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Effect of anchorage and strength of stirrups on shear behavior of high-strength concrete beams

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Abstract. This study investigated possible ways to replace conventional stirrups used on high-strength concrete members with improved reinforcing materials. Headed bar and high-strength steel were chosen to substitute for conventional stirrups, and an experimental comparison between the shear behavior of high-strength concrete large beams reinforced with conventional stirrups and the chosen stirrup substitutes was made. Test results indicated that the headed bar and the high-strength steel led to a significant reserve of shear strength and a good redistribution of shear between stirrups after shear cracking. This is due to the headed bar providing excellent end anchorage and the high-strength steel successfully resisting higher and sudden shear transmission from the concrete to the shear reinforcement. Experimental results presented in this paper were also compared with various prediction models for shear strength of concrete members.

Keywords: shear; high-strength concrete; headed bar; high-strength steel; anchorage

1. Introduction

There are many variables that affect the shear behavior of concrete beams, including span to depth ratio, concrete strength, amount of longitudinal and transverse reinforcement, effective depth, use of fibre-reinforced concrete (FRC), and so on. As the longitudinal reinforcement ratio decreases and the depth of the beam increases, the concrete shear strength at failure decreases. Extensive experimental research investigating the effect of beam size and longitudinal reinforcement ratio on concrete shear strength concluded that the current ACI simplified shear equation was not conservative for large, lightly reinforced concrete beams (Collins *et al*. 1993, Collins and Kuchma 1999, Angelakos *et al*. 2001, Tompos and Frosch 2002). This was because the current ACI shear provisions were developed based on test results performed around 50 years ago using beams of relatively small beam size and higher longitudinal reinforcement ratio. The ACI Committee 445 on shear and torsion (1998) also suggested that the main reason for this size effect was the large width of diagonal cracks in larger beams. The ACI Code (2008) does not account for the size effect and flexural reinforcement ratio, whereas the CSA Standard (2004) and EC2 Code (2003) include

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factors for the influence of the size effect on shear resistance provided by concrete. The EC2 Code (2003) also account for the flexural reinforcement ratio as a factor affecting shear strength carried by concrete for members not requiring shear reinforcement.

It is obvious the shear stress at shear failure increases with increasing concrete strength. The expressions of the ACI 318-08 (2008) and CSA A23.3-04 (2004) code for concrete shear strength are functions of the square root of the concrete compressive strength, whereas the expression for the shear strength provided by concrete of Eurocode 2 (2003) is a function of the cube root of the concrete compressive strength. Even though a higher cracking shear is expected for high-strength concrete beams which are critical in shear, the rate of increase for the concrete shear strength decreases and the brittleness increases as the concrete compressive strength increases. Sufficient shear reinforcement must therefore be employed effectively to reserve shear strength after diagonal cracking of reinforced concrete beams. In a number of experimental studies investigating the brittleness of high-strength concrete beams, it has been proposed that increased minimum shear reinforcement requirements must be put in place to prevent sudden shear failure of high-strength concrete beams (Johnson and Ramirez 1989, Roller and Russell 1990, Yoon *et al*. 1996), and the ACI Code (2008), CSA Standard (2004) and EC2 Code (2003) provide a procedure for calculating the minimum amount of shear reinforcement taking into account the concrete compressive strength.

Recently, as concrete structures are becoming larger, higher, longer, and more specialized due to the increase in the use of high-strength concrete, a concern has arisen about how to compensate for the brittleness of high-strength concrete. As mentioned above, the first action to reduce this concern is to provide sufficient reinforcements. In addition, the performance of reinforcing materials used in high-strength concrete members should be improved. This study investigates possible ways to replace conventional stirrups with improved reinforcing materials for high-strength concrete members. Headed bar and high-strength steel were chosen to substitute for conventional stirrups, and an experimental comparison was made of the shear behavior of high-strength concrete large beams reinforced with conventional stirrups and those substituting stirrups. The test results were also compared with various equations for shear resistance.

2. Headed bar and high-strength steel

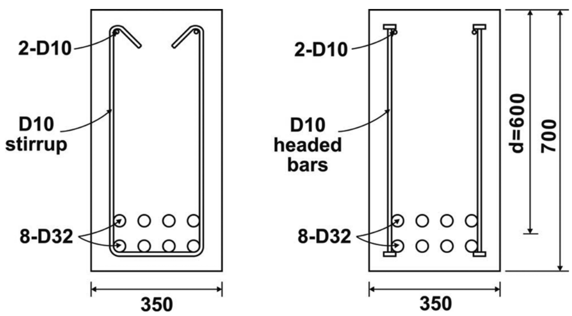
Comprehensive research programs applying headed bars in slabs and footings, beams with thin webs, crossties in columns and walls, precast beams, deep beams and pile caps have substantiated that headed bars conforming to ASTM A970-07 (2007) have many advantages. These include simpler installation, less congestion of reinforcement, more effective anchorage, improved seismic response, and enhanced shear resistance (Ghali and Youakim 2005). Ghali and Dilger (1998) pointed out that conventional stirrups cannot develop the full yield strength at the leg near the hook and bend, while headed bars with relatively large anchor heads can develop the full yield strength at the stem near the anchor head. The high compressive stress, developed on the concrete inside the hook, causes crushing of the concrete and slippage of the hook before the tensile stress in the leg reaches its yield strength. Gayed and Ghali (2004) tested concrete I-beams reinforced with double-headed studs as shear reinforcement, and stated that the head studs provided adequate strength and ductility at ultimate shear. In ACI 318-08 (2008), headed shear reinforcing bars are permitted for punching shear applications.

