Reformatted according to Frosch comment #13 (note this change in numbering results in associated modifications to these references in other chapters).

6.2 Non prestressed reinforcing bars and wire
   6.2.1 Material properties
   6.2.2 Design properties

6.3 Prestressing strands, wires, and bars
   6.2.1 Material properties
   6.2.2 Design properties

6.4 Structural steel, pipe, and tubing for composite columns
   6.2.1 Material properties
   6.2.2 Design properties

CR063 – Modified to address results LB 12-2 as Approved ACI 318 Main Summer 2012. Highlighted text with underline strikeout denotes substantive changes to ACI 318-11 as approved.

Y: 15
C: 12
N: 12 (negatives addressed or withdrawn)
A: 2

Approved ACI 318 Summer Meeting in Detroit June 15, 2012

CHAPTER 6 — STEEL REINFORCEMENT PROPERTIES AND DURABILITY

6.1 — Scope

6.1.1 — Provisions of this chapter shall apply to steel reinforcement and govern
   (a) material properties;
   (b) properties to be used for design; and
   (c) durability requirements for reinforcement including minimum specified cover requirements. <~>

6.1.2 — Construction requirements for the placement and protection of steel reinforcement and embedments shall conform to 23.xx.xx and shall be included in contract documents in accordance with 23.yy.yy. <~>

The references above to be updated when Chapter 23 is completed.
This chapter has been updated to be consistent with ACI 318-11.

6.2 — Non prestressed bars and wires
6.2.1 — Material properties

6.2.1.1 — Nonprestressed bars and wires shall be deformed, except plain bars or wires are permitted for use in spirals. <3.5.1>

6.2.1.2 — Deformed bars shall conform to (a), (b), (c), (d), or (e).

(a) ASTM A615 – carbon steel

(b) ASTM A706 – low-alloy steel

(c) ASTM A996 – axle steel and rail steel. Bars from rail steel shall be Type R.

(d) ASTM A955 – stainless steel

(e) ASTM A1035 – low-carbon chromium steel

<3.5.3.1, 3.5.3.3>

Consider note in commentary that A1035 is for transverse reinforcement in 21.6.4 or spirals in 10.9.3 of ACI 318-11. This information is also included in Table 6.3.4(a)

6.2.1.2.1 — For ASTM A615, A706, A996, and A955 deformed bars with \( f_y \) less than 60,000 psi, yield strength shall be taken as the stress corresponding to a strain of 0.5 percent; for \( f_y \) of at least 60,000 psi, yield strength shall be taken as the stress corresponding to a strain of 0.35 percent. < 3.5.3.2>

6.2.1.3 — Plain bars for spiral reinforcement shall conform to ASTM A615, A706, A955, or A1035. <3.5.4.1>

6.2.1.4 — Welded deformed bar mats shall conform to ASTM A184. Deformed bars used in bar mats shall conform to ASTM A615 or A706. <3.5.3.4>

6.2.1.5 — Headed deformed bars shall conform to ASTM A970 including Annex A1 Requirements for Class HA Head Dimensions. <3.5.9>

6.2.1.6 — Deformed wire, plain wire, welded deformed wire reinforcement, and welded plain wire reinforcement shall conform to (a) or (b):

(a) A1064 – carbon steel

(b) A1022 – stainless steel

6.2.1.6.1 — For plain or deformed wire satisfying 6.2.1.6 with a specified yield strength greater than 60,000 psi, yield strength shall be taken as the stress corresponding to a strain of 0.35 percent. <ACI 318-11 3.5.3.5, 3.5.3.6, 3.5.3.7, 3.5.3.10, 3.5.4.2>

6.2.1.6.2 — Deformed wire sizes D-4 through D-31 shall be permitted.

6.2.1.6.3 — Deformed wire sizes larger than D-31 shall be permitted in welded wire reinforcement if treated as plain wire for calculation of development and splice lengths in accordance with 21.4.7 and 21.5.4, respectively. <3.5.3.6, 3.5.3.7, 3.5.3.11>
6.2.1.6.4 — Spacing of welded intersections in welded wire reinforcement in direction of calculated stress shall not exceed (a) or (b) except as permitted for welded wire reinforcement used as stirrups in accordance with 21.8.1. <3.5.3.6, 3.5.3.7, 3.5.3.10>

(a) 16 in. for welded deformed wire reinforcement, and
(b) 12 in. for welded plain wire reinforcement.

6.2.2 — Design properties

6.2.2.1 — For nonprestressed bars and wires, the stress below $f_y$ shall be taken as $E_s$ times steel strain. For strains greater than that corresponding to $f_y$, stress shall be considered independent of strain and equal to $f_y$. <10.2.4.10>

6.2.2.2 — Modulus of elasticity, $E_s$, for nonprestressed bars and wires shall be permitted to be taken as 29,000,000 psi. <8.5.2>

6.2.2.3 — Yield strength for nonprestressed bars and wires for use in design calculations shall be based on the grade of reinforcement specified by the licensed design professional and shall not exceed the values given in 6.2.2.4 for the associated applications. <~>

6.2.2.4 — Types of nonprestressed bars and wires to be specified for particular structural applications shall be in accordance with Table 6.2.2.4(a) for deformed reinforcement and Table 6.2.2.4(b) for plain reinforcement. <ACI 318-11 3.5.1; 3.5.3.3; 3.5.4.1; 9.4; 10.9.3; 11.4.2; 11.5.3.4; 11.6.6; 19.3.2; 21.1.5.1; 21.1.5.2; 21.1.5.4; and 21.1.5.5>

Table 6.2.2.4(a) —Nonprestressed deformed bars and wires

<table>
<thead>
<tr>
<th>Usage</th>
<th>Application</th>
<th>Maximum value of $f_y$ or $f_{yt}$ permitted for design calculations, psi</th>
<th>Deformed Bars</th>
<th>Deformed Wires</th>
<th>Welded wire</th>
<th>Welded bar mats</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexure, axial force, and shrinkage and temperature</td>
<td>Special seismic systems</td>
<td>60,000</td>
<td>A615§, A706</td>
<td>Not permitted</td>
<td>Not permitted</td>
<td>A184§</td>
</tr>
<tr>
<td></td>
<td>Other</td>
<td>80,000</td>
<td>A615, A706, A955, A996</td>
<td>A1064, A1022</td>
<td>A1064, A1022</td>
<td>A184§</td>
</tr>
<tr>
<td>Lateral support of longitudinal bars or Concrete confinement</td>
<td>Special seismic systems</td>
<td>100,000</td>
<td>A615, A706, A955, A996, A1035</td>
<td>A1064, A1022</td>
<td>A1064§, A1022§</td>
<td>Not permitted</td>
</tr>
<tr>
<td>Concrete confinement</td>
<td>Spirals</td>
<td>100,000</td>
<td>A615, A706, A955, A996,</td>
<td>A1064, A1022</td>
<td>Not permitted</td>
<td>Not permitted</td>
</tr>
<tr>
<td></td>
<td>A1035</td>
<td>A615, A706, A955, A996</td>
<td>A1064, A1022</td>
<td>A1064, A1022</td>
<td>Not permitted</td>
<td>(e)</td>
</tr>
<tr>
<td>----------------</td>
<td>-------</td>
<td>------------------------</td>
<td>--------------</td>
<td>--------------</td>
<td>---------------</td>
<td>----------</td>
</tr>
<tr>
<td>Other</td>
<td>80,000</td>
<td>A615, A706, A955, A996</td>
<td>A1064, A1022</td>
<td>A1064, A1022</td>
<td>Not permitted</td>
<td>(f)</td>
</tr>
<tr>
<td>Spirals</td>
<td>60,000</td>
<td>A615, A706, A955, A996</td>
<td>A1064, A1022</td>
<td>Not permitted</td>
<td>Not permitted</td>
<td>(f)</td>
</tr>
<tr>
<td>Shear friction</td>
<td>60,000</td>
<td>A615, A706, A955, A996</td>
<td>A1064, A1022</td>
<td>A1064, A1022</td>
<td>Not permitted</td>
<td>(f)</td>
</tr>
<tr>
<td>Stirrups, ties, hoops</td>
<td>60,000</td>
<td>A615, A706, A955, A996</td>
<td>A1064, A1022</td>
<td>A1064, A1022 (welded plain wire)</td>
<td>Not permitted</td>
<td>(h)</td>
</tr>
<tr>
<td></td>
<td>80,000</td>
<td>Not permitted</td>
<td>Not permitted</td>
<td>A1064, A1022 (welded deformed wire)</td>
<td>Not permitted</td>
<td>(i)</td>
</tr>
<tr>
<td>Torsion</td>
<td>Longitudinal and transverse</td>
<td>60,000</td>
<td>A615, A706, A955, A996</td>
<td>A1064, A1022</td>
<td>A1064, A1022</td>
<td>Not permitted</td>
</tr>
</tbody>
</table>

