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EVALUATION OF EXISTING REINFORCED CONCRETE COLUMNS

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SUMMARY

An idealized backbone model defining the key damage states of an existing reinforced concrete column (flexural yielding, shear failure, and axial load failure) is developed using drift capacity models. Assumptions and limitations of the drift capacity models are discussed. To demonstrate the application of the idealized backbone model to the evaluation of existing reinforced concrete columns, the model is compared to data from shake table tests and measured drifts from an instrumented building damaged during the Northridge Earthquake. The need for probabilistic drift capacity models is also discussed.

INTRODUCTION

Experimental research and post-earthquake reconnaissance have demonstrated that reinforced concrete columns with light or widely spaced transverse reinforcement are vulnerable to shear failure during earthquakes. Such damage can also lead to a reduction in axial load capacity, although this process is not well understood. As the axial capacity diminishes, the gravity loads carried by the column must be transferred to neighboring elements, possibly leading to a progression of damage, and in turn, collapse of the building.

To assess the collapse potential of a building structure, engineers require models capturing the key damage states of a column during seismic response, namely: flexural yielding; shear failure; and axial load failure. Models have been proposed by Elwood and Moehle [1, 2] to estimate the drifts at shear and axial load failure for existing reinforced concrete columns. This paper will summarize these models and discuss how they can be incorporated into an idealized backbone response for existing reinforced concrete columns. This idealized backbone response has been implemented in an analytical model [3], enabling the nonlinear analysis of existing building frames including the influence of shear and axial load failures.

The models for shear and axial load failure have been developed for columns which are expected to experience flexural yielding prior to shear failure. As such, the idealized backbone model presented here should only be applied to columns with similar characteristics. To demonstrate its application, the backbone model has been applied to two columns: one from dynamic laboratory tests, and a second from an instrumented building damaged during the Northridge Earthquake.

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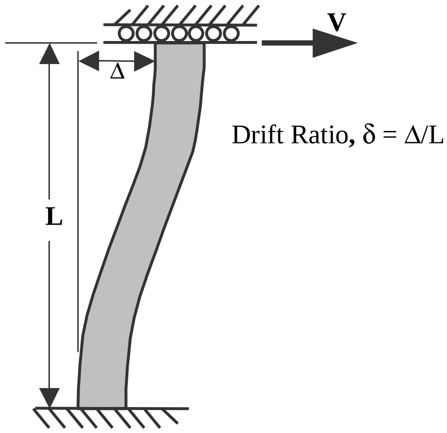


Figure 1. Fixed-fixed column subjected to lateral loads

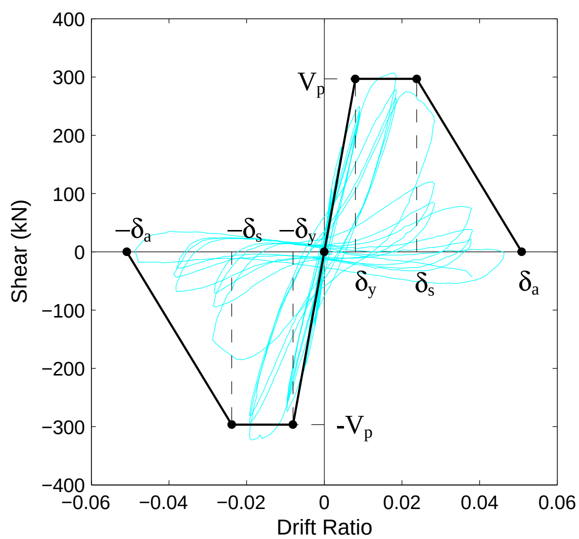


Figure 2. Calculated idealized backbone plotted with column test data (Column data from reference [4])

IDEALIZED BACKBONE RESPONSE

The idealized backbone captures significant changes in the shear-drift response of a column subjected to cyclic lateral loads; namely, flexural yielding, shear failure (defined by a 20% loss of shear strength), and axial load failure (defined by the initial redistribution of axial load supported by the column). The backbone presented herein is developed for a column which is assumed to be fixed against rotation at both ends and deforming in double curvature, as illustrated in Figure 1. The shear, V, is constant over the height of the column, and the drift ratio, δ, is defined as the displacement of the top of the column divided by the clear height of the column.

Figure 2 shows an example of the idealized backbone response compared with pseudo-static test data by Sezen [4]. The backbone is defined by the following coordinates approximating the critical damage states during cyclic response:

- Flexural yielding at (δ_y, V_p)
- Shear failure at (δ_s, V_p)
- Axial load failure at (δ_a, 0.0)

where V_p is the calculated plastic shear capacity, and δ_y, δ_s, and δ_a, are the calculated drift ratios at yielding, shear failure, and axial load failure, respectively. Figure 2 illustrates the application of the idealized backbone for a column with approximately constant axial load, and hence, the backbone is assumed to be symmetric about the origin. For columns with varying axial load, the cyclic response will not be symmetric about the origin and the coordinates of each damage state must be determined using the appropriate axial load. The following sections describe how the coordinates for each of the damages states can be calculated.

