

Nonlinear modeling parameters and acceptance criteria for concrete columns

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Abstract

A database of 490 pseudo-static tests of reinforced concrete columns subjected to load reversals was used to evaluate nonlinear modeling parameters that define the lateral force versus lateral deformation envelope relation of columns under seismic excitations. Based on the modeling parameters, criteria that identify acceptable deformation levels at various performance objectives are proposed. The effects of bi-directional loading and number-of-cycles of the displacement history on the drift ratio at axial failure are discussed, and recommendations are given to account for such effects. Modeling parameters and acceptance criteria are provided in a format that is consistent with provisions of the ASCE 41-06 Standard entitled “Seismic Rehabilitation of Existing Structures”.

Keywords: concrete, columns, nonlinear, modeling parameters, acceptance criteria, bi-directional, cyclic, seismic

1. Introduction

When estimating the performance of existing structures subjected to seismic events, it is often necessary to conduct nonlinear dynamic analyses that require the definition of the lateral force versus lateral deformation relation of frame members. Based on a database comprised of 490 reinforced concrete column tests [1, 2], relations are proposed to define key limiting deformations in the lateral force-deformation relation of reinforced concrete columns subjected to seismic demands. More specifically, relations for evaluating plastic rotations at incipient lateral-strength degradation and incipient axial degradation are proposed. Furthermore, recommendations are provided for selecting acceptable deformations or Acceptance Criteria (AC) below which the performance of reinforced concrete columns is deemed acceptable for selected target performance objectives.

The proposed relations and recommendations are given in a format that is compatible with the standard for seismic rehabilitation ASCE 41-06 Supplement 1 [3, 4]; hereafter referred to as ASCE 41. The standard defines the nonlinear force-deformation backbone relations of concrete columns and other members as illustrated in Fig.1a. In the Figure, the plastic rotation at incipient lateral-strength degradation is given through the Modeling Parameter (MP) a . The plastic rotation in a concrete column at incipient axial degradation is given through the MP b . The residual lateral strength of a column is given by the MP c . MP a and b are given in the current standard as conservative lower-bound estimates of experimental values (Fig. 1b) [5]. To avoid skewing the results of nonlinear simulations, the proposed MP a and b will target median experimental values. AC in the standard were based on fixed percentages of the MP for various performance objectives. Because the error on the estimates of MP exhibit different dispersions for various parameters and members, selecting a fixed fraction of those MP values for AC results in varying probabilities of exceedance for the AC (e.g., 75% of a MP does not always provide the same probabilities of exceedance because that probability depends on the standard

deviation on the MP estimate) . Thus, proposed AC are defined through fixed probabilities of exceedance for various performance objectives and the corresponding fractions of MP values that achieve those probabilities are given.

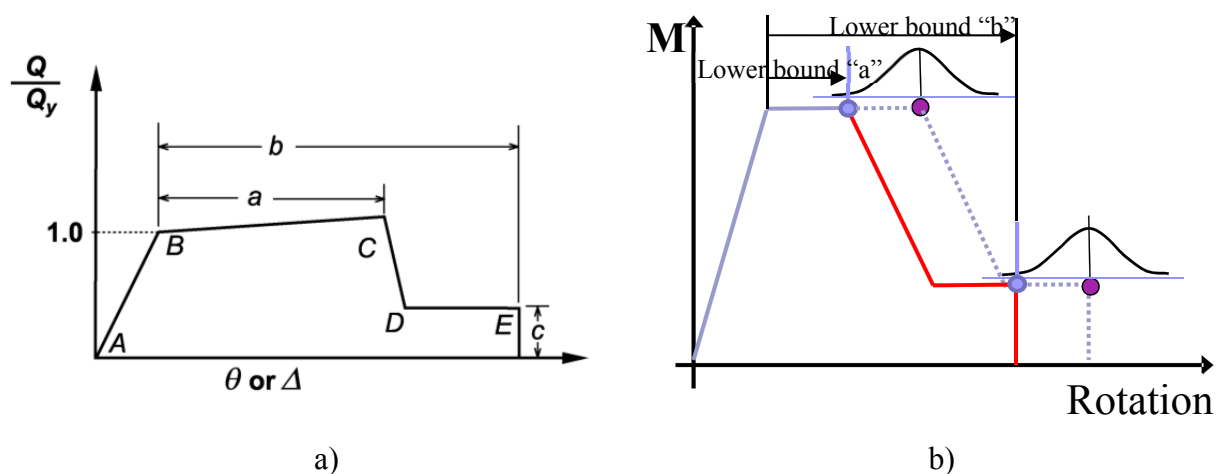


Figure 1: a) ASCE 41-06 backbone response for nonlinear modeling of RC components (from [3]); b) illustration of MP deviation from median values and resulting backbone for modeling RC elements.

2. Column Database

The database used to support the development of the proposed MP and AC contains 319 rectangular column tests and 171 circular column tests for a total of 490 tests [1, 2]. Much of the data was derived from the PEER column database [6]. All tests in the database were conducted quasi-statically. The new database is webcast and accessible to the public ([1, 2]) and additional information about the database can be found in Sivaramakrishnan [7]. The distributions of key parameters of the columns in the data sets are illustrated in the bar charts of Fig. 2. Out of the rectangular columns in the database, 37 can be considered to satisfy the requirements of ACI 318-11 [8] for Special Moment Resisting Frames (SMRF). Out of the circular columns, 24 can be considered to satisfy these requirements. A limited number of rectangular columns (25 out of 171) have reported ties with 90° hooks. The tie hook-angle is unknown for 26 rectangular columns, while 269 rectangular columns have ties with 135° hooks or welded ends. A limited number of circular columns had ties with lapped ends (13 out of 171). Tie details were unknown for one circular column, while the remainder of the circular columns had spirals, ties with welded ends, or hooks anchored into the core. Limited information on hook extensions was available for column tests of the database. Splices and anchorage deficiencies are not within the scope of the work presented, and as such no tests on columns containing anchorage deficiencies were used.

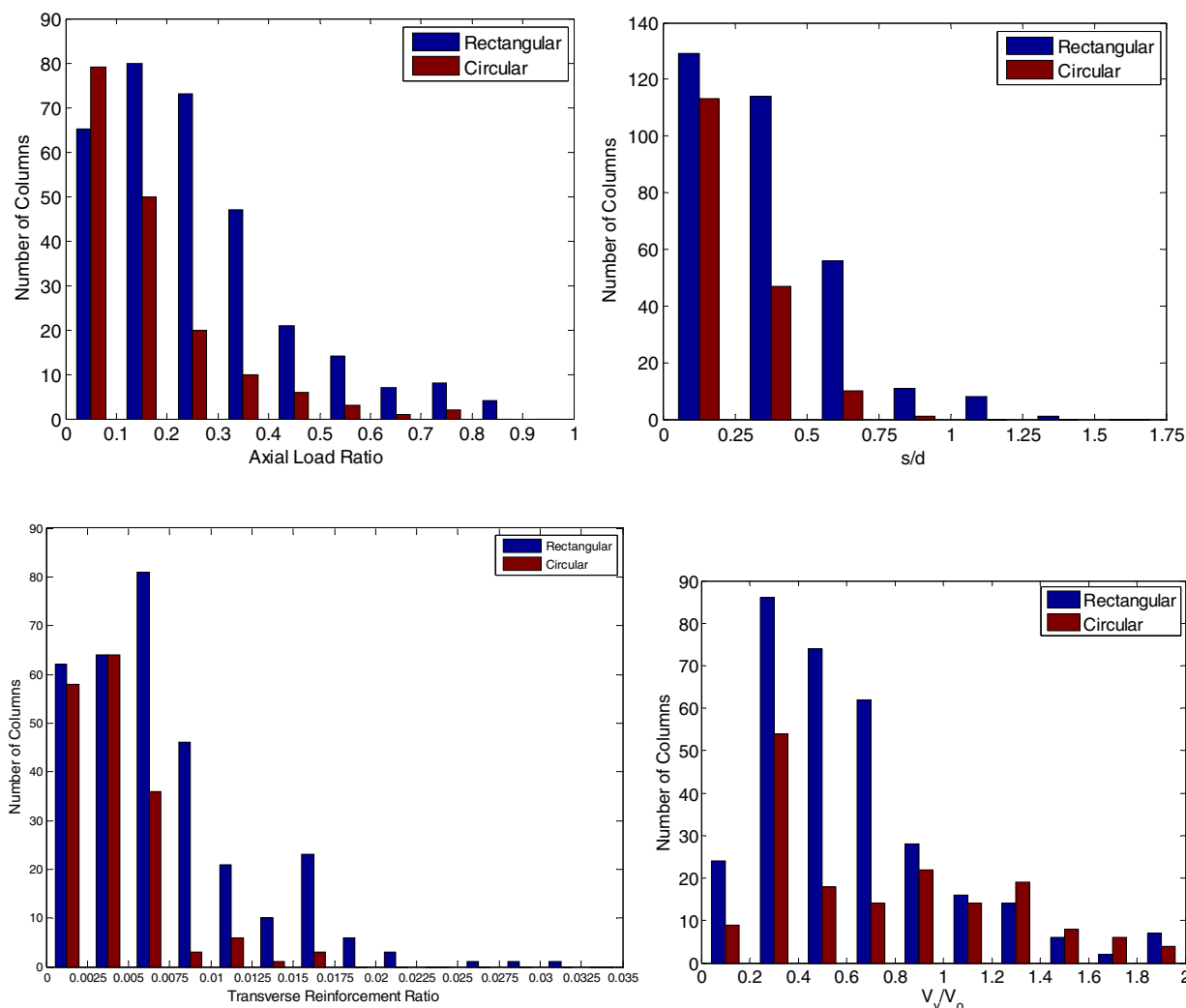


Figure 2: Distribution of key parameters in column database – see *Notation* section for term definitions

3. Data Extraction

The values of a and b were extracted for all column tests. The a values were taken as: $a = (\Delta_{0.8} - \Delta_y)/L$ with $\Delta_{0.8}$ = lateral drift at which the lateral strength of an element degrades by 20% from peak, Δ_y = lateral drift at onset of significant inelastic deformations, and L = column clear length. The drift at onset of inelastic deformation, Δ_y , was obtained as recommended by Sezen and Moehle [9]. A secant line was extended on the lateral force-deformation plot of a column test from the origin to the point on the backbone curve at 70% of the maximum shear ($0.7 V_{max}$). Δ_y was then taken as the drift at the intersection of the secant line with the horizontal line drawn at V_{max} . Δ_y was also evaluated using a secant line passing through $0.6 V_{max}$. Little difference was observed between results from the two intercepts. Results based on a $0.7 V_{max}$ intercept were used in parameter extractions.

The drifts at yield obtained as described above differ from estimates that would be obtained using column stiffness relations provided in the ASCE 41 standard. ASCE 41 stiffness relations were not used for extracting parameters a and b to avoid skewing the extracted plastic rotations by

errors in stiffness estimates inherent in the ASCE 41 stiffness relations [10] (especially for columns with low deformation capacities). The objective of the study was to extract the best estimate of plastic rotations a and b such that the proposed relations defining them would provide median estimates of the plastic rotations and the trends described in those relations would not be artificially altered. The outcome for users of the standard that opt to use ASCE 41 stiffness relations will be a slightly skewed total deformation capacity for elements, but the estimated behavior of the elements would not be skewed (i.e., the degree of inelastic deformations prior to loss of strength estimated by the relations would be the median estimate). Moreover, the proposed relations are intended to be used not only with the ASCE 41 elastic stiffness relations but also with other methods for evaluating stiffness (such as fiber-section models).

Due to the scarcity of column tests conducted to collapse, two sets of b values were produced. The first set originates from column tests that were conducted to axial collapse. If a test was conducted to axial collapse, the plastic rotation at axial failure was taken as $b_1 = (\Delta_{axial} - \Delta_y)/L$; with Δ_{axial} = drift at onset of axial collapse. Only 36 rectangular and 9 circular columns in the database were pushed to axial failure. The webcast database was further bolstered by 12 recent rectangular-column collapse tests [11, 12]. The second set of b values, b_2 , includes the first set but also introduces b values for all other columns in the database based on the following:

- If a test reached a drift at which the lateral strength degraded to 25% of the peak but no axial failure was reported, the plastic rotation at that drift was taken as b_2 . This limit on deformations at axial failure was introduced to mitigate possible errors or omissions in the reporting of axial failure in database tests. Columns that lost 75% of their lateral strength were deemed unstable and close to axial collapse.
- If a column was not tested to collapse or to a deformation causing a reduction in lateral strength to below 25% of the peak lateral strength, the plastic rotation at the largest drift a column was pushed was taken as b_2 .

This methodology provides a lower bound estimate on the parameter b but utilizes all the available column tests. Additional details about the process by which the plastic rotations were extracted from the database can be found in [7].

Values of V_y were extracted from experimental data through analytical means. In all calculations, measured material properties were used. V_y is defined as the shear demand corresponding to the development of moment strength in a column. $V_y = M_y/L_a$ with M_y = column moment capacity, and L_a = column shear span. M_y was calculated using fiber-section analyses that utilized the parabolic stress-strain relation for concrete proposed by Hognestad [13] for concrete in compression. The limiting strain in compression for the concrete was adopted as 0.003. Confinement effects on concrete material properties, as well as the tensile strength of concrete were neglected. An elastic perfectly plastic material model was used for steel fibers and capped at the measured yield stress of longitudinal bars.

ACI 318-11 specifies the use of the ultimate stress of longitudinal bars (approximated as 1.25 the specified yield stress) when evaluating the flexural strength of plastic hinges in flexural members. The increment of 1.25 on yield stress is an arbitrary limit to ensure that an upper bound estimate of the shear demand is used for proportioning the transverse reinforcement. In the study that was performed, measured values of yield stress were used in calculating V_y for following reasons:

- 1) In reinforced concrete columns, longitudinal bars will only reach ultimate stress in well confined columns that are pushed to large inelastic deformations. For columns that sustain shear failure prior to flexural yielding or at relatively low inelastic deformations, evaluating V_y using the yield stress of bars is more appropriate.
- 2) Deriving the modeling parameter and acceptance criteria relations using lower V_y values obtained using the yield stress of bars would provide lower modeling parameters and acceptance criteria should users opt to evaluate V_y using 1.25 the yield stress of longitudinal bars.

The beneficial effects of confinement on flexural strength were not included in calculations of V_y for similar reasons.

V_o was evaluated based on the shear-strength equation of ASCE 41-06 for reinforced concrete columns. The shear strength equation used was:

$$V_o = \Phi \frac{A_v f_{yt} d}{s} + \left(\frac{\alpha \sqrt{f'_c}}{M/Vd} \sqrt{1 + \frac{N_u}{\alpha \sqrt{f'_c} A_g}} \right) 0.8 A_g \quad \text{Eq. 1}$$

where $\alpha = 6$ in psi units and 0.5 in MPa units; N_u = axial compression force (= 0 for tension force); M/Vd is the ratio of moment to shear times the effective depth and was bounded by the values of 4 and 2; d is the effective depth; and A_g is the gross cross-sectional area of the column. For circular columns $d = 0.8D$, where D is the diameter of the column. To account for the inefficacy of transverse reinforcement in resisting shear when spaced beyond 75% of the effective depth of a section, the steel contribution to shear strength (first term in Eq. 1) was modified by a factor of Φ that was taken as 1.0 for $s/d \leq 0.75$, zero for $s/d \geq 1.0$, and linearly interpolated between the two values of s/d .

4. Regression-Based Modeling Parameters (MP)

4.1 Relations

Linear regressions using the most influential parameters were conducted to calculate a median estimate for parameters a and b . In a dataset that has a reasonable spread across parameters, a linear regression plane should intersect the data at around the median (i.e., the regression plane should divide the data into two approximately equal groups, with half the data points above and half the data points below the plane). This was found to be the case for this database. Only minor adjustments on the intercept of the regression equations were necessary to achieve a median fit. Regression equations rather than a tabulated format were selected to avoid the problem of stepping functions at parameter boundaries.

Most rectangular columns in the database contained ties with 135° hooks or welded ties with the welded cases accounting for a small minority of tests. Most circular columns in the database were spirally reinforced. A handful of circular columns had welded or core-anchored ties that are considered to perform similar to spirals. In the following, the “spirally” reinforced column group was considered to contain circular columns reinforced with welded circular ties and those with circular ties adequately anchored in the core.

Regression analyses indicated that spirally reinforced circular columns responded differently to variations in influential parameters than rectangular columns reinforced with rectangular ties. To account for such differences, separate regression equations were developed for spirally reinforced circular columns than for all other columns. The most influential parameters for rectangular and circular columns were: the axial load ratio, the transverse reinforcement ratio (ρ_t), and the ratio V_y/V_o . All three parameters are currently used in the standard ASCE 41 to estimate plastic rotation limits for concrete columns. The maximum shear stress applied to a column section is also used in the standard as a parameter to estimate MP. That parameter was not found to have a determining role in column responses analyzed in this work.

The proposed relations between plastic rotations at incipient loss of lateral strength and axial failure are given below. The subscripts R and C in the equations indicate plastic rotations for rectangular and circular columns, respectively. Only relations for b_2 were derived using linear regression because there are too few b_1 values to provide meaningful regression-based estimates.

Plastic rotation capacities for columns other than spirally reinforced circular columns

$$a_R = 0.042 - 0.043 \frac{P}{A_g f'_c} + 0.63 \rho_t - 0.023 \frac{V_y}{V_o} \geq 0.0 \text{ (rad)} \quad \text{Eq.2}$$

$$b_{2R} = 0.051 - 0.051 \frac{P}{A_g f'_c} + 1.3 \rho_t - 0.023 \frac{V_y}{V_o} \geq a_R \text{ (rad)} \quad \text{Eq.3}$$

Plastic rotation capacities for columns for spirally reinforced circular columns

$$a_C = 0.06 - 0.058 \frac{P}{A_g f'_c} + 1.3 \rho_t - 0.037 \frac{V_y}{V_o} \geq 0.0 \text{ (rad)} \quad \text{Eq.4}$$

$$b_{2C} = 0.064 - 0.07 \frac{P}{A_g f'_c} + 2.85 \rho_t - 0.03 \frac{V_y}{V_o} \geq a_C \text{ (rad)} \quad \text{Eq.5}$$

When evaluating the above parameters, ρ_t should not be taken greater than 0.0175 and V_y/V_o should not be taken smaller than 0.2. The equation is not applicable for $\rho_t \leq 0.0005$. An upper bound on the transverse reinforcement ratio of 0.0175 is prescribed because few columns in the database contained a ratio exceeding that limit. Equations for modeling parameters cannot be used for columns with a transverse reinforcement ratio below 0.0005 because the equations are not intended for unreinforced columns. A lower limit on V_y/V_o of 0.2 is prescribed because few columns in the database had lower values of V_y/V_o .

Values of a and b_2 for rectangular and spirally reinforced column are presented in Table 1 at practical parameter boundaries. In Table 1, the notation is consistent with that used in Eq. 2 to 5 in which the subscript r is used in reference to rectangular columns while the subscript c is used in reference to circular columns.. As can be observed in the table, for a given set of parameters, spirally reinforced columns exhibited significantly larger deformation capacities than rectangular columns.

Table 1: Values of a and b at parameter boundaries

$P/(A_g f'_c)$	ρ_t	V_y/V_o	a_R (rad)	a_C (rad)	b_{2R} (rad)	b_{2C} (rad)
0	0.0005	0.2	0.038	0.058	0.047	0.060
0	0.0005	2.0	0.0	0.0	0.006	0.005
0	0.0175	0.2	0.048*	0.071*	0.069*	0.108*
0	0.0175	2.0	0.007	0.0	0.028	0.054
0.7	0.0005	0.2	0.008	0.016	0.011	0.016
0.7	0.0005	2.0	0.0	0.0	0.0	0.0
0.7	0.0175	0.2	0.018	0.029	0.033	0.059
0.7	0.0175	2.0	0.0	0.0	0.0	0.005

* Maximum permissible values

4.2 Analyses of Fit

4.2.1 Estimates of a

The cumulative distribution of the error difference between experimental and regression estimates of a are plotted for rectangular and circular columns in Fig. 3. Errors are plotted for estimates evaluated using the proposed regression equations as well as for those estimated using the ASCE 41 standard. As can be observed in the figures, the proposed regression equations shifted estimates from conservative ones based on the standard to median estimates; i.e., error = 0 at 0.5 probability of exceedance. For a_R , both methods produced similar spread on the error as evidenced by the similar slopes of the cumulative distribution curves and standard deviations presented in Fig. 3. For a_C however, the proposed relation produced a large shift in estimates from those derived using the standard. This is not surprising given that : 1) the table in the standard defining the a values for reinforced concrete columns was based only on data from rectangular column tests [5], and 2) spirally reinforced circular columns in the data set showed increased deformation capacity over rectangular columns with similar parameters.

