STRUCTURAL PERFORMANCE OF PLYWOOD AND STEEL CEILING DIAPHRAGMS

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Wind-bracing of farm structures with engineered structural diaphragms has not been fully accepted, partly because traditional wood-frame constructions seem to develop considerable wind resistance without the engineered connections promoted by the Canada Plan Service. This paper reports the results of simulated wind loads on three farm building ceilings, as follows: (1) *conventional*, 7.5-mm sheathing Douglas fir plywood nailed directly to trusses spaced at 600 mm, plyclips at panel edges mid-span between trusses; (2) *improved plywood diaphragm*, 7.5-mm sheathing Douglas fir nailed four edges to a 1200 × 1200-mm grid under trusses spaced at 1200 mm; (3) *screwed sheet steel diaphragm*, power-screwed under trusses spaced at 1200 mm. With a typical stud wall height of 2.4 m and a 1/10 hourly wind pressure of 0.64 kN/m² (Lethbridge), the conventional plywood ceiling would be safe to a ceiling length/width ratio (*L*/*W*) up to 3.67, the improved plywood ceiling to 6.04 and the screwed steel ceiling to 5.35.

INTRODUCTION

Structural diaphragms can provide the most convenient way to resist horizontal forces (wind primarily) acting to overturn typical farm buildings. This is particularly true for insulated wood-frame buildings for animal production and food storage; here, the requirement to insulate and finish the interior surfaces usually dictates a cladding material which can also provide a structural diaphragm system for wind-bracing.

Canada Plan Service (CPS) building designs have for many years shown structural diaphragms. These are preferred over wall-to-roof knee bracing which often interferes with mechanized operations in the barn. Unfortunately, installation of engineered ceiling and wall diaphragms is seldom properly completed in practice.

Turnbull (1964) emphasized the importance of connecting all four edges of each ceiling panel to adjacent panels in order to achieve the most effective transmission of diaphragm-ceiling shear forces to the building sidewalls and endwalls. When fastening a ceiling of panel products such as plywood to the underside of roof trusses, practical construction problems become apparent when the designer attempts to provide backing for fastening all four edges of the ceiling panels to each other and to the trusses above.

The 'conventional' method of nailing plywood ceiling panels directly to roof trusses spaced at 0.6 m presents two problems; (1) the butted end joints of the plywood meet each other at the narrow bottom edge (38 mm) of the wood roof truss, which gives (at best) only 19 mm for each row of nails; and (2) longitudinal joints at the edges of each ceiling panel can only be nailed at each truss (at 0.6-m intervals), a spacing usually inadequate for shear transfer.

For these reasons, the 'improved' CPS method is to add a 1.2 × 1.2-m square grid of 38-mm strapping to the lower edge of the roof trusses (see (2) and (3), Fig. 1). This gives the required panel support for two adjacent rows of nails at all four

