Screw Connected Shear Diaphragms

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by

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Introduction

In the past several years, cold-formed steel roof deck and floor panel systems have been used extensively in design. Their primary function has been to resist gravity loads but more and more often they are being relied upon for the secondary function of bracing against lateral loads such as those arising from wind.

To assess this diaphragm role in the overall behavior of a structure, requires an understanding of the in-place shear strength, shear stiffness, and the reliability of the system. While strength is important, stiffness is equally a major consideration because deflection compatibility must be maintained between the structural framing and the diaphragm.

Diaphragm strength and stiffness are dependent, among other parameters, on the panel shape, plan dimensions of the structure, material thickness, panel shape, and type of connections. The reliability is more dependent on connections than any other single item since observed failures almost always have involved the fasteners. Connections fall in two categories, those between panels along their edges and those from the panel to supporting structural members.

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The first of these will be referred to as PP Connections and the latter as PS Connections.

To those who have observed field erection of steel deck using welds, certain problems related to burn through, poor fusion, and other weld control measures are obvious. Screw manufacturers, particularly those producing self drilling screws, have been conscious of these problems and feel that screws result in more reliable connections.

The purpose of this study has been to investigate screw connected diaphragms and to make comparisons between them and similar ones having welds. It should be noted that all test diaphragms were assembled under laboratory conditions making every effort toward optimum control on fastener quality.

Experimental Program

The screw connected tests in this series were made on 16' wide by 20' long diaphragm assemblies using 20, 22, and 26 gage panels. Panels were the intermediate rib type (I) at 30" widths and 24" wide B type for the 26 gage diaphragms. The 20 and 22 gage systems were assembled using TEKS No. 12 self-drilling screws while the 26 gage diaphragms were wide rib type B and had No. 14 screws. All panels were 20' long.

The test frame was a single bay cantilever type indicated schematically in Fig. 1. Frame perimeter members were W 10 x 21 steel sections and purlins were C 6 x 10.5 channels. Internal frame member connections were flexible and considered penned since the frame had no appreciable shear resistance prior to attaching a diaphragm. Purlins were framed between the perimeter members such that all top flanges were at the same level.

The total test load P was supplied by a hydraulic jack system connected to a load cell. The usual array of dial gages was used to arrive at the net shear deflection Δ. The test load is converted to the average diaphragm shear S according to P/b where b is the frame dimension parallel to the line of applied load P. The shear stiffness was determined following AISI[1] amd SDI[2] recommendations:

$$G' = \frac{P}{\Delta}\frac{a}{b} \tag{1}$$

where a and b are the frame dimensions shown in Fig. 1, b also being the test panel length. For the purposes of Eq. 1, P and Δ are evaluated at four-tenths of the ultimate jack load P_u.

Two tests were made using 26 gage Type B panels and loaded under dynamic conditions with an MTS closed loap loading system. The purpose of these tests was to determine the effects of possible screw loosening, hole elongation, or other general deleterious results from cyclic loading. Sinusoidal loads between lower and upper peak values as indicated in Fig. 3 were applied at 1/4 cycles per second for 1650 cycles. A final static load from zero to failure was applied. These loads could be compared to two other identical diaphragms loaded under static conditions only. The average loading schedule for cyclic tests was as follows.

Number of Cycles	(Average Range)/Pu Low Peak	High Peak
1650	0.10	0.16
250	0.14	0.23
100	0.20	0.33
50	0.25	0.40
	Static Load, zero to failure P_u:	1.00

The panels used in screw connected tests were 24 and 32" wide and assembled using number 12 or 14 screws. Screw spacing across panel ends and along panel edges are as indicated in Table 1. The welded diaphragms to which comparisons are made have been reported previously.[2,3] They were assembled using 5/8" diameter or 1 1/4" x 3/8" puddle welds.