† ASTM A615 Grades 40 and 60 reinforcement conforming to 6.3.5 shall be permitted.<21.1.5.2>
§ Welded bar mats shall be permitted to be assembled using A615 or A706 deformed bars. <3.5.3.4>
‡ A1064, A1022 is only permitted where it is wrapped around the element, not where the weld is being called upon to resist shear in response to confinement or lateral buckling action. <~>
Table 6.2.2.4(b) —Nonprestressed plain bars and wires

<table>
<thead>
<tr>
<th>Usage</th>
<th>Application</th>
<th>Maximum value of $f_y$ or $f_{yt}$ permitted for design calculations, psi</th>
<th>Applicable ASTM Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral support of</td>
<td>Spirals in special seismic</td>
<td>100,000</td>
<td>A615, A706, A955, A1035</td>
</tr>
<tr>
<td>longitudinal bars</td>
<td>systems</td>
<td></td>
<td>A1064, A1022 (a)</td>
</tr>
<tr>
<td>or</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete confinement</td>
<td>Spirals</td>
<td>100,000</td>
<td>A615, A706, A955, A1035</td>
</tr>
<tr>
<td>Shear</td>
<td>Spirals</td>
<td>60,000</td>
<td>A615, A706, A955, A1035</td>
</tr>
<tr>
<td>Torsion in nonprestressed</td>
<td>Spirals</td>
<td>60,000</td>
<td>A615, A706, A955, A1035</td>
</tr>
<tr>
<td>beams</td>
<td></td>
<td></td>
<td>A1064, A1022 (d)</td>
</tr>
</tbody>
</table>

6.2.2.5 — ASTM A615 Grades 40 and 60 deformed reinforcement shall be permitted for longitudinal reinforcement in special seismic systems if the actual yield strength based on mill tests does not exceed $f_y$ by more than 18,000 psi and ratio of the actual tensile strength to the actual yield strength is at least 1.25. <21.1.5.2>

6.3 — Prestressing strands, wires, and bars

6.3.1 — Material properties

6.3.1.1 — Prestressing reinforcement shall conform to (a), (b), or (c): <3.5.6.1>

(a) ASTM A416 – strand
(b) ASTM A421 – wire including Supplementary Requirement S1 “Low-Relaxation Wire and Relaxation Testing”
(c) ASTM A722 – high-strength bar

6.3.1.2 — Prestressing strands, wires, and bars that do not conform to not specifically listed in ASTM A421, A416, or A722 are permitted provided they conform to minimum requirements of these specifications and are shown by test or analysis not to impair the performance of the member, and do not have properties that make them less satisfactory than those listed in ASTM
6.3.2 — Design properties

6.3.2.1 — Modulus of elasticity, $E_p$, for prestressing reinforcement shall be determined from tests or as reported by the manufacturer. <8.5.3> <~>

NOTE: Consider including information in the commentary on default values for $E_p$.

6.3.2.2 — Tensile strength, $f_{pu}$, for use in design calculations shall be based on the grade or type of prestressing reinforcement specified by the licensed design professional and shall not exceed the values given in Table 6.3.2.2. <3.5.6.1> <3.5.6.2> <18.1.1> <21.1.5.3>

Table 6.3.2.2 — Prestressing strands, wires, and bars

<table>
<thead>
<tr>
<th>Type</th>
<th>Maximum value of $f_{pu}$ permitted for design calculations, psi</th>
<th>Applicable ASTM Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strand (stress-relieved and low-relaxation)</td>
<td>270,000</td>
<td>A416 (a)</td>
</tr>
<tr>
<td>Wire (stress-relieved and low-relaxation)</td>
<td>250,000</td>
<td>A421 (b)</td>
</tr>
<tr>
<td>A421 including Supplementary Requirement S1 “Low-Relaxation Wire and Relaxation Testing”</td>
<td></td>
<td></td>
</tr>
<tr>
<td>High-strength bar</td>
<td>150,000</td>
<td>A722 (d)</td>
</tr>
</tbody>
</table>
Comment from Bondy to be considered for inclusion in commentary:

ASTM A421 specifies tensile strengths of 235,000 to 250,000 psi depending on the diameter and type of the wire. For the most common diameter, perhaps the ONLY diameter ever used in the US (0.25") ASTM A421 specifies a tensile strength of 240,000 psi.

Also consider including in commentary:
There are two grades of strand Gr 250 and Gr 270.

6.3.2.3—Stress in bonded prestressed reinforcement at nominal flexural strength, $f_{ps}$

6.3.2.3.1 — As an alternative to a more accurate determination of $f_{ps}$ based on strain compatibility, values of $f_{ps}$ calculated in accordance with Eq. (6.3.2.3.1) shall be permitted for members with bonded prestressed reinforcement if $f_{se} \geq 0.5 f_{pu}$. If compression reinforcement is considered for the calculation of $f_{ps}$ by Eq. (6.3.2.3.1), 6.3.2.3.1(a) and 6.3.2.3.1(b) shall apply.

$$f_{ps} = f_{pu} \left\{1 - \frac{\gamma_p}{\beta_p} \left[\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{\rho_p} \frac{f_y}{f'_c} (\rho - \rho')\right]\right\}$$

(6.3.2.3.1)

where $\gamma_p$ is in accordance with Table 6.3.2.3.1

<table>
<thead>
<tr>
<th>$f_{py} / f_{pu}$</th>
<th>$\gamma_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\geq 0.80$</td>
<td>0.55</td>
</tr>
<tr>
<td>$\geq 0.85$</td>
<td>0.40</td>
</tr>
<tr>
<td>$\geq 0.90$</td>
<td>0.28</td>
</tr>
</tbody>
</table>

6.3.2.3.1(a) — If $d'$ exceeds $0.15d_p$, the compression reinforcement shall be neglected in Eq. (6.3.2.3.1).