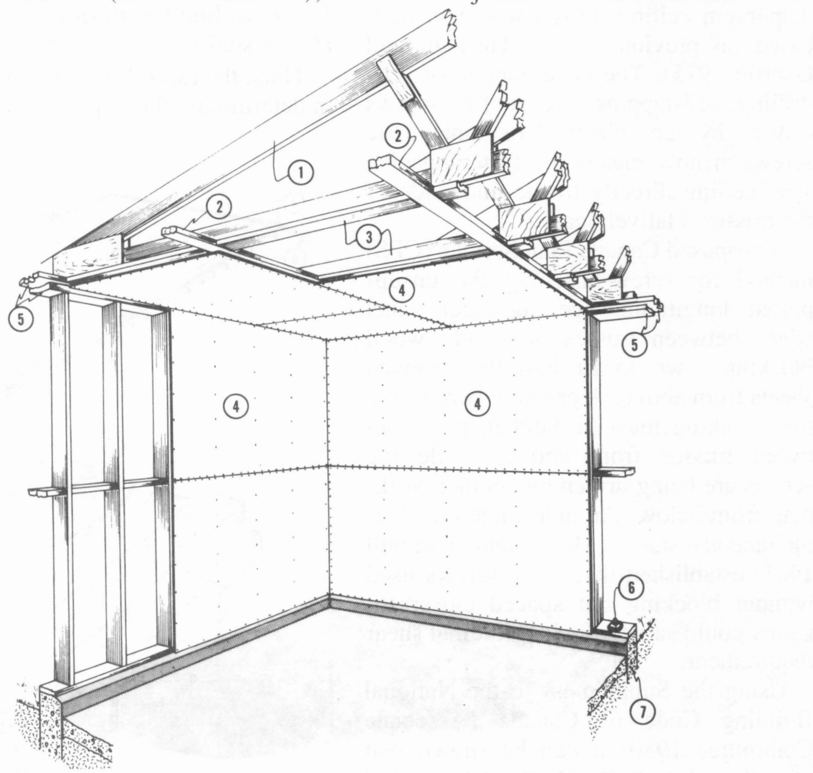

Figure 1. Typical CPS connection details for a plywood ceiling/wall diaphragm system. (1) Trusses spaced at 1200 mm oc; (2) 38 × 89-mm strapping nailed to (1) at 1200-mm spacing; (3) 38 × 64-mm blocking nailed to trusses between (2); (4) ceiling and sidewall, 7.5-mm sheathing plywood, nailed all four edges, carries ceiling shear through to sill; (5) double plate; 38 × 140 and 38 × 184 mm, makes a step for fastening ceiling to end and side walls; (6) pressure-treated sill bolted to transmit shear and uplift to (7); (7) concrete foundation.

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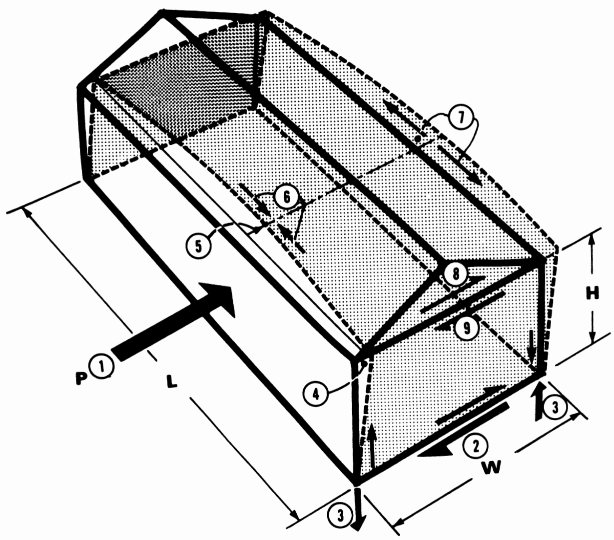

Figure 2. Diaphragm principle for resisting wind perpendicular to the long wall of a typical farm building. (1) Critical wind, direction perpendicular to long wall; (2) foundation horizontal reaction to wind force; (3) foundation reaction to overturning at endwall; (4) shear deformation in end wall diaphragm; (5) shear and bending deformation in ceiling diaphragm; (6) plate beam compression due to ceiling bending; (7) plate beam tension due to ceiling bending; (8) maximum ceiling shear stress; (9) endwall shear stress.

panel edges. However, the extra task of nailing the 38-mm strapping and blocking to the underside of the trusses has not been popular with farm builders.

Consequently a new galvanized steel diaphragm ceiling design was executed, based on previous work (Turnbull and Guertin 1975). The development of self-drilling, self-tapping steel roofing screws driven by an electric or pneumatic screwgun now makes the fastening of a steel ceiling directly to the underside of the trusses relatively easy.

A proposed Canada Plan Service (CPS) method for screw-fastening the unsupported longitudinal ceiling steel panel edges between trusses is to add wood blocking sawn to fit into the grooved sheets from above. A problem here is that this blocking must be held in place between trusses from above, while the screws are being driven by another workman from below. An independent and simultaneous study (Massé and Turnbull 1981) established that stitch-screws used without blocking but spaced closer together could satisfy the longitudinal shear requirement.

Using the Supplement to the National Building Code of Canada (Associate Committee 1980) it can be shown that when based on shear, the allowable wind pressure for a given ceiling diaphragm in a stud-framed farm building with a gable roof slope of 1:3 reduces to:

$$q = 2.2 \frac{SW}{HL} \qquad (1)$$

where, as shown in Fig. 2,

q = allowable wind pressure, (kN/m^2)
S = ceiling shear resistance, (kN/m of width)
W = ceiling width (m)
L = ceiling length (m)
H = stud wall height (m)

Thus, the ratio W/L is a principal factor in determining the required shear strength near the ends of the ceiling where transverse shear forces will be at maximum. In other words, narrow, long buildings will require the greatest shear resistance for a given design wind pressure, q.

