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Validation of CSCT Strain-based Shear Failure Criteria for Prestressed Concrete Members without Shear Reinforcement

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Abstract. The shear provision for members without shear reinforcement in the second generation of Eurocode has been changed to a new set of formulas based on the critical shear crack theory (CSCT). The formula is based on a shear failure criterion originally developed for reinforced concrete members without shear reinforcement. To allow its application as a design code type for formula, the original CSCT failure criterion undergoes several modifications, such that it can be used to verify the shear resistance of prestressed members as well. Since the new Eurocode shear provision will be applied to design and assess prestressed concrete members in Europe and many other countries in the world, it is important to extensively validate this model. This paper presents a validation study of three different variations of the CSCT strain-based failure criteria, including the one eventually employed in the second generation Eurocode shear provision, using the ACI-DAfStb shear database. The results are also compared with the current Eurocode shear provisions. The second generation Eurocode shear formula appears to be able to determine the shear resistance more accurately than the current one, even for prestressed concrete members without shear reinforcement while it was not actually developed for this. However, Annex I.8 shear formula may lead to an overestimation of the shear resistance for higher values of the effective span to depth ratio (a_{cs}/d).

Keywords: second generation Eurocode, critical shear crack theory, shear failure criteria, prestressed concrete, ACI-DAfStb database

1 Introduction

The shear strength formula for concrete members without shear reinforcement presented in the current Eurocode is based on an empirical expression derived from experimental research conducted by Zsutty in 1968 [1]. Although this formula has been used since the early 2000s after being incorporated into the Eurocode [2], it has been criticized for not considering key shear resistance factors such as size effect, the shear slenderness and aggregate interlock [3]. Therefore, the TG4 subcommittee has adopted a new expression based on the critical shear crack theory (CSCT) for concrete members without shear reinforcement for the second generation Eurocode [3, 4]. The new shear formula can directly account for size effects and aggregate size, and consistently consider the influence of the M/V ratio on shear resistance. For prestressed elements, the model takes into account not only the effect of normal force but also the effects of the eccentricity of the prestressed strands. The CSCT formula is a failure criterion originally developed for the purpose of shear and punching shear resistance of reinforced concrete members without shear reinforcement [5], and has been extensively verified. In the process of incorporating it into the second generation Eurocode, several modifications were made to the CSCT original failure criterion, including longitudinal strain and effective shear span, to also enable its use in verifying the shear resistance of prestressed concrete members [3, 6]. The new Eurocode shear provision will be applied for a long time in the design and assessment of concrete members in Europe and many other countries around the world, making extensive validation of this expression very important. Therefore, this study presents a validation research on the shear provisions of the current Eurocode and the second generation Eurocode, including several CSCT failure criteria, using the ACI-DAfStb shear database [7, 8] for prestressed concrete members without shear reinforcement that require further verification.

2 Eurocode 2 Shear Formulas

2.1 Current Eurocode 2 (EN 1992-1-:2002)

According to the EN 1992-1-1 [2] the shear capacity of concrete members without shear reinforcement is determined as:

$$V_{\mathrm{Rd,c}} = \left[C_{\mathrm{Rd,c}} \cdot k \cdot \left(100 \cdot \rho_{\mathrm{l}} \cdot f_{\mathrm{ck}}\right)^{1/3} + k_1 \cdot \sigma_{\mathrm{cp}} \right] \cdot b_{\mathrm{w}} \cdot d \tag{1}$$

where $C_{\mathrm{Rd,c}}$ is the constant value equal to $0.18/\gamma_{\mathrm{c}}$, γ_{c} is the partial factor for concrete, k_1 is a constant value equal to 0.15, f_{ck} is the characteristic compressive strength of concrete measured in cylinder, b_{w} is the smallest width of the cross-section in the tensile area and d is the effective depth of the member. k, ρ_{l}, and σ_{cp} are the size factor, the longitudinal reinforcement ratio, and the axial stress, respectively, and their expressions are as follows:

$$\begin{aligned} k &= 1 + \sqrt{\frac{200}{d}} \leq 2.0 \\ \rho_{\mathrm{l}} &= \frac{A_{\mathrm{sl}}}{b_{\mathrm{w}} d} \leq 0.02 \\ \sigma_{\mathrm{cp}} &= \frac{N_{\mathrm{Ed}}}{A_{\mathrm{c}}} < 0.2 f_{\mathrm{cd}} \end{aligned} \tag{2}$$

where A_{sl} is the area of tensile reinforcement, A_{c} is the area of concrete cross section, N_{Ed} is the axial force ($N_{\mathrm{Ed}} > 0$ for compression) and f_{cd} is the design value of concrete compressive strength.

