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THE INVESTIGATION OF THE STRENGTH REDUCTION FACTOR IN PREDICTING THE SHEAR STRENGTH

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Design codes propose to restrict the nominal probability of failure within specific target structural reliability levels using a load factor and a strength reduction factor. In the current ACI318 Code, the strength reduction factor varies from 0.65 to 0.90, and the value considered in predicting the shear strength equals to 0.75. In this study, the change in the strength reduction factor in predicting the shear strength according to ACI318 has been investigated for different coefficients of variation of concrete compressive strength by using the first-order second moment approach, and the strength reduction factor is proposed for the target values of failure probability.

Keywords: reinforced concrete, beam, shear strength, reduction factor, target reliability

1. Introduction

The safety of a structure can be explained as the probability that the structure will perform its purposes throughout its design lifetime. In order to provide certain reliability levels for structures, design codes use safety factors. Partial safety factors are to be evaluated for a given target reliability index (β). The value of β depends on the relative consequences of failure and the relative costs of safety measures. The range of β for flexural strength of reinforced concrete (RC) beams designed according to ACI318 was investigated by Mirza (1996). The resulting value of β is 3.1 (range 2.5-3.9). The target values of β were set in the study of Mirza (1996) at 3.0 and 3.25 for columns exhibiting tension and compression failures, respectively. MacGregor (1983) took as $\beta = 2.5$-3 for tension failures and $\beta = 3$-3.5 for compression failures. A higher value of β was assigned to members displaying compression failure, reflecting the increased danger due to sudden, brittle behavior of such members at the failure load.

Beck *et al.* (2009) and Oliveira *et al.* (2008) noted that the target β of structures designed according to NBR8800 (2008) lies in the range from 2.3 to 4.5. The AS5104 (2005) and ISO2394 (1998) suggest that the lifetime target β ranges from 3.1 to 4.3 for ultimate (strength) limit states design. According to Vrouwenvelder (2002), the central value of $\beta = 4.2$ ($p_F = 1.33 \cdot 10^{-5}$) should be considered as the most common design situation, and the value of $\beta = 3.8$ ($p_F = 0.7 \cdot 10^{-5}$) is mentioned for a reference period of 50 years in the Eurocode. The target value of β in the studies of Hasofer and Lind (1974), Rackwitz and Fiessler (1978) and Madsen *et al.* (1986) was set at 4.1.

ACI318 and ASCE/SEI (2010) are based on semi-probabilistic approaches to design (Ribeiro and Diniz, 2013). According to the study of Nowak and Szerszen (2003) that is the basis of ACI318 calibration, the target β is 3.5 (range 3.4-3.6) for RC beams. In the ACI318 (1995), the strength reduction factor for shear is 0.85. According to the ACI318 (2002, 2011), the strength reduction factor for shear is 0.75. According to TS500 (2000), the contribution of concrete to shear strength is obtained by reducing diagonal cracking strength with a safety factor of 0.8.

A first-order second moment probabilistic analysis procedure is used to compute the strength reduction factor in predicting the shear strength of RC beams according to the current ACI318 (2011), Section 9.3. The change in the strength reduction factor against the coefficient of variation of concrete compressive strength (V_{fc}) and the failure probability (p_F) is investigated through the database of 375 shear test results collected from 36 references.

2. Design recommendations for RC beams

According to the ACI318, the nominal shear strength (ν_n) is derived from two components: concrete and stirrups. This relationship is given as follows

$$\nu_n = \nu_c + \nu_s \tag{2.1}$$

in which ν_s is the shear strength of stirrup based on yield and ν_c is the shear strength of concrete, respectively. The shear strength of concrete consists of four mechanisms of shear transfer identified by the ASCE-ACI426 (1973) report as follows: the uncracked portion of the concrete, vertical components of the aggregate interlocking force in the cracked portion of concrete, dowel action of the longitudinal steel, and arch action. The shear strength of RC beams is given as follows

$$\nu_n = \frac{1}{6}\sqrt{f_c} + \rho_w f_{yw} \tag{2.2}$$

in which ρ_w is the ratio of stirrups, f_c and f_{yw} are the compressive strength of concrete and yield strength of stirrup in MPa, respectively.

