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Shear strength of slender SCC beams – possible differences from VC beams

Resistência à força cortante de vigas esbeltas de CAA – possíveis diferenças em relação a vigas de CV

T. L. RESENDE [a]
thomas.lima@ufvjm.edu.br

L. C. D. SHEHATA [b]
lidiashehata@vm.uff.br

I. A. E. M. SHEHATA [c]
ibrahim@coc.ufrj.br

Abstract

In comparison with vibrated concrete (VC) of the same strength class, self-compacting concrete (SCC) typically has a lower coarse aggregate content and, eventually, a smaller maximum aggregate size. This may reduce the aggregate interlock between the fracture surfaces in SCC. Since the aggregate interlock plays an important role in the shear strength of slender beams, SCC beams may have a shear strength lower than that of similar VC beams.

This article summarizes experimental studies on the shear strength of reinforced SCC slender beams without and with shear reinforcement. The shear strengths of SCC beams are compared with the ones of VC beams and also to the calculated ones according to different code procedures. It is shown that powder-type SCC beams tend to have lower shear strength than similar VC beams and that the difference depends upon the concretes composition and the characteristics of the beams.

Keywords: self-compacting concrete, slender beams, shear strength.

Resumo

Em comparação com o concreto vibrado (CV) de mesma classe de resistência, o concreto autoadensável (CAA) tipicamente apresenta menor quantidade de agregado graúdo e, eventualmente, menor dimensão máxima deste agregado. Isto pode reduzir o engrenamento de agregados entre as superfícies de ruptura do CAA. Sendo o engrenamento dos agregados um parâmetro de influência importante na resistência à força cortante de vigas esbeltas, as vigas de CAA podem ter resistência à força cortante menor que a de vigas similares de CV.

Este artigo resume estudos experimentais sobre a resistência à força cortante de vigas esbeltas de CAA sem e com armadura de cisalhamento. As resistências à força cortante de vigas de CAA são comparadas com as de vigas de CV e também com as calculadas segundo procedimentos de diferentes normas. É mostrado que as vigas de CAA com menor teor de agregados graúdos tendem a ter menor resistência à força cortante que vigas semelhantes de CV e que a diferença depende da composição dos concretos e das características das vigas.

Palavras-chave: concreto autoadensável, vigas esbeltas, resistência à força cortante.

[a] *Universidade Federal dos Vales do Jequitinhonha e Mucuri, Instituto de Ciência, Engenharia e Tecnologia, Teófilo Otoni, MG, Brasil;*
[b] *Universidade Federal Fluminense, Departamento de Engenharia Civil, Niterói, RJ, Brasil;*
[c] *Universidade Federal do Rio de Janeiro, COPPE, Programa de Engenharia Civil, Rio de Janeiro, RJ, Brasil.*

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1. Introduction

The use of SCC is progressively increasing, especially in the precast industry, and a large amount of research has been done on the fresh and hardened properties of SCC, but relatively little research has been carried out on the structural behaviour of SCC.
As far as shear behaviour is concerned, results from different research works have shown contradictory conclusions. Some have shown that SCC and VC beams with the same characteristics have a similar shear strength, while, according to others, SCC beams have lower shear strength. That is probably due to the different parameters that affect the shear strength of beams and also to the different possible concretes compositions.
In order to obtain the necessary flowability of the concrete, it is more usual to opt for increasing the powder content and reducing the coarse aggregate content (powder-type SCC) and for rounder aggregates and smaller maximum aggregates size. If a viscosity modifying admixture is used, however, SCC may have a coarse aggregate content of the same order of VC (viscosity agent-type or combination-type SCC), but the use of that admixture in SCC is not the common practice of ready-mix concrete suppliers in Brazil. In comparison with a vibrated concrete of the same strength class, the reduction of coarse aggregate content and, eventually, also the aggregate maximum size in SCC may produce a reduction of the aggregate interlock between crack surfaces, but this also depends on the paste and interfacial transition zone, that tend to be denser and more uniform than in VC.
The ultimate nominal shear stress of slender members without transverse reinforcement depends mainly on the concrete strength, the aggregate interlock between surfaces of cracks, the effective depth (size effect) and the longitudinal reinforcement ratio. It has been shown that the aggregate interlock plays an important role in the shear capacity of members without shear reinforcement [1, 2, 3] and that it is affected by the roughness of the crack interfaces, associated with the type and size of the aggregate, as well as crack width. In high strength concrete members, the cracks can go through the aggregates, instead of propagating around them, reducing the roughness of cracks interfaces and, consequently, the interlocking capacity [4, 5].
In order to avoid non-conservative predictions when applied to high strength concrete members, some code procedures limit the concrete compressive strength or maximum aggregate size to be considered in shear strength equations. The UK National Annex to Eurocode 2 [6] limits f_{ck} to 50 MPa unless justified otherwise and the FIB MC 2010 [7] considers an aggregate maximum size equal to zero when the concrete strength exceeds 70MPa (Level of Approximation II).
Members with a higher longitudinal reinforcement ratio have a higher shear capacity, which can be attributed to a combination of additional dowel action (if there is no yielding) and smaller crack widths, that result in an aggregate interlock increase and a larger concrete compression zone.
For the same tensile longitudinal reinforcement ratio and concrete, higher members have wider cracks and aggregate interlock becomes less effective.
Transverse reinforcement contributes itself to shear strength and enhances the contribution of other shear transfer mechanisms, restricts the widening of shear cracks and may mitigate the size effect on the shear strength of beams, but not suppress it [9].
This paper analyses a database for beams tested by different authors who investigated the shear strength of SCC slender beams (shear span to effective depth ratio a/d ≥ 2.5) with no axial force. Some of those authors tested only SCC beams [10, 11, 12] while the others [13 - 21], for the sake of comparison, tested SCC and VC beams and these VC beams are also included in the analysis. The shear strengths of SCC and VC beams are compared and the experimental shear strengths of the beams, V_u, are compared with the ones calculated, V_R, using the provisions of ABNT NBR 6118:2014 [22], ACI 318:2014 [8], EN 1992-1-1:2004 [23] and FIB MC 2010 [7].

