Determining shear capacity of ultra-high performance concrete beams by experiments and comparison with codes

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Abstract. In this research, 19 specimens of ultra-high performance fiber-reinforced concrete rectangular beams were made and their shear resistance was determined experimentally. The results were compared with estimations by ACI 318, RILEM TC 162-TDF, Australian guideline, and Iranian national building regulations. To compare the code estimations, the ratio of experimental shear strength to predicted shear strength was calculated for each code. This ratio is in fact a measure of safety factor on the one hand and a measure of precision of the estimation on the other hand. Based on the results of both studies, the authors concluded that the Australian guideline, with the amount of 2.5, provided the minimum experimental to predicted ratio, while the Iranian National Building Regulations, with the amount of about 10, provided the highest one. The ratios obtained by ACI and RILEM were about 8 and 3.6, respectively. The Iranian and ACI codes basically provided the same strength estimation. Both were very conservative, which might mainly be because the codes were dubious about the precision of their own estimations. However, RILEM and Australian codes estimated the shear resistance with reasonable margin of safety.

1. Introduction

Experimental investigations into shear failure of reinforced concrete beams without transverse reinforcement illustrate that the shear behavior of these beams is complex. More specifically, the recent research on High-Strength Concrete (HSC) and Ultra-High Performance Fiber-Reinforced Concrete (UHPFRC) beams emphasizes their quite complex behavior [1-5].

The shortcomings of UHPC, such as low tensile strength and low ductility, along with other excellent characteristics, such as ultra-high compressive strength, have led to the development of UHPFRC in the developed countries. This is made possible by adding steel fiber to concrete (UHPFRC). The combination of two advanced materials, namely, steel fibers and UHPC, leads to many advantages, e.g., increase in UHPC tensile strength and ductility of the fibers, which provides noteworthy resistance against the initiation of cracks. This has led to the development of more slender structural elements and hence, saving of materials and energy. The inherent qualities of UHPFRC, such as high compressive strength, suitable ductility, and ideal tension strength, have made it more suitable for application in special structures and their elements [2,6].

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Investigation into different mechanical charac-teristics and properties of UHPFRC as well as its potentials, in order to replace conventional concrete, has recently attracted the attention of a large number of researchers [3-5,7]. Among these many properties, those relevant to structural design have been relatively more frequently investigated. Researchers have focused on the compressive and tensile strength, modulus of elasticity, Poisson's ratio, creep, and shrinkage [7]. Furthermore, a large number of studies on UHPC have concentrated on flexural [8-11] and shear behaviors in this development [12-18].

Currently, the fact is that shear failure is difficult to predict accurately. This difficulty is more particu-larly observed in high-strength concrete and UHPFRC beams. In spite of many decades of experimental research, some of which was reviewed above, and the use of highly sophisticated analytical tools, practically accurate enough methods and equations for estimating shear capacity are not fully understood yet. More particularly, there is no adequate knowledge about rectangular UHPFRC beams without stirrups.

In fact, the previous experimental and theoretical studies were mainly concentrated on UHPFRC beams with pre-stressing strands and, basically, I-shaped and P-shaped beams. [19-22]. Taking this gap into account, the present paper seeks to study the shear strength of rectangular UHPFRC beams with longitudinal tension rebar and without transverse reinforcement, for which the estimations of different guidelines, regulations, and codes are compared with the results obtained by recent experiments [23].

