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SHEAR DIAPHRAGMS WITH LIGHTWEIGHT CONCRETE FILL

by

Larry D. Luttrell*

Introduction. Light gage steel shear diaphragms are systems of roof, wall, or floor panels assembled in such a way that the systems have in-plane shear strength. They may be made from open fluted panels, cellular sections, from flat sheets, or corrugated shapes with concrete fill. for structural application, they must possess adequate shear strength and be connected to a supporting system sufficient to remove internal shear forces.

One common application and the subject of this study is in flat roof construction where corrugated panels are welded to joists and beams and the whole system covered with a lightweight insulating concrete. When the whole roof is in place, the system acts somewhat as a horizontal girder transferring loads to end walls and relieving loads on interior frames. To be sure, such "girders" may be weak, possess little shear rigidity, and have many discontinuities but they must be considered in analysis if actual behavior of the overall structure is to be predicted.

Exact mathematical expressions for determining load-deflection characteristics are limited to rather simple cases having well defined boundary conditions. One such solution is for plane rectangular diaphragms continuously connected on the sheet edges (1).* In practical diaphragms made up from roof deck for example, continuity across panel edges is far from perfect. These diaphragms are made from finite width elements joined at discrete points around their edges. A panel connected at a few points is certainly weaker in shear than a similar one connected at many points. Shear forces must be transferred from panel to panel across side laps. If there were three fasteners along a lap, one strength would develop; six fasteners would result in a greater strength though not necessarily twice as great. This is because buckling can be present and the span or distance between fasteners can enter the problem in a non-linear manner.

The purpose here is to examine test procedures, possible failure modes, some common misconceptions in predicting strength, and test data from diaphragms where buckling is severely retarded by the presence of insulating concrete.

Test Procedures. Precise evaluation of shear strength and stiffness, except for a few ideal cases, is presently impossible. It has been necessary to resort to laboratory tests in which load-deflection curves are determined for each type of diaphragm assembly.

Diaphragm behavior may be investigated using either of the test arrangements shown in Fig. 1. The three bay test arrangement requires more space than the single bay cantilever method and is more expensive to fabricate. However, it does give dual results, each of the end diaphragms being loaded by equal shear loads P. An examination of the three bay simple test arrangement reveals that the center bay has no shear. Its only function is to provide internal reactions of the end bays. Part b of the figure shows a free body of the end panel with reactions R. Comparisons between Figs. 1(b) and (c) show that the cantilever arrangement has identical reactions. In either case, the shear S per foot of diaphragm in either direction is

$$S = \frac{P}{b}$$

or

$$S = \frac{R}{a} = \frac{Pa}{b} \cdot \frac{1}{a} = \frac{P}{b} \qquad (1)$$

The shear formulas above presuppose internal frame connections and supports that are free to move prior to attaching the diaphragm; all shear is transferred through the diaphragm.

Evaluation of the shear stiffness G' requires accurate deflection measurement corrected for support movement. The net measured deflection after correction for support movement is

$$\text{Beam} \qquad \Delta = (D_2 + D_3 - D_1 - D_4)/2 \qquad (2)$$

$$\text{Cantilever} \qquad \Delta = D_3 - [D_1 + \frac{a}{b}(D_2 + D_4)] \qquad (3)$$

Where D_1 through D_4 are measured deflections at points indicated in Fig. 1. The above values for Δ include bending deflections which are a function of edge member sizes. Since a particular diaphragm type may be used with many different sizes of edge members, it is necessary to remove the bending deflection influence when evaluating shear stiffness. (Bending deflection must be reconsidered for each design situation.) The net shear deflection is given by

$$\Delta_s' = \Delta' - \Delta_b' \qquad (4)$$

Where Δ' is the average values from all tests considered according to Eqs. 2 or 3 and Δ_b' is the bending deflection which may be estimated conservatively using the following equations.

$$\text{Beam} \qquad \Delta_b' = \frac{5\ Pa^3(12)}{6EI} \qquad (5)$$

$$\text{Cantilever} \qquad \Delta_b' = \frac{Pa^3\ (12)^3}{3EI} \qquad (6)$$

where

P = Jack load, lb.

$E = 29.5 \times 10^6$ psi

I = Moment of inertia considering frame members only $= Ab^2\ (12)^2/2\ \text{in}^4$

A = Sectional area of members CD and GE, in^2.

The shear stiffness G' can be determined as a secant modulus at a particular load, usually at 0.4 of the ultimate laod P_u, as

$$G' = \frac{0.4\ P_u}{\Delta_s'} \ \frac{a}{b} \qquad (7)$$

It was noted in Reference (3) that the shear stiffness is dependent on panel length L yet Eq. (7) appears to be independent of L. The influence of panel length L is contained in the corrected shear deflection. For example, if b/a were 2 and the panels spanned the long direction, the system would be more rigid and a smaller value of Δ_s' would result than if panels spanned the short direction.

