AMERICAN WOOD COUNCIL

January 2021

ERRATA
to the 2018 and Prior Editions of
the National Design Specification® (NDS®) for Wood Construction

Page **Revision**

91 Revise footnote 1 in Table 12.5.1D as follows:

1. The ℓ/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 = ℓ_m/D
   (b) total length of fastener in wood side member(s)/D = ℓ_s /D

222 Catoctin Circle, SE, Suite 201 ▪ Leesburg, VA 20175 ▪ 202-463-2766 ▪ www.awc.org ▪ info@awc.org

AMERICAN WOOD COUNCIL

July 2020

ERRATA
to the 2018 and Prior Editions of
the National Design Specification® (NDS®) for Wood Construction

Page	**Revision**
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 C_D, C_T, C_M, and C_i are assumed to equal 1.0 in this example for calculation of adjusted design values.

$F_t' = 525 \text{ psi } (C_F) = 525(1.5) = 788 \text{ psi}$

$F_v' = 150 \text{ psi}$

Connection Details:
Bolt diameter, D: 1/2 in.
Bolt hole diameter, D_h: 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).

Adjusted ASD Connection Capacity, n $Z_{||}'$:

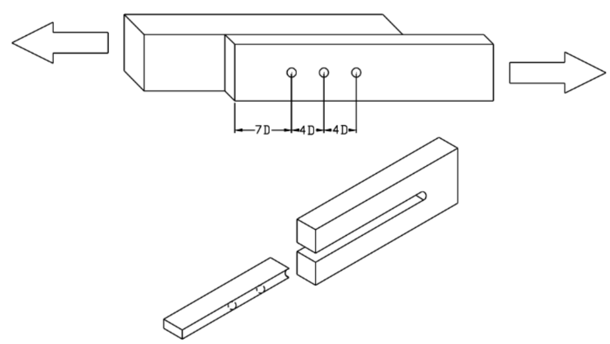
$nZ_{||}' = (3 \text{ bolts})(550 \text{ lbs}) = 1{,}650 \text{ lbs}$

~~Adjusted~~ For side member, adjusted ASD Net Section Area Tension Capacity, Z_{NT}':

$\mathbf{Z_{NT}' = F_t' \, t \, [w - n_{row} D_h]}$

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

Figure E2 Single Row of Bolts



~~Adjusted~~ For side member, adjusted ASD Row Tear-Out Capacity, Z_{RT}':

$\mathbf{Z_{RTi}' = n_i F_v' \, t s_{critical}}$

$Z_{RT1}' = 3(150 \text{ psi})(1.5'')(2'') = 1{,}350 \text{ lbs}$

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

A F & P A

AMERICAN FOREST & PAPER ASSOCIATION
American Wood Council
Engineered and Traditional Wood Products

November 2009

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)

Page(s)	Revision	
64	$Defl_{Max}$ = span · 12 in/ft / ~~240~~ 360	change 240 to 360 in the denominator
	$Defl_{Max}$ = ~~0.9~~ 0.6 in	
65	$Defl_{Max}$ = span · 12 in/ft / ~~240~~ 360	change 240 to 360 in the denominator
	$Defl_{Max}$ = ~~0.9~~ 0.6 in	
66	I_{reqd} = ~~1037~~ 1556 in^4	
	Minimum required I is ~~1037~~ 1556 in^4 for deflection	
67	I_{reqd} = ~~1037~~ 1556 in^4	
	Minimum required I is ~~1037~~ 1556 in^4 for deflection	

1111 Nineteenth Street, NW, Suite 800 - Washington, DC 20036 - 202 463-4713 - Fax: 202 463-2791 - www.awc.org

AF&PA®

AMERICAN FOREST & PAPER ASSOCIATION
American Wood Council
Engineered and Traditional Wood Products

September 2009

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	**Revision**
60	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, $\mathbf{F_{vx}}$ and $\mathbf{F_{vy}}$, 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 <u>and 10.1.2</u>) 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).

