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Comparison of actions and resistances in different building design codes

Mourad M. Bakhoum, Sherif A. Mourad, Maha M. Hassan *

Department of Structural Engineering, Cairo University, Gamaa Street, Giza, Egypt

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A B S T R A C T

Structural design codes of different countries provide engineers with data and procedures for design of the various structural components. Building design codes from USA, Europe, and Egypt are considered. Comparisons of the provisions for actions (loads), and for the resistance (strength) of sections in flexural and compressive axial loading are carried out. Several parameters are considered including variable actions for occupancy and different material strengths. The comparison is made considering both concrete and steel structures. Issues and consequences of mixing actions from one code and resistance from another code are also discussed.

Introduction

Structural design codes of different countries provide engineers with data and procedures for design of the various structural components. Differences, sometimes large ones, could be noticed between the codes in the data given for actions (loads), in the provisions for evaluating resistance of sections, in addition to other code requirements for durability, detailing, etc. This paper presents a quantitative comparison of different design building codes from USA, Europe, and Egypt. The considered codes include ASCE 7-10 [1], ACI 318-14 [2], and AISC-360-10 [3] from USA; EN 1991-1:1996 Eurocode 1 (EC1) [4], EN 1992-2:2001 Eurocode 2 (EC2) [5], EN 1993-1-3:2001 Eurocode 3 (EC3) [6], and EN 1994-1-1:2004 Eurocode 4 (EC4) [7] from European Community; and ECP 201-2011 [8], ECP 203-2007 [9], and ECP 205-2007 [10] from Egypt.

The available literature includes many comparative studies for the provisions included in different design codes. Focus is usually given to evaluating the differences in loads, load factors, resistance values stipulated in design codes from United States, Europe, and Japan. Bakhoum and Shafiek [11] compared concrete building design codes from USA, Britain, and Egypt. Comparison focused on the values of actions (loads) and resistance (strength) of sections in flexural.

Nandi and Guha [12] compared the Indian and European design codes considering the material properties, limits on

* Corresponding author. Tel.: +20 1064149907.
E-mail address: mahamoddather@eng.cu.edu.eg (M.M. Hassan).
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reinforcement area for different elements, and formulas used for calculating ultimate capacity for such elements. El-Shennawy et al. [13] compared the ECP 203-2007 [9] with the equivalent Euro codes through a complete design of a four-storey residential reinforced concrete building. The two designs were evaluated based upon the environmental impact and economical aspects. Hawileh et al. [14] performed a full comparison of the ACI 318 and EC2 design codes considering flexural calculations only. The authors concluded that the EC2 provisions provide a higher safety factor than those for ACI-318. However, the difference is negligible for live/dead load ratios higher than 4. Tabsh [15] focused on comparing the ACI 318 code with the British BS 8110 code regarding the flexural, shear, and axial compressive capacity of members. The study included examining different cross sections while considering different values of live/dead load ratios. The author concluded that the ACI 318 code results in larger cross sections and higher reinforcement ratios. Hassan et al. [16] compared seismic provisions in the Egyptian code for loads (that was under development); Euro Code 8; and Uniform Building Code by focusing on the calculation of lateral forces, member ductility requirements, force reduction factor, and the relevant design accelerations. Bakhoum [17] compared the provisions in American, Japanese, Egyptian, and European codes for highway bridge design. Large differences were highlighted in the traffic action values; however, such differences were considerably reduced when combined with the permanent action values. Bakhoum [18] compared loads used for railway bridge design considering the vertical loads, dynamic factors, longitudinal forces due to traction and braking, and fatigue loads. The comparison included American, Egyptian, and European codes. Bakhoum et al. [19] compared the serviceability limit state requirements in international bridge design codes through analysis of example composite bridges while altering the values of bridge span, bridge width, number of main girders, and the used design code.

This paper focuses on the considered actions (loads) and used design rules for different structural elements including beams and columns while considering steel, concrete, and composite materials. Similarities and differences between the considered design codes are evaluated. The study is meant to provide an insight regarding the applicability of mixing design codes and comparing the safety factors for them. The study also shows the ultimate limit state design for steel elements as a new design philosophy introduced in Egypt in the last few years.

Methodology

Actions and resistances are evaluated and compared for several cases. These include reinforced concrete beams, reinforced concrete columns, steel beams, steel columns, and composite beams. First, the actions and load factors stipulated in different design codes are evaluated. The considered parameters in the study include the following: (i) Permanent actions (D.L.) and variable actions of buildings (L.L.); (ii) Types of building occupancy for variable actions: residential, offices, and shops; and (iii) Action effects: flexural and axial forces. Afterward, the resistances of several structural elements are evaluated for beams and axially loaded short columns. The material properties are fixed throughout the study as follows: Reinforcement yield strength $f_{yk} = 360$ and $500\ \text{N/mm}^2$, structural steel yield strength, $f_y = 240\ \text{N/mm}^2$, and concrete cylinder strength $f_{ck} = 25$ and $40\ \text{N/mm}^2$.

Results and discussion

Actions in the considered codes

Table 1 presents some values of variable actions (L.L.) specified for different types of building occupancy. Comparing the values provided by different codes, differences in values in variable actions can be observed. Large differences in live load intensities are noticed for balconies and corridors in residential buildings; and stair loads in shops. In some cases, the observed differences reached 60% increase in the design live load intensity.

Values of variable actions (L.L.) are combined with permanent actions (D.L.), and then each is multiplied by relevant load factor for ultimate limit state as illustrated in Table 2. The following assumptions are made for evaluating items in

Table 1 Values of variable action intensities for different types of building's occupancy in different studied codes.

