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SHEAR RIGIDITY OF SHEATHED WALLS WITH PNEUMATICALLY DRIVEN PIN CONNECTIONS

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ABSTRACT

The purpose of this experimental and analytical study was to observe the structural behavior of wall panels constructed with cold-formed steel framing members and sheathing fastened with pneumatically driven pins subject to transverse load as typically occurs in residential construction. This study included the key parameters that influence the connection strength: steel thickness (16-, 18- and 20-gauge steel), sheathing thickness (1/2" (1.27-cm) Unipan and 1/2" (1.27cm) Dens-Glass Gold).

The shear design values given in the AISI design specifications (2001) for structural members braced by diaphragms are compared and reviewed with results obtained from a series of static load tests. The findings of this pilot study were used to define future research needed to establish design methodologies for residential shear wall construction. Because this was a pilot study no new design equations or recommendations were developed.

INTRODUCTION

Several manufacturers have been developing new types of fastening systems for cold-formed steel construction utilizing a pneumatically driven pin connection. Such a fastening system would meet the need to reduce time of construction,

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reduce the number of workers required to perform such construction and reduce the overall cost of the finished product. This study was conducted with the intention of comparing the 2001 AISI's Cold-Formed Steel Design Manual Specification provisions for structural members braced by diaphragms with results gained from testing pneumatically driven pin connections in cold formed steel panels.

EVALUATION OF DESIGN RECOMMENDATIONS FOR SHEATHING BRACING DESIGN

The design shear rigidity $\overline{Q}$ for shear wall is calculated using the following equation:

$$\overline{Q} = \overline{q}B \tag{1}$$

The value $\overline{q}$ was defined as the design shear rigidity of two wall boards per inch of stud spacing. Based on Simaan and Pekoz report (Yu, 2000) $\overline{q}$ was defined as:

$$\overline{q} = \frac{2G'}{SF} \tag{2}$$

where G' = diaphragm shear stiffness of a single wallboard for a load of $0.8P_{ult}$,

$= \dfrac{0.8P_{ult}}{\Delta_d}\left(\dfrac{a}{b}\right)$, kips/in.

P_{ult} = ultimate load reached in shear diaphragm test of a given wallboard, kips

Δ_d = shear deflection corresponding to a load of $0.8P_{ult}$, in.

a, b = geometric dimensions of shear diaphragm test frame, ft.

SF = safety factor, = 1.5

Based on previous studies of similar tests (Yu, 2000) the reason for using $0.8P_{ult}$ for G' indicates the shear deflection and thus the shear rigidity at the ultimate load is not well defined and reproducible. In addition to reducing the ultimate load a safety factor of 1.5 was used to avoid premature failure of the wallboard.

Simplifying Eq. 2 the design rigidity can be evaluated as

$$\bar{q} = \frac{0.53 P_{ult}}{\Delta_d}\left(\frac{a}{b}\right) \tag{3}$$

The 1980 and 1986 AISI Specification, provided values of $\bar{q}_0$ based on the results of a series of shear diaphragm tests assembled with different wallboards and self drilling screws 6- to 12-in. apart (Yu, 2000). The value $\bar{q}_0$ was computed by

$$\bar{q}_0 = \frac{\bar{q}}{2 - s/12} \tag{4}$$

In 1994 Miller and Pekoz reported (Yu, 2000) the strength of gypsum wall board braced studs was observed to have a significant correlation to the spacing of the studs. Furthermore the deformations of the wall panel were observed to be localized at the fasteners, and not evenly distributed across the panel. As a result $\bar{Q}_0$ in Table 1 was determined from $\bar{Q}_o = 12\bar{q}_0$.

Table 1. Sheathing Parameters(1)

Sheathing(2)	$\bar{Q}_0$	
	k	kN
3/8 in. (9.5 mm) to 5/8 in. (15.9 mm) thick gypsum	24.0	107.0
Lignocellulosic board	12.0	53.4
Fiberboard (regular or impregnated)	7.2	32.0
Fiberboard (heavy impregnated)	14.4	64.1

(1) The values given are subject to the following limitations:
All values are for sheathing on both sides of the wall assembly.
All fasteners are No. 6, type S-12, self drilling drywall screws with pan or bugle head, or equivalent.

