LATERAL LOAD RESISTING ELEMENTS: DIAPHRAGM DESIGN VALUES

This note was archived November, 2011. The author has developed ASD design tables using an analytical method for wood framing based on the provisions of the 1991 edition of the AFPA National Design Specification for Wood Construction. Additional research is needed to bring the data in line with current codes and standards. The data below is from commentary section D2.1 of American Iron and Steel Institute (AISI) S213: Standard for Cold-Formed Steel Framing – Lateral Design. References to “Lum” refer to the Technical Note on the following pages (558b-1).

AISI S213 Commentary, section D2.1 (part): Available Shear Strength

...It should be noted that the diaphragm design values by Lum were based on the nominal strength of a No. 8 screw attaching wood structural sheathing to 33-mil cold-formed steel framing members. The 1991 NDS calculation methodology, which was used by Lum, yielded a nominal strength of 372 lb and a safety factor of 3.3. However, the NDS methodology was revised in 2001, and the revision greatly reduced the calculated strength of screw connections. Until Lum's work is updated, justification for maintaining the current diaphragm design values in AISI S213 is based, in part, on tests performed by APA (APA, 2005). Test results for single lap shear tests for a No. 8 screw attaching ½ in. plywood to 68-mil sheet steel indicated that the nominal strength of the connection was governed by the strength of the screw in the sheet steel; i.e., the wood structural sheathing did not govern the capacity. Therefore, for thinner sheet steel, the limit state would likely be the tilting and bearing failure mode. For a No. 8 screw installed in 33-mil sheet steel, computations of connection capacity in accordance with AISI S100 [CSA S136] would yield a nominal strength of 492 lbs and a safety factor of 3.0. Additionally, connection tests for plywood attached to 33-mil cold-formed steel framing members were performed by Serrette (1995b) and produced an average ultimate connection capacity of 1177 lbs, and Serrette suggested the use of a safety factor of 6, as given by APA E380D. A review of the allowable strengths, as summarized in Table D2-1 below, indicates that although Lum’s design values are based on an earlier edition of the NDS, the value is conservative when compared to both AISI and Serrette.

<table>
<thead>
<tr>
<th>No. 8 Screw Shear Strength (lbs) for 33-mil Cold-Formed Steel Member</th>
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<tbody>
<tr>
<td>Lum</td>
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<tr>
<td>Nominal</td>
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Introduction

Lateral loads from wind or seismic effects are usually transferred to the vertical lateral force resisting system (the shearwalls or frames) by the floor (or roof) framing system acting as a structural diaphragm. A diaphragm is part of the lateral force-resisting system, and may be thought of as a deep, horizontally oriented beam that spans between the shearwalls or frames of the structural system. In residential or light commercial construction that utilized cold-formed steel (CFS) structural framing, it is usually the case that structural plywood sheathing, along with the floor or roof framing that provides the required diaphragm capacity.

Unblocked diaphragms do not have complete blocking along the panel edges that are perpendicular to the direction of the supporting framing span. Shear transfer along these edges is achieved by a combination of bearing on adjacent panel edges (if the panels are staggered), and lateral resistance of the nails or screws that occur within the field of the panel.

Blocked diaphragms are typically stronger and stiffer due to the more complete edge attachments, but they also are typically more expensive to assemble than unblocked diaphragms. Unblocked diaphragms are used more frequently in residential and light commercial structures mainly because the loads imposed on the diaphragm of the structure are generally low. In addition, the aspect ratio (the ratio between the length of the diaphragm between supporting elements - to width of the panel) also tends to be low. Consequently, diaphragm deflection is usually within acceptable limits.

A plywood diaphragm traditionally has been considered to be “flexible.” That is, it is assumed to be incapable of resisting rotation within its own plane. Within certain limitations imposed by building codes, exceptions to the previous statement include diaphragms that cantilever over the nearest line of shearwalls or frames, and buildings that are completely open on one side (“three-wall” buildings).

Lateral load resisting elements: Diaphragm design values

Summary: The derivation of service load values for structural diaphragms that utilize plywood sheathing over cold-formed steel framing are examined in this Tech Note. Included is a brief discussion of some of the structural issues that the designer should consider while designing and detailing the diaphragm.

A comprehensive discussion of the design of structural plywood diaphragms is considered beyond the scope of this Tech Note. However, it is important to note that diaphragm design considerations normally associated with plywood sheathing over wood framing apply equally to plywood sheathing over cold-formed steel framing.

Plywood sheathed diaphragms fall into two general categories: Blocked and Unblocked.

Blocked diaphragms have blocking or framing members attached to all edges of the plywood panels, and the blocking/framing is attached to transfer shearing loads across the panel boundaries. Blocking may consist of:

a) Cold-formed steel C-stud lengths installed vertically or flat between floor joists under the full length of the unsupported plywood panel edge;

b) Flat cold-formed steel strapping installed continuously or in discreet lengths spaced along the unsupported edges of the plywood sheathing. Strapping may be installed on top of, or beneath, the plywood panel edge.