Use	Code	Floors (kN/m^2)	Corridors (kN/m^2)	Stairs (kN/m^2)	Balconies (kN/m^2)
Residential	ASCE 7-10 [1]	1.92	4.79	4.79	2.88
	ACI 318-14 [2]	1.90	4.80	4.80	4.80
	EC2 [5]	2.00	2.00	3.00	4.00
	ECP 201-2011 [8]	2.00	2.00*	3.00	3.00
Offices	ASCE 7-10 [1]	2.40	3.83	4.79	3.60
	ACI 318-14 [2]	2.40	4.80	4.80	4.80
	EC2 [5]	3.00	3.00	3.00	3.00
	ECP 201-2011 [8]	2.50	2.50*	4.00	4.00
Shops	ASCE 7-10 [1]	6.00**	6.00**	4.79	–
	ACI 318-14 [2]	6.00**	6.00**	4.80	–
	EC2 [5]	5.00	5.00	5.00	–
	ECP 201-2011 [8]	5.00***	5.00***	5.00***	–

* This value is assumed to be same as that of floors.
** This value is assumed for light manufacturing.
*** The variable action intensity for warehouses and stores is given by $\geqslant 10\ \text{kN/m}^2$ (according to the stored materials).

Table 2 Comparison of ultimate loads and partial safety factors for different types of building occupancy.

Use	Dead load (kN/m^2)	ACI318-14/EC2	ECP203-2007/EC2	ECP205-2007/EC2	EC2 ultimate value (kN/m^2)
Residential (Floors)	L.L. Ratio	0.95	1.00	1.00	**2.00**
	3.00	0.94	1.05	0.96	7.05
	4.00	0.93	1.05	0.95	8.40
	7.00	0.92	1.04	0.93	12.45
Residential (Stairs)	L.L. Ratio	1.60	1.00	1.00	**4.00**
	3.00	1.32	1.05	0.98	9.00
	4.00	1.26	1.05	0.97	10.40
	7.00	1.15	1.05	0.95	14.60
Residential (Balconies)	L.L. Ratio	1.20	0.75	0.75	**4.00**
	3.00	1.12	0.90	0.84	9.00
	4.00	1.09	0.91	0.84	10.40
	7.00	1.04	0.94	0.85	14.60
Offices (Floors)	L.L. Ratio	0.80	0.83	0.83	**3.00**
	3.00	0.87	0.96	0.89	8.20
	4.00	0.87	0.97	0.89	9.60
	7.00	0.88	0.99	0.89	13.80

Case	Loads considered	Dead loads (D)		Live loads (L)		Wind loads (W)
		Adverse	Beneficial	Adverse	Beneficial	
ACI 318-14 [2]						
1	D, L	1.2	0.9	1.6	0.0	–
2	D, L, W	1.2	–	1.6	–	0.5
		1.2	–	1.0	–	1.0
3	D, W	–	0.9	–	–	1.0
For simplicity, L refers to floor live load and the roof live load case is neglected						
Case	**Loads considered**	**Permanent loads (G_k)**		**Variable imposed loads (Q_k)**		**Wind loads (W_k)**
		Adverse	Beneficial	Adverse	Beneficial	
EC2 [5]						
1	G_k, Q_k	1.35	1.00	1.50	0.00	–
2	G_k, Q_k, W_k	1.35	1.00	1.35	0.00	1.35
3	G_k, W_k	1.35	1.00	–	–	1.50
Simplified combination rules with only one variable action are considered.						
Case	**Loads Considered**	**Dead Loads (D)**		**Live Loads (L)**		**Wind Loads (W)**
		Adverse	Beneficial	Adverse	Beneficial	
ECP 203-2007 [9]						
1	D, L*	1.4	0.9	1.6	0.0	–
2	D, L, W	0.8 × 1.4	0.8 × 1.4	0.8 × 1.6	0.8 × 1.6	0.8 × 1.6
3	D, W	1.4	0.9	–	–	1.3
Case	**Loads Considered**	**Dead Loads (D)**		**Live Loads (L)**		**Wind Loads (W)**
		Adverse	Beneficial	Adverse	Beneficial	
ECP 205-2007 [10]						
1	D, L	1.2	0.9	1.6	0.0	–
2	D, L, W	1.2	–	1.6	–	0.8
		1.2	–	0.5	–	1.3
3	D, W	–	0.9	–	–	1.3
For simplicity, L refers to floor live load and the roof live load case is neglected						

Notes: Values written in bold font represent the Variable Action Intensity according to EC2 [5] and as indicated in Table 1.
* For cases when live loads does not exceed 0.75 of the dead loads, the ultimate load (U) is calculated as follows: U = 1.5 (D + L).

Table 2: (i) D.L. and L.L. are applied to the same area, (ii) The lower value of D.L. intensities (3 kN/m^2) corresponds to D.L. in thin slab or void slab construction plus the flooring weight, and the higher value (7 kN/m^2) corresponds to dead loads in thick slab constructions plus the flooring weight. Accordingly, the ultimate limit state values are determined and evaluated with respect to ultimate load of EC2 [5]. The last column gives the values of ultimate loads for the EC2 [5] in kN/m^2. The considered ultimate loading combinations as per different standards are as follows:

$$\text{ASCE 7-10 [1]}: \quad 1.2\, \textit{Dead Load} + 1.6\, \textit{Live Load} \tag{1}$$

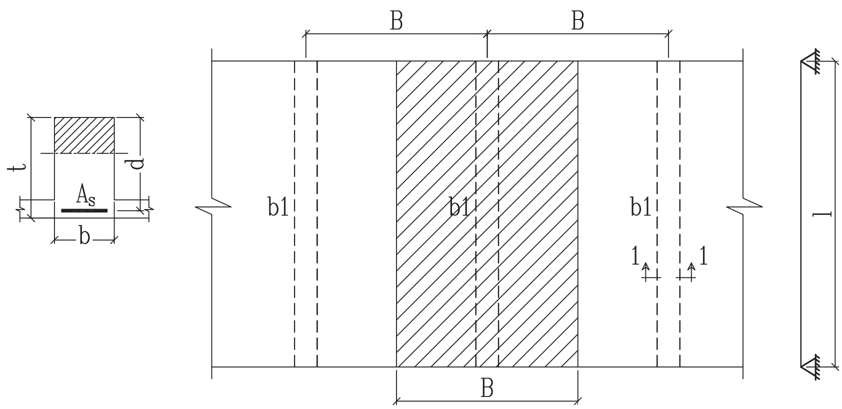
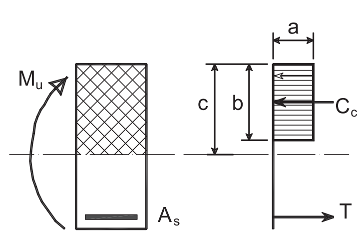


Fig. 1 Considered beam used for comparison between different codes.

