

Guide for the Use of ASTM A1035/A1035M Type CS Grade 100 (690) Steel Bars for Structural Concrete

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Guide for the Use of ASTM A1035/A1035M Type CS Grade 100 (690) Steel Bars for Structural Concrete

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*This guide provides recommendations on design provisions for the use of **ASTM A1035/ASTM A1035M** Type CS Grade 100 (690) deformed steel bars for reinforced concrete members. The recommendations address only those requirements of **ACI 318-14** that limit efficient use of such steel bars. Other code requirements are not affected. Any other ACI 318 versions will be explicitly specified. Although there are limiting ACI 318 requirements, ACI 318-14 Section 1.10 would allow the use of high-strength reinforcement. “Sponsors...shall have the right to present the data on which their design is based to the building official or to a board of examiners appointed by the building official.”*

*The International Building Code (IBC 2012) would allow the same under Section 104.11, “Alternative materials, design and methods of construction and equipment”. To approve an alternative material under this section, a building department would typically require an ICC Evaluation Service (ICC-ES) Evaluation Report, which would be based on an ICC-ES Acceptance Criteria (AC) document (**ICC-ES AC429**) and an Evalua-*

*tion Report (**ICC-ES ESR-2107**) exist, permitting the use of ASTM A1035/A1035M Grade 100 reinforcement.*

This guide includes a discussion of the material characteristics of Grade 100 (690) ASTM A1035/A1035M (CS) deformed steel bars and recommends design criteria for beams, columns, slab, systems, walls, and footings for Seismic Design Category (SDC) A, B, or C, and for structural components not designated as part of the seismic-force-resisting system for SDC D, E, or F.

A structure assigned to SDC A, B, or C is required to be designed for all applicable gravity and environmental loads. In the case of SDC A structures, seismic forces are notional structural integrity forces. This guide addresses all design required for SDC A, B, and C structures.

*Because the modulus of elasticity for ASTM A1035/A1035M (CS) is similar to that of carbon steel (**ASTM A615/A615M**) using higher specified minimum yield strength f_y may result in higher steel stress at service load condition and potentially cause wider cracks and larger deflections, which may be objectionable if aesthetics and water-tightness are critical design requirements. Higher deflection can also contribute to serviceability issues. Also, with higher f_y , the required development length will be longer.*

Keywords: bar; design; guide; high-strength steel; structural.

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CHAPTER 1—INTRODUCTION

1.1—Objective

This guide is based on **ACI ITG-6R-10**, “Design Guide for the Use of **ASTM A1035/A1035M** Grade 100 (690) Steel Bars for Structural Concrete,” reported by ACI Innovation Task Group 6. The ACI ITG-6R guide provides design provisions for the use of ASTM A1035/A1035M Type CS Grade 100 (690) deformed steel bars in reinforced structural members. This guide, which is based on ACI ITG-6R-10, is a stand-alone document, references and addresses only those requirements in ACI 318-14 that limit the use of such steel bars, and should not affect the application of other code requirements. Any other **ACI 318** version will be explicitly specified. This guide includes a discussion of the material characteristics of Grade 100 (690) ASTM A1035/A1035M Type CS (Chromium content 8.0 to 10.9 percent) deformed steel bars, and the design provisions are based on the specific material properties and stress-strain behavior of these bars.

Although there are limiting ACI 318 requirements, ACI 318-14 Section 1.10 would allow the use of high-strength reinforcement. “Sponsors...shall have the right to present the data on which their design is based to the building official or to a board of examiners appointed by the building official.”

The IBC International Building Code (IBC 2012) would allow the same under Section 104.11, “Alternative materials, design and methods of construction and equipment”. To approve an alternative material under this section, a building department would typically require an ICC Evaluation Service (ICC-ES) Evaluation Report that would be based on an ICC-ES Acceptance Criteria (AC) document. An AC document (**ICC-ES AC429**) and an Evaluation Report (**ICC-ES ESR-2107**) exist, permitting the use of ASTM A1035/A1035M Grade 100 reinforcement.

Since publication of ACI ITG-6R, additional Grade 100 (690) ASTM A1035/A1035M Types CM (Chromium content 4.0 percent to 7.9 percent) and CL (chromium content 2.0 percent to 3.9) were introduced to the market. The mechanical properties and stress-strain characteristics of these steels may affect the design criteria herein and limit the applicability of the guide. Additionally, ASTM A615/A615M Grade 100 (690) was introduced to the market. These bars will be addressed in future editions when research on concrete members reinforced with these bar types become available and the impact of the mechanical properties have been documented. Grade 120 (830) ASTM A1035/A1035M Type CS, CM and CL are available, but are not addressed under this guide.

1.2—Scope

This guide presents the material characteristics of ASTM A1035/A1035M (CS) steel bars and recommends design criteria for beams, columns, slab systems, walls, and footings for Seismic Design Category (SDC) A, B, or C. A structure assigned to SDC A, B or C is required to be designed for all applicable gravity and environmental loads. In the case of SDC A structures, seismic forces are notional structural integrity forces. This guide addresses all design required for SDC A, B, and C structures. Due to lack of adequate data,

the application of this guide for SDC D, E, or F is limited to slab systems, foundations, and structural components not designated as part of the seismic-force-resisting system, but explicitly checked for the induced effects of the design displacements. The only exception is the use of transverse reinforcement for concrete confinement with a specified minimum yield strength f_y up to 100,000 psi (690 MPa) in special moment frames, special structural walls, and all components of special structural walls including coupling beams and wall piers as permitted by ACI 318. Refer to 10.1 of this guide for more information on seismic design considerations. Shells, folded plate members, and prestressed concrete are beyond the scope of this guide. However, ASTM A1035/A1035M Type CS can be used as reinforcement in prestressed concrete but not as the prestressing strands. Design examples are included to illustrate design procedures and proper application of the design criteria. Modifications to these design criteria may be justified where the design adequacy within the scope of this guide is demonstrated by successful use, analysis, or test.

1.3—Historical perspective and background

For several decades prior to **ACI 318-71**, the design of structural concrete was restricted to using specified minimum yield strength f_y of 60,000 psi (420 MPa) or less for reinforcing bars. Section A603(e) of ACI 318-56 specified that “Stress in tensile and compressive reinforcement at ultimate load shall not be assumed greater than the yield point or 60,000 psi, whichever is smaller.”

Section 1505 of **ACI 318-63**, specified two requirements:

“(a) When reinforcement is used that has a yield strength, f_y , in excess of 60,000 psi (420 MPa), the yield strength to be used in design shall be reduced to $0.85f_y$ or 60,000 psi (420 MPa), whichever is greater, unless it is shown by tension tests that at a proof stress equal to the specified minimum yield strength, f_y , the strain does not exceed 0.003;

(b) Designs shall not be based on a yield strength, f_y , in excess of 75,000 psi (520 MPa). Design of tension reinforcement shall not be based on a yield strength, f_y , in excess of 60,000 psi (420 MPa) unless tests are made in compliance with Section 1508(b).”

The Commentary on Section 1505 of ACI 318-63 states that:

This section provides limitations on the use of high strength steels to assure safety and satisfactory performance. High strength steels frequently have a strain at yield strength or yield point in excess of the 0.003 assumed for the concrete at ultimate. The requirements of Section 1505(a) are to adjust to this condition.

The maximum stress in tension of 60,000 psi (420 MPa) without test is to control cracking. The absolute maximum is specified as 75,000 psi (520 MPa) to agree with present ASTM specifications and as a safeguard until there is adequate experience with the high stresses.

Then, the Commentary on Section 1508 of ACI 318-63 states that:

When the design yield point of tension reinforcement exceeds 60,000 psi (420 MPa), detailing for crack control becomes even more important. Entirely acceptable structures have been built, particularly in Sweden, with a design yield strength approaching 100,000 psi (690 MPa) but more design criteria for crack control and considerable American practical experience with 60,000 psi (420 MPa) yield strength tension reinforcement are needed before higher yield strengths are approved for general use. The Code, therefore limits *tension* reinforcement to 60,000 psi (420 MPa) yield strength, unless special full-scale tests are made. It was thought that 75,000 psi (520 MPa) yield strength tension reinforcement should be permitted where full-scale testing is economically feasible, such as in precast members. The crack width criteria are not too difficult to meet by proper attention to reinforcing details.

When the use of Grade 40 (280) reinforcing bars in the 1930s and 1940s was replaced by the use of Grade 60 (420) bars in the 1950s and 1960s, there were concerns about fatigue resistance of the higher-strength steel bars. Similar concerns were expressed about the use of **ASTM A1035/A1035M** (CS) steel bars when they were introduced. **El-Hacha and Rizkalla (2002)** and **DeJong et al. (2006)** conducted studies on the fatigue behavior of ASTM A1035/A1035M (CS) steel bars. Their results indicated that a fatigue life of 1×10^6 cycles was observed at a stress range of approximately 44,000 psi (310 MPa) for ASTM A1035/A1035M (CS) steel bars. The ASTM A1035/A1035M (CS) steel bars showed comparable fatigue resistance to Grade 60 (420) reinforcing bars even though their stress at service would be higher than that of Grade 60 (420) bars.

1.4—Reinforcing steel grades availability

The most widely used deformed reinforcing bars conform to **ASTM A615/A615M**, which include Grade 40 (280), Grade 60 (420), Grade 75 (520), and Grade 80 (550). The Grade 60 (420) reinforcement exhibits minimum yield strength of 60,000 psi (420 MPa) with a distinct yield plateau. ACI 318 permits use of reinforcing bars with a specified minimum yield strength f_y exceeding 60,000 psi (420 MPa), but f_y is limited to the lesser of 80,000 psi (550 MPa) or the stress corresponding to a strain of 0.0035, except as follows. **ACI 318** limits the specified minimum yield strength for deformed bars used as shear reinforcement to 60,000 psi (420 MPa). For deformed bars used as confinement reinforcement (ties or spirals) in compression members, ACI 318 permits the use of specified minimum yield strength of up to 100,000 psi (690 MPa).

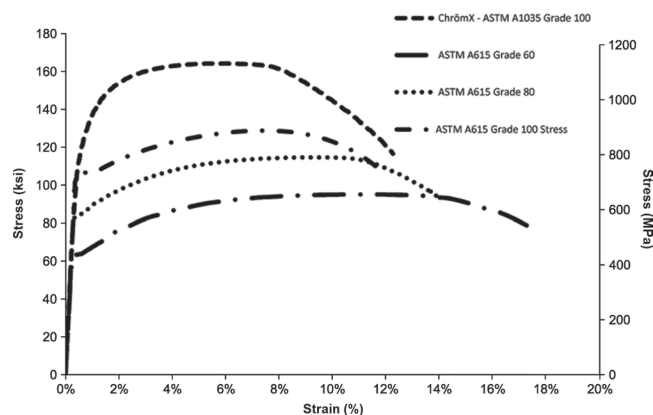


Fig. 1.5—Comparison of typical stress-strain curves for ASTM A615/A615M and ASTM A1035/A1035M (CS) reinforcing bars.

1.5—Introduction of ASTM A1035/A1035M Type CS Grade 100

The introduction of higher-strength steel reinforcing bars with a specified minimum yield strength, $f_y = 100$, the designers might be able to reduce the total cross-sectional area of required reinforcement. The reduced area of reinforcement could result in fewer bars and reduce reinforcement congestion often encountered in mat foundations, shear walls, beam-column joints, and many precast concrete elements. The reduction in reinforcement congestion facilitates concrete placement and consolidation.

The ASTM A1035/A1035M (CS) bar Grade 100 (690) (Fig. 1.5) exhibits a linear stress-strain relationship up to a proportional limit ranging from 60,000 to 80,000 psi (420 to 550 MPa), without a well-defined yield plateau. Refer to **Appendix C** of this guide for a discussion on how the lack of a well-defined yield plateau affects the flexural behavior of beams. Actual yield strength, determined by the 0.2 percent offset method, typically exceeds 115,000 psi (790 MPa) for Grade 100 (690) bars. The tensile strength typically exceeds 155,000 psi (1070 MPa) for Grade 100 (690) bars. The corresponding strain at the peak of the stress-strain curve ranges from 0.04 to 0.06. Refer to Chapter 3 of this guide for more details on the material characteristics of ASTM A1035/A1035M (CS) reinforcing bars.

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

- A_s = area of nonprestressed longitudinal tension reinforcement, in.² (mm²)
- A_s' = area of nonprestressed longitudinal compression reinforcement, in.² (mm²)
- A_{tr} = total cross-sectional area of all transverse reinforcement that is within the spacing s and crosses the potential plane of splitting through the reinforcement being developed or lap spliced, in.² (mm²); refer to Eq. (10.2b)
- a = depth of equivalent rectangular stress block as defined in ACI 318, in. (mm)

b	= width of compression face of member, in. (mm)	R_r	= relative rib area of the reinforcement = ratio of projected rib area normal to bar axis to the product of the nominal bar diameter and the center-to-center rib spacing, may be taken conservatively as 0.07 for design
b_w	= web width, in. (mm)	s	= spacing of transverse (shear) reinforcement, in. (mm)
C	= force in the compression zone of a beam, lbf (N)	T	= force in tension reinforcement of a beam, lbf (N)
c	= distance from extreme compression fiber to neutral axis, in. (mm)	T_s	= additional bond strength provided by the transverse steel, lbf (N)
c_b	= $c_{min} + 0.5d_b$, in. (mm); refer to Eq. (10.2a)	t_d	= bar diameter factor = $0.78d_b + 0.22$ in. (mm); refer to Eq. (10.2d)
c_{bb}	= clear cover of reinforcement being developed or lap spliced, measured to tension face of member, in. (mm)	t_r	= term representing the effect of relative rib area on T_s = $9.6R_r + 0.28 \leq 1.72$; refer to Eq. (10.2c)
c_c	= clear cover to tension steel, in. (mm)	V	= applied shear at critical section, lbf (N)
c_{max}	= maximum value of c_s or c_{bb} , in. (mm)	V_c	= nominal shear strength provided by concrete, lbf (N)
c_{min}	= minimum value of c_s or c_{bb} , in. (mm)	w_u	= factored load per unit length of beam or one-way slab, lb/ft ³ (kg/m ³)
c_s	= minimum value of $c_{si} + 0.25$ in. (6.35 mm) or c_{so} , in. (mm); c_{si} may be used instead of $c_{si} + 0.25$ in. (6.35 mm)	α	= reinforcement location factor
c_{si}	= one-half of average clear spacing between bars or lap splices in a single layer, in. (mm)	β	= ratio of the distance from the neutral axis to the extreme tension face to the distance from the neutral axis to the center of the tension reinforcement factor relating depth of equivalent rectangular compressive stress block to neutral axis depth, as defined in ACI 318
c_{so}	= clear cover of reinforcement being developed or lap spliced, measured to side face of member, in. (mm)	β_c	= coating factor; refer to Eq. (10.2a)
d	= distance from extreme compression fiber to centroid of longitudinal tension reinforcement, in. (mm)	Δ	= deflection, in. (mm)
d'	= distance from extreme compression fiber to centroid of longitudinal compression reinforcement, in. (mm)	ϵ_s	= strain in reinforcement
d_b	= bar diameter, in. (mm)	ϵ_r	= net tensile strain in extreme layer of longitudinal tension steel at nominal strength, excluding strains due to creep, shrinkage, and temperature
d_t	= distance from extreme compression fiber to centroid of extreme layer of longitudinal tension reinforcement, in. (mm)	ϕ	= strength reduction factor
E_c	= modulus of elasticity of concrete, psi (MPa)	λ	= lightweight aggregate concrete factor
E_s	= modulus of elasticity of reinforcement, psi (MPa)	ρ_b	= bias factor = ratio of the average actual value to the specified minimum value of a property being analyzed
f'_c	= specified compressive strength of concrete, psi (MPa)	ρ	= reinforcement ratio = A_s/bd
f_s	= calculated tensile stress in reinforcement, psi (MPa)	ρ_b	= reinforcement ratio producing balanced strain conditions as defined in ACI 318
f_u	= specified ultimate strength of reinforcement, psi (MPa)	ρ_s	= volumetric spiral reinforcement ratio
f_y	= specified minimum yield strength of reinforcement, psi (MPa)	ψ	= curvature
f_{yt}	= specified minimum yield strength of transverse reinforcement, psi (MPa)	ω	= factor reflecting benefit of large cover/spacing perpendicular to controlling cover/spacing = $0.1(c_{max}/c_{min}) + 0.9 \leq 1.25$; refer to Eq. (10.2a)
h	= overall height or thickness of member, in. (mm)		
h_f	= flange thickness of T-beam, in. (mm)		
I_{cr}	= moment of inertia of cracked section transformed to concrete, in. ⁴ (mm ⁴)		
I_e	= effective moment of inertia for computation of deflection, in. ⁴ (mm ⁴)		
I_g	= moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, in. ⁴ (mm ⁴)		
K_{tr}	= transverse reinforcement index; refer to Eq. (10.2b)		
ℓ_d	= development length (also splice length), in. (mm)		
M	= applied moment at critical section, in.-lbf (N·mm)		
M_a	= maximum moment in member due to service loads at stage deflection is computed, in.-lbf (N·mm)		
M_{cr}	= cracking moment, in.-lbf (N·mm)		
M_n	= nominal flexural strength at section, in.-lbf (N·mm)		
n	= number of bars being developed or lap spliced along plane of splitting; refer to Eq. (10.2b)		
P_n	= nominal axial strength of cross section, lbf (N)		

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, ACI Concrete Terminology. Definitions provided herein complement that resource.