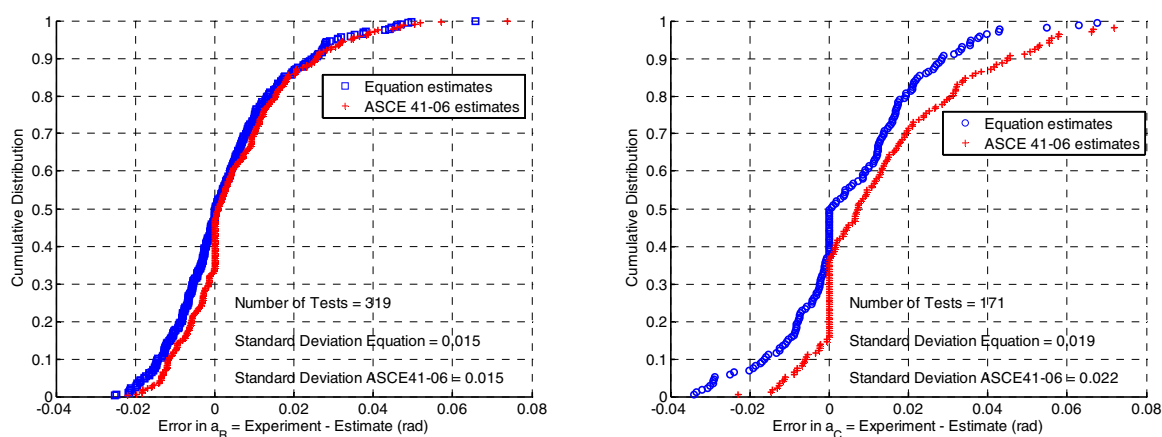


Figure 3: Cumulative distribution of the error between experimental values and estimates for a_R and a_C

The cumulative distribution of the error difference between experimental and regression estimates of a_R are plotted for various bins of rectangular columns in Fig. 4. Test data are divided into bins to illustrate the fit of the proposed relation in various parameter quadrants. As can be observed in the figure, the proposed relation produced estimates that were very close to the median in all bins. Standard deviations on the error of the proposed equation vary from 0.005 to 0.018; which indicates variable accuracy of the proposed equation across bins. Figure 5 presents similar data as Fig. 4 but for circular columns. Similar conclusions can be drawn from Fig. 5 for spirally reinforced circular columns, although the estimates from the proposed equation deviate conservatively from the median estimate for columns with low transverse reinforcement ratio and intermediate values of V_y/V_o (ranging between 0.6 and 1.0). The standard deviations were slightly higher for circular columns than rectangular ones and ranged from 0.005 to 0.029 across bins.

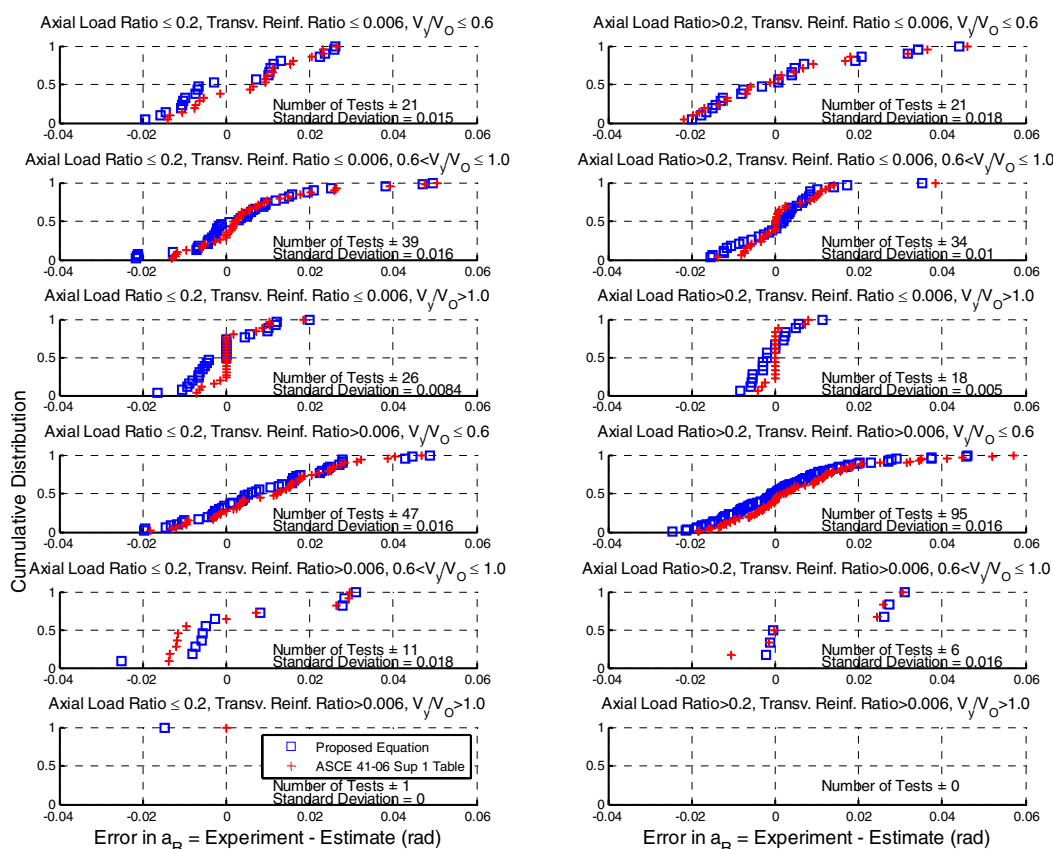


Figure 4: Cumulative distributions of the error between experimental values and estimates for a_R ; data split into bins covering various ranges of parameters and standard deviations given for equation estimates only.

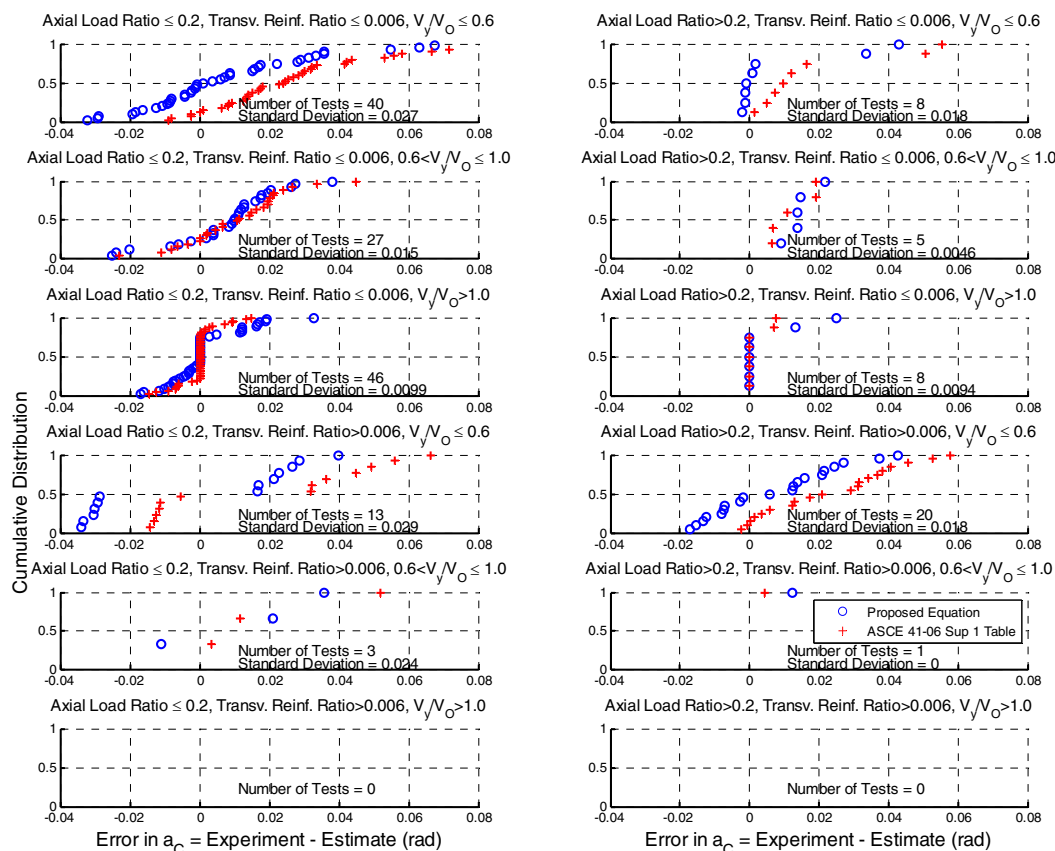


Figure 5: Cumulative distributions of the error between experimental values and estimates for a_C ; data split into bins covering various ranges of parameters and standard deviations given for equation estimates only.

The effects of hook details of transverse reinforcement are explored in Fig. 6. In this figure, the error between experimental values and equation-based estimates of a are plotted versus s/d . Rectangular columns with ties having 135° hooks and 90° hooks are highlighted. Columns with unknown hook details are also indicated. Spirally reinforced circular columns as well as those with lapped ties are highlighted. As can be seen in the figure, there appears to be little overall bias in equation estimates with respect to hook details. In fact, for rectangular columns with 90° hooks the median of the error on the estimate of a_R is 0.005 and the associated standard deviation is 0.011, while for 135° hooks the median of the error on the estimate of a_R is zero and the associated standard deviation is 0.015. Similarly, for circular columns with lapped ties the mean error of the estimate of a_C is -0.002 and its associated standard deviation is 0.011, while for spirally reinforced columns the mean error of the estimate of a_C is 0.006 and its associated standard deviation is 0.019. For large s/d ratios (above 0.75) the proposed equation for rectangular columns provided conservative estimates even for columns with ties having 90° hooks; 90% of all data points in that range have conservative estimates of a_R . One should note that columns in the database with lapped ties or ties having 90° hooks had relatively low transverse reinforcement ratios, ρ_t (as is the case typically in older existing buildings). The median ρ_t for rectangular columns with 90° ties is 0.0017, while that for circular columns with lapped ties is 0.0008. At such low reinforcement ratios, the contribution of ties to the overall

deformation capacity of columns is relatively low, which may explain the observed insensitivity of estimate errors to hook details of ties.

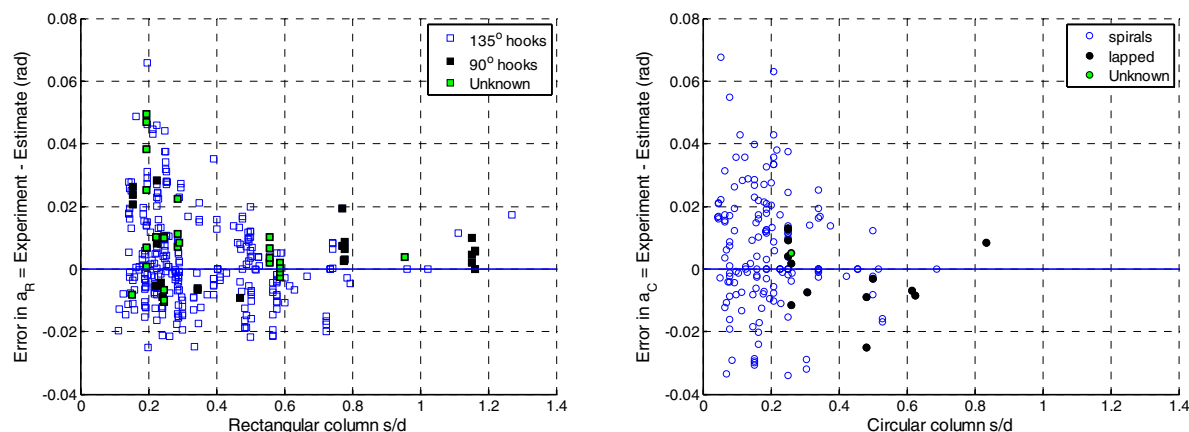


Figure 6: Error between experimental values and equation-based estimates for a plotted versus s/d with transverse reinforcement type highlighted

4.2.2 Estimates of b

The cumulative distribution of the error difference between experimental values and estimates of b_2 are plotted for rectangular and circular columns in Fig. 7. Errors are plotted for estimates evaluated using the proposed regression equations as well as for those estimated using the ASCE 41 standard. The trends observed in Fig. 7 for values of b_2 were similar to those observed for a values. It is apparent from Fig. 7 that the proposed regression equations shifted estimates from conservative ones based on the standard to median estimates; i.e., error = 0 at 0.5 probability of exceedance. For b_{2R} , both methods produced similar spread on estimate error as evidenced by the similar slopes of the cumulative distribution curves and the standard deviations. For b_{2C} however, the proposed relation produced a large shift in estimates from those derived using the standard. For brevity, figures similar to Fig. 4 and Fig. 5 are not presented for b_2 values. Similar trends as those for a values were observed in the various bins for b_2 values.

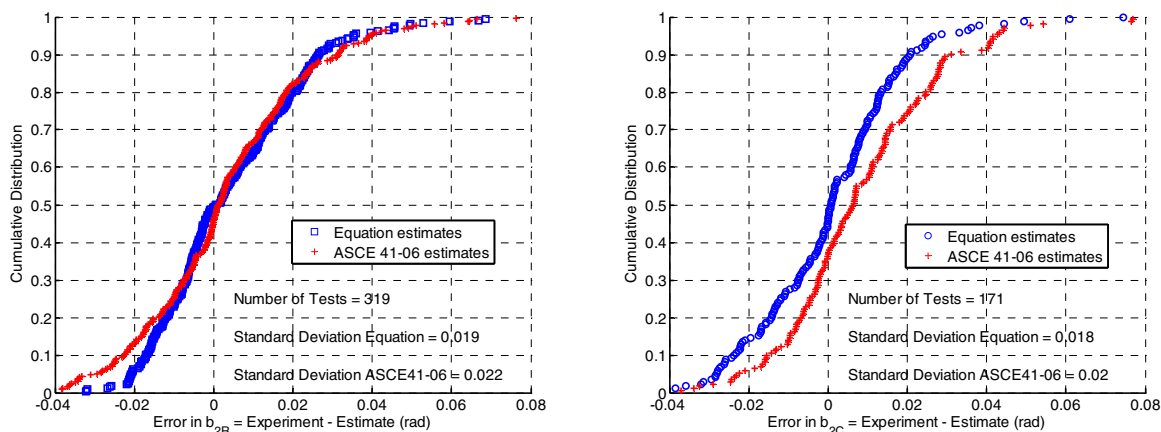


Figure 7: Cumulative distribution of the error between experimental values and estimates for b_{2R} and b_{2C}

In Fig. 8, equation-based estimates of b_2 are plotted versus experimental values. Highlighted in the figures are tests that were conducted to axial collapse. As can be seen in Fig. 8, the equation for b_{2R} provided a conservative estimate of the deformation at axial failure for the columns that were tested to collapse. The equation for b_{2c} on the other hand shows an approximate median estimate for tests conducted to axial collapse.

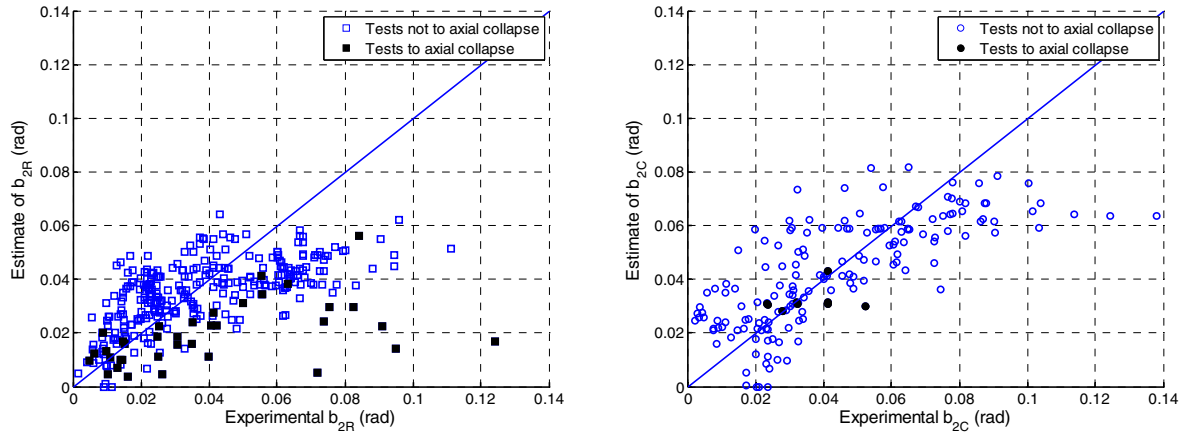


Figure 8: Comparison between experimental and model estimates for plastic rotations at axial collapse b_2

5. Model-Based Modeling Parameter at Onset of Axial Failure b

The limited amount of data makes it difficult to use statistical methods to calibrate an expression to estimate the drift ratio at axial failure of reinforced concrete columns. A behavioral model that estimates the plastic rotation at axial collapse of rectangular columns is proposed instead. The model is mechanics-based and as such is better suited than statistics-based relations for use outside the range of available test data. The proposed behavioral model simplifies the Elwood and Moehle model [14] and is verified using recently produced collapse-test data [12, 15-18] and those available in the database.

The axial failure model by Elwood and Moehle [14] states that the lateral drift ratio at axial failure of a reinforced concrete column is given by:

$$\frac{\Delta}{L} = \frac{4}{100} \frac{1 + (\tan \theta)^2}{\tan \theta + \frac{P}{A_v f_{yt} d_c} \frac{s}{1}} \quad (\text{terms defined in the notations section}) \quad \text{Eq. 6}$$

In Eq. 6, θ is the angle of the critical shear crack from horizontal. Elwood and Moehle recommend adopting a value of θ of 65° . If this value is used, the equation is reduced to:

$$\frac{\Delta}{L} = \frac{0.48}{4.6 + \frac{P}{A_v f_{yt} d_c} \frac{s}{1}} \quad \text{Eq. 7}$$

The previous equation is further simplified to:

$$\frac{\Delta}{L} = \frac{0.5}{5 + \frac{P}{A_v f_{yt} d_c} \frac{s}{1}} \quad \text{Eq. 8}$$

When originally developed, the provisions in the ASCE 41 standard were intended to provide a conservative estimate of the drift ratio at axial failure. The conservative nature of the standard is reflected in the Fig. 9, in which the drift ratio at axial failure calculated following the

provisions in the standard and the measured drift ratio at axial failure for several collapse tests [12, 15-18] are compared. The data set used for the comparison shown in Fig. 9 includes rectangular columns that are both slender and short, columns that are shear-critical, and columns that are flexure-shear critical (i.e., yield in flexure prior to sustaining shear failure).

Figure 9 corroborates that drift ratios at axial failure calculated according to the provisions in ASCE 41 are very conservative for rectangular slender and short columns subjected to uniaxial loading protocols with three cycles per target lateral drift. Contrary to that, estimates of the drift ratio at axial failure for specimens subjected to biaxial loading with three cycles per target drift in each direction or uniaxial loading with six cycles per target drift correspond approximately to the mean, which is consistent with the observation that columns with these types of loading protocols experience axial failure at lower drift ratios than columns subjected to fewer loading cycles [12, 19].

