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Development of the One-Way Shear Design Provisions of ACI 318-19 for Reinforced Concrete

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Relationships in ACI 318-14 for calculating the concrete contribution to shear resistance (V_c) in reinforced concrete (RC) members (that is, non-prestressed) have been replaced in ACI 318-19 by one general relationship that considers the combined effects of member depth, percentage of longitudinal reinforcement, and the effect of axial stress on predicted shear strength capacity. This new relationship is $V_c = (8\lambda_s\lambda[\rho_w]^{1/3}\sqrt{f_c'} + N_u/[6A_g])b_wd$, *where* λ_s *is a size effect factor equal to* $\sqrt{(2/[1 + d/10])}$ *that accounts for a reduction in shear stress capacity with increasing member depth. The frequently used expression in ACI 318-14,* $V_c = 2\lambda\sqrt{f_c'}b_wd$, *may continue to be used in members containing at least the minimum level of shear reinforcement. The one-way shear provisions for prestressed concrete (PC) members were not changed in this code cycle. The primary basis for the new RC provisions are test results compiled in databases developed and analyzed over the past two decades through a collaboration of Joint ACI-ASCE Committee 445, Shear and Torsion, and the German Committee for Structural Concrete (DAbStb). The process for developing these new provisions included an invitation to the ACI community to suggest new one-way shear design provisions. These suggestions were discussed within Joint ACI-ASCE Committee 445, and then evaluated and modified by ACI Subcommittee 318-E, Section and Member Strength, with consideration of their basis, accuracy, safety, ease-of-use, and range of application.*

Keywords: building code; database; design; experiments; shear.

INTRODUCTION TO ACI 318-14 SHEAR PROVISIONS

The philosophy of the one-way reinforced concrete (RC) (that is, non-prestressed) shear design provisions in the "Building Code Requirements for Structural Concrete (ACI 318-14)" have been largely unchanged since the 1963 Code (ACI Committee 318 1963). The calculated one-way shear capacity of a member by ACI 318-14 is the sum of the contribution of the shear reinforcement (V_s), the concrete (V_c), and the vertical component of prestressing steel (V_p), where V_s is based on a 45-degree parallel-chord truss model. When V_c was first introduced into the ACI code more than one century ago, it was taken as a fraction of the concrete compressive strength multiplied by the width and depth of the beam cross-section. This was changed in ACI 318-63 to be the estimated diagonal cracking strength as recommended in the Joint ACI-ASCE Committee 326 report on "Shear and Diagonal Tension" (Joint ACI-ASCE Committee 326 1962a,b,c). With this, V_c was made proportional to the square root of the specified concrete compressive strength. Additional equations for calculating V_c were added over time to consider the effects of moment, axial load, and percentage of longitudinal tension reinforcement. The ACI 318-14 one-way shear design provisions for V_c in RC members are presented in Table 1 in U.S. customary units.

While these V_c relationships may provide a reasonable estimate of diagonal cracking strength, there is no mechanistic basis for why the shear cracking strength is a reliable lower-bound estimate of the concrete contribution to shear resistance in the Ultimate Limit State. The use of these V_c relationships in members with A_v were empirically validated by laboratory beam tests.

This paper presents the process and outcome of a long-term effort within the ACI community to advance the one-way shear design provisions. The paper begins with a review of one-way shear design relationships in other codes of practice and suggested by researchers. A discussion is then presented on the complexity of shear resistance in beams, and the relevance of various potential design parameters. The process used in the selection and setting of provisions is then described. The paper concludes with a statistical evaluation of the accuracy and safety level of the new one-way shear equations in ACI 318-19 versus the ACI 318-14 design equations that were made using a large and vetted database of beam shear test results.

RESEARCH SIGNIFICANCE

Through hundreds of research studies conducted in the past few decades, significant advancements have been made in understanding how shear is carried in structural concrete. Many of these studies have recommended changes in the ACI 318 one-way shear provisions to address concerns regarding the low shear stress capacity observed in large members without shear reinforcement and/or members with a low percentage of longitudinal tension reinforcement. The new one-way reinforced concrete shear design provisions in ACI 318-19 address many of the concerns identified in research studies, and are presented and discussed in this paper.

REVIEW OF SELECTED SHEAR DESIGN METHODS

While the ACI 318 one-way shear provisions have been largely unchanged for decades, there have been significant

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Table 1—ACI 318-14 Reinforced concrete one-way shear design provisions

ACI 318-14*	Simplified method	Detailed method
V_n	$V_n = V_c + V_s$ $V_u \le \phi\left(V_c + 8\sqrt{f_c'}\right)b_w d$	Same as simplified method
V_c	$V_c = 2\lambda\sqrt{f_c'}b_w d$	Lesser of: $V_c = \left[1.9\lambda\sqrt{f_c'} + 2500\rho_w \frac{V_u d}{M_u}\right]b_w d$ $V_c = \left[1.9\lambda\sqrt{f_c'} + 2500\rho_w\right]b_w d$ $V_c \le 3.5\lambda\sqrt{f_c'}b_w d$
V_c with N_u	Axial compression $V_c = 2\lambda\left[1 + \frac{N_u}{2000A_g}\right]\sqrt{f_c'}b_w d$ Axial tension $V_c = 2\lambda\left[1 + \frac{N_u}{500A_g}\right]\sqrt{f_c'}b_w d$	Axial compression (lesser of): $V_c = \left[1.9\lambda\sqrt{f_c'} + 2500\rho_w \frac{V_u d}{M_u - N_u \frac{4h-d}{8}}\right]b_w d$ Equation not applicable if $M_u - N_u \frac{4h-d}{8} < 0$ $V_c = 3.5\lambda\sqrt{f_c'}b_w d\left[1 + \frac{N_u}{500A_g}\right]$
V_s	$V_s = \frac{A_v f_{yt} d}{s}$	

Notes: Not all limits and requirements are indicated. Units: lb, inch, and psi.

changes in shear design provisions in other building code documents including CSA 23.3 in 1994, AASHTO-LRFD in 1994, the *fib* Practical Design of Structural Concrete in 1999, and the *fib* Model Code 2010, as well as in the types of approaches proposed by researchers (Bažant et al. 2007; Choi et al. 2007; Cladera et al. 2016; Pang and Hsu 1995; Wolf and Frosch 2007). The different approaches raise important issues to consider in improving the one-way provisions in ACI 318, including: 1) the size effect in shear; 2) similarity in V_c for members with and without shear reinforcement; 3) effect of axial compression and prestressing on V_c; 4) use of a 45-degree or variable-angle truss model for calculating the contributions of shear reinforcement; 5) the number of relationships for calculating V_c; and 6) other factors that could be considered in code relationships, including effect of distributed longitudinal reinforcement, effect of flanges, crack roughness, crack widths, longitudinal strain, depth of compression, and percentage of longitudinal reinforcement.

