Chapter 6

Proposal for a new shear design method

The behaviour of beams failing in shear has been studied in the previous chapters, with special attention paid to high-strength concrete beams. Some aspects not usually considered have been highlighted, such as the dependence of the size effect on the concrete compressive strength or the non-linear proportionality between stirrups and shear strength. Furthermore, procedures used for calculating the ultimate shear strength from five different codes of practice have been evaluated. In this chapter, a new shear design method based on the observed behaviour is proposed, and an effort is made to keep it simple enough to make it suitable for implementation in a code of practice.

6.1 Beams without web reinforcement

6.1.1 Summary of the observed behaviour

The main conclusions drawn from the observed behaviour of beams failing in shear, that form the basis for our two proposals are that:

- The EHE procedure generally shows a satisfactory correlation for normal-strength concrete beams. However, the correlation can be improved with minor changes.

- The predictions made by the EHE shear procedure are unconservative for members with an effective depth under 100 mm (Table 5.4). The benefit of the size effect should be limited for small members.

- The size effect is related to the concrete compressive strength. For deep beams, the benefit of a higher concrete compressive strength is outweighed by the loss caused by the size effect (Figure 5.20).

- The size effect is also related to the maximum spacing between layers of longitudinal reinforcement.

- The influence of the longitudinal reinforcement is greater than that proposed by the EHE procedure. It would not be necessary to limit its value to 2% for high-strength concrete beams (Figure 5.21).

- The shear span-to-depth ratio, a/d, influences the failure shear strength even for beams with an a/d greater than 2.5 (Figure 5.22). The AASHTO LRFD Specification takes the bending moment into account to calculate the failure shear strength. Nevertheless, the equations proposed will predict a conservative result, since they do not take the shear span-to-depth ratio into consideration.

6.1.2 Shear design method

For members without web reinforcement, the following equation is proposed, directly derived from the analyses carried out in Chapter 5:

$$V_c = \left[0.13\xi\left(100\rho_s\right)^{1/2} f_c^{1/3}\right] b_w d \qquad (6.1)$$

where

$$\xi = \left(\frac{135000 \cdot f_c^{-1,1}}{s_x} \right)^{0.25\left(1+\frac{f_c-25}{75}\right)} \leq 2.75,$$ is the size effect with $f_c \geq 25\ MPa$,

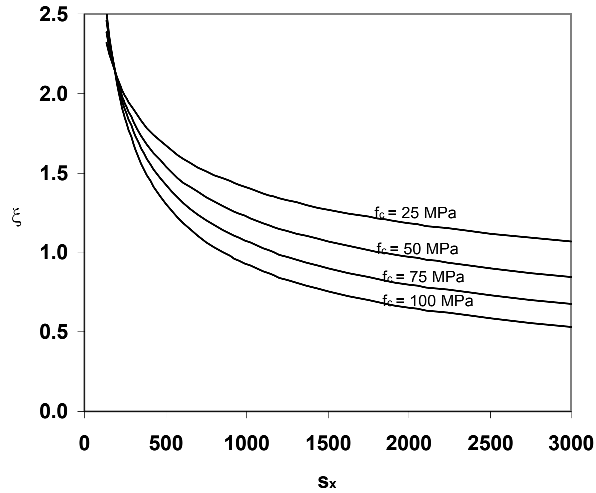

Figure 6.1: Proposed size effect in function of the concrete compressive strength.

s_x is whichever is smaller, d_v or the vertical distance between longitudinal distributed reinforcement as indicated in Figure 6.2,

d is the effective depth in mm,

d_v is the mechanical depth taken to be $0.9 \cdot d$,

$\rho_s = \frac{A_{sl}}{b_w d} \leq 0.02\left(1 + \frac{f_c}{100}\right)$ is the amount of longitudinal reinforcement,

$f_c \leq 100\ MPa$ and

b_w is the smallest width of the cross-section area in mm.

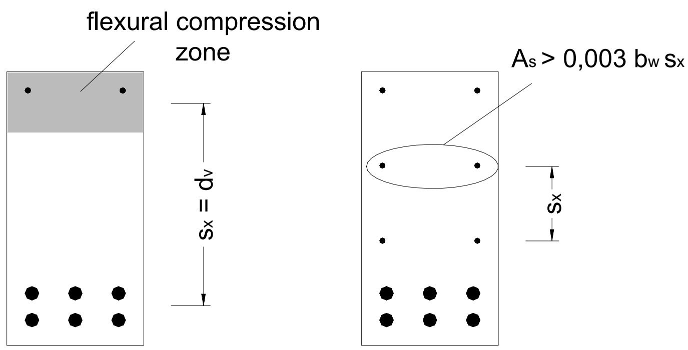

Figure 6.2: Value of s_x for members without web reinforcement.

Equation 6.1 does not take the concrete safety factor into account. To consider it, the first constant should be modified, resulting in the following expression:

$$V_c = \left[0.10\xi(100\rho_s)^{1/2} f_c^{1/3}\right] b_w d \qquad (6.2)$$

6.1.3 Simplified shear design method

Including the size effect in equations 6.1 and 6.2, which was linearly derived in §5.4.3 from equations developed by Fujita et al. (2002) results in equation that are probably too complex to be implemented in a code of practice. For this reason, a simplified shear design method is proposed.

