Structural Behavior of Flexural Member with High Strength Concrete

Chuanbing Sun, Ph.D. Candidate, University of Nebraska-Lincoln, Omaha, NE
Amgad Girgis, Ph.D., University of Nebraska-Lincoln, Omaha, NE
Maher K. Tadros, Ph.D., PE, University of Nebraska-Lincoln, Omaha, NE
Sameh Badie, Ph.D., PE, George Washington University, Washington, DC

ABSTRACT

This paper addresses two independent issues related to use of confined high strength concrete in prestressed concrete flexural members: modeling of the concrete stress-strain relationship when the design strength exceeds 10,000 psi, and quantifying the concrete strength and ultimate strain enhancement due to confinement with transverse reinforcement. Extensive parametric analysis in this research as well as other reported research support continued use of the equivalent rectangular stress block for high strength concrete. This paper presents a proposal for modification of the ACI 318 Code rectangular stress block coefficients to allow for modeling of members with concrete strengths up to 16 ksi. Comparisons are given for application of the proposed model and for its impact on flexural strength and reinforcement limits. Comparisons are also given of the results of the proposed model and those by other researchers and in foreign codes.

The second objective of this paper is to investigate the effect of confinement of the compression zone with spiral reinforcement, rectangular ties and structural steel tubes. Confinement is particularly useful in high strength compression and members, which appear to be less ductile than conventional concrete members. The proposed design method is not based on new experimental evidence or fundamental theory. Rather, the extensive research on confinement reported in the literature has been evaluated and combined into a procedure suitable for practice and for inclusion in the codes. The procedure is applicable for all three confinement alternatives: steel tubes, rectangular ties and circular spirals/ties. Numerical examples are given for illustration and comparison.

Keywords: High Performance Concrete, Flexural Member, Confinement, Steel Tube

INTRODUCTION

High-strength concrete is extensively used worldwide. Designers need an appropriate approximation to simulate the compression block or the uniaxial stress-strain relationship. All codes and many researchers are modeling the stress- strain curve using the rectangular block. The rectangular block simulation is very effective and can reduce errors and mistakes they might happen due to the complexity of the stress strain curve. However this leads to the need of more accurate and conservative parameters for the compression block when high strength concrete is in use. This paper is proposing new parameters for the compression block for high strength concrete. Moreover, high-strength concrete is a more "sensitive" material than the traditional concrete due to its brittle nature. Thus, confinement is introduced to increase the ductility of high strength concrete columns, which is desirable especially for those members expected to undergo a large number of inelastic deformation cycles in seismically active regions.[1, 2]

RESEARCH SIGNIFICANCE

When compared to the parameters proposed by code such as ACI318-99 (02),[3] Canadian[4] and New Zealand standards;[5] the proposed equations for calculating the rectangular compression block are shown to present a major improvement in approximating the ultimate concrete strain and β_1 as calculated using nonlinear analysis, both in terms of accuracy and conservatism. For confinement high strength concrete, so far various methods have been developed for structural design. But no one has assembled the available information to propose a uniform and simple design procedure to account for the confinement including the non-circular compression zones as well as spirals or external steel tube confinement.

BACKGROUND

The ACI318-99 (02) Code,[3] as well as previous AASHTO specifications, specify the ultimate concrete strain as a maximum concrete strain of 0.003. The recommendations of the Comite

Euro-International du Beton (CEB) have adopted an ultimate strain of 0.0035. Experimental investigations by Attard and Steward,[6] Ibrahim and MacGregor,[7] and Mendis and Pendyala,[8] have all confirmed the validity of the numerical nonlinear stress-strain model for high strength concrete developed by Popovics.[9] The Canadian[4] and New Zealand[5] Codes recognize reduced ductility as strength increases by assigning a lower concrete strength intensity factor in the equivalent rectangular stress block, while keeping the maximum strain unchanged. However, the CEB proposal for high strength concrete (greater than 7,000 psi) is to gradually reduce the maximum strain from 0.0035 at (7,250psi) to 0.0025 at (14,000psi) or greater.[10, 11]

The proposed formula for the ultimate strain parallels the above CEB proposal. Directly limiting the ultimate is likely to be more effective in recognizing the reduced ductility with the increased strength. It also offers more direct basis for flexural analysis using the Unified Design Method. The proposed formula gives conservative values relative to the European Code and correlates well with results of non-linear analysis.

The neutral axis depth β_1 theoretically should not fall below 2/3. This limit represents the case when the actual compressive stress distribution is rectangular, corresponding to linear stress-strain diagram to the ultimate strain limit. It is known that high strength concrete approaches, but never reaches, this limiting condition as strength increases. Accordingly, It was decided to limit β_1 to 0.7. This is close to the 0.67 value in the Canadian Code.[4] The proposed equation for β_1 gives a smoother transition from conventional concrete to high strength concrete than the current relationships. It is also well-correlated with nonlinear stress analysis.

As far as the confined concrete is concerned, the ACI 318-99 code has proposed the formula for the required confinement reinforcement as:

$$\frac{4A_{sp}}{d_{sp}s} \geq 0.42\frac{f_{c0}}{f_{sp}}\left(\frac{A_g}{A_{core}} - 1\right) \tag{1}$$

However, the arrangement of transverse reinforcement and the other types of confinement steel rather than Grade 60 rebar are not taken into account in the current ACI 318 building code, which resulted in either unsafe or over-conservative design approach. Basically, the over-conservative design leads to highly congested confinement reinforcement and concrete placement problems.

STRESS-STRAIN TESTING FOR UNCONFINEMENT CONCRETE

The response of concrete in uniaxial compression can usually be determined by loading a 6 in. diameter, 12 in. long cylinder with a rate of loading of approximately 1,000 lb/sec. The formula that describes the stress-strain relationship was originated by Popovics[9] in 1973. This formula went through several subsequent modifications by Collins and Porasz[12] and Collins and Mitchell.[13]

HISTORY OF RECTANGULAR BLOCK

After Popovics[9] had come up with his formula that described the stress- strain relationship in 1973, there were several attempts to modify this formula by Collins and Porasz[12] and Collins and Mitchell.[13] Several researchers tried to simplify the nonlinear behavior of the stress-strain curve to a rectangular block, including Attard and Foster,[14] Ibrahim and MacGregor[7] and Mendis and Pendyala[8] in 1997; Attard and Steward[6] in 1998, AFREM-95.[10] Several Codes[3,4,5] are also using the simplified rectangular compression block approach.

