ERRATA

to the 2018 and Prior Editions of

the National Design Specification® (NDS®) for Wood Construction

Page 91  Revise footnote 1 in Table 12.5.1D as follows:

1. The \( \ell/D \) ratio used to determine the minimum edge distance spacing between rows shall be the lesser of:
   
   (a) length of fastener in wood main member/D = \( \ell_m/D \)
   
   (b) total length of fastener in wood side member(s)/D = \( \ell_s /D \)
ERRATA
to the 2018 and Prior Editions of
the National Design Specification® (NDS®) for Wood Construction

Page 166 Clarifies that the following calculations in Example E.7 Sample Solution of Row of Bolts
is intended for a single-row bolted connection with a 3-1/2" thick main member and 1-1/2" thick side member:

E.7 Sample Solution of Row of Bolts

Calculate the net section area tension and row tear-out adjusted ASD design capacities for the single-shear single-row bolted connection represented in Figure E2.

Main and Side Members:
#2 grade Hem-Fir 2x4 lumber. See NDS Supplement Table 4A – Visually Graded Dimension Lumber for reference design values. Adjustment factors CD, CT, CM, and Ci are assumed to equal 1.0 in this example for calculation of adjusted design values.

\[ F_t' = 525 \text{ psi} \times (1.5) = 788 \text{ psi} \]
\[ F_v' = 150 \text{ psi} \]

Connection Details:
Bolt diameter, D: 1/2 in.
Bolt hole diameter, Dh: 0.5625 in.
Adjusted ASD bolt design value, Z||': 550 lbs
(See NDS Table 12A for 3-1/2" main member thickness and 1-1/2" side member thickness. For this trial design, the group action factor, Cg, is taken as 1.0).

Adjusted ASD Connection Capacity, n Z||':
\[ nZ||' = (3 \text{ bolts})(550 \text{ lbs}) = 1,650 \text{ lbs} \]

Adjusted For side member, adjusted ASD Net Section Area Tension Capacity, ZNT':
\[ Z_{NT'} = F_t' \cdot t \cdot [w - n \cdot \text{row} \cdot D_h] \]
\[ Z_{NT'} = (788 \text{ psi})(1.5'')(3.5'' - 1(0.5625'')) = 3,470 \text{ lbs} \]

Adjusted For side member, adjusted ASD Row Tear-Out Capacity, ZRT':
\[ Z_{RT'} = nF_v' \cdot t \cdot s_{critical} \]
\[ Z_{RT'} = 3(150 \text{ psi})(1.5'')(2'') = 1,350 \text{ lbs} \]

In this sample calculation, the adjusted ASD connection capacity is limited to 1,350 pounds by row tear-out, ZRT'.

\[ \text{Figure E2 Single Row of Bolts} \]
2009 ERRATA

to the

2005 Edition of

the ASD/LRFD STRUCTURAL WOOD DESIGN SOLVED EXAMPLE PROBLEMS

included in the 2005 Wood Design Package

(printed versions dated 08-06 5M, 02-07 10M, 02-08 10M)

<table>
<thead>
<tr>
<th>Page(s)</th>
<th>Revision</th>
</tr>
</thead>
<tbody>
<tr>
<td>64</td>
<td>Defl_{max} = \text{span} \cdot 12 \text{ in/ft} / \frac{240}{360} \quad \text{change 240 to 360 in the denominator}</td>
</tr>
<tr>
<td></td>
<td>Defl_{max} = 0.9 \times 0.6 \text{ in}</td>
</tr>
<tr>
<td>65</td>
<td>Defl_{max} = \text{span} \cdot 12 \text{ in/ft} / \frac{240}{360} \quad \text{change 240 to 360 in the denominator}</td>
</tr>
<tr>
<td></td>
<td>Defl_{max} = 0.9 \times 0.6 \text{ in}</td>
</tr>
<tr>
<td>66</td>
<td>I_{reqd} = 1037 \times 1556 \text{ in}^4</td>
</tr>
<tr>
<td></td>
<td>Minimum required I is 1037 \times 1556 \text{ in}^4 for deflection</td>
</tr>
<tr>
<td>67</td>
<td>I_{reqd} = 1037 \times 1556 \text{ in}^4</td>
</tr>
<tr>
<td></td>
<td>Minimum required I is 1037 \times 1556 \text{ in}^4 for deflection</td>
</tr>
</tbody>
</table>
2009 ERRATA/ADDENDUM

to the

2005 Edition of

the National Design Specification® (NDS®) for Wood Construction Supplement:
Design Values for Wood Construction
(printed versions dated 04-05 2M, 09-05 2M, 08-06 5M, and 02-08 10M)

Page 60  Revision
Revise footnote 4 of Table 5A, footnote 3 of Expanded Table 5A, and footnote 3 of Table 5B as follows:

The design value for shear, $F_{vx}$ and $F_{vy}$, shall be decreased by multiplying by a factor of 0.72 for non-prismatic members, notched members, and for all members subject to impact or cyclic loading. The reduced design value shall be used for design of members at connections (NDS 3.4.3.3 and 10.1.2) that transfer shear by mechanical fasteners. The reduced design value shall also be used for determination of design values for radial tension (NDS 5.2.2).
# 2009 ERRATA/ADDENDUM to the 2005 Edition of the Commentary to the National Design Specification® (NDS®) for Wood Construction

(printed versions dated 04-05 2M, 09-05 2M, 08-06 5M, and 02-08 10M)

<table>
<thead>
<tr>
<th>Page</th>
<th>Revision</th>
</tr>
</thead>
<tbody>
<tr>
<td>249</td>
<td>Revise equation C15.1-1 as follows:</td>
</tr>
</tbody>
</table>

\[ P_{14} = -180.7 + 140.5 \log(P_m) \]
November 2008

## 2008 ERRATA

to the

2005 Edition of

the *ASD/LRFD STRUCTURAL WOOD DESIGN SOLVED EXAMPLE PROBLEMS*

included in the *2005 Wood Design Package*

(printed versions dated 02-08-10M, 08-06 5M, and 02-07 10M)

<table>
<thead>
<tr>
<th>Page(s)</th>
<th>Revision</th>
</tr>
</thead>
</table>
| 264     | $M_{\text{Total}} = M_{\text{load}} + \text{Weight} \cdot \text{Length}^2 / 8 \cdot 12$ | delete “12” in the denominator  
$f_b = 4695 \ 1778$ psi |
| 265     | $M_{\text{Total}} = M_{\text{load}} + 1.2(\text{Weight} \cdot \text{Length}^2) / 8 \cdot 12$ | delete “12” in the denominator  
$f_b = 2575 \ 2674$ psi |
| 274     | $M_{\text{Total}} = M_{\text{load}} + \text{Weight} \cdot \text{Length}^2 / 8 \cdot 12$ | delete “12” in the denominator  
$f_b = 4352 \ 1401$ psi |
| 275     | $M_{\text{Total}} = M_{\text{load}} + 1.2(\text{Weight} \cdot \text{Length}^2) / 8 \cdot 12$ | delete “12” in the denominator and incorporate 1.2 load factor  
$f_b = 2145 \ 2204$ psi |
2008 ERRATA/ADDENDUM

to the

the National Design Specification® (NDS®) for Wood Construction
(printed versions dated 04-05 2M, 09-05 2M, 08-06 5M, and 02-08 10M)

The 1991-2005 Editions of the National Design Specification®(NDS®) for Wood Construction have contained provisions for designing columns loaded simultaneously with axial, edgewise bending, and/or flatwise bending loads using a stress interaction equation. When a flatwise bending load is checked with the third term of the stress interaction equation, the axial and edgewise bending interaction in the denominator can become a negative value. The occurrence of the negative value indicates an overstress. Use of this negative term in the stress interaction equation overlooks the overstress in flatwise bending and incorrectly reduces the overall interaction.