(2) All sheathing is 1/2 in. (12.7 mm) thick excepted as noted.

EXPERIMENTAL INVESTIGATION

This study demonstrates the use of pneumatic driven pins and their properties as they are used to attach sheathing to light gauge steel channels. The study considered sheathing and thickness variables, specifically Dens Glass Gold and Unipan, and 16, 18, and 20 gauge steel thickness. The static load test (racking load assembly) demonstrates the shear resistance of framed wall panels subject to shear and tensile loading, combined. In a previous paper additional studies considered shear capacity and the failure mode of a single pin connection subject to shear and tensile loading, respectively. A minimum of three identical specimens per category was tested.

Pin connectors were pneumatically driven into the sheathing until the pin was securely fastened into the cold-formed steel channel with a minimum penetration 1/4-in. (6.4 mm) through the frame as specified by the sheathing manufacturer.

Dens-Glass Gold is a unique "paperless" sheathing panel made of a patented silicone-treated core, surfaced with inorganic glass mat facings and a "gold" alkali-resistant coating. Unipan is a lightweight concrete backerboard of cement with polymer and lightweight aggregate wrapped in a fiberglass mesh. The materials share durability qualities, in addition to similar moisture resistance, handling and installation ease.

A series of racking assembly tests (Figure 1) were conducted for three different gauges of cold-formed steel (16, 18, and 20 gauge) and two types of sheathing (Unipan and Dens-Glass Gold). A system of identification was developed to differentiate between the various types of specimen. As an example R20-G-100 indicates a 20-gauge Dens-Glass Gold specimen subjected to a racking test using a 0.100-in. (2.54mm) diameter pin.

EXPERIMENTAL PROCEDURE

<u>Static Load Test for Shear Resistance of Framed Wall Panels (Racking Load Assembly)</u>: The wall panel specimen was built in accordance to the guidelines set forth by ASTM E72 (1980) and E564 (1984). The 8 x 8-ft. (2.4 x 2.4 m) panel is composed of steel studs placed 16-in. on center and screwed together into top and bottom tracks using pan head self-tapping screws. The sheathing was connected in accordance with manufacturer's recommendations as follows:

A. Dens-Glass Gold: two 4x8 ft. (1.2 x 2.4 m) sheets were placed so as to have the longer side oriented in a horizontal direction and pins were spaced a maximum of 8-in. (203.0 mm) on center around the perimeter and in the field of the board. Fasteners were installed flush with the surface (not countersunk). Fasteners were located a minimum of 3/8-in. (9.5 mm) from edges and ends of sheathing panel.

B. Unipan: two 4x8 ft. (1.2 x 2.4 m) sheets were placed so as to have the longer side oriented in a horizontal direction and pins were spaced a maximum of 8-in. (203.0 mm) on center around the perimeter and in the field of the board. Fasteners were installed flush with the surface (not countersunk). Fasteners were located a minimum of 3/8-in. (9.5 mm) from edges and ends of sheathing panel.

In accordance with ASTM E72 (1980) and E564 (1984), the panel's upper and lower track was anchored to the apparatus using a ½-in. (12.7 mm) diameter anchor bolt every 4-ft. (1.2 m) on center. Data was recorded in increments of 200 lb. (90.7 kg). Measurements were obtained at 790 lb. (358.3 kg) and 1570 lb. (712.1 kg) and any sustained loads exceeding these values. The recorded loads and deflections of the test were then graphed into load-deflection curves. After each load had been placed on the specimen, the load was removed and any residual deflection in the panel was noted. The loading was then applied up to total deflection of the panel exceeded 4-in. (102.0 mm).

The data recorded due to displacement of the panel was collected by means of dial gages to the nearest 0.01-in. (0.25 mm) The location of the dial gages (Figure 1) were placed at the lower right and left to measure any slippage of the panel and the dial gage at the upper left to measure the total of the displacement of the panel. The horizontal deformation of the panel at any load was the difference in reading of the dial gage at the upper left and the reading of the bottom right dial gage.

