International Journal of Concrete Structures and Materials
Vol.18, No.1E, pp.11~16, June 2006

Concrete Stress Block Parameters for High-Strength Concrete : Recent Developments and Their Impact

Sun-Gjin Bae[1)]

(Received September 3, 2005, and accepted May 30, 2006)

Abstract: The use of the current ACI 318 stress block parameters has been reported to provide unconservative estimations of the moment capacities for high-strength concrete columns. Accordingly, several concrete stress block parameters have been recently proposed. This paper discusses various concrete stress block parameters for high-strength concrete and their influences on the code provisions. In order to adopt the proposed stress block parameters to the design code, it is necessary to understand the impact of the change of the stress block parameters on various aspects of the code provisions. For this purpose, the influence of using of different stress block parameters on the location of the neutral axis and the tensile strain in extreme tension steel as well as the axial and moment capacities are investigated. In addition, the influence on the prestressed concrete members is also elucididated.

Keywords: stress block, high-strength concrete, cover concrete, premature spalling.

1. Introduction

To reduce the section size of a column, the use of high-strength concrete (f_{ck} > 55 MPa) has been increasing steadily in the last two decades. This necessitates a careful examination of the current ACI code provisions regarding nominal moment capacity calculations for high-strength concrete (HSC) columns, as they are primarily based on the experimental data obtained from normal-strength concrete (NSC) column tests.

Several researchers proposed concrete stress block parameters for HSC to consider the different nature of stress-strain relationship and brittleness of HSC, and some of were adopted by design codes.[1,2] Bae and Bayrak[3] reported that the presence of large amount of transverse reinforcement required for HSC columns could contribute to the premature spalling of cover concrete.

The American Concrete Institute formed a task group (ACI 318 TG-5) to investigate and develop new concrete stress block parameters after recognizing this problem of selecting the best concrete stress block parameters for HSC.

This paper discusses various proposals for better concrete stress block parameters, their influences on the estimated moment capacity, the depth of the neutral axis, and the tensile strain in extreme tension steel.

2. Research significance

The use of the current ACI stress block parameters has been reported to provide unconservative estimations of the moment capacities for HSC columns. As the result, the building codes of Canada and New Zealand have already changed their stress block parameters based on experimental/analytical investigations. The ACI also formed a task group to develop new stress block parameters for HSC. Therefore, it is urgent to investigate various proposed stress block parameters for HSC and their influences on the design practice.

3. Previous research

3.1 ACI 318-05 provisions 2005

The concept of the equivalent rectangular concrete stress block was first introduced by Whitney[5] and later experimentally verified by Hognestad et al.[6] and Mattock et al.[7] Mattock et al. conducted an investigation of eccentrically loaded unreinforced concrete columns of NSC (f_{ck} < 55 MPa). Nedderman[8] conducted tests on eccentrically loaded unreinforced concrete columns of concrete strengths ranging between 79 and 98 MPa and proposed a lower limit of 0.65 for β_1 in case of concrete strengths in excess of 55 MPa. This limit was well taken a long time ago to be incorporated into the 1977 version of the ACI code.[9]

Fig. 1 illustrates the rectangular stress block parameters of ACI

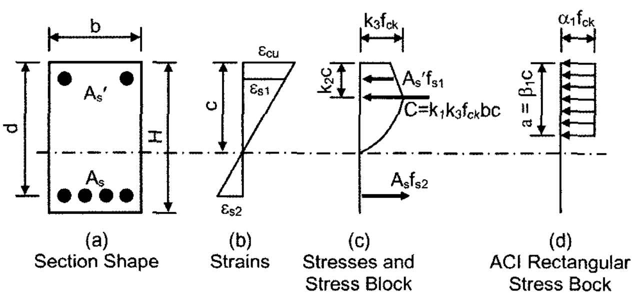

Fig. 1 Rectangular stress block.

