Appendix L

Proposed Changes to Section 5 of the AASHTO LRFD Specification

PROPOSED CHANGES

NOTE: THIS CONTAINS THE PROPOSED CHANGES TO SECTION 5 OF THE AASHTO LRFD SEPCIFICATIONS (V5). ONLY AFFECTED SECTIONS ARE SHOWN. IN THE CASES OF LARGE SECTIONS WHERE THERE ARE MINOR MODIFICATIONS, ONLY THE MODIFICATIONS ARE SHOWN FOR CLARITY. MODIFIED ARTICLES ARE SHOWN IN BOLD IN THE TABLE OF CONTENTS.

5.2—DEFINITIONS

Tension-Controlled Section—A cross-section in which the net tensile strain in the extreme tension steel at nominal resistance is greater than or equal to ~~0.005~~ the tension-controlled strain limit.

Tension-Controlled Strain Limit—The net tensile strain in the extreme tension steel at nominal resistance that results in a tension-controlled section. See Article 5.7.2.1.

5.3—NOTATION

f_y = specified minimum yield strength of reinforcing bars (ksi); specified yield strength of reinforcing bars ≤75 ksi (~~5.5.4.2.1)~~ (5.10.8) unless higher strength is permitted by a specific article (5.4.3.1)

ε_{cl} = compression-controlled strain limit in the extreme tension steel (in./in.) (5.7.2.1)

ε_{tl} = tension-controlled strain limit in the extreme tension steel (in./in.) (5.7.2.1)

5.4.3—Reinforcing Steel

5.4.3.1—General

Reinforcing bars, deformed wire, cold-drawn wire, welded plain wire fabric, and welded deformed wire fabric shall conform to the material standards as specified in Article 9.2 of the *AASHTO LRFD Bridge Construction Specifications*.

Reinforcement shall be deformed, except that plain bars or plain wire may be used for spirals, hoops, and wire fabric.

The nominal yield strength shall be the minimum as specified for the grade of steel selected, except that yield strengths in excess of 75.0 ksi shall not be used for design purposes unless specified yield strengths up to 100.0 ksi are permitted by specific articles for Seismic Zone 1. The yield strength or grade of the bars or wires shall be shown in the contract documents. Bars with yield strengths less than 60.0 ksi shall be used only with the approval of the Owner.

Where ductility is to be assured or where welding is required, steel conforming to the requirements of ASTM A706, "Low Alloy Steel Deformed Bars for Concrete Reinforcement," should be specified.

5.4.3.2—Modulus of Elasticity

The modulus of elasticity, E_s, of steel ~~reinforcing~~ reinforcement shall be assumed as 29,000 ksi for specified yield strengths up to 100 ksi.

5.4.3.3—Special Applications

Reinforcement to be welded shall be indicated in the contract documents, and the welding procedure to be

C5.4.3.1

Unlike reinforcing bars with yield strengths below 75.0 ksi, reinforcing bars with yield strengths exceeding 75.0 ksi usually do not have well-defined yield plateaus. Consequently, different methods are used in different standards to establish yield strengths. These include the 0.2% offset and the 0.35% or 0.50% extension methods. For design purposes, the value of f_y should be the same as the specified yield strength defined in the material standard. Based on research by Shahrooz et al. (2010), certain articles now allow the use of reinforcing steels with yield strengths up to 100.0 ksi for Seismic Zone 1.

ASTM A706 reinforcement should be considered for seismic design because of the greater quality control by which unanticipated overstrength is limited.

C5.4.3.3

In 2004, ASTM published A1035/A1035M, *Standard Specification for Deformed and Plain, Low-*

used shall be specified.

The use of reinforcing steel with specified yield strengths of less than or equal to 100.0 ksi may be used for Seismic Zone 1 where permitted by specific articles. Reinforcement conforming to ASTM A1035/A1035M may only be used as top and bottom flexural reinforcement in the longitudinal and transverse directions of bridge decks in Seismic Zones 1 and 2.

carbon, Chromium, Steel Bars for Concrete Reinforcement. This reinforcement offers the potential for corrosion resistance.

Epoxy-coated reinforcing steel provides a physical barrier to inhibit corrosion of the steel in the presence of chlorides. The handling, placement, and repair of epoxy-coated reinforcing steel requires significant care and attention.

Reinforcement conforming to ASTM A1035/A1035M has a specified minimum yield strength of 100 ksi determined by the 0.2 percent offset method, a specified minimum tensile strength of 150 ksi, and a specified minimum elongation of six or seven percent depending on bar size. There is also a requirement that the stress corresponding to a tensile strain of 0.0035 shall be a minimum of 80 ksi. The reinforcement has a non-linear stress-strain relationship. Article 5.4.3.1 of the Design Specifications states that yield strengths in excess of 75.0 ksi shall not be used for design purposes. Consequently, design is based on a stress of 75.0 ksi, but the actual strength is at least twice that value. This has lead to concerns about the applicability of the existing specifications with ASTM A1035 reinforcement. Consequently, it is proposed that initial usage of the reinforcement be restricted to top and bottom flexural reinforcement in the transverse and longitudinal directions of bridge decks in Seismic Zones 1 and 2.

5.5.3—Fatigue Limit State

5.5.3.1—General

Fatigue need not be investigated for concrete deck slabs in multigirder applications or reinforced-concrete box culverts.

In regions of compressive stress due to permanent loads and prestress in reinforced and partially prestressed concrete components, fatigue shall be considered only if this compressive stress is less than the maximum tensile live load stress resulting from the Fatigue I load combination as specified in Table 3.4.1-1 in combination with the provisions of Article 3.6.1.4.

Fatigue of the reinforcement need not be checked for fully prestressed components designed to have extreme fiber tensile stress due to Service III Limit State within the tensile stress limit specified in Table 5.9.4.2.2-1.

C5.5.3.1

Stresses measured in concrete deck slabs of bridges in service are far below infinite fatigue life, most probably due to internal arching action; see Article C9.7.2.

Fatigue evaluation for reinforced-concrete box culverts showed that the live load stresses in the reinforcement due to Fatigue I load combination did not reduce the member resistance at the strength limit state.

In determining the need to investigate fatigue, Table 3.4.1-1 specifies a load factor of 1.50 on the live load force effect resulting from the fatigue truck for the Fatigue I load combination. This factored live load force effect represents the greatest fatigue stress that the bridge will experience during its life.

Fatigue limit state load factor, girder distribution factors, and dynamic allowance cause fatigue limit state stress to be considerably less than the corresponding value determined from Service Limit State III. For fully prestressed components, the net concrete stress is usually significantly less than the concrete tensile stress limit specified in Table 5.9.4.2.2-1. Therefore, the calculated flexural stresses are significantly reduced. For this situation, the calculated steel stress range, which is equal to the modular ratio times the concrete stress range, is almost always less than the steel fatigue stress range limit specified in Article 5.5.3.3.

For fatigue considerations, concrete members shall satisfy:

$$\gamma(\Delta f) \leq (\Delta F)_{TH} \qquad (5.5.3.1\text{-}1)$$

where:

γ =   load factor specified in Table 3.4.1-1 for the Fatigue I load combination

Δf =   force effect, live load stress range due to the passage of the fatigue load as specified in Article 3.6.1.4 (ksi)

$(\Delta F)_{TH}$ =   constant-amplitude fatigue threshold, as specified in Article 5.5.3.2, 5.5.3.3, or 5.5.3.4, as appropriate (ksi)

For fully prestressed components in other than segmentally constructed bridges, the compressive stress due to the Fatigue I load combination and one-half the sum of effective prestress and permanent loads shall not exceed $0.40f'_c$ after losses.

The section properties for fatigue investigations shall be based on cracked sections where the sum of stresses, due to unfactored permanent loads and prestress, and the Fatigue I load combination is tensile and exceeds $0.095\sqrt{f'_c}$.

5.5.3.2—Reinforcing Bars

The constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, for straight reinforcement and welded wire reinforcement without a cross weld in the high-stress region shall be taken as:

$$\cancel{(\Delta F)_{TH} = 24 - 0.33 f_{min}}$$
$$\underline{(\Delta F)_{TH} = 24 - 20 f_{min} / f_y} \qquad (5.5.3.2\text{-}1)$$

The constant-amplitude fatigue threshold, $(\Delta F)_{TH}$, for straight welded wire reinforcement with a cross weld in the high-stress region shall be taken as:

$$(\Delta F)_{TH} = 16 - 0.33 f_{min} \qquad (5.5.3.2\text{-}2)$$

where:

f_{min} =   minimum live-load stress resulting from the Fatigue I load combination, combined with the more severe stress from either the permanent loads or the permanent loads, shrinkage, and creep-induced external loads; positive if tension, negative if compression (ksi)

$\underline{f_y}$ = $\underline{\text{specified minimum yield strength, not to be}}$

C5.5.3.2

With the permitted use of steel reinforcement having yield stresses above 75.0 ksi, the value of f_{min} is expected to increase. In previous versions of Eq. 5.5.3.2-1, an increase in f_{min} would result in an decrease in $(\Delta F)_{TH}$, regardless of the yield strength of the bar. All available data indicates that steel with a higher yield strength actually has a higher fatigue limit (DeJong and MacDougall, 2006). Eq. 5.5.3.2-1 has been calibrated such that there is no change to the value of $(\Delta F)_{TH}$ from earlier versions of this equation for cases of f_y = 60.0 ksi, but it now provides more reasonable values of $(\Delta F)_{TH}$ for higher strength reinforcing bars.

Bends in primary reinforcement should be avoided in regions of high stress range.

Structural welded wire reinforcement has been increasingly used in bridge applications in recent years, especially as auxiliary reinforcement in bridge I- and box beams and as primary reinforcement in slabs. Design for shear has traditionally not included a fatigue check of the reinforcement as the member is expected to be uncracked under service conditions and the stress range in steel minimal. The stress range for steel bars has existed in previous editions. It is based on Hansen et al. (1976). The simplified form in this edition replaces the (r/h) parameter with the default value 0.3

taken less than 60.0 ksi nor greater than 100.0 ksi.

The definition of the high-stress region for application of Eqs. 5.5.3.2-1 and 5.5.3.2-2 for flexural reinforcement shall be taken as one-third of the span on each side of the section of maximum moment.

recommended by Hansen et al. Inclusion of limits for WWR is based on recent studies by Hawkins et al. (1971, 1987) and Tadros et al. (2004).

Since the fatigue provisions were developed based primarily on ASTM A615 steel reinforcement, their applicability to other types of reinforcement is largely unknown. Consequently, a cautionary note is added to the Commentary.

5.5.4.2—Resistance Factors

5.5.4.2.1—Conventional Construction

The provisions of Article 5.5.4.2.1 are applicable to nonprestressed reinforcing steels with specified yield strengths up to 100.0 ksi for Seismic Zone 1.

Resistance factor ϕ shall be taken as:

- For tension-controlled reinforced concrete sections as defined in Article 5.7.2.1..................0.90
- For tension-controlled prestressed concrete sections as defined in Article 5.7.2.1..................1.00
- For shear and torsion:
        normal weight concrete.........................0.90
        lightweight concrete..............................0.70
    - For compression-controlled sections with spirals or ties, as defined in Article 5.7.2.1, except as specified in Articles 5.10.11.3 and 5.10.11.4.1b for Seismic Zones 2, 3, and 4 at the extreme event limit state...0.75
- For bearing on concrete......................................0.70
- For compression in strut-and-tie models............0.70

C5.5.4.2.1

In applying the resistance factors for tension-controlled and compression-controlled sections, the axial tensions and compressions to be considered are those caused by external forces. Effects of prestressing forces are not included.

