2015/2018 NDS Example Problems

Member Designs and Connection Basics (DES 221)

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Questions related to specific materials, methods, and services will be addressed at the conclusion of this presentation.
LEARNING OBJECTIVES

Upon completion, participants will be better able to identify:

1. **Documentation**
   Discuss application of *NDS* design provisions for beams, columns, connections, and calculating design values.

2. **Resources**
   Apply the reference design values from the *NDS Supplement*.

3. **Adjustment Factors**
   Explain the design value adjustment factors from the *NDS* and *NDS Supplement*.

4. **Updated Provisions**
   Recognize the updated design provisions for connections new to the 2018 NDS.
1. What is your profession?
   a) Architect
   b) Engineer
   c) Code Official
   d) Fire Service
   e) Builder/Product Manufacturer/Other
OUTLINE

• Example Problems Document
• Member Design Examples
• Connection Design Examples
  • Shear
  • Lateral
• 2018 NDS
• 2018 NDS Supplement
• Examples document – **NEW!**
E1.2a - Simply Supported Beam Capacity Check (ASD)

A Select Structural Douglas Fir-Larch (DF-L) nominal 4X16 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. The beam is supported on a 2x4 top plate. Lateral support is provided only at the ends of the member and the ends are considered pinned.

Check beam's capacity to resist shear stress from maximum (moment controlled) load; determine deflection from maximum load and check bearing capacity.

Notes:
Load cases used in this example have been simplified for clarity. Refer to NDS Section 1.4.4 for requirements on load combinations.
**NDS SUPPLEMENT**

- Unadjusted design values

### Table 4A

**Reference Design Values for Visually Graded Dimension Lumber (2" - 4" thick)¹,²,³**

*(All species except Southern Pine — see Table 4B)*

Tabulated design values are for normal load duration and dry service conditions. See NDS 4.3 for a comprehensive description of design value adjustment factors.

#### USE WITH TABLE 4A ADJUSTMENT FACTORS

<table>
<thead>
<tr>
<th>Species and commercial grade</th>
<th>Size classification</th>
<th>Design values in pounds per square inch (psi)</th>
<th>Modulus of Elasticity</th>
<th>Specific Gravity</th>
<th>Grading Rules Agency</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Bending</td>
<td>Tension parallel to grain</td>
<td>Shear parallel to grain</td>
<td>Compression perpendicular to grain</td>
</tr>
<tr>
<td><strong>DOUGLAS FIR-LARCHI</strong></td>
<td></td>
<td>F_b</td>
<td>F_t</td>
<td>F_v</td>
<td>F_p</td>
</tr>
<tr>
<td>Select Structural</td>
<td></td>
<td>1.500</td>
<td>1.000</td>
<td>180</td>
<td>625</td>
</tr>
<tr>
<td>No. 1 &amp; Btr</td>
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<td>800</td>
<td>180</td>
<td>625</td>
</tr>
<tr>
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<td>675</td>
<td>180</td>
<td>625</td>
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<tr>
<td>No. 2</td>
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<td>575</td>
<td>180</td>
<td>625</td>
</tr>
<tr>
<td>No. 3</td>
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<td>0.525</td>
<td>325</td>
<td>180</td>
<td>625</td>
</tr>
<tr>
<td>Stud</td>
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<td>0.700</td>
<td>450</td>
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<td>1.000</td>
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<td>180</td>
<td>625</td>
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<td>0.575</td>
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<tr>
<td></td>
<td></td>
<td>0.275</td>
<td>175</td>
<td>180</td>
<td>625</td>
</tr>
</tbody>
</table>

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2015/2018 NDS Example Problems
E1.2a - Simply Supported Beam Capacity Check (ASD)

A Select Structural Douglas Fir-Larch (DF-L) nominal 4X16 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. The beam is supported on a 2x4 top plate. Lateral support is provided only at the ends of the member and the ends are considered pinned.

Check beam's capacity to resist shear stress from maximum (moment controlled) load; determine deflection from maximum load and check bearing capacity.

Notes:
Load cases used in this example have been simplified for clarity. Refer to NDS Section 1.4.4 for requirements on load combinations.

