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Nominal axial and flexural strengths of high-strength concrete columns

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Abstract. The ACI building code is allowing for higher strength reinforcement and concrete compressive strengths. The nominal strength of high-strength concrete columns is over predicted by the current ACI 318 rectangular stress block and is increasingly unconservative as higher strength materials are used. Calibration of a rectangular stress block to address this condition leads to increased computational complexity. A triangular stress block, derived from the general shape of the stress-strain curve for high-strength concrete, provides a superior solution. The nominal flexural and axial strengths of 150 high-strength concrete columns tests are calculated using the proposed stress distribution and compared with the predicted strength using various design codes and proposals of other researchers. The proposed triangular stress model provides similar level of accuracy and conservativeness and is easily incorporated into current codes.

Keywords: high-strength concrete; flexural and axial strengths; triangular stress block; interaction curve; column; beam

1. Introduction

The use of high-strength concrete (HSC) in buildings and transportation industry has increased in worldwide popularity. HSC has the advantages over normal-strength concrete (NSC) in strength and durability (Myers, 2008). HSC offers reduction in section size for columns when used in high-rise buildings. This gives a strong motivation to examine the current ACI 318 (2014) provisions for nominal strength calculations for HSC columns because they are developed based on NSC columns tests (Bae and Bayrak 2013, ACI 441.1R 2018). Several researchers conducted tests to study the behavior of HSC columns reported that the axial and flexural strengths of HSC columns could be over predicted by the current ACI 318 (2014) rectangular stress block expressions (Wahidi 1995, Ibrahim and MacGregor 1996, Lloyd and Rangan 1996). While the calculation of nominal strength is addressed, the lack of attention to the effects on the design strength lead to solutions where proposed modifications to strength reduction factors minimize the benefits of using high-strength concrete.

Khadiranaikar and Awati (2012) conducted experimental tests of plain concrete columns, reinforced concrete members such as eccentrically loaded columns, and beams in pure flexure. Based on the test results, stress-block parameters for wide range of concrete strength have been developed. Yang *et al*. (2013) proposed a generalized equivalent stress block model that works for both light and normal weight HSC. The coefficients used in the proposed stress block were formulated based on a nonlinear regression analysis through an extensive database of test data.

Recently, Al-Kamal (2019), the author of this paper, proposed a triangular stress distribution to calculate the flexural strength of high-strength concrete beams. Extending this concept, the triangular stress distribution is suggested in this paper to calculate the nominal axial and flexural strengths of HSC columns.

The shape of the ascending part of the stress-strain curve for HSC remains linear up to a stress closer to peak stress than the curve for NSC. Hence a triangular stress distribution is better suited for HSC (Wahidi 1995). In this research, the triangular stress distribution is studied thoroughly and validated using large database consisting of 150 tested HSC columns with concrete strengths above 55 MPa (8,000 psi) and up to 130 MPa (18,800 psi). In addition, the results obtained by using the triangular stress block is compared with the results of recent studies on the equivalent rectangular stress block for HSC columns, i.e., ACI 318 (2014), CEB-FIP Model Code (2010), NZS 3101 (2006), CSA A23.3 (2004), EN 1992 (2004), Mertol *et al*. (2008), Bae and Bayrak (2003), Ozbakkaloglu and Saatcioglu (2004), Ibrahim and MacGregor (1997), Azizinamini *et al*. (1994). Based on the comparison results, a change to the stress block parameters of various codes is examined.

2. Research significance

The current ACI 318-14 provisions allow an equivalent rectangular stress block for calculation of member strength. The shape of the stress-strain curve is adjusted by the factor β_1 to account for the higher strength. Above 55 MPa (8,000 psi) there is no further change in this value, in part because higher strength tests were not available when the limit was established. While other design codes and individuals have proposed alternative stress block models for calculating strength of HSC members, there is no universal agreement

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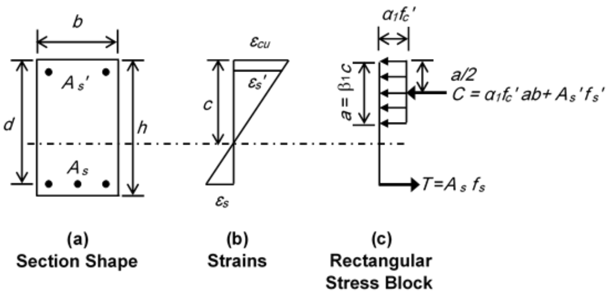

Fig. 1 Rectangular stress block

on a rational stress block model for high-strength concrete nor is there agreement on when a transition in the stress block parameters α_1 and β_1 should occur. This paper proposes to use the ACI 318 rectangular stress block for concrete strength at a transition point of 69 MPa (10,000 psi) and replace the ACI 318 rectangular stress block by the triangular stress block for concrete strength above that point, which provides both a smooth transition in nominal and design strengths.

3. Rectangular stress block parameters

The rectangular stress block parameters are shown in Fig. 1. The intensity of the rectangular block is $\alpha_1 f_c'$. The depth of the stress block is $\beta_1 c$, where c is the neutral axis depth. In ACI 318 (2014), the α_1 parameter is 0.85 for all concrete strengths and the β_1 parameter is equal to 0.85 for concrete strength up to 30 MPa (4,350 psi) and decreases linearly to 0.65 for a concrete strength of 55 MPa (8,000 psi) and remain constant at 0.65. The ultimate compressive strain ε_{cu} at the extreme compression fiber is set to 0.003 for all concrete strengths. The current ACI 318 (2014) provisions do not specify an upper limit for the concrete strength.

