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# Comparison between ACI 318-05 and Eurocode 2 (EC2-94) in flexural concrete design

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**Abstract.** The two major widely used building design code documents of reinforced concrete structures are the ACI 318-05 and Eurocode for the Design of Concrete Structures EC2. Therefore, a thorough comparative analysis of the provisions of these codes is required to confirm their validity and identify discrepancies in either code. In this context, provisions of flexural computations would be particularly attractive for detailed comparison. The provisions of safety concepts, design assumptions, cross-sectional moment capacity, ductility, minimum and maximum reinforcement ratios, and load safety factors of both the ACI 318-05 and EC2 is conducted with parametric analysis. In order to conduct the comparison successfully, the parameters and procedures of EC2 were reformatted and defined in terms of those of ACI 318-05. This paper concluded that although the adopted rationale and methodology of computing the design strength is significantly different between the two codes, the overall EC2 flexural provisions are slightly more conservative with a little of practical difference than those of ACI 318-05. In addition, for the limit of maximum reinforcement ratio, EC2 assures higher sectional ductility than ACI 318-05. Overall, EC2 provisions provide a higher safety factor than those of ACI 318-05 for low values of Live/Dead load ratios. As the ratio increases the difference between the two codes decreases and becomes almost negligible for ratios higher than 4.

**Keywords:** ACI 318-05; EC2-94; Reinforced Concrete; Flexural Design; reinforcement ratio; ductility; safety concepts.

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## 1. Introduction

The American Building Code Requirements for Structural Concrete “ACI 318-05”, and Commentary “ACI 318R-05” (2004) covers the proper design and construction of structural concrete buildings. The commentary of the ACI code discusses some of the considerations of the committee in developing the code with emphasis given to the explanation of new or revised provisions from older editions of the ACI Code.

On the other hand, the design of concrete structures is covered in Part 1-1 of the ENV version of Eurocode 2 (2004) “*General Rules and Rules for Buildings*”. In the Spring of 1992, the British Standard Institute (BSI 1992) published a National Application Document (NAD) in conjunction with ENV. The document had been through several draft revisions and was finally published in 2004 by BSI (BSI 2004). EC2 Part1-1 applies to the design of buildings and civil engineering works in plain, reinforced and prestressed concrete.

Comparative studies by other investigators were conducted between EC2 and the British code BS 8110. Among them, Narayanan (1994), and Moss and Webster (2004) compared the concrete design provisions of EC2 with BS 8110. The conclusions drawn from the comparisons validate both codes of practice and shows the major differences between the provisions of both codes in flexural design, allowable span to depth ratio, beam shear, compression design, punching shear, detailing, and biaxial bending. It was concluded that EC2 scope is more general and its provisions are less restrictive than those of BS 8110 which may encourage innovative design methods. For the design and analysis of flexural concrete members at the ultimate limit state, it was concluded that EC2 provisions are very similar to those of BS 8110 with a very little of practical difference. The major difference between the two codes is that EC2 requires the application of general principles (e.g., bending, shear etc.) for the design method rather than by element types as in BS 8110 (e.g., beams, slabs, columns etc.).

EC2 also allows benefits to be derived from using high strength concretes, which BS 8110 does not. Pam *et al*. (2001) studied the post-peak behavior and flexural ductility of normal and high strength concrete beams by evaluating the complete moment-curvature curves. The analytical method adopted took into account the stress-path of the materials’ constitutive properties. It was concluded that the major factors affecting the flexural ductility are the steel tension and compression reinforcement ratios and concrete grade. Accordingly, Pam *et al*. (2001) developed a formula for direct evaluation of the flexural ductility that can be used for both normal and high strength concrete. It should be noted that generally in the design of reinforced concrete beams, the yield strength of steel is used as the steel tensile strength for the evaluation of the flexural strength. However, the tensile strength of the steel reinforcement is substantially higher than the yield stress due to strain hardening. This design approach will lead to a conservative design resulting in a actual flexural strength higher than the theoretical flexural strength with strain hardening ignored. Ho *et al*. (2001) studied the effects of strain hardening of the reinforcement steel on the flexural strength and ductility of normal and high strength concrete beams. It was concluded that for beams with relatively small tension steel reinforcement, the effects of strain hardening could be quite significant. Recently, researchers are investigating the structural behavior of corrosion affected structures for repair, replacement, and strengthening applications. Ning *et al*. (2001) developed a reliable and accurate method to predict the flexural deformation and response of structural concrete members subject to service load. The method related the extent of concrete cracking, measured as a function of the magnitude of applied moment in a member, to the reduction in the effective moment of inertia of cracked reinforced concrete members under service load conditions. The method was verified with reference to experimental investigations and shoed a good correlation with experimental results.

Bhargava *et al.* (2007) developed formulation to predict the loss of cross-sectional area and weight loss for reinforcing bars. The developed formulation was used to evaluate the ultimate moment and shear capacity of corroded concrete beams. Their analytical predicted values for the ultimate moment and shear capacity of the corroded beams agreed reasonably well the experimental values.

Two philosophies of design have been prevalent by the ACI Code. The working stress method, focusing on conditions at service load (i.e., when the structure is being used), was the principle method used by the ACI Code from the early 1900s until the early 1960s. A structural element is so designed according to this method that the stresses resulting from the action of service loads (also called working loads) and computed by the mechanics of elastic members do not exceed allowable values. Since 1983, with few exceptions, the strength design method is used, focusing on conditions at loads greater than service loads at failure. Mast (1992) developed a unified design provisions for reinforced and prestressed concrete flexural and compression members. The strength design method is deemed conceptually more realistic to establish structural safety and economy. In the strength design method (ultimate strength method), the service loads are increased sufficiently by load factors to obtain the load at which failure is considered to be imminent. This load is called the factored load or factored service load. The structure or structural element is then proportioned such that the strength is reached when the factored load is acting.

In the new ACI 318-05 (2005) code, a unified design method has been adopted to unify the design requirements and methods with respect to beams and columns (flexure and compression) without violating the fundamental principles on which the provisions are based. The concrete beam cross-section is characterized as tension controlled. This method is based on the strain limit rather than stress limits. According to this approach, the nominal flexural strength is reached when the strain in the extreme compression concrete fiber reaches $\varepsilon_c = 0.003$ and the strain in the tension reinforcement is greater than or equal to $\varepsilon_t \geq 0.005$. This corresponds to a reinforcement ratio of $\rho/\rho_b = 0.63$, representing a fully ductile behavior. In order to verify, validate, and to help the designers to understand this new approach the provisions of ACI 318-05 are compared with those of EC2.

Eurocode 2 (EC2) is the key document for future structural design in concrete throughout Europe. The challenge in this research is to facilitate the comparison between the two code provisions. As a result, a technique was adopted that reformat and defines the parameters of EC2 in terms of the parameters of the ACI 318-05 code. The design assumptions, different models of stress distribution at ultimate strength across the section height, design procedures, required strength, and design strength of reinforced concrete members adopted by both codes will be fully presented and compared in this paper.