Table 3 Summary of results for the studied structural elements.

Design code	Dead load (kN/m)	Ultimate load (kN/m)	M_u (kN m)	Ultimate resistance (kN m)
A. studied concrete beam				
ACI 318-14 [2]	21.4	33.89	128.14	160.74
EC2 [5]	21.4	36.99	139.87	149.04
ECP 203-2007 [9]	21.4	38.60	145.96	148.74
B. studied steel beam				
AISC-360-10 [3]	18.94	30.94	116.99	135.65
EC3 [6]	18.94	33.67	127.32	137.02
ECP 205-2007 [10]	18.94	35.16	132.95	128.11
C. studied steel–concrete beam				
AISC-360-10 [3]	18.94	34.73	247.51	315.4
EC4 [7]	18.94	33.67	269.38	350.5
ECP 205-2007 [10]	18.94	35.16	281.2	281.2



	ACI 318-14 [2]	**EC2 [5]**	**ECP 203 – 2007 [9]**
a	$0.85f_{ck}$	$0.85f_{ck}/1.5$ for $f_{ck} \leq 50$MPa $[1- \frac{f_{ck}-50}{200}]x0.85f_{ck}/1.5$ for $50 \leq f_{ck} \leq 90$MPa	$0.56\ f_{ck}$
b	0.85c for $17 \leq f_{ck} \leq 28$MPa $[0.85 - \frac{0.05(f_{ck}-28)}{7}]$c for $28 \leq f_{ck} \leq 55$MPa 0.65c for $f_{ck} \geq 55$MPa	0.8c for $f_{ck} \leq 50$MPa $[0.8- \frac{f_{ck}-50}{400}]$c for $50 \leq f_{ck} \leq 90$MPa	0.85c

Fig. 2a Concrete stress block parameters for different codes.

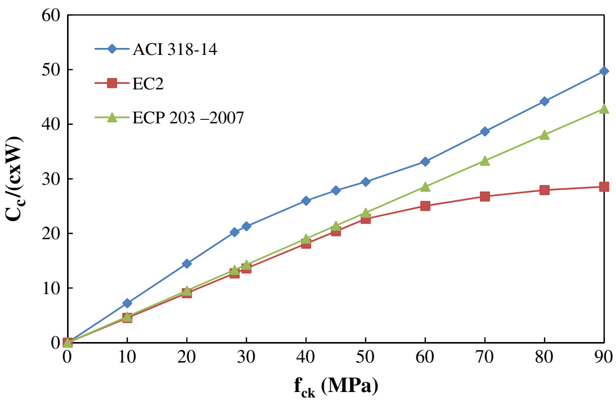


Fig. 2b Force carried by concrete part of the rectangular section.

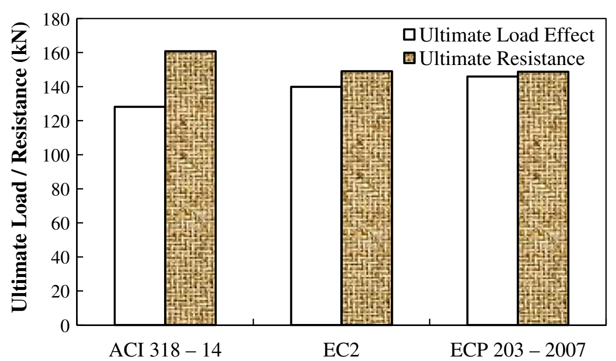


Fig. 3a Ultimate action effect and ultimate section resistance for the studied codes.

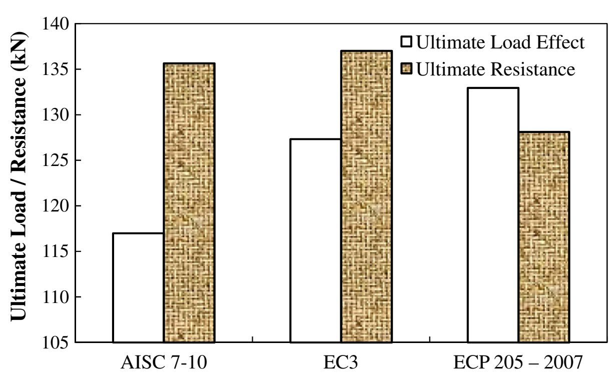


Fig. 3b Ultimate action effect and ultimate section resistance for the studied codes – IPE(300).

$$\text{ACI 318-14 [2]}: \quad 1.2\ \textit{Dead Load} + 1.6\ \textit{Live Load} \tag{2}$$

$$\text{EC2 [5]}: \quad 1.35\ \textit{Permanent Load} + 1.5\ \textit{Variable Load} \tag{3}$$

$$\text{ECP 203-2007 [9]}: \quad 1.2\ \textit{Dead Load} + 1.6\ \textit{Live Load} \tag{4}$$

$$\text{ECP 205-2007 [10]}: \quad 1.2\ \textit{Dead Load} + 1.6\ \textit{Live Load} \tag{5}$$

The following general observations could be made concerning the considered cases:

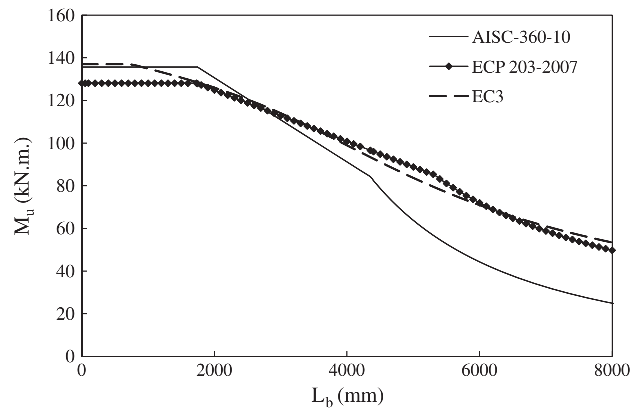


Fig. 3c Ultimate flexural resistance of steel compact section for unrestrained steel beam.

(i) ACI 318-14 [2] gives larger values of ultimate loads for floors, stairs, and balconies of residential buildings, and lower values for office floors in comparison with EC2 [5].
(ii) ECP 203-2007 [9] gives larger values of ultimate loads for floors and stairs of residential buildings, and lower values for balconies of residential buildings and for floors of office buildings in comparison with EC2 [5].
(iii) ECP 205-2007 [10] generally yields lower values, compared to EC2 [5], of ultimate loads for the different studied cases due to the decreased load factor considered for dead load case. This is true for the weight of steel components due to the improved quality control associated with the steel sections manufacturing. However, using the same reduced ultimate load factor for the concrete slab and finishes is questionable.
(iv) The differences between ultimate loads in the three codes decrease, in general, with the increase in the value of D. L.

Table 2 lists the ultimate load factors for the studied codes considering dead load, live load, and wind load cases. The following observations can be summarized:

(i) ACI 318-14 [2] specifies lower values for the ultimate dead load factor compared to the EC2 [5] code; however, higher ultimate live load factor is considered.
(ii) ECP 203-2007 [9] specifies the highest ultimate dead load factors compared to the other codes. Meanwhile, the ultimate live load factors are similar to ACI 318-14 [2]. It is also observed that both dead and live load factors are reduced by 20% when the wind load case is considered.

Resistance in the considered codes

Comparison of the considered codes should include both action and resistance. As illustrated in Table 2, the examined codes provide different ultimate loading actions. This will lead to varying design straining actions on the structural elements. Hence, determining whether a code is more conservative or more liberal has to involve considering both sides of the design

Table 4 Comparison of ultimate moment of resistance and combined effect of action and resistance of singly reinforced concrete sections.

f_{ck} (N/mm^2)	f_{yk} (N/mm^2)	ρ (%)	$M_{\text{ult-Code}}/M_{u\text{-EC2}}$		EC2 value
			$\frac{\text{ACI318-14}}{\text{EC2}}$	$\frac{\text{ECP203-2007}}{\text{EC2}}$	M_u/bd^2 (N mm)
25	360	0.5	1.05	1.00	1.48
		1.0	1.06	1.00	2.78
		1.5	1.08	1.00	3.92
		2.0	1.10	1.00	4.87
25	500	0.5	1.05	1.00	2.01
		1.0	1.08	1.00	3.68
		1.5	1.11	1.00	5.02
		2.0	1.14	1.00	6.02
40	360	0.5	1.04	1.00	1.51
		1.0	1.05	1.00	2.91
		1.5	1.06	1.00	4.21
		2.0	1.07	1.00	5.40
40	500	0.5	1.05	1.00	2.07
		1.0	1.06	1.00	3.93
		1.5	1.08	1.00	5.58
		2.0	1.09	1.00	7.03
Use	Materials (N/mm^2)	Permanent load (kN/m^2)	ρ (%)	$\frac{\text{ACI318-14}}{\text{EC2}}$	$\frac{\text{ECP203-2007}}{\text{EC2}}$
Combined effect of ultimate action and ultimate moment of resistance					
Residential (Floors)	f_{ck} = 25	3	0.5	0.90	1.05
	f_{yk} = 360	3	1.5	0.87	1.05
		7	0.5	0.88	1.04
		7	1.5	0.85	1.05
	f_{ck} = 25	3	0.5	0.89	1.05
	f_{yk} = 500	3	1.5	0.85	1.05
		7	0.5	0.87	1.04
		7	1.5	0.83	1.05
Offices (Floors)	f_{ck} = 25	3	0.5	0.83	0.96
	f_{yk} = 360	3	1.5	0.80	0.96
		7	0.5	0.84	0.99
		7	1.5	0.81	0.99
	f_{ck} = 25	3	0.5	0.83	1.15
	f_{yk} = 500	3	1.5	0.79	1.15
		7	0.5	0.83	0.99
		7	1.5	0.79	0.99

equation: actions and resistance of sections. In the following sections, formulas for calculating resistance of concrete and steel sections subjected to various types of straining actions are exhibited. Consequently, comparison between different studied codes is performed considering both the resistance of sections and effect of actions.

Resistance of reinforced concrete sections in flexure

Consider a beam in a typical one-way slab construction within a residential building, e.g. beam b_l shown in Fig. 1. The assumed structural system is a simply supported inverted beam. The distance between the successive beams (B) and the span (l) of the beam are 2.7 m and 5.5 m, respectively. The width and thickness of the beam are equal to 200 mm and 500 mm, respectively. The characteristic compressive strength of concrete cylinder (f_{ck}) is 25 N/mm^2 and the yield strength of longitudinal reinforcement (f_{yk}) is 500 N/mm^2. It should be mentioned that for the characteristic concrete cylinder strength, and also steel yield strength, the used values may not correspond to the specific grades of the codes considered. However, since the interest of the current study is to compare ultimate moments of resistance according to the provisions of different codes, the same material strength should be used.