Flexural and axial strengths of HSC columns appear to be over predicted by the ACI 318 (2014) stress block parameters (Wahidi, 1995; Ibrahim and MacGregor, 1996; Lloyd and Rangan; 1996). As a result, many researchers have adopted studies to propose alternative rectangular stress block parameters for HSC columns. Table 1 summarizes stress block parameters obtained from various design codes and those from different publications. From Table 1, the parameter α_1 of the ACI 318 (2014) remains constant regardless the concrete strength. The parameter β_1 adopted by Mertol *et al*. (2008) and NZS 3101 (2006) is identical to that of ACI 318-14 provisions; however, the lower limit of α_1 is reduced linearly from 0.85 to 0.75 for concrete strengths above 69 MPa (10,000 psi). Both α_1 and β_1 parameters are adjusted and should not be less than 0.67 according to CSA A23.3 (2004), Bae and Bayrak (2003).

Table 1 Rectangular stress block expressions: codes and proposals

Code or proposal	f_c' in MPa	f_c' in psi	ε_{cu}
ACI 318 (2014)	α_1=0.85 β_1=0.85-0.008(f_c'-30) $0.85 \geq \beta_1 \geq 0.65$	α_1=0.85 β_1=0.85-0.05/1,000X(f_c'-4,000) $0.85 \geq \beta_1 \geq 0.65$	0.003
Mertol *et al*. (2008)	α_1=0.85-0.0029(f_c'-69) $0.85 \geq \alpha_1 \geq 0.75$ β_1=0.85-0.007252(f_c'-28) $0.85 \geq \beta_1 \geq 0.65$	α_1=0.85-0.02/1,000X(f_c'-10,000) $0.85 \geq \alpha_1 \geq 0.75$ β_1=0.85-0.05/1,000X(f_c'-4,000) $0.85 \geq \beta_1 \geq 0.65$	0.003
NZS 3101 (2006)	α_1=0.85-0.004(f_c'-55) $0.85 \geq \alpha_1 \geq 0.75$ β_1=0.85-0.008(f_c'-30) $0.85 \geq \beta_1 \geq 0.65$	α_1=0.85-0.028/1,000X(f_c'-8,000) $0.85 \geq \alpha_1 \geq 0.75$ β_1=0.85-0.05/1,000X(f_c'-4,000) $0.85 \geq \beta_1 \geq 0.65$	0.003
CSA A23.3 (2004)	α_1=0.85-0.0015$f_c' \geq 0.67$ β_1=0.97-0.0025$f_c' \geq 0.67$	α_1=0.85-0.010/1,000X$f_c' \geq 0.67$ β_1=0.97-0.017/1,000X$f_c' \geq 0.67$	0.0035
Bae and Bayrak (2003)	α_1=0.85-0.004(f_c'-70) $0.85 \geq \alpha_1 \geq 0.67$ β_1=0.85-0.004(f_c'-30) $0.85 \geq \beta_1 \geq 0.67$	α_1=0.85-0.028/1,000X(f_c'-10,000) $0.85 \geq \alpha_1 \geq 0.67$ β_1=0.85-0.028/1,000X(f_c'-4,000) $0.85 \geq \beta_1 \geq 0.67$	0.0025 for f_c' > 55 MPa (8,000 psi), otherwise 0.003
Ozbakkaloglu and Saatcioglu (2004)	α_1=0.85-0.0014(f_c'-30) $0.85 \geq \alpha_1 \geq 0.72$ β_1=0.85-0.0020(f_c'-30) $0.85 \geq \beta_1 \geq 0.67$	α_1=0.85-0.010/1,000X(f_c'-4,000) $0.85 \geq \alpha_1 \geq 0.72$ β_1=0.85-0.013/1,000X(f_c'-4,000) $0.85 \geq \beta_1 \geq 0.67$	0.003
Ibrahim and MacGregor (1997)	α_1=0.85-0.00125$f_c' \geq 0.725$ β_1=0.95-0.0025$f_c' \geq 0.70$	α_1=0.85-0.0086/1,000X$f_c' \geq 0.725$ β_1=0.95-0.0172/1,000X$f_c' \geq 0.70$	0.003
Azizinamini *et al*. (1994)	α_1=0.85-0.007(f_c'-69) $0.85 \geq \alpha_1 \geq 0.60$ β_1=0.85-0.008(f_c'-30) $0.85 \geq \beta_1 \geq 0.65$	α_1=0.85-0.05/1,000X(f_c'-10,000) $0.85 \geq \alpha_1 \geq 0.60$ β_1=0.85-0.05/1,000X(f_c'-4,000) $0.85 \geq \beta_1 \geq 0.65$	0.003
CEB-FIP Model Code (2010) or EN 1992 (2004)	α_1=1.0-(f_c'-50)/200 $\alpha_1 \leq 1.0$ β_1=0.80-(f_c'-50)/400 $\beta_1 \leq 0.80$	α_1=1.0-0.0345/1,000X(f_c'-7,250) $\alpha_1 \leq 1.0$ β_1=0.80-0.0172/1,000X(f_c'-7,250) $\beta_1 \leq 0.80$	(2.6 + 35 [(90 – f_c')/100]^4)/1000, f_c' in MPa

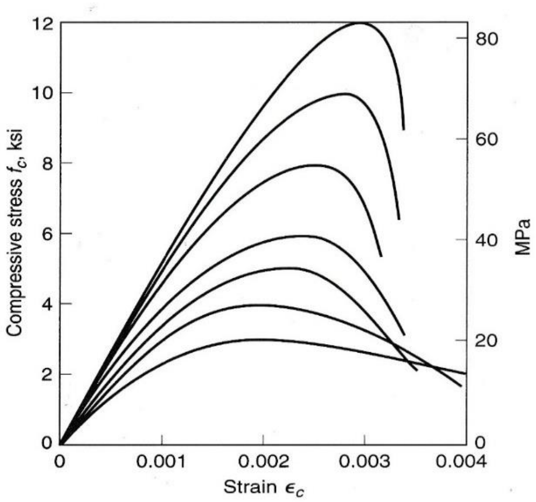

Fig. 2 Typical compressive stress-strain curves for NSC and HSC (Nilson *et al*. 2010)

In addition to ACI 318 (2014), Mertol *et al*. (2008), NZS 3101 (2006), CSA A23.3 (2004), and Bae and Bayrak (2003), alternative stress block parameters have been proposed by CEB-FIP Model Code (2010) or EN 1992 (2004), Ozbakkaloglu and Saatcioglu (2004), Ibrahim and MacGregor (1997), Azizinamini *et al*. (1994). The combined result of the above research indicates that additional modification of the ACI 318 (2014) rectangular stress block is needed for high-strength concrete.