2.2 CSCT failure criterions

CSCT assumes that deformation is localized due to a critical shear crack occurring along the web of concrete members without shear reinforcement and presents the shear capacity ($V_{\mathrm{R,c}}$) of the member as a function of concrete compressive strength, the displacement of crack lips ($\varepsilon \cdot d$), and the roughness of the crack surface (d_{g} and d_{g0}) as follows [3, 5, 6, 9-11]:

$$\frac{V_{\mathrm{R,c}}}{\sqrt{f_{\mathrm{c}}} \cdot b \cdot d} = \frac{1}{3} \cdot \frac{1}{1 + 120 \cdot \dfrac{\varepsilon \cdot d}{d_{\mathrm{g}} + d_{\mathrm{g0}}}} \tag{3}$$

where f_{c} is the compressive strength of concrete, ε refers to the longitudinal strain at a depth of $0.6 \cdot d$ from the compression fiber, reflecting the width of the critical shear crack along with the effective depth of the section. d_{g} and d_{g0} are coefficients considering aggregate interlock, representing the aggregate size and reference value of the aggregate size (=16 mm), respectively.

This formula was originally developed to calculate the shear and punching shear resistance of one-way and two-way slabs and is used to calculate the punching shear strength in a closed-form manner combined with the load-deformation relationship as a general CSCT failure criterion [3, 5, 6]. This closed-form expression is very convenient to use for design and evaluation purposes. Therefore, in FprEN 1992-1-1:2023 (the Second generation Eurocodes for the design of concrete structures), for the purpose of assessment of existing structures, longitudinal strain (ε) and safety factors in Eq. (3) are partially modified and considered in Appendix I.8 as follows:

$$\tau_{\mathrm{Rd,c}} = 0.33\frac{\gamma_{\mathrm{def}}^{2/3}}{\gamma_{\mathrm{v}}}\frac{\sqrt{f_{\mathrm{c}}}}{1+24\cdot\gamma_{\mathrm{def}}\cdot\varepsilon_{\mathrm{v}}\cdot\dfrac{d}{d_{\mathrm{dg}}}} \tag{4}$$

where ε_{v} is the strain in the longitudinal tensile reinforcement and γ_{def} is a partial safety factor which covers the uncertainties related to the calculation of the deformation (the recommend value is 1.33).

Meanwhile, in the original CSCT failure criterion, Eq. (3) uses the longitudinal strain (ε) at a depth of $0.6\cdot d$ from the compression fiber. Instead of this strain, the strain in the longitudinal tensile reinforcement derived by assuming the depth of the compression zone as $0.35\cdot d$ can be applied to simplify as follows [9]:

$$\frac{V_{\mathrm{R,c}}}{\sqrt{f_{\mathrm{c}}}\cdot b_{\mathrm{w}}\cdot d} = \frac{0.3}{1+\varepsilon_{\mathrm{v}}\cdot d\cdot k_{\mathrm{dg}}} \tag{5}$$

where ε_v is the strain of the longitudinal tensile reinforcement, k_{dg} accounts for the concrete type and can be calculated as $48/d_{\mathrm{dg}}$, d_{dg} refers to the average roughness dimension of the critical shear crack, and is determined as the sum of the smallest value of upper sieve size D in an aggregate (D_{lower}) and 16 mm.

Additionally, to facilitate the application in the design of new structures, Eq. (5) has been simplified by further considering a coefficient (k) that accounts for the influence of the inclination of the critical shear crack, leading to the derivation of a general failure criterion (i.e., CSCT power-law) as follows [4, 6]:

$$\begin{gathered} V_{\mathrm{R,c}} = k\left(\frac{f_{\mathrm{c}}\cdot d_{\mathrm{dg}}}{\varepsilon_{\mathrm{v}}\cdot d}\right)^{1/2} b_{\mathrm{w}}\cdot d \\ k = 0.015\left(\frac{a_{\mathrm{cs}}}{d}\right)^{1/4} \end{gathered} \tag{6}$$

where k accounts for the influence of the inclination of the critical shear crack, which is defined as a function of a_{cs}/d in which a_{cs} is the effective shear span at control section and can calculated as $|M_{\mathrm{E}}/V_{\mathrm{E}}|$.

