Updates and Errata
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About American Wood Council
On behalf of the industry it represents, AWC is committed to ensuring a resilient, safe, and sustainable built environment. To achieve these objectives, AWC contributes to the development of sound public policies, codes, and regulations which allow for the appropriate and responsible manufacture and use of wood products. We support the utilization of wood products by developing and disseminating consensus standards, comprehensive technical guidelines, and tools for wood design and construction, as well as providing education regarding their application.

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The International Code Council is a member-focused association. It is dedicated to developing model codes and standards used in the design, build and compliance process to construct safe, sustainable, affordable and resilient structures. Most US communities and many global markets choose the International Codes. ICC Evaluation Service (ICC-ES) is the industry leader in performing technical evaluations for code compliance fostering safe and sustainable design and construction.
2015/2018 Structural Wood Design Examples

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American Wood Council
222 Catoctin Circle, SE, Suite 201
Leesburg, VA 20175
info@awc.org
FOREWORD

This document is intended to aid instruction in structural design of wood structures using both Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD). It contains design examples and complete solutions calculated using ASD and LRFD. Solutions have been developed based on the 2015 and 2018 National Design Specification® (NDS®) for Wood Construction, and the 2015 Special Design Provisions for Wind and Seismic (SDPWS), as appropriate. References are also made to the 2015 and 2018 Wood Frame Construction Manual (WFCM) for One- and Two- Family Dwellings. Copies of these standards produced by the American Wood Council can be obtained at www.awc.org/codes-standards/publications. Procedures will be applicable for both 2015 and 2018 versions of the NDS and WFCM, unless otherwise noted. Example problems range from simple to complex and cover many design scenarios. In the solutions where a particular provision of the NDS or SDPWS is cited, reference is made to the document and corresponding provision number, e.g. NDS 4.3.1. It is intended that this document be used in conjunction with competent engineering design, accurate fabrication, and adequate supervision of construction. Neither the American Wood Council, the International Code Council, nor their members assume any responsibility for errors or omissions in this document, nor for engineering designs, plans, or construction prepared from it. Those using this document assume all liability arising from its use. The design of engineered structures is within the scope of expertise of licensed engineers, architects, or other licensed professionals for applications to a particular structure.

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Solutions have been developed using the MathCAD® 15 software by PTC (https://www.ptc.com/). Some formatting is the result of the program layout, for example the use of “:=” denotes an assigned value, while an “=” denotes a calculated value. Examples may contain notes or comments for instances where program constraints have led to the use of non-standardized terms or subscripts.
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E1.1 - Adjustment Factors (ASD and LRFD)

A No. 1 Douglas Fir-Larch (DF-L) nominal 2x6 is used for a floor joist @ 16” o.c. (supporting only dead and live loads) for an exterior deck. The in-service moisture content is greater than 19%, and the member is not subject to elevated temperatures. The ends are held in place with full-depth blocking. Determine the adjusted bending design value (F’b), adjusted shear design value (F’v), adjusted tension design value (F’t) and moduli of elasticity (E' and E_min') for the member using both Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD). Assume lumber is incised.
A No. 1 Douglas Fir-Larch (DF-L) nominal 2x6 is used for a floor joist @ 16" o.c. (supporting only dead and live loads) for an exterior deck. The in-service moisture content is greater than 19%, and the member is not subject to elevated temperatures. The ends are held in place with full-depth blocking. Determine the adjusted bending design value \( F'_{b} \), adjusted shear design value \( F'_{v} \), adjusted tension design value \( F'_{t} \) and modulii of elasticity \( E' \) and \( E_{min}' \) for the member using both Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD). Assume lumber is incised.

Reference and Adjusted Design Values for No. 1 DF-L 2x6 (NDS Supplement Table 4A)

\[
\begin{align*}
F_b &= 1000 \text{ psi} \\
E &= 1700000 \text{ psi} \\
E_{min} &= 660000 \text{ psi} \\
F_t &= 800 \text{ psi} \\
F_v &= 180 \text{ psi}
\end{align*}
\]

\( \text{(NDS Supplement Table 4A)} \)

**Determine Adjusted Bending Design Value \( F'_{b} \) using ASD factors**

Load Duration Factor

\[
C_D = 1.0 \quad (\text{NDS Table 2.3.2})
\]

Wet Service Factor

\[
C_{Mb} = 0.85 \quad (\text{NDS Supplement Table 4A Adjustment factors})
\]

Temperature Factor

\[
C_T = 1.0 \quad (\text{NDS Table 2.3.3})
\]

Beam Stability Factor

\[
C_L = 1.0 \quad (\text{NDS 3.3.3.2 and 4.4.1.2})
\]

Size Factor

\[
C_F = 1.3 \quad (\text{NDS Supplement Table 4A})
\]

Flat Use Factor

\[
C_{fu} = 1.0 \quad (\text{NDS Supplement Table 4A})
\]

Incising Factor

\[
C_i = 0.8 \quad (\text{NDS Table 4.3.8})
\]

Repetitive Member Factor

\[
C_r = 1.15 \quad (\text{NDS 4.3.9})
\]

\[
F'_{b,ASD} = F_b \cdot C_D \cdot C_{Mb} \cdot C_T \cdot C_L \cdot C_F \cdot C_{fu} \cdot C_i \cdot C_r
\]

\[
F'_{b,ASD} = 1017 \text{ psi} \quad \text{Adjusted ASD Bending Design Value \( F'_{b} \)}
\]
Determine Adjusted Bending Design Value \( (F'_{b}) \) using LRFD factors

Format Conversion Factor

\[ K_F := 2.54 \quad \text{(NDS Table 4.3.1)} \]

Resistance Factor

\[ \phi := 0.85 \quad \text{(NDS Table 4.3.1)} \]

Time Effect Factor

\[ \lambda := 0.8 \quad \text{(NDS Appendix N.3.3 - use combination 1.2D+1.6L+0.5(L_r or S or R), L from occupancy)} \]

\[ F'_{bLRFD} := F_{b} \cdot C_{M} \cdot C_i \cdot C_{L} \cdot C_{F} \cdot C_{fu} \cdot C_i \cdot C_r \cdot K_F \cdot \phi \cdot \lambda \]

\[ F'_{bLRFD} = 1756 \text{ psi} \quad \text{Adjusted LRFD Bending Design Value (\( F'_{b} \))} \]

Determine Adjusted Shear Parallel to Grain Design Value \( (F'_{v}) \) using ASD factors

Load Duration Factor

\[ C_D := 1.0 \quad \text{(NDS Table 2.3.2)} \]

Wet Service Factor

\[ C_{MV} := 0.97 \quad \text{(NDS Supplement Table 4A)} \]

Temperature Factor

\[ C_t := 1.0 \quad \text{(NDS Table 2.3.3)} \]

Incising Factor

\[ C_i := 0.8 \quad \text{(NDS Table 4.3.8)} \]

\[ F'_{vASD} := F_{v} \cdot C_D \cdot C_{MV} \cdot C_t \cdot C_i \quad \text{Adjusted ASD Shear Parallel to Grain Design Value (\( F'_{v} \))} \]

\[ F'_{vASD} = 140 \text{ psi} \]

Determine Adjusted Shear Parallel to Grain Design Value \( (F'_{v}) \) using LRFD factors

Format Conversion Factor

\[ K_{Fv} := 2.88 \quad \text{(NDS Table 4.3.1)} \]

Resistance Factor

\[ \phi_v := 0.75 \quad \text{(NDS Table 4.3.1)} \]

Time Effect Factor

\[ \lambda := 0.8 \quad \text{(NDS Appendix N.3.3 - use combination 1.2D+1.6L+0.5(L_r or S or R), L from occupancy)} \]

\[ F'_{vLRFD} := F_{v} \cdot C_{MV} \cdot C_i \cdot C_r \cdot K_{Fv} \cdot \phi_v \cdot \lambda \quad \text{Adjusted LRFD Shear Parallel to Grain Design Value (\( F'_{v} \))} \]
\[ F'_{v LRFD} = 241 \text{ psi} \]

**Determine Adjusted Tension Parallel to Grain Design Value (F') using ASD factors**

**Load Duration Factor**
\[ C_D := 1.0 \quad \text{(NDS Table 2.3.2)} \]

**Wet Service Factor**
\[ C_{Mt} := 1.0 \quad \text{(NDS Supplement Table 4A)} \]

**Temperature Factor**
\[ C_t := 1.0 \quad \text{(NDS Table 2.3.3)} \]

**Size Factor**
\[ C_F := 1.3 \quad \text{(NDS Supplement Table 4A)} \]

**Incising Factor**
\[ C_i := 0.8 \quad \text{(NDS Table 4.3.8)} \]

\[ F'_{t ASD} := F_t \cdot C_D \cdot C_{Mt} \cdot C_t \cdot C_F \cdot C_i \quad \text{Adjusted ASD Tension Parallel to Grain Design Value (F')} \]

\[ F'_{t ASD} = 832 \text{ psi} \]

**Determine Adjusted Tension Parallel to Grain Design Value (F') using LRFD factors**

**Format Conversion Factor**
\[ K_{Ft} := 2.70 \quad \text{(NDS Table 4.3.1)} \]

**Resistance Factor**
\[ \phi_t := 0.8 \quad \text{(NDS Table 4.3.1)} \]

**Time Effect Factor**
\[ \lambda := 0.8 \quad \text{(NDS Appendix N.3.3 - use combination 1.2D+1.6L+0.5(L_r or S or R), L from occupancy)} \]

\[ F'_{t LRFD} := F_t \cdot C_{Mt} \cdot C_t \cdot C_F \cdot C_i \cdot K_{Ft} \cdot \phi_t \cdot \lambda \quad \text{Adjusted LRFD Tension Parallel to Grain Design Value (F')} \]

\[ F'_{t LRFD} = 1438 \text{ psi} \]

**Determine Adjusted Modulus of Elasticity (E') (same for ASD and LRFD)**

**Wet Service Factor**
\[ C_{ME} := 0.9 \quad \text{(NDS Supplement Table 4A)} \]

**Temperature Factor**

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\[
C_{tE} := 1.0 \quad \text{(NDS Table 2.3.3)}
\]

**Incising Factor**

\[
C_{iE} := 0.95 \quad \text{(NDS Table 4.3.8)}
\]

\[
E' := E \cdot C_{ME} \cdot C_{tE} \cdot C_{iE}
\]

\[
E' = 1453500 \text{ psi} \quad \text{Adjusted ASD/LRFD Modulus of Elasticity (E')} 
\]

**Determine Beam and Column Stability Adjusted Modulus of Elasticity (E_{min}') with ASD Factors**

**Wet Service Factor**

\[
C_{ME} := 0.9 \quad \text{(NDS Supplement Table 4A)}
\]

**Temperature Factor**

\[
C_{tE} := 1.0 \quad \text{(NDS Table 2.3.3)}
\]

**Incising Factor**

\[
C_{iE} := 0.95 \quad \text{(NDS Table 4.3.8)}
\]

**Buckling Stiffness Factor**

\[
C_{T} := 1.0 \quad \text{(NDS 4.4.2)}
\]

\[
E'_{minASD} := E \cdot C_{ME} \cdot C_{tE} \cdot C_{iE} \cdot C_{T}
\]

\[
E'_{minASD} = 1453500 \text{ psi} \quad \text{Adjusted ASD Beam and Column Stability Modulus of Elasticity (E_{min}')} 
\]

**Determine Beam and Column Stability Adjusted Modulus of Elasticity (E_{min}') with LRFD Factors**

**Wet Service Factor**

\[
C_{ME} := 0.9 \quad \text{(NDS Supplement Table 4A)}
\]

**Temperature Factor**

\[
C_{tE} := 1.0 \quad \text{(NDS Table 2.3.3)}
\]

**Incising Factor**

\[
C_{iE} := 0.95 \quad \text{(NDS Table 4.3.8)}
\]

**Buckling Stiffness Factor**

\[
C_{T} := 1.0 \quad \text{(NDS 4.4.2)}
\]
Format Conversion Factor

\[ K_{FE} := 1.76 \text{ (NDS Table 4.3.1)} \]

Resistance Factor

\[ \phi_{E} := 0.85 \text{ (NDS Table 4.3.1)} \]

\[ E'_{\text{min LRFD}} := E \cdot C_{\text{ME}} \cdot C_{\text{T}} \cdot C_{\text{iE}} \cdot C_{\text{T}} \cdot K_{FE} \cdot \phi_{E} \]

\[ E'_{\text{min LRFD}} = 2174436 \text{ psi} \]

Adjusted LRFD Beam and Column Stability Modulus of Elasticity (\( E'_{\text{min}} \))
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.
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)

<table>
<thead>
<tr>
<th>$F_b$</th>
<th>$E$</th>
<th>$E_{min}$</th>
<th>(Table 4A)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1500-psi</td>
<td>1900000-psi</td>
<td>690000-psi</td>
<td></td>
</tr>
<tr>
<td>$F_{c\perp}$</td>
<td>625-psi</td>
<td>$F_v$</td>
<td>180-psi</td>
</tr>
<tr>
<td>$C_D$</td>
<td>1.0</td>
<td>$C_M$</td>
<td>1.0</td>
</tr>
<tr>
<td>$C_f$</td>
<td>1.0</td>
<td>$C_t$</td>
<td>1.0</td>
</tr>
<tr>
<td>$C_T$</td>
<td>1.0</td>
<td>$C_F$</td>
<td>1.0</td>
</tr>
</tbody>
</table>

$E' = E \cdot C_M \cdot C_t \cdot C_i \cdot C_T$  
$E'_{min} = E_{min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T$  

$E' = 1900000$ psi  
$E'_{min} = 690000$ psi

Member dimensions and properties

<table>
<thead>
<tr>
<th>$l$</th>
<th>20-ft</th>
<th>$b$</th>
<th>3.5-in</th>
<th>$d$</th>
<th>15.25-in</th>
<th>$w_{bearing}$</th>
<th>3.5-in (width of bearing)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_g$</td>
<td>$b \cdot d$</td>
<td>$S = \frac{b \cdot d^2}{6}$</td>
<td>$I = \frac{b \cdot d^3}{12}$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| $A_g$ | 53.38-in$^2$ | $S = 135.66$-in$^3$ | $I = 1034$-in$^4$

Beam Stability Factor

$F_{b*} = F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot C_T$  

$F_{b*} = 1500$ psi  

$L_u := 12 \cdot \text{in} \cdot \text{ft}^{-1}$  

$L_u = 240 \cdot \text{in}$  

Laterally unsupported length

$L_u = 15.7$  

$L \cdot d > 7$ (Table 3.3.3)

$L_e := 1.37 \cdot L_u + 3 \cdot d$  

$L_e = 375$-in  

(Table 3.3.3)
\[ R_B := \sqrt{\frac{I_e d}{b^2}} \quad R_B = 21.6 \quad \text{Slenderness ratio for bending (3.3-5)} \]

\[ F_{bE} := \frac{1.20 \cdot E\text{'min}}{R_B} \quad F_{bE} = 1776 \text{ psi} \quad \text{Critical buckling design value for bending (3.7.1)} \]

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

\[ C_L = 0.876 \]

\[ F'_{b} := (F_{b*} \cdot C_{fu} \cdot C_L) \]

\[ F'_{b} = 1313 \text{ psi} \quad \text{Adjusted bending design value with all adjustment factors} \]

**Determine Maximum Moment Allowed on Beam**

Maximum total moment is the adjusted bending design value \( F'_{b} \) times the section modulus \( S \)

\[ M_{\text{max}} := F'_{b} \cdot \frac{S}{12 \text{ in}} \quad M_{\text{max}} = 14849 \text{ ft}\cdot\text{lbf} \]

**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 \text{ lbs/ft}^3 (110\% of tabulated of the specific gravity \( G \) for Southern Pine).

\[ \rho := 37.5 \text{ lbs/ft}^3 \quad w_{\text{beamweight}} := \rho \cdot \frac{b}{12 \text{ in}} \cdot \frac{d}{12 \text{ in}} \quad w_{\text{beamweight}} = 13.9 \text{ plf} \quad \text{Note:} \]

\( w_{\text{beamweight}} \) is self weight of beam

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

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

\[ P := 4 \frac{M_{\text{allow}}}{l} \]

**Result:**

The total allowable concentrated moment-limited midspan load (hoist plus payload) is \( P = 2831 \text{ lbf} \)
1.2A SIMPLY SUPPORTED BEAM CAPACITY CHECK (ASD)

Check Beam’s Capacity to Resist Shear from Maximum (bending controlled) Load

\[ V := \frac{P}{2} \quad V = 1415 \text{lbf} \]
\[ f_v := \frac{3 \cdot V}{2 \cdot b \cdot d} \quad f_v = 40 \text{ psi} \]
\[ F'_v := F_v \cdot C_D \cdot C_M \cdot C_t \cdot C_i \quad F'_v = 180 \text{ psi} \quad f_v < F'_v \text{ okay} \]

Check Compression Perpendicular to Grain at Bearing Points

\[ f_{c,\perp} := \frac{V}{b \cdot w_{bearing}} \quad f_{c,\perp} = 116 \text{ psi} \]
\[ F'_{c,\perp} := F_{c,\perp} \cdot C_M \cdot C_t \cdot C_i \quad F'_{c,\perp} = 625 \text{ psi} \quad f_{c,\perp} < F'_{c,\perp} \text{ okay} \]

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.

\[ \Delta_{\text{beam_weight}} := \frac{5}{384 \cdot E \cdot I} \cdot \frac{w_{\text{beamweight}}}{12} \cdot \frac{\text{ft}}{\text{in}} \cdot \left( \frac{1 \cdot 12 \cdot \text{in}}{\text{ft}} \right)^4 \quad \Delta_{\text{beam_weight}} = 0.025 \text{-in} \]
\[ \Delta_{\text{crane_load}} := \frac{P}{48 \cdot E \cdot I} \cdot \left( \frac{1 \cdot 12 \cdot \text{in}}{\text{ft}} \right)^3 \quad \Delta_{\text{crane_load}} = 0.415 \text{-in} \]
\[ \Delta_{\text{total}} := \Delta_{\text{beam_weight}} + \Delta_{\text{crane_load}} \quad \Delta_{\text{total}} = 0.44 \text{-in} \]

Calculate Span/Deflection Ratio

\[ \frac{12 \cdot 1 \cdot \text{in}}{\text{ft}} = 545 \quad \frac{\text{L}}{\Delta_{\text{total}}} \text{ ratio} \]
E1.2b - Simply Supported Beam Capacity Check (LRFD)

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 load combination 1.2D+1.6L applies (λ=0.8). 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.
1.2B Simply Supported Beam Capacity Check (LRFD)

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 load combination 1.2D+1.6L applies ($\lambda=0.8$). 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)

\[
\begin{align*}
F_b &:= 1500 \text{ psi} \\
F_{c_{\perp}} &:= 625 \text{ psi} \\
\lambda &:= 0.8 \\
C_M &:= 1.0 \\
C_T &:= 1.0 \\
C_{fu} &:= 1.0 \\
C_{\perp} &:= 1.0 \\
\phi_b &:= 0.85 \\
\phi_{c_{\perp}} &:= 0.90 \\
\phi_{E\text{min}} &:= 0.85 \\
F' &:= E\cdot C_M \cdot C_T \cdot C_{\perp} \\
F'_{\text{min}} &:= E_{\text{min}}\cdot C_M \cdot C_T \cdot C_{\perp} \cdot K_{\text{FE}\text{min}} \cdot \phi_{\text{E\text{min}}} \\
F_b^\ast &:= F_b \cdot C_M \cdot C_T \cdot C_{\perp} \cdot C_{fu} \cdot K_{Fb} \cdot \phi_b \cdot \lambda \\
F_b^\ast &:= 2591 \text{ psi}
\end{align*}
\]
1.2B

SIMPLY SUPPORTED BEAM CAPACITY CHECK (LRFD)

Laterally unsupported length

\[ l_u = \frac{12 \text{ in}}{1 \text{ ft}} \]

\[ l_u = 240 \text{ in} \]

\[ \frac{l_u}{d} = 15.7 \]

\[ l_c := 1.37 \cdot l_u + 3 \cdot d \]

\[ l_c = 375 \text{ in} \]

Slenderness ratio for bending (3.3-5)

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

\[ R_B = 21.6 \]

Critical buckling design value for bending (3.7.1)

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

\[ F_{bE} = 2657 \text{ psi} \]

\[ F_{b} = 2143 \text{ psi} \]

\[ F_{b} \] is adjusted bending design value with all adjustment factors.

Determine Maximum Moment Allowed on Beam

Maximum total moment is the adjusted bending design value \( F'_b \) times the section modulus \( S \)

\[ M_{\text{max}} := \frac{F'_b \cdot S}{12 \text{ in} \cdot \text{ft}} \]

\[ M_{\text{max}} = 24230 \text{ ft} \cdot \text{lbf} \]
**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/ft$^3$ (110% of tabulated of the specific gravity $G$ for Southern Pine).

$$
p := 37.5 \frac{\text{lb}}{\text{ft}^3}
$$

$$
w_{\text{beamweight}} := \rho \frac{b}{12 \text{ in}} \frac{d}{12 \text{ in}}
$$

$$
w_{\text{beamweight}} = 13.9 \text{ plf}
$$

$$
M_{\text{beamweight}} := \frac{1.2w_{\text{beamweight}}(l)^2}{8}
$$

$$
M_{\text{beamweight}} = 834 \text{ ft} \cdot \text{lbf}
$$

1.2 factor for load combination 1.2D+1.6L applied here

$$
M_{\text{allow}} := M_{\text{max}} - M_{\text{beamweight}}
$$

$$
M_{\text{allow}} = 23396 \text{ ft} \cdot \text{lbf}
$$

$$
P := 4 \frac{M_{\text{allow}}}{l \cdot 1.6}
$$

1.6 factor for load combination 1.2D+1.6L applied here

**Result:**
The total allowable concentrated moment-limited midspan load (hoist plus payload) is $P = 2925 \text{ lbf}$

**Check Beam’s Capacity to Resist Shear from Maximum (bending controlled) Load**

$$
V := \frac{1.6P}{2}
$$

1.6 factor applied here for live load

$$
V = 2340 \text{ lbf}
$$

$$
f_v := \frac{3 \cdot V}{2 \cdot b \cdot d}
$$

$$
f_v = 66 \text{ psi}
$$

$$
F_v := F_{v} C_M C_t C_i K_{Fv} \phi_v \lambda
$$

$$
F_{v} = 311 \text{ psi}
$$

$$
f_v < F_{v} \text{ OK}
$$

**Check Compression Perpendicular to Grain at Bearing Points**

$$
f_{c,\perp} := \frac{V}{b \cdot w_{\text{bearing}}}
$$

$$
f_{c,\perp} = 191 \text{ psi}
$$

$$
F'_{c,\perp} := F_{c,\perp} C_M C_t C_i K_{Fc,\perp} \phi_{c,\perp}
$$

$$
F'_{c,\perp} = 939 \text{ psi}
$$

$$
f_{c,\perp} < F'_{c,\perp} \text{ OK}
$$

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.

\[
\Delta_{\text{beam_weight}} := \frac{5}{384 \cdot E \cdot I} \cdot \frac{w_{\text{beam weight}}}{12} \cdot \left(\frac{12 \cdot \text{in}}{\text{ft}}\right)^4 \Rightarrow \Delta_{\text{beam_weight}} = 0.025 \cdot \text{in}
\]

\[
\Delta_{\text{crane_load}} := \frac{P}{48 \cdot E \cdot I} \cdot \left(\frac{12 \cdot \text{in}}{\text{ft}}\right)^3 \Rightarrow \Delta_{\text{crane_load}} = 0.429 \cdot \text{in}
\]

\[
\Delta_{\text{total}} := \Delta_{\text{beam_weight}} + \Delta_{\text{crane_load}} \Rightarrow \Delta_{\text{total}} = 0.454 \cdot \text{in}
\]

**Calculate Span/Deflection Ratio**

\[
\frac{12 \cdot \text{in}}{\text{ft}} \cdot \frac{\text{in}}{\Delta_{\text{total}}} = 529 \Rightarrow \frac{L}{\Delta_{\text{total}}} \text{ ratio}
\]
E1.3 - Glued Laminated Timber Beam Design (ASD)

Design a simple roof supporting beam spanning 32 ft, with 5000 lb loads (1000 lb dead load (DL) + 4000 lb snow load (SL)) applied by purlins at 8 ft on center (at 1/4 points plus the ends). Member has lateral supports at the ends and compression edge supports at the purlin locations. Beam supports are 6 inches long. Assume dry service conditions. Temperature is less than 100 degrees (F) but occasionally may reach 150 degrees (F). Use 24F-1.8E structural glued laminated (glulam) Southern Pine timber.