The intensity of permanent action is considered equal to 7 kN/m^2. Hence, the uniform acting load on the beam due to permanent loads is 21.4 kN/m. Third and fourth columns in Table 3A summarize the ultimate loading acting on the beam calculated as per each of the considered codes and the ultimate bending moment considering the simply supported statistical system.

Different codes adopt the equivalent stress block instead of the curved stress block of concrete along with the equations of equilibrium of the section to determine the ultimate resisting moment of beams. Fig. 2a shows the stress distribution of a reinforced concrete section. The figure exhibits the assumptions made by the studied codes regarding the average intensity

Table 5 Comparison of ultimate strength of axially loaded short columns.

f_{ck} (N/mm^2)	f_{yk} (N/mm^2)	ρ (%)	$\frac{ACI318\text{-}14}{EC2}$	$\frac{ECP203\text{-}2007}{EC2}$	EC2 value bd (N/mm^2)
25	500	1.0	0.73	0.77	18.60
25	500	3.0	0.69	0.77	27.30
40	500	1.0	0.74	0.77	27.15
40	500	3.0	0.71	0.77	35.85

– The values shown in the last column should be multiplied by the cross-sectional dimensions (mm) to obtain the ultimate strength of column (N).
– Fourth, fifth, and sixth columns give relative values with respect to EC2 [5].

Table 6 Comparison of the dimensional requirements.

Design code	Minimum concrete slab thickness (mm)	Effective width of slab (mm)
AISC-360-10 [3]	50	Effective width is the sum of the effective width for the two sides of the beam as the minimum of the following: • L/8, where L is the span of the beam • Half the distance to center line of the adjacent beam • The distance to edge of slab
EC4 [7]	80	Effective width is the sum of the effective width for the two sides of the beam in addition to the distance between the outstand shear connectors. The effective width of the concrete flange is taken as the minimum of the following: • L/8, where L is the span of the beam • Half the distance to center line of the adjacent beamThe distance to edge of slab.
ECP 205-2007 [10]	For roof slabs 80 For repeated floors 100 For floors supporting moving loads 120	Effective width is the sum of the effective width for the two sides of the beam as the minimum of the following: • L/8, where L is the span of the beam • Half the distance to center line of the adjacent beam • The distance to edge of slab

(a), depth (b) of the stress block, and location of the neutral axis (c). ECP 203-2007 [9] is the only code using the same equations for the stress block parameters regardless of the value of the compressive concrete strength. Fig. 2b plots the compressive force carried by the concrete portion (C_c) on the section divided by section width (W) and depth of the compression zone (c) versus the compressive strength of concrete cylinder (f_{ck}) for the three studied codes. EC2 [5] and ECP 203-2007 [9] provide comparable results till compressive concrete strength equal to 50 MPa. Afterward, the values estimated by the Egyptian code are larger than the values estimated by EC2 [5]. It is also observed that ACI 318-14 [2] yields the highest results.

The three codes yield comparable results for $f_{ck} < 40$ MPa; however, the difference increases as the compressive strength of concrete cylinder increases. It is also observed that EC2 [5] yields the highest results.

If this beam is designed, for example according to the ACI 318-14 code [2], it is required that at failure (assuming b_1 is a singly reinforced beam) [11]:

$$(1.2w_D + 1.6w_L)\frac{l^2}{8} \leqslant \phi\,\rho\,f_{yk}\left(1 - 0.59\frac{\rho\,f_{yk}}{f_{ck}}\right) b\,d^2 \tag{6}$$

$$\rho = A_s/(b\,d) \tag{7}$$

$$(1.2w_D + 1.6w_L)\,C_1 \leqslant \mu\,b\,d^2 \tag{8}$$

$$\mu = \phi\,\rho\,f_{yk}\left(1 - 0.59\frac{\rho\,f_{yk}}{f_{ck}}\right) \tag{9}$$

$$b\,d^2 \geqslant \frac{(1.2w_D + 1.6w_L)}{\mu}\,C_1 \tag{10}$$

where w_D and w_L are the dead and live uniform loads acting on the studied beam. ϕ is a reduction factor. ρ is the ratio of the longitudinal reinforcement within the studied section.

In Eq. (9), C_1 is a function of the structural system. C_1 does not, in most cases, differ from one code to the other. Numerator of the right hand side of Eq. (9) is a function of the dead load, live load, and the load factors given in different codes. The dead load includes the weight of structural and non-structural elements. Denominator of Eq. (9) is a function of the material properties (f_{ck} and f_{yk}) in addition to the resistance model given by design code including the following: stress–strain relations, limit strain, stress block shape, and partial safety factors for materials. Equations similar to Eq. (9) could be written for different codes and considering different load effects.

Using the right hand side of Eq. (5), the ultimate moment of resistance according to the ACI 318-14 [2] provisions is evaluated. Similar formulas are used to calculate the ultimate moment of resistance for different considered codes as per the following:

$$\text{ACI 318-14 [2]}: \quad 0.90\,\rho f_{yk}\left(1-0.59\frac{\rho f_{yk}}{f_{ck}}\right) b\, d^2 \tag{11}$$

$$\text{EC2 [5]}: \quad 0.87\,\rho f_{yk}\left(1-0.78\frac{\rho f_{yk}}{f_{ck}}\right) b\, d^2 \tag{12}$$

$$\text{ECP 203-2007 [9]}: \quad 0.87\,\rho f_{yk}\left(1-0.78\frac{\rho f_{yk}}{f_{ck}}\right) b\, d^2 \tag{13}$$

The above formulas are used to calculate the ultimate moment of resistance considering reinforcement ratio (ρ) equal to 1%. The results are summarized in the fifth column of Table 3A. Fig. 3a exhibits the ultimate load effect and ultimate resistance for the different considered codes. It can be observed that for the same loading effects and beam dimensions, ACI 318-14 [2] and EC2 [5] codes yield conservative results compared to the ones given by ECP 203-2007 [9] code.