1111 Nineteenth Street, NW, Suite 800 - Washington, DC 20036 - 202 463-4713 - Fax: 202 463-2791 - www.awc.org

A F & P A®

AMERICAN FOREST & PAPER ASSOCIATION
American Wood Council
Engineered and Traditional Wood Products

July 2009

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)

Page	Revision
249	Revise equation C15.1-1 as follows:

$$P_{14} = -180.7 + 140.5 \log(P_m)$$

AF&PA
AMERICAN FOREST & PAPER ASSOCIATION
American Wood Council
Engineered and Traditional Wood Products

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)

Page(s)	Revision	
264	$M_{Total} = M_{load}$ + Weight · Length^2 / 8 · ~~12~~	delete "12" in the denominator
	f_b = ~~1695~~ 1778 psi	
265	$M_{Total} = M_{load}$ + 1.2(Weight · Length^2) / 8 · ~~12~~	delete "12" in the denominator
	f_b = ~~2575~~ 2674 psi	
274	$M_{Total} = M_{load}$ + Weight · Length^2 / 8 · ~~12~~	delete "12" in the denominator
	f_b = ~~1352~~ 1401 psi	
275	$M_{Total} = M_{load}$ + 1.2(Weight · Length^2) / 8 · ~~12~~	delete "12" in the denominator and incorporate 1.2 load factor
	f_b = ~~2145~~ 2204 psi	

AMERICAN FOREST & PAPER ASSOCIATION
American Wood Council
Engineered and Traditional Wood Products

September 2008

2008 ERRATA/ADDENDUM
to the

2005 Edition of

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:

$$\frac{f_c}{F_{cE2}} + \left(\frac{f_{b1}}{F_{bE}}\right)^2 < 1.0$$

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

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 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 $(f_c/F_{cE2}) + (f_{b1}/F_{bE})^2$ 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:

$$\frac{f_c}{F_{cE2}} + \left(\frac{f_{b1} + f_c(6e_1/d_1)}{F_{bE}}\right)^2 < 1.0$$

1111 Nineteenth Street, NW, Suite 800 - Washington, DC 20036 - 202 463-4713 - Fax: 202 463-2791 - www.awc.org

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 $(f_c/F_{cE2}) + ([f_{b1}+ 6f_ce_1/d_1]/F_{bE})^2$ 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.

AMERICAN FOREST & PAPER ASSOCIATION
American Wood Council
Engineered and Traditional Wood Products

September 2008

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)

Page(s)	Revision	
47	In Example C4.2.2-1 revise "Nail load/slip at 1.4 $v_{s(ASD)}$:"	Subscript (ASD)
	In Example C4.2.2-1 revise "= $(129.5/456)^{3.144}$ = 0.0191"	Superscript 3.144
50	In Equations C4.3.2-1 and C4.3.2-2 revise "(tie-down ~~nail~~ slip)"	Delete "nail"
	In the third paragraph of C4.3.2 (second column) revise "…or 3-term equation (SDPW~~D~~S)…"	Change "D" to "S"
53	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.	

1111 Nineteenth Street, NW, Suite 800 - Washington, DC 20036 - 202 463-4713 - Fax: 202 463-2791 - www.awc.org

AMERICAN FOREST & PAPER ASSOCIATION
American Wood Council
Engineered and Traditional Wood Products

September 2008

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)

Page(s)	**Revision**
155	Revise Table M16.2-10 column headers for decking width as follows:

Rating	1-HOUR				1.5-HOUR			2-HOUR	
Decking Width	~~5~~1.5	~~7~~2.5	~~9~~3.5	~~11~~5.5	~~7~~2.5	~~9~~3.5	~~11~~5.5	~~9~~3.5	~~11~~5.5

1111 Nineteenth Street, NW, Suite 800 - Washington, DC 20036 - 202 463-4713 - Fax: 202 463-2791 - www.awc.org
America's Forest & Paper People® - *Improving Tomorrow's Environment Today ®*

AMERICAN FOREST & PAPER ASSOCIATION
American Wood Council
Engineered and Traditional Wood Products

April 2008

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)

Page(s)	Revision
64	Figure M9.2-4 should be renumbered M9.2-5 Figure M9.2-5 should be renumbered M9.2-4

A F & P A
AMERICAN FOREST & PAPER ASSOCIATION
American Wood Council
Engineered and Traditional Wood Products

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)

Page(s)	Revision	
72	C_{M_Fb} := ~~0.85~~ 1.0	Change wet service factor. According to the *NDS Supplement*, when $(F_b)(C_F) \leq 1150$ psi $C_M = 1.0$
	F'_{bx} = ~~1124~~ 1322 psi	This change does not affect the outcome - bending still checks.
73	C_{M_Fb} := ~~0.85~~ 1.0	Change wet service factor. According to the *NDS Supplement*, when $(F_b)(C_F) \leq 1150$ psi $C_M = 1.0$
	F'_{bx} = ~~2428~~ 2856 psi	This change does not affect the outcome - bending still checks.