In editions of and interims to the LRFD Specifications prior to 2005, the provisions specified the magnitude of the resistance factor for cases of axial load or flexure, or both, it terms of the type of loading. For these cases, the ϕ-factor is now determined by the strain conditions at a cross-section, at nominal strength. The background and basis for these provisions are given in Mast (1992) and ACI 318-02.

A lower ϕ-factor is used for compression-controlled sections than is used for tension-controlled sections because compression-controlled sections have less ductility, are more sensitive to variations in concrete strength, and generally occur in members that support larger loaded areas than members with tension-controlled sections.

For sections subjected to axial load with flexure, factored resistances are determined by multiplying both P_n and M_n by the appropriate single value of ϕ. Compression-controlled and tension-controlled sections are defined in Article 5.7.2.1 as those that have net tensile strain in the extreme tension steel at nominal strength less than or equal to the compression-controlled strain limit, and equal to or greater than 0.005 the tension controlled strain limit, respectively. For sections with net tensile strain ε_t in the extreme tension steel at nominal strength between the above limits, the value of ϕ may be determined by linear interpolation, as shown in Figure C5.5.4.2.1-1. The concept of net tensile strain ε_t is discussed in Article C5.7.2.1. Classifying sections as tension-controlled, transition or compression-controlled, and linearly varying the resistance factor in the transition zone between reasonable values for the two extremes, provides a rational approach for determining ϕ and limiting the capacity of over-reinforced sections.

$$\phi = 0.583 + 0.25\left(\frac{d_t}{c} - 1\right)$$

$$\phi = 0.65 + 0.15\left(\frac{d_t}{c} - 1\right)$$

$$0.75 \leq \phi = 0.75 + \frac{0.25\left(\varepsilon_t - \varepsilon_{cl}\right)}{\left(\varepsilon_{tl} - \varepsilon_{cl}\right)} \leq 1.0$$

$$0.75 \leq \phi = 0.75 + \frac{0.15\left(\varepsilon_t - \varepsilon_{cl}\right)}{\left(\varepsilon_{tl} - \varepsilon_{cl}\right)} \leq 0.9$$
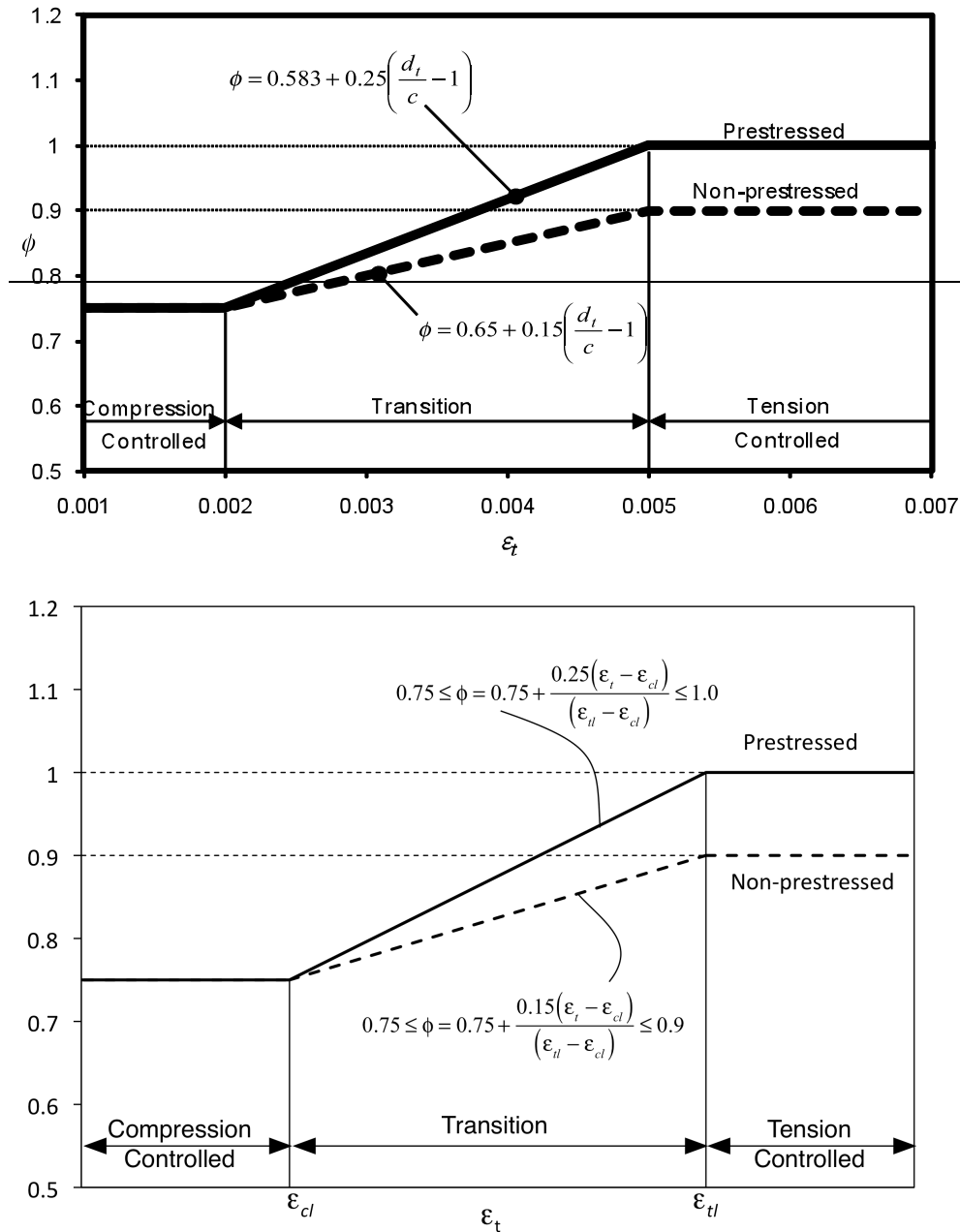
Figure C5.5.4.2.1-1—Variation of ϕ with Net Tensile Strain ε_t ~~and d_t/c~~ for ~~Grade 60~~ <u>Nonprestressed</u> Reinforcement and for Prestressing Steel

- For compression in anchorage zones:
  normal weight concrete 0.80
  lightweight concrete 0.65
- For tension in steel in anchorage zones 1.00
- For resistance during pile driving 1.00

For sections in which the net tensile strain in the extreme tension steel at nominal resistance is between the limits for compression-controlled and tension-controlled sections, ϕ may be linearly increased from 0.75 to that for tension-controlled sections as the net

The ϕ-factor of 0.8 for normal weight concrete reflects the importance of the anchorage zone, the brittle failure mode for compression struts in the anchorage zone, and the relatively wide scatter of results of experimental anchorage zone studies. The ϕ-factor of 0.65 for lightweight concrete reflects its often lower tensile strength and is based on the multipliers used in ACI 318-89, Section 11.2.1.2.

The design of intermediate anchorages, anchorages, diaphragms, and multiple slab anchorages are addressed in Breen et al. (1994).

tensile strain in the extreme tension steel increases from the compression-controlled strain limit, ε_{cl} to ~~0.005~~the tension-controlled strain limit, ε_{tl}.

This variation ϕ may be computed for prestressed members such that:

$$~~0.75 \le \phi = 0.583 + 0.25\left(\frac{d_t}{c} - 1\right) \le 1.0~~$$

$$0.75 \le \phi = 0.75 + \frac{0.25\left(\varepsilon_t - \varepsilon_{cl}\right)}{\left(\varepsilon_{tl} - \varepsilon_{cl}\right)} \le 1.0 \qquad (5.5.4.2.1\text{-}1)$$

and for nonprestressed members such that:

$$~~0.75 \le \phi = 0.65 + 0.15\left(\frac{d_t}{c} - 1\right) \le 0.9~~$$

$$0.75 \le \phi = 0.75 + \frac{0.15\left(\varepsilon_t - \varepsilon_{cl}\right)}{\left(\varepsilon_{tl} - \varepsilon_{cl}\right)} \le 0.9 \qquad (5.5.4.2.1\text{-}2)$$

where:

~~c = distance from the extreme compression fiber to the neutral axis(in.)~~

~~d_t = distance from the extreme compression fiber to the centroid of the extreme tension steel element (in.)~~

ε_t = net tensile strain in the extreme tension steel at nominal resistance

ε_{cl} = compression-controlled strain limit in the extreme tension steel (in./in.) (5.7.2.1)

ε_{tl} = tension-controlled strain limit in the extreme tension steel (in./in.) (5.7.2.1).

For tension-controlled partially prestressed components in flexure, the values of ϕ may be taken as:

$$\phi = 0.90 + 0.10(PPR) \qquad (5.5.4.2.1\text{-}3)$$

in which:

$$PPR = \frac{A_{ps} f_{py}}{A_{ps} f_{py} + A_s f_y} \qquad (5.5.4.2.1\text{-}4)$$

where:

PPR = partial prestress ratio

A_s = area of nonprestressed tension reinforcement (in.^{2})

A_{ps} = area of prestressing steel (in.^{2})

f_y =   specified yield strength of reinforcing bars (ksi)

f_{py} =   yield strength of prestressing steel (ksi)

Resistance factors shall not be applied to the development and splice lengths of reinforcement as specified in Article 5.11.

5.7—DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS

The provisions of this article are applicable to nonprestressed reinforcing steels with specified yield strengths up to 100.0 ksi for Seismic Zone 1.

5.7.1—Assumptions for Service and Fatigue Limit States

The following assumptions may be used in the design of reinforced, prestressed, and partially prestressed concrete components for all compressive strength levels:

- Prestressed concrete resists tension at sections that are uncracked, except as specified in Article 5.7.6.

- The strains in the concrete vary linearly, except in components or regions of components for which conventional strength of materials is inappropriate.

- The modular ratio, n, is rounded to the nearest integer number.

- The modular ratio is calculated as follows:

    ○ E_s/E_c for reinforcing bars

    ○ E_p/E_c for prestressing tendons

- An effective modular ratio of $2n$ is applicable to permanent loads and prestress.

5.7.2—Assumptions for Strength and Extreme Event Limit States

5.7.2.1—General

Factored resistance of concrete components shall be based on the conditions of equilibrium and strain compatibility, the resistance factors as specified in Article 5.5.4.2, and the following assumptions:

- In components with fully bonded reinforcement or prestressing, or in the bonded length of locally debonded or shielded strands, strain is directly

C5.7

The provisions for the use of reinforcing steel with specified yield strengths between 75.0 and 100.0 ksi are based on research by Shahrooz, et al. (2010), which did not consider seismic design.

C5.7.1

Prestressing is treated as part of resistance, except for anchorages and similar details, where the design is totally a function of the tendon force and for which a load factor is specified in Article 3.4.3. External reactions caused by prestressing induce force effects that normally are taken to be part of the loads side of Eq. 1.3.2.1-1. This represents a philosophical dichotomy. In lieu of more precise information, in these Specifications the load factor for these induced force effects should be taken as that for the permanent loads.

Examples of components for which the assumption of linearly varying strains may not be suitable include deep components such as deep beams, corbels, and brackets.

C5.7.2.1

The first paragraph of C5.7.1 applies.

proportional to the distance from the neutral axis, except for deep members that shall satisfy the requirements of Article 5.13.2, and for other disturbed regions.

- In components with fully unbonded or partially unbonded prestressing tendons, i.e., not locally debonded or shielded strands, the difference in strain between the tendons and the concrete section and the effect of deflections on tendon geometry are included in the determination of the stress in the tendons.

- If the concrete is unconfined, the maximum usable strain at the extreme concrete compression fiber is not greater than 0.003.

- If the concrete is confined, a maximum usable strain exceeding 0.003 in the confined core may be utilized if verified. Calculation of the factored resistance shall consider that the concrete cover may be lost at strains compatible with those in the confined concrete core.

- Except for the strut-and-tie model, the stress in the reinforcement is based on a stress-strain curve representative of the steel or on an approved mathematical representation, including development of reinforcing and prestressing elements and transfer of pretensioning.

