ERRATA

to the 2018 and Prior Editions of

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

<table>
<thead>
<tr>
<th>Page</th>
<th>Revision</th>
</tr>
</thead>
<tbody>
<tr>
<td>91</td>
<td>Revise footnote 1 in Table 12.5.1D as follows:</td>
</tr>
</tbody>
</table>

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.

Ft’ = 525 psi (CF) = 525(1.5) = 788 psi
Fv’ = 150 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, \( C_g \), is taken as 1.0).

\[ Z_{\text{NT}}' = F_t' \left[ w - n_{\text{row}} D_h \right] \]
\[ Z_{\text{NT}}' = (788 \text{ psi})(1.5')(3.5" - 1(0.5625")) = 3,470 \text{ lbs} \]

Adjusted For side member, adjusted ASD Row Tear-Out Capacity, \( Z_{RT}' \):
\[ Z_{RT}' = n F_{v'} ts_{\text{critical}} \]
\[ Z_{RT}' = 3(150 \text{ psi})(1.5'')(2") = 1,350 \text{ lbs} \]

In this sample calculation, the adjusted ASD connection capacity is limited to 1,350 pounds by row tear-out, \( Z_{RT}' \).
March 2002

2002 ADDENDUM
to the
1997 NDS and PRIOR EDITIONS

The 2001 Edition of the National Design Specification® (NDS®) for Wood Construction contains editorially clarified provisions for checking stresses in members at connections. The following requirements, included in the 2001 NDS, are also applicable to all prior editions of the NDS:

Stresses in Members at Connections - Structural members shall be checked for load carrying capacity at connections in accordance with all applicable provisions of the NDS. Local stresses in connections using multiple fasteners shall be checked in accordance with principles of engineering mechanics.

One method for determining these stresses is provided in the attached Appendix E from the 2001 NDS. All referenced sections and design values used in sample solutions of this Addendum are based on information in the 2001 NDS.
Appendix E (Non-mandatory) Local Stresses in Fastener Groups

E.1 General

Where a fastener group is composed of closely-spaced fasteners loaded parallel to grain, the capacity of the fastener group may be limited by wood failure at the net section or tear-out around the fasteners caused by local stresses. One method to evaluate member strength for local stresses around fastener groups is outlined in the following procedures.

E.1.1 Tabulated nominal design values for timber rivet connections in Chapter 13 account for local stress effects and do not require further modification by procedures outlined in this Appendix.

E.1.2 The capacity of connections with closely-spaced, large diameter bolts has been shown to be limited by the capacity of the wood surrounding the connection. Connections with groups of smaller diameter fasteners, such as typical nailed connections in wood-frame construction, may not be limited by wood capacity.

E.2 Net Section Tension Capacity

The allowable tension capacity is calculated in accordance with provisions of 3.1.2 and 3.8.1 as follows:

\[ Z_{NT} = F'_{t} A_{net} \]  

(E.2-1)

where:

- \( Z_{NT} \) = allowable tension capacity of net section area
- \( F'_{t} \) = allowable tension design value parallel to grain
- \( A_{net} \) = net section area per 3.1.2

E.3 Row Tear-Out Capacity

The allowable tear-out capacity of a row of fasteners can be estimated as follows:

\[ Z_{RTi} = n_{i} \frac{F'_{v} A_{critical}}{2} \]  

(E.3-1)

where:

- \( Z_{RTi} \) = allowable row tear out capacity of row \( i \)
- \( n_{i} \) = number of fasteners in row \( i \)
- \( F'_{v} \) = allowable shear design value parallel to grain
- \( A_{critical} \) = minimum shear area of any fastener in row \( i \)

E3.1 Assuming one shear line on each side of bolts in a row (observed in tests of bolted connections), Equation E.3-1 becomes:

\[ Z_{RTi} = \frac{F'_{v} t}{2} [n_{i} S_{critical}] (n_{critical} shear lines) \]  

(E.3-2)

where:

- \( S_{critical} \) = minimum spacing in row \( i \) taken as the lesser of the end distance or the spacing between fasteners in row \( i \)
- \( t \) = thickness of member

The total allowable row tear-out capacity of multiple rows of fasteners can be estimated as:

\[ Z_{RT} = \sum_{i=1}^{n_{row}} Z_{RTi} \]  

(E.3-3)

where:

- \( Z_{RT} \) = allowable row tear out capacity of multiple rows
- \( n_{row} \) = number of rows

E3.2 In Equation E.3-1, it is assumed that the induced shear stress varies from a maximum value of \( f_{v} = F'_{v} \) to a minimum value of \( f_{v} = 0 \) along each shear line between fasteners in a row and that the change in shear stress/strain is linear along each shear line. The resulting triangular stress distribution on each shear line between fasteners in a row establishes an apparent shear stress equal to half of the design shear stress, \( F'_{v}/2 \), as shown in Equation E.3-1. This assumption is combined with the critical area concept for evaluating stresses in fastener groups and provides good agreement with results from tests of bolted connections.