6.3.2.3.1(b) — If compression reinforcement is included in Eq. (6.3.2.3.1) the term

$$\left[\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{\rho_p} \frac{f_y}{f'_c} (\rho - \rho')\right]$$

shall not be taken less than 0.17.

6.3.2.3.2 — For pretensioned strands, the strand design stress at sections of members located within a distance $\ell_d$ from the free end of strand shall not exceed that calculated in accordance with 21.4.8.4.

6.3.2.4—Stress in unbonded prestressed reinforcement at nominal flexural strength, $f_{ps}$
6.3.2.4.1 — As an alternative to a more accurate determination of $f_{ps}$ based on strain compatibility, values of $f_{ps}$ calculated in accordance with Table 6.3.2.4.1 shall be permitted to be used for members prestressed with unbonded tendons if $f_{se} \geq 0.5 f_{pu}$. <18.7.2>

Table 6.3.2.4.1 — Approximate values of $f_{ps}$ at nominal flexural strength for unbonded tendons

<table>
<thead>
<tr>
<th>$\ell_u/h$</th>
<th>$f_{ps}$</th>
</tr>
</thead>
</table>
| $\leq 35$  | The least of:
|            | $f_{se} + 10,000 + \frac{f'_c}{100 \rho_p}$ (a) |
|            | ($f_{se} + 60,000$) (b) |
|            | $f_{py}$ (c) |
| $> 35$     | The least of:
|            | $f_{se} + 10,000 + \frac{f'_c}{300 \rho_p}$ (d) |
|            | ($f_{se} + 30,000$) (e) |
|            | $f_{py}$ (f) |

6.3.2.5 — Permissible tensile stresses in prestressed reinforcement

6.3.2.5.1 — The tensile stress in prestressed reinforcement during and immediately after stressing shall not exceed the limits in Table 6.3.1.5.1. <18.5.1>
### Table 6.3.2.5.1 — Maximum permissible tensile stresses in prestressed reinforcement

<table>
<thead>
<tr>
<th>Stage</th>
<th>Location</th>
<th>Maximum tensile stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>During stressing</td>
<td>At parking end</td>
<td>Least of:</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$0.94f_{py}$ (a)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$0.80f_{pu}$ (b)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maximum tensile stress recommended by the manufacturer of prestressing reinforcement steel or anchorage devices (c)</td>
</tr>
<tr>
<td>Immediately after force transfer</td>
<td>At post-tensioning anchorage devices and couplers</td>
<td>$0.70f_{pu}$ (d)</td>
</tr>
</tbody>
</table>

### 6.3.2.6 — Prestress losses

6.3.2.6.1 — Prestress losses shall be considered in calculation of the effective tensile stress in the prestressed reinforcement, $f_{se}$, and shall include: <18.6.1>

- (a) Prestressed reinforcement seating at transfer;
- (b) Elastic shortening of concrete;
- (c) Creep of concrete;
- (d) Shrinkage of concrete; and
- (e) Relaxation of prestressed reinforcement; and
- (f) Friction loss due to intended or unintended curvature in post-tensioning tendons.

6.3.2.6.2 — Calculated friction loss in post-tensioning tendons shall be based on experimentally determined wobble and curvature friction coefficients. < 18.6.2.2>

6.3.2.6.3 — Where loss of prestress in a member is anticipated due to connection of the member to adjoining construction, such loss of prestress shall be included in design calculations. <18.6.3>

### 6.4 — Structural steel, pipe, and tubing for composite columns
6.4—Material properties

6.4.1—Structural steel other than steel pipe or tubing used with reinforcing bars in composite columns with concrete encasing a steel core shall conform to (a), (b), (c), (d), or (e).<3.5.7.1>

(a) ASTM A36 – carbon steel;
(b) ASTM A242 – high-strength low-alloy steel;
(c) ASTM A572 – high-strength, low-alloy, columbium-vanadium steel;
(d) ASTM A588 – high-strength, low-alloy, 50 ksi steel;
(e) ASTM A992 – structural shapes.

6.4.1.2—Steel pipe or tubing used in composite columns to encase a concrete core shall conform to (a), (b) or (c).<3.5.7.2>

(a) ASTM A53 Grade B – black steel, hot-dipped, zinc-coated;
(b) ASTM A500 – cold-formed, welded, seamless;
(c) ASTM A501 – hot-formed, welded, seamless.

6.4.2—Design properties

6.4.2.1—For structural steel in composite columns, maximum value of $f_y$ used in design calculations shall be in accordance with the appropriate ASTM specifications in 6.4.1.<->

6.4.2.2—For structural steel used in composite columns with a structural steel core, with laterally tied or spirally reinforced concrete around a structural steel core, maximum value of $f_y$ used in design calculations shall not exceed 50,000 psi. <10.13.7.1> <10.13.8.1>

6.5—Headed shear stud reinforcement

6.5.1—Headed shear stud reinforcement and stud assemblies shall conform to ASTM A1044. <3.5.5.1>

Note: The value for $f_y$ was added to fit the flow of information provided in this chapter. Negatives brought up in CR062 were persuasive to move this material back to the commentary.

6.6—Anchors for connections to concrete

6.6.1—Material properties and design requirements for anchors used as connections to concrete shall comply with provisions of Chapter 19.<D.1>

6.7—Discontinuous deformed steel fiber reinforcement
6.7.1 — Discontinuous steel fiber reinforcement for concrete shall be deformed and shall conform to ASTM A820. Fibers shall have a length-to-diameter ratio of not smaller than 50 and not greater than 100. <3.5.8>

6.7.2 — Discontinuous deformed steel fibers conforming to 22.5.5 shall be permitted only for resisting shear under conditions specified in 13.6.3.1. <3.5.1>

Note: 6.7.2 as modified above was balloted and approved as CD022 ACI 318 Summer 2012.

Note: Suggested commentary for R6.7.2 (318-11 Section R3.5.1) — For other applications where it is desired to account for the strength or ductility provided by fibers, 1.10 provides a procedure for approval.

6.8 — Provisions for durability of steel reinforcement

6.8.1 — Specified concrete cover

6.8.1.1 — Unless the general building code requires a greater concrete cover for fire protection, the specified concrete cover shall not be less than required in 6.8. <7.7.1> <7.7.8>

6.8.1.2 — It shall be permitted to consider all concrete floor finishes as part of required cover for nonstructural purposes. <8.14.2>

Note: The second half of ACI 318-08 8.14.2 “or total thickness for nonstructural considerations” was not included with this section, and should be relocated to a more appropriate location in the code.

