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# THE VALIDITY OF EXISTING INTERNATIONAL DESIGN CODES IN PREDICTION OF MOMENT CAPACITIES OF HIGH-STRENGTH CONCRETE MEMBERS

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**Abstract:** The availability and advancement of material technology and the acceptance has led to the production of higher grades of concrete. High strength concrete (HSC) offers superior engineering properties i.e. compressive strength, tensile strength, durability, modulus of elasticity and overall better performance when compared to the conventional concrete. Due to its enhanced strength and improved structural properties, high strength concrete has been increasingly used for the past two decades. In this research, many published studies on the behavior of HSC beams have been discussed and analyzed. High strength concrete used in this study is defined as concrete with compressive strength exceeding 50MPa. Although there are many publications proposing stress block models for HSC beams, a universally accepted stress block model is yet to be developed. In most design standards, the conventional rectangular stress block developed for normal strength concrete (NSC) is still being used for design of HSC beams. In this paper, published work has been analyzed to establish some understanding of flexural behavior of HSC beams. Models proposed in various design codes and standards have been analyzed to compare the experimental and theoretical moment capacities. A number of spread sheets in Excel were developed using available data and various graphs were plotted to determine the accuracy of the code provisions for calculating the ultimate moment capacity of beams. Based on this, conclusions are drawn for the design of high strength concrete beams in flexure utilizing different code provisions.

**Keywords:** *Design codes, moment capacities, high-strength concrete beams.*

## 1.0 Introduction

There has been a rapid growth in the use of high strength concrete because of its enhanced material and structural properties and the ability to gain high early strength. But lack of a proper design procedure discourages structural designers to make full use of the material. There are currently no design guidelines in the Egyptian Code (ECP 203-07) and most international existing codes for the design of concrete members with compressive strength in excess of 65MPa. Much experimental work on High strength concrete (HSC) has been carried out to date; however, findings are diverse and require careful analysis prior to proposing any change to the current provisions. Many suggestions have been made concerning the design rules, and there is now a need for finding the most appropriate and feasible design method for the flexural and shear strength capacities of beams.

This paper includes theoretical analysis of reinforced high strength concrete beams for design of normal weight HSC members with compressive strength greater than 50MPa. It will allow the design of high strength concrete beams for flexure, propose recommendations to be considered in the revised most codes and standards.

The aims of this study are to:

1. Review the existing literature and identify the gaps in knowledge. Conduct analytical study on flexure of high strength concrete beams; examine the code equations and suggest recommendations for the higher strength concrete for use by engineers in practice to design concrete members with compressive strength beyond the scope of the Egyptian Code and most international existing codes where no guidelines are currently available;
2. Examine the validity of the ECP 203-07 and other codes provisions for finding the flexural capacity of HSC beams;
3. Suggest the most feasible design method for use by the structural engineers. The general aim is to provide the accuracy of current design provisions to enable industry the use of high strength concrete with confidence and therefore able to utilize its benefits and acknowledge its limitations; and
4. Give an understanding of the behavior of beams in flexure when different stress block parameters are used for beams made of higher concrete strengths. The results from the analysis will add to the body of knowledge currently available and is also significant to building code writers since the current stress block parameters were developed for normal strength concrete.

## 2.0 Concrete Compressive Stress Block

For simplicity, a rectangular stress block is preferred for calculation of the ultimate moment capacity of reinforced concrete members. This ultimate strength is assumed to occur at a particular value of extreme fiber concrete strain, $\varepsilon_{cu}$. The stress block model was introduced by (Hognestad *et al.*, 1955) from experimental investigations making use of normal strength concrete. The rectangular stress block is defined by two parameters: $\alpha 1$ is the intensity of stress in the stress block and $\beta 1$ is the ratio of the depth of the stress block to the depth of the neutral axis. The rectangular stress block is found to be useful only for under-reinforced beams when the neutral axis lies within the cross-section as shown in Figure (1).

The typical stress-strain curve for high strength concrete is more linear than parabolic and the ultimate strain is lower for high strength concrete. Considering the differences in the stress-strain curves and other characteristics of high strength concrete, a modification of the rectangular stress block parameters is necessary. Table 1 summarizes various recommendations for the stress block parameters and $\varepsilon_{cu}$ from various design codes.