The major objective of this paper is to compare the ultimate limit state in flexural design of both the ACI318-05 (2004) and EC2 (2004). A limit state is condition, which represents the limit of structural usefulness. There are two limit states in design:

- Serviceability Limit State: defines functional requirements such as deflections and crack control
- Ultimate Limit State: defines safety against extreme loading during the intended life of structure

Both codes have the same concept in the ultimate limit state as shown qualitively in Fig. 1. The required strength should be smaller or at least equal to the design strength. In ACI 318-05 terminology

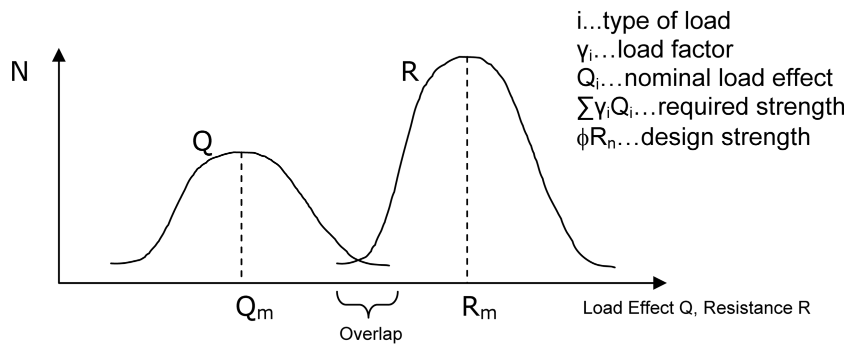



Fig. 1 Frequency of occurrence versus load effects and resistance

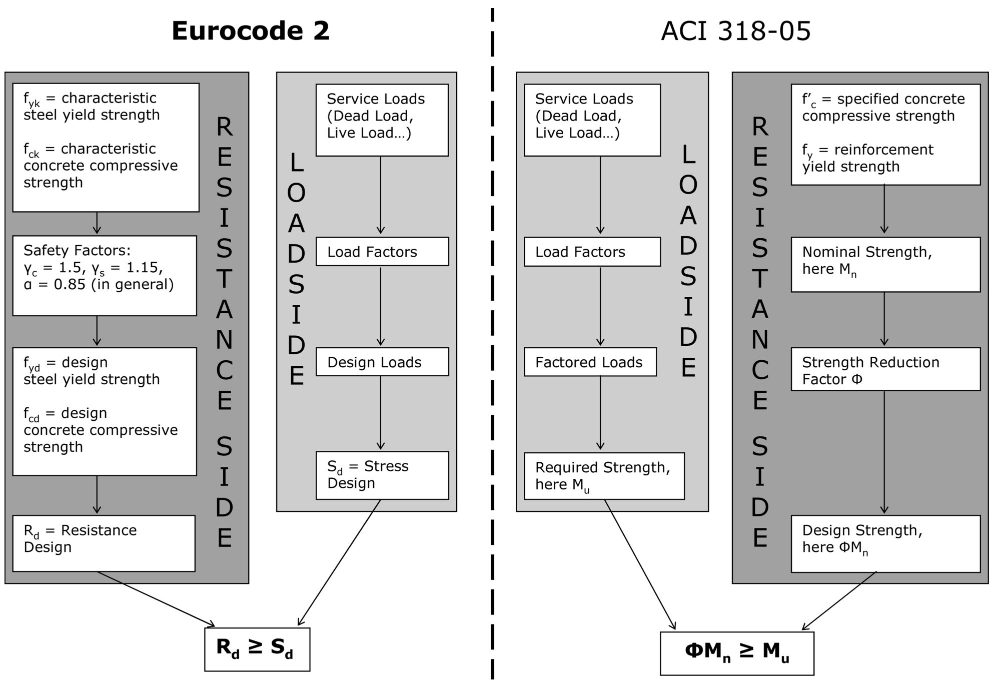



Fig. 2 Comparison of safety concepts adopted by both codes

$$\Sigma \gamma i Q i \leq \phi R n \tag{1}$$

where, $i$ is the type of load, $\gamma_i$ is the load factor, $Q_i$ is the nominal load effect, $\phi$ is the strength reduction factor, and $R_n$ is the nominal section strength (Resistance).

## 2. Safety concepts

Load and resistance safety concepts are adopted by both codes with the steps summarized in Fig. 2. According to EC2, the material strength is reduced by dividing the yield strength of reinforcing steel and the concrete compressive strength by partial safety factors as follows

$$f_{yd} = \frac{f_{yk}}{\gamma_s} \tag{2}$$

$$f_{cd} = \frac{f_{ck}}{\gamma_c} \tag{3}$$

where,

$f_{yd}$ = design steel yield strength
$f_{yk}$ = characteristic steel yield steel
$\gamma_s$ = 1.15 (safety factor for steel yield strength)
$f_{cd}$ = design concrete compressive strength
$f_{ck}$ = characteristic concrete compressive strength
$\gamma_c$ = 1.5 (safety factor for concrete compressive strength)

The ACI 318-05 code on the other hand, reduces the nominal moment strength of the member's cross-section by an overall reduction factor such that

$$M_{\text{des}} = \phi\, M_n \tag{4}$$

where,

$M_{\text{des}}$ = design moment strength
$\phi$ = strength reduction factor
$M_n$ = nominal moment strength

The major difference between the two codes is that EC 2 strength safety factors are applied to both the yield strength of the reinforcing steel and compressive strength of concrete. On the other hand ACI 318-05 has an overall strength reduction factor applied to the nominal moment strength of the cross-section.

Both codes multiply the service loads by load factors. The load factors of the ACI 318-05 code are 1.2 for the dead load (DL) and 1.6 for the live load (LL). EC2 load factors on the other hand are 1.35 for the dead load (DL) and 1.5 for the live load (LL).

## 3. Definitions and limitations

In order to compare the nominal moment and design strength between the two codes, both the concrete compressive strength and steel yield strength should be well defined as follows:

**EC2** (characteristic compressive strength, $f_{ck}$):

- 28 days, strength is tested on cylinders (Dia ≈ 6in., h ≈ 12in.)
- 95% Quantile

EC2 is based on the characterstic cylinder strength rather than the cube strength. The cylinder strength is typically 10-20% less than the corresponding cube strength (e.g., for class C30/37

Table 1 EC2 Compressive cylinder and cube strength, and tensile strength of concrete

| Symbol | Description | Properties | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| $f_{\text{ck}}$ (MPa) | Characteristic cylinder strength | 12 | 16 | 20 | 25 | 30 | 35 | 40 | 45 | 50 |
| $f_{\text{ck,cube}}$ (MPa) | Characteristic cube strength | 15 | 20 | 25 | 30 | 37 | 45 | 50 | 55 | 60 |
| $f_{\text{ctm}}$(MPa) | Mean tensile strength | 1.6 | 1.9 | 2.2 | 2.6 | 2.9 | 3.2 | 3.5 | 3.8 | 4.1 |

concrete the cylinder strength is 30 MPa, whereas the cube strength is 35 MPa). Table 1 provides a listing of concrete properties for the cylinder, cube, and mean tensile strengths up to a class of C50/60. EC2 allows benefits to use high-strength concrete strengths up to a maximu grade class of C90/105. However for classes above C50/60, the code specifies additional rules and variations. Accordingly, concrete grades up to a class of C50/60 are considered in this paper.

**ACI** (specified compressive strength, $f_c'$):

- 28 days, strength is tested on cylinders (Dia = 6in., h = 12in.)
  $\rightarrow$ 95% Quantile

$\rightarrow f_{ck}$ (EC2) $= f_c'$ (ACI 318-05)

Reinforcment Steel strength

$\rightarrow f_{cy}$ (EC2 characterstic yield strength) $= f_y$ (ACI reinforcement yield shtrength)

It should be noted that EC2 can be used with reinforcement characteristic strength ranging from 400 to 600 MPa. In the UK reinforcement industry, characteristic yield strength of 500 MPa has been adopted. The ACI 318-05 minimum yield strengths of the reinforcement steel ranges from 300 to 520 MPa.

