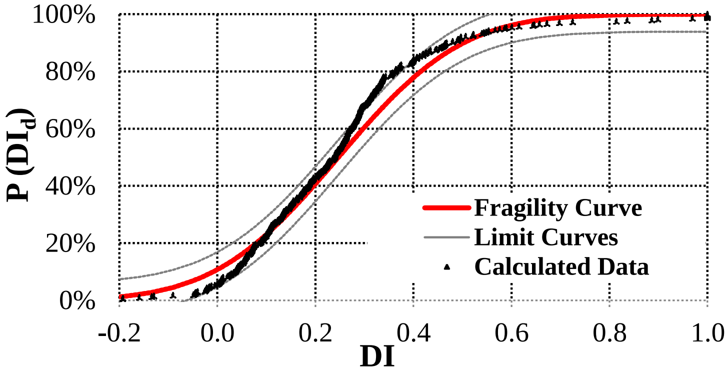


Figure 3. Fragility curve for damage index at different damage state

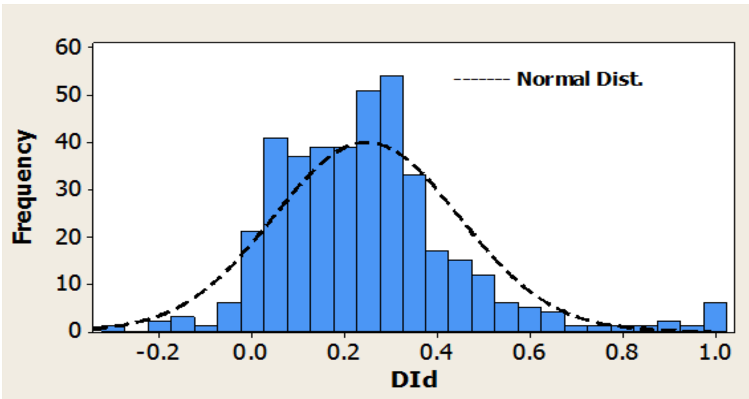


Figure 4. Normal frequency distribution of damage indices

6 RELIABILITY ANALYSIS

The reliability of a structure commonly expressed in terms of reliability index (β) [7]. The reliability analysis presented in this paper is based on large experimental test data and comprehensive analysis of column resistances and load models, respectively. Because the scatter in data was normally distributed, therefore, β corresponds to normal distribution was calculated by utilizing equation Eq. 2 [2, 7]. μ_R and σ_R is the mean and standard deviation of DI corresponds to DS3 in resistance/capacity curve (Fig. 2). μ_Q and σ_Q is the mean and standard deviation of DI corresponds to DS3 in load/demand curve (Fig. 3). The probability of bent failure corresponds to given β can be calculated by using Eq. 3. The β calculated using Eq. 2 does not include probability of earthquake exceednace during the life time of a structure.

$$\beta = \frac{\mu_R - \mu_L}{\sqrt{\sigma_R^2 + \sigma_L^2}} \tag{2}$$

$$\beta = -\Phi^{-1}(P_f) \tag{3}$$

Where, P_f and Φ represent the probability of failure and normal standard distribution function, respectively.

Probability of bent failure is related to the return period or exceedance probability of the design earthquake. In the present paper all bents were designed for an earthquake of 1000 year return period. Therefore, it is important to determine the probability of bent failure combined with the probability of earthquake exceedance during the life time of a structure. Considering annual earthquake events are independent, the probability of earthquake exceedance during lifetime of a structure can be calculated using Eq. 4 [16]:

$$P_{EQ} = 1 - \left(1 - \frac{1}{T}\right)^t \tag{4}$$

Where, P_{EQ}, t and T are the probability of earthquake exceedance during the life time of a structure, life time of a structure and return period, respectively. The P_{EQ} calculated utilizing Equation 4 was equal to 0.072. The probability of bent failure (P_{BF}) and at the same time to have earthquake occurrence can be calculated using conditional probability as:

$$P_{BF} \cap P_{EQ} = \left(P_{BF} \,\middle|\, P_{EQ}\right) \times \left(P_{EQ}\right) \tag{5}$$

$$P_{BF} \,\middle|\, P_{EQ} = 1 - \Phi(\beta) \tag{6}$$

$P_{BF} \mid P_{EQ}$ is the probability of bent failure given the design earthquake has occurred. Using β calculated from Eq. 2, $P_{BF} \mid P_{EQ}$, was calculated utilizing Eq. 6.

Having P_{EQ} and $P_{BF}|P_{EQ}$, the probability of bent failure (P_{BF}) combined with probability of earthquake exceedance during the life time of a structure was calculated using Eq. 5. Having the combined probability of failure, β was back calculated from Eq. 3. The level of reliability at DS3, DS4, and DS5 was also investigated. Fig. 5 shows the cumulative β combining the DI_d data of all TCB models. The reliability indices presented in Fig. 5 are based on target design DI of 0.35 and earthquake of 1000 year return period.