Flexural Yielding

The idealized backbone approximates the gradual yielding of longitudinal reinforcement with an elastic-perfectly-plastic response. Yielding is assumed to occur once the shear demand reaches the plastic shear capacity, V_p. For the assumed boundary conditions shown in Figure 1, the column plastic shear capacity can be determined as follows:

$$V_p = \frac{2M_p}{L} \tag{1}$$

where M_p is the plastic moment capacity based on standard section analysis (assuming plane sections remain plane).

The drift ratio at yielding of the longitudinal reinforcement can be considered as the sum of the drifts due to flexure, bar slip, and shear:

$$\delta_y = \delta_{flex} + \delta_{slip} + \delta_{shear} \tag{2}$$

Assuming the column is fixed against rotation at both ends and assuming a linear variation in curvature over the height of the column, the drift ratio at yield due to flexure can be estimated as follows:

$$\delta_{flex} = \frac{L}{6}\phi_y \tag{3}$$

where ϕ_y is the curvature at yielding of the longitudinal reinforcement.

The drift ratio at yield due to bar slip can be estimated as follows [5]:

$$\delta_{slip} = \frac{d_b f_y \phi_y}{8u} \tag{4}$$

where d_b is the diameter of the longitudinal reinforcement, f_y is the yield stress of the longitudinal reinforcement, and u is the bond stress between the longitudinal reinforcement and the beam-column joint concrete. A bond stress of $0.5\sqrt{f_c'}$ (MPa units) [$6\sqrt{f_c'}$ in psi units] can be assumed for the calculation of δ_{slip} [5].

Finally, assuming the column is fixed against rotation at both ends, the drift ratio at yield due to shear deformations can be estimated by idealizing the column as consisting of a homogeneous material with a shear modulus G:

$$\delta_{shear} = \frac{2M_p}{(5/6)A_g GL} \tag{5}$$

where A_g the gross area of the column section.

The three drift components described above can be estimated using the column cross sectional dimensions and material properties. Given the drift components, Equations 1 and 2 can be used to estimate the point of flexural yielding for the idealized backbone, (δ_y, V_p).

Shear Failure

Since the idealized backbone has been developed for columns which are expected to yield prior to shear failure, some limited ductility can be expected in the cyclic response prior to a significant degradation in shear strength. To capture this limited ductile response prior to shear failure, the idealized backbone is given a horizontal slope between the drift at flexural yielding and the drift at shear failure, and hence, V_p is used to approximate the shear demand at shear failure. The following paragraphs describe the evaluation of the drift at shear failure, δ_s.

Several models have been developed to represent the degradation of shear strength with increasing inelastic deformations [4, 6, 7, 8]. While these shear strength models are useful for estimating the column strength as function of deformation demand, they are less useful for estimating displacement at shear failure. For example, the model by Sezen [4] represents shear strength as a function of displacement ductility, μ_δ, using the relation shown in Figure 3, where V_0 = initial shear strength. A small variation in shear strength (shown by mean plus or minus one standard deviation bounds) or similar variation in flexural strength (not shown) can result in large variation in estimated displacement capacity, $\Delta\mu_\delta$. These models also suggest misleading trends in the relation between some critical parameters (such as axial load) and displacement capacity at shear failure.

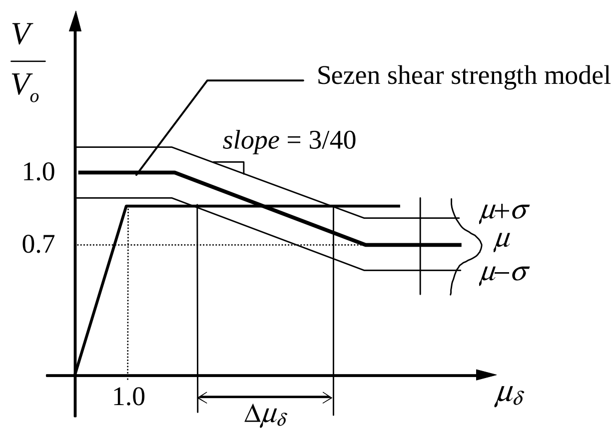


Figure 3. Displacement at shear failure as function of model variability

Pujol et al. [9, 10, 11] have proposed drift capacity models for columns failing in shear. These models make an important contribution by focusing attention directly on displacement capacity and by analyzing data for model development. The database of Pujol et al. includes columns with transverse reinforcement ratios exceeding 0.01, which is larger than that which is of interest in the present study.