There are several practical advantages to using high-strength steel, including a reduction of congestion in heavily reinforced members, savings in cost of labor, and a reduction of construction time (Mast *et al*. 2008, Sumpter *et al*. 2009, Yang *et al*. 2010). Griezic *et al*. (1994) demonstrated through experimental testing of full-scale beams with high-strength deformed welded wire fabric stirrups that deformed welded wire fabric stirrups that have high strength exhibited large enough strains and sufficient ductility to redistribute the stresses in the stirrups to avoid sudden, brittle shear failure. Sumpter *et al*. (2009) concluded that direct replacement of conventional stirrups with high-strength steel stirrups increased the shear load capacity of flexural members and enhanced the serviceability. However, there has been some reluctance to use high-strength steel because of concerns about possible excessive cracking and deflection that might not meet serviceability requirements. Maximum design yield strength for shear reinforcement given in the ACI Code (2008) is 420 MPa, except for welded deformed wire reinforcements which have a maximum of 550 MPa.

3. Test program

3.1 Details of test specimens

Three full-scale beam specimens constructed with high-strength concrete were tested. Fig. 1 shows the details of the 350 mm wide × 700 mm deep specimens. The effective depth of all specimens was 600 mm. Each specimen had different shear reinforcements: normal-strength steel, headed bar, and high-strength steel. Specimen SN was reinforced with normal-strength stirrups, which have 135° standard hooks, as shown in Fig. 1(a). In order to observe the differences between the behavior of the specimens with normal-strength stirrups and high-strength stirrups, SD350 steel, which has a minimum yield strength of 350 MPa, was used in specimen SN as the shear reinforcement. Specimen SNH was reinforced with headed bars with a 31.0 mm head diameter and an 11.0 mm head thickness. The shape and details of the headed bars used in this study are shown in Fig. 2 and Table 1, respectively. The headed bars were made of straight bars, cut from the same stock used for fabricating the stirrups of specimen SN, with circular heads welded to each end. Fig. 1(b) shows the section details of specimen SNH. Specimen SH was constructed with high-strength stirrups, which had the same cross-sectional area and hook details as the stirrups of specimen SN. SD500 steel, which has minimum yield strength of 500 MPa, was used for the high-strength stirrups in specimen SH. In the case of specimen SH, steel fibers were added in the percentage of $v_f = 1\%$ by volume to



(a) Specimen SN and SH (b) Specimen SNH

Fig. 1 Section details (dimensions in mm)

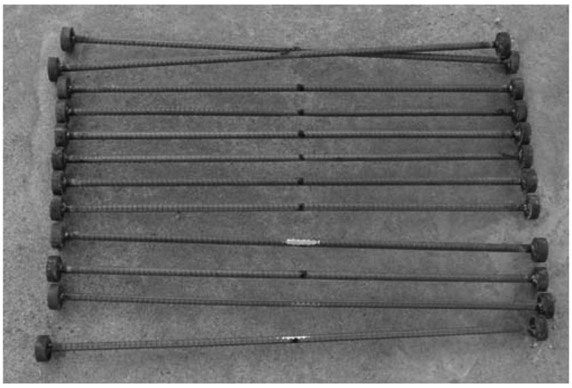
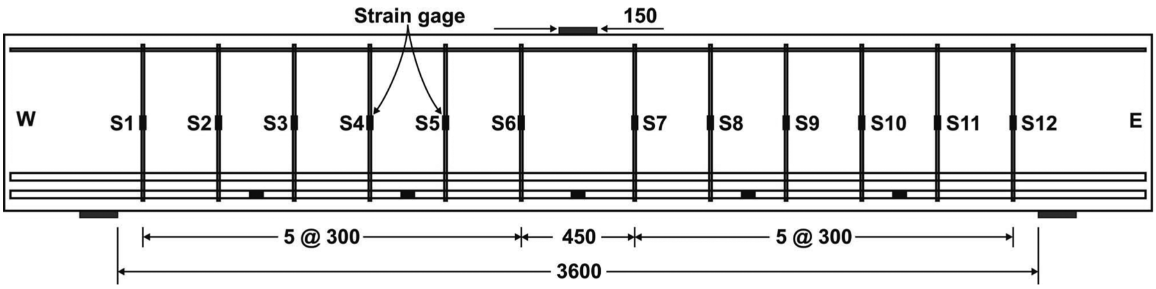


