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Evaluation of Shear Strength of HSRC beams without web reinforcement

Ahmed I. Ramadan*, Aly G. Aly Abd-Elshafy

Abstract

Currently, there is no general agreement on a theory describing the response of reinforced concrete members without web reinforcement. Many structural systems rely on design is usually performed using empirical or semi-empirical expressions provided by codes of practice that do not consider the influence of many governing parameters. In this paper, a comparison between values of current experimental shear strength and those of various international design approaches have been calculated and analyzed on 18 simple span HSRC deep beams without web reinforcement were tested under monotonic two point loads at the mid span to examine the contribution of various parameters on the shear capacity of HSRC beams like; f_{cu}=60 MPa, three values of tension reinforcement (0.73%,1.21% &1.83%) and shear span to effective depth ratio (2,1.5 &1) were selected to mainly study the behavior of deep beams, where typical shear failure can be anticipated.

Keywords*: deep* beams, HSC, tension reinforcement ratio, shear span to effective depth ratio and shear strength.

I. Introduction

There is a general agreement among the researchers in the field of Structural Engineering and Concrete Technology that the shear strength of HSRC beams, unlike the Normal Strength Reinforced Concrete (NSRC) does not increase, in the same proportion as the increase in the compressive strength of concrete, due to brittle behavior of the High Strength Concrete. Hence the current empirical equations proposed by most of the building codes for shear strength of HSRC beams are less conservative as compared to the Normal Strength Reinforced Concrete (NSRC) beams. This major observation by the researcher is the main focus of this research. Extensive research work on shear behavior of normal as well as high-strength concrete beams has been carried out all over the world. The major researchers include Ferguson [11],

Taylor [12], Cossio [13], Berg [14], Mathey and Watstein [15], Zsutty [16], Kani [17], Elzanaty [18], Roller and Russel [19], Ahmad and Lue [20], Barrington[21], Shin et al. [22], Kim and White [23], Tompos and Frosh [24], Ahmad [25], Reineck [26], and many more [27-46]. Despite the extensive research work, shear behavior of high-strength reinforced concrete beams is still controversial and needs further research.

*Ahmed I. RAMADAN, Lecturer *(Author)*
Civil Engineering Dept., Faculty of Engineering / Asiut University, EGYPT
Aihr78@aun.edu.eg, aihr48@mun.ca

In this paper, shear span to effective depth ratio, tensile steel ratio and beam size effect were the main variables considered.

A. Shear Span to Effective Depth Ratio (a/d)

Many researchers have shown that failure mode is strongly dependent on the shear span to depth ratios (a/d). Berg [13] finds increase in shear capacity with decrease of a/d ratio. However Ferguson [10] describes this increased resistance to diagonal tension with small a/d, a local loading effect due to direct transfer of load to supports through concrete compression. Taylor [11] found increase in diagonal cracking load with increase in shear span for concrete compressive strength up to 27.59 MPa.

B. Tensile Steel Ratio (ρ%)

The shear strength of a beam increases with increase in longitudinal steel ratio. Barrington [21] confirmed a strong relationship between cracking shear and steel ratio in lightly reinforced beams having steel ratio < 0.015. Berg [14] found a highly significant correlation between the nominal shear strength and the percentage tension reinforcement. Ahmad and Lue [20] carried out a research and found that for very low steel ratios, the relative flexural strength increases as the tensile steel ratio decreases.

C. Beam Size Effect

the basic theory of size effect in the shear failure of reinforced concrete beams was formulated more than two decades ago and experimental evidence has become great, ACI 318-14 Code has not adopted size effect provisions for beams of depths d up to 0.6 m and even 1 m. The ACI-445F database [26] for shear strength of longitudinally reinforced concrete beams with no stirrups (ACI Committee 445), obtained mostly under three or four-point bending (beams under distributed load are excluded), has a bias of two types: 1) crowding of the data in the small size range: 86% of the 398 data points pertain to beam depths less than 0.5 m and 99% to depths less than 1.1 m, whereas only 1% of data pertains to depths from 1.2 to 2 m; and 2) strongly dissimilar distributions, among different size intervals, of the subsidiary influencing parameters, particularly the longitudinal steel ratio, shear span ratio (a/d).