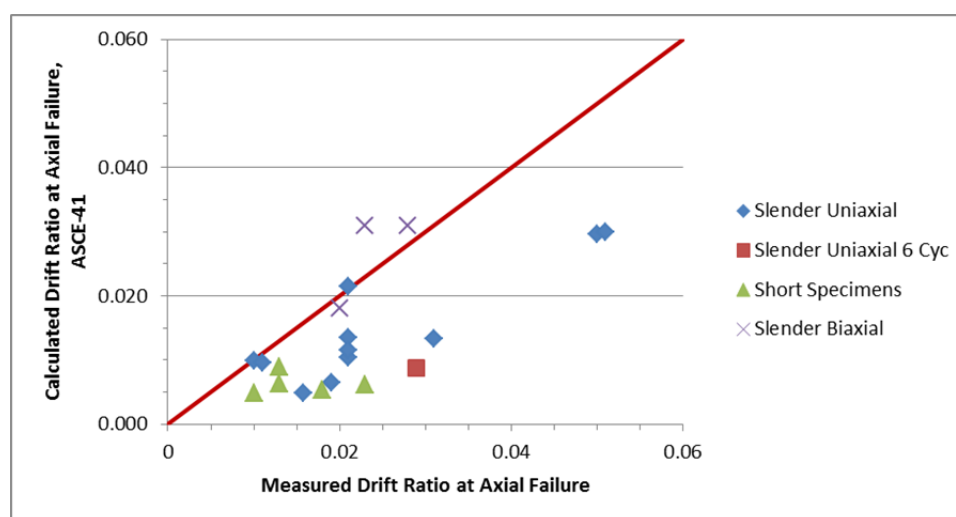


Figure 9: Comparison between measured and calculated drift ratio at axial failure; calculations made using the ASCE 41 provisions. Test data from [12, 15-18].

A similar comparison between experimental drift ratio at axial failure and that calculated using the proposed simplified model for rectangular columns is presented in Fig. 10. A different trend was observed in Fig. 10 than in Fig. 9, whereby calculated drift ratios at axial failure for slender and short columns subjected to uniaxial loading protocols (3 cycles per drift level) corresponded approximately to the mean, while estimates of drift ratio for specimens subjected to biaxial loading and higher number of cycles were unconservative.

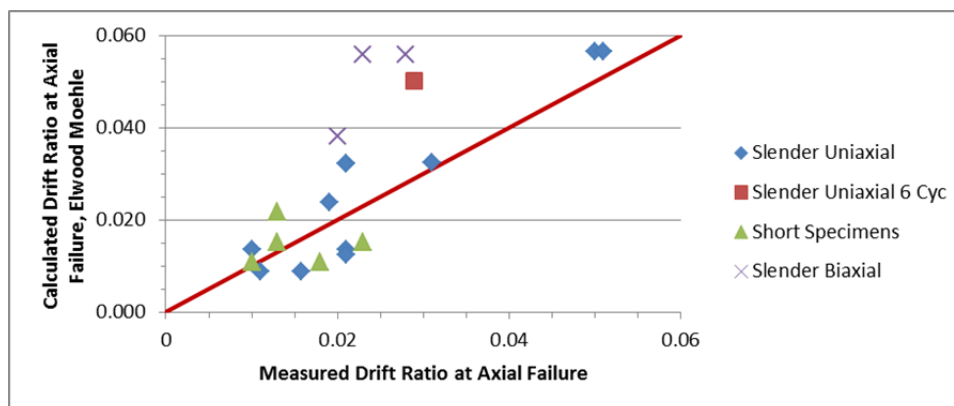


Figure 10: Comparison between measured and calculated drift ratio at axial failure; calculations made using the proposed model. Test data from [12, 15-18].

On the basis of these observations, a reduction factor of 0.5 was applied to drift estimates derived using the proposed model to calculate parameter b for columns subjected to biaxial loading or columns subjected to loading protocols with large numbers of cycles per drift ratio. The results of applying the reduction factor are shown in Fig. 11. As can be seen in the figure, the reduction factor improved the fit of the model drastically. The statistical results for the data set of rectangular columns evaluated are presented in Table 2.

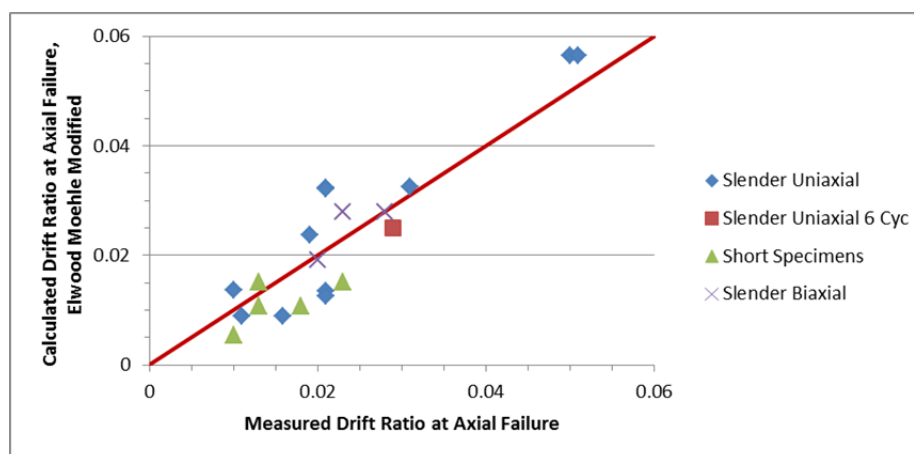


Figure 11: Comparison between measured and calculated drift ratio at axial failure; calculations made using the proposed model with reduction factor for loading protocol. Test data from [12, 15-18].

Table 2: Statistical comparison of measured and calculated drift ratio at axial failure

Model	Average Measured/ Calculated drift ratio at axial failure	Standard Deviation	Coefficient of Variation
ASCE 41-06 Suppl. 1	1.97	0.93	0.47
Elwood-Moehle	0.97	0.44	0.46
Simplified Elwood – Moehle with loading factor	1.13	0.38	0.33

Parameter b_2 values for all rectangular and circular columns in the database are shown in Fig. 12. Estimates of parameter b were obtained using the proposed simplified equation for drift ratio at axial failure. Parameter b was calculated by subtracting the experimentally observed drift ratio at yield Δ_y/L from the drift ratio at axial failure estimated by the model. Tests conducted to axial collapse are highlighted in the figure.

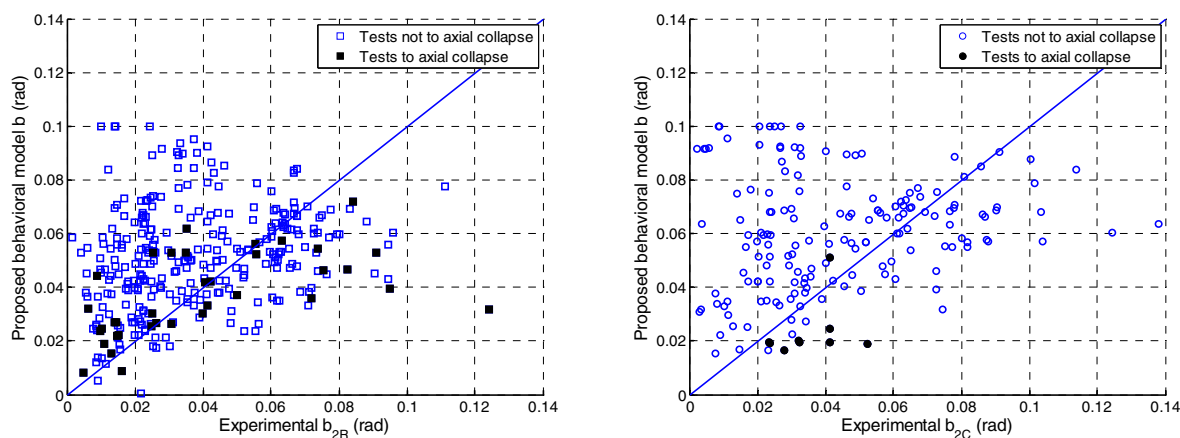


Figure 12: Comparison between experimental and model estimates for plastic rotations at axial collapse; model estimates not adjusted by a factor of 1.3 for circular columns

For rectangular columns, the mean error ratio for $b_1 = 1.07$ (mean experimental b_1 / model estimate), and the corresponding coefficient of variation was 0.71. For circular columns the mean error ratio for b_1 was 1.64 and the corresponding coefficient of variation was 0.57. For a majority of column tests not conducted to collapse (201 rectangular and 117 circular), the model produced higher estimates of the b_2 values (tests to the left of the diagonal lines in Fig. 12). However, given that the experimental b_2 values constitute a lower-bound of the actual plastic rotations at axial failure, it is not possible to conclude whether the model produced conservative or unconservative estimates for those tests. The model produced lower estimates of the b_2 values for many tests that were not conducted to collapse (82 rectangular and 45 circular tests to the right of the diagonal lines in Fig. 12), which indicates conservative estimates of the plastic rotations at axial failure for those tests. Such numbers suggest that the model provided a reasonable estimate of the mean plastic rotations at axial failure for tests conducted to collapse and may not be unconservative over the entire database. For circular columns however, the analysis indicates that the model provided a conservative estimate of the parameter b , particularly considering that the mean error ratio of b_1 values was 1.64. In light of these results, and given the paucity of experimental results to axial failure of circular column, a factor of 1.3 is proposed to increase model estimates for spirally reinforced circular columns. This factor was chosen by comparing the terms of statistically-derived equations for the parameter b_2 of circular and rectangular columns. While the 1.3 factor does generate a mean error ratio still significantly larger than 1.0 ($=1.26$) for b_1 values, it is consistent with the approximate 30% difference between the regression based b_{2R} and b_{2C} estimates. Increasing model estimates by more than 30% was not deemed appropriate given the limited test data (only nine tests on circular columns were conducted to collapse).

6. Proposed Modeling Parameters for Reinforced Concrete Columns

It is proposed that the modeling parameter a for determining the plastic rotation at onset of lateral-strength degradation for reinforced concrete columns be given by equations derived through regression methods. Given the large experimental dataset for parameter a values, statistical methods provide a useful tool for developing expressions for median estimates of that parameter. The alternative to deriving expressions through regressions on influential parameters is to develop a fundamental expression based on mechanics and calibrate it using test data. Several mechanisms however can trigger lateral-strength degradation in reinforced concrete columns such as yielding of the transverse reinforcement crossing a shear crack, or the buckling or fracture of longitudinal bars. Because multiple modes of failure must be considered, a potential mechanics-based model is more complex to develop and calibrate than the simple equations proposed. It is therefore recommended that Eq. 2 and Eq. 4 be used for the evaluation of the plastic rotations at onset of lateral-strength degradation (a) for reinforced concrete columns.

For the plastic rotations at onset of axial collapse (b), the paucity of test data makes difficult the use of statistical methods to estimate that parameter. The accuracy of a mechanics-based model [14] is explored through the available dataset. A simplified version of the model was shown to provide a reasonable estimate of the b parameter for rectangular columns tested to collapse and a conservative estimate for spirally-reinforced circular columns tested to collapse. It is therefore recommended to use the mechanics-based model with the proposed simplifications to estimate the b parameter for all columns with the exception of spirally reinforced circular columns. For the latter type of columns, it is recommended that the b parameter estimated using the mechanics-based model be increased by 30% to account for the observed improved performance over other types of columns. In no case should the a value for a column be taken larger than the b value.

Test results from columns subjected to more than 3 cycles per drift level in one direction or 3 cycles per drift level in two orthogonal directions have been shown to have a drift ratio at axial failure up to 50% lower than observed in similar columns subjected to 3 cycles per drift level in one direction [11, 18]. The loading protocols used in those tests imposed a severe displacement demand on the test columns that is difficult to correlate with displacement histories induced by seismic excitations from various types of earthquake sources. The loading protocols of most tests in the database consisted of three cycles at incrementally increasing lateral drifts and constitute loading protocols that are commonly considered to be severe enough to encompass a large majority of seismic demands [20]. For these reasons, no further adjustments to the proposed relations are introduced in this paper, although engineers are cautioned to reduce modeling parameter b when evaluating the performance of buildings near sources with the potential to generate unusually long duration earthquakes or seismic demands with similar intensities in the fault normal and parallel components.

7. Comparison Between Proposed and ASCE 41 Estimates for Modeling Parameters

In Fig. 13, the difference between a values estimated using the ASCE 41 standard and the proposed equations are illustrated. The figure shows that for the majority of rectangular columns, the proposed Eq. 2 alters estimates from the standard by ± 0.01 rad. For circular columns, the change in estimate is greater and biased towards increasing deformation capacities; which is

expected given the observed higher deformation capacities of circular columns that are not accounted for in the standard.

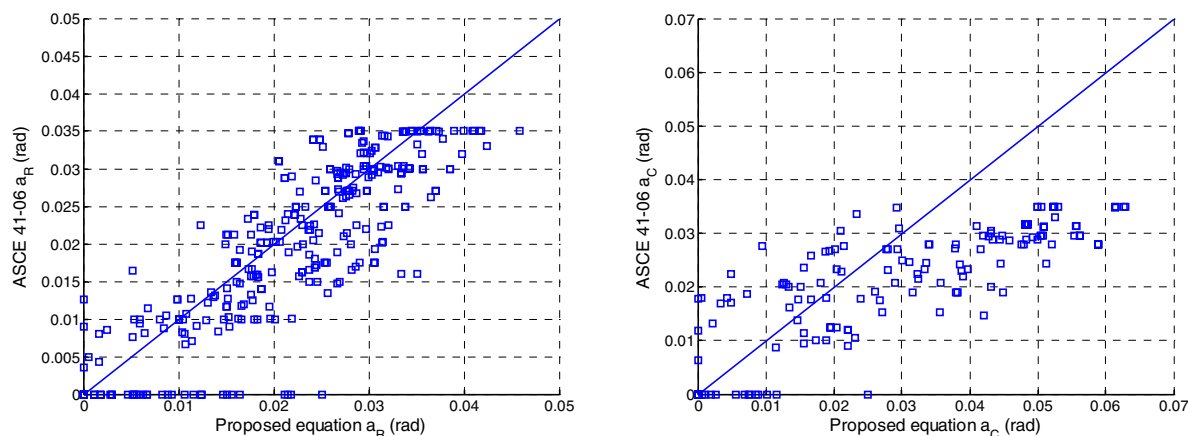


Figure 13: Difference in estimates of a between ASCE41 and the proposed equations

A comparison between b values estimated using of the proposed mechanics-based model and those of the ASCE 41 standard are shown in Fig. 14. For both rectangular and spirally reinforced circular columns, the proposed model estimates increased deformation capacities significantly over standard estimates. This was to be expected because the standard targets conservative deformation capacities while the model targets median estimates.

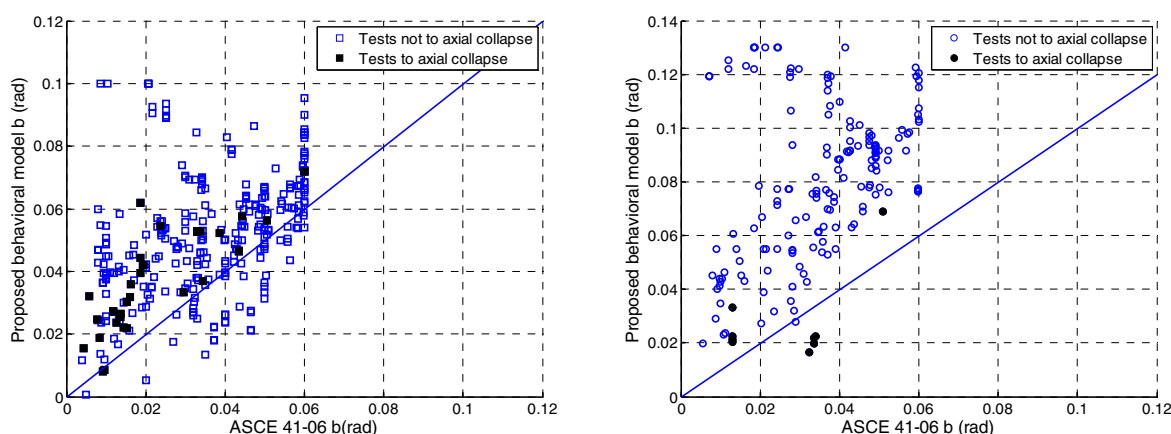


Figure 14: Difference in estimates of b between ASCE 41 and the proposed model (including the 30% adjustment for circular columns)

8. Acceptance Criteria

Acceptance criteria (AC) should reflect the amount of damage that is acceptable for the Immediate Occupancy (IO) performance objective (defined in [3]). For more severe performance objectives related to safety and structural stability, considerations for selecting AC should not only reflect acceptable damage levels but also the probability of exceeding a threshold behavioral state such as a column initiating lateral-strength degradation and exhibiting a negative lateral stiffness. The ASCE 41 standard provides procedures for defining AC based on experimental data. For all members, the standard defines AC for the Immediate Occupancy (IO) performance objective as the critical deformation at which permanent visible damage occurs in a

member. The standard defines AC differently for primary and secondary members at the Life-Safety (LS) and Collapse-Prevention (CP) performance objectives. For primary members, AC are defined as 75% of the MP a for LS and equal to a but not greater than 75% of b for CP. For secondary members, AC are defined as 75% of MP b for LS and equal to b for CP. It is useful to recall here that current MP in the standard were selected conservatively and thus by limiting AC to the MP a or b , the standard in effect aims to limit deformations to below values at which loss of lateral strength or member failure occur.

It is proposed here that AC for IO be based on a percentage of the a value since columns with better detailing and higher deformation capacities tend to exhibit visible damage at larger deformations than less well detailed columns. A conservative value of 10% of the a parameter is selected as the limiting deformation at which a reinforced column is deemed to need repair and no longer satisfy the IO performance objective.

For more severe performance objectives (i.e., Life-Safety (LS) and Collapse-Prevention (CP)), structural stability and safety are of paramount concern. For these performance objectives, ensuring a fixed probability of exceeding the deformation at onset of lateral-strength degradation (a) or the onset of member failure (b) is recommended. Thus for primary structural members, AC should be defined such that the deformation at onset of lateral-strength degradation (MP a) has a low probability of being exceeded. For secondary structural members, AC should be defined so the deformation corresponding to member loss of strength (MP b) has a low probability of being exceeded. Because the errors on the estimates of MP exhibit different dispersions for various parameters, selecting a fixed fraction of those MP values for AC results in different probabilities of exceeding MP deformations for members with deformations not exceeding the AC. Consequently, it is recommended that AC be defined through specific percentiles rather than percentages of MP. Percentiles for selecting AC were selected to remain consistent with the intent of the current ASCE 41 standard definitions for AC (i.e., percentiles were selected to provide percentage values of MP equivalent to those currently defined in the standard). The following summarizes the selected percentiles recommended for the various performance objectives:

- a. For LS of primary members, it is proposed that plastic rotations should not exceed the 20th percentile of a . For a member critical to the stability of a structure, satisfying the AC for LS would indicate a high level of confidence (80%) that the member under consideration has not initiated lateral-strength degradation.
- b. For CP of primary members, it is proposed that plastic rotations should not exceed the 35th percentile of a . The ASCE 41 standard specifies that this AC be equal to the MP a . Given that MP provided in the ASCE 41 standard are conservative estimates of the behavioral milestones rather than median estimates, the AC is given here as lower than the proposed median estimate of that parameter.
- c. For LS of secondary members, it is proposed that plastic rotations should not exceed the 10th percentile of b nor be less than AC for LS of primary members. Due to the more critical nature of the behavioral milestone identified by b , a lower percentile was selected for this AC than for primary members.
- d. For CP of secondary members, plastic rotations should not exceed the 25th percentile of b nor be less than AC for CP of primary members.
- e. In no case should the AC for primary members be larger than those for secondary members.

The percentages of MP a and b that satisfy the selected percentiles for reinforced concrete columns are given in Table 3. As can be seen from the table, the resulting AC are on the order of 30 to 50% of a for LS of primary members, 65 to 75% of a for CP of primary members, 60% of b for LS of secondary members, and 75% of b for CP of secondary members. The range of percentages needed to achieve the same percentiles of MP illustrates the need to move to fixed percentiles and variable percentages to remain consistent with the performance-based philosophy of the ASCE 41 standard.