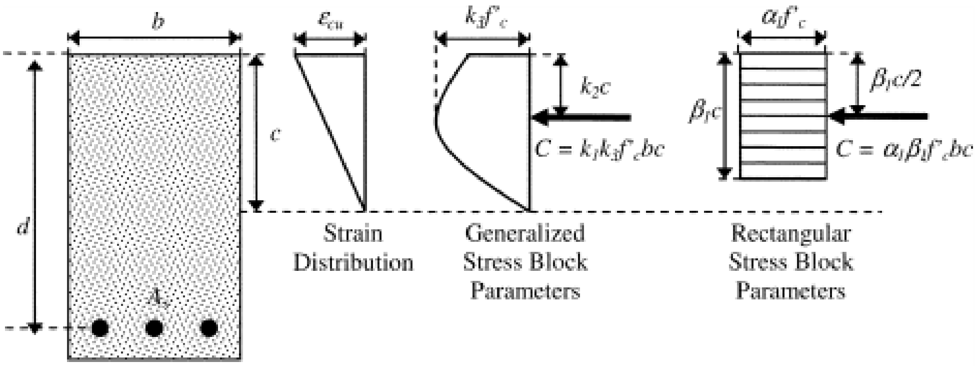


Figure 1: Stress block parameters for rectangular sections

Table 1: Rectangular stress block parameters in different design codes

| ***No.*** | ***Code*** | $\alpha_1$ ***(=k₁k₃)*** | $\beta_1$ ***(=k₂)*** | $\varepsilon_{cu}$ |
|---|---|---|---|---|
| 1 | ECP 203-07 | 0.84 | 0.80 | 0.003 |
| 2 | ACI 318-08 | 0.85 | $1.09 - 0.008 f'_c$ <br> $0.85 \geq \beta_1 \geq 0.65$ | 0.003 |
| 3 | CEB/FIP Model MC90 | $0.85(1 - f'_c / 250)$ | 1 | $0.004 - 0.002 f'_c / 100$ |
| 4 | CAN3-A23.3-M94 | $0.85 - 0.0015 f'_c \geq 0.67$ | $0.97 - 0.0025 f'_c \geq 0.67$ | 0.0035 |
| 5 | Eurocode-2 | 0.85 | $0.9 - f'_c / 500$ | 0.0035 |
| 6 | AS 3600 | 0.85 | $0.85 - 0.007(f'_c - 28)$ <br> $0.85 \geq \beta_1 \geq 0.65$ | 0.003 |
| 7 | NZS3101 | $1.07 - 0.004 f'_c$ <br> $0.85 \geq \alpha_1 \geq 0.75$ | $1.09 - 0.008 f'_c$ <br> $0.85 \geq \beta_1 \geq 0.65$ | 0.003 |

## 3.0 Test Specimens & Methodology

The test specimens consisted of 53 singly-reinforced high-strength concrete beams with rectangular cross section were collected from the literature. The parameters of this study were beam geometry (*b d*), amount of steel reinforcement ( $\rho$ ), concrete compressive strength ( $f'_c$ ). The beams were simply supported and subjected to pure bending. Each beam was loaded by two symmetrical concentrated loads. These Beams have been considered with a view to compare the ultimate strength of them in bending to the capacity predicted by different codes (7 codes). For a comparison to be made between the actual moment capacities and theoretical moment capacities, the theoretical moment capacities had to be based on the same parameters as the actual beams tested. The details and moment capacities have been given in Table (2).