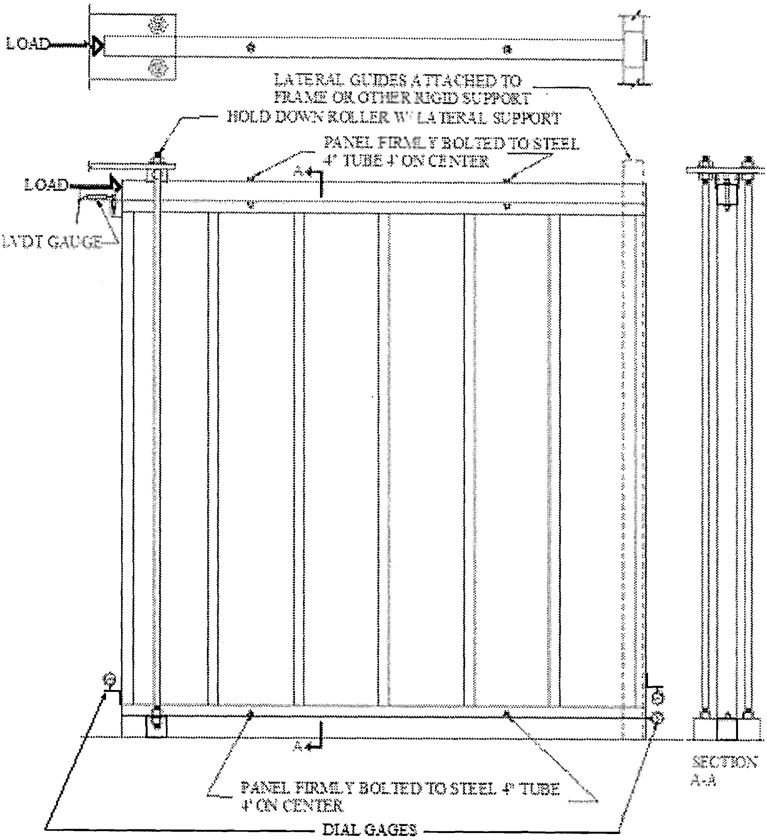

Figure 1. Static load test setup

EVALUATION OF TEST DATA

The static load test for shear resistance of framed walls is designed to evaluate the static shear capacity of a wall under simulated load conditions and to determine the stiffness in shear of the structural assembly. The apparatus was calibrated to record the load (lb.) and the deflection (in.). The data was then plotted on an applied transverse load vs. deflection graph as shown in Figure 2, 3 and 4. From the experimental data the shear stiffness was then computed using the AISI design specifications (2001) for structural members braced by diaphragms.

