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AMERICAN WOOD COUNCIL
*Engineered and Traditional Wood Products*

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# 1995 ERRATA
# to
# 1993 Edition of

## *COMMENTARY*
## *on the*
## *NATIONAL DESIGN SPECIFICATION® (NDS®)*
## *FOR WOOD CONSTRUCTION*
## *1991 EDITION*

The following pages replace corresponding existing pages of the *NDS Commentary*. Shaded text indicates provisions that have changed.

**Example C3.3-1**

A Select Structural Southern Pine 4×16 beam on a 20 ft span supports a hoist located at the center of the span. Determine the maximum allowable load on the hoist (including its weight) based on bending. Assume normal load duration. Lateral support is provided at the ends only.

$F_b$ = 1900 psi $E$ = 1,800,000 psi (Table 4B)

$C_F$ = (0.9)(1.1) $C_D$ = 1.0 $A$ = 53.38 in² $S$ = 135.7 in³

**Beam Stability Factor $C_L$** (3.3.3)

$$F_b^* = F_bC_DC_F = (1900)(1.0)(0.9)(1.1) = 1880 \text{ psi}$$

$$\ell_u/d = (20)(12)/(15.25) = 15.7 > 7$$

$$\ell_e = 1.37\ell_u + 3d \quad \text{(Table 3.3.3)}$$

$$= 1.37(20)(12) + 3(15.25) = 374.6 \text{ in.}$$

$$R_B = \sqrt{\frac{\ell_e d}{b^2}} = \sqrt{\frac{(374.6)(15.25)}{(3.5)^2}} = 21.6$$

$$K_{bE} = 0.438$$

$$F_{bE} = \frac{K_{bE}E'}{R_b^2} = \frac{(0.438)(1{,}800{,}000)}{(21.6)^2} = 1691 \text{ psi}$$

$$C_L = \frac{1+(F_{bE}/F_b^*)}{1.9} - \sqrt{\left[\frac{1+(F_{bE}/F_b^*)}{1.9}\right]^2 - \frac{F_{bE}/F_b^*}{0.95}}$$

$$= \frac{1+1691/1880}{1.9} - \sqrt{\left[\frac{1+1691/1880}{1.9}\right]^2 - \frac{1691/1880}{0.95}}$$

$$= 0.770$$

**Allowable Bending Design Value, $F_b'$** (Table 2.3.1)

$$F_b' = F_bC_DC_LC_F = (1900)(1.0)(0.770)(0.9)(1.1)$$

$$= 1448 \text{ psi}$$

**Maximum Moment**

Assume density of beam = 37.5 lb/ft³

weight of beam = (37.5)(53.38/144) = 13.9 lb/ft

hoist plus payload = $P$

$$M_{max} = P\ell/4 + w\ell^2/8$$

$$= (P)(20)(12)/4 + (13.9)(20)^2(12)/8$$

$$= 60P + 8340$$

**Maximum Allowable Load**

$$M_{allow.} = F_b' S$$

Substituting,

$$P = (F_b' S - 8340)/60$$

$$= ((1448)(135.7) - 8340)/60$$

$$= 3136 \text{ lb} \sim 3100 \text{ lb}$$

**Total allowable concentrated load = 3100 lb (hoist plus payload)**

**Example C3.3-2**

A Select Structural Southern Pine 2×14 beam on a 16 ft span carries five 500 lb (DL+SL) concentrated loads from purlins spaced at 32 in. on center (1/6 points). Determine if the member is adequate for bending. Lateral support is provided at the purlins and the supports.

$F_b$ = 1900 psi $E$ = 1,800,000 psi (Table 4B)

$C_F$ = 0.9 $C_D$ = 1.15 $A$ = 19.88 in² $S$ = 43.89 in³

**Beam Stability Factor $C_L$** (3.3.3)

$$F_b^* = F_bC_DC_F = (1900)(1.15)(0.9) = 1967 \text{ psi}$$

$$\ell_u = 32 \text{ in.}$$

$$\ell_e = 1.73\ell_u = 1.73(32) = 55.4 \text{ in.} \quad \text{(Table 3.3.3)}$$

$$R_B = \sqrt{\frac{\ell_e d}{b^2}} = \sqrt{\frac{(55.4)(13.25)}{(1.5)^2}} = 18.1$$

$$K_{bE} = 0.438$$

$$F_{bE} = \frac{K_{bE}E'}{R_b^2} = \frac{(0.438)(1{,}800{,}000)}{(18.1)^2} = 2418 \text{ psi}$$

$$C_L = \frac{1+(F_{bE}/F_b^*)}{1.9} - \sqrt{\left[\frac{1+(F_{bE}/F_b^*)}{1.9}\right]^2 - \frac{F_{bE}/F_b^*}{0.95}}$$