Shear Strength

In excess of 160 full scale tests on welded diaphragms have been made for the Steel Deck Institute.[3,4] Figure 4 shows shear strength results from some of these tests in which comparisons can be made directly to screw connected diaphragms. The straight lines representing shear strength for each gage are for a standard case with w at 24" and n = 1, where w is the panel width and n is average number of fasteners per foot at the panel ends. It has been found that the shear strength changes proportional to $\sqrt{n\ w/24}$ for 18 and 20 gage panels and by $\sqrt{w/24}$ for thinner panels when $n \neq 1$ and $w \neq 24"$.[2] There are obvious limits to these equations. When L/t becomes large, the straight line equations would finally result in 18 gage diaphragms showing lower shear values than otherwise identical 22 gage diaphragms. The design might be better based on a smooth curve as shown having only L/t as a variable. L is the spacing of (PP) panel-to-panel connections on longitudinal panel edges and t is the base metal thickness.

Table 1 contains a summary of test results from tests on screw connected diaphragms. Values from the first six are plotted on the lower curve in Fig. 4 having been reduced to the standard case with n = 1 and w = 24. Eventhough the range of L/t for tests on the lower curve is 1100 or less and the welded diaphragms above 1100, it is obvious that the welded systems were stronger. This is not surprising since the puddle welds were much greater in diameter than the screws and most failures were related to fastener bearing stress in the panel. It should be noted that puddle welds are very difficult to form between joists or purlins and low L/t values are not practical for these diaphragms unless joists are spaced closely.

The screw connected diaphragms had closely spaced PP connections and buckling was not a problem. Failure resulted from panel deformation around screws and all fasteners along any sidelap exhibited about the same degree of distress as failure loads were approached. Noting the absence of local buckling between PP fasteners, it is not unreasonable to expect that the average shear S_u per foot is related to the individual fastener strength V_u in some manner. If this were the case, S_u would be in the form $12\ V_u/L$ (plf). This might be rearranged to $S_u = (12\ V_u/t)/(L/t)$. Simple shear tests with No. 12 screws indicate that the numerator is about 350000. The best fit test curve yields

$$S_u = 320{,}000/(L/t) \text{ plf.} \tag{2}$$

as expected, the test results yield a lower coefficient indicating less than uniform load distribution along panel edges.

Table 1 contains the results from four identical diaphragms using No. 14 screws where direct comparisons can be made between static and cyclic load effects. Diaphragms S1 and S2 were loaded under static conditions while C1 and C2 were subjected to 1650 load cycles as described earlier. Figure 5 contains the results of final load applications from zero to failure. The cumulative effect of the repeated loading represented a permanent average set of 0.074" in the shear deflection. The final load to failure resulted in an average S_u = 319 plf. versus an average of 290 plf. for the diaphragms loaded under static conditions. The increase may have been due to gradual elongation of holes around screws under repeated loading and a more uniform load distribution to individual screws. It is possible that the difference is in the range of test scatter. Similar tests at W.V.U. with different types of wal panels have resulted in similar comparisons.

It appears that the application of cyclic load below 0.4 P_u does not result in strength decreases.

<u>Shear Stiffness</u>

When welded connections are used, it has been found[3] that the shear stiffness for intermediate rib (I) deck and the standard condition is

$$G' = \frac{a}{b}[14(x)^2\ 10^{-6} - 60(x)\ 10^{-3} + 76],\ x < 2000 \tag{3}$$

and
$$G' = \frac{a}{b}[20 - 4(x)\ 10^{-3}]\quad x \geq 2000 \tag{4}$$

where $x = La/tb$. A plot of these equations is shown in Figure 6. Non-standard conditions are accounted for with a multiplication factor $n\sqrt{w/24}$.

There is more tendency for sidelap slip in screw connected diaphragms than for similar systems with welds. This is particularly true at close sidelap spacing when screws are heavily loaded. At greater spacing, panel distortion is more likely to govern stiffness since panels can warp more easily, buckling is likely, and fasteners will not be loaded heavily at failure. With x above 2000, buckling and warping will control and stiffness should be about the same for both welded and screw connected diaphragms.