Fig. 1 shows a comparison between the three failure criteria of CSCT and Annex I.8 shear formula [Eq. (4)]. However, since the CSCT original failure criterion [Eq. (3)] considers the strain (ε) at the control depth rather than the strain in the tensile reinforcement (ε_{v}), it assumes ε as $0.41\cdot\varepsilon_{\mathrm{v}}$ for comparison with other failure criteria and applies it accordingly. Additionally, as the CSCT power-law failure criterion [Eq. (6)] shows different level of failure criteria depending on the effective shear span (a_{cs}), it has been applied assuming a_{cs}/d as 4. As shown in the figure, Annex I.8 shear formula represents the highest level of failure criteria, while the CSCT hyperbolic-law and power-law failure criteria are generally more conservative compared to Annex I.8 shear formula. Moreover, the three CSCT failure criteria show relatively similar levels of failure criteria, but the CSCT power-law failure criterion, as finally included in FprEN 1992-1-1, presents that the difference between the two criteria increases as $\varepsilon_{\mathrm{v}}\cdot d$ approaches 0 or exceeds 3. This indicates that within this range, FprEN 1992-1-1 may provide somewhat unconservative results compared to the other two failure criteria.

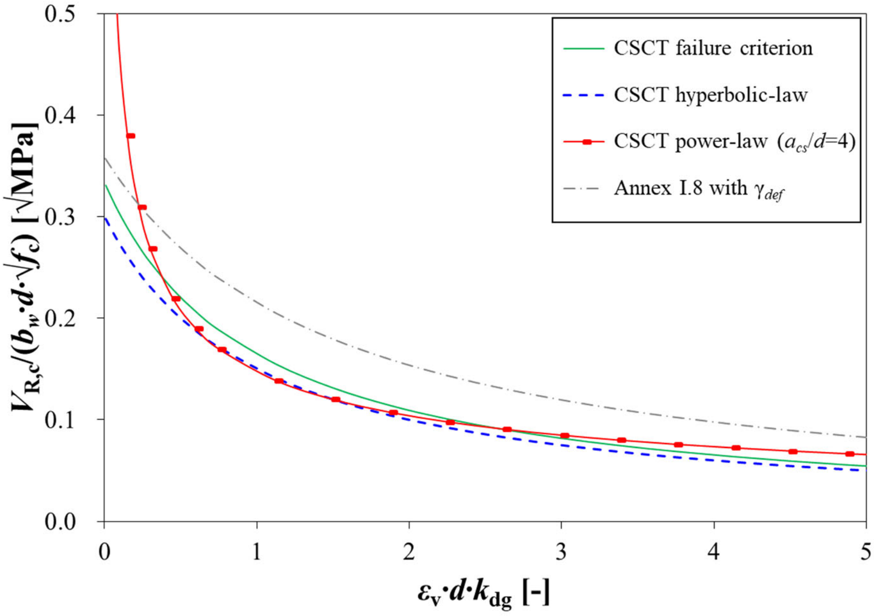


Fig. 1. Comparison of CSCT failure criterion

2.3 FprEN 1992-1-1 shear formula

The New Eurocode shear formula presented in the second generation Eurocode is derived based on the CSCT power-law failure criterion. From Eq. (6) of the CSCT power-law failure criterion, the strain of the longitudinal tensile reinforcement (ε_v) can be calculated according to a linear relationship with the acting bending moment as follows [3, 4]:

$$\varepsilon_v = \frac{M_E}{A_{sl} \cdot E_s \cdot z} = \frac{M_E}{\rho_l \cdot b_w \cdot d \cdot E_s \cdot z} = \frac{v_E \cdot a_{cs}}{\rho_l \cdot E_s \cdot z} \tag{7}$$

where M_E and V_E are the acting bending moment and shear force at the control section, respectively, E_s is the modulus of elasticity of the longitudinal reinforcement, and z is the effective lever arm of the longitudinal internal forces. Additionally, as shown in Fig. 2, when an axial force such as a prestressing force (N_E) acts on a concrete member without shear reinforcement, the strain of the longitudinal tensile reinforcement (ε_v) can be represented as follows.

$$\varepsilon_v = \frac{|M_E| + N_E \cdot e_c}{A_{sl} \cdot E_s \cdot z} = \left(1 + \frac{N_E \cdot e_c}{|M_E|}\right)\frac{|M_E|}{A_{sl} \cdot E_s \cdot z} = k_{vp}\frac{|V_E| \cdot a_{cs}}{A_{sl} \cdot E_s \cdot z} \tag{8}$$

where e_c is the distance from centroid of the cross section to the resultant of the normal compressive stresses, and k_{vp} is the coefficient to take into account the influence of axial forces on the shear resistance.

