Executive Summary

1. Introduction

Sub-standard bridges should be evaluated in detail to identify their deficiencies and to prioritize a retrofitting program. Another important consideration is the serviceability of bridges in the aftermath of earthquakes. A serviceability assessment determines whether the bridge is sufficiently safe to be kept open to traffic without repair, if it is repairable within a reasonable period, or if it should be replaced. Analytical tools are needed to make such assessment.

The purpose of the present study was to develop a practical method for reliability-based seismic assessment of sub-standard bridge columns constructed prior to the development of the current seismic design codes. Sub-standard columns are susceptible to premature failure due to slippage of lap-spliced bars at the base, low shear capacity, low confinement, or a combination of these deficiencies. The assessment is carried out based on apparent damage and column properties with no non-destructive testing required. The study was composed of: (1) development of a capacity (resistance) model through defining simple criteria for visual damage states of sub-standard columns (SubDSs) and correlating them with a seismic response parameter (RP), (2) development of a demand (load) model for sub-standard bridge bents, and (3) a reliability analysis for the columns to determine the extent of their seismic vulnerability.

2. Research Objectives

The overall objective of this study was to develop a method and tools for probabilistic assessment of earthquake damage in sub-standard bridge columns for different earthquake return periods and damage states. Specifically, this study was aimed at:

1- Developing a capacity (resistance) model for sub-standard bridge columns using detailed data from previously tested bridge column models.
2- Developing a relatively simple analytical approach to capture the effect of lap-spliced bars on the response of the sub-standard columns through modifying the stress-strain relationship of steel.
3- Developing a robust demand (load) model for typical sub-standard single-column and multi-column bridge bents in California utilizing the USGS hazard curves and nonlinear dynamic analysis (NLDA).
4- Developing demand distribution curves and reliability charts for existing sub-standard bridge columns through fragility and statistical analyses.
5- Providing illustrative examples on the application of reliability charts in seismic assessment of bridge columns.

3. Seismic Damage States and Response Parameters

The available test data and observations from prior experimental studies of 25 test models that had been tested under cyclic or shake table loading was reviewed to determine the most common type of apparent seismic damage in sub-standard columns. Four distinct damage states were identified as indicators of damage level: (1) flexural cracks in the lap-splice region but no vertical cracks (SubDS-1), (2) minor vertical or shear cracks in the lap-splice region (SubDS-2), (3) extensive vertical (block-shape) or
shear cracks in the lap-splice region (SubDS-3), and (4) slippage of lap-spliced bars or failure due to shear (SubDS-4). An example of each damage state is shown in Fig. 1.

![SubDS-1](image1.png) ![SubDS-2](image2.png)

![SubDS-3](image3.png) ![SubDS-4](image4.png)

Figure 1. Damage states for sub-standard bridge columns

A damage index (DI) was defined to represent the column response. This approach is consistent with previous research on probabilistic damage control approach (PDCA) of standard (modern) columns. The DI for standard columns could not be used in the present study because it involves plastic displacement that is not applicable to relatively brittle sub-standard columns. As a result, the following expression was developed for sub-standard columns:

\[ DI = \frac{D_{\text{max}}}{D_u} \]  

in which \( D_{\text{max}} \) is the column lateral displacement demand and \( D_u \) is the column displacement capacity. The displacement capacity of sub-standard columns was assumed to correspond to the point in which the lateral load on the column drops substantially due to either slippage of spliced bars or shear failure.
4. Development of Capacity Model

To determine the correlation between SubDSs and capacity DIs, the point corresponding to the maximum measured displacement at a given damage state was marked on the force-displacement relationship for the column models, the displacement capacity of test models was determined, and DI at each SubDS was calculated. Figure 2 depicts a sample of force-displacement curves with marked SubDSs.

To develop a capacity model, all the column models were grouped in a single category and distribution curves were developed to correlate the SubDSs with DI using a probabilistic approach of “fragility function.” The curves provide the probability of exceeding a particular damage state given a response parameter. It was assumed that, similar to fragility functions, load distributions take the form of cumulative lognormal distribution functions. To check how well cumulative distributions fits cumulative logarithmic distribution, the Kolmogorov-Smirnov goodness-of-fit test was used. The capacity distribution curves for sub-standard bridge columns at each damage state is shown in Fig. 3.
5. Development of Demand Model

The demand component used in this study was the demand DI for sub-standard bents under different ground motions (GMs). The demand database was developed by analyzing a large number of single-, two-, and four-column bents (SCBs, TCBs, and FCBs, respectively) under GMs with different return periods and including far-field and near-field GMs. To account for uncertainties in the demand model, the effect of the bent and GM properties including the number of columns per bent, support conditions, column aspect ratio, longitudinal steel ratio, lap-splice length, GM type, site class, and the earthquake return period was considered.