To assess if and how to update the ACI 318 one-way shear provisions for reinforced concrete, an examination was made of selected one-way shear design methods in other codes of practice as well as those proposed by researchers. A selected number of these are presented herein.

EuroCode2 (EC2) Part 1 (1992)

For members without shear reinforcement (A_v), the concrete stress contribution considers the influence of concrete strength (f_c'), longitudinal reinforcement ratio (ρ_w), member depth (d), and axial stress (N_u/A_g) as presented in Eq. (1) in U.S. customary units

$$V_c = \left[3.32\left(1 + \sqrt{16.67/d}\right)\left(100\rho_w\left(f_c' - 232\right)\right)^{1/3} + 0.15N_u/A_g\right]b_w d \tag{1}$$

where the value of N_u/A_g is limited to $\approx 0.12 f_c'$.

The diagonal cracking strength for members subjected to axial compression may be derived from elastic stress distributions that consider the influence of longitudinal compression stress on shear, as shown in Eq. (2)

$$V_c = \frac{Ib_w}{S}\sqrt{f_{cr}^2 + \sigma_{cp}f_{cr}} \tag{2}$$

where f_{cr} is the concrete cracking strength; I is the moment of inertia; S is the first moment of area above and about the centroidal axis; and σ_{cp} is the axial compressive stress at the gross centroidal axis equal to N_u/A_g (equivalent to f_{pc} for prestressed concrete members in ACI 318-19).

The contribution of shear reinforcement A_v to shear resistance V_s is determined using a variable-angle truss model as shown in Eq. (3). The designer may choose the angle of diagonal compression (θ) to use, but has to directly check that the diagonal compressive strength is sufficient. These provisions are based partly on plasticity theory as developed by Nielsen (1967) and Lampert and Thürlimann (1969)

$$V_s = \frac{A_v f_{yt} d\cot(\theta)}{s} \tag{3}$$

where s is the spacing of the shear reinforcement

Canadian Standards Association A23.3 (2004)

This method was derived from the Modified Compression Field Theory (MCFT) by Vecchio and Collins (1986). The shear strength is taken as the sum of the contribution from concrete V_c, shear reinforcement V_s, and vertical component of the prestressing reinforcement V_p. This is shown in Eq. (4), where V_c and V_s are controlled by β and θ; in which β is the shear stress coefficient (same as the role of 2 in the ACI 318-14 expression that $V_c = 2\sqrt{f_c'}b_w d$), and θ is the angle of diagonal compression from the longitudinal axis of the member. Both β and θ are functions of the longitudinal strain at middepth (refer to Eq. (5) and (6)), which may be taken as one-half of strain in the longitudinal tension reinforcement ($\varepsilon_x = \varepsilon_s/2$) as shown in Eq. (7). The numerator in Eq. (7) is the demand on the longitudinal reinforcement from all actions (moment, shear, axial loading, and prestressing), and the denominator is the axial stiffness of the reinforcement; this is then divided by 2 to provide the approximate longitudinal strain at middepth. The shear depth d_v may be taken as $0.9d$

$$V_n = V_c + V_s + V_p = \phi_c \lambda \beta \sqrt{f_c'} b_w d_v + \frac{\phi_s A_v f_y d_v \cot\theta}{s} + V_p \quad (4)$$

$$\beta = \frac{4.5}{(1+1500\varepsilon_x)} \cdot \frac{50}{(38+d_v)} \quad (5)$$

$$\theta = 29 + 7000\varepsilon_x \quad (6)$$

$$\varepsilon_x = \frac{\varepsilon_s}{2} = \frac{M_f / d_v + V_f - V_p + 0.5N_f - A_p f_{po}}{2(E_s A_s + E_p A_p)} \quad (7)$$

It is useful to recognize that the equation for β consists of two parts multiplied together. When there is shear reinforcement, or when d_v = 12 in., the second part is set or equal to 1. When ε_x = 0.001 (approximately half of the yield strain in Grade 60 reinforcement), the first part is equal to 2. Thereby, Eq. (4) yields the most commonly used expression for V_c in ACI 318. If there is no shear reinforcement, the concrete contribution in Eq. (4) decreases with increasing depth.

The shear design provisions in the AASHTO-LRFD 1994 specifications are very similar to the CSA A23.3 provisions with the exception that ε_s rather than ε_x is used as the controlling parameter; the relationships for β and θ being adjusted to provide the same values as when the CSA method is used (Kuchma et al. 2008).

fib Model Code 2010

The *fib* Model Code 2010 (*fib* 2012) introduced a "Levels of Approximation" approach to the design and analysis of concrete structures, which recognizes that different levels of design effort and accuracy are warranted for different design situations. Level 1 generally involves the least effort, fewest parameters, and makes conservative assumptions so to provide a safe estimate of capacity; this level is most useful for preliminary and typical design cases. Level 4 generally involves the use of more parameters, additional calculations, and provides a more accurate and less conservative capacity. Level 4 is more appropriate for capacity rating and design optimization.

There are two design requirement levels for members without shear reinforcement, and these are much the same as those in CSA A23.3, where Level 1 uses the conservative fixed value for ε_x of 0.001, and Level 2 evaluates ε_x using Eq. (7).

For members with shear reinforcement, there are three levels. In Level 1, only the contribution of shear reinforcement is considered and fixed minimum values are given for the angle of the diagonal compression field in evaluating V_s. Level 2 also only considers V_s, but the minimum angle of diagonal compression is based the calculated value of ε_x. Level 3 is essentially the CSA general method in which both V_c and V_s are functions of ε_x.

Methods based on depth of compression

Tureyen and Frosch (2003), Choi et al. (2007), and Cladera et al. (2016) have suggested that the depth of uncracked concrete in compression is a better geometric parameter to use than the depth of the beam for calculating the concrete contribution to shear resistance. This method was also adopted in the ACI 440 guidelines (ACI Committee 440 2015), which is intended for the design of beams with fiber-reinforced polymer (FRP) flexural reinforcement. An advantage of this approach is that it directly accounts for the effect of moment, axial load, and stiffness of the longitudinal tension reinforcement. For members without axial loadings, the depth of compression can be taken as the cracked section depth and determined directly from the ratio of longitudinal reinforcement ($\rho_w = A_s/b_w d$) and the ratio of the reinforcement to concrete stiffness modulus ($n = E_s/E_c$) or E_{frp}/E_c) as was proposed by Frosch et al. (2017) in Eq. (8) and (9), and which assumes linear elastic material behavior.

$$V_c = 5\sqrt{f_c'}b_w c \quad (8)$$

$$\text{where } c = kd = \left(\sqrt{2n\rho - (n\rho)^2} + n\rho\right)d \quad (9)$$

For members with flanges, the effective width in compression should be taken into consideration because the compression zone chord extends into the flange. For members subjected to prestressing or axial load, a strain compatibility approach is needed to determine the depth of compression. For these types of members, researchers have proposed relationships for evaluating this depth of compression (Cladera et al. 2017). In Frosch et al. (2017), Eq. (8) is to be multiplied by a size effect factors derived from fracture mechanics.

