

Determining Shear Capacity of Ultra-high Performance Concrete Beams by Experiments and Comparison with Codes

Masoud Pourbaba¹, Abdolreza Joghataie²

Abstract

In this research, nineteen specimens of ultra-high performance fiber-reinforced concrete rectangular beams are made and their shear resistance is determined experimentally. The results are 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 is calculated for each code. This ratio is actually a measure of safety factor on one hand and a measure of precision of the estimation on the other hand. Based on the results of both studies, the authors conclude that the Australian guideline with a ratio of 2.5 provides the minimum experimental to predicted ratio while the Iranian National Building Regulations with a ratio of about 10 provides the highest experimental to predicted ratio. This ratio obtained for ACI and RILEM was about 8 and 3.6 respectively. The Iranian and ACI codes provide basically the same strength estimation but both are very conservative, which may be interpreted as mainly because the codes are dubious about the precision of their own estimations. However, RILEM and Australian codes, estimate the shear resistance with reasonable margin of safety.

Keywords: Ultra-High Performance Fiber-Reinforced Concrete Beams (UHPFRC), Shear Strength, Steel fibers, Regulations, Experimental to predicted shear strength

1. Introduction

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Experimental investigations on shear failure of reinforced concrete beams without transverse reinforcement illustrate that the shear behavior of these beams is complex. More specifically, recent research on high strength concrete (HSC) and ultra-high performance fiber-reinforced concrete (UHPFRC) beams emphasize on their quite complex behavior [1-5].

The shortcomings of UHPC such as its low tensile strength and low ductility along with other excellent characteristics such as its ultrahigh compressive strength have led to the development of UHPFRC in developed countries. This was made possible by adding steel fiber to concrete (UHPFRC). When sufficient steel fibers were mixed with UHPC, many advantages resulted from the combination of these two advanced materials, among which we can refer to the increasing of UHPC tensile strength and ductility through the fibers which provide noteworthy resistance against the initiation of cracks. This has led to the development of more slender structural elements and the saving of materials and energy. The inherent qualities of UHPFRC such as its high compressive strength, suitable ductility and ideal tension strength has made it more suitable for application in special structures and their elements [2, 6].

Investigation of different mechanical characteristics and properties of UHPFRC and its potentials in order to replace conventional concrete has recently attracted the attention of a large number of researchers [3, 4, 5, 7]. From 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 of this development [12-18].

The current fact is that shear failure is difficult to predict accurately. This difficulty is more particularly observed in high strength concrete and UHPFRC beams. In spite of many decades of experimental research, some of which were reviewed above, and the use of highly

sophisticated analytical tools, practically accurate enough methods and equations for estimating shear capacity is not yet fully understood. More particularly, there is no adequate knowledge on 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 article seeks to study the shear strength of rectangular UHPFRC beams with longitudinal tension rebar and without transverse reinforcement where shear strength estimation by different guidelines, regulations and codes are compared with the results obtained from recent experiments [23].

2. Materials and Methods

2.1 Specimens and parameters

Nineteen beam specimens were made at the laboratory of Tabriz University (Iran) and Florida International University (US). The beams were of three different sizes and their dimensions were 152×152×559, 102×203×559 and 152×76×559 mm. The material of specimens consisted of Portland cement, fine sand, silica fume, superplasticizer, steel fibers, and water for UHPFRC, straight high strength steel fiber (13 mm long, 0.18 mm diameter and specified tensile strength of 2700 MPa) for fibers, and deformed steel bars for longitudinal steel with a specified yield point of about 400 MPa (60 psi) and 690 MPa (100 psi). The mix design and curing process of specimens have been detailed in a previous study by the first author [23] which is explained here briefly too. 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 were used and a conventional concrete mixer was used too. This mixer had been reinforced by adding steel plates inside the drum. Moreover, since UHPC requires 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 only. 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, superplasticizer 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 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 is a polycarboxylic ether based superplasticizer.

While the nine beams were made using generic mix of local material in Tabriz, a commercial product known as Ductal® (same material included 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 Tabriz.

Cubes of 100×100×100 mm and cylinders of 100 × 200 mm specimens were made too, to determine the compressive strength of UHPFRC. The specimens were kept under water in Tabriz and in the Lab with almost constant temperature in Miami (because of high humidity). The compressive strengths were obtained as 125 MPa and 137 MPa for the specimens in Tabriz and Miami respectively. Figure 1 and Figure 2 show the mixing process in Tabriz and Miami. It should be noted that as Graybeal reported, cube specimens (100×100 mm) have compressive strengths about 5 percent higher than the cylinder specimens (100×200mm), so the compressive strengths obtained from cube specimens were reduced by five percent [4].

