EXPERIMENTAL TESTS ON SLENDER, PRESTRESSED COLUMNS

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ABSTRACT

The ACI 318 Building Code provides significant guidance for the design of nonprestressed concrete columns; but few provisions exist for prestressed columns, and no provisions exist for slender, prestressed columns. Furthermore, few test results exist for prestressed concrete columns, especially those that are slender. To address this shortcoming, twelve short-term tests of slender, prestressed columns were conducted using three different prestressing arrangements. The tests used typical construction materials and methods and, when necessary, appropriately scaled details. The results of the short-term tests are compared against existing design equations to evaluate their accuracy and conservatism.

Keywords: Concrete Columns, Prestressed Columns, Slender Columns

INTRODUCTION

The ACI 318 Building Code[1] provides significant guidance for the design of nonprestressed concrete columns; but few provisions exist for prestressed columns, and no provisions exist for slender, prestressed columns. Furthermore, few experimental results exist for prestressed concrete columns, especially those that are slender. Twelve short-term tests of slender, prestressed columns were conducted. The results of the short-term tests were compared against design equations recommended by the PCI Design Handbook[2] as well as an equation from ACI 318 intended for the design of nonprestressed columns.

EXPERIMENTAL PROGRAM

An experimental program[3] was developed to test a range of variables contributing to slender column behavior, and the loading parameters were selected to be practical and realistic while advancing the current boundaries of design. Because of laboratory limitations, the tested columns were small-scale. Figure 1 shows a typical cross-section of the columns. The ties, which included 135 degree standard hooks, were designed in accordance with ACI 318. Three types of prestressing layouts were tested: two, four, and six prestressing wires.

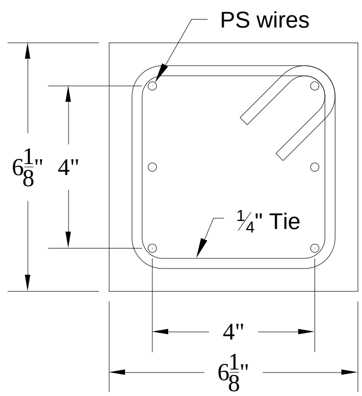

Figure 1: Column Cross-Section

The loading method was simplified to allow for tests that represented theoretical conditions, which correspond better to code provisions. The test setup was designed to simulate the following assumptions: columns braced against sidesway, pinned-pinned loading conditions, and equal end eccentricities.

For the pinned-pinned condition, two steel end plates (1 in. thickness) were fabricated and fastened to the column ends. The steel plates helped to resist the high localized forces caused by the loading and located the loading to allow accurate eccentric loading. Figure 2 shows a view of end plate assembly. As shown, a circular groove was milled into the steel. The groove had a depth of 1/2 in. and a diameter slightly larger than the loading pin. The lowest point of the groove was aligned with the desired eccentricity, measured from the center of the steel plate. A steel rod with a diameter of 1-1/2 in. was used for the pinned end. Based on this setup, the theoretical length of the columns was measured from centerline pin to pin or an additional 1/4 in. from the end of the steel plate on either end of the column.