2. Experimental studies on shear strength of slender SCC beams

Hassan, Hossain and Lachemi [13,14], aiming to compare the shear strength of similar VC and SCC beams without transverse reinforcement, tested ten VC and ten SCC beams. Ready-mixed concretes were used and, apart from the chemical admixtures, the same types of materials were used in the two types of concrete, with the coarse aggregate content of the SCC being about 20% smaller than that used in the VC. The compressive strength of the concretes was about the same, 45MPa (SCC) and 47MPa (VC) and the total depth (h) and shear span (a) of the beams varied in such way that the ratio (a/h) was kept equal to 2.5. For each depth and type of concrete, two different values of tensile longitudinal reinforcement ratio $\rho = A_s/(b_w d)$ were used in the beams. Figures 1 and 2 compare the ultimate shear stress $V_u/(b_w d)$ of the similar beams of SCC and VC with $A_s/(b_w h)$=1.0% and 2.0%, respectively. They show that the ultimate shear stresses of the SCC beams were always lower than those of the VC beams and that the difference increased with decreasing tensile longitudinal reinforcement and increasing depth. The difference for those with a smaller longitudinal reinforcement ratio varied between ≅ 5% and 16%, while for those with a greater longitudinal reinforcement it varied between ≅ 4% and 7%.

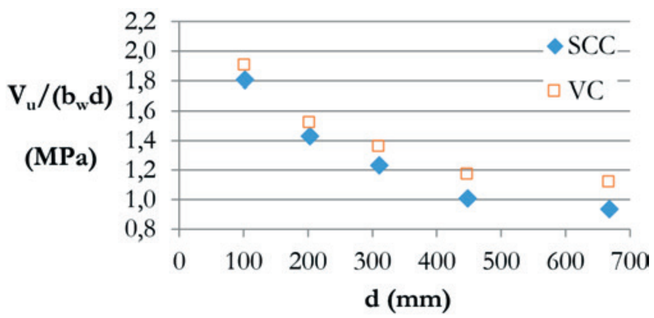

Figure 1
Comparison between the ultimate shear stress of SCC and VC beams tested by Hassan, Hossain and Lachemi (13,14) with $A_s/(b_w h)$=1.0%

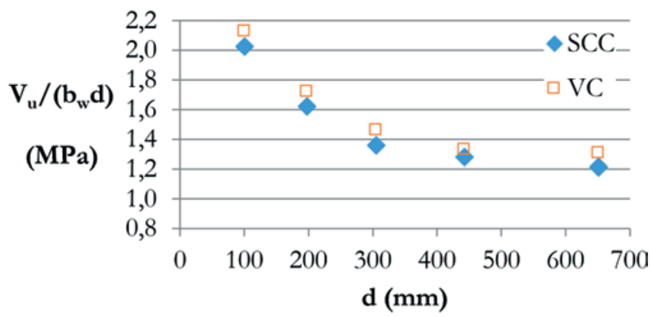

Figure 2
Comparison between the ultimate shear stress of SCC and VC beams tested by Hassan, Hossain and Lachemi (13,14) with $A_s/(b_w h)$=2.0%

Beygi et al. [10] tested four SCC beams with transverse reinforcement, where two different concrete mixes were used in order to have normal strength and high strength SCC. The high strength SCC beams had a higher value of ρ and, for each type of concrete, two beams with different values of transverse reinforcement index $\rho_w f_{yw}$ were tested.