Furthermore, the following limitations and simplifications are considered in the comparative analysis of this study:

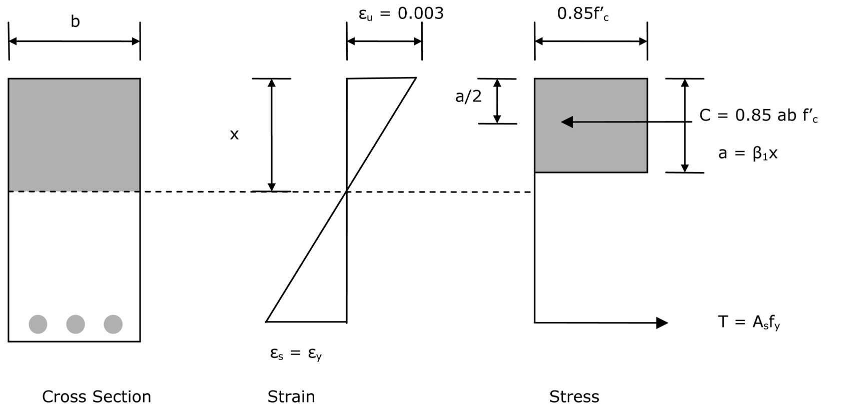
- Same concrete cover
- Same shear reinforcement
- Fully elastic analysis (moment redistribution is not considered, EI is constant)
- Service Loads are assumed to be equal
- A characteristic reinforcement yield strength of 500 MPa is used
- Concrete grades up to a class of C50/60 is considered
- Compression reinforcement is not considered

## 4. Design assumptions

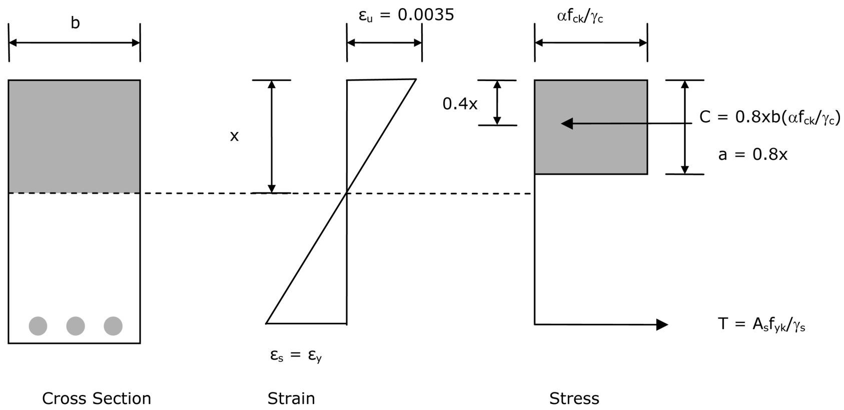
For the analysis of the internal forces and moments, the following assumptions are considered:

- The strain in the reinforcement steel and concrete shall be assumed directly proportional to the distance from the neutral axis. That means plane sections remain plane after deflection. (Bernoulli Hypothesis)
- The bond between the reinforcement steel and concrete should be considered as totally perfect. In other words, there is no relative displacement between the concrete fibre and the steel.
- The stress-strain diagram for reinforcement steel should be assumed to linear elastic until its yield strength $f_y$. For each strain larger than $\varepsilon_y$, the steel strength is equal to $f_y$. Additionally, in EC2 it is permitted to use a curve with a strain hardening effect (not considered in this paper, because it is neglected in most practical purposes).
- The maximum usable strain $\varepsilon_u$ at extreme concrete compression fibre according to ACI 318-05 shall be assumed to be equal to 0.003, and 0.0035 according to EC2.
- The tensile strength of concrete shall be neglected in axial and flexural calculations for reinforced concrete.

The relationship between concrete compressive stress distribution and concrete strain according to the ACI 318-05 shall be assumed to be rectangular, parabolic, trapezoidal, or any other shape that results in prediction of strength in substantial agreement with results of comprehensive tests.

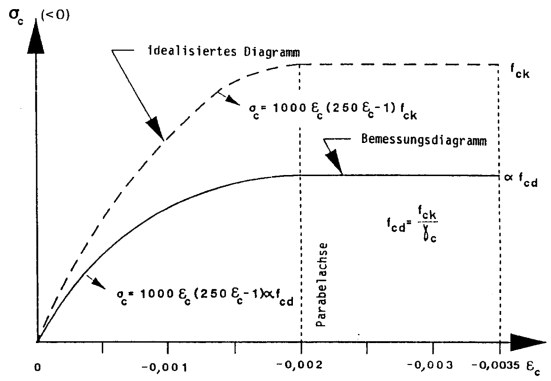



(a) ACI 318-05 Idealized Rectangular Stress Block

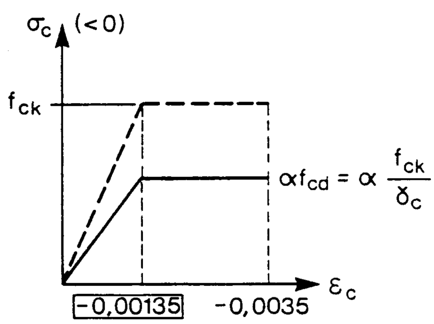



(b) EC2 Idealized Rectangular Stress Block

Fig. 3 Comparison of ACI 318-05 and EC2 idealized rectangular stress distribution




(a) EC2 Parabolic-Rectangular Stress-Strain Diagram




(b) EC2 Bilinear Diagram

Fig. 4 EC2 Parabolic-rectangular and bilinear stress-strain diagrams

Similarly, EC2 describes three possibilities for concrete. The preferred idealization is the parabolic-rectangular diagram, but a bilinear diagram and a rectangular diagram are also permitted. In most cases, the designer will use the simple rectangular stress block. The stress block used in ACI 318-05 is compared with that in Figs. 3(a) and 3(b).

According to EC2, each model can be used for design, but the parabolic rectangular model shown in Fig. 4(a) is the preferred model for rectangular compression zone. In addition, Figure 4b displays the bilinear stress-strain diagram model.

## 5. Comparison of the parameters of the rectangular stress block

From equilibrium of forces at the ultimate limit state $T = C$, the following equations can be obtained with reference to Fig. 3

$$T = C$$
$$A_s f_y = 0.85 a b f_c'$$
$$\Rightarrow a = \frac{A_s f_y}{0.85 b f_c'} \tag{5}$$

where, $T$ is the reinforcement steel tensile force, $C$ is the resultant compression force in the rectangular compressive stress block, $b$ is the width of the cross-section, and $a$ is the depth of the compression zone.