To evaluate the response of columns with low transverse reinforcement ratios, a database of 50 column tests with observed shear distress at failure and tested in single or double curvature was compiled [4]. Columns in the database had the following range of properties:

- shear span to depth ratio: $2.0 \le a/d \le 4.0$
- concrete compressive strength: $13 \le f_c^{'} \le 45$ MPa $[1900 \le f_c^{'} \le 6500$ psi]
- longitudinal reinforcement yield stress: $324 \le f_{yl} \le 524$ MPa $[47 \le f_{yl} \le 76$ ksi]
- longitudinal reinforcement ratio: $0.01 \le \rho_l \le 0.04$
- transverse reinforcement yield stress: $317 \le f_{yt} \le 648$ MPa $[46 \le f_{yt} \le 94$ ksi]
- transverse reinforcement ratio: $0.0010 \le \rho" \le 0.0065$

- maximum shear stress: $0.23 \le \frac{v}{\sqrt{f_c^{'}, MPa}} \le 0.72 \quad [\,2.8 \le \frac{v}{\sqrt{f_c^{'}, psi}} \le 8.6\,]$

- axial load ratio: $0.0 \le \frac{P}{A_g f_c^{'}} \le 0.6$

The model of Sezen [4] can be used to estimate mean shear strength as a function of displacement ductility. As suggested by Figure 3, the intersection of the shear corresponding to flexural strength with mean shear strength can be interpreted to indicate the expected displacement at shear failure. Figure 4 compares results obtained by this procedure with those actually observed during the tests included in the database. The results show relatively high bias and dispersion (mean of the measured drift ratio at shear failure divided by the calculated drift ratio is 1.78 and the coefficient of variation is 0.63).

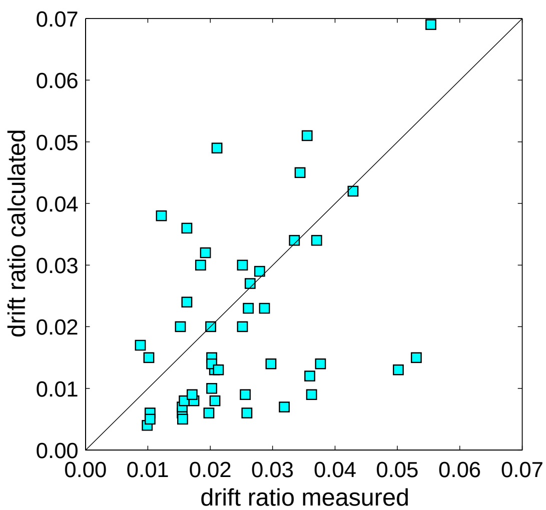


Figure 4. Measured versus calculated drift at shear failure, interpreted from Sezen shear strength model.

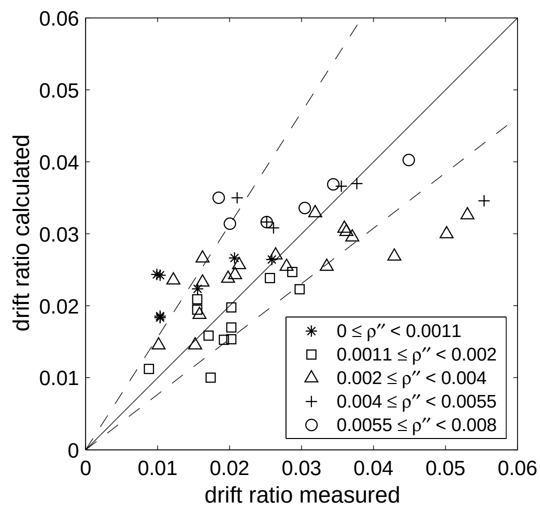


Figure 5. Measured versus calculated drift at shear failure, Equation 6

Elwood and Moehle [1] reevaluated the database test results from a displacement-capacity perspective, and proposed the following relationship to estimate drift ratio at shear failure:

$$\delta_s = \frac{3}{100} + 4\rho'' - \frac{1}{40}\frac{v}{\sqrt{f_c^{'}}} - \frac{1}{40}\frac{P}{A_g f_c^{'}} \ge \frac{1}{100} \qquad \text{(MPa units)} \quad (6)$$

$$[\,\delta_s = \frac{3}{100} + 4\rho'' - \frac{1}{500}\frac{v}{\sqrt{f_c^{'}}} - \frac{1}{40}\frac{P}{A_g f_c^{'}} \ge \frac{1}{100} \qquad \text{(psi units)}]$$

where ρ'' = transverse steel ratio, v = nominal shear stress, f'_c = concrete compressive strength, P is the axial load on the column, and A_g is the gross cross-sectional area. Figure 5 compares Equation 6 with the results from the database. The mean of the measured drift ratio divided by the calculated drift ratio is 0.97, the coefficient of variation is 0.34. The correlation is improved compared with that shown in Figure 4.

Equations 1 and 6 can be used to determine the point of shear failure for the idealized backbone response. When computing the drift ratio at shear failure using Equation 6, the shear stress, v, can be estimated based on the plastic shear capacity, i.e. $v = V_p/(bd)$.

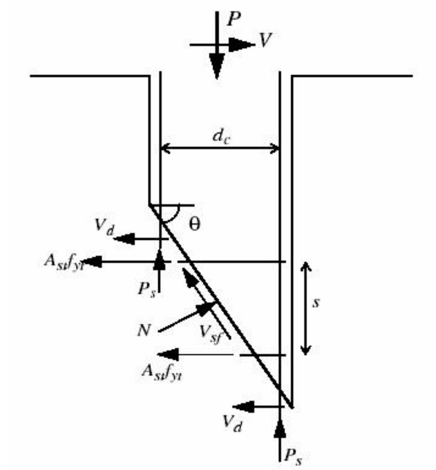


Figure 6. Free-body diagram of column after shear failure

Axial Load Failure

Experimental studies have shown that axial load failure tends to occur when the shear strength degrades to approximately zero [12]. Hence, the final point on the idealized backbone assumes a shear strength of zero, and therefore, only requires the calculation of the drift ratio at axial load failure, δ_a.