[1)] KCI member, Dept. of Civil Engineering, The University of Texas at Austin, USA. *E-mail: sjinbae@yahoo.com*

318-05. The intensity of the equivalent stress block is given by $\alpha_1 f_{ck}$. The depth of the stress block is $\beta_1 c$, where c is the neutral axis depth. According to ACI 318, α_1 is assumed to have a constant value of 0.85. β_1 is equal to 0.85 for concrete strengths up to 30 MPa and is reduced gradually at a rate of 0.08 for each 10 MPa of concrete strength in excess of 30 MPa. β_1 is equal to 0.65 for concrete strengths greater than 55 MPa. The ultimate compressive strain ε_{cu} (Fig. 1(b)) is assumed to have a constant value of 0.003 for all concrete strengths.

$$\alpha_1 = 0.85 \ (f_{ck} \text{ in MPa}) \quad (1)$$

$$\beta_1 = 0.85 - 0.008\,(f_{ck} - 30) \quad (2)$$
in which $0.65 \le \beta_1 \le 0.85$

3.2 Ibrahim and MacGregor 1997

Ibrahim and MacGregor proposed a new set of rectangular stress block parameters that can be applied for both NSC and HSC. They tested 20 HSC column specimens of no or less confinement reinforcement than that required by Chapter 21 of ACI 318-05. The parameter α_1 was derived to provide a conservative lower bound for the experimental data on k_3, while the value of β_1 was derived as an average value of their test data. Eqs. (3) and (4) illustrate the recommendation of Ibrahim and MacGregor.

$$\alpha_1 = 0.85 - \frac{f_{ck}}{800} \ge 0.725 \ (f_{ck} \text{ in MPa}) \quad (3)$$

$$\beta_1 = 0.95 - \frac{f_{ck}}{400} \ge 0.70 \quad (4)$$

3.3 Li et al. 1994

The shape of the stress-strain relationship for HSC is reasonably similar to a triangular stress distribution with peak stress occurring at a strain of about 0.003. With the assumption that the maximum stress of the triangular stress block in HSC is f_{ck}, the equivalent rectangular stress block can be calculated to have the same magnitude and position of the resultant concrete compressive force from the assumed triangular stress block. With this requirement, α_1 can be calculated as: $\alpha_1 \cdot f_{ck} \cdot a = f_{ck} \cdot c/2$ and $a/2 = c/3$, therefore, $\alpha_1 = 0.75$. Hence, the equivalent rectangular stress block was assumed to have a mean stress about $0.75 f_{ck}$ and $\beta_1 = a/c = 0.67$ for HSC. On this basis, the following stress block parameters were suggested by Li et al.

$$\alpha_1 = 0.85 - 0.004\,(f_{ck} - 55) \ (f_{ck} \text{ in MPa}) \quad (5)$$
in which $0.75 \le \alpha_1 \le 0.85$

β_1 = same as the ACI code expression (6)

3.4 Bae and Bayrak 2003

Bae and Bayrak found that premature spalling of the cover concrete could occur at a compressive concrete strain less than 0.003. They concluded that the primary reason for premature spalling of cover concrete was the presence of large confining reinforcement required for HSC columns. Consequently, they reduced the compressive strain limit to 0.0025 for concrete strengths greater than 55 MPa to and developed new stress block parameters for both NSC and HSC.

$$\alpha_1 = 0.85 - 0.004\,(f_{ck} - 70)$$
in which $0.67 \le \alpha_1 \le 0.85$ (f_{ck} in MPa) (7)

$$\beta_1 = 0.85 - 0.004\,(f_{ck} - 30)$$
in which $0.67 \le \beta_1 \le 0.85$ (8)

3.5 ACI 318 TG-5 2004

An Innovation Task Group (later, changed to Task Group) has been formed within ACI for the purpose of developing a non-mandatory standard on the use of HSC in moderate to high seismic applications. According to the ACI 318 TG-5, it is apparent from a review of existing literature that, if the equivalent rectangular stress block of ACI 318-05 is used, the ratio of nominal to experimental column strength decreases as the axial load increases. Experimental results indicate that the nominal moment and axial load strengths of columns calculated with the ACI 318-05 rectangular stress block may be unconservative for compressive strengths greater than 80 MPa.