The simplified shear design method adopts a size effect term similar to the EHE one, and it limits the concrete compressive strength to 60 MPa to keep from being unconservative for deep high-strength concrete beams.

$$V_c = \left[0.225\xi(100\rho_s)^{1/2} f_c^{0.2}\right] b_w d \qquad (6.3)$$

where

$\xi = 1 + \sqrt{\frac{200}{s_x}} \leq 2.75$ is the size effect,

s_x is whichever is smaller, d_v or the vertical distance between longitudinal distributed reinforcement as indicated in Figure 6.2,

d_v is the mechanical depth, taken to be $0.9 \cdot d$,

d is the effective depth in mm,

$\rho_s = \frac{A_{sl}}{b_w d} \leq 0.02\left(1 + \frac{f_c}{100}\right)$ is the amount of longitudinal reinforcement,

$f_c \leq 60\ MPa$ and

b_w is the smallest width of the cross-section area in mm.

Equation 6.3 does not take the concrete safety factor into account. If we factor it in, the resulting equation is:

$$V_c = \left\lfloor 0.18\xi(100\rho_s)^{1/2} f_c^{0.2} \right\rfloor b_w d \qquad (6.4)$$

6.1.4 Verification of the proposed equation using the experimental database

In order to compare the proposed equations with the code procedures described in Chapter 2, the statistical analyses presented in Chapter 5 are here compared with the predictions made by equations 6.1 and 6.3.

Table 6.1 compares the predicted values with the empirical results for the entire database of members without web reinforcement. Proposed equations 6.1 and 6.3 offer very similar results, although the first one gives a slightly better coefficient of variation than the second procedure. Nevertheless, both equations correlate better with the empirical results than do the other procedures. §5.2 explains the meaning of each parameter in Table 6.1.

Procedure	EHE	EC-2	AASHTO	ACI 11-5	ACI 11-3	Eq. 6.1	Eq. 6.3
Average	1.23	1.02	1.28	1.28	1.29	1.15	1.13
Median	1.16	0.99	1.25	1.27	1.25	1.14	1.12
Standard deviation	0.29	0.23	0.22	0.34	0.40	0.18	0.19
COV (%)	23.61	22.03	16.80	26.36	31.21	15.73	16.42
$COV_{LOW\ 50\%}$ (%)	14.83	17.16	12.56	25.17	27.43	13.92	14.30
$COV_{HIGH\ 50\%}$ (%)	33.17	27.71	21.34	27.67	36.59	19.10	17.38
Minimum	0.78	0.57	0.86	0.47	0.42	0.73	0.78
$(V_{test} / V_{pred})_{1\%}$	0.76	0.60	0.89	0.54	0.46	0.76	0.76
Maximum	2.35	1.78	2.14	2.34	2.47	1.69	1.85
$(V_{test} / V_{pred})_{99\%}$	2.04	1.62	1.86	2.08	2.31	1.61	1.60

Table 6.1: Verification of proposed shear procedures using the entire database for beams without web reinforcement.

The Total Demerit Point classification is given in Table 6.2. It can be seen that the simplified shear design procedure (equation 6.3) obtains a vaguely better score, as the general design procedure is slightly more conservative.

$\frac{V_{test}}{V_{pred}}$	Classification	DP	EHE	EC-2	AASHTO	ACI 11-5	ACI 11-3	Eq. 6.1	Eq. 6.3
< 0.50	Extremely dangerous	10	0	0	0	1	2	0	0
0.50 - 0.65	Dangerous	5	0	3	0	2	2	0	0
0.65 – 0.85	Low safety	2	5	16	0	7	9	4	3
0.85 – 1.30	Appropriate safety	0	67	69	61	48	44	76	79
1.30 – 2.00	Conservative	1	26	12	39	40	37	20	18
> 2.00	Extremely conservative	2	2	0	1	2	6	0	0
Total Demerit Points			**40**	**59**	**41**	**78**	**97**	**28**	**24**

Table 6.2: Comparison of Demerit Points classifications for beams without web reinforcement.

The results of the partial set analyses are given in Table 6.3. The proposed methods represent an improvement in terms of the coefficient of variation over all the other code procedures and all partial datasets except for the AASHTO procedure. This code presents the best correlation for two sets: beams with a low amount of longitudinal reinforcement, $\rho_l \leq 1\%$, and for normal-strength concrete beams.

Proposed equation 6.1 gives an almost identical correlation for normal-strength and high-strength concrete beams, with average V_{test}/V_{pred} ratios of 1.14 and 1.16, and coefficients of variation of 15.96% and 15.53% respectively.

The distribution of V_{test}/V_{pred} ratio percentages is plotted in Figure 6.3, and can be compared to the graphs given in Chapter 5 for the different code procedures. Both proposed codes show a higher percentage of predictions in the 1.00-1.30 band than any other code procedure. It can also been seen that, although equation 6.3 gets a better Demerit Point score, the proposed equation 6.1 shows a better distribution, with 60% of the beams falling in the 1.00-1.30 band.