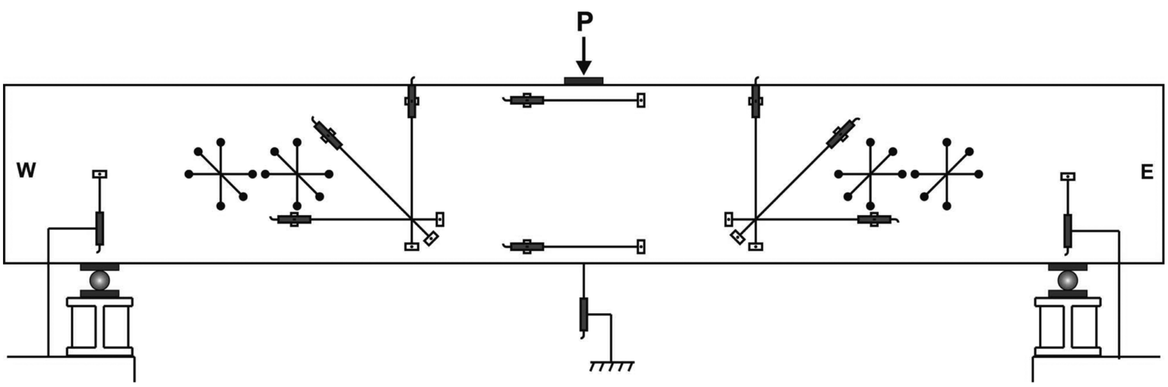
Fig. 2 Photo of headed bars

Table 1 Details of headed bar

Bar diameter (mm)	Head diameter (mm)	Head thickness (mm)	Head area (mm^2)
9.5	31.0	11.0	754.4



(a) Reinforcement details and the locations of strain gages



(b) Test setup and the locations of LVDTs and strain targets

Fig. 3 Details of beam specimens and instrumentation (dimensions in mm) one half of the beam (*W* direction) in order to investigate the effect of steel fibers on the shear behavior of high-strength concrete. The intension was to perform two tests with one specimen. However, after the half (*E* direction) without steel fibers failed, the second phase of the test was not performed due to harsh damage and unskilled repair of the failed half of the beam.

As shown in Fig. 3(a), the overall length of all specimens was 4,550 mm and the distance from the edge of the loading plates to the edge of the supports was 1,725 mm, resulting in clear shear span-to-depth ratios (a/d) of 2.875. The flexural tension reinforcement for all specimens consisted of 8-D32 bars in two layers, giving a longitudinal reinforcement ratio of 3%. This amount of flexural reinforcement was chosen to insure that shear failure would occur before flexural failure. A spacing of 300 mm for shear reinforcements was chosen for all specimens by considering the spacing limits requirement ($d/2$) for shear reinforcement in the ACI Code (2008). The requirement for the minimum amount of shear reinforcement was not considered in order to provide critical conditions to the stirrups with the given spacing.

3.2 Test setup and instrumentation

All test specimens were tested using one point concentrated loading of a 2,000 kN universal testing machine, as shown in Fig. 2. A 150 × 25 mm and two 200 × 25 mm steel bearing plates were placed at the location of the loading and supports, respectively, to prevent local crushing of the concrete. The loading was applied monotonically in small increments, while the loads, deflections and strains were recorded at each increment. At each load stage, the crack pattern and crack widths were also recorded. The width of diagonal shear cracks was measured using a crack width comparator. As shown in Fig. 3(b), the midspan deflection and support settlements were measured with linear voltage differential transformers (LVDTs). Strain rosettes using LVDTs and strain targets were also attached to the side-face of the beam to determine the principal strains. Electrical resistance strain gages were glued to the centre of all stirrups and to the two longitudinal bars in the bottom layer.

3.3 Material properties

The beams were constructed using normal-weight ready-mixed concrete with design strength of 90 MPa, supplied by a local concrete supplier. The beam specimens were covered with moist burlap and plastic, and forms were removed at 5 days after casting. The specimens were cured in an outdoor area and water was provided for 14 days after casting. All beams were tested for 2 days (28~29 days after casting) and tests for mechanical properties of concrete were conducted at 28 days after casting. Standard compressive cylinder tests and split-cylinder tests, using 100 diameter by 200 mm long cylinders, and four point loading tests, using 150 × 150 × 550 mm flexural beams, were conducted.