Table 3: Recommended acceptance criteria

	IO	LS Primary	CP Primary	LS Secondary	CP Secondary
Rectangular	0.1 a_R	0.50 a_R	0.75 a_R	0.60 b_R	0.75 b_R
Circular	0.1 a_C	0.30 a_C	0.65 a_C	0.60 b_C	0.75 b_C

9. Summary and Conclusions

A database of 490 pseudo-static tests of reinforced concrete columns was used to develop relations for nonlinear modeling parameters that define the lateral force-deformation envelope of such columns under seismic excitation. Expressions were developed to estimate the plastic rotation in a concrete column at incipient lateral-strength degradation (Modeling Parameter MP a) and the plastic rotation at incipient axial degradation (MP b). The proposed relations are given in a format that is compatible with the standard for seismic rehabilitation ASCE 41-06 Supplement 1 [3, 4]. Contrary to provisions in the standard, that produce conservative estimates of MP, the proposed equations provide median estimates of MP with the intent to avoid skewing analytical results. Separate relations were derived for spirally reinforced circular columns and all other columns due to observed larger deformation capacities of circular columns.

Regression-based expressions were derived for the MP a and b based test data. Influential parameters for rectangular and circular columns were: the axial load ratio, the transverse reinforcement ratio, and the ratio V_y/V_o . Regression-based relations for a produced median estimates of the parameter and standard deviations on the same order as those of estimates produced using the ASCE 41 standard. The proposed circular-column expression for a produced values significantly higher than those calculated using the ASCE-41 standard because the provisions in the standard were derived based on rectangular column data and do not account for the improved performance of spirally reinforced circular columns.

Due to the scarcity of data on columns tested to axial collapse, the regression-based expressions for the parameter b could only produce a lower-bound estimate of that parameter. An alternative to estimate deformations at axial failure was proposed through a behavioral model. The model is mechanics based and as such is better suited than statistics-based relations for use outside the range of available test data. The model constitutes a simplification of the model by Elwood and Moehle [14]. Model estimates were increased by 30% to account for the higher deformation capacity of spirally reinforced circular columns. The proposed model was shown to produce approximately median estimates of the b parameters for column tests conducted to collapse. Increased numbers of lateral cycles or significant bi-directional excitation have been

shown to reduce the drift at axial collapse by up to 50%. Because the loading protocols that generated such reductions are unusually severe and it is difficult to correlate them to earthquake duration, no further adjustments to the proposed relations were introduced. Engineers are advised to reduce model estimates of drift ratio at axial failure should an unusually demanding displacement history be expected to occur.

Acceptance criteria (AC) were proposed that identify acceptable deformation levels at various performance objectives. Limiting deformations for Life-Safety and Collapse-Prevention performance objectives were defined in terms of the a and b MP such that reaching the AC deformations would correspond to a low probability of exceeding deformations at onset of lateral or axial degradation.

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Notations

a = modeling parameter representing the plastic rotation at incipient lateral-strength degradation; experimentally extracted as $a = (\Delta_{0.8} - \Delta_y)/L$

a_C = modeling parameter a for circular columns

a_R = modeling parameter a for rectangular columns

A_g = concrete gross section area

A_v = area of transverse reinforcement in the direction of loading spaced at s

b = modeling parameter representing the plastic rotation at incipient axial degradation

b_1 = modeling parameter b for column tests conducted to axial collapse = $(\Delta_{axial} - \Delta_y)/L$

b_2 = modeling parameter b for all column tests including those conducted to axial collapse

b_{2C} = modeling parameter b_2 for circular columns

b_{2R} = modeling parameter b for rectangular columns

b_w = section web width

c = modeling parameter representing the residual lateral strength of a column

d = section effective depth from edge of compression region to centroid of tension steel. d is taken as $0.8D$ (D =section diameter) for circular columns

d_c = depth of core (centerline to centerline of ties)

f'_c = concrete compressive strength

f_{yt} = transverse steel yield strength

L = column clear length

L_a = column shear span

M/Vd = ratio of applied moment to applied shear times the effective depth and is bounded by the values of 4 and 2

M_y = column moment strength

N_u = applied axial load used for shear strength evaluation (= 0 for tension loads)

P = axial load (positive in compression)

$P / A_g f'_c$ = axial load ratio

s = transverse reinforcement spacing

V_{max} = maximum applied shear

V_o = nominal shear strength of columns as defined by ASCE 41-06 but with the following modification: the shear strength provided by steel is fully accounted for up to a s/d ratio of 0.75, is taken equal to zero for a s/d ratio above 1.0, and linearly interpolated between full strength and zero strength between $s/d = 0.75$ and $s/d = 1.0$

V_y = shear corresponding to the development of flexural yield moments = M_p / L_a

α = unit adjustment factor = 6 in psi units and 0.5 in MPa units

Δ = drift at axial failure estimated using the proposed mechanics-based model

$\Delta_{0.8}$ = lateral drift at which the lateral strength of an element degrades by 20% from peak

Δ_{axial} = drift at onset of axial collapse

Δ_y = lateral drift at yield of longitudinal bars or at onset of significant inelastic deformations

Φ = factor for adjusting steel shear contribution and is taken as 1.0 for $s/d \leq 0.75$, zero for $s/d \geq 1.0$, and linearly interpolated between the two values of s/d

ρ_t = transverse reinforcement ratio = $A_v/(b_w s)$

θ = angle of critical shear crack from horizontal

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Assessment of ASCE/SEI 41 Concrete Column Provisions using Shaking Table Tests

Y. Li¹, K.J. Elwood², and S.-J. Hwang³

ABSTRACT

A database comprised of 59 reinforced concrete columns subjected to strong ground shaking using earthquake simulators (or shaking tables) is compiled. This paper will focus on insights provided by the database related to the concrete column provisions in ASCE/SEI 41. In particular, the Shaking Table Test Column Database is used to evaluate the accuracy of column effective stiffness models, column classification criteria, and the level of conservatism provided by the plastic rotation capacities specified in ASCE/SEI 41. It is found that the Standard generally overestimates the column effective stiffness, while providing a mean value estimate of the column shear strength regardless of tie spacing. The modeling parameters specified in the standard provide conservative estimate of the column drift capacities and are consistent with the targeted probability of failure. Refinements of the shear strength model and the criteria for column classifications are suggested. This study also compares the measured response of columns subjected to quasi-static cyclic loads and shaking table tests.

Keywords: Shaking table tests; ASCE/SEI 41; reinforced concrete columns; shear failure, axial-load failure

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INTRODUCTION

ASCE/SEI 41 (2013) is commonly used throughout the United States and internationally for the seismic assessment and retrofit of existing concrete buildings. In 2007, the concrete provisions of ASCE/SEI 41 were updated to better reflect the observed performance of concrete components from quasi-static cyclic laboratory tests (Elwood et al. 2007). These updates have been incorporated in the latest release of the seismic rehabilitation standard, ASCE/SEI 41 (2013).

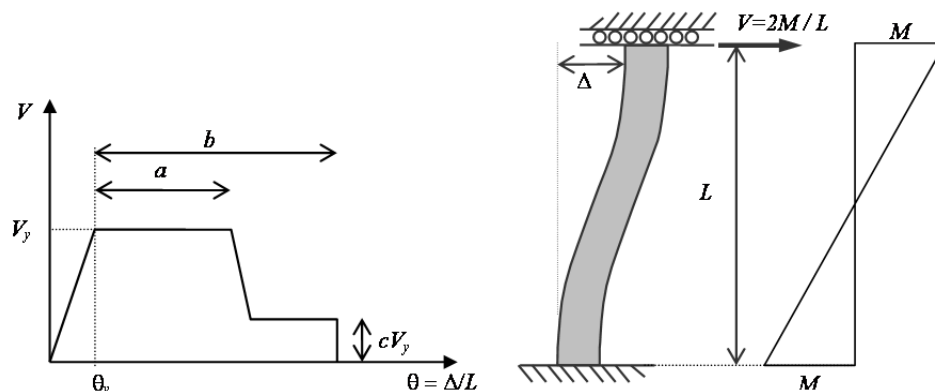


Figure 1 ASCE/SEI 41 Backbone Model

ASCE/SEI 41 recommends generalized load-deformation relation (or backbone) for displacement-controlled components including concrete columns, as shown in Figure 1. According to the ASCE/SEI 41 backbone model, a column is assumed to remain elastic for shear demands below the effective yield strength of the column, V_y , which is taken as the minimum of the plastic shear capacity associated with plastic hinging at both ends of the column, V_p , and the nominal shear strength at low ductility demands, V_0 .

After the rotation demand exceeds the yield rotation θ_y , the column experiences plastic deformations and ASCE/SEI 41 provides the length of the plastic plateau, a , depending on the expected failure mode of the column. For primary components, a also defines the maximum allowable plastic rotation (acceptance criteria) for the Collapse Prevention (CP) performance level. (Note that ASCE/SEI 41-13 no longer provides the nonlinear acceptance criteria for primary components since all components can be considered as secondary when using nonlinear procedures in which any strength degradation is captured in the nonlinear model. The length of the yield plateau, a , is still the basis for the m -factor for primary components used in linear procedures.)

When plastic rotation demands exceed a , the column shear resistance drops to a residual strength of cV_y , and remains at this level of resistance until the ultimate plastic rotation, b , is reached. Quantities b and c are also specified in ASCE/SEI 41 based on the expected failure mode of the column. The ultimate plastic rotation is intended to represent the plastic rotation after which the axial-load carrying capacity of the column should not be relied upon. For secondary components, b also defines the maximum allowable plastic rotation (acceptance criteria) for the Collapse Prevention (CP) performance level.

Examples of other backbone models or acceptance criteria for existing concrete columns include Panagiotakos and Fardis (2001), Elwood and Moehle (2006) and Haselton et al. (2008). Panagiotakos and Fardis (2001), used as the basis for acceptance criteria in Eurocode 8 Part 3 (EN 1998-3 2005), provided empirical equations for estimating the chord rotation at yield and 20% loss of strength. Elwood and Moehle (2006) provided a backbone model for columns experiencing shear failure after flexural yielding, including estimates of the column effective stiffness and drift ratio at shear and axial load failure. Haselton et al. (2008) provided a backbone model for flexure-controlled columns based on a database of 255 quasi-static cyclic column tests.

Due to the prevalence of application in engineering practice, this study will focus on the backbone model and provisions for concrete columns in ASCE/SEI 41. In particular, a database comprised of 59 reinforced concrete columns subjected to strong ground shaking using earthquake simulators (or shaking tables) has been compiled to evaluate the accuracy of column effective stiffness models, the performance of column classification criteria, and the level of conservatism provided by the plastic rotation capacities specified in ASCE/SEI 41. A comparison of the measured response of columns subjected to quasi-static cyclic loads and shaking table tests is also included.

EXPERIMENTAL DATABASE

To assess the ASCE/SEI 41 backbone model, a Shaking Table Test Column Database is compiled (for brevity, this database will be referred to as “Dynamic Database” herein). The Dynamic Database includes results of shaking table tests that have been conducted on simple two-dimensional frames with relatively large-scale reinforced concrete columns (length scale factor no less than 1:3.5), for which critical test data (column axial load, column shear, horizontal drift ratio and column shortening/lengthening) are available. Based on this criteria, seven shaking table test programs were identified, three of which were conducted at UC Berkeley (Elwood 2002, Ghannoum 2007, and Shin 2007) and the remaining four were carried out at National Center for Research on Earthquake Engineering (NCREE) in Taiwan (Su 2007, Kuo 2008, Wu et al. 2006, Yavari 2011). Most of the selected shaking table tests were conducted up to collapse of the frame specimens. Typical test configurations are shown in Figure 2. All specimens were single story frames with the exception of specimens tested by Yavari (2011) and Ghannoum (2007) which were two and three stories, respectively. Details of the shaking table tests are summarised in Li (2012).

Fifty-nine (59) first-story columns out of a total of 97 columns are included in the Dynamic Database (upper story column shear forces and axial loads were not recorded directly and cannot be determined with sufficient reliability to be included in the database). Figure 3 provides the range of column geometry, material properties and reinforcement details for columns in the database; data for each column are provided in Appendix A.

Columns in the Dynamic Database were classified into three types based on the observed failure modes: flexure critical, flexure-shear critical and shear critical columns. The classification criteria based on column properties and recorded response during testing are summarized in Table 1.

Fifteen of the flexure-shear-critical columns were part of a larger multi-story frame and the top of the columns were connected to flexible beams similar to the configuration shown in Figure 2(a). This is relevant to the assessment below since the flexible beam results in additional deformations from beams and joints, contributing to the measured interstory drift. For specimens with rigid beams (e.g. setups similar to Figure 2(b)), the additional deformation from beams and joints can be neglected. All shear-critical columns were connected to rigid beams.

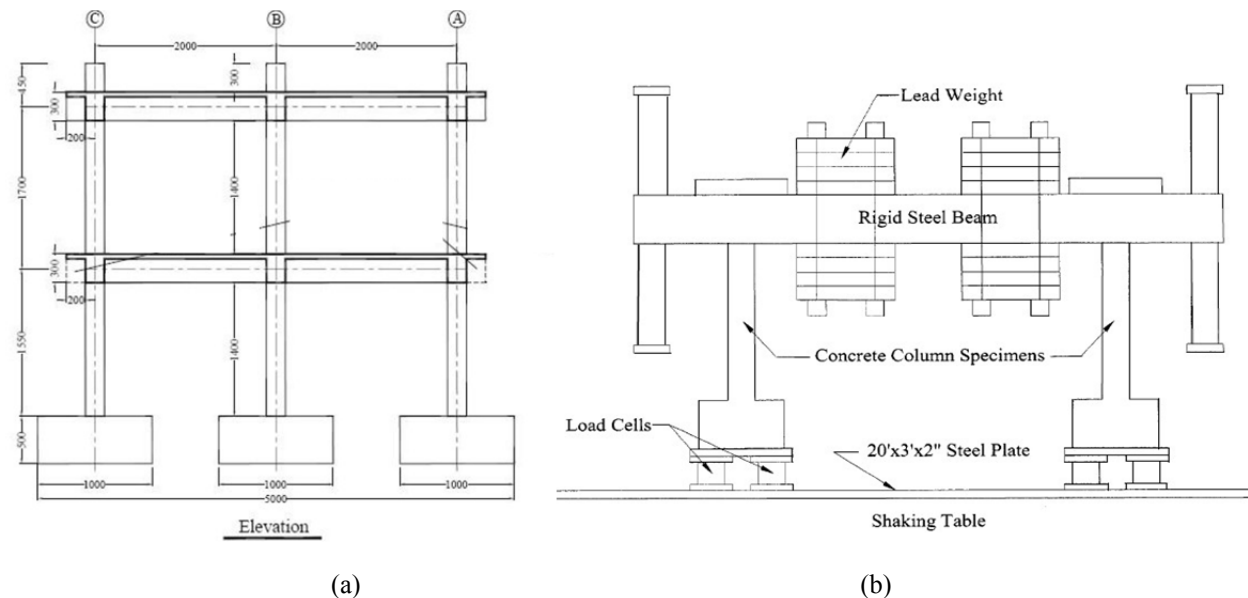
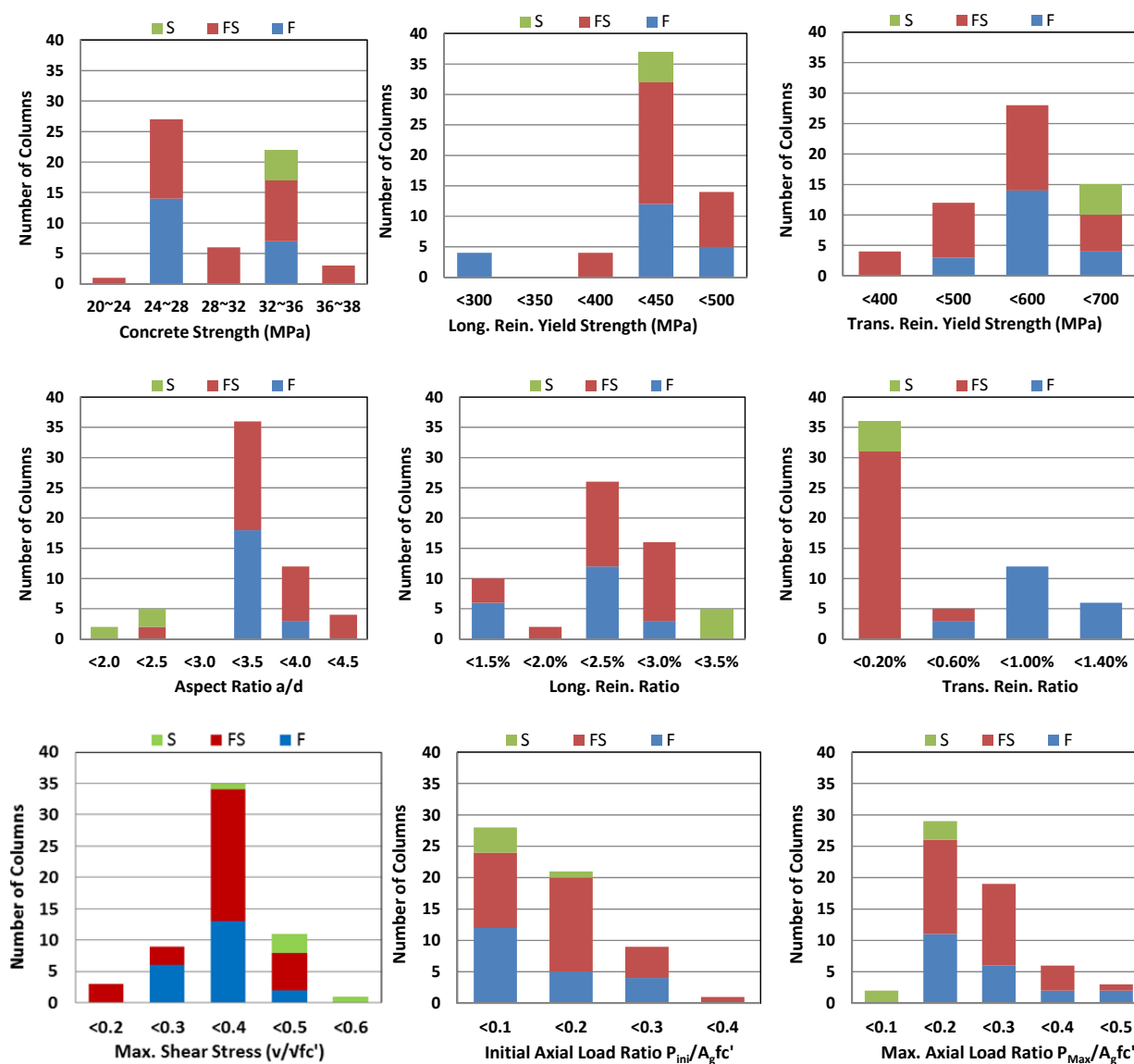


Figure 2 Typical configuration of tests in Dynamic Database (a) columns connected with flexible beams (source: Yavari 2011); (b) columns connected with rigid beam (source: Shin 2007)



Note: F – flexure-critical columns; FS – flexure-shear-critical columns; S – shear-critical columns

Figure 3 Column Properties

Figure 4 shows the normalized envelopes of measured lateral response for columns experiencing shear or flexure-shear failure modes in the Dynamic Database. Column lateral force is corrected for P-Delta effects to give the effective lateral force instead of recorded shear force of each column (unless specified otherwise, the lateral force used throughout this paper is the column shear force corrected for P-Delta effects). The x-axis is the ductility demand, based on the measured yield displacement determined from the maximum force observed during the test and an effective stiffness based on a straight line from the origin through the measured response of the column at 75% of the maximum lateral force. Detailed descriptions of the tests and the development of the envelopes can be found in Li (2012). Despite the fact that all columns in Figure 4 experienced shear failures (before or after flexural yielding), it is noted that all columns achieved ductilities greater than unity and the majority of flexure-shear-critical columns were able to sustain lateral loads beyond a ductility demand of 2.0.