Beam specimens	nº beams	Average V_{test} / V_{pred}							COV V_{test} / V_{pred}						
		EHE	EC-2	AASHTO	ACI 11-5	ACI 11-3	Eq. 6.1	Eq. 6.3	EHE	EC-2	AASHTO	ACI 11-5	ACI 11-3	Eq. 6.1	Eq. 6.3
All	193	1.23	1.02	1.28	1.28	1.29	1.15	1.13	23.61	22.03	16.80	26.36	31.21	15.73	16.42
d ≥ 900 mm	18	1.03	0.83	1.11	0.78	0.76	1.28	1.07	13.63	18.84	14.46	24.75	28.49	10.65	11.49
d ≤ 100 mm	12	0.98	1.18	1.42	1.61	1.58	1.11	1.07	8.09	10.59	10.57	10.62	10.65	10.53	9.16
$\rho_l \leq 1\%$	37	1.09	0.89	1.16	0.97	0.90	1.27	1.17	14.49	17.40	10.13	23.81	25.51	12.96	12.68
f_c > 50 MPa	93	1.31	1.03	1.29	1.30	1.32	1.14	1.17	26.14	25.81	20.10	29.19	34.23	15.96	17.32
$f_c \leq$ 50 MPa	100	1.15	1.01	1.28	1.27	1.27	1.16	1.09	17.86	17.58	12.99	23.36	27.79	15.53	14.69
ρ_l > 2% f_c > 50 MPa	55	1.46	1.15	1.38	1.47	1.54	1.13	1.20	23.54	23.24	19.85	22.63	26.27	17.47	19.59
ρ_l > 2% $f_c \leq$ 50 MPa	34	1.31	1.10	1.35	1.43	1.52	1.15	1.07	16.50	16.10	13.26	17.22	20.68	15.23	16.49

Table 6.3: Verification of different code procedures using partial sets of the database for beams without web reinforcement.

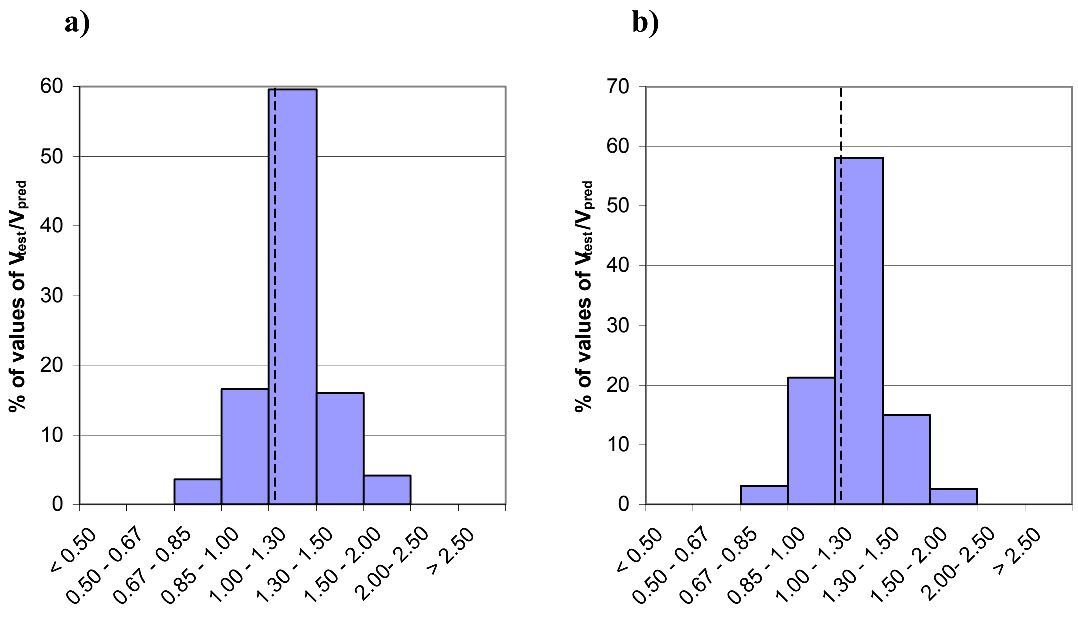

Figure 6.3: Correlation of the proposed equations with empirical tests for beams without web reinforcement. a) Equation 6.1. b) Equation 6.3.

6.1.5 Verification of the proposed equation for elements with longitudinal distributed reinforcement

As was stated in Chapter 2, Collins and Kuchma (1999) carried out an experimental campaign to evaluate the parameters influencing on the size effect. They concluded that it was related to the maximum spacing between the layers of longitudinal reinforcement rather than the overall member depth.

Test beams with longitudinal reinforcement were not included in the database presented in §5.1, as only the AASHTO LRFD Specifications take this effect into account, and therefore the performance of the other codes' procedures would have been poorer. Table 6.4 gives the geometrical parameters of the beams with longitudinal distributed reinforcement, test results, and predictions given by the EHE Code procedure, the AASHTO LRFD Specifications, and the proposed equations. All beams in Table 6.4 had a greater amount of longitudinal reinforcement distributed in the web than the minimum amount given in Figure 6.2. All were tested by Collins and Kuchma (1999) except for the last two beams, tested as part of this thesis.

The EHE shear procedure does not take the effect of distributed longitudinal reinforcement into consideration, and is excessively conservative for the 17 beams containing it, with an average V_{test}/V_{pred} ratio of 1.49. The predictions made by the AASHTO procedure improve the correlation, resulting in an average of 1.06. The standard deviation for both codes is 0.16.

Equations 6.1 and 6.3 do take this effect into account and they improve the performance observed for the EHE shear procedure for members without web reinforcement. The average V_{test}/V_{pred} ratio is 1.09 for equation 6.1 and 1.20 for equation 6.3, and their standard deviations are 0.16 and 0.11 respectively.