While a check for overstress due to bending is a limiting condition of member design for bending per 3.3.1 of the NDS, an explicit limit is provided to clarify limitations on flatwise bending in NDS stress interaction equations as follows:

**Page Revision**
20-21 Add limitation to provisions in NDS 3.9.2:

\[
\frac{f_c}{F_{ce2}} + \left( \frac{f_{bl}}{F_{be}} \right)^2 < 1.0
\]

**Page Revision**
190 Append to NDS Commentary C3.9.2:

The limits on \( f_c \) and \( f_{bl} \) (e.g. \( f_c < F_{ce1}, f_c < F_{ce2}, \) and \( f_{bl} < F_{be} \)) do not address the case where the sum of the \( f_c/F_{ce2} \) stress ratio and the square of the \( f_{bl}/F_{be} \) stress ratio in the denominator of the third term of Equation 3.9-3 exceeds 1.0. In this case, the third term becomes negative indicating overstress in flatwise bending due to combined loading effects. Inclusion of the third term as a negative value overlooks the overstress in flatwise bending and incorrectly reduces the interaction calculation. A limit on \( (f_c/F_{ce2}) + (f_{bl}/F_{be})^2 \) is stated explicitly to clarify limitations on flatwise bending in the stress interaction equation and avoid accidental inclusion of a negative value in the stress interaction equation.

**Page Revision**
137 Add limitation to provisions in NDS 15.4.1:

\[
\frac{f_c}{F_{ce2}} + \left( \frac{f_{bl} + f_c(6e_i/d_i)}{F_{be}} \right)^2 < 1.0
\]
Page Revision
253 Append to *NDS Commentary C15.4.1:

The limits on $f_c$ and $f_{b1}$ (e.g. $f_c < F_{cE1}$, $f_c < F_{cE2}$, and $f_{b1} < F_{bE}$) do not address the case where the sum of the $f_c/F_{cE2}$ stress ratio and the square of the $f_{b1}/F_{bE}$ stress ratio or the $[f_{b1} + f_c(6e_1/d_1)]/F_{bE}$ ratio in the denominator of the third term of Equations 15.4-1 and 15.4-2 exceeds 1.0. In this case, the third term becomes negative indicating overstress in flatwise bending due to combined loading effects. Inclusion of the third term as a negative value overlooks the overstress in flatwise bending and incorrectly reduces the interaction calculation. A limit on $(f_c/F_{cE2}) + ([f_{b1} + 6f_c e_1/d_1]/F_{bE})^2$ is stated explicitly to clarify limitations on flatwise bending in the stress interaction equation and avoid accidental inclusion of a negative value in the stress interaction equations.
2008 ERRATA
to the
2005 Edition of

the SPECIAL DESIGN PROVISIONS FOR WIND AND SEISMIC, ANSI/AF&PA SDPWS-2005
included in the 2005 Wood Design Package
(printed versions dated 08-06 5M, 02-07 10M, and 02-08 10M)

<table>
<thead>
<tr>
<th>Page(s)</th>
<th>Revision</th>
</tr>
</thead>
<tbody>
<tr>
<td>47</td>
<td>In Example C4.2.2-1 revise “Nail load/slip at 1.4 $v_{\text{w(ASD)}}$:”</td>
</tr>
<tr>
<td></td>
<td>In Example C4.2.2-1 revise “= (129.5/456)^3.144 = 0.0191”</td>
</tr>
<tr>
<td>50</td>
<td>In Equations C4.3.2-1 and C4.3.2-2 revise “(tie-down nail slip)”</td>
</tr>
<tr>
<td></td>
<td>In the third paragraph of C4.3.2 (second column) revise “…or 3-term equation (SDPW\overline{D}S)…”</td>
</tr>
<tr>
<td>53</td>
<td>In Example C4.3.2-1, text beginning with “C4.3.2.1” to the end should be moved to page 51 at the end of section C4.3.2.</td>
</tr>
</tbody>
</table>
2008 ERRATA/ADDENDUM
to the

2005 Edition of

the ASD/LRFD MANUAL FOR ENGINEERED WOOD CONSTRUCTION
included in the 2005 Wood Design Package
(printed versions dated 08-06 5M, 02-07 10M, and 02-08 10M)

<table>
<thead>
<tr>
<th>Page(s)</th>
<th>Revision</th>
</tr>
</thead>
<tbody>
<tr>
<td>155</td>
<td>Revise Table M16.2-10 column headers for decking width as follows:</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Rating</th>
<th>1-HOUR</th>
<th>1.5-HOUR</th>
<th>2-HOUR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Decking Width</td>
<td>5</td>
<td>2.5</td>
<td>3.5</td>
</tr>
</tbody>
</table>
2008 ERRATA to the
2005 Edition of

the *ASD/LRFD MANUAL FOR ENGINEERED WOOD CONSTRUCTION*
included in the *2005 Wood Design Package*
(printed versions dated 08-06 5M, 02-07 10M, and 02-08 10M)

<table>
<thead>
<tr>
<th>Page(s)</th>
<th>Revision</th>
</tr>
</thead>
</table>
| 64      | Figure M9.2-4 should be renumbered M9.2-5  
          | Figure M9.2-5 should be renumbered M9.2-4 |
February 2008

2008 ERRATA
to the

2005 Edition of

the ASD/LRFD STRUCTURAL WOOD DESIGN SOLVED EXAMPLE PROBLEMS
included in the 2005 Wood Design Package
(printed versions dated 08-06 5M and 02-07 10M)

<table>
<thead>
<tr>
<th>Page(s)</th>
<th>Revision</th>
</tr>
</thead>
<tbody>
<tr>
<td>72</td>
<td>( C_{M,Fb} := 0.85 ) 1.0 Change wet service factor. According to the NDS Supplement, when ((F_b)(C_F) \leq 1150 \text{ psi} ) ( C_M = 1.0 ) ( F'_{bx} = 4424 , 1322 \text{ psi} ) This change does not affect the outcome - bending still checks.</td>
</tr>
<tr>
<td>73</td>
<td>( C_{M,Fb} := 0.85 ) 1.0 Change wet service factor. According to the NDS Supplement, when ((F_b)(C_F) \leq 1150 \text{ psi} ) ( C_M = 1.0 ) ( F'_{bx} = 2428 , 2856 \text{ psi} ) This change does not affect the outcome - bending still checks.</td>
</tr>
</tbody>
</table>
2007 ERRATA
to the

2005 Edition of

the ASD/LRFD STRUCTURAL WOOD DESIGN SOLVED EXAMPLE PROBLEMS
included in the 2005 Wood Design Package
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<table>
<thead>
<tr>
<th>Page(s)</th>
<th>Revision</th>
</tr>
</thead>
<tbody>
<tr>
<td>91</td>
<td>( \lambda = 1.0 \times 0.8 ) Time effect factor</td>
</tr>
</tbody>
</table>

93

- \( F_{bS, 24.0} = 648.5 \text{ lbf}\cdot\text{in} \)
- \( F_{bS, 24.16} = 832.6 \text{ lbf}\cdot\text{in} \)
- \( F_{bS, 32.16} = 964.8 \text{ lbf}\cdot\text{in} \)
- \( F_{bS, 40.20} = 1620.1 \text{ lbf}\cdot\text{in} \)
- \( F_{bS, 48.24} = 2160.1 \text{ lbf}\cdot\text{in} \)

The maximum moment is 826 in-lb/ft, so the 24/16 \( 40/20 \) span rating works for flexure.