Elwood and Moehle [2] have proposed a shear-friction model to represent the general observation from experimental tests that the drift ratio at axial failure of a shear-damaged column is inversely proportional to the magnitude of the axial load. Figure 6 shows a free-body diagram for the upper portion of a column under shear and axial load. The inclined free surface at the bottom of the free-body diagram is assumed to follow a critical inclined crack associated with shear damage. The "critical" crack is one that, according to the idealized model, results in axial load failure as shear-friction demand exceeds the shear-friction resistance along the crack.

Several assumptions are made to simplify the problem. Dowel forces from the transverse reinforcement crossing the inclined crack are not shown; instead, the dowel forces are assumed to be included implicitly in the shear-friction force along the inclined plane. Shear resistance due to dowel action of the longitudinal bars depends on the spacing of the transverse reinforcement, and reasonably can be ignored for the columns considered in this study. Relative movement across the shear failure plane tends to compress the longitudinal reinforcement. Given the tendency for buckling, especially in the limit, the axial force capacity of the longitudinal reinforcement will be assumed equal to zero. Finally, the horizontal shear force is assumed to have dropped to zero in the limit following shear failure as noted above from experimental evidence [12].

Equilibrium of the forces shown in Figure 6 results in the following expression for the axial capacity of the column:

$$P = \frac{A_{st} f_{yt} d_c}{s} \tan\theta \left(\frac{1 + \mu \tan\theta}{\tan\theta - \mu} \right) \tag{7}$$

in which all variables are shown in Figure 6 except μ = effective shear-friction coefficient. An empirical approach was used to define the crack angle θ and effective shear-friction coefficient μ, with data from twelve columns tested at the University of California, Berkeley [4, 13]. These full-scale shear-critical reinforced concrete columns were subjected to constant or varying axial load with cyclic lateral load reversals in one plane only. Tests were continued past shear failure to the point where axial load could no longer be sustained.

From these relations, Elwood and Moehle [2] developed relations among axial load, transverse reinforcement, and drift ratio at axial load failure as:

$$\delta_a = \frac{4}{100} \frac{1 + \tan^2 \theta}{\tan\theta + P\left(\dfrac{s}{A_{st} f_{yt} d_c \tan\theta}\right)} \tag{8}$$

in which θ = critical crack angle assumed = 65 deg, P = axial load, A_{st} = area of transverse reinforcement parallel to the applied shear and having longitudinal spacing s, f_{yt} = transverse reinforcement yield stress, and d_c = depth of the column core measured parallel to the applied shear. Results of tests reported in [4] and [13] are compared with Equation 8 in Figure 7.

While useful as a design chart for determining drift capacities, Figure 7 must only be used with a full appreciation for the inherent scatter in the results and the limitation that the results are based on unidirectional, pseudo-static tests.

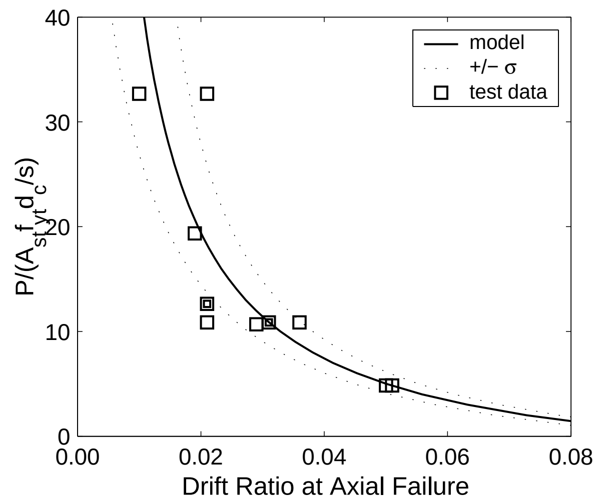


Figure 7. Drift capacity curve based on shear-friction model

Discussion of Idealized Backbone Model

Given column details and the axial load demand, Equations 1, 2, 6, and 8 can be used to calculate the idealized backbone response shown in Figure 2 for a reinforced concrete column experiencing shear and axial load failure after flexural yielding. However, before the idealized backbone model can be used to assess the capacity of an existing reinforced concrete column, it must be determined if the column is expected to experience shear failure *after* yielding of the longitudinal reinforcement, a common characteristic of all the column tests used to develop the drift capacity models described above.