Beam	f_c MPa	b mm	d mm	a/d	ρ_l	s_x[†]	V_{fai} (KN)	$V_{predicted}$				V_{test} / $V_{predicted}$			
								EHE	LRFD	Eq.6.1	Eq.6.3	EHE	LRFD	Eq.6.1	Eq.6.3
B100D	36	300	925	2.92	0.76	170	320	184	288	225	232	1.74	1.11	1.42	1.38
BND100	37	300	925	2.92	0.76	170	258	185	268	227	234	1.39	0.96	1.14	1.10
BND50	37	300	450	3.00	0.81	85	163	105	141	139	143	1.55	1.15	1.17	1.14
BND25	37	300	225	3.00	0.89	40	112	63	72	75	81	1.78	1.56	1.49	1.38
BHD100	99	300	925	2.92	0.76	170	278	218	345	321	257	1.28	0.81	0.87	1.08
BHD100R	99	300	925	2.92	0.76	170	334	218	345	321	257	1.53	0.97	1.04	1.30
BHD50	99	300	450	3.00	0.81	85	193	123	180	198	157	1.57	1.07	0.98	1.23
BHD50R	99	300	450	3.00	0.81	85	205	123	180	198	157	1.66	1.14	1.04	1.30
BH25D	99	300	225	3.00	0.89	40	111	74	103	104	89	1.50	1.07	1.07	1.24
SE100B-45	50	295	920	2.50	1.03	195	281	222	321	274	273	1.27	0.87	1.02	1.03
SE100B-45-R	50	295	920	2.50	1.03	195	316	222	321	274	273	1.42	0.98	1.15	1.16
SE50B-45	53	169	459	2.72	1.03	195	87	73	87	80	79	1.19	1.00	1.09	1.10
SE100B-83	86	295	920	2.50	1.03	195	365	236	361	328	283	1.55	1.01	1.11	1.29
SE100B-83-R	86	295	920	2.50	1.03	195	364	236	361	328	283	1.54	1.01	1.11	1.29
SE50B-83	91	169	459	2.72	1.03	195	101	76	97	95	81	1.32	1.04	1.06	1.25
H50/5	49.9	200	359	3.01	2.24	110	130	87	110	129	124	1.49	1.18	1.00	1.05
H100/5	87	200	359	3.01	2.24	110	141	93	125	167	129	1.52	1.13	0.85	1.09
								Average				**1.49**	**1.06**	**1.09**	**1.20**
								Standard deviation				**0.16**	**0.16**	**0.16**	**0.11**
								Coefficient of var.				**10.95**	**15.17**	**14.94**	**9.45**

[†] Distance between layers of long. reinforcement

Table 6.4: Summary of predictions by EHE, AASHTO LRFD, equation 6.1 and equation 6.3 for elements with longitudinal distributed reinforcement.

6.2 Beams with web reinforcement

6.2.1 Summary of the observed behaviour

A general shear design method is proposed in this section, as well as two simplified shear procedures based on the following observations for members with web reinforcement:

- The EHE, EC-2, ACI 11-5 and ACI 11-3 shear procedures do not correlate satisfactorily for members with web reinforcement, as was discussed in Chapter 5.

- The AASHTO LRFD shear procedure, based on the modified compression field theory, performs much better than the other current codes do. In this procedure, the angle between the compression struts and the longitudinal axis of the beam, θ, is obtained by compatibility, and it depends on the shear stress and the longitudinal strain of the web.

- The concrete contribution to the shear strength is the vertical component of the shear stress transferred across the crack and therefore depends on the crack width. The greater the amount of shear reinforcement, the lesser crack width, and the larger the concrete contribution will be (§2.3.5).

- The influence of the amount of web reinforcement is not linearly proportional to the failure shear strength (§5.5.3). Truss models, like EC-2, could be unconservative for highly reinforced concrete members, as can be seen in Table 5.4.

- The use of high-strength concrete tends to prevent shear-compression failure and to ensure a diagonal tension failure instead, thus increasing the effectiveness of the shear reinforcement.

- For members with low shear reinforcement, the size effect reduces the shear stress at failure, although most codes do not take it into account for members with stirrups (Figure 5.24).

- An increase in the amount of longitudinal reinforcement produces an increase in shear strength. It would not be necessary to limit its value to 2% as is required by the EHE procedure (Figure 5.26).

6.2.2 General shear design method: procedure and justification

Procedure

For members with web reinforcement, the failure shear strength is given by:

$$V = V_c + V_s \tag{6.5}$$

$$V_c = \left[0.17\xi(100\rho_s)^{1/2} f_c^{0.2} \tau^{1/3}\right] b_w d \tag{6.6}$$

where

$\xi = 1 + \sqrt{\frac{200}{s_x}} \leq 2.75$ is the size effect,

s_x is whichever is smaller, d_v or the vertical distance between longitudinal distributed reinforcement as indicated in Figure 6.2,

d_v is the mechanical depth which can be taken as $0.9 \cdot d$,

d is the effective depth in mm,

$\rho_s = \frac{A_{sl}}{b_w d} \leq 0.04$, is the amount of longitudinal reinforcement,

$f_c \leq 100\ MPa$,

$\tau = \frac{V_d}{b_w\, d_v} \leq 3\ MPa$, and

b_w, the smallest width of the cross-section area in mm.