5.1. Bent Properties

Figure 4 depicts the details of the bents. To account for uncertainties in the bent properties, a large number of bents were considered with different longitudinal steel ratios, column diameters, heights, and support conditions. To account for variability in the support conditions, fixed-fixed, fixed-free, and pinned-fixed connections were assumed. To capture the effect of column aspect ratio, three column diameters of 4, 5, and 6 ft [1.22, 1.52, and 1.83 m] and two column heights of 30 and 60 ft [9.14 and 18.29 m] were used in the study resulting in aspect ratios of 5 to 15 for SCBs, 5 to 12 for TCBs, and 7.5 for FCBs. The steel yield strength and concrete compressive strength were not treated as variables in developing the demand models. ASTM A615 Grade 40 steel with an expected yield strength of 48 ksi [330.9 MPa] was assumed for longitudinal and transverse reinforcements. The specified and expected compressive strength of concrete were 3.85 and 5 ksi [26.5 and 34.5 MPa], respectively. The longitudinal steel ratio of the columns was 1%, 2%, and 3%. All the columns were reinforced with No. 11 [bar diameter (db)=35.8 mm] longitudinal bars and No. 4 [db=12.7 mm] hoops at 1 ft [304.8 mm] spacing regardless of the section dimensions. The hoops were assumed to incorporate lap splices in the cover concrete, which rendered them ineffective in providing confinement and shear strength. It was assumed that the column longitudinal
bars were spliced with starter bars extending from the footing with a lap splice length of 20, 24, and 30db. An axial load index (ALI) of 5% was assumed for all the columns. ALI is defined as ratio of the column axial load to the product of the column gross cross-sectional area and the specified compressive strength of concrete.

Figure 4. Details of bents (unit: ft [m])

5.2. Ground Motion Properties

Two categories of site classes were assumed: BC and D. Because sites B and C represent rock, they were lumped into one site class BC in the present study. A VS30 of 2,500 ft/s [760 m/s] and 886 ft/s [270 m/s] was assumed for site BC and D, respectively. To account for uncertainties in ground motions, various parameters were assumed. These parameters were used to distinguish between far-field and near-field GMs, including:

1- The VS30 in the range between 1640 to 4921 ft/s [500 to 1500 m/s] and 656 to 1181 ft/s [200 to 360 m/s] was used to select GMs for site class BC and D, respectively.

2- Only GMs with magnitude exceeding six were selected.

3- A distance to the vertical projection of the fault to the earth surface (Rjb) between
0 to 9.32 mi [0 to 15 km] and 9.32 to 18.64 mi [15 to 30 km] was used in selection of near-filed and far-filed GMs, respectively.

4- The GMs were selected so that scale factor (SF) does not exceed three. The SF is defined as the ratio of spectral acceleration at one-second period (Sa1) for the GM response spectrum and 1000-year design spectrum. The limit of three was applied to help select the GM’s used in demand analysis. Once a record was selected, it was scaled further based on the fundamental period of the bent before each dynamic analysis, as explained below.

Using the above criteria, 10 far-field and 15 near-field GMs were selected for each site class. A larger number of near-field motions was used because this motion type is generally more demanding. Out of the three GM components in each record set, the horizontal motion with the highest Sa1 was selected as the input GM in the analyses. Previous earthquakes and studies have shown that sub-standard bridge columns are susceptible to premature failure even under low-to-moderate earthquakes due to low confinement, small shear strength, and insufficient lap-splice length. To ensure that the earthquakes are in the low and moderate category, the fixed-base bents were analyzed under 60-, 100-, 250-, 500- and 1000-year return period earthquakes to allow for variety in the response. Pinned-base bents do not incorporate lap splices at the base, and thus are stronger. For these bents, only stronger earthquakes (those with longer return periods of 250, 500, and 1000 years) were used in the analysis.

Utilizing the United States Geological Survey (USGS) deaggregation tool, design spectra were developed for a site in Los Angeles with 34.05 degrees in latitude and -118.25 degrees in longitude. A Vs30 of 2,500 ft/s [760 m/s] and 886 ft/s [270 m/s] for site BC and D, respectively, and a damping ratio of 5% were assumed. The input GMs were scaled to match the design spectra at the fundamental period of the bents. Using the dynamic mass and the effective stiffness of the bent, the fundamental period of the bents was calculated. The effective stiffness of the bents was determined as the slope of the idealized elasto-plastic representation of the pushover response of the bents.