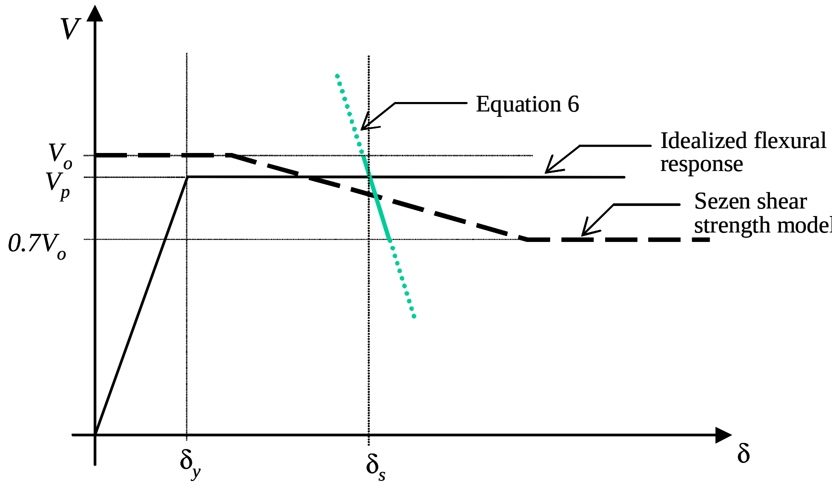


Figure 8. Comparison of Sezen shear strength model and model for drift ratio at shear failure (Equation 6)

To determine if a column is likely to experience shear failure after flexural yielding, the plastic shear capacity, V_p, should be compared with an appropriate shear strength model, such as that proposed by Sezen [4]. As shown in Figure 8, the plastic shear capacity should fall in the range between the initial shear strength, V_o, and the final shear strength, $0.7V_o$. Because there is some dispersion between actual and calculated shear strength, some columns with shear demand less than the calculated shear, or plastic shear capacity greater than V_o, may still experience shear failure after flexural yielding. Currently, engineering judgment is required to select which columns with V_p outside the V_o to $0.7V_o$ range are still expected to experience shear failure after flexural yielding, and hence, can be evaluated using the proposed idealized backbone model. To reduce the reliance on judgment alone, there is a need to develop probabilistic capacity models accounting for the variability in the test results relative to the mean response captured by the model. As described later in this paper, given such probabilistic capacity models, it is possible to define the probability of a given column experiencing shear failure after flexural yielding.

It must be recognized that the axial failure model described above is based on data from only twelve columns. All of the columns were constructed of normal strength concrete, had the same height to width ratio, and were designed to yield the longitudinal reinforcement prior to shear failure. Only limited variation in the spacing and type of transverse reinforcement was possible. The axial failure model presented here may not be appropriate for columns for whom the test specimens are not representative. Furthermore, all twelve columns were tested under unidirectional lateral load parallel to the one face of the column. With the exception of two tests, the loading routine was standardized, with each column subjected to nominally constant axial compression and a series of lateral displacements at increasing amplitude (three cycles at each amplitude). During earthquake excitation columns can experience bidirectional loading and a wide variety of loading histories, which may consist of a single large pulse or many smaller cycles prior to shear and axial load failure. It has been demonstrated that an increase in the number of cycles past the yield displacement can result in a decrease in the drift capacity at shear failure [11]. Equation 6 for the drift ratio at shear failure does not account for the influence of the number of significant loading cycles. It is also anticipated that an increase number of cycles will reduce the drift capacity at axial failure, although not enough test data is available to support or refute this hypothesis. Further testing of reinforced concrete columns to the stage of axial failure is needed to supplement the current database.

The proposed models for the drift ratios at shear and axial load failure (Equations 6 and 8) provide estimates of the *mean* drift ratio at failure for the columns included in the respective databases. As such, approximately half of the columns from the databases used to develop the models experienced failure at drifts *less than* those predicted by the models. Furthermore, actual columns have configurations and loadings that differ from those used in the tests, so that some additional scatter in results seems likely. A model used for design or assessment might be adjusted accordingly to provide additional conservatism as may be appropriate. Probabilistic capacity models, described later in this paper, may assist in selecting an appropriate degree of conservatism.

APPLICATIONS OF THE IDEALIZED BACKBONE MODEL

Two applications of the idealized backbone model developed above will be discussed. First, the idealized backbone model will be compared with shake table test results [14]; and second, the backbone model will be used to assess the capacity of a column from an instrumented building damaged during the Northridge Earthquake.

Comparison with shake table test results

Shake table tests were designed to observe the process of dynamic shear and axial load failures in reinforced concrete columns when an alternative load path is provided for load redistribution. The test specimens were composed of three columns fixed at their base and interconnected by a beam at the upper level (Figure 9). The center column had wide spacing of transverse reinforcement making it vulnerable to shear failure, and subsequent axial load failure, during testing. As the center column failed, shear and axial load would be redistributed to the adjacent ductile columns.

Two test specimens were constructed and tested. The first specimen supported a mass of 300 kN, producing column axial load stresses roughly equivalent to those expected for a seven-story building. The second specimen also supported a mass of 300 kN, but pneumatic jacks were added to increase the axial load carried by the center column from 128 kN (0.10 f'_cA_g) to 303 kN (0.24 f'_cA_g), thereby amplifying the demands for redistribution of axial load when the center column began to fail.