Notes:
Load cases used in this example have been simplified for clarity. Refer to NDS Section 1.4.4 for requirements on load combinations.
E1.3 - Glued Laminated Timber Beam Design (ASD)

Design a simple roof supporting beam spanning 32 ft, with 5000 lb loads (1000 lb dead load (DL) + 4000 lb snow load (SL)) applied by purlins at 8 ft on center (at 1/4 points plus the ends). Member has lateral supports at the ends and compression edge supports at the purlin locations. Beam supports are 6 inches long. Assume dry service conditions. Temperature is less than 100 degrees (F) but occasionally may reach 150 degrees (F). Use 24F-1.8E structural glued laminated (glulam) Southern Pine timber.

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 24F-1.8E structural glued laminated softwood timber

\[
\begin{align*}
F_{bx+} &:= 2400 \text{ psi} \\
F_{bx-} &:= 1450 \text{ psi} \\
F_{cx} &:= 650 \text{ psi} \\
F_{cv} &:= 265 \text{ psi} \\
F_t &:= 1100 \text{ psi} \\
F_c &:= 1600 \text{ psi} \\
E_x &:= 1800000 \text{ psi} \\
E_{x\text{min}} &:= 950000 \text{ psi} \\
C_D &:= 1.15 \\
C_M &:= 1.0 \\
C_t &:= 1.0 \\
C_{fu} &:= 1.0 \\
C_c &:= 1.0 \\
C_b &:= 1.0 \\
E'_{x} &:= E_x \cdot C_M \cdot C_t \\
E'_{x\text{min}} &:= E_{x\text{min}} \cdot C_M \cdot C_t \\

\end{align*}
\]

Temp up to 150 degrees F only occasionally (NDS Table 5.3.1)

Note: \(E_x\) notation has changed to \(E_{x\text{app}}\) for 2018 NDS

E\(x\) = 1800000 psi
E\(x\)\(_{\text{min}}\) = 950000 psi

Member length and properties

\[
\begin{align*}
l &:= 32 \text{ ft} \\
b &:= 5 \text{ in} \\
d &:= 30.25 \text{ in} \\
I_{\text{support}} &:= 6 \text{ in} \\

\end{align*}
\]

Initial iteration

Note: Beam length designated as lower case \(l\) instead of upper case \(L\) used in the Specification nomenclature

\[
\begin{align*}
A_g &:= b \cdot d \\
S_{xx} &:= \frac{b \cdot d^2}{6} \\
I_{xx} &:= \frac{b \cdot d^3}{12} \\

\end{align*}
\]

\[
\begin{align*}
A_g &= 151.3 \cdot \text{in}^2 \\
S_{xx} &= 762.6 \cdot \text{in}^3 \\
I_{xx} &= 11534 \cdot \text{in}^4 \\

\end{align*}
\]

Beam Stability Factor

\[
\begin{align*}
F_{bx+*} &:= F_{bx+} \cdot C_D \cdot C_M \cdot C_t \cdot C_c \cdot C_I \\
F_{bx+*} &= 2760 \text{ psi} \\

F_{b*} & is adjusted bending design value with all adjustment factors except the beam stability factor \(C_I\) flat use factor \(C_{fu}\) and volume factor \(C_v\) applied. \\

l_u &= 12 \cdot \frac{\text{in}}{\text{ft}} \\
l_u &= 96 \cdot \text{in} \\
l_e &:= 1.54 \cdot l_u \\
l_e &= 148 \cdot \text{in} \\
R_B &= \frac{l_e \cdot d}{b^2} \\
R_B &= 13.375 \\

\text{Slenderness ratio for bending (3.3-5)}

Laterally unsupported length

(Table 3.3.3)
1.3 GLUED LAMINATED TIMBER BEAM DESIGN (ASD)

\[ F_{bE} := \frac{1.20 \cdot E'_{\text{min}}}{R_B^2} \]

Critical bucking design value for bending (3.3.3.8)

\[ F_{bE} = 6373 \text{ psi} \]

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

Note: \( C_L \) must be \( \leq 1.0 \) per NDS 5.3-1

\[ C_L = 0.965 \]

**Volume Factor**

\[ x := 20 \]

\[ C_V := \text{min} \left[ \left[ \frac{1}{21 \cdot \text{ft}} \right]^x \left[ \frac{1}{12 \cdot \text{in}} \right]^x \left[ \frac{1}{5.125 \cdot \text{in}} \right]^x \right] \]

\[ C_V = 0.936 \]

**Adjusted Bending Design Value**

\[ F'_b := \left[ F_{bx+*} \cdot \text{min} \left( \frac{C_L}{C_V} \right) \cdot C_{fu} \right] \]

\[ F'_b = 2584 \text{ psi} \]

\( F'_b \) is adjusted bending design value with all adjustment factors.

**Assume Beam Weight and Determine Section Modulus Required to Resist Bending**

Wood density can be estimated based on NDS Supplement 3.1.3. Maximum total moment is the adjusted bending design value \( F'_b \) times the section modulus, \( S \)

\[ w_{\text{beamweight}} := 40 \frac{\text{lbf}}{\text{ft}} \]

Estimated self weight of beam

\[ P := 5000 \text{ lbf} \]

Load from purlin

\[ M_{\text{est}} := \left( P \cdot \frac{1}{2} + \frac{w_{\text{beamweight}} \cdot l^2}{8} \right) \cdot 12 \frac{\text{in}}{\text{ft}} \]

\[ M_{\text{est}} = 1021440 \text{ in} \cdot \text{lbf} \]

\[ S_{\text{reqd}} := \frac{M_{\text{est}}}{F'_b} \]

\[ S_{\text{reqd}} = 395 \text{ in}^3 \]

\( S_{\text{reqd}} \) is \( < 762.6 \text{ in}^3 \) so try a smaller section for efficiency
Try a 5 X 22 member

\[ b_2 := 5 \text{-in} \quad d_2 := 22 \text{-in} \]

Beam dimension for trial section. Subscript "2" used to denote second iteration

\[ A_{g2} := b_2 \cdot d_2 \quad S_{xx2} := \frac{b_2 \cdot d_2^2}{6} \]

\[ A_{g2} = 110 \text{-in}^2 \quad S_{xx2} = 403.3 \text{-in}^3 \]

\[ R_{B2} := \sqrt{\frac{1.20 \cdot E'_{\text{min}}}{b_2^2}} \quad R_{B2} = 11.406 \]

Slenderness ratio for bending (3.3-5)

\[ F_{bE2} := \frac{F_{bE2}}{R_{B2}^2} \quad F_{bE2} = 8763 \text{ psi} \]

Critical bucking design value for bending (3.3.3.8)

\[ C_{L2} := \frac{1 + \left( \frac{F_{bE2}}{F_{bx+*}} \right)}{1.9} \left( 1 + \left( \frac{F_{bE2}}{F_{bx+*}} \right)^2 \right)^{\frac{1}{2}} \left( \frac{F_{bE2}}{F_{bx+*}} \right) \]

Note: \( C_L \) must be \( \leq 1.0 \) per NDS 5.3-1

\[ C_{L2} = 0.978 \]

\[ C_{V2} := \min \left[ \left( \frac{21 \text{-ft}}{1} \right)^x \left( \frac{12 \text{-in}}{d_2} \right)^x \left( \frac{5.125 \text{-in}}{b_2} \right)^x \right] \]

\[ C_{V2} = 0.951 \]

\( C_L \) and \( C_V \) shall not apply simultaneously (5.3.5). \( C_V \) is less than \( C_L \). \( C_V \) controls

Adjusted Bending Design Value

\[ F'_{b2} := F_{bx+*} \cdot \min \left( \frac{C_{L2}}{C_{V2}} \right) \cdot C_{fu} \]

\[ F'_{b2} = 2625 \text{ psi} \]

\( F'_{b2} \) is adjusted bending design value with all adjustment factors for the trial section considered.

\[ w_{\text{beamweight2}} := 30 \text{lbf/ft} \]

\[ M_2 := \left( \frac{1}{2} + \frac{w_{\text{beamweight2}} \cdot 1^2}{8} \right) \cdot 12 \cdot \text{in/ft} \]
\[ M_2 = 1006080 \text{ in} \cdot \text{lb} \]

\[ S_{\text{reqd}} = \frac{M_2}{F'_{\text{b}2}} \]

\[ S_{\text{reqd}} = 383 \text{ in}^3 \]

\[ 383 \text{ in}^3 < 403 \text{ in}^3 \quad \text{Okay} \]

**Shear Parallel to Grain**

The two 5000 pound purlin loads at the ends of the beam are within "d" of the supports and can be ignored for shear (3.4.3.1(a)). Shear determined from remaining purlin loads

\[ V_{\text{purlins}} = \frac{3P}{2} \]

\[ V_{\text{purlins}} = 7500 \text{ lb} \]

\[ V_{\text{beamweight2}} = \frac{w_{\text{beamweight2}}}{l^2} \left[ \frac{1}{2} - \left( \frac{d_2}{2} + \frac{1}{2} \frac{l_{\text{support}}}{2} \right) \right] \]

\[ V_{\text{beamweight2}} = 417.5 \text{ lb} \]

\[ V_{\text{total}} = V_{\text{purlins}} + V_{\text{beamweight2}} \]

\[ V_{\text{total}} = 7918 \text{ lb} \]

**Adjusted (F'_{\text{v}}) and Actual (f_{\text{v}}) Shear Parallel to Grain**

\[ C_{vr} = 1.0 \quad \text{(NDS 5.3.10)} \]

\[ F'_{\text{v}} = F_{\text{vx}} \cdot C_D \cdot C_M \cdot C_t \cdot C_{vr} \]

\[ F'_{\text{v}} = 305 \text{ psi} \]

\[ f_{\text{v}} = \frac{3 \cdot V_{\text{total}}}{2 \cdot b_2 \cdot d_2} \]

\[ f_{\text{v}} = 108 \text{ psi} \]

\[ f_{\text{v}} < F'_{\text{v}} \quad \text{Actual shear stress parallel to grain less than adjusted OK} \]

**Compression Perpendicular to Grain**

**At Bearing Ends**

The bearing ends of the beam transmit all the purlin loads so the two 5000 pound purlin loads at the ends of the beam are included in the bearing load calculations

\[ R_{\text{purlins}} = \frac{1}{2} \cdot 5 \cdot P \]

\[ R_{\text{purlins}} = 12500 \text{ lb} \]

\[ R_{\text{beamweight2}} = \frac{1}{2} \cdot w_{\text{beamweight2}} \left[ l + 2 \left( \frac{l_{\text{support}}}{2} \right) \right] \]

\[ R_{\text{beamweight2}} = 487.5 \text{ lb} \]

\[ R_{\text{total}} = R_{\text{purlins}} + R_{\text{beamweight2}} \]
GLUED LAMINATED TIMBER BEAM DESIGN (ASD)

M2 1006080 in\(^2\)lbf

\[ S_{\text{req}} = \frac{M}{F'b^2} \]

\[ S_{\text{req}} = 383 \text{ in}^3 \]

Okay

Shear Parallel to Grain

The two 5000 pound purlin loads at the ends of the beam are within “d” of the supports and can be ignored for shear (3.4.3.1(a)). Shear determined from remaining purlin loads

\[ V_{\text{purlins}} = \frac{3P}{2} \]

\[ V_{\text{purlins}} = 7500\text{lbf} \]

\[ V_{\text{beamweight}} = \frac{w_{\text{beamweight}}}{l_{\text{support}}} \left( \frac{1}{2} l_{\text{support}} - \frac{1}{2} d \right) \]

\[ V_{\text{beamweight}} = 417.5\text{lbf} \]

\[ V_{\text{total}} = V_{\text{purlins}} + V_{\text{beamweight}} \]

\[ V_{\text{total}} = 7918\text{lbf} \]

Adjusted (\(F'_{v}\)) and Actual (\(f_{v}\)) Shear Parallel to Grain

\[ F'_{v} = (NDS\ 5.3.10) \]

\[ F'_{v} = F_{v} C_{D} C_{M} C_{t} C_{v} \]

\[ F'_{v} = 305\text{psi} \]

\[ f_{v} = \frac{V_{\text{total}}}{b^2 d^2 \left( \frac{1}{2} l_{\text{support}} \right)} \]

\[ f_{v} = 108\text{psi} \]

Actual compression stress perpendicular to grain is less than the adjusted compression perpendicular to grain design value. OK

Compression Perpendicular to Grain

At Bearing Ends

The bearing ends of the beam transmit all the purlin loads so the two 5000 pound purlin loads at the ends of the beam are included in the bearing load calculations

\[ R_{\text{purlins}} = \frac{5\text{P}}{2} \]

\[ R_{\text{purlins}} = 12500\text{lbf} \]

\[ R_{\text{beamweight}} = \frac{w_{\text{beamweight}}}{l_{\text{support}}} \left( \frac{1}{2} l_{\text{support}} - \frac{1}{2} d \right) \]

\[ R_{\text{beamweight}} = 487.5\text{lbf} \]

\[ R_{\text{total}} = R_{\text{purlins}} + R_{\text{beamweight}} \]

\[ R_{\text{total}} = 12988\text{lbf} \]

\[ F'_{c,\perp} = F_{c,\perp} C_{M} C_{t} \]

\[ F'_{c,\perp} = 650\text{psi} \]

\[ f_{c,\perp} = \frac{R_{\text{total}}}{b^2 l_{\text{support}}} \]

\[ f_{c,\perp} = 433\text{psi} \]

At Purlins

Purlins are supported by saddle style hangers that transfer compressive loads to the top of the beam. Determine the area of the hangers required to support each purlin without creating actual compression stresses greater than the adjusted compression perpendicular to grain design value

\[ A_{\text{hanger}} = \frac{P}{F'_{c,\perp}} \]

\[ A_{\text{hanger}} = 7.69\text{in}^2 \]

Assuming that purlins frame in from both sides of the beam, the width of the hanger can be calculated as follows:

\[ w_{\text{hanger}} = \frac{A_{\text{hanger}}}{0.50 b^2} \]

\[ w_{\text{hanger}} = 3.08\text{in} \]
A 3-1/8 wide purlin hanger is adequate. Note: The compression perpendicular to grain design value $F'_{c\perp}$ can be increased by the bearing area factor $C_b$ (5.3.12). For 3 inch bearing the factor is:

$$l_b := 3 \cdot \text{in} \quad C_b := \frac{l_b + 0.375 \cdot \text{in}}{l_b} \quad C_b = 1.125$$

$$\frac{1}{C_b \cdot w_{\text{hanger}}} = 2.735 \cdot \text{in}$$

Using the bearing factor $C_b$ confirms that a 3 inch wide hanger across the beam would be adequate.

At this stage of the calculations, the span of the beam can be reviewed. The 32 foot span was based on the center to center distance between supports. The length of the span used in design is the face to face distance plus 1/2 of the required bearing length at the ends (3.2.1).

In the example, the face to face distance is 32 ft minus 6 inches or 31.5 feet. At the end of the beam the required bearing distance is 12,998 lbs/(5 inches * 650 psi) or 4 inches. At the interior face, half the purlin load is assumed to be transferred to the beam end. Required length in bearing is (2500 lbs + 7500 lbs + 488 lbs)/5 inches * 650 psi) or 3.25 inches. The two required bearing lengths and the face to face distance produces a span of 31.5 ft + 1/2 (4.00/12) + 1/2 (3.25/12) or 31.8 feet which 99.4% of the center to center span. The shorter span reduces moments and bending stresses by 1.25%. The reduction is considered insufficient to allow the use of the next smaller beam.

**Deflection**

The specification does not include specific deflection limits for roofs. In some applications, deflections may be critical and the designer may wish to limit deflections. For this example, a deflection limit of L/240 has been selected.

Dead load deflection is usually calculated to determine the desired camber of the beam. The recommended camber is usually 150% of the dead load deflection. Deflection for the 5000 lb concentrated loads and the beam weight is:

$$\Delta_{\text{purlin}} = \frac{19.9 \cdot P}{384 \cdot E'_{x} \cdot l_{xx}^2} \left( \frac{12 \cdot l_{\text{in}}}{\text{ft}} \right)^3$$

$$\Delta_{\text{beamweight}} = \frac{w_{\text{beamweight}}^2}{384 \cdot E'_{x} \cdot l_{xx}^2} \left( \frac{l \cdot 12 \cdot \text{in}}{\text{ft}} \right)^4$$

$$\Delta_{\text{purlin}} = 1.754 \cdot \text{in} \quad \Delta_{\text{beamweight}} = 0.089 \cdot \text{in} \quad \Delta_{\text{camber}} = 0.375 \cdot \text{in}$$

$$\Delta_{\text{total}} = \Delta_{\text{purlin}} + \Delta_{\text{beamweight}} - \Delta_{\text{camber}}$$

$$\Delta_{\text{total}} = 1.468 \cdot \text{in}$$

**Length/Deflection Ratio**

$$\frac{l \cdot 12 \cdot \text{in}}{\text{ft} \cdot \Delta_{\text{total}}} = 262 \quad L/\Delta > 240. \text{ The length deflection ratio satisfies specified criteria}$$
E1.4 - Compression Members and Column Stability Calculation (ASD)

Compare the axial compression capacity of a nominal 4x4 and nominal 6x6 post being used for an interior column (only carrying gravity loads - dead load (DL) + floor live load (LL)). Both members are No. 2 Southern Pine and have a length of 10 feet. Both ends are assumed to be pinned (\(K_e=1.0\) - NDS 3.7.1.2). Assume all members are loaded concentrically.
E1.4 - Compression Members and Column Stability Calculation (ASD)

Compare the axial compression capacity of a nominal 4x4 and nominal 6x6 post being used for an interior column (only carrying gravity loads - dead load (DL) + floor live load (LL)). Both members are No. 2 Southern Pine and have a length of 10 feet. Both ends are assumed to be pinned ($K_e=1.0$ - NDS 3.7.1.2). Assume all members are loaded concentrically.

Reference and Adjusted Design Values - 4x4 Post

\[ F_c := 1450 \text{psi} \]
\[ E := 1400000 \text{psi} \]
\[ E_{\text{min}} := 510000 \text{psi} \]
\[ C_F := 1.0 \]
\[ C_M := 1.0 \]
\[ C_t := 1.0 \]
\[ C_i := 1.0 \]
\[ C_T := 1.0 \]
\[ C_D := 1.0 \]

\[ E' := E \cdot C_M \cdot C_t \cdot C_i \]
\[ E' = 1.4 \times 10^6 \text{psi} \]
\[ E'_{\text{min}} := E_{\text{min}} \cdot C_M \cdot C_t \cdot C_i \cdot C_T \]
\[ E'_{\text{min}} = 5.1 \times 10^5 \text{psi} \]

\[ d := 3.5\text{in} \]
\[ \text{Actual member dimensions} = 3.5'' \times 3.5'' \]
\[ \text{Area} := 12.25\text{in}^2 \]
\[ K_e := 1.0 \]
\[ \text{Length} := 120\text{in} \]
\[ l_e := K_e \cdot \text{Length} \]
\[ l_e = 120 \cdot \text{in} \]
\[ \frac{l_e}{d} = 34.3 \]

Needs to be less than 50 (NDS 3.7.1.3)
**Column Stability Factor Calculation**

\[
F_{cE} := \frac{(0.822 \cdot E'\text{min})}{\left(\frac{l_e}{d}\right)^2} \quad F_{cE} = 357\text{ psi} \quad \text{(NDS 3.7.1)}
\]

\[
F_{c*} := F_{c} \cdot C_{D} \cdot C_{M} \cdot C_{t} \cdot C_{F} \cdot C_{i} \quad F_{c*} = 1450\text{ psi}
\]

\[
C_{\text{sawn}} := 0.8 \quad \text{(NDS 3.7.1)}
\]

\[
C_{p} := \left(1 + \frac{F_{cE}}{F_{c*}}\right) - \sqrt{\left(1 + \frac{F_{cE}}{F_{c*}}\right)^2 - \frac{\left(F_{cE}/F_{c*}\right)}{c_{\text{sawn}}}}
\]

\[C_{p} = 0.232\]

**Axial Buckling Capacity**

\[
F'_{c} := F_{c} \cdot C_{D} \cdot C_{M} \cdot C_{t} \cdot C_{F} \cdot C_{i} \cdot C_{p}
\]

\[F'_{c} = 336\text{ psi}\]

\[P := F'_{c} \cdot \text{Area}\]

\[P = 4120\text{-lbf}\]

**Bearing Capacity**

\[f_{c} := F'_{c} \quad f_{c} = 336\text{ psi} \quad \text{Actual compression stress parallel to grain assuming column loaded to 100\% calculated buckling capacity}\]

\[0.75 \cdot F_{c*} = 1088\text{ psi}\]

\[f_{c} \text{ is less than } F_{c*} \text{ (NDS 3.10.1) so bearing parallel to grain is OK.}\]

\[f_{c} \text{ is less than } 0.75 F_{c*}, \text{ so no rigid bearing insert is required per NDS 3.10.1.3}\]
Reference and Adjusted Design Values - 6x6 Post

\[ F_{c2} := 525 \text{ psi} \]
\[ E_2 := 1200000 \text{ psi} \]
\[ E_{\text{min}2} := 440000 \text{ psi} \]

\[ C_{F2} := 1.0 \quad \text{Size factor (NDS Supplement Table 4D)} \]
\[ C_{M2} := 1.0 \quad \text{Moisture factor (NDS Supplement Table 4D)} \]
\[ C_{t2} := 1.0 \quad \text{Temperature factor (NDS Table 2.3.3)} \]
\[ C_{i2} := 1.0 \quad \text{Incising factor (NDS Table 4.3.8)} \]
\[ C_{T2} := 1.0 \quad \text{Buckling Stiffness factor (NDS 4.4.2)} \]
\[ C_{D2} := 1.0 \quad \text{Load Duration factor (NDS Table 2.3.2)} \]

\[ E'_2 := E_2 \cdot C_{M2} \cdot C_{t2} \cdot C_{i2} \]
\[ E'_{\text{min}2} := E_{\text{min}2} \cdot C_{M2} \cdot C_{i2} \cdot C_{t2} \cdot C_{T2} \]

\[ d_2 := 5.5 \text{ in} \]
\[ \text{Actual member dimensions} = 5.5" \times 5.5" \]
\[ \text{Needs to be less than} \ 50 \ (\text{NDS 3.7.1.3}) \]

\[ \text{Area}_2 := 30.25 \text{ in}^2 \]
\[ K_{e2} := 1.0 \]
\[ \text{Length}_2 := 120 \text{ in} \]
\[ l_{e2} := K_{e2} \cdot \text{Length}_2 \]
\[ l_{e2} = 120 \cdot \text{in} \]
\[ \frac{l_{e2}}{d_2} = 21.8 \]
**Column Stability Factor Calculation**

\[
F_{CE2} := \frac{(0.822 \cdot E'_{min})}{(\frac{1}{l_c^2})}
\]

\[
F_{CE2} = 760 \text{ psi} \quad \text{(NDS 3.7.1)}
\]

\[
F_{c2*} := F_{c2} \cdot D2 \cdot C_{M2} \cdot C_{t2} \cdot C_{F2} \cdot C_{i2}
\]

\[
F_{c2*} = 525 \text{ psi}
\]

\[
c_{sawn2} := 0.8
\]

\[
C_p2 := \frac{1 + \frac{F_{CE2}}{F_{c2*}}}{2c_{sawn2}} - \frac{\left[1 + \frac{F_{CE2}}{F_{c2*}}\right]^2 - \frac{F_{CE2}}{F_{c2*}}}{c_{sawn2}}
\]

\[
C_p2 = 0.801
\]

**Axial Buckling Capacity**

\[
F'_{c2} := F_{c2} \cdot D2 \cdot C_{M2} \cdot C_{t2} \cdot C_{F2} \cdot C_{i2} \cdot C_{P2}
\]

\[
F'_{c2} = 421 \text{ psi}
\]

\[
P_2 := F'_{c2} \cdot \text{Area}_2
\]

\[
P_2 = 12725 \text{ lbf}
\]

**Bearing Capacity**

\[
f_{c2} := F'_{c2} \quad f_{c2} = 421 \text{ psi}
\]

Actual compression stress parallel to grain assuming column loaded to 100% calculated buckling capacity

\[
0.75 \cdot F_{c2*} = 394 \text{ psi}
\]

\[f_c\] is less than \(F_{c}^{*}\) (NDS 3.10.1) so bearing parallel to grain is OK.

\[f_c\] is greater than 0.75 \(F_{c}^{*}\), so rigid bearing insert such as 20 gage metal plate is required per NDS 3.10.1.3.