For the sake of comparison between different codes, the ratio of the moment of resistance as a function of bd^2 is evaluated for the different studied codes. Then, the ratio of the moment of resistance for a specific code to the moment of resistance for EC2 [5] code is evaluated. If this ratio is larger than 1, then the considered code is more conservative (or less economic) than EC2 [5] code, and vice versa. Repeating the above process for several cases could give an idea on the economy of concrete structures as designed according to different codes. Examples of such comparison are given in Table 4. It is worth mentioning that these values are derived for under reinforced sections ($\rho < \rho_{\text{balanced}}$). The following observations could be summarized:

(i) The ultimate moments of resistance are observed to be 5–14% higher for ACI 318-14 [2] than for the EC2 [5] and ECP 207-2007 [10]. This difference increases slightly with the increase of (ρ).
(ii) The values of ultimate moment of resistance of singly under reinforced concrete sections, M_u, are the same for EC2 [5] and ECP 203-2007 [9]. This is attributed for the fact that, for the cases considered, the two codes use the same equivalent concrete block and the same material partial safety factors.

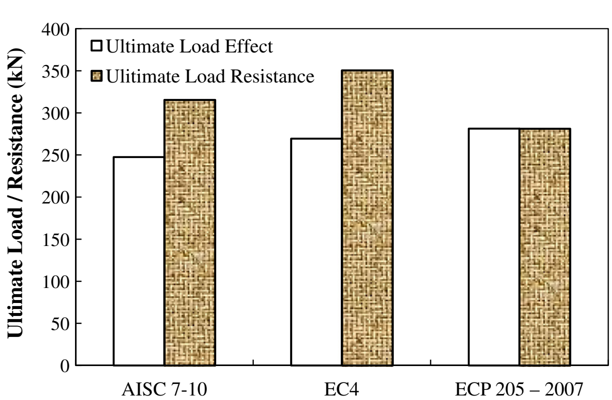


Fig. 4 Ultimate action effect and ultimate section resistance for the studied codes – built-up section.

Table 4 shows comparison of the combined effect of the actions and the ultimate resistance of the studied singly reinforced concrete beam. The varied parameters include the value of the permanent loading, type of usage of the area, yield strength of the reinforcing bars, and the reinforcement ratio. The last three columns exhibit the relative ratio of bd^2 of the studied code with respect to EC2 [5]. The main observations can be summarized as follows:

(i) ACI 318-14 [2] generally requires smaller sections than EC2 [5]. This means that it is less conservative or more economic by 2–10% depending upon the reinforcement ratio and the resistance of steel.
(ii) ECP 203-2007 [9] requires sections that are larger than the ones given by EC2 [5] by about 5% for residential occupancy. For offices, the ratio is smaller by 1–4% considering f_{yk} = 360 MPa; and larger by 15% considering f_{yk} = 500 MPa.

Resistance of steel compact I-sections in flexure

The same structural system illustrated in Fig. 1 is resolved considering steel beams instead of concrete beams. In addition, concrete slab of thickness 80 mm is considered to be poured on steel decking.

The design of the steel beams is performed considering St. 37-2, which is equivalent to A36, and has a yield strength (f_y) and ultimate strength (f_u) equal to 240 N/mm^2 and 360 N/mm^2, respectively. The design formulas shown in Eqs. (13)(15) are performed considering compact I-section beam with section plastic modulus, Z_x.

$$\text{AISC-360-10 [3]}: \quad (1.4w_D + 1.6w_L)\,\frac{l^2}{8} \leqslant 0.9\,f_y\,Z_x \tag{14}$$

$$\text{EC3 [6]}: \quad (1.35w_D + 1.5w_L)\,\frac{l^2}{8} \leqslant f_y/1.1\,Z_x \tag{15}$$

$$\text{ECP 205-2007 [9]}: \quad (1.4w_D + 1.6w_L)\,\frac{l^2}{8} \leqslant 0.85\,f_y\,Z_x \tag{16}$$

The comparison is illustrated in Table 3B considering using IPE 300 steel section for the beam. Fig. 3b exhibits the ultimate load effect and ultimate resistance of the studied beam for the different considered codes.

The following can be observed for the same loading effects:

(i) The ultimate moments of resistance are observed to be 1% lower for AISC-360-2010 [3] than for EC3 [6] and about 6% higher for AISC-360-2010 [3] than for ECP 205-2007 [10].
(ii) ECP 205-2007 [10] evaluates the used section as unsafe and requires the use of a larger section.

Fig. 3c exhibits a general comparison of the ultimate moment of resistance of the steel section IPE 300 related to the unbraced length of the beam. It can be observed that the flexural moment of resistance as per the Euro design code is relatively higher than the American and Egyptian design codes in the first part. However, as the unbraced length increases the flexural moment of resistance of the European and Egyptian codes is compared to each other. Comparing AISC-360-10 [3] and EC3 [6], the percentage of change in the flexural

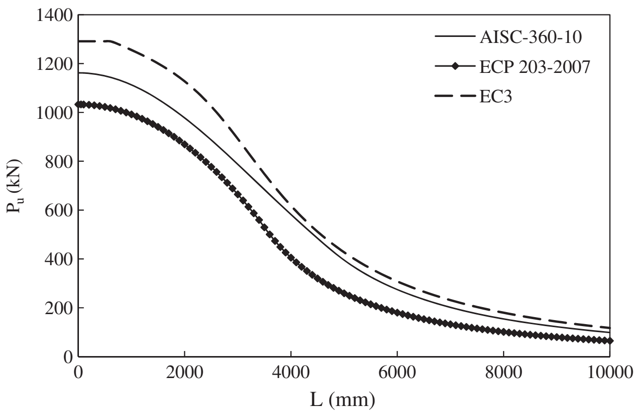


Fig. 5 Ultimate axial compressive resistance of steel compact section.