HEIGH STRENGTH CONCRETE BEHAVIOR

Recent advances in concrete technology have made it possible both technically and economically to obtain high strength concrete up to 16,000 psi. The behaviors of high and low strength concrete are very different. As shown in Figure 1, the higher the concrete strength, the faster the post-peak curve will be. Consequently, high strength concrete reaches 0.85 f_c' significantly faster than the lower strength concrete. Moreover, there is a smaller difference between the ultimate concrete strain and the concrete strain at the maximum stress in high strength concrete than in lower strength concrete.

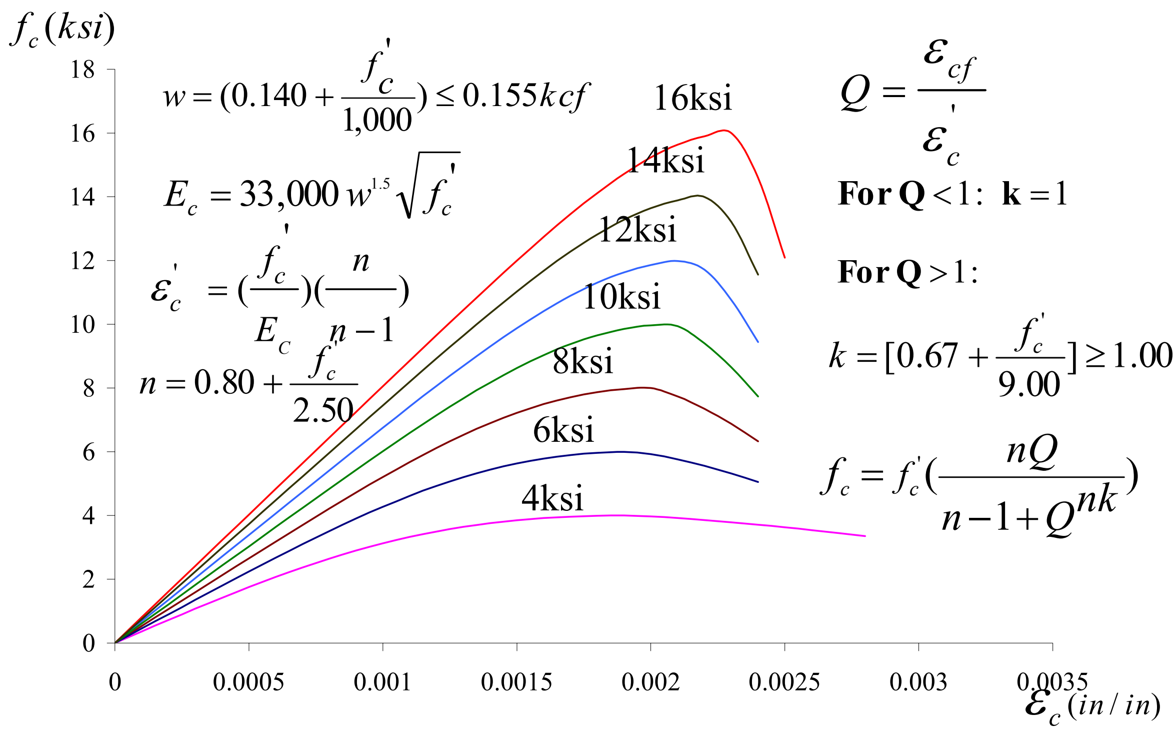

Figure 1 Concrete Stress-Strain Curves Using Popovics Formula

POPOVICS[9]

The response of concrete in uniaxial compression is usually determined by loading a 6 in. (150mm) diameter, 12 in.(300mm) long concrete cylinder, so that the maximum stress, f_c', is reached in 2 to 3 minutes. A convenient expression, which accurately describes the shape of the rising branch of the concrete cylinder stress-strain curve, was proposed by Popovics.[9] This expression is:

$$\frac{f_c}{f_c'} = \frac{\varepsilon_{cf}}{\varepsilon_c'} \frac{n}{n - 1 + \left(\varepsilon_{cf} / \varepsilon_c'\right)^n} \qquad (2)$$

where

f_c = Compressive stress

f_c' = Maximum compressive stress

ε_{cf} = Compressive strain

$\varepsilon_c^{'}$ = Strain when f_c reaches $f_c^{'}$

n = Curve fitting factor, as n becomes higher the rising curve become more linear

In 1987, Thorenfeldt, Tomaszewicz, and Jensen[15] suggested adding the factor k to Popovics' equation to better describe the post-peak decay. Equation (3) represents their modified version of equation (2).

$$\frac{f_c}{f_c^{'}} = \frac{\varepsilon_{cf}}{\varepsilon_c^{'}} \frac{n}{n - 1 + \left(\varepsilon_{cf} / \varepsilon_c^{'}\right)^{nk}} \tag{3}$$

Where k equals to 1 when $\varepsilon_{cf} / \varepsilon_c^{'}$ is less than 1 and k is greater than 1 when $\varepsilon_c / \varepsilon_c^{'}$ exceeds 1.

Collins and Poraz[12] and Collins and Mitchell[13] suggested that for $\varepsilon_{cf} / \varepsilon_c^{'} > 1$

$$k = [0.67 + \frac{f_c^{'}}{9.00}] \geq 1.00 \quad \text{ksi units} \tag{4}$$

and that

$$n = 0.80 + \frac{f_c^{'}}{2.50} \quad \text{ksi units} \tag{5}$$

CODE EQUATIONS FOR UNCONFINED CONCRETE

This section reviews the parameters proposed by some of the current codes for the concrete rectangular block.