0.2% offset method—method for determining a yield strength value for a material that does not exhibit a distinct yield plateau. The yield strength is the stress on the engineering stress-strain curve at its intersection with a line having a slope equal to the initial modulus of elasticity and offset from the linear elastic portion of the engineering stress-strain curve by a strain of 0.2%.



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bias factor—ratio of the average actual value to the specified minimum value of a property being considered in risk analysis.

coefficient of variation (COV)—the standard deviation divided by the mean value of a variable.

cumulative distribution function (CDF)—a function or graph describing the probability distribution of a real-valued random variable taking on a value less than or equal to a particular value.

Seismic Design Category (SDC)—classification assigned to a structure based on its occupancy category, and the severity of the design earthquake ground motion. The category assignment can range from A to F.

CHAPTER 3—MATERIAL PROPERTIES

3.1—Introduction

This guide addresses high-strength deformed reinforcing bars of Type CS as defined by **ASTM A1035/A1035M**, “Standard Specification for Deformed and Plain, Low-Carbon, Chromium, Steel Bars for Concrete Reinforcement.” As with other specifications for steel reinforcing bars, this standard includes requirements for nominal weights and dimensions, tensile properties, chemical composition, and deformations. This chapter reviews properties of primary interest to the structural designer who may specify the use of ASTM A1035/A1035M bars.

Since the publication of **ACI ITG-6R**, additional Grade 100 (690) reinforcing steels were introduced to the market, including ASTM A1035/A1035M Types CM and CL and **ASTM A615/A615M** Grade 100 (690). The mechanical properties and stress-strain characteristics of these steels may impact the applicability of this design guide. These bars will be addressed in future editions when research on concrete members reinforced with these bar types become available and the impact of the mechanical properties have been documented.

3.2—Weights, dimensions, and deformations

ASTM A1035/A1035M (CS) specifies nominal weights (mass), areas, and dimensions. Deformation requirements are the same as those specified by ASTM A615/A615M for carbon-steel bars and **ASTM A706/A706M** for low-alloy steel bars. Consequently, this section does not discuss bar deformations and related aspects.

3.3—Specified tensile properties

Tensile properties are paramount for structural design. Tables 3.3a and 3.3b contain a summary of the specified tensile properties for ASTM A1035/A1035M (CS) and other reinforcement (**ACI 439.4R**). Table 3.3a shows that the tensile strength properties for ASTM A1035/A1035M (CS) bars are significantly greater than those for ASTM A615/A615M Grade 60 (420), Grade 80 (550), and Grade 100 (690) bars. The requirements for elongation in 8 in. (200 mm) across the fracture, as shown in Table 3.3b, are comparable to those for ASTM A615/A615M Grade 80 (550) and Grade 100 (690) reinforcement, and in some cases lower

Table 3.3a—Specified tensile and yield strength

Bar type	Tensile strength, minimum, psi (MPa)	Yield strength*	
		Minimum, psi (MPa)	Maximum, psi (MPa)
ASTM A615/A615M Grade 60 (420)	90,000 (620)	60,000 (420)	—
ASTM A615/A615M Grade 80 (550)	105,000 (725)	80,000 (550)	—
ASTM A615/A615M Grade 100 (690)	115,000 (790) [†]	100,000 (690)	—
ASTM A706/A706M Grade 60 (520)	80,000 [‡] (550) [‡]	60,000 (420)	78,000 (540)
ASTM A706/A706M Grade 80 (520)	100,000 [‡] (690) [‡]	80,000 (550)	98,000 (675)
ASTM A1035/A1035M (CS, CM, and CL) Grade 100 (690)	150,000 (1030)	100,000 (690)	—

*Observed yield point for ASTM A615/A615M and ASTM A706/A706M bars, and yield strength according to 0.2 percent offset method for ASTM A1035/A1035M bars, which is applicable to ASTM A615/A615M and ASTM A706/A706M bars only when steel bar tested does not exhibit a well-defined yield point.

[†]Grade 100 (690) reinforcing bars have a ratio of specified tensile strength to specified yield strength of 1.15. Designers should be aware that there will, therefore, be a lower margin of safety and reduced warning of failure following yielding when Grade 100 (690) bars are used in structural members.

[‡]Tensile strength for ASTM A706/A706M bars should also be not less than 1.25 times measured yield strength.

Table 3.3b—Specified elongation in 8 in. (200 mm) across fracture

Bar type	Bar size no.			
	3, 4, 5, 6 (10, 13, 16, 19)	7, 8 (22, 25)	9, 10, 11 (29, 32, 36)	14, 18 (43, 57)
	Elongation in 8 in. (200 mm) across fracture, minimum, percent			
ASTM A615/A615M Grade 60 (420)	9	8	7	7
ASTM A615/A615M Grade 80 (550), 100 (690)	7	7	6	6
ASTM A706/A706M Grade 60 (420)	14	12	12	10
ASTM A706/A706M Grade 80 (550)	12	12	12	10
ASTM A1035/A1035M (CS, CM, and CL) Grade 100 (690)	7	7	7	6

than those for ASTM A615/A615M Grade 60 (420) reinforcement. As described in 1.3 of this guide, ASTM A1035/A1035M (CS) reinforcing steel does not exhibit a distinct yield plateau (Fig. 1.5). Consequently, ASTM A1035/A1035M (CS) specifies minimum yield strength according to the 0.2 percent offset method.

3.4—Measured tensile properties

Understanding the differences in tensile behavior of higher strength steels is essential for the safe and serviceable design

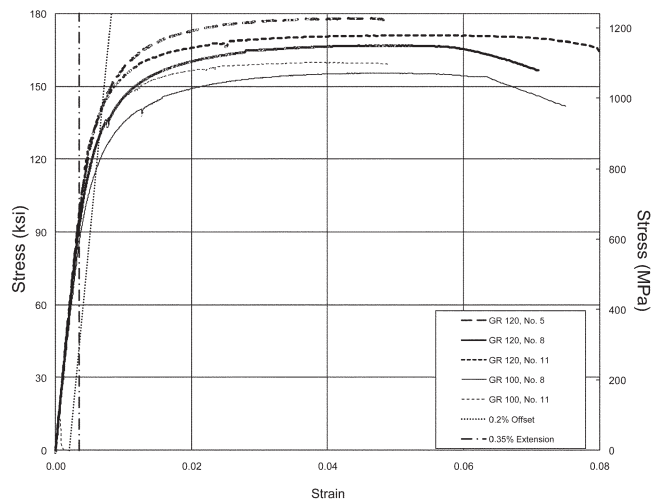


Fig. 3.4a—Actual stress-strain curves for ASTM A1035/A1035M Type CS reinforcing bars of different grades and sizes (WJE 2008).

of concrete members. Tensile properties are necessary in the probabilistic studies that establish the strength reduction factors used in reinforced concrete design.

Figure 1.5 shows comparisons of stress-strain curves recorded for samples of **ASTM A1035/A1035M** (CS) Grade 100 (690) bars to similar curves for samples of **ASTM A615/A615M** bars in Grades 60 (420) and 80 (550). ASTM A1035/A1035M (CS) bars have a greater tensile strength and lack a well-defined yield point and yield plateau. ASTM A1035/A1035M (CS) bars reach a proportional limit at a stress from 60,000 to 80,000 psi (420 to 550 MPa), which is similar to the yield stress of ASTM A615/A615M Grade 60 (420) and **ASTM A706/A706M** Grade 60 (420) bars (WJE 2008) and the strain at the peak tensile stress in the bar ranges from 0.04 to 0.06. By comparison, strains at peak tensile stress for ASTM A615/A615M Grade 60 (420) bars range from 0.07 to 0.10, and those of ASTM A706/A706M Grade 60 (420) bars range from 0.10 to 0.14.

For ASTM A1035/A1035M (CS) bars, the elongation in 8 in. (200 mm) across the fracture ranges from 0.08 to 0.13, whereas the elongation in 8 in. (200 mm) across the fracture for ASTM A615/A615M Grade 60 (420) and ASTM A706/A706M Grade 60 (420) bars range from 0.09 to 0.12 and 0.14 to 0.20, respectively. The modulus of elasticity value of 29,000,000 psi (200,000 MPa) as defined in ACI 318 is applicable to define the elastic modulus for ASTM A1035/A1035M (CS) Grade 100 (690) bars.

Figure 3.4a shows actual stress-strain curves recorded for samples of ASTM A1035/A1035M reinforcing bars (WJE 2008). Yield strength of ASTM A1035/A1035M (CS) bars, determined by the 0.2 percent offset method, exceeds 115,000 psi (790 MPa). The tensile strength for ASTM A1035/A1035M (CS) Grade 100 (690) bar exceeds 155,000 psi (1070 MPa).

An approximate lower bound for the stress-strain curves of Grade 100 (690) bars can be represented by the following three equations, as shown in Fig. 3.4b.

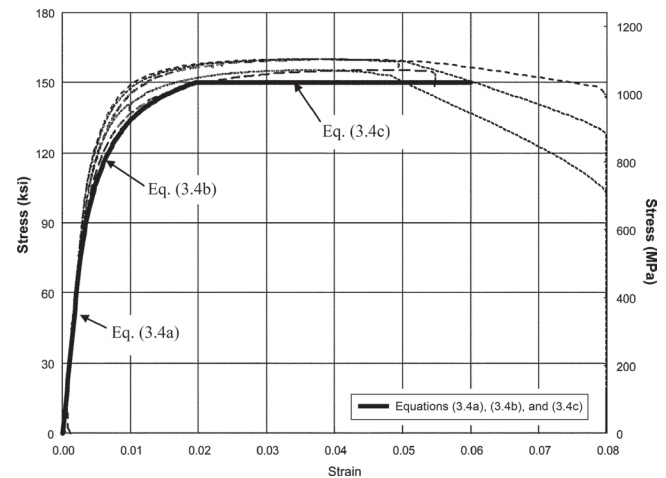


Fig. 3.4b—Equations (3.4a), (3.4b), and (3.4c) compared with actual stress-strain curves from samples of No. 8 (25) and No. 11 (36) bars of ASTM A1035/A1035M Grade 100 (690) Type CS.

Equations (3.4a) through (3.4c) represent a lower bound to the stress-strain behavior of ASTM A1035/A1035M (CS) Grade 100 (690) bar. The equations are based on an assumed proportional limit of 70,000 psi (480 MPa) and an assumed tensile strength of 150,000 psi (1030 MPa), which is reached at the strain of 0.02, the upper limit for Eq. (3.4b).

$$f_s = 29,000\varepsilon_s \text{ (ksi) for } \varepsilon_s \leq 0.0024 \text{ (in.-lb)} \quad (3.4a)$$

$$f_s = 200,000\varepsilon_s \text{ (MPa) for } \varepsilon_s \leq 0.0024 \text{ (SI)}$$

$$f_s = 170 - \frac{0.4317}{\varepsilon_s + 0.0019} \text{ (ksi) for } 0.0024 < \varepsilon_s \leq 0.02 \text{ (in.-lb)}$$

$$f_s = 1170 - \frac{2.9670}{\varepsilon_s + 0.0019} \text{ (MPa) for } 0.0024 < \varepsilon_s \leq 0.02 \text{ (SI)} \quad (3.4b)$$

$$f_s = 150 \text{ (ksi) for } 0.02 < \varepsilon_s \leq 0.06 \text{ (in.-lb)} \quad (3.4c)$$

$$f_s = 1040 \text{ (MPa) for } 0.02 < \varepsilon_s \leq 0.06 \text{ (SI)}$$

Accordingly, an analysis was performed on the results of 137 mill tests on ASTM A1035/A1035M (CS) Grade 100 (690) bars produced after 2004 that were provided by a manufacturer. Based on the analysis of the mill test data, it was shown that Eq. (3.4b) and (3.4c) would provide a lower tolerance limit on actual stress corresponding to a strain of 0.0035, actual yield strength, and actual tensile strength for ASTM A1035/A1035M (CS) Grade 100 (690) bars such that at least 95 percent of the data are greater than the corresponding values calculated by these equations with a confidence level of 90 percent.

Cumulative distribution functions (CDFs) for the yield strength by the 0.2 percent offset method and the tensile strength were developed for **ASTM A1035/A1035M** (CS) Grade 100 (690) bars.

Table 3.4—Statistical analysis of mill test data for ASTM A1035/A1035M (CS) Grade 100 (690) steel reinforcing bars*

Bar designation	No. of samples	0.2% offset yield strength			Tensile strength	
		Mean, ksi (MPa)	Bias factor λ_b	COV	Mean, ksi (MPa)	COV
No. 3 (10)	10	139.8 (964)		0.044	174.3 (1202)	0.046
No. 4 (13)	20	129.9 (895)		0.026	162.1 (1118)	0.027
No. 5 (16)	28	130.9 (903)		0.036	164.7 (1135)	0.035
No. 6 (19)	16	129.9 (895)		0.067	164.1 (1132)	0.045
No. 7 (22)	15	124.2 (857)		0.061	162.5 (1120)	0.028
No. 8 (25)	9	128.4 (886)		0.032	161.6 (1114)	0.019
No. 9 (29)	8	127.1 (877)		0.029	161.7 (1115)	0.039
No. 10 (32)	3	133.7 (922)		0.050	169.5 (1169)	0.051
No. 11 (36)	28	132.8 (916)		0.040	168.5 (1162)	0.031
All sizes	137	130.6 (901)		0.051	165.2 (1139)	0.040
Lower tail of data for all sizes	—	—	1.159	0.043	—	—
Lower tail of data for ASTM A615/A615M Grade 60 (420)*	—	—	1.145	0.055	—	—

*Statistical determination used for establishing strength reduction factors ϕ as reported by Nowak and Szerszen (2003).