In the ACI318, the strength design philosophy states that the design shear capacity of a member must exceed the shear demand as shown in Eq. (2.3)

$$\phi\nu_n \geqslant \nu_u \tag{2.3}$$

in which ϕ is the shear strength reduction factor and given as 0.75 in ACI318 (2011) and 0.85 in ACI318 (1995). In this study, the change in the strength reduction factor considered in predicting the shear strength according to the ACI318 (2011) is investigated and compared for different failure probabilities and coefficients of variation of concrete compressive strength.

3. Reliability analysis

3.1. Analysis method

In probability theory, the capacity R and the load S involve different basic variables. Hence the performance function, $Z = R - S = g(X_1, X_2, \ldots, X_n)$, contains uncertainties in all design variables. When the performance function equals to zero, $Z = 0$, it is called a failure surface. The safety or reliability is defined by the condition $Z > 0$ and therefore, failure by $Z < 0$. The calculation of probabilities of reliability or failure requires the knowledge of the joint probability distribution of all basic variables in the performance function. However, in many cases, these probability distributions are unavailable or difficult to obtain due to general lack of data. Besides, even though distributions of the variables are known, if the performance function is highly nonlinear, the evaluation of failure probability by numerical methods is difficult (Ranganathan, 1990; Ang and Tang, 1984).

Because of these difficulties, the approximate methods for evaluation of structural reliability have been improved. In these methods, the random variables are represented by their first and second moments. In evaluating the first and second moments of the failure function, the first order approximation is used. That is why these methods are called first-order second moment methods. Therefore, this method is generally used by committees in calibrating codes for the evaluation of partial safety factors (Ranganathan, 1990).

3.2. Determination of partial safety factors

In this study, the determination of the strength reduction factor has been developed using the first-order second moment method. The strength reduction factor may be called as partial safety factor in the reliability based design. Partial safety factors are to be evaluated for a given β. At the same time, β is the safety measure that corresponds to a given probability of failure. Hence, in the reliability based design, the problem of the partial safety factors is reverse. If x_i^* is the design value of the original variable X_i, the failure surface equation is defined as

$$g(x_1^*, x_2^*, \ldots, x_n^*) = 0 \qquad i = 1, 2, \ldots, n \tag{3.1}$$

where $x_i^*(= \gamma_i m_{X_i})$ is the most probable failure point on the failure surface, and the determination of x_i^* requires an iterative solution. Thus, it is required to find the design point $(\gamma_i m_{X_i})$ corresponding to the target β. The most general design format is to apply a safety factor on each of all design variables. The performance function must satisfy

$$g(\gamma_1 m_{X_1}, \gamma_2 m_{X_2}, \ldots, \gamma_i m_{X_i}) = 0 \qquad i = 1, 2, \ldots, n \tag{3.2}$$

The design point should be the most probable failure point. In the space of the reduced variates, the most probable failure point is $x_i^{'*} = -\alpha_i^* \beta$, and β is defined as the shortest distance from the failure surface to the origin.

Sensitivity coefficient α_i^* is defined by as (Ang and Tang, 1984)

$$\alpha_i^* = \frac{\partial g}{\partial X_i'} \left[\sum_{i=1}^{n} \Big(\frac{\partial g}{\partial X_i'} \Big)_*^2 \right]^{-1/2} \tag{3.3}$$

The partial safety factors required for a given β are defined as $\gamma_i(= x_i^*/m_{X_i})$. The original variables are given by $x_i^* = m_{X_i}(1 - \alpha_i^* \beta V_{X_i})$, in which m_{X_i} and V_{X_i} are the mean value and the variance coefficient of the original variable X_i with normal distribution, respectively. V_{X_i} is the ratio of standard deviation (σ_{X_i}) to the mean value (m_{X_i}). The partial safety factors are calculated as $\gamma_i = (1 - \alpha_i^* \beta V_{X_i})$.

In this study, it is assumed that the distributions of variables in the performance function are normal and lognormal. In lognormal distributions, m_{X_i} and σ_{X_i} should be replaced by the equivalent normal mean $m_{X_i}^N$ and standard deviation $\sigma_{X_i}^N$. In addition, it is also assumed that the all variables are statistically independent (Ang and Tang, 1984).