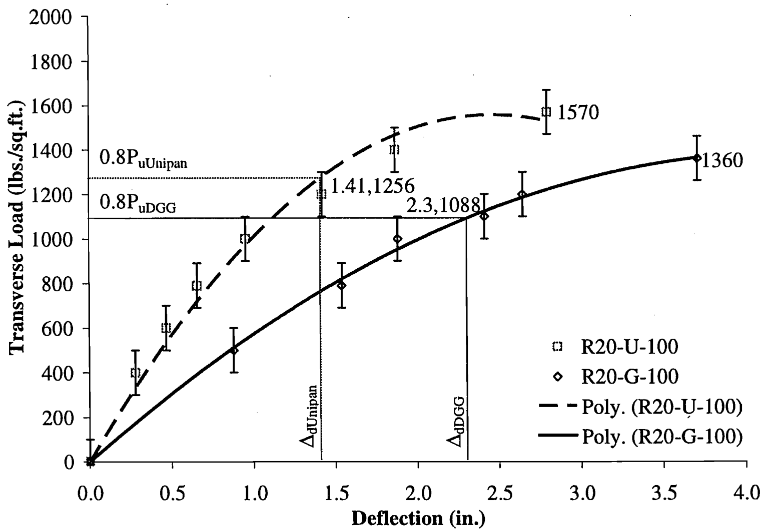

Figure 2. Racking Test for 20-Gauge Cold-Formed Steel

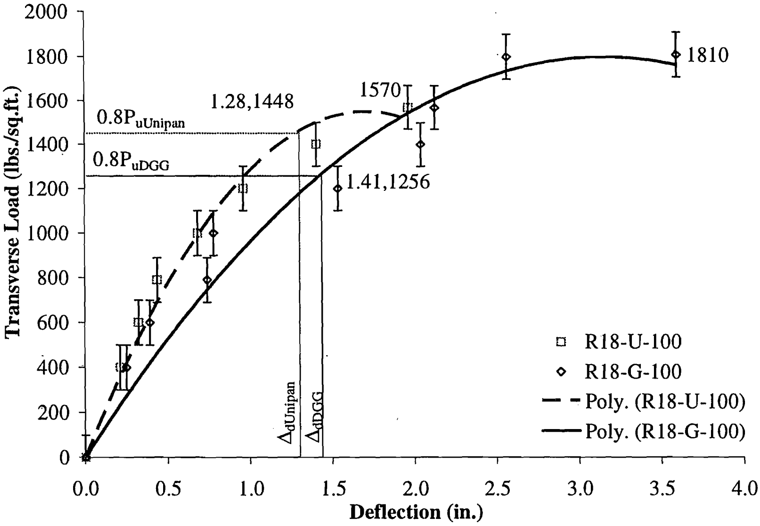

Figure 3. Racking Test for 18-Gauge Cold-Formed Steel

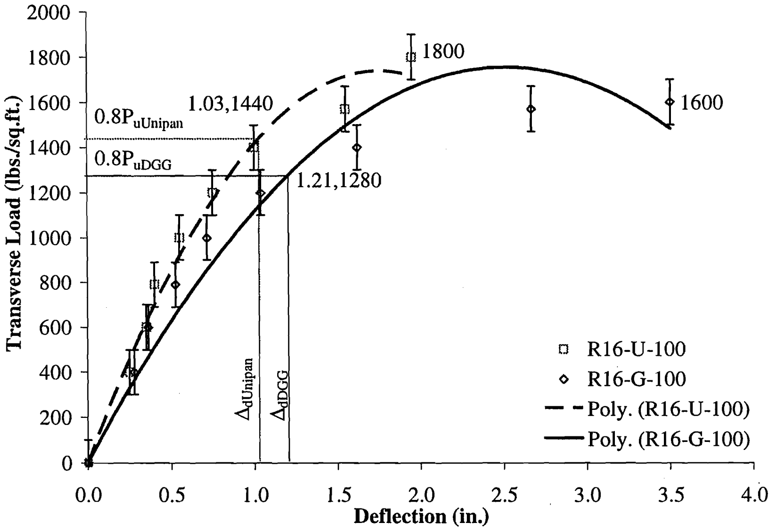

Figure 4. Racking test for 16-Gauge Cold-Formed Steel

Several assumptions were made in the evaluation of the load vs. deflection data. First the apparatus was considered to be frictionless, losses due to friction resulted from the hold-down roller located at the upper left corner in Figure 1.

The roller applies a load on the wall panel in order to prevent uplift. In order to minimize the amount of friction loss due to contact with the side supports of the roller, the wall was aligned to the center of the roller before loading.

Secondly, all of the test assemblies were loaded eccentrically to the wall's center of gravity. Imperfections such as differences in lengths and the misalignment of joined members caused unevenness along the top and bottom edges of the wall. In accordance with ASTM, the wall panel is only required to have sheathing on one side of the frame. ASTM guidelines specify the use of lateral bracing to prevent out-of-plane deformation perpendicular to the direction of loading. As a result, the wall produces inner moments causing torque throughout the panel. The rotational forces in certain instances caused local web buckling. In determining the effects the rotational forces have on the overall results of the wall assembly, an alternative approach is needed.

The next assumption made was the determination at which point the load had leveled-off. The load was calibrated by exceeding the necessary load and then allowing the assembly to relax for a set time of one-minute permitting the assembly to come to a substantial rest prior to taking the specified readings. In some cases, the period of set differed due to the difficulty of acquiring a stable point. The actual test loads would fluctuate due to the residual stress momentarily exceeding the counteracting frictional force resulting in fluctuations throughout the process. To compensate for the margin of error due to friction loss and fluctuation, the assumption was made to incorporate an allowance of 100-lbs (45.4-kg) in every case.