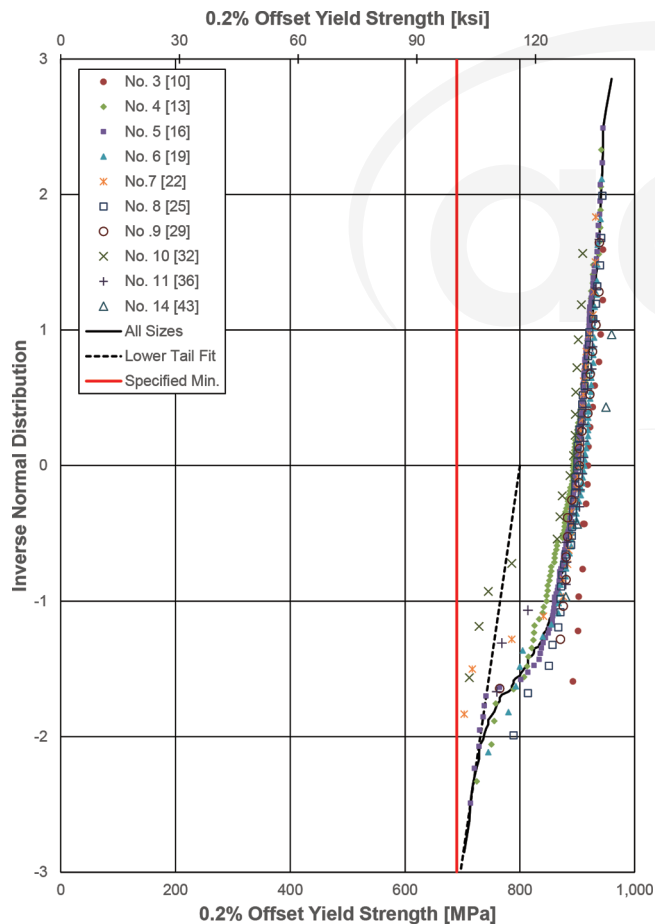


Fig. 3.4c—Cumulative distribution function for yield strength of ASTM A1035/A1035M (CS) Grade 100 (690) bars by 0.2 percent offset method. (Note: 1 MPa = 145 psi.)

(CS) Grade 100 (690) bar mill test data. The results are summarized in Table 3.4, Fig. 3.4c, and Fig. 3.4d. The summarized statistical data include the bias factor λ_b and the coefficient of variation (COV).

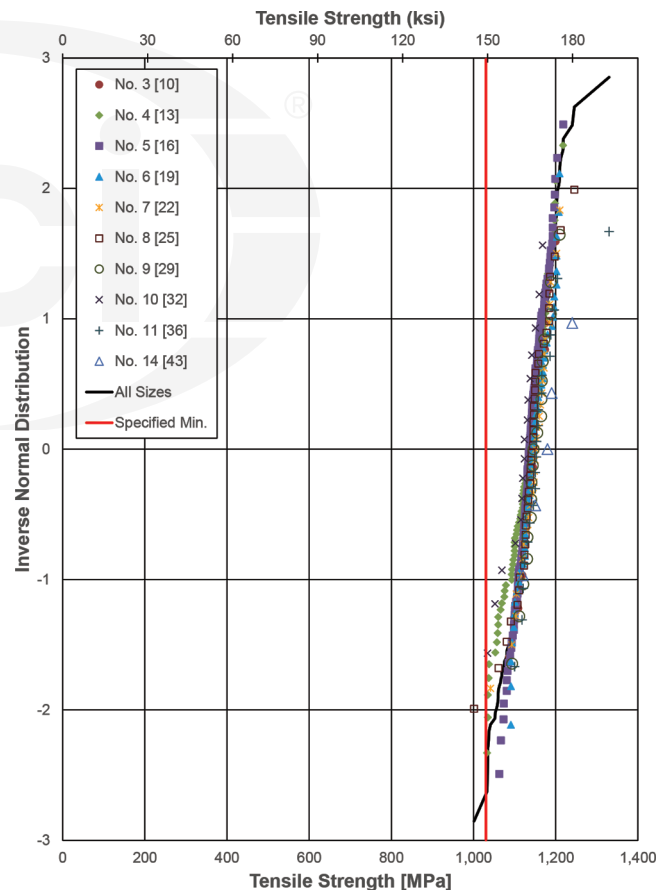


Fig. 3.4d—Cumulative distribution function for tensile strength of ASTM A1035/A1035M (CS) Grade 100 (690) bars. (Note: 1 MPa = 145 psi.)

Nowak and Szerszen (2003) reported the comparable statistical analysis for yield strength of ASTM A615/A615M Grade 60 (420) bars as used for code-calibration reliability analysis. They recommended using a bias factor λ_b of 1.145

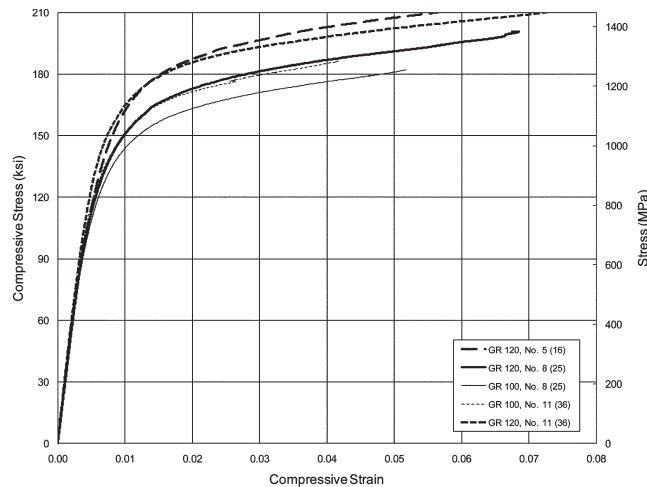


Fig. 3.5—Actual stress-strain curves in compression for ASTM A1035/A1035M (CS) reinforcing bars (WJE 2008).

and a COV of 0.05, based on the lower tail of distribution of the data used for their analysis, in the development of strength reduction factors for ACI 318. The summarized statistical data in Table 3.4 include the bias factor λ_b of 1.159 and COV of 0.043 for ASTM A1035/A1035M (CS) Grade 100 (690) reinforcement, based on a curve fit to the lower tail of the pooled data. The lower tail fit curve, shown in Fig. 3.4c, was determined in the same manner as used by Nowak and Szerszen (2003) for ASTM. The values of the bias factors for ASTM A1035/A1035M (CS) Grade 100 (690) bar is larger than 1.145 established for ASTM A615/A615M Grade 60 (420) bars, whereas the value for COV is less than the recommended value of 0.05 for ASTM A615/A615M Grade 60 (420) bars. Use of the same strength reduction factors with ASTM A1035/A1035M (CS) Grade 100 (690) bars as used for ASTM A615/A615M Grade 60 (420) bars is therefore recommended.

3.5—Actual compressive properties

WJE (2008) studied the stress-strain behavior in compression of representative samples of ASTM A1035/A1035M (CS) bars in size No. 5 (16), 8 (25), and 11 (36) in both Grades 100 (690) and 120 (830). The typical length of a compression test specimen was twice its nominal diameter. Figure 3.5 shows recorded compressive stress-strain curves. The proportional limit ranges from 60,000 to 80,000 psi (420 to 550 MPa) for both grades. Yield strength in compression by the 0.2 percent offset method is approximately 130,000 psi (900 MPa) for Grade 100 (690) bars and ranges from 135,000 to 140,000 psi (930 to 970 MPa) for Grade 120 (830) bars. The initial slopes of the stress-strain curves in compression are consistent with 29,000,000 psi (200,000 MPa) defined by ACI 318 for the modulus of elasticity.

3.6—Chemical composition

Table 3.6 shows the comparison of the specified chemical composition for ASTM A1035/A1035M (CS) low-carbon, chromium-steel bars with that of ASTM A615/A615M carbon-steel bars and ASTM A706/A706M low-alloy steel

Table 3.6—Specified chemical composition by heat analysis

Element	Bar type		
	ASTM A1035/ A1035M (CS)	ASTM A615/ A615M	ASTM A706/ A706M
	Maximum content, percent		
Carbon	0.15	†	0.30
Chromium	8.0 to 10.9*	—	—‡
Manganese	1.50	†	1.50‡
Nitrogen	0.05	—	—
Phosphorus	0.035	0.06	0.035
Sulfur	0.045	†	0.045
Silicon	0.50	—	0.50

*Chromium content should also be a minimum of 8.0 percent and maximum of 10.9 percent.

†Content should be reported but no limit is established.

‡Regulated by maximum carbon equivalent of 0.55 percent according to the following formula: $CE = \%C + \%Mn/6 + \%Cu/40 + \%Ni/20 + \%Cr/10 - \%Mo/50 - \%V/10$.

bars. The specified chemical content of ASTM A1035/A1035M (CS) bars differs from the other two primarily in that there is a required amount of chromium (8.0 to 10.9 percent). In addition, carbon content is limited to 0.15 percent and a maximum limit is specified for nitrogen (0.05 percent). Other than the chromium requirement, the chemical composition specified by ASTM A1035/A1035M (CS) is similar to ASTM A706/A706M. ASTM A1035/A1035M (CS) bars are not readily weldable because of metallurgical reasons, as discussed in 10.4 of this guide.

CHAPTER 4—BEAMS

4.1—Introduction

Although the previous chapters address Grade 100 (690) and Grade 120 (830) of ASTM A1035/A1035M (CS) bars, the procedures in 4.2 through 4.11 of this guide apply only to the use of ASTM A1035/A1035M (CS) Grade 100 (690) bars. Research data and experience are insufficient to allow proper evaluations of the procedures for Grade 120 (830) bars.

With the simplified strength design procedures in 4.2 and 4.3, flexural strength design using ASTM A1035/A1035M (CS) Grade 100 (690) bars is similar to that using ASTM A615/A615M Grade 60 (420) bars, whether by manual calculations or by using design software. Only certain inputs and limitations are modified.

4.2—Flexural strength

As discussed in 3.4 of this guide, Eq. (3.4a) through (3.4c) represent a lower bound to the stress-strain behavior of ASTM A1035/A1035M (CS) Grade 100 (690) bar.

For a given reinforced concrete section, the designer may perform a nonlinear flexural analysis by using the stress-strain relationship defined by Eq. (3.4a) and (3.4b) and satisfying the requirements of force equilibrium and strain compatibility. However, the analysis procedure requires an iterative process of trial and error. If such a nonlinear

analysis is used for flexural design, the tensile stress in the reinforcement under service load should be determined to evaluate satisfaction of serviceability criteria such as crack control or control of member deflections. Furthermore, evaluation procedures for other strength criteria at ultimate limit state, such as shear strength, should consider nonlinear behavior of the reinforcement. Refer to [Appendix B](#) of this guide for detailed discussions of a nonlinear flexural analysis approach and for design examples comparing the nonlinear flexural analysis approach with elastic-plastic approach.

A simplified design method using [ASTM A1035/A1035M](#) (CS) bars based on the study by [Mast et al. \(2008\)](#) uses an idealized elastic-plastic stress-strain curve, similar to that used for [ASTM A615/A615M](#) Grades 60 (420) and 75 (520) bars (Fig. 4.2). Limitations are set on the range of applicability for this method to protect against harmful effects resulting from the actual steel stress being higher than assumed, and effects from larger steel strains at service load. Refer to 4.3 of this guide for a discussion of these limitations.

The idealized elastic-plastic stress-strain curve shown in Fig. 4.2 has an elastic portion with a modulus E_s of 29,000,000 psi (200,000 MPa) (the same as for [ASTM A615/A615M](#) bar), and a perfectly plastic behavior after reaching f_y equal to 100,000 psi (690 MPa). These parameters allow computation of flexural strength with f_y set at 100,000 psi (690 MPa) for a beam that is tension-controlled. [Appendix A](#) illustrates a series of design examples using the idealized elastic-plastic stress-strain curve of [ASTM A1035/A1035M](#) (CS) Grade 100 (690) reinforcement.

4.3—Tension- and compression-controlled limits

4.3.1 Historical—It has long been accepted that reinforced concrete flexural members should behave elastically at service load but have the capability to deform inelastically before reaching maximum capacity. This is accomplished by limiting the reinforcement ratio so that the reinforcement yields before the concrete crushes. Beginning with [ACI 318-63](#), flexural member reinforcement ratio was limited to 75 percent of the balanced reinforcement ratio ρ_b . By 1995, this criterion had been used for over 30 years, and flexural member behavior was judged to be satisfactory.

4.3.2 Tension- and compression-controlled strain limits—Beginning with [ACI 318-95](#), the tension-controlled criterion based on tensile strain in the reinforcement was selected to provide behavior similar to that experienced under the 0.75 ρ_b criterion. Nonlinear analyses of the behavior between service load and nominal strength were made for the old maximum reinforcement ratio and the new tension-controlled strain criteria. The ratio of deformation at nominal strength to that at service load for steel strain ϵ_s , curvature ψ , and deflection Δ were calculated and compared. Setting the tension-controlled strain limit at 0.005 for Grade 60 (420) steel provided more deformation than the $\rho = 0.75\rho_b$ limit. For Grade 60 (420) steel, the compression-controlled limit was set at 0.002, which is the strain (rounded) at the balanced condition ([Mast 1992](#)).

4.3.3 Strain limits for sections with [ASTM A1035/A1035M](#) (CS) bar—The definition of tension-controlled sections in

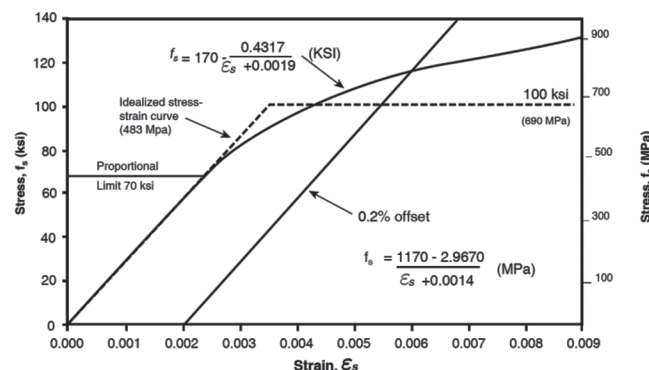


Fig. 4.2—Approximated nonlinear stress-strain relationship of [ASTM A1035/A1035M](#) (CS) Grade 100 (690) steel and idealized bilinear elastic-plastic stress-strain relationship for simplified design. (Note: 1 ksi = 6.9 MPa.)

[ACI 318](#) is based on [ASTM A615/A615M](#) Grades 60 (420) and 75 (520) bars. These were the reinforcing bars available when [ACI 318](#) strain limits were originally developed. For [ASTM A1035/A1035M](#) (CS) bar, the process to determine [ACI 318](#) strain limits was repeated using equations similar to Eq. (3.4a) and (3.4b) of this guide and the idealized elastic-plastic stress-strain relationship for [ASTM A1035/A1035M](#) (CS) bar. It was found that a tension-controlled strain limit of 0.0066 (corresponding to $c/d = 5/16$) produced behavior similar to members designed by [ACI 318](#) with Grades 60 (420) and 75 (520) bars and a strain limit of 0.005 ([Mast et al. 2008](#)). The strain limit for compression-controlled sections is discussed in the next section.