3.3. Strength reduction factor

The performance function $g(X)$ for the shear failure mode is expressed as

$$g(X) = \gamma_i \nu_n - \gamma_j \nu_{u,exp} \tag{3.4}$$

in which ν_n is the nominal shear strength, $\nu_{u,exp}$ is the experimental shear strength, γ_i and γ_j are the safety factors corresponding to the related variables. By calculating weighted averages of these factors (γ_i), the strength reduction factor ϕ, defined in Eq.(3.4) is determined. The change in the ϕ considered in predicting the shear strength according to the ACI318 against the different V_{f_c} $(0.10, 0.12, 0.15, 0.18)$ and p_F $(10^{-7}, 10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}, 10^{-2})$ has been investigated by using experimental studies available in the literature.

4. Uncertainties of random variables

The uncertainties included in the prediction of shear strength are modeled as random variables. Since there is no information about the measurement sensivity in the experiments, the values of the coefficient of variation taken into account in the calculations are determined by considering the previous statistical studies.

The coefficient of variation of concrete compressive strength (V_{f_c}) under average construction quality control usually depends on the concrete strength and varies in between 0.10 and 0.21 through the literature. The V_{f_c} was taken as 0.10 by Nowak and Szerszen (2003) and Ribeiro and Diniz (2013), 0.11 by Hao *et al.* (2010), 0.12 by Neves *et al.*(2008) and Soares *et al.* (2002), 0.13 by Val *et al.* (1997), 0.15 by Mirza (1996), Mirza *et al.* (1979), Mirza and MacGregor (1979a,b), 0.16 by Val and Chernin (2009) and Hosseinnezhad *et al.* (2000), 0.18 by Enright and Frangopol (1998) and Ramsay *et al.* (1979), 0.20 by Melchers (1999) and 0.21 by Ellingwood (1978).

Although the reinforcement ratios depend on the structural dimensions, they are assumed to be statistically independent from each other and from the other random structural parameters. In the study of Hao *et al.* (2010), it was assumed that the coefficient of variation of stirrup ratio (V_{ρ_w}) is 0.15, which is the value used in this study.

The coefficient of variation of reinforcement strength (V_{f_y}) was also reported by many researchers. Slightly different values were given by different researchers, where the V_{f_y} ranges from 0.05 to 0.15. The V_{f_y} was taken as 0.05 by JCSS (2000), 0.08 by Val *et al.* (1997), Hosseinnezhad *et al.* (2000) and Low and Hao (2001), 0.06 by Soares *et al.* (2002), 0.08-0.11 by Ostlund (1991), MacGregor *et al.* (1983), 0.12 by Enright and Frangopol (1998) and 0.15 by Mirza (1996), Mirza *et al.* (1979), Mirza and MacGregor (1979a,b). The V_{f_y} is taken as 0.10 in the present study. In the studies of Hognestad (1951) and Mirza (1996), it was assumed that the coefficient of variation of strength due to test procedure was 0.04, which is the value used in this study.

5. Investigation of the strength reduction factor in predicting the shear strength

The distributions of main properties of the beams in the database of 375 shear test results (Adebar and Collins, 1996; Anderson and Ramirez, 1989; Angelakos *et al.*, 2001; Bahl, 1968; Bresler and Scordelis, 1961; Bresler and Scordelis, 1966; Cladera and Mari, 2005, 2007; Collins and Kuchma, 1999; Cucchiara *et al.*, 2004; Elzanaty *et al.*, 1986; Guralnick, 1960; Gonzalez, 2002; Haddadin *et al.*, 1971; Johnson and Ramirez, 1989; Leonhardt and Walter, 1962; Karayiannis and Chalioris, 1999; Kong and Rangan, 1998; Krefeld and Thurston, 1966; Lee and Kim, 2008; Mattock and Wang, 1984; McGormley *et al.*, 1996; Mphonde and Frantz, 1985; Placas and Regan, 1971; Palakas and Darwin, 1980; Rajagopalan and Ferguson, 1968; Swamy and Andriopoulos, 1974; Ozcebe *et al.*, 1999; Roller and Russell, 1990; Sarzam and Al-Musawi, 1992; Shin *et al.*, 1999; Tan *et al.*, 1997; Xie *et al.*, 1994; Yoon *et al.*, 1996; Zararis and Papadakis, 1999; Zararis, 2003) are shown in Fig. 1. The frequency distribution of f_c varies from 12 MPa to 103 MPa, so covers a wide range of RC properties. In this study, the normal strength concrete (NSC) is defined as concrete having compressive strength less than 55 MPa, and high strength concrete (HSC) having compressive strength equal to or more than 55 MPa. Only 5% of the NSC beam tests (14 of 281 tests) were conducted for $f_c \leqslant 20$ MPa and 20% of the HSC beam tests (19 of 94 tests) were conducted for $f_c \geqslant 80$ MPa. It can be stated that the f_c values are not equally distributed in the range from 45 MPa to 75 MPa. A large amount of beams is characterized by 30 MPa for the NSC beams and 75 MPa for the HSC beams.