1. ACI318-99 (02) without modification for high strength concrete

$$\varepsilon_{cu} = (3000)10^{-6} = 0.003 \tag{6}$$

$$\alpha = 0.85 \tag{7}$$

$$\beta_1 = (0.85 - 0.05(f_c^{'} - 4)), \quad \text{Where} \quad 0.65 \leq \beta_1 \leq 0.85 \tag{8}$$

2. Canadian Standards[4]

$$\varepsilon_{cu} = .0035 \tag{9}$$

$$\alpha = 0.85 - 0.0015 f_c^{'} \geq 0.67 \tag{10}$$

$$\beta_1 = 0.97 - 0.0025 f_c' \geq 0.67 \quad (11)$$

3. Standards New Zealand[5]

$$\varepsilon_{cu} = (3000)10^{-6} = .003 \quad (12)$$

$$\alpha = 1.07 - 0.004 f_c' \text{, Where } 0.75 \leq \alpha \leq 0.85 \quad (13)$$

$$\beta_1 = 1.09 - 0.008 f_c' \quad (14)$$

THE PROPOSED EQUATIONS FOR UNCONFINED HSC

The proposed parameters for the concrete rectangular block are as follows:

$$\varepsilon_{cu} = \frac{0.033}{7 + f_c'} \text{, Where } 0.0022 \leq \varepsilon_{cu} \leq 0.0030 \quad (15)$$

$$\beta_1 = \frac{28}{30 + f_c'} \text{, Where } 0.70 \leq \beta_1 \leq 0.85 \quad (16)$$

$$\alpha = 0.85 \quad (17)$$

COMPARISONS OF THE PROPOSED METHOD WITH THE CURRENT METHODS AND MODIFIED POPOVICS FOR UNCONFINED CONCRETE

This section presents detailed comparisons between the proposed parameters of the compression block and those proposed by other codes. The proposed ultimate concrete equation result is significantly closer to that of the Popovics formula than that of the recent codes. At 7 ksi, which is an economic mix, the result from the proposed equation is the same as the Popovics formula, as shown in Figures 2 .

Figures 3 show that the proposed equation for β_1 result is also significantly closer to the Popovics formula than that of the recent codes. At 8 ksi, which is an economic mix, the result from the proposed equation is the same as the Popovics formula.

$$a_c = \frac{\text{Area under Stress} - \text{Strain curve}}{f_c' \varepsilon_{cu}} \quad (18)$$

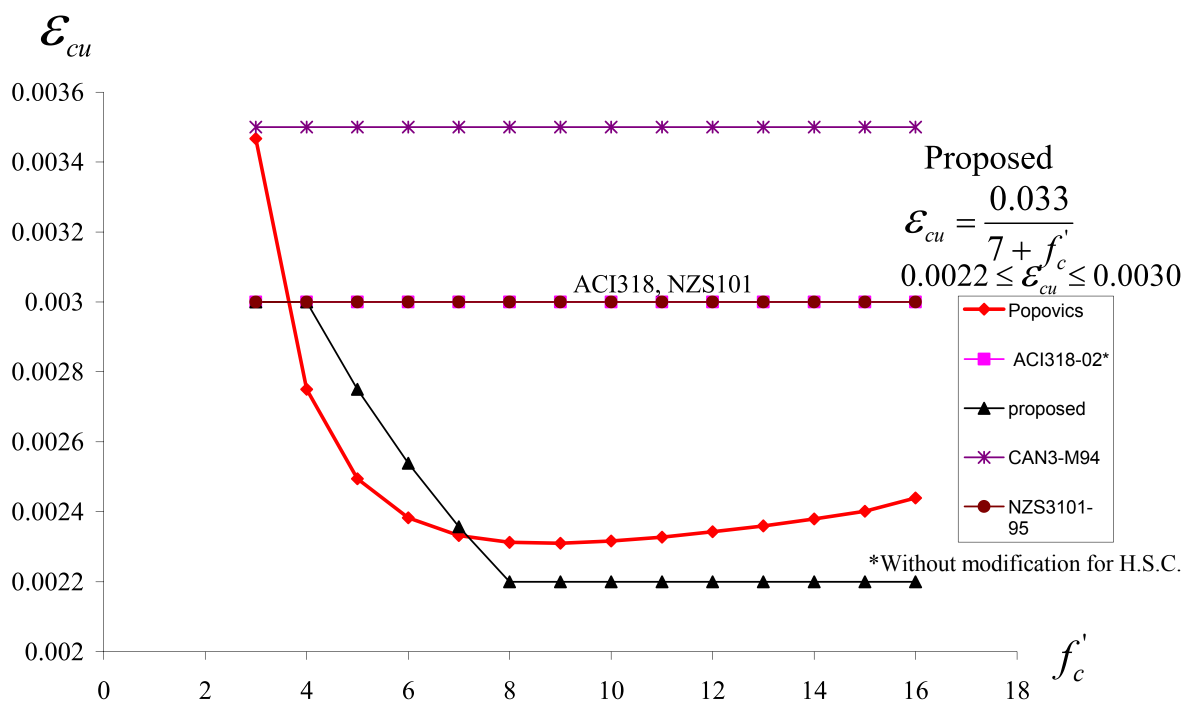

Figure 2 Comparison of proposed Ultimate Strain with Current Codes and with Popovics Nonlinear Curves