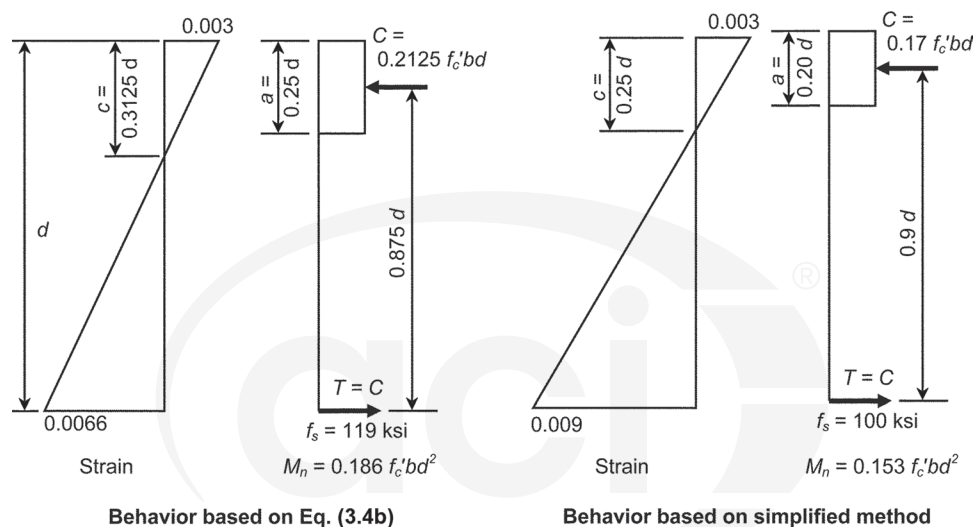
4.3.4 Simplified design strain limits—The tension- and compression-controlled strain limits need to be modified for simplified design using the idealized elastic-plastic stress-strain relationship described in 4.2. The tension-controlled strain limit should be adjusted to 0.009 ($c/d = 0.25$) to compensate for the fact that the actual steel stress at nominal strength is higher than the assumed stress of 100,000 psi (690 MPa). This adjustment ensures ductility and deformability are comparable to designs based on [ACI 318](#) using [ASTM A615/A615M](#) Grade 60 (420) bars. Table 4.3.4 and Fig. 4.3.4 show that the simplified method using $\epsilon_t = 0.009$ and $f_y = 100,000$ psi (690 MPa) results in a value of nominal flexural strength 18 percent less than the same section designed using $\epsilon_t = 0.0066$ and the corresponding $f_s = 119,000$ psi (820 MPa) according to Eq. (3.4b). Using the simplified method avoids having to perform an analysis using the nonlinear stress-strain relationship of the steel.

[ACI 318](#) defines the compression-controlled strain limit as the net tensile strain at balanced strain conditions. For f_y of 100,000 psi (690 MPa) and $E_s = 29,000,000$ psi (200,000 MPa), this is a strain of 0.00345. It is simpler and conservative to round the compression-controlled strain limit for the [A1035/A1035M](#) (CS) reinforcement to 0.004.

Using the limits defined previously for the simplified method, the tension-controlled limit occurs at a $c/d = 0.25$ (instead of 0.375 for Grade 60 [420] steel), and the compression-controlled limit is at a $c/d = 3/7$, or 0.43 (instead of 0.600 for Grade 60 [420] steel).

Table 4.3.4—Comparison of design methods using ASTM A1035/A1035M (CS) Grade 100 (690) steel

	Using Eq. (3.4a) and (3.4b)	Simplified method
Tension-controlled strain limit	0.0066	0.009
Steel tensile stress f_s , ksi (MPa)	119 (820)	100 (690)
Neutral axis depth c , in. (mm)	$0.3125d$	$0.25d$
Stress block depth $a = \beta_1 c$, in. (mm)	$0.3125\beta_1 d$	$0.25\beta_1 d$
Compression force C , kip (N)	$0.85f'_c ab$	$0.85f'_c ab$
Steel area $A_s = C/f_s$, in. ² (mm ²)	$0.85(f'_c/f_s)(0.3125\beta_1\delta)b$	$0.85(f'_c/f_s)(0.25\beta_1\delta)b$
Tension-controlled reinforcement ratio $\rho_t = A_s/bd$	$0.002232f'_c/\beta_1$ ($0.0003239f'_c/\beta_1$)	$0.002125f'_c/\beta_1$ ($0.0003079f'_c/\beta_1$)
$T = C = A_s f_s = \rho_t b d f_s$, kip (N)	$0.2656f'_c\beta_1 bd$	$0.2125f'_c\beta_1 bd$
Lever arm $= d - a/2$, in. (mm)	$d(1 - 0.156\beta_1)$	$d(1 - 0.125\beta_1)$
M_n for $f'_c = 5$ ksi (34.5 MPa); $\beta_1 = 0.8$, in.-kip (mm-N)	$0.186f'_c bd^2$	$0.153f'_c bd^2$

*Fig. 4.3.4—Tension-controlled strain limits with $f'_c = 5$ ksi and $\beta_1 = 0.8$. (Note: 1 ksi = 6.9 MPa.)*

4.4—Strength reduction factor ϕ

With the aforementioned modified tension- and compression-controlled limits for use with the simplified method, it is necessary to revise the equation for the transition in ϕ between these strain limits. The revised equation is given as follows

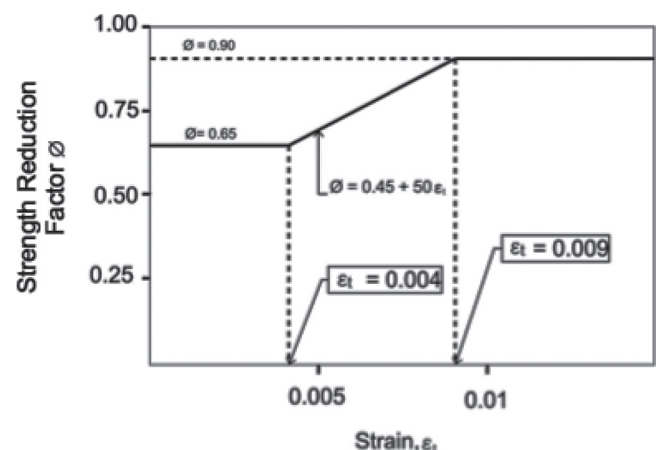
$$0.65 \leq (\phi = 0.45 + 50\varepsilon_t) \leq 0.9 \quad (4.4)$$

Note that for $\varepsilon_t \geq 0.009$ (tension-controlled section), $\phi = 0.9$, and for $\varepsilon_t \leq 0.004$ (compression-controlled section), $\phi = 0.65$. Figure 4.4 illustrates this relationship.

If the designer uses nonlinear analysis for flexural design, Eq. (4.4) should be modified for the value of ϕ in the transition between the tension- and compression-controlled strain limits. The modified equation for ϕ is given in **Appendix B**.

4.5—Stress in steel due to flexure

For an efficient use of flexural reinforcement, it is advisable to proportion all flexural members as tension-controlled members. Figure 4.5 shows steel strain and stress diagrams for a member at the tension-controlled limit. The steel stress diagram shows both the idealized elastic-plastic and nonlinear stress distributions. The stress diagrams below the

*Fig. 4.4—Relationship between strength reduction factor ϕ and strain limits for use only with simplified design based on idealized elastic-plastic stress-strain relationship for A1035/A1035M (CS) Grade 100 (690) steel reinforcement.*

neutral axis are the same as in Fig. 4.2. There are two regions in the steel stress diagram where the idealized elastic-plastic design stress may exceed the nonlinear design stress given

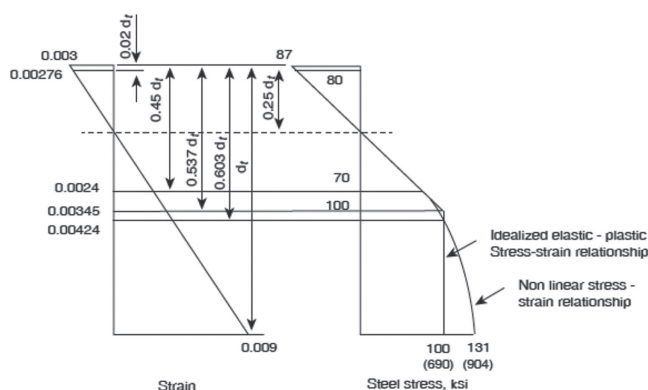


Fig. 4.5—Strain and steel stress at tension-controlled limits.
(Note: 1 ksi = 6.9 MPa.)

by Eq. (3.4a) and (3.4b) of this guide. On the tension side, this occurs between $0.45d_t$ and $0.603d_t$ below the extreme compression fiber. Ordinarily, beams do not have primary tension steel at this location, but if there is, underestimating steel stress near the tension face when using the idealized elastic-plastic stress-strain model offsets strength overestimation between $0.45d_t$ and $0.603d_t$. On the compression side, in relation to a maximum usable compression steel stress of 80,000 psi (550 MPa), the steel stress is overestimated for steel with strains from 0.00276 to 0.003. For a section at the tension-controlled limit, this occurs only for steel located within $0.02d_t$ measured from the compression face. The influence of this overestimation is typically insignificant.

4.6—Compression stress limit

ACI 318 limits the maximum usable compression strain in concrete to 0.003. For this reason, ACI 318 limits the compression steel stress to 80,000 psi (550 MPa), and this limit should also apply for ASTM A1035/A1035M (CS) bars in compression.

Although most design software accepts only a single value of f_y for tension and compression, using $f_y = 100,000$ psi (690 MPa) as input for design software should not be a problem. For a maximum concrete compressive strain of 0.003, the steel stress in compression does not exceed 80,000 psi (550 MPa) unless the depth from the extreme compression fiber to the compression steel, d' , is less than $0.08c$, where c is the depth of the compression zone. This is unlikely to happen and can be easily checked.

4.7—Moment redistribution

The moment redistribution technique in ACI 318 allows moments in continuous members to be increased or decreased by prescribed amounts relative to the magnitudes determined from the elastic theory, where sufficient rotational capacity is provided at plastic hinge locations. Corresponding adjustments are made to the moment demand envelopes in the span. As of the writing of this guide, no data are available to judge if moment redistribution is applicable to members with ASTM A1035/A1035M (CS) bars. In particular, the simplified strength design procedures in 4.2 and 4.3 of this guide may not provide adequate estimates of

the actual loading conditions at which flexural hinges form. Furthermore, the inelastic rotation capacity of high-moment regions reinforced with ASTM A1035/A1035M (CS) bars has not been evaluated. Therefore, do not use moment redistribution for members containing ASTM A1035/A1035M (CS) bars until further data are available.

When determining the design forces in members with ASTM A1035/A1035M (CS) bars or other reinforcement constructed integrally with other members or elements that contain ASTM A1035/A1035M (CS) longitudinal bars, the actual forces transferred from the member containing ASTM A1035/A1035M (CS) bars may be difficult to determine at the strength limit state because ASTM A1035/A1035M (CS) bars lack a well-defined yield plateau. This situation could occur with beam-column joints of frame structures, the interface between slabs and spandrel beams, two-way slab construction, or continuous members where ASTM A1035/A1035M (CS) bars and other reinforcement types are used in different regions.

4.8—Deflection

For one-way members, ACI 318 provides two methods for controlling deflections at the service load level:

- (1) Compute the expected deflections and compare against appropriate limits
- (2) Implicitly control deflection through minimum thicknesses

When deflections are not computed, ACI 318 provides the minimum height or thickness h of one-way concrete members to implicitly control deflections. The minimum thickness values apply only to members not supporting or attached to construction likely to be damaged by deflections, and are given as functions of the span length, member type, and support condition. The table is based on normalweight concrete and ASTM A615/A615M Grade 60 (420) bar. For f_y other than 60,000 psi (420 MPa), footnote (b) of the table permits modification of the tabulated values by the multiplier $(0.4 + f_y/100,000)$. This table was first presented in ACI 318-71 where accompanying commentary stated:

The modification for yield strength in Footnote (b) is based on judgment, experience, and studies of the results of tests and of unpublished analyses. The simple expression given is approximate but should yield conservative results for the types of members considered in the table, for typical reinforcement ratios, and for values of f_y between 40 and 80 ksi.

The range of f_y between 40,000 and 80,000 psi (280 and 550 MPa) corresponded to the range typically available in 1971.

When using ASTM A1035/A1035M (CS) longitudinal bars in designs based on the method described in 4.2 and 4.3 of this guide, the strain in the reinforcement at the service condition is higher than that in comparable members designed with Grade 60 (420) reinforcement. Under service load condition, the steel stress is usually taken as $0.4f_y$. Using the multiplier $(0.4 + f_y/100,000)$ as a member

thickness modification factor results in similar maximum member curvature for members with different reinforcement strengths. The guide by Mast (2006) shows that for $f_y = 100,000$ psi (690 MPa), the deflection at service load is 1.4 times that for members with $f_y = 60,000$ psi (420 MPa). This confirms the applicability of the multiplier of $(0.4 + f_y/100,000) = 1.4$ on the minimum height or thickness h in ACI 318 minimum thickness recommendations of Chapter 8 when using ASTM A1035/A1035M (CS) bars with $f_y = 100,000$ psi (690 MPa). Desalegne and Lubell (2012) recommended that direct deflection calculations should be used for the design of concrete slabs reinforced with ASTM A1035/A1035M (CS), which is discussed in Chapter 6.

For direct deflection calculation, the ACI 318 method uses an effective moment of inertia proposed by Branson (1977) to account for variable cracks at different sections along the member length. Bischoff (2005), however, showed that using Branson's equation for I_e underestimates deflections, especially when the reinforcement ratio is less than 1 percent. Bischoff (2005) proposed an alternate formulation of I_e

$$I_e = \frac{I_{cr}}{1 - \left(1 - \frac{I_{cr}}{I_g}\right) \left(\frac{M_{cr}}{M_a}\right)^2} \leq I_g \quad (4.8)$$

Calculated deflections using Eq. (4.8) correlate well with deflections measured in laboratory tests for members with a range of reinforcement ratios (Bischoff and Scanlon 2007). Because slabs and beams using ASTM A1035/A1035M (CS) longitudinal bars may result in lightly reinforced members, Eq. (4.8) may be used in place of ACI 318-14 Eq. (24.2.3.5a). This method is suitable when the steel stress at the load condition checked for deflection is below the proportional limit.

When ASTM A1035/A1035M (CS) steel is used as longitudinal reinforcement, the designer should check that the member deflection limits are not exceeded for lightly reinforced members.

4.9—Crack control

4.9.1 Historical—In the early 1960s, the developments of ultimate strength design and ASTM A615/A615M Grade 60 (420) reinforcement caused concern about crack widths. These two changes in design practice resulted in steel stress in flexural members at the service load of approximately 36,000 psi (250 MPa), almost twice the 20,000 psi (140 MPa) allowed previously for ASTM A615/A615M Grade 40 (280) steel under working stress design. Numerous research papers led to the crack control requirements of Chapter 10 in ACI 318-71. These requirements remained mostly unchanged until 1999.

Researchers agree that strain in the tension steel is one of the primary variables for controlling crack width. Because the elastic relationship between strain and stress for steel is a constant for strains smaller than the proportional limit, most crack width formulas use f_s instead of ϵ_s . In addition, the

interaction of bond to the reinforcement and tensile stress in the concrete surrounding the reinforcement also affect crack spacing and crack width. Numerous equations have been proposed for predicting crack spacings and widths including the Gergely-Lutz equation (Gergely and Lutz 1968), which was adopted by ACI 318-71.

4.9.2 Crack widths—There are three reasons for limiting crack width: 1) appearance; 2) control of reinforcement corrosion; and 3) water-tightness. Aesthetic considerations by owners and designers often demand tight control of crack width. For liquid-retaining structures and building envelopes, water-tightness is essential. Control of reinforcement corrosion is paramount to achieve durability.