In the shear design of new concrete members, the failure criterion can determine the required shear strength through the relationship with the load-deformation relationship ($V_E \leq V_{R,c}$). However, an iterative process is needed to derive the actual shear strength where V_E equals $V_{R,c}$. If the strain of the longitudinal tensile reinforcement (ε_v) in Eq. (6) is replaced with Eq. (8), then the $V_{R,c}$ presented in FprEN 1992-1-1 [3, 4] can be rewritten as follows.

$$
\begin{aligned}
V_{\mathrm{R,c}} &= \frac{0.66}{\gamma_{\mathrm{v}}}\left(\frac{100\cdot\rho_{\mathrm{l}}\cdot f_{\mathrm{c}}\cdot d_{\mathrm{dg}}}{k_{\mathrm{vp}}\sqrt{a_{\mathrm{cs}}d/4}}\right)^{1/3} b_{\mathrm{w}}\cdot z \geq V_{\mathrm{Rc,min}} \\
k_{\mathrm{vp}} &= 1+\frac{N_{\mathrm{E}}}{|V_{\mathrm{E}}|}\frac{d}{3a_{\mathrm{cs}}} \geq 0.1
\end{aligned} \tag{9}
$$

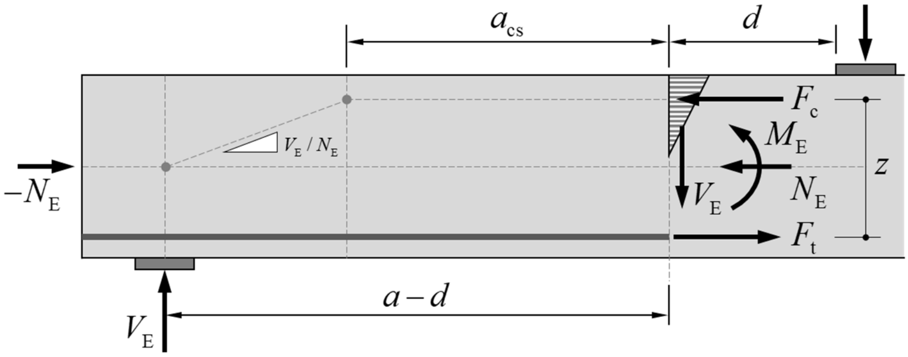


Fig. 2. Equilibrium of internal forces with centroid axial force.

FprEN 1992-1-1 shear formula [Eq.] allows for the determination of shear resistance by considering V_{E} and N_{E} at the control section and the effective shear span, using the coefficient k_{vp}. Fig. 3 illustrates the CSCT power-law failure criterion as it varies under specific design conditions according to the effective shear span (a_{cs}). In this figure, the intersection of the CSCT power-law failure criterion [Eq. (6)] and the load-deformation relationship [Eq. (8)] represents the shear resistance value determined by Eq. . According to CSCT power-law failure criterion [Eq. (6)], as a_{cs}/d increases, the level of the failure criterion rises, but the actual shear resistance determined decreases, similar to the trend of the bold solid line connecting the intersections on the load-deformation path. In addition, in prestressed concrete members, i.e., when axial forces are applied, the k_{vp} value decreases, leading to an increase in the determined shear resistance values, but it can be seen that the decrease in shear resistance according to a_{cs}/d increases significantly.

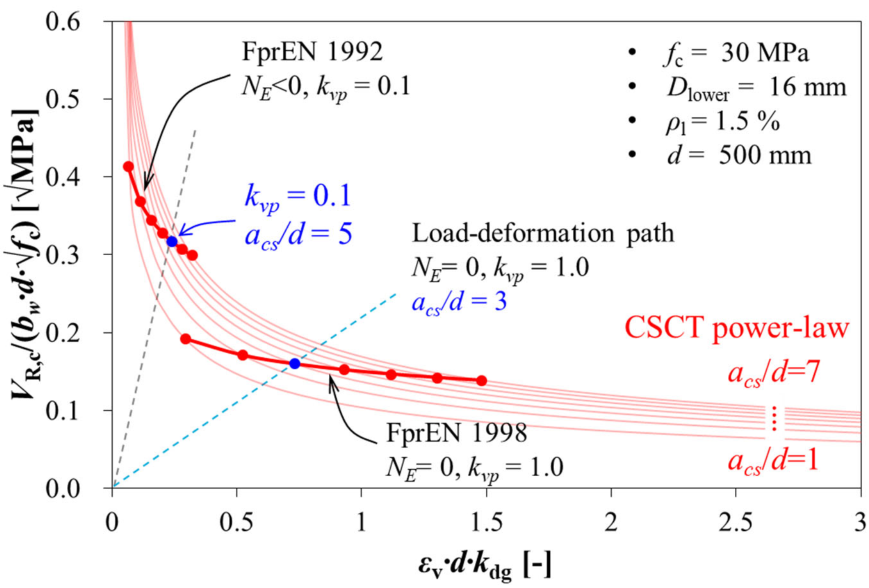


Fig. 3. Shear resistance of FprEN 1992-1-1 for different a_{cs}/d.