Table 2: Details of beams and ultimate moment predictions using different codes

| *Reference* | *Beam Name* | $f'_c$ | *b* | *d* | $\rho$ | $f_y$ | $M_{exp}$ | $M_{exp}/M_{pred}$ | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | *MPa* | *mm* | *mm* | *%* | *N/mm²* | *kN.m* | *1* | *2* | *3* | *4* | *5* | *6* | *7* |
| *(Pam et al., 2001)* | 1 | 37.4 | 200 | 264 | 0.76 | 579 | 77.6 | 1.36 | 1.36 | 1.37 | 1.36 | 1.36 | 1.36 | 1.35 |
| | 2 | 36.8 | 200 | 264 | 1.14 | 579 | 103.5 | 1.26 | 1.26 | 1.28 | 1.27 | 1.26 | 1.26 | 1.24 |
| | 3 | 36.4 | 200 | 260 | 1.89 | 578 | 126.5 | 1.04 | 1.04 | 1.08 | 1.06 | 1.04 | 1.04 | 1.03 |
| | 4 | 42.3 | 200 | 260 | 1.89 | 536 | 129 | 1.10 | 1.10 | 1.13 | 1.11 | 1.10 | 1.10 | 1.12 |
| | 5 | 46.4 | 200 | 260 | 2.28 | 300 | 142.8 | 1.69 | 1.16 | 1.05 | 1.03 | 1.01 | 1.00 | 1.01 |
| | 7 | 58.6 | 200 | 260 | 2.49 | 300 | 164.6 | 1.76 | 1.09 | 1.13 | 1.10 | 1.08 | 1.07 | 1.08 |
| | 8 | 57.1 | 200 | 260 | 2.86 | 300 | 166.2 | 1.57 | 1.02 | 1.04 | 1.00 | 0.98 | 0.98 | 0.99 |
| | 9 | 58.6 | 200 | 256 | 3.53 | 300 | 171.6 | 1.39 | 0.70 | 0.94 | 0.90 | 0.87 | 0.87 1 | 0.90 |
| | 14 | 95.5 | 200 | 260 | 1.89 | 300 | 138 | 1.87 | 1.46 | 1.05 | 1.02 | 1.00 | 1.00 | 1.01 |
| | 15 | 98 | 200 | 260 | 2.84 | 300 | 200.7 | 1.84 | 0.91 | 1.08 | 1.03 | 1.01 | 1.00 | 1.02 |
| | 16 | 102.5 | 200 | 260 | 2.84 | 300 | 181.7 | 1.66 | 0.86 | 0.98 | 0.93 | 0.91 | 0.90 | 0.92 |
| | 17 | 87 | 200 | 256 | 3.14 | 300 | 172 | 1.49 | 0.96 | 0.93 | 0.89 | 0.87 | 0.86 | 0.88 |
| *(Sarkar et al., 1997)* | HSC 1-1 | 107 | 150 | 220 | 1.03 | 470 | 38.94 | 1.14 | 1.14 | 1.16 | 1.15 | 1.14 | 1.14 | 1.15 |
| | HSC 1-2 | 97 | 150 | 220 | 1.03 | 470 | 35.64 | 1.05 | 1.05 | 1.07 | 1.05 | 1.05 | 1.05 | 1.05 |
| | HSC | 85 | 150 | 220 | 1.03 | 442 | 37.62 | 1.18 | 1.18 | 1.20 | 1.19 | 1.18 | 1.18 | 1.18 |

| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1-3 | | | | | | | | | | | | |
| | HSC 2-1 | 105 | 150 | 213 | 1.42 | 470 | 46.33 | 1.06 | 1.07 | 1.10 | 1.08 | 1.07 | 1.07 | 1.07 |
| | HSC 2-2 | 100 | 150 | 213 | 1.42 | 470 | 46.86 | 1.07 | 1.08 | 1.11 | 1.09 | 1.08 | 1.08 | 1.09 |
| | HSC 2-3 | 77 | 150 | 213 | 1.42 | 442 | 43.56 | 1.07 | 1.08 | 1.10 | 1.09 | 1.08 | 1.08 | 1.08 |
| | HSC 2-4 | 90 | 150 | 213 | 1.42 | 442 | 48.84 | 1.19 | 1.20 | 1.23 | 1.21 | 1.20 | 1.20 | 1.21 |
| | HSC 3-1 | 107 | 150 | 215 | 1.94 | 470 | 67.32 | 1.12 | 1.12 | 1.16 | 1.13 | 1.12 | 1.12 | 1.12 |
| | HSC 3-2 | 85 | 150 | 215 | 1.94 | 470 | 66 | 1.12 | 1.11 | 1.15 | 1.12 | 1.11 | 1.11 | 1.12 |
| | HSC 3-3 | 78 | 150 | 215 | 1.94 | 442 | 64.68 | 1.16 | 1.16 | 1.20 | 1.17 | 1.16 | 1.16 | 1.17 |
| | HSC 4-1 | 101 | 150 | 208 | 4.04 | 470 | 92.42 | 0.84 | 0.85 | 0.93 | 0.87 | 0.85 | 0.85 | 0.86 |
| | HSC 4-2 | 87 | 150 | 208 | 4.04 | 470 | 89.6 | 0.84 | 0.84 | 0.91 | 0.86 | 0.84 | 0.84 | 0.86 |
| | HSC 4-3 | 82 | 150 | 208 | 4.04 | 442 | 111.63 | 1.11 | 1.11 | 1.20 | 1.14 | 1.11 | 1.11 | 1.12 |