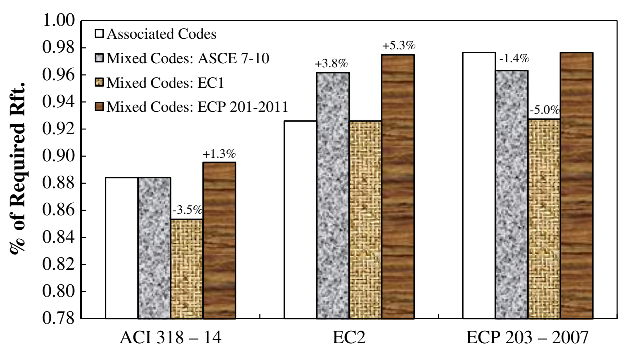


Fig. 6a Comparison of the reinforcement ratio [%] for a singly reinforced beam section in flexure calculated by associated codes and mixed codes.

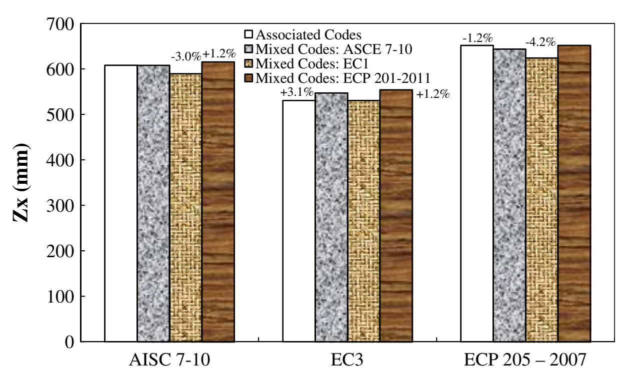


Fig. 6b Comparison of the required steel section plastic modulus for a steel beam by associated codes and mixed codes.

moment of resistance ranges between a decrease of 6% and an increase of 60%. Meanwhile, values calculated considering ECP 205-2007 [9] and EC3 [6] are compared to each other; however, EC3 [6] yields higher results at small values of the unbraced length (L_b) compared to the Egyptian code by about 7%. For unbraced lengths between 4000 and 6000 mm, the values calculated by ECP 205-2007 [9] are higher than the ones calculated by EC3 [6] by a percentage reaching 8%. The Egyptian code provides moment resistance values higher than the American standard except for beams with small values of unbraced lengths.

Comparison between the studied codes considering both actions and resistances is not straightforward for steel beams as it depends upon the unbraced length as seen from Fig. 3c. Different codes specify different regions for calculating the resistance with different limits. Hence, when combining the actions and resistances in comparison, it is expected to have different ratios depending upon the unbraced length value. The following observations can be listed:

(i) AISC-360-10 [3] requires larger sections compared to EC3 [6]. However, the ratio depends upon the type of occupancy. Also, a small increase in the required section is observed as the Dead to live load ratio increases.
(ii) Comparing ECP 205-2007 [10] and EC3 [6], the same ratio yielded for residential floors as both codes use the same variable action ratio. However, for office floors, the ratio is dependent upon the dead to live load ratio. It can also be observed that the Egyptian standard is more conservative than the European.

Resistance of steel composite sections in flexure

Composite steel beams are used to support the structural system as shown in Fig. 1. The span of the beam (l) is considered equal to 8 m. Thickness of concrete slab is considered equal to 100 mm poured on steel decking. The characteristic compressive strength of concrete cylinder (f_{ck}) is 25 N/mm^2 and the yield strength of longitudinal reinforcement (f_{yk}) is 500 N/mm^2. St. 37-2 is used for the steel elements including the steel beam and the shear studs. A suitable built-up section is chosen according to the guidelines provided by different codes. Compact web section is chosen such that the plastic design method enlisted in AISC-360-10 [3], ECP 205-2007 [10], and EC4 [7] is used.

Table 6 summarizes the requirements for the three studied codes regarding the minimum slab thickness and the effective slab width. The provisions of the effective width for AISC-360-10 [3] and ECP 205-2007 [10] are identical. Meanwhile, EC4 [7] adds the distance between the outstand shear connectors yielding a larger effective width for the same section.

The design formulas shown in Eqs. (17)(19) are performed considering the same built-up section. The plastic design moments are calculated considering the effective part of the concrete slab and the used steel section. The main differences lie in the value of the effective width and the reduction factor.

$$\text{AISC-360-10 [3]}: \quad (1.4w_D + 1.6w_L)\,\frac{l^2}{8} \leqslant 0.9\,M_p \tag{17}$$

$$\text{EC4 [7]}: \quad (1.35w_D + 1.5w_L)\,\frac{l^2}{8} \leqslant M_p \tag{18}$$

$$\text{ECP 205-2007 [10]}: \quad (1.4w_D + 1.6w_L)\,\frac{l^2}{8} \leqslant 0.8\,M_p \tag{19}$$

Table 3C shows a comparison for the ultimate and resisting moments considering the different studied codes. Fig. 4 exhibits such values.

The following can be observed for the same loading effects:

(i) The ultimate moments of resistance are observed to be 10% and 19.7% lower for AISC-360-2010 [3] and ECP 205-2007 [10] than for EC3 [6], respectively. This is mainly due to the difference in the effective width value and the strength reduction factor.
(ii) ECP 205-2007 [10] yields the least resistance and the highest ultimate moment due to the same applied loads.