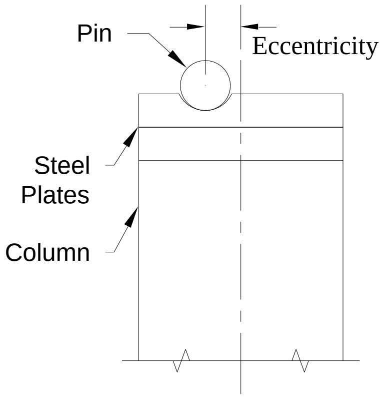

Figure 2: Column End Regions

The loading frame (

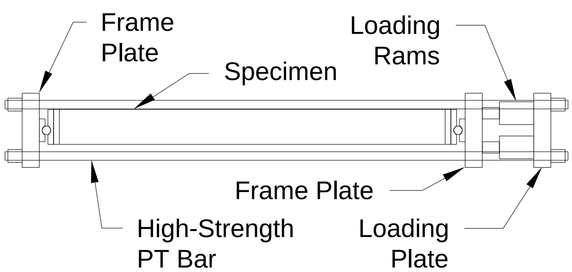

Figure 3), consisted of four post-tensioning bars and three steel plates. The columns were positioned between two steel plates, and the loading rams were positioned between two steel plates. The column was located in the center of the four post-tensioning bars. Using hex nuts to resist the post-tensioning, the loading rams were extended to apply load to the column. To hold the column in place before initial loading, wooden pedestals were used. The pedestals were topped with two separate sheets of Teflon to reduce the influence of friction in the direction of bending.

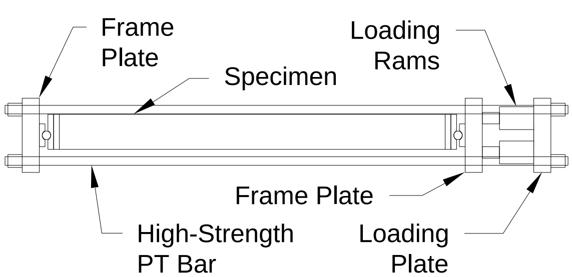

Figure 3: Column Loading Frame

The concrete mix was designed to represent typical structural concrete and have a target compressive strength of 6000 psi. Due to the small scale of the tests, a nominal maximum aggregate size of 3/8 in. was used as well as a high-range water-reducing admixture (HRWRA) to increase flowability while maintaining a low water-cement ratio. The concrete was a five-and-a-half bag mix (517 lb per cubic yd) of Type I cement with a designed water-cement ratio was 0.467. The coarse aggregate was a gradated mix with a maximum aggregate size of 3/8 in. river gravel, and the fine aggregate was a gradated sand mix with 96.6% passing a #4 sieve.

The design of the prestressing was intended to represent typical prestressed columns in service. Because the columns in this project were small-scale, however, standard 0.5 in. or 0.6 in. prestressing strands were not appropriate. Instead, 5.32 mm diameter, steel wire conforming to ASTM A881[4] was used. Similar to typically used prestressing strand, ASTM A881wire is high-strength (≈ 262 ksi) and low-relaxation. Additionally, the wire is indented to improve transfer and development length. The wires were stressed to 75% of the nominal, ultimate capacity of the wires (≈ 196 ksi).

The concrete was allowed to cure for three days before releasing the strands. The strands were released by heating with an oxy-acetylene torch and one at a time in an order that minimized the effective eccentricity. Care was taken to slowly heat the wires to encourage gradual release, but because they were single wires, the release was rather sudden. The wires were torched at one end adjacent to the abutment. After all the wires were released, the wires in gaps between columns were cut with a cutting wheel to ensure there was no residual stress in any section of the wire due to friction between the columns and the plywood formwork base.

Table 1 provides a summary of the tests. The Column ID provides the following information on the column tests: Prestressing (number of wires) – Slenderness ratio (kl_u/r) – Eccentricity Ratio (e/h). For columns braced against sidesway and with equal end moments, ACI 318 permits slenderness effects to be neglected if the slenderness ratio (kl_u/r) is equal to or less than 22. Considering this, any column with a slenderness ratio equal to or less than 22 is considered short. Columns with a slenderness ratio of just greater than 22, though effects cannot be neglected for design, do not exhibit significant slenderness effects for the purpose of this project. Based on preliminary calculations, a lower slenderness ratio of 40 (72 in.) was chosen as a length that begins to show significant effects. The higher value of 70 (126 in.) was chosen as a practical maximum.