Because experimental results showed that the ACI 318-05 rectangular stress block is safe for NSC, and to minimize the change of the current stress block parameters, the β_1 expression is kept as the same, since it provides lower bound values of experimental data. Bearing this in mind, several suggestions for the α_1 parameter have been made. Eqs. (9) and (10) are one of the discussed expressions.

$$\alpha_1 = 0.85 - 0.004\,(f_{ck} - 55)$$
in which $0.85 \ge \alpha_1 \ge 0.70$ (f_{ck} in MPa) (9)

β_1 = same as the ACI code expression (10)

4. Discussion

Various proposals on concrete stress block parameters for HSC have been discussed. In this section, their impact on the *P-M* interaction curves, the depths of the neutral axis and the tensile strains of extreme tension steel are discussed.

4.1 *P-M* interaction curves

The *P-M* interaction curves are generated using various concrete stress block parameters and compared. Concrete columns with 610 × 610 mm square sections are used (Fig. 2). Twelve bars with steel ratio of 2% are assumed. Two concrete strengths of 80 and 100 MPa are studied.

Fig. 3 illustrates the results. As expected, the calculated axial and moment capacities of the ACI 318 code are larger than those of other stress block parameters. The difference of the ACI *P-M* interaction curve, from the other *P-M* interaction curves becomes larger as the concrete strength increases.

It is interesting to note that *P-M* interaction curves at a low axial load level are almost identical regardless of the difference in the concrete stress block parameters. Bae and Bayrak[13] stated that the *P-M* interaction curves are less sensitive to the concrete stress block parameters at a low axial load level ($P < P_b$) than at high axial load level ($P \ge P_b$). This is because the tensile force capacity of the tension reinforcement is likely to govern the moment capacity at a low axial load. When this is coupled with the fact that the premature cover spalling in columns are mostly reported at moderate to high levels of axial load[13], it becomes apparent that the HSC columns subjected to moderate to high axial load levels or the HSC beams

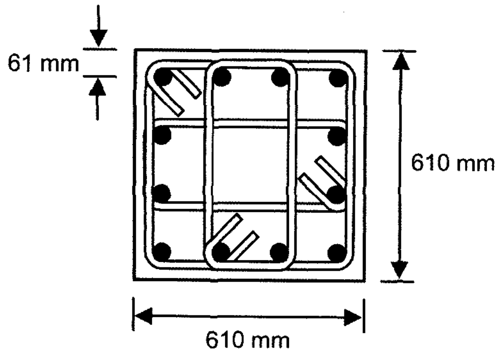

Fig. 2 Concrete column section.

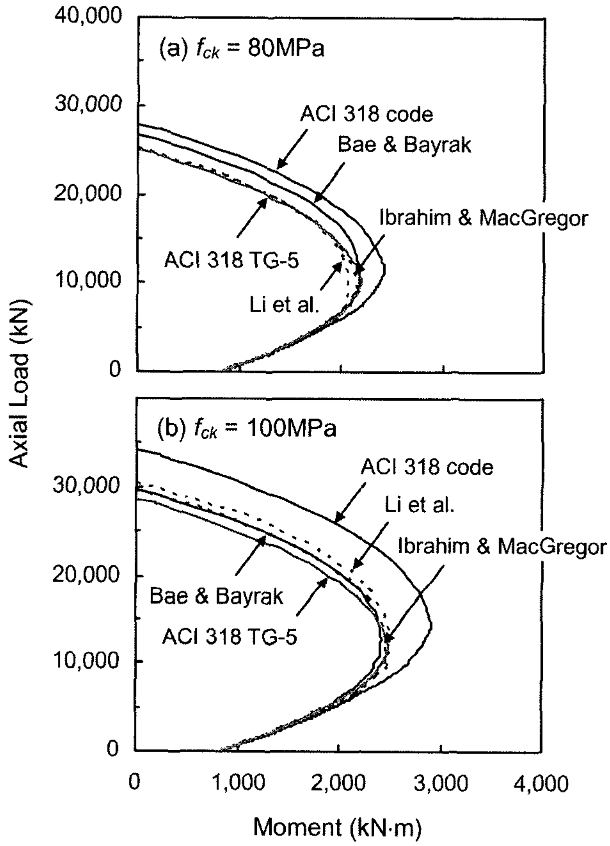

Fig. 3 P-M Interaction curves.

with compression-controlled sections are in an urgent need to modify the concrete stress block parameters.

