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Shear design procedure for reinforced normal and high-strength concrete beams using artificial neural networks. Part I: beams without stirrups

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Abstract

Over the last decades, a great number of experimental campaigns on the behavior of high- and normal-strength reinforced concrete beams without shear reinforcement failing in shear have been published, and some excellent rational models to explain the physical phenomena have been developed. However, their implementation into design codes still requires considerable simplification. With the aim of taking into account this large amount of information available and to re-evaluate the current codes of practice extensive research was performed. An artificial neural network was developed to predict the shear strength of reinforced beams and, based on its results, a parametric study was carried out to determine the influence of each parameter affecting the failure shear strength of beams without web reinforcement. Finally, new simple expressions are proposed for the design of high-strength and normal-strength reinforced concrete beams without shear reinforcement. The new expressions correlate with the empirical tests better than any current code of practice does.

Keywords: Reinforced concrete; Shear strength; High-strength concrete; Structural design; Size effect; Artificial neural networks

1. Introduction

Although high-strength concrete (HSC) has been increasingly used in the construction industry during the last few years, there are still some concerns about the application of some empirical rules obtained for concrete beams with compressive strengths largely below 50 MPa. An increase in the strength of concrete is directly associated with an improvement in most of its properties, in particular the durability, but this also produces an increase in its brittleness and smoother crack surfaces which affects significantly the shear strength. Moreover, the failure of high-strength concrete beams without web reinforcement occurs suddenly, and some current codes limit the magnitude of compressive strength used in the design formulae to around 60 MPa or 70 MPa to prevent it. Table 1 summarises the shear provisions of different current codes of practice. It is remarkable that most of these codes are not based on any of the excellent rational models that have been developed [5–7], as they need substantial simplification.

The size effect, raised by Kani in 1967 [8], is also related to the concrete compressive strength. Some authors found out that high-strength concrete beam specimens showed a more significant size effect in shear than normal-strength concrete members [9,10]. Collins and Kuchma [9] also suggested that the reduction in shear stress at failure was related more directly to the maximum spacing between the layers of longitudinal reinforcement rather than the overall member depth.

Experimental tests carried out by Fujita et al. [10] showed that shear fracture in HSC is characterised by a conspicuous localisation of cracking in comparison with ordinary strength concrete, and that the propagation of these cracks was rapid, resulting in a more brittle fracture. A study using Fracture Mechanics was

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Table 1
Summary of current code of practice procedures

Code	Predicted failure shear strength	Comments
MC-90 [1]	$V_c = 0.15\xi\ (100\rho_l f_c)^{1/3}\left(\frac{3}{a/d}\right)^{1/3} b_w d$	$\xi = 1 + \sqrt{\frac{200}{d}}$ $\rho_l = \frac{A_l}{b_w d}$
2002 Final Draft of the EC–2 [2]	$V_{Rd,c} = 0.18\ k\ (100\rho_l f_c)^{1/3} b_w d \geq 0.035\ k^{3/2} f_c^{1/2} b_w d$	$f_c = 100$ MPa $k = 1 + \sqrt{\frac{200}{d}} \leq 2.0$ $\rho_l = \frac{A_l}{b_w d} \leq 0.02$
AASHTO LRFD 2000, [3]	$V_c = \beta\sqrt{f_c}\ b_w d_v$	β is given in a table as a function of the equivalent crack spacing and the longitudinal strain in the web.
ACI 318-02, Eq. 11-3 [4]	$V_c = \left(\frac{\sqrt{f_c}}{6}\right) b_w d$	$f_c < 70$ MPa
ACI 318-02, Eq. 11-5 [4]	$V_c = \left(0.16\sqrt{f_c} + 17\rho_l \frac{V \cdot d}{M}\right) b_w d$	$f_c < 70$ MPa, $Vd/M \leq 1$

conducted to determine the relationship between size effect and concrete compressive strength, leading to the following expressions [10]:

$$\frac{V}{bd} \propto \left(\frac{l_{ch}}{d}\right)^{1/4} \quad \text{for NSC} \tag{1}$$

$$\frac{V}{bd} \propto \left(\frac{l_{ch}}{d}\right)^{1/2} \quad \text{for HSC} \tag{2}$$

where l_{ch} (mm), the characteristic length, is equal to:

$$l_{ch} = 30700 \cdot f_c^{-1.1}\ (f_c \text{ in MPa}) \tag{3}$$

In order to evaluate these and other effects, an artificial neural network was developed based on a database with 193 test beams. The analyses rely on the databases developed by Bentz [11] and Kuchma [12]. However, more than 30 new tests were added, and many experimental programmes carried out during the 1950s and 1960s have been omitted as the reinforcement steel had a very low yielding stress. To summarize, the database includes the beams reported in references [9] and [13–31].