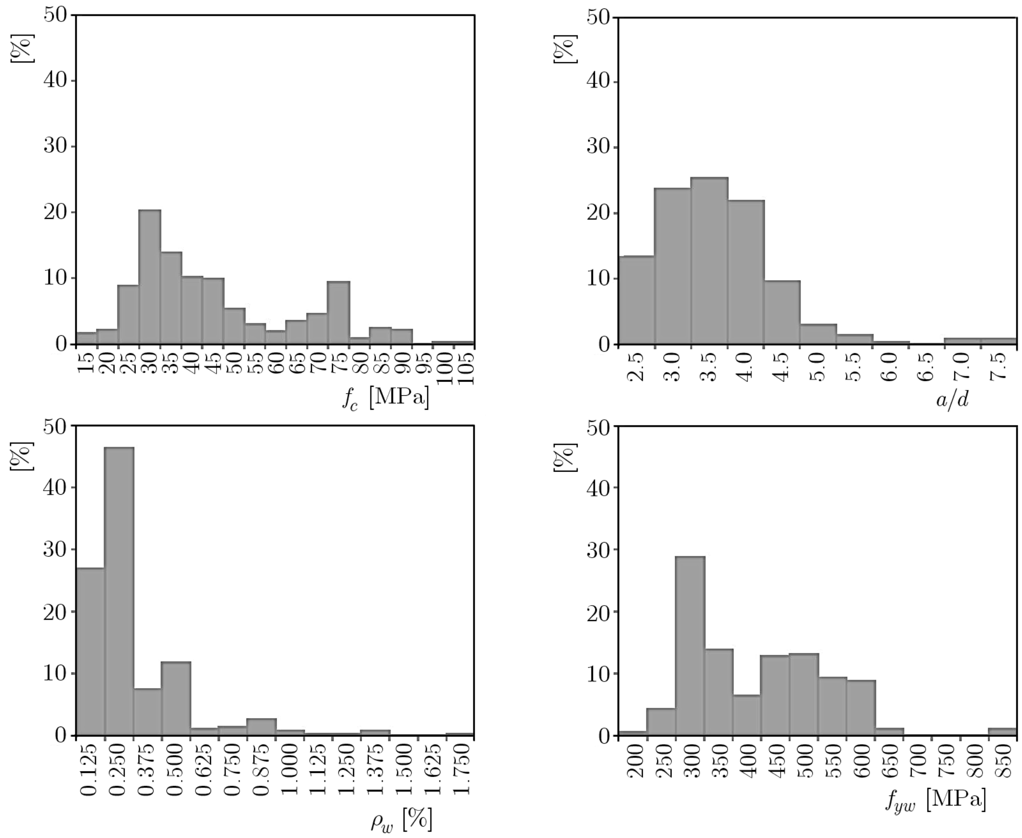

Fig. 1. Data frequency distributions: f_c, a/d, ρ_w and f_{yw}

The frequency distribution of shear span-to-depth ratio (a/d) varies from 2.5 to 7.5. It is worth noting that a/d values are not equally distributed in the range from 2.5 to 7.5 and most of the beams are characterized by small a/d. The beams with a/d higher than 6 ($a/d \geqslant 6$) are limited for all (NSC and HSC) beams; further research is therefore required to verify the found p_F.

