compression strength is required, the designer should increase member size, decrease framing member spacing, provide additional lateral support, or select a different grade and species of lumber with higher allowable stresses. Improving lateral support is usually the most efficient solution when stability controls the design (disregarding any architectural limitations). The need for improved lateral support will become evident when the designer performs the calculations necessary to determine the stability factor, C_P, in accordance with NDS•3.7. When a column has continuous lateral support in two directions, buckling is not an issue and C_p = 1.0. If, however, the column is free to buckle in one or more directions, C_p must be evaluated for each direction of possible buckling. The evaluation must also consider the spacing of intermediate bracing, if any, in each direction.

[NDS•3.7]

$f_c \le F_c'$ basic design check for compression parallel to grain

$F_c' = F_c$ x (applicable adjustment factors from Section 5.2.4, including C_p)

$f_c = \dfrac{P}{A}$ compressive stress parallel to grain due to axial load, P, acting on the member's cross-sectional area, A.

$$C_p = \frac{1+\left(F_{cE}/F_c^*\right)}{2c} - \sqrt{\left[\frac{1+\left(F_{cE}/F_c^*\right)}{2c}\right]^2 - \frac{F_{cE}/F_c^*}{c}}$$ column stability factor

$$F_{cE} = \frac{K_{cE}E'}{\left(\ell_e/d\right)^2}$$

$F_c^* = F_c$ x (same adjustment factors for F'_c except C_p is not used)

Tension

Relatively few members in light-frame construction resist tension forces only. One notable exception occurs in roof framing where cross-ties or bottom chords in trusses primarily resist tension forces. Other examples include chord and collector members in shear walls and horizontal diaphragms as discussed in Chapter 6. Another possibility is a member subject to excessive uplift loads such as those produced by extreme wind. In any event, connection design is usually the limiting factor in designing the transfer of tension forces in light-frame construction (refer to Chapter 7). Tension stresses in wood members are checked by using the equations below in accordance with NDS•3.8.

[NDS•3.8]

$f_t \le F_t'$ basic design check for tension parallel to grain

$F_t' = F_t$ x (applicable adjustment factors per Section 5.2.4)

$f_t = \dfrac{P}{A}$ stress in tension parallel to gain due to axial tension load, P, acting on the member's cross-sectional area, A

The NDS does not provide explicit methods for evaluating cross-grain tension forces and generally recommends the avoidance of cross-grain tension in lumber even though the material is capable of resisting limited cross-grain stresses. Design values for cross-grain tension may be approximated by using one-third of the unadjusted horizontal shear stress value, F_v. One application of cross-grain tension in design is in the transfer of moderate uplift loads from wind through the band or rim joist of a floor to the construction below. If additional cross-grain tension strength is required, the designer should increase member size or consider alternative construction details that reduce cross-grain tension forces. When excessive tension stress perpendicular to grain cannot be avoided, the use of mechanical reinforcement or design detailing to reduce the cross-grain tension forces is considered good practice (particularly in high-hazard seismic regions) to ensure that brittle failures do not occur.

5.3.2 Structural Serviceability

Deflection Due to Bending

The NDS does not specifically limit deflection but rather defers to designer judgment or building code specifications. Nonetheless, with many interior and exterior finishes susceptible to damage by large deflections, reasonable deflection limits based on design loads are recommended herein for the design of specific elements.

The calculation of member deflection is based on the section properties of the beam from NDS-S and the member's modulus of elasticity with applicable adjustments. Generally, a deflection check using the equations below is based on the estimated maximum deflection under a specified loading condition. Given that wood exhibits time- and load-magnitude-dependent permanent deflection (creep), the total long-term deflection can be estimated in terms of two components of the load related to short- and long-term deflection using recommendations provided in NDS•3.5.

[NDS•3.5]

$$\Delta_{estimate} \leq \Delta_{allow} = \frac{\ell}{(120 \text{ to } 600)} \quad \text{(see Table 5.5 for value of denominator)}$$

$$\Delta_{estimate} \cong f\left(\frac{\text{load and span}}{\text{EI}}\right) \quad \text{(see beam equations in Appendix A)}$$

If a deflection check proves unacceptable, the designer may increase member depth, decrease the clear span or spacing of the member, or select a grade and species of wood with a higher modulus of elasticity (the least effective option). Typical denominator values used in the deflection equation range from 120 to 600 depending on application and designer judgment. Table 5.5 provides recommended deflection limits. Certainly, if a modest adjustment to a deflection limit results in a more efficient design, the designer should exercise discretion with respect to a possible negative consequence such as vibration or long-term creep. For lateral bending loads on walls, a serviceability load for a deflection check may be considered as a fraction of the nominal design wind load for