In order to calculate the depth of the neutral axis x measured from the top compression fiber and which is needed to calculate the resulting tensile strain (ductility) in the steel reinforcement, the depth of the compression stress block "$a$" is divided by a factor $\beta_1$ in ACI 318-05 terminology and 0.8 in EC2, respectively. The ACI 318-05 $\beta_1$ factor is a function of the specific concrete compressive strength $f_c'$, and can be obtained using the following conditions:

For $f_c'$ values of 30 MPa or less, $\beta_1$ = 0.85, and for concrete with $f_c'$ > 30 MPa psi

$$\beta_1 = 0.85 - 0.008(f_c' - 30) \geq 0.65 \tag{6}$$

Table 2 provides a listing of the ACI 318-05 $\beta_1$ factor compared with that of EC2 for different values of $f_c'$. It is clear from Table 2 that for $f_c'$ values of 35 MPa and less, the ACI 318-05 factor is less than that of EC2, and smaller for higher values of $f_c'$. It should be noted that the deeper the depth of the neutral axis $x$, results in lower strain (less ductility) in the tensile reinforcement. This means that ACI 318-05 provisions yields higher computed ductility for $f_c'$ of 35 MPa and less, and

Table 2 EC2 and ACI 318-05 $b_1$ factor for different values of concrete compressive strength

| $f_c'$ | ACI $\beta_1$ | EC2 |
|---|---|---|
| 25 | 0.85 | 0.8 |
| 30 | 0.85 | 0.8 |
| 35 | 0.81 | 0.8 |
| 40 | 0.77 | 0.8 |
| 45 | 0.73 | 0.8 |
| 50 | 0.69 | 0.8 |

lower ductility for higher concrete compressive strengths than that of EC2. The following additional equations for both ACI 318-05 and EC2 can derived from Figs. 3(a) and 3(b)

Let,
$$\rho = \frac{A_s}{bd} \tag{7}$$

where, $\rho$ is the reinforcement ratio.

*ACI 318-05*: (refer to Fig. 3(a))

$$\frac{x}{d} = \frac{\rho}{0.85\beta_1}\frac{f_y}{f_c'} \tag{8}$$

and the following expression can be derived from the strain diagram

$$\frac{x}{d} = \frac{0.003}{0.003 + \varepsilon_t} \tag{9}$$

where $\varepsilon_t$ is the strain in the tensile steel reinforcement. Equating Eq. (8) to Eq. (9) yields

$$\rho = \frac{0.00255}{(0.003 + \varepsilon_t)}\frac{f_c'}{f_y} \tag{10}$$

Similarly according to *EC2*: (refer to Fig. 3(b))

$$\frac{x}{d} = 1920\rho\frac{f_y}{f_c'} \tag{11}$$

and

$$\frac{x}{d} = \frac{0.0035}{0.0035 + \varepsilon_t} \tag{12}$$

Equating Eq. (11) to Eq. (12) yields

$$\rho = \frac{0.00182}{(0.0035 + \varepsilon_t)}\frac{f_{ck}}{f_{yk}} \tag{13}$$

Table 3 Comparison of EC2 and ACI-318-05 stress block parameters

| Concrete Grade | Parabolic-rectangular EC2 | | Bilinear EC2 | | Rectangular EC2 | | Rectangular ACI | |
|---|---|---|---|---|---|---|---|---|
| | Average stress (MPa) | Centroid | Average stress (MPa) | Centroid | Average stress (MPa) | Centroid ($a/2x$) | Average stress (MPa) | Centroid ($a/2x$) |
| 20/25 | 9.2 | 0.416 | 9.2 | 0.411 | 9.1 | 0.400 | 14.5 | 0.425 |
| 25/30 | 11.5 | 0.416 | 11.4 | 0.411 | 11.3 | 0.400 | 18.1 | 0.425 |
| 30/37 | 13.8 | 0.416 | 13.7 | 0.411 | 13.6 | 0.400 | 21.7 | 0.425 |
| 35/45 | 16.1 | 0.416 | 16.0 | 0.411 | 15.9 | 0.400 | 24.1 | 0.405 |
| 40/50 | 18.4 | 0.416 | 18.3 | 0.411 | 18.1 | 0.400 | 26.2 | 0.385 |
| 45/55 | 20.6 | 0.416 | 20.6 | 0.411 | 20.4 | 0.400 | 27.9 | 0.365 |
| 50/60 | 22.9 | 0.416 | 22.9 | 0.411 | 22.7 | 0.400 | 29.3 | 0.345 |

Table 3 compares the three permitted EC2 idealizations as well as the ACI 318-05 rectangular stress block idealization in terms of the average stress over a rectangular compression zone and the distance from the compression center to the compression face of the cross-section. In addition, Table 3 provides useful information for design calculations. It is obvious from Table 3 that the results obtained from the three idealizations of EC2 are very close and indistinguishable for all normal purposes. It should be also noticed that the computed ACI 318-05 average stresses are greater than those to EC2. This is due to the fact that EC2 divides the materials' strength $f_{ck}$ and $f_{yk}$ by partial safety factors of 1.5 and 1.15 respectively. The overall effect of the above differences between the two codes on the design bending strength can be best seen by comparing $M/bd^2$ using these various assumptions for given values of steel percentages $\rho$. A detailed comparison between the provisions of both codes for the overall flexural moment computation and ductility will be provided in the subsequent sections.

## 6. Comparison of cross-sectional moment capacity

In order to compare the moment capacity of both codes, a dimensionless set of design equations should be developed and derived using force and moment equilibrium. The following basic equations for both the ACI 318-05 and EC2 are derived for the section's design flexural strength:

*ACI 318-05*

$$\frac{M_u}{bd^2} = \phi\rho f_y\left(1 - \frac{f_y}{1.7f'_c}\right) \tag{14}$$

where,

$\rho$ = reinforcement ratio = Eqs. (7) and (10)
$A_s$ = area of tension reinforcement, mm$^2$
$b$ = width of compression face of member, mm
$d$ = distance from extreme compression fiber to centroid of tension reinforcement, mm

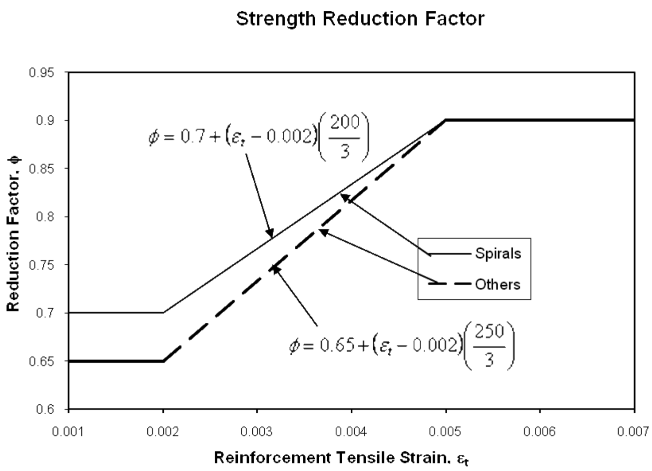



Fig. 5 ACI 318-05 Strength reduction factor ($\phi$) versus the net tensile strain of the reinforcing steel