Table 2 Steel properties

Designation	Area (mm^2)	f_y (MPa)	ε_y (%)	f_u (MPa)	ε_u (%)	Function
SD350 - D10	71	375	0.19	525	4.02	Stirrups for SN and SNH
SD400 - D32	794	428	0.22	735	4.13	Longitudinal reinforcements
SD500 - D10	71	569	0.29	754	4.25	Stirrups for SH

The mean values of the concrete compressive strength, f_c', splitting tensile strength, f_{sp}, and modulus of rupture, f_r, were 97.0, 5.8 and 7.7 MPa, respectively. Table 2 gives the mechanical properties of the steel reinforcing bars used in the construction of the test specimens. These average values were determined by testing three random samples for each bar size of each specimen.

4. Test results and discussion

4.1 Behavior of test specimens

In all specimens, flexural cracking began at approximately the same applied load. As the load was increased further, new flexural cracks formed in the shear span. These flexural cracks increased in depth and began to incline towards the applied load, becoming flexural shear cracks. Table 3 summarizes the measured shears at the formation of the primary shear crack, V_{cr}, and the ultimate shear, V_u. Fig. 4 shows the shear versus midspan deflection relationships for all specimens. Most of shear stress is resisted by concrete at first. After the formation of primary shear crack, most of shear stress is resisted by stirrups. The formation of primary shear crack can be regarded as a shear failure point for concrete beams without stirrups. The formation of the primary shear crack was observed in the readings taken from the strain gages on the stirrups as well as sudden increase in midspan deflections in the shear-deflection curve of Fig. 4.

In specimen SN, in which normal-strength steel is used as shear reinforcement, the primary shear crack occurred at a shear of 286.2 kN. After formation of the primary shear crack, the deflection and stirrup strain rapidly increased, and the ultimate failure occurred at a shear of 385.2 kN with crushing and separating of the top concrete cover, bucking of the compressive steel, and brittle shear-splitting cracks in the end region of the beam as shown in Fig. 5. Specimens SNH and SH displayed similar shear of the primary shear cracking as that of specimen SN, but they displayed 64% and 57% higher ultimate shear, respectively, than specimen SN. Unlike specimen SN, which failed ultimately with a small additional shear of 99.0 kN after the formation of the primary shear

Table 3 Test results and comparison with predicted results

Specimen	Test results		ACI 318-08		CSA A23.3-04 Simplified method		MCFT		Eurocode 2
	V_{cr} (kN)	V_u (kN)	$V_{c,ACI}$ (kN)	$V_{n,ACI}$ (kN)	$V_{c,CSA}$ (kN)	$V_{n,CSA}$ (kN)	$V_{c,MCFT}$ (kN)	$V_{n,MCFT}$ (kN)	$V_{n,EC2}$ (kN)
SN	286.2	385.2	296.3 (0.97)	403.3 (0.96)	227.9 (1.26)	380.6 (1.01)	283.6 (1.01)	437.1 (0.88)	240.6 (1.60)
SNH	292.7	631.1	296.3 (0.99)	403..3 (1.57)	227.9 (1.28)	380.6 (1.66)	283.6 (1.03)	452.8 (1.39)	240.6 (2.62)
SH	291.7	605.0	296.3 (0.98)	458.6 (1.32)	227.9 (1.28)	517.4 (1.17)	283.6 (1.03)	504.1 (1.20)	365.1 (1.66)
Mean			0.98	1.28	1.27	1.28	1.02	1.16	1.96
COV			0.012	0.239	0.011	0.265	0.011	0.223	0.293

V_{test}/V_{pred} given in parenthesis

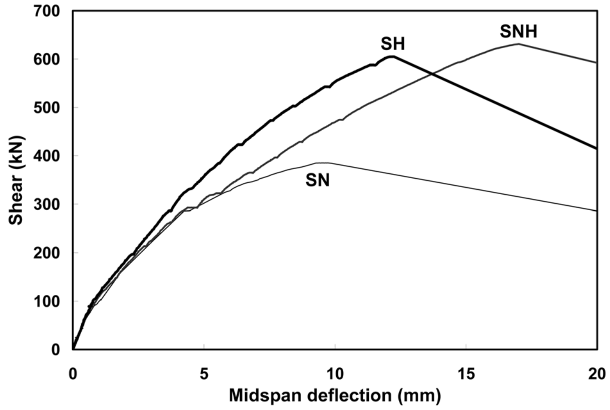


Fig. 4 Shear versus midspan deflection relationships

(a) Crushing of concrete, buckling of compressive steel, and slip of hook in stirrup

(b) Shear-splitting crack

Fig. 5 Failure pattern of specimen SN

crack, specimens SNH and SH showed ultimate failure with a much larger additional shear of 338.4 kN and 313.3 kN after formation of the primary shear crack, respectively. In addition, SNH and SH showed a smooth increase of midspan deflection without a rapid decrease of stiffness that appeared in the shear-deflection relationship of the SN.