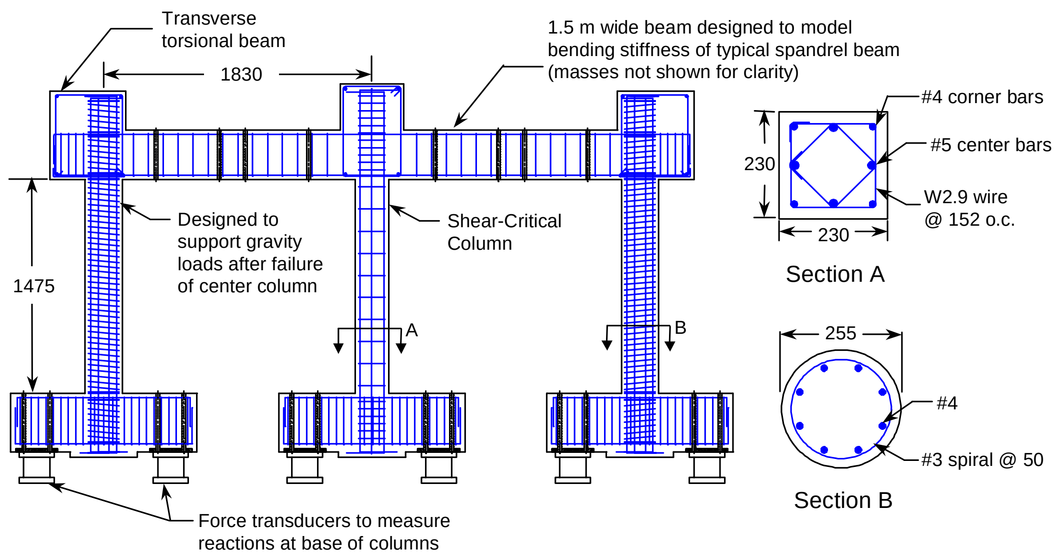


Figure 9. Shake table test specimen (units in mm).

Figure 10a. Top of center column, Specimen 1 at 24.9 sec

Figure 10b. Top of center column, Specimen 2 at 24.9 sec

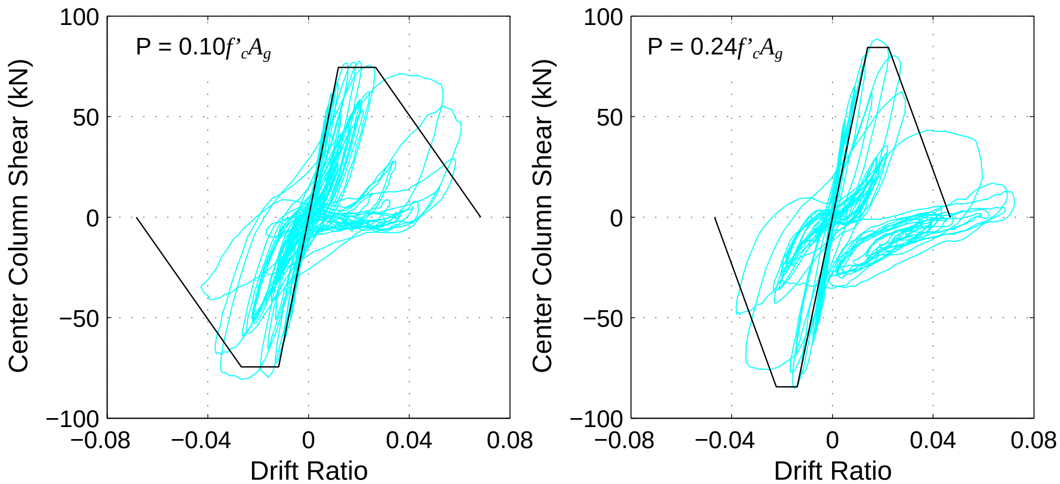


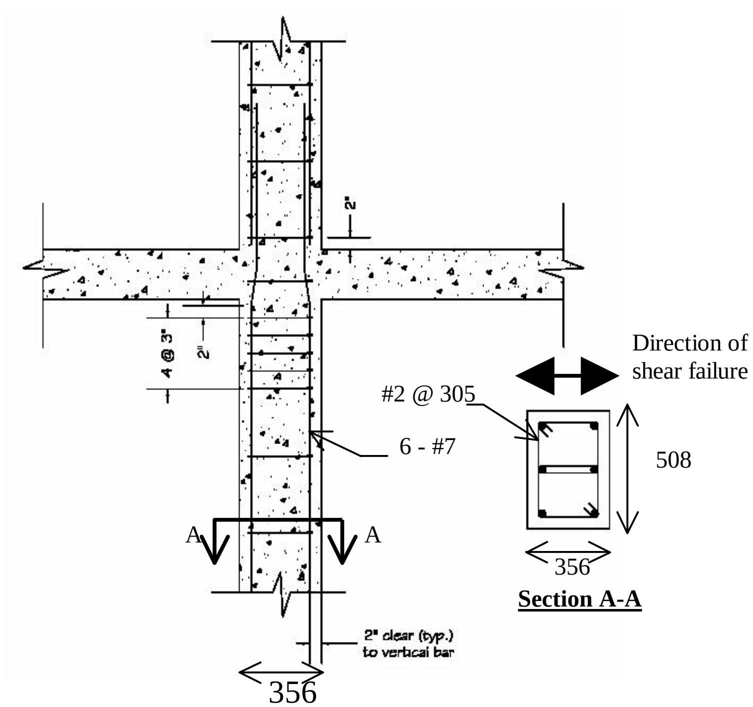
Figure 11. Center column hysteretic response compared with idealized backbone

The shear-critical center column was designed as a one-half scale reproduction of the 2.95 m tall, 457 mm x 457 mm square-cross-section columns reported in [4] and [13]. From those previous tests, it was expected that the center column would sustain flexural yielding prior to shear failure. Axial load failure was expected to be more gradual for the column with low axial load and more sudden for the column with higher axial load. The remaining frame elements (i.e., the beams, outside columns, and footings) were not scaled from prototype designs, but instead were designed to achieve the desired response. Both specimens were subjected to one horizontal component from a scaled ground motion recorded at Viña del Mar during the 1985 Chile earthquake.

