AUBURN UNIVERSITY
Samuel Ginn College of Engineering

Final Research Report

RELIABILITY EVALUATION OF ACI 318 STRENGTH REDUCTION FACTOR FOR ONE-WAY SHEAR

Submitted to

Concrete Research Council
ACI Foundation

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16. Abstract

A collaborative effort of several ACI technical committees led to the adoption of new ACI 318-19 one-way shear strength design provisions that addressed the influence of size effect and longitudinal reinforcement. These improved provisions provided an opportunity for a rational re-evaluation of the strength reduction (“phi”) factor for shear. This report describes a study with the objectives of providing a statistical basis for evaluating and improving the strength reduction factor for one-way shear and recommending an updated strength reduction factor if justified.

The reliability analysis procedure employed Monte Carlo simulation. The in-place concrete strength uncertainty was updated to reflect relevant test data. Considerations for uncertainty in effective depth were specifically revised for one-way shear strength. A CRSI database was analyzed to update the statistical parameters of bars commonly used as shear reinforcement. Data-driven professional factors, based on analyses of the ACI 445 databases, were used instead of expert opinion estimates as in the past. Simulated scenarios included: small, medium, and large size members; light, moderate, and heavy flexural reinforcement; no, light, moderate, and heavy shear reinforcement; and dead-to-total load ratios ranging from 0 to 1.

It was found that the ACI 318-19 design provisions represent a major improvement in the accuracy and reliability of one-way shear design across the entire size range of reinforced concrete slabs and beams. An increase in the one-way shear strength reduction factor to 0.80 is justifiable for beams with shear reinforcement, and for small- to medium-size members without shear reinforcement. However, this factor should not be increased beyond the current value of 0.75 for large members without shear reinforcement. The reliability indices for these members remain less than the desirable target value. More one-way shear tests of large beams and slabs are needed to improve the statistical variability or the design expressions.

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Lawrence Novak	International Code Council, Project Advisory Group
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ABSTRACT

A collaborative effort of several ACI technical committees led to the adoption of new ACI 318-19 one-way shear strength design provisions that addressed the influence of size effect and longitudinal reinforcement. These improved provisions provided an opportunity for a rational re-evaluation of the strength reduction ("phi") factor for shear. This report describes a study with the objectives of providing a statistical basis for evaluating and improving the strength reduction factor for one-way shear and recommending an updated strength reduction factor if justified.

The reliability analysis procedure employed Monte Carlo simulation. The in-place concrete strength uncertainty was updated to reflect relevant test data. Considerations for uncertainty in effective depth were specifically revised for one-way shear strength. A CRSI database was analyzed to update the statistical parameters of bars commonly used as shear reinforcement. Data-driven professional factors, based on analyses of the ACI 445 databases, were used instead of expert opinion estimates as in the past. Simulated scenarios included: small, medium, and large size members; light, moderate, and heavy flexural reinforcement; no, light, moderate, and heavy shear reinforcement; and dead-to-total load ratios ranging from 0 to 1.

It was found that the ACI 318-19 design provisions represent a major improvement in the accuracy and reliability of one-way shear design across the entire size range of reinforced concrete slabs and beams. An increase in the one-way shear strength reduction factor to 0.80 is justifiable for beams with shear reinforcement, and for small- to medium-size members without shear reinforcement. However, this factor should not be increased beyond the current value of 0.75 for large members without shear reinforcement. The reliability indices for these members remain less than the desirable target value. More one-way shear tests of large beams and slabs are needed to improve the statistical variability or the design expressions.

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INTRODUCTION

1.1 PROJECT BACKGROUND

Reliability analyses provided a rational basis for increases in the ACI 318 strength reduction ("phi") factor for a moment and axial force in 2002 (tension-controlled) and 2008 (compression-controlled with spirals), which in turn resulted in the more efficient design of concrete structures. However, a corresponding improvement in the strength reduction factor for shear was not justifiable during this period because of well-founded concerns about the level of safety associated with the ACI 318 one-way shear strength expressions—particularly for large and lightly reinforced beams and slabs—which were first introduced in 1963. After several ACI technical committees (ACI 318-E, ACI-ASCE 445, and ACI 446) sustained collaborative efforts to address these safety concerns, improved one-way shear strength expressions were adopted in ACI 318-19 (ACI Committee 318 2019). It has been shown that these new expressions more reliably describe collected experimental test data on the one-way shear strength of RC members. This improved reliability now presents an opportunity for a rational re-evaluation of the strength reduction factor for shear.

1.2 RESEARCH OBJECTIVES

The objectives of the proposed project are

1) To provide an objective, statistical basis for improving the strength reduction factor for one-way shear, and
2) To propose a new strength reduction factor for one-way shear, if justified.

1.3 RESEARCH SIGNIFICANCE

A reliability-justified increase in the resistance factor for shear and torsion would result in more economically competitive and readily constructible concrete structural systems—including decreased transverse reinforcement requirements (amount and congestion). This study is a significant step in this effort as it aims to identify the inherent level of safety in the new one-way shear provisions and start down the path of providing an appropriate level of safety for all shear-based failure modes.

1.4 RESEARCH SCOPE

The objectives were achieved by developing a reliability model for one-way shear strength consistent with practices used to develop the current ACI 318 strength reduction factors for moment and axial force.

Statistical parameters of building loads needed for the reliability analysis were taken from NBS Report 577 (Ellingwood et al. 1980) and related sources. The mechanical properties of reinforcement and concrete and as-built dimensions were treated as random variables in the resistance models, incorporating the ACI 318-19 design provisions for shear strength applied to a practical spectrum of reinforced concrete beams and slabs. Mechanical property statistics were determined using recent material test data provided by the Portland Cement Association (PCA) and the Concrete Reinforcing Steel Institute (CRSI). Normalweight and lightweight concrete data were considered. Tolerances and geometric variability were incorporated.

In previous calibrations for one-way shear (Ellingwood et al. 1980; Nowak et al. 2005), the statistical parameters of the professional (model) factor were estimated rather than based on a large body of experimental data. In the research study reported herein, the extensive ACI-ASCE Committee 445 shear test databases supported the reliability analyses. This collection of shear test results was not available when the current strength reduction factor for shear was determined.

These seven research tasks were proposed:

Task 1—Collection and Review of Relevant Test Data

The research team will use extensive shear test databanks collected by ACI Committee 445 Shear and Torsion to assemble test databases that are relevant and representative of modern concrete structures.

Task 2—Selection of Appropriate Statistical Models for Loads

Statistical parameters of building loads needed for the reliability analysis will be selected from NBS Report 577 (Ellingwood et al. 1980) and related sources.

Task 3—Determination of Mechanical Properties Characteristics and Representative Geometric Variability

The mechanical properties of reinforcement and concrete and as-built dimensions will be treated as random variables in the resistance models, incorporating the ACI 318-19 design provisions for shear strength applied to a practical spectrum of reinforced concrete beams and slabs. Mechanical property statistics will be determined using the relevant material test data provided by PCA and CRSI. Normalweight and lightweight concrete data will be considered. Tolerances and geometric variability will be incorporated as in previous code calibration efforts.

Task 4—Selection of Reliability Analysis Procedure

An appropriate Monte Carlo-based reliability analysis procedure will be selected and applied.

Task 5—Execution of Reliability Analysis Procedure

The selected reliability analysis procedure will be executed using the new ACI 318 shear strength provisions as prediction models. The reliability analysis procedure will be repeated for different ranges of key parameters to assess the sensitivity of the reliability to changes in these parameters.

Task 6—Determination of Reliability Index Corresponding to Various Strength Reduction Factors for One-Way Shear

The target reliability index for shear will be determined from the reliability index associated with designs in compliance with ACI 318-14 (ACI Committee 318 2014) specification. The ideal value is $\beta_T = 4.0$, consistent with the long-standing code philosophy that the probability of nonductile shear failure shall be much less than for a flexural failure (reliability index of 3.5). The target reliability index will be used to determine the appropriate strength reduction factor for one-way shear behavior in reinforced concrete.

Task 7—Preparation of Final Report and Journal Article Manuscript

The investigators will work with the ACI 318-E task group to prepare the resulting code change proposals and an *ACI Structural Journal* article to explain the code changes.

1.5 REPORT ORGANIZATION

Chapter 2 of this report consists of an overview of the basic one-way shear design provisions of ACI 318-14 and ACI 318-19. Chapter 3 is a brief discussion of the philosophy of safe design of structures. The collection and review of relevant test data (Task 1) is described in Chapter 4. Chapter 5 addresses statistical models for dead and live loads (Task 2). The determination of how mechanical and dimensional variability, including the professional factor, were included in the reliability analysis (Task 3) is summarized in Chapter 6. The overall reliability analysis procedure and simulation steps (Tasks 4 and 5) are described in Chapter 7. Chapter 8 contains the results of the reliability analysis, organized according to each simulation step. Chapter 9 includes a discussion of the relationship between reliability index and various strength reduction factors for the ACI 318-19 one-way shear design provisions (Task 6). Chapter 10 contains a summary of the research and the conclusions drawn from the results.

1.6 PROJECT ADVISORY GROUP

This research study was endorsed by Subcommittee E, *Section and Member Strength*, of ACI Committee 318, *Structural Concrete Building Code*. ACI 318-E Chair David H. Sanders appointed three subcommittee members to form the advisory group for the project:

- John F. Bonacci, Karins Engineering Group, Sarasota, Florida,
- Daniel A. Kuchma, Tufts University, Medford, Massachusetts, and
- Lawrence C. Novak, International Code Council, Country Club Hills, Illinois.

The researchers appreciate the insight and experience of the advisory group; this study was greatly improved by the advice provided by this group and Professor Sanders.

Chapter 2
ONE-WAY SHEAR STRENGTH IN ACI 318

A brief overview of the ACI 318-14 (ACI Committee 318 2014) methods and the new ACI 318-19 (ACI Committee 318 2019) method is provided in this chapter. Significant differences are highlighted.

2.1 ACI 318-14 Simplified Method

In accordance with ACI 318-14 code, the calculation of the nominal one-way shear capacity *(V_n)* of a nonprestressed member is based on the sum of the contribution of the concrete *(V_c)* and the shear reinforcement *(V_s)* as follows:

$$V_n = V_c + V_s \le V_c + 8\sqrt{f_c^{'}}b_w d \tag{1}$$

The concrete contribution in reinforced concrete is calculated with Eq. (2),

$$V_c = 2\lambda\sqrt{f_c^{'}}b_w d \tag{2}$$

and the reinforcement contribution is calculated as follows:

$$V_s = \frac{A_v f_{yt} d}{s} \tag{3}$$

2.2 ACI 318-14 Detailed Method

In accordance with ACI 318-14, a more complex method for determining V_c is also permitted. In this method, denoted the "ACI 318-14 Detailed Method" in this report, V_c is computed using Eq. (4), and V_s remains the same as Eq. (3).

$$V_c = Lesser\,of \begin{cases} \left[1.9\lambda\sqrt{f_c^{'}} + 2500\rho_w \dfrac{V_u d}{M_u}\right]b_w d \\ \left[1.9\lambda\sqrt{f_c^{'}} + 2500\rho_w\right]b_w d \\ 3.5\lambda\sqrt{f_c^{'}}b_w d \end{cases} \tag{4}$$

2.3 ACI 318-19 METHOD

The ACI 318-19 one-way shear design method follows the dual-contribution model indicated in Eq. (1). However, in accordance with ACI 318-19, if minimum shear reinforcement is not provided, the concrete contribution is computed as follows,

$$V_c = \left[8\lambda_s \lambda \rho_w^{\frac{1}{3}} \sqrt{f_c^{'}} + \frac{N_u}{6A_g} \right] b_w d \tag{5}$$

where λ_s is the size effect factor defined in Eq.(6).

$$\lambda_s = \frac{1.4}{\sqrt{1 + \frac{d}{10}}} \leq 1.0 \tag{6}$$

If a specified minimum amount of web reinforcement is provided, the concrete contribution is calculated using Eq. (7). For cases with no axial force, the first line of Eq. (7) is the same as Eq. (2). The second line of Eq. (7) is the same as Eq. (5) without the size effect factor.

$$V_c = Either\ of \begin{cases} \left[2\lambda \sqrt{f_c^{'}} + \frac{N_u}{6A_g} \right] b_w d \\ \left[8\lambda \rho_w^{\frac{1}{3}} \sqrt{f_c^{'}} + \frac{N_u}{6A_g} \right] b_w d \end{cases} \tag{7}$$

The shear reinforcement contribution *(V_s)* is computed according to Eq. (3), just as it was in ACI 318-14.

2.4 COMPARISON BETWEEN ACI 318-14 AND ACI 318-19

There are three significant changes between ACI 318-14 and ACI 318-19 one-way shear equations: (1) the moment-to-shear ratio *(M/Vd)* dependency was removed; (2) a size effect factor was incorporated for members without shear reinforcement; and (3) the shear strength depends on the cube root of the longitudinal steel to cross-section area ratio *(ρ_w)*, instead of the previous linear relationship.

2.5 SHEAR REINFORCEMENT THRESHOLD AND EXCEPTIONS

The minimum shear reinforcement requirements for beams in ACI 318-14 can be found in Article 9.6.3. A minimum area of shear reinforcement shall be provided in all regions where V_u exceeds $0.5\phi V_c$, except for the cases stipulated in Table 9.6.3.1 of the code. For these specific cases, a minimum area of shear reinforcement is only required where V_u exceeds ϕV_c. The special cases stipulated in Table 9.6.3.1 are shallow-depth beams (h ≤ 10 in.), shallow beams integral with slabs (h ≤ 24 in.), beams constructed with steel fiber-reinforced concrete, and one-way joist systems as defined in the code. If the simplified equation for one-way shear strength is taken as reference, the threshold for minimum shear reinforcement for the general case can be rewritten as $V_u > \phi\lambda\sqrt{f_c'}b_w d$; while for the exception cases, the limit can be rewritten as $V_u > \phi\, 2\lambda\sqrt{f_c'}b_w d$. The ACI 318-14 minimum shear reinforcement requirements for one-way slabs are in Article 7.6.3. Solid, nonprestressed slabs and footings are treated the same as the exception cases for beams, where a minimum area of shear reinforcement is only required where V_u exceeds ϕV_c.

Similarly, the minimum shear reinforcement requirements for beams in ACI 318-19 can also be found in the same location, Article 9.6.3, and the exceptions are also detailed in Table 9.6.3.1. The specification establishes that a minimum area of shear reinforcement shall be provided in all regions where V_u exceeds $\phi\lambda\sqrt{f_c'}b_w d$, except for the cases in Table 9.6.3.1. For the exception cases, a minimum area of shear reinforcement is required only where V_u exceeds ϕV_c, which for ACI 318-19 can be rewritten as $V_u > \phi 8\lambda_s\lambda\rho_w^{\frac{1}{3}}\sqrt{f_c'}b_w d$. As in ACI 318-14, ACI 318-19 Article 7.6.3 treats solid, nonprestressed slabs and footings the same as the exception cases for beams, where a minimum area of shear reinforcement is only required where V_u exceeds ϕV_c, which was changed to $\phi 8\lambda_s\lambda\rho_w^{\frac{1}{3}}\sqrt{f_c'}b_w d$ in ACI 318-19.

Chapter 3

SAFETY MANAGEMENT AND RELIABILITY OF STRUCTURES

Traditionally, such as in allowable stress design (ASD), uncertainties in the calculations of loads and resistances were accounted for through a single global factor of safety. Later, a partial factor of safety methodology was developed. This design approach includes load factors that amplify each load component (dead, live, wind, snow, earthquake, etc.), accounting for uncertainties associated with them, and the resistance factor (or strength reduction factor) that accounts for uncertainties related to the load-carrying capacity. This methodology is called load and resistance factor design (LRFD). The latest editions of principal design and construction codes follow the LRFD philosophy. The objective of the LRFD philosophy is to design more efficiently and manage risk in design and construction in a rational manner.

In addition to the evident variability in loads that affect structures, there are many sources of uncertainty inherent to structural design. For example, concrete compressive strength, yield strength, and material unit weight rarely have the same observed values under the same test conditions. Thus, design parameters are random variables. The concept of a random variable is closely related to conducting an experiment. If an experiment is performed repeatedly (with all conditions maintained as precisely as possible) and the measured results are identical, then the measured variables are said to be deterministic. However, if the numerical results vary, the item is random (Hart 1982). Since random variables are inevitable, absolute safety (or zero probability of failure) cannot be achieved. Consequently, structures must be designed to serve their functions with a finite probability of failure (Nowak and Collins 2013).

How engineers treat the uncertainty of a given phenomenon depends upon the situation. If the degree of variability is small, and the consequences of any variability will not be significant, the uncertainty can be ignored by merely assuming that the variable will be equal to the best available estimate. This is typically done with the material elastic modulus or with the physical dimensions of components constructed with a high level of quality control. If the uncertainty is significant, it is necessary to use a conservative estimate of the variable. This has been done by setting specified minimum strength properties of materials and members. Therefore, some questions arise: how can engineers maintain consistency in their conservatism from one situation to another? For instance, separate professional committees set the specified minimum concrete compressive strength and the specified minimum bending strength of wood (Benjamin and Cornell 1970). How can engineers maintain consistency in their conservatism within the same construction project? For example, in a building, the specified minimum flexural and shear strength of beams, slabs, corbels, walls, and foundations should be consistent. Reliability of structures is the probability-based framework that has been used in the development of modern design codes to answer those questions. This study addresses a reliability-based calibration for the one-way shear strength limit state.

Chapter 4

COLLECTION AND REVIEW OF RELEVANT TEST DATA

4.1 INTRODUCTION

There have been significant efforts to collect data for reliability-based calibration in the past. NBS Report 577 (Ellingwood et al. 1980) contains load effects and reinforced concrete building construction data, including statistical characterization of the material, fabrication, and analysis uncertainties. Nowak et al. (2005) carried out a reliability-based calibration of the ACI 318 building code requirements for structural concrete, in which material data was updated with the contribution of several professional associations, e.g., the Portland Cement Association (PCA), Precast/Prestressed Concrete Institute (PCI), and Concrete Reinforcing Steel Institute (CRSI). Nowak and Rakoczy (2011) reported uncertainties in the compressive strength of lightweight concrete and its effect on flexural strength and shear strength. Greene and Graybeal (2013) collected significant data to study several mechanical properties of lightweight concrete for an update of the AASHTO LRFD Bridge Design Specifications. The Concrete Reinforcing Steel Institute (CRSI) maintains a database of mechanical properties of steel bars fabricated domestically. This database is available for research initiatives, and its use was approved for this study (CRSI 2019). Furthermore, an extensive shear test databank collected by ACI Committee 445 Shear and Torsion (Reineck et al. 2003, Reineck et al. 2014) was considered in this study to assemble test databases that are relevant and representative of modern concrete structures.

The research team reviewed the most up-to-date data available and reconsidered assumptions and simplifications in past calibrations, e.g., regarding dimensional fabrication accuracy, analysis accuracy, and in-place concrete strength. Also, advances in reliability analysis and computational capabilities allowed the efficient performance of more sophisticated analyses.

4.2 MATERIAL PROPERTY DATA

The data available to characterize the uncertainty in material mechanical properties include results of tests routinely performed as a part of the construction process and as quality control for material producers. Each database available is described in this section.

4.2.1 Concrete Cylinder Strengths—Normalweight Concrete

The available data for normalweight concrete are test results provided by the industry through the National Ready Mix Concrete Association (NRMCA) in a study sponsored by Portland Cement Association (PCA) and co-sponsored by the Precast/Prestressed Concrete Institute (PCI) reported by Nowak et al. (2005). Test results for 28-day compressive strength determined for standard 6 in. x 12 in. cylinders were given for ordinary concrete (nominal strength ranging from 3000 to 6500 psi) for cast-in-place ready-mix concrete (11,098 samples) and plant-cast concrete (1,174 samples). Table 4-1 and Table 4-2 show the nominal f'_c value, the number of samples tested, and the actual mean value of f'_c for ordinary ready-mix concrete and ordinary plant-cast concrete, respectively. In addition, for reliability analysis purposes, the tables contain bias factors (λ) and coefficients of variation (V) of normal distributions fitted to the lower tail of each dataset (Nowak et al. 2005). Throughout this report, the bias factor (λ) represents the ratio of mean value to the nominal value. The data in Table 4-3 for high-strength concrete (nominal strength ranging from 7000 to 12000 psi) were obtained from precast concrete plants (2,052 samples).