$$= \frac{1+2418/1967}{1.9} - \sqrt{\left[\frac{1+2418/1967}{1.9}\right]^2 - \frac{2418/1967}{0.95}}$$

$$= 0.886$$

**Allowable Bending Design Value, $F_b'$** (Table 2.3.1)

$$F_b' = F_bC_DC_LC_F = (1900)(1.15)(0.886)(0.9)$$

$$= 1742 \text{ psi}$$

*(cont.)*

the highest $E$ value grade and species of visually graded lumber was used, the 1977 $C_T$ equations were applicable to all grades and species of this material. Machine stress rated lumber has higher $E$ grades than the highest such grade for visually graded material and the coefficient of variation associated with machine stress rated $E$ values is only 11 percent. Therefore many of the stiffer grades of machine stress rated lumber had a higher 5 percent exclusion value of $E$ than that used in the derivation of the 1977 equations. As a result, the equations were applicable only to those machine stress rated grades having a design $E$ of 1,400,000 psi or less.

Based on additional analyses of the effects of various levels of chord $E$ on the stiffness contribution of sheathing to chord buckling (79), the $C_T$ equations were revised in the 1982 edition to apply to all grades of machine stress rated lumber and to any grade or species of visually graded lumber. As shown below, the equations are entered with a nominal 5 percent exclusion value, $E_{0.05}$, which is computed from tabulated $E$ values (Tables 4A, 4B and 4C) as

$$E_{0.05_{VG}} = [1 - 1.645(0.25)]E_{table} = 0.59\,E_{table}$$

and

$$E_{0.05_{MSR}} = [1 - 1.645(0.11)]E_{table} = 0.82\,E_{table}$$

for visually graded and machine stress rated material, respectively.

For lumber dry at time of plywood attachment:

$$C_T = 1 + \frac{2300\,\ell_e}{E_{0.05}} \qquad \text{(C4.4-12)}$$

For lumber unseasoned or partially seasoned at time of plywood attachment:

$$C_T = 1 + \frac{1200\,\ell_e}{E_{0.05}} \qquad \text{(C4.4-13)}$$

The 1982 $C_T$ equations have been carried forward unchanged to the 1991 edition. Application is limited to 2 inch thick lumber 4 inches or less in depth. The equations have not been verified beyond chord lengths of 96 inches and therefore, $C_T$ factors for longer chord lengths should be based on a length of 96 inches.

The $C_T$ adjustment for $E$ is intended for use in checking loads on the chords of trusses subject to combined bending and compression. The interaction equation of 3.9.2 is entered with an $F_c'$ allowable stress and Euler buckling stress in the primary ($d_1$) plane of bending computed with $E'$ equal to $C_T E$.

The equations of 4.4.3 apply only to plywood sheathing panels. Buckling stiffness factors for other types of structural panels, which have different nail load-slip relationships than plywood, have not been established.

**4.4.3.2** The design of triangular and parallel chord wood trusses made with metal connector plates are governed by specific national standards of practice for these components (186,187). Such practices employ provisions of this Specification that relate to the performance characteristics of individual pieces of lumber used as chord or web members.

**Example C12.1-1**

Yield mode lateral design values for wood-to-wood single shear nailed connections:

Single and mixed species joints of southern pine and spruce-pine-fir made with 8d common nails in side member thicknesses of 3/8, 1/2 and 5/8 inches and with 60d common nails in side member thicknesses of 1/2, 1-1/2, and 2-1/2 inches

$F_{em}$, $F_{es}$ = 5550 psi southern pine (SP)
= 3350 psi spruce-pine-fir (SPF)
$F_{yb}$ = 100,000 psi 8*d*
= 70,000 psi 60*d*
$D$ = 0.131 8*d*
= 0.263 60*d*
$K_D$ = 2.2 8*d*
= 3.0 60*d*
$p$ = nail length - side member thickness