Resistance of reinforced concrete sections in axial compression

Table 6 presents a comparison of the ultimate axial strength of columns, P_u, for the different studied codes. The columns are considered to be short; consequently the effect of buckling is neglected. This can be done commonly by limiting the height to width or depth ratio of the column. The ultimate axial capacity formulas (P_u), given by different codes, are shown in Eqs. (20)–(22).

$$\text{ACI 318-14 [2]}: \quad P_u = 0.44 f_{ck} A_c + 0.52 f_{yk} A_s \tag{20}$$

$$\text{EC2 [5]}: \quad P_u = 0.57 f_{ck} A_c + 0.87 f_{yk} A_s \tag{21}$$

$$\text{ECP 203-2007 [9]}: \quad P_u = 0.44 f_{ck} A_c + 0.67 f_{yk} A_s \tag{22}$$

The considered parameters in Table 5 include concrete compressive strength, reinforcement yield strength, and reinforcement ratio. The results show that for the same section dimensions, EC2 [5] yields the highest axial strength compared to ACI318-14 and ECP203-2007 by 30% and 23%, respectively. For the Egyptian standard, the ratio is constant and independent of the concrete compressive strength, reinforcement yield strength, or reinforcement ratio.

Resistance of steel columns

IPE 300 steel shape is assumed to be used as a pinned column with different buckling lengths. The ultimate compressive strength is calculated using the formulas provided by the studied codes considering both yielding and buckling limit states as shown in Fig. 5. Comparing the three curves, the following observations can be elaborated:

(i) The ultimate compressive strength curve according to European standards is higher than the curves calculated as per the American and Egyptian standards.
(ii) The ultimate yielding limit state according to EC3 is larger than AISC-360-10 and ECP 203-2007 by 10% and 20%, respectively. This is attributed to the difference in the reduction factor value given by different codes.
(iii) The ultimate buckling limit state according to EC3 is larger than AISC-360-10 and ECP 203-2007 by values ranging between 1.6–14% and 20–45%, respectively.

Mixing design codes

This section is meant to show the consequences of mixing design codes by taking actions from one code and resistances from another code. The comparison is illustrated for a representative example for concrete and steel structures design.

Fig. 6a shows the percentage of the needed reinforcement for a singly reinforced concrete beam. The horizontal axis stands for the design code used to calculate the ultimate moment of resistance, while the vertical axis represents the ratio of required reinforcement in the section. The first bar chart exhibits the reinforcement ratio in case of using associated design codes, i.e. calculating the straining actions and the ultimate resistance with the same code. The rest of bars represent the mixed designs according to the loads calculated as per the shown in figure. Percentage values above the bar chart stand for the variation in the reinforcement ratio for each mixed code case. Negative values indicate unsafe situation, while positive values indicate uneconomic situation with respect to the considered design code. It can be observed that:

(i) ACI 318-14 yields unsafe results upon using the European loading criteria, while conservative results are noticed when using the Egyptian loading criteria.
(ii) Using the American and Egyptian loading criteria along with the European design moment of resistance yields conservative results by 3.8% and 5.3%, respectively.
(iii) Combining the Egyptian standards for resistance with loading criteria other than the Egyptian code leads to unsafe designs.

Fig. 6b illustrates the required section modulus for restrained compact steel sections considering the three studied codes. The comparison is made in the same concept as for the concrete beam. It is apparent that the differences between the different codes for this case are not large. Egyptian specification yields unsafe results when mixed with other loading codes. Meanwhile, European specification yields conservative results when mixed with other codes.

Conclusions

Three building design codes and the corresponding codes for actions are considered. It was shown that comparing variable actions and ultimate resistance of sections separately is useful; however, including the combined effect of both actions and resistances as stipulated by different codes is crucial for better comparison. There are many similarities between design codes in concepts and design formulas. It is a common practice to use provisions according to a certain design code if it is missing from the local design code. However, not only this is illegal, but it could lead to unsafe or uneconomic designs as seen in the previous sections. Differences not only are observed in the safety factors used in calculating the resistance of different sections, but they are also observed in the values of the imposed actions in different design codes. Large differences in live load intensities were noticed after comparing the values stipulated in different codes.

Based upon the comparisons made for the considered cases in this study, the following conclusions could be drawn:

- Concerning variable actions, large differences in intensities exist in some of the studied cases in the current research work. The Egyptian code stipulates values that are same as the European code except for office buildings.
- When variable actions are combined with permanent actions and considering the adverse and beneficial safety

factors, some differences are still observed. Comparing the ultimate load combination of dead and live loads as defined by the studied codes, it was found that ACI 318-14 yields the largest values for residential stairs and balconies. Meanwhile, the Egyptian codes yields the largest values for the residential floors. However, the observed difference decreases as the permanent action to variable action ratio increases.

- Using actions from one code and resistances from another code could lead to unsafe designs. Different safety factors are considered which leads to large variations in the calculated resistance of sections and ultimate load combinations for the cases considered in the current study. Hence, a section might evaluate as safe according to a certain design specification and unsafe according to another.
- For the Egyptian standards, it is recommended to unify the ultimate load factors used for steel and concrete standards. The different reliability levels of permanent loads can be accounted for by distinctive ultimate load factors for the own weight component and the superimposed component.
- The Egyptian standards generally yield the largest section dimensions with the heaviest values of steel reinforcement.
- ECP 203-2007 is the only code that uses the same formulas for the equivalent concrete stress block regardless of the compressive strength of concrete value. Meanwhile, ACI 318-14 yields the largest compression component for a singly reinforced concrete section among the studied codes.
- For steel flexural components, the Egyptian standards yield the largest ultimate load combinations and the lowest sectional capacity considering same rolled section.
- AISC-360-10 yields the smallest sectional capacity for steel flexural members at large values of the unbraced length.
- For steel columns, the axial capacity calculated by EC3 is larger than AISC-360-10 and ECP 203-2007 by a percentage ranging between 1.6% and 45%.
- Mixing the use of different design codes could lead to conservative or unconservative results for the required dimensions, reinforcement, or section modulus.

Conflict of interest

The authors have declared no conflict of interest.

Compliance with Ethics Requirements

This article does not contain any studies with human or animal subjects.

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