Table 4-1: Statistical Parameters for Ordinary Ready-Mix Concrete

f'_c	Number of samples	Mean f'_c	Bias factor, λ	V
3000 psi	4016	4000 psi	1.33	0.145
3500 psi	527	4350 psi	1.24	0.115
4000 psi	2784	4850 psi	1.21	0.155
4500 psi	1919	5350 psi	1.19	0.16
5000 psi	1722	6100 psi	1.22	0.125
6000 psi	130	7300 psi	1.22	0.075

Table 4-2: Statistical Parameters for Ordinary Plant-Cast Concrete

f'_c	Number of samples	Mean f'_c	Bias factor, λ	V
5000 psi	330	6600 psi	1.32	0.105
5500 psi	26	6570 psi	1.195	0.045
6000 psi	493	6950 psi	1.16	0.08
6500 psi	325	7000 psi	1.08	0.07

Table 4-3: Statistical Parameters for High-Strength Concrete

f'_c	Number of samples	Compressive strength @ 28 days		
		Mean f'_c	Bias factor, λ	V
7000 psi	210	8340 psi	1.191	0.116
8000 psi	753	8740 psi	1.093	0.088
9000 psi	73	10,410 psi	1.157	0.100
10,000 psi	635	11,280 psi	1.128	0.115
12,000 psi	381	12,440 psi	1.037	0.109

4.2.2 Concrete Cylinder Strengths—Lightweight Concrete

The available data for lightweight concrete are from tests performed by materials producers and submitted through the Expanded Shale, Clay and Slate Institute (ESCSI), as reported by Nowak and Rakoczy (2011). Test results for 28-day compressive strength determined for standard 6 in. x 12 in. cylinders were collected for lightweight concrete with nominal strength ranging from 3000 to 7000 psi. The database includes 6991 samples. Table 4-4 shows the nominal f'_c value, the number of samples tested, and the actual mean value of f'_c for the lightweight concrete. In addition, the bias factors (λ) and coefficients of variation (V) of normal distributions fitted to the lower tail of each dataset are reported.

Table 4-4: Statistical Parameters for Lightweight Concrete

f'_c	Number of samples	Bias factor, λ	V
3000 psi	609	1.43	0.16
3500 psi	733	1.3	0.12
4000 psi	2212	1.34	0.12
4500 psi	1230	1.33	0.12
5000 psi	422	1.11	0.08
6300 psi	876	1.18	0.11
6800 psi	392	1.19	0.12
7000 psi	517	1.14	0.10

4.2.3 Modulus of Rupture—Normalweight Concrete

Legeron and Paultre (2000) presented a study with 395 data points from a large number of research programs with compressive strengths ranging from 20 to 130 MPa. The data are shown in Figure 4-1 (MPa units).

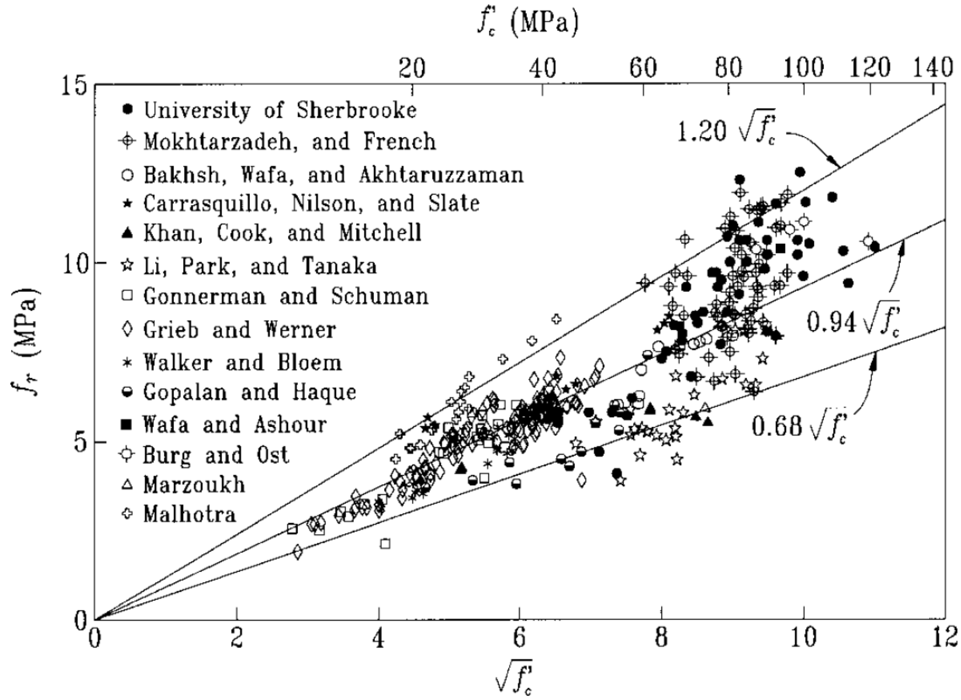


Figure 4-1: Modulus of Rupture as a Function of $\sqrt{f'_c}$ (Legeron and Paultre 2000)

4.2.4 Modulus of Rupture—Lightweight Concrete

Greene and Graybeal (2013) performed a thorough literature review that included tests, analyses, and discussions of lightweight concrete (LWC) to suggest revisions to the AASHTO LRFD Bridge Design Specifications. These documents were reviewed for LWC data consisting of a compressive strength value and data from at least one other mechanical test. The recorded mechanical tests included compressive strength (f'_c), modulus of elasticity (E_c), splitting tensile strength (f_{ct}), modulus of rupture (f_r), and Poisson's ratio. The concrete density was also recorded. The Turner-Fairbank Highway Research Center (TFHRC) database consists of 3,835 data lines. A much smaller set of data from Auburn University research on concrete for bridge decks is also included in this analysis of the modulus of rupture (Byard 2011; Byard et al. 2012; Tankasala 2017; Tankasala and Schindler 2020). Thus, the numbers of the data gathered are as follows: modulus of rupture in sand-lightweight concrete (SLW) 221, modulus of rupture in all-lightweight concrete (ALW) 227.

Figure 4-2 shows the measured modulus of rupture against the square root of the measured compressive strength for the test data available. For SLW, the data ranges from $f_r = 0.102\sqrt{f_c'}$ to $f_r = 0.441\sqrt{f_c'}$ (ksi units), with the best-fit line as $f_r = 0.278\sqrt{f_c'}$ (ksi units). Assuming the best-fit line represents the mean of a distribution and the variation around this value is normally distributed, the maximum and minimum values are within ±3σ. Therefore, the coefficient of variation of f_r as a function of $\sqrt{f_c'}$ can be estimated as V = 0.20. For ALW, the data ranges from $f_r = 0.175\sqrt{f_c'}$ to $f_r = 0.421\sqrt{f_c'}$, with the best-fit line as $f_r = 0.263\sqrt{f_c'}$. Similarly, under the same assumptions as before, the coefficient of variation of f_r as a function of $\sqrt{f_c'}$ can be estimated as V = 0.16.

(a)

(b)

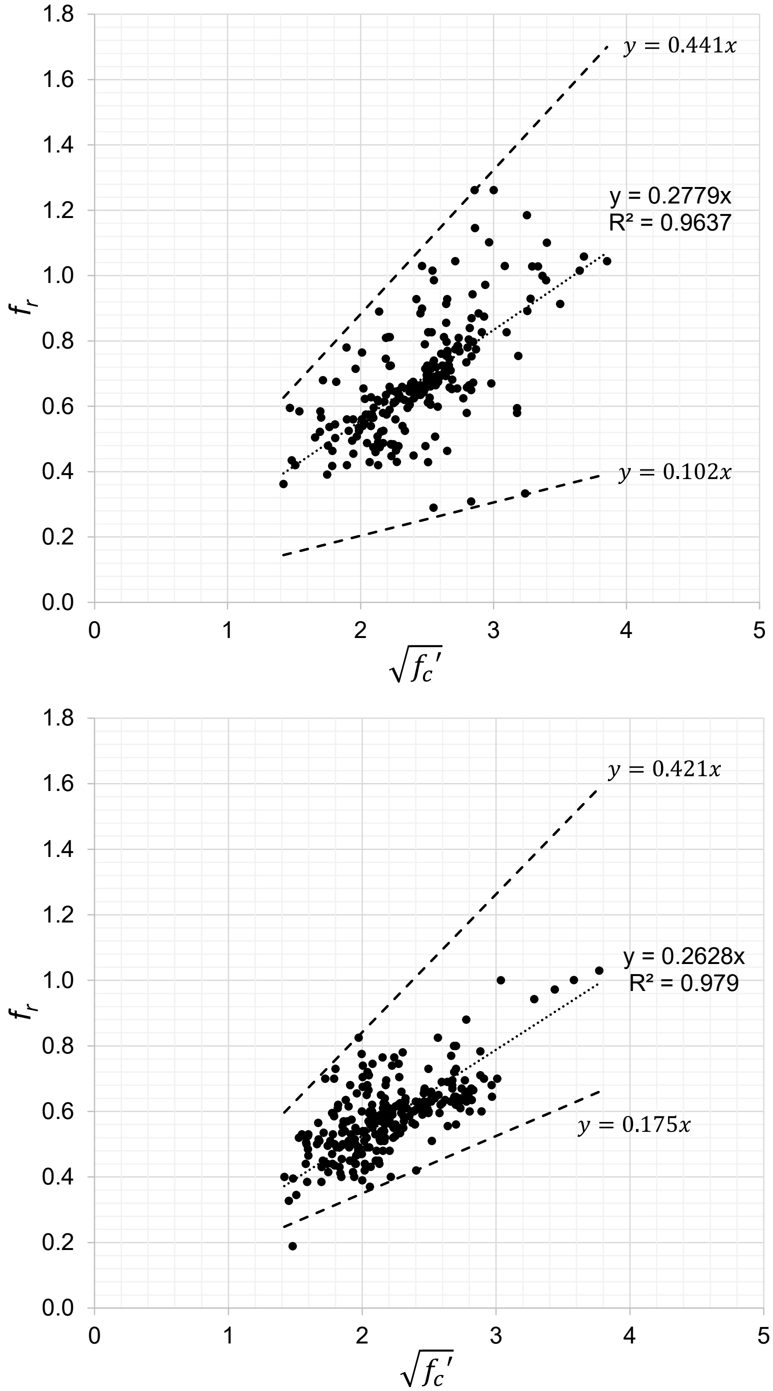


Figure 4-2: Modulus of Rupture as a Function of $\sqrt{f_c'}$ in LWC: (a) SLW, (b) ALW (ksi)

4.2.5 In-Place Concrete Strengths

The strength of concrete in the built structural component is called in-place concrete strength, and it is different from the specified strength for several reasons. Bartlett and MacGregor (1996) studied the relationship between the in-place compressive strength of concrete in structures and the specified strength f'_c using factors F_1 and F_2 defined as shown in Figure 4-3. The factor F_1, i.e., the relationship between specified (nominal) concrete compressive strength and the standard cylinder 28-day compressive strength, is discussed in Sections 4.2.1 and 4.2.2. The factor F_2 is the ratio of in-place strength to cylinder strength. In the previous development of a statistical resistance model for reliability-based calibration (Nowak and Szerszen 2003), a simple approach was used to represent the factor F_2. It was assumed that the strength of concrete in the actual structural component is less than the concrete compressive strength measured using standard cylinder 28-day tests by approximately 10 percent. More recently, F_2 was evaluated by Bartlett and MacGregor (1996) using core and cylinder data representing 108 concrete mixes with strengths less than 8,000 psi from several sources. A statistical description of the factor F_2 was recommended as lognormal with λ = 1.03, V = 0.14. Bartlett and MacGregor (1999) used 500 data points to study the systematic within-member strength variation in laboratory-cast columns and 1062 data points to study the systematic within-member strength variation in laboratory-cast “shallow” elements (blocks, beams, slabs, and walls). Coefficients of variation that represent the overall variation of the in-place concrete strength in a structure vary from 7% for one member cast from one batch of concrete to 13% for a structure consisting of many members cast from many batches of cast-in-place concrete.

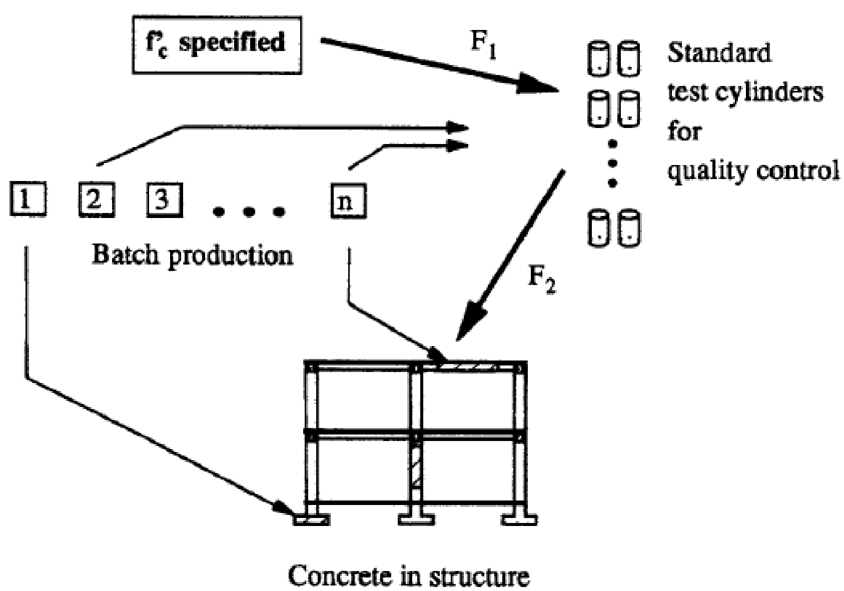


Figure 4-3: Nature of Relationship between Specified and In-Place Strength (Bartlett and MacGregor 1996)

Ergun and Kurklu (2012) performed 84 tests to evaluate the strength of concrete cores varying several parameters. The effect of diameter, length-to-diameter ratio, test age, and coring orientation was assessed on molded cylinder and concrete cube specimens (see Figure 4-4). They suggested a value for in-situ concrete strength of $f'_c/0.86$ by using compressive concrete strength of cores with length-to-diameter ratio of 1.0 drilled perpendicular to the casting direction.

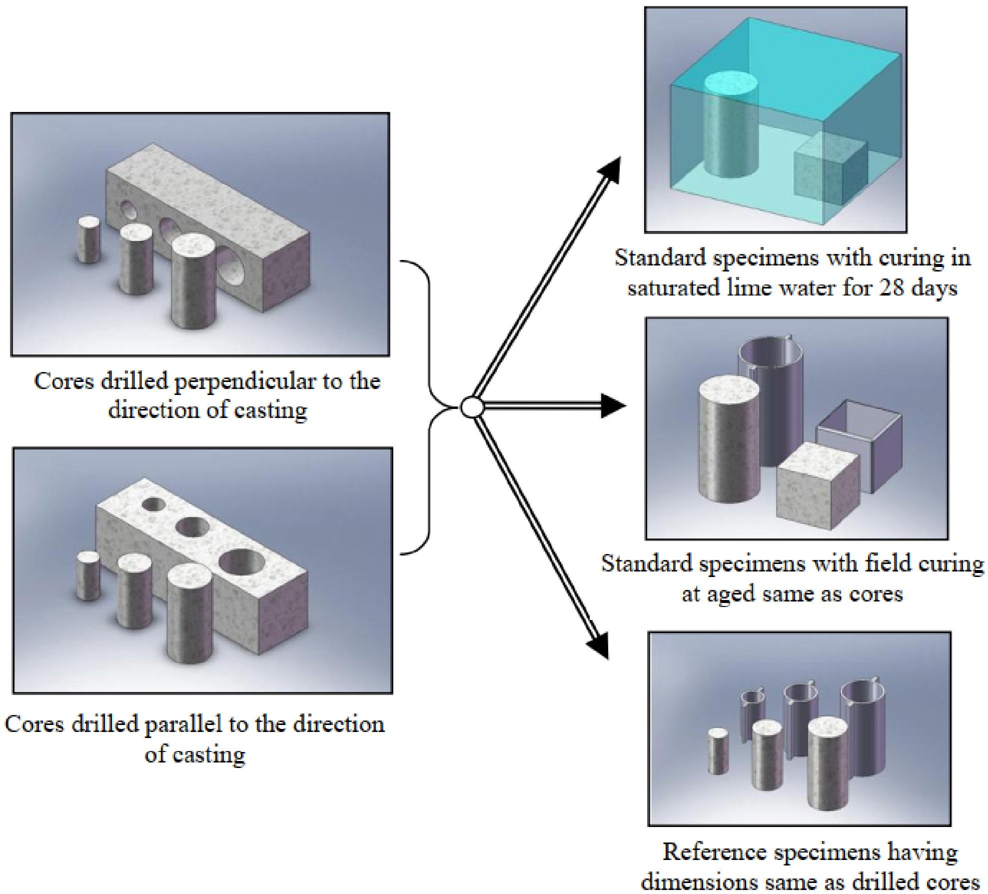


Figure 4-4: Test Specimen Scheme (Ergun and Kurklu 2012)

Grubbs et al. (2016) documented an experimental study on in-place concrete strength in field-cast slabs performed at Auburn University. The relationship between the molded cylinder and cast-in-place cylinder strength was analyzed. The strengths of 28- and 365-day exterior and interior cast-in-place cylinders (Figure 4-5) were matched with the strength of standard molded 28-day cylinders of corresponding batches by material composition.

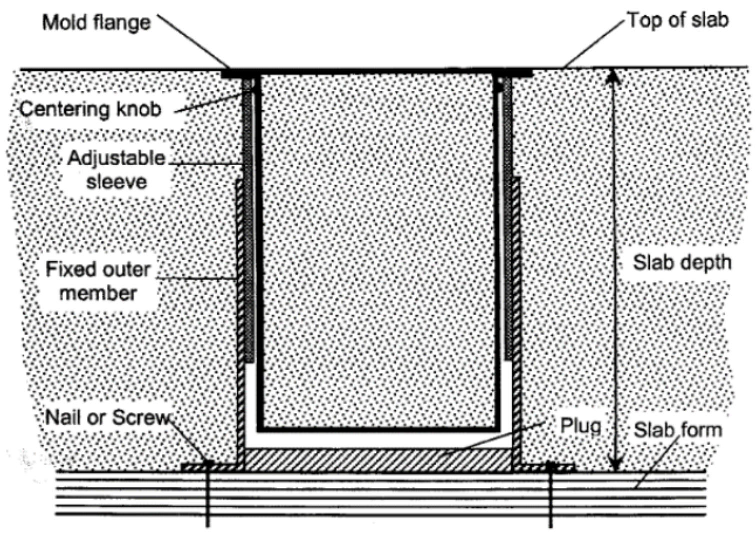


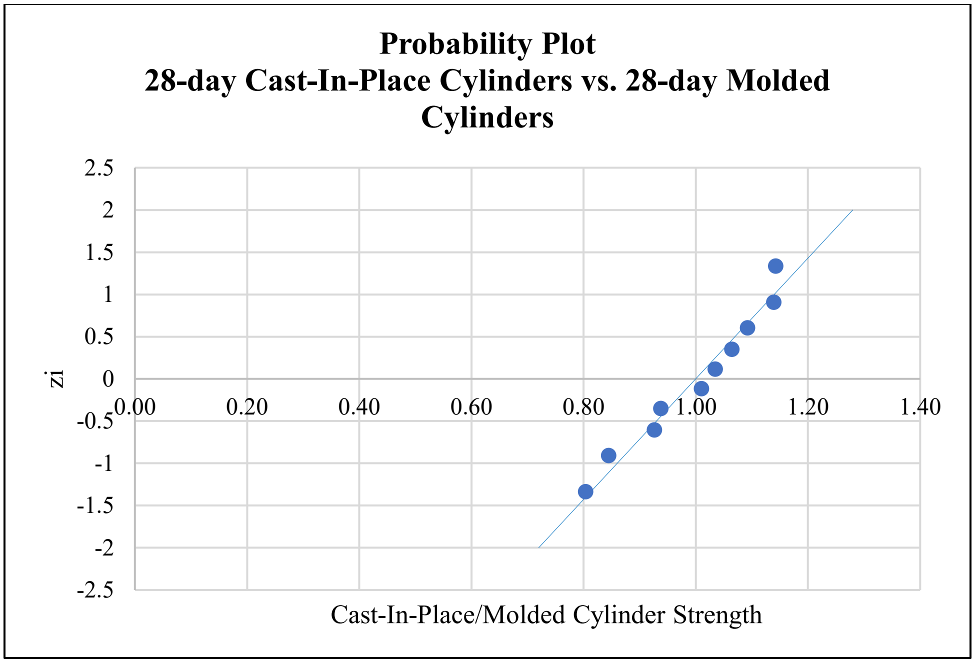
Figure 4-5: Schematic of Cast-In-Place Cylinder Assembly (ASTM C873 2011)

Based on the data reported by Grubbs et al. (2016), statistical parameters for the cast-in-place cylinder strength relative to the standard 28-day molded cylinder strength ratio were determined:

- at 28 days (in-place): μ = 1.00, V = 0.12;
- at 365 days (in-place): μ = 1.21, V = 0.18.