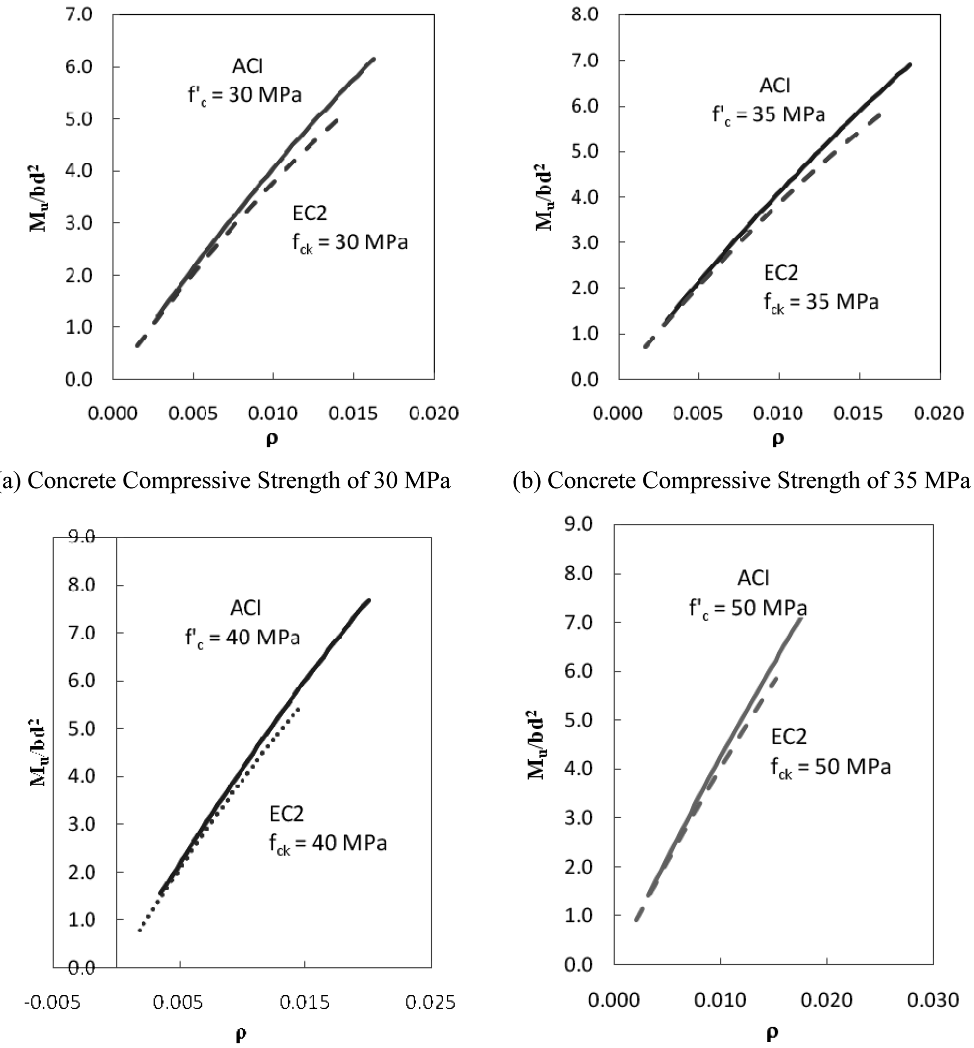
*EC2:*

$$\frac{M_u}{bd^2} = \rho\frac{f_{yk}}{1.15}\left(1-0.768\rho\frac{f_{yk}}{f_{ck}}\right) \tag{15}$$

where,

$\rho$ = reinforcement ratio = Eqs. (7) and (13).

The major difference between the two codes is that EC2 (Eq. (15)) includes material's partial safety factors, while on the ACI 318-05 provisions (Eq. (14)) reduces the nominal moment capacity of the section by an overall strength reduction factor $\phi$. This factor is a function of the net tensile strain in the reinforcing steel. The computation of the $\phi$ factor for different types of reinforcement is shown in




(a) Concrete Compressive Strength of 30 MPa (b) Concrete Compressive Strength of 35 MPa

(c) Concrete Compressive Strength of 40 MPa (d) Concrete Compressive Strength of 50 MPa

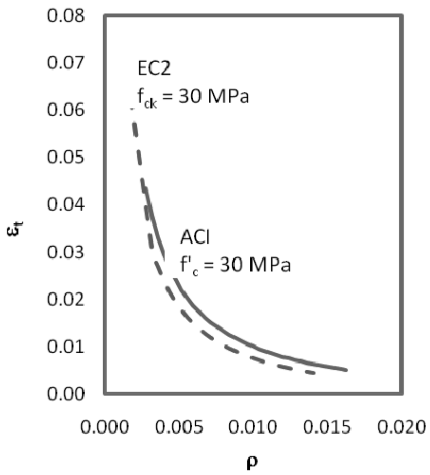
Fig. 6 Comparison of EC2 and ACI 318-05 moment design strength for different values of reinforcement ratio $\rho$ and $f_{ck}$ for $f_{yk}$ = 500 MPa

Fig. 5. The ACI 318-05 defines three strain zones that cover all situations. These zones are:

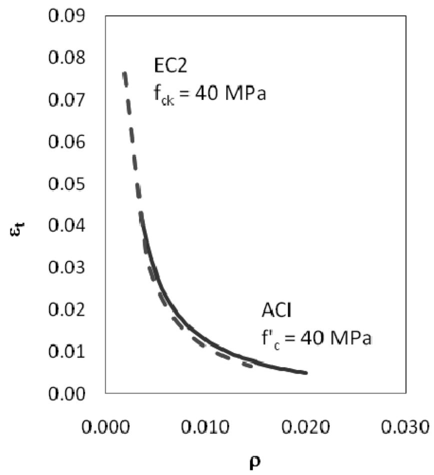
1. Compression controlled (e.g., columns and bearing on concrete): $\varepsilon_t \leq 0.002$
2. Transition (e.g., columns supporting very small axial loads): $0.002 < \varepsilon_t \leq 0.005$
3. Tension controlled (e.g., beams and slabs): $\varepsilon_t > 0.005$

The reduction $\phi$ factor for the tension controlled zone is equal to 0.9. The intent of this paper is to compare flexural provisions of both codes. Thus, the ACI 318-05 tension controlled zone provisions (Eq. (14) with $\phi = 0.9$) is compared with EC2 Eq. (15).

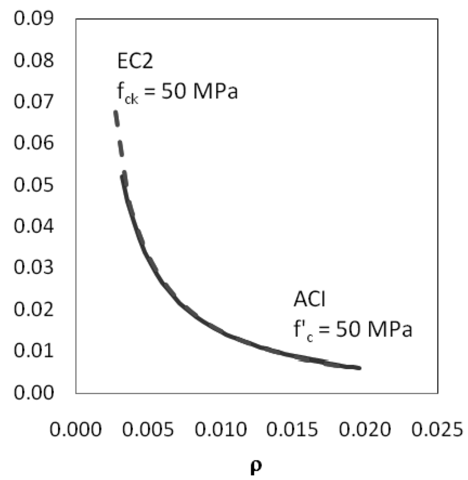
The overall effect of the various stress block parameters differences between the two codes on the flexural design strength of members subjected only to bending can be best seen by comparing the reinforcement tensile strain $\varepsilon_t$, the $x/d$ ratio, and $M_u/bd^2$ (where $M_u$ is the factored applied bending moment) for different given values of reinforcement ratios $\rho$ and $f_c$. This is done in Figs. 6 through 8