| ***Reference*** | ***Beam Name*** | $f'_c$ | ***b*** | ***d*** | $\rho$ | $f_y$ | $M_{exp}$ | $M_{exp}/M_{pred}$ | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ***MPa*** | ***mm*** | ***mm*** | ***%*** | ***N/mm²*** | ***kN.m*** | ***1*** | ***2*** | ***3*** | ***4*** | ***5*** | ***6*** | ***7*** |
| ***(Pam et al., 2001)*** | 1 | 37.4 | 200 | 264 | 0.76 | 579 | 77.6 | 1.36 | 1.36 | 1.37 | 1.36 | 1.36 | 1.36 | 1.35 |
| | 2 | 36.8 | 200 | 264 | 1.14 | 579 | 103.5 | 1.26 | 1.26 | 1.28 | 1.27 | 1.26 | 1.26 | 1.24 |
| | 3 | 36.4 | 200 | 260 | 1.89 | 578 | 126.5 | 1.04 | 1.04 | 1.08 | 1.06 | 1.04 | 1.04 | 1.03 |
| | 4 | 42.3 | 200 | 260 | 1.89 | 536 | 129 | 1.10 | 1.10 | 1.13 | 1.11 | 1.10 | 1.10 | 1.12 |
| | 5 | 46.4 | 200 | 260 | 2.28 | 300 | 142.8 | 1.69 | 1.16 | 1.05 | 1.03 | 1.01 | 1.00 | 1.01 |
| | 7 | 58.6 | 200 | 260 | 2.49 | 300 | 164.6 | 1.76 | 1.09 | 1.13 | 1.10 | 1.08 | 1.07 | 1.08 |
| | 8 | 57.1 | 200 | 260 | 2.86 | 300 | 166.2 | 1.57 | 1.02 | 1.04 | 1.00 | 0.98 | 0.98 | 0.99 |
| | 9 | 58.6 | 200 | 256 | 3.53 | 300 | 171.6 | 1.39 | 0.70 | 0.94 | 0.90 | 0.87 | 0.871 | 0.90 |
| | 14 | 95.5 | 200 | 260 | 1.89 | 300 | 138 | 1.87 | 1.46 | 1.05 | 1.02 | 1.00 | 1.00 | 1.01 |
| | 15 | 98 | 200 | 260 | 2.84 | 300 | 200.7 | 1.84 | 0.91 | 1.08 | 1.03 | 1.01 | 1.00 | 1.02 |

| | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 16 | 102.5 | 200 | 260 | 2.84 | 300 | 181.7 | 1.66 | 0.86 | 0.98 | 0.93 | 0.91 | 0.90 | 0.92 |
| | 17 | 87 | 200 | 256 | 3.14 | 300 | 172 | 1.49 | 0.96 | 0.93 | 0.89 | 0.87 | 0.86 | 0.88 |
| ***(Sarkar et al., 1997)*** | HSC 1-1 | 107 | 150 | 220 | 1.03 | 470 | 38.94 | 1.14 | 1.14 | 1.16 | 1.15 | 1.14 | 1.14 | 1.15 |
| | HSC 1-2 | 97 | 150 | 220 | 1.03 | 470 | 35.64 | 1.05 | 1.05 | 1.07 | 1.05 | 1.05 | 1.05 | 1.05 |
| | HSC 1-3 | 85 | 150 | 220 | 1.03 | 442 | 37.62 | 1.18 | 1.18 | 1.20 | 1.19 | 1.18 | 1.18 | 1.18 |
| | HSC 2-1 | 105 | 150 | 213 | 1.42 | 470 | 46.33 | 1.06 | 1.07 | 1.10 | 1.08 | 1.07 | 1.07 | 1.07 |
| | HSC 2-2 | 100 | 150 | 213 | 1.42 | 470 | 46.86 | 1.07 | 1.08 | 1.11 | 1.09 | 1.08 | 1.08 | 1.09 |
| | HSC 2-3 | 77 | 150 | 213 | 1.42 | 442 | 43.56 | 1.07 | 1.08 | 1.10 | 1.09 | 1.08 | 1.08 | 1.08 |
| | HSC 2-4 | 90 | 150 | 213 | 1.42 | 442 | 48.84 | 1.19 | 1.20 | 1.23 | 1.21 | 1.20 | 1.20 | 1.21 |
| | HSC 3-1 | 107 | 150 | 215 | 1.94 | 470 | 67.32 | 1.12 | 1.12 | 1.16 | 1.13 | 1.12 | 1.12 | 1.12 |
| | HSC 3-2 | 85 | 150 | 215 | 1.94 | 470 | 66 | 1.12 | 1.11 | 1.15 | 1.12 | 1.11 | 1.11 | 1.12 |
| | HSC 3-3 | 78 | 150 | 215 | 1.94 | 442 | 64.68 | 1.16 | 1.16 | 1.20 | 1.17 | 1.16 | 1.16 | 1.17 |
| | HSC 4-1 | 101 | 150 | 208 | 4.04 | 470 | 92.42 | 0.84 | 0.85 | 0.93 | 0.87 | 0.85 | 0.85 | 0.86 |
| | HSC 4-2 | 87 | 150 | 208 | 4.04 | 470 | 89.6 | 0.84 | 0.84 | 0.91 | 0.86 | 0.84 | 0.84 | 0.86 |
| | HSC 4-3 | 82 | 150 | 208 | 4.04 | 442 | 111.63 | 1.11 | 1.11 | 1.20 | 1.14 | 1.11 | 1.11 | 1.12 |

| ***Reference*** | ***Beam Name*** | ***$f'_c$*** | ***b*** | ***d*** | $\rho$ | ***$f_y$*** | ***$M_{exp}$*** | $M_{exp}/M_{pred}$ | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ***MPa*** | ***mm*** | ***mm*** | ***%*** | ***$N/mm^2$*** | ***kN.m*** | ***1*** | ***2*** | ***3*** | ***4*** | ***5*** | ***6*** | ***7*** |
| ***(Ashour, 2000)*** | B-N2 | 48.61 | 200 | 215 | 1.18 | 530 | 58.17 | 1.09 | 1.09 | 1.11 | 1.09 | 1.09 | 1.09 | 1.09 |
| | B-N3 | 48.61 | 200 | 215 | 1.77 | 530 | 57.95 | 0.76 | 1.05 | 1.08 | 1.06 | 1.05 | 1.05 | 1.05 |
| | B-N4 | 48.61 | 200 | 215 | 2.37 | 530 | 56.8 | 0.58 | 1.01 | 1.06 | 1.03 | 1.01 | 1.01 | 1.01 |
| | B-M2 | 78.5 | 200 | 215 | 1.18 | 530 | 80.6 | 1.46 | 1.05 | 1.07 | 1.06 | 1.05 | 1.05 | 1.05 |
| | B-M3 | 78.5 | 200 | 215 | 1.77 | 530 | 79.91 | 0.99 | 0.99 | 1.02 | 1.00 | 0.99 | 0.99 | 0.99 |

| | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | B-M4 | 78.5 | 200 | 215 | 2.37 | 530 | 82.76 | 0.79 | 0.99 | 1.04 | 1.00 | 0.99 | 0.99 | 0.99 |
| | B-H2 | 102.4 | 200 | 215 | 1.18 | 530 | 99.55 | 1.79 | 1.02 | 1.04 | 1.02 | 1.02 | 1.02 | 1.02 |
| | B-H3 | 102.4 | 200 | 215 | 1.77 | 530 | 103.77 | 1.27 | 1.01 | 1.05 | 1.02 | 1.01 | 1.01 | 1.01 |
| | B-H4 | 102.4 | 200 | 215 | 2.37 | 530 | 108.1 | 1.00 | 1.00 | 1.06 | 1.02 | 1.00 | 1.00 | 1.00 |