6.8.1.3 — Specified concrete cover requirements

6.8.1.3.1 — Non prestressed cast-in-place concrete members shall have specified concrete cover for reinforcement at least that given in Table 6.8.1.3.1. <7.7.1>
### Table 6.8.1.3.1 — Specified concrete cover for cast-in-place nonprestressed concrete members

<table>
<thead>
<tr>
<th>Concrete exposure</th>
<th>Member</th>
<th>Reinforcement</th>
<th>Specified Cover in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cast against and permanently in contact with ground</td>
<td>All</td>
<td>All</td>
<td>3 (a)</td>
</tr>
<tr>
<td>Exposed to weather or in contact with ground</td>
<td>All</td>
<td>No. 6 through No. 18 bars</td>
<td>2 (b)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No. 5 bar, W31 or D31 wire, and smaller</td>
<td>1-1/2 (c)</td>
</tr>
<tr>
<td>Not exposed to weather or in contact with ground</td>
<td>Slabs, joists and walls</td>
<td>No. 14 and No. 18 bars</td>
<td>1-1/2 (d)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No. 11 bar and smaller</td>
<td>3/4 (e)</td>
</tr>
<tr>
<td></td>
<td>Beams, columns, pedestals and tension ties</td>
<td>Primary reinforcement, stirrups, ties, spirals and hoops</td>
<td>1-1/2 (f)</td>
</tr>
</tbody>
</table>

### Table 6.8.1.3.2 — Specified concrete cover for cast-in-place prestressed concrete members

<table>
<thead>
<tr>
<th>Concrete exposure</th>
<th>Member</th>
<th>Reinforcement</th>
<th>Specified Cover in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cast against and permanently in contact with ground</td>
<td>All</td>
<td>All</td>
<td>3 (a)</td>
</tr>
<tr>
<td>Exposed to weather or in contact with ground</td>
<td>Slabs, joists and walls</td>
<td>All</td>
<td>1 (b)</td>
</tr>
<tr>
<td></td>
<td>All other</td>
<td>All</td>
<td>1-1/2 (c)</td>
</tr>
<tr>
<td>Not exposed to weather or in contact with ground</td>
<td>Slabs, joists and walls</td>
<td>All</td>
<td>3/4 (d)</td>
</tr>
<tr>
<td></td>
<td>Beams, columns and tension ties</td>
<td>Primary reinforcement, stirrups, ties, spirals and hoops</td>
<td>1-1/2 (e)</td>
</tr>
</tbody>
</table>

### 6.8.1.3.3 — Precast nonprestressed or prestressed concrete members manufactured under plant conditions shall have specified concrete cover for reinforcement, ducts, and end fittings at least that given in Table 6.8.1.3.3. <7.7.3>
Table 6.8.1.3.3 — Specified concrete cover for precast nonprestressed or prestressed concrete members manufactured under plant conditions

<table>
<thead>
<tr>
<th>Concrete exposure</th>
<th>Member</th>
<th>Reinforcement</th>
<th>Specified Cover in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exposed to weather or in contact with ground</td>
<td>Walls</td>
<td>No. 14 and No. 18 bars; Tendons larger than 1-1/2 in. diameter</td>
<td>1-1/2 (a)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No. 11 bars and smaller; W31 and D31 wire and smaller; Tendons and strands 1-1/2 in. diameter and smaller</td>
<td>3/4 (b)</td>
</tr>
<tr>
<td></td>
<td>All other</td>
<td>No. 14 and No. 18 bars; Tendons larger than 1-1/2 in. diameter</td>
<td>2 (c)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No. 6 through No. 11 bars; Tendons and strands larger than 5/8 in. diameter through 1-1/2 in. diameter</td>
<td>1-1/2 (d)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No. 5 bar, W31 or D31 wire, and smaller; Tendons and strands 5/8 in. diameter and smaller</td>
<td>1-1/4 (e)</td>
</tr>
<tr>
<td>Not exposed to weather or in contact with ground</td>
<td>Slabs, joists and walls</td>
<td>No. 14 and No. 18 bars; Tendons larger than 1-1/2 in. diameter</td>
<td>1-1/4 (f)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tendons and strands 1-1/2 in. diameter and smaller</td>
<td>3/4 (g)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No. 11 bar, W31 or D31 wire, and smaller</td>
<td>5/8 (h)</td>
</tr>
<tr>
<td></td>
<td>Beams, columns, pedestals and tension ties</td>
<td>Primary reinforcement</td>
<td>Greater of $d_b$ and 5/8 and need not exceed 1-1/2 (i)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Stirrups, ties, spirals and hoops</td>
<td>3/8 (j)</td>
</tr>
</tbody>
</table>

6.8.1.3.4 — For bundled bars, specified concrete cover shall not be less than the smaller of (a) and (b), and for concrete cast against and permanently in contact with ground, the specified cover shall be 3 in. <7.7.4>
(a) the equivalent diameter of the bundle
(b) 2 in.

6.8.1.3.5— For headed shear stud reinforcement, specified concrete cover for the heads or base rails shall be at least that required for the reinforcement in the member. <7.7.5>

6.8.1.3.6— For pipes, conduits and fittings, specified concrete cover shall be at least 1-1/2 in. for concrete exposed to weather or in contact with ground; and at least 3/4 in. for concrete not exposed to weather or in contact with ground. <6.3.10>

Remove ACI 318-11 <6.3.10> cover for pipes, conduits and fittings from Chapter 6 as per...
Seguirant N #152. Pipes, conduits and fittings do not fit the definition of reinforcement. This section is already in Chapter 23. Consider adding information in the commentary about other cover concerns in the code.

6.8.1.4 — Specified concrete cover requirements for corrosive environments

6.8.1.4.1 — In corrosive environments or other severe exposure conditions, the specified concrete cover shall be increased as deemed necessary and specified by the licensed design professional. The applicable requirements for concrete based on exposure categories in 5.3 shall be satisfied, or other protection shall be provided. <ACI 318-11 7.7.6>

6.8.1.4.2 — For prestressed concrete members classified as Class T or C in 10.5.2 and exposed to corrosive environments or other severe exposure categories such as those defined in 5.3, the specified concrete cover shall be at least 1.5 times the cover for prestressed reinforcement required by 6.8.1.3.2 for cast-in-place prestressed concrete members and 6.8.1.3.3 for precast concrete members. <7.7.6.1>

6.8.1.4.3 — The increased cover requirement of 6.8.1.4.2 need not be satisfied if the precompressed tensile zone is not in tension under sustained loads. <7.7.6.1>

6.8.2 — Nonprestressed coated reinforcement

6.8.2.1 — Nonprestressed coated reinforcement shall conform to Table 6.8.2.1. <3.5.3.8> <3.5.3.9>

Table 6.8.2.1 — Nonprestressed coated reinforcement

<table>
<thead>
<tr>
<th>Type of Coating</th>
<th>Applicable ASTM Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Bar</td>
</tr>
<tr>
<td>Zinc-Coated (Galvanized)</td>
<td>A767 Not permitted</td>
</tr>
<tr>
<td>Epoxy-Coated</td>
<td>A775 or A934</td>
</tr>
<tr>
<td>Zinc and Epoxy Dual-Coated</td>
<td>A1055 Not permitted</td>
</tr>
</tbody>
</table>

6.8.2.2 — Deformed bars to be zinc-coated (galvanized), epoxy-coated, or zinc and epoxy dual-coated shall conform to 6.2.1.2 (a), (b) or (c). <3.5.3.8>

6.8.2.3 — Wire and welded wire reinforcement to be epoxy-coated shall conform to 6.2.1.6(a). <3.5.3.9>

Note: Consider adding information in the commentary about other corrosion-resistant reinforcement (e.g., stainless steel, etc.)