Reference and Adjusted Design Values for 4x16 Select Structural DF-L (size adjusted 4x12 values)

\[
\begin{align*}
F_b &= 1500 \text{ psi} \\
E &= 1900000 \text{ psi} \\
E_{\text{min}} &= 690000 \text{ psi} \\
E' &= 1900000 \text{ psi} \\
F_{cL} &= 625 \text{ psi} \\
F_V &= 180 \text{ psi} \\
C_D &= 1.0 \\
C_M &= 1.0 \\
C_t &= 1.0 \\
C_i &= 1.0 \\
C_T &= 1.0 \\
C_F &= 1.0 \\
F'_{b*} &= F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_i \cdot C_T \\
E'_{\text{min}} &= E_{\text{min}} \cdot C_M \cdot C_t \cdot C_i \cdot C_T \\
E' &= 1900000 \text{ psi} \\
E'_{\text{min}} &= 690000 \text{ psi}
\end{align*}
\]

Member dimensions and properties

\[
\begin{align*}
l &= 20 \text{ ft} \\
b &= 3.5 \text{ in} \\
d &= 15.25 \text{ in} \\
w_{\text{bearing}} &= 3.5 \text{ in} \\
A_g &= b \cdot d \\
S &= \frac{b \cdot d^2}{6} \\
I &= \frac{b \cdot d^3}{12} \\
A_g &= 53.38 \text{ in}^2 \\
S &= 135.66 \text{ in}^3 \\
I &= 1034 \text{ in}^4
\end{align*}
\]

Beam Stability Factor

\[
F'_{b*} = F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_i \cdot C_T \\
F'_{b*} = 1500 \text{ psi}
\]

F'_{b*} is adjusted bending design value with all adjustment factors except the beam stability factor C_L and flat use factor C_t applied.

\[
\begin{align*}
l_u &= 12 \text{ in} \cdot 1 \text{ ft} \\
l_u &= 240 \text{ in} \\
l_u/d &= 15.7 \\
l_c &= 1.37 \cdot l_u + 3 \cdot d \\
l_c &= 375 \text{ in}
\end{align*}
\]

Laterally unsupported length

L_y/d > 7 (Table 3.3.3)
Determine Maximum Moment Allowed on Beam

Maximum total moment is the adjusted bending design value $F'_b$ times the section modulus $S$.

$$M_{\text{max}} := F'_b \cdot \frac{S}{12 \text{ in}} \quad M_{\text{max}} = 14849 \text{ ft \cdot lbf}$$

Determine Maximum Hoist Load $P$

Maximum hoist load $P$ is determined from subtracting moment due to beam weight from the maximum total moment allowed on the beam and solving for hoist load $P$. Load $P$ creates a moment on beam length $L$ of $PL/4$. Assume density of beam material is 37.5 lbs/ft³ (110% of tabulated of the specific gravity $G$ for Southern Pine).

$$\rho := 37.5 \frac{\text{lbf}}{\text{ft}^3} \quad w_{\text{beamweight}} := \rho \cdot \frac{b}{12 \text{ in}} \cdot \frac{d}{12 \text{ in}} \quad w_{\text{beamweight}} = 13.9 \cdot \text{plf}$$

Note:
- $w_{\text{beamweight}}$ is self weight of beam
- $M_{\text{beamweight}}$ is moment due to self weight
- $M_{\text{allow}}$ is maximum allowable moment due to applied hoist load

$$M_{\text{beamweight}} := \frac{w_{\text{beamweight}} (L)^2}{8} \quad M_{\text{beamweight}} = 695 \cdot \text{ft \cdot lbf}$$

$$M_{\text{allow}} := M_{\text{max}} - M_{\text{beamweight}} \quad M_{\text{allow}} = 14154 \cdot \text{ft \cdot lbf}$$

$$P := 4 \frac{M_{\text{allow}}}{L}$$

Result:
The total allowable concentrated moment-limited midspan load (hoist plus payload) is $P = 2831 \text{ lbf}$
Check Beam's Capacity to Resist Shear from Maximum (bending controlled) Load

\[ V := \frac{P}{2} \quad V = 1415 \text{ lbf} \]

\[ f_v := \frac{3 \cdot V}{2 \cdot b \cdot d} \quad f_v = 40 \text{ psi} \]

\[ F'_v := F_v \cdot C_D \cdot C_M \cdot C_t \cdot C_i \]

\[ F'_v = 180 \text{ psi} \quad f_v < F'_v \text{ okay} \]

Check Compression Perpendicular to Grain at Bearing Points

\[ f_{c,\perp} := \frac{V}{b \cdot w_{\text{bearing}}} \quad f_{c,\perp} = 116 \text{ psi} \]

\[ F'_{c,\perp} := F_{c,\perp} \cdot C_M \cdot C_t \cdot C_i \]

\[ F'_{c,\perp} = 625 \text{ psi} \quad f_{c,\perp} < F'_{c,\perp} \text{ okay} \]

Note: NDS Section 4.3.12 allows \( F_{c,\perp} \) to be increased by \( C_b \) as specified in Section 3.10.4. That increase was not used in this example.