- The tensile strength of the concrete is neglected.

- The concrete compressive stress-strain distribution is assumed to be rectangular, parabolic, or any other shape that results in a prediction of strength in substantial agreement with the test results.

- The development of reinforcing and prestressing elements and transfer of pretensioning are considered.

- Balanced strain conditions exist at a cross-section when tension reinforcement reaches the strain corresponding to its specified yield strength f_y just as the concrete in compression reaches its assumed ultimate strain of 0.003.

- Sections are compression-controlled when the net tensile strain in the extreme tension steel is equal to or less than the compression-controlled strain limit, ε_{cl}, at the time the concrete in compression reaches its assumed strain limit of 0.003. The compression-controlled strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For Grade 60 reinforcement, and for all prestressed reinforcement, the compression-controlled strain limit may be set equal to 0.002. ε_{cl} = 0.002. For nonprestressed reinforcing steel with a specified yield strength of 100.0 ksi, the compression-controlled strain limit may be taken as ε_{cl} = 0.004. For nonprestressed reinforcing steel with a specified

Research by Bae and Bayrak (2003) has shown that, for well-confined High Strength Concrete (HSC) columns, the concrete cover may be lost at maximum useable strains at the extreme concrete compression fiber as low as 0.0022. The heavy confinement steel causes a weak plane between the concrete core and cover, causing high shear stresses and the resulting early loss of concrete cover.

The nominal flexural strength of a member is reached when the strain in the extreme compression fiber reaches the assumed strain limit of 0.003. The net tensile strain ε_t is the tensile strain in the extreme tension steel at nominal strength, exclusive of strains due to prestress, creep, shrinkage, and temperature. The net tensile strain in the extreme tension steel is determined from a linear strain distribution at nominal strength, as shown in Figure C5.7.2.1-1, using similar triangles.

Reinforcement closest to the tension face
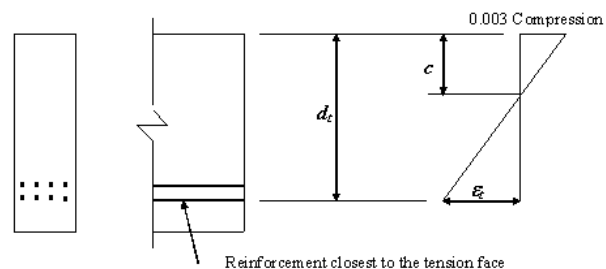
yield strength between 60.0 and 100.0 ksi, the compression controlled strain limit may be determined by linear interpolation based on specified yield strength.

- Sections are tension-controlled when the net tensile strain in the extreme tension steel is equal to or greater than ~~0.005~~the tension-controlled strain limit, ε_{tl} just as the concrete in compression reaches its assumed strain limit of 0.003. Sections with net tensile strain in the extreme tension steel between the compression-controlled strain limit and ~~0.005 constitute a transition region between compression-controlled and tension-controlled sections~~ the tension-controlled strain limit constitute a transition region between compression-controlled and tension-controlled sections. The tension-controlled strain limit, ε_{tl}, shall be taken as 0.005 for nonprestressed reinforcing steel with a specified yield strength, $f_y \leq$ 75.0 ksi and prestressed reinforcing steel. The tension-controlled strain limit, ε_{tl}, shall be taken as 0.008 for nonprestressed reinforcing steel with a specified yield strength, $f_y = 100.0$ ksi. For nonprestressed reinforcing steel with a specified yield strength between 75.0 and 100.0 ksi, the tension-controlled strain limit shall be determined by linear interpolation based on specified yield strength.

Figure C5.7.2.1-1—Strain Distribution and Net Tensile Strain

When the net tensile strain in the extreme tension steel is sufficiently large (equal to or greater than ~~0.005~~the tension-controlled strain limit), the section is defined as tension-controlled where ample warning of failure with excessive deflection and cracking may be expected. When the net tensile strain in the extreme tension steel is small (less than or equal to the compression-controlled strain limit), a brittle failure condition may be expected, with little warning of impending failure. Flexural members are usually tension-controlled, while compression members are usually compression-controlled. Some sections, such as those with small axial load and large bending moment, will have net tensile strain in the extreme tension steel between the above limits. These sections are in a transition region between compression- and tension-controlled sections. Article 5.5.4.2.1 specifies the appropriate resistance factors for tension-controlled and compression-controlled sections, and for intermediate cases in the transition region.

Before the development of these provisions, the limiting tensile strain for flexural members was not stated, but was implicit in the maximum reinforcement limit that was given as $c/d_e \leq 0.42$, which corresponded to a net tensile strain at the centroid of the tension reinforcement of 0.00414. The net tensile strain limit of 0.005 for tension-controlled sections was chosen to be a single value that applies to ~~all types of steel (prestressed and nonprestressed) permitted by this Specification.~~ nonprestressed reinforcing steel with a specified yield strength of 75.0 ksi or less and prestressed reinforcing steel. Research by Shahrooz, et al. (2010) and Mast (2008) supports the values stated for compression- and tension-controlled strain limits for steel with higher specified yield strengths.

Unless unusual amounts of ductility are required, the ~~0.005~~tension-controlled strain limit will provide ductile behavior for most designs. One condition where greater ductile behavior is required is in design for redistribution of moments in continuous members and frames. Article 5.7.3.5 permits redistribution of negative moments. Since moment redistribution is dependent on adequate ductility in hinge regions, moment redistribution is limited to sections that have a net tensile strain of at least ~~0.0075~~$1.5\varepsilon_{cl}$.

For beams with compression reinforcement, or T-beams, the effects of compression reinforcement and flanges are automatically accounted for in the computation of net tensile strain ε_t.

- In the approximate flexural resistance equations of Articles 5.7.3.1 and 5.7.3.2, f_y and f'_y may replace f_s and f'_s, respectively, subject to the following

When using the approximate flexural resistance equations in Articles 5.7.3.1 and 5.7.3.2, it is important to assure that both the tension and compression mild steel reinforcement are yielding to obtain accurate

conditions:

○ f_y may replace f_s when, using f_y in the calculation, the resulting ratio c/d_s does not exceed ~~0.6.~~:.

$$\frac{c}{d_s} \leq \frac{0.003}{0.003 + \varepsilon_{cl}} \qquad (5.7.2.1\text{-}1)$$

where:

c = distance from the extreme compression fiber to the neutral axis (in.)

d_s = distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement (in.)

ε_{cl} = compression-controlled strain limit as defined above.

If c/d_s exceeds ~~0.6~~this limit, strain compatibility shall be used to determine the stress in the mild steel tension reinforcement.

○ f'_y may replace f'_s when, using f'_y in the calculation, if $c \geq 3d'_s$~~.~~ and $f_y \leq 60.0$ ksi. If $c < 3d'_s$~~,~~ or $f_y > 60.0$ ksi, strain compatibility shall be used to determine the stress in the mild steel compression reinforcement. ~~The~~ Alternatively, the compression reinforcement ~~shall~~ may be conservatively ignored, i.e., $A'_s = 0$.

When using strain compatibility, the calculated stress in the nonprestressed reinforcing steel may not be taken as greater than the specified yield strength.

Additional limitations on the maximum usable extreme concrete compressive strain in hollow rectangular compression members shall be investigated as specified in Article 5.7.4.7.

5.7.3.2—Flexural Resistance

5.7.3.2.5—Strain Compatibility Approach

Alternatively, the strain compatibility approach may be used if more precise calculations are required. The appropriate provisions of Article 5.7.2.1 shall apply.

The stress and corresponding strain in any given layer of reinforcement may be taken from any representative stress-strain formula or graph for mild reinforcement and prestressing strands.

When using strain compatibility, the calculated stress in the nonprestressed reinforcing steel shall not

results. In previous editions of the *AASHTO LRFD Bridge Design Specifications*, the maximum reinforcement limit of $c/d_e \leq 0.42$ assured that the mild tension steel would yield at nominal flexural resistance, but this limit was eliminated in the 2006 interim revisions. The current limit ~~of~~ on $c/d_s \leq$ ~~0.6~~ assures that the mild tension steel will be at or near yield~~, while~~. The ratio $c \geq 3d'_s$ assures that ~~the~~ mild compression steel with $f_y \leq 60.0$ ksi will yield. For yield strengths above 60.0 ksi, the yield strain is close to or exceeds 0.003, so the compression steel may not yield. It is conservative to ignore the compression steel when calculating flexural resistance. In cases where either the tension or compression steel does not yield, it is more accurate to use a method based on the conditions of equilibrium and strain compatibility to determine the flexural resistance.

The mild steel tension reinforcement limitation does not apply to prestressing steel used as tension reinforcement. The equations used to determine the stress in the prestressing steel at nominal flexural resistance already consider the effect of the depth to the neutral axis.

excexed the specified yield strength.

5.7.3.3—Limits for Reinforcement

5.7.3.3.1—Maximum Reinforcement

[PROVISION DELETED IN 2005]

C5.7.3.3.1

In editions of and interims to the LRFD Specifications prior to 2005, Article 5.7.3.3.1 limited the tension reinforcement quantity to a maximum amount such that the ratio c/d_e did not exceed 0.42. Sections with $c/d_e > 0.42$ were considered over-reinforced. Over-reinforced nonprestressed members were not allowed, whereas prestressed and partially prestressed members with PPR greater than 50 percent were if "it is shown by analysis and experimentation that sufficient ductility of the structure can be achieved." No guidance was given for what "sufficient ductility" should be, and it was not clear what value of ϕ should be used for such over-reinforced members.

The current provisions of LRFD eliminate this limit and unify the design of prestressed and nonprestressed tension- and compression-controlled members. The background and basis for these provisions are given in Mast (1992). ~~Below a~~ When the net tensile strain in the extreme tension steel ~~of 0.005~~ is below the tension-controlled strain limit, as the tension reinforcement quantity increases, the factored resistance of prestressed and nonprestressed sections is reduced in accordance with Article 5.5.4.2.1. This reduction compensates for decreasing ductility with increasing overstrength. Only the addition of compression reinforcement in conjunction with additional tension reinforcement can result in an increase in the factored flexural resistance of the section.

5.7.3.4—Control of Cracking by Distribution of Reinforcement

The provisions specified herein shall apply to the reinforcement of all concrete components, except that of deck slabs designed in accordance with Article 9.7.2, in which tension in the cross-section exceeds 80 percent of the modulus of rupture, specified in Article 5.4.2.6, at applicable service limit state load combination specified in Table 3.4.1-1.

C5.7.3.4

All reinforced concrete members are subject to cracking under any load condition, including thermal effects and restraint of deformations, which produces tension in the gross section in excess of the cracking strength of the concrete. Locations particularly vulnerable to cracking include those where there is an abrupt change in section and intermediate post-tensioning anchorage zones.

Provisions specified, herein, are used for the distribution of tension reinforcement to control flexural cracking.

Crack width is inherently subject to wide scatter, even in careful laboratory work, and is influenced by shrinkage and other time-dependent effects. Steps should be taken in detailing of the reinforcement to control cracking. From the standpoint of appearance, many fine cracks are preferable to a few wide cracks. Improved crack control is obtained when the steel

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

$$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \qquad (5.7.3.4\text{-}1)$$

in which:

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

where:

γ_e = exposure factor
    = 1.00 for Class 1 exposure condition
    = 0.75 for Class 2 exposure condition

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)

f_{ss} = tensile stress in steel reinforcement at the service limit state (ksi) not to exceed 60.0 ksi.

h = overall thickness or depth of the component (in.)

d_ℓ = distance from the extreme compression fiber to the centroid of extreme tension steel element (in.)