The first specimen with lower axial load sustained severe diagonal cracking (Figure 10a) and a loss of lateral load capacity, however, was able to maintain most of its axial load demand. The center column from the second specimen with moderate axial load, also failed in shear, however, was unable to support most of its axial load demand, resulting in shortening of the center column (Figure 10b). Further results from the shake table tests are reported elsewhere [14].

Figure 11 indicates that the idealized backbones, calculated using Equations 1, 2, 6, and 8, provide a reasonable estimate of the measured hysteretic response of the center columns. The model underestimates the drift ratio at shear failure for the specimen with low axial load, but provides a good estimate of the drift ratio at shear failure for the specimen with moderate axial load. These results suggest that the model for the drift ratio at shear failure (Equation 6) may underestimate the influence of the axial load on the drift ratio at shear failure, particularly for low axial loads. Figure 11 also indicates that the observed response of the test specimens was consistent with the calculated drifts at axial failure based on Equation 8. The measured drifts for the specimen with low axial load (and no axial failure) remain below the calculated drift ratio at axial load failure, while the measured drifts for the specimen with moderate axial load (and experiencing axial failure) exceed the calculated drift ratio at axial load failure.

Figure 12. Building frame with columns damaged during Northridge Earthquake



Column C4 Properties:

$f_{y\ long}$ =	496 MPa
$f_{y\ trans}$ =	345 MPa
f'_c =	28 MPa
P =	$0.13\ A_g\ f'_c$
L =	2.08 m

Figure 13. Details and assumed properties for fourth story column

Column damaged during the Northridge Earthquake

In this section the idealized backbone model will be used to assess the capacity of a column from a building damaged during the Northridge Earthquake. The seven-story moment frame building, shown in Figure 12 and located in Van Nuys, California, experienced significant ground shaking during both the San Fernando Earthquake and the Northridge Earthquake. Instrumentation distributed throughout the structure prior to both earthquakes has provided very valuable data on the inelastic response of moment frame buildings. Extensive literature on the structure is available elsewhere [15, 16, 17, etc.]. Only details on the damaged fourth-story exterior columns will be discussed here. An exterior column from the fourth story of the building (C4, circled in Figure 12) will be evaluated using the idealized backbone model. Typical details for this column are shown in Figure 13. The axial load was estimated based on the dead load from a three-dimensional static analysis.

Significant shear distress at the top of the column was observed after the Northridge Earthquake (Figure 14). Based on this damaged state, it is reasonable to expect that the column has lost the majority of its lateral load capacity due to shear failure. However, it is unclear whether the column experienced loss of axial load capacity resulting in gravity load redistribution to neighboring elements. Based on interpolation of the recorded motions on the third and sixth floors, the maximum drift ratio at the fourth story during the Northridge Earthquake was estimated to be 0.018. Note that 0.018 is expected to be a lower-bound value since the drifts were likely higher at the fourth story due to the concentration of damage at this level.

Figure 14. Damage to Column C4 after Northridge Earthquake [17]

As emphasized earlier, the idealized backbone model has been developed for columns which are expected to experience shear failure *after* flexural yielding. To determine if column C4 experienced shear failure *after* flexural yielding, the plastic shear capacity (assuming the column is fixed against rotation at both ends), V_p, is compared with the shear strength according to the Sezen model, V_o, as illustrated in Figure 8. The ratio V_p/V_o is found to be 1.04, indicating that the Sezen shear strength model suggests that the column may have failed in shear *prior to* flexural yielding. Currently no model exists to determine the drift at axial load failure for columns experiencing this mode of failure. As will be discussed in the subsequent section, due to the variability in the column shear strength (and, indeed, in the flexural strength), finding V_p/V_o greater than unity does not preclude the possibility of shear failure occurring *after* flexural yielding. To illustrate the application of the idealized backbone model to column C4, it will be assumed that there exists sufficient probability that shear failure occurred after flexural yielding to warrant the investigation of this failure mode. This case highlights the importance of developing drift capacity models for columns expected to fail in shear prior to flexural yielding.

Applying Equations 2, 6, and 8, the following drift capacities were determined for column C4:

- $\delta_y = 0.010$
- $\delta_s = 0.023$
- $\delta_a = 0.022$

Note that the calculated drift at shear failure is slightly larger than the drift at axial load failure. This result is inconsistent with the expected behavior, since Equation 8 assumes shear failure has already occurred. Due to the lack of coupling between the models for the drift at shear failure (Equation 6) and the drift at axial failure (Equation 8), there is nothing requiring $\delta_s < \delta_a$ when the drift ratios are calculated.

