Flexural Design of Concrete Beams with High-Strength Reinforcement

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Abstract

This paper examines the ACI 318 Code methodology for flexural strength design of reinforced concrete beams with high-strength steel reinforcement conforming to ASTM A1035/A1035M-16. Tensile steel strains corresponding to strain limits for tension-controlled and transitional members are studied for beams with high- and normal-strength steel reinforcement. For normal strength steel reinforcement typically corresponding to Grade 60 (400) and Grade 75 (520), the minimum tension-controlled steel strain is 0.005. ACI flexural members may be designed with tensile strain as low as 0.004. To provide the same level of structural safety for concrete beams with high-strength reinforcement, the strain limits per ACI 318 Code will need to be increased as compared to beams with normal strength rebar. Having different ACI strain limits for high and normal strength steel reinforcement for members in flexure is necessary. However, this complicates the ACI resistance factor versus tensile steel strain relationship. To simplify the resulting relationship, a modification is proposed, which results in one simplistic expression that accounts for various strengths of grades of tensile steel reinforcement. The resulting ACI modification provides a clear and consistent means of ensuring that all flexural members are designed to be under-reinforced by increasing the limiting tensile strain based on the grade of the steel reinforcement.

The proposed modification to the strain limits for flexural members with high and normal strength reinforcement follows the procedures of ACI 318 and is discussed in detail herein, examining results performed by others. An ultimate strain in the tensile reinforcement is proposed, which eliminates the need to determine the minimum steel ratio and is more consistent with the strain limit approach for flexure in ACI 318. A numerical design example showing the application of the proposed modification is presented.

Introduction

According to IBC-2018, the maximum allowable design strength of nonprestressed tendon steel reinforcement is 80 ksi (550 MPa) (IBC, 2018). This maximum was established in ACI

318-71 by Committee 318 since this steel strength is nearly equal to the product of the ultimate strain in the concrete, 0.003, and the modulus of elasticity of steel, 29,000 ksi (200 GPa) (ACI Committee 318, 1971). For nearly 50 years, this limitation has been in practice. At the time of implementation, this requirement worked well since high-strength steel reinforcement was not available. In Europe, flexural rebar with a grade of 72.5 ksi (500 MPa) is commonly used, and the maximum permissible strength is 94.3 ksi (650 MPa).

In recent years, steel reinforcement with enhanced properties has been developed, which may lead to the possibility of designers using steel reinforcement with greater strengths as compared to traditional reinforcement (Faza, Kwok, & Salah, 2008; CRSI, 2017; Risser & Humphreys, 2008). High-strength steel reinforcement meeting the requirements of ASTM A1035/A1035M-16 have been developed (MMFX Technologies, 2012; MMFX Technologies, 2015; ASTM A1035/A1035M, 2016). Stress-strain characteristics, including pre-peak and post-peak behavior, are fundamentally different from the behavior of Grade 60 (400) steel reinforcement. Steels with enhanced strength properties are stronger. However, they tend to lack a well-defined yield point. Up to a strain of about 0.0015, the stress-strain relationship for reinforcing steels is predominantly linear. This applies to both normal and high-strength reinforcement. However, from a strain of about 0.0015 to 0.003, this relationship for high-strength steels gradually becomes nonlinear as compared to normal strength steels such as Grade 60 (400) and Grade 75 (520), which exhibit a more immediate plastic behavior. Normal strength reinforcing steels tend to exhibit an initial stress-strain relationship that is linear elastic and is immediately followed by plastic-like behavior. High-strength steels tend to exhibit a stress-strain relationship that is initially linear elastic but as strains increase becomes gradually nonlinear, eventually reaching a more plastic-like behavior.

Figure 1 presents typical stress-strain relationships for several different high-strength reinforcing steels, along with the stress-strain relationships for normal strength reinforcing steels such as Grade 60 (400) steel and Grade 75 (520). In addition, Figure 1 illustrates the stress-strain relationship for traditional prestressing steel. As shown in the figure, the stress-strain relationship for high-strength steel is predominantly characterized by an initial linear portion followed by a gradual nonlinear portion that eventually reaches a more plastic-like behavior. The absence of a distinct yield plateau is typical of high-strength steel. Like normal-strength reinforcing steels, high-strength reinforcing steels are capable of achieving ultimate strains of up to 0.050 and higher before final failure (Mast, Dawood, Rizkalla, & Zia, 2008). In general, as the strength of steel reinforcement increases, ductility decreases. High-strength steels have a reduced capacity to permanently deform before fracture and ultimately failure as compared to normal strength steel reinforcement. High-strength steels are capable of deforming considerably prior to failure and are considered to be ductile in comparison to normal concrete, which is a quasi-brittle material with limited ability to strain prior to the onset of cracking and eventual failure.

ACI 318-14 Code limits the maximum yield strength of steel reinforcement to 80 ksi (550 MPa) (ACI Committee 318, 2014). A simplified elastic-plastic relationship and method has been proposed for design purposes by Mast when designing flexural members with tensile steel reinforcement with a maximum yield strength up to 100 ksi (690 MPa) (Mast et al., 2008; Mast, 2007). The simplified model consists of an initial linear elastic portion with a modulus of elasticity of 29,000 ksi (200 GPa) followed by a perfectly plastic yield plateau with a yield strength of 100 ksi (690 MPa) as shown in Figure 2.