Check Deflection

Total deflection is the combination of deflection from beam weight and deflection from the applied crane load. Deflection from beam weight is considered long term deflection. Deflection from crane load may be considered short-term.

\[ \Delta_{\text{beam_weight}} := \frac{5}{384 \cdot E \cdot I} \cdot \frac{w_{\text{beamweight}}}{12} \cdot \left( \frac{1}{12 \cdot \text{in}} \right)^4 \]

\[ \Delta_{\text{beam_weight}} = 0.025 \text{ in} \]

\[ \Delta_{\text{crane_load}} := \frac{P}{48 \cdot E \cdot I} \cdot \left( \frac{1}{12 \cdot \text{in}} \right)^3 \]

\[ \Delta_{\text{crane_load}} = 0.415 \text{ in} \]

\[ \Delta_{\text{total}} := \Delta_{\text{beam_weight}} + \Delta_{\text{crane_load}} \]

\[ \Delta_{\text{total}} = 0.44 \text{ in} \]

Calculate Span/Deflection Ratio

\[ \frac{12 \cdot \text{in}}{\text{ft}} \cdot \frac{\text{ft}}{\Delta_{\text{total}}} = 545 \quad \frac{L}{\Delta_{\text{total}}} \text{ ratio} \]
E1.5b - Compression Member Analysis (LRFD)

A No 2 Spruce Pine Fir (SPF) nominal 2X6 interior bearing stud, 91.5 inches long, sheathed on both sides with gypsum board, carries dead load and snow load from the roof (assume load combination 1.2D + 1.6S, λ=0.8). Determine $C_p$ and the allowable compression parallel to grain design value ($F_{c'}$) for the stud. Assume studs are placed 16” on center and top and bottom plates are the same grade and species. Determine axial loads based on buckling and bearing limit states.
E1.5b - Compression Member Analysis (LRFD)

A No 2 Spruce Pine Fir (SPF) nominal 2X6 interior bearing stud, 91.5 inches long, sheathed on both sides with gypsum board, carries dead load and snow load from the roof (assume load combination 1.2D + 1.6S, λ=0.8). Determine CP and the allowable compression parallel to grain design value (F_c') for the stud. Assume studs are placed 16" on center and top and bottom plates are the same grade and species. Determine axial loads based on buckling and bearing limit states.

Reference and Adjusted Design Values for No. 2 SPF 2x6

F_c := 1150 psi  \ E_{min} := 510000 psi  \ F_{c,\perp} := 425 psi  \ (NDS Table 4A)

\[ \lambda := 0.8 \quad C_M := 1.0 \quad C_t := 1.0 \quad C_F := 1.1 \quad C_i := 1.0 \quad C_T := 1.0 \quad \]  \ (NDS Table 4.3.1 and Appendix N)

K_{F_c} := 2.40 \quad \phi_c := 0.9 \quad K_{F,E_{min}} := 1.76 \quad \phi_{E_{min}} := 0.85

K_{c,\perp} := 1.67 \quad \phi_{c,\perp} := 0.9

\[ E'_{min} := \frac{E_{min} C_M C_t C_i C_T K_{F,E_{min}} \phi_{E_{min}}}{F_{c,\perp}} \quad E'_{min} = 762960 psi \]

\[ F'_{c,\perp} := F_{c,\perp} C_M C_t C_i K_{c,\perp} \phi_{c,\perp} \quad F'_{c,\perp} = 638.775 psi \]

Member length and properties

l := 91.5-in  \quad b := 1.5-in  \quad d := 5.5-in

Column Stability Factor

F_{c,*} := F_c C_M C_t C_i C_T K_{F_c} \phi_c \lambda  \quad F_{c,*} is adjusted bending design value with all adjustment factors except the column stability factor C_P.

F_{c,*} = 2186 psi

\[ l_{e2} := 0 \quad I_{e1} := 1 \]

\[ \frac{l_{e2}}{b} = 0 \quad \frac{l_{e1}}{d} = 16.636 \]

F_{c,E} := \frac{0.822 E'_{min}}{l_{e1}^2 d}  \quad F_{c,E} = 2266 psi

C := 0.8

Sawn lumber (NDS 3.7.1.5)

\[ 1 + \frac{F_{c,E}}{F_{c,*}} - \sqrt{1 + \left(\frac{F_{c,E}}{F_{c,*}}\right)^2 - \frac{F_{c,E}}{c}} \quad \text{Column Stability Factor (NDS 3.7-1)} \]

C_P = 0.703
\[ F'_c := (F_c \ast C_p) \]

\[ F'_c = 1537 \text{ psi} \]

\( F'_c \) is adjusted compression design value with all adjustment factors.