AF&PA
AMERICAN FOREST & PAPER ASSOCIATION
American Wood Council
Engineered and Traditional Wood Products

October 2007

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)

Page(s)	Revision
91	$\lambda = \cancel{1.0}\ \underline{0.8}$ Time effect factor
93	$F'_{bS\_24\_0} = \cancel{648}\ \underline{518}$ lbf·in $F'_{bS\_24\_16} = \cancel{832}\ \underline{665}$ lbf·in $F'_{bS\_32\_16} = \cancel{961}\ \underline{769}$ lbf·in $F'_{bS\_40\_20} = \cancel{1620}\ \underline{1296}$ lbf·in $F'_{bS\_48\_24} = \cancel{2160}\ \underline{1728}$ lbf·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. $F_{s\_IbQ} := \cancel{150}\ \underline{205}$ lbf $F'_{s\_IbQ} = \cancel{324}\ 354$ lbf **The ~~24/16~~ 40/20 span rating is OK in shear.** $EI_{40\_20} := \cancel{78000}\ \underline{225000}\ \text{lbf}\cdot\text{in}^2$ — EI = 225000 ~~78000~~ lbf in^2 per foot of width for ~~24/16~~ 40/20 span rating. Table M9.2-1. $EI'_{40\_20} := \cancel{78000}\ \underline{225000}\ \text{lbf}\cdot\text{in}^2$ — ~~24/16~~ 40/20 bending stiffness
96	$\Delta < L/240 = \cancel{0.95}\ \underline{0.095}$ in, thus deflection is OK
97	$\Delta = 0.00677\ w_{load}\ span_{defl}^{4} / EI'_{40\_20}$ $\Delta = \cancel{0.213}\ \underline{0.074}$ in

1111 Nineteenth Street, NW, Suite 800 - Washington, DC 20036 - 202 463-4713 - Fax: 202 463-2791 - www.awc.org

AMERICAN FOREST & PAPER ASSOCIATION
American Wood Council
Engineered and Traditional Wood Products

May 2007

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)

Page(s)	Revision
376	$\text{Weight}_{curved} = \pi \cdot R_{center}\cancel{/2} \cdot \text{Depth} \cdot \text{Width} \cdot 40 \cdot \text{lbf/ft}^3$ (delete “2” in the denominator)
	$\text{Weight}_{curved} = \cancel{353}\ \underline{706}$ lbf
	$M_{max} = \cancel{66244}\ \underline{97288}$ in·lbf
377	$\text{Weight}_{curved} = \pi \cdot R_{center}\cancel{/2} \cdot \text{Depth} \cdot \text{Width} \cdot 40 \cdot \text{lbf/ft}^3$ (delete “2” in the denominator)
	$\text{Weight}_{curved} = \cancel{353}\ \underline{706}$ lbf
	$M_{max} = \cancel{92742}\ \underline{136204}$ in·lbf
380	$f_b = \cancel{343}\ \underline{504}$ psi
	$V_{load} = \cancel{376}\ \underline{553}$ lbf
	$f_v = \cancel{5}\ \underline{8}$ psi
	$f_r = \cancel{10}\ \underline{15}$ psi
381	$f_b = \cancel{481}\ \underline{706}$ psi
	$V_{load} = \cancel{527}\ \underline{774}$ lbf
	$f_v = \cancel{7}\ \underline{11}$ psi
	$f_r = \cancel{14}\ \underline{21}$ psi

1111 Nineteenth Street, NW, Suite 800 - Washington, DC 20036 - 202 463-4713 - Fax: 202 463-2791 - www.awc.org

384 $Weight_{curved} = \pi \cdot R_{center}\cancel{/2} \text{ Depth} \cdot \text{Width} \cdot 40 \cdot \text{lbf/ft}^3$ (delete "2" in the denominator)

$Weight_{curved} = \cancel{353}\ \underline{706}$ lbf

$M_{max} = \cancel{66244}\ \underline{97288}$ in·lbf

$P_{max} = \cancel{553}\ \underline{906}$ lbf

385 $Weight_{curved} = \pi \cdot R_{center}\cancel{/2} \text{ Depth} \cdot \text{Width} \cdot 40 \cdot \text{lbf/ft}^3$ (delete "2" in the denominator)