2. Materials and methods

2.1. Specimens and parameters

Nineteen beam specimens were made at the labora-tories of Tabriz University, Iran, and Florida Inter-national University, US. The beams were of 3 dif-ferent sizes and their dimensions were $152\times152\times559$, $102\times203\times559$, and $152\times76\times559$ mm. The materials of specimens consisted in Portland cement, fine sand, silica fume, superplasticizer, steel fibers, and water for UHPFRC; straight high-strength steel fiber (with the length of 13 mm, diameter of 0.18 mm, and specified tensile strength of 2700 MPa) for fibers; and deformed steel bars for longitudinal steel with specified yield points of about 400 MPa (60 ψ) and 690 MPa (100 ψ). The mix design and curing process of specimens were detailed in a previous study by the first author [23], which is also explained briefly here. Nine beams were cast from one batch of UHPC in Tabriz using the above-mentioned materials. In order to make the project economical, local materials and a conventional concrete mixer were used. The mixer was reinforced by adding steel plates inside the drum. Moreover, since

UHPC required increased energy input compared to conventional concrete, the mixing time was increased. To ensure that the UHPC did not overheat during mixing and to make the process more convenient, the temperature of the constituents was lowered and a mixture of ice and water was used, instead of water alone. As recommended by Graybeal, first, silica fume was mixed with all the sand for approximately 5 minutes [3,5]. Then, before adding water, cement and ground quartz were added and dry mixed for at least 5 minutes. Then, in order to improve flowability, su-perplasticizer was added gradually. After a number of trials, a water-cement ratio (w/c) of 0.24 was obtained for the final mixture. Straight high-strength steel fiber was added by 6.1% of the weight in order to improve the mechanical properties of concrete, especially in terms of tensile strength and ductility. The superplasticizer used was AURAMIX 4450 (FOSROC), which was a polycarboxylic ether based superplasticizer.

While the 9 beams were made using generic mix of local material in Tabriz, a commercial product, known as Ductal® (similarly including cement, silica fume, ground quartz, and sand), was used in Miami to make specimens with a w/c of 0.20 and steel fiber of 6.4% by weight. The rest of the process, including mixing procedure in Miami, was similar to that in Tabriz.

Cubes of $100\times100\times100$ mm and cylinders of 100×200 mm specimens were also made to determine the compressive strength of UHPFRC. The specimens were kept under water in Tabriz, and in the laboratory with almost constant temperature in Miami (because of high humidity). The compressive strengths were obtained to be 125 MPa and 137 MPa for the specimens in Tabriz and Miami, respectively. Figures 1 and 2 show the mixing process in Tabriz and Miami. It should be noted that, as Graybeal reported, cube specimens (100×100 mm) had compressive strengths about 5% higher than the cylinder specimens (100×200 mm), so

Figure 1. Drum strengthened by plate used for UHPFRC mixing in Tabriz (Iran).

Figure 2. Site with equipment used for UHPFRC mixing in Miami (US).

the compressive strengths obtained for cube specimens were reduced by 5% [4].

The shear strength of each of the 19 specimens was determined by 4-point loading test at the laboratories of Tabriz University and Florida International University. Also, the shear strength of each specimen was estimated using various codes and regulations including ACI 318, RILEM TC 162-TDF, Australian guideline, and Iranian National Building Regulations, as will be explained in the next sections. To assess the estimation capability of the codes and regulations, the ratio of experimental shear strength to predicted shear strength (EP) was calculated. This ratio, EP, was used to compare the codes.

Table 1 provides the information about the prop-

erties of specimens tested by Pourbaba [23]. All the beams were 559 mm in total length, having a span of 457 mm. While the widths of specimens were only 152 and 102 mm, they had different total depths of 152, 203, and 76 mm. Also, given that different size bars were used, the effective depth of reinforcement was also different from specimen to specimen. The effective depths were about 126, 180, and 55 mm. As indicated in Table 1, the longitudinal reinforcements included 3∅25, 3∅22, 3∅20, 3∅19, 3∅18, 2∅20, 2∅16, 3∅14, 3∅12, and 3∅10. Figure 3 shows the three-dimensional view and bar placement of the specimens. It is worth mentioning that the specimens used in the above-mentioned research contained no transverse reinforcing bars.