4.9.3 ACI 318-99 revisions—While the z -factor determined from Eq. (10-2) of ACI 318-71 provided adequate reinforcement distribution for crack control of normal-size beams and one-way slabs with standard concrete clear covers not exceeding 1.5 in. (38 mm), the z -factor method based on the Gergely-Lutz (Gergely and Lutz 1968) equation produced unrealistic bar spacings when larger concrete covers were used. This led designers to either ignore the provisions when larger covers were used or simply use the code-minimum cover or a standard value—typically 2.5 in. (63 mm)—rather than the actual cover in the calculations. Additionally, some building inspectors continued the old practice of comparing measured crack widths in the field against the limiting crack widths according to the z -factor as reasons for rejecting members or projects, even though research showed that there was little if any correlation between corrosion and crack width for the range of crack widths being considered. Further, the limiting crack widths implied by the z -factor—0.016 in. (0.41 mm) for interior exposure and 0.013 in. (0.33 mm) for exterior exposure (refer to Commentary on Section 10.6.4 of ACI 318-71)—represent a difference of 0.003 in. (0.08 mm), which implies an unrealistic accuracy. To find a simpler, more direct way of defining bar spacing to control cracking than Eq. (10-2), ACI Committee 318 examined the work of Kaar and Mattock (1963), Gergely and Lutz (1968), Beeby (1979), and Frosch (1999). A simple bilinear relationship between concrete clear cover and bar spacing for a maximum crack width of 0.016 in. (0.41 mm) was adopted as Eq. (10-5) in ACI 318-99. Figure 4.9.3 illustrates this bilinear relationship and three other nonlinear relationships. In Fig. 4.9.3, f_s is the bar stress at service load, assumed as 60 percent of specified minimum yield strength, and is the ratio of the distance from neutral axis to the extreme tension face to the distance from neutral axis to the center of the tension reinforcement. The bilinear relationship (ACI 318-99 Eq. (10-5)) is independent of β and bar size, whereas the other nonlinear relationships depend on these variables. A modified version of Eq. (10-5) of ACI 318-99—valid for service load steel stresses ranging from 24,000 to 48,000 psi (170 to 330 MPa)—became Eq. (10-4) of ACI 318-05.

4.9.4 Applicability with ASTM A1035/A1035M (CS) bars—Figure 4.9.4 is similar to the bilinear equation shown in Fig. 4.9.3, but with f_s taken as 67,000 psi (462 MPa), which is two-thirds of the specified minimum yield strength f_y . ACI

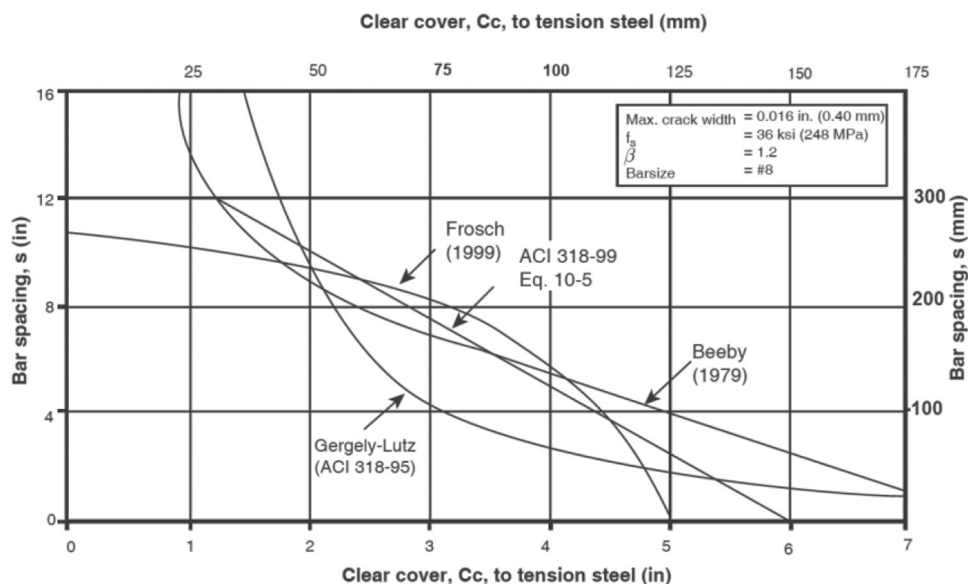


Fig. 4.9.3—Bar spacing versus clear cover for crack control. (Note: 1 in. = 25 mm; 1 ksi = 6.9 MPa.)

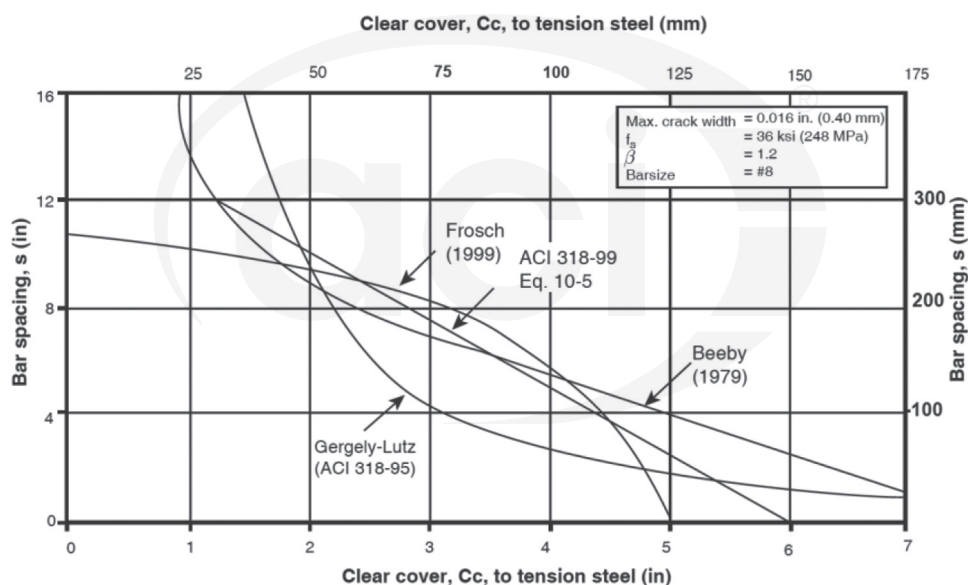


Fig. 4.9.4—Bar spacing versus clear cover for crack control ($f_s = 67$ ksi). (Note: 1 in. = 25 mm; 1 ksi = 6.9 MPa.)

318 gives more conservative results compared with those shown in Fig. 4.9.3, but the results are restrictive in design choices. For example, the bar spacing s cannot exceed 6.5 in. (165 mm) at 1 in. (25 mm) cover c_c , and smaller bar spacing is required at greater cover to meet the 0.016 in. (0.41 mm) crack width criterion. Any combination of c_c and s above the bilinear curve leads to larger crack widths.

When crack control is critical, use of ACI 318 is appropriate. For members with minimal cover, such as slabs and walls, this is feasible when designing for $f_y = 100,000$ psi (690 MPa) and $f_s = 67,000$ psi (460 MPa). To provide crack control at a reasonable bar spacing for members with increased cover, it will be necessary to limit the steel stress at service load to less than 67,000 psi (460 MPa).

4.10—Minimum reinforcement

A minimum area of flexural reinforcement in accordance to ACI 318, A_{smin} , shall be provided at every section where tension reinforcement is required by analysis with $f_y = 100,000$ psi (690 MPa). When the minimum reinforcing requirements for temperature and shrinkage of ACI 318 are applied, the minimum reinforcing ratio of 0.0014 will control.

4.11—Strength design for shear

Design for shear and torsion in reinforced concrete beams using ASTM A1035/A1035M (CS) bars as reinforcement should follow ACI 318, with a few exceptions noted as follows.

The shear failure of reinforced concrete beams without shear reinforcement is brittle and the shear stress at failure is

influenced by several design parameters, including member depth, longitudinal reinforcement ratio, and concrete strength (Rajagopalan and Ferguson 1968; Collins and Kuchma 1999; Reineck et al. 2003; Lubell et al. 2004). Unsafe predictions of shear capacity can result from the use of ACI 318 for lightly reinforced beams that are generally with ρ less than 1 percent. Laboratory tests (Hassan et al. 2008; Desalegne and Lubell 2010) demonstrate that these trends also occur for members containing ASTM A1035/A1035M (CS) bars as longitudinal reinforcement; therefore, all lightly reinforced beams should contain minimum shear reinforcement.

For members designed according to the simplified method for flexural design in 4.2 of this guide, the shear capacity can be determined from ACI 318. Such members should include shear reinforcement that satisfies the spacing and quantity requirements of ACI 318. ACI 318 limits the design yield strength f_{yt} for shear reinforcement to 60,000 psi (420 MPa), but the value can be increased to 80,000 psi (550 MPa) for welded deformed wire reinforcement. Similarly, ACI 318 limits the design yield strength f_{yt} for torsional reinforcement to 60,000 psi (420 MPa). The limitation of 60,000 psi (420 MPa) has been specified since ACI 318-71, in part to provide a control on diagonal crack width at service load levels.

ACI 318-95 raised the f_{yt} limit to 80,000 psi (550 MPa) for welded deformed wire reinforcement. Full-scale beam tests (Griezic et al. 1994) indicated that inclined shear crack widths at service load levels were less for beams with welded deformed wire reinforcement cages designed with a yield strength of 75,000 psi (520 MPa) than for beams reinforced with deformed Grade 60 (420) stirrups. For lack of comparative data, the limitation of 60,000 psi (420 MPa) remains for torsional reinforcement even when using welded deformed wire reinforcement.

Tests of full-scale beams by Munikrishna (2008) and Sumpter et al. (2009) compared the behavior of beams reinforced with ASTM A1035/A1035M (CS) Grade 100 (690) bars as stirrups with 135-degree hooks and designed with f_{yt} of 80,000 psi (550 MPa) with the behavior of similar beams reinforced with deformed Grade 60 (420) stirrups. The results indicated that the behavior of the two groups of beams was similar at failure. At service load levels, inclined shear crack widths were larger, as expected, for beams reinforced with ASTM A1035/A1035M (CS) Grade 100 (690) stirrups designed with a yield strength of 80,000 psi (550 MPa). In all cases, the crack widths were less than the commonly accepted limit of 0.016 in. (0.41 mm).

A design yield strength f_{yt} of 80,000 psi (550 MPa) for ASTM A1035/A1035M (CS) Grade 100 (690) stirrups as shear reinforcement is appropriate if appearance and serviceability due to shear cracking is not a critical design consideration. Otherwise, f_{yt} should be limited to 60,000 psi (420 MPa). For lack of research data, f_{yt} should also be limited to 60,000 psi (420 MPa) for shear reinforcement designed for torsion.

CHAPTER 5—COLUMNS

5.1—Introduction

Column design using ASTM A1035/A1035M (CS) bars should follow ACI 318, with modifications made to the specified minimum yield strength, f_y , for the longitudinal reinforcement and f_{yt} for the lateral reinforcement.

5.2—Specified minimum yield strength for longitudinal reinforcement

For members under axial load, the maximum compressive stress of the longitudinal reinforcement should be limited to 80,000 psi (550 MPa) when computing the design axial strength ϕP_n using ACI 318.

The steel strain ϵ_s corresponding to this stress is approximately 0.0028, nearly equal to the maximum usable strain (0.003) for concrete in compression assumed by ACI 318.

For members subjected to axial load combined with moment, f_y should be 80,000 psi (550 MPa) for the longitudinal reinforcement in compression for the reason given previously, and 100,000 psi (690 MPa) for the longitudinal reinforcement in tension as in the case of flexural design discussed in Chapter 4 of this guide.

5.3—Specified minimum yield strength for transverse reinforcement

Transverse reinforcement in a column serves three functions:

- (1) Lateral support to the longitudinal reinforcement
- (2) Confinement to the core concrete when the load supported by the column approaches its axial strength
- (3) Shear reinforcement when the column is subjected to shear

The transverse reinforcement is most commonly rectangular or circular ties and spirals.

ACI 318 provides minimum size and maximum spacing of ties in compression members, irrespective of the bar strength. These requirements apply for ASTM A1035/A1035M (CS) bars.

When a column is required to resist shear, provide ties for the required shear strength as discussed in 4.11 of this guide.

For columns reinforced with ASTM A1035/A1035M (CS) spirals, determine the volumetric spiral reinforcement ratio ρ_s of ACI 318 with the specified minimum yield strength f_{yt} as 100,000 psi (690 MPa). Spirals should be continuous and lap splices should not be used.

5.4—Slenderness effect

When slenderness effect is considered, design the column by using the moment magnification procedure of ACI 318. Use the magnified moment to design the column according to the procedures outlined in Chapter 4 of this guide.

CHAPTER 6—SLAB SYSTEMS

6.1—One-way slabs

Slab depth is typically determined by a deflection and considerations including ACI 318 depth limits table for

one-way slabs modified for $f_y = 100,000$ psi (690 MPa), or by the depth required to provide adequate shear strength without shear reinforcement. Design for flexure and deflection in one-way slabs, with or without joists, is based on the same principles as for beams. Thus, the recommendations in Chapter 4 of this guide apply. To control flexural cracking, the distribution of flexural reinforcement should conform to ACI 318, as discussed in 4.9 of this guide. In addition, shrinkage and temperature reinforcement should be provided to meet the requirements of ACI 318.

An analytical study of one-way slab deflection at service load level (Tang and Lubell 2008) using Eq. (4.8) for the effective moment of inertia, I_e , indicated that minimum slab thickness for deflection control was less sensitive to reinforcement strength than suggested by ACI 318 and more heavily influenced by applied loading, span length, and the assumptions for cracking moment. Desalegne and Lubell (2012) showed similar trends for members with A1035/A1035M (CS) Grade 100 reinforcement.

6.2—Shear design of one-way slabs

Shear design of one-way slabs is the same as shear design of beams and should follow the provisions of ACI 318. Shear reinforcement, however, is rarely used in one-way slabs. As indicated in 4.11 of this guide, research has demonstrated that for members without shear reinforcement, the shear stress at failure decreases as the member depth increases and as the reinforcement ratio decreases.

Bentz et al. (2006) and Bentz and Collins (2006) developed a simplified shear capacity model that accounts for the influence of member depth, concrete strength, aggregate size, crack width, and longitudinal reinforcement strains on shear strength. Hoult et al. (2008) presented an enhanced version of the model to better predict shear strengths for members with larger reinforcement strains.

For one-way slabs containing ASTM A1035/A1035M (CS) longitudinal bars designed in accordance with Chapter 4 of this guide, a simplification to the Hoult et al. (2008) model for shear strength was developed by Desalegne and Lubell (2010). This simplification assumed a longitudinal reinforcement strain of 0.0042, which corresponds to a stress of 100,000 psi (690 MPa) according to Eq. (3.4b) of this guide. For lightly reinforced members (generally with ρ less than 1 percent) without significant axial load and not containing shear reinforcement, the shear strength may be taken as

$$V_c = \frac{73}{(39 + 2.1d)} \sqrt{f'_c} b_w d \quad (\text{in.-lb})$$

$$V_c = \frac{154}{(1000 + 2.1d)} \sqrt{f'_c} b_w d \quad (\text{SI})$$
(6.2)

Because Eq. (6.2) is based on the simplified flexural model from Mast et al. (2008) (4.2), it is not appropriate for members designed according to Appendix B of this guide. Desalegne and Lubell (2010) provide a generalized

shear capacity model that can also be applied to members designed for flexure using Appendix B of this guide.