4x4 Post Capacity = 4120 lbs (no bearing plate required)

6x6 Post Capacity = 12725 lbs (bearing plate required)

Note: for eccentrically loaded columns, see NDS Chapter 15
E1.5a - Compression Member Analysis (ASD)

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. 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.
**E1.5a - Compression Member Analysis (ASD)**

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. 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.

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

\[
F_c := 1150 \text{ psi} \quad Emin := 510000 \text{ psi} \quad F_{c\perp} := 425 \text{ psi} \quad (\text{NDS Table 4A})
\]

\[
C_D := 1.15 \quad C_M := 1.0 \quad C_t := 1.0 \quad C_F := 1.1 \quad C_i := 1.0 \quad C_T := 1.0 \quad (\text{NDS Table 4.3.1})
\]

\[
E'_{\text{min}} := Emin \cdot C_M \cdot C_t \cdot C_F \cdot C_i \quad F'_{c\perp} := F_{c\perp} \cdot C_M \cdot C_t \cdot C_i \quad F'_{c\perp} = 425 \text{ psi}
\]

**Member length and properties**

\[
l := 91.5 \text{ in} \quad b := 1.5 \text{ in} \quad d := 5.5 \text{ in}
\]

**Column Stability Factor**

\[
F_{c*} := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i
\]

\[
F_{c*} = 1455 \text{ psi}
\]

\[
\frac{l_{c2}}{b} := 0 \quad \frac{l_{c1}}{d} = 16.636
\]

\[
F_{cE} := \frac{0.822 \cdot E'_{\text{min}}}{\left(\frac{l_{c1}}{d}\right)^2} \quad F_{cE} = 1515 \text{ psi}
\]

\[
c := 0.8
\]

\[
C_p := \frac{1 + \left(\frac{F_{cE}}{F_{c*}}\right)}{2 \cdot c} - \sqrt{1 + \left(\frac{F_{cE}}{F_{c*}}\right)^2} \quad F_{c*} = 1025 \text{ psi}
\]

\[
F_{c} := (F_{c*} \cdot C_p)
\]

\[
F_{c} = 1025 \text{ psi}
\]

$F_{c*}$ is adjusted bending design value with all adjustment factors except the column stability factor $C_p$.

Effective lengths of compression member in planes of lateral support. Strong axis buckling controls. See NDS A.11.3 regarding lateral support of the weak axis due to gypsum sheathing.

\[
I_b = 50 \text{ OK (NDS 3.7.1.4)}
\]

Critical bucking design value for compression members (3.7.1.5)

Sawn lumber (3.7.1.5)

Column Stability Factor (3.7-1)

$F_c$ is adjusted compression parallel to grain design value with all adjustment factors.
Determine Axial Loads Based on Buckling and Bearing

\[ P_{\text{Buckling}} := b \cdot d \cdot F'_c \]
\[ P_{\text{Bearing}} := b \cdot d \cdot F'_{c,\perp} \]
\[ P_{\text{Bearing2}} := b \cdot d \cdot F'_c* \]

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 (Equation 3.10-2 and Table 3.10.4)

\[ C_b := 1.25 \]
\[ P_{\text{BearingIncreased}} := b \cdot d \cdot F'_{c,\perp} \cdot C_b \]

Note: With a 3:1 snow to dead load ratio, this translates to 3,287 lbs snow load and 1,096 lbs dead load.
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 \( C_p \) 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.

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

\[
F_c := 1150 \text{-psi} \quad E_{\min} := 510000 \text{-psi} \quad F_{c_{\perp}} := 425 \text{-psi}
\]

(NDS Table 4A)

\[
\lambda := 0.8 \quad C_M := 1.0 \quad C_t := 1.0 \quad C_F := 1.1 \quad C_i := 1.0 \quad C_T := 1.0
\]

(NDS Table 4.3.1 and Appendix N)

\[
K_{F_c} := 2.40 \quad \phi_c := 0.9 \quad K_{F_{E\min}} := 1.76 \quad \phi_{E\min} := 0.85
\]

\[
K_{c_{\perp}} := 1.67 \quad \phi_{c_{\perp}} := 0.9
\]

\[
E'_{\min} := E_{\min} \cdot C_M \cdot C_t \cdot C_i \cdot C_T \cdot K_{F_{E\min}} \cdot \phi_{E\min} \quad E'_{\min} = 762960 \text{-psi}
\]

\[
F'_{c_{\perp}} := F_{c_{\perp}} \cdot C_M \cdot C_t \cdot C_i \cdot K_{c_{\perp}} \cdot \phi_{c_{\perp}} \quad F'_{c_{\perp}} = 638.775 \text{-psi}
\]

**Member length and properties**

\[
l := 91.5 \text{-in} \quad b := 1.5 \text{-in} \quad d := 5.5 \text{-in}
\]

**Column Stability Factor**

\[
F_{c^*} := F_c \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot K_{F_c} \cdot \phi_c \cdot \lambda
\]

\[
F_{c^*} = 2186 \text{-psi}
\]

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

Effective lengths of compression member in planes of lateral support. Strong axis buckling controls

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

<50 OK (NDS 3.7.1.4)

\[
F_{cE} := \frac{0.822 \cdot E'_{\min}}{\left(\frac{l_{e1}}{d}\right)^2} \quad F_{cE} = 2266 \text{-psi}
\]

Critical buckling design value for compression members (NDS 3.7.1.5)

\[
c := 0.8
\]

Sawn lumber (NDS 3.7.1.5)

\[
C_p := \frac{1 + \left(\frac{F_{cE}}{F_{c^*}}\right)}{2 \cdot c} - \left[1 + \left(\frac{F_{cE}}{F_{c^*}}\right)\right]^2 \cdot \frac{\left(\frac{F_{cE}}{F_{c^*}}\right)}{c}
\]

Column Stability Factor (NDS 3.7-1)

\[
C_p = 0.703
\]
\[ F'_{c} := (F_{c} \cdot C_{P}) \]
\[ F'_{c} = 1537 \text{ psi} \]

\( F'_{c} \) is adjusted compression design value with all adjustment factors.

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

\[ P_{\text{Buckling}} := b \cdot d \cdot F'_{c} \quad P_{\text{Buckling}} = 12683 \text{ lbf} \]
\[ P_{\text{Bearing}} := b \cdot d \cdot F'_{c\perp} \quad P_{\text{Bearing}} = 5270 \text{ lbf} \quad \text{Perpendicular to grain bearing} \]
\[ P_{\text{Bearing2}} := b \cdot d \cdot F'_{c\parallel} \quad P_{\text{Bearing2}} = 18034 \text{ lbf} \quad \text{Parallel to grain bearing} \]

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)

\[ C_{b} := 1.25 \]
\[ P_{\text{BearingIncreased}} := b \cdot d \cdot F'_{c\perp} \cdot C_{b} \quad P_{\text{BearingIncreased}} = 6587 \text{ lbf} \quad \text{Controlling Value} \]

Note: With a 3:1 snow to dead load ratio, this translates to 3,294 lbs snow load and 1098 lbs dead load.
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.
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

\[
\begin{align*}
F_b &:= 850 \text{ psi} & E &= 1300000 \text{ psi} & E_{\text{min}} &= 470000 \text{ psi} \\
F_t &:= 525 \text{ psi} & C_M &= 1.0 & C_t &= 1.0 & C_F &= 1.2 \\
C_{fu} &= 1.0 & C_i &= 1.0 & C_r &= 1.0 \\
C_T &= 1.0
\end{align*}
\]

\[E' = E \cdot C_M \cdot C_t \cdot C_i \quad \text{and} \quad E'_{\text{min}} = E_{\text{min}} \cdot C_M \cdot C_t \cdot C_i \cdot C_T\]

Member length and properties

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

Applied Loads

\[
\begin{align*}
w_D &= 8 \text{ lb/ft}^2 & w_{\text{trib}} &= 4 \text{ ft} & T_{\text{wind}} &= 880 \text{ lb} & T_{\text{live}} &= 880 \text{ lb} & T_{\text{dead}} &= 1420 \text{ lb}
\end{align*}
\]
Load Case 1: DL + RLL + WL

\[ C_D := 1.6 \]

**Tension**

\[ F'_t := F_T \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \]

\[ F'_t = 1008 \text{ psi} \]

\[ T_1 := T_{\text{wind}} + T_{\text{Live}} + T_{\text{Dead}} \]

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

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

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

**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)

**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 \( C_L \)**

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

\[ l_u \]

\[ d = 19.9 \]

\[ l_c := 1.63 \cdot l_u + 3 \cdot d \]

\[ l_c = 256.5 \text{ in} \]

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

\[ R_B = 28.75 \]

\[ R_B < 50 \text{ OK (NDS 3.3.3.7)} \]

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

(NDS 3.3.3.6)

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

(NDS Equation 3.3-6)

\[ C_L = 0.404 \]

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

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

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

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

\[ M_{max} := \frac{wDw_{trib}}{8} \left( \frac{1}{12} \cdot \frac{\text{in}}{\text{ft}} \right) \]

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

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

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

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

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

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

\[ \text{Since } C_v \text{ does not apply to solid sawn lumber, } F'_b** \text{ is equal to } F'_b \]

Bending resulting from dead load

\[ \frac{f_{t1}}{F't'} + \frac{f_b}{F'_b*} = 0.61 \]

\[ \frac{f_b - f_{t1}}{F'_b**} = 0.354 \]

\[ \text{Combined Bending and Axial Tension} \]

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

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

\[ C_D := 1.25 \]

Tension

\[ Ft' = F_t \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \]
\[ Ft' = 787.5 \text{ psi} \]

\[ T_2 := T_{Live} + T_{Dead} \]
\[ T_2 = 2300 \text{ lbf} \]

\[ f_{t2} := \frac{T_2}{A_g} \]

\[ f_{t2} = 211 \text{ psi} \quad \text{Ft'} = 787 \text{ psi} \]

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

\[ F'_{b*} = 1275 \text{ psi} \]

\[ C_L := \frac{1 + \frac{F_{bE}}{F'_{b*}}}{1.9} - \sqrt{\left( \frac{1 + \frac{F_{bE}}{F'_{b*}}}{1.9} \right)^2 - \frac{F_{bE}}{0.95}} \]

\[ C_L = 0.509 \]

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

\[ F_{b} = 649 \text{ psi} \]

\[ F'b** := F'_{b} \]

\[ F'b** = 649 \text{ psi} \]

\[ f_{b} = 526 \text{ psi} \quad \text{F}'_{b} = 649 \text{ psi} \]

Combined Bending and Axial Tension

\[ \frac{f_{t2}}{Ft'} + \frac{f_{b}}{F'_{b*}} = 0.68 \quad <1.0 \text{ ok} \]

\[ \frac{f_{b} - f_{t2}}{F'b**} = 0.48 \quad <1.0 \text{ ok} \]

Appendix B Section B.2 (non-mandatory). Roof Live Load is a construction load.

Adjusted tension parallel to grain design value for short duration loads (NDS 2.3.1 and 4.3.1)
**Load Case 3: DL only**

\[ C_D := 0.9 \]

**Tension**

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

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

\[ T_3 := T_{\text{Dead}} \]

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

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

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

**Bending**

\[ F_{b^*} := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot C_t \]

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

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

\[ C_L = 0.674 \]

\[ F_b := F_{b^*} \cdot C_L \cdot C_{f_u} \]

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

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

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

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

**Combined Bending and Axial Tension**

\[ \frac{f_{t3}}{F_t'} + \frac{f_b}{F_{b^*}} = 0.8 \quad <1.0 \quad \text{ok} \]

\[ \frac{f_b - f_{t3}}{F_{b^{**}}} = 0.64 \quad <1.0 \text{ ok} \]

**Results:** No 2 Hem-Fir 2 x 8 satisfies NDS Criteria for combined bending and axial tension.
E1.7 - Combined Bending and Axial Compression (ASD)

No. 1 Southern Pine nominal 2x6 beam-columns are being designed to carry an axial compressive load of 840 lb (snow) and 560 lb (dead) plus a 25 psf wind load on their narrow face. Columns are 9 ft long and spaced 4 feet o.c. Their ends are held in position and lateral support is provided along the entire narrow face.

Check the adequacy of the beam-column for bending and compression for the appropriate load cases.

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

The column being considered is not subjected to especially severe service conditions or extraordinary hazard.
**E1.7 - Combined Bending and Axial Compression (ASD)**

No. 1 Southern Pine nominal 2x6 beam-columns are being designed to carry an axial compressive load of 840 lb (snow) and 560 lb (dead) plus a 25 psf wind load on their narrow face. Columns are 9 ft long and spaced 4 feet o.c. Their ends are held in position and lateral support is provided along the entire narrow face.

Check the adequacy of the beam-column for bending and compression for the appropriate load cases.

Notes:
Load cases used in this example have been simplified for clarity. Refer to NDS Section 1.4.4 for requirements on load combinations.
The column being considered is not subjected to especially severe service conditions or extraordinary hazard.

Reference and Adjusted Design Values for No. 1 Southern Pine 2x6

<table>
<thead>
<tr>
<th>Property</th>
<th>Value (psi)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fb</td>
<td>1350</td>
<td>(NDS Supplement Table 4B)</td>
</tr>
<tr>
<td>Ec</td>
<td>1550</td>
<td>(NDS Table 4.3.1)</td>
</tr>
<tr>
<td>Cfu</td>
<td>1.0</td>
<td>Flat use factors are for weak axis bending. However, since lateral support is provided along the entire narrow face, Cfu = 1.0</td>
</tr>
<tr>
<td>Cm</td>
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<td></td>
</tr>
<tr>
<td>Ct</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Cf</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Emin</td>
<td>580000</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>1600000</td>
<td></td>
</tr>
<tr>
<td>Emin'</td>
<td>E_min Cm Ct C1 C1 C1</td>
<td>equation for sawn lumber. (3.7.1)</td>
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</table>

Member length and properties

<table>
<thead>
<tr>
<th>Length</th>
<th>Width</th>
<th>Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>l : 9 ft</td>
<td>b : 1.5 in</td>
<td>d : 5.5 in</td>
</tr>
</tbody>
</table>

\[ A_g = b \cdot d \]
\[ S_x = \frac{b \cdot d^2}{6} \]
\[ S_y = \frac{d \cdot b^2}{6} \]

\[ A_g = 8.25 \text{ in}^2 \]
\[ S_x = 7.562 \text{ in}^3 \]
\[ S_y = 2.062 \text{ in}^3 \]

Applied Loads

\[ w_{strong} := \frac{25 \text{ lbf}}{\text{ft}^2} \]
\[ w_{trib} := 4 \text{ ft} \]
\[ P_{snow} := 840 \text{ lbf} \]
\[ P_{Dead} := 560 \text{ lbf} \]

\[ w_{strong} \cdot w_{trib} = 100 \text{ lbf/ft} \]

\[ w_{weak} := 0 \]

Load applied to weak axis of beam column.
In this example, no load is applied.
Load Case 1: DL + SL + WL

\[ C_D := 1.6 \]

**Compression**

\[ F_{c\ast} := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \]

\[ F_{c\ast} = 2480 \text{ psi} \]

\[ P_1 := P_{\text{snow}} + P_{\text{Dead}} \]

\[ P_1 = 1400 \text{ lbf} \]

\[ f_{c1} := \frac{P_1}{A_g} \]

\[ f_{c1} = 170 \text{ psi} \]

**Appendix B Section B.2 (non-mandatory)**

\[ F_{c\ast} \] is reference compression parallel to grain design value adjusted with all adjustment factors except the column stability factor \( C_P \). The following calculations determine the column stability factor \( C_P \):

Subscripts refer to Load Case

**Determine Effective Lengths and Critical Buckling Design Values**

\[ K_e := 1.0 \]

\[ d_1 := d \quad l_1 := 1 \quad l_{e1} := K_e \cdot l_1 \cdot \left(\frac{12 \text{ in}}{\text{ft}}\right) \]

\[ d_2 := b \quad l_2 := \frac{7}{12} \text{ ft} \quad l_{e2} := K_e \cdot l_2 \cdot \left(\frac{12 \text{ in}}{\text{ft}}\right) \]

\[ l_{e1} = 108 \text{ in} \quad l_{e2} = 0 \]

\[ l_e := \max \left(\frac{l_{e1}}{l_{e2}}\right) \quad l_e = 108 \text{ in} \]

\[ F_{cE} := \frac{0.822 \cdot E'_{\min} \cdot l_{e1}^2}{d_1} \quad F_{cE} = 1236 \text{ psi} \]

\[ R_B := \frac{l_e \cdot d}{b^2} \quad R_B = 16 \]

\[ F_{bE} := \frac{1.20 \cdot E'_{\min} \cdot R_B^2}{R_B} \quad F_{bE} = 2636 \text{ psi} \]

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Determine Column Stability Factor $C_p$

\[
C_p := \frac{1 + \left( \frac{F_{cE}}{F_{c^*}} \right)}{2 \cdot c} - \sqrt{\left[ \frac{1 + \left( \frac{F_{cE}}{F_{c^*}} \right)}{2 \cdot c} \right]^2 - \frac{F_{cE}}{F_{c^*}}} \]

\[C_p = 0.433\]

\[F_{c'} := F_{c^*} \cdot C_p\]

\[F_{c'} = 1073 \text{ psi}\]

\[f_{c1} = 170 \text{ psi}\]

\[F_{c'} = 1073 \text{ psi}\]

Actual compression stress $f_c$ does not exceed adjusted compression design value $F'_c$.

**Bending**

\[C_L := 1.0\]

\[F_{b1}' := F_b \cdot \bar{C}_D \cdot C_M \cdot C_L \cdot C_t \cdot C_F \cdot C_i \cdot C_r\]

\[F_{b1}' = 2160 \text{ psi}\]

\[M_{\text{max}1} := \frac{\left( w_{\text{strong}} w_{\text{trib}} \right)^2 \cdot 12 \cdot \text{in} \cdot \text{ft}}{8}\]

\[M_{\text{max}1} = 12150 \text{ in} \cdot \text{lbf}\]

\[f_{b1} := \frac{M_{\text{max}1}}{S_x}\]

\[f_{b1} = 1607 \text{ psi}\]

\[f_{b1} = 1607 \text{ psi}\]

\[F_{b1}' = 2160 \text{ psi}\]

Actual bending stress in strong direction resulting from wind load applied to narrow face of beam-column.

**Combined Bending and Axial Compression**

\[
\frac{\left( f_{c1} \right)^2}{F_{c'}} + \frac{f_{b1}}{F_{b1}' \cdot \left[ 1 - \left( \frac{f_{c1}}{F_{cE}} \right) \right]} = 0.89
\]

\[f_{c1} = 170 \text{ psi}\]

\[F_{cE} = 1236 \text{ psi}\]

\[f_{b1} = 1607 \text{ psi}\]

\[F_{bE} = 2636 \text{ psi}\]

Actual compression stress is less than critical bucking design values for strong axis buckling. OK (NDS 3.9.2)

Actual bending stress does not exceed adjusted parallel to grain design value. OK (NDS 3.9.2)
Load Case 2: DL+SL

Load duration is not included in $E'$ calculation. It is important to evaluate multiple combinations such that all load duration effects are considered.

**Compression**

\[
F_{c*} := F_c C_D C_M C_t C_F C_i \quad P_2 := P_{snow} + P_{Dead} \quad f_{c2} := \frac{P_2}{A_g}
\]

\[
F_{c*} = 1782 \text{ psi} \quad P_2 = 1400 \text{-lbf} \quad f_{c2} = 170 \text{-psi}
\]

Determine Column Stability Factor $C_p$

\[
C_p := \frac{1 + \left( \frac{F_{cE}}{F_{c*}} \right)}{2c} - \sqrt{\left[ 1 + \left( \frac{F_{cE}}{F_{c*}} \right) \right]^2 - \left( \frac{F_{cE}}{F_{c*}} \right)}
\]

$C_p = 0.555$

$F_{c'} := F_{c*} \cdot C_p$

$F_{c'} = 990 \text{-psi}$

$f_{c2} = 170 \text{-psi} \quad F_{c'} = 990 \text{-psi}$

Actual compression parallel to grain stress $f_c$ does not exceed adjusted compression parallel to grain design value $F_c'$. OK

Load Case 3: DL only

$C_D := 0.90$

**Compression**

\[
F_{c*} := F_c C_D C_M C_t C_F C_i \quad P_3 := P_{Dead} \quad f_{c3} := \frac{P_3}{A_g}
\]

\[
F_{c*} = 1395 \text{ psi} \quad P_3 = 560 \text{-lbf} \quad f_{c3} = 68 \text{-psi}
\]

Determine Column Stability Factor $C_p$

\[
C_p := \frac{1 + \left( \frac{F_{cE}}{F_{c*}} \right)}{2c} - \sqrt{\left[ 1 + \left( \frac{F_{cE}}{F_{c*}} \right) \right]^2 - \left( \frac{F_{cE}}{F_{c*}} \right)}
\]

$C_p = 0.648$

$F_{c'} := F_{c*} \cdot C_p$

$F_{c'} = 904 \text{-psi}$

$f_{c3} = 68 \text{-psi} \quad F_{c'} = 904 \text{-psi}$

Actual compression stress does not exceed adjusted compression parallel to grain design values for strong axis buckling. OK (NDS 3.9.2)
E1.8 - Combined Bi-axial Bending and Axial Compression (ASD)

A No. 2 Southern Pine nominal 2x4 oriented flatwise is being considered for use as a member within the top chord of a parallel chord gable end truss. The member is 3 ft long (between panel points) and will be subjected to axial compression forces of 300 lb dead load (DL) and 600 lb snow load (SL), concentrated loads of 50 lb (DL) and 100 lb (SL) at the midpoint of the member on its wide face and 120 lb wind load (WL) at the midpoint of the member on its narrow face. Lateral support is provided only at the ends of the member and the ends are considered pinned.

Check the adequacy of the beam-column for bi-axial bending and axial compression for the appropriate load cases.

Notes:
Load cases used in this example have been simplified for clarity. Refer to NDS Section 1.4.4 for requirements on load combinations.
E1.8 - Combined Bi-axial Bending and Axial Compression (ASD)

A No. 2 Southern Pine nominal 2x4 oriented flatwise is being considered for use as a member within the top chord of a parallel chord gable end truss. The member is 3 ft long (between panel points) and will be subjected to axial compression forces of 300 lb dead load (DL) and 600 lb snow load (SL), concentrated loads of 50 lb (DL) and 100 lb (SL) at the midpoint of the member on its wide face and 120 lb wind load (WL) at the midpoint of the member on its narrow face. Lateral support is provided only at the ends of the member and the ends are considered pinned.

Check the adequacy of the beam-column for bi-axial bending and axial compression for the appropriate load cases.

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 No. 2 Southern Pine 2x4

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_b$</td>
<td>1100-psi</td>
</tr>
<tr>
<td>$E$</td>
<td>1400000-psi</td>
</tr>
<tr>
<td>$E_{min}$</td>
<td>510000-psi</td>
</tr>
<tr>
<td>$F_c$</td>
<td>1450-psi</td>
</tr>
<tr>
<td>$C_M$</td>
<td>1.0</td>
</tr>
<tr>
<td>$C_t$</td>
<td>1.0</td>
</tr>
<tr>
<td>$C_F$</td>
<td>1.0</td>
</tr>
<tr>
<td>$C_{fu}$</td>
<td>1.10</td>
</tr>
<tr>
<td>$C_i$</td>
<td>1.0</td>
</tr>
<tr>
<td>$C_r$</td>
<td>1.0</td>
</tr>
<tr>
<td>$c$</td>
<td>0.8</td>
</tr>
</tbody>
</table>

\[ E'_{min} = E_{min} C_M C_t C_i \]

\[ E'_{min} = 510000 \text{ psi} \]

Member length and properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$l$</td>
<td>3-ft</td>
</tr>
<tr>
<td>$b$</td>
<td>1.5-in</td>
</tr>
<tr>
<td>$d$</td>
<td>3.5-in</td>
</tr>
</tbody>
</table>

\[ A_g = b \cdot d \]

\[ S_x = \frac{b \cdot d^2}{6} \]

\[ S_y = \frac{d \cdot b^2}{6} \]

\[ A_g = 5.25-\text{in}^2 \]

\[ S_x = 3.062-\text{in}^3 \]

\[ S_y = 1.312-\text{in}^3 \]

Applied Loads

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>DLAxial</td>
<td>300-lbf</td>
</tr>
<tr>
<td>SLAxial</td>
<td>600-lbf</td>
</tr>
<tr>
<td>DLWeak</td>
<td>50-lbf</td>
</tr>
<tr>
<td>SLWeak</td>
<td>100-lbf</td>
</tr>
<tr>
<td>WLStrong</td>
<td>120-lbf</td>
</tr>
</tbody>
</table>

Applied loads. Subscripts depict load type (DL-dead load, SL-snow load and WL-wind load) and application in relation to the member (applied to strong or weak axis).
Load Case 1: DL + SL + WL

$C_D := 1.6$

Compression

$F_{c^*} := \frac{F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i}{A_g}$

$F_{c^*} = 2320 \text{ psi}$

$P_1 := DL_{Axial} + SL_{Axial}$

$P_1 = 900 \cdot \text{lbf}$

$f_{c1} := \frac{P_1}{A_g}$

$f_{c1} = 171 \cdot \text{psi}$

Notes:

- Check the adequacy of the beam-column for bi-axial bending and axial compression for the appropriate load considered pinned.
- Lateral support is provided only at the ends of the member and the ends are of the member on its narrow face.
- Load cases used in this example have been simplified for clarity. Refer to NDS Section 1.4.4 for requirements.