4. Proposed stress block model

It is assumed that the stress distribution of the compressive zone of a cross section is the same as the stress-strain curve in uniaxial compression. Therefore, the concrete stress-strain relationship plays a significant role in developing a stress block model. For this purpose, a typical set of stress-strain curves for normal-strength concrete (NSC) and high-strength concrete (HSC) is shown in Fig. 2 (Nilson *et al*. 2010). From this figure it is clear that as the concrete strength increases, the shape of the ascending part of the relationship becomes more linear and steeper and the strain increases and reaches a peak value near 0.003. The slope of the descending branch of the curve is steeper for high strength concrete. As a result, the general shape of the stress-strain relationship for HSC is more like a triangle. A triangular stress block is, therefore, proposed in this study as shown in Fig. 3. This model is simple and slightly conservative since the actual area under the stress-strain curve is slightly greater than a triangle.

Failure becomes more brittle as the concrete strength increases because unloading beyond the peak stress becomes more rapid as shown in Fig. 2. Testing of several HSC cylinders is performed by Chen (1995). Chen (1995) reported that these cylinders exploded suddenly when they reached their nominal strength. To account for this brittle behavior, the triangular stress block may need adjustment because brittle failure is undesirable.

Tests have shown that the concrete prisms and cylinders

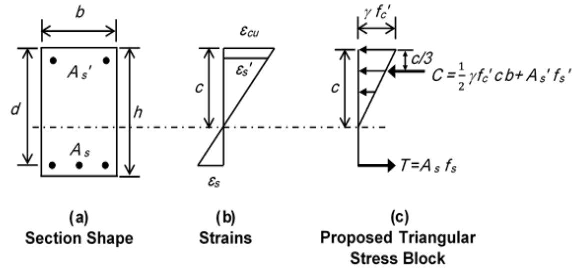

Fig. 3 Triangular stress block

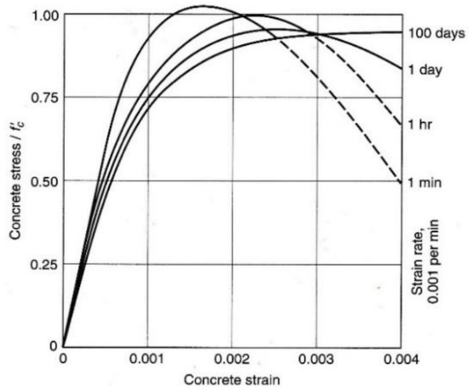

Fig. 4 Stress-strain curves at various strain rates (Nilson *et al*. 2010)

strengths under sustained loads fail at lower value than f_c' determined at normal rates of test loading as shown in Fig. 4 (Nilson *et al*. 2010, Rüsch 1968). A column test may go for minutes or hours compared to the long-term loading of a building. Hence, the effect of sustained load needs to be considered.

A value less than 1.0 is appropriate for evaluating test results that are conducted in a short duration. For this purpose, a factor γ is defined as shown in Fig. 3. The strength reduction factor γ can be determined by performing tests on HSC for normal load testing and sustained load testing similar to that done for NSC by Rüsch (1968). Tests conducted by Han (1996) show that failure occurred after about 1.5 to 2.5 minutes for HSC prisms loaded with $0.95f_c'$, while for HSC prisms loaded with $0.85f_c'$ failure occurred after about 0.5 to 2.5 hours. Therefore, a value of 0.85 for the reduction factor γ is selected for evaluating HSC columns tests. This value is consistent with the ACI building code and results in no change to current practice. In addition, Danica (2016), Brachmann and Empelmann (2018) recommend the value of 0.85 for the factor (γ). They have been found that the value of 0.85 leads to a safe design for both NSC and HSC.

The ultimate concrete strain varies with the concrete strength as shown in Fig. 2. As the concrete strength increases, the ultimate concrete strain would not remain constant but would gradually decrease (Ho *et al*. 2002). However, the majority of the proposed stress block parameters shown in Table 1 use a value of 0.003 for the ultimate concrete strain at failure. Therefore, the 0.003 value is retained in this research to maintain consistency with current practice.

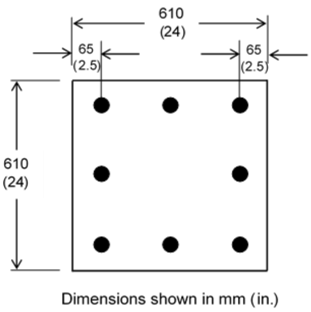

Fig. 5 Column cross section used in parametric study

5. Parametric study

A parametric study similar to that done in the ACI 441.1R (2018) report compares the proposed triangular model with stress blocks of various codes and proposals of researchers is carried out in this research. Normalized P-M interaction curves without using the strength reduction factor ϕ are developed for this purpose. Fig. 5 shows a typical column cross section that is used for the parametric study. The column is analyzed for steel ratio ρ of 1 percent and concrete compressive strengths of 55, 83, and 110 MPa (8,000, 12,000, and 16,000 psi). The yield strength f_y of the reinforcement is taken as 410 MPa (60,000 psi). Results of the parametric study are shown in Fig. 6.

For concrete strength of 55 MPa (8,000 psi), in Fig. 6(a), the column nominal strength obtained by the proposed triangular stress block is more conservative than other stress blocks. The ACI 318 (2014) and the CEB-FIP Model Code (2010) or EN 1992 (2004) stress blocks strength is noticeably higher than the alternative stress blocks; CSA A23.3 (2004) Ozbakkaloglu and Saatcioglu (2004), and Ibrahim and MacGregor (1997).