Figure 4 presents the test setup designed for the experiments explained in Pourbaba's dissertation [23]. As shown in Figure 4, the shear span was $a = 153$ mm (203-51) for all the specimens; however, the ratio of shear span to depth, a/d, was different for different specimens.

2.2. Review of various codes, regulations, guidelines, and design methods

2.2.1. ACI 318 (building code requirements for structural concrete, American concrete institute)

ACI Code presents the basic shear equations in terms of shear forces, not shear stresses. In fact, in order to obtain the total shear forces, the average shear stresses are multiplied by the effective beam areas. The shear strength of the concrete, denoted by V_c, is obtained by

Table 1. Properties of tested UHPC beams by Pourbaba [23]*.

Name	Section $b \times h$ (mm)	Rebars (s)	d (mm)	A_s (mm^2)	f'_c (MPa)	f_y (MPa)	ρ	a/d
B1**	152×152	3∅25	125	1473	137	690	0.078	1.2
B2**	152×152	3∅22	126	1473	137	690	0.060	1.2
B3**	152×152	3∅25	125	1473	137	414	0.078	1.2
B4**	152×152	3∅22	126	1473	137	414	0.060	1.2
B5**	152×152	3∅19	128	1473	137	414	0.044	1.2
B21	152×152	3∅20	127	942.5	125	400	0.049	1.2
B22	152×152	3∅18	128	763.4	125	400	0.039	1.2
B23	152×152	3∅20	127	942.5	125	400	0.049	1.2
B24	152×152	3∅18	128	763.4	125	400	0.039	1.2
B29	102×203	2∅20	178	628.3	125	400	0.035	0.9
B30	102×203	2∅16	180	402.1	125	400	0.022	0.8
B35	152×76	3∅14	54	461.8	125	400	0.056	2.8
B36	152×76	3∅12	55	339.3	125	400	0.040	2.8
B37	152×76	3∅10	56	235.6	125	400	0.027	2.7

* Based on data from Pourbaba's dissertation [23].