Table 3: Summary of correlation for all beams

| No. | Code | $M_{exp}/M_{pred}$ | |
|---|---|---|---|
| | | **Mean** | **COV, %** |
| 1 | ECP 203-07 | 1.376 | 24.35 |
| 2 | ACI 318-08 | 0.976 | 16.96 |
| 3 | CEB/FIP Model MC90 | 1.011 | 13.53 |
| 4 | CAN3-A23.3-M94 | 0.982 | 14.41 |
| 5 | Eurocode-2 | 0.968 | 14.92 |
| 6 | AS 3600 | 0.967 | 14.93 |
| 7 | NZS3101 | 0.974 | 14.41 |

Table 4: Summary of correlation excluding beams tested by (Bernardo and Lopes, 2004)

| No. | Code | $M_{exp}/M_{pred}$ | |
|---|---|---|---|
| | | **Mean** | **COV, %** |
| 1 | ECP 203-07 | 1.227 | 26.72 |
| 2 | ACI 318-08 | 1.062 | 13.51 |
| 3 | CEB/FIP Model MC90 | 1.091 | 9.11 |
| 4 | CAN3-A23.3-M94 | 1.063 | 10.13 |
| 5 | Eurocode-2 | 1.05 | 10.65 |
| 6 | AS 3600 | 1.048 | 10.76 |
| 7 | NZS3101 | 1.054 | 10.18 |

## 4.0 Correlation of Test Moment Capacity with Predictions by Various Codes

Various code provisions for flexural capacity of concrete beams have been described. The experimental moment capacities of the 53 beams tested by (Pam *et al.*, 2001), (Sarkar *et al.*, 1997), (Bernardo and Lopes, 2004), and (Ashour, 2000) have been compared to the predictions by the codes. Although various recommendations made by the design codes are considerably different in nature, capacities of reinforced concrete members can be predicted with similar level of accuracy by using any of the aforementioned recommendations. The effect of using accurate and conservative stress block parameters for high strength concrete beams will be pronounced. For this purpose, a comparison of test moment capacity to predictions by the various codes has been carried out using spread sheets in Excel. Details of beams and ultimate moment predicted using different codes have been given in Table (2).

A summary of the correlation has been given in Table (3). The summary of correlation indicates significant scatter in the predictions by the above methods. Figure (2) show the correlation of the test moment capacity versus the predicted moment capacities of the beams. The mean values of $M_{exp}/M_{pred}$ are also given and the coefficient of variation determined. CEB/FIP Model MC90 gave the best prediction with the smallest scatter. The mean value $M_e/M_p$ is 1.011 with a smallest coefficient of variation equal 13.53%. Most of the results fall either within the ±20% band of the ideal 1:1 test moment capacity versus predicted moment capacity line, or above this band as shown in Figure (2). On the contrary, ECP 203-07 gave the worse prediction (more conservative), it strongly underestimated the capacities of the HSC beams, with a larger scatter and the results fall within the -50% band. It is found that for beams tested by (Bernardo and Lopes, 2004), the predictions from the theoretical point of view were a bit unconservative.

All of the methods used to determine the strength of beams in bending tested by (Bernardo and Lopes, 2004), overestimate the capacity of the beam and produce theoretical moment capacity which is more than the actual capacity of the beam. It is believed that design guidelines should provide similar levels of conservativeness for NSC as well as HSC (Bae and Bayrak, 2003). In this regard, it is said that the current code provisions are unsuitable for use in designing HSC beams subjected to flexure. From Table (3), it is noted that all the code provisions involving different stress block parameters used to predict the ultimate strength show a coefficient of variation (COV). The mean value of $M_{exp}/M_{pred}$ is found to be less than 1 for all the methods except for CEB/FIP Model MC90 and ECP 203-07 codes, where $M_{exp}/M_{pred}$ is found to be 1.011 and 1.376 for both codes respectively. It is concluded that all the methods involving different stress block parameters are unconservative for use with high strength concrete beams.