Eccentricity ratios ($M/Ph = e/h$) of 10% and 25% were selected, which approximately corresponds to eccentricities of 0.6 in. and 1.5 in. ACI 318 requires the use of a minimum eccentricity to account for out-of-straightness and unknown end conditions. The minimum eccentricity approaches 3% for larger columns but is closer to 6% to 8% for typically sized columns. Based on this evaluation, a smaller value of 10% was selected because initial computational modeling showed smaller eccentricities resulted in negligible second-order effects. The higher eccentricity was selected to be outside of the kern, which introduces tensile stresses in a cross-section immediately upon loading for nonprestressed sections, but the value should not be exceedingly high and thus unlikely to occur in service conditions. Upon consultation with designers and based on preliminary calculations, an eccentricity ratio of 25% was selected. This ratio results in the failure of several columns at or below the balanced point resulting in behavior becoming more similar to beams rather than columns. For this reason, greater ratios are of little value for the investigation of second-order effects.

Table 1: Testing Matrix

Column ID	Reinforcement	Concrete Strength, f_c'	Column Length, in.	Eccentricity, in.
P2-40-10	2 – Prestressing Wires	6570	72	0.6
P2-40-25				1.5
P2-70-10			126	0.6
P2-70-25				1.5
P4-40-10	4 – Prestressing Wires	6370	72	0.6
P4-40-25				1.5
P4-70-10			126	0.6
P4-70-25				1.5
P6-40-10	6 – Prestressing Wires	6570	72	0.6
P6-40-25				1.5
P6-70-10			126	0.6
P6-70-25				1.5

EXPERIMENTAL RESULTS

The results of the short-term tests are summarized by plotting the load-deflection and interaction diagrams. Because of the use of equal end moments, all columns should theoretically deflect symmetrically, with the maximum deflection occurring at the column midspan. As a result, the deflections of the columns were only measured at the column midspans. Figure 4 shows the load versus midspan deflection of the columns with two prestressing wires. Because column design focuses on load-moment interaction, these results were used to create an interaction diagram, which represents the load versus maximum moment in a column.

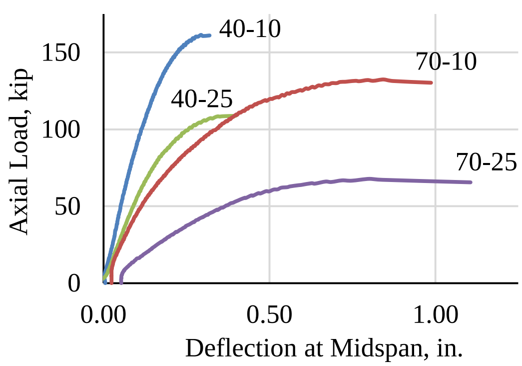

Figure 4: Load v. Deflection (2 – PS Wires)

The moment demands on the columns were computed by multiplying the axial load by the sum of the eccentricity and deflection ($M = P\ (e + \delta)$).The interaction diagrams also plot the nominal strength computed in accordance with ACI 318 and used the as-measured, test-day material properties of the concrete and prestressing steel. The interaction diagram for the columns with two prestressing wires is shown in Figure 5. The model estimated the nominal strength conservatively for all four columns. For the P2-40-25 and P2-70-10 columns, the failure moment was much higher than the computed nominal strength, but the P2-70-10 column approached global instability once it exceeded the nominal strength, shown by zero slope towards the end of its behavior. Because of this, while the failure moment was conservatively estimated, the estimated failure load was more accurate.

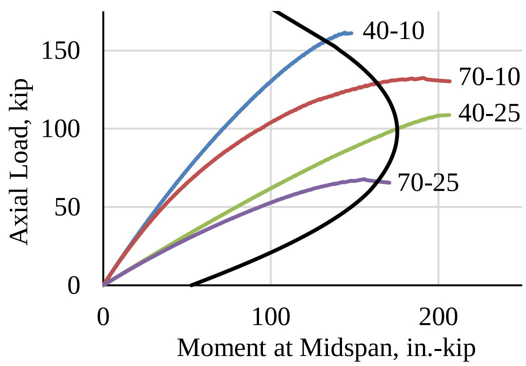

Figure 5: Interaction Diagram (2 – PS Wires)

The interaction diagram for the columns with four prestressing wires is shown in Figure 6. The nominal strength was estimated conservatively for all four columns but was more accurate than for the P2 columns. Also similar to the P2 columns, the P4-70 columns experienced global instability near failure.