Methods based on fracture mechanics

Fracture mechanics can be used to predict the propagation of cracks and when they become critical. These methods have been successfully applied to brittle and quasi-brittle materials such as plain concrete. For more than three decades, members of the fracture mechanics community have been advocating for the ACI 318 one-way shear provisions to consider a size effect (Bažant 1984). While there have been some modifications since 1984, the core size effect on shear stress contribution of the concrete at ultimate has remained the same as shown in Eq. (10), where d_o is a material constant; this approach continued to be endorsed by ACI Committee 446, Fracture Mechanics (Bažant et al. 2007). These methods suggest that there is still a size effect in members that contain shear reinforcement but that this effect is modest in comparison to that in members without shear reinforcement

$$v_c \propto \frac{1}{\sqrt{1+\frac{d}{d_o}}} \qquad (10)$$

COMPLEXITY OF SHEAR RESISTANCE IN PANELS VERSUS BEAMS

There has been tremendous advancement in understanding how shear is carried in panels, and there is now close agreement between the leading models such as those by Vecchio and Collins (1986), Pang and Hsu (1995), and Kaufmann and Marti (1998). All of these smeared cracking models combine compatibility, material constitutive relationships, and equilibrium such that they can predict the full shear force versus shear deformation of panels including the state of stress in the reinforcement and in the cracked concrete. By contrast, there are significant differences in the calculated shear strength of beams such as by the methods presented in the previous section in which the calculated shear strength of a beam by these relationships can differ by more than a factor of 2. The reason that the shear strength of beams is more difficult to predict than panels is because in beams: 1) there is a variation in the longitudinal strain over the depth of beams and along the length of beams, as shown in Fig. 1; 2) the longitudinal reinforcement is usually not uniformly distributed over the depth of the beam; 3) the points of loading and support create a more complex state of stress; 4) there may be flanges; 5) prestressing can introduce non-uniform stress states; and 6) there are complex interactions between these and other factors. Researchers do agree that the V_c contribution at nominal strength in beams is through a combination of aggregate interlock (interface shear transfer), shear in the compression zone, dowel action, and residual tensile stresses across cracks, as illustrated in Fig. 2. The first two of these contributions are generally accepted to provide most of the "concrete contribution" to shear resistance as discussed in the Joint ACI-ASCE Committee 445 report, "Recent Approaches to the Shear Design of Structural Concrete" (Joint ACI-ASCE Committee 445 1999). Because of these complexities and all of the influencing factors, it is not surprising that researchers have examined different factors in their testing programs and suggested different design relationships and methods that focus on the effects of the parameters investigated in their particular study.

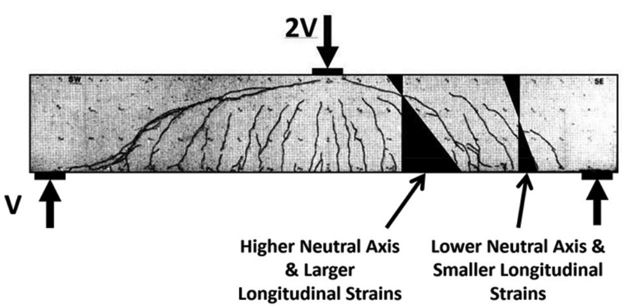


Fig. 1—Longitudinal straining at two sections of reinforced concrete beam.

FACTORS CONSIDERED IN REVISING THE ACI 318 ONE-WAY SHEAR PROVISIONS

The role of technical committees such as committee Joint ACI-ASCE Committee 445, Shear and Torsion, is to assemble experimental evidence and various proposed approaches, and discuss the technical merits of different methods (Joint ACI-ASCE Committee 445 1999). The role of building code committees and subcommittees such as ACI Subcommittee 318-E, Sectional and Member Strength, is to consider the technical state-of-the-art, complexity in use, and range of application so to provide clear and sufficiently conservative design provisions. In this one-way shear effort,

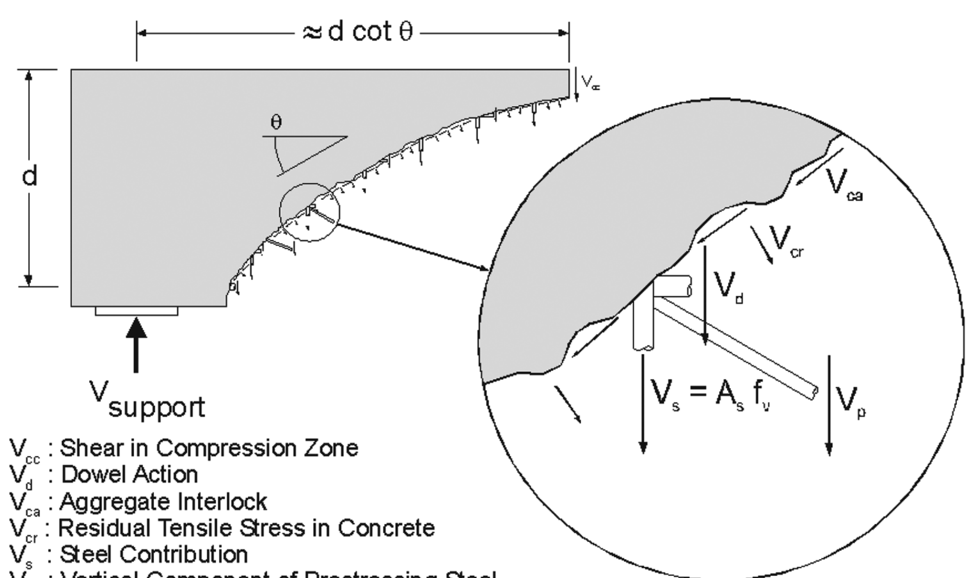


Fig. 2—Components of shear resistance in structural concrete beams.