E3.3 Use of the minimum shear area of any fastener in a row for calculation of row tear-out capacity is based on the assumption that the smallest shear area between fasteners in a row will limit the capacity of the row of fasteners. Limited verification of this approach is provided from tests of bolted connections.
**E.4 Group Tear-Out Capacity**

The allowable tear-out capacity of a group of “n” rows of fasteners can be estimated as:

\[
Z_{GT}' = \frac{Z_{RT-1}'}{2} + \frac{Z_{RT-n}'}{2} + F_i A_{\text{group-net}} \tag{E.4-1}
\]

where:

- \(Z_{GT}'\) = allowable group tear-out capacity
- \(Z_{RT-1}'\) = allowable row tear-out capacity of Row 1 of fasteners bounding the critical group area
- \(Z_{RT-n}'\) = allowable row tear-out capacity of Row n of fasteners bounding the critical group area
- \(A_{\text{group-net}}\) = critical group net section area between Row 1 and Row n

**E.4.1** For groups of fasteners with non-uniform spacing between rows of fasteners various definitions of critical group area should be checked for group tear-out in combination with row tear-out to determine the allowable capacity of the critical section.

**E.5 Effects of Fastener Placement**

**E 5.1** Modification of fastener placement within a fastener group can be used to increase row tear-out and group tear-out capacity limited by local stresses around the fastener group. Increased spacing between fasteners in a row is one way to increase row tear-out capacity. Increased spacing between rows of fasteners is one way to increase group tear-out capacity.

**E 5.2** Footnote 2 to Table 11.5.1D limits the spacing between outer rows of fasteners paralleling the member on a single splice plate to 5 inches. This requirement is imposed to limit local stresses resulting from shrinkage of wood members. When special detailing is used to address shrinkage, such as the use of slotted holes, the 5 inch limit can be adjusted.

**E.6 Sample Solution of Staggered Bolts**

Calculate the net section area tension, row tear-out, and group tear-out allowable design capacities for the double-shear bolted connection in Figure E1.

**Main Member:**
Combination 3 Douglas fir 3-1/8 x 12 glued laminated timber member
\(F'_{v} = 1450 \text{ psi}\)
\(F'_{t} = 240 \text{ psi}\)
Main member thickness, \(t_m\): 3.125 inches
Main member width, \(w\): 12 inches

**Side Member:**
A36 steel plates on each side
Side plate thickness, \(t_s\): 0.25 inches

**Connection Details:**
Bolt diameter, \(D\): 1 inch
Bolt hole diameter, \(D_h\): 1.0625 inches
Allowable bolt design value, \(Z_{||}'\): 4380 lbs. (see NDS Table 11I. For this trial design, the group action factor, \(C_g\), is taken as 1.0).
Spacing between rows: \(S_{row} = 2.5D\)

**Allowable Connection Capacity, \(nZ_{||}'\):**
\(nZ_{||}' = (8 \text{ bolts})(4380 \text{ lbs.}) = 35,040 \text{ lbs.}\)

**Figure E1 Staggered Rows of Bolts**

Allowable Net Section Area Tension Capacity, \(Z_{NT}':\)
\(Z_{NT}' = F_{v} [w - n_{row}D_h]\)
\(Z_{NT}' = (1450 \text{ psi})(3.125\")(12\" - 3(1.0625\")) = 39,930 \text{ lbs.}\)

Allowable Row Tear-Out Capacity, \(Z_{RT}':\)
\(Z_{RT}' = n F_{t} t S_{critical}\)
\(Z_{RT-1}' = 3(240 \text{ psi})(3.125\")(4\") = 9,000 \text{ lbs.}\)
Calculate the net section area tension and row tear-out allowable design capacities for the single-shear single-row bolted connection represented in Figure E2.