95

Check the 24/16 \( 40/20 \) span rating for shear.

- \( F_s_{lbQ} := 150.205 \text{ lbf} \)
- \( F'_s_{lbQ} = 324.354 \text{ lbf} \)

The 24/16 \( 40/20 \) span rating is OK in shear.

96

\( \Delta < L/240 = 0.95 \times 0.095 \text{ in}, \text{ thus deflection is OK} \)

97

\( \Delta = 0.00677 \text{ wload span}_{\text{defl}}^4 / EI'_{40.20} \)

\( \Delta = 0.243 \times 0.074 \text{ in} \)
2007 ERRATA

to the
2005 Edition of

the *ASD/LRFD STRUCTURAL WOOD DESIGN SOLVED EXAMPLE PROBLEMS*

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<table>
<thead>
<tr>
<th>Page(s)</th>
<th>Revision</th>
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</thead>
<tbody>
<tr>
<td>376</td>
<td>Weight curved = ( \pi \cdot \frac{R_{center}}{2} \cdot \text{Depth} \cdot \text{Width} \cdot 40 \cdot \text{lbf/ft}^3 ) (delete “2” in the denominator)</td>
</tr>
<tr>
<td></td>
<td>Weight curved = 353 706 lbf</td>
</tr>
<tr>
<td></td>
<td>( M_{\text{max}} = 66244 \ 97288 \ \text{in} \cdot \text{lbf} )</td>
</tr>
<tr>
<td>377</td>
<td>Weight curved = ( \pi \cdot \frac{R_{center}}{2} \cdot \text{Depth} \cdot \text{Width} \cdot 40 \cdot \text{lbf/ft}^3 ) (delete “2” in the denominator)</td>
</tr>
<tr>
<td></td>
<td>Weight curved = 353 706 lbf</td>
</tr>
<tr>
<td></td>
<td>( M_{\text{max}} = 92742 \ 136204 \ \text{in} \cdot \text{lbf} )</td>
</tr>
<tr>
<td>380</td>
<td>( f_b = 343 \ 504 \ \text{psi} )</td>
</tr>
<tr>
<td></td>
<td>( V_{\text{load}} = 376 \ 553 \ \text{lbf} )</td>
</tr>
<tr>
<td></td>
<td>( f_c = 5 \ 8 \ \text{psi} )</td>
</tr>
<tr>
<td></td>
<td>( f_r = 40 \ 15 \ \text{psi} )</td>
</tr>
<tr>
<td>381</td>
<td>( f_b = 484 \ 706 \ \text{psi} )</td>
</tr>
<tr>
<td></td>
<td>( V_{\text{load}} = 527 \ 774 \ \text{lbf} )</td>
</tr>
<tr>
<td></td>
<td>( f_c = 7 \ 11 \ \text{psi} )</td>
</tr>
<tr>
<td></td>
<td>( f_r = 44 \ 21 \ \text{psi} )</td>
</tr>
</tbody>
</table>
384 \[ \text{Weight}_{\text{curved}} = \pi \cdot \frac{R_{\text{center}}}{2} \text{ Depth} \cdot \text{Width} \cdot 40 \cdot \text{lbf/ft}^3 \text{ (delete “2” in the denominator)} \]

\[ \text{Weight}_{\text{curved}} = 333,706 \text{ lbf} \]

\[ M_{\text{max}} = 66244,97288 \text{ in} \cdot \text{lbf} \]

\[ P_{\text{max}} = 553,906 \text{ lbf} \]

385 \[ \text{Weight}_{\text{curved}} = \pi \cdot \frac{R_{\text{center}}}{2} \text{ Depth} \cdot \text{Width} \cdot 40 \cdot \text{lbf/ft}^3 \text{ (delete “2” in the denominator)} \]

\[ \text{Weight}_{\text{curved}} = 333,706 \text{ lbf} \]

\[ M_{\text{max}} = 92742,136204 \text{ in} \cdot \text{lbf} \]

\[ P_{\text{max}} = 774,1268 \text{ lbf} \]

388 \[ f_c = 88 \text{ psi} \]

389 \[ f_c = 711 \text{ psi} \]

390 \[ f_{b1} = 343,504 \text{ psi} \]

\[ \left( \frac{f_c}{F_c} \right)^2 + \frac{f_{b1}}{F_b \left[ 1 - \left( \frac{f_c}{F_{cE1}} \right) \right]} = 0.314 \cdot 0.469 \]

391 \[ f_{b1} = 484,706 \text{ psi} \]

\[ \left( \frac{f_c}{F_c} \right)^2 + \frac{f_{b1}}{F_b \left[ 1 - \left( \frac{f_c}{F_{cE1}} \right) \right]} = 0.305 \cdot 0.454 \]

392 \[ \left( \frac{f_c}{F_c} \right)^2 = 0.00056 \cdot 0.00151 \]

\[ \frac{f_{b1}}{F_b \left[ 1 - \left( \frac{f_c}{F_{cE1}} \right) \right]} = 0.313 \cdot 0.467 \]

393 \[ \left( \frac{f_c}{F_c} \right)^2 = 0.00049 \cdot 0.00132 \]

\[ \frac{f_{b1}}{F_b \left[ 1 - \left( \frac{f_c}{F_{cE1}} \right) \right]} = 0.304 \cdot 0.453 \]
2007 ERRATA to the 2005 Edition of the ASD/LRFD STRUCTURAL WOOD DESIGN SOLVED EXAMPLE PROBLEMS included in the 2005 Wood Design Package (printed versions dated 08-06 5M and 02-07 10M)

<table>
<thead>
<tr>
<th>Page(s)</th>
<th>Revision</th>
</tr>
</thead>
<tbody>
<tr>
<td>276</td>
<td>$F'<em>{v} = \frac{2}{3} F</em>{v_a} \ C_{D} \ C_{M} \ C_{r_{Fv}}$</td>
</tr>
<tr>
<td></td>
<td>$F'_{v} = 200 \ 300$ psi</td>
</tr>
<tr>
<td></td>
<td>This change does not affect the final results for shear calculations since $f_{v} &lt; F'_{v}$.</td>
</tr>
<tr>
<td>277</td>
<td>$F'<em>{v} = \frac{2}{3} \lambda \ K</em>{Fv} \ \phi_{v} \ F_{v_a} \ C_{M} \ C_{r_{Fv}}$</td>
</tr>
<tr>
<td></td>
<td>$F'_{v} = 345.6 \ 518.4$ psi</td>
</tr>
<tr>
<td></td>
<td>This change does not affect the final results for shear calculations since $f_{v} &lt; F'_{v}$.</td>
</tr>
</tbody>
</table>
2007 ERRATA

to the

2005 Edition of

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<table>
<thead>
<tr>
<th>Page(s)</th>
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<tbody>
<tr>
<td>126</td>
<td>Change the IIC rating for assemblies “Without Gypsum Concrete” with “Carpet &amp; Pad” from 62 to 66.</td>
</tr>
<tr>
<td>129</td>
<td>Change the STC rating for assemblies “With Gypsum Concrete” with “Carpet &amp; Pad” from 68 to 58c.</td>
</tr>
</tbody>
</table>