Class 1 exposure condition applies when cracks can be tolerated due to reduced concerns of appearance and/or corrosion. Class 2 exposure condition applies to transverse design of segmental concrete box girders for any loads applied prior to attaining full nominal concrete strength and when there is increased concern of appearance and/or corrosion.

In the computation of d_c, the actual concrete cover thickness is to be used.

When computing the actual stress in the steel reinforcement, axial tension effects shall be considered, while axial compression effects may be considered.

The minimum and maximum spacing of reinforcement shall also comply with the provisions of Articles 5.10.3.1 and 5.10.3.2, respectively.

The effects of bonded prestressing steel may be considered, in which case the value of f_s used in Eq. 5.7.3.4-1, for the bonded prestressing steel, shall be the stress that develops beyond the decompression state calculated on the basis of a cracked section or strain

reinforcement is well distributed over the zone of maximum concrete tension. Several bars at moderate spacing are more effective in controlling cracking than one or two larger bars of equivalent area.

Extensive laboratory work involving deformed reinforcing bars has confirmed that the crack width at the service limit state is proportional to steel stress. However, the significant variables reflecting steel detailing were found to be the thickness of concrete cover and spacing of the reinforcement.

Eq. 5.7.3.4-1 is expected to provide a distribution of reinforcement that will control flexural cracking. The equation is based on a physical crack model (Frosch, 2001) rather than the statistically-based model used in previous editions of the specifications. It is written in a form emphasizing reinforcement details, i.e., limiting bar spacing, rather than crack width. Furthermore, the physical crack model has been shown to provide a more realistic estimate of crack widths for larger concrete covers compared to the previous equation (Destefano 2003).

Eq. 5.7.3.4-1 with Class 1 exposure condition is based on an assumed crack width of 0.017 in. Previous research indicates that there appears to be little or no correlation between crack width and corrosion, however, the different classes of exposure conditions have been so defined in order to provide flexibility in the application of these provisions to meet the needs of the Authority having jurisdiction. Class 1 exposure condition could be thought of as an upper bound in regards to crack width for appearance and corrosion. Areas that the Authority having jurisdiction may consider for Class 2 exposure condition would include decks and substructures exposed to water. The crack width is directly proportional to the γ_e exposure factor, therefore, if the individual Authority with jurisdiction desires an alternate crack width, the γ_e factor can be adjusted directly. For example a γ_e factor of 0.5 will result in an approximate crack width of 0.0085 in.

Where members are exposed to aggressive exposure or corrosive environments, additional protection beyond that provided by satisfying Eq. 5.7.3.4-1 may be provided by decreasing the permeability of the concrete and/or waterproofing the exposed surface.

Cracks in segmental concrete box girders may result from stresses due to handling and storing segments for precast construction and to stripping forms and supports from cast-in-place construction before attainment of the nominal f'_c.

The β_s factor, which is a geometric relationship between the crack width at the tension face versus the crack width at the reinforcement level, has been incorporated into the basic crack control equation in order to provide uniformity of application for flexural member depths ranging from thin slabs in box culverts to deep pier caps and thick footings. The theoretical definition of β_s may be used in lieu of the approximate expression provided.

compatibility analysis.

Where flanges of reinforced concrete T-girders and box girders are in tension at the service limit state, the flexural tension reinforcement shall be distributed over the lesser of:

- The effective flange width, specified in Article 4.6.2.6, or

- A width equal to 1/10 of the average of adjacent spans between bearings.

If the effective flange width exceeds 1/10 the span, additional longitudinal reinforcement, with area not less than 0.4 percent of the excess slab area, shall be provided in the outer portions of the flange.

If d_ℓ of nonprestressed or partially prestressed concrete members exceeds 3.0 ft, longitudinal skin reinforcement shall be uniformly distributed along both side faces of the component for a distance $d_\ell/2$ nearest the flexural tension reinforcement. The area of skin reinforcement A_{sk} in in.^{2}/ft of height on each side face shall satisfy:

$$A_{sk} \geq 0.012 \ (d_\ell - 30) \leq \frac{A_s + A_{ps}}{4} \qquad (5.7.3.4\text{-}2)$$

where:

A_{ps} = area of prestressing steel (in.^{2})

A_s = area of tensile reinforcement (in.^{2})

However, the total area of longitudinal skin reinforcement (per face) need not exceed one-fourth of the required flexural tensile reinforcement $A_s + A_{ps}$.

The maximum spacing of the skin reinforcement shall not exceed either $d_e/6$ or 12.0 in.

Such reinforcement may be included in strength computations if a strain compatibility analysis is made to determine stresses in the individual bars or wires.

5.7.3.5—Moment Redistribution

In lieu of more refined analysis, where bonded reinforcement that satisfies the provisions of Article 5.11 is provided at the internal supports of continuous reinforced concrete beams, negative moments determined by elastic theory at strength limit states may be increased or decreased by not more than

Research by Shahrooz et al. (2010) indicated that Eq. 5.7.3.4-1 can be applied to reinforcement with specified yield strengths up to 100.0 ksi but the tensile stress in the steel reinforcement at the service limit state, f_{ss}, cannot exceed 60.0 ksi. Moreover, when using reinforcing steel acknowledged to have greater resistance to corrosion (i.e., ASTM A955 and A1035), only Class 1 requirement needs to be satisfied.

Distribution of the negative reinforcement for control of cracking in T-girders should be made in the context of the following considerations:

- Wide spacing of the reinforcement across the full effective width of flange may cause some wide cracks to form in the slab near the web.

- Close spacing near the web leaves the outer regions of the flange unprotected.

The 1/10 of the span limitation is to guard against an excessive spacing of bars, with additional reinforcement required to protect the outer portions of the flange.

The requirements for skin reinforcement are based upon ACI 318-95. For relatively deep flexural members, some reinforcement should be placed near the vertical faces in the tension zone to control cracking in the web. Without such auxiliary steel, the width of the cracks in the web may greatly exceed the crack widths at the level of the flexural tension reinforcement.

C5.7.3.5

In editions and interims to the LRFD Specifications prior to 2005, Article 5.7.3.5 specified the permissible redistribution percentage in terms of the c/d_e ratio. The current specification specifies the permissible redistribution percentage in terms of net tensile strain ε_t. ~~The background and basis for these provisions are given~~

$1000\varepsilon_t$ percent, with a maximum of 20 percent. Redistribution of negative moments shall be made only when ε_t is equal to or greater than ~~0.0075~~ $1.5\varepsilon_{tl}$ at the section at which moment is reduced, where ε_{tl} is the tension-controlled strain limit defined in Article 5.7.2.1.

Positive moments shall be adjusted to account for the changes in negative moments to maintain equilibrium of loads and force effects.

~~in Mast (1992).~~

Mast (1992) provides an equation for the minimum strain in the tensile steel required for moment redistribution, which is based on the yield strain of the reinforcing steel and the assumption that $\rho/\rho_{bal} = 0.5$. Using reasonable values of yield strain for reinforcing steel with $f_y = 100.0$ ksi, the minimum tensile strain for moment redistribution can be found as 0.0012, which is $1.5\varepsilon_{tl}$ (Shahrooz, et al. 2010) Previous versions of this article set the minimum tensile strain at 0.0075 when the tension-controlled strain limit was set to 0.005. Thus, $1.5\varepsilon_{tl}$ gives the same value of minimum tensile strain for moment redistribution for reinforcing steel with $f_y \leq 75.0$ ksi as in previous editions, but also provides a reasonable value of minimum tensile strain for moment redistribution for higher strength reinforcing steels.

5.7.4.2—Limits for Reinforcement

C5.7.4.2

Additional limits on reinforcement for compression members in Seismic Zones 2, 3, and 4 shall be considered as specified in Articles 5.10.11.3 and 5.10.11.4.1a.

The maximum area of prestressed and nonprestressed longitudinal reinforcement for noncomposite compression components shall be such that:

$$\frac{A_s}{A_g} + \frac{A_{ps} f_{pu}}{A_g f_y} \leq 0.08 \qquad (5.7.4.2\text{-}1)$$

and:

$$\frac{A_{ps} f_{pe}}{A_g f_c'} \leq 0.30 \qquad (5.7.4.2\text{-}2)$$

The minimum area of prestressed and nonprestressed longitudinal reinforcement for noncomposite compression components shall be such that:

$$\frac{A_s f_y}{A_g f_c'} + \frac{A_{ps} f_{pu}}{A_g f_c'} \geq 0.135 \qquad (5.7.4.2\text{-}3)$$

where:

A_s = area of nonprestressed tension steel (in.²)

A_g = gross area of section (in.²)

A_{ps} = area of prestressing steel (in.²)

f_{pu} = specified tensile strength of prestressing steel (ksi)

According to current ACI codes, the area of longitudinal reinforcement for nonprestressed noncomposite compression components should be not less than $0.01 A_g$. Because the dimensioning of columns is primarily controlled by bending, this limitation does not account for the influence of the concrete compressive strength. To account for the compressive strength of concrete, the minimum reinforcement in flexural members is shown to be proportional to f_c'/f_y in Article 5.7.3.3.2. This approach is also reflected in the first term of Eq. 5.7.4.2-3. For fully prestressed members, current codes specify a minimum average prestress of 0.225 ksi. Here also the influence of compressive strength is not accounted for. A compressive strength of 5.0 ksi has been used as a basis for these provisions, and a weighted averaging procedure was used to arrive at the equation.

f_y =   specified yield strength of reinforcing bars (ksi)

f'_c =   specified compressive strength of concrete (ksi)

f_{pe} =   effective prestress (ksi)

The minimum number of longitudinal reinforcing bars in the body of a column shall be six in a circular arrangement and four in a rectangular arrangement. The minimum size of bar shall be No. 5.

For bridges in Seismic Zone 1, a reduced effective area may be used when the cross-section is larger than that required to resist the applied loading. The minimum percentage of total (prestressed and nonprestressed) longitudinal reinforcement of the reduced effective area is to be the greater of one percent or the value obtained from Eq. 5.7.4.2-3. Both the reduced effective area and the gross area must be capable of resisting all applicable load combinations from Table 3.4.1-1.

Where columns are pinned to their foundations, a small number of central bars have sometimes been used as a connection between footing and column.

For low risk seismic zones, the one percent reduced effective area rule, which has been used successfully since 1957 in the Standard Specifications, is implemented, but modified to account for the dependency of the minimum reinforcement on the ratio of f'_c / f_y.

For columns subjected to high, permanent axial compressive stresses where significant concrete creep is likely, using an amount of longitudinal reinforcement less than that given by Eq. 5.7.4.2-3 is not recommended because of the potential for significant transfer of load from the concrete to the reinforcement as discussed in the report of ACI Committee 105.

When using high strength reinforcing steel in axially loaded members, designers should account for the fact that these members may have smaller areas of steel and/or smaller gross dimensions for the same resistance. These reductions may affect axial deformations, slenderness effects, and/or the effects of creep and shrinkage.

5.7.4.4—Factored Axial Resistance

The factored axial resistance of concrete compressive components, symmetrical about both principal axes, shall be taken as:

$$P_r = \phi P_n \qquad (5.7.4.4\text{-}1)$$

in which:

- For members with spiral reinforcement:

$$P_n = 0.85 \left[\begin{array}{c} 0.85 f'_c \left(A_g - A_{st} - A_{ps} \right) \\ + f_y A_{st} - A_{ps} \left(f_{pe} - E_p \varepsilon_{cu} \right) \end{array} \right] \qquad (5.7.4.4\text{-}2)$$

- For members with tie reinforcement:

$$P_n = 0.80 \left[\begin{array}{c} 0.85 f'_c \left(A_g - A_{st} - A_{ps} \right) \\ + f_y A_{st} - A_{ps} \left(f_{pe} - E_p \varepsilon_{cu} \right) \end{array} \right] \qquad (5.7.4.4\text{-}3)$$

where:

P_r =   factored axial resistance, with or without flexure (kip)

C5.7.4.4

The values of 0.85 and 0.80 in Eqs. 5.7.4.4-2 and 5.7.4.4-3 place upper limits on the usable resistance of compression members to allow for unintended eccentricity.