3 Shear test Database

In this study, 176 prestressed concrete specimens without shear reinforcement were collected from the ACI-DAfStb database, which has been officially established by joint ACI-ASCE Committee 445 and Deutscher Ausschuss für Stahlbeton [7, 8], to verify the shear regulations of the current Eurocode and second generation Eurocode, including the three failure criteria of CSCT. All collected specimens were tested under simple support conditions and failed in shear.

Table 1 and Fig. 4 show the ranges of major parameters for the collected specimens. The cross-sectional height (h) of the collected specimens mostly ranges from 200 mm to 500 mm, with a maximum height of 1.1 m. The compressive strength (f_c) of concrete ranged from 17.8 MPa to 105.4 MPa, and the ratio of prestressed reinforcement (ρ_p) ranged from 0.2 % to 4.5 %. Regarding the cross-sectional shape, 85 are rectangular cross-sections, and 91 are I-shaped or T-shaped profiled cross-sections. In addition, the shear slenderness (a/d) ranged from 2.4 to 7.3, and there were 36 specimens with a/d of 3 or less, and most specimens had a/d between 3 and 6.

Table 1. Dimensions and material properties of collected test specimens without shear reinforcement.

Reference: Authors (years)	Nr. of specimens: All(a/d>3)	Section type* / PS method**	h (mm)	f_c (MPa)	ρ_p (%)	a/d (-)
Sozen *et al.* (1959)	50(44)	I / Post	305	13.8 - 51.1	0.18 - 0.96	2.7 - 6.5
Arthur *et al.* (1965)	9(4)	I / Pre	229 - 305	27.4 - 49.1	0.25 - 0.34	2.5 - 4.6
Olesen *et al.* (1967)	2(2)	I / Pre	304.8	15.8 - 16.3	0.29 - 0.40	3.5 - 3.6
Kar *et al.* (1968)	29(25)	I or R / Post	203 - 305	24.0 - 34.6	0.45 - 1.25	2.5 - 6.0
Mahgoub *et al.* (1975)	20(17)	I / Pre	300	20.9 - 41.5	0.37 - 0.55	2.7 - 5.9
Elzanaty *et al.* (1985)	14(13)	I / Pre	356 - 457	37.4 - 74.6	0.31 - 0.75	2.8 - 6.8
Evans *et al.* (1985)	7(4)	I or R / Pre	152 - 305	24.6 - 44.3	0.90 - 4.54	2.4 - 4.7
Funakoshi *et al.* (1981, 1982)	6(1)	I / Post	220	36.4 - 79.2	1.60 - 1.64	2.4 - 3.4
Sato *et al.* (1987)	9(8)	R / Post	400 - 450	32.4 - 40.2	0.52 - 1.62	2.8 - 3.3
PWRI (1995)	10(0)	R / Pre	425 - 1100	35.3 - 85.1	0.56 - 1.20	2.8 - 3.0
Ito *et al.* (1996, 1997)	3(1)	R or T / Post	300 - 420	36.2 - 50.1	0.52 - 0.60	2.5 - 3.7
Zink *et al.* (2000)	3(3)	R / Post	400 - 800	85.5 - 101.3	0.23 - 0.65	3.5 - 3.5
Saqan *et al.* (2009)	6(6)	R / Pre	711	47.7 - 49.5	0.17 - 0.45	3.1 - 3.3
Joergensen (2021)	8(8)	R / Post	700	56.3 - 60.7	0.78	4.9 - 7.3

*section type: R = rectangular cross section, I = I-shape cross section, T = T-shape cross section
** PS method: Pre = prestressing strands, Post = post-tensioning strands

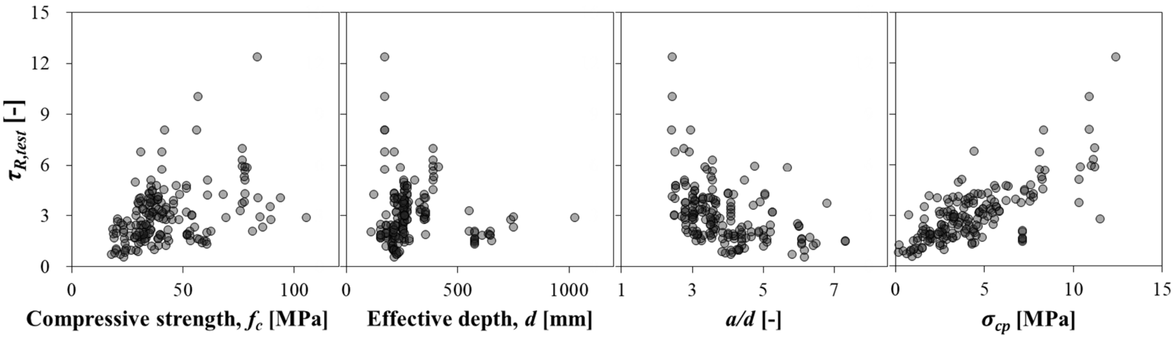


Fig. 4. Data distribution by main parameters for test specimens.