**Page 4** In Table 2.1, revise the following:

<table>
<thead>
<tr>
<th>Species or Species Combination</th>
<th>Species That May Be Included in Combination</th>
<th>Grading Rules Agencies</th>
<th>Design Values Provided in Tables</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coast Sitka Spruce</td>
<td>Coast Sitka Spruce</td>
<td>NLGA</td>
<td>4A, 4D, 4E</td>
</tr>
<tr>
<td>Yellow Cedar</td>
<td>Yellow Cedar</td>
<td>NLGA</td>
<td>4A</td>
</tr>
</tbody>
</table>

**Page 32** In Table 4A, add the following design values for Coast Sitka Spruce:

<table>
<thead>
<tr>
<th>Species and commercial grade</th>
<th>Size classification</th>
<th>Design values in pounds per square inch (psi)</th>
<th>Modulus of Elasticity</th>
<th>Grading Rules Agency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coast Sitka Spruce</td>
<td></td>
<td>Design values in pounds per square inch (psi)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Select Structural</td>
<td></td>
<td>Bending $F_b$</td>
<td>Tension parallel to grain $F_t$</td>
<td>Shear parallel to grain $F_v$</td>
</tr>
<tr>
<td>No. 1 / No. 2</td>
<td>2&quot; &amp; wider</td>
<td>1300</td>
<td>950</td>
<td>125</td>
</tr>
<tr>
<td>No. 3</td>
<td>2&quot; &amp; wider</td>
<td>925</td>
<td>550</td>
<td>125</td>
</tr>
<tr>
<td>Stud</td>
<td>2&quot; &amp; wider</td>
<td>525</td>
<td>325</td>
<td>125</td>
</tr>
<tr>
<td>Construction</td>
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</tr>
<tr>
<td>Standard</td>
<td>2&quot;-4&quot; wide</td>
<td>725</td>
<td>450</td>
<td>125</td>
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<td></td>
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<td>275</td>
<td>175</td>
<td>125</td>
</tr>
</tbody>
</table>

**Page 34** In Table 4A, revise the following design values for Northern Species:

<table>
<thead>
<tr>
<th>Species and commercial grade</th>
<th>Size classification</th>
<th>Design values in pounds per square inch (psi)</th>
<th>Modulus of Elasticity</th>
<th>Grading Rules Agency</th>
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<tbody>
<tr>
<td>Northern Species</td>
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<td>Design values in pounds per square inch (psi)</td>
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</tr>
<tr>
<td>Select Structural</td>
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<td>Bending $F_b$</td>
<td>Tension parallel to grain $F_t$</td>
<td>Shear parallel to grain $F_v$</td>
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<tr>
<td>No. 1 / No. 2</td>
<td>2&quot; &amp; wider</td>
<td>1000</td>
<td>975</td>
<td>110</td>
</tr>
<tr>
<td>No. 3</td>
<td>2&quot; &amp; wider</td>
<td>600</td>
<td>625</td>
<td>110</td>
</tr>
<tr>
<td>Std</td>
<td>2&quot; &amp; wider</td>
<td>350</td>
<td>350</td>
<td>110</td>
</tr>
<tr>
<td>Construction</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Standard</td>
<td>2&quot;-4&quot; wide</td>
<td>475</td>
<td>225</td>
<td>110</td>
</tr>
<tr>
<td>Utility</td>
<td></td>
<td>700</td>
<td>300</td>
<td>110</td>
</tr>
<tr>
<td></td>
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<td>400</td>
<td>175</td>
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<td></td>
<td></td>
<td>175</td>
<td>75</td>
<td>110</td>
</tr>
</tbody>
</table>
36 In Table 4A, add the following design values for Yellow Cedar

<table>
<thead>
<tr>
<th>Species and commercial grade</th>
<th>Size classification</th>
<th>Bending to grain (F_b)</th>
<th>Tension parallel to grain (F_t)</th>
<th>Shear parallel to grain (F_s)</th>
<th>Compression perpendicular to grain (F_cL)</th>
<th>Compression parallel to grain (F_c)</th>
<th>Modulus of Elasticity E</th>
<th>E_min</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yellow Cedar</td>
<td>Select Structural</td>
<td>1200</td>
<td>725</td>
<td>175</td>
<td>540</td>
<td>1200</td>
<td>1,600,000</td>
<td>580,000</td>
</tr>
<tr>
<td></td>
<td>No. 1 / No. 2</td>
<td>800</td>
<td>475</td>
<td>175</td>
<td>540</td>
<td>1000</td>
<td>1,400,000</td>
<td>510,000</td>
</tr>
<tr>
<td></td>
<td>No. 3</td>
<td>475</td>
<td>275</td>
<td>175</td>
<td>540</td>
<td>575</td>
<td>1,200,000</td>
<td>440,000</td>
</tr>
<tr>
<td></td>
<td>Stud</td>
<td>2&quot; &amp; wider</td>
<td>625</td>
<td>375</td>
<td>175</td>
<td>540</td>
<td>650</td>
<td>1,200,000</td>
</tr>
<tr>
<td></td>
<td>Construction</td>
<td>925</td>
<td>550</td>
<td>175</td>
<td>540</td>
<td>1200</td>
<td>1,300,000</td>
<td>470,000</td>
</tr>
<tr>
<td></td>
<td>Standard</td>
<td>525</td>
<td>300</td>
<td>175</td>
<td>540</td>
<td>1050</td>
<td>1,200,000</td>
<td>440,000</td>
</tr>
<tr>
<td></td>
<td>Utility</td>
<td>2&quot;-4&quot; wide</td>
<td>250</td>
<td>150</td>
<td>175</td>
<td>540</td>
<td>675</td>
<td>1,100,000</td>
</tr>
</tbody>
</table>

36 In Table 4A, add the following footnote:

4. **SPECIFIC GRAVITY, G.** Specific gravity values are provided below for visually graded dimension lumber. Note that the value for Coast Sitka Spruce is applicable only for visually graded dimension lumber (2” – 4” thick). See NDS Table 11.3.2A for the specific gravity value applicable to Coast Sitka Spruce used as visually graded timber (5”x5” and larger) and visually graded decking.

<table>
<thead>
<tr>
<th>Species</th>
<th>Specific Gravity, G</th>
<th>Grading Rules Agency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coast Sitka Spruce</td>
<td>0.43</td>
<td>NLGA</td>
</tr>
<tr>
<td>Yellow Cedar</td>
<td>0.46</td>
<td>NLGA</td>
</tr>
</tbody>
</table>
2007 ERRATA
to the

2005 Edition of

the SPECIAL DESIGN PROVISIONS FOR WIND AND SEISMIC, ANSI/AF&PA SDPWS-2005
included in the 2005 Wood Design Package
(printed version dated 08-06 5M)

<table>
<thead>
<tr>
<th>Page(s)</th>
<th>Revision</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>$x = \text{distance from chord splice to nearest support, in. ft}$</td>
</tr>
<tr>
<td>13</td>
<td>$x = \text{distance from chord splice to nearest support, in. ft}$</td>
</tr>
<tr>
<td>23</td>
<td>$C_o = \text{shear capacity adjustment factor from Table 4.3.2.1, 4.3.3.4}$</td>
</tr>
<tr>
<td>44</td>
<td>$x = \text{distance from chord splice to nearest support, in. ft}$</td>
</tr>
<tr>
<td>46</td>
<td>$x = \text{distance from chord splice to nearest support, in. ft}$</td>
</tr>
</tbody>
</table>
2007 ERRATA

to the

2005 Edition of

the ASD/LRFD STRUCTURAL WOOD DESIGN SOLVED EXAMPLE PROBLEMS

included in the 2005 Wood Design Package

(printed version dated 08-06 5M)