** These specimens were tested in duplicates (*a* and *b*) in Miami to confirm repeatability.
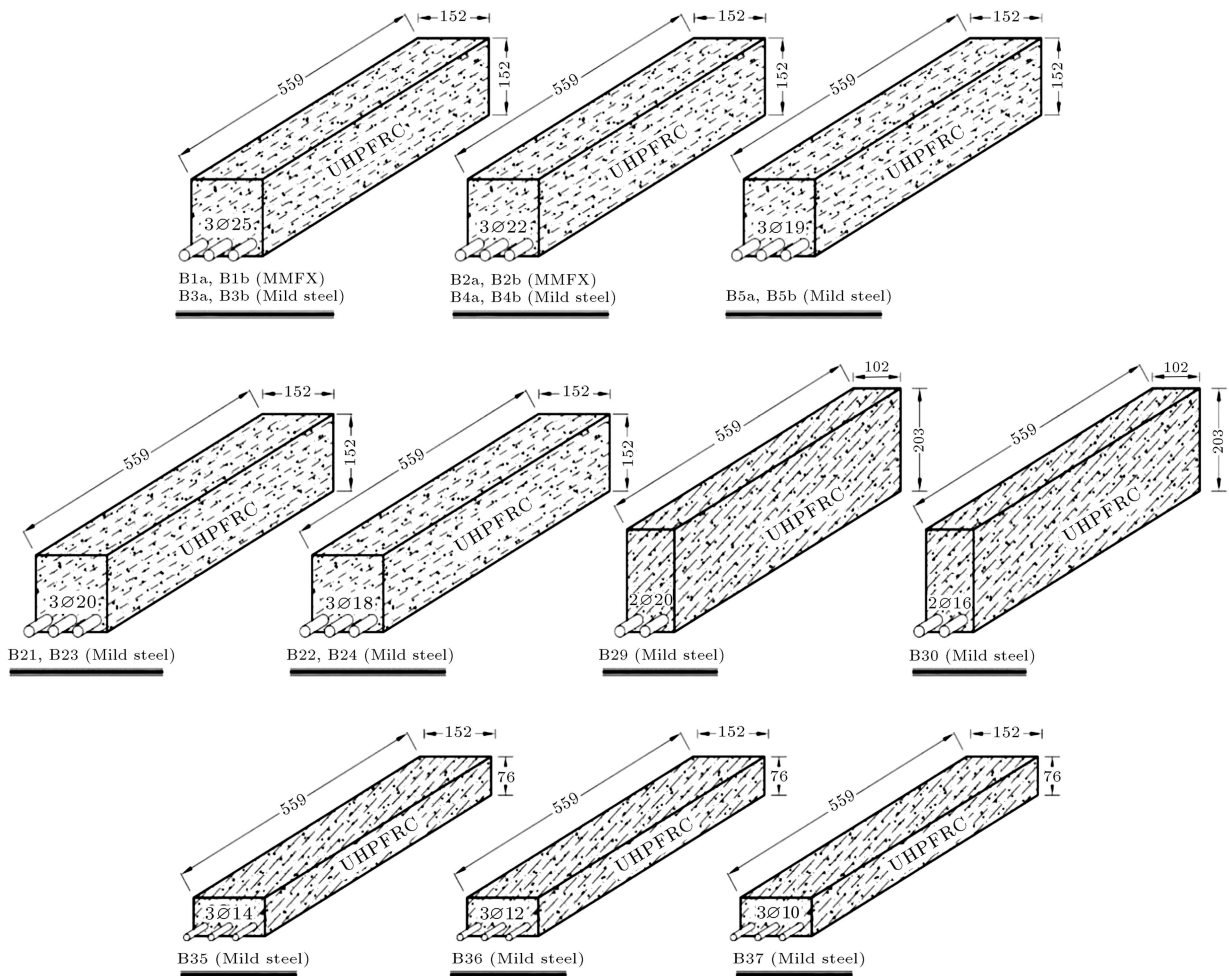
Figure 3. Three-dimensional view of specimens tested by Pourbaba [23] (units are in mm).
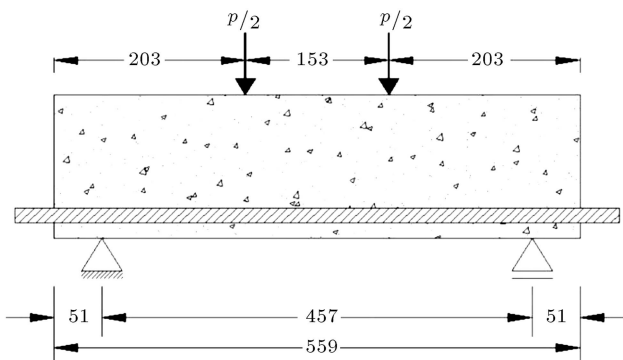

Figure 4. Test setup used by Pourbaba [23] to test UHPC specimens.

the following equation:

$$V_c = \frac{\sqrt{f'_c}}{6} b_w d, \qquad (1)$$

where is the specified compressive strength of concrete at age of 28 days, b_w is the width of a rectangular beam, and d is the effective depth.

Furthermore, according to ACI Code, V_c can go

even higher and consequently, be obtained by Eq. (2) in the following, in which the effects of the longitudinal reinforcing as well as the moment and shear magnitudes have been taken into consideration [24,25]:

$$V_c = \left(\sqrt{f'_c} + 120 \rho_w \frac{V_u d}{M_u} \right) \frac{b_w d}{7} \le 0.30 \sqrt{f'_c} b_w d, \qquad (2)$$

where $\rho_w = As/(b_w d)$ is the reinforcement ratio and M_u is the moment occurring in combination with shear force V_u at the cross section considered. Also, according to ACI, in the above equation for V_c, $V_u d / M_u$ shall not be taken greater than unity [23,24]. Taking Figure 4 into account, in our case, $V_u = P/2$ and $M_u = V_u \times a = Pa/2$; consequently, $V_u d/M_u = d/a$. According to the last column of Table 1, $V_u d/M_u = d/a = 1/(d/a)$ is smaller than 1, except for B29 and B30, where the ratios are 1.11 and 1.25, i.e., marginally above the ACI limit.