Probability plots for both cases of the present study show that the distribution obtained is approximately normal (Figure 4-6). After fitting the normal distribution to the lower tail of the data, the following adjusted statistical parameters were obtained:

- at 28 days (in-place): μ = 1.00, V = 0.14;
- at 365 days (in-place): μ = 1.21, V = 0.20.



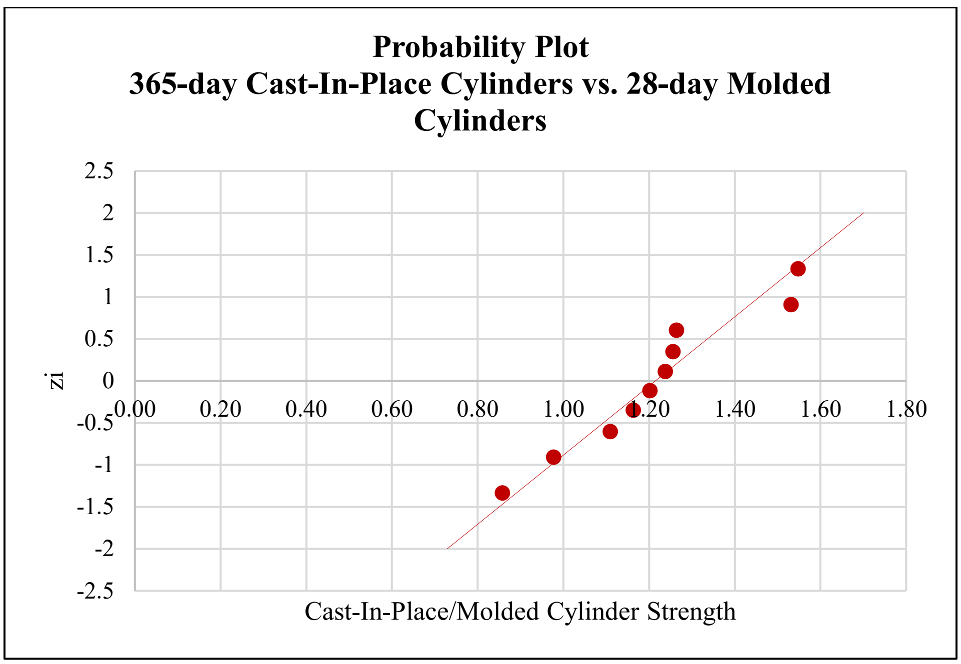


Figure 4-6: Probability Plots for 28- and 365-Day Cast-In-Place Strength to Molded Cylinder Strength Ratio

According to Bartlett and MacGregor (1996), the ratio F_2 between the average in-place strength and the strength of standard cylinders tested follows a lognormal distribution and the statistical parameters are:

- at 28 days (in-place): μ= 1.03, V = 0.14;
- at 365 days (in-place): μ= 1.29, V = 0.14.

Although based on a small set of samples, the statistical parameters obtained by analyzing the data of Grubbs et al. (2016) are close to those found in the comprehensive study performed by Bartlett and MacGregor (1996). Therefore, in the present study, F_2 is considered a lognormal distribution with λ = 1.03 and V = 0.14, which corresponds to the 28-day strength.

In addition, Bartlett and MacGregor (2016) reported a 25 percent increase in the in-place strength between 28 days and 1 year. The more limited data reported by Grubbs et al. (1996) showed a 21 percent increase in the mean value over the same duration. Thus, the conservatism of the concrete compressive strength estimate increases significantly over time.

4.2.6 Reinforcing Steel

Ellingwood et al. (1980) collected steel reinforcing bar data for reliability analysis more than 40 years ago. The bias factor for f_y selected for use in that study was λ = 1.125, and the coefficient of variation was V = 0.10. In 2005, Nowak et al. (2005) reported newer test data for reinforcing bars with the nominal yield strength of 60 ksi, for sizes from No. 3 to No. 14 (15,769 samples), obtained from the Concrete Reinforcing Steel Institute (CRSI) for a project sponsored by PCA and co-sponsored by PCI. Table 4-5 shows the number of samples for each bar size and the mean value of the measured yield strength. In addition, for reliability analysis purposes, Nowak et al. (2005) inferred bias factors and coefficients of variation of normal distributions fitted to the lower tail of each bar size dataset.

Table 4-5: Statistical Parameters for Reinforcing Steel, Grade 60 ksi (Nowak et al. 2005)

Bar size	Number of samples	Mean yield f_y (ksi)	Bias factor, λ	V
No. 3 (0.375 in)	864	71.0	1.18	0.04
No. 4 (0.500 in)	2685	67.5	1.13	0.03
No. 5 (0.625 in)	3722	67.0	1.12	0.02
No. 6 (0.750 in)	1455	67.0	1.12	0.02
No. 7 (0.875 in)	1607	68.5	1.14	0.03
No. 8 (1.000 in)	1446	68.0	1.13	0.025
No. 9 (1.128 in)	1573	68.5	1.14	0.02
No. 10 (1.270 in)	1089	68.0	1.13	0.02
No. 11 (1.410 in)	1316	68.0	1.13	0.02
No. 14 (1.693 in)	12	68.5	1.14	0.02

For the current research study, CRSI approved the use of its updated CRSI Mill Database (2009–2019). This database includes more than 950,000 test results for steel bars fabricated according to ASTM A615 (2022) and ASTM A706 (2022) for grades 40, 60, 75, 80, and 100, and bar sizes from No. 3 to No.

18 from 33 mills across the U.S. These data were used to generate the statistical parameters used in the reliability analysis for this project.

After an exploratory analysis of the available data, numerous (understrength) outliers were found in the Grade 60 bar tests from a single mill, labeled as Mill 304, for multiple bar sizes in one year, 2010. It is possible that these records represented Grade 40 steel but were mislabeled as Grade 60 for the database. These records start occurring at a specific location in the database. Around this location, all the tests marked Grade 60, which were few samples, have results very similar to adjacent, and more numerous, Grade 40 tests on the same day. Conversely, occasional tests marked Grade 40 have test results that seem characteristic of Grade 60 steel. This also occurred on the same testing day.

Mislabeling occurs today in data reported by mills when it is sent to the CSRI Mill Database; in response, there is now a validation procedure when registering the data from mills into the database. However, validation issues from 2010 were not monitored as well as they have been recently, so it seems likely that several test results for Mill 304 were mislabeled and were not caught at the time. Further exploratory analyses revealed that this issue of understrength data was limited to Mill 304 and only for the year 2010.

An overstrength data issue was associated with several mills (Mill 304, Mill 102, Mill 243, and Mill 420) for two years: 2010 and 2011. Mill 102 data represent roughly 6% of ASTM A615 data for No. 3, No. 4, No. 5, Grade 40, and Grade 60 bars for all years combined. In Mill 243 data, the issue was limited to Grade 40 No. 5 bars, and only in 2011, which was the only year this mill reported producing this combination of size and grade. Mill 420 data represent less than 0.5% of the data available.

Given the issues mentioned above and the abundance of data, the simplest and most objective approach is to consider only the data from 2012 to 2019 for determining statistical parameters for further analyses and the reliability-based calibration. Hence, data from 2012 to 2019 for bar sizes No. 3, No. 4, and No. 5, Grade 40 and Grade 60, fabricated according to the standard ASTM A615, were analyzed. These smaller bar sizes were selected because these are the sizes primarily used for shear reinforcement.

Failure of a structural member is a consequence of some combination of overload and understrength; therefore, the probability of failure is determined by the upper tail of the load effect distribution and the lower tail of the resistance (strength) distribution. For this reason, accurately modeling the lower tail of each source of uncertainty in the statistical resistance model is essential. Appropriate distributions were fitted to the lower tail of each year's f_y dataset with good agreement. Table 4-6 summarizes the resulting bias factors and coefficients of variation that represent each year of data. The values are plotted in Figure 4-7 and Figure 4-8.

Table 4-6: Statistical Parameters for Reinforcing Steel, 2012–2019

Bar size	Year	Grade 40			Grade 60		
		Count	λ	V	Count	λ	V
No. 3	2012	510	1.32	0.07	2270	1.18	0.05
	2013	811	1.35	0.08	2696	1.20	0.05
	2014	568	1.38	0.08	2446	1.17	0.04
	2015	499	1.35	0.07	2344	1.17	0.04
	2016	682	1.33	0.07	2445	1.17	0.04
	2017	959	1.34	0.08	3053	1.17	0.04
	2018	1215	1.31	0.08	3809	1.15	0.04
	2019	513	1.35	0.07	3359	1.16	0.04
No. 4	2012	1910	1.25	0.08	14097	1.14	0.04
	2013	2526	1.26	0.06	14320	1.15	0.04
	2014	2179	1.29	0.08	15995	1.15	0.04
	2015	1497	1.25	0.06	15017	1.14	0.04
	2016	2017	1.30	0.08	15755	1.14	0.04
	2017	2383	1.24	0.07	15860	1.14	0.04
	2018	2575	1.21	0.06	19249	1.12	0.04
	2019	1651	1.31	0.08	14884	1.13	0.04
No. 5	2012	670	1.31	0.09	17216	1.13	0.03
	2013	622	1.31	0.09	18295	1.13	0.04
	2014	508	1.34	0.08	19720	1.13	0.03
	2015	362	1.32	0.07	18456	1.13	0.04
	2016	569	1.34	0.08	20736	1.14	0.04
	2017	556	1.35	0.10	19702	1.13	0.04
	2018	792	1.34	0.11	22326	1.12	0.03
	2019	523	1.33	0.09	17922	1.13	0.03

(a)

(b)

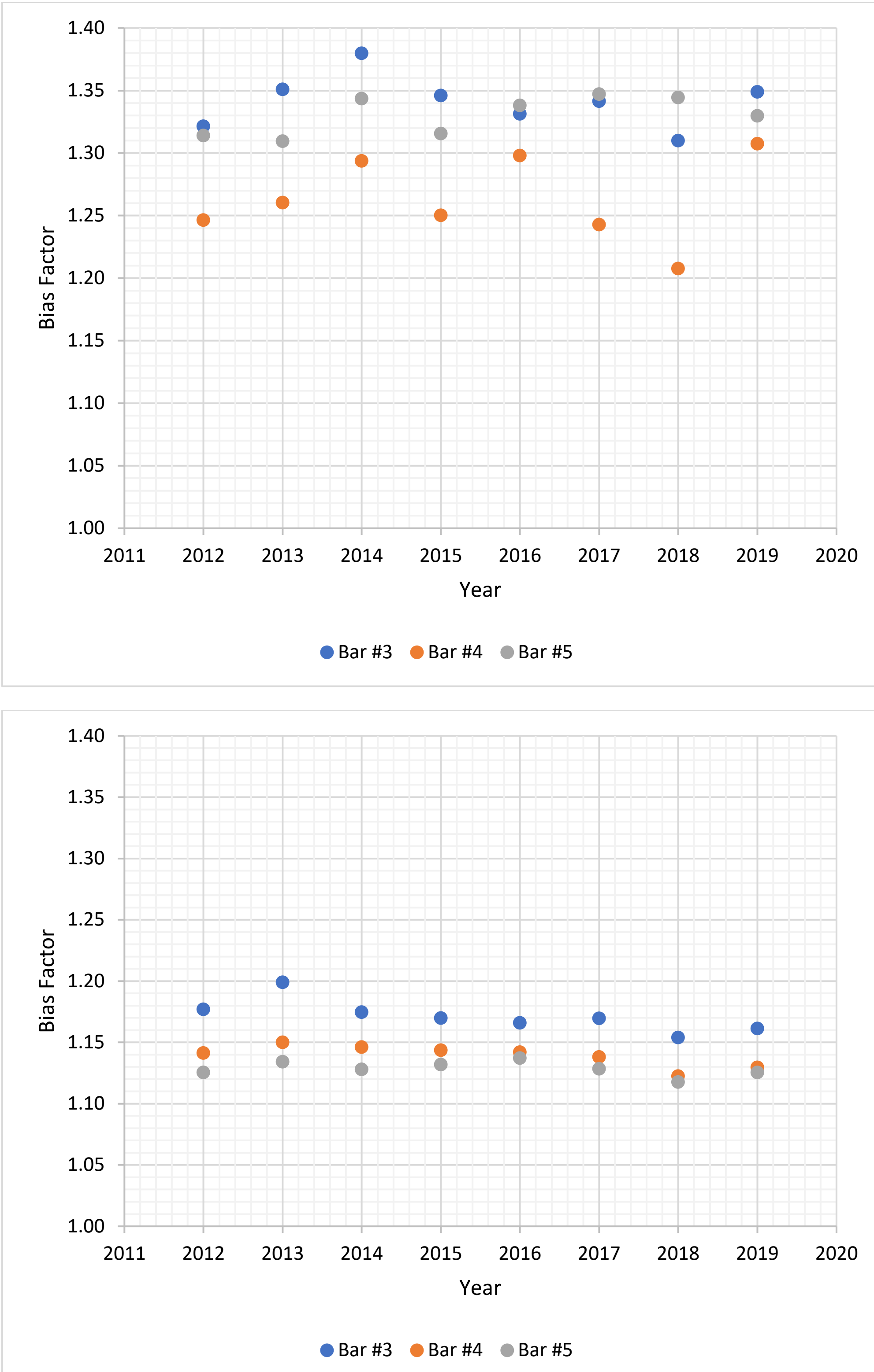


Figure 4-7: Bias Factor of f_y for Bar Sizes No. 3–No. 5, 2012–2019. (a) Grade 40 and (b) Grade 60.

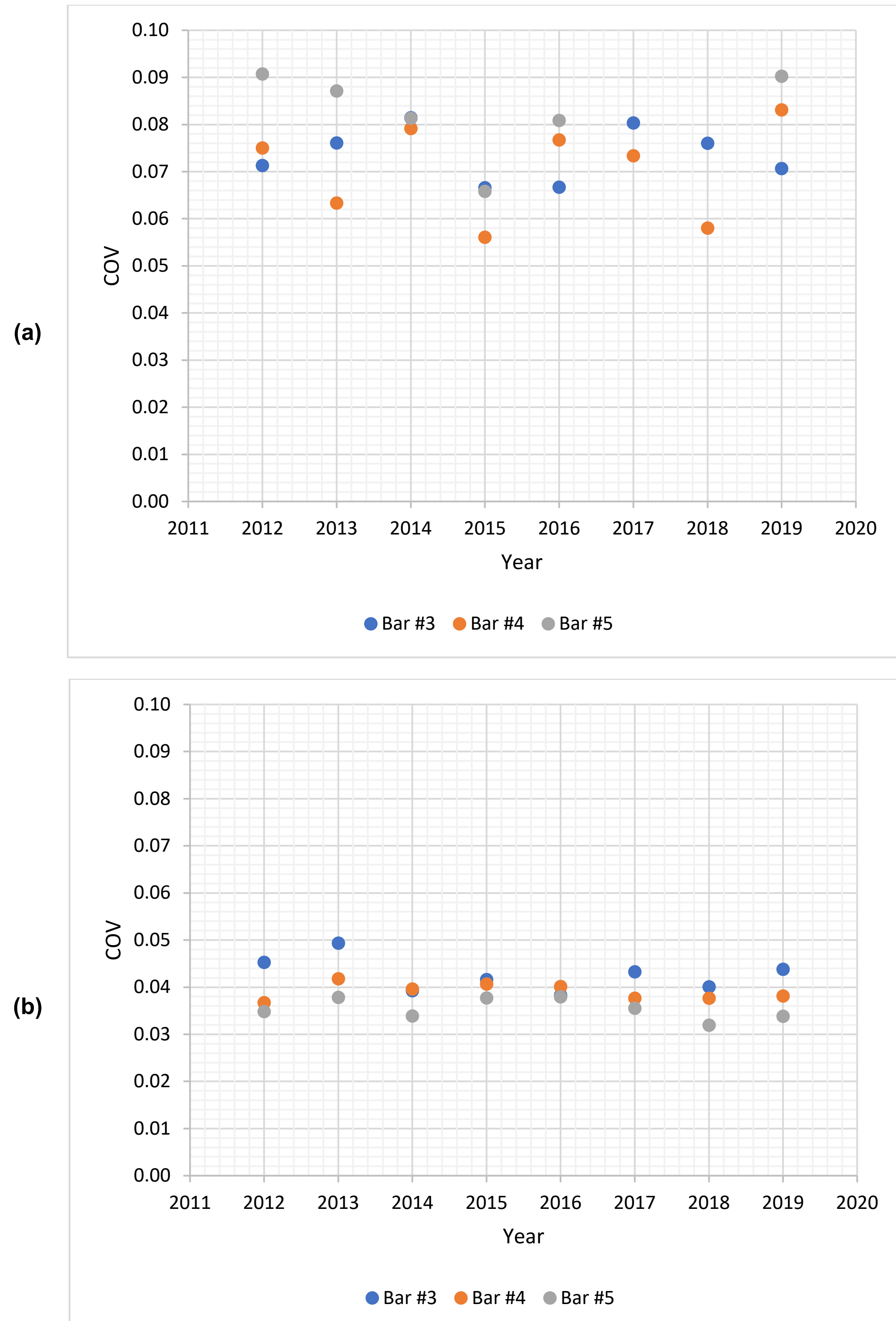


Figure 4-8: Coefficient of Variation of f_y for Bar Sizes No. 3–No. 5, 2012–2019 (a) Grade 40 and (b) Grade 60.

The bias factor ranges from λ = 1.21–1.38 for Grade 40 and from λ = 1.12–1.20 for Grade 60; and the coefficient of variation ranges from V = 0.06–0.11 for Grade 40, and from V = 0.03–0.05 for Grade 60. For each year of Grade 40 steel, the data count ranges from 1,497 to 2,575, while for each year of Grade 60 steel, the data count is much greater: 14,097 to 19,249. No significant yearly trend can be observed in Figure 4-7 or Figure 4-8; hence, the recommended λ and V are the weighted averages of each parameter over time. The resulting statistical parameters for each bar size considered in this analysis are reported in Table 4-7.

Table 4-7: Statistical Parameters for Reinforcing Steel, Weighted Averages, 2012–2019.

Bar size	Grade 40		Grade 60	
	λ	V	λ	V
No. 3	1.34	0.07	1.17	0.04
No. 4	**1.26**	**0.07**	**1.14**	**0.04**
No. 5	1.33	0.09	1.13	0.04

As shown in Table 4-7, for Grade 40 steel, the bias factors for No. 3 and No. 5 bars are greater than those for No. 4 bars. Therefore, the bias factor λ = 1.26 for modeling shear reinforcement was selected because it is conservative and represents the most prevalent stirrup size. The coefficients of variation, V, associated with No. 3 and No. 5 bars (0.07 and 0.09, respectively) are equal to or slightly larger than for No. 4; however, these are based on fewer data than for No. 4 bars. Also, No. 4 bars are more commonly used as shear reinforcement than No. 3 and No. 5. Therefore, it was decided to recommend V = 0.07 for the statistical model of Grade 40 shear reinforcement.

For Grade 60 steel, the coefficient of variation is 0.04 for all three bar sizes, but the bias factor varies. The bias factors associated with No. 5 and 4 bars are almost equal, 1.13 and 1.14, respectively. The bias factor of the No. 3 bars (1.17) is slightly greater. Given the prevalence of No. 4 bars as stirrups, λ = 1.14 was selected as a representative value.

Therefore, the selected statistical model for ASTM A615 Grade 40 shear reinforcement is a normal distribution with λ = 1.26 and V = 0.07. The selected statistical model for ASTM A615 Grade 60 shear reinforcement is a normal distribution with λ = 1.14 and V = 0.04. The selected values are indicated in bold in Table 4-7.

4.3 Dimensional Uncertainty Data

Reliability analysis needs to incorporate the uncertainty of the geometric properties of the structural elements. These differences between the nominal and as-built dimensions can be characterized by the mean and standard deviation of the error. Ellingwood et al. (1980) found the standard deviations of the dimensions are roughly independent of the size of the members and that coefficients of variation decrease as the size increases. However, the overall (mean) variability in dimensions is size-dependent. Table 4-8 summarizes statistical parameters of dimensional uncertainty for several structural members reported by Ellingwood et al. (1980). This data contains Swedish studies reported in 1953 and 1968, presenting mean errors and standard deviations. Even though the 1968 dataset is small, it is important because the building was designed to satisfy the design and construction requirements of the ACI 1953 Code.