5.3. Analytical Method for Slippage of Lap-spliced Bars

A new approach was developed in the present study to implicitly account for the effects of the slippage of spliced bars on the response of sub-standard columns with inadequate lap-splice length. It was assumed that footing starter bars and the column longitudinal bars are attached to the surrounding concrete using distributed uniaxial springs placed at location of the bar ribs [Fig. 5(a)]. The force-displacement relationship of each spring was defined using the cyclic local bond stress-slipage model developed by Xiao and Ma (1997). A schematic of the Xia and Ma bond-slip relationship is shown in Fig. 5(b). The bond stress-slip relationship in this model is expressed in the form of Popovics’ equation. To determine the force-deformation relationship of the lap-spliced bars under net tensile force, OpenSees program was utilized. The overall deformation of the bar consisted of the bar elongation and slippage represented by displacement of the springs. The force-deformation relationship of the bar was converted to the corresponding stress-strain relationship of the longitudinal steel.

The modified stress-strain relationships for ASTM A615 Grade 40 No. 11 [db=35.8] longitudinal bars with lap-splice length (Ls) of 20db, 24db, 30db, and 35db is depicted in Figs. 6. It was assumed that the column transverse steel in all cases was No.
4 [db=12.7 mm] spliced hoops at 1 ft [305 mm] spacing, which did not provide any confinement to the core concrete.

![Figure 5](image1)

**Figure 5.** (a) Schematic of analytical model for lap-spliced bar; (b) schematic of Xiao and Ma bond stress-slip relationship

![Figure 6](image2)

**Figure 6.** Modified stress-strain relationship for No. 11 [db=35.8 mm]

### 5.4. Shear Failure Modeling

The possibility of shear failure was accounted for implicitly in the pushover and dynamic analyses. It was assumed that the hoops were ineffective because they were closed by lap splices in the cover concrete. As a result, the contribution of the transverse steel to the shear capacity was ignored. The concrete shear capacity was determined based on the Caltrans SDC (2013) method, which is dependent on the displacement ductility. The shear capacity of the bents was determined taking into account that lap splice slippage is the dominant failure mode. If the calculated shear capacity was lower
than the corresponding lateral load at slippage, shear failure controlled the lateral load capacity of the bent. Consequently, the displacement ductility and thus the shear capacity were updated based on the new ultimate lateral load and the “effective yield” displacement. This process was repeated until the displacement ductility converged to within 1% of the displacement ductility previously determined. The displacement corresponding to the shear capacity (displacement capacity) was used in calculation of demand DIs in these cases.

5.5. Analytical Modeling

OpenSees v2.4.5 was used for analytical modeling of the bents. The columns were modeled using nonlinear beam-column elements with fiber sections. The cap beam in multi-column bents was modeled using elastic beam-column element. The analytical modeling method for all the bents was the same except for the connection of the columns to the base and cap beam. In all the models, the expected material properties were assigned to the fibers. A dead load corresponding to 5% ALI was applied as a vertical point load on the columns. The P-Delta effect was included in the analyses. A mass corresponding to 5% ALI was lumped at the end nodes of the cap beams. Rotational inertial masses were not included. A damping ratio of 5% was used in dynamic analyses. The bond-slip effect at column-to-cap beam connections was modeled using a modified stress-strain relationship for reinforcing steel bars developed by Tazarv and Saiidi (2014).

5.6. Pushover Analysis

Pushover analysis was conducted to determine the dominant failure mode, the displacement capacity, and the initial stiffness that was used in calculating the fundamental period of the bents. The shear failure and slippage of lap-spliced bars at the base were the dominant modes of failure in fixed-base bents. For pinned-base bents, the failure was due to shear, core concrete crushing, reinforcement fracture or significant drop in the lateral load capacity of the bent.

5.7. Dynamic Analysis

Nonlinear dynamic analysis was conducted on SCBs, TCBs, and FCBs to determine the demand DIs under different earthquake records. The results were used in developing the demand model. The effects of the slippage of spliced bars at fixed-base connections, column shear failure, and bond-slip at column-to-cap beam connections were included in the analyses. Each bent was subjected to 25 scaled GM records (15 near-field and 10 far-field GMs) to account for uncertainties in the GM type (near field and far field), site class (BC and D), and earthquake return period (60 to 500 years for fixed-fixed and fixed-free bents, and 250 to 1000 years for pinned-fixed bents).