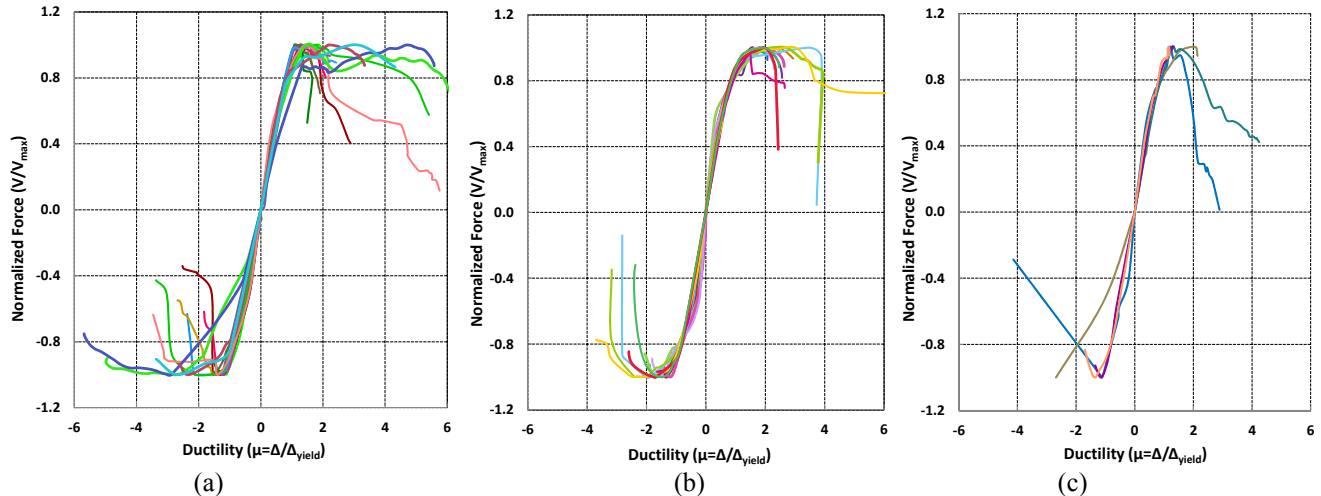


Figure 4 Experimental envelopes for (a) flexure-shear-critical columns connected with rigid beam; (b) flexure-shear-critical columns connected with flexible beam; (c) shear-critical columns.

Table 1 Column classification

	Failure mode		
	Flexure-critical	Flexure-shear critical	Shear-critical
Column Properties	Adequate transverse reinforcement ($\rho_t > 0.002$; 135° hooks)	- Inadequate transverse reinforcement for ductile response ($\rho_t \leq 0.002$; 90° hooks) - Considered to be representative of columns found in older reinforced concrete structures.	- Non-ductile columns with inadequate transverse reinforcement (transverse reinforcement ratio $\rho_t \leq 0.002$; 90° hooks) - Usually have low aspect ratio (i.e. "short column")
Column Response	- Demonstrate extended deformation after yielding - Experience failure associated with flexural damage (e.g., concrete spalling, buckling of longitudinal reinforcement) - Shear failure is unlikely to occur - Have a plastic plateau after initial elastic response in lateral force-deformation relations	- Vulnerable to damage associated with shear failure (e.g., diagonal crack, fracture of transverse reinforcement) - Develop limited inelastic deformation beyond flexural yielding - As shear-failure planes develop, axial load support is generally lost	- Vulnerable to damage associated with shear failure (e.g., diagonal crack, fracture of transverse reinforcement) - Experience sudden shear failure before flexural yielding - Lose gravity load support shortly after shear failure - Develop very limited plastic drift before column failure
Observation	- A total of 21 columns are included in Dynamic Database - No column failed during the test (i.e. no more than 20% of lateral force degradation after P-Δ correction)	- A total of 33 columns are included, of which 27 columns experienced shear failure and axial-load failure. The remaining 6 columns did not fail in tests. - Severe lateral force degradation (loss of more than 20% of its peak resistance) - longitudinal reinforcement buckled at the top or bottom of some columns	- A total of 5 columns with aspect ratio ≤ 2.0 all from NCREE 2007 tests (Su 2007). - All five columns experienced sudden shear failure before yielding of longitudinal reinforcement - Axial-load failure occurred shortly after column shear failure
Examples (Refer to Appendix A)	- NCREE 2009-MUF-A1 - Ghannoum 2006-C1	- NCREE 2009-MCFS-A1 - Elwood 2002-S1-CenC	- NCREE 2007-S1-C2 - NCREE 2007-S3-C1

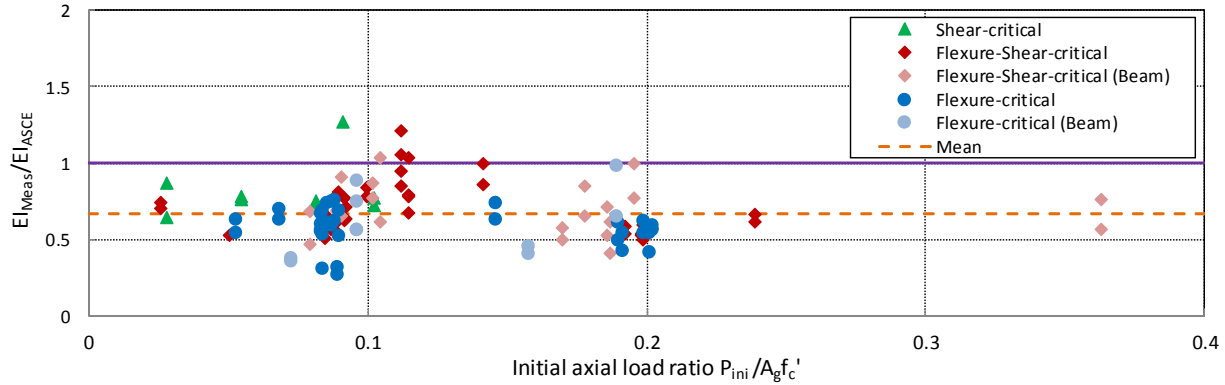


Figure 5 Evaluation of column effective stiffness estimated by ASCE/SEI 41

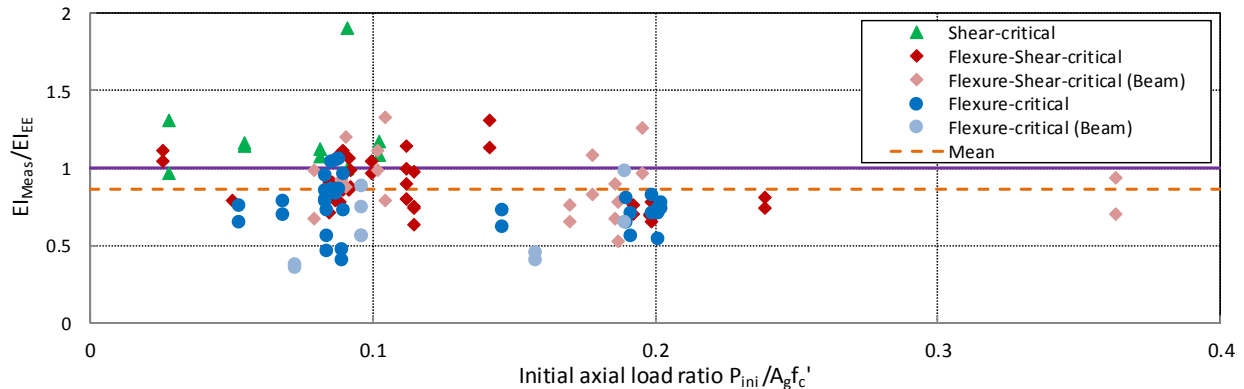


Figure 6 Evaluation of column effective stiffness estimated by Elwood and Eberhard (2009)

EFFECTIVE STIFFNESS

Most columns included in the Dynamic Database could be considered as fixed against rotation at both ends, thus the relationship between measured column lateral stiffness, k_{Meas} , and effective flexural rigidity, EI_{eff} , can be expressed by Equation 1:

$$k_{Meas} = V_y / \theta_y = 12EI_{eff} / L^2 \quad \text{Equation 1}$$

where V_y is the effective yield strength of the column; θ_y is the yield rotation; L is the column clear height.

The effective flexural rigidity (EI_{eff}) is often expressed as a fraction of the gross-section flexural rigidity EI_g . In the interest of using common terminology from engineering practice, the ratio (EI_{eff}/EI_g) will herein be referred to as *effective stiffness ratio*.

In this research, the measured effective stiffness ratio, EI_{Meas}/EI_g , and yield drift for columns in the Dynamic Database are based on a secant stiffness to 75% of the maximum lateral force and corresponding drift on the measured lateral force-drift envelope. According to ASCE/SEI 41, the effective stiffness ratio (EI_{eff}/EI_g) for columns can be estimated as 0.3 for axial load ratios ($P/A_g f'_c$) less than 0.1, and 0.7 for axial load ratios higher than 0.5. Linear interpolation of the effective stiffness ratio is used between these limits.

Results of measured to calculated effective stiffness ratio, EI_{Meas}/EI_{ASCE} , are presented in Figure 5 and grouped by column failure mode. For specimens subjected to multiple ground motions, only the first test is considered here to avoid consideration of stiffness reductions associated with previous testing. Specimens connected to a flexible beam are indicated by a different symbol in Figure 5 (i.e. “Flexure-critical (Beam)” and “Flexure-shear-critical (Beam)”), since such cases are expected to result in a lower bound estimate of the column stiffness due to the additional flexibility of the beam and joint. Statistics are provided in Table 2 for all specimens and just those specimens with rigid beams to remove the additional variability introduced by the beam flexibility.

Based on the test results of 150 columns subjected to quasi-static cyclic loads, Elwood and Eberhard (2009) proposed an effective stiffness model taking into account the deformation due to flexure, shear and bar slip, as shown in Equation 2:

$$0.2 \leq \frac{EI_{EE}}{EI_g} = \frac{0.45 + 2.5 P / A_g f'_c}{1 + 110 \left(\frac{d_b}{D} \right) \left(\frac{D}{a} \right)} \leq 1.0 \quad \text{Equation 2}$$

where P is the column axial load; A_g is the gross cross-section area of the column; f'_c is the concrete compressive strength; d_b is the nominal diameter of longitudinal bars; D is the diameter of circular column or column depth in direction of loading and a is the shear span of the column.

Ratio of measured to calculated effective stiffness EI_{Meas}/EI_{EE} by Elwood and Eberhard (2002) for the Dynamic Database is presented in Figure 6 and Table 2.

ASCE/SEI 41 tends to overestimate the stiffness of nearly all columns regardless of failure mode. Elwood and Eberhard (2009) gives an improved estimation (closer to mean value of the test results) but still generally overestimates the measured stiffness, except for shear critical columns. Since the Elwood and Eberhard model was developed to provide a close estimate of the mean response from quasi-static cyclic tests, these results suggest that columns subjected to real ground motions tend to exhibit a slightly lower effective stiffness compared to quasi-static cyclic tests. This result may be attributed to the large number of below-yield cycles typically imposed on columns during real ground motions; however, further testing would be required to generalize this conclusion.

Table 2 Statistical results of effective stiffness ratio

	Min	Max	Mean	Median	Standard Deviation	Coefficient of Variation(COV)
<i>Flexure-critical columns</i>						
EI_{Meas}/EI_{ASCE}	0.27	0.98	0.58	0.57	0.14	25%
EI_{Meas}/EI_{ASCE} (Rigid beam only)	0.27	0.76	0.58	0.60	0.12	21%
EI_{Meas}/EI_{EE}	0.41	1.24	0.75	0.73	0.20	26%
EI_{Meas}/EI_{EE} (Rigid beam only)	0.41	1.07	0.75	0.74	0.16	21%
<i>Flexure-shear-critical columns</i>						
EI_{Meas}/EI_{ASCE}	0.41	1.21	0.70	0.67	0.17	24%
EI_{Meas}/EI_{ASCE} (Rigid beam only)	0.50	1.21	0.71	0.65	0.17	24%
EI_{Meas}/EI_{EE}	0.52	1.33	0.89	0.87	0.18	20%
EI_{Meas}/EI_{EE} (Rigid beam only)	0.63	1.31	0.88	0.86	0.16	18%
<i>Shear-critical columns</i>						
EI_{Meas}/EI_{ASCE} (Rigid beam only)	0.65	1.27	0.79	0.75	0.18	23%
EI_{Meas}/EI_{EE} (Rigid beam only)	0.97	1.90	1.19	1.13	0.27	22%

COLUMN CLASSIFICATION

To establish the deformation capacity of a concrete column, ASCE/SEI 41 classifies columns into three “conditions”, or failure modes, based on the ratio of the shear strength at low ductility demand, V_o , to the plastic shear demand, V_p . The plastic shear demand, V_p , is the shear force corresponding to flexural yielding of plastic hinges. As shown in Figure 1, for columns with both ends fixed (as assumed for columns in the Dynamic Database), V_p is calculated by dividing the maximum moment strength obtained from section analysis by shear span, $a=L/2$.

$$V_p = 2 M_p / L \quad \text{Equation 3}$$

ASCE/SEI 41 adopts the shear strength model developed by Sezen and Moehle (2004) and recommends calculating column shear strength, V_n , by summing the contribution from concrete, V_c , and transverse reinforcement,

V_s , as shown in Equation 4. The term $V_0 = V_n / k$ is the shear strength excluding the reduction factor related to ductility demand.

$$V_n = kV_0 = k(V_c + V_t)$$

$$V_c = \lambda \left(\frac{0.5\sqrt{f'_c}}{M/Vd} \sqrt{1 + \frac{N_u}{0.5\sqrt{f'_c}A_g}} \right) 0.8A_g$$

$$V_s = \begin{cases} \frac{A_v f_y d}{s} \leftarrow (s/d) \leq 0.5 \\ 0.5 \frac{A_v f_y d}{s} \leftarrow 0.5 < (s/d) \leq 1.0 \\ 0 \leftarrow (s/d) > 1.0 \end{cases} \quad \text{Equation 4}$$

As indicated in Equation 4, ASCE/SEI 41 specifies that if the spacing of column transverse reinforcement is larger than half of the effective depth of column section but less than effective depth ($d/2 < s \leq d$) in the direction of applied shear force, the transverse reinforcement should be taken as no more than 50% effective when resisting shear or torsion, and thus V_s is set to $V_s = 0.5A_v f_y d / s$. If the spacing is larger than the effective depth of the column section ($s > d$), the transverse reinforcement shall be considered ineffective and $V_s = 0$. This modification of V_s is not included in the original model by Sezen and Moehle (2004).

Su (2007) observed that the shear strengths for shear-critical columns in the NCREE 2007shake table tests were higher than for columns subjected to quasi-static test with similar configurations. This difference may be attributed to the higher strain rate and fewer pre-failure cycles that shear-critical columns experienced during the dynamic tests. This observation suggests that the Sezen and Moehle model may be conservative for the columns in the Dynamic Database as this original model was developed based on a database of quasi-static cyclic tests.

Based on the calculated shear strength, V_o , and plastic shear demand, V_p , ASCE/SEI 41 classifies columns with 135-degree hooks on transverse reinforcement into three conditions: condition i, ii and iii. These conditions are intended to correspond to the following failure modes:

- Condition i: flexure failure;
- Condition ii: flexure-shear failure, and;
- Condition iii: shear failure

Table 3 ASCE/SEI 41 column classification

	Transverse Reinforcement Details		
	ACI Conforming details with 135° hooks	Closed hoops with 90° hooks	Other (including lap spliced trans. rein.)
$V_p/V_0 \leq 0.6$	i	ii	ii
$0.6 < V_p/V_0 \leq 1.0$	ii	ii	iii
$V_p/V_0 > 1.0$	iii	iii	iii

Poor transverse reinforcement detailing reduces the column lateral deformation capacity and therefore, columns with transverse reinforcement including 90-degree hooks or having lap spliced transverse reinforcement are adjusted to a poorer condition (e.g., from condition i to condition ii) as shown in Table 3.

The accuracy of the ASCE/SEI 41 classification method is evaluated using the matrix shown in Table 4. Only columns that were observed to experience column failure (i.e. 20% drop in lateral resistance) during the tests are included. The data along the diagonal in the blue cells indicates the number of columns that are classified properly by ASCE/SEI 41. The column data that falls in the green cells provides a “conservative” estimation of the column failure type in terms of column drift capacity (for example “23” indicates that there were 23 columns that are classified as shear-critical columns, yet were observed to have experienced flexure-shear failures during the tests).

While “green zone” classification may result in unnecessary retrofit costs, it is unsafe and hence unacceptable to have column tests falling in the red cells in Table 4. Results in Table 4 show that ASCE/SEI 41 is over-conservative since the 23 of the 27 flexure-shear-critical columns are categorized in Condition iii.

Table 4 Evaluation of ASCE/SEI 41 column classification method

		ASCE/SEI 41 Classification			
		Flexure (Condition i)	Flexure-Shear (Condition ii)	Shear (Condition iii)	Total
Observed Failure Type	Flexure	0	2	0	2
	Flexure-Shear	0	4	23	27
	Shear	0	0	5	5
	Total	0	6	28	34

Table 5 Evaluation of modified ASCE/SEI 41 classification method

		Modified ASCE/SEI 41 Classification			
		Flexure (Condition i)	Flexure-Shear (Condition ii)	Shear (Condition iii)	Total
Observed Failure Type	Flexure	0	2	0	2
	Flexure-Shear	0	27	0	27
	Shear	0	0	5	5
	Total	0	29	5	34

To provide a better match with the observed failure modes from the Dynamic Database, two modifications to the classification methodology in ASCE/SEI 41 are proposed. First, the shear strength, V_o , is not reduced for widely spaced ties ($d > s > d/2$). This is consistent with the development of the shear strength model in Sezen and Moehle (2004) where columns with widely spaced ties are included in the database without modification. Note that the Dynamic Database does not include any columns with $s > d$, hence the impact of the penalty for such large spacing cannot be evaluated here. It is considered prudent to maintain this limit considering the potential for a diagonal crack not crossing any hoops for such wide spacing.

Secondly, it is found that most of the flexure-shear-critical columns that are misclassified as Condition iii have ratio V_p/V_o only slightly over unity. If the criterion for flexure-shear-critical columns is modified to $1.1 \geq V_p/V_o > 0.6$ instead of $1.0 \geq V_p/V_o > 0.6$, the classification improves substantially. This relaxation of the upper limit on the shear ratio for Condition ii reflects the fact that columns with plastic shear demand very close to the shear capacity are expected to exhibit some limited plastic deformation capacity even if post yield deformations are dominated by shear deformations. The relaxation of the upper limit is also consistent with the observation noted above that columns subjected to dynamic loads tend to exhibit slightly higher shear strengths (Su 2007).

As shown in Table 5, by using the unreduced column shear strength and the modified upper limit on V_p/V_o for condition ii, flexure-shear-critical columns could be properly identified, resulting in more accurate modeling parameters and acceptance criteria, and subsequently, a more economical retrofit solution. The modified classification criteria are used to classify columns and evaluate the recommended modeling parameters in the following section.

MODELING PARAMETERS FOR CONCRETE COLUMNS

By classifying columns into the aforementioned three conditions based on failure modes, ASCE/SEI 41 provides recommendations on modeling parameters and allowable plastic rotations for columns in each condition. As shown in Figure 1, the modeling parameters a and b are provided to represent the plastic rotation at significant loss of lateral resistance and loss of axial load resistance, respectively. Plastic Rotation Ratio (PRR), calculated as the ratio of measured plastic rotation to the modeling parameter, is adopted here to assess the accuracy of the ASCE/SEI 41 modeling parameters.

Assessment of Modeling Parameter a : loss of significant lateral resistance

Non-ductile columns with inadequate transverse reinforcement are vulnerable to damage associated with shear failure when subjected to seismic loads. Consistent with the definition of modeling parameter a in ASCE/SEI 41, the column shear failure point is taken as the point that column lateral resistance degrades to 80% of its peak lateral resistance. Most drift capacity models available also have been developed to provide similar estimations of column drift at 80% of maximum effective force (Elwood and Moehle 2005; Kato and Ohnishi 2002, etc.).