II. Experimental Work

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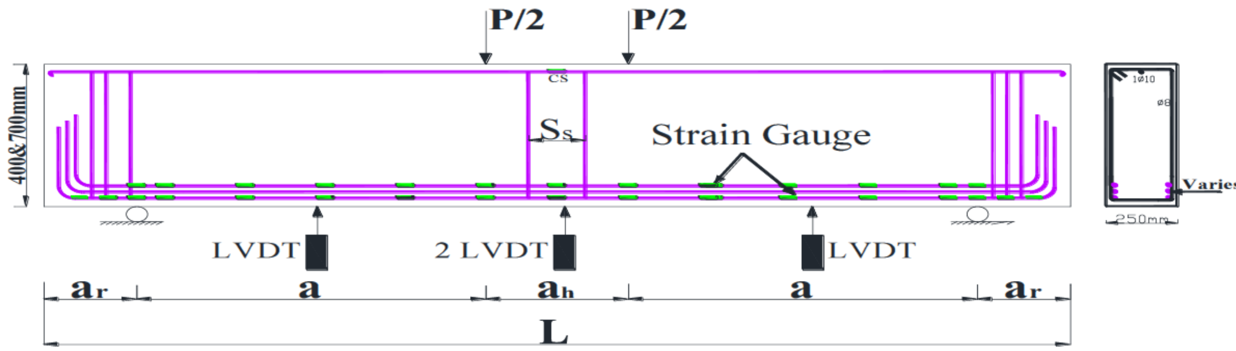
A. Test Specimens

Eighteen high strength reinforced concrete deep beams, two groups; nine deep beams each, without web reinforcement, summarized in **Table 1** and dimension details shown in **Fig. 1**, was



tested. the first group h=700 mm, 3600 mm length, the second group h=400 mm, 3000 mm length, and all groups with three values of tensile reinforcement (0.73%,1.21% &1.83 %) and three values of shear span to effective depth ratio (2,1.5 &1) were selected to mainly study the behavior of short beams, where typical shear failure can be anticipated. These beams were tested under monotonic two point loads at the mid span to examine the contribution of various parameters like longitudinal steel, shear span to depth ratio, and beam span, on the shear capacity of HSRC beams.

Fig. 1 Details of Specimen

B. Materials

The beams are constructed using concrete provided by a local ready-mix supplier. The concrete mix was placed in the forms and vibrated to ensure workability of the concrete. Concrete cylinders 150×300 mm are cast during casting the beams and cured under the same conditions, at room temperature for 28 days, as the tested beams. The concrete strength was monitored by compression testing of the cylinders. The strength of the concrete ranged from 48 MPa to 52 MPa with an average value of 50 MPa at the age of 28 days. Four diameters of high strength deformed bars 10, 12, 14, 18, and 20 mm and of 765, 650, 670, 670, and 670 MPa proof strengths respectively were used for longitudinal reinforcement. 8 mm plain bars were used for transverse reinforcement.

C. Test Procedure

Each specimen was tested as a simply supported beam under four point loading, **Fig. 2**. Two point loads were applied by hydraulic jacks in a load frame. In testing, four LVDT was calibrated, two at middle of span and one at each of middle of shear span. Specimens were loaded at a constant rate and deflection was recorded. The cracks and crack pattern was noted at each increment of load. The test was continued in the same manner until the specimen failed. On the day of testing, all cylinders were also tested in accordance with ASTM C39-86.