In order to investigate the shear behaviour of VC and SCC reinforced I beams produced in a precast factory, Cuenca, Serna and Pelufo [15] tested one beam of each type of concrete (a/d≅3.2, ρ≅3%, $\rho_w f_{yw}$≅0.85MPa). Apart from the sand, the same types of materials were used in both concretes (natural sand in the VC and artificial sand in the SCC). The coarse aggregate content of the SCC was ≅11% smaller than that of the VC, the compressive strengths of both concretes was around 50MPa and the beams had a similar shear behaviour.

Boel et al. [16] and Helincks et al. [17] tested beams produced with two mixes of VC and four mixes of SCC. For each mixture type, a/d (2.5 or 3.0) and ρ (1.2%, 1.7% or 2.3% for SCC beams and 1.2% for VC beams) values, three or four beams were produced, but only the average value of shear strength of each group was reported. One VC (maximum aggregate size d_{max}=16mm) and one SCC (d_{max}=8mm) were supplied by a ready-mix concrete company; the others were made in the laboratory. The laboratory SCC mixtures had the same coarse aggregate content (around 43% less than the VC) and the differences between them were the contents of cement, limestone filler and superplasticizer. Part of the coarse aggregate of the VC and SCC (38%) had maximum size d_{max}= 8mm and the other part d_{max}=16mm. All the beams had the same dimensions and no transverse reinforcement and the compressive strength of the concretes varied between around 50MPa and 60MPa. All the beams had the same values of b_w and d and the normalized shear strengths ($V_u/\sqrt{f_c}$) of the SCC beams were about 95% of those of the similar VC beams (a/d=2.5 or 3.0 and ρ=1.2%).

From the three VC and six SCC used in the beams tested by Lin and Chen [18], only the SCC had mineral additions, superplasticizer and smaller maximum coarse aggregate size. Three types of SCC (SCC-1) had about the same coarse aggregate content of the three VC ones whilst the others (SCC-2) had 14% less. The SCC-1 mixtures had three types of mineral additions (fly ash+slag+silica fume) and the SCC-2 mixtures had two (fly ash+slag), and the

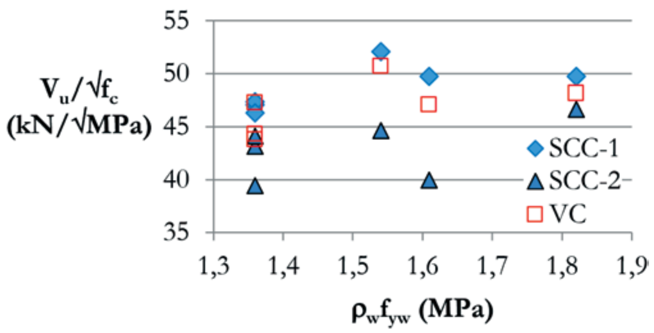

Figure 3
Comparison between the normalized shear strength of VC and SCC beams tested by Lin and Chen (18) with a/d=3.0

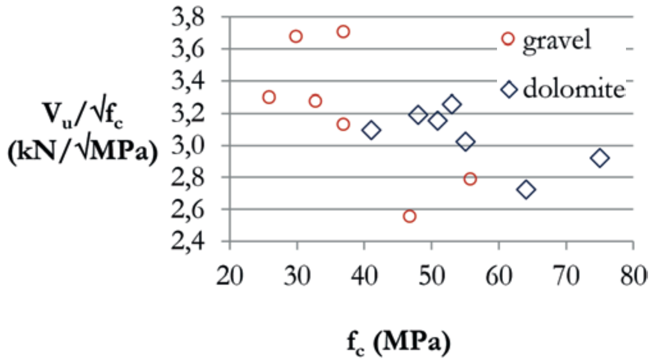

Figure 4
Normalized shear strength of the SCC beams with different types of coarse aggregate and ρ = 1.68% tested by Safan (11)

compressive strength of the concretes varied from about 30MPa to 50MPa. The dimensions and the longitudinal reinforcement of the beams were kept constants and, besides the type of the concrete, the values of a/d (2.5, 3.0 or 3.5) and $\rho_w f_{yw}$ (from 1.36MPa to 1.82MPa) were varied. The beams tested with a/d=2.5 and 3.5 always had $\rho_w f_{yw}$=1.36MPa and the beams tested with a/d=3.0 had four different values of $\rho_w f_{yw}$. The normalized shear strengths ($V_u/\sqrt{f_c}$) of the beams tested with a/d=3.0, plotted in figure 3 as a function of $\rho_w f_{yw}$, show that the SCC beams with a lower coarse aggregate content (SCC-2) tended to have a lower shear strength.