5.8. Demand Distribution Curves

To model the scatter in the demand damage index (DI), demand distribution curves were developed for each bent using statistical analyses. It was assumed that the curves take the form of cumulative log-normal distribution functions. The Smirnov-Kolmogorov goodness-of-fit test was selected as an acceptance criterion for continuous cumulative log-normal distributions. To use this test, it was assumed that the analytical DIs represents a sample of a population. This assumption was satisfied by conducting
extensive dynamic analysis to develop reasonable scatter in data. The acceptable demand distribution curves serve as a demand (load) model in the reliability analysis. Figure 7 shows a sample of demand distribution curves for fixed-fixed SCBs with 4-ft [1.22-m] column diameter, 30- and 60-ft [9.14- and 18.29-m] column height, 1% to 3% steel ratio, 20db splice length in site class BC under 60-year return period earthquakes.

![Figure 7. Load distribution curves for fixed-fixed SCBs with 4-ft [1.22-m] column diameter and 20db lap-splice length in site class BC under 60-year return period earthquakes](image)

6. Reliability Analysis

Reliability refers to the probability that sub-standard bridge bents will resist seismic loads resulting from a given earthquake level. A measure of structural reliability is defined by reliability index, $\beta$. Utilizing normally distributed capacity and demand distribution curves, reliability of exceeding a certain damage state for a specific bent category, site class, and earthquake return period was determined based on the first-order reliability method. In seismic assessment of sub-standard bridge columns, $\beta$ is used as a measure of structural safety. When $\beta$ is zero, the probabilities of failure and survival are the same, which is not acceptable. Columns with relatively small reliability index are more susceptible to damage, and thus are of higher priority for retrofit. Using conditional probability, the effect of the probability of occurrence of a particular earthquake in the lifetime of a bridge [(assumed to be 75 years based on AASHTO LRFD (2012))] was also accounted for in calculation of reliabilities (combined reliabilities, $\beta'$). Because the knowledge of the probability of exceedance of a damage state is important from downtime perspective for either retrofit or repair of bridges, $\beta'$ was determined against all SubDSs. The effects of support condition, site class, longitudinal steel ratio, lap-splice length, column diameter, column height, and earthquake return period on combined reliabilities were described. A sample of reliability analysis for fixed-fixed SCBs with 4-ft [1.22-m] column diameter, 30- and 60-ft [9.14- and 18.29-m] column height, 1% to 3% steel ratio, 20db splice length in site class BC under 60-year return period earthquakes is presented in Fig. 8. Complete results are presented in the main body of this report.
7. Illustrative Example

The database developed in this study can be utilized as a tool to (1) identify sub-standard bridge piers that are more susceptible to damage prior to occurrence of earthquakes and to prioritize the retrofitting program and (2) assess the level of expected damage to existing sub-standard bridge piers in the aftermath of earthquakes to prioritize the repair process. The following example shows how the combined reliabilities can be utilized to fulfill the foregoing goals.

Example: Consider three, two-span bridges with integral bent-superstructure connections. Figure 9 shows a schematic of the bridges. All the bridges are sub-standard and are located in site class D. Column longitudinal bars are lap spliced at the base and hoops provide negligible strength. The properties of the columns of the bents are listed in Table 1.

Under earthquakes with return period of 60 and 250 years, determine:
(a) The combined reliability index against failure ($\beta^*_4$) for the columns.
(b) The combined reliability for SubDS-1, SubDS-2, and SubDS-3.
(c) The probability of exceedance of SubDS-1 to SubDS-4.
(d) The most vulnerable bridge.
Figure 9. Schematic of bridges (a) Bridge-1 - single-column bent (SCB); (b) Bridge-2 - two-column bent (TCB); (c) Bridge-3 - four-column bent (FCB)

Table 1. Properties of columns

<table>
<thead>
<tr>
<th>Bridge No.</th>
<th>Bent Type</th>
<th>Column Diameter</th>
<th>Column Height</th>
<th>Splice Length</th>
<th>Longitudinal Steel Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>SCB</td>
<td>1.52 [ft]</td>
<td>30 [ft]</td>
<td>20db</td>
<td>1%</td>
</tr>
<tr>
<td>2</td>
<td>TCB</td>
<td>1.83 [ft]</td>
<td>60 [ft]</td>
<td>30db</td>
<td>3%</td>
</tr>
<tr>
<td>3</td>
<td>FCB</td>
<td>1.52 [ft]</td>
<td>30 (18.29) [m]</td>
<td>30db</td>
<td>3%</td>
</tr>
</tbody>
</table>