Main and Side Members:
#2 grade Hem-Fir 2x4 lumber
$F'_{v} = 788$ psi
$F'_{v} = 145$ psi
Main member thickness, $t_m$: 3.5 inches
Side member thickness, $t_s$: 1.5 inches
Main and side member width, $w$: 3.5 inches

Connection Details:
Bolt diameter, $D$: 1/2 inch
Bolt hole diameter, $D_h$: 0.5625 inches
Allowable bolt design value, $Z'_v$: 550 lbs. (See NDS Table 11A. For this trial design, the group action factor, $C_g$, is taken as 1.0).

Allowable Connection Capacity, $nZ'_v$:
$$nZ'_v = (3 \text{ bolts})(550 \text{ lbs.}) = 1,650 \text{ lbs.}$$

Allowable Net Section Area Tension Capacity, $Z_{NT}'$:
$$Z_{NT}' = F'_{v} [w - n_{row} D_h]$$

Allowable Group Tear-Out Capacity, $Z_{GT}'$:
$$Z_{GT}' = \frac{Z_{RT}' - 1}{2} + \frac{Z_{RT}' - 1}{2} + F'_v \{(n_{row} - 1)(S_{row} - D_h)\}$$

$$Z_{GT}' = (9,000 \text{ lbs.})/2 + (9,000 \text{ lbs.})/2 + (1450 \text{ psi})(3.125\text{")[(3-1)(2.5\text{-}1.0625\text{")\}] = 22,030 \text{ lbs.}$$

In this sample calculation, the connection capacity is limited to 22,030 pounds by group tear-out, $Z_{GT}'$.

**E.7 Sample Solution of Row of Bolts**

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

**Figure E2 Single Row of Bolts**

$Z_{RT}' = (788 \text{ psi})(1.5\text{")}[3.5\text{-}1(0.5625\text{")\}] = 3,470 \text{ lbs.}$

Allowable Row Tear-Out Capacity, $Z_{RT}'$:
$$Z_{RT}' = nF'_{v} tS_{critical}$$
$$Z_{RT}' = 3(145 \text{ psi})(1.5\text{")})(2\text{")\} = 1,310 \text{ lbs.}$$

In this sample calculation, the connection capacity is limited to 1,310 pounds by row tear-out, $Z_{RT}'$. 
E.8 Sample Solution of Row of Split Rings

Calculate the net section area tension and row tear-out allowable design capacities for the single-shear single-row split ring connection represented in Figure E3.

**Main and Side Members:**
- #2 grade Southern Pine 2x4 lumber
- $F'_t = 825 \text{ psi}$
- $F'_v = 175 \text{ psi}$
- Main member thickness, $t_m$: 1.5 inches
- Side member thickness, $t_s$: 1.5 inches
- Main and side member width, $w$: 3.5 inches

**Connection Details:**
- Split ring diameter, $D$: 2.5 inch (see Appendix K for connector dimensions)
- Allowable split ring design value, $P'_c$: 2730 lbs. (see NDS Table 12.2A. For this trial design, the group action factor, $C_g$, is taken as 1.0).

**Allowable Connection Capacity, $nP'_c$:**
$$nP'_c = (2 \text{ split rings})(2730 \text{ lbs.}) = 5,460 \text{ lbs.}$$

**Allowable Net Section Area Tension Capacity, $Z_{NT}'$:**
$$Z_{NT}' = F'_t A_{net}$$
$$Z_{NT}' = (825 \text{ psi})[5.25 \text{ in.}^2 - 1.5" (0.5625") - 1.1 \text{ in.}^2]$$
$$= 2,728 \text{ lbs.}$$

**Allowable Row Tear-Out Capacity, $Z_{RT}'$:**
$$Z_{RT}' = n P'_c \frac{A_{critical}}{2}$$
$$Z_{RT}' = [(2 \text{ connectors})(175 \text{ psi})/2] \times [(2 \text{ shear lines}) (0.375") (5.5") + (1 \text{ shear line}) (17.6 \text{ in.}^2)]$$
$$= 3,802 \text{ lbs.}$$

In this sample calculation, the connection capacity is limited to 2,728 pounds by net section area tension capacity, $Z_{NT}'$. 