ACI Subcommittee 318-E, with support from ACI Subcommittee Joint ACI-ASCE Subcommittee 445-D, Shear Databases, examined the safety of the suggested shear design methods for the code by comparing the ratio of the measured shear strength of test beams with the calculated strength of each suggested method for the ACI 318-19 building code. This is referred to as the strength ratio (SR) = V_{test}/V_{code} throughout this paper. In this process, plots were made of the SR as a function of current and potential design parameters such as d, ρ_w, f_c', $V_u d/M_u$, ε_x, N_u/A_g, and θ, so to assess trends and thereby the importance of including these parameters in shear design formula. In the selection and development of the new ACI 318-19 one-way shear requirements, four considerations were given particular attention, as discussed in the following.

1. *Influence of percentage of longitudinal tension reinforcement ($\rho_w = A_s/b_w d$), moment, or longitudinal strain*—There is no consensus on if and how to consider the effect of these factors. The effect of ρ_w is considered in evaluating V_c in some ACI 318-14 provisions, in methods based on shear being primarily carried over the depth of compression, in longitudinal strain-based methods, and in other empirical methods. It is also used in evaluating V_s in some methods. However, the form and magnitude of this influence varies. For example, in the detailed method of ACI 318-14, and in CSA, AASHTO-LRFD, and MC2010 (Level 3) methods, shear capacity is a function of both ρ_w and M_u, but then doubling both has no effect on shear capacity (other factors being constant). In EC2, and in methods where V_c is proportional to the depth of compression c, there is an increase in shear capacity with ρ_w regardless of the magnitude of the bending moment. It is also worthwhile to note that both EC2 (basis being aggregate interlock), and depth of compression approaches suggest that shear stress resistance is proportional to $\approx (\rho_w)^{1/3}$.

2. *Influence of axial load and prestressing*—The effect of axial compressive stress on V_c is considered in five ACI 318-14 equations for V_c, and the effect from compressive stress from external loading and prestressing is not treated the same. EC2 considers the effect of axial load on both V_c and V_s, and treats the effects of prestressing and axial load in a consistent manner for members that are cracked in flexure. CSA has V_c and V_s values be a function of longitudinal strain which is calculated considering the effects of axial load, moment, shear, prestressing, stiffness of reinforcement, and axial stress.

3. *45-degree or variable angle truss model*—The ACI 318 code has always used a 45-degree truss model for evaluating the contributions of transverse reinforcement to shear resistance. By contrast, European codes have permitted the use of a variable-angle truss model based on the theory of plasticity. With EC2, the designer may choose the angle of diagonal compression θ, with limits placed on this angle to guard against diagonal compression failures. CSA A23.3 provides two options: 1) a simplified method with a 35-degree fixed angle truss model; and 2) a general method in which the angle of the truss is a function of the longitudinal strain near middepth. The *fib* Model Code 2010 is a hybrid of both EC2 and the CSA method, and provides guidance for minimum angles of diagonal compression, freedom for the designer to choose this angle, and a CSA-type method for calculating V_s based the longitudinal strain at middepth. Because design requirements are evaluated by test data, it is important to recognize the interaction between V_c and the assumed V_s when interpreting test data. For example, one may consider that $V_{c,test} = V_{test} - V_s$, where V_s is evaluated using a 45-degree truss model. The actual V_s from the test may be quite different from this because of the effect of axial compression that decreases the angle of diagonal cracking (or compression) such that the number of stirrups that are picking up the load is $d\cot(\theta)/s$ and thereby different than what ACI would suggest (d/s). This difference can be more than a factor of 2. There are other interacting effects such as the influence of the amount of V_s on resistance to shear sliding, which makes V_c difficult to isolate.

4. *Number of shear design equations and methods*—If limiting criteria are neglected, then ACI 318-14 has eight one-way shear design relationships for evaluating V_c for reinforced concrete, and one for evaluating V_s. The V_c in ACI is generally considered to be the diagonal cracking load and V_s is based on a 45-degree parallel chord truss model. The eighth edition of the AASHTO-LRFD bridge design specifications (AASHTO 2017) has two different types of methods for evaluating shear resistance, one very much like ACI 318-14 but with only two different equations for V_c, and one for V_s that uses a variable angle truss model where the angle of diagonal compression is taken as the estimated angle of shear cracking (Kuchma et al. 2008). It also employs the equivalent to the CSA general method, and provides an earlier version of the CSA method in an appendix, in which V_c and V_s are taken from tabular values and iteration is often required for design. For members without shear reinforcement, *fib* Model Code 2010 has one method for calculating V_c that is similar to the CSA but with two levels of approximation. For members with shear reinforcement, *fib* Model Code 2010 has one method for evaluating V_c as a function of longitudinal strain, and then three levels of approximation for calculating V_s and how it is added to V_c.

The influence of each of these four factors was considered in detail during the evaluation process using plots of SRs with relevant parameters to examine trends.

PROCESS USED IN SELECTION AND EVALUATION OF ONE-WAY SHEAR PROVISIONS

The steps in the development of the ACI 318-19 one-way shear provisions were: 1) the development of shear test databases; 2) an open call by Joint ACI-ASCE Committee 445 and ACI Subcommittee 318-E for proposals to improve ACI 318 one-way shear design provisions; 3) comparative assessment of the accuracy of different proposals for calculating one-way shear capacity; 4) discussions primarily within ACI Subcommittee 318-E on the relative merits of the different methods and how they could be modified to balance accuracy, ease of use, and transparency; and 5) adjusting coefficient values in shear strength equations to achieve the necessary level of safety and checking trends in SR (V_{test}/V_{code}) with key design or behavior characteristics. Each of these five elements of the process is now briefly described.

A database of one-way shear test results was created and examined through a joint effort of Joint ACI-ASCE Committee 445 and the German Committee of Reinforced Concrete (DAbStb), as presented in Reineck et al. (2013, 2014). The datasets from this database used consisted of 784 RC members without shear reinforcement (A_v), 170 tests on RC members with $A_v \geq A_{v,min}$, 214 PC members without A_v, and 117 prestressed members with $A_v \geq A_{v,min}$. These databases did not include tests in which yielding of the flexural reinforcement was suspected, beams that were very small, beams with suspected anchorage failures, and for other reasons as explained in Reineck et al. (2013, 2014). Because prestressing introduces an axial stress as does axial compression (N_u), the tests were characterized as follows: 1) without A_v and no N_u; 2) with A_v and no N_u; 3) without A_v and with N_u; and 4) with A_v and with N_u. Members subjected to cyclic loads were excluded.