The remarkably low shear resistance provided by the shear reinforcement of specimen SN could

Table 4 Minimum shear reinforcement requirements

Specimen	A_v (mm^2)	$A_{v,\min}$ (mm^2)		
		ACI (2008)	CSA (2004)	EC2 (2003)
SN	142.6	171.0	165.5	218.8
SNH	142.6	171.0	165.5	218.8
SH	142.6	112.7	109.0	144.2

It was assumed that the concrete characteristic strength was taken as $f_{ck} = f_c' - 1.6$ MPa (Gardner 2005)

be due to an insufficient amount of shear reinforcement and anchorage loss of the hook in the U shape stirrup. Table 4 gives the area of shear reinforcement and required minimum area of shear reinforcement for all specimens according to the ACI Code (2008), CSA Standard (2004) and EC2 Code (2003). As shown in Table 4, the shear reinforcement of specimen SN was less than the minimum shear reinforcement requirement. Because the shear cracking of high-strength concrete appears at a high shear, if the beam section does not contain an increased minimum amount of transverse reinforcement, shear strength after shear cracking would be difficult to reserve by stirrups. Specimen SH, which had stirrups that provided similar or more than the minimum amount of shear reinforcement due to the high stirrup strength, showed significant reserves of strength after cracking and a much higher ultimate shear strength.

An anchorage loss of a hook in a stirrup could result in specimen SN becoming weak and brittle. Conventional stirrups cannot develop full yield strength in the leg adjacent to the hook (Ghali and Dilger 1998). The high compressive stress, developed on the concrete inside the hook, causes crushing of the concrete and slippage of the hook before the tensile stress in the leg reaches its yield strength. Especially in high-strength concrete beams, the combination of higher compressive stress and higher shear after formation of the primary shear crack could lead to a sudden increase of hook slippage near the shear crack that has propagated deeply into the compression zone. As shown in Fig. 5(a), specimen SN displayed a noticeable slip of the hook in the stirrup and a buckling of the compressive steel. However, it should be noted that the headed bars in SNH provided excellent end anchorage, thereby redistributing the stress between the stirrups that cross the primary shear crack, and reserving the shear strength after formation of the primary shear crack. All this was achieved despite the shear reinforcement amount in SNH being less than the minimum shear reinforcement requirements.

4.2 Stirrup strains

Immediately after formation of the primary shear crack, the first yielding of a stirrup appeared in specimens SN and SNH. However, in specimen SH the first yielding of the stirrup occurred with an additional shear of 45 kN after formation of the primary shear crack due to the high yield strength of the stirrups. This indicated that high-strength stirrups resist higher, sudden shear transmission from concrete to shear reinforcement better in high-strength concrete beams. Fig. 6 shows the stirrup strains measured at ultimate load. This figure demonstrates that, contrary to specimens SNH and SH, which showed good redistribution of shear stress between stirrups, only one stirrup (S10) developed its full strength and resisted shear load until ultimate failure in specimen SN.

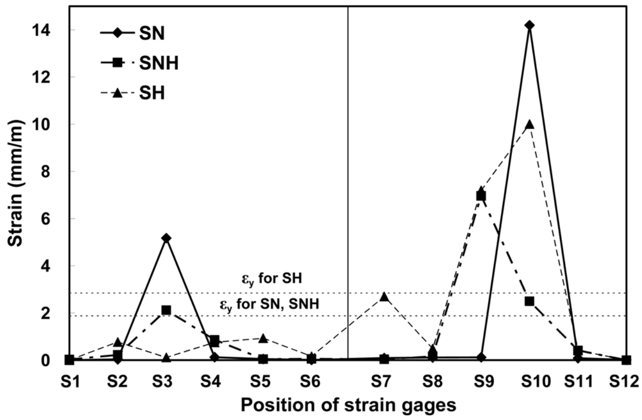
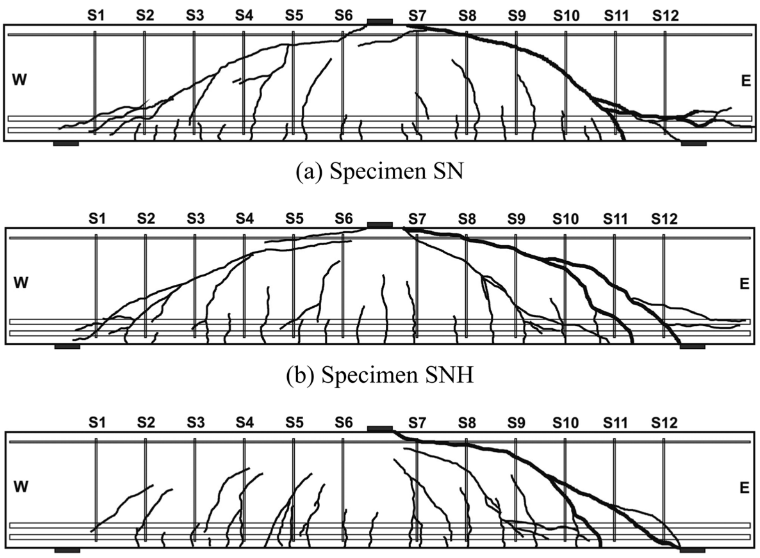