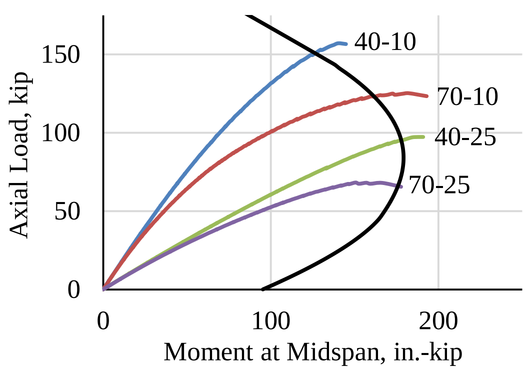

Figure 6: Interaction Diagram (4 – PS Wires)

The interaction diagram for the columns with six prestressing wires is shown in Figure 7. The nominal strength was estimated conservatively for all four columns but was less accurate than when compared to the P2 and P4 column strengths. The P6-40 columns showed greater axial load and moment strength at failure relative to the computations, while the P6-70 columns showed greater moment strength but experienced global instability near failure.

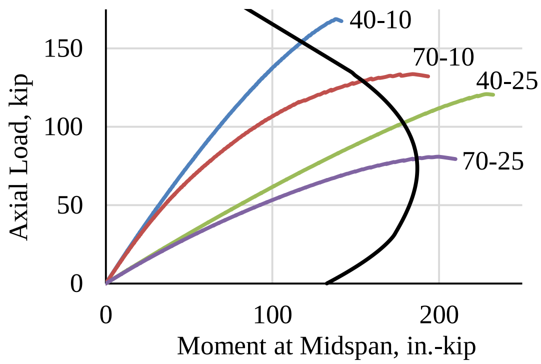

Figure 7: Interaction Diagram (6 – PS Wires)

DESIGN METHODOLOGY

As mentioned, ACI 318[1] includes little guidance on the design of prestressed columns. Most of the provisions provide for the definition of a prestressed column and specify its detailing requirements. In general, columns with an average compressive stress greater than 225 psi, due to effective prestressing force only, do not require the typical, minimum, longitudinal reinforcement. Transverse reinforcement requirements are also modified by eliminating the 16 bar diameters spacing requirements. This compressive stress limit generally defines a prestressed column, and all columns constructed for this experimental program are classified as prestressed columns according to this definition.

EXISTING DESIGN METHODOLOGY

Designers typically refer to the PCI Design Handbook[2] for the design of prestressed construction. The Handbook is not a code but provides guidance on design that is in accordance with ACI 318. Because most prestressed columns do not satisfy the minimum longitudinal steel requirements of ACI 318, the Handbook states the use of the ACI 318 moment magnification procedure is generally not recommended. The Handbook suggests designers use elastic, second-order elastic analysis, but this analysis is typically only satisfactory for the sway effects of frames, not second-order effects due to moment magnification between the column ends.

Consequently, the PCI Committee on Prestressed Concrete Columns[5] provided an alternative method of design. This method, summarized by Equations 1 through 4, computes a flexural stiffness for use in the moment magnification procedure outlined in ACI 318. The Committee explains that these equations, developed by and based on an analytical study, are recommended if using the moment magnification procedure in ACI 318. It should be noted that Equation 4 corresponds with a cross-section without a compression flange. Also, the calculation assumes only prestressed reinforcement is included, or in other words, the use of nonprestressed reinforcement and ducts are not included.

$$EI = E_c I_g / \lambda \tag{1}$$

$$\lambda = \eta\theta \geq 3.0 \tag{2}$$

$$\eta = 2.5 + \frac{1.6}{P / P_0} \tag{3}$$

$$\theta = \frac{27}{kl_u / r} - 0.05 \tag{4}$$

where:

$6 \leq \eta \leq 70$

$A_{pt} =$ total area of prestressing reinforcement, in.^{2}

$E_c = 57{,}000\sqrt{f_c^{'}}$ psi $=$ modulus of elasticity of concrete, psi

$E_p =$ modulus of elasticity of prestressing reinforcement, psi

$f_{se} =$ effective stress in prestressing reinforcement, psi

$I_g =$ moment of inertia of gross concrete section, in.^{4}

$l_u =$ unsupported length of compression member, in.