(a) Concrete Compressive Strength of 30 MPa



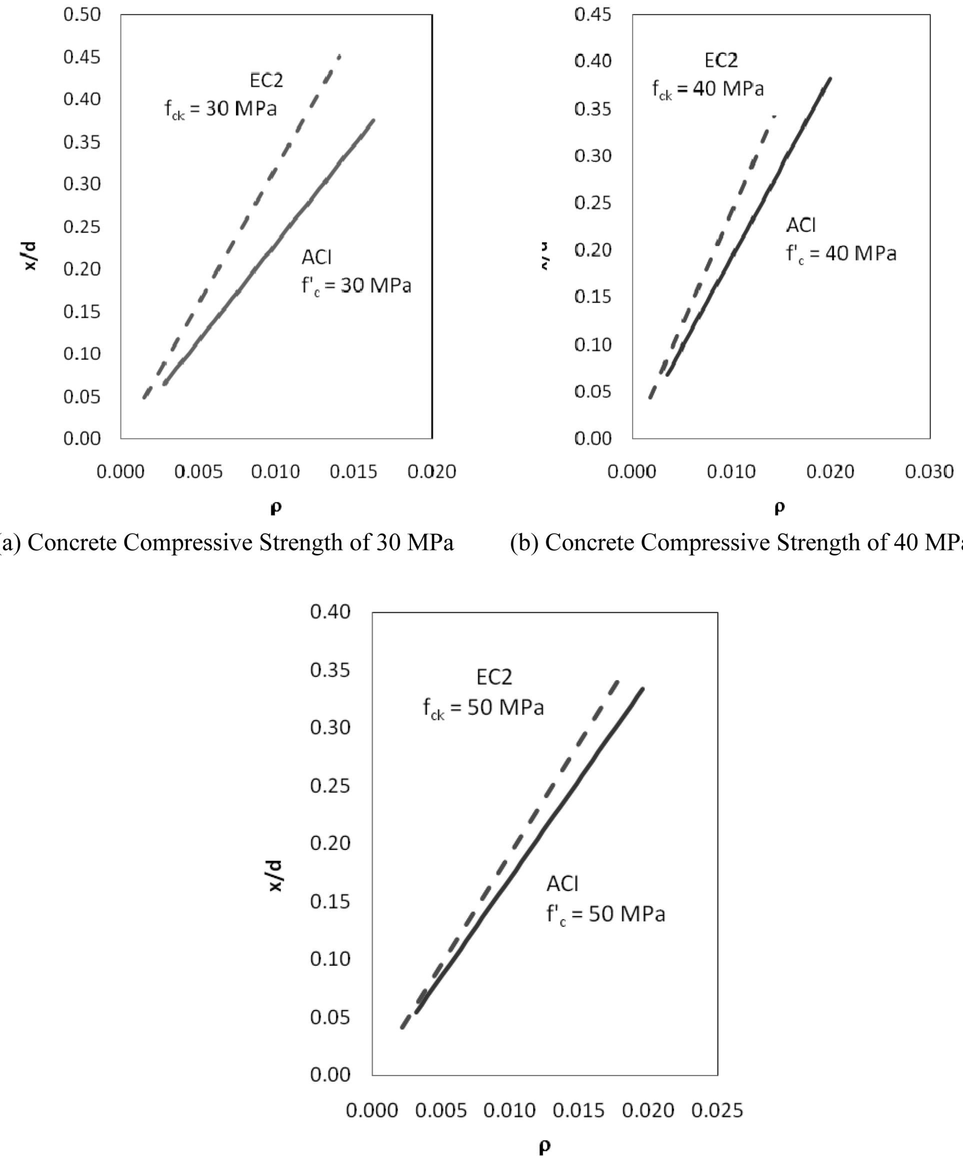

(b) Concrete Compressive Strength of 40 MPa




(c) Concrete Compressive Strength of 50 MPa

Fig. 7 Comparison of EC2 and ACI 318-05 net reinforcement tensile strain $\varepsilon_t$ for different values of reinforcement ratio $\rho$ and $f_{ck}$ for $f_{yk}$ = 500 MPa for a reinforcement yield strength $f_{yk}$ of 500 MPa.

Fig. 6 compares the ACI 318-05 and EC2 calculated sectional moment capacity versus $\rho$ for different values $f_c'$. Fig. 6 shows that as expected, the moment capacity of a section increases by increasing the reinforcement ratio $\rho$. Overall, the trend of the results in Fig. 6 indicate that the ACI 318-05 computed moment capacity is slightly higher than that of EC2 for a specified value of $\rho$ ($\approx$5% at $\rho$ = 0.005 and $\approx$7% at $\rho$ = 0.01%, and $\approx$8% at $\rho$ = 0.015%). Thus, the EC2 provisions are slightly more conservative than those of ACI 318-05. Therefore, there is a very little practical




(a) Concrete Compressive Strength of 30 MPa (b) Concrete Compressive Strength of 40 MPa

(c) Concrete Compressive Strength of 50 MPa

Fig. 8 Comparison of EC2 and ACI 318-05 *x/d* for different values of reinforcement ratio $\rho$ and concrete compressive strength for $f_{yk}$ = 500 MPa difference between EC2 and ACI 318-05 in computing the flexural moment capacity.

Figs. 7 and 8 compare the ACI 318-05 and EC2 provisions for calculating the net tensile strain $\varepsilon_t$ and the *x/d* ratio for given values of $\rho$ and $f_c'$. The most significant difference between the two codes in this regard is the variation of the $\beta_1$ factor depending on $f_c'$ in the ACI 318-05 provisions, which on the other hand is constant and equals to 0.8 in EC2 provisions. Recall that according to ACI 318-05, as $f_c'$ increases the $\beta_1$ factor decreases with a lower limit of 0.65. In addition, it is clear from Fig. 3 that as the *x/d* ratio increases the strain in the reinforcement steel $\varepsilon_t$ decreases resulting in lower ductility.

The conclusion from Figs. 7 and 8 is that the ACI 318-05 computed *x/d* ratio is lower than that of EC2 for all values of $f_c'$ resulting in higher values of $\varepsilon_t$ (i.e., ACI 318-05 is predicting higher ductility than that of EC2 for a specified $\rho$). It can be also noticed that as the reinforcement ratio $\rho$ increases the deviation between the results of the two codes increases. In addition, as the concrete compressive strength $f_c'$ increases, the difference between the two codes in calculating *x/d* and $\varepsilon_t$ are decreasing. For $f_c'$ of 50 MPa the differences between the two codes are indistinguishable.

## 7. Minimum reinforcement ratio

A minimum reinforcement ratio for flexural members is required by both codes for crack opening control and to avoid plain concrete behavior. In ACI 318-05, the minimum reinforcement ratio is the larger of (In SI units)

$$\rho_{\min} = \frac{A_{s,\min}}{b_w d} = \frac{\sqrt{f_c'}}{4 f_y} \tag{16}$$

$$\rho_{\min} = \frac{A_{s,\min}}{b_w d} = \frac{1.4}{f_y} \tag{17}$$

Similarly, The minimum reinforcement ratio is obtained using the following formula

$$\rho_{\min} = \frac{A_{s,\min}}{b_w d} = 0.26\frac{f_{ctm}}{f_{yk}} \tag{18}$$

It is clear that The ACI 318-05 Eq. (16) is a function of $f_c'$ while EC2 Eq. (18) doesn't is a function of the concrete mean tensile strength $f_{ctm}$. The ACI 318-05 and EC2 provisions for the minimum flexural reinforcement ratio are compared in Table 4 for given values of $f_c'$ and the results