![Figure E3: Single Row of Split Ring Connectors](image-url)
Page Revision

38–42  Revise Table 4D Design values as follows:

<table>
<thead>
<tr>
<th>Species and Commercial Grade</th>
<th>Size Classification</th>
<th>Design values in pounds per square inch (psi)</th>
<th>Grading Rules Agency</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Bending $F_b$</td>
<td>Tension $F_t$</td>
</tr>
<tr>
<td>DOUGLAS FIR-LARCH</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dense Select Structural</td>
<td>Beams and Stringers</td>
<td>1850 1900</td>
<td>1100</td>
</tr>
<tr>
<td>Dense No. 2 No. 2 Posts and Timbers</td>
<td>800 850 700 750</td>
<td>550</td>
<td>425 700</td>
</tr>
<tr>
<td>DOUGLAS FIR-SOUTH</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 2 Beams and Stringers</td>
<td>825</td>
<td>425</td>
<td>525 550</td>
</tr>
<tr>
<td>Select Structural No. 2 Posts and Timbers</td>
<td>1400 1450 650 675</td>
<td>950</td>
<td>1050</td>
</tr>
<tr>
<td>HEM-FIR</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Select Structural No. 1 Beams and Stringers</td>
<td>1250 1300 1050 675</td>
<td>725 750 525 325 350</td>
<td>925 725 425 500</td>
</tr>
<tr>
<td>No. 1 No. 2 Posts and Timbers</td>
<td>950 975 625 575</td>
<td>650 375</td>
<td>850 575</td>
</tr>
<tr>
<td>WESTERN CEDARS</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Select Structural Beams and Stringers</td>
<td>1150</td>
<td>700 675</td>
<td>875</td>
</tr>
<tr>
<td>No. 2 Posts and Timers</td>
<td>800 550</td>
<td>350</td>
<td>325 550</td>
</tr>
</tbody>
</table>

Future Updates will be available at www.awc.org
Shear Design Values for Lumber

Shear design values for lumber have recently been revised and approved by the American Lumber Standard Committee, in accordance with changes to ASTM Standard D 245, Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber. These new lumber shear design values are higher than earlier assigned values. To obtain more information on new lumber shear design values, contact any of the following agencies:

Canadian Wood Council
Southern Pine Inspection Bureau
West Coast Lumber Inspection Bureau
Western Wood Products Association

Design provisions, including requirements for shear design of lumber, are published by the American Forest & Paper Association (AF&PA) in the National Design Specification ® for Wood Construction (NDS®), an ANSI national consensus standard. NDS shear provisions are being revised in the next edition of the NDS in order to utilize new lumber shear design values. However, until revision of the NDS has been completed, 1997 NDS design provisions are only to be used with design values in the 1997 NDS Supplement: Design Values for Wood Construction, or similar values.

Until appropriate revisions to the NDS can be fully implemented, the American Forest & Paper Association recommends the following as guidance when using new shear design values with the 1997 or earlier editions of the NDS:

- The shear stress adjustment factor, C_H, for splits, checks, and shakes does not apply to the new shear design values.
- Tension-side notching equations (3.4-3 & 3.4-4) are only to be applied with 1997 NDS Supplement design values. These equations are being revised to permit the use of higher lumber shear design values. Until these design provisions are approved, tension-side notches should be designed using existing 1997 NDS design procedures and 1997 NDS Supplement design values.
- Provisions in 3.4.5 on shear design for bending members at connections, including equations (3.4-6 & 3.4-7), are only to be applied with 1997 NDS Supplement design values. These provisions are being revised to permit the use of higher lumber shear design values. Until these design provisions are approved, shear design for bending members at connections should be in accordance with existing 1997 NDS design procedures and 1997 NDS Supplement design values.
- Alternate design procedures in 4.4.2 do not apply to new shear design values.

For more information on the new shear design provisions for lumber, contact the American Wood Council (AWC) Helpdesk at 202/463-4713 or email awcinfo@afandpa.org.
32-33  Revise Table 4B as follows:

<table>
<thead>
<tr>
<th>Species and Commercial Grade</th>
<th>Size Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>SOUTHERN PINE</td>
<td></td>
</tr>
<tr>
<td>Construction Standard Utility²</td>
<td>2” - 4” thick 2” - 4” wide</td>
</tr>
<tr>
<td>MIXED SOUTHERN PINE</td>
<td></td>
</tr>
<tr>
<td>Construction Standard Utility²</td>
<td>2” - 4” thick 2” - 4” wide</td>
</tr>
</tbody>
</table>

5. Design values apply to 4” widths only.
2000 Errata
to
1999 Edition of

ASD STRUCTURAL-USE PANELS SUPPLEMENT to the
ALLOWABLE STRESS DESIGN (ASD) MANUAL FOR ENGINEERED WOOD
CONSTRUCTION

Page  Revision

4  Revise section 2.1 under Span Ratings as follows:

   Span ratings indicate a maximum recommended support spacing, in inches, for specific
   applications. The span rating system applies when the panel is applied with the strength axis
   across two three or more supports. The strength axis is usually the primary axis (which is usually
   the long dimension) of the panel.