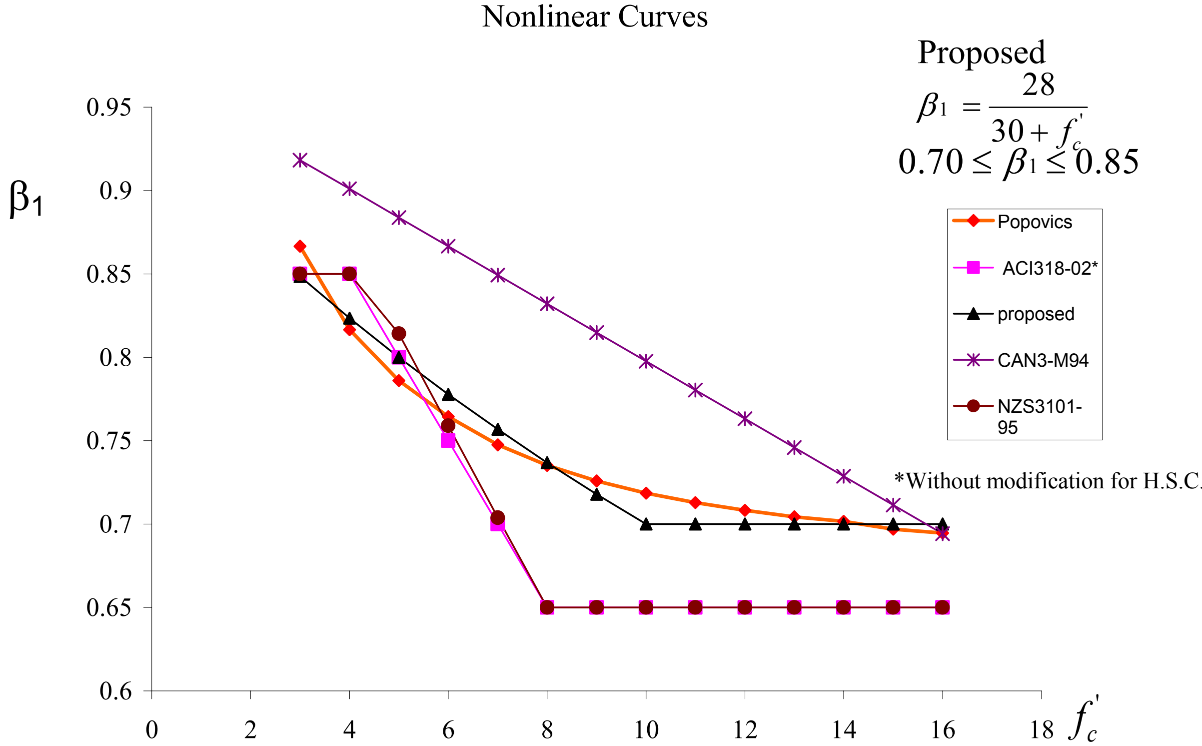

Figure 3 Comparison of proposed β_1 with Current Codes and with Popovics Nonlinear Curves

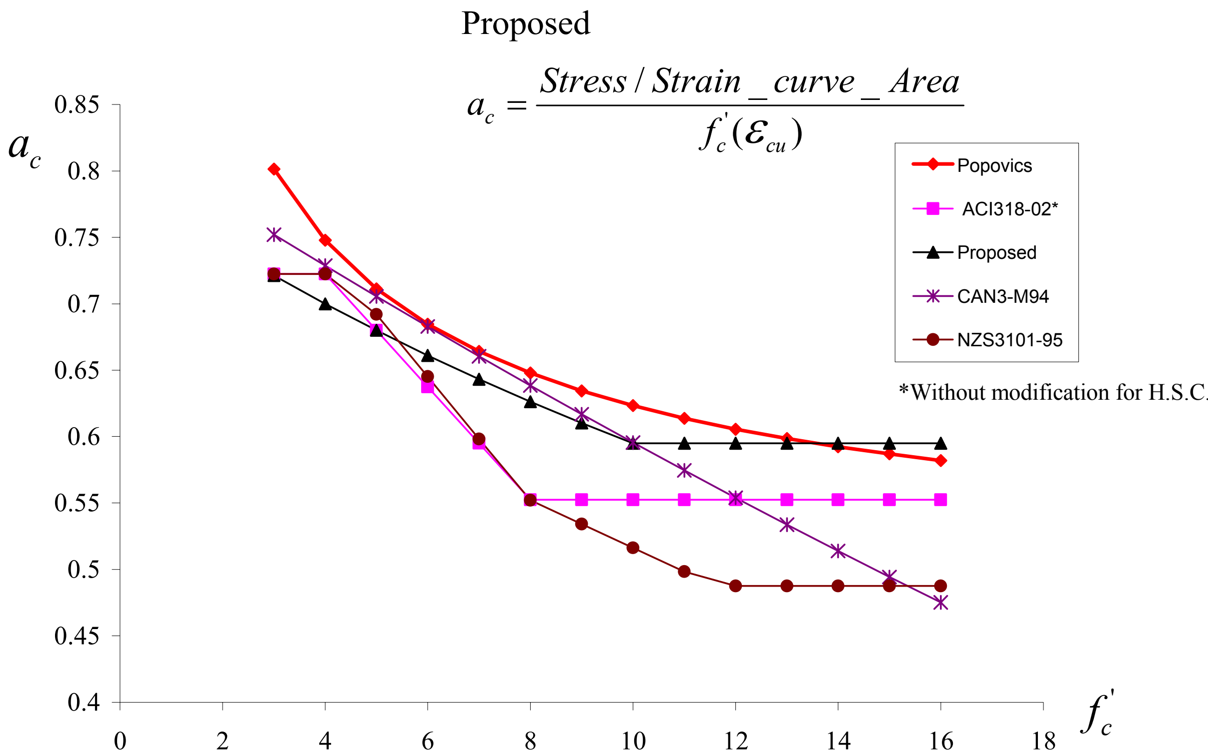

Figure 4 Comparison of proposed a_c with Current Codes and Popovics Nonlinear Curves