The other possible mode of diaphragm ceiling failure is bending, which will develop maximum compression and tension forces in the ceiling edges at the building mid-length (see (6) and (7), Fig. 2). If the ceiling is not continuously connected to the long side walls (as when side air inlets form a slot between ceiling and sidewalls) it becomes necessary to add a 'flange' structure to handle the edge forces due to bending for which the thin ceiling may not be adequate. If, however, the ceiling can be continuously connected to both side and end walls (as in Fig. 1, for example), shear resistance of the sidewalls can easily be made to transmit the ceiling edge forces down to the leeward and windward foundation walls. Either situation may exist in a particular design, and the designer must ensure that ceiling bending resistance is incorporated, one way or the other.

Extension engineers indicated the importance of diaphragm load tests approaching full scale, as a check on the design methods and structural performance of conventional and special ceiling designs. The objectives of this work were: (1) to check the design equations and working stresses derived from previous single-panel shear tests, and (2) to dem-

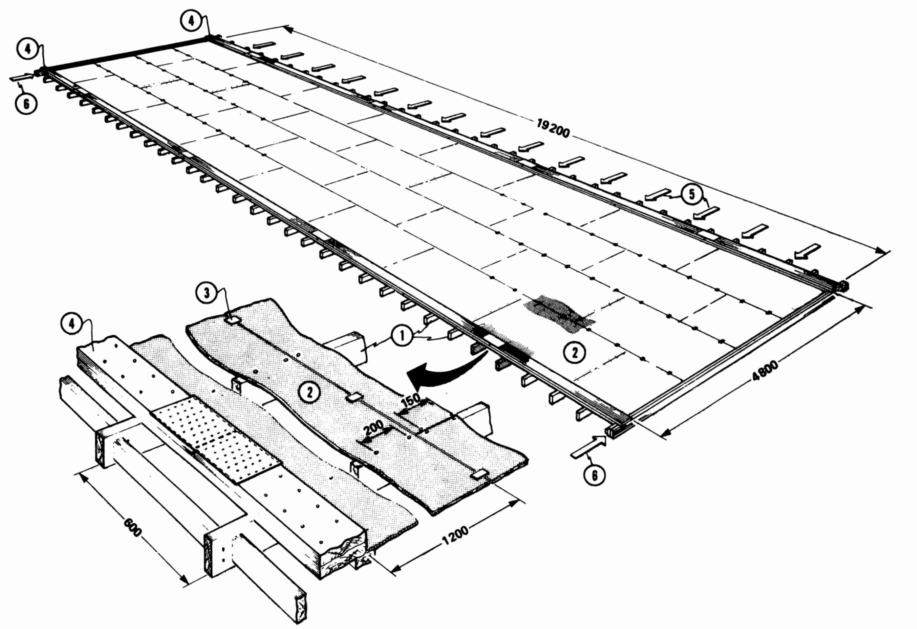

Figure 3. Details of conventional plywood ceiling model. (1) 38 × 140-mm members represent ceiling joists or roof trusses; (2) 7.5-mm sheathing fir plywood, nailed to (1) at 150-mm spacing across ends and 200-mm spacing to others; (3) 7.5-mm steel plyclips midway between (1); (4) beam edge of 2.38 × 184-mm planks with steel splice plates at each staggered butt joint; (5) 15 hydraulic cylinders spaced at 1200 mm; (6) two reaction pads with load cells, anchored to warehouse floor.

onstrate possible size effects resulting from many panels interacting within diaphragm specimens more like full scale.

THE EXPERIMENT

Within budget limitations it was determined that only three tests could be performed, as detailed in Fig. 3, 4 and 5. The Fig. 3 test was designed to simulate the conventional plywood ceiling/truss system most popular in the western provinces, where trusses are usually spaced at 600 mm. Plyclips were added midway between the 'trusses' to provide vertical plywood-edge support. Trusses were simulated by 38 × 140-mm members placed on edge and vertically supported to the concrete floor of the rented warehouse used as a laboratory. All ceiling models were built and tested upside down, for ease of fabrication and for observing the effects of loading.

Figure 4 shows a test to simulate the 'improved' plywood diaphragm with a four-edge panel support grid spaced 1200 × 1200 mm, as in CPS plan M-9374. In both conventional and improved models, nailing of the plywood to the supporting framing was done with 3.2 × 38-mm large-head galvanized roofing nails.