The shear strength of each of the nineteen specimens were determined from four point loading test at the laboratory 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 contains the information about the properties of specimens tested by Pourbaba [23]. All the beams were 559 mm in total length, having a span of 457 mm. While the width of specimens were only 152 and 102 mm, they had different total depths of 152 mm, 203 mm and 76 mm. Also, noticing 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 3Ø25, 3Ø22, 3Ø20, 3Ø19, 3Ø18, 2Ø20, 2Ø16, 3Ø14, 3Ø12 and 3Ø10. Figure 3 shows the three-dimensional view and bar placement of the specimens. It is worth mentioning that the specimens used in that 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 is $a=153$ 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 and 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 provided by concrete, denoted by V_c , is obtained by the following equation:

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

Where f'_c 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 even go higher and consequently be obtained by the following equation 2 in which the effects of the longitudinal reinforcing and the moment and shear magnitudes have been taken into consideration [24, 25]:

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

where $\rho_w = A_s / (b_w d)$ 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 $V_u d / M_u = d/a$. From 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 ratio is 1.11 and 1.25 that are marginally above the ACI limit.

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

2.2.2 RILEMTC 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], were conducted by Pourbaba which their results are used in the current research [23].

The methods of conducting the above-mentioned test is 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 were conducted in accordance with EN 14651(2005) to determine the post-cracking behavior under tension and were 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 mid [23].

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

$$f_{R,i} = \frac{3F_{R,i} \times L}{2b \times h_{sp}^2} \quad \frac{N}{mm^2} \quad (3)$$

where b is the width of the specimen in mm; h_{sp} is the distance between the tip of the notch and the top of the cross section in mm and L is the span of the specimen 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]:

$$V_u = V_c + V_f + V_s \quad (4)$$

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

$$V_f = V_{fibers} = 0.7k_f k \tau_{fd} b_w d \quad (6)$$

$$\text{where } k = 1 + \sqrt{\frac{200}{d}} \leq 2, \rho_l = \frac{A_s}{b_w d} \leq 0.02 \text{ and } \tau_{fd} = 0.12 f_{Rk,4} \quad (7)$$

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 the section width and the effective depth in mm respectively; 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 formulae gives the shear strength of a prestressed concrete section:

$$V_u = V_{uc} + V_{us} + P_v \quad (8)$$

Where V_{uc} is the contribution of the concrete to the shear strength; V_{us} is the contribution of the transverse shear reinforcement; 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:

$$V_u = V_{uc} \quad (9)$$

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]:

$$v_c = 5.0 + 0.13\sqrt{f_c} \quad (10)$$

$$V_c = v_c b_w d \quad (11)$$

The results are tabulated in the sixth columns 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:

$$V_c = v_c b_w d \quad (12)$$

$$v_c = 0.2\phi_c \sqrt{f_c} \quad (13)$$

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. 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:

$$V_c = (0.95v_c + 12\rho_w \frac{V_u d}{M_u})b_w d \quad (14)$$

where ρ_w is the reinforcement ratio and $\rho_w = A_s / (b_w d)$; and M_u is the moment occurring in combination with shear force V_u at the cross section. Also, similar to ACI code, the Iranian

National Building Regulations limits the value of $\frac{V_u d}{M_u}$ to 1.0. In addition, the Iranian

regulations requires that V_c should be equal to or less than $1.75v_c b_w d$ [30]. In our case

when considering 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 they present 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 from 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 of all the specimens. Studying more details reveals

that the Australian guideline has given the closest and nearest predictions to the experimental results, while the Iranian national building regulations has predicted 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 from Table 3, the minimum average of the experimental shear force to the predicted shear strength is 2.9 which is related to Australian guideline. This shows that the Australian guideline is reasonably conservative. The minimum ratio of V_{exp}/V_{pre} among all the specimens is 1.3 which also belongs to Australian guideline and related to 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 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 in the Table 3) and first Iranian regulations Eq.12 (column 7 in the Table 3), which do not consider the effect of longitudinal reinforcement and effect of moment and shear magnitudes, are greater and more conservative than the other equation in the same code (Eq. 2 and Eq.13). This shows that equations taking into account the effect of moment and shear and also longitudinal reinforcement give better predictions than others do.