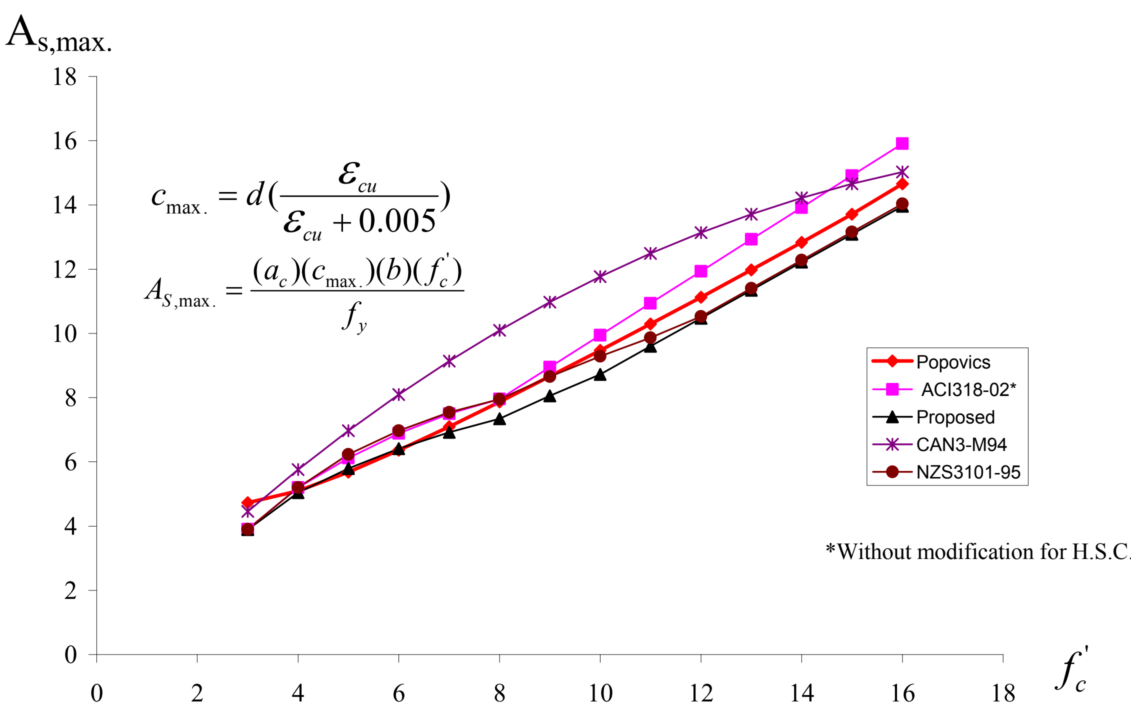

Figure 5 Comparison of proposed $A_{s,max.}$ with Current Codes and Popovics

Figure 4 shows that the proposed equation gives a normalized area result that is close to Popovics beyond 10 ksi concrete strength. However, for concrete strength less than 10 ksi, the Canadian code gives a slightly closer result.

Maximum reinforcement is calculated based on the proposed parameters for the rectangular block and based on the current codes and other researchers. The reinforcement strain is taken as 0.005. Figure 5 shows a comparison between the maximum reinforcement obtained from the proposed equation and that of the current codes. This figure clearly indicates that the proposed equation gives a more conservative maximum steel reinforcement compared to Popovics. It is worth saying that the ACI318-99 (02)[3] and the Canadian codes[4] give non-conservative results. The New Zealand code[5] gives non-conservative results from 4 ksi to 8 ksi concrete strength.

DESIGN CRITERION OF ACI 318-99 (02) CODE FOR CONFINEMENT CONCRETE

The ACI 318-99 (02)[3] assumes that a column essentially fails when cover spalls off. The capacity of confined concrete core alone need not be greater than that of total unconfined section, which is the basis for the following equation:

$$A_{core}\left(0.85 f_{c0} + 4.1\frac{2A_{sp}f_{sp}}{d_{sp}s}\right) = 0.85 f_{c0} A_g \tag{19}$$

Solving for A_{sp}:

$$\frac{4A_{sp}}{d_{sp}s} \geq 0.42\frac{f_{c0}}{f_{sp}}\left(\frac{A_g}{A_{core}} - 1\right)$$

This is the same as that in ACI code except 0.42 is increased to 0.45. The confinement effect due to spiral is shown as the equation:

$$f_{c2} = f_{c0} + 4.1 f_{22} \tag{20}$$

Where:

f_{c2} = confined concrete strength with confinement;

f_{c0} = unconfined concrete strength;

f_{22} = lateral confinement pressure;

Figure 6 illustrates the increase of the concrete strength due to the lateral confinement strength f_{22}. According to the past research, it is concluded that columns with the same amount and spacing of confinement reinforcement showed significantly strength and deformability when confined by the different arrangements of transverse reinforcement. [2] The formulas given in ACI 318-99 building code did not include the arrangement and the variable strength of the confinement steel, which resulted in unsafe or over-conservative design approach.

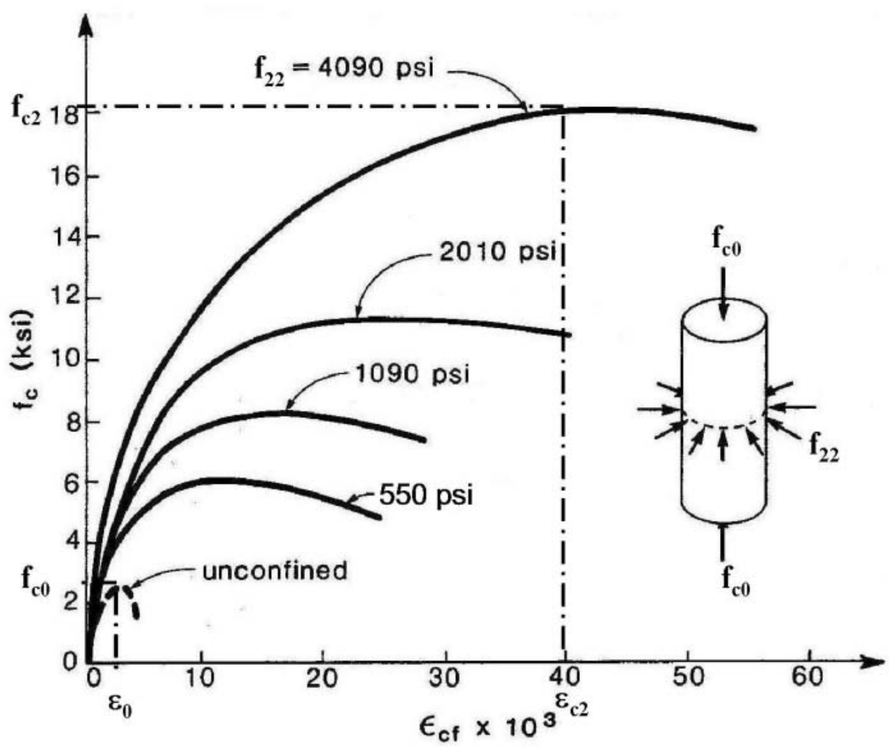

Figure 6. Lateral confinement concrete strength [13]

THE PROPOSED UNINFORM EQUATIONS FOR CONFINED CONCRETE

According to the ACI 318-99 building code and the latest research done by Pessiki[15], Saatcioglu[16], etc., a possible uniform equation and design procedure was proposed to account for the confined concrete. The proposed equation takes into account for both circular section and non-circular section. In connection with circular section, no change is made for the confined concrete strength in comparison with the current ACI code. Related to the confined non-circular section, the proposed equation is given by simplifying the formula suggested by Saatcioglu[2], where the amount, grade, spacing and arrangement of transverse reinforcement were considered as parameter of confinement.