As the wall panel was tested, the geometry of the 8 x 8–ft (2.4 x 2.4-m) square panel was altered into a parallelogram causing a sequence of failures. The initial failure condition was edge-tearing (Figure 5), followed by tilting and pull-over failures (Figure 6 and 7) which are diagrammatically illustrated in Figure 8.

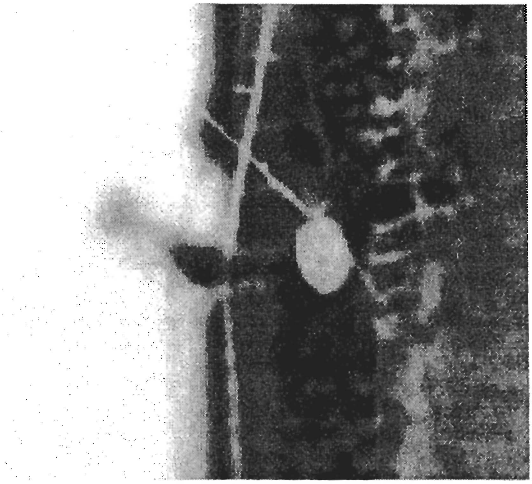

Figure 5. Edge Tearing – Unipan

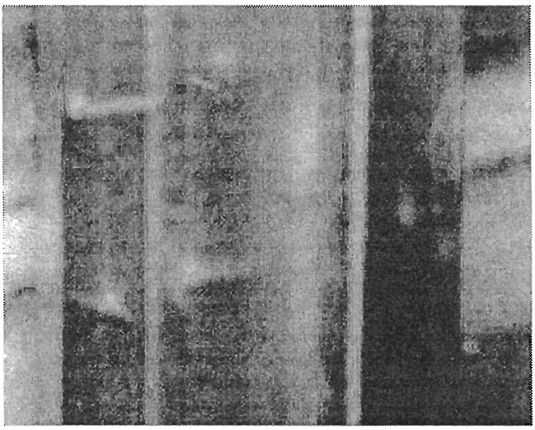

Figure 6. Tilting - Unipan

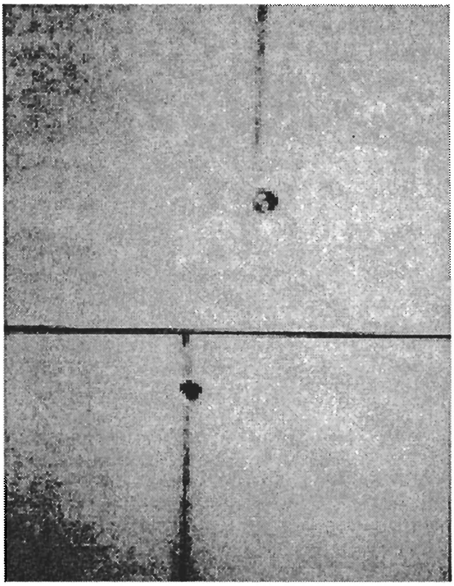

Figure 7. Failure Mode Tilting and Pull-Over - Unipan

The increase in thickness of the steel and the increase in yield strength resulted in additional stiffness reducing the tilt associated with pin connections and allowing for increased resistance. The common differences in the sheathings tested are the residual effect during and after loading. In most of the analyses, Dens-Glass Gold deflected greater than Unipan. The permanent deformation upon the removal of applied load of each series revealed minimal elastic range with minimal differences in the graphs for both Unipan and Dens-Glass Gold. Other slight variations extend from the failure load for both types of sheathing with both yielding similar ultimate loads.

Table 2 summarizes the results of the tests and the comparison of the experimental versus the design shear rigidity $\overline{Q}_0$ as provided by the Table 1. The measured capacity for shear versus the AISI shear design equations varied with a mean of 0.55 for Unipan and 0.43 for Dens Glass Gold.

