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Assessment of statistical variations in impact resistance of high-strength concrete and high-strength steel fiber-reinforced concrete

P.S. Song[a,*], J.C. Wu[b], S. Hwang[c], B.C. Sheu[c]

[a]Department of Civil Engineering, Dahan Institute of Technology, Sincheng, Hualien, Taiwan 971, R.O.C.
[b]Department of Industrial Engineering and Management, Cheng Shiu Institute of Technology, Kaohsiung, Taiwan 833, R.O.C.
[c]Department of System Engineering, Chung Cheng Institute of Technology, National Defense University, Dashi, Taoyuan, Taiwan 335, R.O.C.

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Abstract

The impact resistance variations of high-strength steel fiber-reinforced concrete (HSFRC), versus those of high-strength concrete (HSC), commanded this research. The impact resistance of the high-strength steel fiber-reinforced concrete improved satisfactorily over that of the high-strength concrete; the failure strength improved most, followed by first-crack strength and percentage increase in the number of post-first-crack blows. The two concretes resembled each other on the coefficient of variation values, respectively, on the two strengths, whereas the high-strength concrete was much higher in the value on the percentage increase. The Kolmogorov–Smirnov test indicates that the high-strength concrete was approximately normally distributed in first-crack and failure strengths, high-strength steel fiber-reinforced concrete was poorly normally distributed in the two strengths, and both concretes were hardly normally distributed in the percentage increase. Finally, for both concretes, failure strength regression models were developed, and then, the accompanying 95% prediction intervals for the strength were established.

Keywords: Concrete; Mechanical properties; Fiber reinforcement; Kolmogorov–Smirnov test

1. Introduction

The term “high-strength concrete” (HSC) refers to concrete that has a compressive strength of over 42 MPa (6000 psi). In the construction industry, HSC has been beneficially adopted for precasted and prestressed products, reinforced and prestressed structures, columns and shear walls of high-rise buildings, etc. [1,2]. The adoption of HSC favors the design of concrete components with reduced sections, thus decreasing the dead weight and increasing the capacity of structures. Reduction in mass also plays an important part in the economical design of earthquake-resistant structures [3,4]. The HSC is a brittle construction material, typically characterized by a steep descending stress–strain curve in compression. The strength of the concrete generally goes up with the descending portion of the curve, growing steeper or even disappearing, which indicates that HSC strength increases in exchange for decreasing ductility [5–7]. To improve the ductility, engineers came up with an age-old concept of reinforcing a brittle matrix with discrete fibers. When the resulting

Table 1
Concrete mix proportions

Material	Quantity
Type I Cement (kg/m^3)	475
Coarse aggregate (kg/m^3)	1052
Fine aggregate (kg/m^3)	739
Silica fume (kg/m^3)	25
Water (kg/m^3)	140
Superplasticizer (kg/m^3)	14
Water/(C+S) ratio	0.28
Slump (mm)	9

* Corresponding author. Tel.: +886 3 8263936; fax: +886 3 8263936.
E-mail address: pssong@ms01.dahan.edu.tw (P.S. Song).

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fiber-reinforced concrete develops cracks under service loads, the crack bridging fibers act as a load transfer mechanism. The mechanism carries the service loads, transmitting the loads from one side of the crack to the other, reducing the impetus to crack development, and contributing to ductility and strength improvements [8,9].

Steel fiber-reinforced concrete has gained acceptance for a variety of applications, including industrial floors, hydraulic structures, bridge deck overlays, pavement and overlays, explosive and penetration resistant structures, etc. [10]. The acceptance rests primarily on the impact resistance [11]. The resistance is assessed through different types of test proce-