The measured plastic rotation capacity was taken equal to the measured drift ratio corresponding to a 20% reduction in the maximum measured shear resistance (θ) minus the calculated yield drift ratio (θ_y) using the recommended effective stiffness from ASCE/SEI 41 and the plastic shear demand, V_p , as defined previously. Hence, the plastic rotation ratio at loss of significant lateral resistance is defined as:

$$\text{Plastic Rotation Ratio (PRR)} = (\theta_i - \theta_y)/a \quad \text{Equation 5}$$

Figure 7 shows the PRR at loss of significant lateral resistance for columns in the Dynamic Database. Data in solid markers provide the PRR for columns that experienced failure (20% loss in lateral resistance) during the test, while the hollow markers represent the lower bound of PRR based on the maximum recorded drifts for columns that did not fail in the shake table test. It is noted that all flexure-critical columns (Condition i) did not experience significant loss of lateral strength during the shaking table tests. The red solid diamond marker (Condition ii) represents the PRR for flexure-shear critical columns that were connected by a rigid beam in the reinforced concrete frame specimens. The PRR of flexure-shear-critical columns that were connected by flexible beam are represented by purple solid diamond marker, namely “Condition ii (Beam)”. In such cases the measured drift ratio is influenced by the flexibility of the beams and joints, indicating that the resulting PRR is likely higher than the true value for the column alone and should be evaluated with caution.

To compare the PRR for columns subjected to dynamic loads and static cyclic loads, 50 non-ductile columns subjected to quasi-static cyclic lateral loads are selected from the PEER Structural Performance Database (Berry et al. 2004) to form a Static Dataset A (detailed column properties could be found in Appendix B). Figure 8 plots the PRR at loss of significant lateral resistance for columns in Static Dataset A.

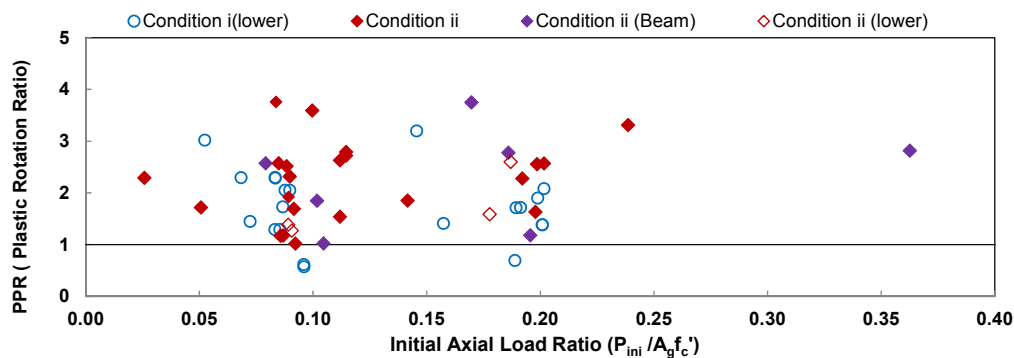


Figure 7 Plastic rotation ratio at significant loss of lateral resistance for Dynamic Database

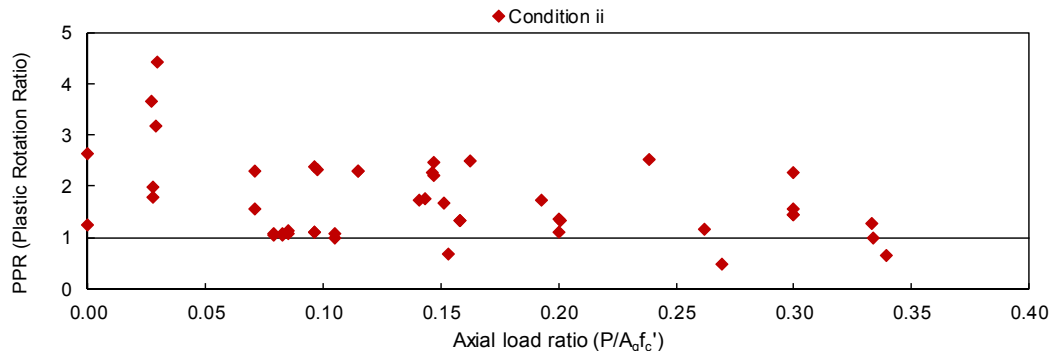


Figure 8 Plastic rotation ratio at significant loss of lateral resistant force for Static Dataset A

Significant scatter exists among the data for PRR in Figure 7 and Figure 8, emphasizing the need to determine the level of safety provided by ASCE/SEI 41 provisions. Results of plastic rotation ratio can be represented by a lognormal cumulative distribution, commonly referred to as a fragility curve. The fragility curve in Figure 9 focuses on the probability of failure for flexure-shear critical columns (i.e. Condition ii). In Figure 9(a), the fragility curve is constructed based on the test data shown in red diamonds in Figure 7 (Condition ii columns connected to rigid beams and observed to experience flexure-shear failures). The fragility curves for flexure-shear critical columns in the Dynamic Database and Static Dataset A are compared in Figure 9 (b).

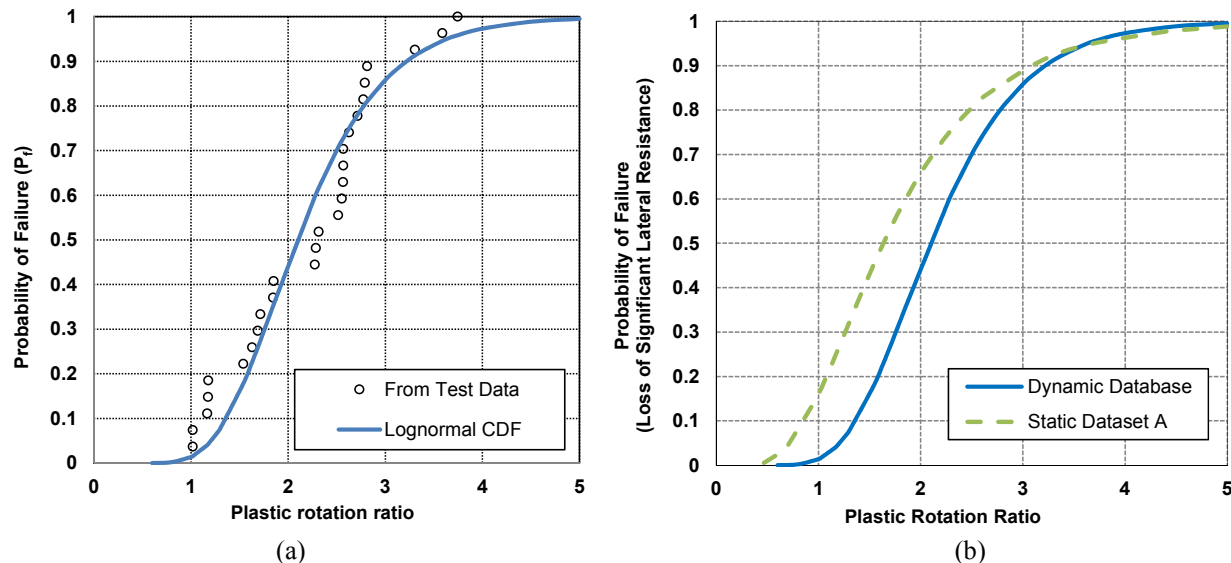


Figure 9 Lognormal fragility curve for plastic rotation at significant loss of lateral resistance in Dynamic Database and Static Dataset A

According to the target safety levels selected in the development of the ASCE/SEI 41 tables (as reported in Elwood et al 2007), at a plastic rotation demand equal to parameter a the probability of failure, P_f , should be less than 15% for columns vulnerable to shear failure, (i.e. columns in condition ii and iii); while for flexure-critical columns in condition i, P_f up to 35% is acceptable since flexure failure is less sudden and brittle. (The “failure” here refers to the point at which the column has lost more than 20% of its lateral resistance.) Based on Figure 9 (a), there is a probability of failure of 3% for “Condition ii” columns in the Dynamic Database when subjected to plastic rotation demand equal to parameter a specified in ASCE/SEI 41 (PRR=1), and thus, is lower than the ASCE/SEI 41 targets. As shown in Figure 9 (b), the probability of failure at PRR =1 for columns in the Static Dataset A is more than four times that of the Dynamic Database, but still consistent with the ASCE/SEI 41 targets.

Most quasi-static cyclic tests are displacement-controlled (column specimens are typically subjected to two or three reversed lateral force-deformation cycles at each horizontal drift ratio level until failure) and commonly include more cycles at high displacement demands than expected from real ground motions used in the shaking table tests. During multiple cycles in quasi-static cyclic tests, diagonal shear cracks open and close repeatedly, increasing the column shear damage and thus resulting in faster degradation of column lateral resistance. This provides a possible explanation for why the fragility curve for columns subjected to shaking table tests shifts to lower probabilities of failure compared to that from quasi-static cyclic tests.

For columns expected to experience shear failure prior to flexural yielding (Condition iii), plastic deformations cannot be relied upon prior to shear failure according to ASCE/SEI 41; and hence, parameter a is set to zero and the drift ratio at lateral load failure is taken as V_0/k_{eff} . Hence, the Total Rotation Ratio (TRR, Equation 6) is used here in place of the Plastic Rotation Ratio to assess the level of conservatism provided by ASCE/SEI 41 for Condition iii columns.

$$\text{Total Rotation Ratio} = \theta_l / (V_0/k_{eff})$$

Equation 6

It is observed that all Condition iii columns exhibited TRR larger than unity, indicating that the ASCE/SEI 41 limits for shear-critical columns are conservative. Since only five shear-critical columns are included in the database, the data are not sufficient to construct a fragility curve or recommend revisions to ASCE/SEI 41.

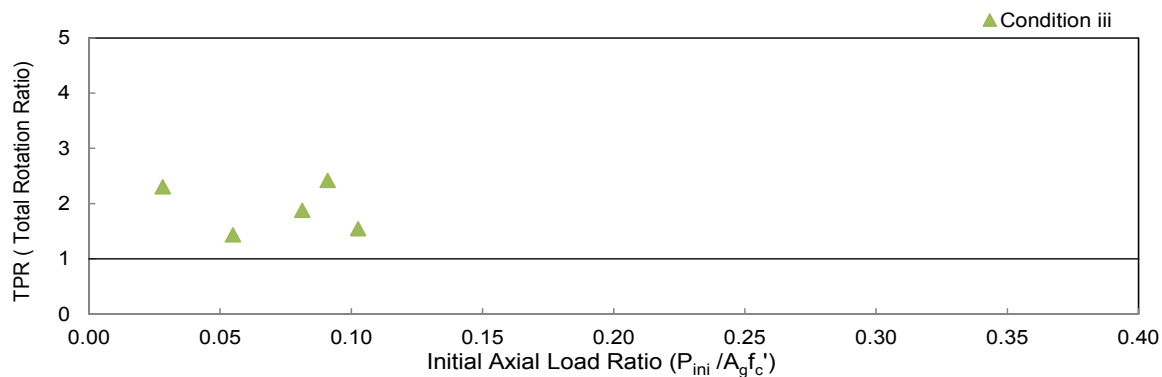


Figure 10 Total Rotation Ratio at significant loss of lateral resistance for Condition iii columns in Dynamic Database

Assessment of Modeling Parameter b : loss of axial-load support

Modeling parameter b in ASCE/SEI 41 represents the plastic rotation at the point that a column experiences axial-load failure and loses axial-load support. While it is intuitive to define the axial-load failure based on the axial load response, load redistribution between columns in frame systems and column elongation/shortening due to imposed lateral deformations can significantly influence the axial load data, rendering it difficult to capture the true point of column axial-load failure from frame tests such as those in the Dynamic Database. For consistency among all tests, this research adopts the method used in Yavari (2011) to define the column axial-load failure point as “the point at which the peak vertical elongation of the column is recorded immediately prior to a sudden shortening of the column”. Yavari (2011) found that this definition resulted in a failure point corresponding to a sudden drop in axial load and visual failure of the column in videos. An example of the application of this definition is shown in Figure 11.

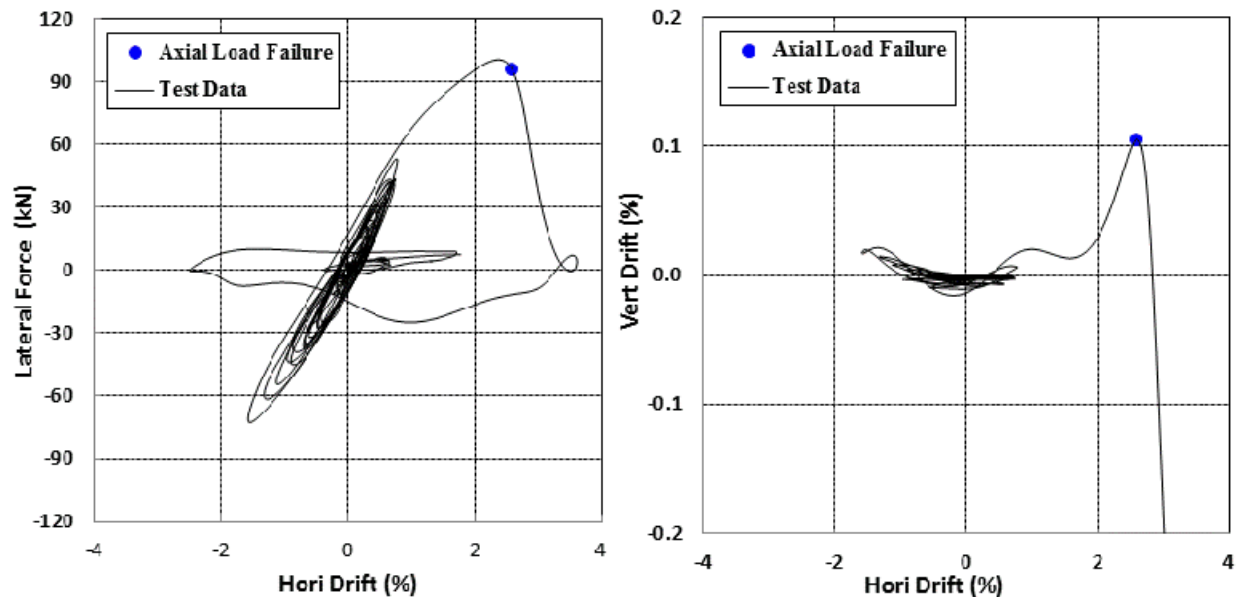


Figure 11 Interpretation of column axial-load failure

The measured plastic rotation capacity at axial-load failure was taken equal to the measured drift ratio at loss of axial-load support (θ_a) minus the calculated yield drift ratio using the recommended effective stiffness from ASCE/SEI 41 and the plastic shear demand, V_p , as defined previously. Hence, the plastic rotation ratio at loss of axial load support is defined as:

$$\text{Plastic Rotation Ratio (PRR)} = (\theta_a - \theta_y)/b \quad \text{Equation 7}$$

To compare the plastic rotation at axial-load failure for columns subjected to quasi-static cyclic tests and shaking table tests, Static Dataset B consisting of 21 Condition ii columns is introduced and summarized in Appendix C. These columns are subjected to unidirectional displacement cycles and constant axial load and experience axial-load failure after shear failure. Static Dataset A could not be used for this assessment since only a small subset of these tests experienced axial load failure. Figure 12 and Figure 13 plot the PRR (Plastic Rotation Ratio) at loss of axial-load support for columns in the Dynamic Database and Static Dataset B, respectively.

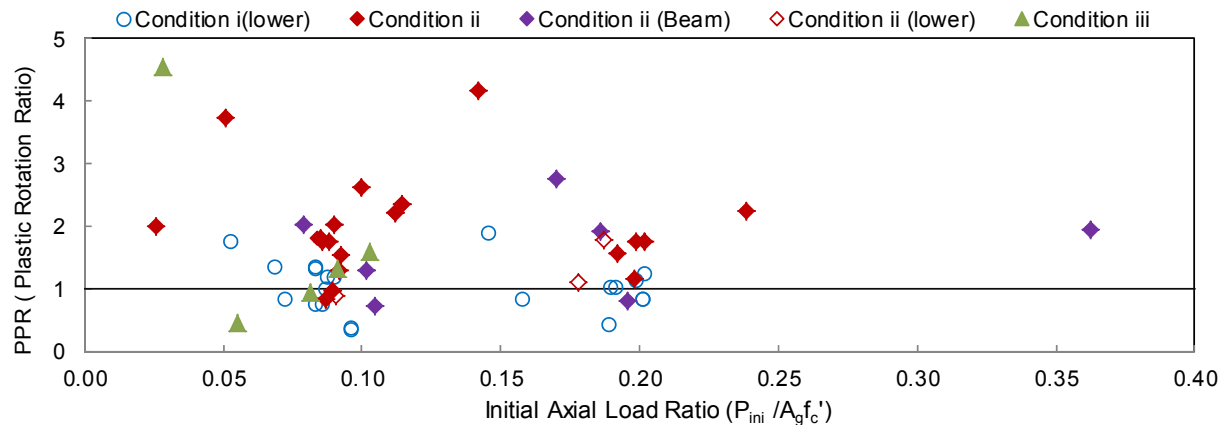


Figure 12 Plastic rotation ratio at loss of axial-load support for Dynamic Database

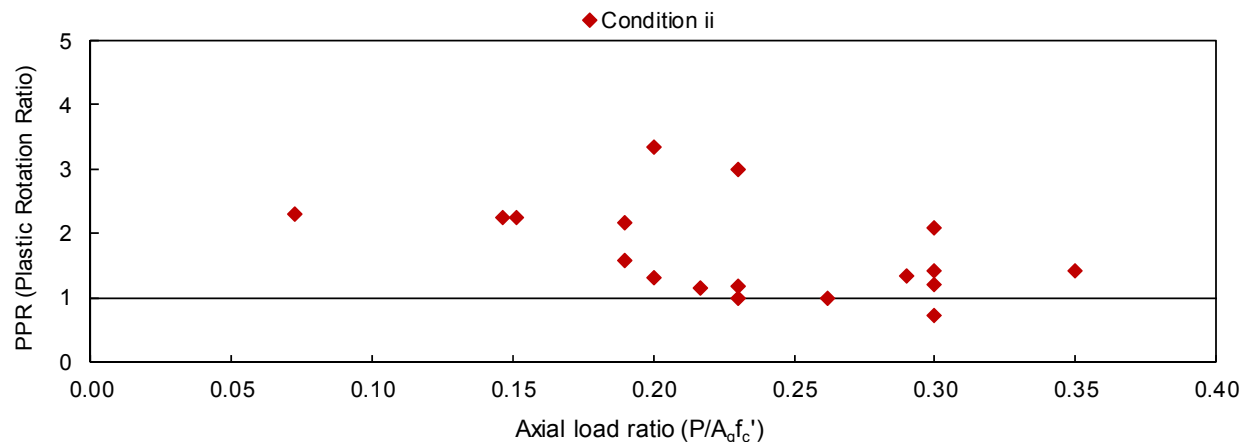


Figure 13 Plastic rotation ratio at loss of axial-load support for Static Dataset B

Similar to the previous section for modeling parameter a , results of plastic rotation ratio for Condition ii columns in Dynamic Database are represented by a fragility curve, as shown in Figure 14 (a). The fragility curve is constructed based on the test data of flexure-shear-critical columns that experienced axial-load failure during the tests (data in red diamond marker in Figure 12).