5  Revise section 2.1 under Span Ratings as follows:

   Single Floor: The Single Floor span rating is an index number that provides the
   maximum recommended support spacing with the strength axis across two three or more
   supports. Typical Single Floor span ratings are 20 oc and 24 oc, although 16 oc, 32 oc, and 48 oc
   Single Floor panels are also available.

21  Revise Table 4.5 Panel Size Factor, C_s, as follows:

<table>
<thead>
<tr>
<th>Panel Width, w</th>
<th>C_s</th>
</tr>
</thead>
<tbody>
<tr>
<td>w≤8 inches</td>
<td>(8+w)</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
</tr>
<tr>
<td>8 inches &lt; w &lt; 24 inches</td>
<td>(8+w)</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
</tr>
<tr>
<td>w≥24 inches</td>
<td>1.0</td>
</tr>
</tbody>
</table>
2000 Errata
to
1999 Edition of

ASD STRUCTURAL CONNECTIONS SUPPLEMENT to the
ALLOWABLE STRESS DESIGN (ASD) MANUAL FOR ENGINEERED WOOD
CONSTRUCTION

<table>
<thead>
<tr>
<th>Page</th>
<th>Revision</th>
</tr>
</thead>
<tbody>
<tr>
<td>31</td>
<td>Revise Table 12.3A as follows:</td>
</tr>
</tbody>
</table>

**Table 12.3A Common Wire or Box Nail Design Values (Z) for Single Shear (two member) Structural Connections**

| Structural panel to lumber with structural panel side members with an effective G=0.50⁰ |
|---------------------------------|-----------------|
| Side Member Thickness t, inches | Penny Weight² |
|                                 | Common          | Box               |
| 1                               |                 |
| 6d²                            | 6d²             |
| 8d²                            | 8d              |
| 10d, 12d                       | 10d, 12d        |
| 1-1/8                           |                 |
| 6d²                            | 6d²             |
| 8d²                            | 8d              |
| 10d, 12d                       | 10d, 12d        |
| 1-1/4                           |                 |
| 6d²                            | 6d²             |
| 8d²                            | 8d              |
| 10d, 12d                       | 10d, 12d        |
| 10d ᵃ², 12d                    | 20d, 30d        |

Future Updates will be available at www.awc.org
Revise Table 12.3C as follows:

**Table 12.3C Common Wire and Box Nail Lengths, L, and Diameters, D**

<table>
<thead>
<tr>
<th>Common Wire Nail</th>
<th>30d L, in.</th>
<th>30d D, in.</th>
<th>40d L, in.</th>
<th>40d D, in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>4-1/2</td>
<td>0.225</td>
<td>0.207</td>
<td>5-1/2</td>
<td>0.235</td>
</tr>
</tbody>
</table>
5. Definition for $Z$ should be:

$$Z = \text{nominal lateral design value for a single bolt or lag screw wood-to-wood, wood-to-metal, or wood-to-concrete connection with all wood member(s) loaded perpendicular to grain, lbs.}$$

45. Table 7.3.3 Wet Service Factors, $C_M$, for Connections, should be revised as follows. (Note: only changes to table are shown)

<table>
<thead>
<tr>
<th>Fastener Type</th>
<th>Moisture Content At Time of Fabrication</th>
<th>Load In-Service</th>
<th>Lateral</th>
<th>Withdrawal</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bolts &amp; Drift Pins &amp;</td>
<td>any $\leq 19%$</td>
<td>$\leq 19%$</td>
<td>$1.0^3$</td>
<td>-</td>
</tr>
<tr>
<td>Drift Bolts &amp;</td>
<td>$&gt; 19%$</td>
<td></td>
<td>$0.4^3$</td>
<td>-</td>
</tr>
<tr>
<td>Lag Screws &amp; Wood Screw</td>
<td>any $\leq 19%$</td>
<td>$\leq 19%$</td>
<td>$1.0^3$</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>$&gt; 19%$</td>
<td>$\leq 19%$</td>
<td>$0.4^3$</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>any $&gt; 19%$</td>
<td>$&gt; 19%$</td>
<td>$0.7$</td>
<td>0.7</td>
</tr>
</tbody>
</table>

3. For two or more rows of bolts or lag screws with single steel side plate(s) installed in wood with moisture content $>19\%$ at time of fabrication and $\leq 19\%$ in service, $C_M = 0.4$.