Table 4-8: Fabrication Factors according to Ellingwood et al. (1980)

Fabrication Factors		
$h_{As\text{-}built} - h_{Nominal}$	μ	σ
1969 Swedish Slabs	+0.03 in	0.47 in
99 Slabs	+0.21 in	0.26 in
108 Beams	-0.12 in	0.25 in
24 Beams	+0.81 in	0.55 in
$d_{As\text{-}built} - d_{Nominal}$ in one-way Slab; Top Bars		
1969 Swedish Slabs	-0.75 in	0.63 in
99 Slabs	-0.04 in	0.37 in
Recommended in NBS 577	-0.40 in	0.50 in
$d_{As\text{-}built} - d_{Nominal}$ in one-way Slab; Bottom Bars		
2805 Swedish Slabs	-0.13 in	0.34 in
96 Slabs	-0.16 in	0.35 in
Recommended in NBS 577	-0.13 in	0.35 in
$d_{As\text{-}built} - d_{Nominal}$ in Beam, Top Bars	-0.22 in	0.53 in
$b_{w,As\text{-}built} - b_{w,Nominal}$	+0.10 in	0.15 in
Clear cover for bottom Steel in Beams	+0.06 in	0.45 in
	-0.35 in	0.28 in

For comparison purposes, Allen (1970) assumed an effective depth value equal to the nominal value and a coefficient of variation of $0.025+0.20/d$ in his probabilistic study of reinforced concrete bending. Ellingwood (1978) suggested that the coefficient of variation for member dimensions is $0.4/h_n$ and $0.68/h_n$ for the effective depth of reinforcement in flexural members.

Nowak and Collins (2013), based on this previous research, suggested the fabrication factors presented in Table 4-9.

Table 4-9: Fabrication Factors suggested by Nowak and Collins (2013)

Fabrication Factor	λ_F	V_F
b_w	1.01	0.04
d beams	0.99	0.04
d slabs	0.92	0.12
s stirrups	1.00	0.04
A_s	1.00	0.015
A_v	1.00	0.015

The degree of uncertainty of the cross-sectional properties of the structural elements is significantly related to tolerances during concrete production and construction. The two main categories of tolerances that are of interest in terms of fabrication factors are tolerances of reinforcement placement and cross-sectional dimensions.

Table 4-10 and Table 4-11 summarize the ACI 318-19 26.6.2.1 tolerances on the reinforcement location within the cross section and longitudinal location of bends and ends of reinforcement.

Table 4-10: Tolerances on d and Specified Cover (ACI 318-19)

d, in.	Tolerance for d, in.	Tolerance for specified concrete cover, in.
≤ 8	±3/8	Smaller of: –3/8 –(1/3) · specified cover
> 8	±1/2	Smaller of: –1/2 –(1/3) · specified cover

Table 4-11: Tolerances for Longitudinal Location of Bends and Ends of Reinforcement

Location of bends or reinforcement ends	Tolerances, in.
Discontinuous ends of brackets and corbels	±1/2
Discontinuous ends of other members	±1
Other locations	±2

The tolerances also apply for precast concrete for tendons and post-tensioning ducts in accordance with ACI 318-19 26.6.2.1(a).

Cross-sectional dimensional tolerances suggested by ACI for reinforced concrete buildings are described in "Tolerances in Concrete Construction" by the Editorial Staff of *Concrete Construction* (1979). According to this document, the variation in cross-sectional dimensions of columns and beams and the thickness of slabs and walls is –¼ in, +½ in.

The more recent ACI 117-10 "Specification for Tolerances for Concrete Construction and Materials" (ACI Committee 117 2010) provides the following tolerances of deviation from cross-sectional dimensions:

1. The thickness of elements, except slabs, where the specified cross-sectional dimension is
    - 12 in. (300 mm) or less: +3/8 in. (10 mm), –1/4 in. (6 mm);
    - More than 12 in. (300 mm) but not more than 36 in. (900 mm): +1/2 in. (13 mm), – 3/8 in. (10 mm);
    - More than 36 in. (900 mm): +1 in. (25 mm), –3/4 in. (20 mm).
2. The thickness of suspended slabs: –1/4 in. (6 mm).

4.4 ONE-WAY SHEAR ANALYSIS MODEL DATA

The Joint ACI-ASCE Committee 445, in collaboration with the German Committee of reinforced concrete (DAbStb), has compiled a database of slender beams tests (a/d > 2.4) clearly exhibiting a shear failure. The datasets used in this research study include reinforced concrete members without shear reinforcement (784 samples) and reinforced concrete members with shear reinforcement (large dataset: 170 samples and small dataset: 87 samples). The ranges of the most relevant input parameters in the databases are shown in Table 4-12, and their distributions are shown in Figure 4-9. The criteria for collecting the experimental information and a complete description of the data are given elsewhere (Reineck et al. 2003; Reineck et al. 2014). There are two databases for members with shear reinforcement: the large database and the small database. The small database is a subset of the large database where there is a certainty that the shear reinforcement yielded before the shear failure occurred.

Table 4-12: Range of the Main Input Parameters in the Databases

Parameter	Range	
	Without shear reinforcement	With shear reinforcement
f_c' (psi)	1,520−19,810	1,600−17,830
d_{agg} (in.)	0.10−2.01	0.28−1.26
b_w (in.)	2.0−118.3	2.5−18.0
d (in.)	2.2−118.1	6.3−53.9
L (in.)	19.5−1417.3	42.3−860.2
ρ_w	0.0014−0.0664	0.0050−0.1561
a/d	2.4−8.1	2.4−7.1
s_w (in.)	N/A	2.4−11.8
ρ_v	N/A	0.0007−0.0369
V_{test} (kip)	1.6−294.1	18.2−760.8

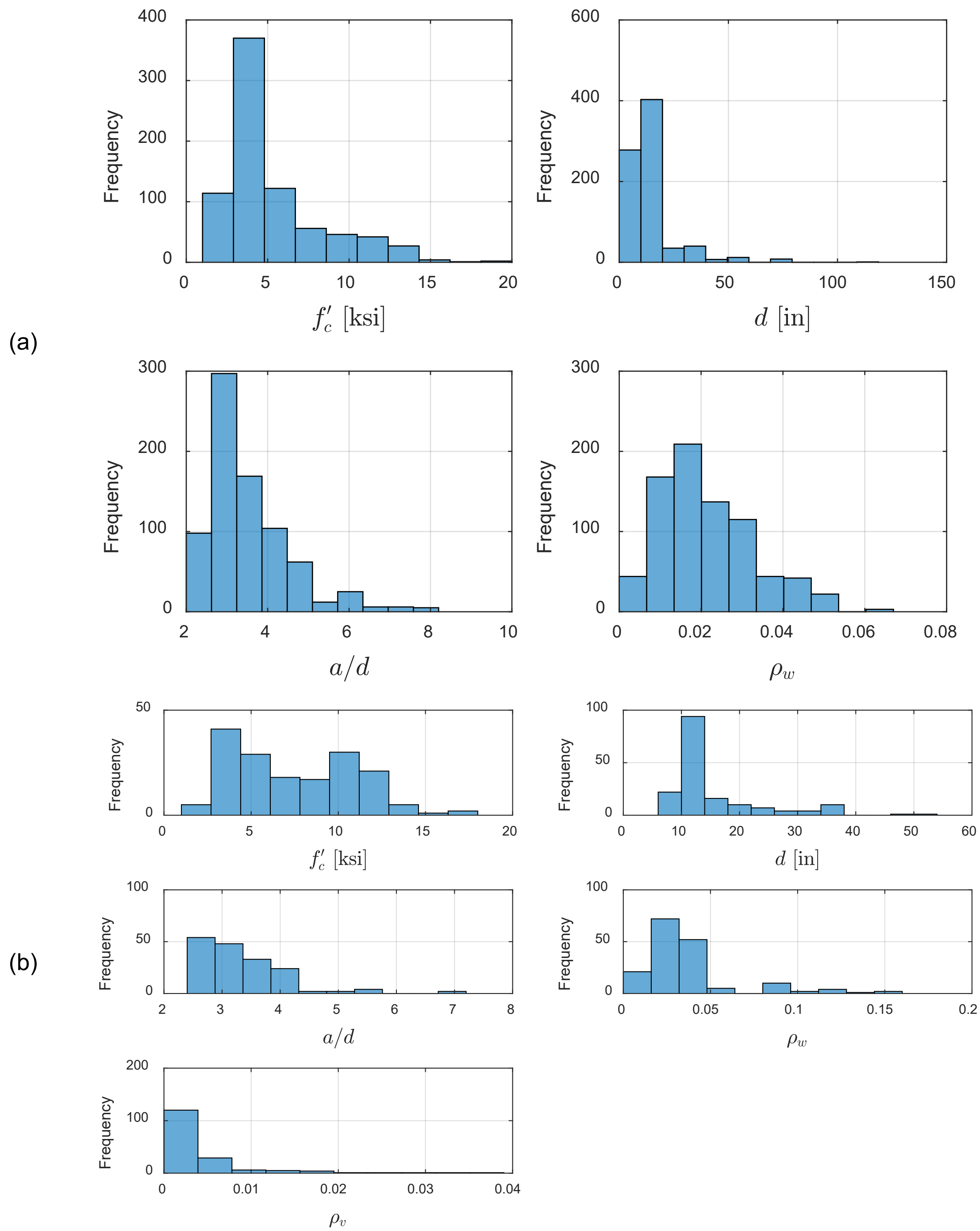


Figure 4-9: Distribution of the Main Input Parameters in the Databases: (a) Members without Shear Reinforcement; and (b) Members with Shear Reinforcement

Chapter 5

SELECTION OF APPROPRIATE STATISTICAL MODELS FOR LOAD

5.1 Gravity Load Components

The fundamental load case for calibration is a combination of dead load and live load. The statistical parameters of the load components used in this study are taken from literature. The statistical parameters of the total load effect ($Q = D+L$) depend on the amount of dead load relative to the total load. As established in previous ACI 318 calibration studies (Szerszen and Nowak 2003), $D_n/(D_n+L_n)$ ranges typically from 0.3 to 0.6 for slabs and from 0.3 to 0.7 for beams.

5.2 Statistical Parameters of Dead Load

For the distribution of dead load effect (D), the bias factor is $\lambda = 1.05$, and the coefficient of variation (V = standard deviation/mean) is $V = 0.10$ for cast-in-place concrete buildings (Nowak 1999). Cast-in-place concrete is used as a reference for calibration because precast concrete members tend to have better statistical resistance parameters than cast-in-place members (i.e., lower λ and V). In addition, the material data available for cast-in-place concrete are significantly more robust than for precast; therefore, the reliability-based calibration is focused on cast-in-place construction.

5.3 Statistical Parameters of Live Load

For the distribution of the live load effect *(L)*, the statistical parameters depend on the occupancy type and the influence area for a given structural member. For the maximum 50-year live load (extreme type I), the bias factor is $\lambda = 1.0$, and V depends on the influence area of the member. Previous researchers have suggested appropriate V values, as summarized in Table 5-1. In the present study, after consideration and discussion with the project advisory group, $\lambda = 1.0$ and $V = 0.18$ were assumed to represent the live load in statistical terms.

Table 5-1: Coefficient of Variation for Maximum Live Load in Office Buildings According to Different Sources (adapted from Ellingwood et al. 1980)

Reference	Influence area (ft^2)			
	200	**1,000**	**5,000**	**10,000**
McGuire and Cornell (1973)	0.14	0.13	0.15	0.15
Ellingwood and Culver (1977)	0.19	0.16	0.16	0.16
Chalk and Corotis (1979)	0.18	0.13	0.10	0.09
Sentler (1975)	0.23	0.18	0.14	0.12

Chapter 6

DETERMINATION OF MECHANICAL AND DIMENSIONAL VARIABILITY

This chapter describes the statistical models selected to represent the sources of uncertainty in the one-way shear strength based on the collected data and reviewed literature.

6.1 MATERIAL FACTORS

In accordance with Nowak et al. (2005) and Nowak and Rakoczy (2011), the recommended bias factors and coefficients of variation for compressive strength of concrete are summarized in Table 6-1. These selected parameters are based on an analysis of the concrete data outlined in Section 4.2.

Table 6-1: Recommended Bias Factors and Coefficients of Variation for Compressive Strength

Normalweight		λ_M	V_M
f'_c(psi)	3000	1.31	0.170
	3500	1.27	0.160
	4000	1.24	0.150
	4500	1.21	0.140
	5000	1.19	0.135
	5500	1.17	0.130
	6000	1.15	0.125
	6500	1.14	0.120
	7000	1.13	0.115
	8000	1.11	0.110
	9000	1.10	0.110
	10000	1.09	0.110
	12000	1.08	0.110
Lightweight		λ_M	V_M
f'_c(psi)	3000	1.38	0.155
	3500	1.33	0.145
	4000	1.29	0.140
	4500	1.25	0.135
	5000	1.22	0.130
	6300	1.20	0.125
	6800	1.18	0.120
	7000	1.16	0.120

Material statistical parameters for reinforcing steel were determined for shear reinforcement based on the CRSI (2019) database as described in Section 4.2.6. The focus of this study was on small bar sizes because this is typically used for shear reinforcement (stirrups). The statistical parameters selected to simulate the yield strength of reinforcing steel in further analyses are listed in Table 6-2.

Table 6-2: Statistical Parameters for Reinforcing Steel Yield Strength

Reinforcement type	Grade 40		Grade 60	
	λ	V	λ	V
Shear (stirrups)	1.26	0.07	1.14	0.04

6.2 Fabrication Factors for Dimensional Uncertainty

The statistical parameters to represent the geometric dimensional uncertainty in the reliability analysis were determined in collaboration with the advisory group based on the historical data summarized in Section 4.3. The values reported in Table 6-3 were implemented in the reliability analysis:

Table 6-3: Fabrication Factors

Fabrication Factor	λ_F	V_F	μ *as-built, nominal*	σ *as-built, nominal*
b_w	1.01	0.04		
d for beams			- 1/4 in.	0.50 in.
d for slabs			- 3/8 in.	0.50 in.
s stirrups	1.00	0.04		
A_s	1.00	0.015		
A_v	1.00	0.015		

In previous calibrations, the variation in d was handled by employing a bias factor and a coefficient of variation. This implies that the absolute uncertainty in the effective depth grows with beam depth, which is inconsistent with the original recommendations in the literature described in Section 4.3. Therefore, the mean and standard deviation of d were implemented in the reliability analysis instead of the bias factor and coefficient of variation. This is important because one-way shear strength is particularly sensitive to d, and one of the motivating factors for the new ACI 318-19 one-way shear expressions was a desire to capture the size effect more accurately.

6.3 PROFESSIONAL FACTORS

The professional factor, P, represents the uncertainty in the analytical model for predicting the strength. The ACI 318 equations for predicting the one-way shear strength of reinforced concrete members are semi-empirical relationships based on fundamental mechanics adjusted to fit available test results. This analytical model includes simplifications from classical assumptions of structural analysis and reinforced concrete behavior; moreover, the curve fitting to select coefficients also included simplifications and professional judgment. Hence, there is uncertainty in the analytical model itself that arises from those approximations.

In previous reliability-based calibrations, the professional factor for flexural strength of reinforced beams was assumed to have a normal distribution with λ = 1.02 and V = 0.06. Rakoczy and Nowak (2013) collected experimental data from 14 studies (a total of 45 test results) and computed the mean and V of the ratio between flexural strengths determined from these tests. The predicted flexural strengths (M_{test}/M_{pred}) are 1.01 and 0.04, respectively. Thus, these selected experimental data validated the assumed professional factor for the flexural strength of reinforced concrete beams.

For one-way shear, the professional factor was previously assumed to have a normal distribution with λ = 1.16 and V = 0.11 for slabs and beams without web reinforcement and λ = 1.075 and V = 0.10 for beams with web reinforcement. However, Kuchma et al. (2019) evaluated the performance of the ACI 318-14 one-way shear method against the available database (Reineck et al. 2003; Reineck et al. 2014), and the mean and V of the ratio V_{test}/V_{pred} were 1.51 and 0.38 for beams without web reinforcement, and 1.47 and 0.24 for beams with web reinforcement. These values of V indicate that the professional factor assumed in the past was too optimistic regarding dispersion. Moreover, several researchers have found that the accuracy of the ACI 318-14 one-way shear model varies significantly with effective depth and longitudinal reinforcement ratio.

To develop a professional factor for one-way shear strength, the shear force at failure measured in laboratory tests was compared to predictions made with the ACI 318 one-way shear models. The most accurate information about material properties and dimensions, loading, and support conditions reported by researchers was used to predict the strength that was used in the comparisons. This process was used to determine the statistical parameters of P.

6.3.1 Laboratory Variability

The ratio V_{test}/V_{pred} is an indication of the accuracy of the analytical model, but it does not render a complete picture. The reported laboratory test strength may vary from the actual strength slightly because of errors involved in the testing and measurement procedure, such as inaccuracies in sensors or recordings and the definition of failure. Likewise, the predicted strength may vary slightly from the actual testing conditions due to differences between the in-situ (concrete or reinforcement) material strength and the samples used to ascertain this strength or variation in the actual specimen dimensions relative to the reported dimensions (Ellingwood et al. 1980; Somo and Hong 2006). Even in a well-controlled environment such as a research laboratory, there is some inherent variability in material, fabrication, and testing procedures. Therefore, the professional factor is estimated based on the distribution of V_{test}/V_{pred}, but only after discounting the degree of variation in the laboratory testing (V_{Lab}) as indicated in Eq. (8).

$$V_p = \sqrt{V_{test}^2 - V_{Lab}^2} \tag{8}$$

Bentz and Collins (2018) quantified and reported variability in shear strength measurement as 0.06. Ellingwood et al. (1980) suggested a COV of 0.04 to represent testing equipment and procedure inaccuracies and a COV of 0.04 to account for the size and material uncertainties in a laboratory environment. Combining these two effects gives an approximate value of V_{Lab} = 0.06, which is consistent with the finding of Bentz and Collins (2018). In order to estimate the variability attributable to laboratory procedures in the database considered for the current study, non-unique test specimens with more than three replicates were identified in the database. The COVs for these five groups of replicate tests are 0.04, 0.06, 0.11, 0.12, and 0.14. Hence, two potential levels of variability in shear strength measurement could be assumed for further analysis, i.e., V_{Lab} = 0.06 and V_{Lab} = 0.10.

6.3.2 Size Effect

The nonuniform level of accuracy related to the size of the members has been accounted for by subdividing the database into three size groups as originally suggested by Aguilar (2020); however, the group limits were revised in accordance with the advisory panel recommendations. As a result, the data were grouped into three size categories by member effective depth: $d \leq 10$ in., 10 in. < $d \leq 33$ in., and d > 33 in. The d = 10 in. boundary was selected because it is the threshold depth beyond which the ACI 318-19 size effect factor influences the predicted strength in members without minimum shear reinforcement. The d = 33 in. limit was selected to approximate the threshold (h = 36 in.) for when ACI 318 provisions require longitudinal skin reinforcement to control side face crack widths for relatively deep beams. The professional factors associated with each size grouping defined above were determined and reported by Aguilar et al. (2023).

6.3.3 Professional Factor Selected for Reliability Analysis

The resulting data-driven professional factors are suggested for ACI 318-14 and ACI 318-19 one-way shear models based on the analysis of the test results in the ACI-ASCE 445/DAbStb databases described in Section 4.4. These proposed professional factors are given in Table 6-4 and Table 6-5, for ACI 318-14 and ACI 318-19, respectively.