Table 4 Comparison of EC2 and ACI-318-05 Minimum and Maximum Reinforcement Ratios

| $f_{ck}$ (MPa) | $f_{ctm}$ (MPa) | $\rho_{\min}$ EC2 | $\rho_{\min}$ ACI | $\rho_{\max}$ EC2 | $\rho_{\max}$ ( $0.75\rho_b$) ACI |
|---|---|---|---|---|---|
| 25 | 2.6 | 0.001352 | 0.0028 | 0.0156 | 0.0148 |
| 30 | 2.9 | 0.001508 | 0.0028 | 0.0188 | 0.0177 |
| 35 | 3.2 | 0.001664 | 0.0030 | 0.0219 | 0.0197 |
| 40 | 3.5 | 0.00182 | 0.0032 | 0.0250 | 0.0214 |
| 45 | 3.8 | 0.001976 | 0.0034 | 0.0281 | 0.0228 |
| 50 | 4.1 | 0.002132 | 0.0035 | 0.0313 | 0.0240 |

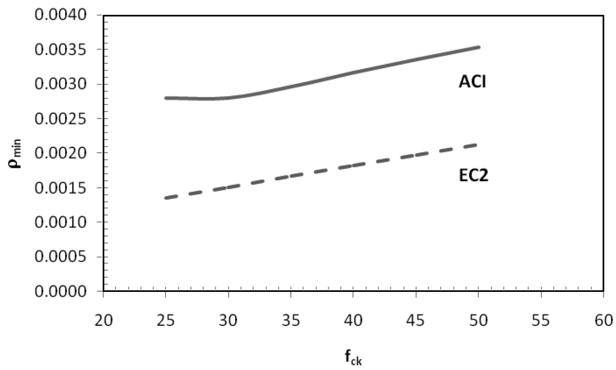



Fig. 9 Comparison of EC2 and ACI 318-05 minimum reinforcement ratio $\rho_{\min}$ versus $f_{ck}$ for $f_{yk}$ = 500 MPa

are displayed in Fig. 9. It is clear from Fig. 9 that the minimum reinforcement ratio required by the ACI 318-05 code is larger than that of EC2 for all values of $f_c'$. The difference increases as $f_c'$ increases.

## 8. Maximum reinforcement ratio (adequate ductility)

The two codes differ in their limitations imposed on the maximum reinforcement ratio (or maximum neutral axis depth) in order to ensure adequate ductility.

For the ACI 318-05, there is no particular limitation for the reinforcement ratio, but the design philosophy of the ACI 318-05 demands a so-called tension-controlled failure. Tension-controlled failure is an ultimate limit state failure, which could be noticed in advance by large deflection of the structure and only occurs if the strain in the reinforcement steel is excessive. Accordingly, the ACI 318-05 provisions limit the reinforcement ratio $\rho_{\max}$ to 0.75 $\rho_b$ with a recommended practical economical reinforcement ratio of $0.5\rho_{\max}$, where, $\rho_b$ is the percentage of steel required for a balanced design at the ultimate load (i.e., the concrete will theoretically fail at a strain of 0.003 and the reinforcement steel will simultaneously yield). Equating Eq. (8) with Eq. (9) with $\varepsilon_t = f_y/E_{st}$ yields the following formula for $\rho_b$

$$\rho_b = \left(\frac{0.85\beta_1 f_c'}{f_y}\right)\left(\frac{600}{600+f_y}\right) \tag{19}$$

The EC2 on the other hand, limited the neutral axis depth ratio ($x/d$) to a maximum permissible value of

$$\frac{x_{\max}}{d} = \delta - 0.4 \tag{20}$$

where,

$x_{\max}$ = maximum permissible neutral axis depth before compression steel is to be provided.

$\delta$ = amount of assumed redistribution. For example $\delta$ = 1.0 means no redistribution and $\delta$ = 0.9 for 10% redistribution.

For the sake of comparison, assume there is no redistribution, thus substitute $\delta$ = 1.0 in Eq. (20), accordingly

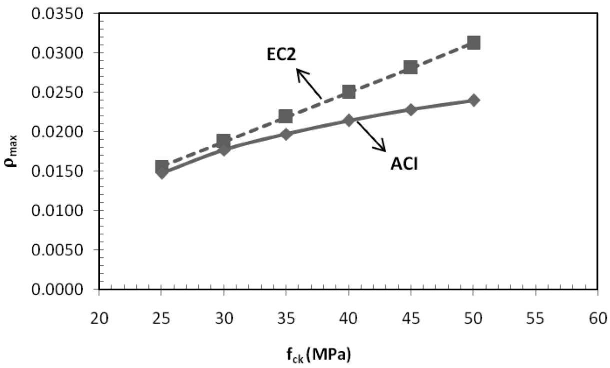



Fig. 10 Comparison of EC2 and ACI 318-05 maximum reinforcement ratio $\rho_{\max}$ versus $f_{ck}$ for $f_{yk}$ = 500 MPa

$$\frac{x_{\min}}{d} = 1 - 0.4 = 0.6 \tag{21}$$

Substituting Eq. (21) in Eq. (11), yields

$$\rho_{\min} = \frac{f_{ck}}{3.2 f_{yk}} \tag{22}$$

The ACI 318-05 and EC2 provisions for the maximum flexural reinforcement ratio are compared in Table 4 for given values of $f_c'$ and the results are displayed in Fig. 10. It is clear from Fig. 10 that $\rho_{\max}$ provided by EC2 is higher than that of ACI 318-05 especially for concrete compressive strengths higher than 35 MPa. This means that the EC2 maximum limitations on the neutral axis depth allow the designer to use a maximum steel reinforcement percentage ratio higher than that of ACI 318-05. Therefore, ACI 318-05 limitations ensures higher ductility (strain) in the steel reinforcement at the ultimate load than that of EC2.

## 9. Comparison of the load safety factors

An overall comparison of the (Load factors/Strength reduction factors) is required for both the ACI 318-05 and EC2. The strength reduction factor $\phi$ for ACI 318-05 is 0.9. On the other hand, EC2 reduces the steel and concrete strengths by partial safety factor of 1.15 and 1.5 respectively. In order to compare the provisions of both codes, an equivalent strength reduction factor $\phi_{eq}$ ($M_u/M_n$) for EC2 should be derived by dividing Eq. (15) by $M_n/bd^2$ (Eq. (23)), where $M_n/bd^2$ is calculated without the inclusion of the partial safety factors 1.15 and 1.5, such that

$$\frac{M_n}{bd^2} = f_{yk}\left(1 - 0.588\rho\frac{f_{yk}}{f_{ck}}\right) \tag{23}$$

and,

$$\phi_{eq} = \frac{\left(1 - 0.768\rho\frac{f_{yk}}{f_{ck}}\right)}{1.15\left(1 - 0.588\rho\frac{f_{yk}}{f_{ck}}\right)} \tag{24}$$

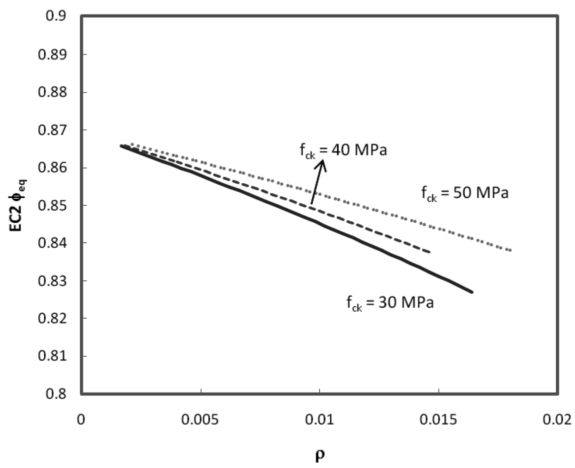



Fig. 11 EC2 Equivalent flexural strength reduction factor $\phi_{eq}$ for different values of reinforcement ratio $\rho$ and $f_{ck}$ for $f_{yk}$ = 500 MPa

The EC2 $\phi_{eq}$ strength reduction factor was calculated for given values of $\rho$ and $f_{ck}$ for $f_{yk}$ = 500 MPa. The results are shown in Fig. 11. The computed $\phi_{eq}$ ranged from 0.865 to 0.827. It can also be noticed from Fig. 11 that $\phi_{eq}$ decreases with increasing the reinforcement ratio and increases with increasing the compressive strength of concrete. It is clear from the parametric analysis that the EC2 $\phi_{eq}$ is less than the ACI 318-05 $\phi$ factor of 0.9 for all the studied cases.