3. $C_M = 1.0$ for wood screws. For bolt and lag screw connections with: 1) one fastener only, or 2) two or more fasteners placed in a single row parallel to grain, or 3) fasteners placed in two or more rows parallel to grain with separate splice plates for each row, $C_M = 1.0$. 
54 The second sentence of 8.2.3.1 should be revised as follows: "Table 8.2E provides nominal design values for various single shear bolted wood to concrete or masonry connections."

73 In Table 8.3D, “Z_{1/2ν/2}” should be “Z_{1/2}” and “Z_{X}” should be “Z_{X}”.

89 In Table 10.2B, the heading for the first column should be “Shear plate diameter”.

132 Section 13.1 - General, should be revised as follows:

“The design criteria for timber rivet joints apply to timber rivets that satisfy the requirements of 13.1.1 loaded in single shear, with steel side plates on Western Species Douglas Fir - Larch or Southern Pine glued laminated timber manufactured in accordance with ANSI/AITC A190.1-1992 (Reference 4).”

156 Appendix B, item (e) should be:

(e) Load duration factors greater than 1.6 shall not apply to structural members pressure-treated with water-borne preservatives (see Reference 29), or fire retardant chemicals. The impact load duration factor shall not apply to connections.

159 In Equation D-3, “F_{cE}” should be “F_{cE}”.


John “Buddy” Showalter, P.E., and Phil Line, P.E.

Introduction

The 1997 edition of the *National Design Specification (NDS)* for Wood Construction has been published by the American Forest & Paper Association’s (AF&PA) American Wood Council (AWC). This culminates 5 years of development by the wood products industry to provide state-of-the-art information for wood design, improve the design process, and more accurately reflect the field performance of wood members and connections. This latest edition of the NDS was approved as an American National Standard on August 7, 1997, and designated ANSI/AF&PA NDS-1997. The *NDS Supplement Design Values for Wood Construction*, an integral part of the NDS, has also been updated to provide the latest design values for lumber and glued laminated timber. Changes to the NDS include:

- new provisions for timber rivets;
- new provisions for wood-to-concrete connections;
- new provisions for designing notches on the compression face of a bending member;
- new incising adjustment factors;
- simplified wet service factors for connectors;
- new combined lateral/withdrawal equation for nails;
- modification of nail clinching provisions;
- clarification of built-up column provisions;
- revised upper limit on load duration factors for pressure-preservation and fire retardant-treated wood;
- update of lumber and glued laminated timber design values (*NDS Supplement*); and
- permissive language replaced with mandatory language.

**Timber Rivets**

Timber rivet connections have been used in Canada for decades. New design criteria introduced in Chapter 13 of the NDS apply to joints with steel side plates for glued laminated timber made from either Southern Pine or western species. Originally called the “glulam rivet,” the term “timber rivet” was chosen to accommodate future application to sawn lumber as well.

The high-strength nails used to attach steel side plates to wood members are rectangular in cross-section (Fig. 1) and must have a minimum ultimate tensile strength of 145,000 psi. Steel side plates must conform to ASTM A36 (ASTM 1996) and be at least 1/8-inch thick. Timber rivet connections are evaluated for both wood capacity and rivet capacity. Minimum end and edge distance and spacing are prescribed, as well as fabrication sequence. Connection capacities as high as 129,000 pounds can be developed using timber rivets. Joint configurations can be developed to allow parallel-to-grain loads, perpendicular-to-grain loads, or loads at an angle to the grain (Fig. 2).

**Wood-to-Concrete Connections**

New provisions have been incorporated for wood-to-concrete connections. In previous editions of the NDS, a designer assumed a main member thickness (concrete) equal to twice the thickness of the side member (wood), with the same dowel-bearing strength as that of the wood member. In section 8.2.3 of the 1997 NDS, connection values may be calculated specifically using dowel-bearing strengths for the wood member and the concrete. Design values are tabulated assuming a concrete dowel-bearing strength of 6,000 psi for concrete with 6 inches and greater embedment depth. Wood member sizes of 1.5, 2.5, and 3.5 inches are tabulated. This will increase bolt design values (Z) by up to 30 percent for some common wood-to-concrete connections (Table 1).

**Notches and Taper Cuts**

New provisions for compression side notches and taper cuts at the ends of bending members have been added to section 3.4.4.5 of the 1997 NDS. These provisions are based on similar provisions included in AITC’s *Timber Construc-
tion Manual (ATTC 1994) and apply to both glued laminated timber and dimension lumber. The new provisions recognize that a compression side notch is less severe than a tension side notch, and that a taper cut on the compression side is less severe than a notch on the compression side.