And,

$$V_s = d_v \frac{A_{sw}}{s} f_{ywd} \cot\theta \tag{6.7}$$

where

A_{sw} is the cross-sectional area of the shear reinforcement.

s is the spacing of the stirrups

f_{ywd} is the design yielding strength of the shear reinforcement, and

θ is the angle of the compression struts, derived as follows:

$$\theta = 20 + 15\,\varepsilon_x + 45\frac{\tau}{f_{ck}} \leq 45° \tag{6.8}$$

where

ε_x is the longitudinal strain in the web, expressed in 1/1000, calculated by the following expression:

$$\varepsilon_x \approx 0.5 \frac{\frac{M_d}{d_v} + V_d}{E_s A_{sl}} \cdot 1000 \leq 1 \tag{6.9}$$

$$\frac{\tau}{f_{ck}} \geq 0.05$$

The expression of the longitudinal strain in the web is a conservative simplification of the real strain. It assumes that in the web the strain is equal to one half the strain in the tension reinforcement, and that the maximum longitudinal strain of the reinforcement is 0.002.

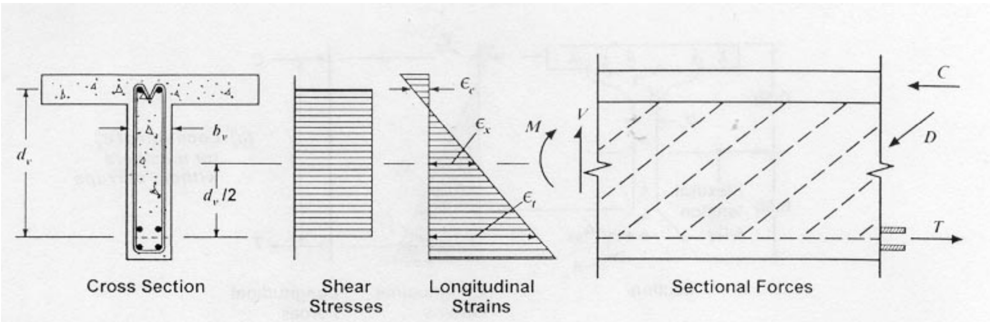

Figure 6.4: Longitudinal strain in the web (from Collins 2001)

Equation 6.6 does not take the concrete safety factor into account. To consider it, the first constant should be modified, resulting in the following expression::

$$V_c = \left[0.14\xi(100\rho_s)^{1/2} f_c^{0.2} \tau^{1/3}\right] b_w d \tag{6.10}$$

Justification

Equation 6.8 was directly derived from the AASHTO LRFD Table given in Figure 2.29, in an attempt to find the simplest equation that still followed the general trend. The proposed value for θ is always conservative compared with the AASHTO predictions. Once the angle was obtained, the steel contribution was able to be determined.

Equation 6.3, from the simplified shear design method for beams without stirrups, was taken to be a good procedure for evaluating the concrete contribution for a beam with only longitudinal reinforcement. An extra term was added to take into account the stirrups' influence on the shear friction. As the amount of transversal reinforcement is unknown during the design process, the new term is a function of the designing shear stress, τ to the power of 1/3. This value of the power and the constant 0.17, used to multiply V_c, were derived empirically to adjust the test beam results.

6.2.3 Simplified shear design method

To apply the procedure presented in 6.2.2 it is necessary to evaluate the shear strength in different sections of the beam, due to the interaction between the bending moment and the shear strength. The simplified shear design method assumes that the longitudinal strain in the web, ε_x, is equal to 1, and therefore that the longitudinal reinforcement yields; this is the worst condition under which to calculate the shear strength. Hence:

$$\theta = 35 + 45 \frac{\tau}{f_{ck}} \leq 45° \tag{6.11}$$

where

$$\frac{\tau}{f_{ck}} \geq 0.05$$

With the value of the angle of the compression struts given by 6.11, the failure shear strength can be calculated using equations 6.5, 6.6 and 6.7.

6.2.4 Simplified shear verification method

To verify the ultimate shear strength of a given section it would be possible to use the expressions given in §6.2.2, although it would be necessary to iterate to find the solution, as V_d is an input to obtain both the concrete and the steel contributions. Moreover, the ultimate shear strength would depend not only on the cross-section of the beam, but also on the bending moment in that section.

The simplified shear verification method estimates τ, so the ultimate shear strength can be calculated from equations 6.5, 6.6 and 6.7, assuming that ε_x = 1:

$$\tau_{est} = 3.5\sqrt{\frac{200}{s_x}\frac{f_{ywd}}{b_w}\left(\frac{A_{sw}}{s}\right)^{0.5}} \quad (6.12)$$

The above estimation is equivalent to a truss model using a variable angle of inclination for the trusses. For small members, such as s_x = 200 mm, the estimated shear failure would be given by a truss model in which cot θ = 3.5. For a bigger beam, for example s_x = 1000 mm, the inclination of the truss would be given by cot θ = 1.57.

6.2.5 Verification of the proposed equation with the experimental database

The proposed equations are compared in Table 6.5 with the database's 123 test beams with shear reinforcement. The three proposed procedures correlate much better with empirical tests than do the EHE, EC-2, ACI 11-5, or ACI 11-3 procedures. For example, the V_{test}/V_{pred} ratio for the current EHE code is 1.64, with a standard deviation of 0.43, while, it is 1.11, with a standard deviation of 0.21 for the general shear design method (§6.2.2). Nevertheless, the AASHTO LRFD shear procedure performs very similarly to the proposed equations.

It can also be seen in Table 6.5, that the two simplified methods are slightly more conservative than the general design method, as they do not take into account the influence of the bending moment, and they assume that the longitudinal rebars yield.