The frequency distribution of the stirrup yielding strength (f_{yw}) varies from 179 MPa to 840 MPa. It is worth noting that f_{yw} values are not equally distributed in the range from 300 MPa to 500 MPa. A large amount of beams is characterized by 300 and 500 MPa. Thus, regarding the stirrup, these two values can be good representatives of typical yielding strengths of stirrups for existing buildings (300 MPa) and more recent ones (500 MPa). The database is characterized by percentage of ρ_w that ranges from 0.040 to 1.750 with a large amount of beams characterized by 0.250.

In order to determine a more accurate shear strength reduction factor for the shear design method, the change in the ϕ obtained from the analysis is compared in Table 1 for different values of V_{f_c} and p_F. ϕ decreases as β increases, and the reduction in the ϕ increases with V_{f_c}. For given V_{f_c} and p_F, the ϕ for the HSC beams are found to be greater than the one for the NSC beams, so it can be inferred that the ϕ for HSC beams is more safe than the one for the NSC beams. In the ACI318 (1995), the ϕ considered in predicting the shear strength equals to 0.85.

It is indicated that this value corresponds to the target values of $p_F = 10^{-2}$ ($\beta = 2.33$) and $V_{f_c} = 0.10$. In the ACI318 (2002) and ACI318 (2011), the factor of 0.85 was replaced by a factor of 0.75, which corresponds to the target values of $p_F = 10^{-5}$ ($\beta = 4.27$) and $V_{f_c} = 0.10$ for all beams. It is observed that this value is conservative for $p_F > 10^{-5}$ and a variation coefficient of 0.10. Ninety seven percent of the beams have strengths that exceed 0.75 times the calculated strength. The effects of f_c, a/d, ρ_w and f_{yw} on the ϕ are discussed below.

Table 1. Changing the average values of ϕ

V_{fc}	p_F						Beams
	10^{-7}	10^{-6}	10^{-5}	10^{-4}	10^{-3}	10^{-2}	
0.10	0.716	0.736	0.758	0.784	0.815	0.854	NSC (281 beams)
0.12	0.703	0.723	0.746	0.773	0.806	0.847	
0.15	0.681	0.703	0.728	0.756	0.791	0.836	
0.18	0.659	0.683	0.708	0.739	0.776	0.823	
0.20	0.645	0.669	0.695	0.727	0.766	0.815	
0.10	0.749	0.766	0.785	0.808	0.835	0.870	HSC (94 beams)
0.12	0.733	0.751	0.772	0.796	0.825	0.862	
0.15	0.708	0.728	0.750	0.776	0.808	0.849	
0.18	0.682	0.704	0.727	0.756	0.790	0.834	
0.20	0.666	0.688	0.713	0.742	0.778	0.825	
0.10	0.724	0.744	0.765	0.790	0.820	0.858	NSC and HSC (375 beams)
0.12	0.710	0.730	0.753	0.779	0.810	0.851	
0.15	0.688	0.709	0.733	0.761	0.795	0.839	
0.18	0.665	0.688	0.713	0.743	0.780	0.826	
0.20	0.650	0.674	0.700	0.731	0.769	0.817	