Fig. 2 Test setup of Specimens

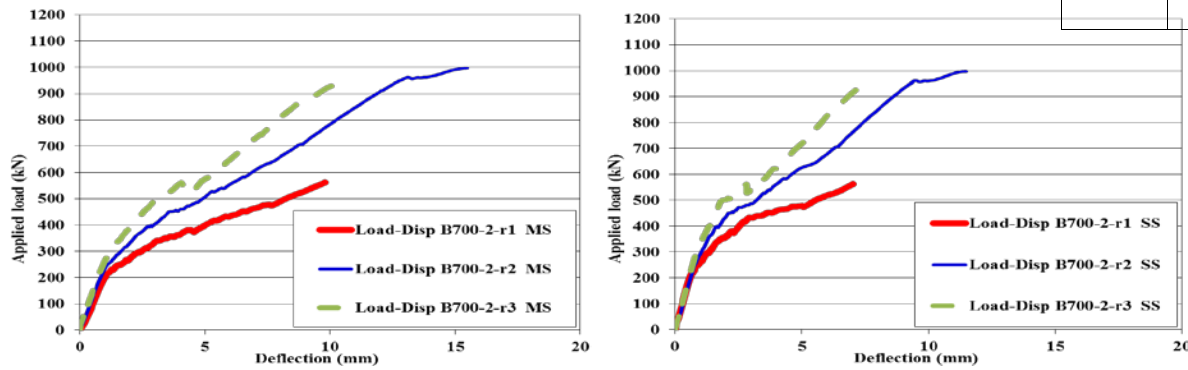
III. Results and Discussions

A. Results

The measured load, deflection, crack development and failure of each of the eighteen tested specimens were recorded. Cracks were marked on each of the beams throughout testing to failure. All the calculations have been done based on the compressive strength of concrete cylinders. Moreover, the shear strength of the concrete beams has been calculated using different design approaches and compared with the experimental results. The tests results for the experimental program are summarized in **Table 2**; numbering and beam designation refer to beam depth, shear span to effective depth ratio, concrete comp. Strength and reinforcement model.

B. Mode of failures

Four failure modes are identified, **Fig. 3**, diagonal splitting (shear) failure, shear-flexure failure, flexure and shear compression failure. The diagonal-splitting failure, characterized as shear failure, is brittle, sudden and hence treacherous. A critical diagonal crack joining the loading point at the top and support point at bottom is developed. In the shear-compression mode of failure, a/d =1.5, after the appearance of the inclined crack, the concrete portion between the top load point experiences high compression and it then finally fails. This mode of failure is equally a brittle mode of failure. The shear-flexure mode of failure is the



combined failure in shear and flexure. Flexural cracks are formed followed by the partly diagonal crack. This is ductile mode of failure in which the beam deflects at the center and no explosive sound was heard at the time of failure.

C. Load-Deflection Curves

The load-deflection responses of all the beams appear to be non-linear. The deflection increases at beginning linearly then trend be non-linear with loading. Some of the load deflection curves have been given in **Figs.4 & 5 –a) ,b)** show the mid-span & mid-shear span deflections against the applied loads for beams having varying steel ratio of ρ_s % and constant a/d ratios. The load-deflection curves for beams with a/d = 1 are steeper than those with a/d of 1.5 and 2. The deflections at ultimate loads of beams with a/d of 1.5 and 2 are greater than those when a/d = 1. Thus stiffness, as represented by the load deflection curves, reduces as a/d increases. The ultimate load decreased as the a/d increased. This is due to the strut and tie action (tied-arch action) effect which becomes greater as the a/d gets smaller.

Fig. 3 Failure modes of some tested beams

a) at mid-span b) at mid-shear span

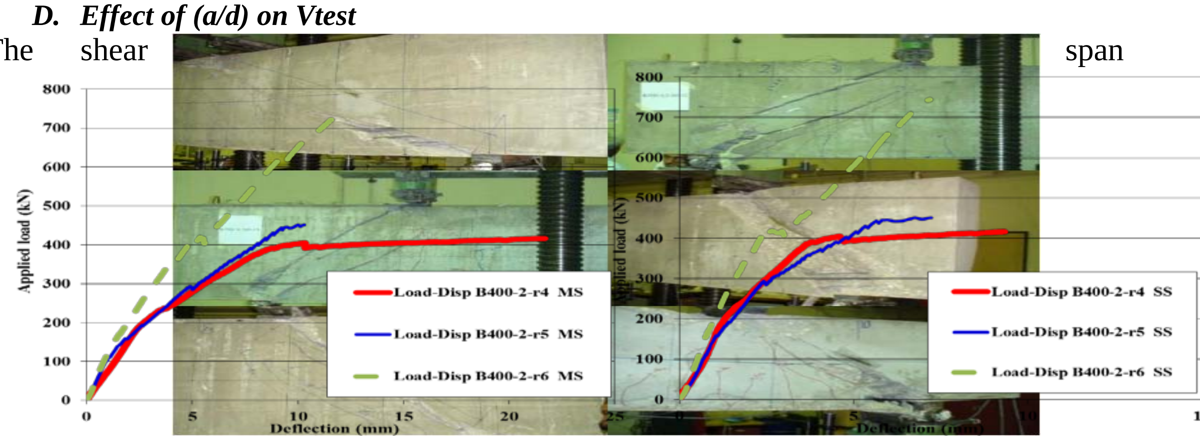
Fig.4 Load-Defl. relationship for Beams 1,2 & 3

a) at mid-span b) at mid-shear span

Fig.5 Load-Defl. relationship for Beams 10,11 & 12

D. Effect of (a/d) on Vtest

The shear span to



depth a/d ratio has a strong influence on the shear strength of HSRC beams like NSRC beams. The shear strength decreases with the increase of a/d values for the same longitudinal steel. The increase in shear span increases the number of cracks formed and as result more cantilever force applied at the cracked concrete, reducing the shear strength of concrete to greater extent. The effect of a/d values on the shear strength of HSRC beams has been shown in **Fig.6 a), b)**.