Fourteen different SCC mixes were used in the experimental programme of Safan [11]; seven had natural gravel as coarse aggregate and seven had crushed dolomite with the same maximum size. For each type of coarse aggregate, there was one mixture with no mineral additions, two with different dolomite powder contents; two with dolomite powder and silica fume (two different dolomite powder contents) and two with dolomite powder and fly ash. The compressive strength of the concretes varied from about 25MPa to 75MPa and the compressive strengths of the SCC with gravel were always higher than the ones with the same composition but with crushed dolomite as coarse aggregate. The dimensions of the beams, that had no shear reinforcement, were kept constants and, for each type of SCC, two beams with different tensile longitudinal reinforcement ratios were tested having a/d=2.6. The crushed dolomite SCC beams had failure surfaces passing through the coarse aggregate, while the gravel SCC beams had rough failure surfaces going around the coarse aggregate. Figure 4 shows that the normalized shear strengths ($V_u/\sqrt{f_c}$) of beams with the same coarse aggregate and tensile longitudinal reinforcement tended to decrease with increasing f_c.

Salman, Jarallah and Delef [19] tested VC and SCC beams with same dimensions and transverse reinforcement and variable tensile longitudinal reinforcement. Apart from the limestone powder and superplasticizer used only in the two SCC types, the same materials were used in the concretes and the two SCC had 23% less coarse aggregate than the VC. The values of f_c were around 25MPa (VC and SCC) or 50MPa (SCC). The VC and SCC beams with f_c ≅ 25MPa and same tensile longitudinal reinforcement had similar shear behaviour.

The SCC mixtures used in the beams tested by Arezoumandi and

Volz [20, 21] were obtained by adding chemical admixtures (viscosity agent, air-entraining and high-range water-reducing) to the VC mixture (viscosity agent-type SCC). The f_c values varied from around 34MPa to 54MPa. The beams with no shear reinforcement (six with VC and six with SCC) had three different values of ρ (two for each ρ) and the ones with shear reinforcement (two of VC and two of SCC) had the same value of ρ. The values of the average normalized shear stress $V_u/(b_w d\sqrt{f_c})$ for the SCC beams without shear reinforcement were close to those of VC with the same ρ (difference from ≅ 0 to 6%), while for the beams with shear reinforcement the difference was ≅ 12%. Considering the shear database for VC beams without shear reinforcement of Reineck et al. [23] and their SCC tested beams, Arezoumandi and Volz [20] concluded that there is no statistically significant difference between the ultimate normalized shear stress of VC and SCC beams.

Resende, Shehata and Shehata [12] aimed to investigate the shear behaviour of high strength SCC beams and small amounts of shear reinforcement, case that had not been studied (only one beam with small value of $\rho_w f_{yw}$ and f_c≈54MPa had been tested by Cuenca, Serna and Pelufo [15]). The six beams tested were produced with a SCC provided by a ready-mixed concrete supplier that had been used in a cast-in-place structure. Half the coarse aggregate used in the SCC had a maximum size of 9.5mm and the other half 19mm and the concrete compressive strength was about 70MPa. All the beams had the same dimensions (b_w=175mm and h=500mm) and a/d=2.8, their main variable was the shear reinforcement index (0.508MPa to 0.975MPa) and its four values aimed to cover the range given by $\rho_{w,min} f_{ywk}$ according to different codes of practice. The stirrups spacing ranged from approximately 0.3d to 0.5d. Two beams had the same $\rho_w f_{yw}$ but different stirrups spacing and diameter and two had the same $\rho_w f_{yw}$ and different ρ. Comparisons of the experimental shear strengths with calculated ones showed that the ACI 318:2014 [8] and the FIB MC 2010 [7] Level III of Approximation may lead to a non-conservative evaluation of the shear strength of SCC beams with small values of $\rho_w f_{yw}$, particularly if they also have smaller value of ρ.

Resende, Shehata and Shehata [12] compared the ultimate shear stresses of the SCC beams with the ones of VC beams tested by Garcia [25]. This VC had a 90mm slump, the same type of coarse aggregate with d_{max}=19mm used in the SCC and similar value of f_c. The VC beams were tested with a/d=3.0, had the cross section dimensions (b_w=150mm, h=450mm) a little smaller than those of the SCC beams, ρ (2.60%) close to that of five SCC beams (2.48%) and also small values of $\rho_w f_{yw}$. The comparison showed that the SCC

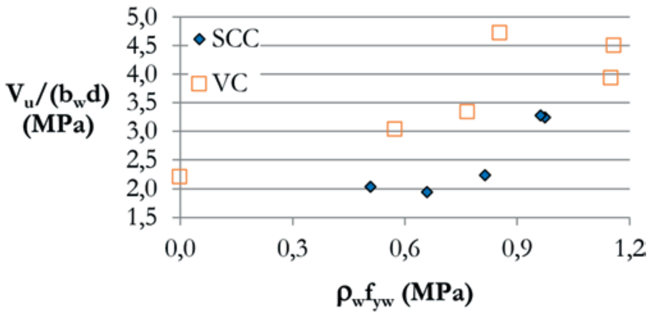

Figure 5
Comparison of the ultimate shear stresses of high strength SCC and VC beams with f_c ≅ 70 MPa and small values of $\rho_w f_{yw}$ (12)

beams had shear strength lower than that of the VC beams (figure 5) and that the differences could reach values of the order of 30%. According to the authors, such difference can neither be attributed to the small differences between the values of ρ and a/d of the groups of SCC and VC beams, nor to the difference between the effective depths as the stirrups tend to reduce the size effect [9].