Determine Column Stability Factor $C_p$

$F_c := \min\left(\frac{F_{cE1}}{F_{cE2}}\right)$

$F_c = 728 \text{ psi}$

$F_{cE} := \min\left(\frac{F_{cE1}}{F_{cE2}}\right)$

$F_{cE} = 728 \text{ psi}$

$F_{c^*}$ is reference compression parallel to grain design value adjusted with all adjustment factors except the column stability factor $C_p$.

Critical bucking compression design values for compression member in planes of lateral support (NDS 3.7.1)

$K_e := 1.0$

$d_1 := d \quad l_1 := 1 \quad l_{e1} := K_e \cdot l_1 \cdot \left(\frac{12 \text{ in}}{\text{ft}}\right)$

$d_2 := b \quad l_2 := 1 \quad l_{e2} := K_e \cdot l_2 \cdot \left(\frac{12 \text{ in}}{\text{ft}}\right)$

$l_{e1} = 36 \text{ in}$

$l_{e2} = 36 \text{ in}$

$\frac{l_{e1}}{d_1} = 10 \quad \frac{l_{e2}}{d_2} = 24$

$F_{cE1} := \frac{0.822 \cdot F_{c^*}}{\left(\frac{l_{e1}}{d_1}\right)^2}$

$F_{cE1} = 3963 \text{ psi}$

$F_{cE2} := \frac{0.822 \cdot F_{c^*}}{\left(\frac{l_{e2}}{d_2}\right)^2}$

$F_{cE2} = 728 \text{ psi}$

Effective lengths in each axis

$l_{e1}/d$ is less than 50 for each axis (NDS 3.7.1.4)

Critical buckling compression design values for compression member in planes of lateral support (NDS 3.7.1)

Buckling length coefficient $K_e$ for rotation free/translation fixed (pinned/pinned) column (NDS Appendix G Table G1)

Beam dimensions, laterally unsupported lengths (Figure 3F) and effective column lengths (NDS 3.7.1) for buckling in each direction. Subscript 1 is strong (but unsupported) axis; Subscript 2 is the weak but laterally supported axis.

Determine Effective Lengths and Critical Buckling Design Values

$K_e := 1.0$

$\frac{l_{e1}}{d_1} = 10$

$\frac{l_{e2}}{d_2} = 24$

$F_{cE1} := \frac{0.822 \cdot F_{c^*}}{\left(\frac{l_{e1}}{d_1}\right)^2}$

$F_{cE1} = 3963 \text{ psi}$

$F_{cE2} := \frac{0.822 \cdot F_{c^*}}{\left(\frac{l_{e2}}{d_2}\right)^2}$

$F_{cE2} = 728 \text{ psi}$

$F_{c^*}$ is reference compression parallel to grain design value adjusted with all adjustment factors except the column stability factor $C_p$.

Critical bucking compression design values for compression member in planes of lateral support (NDS 3.7.1)

Buckling length coefficient $K_e$ for rotation free/translation fixed (pinned/pinned) column (NDS Appendix G Table G1)

Beam dimensions, laterally unsupported lengths (Figure 3F) and effective column lengths (NDS 3.7.1) for buckling in each direction. Subscript 1 is strong (but unsupported) axis; Subscript 2 is the weak but laterally supported axis.

Effective lengths in each axis

$l_{e1}/d$ is less than 50 for each axis (NDS 3.7.1.4)

Critical buckling compression design values for compression member in planes of lateral support (NDS 3.7.1)
**Determine Adjusted Compression Parallel to Grain Design Value** $F'_c$

\[
F_c' := F_c \cdot C_p
\]

Adjusted compression parallel to grain design value

\[
f_{c1} = 171 \cdot \text{psi} \quad F_c' = 673 \cdot \text{psi}
\]

Actual compression stress $f_c$ does not exceed adjusted compression design value $F_c'$ (NDS 3.6.3). OK

**Narrow Face (Strong Axis) Bending - (load parallel to wide face)**

\[
F'_b* := F_b \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \cdot C_r
\]

$F'_b*$ is adjusted bending design value with all adjustment factors except the beam stability factor $C_L$ and flat use factor $C_{fu}$ applied.

$F'_b* = 1760 \text{ psi}$

**Determine Strong Axis Beam Stability Factor** $C_L$ **and Adjusted Bending Design Values**

\[
l_u := \frac{12 \text{ in}}{\text{ft}} \quad l_u = 36 \text{ in} \quad \text{Laterally unsupported length}
\]

\[
\frac{l_u}{d} = 10.3
\]

\[
l_e := 1.37 \cdot l_u + 3 \cdot d \quad l_e = 59.8 \text{ in} \quad (\text{Table 3.3.3})
\]

\[
R_B := \frac{l_e \cdot d}{b^2} \quad R_B = 9.6 \quad \text{Slenderness ratio for bending (3.3-5) } R_B < 50 \text{ (NDS 3.3.3.7)}
\]

\[
F_{bE} := \frac{1.20 \cdot F'_{min}}{R_B^2} \quad F_{bE} = 6577 \text{ psi} \quad \text{Critical bucking design value for bending (3.7.1)}
\]

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

\[
C_L = 0.982
\]

Resulting beam stability factor $C_L$. As an alternative, 4.4.1.2 (b) allows $C_L = 1.0$ for $d/b = (4/2)$ when the ends are held in position by full depth solid blocking, bridging, hangers, nailing, bolting or other suitable means.

$F'_b1 := F_b \cdot C_D \cdot C_M \cdot C_L \cdot C_t \cdot C_F \cdot C_i \cdot C_r$

$F'_b1 = 1729 \text{ psi}$

$F'_b1$ is adjusted edgewise bending design value with all adjustment factors.
**Determine Bending Load and Resulting Bending Stress - Strong Axis Bending**

\[
M_{1\text{Strong}} = \frac{W_{\text{L-Strong}} \cdot 1 \cdot 12 \text{ in}}{4 \text{ ft}}
\]

\[
M_{1\text{Strong}} = 1080 \text{ in} \cdot \text{lb}
\]

\[
f_{b1} = \frac{M_{1\text{Strong}}}{S_x}
\]

\[
f_{b1} = 353 \text{ psi} \quad F'_{b1} = 1729 \text{ psi}
\]

Actual bending stress in strong direction resulting from wind load applied to narrow face of beam-column.

Ok. Actual bending stress \( f_b \) does not exceed adjusted edgewise compressive design value \( F'_{b1} \).

**Wide Face (Weak Axis) Adjusted Bending Design Value - (load parallel to narrow face)**

\[
C_L = 1.0
\]

\[
C_{fu} = 1.1
\]

\[
F'_{b2} = F_b \cdot C_D \cdot C_M \cdot C_L \cdot C_t \cdot C_{fu} \cdot C_F \cdot C_r
\]

\[
F'_{b2} = 1936 \text{ psi}
\]

Since \( d < b \) (2 in < 4 in) \( C_L = 1.0 \)

Flat use factor \( C_{fu} \) applies for weak axis bending.

\( F'_{b2} \) is adjusted flatwise bending design value with all adjustment factors.

**Determine Bending Load and Resulting Bending Stress - Weak Axis Bending**

\[
M_{1\text{Weak}} = \frac{(D_{\text{L-Weak}} + S_{\text{L-Weak}}) \cdot 1 \cdot 12 \text{ in}}{4 \text{ ft}}
\]

\[
M_{1\text{Weak}} = 1350 \text{ in} \cdot \text{lb}
\]

\[
f_{b2} = \frac{M_{1\text{Weak}}}{S_y}
\]

\[
f_{b2} = 1029 \text{ psi} \quad F'_{b2} = 1936 \text{ psi}
\]

Bending stress in strong direction resulting from dead and snow load applied to narrow face of beam-column.

Ok. Actual bending stress \( f_b \) does not exceed adjusted bending design value \( F_b \) (NDS 3.3.1).
Combined Bi-axial Bending and Axial Compression

\[
\left(\frac{f_{c1}}{F_c'}\right)^2 + \frac{f_{b1}}{F'_{b1}} \left[ 1 - \left(\frac{f_{c1}}{F_{cE1}}\right) \right] + \frac{f_{b2}}{F'_{b2}} \left[ 1 - \left(\frac{f_{c1}}{F_{cE2}}\right) - \left(\frac{f_{b1}}{F_{bE}}\right)^2 \right] = 0.98 < 1.0 \text{ ok (3.9-3)}
\]

\[
\frac{f_{c1}}{F_{cE2}} + \left(\frac{f_{b1}}{F_{bE}}\right)^2 = 0.24 < 1.0 \text{ ok (3.9-4)}
\]

- \(f_{c1} = 171\) psi \(\quad F_{cE1} = 3963\) psi \quad Actual compression stress is less than critical buckling design values for both weak and strong axis buckling. Ok
- \(f_{c1} = 171\) psi \(\quad F_{cE2} = 728\) psi
- \(f_{b1} = 353\) psi \(\quad F_{bE} = 6577\) psi \quad Actual bending stress is less than critical buckling design value. Ok
**Load Case 2: DL+SL**

\[ C_D := 1.15 \]

\[ P_2 := \text{DL}_{\text{Axial}} + \text{SL}_{\text{Axial}} \quad P_2 = 900 \text{ lbf} \]

\[ f_{c2} := \frac{P_1}{A_g} \]

**Compression**

\[ F_{c*} := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \]

\[ F_{c*} = 1667 \text{ psi} \quad f_{c2} = 171 \cdot \text{psi} \]

**Determine Column Stability Factor \( C_p \)**

\[ C_p := \frac{1 + \left( \frac{F_{cE}}{F_{c*}} \right)}{2 \cdot c} - \sqrt{\left[ 1 + \left( \frac{F_{cE}}{F_{c*}} \right) \right]^2 - \left( \frac{F_{cE}}{F_{c*}} \right)} \]

\[ C_p = 0.387 \]

\[ F_{c'} := F_{c*} \cdot C_p \]

\[ F_{c'} = 646 \cdot \text{psi} \]

\[ f_{c2} = 171 \cdot \text{psi} \quad F_{c'} = 646 \cdot \text{psi} \]

Actual compression stress \( f_c \) does not exceed adjusted compression design value \( F'_c \) OK

**Weak Axis Bending**

\[ F'_{b2} := F_b \cdot C_D \cdot C_M \cdot C_L \cdot C_{fu} \cdot C_F \cdot C_t \cdot C_r \]

\[ F'_{b2} = 1392 \text{ psi} \]

\[ M_{2\text{Weak}} := \frac{(\text{DL}_{\text{Weak}} + \text{SL}_{\text{Weak}}) \cdot 12 \text{ in}}{4} \]

\[ M_{2\text{Weak}} = 1350 \cdot \text{in-lbf} \]

\[ f_{b2} := \frac{M_{2\text{Weak}}}{S_y} \]

\[ f_{b2} = 1029 \cdot \text{psi} \quad F'_{b2} = 1392 \text{ psi} \]

Bending stress in strong direction resulting from snow load applied to narrow face of beam-column

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

**Strong Axis Bending**

Wind load is zero. No strong axis bending \( f_{b1} \) in following equations set to zero
Combined Bending and Compression

\[
\left( \frac{f_{c2}}{F_{c'}} \right)^2 + \frac{f_{b1} \cdot 0}{F'_{b1}} + \frac{f_{b2}}{F'_{b2}} \left[ 1 - \left( \frac{f_{c2}}{F_{cE1}} \right) \right] \left[ 1 - \left( \frac{f_{c2}}{F_{cE2}} \right) - \left( \frac{f_{b1} \cdot 0}{F_{bE}} \right)^2 \right] = 1.04
\]

\[ \approx 1.0 \text{ OK (NDS 3.9-3)} \]

Engineering judgement required for rounding.

Checking NDS Equation 3.9-4 not necessary since \( f_{b1} = 0 \)

\[ f_{c2} = 171 \text{ psi} \quad F_{cE1} = 3963 \text{ psi} \quad \text{Actual compression stress is less than critical bucking design values for both weak and strong axis buckling. OK} \]

\[ f_{c2} = 171 \text{ psi} \quad F_{cE2} = 728 \text{ psi} \]

\[ f_{b2} = 1029 \text{ psi} \quad F_{bE} = 6577 \text{ psi} \quad \text{Actual bending stress is less than critical buckling design value. OK} \]
**Load Case 3: DL only**

\[ C_D := 0.90 \]

**Compression**

\[ F_{c*} := F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i \quad P_3 := DL_{Axial} \]

\[ F_{c*} = 1305 \text{ psi} \quad P_3 = 300 \cdot \text{lbf} \]

\[ f_c := \frac{P_3}{A_g} \]

\[ f_c = 57 \cdot \text{psi} \]

**Determine Column Stability Factor \( C_p \)**

\[
C_p := \frac{1 + \left( \frac{F_{cE}}{F_{c*}} \right)}{2 \cdot c} - \sqrt{\left( 1 + \left( \frac{F_{cE}}{F_{c*}} \right) \right)^2 - \left( \frac{F_{cE}}{F_{c*}} \right)^2} - \frac{2 \cdot F_{cE}}{c}
\]

\[ C_p = 0.473 \]

\[ F_{c'} := F_{c*} \cdot C_p \]

\[ F_{c'} = 617 \cdot \text{psi} \]

\[ f_{c3} = 57 \cdot \text{psi} \quad F_{c'} = 617 \cdot \text{psi} \]

Actual compression parallel to grain stress \( f_c \) does not exceed adjusted compression parallel to grain design value \( F_c' \) OK

**Weak Axis Bending**

**\( F_{b2} := F_b \cdot C_D \cdot C_M \cdot C_L \cdot C_t \cdot C_{fu} \cdot C_F \cdot C_i \cdot C_r \)**

\[ F_{b2} = 1089 \text{ psi} \]

\[ M_{3\text{Weak}} := \frac{(DL_{Weak}) \cdot 1.12 \cdot \text{in} \cdot \text{ft}}{4} \]

\[ M_{3\text{Weak}} = 450 \cdot \text{in} \cdot \text{lbf} \]

\[ f_{b3} := \frac{M_{3\text{Weak}}}{S_y} \]

\[ f_{b3} = 343 \cdot \text{psi} \quad F_{b2} = 1089 \text{ psi} \]

Actual bending stress \( f_b \) does not exceed adjusted bending design value \( F_b \) OK

**Strong Axis Bending**

Wind load is zero. No strong axis bending \( (f_{b1} = 0) \)
**Combined Bending and Compression**

\[
\frac{f_{c3}^2}{F_{cE1}} \left(1 - \frac{f_{c3}}{F_{cE1}}\right) + \frac{f_{b1} \cdot 0}{F_{b1}} \left(1 - \frac{f_{c3}}{F_{cE2}} - \frac{f_{b1} \cdot 0}{F_{bE}}\right)^2 = 0.35 < 1.0 \text{ OK (NDS 3.9-3)}
\]

Checking NDS Equation 3.9-4 not necessary since \( f_{b1} = 0 \)

- \( f_{c3} = 57 \text{ psi} \) \( F_{cE1} = 3963 \text{ psi} \) Actual compression stress is less than critical buckling design values for both weak and strong axis buckling. OK
- \( f_{c3} = 57 \text{ psi} \) \( F_{cE2} = 728 \text{ psi} \)
- \( f_{b3} = 343 \text{ psi} \) \( F_{bE} = 6577 \text{ psi} \) Actual bending stress is less than critical buckling design value. OK
E1.9 - Loadbearing Wall Wood Stud Resisting Wind and Gravity Loads

Background: The 2 story home considered in the Design of Wood Frame Buildings for High Wind, Snow, and Seismic Loads (WFCM Workbook) has a Foyer with a vaulted ceiling. The south bearing wall of the Foyer must support gravity loads from the roof and attic above, must resist reactions from uplift wind forces on the roof and must resist out-of-plane wind pressures. The home is located in an area with a basic wind speed of 160 mph - Exposure B.

The Foyer originally had a 4 foot wide plant shelf at the second floor level. The plant shelf was platform framed as part of the floor diaphragm and limited stud length to one story. The resulting configuration was within the limitations of the prescriptive provisions of the Wood Frame Construction Manual (WFCM) and the wall framing could be determined from Chapter 3 of the WFCM.

Removing the plant shelf requires the wall to be balloon framed and will increase the stud lengths to 19 feet where they are no longer within the limitations of the prescriptive provisions of the WFCM.

Goal: Determine requirements for studs that are balloon framed from the first floor to the roof.

Approach: Analyze wall framing as part of the Main Wind Force Resisting System (MWFRS) exposed to in-plane and out-of-plane load combinations specified by ASCE 7 Minimum Design Loads for Buildings and Other Structures. Analyze wall framing as Components and Cladding (C&C) exposed to out of plane C&C wind pressures only. Design wall framing per the National Design Specification (NDS) for Wood Construction.

In this example, the following loads are assumed:

<table>
<thead>
<tr>
<th>Roof Loads</th>
<th>Attic/Ceiling</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Load</td>
<td>10 psf</td>
</tr>
<tr>
<td>Live Load</td>
<td>20 psf</td>
</tr>
<tr>
<td>Ground Snow Load</td>
<td>30 psf</td>
</tr>
<tr>
<td>Dead Load</td>
<td>15 psf</td>
</tr>
<tr>
<td>Attic Live Load</td>
<td>30 psf</td>
</tr>
</tbody>
</table>

Additionally, the following design assumptions apply:

- Stud spacing = 16" o.c.
- Exterior Sheathing = Wood Structural Panels
- Interior Sheathing = 1/2" Gypsum Wallboard

The analysis involves an iterative approach. Initial values are selected for the member properties (depth, number of members and their species group and grade); initial analyses are completed and stresses and deflections determined and compared to allowable values. The member properties are then varied and analyses repeated until stress and deflection criteria are satisfied.

Note: As stated in the Foreword, examples are applicable to both the 2015 and 2018 versions of the NDS and WFCM unless otherwise noted. This example also references ASCE 7 and is applicable to both the 2010 and 2016 versions. Where section, figure, and table designations vary between the two versions of ASCE 7, they are noted. Reference to the International Building Code (IBC) is also applicable to the 2015 and 2018 versions.
Reference and Adjusted Design Values (values were revised during iterations - final values are for No. 2 SP 2x8)

\[ F_b := 925 \text{ psi} \quad E := 1400000 \text{ psi} \quad \text{Emin} := 510000 \text{ psi} \]

(NDS Supplement Table 4B)

\[ F_c := 1350 \text{ psi} \quad F_t := 550 \text{ psi} \]

(NDS Table 4.3.1)

\[ C_M := 1.0 \quad C_t := 1.0 \]

C\textsubscript{t} used to represent Wall Stud

\[ C_{fu} := 1.0 \quad C_i := 1.0 \quad C_r := 1.25 \]

Repetitive Member Factors (SDPWS 3.1.1.1)

\[ C_f := 1.0 \quad C_T := 1.0 \quad c := 0.8 \]

factor “c” in column stability factor C\textsubscript{P}
equation for sawn lumber. (NDS 3.7.1)

\[ E' \text{min} := E' \text{min} \cdot C_M \cdot C_i \cdot C_t \cdot C_T \quad E' \text{min} = 510000 \text{ psi} \]

\[ E' \text{min} := E' \text{min} \cdot C_M \cdot C_i \cdot C_t \cdot C_T \]

(Factor “c” in column stability factor C\textsubscript{P})

Member Properties

\[ n := 1 \quad b := 1.5 \text{ in} \quad d := 7.25 \text{ in} \]

n is the number of full length studs

\[ A_g := n \cdot b \cdot d \quad S_x := \frac{n \cdot b \cdot d^2}{6} \quad I_x := \frac{n \cdot b \cdot d^3}{12} \]

\[ A_g = 10.9 \text{ in}^2 \quad S_x = 13.1 \text{ in}^3 \quad I_x = 47.6 \text{ in}^4 \]

Home Dimensions

\[ L := 19 \text{ ft} \quad \text{length of balloon-framed studs} \quad W := 32 \text{ ft} \quad \text{building width} \quad W_{ovhg} := 2 \text{ ft} \quad \text{width of roof overhang} \]

Determine Distributed Loads Supported by the South Wall

Dead and Live Loads

\[ w_{DLAttic} := 15 \cdot \frac{\text{lbf}}{\text{ft}^2} \cdot \frac{1}{2} \cdot 16 \text{ ft} \]

\[ w_{DLRoof} := 10 \cdot \frac{\text{lbf}}{\text{ft}^2} \cdot \frac{1}{2} \cdot 36 \text{ ft} \]

\[ w_{DLAttic} = 120 \cdot \text{plf} \quad w_{DLRoof} = 180 \cdot \text{plf} \]

\[ w_{LLAttic} := 30 \cdot \frac{\text{lbf}}{\text{ft}^2} \cdot \frac{1}{2} \cdot 16 \text{ ft} \]

\[ w_{LLRoof} := 20 \cdot \frac{\text{lbf}}{\text{ft}^2} \cdot \frac{1}{2} \cdot 36 \text{ ft} \]

\[ w_{LLAttic} = 240 \cdot \text{plf} \quad w_{LLRoof} = 360 \cdot \text{plf} \]

\[ w_{totalDead} := w_{DLAttic} + w_{DLRoof} \]

\[ w_{totalDead} = 300 \cdot \text{plf} \]

Rain Load

\[ R := 0 \cdot \text{plf} \]

Earthquake Load

\[ E_i := 0 \cdot \text{plf} \]

Rain and earthquake loads are included in ASCE 7 load combinations. The subscript for the earthquake load is used to differentiate the earthquake load from the modulus of elasticity.
Snow Load

\[ p_g := 30 \cdot \frac{\text{lbf}}{\text{ft}^2} \]

\[ C_e := 1.0 \quad C_{ts} := 1.0 \quad I_s := 1.0 \]

\[ p_f := 0.7 \cdot C_e \cdot C_{ts} \cdot I_s \cdot p_g \]

\[ p_f = 21 \cdot \text{psf} \]

\[ C_s := 1.0 \]

\[ p_{sBal} := C_s \cdot p_f \quad p_{sBal} = 21 \cdot \text{psf} \]

\[ p_{sUnBal} := I_s \cdot p_g \quad p_{sUnBal} = 30 \cdot \text{psf} \]

\[ p_{sBal} \left( \frac{1}{2} \cdot 36 \cdot \text{ft} \right) = 378 \cdot \text{plf} \]

\[ p_{sUnBal} \left( \frac{3}{4} \cdot 18 \cdot \text{ft} \right) = 405 \cdot \text{plf} \]

\[ w_{\text{snow}} := \max \left( p_{sBal} \left( \frac{1}{2} \cdot 36 \cdot \text{ft} \right), p_{sUnBal} \left( \frac{3}{4} \cdot 18 \cdot \text{ft} \right) \right) \]

\[ w_{\text{snow}} = 405 \cdot \text{plf} \]

Calculate MWFRS Wind Loads

MWFRS Wind Pressures are calculated using the Envelope Procedure contained in Chapter 28 of ASCE 7. The wind pressure equation is:

\[ p = q_h[(G_{pL})-(G_{pI})] \quad (\text{ASCE 7-10 Eq. 28.4-1 or ASCE 7-16 Eq. 28.3-1}) \]

Where:

- \( q_h \) is the velocity pressure
- \( G_{pL} \) is the external pressure coefficient for the surface being analyzed and
- \( G_{pI} \) is the internal pressure coefficient

Determine Velocity Pressure \( q_h \)

\[ V := 160 \]

\[ K_z := 0.70 \]

\[ K_d := 0.85 \quad \text{ASCE 7 Table 26.6-1} \]

\[ K_{zt} := 1.0 \quad \text{ASCE 7 Section 26.8.2} \]

\[ q_h := (0.60) \cdot 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot \frac{\text{lbf}}{\text{ft}^2} \]

\[ q_h = 23.4 \cdot \text{psf} \]

Note: The 160 Exp B velocity pressures \( q_h \) in the WFCM is 24.06 psf and is based on a 33 ft MRH where the velocity pressure exposure coefficient \( K_z \) for Exp B is 0.72.

\[ K_z \] for Exp B evaluated at 25 ft MRH in this example is 0.70 per ASCE 7-10 Table 28.3.1 (ASCE 7-16 Table 26.10-1) and produces a slightly lower velocity pressure of 23.4 psf.