| Nail Penny-Weight | Side Member Thickness in. | Species Main | Species Side | Yield Mode Design Values, lbs $Z_{I_s}$ | $Z_{III_m}$ | $Z_{III_s}$ | $Z_{IV}$ |
|---|---|---|---|---|---|---|---|
| 8d | 3/8 | SP | SP | 124 | 242 | 78 | 106 |
| | | SPF | SP | 124 | 160 | 69 | 92 |
| | | SP | SPF | 75 | 219 | 65 | 92 |
| | | SPF | SPF | 75 | 149 | 59 | 82 |
| | 1/2 | SP | SP | 165 | 229 | 85 | 106 |
| | | SPF | SP | 165 | 152 | 76 | 92 |
| | | SP | SPF | 100 | 207 | 68 | 92 |
| | | SPF | SPF | 100 | 141 | 61 | 82 |
| | 5/8 | SP | SP | 207 | 216 | 94 | 106 |
| | | SPF | SP | 207 | 144 | 84 | 92 |
| | | SP | SPF | 125 | 195 | 72 | 92 |
| | | SPF | SPF | 125 | 134 | 65 | 82 |
| 60d | 1/2 | SP | SP | 243 | 905 | 188 | 262 |
| | | SPF | SP | 243 | 593 | 165 | 228 |
| | | SP | SPF | 147 | 821 | 161 | 228 |
| | | SPF | SPF | 147 | 551 | 144 | 204 |
| | 1-1/2 | SP | SP | 730 | 745 | 288 | 262 |
| | | SPF | SP | 730 | 491 | 260 | 228 |
| | | SP | SPF | 441 | 676 | 207 | 228 |
| | | SPF | SPF | 441 | 456 | 191 | 204 |
| | 2-1/2 | SP | SP | 1216 | 588 | 433 | 262 |
| | | SPF | SP | 1216 | 391 | 392 | 228 |
| | | SP | SPF | 734 | 533 | 294 | 228 |
| | | SPF | SPF | 734 | 363 | 272 | 204 |

this will result in significant underestimation of joint capacity.

**Comparison of 1991 and Earlier Edition Lateral Design Values.** Differences in lateral design values for single shear nailed connections in the 1991 and 1986 editions resulting from the change to the yield limit model and the use of individual species rather than fastener group values is illustrated in Table C12.3-1.

**Table C12.3-1 - Comparison of 1991 and 1986 NDS Wood-to-Wood Single Shear Nail Lateral Design Values**

| Side Member Thickness, in. | Nail Penny-Weight | Nail Diam. in. | Nail Lateral Design Value, lbs — Southern Pine 1991 | 1986 | Ratio | Spruce-Pine-Fir 1991 | 1986 | Ratio |
|---|---|---|---|---|---|---|---|---|
| Box Nails: | | | | | | | | |
| 1/2 | 8*d* | 0.113 | 67 | 63 | 1.06 | 47 | 51 | 0.92 |
| | 20*d* | 0.148 | 101 | 94 | 1.07 | 73 | 77 | 0.95 |
| 3/4 | 8*d* | 0.113 | 79 | 63 | 1.25 | 57 | 51 | 1.12 |
| | 20*d* | 0.148 | 121 | 94 | 1.29 | 83 | 77 | 1.08 |
| Common Nails: | | | | | | | | |
| 1/2 | 8*d* | 0.131 | 85 | 78 | 1.09 | 61 | 64 | 0.95 |
| | 20*d* | 0.192 | 137 | 139 | 0.99 | 103 | 114 | 0.90 |
| 1-1/2 | 20*d* | 0.192 | 185 | 139 | 1.33 | 144 | 114 | 1.26 |
| | 60*d* | 0.263 | 262 | 223 | 1.17 | 191 | 182 | 1.05 |
| Threaded Hardened Nails: | | | | | | | | |
| 1/2 | 8*d* | 0.120 | 80 | 78 | 1.03 | 58 | 64 | 0.91 |
| | 20*d* | 0.177 | 147 | 139 | 1.06 | 111 | 114 | 0.97 |
| 1-1/2 | 16*d* | 0.148 | 145 | 108 | 1.34 | 113 | 88 | 1.28 |
| | 70*d* | 0.207 | 227 | 176 | 1.30 | 171 | 144 | 1.19 |
| Spikes: | | | | | | | | |
| 1/2 | 20*d* | 0.225 | 162 | 176 | 0.92 | 123 | 144 | 0.85 |
| | 60*d* | 0.283 | 201 | 248 | 0.81 | 155 | 203 | 0.76 |
| 1-1/2 | 40*d* | 0.263 | 262 | 223 | 1.17 | 191 | 182 | 1.05 |
| | 3/8 | 0.375 | 428 | 379 | 1.13 | 290 | 310 | 1.07 |

The consistently higher 1991/1986 lateral design value ratios for the 3/4 and 1-1/2 inch side member thicknesses relative to the ratios for the 1/2 inch side member thickness reflect the use of side member thickness as a variable in the yield mode equations whereas previous lateral design values were independent of member thickness. The generally lower lateral design value ratios for the fasteners with diameters over 0.25 inches relative to those with diameters less than 0.17 inches represents the effect of the larger $K_D$ factor (3.0) used in the yield mode equations for the former as compared to the factor (2.2) used with the latter.