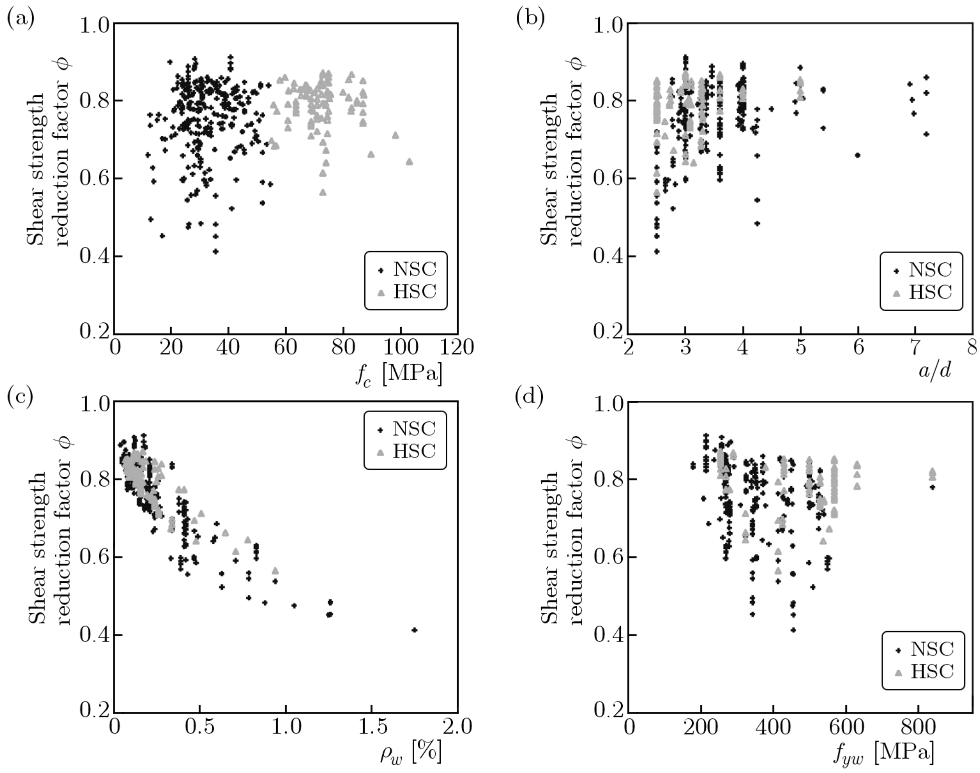

Fig. 2. Range of ϕ values determined using the evaluation database for $\beta = 4.27$ and $V_{fc} = 0.10$

Figures 2a and 3a show the variation of ϕ with f_c for $\beta = 4.27$ ($p_F = 10^{-5}$), $V_{fc} = 0.10$ and $\beta = 2.33$ ($p_F = 10^{-2}$), $V_{fc} = 0.10$, respectively. The ϕ for the existing test data yields large scatter and is not influenced significantly by f_c for all (NSC and HSC) beams. Figures 2b and 3b show the variation of ϕ with a/d for $\beta = 4.27$ ($p_F = 10^{-5}$), $V_{fc} = 0.10$ and $\beta = 2.33$ ($p_F = 10^{-2}$), $V_{fc} = 0.10$, respectively. The ϕ for the existing test data yields large scatter in the results for all (NSC and HSC) beams. 29% of the NSC beam tests (81 of 281 tests) and only

11% of the HSC beam tests (10 of 94 tests) were conducted for $(a/d) \geqslant 4.0$. The corresponding ϕ of 13 of 81 NSC beams are less than 0.75 for $p_F = 10^{-5}$, $V_{fc} = 0.10$ and the corresponding ϕ of 11 of 81 NSC beams are less than 0.85 for $p_F = 10^{-2}$, $V_{fc} = 0.10$. The corresponding ϕ of HSC beams are higher than 0.75 for $p_F = 10^{-5}$, $V_{fc} = 0.10$ and 0.85 for $p_F = 10^{-2}$, $V_{fc} = 0.10$.