a) B1-B9 **b) B10-B9**

Fig. 6 (V_{test}-a/d)

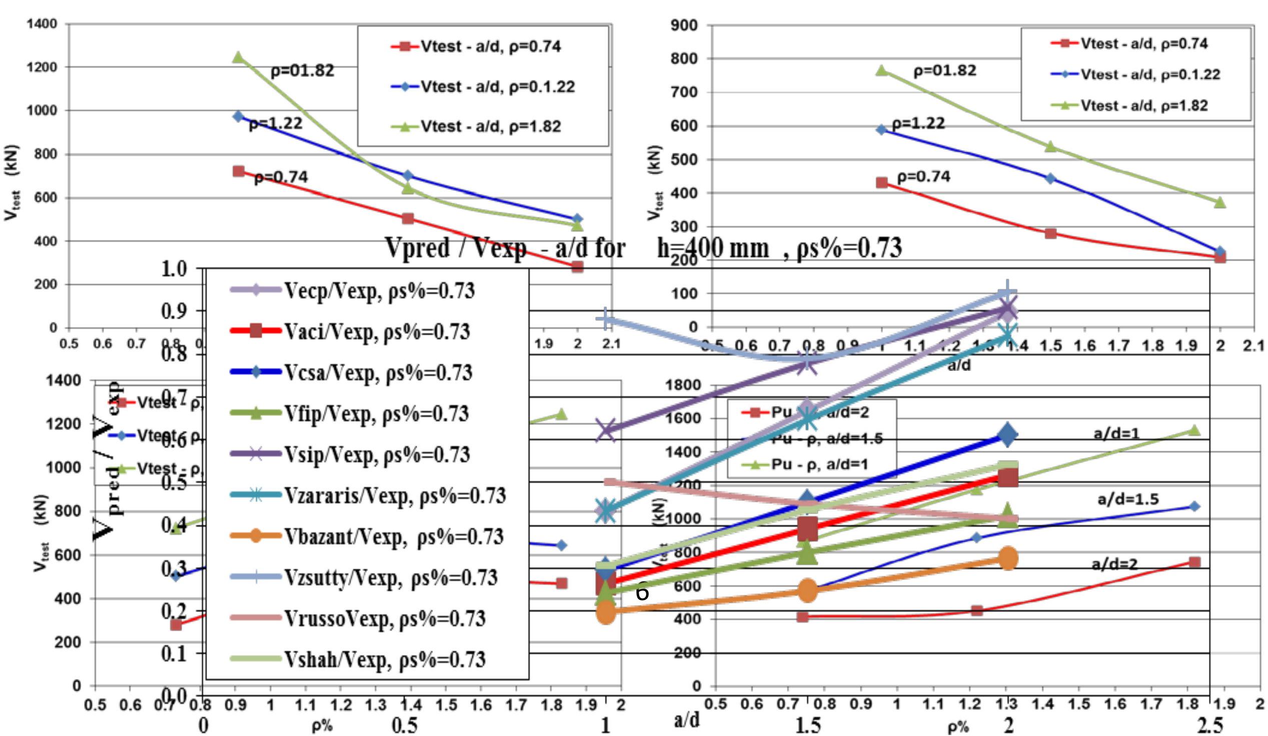
E. Effect of (ρ%) on V_{test}

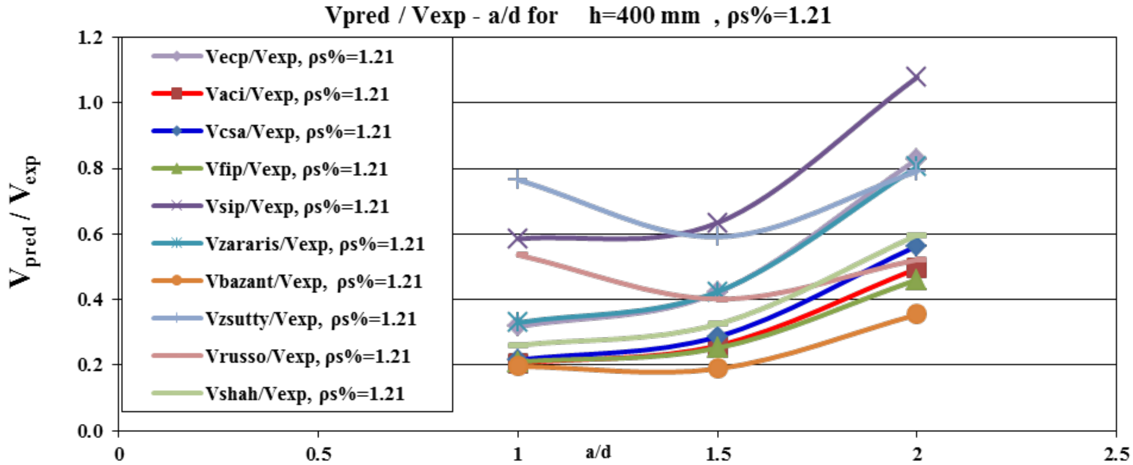
The tests have demonstrated that the beams reinforced with higher ρ_s % exhibited fewer strains in the longitudinal steel than those reinforced with lower ρ_s % due to increases in the ultimate shear capacity and reduces the deflection. An increase was recorded in values of $\mathbf{V_{test}/V_{Code}}$ as the steel percentage was increased, Table 3&4. The increase is mainly due to the dowel action which improves with the amount of longitudinal steel