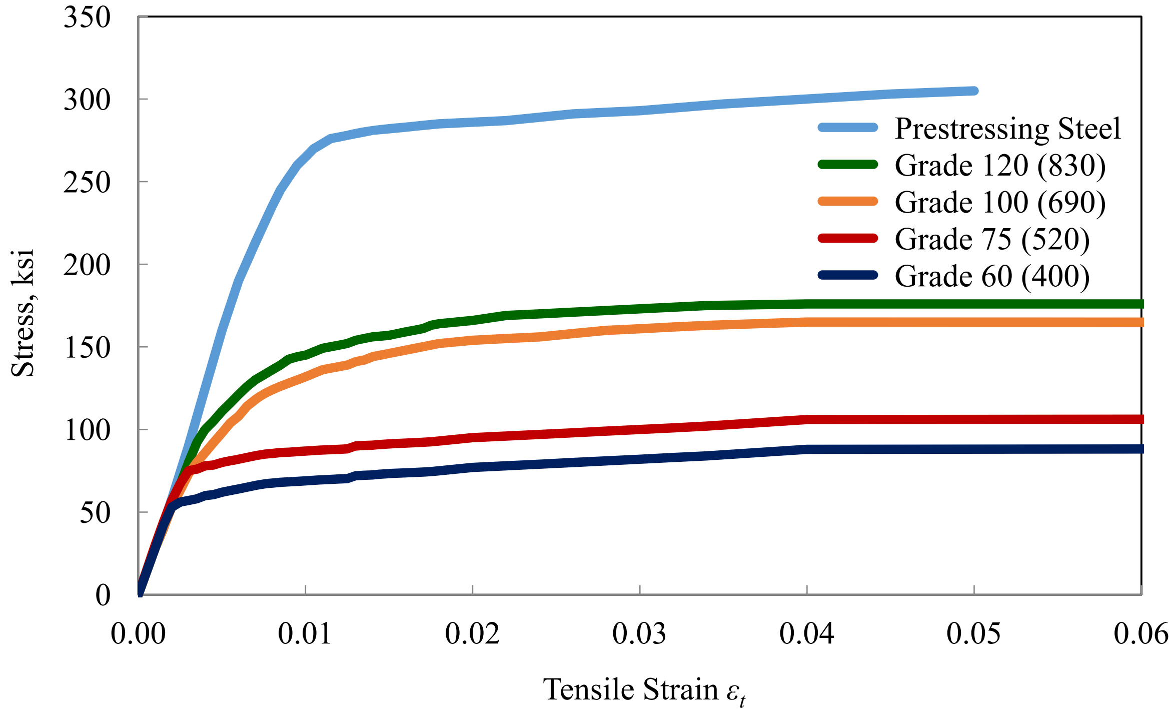

Figure 1. Material stress-strain behavior of reinforcing steels.

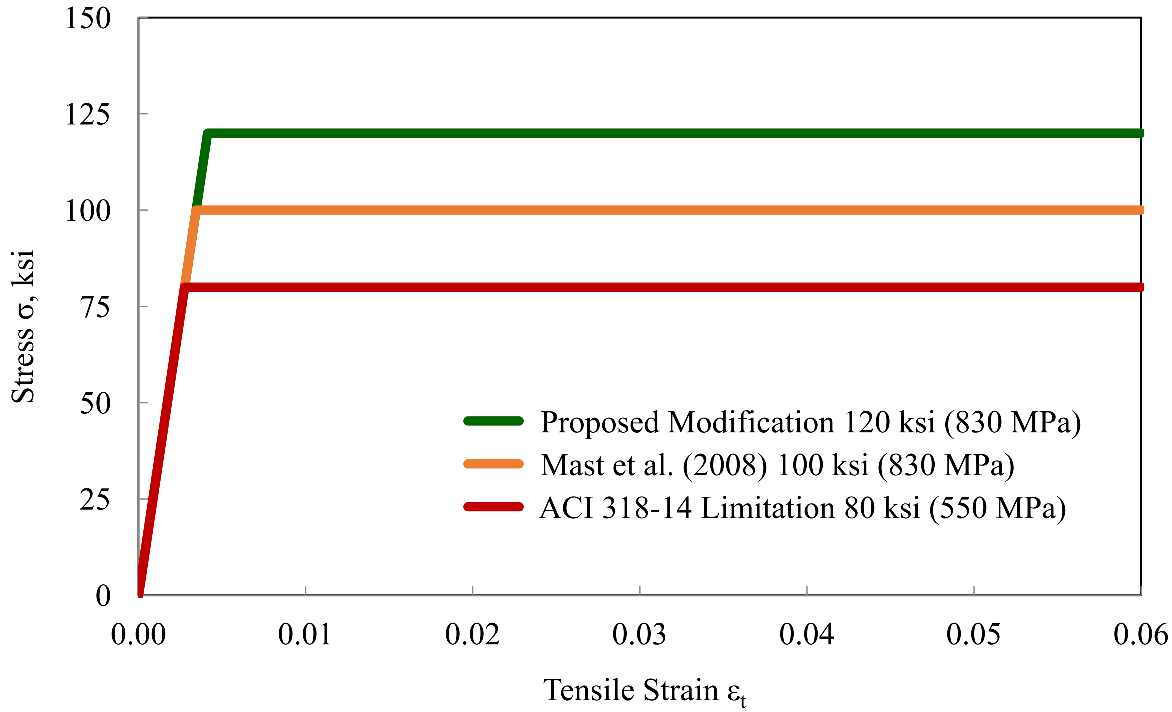

Figure 2. Elastic-plastic design relationship.

In the near future, Committee 318 will need to consider the use of high-strength reinforcing steels in the design of concrete structures. To increase the 80 ksi (550 MPa) yield strength limitation on reinforcing steel rebar used in flexural members in concrete structures as required by the current ACI Code, a modification of the proposed method by Mast is presented where flexural members with grade reinforcing steels of up to 120 ksi (520 MPa) are considered (Mast et al., 2008; Mast, 2007). See Figure 2 for the proposed modification and the ACI limitation.

Throughout this paper, all numbers, equations, and tables are presented using English units with soft metric conversions. This paper proposes increasing the yield strength limit for high-strength steel reinforcing bars in flexural members for tension only up to 120 ksi (830 MPa). A comparative numerical beam example showing the use of Grade 60 (400), 75 (520), 100 (690), and 120 (830) tensile reinforcement is presented. Other reinforcing steels in flexural members, such as those used for compression and shear strength, need to follow current recommendations for strength as required by ACI 318. For reinforcing bars used in compression, the limit of 80 ksi (550 MPa) is reasonable since the stress in the compression steel is controlled by the ultimate compressive strain in the concrete, which is specified by ACI 318 as 0.003 for the flexural design of reinforced concrete members. For the flexural beams studied herein, concrete strengths up to 10,000 psi (70 MPa) were considered. Additional physical testing of actual concrete beams with high-strength reinforcement is needed for verification purposes.