6.8.3 — Corrosion protection for unbonded prestressing reinforcement
6.8.3.1 — Unbonded prestressing reinforcement shall be encased in sheathing, and the space between the strand and the sheathing shall be completely filled with a suitable material formulated to inhibit corrosion. Sheathing shall be watertight and continuous over the entire length to be unbonded. <18.16.1> <18.16.2>

6.8.3.2 — For applications in corrosive environments, the sheathing shall be connected to all stressing, intermediate, and fixed anchorages in a watertight fashion. <18.16.3>

6.8.3.3 — Unbonded single-strand tendons shall be protected against corrosion in accordance with ACI 423.7. <18.16.4>

6.8.4 — Corrosion protection for grouted tendons

6.8.4.1 — Ducts for grouted tendons shall be mortar grout-tight and nonreactive with concrete, prestressing reinforcement, grout, and corrosion inhibitor. <18.17.1>

6.8.4.2 — Ducts shall be maintained free of ponded water if members to be grouted are exposed to temperatures below freezing prior to grouting. <18.17.4>

6.8.4.3 — Ducts for grouted single-wire, single-strand, or single-bar tendons shall have an inside diameter at least 1/4 in. larger than the diameter of the prestressing reinforcement. <18.17.2>

6.8.4.4 — Ducts for grouted multiple wire, multiple strand, or multiple bar tendons shall have an inside cross-sectional area at least two times the cross-sectional area of the prestressing reinforcement. <18.17.3>

6.8.5 — Corrosion protection for post-tensioning anchorages, couplers, and end fittings.

6.8.5.1 — The licensed design professional shall detail Anchorages, couplers, and end fittings shall be protected to provide long-term resistance to corrosion to resist long-term corrosion. <18.21.4>

6.8.6 — Corrosion protection for external post-tensioning anchorages, couplers, and end fittings.

6.8.6.1 — External tendons and tendon anchorage regions shall be protected against corrosion. <18.22.4>
Chapter 6 Commentary was approved at ACI 318 Main meeting in San Antonio, Tuesday, January 15th. It is to be posted for 30-day review.

Ballot History:

Chapter 6 Commentary was balloted in LB 12-8 as CR064.
The result was Y/C/N/A: 17/13/8/3
N: Becker, Cleland, Fiorato, Frosch, Parra, Rabbat, Seguirant, Wyllie
A: Barth, Ghosh, Hwang
The negative ballots were resolved at the ACI 318 Main meeting in Toronto. The final document needed approval by ACI 318Main, but overall vote was not completed at the main meeting, so the commentary was sent out for approval in LB12-10.

*Note, to resolve Fiorato’s negative, he requested that “total thickness” of floor in existing 8.14.2 is addressed in appropriate chapter (e.g., as it relates to serviceability). See note associated with 6.8.1.2. This note is provided to ensure that the text is picked up by the appropriate chapter.*

Chapter 6 Commentary was reviewed in LB 12-10 as CR065.
The result was Y/C/N/A: 20/9/7/2
N: Dolan, Fiorato, Frosch, Holland, Moehle, Seguirant, Wyllie
A: Ghosh, Wood
The negative ballots and comments were resolved at the ACI 318 Main meeting in San Antonio. The document is to be posted for 30-day review. *The yellow highlighted changes shown below are those made to address the negatives and comments associated with LB12-10.*

CHAPTER 6 — STEEL REINFORCEMENT PROPERTIES AND DURABILITY

Commentary

R6.1.1 — Materials permitted for use as reinforcement are specified. Other metal elements, such as inserts, anchor bolts, or plain bars for dowels at isolation or contraction joints, are not normally considered to be [2] reinforcement under the provisions of this Code. Fiber-reinforced polymer (FRP) reinforcement is not addressed in this Code. ACI Committee 440 has developed guidelines for the use of FRP reinforcement.\(^3\)\(^2\),\(^3\)\(^3\) <Part of R3.5.1> <R3.5.1>

R6.2 — Nonprestressed bars and wires

R6.2.1 — Material properties

R6.2.1.2 — Low-alloy steel deformed bars conforming to ASTM A706 are intended for applications where controlled tensile properties, restrictions on chemical composition to enhance weldability, or both, are required.

Rail-steel deformed bars used with this Code are required to conform to ASTM A996 including the provisions for Type R bars. Type R bars are required to meet more restrictive provisions for bend tests than other types of rail steel.
Stainless steel deformed bars are used in applications where high corrosion resistance or controlled magnetic permeability are required. The physical and mechanical property requirements for stainless steel bars under ASTM A955 are the same as those for carbon-steel bars under ASTM A615. [R3.5.3.1]

Low-carbon chromium steel is a high-strength material that is permitted for use as transverse reinforcement for confinement in special earthquake-resistant structural systems and spirals in columns. See Tables 6.2.2.4(a) and (b). ASTM A1035 provides requirements [3] for bars of two minimum yield strength levels, 100,000 psi, and 120,000 psi designated as Grade 100 and Grade 120, respectively, but only Grade 100 is permitted by a maximum \( f_y \) of 100,000 psi is permitted for design calculations in this Code. [4,5,6]

**R6.2.1.2.1** — The ASTM specifications require that yield strength be determined by the offset method (0.2 percent offset) and also include, for bars with \( f_y \) of 60,000 psi or more, the additional requirement that the stress corresponding to a tensile strain of 0.35 percent be at least \( f_y \). The 0.35 percent strain limit is necessary to ensure that the assumption of an elasto-plastic stress-strain curve in 6.2.2.1 will not lead to unconservative [8] values of the member strength. Therefore, the Code defines yield strength in terms of the stress corresponding to a strain of 0.35 percent for \( f_y \) less than 60,000 psi and the stress corresponding to a strain of 0.35 percent for \( f_y \) of 60,000 psi or more [7]. [R3.5.3.2]

**Note to staff:**
In approved version of Code Chapter 6, Line 89, change 10.2.4.10 to 10.2.4.

**R6.2.1.3** — Plain bars are permitted only for spiral reinforcement either [10] used as transverse reinforcement for compression members, columns [12], transverse reinforcement for torsion, or and [11] confining reinforcement for splices. [R3.5.4]

**R6.2.1.5** — The limitation to Class HA head dimensions from Annex A1 of ASTM A970 is due to a lack of test data for headed deformed bars that do not meet Class HA dimensional requirements. Heads not conforming to Class HA limits on bar deformation obstructions and bearing face features could cause unintended splitting forces in the concrete that may not be characteristic of the heads used in the tests that were the basis for 21.4.4. For heads conforming to Class HA dimensional requirements, the net bearing area of the head can be assumed to be equal to the gross area of the head minus the area of the bar. This assumption may not be valid for heads not conforming to Class HA dimensional requirements. [R3.5.9]

**R6.2.1.6** — Plain wire is permitted only for spiral reinforcement and in welded plain [13]wire reinforcement, which is considered deformed. Stainless steel wire and stainless steel welded wire reinforcement are used in applications where high corrosion resistance or controlled magnetic permeability is required. The physical and mechanical property requirements for deformed stainless steel wire and deformed and plain welded wire reinforcement under ASTM A1022 are the same as those for deformed wire, deformed welded wire reinforcement, and plain welded wire reinforcement under ASTM A1064. [R3.5.3.10]

**R6.2.1.6.1** — Welded plain and deformed wire reinforcement is made of wire conforming to ASTM A1064, which specifies a minimum yield strength of 70,000 psi. The Code has assigned a yield strength value of 60,000 psi, but makes provision for the use of higher yield strengths provided the stress corresponds to a strain of 0.35 percent. See Tables 6.2.2.4(a) and (b). [R3.5.3.6, R3.5.3.7]

**R6.2.1.6.2** — An upper limit is placed on the size of deformed wire because tests show that D-45 wire will achieve only approximately 60 percent of the bond strength in tension given by Eq. (21.4.2.3.a). [R3.5.3.5]
R6.2.2 — Design properties