Table 6-4: Professional Factors for One-Way Shear Methods in ACI 318-14

<table>
<tr><th>Assumed laboratory uncertainty</th><th>Size range</th><th>Members without shear reinforcement</th><th>Members with shear reinforcement</th></tr>
<tr><td rowspan="3">$V_{Lab} = 0.10$</td><td>$d \leq 10$ in.</td><td>$\lambda_P = 1.71$ and $V_P = 0.18$</td><td>$\lambda_P = 1.71$ and $V_P = 0.17$</td></tr>
<tr><td>10 in. $< d \leq 33$ in.</td><td>$\lambda_P = 1.35$ and $V_P = 0.24$</td><td rowspan="2">$\lambda_P = 1.42$ and $V_P = 0.21$</td></tr>
<tr><td>$d > 33$ in.</td><td>$\lambda_P = 0.58$ and $V_P = 0.21$</td></tr>
<tr><td rowspan="3">$V_{Lab} = 0.06$</td><td>$d \leq 10$ in.</td><td>$\lambda_P = 1.71$ and $V_P = 0.20$</td><td>$\lambda_P = 1.71$ and $V_P = 0.19$</td></tr>
<tr><td>10 in. $< d \leq 33$ in.</td><td>$\lambda_P = 1.35$ and $V_P = 0.25$</td><td rowspan="2">$\lambda_P = 1.42$ and $V_P = 0.22$</td></tr>
<tr><td>$d > 33$ in.</td><td>$\lambda_P = 0.58$ and $V_P = 0.22$</td></tr>
</table>

Table 6-5: Professional Factors for One-Way Shear Methods in ACI 318-19

<table>
<tr><th>Assumed laboratory uncertainty</th><th>Size range</th><th>Members without shear reinforcement</th><th>Members with shear reinforcement</th></tr>
<tr><td rowspan="3">$V_{Lab} = 0.10$</td><td>$d \leq 10$ in.</td><td>$\lambda_P = 1.33$ and $V_P = 0.12$</td><td>$\lambda_P = 1.45$ and $V_P = 0.17$</td></tr>
<tr><td>10 in. $< d \leq 33$ in.</td><td>$\lambda_P = 1.42$ and $V_P = 0.16$</td><td rowspan="2">$\lambda_P = 1.28$ and $V_P = 0.18$</td></tr>
<tr><td>$d > 33$ in.</td><td>$\lambda_P = 1.38$ and $V_P = 0.22$</td></tr>
<tr><td rowspan="3">$V_{Lab} = 0.06$</td><td>$d \leq 10$ in.</td><td>$\lambda_P = 1.33$ and $V_P = 0.15$</td><td>$\lambda_P = 1.45$ and $V_P = 0.19$</td></tr>
<tr><td>10 in. $< d \leq 33$ in.</td><td>$\lambda_P = 1.42$ and $V_P = 0.18$</td><td rowspan="2">$\lambda_P = 1.28$ and $V_P = 0.20$</td></tr>
<tr><td>$d > 33$ in.</td><td>$\lambda_P = 1.38$ and $V_P = 0.23$</td></tr>
</table>

As noted previously, the determination of the professional factor depends on the assumed coefficient of variation attributed to laboratory testing procedures. Therefore, the analysis was performed using two different assumed values of V_{Lab}: 6 percent and 10 percent. After the project advisory panel discussion and review of the professional factor results, it was decided to apply V_{lab} = 0.06. This is the conservative choice because it assigns more of the experimental dispersion to the professional factor associated with the one-way shear model.

These professional factors reflect the fact that the new ACI 318-19 one-way shear provisions (Table 6-5) are much improved over the full range of member sizes when compared to the older provisions (Table 6-4), and the main difference is evident in members without shear reinforcement. Test specimens with $d \leq 10$ in. are considered representative of slabs in buildings. Specimens with $10 < d \leq 33$ in. are considered representative of common beam sizes or even thick slabs in buildings. Specimens with $d > 33$ in. represent sizes that are less common in buildings, yet thick slabs and large transfer beams might be common in foundations or underground structures.

Although the professional factors improved significantly for members without shear reinforcement due to the changes in the ACI 318-19 one-way shear design provisions—there is one major exception: the coefficient of variation, V_P, *increased* slightly for the set of test specimens with the greatest d (even though the bias factor became more conservative for these deep members). One reason for this large coefficient of variation ($V_P = 0.23$) is the scarcity of tests in this size range ($d > 33$ in.). As noted in Section 9.2, a V_P of such large magnitude causes the reliability index to be relatively insensitive to changes in the strength reduction factor. Therefore, much more testing of large specimens should yield significant improvements in V_P and in the accuracy and efficiency of one-way shear design provisions. The professional factors used in this reliability analysis and strength reduction factor calibration are data driven. They were determined by processing and analyzing the available shear test databases. The professional factors are only as realistic as the input data are representative of the actual member behavior.

Chapter 7
RELIABILITY ANALYSIS PROCEDURE

This chapter presents a brief overview of reliability analysis fundamentals and describes the procedure selected to perform reliability analyses.

7.1 RELIABILITY ANALYSIS BACKGROUND

A structure or structural component is designed to resist an expected load. If the load effect is represented by Q and resistance (load carrying capacity) by R, then the condition of the structure can be determined by considering the function g as,

$$g = R - Q \tag{9}$$

and if $g \geq 0$ the structure is safe; otherwise, ($g < 0$) there is a failure. The limit state of the structure, i.e., the border between safe and unsafe, can be mathematically formulated using the limit state function g = 0.

$$g = R - Q = 0 \tag{10}$$

Hence, the probability of failure, P_F, is given by the probability of $g < 0$.

$$P_F = P(g < 0) \tag{11}$$

If R and Q are normal random variables, then g is also a normal variable, and the probability of failure can be calculated with Eq. (12),

$$P_F = P(g < 0) = \Phi\left(-\frac{\mu_g}{\sigma_g}\right) \tag{12}$$

where μ_g is the mean of g; σ_g is the standard deviation of g; and Φ is the standard normal cumulative distribution function (CDF). The reliability index β is defined as follows.

$$\beta = \frac{\mu_g}{\sigma_g} \tag{13}$$

Thus, the reliability index, β, is a function of the probability of failure (P_F), and as the probability of failure decreases, the reliability index increases. Table 7-1 provides the relationship among these concepts.

Table 7-1: Reliability Index, Reliability, and Probability of Failure (Nowak 1999)

Reliability Index β	Reliability $S = 1 - P_F$	Probability of Failure P_F
0.0	0.500	0.500×10^{0}
0.5	0.691	0.309×10^{0}
1.0	0.841	0.159×10^{0}
1.5	0.933 2	0.668×10^{-1}
2.0	0.977 2	0.228×10^{-1}
2.5	0.993 79	0.621×10^{-2}
3.0	0.998 65	0.135×10^{-2}
3.5	0.999 767	0.233×10^{-3}
4.0	0.999 968 3	0.317×10^{-4}
4.5	0.999 996 60	0.340×10^{-5}
5.0	0.999 999 713	0.287×10^{-6}
5.5	0.999 999 981 0	0.190×10^{-7}
6.0	0.999 999 999 013	0.987×10^{-9}

The reliability index or probability of failure values can be used to make judgments regarding the target level of safety or target reliability index. The acceptable minimum safety margin is selected in terms of a target reliability index (β_T). In previous code calibration efforts, the target reliability was selected as β_T = 3.5 for the flexural strength limit state in structural components such as cast-in-place concrete beams or girders (Szerszen and Nowak 1999). As shown in Table 7-1, this corresponds to a probability of failure of $P_f = 0.233\times10^{-3}$ (1 out of 4300). Increased target reliability index values (decreased P_f) are warranted for structural members for which failure has more severe consequences. In addition, an increased target reliability index can be selected to reduce the probability of failure for strength limit states associated with nonductile failures, as is the case with shear failures in structural concrete. For consistency, a target reliability index can also be selected based on successful past design practice by calculating the reliability indices associated with members designed in accordance with historically implemented design provisions.

The design parameters vary due to uncertainties, and they can be treated as random variables, including the load components, concrete compressive strength, yield strength of steel, location of reinforcement, etc. In this study, Q is considered as a sum of dead load and live load. Therefore, the limit state function is rewritten as

$$g = R - (D + L) = 0 \tag{14}$$

where D, L, and R are random variables corresponding to dead load effect, live load effect, and corresponding resistance, respectively.

A typical design formula for gravity loads is

$$\phi R_n \geq max\ (1.2D_n + 1.6L_n\ , 1.4D_n) \tag{15}$$

where ϕ is the strength reduction factor; D_n, L_n, and R_n are the nominal values of the dead load effect, live load effect, and corresponding resistance, respectively. Nominal values are what are used in design.

In previous calibrations, gravity load combinations were used as the reference case to determine strength reduction factors. The resulting strength reduction factor selected for the design standard was then extended for use with other load combinations, such as those including wind or earthquake loads, based on the principle that the variability associated with different types of loads is captured in the specific load factors. In certain situations, such as outlined in ACI 318-19 Section 21.2.4 for components of seismic-force-resisting systems, the strength reduction factor (determined from the reference case) may be modified to further reduce the probability of undesirable failure modes. This study is focused on evaluating the strength reduction factor for one-way shear with respect to the reference gravity load combinations.

7.2 Monte Carlo Approach

The Monte Carlo method is a technique to numerically simulate outcomes without performing extensive physical testing. Prior test results can be used to determine statistical parameters with specific probability distributions of the important parameters in the problem to be solved. This information is then used to generate samples of numerical data.

Nowak and Collins (2013) provide a further explanation of the procedure. This explanation involves consideration of a series of actual tests of concrete cylinders to determine the compressive strength f'_c. Assume that a relative frequency histogram has been drawn using the data, and a lognormal probability distribution, as shown in Figure 7-1, appears to fit the data reasonably well.

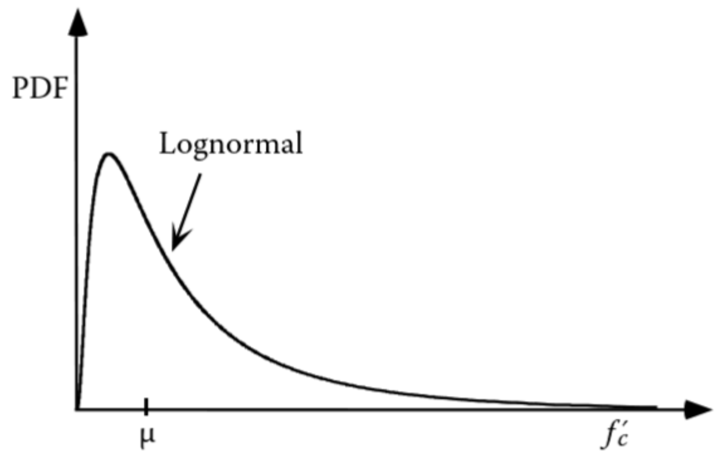


Figure 7-1: Lognormal Probability Density Function (PDF) for the Example Considered

Now consider a concrete column. The compressive load carrying capacity is $0.85f'_cA_c$, where A_c is the cross-sectional area of the column concrete and is assumed for simplicity to be deterministic. Assume that the applied load, Q, is normally distributed with a mean value μ_Q and a coefficient of variation V_Q. What is the probability of failure of this column, P_F?

Since the problem involves different probability distributions, Monte Carlo simulation can be used to solve the problem following this general procedure:

1. Randomly generate a value of f'_c (using the appropriate lognormal probability distribution) and calculate $R = 0.85f'_cA_c$.
2. Randomly generate a value for Q, using its probability distribution.
3. Compute $Y = R - Q$.
4. Store the computed value.
5. Repeat steps 1–4 until a sufficient number of Y values have been simulated.
6. If a sufficient number of simulations have been performed, the probability of failure can be estimated as:

$$\overline{P}_F = \frac{\text{Total simulated cases that } Y < 0}{\text{Total simulated cases}}$$

Traditionally, the number of simulations is considered sufficient when there are at least ten negative cases in step 3. Therefore, the required number of simulations is a function of the expected probability of failure for a given scenario. If the reliability indices of one-way shear critical concrete members are expected to be approximately $\beta = 4.0$, then the probability of failure of $P_F = 0.32\times10^{-4}$ needs to be observable in the simulation. For a minimum of ten expected negative cases, the total number of simulated cases should be around 320,000. Hence, to enhance accuracy, a minimum of 500,000 simulations was selected for use in each Monte Carlo analysis performed for this study.

7.3 RELIABILITY ANALYSIS PROCEDURE

The reliability analysis procedure included simulating the resistance by using the Monte Carlo method (for the appropriate one-way shear strength equation), adjusting the resistance statistical parameters (mean and standard deviation) to the lower tail of the simulated distribution, and then computing the approximate reliability index with Eq. (13) assuming load and resistance are uncorrelated:

$$\beta = \frac{\mu_g}{\sigma_g} = \frac{\mu_R - \mu_Q}{\sqrt{\sigma_R{}^2 + \sigma_Q{}^2}} = \frac{\mu_R - (\mu_D + \mu_L)}{\sqrt{\sigma_R{}^2 + \left(\sigma_D{}^2 + \sigma_L{}^2\right)}} \tag{16}$$

where μ_R is the mean of R; σ_R is the standard deviation of R; μ_Q is the mean of Q; σ_Q is the standard deviation of Q. Given modern computational resources, this method can be feasibly performed in a reasonable amount of time.

7.4 SIMULATIONS PERFORMED

Numerous simulations were performed to cover a wide range of design scenarios. Also, validation steps are considered. A descriptive list of the primary simulation steps is provided here. The representative cross sections and reinforcement amounts selected for the simulations are also outlined in this section.

Step 1. Simulation of flexural strength limit state as in Nowak et al. (2005)

To validate the simulation programming for this study, one scenario was taken from Nowak et al. (2005) and replicated with the same conditions and statistical parameters.

Step 2. Simulation of flexural strength limit state including a refined consideration for in-situ compressive strength

A simple reduction factor of 0.90 was implemented in previous calibrations to account for differences between the strength of standard test cylinders and in-situ compressive strength. The literature review for the present study revealed that a statistical factor, called F_2, that follows a lognormal distribution with λ = 1.03 and V = 0.14 is appropriate for this purpose (see Section 4.2.5). The objective of this step is to study the influence of the implementation of this refined approach relative to the simple historical approach.

Step 3. Simulation of one-way shear strength limit state with traditional shear design method ACI 318-14 and ϕ = 0.75.

This is a baseline simulation to determine the reliability indices associated with ACI 318-14 design practices. The objective is to develop a background for the selection of a target reliability index for one-way shear.

Step 4. Simulation of one-way shear strength limit state with new shear design method ACI 318-19 and ϕ = 0.75

Because the new design method offers more accurate predictions of one-way shear strength than the traditional approach, it is expected that the reliability of members designed in accordance with the new method is greater than those designed in accordance with the traditional method. However, this might not be the case in all scenarios. The objective is to quantify the change in the reliability index attributable to the implementation of the ACI 318-19 design method.

As noted in Section 2.3, the ACI 318-19 design method offers two alternative approaches for the computation of V_c in members with shear reinforcement, both of which are given in Eq. (7). One computation includes the amount of longitudinal flexural reinforcement; the other does not. In the simulations performed for this study, it was assumed that the designer would choose the greater of the two calculated V_c values. This represents the least conservative design choice, but it yields the most conservative estimate of reliability index in the simulation (i.e., the smaller reliability index).

Step 5. Simulation of one-way shear strength limit state with new shear method ACI 318-19 for different strength reduction factors: ϕ = 0.70, ϕ = 0.80.

Based on the increased reliability attributable to the improved accuracy of the new design method, an increased strength reduction factor might be used and still maintain (or exceed) the reliability level of past successful designs. The objective is to try different practical ϕ values and determine which strength reduction factor best agrees with the selected target reliability index.

7.4.1 Selected Cross Sections

The slab and beam cross sections summarized in Table 7-2 were selected as broadly representative of reinforced concrete buildings and other structures for which ACI 318 might apply.

Table 7-2: Cross Sections for Slabs and Beams Used in Simulations

Shear reinforcement	**Member type**	**Size range**	**Selected Dimensions**
without shear reinforcement	slabs	*Small* $d \leq 10$ in.	$d = 6$ in.
		Medium 10 in. $< d \leq 33$ in.	$d = 16$ in. and $d = 21.5$ in.
		Large $d > 33$ in.	$d = 45$ in.
	beams	*Small* $d \leq 10$ in.	$b_w = 12$ in. and $d = 10$ in.
		Medium 10 in. $< d \leq 33$ in.	$b_w = 36$ in. and $d = 27.5$ in.
		Large $d > 33$ in.	$b_w = 24$ in. and $d = 45$ in.
with shear reinforcement	beams	*Small* $d \leq 10$ in.	$b_w = 12$ in. and $d = 10$ in.
		Medium 10 in. $< d \leq 33$ in.	*(a)* $b_w = 18$ in. and $d = 25.5$ in. *(b)* $b_w = 36$ in. and $d = 27.5$ in.
		Large $d > 33$ in.	$b_w = 24$ in. and $d = 45$ in.

7.4.2 Amounts of Reinforcement

The reinforcement amounts were selected to represent reinforced concrete buildings and other structures for which ACI 318 might apply. Reinforcement amounts were selected to simulate (a) lightly, (b) moderately, and (c) heavily reinforced one-way slabs and beams with respect to both flexural and shear strength. Table 7-3 shows the reinforcement amounts selected for the simulations.

Table 7-3: Reinforcement Amounts for Slabs and Beams Used in Simulations

Member	Reinforcement	Variable	Value
without shear reinforcement	flexural for slabs	$\rho_{w\ light}$	0.002
		$\rho_{w\ mod}$	0.015
		$\rho_{w\ heavy}$	0.030
	flexural for beams	$\rho_{w\ light}$	0.003
		$\rho_{w\ mod}$	0.015
		$\rho_{w\ heavy}$	0.040
with shear reinforcement	flexural for beams	$\rho_{w\ light}$	0.003
		$\rho_{w\ mod}$	0.015
		$\rho_{w\ heavy}$	0.040
	shear for beams	$(A_v/s)_{light}$	$50b_w/f_y$ and s = min ($d/2$, 24 in.)
		$(A_v/s)_{mod}$	$250b_w/f_y$ and s = min ($d/2$, 24 in.)
		$(A_v/s)_{heavy}$	$500b_w/f_y$ and s = min ($d/4$, 12 in.)

For longitudinal reinforcement, $\rho_{w\ light}$ for slabs was selected based on the specification of minimum flexural slab reinforcement of $0.0018A_g$ (see ACI 318-19 Table 7.6.1.1). $\rho_{w\ light}$ for beams was selected based on the minimum reinforcement ratio requirement of $200/f_y$ (ACI 318-19 9.6.1.2). $\rho_{w\ mod}$ for slabs and beams is the flexural steel ratio at which $8\rho_{w\ mod}^{1/3} = 2$, which is, therefore, the ratio at which the traditional one-way shear design method and new method give the same V_c (without size effect). $\rho_{w\ heavy}$ for slabs and beams was selected by estimating the most flexural reinforcement that would result in a tension-controlled failure at ε_t = 0.005. For slabs, 8,000 psi concrete requires that the flexural steel ratio be less than approximately 0.030. For beams, since there is usually some amount of compression steel, the ratio could be greater; therefore, 0.040 was selected. This also approximates the limit for a 12,000 psi concrete with no compression steel.

Likewise, “light,” “moderate,” and “heavy” amounts of shear reinforcement were designated for simulation. For shear reinforcement ranges, $(A_v/s)_{light}$ corresponds to the $50b_w/f_y$ minimum shear reinforcement requirement in the code (ACI 318-19 9.6.3.3). The moderate level corresponds to the shear force where $V_s = 4\sqrt{f_c'}b_w d$, thus approximating the boundary at which the maximum stirrup spacing requirements change. Accordingly, $(A_v/s)_{mod}$ approximates $250b_w/f_y$ when 4,000 psi concrete is specified. Similarly, $(A_v/s)_{heavy}$ corresponds to the maximum allowed shear force level at which $V_s = 8\sqrt{f_c'}b_w d$. Accordingly, $(A_v/s)_{heavy}$ approximates $500b_w/f_y$ when 4,000 psi concrete is specified.

Chapter 8
RESULTS OF RELIABILITY ANALYSES

The selected reliability analysis procedure was executed using the ACI 318-19 shear strength provisions as prediction models as described in Chapter 7. The results are summarized in this chapter.

8.1 Simulation of Flexural Strength Limit State

According to the simulations performed by Nowak et al. (2005), the statistical parameters for resistance for a beam with b_w = 10 in. and d = 20 in., made of 4000 psi concrete with a reinforcement ratio of ρ = 0.006 are λ_R = 1.137 and V_R = 0.08. According to the simulations performed in this study, for the same member, the parameters obtained are λ_R = 1.13 and V_R = 0.08. The slight difference in the bias factor could be explained by the different number of simulations or number truncation. Overall, the simulation processes formulated in R and MATLAB for this study were considered to be correct after this benchmark step.

8.2 Simulation of Flexural Strength Limit State Including a Refined Consideration for In-Situ Compressive Strength

When using a simple reduction factor of 0.90 for accounting for differences between the strength of standard test cylinders and in-situ compressive strength, the statistical parameters for resistance for a beam with b_w = 10 in. and d = 20 in., made of 4000 psi concrete with a reinforcement ratio of ρ = 0.006 are λ_R = 1.13 and V_R = 0.08. According to the simulations employing the newly selected (see Section 4.2.5) lognormal distribution (with λ = 1.03 and V = 0.14) for the same purpose, the parameters obtained are λ_R = 1.14 and V_R = 0.08. Therefore, the simplified approach used in previous calibrations is deemed conservative at this strength level. The newer lognormal approach for representing in-situ strength was used for the rest of the reliability analyses in this study.