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There has been some discussion in recent years as to whether or not certain plywood sheathed diaphragms should be classified as “semi-rigid”, or having the ability to resist some level of in-plane rotation. The 1997 edition of the UBC provides classifications for a diaphragm’s flexibility, based on its mid-span deflection relative to the span between supporting elements. The designer should note that there is currently no accepted method for calculating the deflection of an unblocked diaphragm, and thus the classification of an unblocked diaphragm as anything but flexible should be avoided. The use of an unblocked diaphragm implies a higher level of diaphragm deflection.
DETAILING

Figure 1 illustrates the various planes that shearing stresses must transfer across to connect the diaphragm to adjoining lateral force-resisting elements. In addition, diaphragm performance relies on:

a) Proper sizing and detailing of chord elements to accommodate the induced loads.

b) Proper sizing and detailing of drag-strut/collectors to develop the induced loads from the diaphragm into the vertical elements of the lateral force resisting system (the shearwalls or frames).

A more comprehensive discussion of the design and detailing concepts for structural plywood diaphragms can be found in the publications cited in the References section at the end of this publication.

DIAPHRAGM SERVICE LOAD VALUES

While tables of service load diaphragm values exist in the various codes for plywood attached to wood framing. Values for plywood attached to cold-formed steel framing were not available in any of the nationally recognized building codes at the time of this writing. However, diaphragm shear values for plywood / cold-formed steel framing assemblies can be calculated using the methods outlined in a document published by the American Plywood Association, titled “Calculation by Principles of Mechanics” (see notation in the References section of this Tech Note). This document is a valuable tool for all designers of cold-formed steel structures.

Load values for plywood over cold-formed steel framing based on the methods outlined in this publication are shown in Table 1. Connection design shear calculations can be found in Table 2. The basic design method as outlined in this document involves determining the appropriate fastener strength, and then finding the appropriate load per lineal foot based on fastener strength and spacing. In addition, determining the appropriate diaphragm allowable loads requires the use of some reduction factors to account for the effect of various grades of plywood, plywood panel buckling, and close fastener spacing. Reduction factor values used in the calculations, and the derivation of fastener strength are as follows:

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**Reduction Factor for Species Group II Plywood**

Design shears for Group II and C-DX plywood are 90 percent of the values for STRUCT I plywood.

**Reduction Factor for 2 Inch Nominal Width**

The reduction for 2 inch lumber is needed because of the reduced fastener strength in the thinner lumber width. A screw in a 2 inch nominal width CFS stud is not vulnerable to the same capacity reducing factor. No reduction in design shears was made for this load effect.

**Reduction Factor for Close Fastener Spacing**

No reduction in boundary attachment values have been taken for screws spaced at 2 inches on-center.

**Reduction Factor for Lightly Loaded Diaphragms**

a) For blocked diaphragms with 6 inch boundary and panel edge attachments, use 75 percent of the shear values of the 4 inch and 6 inch fastener spacing.

b) For unblocked diaphragms:

1. For load perpendicular to unblocked edges and continuous panel joints, use 67 percent of the shear values of the 4 inch and 6 inch fastener spacing.
2. For all other load conditions on unblocked diaphragms, use 50 percent of the shear values of the 4 inch and 6 inch fastener spacing.
Reduction Factor for Panel Buckling

Design shears for 3/8 inch panels were reduced by 17 percent for joists spaced at 24 inches on-center, and design shears for 7/16 inch panels were reduced by 8.5 percent for joists spaced at 24 inches on-center.

Calculation of Allowable Fastener Loads

Table 3 shows the allowable load calculations for screws used to derive the shearwall values. The calculations utilize the formulas in part XI, Section 11.3.2 of the 1991 edition of the National Design Specification (NDS) for wood construction (wood screws with metal side plates). Loads were calculated for #8 screws. Although it is theoretically possible to calculate values for #6 screws, caution should be exercised if #6 screws are used. Tests at the Santa Clara University suggest that the #6 screws exhibit excessively brittle behavior.

Preliminary fastener computations based on the thicker gauges of sheet steel did not yield significantly higher load values than the fasteners in the 20 gauge material. Therefore, the diaphragm schedules do not have different allowable load values for the thicker gauge studs. In addition, it was assumed that a #8 screw, having roughly the same diameter as an 10d nail, would fully develop the strength of a 15/32 inch STRUCT I, or C-DX plywood panel.

The allowable load values shown in Table 1 are for wind and seismic loads. The values are also limited to 4:1 aspect ratios.