R6.2.2.1 — For deformed reinforcement, it is reasonably accurate to assume that the stress in reinforcement is proportional to strain below the specified yield strength $f_y$. The increase in strength due to the effect of strain hardening of the reinforcement is neglected for strength calculations. In strength calculations, the force developed in tension or compression reinforcement is calculated as:

if $\varepsilon_s < \varepsilon_y$ (yield strain)

$$A_s f_s = A_s E_s \varepsilon_s$$

if $\varepsilon_s \geq \varepsilon_y$

$$A_s f_s = A_s f_y$$

where $\varepsilon_s$ is the value from the strain diagram at the location of the reinforcement. <R10.2.4>

Note substantial change proposed to be made to table in Chapter 6:
Change the heading in Table 6.2.2.4(a) to “Nonprestressed deformed reinforcement bars and wires” Because it includes welded wire reinforcement, which is not clear from the current heading
Change the heading in Table 6.2.2.4(b) to “Nonprestressed plain spiral reinforcement bars and wires” Because it makes it consistent with proposed change to title of Table 6.2.2.4(a) and also makes clear it is only to be used for spiral reinforcement.

R6.2.2.4 — Tables 6.2.2.4(a) and (b) limit the maximum values of yield strength to be used in design calculations for nonprestressed deformed reinforcement and nonprestressed plain spiral reinforcement, respectively. [Added this sentence to provide better flow.]

In Table 6.2.2.4(a), for deformed reinforcement in special moment frames and special structural walls, the use of longitudinal reinforcement with strength substantially higher than that assumed in design will lead to higher shear and bond stresses at the time of development of yield moments. These conditions may lead to brittle failures in shear or bond and should be avoided even if such failures may occur at higher loads than those anticipated in design. Therefore, a ceiling limit [20] is placed on the actual yield strength of the steel [see 6.2.2.5]. ASTM A706 for low-alloy steel reinforcing bars now includes both Grade 60 and Grade 80; however, only Grade 60 is generally [22] permitted because of insufficient data to confirm applicability of existing code provisions for structures using the higher grade. Section 20.2.1.7 permits alternative material such as ASTM A706 Grade 80 if results of tests and analytical studies are presented in support of its use. <Part of R21.1.5> [23] For flexural members beams [24] the deflection provisions of 10.2 and the limitations on distribution of flexural reinforcement of 10.3 become increasingly critical as $f_y$ increases. <R9.4> [reordered this to be more consistent with Table 6.2.2.4(a).]

Limiting the values of $f_y$ and $f_{yt}$ used in design of deformed and plain spiral shear reinforcement to 60,000 psi and welded wire reinforcement to 80,000 psi for welded wire reinforcement (see Table 6.2.2.4(a)) and plain spiral shear reinforcement to 60,000 psi (see Table 6.2.2.4(b)) provides a control on
diagonal inclined [38] crack width. The higher yield strength for welded deformed wire reinforcement is
based on References 11.18-11.20. The research that has indicated that the performance of higher-strength
welded deformed wire reinforcement as shear reinforcement has been satisfactory is given in References
11.18-11.20. [35] In particular, full-scale beam tests described in Reference 11.19 indicated that the
widths of inclined shear cracks at service load levels were less for beams reinforced with smaller-
diameter welded deformed wire reinforcement cages designed on the basis of a yield strength of 75,000
psi than beams reinforced with deformed Grade 60 stirrups. <R11.4.2>

Limiting the values of \( f_y \) and \( f_{yt} \) used in design of deformed and plain spiral torsion reinforcement (see
Tables 6.2.2.4(a) and (b)) [40] to 60,000 psi provides a control on diagonal crack width. <R11.5.3.4>

Note to staff: Fix reference in Chapter 6 Table 6.2.2.4(a) footnote to reference 6.3.5-6.2.2.5 [41]

R6.2.2.5 — The requirement for the tensile strength to be greater than the yield strength of the
reinforcement by a factor of 1.25 is based on the assumption that the capability of a structural member to
develop inelastic rotation capacity is a function of the length of the yield region along the axis of the
member. In interpreting experimental results, the length of the yield region has been related to the relative
magnitudes of nominal probable and yield moments. [21.8] According to this interpretation, the greater the
ratio of nominal probable to yield moment, the longer the yield region. Members with reinforcement
not satisfying this condition can also develop inelastic rotation, but their behavior is sufficiently different
to exclude them from direct consideration on the basis of rules derived from experience with members
reinforced with strain-hardening steel. <Part of R21.1.5>

R6.3 — Prestressing strands, wires, and bars

R6.3.1 — Material properties

Fix Code side 6.3.1.1

Reason: Missing reference to ASTM A 421 wire excluding the low-relaxation wire supplement. See
commentary and Table 6.3.2.2 which has a separate line for ASTM 421 and ASTM 421 including
supplement.

6.3.1.1 — Prestressing reinforcement shall conform to (a), (b), or (c), or (d): <3.5.6.1>
(a) ASTM A416 – strand
(b) ASTM A421 - wire
(b) (c) ASTM A421 – low-relaxation wire including Supplementary Requirement S1 “Low-Relaxation Wire
and Relaxation Testing”
(c) (d) ASTM A722 – high-strength bar
<3.5.6.1>

R6.3.1.1 — Because low-relaxation prestressing steel reinforcement [45] is addressed in a supplementary
requirement to ASTM A421, which applies only when low-relaxation material is specified, the
appropriate ASTM reference is listed as a separate entity. <R3.5.6.1>

R6.3.2 — Design properties

R6.3.2.1 — Default values of \( E_p \) between 28,500,000 and 29,000,000 psi are commonly used for design
purposes. More precise accurate [46] values based on tests or manufacturer’s reports may be needed for
elongation checks during stressing.

Reason: ACI 318-11 21.1.5.3 limits prestressing steel resisting earthquake-induced flexural and axial
loads in frame members and in precast structural walls to ASTM A416 or A722. This is not clear from 6.3.2.2 so it should be clarified.

**Proposed changes:**

6.3.2.2 — Tensile strength, $f_{pu}$, for use in design calculations shall be based on the grade or type of prestressing reinforcement specified by the licensed design professional and shall not exceed the values given in Table 6.3.2.2. Prestressing reinforcement in members resisting earthquake-induced moment, axial force, or combinations of both \[49\] shall conform to ASTM A416 or A722. <3.5.6.1> <3.5.6.2> <18.1.1> <21.1.5.3>

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R6.3.2.2 — ASTM 416 specifies two grades of strand tensile strength, 250,000 and 270,000 psi ksi \[52\].

Proposed commentary from Bondy.

ASTM A421 specifies tensile strengths of 235,000, 240,000 and 250,000 psi depending on the diameter and type of wire. For the most common diameter, 0.25in., ASTM A421 specifies a tensile strength of 240,000 psi. Proposed commentary from Bondy.

R6.3.2.3—Stress in bonded prestressed reinforcement at nominal flexural strength, $f_{ps}$

R6.3.2.3.1 — Use of \[53\] Equation (6.3.2.3.1) may underestimate the strength of beams with high percentages of reinforcement and, for more accurate evaluations of their strength, the strain compatibility and equilibrium method should be used. Use of Eq. (6.3.2.3.1) is appropriate when all of the prestressed reinforcement is in the tension zone. When part of the prestressed reinforcement is in the compression zone, a strain compatibility and equilibrium method should be used.