The shear-span-to-depth ratio, a/d, for all of these beam specimens, is greater than 2.48, and all of them failed in shear. Although great efforts were made in order to achieve a homogeneous database, most beams had a small effective depth and a high amount of longitudinal reinforcement. For instance, only 11% of the data had an effective depth greater or equal to 600 mm. However, with the aim of reducing the influence of the database's lack of heterogeneity, the analyses were performed not only on the full set of beams, but also on partial subsets of the database. Related to the concrete compressive strength, indeed an important parameter in this study, it is remarkable that 20% of the beams presented f_c higher than 70 MPa.

2. Artificial neural network

An artificial neural network (ANN) is a computational tool that attempts to simulate the architecture and internal features of the human brain and nervous system [32]. ANNs are made up of a number of simple, highly-interconnected processing elements, representing neurons, which constitute a network (Fig. 1). Each

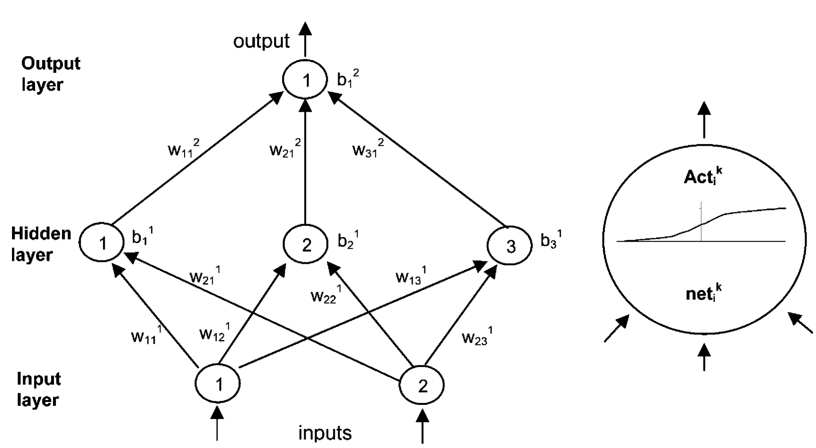

Fig. 1. Typical neural network model. Adapted from [32].

neuron receives several inputs from neighbouring elements, but only sends one output. The training process of an ANN involves presenting a set of examples with known inputs and outputs. The system adjusts the weights of the internal connections to minimise errors between the network output and target output. Moreover, this learning occurs even when the input data contains errors or is incomplete, which is one of the problems we must address when talking about shear strength. The ANN described in this paper was developed using the program PDP++ by O'Reilly et al. [33].

2.1. Data selection

In total, 177 test beams were used to train and test the artificial neural network out of the 193 beams of the experimental database. In other words, 16 beams had to be removed to ensure a satisfactory generalization. The 18 beams removed were those tested by Bazant et al. [17] with a web width equal to 38.1 mm, much smaller than the next lowest width of 102 mm. Although it was initially tried to use these data, it was seen that the network performance was much better when they were removed, as the ANN was not able to ensure generalisation for beams with the web width between 38 and 100 mm.

The input parameters considered are the effective depth (d); the web width, b, introduced as a web slenderness factor d/b; the shear span-to-depth ratio a/d; the reinforcement ratio of longitudinal tensile steel (ρ_l); and the concrete cylinder compressive stress (f_c). The output value is the failure shear strength V_{fail}. Table 2 summarizes the ranges of each different variable. The 177 test beams were divided into two sets: a training set containing 147 beams, and a validating set with 30 beams.

2.2. Topology of the artificial neural network, learning process and validation

A method of trial and error was carried out to define the optimum topology, learning procedure, and duration of the learning process.