Tensile Steel Ratio

In 1963, the ACI 318 Code required all flexural members to limit the tensile steel reinforcement ratio, ρ, to a maximum of 75% of the balanced steel ratio, ρ_b (ACI Committee 318, 1963). This requirement remained in the ACI Code for more than 30 years. The current requirement for the upper bound limit on the amount of tensile strain reinforcement was selected based on approximately 75% of the balanced tensile steel ratio. Currently, the actual upper bound limit for the tensile steel ratio is 71.4% of the balance steel ratio, which corresponds to a minimum strain in the tensile steel of 0.004 (ACI Committee 318, 2014). According to the ACI 318 Code, the lower bound limit on the amount of tensile steel reinforcement is provided by the lower limit on the steel ratio. The minimum reinforcement ratio of $200/f_y$ was based on flexural members having a minimum steel area of 0.5% of the strength cross-section, assuming Grade 40 (280) steel reinforcement, where f_y is the yield strength of the reinforcement in units of psi. In 1995, the ACI 318 Code modified this requirement for concrete compressive strengths greater than approximately 5000 psi (35 MPa) (ACI Committee 318, 1995). The current ACI requirement for the minimum steel reinforcement ratio is

$$\rho_{min} = \min \left(\frac{3\sqrt{f_c'}}{f_y} \quad \frac{200}{f_y} \right) \tag{1}$$

Where f_c' is the compressive strength of the concrete and f_y both have units of psi. Above concrete strengths of 4,444 psi (30 MPa), the first quantity governs. Below concrete strengths of 4,444 psi (30 MPa), the second quantity governs. Based on the Whitney stress block for flexure, the tensile steel ratio for flexural members can be calculated as follows,

$$\rho = \frac{A_s}{bd} \tag{2}$$

Where A_s is the area of the tensile steel reinforcement, and b and d correspond to the beam width and beam depth from the compression face to the tensile steel, respectively.
Based on the equilibrium, where the tension force in the tensile steel reinforcement must be equal to the compression force of the Whitney stress block for a singly reinforced concrete beam, the tensile steel ratio for flexural members made be determined as follows,

$$\rho = 0.85\, \beta_1 \left(\frac{f_c'}{f_y} \right) \left(\frac{\varepsilon_u}{\varepsilon_u + \varepsilon_t} \right) \tag{3}$$

Where β_1 is the ratio of the depth of the Whitney stress block to the distance from the compression face to the neutral axis, ε_u is the ultimate strain at the compression face of the beam section, and ε_t is the strain in the tensile steel reinforcement. From Equation 3, as the strain in the steel tensile reinforcement, ε_t, increases beyond the strain at yield, ε_y, the steel ratio, ρ, decreases below ρ_b. This inverse relationship is shown in Figure 3, where the ratio ρ/ρ_b is plotted versus tensile steel strain for different grades of reinforcement strength.

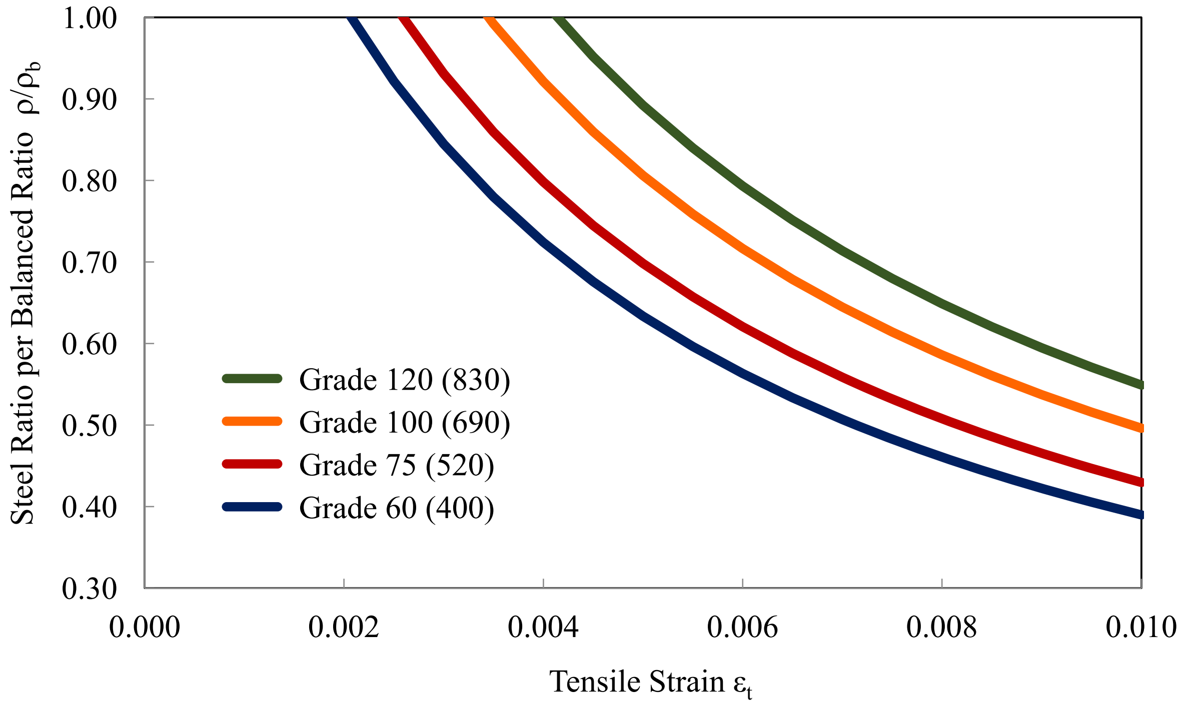

Figure 3. Steel ratio per balanced ratio versus tensile steel strain.