Failure Modes. At least three different types of failure can occur for a particular deck used as a shear diaphragm. The failure mode is dependent on the spacing of purlins or girts, connection layout, and the connection quality. The ultimate shear strength is usually controlled by connection strength in stiff diaphragm where the purlins are rather closely spaced and by buckling when purlins are far apart. A third and mixed failure mode can be initiated by local buckling leading to redistribution of internal forces with subsequent fastener failure.

* Numbers in parentheses refer to the list of References in Appendix I.

* Associate Professor of Civil Engineering, West Virginia University, Morgantown, West Virginia 26506.

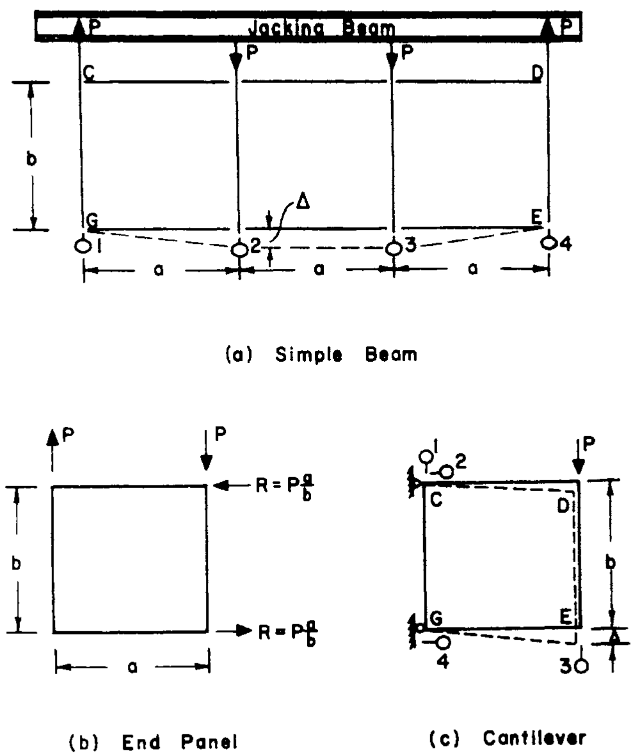


Figure 1. Diaphragm Test Arrangements.

Force distribution among welds in a diaphragm is strongly dependent on diaphragm stiffness. Flexible daiphragms may develop local buckling at a very low percentage of the ultimate load reducing loads on many connections through redistribution in the flexible system. Very stiff diaphragms on the other hand may have all welds on a given sidelap loaded almost equally but the premature failure on one weak weld may lead to sudden load transfer to adjacent welds and a "domino-like" rapid failure. It is sometime observed in tests that diaphragms with large numbers of welds exhibit only slightly higher strength than similar systems with fewer welds for this reason.

Common Misconceptions in Predicting Behavior. There is a tendency among engineers to estimate the strength for individual welded diaphragm connections on the basis of well established criteria for plug welds in heavy steel construction. Then, the further tendency is to assume all connections in a particular line share the loads equally; multiply strength per connection times number of fasteners to arrive at the shear strength for the system. This concept is wrong on both counts.

The allowable shear stress on a plug weld is well defined and since such welds are made through holes of a given diameter, the shear area A is very close to $\pi d^2/4$ where d is the hole diameter. Puddle welds for light gage material on the other hand, are made by burning through the panel during the welding operation. The apparent weld diameter may vary greatly depending on the electrode amperage and the actual fusion area and be much less than the apparent area. Two things are essential for high quality welds. First, the light gage panel must be flat against the supporting member else the hole will burn faster than weld material can be deposited; the result is that fusion to the panel occurs on only part of the weld perimeter. Secondly, welding must be slow enough that the lower member in a lap reaches the fusion temperature. Otherwise, the weld has only a stem of fused area and would appear somewhat like a mushroom in cross section. Unpublished data from tests at West Virginia University show that fusion to the panels usually is over about 75% of the weld perimeter and that the "stem" has a diameter of about 1/4 to 1/2 of the apparent diameter. Considering the problems in controlling field welding quality, an obvious conslusion is that weld strength in light gage steel diaphragms cannot be predicted by direct use of established plug weld criteria for heavy steel welding.

Even if individual weld strength could be predicted, it could be used to predict diaphragm shear strength in very few if any cases. Consider Fig. 2 which has a schematic drawing representing a shear diaphragm; arrows represent the general direction of diagonal tension fields at the higher levels of loading. Panel ABCG tends toward tensile stresses along diagonal BC and compressive stresses on the AG diagonal. Panels are almost invariably loaded with eccentricity since connections are in the valleys. Consider the edge free body in the figure. At very low shear loads P and before any buckling develops, it is possible that the weld loads p_1, p_2, p_3, and p_4 are about equal. With continued loading however, p_1 causes increased bending and may reach some critical value p_{1c} which will cause local buckling or "kinking" on the free edge with the edge flute acting somewhat as a strut. The force p_1 may remain essentially constant or decrease with further increase in the load P leading to possible buckling near the weld indicated by force p_2. Beyond this load level, most of the force is transferred through p_3 and p_4. No buckling will occur due to forces p_3 and p_4 since they constitute part of the diagonal tension field and load the "strut" in tension.