4.2 Depth to neutral axis and tensile strain in extreme tension steel

The influence on the depth to the neutral axis (c in Fig. 1) and the tensile strain in extreme tension steel is investigated for concrete strengths of 80 and 100 MPa. The same column section properties, which are illustrated in Fig. 2, are considered. The results are shown in Figs. 4 and 5. Both figures indicate that the predicted depth to the neutral axis c from the ACI code expressions is smaller than the other predicted locations of neutral axes for a given axial load. The increase in the depth to the neutral axis reduces the tensile strain ε_t in the extreme tension steel at the nominal strength, as illustrated in the Figs. 4(b) and 5(b).

The strength reduction factor ϕ of the ACI 318-05 code is related to the tensile strain in extreme tension steel, as shown in Eq. (11).

$$\phi = 0.48 + 83\varepsilon_t \text{, for rectilinear ties} \quad (11)$$

where ϕ varies between 0.65 and 0.90

Therefore, the use of the stress block parameters other than the ACI code expressions provides smaller strength reduction factors for a given axial load, resulting in more conservative estimates for factored nominal column capacities.

4.3 Comparison with experimental data

The accuracy and conservativeness of the proposed stress block parameters are evaluated using data from 224 column tests reported in the literature. All of the 224 columns had reasonably large rectangular sections. The degree of accuracy of the concrete stress block parameters is estimated by Eq. (12).

$$\text{Error} = \frac{M_{predicted} - M_{test}}{M_{test}} \times 100(\%) \quad (12)$$

The predicted moment capacities are calculated from the *P-M* interaction curves using constant axial load approach. Detailed description on the column test data and the measurement of the accuracy can be found elsewhere.[3] Fig. 6 indicates that the use of ACI 318-05 stress block parameters results in inaccurate and

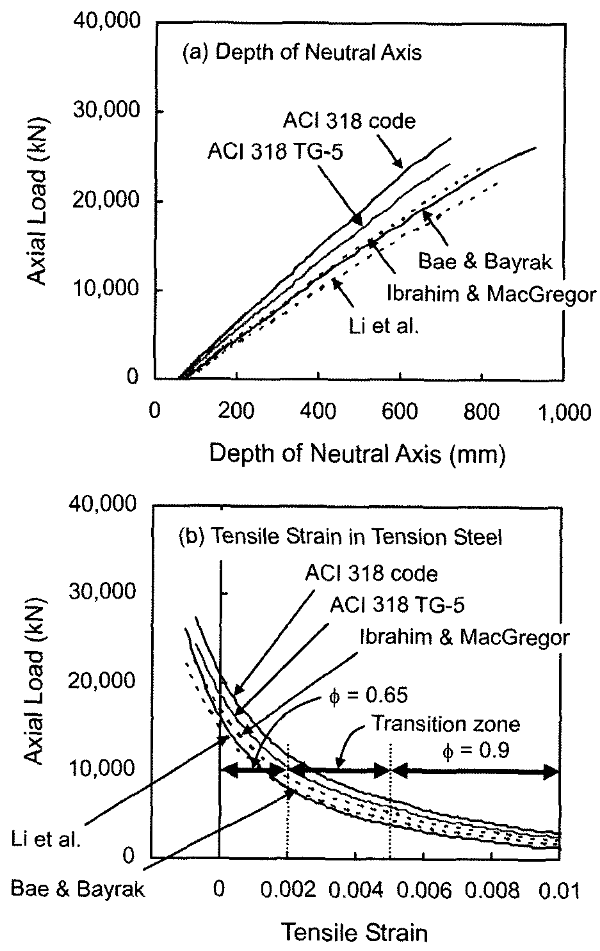

Fig. 4 Effect on neutral axis and tensile strain in extreme steel (f_{ck} = 80 MPa).