Fig. 6 Stirrup strains measured at ultimate loads

4.3 Cracking responses

Fig. 7 describes the crack patterns at ultimate failure for all specimens. All specimens showed typical shear cracking patterns, which started with the formation of a number of small flexural cracks and expanded to a diagonal shear crack. All specimens displayed the same angle at which the primary shear crack formed as well as smooth shear crack surfaces, which is a feature of high-strength concrete. Due to the high strength of the concrete matrix, cracks pass through instead of around the



(a) Specimen SN

(b) Specimen SNH

(c) Specimen SH

Fig. 7 Crack patterns at failure

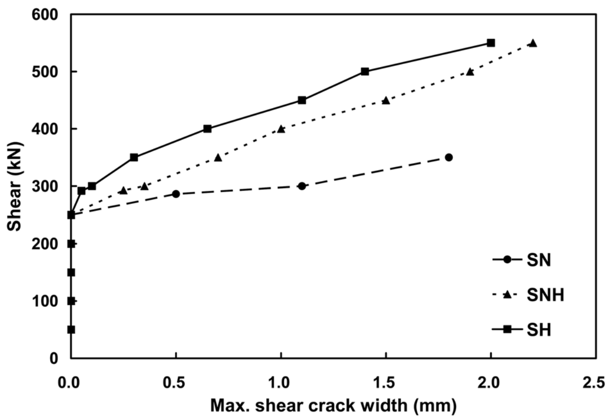


Fig. 8 Shear versus maximum shear crack width

aggregates, thereby reducing the aggregate interlock and shear carried by the concrete. This reduced aggregate interlock induces higher dowel forces in the longitudinal reinforcing bars, and results in brittle shear-splitting cracks in the end regions of the beams (Johnson and Ramirez 1989, Yoon *et al*. 1996). As shown in Fig. 5(b), specimen SN showed conspicuous shear-splitting cracks, and this is evidence of brittle shear failure at relatively low shear.

Fig. 8 shows the shear versus maximum shear crack width for all of the specimens tested. The maximum shear crack width of specimen SN increased rapidly after formation of the primary shear crack, while specimens SNH and SH showed a relatively slow increase in maximum shear crack width.

5. Comparison of test results and predicted results

The experimental test results of this study were compared with the shear strength predictions based on the modified compression field theory (MCFT; Collins and Mitchell 1991) and the equations specified in ACI 318-08 (2008), CSA A23.3-04 (2004) and Eurocode 2 (2003). A summary of the experimental and predicted results is presented in Table 3.

The ACI Code (2008) and the simplified method of the CSA Standard (2004) provide the same expression for the nominal shear strength, given by

$$V_n = V_c + V_s = 0.17\sqrt{f_c'}b_w d + \frac{A_v f_{yt} d}{s}\cot\theta \tag{1}$$

where the term $\sqrt{f_c'}$ should not exceed 8.3 and 8 MPa, and θ is assumed as 45° and 35° in the ACI Code and the simplified method of CSA Standard, respectively. The ACI Code does not consider the size effect, while a size effect factor of [217/(1000+d)] should substitute for a constant of 0.17 if the section contains no transverse reinforcement or less than the minimum amount of transverse reinforcement in the CSA Standard simplified method.

The Eurocode 2 (2003) uses a variable angle truss model to assign all shear capacity to the truss for members with shear reinforcement, thereby ignoring any contribution from the concrete shearing mechanism, as given by

$$V_n = \frac{A_v f_{yt} z}{s} \cot\theta \tag{2}$$

where the lever arm, z, may be considered as $0.9d$, and $1 \le \cot\theta \le 2.5$.

The MCFT account for strain compatibility and uses both tensile and compressive stress-strain relationships for the diagonally cracked concrete (Collins and Mitchell 1991). In order to predict shear strength using the MCFT, the tensile stress factor β and the angle of principal compression θ should be calculated. The values of both β and θ are related to the longitudinal strain and shear stress in the web, hence these two values are not constant. The computer program "RESPONSE 2000" (Bentz and Collins 2000) based on the MCFT was used to calculate the predicted values of the MCFT method. This program combines a plane section analysis for flexure and modified compression field analysis for shear. The calculations were performed at a section located at a distance d from the face of the loading plate, that is, where the shear-to-moment ratio was 1.2 m. In calculating the MCFT prediction of specimen SNH using the RESPONSE 2000 program, the "T Headed single leg" option was selected to define the transverse reinforcements.