Very stiff diaphragms made from heavy gage material may not exhibit the local buckling failures usually noted for panels of about 20 gage (0.0359") and thinner. Concrete fill can also restrict local buckling and produce a favorable force distribution between the edge welds. In no case however, can the shear strength exceed the ability of the system to transfer force in and out of the diaphragm along edges DE and CG respectively.

Shear strength can be increased by adding sidelap fasteners but there is no point in adding them only along one sidelap line. Consider Fig. 3c in which a free body is taken from one end of a three bay system. Each line of welds CG, AB and mn must transfer a total force P. Since all welds are made between panels and support members, they can be of equal quality. It should be apparent that if the intermediate welds marked x were omitted, failure might occur along any of the three lines but their use would restrict failure to either line AB or mn.

Figure 3c shows a free body of the edge panel having nine edge and five sidelap welds. for equilibrium, the average weld load along line AB should be 9/5 of the average on line CG. A greater strength could be realized by adding intermediate fasteners on AB such as button punches, welds, or by using concrete fill for shear transfer between adjacent panels. Such devices must exhibit at least as much shear strength as intermediate edge fasteners and have similar shear characteristics. Concrete for example would act compositely with steel and due consideration would have to be given to the shear modulii in predicting stiffness.

Diaphragms with Insulating Concrete. Roof assemblies sometimes

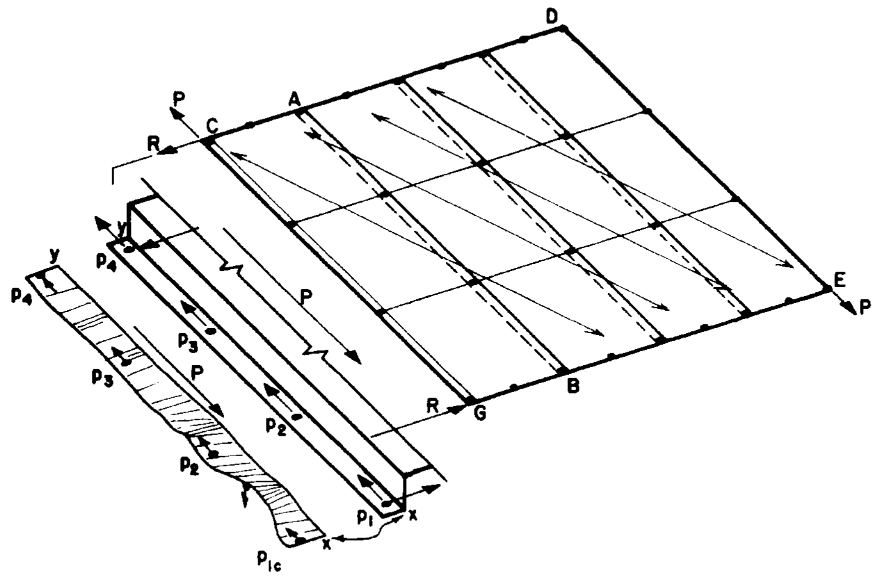
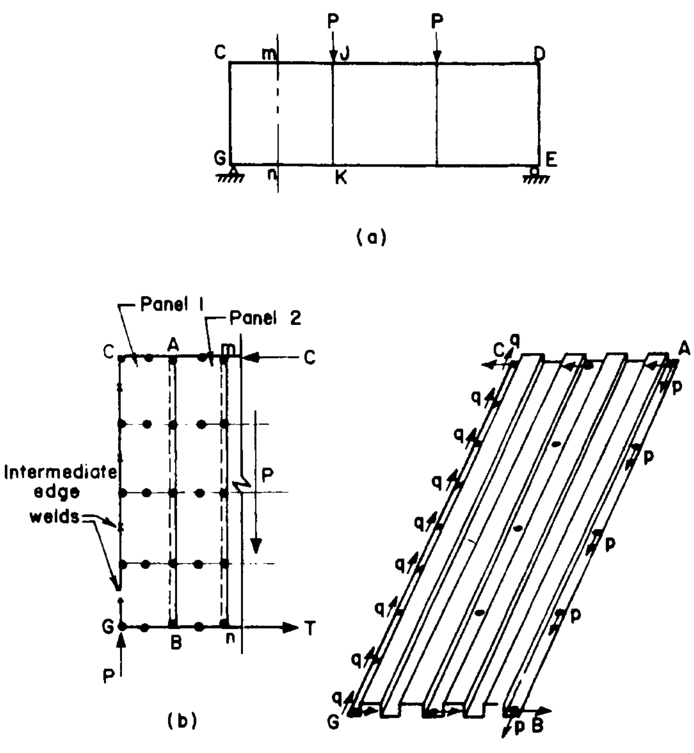


Figure 2. Freebody of Diaphragm and Edge Flute.



(c)

Figure 3. Section and Panel Freebody.