Due to the variability associated with both drift capacity models, the results can be interpreted to imply that axial load failure is expected to occur almost immediately after shear failure at a drift of approximately 0.022.

The idealized backbone response and the estimated maximum drift ratio during the Northridge Earthquake are shown in Figure 15. Given that the maximum drift ratio of 0.018 is a lower-bound value, and the variability in the drift capacities, these results suggest that column C4 may have experienced axial load failure and redistributed some load to neighboring elements. The results also reinforce the need to directly account for the variability in the capacity models during the evaluation of existing reinforced concrete columns.

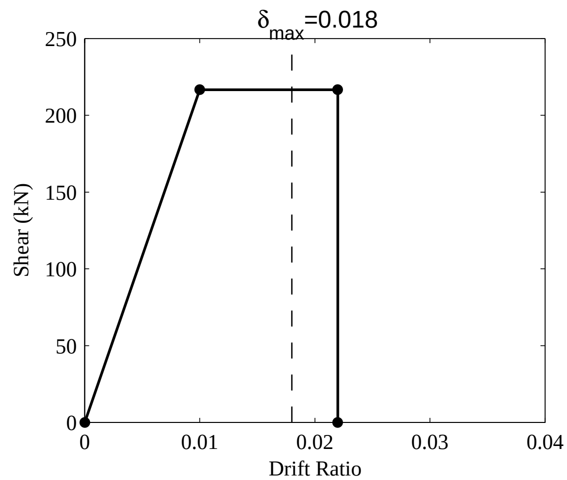


Figure 15. Idealized backbone model for column C4.

PROBABILISTIC CAPACITY MODELS

The capacity models described above only represent the *mean* response from the tests used to develop the models. Probabilistic capacity models, defining not just the mean response but also the variation in capacity about the mean value, are required to assess the probability of exceeding the column capacity. Given such a probability, an engineer will be better equipped to make an informed decision regarding the need for seismic retrofit.

The application of a probabilistic capacity model for the drift at shear failure is schematically represented in Figure 16. As discussed above, Equation 6 can be used to estimate the mean drift capacity at shear failure, δ_s. If the drift demand on the column, δ_{dem}, is less than δ_s, as shown in Figure 16, then, without a probabilistic drift capacity model it is be left to the engineer's judgment to determine if the column has sufficient safety against shear failure. With the development of a probabilistic drift capacity model (shown schematically in Figure 16 as a probability density function, $f(\delta)$, with a mean δ_s and standard deviation σ), it is possible to determine the probability of shear failure, P_f, by finding the shaded area under the probability density function:

$$P_f = \int_0^{\delta_{dem}} f(\delta)\, d\delta$$

(Note that for illustration purposes, Figure 16 ignores variability in the drift demand or plastic shear capacity).

Gardoni et al. [18] have introduced a methodology to develop probabilistic capacity models. Ongoing work by Zhu et al. [19] will apply this methodology to develop probabilistic drift capacity models for shear and axial load failure.

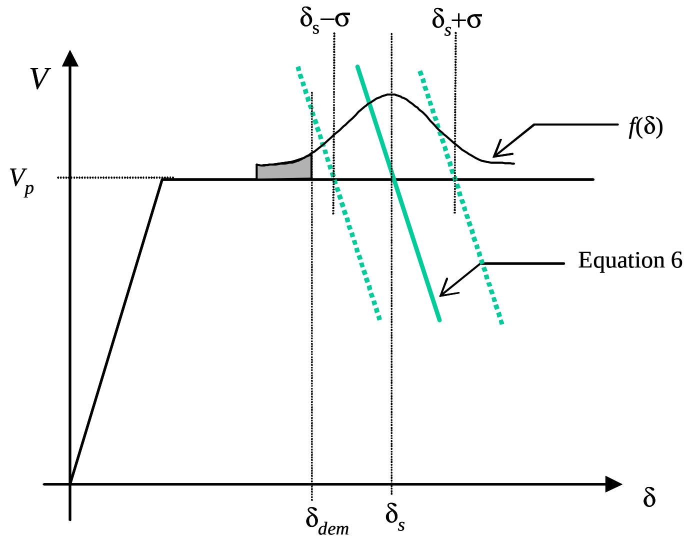


Figure 16. Illustration of the application of a probabilistic capacity model for the drift at shear failure

CONCLUSIONS

An idealized backbone model to assess the capacity of existing reinforced concrete columns with light transverse reinforcement has been presented. The model approximates the true backbone response of a column by capturing only the key damage states during cyclic response, namely, flexural yielding, shear failure, and axial load failure. Comparison of the model with shake table test data indicates that the idealized backbone adequately represents the cyclic behavior and accounts for the change in response with a change in axial load. Application of the model to a column damaged during the Northridge Earthquake highlights the need to account for the variability in the capacity models when evaluating the response of reinforced concrete columns. The development of probabilistic capacity models will enable the calculation of the probability of failure, and provide the engineer with better tools to assess the seismic response of a structure.

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