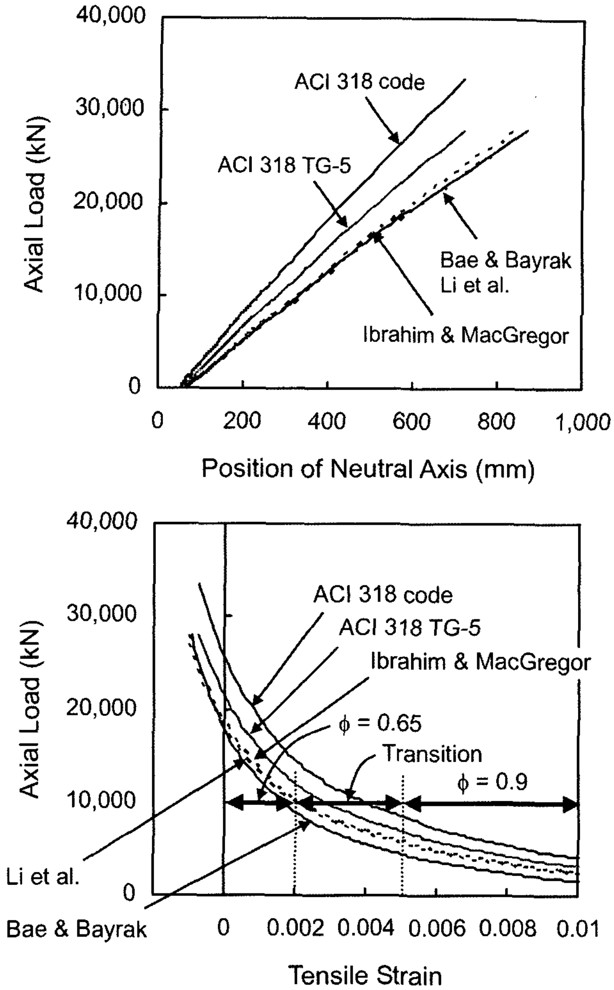

Fig. 5 Effect on neutral axis and tensile strain in extreme steel (f_{ck} = 100 MPa).

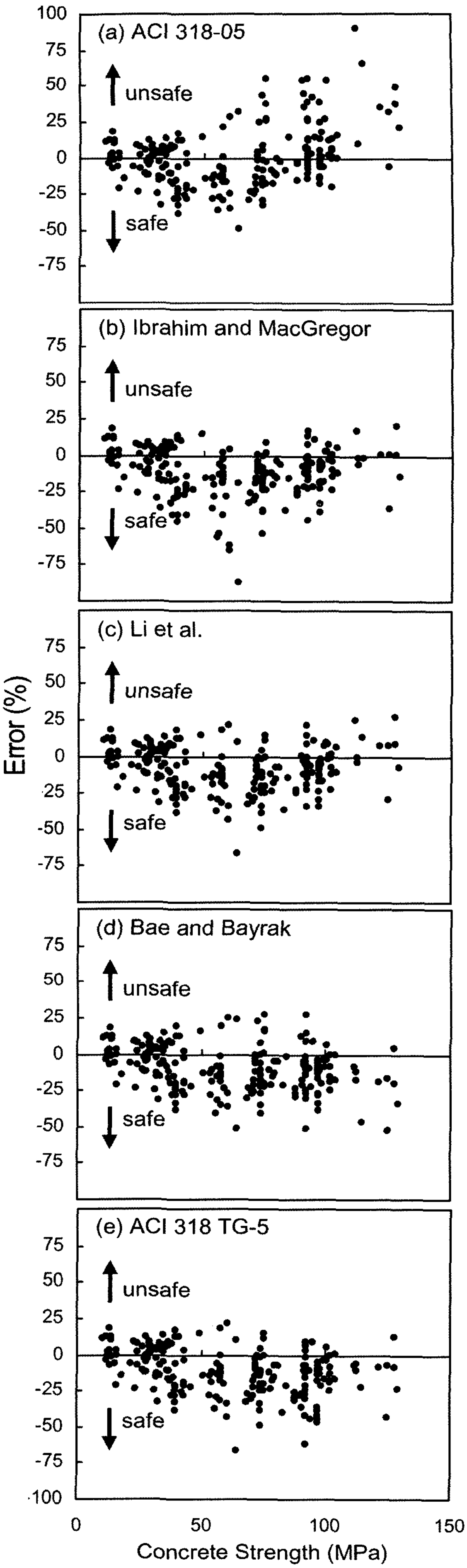

Fig. 6 Error in moment capacity predictions[3].

progressively increasing overestimations of the moment capacities as the concrete strength increases. The use of other stress block parameters provides more consistent, safe estimations for the entire range of concrete strength.