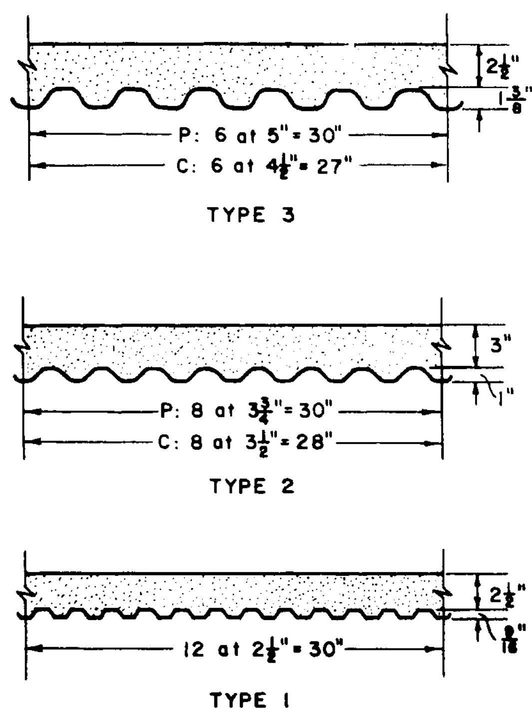


Figure 4. Nominal Panel Dimensions.

have a cast in place light weight insulating concrete. Eventhough the concrete may have very low compressive strength in the range of 100 to 200 psi, it can have significant effect on diaphragm performance. The most noticeable influences are on the shear stiffness and to a lesser extent, strength.

To investigate the effect of light weight fill, eleven tests were made on corrugated diaphragms, nine having filler concrete and two without. The diaphragms were assembled with 20' long panels on a 16' x 20' cantilever type test frame shown schematically in Fig. 1. Typical panel shapes with nominal dimensions are given in Fig. 4. Slight variations were present in the cross sections designated by a P or C in Fig. 4 as well as in the test numbers.

The test frame had W 10x21 perimeter members and C 6x10.5 purlins connected by light clip angles. The frame was supported on a roller system such that it had negligible shear resistance prior to attaching the diaphragm panels. A photograph of the frame with diaphragm is shown in Fig. 5.

The steel deck used in these tests was galvanized with a one ounce coating for all tests except P3-6 which was uncoated steel. The panels were corrugated with nominal depths of 9/16", 1", and 1-3/8" respectively for types 1, 2, and 3. The type is indicated as the second digit in each test number. Standard tensile test coupons were taken from each shipment of material. The coating was removed and tests made resulting in the material properties shown in Table 1 .

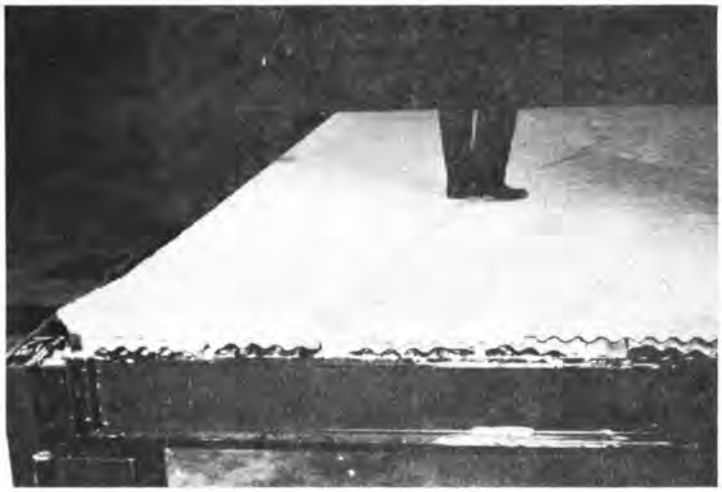


Figure 5. Diaphragm C1-1 at failure.

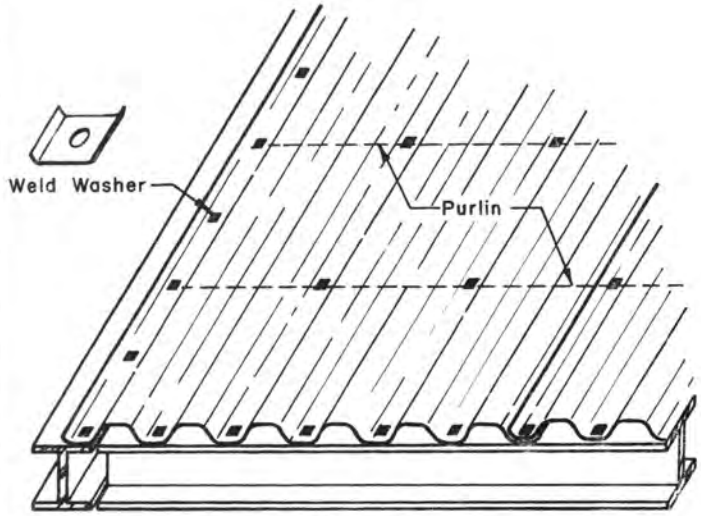


Figure 6. Typical Weld Layout.