$Weight_{curved} = \cancel{353}\ \underline{706}$ lbf

$M_{max} = \cancel{92742}\ \underline{136204}$ in·lbf

$P_{max} = \cancel{774}\ \underline{1268}$ lbf

388 $f_c = \cancel{5}\ \underline{8}$ psi

389 $f_c = \cancel{7}\ \underline{11}$ psi

390 $f_{b1} = \cancel{343}\ \underline{504}$ psi

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

391 $f_{b1} = \cancel{481}\ \underline{706}$ psi

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

392 $\left(\frac{f_c}{F_c'}\right)^2 = \cancel{0.00056}\ \underline{0.00151}$ $\quad \frac{f_{b1}}{F_b'\left[1-\left(\frac{f_c}{F_{cE1}}\right)\right]} = \cancel{0.313}\ \underline{0.467}$

393 $\left(\frac{f_c}{F_c'}\right)^2 = \cancel{0.00049}\ \underline{0.00132}$ $\quad \frac{f_{b1}}{F_b'\left[1-\left(\frac{f_c}{F_{cE1}}\right)\right]} = \cancel{0.304}\ \underline{0.453}$

AF&P A
AMERICAN FOREST & PAPER ASSOCIATION
American Wood Council
Engineered and Traditional Wood Products

March 2007

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)

Page(s)	Revision
276	$F'_v = \cancel{2/3}\ F_{vx}\ C_D\ C_M\ C_{t\_Fv}$
	$F'_v =$ ~~200~~ 300 psi
	This change does not affect the final results for shear calculations since $f_v < F'_v$.
277	$F'_v = \cancel{2/3}\ \lambda\ K_{F\_Fv}\ \phi_v\ F_{vx}\ C_M\ C_{t\_Fv}$
	$F'_v =$ ~~345.6~~ 518.4 psi
	This change does not affect the final results for shear calculations since $f_v < F'_v$.

1111 Nineteenth Street, NW, Suite 800 - Washington, DC 20036 - 202 463-4713 - Fax: 202 463-2791 - www.awc.org
America's Forest & Paper People® - Improving Tomorrow's Environment Today ®

AMERICAN FOREST & PAPER ASSOCIATION
American Wood Council
Engineered and Traditional Wood Products

February 2007

2007 ERRATA
to the

2005 Edition of

the *ASD/LRFD MANUAL FOR ENGINEERED WOOD CONSTRUCTION*
included in the *2005 Wood Design Package*
(printed version dated 08-06 5M)

Page(s)	Revision
126	Change the IIC rating for assemblies "Without Gypsum Concrete" with "Carpet & Pad" from ~~62~~ to 66.
129	Change the STC rating for assemblies "With Gypsum Concrete" with "Carpet & Pad" from ~~68~~[c] to 58[c].

February 2007

2007 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, and 08-06 5M)

Page **Revision**

4 In Table 2.1, revise the following:

Species or Species Combination	Species That May Be Included in Combination	Grading Rules Agencies	Design Values Provided in Tables
Coast Sitka Spruce	Coast Sitka Spruce	NLGA	4A, 4D, 4E
Yellow Cedar	Yellow Cedar	NLGA	4A

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

Species and commercial grade	Size classification	Design values in pounds per square inch (psi)							Grading Rules Agency
		Bending F_b	Tension parallel to grain F_t	Shear parallel to grain F_v	Compression perpendicular to grain $F_{c\perp}$	Compression parallel to grain F_c	Modulus of Elasticity E	Modulus of Elasticity E_{min}	
Coast Sitka Spruce[4]									
Select Structural	2" & wider	1300	950	125	455	1200	1,700,000	620,000	NLGA
No. 1 / No. 2		925	550	125	455	1100	1,500,000	550,000	
No. 3		525	325	125	455	625	1,400,000	510,000	
Stud	2" & wider	725	450	125	455	675	1,400,000	510,000	
Construction	2"-4" wide	1050	650	125	455	1300	1,400,000	510,000	
Standard		600	350	125	455	1100	1,300,000	470,000	
Utility		275	175	125	455	725	1,200,000	440,000	