**Determine Axial Loads Based on Buckling and Bearing**

\[ P_{\text{Buckling}} := b \cdot d \cdot F'_c \]

\[ P_{\text{Bearing}} := b \cdot d \cdot F'_{c \perp} \]

\[ P_{\text{Bearing2}} := b \cdot d \cdot F'_c \ast \]

\[ P_{\text{Buckling}} = 12683 \text{ lbf} \]

\[ P_{\text{Bearing}} = 5270 \text{ lbf} \]

Perpendicular to grain bearing

\[ P_{\text{Bearing2}} = 18034 \text{ lbf} \]

Parallel to grain bearing

Note: Bearing area factor \((C_b)\) can be used to increase the bearing controlled load on interior studs. The bearing factor for the 1-1/2 bearing length measured parallel to grain is 1.25 (NDS Equation 3.10-2 and Table 3.10.4)

\[ C_b := 1.25 \]

\[ P_{\text{BearingIncreased}} := b \cdot d \cdot F'_{c \perp} \cdot C_b \]

\[ P_{\text{BearingIncreased}} = 6587 \text{ lbf} \]

Controlling Value

Note: With a 3:1 snow to dead load ratio, this translates to 3,294 lbs snow load and 1098 lbs dead load.
2. The aspect ratio (length divided by depth) of a compression member needs to be less than 50.

a) True
b) False
A No. 2 Hem-Fir nominal 2x8 is considered for use as the bottom chord of a 24-ft roof truss (12 ft between panel points). The chord will be subject to a uniform dead load of 8 psf as well as tension forces (assuming pinned connections) of 880 lb from roof wind loads (WL), 880 lb from roof live (RLL) and 1420 lb from dead loads (DL). Trusses are to be spaced 4 ft on center. Framing will have a 19% (max) moisture content. Check the adequacy of the bottom chord member for bending and tension for the appropriate load cases.

Note: Load cases used in this example have been simplified for clarity. Refer to NDS Section 1.4.4 for requirements on load combinations.
E1.6 - Combined Bending and Axial Tension Loading of a Truss Chord Member (ASD)

A No. 2 Hem-Fir nominal 2x8 is considered for use as the bottom chord of a 24-ft roof truss (12 ft between panel points). The chord will be subject to a uniform dead load of 8 psf as well as tension forces (assuming pinned connections) of 880 lb from roof wind loads (WL), 880 lb from roof live (RLL) and 1420 lb from dead loads (DL). Trusses are to be spaced 4 ft on center. Framing will have a 19% (max) moisture content. Check the adequacy of the bottom chord member for bending and tension for the appropriate load cases.

Note: Load cases used in this example have been simplified for clarity. Refer to NDS Section 1.4.4 for requirements on load combinations.

Reference and Adjusted Design Values for No. 2 Hem-Fir 2x8

\[ E = 1300000 \text{ psi} \]
\[ E_{\text{min}} = 470000 \text{ psi} \]
\[ F_b = 850 \text{ psi} \]
\[ F_t = 525 \text{ psi} \]
\[ C_M = 1.0 \]
\[ C_t = 1.0 \]
\[ C_i = 1.0 \]
\[ C_f = 1.2 \]
\[ C_R = 1.0 \]
\[ C_T = 1.0 \]
\[ (\text{NDS Supplement Table 4A}) \]

\[ E' = E \cdot C_M \cdot C_t \cdot C_i \]
\[ E_{\text{min}}' = E_{\text{min}} \cdot C_M \cdot C_t \cdot C_i \cdot C_T \]
\[ E' = 1300000 \text{ psi} \]
\[ E_{\text{min}}' = 470000 \text{ psi} \]

Member length and properties

\[ l = 12 \text{ ft} \]
\[ b = 1.5 \text{ in} \]
\[ d = 7.25 \text{ in} \]
\[ A_g = b \cdot d \]
\[ S = \frac{b \cdot d^2}{6} \]
\[ A_g = 10.875 \text{ in}^2 \]
\[ S = 13.141 \text{ in}^3 \]

Applied Loads

\[ w_D = \frac{8 \text{ lbf}}{\text{ft}^2} \]
\[ w_{\text{trib}} = 4 \text{ ft} \]
\[ T_{\text{wind}} = 880 \text{ lbf} \]
\[ T_{\text{Live}} = 880 \text{ lbf} \]
\[ T_{\text{Deaq}} = 1420 \text{ lbf} \]
Load Case 1: DL + RLL + WL

\[ C_D := 1.6 \quad \text{NDS Appendix B Section B.2 (non-mandatory)} \]

**Tension**

\[ \text{Ft'} := F_t \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \]

\[ \text{Ft'} = 1008 \text{ psi} \]

\[ T_1 := T_{\text{wind}} + T_{\text{live}} + T_{\text{dead}} \]

\[ T_1 = 3180 \text{ lbf} \]

\[ f_{t1} := \frac{T_1}{A_g} \]

\[ f_{t1} = 292 \text{ psi} \quad \text{Ft'} = 1008 \text{ psi} \]

**Bending**

\[ F'_{b*} := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot C_r \]

\[ F'_{b*} = 1632 \text{ psi} \]