8.3 Simulation of One-Way Shear Strength Limit State with Traditional Shear Design Method ACI 318-14 and ϕ = 0.75

The reliability indices associated with ACI 318-14 design practices were determined for several scenarios. Figure 8-1 shows the reliability index for a small beam (a) and a large beam (b) without shear reinforcement as a function of the load ratio (i.e., dead load to total load). It can be observed that there is a significant decrease in the reliability index for the beam with greater depth (at all load ratios). Also, the maximum reliability index is obtained for the theoretical case of load ratio equal to zero (i.e., only live load), and the minimum value occurs for scenarios where the dead load is about 90% of the total load. Load ratios for most structures lie between these two boundaries.

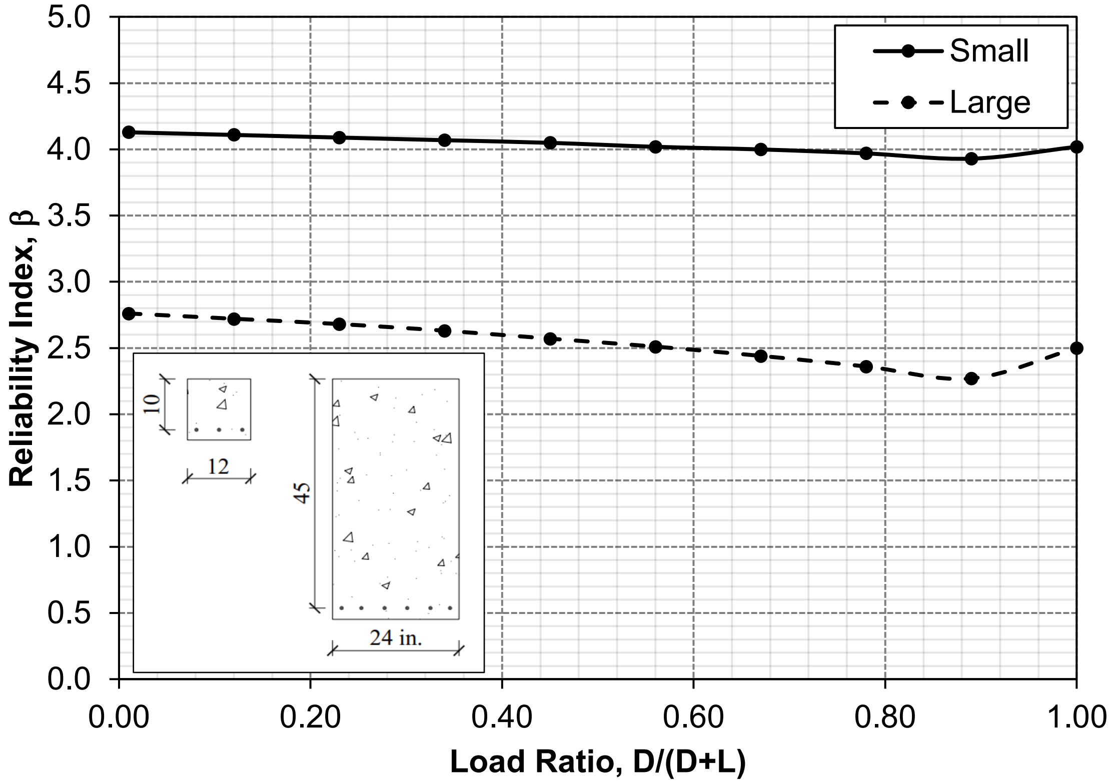


Figure 8-1: ACI 318-14 Reliability Index as a Function of the Load Ratio for: (a) Small Beam without Shear Reinforcement; and (b) Large Beam without Shear Reinforcement

The results of the numerous ACI 318-14 simulations are summarized in Figure 8-2 and Figure 8-3. For slabs (Figure 8-2a), a significant decrease in the reliability of the members is evident as the effective depth increases. The two lines represent the upper and lower bounds of the range of simulated values at each member depth. The reliability index of thick, one-way slabs without shear reinforcement appears to be dangerously low: ranging from 0 to 1 for d = 45 in.

(a)

(b)

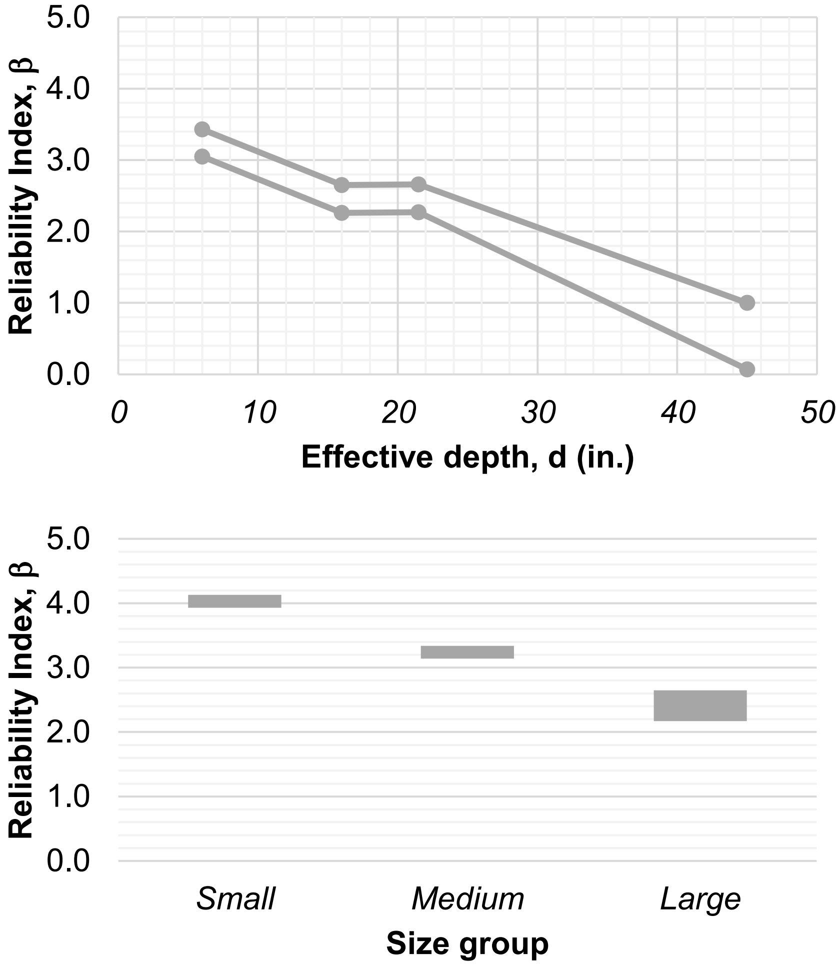


Figure 8-2: Reliability Index Range Associated with ACI 318-14 for (a) Slabs and (b) Beams without Shear Reinforcement.

For beams that lack shear reinforcement and remain below the $V_u \leq \phi \frac{V_c}{2}$ shear demand threshold (Figure 8-2b), the results are shown as a range for each size group defined in Table 7-2. Here, the reliability index is also sensitive to the size of the member, and the reliability of large beams is relatively low (approximately 2.5), but significantly better than in slabs, which are not required to satisfy the $V_u \leq \phi \frac{V_c}{2}$ threshold.

For beams with shear reinforcement designed in accordance with ACI 318-14, two sets of simulations were performed. First, the professional factor was applied to the total shear strength of the members, i.e., V_c + V_s, (Figure 8-3a), as they were originally developed by Aguilar et al. (2023). In this case, the reliability of small beams is somewhat greater than for medium and large beams. The load ratio range explains the range between minimum and maximum reliability. There are two facts that are unrealistic or inappropriate in these results: (1) the obtained reliability indices are mostly lower than 3.0, which is also lower than the reliability index for the flexural ultimate limit state (β = 3.5), which would imply that shear failures occur more frequently than flexural failures in practice; (2) the reliability level is not affected by the amount of transverse reinforcement. The slight vertical change in the reliability index in Figure 8-3a is only due to the load ratio range. Hence, these observations led the authors to consider a second approach.

(a)

(b)

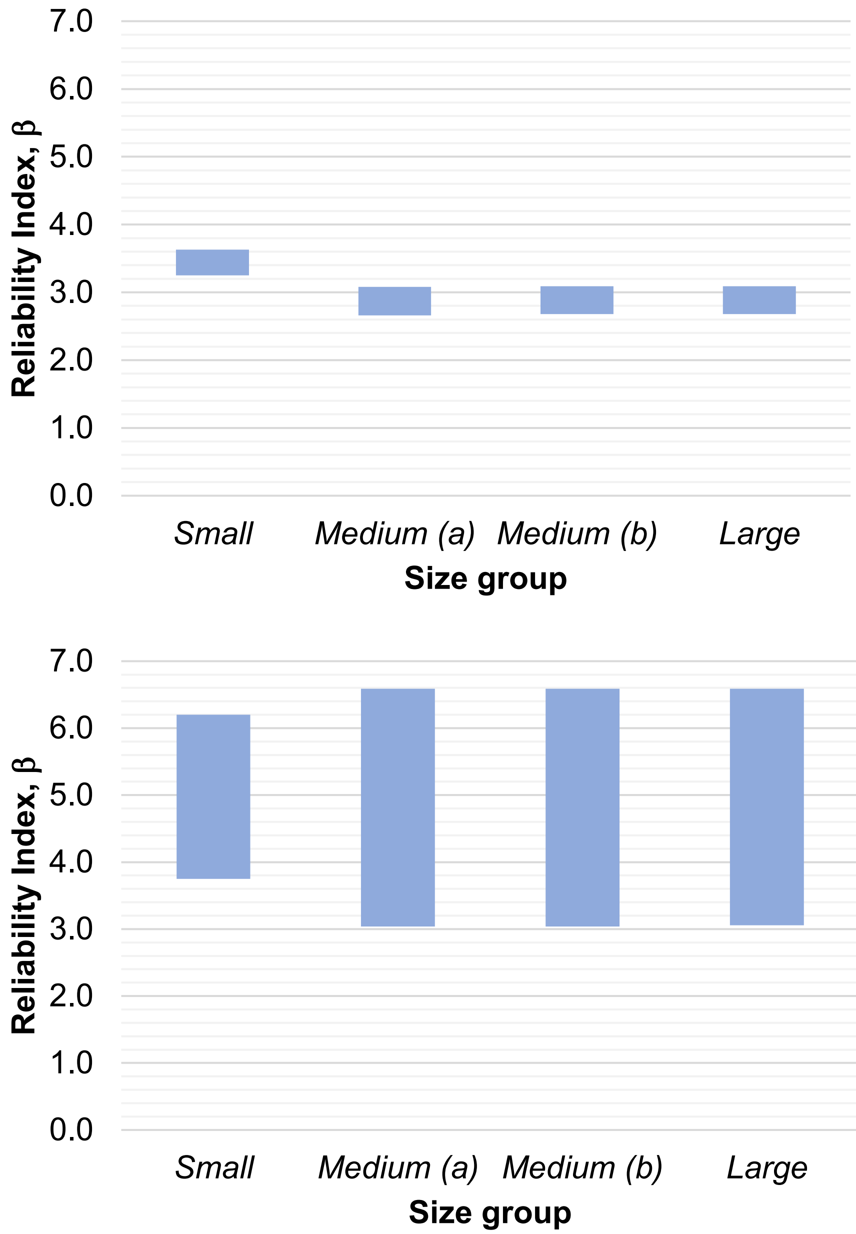


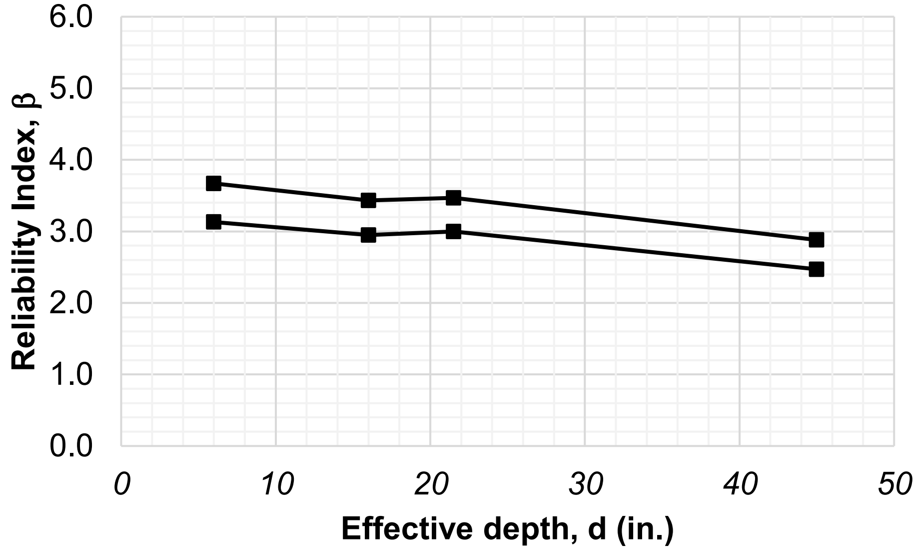
Figure 8-3: Reliability Index Range Associated with ACI 318-14 for Beams with Shear Reinforcement using: (a) Professional Factor on V_n (b) Professional Factor on V_c.

The second approach was to apply the professional factor to the concrete contribution only, i.e., V_c only, assuming that most of the uncertainty in the analysis is attributable to the concrete contribution to the shear strength. The reliability indices for this second approach are presented in Figure 8-3b. The reliability indices are greater than 3.0, and there is a significant change in reliability based on the amount of transverse reinforcement. In this case, the minimum reliability indices correspond to beams with light shear reinforcement and a high dead load to total load ratio. In contrast, the maximum reliability indices correspond to beams heavily reinforced for shear and a load ratio of about 0.30–0.50.

8.4 Simulation of One-Way Shear Strength Limit State with New Shear Design Method ACI 318-19 and $\phi = 0.75$

Next, the reliability indices associated with implementing the ACI 318-19 design procedures were determined. The results of the simulations performed are summarized in Figure 8-4 and Figure 8-5.

(a)



(b)

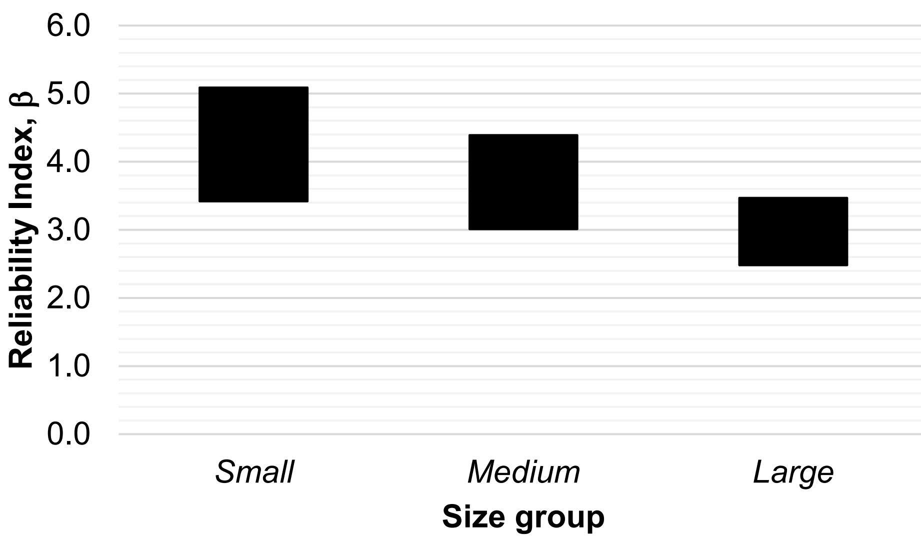


Figure 8-4: Reliability Index Range Associated with ACI 318-19 for: (a) Slabs and (b) Beams without Shear Reinforcement.

For slabs (Figure 8-4a), the reliability index is reported as a function of the effective depth. The range between minimum and maximum bounds is related to the load ratio range. Furthermore, there is a decrement of approximately 0.5 points in the reliability index comparing the smallest to the largest slab considered, which is far less extreme than what was found regarding the reliability of slabs designed according to the ACI 318-14 method (Figure 8-2a).

For beams that lack shear reinforcement and remain below the ACI 318-19 $V_u \leq \phi\sqrt{f_c'}b_w d$ shear demand threshold (Figure 8-4b), the smallest reliability indices were obtained for beams with light longitudinal reinforcement and high load ratios of about 0.90. The greatest reliability indices were found for beams with heavy longitudinal reinforcement with very low load ratios approaching zero. The size of the member influences the reliability index; however, this influence is less pronounced than in the case of the ACI 318-14 design method (Figure 8-2b).

For beams with shear reinforcement designed in accordance with ACI 318-19, two sets of simulations were performed. First, the professional factor was applied to the total shear strength of the members (Figure 8-5a), as they were originally developed by Aguilar et al. (2023). The reliability indices obtained for the ACI 318-19 new one-way shear design method are almost identical to those obtained for the ACI 318-14 method. Next, assuming that most of the uncertainty in the analysis is attributable to the concrete contribution to the shear strength, the professional factor was applied to the concrete contribution only; the results are shown in Figure 8-5b. The minimum reliability indices here were obtained for heavily longitudinally reinforced beams with light shear reinforcement and high load ratios of about 0.90. The maximum reliability indices were found for lightly longitudinally reinforced beams with heavy shear reinforcement and low load ratios of about 0.20. The ranges of reliability indices obtained for the ACI 318-19 one-way shear design method are very similar to those obtained for the ACI 318-14 method (Figure 8-3b). Recall that for the ACI 318-19 method, it was assumed within the simulations that the designer would choose the greater of the two V_c alternatives. Thus, these reliability indices represent the least possible value based on this choice.

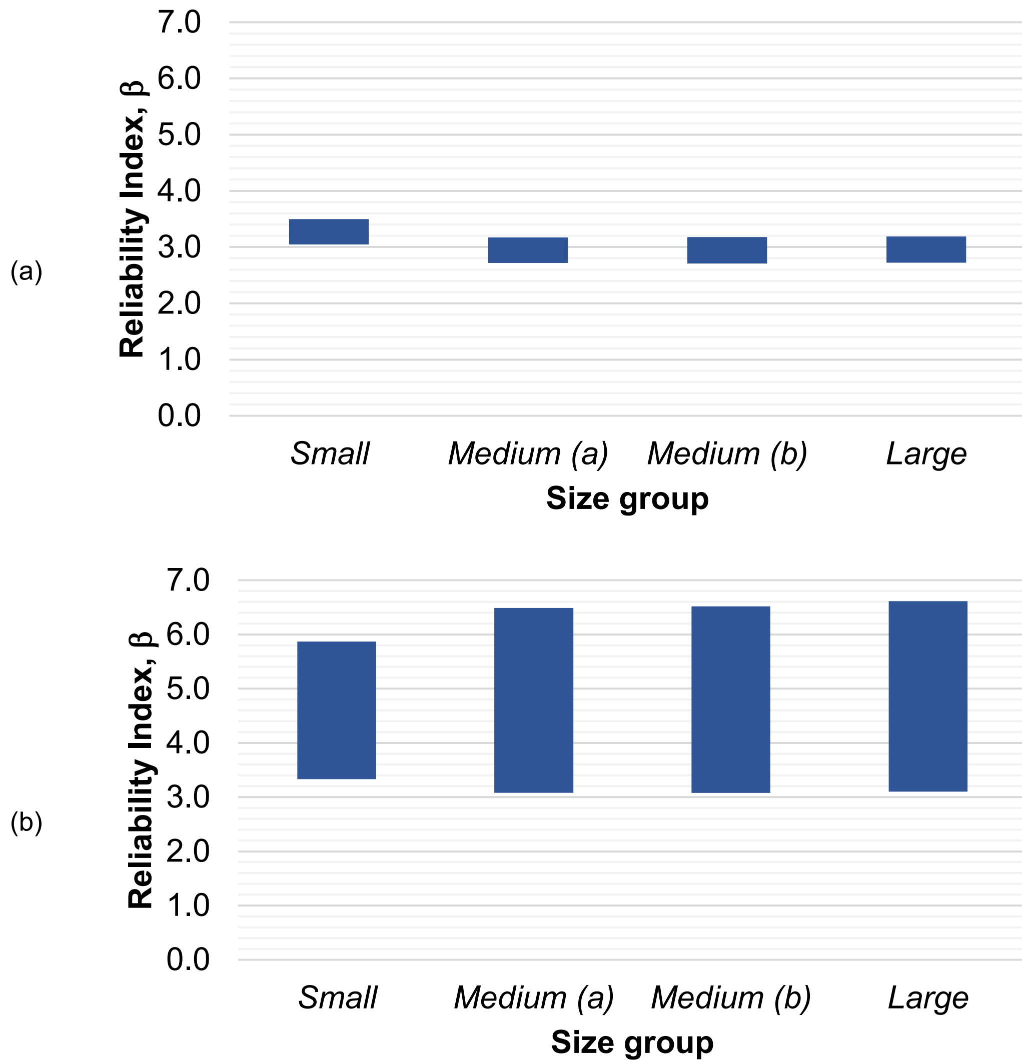
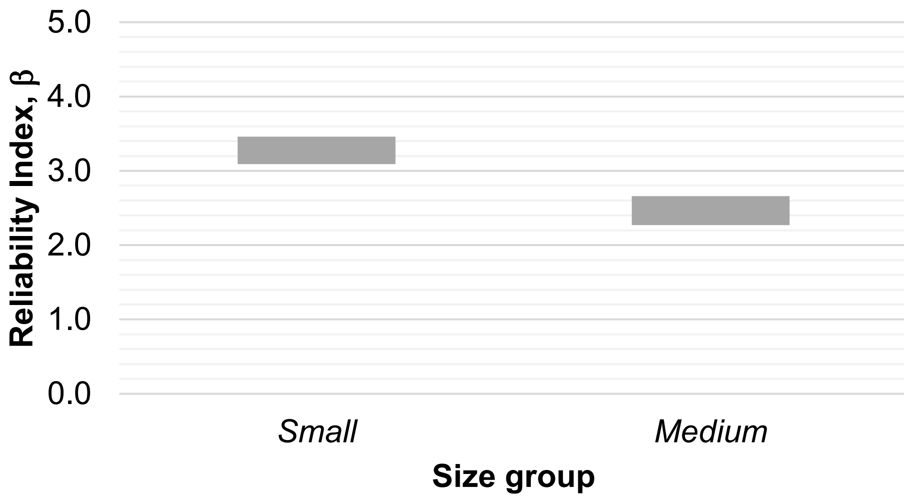


Figure 8-5: Reliability Index Range Associated with ACI 318-19 for Beams with Shear Reinforcement using: (a) Professional Factor on V_n (b) Professional Factor on V_c.