From a seismic design point of view, overestimating the flexural strengths of columns has two impacts: (1) the increase of the shear demand on the column calculated on the basis of the probable flexural strength and (2) the overestimation of the ratio of column-to-beam moment strengths. Overestimating the shear demand is conservative because it will lead to a larger amount of confining reinforcement. Contrarily, overestimating the ratio of column- to-beam moment strengths has a negative effect, because it will increase the probability of hinging in columns. ACI 318-05 requires a minimum ratio of column-to-beam moment strengths of 1.2. Overestimating column flexural strength will decrease that ratio and may even result in a strong beam-weak column mechanism.

4.4 Stress in prestressed reinforcement at nominal strength

The stress block parameters affect the provision on the prestressed concrete as well as the reinforced concrete. The ACI 318-05 code provides an equation to estimate the stress in prestressed tendons at the nominal strength in relation to the β_1 parameter in Chapter 18 as follows:

$$f_{ps} = f_{pu}\left\{1 - \frac{\gamma_p}{\beta_1}\left[\rho_p\frac{f_{pu}}{f_{ck}} + \frac{d}{d_p}(\omega - \omega')\right]\right\} \quad (13)$$

If any compression reinforcement is taken into account, the term $[\rho_p f_{pu}/f_{ck} + d/d_p(\omega - \omega')]$shall be taken not less than 0.17.

To investigate the effect of different stress block parameters on the calculated stress in prestressed tendons, two prestressed concrete sections are studied, as shown in Fig. 7. Results are illustrated in Fig. 8. The difference in the calculated stresses in tendons is negligible for both cases. The largest difference in the calculated stresses is less than 2%. Therefore, it can be concluded that the use of different concrete stress block parameters does not significantly affect the calculated stresses in prestressed tendons at the nominal moment, which indicates that the nominal moment capacity is not affected either. This is because the concrete stress block parameters are less significant for the columns with a low axial load level or for the beams with tension-controlled sections, as aforementioned.

5. Conclusions

The recently proposed concrete stress block parameters for HSC(high-strength concrete) are introduced and their impact on various aspects of ACI code provisions is discussed. On the basis of the results obtained from this study, the following conclusions can be drawn.

1) For the 224 columns considered in this study, the use of ACI 318-05 stress block parameters resulted in progressively increasing overestimations of moment capacities as the concrete strength increased.

2) The various stress block parameters for HSC, discussed in this paper, provide similar levels of conservativeness and accuracy for NSC and HSC. The difference in the calculated column capacities are investigated through the *P-M* interaction curves, and the curves showed that the ACI 318-05 stress block parameters produced larger column capacities than the other proposals for HSC.

3) The use of stress block parameters other than the ACI code resulted in the increase of the depth of neutral axes, the decrease of the tensile strains in extreme tension steel, and the decrease of the strength reduction factors for a given axial load. Therefore, more conservatively factored nominal capacities can be obtained by the use of stress block parameters other than those of ACI codes.

4) The use of different stress block parameters does not cause any significant difference in the calculated capacities for prestressed concrete members. The most critical members which are sensitive to the stress block parameters are identified as the columns with high axial load level or the beams with compression-controlled sections.

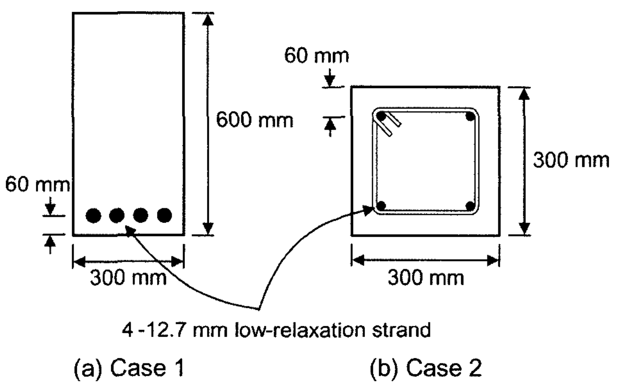

Fig. 7 Prestressed member sections.