**Determine Beam Stability Factor \( C_L \)**

\[ l_u := \frac{12}{\text{in}} \cdot \frac{1}{\text{ft}} \quad l_{u} = 144 \text{ in} \]

\[ \frac{l_u}{d} = 19.9 \]

\[ l_c := 1.63 \cdot l_u + 3 \cdot d \quad l_c = 256.5 \text{ in} \]

\[ R_B := \sqrt{\frac{l_c \cdot d}{b^2}} \quad R_B = 28.75 \]

\[ F_{bE} := \frac{1.20 \cdot E_{\text{min}}}{R_B^2} \quad F_{bE} = 682 \text{ psi} \]

\[ C_L := \frac{1 + \left( \frac{F_{bE}}{F'_{b*}} \right)}{1.9} - \sqrt{\left[ 1 + \left( \frac{F_{bE}}{F'_{b*}} \right) \right]^2 - \left( \frac{F_{bE}}{F'_{b*}} \right)^2} \]

\[ C_L = 0.404 \]

Adjusted tension parallel to grain design value for short duration loads (NDS 2.3.1 and 4.3.1)

Subscripts refer to Load Case

Tensile stress in bottom chord

Actual tension stress is less than adjusted tension parallel to design value. OK (NDS 3.8.1)

\( F'_{b*} \) is adjusted bending design value with all adjustment factors except the beam stability factor \( C_L \) and flat use factor \( C_u \) applied. The following calculations determine the beam stability factor \( C_L \):

(NDS 3.3.3)

Laterally unsupported length

\[ \frac{l_u}{d} > 7 \quad (\text{NDS Table 3.3.3}) \]

\( R_B < 50 \) OK (NDS 3.3.3.7)

(NDS 3.3.3.6)

Resulting beam stability factor \( C_L \).
\[ F'_b := F'_b \cdot C_L \cdot C_{fu} \]

\[ F'_b = 660 \text{ psi} \]

\[ F'_b \cdot ** := F'_b \]

\[ F'_b \cdot ** = 660 \text{ psi} \]

\[ M_{\text{max}} := \frac{(wD \cdot w_{\text{trib}})^2}{8} \cdot \text{in} \cdot \text{ft} \]

\[ M_{\text{max}} = 6912 \text{ in} \cdot \text{lbf} \]

\[ f_b := \frac{M_{\text{max}}}{S} \]

\[ f_b = 526 \text{ psi} \]

\[ f_b = 526 \text{ psi} \]

\[ F'_b = 660 \text{ psi} \]

\[ F'_b \] is the fully adjusted bending design value with all adjustment factors including the beam stability factor \( C_L \) and flat use factor applied.

Since \( C_v \) does not apply to solid sawn lumber, \( F'_b \cdot ** \) is equal to \( F'_b \).

Bending resulting from dead load:

\[ M_{\text{max}} \]

\[ f_b \]

\[ f_b = 526 \text{ psi} \]

\[ F'_b = 660 \text{ psi} \]

Ok. Actual bending stress \( f_b \) does not exceed adjusted bending design value \( F'_b \).

**Combined Bending and Axial Tension**

\[ \frac{f_{t1}}{F_t} + \frac{f_b}{F'_b \cdot *} = 0.61 \]

\[ f_b - f_{t1} \]

\[ \frac{f_b}{F'_b \cdot **} = 0.354 \]

\[ <1.0. \text{ Ok (NDS Equation 3.9-1)} \]

\[ <1.0 \text{ ok (NDS Equation 3.9-2)} \]
Load Case 2: DL+RLL

C_D := 1.25

Tension

F_t := F_t C_D C_M C_t C_F C_i
F_t = 787.5·psi

T_2 := T_{Live} + T_{Dead}
T_2 = 2300·lbf

\[ f_{t2} := \frac{T_2}{A_g} \]
\[ f_{t2} = 211\cdot\text{psi} \quad \text{F_t} = 787\cdot\text{psi} \]

F_{b*} := F_b C_D C_M C_t C_F C_i C_r
F_{b*} = 1275 psi

\[ C_L := \frac{1 + \left( \frac{F_b E}{F_{b*}} \right)}{1.9} - \sqrt{\frac{1 + \left( \frac{F_b E}{F_{b*}} \right)}{1.9} - \left( \frac{F_b E}{F_{b*}} \right) \cdot 0.95} \]

C_L = 0.509

F_b := F_{b*} C_L C_{fu}
F_b = 649·psi

F_{b**} := F_b
F_{b**} = 649 psi

\[ f_b = 526\cdot\text{psi} \quad F_{b} = 649\cdot\text{psi} \]

Combined Bending and Axial Tension

\[ \frac{f_{t2} + f_b}{F_{b*}} = 0.68 \quad <1.0 \text{ ok} \]

\[ \frac{f_b - f_{t2}}{F_{b**}} = 0.48 \quad <1.0 \text{ ok} \]
Load Case 3: DL only