The confined concrete strength is taken as:

$$f_{c2} = f_{co} + 4.1kf_{22} \tag{21}$$

where:

k = 1.0 for circular section confined with spiral or steel tube (22)

$$k = 0.17\left(\frac{b_c}{s}\frac{b_c}{s_l}\right)^{0.4} f_{22}^{-0.6} \quad \text{for rectangular sections} \tag{23}$$

$$f_{22} = \frac{\sum A_s f_s}{sb_c} \tag{24}$$

For external tube, f_{22} can be simplified by substitute (t)(s) for A_s,

$$f_{22} = \frac{2t}{b_c} f_s \tag{25}$$

k = confinement efficient factor.

b_c = core dimension, center-to-center of perimeter tie, see Figure 7 (a) & (b)

s_l = center-to-center spacing of longitudinal reinforcement, laterally supported by corner of hoop or hook of crosstie [2]

f_{22} = lateral confinement strength.

s = pitch of confined reinforcement

t = steel pipe thickness

A_s = area of confinement steel

f_s = confinement steel strength

Note that, for non-circular sections, the smaller of the two "kf_{22}" values from the two direction of the section should be applied. One added requirement for using the equation (21), except for the steel tube confinement case, is to guarantee that: $A_g/A_c \geq 1.3$, where A_g is the gross area of column concrete section and A_c is the area of core concrete within perimeter transverse reinforcement according to the research by Razvi[16].

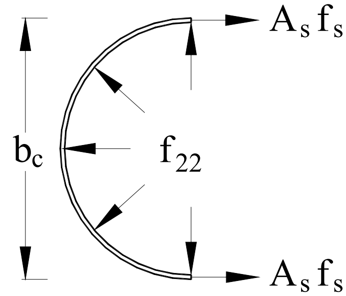

Figure 7-(a). Steel tube confined with concrete

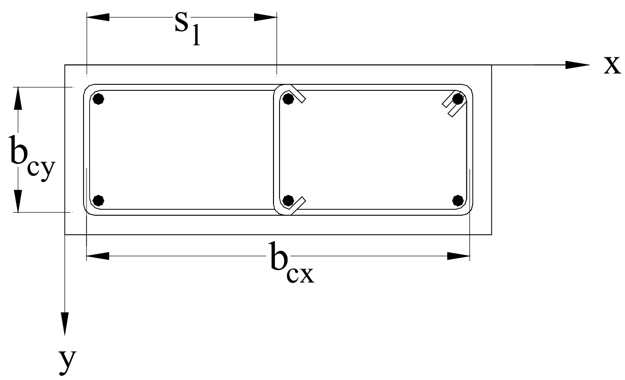

Figure 7-(b). Confined rectangular section

.

In equation (21), the actual confinement steel strength instead of the yield strength is incorporated considering the implementation of high-strength confinement steel and the fact that the high strength steel is very possibly not yielded yet before the column collapsed.

According to the research done by Richart and Brown[17], the equation that relates to the longitudinal strain at the confined concrete strength ε_{c2} to the confined strength f_{c2}, the unconfined strength f_{c0}, and the corresponding unconfined longitudinal ε_{c0} is as follows:

$$\varepsilon_{c2} = \varepsilon_{c0}\left(5\frac{f_{c2}}{f_{c0}} - 4\right) \tag{26}$$

ε_{c2} - longitudinal strain at the confined concrete strength;

ε_{c0} - longitudinal strain at the unconfined concrete strength;

Further, the transverse strain is a function of axial strain, ε_{c0}, and degree of confinement, shown as equation (27). This equation is given according to the work by Pessiki[15], which was done based on to the spiral confinement.

$$\varepsilon_{ct2} = \varepsilon_{c0}\left(0.3 + 17\frac{A_s f_s}{b_c s f_{c0}}\right) \quad (27)$$

This equation is extended to analyze the non-circular section so that the actual confinement steel strength, f_s, can be obtained according to the steel stress-strain diagram. For high-strength steel, the power formula provided by Tadros and Skogman [18] can be applied to get the steel strength with the transverse strain from equation (27).

DESIGN STEPS FOR HSC WITH CONFINEMENT

A possible design procedure is illustrated as follows to analyze a steel pipe filled with high strength concrete. And the Popovics formula is applied to obtain the confined concrete stress-strain relationship with the given confined steel tube dimension and material properties. Similar procedure can be applied to the other types of confinement steel.

Step 1: Evaluate ε_{ct2} by assuming $f_s = f_y$.

$$\varepsilon_{ct2} = \varepsilon_{c0}\left(0.3 + 17\frac{A_s f_s}{b_c s f_{c0}}\right)$$

Where: $f_{c0} = f_c^{'}$

$$\varepsilon_{c0} = (\frac{f_c^{'}}{E_c})(\frac{n}{n-1})$$

$$n = 0.80 + \frac{f_c^{'}}{2.50}$$

$$w = (0.140 + \frac{f_c^{'}}{1{,}000}) \le 0.155 kcf$$

$$E_c = 33 w^{1.5}\sqrt{f_c^{'}}/1000$$

Step 2: Check the assumption in Step 1.

$$f_s = E_s \varepsilon_{ct2} \le f_y$$

Notice that power formula [17] is recommended for high strength confinement steel.

Step 3: Reiterate step 1, if necessary.