Procedure	EHE	EC-2	AASHTO	ACI 11-5	ACI 11-3	§6.2.2	§6.2.3	§6.2.4
Average	1.64	1.83	1.18	1.36	1.41	1.11	1.17	1.18
Median	1.62	1.72	1.17	1.37	1.42	1.11	1.17	1.19
Standard deviation	0.43	0.74	0.23	0.34	0.38	0.21	0.23	0.22
COV (%)	26.26	40.29	19.23	24.60	26.70	18.77	19.56	18.71
$COV_{LOW\ 50\%}$ (%)	23.61	31.05	17.13	23.60	25.91	17.39	17.98	17.55
$COV_{HIGH\ 50\%}$ (%)	29.40	53.18	21.71	25.52	27.23	20.06	21.07	19.66
Minimum	0.62	0.50	0.69	0.69	0.67	0.67	0.67	0.73
$(V_{test} / V_{pred})_{1\%}$	0.74	0.49	0.71	0.63	0.57	0.66	0.69	0.71
Maximum	3.27	4.85	1.96	2.66	2.83	2.01	2.20	2.14
$(V_{test} / V_{pred})_{99\%}$	2.72	3.83	1.75	2.17	2.31	1.62	1.74	1.73

Table 6.5: Verification of proposed shear procedures for beams with web reinforcement using the entire database.

The Total Demerit Point classification for beams with web reinforcement is given in Table 6.6. The AASHTO procedure obtains the best score, 36 points, followed by the general shear design method which gets 37 points. The two simplified proposed methods score 41 and 39 Demerit Points.

$\frac{V_{test}}{V_{pred}}$	Classification	DP	EHE	EC-2	AASHTO	ACI 11-5	ACI 11-3	§6.2.2	§6.2.3	§6.2.4
< 0.50	Extremely dangerous	10	0	1	0	0	0	0	0	0
0.50 - 0.65	Dangerous	5	1	2	0	0	0	0	0	0
0.65 – 0.85	Low safety	2	2	2	5	6	6	10	9	8
0.85 – 1.30	Appropriate safety	0	14	15	69	36	31	75	69	70
1.30 – 2.00	Conservative	1	69	48	26	56	58	15	21	21
> 2.00	Extremely conservative	2	14	32	0	2	6	1	1	1
Total Demerit Points			**106**	**136**	**36**	**72**	**82**	**37**	**41**	**39**

Table 6.6: Comparison of Demerit Points classification for beams with web reinforcement.

Beam specimens	nº beams	Average V_{test} / V_{pred}								COV V_{test} / V_{pred}							
		EHE	EC-2	LRFD	ACI 11-5	ACI 11-3	§ 6.2.2	§ 6.2.3	§ 6.2.4	EHE	EC-2	LRFD	ACI 11-5	ACI 11-3	§ 6.2.2	§ 6.2.3	§ 6.2.4
All	123	1.64	1.83	1.18	1.36	1.41	1.11	1.17	1.18	26.26	40.29	19.23	24.60	26.70	18.77	19.56	18.71
d ≥ 750 mm	12	1.29	1.34	1.00	0.91	0.88	1.08	1.12	1.14	16.22	24.66	20.38	18.73	20.97	16.83	16.26	15.05
$\rho_w \leq$ 1MPa	93	1.71	2.05	1.18	1.37	1.42	1.12	1.19	1.20	24.60	34.28	19.84	26.01	28.42	18.63	19.17	18.54
$\rho_w >$ 1MPa $\rho_w \leq$ 2MPa	23	1.57	1.28	1.22	1.38	1.42	1.10	1.14	1.17	21.89	22.76	15.89	17.68	18.66	15.68	16.38	15.51
$\rho_w >$ 2MPa	7	0.98	0.78	1.07	1.18	1.23	0.99	1.02	1.06	19.63	19.63	20.91	22.29	23.84	29.50	31.08	29.62
$f_c \leq$ 50 MPa	38	1.47	1.44	1.13	1.30	1.33	1.08	1.13	1.13	22.01	29.70	17.99	22.48	23.71	16.22	15.80	15.79
$f_c >$ 50 MPa	85	1.72	2.01	1.21	1.39	1.44	1.12	1.19	1.21	26.33	38.92	19.51	25.21	27.57	19.65	20.73	19.52
$\rho_l \leq$ 2 %	19	1.24	1.33	0.99	0.98	0.96	1.05	1.08	1.08	17.97	32.24	15.54	21.60	23.37	17.62	17.66	16.02

Table 6.7: Verification of different code procedures using partial sets of the database for beams with web reinforcement.

To check the ability of the proposed methods to predict the shear strength for different types of beams, the results of the partial set analyses are given in Table 6.7. The proposed procedures represent an improvement over the performance of the EHE, EC-2, ACI 11-5, and ACI 11-3 procedures for all the groups of beams studied.

For the biggest members, where d ≥ 750 mm, most codes do not take into account the size effect when stirrups are provided. It was shown in Figure 5.24 that, for members with low shear reinforcement, the size effect causes a reduction in shear strength. The proposed equations correctly reproduce this behaviour, as very little reduction in the overall safety factor (V_{test}/V_{pred}) is observed.

Another set of beams that requires special attention is the group of seven beams with high shear reinforcement ($\rho_w > 2$ MPa). The EC-2 procedure is absolutely unconservative with an average V_{test}/V_{pred} ratio of 0.78. The EHE code, with a ratio of 0.99, is also somewhat unconservative, signifying an approximate 40% reduction in the safety factor with respect to the average coefficient for all the 123 beams. The general shear design method presents the same ratio as the EHE code, but the decrease in the safety factor is only by about 11%. The simplified shear design methods are not unconservative for this set of beams, although the best performance is achieved by the AASHTO procedure.