Figure 6 indicates the average of V_{exp}/V_{pre} for all the specimens using various codes and

regulations. As illustrated clearly by the graph, the Australian guideline gives the nearest prediction ratio, with a ratio 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 strengths is from 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 authors. Moreover, the ratio of experimental to predict shear strengths were determined by various well-known regulations and the obtained safety factors were compared together. Based on the results of this research on 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. Which means that 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 concluded that the Australian design method is quite reasonably conservative while the RILEM TC 162-TDF is slightly more conservative but the other codes (ACI code and Iranian regulations) are drastically conservative. A reason is because the Australian design method and RILEM consider the effect of fibers but the other codes do not consider this important parameter and so, taking into account the effect of steel fiber reinforcement in the UHPC beams in

ACI code and Iranian regulations or providing new codes and guidelines specifically for ultra- high performance fiber-reinforced concrete structures seems completely essential.

3. In 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 V_c obtained by RILEM TC 162-TDF equation is 3.8 times less than the experimentally obtained V_c . The two ACI equations (Eq.1 and Eq.2) have given the experimental to estimated shear strength ratios of 9.9 and 8.0 respectively. These ratios are 13.7, 10.4 for two Iranian regulations approaches (Eq.12 and Eq.13) respectively. (Figure 6)
4. The predictions in accordance with ACI equation 1 and Iranian regulations equation 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 to the average ratio obtained from application of RILEM and Australian equations.

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Biography:

Masoud Pourbaba obtained his BS and MS degrees in Civil Engineering from Tabriz and Mazandaran University in 1998 and 2001, respectively. He is currently a PhD student in the

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

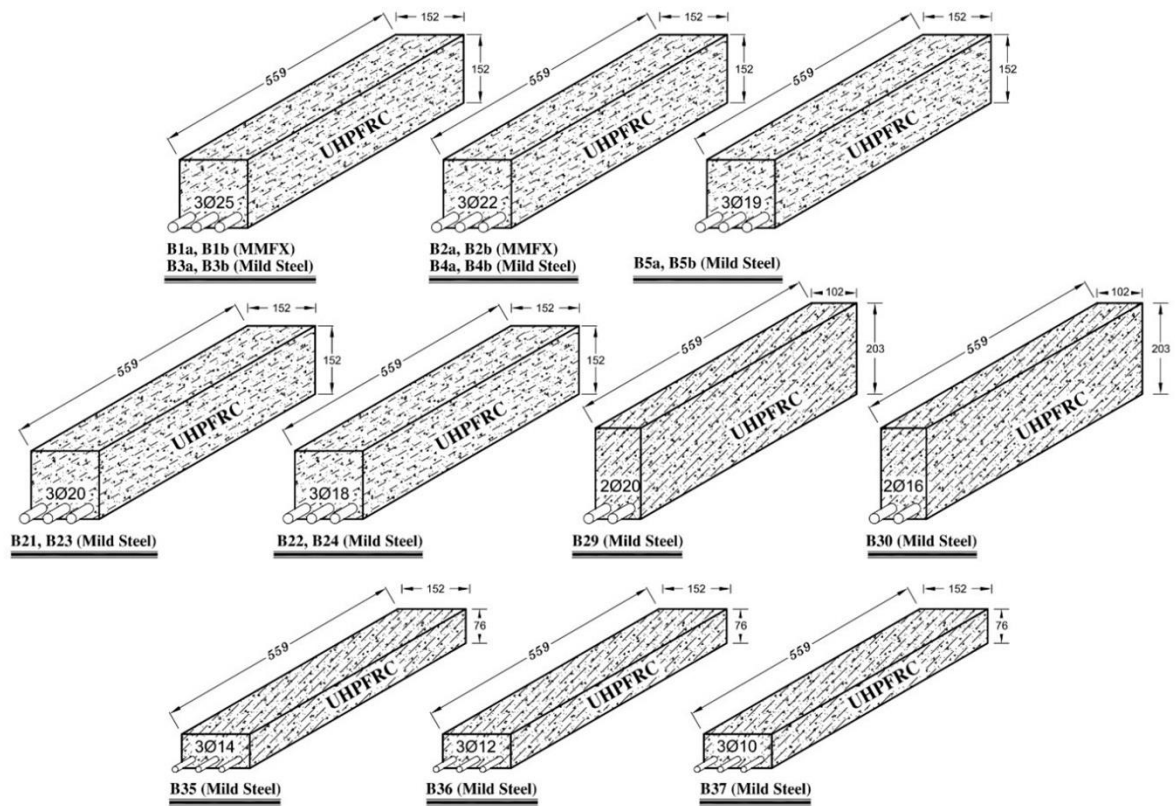


Figure 3. Three-dimensional view of specimens tested by Pourbaba et al. (units are in mm).