Tensile Steel Strain

Ultimate strength design, now referred to as strength design, as a methodology was first introduced in the ACI Code in 1963. Based on the balanced steel ratio, ρ_b, for flexural members, ACI beams were required to be under-reinforced, meaning that when the strain in the concrete at the compression face reaches 0.003, the strain in the tension steel, ε_t, must be significantly above the strain at yield, ε_y. At a concrete strain of 0.003, this is the ultimate strain in the concrete, ε_u, according to the ACI Code. This condition requires the tensile steel reinforcement to fail first and yield, prior to the concrete at the compression face from crushing. As a result, failure would be gradual where permanent deformation in the tensile rebar would occur and accumulate, prior to the failure of the concrete.

In 1995, the ACI 318 Code introduced into Appendix B an alternative requirement that limited the maximum steel ratio, ρ_{max}, to a minimum tensile strain in the reinforcement at the nominal moment strength for flexure (ACI Committee 318, 1995). Prior to this, the maximum steel ratio was limited to a maximum of 75% of the balance steel ratio. A tension-controlled member was defined such that the tensile strain in the tension steel at nominal strength was equal to a minimum of 0.005 or greater. This corresponded to a resistance factor, ϕ, of 0.90. For flexural members, the tensile strain in the reinforcement was permitted to be as low as 0.004. However, as the tensile steel strain decreased from 0.005 to 0.004, the resistance factor for rectangular beam sections decreased linearly from 0.90 to 0.82, respectively. Flexural members with tensile strains equal to 0.004 to but not including 0.005 are referred to as transition zone members, not tension-controlled members. Figure 4 presents

ACI strain limits for reinforced concrete members.

Research supporting the change in the ACI 318 Code from the steel ratio requirement to a tensile steel strain requirement was based on studies using Grade 60 (400 MPa) rebar. However, this approach was extended and permitted for flexural members with Grade 75 (520) steel reinforcement despite the difference in the strains at yield. For Grade 60 (400) steel reinforcement, the strain at yield is 0.00207. For Grade 75 (520) steel reinforcement, the strain at yield is 0.00259. The change in the strain at yield corresponds to a 25% increase in strain. In 2002, the ACI 318 Code made this alternative mandatory by moving Appendix B to the body of the ACI Code (ACI Committee 318, 2002). Thus, rather than designing flexural members on the basis of the maximum steel ratio, ρ_{max}, this was replaced by limits placed on the tensile steel strain, ε_t, in the tension reinforcement.

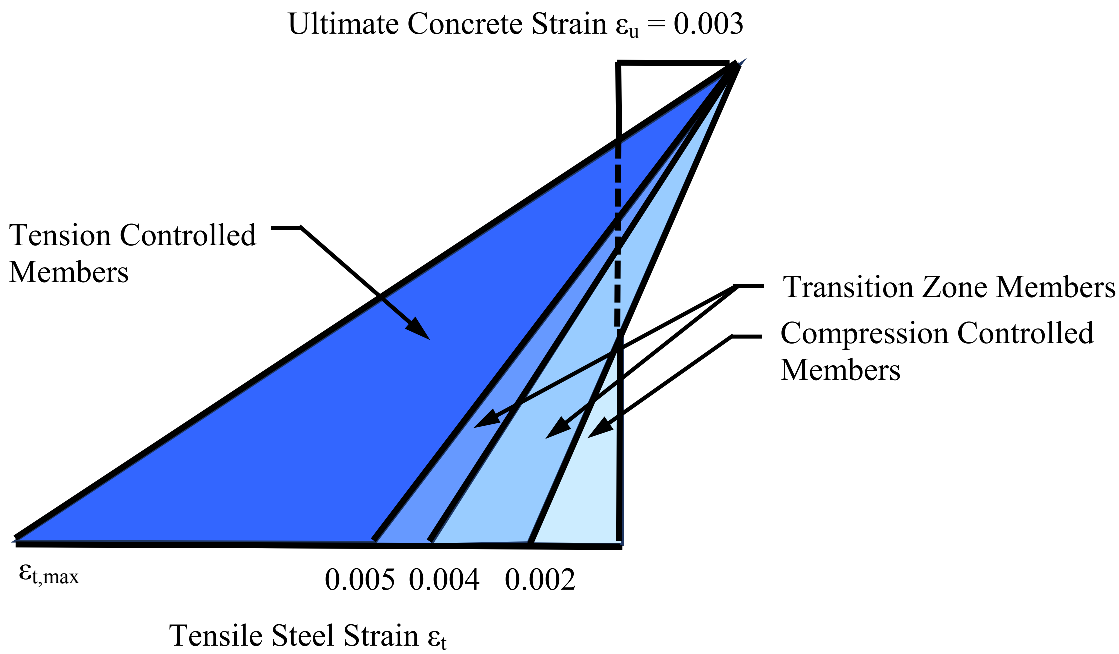

Figure 4. ACI tensile strain limits for reinforced concrete members.

While this approach is practical and feasible for Grade 60 (400) steel reinforcement (and was deemed acceptable for Grade 75 (520) steel reinforcement), changes in strain limits are needed to ensure the same level of structural safety if reinforcing steels with higher grades beyond 80 ksi (550 MPa) are to be used in practice for the flexural design of reinforced concrete members. Researchers have already proposed modifying the strain limits for tension reinforcement in flexural members (Mast et al., 2008; Shahrooz, Reis, Wells, Miller, Harries, & Russell, 2010). Mast et al. proposed modifying the existing strain limits for using Grade 100 (690) steel reinforcement meeting the requirements of ASTM A1035/A1035M in flexural members (2008). For Grade 100 (690) steel rebar, the strain at yield is 0.00345. This represents an increase in the strain at yield of 40% in comparison to Grade 60 (400) steel reinforcement. Not increasing the strain limits would decrease the structural safety of the flexural member, potentially leading to more failures. When using tensile steel reinforcement that is Grade 100 (690), Mast el al. proposed increasing the strain limit for compression controlled members from 0.002 to 0.004 and for tension-controlled members from 0.005 to 0.009 (2008). Thus, the 0.004 minimum strain limit for a flexure member would need to be increased to approximately 0.0065 for consistency. Similar results were reported by Shahrooz et al. for future updates to the AASHTO *LRFD Bridge Design Specification* (2010). For compression-controlled members, a recommendation was made to increase the strain limit from 0.002 to 0.004, the same as proposed by Mast et al. For tension-controlled members, a recommendation was made to increase the strain limit from 0.005 to 0.008.