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

Species and commercial grade	Size classification	Design values in pounds per square inch (psi)							Grading Rules Agency
		Bending F_b	Tension parallel to grain F_t	Shear parallel to grain F_v	Compression perpendicular to grain $F_{c\perp}$	Compression parallel to grain F_c	Modulus of Elasticity E	Modulus of Elasticity E_{min}	
Northern Species									
Select Structural	2" & wider	~~1,000~~ 975	~~450~~ 425	110	350	1,100	1,100,000	400,000	NLGA
No. 1 / No. 2		~~600~~ 625	275	110	350	850	1,100,000	400,000	
No. 3		350	150	110	350	500	1,000,000	370,000	
Stud	2" & wider	475	225	110	350	550	1,000,000	370,000	
Construction	2"-4" wide	700	~~300~~ 325	110	350	1,050	1,000,000	370,000	
Standard		400	175	110	350	875	900,000	330,000	
Utility		175	75	110	350	575	900,000	330,000	

Page **Revision**

36 In Table 4A, add the following design values for Yellow Cedar

Species and commercial grade	Size classification	Design values in pounds per square inch (psi)							
		Bending F_b	Tension parallel to grain F_t	Shear parallel to grain F_v	Compression perpendicular to grain $F_{c\perp}$	Compression parallel to grain F_c	Modulus of Elasticity E	Modulus of Elasticity E_{min}	Grading Rules Agency
Yellow Cedar[4]									
Select Structural	2" & wider	1200	725	175	540	1200	1,600,000	580,000	NLGA
No. 1 / No. 2	2" & wider	800	475	175	540	1000	1,400,000	510,000	NLGA
No. 3	2" & wider	475	275	175	540	575	1,200,000	440,000	NLGA
Stud	2" & wider	625	375	175	540	650	1,200,000	440,000	NLGA
Construction	2"-4" wide	925	550	175	540	1200	1,300,000	470,000	NLGA
Standard	2"-4" wide	525	300	175	540	1050	1,200,000	440,000	NLGA
Utility	2"-4" wide	250	150	175	540	675	1,100,000	400,000	NLGA

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.

Species	**Specific Gravity, G**	**Grading Rules Agency**
Coast Sitka Spruce	0.43	NLGA
Yellow Cedar	0.46	NLGA

AMERICAN FOREST & PAPER ASSOCIATION
American Wood Council
Engineered and Traditional Wood Products

February 2007

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)

Page(s)	Revision
6	x = distance from chord splice to nearest support, ~~in.~~ <u>ft</u>
13	x = distance from chord splice to nearest support, ~~in.~~ <u>ft</u>
23	C_o = shear capacity adjustment factor from Table ~~4.3.2.1~~ <u>4.3.3.4</u>
44	x = distance from chord splice to nearest support, ~~in.~~ <u>ft</u>
46	x = distance from chord splice to nearest support, ~~in.~~ <u>ft</u>

AF&PA

AMERICAN FOREST & PAPER ASSOCIATION
American Wood Council
Engineered and Traditional Wood Products

January 2007

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)

Page(s)	**Revision**
120-121	The third part (minor axis bending component) of the biaxial bending and compression interaction equation should be revised as follows:

$$\frac{f_{b2}+f_c(6e_2/d_2)\left\{1+0.234(f_c/F_{cE2})+0.234\left[\frac{f_{b1}+f_c(6e_1/d_1)}{F_{bE}}\right]^2\right\}}{F_{b2}'\left\{1-(f_c/F_{cE2})-\left[\frac{f_{b1}+f_c(6e_1/d_1)}{F_{bE}}\right]^2\right\}}\le 1.0$$

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:

$$\left(\frac{f_c}{F_c'}\right)^2+\frac{f_{b1}+f_c(6e_1/d_1)[1+0.234(f_c/F_{cE1})]}{F_{b1}'\left[1-(f_c/F_{cE1})\right]}+\frac{f_{b2}+f_c(6e_2/d_2)\left\{1+0.234(f_c/F_{cE2})+0.234\left[\frac{f_{b1}+f_c(6e_1/d_1)}{F_{bE}}\right]^2\right\}}{F_{b2}'\left\{1-(f_c/F_{cE2})-\left[\frac{f_{b1}+f_c(6e_1/d_1)}{F_{bE}}\right]^2\right\}} = \cancel{0.945}\ \underline{0.927}$$

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).