In order to approximate the exception cases in Table 9.6.3.1 of the ACI 318 code, members lacking shear reinforcement that satisfy the less stringent $V_u \leq \phi V_c$ shear demand threshold were simulated, i.e., shallow depth beams (h ≤ 10 in.), shallow beams integral with slab (h ≤ 24 in.), beams constructed with steel fiber, and one-way joist systems as defined in the code. However, these members might exhibit behavior that is not captured by the test databases or by the reliability analyses performed in this study.

Figure 8-6 shows that ACI 318-19 provisions for the design of members lacking shear reinforcement that satisfy $V_u \leq \phi V_c$ lead to designs that are equally or more reliable than those resulting from the application of the $V_u \leq \phi V_c$ shear demand threshold in the ACI 318-14 design method. This is particularly evident for the medium-depth beams. The range observed in the figure is attributable to the live load range considered.

(a)



(b)

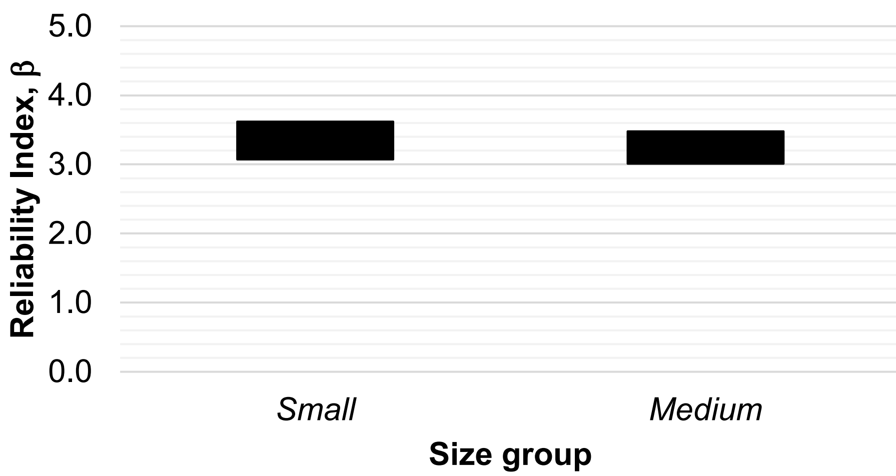


Figure 8-6: Reliability Index Range Associated with Members without Shear Reinforcement designed for $V_c \geq (V_u/\phi)$: (a) ACI 318-14 and (b) ACI 318-19.

8.5 Simulation of One-Way Shear Strength Limit State with New Shear Design Method for Different Strength Reduction Factors: ϕ = 0.70, ϕ = 0.80

It was hypothesized when this study was proposed that based on the increased reliability attributable to the improved accuracy of the new design method, an increased strength reduction factor might be used and still maintain (or exceed) the reliability level of past successful designs. The objective of this step was to try different practical ϕ values and determine which strength reduction factor best agrees with the selected target reliability index.

Figure 8-7, Figure 8-8, and Figure 8-9 show a comparison of the range of reliability indices obtained when applying strength reduction factors of ϕ = 0.70, ϕ = 0.75, and ϕ = 0.80 for slabs, beams without shear reinforcement, and beams with shear reinforcement. In all cases, the increase in ϕ from the current value of 0.75 to 0.80 decreases the lower bound of the reliability index by approximately 0.2.

ϕ = 0.70

ϕ = 0.75

ϕ = 0.80

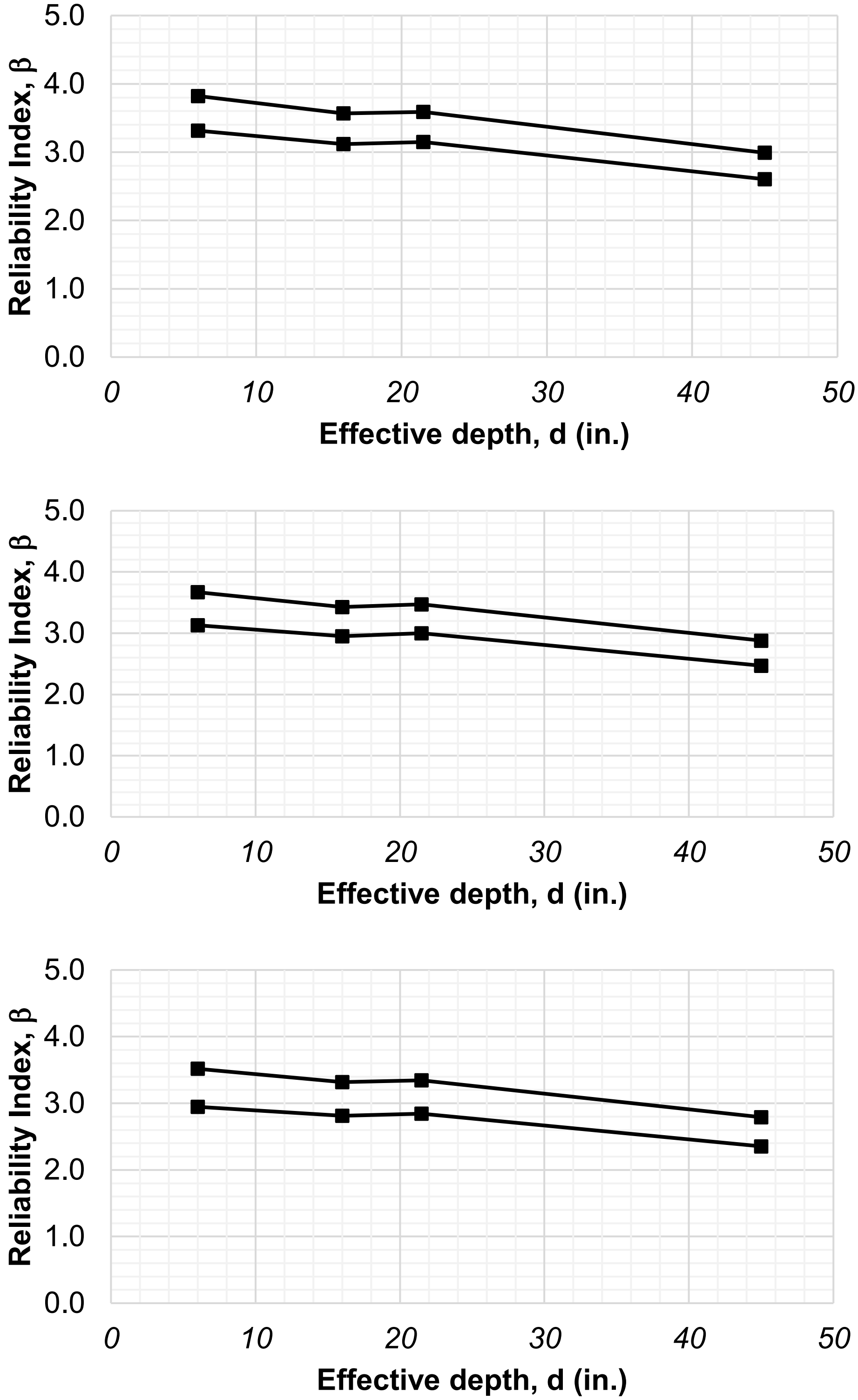


Figure 8-7: Reliability Index Range Associated with ACI 318-19 for Slabs for Different Practical Strength Reduction Factors

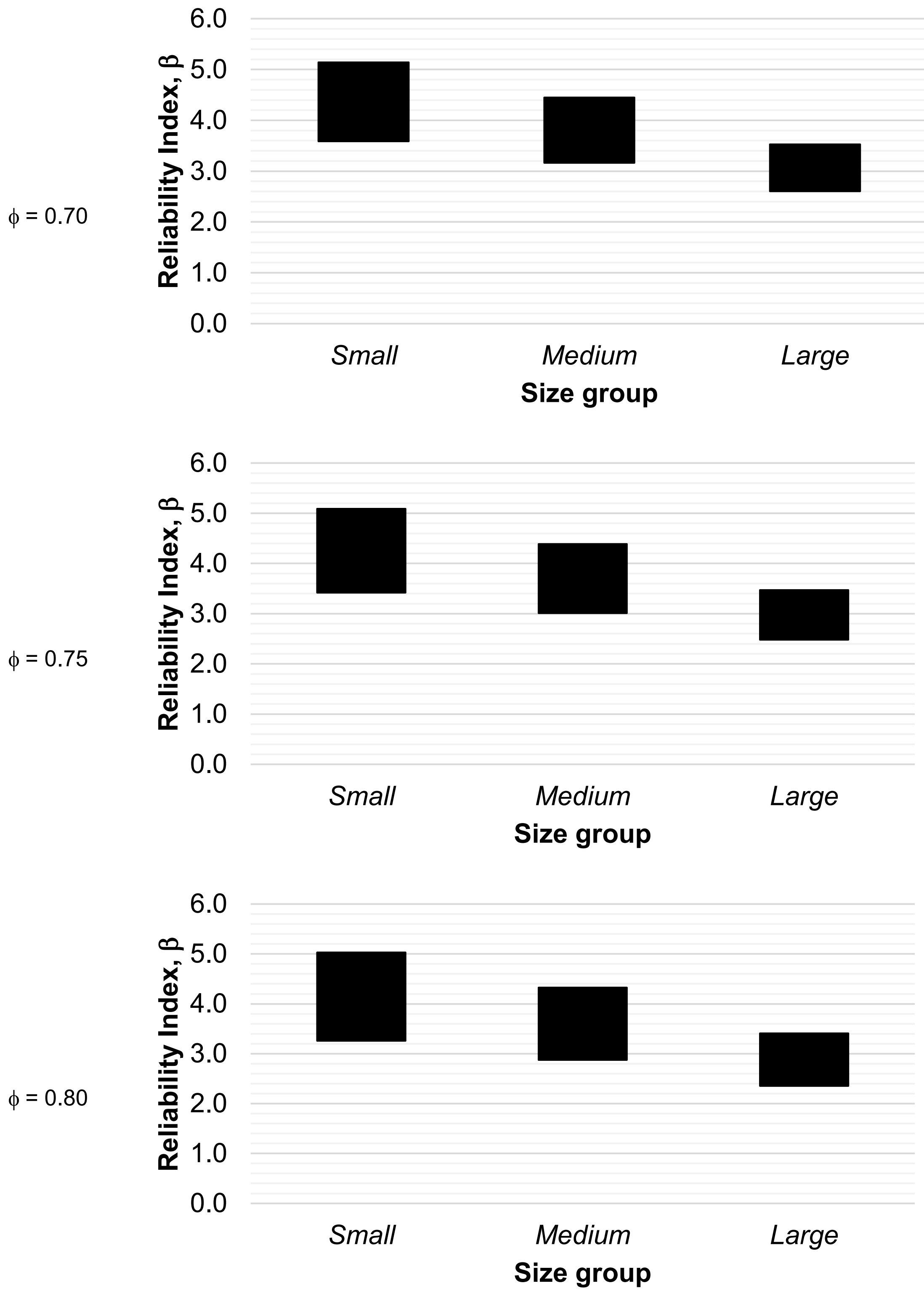
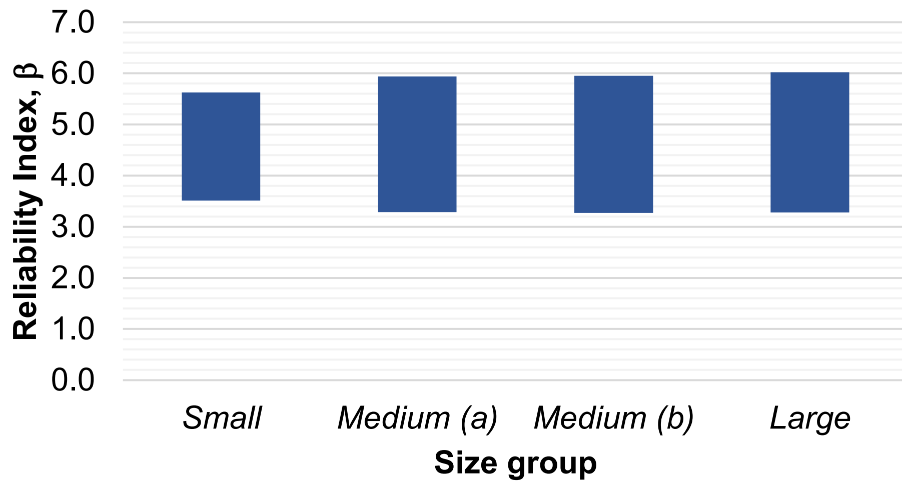
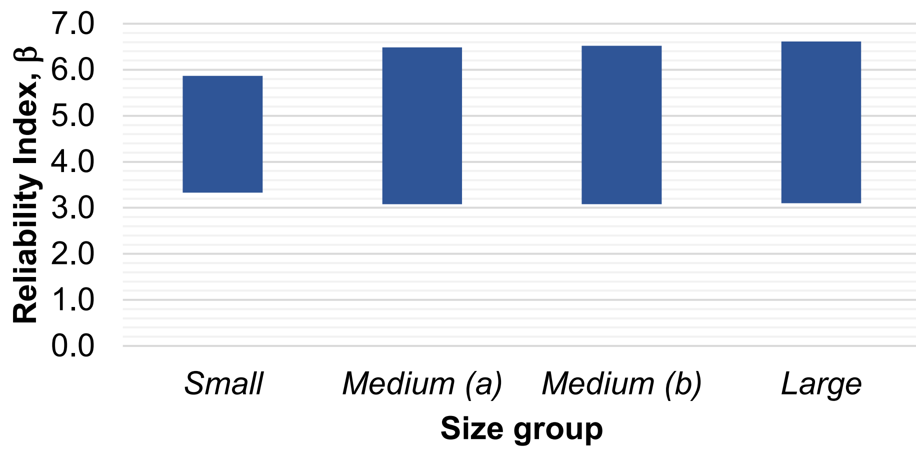


Figure 8-8: Reliability Index Range Associated with ACI 318-19 for Beams without Shear Reinforcement for Different Practical Strength Reduction Factors

ϕ = 0.70



ϕ = 0.75



ϕ = 0.80

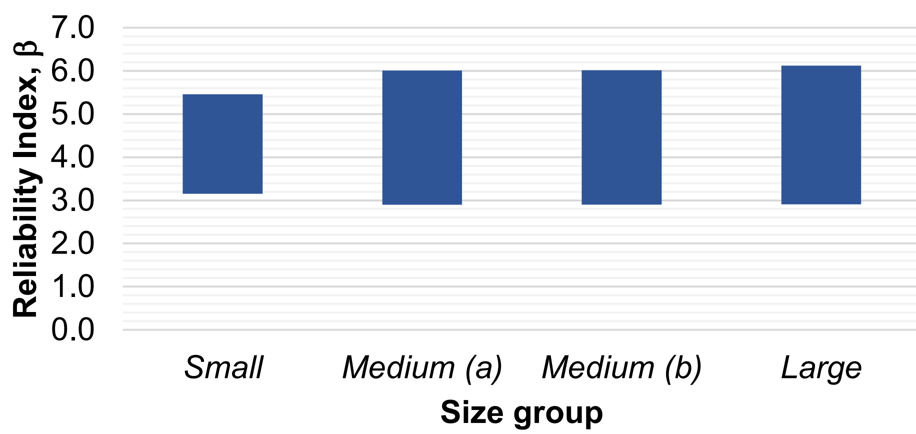


Figure 8-9: Reliability Index Range Associated with ACI 318-19 for Beams with Shear Reinforcement for Different Practical Strength Reduction Factors

Increasing the ϕ factor to 0.80 decreases the lower bound of the reliability index range to just less than 3.0 for medium-size beams without shear reinforcement (Figure 8-8) and for medium and large-size beams with shear reinforcement (Figure 8-9). In the case of slabs without shear reinforcement (Figure

8-7), increasing the ϕ factor to 0.80 decreases the lower bound of the reliability index range to slightly less than 3.0 for normal-thickness slabs. The reliability index for very thick slabs is less than 3.0 even if the ϕ factor is *decreased* to 0.70. As noted in Section 6.3.3, the scarcity of tests in this size range (d > 33 in.) accompanies a large coefficient of variation (V_P = 0.23) in the professional factor. The professional factors used in this reliability analysis and strength reduction factor calibration are data driven. They were determined by processing and analyzing the available shear test databases. The professional factors in this range are only as realistic as the small number of input data are representative of the member behavior.

Chapter 9
STRENGTH REDUCTION FACTORS FOR ONE-WAY SHEAR

9.1 SELECTION OF TARGET RELIABILITY INDEX

The target reliability index for the one-way shear strength limit state was determined after investigating the reliability index associated with designs in compliance with ACI 318-14 specification corresponding to the safety margin inherent in past shear design practice. These values were presented in Section 8.3.

The reliability index range obtained for ACI 318-14 for slabs and beams with d > 33 in. and without shear reinforcement indicates that these designs are very unconservative; therefore, this range of reliability was deemed inappropriate as a target value. The reliability values associated with past practice for small and medium-size members are therefore used as reference values. The target reliability indices, as indicated in Table 9-1, were determined as the minimum reliabilities from designs in compliance with ACI 318-14, but not less than 3.0 (which is necessary to filter out the undesirable reliability of slabs with d greater than approximately 10 in.).

Table 9-1: Target Reliability Index for One-Way Shear Strength Limit State

Member type	Target reliability Index, β_T	Reference
Slabs	3.0	Figure 8-2a
Beams without shear reinforcement	3.0	Figure 8-2b
Beams with shear reinforcement	3.0	Figure 8-3

For slabs, the target reliability index for the flexural strength limit state was β_T = 2.5 in the previous calibration of ACI 318 (Szerszen and Nowak 2003). Hence, target reliability for one-way shear strength of β_T = 3.0 is ideal as it is consistent with the long-standing code philosophy that the probability of nonductile shear failure should be much less than for a flexural failure. However, for beams with and without shear reinforcement, these analyses showed shear limit state reliability indices that are lower than the target reliability index for flexural strength limit state that was selected as β_T = 3.5 in the past. An ideal value for the one-way shear strength limit state would be β_T = 4.0. Nevertheless, the engineering community has not reported issues or concerns regarding the shear design of beams; therefore, there is little justification for increasing the target reliability index of these members beyond the reliability of successful past practice. Hence, the recommended values in Table 9-1 correspond to the minimum reliability indices obtained from designs in compliance with ACI 318-14, but not less than 3.0 (which limits the case of ACI 318-14 slabs with d greater than approximately 10 in.).