The $\gamma_p$ term in Eq. (6.3.2.3.1) and Table 6.3.2.3.1 reflects the influence of different types of prestressing reinforcement on the value of $f_{ps}$. Table R6.3.2.3.1 \[54\] associates the shows prestressed reinforcement type with the associated ratio of $f_{py}/f_{pu}$.

<table>
<thead>
<tr>
<th>Prestressed Reinforcement Type</th>
<th>$f_{py}/f_{pu}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>High-strength prestressing bars</td>
<td>ASTM A722 Type I (Plain) [184]</td>
</tr>
<tr>
<td></td>
<td>ASTM A722 Type II (Deformed) [184]</td>
</tr>
<tr>
<td>Stress-relieved strand and wire</td>
<td>ASTM A416</td>
</tr>
<tr>
<td></td>
<td>ASTM A421</td>
</tr>
</tbody>
</table>

The $\rho^*$ term in Eq. (6.3.2.3.1) reflects the increased value of $f_{ps}$ obtained when compression reinforcement is provided in a beam with a large reinforcement index. \[R18.7.2\] The compression reinforcement may be conservatively neglected when using Eq. (6.3.2.3.1) by taking $\rho^*$ as zero. If the term $\left[\rho_p (f_{pu}/f_{y}) + (d/d_p)(f_{y}/f_{pu}) (p–\rho^*)\right]$ in Eq. (6.3.2.3.1) is small, the neutral axis depth is small, the compression reinforcement does not develop its yield strength, and Eq. (6.3.2.3.1) becomes unconservative. This is the reason why the term $\left[\rho_p (f_{ps}/f_{y}) + (d/d_p)(f_{y}/f_{ps}) (p–\rho^*)\right]$ may not be taken less than 0.17 if compression reinforcement is taken into account when computing $f_{ps}$. By taking $\rho^*$ as zero, then the term $\left[\rho_p (f_{ps}/f_{y}) + (d/d_p)(f_{y}/f_{ps}) (p–\rho^*)\right]$
(d/d_p) (f_y/f'_c) (ρ) may be less than 0.17 and an increased and correct value of fps is obtained. <R18.7.2>

R6.3.2.3.1(a) — When d' is large, the strain in compression reinforcement can be considerably less than its yield strain. In such a case, the compression reinforcement does not influence fps as favorably as implied by Eq. (6.3.2.3.1). For this reason, if d' exceeds 0.15d_p, the applicability of Eq. (6.3.2.3.1) is limited to beams in which d' is less than or equal to 0.15d_p only if the compression reinforcement is neglected [62].<Portion of R18.7.2>

R6.3.2.3.1(b) — The ρ' term in Eq. (6.3.2.3.1) reflects the increased value of fps obtained when compression reinforcement is provided in a beam with a large reinforcement index. <R18.7.2> If when the term [ρ_p (f_p/f'_c) + (d/d_p) (f_y/f'_c) (ρ - ρ')] in Eq. (6.3.2.3.1) is small, the neutral axis depth is small, the compressive reinforcement does not develop its yield strength, and Eq. (6.3.2.3.1) becomes unconservative. For this reason, the term [ρ_p (f_p/f'_c) + (d/d_p) (f_y/f'_c) (ρ - ρ')] in Eq. (6.3.2.3.1) may not be taken less than 0.17 if compression reinforcement is taken into account when calculating fps. <Portion of R18.7.2> The compression reinforcement may be conservatively neglected when using Eq. (6.3.2.3.1) by taking ρ' as zero, in which case the term [ρ_p (f_p/f'_c) + (d/d_p) (f_y/f'_c) (ρ)] may be less than 0.17 and an increased and correct acceptable value of fps is obtained. <R18.7.2> [58]

Fix references to Code side 6.3.2.3.2
Reason: Correct reference.

6.3.2.3.2 — For pretensioned strands, the strand design stress at sections of members located within a distance ℓ_d from the free end of strand shall not exceed that calculated in accordance with 21.4.8.4 21.4.8.3. <12.9.1.1>

R6.3.2.4 — Stress in unbonded prestressed reinforcement at nominal flexural strength, fps

R6.3.2.4.1 — The term [f_w + 10,000 + f'_c/(300 ρ_p)] reflects results of tests on members with unbonded tendons and span-to-depth ratios greater than 35 (one-way slabs, flat plates, and flat slabs). These tests also indicate that the term [f_w + 10,000 + f'_c/(100 ρ_p)], formerly used for all span-depth ratios, overestimates the amount of stress increase in such members. Although these same tests indicate that the moment strength of those shallow members designed using [f_w + 10,000 + f'_c/(100 ρ_p)] meets the factored load strength requirements, this reflects the effect of the Code requirements for minimum bonded reinforcement, as well as the limitation on concrete tensile stress that often controls the amount of prestressing force provided. <Portion of R18.7.2> <R18.7.2>

R6.3.2.5 — Permissible tensile stresses in prestressed reinforcement

Fix references to Code side 6.3.2.5.1
Reason: Correct reference.

6.3.2.5.1 — The tensile stress in prestressed reinforcement during and immediately after stressing shall not exceed the limits in Table 6.3.1.5-16.3.2.5.1. <18.5.1>

R6.3.2.5.1 — Because of the high yield strength of low-relaxation wire and strand and wire meeting the requirements of ASTM A416 and A421 including Supplementary Requirement S1
“Low-Relaxation Wire and Relaxation Testing” [68], it is appropriate to specify permissible stresses
in terms of specified minimum ASTM yield strength along with the specified minimum ASTM tensile
strength.

Because of the higher allowable initial prestressing steel stresses permitted since the 1983 Code, final
stresses can be greater. For structures subject to corrosive conditions or repeated loadings, limiting the
final stress should be considered [70]. <R18.5.1>

R6.3.2.6 — Prestress losses

R6.3.2.6.1 — For an explanation of how to calculate prestress losses, see References 18.6 through 18.9.
Reasonably accurate estimates of prestress losses can be calculated in accordance with the
recommendations in Reference 18.9, which include consideration of initial stress level (0.7fpu or higher),
type of steel (stress-relieved or low-relaxation wire, strand, or bar), exposure conditions, and type of
construction (pretensioned, bonded post-tensioned, or unbonded post-tensioned).

Actual losses, greater or smaller than the computed values, have little effect on the design strength of the
member, but affect service load behavior (deflections, camber, cracking load) and connections. At service
loads, overestimation of prestress losses can be almost as detrimental as underestimation, because the
former can result in excessive camber and horizontal movement. <R18.6.1>

R6.3.2.6.2 — Estimation of friction losses in post-tensioned tendons is addressed in Reference 18.10.
Values of the wobble and curvature friction coefficients to be used for the particular types of prestressing
reinforcement and particular types of ducts should be obtained from the manufacturers of the tendons. An
unrealistically low estimate of the friction loss can lead to improper camber, or potential deflection, of the
member and inadequate prestress. Overestimation of the friction may result in extra prestressing force.
This could lead to excessive camber and excessive shortening of a member. If the friction factors are
determined to be less than those assumed in the design, the tendon stressing should be adjusted to provide
only that prestressing force in the critical portions of the structure required by the design.