In response to the call by Joint ACI-ASCE Committee 445 for suggested shear design provisions in 2014, 10 different proposals were received. Individuals and groups with similar suggestions were encouraged to combine their ideas, and this reduced the number to six proposals: Bentz and Collins (2017), Cladera et al. (2017), Frosch et al. (2017), Li et al. (2017), Park and Choi (2017), and Reineck (2017). These proposals were published in *Concrete International*; refer to Belarbi et al. (2017).

ACI Subcommittee 318-E reviewed the six proposals as well as provisions in other national codes-of-practice. The subcommittee established an assessment process that included examining accuracy for several parameters or range of values.

One of the significant assumptions that had to be made in the assessment process was the definition of the critical section to use for computing SR (V_{test}/V_{code}). This is particularly important for evaluating methods in which the suggested strength equation is a function of the level of moment M_u or M_u/V_ud at each section. For these cases, the location for the lowest SR is likely to be at $d\cot\theta/2$ from the point of loading. Other arguable locations for evaluation are at the location of observed failure or at d from the support, where the maximum shear force was supported. In the evaluation presented in this paper, the critical section was taken to be at one half of the length of the shear-span from the support except as noted otherwise.

COMPARATIVE ACCURACY AND SAFETY OF ACI 318-19 AND ACI 318-14 PROVISIONS

A comparison was made between the between the one-way shear provisions in ACI 318-19 versus the corresponding simplified and detailed methods in ACI 318-14. The ACI 318-19 provisions are presented in Table 2 with the corresponding footnotes, while the ACI 318-14 provisions are presented in Table 1.

The size effect factor, λ_s, is given by Eq. (11), which identifies that the reduction in shear strength with depth begins at 10 in.

Table 2—ACI 318-19 one-way shear provisions for reinforced concrete members

Criteria		V_c	
$A_v \geq A_{vmin}$	Either of:	$\left[2\lambda\sqrt{f_c'} + \frac{N_u}{6A_g}\right]b_w d$	(a)
		$\left[8\lambda(\rho_w)^{1/3}\sqrt{f_c'} + \frac{N_u}{6A_g}\right]b_w d$	(b)
$A_v < A_{vmin}$		$\left[8\lambda\lambda_s(\rho_w)^{1/3}\sqrt{f_c'} + \frac{N_u}{6A_g}\right]b_w d$	(c)

Notes: Axial load N_u is positive for compression and negative for tension; V_c shall not be taken less than zero; V_c shall not be taken greater than $5\lambda b_w d$; and $N_u/(6A_g)$ shall not be taken greater than $0.05f_c'$.

Table 3—Cases where $A_{v,min}$ is not required if $V_u \leq \phi V_c$

Beam type	Condition
Shallow beam	$h \leq 10$ in.
Integral with slab	$h \leq$ greater of $2.5t_f$ or $0.5b_w$ and $h \leq 24$ in.
Constructed with steel fiber-reinforced normal-weight concrete conforming to specific requirements and with $f_c' \leq 6000$ psi	$h \leq 24$ in. and $V_u \leq \phi 2\sqrt{f_c'}b_w d$
One-way joist system	Conforming to specific requirements

Notes: 1 in. = 25.4 mm; 1 psi = 0.0069 MPa.

$$\lambda_s = \sqrt{\frac{2}{1+\frac{d}{10}}} \quad (11)$$

For nonprestressed beams, a minimum area of shear reinforcement, $A_{v,min}$, is required in all regions where $V_u > \phi\lambda\sqrt{f_c'}b_w d$, except for the cases in Table 3. When a minimum amount of shear reinforcement is provided, the longstanding relationship $Vc = 2\lambda\sqrt{f_c'}b_w d$ may continue to be used.

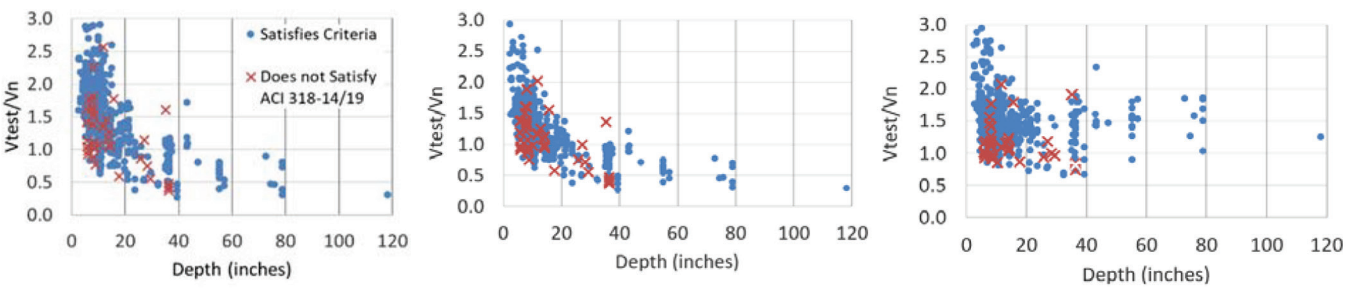
The cases in Table 3 are the same as in ACI 318-14, but $A_{v,min}$ is required when $V_u > \phi\lambda\sqrt{f_c'}b_w d$ instead of $\phi V_c/2$. In ACI 318-14, V_c is usually taken as $2\lambda\sqrt{f_c'}b_w d$, and therefore the value of $\phi V_c/2$ will not change for many cases.

In Table 4, the Mean SR (V_{test}/V_{code}) and associated coefficient of variation (COV) are presented for the new ACI 318-19 provisions (case (b) or (c) in Table 2), and the ACI 318-14 Simplified and Detailed provisions given in Table 1 for each of the four databases previously introduced. In Table 4, the percentage of beam test data with shear capacities less than $\phi V_n = 0.75V_n$ are shown. Test results were not included in the statistical calculations when:

- f_c' was greater than 12,000 psi for members without shear reinforcement;
- Yield strength of the provided shear reinforcement, f_{yt}, exceeded 80,000 psi;
- Strength of the provided shear reinforcement exceeded $V_s = 8\sqrt{f_c'}b_w d$.