Table 2
Impact resistance test results and predicted failure strength for HSFRC

Specimen number	Impact resistance test results			Predicted failure strength		
	Number of blows for first-crack strength	Number of blows for failure strength	Percentage increase in number of post-first-crack blows	Number of blows for failure strength	95% Prediction interval on number of blows for failure strength	
					Lower prediction bound	Upper prediction bound
1	1450	1650	13.8	1603	1372	1834
2	1510	1590	5.3	1665	1434	1895
3	2012	2088	3.8	2182	1951	2413
4	2890	3590	24.2	3088	2852	3324
5	1306	1525	16.8	1454	1223	1685
6	1104	1238	12.1	1246	1014	1478
7	1250	1336	6.9	1396	1165	1628
8	2504	2718	8.5	2690	2457	2923
9	1274	1385	8.7	1421	1190	1653
10	2750	2832	3.0	2943	2709	3178
11	1367	1424	4.2	1517	1286	1748
12	1683	1780	5.8	1843	1612	2074
13	1991	2074	4.2	2161	1930	2391
14	1463	1617	10.5	1616	1385	1847
15	1396	1490	6.7	1547	1316	1778
16	1429	1478	3.4	1581	1350	1812
17	1450	1580	9.0	1603	1372	1834
18	1147	1224	6.7	1290	1058	1522
19	1255	1316	4.9	1402	1170	1633
20	1335	1449	8.5	1484	1253	1715
21	2895	2967	2.5	3093	2857	3329
22	1872	2108	12.6	2038	1807	2269
23	1287	1386	7.7	1435	1203	1666
24	1750	1871	6.9	1912	1682	2143
25	1085	1407	29.7	1226	994	1459
26	1783	1897	6.4	1946	1716	2177
27	1802	1987	10.3	1966	1735	2196
28	2112	2433	15.2	2285	2054	2517
29	1262	1501	18.9	1409	1177	1640
30	1258	1292	2.7	1405	1173	1636
31	3739	3900	4.3	3963	3717	4210
32	5408	5609	3.7	5685	5405	5965
33	1038	1223	17.8	1178	945	1410
34	2346	2535	8.1	2527	2295	2759
35	1624	1763	8.6	1782	1552	2013
36	1514	1600	5.7	1669	1438	1899
37	1650	1874	13.6	1809	1578	2040
38	1155	1370	18.6	1298	1067	1530
39	1676	2150	28.3	1836	1605	2066
40	1306	1429	9.4	1454	1223	1685
41	1700	1900	11.8	1861	1630	2091
42	1285	1456	13.3	1433	1201	1664
43	1141	1274	11.7	1284	1052	1516
44	1521	1775	16.7	1676	1445	1907
45	1647	1741	5.7	1806	1575	2036
46	1444	1561	8.1	1596	1366	1827
47	1894	1993	5.2	2061	1830	2291
48	1476	1606	8.8	1629	1399	1860

dures, such as explosive test, drop-weight test, projectile impact test, constant strain rate test, etc. [12]. Among these tests, the drop-weight test is the simplest. This ACI devised test applies repeated blows to a concrete disc and records the number of blows to develop the first visible crack on the top end of the disc and the number to cause the ultimate failure of the disc [13]. Ramakrishnan et al. [14] indicated that hooked-end steel fibers have a good potential for enabling the concretes to withstand more impact loads and that the fibers provide at least a fivefold increase in the impact resistance, compared with straight steel fibers. Ramakrishnan et al. [15] also reported that the steel fibrous concrete is six times better in receiving impact loads than the nonfibrous concrete is. Practically, many variables in concrete encourage the change

Table 3
Impact resistance test results and predicted failure strength for HSC

Specimen number	Impact resistance test results			Predicted failure strength		
	Number of blows for first-crack strength	Number of blows for failure strength	Percentage increase in number of post-first-crack blows	Number of blows for failure strength	95% Prediction interval on number of blows for failure strength	
					Lower prediction bound	Upper prediction bound
1	720	729	1.3	725	716	734
2	264	272	3.0	274	265	283
3	413	425	2.9	421	413	430
4	489	496	1.4	497	488	505
5	173	192	11.0	184	175	193
6	437	447	2.3	445	436	454
7	495	498	0.6	502	494	511
8	658	664	0.9	664	655	673
9	536	542	1.1	543	534	552
10	374	385	2.9	383	374	391
11	149	155	4.0	160	151	169
12	578	583	0.9	585	576	593
13	169	179	5.9	180	171	189
14	638	647	1.4	644	635	653
15	249	256	2.8	259	250	268
16	143	151	5.6	154	145	163
17	303	320	5.6	312	304	321
18	640	648	1.3	646	637	655
19	428	432	0.9	436	427	445
20	380	392	3.2	389	380	397
21	337	347	3.0	346	337	355
22	251	261	4.0	261	252	270
23	104	133	27.9	115	106	124
24	333	347	4.2	342	333	351
25	567	572	0.9	574	565	583
26	181	188	3.9	192	183	200
27	482	490	1.7	490	481	498
28	741	750	1.2	746	737	755
29	900	910	1.1	904	894	913
30	537	543	1.1	544	535	553
31	557	563	1.1	564	555	573
32	567	572	0.9	574	565	583
33	355	360	1.4	364	355	373
34	546	550	0.7	553	544	562
35	574	583	1.6	581	572	589
36	569	574	0.9	576	567	584
37	302	307	1.7	311	303	320
38	450	455	1.1	458	449	467
39	512	517	1.0	519	511	528
40	240	249	3.8	250	241	259
41	372	376	1.1	381	372	389
42	275	280	1.8	285	276	293
43	598	603	0.8	604	596	613
44	397	400	0.8	405	397	414
45	597	601	0.7	603	595	612
46	570	577	1.2	577	568	585
47	864	867	0.3	868	859	877
48	398	407	2.3	406	398	415