For beams with a low amount of longitudinal reinforcement, $\rho_l \leq 2\%$, the proposed methods perform satisfactorily, while other codes present slightly unconservative results.

Finally, the distribution in percentages of the V_{test}/V_{pred} ratio is plotted in Figure 6.5, and it can be compared with the Figures given in Chapter 5 for the different code procedures. The three proposed methods present the highest prediction percentage in the 1.00-1.30 band compared with other codes.

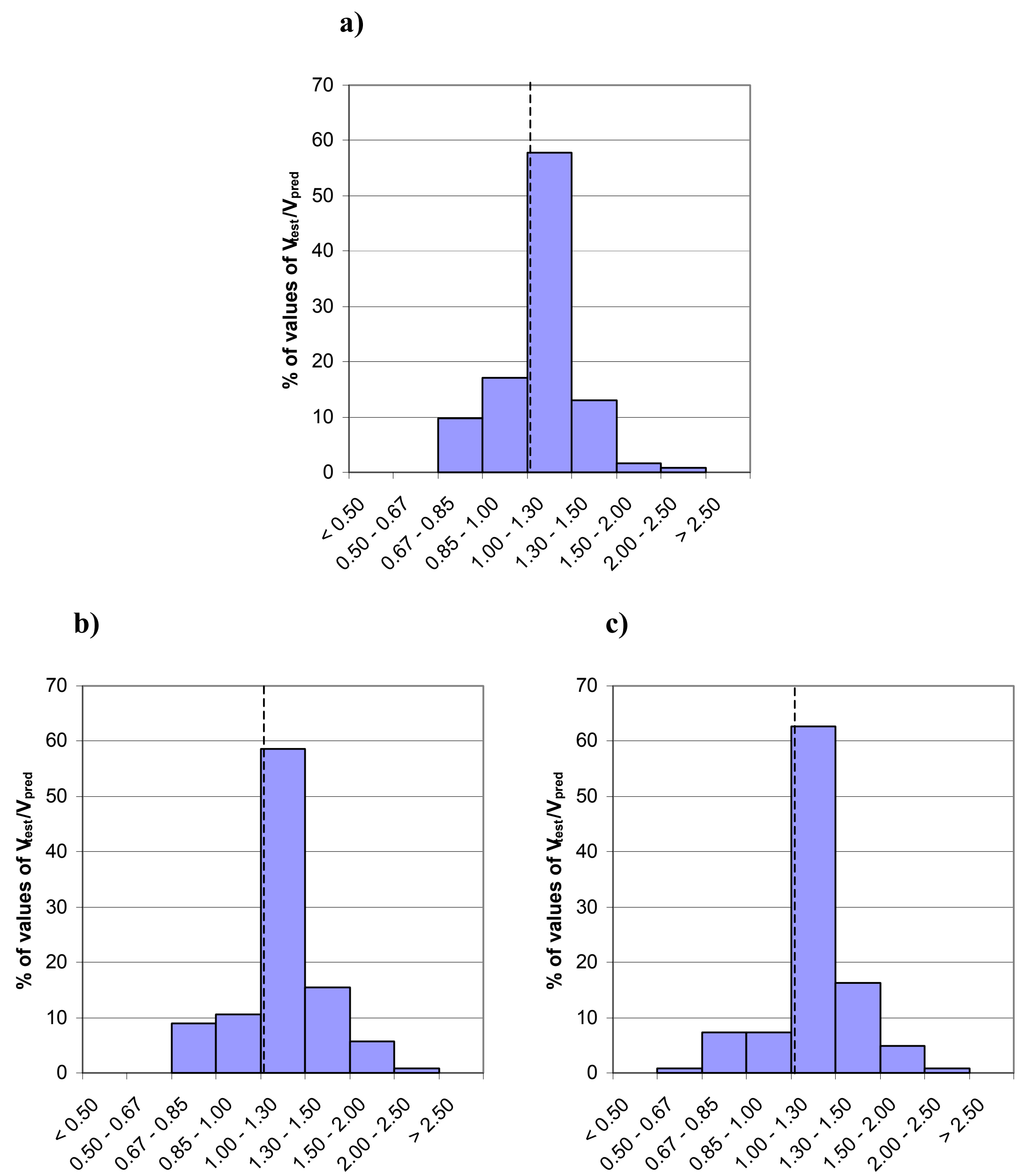

Figure 6.5: Correlation of the proposed equations with empirical tests for beams with web reinforcement. a) Equation §6.2.2. b) Equation §6.2.3. c) Equation §6.2.4

6.2.6 Equivalence between the simplified shear design method and the simplified shear verification method

The simplified shear procedures given in §6.2.3 and §6.2.4 are respectively intended for design and verification. Both procedures assume the longitudinal strain in the web to be equal to 1, and, that the longitudinal reinforcement will yield. In this section it will be shown that the results obtained by the two methods are practically identical, the verification procedure being slightly (1%) more conservative than the design method.