Table 4—One-way shear strength ratio (V_{test}/V_{code}) statistics

	$N_u = 0$ (non-prestressed)						Compressive N_u (prestressed)					
	No Av 742 beams			With Av 101 beams			No Av 202 beams			With Av 81 beams		
Method	Mean	COV	<0.75	Mean	COV	<0.75	Mean	COV	<0.75	Mean	COV	<0.75
ACI 318-14 Simplified at a/2	1.51	0.38	6.2%	1.46	0.24	2.0%	2.33	0.36	0.0%	1.81	0.19	0.0%
ACI 318-14 Detailed at d	1.10	0.30	9.6%	1.20	0.20	3.0%	1.29	0.33	5.9%	1.20	0.19	1.2%
ACI 318-14 Detailed at a/2	1.25	0.30	7.0%	1.30	0.21	3.0%	1.52	0.27	0.0%	1.42	0.14	0.0%
ACI 318-14 Detailed at a-d/2	1.37	0.32	6.2%	1.38	0.22	3.0%	1.98	0.25	0.0%	1.85	0.16	0.0%
ACI 318-19 expression with ρw	1.48	0.24	0.5%	1.32	0.20	0.0%	1.94	0.29	0.0%	1.53	0.18	0.0%



(a) ACI 318-14 Simplified (b) ACI 318-14 Detailed (c) ACI 318-19

Fig. 3—Impact of depth on strength ratios for members without A_v *or* N_u.

While these limits on f_c' and f_{yt} are greater than the ACI 318-14 limits, they are included so to have a large number of test results to evaluate accuracy of strength calculations.

When considering the statistical values shown in Table 4, it must be recognized that databases are biased towards the types of beams usually tested in laboratories (that is, small, over-reinforced in flexure, and without shear reinforcement). To examine such bias, trends in SR with key design parameters were plotted as a function of the following parameters: depth (d); shear span-to-depth ratio (a/d) or (M_u/V_ud); longitudinal reinforcement ratio (ρ_w); concrete compressive strength (f_c'); normalized amount of shear reinforcement index ($\rho_v f_{yt}/\sqrt{f_c'}$); yield strength of transverse shear reinforcement (f_{yt}); and level of axial stress at the centroidal axis (N_u/A_g). This was done for the SRs associated with the ACI 318-14 Simplified and Detailed provisions, as well as the new ACI 318-19 provisions using test results in the four aforementioned databases. The results of this are presented in the next section of this paper. In these upcoming plots, all of the test data was included and no limitations were placed on f_c', f_{yt}, and V_s when evaluating SRs (that is, $V_n = V_c + V_s$) for the points in these plots even if these values are limited in ACI 318. If limits had been placed, then this would have led to higher than realistic impressions of safety. For example, if the actual yield strength of the transverse reinforcement was 70,000 psi, whereas the ACI 318 limit is 60,000 psi, the use of 60,000 psi in calculating V_s by ACI 318 would likely underestimate the actual stress in this reinforcement and thereby V_s in the test beam at ultimate. To distinguish between the test results used in generating the statistics in Table 4, a round marker is used for points satisfying $f_c' \leq$ 12,000 psi (no A_v only), $f_{yt} \leq$ 80,000 psi, and $V_s \leq 8\sqrt{f_c'}b_wd$, whereas an X-shaped marker is used for test results where any of these limits is exceeded.

Comparative assessment of ACI 318-14 versus ACI 318-19 for members without A_v or N_u

In Fig. 3, the SR versus depth is plotted for RC members without any shear reinforcement or axial load. For both the ACI 318-14 Simplified (Fig. 3(a)) and Detailed (Fig. 3(b)) methods, there are many cases in which the beam failed under a shear force lower than the strength calculated by the code (SR < 1.0), some with failure loads less than 50% of the code (SR < 0.5), and a strong trend of lower SRs with increasing depth. In contrast, for ACI 318-19 (Fig. 3(c)), there are only a small number of SRs less than 1.0 and very few SRs less than 0.75 (φ factor for shear); there is also no obvious trend of decreasing SR with depth.

In Fig. 4, the influence of a/d (equivalent to M_u/V_ud) on SR is shown. The ACI 318-19 and ACI 318-14 Simplified methods do not consider the impact of a/d. The results suggest that there is no appreciable benefit in considering a/d, and that the ACI 318-19 provisions result in the narrowest range of SRs over the range of a/d.

In Fig. 5, the influence of longitudinal reinforcement ratio on SR is shown in which $\rho_w = A_s/(b_wd)$. As shown in

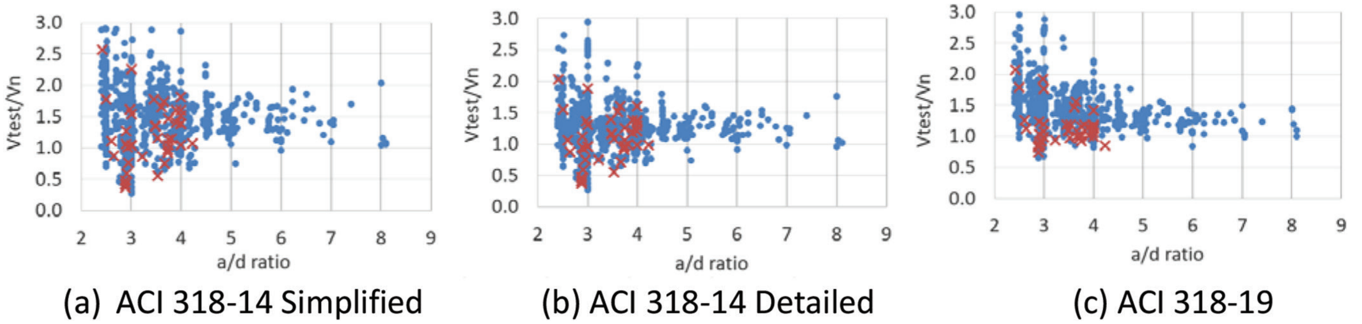


Fig. 4—Impact of a/d on strength ratios for members without A_v *or* N_u.

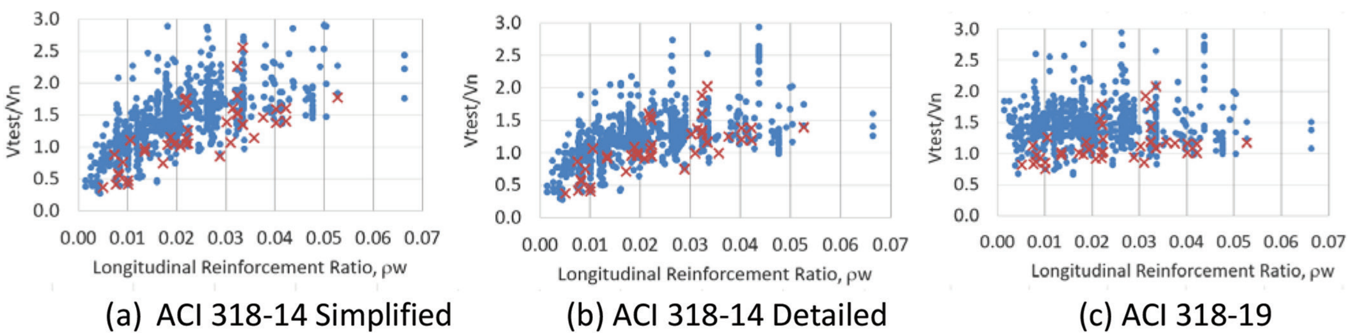


Fig. 5—Impact of ρ_w *on strength ratios for members without* A_v *or* N_u.