Since the ACI Code now requires a minimum strain in the tensile reinforcement for flexural members instead of calculating a limiting steel ratio, a maximum strain in the tensile reinforcement should also be adopted rather than calculating a minimum steel ratio, which corresponds to a maximum strain in the tensile steel. By setting the minimum ACI steel ratio, Equation 1, equal to the steel ratio, Equation 2, the strain in the tensile steel, ε_t, becomes the maximum strain in the tensile steel, $\varepsilon_{t,max}$. Solving for $\varepsilon_{t,max}$, the following expression results. For $f_c^{'}$ less than or equal to 4,444 psi (30 MPa),

$$\varepsilon_{t,max} = \frac{0.85}{200} \beta_1 f_c^{'} \varepsilon_u \tag{4}$$

For $f_c^{'}$ greater than 4,444 psi (30 MPa),

$$\varepsilon_{t,max} = \frac{0.85}{3} \beta_1 \sqrt{f_c^{'}} \varepsilon_u - \varepsilon_u \tag{5}$$

Where $f_c^{'}$ is the concrete compressive strength in units of psi, β_1 is a function of the compressive strength of the concrete, and ε_u is the ultimate strain in the concrete at the compression face which is equal to 0.003 according to ACI 318-14. Equations 4 and 5 determine the ACI maximum strain in the tensile steel reinforcement. These equations are presented in Figure 5. The ACI maximum strain in the tensile steel has three distinct segments resulting from the change in the value of β_1 as concrete compressive strength increases. The value for β_1 is a function of the compressive strength of the concrete. For $f_c^{'}$ less than or equal to 4,000 psi (28 MPa), β_1 equals 0.85. For $f_c^{'}$ greater than or equal to 8,000 psi (55 MPa), β_1 equals 0.65. For $f_c^{'}$ between and including 4,000 psi (28 MPa) and 8,000 psi (55 MPa), β_1 varies linearly. A simple relationship for the maximum strain in the tensile steel reinforcement may be conservatively determined as

$$\varepsilon_{t,max} = 0.00052 \sqrt{f_c^{'}} \tag{6}$$

Where $f_c^{'}$ is the concrete compressive strength in units of psi. This is the proposed maximum strain in the tensile steel and is also presented in Figure 5.

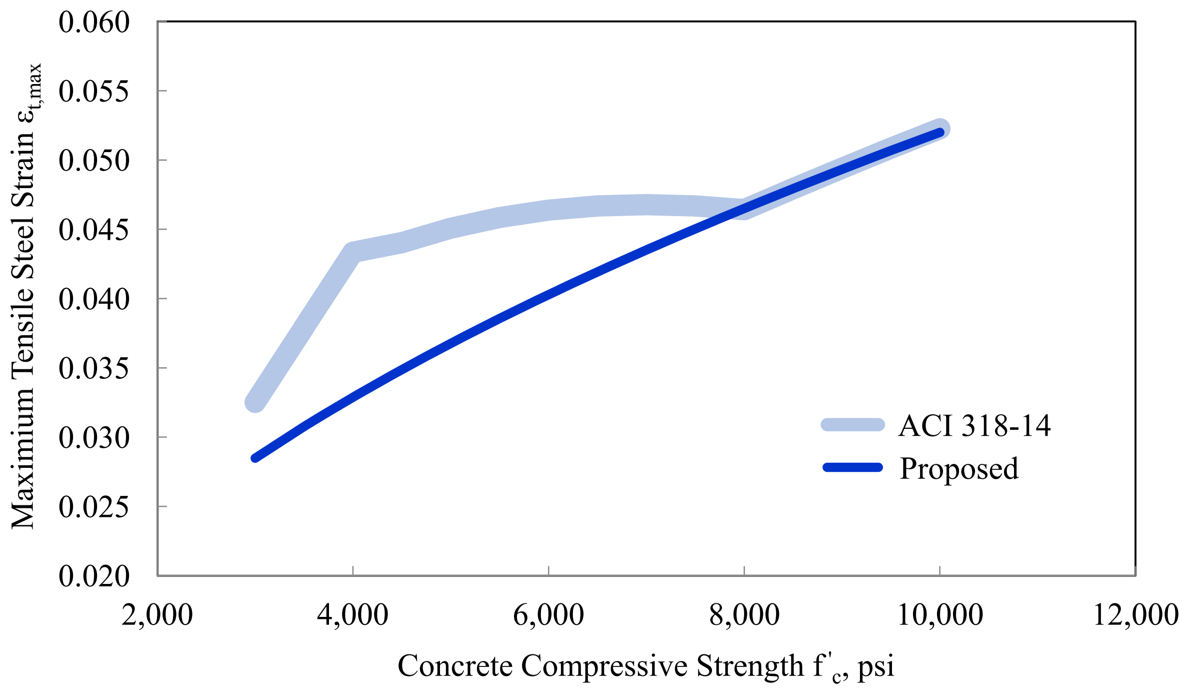

Figure 5. Maximum tensile steel strain versus concrete compressive strength.

Will future designers have to consider two different sets of strain limits based on the use of normal or high-strength steel reinforcement for flexure? What about higher strength rebar with Grade 120 (830) or more? Perhaps a more consistent and simplistic method may be to modify the ACI relationship for the resistance factor, ϕ, versus tensile steel strain, ε_t, and calculate a maximum tensile steel strain, $\varepsilon_{t,max}$, as discussed in the next section.

Resistance Factor

Since 1963, the ACI 318 Code has defined the factored nominal moment strength as the product of the resistance factor and the nominal moment. In 2002, ACI 318 specified the resistance factor, ϕ, as a dependent function on the tensile strain, ε_t, in the steel reinforcement with a grade of 80 ksi (550 MPa) or less (ACI Committee 318, 2002). From the current ACI 318 Code, the graph of the resistance factor versus tensile strain in the steel reinforcement is shown in Figure 6 (ACI Committee 318, 2014).