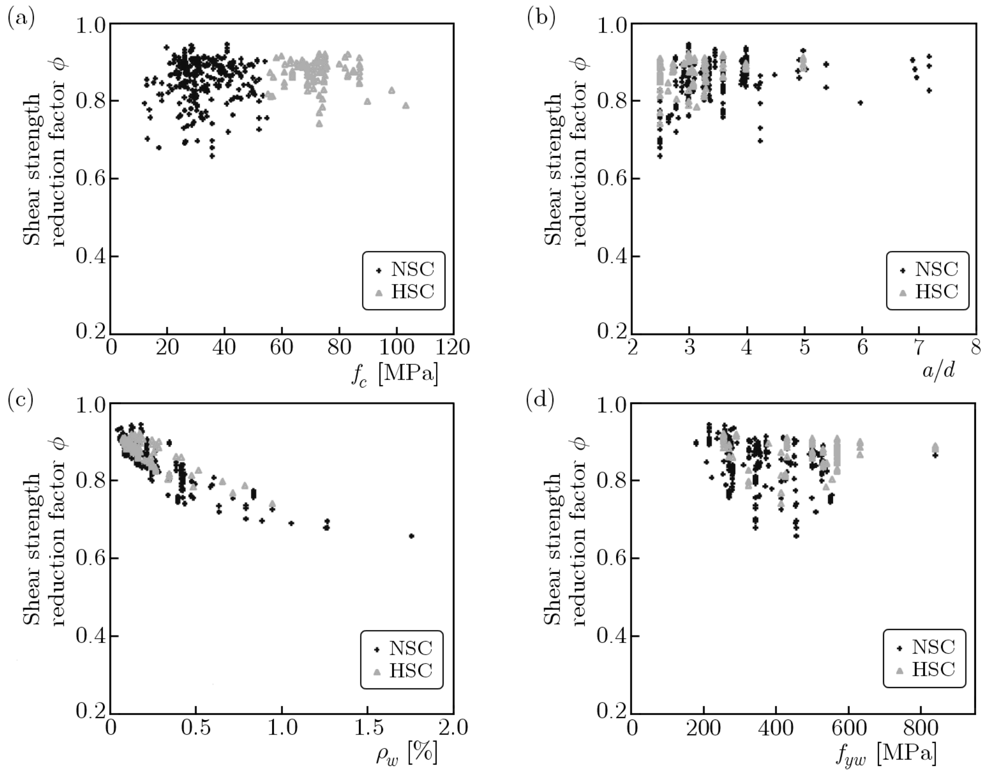

Fig. 3. Range of ϕ values determined using the evaluation database for $\beta = 2.33$ and $V_{fc} = 0.10$

Figures 2c and 3c show the variation of ϕ with the ρ_w for $\beta = 4.27$ ($p_F = 10^{-5}$), $V_{fc} = 0.10$ and $\beta = 2.33$ ($p_F = 10^{-2}$), $V_{fc} = 0.10$, respectively. 10% of the NSC beam tests (28 of 281 tests) were conducted for $\rho_w \geqslant 0.5\%$ and only 7% of the HSC beam tests (7 of 94 tests) were conducted for $\rho_w \geqslant 0.5\%$. The corresponding ϕ of 28 NSC and 7 HSC beams are less than 0.75 for $p_F = 10^{-5}$, $V_{fc} = 0.10$ and less than 0.85 for $p_F = 10^{-2}$, $V_{fc} = 0.10$. It is observed that the ϕ decreases with increasing ρ_w for all beams. Figures 2d and 3d show the variation of ϕ with f_{yw} for $\beta = 4.27$ ($p_F = 10^{-5}$), $V_{fc} = 0.10$ and $\beta = 2.33$ ($p_F = 10^{-2}$), $V_{fc} = 0.10$, respectively. The ϕ for existing test data yields large scatter in the results for all (NSC and HSC) beams.

It can be stated that the ϕ values are not equally distributed with respect to f_c, a/d and f_{yw}. The beams with a/d higher than 6 ($a/d \geqslant 6.0$) are limited for all (NSC and HSC) beams; further research is therefore required to verify the found p_F.

6. Conclusions

The change in the shear strength reduction factor according to the ACI318 is investigated for different coefficients of variation and failure probabilities. The following conclusions can be drawn from the results of this study.

- It is found that ϕ of 0.75, which is a value recommended by the ACI318 (2002) and ACI318 (2011), corresponds to the target values of $p_F = 10^{-5}$ ($\beta = 4.27$) and $V_{fc} = 0.10$, whereas ϕ of 0.85, which is a value recommended by the ACI318 (1995), corresponds to the target values of $p_F = 10^{-2}$ ($\beta = 2.33$) and $V_{fc} = 0.10$.

- The values of ϕ for the considered beams are largely scattered and are not influenced significantly by f_c, a/d and f_{yw}. The ϕ of HSC beams with $(a/d) \geqslant 4.0$ are higher than 0.75 for $p_F = 10^{-5}$, $V_{fc} = 0.10$ and 0.85 for $p_F = 10^{-2}$, $V_{fc} = 0.10$. The beams with a/d higher than 6 ($a/d \geqslant 6.0$) and with $\rho_w \geqslant 0.5\%$ are limited for all (NSC and HSC) beams; further research is therefore required to verify the found p_F.
- It is observed that the ϕ decreases with increasing ρ_w for all beams.
- For given V_{f_c} and p_F, ϕ for the HSC beams are found to be greater than the one for the NSC beams, so it can be inferred that ϕ for the HSC beams is more safe than the one for the NSC beams.

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