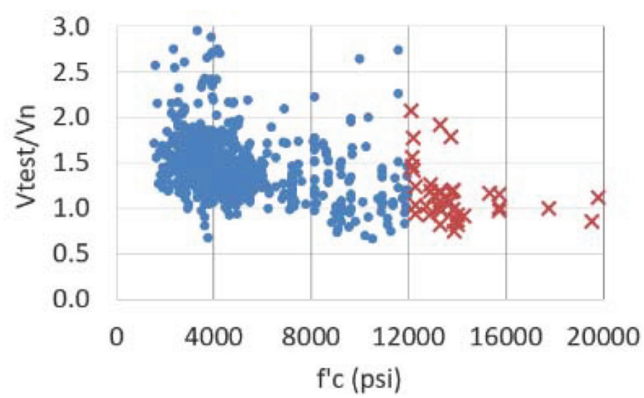


Fig. 6—Impact of f_c' *on strength ratios of members without* A_v *or* N_u *(ACI 318-19).*

Fig. 5(a), the ACI 318-14 Simplified equation, which does not consider ρ_w, shows a very strong trend with ρ_w. The ACI 318-14 Detailed method includes ρ_w but underestimates its effect, as shown in Fig. 5(b). The ACI 318-19 method directly considers the longitudinal reinforcement ratio by making V_c proportional to $(\rho_w)^{1/3}$, and with this there is no appreciable trend in SR with ρ_w for the ACI 318-19 provisions, as shown in Fig. 5(c).

In Fig. 6, there is a very slight downward trend of decreasing SRs with increasing concrete strength for the ACI 318-19. A similar trend was also observed for ACI 318-14 relationships because both methods are based on shear stress capacity being proportional to $(f_c')^{1/2}$. This trend is not present in EC2 because shear stress capacity is proportional to $(f_c')^{1/3}$. It was decided to keep V_c a function of $(f_c')^{1/2}$ for consistency with other provisions.

Key observations on effectiveness of ACI 318-19 one-way shear provisions

When $A_v \geq A_{v,min}$, ACI 318-19 does not consider a size effect in shear (for example, shear stress capacity decreasing with member depth). Figure 7(a) shows a slight downward trend in the ratio of V_{test}/V_{code} for the shear database without prestressing and for which $A_v > A_{v,min}$, and there are a few test results for which the beam test strength are between 70 and 75% of the calculated strength by the ACI318-19 method. Because only a few tests yielded low shear strength ratios, it was not considered necessary to include a size effect for members with minimum shear reinforcement in the new provisions. There is also no trend in SR with ρ_w, f_{yt}, or $\rho_v f_{yt}/\sqrt{f_c'}$ as shown in Fig. 7(b) to (d). The results presented in Fig. 7(c) suggest that f_{yt} up to 80,000 psi could be used with no significant change in the range of SR values. Figure 7(d) shows that the ACI 318-14 limit on v_s of $8\sqrt{f_c'}$ is very conservative as there is no trend in SRs for V_s of up to $20\sqrt{f_c'}$. The ACI 318-14 limit on v_s of $8\sqrt{f_c'}$ was retained in ACI 318-19.

In Fig. 8, SRs are shown for members without A_v but with net axial compressive stress which was due to prestressing in the test database, and is considered as N_u/A_g in the use of the ACI 318-19 provisions. As shown in Fig. 8(a), there is no significant trend in SR with depth. An increasing SR with decreasing a/d less than 4 is shown in Fig. 8(b); these much higher SRs at low a/d are expected to be from arch action that is more effective when there is a net axial compression in members and where the member may be uncracked in flexure. The impact of a/d was not directly considered in the new provisions. Because most members have a slenderness ratio $a/d > 4$, the performance of the ACI 318-19 shear provisions is considered reasonable in this range. Figure 8(c) shows no discernable trend with increasing levels of axial compressive stress.