In a comparison of test results and predicted results for the concrete shear strength, V_c, all predictions provided generally reasonable results. However, it is noted that the ACI Code prediction slightly overestimated the concrete shear strength although the longitudinal reinforcement ratios of all specimens were significantly higher and the concrete strength was limited to 69 MPa. This is because the ACI Code does not account for the effect of large beam size. The predictions of CSA Standard simplified method, which considers the size effect, showed the most conservative results with an average test-to-predicted strength ratio of 1.27. In a comparison of nominal shear strength, V_n, the predictions of the ACI Code and MCFT method for Specimen SN showed unconservative results because Specimen SN was reinforced with less than the minimum transverse reinforcement. Eurocode 2 (2003) predicted the most conservative results for all specimens. This is attributed to its basic assumption ignoring any contribution from the concrete shearing mechanism.

6. Shear strength of transverse reinforcement

Tompos and Frosch (2002) pointed out that the d/s value used to compute the number of stirrups crossing a shear crack should be rounded down to an integer quantity when the stirrup shear strength is predicted using the ACI Code (2008). This is because a crack cannot cross a fractional portion of a stirrup. Intuitively, if a stirrup is crossed by the shear crack, it contributes strength; if a stirrup is not crossed by a shear crack, it should not contribute strength. They also mentioned that the number of stirrups that effectively transfer shear across the inclined crack should be limited by the development length of the shear reinforcement. ACI code method assumes that all stirrups of d/s can develop full yield strength although actual steel can develop yield strength when full anchorage is provided. Based on those two viewpoints, Tompos and Frosch (2002) proposed a new calculating method for the stirrup shear strength as shown in Eqs. (3) and (4).

$$V_s = A_v f_{yt} N_v \tag{3}$$

where

$$N_v = \mathrm{INT}\left(\frac{d - l_{dv}}{s}\right) \tag{4}$$

Table 5 Analysis of stirrup capacity

Specimen	V_u (kN)	V_{cr} (kN)	V_s (kN)	$V_{s, Eq.(3)}$ (kN)	$V_{s,ACI}$ (kN)
SN	385.2	286.2	99.0	53.5 (1.85)	107.0 (0.93)
SNH	631.1	292.7	338.4	53.5 (6.33)	107.0 (3.16)
SH	605.0	291.7	313.3	81.1 (3.86)	162.3 (1.93)
Mean				4.01	2.01

V_{test}/V_{pred} given in parenthesis

Table 5 summarizes the calculated results for the stirrup shear strength using Eq. (3) and the ACI Code. The method of Tompos and Frosch (2002) was more conservative especially in the case of specimen SN, in which the ACI Code slightly overestimated the shear strength developed by the stirrups. However, their method considerably underestimates the shear strength for specimens SNH and SH. This is because the SNH and SH had two primary shear cracks and Eq. (1) does not consider the contributions of the strain hardening of effective stirrups. As shown in Fig. 7, the primary shear cracks cross more than two stirrups in specimens SNH and SH, while Eq. (1) predicts that the number of effective stirrups to be just one.

7. Conclusions

The following conclusions were drawn from the shear tests on three high-strength concrete beams with normal-strength stirrups, high-strength stirrups, and headed bars as shear reinforcement, respectively:

1. The specimen reinforced with normal-strength U shape stirrups showed sudden shear failure with lower shear strength due to anchorage loss of the hooks in the stirrup after formation of the primary shear crack and the amount of shear reinforcement less than the minimum shear reinforcement requirement.
2. The headed bar as shear reinforcement provided excellent end anchorage, and the beam that had headed bars as shear reinforcement therefore showed significant reserves of shear strength and good redistribution of shear between stirrups after shear cracking.
3. The high-strength stirrup resisted high, sudden shear transmission from concrete to shear reinforcement more successfully in the high-strength concrete beams, resulting in a higher ultimate shear strength of beams.
4. The headed bar stirrup and high-strength stirrup can be better reinforcements to replace conventional stirrups for high-strength concrete beams.
5. Further experimental research, which have many variables of beam size, longitudinal and transverse reinforcement ratio, concrete strength, and so on, are required to verify the effectiveness of headed bar and high-strength steel as a shear reinforcement for high-strength concrete beams.
6. As the concrete shear strength equation of the ACI Code does not account for the size effect, the ACI Code prediction slightly overestimated the concrete shear strength for all specimens. All

code predictions for the nominal shear resistances are generally conservative for tested beams reinforced with headed bars and high-strength steels. The prediction method for stirrup shear strength proposed by Tompos and Frosch was much more conservative than that of the ACI code.

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References

American Concrete Institute (ACI) (2008), *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-08 and ACI 318M-08, Farmington Hills, MI.

Angelakos, D., Bentz, E.C. and Collins, M.P. (2001), "Effect of concrete strength and minimum stirrup on shear strength of large members", *ACI Struct. J.*, **98**(3), 290-300.

ASTM International (2007), *Standard Specification for Headed Steel Bars for Concrete Reinforcement*, ASTM A970-07, West Conshohocken, PA.

Bentz, E.C. and Collins, M.P. (2000), *RESPONSE 2000*, Version 1.05, University of Toronto, Ontario.