In the absence of concurrent bending due to external loads or eccentric application of prestress, the ultimate strain on a compression member is constant across the entire cross-section. Prestressing causes compressive stresses in the concrete, which reduces the resistance of compression members to externally applied axial loads. The term, $E_p \varepsilon_{cu}$, accounts for the fact that a column or pile also shortens under externally applied loads, which serves to reduce the level of compression due to prestress. Assuming a concrete compressive strain at ultimate, ε_{cu} = 0.003, and a prestressing steel modulus, E_p = 28,500 ksi, gives a relatively constant value of 85.0 ksi for the amount of this reduction. Therefore, it is acceptable to reduce the effective prestressing by this amount. Conservatively, this reduction can be ignored.

Research by Shahrooz, et al. (2010) and Ward (2008) showed that these provisions are applicable to columns using reinforcing steel with specified yield strengths up to 100.0 ksi. Designers should account for

P_n = nominal axial resistance, with or without flexure (kip)

f'_c = specified strength of concrete at 28 days, unless another age is specified (ksi)

A_g = gross area of section (in.^{2})

A_{st} = total area of longitudinal reinforcement (in.^{2})

f_y = specified yield strength of reinforcement (ksi)

ϕ = resistance factor specified in Article 5.5.4.2

A_{ps} = area of prestressing steel (in.^{2})

E_p = modulus of elasticity of prestressing tendons (ksi)

f_{pe} = effective stress in prestressing steel after losses (ksi)

ε_{cu} = failure strain of concrete in compression (in./in.)

5.7.4.6—Spirals and Ties

The area of steel for spirals and ties in bridges in Seismic Zones 2, 3, or 4 shall comply with the requirements specified in Article 5.10.11.

Where the area of spiral and tie reinforcement is not controlled by:

- Seismic requirements,

- Shear or torsion as specified in Article 5.8, or

- Minimum requirements as specified in Article 5.10.6,

the ratio of spiral reinforcement to total volume of concrete core, measured out-to-out of spirals, shall satisfy:

$$\rho_s \geq 0.45 \left(\frac{A_g}{A_c} - 1 \right) \frac{f'_c}{f_{yh}} \qquad (5.7.4.6\text{-}1)$$

where:

A_g = gross area of concrete section (in.^{2})

A_c = area of core measured to the outside diameter of the spiral (in.^{2})

f'_c = specified strength of concrete at 28 days, unless another age is specified (ksi)

the fact that columns using high strength reinforcing steel may have smaller areas of steel and/or smaller gross dimensions for the same resistance. These reductions may affect axial deformations and/or slenderness effects.

f_{yh} = specified yield strength of spiral reinforcement (ksi) $\leq$ 100.0 ksi for Seismic Zone 1; $\leq$ 75.0 ksi for Seismic Zones 2, 3, and 4)

Other details of spiral and tie reinforcement shall conform to the provisions of Articles 5.10.6 and 5.10.11.

5.8—SHEAR AND TORSION

5.8.2.4—Regions Requiring Transverse Reinforcement

Except for slabs, footings, and culverts, transverse reinforcement shall be provided where:

- $V_u > 0.5\phi\left(V_c + V_p\right)$ (5.8.2.4-1)

    or

- Where consideration of torsion is required by Eq. 5.8.2.1-3 or Eq. 5.8.6.3-1

where:

V_u = factored shear force (kip)

V_c = nominal shear resistance of the concrete (kip)

V_p = component of prestressing force in direction of the shear force; V_p = 0 when the simplified method of 5.8.3.4.3 is used (kip)

ϕ = resistance factor specified in Article 5.5.4.2

For members subjected to flexural shear without torsion, reinforcing steel with specified yield strengths up to 100.0 ksi may be used for transverse reinforcement for Seismic Zone 1.

C5.8.2.4

Transverse reinforcement, which usually consists of stirrups, is required in all regions where there is a significant chance of diagonal cracking.

Testing by Shahrooz, et al. (2010) has verified the use of transverse reinforcement with specified yield strengths up to 100.0 ksi for both prestressed and nonprestressed members subjected to flexural shear without torsion for nonseismic applications. No torsional or combined torsion/flexure tests of members using transverse reinforcement with specified yield strengths between 75.0 and 100.0 ksi have been conducted.

5.8.2.5—Minimum Transverse Reinforcement

Except for segmental post-tensioned concrete box girder bridges, where transverse reinforcement is required, as specified in Article 5.8.2.4, the area of steel shall satisfy:

$$A_v \geq 0.0316\ \sqrt{f_c'}\ \frac{b_v s}{f_y}$$ (5.8.2.5-1)

where:

A_v = area of a transverse reinforcement within distance s (in.^{2})

C5.8.2.5

A minimum amount of transverse reinforcement is required to restrain the growth of diagonal cracking and to increase the ductility of the section. A larger amount of transverse reinforcement is required to control cracking as the concrete strength is increased.

Additional transverse reinforcement may be required for transverse web bending.

Testing by Shahrooz, et al. (2010) has verified these minimum values of transverse reinforcement for reinforcing steel with specified yield strengths up to 100.0 ksi for both prestressed and nonprestressed members subjected to flexural shear without torsion for

b_v = width of web adjusted for the presence of ducts as specified in Article 5.8.2.9 (in.)

s = spacing of transverse reinforcement (in.)

f_y = yield strength of transverse reinforcement (ksi) ≤ 100.0 ksi

For segmental post-tensioned concrete box girder bridges, where transverse reinforcement is required, as specified in Article 5.8.6.5, the area of transverse reinforcement shall satisfy:

$$A_v \geq 0.05 \frac{b_w s}{f_y} \qquad (5.8.2.5\text{-}2)$$

where:

A_v = area of a transverse shear reinforcement per web within distance s (in.^{2})

b_w = width of web (in.)

s = spacing of transverse reinforcement (in.)

f_y = yield strength of transverse reinforcement (ksi)

For segmental post-tensioned concrete box girder bridges, where transverse reinforcement is not required, as specified in Article 5.8.6.5, the minimum area of transverse shear reinforcement per web shall not be less than the equivalent of two No. 4 Grade 60 reinforcement bars per foot of length.

5.8.2.7—Maximum Spacing of Transverse Reinforcement

The spacing of the transverse reinforcement shall not exceed the maximum permitted spacing, s_{max}, determined as:

- If $v_u < 0.125 f'_c$, then:

$$s_{max} = 0.8d_v \leq 24.0 \text{ in.} \qquad (5.8.2.7\text{-}1)$$

- If $v_u \geq 0.125 f'_c$, then:

$$s_{max} = 0.4d_v \leq 12.0 \text{ in.} \qquad (5.8.2.7\text{-}2)$$

where:

v_u = the shear stress calculated in accordance with 5.8.2.9 (ksi)

d_v = effective shear depth as defined in Article 5.8.2.9 (in.)

nonseismic applications.

C5.8.2.7

Sections that are highly stressed in shear require more closely spaced reinforcement to provide crack control.

These spacing requirements were verified by Shahrooz, et al. (2010) for transverse reinforcement with specified yield strengths up to 100.0 ksi for prestressed and nonprestressed members subjected to flexural shear without torsion for nonseismic applications.

For segmental post-tensioned concrete box girder bridges, spacing of closed stirrups or closed ties required to resist shear effects due to torsional moments shall not exceed one-half of the shortest dimension of the cross-section, nor 12.0 in.

5.8.2.8—Design and Detailing Requirements

Transverse reinforcement shall be anchored at both ends in accordance with the provisions of Article 5.11.2.6. For composite flexural members, extension of beam shear reinforcement into the deck slab may be considered when determining if the development and anchorage provisions of Article 5.11.2.6 are satisfied.

For members in Seismic Zone 1 subjected to flexural shear without torsion, the design yield strength of nonprestressed transverse reinforcement shall be taken as the specified yield strength, but not to exceed 100.0 ksi.

In all other cases, the The design yield strength of nonprestressed transverse reinforcement shall be taken equal to the specified yield strength when the latter does not exceed 60.0 ksi. For nonprestressed transverse reinforcement with yield strength in excess of 60.0 ksi the design yield strength shall be taken as the stress corresponding to a strain of 0.0035, but not to exceed 75.0 ksi.

When welded wire reinforcement is used as transverse reinforcement, it shall be anchored at both ends in accordance with Article 5.11.2.6.3. No welded joints other than those required for anchorage shall be permitted.

Components of inclined flexural compression and/or flexural tension in variable depth members shall be considered when calculating shear resistance.

C5.8.2.8

To be effective, the transverse reinforcement should be anchored at each end in a manner that minimizes slip. Fatigue of welded wire reinforcement is not a concern in prestressed members as long as the specially fabricated reinforcement is detailed to have welded joints only in the flanges where shear stress is low.

Some of the provisions of Article 5.8.3 are based on the assumption that the strain in the transverse reinforcement has to attain a value of 0.002 to develop its yield strength. For prestressed tendons, it is the additional strain required to increase the stress above the effective stress caused by the prestress that is of concern. Limiting Previous versions limited the design yield strength of nonprestressed transverse reinforcement to 75.0 ksi or a stress corresponding to a strain of 0.0035 provides to provide control of crack widths at service limit state. For Shahrooz et al. (2010) compared the performance of transverse reinforcement without a well-defined with specified yield point, the yield strength is determined strengths of 60.0 ksi and 100.0 ksi in prestressed and nonprestressed members subjected to flexural shear only, under nonseismic conditions. The results do not show any discernable difference between the performance of the two types of transverse reinforcing steel at a strain of 0.0035 at either service or strength limit states. Research by Griezic (1994), Ma (2000), and Bruce (2003) has indicated that the performance of higher strength steels as shear reinforcement has been satisfactory. Use of relatively small diameter deformed welded wire reinforcement at relatively small spacing, compared to individually field tied reinforcing bars results in improved quality control and improved member performance in service.

The components in the direction of the applied shear of inclined flexural compression and inclined flexural tension can be accounted for in the same manner as the component of the longitudinal prestressing force, V_p.

5.8.3.3—Nominal Shear Resistance

The nominal shear resistance, V_n, shall be determined as the lesser of:

C5.8.3.3

The shear resistance of a concrete member may be separated into a component, V_c, that relies on tensile stresses in the concrete, a component, V_s, that relies on tensile stresses in the transverse reinforcement, and a

$$V_n = V_c + V_s + V_p \qquad (5.8.3.3\text{-}1)$$

$$V_n = 0.25 f'_c b_v d_v + V_p \qquad (5.8.3.3\text{-}2)$$

in which:

$V_c = 0.0316\beta\sqrt{f'_c}\ b_v d_v$, if the procedures of
Articles 5.8.3.4.1 or 5.8.3.4.2 are used $\qquad (5.8.3.3\text{-}3)$

V_c = the lesser of V_{ci} and V_{cw}, if the procedures of
Article 5.8.3.4.3 are used

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \qquad (5.8.3.3\text{-}4)$$

Where transverse reinforcement consists of a single longitudinal bar or a single group of parallel longitudinal bars bent up at the same distance from the support, the shear resistance V_s provided by these bars shall be determined as:

$$V_s = A_v f_y \sin\alpha \le 0.095\sqrt{f'_c}b_v d_v \qquad (5.8.3.3\text{-}5)$$

where:

b_v = effective web width taken as the minimum web width within the depth d_v as determined in Article 5.8.2.9 (in.)

d_v = effective shear depth as determined in Article 5.8.2.9 (in.)

s = spacing of transverse reinforcement measured in a direction parallel to the longitudinal reinforcement (in.)