The 0.60 factor in the velocity pressure equation incorporates ASCE 7 load factors for allowable stress design (ASD) load combinations.
Determine MWFRS Roof Pressure Coefficients ($GC_{pf}$)

ASCE 7-10 Figure 28.4-1 (ASCE 7-16 Figure 28.3-1) shows the external pressure coefficient for interior and end zones for two load cases. Load Case A is for wind perpendicular to the ridge; Load Case B is for wind parallel to the ridge. By observation, $GC_{pf} = -0.18$ results in the highest positive reaction at roof supports for Case A and $GC_{pf} = +0.18$ results in the highest positive reaction at roof supports for Case B.

<table>
<thead>
<tr>
<th>Zone 2 (windward)</th>
<th>Zone 3 (leeward)</th>
<th>Roof Overhang</th>
<th>Internal</th>
</tr>
</thead>
<tbody>
<tr>
<td>$GC_{pfAWW}$ := 0.21</td>
<td>$GC_{pfALW}$ := -0.43</td>
<td>$GC_{pOH}$ := -0.70</td>
<td>$GC_{pi}$ := -0.18</td>
</tr>
</tbody>
</table>

(Determine MWFRS Wind Pressures on Roof for Load Cases A and B)

Load Case A - wind perpendicular to ridge

windward roof overhang

$$WRO_A := q_h \left( GC_{pfAWW} + GC_{pOH} \right)$$

$$WRO_A = -11.5 \text{ psf}$$

windward roof

$$WR_A := q_h \left( GC_{pfAWW} - GC_{pi} \right)$$

$$WR_A = 9.1 \text{ psf}$$

leeward roof

$$LR_A := q_h \left( GC_{pfALW} - GC_{pi} \right)$$

$$LR_A = -5.8 \text{ psf}$$

Case A – Wind Perpendicular to Ridge
Load Case B - wind parallel to ridge

<table>
<thead>
<tr>
<th>Zone 2 (windward)</th>
<th>Zone 3 (leeward)</th>
<th>Roof Overhang</th>
<th>Internal</th>
</tr>
</thead>
<tbody>
<tr>
<td>$G_{CpfBWW} := -0.69$</td>
<td>$G_{CpfBLW} := -0.37$</td>
<td>$G_{CpOH} := -0.70$</td>
<td>$G_{Cpi} := 0.18$</td>
</tr>
</tbody>
</table>

windward roof overhang

$W_{ROB} := q_h \cdot (G_{CpfBWW})$

$W_{ROB} = -16.1 \text{ psf}$

windward roof

$W_{RB} := q_h \cdot (G_{CpfBWW} - G_{Cpi})$

$W_{RB} = -20.4 \text{ psf}$

leeward roof

$L_{RB} := q_h \cdot (G_{CpfBLW} - G_{Cpi})$

$L_{RB} = -12.9 \text{ psf}$

**Windward Wall and Roof**

\[
\begin{align*}
G_{Cpf} &= -0.69 \\
G_{Cpf} &= -0.37 \\
G_{Cpf} &= -0.45 \\
G_{Cpi} &= 0.18 \\
G_{Cpf} &= -0.45
\end{align*}
\]
Determine Wind Load Reactions

Reactions at the top of the bearing wall are determined by summing overturning moments about the top of leeward wall for both load cases and determining the controlling reaction to use in the design. Horizontal projections are used in the analysis.

Note: The component of the overturning moment that results from wind pressures on the leeward roof overhang was not considered because: (1) it has a short (1 ft) moment arm and (2) the uplift pressures on the overhang occur downwind of the leeward wall and reduce the net overturning moment reaction slightly. This approach provides slightly conservative results.

Load Case A

\[
R_{\text{windwardA}} := \frac{1}{W} \left[ W_{\text{ovhg}} \left( W + \frac{W_{\text{ovhg}}}{2} \right) \left( W_{\text{RA}} \right) \ldots \right] + \left[ \frac{W}{2} \cdot \frac{3 \cdot W}{4} \left( W_{\text{RA}} \right) + \frac{W}{2} \cdot \frac{1 \cdot W}{4} \left( L_{\text{RA}} \right) \right]
\]

\[
R_{\text{windwardA}} = 62 \text{ plf}
\]

Load Case B

\[
R_{\text{windwardB}} := \frac{1}{W} \left[ W_{\text{ovhg}} \left( W + \frac{W_{\text{ovhg}}}{2} \right) \left( W_{\text{RB}} \right) \ldots \right] + \left[ \frac{W}{2} \cdot \frac{3 \cdot W}{4} \left( W_{\text{RB}} \right) + \frac{W}{2} \cdot \frac{1 \cdot W}{4} \left( L_{\text{RB}} \right) \right]
\]

\[
R_{\text{windwardB}} = -329 \text{ plf}
\]

Note: The uplift reaction on the windward wall can be determined from WFCM Table 2.2A by interpolating the uplift connection loads between the 24 and 36 foot roof spans for the 0 psf roof/ceiling dead load and multiplying the uplift by 0.75 to account for the wall framing not being located in an exterior zone (footnote 1). That approach produces an uplift reaction of 391 plf which is higher than the results of these calculations. The higher reactions result primarily because uplift values in Table 2.2A are based on the worst case (20°) roof slope. The velocity pressure being calculated at 33 ft instead of 25 ft also contributes to slightly higher values.
Determine MWFRS Wind Pressures on Walls for Load Cases A and B

External and internal pressure coefficients (GC_pW and GC_pi) are from ASCE 7-10 Figure 28.4-1 and Table 26.11-1 (ASCE 7-16 Figure 28.3-1 and Table 26.13-1).

Load Case A - wind perpendicular to ridge

<table>
<thead>
<tr>
<th>Zone 1 (windward)</th>
<th>Zone 4 (leeward)</th>
<th>Internal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Case A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>GC_pW := 0.56</td>
<td>GC_p := -0.37</td>
<td>GC_pi := -0.18</td>
</tr>
<tr>
<td>windward wall</td>
<td></td>
<td></td>
</tr>
<tr>
<td>p_mwfrsW := q_h × (GC_pW - GC_pi)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>p_mwfrsW = 17.3 psf</td>
<td></td>
<td></td>
</tr>
<tr>
<td>leeward wall</td>
<td></td>
<td></td>
</tr>
<tr>
<td>p_mwfrsL := q_h × (GC_pL - GC_pi)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>p_mwfrsL = -4.4 psf</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Load Case B - wind parallel to ridge

<table>
<thead>
<tr>
<th>Zone 1 (windward)</th>
<th>Zone 4 (leeward)</th>
<th>Internal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Case B</td>
<td></td>
<td></td>
</tr>
<tr>
<td>GC_pW := -0.45</td>
<td>GC_p := -0.45</td>
<td>GC_pi := 0.18</td>
</tr>
<tr>
<td>windward wall</td>
<td></td>
<td></td>
</tr>
<tr>
<td>p_mwfrsW := q_h × (GC_pW - GC_pi)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>p_mwfrsW = -14.7 psf</td>
<td></td>
<td></td>
</tr>
<tr>
<td>leeward wall</td>
<td></td>
<td></td>
</tr>
<tr>
<td>p_mwfrsL := q_h × (GC_pL - GC_pi)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>p_mwfrsL = -14.7 psf</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Determine the Distributed Loads Supported by the Bearing Wall for ASCE 7 ASD Load Combinations

ASCE 7 (section 2.4.1) includes the following ASD load combinations. Respective NDS load duration factors are shown in [brackets] next to load combination.

1. \( D \) \([C_D = 0.9]\)
2. \( D + L \) \([C_D = 1.0]\)
3. \( D + (L_r \text{ or } S \text{ or } R) \)
   3a. \( D + (L_r) \) \([C_D = 1.25]\)
   3b. \( D + (S) \) \([C_D = 1.15]\)
4. \( D + 0.75L + 0.75(L_r \text{ or } S \text{ or } R) \)
   4a. \( D + 0.75L + 0.75(L_r) \) \([C_D = 1.25]\)
   4b. \( D + 0.75L + 0.75(S) \) \([C_D = 1.15]\)
5. \( D + (0.6W \text{ or } 0.7E) \)
   5a. \( D + (0.6W \text{ or } 0.7E) \) (wind parallel to ridge) \([C_D = 1.6]\)
   5b. \( D + (0.6W \text{ or } 0.7E) \) (wind perpendicular to ridge) \([C_D = 1.6]\)
6. \( D + 0.75L + 0.75(0.6W \text{ or } 0.7E) + 0.75(L_r \text{ or } S \text{ or } R) \)
   6a1. \( D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S) \) (wind parallel to ridge) \([C_D = 1.6]\)
   6a2. \( D + 0.75L + 0.75(0.6W) + 0.75(L_r \text{ or } S) \) (wind perpendicular to ridge) \([C_D = 1.6]\)
   6b. \( D + 0.75L + 0.75(0.7E) + 0.75S \) \([C_D = 1.6]\)
7. \( 0.6D + 0.6W \)
   7a. \( 0.6D + 0.6W \) (wind parallel to ridge) \([C_D = 1.6]\)
   7b. \( 0.6D + 0.6W \) (wind perpendicular to ridge) \([C_D = 1.6]\)
8. \( 0.6D + 0.7E \) \([C_D = 1.6]\)

where

- \( D \) = dead load
- \( L \) = live load
- \( L_r \) = roof live load
- \( W \) = wind load (note the 0.6 load factor has already been included in the velocity pressure \( q_h \))
- \( S \) = snow load
- \( R \) = rain load
- \( E \) = earthquake load

Load combination are included in the following array (note that rain and earthquake load are neglected \( E_1 = R = 0 \))
Load Combinations 1, 2, 3, 4a, 4b, 6b, and 8 model gravity only loads (dead load, live load and/or snow load). Load Combinations 5, 6a1, 6a2, 7a, and 7b include MWFRS wind loads. Load Combinations are keyed to the array as follows:

<table>
<thead>
<tr>
<th>Load Combo</th>
<th>P_{dist}</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>3a</td>
<td>3</td>
</tr>
<tr>
<td>3b</td>
<td>4</td>
</tr>
<tr>
<td>4a</td>
<td>5</td>
</tr>
<tr>
<td>4b</td>
<td>6</td>
</tr>
<tr>
<td>5a</td>
<td>7</td>
</tr>
<tr>
<td>5b</td>
<td>8</td>
</tr>
<tr>
<td>6a1</td>
<td>9</td>
</tr>
<tr>
<td>6a2</td>
<td>10</td>
</tr>
<tr>
<td>6b</td>
<td>11</td>
</tr>
<tr>
<td>7a</td>
<td>12</td>
</tr>
<tr>
<td>7b</td>
<td>13</td>
</tr>
<tr>
<td>8</td>
<td>14</td>
</tr>
</tbody>
</table>

All combinations that include wind will use a 1.6 load duration factor. Since load combinations 1-4, 6b (which is identical to 4), and 8 each have different load duration factors, those combinations will be analyzed individually.
Analyze Framing for Load Combinations 1-4 and 6b
(compute actual and allowable stresses and deflections. Iterate material properties to develop design)

The bearing walls must resist distributed loads from the attic floor and roof and out-of-plane MWFRS wind loads proportional to the width of their tributary areas. The analyses are conducted for 16 inch stud spacing. The number of studs on either side of the framed openings in the south wall shall be determined from the WFCM Table 3.23C. Reductions allowed by WFCM Section 3.4.1.4.2 and Table 3.23D are acceptable.

By inspection, Load Combination 1 controls over Load Combination 8, so Load Combination 8 will not be analyzed.

**Load Combination 1: D**

**Determining compression force in framing for load combination 1**

\[ P_1 := \frac{(16)}{12} \cdot \text{ft} \cdot (P_{\text{dist1}}) \quad P_1 = 400\text{-lbf} \]

Compression force in the framing on each side of the wall openings for Load Combination 1.

**Calculate Reference & Adjusted Compression Design Values for Load Combination 1**

\[ C_{D1} := 0.9 \]

Dead load duration factor \( C_D \) for Load Combination 1. NDS Appendix B Section B.2

\[ F_{c1*} := F_c \cdot C_{D1} \cdot C_M \cdot C_t \cdot C_F \cdot C_i \]

\( F_{c1*} = 1215\text{ psi} \)

\( F_{c1*} \) is reference compression design value for Load Combination 1 adjusted with all adjustment factors except the column stability factor \( C_P \)

\[ K_e := 1.0 \]

Buckling length coefficient \( K_e \) for strong axis bending (pinned/pinned) column (Appendix G Table G1)

\[ d_1 := d \quad l_1 := L \quad l_e := K_e \cdot l_1 \cdot \left( \frac{12 \text{ in}}{\text{ft}} \right) \]

Stud dimensions, laterally unsupported lengths (NDS Figure 3F) and effective column lengths (NDS 3.7.1) for buckling in each direction. Subscript 1 is strong (but laterally unsupported) axis;

\[ l_e = 228\text{-in} \]

Effective length in the strong axis. Assume gypsum wallboard is adequately connected to the studs and provides lateral support (NDS A.11.3)

\[ F_{cE} := \frac{0.822 \cdot E'_{\text{min}}}{\left( \frac{l_e}{d_1} \right)^2} \quad F_{cE} = 424\text{ psi} \]

Critical buckling design value for compression member

**Determine Column Stability Factor \( C_p \) for Load Combination 1**

\[ C_{P1} := \frac{1}{2 \cdot c} \left[ \left( \frac{F_{cE}}{F_{c1*}} \right) + \sqrt{\left[ 1 + \left( \frac{F_{cE}}{F_{c1*}} \right) \right]^2 - \left( \frac{F_{cE}}{F_{c1*}} \right)} \right] \]

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\[ C_{P1} = 0.319 \] Column Stability Factor for Load Combination 1 (NDS 3.7-1)

**Compare Actual Compression Stress to Adjusted Compression Design Value**

\[ F_{c1}' := F_{c1} \cdot C_{P1} \quad F_{c1}' = 388 \text{-psi} \] Adjusted compression design value

\[ f_{c1} := \frac{P_1}{A_g} \quad f_{c1} = 37 \text{-psi} \] Actual compression stress

\[ \frac{f_{c1}}{F_{c1}'} = 0.09 \] Actual compression stress \( f_{c1} \) is less than the adjusted compression design value \( F_{c1}' \). Ratio of actual stress to adjusted compression design value < 1. OK

**Load Combination 2: D + L**

Determining compression force in framing for load combination 2

\[ P_2 := \frac{(16)}{12} \cdot ft \cdot (P_{\text{dist}}) \quad P_2 = 720 \text{-lbf} \] Compression force in the framing on each side of the wall openings for Load Combination 2.

**Calculate Reference & Adjusted Compression Design Values for Load Combination 2**

\[ C_{D2} := 1.0 \] Live load duration factor \( C_D \) for Load Combination 2. NDS Appendix B Section B.2

\[ F_{c2}^* := F_c \cdot C_{D2} \cdot C_M \cdot C_T \cdot C_F \cdot C_i \] \( F_{c2}^* \) is reference compression design value for Load Combination 2 adjusted with all adjustment factors except the column stability factor \( C_P \)

\[ F_{c2}^* = 1350 \text{ psi} \]

\[ K_e := 1.0 \] Buckling length coefficient \( K_e \) for strong axis bending (pinned/pinned) column (Appendix G Table G1)

\[ d_1 := d \quad l_1 := L \quad l_e := K_e \cdot l_1 \left( \frac{12 \text{-in}}{\text{ft}} \right) \] Stud dimensions, laterally unsupported lengths (NDS Figure 3F) and effective column lengths (3.7.1) for buckling in each direction. Subscript 1 is strong (but laterally unsupported) axis;

Effective length in the strong axis. Assume gypsum wallboard is adequately connected to the studs and provides lateral support (NDSA.11.3)

\[ l_e = 228 \text{-in} \]

\[ F_{cE} := \frac{0.822 \cdot E' \text{min}}{\left( \frac{l_e}{d_1} \right)^2} \quad F_{cE} = 424 \text{ psi} \] Critical buckling design value for compression member
Determine Column Stability Factor $C_p$ for Load Combination 2

\[
C_{p2} := \frac{1 + \left( \frac{F_{cE}}{F_{c2}^*} \right)}{2 \cdot c} - \sqrt{\left[ 1 + \left( \frac{F_{cE}}{F_{c2}^*} \right) \right]^2 - \left( \frac{F_{cE}}{F_{c2}^*} \right)}
\]

$C_{p2} = 0.29$  

Column Stability Factor for Load Combination 2 (NDS 3.7-1)

Compare Actual Compression Stress to Adjusted Compression Design Value

\[
F_{c2}' := F_{c2} \cdot C_{p2}
\]

Adjusted compression design value

\[
f_{c2} := \frac{P_2}{A_g}
\]

Actual compression stress

\[
\frac{f_{c2}}{F_{c2}^*} = 0.17
\]

Actual compression stress $f_{c2}$ is less than the adjusted compression design value $F_{c2}'$. Ratio of actual stress to adjusted compression design value < 1. OK

Load Combination 3a: $D + L$

Determing compression force in framing for load combination 3a

\[
P_{3a} := \frac{(16)}{12} \cdot \text{ft} \left( P_{\text{dist3}} \right)
\]

Compression force in the framing on each side of the wall openings for Load Combination 3.

Calculate Reference & Adjusted Compression Design Values for Load Combination 3a

\[
C_{D3a} := 1.25
\]

Roof live load duration factor $C_D$ for Load Combination 3a. NDS Appendix B Section B.2

\[
F_{c3a} := F_c \cdot C_{D3a} \cdot C_M \cdot C_t \cdot C_F \cdot C_i
\]

$F_{c3a}^*$ is reference compression design value for Load Combination 3a adjusted with all adjustment factors except the column stability factor $C_p$

\[
F_{c3a}^* = 1688 \text{ psi}
\]

Buckling length coefficient $K_e$ for strong axis bending (pinned/pinned) column (Appendix G Table G1)

\[
d_1 := d \quad l_1 := L \quad l_e := K_e \cdot 11 \left( \frac{12 \text{ in}}{\text{ft}} \right)
\]

Stud dimensions, laterally unsupported lengths (NDS Figure 3F) and effective column lengths (NDS 3.7.1) for buckling in each direction. Subscript 1 is strong (but laterally unsupported) axis;
\[ l_e = 228 \text{ in} \]

Effective length in the strong axis. Assume gypsum wallboard is adequately connected to the studs and provides lateral support (NDS A.11.3)

\[ F_{cE} := \frac{0.822 \cdot E' \cdot \text{min}}{l_e^2} \]

\[ F_{cE} = 424 \text{ psi} \]

Critical bucking design value for compression member

**Determine Column Stability Factor \( C_p \) for Load Combination 3a**

\[
C_{P3a} := \frac{1 + \left( \frac{F_{cE}}{F_{c3a}^*} \right)}{2 \cdot c} - \sqrt{\left( 1 + \left( \frac{F_{cE}}{F_{c3a}^*} \right) \right)^2 - \frac{F_{cE}}{F_{c3a}^*}}
\]

\[ C_{P3a} = 0.237 \]

Column Stability Factor for Load Combination 3a (NDS 3.7-1)

\[ F_{c3a} := F_{c3a}^* \cdot C_{P3a} \quad F_{c3a}' = 399 \text{ psi} \]

**Compare Actual Compression Stress to Adjusted Compression Design Value**

\[ f_{c3a} := \frac{P_{3a}}{A_g} \quad f_{c3a} = 81 \text{ psi} \]

Actual compression stress

\[ \frac{f_{c3a}}{F_{c3a}'} = 0.2 \]

Actual compression stress \( f_{c3a} \) is less than the adjusted compression design value \( F_{c3a}' \). Ratio of actual stress to adjusted compression design value < 1. OK

**Load Combination 3b: D + S**

Determining compression force in framing for load combination 3b

\[ P_{3b} := \frac{(16)}{12} \cdot \text{ft} \cdot \left( \frac{P_{\text{dist4}}}{12} \right) \quad P_{3b} = 940 \text{ lbf} \]

Compression force in the framing on each side of the wall openings for Load Combination 3b.

**Calculate Reference & Adjusted Compression Design Values for Load Combination 3b**

\[ C_{D3b} := 1.15 \]

Snow load duration factor \( C_D \) for Load Combination 3b.

NDS Appendix B Section B.2

\[ F_{c3b}^* := F_c \cdot C_{D3b} \cdot C_M \cdot C_t \cdot C_F \cdot C_i \]

\[ F_{c3b}^* = 1552 \text{ psi} \]

\[ K_e := 1.0 \]

Buckling length coefficient \( K_e \) for strong axis bending (pinned/pinned) column (Appendix G Table G1)
\[ d_1 := d \quad l_1 := L \quad l_e := K_e \cdot l_1 \left( \frac{12 \text{ in}}{\text{ft}} \right) \]

Stud dimensions, laterally unsupported lengths (NDS Figure 3F) and effective column lengths (3.7.1) for buckling in each direction. Subscript 1 is strong (but laterally unsupported) axis;

\[ l_e = 228 \text{ in} \]

Effective length in the strong axis. Assume gypsum wallboard is adequately connected to the studs and provides lateral support (NDS A.11.3)

\[ F_{cE} := \frac{0.822 \cdot E' \min}{l_e^2} \quad F_{cE} = 424 \text{ psi} \]

Critical bucking design value for compression member

\[ \text{Determine Column Stability Factor } C_p \text{ for Load Combination 3b} \]

\[ C_{P3b} := \frac{1 + \left( \frac{F_{cE}}{F_{c3b}^*} \right)}{2 \cdot c} - \sqrt{\left[ 1 + \left( \frac{F_{cE}}{F_{c3b}^*} \right) \right]^2 - c} \]

\[ C_{P3b} = 0.255 \quad \text{(Column Stability Factor for Load Combination 3b (NDS 3.7-1))} \]

\[ \text{Compare Actual Compression Stress to Adjusted Compression Design Value} \]

\[ F_{c3b} := F_{c3b}^* \cdot C_{P3b} \quad F_{c3b} = 397 \text{ psi} \quad \text{(Adjusted compression design value)} \]

\[ f_{c3b} := \frac{P_{3b}}{A_g} \quad f_{c3b} = 86 \text{ psi} \quad \text{(Actual compression stress)} \]

\[ \frac{f_{c3b}}{F_{c3b}^*} = 0.22 \quad \text{(Actual compression stress } f_{c3b} \text{ is less than the adjusted compression design value } F_{c3b}^* \text{. Ratio of actual stress to adjusted compression design value } < 1 \text{. OK)} \]

\[ \text{Load Combination 4a: } D + 0.75 L + 0.75 L_r \]

\[ \text{Determining compression force in framing for load combination 4a} \]

\[ P_{4a} := \frac{(16)}{12} \cdot \text{ft} \cdot P_{\text{dist}5} \quad P_{4a} = 1000 \text{-lbf} \quad \text{(Adjusted compression design value)} \]

\[ \text{Calculate Reference & Adjusted Compression Design Values for Load Combination 4a} \]

\[ C_{D4a} := 1.25 \quad \text{roof live load duration factor } C_D \text{ for Load Combination 4a. NDS Appendix B Section B.2} \]
\[ F_{c4a}^* := F_c \cdot C_{D4a} \cdot C_{M} \cdot C_i \cdot C_f \cdot C_i \]

\[ F_{c4a}^* = 1688 \text{ psi} \]

\[ K_e := 1.0 \]

\[ d_1 := d \quad l_1 := L \quad l_e := K_e \cdot l_1 \left( \frac{12 \text{ in}}{\text{ft}} \right) \]

\[ l_e = 228 \cdot \text{in} \]

\[ F_{cE} := \frac{0.822 \cdot E_{min}}{d_1^2} \quad F_{cE} = 424 \text{ psi} \]

Critical buckling design value for compression member

Determine Column Stability Factor \( C_p \) for Load Combination 4a

\[ C_{P4a} := \frac{1 + \left( \frac{F_{cE}}{F_{c4a}^*} \right)}{2 \cdot c} - \sqrt{\left[ 1 + \left( \frac{F_{cE}}{F_{c4a}^*} \right) \right]^2 - \left( \frac{F_{cE}}{F_{c4a}^*} \right)} \]

\[ C_{P4a} = 0.237 \]

Column Stability Factor for Load Combination 4a (NDS 3.7-1)

Compare Actual Compression Stress to Adjusted Compression Design Value

\[ F_{c4a}' := F_{c4a}^* \cdot C_{P4a} \quad F_{c4a}' = 399 \cdot \text{psi} \]

Adjusted compression design value

\[ f_{c4a} := \frac{P_{4a}}{A_g} \quad f_{c4a} = 92 \cdot \text{psi} \]

Actual compression stress

\[ \frac{f_{c4a}}{F_{c4a}'} = 0.23 \]

Actual compression stress \( f_{c4a} \) is less than the adjusted compression design value \( F_{c4a}' \). Ratio of actual stress to adjusted compression design value < 1. OK
Load Combination 4b: D + 0.75 L + 0.75 S

Determining compression force in framing for load combination 4b

\[ P_{4b} := \frac{(16)}{12} \cdot \text{ft} \left( P_{\text{dist},6} \right) \quad P_{4b} = 1045\text{-lbf} \]

Compression force in the framing on each side of the wall openings for Load Combination 4b.