From Fig. 6(b), for concrete strength of 83 MPa (12,000 psi), the proposed triangular stress yielded more conservative results than other stress blocks in a critical zone of 20% to 70% of $P_n/(bhf_c')$. All stress blocks including the proposed one resulted in conservative estimates for the nominal strength as compared with the ACI 318 (2014) stress block and the CEB-FIP Model Code (2010) or EN 1992 (2004). Less conservative estimates are obtained by Mertol *et al*. (2008) stress block as compared with other stress blocks, except for ACI 318 (2014). The stress block of Mertol *et al*. (2008) is close to identical compared with that of Bae and Bayrak (2003). The stress blocks of NZS 3101 (2006), Ibrahim and MacGregor (1997), and Azizinamini *et al*. (1994) are identical at the concrete strength of 83 MPa (12,000 psi). In the critical zone of 20% to 70% of $P_n/(bhf_c')$, column nominal strength estimates obtained by CSA A23.3 (2004) are larger than obtained by the proposed triangular stress block. If the column is concentrically loaded, the nominal axial strength obtained by using the proposed triangular stress is identical with that of ACI 318-14 provisions.

For concrete strength of 110 MPa (16,000 psi), in Fig. 6(c), Azizinamini *et al*. (1994) stress block resulted in more conservative estimates when compared with the proposed triangular stress block in a critical zone of 20% to 65% of $P_n/(bhf_c')$. All stress blocks yielded conservative results as compared with the ACI 318 (2014) stress block. Above a line of 75% of $P_n/(bhf_c')$, the proposed triangular stress block expressed less conservative estimates as compared with the other stress blocks, except for ACI 318 (2014).

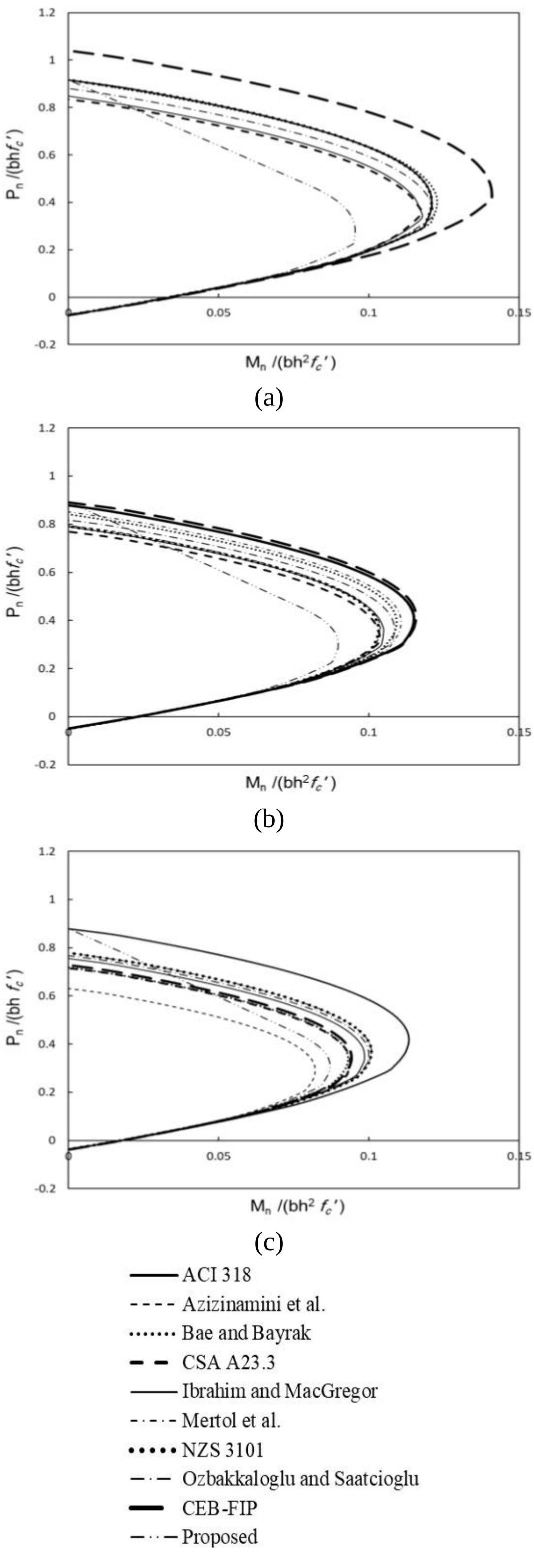

Fig. 6 Column strength interaction diagrams comparing different stress blocks (a) f_c'=55 MPa (8,000 psi) (b) f_c'=83 MPa (12,000 psi) (c) f_c'=110 MPa (16,000 psi)

From the parametric study discussed above, it is difficult to find consensus among different stress blocks of codes and proposals of researchers including the proposed triangular stress block except that the ACI 318 (2014) stress block is less conservative than the others. Also, CEB-FIP Model Code (2010) or EN 1992 (2004) stress block is less conservative than the others specially for concrete strengths of 55 MPa (8,000 psi) and 83 MPa (12,000 psi).

For low level of axial load ($P/P_0 \leq 0.1$), interaction curves are less sensitive to the difference of stress block models for all concrete strengths as shown in Fig. 6. Therefore, all stress block models can be used for nominal flexural strength predictions of HSC beams with the same degree of accuracy and conservativeness

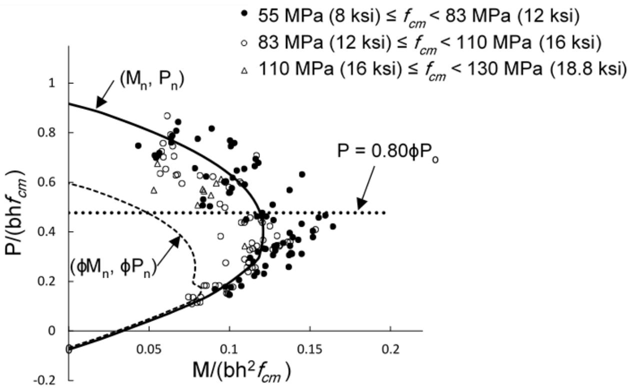

Fig. 7 Normalized *P-M* curve: ACI 318 (2014)