Using the recommendations proposed by Mast et al. (2008) and Shahrooz et al. (2010) for high-strength steel reinforcement with Grade 100 (690) rebar and superimposing this on Figure 6, the resulting graph is shown in Figure 7. Figure 7 is congested and unnecessarily complicated. One of the primary goals of using high-strength reinforcement as compared to nominal-strength reinforcement is to reduce rebar congestion. Figure 7 may be significantly simplified to a single relationship by plotting the resistance factor versus tensile steel strain where the x-axis is also a function of the grade of the rebar, f_y, in units of ksi. Figure 8 shows the resulting graph, calibrated for reinforcing steel for grades up to and including 120 ksi (830 MPa).

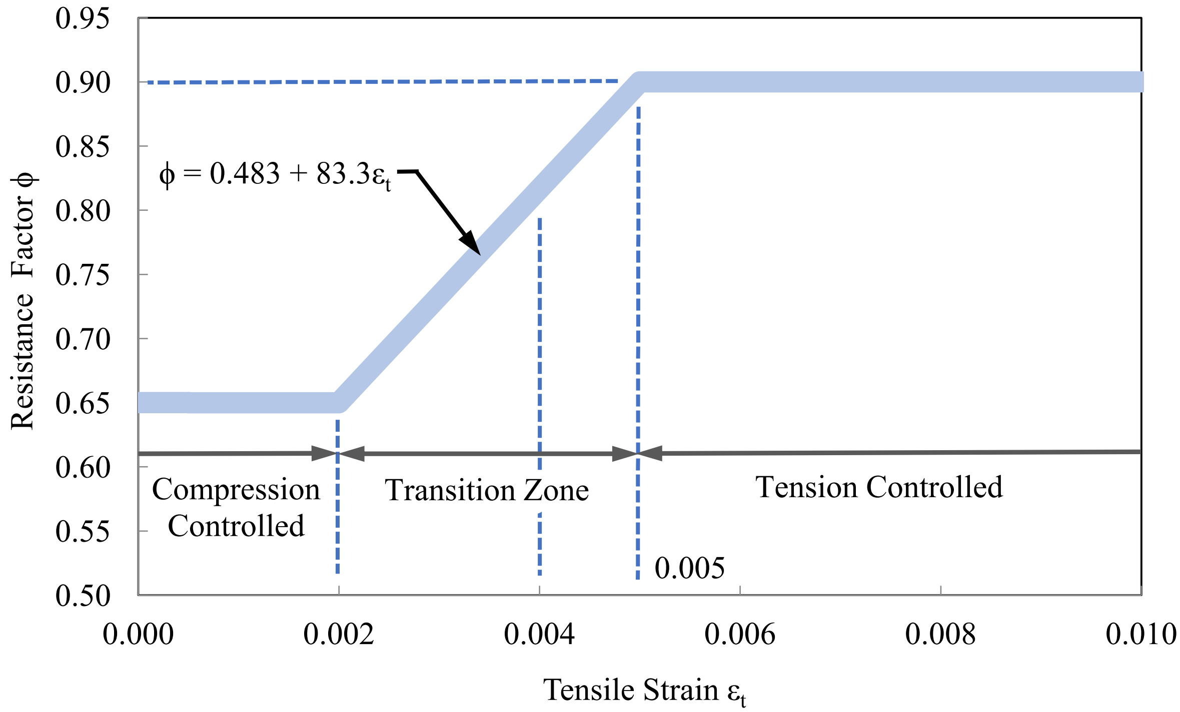

Figure 6. ACI resistance factor versus tensile steel strain.

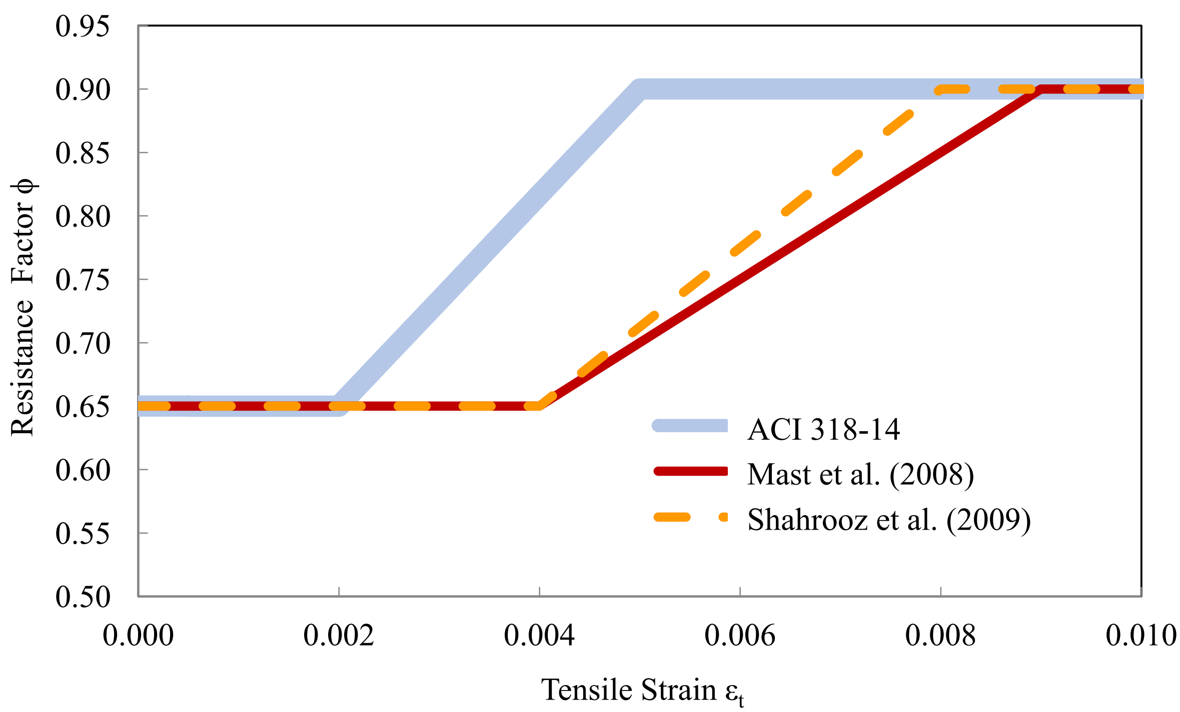

Figure 7. Resistance factor versus tensile steel strain for differing grades of rebar.