Below the 2000 limit, high contact stresses can result around the screws. Slip and the attendant reduction in shear stiffness ensue To this date, no stiffness function has been generated to adequately account for panel slip. It does appear that it is a function of panel thickness, thinner panels having less stability around screws and greater tendency to permit tipping and fastener loosening. The following formula is presented as a best fit to the few tests available. It does reflect a thickness influence and is similar to the SDI stiffness equation.

$$G' = 33.3\,\frac{at}{b}[20 - 4(x)\ 10^{-3}] \tag{5}$$

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REFERENCES

1. "Design of Light Gage Steel Diaphragms," AISI, New York, 1967.

2. "Tentative Recommendations for the Design of Steel Deck Diaphragms," SDI, Chicago, 1972.

3. Ellifritt, D. S., and Luttrell, L. D., "Strength and Stiffness of Steel Deck Shear Diaphragms," Proceedings, 1st Spec. Conference on Cold-Formed Steel Structures, Rolla, Mo., 1971.

4. Ellifritt, D. S., and Luttrell, L. D., "Strength and Stiffness of Steel Deck Subjected to In-Plane Loading," C. E. Report 2011, Steel Deck Institute, Chicago, 1970.

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1. "Design of Light Gage Steel Diaphragms," AISI, New York, 1967.

2. "Tentative Recommendations for the Design of Steel Deck Diaphragm SDI, Chicago, 1972.

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APPENDIX

a	diaphragm width, ft.
b	diaphragm and panel length, ft.
n	average number of panel end connections per foot
t	base metal thickness, ft.
w	deck panel width, in.
x	La/tb
G'	diaphragm shear stiffness, k/in.
L	faster spacing on panel edges, in.
P	applied jack load, kips
S	average shear, plf.
S_u	average ultimate shear, plf.
Δ	shear deflection, in.

Table 1. Screw Connected Diaphragm Test Results

Test No.*	Purlin Sp. (in.)	L (in.)	t (in.)	w (in.)	End** Conn.	C (k/in.)
1	60	30	0.0342	30	12-6-12	22.37
2	60	20	0.0342	30	"	21.78
3	60	30	0.0273	30	"	16.00
4	60	20	0.0273	30	"	16.67
5	48	24	0.0273	30	"	15.31
6	48	24	0.0342	30	"	22.96
S1	80	24	0.0180	24	12-12	4.8
C2	80	"	"	"	"	4.0
S2	80	"	"	"	"	5.3
C2	80	"	"	"	"	4.0

*S1 - static load test, C2 dynamic load test.

