Cyclic Shear and Axial Strength of Lightly-Confined Concrete Columns

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Overview

Before the introduction of special requirements in the 1970s, reinforced concrete building frames constructed in zones of high seismicity in the US had details and proportions similar to frames designed solely for gravity loads. Columns generally were not designed to have strengths exceeding beam strengths, so column failure mechanisms often prevail. Relatively wide spacing of transverse reinforcement was common, such that column failures may involve some form of shear or flexure-shear failure, in some cases followed by loss of axial load capacity. This study examines laboratory behavior of columns with light transverse reinforcement and proposes models for shear strength and subsequent axial load failure that may be suitable for evaluation of existing building frames.

Applicability

The presented results are intended to be applicable to reinforced concrete columns under earthquake loading where strength is governed either by shear or by flexure followed by shear failure. Column shear span (length measured from point of maximum moment to point of inflection) should not deviate significantly from the tested range (in the tests, the shear span varied between two and four times the section depth in the direction of loading). Lateral loads in the tests were applied in a single horizontal direction; more rapid degradation is possible under biaxial loading.

Figure 1 shows a typical test column configuration.

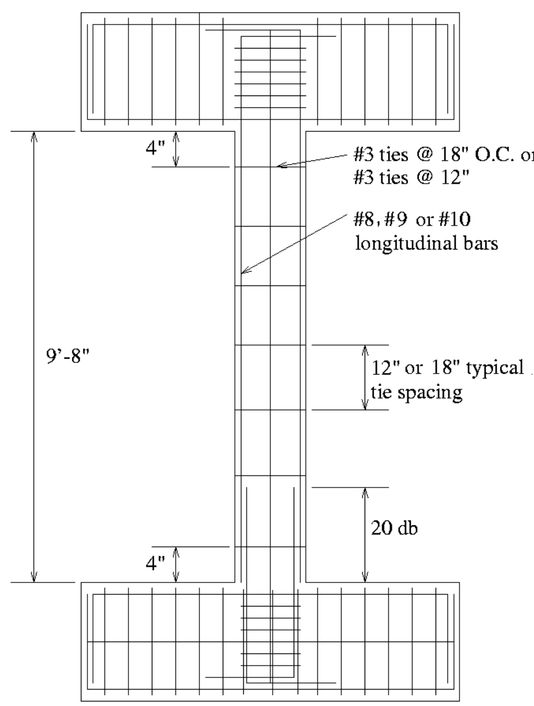

Figure 1 – Typical test column details

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Shear Strength Model

Shear strength is defined by Equations 1 through 3.

$$V_n = V_c + V_s \qquad \textbf{Equation 1}$$

$$V_c = k\left(\frac{6\sqrt{f_c'}}{a/d}\sqrt{1+\frac{P}{6\sqrt{f_c'}A_g}}\right)A_g \text{ psi} \qquad \textbf{Equation 2}$$

$$V_s = k\frac{A_{sw}f_y d}{s} \qquad \textbf{Equation 3}$$

V_n = nominal shear strength, V_c = contribution from concrete, V_s = contribution from ties, f_c' = concrete compressive strength (psi), a = shear span (column length from point of maximum moment to inflection point), d = column effective depth (may be taken equal to 0.8h), h = section depth measured parallel to shear, P = axial load, A_g = gross concrete area, A_{sw} = area of the tie steel, f_y = yield strength of the tie steel, s = tie spacing, and k is defined below.

No bounds are placed on the aspect ratio a/d, though it is noted that the range of values was limited to between 2 and 4 in the database. Some limits may be appropriate for columns having aspect ratios outside this range.

In Equations 2 and 3, the term k is a modifier to account for strength degradation within the flexural plastic hinges. For this data set, k was defined as shown in Figure 2.

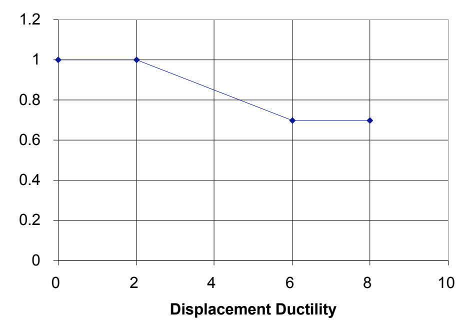

Figure 2 – Variation of parameter k

Shear Strength Results

Figure 3 plots ratios of measured to calculated shear strengths using the procedure defined by Equations 1 through 3. The correlation is relatively uniform for the range of ductilities shown. The mean ratio of test to calculated strength is 1.01; the coefficient of variation is 0.11.