$$\frac{f_{b2}+f_c(6e_2/d_2)\left\{1+0.234(f_c/F_{cE2})+0.234\left[\frac{f_{b1}+f_c(6e_1/d_1)}{F_{bE}}\right]^2\right\}}{F_{b2}'\left\{1-(f_c/F_{cE2})-\left[\frac{f_{b1}+f_c(6e_1/d_1)}{F_{bE}}\right]^2\right\}} = \cancel{0.119}\ \underline{0.101}$$

Page	**Revision**
133	The third part (minor axis bending component) of the biaxial bending and compression interaction equation should be revised as follows:

$$\left(\frac{f_c}{F_c'}\right)^2 + \frac{f_{b1} + f_c(6e_1/d_1)[1+0.234(f_c/F_{cE1})]}{F_{b1}'\left[1-(f_c/F_{cE1})\right]} + \frac{f_{b2} + f_c(6e_2/d_2)\left\{1+0.234(f_c/F_{cE2})+0.234\left[\frac{f_{b1}+f_c(6e_1/d_1)}{F_{bE}}\right]^2\right\}}{F_{b2}'\left\{1-(f_c/F_{cE2})-\left[\frac{f_{b1}+f_c(6e_1/d_1)}{F_{bE}}\right]^2\right\}} = \cancel{0.785}\ \underline{0.772}$$

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).

$$\frac{f_{b2} + f_c(6e_2/d_2)\left\{1+0.234(f_c/F_{cE2})+0.234\left[\frac{f_{b1}+f_c(6e_1/d_1)}{F_{bE}}\right]^2\right\}}{F_{b2}'\left\{1-(f_c/F_{cE2})-\left[\frac{f_{b1}+f_c(6e_1/d_1)}{F_{bE}}\right]^2\right\}} = \cancel{0.156}\ \underline{0.142}$$

284-285	The third part (minor axis bending component) of the biaxial bending and compression interaction equation should be revised as follows:

$$\frac{f_{b2} + f_c(6e_2/d_2)\left\{1+0.234(f_c/F_{cE2})+0.234\left[\frac{f_{b1}+f_c(6e_1/d_1)}{F_{bE}}\right]^2\right\}}{F_{b2}'\left\{1-(f_c/F_{cE2})-\left[\frac{f_{b1}+f_c(6e_1/d_1)}{F_{bE}}\right]^2\right\}} \le 1.0$$

The square symbol in the denominator goes between the last 2 brackets in the equation.

2005 National Design Specification® (NDS®) for Wood Construction

Phil Line, P.E., John "Buddy" Showalter, P.E., and Robert J. Taylor, Ph.D., P.Eng.

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_{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 $f_b \le F_b'$. 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_b, F_t, F_v, F_c, $F_{c\perp}$, E, E_{min}) are multiplied by adjustment factors to determine adjusted design values (F_b', F_t', F_v', F_c', $F_{c\perp}'$, E', E_{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:

Table 4.3.1 Applicability of Adjustment Factors for Sawn Lumber.

		ASD only	ASD and LRFD										LRFD only		
		Load duration factor	Wet service factor	Temperature factor	Beam stability factor	Size factor	Flat use factor	Incising factor	Repetitive member factor	Column stability factor	Buckling stiffness factor	Bearing area factor	Format conversion factor	Resistance factor	Time effect factor
$F_b' = F_b$	x	C_D	C_M	C_t	C_L	C_F	C_{fu}	C_i	C_r	–	–	–	K_F	ϕ_b	λ
$F_t' = F_t$	x	C_D	C_M	C_t	–	C_F	–	C_i	–	–	–	–	K_F	ϕ_t	λ
$F_v' = F_v$	x	C_D	C_M	C_t	–	–	–	C_i	–	–	–	–	K_F	ϕ_v	λ
$F_{c\perp}' = F_{c\perp}$	x	–	C_M	C_t	–	–	–	C_i	–	–	–	C_b	K_F	ϕ_c	λ
$F_c' = F_c$	x	C_D	C_M	C_t	–	C_F	–	C_i	–	C_P	–	–	K_F	ϕ_c	λ
$E' = E$	x	–	C_M	C_t	–	–	–	C_i	–	–	–	–	–	–	–
$E_{min}' = E_{min}$	x	–	C_M	C_t	–	–	–	C_i	–	–	C_T	–	K_F	ϕ_s	–

F_b = 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:

$$Z' = Z\, C_D\, C_M\, C_t\, C_g\, C_\Delta\, C_{eg}\, C_{di}\, C_{tn}$$

For LRFD:

$$Z' = Z\, K_F\, \phi\, \lambda\, C_M\, C_t\, C_g\, C_\Delta\, C_{eg}\, C_{di}\, C_{tn}$$

where:

Z = the reference design value based on normal load duration. Z may be taken from connection tables directly or calculated using yield mode equations.