β = factor indicating ability of diagonally cracked concrete to transmit tension and shear as specified in Article 5.8.3.4

θ = angle of inclination of diagonal compressive stresses as determined in Article 5.8.3.4 (degrees); if the procedures of Article 5.8.3.4.3 are used, $\cot\theta$ is defined therein

α = angle of inclination of transverse reinforcement to longitudinal axis (degrees)

A_v = area of shear reinforcement within a distance s (in.^{2})

V_p = component in the direction of the applied shear of the effective prestressing force; positive if resisting the applied shear; $V_p = 0$ when Article 5.8.3.4.3 is applied (kip)

component, V_p, that is the vertical component of the prestressing force.

The expressions for V_c and V_s apply to both prestressed and nonprestressed sections, with the terms β and θ depending on the applied loading and the properties of the section.

The upper limit of V_n, given by Eq. 5.8.3.3-2, is intended to ensure that the concrete in the web of the beam will not crush prior to yield of the transverse reinforcement.

where $\alpha = 90$ degrees, Eq. 5.8.3.3-4 reduces to:

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} \qquad (C5.8.3.3\text{-}1)$$

As noted in Article 5.8.2.4 for members subjected to flexural shear without torsion, transverse reinforcement with specified yield strengths up to 100.0 ksi is permitted for Seismic Zone 1.

The angle θ is, therefore, also taken as the angle between a strut and the longitudinal axis of a member.

V_p is part of V_{cw} by the method in Article 5.8.3.4.3 and thus V_p need be taken as zero in Eq. 5.8.3.3-1.

Where bent longitudinal reinforcement is used, only the center three-fourths of the inclined portion of the bent bar shall be considered effective for transverse reinforcement.

Where more than one type of transverse reinforcement is used to provide shear resistance in the same portion of a member, the shear resistance V_s shall be determined as the sum of V_s values computed from each type.

Where shear resistance is provided by bent longitudinal reinforcement or a combination of bent longitudinal reinforcement and stirrups, the nominal shear resistance shall be determined using the simplified procedure in accordance with Article 5.8.3.4.1.

Requirements for bent bars were added to make the provisions consistent with those in AASHTO (2002).

5.8.3.5—Longitudinal Reinforcement

At each section the tensile capacity of the longitudinal reinforcement on the flexural tension side of the member shall be proportioned to satisfy:

$$A_{ps}f_{ps} + A_s f_y \geq \frac{|M_u|}{d_v \phi_f} + 0.5 \frac{N_u}{\phi_c} + \left(\left| \frac{V_u}{\phi_v} - V_p \right| - 0.5 V_s \right) \cot \theta$$

$$(5.8.3.5-1)$$

where:

V_s = shear resistance provided by the transverse reinforcement at the section under investigation as given by Eq. 5.8.3.3-4, except V_s shall not be taken as greater than V_u/ϕ (kip)

θ = angle of inclination of diagonal compressive stresses used in determining the nominal shear resistance of the section under investigation as determined by Article 5.8.3.4 (degrees); if the procedures of Article 5.8.3.4.3 are used, cot θ is defined therein

ϕ_f, ϕ_v, ϕ_c = resistance factors taken from Article 5.5.4.2 as appropriate for moment, shear and axial resistance

The area of longitudinal reinforcement on the flexural tension side of the member need not exceed the area required to resist the maximum moment acting alone. This provision applies where the reaction force or

C5.8.3.5

Shear causes tension in the longitudinal reinforcement. For a given shear, this tension becomes larger as θ becomes smaller and as V_c becomes larger. The tension in the longitudinal reinforcement caused by the shear force can be visualized from a free-body diagram such as that shown in Figure C5.8.3.5-1.

Taking moments about Point 0 in Figure C5.8.3.5-1, assuming that the aggregate interlock force on the crack, which contributes to V_c, has a negligible moment about Point 0, and neglecting the small difference in location of V_u and V_p leads to the requirement for the tension force in the longitudinal reinforcement caused by shear.
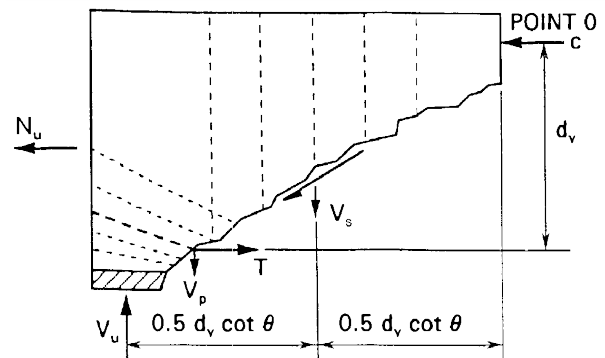

**Figure C5.8.3.5-1—Forces Assumed in Resistance Model Caused by Moment and Shear**

At maximum moment locations, the shear force changes sign, and hence the inclination of the diagonal compressive stresses changes. At direct supports including simply-supported girder ends and bent/pier

the load introduces direct compression into the flexural compression face of the member.

Eq. 5.8.3.5-1 shall be evaluated where simply-supported girders are made continuous for live loads. Where longitudinal reinforcement is discontinuous, Eq. 5.8.3.5-1 shall be re-evaluated.

caps pinned to columns, and at loads applied directly to the top or bottom face of the member, this change of inclination is associated with a fan-shaped pattern of compressive stresses radiating from the point load or the direct support as shown in Figure C5.8.3.5-2. This fanning of the diagonal stresses reduces the tension in the longitudinal reinforcement caused by the shear; i.e., angle θ becomes steeper. The tension in the reinforcement does not exceed that due to the maximum moment alone. Hence, the longitudinal reinforcement requirements can be met by extending the flexural reinforcement for a distance of $d_v \cot\theta$ or as specified in Article 5.11, whichever is greater.
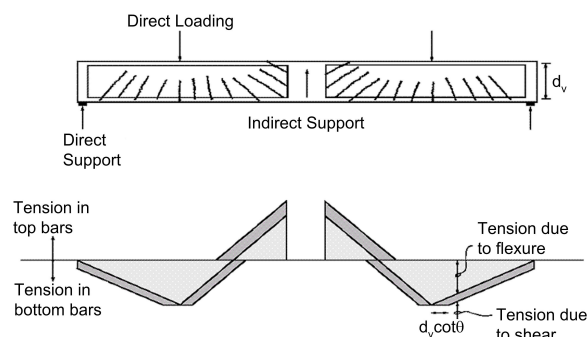
Figure C5.8.3.5-2—Force Variation in Longitudinal Reinforcement Near Maximum Moment Locations

At the inside edge of the bearing area of simple end supports to the section of critical shear, the longitudinal reinforcement on the flexural tension side of the member shall satisfy:

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\phi_v} - 0.5 V_s - V_p \right) \cot\theta \qquad (5.8.3.5-2)$$

Eqs. 5.8.3.5-1 and 5.8.3.5-2 shall be taken to apply to sections not subjected to torsion. Any lack of full development shall be accounted for.

The use of longitudinal and/or transverse reinforcing steel with specified yield strengths up to 100.0 ksi is permitted for Seismic Zone 1.

In determining the tensile force that the reinforcement is expected to resist at the inside edge of the bearing area, the values of V_u, V_s, V_p, and θ, calculated for the section d_v from the face of the support may be used. In calculating the tensile resistance of the longitudinal reinforcement, a linear variation of resistance over the development length of Article 5.11.2.1.1 or the bi-linear variation of resistance over the transfer and development length of Article 5.11.4.2 may be assumed.

This provision allows the use of longitudinal and transverse reinforcing steel with specified yield strengths up to 100.0 ksi for Seismic Zone 1; however, the use of higher strength longitudinal reinforcing steel may not be practical due to longer required development lengths.

5.8.4—Interface Shear Transfer—Shear Friction

5.8.4.1—General

Interface shear transfer shall be considered across a given plane at:

C5.8.4.1

Shear displacement along an interface plane may be resisted by cohesion, aggregate interlock, and shear-friction developed by the force in the reinforcement

- An existing or potential crack,

- An interface between dissimilar materials,

- An interface between two concretes cast at different times, or

- The interface between different elements of the cross-section.

If a member has transverse reinforcing steel with a specified yield strength greater than 60.0 ksi for flexural shear resistance, interface reinforcement may be provided by extending the transverse reinforcement across the interface zone. In this case, the value of f_y in Equation 5.8.4.1-3 shall not be taken as greater than 60.0 ksi.

REMAINDER OF THE ARTICLE IS UNCHANGED AND IS ELIMINATED FOR BREVITY.

crossing the plane of the interface. Roughness of the shear plane causes interface separation in a direction perpendicular to the interface plane. This separation induces tension in the reinforcement balanced by compressive stresses on the interface surfaces.

Adequate shear transfer reinforcement must be provided perpendicular to the vertical planes of web/flange interfaces in box girders to transfer flange longitudinal forces at the strength limit state. The factored design force for the interface reinforcement is calculated to account for the interface shear force, ΔF, as shown in Figure C5.8.4.1-1, as well as any localized shear effects due to the prestressing force anchorages at the section.
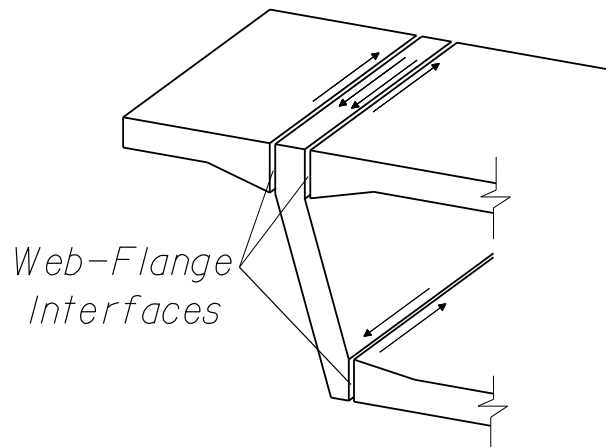
Figure C5.8.4.1-1—Longitudinal Shear Transfer between Flanges and Webs of Box Girder Bridges

5.10—DETAILS OF REINFORCEMENT

5.10.1—Concrete Cover

Minimum concrete cover shall be as specified in Article 5.12.3.

5.10.2—Hooks and Bends

5.10.2.1—Standard Hooks

For the purpose of these Specifications, the term "standard hook" shall mean one of the following:

- For longitudinal reinforcement:

  (a) 180- degree bend, plus a $4.0d_b$ extension, but not less than 2.5 in. at the free end of the bar, or

  (b) 90- degree bend, plus a $12.0d_b$ extension at the

C5.10.2.1

These requirements are consistent with the requirements of ACI 318 and CRSI's *Manual of Standard Practice*.

free end of the bar.

- For transverse reinforcement:

    (a) No. 5 bar and smaller—90-degree bend, plus a
        $6.0d_b$ extension at the free end of the bar,

    (b) No. 6, No. 7 and No. 8 bars—90-degree bend,
        plus a $12.0d_b$ extension at the free end of the
        bar; and

    (c) No. 8 bar and smaller—135-degree bend, plus a
        $6.0\ d_b$ extension at the free end of the bar.

where:

d_b = nominal diameter of reinforcing bar (in.)

Standard hooks may be used with reinforcing steel
having a specified yield strength up to 100.0 ksi.

Tests by Shahrooz et al. (2010) showed that
standard hooks are adequate for reinforcing steel with
specified yield strengths up to 100.0 ksi if transverse,
confining reinforcement is provided.

5.10.2.2—Seismic Hooks

Seismic hooks shall consist of a 135-degree bend,
plus an extension of not less than the larger of $6.0d_b$ or
3.0 in. Seismic hooks shall be used for transverse
reinforcement in regions of expected plastic hinges.
Such hooks and their required locations shall be detailed
in the contract documents.

For seismic hooks, f_y shall not be taken greater than
75.0 ksi.