Calculate Reference & Adjusted Compression Design Values for Load Combination 4b

\[ C_{D4b} := 1.15 \]

Snow load duration factor \( C_D \) for Load Combination 4b.  
NDS Appendix B Section B.2

\[ F_{c4b*} := F_c \cdot C_{D4b} \cdot C_M \cdot C_t \cdot C_F \cdot C_i \]

\( F_{c4b*} \) is reference compression design value for Load Combination 4b adjusted with all adjustment factors except the column stability factor \( C_p \)

\[ F_{c4b*} = 1552 \text{ psi} \]

Buckling length coefficient \( K_e \) for strong axis bending (pinned/pinned) column (Appendix G Table G1)

\[ d_1 := d \quad l_1 := L \quad l_e := K_e \cdot l_1 \left( \frac{12}{\text{in}} \right) \]

Stud dimensions, laterally unsupported lengths (NDS Figure 3F) and effective column lengths (3.7.1) for buckling in each direction.  Subscript 1 is strong (but laterally unsupported) axis;

\[ l_e = 228\text{-in} \]

Effective length in the strong axis.  Assume gypsum wallboard is adequately connected to the studs and provides lateral support (NDS A.11.3)

\[ F_{cE} := \frac{0.822 \cdot E' \cdot \min \left( \frac{l_e}{d} \right)}{2} \quad F_{cE} = 424 \text{ psi} \]

Critical buckling design value for compression member

Determine Column Stability Factor \( C_p \) for Load Combination 4b

\[
C_{p4b} := \frac{1 + \left( \frac{F_{cE}}{F_{c4b*}} \right)}{2 \cdot c} - \sqrt{\left[ 1 + \left( \frac{F_{cE}}{F_{c4b*}} \right) \right]^2 - \left( \frac{F_{cE}}{F_{c4b*}} \right)}
\]

\[ C_{p4b} = 0.255 \]

Column Stability Factor for Load Combination 4b (NDS 3.7-1)

Compare Actual Compression Stress to Adjusted Compression Design Value

\[ F_{c4b} := F_{c4b*} \cdot C_{p4b} \quad F_{c4b} = 397 \text{ psi} \]

Adjusted compression design value

\[ f_{c4b} := \frac{P_{4b}}{A_g} \quad f_{c4b} = 96 \text{ psi} \]

Actual compression stress

\[ \frac{f_{c4b}}{F_{c4b}} = 0.24 \]

Actual compression stress \( f_{c4b} \) is less than the adjusted compression design value \( F_{c4b} \).  Ratio of actual stress to adjusted compression design value < 1. OK
Load Combination | Applied Stress/Allowable Stress
--- | ---
1 | 0.09
2 | 0.17
3a | 0.20
3b | 0.22
4a | 0.23
4b | 0.24

Table comparing ratio of applied stress to allowable stresses for gravity load controlled combinations.

**Load Case 5a: D + 0.6 W (wind parallel to ridge)**

Determining compression load in framing for load combination 5a

\[ P_{5a} := \frac{(16)}{12} \cdot \left( P_{\text{dist},7} \right) \quad P_{5a} = 483 \text{ lbf} \]

Determine Reference and Adjusted Compression Design Values for Load Combination 5a

\[ C_{D5} := 1.6 \quad \text{Wind load duration factor } C_D \text{ controls for Load Combination 5a - NDS Appendix B Section B.2} \]

\[ F_{c5a} := F_c \cdot C_{D5} \cdot C_M \cdot C_t \cdot C_F \cdot C_i \quad F_{c5a} = 2160 \text{ psi} \]

Determine Column Stability Factor \( C_p \) for Load Combination 5a

\[ C_{P5} := \left( \frac{1 + \left( \frac{F_cE}{F_{c5a}} \right)}{2 \cdot c} \right) - \sqrt{\left( \frac{1 + \left( \frac{F_cE}{F_{c5a}} \right)}{2 \cdot c} \right)^2 - \frac{\left( \frac{F_cE}{F_{c5a}} \right)^2}{c}} \]

\[ C_{P5} = 0.188 \quad \text{Column Stability Factor for Load Combination 5a (NDS 3.7-1)} \]

Determine Adjusted Compression Design Value for Load Combination 5a

\[ F_{c5a} := F_{c5a} \cdot C_{P5} \quad F_{c5a} = 405 \text{ psi} \quad \text{Adjusted compression design value for Load Combination 5a} \]

Compare Actual Compression Stress with Adjusted Compression Design Value

\[ f_{c5a} := \frac{P_{5a}}{A_g} \quad f_{c5a} = 44 \text{ psi} \]

\[ F_{c5a} = 405 \text{ psi} \]

\[ \frac{f_{c5a}}{F_{c5a}} = 0.11 \quad \text{Actual compression stress } f_{c5a} \text{ does not exceed adjusted compression design value } F_{c5a}. \text{ Ratio of actual compression stress to adjusted compression design value } \frac{f_{c5a}}{F_{c5a}} < 1. \text{ OK} \]
**Determine Bending Stress from Out-of-Plane MWFRS Wind Pressures**

**Moment**

\[
\frac{w_{\text{windAWW}}}{12} = \frac{16}{12} \cdot \text{ft} \cdot p_{\text{mwfrsAWW}}
\]

\[w_{\text{windAWW}} = 23.08 \cdot \text{plf}\]

\[
M_{\text{mwfrsAWW}} := \frac{w_{\text{windAWW}} \cdot L^2}{8} \cdot 12 \cdot \text{in} \cdot \text{ft}
\]

\[M_{\text{mwfrsAWW}} = 12500 \cdot \text{in} \cdot \text{lbf}\]

**Determine Reference and Adjusted Bending Design Values for Load Combination 5a**

\[C_L := 1.0\]

Depth to breadth (d/b) ratio 2<d/b<4  End restraints for the beam-column satisfy NDS 4.4.1.2 (b) and sheathing/gypsum wall board nailing provides lateral support for the compression edges NDS 4.4.1.2 (c)

\[F'_{b5a} := F_b \cdot D^5 \cdot C_M \cdot C_T \cdot C_i \cdot C_r\]

\[F'_{b5a} = 1850 \text{ psi}\]

**Compare Actual Bending Stress with Adjusted Bending Design Values for Load Combination 5a**

\[f_{b5a} := \frac{M_{\text{mwfrsAWW}}}{S_x}\]

\[f_{b5a} = 951 \text{ psi}\]

\[F'_{b5a} = 1850 \text{ psi}\]

\[\frac{f_{b5a}}{F'_{b5a}} = 0.51\]

Bending stress resulting from out-of-plane MWFRS wind loads

Ok. Actual bending stress \(f_{b5a}\) is less than adjusted bending design value \(F'_{b5a}\). Ratio of actual bending stress to adjusted bending design value < 1

**Check Combined Uniaxial Bending and Axial Compression**

\[
\left(\frac{f_{c5a}}{F_{c5a}}\right)^2 + \frac{f_{b5a}}{F'_{b5a}} \left[1 - \left(\frac{f_{c5a}}{F_{cE}}\right)\right] = 0.59 < 1.0 \text{ ok (NDS 3.9-3)}\]

\[f_{c5a} = 44 \text{ psi} \quad F_{cE} = 424 \text{ psi}\]

Actual compression stress \(f_{c5a}\) does not exceed adjusted compression design value in plane of lateral support for edgewise bending \(F_{cE}\). OK
Load Case 5b: D + 0.6 W (wind perpendicular to ridge)
Determining axial load in framing for load combination 5b

\[ P_{5b} = \left( \frac{16}{12} \right) \cdot \left( P_{\text{dist}} \right) \Rightarrow P_{5b} = -39 \text{ lbf} \]

Note: by inspection, Load Combination 7b controls and is analyzed below.

Load Case 6a1: D + 0.75L + 0.75 W (wind parallel to ridge)
Determining compression load in framing for load combination 6a1

\[ P_{6a1} = \left( \frac{16}{12} \right) \cdot \left( P_{\text{dist}} \right) \Rightarrow P_{6a1} = 1107 \text{ lbf} \]

Determine Reference and Adjusted Compression Design Values for Load Combination 6a1

\[ C_{D6} = 1.6 \]

Wind load duration factor \( C_D \) controls for Load Combination 6a - NDS Appendix B Section B.2

\[ F_{c6a1} = F_c \cdot C_{D6} \cdot C_M \cdot C_I \cdot C_F \cdot C_i \]

\[ F_{c6a1} = 2160 \text{ psi} \]

Determine Column Stability Factor \( C_p \) for Load Combination 6a1

\[ C_p = 0.188 \]

Column Stability Factor for Load Combination 6a1 (NDS 3.7-1)

Determine Adjusted Compression Design Value for Load Combination 6a1

\[ F_{c6a1} = F_{c6a1} \cdot C_p \]

\[ F_{c6a1} = 405 \text{ psi} \]

Adjusted compression design value for Load Combination 6a

Compare Actual Compression Stress with Adjusted Compression Design Value

\[ f_{c6a1} = \frac{P_{6a1}}{A_g} \]

\[ f_{c6a1} = 102 \text{ psi} \]

Actual compression stress \( f_{c6a1} \) does not exceed adjusted compression design value \( F_{c6a1} \). Ratio of actual compression stress to adjusted compression design value < 1. OK

\[ \frac{f_{c6a1}}{F_{c6a1}} = 0.25 \]
Determine Bending Stress from Out-of-Plane MWFRS Wind Pressures

**Moment**

\[ w_{\text{windAWW}} := 0.75 \frac{16}{12} \text{ft} \cdot p_{\text{mwfrsAWW}} \]

\[ w_{\text{windAWW}} = 17.31 \text{ plf} \]

\[ M_{\text{mwfrsAWW}} := \frac{w_{\text{windAWW}} \cdot L^2}{8} \cdot \frac{12 \cdot \text{in}}{\text{ft}} \]

\[ M_{\text{mwfrsAWW}} = 9375 \text{ in}\cdot\text{lbf} \]

**Determine Reference and Adjusted Bending Design Values for Load Combination 6a1**

\[ C_L := 1.0 \]

Depth to breadth (d/b) ratio 2<d/b<4  End restraints for the beam-column satisfy NDS 4.4.1.2 (b) and sheathing/gypsum wall board nailing provides lateral support for the compression edges NDS 4.4.1.2 (c)

\[ F'_{b6a1} := F_b \cdot C_D \cdot C_M \cdot C_L \cdot C_t \cdot C_F \cdot C_i \cdot C_r \]

\[ F'_{b6a1} = 1850 \text{ psi} \]

**Compare Actual Bending Stress with Adjusted Bending Design Values for Load Combination 6a1**

\[ f_{b6a1} := \frac{M_{\text{mwfrsAWW}}}{S_x} \]

\[ f_{b6a1} = 713 \text{ psi} \]

\[ F'_{b6a1} = 1850 \text{ psi} \]

\[ \frac{f_{b6a1}}{F'_{b6a1}} = 0.39 \]

Bending stress resulting from out-of-plane MWFRS wind loads

Ok. Actual bending stress \( f_{b6a1} \) is less than adjusted bending design value \( F'_{b6a1} \). Ratio of actual bending stress to adjusted bending design value < 1

**Check Combined Uniaxial Bending and Axial Compression**

\[ \left( \frac{f_{c6a1}}{F_{c6a1}} \right)^2 + \frac{f_{b6a1}}{F'_{b6a1} \left[ 1 - \left( \frac{f_{c6a1}}{F_{cE}} \right) \right]} = 0.57 \]

\[ \left( \frac{102}{424} \right)^2 + \frac{713}{1850 \left[ 1 - \left( \frac{102}{424} \right) \right]} = 0.57 \]

\[ f_{c6a1} = 102 \text{ psi} \quad F_{cE} = 424 \text{ psi} \]

Actual compression stress \( f_{c6a1} \) does not exceed adjusted compression design value in plane of lateral support for edgewise bending \( F_{cE} \). OK
Load Case 6a2: D + 0.75L + 0.75 W (wind perpendicular to ridge)

Determining compression load in framing for load combination 6a2

\[ P_{6a2} := \frac{(16)}{12} \cdot \frac{p_{\text{dist}}}{P_{\text{dist}}_{10}} \quad P_{6a2} = 716 \text{-lbf} \]

Determine Reference and Adjusted Compression Design Values for Load Combination 6a2

\[ C_{D6} := 1.6 \quad \text{Wind load duration factor } C_D \text{ controls for Load Combination 6a - NDS Appendix B Section B.2} \]

\[ F_{c6a2^*} := F_c \cdot C_{D6} \cdot C_M \cdot C_t \cdot C_F \cdot C_i \quad F_{c6a2^*} \text{ is reference compression design value adjusted with all adjustment factors except the column stability factor } C_P. \]

\[ F_{c6a2^*} = 2160 \text{ psi} \]

Determine Column Stability Factor \( C_p \) for Load Combination 6a2

\[ C_P_{6} := \frac{1 + \left( \frac{F_c E}{F_{c6a2^*}} \right)}{2 \cdot c} - \sqrt{\left[ 1 + \left( \frac{F_c E}{F_{c6a2^*}} \right) \right]^2 - \left( \frac{F_c E}{F_{c6a2^*}} \right)} \]

\[ C_P_{6} = 0.188 \quad \text{Column Stability Factor for Load Combination 6a2 (NDS 3.7-1)} \]

Determine Adjusted Compression Design Value for Load Combination 6a2

\[ F_{c'6a2} := F_{c6a2^*} \cdot C_P_{6} \quad F_{c'6a2} = 405 \text{-psi} \quad \text{Adjusted compression design value for Load Combination 6a} \]

Compare Actual Compression Stress with Adjusted Compression Design Value

\[ f_{c6a2} := \frac{P_{6a2}}{A_g} \quad f_{c6a2} = 66 \text{-psi} \quad F_{c'6a2} = 405 \text{-psi} \]

\[ \frac{f_{c6a2}}{F_{c'6a2}} = 0.16 \]

Determine Bending Stress from Out-of-Plane MWFRS Wind Pressures

**Moment**

\[ w_{\text{windBW}} := 0.75 \cdot \frac{(16)}{12} \cdot \text{ft} \cdot \text{p}_{\text{mwfrsBW}} \]

\[ w_{\text{windBW}} = -14.74 \text{-plf} \]

\[ M_{\text{mwfrsBW}} := \frac{w_{\text{windBW}} \cdot L^2}{8} \cdot 12 \cdot \text{in} \frac{\text{ft}}{\text{in}} \quad M_{\text{mwfrsBW}} = -7982 \text{-in} \cdot \text{lb} \]

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Determine Reference and Adjusted Bending Design Values for Load Combination 6a2

\[ C_L := 1.0 \]

Depth to breadth (d/b) ratio 2 < d/b < 4. End restraints for the beam-column satisfy NDS 4.4.1.2 (b) and sheathing/gypsum wall board nailing provides lateral support for the compression edges NDS 4.4.1.2 (c)

\[ F_{b6a2} := F_b \cdot C_D \cdot C_M \cdot C_{M} \cdot C_t \cdot C_F \cdot C_i \cdot C_r \]
\[ F_{b6a2} = 1850 \text{ psi} \]

\[ F_{b6a2} \] is adjusted bending design value for Load Combination 6a2

Compare Actual Bending Stress with Adjusted Bending Design Values for Load Combination 6a2

\[ f_{b6a2} := \frac{M_{mwfrsBWW}}{S_x} \]
\[ f_{b6a2} = 607 \text{ psi} \]
\[ F_{b6a2} = 1850 \text{ psi} \]

Bending stress resulting from out-of-plane MWFRS wind loads

\[ \frac{f_{b6a2}}{F_{b6a2}} = 0.33 \]

Ok. Actual bending stress \( f_{b6a2} \) is less than adjusted bending design value \( F_{b6a2} \). Ratio of actual bending stress to adjusted bending design value < 1

Check Combined Uniaxial Bending and Axial Compression

\[ \left( \frac{f_{c6a2}}{F_{c6a2}} \right)^2 + \frac{f_{b6a2}}{F_{b6a2}} \left[ 1 - \left( \frac{f_{c6a2}}{F_{cE}} \right) \right] = 0.42 \]

\[ < 1.0 \text{ ok (NDS 3.9-3)} \]

Actual compression stress \( f_{c6a2} \) does not exceed adjusted compression design value in plane of lateral support for edgewise bending \( F_{cE} \). OK

\[ f_{c6a2} = 66 \text{ psi} \]
\[ F_{cE} = 424 \text{ psi} \]

Load Case 7a: 0.6 D + 0.6 W (wind parallel to ridge)

Determining compression load in framing for load combination 7a

\[ P_{7a} := \frac{(16)}{12} \cdot \text{ft} \cdot (P_{dist12}) \]
\[ P_{7a} = 323 \text{ lbf} \]

Note: by inspection, Load Case 5a controls and calculations will not be repeated for Load Case 7a.
Load Case 7b: 0.6 D + 0.6 W (wind perpendicular to ridge)

Determining axial load in framing for load combination 7b

\[ P_{7b} = \frac{(16)}{12} \text{ ft} \left( p_{\text{dist}_{13}} \right) \]

\[ P_{7b} = -199 \cdot \text{lbf} \]

Determine Adjusted Tension Design Value for Load Combination 7b

\[ C_D 7 = 1.6 \]

Wind load duration factor \( C_D \) controls for Load Combination 7b - NDS Appendix B Section B.2

\[ F'_{t7b} = F_t \cdot C_D 7 \cdot C_M \cdot C_t \cdot C_F \cdot C_i \]

\[ F'_{t7b} \text{ is adjusted tension design value} \]

\[ F'_{t7b} = 880 \text{ psi} \]

Compare Actual Tension Stress with Adjusted Tension Design Value

\[ f_{t7b} = \frac{P_{7b}}{A_g} \]

\[ f_{t7b} = -18 \text{ psi} \]

Actual tension stress \( f_{t7b} \) does not exceed adjusted tension design value \( F'_{t7b} \). Ratio of actual tension stress to adjusted tension design value \( < 1 \). OK

\[ \frac{f_{t7b}}{F'_{t7b}} = -0.02 \]

Determine Bending Stress from Out-of-Plane MWFRS Wind Pressures

Moment

\[ w_{\text{windBW}} := \frac{(16)}{12} \cdot \text{ft} \cdot p_{\text{mwfrsBW}} \]

\[ w_{\text{windBW}} = -19.65 \cdot \text{plf} \]

\[ M_{\text{mwfrsBW}} := \frac{w_{\text{windBW}} \cdot L^2}{8} \cdot 12 \cdot \text{in} \cdot \text{lbf} \]

\[ M_{\text{mwfrsBW}} = -10642 \cdot \text{in} \cdot \text{lbf} \]

Determine Reference and Adjusted Bending Design Values for Load Combination 7b

\[ C_L := 1.0 \]

Depth to breadth (d/b) ratio \( 2 < d/b < 4 \). End restraints for the beam-column satisfy NDS 4.4.1.2 (b) and sheathing/gypsum wall board nailing provides lateral support for the compression edges NDS 4.4.1.2 (c)

\[ F'_{b7b} := F_b \cdot C_D 7 \cdot C_M \cdot C_L \cdot C_t \cdot C_F \cdot C_i \cdot C_r \]

\[ F'_{b7b} \text{ is adjusted bending design value for Load Combination 7b} \]

\[ F'_{b7b} = 1850 \text{ psi} \]
1.9 LOADBEARING WALL WOOD STUD RESISTING WIND AND GRAVITY LOADS

Compare Actual Bending Stress with Adjusted Bending Design Values for Load Combination 7b

\[ f_{b7b} := \frac{M_{mwfrsBW}}{S_x} \]

- Bending stress resulting from out-of-plane MWFRS wind loads
- \( f_{b7b} = 810 \text{ psi} \)
- \( F'_{b7b} = 1850 \text{ psi} \)

\[ \frac{f_{b7b}}{F'_{b7b}} = 0.44 \]

- Ok. Actual bending stress \( f_{b7b} \) is less than adjusted bending design value \( F'_{b7b} \). Ratio of actual bending stress to adjusted bending design value < 1

Check Combined Uniaxial Bending and Axial Tension

\[ \frac{f_{t7b}}{F'_{t7b}} + \frac{f_{b7b}}{F'_{b7b}} = 0.417 \]

- < 1.0 ok (NDS 3.9-1)

\[ \frac{f_{b7b} - f_{t7b}}{F'_{b7b}} = 0.448 \]

- < 1.0 ok (NDS 3.9-2)

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Applied Stress/ Allowable Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>5a</td>
<td>0.59</td>
</tr>
<tr>
<td>6a1</td>
<td>0.57</td>
</tr>
<tr>
<td>6a2</td>
<td>0.42</td>
</tr>
<tr>
<td>7b</td>
<td>0.45</td>
</tr>
</tbody>
</table>

Table comparing ratio of applied stress to allowable stresses for combined bending and axial load combinations. By inspection and comparison to gravity load combinations analyzed earlier, Load Combination 5a controls so far with combined dead plus MWFRS loads parallel to ridge.

Check Adequacy of Framing to Resist Components and Cladding (C&C) loads

Calculate C&C Pressures on Wall

\[ C_{d, DCC} := 1.6 \]

- \( C_D \) for C&C loading

Determine External C&C Pressure Coefficient

\[ EWA := \frac{L^2}{3} \times \frac{1}{\text{ft}^2} \]

- \( EWA = 120 \)

\[ GC_p(EWA) := -0.8 - 0.3 \cdot \left( \frac{\log \left( \frac{EWA}{500} \right)}{\log \left( \frac{10}{500} \right)} \right) \]

- External pressure coefficient for full height studs in Foyer wall

\[ GC_p(EWA) = -0.909 \]

\[ p_{CC}(EWA) := q_h \left[ GC_p(EWA) - GC_p(\text{pl}) \right] \]

- Equation for C&C pressures for framing in the Foyer wall

By observation negative external pressure coefficients (\( GC_p \)) are greater than positive external pressure coefficients. So negative external pressures and positive internal pressures (windward) create the greatest C&C pressures.
\[ p_{CC}(EWA) = -25.48 \text{ psf} \]

C & C pressures for full height framing in the south wall

**Apply C&C Pressures to Wall Framing and Check Bending and Deflection**

**Bending**

\[ w_{CC} = \frac{(16)}{12} \cdot ft \cdot p_{CC}(EWA) \quad w_{CC} = 34 \text{ plf} \]

\[ M_{CC} = \frac{\left( w_{CC} \right)^2 \cdot L^2 \cdot 12 \cdot \text{in}}{8} \]

\[ M_{CC} = 18399 \text{ in} \cdot \text{lbf} \]

**Determine Reference and Adjusted Bending Design Values for C&C loading**

\[ F'_{bCC} = F_b \cdot C_{DC} \cdot C_M \cdot C_L \cdot C_T \cdot (1.0) \cdot C_F \cdot C_i \cdot C_r \]

\[ F'_{bCC} = 1850 \text{ psi} \]

**Compare Actual Bending Stress with Adjusted Bending Design Values for C&C loading**

\[ f_{bCC} = \frac{M_{CC}}{S_x} \quad f_{bCC} = 1400 \text{ psi} \]

\[ \frac{f_{bCC}}{F'_{bCC}} = 0.76 \]

Ok. Actual bending stresses that result from C&C pressures \( f_{bCC} \) do not exceed adjusted bending design value \( F'_{bCC} \)

\[ f_{bCC} = 1850 \text{ psi} \]

The \( fb/F'b \) ratio is greater for C&C loading than for MWFRS loading. C&C controls for strength calculations.