<table>
<thead>
<tr>
<th>Page(s)</th>
<th>Revision</th>
</tr>
</thead>
<tbody>
<tr>
<td>120-121</td>
<td>The third part (minor axis bending component) of the biaxial bending and compression interaction equation should be revised as follows:</td>
</tr>
</tbody>
</table>
|         | \[
| f_{ls} + f_c \left( \frac{6e_x}{d_z} \right) \left[ 1 + 0.234 \left( \frac{f_c}{F_{ce}} \right) \right] + 0.234 \left[ \frac{f_{ls} + f_c \left( \frac{6e_x}{d_z} \right)}{F_{le}} \right]^2 \] |
|         | \[
| \frac{F_{ls}}{1 - \left( \frac{f_c}{F_{ce}} \right)} - \left[ \frac{f_{ls} + f_c \left( \frac{6e_x}{d_z} \right)}{F_{le}} \right]^2 \] |
|         | \leq 1.0 |
|         | The square symbol in the denominator goes between the last 2 brackets in the equation. |

| 132     | The third part (minor axis bending component) of the biaxial bending and compression interaction equation should be revised as follows: |
|         | \[
| \frac{\left( \frac{f_c}{F_{ce}} \right)^2 + f_c \left( \frac{6e_x}{d_z} \right) \left[ 1 + 0.234 \left( \frac{f_c}{F_{ce}} \right) \right]}{F_{ls} \left[ 1 - \left( \frac{f_c}{F_{ce}} \right) \right]} + \frac{f_{ls} + f_c \left( \frac{6e_x}{d_z} \right) + 0.234 \left[ \frac{f_{ls} + f_c \left( \frac{6e_x}{d_z} \right)}{F_{le}} \right]^2}{F_{ls} \left[ 1 - \left( \frac{f_c}{F_{ce}} \right) - \left[ \frac{f_{ls} + f_c \left( \frac{6e_x}{d_z} \right)}{F_{le}} \right]^2 \right]} = 0.945 \quad 0.927 |
|         | The square symbol in the denominator goes between the last 2 brackets in the equation. This changes the overall result to 0.927 (above) and 0.101 for the minor axis bending term (below). |
|         | \[
| f_{ls} + f_c \left( \frac{6e_x}{d_z} \right) \left[ 1 + 0.234 \left( \frac{f_c}{F_{ce}} \right) \right] + 0.234 \left[ \frac{f_{ls} + f_c \left( \frac{6e_x}{d_z} \right)}{F_{le}} \right]^2 \] |
|         | \[
| \frac{F_{ls}}{1 - \left( \frac{f_c}{F_{ce}} \right) - \left[ \frac{f_{ls} + f_c \left( \frac{6e_x}{d_z} \right)}{F_{le}} \right]^2} \] |
|         | = 0.119 \quad 0.101 |
The third part (minor axis bending component) of the biaxial bending and compression interaction equation should be revised as follows:

\[
\left( \frac{f_c}{F_c} \right)^2 + \frac{f_{c1} + f_{c2}(6e_1/d_1)[1 + 0.234(f_c/F_{c1})]}{F_{c1} \left[ 1 - (f_c/F_{c1}) \right]} + \frac{f_{c2} + f_c(6e_2/d_2)[1 + 0.234(f_c/F_{c2}) + 0.234 \left( \frac{f_{c1} + f_{c2}(6e_1/d_1)}{F_{c1}} \right)^2]}{F_{c2} \left[ 1 - (f_c/F_{c2}) - \left( \frac{f_{c1} + f_{c2}(6e_1/d_1)}{F_{c1}} \right)^2 \right]} \leq 0.785 \]

The square symbol in the denominator goes between the last 2 brackets in the equation. This changes the overall result to 0.772 (above) and 0.142 for the minor axis bending term (below).

\[
\frac{f_{c2} + f_c(6e_2/d_2)[1 + 0.234(f_c/F_{c2}) + 0.234 \left( \frac{f_{c1} + f_{c2}(6e_1/d_1)}{F_{c1}} \right)^2]}{F_{c2} \left[ 1 - (f_c/F_{c2}) - \left( \frac{f_{c1} + f_{c2}(6e_1/d_1)}{F_{c1}} \right)^2 \right]} = 0.156 \]

\[
\leq \frac{f_{c1} + f_c(6e_1/d_1)[1 + 0.234(f_c/F_{c1})]}{F_{c1} \left[ 1 - (f_c/F_{c1}) \right]} \leq 1.0
\]

The square symbol in the denominator goes between the last 2 brackets in the equation.
The 2005 Edition of the *National Design Specification for Wood Construction* is expected to be approved as an American National Standard in December 2004 or early 2005, with a designation ANSI/AF&PA NDS-2005. The 2005 NDS was developed as a dual format specification incorporating design provisions for both allowable stress design (ASD) and load and resistance factor design (LRFD). AF&PA’s Wood Design Standards Committee (WDSC) guided it through the consensus process over the course of 2-1/2 years. The primary change in the 2005 NDS is the introduction of LRFD methods to the Specification.

Several format changes to the NDS to accommodate addition of LRFD are summarized in this article and include:

- Revised terminology,
- Expanded applicability of adjustment factor tables,
- Re-format of radial tension design values,
- Revised format of beam and column stability provisions (addition of $E_{\text{min}}$ property), and
- Addition of NDS Appendix N – Load and Resistance Factor Design.

A number of other changes introduced in the 2005 Edition include:

- Removal of form factor,
- Revision of repetitive member factor for I-joists,
- Revision of full-design value terminology, and
- Clarification of built-up column provisions.

The NDS Supplement, *Design Values for Wood Construction* has also been updated to provide the latest design values for sawn lumber and glued laminated timber.

**Introducing LRFD to NDS – An Overview**

Over the years, the WDSC identified benefits of developing a dual format specification which would include: addressing user needs for consistent design information regardless of design format (ASD or LRFD); better utilizing standards committee resources; and providing current design information for the academic community. The 2005 NDS maintains the current 2001 NDS format, familiar to most wood designers. As a result, NDS 2005 is very similar to the 2001 NDS for ASD design, with few exceptions.

Users familiar with the NDS ASD provisions will also find transition to LRFD straightforward. Behavioral equations, such as those for member and connection design, are the same for both ASD and LRFD. Adjustment factor tables now include applicable factors for determining an adjusted ASD design value or an adjusted LRFD design value. A new Appendix N – Mandatory Appendix for Load and Resistance Factor Design (LRFD) outlines requirements that are unique to LRFD and adjustment factors for LRFD.

**Terminology**

Basic requirements for checking strength are revised to use terminology applicable to both ASD and LRFD. For example, the basic requirement for strength in bending is revised as follows:

> 3.3.1 The actual bending stress or moment shall not exceed the adjusted allowable bending design value.

In equation format, this takes the standard form $f_b \leq F_b'$. The term “allowable,” typically associated with ASD, is replaced by “adjusted” to be more generally applicable to either ASD or LRFD and to better describe the approach of applying adjustment factors to reference design values. Reference design values ($F_{b0}, F_{c0}, F_{d0}, F_{c,\text{ult}}, E, E_{\text{min}}$) are multiplied by adjustment factors to determine adjusted design values ($F_{b}' , F_{c}' , F_{d}' , F_{c,\text{ult}}' , E' , E_{\text{min}}'$).