Detailing of seismic hooks has not been verified for
reinforcing steels with yield strengths exceeding 75.0
ksi.

5.10.6—Transverse Reinforcement for Compression Members

5.10.6.1—General

The provisions of Article 5.10.11 shall also apply to
design and detailing in Seismic Zones 2, 3, and 4.

Transverse reinforcement for compression members
may consist of either spirals or ties.

In Seismic Zone 1, spirals and ties may be designed
for specified yield strengths up to 100.0 ksi.

C5.10.6.1

Article 5.10.11.2 applies to Seismic Zone 1 but has
no additional requirements for transverse reinforcement
for compression members.

Spirals and ties with specified yield strengths of up
to 100.0 ksi are permitted based on research by
Shahrooz et al. (2010). Since this research did not
consider seismic design, the use of high strength spirals
and ties is limited to Seismic Zone 1.

5.10.11.1—General

The provisions of these Articles shall apply only to the extreme event limit state.

In addition to the other requirements specified in Article 5.10, reinforcing steel shall also conform to the seismic resistance provisions specified herein.

Displacement requirements specified in Article 4.7.4.4 or longitudinal restrainers specified in Article 3.10.9.5 shall apply.

Bridges located in Seismic Zone 2 shall satisfy the requirements in Article 5.10.11.3. Bridges located in Seismic Zones 3 and 4 shall satisfy the requirements specified in Article 5.10.11.4.

The use of reinforcing steel with specified yield strengths of less than or equal to 100.0 ksi may be used in Seismic Zone 1, where permitted by specific articles.

C5.10.11.1

These Specifications are based on the work by the Applied Technology Council in 1979–1980. The Loma Prieta earthquake of 1989 provided new insights into the behavior of concrete details under seismic loads. The California Department of Transportation initiated a number of research projects that have produced information that is useful for both the design of new structures and the retrofitting of existing structures. Much of this information has formed the basis of recent provisions published by NCHRP (2002, 2006), MCEER/ATC (2003), and FHWA (2006).

This new information relates to all facets of seismic engineering, including design spectra, analytical techniques, and design details. Bridge designers working in Seismic Zones 2, 3, and 4 are encouraged to avail themselves of current research reports and other literature to augment these Specifications.

The Loma Prieta earthquake confirmed the vulnerability of columns with inadequate core confinement and inadequate anchorage of longitudinal reinforcement. New areas of concern that emerged include:

- Lack of adequate reinforcement for positive moments that may occur in the superstructure over monolithic supports when the structure is subjected to longitudinal dynamic loads;

- Lack of adequate strength in joints between columns and bent caps under transverse dynamic loads; and

- Inadequate reinforcement for torsion, particularly in outrigger-type bent caps.

The purpose of the additional design requirements of this Article is to increase the probability that the design of the components of a bridge are consistent with the overall design philosophy of ATC 6, especially for bridges located in Seismic Zones 2, 3, and 4, and that the potential for failures observed in past earthquakes is minimized. The additional column design requirements of this Article for bridges located in Seismic Zones 2, 3, and 4 are to ensure that a column is provided with reasonable ductility and is forced to yield in flexure and that the potential for a shear, compression, or loss of anchorage mode of failure is minimized. The additional design requirements for piers provide for some inelastic resistance; however, the R-factor specified for piers in Section 4 is to ensure that the anticipated inelastic resistance is significantly less than that of columns.

The actual ductility demand on a column or pier is a complex function of a number of variables, including:

- Earthquake characteristics,

- Design force level,

- Periods of vibration of the bridge,

- Shape of the inelastic hysteresis loop of the columns,

- Elastic damping coefficient,

- Contributions of foundation and soil conditions to structural flexibility, and

- Plastic hinge length of the column.

The damage potential of a column is also related to the ratio of the duration of strong motion shaking to the natural period of vibration of the bridge. This ratio will be an indicator of the number of yield excursions and hence of the cumulative ductility demand.

5.10.11.4.1d—Transverse Reinforcement for Confinement at Plastic Hinges

C5.10.11.4.1d

The cores of columns and pile bents shall be confined by transverse reinforcement in the expected plastic hinge regions. The transverse reinforcement for confinement shall have a yield strength not more than that of the longitudinal reinforcement, and the spacing shall be taken as specified in Article 5.10.11.4.1e.

For a circular column, the volumetric ratio of spiral or seismic hoop reinforcement, ρ_s, shall satisfy either that required in Article 5.7.4.6 or:

$$\rho_s \geq 0.12 \frac{f'_c}{f_y}$$ (5.10.11.4.1d-1)

where:

f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (ksi)

f_y = yield strength of reinforcing bars (ksi) ≤ 75.0 ksi

Within plastic hinge zones, splices in spiral reinforcement shall be made by full-welded splices or by full-mechanical connections.

Plastic hinge regions are generally located at the top and bottom of columns and pile bents. The largest of either these requirements or those of Article 5.10.11.4.1c should govern; these requirements are not in addition to those of Article 5.10.11.4.1c.

The main function of the transverse reinforcement specified in this Article is to ensure that the axial load carried by the column after spalling of the concrete cover will at least equal the load carried before spalling and to ensure that buckling of the longitudinal reinforcement is prevented. Thus, the spacing of the confining reinforcement is also important.

Careful detailing of the confining steel in the plastic hinge zone is required because of spalling and loss of concrete cover. With deformation associated with plastic hinging, the strains in the transverse reinforcement increase. Ultimate-level splices are required. Similarly, rectangular hoops should be anchored by bending ends back into the core.

Figures C5.10.11.4.1d-2 and C5.10.11.4.1d-4 illustrate the use of Eqs. 5.10.11.4.1d-2 and 5.10.11.4.1d-3. The required total area of hoop reinforcement should be determined for both principal axes of a rectangular or oblong column. Figure C5.10.11.4.1d-4 shows the distance to be utilized for h_c and the direction of the corresponding reinforcement for both principal directions of a rectangular

For a rectangular column, the total gross sectional area, A_{sh}, of rectangular hoop reinforcement shall satisfy either:

$$A_{sh} \geq 0.30 \, sh_c \frac{f'_c}{f_y} \left[\frac{A_g}{A_c} - 1 \right]$$ (5.10.11.4.1d-2)

or

$$A_{sh} \geq 0.12\, s h_c \frac{f_c'}{f_y} \qquad (5.10.11.4.1d\text{-}3)$$

where:

s = vertical spacing of hoops, not exceeding 4.0 in. (in.)

A_c = area of column core (in.^{2})

A_g = gross area of column (in.^{2})

A_{sh} = total cross-sectional area of tie reinforcement, including supplementary cross-ties having a vertical spacing of s and crossing a section having a core dimension of h_c (in.^{2})

f_y = yield strength of tie or spiral reinforcement (ksi) $\leq$ 75.0 ksi.

h_c = core dimension of tied column in the direction under consideration (in.)

A_{sh} shall be determined for both principal axes of a rectangular column.

Transverse hoop reinforcement may be provided by single or overlapping hoops. Cross-ties having the same bar size as the hoop may be used. Each end of the cross-tie shall engage a peripheral longitudinal reinforcing bar. All cross-ties shall have seismic hooks as specified in Article 5.10.2.2.

Transverse reinforcement meeting the following requirements shall be considered to be a cross-tie:

- The bar shall be a continuous bar having a hook of not less than 135 degrees, with an extension of not less than six diameters but not less than 3.0 in at one end and a hook of not less than 90 degrees with an extension of not less than six diameters at the other end.

- The hooks shall engage peripheral longitudinal bars.

- The 90-degree hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end.

Transverse reinforcement meeting the following requirements shall be considered to be a hoop:

- The bar shall be closed tie or continuously wound tie.

- A closed tie may be made up of several reinforcing

column.

While these Specifications allow the use of either spirals or ties for transverse column reinforcement, the use of spirals is recommended as the more effective and economical solution. Where more than one spiral cage is used to confine an oblong column core, the spirals should be interlocked with longitudinal bars as shown in Figure C5.10.11.4.1d-3. Spacing of longitudinal bars of a maximum of 8.0 in. center-to-center is also recommended to help confine the column core.

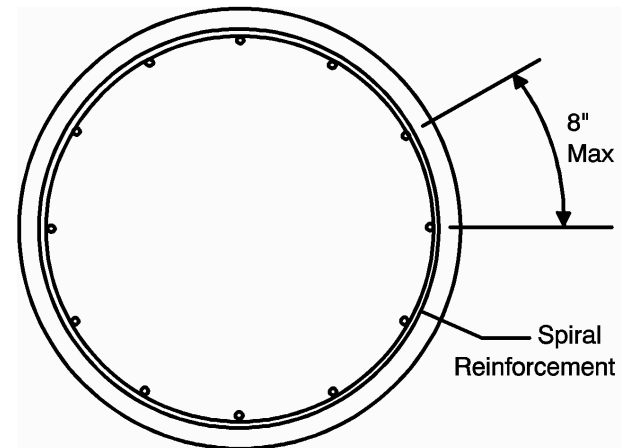Examples of transverse column reinforcement are shown herein.
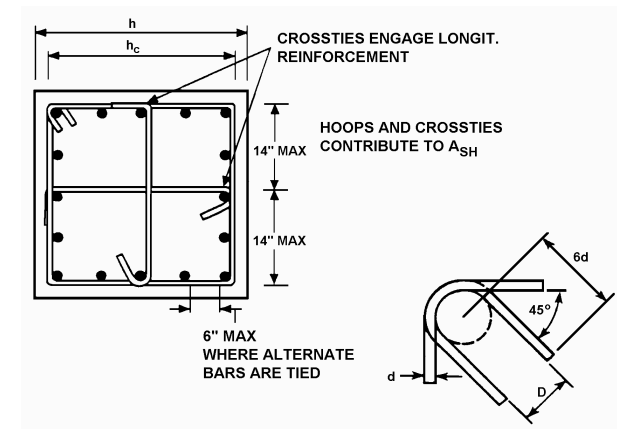

Figure C5.10.11.4.1d-1—Single Spiral

Figure C5.10.11.4.1d-2—Column Tie Details

elements with 135- degree hooks having a six diameter but not less than a 3.0 in. extension at each end.

- A continuously wound tie shall have at each end a 135-degree hook with a six diameter but not less than a 3.0 in. extension that engages the longitudinal reinforcement.
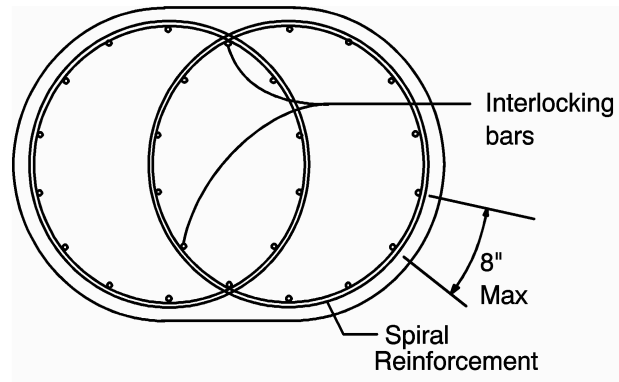

Figure C5.10.11.4.1d-3—Column Interlocking Spiral Details
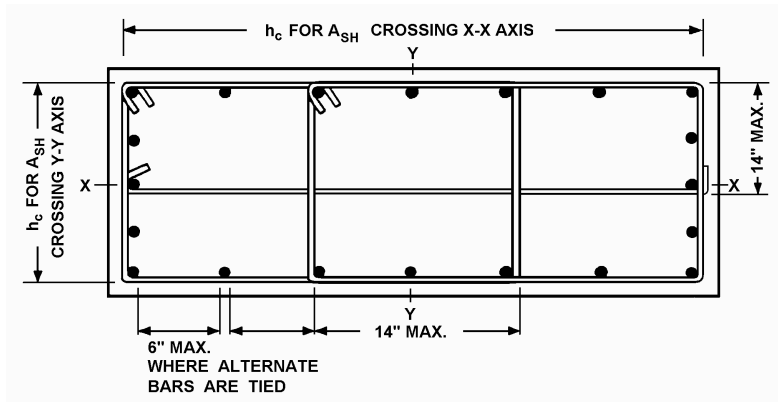

Figure C5.10.11.4.1d-4—Column Tie Details

5.11—DEVELOPMENT AND SPLICES OF REINFORCEMENT

5.11.1—General

5.11.1.1—Basic Requirements

The calculated force effects in the reinforcement at each section shall be developed on each side of that section by embedment length, hook, mechanical device, or a combination thereof. Hooks and mechanical anchorages may be used in developing bars in tension only. This article is applicable to nonprestressed reinforcing steel having a specified yield strength up to 100.0 ksi for Seismic Zone 1.