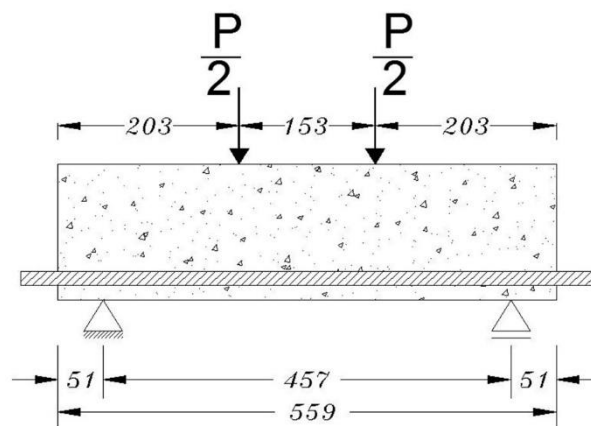


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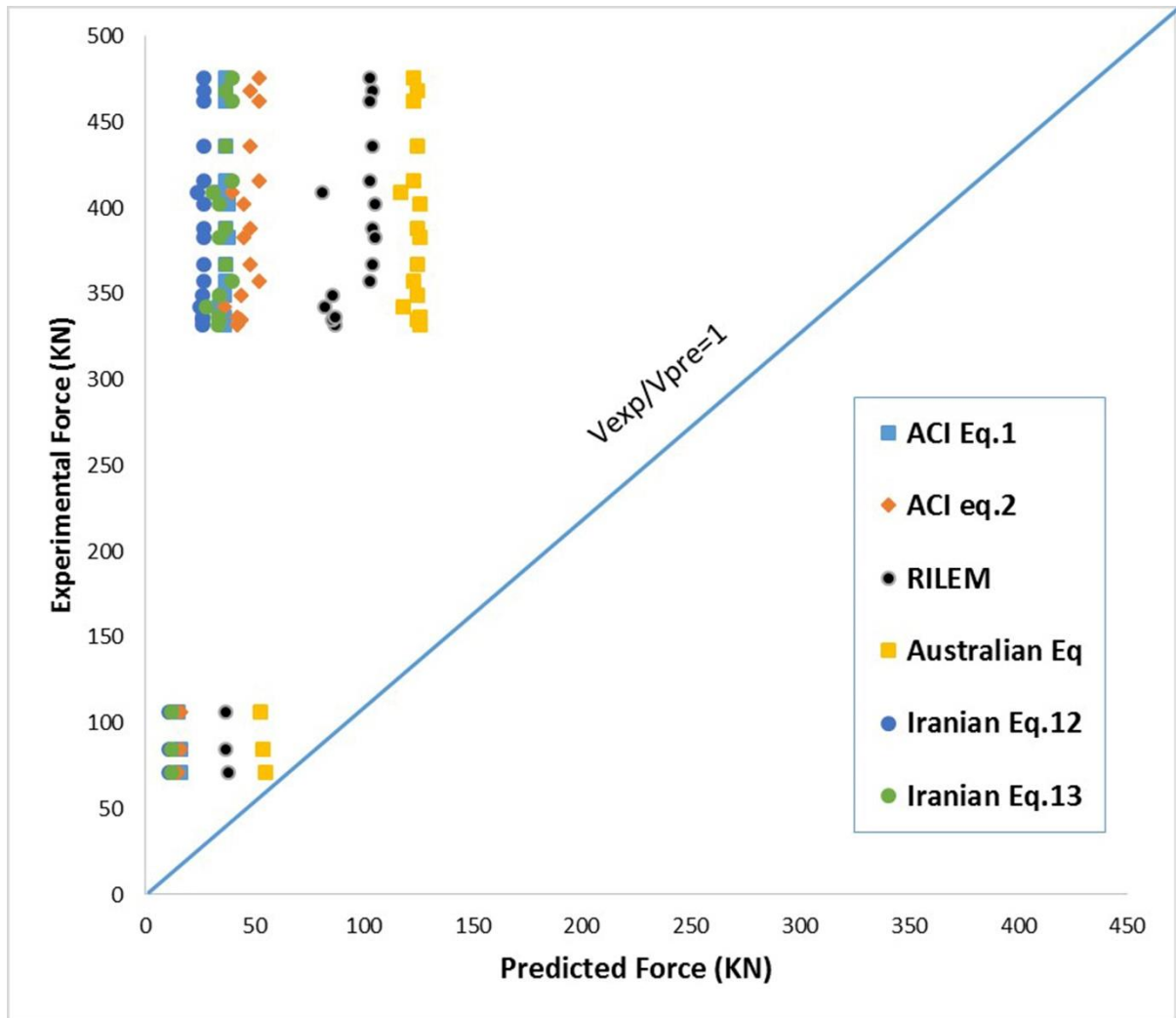