Another analysis of the data excluding the beams tested by (Bernardo and Lopes, 2004) has been done to investigate the effect of the stress block parameters. It is noted that all methods give conservative results as depicted in Table (4). The theoretical moment capacity is found to be less than the actual moment capacity and the $M_{exp}/M_{pred}$ ratio is found to be always greater than 1.

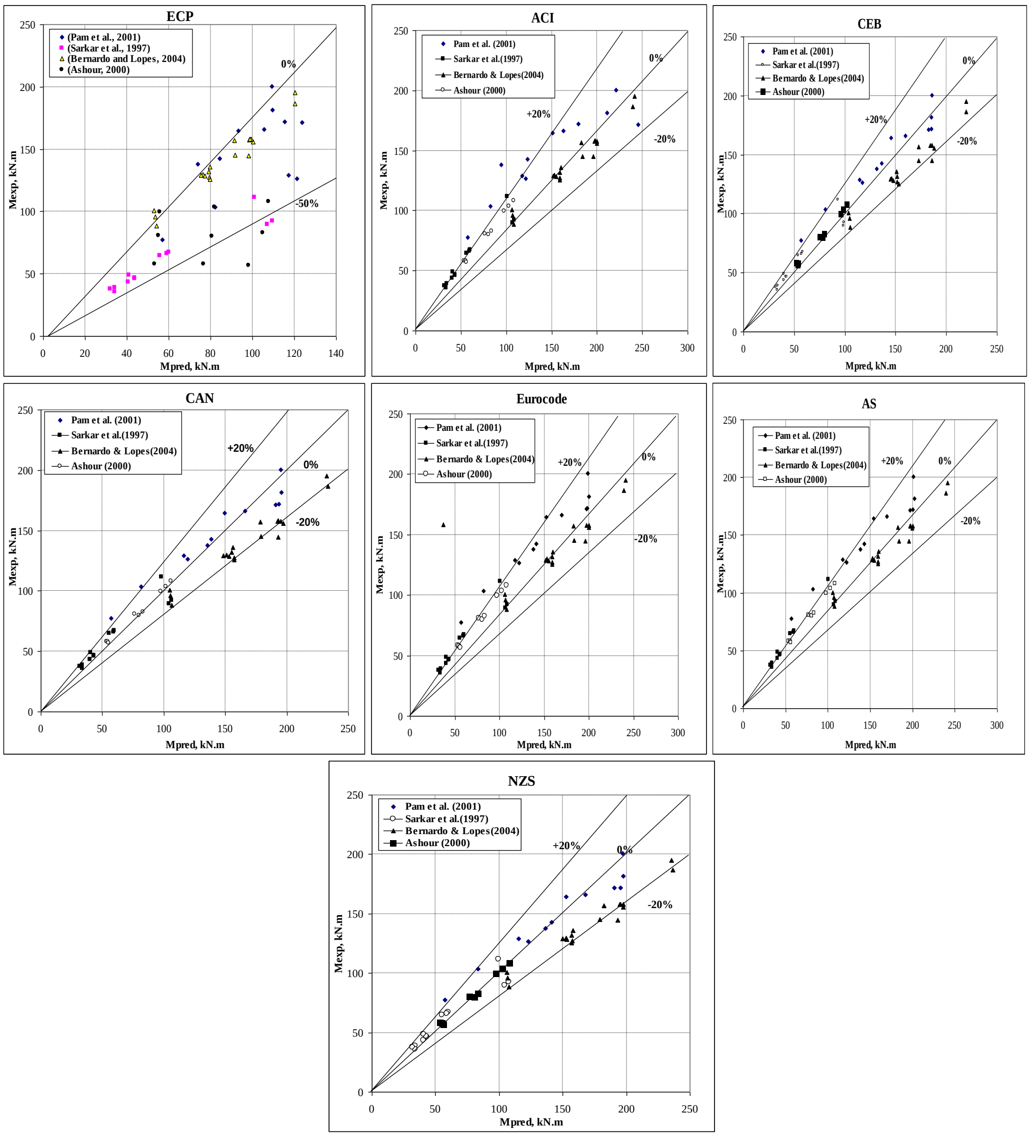


Figure 2: Correlation of experimental moment capacity vs. moment capacity predicted by different codes: ECP 203-07, ACI 318-08, CEB/FIP Model MC90, CAN 3-A23.3-M94, Eurocode-2, AS 3600, and NZS3101respectively.