Welded connections were made with E6013 electrodes through welding washers having 3/8" holes. Welds along the longitudinal edge of the diaphragm were often spaced closer than on the sheet sidelaps and in all cases, sidelap welds were made over the purlins. No sidelap welds were made between purlins. Weld layouts following the scheme in Fig. 6 are shown in Table 1. The following definitions apply:

- Longitudinal Edge Weld: along the edge of the test frame
- Sidelap Weld: on sheet sidelaps at purlin supports
- End Weld: across panel ends or perimeter members
- Purlin Weld: across panels at purlins.

The insulating concrete was placed on the diaphragm panels without any mesh or rod reinforcement except for tests P3-6 and P3-7. These were reinforced by a No. 3 plain bar across the panel ends and two inch hexagonal wire mesh woven from 19 gage steel wire. The tests were identical except that P3-6 had uncoated panels and the other had galvanized panels.

Tests were conducted following procedures outlined in "Design of Light Gage Steel Diaphragms" (3) and deflection data were reduced according to Eqs. 4 and 6.

<u>Test Data</u>. Data from these tests are presented in the form of total test load P versus deflection in Figs. 7 through 11 and summarized in Table 1. The figures show test results from the seven tests made on diaphragms with vermiculite concrete fill and where appropriate, results are compared to similar diaphragms without concrete fill.

Three 6' x 12' compression test cylinders were taken from each concrete mix. The concrete was a standard 1:6 mix made in accordance with the Vermiculite Institute Specification, "American Standard Specifications for Vermiculite Concrete Roofs and Slabs-on-Grade" (4) as approved by the American Standards Association in April, 1965. The cylinders were tested on the date of the diaphragm test in the air dry state. Separate cylinders were oven dried at 110 to 120°C for 24 hours to determine dry density. In one case, two inch cubes were removed from the diaphragm after completing the tests and compression tests made on them. Concrete test results are given in Table 1.

<u>Discussion</u>. The most noticeable influence on the concrete fill is its effect on diaphragm stiffness. As can be seen in Figs. 7 and 8, the fill resulted in virtually rigid diaphragms compared to otherwise identical test specimens. The stiffness as measured by the secant

Table 1. Summary of Test Data

Test No.		C1 - 1	C3 - 2	P1 - 1	P3 - 2	P3 - 3	P3 - 4	P3 - 5	P3 - 6	P3 - 7	C1 - 1A	C3 - 2A
Purlin Sp. (ft)		4'-0"	6'-8"	5'-0"	5'-0"	5'-0"	4'-0"	3'-4"	5'-0"	5'-0"	4'-0"	6'- 8"
Welds[a]	ends	1'-3"	13½"	each valley	each valley	each valley	each valley	each valley	each valley	each valley	1'-3"	13½"
	Purlin	1'-3"	13½"	alt. valley	each valley	each valley	each valley	alt. valley	each valley	each valley	1'-3"	13½"
	Longitud. edges	4'-0"	6'-8"	2'-6"	2'-6"	2'-6"	2'-0"	1'-8"	0'-8"	0'-8"	4'-0"	6'-8"
Conc. f'_c (psi)		171	154	157	160	142	105	110	158	162[b]	none	none
Conc. Den. (pcf)[c]		-	25.2	27.0	26.6	25.0	24.1	26.0	31.1	-	none	none
Steel Yield (ksi)		101	112	105	-	96	96	96	96	96	101	112
Steel Thick.(in)[d]		0.0165	0.0246	0.0192	0.0355	0.0243	0.0243	0.0243	0.0243	0.0243	0.0165	0.0246
S_u (#/ft)		585	640	1055	1650	1350	1440	1550	2400	2280	190	295
G' (k/in)		58.8	180	193	1056	259	307	354	384	429	13.4	7.7

[a] Welds made through 16 gage washers having 3/8" holes.
[b] Three tests on 2" cubes showed strengths of 225 psi.
[c] Oven dried
[d] Uncoated thickness.