Table 4
Statistical evaluation of impact strength for HSFRC and HSC[a]

Property	First-crack strength	Failure strength	PINPB
Mean (x) [blows]	1734/446	1896/454	10/3
Standard deviation (σ) [blows]	770/187	802/185	6/4
Coefficient of variation (σ/x) [%]	44/42	42/41	60/133
Standard error of mean [blows]	111/27	116/27	1/1
95% Confidence interval			
Upper bound [blows]	1958/500	2128/508	12/4
Lower bound [blows]	1511/392	1663/400	8/2

[a] Left number is for HSFRC, right number is for HSC.

in impact resistance, including fiber reinforcement, aggregate type and shape, etc. [16]. Therefore, Nataraja et al. [17] statistically analyzed the impact test results on plain and steel fiber-reinforced concretes, indicating that the impact resistance of the concretes exhibited large coefficient of variation values. However, the impact resistance of high-strength steel fiber-reinforced concrete, HSFRC, is explored rarely in statistics sense.

The impact resistance variability of HSFRC statistically commanded this paper in comparison with that of HSC, thus enabling understanding the impact-resistance improving potential of steel fibers in HSC. Moreover, for both concretes, two reliable regression models were developed to predict the failure strength through the first crack strength.

2. Experimental procedures

Ordinary Portland cement (ASTM Type 1) was used with crushed stone having a maximum particle size of 19 mm and fine sand having a fineness modulus of 3.2. The silica fume, a commercial available byproduct of the silicon and ferrosilicon industries, was employed with a sulphonated naphthalene formaldehyde superplasticizer. The 3.5-mm-long hooked-end steel fibers register an aspect ratio of 40; the fiber volume fraction was 1%. The mixture proportions for the HSC are presented in Table 1. The concrete had an average 28-day compressive strength of 66 MPa; the HSFRC, 76 MPa.

After the HSC ingredients were mixed, the fibers were added, and the ASTM Standard mixing procedure was followed of 3-min mixing, 3-min rest, and 2-min mixing.

The freshly mixed steel fiber-reinforced concrete was placed in molds to cast 150×300 mm cylinders. The molds were filled in two equal layers; each layer was consolidated using a vibrating table. After consolidation, the specimens were kept in the molds for 24 h, then removed from the molds and cured at approximately 25 °C in water until the test age of 28 days.

The HSFRC and its HSC counterpart were, respectively, mixed in one batch. From each batch, twelve 150×300 mm cylinders were cast, and each cylinder was cut using a diamond cutter into four 150-mm-diameter-by-64-mm-thick discs. For each concrete, a total of 48 discs was subject to the drop-weight test following the ACI committee 544 Report "Measurement of Properties of Fiber Reinforced Concrete" [13]. In the test, the number of blows to cause the first visible crack on the disc is recorded as the first-crack strength, and the number of blows to cause ultimate failure of the disc is recorded as the failure strength. The percentage increase in the number of post-first-crack blows, PINPB, judges how strongly an impact-cracked disc played defense against the blows when the first crack gave way to ultimate failure.

3. Results and discussion

The drop-weight test results on 48 HSFRC discs appear in Table 2, those on the 48 HSC discs, in Table 3. In face of the marked variability displayed in both tables, the results were under statistical evaluation, as seen in Table 4.

3.1. First-crack strength

The first-crack strength of HSFRC was approximately 3.9 times that of HSC, following the mean values of the strength in Table 4. This is because the steel fibers provided three-dimensional fibrous reinforcement, which assisted a disc in absorbing the impact energy of repeated blows, thus downplaying the impetus of the disc to cracks and postponing the presence of the first crack. The coefficient of variation values declare that HSFRC presented a narrowly greater dispersion in first-crack strength than HSC did, indicating that the steel fiber addition little widened the first-crack strength variability. For the HSFRC, the 95% confidence interval on the mean ranged from 1511 to 1958 blows, defining a 95% chance that the true first-crack strength was within this range, and for HSC, the true strength was between 392 and 500 blows.