Table 2
Artificial neural networks for beams without web reinforcement. Range of parameters in the database

Parameter	Minimum	Maximum
d (mm)	101.6	1090
d/b	0.37	7.17
ρ_l (%)	0.50	6.64
f_c (MPa)	14.7	101.8
a/d	2.48	7.86
V_{fail} (kN)	19.52	332.14

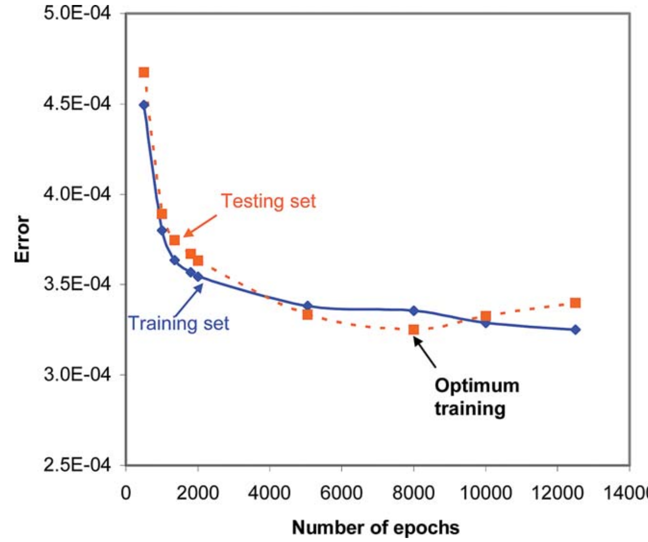

Fig. 2. Artificial neural network for beams without web reinforcement. Learning process for network with 10 hidden units.

After checking more than 50 different trained networks, the best solution was obtained with a neural network containing 10 hidden units and after 8000 iterations (epochs). The summed square error per number of data in both sets is plotted in Fig. 2 against the number of iterations or epochs for a ANN with 10 hidden neurons. While the error in the training set always decreases, there is a low point for the testing set. The final network, with 8000 epochs, shows a satisfactory generalisation, as the testing set event gives a lower error than the training set. The average $V_{\text{test}}/V_{\text{pred}}$ ratio is equal to 0.99 for the training set and 1.02 for the validating set. The CoV are 12.79% and 12.53%. respectively.

3. Parametrical analyses based on the ANN results

After the network has been adequately trained, it is possible to implement the activation functions and the weights in a simple spreadsheet, and to generate new beams to study the influence of the different parameters which affect the failure shear strength. The most important conclusions of the parametrical analyses are now presented.

3.1. Size effect. Influence of the effective depth, d

As was mentioned earlier, the size effect on the shear capacity is linked to the concrete compressive strength. However, the Model Code 90 [1] shear procedure does not consider the size effect to be related to the compression strength. On the other hand, the ACI 11-3 expression [4] does not take into account the size effect. These two procedures are compared in Fig. 3 to the

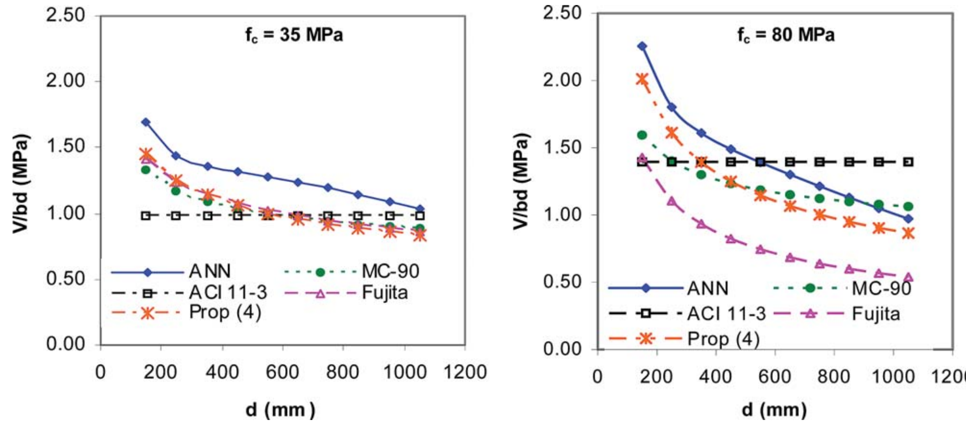

Fig. 3. ANN results compared to the Model Code 90, ACI (Eq. 11-3), Fujita et al. [10] and proposed equation predictions for beams without web reinforcement. Size effect related to the concrete compressive strength.