Using Eqs. (1) and (2), the shear strength of the beam specimens was determined, of which the results are tabulated in the third and fourth columns of Table 2.

Table 2. Maximum experimental shear forces and predicted shear forces using various codes.

Specimens	Ultimate shear strength* (kN)	Predicted shear strength (kN)					
		ACI		RILEM*	Australian guideline	Iranian national building regulations	
		Eq. (1)	Eq. (2)			Eq. (12)	Eq. (13)
B1a	416	37	52	103	123	27	40
B1b	476	37	52	103	123	27	40
B2a	468	37	48	104	125	27	37
B2b	436	37	48	104	125	27	37
B3a	462	37	52	103	123	27	40
B3b	357	37	52	103	123	27	40
B4a	367	37	48	104	125	27	37
B4b	388	37	48	104	125	27	37
B5a	402	38	45	105	126	27	34
B5b	383	38	45	105	126	27	34
B21	349	36	44	86	125	26	34
B22	332	36	42	87	126	26	33
B23	335	36	44	86	125	26	34
B24	336	36	42	87	126	26	33
B29	409	34	40	81	117	24	31
B30	342	34	36	82	118	25	28
B35	106	15	16	37	53	11	12
B36	85	16	16	37	54	11	12
B37	71	16	15	38	55	11	12

* Based on data from Pourbaba's dissertation [23].

2.2.2. RILEM TC 162-TDF (test and design methods for steel fiber reinforced concrete)

The residual flexural tensile strength $f_{R,i}$ is defined as an important parameter, which characterizes the post cracking behavior of steel fiber reinforced concrete. To achieve this property, three-point bending test on notched beams, according to EN 14651 (2005) [26], was conducted by Pourbaba the results of which are used in the current research [23].

The methods of conducting the above-mentioned test have been discussed in some sources such as RILEM TC 162-TDF recommendation (2003) [27], EN 14651 (2005) [26], and fib model code for concrete structures 2010 [28].

The three-point bending test on notched prisms was conducted in accordance with EN 14651(2005) to determine the post-cracking behavior under tension and used to predict shear resistance of the beams without shear reinforcement. The specimens had a height of 150 mm, a width of 150 mm, a span of 500 mm, and a length of 550 mm with an initial notch of 25 mm in the middle [23].

According to RILEM TC 162-TDF (2003), the residual flexural tensile strengths $f_{R,1}$ and $f_{R,4}$ are respectively defined at 0.5 mm and 3.5 mm crack mouth opening displacements and can be determined by means of the following expression:

$$f_{R,i} = \frac{3F_{R,i} \times L}{2b \times h_{sp}^2} \qquad \text{N/mm}^2, \tag{3}$$

where b is the width of the specimen, h_{sp} is the distance between the tip of the notch and the top of the cross section, and L is the span of the specimen all in mm.

Hence, the following RILEM TC TDF-162 (2003) equations (standard method) have been used to obtain the nominal shear strength of UHPC beams [14,27]:

$$V_u = V_c + V_f + V_s, \tag{4}$$

$$V_c = V_{\text{concrete}} = [0.12k(100\rho_l f_{fck})^{1/3} + 0.15\sigma_{cp}]b_w d, \tag{5}$$

$$V_f = V_{\text{fibers}} = 0.7k_f k\tau_{fd}b_w d, \tag{6}$$

where:

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2, \quad \rho_l = \frac{A_s}{b_w d} \leq 0.02,$$

$$\tau_{fd} = 0.12f_{Rk,4}, \tag{7}$$

where k_f is for T-sections and A_s is the tension reinforcement in the section considered in mm^2, b and d are respectively the section width and the effective depth in mm, and V_s is contribution of the shear reinforcement due to stirrups, which in our case was equal to 0.