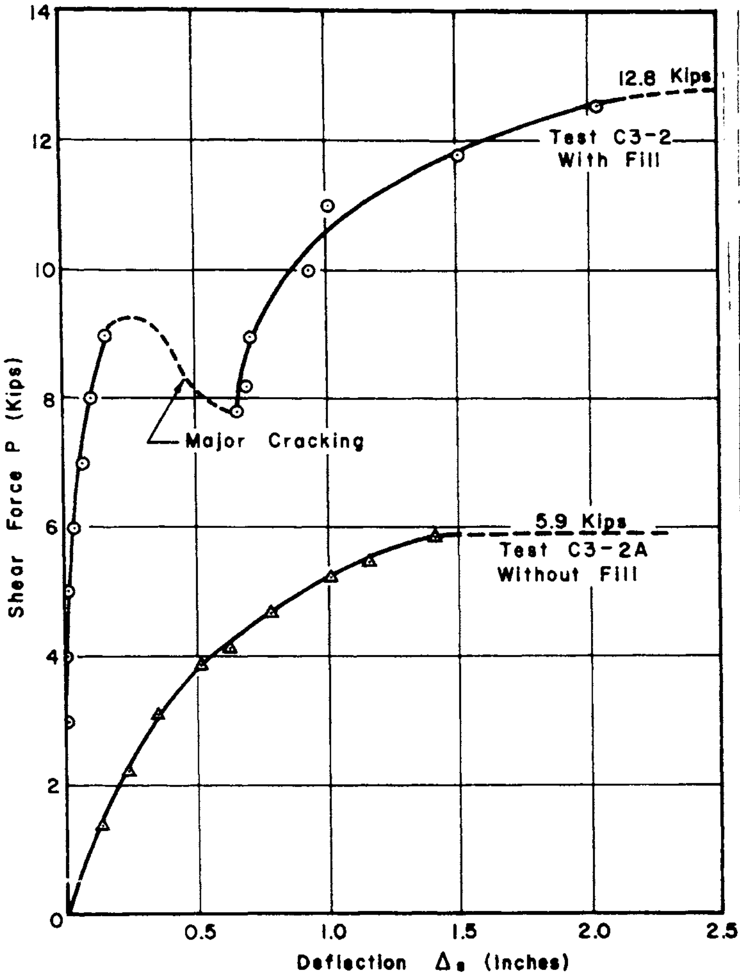


Figure 7. Type C3 Diaphragm Tests.

procedure outlined earlier, increased by 4.4 and 23.4 times over the values for specimens without concrete.

Discussions by Ellifritt (5) have shown that stiffness and failure modes are dependent on the warping restraints present in the system. A very weak connection added at some critical point might be sufficient to change the failure mode from buckling to one involving weld failures. This is what the concrete fill does. Eventhough weak and having compressive strengths on the order of 150 psi, it retarded buckling and out-of-plane bending and resulted in a failure mode involving weld strength for every case tested.

The results in Fig. 9 are from diaphragms with the major variable being panel thickness which was 0.0192, 0.0243, and 0.0355 inches for P1-1, P3-3 and P3-2 respectively. The loads at failure divided by the thickness are 1100, 930, and 1110, indicating that panel thickness may be more or less linearly related to diaphragm strength.

Figure 10 compares results from similar diaphragms tested with purlin spacing as the primary variable. Since interior sidelap connections were made at the purlins only and longitudinal edge welds at half the purlin spacing, strength should be examined in terms of this spacing. Comparing strengths at 0.2" deflection times the purlin spacing gives 2.5 x 21.4 = 54, 2 x 25.6 = 51, amd 1.67 x 31 = 52 for P3-3, P3-4, and P3-5. The consistency of these numbers and the previous comparisons indicate that strength for the unreinforced slabs is directly related to panel thickness and the purlin or edge weld spacing. An attempt was made to correlate strength to the welds along

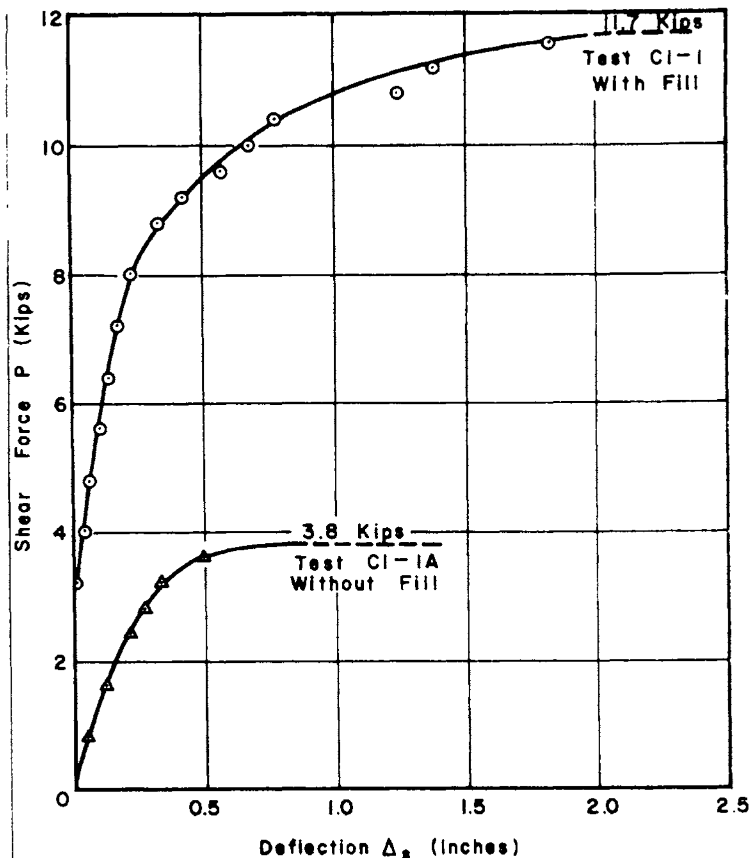


Figure 8. Type C1 Diaphragm Tests.

the longitudinal edge of the diaphragm or the welds around the perimeter of an interior panel. The first resulted in a predicted ultimate shear strength per foot of

$$S_{u1} = 12.4\ (10^4 t)\ \frac{n}{\ell} \quad \text{(plf)} \tag{8}$$

where n is the number of fasteners along the longitudinal edge of the edge panel, t is the panel thickness in inches, and ℓ is the panel length in feet. This formula would govern when edge welds control in unreinforced diaphragms.