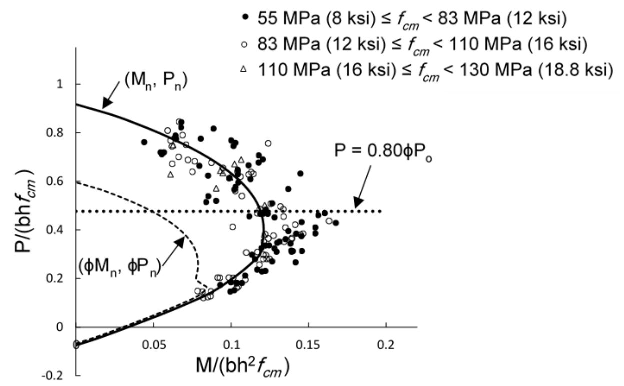

Fig. 8 Normalized *P-M* Curve: Mertol *et al*. (2008)

6. Experimental validation

6.1 Experimental data

The nominal axial and flexural strengths obtained by using the triangular stress block and other stress blocks of various codes and proposals of researchers are evaluated using a data base contains results from 150 HSC columns tests reported in the literature (Ozden 1992, Azizinamini *et al*. 1994, Sheikh *et al*. 1994, Wahidi 1995, Basappa and Rangan 1995, Ibrahim and MacGregor 1996, Lloyd and Rangan 1996, Foster and Attard 1997, Bayrak 1997, Legergon and Paultre 2000). The data base considered in this research is for HSC columns with concrete compressive strength above 55 MPa (8,000) and up to 130 MPa (18,800 psi), which is derived from a bigger data base used by Bae and Bayrak (2003), Bae and Bayrak (2013) that consists of 224 NSC and HSC columns. The data base consists eccentrically loaded columns and columns subjected to combined axial and flexural loads. More details about the columns data base can be found in Bae and Bayrak (2003), Bae and Bayrak (2013).

6.2 Results and discussions

The nominal axial and flexural strengths are calculated using stress block parameters of ACI 318 (2014), NZS 3101 (2006), CSA A23.3 (2004), stress block parameters proposed by CEB-FIP Model Code (2010) or EN 1992 (2004), Mertol *et al*. (2008), Bae and Bayrak (2003), Ozbakkaloglu and Saatcioglu (2004), Ibrahim and MacGregor (1997), Azizinamini *et al*. (1994), and the proposed triangular stress block. Numerous P-M interaction curves are generated to compare the predicted axial and flexural strengths with the columns test results. In order to overcome the challenge of presenting several interaction curves, a normalized interaction curve is developed and the axial and flexural strengths from the column tests are converted to the normalized interaction curve format. Details about the process of normalizing the test results can be found in Bae and Bayrak (2013). The 55 MPa (8,000 psi) column section shown in Fig. 5 used for the parametric study is also used for the normalized interaction curve.

The comparisons of the normalized interaction curves and the converted test data points are shown in Fig. 7 through Fig. 16. For columns with tie reinforcement, design axial strength in compression is limited by the ACI 318 (2014) to ($0.8\phi P_0$). This limit is included in the figures. If a data point is located outside the normalized nominal interaction curve, the calculated column strength is underestimated. Column strength is overestimated when a data point is located inside the normalized nominal interaction curve.

The results for the ACI 318 (2014) stress block are shown in Fig. 7. From this figure, column strengths are over predicted by the ACI 318 (2014) stress block in the range of concrete strength from 55 MPa (8,000) to 83 MPa (12,000 psi). Examination of the columns test results indicates that the ACI 318 (2014) stress block underestimates columns strengths for the concrete strength of 69 MPa (10,000 psi). The ACI 318 (2014) stress block is acceptable for the concrete strength of 83 MPa (12,000 psi) only if the design axial strength limit ($0.8\phi P_0$) is applied. With this restriction, the ACI 318 (2014) stress block can be used for the design of HSC columns up to the concrete strength of 83 MPa (12,000 psi). Near the balance point of the interaction diagram, the ACI 318 (2014) stress block overestimates the member strength, even when the strength reduction factor is applied.

The stress block proposed by Mertol *et al*. (2008) underestimates member strength for concrete strength of 83 MPa (12,000 psi). However, strengths are overestimated for concrete strength above 83 MPa (12,000 psi) as shown in Fig. 8. The stress block of NZS 3101 (2006) provides improved level of conservatism than the ACI 318 2014) and

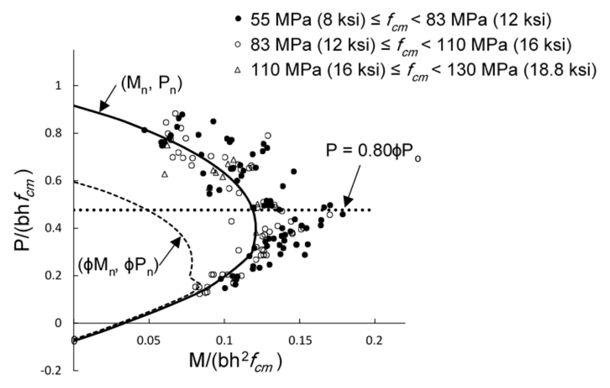

Fig. 9 Normalized *P-M* Curve: NZS 3101 (2006)

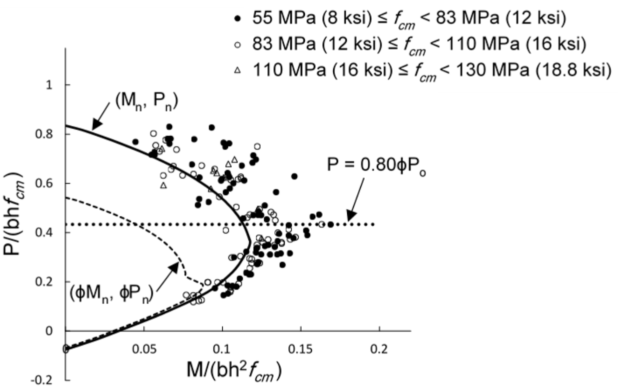

Fig. 10 Normalized *P-M* Curve: CSA A23.3 (2004)