Canadian Standards Association (CSA) (2004), *Design of Concrete Structures,* CSA A23.3-04, Mississauga, Ontario.

Collins, M.P. and Kuchma, D. (1999), "How safe are our large, lightly reinforced concrete beams, slabs, and footings?" *ACI Struct. J.*, **96**(4), 482-490.

Collins, M.P. and Mitchell, D. (1991), *Prestressed Concrete Structures*, Prentice-Hall, Englewood Cliffs, NJ.

Collins, M.P., Mitchell, D. and MacGregor, J.G. (1993), "Structural design consideration for high-strength concrete", *Concrete Int.*, **15**(5), 27-34.

European Standard (2003), *Eurocode 2: Design of Concrete Structures – Part 1: General Rules and Rules for Buildings*, prEN 1992-1-1, Final draft, European Committee for Standardization, Brussels.

Gardner, N.J. (2005), ACI 318-05, CSA A23.3-04, Eurocode 2 (2003), DIN 1045-1 (2001), BS 8110-97 and CEB-FIP MC 90 provisions for punching shear of reinforced concrete flat slabs. *Punching Shear in Reinforced Concrete Slabs*", *SP-232* (Ed. Polak MA), American Concrete Institute, Farmington Hills, MI.

Gayed, R.B. and Ghali, A. (2004), "Double-head studs as shear reinforcement in concrete I-beams", *ACI Struct. J.*, **101**(4), 549-557.

Ghali, A. and Dilger, W. (1998), "Anchoring with double-head studs", *Concrete Int.*, **20**(11), 21-24.

Ghali, A. and Youakim, S.A. (2005), "Headed studs in concrete: state of the art", *ACI Struct. J.*, **102**(5), 657-667.

Griezic, A., Cook, W.D. and Mitchell, D. (1994), "Tests to determine performance of deformed welded wire fabric stirrups", *ACI Struct. J.*, **91**(2), 211-220.

Johnson, M.K. and Ramirez, J.A. (1989), "Minimum shear reinforcement in beams with higher strength concrete", *ACI Struct. J.*, **86**(4), 376-382.

Joint ACI-ASCE Committee 445 (1998), "Recent approaches to shear design of structural concrete", *J. Struct. Eng.*, **124**(12), 1375-1417.

Mast, R.F., Dawood, M., Rizkalla, S.H. and Zia, P. (2008), "Flexural strength design of concrete beams reinforced with high-strength steel bars", *ACI Struct. J.*, **105**(4), 570-577.

Roller, J.J. and Russel, H.G. (1990), "Shear strength of high-strength concrete beams with web reinforcement", *ACI Struct. J.*, **87**(2), 191-198.

Sumpter, M.S., Rizkalla, S.H. and Zia, P. (2009), "Behavior of high-performance steel as shear reinforcement for concrete beams", *ACI Struct. J.*, **106**(2), 171-177.

Tompos, E.J. and Frosch, R.J. (2002), "Influence of beam size, longitudinal reinforcement, and stirrup effectiveness

on concrete shear strength", *ACI Struct. J.*, **99**(5), 559-567.
Yang, J.M., Yoon, Y.S., Cook, W.D. and Mitchell, D. (2010), "Punching shear behavior of two-way slabs reinforced with high-strength steel", *ACI Struct. J.*, **107**(4), 468-475.
Yoon, Y.S., Cook, W.D. and Mitchell, D. (1996), "Minimum shear reinforcement in normal, medium, and high-strength concrete beams", *ACI Struct. J.*, **93**(5), 576-584.

Notations

a = shear span length, mm;
A_v = area of shear reinforcement spacing s, mm^2;
$A_{v,\,\min}$ = minimum area of shear reinforcement within spacing s, mm^2;
b_w = web width of beam, mm;
d = distance from extreme compression fiber to centroid of main tension tie, mm;
f_c' = compressive strength of concrete at time of testing corbel specimens, MPa;
f_{ck} = characteristic concrete cylinder strength, MPa;
f_r = modulus of rupture of concrete, MPa;
f_{sp} = splitting tensile strength of concrete, MPa;
f_u = ultimate strength of reinforcement, MPa;
f_{yt} = yield strength of shear reinforcement, MPa;
l_{dv} = development length of transverse reinforcement, mm;
s = center-to-center spacing of transverse reinforcement, mm;
V_c = nominal shear strength provided by concrete, kN;
V_{cr} = shear corresponding to the formation of primary shear cracking of concrete, kN;
V_n = nominal shear strength, kN;
V_s = nominal shear strength provided by shear reinforcement, kN;
V_u = ultimate shear strength obtained from test, kN;
z = lever arm, mm;
ε_u = strain corresponding to ultimate strength of reinforcement;
ε_y = reinforcement yield strain;
θ = angle between diagonal shear crack and axis of longitudinal reinforcement;