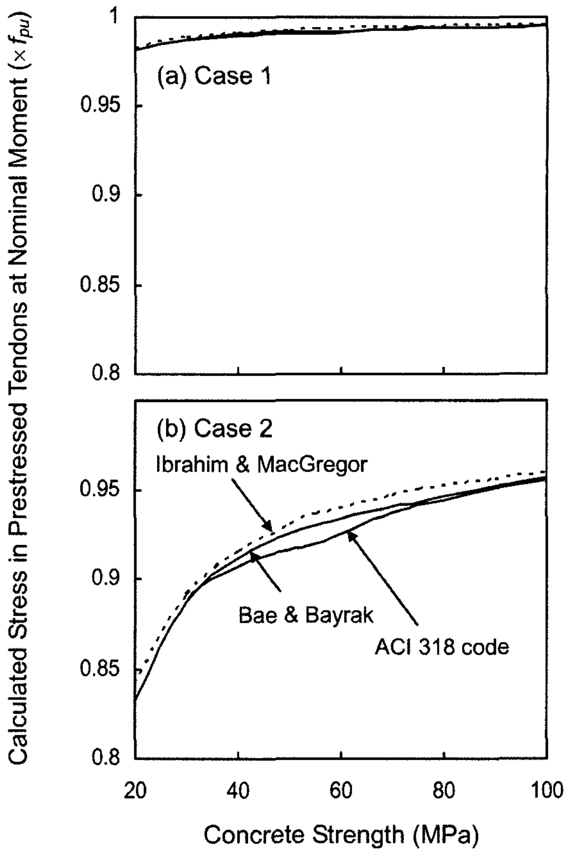

Fig. 8 Calculated stress in tendons.

References

1. Canadian Standards Association, *Design of Concrete Structures in Buildings*, (CAN 3-A23.3-M94), Rexdale, Ontario, Canada, 1994.

2. Standards New Zealand, *Design of Concrete Structure*, (NZS 3101-1995), Wellington, New Zealand, 1995.

3. Bae, S. and Bayrak, O., "Stress Block Parameters for HSC Members," *ACI Structural Journal*, Vol.100, No.5, 2003, pp.626~636.

4. American Concrete Institute, *Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05)*, Farmington Hills, Michigan, 2005.

5. Whitney, C. S., "Design of Reinforced Concrete Members Under Flexure and Combined Flexure and Direct Compression," *Journal of The American Concrete Institute*, Vol.33, No.4, 1937, pp.483~498.

6. Hognestad, E., Hanson, N. W., and McHenry, D., "Concrete Stress Distribution in Ultimate Strength Design," *Journal of the American Concrete Institute*, Vol.52, No.28, 1955, pp.455~479.

7. Mattock, A. H., Kriz, L. B., and Hognestad, E., "Rectangular Concrete Stress Distribution in Ultimate Strength Design," *Journal of The American Concrete Institute*, Vol.57, No.8, 1961, pp.875~928.

8. Nedderman, H., *Flexural Stress Distribution in Very High Strength Concrete*, M.A.Sc. Thesis, Department of Civil Engineering, University of Texas, Arlington, Texas, 1973.

9. American Concrete Institute, *Building Code Requirements for Reinforced Concrete (ACI 318-77)*, American Concrete Institute, 1977.

10. 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, 1997, pp.40~48.

11. Li B., Park, R., and Tanaka, H., *Strength and Ductility of Reinforced Concrete Members and Frames Constructed using High-Strength Concrete*, Research Report No.94-5, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand, 1994, 373pp.

12. ACI 318 TG-5, in the committee meeting, *ACI fall conference*, San Francisco, California, 2004.

13. Bae, S. and Bayrak, O., "Early Cover Spalling in High-Strength Concrete Columns," *Journal of Structural Engineering, ASCE*, Vol.129, No.3, 2003, pp.314~323.