Recall that the service loads in both codes are also magnified by load factors. The load factors for the combination of the dead (D) and live (L) loads in both codes are:

$$\text{Load factor (ACI 318-05)} = 1.2\text{D} + 1.6\text{L} \tag{25}$$

$$\text{Load factor (EC2)} = 1.35\text{D} + 1.5\text{L} \tag{26}$$

An overall factor of safety (F.S.) can be calculated for both codes by dividing the load factors by $\phi$ and $\phi_{eq}$ respectively as follows:

<u>ACI 318-05</u>

$$\text{F.S.} = \left(\frac{\text{Load Factor}}{\text{Strength Reduction Factor}}\right) = \frac{1.2 + 1.6\frac{L}{D}}{0.9\left(1 + \frac{L}{D}\right)} \tag{27}$$

<u>EC2</u>

$$\text{F.S.} = \left(\frac{\text{Load Factor}}{\text{Strength Reduction Factor}}\right) = \frac{1.35 + 1.5\frac{L}{D}}{\phi_{eq}\left(1 + \frac{L}{D}\right)} \tag{28}$$

Eqs. (27) and (28) are compared in Fig. 12 for given values of L/D with EC2 selected $\phi_{eq}$ values of 0.827, 0.85, and 0.865 respectively. It can be concluded from Fig. 12 that EC2 provisions for flexural design strength computations provide a higher factor of safety than those of ACI 318-05 for

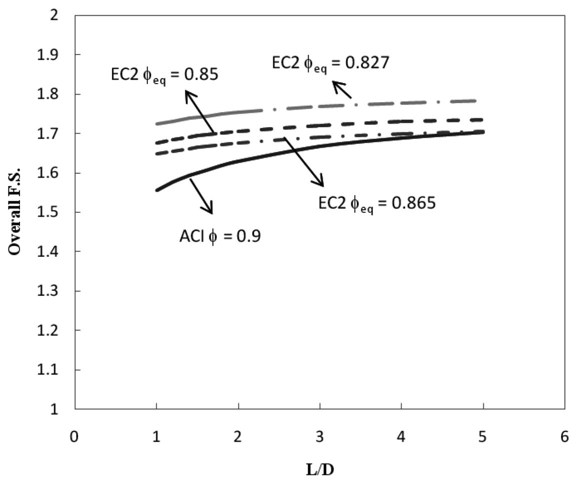



Fig. 12 Comparison of EC2 and ACI-318-05 overall factors of safety for different values of L/D load ratios for $f_{yk}$ = 500 MPa

low L/D ratios. As the ratio of L/D increases the computed safety factor difference between the two codes decreases and becomes almost negligible for $\phi_{eq}$ of 0.865 when the L/D ratio approaches 4. In addition the difference in the calculated safety factor between the two codes decreases for higher values of $\phi_{eq}$ (i.e., with increasing $f_{ck}$).

## 9. Conclusions

ACI 318-05 and EC2 are the two mostly used building design code documents of reinforced concrete structures worldwide. In this paper, a comprehensive detailed comparative study in the flexural provisions of both codes was carried based on non-dimensional parametric analysis. Such comparison will confirm the validity of the flexural provisions of both codes. The major challenge in facilitating such comparison is to develop a technique that can define the parameters of one code in terms of the parameters of the other. In this paper, an analytical procedure was adopted to express the parameters of EC2 into equivalent ACI 318-05 parameters. Accordingly, the provisions of the safety concepts, design assumptions, cross-sectional moment capacity, minimum reinforcement ratios, ductility provisions (maximum reinforcement ratios), and load safety factors of flexural members were compared with extensive parametric analysis. The conclusions based on this comparative study are:

1. EC2 doesn't have implicitly an overall moment strength reduction factor $\phi$ similar to ACI 318-05, which is equal to 0.9 in the tension controlled zone. The safety factors of EC2 are implemented by dividing the materials' (concrete and steel) characteristic strength by partial safety factors of 1.5 and 1.15 respectively resulting in an equivalent reduction factor $\phi_{eq}$ in the range from 0.865 to 0.827.
2. The investigated standards show large differences in their safety concepts and also in the assurance of ductility. But these differences didn't have a significant practical impact on the computation of the moment design strength of flexural members.
3. EC2 provisions are slightly more conservative than those of ACI 318-05 in computing the

flexural design strength of flexural members.

4. ACI 31805 minimum required reinforcement ratio $\rho_{\min}$ is greater that of EC2. The difference increases as $f_c'$ increases.
5. The major difference between the two codes is the limitations imposed on the maximum reinforcement ratio $\rho_{\max}$ to ensure adequate ductility. ACI 318-05 limitations ensures higher ductility in the steel reinforcement at the ultimate limit state than that of EC2.
6. Overall, EC2 provisions for flexural design provide a higher factor of safety than those of ACI 318-05 for low L/D ratios. As the ratio of L/D increases the computed safety factor difference between the two codes decreases and becomes almost negligible for $\phi_{\mathrm{eq}}$ of 0.865 when the L/D ratio approaches 4.
7. In order to make an exact comparison between the two codes further research should be accomplished, in which many other design limit states, must be considered.

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## Notations

$A_s$ : area of tension reinforcement.
$a$ : depth of equivalent rectangular block.
$A_{s,\min}$: minimum amount of flexural reinforcement.
$b$ : width of compression face of member.
$b_w$ : beam web width.
$x$ : distance from extreme compression fiber to neutral axis.
$d$ : distance from extreme compression fiber to centroid of tension reinforcement.
$d_b$ : nominal diameter of bar or wire.
$D$ : dead loads, or related internal moments and forces.
$E_s$ = 200 GPa : modulus of elasticity of reinforcement.
$f_c' = f_{ck}$ specified characteristic compressive strength of concrete (MPa).

$f_y = f_{yk}$ = 500 MPa : specified characteristic yield strength of reinforcement (MPa).
$f_{ctm}$ : concrete mean tensile strength (MPa).
$h$ : overall thickness of member.
$L$ : live loads, or related internal moments and forces.
$M_u$ : factored moment at section.
$Q_{ind}$ : indirect variable action.
$\varepsilon_t$ : net tensile strain in extreme tension steel at nominal strength.
$\rho$ : ratio of tension reinforcement ($A_s/b_w d$).
$\rho_{\min}$ : minimum reinforcement ratio
$\rho_{\max}$ : maximum reinforcement ratio
$\phi$ : ACI 318-05 strength reduction factor.
$\gamma_G$ : partial safety factor for permanent actions.
$\gamma_A$ : partial safety factor for accidental design situations.
$\gamma_m$ : partial safety factor for the material property.
$\gamma_Q$ : partial safety factor for any variable action.