6.3—Two-way slabs

ACI 318 governs design of two-way slab systems. These provisions should be applicable when using ASTM A1035/A1035M (CS) bars with specified minimum yield strength of 100,000 psi (690 MPa) for tension and 80,000 psi (550 MPa) for compression. ACI 318 permits design of two-way slabs by the Direct Design Method (Section 8.10 of ACI 318-14) or the Equivalent Frame Method (Section 8.11 of ACI 318-14). Both methods are suitable for analysis of two-way slabs containing ASTM A1035/A1035M (CS) steel. As noted in 4.7 of this guide, however, the use of moment redistribution should not be used for members containing ASTM A1035/A1035M (CS) bars until further test data are available. Therefore, allowing up to 10 percent moment redistribution in accordance to ACI 318-14 in slabs with Grade 60 (420), steel should not be followed.

ACI 318 provides guidance on slab reinforcement details and illustrates the minimum extensions of reinforcement that are required in two-way slabs without beams. These provisions should also apply to slabs with ASTM A1035/A1035M (CS) steel if the designer directly checks for the impact on flexural capacity due to the longer bar development lengths compared with steel with $f_y = 80,000$ psi (550 MPa).

Only limited test data are available that examine the punching shear strength of two-way slab systems containing ASTM A1035/A1035M (CS) steel as flexural reinforcement without any shear reinforcement (Seliem et al. 2008). The punching shear strength of two specimens containing ASTM A1035/A1035M (CS) steel were similar to that of a third specimen containing Grade 60 (420) steel, and the capacities were in agreement with the existing ACI 318 punching shear analytical model. One-way shear should also be checked in accordance with 6.2 of this guide. Because of a lack of adequate research data, the specified minimum yield strength f_{yt} for shear reinforcement should be limited to 60,000 psi (420 MPa), where shear reinforcement is required to provide sufficient shear strength for a two-way slab system. Yang et al. (2010) reported that direct replacement of conventional steel bars with high-strength steel bars, having the same area, resulted in a 27 percent increase of the punching shear strength. This increase of punching shear resistance is because the higher-strength bars did not yield prior to punching failure.

When $f_y = 100,000$ psi (690 MPa) is used in the design of two-way slab systems, it is recommended that the minimum slab thickness without interior beams be determined according to ACI 318. When using ACI 318 depth limits table, direct extrapolation from $f_y = 75,000$ psi (520 MPa) to $f_y = 100,000$ psi (690 MPa) is recommended. When using ASTM A1035/A1035M (CS) bars as longitudinal reinforcement and members are lightly reinforced (generally with ρ less than 1 percent), direct calculations of deflection is recommended.

Due to the higher reinforcement stresses of ASTM A1035/A1035M (CS) steel at the service load level, it is essential

to check for adequate control of cracking. Additional guidance on crack control in two-way structures is provided by [ACI 224R](#).

CHAPTER 7—WALLS

7.1—Introduction

Design of walls using ASTM A1035/A1035M (CS) Grade 100 (690) bars should follow ACI 318, except that f_y for flexural reinforcement and for shear reinforcement should be limited as indicated in 7.2 through 7.4 of this guide.

7.2—Vertical reinforcement

Limit specified minimum yield strength f_y to 80,000 psi (550 MPa) for vertical reinforcement in compression, and to 100,000 psi (690 MPa) for vertical reinforcement in tension due to out-of-plane flexure or in-plane overturning forces.

7.3—Horizontal reinforcement

Limit the specified minimum yield strength f_y to 80,000 psi (550 MPa) for horizontal reinforcement in compression, and to 100,000 psi (690 MPa) for horizontal reinforcement in tension due to flexure.

7.4—Shear reinforcement

Limit the specified minimum yield strength f_y to 60,000 psi (420 MPa) where the vertical or horizontal reinforcement is required to resist shear. If appearance and serviceability due to shear cracking are not critical design considerations, f_y may be taken as 80,000 psi (550 MPa). Larger diagonal crack widths under service load are expected if f_y is taken as 80,000 psi (550 MPa). Refer to the ACI 318 Commentary on crack width and shear reinforcement.

7.5—Minimum reinforcement

Minimum reinforcement for the control of shrinkage cracking should meet the requirements of ACI 318.

CHAPTER 8—FOOTINGS AND PILE CAPS

8.1—Design

The design of footings and pile caps is governed by the provisions of ACI 318. The depth of footings is generally determined by its required shear strength.

For flexural design of shallow footings and pile caps, follow the design principles from [Chapter 4](#) of this guide. For reinforcement in tension due to flexural forces, f_y may be taken up to 100,000 psi (690 MPa). For reinforcement in compression due to flexural forces, f_y should not be taken greater than 80,000 psi (550 MPa). For shear design of shallow footings, follow the principles in [4.11](#) and [6.2](#) of this guide.

For deep footings or regions of foundation structures meeting the criteria of ACI 318, the strut-and-tie modeling procedure in ACI 318 can be applied. [Hassan et al. \(2008\)](#) and [Garay-Moran and Lubell \(2008, 2016\)](#) reported tests of deep beams with ASTM A1035/A1035M (CS) bars as the primary tension ties, but without web reinforcement. In all

cases, the failure mode was brittle and the ACI 318 provisions over-predicted the strength for beams with higher concrete strengths and larger shear span-depth ratios. Therefore, all members with ASTM A1035/A1035M (CS) ties should contain minimum web reinforcement in accordance with ACI 318 until additional test data become available.

[Garay-Moran and Lubell \(2008, 2016\)](#) reported tests of deep beams containing ASTM A1035/A1035M (CS) ties and Grade 60 (420) web reinforcement. Ductile failure modes were achieved, and ACI 318 strut-and-tie modeling provisions predicted strengths safely when the maximum tension tie stress was limited to 100,000 psi (690 MPa). No test data are available for the case of deep members with ASTM A1035/A1035M (CS) bars as the distributed web reinforcement. The strut-and-tie models should not be used in cases where the elastic flow of forces is not followed and also where considerable deformation capacity redistribution is needed based on the model geometry.

CHAPTER 9—MAT FOUNDATIONS

9.1—Design

ACI 318 governs the design of mat foundations. For design with ASTM A1035/A1035M (CS) bars, limit steel stress to 80,000 psi (550 MPa) for reinforcement in compression and 100,000 psi (690 MPa) for reinforcement in tension due to flexure.

Provide minimum reinforcement in each principal direction, with maximum spacing of 18 in. (460 mm), in accordance with ACI 318. Distribute reinforcement near the top or bottom of the section or allocate between the two section faces for specific conditions, such that the total area of continuous reinforcing steel satisfies ACI 318.

For shear design of mat foundations, follow the principles in [4.11](#), [6.2](#), and [6.3](#) of this guide.

CHAPTER 10—OTHER DESIGN CONSIDERATIONS

10.1—Seismic design limitations

ACI 318 provides design requirements for earthquake resistance but does not apply to structures assigned to Seismic Design Category (SDC) A. Structures assigned to SDC B through F should satisfy the applicable provisions of ACI 318. The recommendations presented in previous sections are applicable to structures assigned to SDC A, B, and C. For structures assigned to SDC D, E, or F, the application of this guide is limited to slab systems, foundations, and structural components not designated as part of the seismic-force-resisting system, but explicitly checked for the induced effects of the design displacements. The exception to this is the use of transverse reinforcement for concrete confinement with f_{yt} up to 100,000 psi (690 MPa) for special moment frames and special structural walls as permitted by [ACI 318](#).

Numerous studies ([Muguruma and Watanabe 1990](#); [Muguruma et al. 1991](#); [Sugano et al. 1990](#); [Budek et al. 2002](#); [Stephan et al. 2003](#)) conducted tests of beams with strength reinforcement (f_{yt} up to and greater than 120,000 psi

[827 MPa]) show that there is no detriment to using high-strength transverse reinforcement as column confinement. The test program by [Lepage et al. \(2008\)](#) has demonstrated successful use of high-strength steel (HSS) in beams and columns subjected to reverse cyclic loadings ([Rautenberg et al. 2010](#)). With the exception of using f_{yt} up to 100,000 psi (690 MPa) for confinement, the following two recommendations are appropriate until comprehensive and systematic tests performed using **ASTM A1035/A1035M** (CS) bars justify its use in structural members of buildings assigned to SDC D and higher:

1. ASTM A1035/A1035M (CS) bars should not be used as longitudinal reinforcement in a structural member that is part of the seismic-force-resisting system of a building assigned to SDC D, E, or F. The use of ASTM A1035/A1035M (CS) bars as transverse reinforcement is permitted, provided f_{yt} is limited to 60,000 psi (420 MPa) for computing shear strength.

2. If Grade 60 (420) bar is used for column longitudinal reinforcement and ASTM A1035/A1035M (CS) bar is used for beam longitudinal reinforcement in an intermediate moment frame, it should be ensured that the beam-column joints exhibit strong column-weak beam behavior because the actual flexural strengths of beams using ASTM A1035/A1035M (CS) reinforcement exceed the nominal flexural strengths computed by the simplified method in [Chapter 4](#) of this guide. In addition, the required shear strength of the beam should be determined based on the nominal moment M_n obtained from nonlinear analysis, as given in [Appendix B](#) of this guide.

10.2—Development and lap splice length

Design for tension development and splices of ASTM A1035/A1035M (CS) reinforcement should follow ACI 318. Some of the provisions require additional testing and evaluation for validation, as noted in the following.

There have been several investigations on bond and development length of ASTM A1035/A1035M bars of different sizes and with different concrete strengths. Earlier studies include 130 bond tests of beam-end specimens ([Ahlborn and DenHartigh 2002](#)), 15 pullout tests ([El-Hacha and Rizkalla 2002](#)), and six beam tests with unconfined spliced bars ([Peterfreund 2003](#)). Although these studies used pre-standardization high-strength bars, the bar deformations have not changed and the research results are still relevant. The results of these investigations indicated that the development length computed by Eq. (12-1) of **ACI 318-99** and **ACI 318-02** was sufficient for pre-standardized ASTM A1035/A1035M (CS) bars to develop the 0.2 percent offset yield strength of 120,000 psi (830 MPa).

However, a coordinated research program of bond tests of 69 large-scale specimens using ASTM A1035/A1035M bars ([Seliem et al. 2009](#)) indicated that ACI 318 should be used to predict only the strength of confined splices, whereas a similar equation proposed by **ACI 408R** provides a better strength estimate for both unconfined and confined splices using a strength reduction factor ϕ of 0.82.

Based on the aforementioned studies of ACI 318, it is found applicable for calculating the development and splice lengths of ASTM A1035/A1035M (CS) bars to develop the specified minimum yield strength of 100,000 psi (690 MPa) if the bars are confined. ACI 318 should not be applied to unconfined bars, as discussed in the following.

Alternatively, for both confined and unconfined spliced bars, the equation in ACI 408R with a revised strength reduction factor ϕ of 0.80, instead of 0.82, is recommended. The equations in ACI 408R for development length in the customary in.-lbf units are shown as Eq. (10.2a) through (10.2e).

$$\ell_d = \frac{\left(\frac{f_y}{f'_c}\right)^{1/4} - \phi 2400\omega}{\phi 76.3 \left(\frac{c_b\omega + K_{tr}}{d_b}\right)} \alpha \beta_c \lambda d_b \quad (\text{in.-lbf}) \quad (10.2a)$$

$$\ell_d = \frac{\left(\frac{f_y}{f'_c}\right)^{1/4} - \phi 57.4\omega}{\phi 1.83 \left(\frac{c_b\omega + K_{tr}}{d_b}\right)} \alpha \beta_c \lambda d_b \quad (\text{SI})$$

where $\alpha = 1.3$ for top cast bars, $\beta_c = 1.0$ for uncoated bars, $\lambda = 1.0$ for normalweight concrete, and

$$K_{tr} = (0.52 t_r t_d A_{tr} / s n) \sqrt{f'_c} \quad (\text{in.-lbf}) \quad (10.2b)$$

$$K_{tr} = (6.26 t_r t_d A_{tr} / s n) \sqrt{f'_c} \quad (\text{SI})$$

$$t_r = 9.6 R_r + 0.28 \leq 1.72 \quad (10.2c)$$

$$t_d = 0.78 d_b + 0.22 \quad (\text{in.-lbf}) \quad (10.2d)$$

$$t_d = 0.03 d_b + 0.22 \quad (\text{SI})$$

$$\left(\frac{c_b\omega + K_{tr}}{d_b}\right) \leq 4.0 \quad (10.2e)$$

Although no test data are available, design for development and splices of ASTM A1035/A1035M (CS) Grade 100 (690) bars in compression may follow ACI 318, but the specified minimum yield strength f_y in compression is limited to 80,000 psi (550 MPa).

Tests by [Harries et al. \(2010\)](#) have shown that **ASTM A1035/A1035M** (CS) bars with 90- and 180-degree standard hooks can develop tensile bar stresses of at least 140,000 psi (965 MPa) and, in many cases, the strength of the bar. Instead of hooked bars, headed bars may be considered as an alternative. Section 10.3 of this guide discusses development of headed ASTM A1035/A1035M (CS) bars.

10.3—Mechanically spliced bars and headed bars

According to 10.2 of this guide, designs with f_y of 100,000 psi (690 MPa) for ASTM A1035/A1035M (CS) bars require

relatively long development or splice lengths. Because these lengths may be uneconomical or impractical, the designer may consider using mechanical splices (mechanical connections) and mechanically headed bars.

ACI 318 provides minimum tensile strength requirements for mechanical splices and specifies these requirements as $1.25f_y$ for the Type 1 splice and $1.0f_u$ for the Type 2 splice. Accordingly, the requirements for Type 1 mechanical splices are intended to avoid a splice failure when the reinforcement is subject to expected stress levels in yielding regions. The additional requirement for a Type 2 mechanical splice is intended to result in a mechanical splice capable of sustaining inelastic strains through multiple cycles.

These requirements are functions of the specified strengths of the reinforcing bars being spliced (not actual bar strengths) and do not include any ductility requirements. These functions of the specified strengths were historically developed based on reinforcement with stress-strain behavior that is sharply yielding with a definite yield plateau. For gradually yielding reinforcement, such as ASTM A1035/A1035M (CS) reinforcement, strain-based requirements are more appropriate. Equivalent strain criteria for the typical Type 1 and Type 2 mechanical splices (ACI 318) can be developed for ASTM A1035/A1035M (CS) reinforcement by examination of the actual stress-strain behavior for sharply-yielding reinforcement, and establishing a strain that corresponds to the specified minimum strengths for the mechanical splice. On this basis, the Type 1 mechanical splice minimum strength corresponds to a strain in the spliced bars that is beyond the yield plateau and into the onset of strain hardening; the onset of strain hardening occurs at strains in the range of 1 to 2 percent for **ASTM A615/A615M** Grade 60 and **ASTM A706/A706M** Grade 60 reinforcement. The Type 2 mechanical splice minimum strength requirement corresponds to a strain in the spliced bars that is even larger and beyond the onset of strain hardening and into the strain hardening range, corresponding to strains in the range of 2.5 to 4 percent for ASTM A615/A615M Grade 60 and ASTM A706/A706M Grade 60 reinforcement. Consequently, to achieve stress-strain performance similar to those of the strength-based requirements for mechanical splices specified in ACI 318, new designated Type A and B mechanical splices are recommended by this guide with monotonic strain limits based on strain of the bars.

Based on the historical statistical analysis of ASTM A1035/A1035M (CS) bars' test data, it is recommended that bars be mechanically spliced to develop 2 percent static tensile strain for Type A connections and 3 percent static tensile strain for Type B connections. Testing of the mechanical splices for strain capacity should be performed under static tensile test loading, as any multiple inelastic cyclic excursions beyond the actual yield point of the bar (measured through 0.2% offset), such as defined in **ICC-ES AC133**, **ICC-ES AC429**, **ASTM A1034/A1034M**, or **ISO 15835-2**, will strain harden the reinforcing bar and reduce the strain capacity of the reinforcing steel. Inelastic cyclic testing is therefore not recommended for use in evaluation

of strain capacity of mechanically spliced A1035/A1035M (CS) Grade 100 (690) bars. Only test samples cut from original straight lengths of reinforcing steel, not from coil or post-fabricated steel, should be used for tensile strain evaluation.