**end fastener spacing, inches, measured from panel edge.

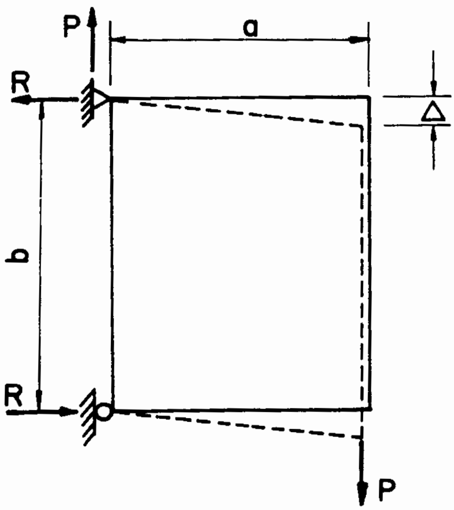


FIG. 1 CANTILEVER TEST FRAME

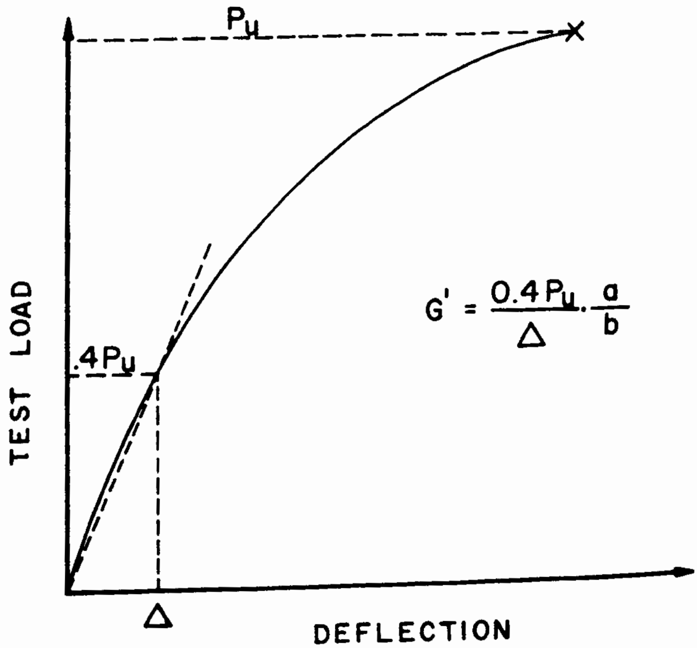
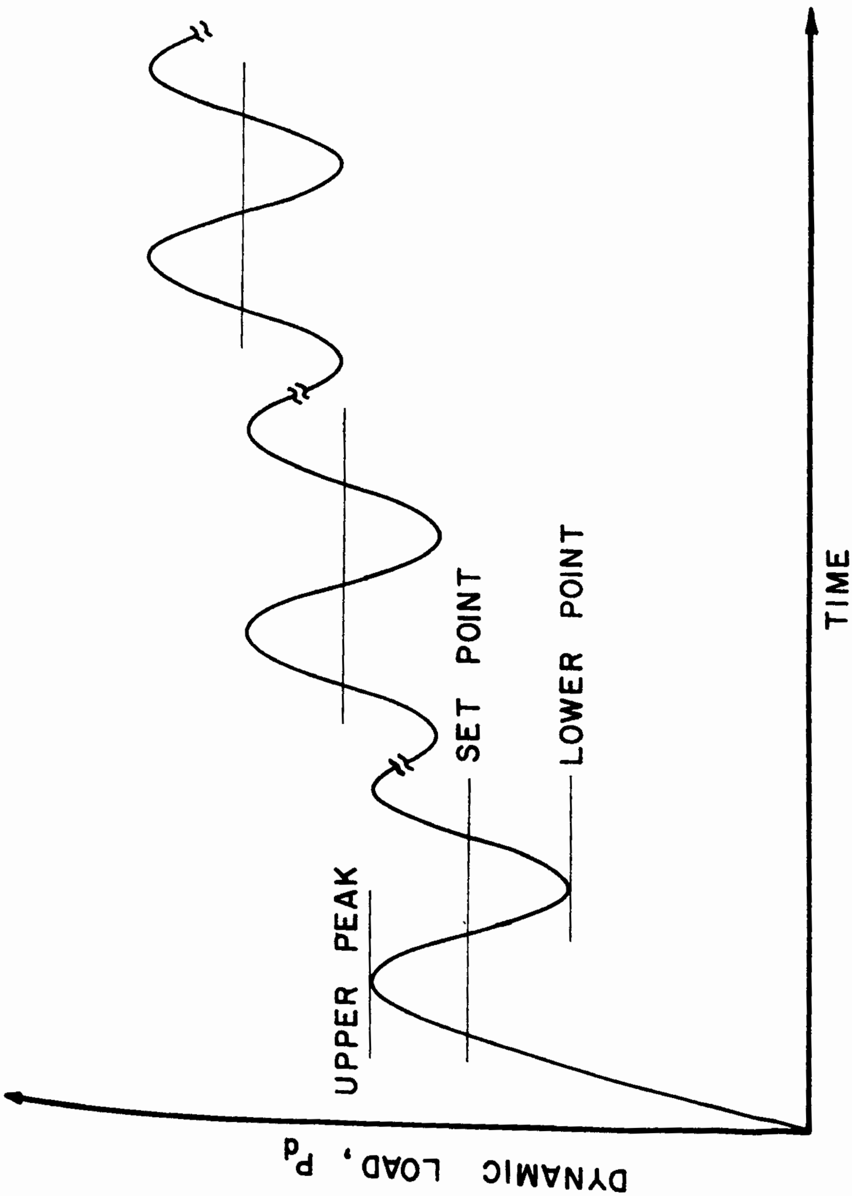