**Applicability of Adjustment Factor Tables**

For member design, the adjusted bending design value, $F_{b}'$, of a sawn lumber bending member is determined using Table 4.3.1 (shown on the next page) as follows:

For ASD:

$$ F_{b}' = F_{b} C_D C_M C_t C_l C_F C_{fu} C_i C_r $$

For LRFD:

$$ F_{b}' = F_{b} K_F \phi_b \lambda C_M C_t C_l C_F C_{fu} C_i C_r $$

where:
Fb = the reference bending design value based on normal load duration.

For connection design, the adjusted lateral design value, Z′, of a dowel connection is determined using Table 10.3.1 Applicability of Adjustment Factors for Connections as follows:

For ASD:

\[ Z' = Z C_D C_M C_t C_l C_p C_{fu} C_i C_T - - - - K_F \phi_b \lambda \]

For LRFD:

\[ Z' = Z K_F \phi \lambda C_M C_t C_p C_i C_T - - - - C_b K_F \phi_c \lambda \]

where:

\[ Z = \text{the reference design value based on normal load duration.} \]

The value \( F_{bE} = 1.20E_{min}' / R_b^2 \) is algebraically equivalent to and replaces \( F_{bE} = K_{bE} E' / R_b^2 \) used in the 2001 NDS. Because the design equation for \( K_{bE} \) includes a reduction for safety, two different formats of the 2001 NDS equation would be needed to address both ASD and LRFD. Instead, the 2005 NDS utilizes \( E_{min}' \), which is adjusted for safety, so the safety factor is not part of the basic design equation. Applicable adjustments to \( E_{min}' \), based on applicability of adjustment factor tables are used to establish the appropriate adjusted modulus of elasticity for beam and column stability, \( E_{min}' \) for either ASD or LRFD.

### Table 4.3.1 Applicability of Adjustment Factors for Sawn Lumber.

<table>
<thead>
<tr>
<th>ASD only</th>
<th>ASD and LRFD</th>
<th>LRFD only</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load duration factor</td>
<td>Wet service factor</td>
<td>Temperature factor</td>
</tr>
<tr>
<td>( F'_b = F_b )</td>
<td>x</td>
<td>( C_D )</td>
</tr>
<tr>
<td>( F'_i = F_i )</td>
<td>x</td>
<td>( C_D )</td>
</tr>
<tr>
<td>( F'_i = F'_v )</td>
<td>x</td>
<td>( C_D )</td>
</tr>
<tr>
<td>( F'_c = F'_c )</td>
<td>x</td>
<td>( C_D )</td>
</tr>
<tr>
<td>( E' = E )</td>
<td>x</td>
<td>-</td>
</tr>
<tr>
<td>( E_{min}' = E_{min} )</td>
<td>x</td>
<td>-</td>
</tr>
</tbody>
</table>

### Revised Format of NDS Beam and Column Stability Provisions

The 2005 NDS includes a revised format for column and beam behavioral equations to address both ASD and LRFD:

**NDS 2005 3.3.3.8:**

3.3.3.8 The beam stability factor shall be calculated as follows:

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

where:

- \( F_{bE}^* = \text{reference bending design value multiplied by all applicable adjustment factors except} \ C_{fu}, C_V, \text{and} \ C_L \) (see 2.3) and
- \( F_{bE} = 1.20E_{min}' / R_b^2 \).
3.7.1.5 The column stability factor shall be calculated as follows:

\[ C_P = \frac{1 + \left( \frac{F_{cE}}{F_{cE}'} \right)}{2c} - \sqrt{\frac{1 + \left( \frac{F_{cE}}{F_{cE}'} \right)^2}{2c}} - \frac{F_{cE}}{F_c} \]

where:

- \( F_{cE}' \) = reference compression design value parallel to grain multiplied by all applicable adjustment factors except \( C_P \) (see 2.3) and
- \( F_{cE} = 0.822E_{min}/(l_e/d)^2 \).

The value \( F_{cE} = 0.822E_{min}'/(l_e/d)^2 \) is algebraically equivalent to and replaces \( F_{cE} = K_{cE}E'/ (l_e/d)^2 \) used in the 2001 NDS. The background justification for this change is identical to that for the beam equation in 3.3.3.8.

### Modulus of Elasticity for Beam and Column Stability, \( E_{min}' \)

For sawn lumber and glulam, reference modulus of elasticity for beam and column stability, \( E_{min} \) (which represents an approximate 5% lower exclusion value on pure bending modulus of elasticity, divided by a 1.66 factor of safety), is tabulated in the NDS Supplement. However, it can also be calculated as follows:

\[ E_{min} = 1.03E (1 – 1.645(COV_E)) / 1.66 \]

where:

- \( E \) = reference modulus of elasticity,
- 1.03 = adjustment factor to convert \( E \) to a pure bending basis except that the factor is 1.05 for glued laminated timber,
- 1.66 = factor of safety, and
- \( COV_E \) = coefficient of variation in modulus of elasticity (see NDS Appendix F).

### Table N3 Time Effect Factor, \( \lambda \) (LRFD only).

<table>
<thead>
<tr>
<th>Load combination</th>
<th>( \lambda )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.4(D+F)</td>
<td>0.6</td>
</tr>
<tr>
<td>1.2(D+F) + 1.6(H) + 0.5(L or S or R)</td>
<td>0.6</td>
</tr>
<tr>
<td>1.2(D+F) + 1.6(L+H) + 0.5(L or S or R)</td>
<td>0.7 when L is from storage</td>
</tr>
<tr>
<td></td>
<td>0.8 when L is from occupancy</td>
</tr>
<tr>
<td></td>
<td>1.25 when L is from impactb</td>
</tr>
<tr>
<td>1.2D + 1.6(L or S or R) + (L or 0.8W)</td>
<td>0.8</td>
</tr>
<tr>
<td>1.2D + 1.6W + L + 0.5(L or S or R)</td>
<td>1.0</td>
</tr>
<tr>
<td>1.2D + 1.0E + L + 0.2S</td>
<td>1.0</td>
</tr>
<tr>
<td>0.9D + 1.6W + 1.6H</td>
<td>1.0</td>
</tr>
<tr>
<td>0.9D + 1.0E + 1.6H</td>
<td>1.0</td>
</tr>
</tbody>
</table>

\( a \) Load combinations and load factors consistent with ASCE 7-02 are listed for ease of reference. Nominal loads shall be in accordance with N.1.2.

\( b \) Time effect factors, \( \lambda \), greater than 1.0 shall not apply to connections or to structural members pressure-treated with water-borne preservatives (see Reference 30) or fire retardant chemicals.
be an uncommon design case, better handled with the bi-
axial bending equation.

**Repetitive Member Factor for I-joists**

Revision to the NDS 2005 repetitive member factor, $C_r$, for I-joists corresponds to revisions in ASTM D5055-02 setting the factor equal to 1.0:

7.3.6 Repetitive Member Factor, $C_r$

For prefabricated wood I-joists with structural composite lumber flanges or sawn lumber flanges, adjusted moment design resistances shall be multiplied by the repetitive member factor, $C_r = 1.0$

In lieu of complete removal of the $C_r$ factor for I-joists in the 2005 NDS, the repetitive member factor was set to 1.0 for clarity since past practice has permitted other $C_r$ factors. For example, in the 2001 NDS, $C_r = 1.04$, for I-joists with structural composite lumber flanges and $C_r = 1.07$, for I-joists with sawn lumber flanges.