ANN predictions for a set of 10 beams with $b = 200$ mm, $\rho_l = 2\%$, $a/d = 3$. Parameters d and f_c varied as indicated in the graphs. The ACI 11-3 equation does not correlate properly with the test result, and for beams with a high effective depth it may be unconservative.

For normal strength concrete ($f_c = 35$ MPa) the Model Code 90 predictions show satisfactory agreement with the ANN results. Nevertheless, the size effect is under-estimated for HSC beams, as shown in Fig. 3. If the size effect factor given in the MC-90 code is replaced with the factor given by Fujita et al. the curves correlate better, as shown in Fig. 3, although it is slightly conservative for HSC beams. Besides, the Fujita equation is not continuous for different concrete compressive strengths, and this makes it difficult to implement it in a shear design procedure.

A new size effect factor able to be implemented in the Model Code expression was developed to take into account the influence of the concrete compressive strength using Fujita's and the artificial neural network predictions:

$$V_c = \left[0.15\xi\ (100\rho_l f_c)^{1/3}\right]\ b_w d \qquad (4)$$

$$\xi = \left(\frac{135000 \cdot f_c^{-1.1}}{d_v}\right)^{0.25\left(1+\frac{f_c-25}{75}\right)} \leq 2.75 \qquad (5)$$

where Eq. (4) is a direct derivation of the MC-90 procedure just removing the shear span to depth ratio, a/d, Eq. (5) is a generalization of the Fujita Eqs. (1)–(3), and b_w is web width; d, effective depth; d_v, mechanical depth taken to be $0.9{\cdot}d$; f_c, concrete compressive strength; ρ_l, amount of longitudinal reinforcement; ξ, size effect (Fig. 4).

The satisfactory agreement between this proposed equation and the ANN results is shown in Fig. 5. However, this expression was considered too complex to be implemented in a code of practice and some simplification is studied later in this paper.

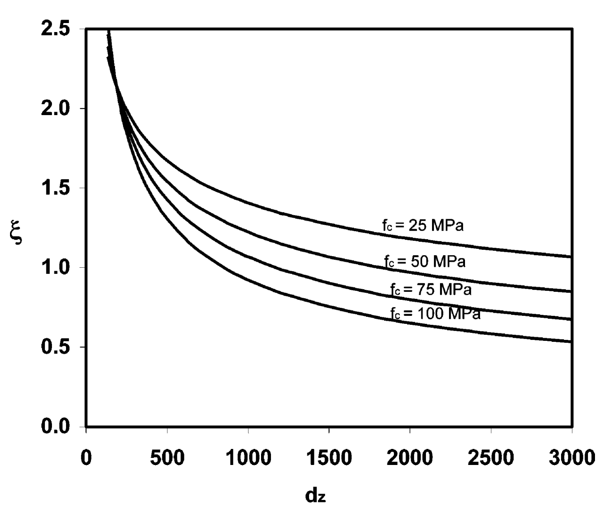

Fig. 4. Proposed size effect in function of the concrete compressive strength.

3.2. Influence of the concrete compressive strength, f_c

Fig. 5 plots the failure shear strength of a series of reinforced concrete beams without web reinforcement as a function of f_c and d as indicated in the graphs. The web width, amount of longitudinal reinforcement and slenderness ratio were 200 mm, 2% and 3, respectively. For the 250 mm effective depth beam, the response is almost linear. However, for the 900 mm-effective depth series the shear strength increases until the concrete compressive strength reaches 50 MPa. For HSC beams, the increase in compressive strength produces a decrease in failure shear strength. Eqs. (4) and (5), proposed above, agree with the ANN results as can also be seen in Fig. 5.

This behavior may be explained, in the authors' opinion, as follows: the influence of the size effect becomes