2.2.3. Australian design guidelines for ductal prestressed concrete beams

According to the Australian design guidelines for ductal prestressed concrete beams [29], the following formula gives the shear strength of a prestressed concrete section:

$$V_u = V_{uc} + V_{us} + P_v, \tag{8}$$

where V_{uc} is the contribution of the concrete to the shear strength, V_{us} is the contribution of the transverse shear reinforcement, and P_v is the transverse component of the prestressing force.

When shear reinforcement and inclined tendons are absent, for pretensioned beams, the shear strength is determined from:

$$V_u = V_{uc}. \tag{9}$$

The shear strength of UHPC in beams depends on limiting the principal tensile stress at the centroidal axis or at the junction of the web and flange to a maximum value based on the uncracked section in flexure. This maximum value is given in the following equations [7,29]:

$$v_c = 5.0 + 0.13\sqrt{f'_c}, \tag{10}$$

$$V_c = v_c b_w d. \tag{11}$$

The results are tabulated in the sixth column of Table 2.

2.2.4. Iranian national building regulations (design and construction of concrete structures)

The following equations are suggested by Iranian national building regulations to predict the nominal shear strength and shear stress:

$$V_c = v_c b_w d, \tag{12}$$

$$v_c = 0.2\phi_c\sqrt{f_c}, \tag{13}$$

where b_w and d are the width of rectangular beam section and effective depth, respectively; v_c is the nominal shear stress; and ϕ_c is the safety factor for concrete that equals 0.60. Also, f_c is the 28-day compressive strength of concrete (standard cylinder strength).

Moreover, there is another equation in the Iranian national building regulations (design and construction

of concrete) for concrete beams subjected to shear combined with bending:

$$V_c = \left(0.95 v_c + 12\rho_w \frac{V_u d}{M_u}\right) b_w d, \tag{14}$$

where ρ_w is the reinforcement ratio and $\rho_w = A_s/(b_w d)$; also, M_u is the moment occurring in combination with shear force V_u at the cross section. Similar to ACI code, the Iranian National Building Regulations limit the value of $\frac{V_u d}{M_u}$ to 1.0. In addition, the Iranian Building Regulation require that V_c should be equal to or less than 1.75 $v_c b_w d$ [30]. In our case, when considering the overview of beam and applied loads shown in Figure 4, $V_u = P/2$ and $M_u = V_u \times a = Pa/2$ (a is shear span of specimens); therefore, $V_u d/M_u = d/a$.

3. Results and discussions

Table 2 indicates the maximum shear load recorded ($P/2$) during the testing of the specimens [23]. Also, it presents the predicted shear loads, which were determined by applying ACI, RILEM, Australian, and Iranian equations.

As Table 2 and Figure 5 point out, all predicted shear strengths using various models (ACI, RILEM, Australian, and Iranian equations) are less than the experimental shear forces. In Figure 5, the shear strength from testing the experiments is plotted versus its estimated value by each code. The 45° line drawn from the origin shows the points for a hypothetical situation where experiment and code prediction could determine the same shear strength. As can be seen, all the points are above the 45° line. This means that all the codes have underestimated the shear capacity for all the specimens. Studying more details
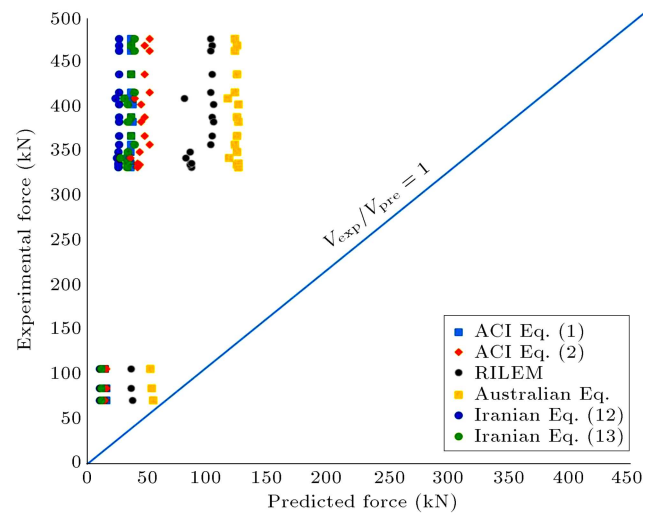
Figure 5. Experimental shear force versus predicted shear strength of existing predictive models for each specimen.