Step 4: Determine the lateral confinement strength.

$$f_{22} = \frac{2t}{b_c} f_s$$

Step 5: Determine the confined concrete strength.

$$f_{c2} = f_{c0} + 4.1kf_{22}$$

Step 6: Determine the confined concrete longitudinal strain.

$$\varepsilon_{c2} = \varepsilon_{c0}\left(5\frac{f_{c2}}{f_{c0}} - 4\right)$$

Step 7: Use f_{c2} and ε_{c2} in place of f'_c and ε'_c in Popovics formula to determine confined concrete stress-strain relationship. Note that $n=E_c/(E_c-E'_c)$; E_c is given in step 1. And $E'_c=f_{c2}/\varepsilon_{c2}$.

NUMERICAL EXAMPLE FOR HSC WITH CONFINEMENT

A numerical example is given to show the procedure of analyzing a given steel tube filled with concrete. With the given tube dimension (see Figure 8), steel and concrete material properties, and unconfined concrete stress-strain relationship (shown in Figure 9), to determine the confined concrete strength and the confined concrete stress-strain relationship. The proposed solution is given herein.

Step 1: Assume $f_s = f_y = 50$ ksi

From $\varepsilon_{ct2} = \varepsilon_{c0}\left(0.3 + 17\frac{A_s f_s}{b_c s f_{c0}}\right)$

Substitute (t)(s) for A_s, then

$$\varepsilon_{ct2} = \varepsilon_{c0}\left(0.3 + 17\frac{tf_s}{b_c f_{c0}}\right)$$

Where

$f_{co} = f'_c = 8$ ksi

$$\varepsilon_{c0} = \left(\frac{f'_c}{E_C}\right)\left(\frac{n}{n-1}\right)$$

$$w = \left(0.140 + \frac{f'_c}{1{,}000}\right) = \left(0.140 + \frac{8}{1{,}000}\right) = 0.148 \le 0.155 \quad kcf$$

$$\varepsilon_{c0} = \left(\frac{f'_c}{E_C}\right)\left(\frac{n}{n-1}\right) = \left(\frac{8}{5{,}314.37}\right)\left(\frac{4.0}{4.0-1}\right) = 0.00201$$

$$E_c = 33w^{1.5}\sqrt{f_c^{'}}/1000 = 33(148)^{1.5}\sqrt{8{,}000}/1{,}000 = 5{,}314.37 \quad ksi$$

$$n = 0.80 + \frac{f_c^{'}}{2.50} = 0.80 + \frac{8}{2.50} = 4.0$$

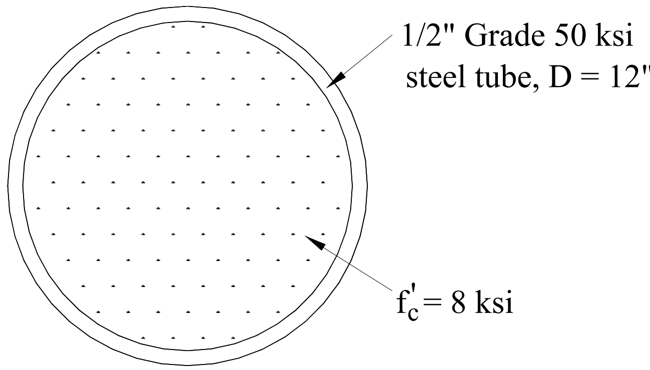

Figure 8. Steel tube filled with concrete

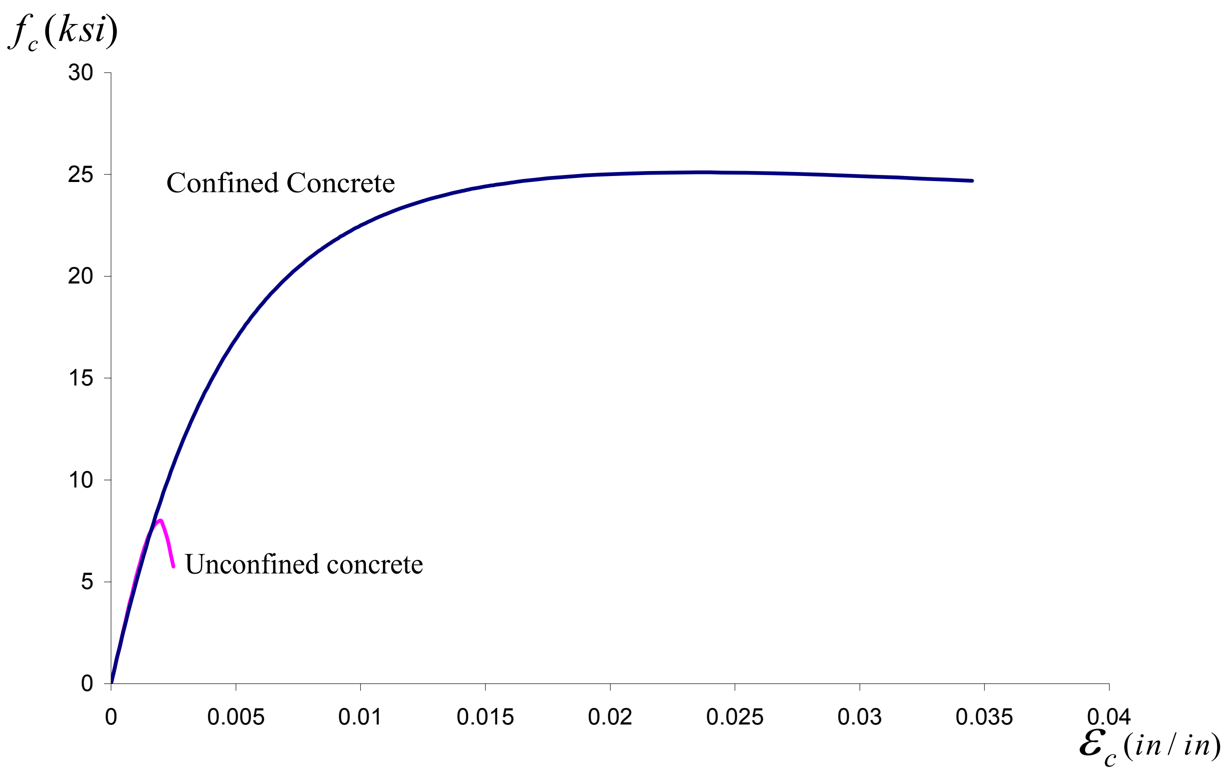

Figure 9 Stress-strain relationships using Popovics Formula for confined and unconfined concrete