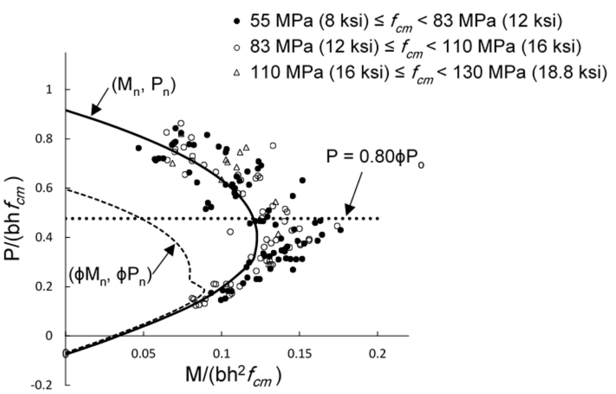

Fig. 11 Normalized *P-M* Curve: Bae and Bayrak (2003)

Mertol *et al*. (2008) stress blocks as shown in Fig. 9. However, nominal strengths for some tested columns with concrete strength of 75 MPa (10,800 psi) are over predicted by the NZS 3101 (2006). The stress block of NZS 3101 (2006) over predicts columns strengths for concrete strength above 83 MPa (12,000 psi), but with better performance than the ACI 318 (2014) and Mertol *et al*. (2008) stress blocks.

The stress blocks of CSA A23.3 (2004), Bae and Bayrak (2003), Ozbakkaloglu and Saatcioglu (2004), and Ibrahim and MacGregor (1997) provide similar level of performance with the NZS 3101 (2006) stress block, but with more conservative strength predictions for concrete strength above 83 MPa (12,000 psi) (Figs. 10, 11, 12, and 13). The stress block of Azizinamini *et al*. (1994) results in severely

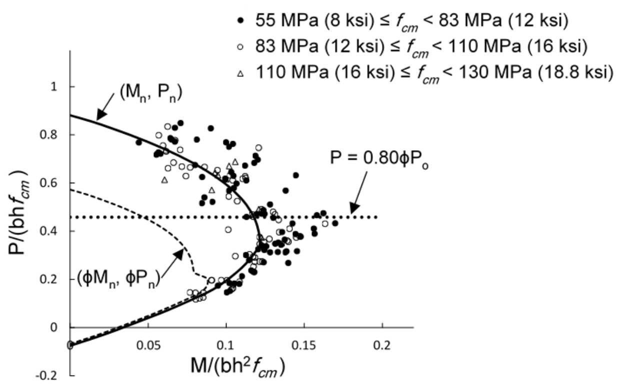

Fig. 12 Normalized *P-M* Curve: Ozbakkaloglu and Saatcioglu (2004)

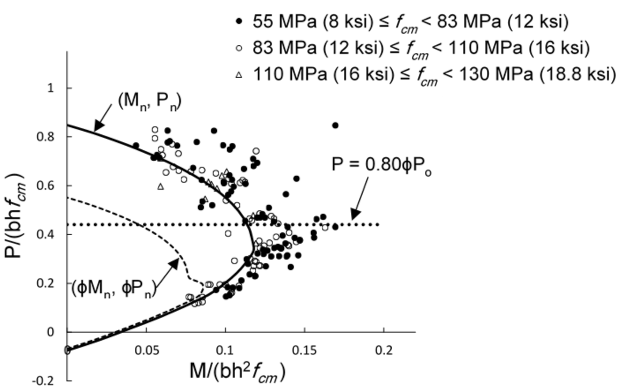

Fig. 13 Normalized *P-M* Curve: Ibrahim and MacGregor (1997)

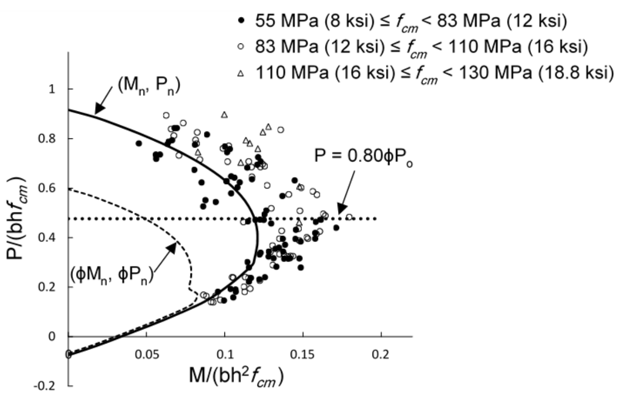

Fig. 14 Normalized *P-M* Curve: Azizinamini *et al*. (1994)

underestimates columns strengths when concrete strength is above 110 MPa (16,000 psi) as shown in Fig. 14.

The CEB-FIP Model Code (2010) moderately over predicts column strengths in the range of concrete strength from 55 MPa (8,000) to 110 MPa (16,000 psi) as compared with the other models. The CEB-FIP Model Code (2010) has similar underestimation for the columns strengths as compared with Azizinamini *et al*. (1994) when concrete strength is above 110 MPa (16,000 psi) as shown in Fig. 15.