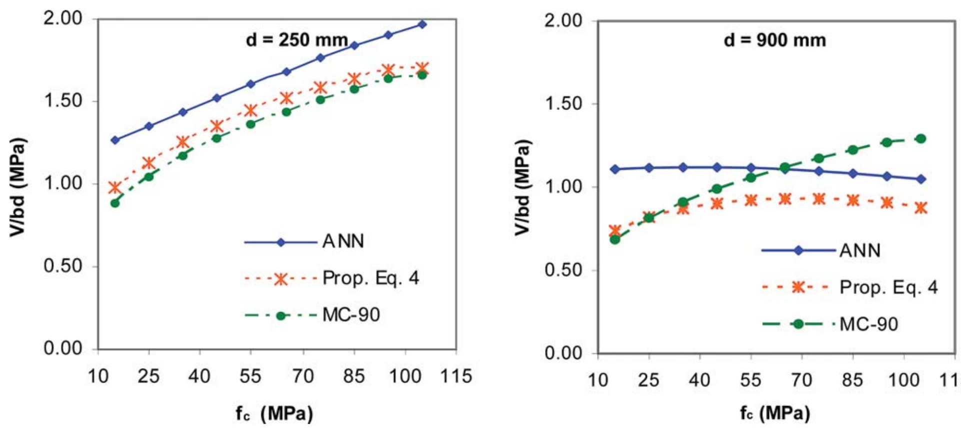

Fig. 5. ANN results compared to the predictions made by the CM-90 and the method proposed in this section for beams without web reinforcement. Influence of the concrete compressive strength as related to the size effect.

bigger when the concrete compressive strength increases, to such an extent that, for deep beams, the benefit of having a higher concrete compressive strength is lower than the loss caused by the size effect.

3.3. *Influence of the amount of longitudinal reinforcement, ρ_l*

The influence of the amount of longitudinal reinforcement predicted by the ANN results is analysed here and compared with the current MC-90 [1] procedure and the equation 11-3 of the ACI 318-02 code [4]. The MC-90 shear procedure suggests that the influence of the amount of longitudinal reinforcement is proportional to $\rho_l^{1/3}$. On the other hand, the ACI 11-3 equation does not take into account its influence.

The ANN predictions propose that the longitudinal reinforcement has a greater influence. No significant difference has been found for different beam depths. In order to adapt the proposed Eq. (4) to take into account the higher influence of the longitudinal reinforcement, that term has been raised to a different power:

$$V_c = \left[0.13\xi(100\rho_l)^{1/2}f_c^{1/3}\right]\, b_w d \qquad (6)$$

where ξ is defined by Eq. (5). The study suggested that the value limiting ρ_l could be related to the concrete compressive strength, since for high-strength concrete beams the limit would be higher than for normal-strength concrete beams, as it will be seen in the general shear design method (Eq. (7)).

4. Proposal for a new shear design equation

Two new shear design methods, a general and a simplified procedure, are proposed based on the observed behaviour from the analyses carried out with the ANN and taking into account the structure of the MC-90 shear procedure. Both methods also consider the Collins and Kuchma [9] observation about the size effect and the distributed longitudinal reinforcement.

4.1. *General shear design method*

The shear strength of reinforced normal and high-strength concrete beams without web reinforcement is given by the following equation:

$$V_c = \left[0.13\xi(100\rho_l)^{1/2}f_c^{1/3}\right]\, b_w d \qquad (7)$$

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, s_x is whichever is smaller, d_v or the vertical distance between longitudinal distributed reinforcement as indicated in Fig. 6, d is the effective depth in mm, d_v is the mechanical depth taken to be $0.9{\cdot}d$,

$$\rho_l = \frac{A_l}{b_w d} \leq 0.02\left(1+\frac{f_c}{100}\right)$$

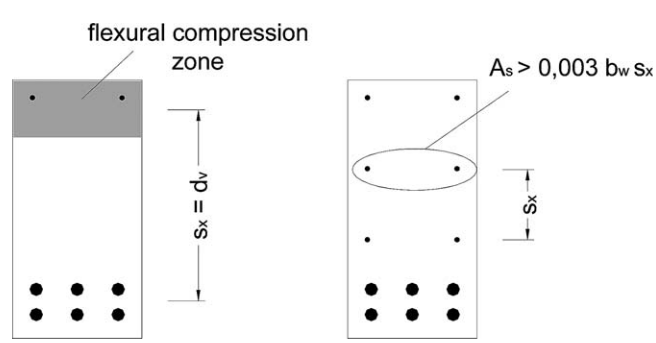

Fig. 6. Value of s_x for members without web reinforcement.

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.

The constant 0.13 in Eq. (7) was the best fit with the experimental database. However, as it will be commented later, the verification of the model was also conducted with databases developed by other authors. All the other parameters were derived from the parametrical study above commented. For beams containing a very low amount of longitudinal reinforcement, ρ_l, a minimum value for Eq. (7) could be defined similarly to Eurocode 2 [2] (Table 1).