Tension

\[ F_t' := F_t \cdot C_D \cdot C_f \cdot C_M \cdot C_t \cdot C_i \]

\[ F_t' = 567 \text{ psi} \]

\[ T_3 := T_{Dead} \]

\[ T_3 = 1420 \text{ lbf} \]

\[ f_{t3} := \frac{T_3}{A_g} \]

\[ f_{t3} = 131 \text{ psi} \quad F_t' = 567 \text{ psi} \]

Bending

\[ F_{b*}' := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_f \cdot C_i \cdot C_r \]

\[ F_{b*}' = 918 \text{ psi} \]

\[ C_L := \frac{1 + \left( \frac{F_{bE}}{F_{b*}'} \right)}{1.9} - \sqrt{\frac{1 + \left( \frac{F_{bE}}{F_{b*}'} \right)^2}{1.9} - \frac{F_{bE}}{F_{b*}'}} \]

\[ C_L = 0.674 \]

\[ F_b' := F_{b*}' \cdot C_L \cdot C_{fu} \]

\[ F_b' = 619 \text{ psi} \]

\[ F_{b**}' := F_b' \]

\[ F_{b**}' = 619 \text{ psi} \]

\[ f_b = 526 \text{ psi} \quad F_b' = 619 \text{ psi} \]

Combined Bending and Axial Tension

\[ \frac{f_{t3} + f_b}{F_t'} < 1.0 \quad \text{ok} \]

\[ \frac{f_b - f_{t3}}{F_{b**}'} < 1.0 \quad \text{ok} \]

Results: No 2 Hem-Fir 2 x 8 satisfies NDS Criteria for combined bending and axial tension.
E2.1b - Fastener Uplift Capacity - Roof Sheathing
Ring Shank Nail in 5/8" WSP (2018 NDS Only)

Using 2018 NDS section 12.2, calculate the Allowable Stress Design (ASD) reference withdrawal
design value in pounds (capacity) and head pull-through design value in pounds (capacity) of a
0.131" diameter, 3" long roof sheathing ring shank (RSRS-05) nail in the narrow face of a Douglas
Fir-Larch nominal 2x6 with a 5/8 in. thick Douglas-Fir Wood Structural Panel (plywood or
oriented strand board) side member. Assume all adjustment factors are unity.

Main member:
Douglas Fir-Larch (DFL) 2x6 (G = 0.5)

Side member:
5/8 in. thick Wood Structural Panel (WSP) (G = 0.5)

Fastener Dimensions:
Dash No. 05 (NDS Table L6)
Length = 3 in.
Diameter = 0.131 in.
Head diameter = 0.281 in.
TL = 1.5 in.

![Diagram of Fastener Uplift Capacity](image)
E2.1b - Fastener Uplift Capacity - Roof Sheathing
Ring Shank Nail in 5/8” WSP (2018 NDS Only)

Using 2018 NDS section 12.2, calculate the Allowable Stress Design (ASD) reference withdrawal design value in pounds (capacity) and head pull-through design value in pounds (capacity) of a 0.131” diameter, 3” long roof sheathing ring shank (RSRS-05) nail in the narrow face of a Douglas Fir-Larch nominal 2x6 with a 5/8 in. thick Douglas-Fir Wood Structural Panel (plywood or oriented strand board) side member. Assume all adjustment factors are unity.

Main member:
Douglas Fir-Larch (DFL) 2x6 (G = 0.5)

Side member:
5/8 in. thick Wood Structural Panel (WSP) (G = 0.5)

Fastener Dimensions:
Dash No. 05 (NDS Table L6)
Length = 3 in.
Diameter = 0.131 in.
Head diameter = 0.281 in.
TL = 1.5 in.

\[ D := 0.131 \quad \text{Fastener diameter (in.)} \]
\[ D_H := 0.281 \quad \text{Fastener head diameter (in.)} \]
\[ TL := 1.5 \quad \text{Deformed Shank Length (in.)} \]
\[ t_{ns} := 0.625 \quad \text{Net Side Member thickness (in.)} \]
\[ G := 0.5 \quad \text{Specific gravity, main and side members (NDS Table 12.3.3A and 12.3.3B)} \]

Checking Fastener Withdrawal

\[ W := 1800 \cdot G^2 \cdot D \quad \text{NDS Equation 12.2-5} \]
\[ W = 59 \quad \text{Reference withdrawal design value. Compare to NDS Table 12.2E, } W = 59 \text{ lbs/in} \]
\[ W \cdot TL = 88 \quad \text{Reference withdrawal design value based on deformed shank fastener penetration (TL) in main member (lbs)} \]