9.2 SELECTION OF STRENGTH REDUCTION FACTOR

In this section, the reliability of the most common designs is reported. The full range of reliability indices is reported for several scenarios in Chapter 8. Figure 9-1 shows the reliability index of members in which dead load and live load are equal (load ratio equal 0.5) for (a) slabs; (b) lightly longitudinally reinforced beams without shear reinforcement; and (c) lightly longitudinally reinforced beams with minimum shear reinforcement (in which case the professional factor is applied to V_c only). In each graph, the reliability index is presented as a function of the size group for different practical possible strength reduction factors, i.e., ϕ = 0.70, ϕ = 0.75, and ϕ = 0.80.

Based on the results for the most common members:

(1) The reliability indices obtained with the ACI 318-19 design provisions for slabs are greater and more uniform than with the ACI 318-14 provisions. The reliability indices obtained with ACI 318-19 design provisions exceed the selected target reliability index of 3.0, except for very thick slabs (d of approximately 33 in. or larger).

(2) The reliability indices obtained with the ACI 318-19 provisions for beams that lack shear reinforcement (and remain below the ACI 318-19 $V_u \leq \phi\sqrt{f_c'}b_w d$ shear demand threshold) are more uniform than with the ACI 318-14 provisions across the complete size range. The reliability indices obtained with ACI 318-19 design provisions exceed the selected target reliability index of 3.0, except for deep beams (d of approximately 33 in. or larger) without shear reinforcement.

(3) The reliability indices obtained with the ACI 318-19 provisions for beams with shear reinforcement are lower than with ACI 318-14 for small beams (d of approximately 10 in.) and higher than with ACI 318-14 for medium and large beams. Although there is a small apparent increase in reliability for medium and large beams with shear reinforcement, this is only observed when the analysis uncertainty is assumed to be assigned only to the concrete contribution. When the analysis uncertainty is applied to the total shear strength, the reliability of designs associated with ACI 318-14 and ACI 318-19 are almost identical.

Hence, the implementation of the new ACI 318-19 one-way shear design method resulted in greater or equal reliability indices relative to ACI 318-14 in most cases. The major differences are in one-way slabs, and a more uniform reliability level is obtained for beams without shear reinforcement.

(a)

(b)

(c)

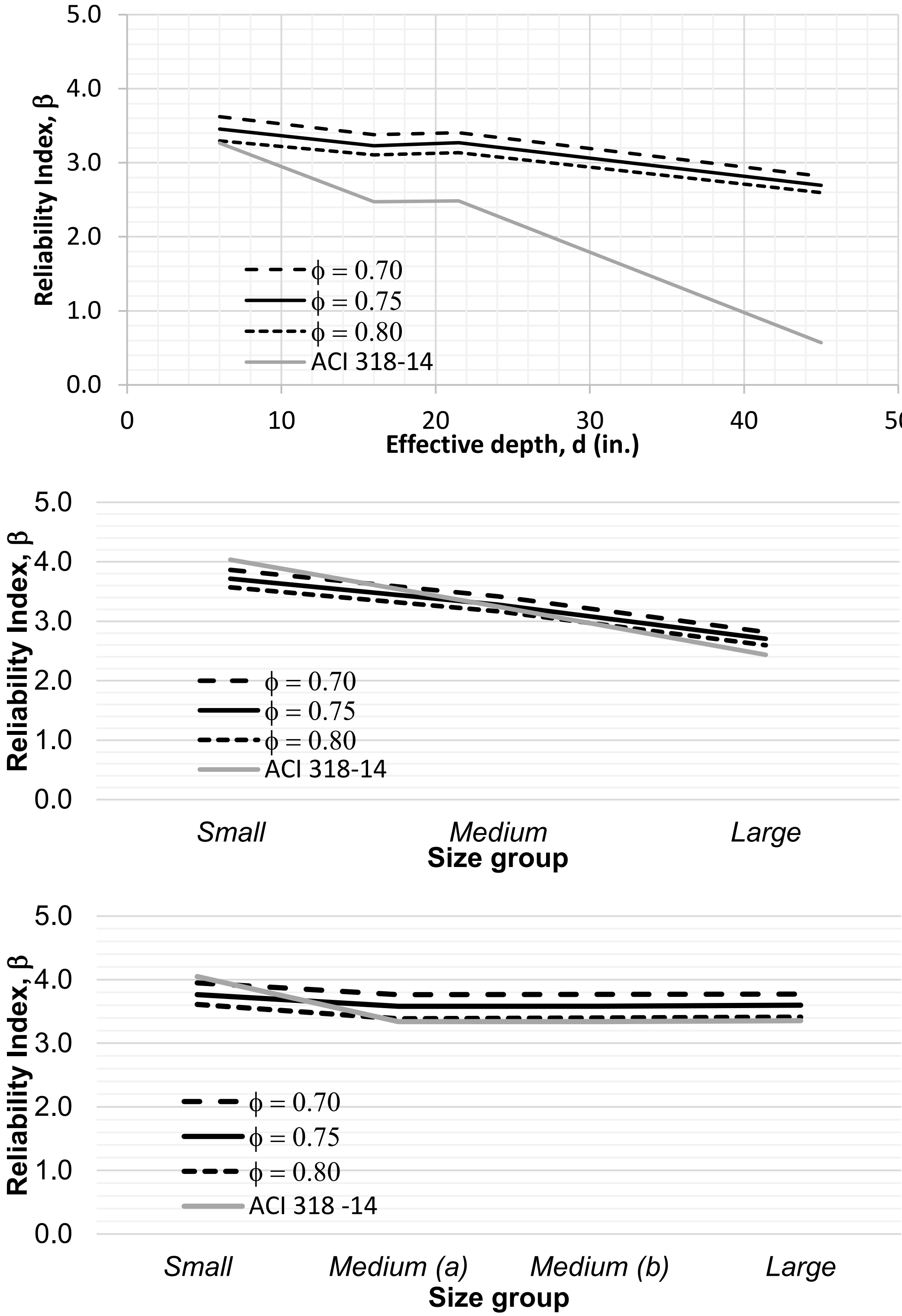


Figure 9-1: Reliability Index Associated Common Members with Different Practical Strength Reduction Factors: (a) Slabs; (b) Lightly Reinforced Beams without Shear Reinforcement; and (c) Lightly Reinforced Beams with Minimum Shear Reinforcement

Figure 9-1a shows that the design provisions in ACI 318-19 are a major improvement relative to ACI 318-14 for one-way shear design of slabs. The strength reduction factor of ϕ = 0.75 used in conjunction with ACI 318-19 provisions leads to a reliability index of β = 3.5 for a typical slab effective depth less than 10 in. and β = 3.3 for less common slabs with effective depths between 10 in. and 33 in. However, for the uncommon cases of larger one-way slabs, the reliability is less than β = 3.0; therefore, a lower strength reduction factor would be necessary to increase β to the target value of 3.0. Because the coefficient of variation for one-way shear strength relative to the ACI 318-19 prediction model (Table 6-5) remains quite large ($V_R \geq 0.20$) for deep members without shear reinforcement, the effectiveness of changing the ϕ-factor for increasing the reliability is quite small. As noted in Section 6.3.3, this large coefficient of variation is at least partly attributable to having a very small set of test data in this size range.

Figure 9-1b shows that the new design provisions in ACI 318-19 are an improvement for one-way shear design of lightly reinforced beams that lack shear reinforcement (and remain below the ACI 318-19 $V_u \leq \phi\sqrt{f_c'}b_w d$ shear demand threshold) when considering the uniformity across the size range considered. With a strength reduction factor of ϕ = 0.75, the new provisions lead to a reliability index of β = 3.7 for small members, which is slightly less than for the ACI 318-14 provisions (β = 4.0), but well above the target value of 3.0. For medium-size members, the reliability associated with designs that comply with the new or the previous provisions is almost identical. However, for the uncommon cases of large beams without shear reinforcement beyond 33 in. effective depth, the reliability is less than β = 3.0 (for both current and previous provisions); therefore, a decreased strength reduction factor would be necessary to increase the computed reliability index to β = 3.0. As in the slab case, because the coefficient of variation for one-way shear strength without shear reinforcement is quite large, the effect of changing the ϕ-factor only ±0.05 is quite small (±0.01–0.02 points in β).

Figure 9-1c shows the reliability of lightly longitudinally reinforced beams with minimum shear reinforcement (applying the professional factor to V_c only), that comply with ACI 318-14 compared with ACI 318-19 for several practical strength reduction factor options. There is an increment in reliability from β = 3.3 to β = 3.6 for medium and large members, while there is a decrement in reliability from β = 4.0 to β = 3.8 for small members. Because the coefficient of variation for one-way shear strength is significant, the effect of changing the ϕ-factor ±0.05 is small (±0.02–0.03 points in β), although it is slightly more sensitive than cases without shear reinforcement.

Based on the target reliability indices noted in Table 9-1, an increase in the one-way shear strength reduction factor to ϕ = 0.80 is justified for (a) members with shear reinforcement and (b) for members of small to medium depth without minimum shear reinforcement. However, the implementation of an increased ϕ factor for large beams and slabs without minimum shear reinforcement remains unjustified. In fact, for large members (d > 33 in.) without shear reinforcement, a smaller strength reduction factor would be required to move the reliability index to a value greater than the target reliability index of 3.0.

9.3 LARGE MEMBERS WITHOUT SHEAR REINFORCEMENT

Although the situation improved with the implementation of the ACI 318-19 one-way shear provisions, large slabs and beams (with d > 33 in.) lacking shear reinforcement remain distinct from members of small and medium sizes. These large members are much less common in practice than small- and medium-size members, but they do have practical applications, such as in large, thick retaining walls. Increasing the strength reduction factor for this class of members beyond the current value of 0.75 remains unwarranted. When analyzed strictly in accordance with the reliability analysis procedure implemented in this study, to obtain the target reliability index $\beta_T = 3.0$ requires $\phi = 0.60$ or less for large slabs or beams without shear reinforcement. However, as noted previously, the large coefficient of variation of the professional factor for both the ACI 318-14 and ACI 318-19 one-way shear provisions for this size range (d > 33 in.) limits the effectiveness of reliability analysis for determining an appropriate strength reduction factor for these large members without minimum shear reinforcement. Until a more accurate model for one-way shear strength of large members is implemented, or more test data are available to refine the professional factors associated with existing provisions, achieving a reliability index of 3.0 will remain elusive without employing a low strength reduction factor for these members.

Chapter 10
SUMMARY AND CONCLUSIONS

A reliability-based calibration of a strength reduction factor for one-way shear strength limit state was not justifiable in the past because of well-founded concerns about the level of safety associated with the ACI 318 traditional one-way shear strength expressions—particularly for large and lightly reinforced beams and slabs—which were first introduced in 1963. After a sustained, collaborative effort of several ACI technical committees (318-E, 445, and 446) to address these safety concerns, improved one-way shear strength expressions were adopted in ACI 318-19. The ACI 318-19 one-way shear design equations are a significant improvement relative to the previous shear design equations in former editions of the ACI 318 building code requirements; however, the reliability of members regarding one-way shear strength limit state remained unknown. Therefore, the objectives of this study were: (1) to provide a statistical basis for improving the strength reduction factor for one-way shear, and (2) to propose a new strength reduction factor for one-way shear, if justified.

In this study, relevant test data available in the literature were collected to characterize uncertainties regarding material mechanical properties and cross-sectional dimensions. Although most parameters were statistically represented in the same manner as in previous ACI 318 code calibration, the in-place concrete strength uncertainty was updated, and the considerations for uncertainty in effective depth were specifically revised for one-way shear strength. A large database of yielding strength of reinforcing bars of sizes commonly used as shear reinforcement was analyzed, and new statistical parameters were developed. In this assessment, data-driven professional factors, which represent the uncertainty in the analytical model, were used instead of expert opinion estimates as in the past. Several scenarios were considered throughout the analyses: small, medium, and large size members; light, moderate, and heavy longitudinal reinforcement; no, light, moderate, and heavy shear reinforcement; and dead to total load ratios from 0 to 1.

Based on the results of this study, the following conclusions are warranted:

1. The design provisions in ACI 318-19 represent a major improvement for one-way shear design of members without shear reinforcement—including accuracy, reliability, and uniformity of reliability across the entire size range.
2. There is also a significant improvement in ACI 318-19 regarding the reliability of those members described in Table 9.6.3.1 as being excepted from the typical threshold for requiring minimum shear

reinforcement, i.e., shallow depth beams ($h \leq 10$ in.), shallow beams integral with slab ($h \leq 24$ in.), and one-way joist systems.

3. To ensure a minimum acceptable safety margin and a safety margin at least as large as in past satisfactory practice, a target reliability index of 3.0 is appropriate for evaluating appropriate strength reduction factors to be used in conjunction with the ACI 318-19 one-way shear provisions.

4. The target reliability index of 3.0 is consistent with an increase in the strength reduction factor for one-way shear strength to $\phi = 0.80$ for members with shear reinforcement.

5. The target reliability index of 3.0 is consistent with an increase in the strength reduction factor for one-way shear strength to $\phi = 0.80$ for members without shear reinforcement where the effective depth, d, does not exceed 33 in.

6. The strength reduction factor for one-way shear strength should not be increased beyond the current value of $\phi = 0.75$ for large slabs or beams ($d > 33$ in.) without minimum shear reinforcement.

7. Despite the improvement in accuracy for one-way shear design of large members without shear reinforcement, the reliability index for these members remains less than the desirable target value, and designers should be aware of this. More one-way shear tests of very large beams and slabs are needed to improve the statistical variability or the analytical model or both.

REFERENCES

ACI Committee 117. 2019. *Specification for Tolerances for Concrete Construction and Materials (ACI 117-10) and Commentary*. Farmington Hills, Michigan: American Concrete Institute (ACI).

ACI Committee 318. 2019. *Building Code Requirements for Structural Concrete (ACI 318-19)*. Farmington Hills, Michigan: American Concrete Institute (ACI).

ACI Committee 318. 2014. *Building Code Requirements for Structural Concrete (ACI 318-14)*. Farmington Hills, Michigan: American Concrete Institute (ACI).

Aguilar Vidal, V.H. 2020. "Reliability of Shear-Critical Reinforced Concrete Members." Ph.D. Dissertation, Auburn University, Auburn, Alabama.

Aguilar, V., Barnes, R. W., and Nowak, A. S. 2023. "Data-Driven Professional Factors for ACI 318 One-Way Shear Strength," *Structure and Infrastructure Engineering*, V. 19, No. 9, 1205–1217.

Allen, D.E. 1970. "Probabilistic Study of Reinforced Concrete in Bending," *ACI Journal, Proceedings*, V. 67, No. 12, December 1970, 989–993.

ASTM A615. 2022. ASTM A615/A615M-22— Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement. West Conshohocken, PA: ASTM International.

ASTM A706. 2022. ASTM A706/A706M-22a— Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement. West Conshohocken, PA: ASTM International.

ASTM C873. 2011. ASTM C873/C873M-11—Standard Test Method for Compressive Strength of Concrete Cylinders Cast in Place in Cylindrical Molds. West Conshohocken, PA: ASTM International.

Bartlett, F.M., and MacGregor, J.G. 1996. "Statistical Analysis of the Compressive Strength of Concrete in Structures," *ACI Materials Journal*, V. 93, No. 2, 158–68.

Bartlett, F.M., and MacGregor, J.G. 1999. "Variation of In-Place Concrete Strength in Structures," *ACI Materials Journal*, V. 96, No. 2, 261–70.

Benjamin, J.R., and Cornell, C.A. 1970. *Probability, Statistics, and Decision for Civil Engineers*. New York: McGraw-Hill.

Bentz, E.C., and Collins, M.P. 2018. "The Toronto Size Effect Series," *Shear in Structural Concrete*, SP-328, Mitchell, D., and Belarbi, A. ed., American Concrete Institute, Farmington Hills, MI, 2.1–2.12.

Byard, B.E. 2011. "Early-Age Behavior of Lightweight Aggregate Concrete." Ph.D. Dissertation, Auburn University, Auburn, Alabama.

Byard, B.E., Schindler, A.K., and Barnes, R.W. 2012. "Early-age cracking tendency and ultimate degree of hydration of internally cured concrete," *Journal of Materials in Civil Engineering*, V. 24, No. 8, 1025–33.

Concrete Construction. 1979. "Tolerances in Concrete Construction," *Concrete Construction.*

CRSI. 2019. *Steel Mill Property Database, Years 2010-2019*. Schaumburg, Illinois: Concrete Reinforcing Steel Institute (CRSI).

Ellingwood, B., Galambos, T.V., MacGregor, J.G., and Cornell, C.A. 1980. *Development of a Probability-Based Load Criterion for American National Standard A58*. NBS Special Publication 577. Washington, DC: National Bureau of Standards (NBS).

Ellingwood, B. 1978. “Reliability Basis of Load and Resistance Factors for Reinforced Concrete Design,” *NBS Building Science Series* 110. Washington, DC: National Bureau of Standards.

Ergun, A., and Kurklu, G. 2012. “Assessing the Relationship between the Compressive Strength of Concrete Cores and Molded Specimens,” *Gazi University Journal of Science*, V. 25, No. 3, 737–50.

Greene, G., and Graybeal, B.A. 2013. “Lightweight Concrete: Mechanical Properties,” FHWA-HRT-13-061. McLean, Virginia: Federal Highway Administration (FHWA).

Grubbs, A.R., Carroll, A.C., Schindler, A.K., and Barnes, R.W. 2016. “Evaluation of In-Place Concrete Strength by Core Testing,” Research Report No. 2 for ALDOT Project 930-828. Auburn, AL: Auburn University Highway Research Center.

Hart, G.C. 1982. *Uncertainty Analysis, Loads, and Safety in Structural Engineering*. Englewood Cliffs, New Jersey: Prentice-Hall.

Kuchma, D.A., Wei, S., Sanders, D.H., Belarbi, A., and Nowak, L.C. 2019. “Development of the One-Way Shear Design Provisions of ACI 318-19 for Reinforced Concrete,” *ACI Structural Journal*, V. 116, No. 4, 285–295.

Legeron, F., and Paultre, P. 2000. “Prediction of Modulus of Rupture of Concrete,” *ACI Materials Journal*, V. 97, No. 2, 193–200.

Nowak, A.S. 1999. “Calibration of LRFD Bridge Design Code,” NCHRP Report 368, Transportation Research Board. Washington, DC: National Academy Press.

Nowak, A.S., and Collins, K.R. 2013. *Reliability of Structures*. Boca Raton, Florida: CRC Press.

Nowak, A.S., and Rakoczy, A.M. 2011. “Statistical Model for Compressive Strength of Lightweight Concrete,” *Architecture Civil Engineering Environment*, V. 4, No. 4, 73–80.

Nowak, A.S., and Szerszen, M.M. 2003. “Calibration of Design Code for Buildings (ACI 318): Part 1—Statistical Models for Resistance,” *ACI Structural Journal*, V. 100, No. 3, 377–382.

Nowak, A.S., Szerszen, M.M., Szeliga, E.K., Szwed, A., and Podhorecki, P.J. 2005. *Reliability-based Calibration for Structural Concrete*. UNLCE 05-3. Lincoln, Nebraska: University of Nebraska.

Reineck, K.-H., Bentz, E., Fitik, B., Kuchma, D.A., and Bayrak, O. 2014. “ACI-DAfStb Databases for Shear Tests on Slender Reinforced Concrete Beams with Stirrups,” *ACI Structural Journal*, V. 111, No. 5, 1147–1156.

Reineck, K.-H., Kuchma, D. A., Kim, K. S., and Marx, S. 2003. “Shear Database for Reinforced Concrete Members without Shear Reinforcement,” *ACI Structural Journal*, V. 100, No. 2, 240–249.

Rakoczy, A. M., and Nowak, A. S. 2013. “Resistance Model of Lightweight Concrete Members,” *ACI Materials Journal*, V. 110, No. 1, 99–108.

Somo, S., and Hong, H.P. 2006. “Modeling Error Analysis of Shear Predicting Models for RC Beams,” *Structural Safety*, V. 28, No. 3, 217–30.

Szerszen, M.M., and Nowak, A.S. 2003. “Calibration of Design Code for Buildings (ACI 318): Part 2—Reliability Analysis and Resistance Factors,” *ACI Structural Journal*, V. 100, No. 3, 383–391.

Tankasala, A. 2017. “Effect of lightweight aggregate on early-age cracking of mass concrete.” Ph.D. Dissertation, Auburn University, Auburn, Alabama.

Tankasala, A., and Schindler, A.K. 2020. “Early-Age Cracking of Lightweight Mass Concrete.,” *ACI Materials Journal*, V. 117, No. 1, 223–232.