When the interior panel failure governs in unreinforced diaphragms

$$S_{u2} = 15.97\ (10^4 t)\ \frac{k}{w\ell} \quad \text{(plf)} \tag{9}$$

where w is the panel width in inches and k is determined following Fig. 12. The purlins were assumed flexible and the frame rigid across the panel ends. The panel would tend to deform as shown transferring forces into welds made on the test frame. Assigning a value of 1 is assigned to welds most remote from the panel centerline and smaller values to closer welds. For example in Test P3-3, k is

$$k = 10 \times 1 \times 15 + 4 \times \frac{2}{3} \times 10 + 4 \times \frac{1}{3} \times 5 = 183$$

other values are similarly determined. Using these formulas the following comparisons obtain.

Table 2. Unreinforced Diaphragm Comparisons

Test	n	S_{u1}	k	S_{u2}	Test S_u
C1-1	6	615	180	790	585
C3-2	4	610	108	785	640
P1-1	9	1070	203	1035	1055
P3-2	9	1980	172	1625	1650
P3-3	9	1355	183	1185	1350
P3-4	11	1655	213	1375	1440
P3-5	13	1960	243	1570	1550

Using Eqs. 8 or 9, whichever yields the smaller value, is seen to give results close to the test value. The worst estimate is for Test P3-3 where the error is 12%. However, the average error in predicting the shear strength is about 3% neglecting P3-3.

The test results from P3-6 and P3-7 are shown in Fig. 11. These diaphragms were reinforced using #19 2" x 2" wire mesh and #3 bars across the panel ends. They were heavily welded along the diaphragm longitudinal edge. Using Eq. 8 would give a predicted ultimate shear of 4670 plf. Equation 9 gives S_u = 1185 indicating that the welds on interior panels should control strength. Even-though the concrete fill was quite weak, the reinforcing was successful in the transfer of shear across interior laps reducing loads on interior welds. Data are insufficient to develop any predictions but it seems that such reinforcement may increase the strength 50 to 60% if sufficient edge welds are present.

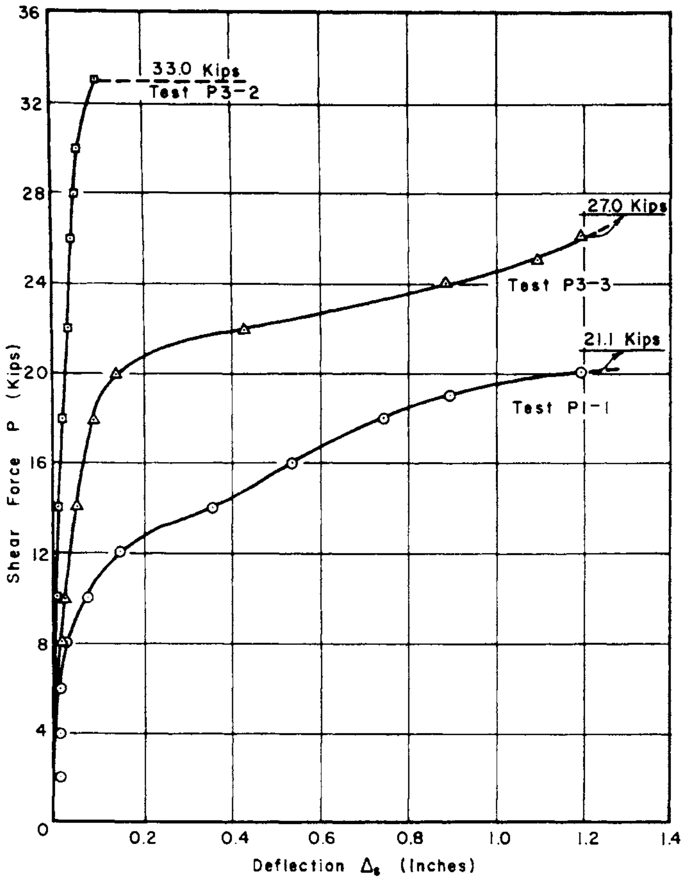


Figure 9. Type P Diaphragms with Fill.