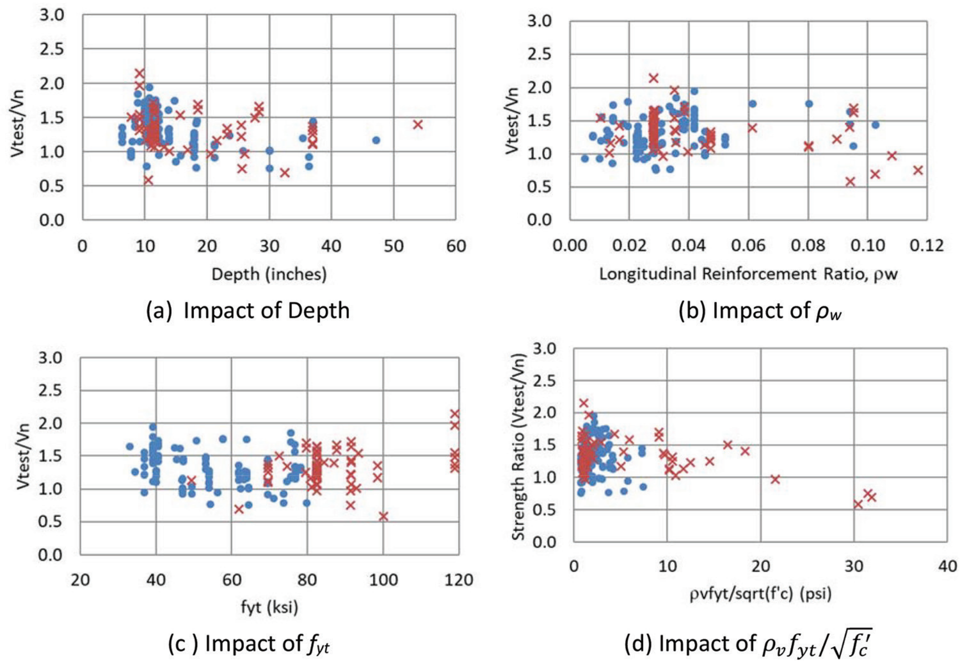


Fig. 7—Strength ratios in members with A_v *but no* N_u *(ACI 318-19).*

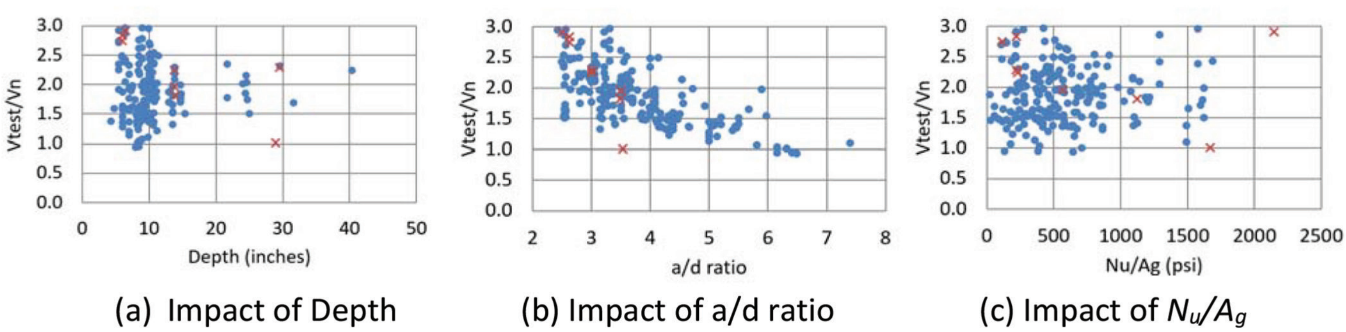


Fig. 8—Strength ratios in members without A_v *and with* N_u *(ACI 318-19).*

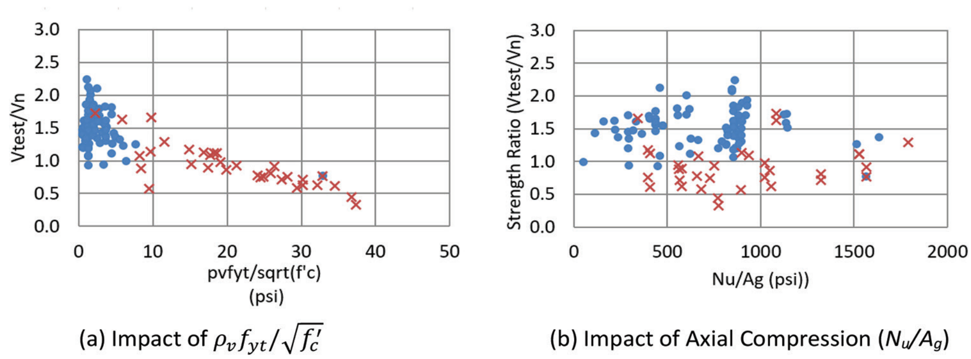


Fig. 9—Strength ratios in members with A_v *and* N_u *(ACI 318-19).*

In Fig. 9, SRs are shown for the new ACI 318-19 provisions for members with $A_v \geq A_{v,min}$ and $N_u > 0$. The SRs for all test beams in which V_s is greater $8\sqrt{f_c'}$ (ACI limit) are shown with an X-shaped marker. As shown in Fig. 9(a), there is no significant downward trend until V_s exceeds $20\sqrt{f_c'}$, suggesting that ACI 318 may be unnecessarily conservative.

SUMMARY AND CONCLUSIONS

There are significant differences in shear design provisions in various codes of practice, and in the parameters used in calculating shear strength such as depth, percentage of longitudinal reinforcement, axial loadings, slenderness, compressive strength, angle of diagonal compression, and magnitude of moment. The differences in calculated strengths by different codes can differ my more than a factor of 2.

Many different concerns have been expressed with the ACI 318 one-way shear provisions over the past few decades, including: 1) V_c does not consider a size effect in shear stress capacity for members without shear reinforcement; 2) V_c is the same for members with and without shear reinforcement;

3) V_c at nominal is taken as the diagonal cracking strength; 4) the effect of axial compression and prestressing on V_c is considered differently; 5) the angle of diagonal compression is fixed at 45 degree; 6) there are too many relationships for V_c for different conditions; 7) several influencing factors are not directly considered; and 8) design relationships are calibrated by laboratory test beams that are not representative of what is common in practice.

The process used in ACI for the updating of the one-way shear provisions in ACI 318 included: 1) the development of an extensive and vetted experimental database; 2) an open call for proposals for new provisions; 3) public presentation and discussions of the proposals at ACI committee meetings and sessions; and 4) an assessment of the accuracy of different approaches for calculating shear strength including multiple ballots within ACI Subcommittee 318-E and ACI Committee 318. A new relationship for one-way shear strength for non-prestressed members was adopted that considers the influence of the size effect with depth, percentage of longitudinal reinforcement, and the effect of axial loading on predicted shear stress capacity, as presented in Eq. (12)

$$V_c = \left(8\lambda_s\lambda\left(\rho_w\right)^{\frac{1}{3}}\sqrt{f_c'} + N_u/\left(6A_g\right)\right)b_w d \qquad (12)$$

In this expression, λ_s is a size effect factor equal to $\sqrt{2/\left(1+\frac{d}{10}\right)}$, which considers the effect of shear stress capacity decreasing with increasing depth. Equation (12) made it possible to remove some of the expressions for V_c in reinforced concrete members and to simplify design by removing dependence on moment-to-shear ratio.

Other significant findings and observations are:

1. The use of minimum shear reinforcement was deemed to sufficiently reduce the size effect for concrete shear strength such that the long-standing relationship that basic $V_c = 2\lambda\sqrt{f_c'}b_w d$ is retained.

2. The limits used in ACI 318-14, such as $f_c' \leq 10{,}000$ psi for no A_v, $f_{yt} \leq 60{,}000$ psi, and $V_s \leq 8\sqrt{f_c'}b_w d$, were maintained. The statistical evaluation suggests that some of these limits may be unnecessarily conservative, and should be further evaluated.

3. The 45-degree parallel chord truss model is effective and conservative. In the future, the addition of a variable-chord truss model may wish to be considered, particular, if the limit on V_s is raised.

4. While the shear strength of beams is dependent on many factors including those listed earlier in the conclusions, it was determined to not be practical to consider these effects in shear design requirements for new construction. However, the development of improved model-based and empirically derived relationships for calculating shear resistance would improve the capacity rating of existing structures such as is done in *fib* Model Code (2010).

5. Future experimental research studies should focus on understanding the shear behavior of members that reflect typical practices including members with flanges, continuous members, large members, flexure-critical design, new types of reinforcement, openings, prestressed and post-tensioned members, members subjected to cyclic loading, and member simultaneously subjected to significant torsion.

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CONVERSIONS

1 lb = 4.45 N

1 in. = 25.4 mm

1 psi = 0.00689 MPa

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