Figure 5 illustrates a galvanized sheet steel ceiling based on CPS plan M-9371. The steel was a deep-rib galvanized siding profile called 'W-R-L Diamond Rib,' by Westeel-Rosco Ltd. This profile (and roughly similar profiles by other Canadian manufacturers) is now a popular ceiling material in farm buildings. Steel base thickness (before galvanizing) was 0.30 mm (30 gauge). The sheets covered 914 mm wide (allowing for edge-laps) and were precut to 2590-mm lengths to ensure end-laps when spanning two truss spaces totalling 2400 mm. Steel sheets were lapped at all four edges (except at the ceiling perimeter) and screwed to the framing with 4 × 25-mm (no. 8 × 1-inch) self-drilling, self-tapping hex-head roofing screws with neoprene washers. The screws were driven with a variable-speed electric hand drill with socket wrench attached to fit the screw-heads. (A proper screw-gun equipped with a slip-clutch adjustable to just tighten the screws without stripping threads would have been a real improvement.)

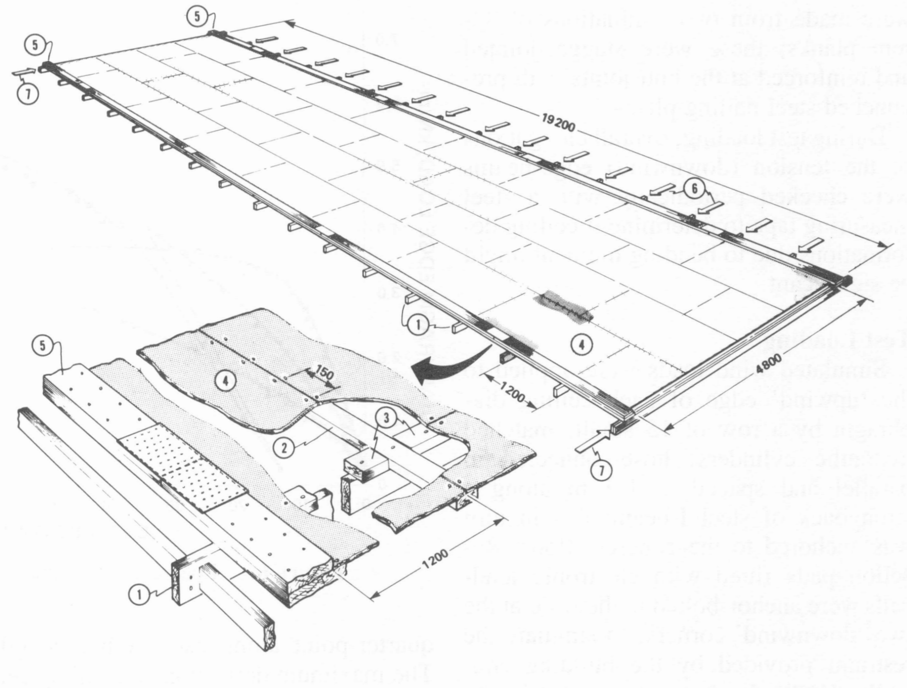

Figure 4. Details of improved plywood diaphragm model. (1) 38 × 140-mm members represent ceiling joists or roof trusses; (2) 38 × 89-mm longitudinal strapping spaced at 1200 mm; (3) 38 × 63-mm blocking at (1) and between (2); (4) 7.5-mm sheathing fir plywood, perimeter of each sheet nailed to (2) and (3) at 150-mm spacing; (5) beam edge of 38 × 184 and 38 × 235 mm with steel splice plate at each staggered butt joint; (6) 15 hydraulic cylinders spaced at 1200 mm; (7) two reaction pads with load cells, anchored to warehouse floor.