Step 2: $f_s = E_s \varepsilon_{ct2} = 29{,}000 \times 0.00950 = 275.50 \quad ksi > f_y = 50 \quad ksi$

so, use $f_s = f_y = 50$ ksi

Step 3:

$$f_{22} = \frac{2t}{b_c} f_s = \frac{2 \times 1/2}{12} \times 50 = 4.17 \quad ksi$$

Step 4: $f_{c2} = f_{c0} + 4.1kf_{22} = 8 + 4.1(1)(4.17) = 25.10 \quad ksi$

Step 5: $\varepsilon_{c2} = \varepsilon_{c0}\left(5\frac{f_{c2}}{f_{c0}} - 4\right) = 0.00201\left(5 \times \frac{25.10}{8} - 4\right) = 0.02349$

Step 6: $f_c = f_{c2}\left(\frac{nQ}{n - 1 + Q^n}\right)$

Where

$$E_c^{'} = \frac{f_c^{'}}{\varepsilon_c^{'}} = \frac{25.10}{0.02349} = 1{,}068.54 \quad ksi$$

$$n = \frac{E_c}{E_c - E_c^{'}} = \frac{5{,}314.37}{5{,}314.37 - 1{,}068.54} = 1.25$$

$$Q = \frac{\varepsilon_{cf}}{\varepsilon_{c2}} = \frac{\varepsilon_{cf}}{0.02349}$$

$$f_c = 25.10 \times \frac{1.25\left(\frac{\varepsilon_{cf}}{0.02349}\right)}{1.25 - 1 + \left(\frac{\varepsilon_{cf}}{0.02349}\right)^{1.25}} = \frac{31.38\left(\frac{\varepsilon_{cf}}{0.02349}\right)}{0.25 + \left(\frac{\varepsilon_{cf}}{0.02349}\right)^{1.25}}$$

Figure 9 shows the stress-strain relationship for the confined concrete in comparison with the unconfined concrete.

CONCLUSIONS AND RECOMMENDATIONS

Based on these comparisons the following can be concluded:

1. New rectangular stress block parameters were proposed and compared to the non-linear Popovics equations and other researchers and current codes.
2. The proposed ultimate concrete equation result is significantly closer to the Popovics formula than that of the recent codes.

3. The proposed equation for β_1 result is significantly closer to the Popovics formula than that of the recent codes.
4. The proposed parameters give conservative results in calculating the maximum steel reinforcement, while most of the parameters of the current code have non-conservative results compared to the Popovics nonlinear solution.
5. A uniform equation was proposed to account for the confined high strength concrete for both circular and non-circular sections. A possible design procedure as well as a numerical example was provided for the confined high strength concrete.

REFERENCES

1. Nehikhare Lambert-Aikhionbare and Sami W. Tabsh; "*Confinement of High-strength Concrete with Welded Wire Reinforcement*". ACI Structural Journal, September-October 2001, pp. 677
2. Murat Saatcioglu, et al. "*Displacement-Based Design of Reinforcement Concrete Columns for Confinement*". ACI Structural Journal Jan.-Feb. 2002
3. ACI Committee 318, *Building Code Requirements for Structural Concrete (318-99,02) and Commentary (318R-99,02)*, American Concrete Institute, Farmington Hills, MI, 1999.
4.Canadian Standards Associations, "*CSA A23.3-94 Design of Concrete Structures in Buildings,*" Rexdale, Ontario, 1994,199 pp.
5 .Standards New Zealand, "*NZS3101 Design of Concrete Structures*," Wellington, N.Z., 1995.
6.Attard, M.M. and Steward, M.G., "*A Two parameter stress block for High-Strength Concrete*," ACI Structural Journal, Vol. 95, No. 3, May-June 1998, pp. 305-317.
7. Ibrahim, H.H.H. and MacGregor, J.G., "*Modification of the ACI Rectangular Stress Block for High-Strength Concrete*," ACI Structural Journal, Vol. 94, No. 1, January-February 1997, pp. 40-48.
8. Mendis, P.A. and Pendyala, R.S., "*Structural Design with High- strength/ High Performance Concrete-Beams and Columns*," Concrete in Australia, Vol. 23, No. 2, 1997, pp. 26-28.
9. Popovics, S., "*A Numerical Approach to the complete Stress-Stain Curve of Concrete*," Cement and Concrete Research, V.3, No. 5 May 1973, pp. 583-599.
10. AFREM-95, *Proposition d'Annexe BHP du Reglement Bael*, Final Report, Connaissance et Utilisation des BHP, LCPC, Paris, 1997.
11. "*High Performance Concrete, Recommended Extensions to the Model Code90 an Research Needs*," Committee Euro-International du Beton (CEB), Bulletin No. 228, July 1995, Lausanne, Switzerland, 4pp.
12. Collins, M., and Porasz, A., "*Shear Design for High Strength Concrete*," CEB Bulletin d'Information, No. 193, Dec. 1989, pp. 77-83.

13. Collins, M., and Mitchell, D., *Prestressed Concrete Structures*, Prentice-Hall Inc., Englewood Cliffs, New Jersey, 1991, 766 pp.
14. Attard, M.M. and Foster, S. J., "*The Effect of Cover on The Strength of High Strength Concrete*," pp. 127-136.
15. Stephen Pessiki, et al. "*Proposed Design of High-Strength Spiral Reinforcement in Compression Members*". ACI Structural Journal Nov-Dec. 2001.
16. Razvi, S. R. and Saatcioglu, M., "*Design of R/C Columns for Confinement Based on Lateral Drift*," Report OCEERC 96-02, Ottawa-Carleton Earthquake Engineering Research Center, Department of Civil Engineering, University of Ottawa, Canada, pp 92
17. Richart, F.E., Brown R. L., "*An Investigation of Reinforced Concrete Columns*", University of Illinois Bulletin, No. 40, 1934
18. Skogman, C. B., Tadros, M. K., and Grasmick, R., "*Flexural Strength of Prestressed Concrete Members*," PCI JOURNAL, V.33, No.5, Sept.-Oct. 1988, pp. 96-123.