4.2. Simplified shear design method

Including the size effect in Eq. (7) results in equations that are probably too complex to be implemented in a code of practice.

The simplified shear design method proposed adopts a size effect term independent of the concrete compression strength similar to the proposed by the Model Code 90 [1], it raises the compressive strength factor to a different power and it limits the magnitude of the concrete compressive strength to 60 MPa, although the formulation is intended for concretes up to 100 MPa, to keep from being unconservative for deep high-strength concrete beams:

$$V_c = \left[0.225\xi(100\rho_l)^{1/2}f_c^{0.2}\right]b_w d \qquad (8)$$

where

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

and $f_c \leq 60$ MPa and the other terms as defined previously.

5. Verification of the proposed equation using the experimental database

Table 3 compares the predicted values with the empirical results for the entire database of members without web reinforcement, including those members removed to build the artificial neural network. Proposed Eqs. (7) and (8) 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 the other procedures. Moreover, it is important to highlight that if the a/d factor is taken into account in the proposed formulation (adding it in the same way that the MC-90 does), the correlation of the proposed method with the empirical tests improves, leading to a coefficient of variation as low as 13.58%.

The results of the partial set analyses are shown in Table 4. An attempt has been made to study the size effect, the influence of the amount of longitudinal reinforcement, and the concrete compressive strength separately. The proposed methods represent an improvement in terms of the coefficient of variation over all the other code procedures and practically over all partial datasets.

For the current code procedures, in particular for the ACI [4] and EC-2 [2] shear methods, the safety is significantly reduced for larger members ($d \geq 900$ mm) and for elements with a low amount of longitudinal reinforcement ($\rho_l \leq 1\%$). MC-90 [1] is also slightly unconservative for the larger members. In contrast, the average value of $V_{\text{fail}}/V_{\text{pred}}$ is higher for members with more than 2% of longitudinal reinforcement, especially for high strength concrete beams, and for small members ($d \leq 100$ mm). These effects are corrected using the proposed equations, as shown in Table 4.

Proposed Eq. (7) gives almost an identical correlation for normal-strength and high-strength concrete beams, with average $V_{\text{fail}}/V_{\text{pred}}$ ratios of 1.14 and 1.16, and coefficients of variation of 15.96 and 15.53%, respectively.

6. Verification of the proposed equations for elements with longitudinal distributed reinforcement

As was stated earlier, Collins and Kuchma carried out an experimental campaign to evaluate the parameters influencing on the size effect [9]. They concluded that it was related to the maximum spacing between

Table 3
Verification of proposed shear procedures using the entire database

Procedure	ACI 11-5	ACI 11-3	MC-90	EC-2	AASHTO	Eq. (7)	Eq. (8)
Average	1.16	1.29	1.15	1.02	1.28	1.15	1.13
Median	1.15	1.25	1.16	0.99	1.25	1.14	1.12
Standard deviation	0.31	0.40	0.19	0.23	0.22	0.18	0.19
CoV (%)	26.89	31.21	16.57	22.03	16.80	15.73	16.42
Minimum	0.42	0.42	0.65	0.57	0.86	0.73	0.78
Maximum	2.14	2.47	1.78	1.78	2.14	1.69	1.85

Table 4
Verification of different code procedures using partial sets of the database for beams without web reinforcement

Beam specimens	No. beams	Average V_{test}/V_{pred}							CoV V_{test}/V_{pred}						
		ACI 11-5	ACI 11-3	MC 90	EC2	AASHTO	Eq. (7)	Eq. (8)	ACI 11-5	ACI 11-3	MC90	EC2	AASHTO	Eq. (7)	Eq. (8)
All	193	1.16	1.29	1.15	1.02	1.28	1.15	1.13	26.89	31.21	16.57	22.03	16.80	15.73	16.42
$d \geq 900$ mm	18	0.71	0.76	0.99	0.83	1.11	1.28	1.07	25.08	28.49	20.57	18.84	14.46	10.65	11.49
$d \leq 100$ mm	12	1.46	1.58	0.98	1.18	1.42	1.11	1.07	10.63	10.65	8.09	10.59	10.57	10.53	9.16
$\rho_l \leq 1\%$	37	0.87	0.90	1.08	0.89	1.16	1.27	1.17	23.98	25.51	15.15	17.40	10.13	12.96	12.68
$f_c > 50$ MPa	93	1.18	1.32	1.15	1.03	1.29	1.14	1.17	29.90	34.23	19.19	25.81	20.10	15.96	17.32
$f_c \leq 50$ MPa	100	1.15	1.27	1.16	1.01	1.28	1.16	1.09	23.71	27.79	13.83	17.58	12.99	15.53	14.69
$\rho_l > 2\%$, $f_c > 50$ MPa	55	1.35	1.54	1.22	1.15	1.38	1.13	1.20	23.00	26.27	17.33	23.24	19.85	17.47	19.59
$\rho_l > 2\%$, $f_c \leq 50$ MPa	34	1.31	1.52	1.26	1.10	1.35	1.15	1.07	17.42	20.68	11.86	16.10	13.26	15.23	16.49