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<table>
<thead>
<tr>
<th>PANEL GRADE</th>
<th>Screw Size (Dia. = in.)</th>
<th>Plywood Thickness (in.)</th>
<th>BLOCKED DIAPHRAGMS</th>
<th>UNBLOCKED DIAPHRAGMS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>#8 (D = 0.161&quot;)</td>
<td></td>
<td>Screw spacing at diaphragm boundary edges and at all continuous panel edges (in.)</td>
<td>Screws spaced at 6 inches max. at supported ends (in.)</td>
</tr>
<tr>
<td></td>
<td>3/8</td>
<td>307 409 654 818</td>
<td>6 4 2.5 2</td>
<td>Load perpendicular to unblocked edges and continuous panel joint</td>
</tr>
<tr>
<td></td>
<td>7/16</td>
<td>307 451 721 902</td>
<td>6 4 3</td>
<td>Other configurations</td>
</tr>
<tr>
<td></td>
<td>15/32</td>
<td>370 493 788 986</td>
<td></td>
<td>Stud Thicknesses</td>
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<td></td>
<td>33 mil, 43 mil &amp; 54 mil</td>
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<tr>
<td>Structural 1</td>
<td>#8 (D = 0.161&quot;)</td>
<td>3/8</td>
<td>274 205</td>
<td>33 mil, 43 mil &amp; 54 mil</td>
</tr>
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<td></td>
<td>7/16</td>
<td>307 406 649 812</td>
<td>247 184</td>
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<td></td>
<td>15/32</td>
<td>333 444 709 887</td>
<td>297 222</td>
<td>33 mil, 43 mil &amp; 54 mil</td>
</tr>
<tr>
<td>C-D, C-C Sheathing, plywood panel siding, and other grades covered in UBC Standard 23-2 or 23-3</td>
<td>#8 (D = 0.161&quot;)</td>
<td>3/8</td>
<td>276 368 589 736</td>
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1. All panel edges backed with 1.5 inch or wider framing. Space screws at 6 inches (152 mm) on center along intermediate framing members for 3/8 inch (9.5 mm) and 7/16 inch (11 mm) panels and 12 inches (305 mm) for other panel thicknesses.
2. Screws shall be self-drilling, self-tapping wafer head or bugle head screws of the minimum diameter shown.
3. Corresponds to 20 gauge, 18 gauge, and 16 gauge.

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Connection Strength Calculation

Table 2

<table>
<thead>
<tr>
<th>F_em</th>
<th>F_es</th>
<th>F_yb</th>
<th>K_D</th>
<th>Re</th>
<th>k</th>
<th>Z</th>
</tr>
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<tbody>
<tr>
<td>4,450 lb/in²</td>
<td>45,000 lb/in²</td>
<td>90,000 lb/in²</td>
<td>2.2</td>
<td>-1+ \sqrt{2 (1 + Re) + F_yb (2 + Re) D^2 / 2 F_em \cdot t_s^2}</td>
<td>\frac{1.75 F_em F_yb}{K_D (3 (1 + Re) D^2)}</td>
<td>112 lb</td>
</tr>
</tbody>
</table>

PANEL GRADE | Screw Size (Dia. = in.) | Plywood Thickness (in.) | Screw spacing at diaphragm boundary edges and at all continuous panel edges (in.) | Screws spaced at 6 inches max. at supported ends (in.) |
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3. Corresponds to 20 gauge, 18 gauge, and 16 gauge.
SAMPLE CALCULATION:  STRUCT I PLYWOOD
with Framing at 24 Inches On Center and #8 Screws

Table 3

Load per fastener
for #8 screw and
Steel side plate =>

Reduction for 24 inches
on center spacing of studs =>
q := 112 (1 - .17) lb. <= 3/8 inch plywood
(17 % for 3/8 inch panels,
8.5 % for 7/16 inch panels)

LOAD DURATION FACTOR => LDF = \frac{4}{3}
REDUCTION FOR 2 INCH
NOMINAL WIDTH OF FRAMING => REDF2NW := 0

DIAPHGRAM FACTOR => C_{di} = 1.1

REDDUCTION FOR 2 INCH
NAIL SPACING AT BOUNDARY => REDF2NS := .00

ON-CENTER FASTENER SPACING
AT BOUNDARY => SB := 4 inches
ON-CENTER SPACING
AT INTERIOR PANEL EDGE => SI := 6 inches

Determine Allowable Shear at Interior Panel Edge Nailing

\[ V_n = \left( \frac{12 \text{ in.}}{\text{ft}} \right) \left( q \cdot (LDF) \cdot C_{di} \cdot (1 - REDF2NW) \right) + \left( \frac{\text{en} \cdot 24 \cdot G \cdot t}{L} \right) \]

\[ nV_n^{\text{interior}} = V = 2,866 \text{ lb./ft.}^{-1} \]

Determine Allowable Shear at Boundary Edge Nailing

\[ V := \left( \frac{12 \text{ in.}}{\text{ft}} \right) \cdot \left( q \right) \cdot (LDF) \cdot C_{di} \]

\[ nV_{n,\text{boundary}} = V = 409 \text{ lb./ft.}^{-1} \]

REFERENCES

1. “Cold-Formed Steel Design Manual,” American Iron and Steel Institute, 1996.
2. LRFD “Cold-Formed Steel Design Manual,” American Iron and Steel Institute, 1996.

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