ASTM A1035/A1035M (CS) bar can be developed in tension by using a mechanically attached head, as is the case with other ASTM types and grades of deformed reinforcing bars. Provisions for computing development length of headed deformed reinforcing bars were introduced in ACI 318, but they are currently limited to reinforcing bars with f_y not exceeding 60,000 psi (420 MPa) and, thus, precludes the use of these provisions with ASTM A1035/A1035M (CS) bars and other reinforcing bars with f_y in excess of 60,000 psi (420 MPa), such as ASTM A615/A615M Grade 75 (520) and Grade 80 (550) bars. However, headed ASTM A1035/A1035M (CS) bars may be used in accordance with ACI 318 "...provided that test results showing the adequacy of such attachment or device are approved by the building official."

ACI 439.3R summarizes available mechanical splices. Some manufacturers use certain methods to produce mechanical splices or headed bars that excessively heat the reinforcing bar, causing an unfavorable alteration to the microstructure of ASTM A1035/A1035M (CS) steel. The applicability of a mechanical splice or headed bar should be determined by consulting the manufacturer of the mechanical splices for the applicability with ASTM A1035/A1035M (CS) bar.

Mechanical splices and headed bars should be placed in accordance with the design drawings or as approved by the licensed design professional.

10.4—Bending and welding of bars

Experiments and field applications have shown that the ASTM A1035/A1035M (CS) reinforcing bars can be bent into standard hooks by following the requirements of ACI 318. Note, however, that because of the strain-hardening characteristics of the material, it is usually necessary to overbend the bar to achieve the desired bent angle. As it is with other reinforcing bars straightening of a bent bar embedded in concrete should be avoided, as it may cause the bar to break.

Although the chemistry (other than the chromium content) of ASTM A1035/A1035M (CS) steel is similar to that of ASTM A706/A706M steel, ASTM A1035/A1035M bars should not be welded. The heat of welding alters the microstructure of the steel. Welding of ASTM A1035 bars should be approached with caution because no specific provisions have been included in the ASTM A1035 to enhance weldability of the bars.

10.5—Use of ASTM A1035/A1035M (CS) bars with ASTM A615/A615M bars

ASTM A1035/A1035M (CS) bars may be used in direct contact with **ASTM A615/A615M** bars in a concrete structure because the difference in corrosion potential between these dissimilar metals is approximately 100 mV, which is well within the acceptable value of 250 mV (**ASTM A615/A615M**).

Table 11—Specified minimum yield strengths for design of members using ASTM A1035/A1035M (CS) reinforcement

Type of member	Longitudinal reinforcement		Transverse reinforcement		
	Tension, psi (MPa)	Compression, psi (MPa)	Shear, psi (MPa)	Torsion, psi (MPa)	Confinement, psi (MPa)
Beams and one-way slabs	100,000 (690)	80,000 (550)	80,000 (550)	60,000 (420)	N/A
Columns	100,000 (690)	80,000 (550)	80,000 (550)	60,000 (420)	100,000 (690)
Tension ties	80,000 (550)	N/A	N/A	N/A	N/A
Compression struts	N/A	80,000 (550)	N/A	N/A	N/A
Two-way slabs	100,000 (690)	80,000 (550)	60,000 (420)	60,000 (420)	N/A
Walls	100,000 (690)	80,000 (550)	80,000 (550)	N/A	100,000 (690)
Footings and pile caps	100,000 (690)	80,000 (550)	80,000 (550)	60,000 (420)	N/A
Mat foundations	100,000 (690)	80,000 (550)	80,000 (550)	N/A	N/A

CHAPTER 11—SUMMARY

Presented in this document are guidelines for using ASTM A1035/A1035M (CS) Grade 100 (690) deformed reinforcing bars in reinforced concrete members, including beams, columns, slab systems, walls, footings, and mat foundations, for Seismic Design Category (SDC) A, B, or C. A structure assigned to SDC A, B, or C is required to be designed for all applicable gravity and environmental loads. In the case of SDC A structures, seismic forces are notional structural integrity forces. This document addresses all design required for SDC A, B, and C structures. Application of these guidelines for SDC D, E, or F is limited to slab systems, foundations, and structural components that are not designed as part of seismic-force-resisting system. Both advantages and disadvantages of using ASTM A1035/A1035M (CS) deformed bars are discussed in [Chapter 1](#) of this guide. Design examples included in [Appendixes A and B](#) of this guide illustrate design procedures and proper applications of the recommended design criteria.

Tests indicated that ASTM A1035/A1035M (CS) Grade 100 (690) reinforcing bars exhibit a linear stress-strain relationship up to a proportional limit ranging from 60,000 to 80,000 psi (420 to 550 MPa) with a modulus of elasticity of 29,000,000 psi (200,000 MPa), but without a well-defined yield point. The yield strength determined by the 0.2 percent offset method exceeds 115,000 psi (790 MPa). The tensile strength exceeds 155,000 psi (1070 MPa) with corresponding strain ranging from 0.04 to 0.06. The elongation in 8 in. (200 mm) ranges from 0.08 to 0.13. An approximate lower-bound representation of the stress-strain curve of the Grade 100 (690) bar is given in Eq. (3.4a) through (3.4c) of this guide.

A simplified design method for flexural tension is developed based on an idealized “elastic-perfectly plastic” stress-strain curve with a yield plateau at $f_y = 100$ ksi (690 MPa). Because the yield plateau neglects the effect of strain hardening of the steel, the tension-controlled and compression-controlled strain limits and the strength reduction factor ϕ are adjusted to ensure satisfactory member behavior. The adjusted tension and compression strain limits are 0.009 and 0.004, respectively, and the adjusted strength reduction

factor ϕ is given by Eq. (4.4) of this guide. For flexural and direct compression, f_y is taken as 80,000 psi (550 MPa) so that the compatible strain in concrete under compression will not exceed the maximum value of 0.003. Due to a lack of research data and experience, moment redistribution for flexural design using ASTM A1035/A1035M (CS) reinforcement is not recommended.

Flexural design using ASTM A1035/A1035M (CS) Grade 100 (690) bars with $f_y = 100,000$ psi (690 MPa) would result in higher steel stress at service load conditions, so the designer should exercise caution in applying the [ACI 318](#) provisions on crack and deflection controls, especially for shallow members with reinforcement ratio less than 1 percent and where diagonal shear cracking is expected at the service load level.

For shear design of beams, ASTM A1035/A1035M (CS) Grade 100 (690) bars can be used as shear reinforcement with $f_y = 80,000$ psi (550 MPa), but at service load levels, slightly increased shear crack widths should be expected. In addition, the concrete contribution to shear strength, V_c , of lightly reinforced deep beams with longitudinal reinforcement ratio less than 1 percent may be less than predicted by ACI 318. It is advisable that such beams should contain minimum shear reinforcement as determined by ACI 318.

Development and splice length for ASTM A1035/A1035M (CS) reinforcement in tension may be determined by ACI 318, provided the splice is confined. Better predictions for both unconfined and confined splices, however, can be obtained by using Eq. (10.2a) through (10.2e) of this guide, which are recommended in [ACI 408R](#). Splice lengths shall comply with ACI 318 if ACI 408R-modified equations provide shorter lengths.

Design of slab systems using ASTM A1035/A1035M (CS) Grade 100 (690) steel bars should follow ACI 318 provisions for shear in one-way slabs and for two-way slabs. Using $f_y = 100,000$ psi (690 MPa) for tension reinforcement in one-way slabs, the section may become lightly reinforced. If ρ is less than 1 percent, it is recommended that the reduced concrete contribution to shear strength, V_c , as given in Eq. (6.2) of this guide, be considered. For two-way slabs, the slab thickness should be determined according to

ACI 318 based on actual deflection calculations. When the member is lightly reinforced (generally with ρ less than 1 percent), it is recommended that direct deflection calculations be used (Yang et al. 2010). If shear reinforcement is required to provide sufficient shear strength for a two-way slab system, the specified minimum yield strength, f_{yt} , for shear reinforcement should be limited to 60,000 psi (420 MPa) because adequate research data are not available to justify a higher value.

Designs for columns, walls, footings and pile caps, and mat foundations using ASTM A1035/A1035M (CS) reinforcement are discussed in Chapters 5, 7, 8, and 9, respectively, of this guide. There are no exceptions from the requirements of ACI 318 for these structural members other than the adjustments of specified minimum yield strength f_y for tension and compression reinforcements, and f_{yt} for shear reinforcement. Some other design considerations, including seismic design limitations, spliced and headed bars, bending of bars, and use of ASTM A1035/A1035M (CS) bars together with ASTM A615/A615M bars, are discussed in Chapter 10 of this guide.

The recommended specified minimum yield strengths for design of various structural members using ASTM A1035/A1035M (CS) reinforcement are summarized for convenient reference in Table 11.

CHAPTER 12—REFERENCES

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- ACI 318-95—Building Code Requirements for Structural Concrete and Commentary
- ACI 318-99—Building Code Requirements for Structural Concrete and Commentary
- ACI 318-02—Building Code Requirements for Structural Concrete and Commentary
- ACI 318-05—Building Code Requirements for Structural Concrete and Commentary
- ACI 318-08—Building Code Requirements for Structural Concrete and Commentary
- ACI 318-11—Building Code Requirements for Structural Concrete and Commentary
- ACI 318-14—Building Code Requirements for Structural Concrete and Commentary
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- ACI 439.3R-07—Types of Mechanical Splices for Reinforcing Bars

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ACI ITG-6R-10—Design Guide for the Use of ASTM A1035/A1035M Grade 100 (690) Steel Bars for Structural Concrete

ASTM International

- A615/A615M-18—Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement
- A706/A706M-16—Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement
- A1034/A1034M-10a(2015)—Standard Test Methods for Testing Mechanical Splices for Steel Reinforcing Bars
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APPENDIX A—DESIGN EXAMPLES

A.1—Introduction

The following design examples illustrate the various design procedures recommended in this guide. The problems selected for these examples are from a Portland Cement Association (PCA) publication, *PCA Notes on ACI 318-05 Building Code Requirements for Structural Concrete* (PCA 2005). The term “CS” used in the examples refers to ASTM A1035/A1035M (CS) Grade 100 steel reinforcement.

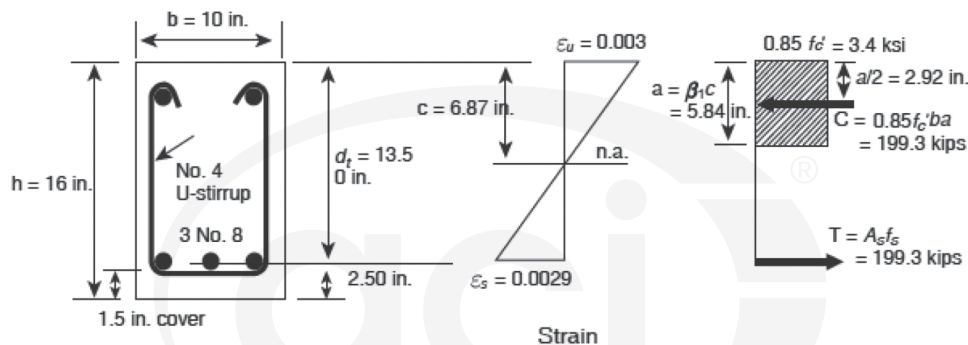
Examples are numbered as the chapter number of this guide. Examples are grouped to illustrate the various aspects of the recommended design procedures, not in ascending numerical order. Examples appear in customary in.-lb units except Examples 6.1(SI) and 4.6(SI), which are the same as Example 6.1 and Example 4.6, but in SI units for purposes of illustration.

Where appropriate, commentaries are added to indicate the differences in design when using ASTM A1035/A1035M (CS) bars as opposed to the ASTM A615/A615M Grade 40 and Grade 60 steel bars.

A.2—Design examples

Example 4.1—Moment strength in beams using equivalent rectangular stress distribution

Similar to Example 6.1 in the *PCA Notes*, this example represents direct bar-for-bar replacement of Grade 60 reinforcing bars by CS bars.



(a) For the beam section shown, calculate moment strength based on static equilibrium using the ACI equivalent rectangular stress distribution. Assume $f'_c = 4000$ psi and $f_y = 100,000$ psi. For simplicity, neglect hanger bars in the compression zone.

(b) Calculate the moment strength for the same concrete section reinforced with two No. 8 high-strength bars ($f_y = 100,000$ psi) instead of three No. 8 Grade 60 bars. For simplicity, neglect hanger bars in the compression zone.

This example represents replacement of the longitudinal tension force, $A_s f_y$, provided by the conventional reinforcement, which results in the use of fewer high-strength reinforcing bars.

Calculations and discussion

(a) Section reinforced with three No. 8, Grade 100 tension bars

1. Define rectangular concrete stress distribution.

$$d = d_t = 16 - 2.5 = 13.50 \text{ in.}$$

$$A_s = 3 \times 0.79 = 2.37 \text{ in.}^2$$

$$\text{Assuming } \epsilon_s < \epsilon_y, (\epsilon_y = 100/29,000 = 0.00345)$$

$$T = A_s E_s \epsilon_s = 2.37 \times 29,000 \times \epsilon_s = 68,730 \epsilon_s$$

$$C = 0.85 f'_c b a = 0.85 \times 4 \times 10 \times a = 34a$$

$$\epsilon_s = \left(\frac{d_t - c}{c} \right) 0.003 = \left(\frac{d_t - a/\beta_1}{a/\beta_1} \right) 0.003$$

Satisfying equilibrium $T = C$

$$68,730 \left(\frac{13.50 - a/0.85}{a/0.85} \right) 0.003 = 34a$$

$$40a^2 + 242.58a - 2783.57 = 0$$

$$a = 5.84 \text{ in.}$$

2. Determine net tensile strain ϵ_s and ϕ .

$$c = a/\beta_1 = 5.84/0.85 = 6.87 \text{ in.}$$

	c , in.	ϕ , curvature
Grade 60, PCA	4.92	0.00061
Grade 100, CS	6.87	0.00044

$$\epsilon_s = \left(\frac{d_t - c}{c} \right) 0.003 = \left(\frac{13.50 - 6.87}{6.87} \right) 0.003 = 0.0029 < 0.004$$

(Refer to Section 4.4 of this guide.)

Therefore, the section is compression-controlled. This also confirms that $\epsilon_s < \epsilon_y = 0.00345$.

$$\phi = 0.65$$

(Refer to Section 4.4 of this guide.)

3. Determine nominal moment strength M_n and design moment strength ϕM_n .

$$M_n = A_s E_s \epsilon_s (d_t - (a/2)) = 68,730 \times 0.0029 \times (13.50 - 2.92) = 2108.8 \text{ in.-kip} = 175.7 \text{ ft-kip}$$

$$\phi M_n = 0.65(175.7) = 114.2 \text{ ft-kip}$$

	M_n , ft-kip	ϕM_n , ft-kip
PCA, Grade 60	135.2	121.9
CS, Grade 100	175.7	114.2

4. Minimum reinforcement.

$$A_{s,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d \geq \frac{200b_w d}{f_y}$$

Because $f'_c = 4000$ psi, $3\sqrt{4000} = 189.7 < 200$. Therefore, $200b_w d/f_y$ governs

$$\frac{200 b_w d}{f_y} = \frac{200 \times 10 \times 13.50}{100,000} = 0.27 \text{ in.}^2$$

$$A_s (\text{provided}) = 2.37 \text{ in.}^2 > A_{s,min} = 0.27 \text{ in.}^2 \quad \text{OK}$$

The nominal moment strength of the beam reinforced with CS is 30 percent higher than the nominal moment strength of 135.2 ft-kip for the same beam reinforced with Grade 60 steel (*PCA Notes*, Example 6.1). The change of failure mode from tension-controlled to compression-controlled, however, reduces the ϕ value, resulting in the beam design moment with CS bars 6 percent lower than the beam reinforced with Grade 60 steel bars.