## 5.0 Discussion

The use of formulae for calculating the moment capacity of reinforced concrete beams by engineers makes it obvious that the theoretical moment capacity should be less than the actual moment capacity. The results obtained theoretically for the calculation of ultimate strength must be conservative. The design rules should provide similar level of conservativeness for normal and high strength concrete (Bae and Bayrak, 2003). From Table (2), it is seen that the rectangular stress block approach is satisfactory for all the beams tested by (Pam *et al*., 2001), (Sarkar *et al*., 1997) and (Ashour, 2000) considered in this study except for the beams tested by Bernardo and Lopes (2004).

It has been found that code provisions conservatively predict the ultimate strength of most of the beams tested by (Pam *et al*., 2001), (Sarkar *et al*., 1997) and (Ashour, 2000) and unconservatively predict the ultimate strength of beams tested by (Bernardo and Lopes, 2004). All the methods produce theoretical moment capacities which are less than the actual moment capacity for the beams. In case of beams tested by (Bernardo and Lopes, 2004), the theoretical moment capacity that is calculated by each method (except ECP 203-07) is greater than the actual moment capacity. The reason for this has been attributed to the inaccuracy of the experimental moment capacity that is calculated from the ultimate load obtained from the test results.

It is noted that there is not much difference between the various stress block parameters. The ACI318-08 stress block parameters show a high coefficient of variation (16.96%). The use of different stress block parameters have been found to yield unconservative estimations for beam capacity as concrete compressive strength increases. Hence, it is concluded that all the stress block parameters used to predict the ultimate strength are unconservative. But the extent of unconservativeness is not large. The degree of accuracy and unconservativeness of all the stress block parameters considered for this study appears similar from the figures.

Verification of the Egyptian Code (ECP 203-2007) and others in terms of strength
A comparison between the measured values of the ultimate moment capacity for the tested beams from the literature and those predicted by the ECP 203-2007 and others are given as mention earlier in Table (2).

Generally, the predicted values underestimate the experimental ones. The results show that the ECP 203-07 is more conservative compared to the all design codes providing safer estimate of the flexural capacity. Because of higher factor of safety are taken resulting in lower internal forces and internal moment arm, thus the internal moment of resistance produced by the section is reduced.

## 6.0 Conclusions

Based on this study, the following conclusions can be made:

1. The rectangular stress block approximation will not give a conservative estimate of the moment capacity of HSC beams. Until future work is completed to address this and develop a conservative model, it is recommended that a reduction factor of 0.8 be used on all HSC beams designed using the rectangular stress block theory.
2. CEB/FIP Model MC90 code gave the best prediction of flexural capacities of HSC beams compared with other international design codes.
3. Using the equivalent rectangular stress block proposed in the Egyptian Code (ECP 203-07) provides more conservative estimate of the flexural capacity of singly reinforced high strength concrete beams with $f'_c$ up to 107 MPa compared to the other codes. The factor of safety by the ECP 203-07 must be reduced for high strength concrete.

## List of Symbols

| | |
|---|---|
| $A_s$ | Area of tensile reinforcement |
| $b$ | Breadth of section |
| $c$ | Depth to neutral axis |
| $C$ | Concrete compressive force |
| $d$ | Effective depth to reinforcement |
| $M_{exp}$ | Experiment ultimate moment capacity results from the literatures |
| $M_{pred}$ | Predicted ultimate moment capacity by different codes |
| $f'_c$ | Cylinder compressive strength of concrete |
| $f_s$ | Ultimate strength of longitudinal steel |
| $T$ | Tension force in reinforcement |
| $k_1, k_2, k_3$ | Rectangular stress block parameters |
| $\varepsilon_{cu}$ | Extreme fibre concrete ultimate compressive strain |
| $\varepsilon_s$ | Strain in steel |
| $\alpha_1$ | Coefficient that defines width of rectangular stress block |
| $\beta_1$ | Coefficient that defines height of rectangular stress block |

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