The proposed triangular stress block produces conservative estimates for the columns strengths for all concrete strengths above 55 MPa (8,000 psi), Fig. 16. The degree of conservatism varies with the concrete strengths

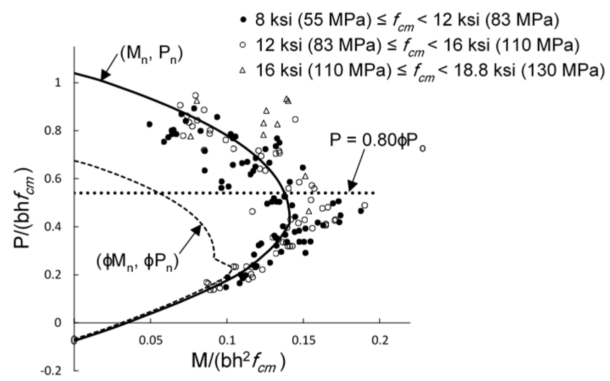

Fig. 15 Normalized *P-M* Curve: CEB-FIP Model Code (2010) or EN 1992 (2004)

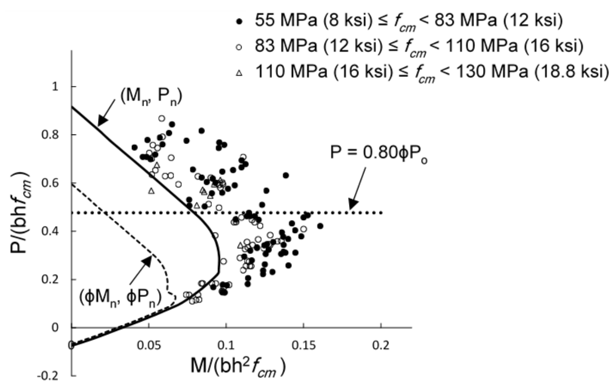

Fig. 16 Normalized *P-M* Curve: Proposed (Triangular Stress Distribution)

and decrease as the concrete strength increased. The proposed triangular stress block lies between severely to moderately underestimates columns strengths in the range of concrete strength from 55 MPa (8,000) to 83 (12,000 psi) as compared with CSA A23.3 (2004), Bae and Bayrak (2003), Ozbakkaloglu and Saatcioglu (2004), and Ibrahim and MacGregor (1997) stress blocks. Examination of the test results indicates that the severe underestimation occurs when the concrete strength is below 69 MPa (10,000 psi). The proposed triangular stress block is slightly more conservative than CSA A23.3 (2004), Bae and Bayrak (2003), Ozbakkaloglu and Saatcioglu (2004), and Ibrahim and MacGregor (1997) stress blocks for concrete strengths above 83 MPa (12,000 psi). For concrete strength above 110 MPa (16,000), the proposed triangular stress block shows identical or close to identical degree of conservatism as compared with CSA A23.3 (2004), Bae and Bayrak (2003), Ozbakkaloglu and Saatcioglu (2004), and Ibrahim and MacGregor (1997) stress blocks.

A way to transition is to generate the interaction diagram, then a limit can be placed on the total concrete contribution to cap the curve similar to the stress-rupture limit e.g., 0.85. However, this makes the use of proposed triangular stress in generating an interaction diagram difficult as compared with the rectangular stress blocks of various codes and proposals of researchers. Therefore, the sustained load factor γ is kept as 0.85 for this range of concrete strength.

7. Modifications to various codes

The results discussed in the previous section revealed that the ACI 318 (2014) provisions overestimates HSC nominal columns strengths when concrete strength above 69 MPa (10,000 psi). However, the ACI 318 (2014) stress block can be used up to concrete strength of 83 MPa (12,000 psi) because of the design axial strength limit ($0.8\phi P_0$). Despite of the axial strength limit similar to the other codes, the ACI 318 (2014) stress block parameters need to be changed to account for the over predicted nominal columns strengths above 69 MPa (10,000 psi). Therefore, this study proposes to replace the ACI 318 (2014) rectangular stress block by the triangular stress block proposed in this paper for concrete strength above 69 MPa (10,000 psi).

A triangular stress block is compatible with ACI 318 (2014) section 22.2.2.3, which reads "The relationship between concrete compressive stress and strain shall be represented by a rectangular, trapezoidal, parabolic, or other shape that results in prediction of strength in substantial agreement with results of comprehensive tests". Further, the $0.85f_c'$ is in accordance with section 22.2.2.4.1 for axially loaded members which reads "Concrete stress of $\mathbf{0.85f_c'}$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a line parallel to the neutral axis located a distance $\boldsymbol{a}$ from the fiber of maximum compressive strain, as calculated by $a=\beta_1c$". Addition of a new section 22.2.2.4.4 could implement a triangular stress block. Section 22.2.2.4.4 would read, above 69 MPa (10,000 psi) a triangular stress block with a maximum stress of $0.85f_c'$ shall be permitted.

The NZS 3101 (2006) stress block tends to over predict nominal columns strengths when compressive strength above 83 MPa (12,000 psi). This is happened because this code stops changing α_1 stress block parameter for concrete strength above 80 MPa (11,600 psi). This parameter is set to 0.75 for concrete strength above 80 MPa (11,600 psi) as shown in Table 1. The lower limits for the stress block parameters of the NZS 3101 (2006) are obtained from converting the triangular stress block with a maximum stress of f_c' into a rectangle. However, in this research the maximum stress for the proposed triangular stress block is set to $0.85f_c'$ considering the sustained load effects. Therefore, as it has been proposed for the ACI 318 (2014), the proposed triangular stress block is suggested as a replacement to the NZS 3101 (2006) stress block for concrete strength above 69 MPa (10,000 psi). Regarding this change, there will be no nominal strengths over predictions by the NZS 3101 (2006) for some tested columns at the concrete strength of 75 MPa (10,800 psi).

The CSA A23.3 (2004) estimations for the strength are conservative for all concrete strength and no further changes are required. Changes to the NZS 3101 (2006) and CSA A23.3 (2004) standards could be similar to those in the ACI building code.

The upper limits for the stress block parameters α_1 and

β_1 for the CEB-FIP Model Code (2010) or EN 1992 (2004) are considered high compared with stress block parameters of other stress blocks as shown in Table 1. This resulted in moderate over predictions for the columns strengths in the range of concrete strength from 55 MPa (8,000) to 110 MPa (16,000 psi). Also, the CEB-FIP Model Code (2010) or EN 1992 (2004) limits the concrete strength for high-strength concrete to 100 MPa (14,500 psi). This resulted in severely underestimated columns strengths and it is similar to that of Azizinamini *et al*. (1994) stress block for concrete strength above 110 MPa (16,000 psi). Therefore, the lower and the upper limits for CEB-FIP Model Code (2010) or EN 1992 (2004) stress block parameters need to be reconstituted.