**Determine Deflection for C&C loading**

\[ \Delta_{CC} = \frac{5}{384 \cdot C_T \cdot E \cdot I_x} \cdot \frac{0.70 \cdot w_{CC} \cdot \text{ft}}{12 \cdot \text{in}} \cdot \left( \frac{L \cdot 12 \cdot \text{in}}{\text{ft}} \right)^4 \]

\[ \Delta_{CC} = 0.84 \text{ in} \]

\[ \frac{L \cdot 12 \cdot \text{in}}{\text{ft}} = 273 \]

OK - Span to deflection ratio is greater than \( L / 180 \)

**Results** - Framing the south wall of the Foyer using No.2 SP 2x8 studs on 16 inch centers is adequate to resist ASCE 7 loads.
E2.1a - Withdrawal Design Value - Plain Shank Nail

Using 2015/2018 NDS section 12.2, calculate the Allowable Stress Design (ASD) reference withdrawal design value in pounds (capacity) of an 8d common smooth-shank nail in the connection below. Assume all adjustment factors are unity:

Main member:
Spruce-Pine-Fir Nominal 4x (Actual dimension 3.5 in.) (G = 0.42)

Side member:
12 gage (0.105 in. thick) ASTM A653 Grade 33 steel side plate

Fastener Dimensions:
8d common nail (NDS Table L4)
Length = 2.5 in.
Diameter = 0.131 in.
E2.1a - Withdrawal Design Value - Plain Shank Nail

Using 2015/2018 NDS section 12.2, calculate the Allowable Stress Design (ASD) reference withdrawal design value in pounds (capacity) of an 8d common smooth-shank nail in the connection below. Assume all adjustment factors are unity:

Main member:
Spruce-Pine-Fir Nominal 4x (Actual dimension 3.5 in.) \( G = 0.42 \)

Side member:
12 gage (0.105 in. thick) ASTM A653 Grade 33 steel side plate

Fastener Dimensions:
8d common nail (NDS Table L4)
Length = 2.5 in.
Diameter = 0.131 in.

\[
\begin{align*}
D &= 0.131 & \text{Fastener diameter (in.)} \\
G &= 0.42 & \text{Specific gravity (NDS Table 12.3.3A)} \\
L &= 2.5 & \text{Nail Length (in.)} \\
L_s &= 0.105 & \text{Side Member thickness (in.)} \\
p_t &= L - L_s & \text{Nail penetration into main member (in.)}
\end{align*}
\]

\[
W := 1380 \cdot G^\frac{5}{2} \cdot D \\
\text{NDS Equation 12.2-3}
\]

\[W = 20.7\]
Reference withdrawal design value. Compare to NDS Table 12.2C, \( W = 21 \text{ lbs/in} \)

\[W \cdot p_t = 49.5\]
Reference withdrawal design value based on nail penetration into main member (lbs)

See NDS Table 11.3.1 for application of additional adjustment factors for connections based on end use conditions.
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.
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

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

Checking Fastener Head Pull-Through

\[ t_{ns} = 0.625 \]
\[ 2.5D_H = 0.703 \quad 2.5D_H \text{ greater than } t_{ns}, \text{ so NDS Equation 12.2-6a applies} \]

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

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.
E2.1c - Fastener Uplift Capacity - Roof Sheathing Ring Shank Nail in 7/16" 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 7/16 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:
7/16 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.
E2.1c - Fastener Uplift Capacity - Roof Sheathing
Ring Shank Nail in 7/16" 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 7/16 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:
7/16 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 Fastener diameter (in.)
DH := 0.281 Fastener head diameter (in.)
TL := 1.5 Deformed Shank Length (in.)
tns := 0.4375 Net Side Member thickness (in.)
G := 0.5 Specific gravity, main and side members (NDS Table 12.3.3A)

Checking Fastener Withdrawal

W := 1800 · G² · D 
W = 59 Reference withdrawal design value. Compare to NDS Table 12.2E, W = 59 lbs/in
W · TL = 88 Reference withdrawal design value based on deformed shank fastener
penetration (TL) in main member (lbs)

Checking Fastener Head Pull-Through

tns = 0.438

2.5DH = 0.703 2.5DH greater than tns, so NDS Equation 12.2-6a applies

WH := 690 · π · DH² · G² · tns 
WH = 67 Head pull-through design value (lbs). Compare to NDS Table 12.2F, WH = 67 lbs

Fastener head pull-through design value of 67 lbs is less than withdrawal design value of 88 lbs;
fasterener head pull-through controls design capacity. See NDS Table 11.3.1 for application of
additional adjustment factors for connections based on end use conditions.
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)

\[ R_c := \frac{F_{em}}{F_{es}} \]  
\[ R_c = 1 \]

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

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

\[ \text{Tip} := 2 \cdot D \]  
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)

\[ L_m := 3 - L_s - \frac{\text{Tip}}{2} \]  
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)

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

$$R_t = 2.803$$

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

$$k_1 = 0.935$$

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

$$k_2 = 1.047$$

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

$$k_3 = 1.347$$

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

**Mode $I_m$**

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

$$Z_{I_m} = 658$$

Yield Mode $I_m$ Solution (lbs)

**Mode $I_s$**

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

$$Z_{I_s} = 235$$

Yield Mode $I_s$ Solution (lbs)
Mode II

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

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

Mode III_m

\[ Z_{III_m} := \frac{k_2 \cdot D \cdot L_m \cdot F_{em}}{(1 + 2 \cdot R_c) \cdot R_d} \]

\[ Z_{III_m} = 230 \quad \text{Yield Mode III_m Solution (lbs)} \]

Mode III_s

\[ Z_{III_s} := \frac{k_3 \cdot D \cdot L_s \cdot F_{em}}{(2 + R_c) \cdot R_d} \]

\[ Z_{III_s} = 105 \quad \text{Yield Mode III_s Solution (lbs)} \]

Mode IV

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

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

\[ Z_{\text{dist}} := \begin{pmatrix} Z_{I_m} \\ Z_{I_s} \\ Z_{II} \\ Z_{III_m} \\ Z_{III_s} \\ Z_{IV} \end{pmatrix} = \begin{pmatrix} 658 \\ 235 \\ 219 \\ 230 \\ 105 \\ 118 \end{pmatrix} \]

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

\[ Z = 105 \]

Minimum value of all Yield Modes provides Z-reference lateral design value (lbs). Mode III_s 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.
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". 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.

\[
W := 1800 \cdot G^2 \cdot D^4 \quad \text{NDS Equation 12.2-1}
\]
\[
W = 436.6
\]

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

\[
W \cdot p_t = 955 \quad \text{Withdrawal design value based on main member penetration (lbs)}
\]

See NDS Table 11.3.1 for application of additional adjustment factors for connections based on end use conditions.
**E2.4 - Single Wood Screw Lateral Design Value - Double Shear Wood-to-wood Connection**

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

Main member
Actual 3 in. Structural Composite Lumber Member (G = 0.5) (NDS 12.3.3.3)

Side members
Nominal 2x Douglas Fir-Larch (DFL) (Actual thickness = 1.5 in.) (G = 0.5) (NDS 12.3.3A)

Fastener Dimensions:
Number 10 Wood Screw (NDS Table L3)
D = 0.19 in.
D, = 0.152 in.
Length = 6 in.
E2.4 - Single Wood Screw Lateral Design Value - Double Shear Wood-to-wood Connection

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

Main member
Actual 3 in. Structural Composite Lumber Member (G = 0.5) (NDS 12.3.3.3)

Side members
Nominal 2x Douglas Fir-Larch (DFL) (Actual thickness = 1.5 in.) (G = 0.5) (NDS 12.3.3A)

Fastener Dimensions:
Number 10 Wood Screw (NDS Table L3)
D = 0.19 in.
Dr = 0.152 in.
Length = 6 in.

Define parameters:
\[ F_{em} := 5550 \] Main member Dowel Bearing Strength (NDS Table 12.3.3) (psi)
\[ F_{es} := 5550 \] Side member Dowel Bearing Strength (NDS Table 12.3.3) (psi)
\[ R_c := \frac{F_{em}}{F_{es}} \] \[ R_c = 1 \]
\[ t_m := 3.0 \] Main Member thickness (in.)
\[ t_s := 1.5 \] Side Member thickness (in.)
\[ F_{yb} := 80000 \] Fastener dowel bending yield strength (psi) (NDS Table I1)
\[ D := 0.19 \] Screw Diameter (in.)
\[ D_r := 0.152 \] Screw Root Diameter (in.)
\[ L_{screw} := 6 \] Screw Length (in.)
\[ Tip := 2 \cdot D \] Length of tapered fastener tip (in.) (NDS 12.3.5.3b)
\[ L_m := t_m \] \[ L_m = 3 \] Main member Dowel Bearing Length (in.) (NDS 12.3.5.3)
\[ L_s := L_{screw} - L_m - t_s - \frac{Tip}{2} \] Side member (holding the screw tip) Dowel Bearing Length (in.) (NDS 12.3.5)
\[ L_s = 1.31 \] NDS 12.1.5.6 requires minimum 6D penetration, \( L_s > 1.14 \) in. OK
\[ R_d := 10D + 0.5 \] \[ R_d = 2.4 \] Reduction Term (NDS Table 12.3.1B)
Calculate \( k_3 \) (NDS Table 12.3.1A) \((k_2 \text{ and } k_4 \text{ not used})\)

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

\[k_3 = 1.095\]

Yield Mode Calculations (NDS Table 12.3.1A)

**Mode I_m**

\[
Z_{I_m} := \frac{D_r \cdot L_m \cdot F_{em}}{R_d}
\]

\[Z_{I_m} = 1055 \quad \text{Yield Mode } I_m \text{ Solution (lbs)}\]

**Mode I_s**

\[
Z_{I_s} := \frac{2D_r \cdot L_s \cdot F_{es}}{R_d}
\]

\[Z_{I_s} = 921 \quad \text{Yield Mode } I_s \text{ Solution (lbs)}\]

**Mode III_s**

\[
Z_{III_s} := \frac{2k_3 \cdot D_r \cdot L_s \cdot F_{em}}{(2 + R_e) R_d}
\]

\[Z_{III_s} = 336 \quad \text{Yield Mode } III_s \text{ Solution (lbs)}\]

**Mode IV**

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

\[Z_{IV} = 234 \quad \text{Yield Mode IV Solution (lbs)}\]
Creating an array with all Yield Mode Solutions

\[ Z_{\text{dist}} := \begin{pmatrix} Z_{\text{Im}} \\ Z_{\text{Is}} \\ Z_{\text{Ils}} \\ Z_{\text{IV}} \end{pmatrix} \quad Z_{\text{dist}} = \begin{pmatrix} 1055 \\ 921 \\ 336 \\ 234 \end{pmatrix} \]

\[ Z := \min(Z_{\text{dist}}) \]

\[ Z = 234 \]

Minimum value of all Yield Modes provides Z-reference lateral design value (lbs). Mode IV controls.

There are no tabulated values in the NDS to compare. Note that a single shear wood screw design value in NDS Table 12L is 117 lbs, which is half the value calculated here, since it is also Mode IV controlled. See NDS Table 11.3.1 for application of additional adjustment factors for connections based on end use conditions.
E2.5 - Single Bolt Lateral Design Value - Single Shear Wood-to-Wood Connection

Using the 2015/2018 NDS Yield Limit Equations (NDS 12.3), determine the Allowable Stress Design (ASD) reference lateral design value of a single shear connection with the following configuration:

Main member
Nominal 4x Hem-Fir (Actual thickness = 3.5”)

Side member
Nominal 4x Hem-Fir (Actual thickness = 3.5”)

Both members loaded parallel to grain
G = 0.43 for Hem-Fir (NDS Table 12.3.3A)

Fastener Dimensions:
1/2 in. diameter bolt
8 in. Bolt with 1.5 in. thread length (NDS Table L1)
E2.5 - Single Bolt Lateral Design Value - Single Shear Wood-to-Wood Connection

Using the 2015/2018 NDS Yield Limit Equations (NDS 12.3), determine the Allowable Stress Design (ASD) reference lateral design value of a single shear connection with the following configuration:

Main member
Nominal 4x Hem-Fir (Actual thickness = 3.5")

Side member
Nominal 4x Hem-Fir (Actual thickness = 3.5")

Both members loaded parallel to grain
G = 0.43 for Hem-Fir (NDS Table 12.3.3A)

Fastener Dimensions:
1/2 in. diameter bolt
8 in. Bolt with 1.5 in. thread length (NDS Table L1)

Define parameters:

\[ F_{em} := 4800 \quad \text{Main member Dowel Bearing Strength (NDS Table 12.3.3) (psi)} \]
\[ F_{es} := 4800 \quad \text{Side member Dowel Bearing Strength (NDS Table 12.3.3) (psi)} \]
\[ R_c := \frac{F_{em}}{F_{es}} \quad R_c = 1 \]
\[ F_{yb} := 45000 \quad \text{Fastener dowel bending yield strength (psi) (NDS Table I)} \]
\[ D := 0.5 \quad \text{Bolt Diameter (in.)} \]

Per NDS 12.3.7.2, check that threads are less than 1/4 the bearing length in the member holding the threads. In this case, 3.5 in./4 > 0.5 in. Therefore, OK to use D instead of Dr in calculations.

\[ L_s := 3.5 \quad \text{Side member Dowel Bearing Length (in.) (NDS 12.3.5)} \]
\[ L_m := 3.5 \quad \text{Main member Dowel Bearing Length (in.) (NDS 12.3.5)} \]
\[ R_{d1} := 4.0 \]
\[ R_{d2} := 3.6 \quad \text{Reduction Terms (NDS Table 12.3.1B)} \]
\[ R_{d3} := 3.2 \]
Calculate $k_1$, $k_2$, and $k_3$ (NDS Table 12.3.1A)

$$R_t := \frac{L_m}{L_s} \quad \Rightarrow \quad R_t = 1$$

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

$k_1 = 0.414$

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

$k_2 = 1.093$

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

$k_3 = 1.093$

Yield Mode Calculations (NDS Table 12.3.1A)

**Mode $I_m$**

$$Z_{Im} := \frac{D \cdot L_m \cdot F_{em}}{R_{d1}}$$

$Z_{Im} = 2100 \quad \text{Yield Mode } I_m \text{ Solution (lbs)}$

**Mode $I_s$**

$$Z_{Is} := \frac{D \cdot L_s \cdot F_{es}}{R_{d1}}$$

$Z_{Is} = 2100 \quad \text{Yield Mode } I_s \text{ Solution (lbs)}$
2.5 SINGLE BOLT LATERAL DESIGN VALUE - SINGLE SHEAR WOOD-TO-WOOD CONNECTION

Mode II

$$Z_{II} := \frac{k_1 \cdot D \cdot L_s \cdot F_{es}}{R_{d2}}$$

$$Z_{II} = 966 \quad \text{Yield Mode II Solution (lbs)}$$

Mode III_m

$$Z_{III_m} := \frac{k_2 \cdot D \cdot L_m \cdot F_{em}}{(1 + 2 \cdot R_e) \cdot R_{d3}}$$

$$Z_{III_m} = 957 \quad \text{Yield Mode III_m Solution (lbs)}$$

Mode III_s

$$Z_{III_s} := \frac{k_3 \cdot D \cdot L_s \cdot F_{em}}{(2 + R_e) \cdot R_{d3}}$$

$$Z_{III_s} = 957 \quad \text{Yield Mode III_s Solution (lbs)}$$

Mode IV

$$Z_{IV} := \left( \frac{D^2}{R_{d3}} \right) \cdot \sqrt{\frac{2 \cdot F_{em} \cdot F_{yb}}{3 \cdot (1 + R_e)}}$$

$$Z_{IV} = 663 \quad \text{Yield Mode IV Solution (lbs)}$$

$$Z_{dist} := \begin{pmatrix}
Z_{Im} \\
Z_{Is} \\
Z_{II} \\
Z_{III_m} \\
Z_{III_s} \\
Z_{IV}
\end{pmatrix}$$

Creating an array with all Yield Mode Solutions

$$Z := \min(Z_{dist}) \quad Z = 663$$

Minimum value of all Yield Modes provides Z-reference lateral design value (lbs). Mode IV controls.
Repeat same problem, but solve using Technical Report 12 - General Dowel Equations for Calculating Lateral Connection Values (TR-12) Equations for comparison

\[ q_s := F_{cs} \cdot D \]  
Side member dowel bearing resistance, lbs/in.

\[ q_m := F_{cm} \cdot D \]  
Main member dowel bearing resistance, lbs/in.

\[ M := \frac{F_{yb} \cdot D^3}{6} \]  
Side and Main member dowel resistance (equal due to equivalent dowel diameter in both members), in.-lbs

\[ \text{gap} := 0 \]  
Gap between member shear planes, in.

The limiting wood stresses used in the yield model are based on the load at which the load-deformation curve from a fastener embedment test intersects a line represented by the initial tangent modulus offset 5% of the fastener diameter. The reduction term, \( R_d \), reduces the values calculated using the yield limit equations to approximate estimates of the nominal proportional limit design values in previous NDS editions.

**Yield Mode Calculations (TR-12 Table 1-1)**

**Mode \( I_m \)**

\[ P_{1m} := q_m \cdot L_m \]  
TR-12 Yield Mode \( I_m \) Solution (lbs)

\[ P_{1m} = 8400 \]

**Mode \( I_s \)**

\[ P_{1s} := q_s \cdot L_s \]  
TR-12 Yield Mode \( I_s \) Solution (lbs)

\[ P_{1s} = 8400 \]
Mode II

\( A_{II} := \frac{1}{4 \cdot q_s} + \frac{1}{4 \cdot q_m} \)

\( B_{II} := \frac{L_s}{2} + \text{gap} + \frac{L_m}{2} \)

\( C_{II} := \frac{-q_s \cdot L_s^2}{4} - \frac{q_m \cdot L_m^2}{4} \)

\( P_{II} = \frac{-B_{II} + \sqrt{B_{II}^2 - 4 \cdot A_{II} \cdot C_{II}}}{2 \cdot A_{II}} \)

\( P_{II} = 3479 \quad \text{TR-12 Yield Mode II Solution (lbs)} \)

Mode III\(_m\)

\( A_{III_m} := \frac{1}{2 \cdot q_s} + \frac{1}{4 \cdot q_m} \)

\( B_{III_m} := \text{gap} + \frac{L_m}{2} \)

\( C_{III_m} := -M - \frac{q_m \cdot L_m^2}{4} \)

\( P_{III_m} := \frac{-B_{III_m} + \sqrt{B_{III_m}^2 - 4 \cdot A_{III_m} \cdot C_{III_m}}}{2 \cdot A_{III_m}} \)

\( P_{III_m} = 3062 \quad \text{TR-12 Yield Mode III\(_m\) Solution (lbs)} \)

Mode III\(_s\)

\( A_{III_s} := \frac{1}{4 \cdot q_s} + \frac{1}{2 \cdot q_m} \)

\( B_{III_s} := \text{gap} + \frac{L_s}{2} \)

\( C_{III_s} := -M - \frac{q_s \cdot L_s^2}{4} \)

\( P_{III_s} := \frac{-B_{III_s} + \sqrt{B_{III_s}^2 - 4 \cdot A_{III_s} \cdot C_{III_s}}}{2 \cdot A_{III_s}} \)
\[ P_{III_s} = 3062 \quad \text{TR-12 Yield Mode III}_s \text{ Solution (lbs)} \]

**Mode IV**

\[ A_{IV} := \frac{1}{2 \cdot q_s} + \frac{1}{2 \cdot q_m} \]

\[ B_{IV} := \text{gap} \]

\[ C_{IV} := -M - M \]

\[ P_{IV} := \frac{-B_{IV} + \sqrt{B_{IV}^2 - 4 \cdot A_{IV} \cdot C_{IV}}}{2 \cdot A_{IV}} \]

\[ P_{IV} = 2121 \quad \text{TR-12 Yield Mode IV Solution (lbs)} \]

\[
\begin{pmatrix}
  P_{Im} \\
  R_{d1} \\
  P_{Is} \\
  R_{d1} \\
  P_{II} \\
  R_{d2} \\
  P_{III} \\
  R_{d3} \\
  P_{III_s} \\
  R_{d3} \\
  P_{IV} \\
  R_{d3}
\end{pmatrix}
\]

\[ Z_{\text{dist}2} := \left( \begin{array}{c}
  2100 \\
  2100 \\
  966 \\
  957 \\
  957 \\
  663
\end{array} \right) \]

Converting from TR-12 "P" values to NDS "Z" values and creating an array. Shows TR-12 results equal NDS results for each Yield Mode. All values in units of lbs.

\[ Z_2 := \min(Z_{\text{dist}2}) \]

\[ Z_2 = 663 \]

Z value from TR-12 equations is equivalent to Z value from NDS equations and comparable to NDS Table 12A value \( Z_{\text{parallel}} = 660 \) lbs. See NDS Table 11.3.1 for application of additional adjustment factors for connections based on end use conditions.
For comparison, $Z_{\text{dist}}$ shows NDS equation results, $Z_{\text{dist2}}$ shows TR-12 equation results, for Modes I, II, III, III, and IV, respectively. All values in units of lbs.
**E2.6 - Bolted Wood-to-Wood Tension Splice Connection Capacity**

Determine the ASD Capacity of a tension splice shown in the Figure below.

Assume 1" diameter x 5" long bolts
Main and side members are nominal 2x12 No. 2 Southern Pine (G = 0.55)
Dead and construction live loads apply
Normal moisture and temperature conditions
Members loaded parallel to grain

![Diagram of bolted wood-to-wood tension splice connection capacity](image-url)
E2.6 - Bolted Wood-to-Wood Tension Splice Connection Capacity

Determine the ASD Capacity of a tension splice shown in the Figure below.

Assume 1" diameter x 5" long bolts
Main and side members are nominal 2x12 No. 2 Southern Pine (G = 0.55)
Dead and construction live loads apply
Normal moisture and temperature conditions
Members loaded parallel to grain

Define parameters:

\[ D := 1.0 \] Bolt Diameter (in.)
\[ s := 4 \] Fastener row spacing (in.)
\[ L_s := 1.5 \] Side member Dowel Bearing Length (NDS 12.3.5)
\[ L_m := 1.5 \] Main member Dowel Bearing Length (NDS 12.3.5.3)
\[ C_D := 1.25 \] Construction load duration (NDS Table 2.3.2)
\[ C_M := 1.0 \]
\[ C_t := 1.0 \]

Group Action Factor (\( C_g \))
\[ A_s := 2 \cdot 16.875 \] Area of side members (in\(^2\))
\[ A_m := 16.875 \] Area of main member (in\(^2\))

NDS Table 11.3.6A Footnote 1 states when \( A_s/A_m > 1.0 \), use \( A_m/A_s \) and \( A_m \) instead of \( A_s \).
Assuming 3 fasteners per row, interpolate results from Table 11.3.6A

\[ C_g := 0.97 \]
Geometry Factor ($C_\Delta$)

Geometry factor provides reduction of reference lateral design values for less than full end distance, edge distance, or spacing (NDS 12.5.1). Smallest calculated geometry factor will control.

End distance

NDS 12.5.1.2(a), Table 12.5.1A

Minimum end := 7  
Minimum end distance required for $C_\Delta = 1.0$ (in.)
Minimum end distance = 7D for softwoods loaded parallel to grain

Provided end := 4  
Provided end distance (in.)

$$C_{\Delta_{\text{end}}} := \frac{\text{Provided end}}{\text{Minimum end}}$$

$C_{\Delta_{\text{end}}} = 0.57$  
Geometry factor based on end distance

Spacing between bolts

NDS 12.5.1.2(c), Table 12.5.1B

Minimum bolt := 4  
Minimum spacing between bolts required for $C_\Delta = 1.0$ (in.)
Minimumum spacing = 4D for parallel to grain load

Provided bolt := 4  
Provided spacing between bolts (in.)

$C_{\Delta_{\text{bolt}}} := 1.0$  
Provided spacing greater than or equal to minimum spacing for $C_\Delta = 1.0$

Edge distance

NDS 12.5.1.3, Table 12.5.1C

Minimum edge := 1.5  
Minimum edge distance required for $C_\Delta = 1.0$ (in.)
Minimumum edge distance = 1.5D for parallel to grain load and $L/D \leq 6$

Provided edge := 3.625  
Provided edge distance between bolts (in.)