FIG. 2 TYPICAL LOAD-DEFLECTION


UPPER PEAK
SET POINT
LOWER POINT
TIME
DYNAMIC LOAD, P_d

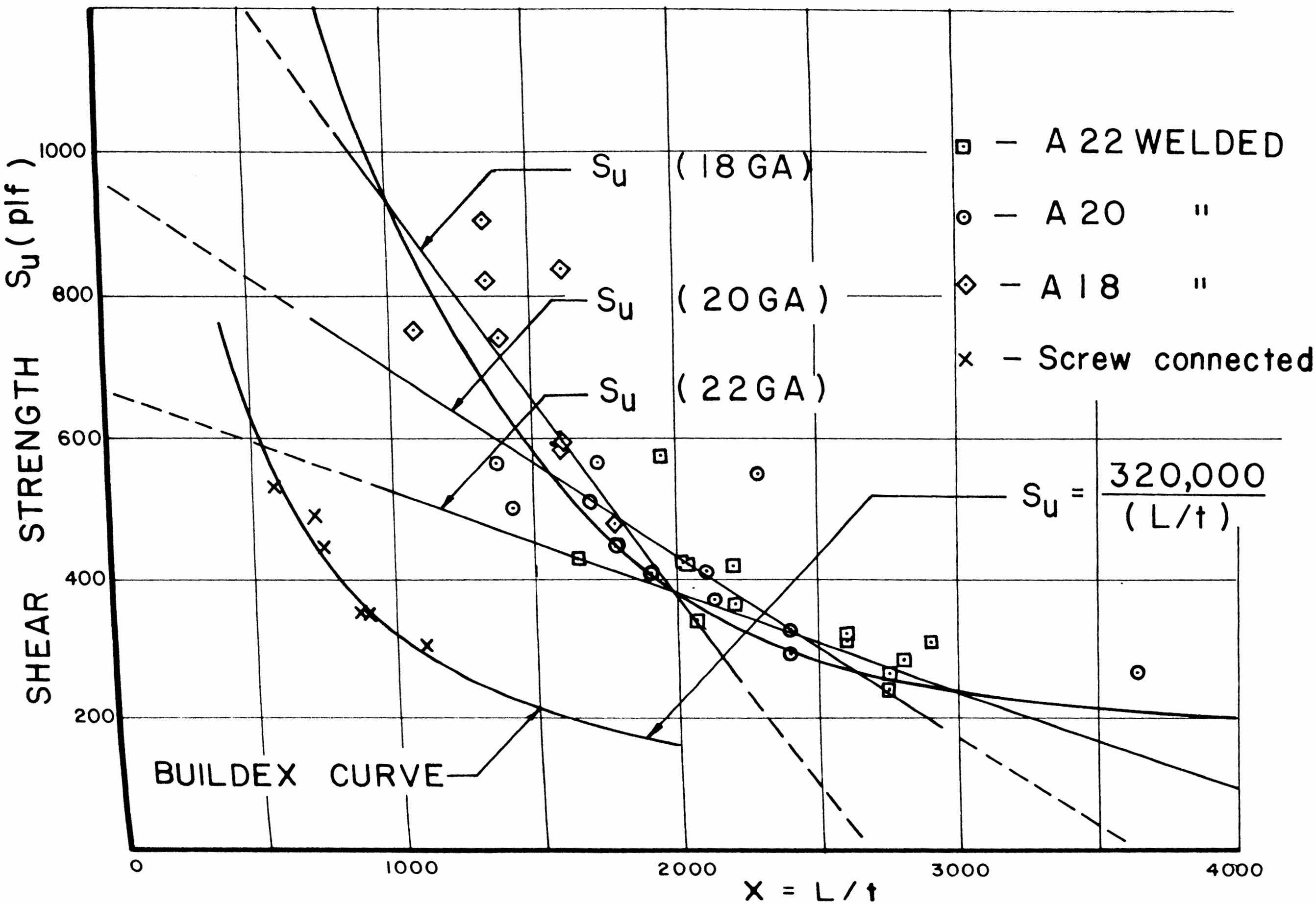