**Solution:** The reliability indices, combined reliability indices, and the corresponding probabilities of exceedance of each damage state for Bridge-1 to Bridge-3 were determined as shown in Figs. 10 through 12, respectively. The graphs show that, under earthquakes with a 60-year return period, the combined reliability indices against failure for Bridge-1, 2, and 3 are 1.7, 3.88, and 4.46 meaning that Bridge-1 is substantially more susceptible to failure than the other two bents because of the lower combined reliability index. The improved redundancy realized by two or more columns makes multi-column bent less vulnerable. However, under earthquakes with a 250-year return period, the combined reliability index is low for all the bents because these
Earthquakes are stronger and are more likely to cause failure in all the bents regardless of the number of columns. Earthquakes with a 250-year return period also impose the same level of damage and relatively small reliability index for other damage states regardless of the number of the columns in the bent as shown in Fig. 11(b). Fig. 11(a) shows that the reliability increases from SubDS-1 to SubDS-3 for all the bents under the weaker, more frequent earthquakes with a 60-year return period. Moreover, bents with smaller number of columns are expected to undergo a higher level of damage. Figure 11 shows that the combined reliability against failure for Bridge-1 is nearly the same for both return periods. This occurs due to the effect of the earthquake return periods on the reliabilities, which is revealed in comparing Figs. 10 and 11. Contrary to combined reliabilities, the reliabilities for 250-year return period is smaller than those of 60-year return period, as expected.

Figure 12 shows that the probability of exceedance of SubDS-4 (failure) for Bridge-1, 2, and 3 is very small meaning that failure is unlikely for these bents under 60- and 250-year return period earthquakes. The probability of exceedance of other SubDSs for all the bents under 250-year return period earthquakes is approximately the same because the bents fail under most of the earthquakes. Under 60-year return period earthquakes [Fig. 12(a)], higher damage states are less likely to be exceeded because the earthquakes are not strong. However, under stronger earthquakes with 250-year return period, more severe damage is expected, and hence probability of exceeding SubDS-3 is nearly the same as that for SubDS-1. Comparing Figs. 12(a) and 12(b) shows that for SubDS-1 and SubDS-2, the probability of exceedance is higher under earthquakes with a 60-year return period than that for a 250-year return period. This is because the latter group put higher demands on the bents.

This example validates the steel-jacketing of SCBs carried out by Caltrans after the 1989 Loma Prieta earthquake.

Figure 10. Reliability indices at different earthquake return periods (a) 60 years; (b) 250 years
Figure 11. Combined reliability indices at different earthquake return periods (a) 60 years; (b) 250 years

Figure 12. Probability of exceedance of SubDSs at different earthquake return periods (a) 60 years; (b) 250 years

8. Conclusions

The key conclusions of this study are highlighted as follows:
1- The proposed damage states were reasonable in terms of the number of distinct apparent damage in sub-standard columns.
2- The lateral force-displacement relationship is a simple and reliable reference to help introduce the new proposed damage index for sub-standard columns.
3- Due to either brittle failure prior to yielding or low displacement ductility capacity of sub-standard columns, the damage index defined for standard (modern) columns is not appropriate for sub-standard columns.
4- The proposed analytical method to account for the effects of the slippage of spliced bars on the response of sub-standard columns through modified steel properties is simple and practical.
5- Fixed-fixed and pinned-fixed sub-standard TCBs with relatively high longitudinal steel ratios and larger column diameters are more susceptible to shear failure than other multi-column bents.
6- The capacity distribution curves and reliability charts developed for the first time in the present study can be utilized as useful means to prioritize retrofitting
program of sub-standard columns and assess the expected damage to existing columns in the aftermath of earthquakes.

7- Contrary to standard columns, the combined reliability of sub-standard columns (the reliability that includes the probability of occurrence of a particular earthquake during bridge design life) is sensitive to the earthquake return period.

8- In general, reliability against failure for sub-standard bents increases for earthquakes with shorter return periods and longer lap splices.

9- The reliability of sub-standard bents in site class BC is higher than those in site class D.

10- Under relatively long return periods, variation of reliability for different steel ratios and damage states diminishes.

11- The combined reliability index against failure ($\beta_1$) for bents failing under all GM records in the database is 1.21, 1.38, 1.74, 2.01, and 2.27 for 60-, 100-, 250-, 500- and 1000-year return period earthquakes, respectively, corresponding to the probability of exceedance of 11.3, 8.4, 4.1, 2.2, and 1.2% against failure.

12- The reliabilities of sub-standard bents are smaller than those of standard columns due to different definition of DI, brittle failure, and the significant effect of earthquake return period on the reliability.