**Revised “Full-Design Value” Terminology and Added Reference to Provisions for Checking Wood Stresses**

Phrases such as “minimum spacing for full design value” and “minimum end distance for full design value” are replaced with alternate descriptions since other provisions for evaluating wood strength must also be checked to ensure that the “full-design value” can be developed. Multiple references to section 10.1.2 are added as a reminder to check wood strength at connections. Example revisions follow:

10.2.2 Multiple Fastener Connections

When a connection contains two or more fasteners of the same type and similar size, each of which exhibits the same yield mode (see Appendix I), the total adjusted design value for the connection shall be the sum of the adjusted design values for each individual fastener. Local stresses in connections using multiple fasteners shall be checked in accordance with principles of engineering mechanics (see 10.1.2).

11.1.2.4 Edge distance, end distance, and fastener spacing required to develop full design values shall not be less than the requirements in 11.5.1A-D.

These revisions do not change methods for calculating strength of connections, but remove language that is potentially confusing. For example, there are additional requirements for checking wood strength at connections based on principles of engineering mechanics and procedures outlined in Appendix E for evaluating member strength around fastener groups.

**Clarify 15.3.2.2 Built-Up Column Design**

Built-up column provisions were revised to correct an obvious but unintended limitation on short built-up columns.

15.3.2.2.... $F_c'$ for built-up columns need not be less than $F_c$ for the individual laminations designed as individual solid columns per section 3.7.

This change permits individual laminations in a built-up column to be designed using provisions of section 3.7 for solid columns. With this change, built-up columns are not unnecessarily limited to design capacities less than the sum of individual member capacities.

**Additional Design Tools**

The revised NDS will be packaged with additional publications as follows:

- ANSI/AF&PA NDS-2005 National Design Specification (NDS) for Wood Construction – with Commentary,
- ANSI/AF&PA SDPWS-05 AF&PA Supplement – Special Design Provisions for Wind and Seismic (SDPWS) with Commentary, and

See the other articles in this issue of Wood Design Focus (WDF) for additional details on the content of these publications.

The 2005 Wood Design Package will be available first quarter of 2005. Call 1-800-890-7732 to order or shop online at www.awc.org.

A related design tool is also being developed to assist designers with the use of the 2005 NDS. A workbook titled LRFD Solved Example Problems for Wood Structures has been updated and renamed Structural Wood Design Using ASD and LRFD to include parallel ASD solutions to the 40 LRFD example problems shown in the former. See the related article in this issue of WDF for details.

**Conclusion**

The primary change in the 2005 NDS is the introduction of LRFD methods to the Specification. Several format changes to the NDS to accommodate the addition of LRFD have been summarized. Users will find very minimal impact on the ASD process as a result, with the added benefit of having a transparent approach to learn and use LRFD. An integrated commentary and other design tools will be available for the new standard.

The authors are Senior Manager of Engineering Research, Director of Technical Media, and Director of Technology Transfer, respectively, with AF&PA’s American Wood Council.
**2005 Special Design Provisions for Wind and Seismic (SDPWS)**

*Philip Line, P.E. and James E. Russell, P.E.*

**Introduction**

AF&PA’s 2005 *Special Design Provisions for Wind and Seismic (SDPWS)* is a dual format, allowable stress design (ASD) / load and resistance factor design (LRFD) specification, developed as an ANSI consensus standard by AF&PA’s Wood Design Standards Committee. In the 2006 *International Building Code (IBC)*, provisions of SDPWS are permitted as an alternative to provisions in *IBC* Section 2305 (see “Seismic Requirements for Wood Building Design – Recent Changes to ASCE 7 and IBC” in this issue of Wood Design Focus). SDPWS provisions are generally consistent with design provisions of the *IBC* Section 2305; however, refinement of SDPWS provisions occurred over its three-year development cycle. As SDPWS continues to develop, it is expected that future editions of the *IBC* will either match provisions contained in SDPWS or be replaced by reference to SDPWS.

**Design Value Format**

In SDPWS, nominal design values are tabulated for shear wall and diaphragm shear capacities. For ASD, nominal design values are divided by an ASD reduction factor to establish an allowable design value. For LRFD, nominal design values are multiplied by the resistance factor, $\phi$, to establish a LRFD factored resistance value.

**Nominal, ASD, and LRFD Unit Shear Capacities for Shear Walls and Diaphragms**

The tabulated nominal unit shear capacity for wind, $(v_{nw})$, equals the *IBC* allowable stress design value times a factor of 2.8. A factor of 2.8, based on minimum performance requirements of *Performance Standard for Wood-Based Structural Use Panels, PS-2*, has commonly been considered the minimum safety factor associated with allowable unit shear values for wood structural panel shear walls and diaphragms. A reduced value equal to 1/1.4 times the nominal unit shear capacity for wind is assigned as the nominal unit shear capacity for seismic, $(v_s)$, maintaining the current ratio of wind and seismic design capacities for wood structural panel shear walls and diaphragms in the *IBC*.

For ASD, nominal unit shear capacities for shear walls and diaphragms are divided by an ASD reduction factor of 2.0. Resulting allowable unit shear capacities for wood structural panel shear walls and diaphragms are identical to those provided in the *IBC*. For LRFD, nominal unit shear capacities for shear walls and diaphragms are multiplied by $\phi = 0.80$. This value, $\phi = 0.80$, provides exact calibration to ASD for wind design. For seismic design, application of $\phi = 0.80$ coupled with reduced nominal unit shear values results in an effective $\phi = 0.57$ (e.g. $0.80/1.4 = 0.57$). When compared to ASD, LRFD will have up to a 12 percent benefit for seismic design for shear when using the following *IBC* load factors: (for LRFD: 1.0E or 1.6W; for ASD: 0.7E or 1.0W). This benefit is reduced where drift limits govern in design.

**Shear Wall and Diaphragm Deflection Equations**

The same tables that specify nominal unit shear capacities also contain values of apparent shear stiffness, $(G_a)$. Re-formatted deflection equations make use of these tabulated values of apparent shear stiffness to simplify deflection calculations and to extend deflection calculations to a variety of sheathing materials and unblocked wood structural panel diaphragms. The SDPWS three-term deflection equations are an algebraic simplification of the four-term equations in the *IBC*; however, SDPWS provides an option to calculate deflections using other methods such as the four-term equations. In SDPWS, total diaphragm deflection, $\delta_{dia}$, is calculated in accordance with Equation [1]:

$$\delta_{dia} = \frac{5vL^3}{8EAW} + \frac{0.25vL}{1000G_a} + \sum \frac{x\Delta_c}{2W}$$  \[1\]

where:

- $5vL^3/(8EAW)$ = deflection due to bending;
- $0.25vL/(1000G_a)$ = deflection due to shear deformation (from panel shear and nail slip); and
- $\Sigma(x\Delta_c)/(2W)$ = deflection due to chord splice slip.