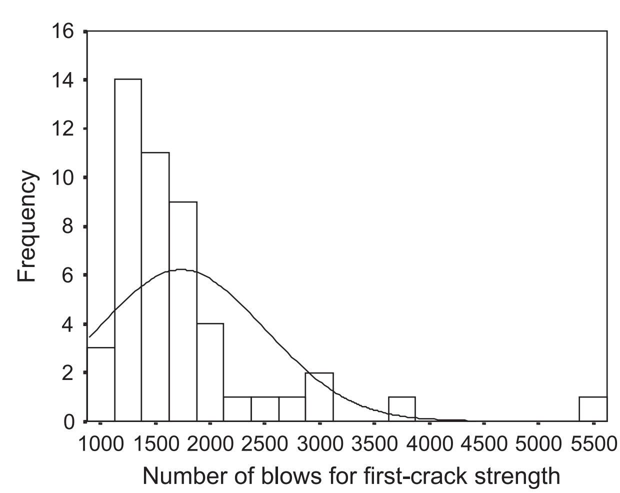

Fig. 1. Frequency histogram and fitted normal curve of the first-crack strength distribution for HSFRC.

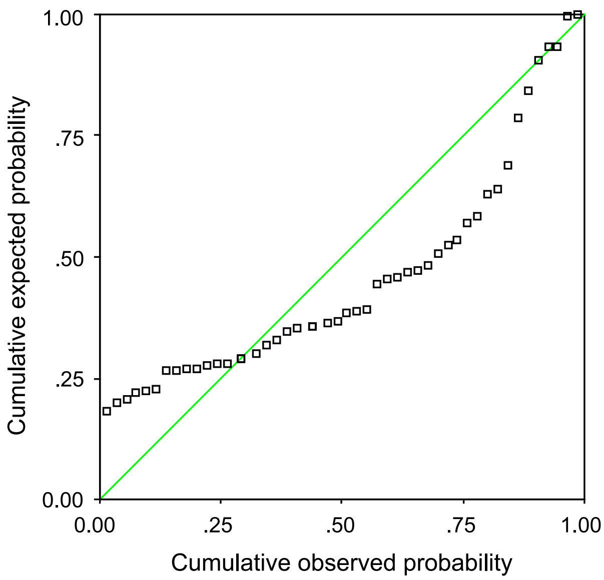

Fig. 2. Normal probability plot for the first-crack strength of HSFRC.

Fig. 1 contains the histogram of first-crack strength for HSFRC, with the fitted normal curve superimposed, suggesting that the first-crack strength distribution was hardly described using the normal distribution. The histogram in Fig. 1 reappears as the normal probability plot in Fig. 2; the plot shows most points falling off the line, seconding the suggestion from Fig. 1. For more accuracy, the first-crack strength distribution was reinvestigated numerically using the Kolmogorov–Smirnov test [18] at a significance level α=0.05. The test gave a p-value next to zero, affirming that the first-crack strength distribution of HSFRC was hardly normal.

For HSC, the first-crack strength histogram is present in Fig. 3 with the fitted normal curve. The curve was seemingly an approximation to the first-crack strength distribution of HSC, supported by the p-value of 0.2, which is much higher than α=0.05.

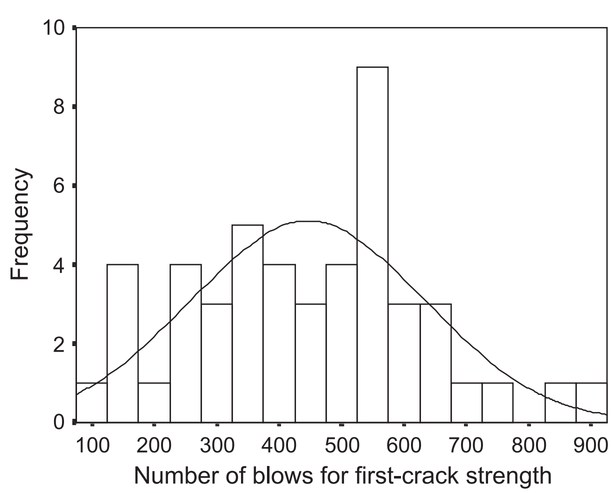

Fig. 3. Frequency histogram and fitted normal curve of the first-crack strength distribution for HSC.