4 Evaluation results

All specimens in the database were classified into rectangular (R-section) and I or T shapes (I/T-section) in terms of cross-sectional shape, and the shear span-to-depth ratio (a/d) was limited to 3. Fig. 5 shows a comparison of CSCT failure criteria and experimental results, where $V_{R,c}$ on the y-axis is a value calculated through experimental results (V_{Rm}) for the test specimens, and the partial factor for shear (γ_v) was not considered. As shown in the figure, the CSCT hyperbolic-law failure criterion presents the lowest level of failure criterion and provides conservative results for almost all experimental results. In order to compare with other failure criteria, the CSCT original failure criterion was applied assuming $0.41 \cdot \varepsilon_v$ instead of the longitudinal strain at the control depth, and as shown in the figure, it shows a failure criterion at a similar level to CSCT hyperbolic-law and power-law. The FprEN 1992-1-1 shear formula is likely to be primarily applied to slabs that do not require shear reinforcement, making specimens with rectangular sections exceeding an a/d ratio of 3 the most critical according to the classified database. For the relevant test specimens, as shown in the figure, it can be seen that both CSCT failure criterion and hyperbolic-law provide conservative results. However, it was found that the Annex I.8 shear formula did not cover a large number of test specimens for the highly important experimental group. The CSCT power-law failure criterion, that is, the FprEN 1992-1-1 shear formula, provides different failure criteria depending on the effective shear span to depth ratio (a_{cs}/d), and the a_{cs}/d values of the specimens differ. Therefore, it is difficult to directly evaluate it from this figure.