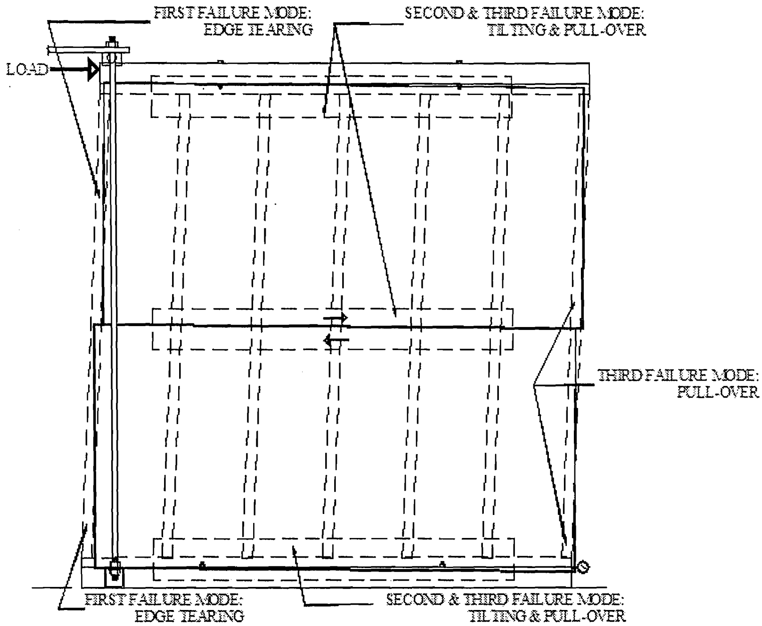

Figure 8. Diagram of Wall Diaphragm after Testing

CONCLUSIONS

The objective of this pilot study was to develop a better understanding of the structural behavior of sheathing attached to cold-formed steel structures using pin connections according to the manufacturer's recommendations and the AISI Design Specifications. Based upon the limited number of tests the following observations have been developed:

- As in some of the previous research conducted by Miller and Pekoz (Yu, 2000) the deformations of the wall panel were observed to be localized at the fasteners, with the first failure mode for the panel tests edge-tearing, followed by tilting and pull-over.

Table 2. Overall test results for shear using the racking assembly

Assembly	Measured Capacity P_{ult}	$0.8P_{ult}$	Shear Analysis Δ_d	G'	$\bar{Q}_0$	Ratio* $\bar{Q}_0/24.0$
	kips (kg)	kips (kg)	in. (mm)	kips/in. (kg./mm)	kips (kg.)	
R20-U-100	1.57 (712.4)	1.27 (569.0)	1.41 (35.8)	0.90 (15.89)	10.8 (190.7)	0.45
R18-U-100	1.57 (712.4)	1.27 (569.9)	1.28 (32.5)	0.99 (18.52)	11.88 (222.2)	0.50
R16-U-100	1.80 (816.5)	1.44 (653.2)	1.03 (26.2)	1.40 (24.93)	16.83 (299.2)	0.70
R20-G-100	1.36 (616.9)	1.09 (493.5)	2.30 (58.4)	0.47 (8.45)	5.67 (101.4)	0.24
R18-G-100	1.81 (821.0)	1.45 (656.8)	1.41 (35.8)	1.03 (18.34)	12.33 (220.1)	0.51
R16-G-100	1.60 (725.7)	1.28 (580.6)	1.21 (30.7)	1.06 (18.91)	12.69 (227.0)	0.53

Means for Unipan 0.55

Means for Dens Glass Gold 0.43

* Both Unipan and Dens-Glass Gold were considered a gypsum product thus $\bar{Q}_0 = 24.0$

-The shear design values given in the AISI design specifications (2001) for structural members braced by diaphragms were compared with test results. The design shear rigidity were consistently higher when compared to the experimental values.

-The thickness of the steel channel and the types of sheathing used did affect the shear capacity of the section, i.e. as the steel thickness is increased, a decrease in tilting was observed accompanied by an increase in loads.

In conclusion, the AISI design specifications (2001) for structural members braced by diaphragms provide design shear rigidity for limited types of sheathing. When compared to the experimental values the design shear rigidity was consistently higher.

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