FIG. 4 SHEAR STRENGTH RESULTS

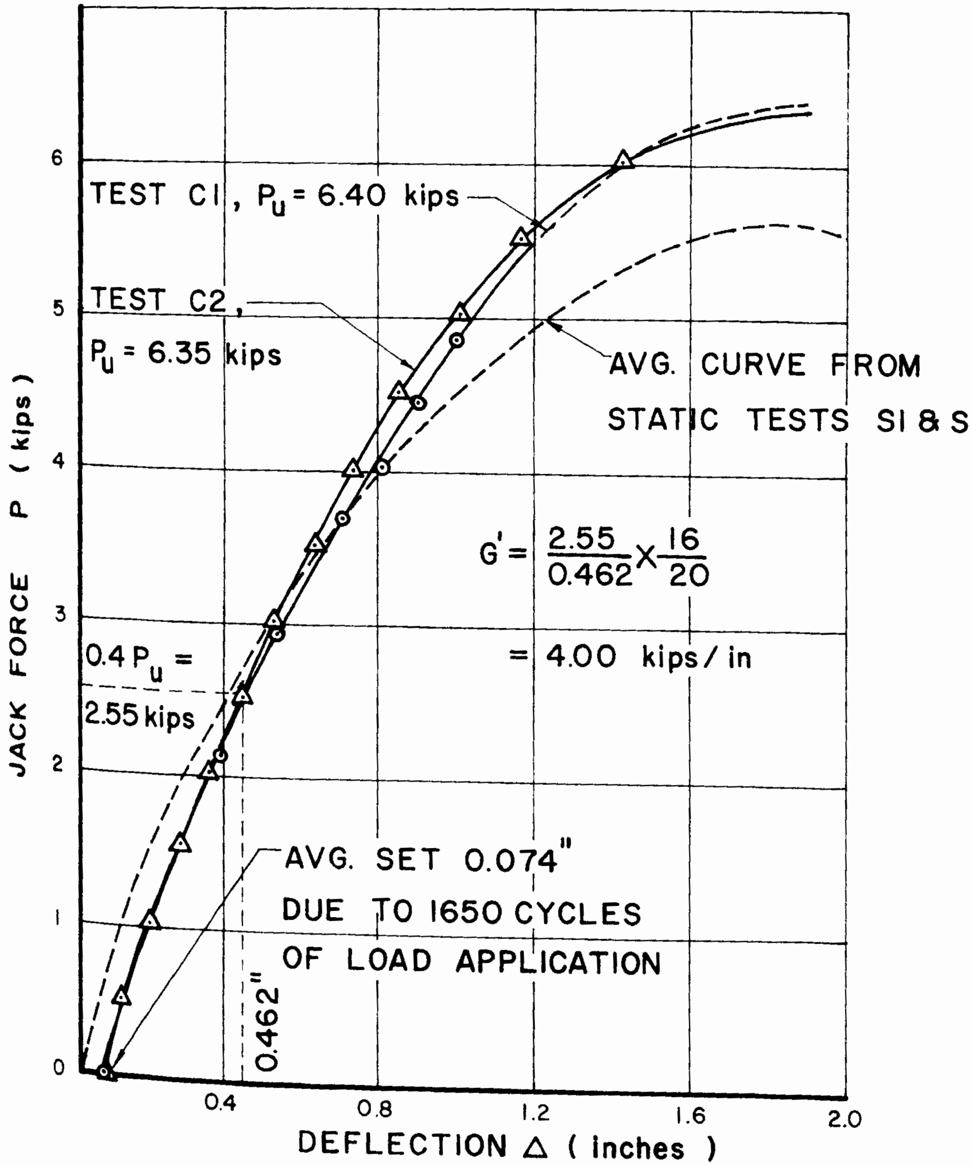


FIG. 5 STATIC LOAD DEFLECTION CURVES FOLLOWING DYNAMIC TESTS