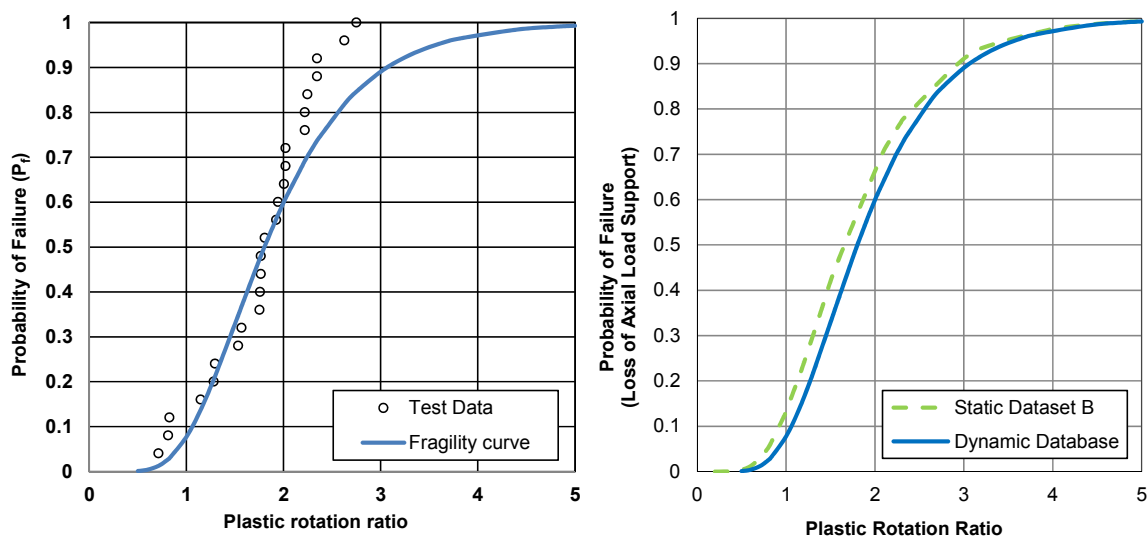


Figure 14 Lognomal fragility curve for plastic rotation at loss of axial load support in Dynamic Database and Static Dataset B

According to the target safety levels selected in the development of the ASCE/SEI 41 tables, when columns experience plastic rotation demand equal to the specified parameter b , the probability of failure, P_f , should be less than 15% (Elwood et al. 2007). (“Failure” here refers to the point that the column loses axial-load support.) Figure 14 (a) shows a 9% probability of failure for $PRR=1.0$, indicating that the probability of failure at loss of axial-load support for flexure-shear-critical columns in the Dynamic Database is consistent with ASCE/SEI 41 targets. The probability of failure for $PRR = 1.0$ for Static Dataset B is slightly higher, at 11%, as shown in Figure 14 (b). The results indicate that the level of conservatism against axial load failure in ASCE/SEI 41 is consistent regardless of the dataset used in the assessment, suggesting that axial load failure is not sensitive to the form of load application (dynamic or static) or the number of cycles.

Considering the conclusions for the drift at lateral load failure (drifts from dynamic tests greater than drifts from static tests) and axial load failure (drifts from dynamic tests similar to drifts from static tests) together, it is noted that the gap between shear failure and axial load failure for columns subjected to shaking table tests is smaller than that for columns in static cyclic tests. Indeed 56% of the flexure-shear-critical columns in the Dynamic Database experience axial-load failure at a drift ratio within $\pm 0.1\%$ of the drift at 20% drop in the lateral resistance. This limited “buffer” between lateral and axial load failure for the Dynamic Database columns may be a result of dynamic amplification of drift demands with the development of a negative stiffness (softening) and a rapid acceleration of response after lateral load failure. Caution should be exercised in interpreting this observation, however, since the specimens in the database included only a limited number of columns and the dynamic response characteristics were heavily influenced by strength degradation in any one column. In a larger building structure, particularly one with walls, failure of one column will not typically result in the same degree of softening and associated dynamic amplification of drift demands.

CONCLUSIONS

Acceptance criteria in codes and standards (e.g. ASCE 41-13) have generally been developed and validated based on comparisons with test data from quasi-static tests. To investigate the influence of real earthquake loading on the performance of concrete columns a database consisting of 59 large-scale reinforced concrete columns tested on a shaking table is compiled. This “Dynamic Database” is used to evaluate the accuracy and conservatism of the concrete column provisions in ASCE/SEI 41-13.

The conclusions of this study include the following:

- ASCE/SEI 41 overestimates the effective stiffness ratio for columns in Dynamic Database. The mean value of EI_{Meas}/EI_{ASCE} for flexural-shear-critical columns connected with rigid beam in Dynamic

Database is 0.65, with coefficient of variation of 24%. This result is consistent with the observation for columns subjected to quasi-static cyclic test.

- Based on the current criteria of ASCE/SEI 41, most of the flexure-shear-critical columns in the Dynamic Database are categorized into Condition iii (pure shear failure). This is too conservative, since Condition iii columns are assumed to have zero plastic rotation capacity prior to shear failure. The following modifications to the ASCE/SEI 41 column classification criteria are recommended: (1) remove the penalty on column shear strength (V_0) for columns transverse reinforcement spaced at greater than $d/2$, while maintaining the penalty ($V_s = 0$) for $s > d$; (2) change the criteria for Condition ii from $0.6 < V_p/V_0 \leq 1.0$ to $0.6 < V_p/V_0 \leq 1.1$.
- Flexure-shear critical columns tend to develop larger plastic drifts at loss of significant lateral resistance in shaking table tests, compared with columns subjected to quasi-static cyclic loads. Based on Condition ii columns in the Dynamic Database, modeling parameter a in ASCE/SEI 41 provided a 3% probability of failure at significant loss of lateral resistance, which is significantly lower than the probability of failure assessed based on columns subjected to quasi-static cyclic loads.
- Flexure-shear critical columns tend to develop similar plastic drifts at loss of axial-load support in shaking table tests, compared with columns subjected to quasi-static cyclic loads. Based on Condition ii columns in the Dynamic Database, modeling parameter b in ASCE/SEI 41 provided a similar probability of failure (9%) at loss of axial-load support compared to that assessed using columns subjected to quasi-static cyclic loads.
- Dynamic tests tend to result in a smaller difference between the drifts at lateral-load and axial-load failure compared with static cyclic tests. For systems where strength deterioration of columns may dominate the dynamic response of the structure, this observation suggests that real earthquakes may result in more rapid degradation of lateral resistance for flexure-shear columns after shear failure than had been previously assumed based on quasi-static cyclic tests.

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APPENDIX A: PROPERTIES OF THE SHAKING TABLE TESTS COLUMN DATABASE

Column Information				Geometry				Long. Rein.				Trans. Rein.					Column Drift*		Failure Mode
	Specimen	Column	Scale	D	b	L	f _c '	d _l	c	f _{yt}	ρ _l	d _t	Tie	s	f _{yt}	ρ _t	Δ _{80%/L}	Δ _{a/L}	
				(mm)	(mm)	(mm)	(MPa)	(mm)	(mm)	(MPa)		(mm)	Type	(mm)	(MPa)	(%)	(%)	(%)	
NCREE 2009 Tests (Yavari 2011)																			
1	MCFS	A1	1/2.25	200	200	1400	34.0	12.7	17	439.0	2.53%	5	r90	120	469.0	0.16%	3.59	3.59	FS
2	MCFS	B1	1/2.25	200	200	1400	34.0	12.7	17	439.0	2.53%	5	r90	120	469.0	0.16%	2.15	2.15	FS
3	MCFS	C1	1/2.25	200	200	1400	34.0	12.7	17	439.0	2.53%	5	r90	120	469.0	0.16%	2.39	2.39	FS
4	HCFS	A1	1/2.25	200	200	1400	34.4	12.7	17	439.0	2.53%	5	r90	120	469.0	0.16%	N/A	3.87	FS
5	HCFS	B1	1/2.25	200	200	1400	34.4	12.7	17	439.0	2.53%	5	r90	120	469.0	0.16%	2.59	2.59	FS
6	HCFS	C1	1/2.25	200	200	1400	34.4	12.7	17	439.0	2.53%	5	r90	120	469.0	0.16%	4.09	4.10	FS
7	MUF	A1	1/2.25	200	200	1400	35.8	12.7	17	467.0	2.53%	5	r135	40	475.0	0.49%	N/A	2.97	F
8	MUF	B1	1/2.25	200	200	1400	35.8	12.7	17	467.0	2.53%	5	r135	40	475.0	0.49%	N/A	2.80	F
9	MUF	C1	1/2.25	200	200	1400	35.8	12.7	17	467.0	2.53%	5	r135	40	475.0	0.49%	N/A	2.82	F
10	MUFS	A1	1/2.25	200	200	1400	36.5	12.7	17	467.0	2.53%	5	r90	120	475.0	0.16%	N/A	2.96	FS
11	MUFS	B1	1/2.25	200	200	1400	36.5	12.7	17	467.0	2.53%	5	r90	120	475.0	0.16%	N/A	2.75	FS
12	MUFS	C1	1/2.25	200	200	1400	36.5	12.7	17	467.0	2.53%	5	r90	120	475.0	0.16%	N/A	2.80	FS
NCREE 2007 Tests (Su 2007)																			
13	S1	C1	1/2	250	250	1000	29.9	12.7	16	436.4	1.62%	4	r135	50	643.3	0.30%	N/A	5.93	FS
14	S1	C2	1/2	250	250	1000	29.9	12.7	16	436.4	3.24%	4	r90	150	643.3	0.07%	1.41	3.35	S
15	S1	C3	1/2	250	250	1000	29.9	12.7	16	436.4	1.62%	4	r135	50	643.3	0.30%	N/A	3.37	FS
16	S3	C1	1/2	250	250	750	29.9	12.7	16	436.4	3.24%	4	r90	150	643.3	0.07%	1.19	1.28	S
17	S3	C2	1/2	250	250	750	29.9	12.7	16	436.4	3.24%	4	r90	150	643.3	0.07%	0.76	1.44	S
18	S4	C1	1/2	250	250	1000	29.9	12.7	16	436.4	3.24%	4	r90	150	643.3	0.07%	0.91	0.91	S
19	S4	C2	1/2	250	250	1000	29.9	12.7	16	436.4	3.24%	4	r90	150	643.3	0.07%	1.22	1.22	S
NCREE 2005 Tests (Kuo 2008)																			
20	P2	C1	1/3	150	150	1000	33.8	9.53	14	470.9	2.53%	3.2	r90	100	548.1	0.16%	4.17	4.98	FS
21	P2	C2	1/3	150	150	1000	33.8	9.53	14	470.9	2.53%	3.2	r90	100	548.1	0.16%	4.22	4.98	FS
22	P2	C3	1/3	150	150	1000	33.8	6	12	231.7	1.39%	5	r135	33	661.9	1.19%	N/A	11.00	F
23	P2	C4	1/3	150	150	1000	33.8	6	12	231.7	1.39%	5	r135	33	661.9	1.19%	N/A	11.00	F
24	L	C1	1/3	150	150	1000	32.3	9.53	14	470.9	2.53%	3.2	r90	100	548.1	0.16%	3.34	8.50	FS
25	L	C2	1/3	150	150	1000	32.3	9.53	14	470.9	2.53%	3.2	r90	100	548.1	0.16%	3.18	8.50	FS

Column Information				Geometry			Long. Rein.					Trans. Rein.					Column Drift*		Failure Mode
	Specimen	Column	Scale	D	b	L	f _c '	d _l	c	f _{yl}	ρ_l	d _t	Tie	s	f _{yt}	ρ_t	$\Delta_{80\%}/L$	Δ_a/L	
				(mm)	(mm)	(mm)	(MPa)	(mm)	(mm)	(MPa)		(mm)	Type	(mm)	(MPa)	(%)	(%)	(%)	
26	L	C3	1/3	150	150	1000	32.3	6	12	231.7	1.39%	5	r135	33	661.9	1.19%	N/A	8.50	F
27	L	C4	1/3	150	150	1000	32.3	6	12	231.7	1.39%	5	r135	33	661.9	1.19%	N/A	8.50	F
NCREE 2004 Tests (Wu et al. 2006)																			
28	S2	C1	1/2	200	200	1730	31.8	9.53	9.5	385.9	1.43%	4	r90	125	385.9	0.10%	2.83	4.31	FS
29	S2	C2	1/2	200	200	1730	31.8	9.53	9.5	385.9	1.43%	4	r90	125	385.9	0.10%	4.30	4.31	FS
30	S3	C1	1/2	200	200	1730	31.8	9.53	9.5	385.9	1.43%	4	r90	125	385.9	0.10%	4.39	4.49	FS
31	S3	C2	1/2	200	200	1730	31.8	9.53	9.5	385.9	1.43%	4	r90	125	385.9	0.10%	4.49	4.49	FS
Ghannoum 2006 Tests (Ghannoum 2007)																			
32	S1	A1	1/3	152	152	991	24.6	9.53	17	441.3	2.45%	3.18	r90	102	655.0	0.15%	4.16	4.66	FS
33	S1	B1	1/3	152	152	991	24.6	9.53	17	441.3	2.45%	3.18	r90	102	655.0	0.15%	4.72	5.11	FS
34	S1	C1	1/3	152	152	991	24.6	6.35	17	482.6	1.09%	4.76	r135	31.8	556.3	1.10%	N/A	5.05	F
35	S1	D1	1/3	152	152	991	24.6	6.35	17	482.6	1.09%	4.76	r135	31.8	556.3	1.10%	N/A	5.58	F
Elwood 2002 Tests (Elwood 2003)																			
36	S1	CenC	1/2	229	229	1473	24.6	12.7	25	479.3	2.48%	4.76	d90	152	689.7	0.18%	5.27	6.07	FS
37	S2	CenC	1/2	229	229	1473	23.9	12.7	25	479.3	2.48%	4.76	d90	152	689.7	0.18%	3.43	3.81	FS
Shin 2005 Tests (Shin 2007)																			
38	Setup I	EastC	1/3	152	152	991	25.6	9.53	12.2	444.1	2.46%	4.76	r135	38.1	549.0	0.92%	N/A	5.39	F
39	Setup I	WestC	1/3	152	152	991	25.6	9.53	12.2	444.1	2.46%	4.76	r135	38.1	549.0	0.92%	N/A	5.39	F
40	Setup I	EastC	1/3	152	152	991	25.6	9.53	12.2	444.1	2.46%	4.76	r135	38.1	549.0	0.92%	N/A	8.07	F
41	Setup I	WestC	1/3	152	152	991	25.6	9.53	12.2	444.1	2.46%	4.76	r135	38.1	549.0	0.92%	N/A	8.07	F
42	Setup I	EastC	1/3	152	152	991	25.6	9.53	12.2	444.1	2.46%	4.76	r135	38.1	549.0	0.92%	N/A	4.84	F
43	Setup I	WestC	1/3	152	152	991	25.6	9.53	12.2	444.1	2.46%	4.76	r135	38.1	549.0	0.92%	N/A	4.84	F
44	Setup I	EastC	1/3	152	152	991	25.6	9.53	12.2	444.1	2.46%	4.76	r135	38.1	549.0	0.92%	N/A	5.95	F
45	Setup I	WestC	1/3	152	152	991	25.6	9.53	12.2	444.1	2.46%	4.76	r135	38.1	549.0	0.92%	N/A	5.95	F
46	Setup II	EastC	1/3	152	152	991	25.6	9.53	12.2	444.1	2.46%	4.76	r135	38.1	549.0	0.92%	N/A	6.94	F
47	Setup II	WestC	1/3	152	152	991	25.6	9.53	13.8	444.1	2.46%	3.17	r90	102	554.5	0.15%	2.46	4.26	FS
48	Setup II	EastC	1/3	152	152	991	25.6	9.53	12.2	444.1	2.46%	4.76	r135	38.1	549.0	0.92%	N/A	8.93	F
49	Setup II	WestC	1/3	152	152	991	25.6	9.53	13.8	444.1	2.46%	3.17	r90	102	554.5	0.15%	4.36	4.36	FS
50	Setup II	EastC	1/3	152	152	991	25.6	9.53	12.2	444.1	2.46%	4.76	r135	38.1	549.0	0.92%	N/A	6.93	F

Assessment of ASCE/SEI 41 Concrete Column Provisions using Shaking Table Tests

Column Information				Geometry			Long. Rein.					Trans. Rein.					Column Drift*		Failure Mode
	Specimen	Column	Scale	D	b	L	f _c '	d _l	c	f _{yl}	ρ_l	d _t	Tie	s	f _{yt}	ρ_t	$\Delta_{80\%}/L$	Δ_a/L	
				(mm)	(mm)	(mm)	(MPa)	(mm)	(mm)	(MPa)		(mm)	Type	(mm)	(MPa)	(%)	(%)	(%)	
51	Setup II	WestC	1/3	152	152	991	25.6	9.53	13.8	444.1	2.46%	3.17	r90	102	554.5	0.15%	3.40	3.40	FS
52	Setup II	EastC	1/3	152	152	991	25.6	9.53	12.2	444.1	2.46%	4.76	r135	38.1	549.0	0.92%	N/A	6.42	F
53	Setup II	WestC	1/3	152	152	991	25.6	9.53	13.8	444.1	2.46%	3.17	r90	102	554.5	0.15%	3.41	3.41	FS
54	Setup III	EastC	1/3	152	152	991	25.6	9.53	13.8	444.1	2.46%	3.17	r90	102	554.5	0.15%	2.47	2.47	FS
55	Setup III	WestC	1/3	152	152	991	25.6	9.53	13.8	444.1	2.46%	3.17	r90	102	554.5	0.15%	2.26	3.83	FS
56	Setup III	EastC	1/3	152	152	991	25.6	9.53	13.8	444.1	2.46%	3.17	r90	102	554.5	0.15%	3.16	3.35	FS
57	Setup III	WestC	1/3	152	152	991	25.6	9.53	13.8	444.1	2.46%	3.17	r90	102	554.5	0.15%	4.28	4.28	FS
58	Setup III	EastC	1/3	152	152	991	25.6	9.53	13.8	444.1	2.46%	3.17	r90	102	554.5	0.15%	3.16	3.16	FS
59	Setup III	WestC	1/3	152	152	991	25.6	9.53	13.8	444.1	2.46%	3.17	r90	102	554.5	0.15%	2.43	2.48	FS

* For flexure-critical columns (Type “F” columns) and flexure-shear-critical columns (Type “FS” columns) that did not experience 20% degradation in lateral resistance during the test, $\Delta_{80\%}/L$ is noted as “N/A” and Δ_a/L is the maximum horizontal drift measured during the tests.

Note: a = the shear span (equals to $L/2$ for fixed-fixed columns); b = width of column section; c = clear cover (measured from the outer face of the concrete to outer face of longitudinal reinforcement); D = height of column section; d_l = the diameter of longitudinal reinforcement; d_t = the diameter of transverse reinforcement; f_c' = concrete compressive stress at test day; f_{yl} = yield stress of longitudinal reinforcement; f_{yt} = yield stress of transverse reinforcement; L = clear height of the column; s = spacing of transverse reinforcement; ρ_l = longitudinal reinforcement ratio; ρ_t = volume transverse reinforcement ratio; Tie Type = the type of the transverse reinforcement, “r135” means rectangular-shaped hoops with 135-degree hooks and “d90” means diamond-shaped transverse reinforcement with 90-degree hooks; s = spacing of transverse reinforcement; $\Delta_{80\%}$ = column displacement at 80% of peak lateral resistance.