C5.11.1.1

Most of the provisions in this Article are based on ACI 318-89 and its attendant commentary. Shahrooz et al. (2010) verified that development length equations for reinforcing bars, splice lengths, and hooks in tension are applicable for specified yield strengths up to 100.0 ksi in nonseismic applications.

5.11.2—Development of Reinforcement

~~For reinforcement conforming to the requirements of ASTM A1035/A1035M, the value of f_y used in this Article shall be taken as 100 ksi.~~

~~Although the specified yield strength of reinforcing bars used in design shall not exceed 75.0 ksi, tests have shown that a longer development length is needed with reinforcement conforming to ASTM A1035/A1035M to achieve a ductility comparable to that achieved with~~

Development lengths shall be calculated using the specified yield strength of the reinforcing steel. Use of nonprestressed reinforcing steel with a specified yield strength up to 100.0 ksi is permitted for Seismic Zone 1.

~~reinforcement conforming to AASHTO M 31. Limited tests have shown a lack of ductility in tension splices of reinforcement conforming to ASTM A1035 when compared to the behavior of splices with reinforcement conforming to AASHTO M 31, when the splice length is calculated using the maximum design yield strength of 75.0 ksi. However, when the splice length of the ASTM A1035/A1035M reinforcement is determined using its specified minimum yield strength of 100 ksi, more ductility is achieved. Consequently, it is proposed to use 100 ksi until additional research indicates an alternative value.~~

Research by Shahrooz et al. (2010) showed that calculated tensile splice lengths and calculated tensile development lengths for both straight bars and standard hooks are adequate in nonseismic applications for reinforcing steels with yield strengths up to 100.0 ksi combined with concrete strengths up to 15.0 ksi.

5.11.2.1—Deformed Bars and Deformed Wire in Tension

For straight bars having a specified yield strength greater than 75.0 ksi, transverse reinforcement satisfying the requirements of Article 5.8.2.5 in beams and Article 5.10.6.3 in columns shall be provided over the required development length.

5.11.2.4—Standard Hooks in Tension

For hooks in reinforcing bars having a specified yield strength greater than 60.0 ksi, ties satisfying the requirements of Article 5.11.2.4.3 shall be provided. For hooks not located at the discontinuous end a member, the modification factors of Article 5.11.2.4.2 may be applied.

5.11.5—Splices of Bar Reinforcement

Use of reinforcing steel with specified yield strengths up to 100.0 ksi is permitted in this article for Seismic Zone 1. For spliced bars having a specified yield strength greater than 75.0 ksi, transverse reinforcement ~~conforming to~~ satisfying the requirements of ~~ASTM A1035/A1035M, the value of f_y used in this~~ Article 5.8.2.5 in beams and Article 5.10.6.3 in columns shall be ~~taken as 100 ksi.~~ provided over the required splice length.

C 5.11.5

Research by Shahrooz et al. (2010) verified the use of these provisions for tensile splices for reinforcing steels with specified yield strengths up to 100.0 ksi in nonseismic applications

5.11.5.3—Splices of Reinforcement in Tension

C5.11.5.3

The tension development length, ℓ_d, used as a basis for calculating splice lengths should include all of the modification factors specified in Article 5.11.2.

Research by Shahrooz et al. (2010) verified these provisions for nonseismic applications for reinforcing steels with specified yield strengths up to 100.0 ksi combined with concrete strengths up to 15.0 ksi.

5.11.5.3.1—Lap Splices in Tension

The length of lap for tension lap splices shall not be less than either 12.0 in. or the following for Class A, B or C splices:

Class A splice………………………………... 1.0 ℓ_d

Class B splice…………………………………1.3 ℓ_d

Class C splice…………………………………1.7 ℓ_d

The tension development length, ℓ_d, for the specified yield strength shall be taken in accordance with Article 5.11.2.

The class of lap splice required for deformed bars and deformed wire in tension shall be as specified in Table 5.11.5.3.1-1.

Table 5.11.5.3.1-1—Classes of Tension Lap Splices

Ratio of $\dfrac{(A_s \text{ as provided})}{(A_s \text{ as required})}$	Percent of A_s Spliced with Required Lap Length		
	50	75	100
≥2	A	A	B
<2	B	C	C

For splices having $f_y > 75.0$ ksi, transverse reinforcement satisfying the requirements of Article 5.8.2.5 in beams and Article 5.10.6.3 in columns shall be provided over the required development length.

5.11.5.3.2—Mechanical Connections or Welded Splices in Tension

Mechanical connections or welded tension splices, used where the area of reinforcement provided is less than twice that required, shall meet the requirements for full-mechanical connections or full-welded splices.

Mechanical connections or welded splices, used where the area of reinforcement provided is at least twice that required by analysis and where the splices are staggered at least 24.0 in., may be designed to develop not less than either twice the tensile force effect in the bar at the section or half the minimum specified yield strength of the reinforcement.

C5.11.5.3.2

In determining the tensile force effect developed at each section, spliced reinforcement may be considered to resist the specified splice strength. Unspliced reinforcement may be considered to resist the fraction of f_y defined by the ratio of the shorter actual development length to the development length, ℓ_d, required to develop the specified yield strength f_y.

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APPENDIX D5—ARTICLES MODIFIED TO ALLOW THE USE OF REINFORCING STEEL WITH A SPECIFIED YIELD STRENGTH UP TO 100 KSI

Article	Brief summary of changes
5.2 DEFINITIONS	Modified the definition of tension-controlled section by changing "0.005" to "tension-controlled strain limit". Added definition of tension-controlled strain limit.
5.3 NOTATION	Modified the definition of f_y to allow higher yield strengths. Added definitions of ε_{cl} and ε_{tl}; compression- and tension-controlled strain limits, respectively.
5.4.3.1 and C5.4.3.1 Reinforcing Steel, General	Permits the use of reinforcing steel with specified yield strengths up to 100.0 ksi when allowed by specific articles.
5.4.3.2 Reinforcing Steel, Modulus of Elasticity	E_s=29,000 may be used for specified yield strengths up to 100.0 ksi.
5.4.3.3 and C5.4.3.3 Reinforcing Steel, Special Applications	Permits the use of reinforcing steel with specified yield strengths up to 100.0 ksi in Seismic Zone 1.
5.5.3.2 and C5.5.3.2 Fatigue Limit State, Reinforcing Bars	Modifies the fatigue equation for reinforcing bars to allow the equation to be used for specified yield strengths up to 100.0 ksi.
5.5.4.2.1 and C5.5.4.2.1 Resistance factors, Conventional Construction	Allows the use of reinforcing steel with specified yield strengths up to 100.0 ksi in Seismic Zone 1. Modifies the equation, figure, and commentary for ϕ. These now use ε_{cl} and ε_{tl}, (compression- and tension-controlled strain limits) in place of 0.002 and 0.005.
5.7 and adds C5.7 DESIGN FOR FLEXURAL AND AXIAL FORCE EFFECTS	Allows the use of reinforcing steel with specified yield strengths up to 100.0 ksi in Seismic Zone 1.
5.7.2.1 and C5.7.2.1 Assumptions for Strength and Extreme Event Limit States	Keeps compression- and tension- controlled strain limits of 0.002 and 0.005 for reinforcing steels with specified yield strengths up to 60.0 and 75.0 ksi, respectively. Provides compression- and tension-controlled strain limits of 0.004 and 0.008 for reinforcing steel with a specified yield strength equal to 100.0 ksi. Linear interpolation is used for reinforcing steels with specified yield strengths between 60.0 or 75.0 ksi and 100.0 ksi. Equations are provided for when f_y may replace f_s or f_s' in 5.7.3.1 and 5.7.3.2.
5.7.3.2.5 Strain Compatibility Approach	Limits the steel stress in a strain compatibility calculation to the specified yield strength.
C5.7.3.3.1 Maximum Reinforcement	Replaces 0.005 with "tension-controlled strain limit".
5.7.3.5 and C5.7.3.5 Moment Redistribution	Adjusts strain limit to allow moment redistribution in structures using reinforcing steel with specified yield strengths up to 100.0 ksi.
C5.7.4.2 and C5.7.4.4. Limits for Reinforcement	Warns that designs should consider that columns using higher strength reinforcing steel may be smaller and have lower axial stiffness.
5.7.4.6 Spirals and Ties	Permits spirals and ties made of reinforcing steel with specified yield strengths up to 100.0 ksi in Seismic Zone 1.
5.8.2.4 and C5.8.2.4 Regions Requiring Transverse Reinforcement 5.8.2.5 and C5.8.2.5 Minimum Transverse Reinforcement	Permits transverse reinforcement with specified yield strengths up to 100.0 ksi in applications with flexural shear without torsion.
C5.8.2.7 Maximum Spacing of Transverse Reinforcement	Indicates that spacing requirements have been verified for transverse reinforcement with specified yield strengths up to 100.0 ksi in applications of shear without

	torsion.
5.8.2.8 and C5.8.2.8 Design and Detailing Requirements.	Permits transverse reinforcement with specified yield strengths up to 100.0 ksi in applications with flexural shear without torsion.
C5.8.3.3 Nominal Shear Resistance	Identifies that transverse reinforcement with specified yield strengths up to 100.0 ksi may be used in applications with flexural shear without torsion.
5.8.3.5 Longitudinal Reinforcement	Permits longitudinal reinforcing steel with specified yield strengths up to 100.0 ksi.
5.8.4.1 Interface Shear Transfer, General	Clarifies that f_y is limited to 60.0 ksi in Eq. 5.8.4.1.3.
5.10.2 and C5.10.2 Hooks and Bends	Permits hooks with specified yield strengths up to 100.0 ksi with transverse confining steel in Seismic Zone 1.
5.10.6.1 and C5.10.6.1 Transverse Reinforcement for Compression Members, General	Permits spirals with specified yield strengths up to 100.0 ksi in Seismic Zone 1.
5.10.11.1 Provisions for Seismic Design, General	Permits the use of reinforcing steel with specified yield strengths up to 100.0 ksi in Seismic Zone 1.
5.11.1.1 and C5.11.1.1 DEVELOPMENT AND SPLICES OF REINFORCEMENT, Basic Requirements 5.11.2 and C5.11.2 Development of Reinforcement	Permits the development length equations to be used for reinforcing steel with specified yield strengths up to 100.0 ksi.
5.11.2.1 Deformed Bar and Wire in Tension	Requires transverse confining steel for development of reinforcing steel with specified yield strengths greater than 75.0 ksi.
5.11.5 and adds C5.11.5 Splices of Bar Reinforcement 5.11.5.3 and C5.11.5.3 Splices of Reinforcement in Tension	Permits splices in reinforcing steel with specified yield strengths up to 100.0 ksi and requires transverse confining steel.
Table 5.11.5.3.1-1 Classes of Tension Lap Splices	Requires transverse confining steel in splices of reinforcing steel with specified yield strengths exceeding 75.0 ksi.