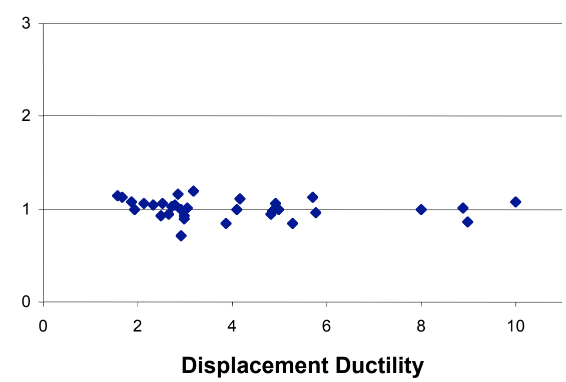

Figure 3 Ratios of measured shear strengths to shear strengths calculated by Equations 1 through 3.

Figure 4 plots ratios of measured to calculated shear strengths calculated by ACI 318-99 provisions (non-seismic provisions of Chapter 11) and as calculated by FEMA 273. For ACI 318-99, the mean ratio of test to calculated strength is 1.11; the coefficient of variation is 0.22. For FEMA 273, the correlation is very poor.

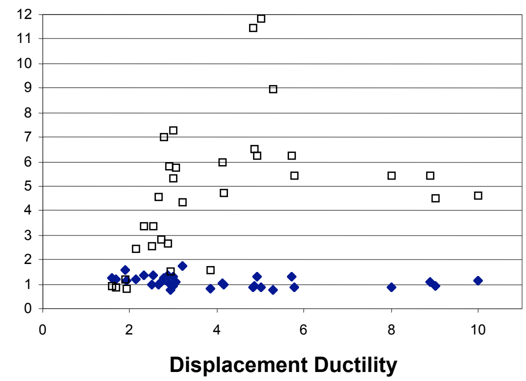

Figure 4 – Ratios of measured shear strengths to shear strengths calculated by ACI 318-99 and FEMA 273.

Axial Failure Model

In seismic evaluation and rehabilitation of existing concrete frames it may be of interest to know the lateral displacement capacity at which reinforced concrete columns can no longer support vertical loads. A simple model was developed to gain understanding of the problem. The model, depicted in the free-body diagram of Figure 5, defines vertical load capacity in terms of sliding resistance along an inclined plane. The inclination of the inclined plane was observed from twelve test columns. Column transverse reinforcement provides clamping action along the sliding plane. The sliding shear friction coefficient along the inclined plane was determined empirically, and was found to vary with imposed deformation level.

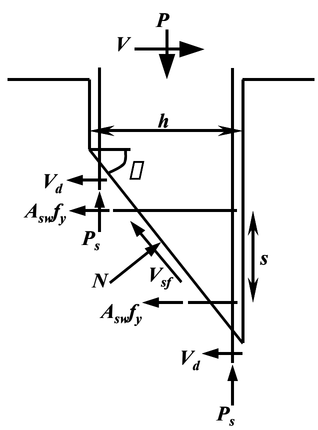

Figure 5 – Shear-friction model for axial load failure.

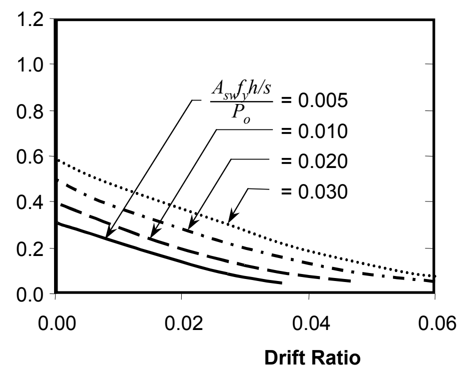

Figure 6 – Computed relation between axial load, transverse reinforcement, and drift ratio at axial load failure.

Figure 6 shows results of the model in terms of an alignment chart. P = axial load, P_o = concentric axial load capacity, A_{sw} = cross-sectional area of transverse reinforcement within spacing s, f_y = yield stress of transverse reinforcement, h = dimension of section parallel to shear that resulted in the inclined crack. To use the chart enter at the axial load ratio on the vertical axis, read across to intersect the appropriate curve, and read the drift capacity from the horizontal axis.

Figure 7 compares calculated and measured drift capacities at failure for the tests from which the relation was derived. It should be apparent that the proposed relation has limited accuracy. Somewhat greater scatter should be anticipated for columns with different geometries and loading histories.

For further information

See Moehle, Elwood, and Sezen reference below, or contact Jack Moehle by email at moehle@peer.berkeley.edu.

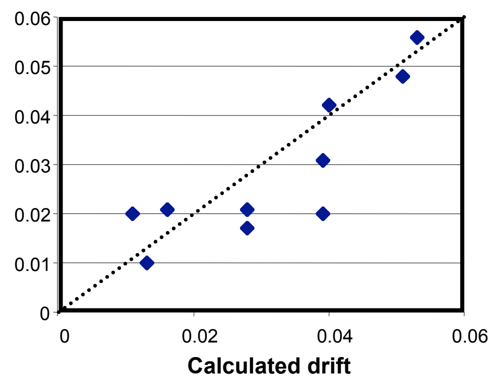

Figure 7 – Comparison of calculated and measured drift at failure.

Acknowledgment

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Keywords

buildings, columns, evaluation, existing buildings, frames, rehabilitation, Reinforced concrete, shear, tests.