Figure5. Experimental shear force versus predicted shear strength of existing predictive models for each specimens

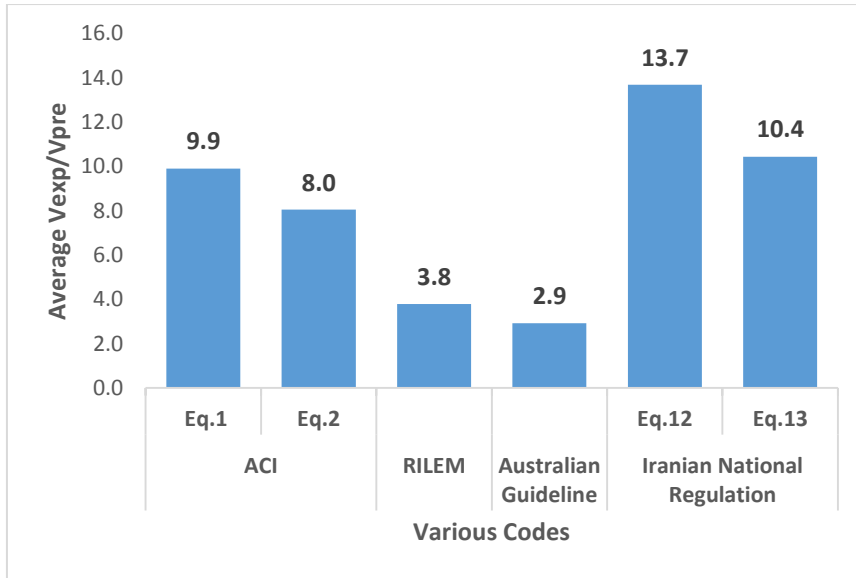


Figure 6. Average ratios of experimental shear strength to predicted shear force obtained from various codes

Table 1*. Properties of tested UHPC beams

Name	Section b×h (mm)	Rebar s	d (mm)	A _s (mm ²)	f' _c (MPa)	f _y (MPa)	ρ	a/d
B1**	152×152	3Ø25	125	1473	137	690	0.078	1.2
B2**	152×152	3Ø22	126	1473	137	690	0.060	1.2
B3**	152×152	3Ø25	125	1473	137	414	0.078	1.2
B4**	152×152	3Ø22	126	1473	137	414	0.060	1.2
B5**	152×152	3Ø19	128	1473	137	414	0.044	1.2
B21	152×152	3Ø20	127	942.5	125	400	0.049	1.2
B22	152×152	3Ø18	128	763.4	125	400	0.039	1.2
B23	152×152	3Ø20	127	942.5	125	400	0.049	1.2
B24	152×152	3Ø18	128	763.4	125	400	0.039	1.2
B29	102×203	2Ø20	178	628.3	125	400	0.035	0.9
B30	102×203	2Ø16	180	402.1	125	400	0.022	0.8
B35	152×76	3Ø14	54	461.8	125	400	0.056	2.8
B36	152×76	3Ø12	55	339.3	125	400	0.040	2.8
B37	152×76	3Ø10	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

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]

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

Specimens	V_{exp}/V_{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

Biography:

Masoud Pourbaba obtained his BS and MS degrees in Civil Engineering from Tabriz and Mazandaran University in 1998 and 2001, respectively. He is currently a PhD student in the Sharif University of Technology, International Campus, and he is also a faculty member at Islamic Azad University (Maragheh Branch) from 2004. He is also a member of ASCE and

ACI. He took a sabbatical leave at Florida International University to work on UHPC in 2014. His main research interests are focused on UHPC, rehabilitation and retrofitting of structures, Large-scale experimental testing.

Abdolreza Joghataie is a faculty member in CivilEngineering Department at Sharif University of Technology.His research interests include structural healthmonitoring and optimization, numerical methods, andartificial neural networks.