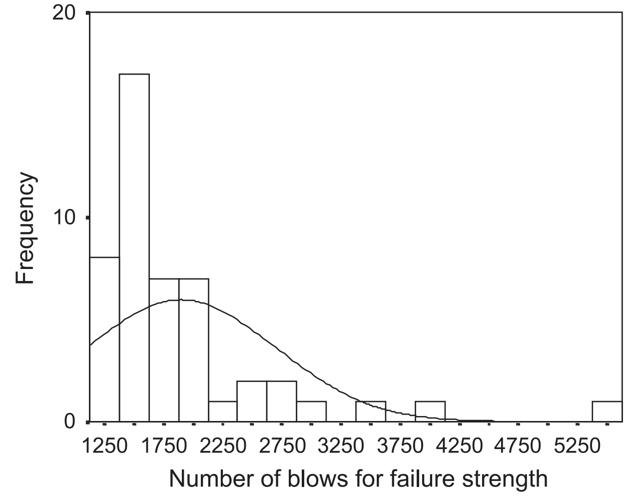

Fig. 4. Frequency histogram and fitted normal curve of the failure strength distribution for HSFRC.

3.2. Failure strength

The failure strength of HSFRC was approximately 4.2 times that of the HSC rival, from the mean values of the strength in Table 4. This came also through the three-dimensional reinforcement, which tied impact-induced cracks together, trapped them in retardation, and therefore defended the disc against the growing trend for ultimate failure. The coefficients of variation indicate that the dispersion of failure strength of HSFRC almost copied that of HSC. The true failure strength of HSFRC ran between 1663 and 2128 blows; that of HSC, 400 and 508 blows.

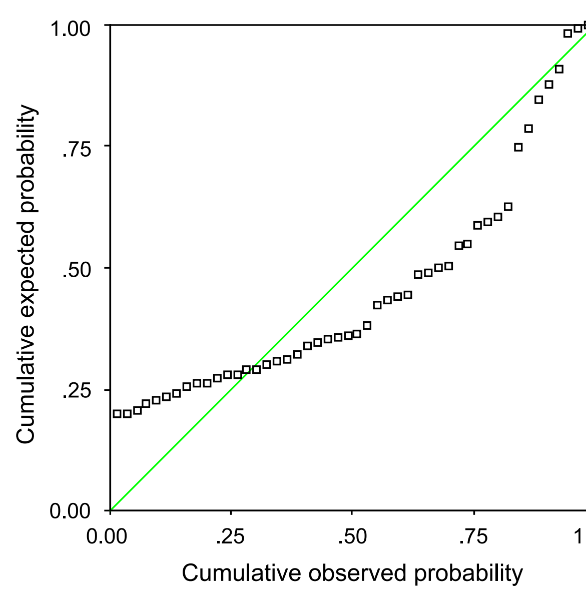

Fig. 5. Normal probability plot for the failure strength of HSFRC.

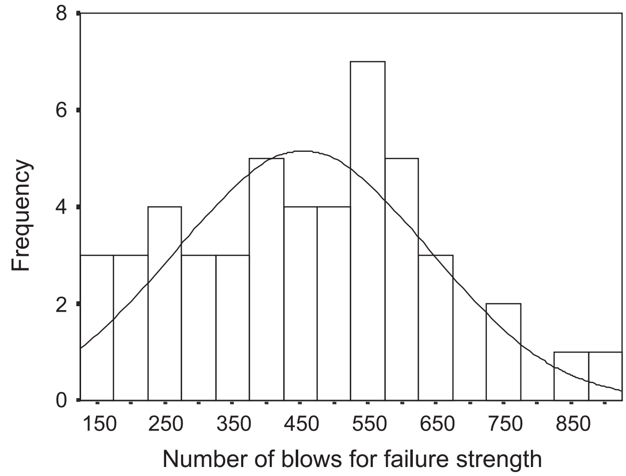

Fig. 6. Frequency histogram and fitted normal curve of the failure strength distribution for HSC.

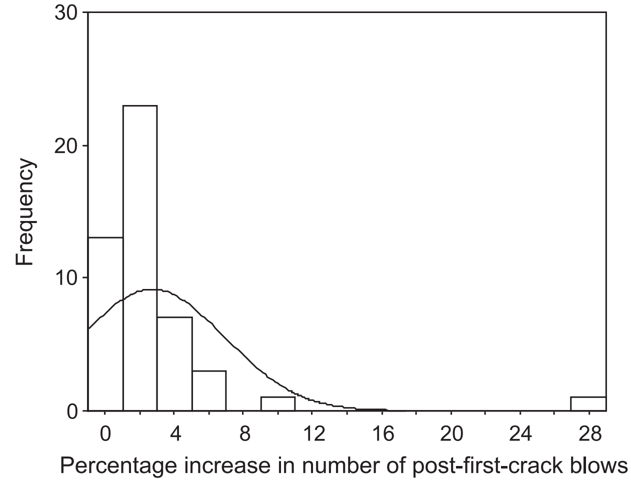

Fig. 8. Frequency histogram and fitted normal curve of the PINPB distribution for HSC.