3. Commentaries on the experimental studies

From the studies cited above, with the exception of the SCC used by Arezoumandi and Volz [20, 21], that were of the viscosity agent-type, the SCC of the beams were of the powder-type. All the beams were simply supported under one or two concentrated loads and, apart from one case [15], had a rectangular cross section.

The majority of the tested beams had a total depth smaller than those that beams usually have and no shear reinforcement and only few were of high strength SCC. There were 91 SCC beams (56 without and 35 with shear reinforcement) and 34 VC beams (20 without and 14 with shear reinforcement). From those beams, 54 of SCC and 11 of VC had h<300mm and 16 of SCC and 8 of VC had ρ>4%, values that do not correspond to real cases of beams and that lead to improved ultimate shear stress. Only 7 of the SCC beams had $\rho_w f_{yw}$ close to the minimum recommended by codes of practice and high strength concrete, besides usual height value [12, 15]. No beam had skin reinforcement. The ranges of the relevant parameters to shear strength are given in table 1.

Table 1
Ranges of relevant parameters of the beams

Parameter	Range	Parameter	Range
Shear span to effective depth ratio a/d	2.5 to 3.8	Effective depth to tensile reinforcement d (mm)	100 to 668
Cylinder compressive strength f_c (MPa)	25 to 75	Longitudinal tensile reinforcement ratio ρ (%)	1.0 to 4.5
Maximum aggregate size d_{max} (mm)	10 to 19	Transverse reinforcement index ρ_{wfyw} (MPa)	0.0 to 1.8

4. Comparison between experimental and calculated shear strengths

The experimental shear strengths (V_u) are compared with the calculated ones (V_R) using the procedures of the ABNT NBR 6118:2014 (models I and II, for beams with shear reinforcement) [22], ACI 318:2014 (simplified and detailed formulas for V_c) [8], EN 1992-1-1:2004 [23] and FIB MC 2010 [7] (Levels of Approximation I and II for beams without stirrups and I, II and III for beams with stirrups), with safety factors considered equal to one and adopting the experimental mean concrete compressive strength and steel yielding stress instead of the characteristic ones.
For the shear strength of beams with transverse reinforcement, the procedures of EN 1992-1-1:2004 [22] and the Levels I and II Approximation of FIB MC 2010 [7] consider only the web reinforcement contribution ($V_R = V_s$), while the ABNT NBR 6118:2014 [22], ACI 318:2014 [8] and Level of Approximation III of FIB MC 2010 [7] consider the web concrete and steel contributions ($V_R = V_c + V_s$), where V_s is based on a modified truss analogy. Table 2 gives the values of the angle between the concrete compression struts and the beam axis of the truss model (θ) and the equations of V_c for the different calculation methods. The ABNT NBR 6118:2014 equation of V_c in the table is for beams; for slabs, members where transverse reinforcement may be omitted, the code considers the equation of the previous European code, which gives more conservative values of V_c. The other codes adopt the same equation of V_c for beams and slabs. A maximum shear strength associated with the compressive struts stress limit and a minimum transverse

Table 2
Summary of procedures for shear strength calculation

Procedure	θ (°)	V_c (units: mm ; MPa)	V_R
ABNT NBR 6118:2014	Model I – 45	$\frac{0.42f_{ctm}}{\gamma_c}b_w d$	–
ABNT NBR 6118:2014	Model II – 30 to 45	$0 \text{ to } \frac{0.42f_{ctm}}{\gamma_c}b_w d$	–
ACI 318:2014*	45	$\Phi 0.17\sqrt{f_{ck}}b_w d$ or $\Phi\left[0.16\sqrt{f_{ck}} + 17\rho V\frac{d}{M}\right]b_w d \le \begin{cases}\Phi[0.29\sqrt{f_{ck}}]b_w d\\ \Phi[0.16\sqrt{f_{ck}} + 17\rho]b_w d\end{cases}$	$V_c + V_s$
EN 1992-1-1:2004**	21.8 to 45	$\left[\frac{0.18}{\gamma_c}\left(1+\sqrt{\frac{200}{d}}\right)(100\rho f_{ck})^{\frac{1}{3}}\right]b_w d \ge \left[0.035\left(1+\sqrt{\frac{200}{d}}\right)^{3/2} f_{ck}^{1/2}\right]b_w d$	V_c or V_s
FIB MC 2010***	Level I – 30 to 45	$0.9\left(\frac{180}{1000+1.125d}\right)\frac{\sqrt{f_{ck}}}{\gamma_c}b_w d$	V_c or V_s
FIB MC 2010***	Level II – (20+10000ε_x) to 45	$0.9\left(\frac{0.4}{1+1500\varepsilon_x}\frac{1300}{1000+0.9d\left(\frac{32}{16+d_{max}}\right)}\right)\frac{\sqrt{f_{ck}}}{\gamma_c}b_w d$	V_c or V_s
FIB MC 2010***	Level III – (20+10000ε_x) to 45	$0.9\left[\frac{0.4}{1+1500\varepsilon_x}\left(1-\frac{V}{V_{Rd,max,\theta min}}\right)\right]\frac{\sqrt{f_{ck}}}{\gamma_c}b_w d$	$V_c + V_s$