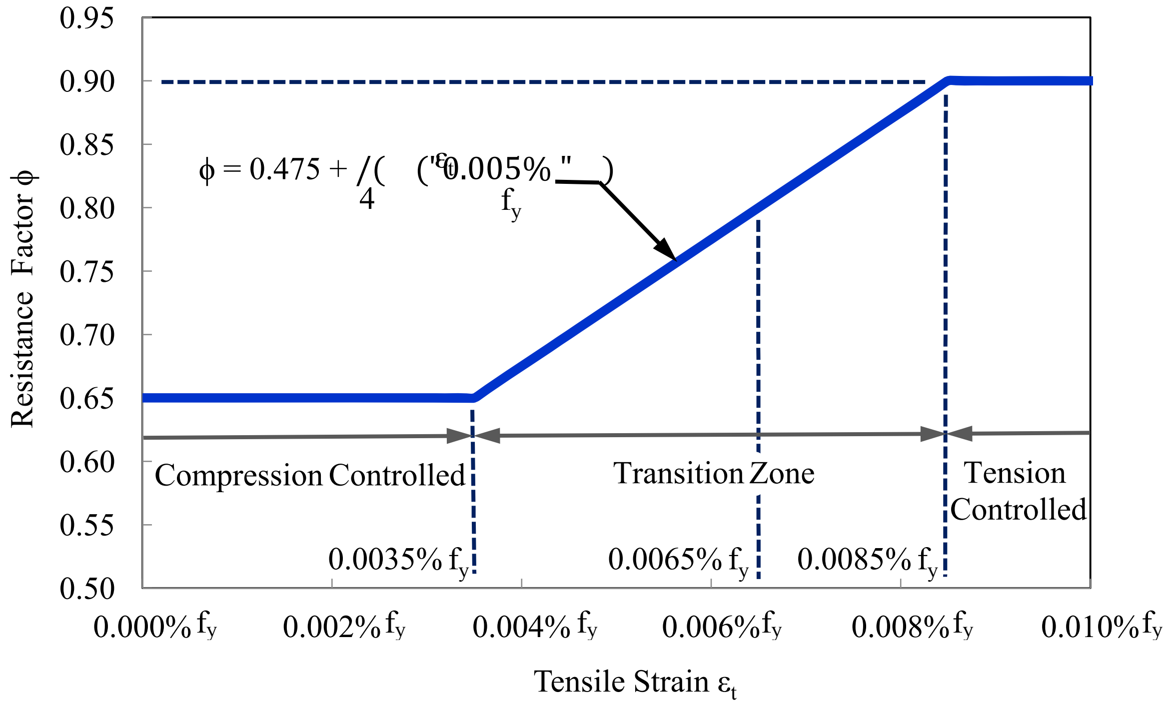

Figure 8. Proposed resistance factor versus tensile steel strain for up to Grade 120 (830).

Table 1 presents the tensile strain limit results of Figure 8 for differing grades of steel reinforcement. For comparison purposes, the tensile strain limits in ACI 318-14 are also presented in Table 1.

Table 1. Tensile steel strain member limits.

ACI / Rebar Grade	Tensile Strain Limit		
	Compression Controlled	Transition Zone for Flexure	Tension Controlled
ACI 318-14	0.0020	0.0040	0.0050
Grade 60 (400)	0.0021	0.0039	0.0051
Grade 75 (520)	0.0026	0.0049	0.0064
Grade 100 (690)	0.0035	0.0065	0.0085
Grade 120 (830)	0.0042	0.0078	0.0102

Using this proposed approach for grades of tensile steel reinforcement up to and including Grade 120 (830) and including the proposed maximum tensile strain, $\varepsilon_{t,max}$, from Equation 6, Figure 3 can be modified as shown in Figure 9.

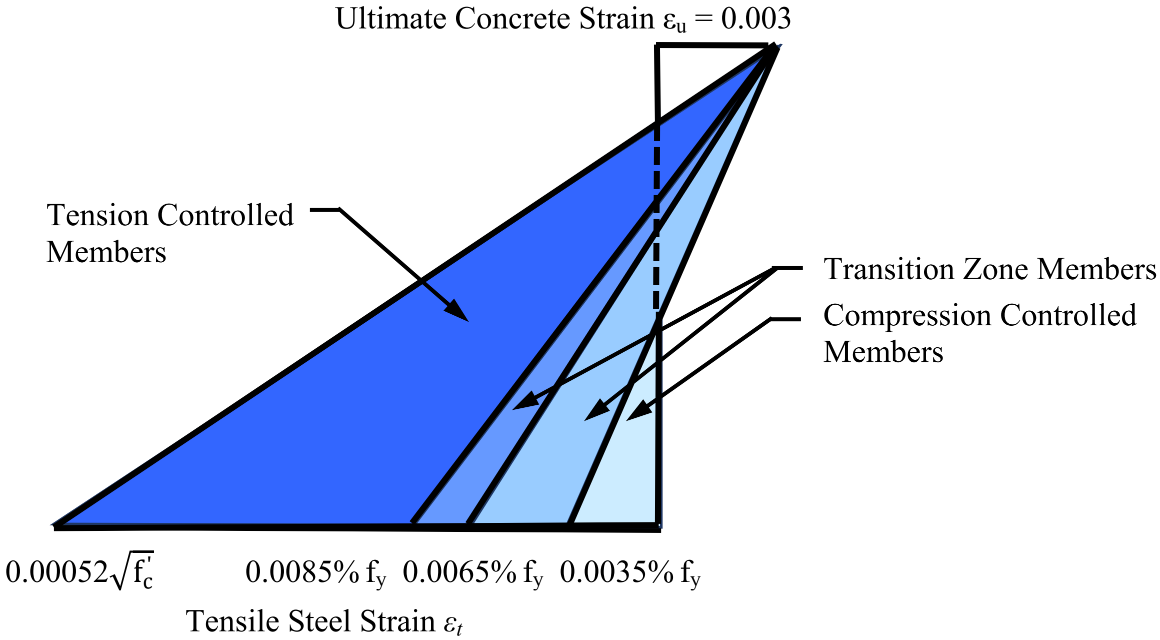

Figure 9. Proposed tensile strain limits for reinforced concrete members.