Checking Fastener Head Pull-Through

\[ t_{ns} = 0.625 \]
\[ 2.5D_H = 0.703 \quad 2.5D_H \text{ greater than } t_{ns}, \text{ so NDS Equation 12.2-6a applies} \]
\[ W_H := 690 \cdot \pi \cdot D_H \cdot G^2 \cdot t_{ns} \quad \text{NDS Equation 12.2-6a} \]
\[ W_H = 95 \quad \text{Head pull-through design value (lbs). Compare to NDS Table 12.2F, } W_H = 95 \text{ lbs} \]

Fastener head pull-through design value of 95 lbs is greater than withdrawal design value of 88 lbs; withdrawal controls design capacity. See NDS Table 11.3.1 for application of additional adjustment factors for connections based on end use conditions.
E2.3 - Withdrawal Design Value - Lag Screw

Using 2015/2018 NDS provisions (NDS 12.2) calculate the Allowable Stress Design (ASD) withdrawal design value of a lag screw in the connection below. Assume all adjustment factors are unity.

Main member:
Southern Pine Nominal 6x (Actual thickness = 5.5 in.) (G = 0.55) (NDS Table 12.3.3A)

Side member:
Southern Pine Nominal 2x (Actual thickness = 1.5 in.) (G = 0.55) (NDS Table 12.3.3A)

Fastener Dimensions:
1/2 in. diameter lag screw (NDS Table L2)
Length = 4 in.
Tip Length = 0.3125 in.
E2.3 - Withdrawal Design Value - Lag Screw

Using 2015/2018 NDS provisions (NDS 12.2) calculate the Allowable Stress Design (ASD) withdrawal design value of a lag screw in the connection below. Assume all adjustment factors are unity.

Main member:
Southern Pine Nominal 6x (Actual thickness = 5.5 in.) (G = 0.55) (NDS Table 12.3.3A)

Side member:
Southern Pine Nominal 2x (Actual thickness = 1.5 in.) (G = 0.55) (NDS Table 12.3.3A)

Fastener Dimensions:
1/2 in. diameter lag screw (NDS Table L2)
Length = 4 in.
Tip Length = 0.3125 in.

\[
D := 0.5 \quad \text{Fastener diameter (in.)}
\]
\[
tip := 0.3125 \quad \text{Fastener tapered tip length (in.)}
\]
\[
G := 0.55 \quad \text{Specific gravity (NDS Table 12.3.3A)}
\]
\[
L := 4 \quad \text{Lag screw length (in.)}
\]
\[
L_s := 1.5 \quad \text{Side Member thickness (in.)}
\]
\[
p_t := L - L_s - tip \quad \text{Lag screw penetration into main member (in.)}
\]
\[
p_t = 2.188
\]

Note: Per Table L2, the unthreaded body length, \(S = 1.5\text{"}\). Therefore, the threaded portion begins at the wood-to-wood interface. Note \(p_t\) also matches Table L2 dimension T-E = 2-3/16". Longer lag screws have different thread length dimensions requiring evaluation to determine actual thread penetration.

\[
W := 1800 \cdot G^2 \cdot D^4
\]

NDS Equation 12.2-1

\[
W = 436.6
\]

Compare to NDS Reference Withdrawal Design Value Table 12.2A, \(W = 437\text{ lbs/in.}\).

\[
W \cdot p_t = 955
\]

Withdrawal design value based on main member penetration (lbs)

See NDS Table 11.3.1 for application of additional adjustment factors for connections based on end use conditions.
3. Fastener head pull-through analysis is required per the 2018 NDS only.

a) True
b) False
E2.2 - Single Common Nail Lateral Design Value - Single Shear Wood-to-wood Connection

Using the 2015/2018 NDS yield limit equations in section 12.3, determine the Allowable Stress Design (ASD) reference lateral design value of a single shear connection with the following configuration. Assume all adjustment factors are unity.

Main member
Nominal 3x Douglas Fir-Larch (Actual thickness = 2.5 in.) (G = 0.5) (NDS Table 12.3.3A)

Side member
Nominal 1x Douglas Fir-Larch (Actual thickness = 0.75 in.) (G = 0.5) (NDS Table 12.3.3A)

Fastener Dimensions:
10d Common Nail (NDS Table L4)
D = 0.148 in.
Length = 3 in.
E2.2 - Single Common Nail Lateral Design Value - Single Shear Wood-to-wood Connection

Using the 2015/2018 NDS yield limit equations in section 12.3, determine the Allowable Stress Design (ASD) reference lateral design value of a single shear connection with the following configuration. Assume all adjustment factors are unity.

Main member
Nominal 3x Douglas Fir-Larch (Actual thickness = 2.5 in.) (G = 0.5) (NDS Table 12.3.3A)

Side member
Nominal 1x Douglas Fir-Larch (Actual thickness = 0.75 in.) (G = 0.5) (NDS Table 12.3.3A)

Fastener Dimensions:
10d Common Nail (NDS Table L4)
D = 0.148 in.
Length = 3 in.