In Fig. 4, the fitted normal curve impossibly approximated the failure strength distribution of HSFRC. This received support from the normal probability plot in Fig. 5, carrying pronounced nonlinearity, and also from the p-value closing on zero. The distribution of HSC failure strength was approximately normal, as Fig. 6 illustrates; the p-value was 0.2, much higher than α=0.05.

3.3. Percentage increase in the number of post-first-crack blows

From the PINPB comparison in Table 4, the defense played by HSFRC was about 3.3 times stronger compared with HSC, again acknowledging the effectiveness of the three-dimensional reinforcement. The PINPB variability was much milder for HSFRC than for HSC. The true PINPB value of HSFRC was between 8 and 12 blows; that of HSC, 2 and 4 blows.

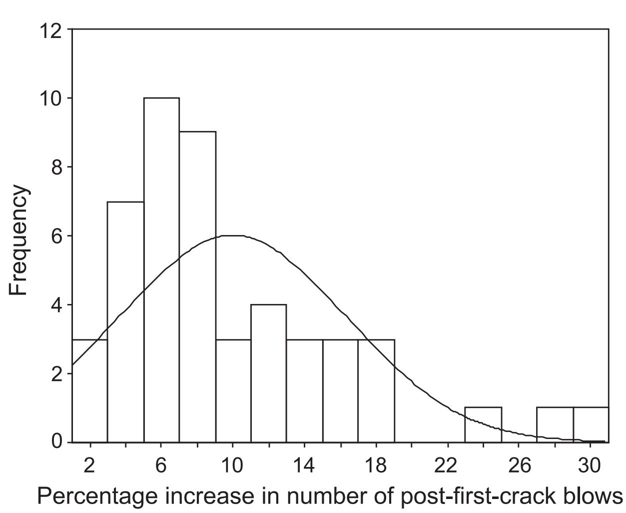

Fig. 7. Frequency histogram and fitted normal curve of the PINPB distribution for HSFRC.

The PINPB distribution of HSFRC was impossibly normal, as seen in Fig. 7, backed by the p-value of 0.02. The impossibility reemerged for the PINPB distribution of HSC, as Fig. 8 shows, with the p-value approaching zero.

3.4. Failure strength predictions

The linear relationship between the two strengths of HSFRC was markedly strong, with the strength correlation coefficient r hitting 0.99. The relationship carried the accompanying equation, through the regression analysis [19], on HSFRC test results.

$$\hat{N}_2 = 107.254 + 1.031N_1 \tag{1}$$

Where $\hat{N}_2$ is the to-be-predicted number of blows to ultimate failure, with N_1 as the actual number of blows to the first crack. For Eq. (1), the coefficient of determination r^2 was 0.980, manifesting that Eq. (1) captured almost 100% of the experimentally observed failure strength variability. The close predictions from Eq. (1) appear in Table 2 with the upper and lower bounds of 95% prediction interval on each prediction.

The failure to first-crack strength relationship for HSC was also remarkably strong, confirmed by the r-value at 0.99. The relationship registered the following equation with the r^2-value up to 0.999.

$$\hat{N}_2 = 12.365 + 0.990N_1 \tag{2}$$

The predictions from Eq. (2) appear in Table 3, accompanied by the two bounds of 95% prediction intervals.

4. Conclusions

1. The impact resistance of HSFRC was superior to that of HSC. The first-crack strength of HSFRC was about 3.9

times that of HSC, with the failure strength about 4.2 times, and PINPB, about 3.3 times.

2. The first-crack strength variability of HSFRC and that of HSC were almost evenly matched; the match reoccurred for the failure strength variability. However, the PINPB variability of HSFRC was much milder versus HSC.
3. For the two concretes, the 95% confidence intervals on the true first-crack strength were identified, which were also identifiable for the failure strength and the percentage increase.
4. The HSFRC hardly followed normal distribution in the first-crack and failure strengths and PINPB; whereas the HSC approximately did in the two strengths and hardly in PINPB.
5. The two regression models enabled point and interval estimates for the number of blows to ultimate failure in the concretes, providing insight into the potential impact resistance of the concretes to ultimate failure.

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