Adjustment Factors for Incised Lumber

New design value adjustment factors for timber and lumber that is incised to increase penetration of preservatives have been added to section 2.3.11 of the NDS. These provisions apply to structural sawn lumber with incisions made parallel-to-grain that have a maximum depth of 3/4-inch, a maximum length of 3/8-inch, and a maximum density of 357 incisions per square foot. Reductions are 5 percent for modulus of elasticity and 15 percent for bending, tension, and compression parallel-to-grain design values.

Wet Service Factors for Connectors

Table 7.3.3 of the 1997 NDS, which outlines wet service factors for connections, has been simplified from 23 to 15 cases. The technical committees of AF&PA and ANSI determined that the degree of accuracy being portrayed by showing, for example, a 0.75 factor for wood screws “exposed” to moisture in service versus a 0.67 factor for wood screws that are “wet” in service, was too precise. These factors, therefore, were combined to 0.7 for any in-service application where wood moisture content exceeds 19 percent. This change was made to all connectors included in the table. Factors for timber rivets were added to this table as well.

Combined Lateral/Withdrawal Equation for Nails

A new equation for calculating combined lateral/withdrawal design values for nails and spikes was developed consistent with those added to the 1991 NDS for wood screws and lag screws. Section 12.3.8 of the 1997 NDS incorporates this new provision.

Nail Clinching

Modification of nail clinching provisions to allow only a 1.75 increase instead of 2.0 for double shear nailed connections is included in section 12.3.3 of the latest edition of the NDS. Re-evaluation of the research used to develop the original provisions indicated that the increased capacity for clinched nails depended on the quality of the clinch (how far the nail was bent relative to 90°) and the direction of the clinch relative to grain orientation. The technical committees of AF&PA and ANSI determined that the 1.75 factor better reflected test data.

Built-Up Columns

Provisions for built-up columns in section 15.3 of the 1997 NDS have been updated to clarify the application of column stability coefficients for nailed and bolted built-up columns. Column stability coefficients, $K_f$, of 0.6 and 0.75 are applicable to the potential buckling direction perpendicular to the grain.
Figure 2.—Timber rivet joint configurations (AF&PA 1997).

Table 1.—Increases in bolt design values (Z) for single shear (two member) wood-to-concrete connections made with Southern Pine (G=0.55)\(^a\).

| \( t_m \) | \( t_s \) | \( D \) | \( Z_{||} \) | \( Z_L \) | \( Z_{||} \) | \( Z_L \) | Ratio | \( Z_{||} \) | \( Z_L \) |
|---|---|---|---|---|---|---|---|---|---|
| 6+ | 1.5 | 0.50 | 660 | 400 | 660 | 360 | 1.00 | 1.1 |
| 0.75 | 1,270 | 660 | 1,270 | 500 | 1.00 | 1.3 |
| 1.00 | 2,140 | 760 | 1,740 | 580 | 1.23 | 1.3 |

\(^a\) Excerpted from Table 8.2E of the 1997 NDS; \( t_m \) equals embedment depth in concrete, \( t_s \) equals thickness of wood member, \( D \) equals bolt diameter.

Parallel to the weak axis of individual members for nailed and bolted built-up columns, respectively. A column stability coefficient of 1.0 is used for the potential buckling direction parallel to the strong axis of individual members.

Load Duration Factors for Treated Lumber

Recent research at the USDA Forest Products Laboratory, Madison, Wisconsin, indicated that the load duration factor for pressure-preservative treated wood should be limited to 1.6 or less. As a result, footnote 2 of NDS Table 2.3.2 now states “Load duration factors greater than 1.6 shall not apply to structural members pressure-treated with waterborne preservatives, or fire retardant chemicals. The impact load duration factor shall not apply to connections.” Frequently used load duration factors for wood design are presented in Table 2.3.2 of the NDS.

Design Values for Lumber and Glued Laminated Timber

Design values for lumber and glued laminated timber have been revised to reflect current data from the wood industry. Changes, for the most part, are minor and include rounding increases; new machine stress-rated (MSR) lumber and mechanically evaluated lumber (MEL) grades; higher specific gravity (G), horizontal shear (\( F_h \)), and compression perpendicular-to-grain (\( F_{\perp} \)) design values for certain MSR and MEL grades; and new glued laminated timber combinations and design values.

New grades of MSR lumber include 1700f-1.6E, 1750f-2.0E, 2250f-1.7E, and 2250f-1.8E. MSR grades that were eliminated include 3150f-2.5E and 3300f-2.6E, and all 6-inch and wider sizes. New grades of MEL include M-5 through M-9, M-28, and M-29. Additionally, design values for certain species and grades of MSR and MEL have increased (Table 2).