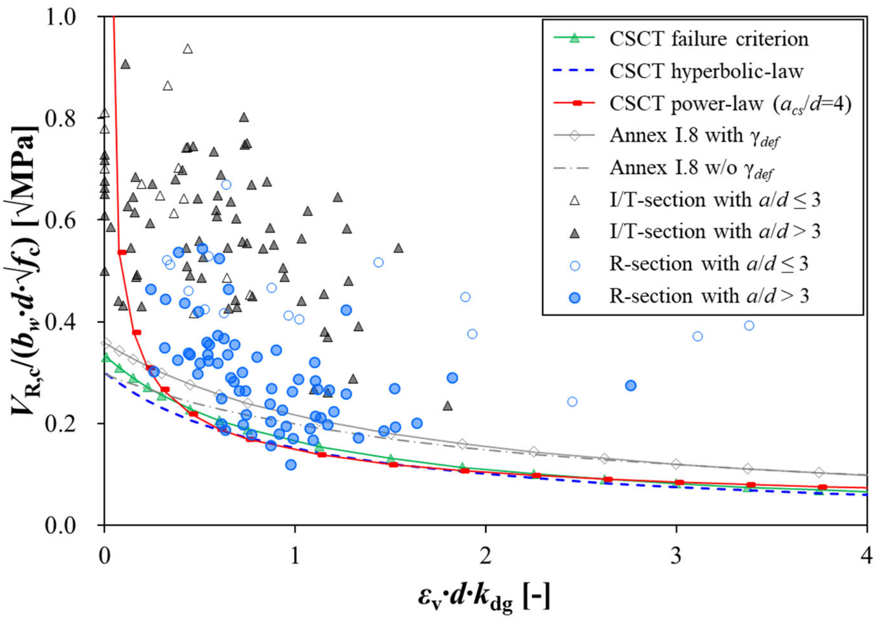
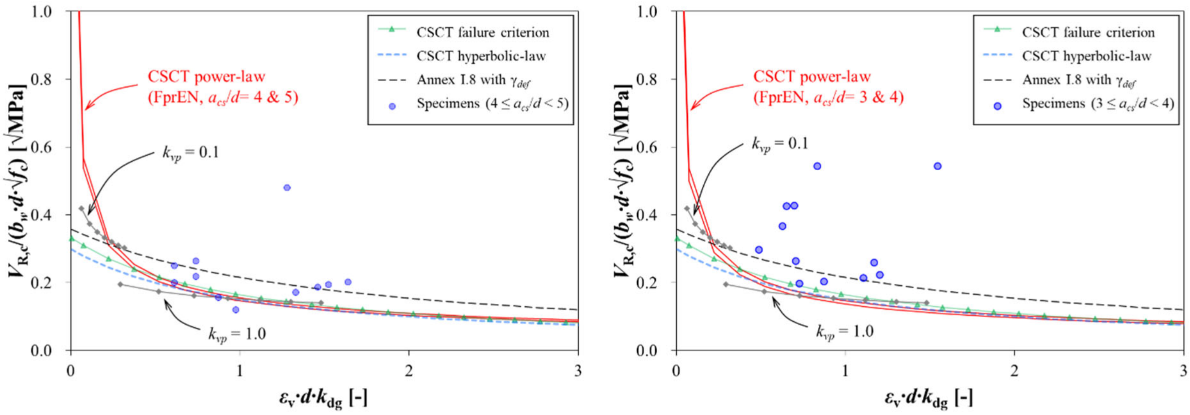
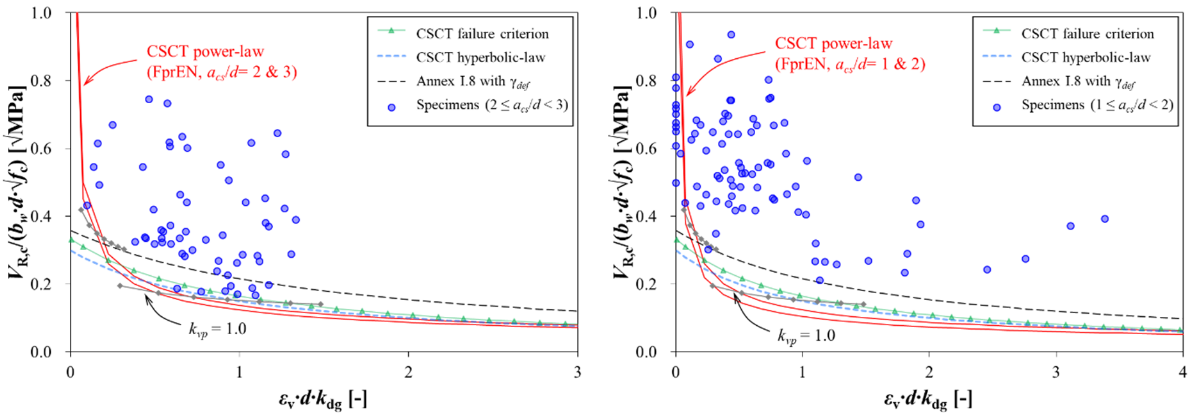


Fig. 5. Comparison between test specimens with failure criteria.

Therefore, as shown in Fig. 6, the specimens from the database were classified into four groups based on the effective shear span to depth ratio (a_{cs}/d) ranging from 1 to 5, for comparison. Fig. 6a shows a comparison of test specimens with a_{cs}/d between 1 and 2, and it can be seen that all failure criteria of CSCT provide conservative results. Fig. 6b, Fig. 6c and Fig. 6d show that, as previously observed in Fig. 5, Annex I.8 shear formula presents somewhat unconservative results for specimens with an a_{cs}/d higher than 2. In the case of the CSCT power-law failure criterion (FprEN 1992-1-1), it is observed that most specimens are provided relatively conservative results. From a_{cs}/d equal to 4, some samples show an overestimation of the shear resistance.



(a) Specimens with $1 \le a_{cs}/d < 2$ (b) Specimens with $2 \le a_{cs}/d < 3$



(c) Specimens with $3 \le a_{cs}/d < 4$ (d) Specimens with $4 \le a_{cs}/d < 5$

Fig. 6. Comparison between test specimens with failure criteria according to a_{cs}/d.

Fig. 7 presents the ratio of experimentally found and calculated shear strength ($V_{\mathrm{Rm,test}}/V_{\mathrm{calc.}}$) calculated according to the Current EC2, FprEN 1992-1-1, and Annex I.8 shear formula for the specimens in the database, plotted against the a_{cs}/d. Table 1 shows the ratio of experimentally found and calculated shear strength ($V_{\mathrm{Rm,test}}/V_{\mathrm{calc.}}$) and the coefficient of variation (CoV) for test specimens with an a/d exceeding 3. Test specimens that experienced shear failure near the supports due to web-shear were excluded from the database. The control section was set at a distance d from the loading point and all resistance factors and strength reduction factors were applied as 1.0 during the strength calculations.