$P =$ axial force, lb

$P_0 = 0.85 f_c^{'}\left(A_g - A_{pt}\right) - \left(f_{se} - 0.003E_p\right)A_{pt} =$ nominal axial strength, lb

$r =$ radius of gyration of gross cross-section, in.

EVALUATION OF DESIGN METHODOLOGY

The experimental results were used to evaluate the accuracy of the existing design methods by comparing the load-moment results of the tested columns to the behavior computed using the ACI 318 moment magnification procedure. In addition to Equation 1, the behavior was also computed using Equation 5. This equation from ACI 318 is not based on prestressed column behavior; however, the code is not clear regarding its use for prestressed columns. In fact, the code indicates its use for non-composite columns. As a result, this equation could be used by design engineers for prestressed columns due to the ambiguity of the code.

$$EI = 0.4E_c I_g \tag{5}$$

To provide a baseline for the comparison of the results and calculations, the nominal strength was computed in accordance with ACI 318. In addition to the nominal strength, the design strength was also computed using strength reduction factors (ϕ). The strength reduction factors were applied to the nominal axial and moment strengths. Comparing the results and procedure to design level loads provides a perspective of the accuracy of the equations under lower loads that would be the maximum strength considered in design. Actual service loads, due to load factors, would be even lower.

The results of Column P2-70-10 compared with corresponding estimations based on the moment magnification procedure for Equations 1 and 5 are shown in Figure 8. The axial load capacities noted in the figure were determined when the test result or equation estimations passed through the corresponding strength curve (design or nominal). Therefore, the capacity of the test result at nominal strength is shown as this value, not the maximum load achieved during the test. As shown, Equation 5 was more accurate than Equation 1. Equation 1 results in a maximum stiffness that is less than that of Equation 5, but Equation 5 always remained conservative. Equation 1 led to excessively conservative results.

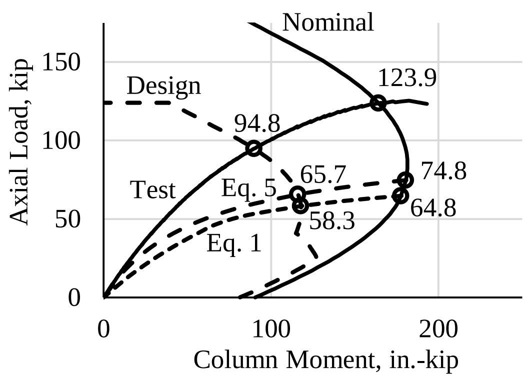

Figure 8: Qualitative Evaluation of Equations

The same calculations and methods of analysis were used to evaluate all of the tested columns, and the capacities were compiled at both nominal and design strengths. Table 2 and Table 3 present the computed capacities of the prestressed, short-term tests at nominal and design strengths, respectively. The ratios of the test capacities to the computed equation capacities are also included. This ratio represents the relative conservatism of the equation to the test result. As such, a value of 1.0 indicates perfect accuracy, while values greater than 1.0 are conservative and values less than 1.0 are unconservative.