For ASD member and connection design, this approach is identical to that used in prior Editions of the *NDS*. For LRFD member and connection design, adjustment factors applicable to reference design values, make conversion between ASD and LRFD-based design values transparent.

In the *2005 NDS*, "reference design value" designates the allowable stress design value based on normal load duration and replaces terms such as *tabulated*, *nominal*, *base*, and *published*, which were also based on normal load duration. The variety of terms was considered potentially confusing. For example, tabulated and published values outside of the Specification may already include adjustment factors. Nominal may be interpreted as nominal strength (especially with the addition of LRFD) rather than in current *NDS* use where it means unadjusted. To avoid confusion, the descriptor "reference" is used and serves as a reminder that design value adjustment factors are applicable for design values in accordance with referenced conditions specified in the *NDS* – such as normal load duration.

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:

$$C_L = \frac{1 + (F_{bE} / F_b^*)}{1.9} - \sqrt{\left[\frac{1 + F_{bE} / F_b^*}{1.9}\right]^2 - \frac{F_{bE} / F_b^*}{0.95}} \qquad [3.3\text{-}6]$$

where:

F_b^* = reference bending design value multiplied by all applicable adjustment factors except C_{fu}, C_V, and C_L (see 2.3) and

F_{bE} = $1.20E_{min}' / R_b^2$.

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.

NDS 2005 3.7.1.5:

3.7.1.5 The column stability factor shall be calculated as follows:

$$C_P = \frac{1+(F_{cE}\ /\ F_c^*)}{2c} - \sqrt{\left[\frac{1+F_{cE}\ /\ F_c^*}{2c}\right]^2 - \frac{F_{cE}\ /\ F_c^*}{c}} \quad [3.7\text{-}1]$$

where:

F_c^* = 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:

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

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).

Reformat of Radial Tension Design Values

Glulam radial tension values have been added to Table 5.3.1 *Applicability of Adjustment Factors for Glued Laminated Timber* to clarify use for both ASD and LRFD.

Reference values for radial tension, F_{rt}, are provided in the glulam chapter to match the format of other reference design values in the *NDS*. In prior editions, equations for determining allowable design values for radial tension were specified.

New Appendix N – Load and Resistance Factor Design

Applicable for LRFD only, Appendix N specifies format conversion factors (K_F) and resistance factors (ϕ) consistent with those in *ASTM D5457 – Standard Specification for Computing the Reference Resistance of Wood-Based Materials and Structural Connections for Load and Resistance Factor Design*. Applicable time effect factors are associated with load combinations of *ASCE 7-02 – Minimum Design Loads for Buildings and Other Structures* and are provided in Table N3 (shown below).

Application of the time effect factor (λ) was also clarified for cases involving load due to lateral earth pressure, ground water pressure, or pressure of bulk materials, H. For cases where H is not in combination with L, a $\lambda = 0.6$ is applicable.

Removal of Form Factor

The form factor, C_f, has been removed from the *2005 NDS*. Plastic deformation of small clear test specimens provided theoretical justification; however, plastic deformation observed in small clear test specimens may not be applicable to full size members.

Additionally, applicability of form factors to standard wood products addressed in the *NDS* was limited. Pole and pile design values do not permit further adjustment by the form factor and the form factor for the specific case of a square member loaded about its diagonal was considered to

Table N3 Time Effect Factor, λ (LRFD only).