crossing the cracks. Hence, it may be noted that the tensile reinforcement significantly affects the deflection of a beam, thus this is the most important parameter in controlling deflections of HSC beams as be shown in **Fig. 7(a),(b)**. The shear carrying capacity of HSC beams was observed to decrease at a greater rate with the increase in a/d ratio, and thereafter a gradual decrease was noted. **Fig. 8 (a) to (f)** shows the variation in $\mathbf{V_{exp}/V_{code}}$ with a/d ratio for different tensile steel ratios.

a) B1-B9 **b) B10-B9**

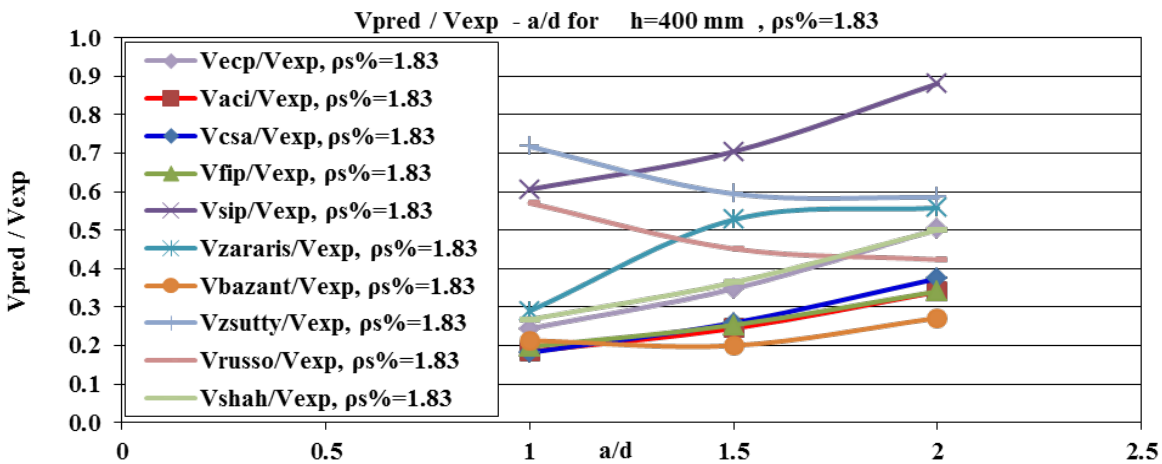
Fig. 7 (V_{test}-a/d)