APPENDIX B: CONDITION II COLUMNS IN STATIC DATASET A

Specimens	b (mm)	D (mm)	L (mm)	f _c (MPa)	Aspect Ratio (a/D)	ρ_t	f _{yt} (MPa)	s (mm)	ρ_t	f _{yt} (MPa)	P/A _g f _c	$\Delta_{80\%}/L$ ($\times 10^{-2}$)
Bechtoula, Kono, Arai and Watanabe, 2002, D1N30	250	250	625	37.6	2.50	0.0243	461	40	0.0050	485	0.30	3.96
Esaki, 1996 H-2-1/3	200	200	400	23.0	2.00	0.0253	363	40	0.0065	364	0.33	1.99
Esaki, 1996 H-2-1/5	200	200	400	23.0	2.00	0.0253	363	50	0.0052	364	0.20	2.52
Esaki, 1996 HT-2-1/3	200	200	400	20.2	2.00	0.0253	363	60	0.0065	364	0.33	2.49
Esaki, 1996 HT-2-1/5	200	200	400	20.2	2.00	0.0253	363	75	0.0052	364	0.20	2.94
Galeota et al. 1996, AB2	250	250	1140	80.0	4.56	0.0603	579	150	0.0054	579	0.30	4.02
Galeota et al. 1996, AB3	250	250	1140	80.0	4.56	0.0603	579	150	0.0054	579	0.30	3.74
Galeota et al. 1996, AB4	250	250	1140	80.0	4.56	0.0603	579	150	0.0054	579	0.20	4.06
Lynn et al. 1998, 3CMD12	457.2	457.2	1473	27.6	3.22	0.0303	331	305	0.0017	400	0.26	1.74
Nosho et al. 1996, No. 1	279.4	279.4	2134	40.6	7.64	0.0101	407	229	0.0010	351	0.34	1.61
Ohue et al. 1985, 2D16RS	200	200	400	32.0	2.00	0.0201	369	50	0.0048	316	0.14	3.83
Ohue et al. 1985, 4D13RS	200	200	400	29.9	2.00	0.0265	370	50	0.0048	316	0.15	1.78
Pujol 2002, No. 10-1-2.25N	152.4	304.8	686	36.5	2.25	0.0245	453	57	0.0073	411	0.08	3.21
Pujol 2002, No. 10-1-2.25S	152.4	304.8	686	36.5	2.25	0.0245	453	57	0.0073	411	0.08	3.14
Pujol 2002, No. 10-2-2.25N	152.4	304.8	686	34.9	2.25	0.0245	453	57	0.0073	411	0.08	3.21
Pujol 2002, No. 10-2-2.25S	152.4	304.8	686	34.9	2.25	0.0245	453	57	0.0073	411	0.08	3.17
Pujol 2002, No. 10-2-3N	152.4	304.8	686	33.7	2.25	0.0245	453	76	0.0055	411	0.09	3.19
Pujol 2002, No. 10-2-3S	152.4	304.8	686	33.7	2.25	0.0245	453	76	0.0055	411	0.09	3.05
Pujol 2002, No. 10-3-2.25N	152.4	304.8	686	27.4	2.25	0.0245	453	57	0.0073	411	0.10	3.05
Pujol 2002, No. 10-3-2.25S	152.4	304.8	686	27.4	2.25	0.0245	453	57	0.0073	411	0.10	3.22
Pujol 2002, No. 10-3-3N	152.4	304.8	686	29.9	2.25	0.0245	453	76	0.0055	411	0.10	3.13
Pujol 2002, No. 10-3-3S	152.4	304.8	686	29.9	2.25	0.0245	453	76	0.0055	411	0.10	3.15
Pujol 2002, No. 20-3-3N	152.4	304.8	686	36.4	2.25	0.0245	453	76	0.0055	411	0.16	3.33
Pujol 2002, No. 20-3-3S	152.4	304.8	686	36.4	2.25	0.0245	453	76	0.0055	411	0.16	3.36
Ramirez and Jirsa, 1980, 00-U	305	305	458	34.5	1.50	0.0245	374	65	0.0032	455	0.00	3.71
Saatcioglu and Ozcebe 1989, U1	350	350	1000	43.6	2.86	0.0321	430	150	0.0030	470	0.00	4.87
Saatcioglu and Ozcebe 1989, U2	350	350	1000	30.2	2.86	0.0321	453	150	0.0030	470	0.16	4.20
Saatcioglu and Ozcebe 1989, U3	350	350	1000	34.8	2.86	0.0321	430	75	0.0060	470	0.14	5.11
Sezen and Moehle No. 1	457.2	457.2	1473	21.1	3.22	0.0247	434	305	0.0017	476	0.15	2.56
Sezen and Moehle No. 2	457.2	457.2	1473	21.1	3.22	0.0247	434	305	0.0017	476	0.60	1.33

Assessment of ASCE/SEI 41 Concrete Column Provisions using Shaking Table Tests

Specimens	b (mm)	D (mm)	L (mm)	f'_c (MPa)	Aspect Ratio (a/D)	ρ_l	f_{yl} (MPa)	s (mm)	ρ_t	f_{yt} (MPa)	$P/A_g f'_c$	$\Delta_{80\%}/L$ ($\times 10^{-2}$)
Sezen and Moehle No. 4	457.2	457.2	1473	21.8	3.22	0.0247	434	305	0.0017	476	0.15	3.49
Soesianawati et al. 1986, No. 4	400	400	1600	40.0	4.00	0.0151	446	94	0.0030	255	0.30	2.74
Takemura and Kawashima, 1997, Test 1 (JSCE-4)	400	400	1245	35.9	3.11	0.0158	363	70	0.0020	368	0.03	3.51
Takemura and Kawashima, 1997, Test 2 (JSCE-5)	400	400	1245	35.7	3.11	0.0158	363	70	0.0020	368	0.03	3.90
Takemura and Kawashima, 1997, Test 3 (JSCE-6)	400	400	1245	34.3	3.11	0.0158	363	70	0.0020	368	0.03	5.96
Takemura and Kawashima, 1997, Test 4 (JSCE-7)	400	400	1245	33.2	3.11	0.0158	363	70	0.0020	368	0.03	8.15
Takemura and Kawashima, 1997, Test 5 (JSCE-8)	400	400	1245	36.8	3.11	0.0158	363	70	0.0020	368	0.03	6.79
Umehara and Jirsa 1982, 2CUS	230	410	455	42.0	1.11	0.0301	441	89	0.0055	414	0.27	1.04
Wehbe et al. 1998, A1	380	610	2335	27.2	3.83	0.0222	448	110	0.0027	428	0.10	5.23
Wehbe et al. 1998, A2	380	610	2335	27.2	3.83	0.0222	448	110	0.0027	428	0.24	4.38
Wight and Sozen 1973, No. 00.033(East)	152.4	304.8	876	32.0	2.88	0.0245	496	127	0.0032	345	0.00	3.00
Wight and Sozen 1973, No. 00.033(West)	152.4	304.8	876	32.0	2.88	0.0245	496	127	0.0032	345	0.00	5.50
Wight and Sozen 1973, No. 25.033(East)	152.4	304.8	876	33.6	2.88	0.0245	496	127	0.0032	345	0.07	3.59
Wight and Sozen 1973, No. 25.033(West)	152.4	304.8	876	33.6	2.88	0.0245	496	127	0.0032	345	0.07	4.80
Wight and Sozen 1973, No. 40.048(East)	152.4	304.8	876	26.1	2.88	0.0245	496	89	0.0046	345	0.15	4.88
Wight and Sozen 1973, No. 40.048(West)	152.4	304.8	876	26.1	2.88	0.0245	496	89	0.0046	345	0.15	5.49
Wight and Sozen 1973, No. 40.067(East)	152.4	304.8	876	33.4	2.88	0.0245	496	64	0.0064	345	0.11	6.85
Wight and Sozen 1973, No. 40.067(West)	152.4	304.8	876	33.4	2.88	0.0245	496	64	0.0064	345	0.11	6.88
Xiao and Martirosyan 1998, HC4-8L16-T6-0.1P	254	254	508	86.0	2.00	0.0246	510	51	0.0075	449	0.10	6.40
Xiao and Martirosyan 1998, HC4-8L16-T6-0.2P	254	254	508	86.0	2.00	0.0246	510	51	0.0075	449	0.19	4.26

Note: a = the shear span (equals to $L/2$ for fixed-fixed columns); b = width of column section; D = height of column section; f'_c = concrete compressive stress at test day; f_{yl} = yield stress of longitudinal reinforcement; f_{yt} = yield stress of transverse reinforcement; L = clear height of the column; s = spacing of transverse reinforcement; ρ_l = longitudinal reinforcement ratio; ρ_t = volume transverse reinforcement ratio; P = column axial load; A_g = area of column cross section; $\Delta_{80\%}$ = column displacement at 80% of peak lateral resistance.

APPENDIX C: CONDITION II COLUMNS IN STATIC DATASET B

Specimen Name	b (mm)	D (mm)	L (mm)	(a/D)	f _c (MPa)	ρ _l	f _{yl} (Mpa)	s (mm)	ρ _t	f _{yt} (MPa)	P/(f _c A _g)	Δ _a /L (×10 ⁻²)
Kato et al. 2000, No.2	350	350	875	1.3	40.5	0.0328	404	45	0.01016	309	0.65	2.00
Kato et al. 2002, LC	250	250	1000	2.0	27.9	0.0127	362	60	0.00420	390	0.43	2.40
Sezen 2002, 2CHD12	457.2	457.2	2946	3.2	21.1	0.0247	434.4	304.8	0.00176	476	0.61	1.90
Kabeyasawa et al. 2002, A-1	420	150	1260	1.5	18.3	0.0100	349	200	0.00133	289	0.29	1.48
Ousalem et al.2004, C4	300	300	900	1.5	13.5	0.0169	340	75	0.00280	384	0.30	4.03
Ousalem et al.2004, C8	300	300	900	1.5	18.0	0.0169	340	75	0.00280	384	0.30	1.52
Ousalem et al.2004, C12	300	300	900	1.5	18.0	0.0169	340	75	0.00280	384	0.20	8.07
Ousalem et al.2004, D16	300	300	600	1.0	26.1	0.0169	447	50	0.00420	398	0.23	4.00
Ousalem et al.2004, D1	300	300	600	1.0	27.7	0.0169	447	50	0.00420	398	0.22	4.06
Ousalem et al.2004, D13	300	300	900	1.5	26.1	0.0226	447	50	0.00420	398	0.23	3.50
Ousalem et al.2004, D14	300	300	900	1.5	26.1	0.0226	447	50	0.00420	398	0.23	10.06
Ousalem et al.2004, C1	300	300	900	1.5	13.5	0.0169	340	160	0.00083	587	0.30	1.01
Sezen 2002, 2CLD12	457.2	457.2	2946	3.2	21.1	0.0247	434.4	304.8	0.00176	476	0.15	4.99
Sezen 2002, 2CLD12M	457.2	457.2	2946	3.2	21.8	0.0247	434.4	304.8	0.00176	476	0.15	5.09
Lynn et al. 1992, 3CMD12	457.2	457.2	2946	3.2	27.6	0.0312	331	304.8	0.00176	400	0.26	2.07
Yoshimura et al. 2002, 2C13	300	300	600	1.0	25.2	0.0169	350	100	0.00210	392	0.19	3.00
Yoshimura et al. 2002, 2M13	300	300	600	1.0	25.2	0.0169	350	100	0.00210	392	0.19	4.10
Lynn et al. 1992, 2SLH18	457.2	457.2	2946	3.2	33.1	0.0194	331	457.2	0.00068	400	0.07	3.63
Yoshimura et al. 2003, No.7	300	300	1200	2.0	30.7	0.0169	409	150	0.00140	392	0.20	2.00
Yoshimura et al. 2003, No.5	300	300	1200	2.0	30.7	0.0268	402	100	0.00210	392	0.35	2.00
Yoshimura et al. 2003, No.4	300	300	1200	2.0	30.7	0.0268	402	100	0.00210	392	0.30	2.00

Note: b = width of column section; D = height of column section; f'_c = concrete compressive stress at test day; f_{yl} = yield stress of longitudinal reinforcement; f_{yt} = yield stress of transverse reinforcement; L = clear height of the column; s = spacing of transverse reinforcement; ρ_l = longitudinal reinforcement ratio; ρ_t = volume transverse reinforcement ratio; P = column axial load; A_g = area of column cross section; Δ_a = column displacement at axial load failure.

Numerical Models for Beam-Column Joints in Reinforced Concrete Building Frames

Jong-Su Jeon, Laura N. Lowes, Reginald DesRoches

Synopsis: The results of laboratory testing and earthquake reconnaissance studies of reinforced concrete frames indicate that beam-column joint deformation can determine total frame deformation and that for older buildings joint failure can result in frames losing lateral and gravity load carrying capacity. Given the impact of joints on frame response, numerical models used to evaluate the earthquake performance of reinforced concrete frames must include nonlinear joint models. This paper reviews previously proposed models for simulating joint response with the objective of identifying models that provide i) accurate simulation of response to earthquake loading, ii) simple implementation in nonlinear analysis software, iii) numerical robustness, iv) computational efficiency, and v) objective calibration procedures. Ultimately, no set of models was identified that met all of these requirements for the range of geometric and design parameters found in reinforced concrete buildings in the United States. With the objective of extending current modeling capabilities for interior joints, an experimental data set was assembled. The data set was used to evaluate existing envelope response models and used to calibrate cyclic response parameters for use with the preferred existing model. A new response model for interior beam-column joints is presented that meets the above requirements for the range of geometric and design parameters found in reinforced concrete buildings in the United States.

Key words: Reinforced concrete, beam-column joint, earthquake, seismic behavior, model, simulation, bond, shear strength

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Introduction

Numerical simulation to evaluate the earthquake performance of reinforced concrete frames requires simulation of the nonlinear response of beam-column joints. Laboratory testing of building frame subassemblages shows that beam-column joints with design details typical of pre-1967¹ construction exhibit severe stiffness and strength loss (Walker 2001; Leon 1990; Meinheit and Jirsa 1981; Pantelides et al. 2002). Significant strength and stiffness loss have been observed also for joints designed to achieve superior performance, such as those compliant with ACI 318 Code requirements (e.g., Park and Ruitong 1988; Durrani and Wight 1985). Finally, earthquake damage to reinforced concrete frames observed during post-earthquake reconnaissance efforts (e.g., Hall 1996; Holmes and Somers 1996) suggests also that joint failure may result in structural collapse. Given the potential for beam-column joints to determine frame performance, simulation of nonlinear joint response is required for accurate evaluation of the earthquake performance of older and modern reinforced concrete frame buildings.

Experimental investigation of the earthquake response of reinforced concrete beam-column joints has been the focus of numerous research studies during the last 40 years. Experimental testing has resulted in data characterizing the response of frame subassemblages, which comprise column and beam segments as well as joints and, in some cases, characterizing the local response of the beam-column joint. Experimental tests have addressed a range of joint configurations and designs, including interior and exterior joints from two-dimensional and three-dimensional frames with detailing representing pre-1967 and modern construction. However, the objective of many of these laboratory tests was to establish that modern building code requirements result in acceptable earthquake performance; thus, many of these tests employed joint designs representative of modern construction, including transverse reinforcement, moderate joint shear stress demands, and moderate bond stress demands for beam

¹ In 1967 UBC provisions changed to include requirements that i) joint shear strength defined using the column shear strength equation exceed joint shear demand, ii) transverse reinforcement be provided in the joint to achieve required shear strength, iii) the sum of column flexural strengths exceed the sum of beam flexural strengths, and iv) splices in beam and column longitudinal reinforcement be located away from the joint. Similar revisions were made to the ACI Code in 1971. These Code changes are reflected in drawings for buildings designed for construction on the West Coast from 1923 – 1979 (Mosier 2000).

longitudinal reinforcement. Fewer tests have investigated the earthquake response of older joints with no transverse reinforcement, high shear stress demands and high bond stress for beam longitudinal reinforcement anchored in the joint. Additionally, most experimental investigations have employed interior beam-column joint subassemblages from two-dimensional frames (the test specimen is cruciform-shaped and comprises a column that is continuous through the joint, a beam that is continuous through the joint, and the joint region with no out-of-plane beam segments). Fewer investigations have employed interior joints from three-dimensional frames (the test specimen comprises a column that is continuous through the joint, a beam that is continuous through the joint, a second orthogonal beam that is continuous through the joint or terminates in the joint, and the joint region). A very few tests have considered the behavior of exterior corner joints in three-dimensional frames (the test specimen comprises a column that is continuous through the joint, two orthogonal beam segments that terminate in the joint, and the joint region). Despite the fact that a plethora of experimental data do not exist for all joint geometries and designs, sufficient data exist to enable calibration of numerical models for use in evaluating the earthquake performance of concrete frames.

Given the impact on frame response of joint flexibility and strength loss, many previous studies have addressed the development of beam-column joint models for use in assessing the earthquake behavior and performance of concrete frames. Previously proposed models have ranged in sophistication from i) “super-elements” comprising multi-dimensional continuum elements in which model response is defined purely by fundamental material properties and joint geometry (Pantazopoulou and Bonacci 1994; Mitra 2007; Baglin and Scott 2000), to ii) “super-elements” comprising one-dimensional springs and employing simplifying assumptions about joint response mechanisms (Lowes and Altoontash 2003; Altoontash 2004; Mitra and Lowes 2007; Tajiri et al. 2006), to iii) rotational hinge models calibrated to fit experimental data (Otani 1974; Anderson and Townsend 1977). In recent years, validation and calibration efforts have employed larger and larger data sets; for example for interior joints, Mitra and Lowes (2007), Shin and LaFave (2004), and Jeon (2013) employed data from 57, 26 and 124 tests, respectively. Unfortunately, however, previous research has not resulted in a model or set of models that fully meets the needs of the earthquake engineering community with respect to modeling accuracy and precision, computational efficiency and robustness, and ease of use for the full range of beam-column joint configurations found in the building inventory and for the case of dynamic cyclic as well as quasi-static monotonic loading.

Modeling Objectives and Research Process

The objective of the research presented here was the development of joint modeling recommendations for evaluation of the earthquake performance of concrete frames. Specifically, joint models were sought that met the following criteria:

1. Accurate simulation of the fundamental characteristics of joint response to earthquake loading. Multiple models may be required to provide accurate response simulation for a range of joint designs and configurations. Simulation of response through significant loss of joint shear strength was considered to be an important attribute; simulation of the loss of joint axial load carrying capacity was not considered to be an important attribute. In the laboratory, beam-column joint subassemblages have typically maintained moderate axial loads following significant loss of joint shear strength. A recent study by Hassan (2011) found that for joints with beam-to-column depth ratios less than 2.5, joint axial load failure was unlikely to precede column axial load failure.
2. Simple implementation of the model in OpenSees using existing element formulations and material models. The OpenSees software platform was chosen for use because it is free, widely

used within the earthquake engineering research community, includes element and material models that can readily be employed to simulate joint response, and provides efficient and robust solution algorithms to support analysis of building frames subjected to quasi-static and dynamic loads. In particular, OpenSees includes joint elements and 1D spring elements that can be used to simulate joint kinematics and material models that are capable of simulating the stiffness and strength loss and pinched hysteretic response exhibited by joints subjected to cyclic loading.

3. Numerical robustness and stability, so that analyses are not hampered by “failures to converge”, and computational efficiency.
4. Objective calibration procedures, so that the model-building process is as efficient as possible and modeling parameters do not need to be adjusted to define the model for a specific beam-column joint with a specific set of design characteristics.

To accomplish the above objective, previously proposed models were reviewed and evaluated with respect to the above criteria. As needed, model development activities were accomplished to fill gaps in the capabilities of existing models.

Modeling Joint Geometry

Previous research has resulted in several approaches for modeling the geometry of the beam-column joint region for frame analysis. This research has primarily addressed analysis of 2D frames, with analysis of 3D systems accomplished by duplicating the 2D modeling approach in the two orthogonal framing directions. Few studies have addressed modeling of the 3D joint volume to support modeling of joint behavior under 3D loading scenarios; few 3D joint elements exist. Most research addressing joint modeling in 2D has employed either a four-node, finite-area joint element or a zero-length rotational spring element. Zero-length elements have been used at the beam-joint and column-joint interface to simulate stiffness and strength loss associated with bond-slip within the joint.

Using a four-node, finite-area joint element, joint behavior is typically simulated through the action of multiple 1D spring elements that compose the joint element (Altoontash 2004; Youssef and Ghobarah 2001; Shin and LaFave 2004; Lowes and Altoontash 2003; Mitra and Lowes 2007). However, the joint element may comprise only a single rotational spring (Celik 2007; Park 2010; Jeon 2013) or a mesh of continuum elements (Baglin and Scott 2000; Hegger et al. 2004; Deaton 2013; Mitra 2007). Joint *super-elements* that comprise more than one spring or continuum element typically require an intra-element solution². Typically the user has minimal control of the intra-element solution algorithm; as such, the use of super-elements introduces the potential for numerical problems and reduced numerical robustness and efficiency. For 2D analysis, element nodes are typically located at the joint-beam and joint-column interface; thus, no additional modeling effort is required to ensure that inelastic flexural action in beams and columns occurs at the beam-joint / column-joint interface and that beam and column elements do not contribute flexural deformation within the joint region. Appropriate element formulations are required to enable simulation of geometric nonlinearity.

² The OpenSees joint elements are essentially small structural subassemblages within the global structural model. These subassemblages comprise multiple nodes and multiple nonlinear springs that are not (explicitly) part of the global structural model. For each iteration to establish equilibrium of the global structural model, it is necessary to solve for equilibrium of the joint subassemblage model. While the user has control over the global solution algorithm, with the ability to change the tolerance or step size to improve the rate of convergence, the user has no control over the solution algorithm used to solve the joint subassemblage model. Thus, the user has limited ability to continue the analysis when a “failure to converge” occurs within the joint element.