Define parameters:

\[ F_{em} := 4650 \] Main member Dowel Bearing Strength (NDS Table 12.3.3) (psi)

\[ F_{es} := 4650 \] Side member Dowel Bearing Strength (NDS Table 12.3.3) (psi)

\[ R_e := \frac{F_{em}}{F_{es}} \] \( R_e = 1 \)

\[ F_{yb} := 90000 \] Fastener dowel bending yield strength (psi) (NDS Table I1)

\[ D := 0.148 \] Nail Diameter (in.)

\[ Tip := 2 \cdot D \] Length of tapered fastener tip (in.) (NDS 12.3.5.3b)

\[ L_s := 0.75 \] Side member Dowel Bearing Length (in.) (NDS 12.3.5)

\[ L_m := 3 - L_s - \frac{Tip}{2} \] Main member Dowel Bearing Length (in.) (NDS 12.3.5.3)

\[ L_m = 2.1 \]

2015 NDS 12.1.6.5 (2018 NDS 12.1.6.4) Requires minimum main member penetration equal to 6D, \( L_m > 0.89 \) in.

\[ R_d := 2.2 \] Reduction Term (NDS Table 12.3.1B)
Calculate $k_1$, $k_2$, and $k_3$ (NDS Table 12.3.1A)

$$R_t := \frac{L_m}{L_s} \quad R_t = 2.803$$

$$k_1 := \frac{\sqrt{R_e + 2 \cdot R_e \cdot (1 + R_t + R_t^2) + R_t^2 \cdot R_e^3} - R_e \cdot (1 + R_t)}{1 + R_e}$$

$k_1 = 0.935$

$$k_2 := -1 + \sqrt{2 \cdot (1 + R_e) + \frac{2 \cdot F_{yb} \cdot (1 + 2 \cdot R_e) \cdot D^2}{3 \cdot F_{em} \cdot L_m^2}}$$

$k_2 = 1.047$

$$k_3 := -1 + \sqrt{\frac{2(1 + R_e)}{R_e} + \frac{2 \cdot F_{yb} \cdot (2 + R_e) \cdot D^2}{3 \cdot F_{em} \cdot L_s^2}}$$

$k_3 = 1.347$

**Yield Mode Calculations (NDS Table 12.3.1A)**

**Mode \( I_m \)**

$$Z_{I_m} := \frac{D \cdot L_m \cdot F_{em}}{R_d}$$

$Z_{I_m} = 658$ \hspace{1cm} Yield Mode \( I_m \) Solution (lbs)

**Mode \( I_s \)**

$$Z_{I_s} := \frac{D \cdot L_s \cdot F_{es}}{R_d}$$

$Z_{I_s} = 235$ \hspace{1cm} Yield Mode \( I_s \) Solution (lbs)
Mode II

\[ Z_{II} := \frac{k_1 \cdot D \cdot L_s \cdot F_{cs}}{R_d} \]

\[ Z_{II} = 219 \quad \text{Yield Mode II Solution (lbs)} \]

Mode III,

\[ Z_{III,m} := \frac{k_2 \cdot D \cdot L_m \cdot F_{em}}{(1 + 2 \cdot R_e) R_d} \]

\[ Z_{III,m} = 230 \quad \text{Yield Mode III}_m \text{ Solution (lbs)} \]

Mode III,

\[ Z_{III,s} := \frac{k_3 \cdot D \cdot L_s \cdot F_{em}}{(2 + R_e) R_d} \]

\[ Z_{III,s} = 105 \quad \text{Yield Mode III}_s \text{ Solution (lbs)} \]

Mode IV

\[ Z_{IV} := \left( \frac{D^2}{R_d} \right) \sqrt{\frac{2 \cdot F_{em} \cdot F_{yb}}{3 \cdot (1 + R_e)}} \]

\[ Z_{IV} = 118 \quad \text{Yield Mode IV Solution (lbs)} \]

\[ Z_{dist} := \begin{pmatrix} Z_{Im} \\ Z_{Is} \\ Z_{II} \\ Z_{III,m} \\ Z_{III,s} \\ Z_{IV} \end{pmatrix} = \begin{pmatrix} 658 \\ 235 \\ 219 \\ 230 \\ 105 \\ 118 \end{pmatrix} \]

Creating an array with all Yield Mode Solutions
\[ Z := \min(Z_{\text{dist}}) \]

Minimum value of all Yield Modes provides Z-reference lateral design value (lbs). Mode III controls. Compare to NDS Table 12N value = 105 lbs. See NDS Table 11.3.1 for application of additional adjustment factors for connections based on end use conditions.
4. The new equations for fastener head pull-through are based on which of the following.

a) Fastener head diameter  
b) Species and weight of side member  
c) Net side member thickness  
d) All of the above.  
e) a. and c.