a) h=400mm,ρs%=0.73

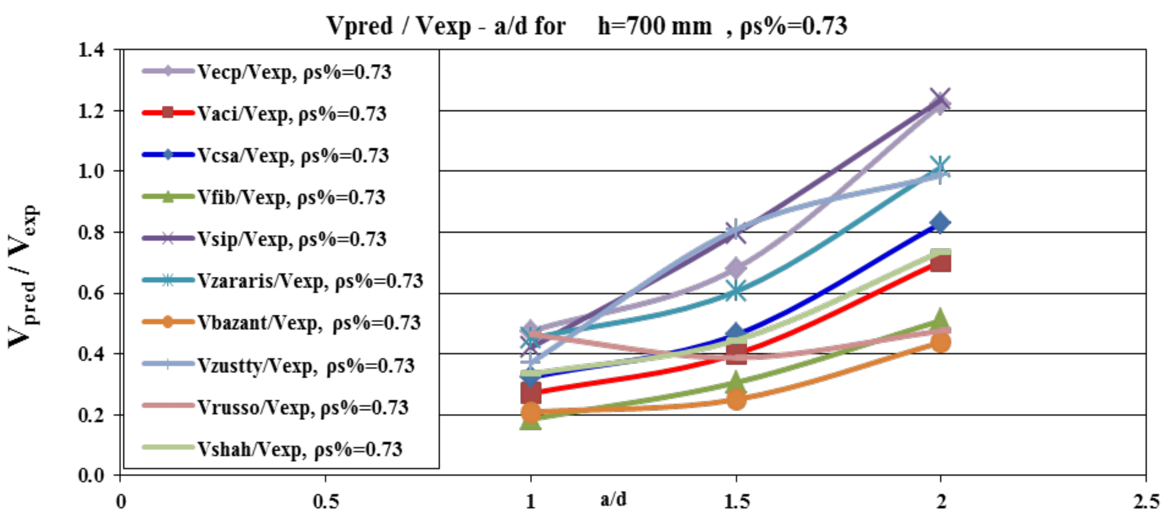




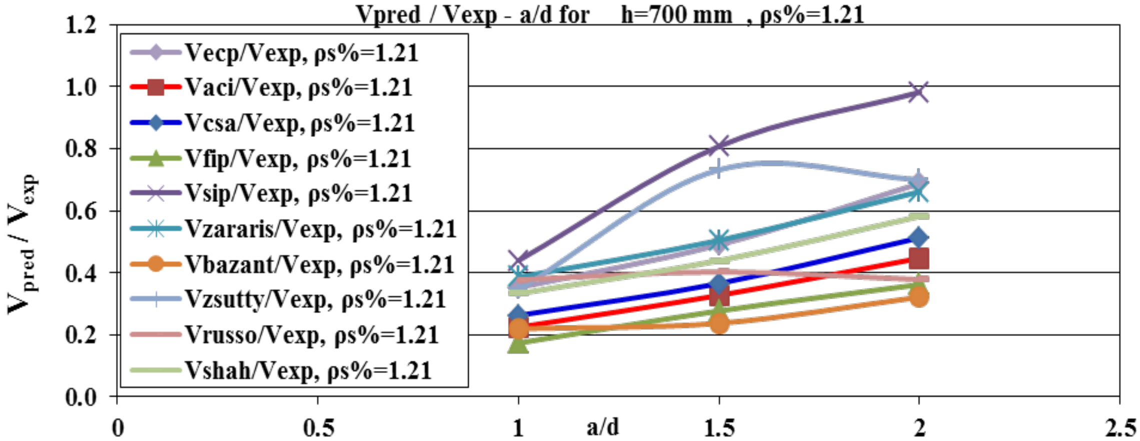
b) h=400mm,ρs%=1.21



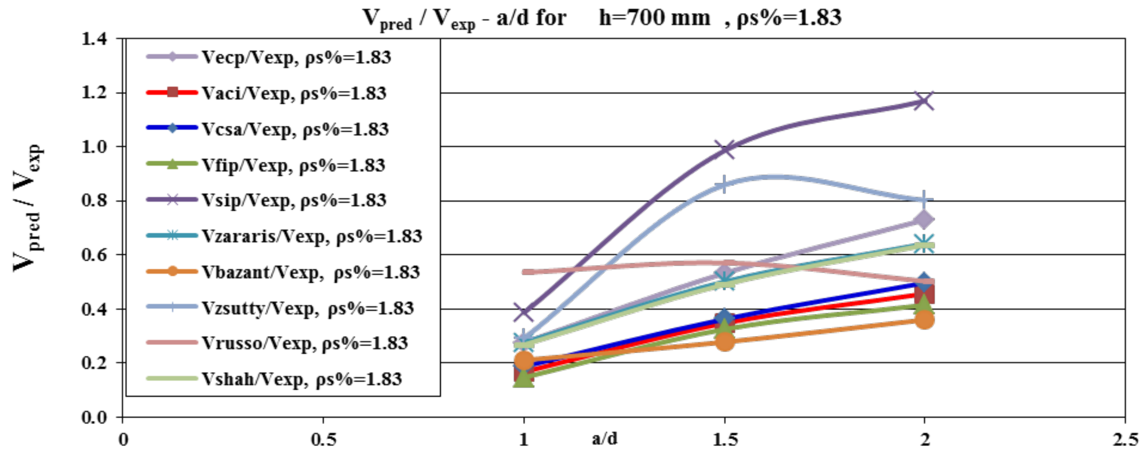
c) h=400mm,ρs%=1.83



d) h=700mm,ρs%=0.73



e) h=700mm,ρs%=1.21



f) h=700mm,ρs%=1.83

Fig. 8 (V_{pred}/V_{test} -a/d)