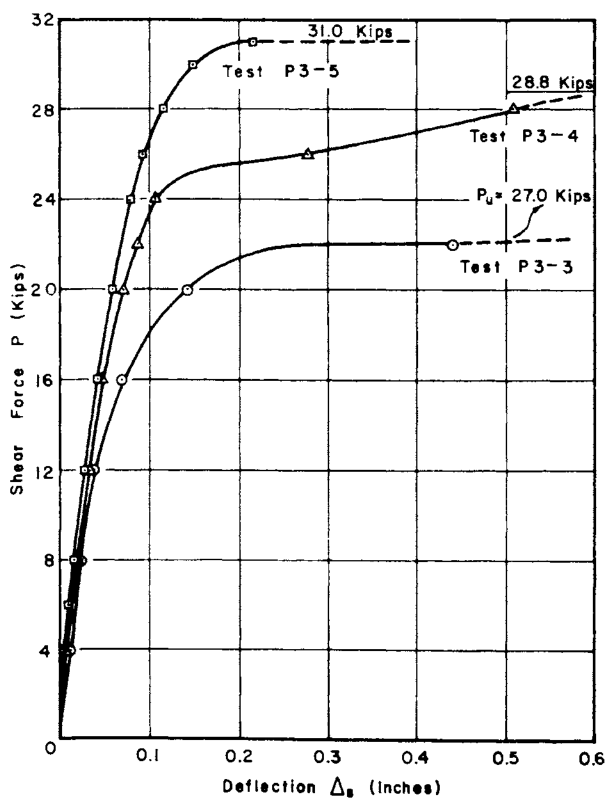


Figure 10. Effects of Purlin Spacing.

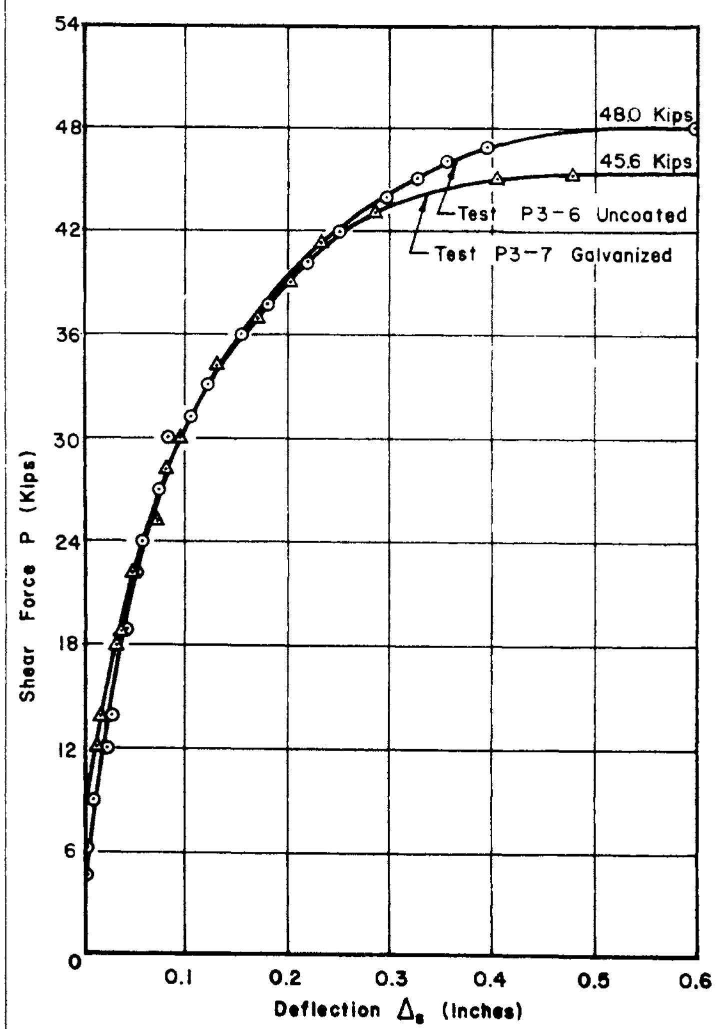


Figure 11. Effect of Coating.

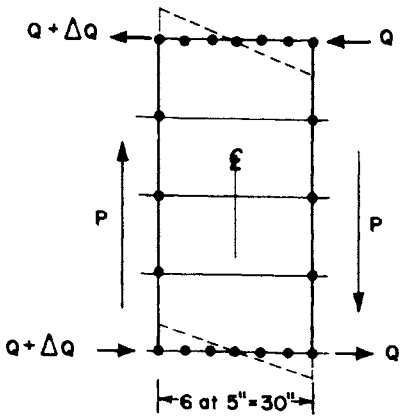


Figure 12. Perimeter Welds on Interior Panel (P3—3).

Acknowledgement. Mr. Robert V. Ault of the Bowman Building Products Division of Cyclops Corporation and Mr. Thomas J. Jones of the Wheeling Corrugating Company have spent many hours working on these projects and are to be commended for their interest in and support of these studies.

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Appendix II - Notation

a = frame dimension perpendicular to load direction (ft)

b = frame dimension parallel to load direction (ft)

D_1 - - D_4 = dial gage displacement (in)

f'_c = concrete strength (psi)

G' = shear stiffness as in Eq. 7 (k/in)

k = measure of weld pattern value

ℓ = panel length (ft)

n = number of welds per panel on exterior panel edge

P_u = ultimate jack load (kips)

S = shear per foot (plf)

S_u = ultimate shear per foot (plf)

t = panel base metal thickness (in)

Δ_s = shear deflection (in)