Table 5
Summary of predictions by MC-90, AASHTO LRFD, Eq. (7) and Eq. (8) for elements with longitudinal distributed reinforcement

Beam	f_c MPa	b mm	$d$mm	a/d	ρ_l	s_x[a]	V_{fail} (kN)	$V_{predicted}$				$V_{test}/V_{predicted}$			
								MC90	LRFD	Eq. (7)	Eq. (8)	MC90	LRFD	Eq. (7)	Eq. (8)
B100D	36	300	925	2.92	0.76	170	320	185	288	225	232	1.73	1.11	1.42	1.38
BND100	37	300	925	2.92	0.76	170	258	187	268	227	234	1.38	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	260	345	321	257	1.07	0.81	0.87	1.08
BHD100R	99	300	925	2.92	0.76	170	334	260	345	321	257	1.29	0.97	1.04	1.30
BHD50	99	300	450	3.00	0.81	85	193	146	180	198	157	1.33	1.07	0.98	1.23
BHD50R	99	300	450	3.00	0.81	85	205	146	180	198	157	1.41	1.14	1.04	1.30
BH25D	99	300	225	3.00	0.89	40	111	88	103	104	89	1.27	1.07	1.07	1.24
SE100B-45	50	295	920	2.50	1.03	195	281	236	321	274	273	1.19	0.87	1.02	1.03
SE100B-45-R	50	295	920	2.50	1.03	195	316	236	321	274	273	1.34	0.98	1.15	1.16
SE50B-45	53	169	459	2.72	1.03	195	87	76	87	80	79	1.15	1.00	1.09	1.10
SE100B-83	86	295	920	2.50	1.03	195	365	283	361	328	283	1.29	1.01	1.11	1.29
SE100B-83-R	86	295	920	2.50	1.03	195	364	283	361	328	283	1.29	1.01	1.11	1.29
SE50B-83	91	169	459	2.72	1.03	195	101	91	97	95	81	1.11	1.04	1.06	1.25
H50/5	49.9	200	359	3.01	2.24	110	130	91	110	129	124	1.44	1.18	1.00	1.05
H100/5	87	200	359	3.01	2.24	110	141	109	125	167	129	1.29	1.13	0.85	1.09
									Average			1.35	1.06	1.09	1.20
									Standard deviation			0.19	0.16	0.16	0.11
									Coefficient of var.			14.36	15.17	14.94	9.45

[a] Distance between layers of longitudinal reinforcement.

the layers of longitudinal reinforcement rather than the overall member depth.

Test beams with longitudinal distributed reinforcement were not included in the database, as only the AASHTO LRFD specifications [3] take this effect into account, and therefore the performance of the other codes' procedures would have been poorer. Neither were used to train the ANN. Table 5 gives the geometrical parameters of the beams with longitudinal distributed reinforcement, test results, and predictions given by the MC-90 procedure, the AASHTO LRFD specifications, and the proposed equations. All beams in Table 5 had a greater amount of longitudinal reinforcement distributed in the web than the minimum amount given in Fig. 6. All were tested by [9] except for the last two beams, tested by Cladera [18] as a part of a larger experimental campaign.

The MC-90 shear procedure does not take the effect of distributed longitudinal reinforcement into consideration, and it is excessively conservative for the 17 beams containing it, with an average V_{fail}/V_{pred} ratio of 1.35 compared to the average of 1.15 for the full database. The predictions made by the AASHTO procedure improve the correlation, resulting in an average of 1.06. The CoV are 14.36 and 15.17%, respectively. Eqs. (7) and (8) do take this effect into account and they improve the performance observed for the MC-90 shear procedure for members without web reinforcement. The average V_{test}/V_{pred} ratio is 1.09 for Eqs. (7) and 1.20 for Eq. (8), and their CoV are 14.94 and 9.45%, respectively.