$C_{\Delta_{\text{edge}}} := 1.0$  
Provided edge distance greater than or equal to minimum spacing for $C_\Delta = 1.0$
2.6 BOLTED WOOD-TO-WOOD TENSION SPLICE CONNECTION CAPACITY

Spacing between rows

NDS 12.5.1.3, Table 12.5.1D

Minimum_row := 1.5 Minimum spacing between rows required for $C_\Delta = 1.0$ (in.)

Minimum spacing = 1.5D for parallel to grain load

Provided_row := 4 Provided spacing between bolts (in.)

$C_\Delta_{row} := 1.0$ Provided spacing greater than or equal to minimum spacing for $C_\Delta = 1.0$

$$C_\Delta := \min(C_{\Delta\text{end}}, C_{\Delta\text{bolt}}, C_{\Delta\text{edge}}, C_{\Delta\text{row}})$$

$C_\Delta = 0.57$

Reference Lateral Design Value

$Z := 2310$ Reference Lateral Design Value (NDS Table 12F)

Adjusted Multiple Bolt Capacity

$n := 6$ $n =$ number of bolts on each side

$Z_{adj} := n \cdot Z \cdot C_D \cdot C_g \cdot C_\Delta \cdot C_M \cdot C_t$ Adjusted ASD lateral design value (lbs)

$Z_{adj} = 9603$

Local Stresses in Fastener Groups

NDS Appendix E calls for net section tension, row tear-out, and group tear-out capacity checks. The only applicable adjustment factor for these checks will be the load duration factor, $C_D$.

Net Section Tension Check

$A_{\text{net}} := (11.25 - 1.0625 \cdot 2) \cdot 1.5$ Net area = (member depth - bolt hole width) x member width

Note: hole size includes 1/16" oversizing per 2015/2018 NDS 12.1.3.2

$A_{\text{net}} = 13.7$ Net area (in.$^2$)

$F_t := 450$ Nominal tension parallel to grain design value (psi)

$F_{adj} := F_t \cdot C_D \cdot C_M \cdot C_t$ Adjusted tension parallel to grain design value (psi)

$F_{adj} = 562.5$

$Z_{NTadj} := F_{adj} \cdot A_{\text{net}}$ Adjusted Net Section Tension Capacity (lbs)

$Z_{NTadj} = 7699$

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Row Tear-Out Check

\[ t := 1.5 \]  
Member thickness (in.)

\[ F_v := 175 \]  
Nominal shear parallel to grain design value (psi)

\[ F_{vadj} := F_v \cdot C_D \cdot C_M \cdot C_t \]  
Adjusted shear parallel to grain design value (psi)

\[ F_{vadj} = 219 \]

\[ s_{critical} := 4 \]  
\( s_{critical} \) is the minimum of the end distance and the in-row bolt spacing

\[ n_i := 3 \]  
Number of fasteners in a single row

\[ Z_{RTiadj} := n_i \cdot F_{vadj} \cdot t \cdot s_{critical} \]  
Adjusted row tear-out capacity for single row (lbs)

\[ Z_{RTiadj} = 3938 \]

\[ Z_{RTadj} := 2 \cdot n_i \cdot F_{vadj} \cdot t \cdot s_{critical} \]  
Adjusted row tear-out capacity for two rows (lbs)

\[ Z_{RTadj} = 7875 \]

Group Tear-Out Check

\[ Z_{RT1adj} := Z_{RTiadj} \]  
\( Z_{RT1} \) is row tear-out capacity for 1st row of bolts, \( Z_{RT2} \) is row tear-out capacity for 2nd row of bolts, both values in lbs

\[ Z_{RT2adj} := Z_{RTiadj} \]

\[ A_{groupnet} := (4 - 1.0625) \cdot 1.5 \]  
\( A_{groupnet} \) is equal to distance between bolt rows times member thickness (in.²)

\[ F_{tadj} = 562.5 \]  
Adjusted tension parallel to grain design value (psi)

\[ Z_{GTadj} := \frac{Z_{RT1adj}}{2} + \frac{Z_{RT2adj}}{2} + F_{tadj} \cdot A_{groupnet} \]  
Adjusted group tear-out capacity (lbs)

\[ Z_{GTadj} = 6416 \]

Connection capacity is minimum of calculated capacities (lbs). Note group tear-out controls. However, increasing the spacing between bolt rows, up to a maximum of 5" for dimension lumber, can increase group tear-out capacity. Bolt sizes could be reduced to more closely match net section capacities.
E3.1 - Segmented Shear Wall Design - Wind

Using the 2015 Special Design Provisions for Wind and Seismic (SDPWS), design the first floor wall shown in the diagram below as a segmented shear wall for a two-story house using Allowable Stress Design (ASD) provisions.

Design wind speed = 160 mph (700-yr wind speed, 3-second gust)
Exposure Category B
Building dimensions:

- L = 40 ft
- W = 32 ft
- Roof pitch = 7:12
- Top plate to ridge height = 9.3 ft
- Wall height = 9 ft
- Door height = 7 ft 6 in.
- Window height = 4.5 ft
- Stud spacing = 16 in. o.c.
- Studs are Southern Pine (G=0.55)

Check design with and without interior gypsum and neglect deflection.

Use ASCE 7-10 Minimum Design Loads for Buildings and Other Structures to determine wind loads.
Design with Interior Gypsum

Check maximum segment length based on Aspect Ratio Limits

Maximum aspect ratio for Wood Structural Panel Shear Walls = 3.5:1 (SDPWS 4.3.4)

Minimum segment length = Wall Height/Aspect Ratio

\[ L_{\text{min}} := \frac{9}{3.5} \quad L_{\text{min}} = 2.6 \quad \text{Minimum full height wall segment length (ft)} \]

All full height segments satisfy aspect ratio requirements. Maximum aspect ratio for WSP shear walls to avoid capacity adjustments = 2:1, so 3 foot wide segments will require capacity adjustments.

\[ V_W := 5520 \quad \text{Applied shear load on each shear wall due to wind force (lbs)} \]

\[ h := 9 \quad \text{Wall height (ft)} \]

(Note: Shear load calculated using Table 2.5B of 2015 Wood Frame Construction Manual)

Assume 15/32 in. thick Wood Structural Panel (WSP) Sheathing, 8d nails @ 4 in. o.c. edge spacing. SDPWS Table 4.3A nominal capacity = 1065 lbs/ft (Wind)

\[ v_{w,\text{ASD}}^{\text{WSP}} := \frac{1065}{2} \quad v_{w,\text{ASD}}^{\text{WSP}} = 532.5 \quad \text{ASD Shearwall Capacity, WSP (lbs/ft)} \]

Maximum aspect ratio for gypsum wallboard = 2:1 (SDPWS 4.3.4) and segments exceeding 1.5:1 shall be blocked. Since no segment in the wall meets the 1.5:1 aspect ratio, assume all gypsum panels will be blocked.

Assume 1/2 in. thick Gypsum Wallboard (GWB) Sheathing, 5d cooler nail @ 7 in. o.c. edge spacing, 16 in. o.c. studs, blocked

SDPWS Table 4.3C nominal capacity = 250 lb/ft

\[ v_{w,\text{ASD}}^{\text{GWB}} := \frac{250}{2} \quad v_{w,\text{ASD}}^{\text{GWB}} = 125 \quad \text{ASD Shearwall Capacity, Gypsum (lbs/ft)} \]

Capacity of WSP and GWB can be added (SDPWS 4.3.3.3.2)

\[ v_{\text{W,ASD}} := v_{w,\text{ASD}}^{\text{WSP}} + v_{w,\text{ASD}}^{\text{GWB}} \]

\[ v_{\text{W,ASD}} = 657.5 \quad \text{ASD Shearwall Capacity, WSP and GWB combined (lbs/ft)} \]

Need minimum 5520 lbs capacity to resist applied wind shear. Provide full height segments on either side of door for 10 ft of full height sheathing.

Capacity := 2.5 \cdot v_{\text{W,ASD}}

Capacity = 6575 \quad \text{Wall capacity (lbs)}

Wall capacity exceeds demand, so design is OK.
Hold down capacity is based on induced unit shear (hold downs at each end of each panel)

\[ L_{\text{eff}} := 10 \quad \text{Effective length of Full Height Segments (ft)} \]
\[ T := \left( \frac{V_w}{L_{\text{eff}}} \right) h \quad T = 4968 \quad \text{Required Hold down capacity (lbs)} \]

Hold down would need to be combined with 2nd floor hold down requirements. Dead load offset has been neglected in this example.

**Design without Interior Gypsum**

Check maximum segment length based on Aspect Ratio Limits

Maximum aspect ratio for Wood Structural Panel Shear Walls = 3.5:1 (SDPWS 4.3.4)

Minimum segment length = Wall Height/Aspect Ratio

\[ L_{\text{min}} := \frac{9}{3.5} \quad L_{\text{min}} = 2.6 \quad \text{Minimum full height wall segment length (ft)} \]

All full height segments satisfy aspect ratio requirements. Maximum aspect ratio for WSP shear walls to avoid capacity adjustments = 2:1, so 3 foot wide segments will require capacity adjustments.

\[ V_w := 5520 \quad \text{Applied shear load on each shear wall due to wind force (lbs)} \]
\[ h := 9 \quad \text{Wall height (ft)} \]

(Note: Shear load calculated using Table 2.5B of 2015 Wood Frame Construction Manual)

Assume 15/32 in. thick Wood Structural Panel (WSP) Sheathing, 8d nails @ 4 in. o.c. edge spacing. SDPWS Table 4.3A nominal capacity = 1065 lbs/ft (Wind)

\[ V_{w\text{ASDWSP}} := \frac{1065}{2} \quad V_{w\text{ASDWSP}} = 532.5 \quad \text{ASD Shearwall Capacity, WSP (lbs/ft) (SDPWS 4.3.3)} \]

Provide two 5 ft wide by 9 ft height panels and two 3 ft wide by 9 ft height panels. Three ft wide panels require adjustment per SDPWS 4.3.3.4.1. Capacity of each 3 ft wide segment is multiplied by 2b_s/h adjustment factor.

\[ b_s := 3 \quad \text{Width of panel requiring capacity adjustment (ft)} \]

\[ \text{Capacity} := 2 \cdot 5 \cdot V_{w\text{ASDWSP}} + \left( \frac{2b_s}{h} \right) (2)(3) \cdot V_{w\text{ASDWSP}} \]
\[ \text{Capacity} = 7455 \quad \text{Wall capacity (lbs)} \]

Wall capacity exceeds demand, so design is OK.

Hold down capacity is based on induced unit shear (hold downs at each end of each panel).
\[ L_{\text{eff}} := 2(5) + \left( \frac{2b_s}{h} \right)(2)(3) \quad L_{\text{eff}} = 14 \]

Provided effective Full Height Sheathing Length (ft)

\[ T := \left( \frac{V_w}{L_{\text{eff}}} \right) \cdot h \quad T = 3549 \]

Required Hold down capacity (lbs) (SDPWS 4.3.6.1.2)

Hold down would need to be combined with 2nd floor hold down requirements. Dead load offset has been neglected in this example. (SDPWS 4.3.6.4.2)
E3.2 - Segmented Shear Wall Design - Seismic

Using the 2015 Special Design Provisions for Wind and Seismic (SDPWS), design the first floor wall shown in the diagram below as a segmented shear wall for a two-story house using Allowable Stress Design (ASD) provisions.

Seismic Design Category D₁ (Based on 2015 IRC)

Building dimensions:

- L = 40 ft
- W = 32 ft
- Roof pitch = 7:12
- Top plate to ridge height = 9.3 ft
- Wall height = 9 ft
- Door height = 7 ft 6 in.
- Window height = 4.5 ft
- Stud spacing = 16 in. o.c.
- Studs are Southern Pine (G=0.55)

Neglect deflection check.

Check maximum segment length based on Aspect Ratio Limits

Maximum aspect ratio for Wood Structural Panel Shear Walls = 3.5:1 (SDPWS 4.3.4)

Minimum segment length = Wall Height/Aspect Ratio

\[ L_{\text{min}} = \frac{9}{3.5} = 2.6 \]  
Minimum full height wall segment length (ft)

All full height segments satisfy aspect ratio requirements. Maximum aspect ratio for WSP shear walls to avoid capacity adjustments = 2:1, so 3 foot wide segments will require capacity adjustments.

\[ V_s = 4733 \quad \text{Applied shear load on each shear wall due to seismic force (lbs)} \]
\[ h = 9 \quad \text{Wall height (ft)} \]

(Note: Seismic force calculated using WFCM Table 2.6 and WFCM Commentary.)

Assume 7/16 in. thick Wood Structural Panel (WSP) Sheathing, 8d nails @ 4 in. o.c. edge spacing. Studs @ 16 in. o.c. triggers Footnote 2 which allows for use of 15/32 in. panel shear values. SDPWS Table 4.3A nominal capacity = 760 lbs/ft (Seismic). Unlike wind design, gypsum capacity is not included for seismic shear wall design.

\[ v_{s,\text{ASDWSP}} = \frac{760}{2} = 380 \quad \text{ASD Shearwall Capacity (lbs/ft) (SDPWS 4.3.3)} \]

Provide two 5 ft wide by 9 ft height panels and two 3 ft wide by 9 ft height panels. Three ft wide panels require adjustment per SDPWS 4.3.3.4.1. Capacity of each 3 ft wide segment is multiplied by \(2b_s/h\) adjustment factor.

\[ b_s = 3 \quad \text{Width of panel requiring capacity adjustment (ft)} \]

Capacity = \(2.5 \cdot v_{s,\text{ASDWSP}} + \left(\frac{2b_s}{h}\right)(2)(3) \cdot v_{s,\text{ASDWSP}}\)

Capacity = 5320  
Wall capacity (lbs)

Wall capacity exceeds demand, so design is OK.

Hold down capacity is based on induced unit shear of effective length of Full Height Segments.

\[ L_{\text{eff}} = 2(5) + \left(\frac{2b_s}{h}\right)(2)(3) = 14 \quad \text{Provided effective Full Height Sheathing Length (ft)} \]
\[ T = \frac{V_s}{L_{\text{eff}}} \quad T = 3043 \quad \text{Required Hold down capacity (lbs) (SDPWS 4.3.6.1.2)} \]

Hold down would need to be combined with 2nd floor hold down requirements. Dead load offset has been neglected in this example. (SDPWS 4.3.6.4.2)
E3.3 - Perforated Shear Wall Design - Wind

Using the 2015 Special Design Provisions for Wind and Seismic (SDPWS), design the first floor wall shown in the diagram below as a perforated shear wall (PSW) for a two-story house using Allowable Stress Design (ASD) provisions.

Design Wind Speed = 160 mph (3 sec. gust, 700 year return)
Exposure B

Building dimensions:

- L = 40 ft
- W = 32 ft
- Roof pitch = 7:12
- Top plate to ridge height = 9.3 ft
- Wall height = 9 ft
- Door height = 7 ft 6 in.
- Window height = 4.5 ft
- Stud spacing = 16 in. o.c.
- Studs are Southern Pine (G=0.55)

Check design with and without interior gypsum, neglect deflection.

Use Minimum Design Loads for Building and Other Structures (ASCE 7-10) to determine loads.
Check maximum segment length based on Aspect Ratio Limits

Maximum aspect ratio for Wood Structural Panel Shear Walls = 3.5:1 (SDPWS 4.3.4)

Minimum segment length = Wall Height/Aspect Ratio

\[ L_{\text{min}} := \frac{9}{3.5} \]

Minimum full height wall segment length (ft)

All full height segments satisfy aspect ratio requirements. Maximum aspect ratio for WSP shear walls to avoid capacity adjustments = 2:1, so 3 foot wide segments will require capacity adjustments.

Design with Interior Gypsum

\[ V_{w} := 5520 \]

Applied shear load on each shear wall due to wind force (lbs)

\[ h := 9 \]

Wall height (ft)

(Note: Shear load calculated using Table 2.5B of 2015 Wood Frame Construction Manual)

Assume 15/32 in. thick Wood Structural Panel (WSP) Sheathing, 8d nails @ 4 in. o.c. edge spacing. SDPWS Table 4.3A nominal capacity = 1065 lbs/ft (Wind)

\[ V_{\text{ASD WSP}} := \frac{1065}{2} \]

ASD Shear wall Capacity (lbs/ft) (SDPWS 4.3.3)

Maximum aspect ratio for gypsum wallboard = 2:1 (SDPWS 4.3.4) and segments exceeding 1.5:1 shall be blocked. Since no segment in the wall meets the 1.5:1 aspect ratio, assume all gypsum panels will be blocked.

Assume 1/2 in. thick Gypsum Wallboard (GWB) Sheathing, 5d cooler nail @ 7 in. o.c. edge spacing, 16 in. o.c. studs, blocked

SDPWS Table 4.3C nominal capacity = 250 lb/ft

\[ V_{\text{ASD GWB}} := \frac{250}{2} \]

ASD Shear wall Capacity, Gypsum (lbs/ft) (SDPWS 4.3.3)

Capacity of WSP and GWB can be added (SDPWS 4.3.3.2)

\[ V_{\text{CASD}} := V_{\text{ASD WSP}} + V_{\text{ASD GWB}} \]

\[ V_{\text{CASD}} = 657.5 \]

ASD Shear wall Capacity, WSP and GWB combined (lbs/ft)

Calculate PSW Shear Capacity Adjustment Factor (C_o)

\[ L_{i} := 2(5) + 4 \left( \frac{2 \cdot 3}{9} \right) \cdot 3 \]

Effective length of Full Height Segments (ft) using adjustment from SDPWS 4.3.4.3

\[ L_{i} = 18 \]

\[ L_{\text{tot}} := 40 \]

Wall length (ft)
3.3 PERFORATED SHEAR WALL DESIGN - WIND

\[
A_0 := 4(4.5-3) + (6-7.5) \quad \text{Area of openings (ft}^2\text{)}
\]

\[
A_0 = 99
\]

\[
r := \frac{1}{1 + \frac{A_0}{h \cdot L_i}} \quad r = 0.621 \quad \text{(SDPWS Eqn. 4.3-6)}
\]

\[
C_o := \left(\frac{r}{3 - 2r}\right) \cdot \frac{L_{\text{tot}}}{L_i} \quad C_o = 0.78 \quad \text{(SDPWS Eqn. 4.3-5)}
\]

\[
V_{\text{PSW}} := V_{\text{WCASD}} C_o \quad V_{\text{PSW}} = 516 \quad \text{Perforated Shearwall (PSW) Capacity (lbs/ft)}
\]

\[
L_{\text{PSW}} := \frac{V_w}{V_{\text{PSW}}} \quad L_{\text{PSW}} = 10.7 \quad \text{Required length of Full Height Sheathing (L_{PSW}) (ft)}
\]

Provided effective length 18 ft of FHS is greater than required 10.7 ft

\[
T := \frac{(V_w \cdot h)}{C_o L_i} \quad \text{Required Hold-down capacity (lbs) (SDPWS Eqn. 4.3-8)}
\]

**Design without Interior Gypsum**

\[
V_w := 5520 \quad \text{Wind reaction on shear wall (lbs)}
\]

\[
h := 9 \quad \text{Wall height (ft)}
\]

Assume 15/32 in. thick Wood Structural Panel (WSP) Sheathing, 8d nails @ 4 in. o.c. edge spacing. SDPWS Table 4.3A nominal capacity = 1065 lbs/ft (Wind)

\[
V_{\text{wASDWSP}} := \frac{1065}{2} \quad V_{\text{wASDWSP}} = 532.5 \quad \text{ASD Shear wall Capacity (lbs/ft) (SDPWS 4.3.3)}
\]

\[
C_o = 0.78 \quad \text{Calculated PSW Shear Capacity Adjustment Factor (same as above)}
\]

\[
V_{\text{PSW}} := V_{\text{wASDWSP}} C_o
\]

\[
V_{\text{PSW}} = 418 \quad \text{Perforated Shearwall (PSW) Capacity (lbs/ft)}
\]

\[
L_{\text{PSW}} := \frac{V_w}{V_{\text{PSW}}} \quad \text{Required length of Full Height Sheathing (L_{PSW}) (ft)}
\]

Provided effective length 18 ft of FHS is greater than required 13.2 ft

\[
T := \frac{(V_w \cdot h)}{C_o L_i} \quad \text{Required Hold-down capacity (lbs) (SDPWS Eqn. 4.3-8)}
\]

Hold-down would need to be combined with 2nd floor hold-down requirements. Dead load offset has been neglected in this example (SDPWS 4.3.6.4.2)
E3.4 - Perforated Shear Wall Design - Seismic

Using the 2015 Special Design Provisions for Wind and Seismic (SDPWS), design the first floor wall shown in the diagram below as a perforated shear wall for a two-story house using Allowable Stress Design (ASD) provisions.

Seismic Design Category D₁ (Based on 2015 IRC)

Building dimensions:

L = 40 ft  
W = 32 ft  
Roof pitch = 7:12  
Top plate to ridge height = 9.3 ft  
Wall height = 9 ft  
Door height = 7 ft 6 in.  
Window height = 4.5 ft  
Stud spacing = 16 in. o.c.  
Studs are Southern Pine (G=0.55)

Neglect deflection.

Use Minimum Design Loads for Building and Other Structures (ASCE 7-10) to determine loads.
Check maximum segment length based on Aspect Ratio Limits

Maximum aspect ratio for Wood Structural Panel Shear Walls = 3.5:1 (SDPWS 4.3.4)

Minimum segment length = Wall Height/Aspect Ratio

\[ L_{\text{min}} := \frac{9}{3.5} \quad L_{\text{min}} = 2.6 \quad \text{Minimum full height wall segment length (ft)} \]

All full height segments satisfy aspect ratio requirements. Maximum aspect ratio for WSP shear walls to avoid capacity adjustments = 2:1, so 3 foot wide segments will require capacity adjustments.

\[ V_s := 4733 \quad \text{Applied shear load on each shear wall due to seismic force (lbs)} \]

\[ h := 9 \quad \text{Wall height (ft)} \]

(Note: Seismic force calculated using WFCM Table 2.6 and WFCM Commentary.)

Assume 7/16 in. thick Wood Structural Panel (WSP) Sheathing, 8d nails @ 4 in. o.c. edge spacing. Studs @ 16 in. o.c. triggers Footnote 2 allows for use of 15/32 in. panel shear values. SDPWS Table 4.3A nominal capacity = 760 lbs/ft (Seismic). Unlike wind design, gypsum capacity is not included for seismic shear wall design.

\[ v_{sASDWP} := \frac{760}{2} \quad v_{sASDWP} = 380 \quad \text{ASD Shearwall Capacity (lbs/ft) (SDPWS 4.3.3)} \]

Calculate PSW Shear Capacity Adjustment Factor \( C_o \)

\[ L_i := 2(5) + 4 \left[ \frac{2.3}{9} \right] \cdot 3 \quad \text{Effective length of Full Height Segments (ft) using adjustment from SDPWS 4.3.4.3} \]

\[ L_i = 18 \]

\[ L_{\text{tot}} := 40 \quad \text{Total wall length (ft)} \]

\[ A_o := 4(4.5-3) + (6-7.5) \quad \text{Area of openings (ft}^2) \]

\[ A_o = 99 \]

\[ r := \frac{1}{1 + \frac{A_o}{h \cdot L_i}} \quad \text{(SDPWS Eqn. 4.3-6)} \]

\[ C_o := \left( \frac{r}{3 - 2r} \right) \cdot \frac{L_{\text{tot}}}{L_i} \quad \text{(SDPWS Eqn. 4.3-5)} \]

\[ C_o = 0.78 \]
\[ v_{PSW} := v_{sASDWS} \cdot C_o \quad v_{PSW} = 298 \]  
Perforated Shearwall (PSW) Capacity (lbs/ft)

\[ L_{PSW} := \frac{V_s}{v_{PSW}} \quad L_{PSW} = 15.9 \]  
Required length of Full Height Sheathing (FHS) (ft)

Provided effective length 18 ft of FHS is greater than required 15.9 ft

Hold down capacity for Perforated Shearwalls specified in SDPWS Eqn. 4.3-8

\[ T := \frac{(V_s \cdot h)}{C_o \cdot L_i} \quad T = 3017 \]  
Required Hold down capacity (lbs)

Hold down would need to be combined with 2nd floor hold down requirements. Dead load offset has been neglected in this example. (SDPWS 4.3.6.4.2)
American Wood Council

AWC Mission Statement
To increase the use of wood by assuring the broad regulatory acceptance of wood products, developing design tools and guidelines for wood construction, and influencing the development of public policies affecting the use and manufacture of wood products.