* $\sqrt{f_{ck}} \le 8$ (exception to this limit is permitted in beams with $\rho_w > \rho_{w,min}$)
** $(1+\sqrt{\frac{200}{d}}) \le 2$; $\rho \le 0.02$
*** $\sqrt{f_{ck}} \le 8$; ε_x is the longitudinal strain at the mid-depth, approximated to half the strain in the tensile longitudinal reinforcement; $V_{R,max,\theta min}$ is the upper limit to the shear resistance correspondent to the limit stress in the diagonal concrete struts considering $\theta = \theta_{min}$.
For members without transverse reinforcement, only Levels I and II are applicable.

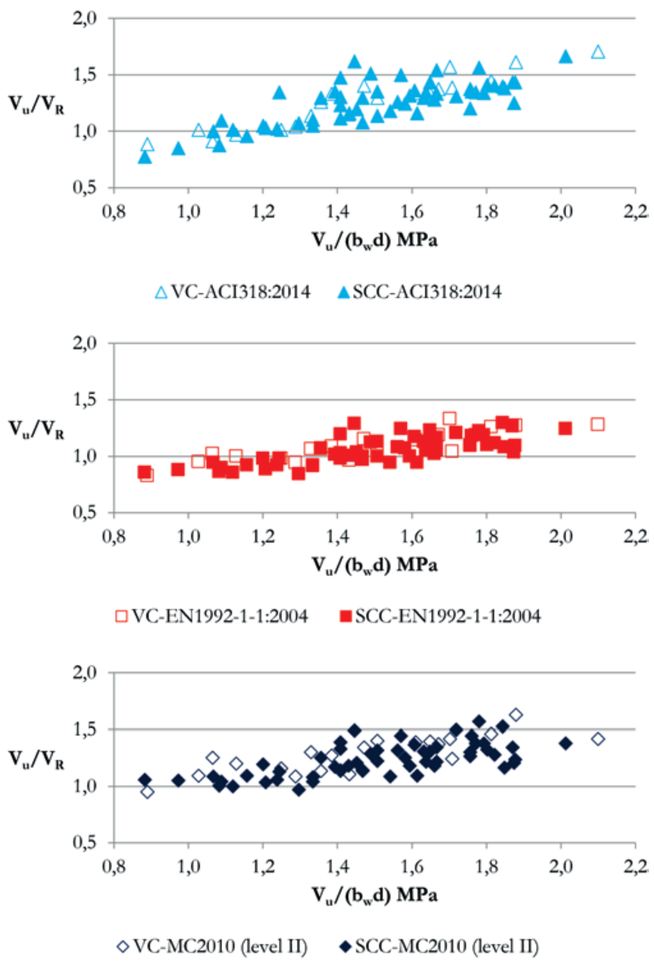

Figure 6
Values of V_u/V_R of beams without shear reinforcement as a function of $V_u/(b_wd)$