7. Verification of the proposed equations with other databases

The correlation of the simplified shear design method and the new draft of the EC-2 procedure [2] with the empirical result data base developed by Reineck et al. [34] are compared to verify the proposed formulation. For the entire database (398 beam specimens), the average of the V_{fail}/V_{pred} ratio is equal to 1.20 for the proposed equation and 1.11 for the EC-2. The coefficients of variation are 22.39 and 25.48%, respectively. The average value of d is 345 mm in the database. If only high-strength concrete beams ($f_c > 50$ MPa) with $d > 500$ mm are considered, the average value of V_{fail}/V_{pred} is equal to 1.02 for the proposed equation and 0.75 for the EC-2 and the coefficients of variation are 14.36 and 25.00%, respectively, which demonstrate that the new EC-2 procedure is unconservative for deep high-strength concrete beams, as it does not consider the influence of the concrete compression strength on the size effect.

Lubell et al. [35] summarized the results of 40 beam specimens with an average depth of 828 mm. For this set of beams, the average of the V_{fail}/V_{pred} ratio is equal to 1.14 for the proposed equation and 0.84 for the EC-2. The coefficients of variation are 18.51 and 20.16%, respectively.

8. Design example

The 1955 shear failures of the reinforced beams in the warehouses used by the US Air Force are a frequently referred structural collapse. The beams, not included in the database used in this paper, failed under dead load only at a shear of about 205 kN and a moment of 102 kN·m [36]. The mechanical characteristics of the beam were: $f_c \approx 25$ MPa, $b_w = 508$ mm, $h = 914$ mm, $d = 850$ mm and $A_l = 1935$ mm^2. Adopting the simplified shear design method (Eq. (8)):

$$V_c = \left[0.225\left(1+\sqrt{\frac{200}{0.9\cdot 850}}\right)\left(100\frac{1935}{508\cdot 850}\right)^{1/2} 25^{0.2}\right] \times 508\cdot 850 = 187 \quad \text{KN}$$

which compared to the shear failure gives a $V_{\text{fail}}/V_{\text{pred}}$ ratio equal to 1.10. The shear strengths obtained using different codes are: $V_{\text{ACI 11-3}} = 367$ kN ($V_{\text{fail}}/V_{\text{pred}} = 0.56$), $V_{\text{MC-90}} = 215$ kN ($V_{\text{fail}}/V_{\text{pred}} = 0.95$), $V_{\text{EC-2}} = 258$ kN ($V_{\text{fail}}/V_{\text{pred}} = 0.83$), and $V_{\text{AASHTO}} = 244$ kN ($V_{\text{fail}}/V_{\text{pred}} = 0.84$). For this beam the general method would be slightly more conservative with a $V_{\text{fail}}/V_{\text{test}}$ ratio of 1.24.

9. Conclusions

A study related to the shear strength of normal and high-strength concrete beams without stirrups has been performed. An artificial neural network has been developed to predict the shear strength of RC beams using a large database of experimental results available. Based on the parametric study made using the ANN, design formulae were developed. The following conclusions can be drawn for the present study:

- Artificial neural networks have been shown to be a powerful tool for predicting the shear strength of reinforced concrete beams. Moreover, blind fitting to the data is avoided by means of a parametrical study.
- Size effect was related to the concrete compressive strength. For deep beams, the benefit of a higher concrete compressive strength was outweighed by the loss caused by the size effect.
- A general shear design procedure was derived directly from the artificial neural network results taking into account an expression of the size effect originally derived by Fujita et al. [10]. The CoV of the $V_{\text{test}}/V_{\text{pred}}$ ratio for the entire database was 15.73%.
- A simplified shear design procedure was derived from the general procedure by simplifying the size effect term. The coefficient of variation of $V_{\text{test}}/V_{\text{pred}}$ ratio for the entire database was 16.42% for the entire database. Because of its simplicity, it is recommended using the use of this shear design method as a general procedure to obtain the shear strength of normal-strength and high-strength concrete beams without web reinforcement.
- Both methods take into account that the size effect is related to the maximum vertical spacing between layers of longitudinal reinforcement rather than the overall member depth.

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