As a result of the evaluation, Current EC2 showed an average of 1.7 and CoV 27% for all specimens with a/d exceeding 3. Although it shows a high accuracy for I/T-shaped section specimens with a CoV of 16%, it still demonstrates low accuracy for rectangular section specimens with a CoV of 23%. Notably, it provides unconservative results for all specimens with an a_{cs}/d exceeding 4.

In the case of Annex I.8, the average is 1.4 and CoV 26% for all specimens with a/d exceeding 3, showing slightly improved accuracy compared to the current EC2. In addition, it shows a high accuracy with a CoV of 17.6% for specimens with a rectangular cross-section compared to specimens with an I/T-shaped cross-section. However, it can be confirmed that specimens with a_{cs}/d greater than 2.5 show unconservative results. FprEN 1992-1-1 shear formula shows excellent accuracy compared to other shear formulas, with an average of 1.4 and CoV of 17.6% for all specimens with a/d exceeding 3. Particularly for rectangular sections, it exhibits very good results with a CoV of 14.7%. Moreover, it provides conservative results across all ranges of a_{cs}/d.

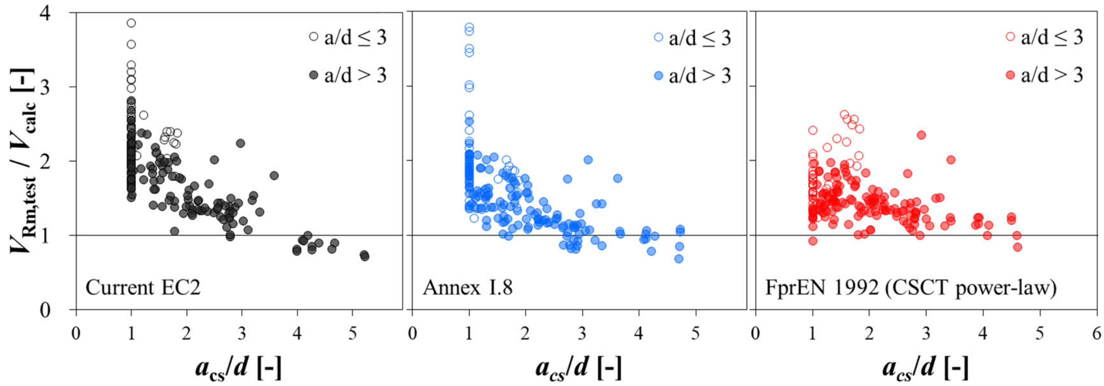


Fig. 7. Ratio of experimental to calculated shear resistance as a function of the effective shear span to depth ratio (a_{cs}/d) for Current EC2, Annex I.8, and FprEN 1992-1-1.

Table 2. Comparison of the calculated shear resistance

		Current EC2	Annex I.8	FprEN 1992-1-1
$a/d > 3$ All section	AVG.	1.67	1.40	1.41
	STD	0.45	0.37	0.25
	CoV	0.27	0.26	0.18
$a/d > 3$ I/T-section	AVG.	1.98	1.66	1.48
	STD	0.32	0.29	0.27
	CoV	0.16	0.17	0.18
$a/d > 3$ R-section	AVG.	1.33	1.13	1.34
	STD	0.31	0.20	0.20
	CoV	0.23	0.18	0.15

5 Conclusions

In this study, experimental results for prestressed concrete members without shear reinforcement were collected, and a validation study was conducted for the Current Eurocode 2, the second generation Eurocode and the associated Annex I.8 shear formulas. The conclusions drawn from this study are as follows:

- As a result of evaluating the Current Eurocode 2 shear formula through the database, it showed a higher average shear strength ratio value than the FprEN 1992-1-1 and Annex I.8 shear formulas. The CoV is comparable to Annex I.8 but higher than according to the FprEN 1992-1-1.

- Annex I.8 was written based on the CSCT failure criterion, but the curve appears to have been adjusted. As a result of evaluating Annex I.8 through the database, the average and CoV values were slightly improved compared to the current Eurocode 2, but the shear resistance was increasingly overestimated for specimens for specimen starting at a_{cs}/d equal to 2.

- As a result of comparing the three failure criteria of CSCT, the failure criteria were relatively similar. Additionally, as a result of evaluation using the database, all CSCT failure criteria except for Annex I.8 showed conservative results.

- In other words, the FprEN 1992-1-1 shear formula showed conservative results in all test specimens, and for prestressed concrete members with a/d exceeding 3 and a rectangular cross-section, it showed a a high accuracy with CoV of 14.7%.

- For all specimens with small k_{vp} or a_{cs}/d, all shear formulas showed conservative results, but as a_{cs}/d increased, the degree of conservatism tended to gradually decrease. In particular, cases where a_{cs}/d is 4 or more require detailed review.

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