Load combination[a]	λ
1.4(D+F)	0.6
1.2(D+F) + 1.6(H) + 0.5(L_r or S or R)	0.6
1.2(D+F) + 1.6(L+H) + 0.5(L_r or S or R)	0.7 when L is from storage 0.8 when L is from occupancy 1.25 when L is from impact[b]
1.2D + 1.6(L_r or S or R) + (L or 0.8W)	0.8
1.2D + 1.6W + L + 0.5(L_r or S or R)	1.0
1.2D + 1.0E + L + 0.2S	1.0
0.9D + 1.6W + 1.6H	1.0
0.9D + 1.0E + 1.6H	1.0

[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, λ, 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 biaxial 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 ~~be in accordance with~~ Table 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,
- *NDS Supplement – Design Values for Wood Construction, 2005 Edition,*
- *ANSI/AF&PA SDPWS-05 AF&PA Supplement – Special Design Provisions for Wind and Seismic (SDPWS)* with Commentary, and
- *ASD/LRFD Manual for Engineered Wood Construction, 2005 Edition.*

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, ϕ, 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, (ν_W), 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, (ν_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, δ_{dia}, is calculated in accordance with Equation [**1**]:

$$\delta_{dia} = \frac{5\nu L^3}{8EAW} + \frac{0.25\nu L}{1000G_a} + \frac{\sum(x\Delta_c)}{2W} \quad [\mathbf{1}]$$

where:

$5\nu L^3/(8EAW)$ = deflection due to bending;

$0.25\nu L/(1000G_a)$ = 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, δ_{SW}, is calculated as shown in Equation [**2**]:

$$\delta_{SW} = \frac{8\nu h^3}{EAb} + \frac{\nu h}{1000G_a} + \frac{h\Delta_a}{b} \quad [\mathbf{2}]$$

where:

$8vh^3/(EAb)$ = deflection due to bending;
$vh/(1000G_a)$ = deflection due to shear deformation (from panel shear and nail slip); and
$h\Delta_a/b$ = 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:

$$G_a = \frac{\nu}{\dfrac{\nu}{G_\nu t_\nu} + 0.75e_n} \qquad [3]$$

where:

$G_v t_v$ = panel shear stiffness, lb/inch of panel depth;
e_n = nail slip, inches; and
ν = 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 × 1370 plf = 2740 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 ϕ 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 Ω_o are limited to low values (R = 1.5 and Ω_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**).

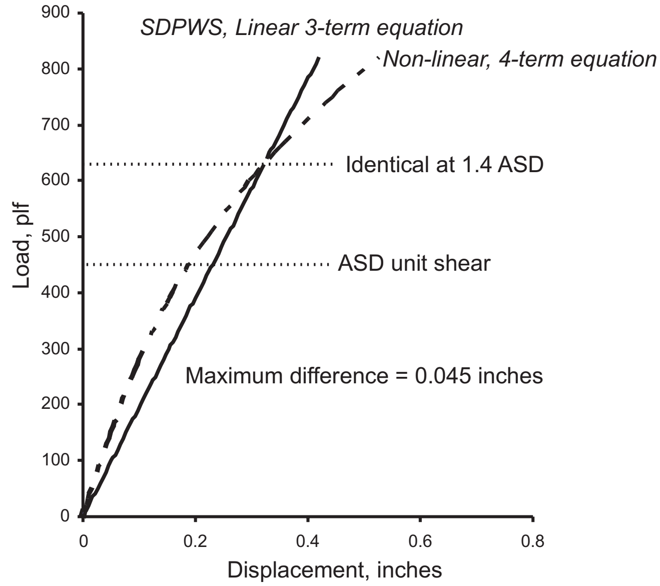


Figure 1.—Comparison of shear wall deflection (8 ft by 8 ft, 7/16-inch OSB sheathing, 8d common nails, 3 inches o.c. at edges).

Table 1.—Adhesive attachment of shear wall sheathing.

Seismic design category	Seismic design coefficient	
	SDPWS	*IBC 2006*
A, B, and C	R = 1.5, Ω_o = 2.5	--
D	Not permitted	Not permitted

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 × 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, (ν_{sc}) or (ν_{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

APA 157, Wood Structural Panel Shear Walls with Gypsum Wallboard and Window/Door Openings, 1996. APA- The Engineered Wood Association, Tacoma, WA, 98466.

APA T2005-08, Using Narrow Pieces of Wood Structural Panel Sheathing in Wood Shear Walls, 2005. APA- The Engineered Wood Association, Tacoma, WA, 98466.

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.

IBC-06, International Building Code. International Code Council, Falls Church, VA, 22041.

IBC-03, International Building Code. International Code Council, Falls Church, VA, 22041.

Philip Line, P.E., Manager, Engineering Research, American Forest & Paper Association, Washington, DC and James E. Russell, P.E., Building Code Consultant, Concord, CA.