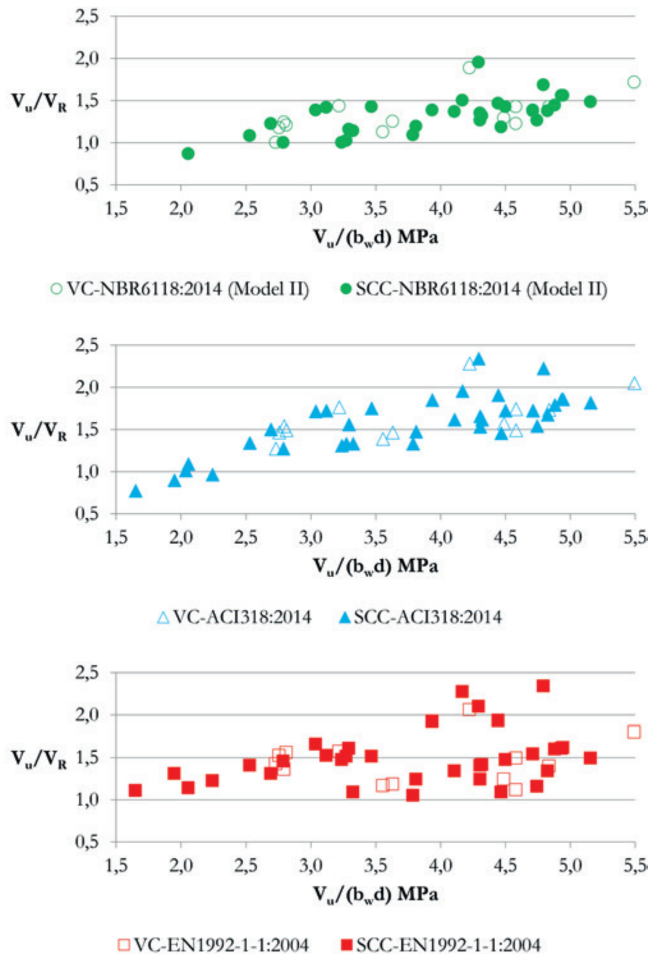

Figure 7a
Values of V_u/V_R of beams with shear reinforcement as a function of $V_u/(b_wd)$ (Continue)

reinforcement ratio also are defined in the codes, but they are not included in table 2 for the sake of conciseness.
Some formulas of V_c listed in table 2 take into consideration only f_{ck} while others include other parameters that affect the shear resistance and they may give quite different values of V_c.
It is worth mentioning that, from the codes cited above, only the ACI 318:2014 [8] allows cases of beams without shear reinforcement (beams with smaller height and shear); the others consider compulsory the use of the indicated minimum transverse reinforcement ratio. The analysis of beams with no shear reinforcement may, however, provide information on the behaviour of one-way slabs and on the minimum transverse reinforcement needed in beams to avoid sudden shear failure soon after the formation of the critical diagonal crack.
In calculating V_s of Model II of ABNT NBR 6118:2014 [22], EN 1992-1-1:2004 [23], and Level I Approximation of FIB MC 2010 [7], the minimum allowed angle between the concrete compression struts and the beam axis of the truss model was used (θ=30°, 21.8° and 30°, respectively); for Model I of ABNT NBR 6118:2014 [22] and ACI 318:2014 [8] this angle is 45°. The values of V_R given by Model II of ABNT NBR 6118:2014 [22] and Levels II and III Approximation of FIB MC 2010 [7] were obtained in an iterative manner.
Figures 6 and 7 show the V_u/V_R ratio for the beams without and with transverse reinforcement, respectively, as a function of $V_u/(b_wd)$. The values of V_R used in the graphics were calculated using the detailed formula of ACI 318:2014 [8] for V_c and Level II Approximation of FIB MC 2010 [7] for beams without stirrups and Level III Approximation of FIB MC 2010 [7] and Model II of ABNT NBR 6118:2014 [22] for beams with stirrups, methods that are supposed to give more realistic shear strengths than the more simplified ones of those codes. In the graphics for beams with shear reinforcement, only the beams with $\rho_w f_{yw}$ greater than or approximately equal to the minimum given by each code are considered.
The average, median and coefficient of variation of the V_u/V_R ratios for the groups of SCC and VC beams are in table 3. The percentage of cases with $V_u/V_R < 1$ is also given.
The graphics related to the beams without shear reinforcement and table 3 indicate that the values of V_u/V_R have smaller coefficient of variation and are more concentrated around the unit when the EN 1992-1-1:2004 [23] is used. On the other hand, this

Table 3
Statistical data of V_u/V_R

Procedure	$\rho_{wfyw} = 0$				$\rho_{wfyw} \geq \rho_{w,minfywk}$			
	Average	Median	Coef. Var.	$V_u/V_R<1$	Average	Median	Coef. Var.	$V_u/V_R<1$
SCC								
NBR 6118:2014 I	—	—	—	—	1.55	1.59	0.172	0.0%
NBR 6118:2014 II	—	—	—	—	1.32	1.37	0.170	3.2%
ACI 318:2014*	1.29	1.31	0.158	5.4%	1.64	1.75	0.228	5.7%
ACI 318:2014	1.26	1.29	0.149	7.1%	1.56	1.62	0.225	8.6%
EN 1992-1-1:2004	1.05	1.04	0.116	32%	1.49	1.46	0.215	0.0%
FIB MC 2010 I	1.64	1.63	0.145	0.0%	2.14	2.11	0.215	0.0%
FIB MC 2010 II	1.23	1.22	0.118	1.8%	1.90	1.83	0.237	0.0%
FIB MC 2010 III	—	—	—	—	1.48	1.46	0.230	8.8%
VC								
NBR 6118:2014 I	—	—	—	—	1.58	1.52	0.178	0.0%
NBR 6118:2014 II	—	—	—	—	1.34	1.27	0.174	0.0%
ACI 318:2014*	1.30	1.33	0.196	15%	1.74	1.64	0.168	0.0%
ACI 318:2014	1.26	1.31	0.187	15%	1.64	1.56	0.164	0.0%
EN 1992-1-1:2004	1.09	1.08	0.121	25%	1.46	1.46	0.175	0.0%
FIB MC 2010 I	1.81	1.73	0.153	0.0%	2.11	2.11	0.175	0.0%
FIB MC 2010 II	1.28	1.29	0.127	5.0%	1.85	1.85	0.184	0.0%
FIB MC 2010 III	—	—	—	—	1.50	1.46	0.177	0.0%