New Southern Pine glued laminated timber combinations include 26F visual rated, and 28F and 30F E-rated. Also, compression perpendicular-to-grain values for most Southern Pine glued laminated timber combinations have increased from 650 psi to 740 psi. Horizontal shear values for certain Southern Pine glued laminated timber combinations have increased from 200 psi to 240 psi, and those for Douglas Fir-Larch from 165 psi to 190 psi.

Mandatory Language

Of the 130 changes balloted to the ANSI committee for the NDS, approximately two-thirds requested modification of NDS language to make the document more suitable for building code enforcement. Some building code policies require mandatory language rather than permissive statements to ensure enforceability. Therefore, terms like “may” were changed to “shall” in various sections of the NDS to accommodate this requirement. Much of the background information contained in the NDS was moved to the NDS Commentary.
Table 2.—Increases in design values for MEL and MSR lumber.

<table>
<thead>
<tr>
<th>Species</th>
<th>Grade</th>
<th>$G$</th>
<th>$F_{c,l}$</th>
<th>$F_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spruce-Pine-Fir</td>
<td>$E \geq 2,000,000$</td>
<td>0.50</td>
<td>0.42</td>
<td>615</td>
</tr>
<tr>
<td>Southern Pine</td>
<td>$E \geq 1,900,000$</td>
<td>0.57</td>
<td>0.55</td>
<td>425</td>
</tr>
<tr>
<td>Engelmann Spruce-Lodgepole Pine</td>
<td>$F_b \geq 1650f$</td>
<td>0.46</td>
<td>0.38</td>
<td>805</td>
</tr>
</tbody>
</table>

**NDS Commentary**

The *Commentary* to the NDS is being updated to reflect changes in the 1997 edition. An addendum to the *NDS Commentary* should be available in 1998.

**WoodWorks® Software**

WoodWorks Sizer has been updated to incorporate the latest provisions of the 1997 NDS. New features of Sizer 97 include:

- design for fire resistance in accordance with AWC's *Design of Fire-Resistive Exposed Wood Members—DCA No. 2*;
- ability to design multi-span joists;
- viewing results anywhere along a member length;
- printout of materials list;
- walls supported by joist areas away from bearing supports; and
- output for reactions on foundations.

Sizer is available on CD-ROM as part of the WoodWorks Design Office 97 package. Additional features of WoodWorks include:

- Sizer—designs beams, columns, and beam-columns;
- Connections—designs bolted and nailed connections;
- Shearwalls—designs traditional and perforated shearwalls for high wind applications;
- 1997 NDS in Adobe® Acrobat® PDF format—electronic version of the NDS;
- Code Conforming Wood Design—determines allowable heights and areas for wood structures; and
- Adobe Acrobat Reader—reads PDF files (for the electronic version of the NDS).

The NDS, NDS Supplement Design Values for Wood Construction, and NDS Commentary are available from AWC and can be ordered by calling 1-800-890-7732. WoodWorks Design Office 97 can be ordered by calling 1-800-844-1275. A demonstration copy of WoodWorks software can be downloaded from AWC's website at www.awc.org. Contact the AWC Technical Inquiry Clearinghouse at 1-800-AWC-AFPA or email awcinfo@afandpa.org for more information.

**Literature Cited**


John "Buddy" Showalter, P.E., Director, Technology Transfer, and Phil Line, P.E., Manager, Engineering Research, American Forest & Paper Association, Washington, D.C. This article is a slightly modified version of an article that originally appeared in the January 1998 issue of Frame Building News, the official publication of the National Frame Builders Association.

**News**

**Leading Engineering Societies Worldwide Develop First International Conference for Structural Engineers**

Six worldwide structural engineering related organizations have come together to develop the first global conference for structural engineers. The Structural Engineers World Congress (SEWC) will take place July 19-23, 1998, at the San Francisco Marriott in the heart of San Francisco's business district. SEWC will feature a comprehensive technical program, addressing every major structural issue from seismic design to state-of-the-art materials.

SEWC also boasts the largest number of sessions devoted to the business and practice of structural engineering ever presented at one conference—30 sessions and more than 100 papers. The congress will also feature the largest exhibition of suppliers of structural materials and systems and computer modeling programs ever organized for practicing structural engineers. The conference is designed to be one of the most user-friendly ever, featuring a take-home package that includes a CD-