Table 3. Ratios of experimental shear strength to predicted shear force using various codes.

Specimens	$V_{\exp}/V_{\mathrm{pre}}$					
	ACI		RILEM	Australian guideline	Iranian regulation	
	Eq. (1)	Eq. (2)			Eq. (12)	Eq. (13)
B1a	11.2	8.0	4.0	3.4	15.4	10.4
B1b	12.9	9.2	4.6	3.9	17.6	11.9
B2a	12.6	9.8	4.5	3.7	17.3	12.6
B2b	11.8	9.1	4.2	3.5	16.1	11.8
B3a	12.5	8.9	4.5	3.8	17.1	11.6
B3b	9.6	6.9	3.5	2.9	13.2	8.9
B4a	9.9	7.6	3.5	2.9	13.6	9.9
B4b	10.5	8.1	3.7	3.1	14.4	10.5
B5a	10.6	8.9	3.8	3.2	14.9	11.8
B5b	10.1	8.5	3.6	3.0	14.2	11.3
B21	9.7	7.9	4.1	2.8	13.5	10.3
B22	9.2	7.9	3.8	2.6	12.7	10.2
B23	9.3	7.6	3.9	2.7	12.9	9.8
B24	9.3	8.0	3.9	2.7	12.9	10.3
B29	12.1	10.3	5.1	3.5	16.8	13.3
B30	10.0	9.4	4.2	2.9	13.9	12.1
B35	6.9	6.7	2.9	2.0	9.6	8.5
B36	5.5	5.5	2.3	1.6	7.6	7.0
B37	4.5	4.7	1.9	1.3	6.2	6.0
Average	**9.9**	**8.0**	**3.8**	**2.9**	**13.7**	**10.4**

reveals that the Australian guideline has given the closest predictions to the experimental results, while the Iranian national building regulations have made predictions very conservatively, as there is a large and wide gap between the predicted shear strength and the shear force obtained from tests.

Table 3 presents the ratios of maximum experimental shear strength to predicted shear force for all the tested specimens using ACI, RILEM, Australian, and Iranian equations, separately. Table 3 also shows the average of the above-mentioned results obtained from each guideline, separately.

As can be seen in Table 3, the minimum average of the experimental shear forces to the predicted shear strength is 2.9, which belongs to Australian guideline. This shows that the Australian guideline is reasonably conservative. The minimum $V_{\exp}/V_{\mathrm{pre}}$ ratio among all the specimens is 1.3, which also belongs to the Australian guideline, for the B37 specimen. Moreover, the second nearest prediction belongs to RILEM results, which are just slightly conservative in comparison with ACI and Iranian equations that are extremely conservative. This is due to the fact that unlike in the ACI and Iranian regulations, in the Australian guideline and RILEM design methods, the effect of steel fiber reinforcement contribution has been taken into consideration.

The average predictions of the first equation of

ACI Code (Eq. (1)) (column 2 of Table 3) and the first equation of Iranian regulations (Eq. (12)) (column 7 of Table 3), which do not consider the effects of longitudinal reinforcement, and moment and shear magnitudes, are greater and more conservative than the predictions of other equations in the same codes (Eq. (2) and Eq. (13)). This shows that equations taking into account the effect of moment and shear as well as longitudinal reinforcement give better predictions than others do.