Comparison of the experimental results with ACI, Canadian, FIP, and the equations proposed by SIP, Zararis, Bazant, Zsutty, Russo, and Shah show that the a/d ratio significantly effects the shear carrying capacity and mode of failure of the tested beams. The shear strength of the beams decreases on increasing the shear span to depth ratio (a/d), where shear strength increased as compared to the various design approaches and brittle failure of the beams was observed.

It can be observed that the average values of Vexp/Vcode increases steadily with increasing in longitudinal reinforcement ratio, which shows that, there is a pronounced effect of tensile steel on the ultimate load and shear capacity of members without shear reinforcement. For a constant value of a/d ratio, the relative flexural strength decreases and failure load increases with an increase in longitudinal reinforcement ratio therefore, quantitative effect of tensile steel was observed on shear capacity of reinforced concrete beams.

ACI 318-14 shows underestimate on shear capacity of a beam without web reinforcement, where experimental results show that the tensile steel has significant effect on shear carrying capacity. Also, it can be observed that the current ACI shear provision is unconservative for HSC beams without web reinforcement with lower values of longitudinal reinforcement ratios. It can be observed that Canadian and FIP codes also underestimate the shear strength of reinforced concrete beams for lower a/d ratios up to 2, and thereafter overestimate.

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IV. Conclusion

In this study eighteen HSRC deep beams were tested to evaluate the contributions of a/d and ρs% on the global behavior in shear. Based on the experimental results obtained, the following conclusions are drawn:

1) HSRC deep beams without stirrups exhibit a brittle behavior;

2) The mode of failure was significantly altered by changing the beam depth. Sufficient ductility was achieved in small size beams, whereas relatively very high brittleness was observed in large size beams.

3) The failure in most of the beams has been caused due to diagonal tension cracking; however it was more dominant failure mode for beams without web reinforcement and having ρ=1.21&1.83%. For beams with ρ=0.73%, flexural shear failure was obvious failure mode.

4) For beams have large values of longitudinal steel, the shear failure is more brittle and sudden, giving no sufficient warning.

5) An increase in longitudinal steel ratio increases the ultimate shear capacity and reduces the deflection at mid-span; an increase of 73% was recorded between beam B700-1-50-r1 and beam B700-1-50-r3 where the steel percentage increased from 0.73 to 1.83%;

6) Ultimate load decreases as a/d increases. In the same manner, mid-span deflections at ultimate load increase as the values of a/d increase; flexural behavior is more associated with a beam action as a/d increases.

7) The three major code provisions for shear in HSC are safe for use with the exception that CSA should be used with care, it might have a tight safety margin against brittle shear failures;

8) The different design equations considered in this study do not accurately reflect the increase in shear capacity of beams with shorter shear spans (a/d = 1.5). Most of the

design models are excessively conservative, and the code predictions only seem to be more accurate as a/d increases beyond a value of 2.0.

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NOTATIONS

a = Shear span, distance between concentrated load and face of support, in mm
$\mathbf{a_h}$ = distance between two concentrated loads , in mm
$\mathbf{a_r}$ = distance between end of beam and face of support, in mm
b = Beam width, in mm
d = Effective beam depth, in mm
$\mathbf{d_o}$ = maximum aggregate size, in mm
$\mathbf{f'_c}$ = Cylindrical compressive strength of concrete, in MPa
$\boldsymbol{\rho_s}$ = Ratio of Longitudinal reinforcement ratio; = As /bd
$\mathbf{S_s}$ = distance between two stirrups under concentrated loads, in mm