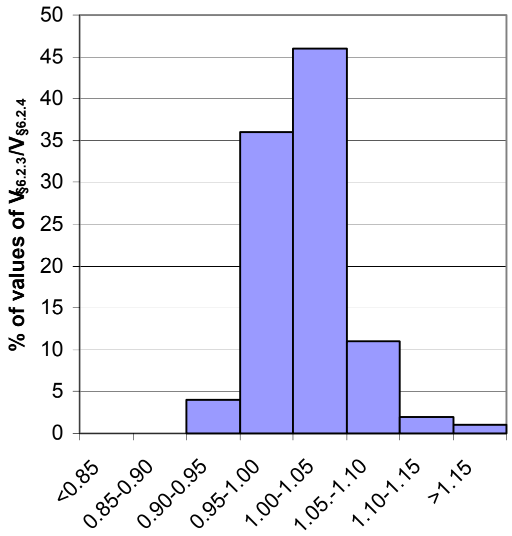

Figure 6.6: Correlation between the simplified shear design method (§6.2.3) *and the simplified shear verification method (*§6.2.4).

The average $V_{§6.2.3}/V_{§6.2.4}$ ratio for the database's 123 test beams with web reinforcement, where $V_{§6.2.3}$ is the shear strength predicted by the simplified shear design method and $V_{§6.2.3}$ is the shear strength predicted by the simplified shear verification method, is equal to 1.01, and its coefficient of variation is 3.85%. The distribution of $V_{§6.2.3}/V_{§6.2.4}$ ratio percentages is given in Figure 6.6. For 82% of the test beams, the value of the ratio falls in the 0.95-1.05 band.

6.3 Comparison of the proposed method with beams tested at the Structural Technology Laboratory.

Table 6.8 summarises the predictions made by the EHE procedure, the 2002 Final Draft of the Eurocode-2, the AASHTO LRFD, ACI 318-99, and proposed general shear design procedures, in addition to the simplified procedures for beams with and without web reinforcement. For beams with stirrups the verification procedure was used.

It can be seen that the proposed equations correlate satisfactorily with the beams tested at the Structural Technology Laboratory of the Technical University of Catalonia, with a coefficient of variation lower than 10%.

Beam	f_c MPa	b mm	d mm	a/d	ρ_w	ρ_l	V_{fai} KN	$V_{predicted}$						V_{test} / $V_{predicted}$					
								EHE	EC	LRFD	ACI	Gen*	Sim+	EHE	EC	LRFD	ACI	Gen	Sim+
H50/1	49.9	200	359	3.01	0	2.24	100	87	110	90	86	90	94	1.15	0.91	1.11	1.16	1.11	1.06
H50/2	49.9	200	353	3.06	0.577	2.28	178	108	91	138	125	150	149	1.65	1.96	1.29	1.42	1.19	1.19
H50/3	49.9	200	351	3.08	1.291	2.29	242	163	203	179	175	207	200	1.48	1.19	1.35	1.38	1.17	1.21
H50/4	49.9	200	351	3.08	1.291	2.99	246	163	203	197	179	228	215	1.51	1.21	1.25	1.37	1.08	1.14
H50/5	49.9	200	359	3.01	0	2.24	130	87	110	102	86	129	124	1.49	1.18	1.27	1.51	1.01	1.05
H60/1	60.8	200	359	3.01	0	2.24	108	93	116	95	95	95	98	1.16	0.93	1.11	1.14	1.14	1.10
H60/2	60.8	200	353	3.06	0.747	2.28	180	124	119	156	145	171	167	1.45	1.51	1.15	1.24	1.05	1.08
H60/3	60.8	200	351	3.08	1.267	2.29	259	160	200	182	180	211	206	1.62	1.30	1.42	1.44	1.23	1.26
H60/4	60.8	200	351	3.08	1.267	2.99	309	160	200	214	184	232	221	1.93	1.55	1.44	1.68	1.33	1.40
H75/1	68.9	200	359	3.01	0	2.24	100	93	145	101	99	98	98	1.08	0.69	0.99	1.01	1.02	1.02
H75/2	68.9	200	353	3.06	0.747	2.28	204	124	119	160	150	174	171	1.65	1.71	1.28	1.36	1.17	1.19
H75/3	68.9	200	351	3.08	1.267	2.29	269	160	200	185	185	214	210	1.68	1.35	1.45	1.45	1.26	1.28
H75/4	68.9	200	351	3.08	1.267	2.99	255	160	200	206	189	236	226	1.59	1.28	1.24	1.35	1.08	1.13
H100/1	87.0	200	359	3.01	0	2.24	118	93	156	110	118	102	98	1.27	0.76	1.07	1.00	1.16	1.20
H100/2	87.0	200	353	3.06	0.906	2.28	226	129	144	175	149	180	174	1.75	1.57	1.29	1.52	1.26	1.30
H100/3	87.0	200	351	3.08	1.291	2.29	254	163	204	192	175	207	200	1.56	1.25	1.32	1.45	1.23	1.27
H100/4	87.0	200	351	3.08	1.291	2.99	267	163	204	215	179	228	215	1.64	1.31	1.24	1.49	1.17	1.24
H100/5	87.0	200	359	3.01	0	2.24	141	93	156	125	118	167	129	1.51	0.90	1.12	1.19	0.84	1.09
* General proposed procedure												**Average**		**1.51**	**1.25**	**1.25**	**1.34**	**1.14**	**1.18**
+ Simplified shear procedure (verification)												**Stand. Deviation**		**0.22**	**0.33**	**0.13**	**0.18**	**0.11**	**0.10**
												COV (%)		**14.8**	**26.7**	**10.5**	**13.7**	**9.98**	**8.67**

Table 6.8: Comparison of the proposed general and simplified shear procedures and current codes with test results of the experimental campaign.