(b) Section reinforced with two No. 8, Grade 100 tension bars.

1. Define rectangular concrete stress distribution.

$$d = d_t = 16 - 2.5 = 13.50 \text{ in.}$$

$$A_s = 2 \times 0.79 = 1.58 \text{ in.}^2$$

Assuming $\epsilon_s > \epsilon_y$

$$T = A_s f_y = 1.58 \times 100 = 158 \text{ kip}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{158}{0.85 \times 4 \times 10} = 4.65 \text{ in.}$$

2. Determine net tensile strain ϵ_s and ϕ .

$$c = a/\beta_1 = 4.65/0.85 = 5.47 \text{ in.}$$

	c, in.	ϕ , Curvature
PCA, 3 No. 8 Grade 60	4.92	0.00061
CS, 3 No. 8 Grade 100	5.47	0.00055

$$\epsilon_s = \left(\frac{d_t - c}{c} \right) 0.003 = \left(\frac{13.50 - 5.47}{5.47} \right) 0.003 = 0.0044 > 0.004$$

Therefore, the section is in the transition zone. This also confirms that $\epsilon_s > \epsilon_y = 0.00345$

$$\phi = 0.45 + 50\epsilon_s = 0.45 + 50 \times 0.0044 = 0.67$$

(Refer to **Section 4.4** of this guide.)

3. Determine nominal moment strength M_n and design moment strength ϕM_n .

$$M_n = A_s f_y \left(d_t - \frac{a}{2} \right) = 158 \times (13.50 - 2.33) = 1764.9 \text{ in.-kip} = 147.1 \text{ ft-kip}$$

$$\phi M_n = 0.67(147.1) = 98.5 \text{ ft-kip}$$

	M_n , ft-kip	ϕM_n , ft-kip
PCA, 3 No. 8 Grade 60	135.2	121.9
CS, 2 No. 8 Grade 100	147.1	98.5

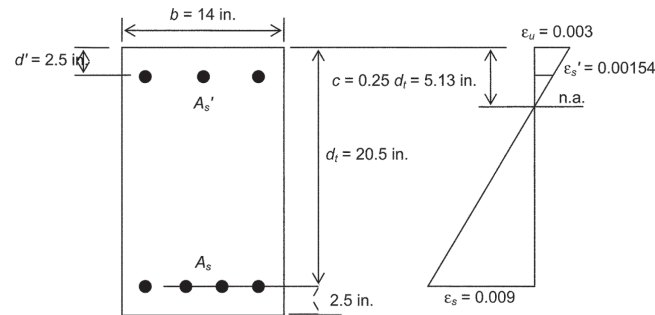
4. Minimum reinforcement.

$$A_s (\text{provided}) = 1.58 \text{ in.}^2 > A_{s,min} = 0.27 \text{ in.}^2 \quad \text{OK}$$

In this case, even though there is a reduction of one-third of the longitudinal steel area, the failure mode is in the transition region between compression control and tension control. The corresponding reduction of the ϕ value causes the beam design moment with two No. 8 CS bars to be 19 percent lower than for the beam reinforced with three No. 8 Grade 60 steel bars.

Example 4.2—Design of beam with compression reinforcement

Similar to Example 6.2 in *PCA Notes*, this example represents replacement of Grade 60 reinforcing bars by high-strength bars. This example demonstrates the benefit of using high-strength concrete with high-strength bars.



(a) A beam cross section is limited to the maximum size shown. Determine the required area of reinforcement for a factored moment $M_u = 516$ ft-kip; $f_c' = 4000$ psi; $f_y = 100,000$ psi (tension); and $f_y = 80,000$ psi (compression).

(b) Determine the required area of reinforcement for the same section using $f_c' = 8000$ psi.

(c) Determine the required area of reinforcement for the same section using $f_c' = 8000$ psi and $f_y = 60,000$ psi for comparison.

Calculations and discussion**(a) Grade 100 steel and $f_c' = 4000$ psi.**

1. Determine the required nominal strength.

Design the section to be tension controlled, $\epsilon_s = 0.009$ and $\phi = 0.9$.

(Refer to [Section 4.4](#) of this guide.)

$$M_n = M_u / \phi = 516 / 0.9 = 573 \text{ ft-kip}$$

2. Determine maximum moment without compression reinforcement.

$$c = 0.25d = 0.25 \times 20.5 = 5.13 \text{ in.}$$

$f_c' = 4000$ psi	c , in.	ϕ , curvature
Grade 60, PCA	7.69	0.00040
Grade 100, CS	5.13	0.00058

$$a = \beta_1 c = 0.85 \times 5.13 = 4.36 \text{ in.}$$

$$C = T = 0.85 f_c' a b = 0.85 \times 4 \times 4.36 \times 14 = 208 \text{ kip}$$

$$M_n = T \left(d_t - \frac{a}{2} \right) = 208 \left(20.5 - \frac{4.36}{2} \right) = 3811 \text{ in.-kip} = 318 \text{ ft-kip}$$

3. Required area of tension steel to develop M_n .

$$A_{s,nl} = 208 / 100 = 2.08 \text{ in.}^2$$

4. Additional moment ($573 - 318 = 255$ ft-kip) should be developed in T-C couple between tension steel and compression steel.

$$\text{Additional tension steel required: } \Delta A_s = \frac{255(12)}{(20.5 - 2.5)(100)} = 1.70 \text{ in.}^2$$

$$\text{Total tension steel required: } A_s = 2.08 + 1.70 = 3.78 \text{ in.}^2$$



The strain in the compression steel is 0.00154, as shown in the aforementioned strain diagram. The corresponding steel stress is 44.6 ksi.

$$\text{Compression steel required: } A'_s = \frac{255(12)}{(20.5 - 2.5)(44.6)} = 3.81 \text{ in.}^2$$

PCA Notes solution: using Grade 60 steel, $A_s = 6.59 \text{ in.}^2$ and $A'_s = 1.43 \text{ in.}^2$

Total area of reinforcement required for this design, including the tension steel of 3.78 in.^2 and the compression steel of 3.81 in.^2 , is 7.59 in.^2 . The original design in the *PCA Notes*, which uses Grade 60 steel, required a tension steel of 6.59 in.^2 and a compression steel of 1.43 in.^2 for a total 8.02 in.^2 of reinforcing steel. The use of CS results in a reinforcing steel reduction of 5 percent. The total reduction is small because the compression steel increases to offset the smaller compressive force provided by the normal-strength concrete. High-strength steel (HSS) for flexural capacity is most advantageous when used with high-strength concrete, as illustrated in the following section.

(b) Grade 100 steel and $f'_c = 8000 \text{ psi}$

1. Determine the required nominal strength.

$$M_n = M_u / \phi = 516 / 0.9 = 573 \text{ ft-kip}$$

2. Determine maximum moment without compression reinforcement.

$$c = 0.25d = 0.25 \times 20.5 = 5.13 \text{ in.}$$

$f'_c = 8000 \text{ psi}$	$c, \text{ in.}$	$\phi, \text{ curvature}$
Grade 60, PCA	5.98	0.00050
Grade 100, CS	5.13	0.00058

$$a = \beta_1 c = 0.65 \times 5.13 = 3.33 \text{ in.}$$

$$C = T = 0.85f'_c ab = 0.85 \times 8 \times 3.33 \times 14 = 317 \text{ kip}$$

$$M_{nt} = 5971 \text{ in.-kip} = 498 \text{ ft-kip}$$

3. Required area of tension steel to develop M_{nt} .

$$A_{s,nt} = 317 / 100 = 3.17 \text{ in.}^2$$

4. Additional moment ($573 - 498 = 75 \text{ ft-kip}$) should be developed in T-C couple between tension steel and compression steel.

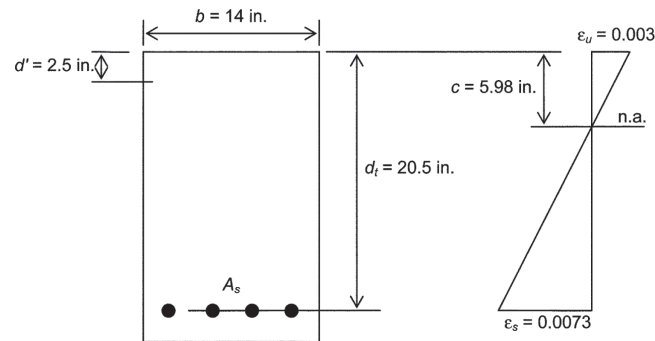
$$\text{Additional tension steel required: } \Delta A_s = \frac{75(12)}{(20.5 - 2.5)(100)} = 0.05 \text{ in.}^2$$

$$\text{Total tension steel required: } A_s = 3.17 + 0.50 = 3.67 \text{ in.}^2$$

$$\text{Compression steel required: } A'_s = \frac{75(12)}{(20.5 - 2.5)(44.6)} = 1.12 \text{ in.}^2$$

By using Grade 100 steel with 8000 psi concrete, both the tension and the compression steels are reduced.

(c) $f'_c = 8000 \text{ psi}$ and $f_y = 60,000 \text{ psi}$



1. Determine the required nominal strength.

$$M_n = M_u / \phi = 516 / 0.9 = 573 \text{ ft-kip} = 6876 \text{ in.-kip}$$

2. Define rectangular concrete stress distribution.

$$C = T = 0.85 f'_c ab$$

$$M_n = T \left(d_t - \frac{a}{2} \right) = 0.85 f'_c ab \left(d_t - \frac{a}{2} \right)$$

$$6876 = 0.85 \times 8 \times a \times 14 \times (20.5 - a/2)$$

$$47.6 a^2 - 1951.6 a + 6876 = 0$$

$$a = 3.89 \text{ in.}$$

3. Determine required area of tension steel.

$$c = a / \beta_1 = 3.89 / 0.65 = 5.98 \text{ in.}$$

$$\epsilon_s = \left(\frac{d_t - c}{c} \right) (0.003) = \left(\frac{20.5 - 5.98}{5.98} \right) 0.003 = 0.0073 > 0.00207 = e_y$$

$$A_s = T / f_y = 0.85 f'_c ab / f_y = 0.85 \times 8 \times 3.89 \times 14 / 60 = 6.17 \text{ in.}^2 \text{ No compression steel is required.}$$

PCA Notes solution: using Grade 60 steel with 4000 psi concrete, $A_s = 6.59 \text{ in.}^2$ and $A_s' = 1.43 \text{ in.}^2$

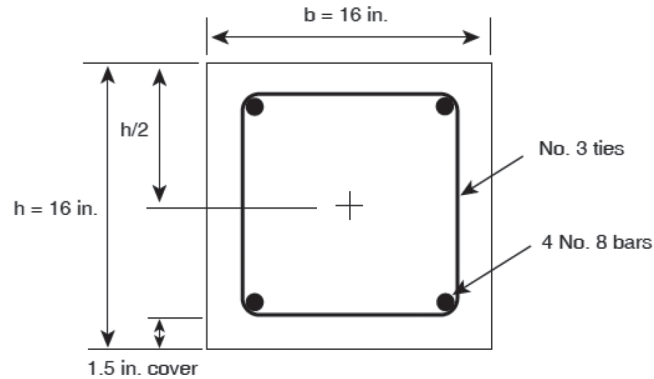
Using CS for flexural capacity is most beneficial when used with high-strength concrete, as illustrated in this example. Using CS requires a total area of reinforcement, including tension and compression steel, of 4.79 in.^2 . Grade 60 reinforcement requires a total of 6.17 in.^2 of steel. The design using CS results in a 25 percent reduction of the required flexural reinforcement.

The design using Grade 60 steel does not require compression steel but using CS does. The tension-controlled strain limit for CS is 0.009 instead of 0.005 for Grade 60 steel. This high strain level results in a smaller compression zone depth c . Consequently, the high-strength compression reinforcement should provide additional compression force if $c > d'$.

	$A_s, \text{in.}^2$	$A_s', \text{in.}^2$	Total $A_s, \text{in.}^2$
Grade 60, PCA, $f'_c = 4000 \text{ psi}$	6.59	1.43	8.02
Grade 100, CS, $f'_c = 4000 \text{ psi}$	3.78	3.81	7.59
Grade 60, PCA, $f'_c = 8000 \text{ psi}$	6.17	Not needed	6.17
Grade 100, CS, $f'_c = 8000 \text{ psi}$	3.67	1.12	4.79

Example 5.1—Axial load-moment strength, P_n and M_n , for given strain conditions

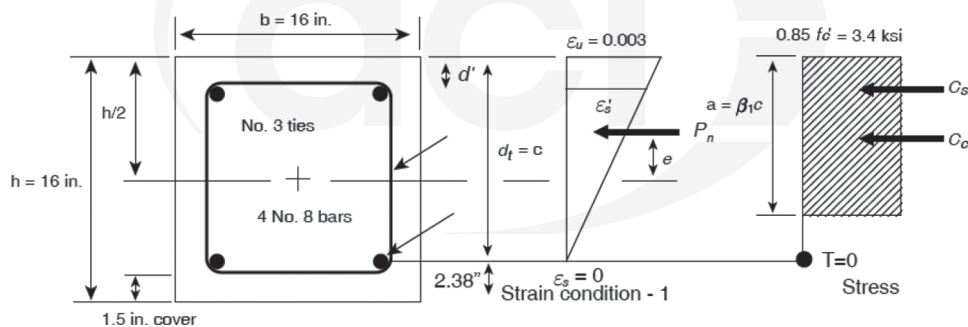
Similar to Example 6.4 in *PCA Notes*, this example represents direct bar-for-bar replacement of Grade 60 reinforcing bars by high-strength bars.



For the column section shown, calculate the load-moment strength, P_n and M_n , for four strain conditions.

- (1) Bar stress near tension face of member equal to zero, $f_s = 0$
- (2) Bar stress near tension face of member equal to $0.5f_y$ ($f_s = 0.5f_y$)
- (3) At limit for compression-controlled section ($\epsilon_t = 0.004$)
- (4) At limit for tension-controlled section ($\epsilon_t = 0.009$)

Use $f'_c = 4000$ psi, $f_y = 100,000$ psi in tension and $f_y = 80,000$ psi in compression.

Calculations and discussion**(a) Axial load-moment strength, P_n and M_n , for Strain Condition 1: $\epsilon_s = 0$.****1. Define stress distribution and determine force values.**

$$d' = \text{Cover} + \text{No. 3 tie diameter} + d_b/2 = 1.5 + 0.375 + 0.5 = 2.38 \text{ in.}$$

$$d_t = 16 - 2.38 = 13.62 \text{ in.}$$

$$\text{Because } \epsilon_s = 0, c = d_t = 13.62 \text{ in.}$$

$$a = \beta_1 c = 0.85 \times 13.62 = 11.58 \text{ in., where } \beta_1 = 0.85 \text{ for } f'_c = 4000 \text{ psi.}$$

$$C_c = 0.85 f'_c b a = 0.85 \times 4 \times 16 \times 11.58 = 630 \text{ kip}$$

$$\text{Compression: } \epsilon_y' = f_y'/E_s = 80/29,000 = 0.00276$$

$$\text{Tension: } \epsilon_y = f_y/E_s = 100/29,000 = 0.00345$$

From strain compatibility