* More simplified formula of V_c; I, II, III: Model or level of approximation.

procedure leads to the greatest number of $V_u/V_R < 1$, but replacement of the coefficient 0.18 by 0.15, coefficient previously suggested [26], would lead to no value of $V_u/V_R < 1$. The ACI 318:2014 [8] formulas may provide unsafe values of shear strength for beams with smaller values of ρ and greater effective depth, while the Level I Approximation of the FIB MC 2010 [7] procedure is quite conservative, giving V_R of about 20% to 70% lower than Level II.

For the SCC beams with shear reinforcement, the graphics and table 3 show that the values of V_u/V_R have smaller coefficient of

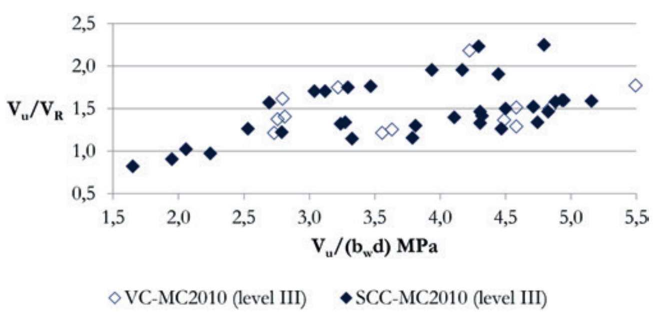

Figure 7b
Values of V_u/V_R of beams with shear reinforcement as a function of $V_u/(b_wd)$ (Continuation)

variation and are closer to the unit when the Model II of ABNT NBR 6118:2014 is adopted. The procedures of ACI318:2014 and Level III Approximation of the FIB MC 2010 give unsafe values of V_R for the beams with lower values of $\rho_w f_{yw}$ and the most conservative one is the Level I Approximation of the FIB MC 2010. Levels I and II Approximation of the FIB MC 2010 may give V_R quite smaller than Level III, mainly for beams with low $\rho_w f_{yw}$, in which case the V_R calculated according to Level I may be lower than half the one obtained using Level III.

Although the ABNT NBR 6118:2014 procedures lead to values of V_c higher than the procedures of ACI 318:2014 and FIB MC 2010 Level of Approximation III (where $V_R = V_c + V_s$), the analysis of V_R given by ABNT NBR 6118:2014 procedures for beams with lower values of $\rho_w f_{yw}$ is favoured because that code defines higher values of $\rho_{w,min} f_{ywk}$ (see figure 8) and only beams with $\rho_w f_{yw} \geq \rho_{w,min} f_{ywk}$ are considered. The smallest values of V_u/V_R correspond to the beams with smaller $\rho_w f_{yw}$, that are included in the analysis of V_u/V_R relative to the other codes and not in the one relative to ABNT NBR 6118:2014.

5. Conclusions

Although there is evidence that SCC beams tend to have lower shear strength than similar VC beams, table 3 shows that there are

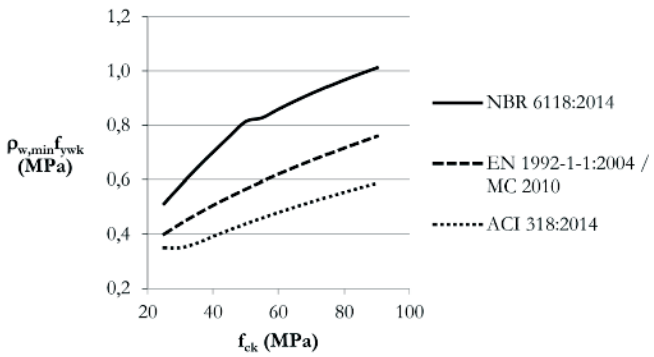

Figure 8
Values of $\rho_{w,min}f_{ywk}$ specified in different codes as a function of f_{ck}

no relevant differences between the statistical data of the groups of SCC and VC beams analysed.

From the limited available test results, it can be inferred that members of the powder type high strength SCC and with minimum or no shear reinforcement, greater height and lower tensile longitudinal reinforcement seem to be more prone to have lower shear strength than similar VC members and, apart from those specific uncommon cases, the code procedures used here may evaluate the shear strength of SCC members as safely as of VC members.

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