Table 2: Equations at Nominal Strength

Column ID	Axial Capacity, kip			P_{test} / $P_{Eq.}$	
	Test	Eq. 1	Eq. 5	Eq. 1	Eq. 5
P2-40-10	159.7	119.8	143.1	1.33	1.12
P2-40-25	102.2	65.8	85.8	1.55	1.19
P2-70-10	130.6	64.4	74.8	2.03	1.75
P2-70-25	67.1	36.9	49.9	1.82	1.34
P4-40-10	152.2	116.8	137.3	1.30	1.11
P4-40-25	96.0	71.2	86.6	1.35	1.11
P4-70-10	123.9	64.8	74.8	1.91	1.66
P4-70-25	65.5	46	53.1	1.42	1.23
P6-40-10	157.2	118.8	138.6	1.32	1.13
P6-40-25	104.9	75.9	90.1	1.38	1.16
P6-70-10	130.9	66.7	76.9	1.96	1.70
P6-70-25	80.5	50.4	56.2	1.60	1.43

Table 3: Equations at Design Strength

Column ID	Axial Capacity, kip			P_{test} / $P_{Eq.}$	
	Test	Eq. 1	Eq. 2	Eq. 1	Eq. 5
P2-40-10	110.2	93.1	102.6	1.18	1.07
P2-40-25	71.4	48.6	62.5	1.47	1.14
P2-70-10	100.7	58.8	66.7	1.71	1.51
P2-70-25	59.0	34.5	44.5	1.71	1.33
P4-40-10	104.7	89.3	97.5	1.17	1.07
P4-40-25	68.5	50.1	62.2	1.37	1.10
P4-70-10	94.8	58.3	65.7	1.63	1.44
P4-70-25	59.2	36.2	43.5	1.64	1.36
P6-40-10	105.8	90.2	97.9	1.17	1.08
P6-40-25	71.4	53.2	64.3	1.34	1.11
P6-70-10	97.7	59.8	67.2	1.63	1.45
P6-70-25	62.4	37.6	45.7	1.66	1.37

The equations estimated the behavior very conservatively. For Equation 1, the ratios range from 17% to 71% conservative. For Equation 5, the ratios range from 7% conservative to 51% conservative. In general, the equations are more accurate for the less slender columns than for the more slender columns. The accuracy and conservativism of the equations did not change significantly between prestressing layouts. For every column, Equation 1 was more than 10% conservative than Equation 5.

Table 4 presents a statistical summary of the capacity ratios for the tested columns at nominal and design strengths. The averages and standard deviations of the ratios are listed, which provide a perspective on the accuracy and conservatism of the equations when compared to the tested columns.

Table 4: Summary of Equation Evaluations

Analysis		Eq. 1	Eq. 5
Nominal Strength, S_n	Average	1.58	1.33
	Std. Dev.	0.26	0.24
Design Strength, ϕS_n	Average	1.47	1.25
	Std. Dev.	0.21	0.16

Equations 1 and 5 computed excessively conservative column capacities at nominal and design strengths. While Equation 5 was approximately 25% conservative at design strength, Equation 1 was even more conservative. This result is expected, however, because Equation 1 permits a maximum stiffness that is lower than the stiffness of Equation 5. While Equation 1 is more complicated than Equation 5, it did not provide additional accuracy and was found to be excessively conservative.

CONCLUSIONS

Twelve slender columns were tested under short-term loading to failure. The columns varied in slenderness ratio, eccentricity ratio, and prestressing arrangement. The test results were compared against calculated values using current design equations. Based on the results of the tests and their comparison to the results of the design methods, the following conclusions were made:

1. Nominal strengths computed in accordance with ACI 318 conservatively estimated the strengths of all columns tested. All columns failed outside of the nominal axial-moment interaction curve.
2. Columns with the higher slenderness (kl_u/r = 70) experienced global instability and zero stiffness before failure.
3. Current design methods were excessively conservative when compared to the test results at both nominal and design strengths. Equations 1 and 5 showed similar standard deviations at design strengths, but Equation 5 was more accurate at this strength level. Both equations have increased accuracy and reduced conservatism at design strength compared to nominal strength. The increase in complexity of Equation 1 compared to Equation 5 did not result in increased accuracy and was excessively conservative. Based on these results, Equation 5 is considered appropriate for the design of prestressed concrete columns. Improved stiffness expressions for the design of prestressed concrete columns, however, are needed if accurate estimates of strength are required.

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