Comparative Numerical Design Example

A simply supported, singly reinforced concrete beam with span length of 25 feet (8 m) was designed for flexure as shown in Figure 10. The beam was loaded with a uniformly distributed live load of 2.0 k/ft (30 kN/m) and a uniformly distributed dead load of 1.2 k/ft (18 kN/m), not including beam self weight. Gross beam dimensions were held constant where the height of the beam was 26 in (0.65 m), beam width was 14 in (0.35 m), and the depth from the compression face to the neutral axis was 24 in (0.60 m). The beam was designed using four different grades of tensile steel reinforcement. For design, the grades of tensile steel reinforcement included 60 (400), 75 (520), 100 (690), and 120 (830). Factoring the loads, the ultimate moment was determined to be 398 k-ft (540 kN/m). The compressive strength of the concrete was 6,000 psi (40 MPa), and the modulus of elasticity of the steel reinforcement was 29,000 ksi (200 GPa). The resulting properties of the beam with varying grades of tensile steel reinforcement are presented in Table 2.

Based on the ACI 318-14, the maximum tensile strain in the steel as per equation (5) was 0.0464. Using the proposed equation, equation (6), the maximum tensile strain in the steel was 0.0403. Compared to the beam with Grade 60 (400) steel reinforcement, a decrease of 43.7% in the required steel area was found for the beam with Grade 100 (690) rebar. In comparison to using Grade 120 (830) rebar, the decrease was 53.1%. Using rebar with high-strength resulted in a beam design that was far less congested than using traditional normal-strength steel reinforcement.

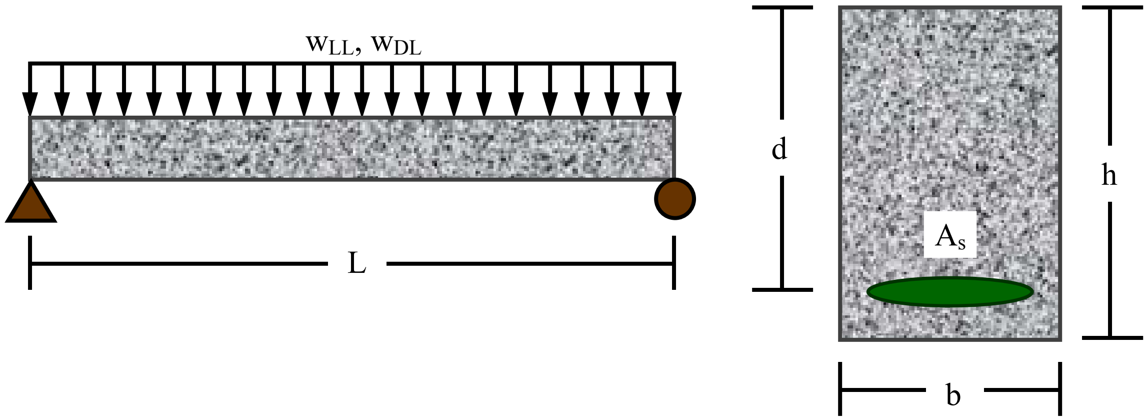

Figure 10. Design example of a flexural member with high-strength reinforcement.

Table 2. Beam design properties for differing grades of rebar reinforcement.

Beam Property	Grade of Tensile Steel Reinforcement, ksi (MPa)			
	60 (400)	75 (520)	100 (690)	120 (830)
ρ_{min}	0.0039	0.0031	0.0023	0.0019
ρ	0.0140	0.0107	0.0079	0.0066
ρ_{max}	0.0239	0.0191	0.0143	0.0120
ρ_b	0.0377	0.0274	0.0178	0.0134
A_s, in^2 (mm^2)	4.71 (3040)	3.61 (2330)	2.65 (1710)	2.21 (1430)
Tensile Rebars	6 #8 (#25)	6 #7 (#22)	6 #6 (#19)	5 #6 (#19)
ε_t	0.0106	0.0112	0.0115	0.0115
$\varepsilon_{min,}$ $\phi = 0.90$	0.0051	0.0064	0.0085	0.0102
ϕM_n, k-ft (kNm)	467 (630)	449 (610)	440 (600)	440 (600)

Conclusions

In recent years, steel reinforcement has been developed with strengths greater than 80 ksi (550 MPa), which is the strength limit for reinforced concrete structures according to ACI 318-14. In this paper, a proposed modification to the modification by Mast et al. was presented where flexural members with tensile steel reinforcement of Grade 120 (830) were examined. A simple maximum tensile steel strain limit relationship was also proposed, based on the minimum tensile steel ratio per ACI 318-14. Finally, a modification to the ACI Code for the resistance factor versus tensile steel strain relationship was proposed, which allows for rebar strength grades up to 120 ksi (830 MPa).

Results of a numerical reinforced concrete beam example designed with differing grades of rebar indicated that using higher strength rebar above the ACI limitation resulted in less flexural steel congestion within the cross-section of the beam. According to this numerical example when compared to using Grade 60 (410) rebar, a reduction of 23%, 44% and 53% in steel area was determined when using Grade 75 (520), 100 (690) and 120 (830) rebar, respectively. When compared to using Grade 75 (520) rebar, a reduction of 27% and 39% in steel area was determined when using Grade 100 (690) and 120 (830) rebar, respectively. This is a significant reduction in flexural steel area, which will lead to improved concrete placement around the reinforcement in the field. Additional analysis and physical testing verification is needed to allow for high-strength tensile steel reinforcement in flexural members in concrete structures. From an analytical design perspective, using high-strength reinforcement is promising.

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