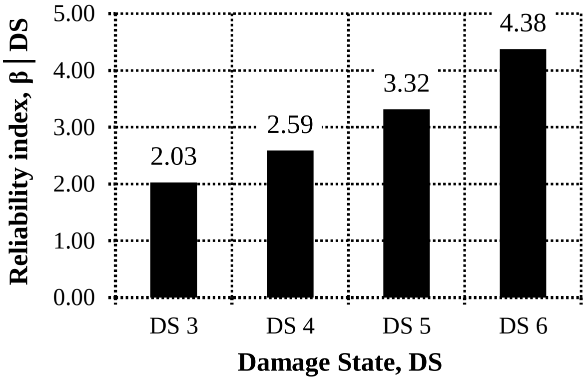


Figure 5. cumulative reliability index at each damage state

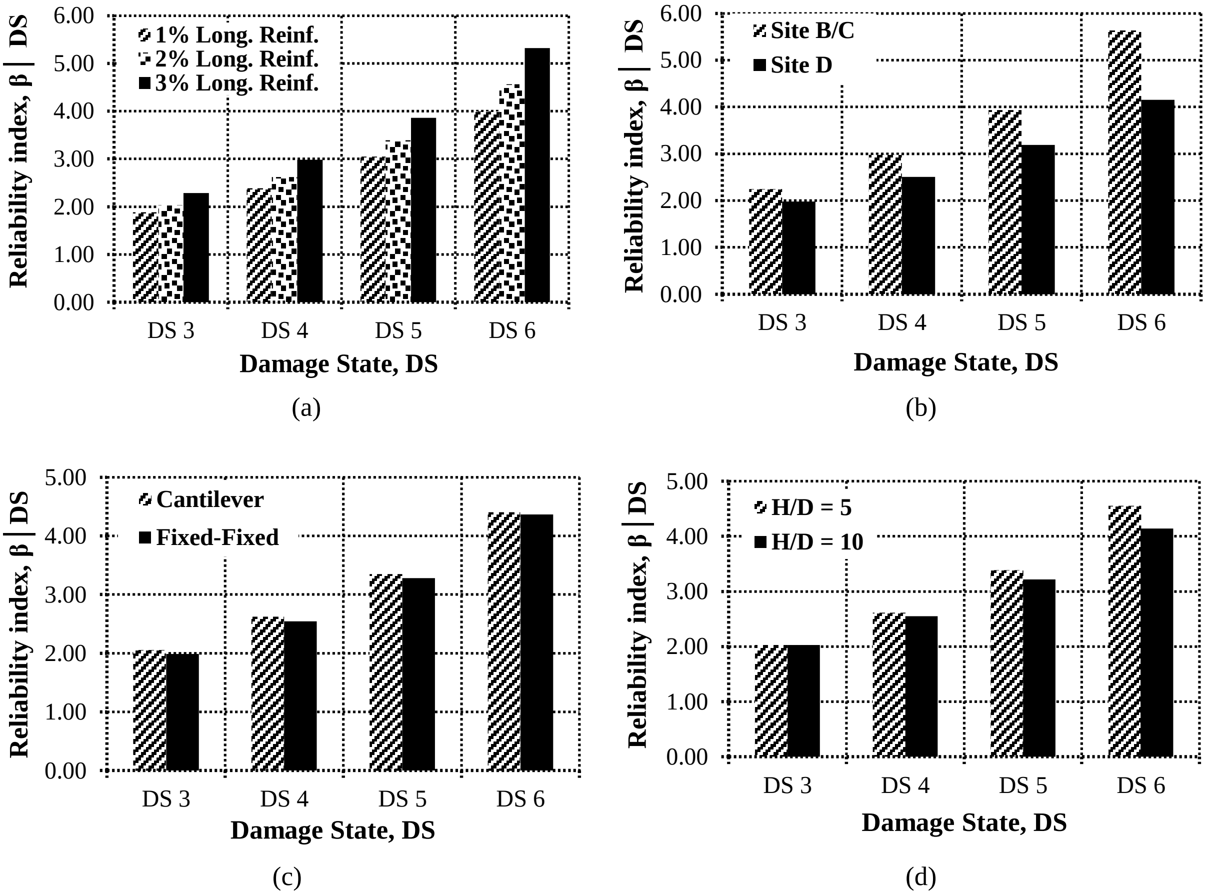


Figure 6. The effect of different parameters on reliability indices at different damage states

Figs. 6 (a) to 6 (d) represent the β for TCB's under various longitudinal steel ratio, site class, support conditions, and column height to diameter ratio (H/D) ratio, respectively. Figs. 6(a) and 6(c) indicate that the reliability indices increase by increasing longitudinal steel ratio and are independent of support conditions, respectively. Fig. 6(b) indicates that TCB's for site class B are more reliable than TCB's for site class D, and Fig. 6(d) indicates that TCB's with H/D = 5 are more reliable than TCB's with H/D = 10.

7 CONCLUSION

The following conclusions were made based on the study presented in this paper:

- Seismic demand of bridge columns is sensitive to bent properties and site class parameters.
- Seismic response of bridges columns is sensitive to bent material properties and bent configurations.
- Reliability index against failure in two column bents is much higher when two column bents are designed for 50% probability of exceeding DS-3 (damage index of 0.35) under 1000-year earthquake. Therefore, to achieve an optimum reliability index against failure, two column bents can be designed for higher probability of exceeding DS-3 without causing a concern for failure.
- The new approach of PDCA by incorporating reliability index provides much flexibility to the structural designers to design a bridge column to reach a given damage state with a specified reliability under an earthquake with a given return period.

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