Figure 6 indicates the average values of $V_{\exp}/V_{\mathrm{pre}}$ for all the specimens using various codes and reg-
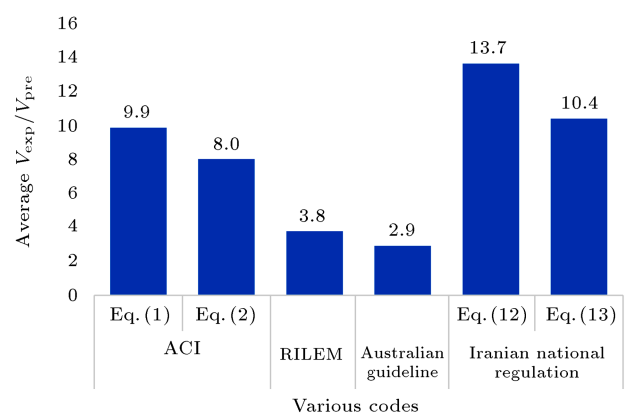
Figure 6. Average ratios of experimental shear strength to predicted shear force obtained by various codes.

ulations. As clearly illustrated by the graph, the Australian guideline gives the nearest prediction ratio with the value of 2.9. It is followed by RILEM design methods with an average of 3.8, while the third closest prediction belongs to ACI code with 8.0 and 9.9 corresponding to its two different equations. Finally, the greatest ratio of experimental to predicted shear strength is for the Iranian national building regulations with 10.4 and 13.7 for its two various equations.

4. Conclusions

The predicted shear strength of ultra-high performance concrete rectangular beams using various international codes was studied. The predicted shear strengths were compared with the obtained experimental shear strengths, tested by the authors. Moreover, the ratio of experimental to predicted shear strength was determined by various well-known regulations and the obtained safety factors were compared with each other. Based on the results of this research for the Ultra-High Performance Fiber-Reinforced Concrete (UHPFRC) beams without stirrups, the following conclusions can be drawn:

1. All the predicted shear strengths by using various models, including ACI, RILEM TC 162-TDF, Australian guideline (design guidelines for ductal prestressed concrete beams), and Iranian national building regulations (design and construction of concrete structures), are less than experimental maximum shear forces. That is, all the codes intend to be on the conservative side when estimating shear strength;

2. According to the results of the predicted shear forces obtained by various codes, design methods, and regulations, for the beams tested, it can be stated that the Australian design method is quite reasonably conservative, while the RILEM TC 162-TDF is slightly more conservative; however, the other codes (ACI code and Iranian regulations) are drastically conservative. A reason is that the Australian design method and RILEM consider the effect of fibers, but the other codes do not consider this important parameter; therefore, taking into account the effect of steel fiber reinforcement in the UHPC beams by ACI code and Iranian regulations or providing new codes and guidelines, specifically for ultra-high performance fiber-reinforced concrete structures, seems absolutely essential;

3. On average, for all the tested UHPC beams, the V_c obtained by Australian guideline is 2.9 times less than the experimentally obtained V_c, while the V_c obtained by RILEM TC 162-TDF equation is 3.8 times less than the experimentally obtained V_c. The two ACI equations (Eqs. (1) and (2)) give

the experimental to estimated shear strength ratios of 9.9 and 8.0, respectively. These ratios are 13.7 and 10.4 for two approaches of Iranian regulations (Eq. (12) and Eq. (13)), respectively (Figure 6);

4. The predictions of ACI Eq. 1 and Iranian regulations Eq. 13 give almost similar averages of the ratio of experimental shear strength to predicted shear strength (about 10), both of which are quite conservative as compared with the average ratios obtained by application of RILEM and Australian equations.

Acknowledgment

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