8. Conclusions

Proposed triangular stress block is developed in this research to predict the nominal axial and flexural strengths of high-strength concrete (HSC) columns. Using a database of 150 HSC columns available in the literature, the proposed triangular stress block and stress blocks of various codes and proposals of researchers were examined. In addition, a possible change to models of various codes are presented. Based on the work done in this research, the following conclusions are drawn:

1. For low level of axial load ($P/P_0 \leq 0.1$), interaction curves are less sensitive to the difference of stress block models for all concrete strengths. As a result, all stress block models including the ACI 318 stress block can be used for nominal flexural strength predictions of HSC beams with the same degree of accuracy and conservativeness.
2. The ACI 318 stress block over predicts nominal axial and flexural strengths of HSC columns with concrete strength above 69 MPa (10,000 psi). Despite of this fact, the ACI 318 stress block can be used for the design of HSC columns up to the concrete strength of 83 MPa (12,000 psi) only if the design axial strength limit ($0.8\phi P_0$) is applied.
3. The stress block proposed by Mertol *et al*. overestimates member strengths of HSC columns for concrete strength above 83 MPa (12,000 psi). The stress block of NZS 3101 provides improved level of conservatism than the ACI 318 and Mertol *et al*. stress blocks. However, the stress block of NZS 3101 over predicts columns strengths for concrete strength above 83 MPa (12,000 psi), but with better performance than the ACI 318 and Mertol *et al*. stress blocks.
4. The stress blocks of CSA A23.3, Bae and Bayrak, Ozbakkaloglu and Saatcioglu, and Ibrahim and MacGregor provide improved level of accuracy and conservatism for the axial and flexural strengths predictions for all concrete strengths above 55 MPa (8,000 psi). On the other hand, the stress block proposed by Azizinamini *et al*. results in severely underestimates columns strengths when concrete strength above 110 MPa (16,000 psi).
5. The proposed triangular stress block provides accurate and conservative results for the axial and flexural strengths estimates. The degree of conservativeness is decreased as the concrete strength increased. The proposed triangular stress block lies between severely to moderately under estimates columns strengths in the range of concrete strength from 55 MPa (8,000) to 83 (12,000 psi). The severe conservativeness is for columns with concrete strengths below 69 MPa (10,000 psi). For concrete strength above 83 MPa (12,000), the proposed triangular stress block is slightly conservative. The proposed triangular stress block shows identical or close to identical results as compared with CSA A23.3, Bae and Bayrak, Ozbakkaloglu and Saatcioglu, and Ibrahim and MacGregor stress blocks for concrete strength above 110 MPa (16,000).
6. Considering the axial strength limit ($0.8\phi P_0$), the ACI 318 stress block should be avoided in column design for concrete strength above 83 MPa (12,000 psi). The proposed triangular stress block, CSA A23.3-04, Bae and Bayrak, Ozbakkaloglu and Saatcioglu, and Ibrahim and MacGregor stress blocks are recommended in such a case.
7. Despite of the axial strength limit ($0.8\phi P_0$) similar to other codes, the ACI 318 stress block parameters need to be changed to account for the overestimated nominal column strength for concrete strength above 69 MPa (10,000 psi). This study suggests to replace the ACI 318 rectangular stress block by the triangular stress block proposed herein for concrete strength above 69 MPa (10,000 psi). Addition of a new section 22.2.2.4.4 to the current ACI 318-14 provisions could implement a triangular stress block. Section 22.2.2.4.4 would read, above 69 MPa (10,000 psi) a triangular stress block with a maximum stress of $0.85f_c'$ shall be permitted.
8. The NZS 3101 code stops changing the stress block parameter α_1 for concrete strength above 80 MPa (11,600 psi). This resulted in nominal strength over predictions for HSC columns with concrete strength above 83 MPa (12,000 MPa). Also, nominal strength for some tested columns are over predicted by the NZS 3101 for the concrete strength of 75 MPa (10,800 psi). Therefore, as it has been suggested for the ACI 318, the proposed triangular stress block can be used as a replacement to the NZS 3101 stress block for concrete strength above 69 MPa (10,000 psi).
9. CEB-FIP Model Code or EN 1992 stress block results in moderate over predictions for the columns strengths in the range of concrete strength from 55 MPa (8,000) to 110 MPa (16,000 psi). Also, the CEB-FIP Model Code (2010) or EN 1992 (2004) stress block results in severely underestimated columns strengths for concrete strength above 110 MPa (16,000 psi). Therefore, the lower and the upper limits for CEB-FIP Model Code (2010) or EN 1992 (2004) stress block parameters need to be reconstituted

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Notations

A_s = area of tension reinforcement
A_s' = area of compression reinforcement
b = width of compression face of member
c = distance from extreme compression fiber to neutral axis
d = distance from extreme compression fiber to centroid of tension reinforcement
d' = distance from extreme compression fiber to centroid of compression reinforcement
f_c' = specified compressive strength of concrete
f_{cm} = measured compressive strength of concrete
f_s = stress in tension reinforcement
f_s' = stress in compression reinforcement
f_y = yield strength reinforcement
h = overall depth of member
M = moment
M_n = nominal flexural strength of the section

P = axial load
P_n = nominal axial strength
P_0 = nominal axial strength at zero eccentricity
α_1 = factor relating magnitude of uniform stress in equivalent rectangular concrete stress block to specified compressive strength of concrete nominal axial strength at zero eccentricity
β_1 = factor relating depth of equivalent rectangular concrete stress block to depth of neutral axis
γ = sustained load factor
ρ = ratio of A_s to bd
ρ' = ratio of A_s' to bd
ϕ = strength reduction factor