TABLE 1. Specimen Details

No	*Beam Designation*	*h mm*	*a mm*	*d*	*a/d*	a_h	a_r	S_s	*r*	*L mm*	ρ_s *(%)*	f'_{cu} *KN*
1	B700-2-50-r1	700	1224	660	2	500	326	200	1	3600	0.73	51.8
2	B700-2-50-r2	700	1224	660	2	500	326	200	2	3600	1.21	51.8
3	B700-2-50-r3	700	1224	660	2	500	326	200	3	3600	1.83	51.8
4	B700-1.5-50-r1	700	918	660	1.5	1100	332	800	1	3600	0.73	51.8
5	B700-1.5-50-r2	700	918	660	1.5	1100	332	800	2	3600	1.21	51.8
6	B700-1.5-50-r3	700	918	660	1.5	1100	332	800	3	3600	1.83	51.8
7	B700-1-50-r1	700	1600	660	1	674	326	1000	1	3600	0.73	51.8
8	B700-1-50-r2	700	1600	660	1	674	326	1000	2	3600	1.21	51.8
9	B700-1-50-r3	700	1600	660	1	674	326	1000	3	3600	1.83	51.8
10	B400-2-50-r1	400	670	360	2	1000	330	800	1	3000	0.73	48.35
11	B400-2-50-r2	400	670	360	2	1000	330	800	2	3000	1.21	48.35
12	B400-2-50-r3	400	670	360	2	1000	330	800	3	3000	1.83	48.35
13	B400-1.5-50-r1	400	502.5	360	1.5	1300	348	1000	1	3000	0.73	48.35
14	B400-1.5-50-r2	400	502.5	360	1.5	1300	348	1000	2	3000	1.21	48.35
15	B400-1.5-50-r3	400	502.5	360	1.5	1300	348	1000	3	3000	1.83	48.35
16	B400-1-50-r1	400	335	360	1	1600	365	1000	1	3000	0.73	48.35
17	B400-1-50-r2	400	335	360	1	1600	365	1000	2	3000	1.21	48.35
18	B400-1-50-r3	400	335	360	1	1600	365	1000	3	3000	1.83	48.35

TABLE 2. COMPARISON OF VTEST RESULTS WITH PROPOSED EQUATION AND SHEAR DESIGN EQUATIONS

Beam No.	*Test Beam Designation*	*Shear Strength (KN)*									
		V_{ACI}	V_{CSA}	V_{FIB}	V_{SIP}	$V_{Zararis}$	V_{Bazant}	V_{Zsutty}	V_{Russo}	V_{Shah}	V_{Test}
1	B700-2-50-r1	197.7	233.3	143.4	347.2	284.9	123.1	277.6	133.8	206.9	281.0
2	B700-2-50-r2	205.0	233.3	169.7	447.0	295.3	146.4	328.5	181.9	247.1	498.5
3	B700-2-50-r3	214.4	233.3	194.7	549.8	301.3	169.0	377.1	235.9	298.9	469.9
4	B700-1.5-50-r1	201.4	233.3	154.1	400.9	305.0	125.9	407.3	195.1	224.1	503.5
5	B700-1.5-50-r2	211.1	233.3	182.3	516.2	316.1	151.9	482.1	275.2	264.2	699.9
6	B700-1.5-50-r3	223.6	233.3	209.2	634.8	322.5	178.7	553.3	367.5	316.1	644.1
7	B700-1-50-r1	195.1	233.3	134.1	303.7	325.1	150.4	267.5	334.5	241.2	721.8
8	B700-1-50-r2	200.7	233.3	158.7	391.0	337.0	200.0	316.6	367.7	281.4	974.5
9	B700-1-50-r3	207.9	233.3	182.1	480.8	343.8	262.1	363.4	666.4	333.3	1246.2
10	B400-2-50-r1	107.8	127.3	88.0	189.1	175.3	67.3	196.7	86.6	112.9	208.1
11	B400-2-50-r2	111.8	127.3	104.1	243.4	181.7	80.0	178.5	117.7	134.8	225.7
12	B400-2-50-r3	116.9	127.3	119.5	299.3	185.4	92.6	204.9	152.7	163.1	372.3
13	B400-1.5-50-r1	109.8	127.3	94.5	218.3	181.3	69.1	221.4	126.3	122.2	280.4
14	B400-1.5-50-r2	115.1	127.3	111.8	281.1	187.9	83.7	262.0	178.1	144.1	443.0
15	B400-1.5-50-r3	121.9	127.3	128.4	345.6	1427.7	99.0	300.8	237.9	172.4	537.7
16	B400-1-50-r1	113.8	127.3	104.6	267.4	187.3	85.5	380.2	216.5	131.6	431.4
17	B400-1-50-r2	121.7	127.3	123.8	344.2	194.1	115.8	449.9	315.4	153.5	588.0
18	B400-1-50-r3	131.9	127.3	142.1	423.3	198.0	154.5	516.4	431.3	181.8	765.2