Total shear wall deflection, $\delta_{sw}$, is calculated as shown in Equation [2]:

$$\delta_{sw} = \frac{8vh^3}{EAb} + \frac{vh}{1000G_a} + \frac{h\Delta_a}{b}$$  \[2\]
where:
\[ 8v^2/(EAb) = \text{deflection due to bending}; \]
\[ vh/(1000Ga) = \text{deflection due to shear deformation} \]
(from panel shear and nail slip); and
\[ h\Delta a/b = \text{deflection due to tie-down slip}. \]

**Apparent Shear Stiffness, \( G_a \)**

Tabulated \( G_a \) values, used to calculate the component of deflection due to shear deformation, replace the need for intermediate calculations that separately account for nail slip and panel shear stiffness. Equation [3] relates \( G_a \) to nail slip and panel shear stiffness:

\[
G_a = \frac{v}{G_v t_v + 0.75 e_n} \tag{3}
\]

where:
\( G_v t_v \) = panel shear stiffness, lb/inch of panel depth;
\( e_n \) = nail slip, inches; and
\( v = 1.4 \) times the ASD unit shear value of the shear wall or diaphragm, plf.

Identical values of calculated deflection are given by the three-term and four-term deflection equations at 1.4 times the ASD unit shear value (or, strength level force), as shown in Figure 1, for a wood structural panel shear wall. For the case shown, the maximum difference in calculated deflection is 0.045 inch. This small difference is not a significant factor in design; however, users should be aware that calculated deflection values will be different except at 1.4 times the ASD unit shear value.

**Perforated Shear Wall**

Limits on shear capacity are given in terms of nominal unit strength for single-sided and double-sided perforated shear walls. For single-sided walls, the nominal unit shear capacity shall not exceed 980 plf for seismic or 1370 plf for wind. For double-sided walls, the nominal unit shear capacity shall not exceed two times 980 plf or 1960 plf for seismic or 2000 plf for wind. The double-sided limit on nominal unit shear capacity for wind is not two times the single-sided value (e.g., \( 2 \times 1370 \text{ plf} = 2740 \text{ plf} \)) because test data for double-sided walls was limited to 2000 plf (APA 157). Nominal unit shear values must be divided by an ASD reduction factor or multiplied by the LRFD factor \( \phi \) to arrive at an appropriate design value.

**Adhesive Attachment of Shear Wall Sheathing**

SDPWS wording clarifies that adhesive attachment of shear wall sheathing shall not be used regardless of presence of mechanical fasteners except in SDC A, B, or C. Approved adhesive attachment systems are permitted in SDC A, B, and C for wind and seismic design; however, seismic design coefficients \( R \) and \( \Omega_o \) are limited to low values (\( R = 1.5 \) and \( \Omega_o = 2.5 \)) unless other values are approved. This tightens conditions under which adhesives are permitted compared to 2003 IBC where adhesive attachment is permitted in SDC A, B, or C without assignment of special seismic design coefficients (Table 1).

**Panel Widths for Shear Walls**

For wood-framed shear walls, which are required to have all panel edges nailed to blocking or framing, a minimum panel width dimension is not specified in SDPWS:

“4.3.7.1 …Panels shall not be less than 4 feet \( \times \) 8 feet, except at boundaries and changes in framing. Framing members or blocking shall be provided at the edges of all panels.”

SDPWS language provides construction flexibility by permitting use of other than full size panels in shear wall construction such as may occur in a wall that is 8 feet-6 inches tall or 4 feet-6 inches wide. Cyclic tests (APA T2005-08) of shear walls constructed with panel widths of 6 inches and 24 inches showed no significant differences in performance.

**Wood Structural Panel Over Lumber Planking or Laminated Decking**

Expanded provisions, compared to IBC Section 2305, are provided for minimum fastener penetration, aspect ratio, and requirements to maintain load path where wood structural panels are applied over solid lumber planking or laminated decking:
“4.2.7.1…Wood structural panel sheathing in a diaphragm is permitted to be fastened over solid lumber planking or laminated decking provided the following requirements are met:
a) panel edges do not coincide with joints in the lumber planking or laminated decking;
b) adjacent panel edges parallel to the planks or decking are fastened to a common member;
c) the planking or decking shall be of sufficient thickness to satisfy minimum fastener penetration in framing requirements as given in SDPWS Table 4.2A;
d) diaphragm aspect ratio (L/W) does not exceed that for a blocked wood structural panel diaphragm (4:1);
e) diaphragm forces are transferred to diaphragm boundary elements through planking or decking by other methods.”

**Forces Contributed by Concrete and Masonry Walls**

SDPWS Section 4.1.5 limitations on use of wood systems to resist forces contributed by concrete or masonry walls are similar to requirements in 2006 IBC; however, editorial revision clarifies requirements:

“4.1.5 Wood Members and Systems Resisting Seismic Forces Contributed by Masonry and Concrete Walls: Wood shear walls, diaphragms, trusses, and other wood members and systems shall not be used to resist seismic forces contributed by masonry or concrete walls in structures over one story in height.

Exceptions:
1. Wood floor and roof members shall be permitted to be used in diaphragms and horizontal trusses to resist horizontal seismic forces contributed by masonry or concrete walls provided such forces do not result in torsional force distribution through the truss or diaphragm.
2. Vertical wood structural panel sheathed shear walls shall be permitted to be used to provide resistance to seismic forces in two-story structures of masonry or concrete walls provided the following requirements are met…”

Since only forces contributed by concrete or masonry walls are addressed in 4.1.5, SDPWS Section 4.1.6 clarifies that wood members and systems must be designed when resisting forces from other structural and non-structural concrete or masonry construction:

“4.1.6 Wood Members and Systems Resisting Seismic Forces from Other Concrete or Masonry Construction: Wood members and systems shall be designed to resist seismic forces from other concrete or masonry components, including but not limited to: chimneys, fireplaces, concrete or masonry veneers, and concrete floors.”

Language “including but not limited to” is provided to avoid an exhaustive list of other concrete or masonry components or cement-based products such as roof tile and siding, which might be present in wood-frame construction.

**Summing Shear Capacities**

Provisions for summing shear capacities expand upon the current rule of thumb approach in IBC Section 2305. For similar materials and construction on opposite sides of a wall, nominal unit shear capacity is based on relative stiffness (and associated strength) of each side - consistent with general principles of analysis in ASCE 7 and IBC Chapter 16. The approach given in SDPWS Section 4.3.3.2 results in consistent treatment of strength and stiffness contribution whether a given panel is located on one side of a two-sided wall or whether the same panel is part of a one-sided wall segment located elsewhere in the wall line. For seismic design of walls sheathed on opposite sides with dissimilar materials, the strength-based rule of thumb is applied:

“4.3.3.2 Summing Shear Capacities…For shear walls sheathed with dissimilar materials on opposite sides, the combined nominal unit shear capacity, \(v_{sc}\) or \(v_{wc}\), shall be either two times the smaller nominal unit shear capacity or the larger nominal unit shear capacity, whichever is greater.”

Where design is based on the combined capacity of dissimilar materials on opposite sides as part of a wood-frame bearing wall system, the corresponding R factor equals 2 and the system is limited to SDC A-D (walls utilizing fiberboard capacity are limited to SDC A, B, and C).

**Conclusion**

New aspects of the 2005 SDPWS include simplified equations for determining shear wall and diaphragm deflection and the use of nominal design values to derive ASD and LRFD capacities for shear walls and diaphragms. It also includes stiffness properties for construction types other than wood structural panel shear walls and blocked diaphragms to assist designers and regulators attempting to ensure compliance with existing seismic criteria.

**References**


DOC PS2-92, Performance Standard for Wood-Based Structural Use Panels. United States Department of Commerce, National Institute of Standards and Technology, Gaithersburg, MD, 20899.