When safety or serviceability of the structure may be involved, the acceptable range of prestressing
reinforcement jacking forces or other limiting requirements should either be given or approved by the
licensed design professional in conformance with the permissible stresses of 6.3.2.5 and 10.5. <R18.6.2>

R6.4 — Structural steel, pipe, and tubing for composite columns

R6.4.1 — Material properties

R6.4.2 — Design properties

R6.4.2.2 — The design yield strength of the steel core should be limited to that which would not generate
spalling of the concrete. It has been assumed that axially compressed concrete will not spall at strains less
than 0.0018. The yield strength of 0.0018 × 29,000,000, or 52,000 psi, represents an upper limit of the
useful maximum steel stress. <1st paragraph of R10.13.8>

R6.5 — Headed shear stud reinforcement

R6.5.1 — The configuration of the studs for headed shear stud reinforcement differs from the
configuration of the headed-type shear studs prescribed in Section 7 of AWS D1.1 and referenced for use
in Chapter 19 of this Code (Fig. R6.5.1). Ratios of the head to shank cross-sectional areas of the AWS
D1.1 studs range from about 2.5 to 4. In contrast, ASTM A1044 requires the area of the head of headed
shear reinforcement studs to be at least 10 times the area of the shank. Thus, the AWS D1.1 headed studs are not suitable for use as headed shear stud reinforcement. The base rail, where provided, anchors one end of the studs; ASTM A1044 specifies material width and thickness of the base rail that are sufficient to provide the required anchorage without yielding for stud shank diameters of 0.375, 0.500, 0.625, and 0.750 in. In ASTM A1044, the minimum specified yield strength of headed shear studs is 51,000 psi.  

<The value for \( \sigma_{yf} \) was added to fit the flow of information provided in this chapter. Negatives brought up in CR062 were persuasive to move this material back to the commentary.>
bars if more than one layer is used without stirrups or ties; to the metal end fitting or duct on post-
tensioning prestressing steel tendons [79]; or to the outermost part of the head on headed bars.

The condition “exposed to weather or in contact with ground” refers to direct exposure to moisture
changes and not just to temperature changes. Slab or soffits are not usually considered directly
exposed unless subject to alternate wetting and drying, including that due to condensation conditions or
direct leakage from exposed top surface, run off, or similar effects.

Alternative methods of protecting the reinforcement from weather may be provided if they are equivalent
to the additional concrete cover required by the Code. When approved by the building official under the
provisions of 1.10, reinforcement with alternative protection from the weather may not have concrete
cover not less than the cover required for reinforcement not exposed to weather. [81]

Development length provisions given in Chapter 21 are a function of bar cover over the reinforcement. In
some cases, order to meet needs requirements for development length, it may be necessary to use larger
than minimum cover greater than the minimums specified in 6.8.1. [82] <R7.7>

Note:
6.8.1.2 — It shall be permitted to consider concrete floor finishes as part of required cover for
nonstructural purposes.
<8.14; 8.14.2>

The second half of ACI 318-08 8.14.2 “or total thickness for nonstructural considerations” was not
included with this section, and should be relocated to a more appropriate location in the code.>

R6.8.1.2 — Concrete floor finishes may be considered for nonstructural purposes such as cover for
reinforcement and fire protection. Provisions should be made, however, to ensure that the concrete finish
will not spall off, thus resulting in decreased cover. Furthermore, considerations for development of
reinforcement require [86] minimum monolithic concrete cover in accordance with 21.4.
<R8.14>

R6.8.1.3 — Specified concrete cover requirements

R6.8.1.3.3 — The lesser cover thicknesses for precast construction reflect the greater control for
proportioning, placing, and curing inherent in precasting. The term “manufactured under plant
conditions” does not specifically imply that precast members should be manufactured in a plant.
Structural elements precast at the job site will also qualify under this section if the control of form
dimensions, placing of reinforcement, quality control of concrete, and curing procedures [88] are equal to
that normally expected in a plant.

Concrete cover to pretensioned strand as described in this section is intended to provide minimum
protection against weather and other effects. Such cover may not be sufficient to transfer or develop the
stress in the strand, and it may be necessary to increase the cover accordingly. <R7.7.3>

R6.8.1.3.5 — Concrete cover requirements for headed shear stud reinforcement are illustrated in See
Figure R6.8.1.3.5. <Paraphrased portion of R7.7.5, other information in R7.7.5 should be located in
Chapter 9 on Sectional Strength or appropriate member chapters.>

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Fig. R6.8.1.3.5—Concrete cover requirements for headed shear stud reinforcement.

**R6.8.1.4 — Specified concrete cover requirements for corrosive environments**

Corrosive environments are defined in Sections 5.3.1, R5.3.1, and R5.3.2. Additional information on corrosion in parking structures is given in ACI 362.1R.7.12 <R7.7.6.1> [moved from R6.8.1.4.2]

**R6.8.1.4.1 —** Where concrete will be exposed to external sources of chlorides in service, such as deicing salts, brackish water, seawater, or spray from these sources, concrete should be proportioned to satisfy the requirements for the applicable exposure class in Chapter 5. **These include maximum w/cm, minimum strength for normalweight and lightweight concrete, and maximum chloride ion in concrete.** [90]

Additionally, for corrosion protection, a specified concrete cover for reinforcement not less than 2 in. for walls and slabs and not less than 2-1/2 in. for other members is recommended. For precast concrete members manufactured under plant control conditions, a specified concrete cover not less than 1-1/2 in. for walls and slabs and not less than 2 in. for other members is recommended. <R7.7.6>

**R6.8.2 — Nonprestressed coated reinforcement**

<Consider adding information in the commentary about other corrosion-resistant reinforcement (e.g., stainless steel, etc.>>
Zinc-coated (hot-dipped [92] galvanized) reinforcing bars (ASTM A767), epoxy-coated reinforcing bars (ASTM A775 and A934), and zinc and epoxy dual-coated reinforcing bars (ASTM A1055) are used in applications where corrosion resistance of reinforcement is of particular concern. They have typically been used such as [91] in parking structures, bridge structures, and other highly corrosive environments. <R3.5.3.8>

Corrosion protection for unbonded prestressing reinforcement

Material for corrosion protection of unbonded prestressing [93] reinforcement should have the properties identified in Section 5.1 of Reference 18.29. <R18.16.1>

Typically, sheathing is a continuous, seamless, high-density polyethylene material that is extruded directly onto the coated prestressing reinforcement [95]. <R18.16.2>

Corrosion protection for grouted tendons

Water in ducts may cause distress to the surrounding concrete upon freezing. When prestressing reinforcement is [97] present, ponded water in ducts should also be avoided. A corrosion inhibitor should be used to provide temporary corrosion protection if prestressing reinforcement is exposed to prolonged periods of moisture in the ducts before grouting. <R18.17.4>

Corrosion protection for post-tensioning anchorages, couplers, and end fittings.

For recommendations regarding protection, see Sections 4.2 and 4.3 of Reference 18.11 and Sections 3.4, 3.6, 5, 6, and 8.3 of Reference 18.29. <R18.21.4>

Permanent corrosion protection can be achieved by a variety of methods. The corrosion protection provided should be suitable to the environment in which the tendons are located. Some conditions will require that the prestressing reinforcement be protected by concrete cover or by cement grout in polyethylene or metal tubing; other conditions will permit the protection provided by coatings such as paint or grease. Corrosion protection methods should meet the fire protection requirements of the General Building Code, unless the installation of external post-tensioning is to only improve serviceability. <R18.22.4>