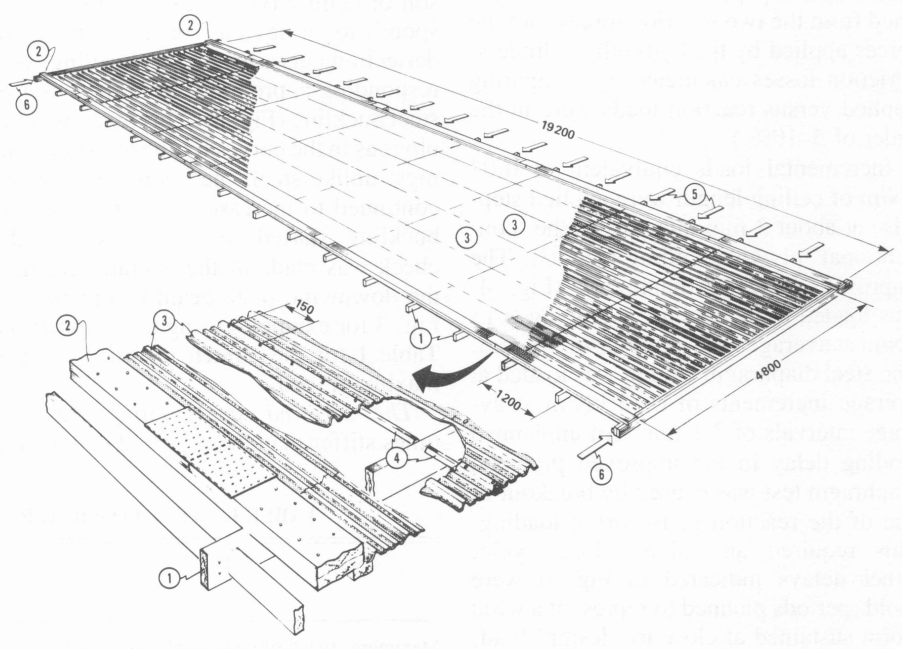

Figure 5. Details of screwed sheet steel diaphragm. (1) 38 × 140-mm members represent ceiling joists or roof trusses; (2) beam edge of 2.38 × 184 mm with steel splice plate at each staggered butt joint; (3) 0.30 × 900 × 2540-mm galvanized steel siding sheets, with centerline, lapped edges and ends screwed at 150-mm spacing to trusses (1) and blocking (4); (4) blocking cut to fit grooves, from 38 × 38 mm, loose fit between (1); (5) 15 hydraulic cylinders spaced at 1200 mm; (6) two reaction pads with load cells, anchored to warehouse floor.

Ceiling Bending

No attempt was made in any of the three tests to precisely model the binding resistance of the two long building walls to a bending moment in the ceiling. To resist bending tension and compression at the long ceiling edges, built-up edge beams were made from two laminations of 38-mm planks; these were stagger-jointed and reinforced at the butt joints with pre-punched steel nailing plates.

During test loading, overall elongations of the tension (downwind) edge-beams were checked periodically with a steel measuring tape to determine if ceiling deformations due to bending moment might be significant.

Test Loading

Simulated wind loads were applied to the 'upwind' edge of each ceiling diaphragm by a row of 15 small, matched hydraulic cylinders, hose-connected in parallel and spaced at 1.2 m along a strongback of steel I-beam; this in turn was anchored to the concrete floor. Reaction pads fitted with electronic load-cells were anchor-bolted to the floor at the two 'downwind' corners, to simulate the restraint provided by the building end-walls. Hydraulic force developed in the loading cylinders was indicated by an additional 'dummy' cylinder connected in parallel to the same hydraulic circuit but applying its load to a third electronic load cell. This duplication of force-measuring equipment permitted a check on load losses due to friction between each 'ceiling' and the vertical restraints provided by spaced, omni-directional roller bearings running on the floor. Distributed loads tabled in this paper were net loads calculated from the two reaction forces, not the forces applied by the hydraulic cylinders. (Friction losses calculated by comparing applied versus reaction loads were in the order of 5–10%.)

Incremental loads equivalent to 0.07 kN/m of ceiling length were applied stepwise at about 3-min intervals to the 'conventional' plywood ceiling (Fig. 3). The improved plywood diaphragm (Fig. 4) was loaded at increments averaging 0.13 kN/m at average time intervals of 3.6 min. The steel diaphragm ceiling was loaded at average increments of 0.13 kN/m at average intervals of 3.1 min. An unplanned loading delay in the improved plywood diaphragm test was caused by breakout of one of the reaction pads during loading. This required an unload/reload cycle. Other delays indicated in Fig. 6 were 'hold' periods planned to represent a wind storm sustained at close to 'design' load, to determine the creep characteristics of each construction.

During loading, lateral deflections at the longitudinal centerline of the ceilings were read by means of a surveyor's transit, reading scales fixed to each ceiling centerline, at both ends and at each

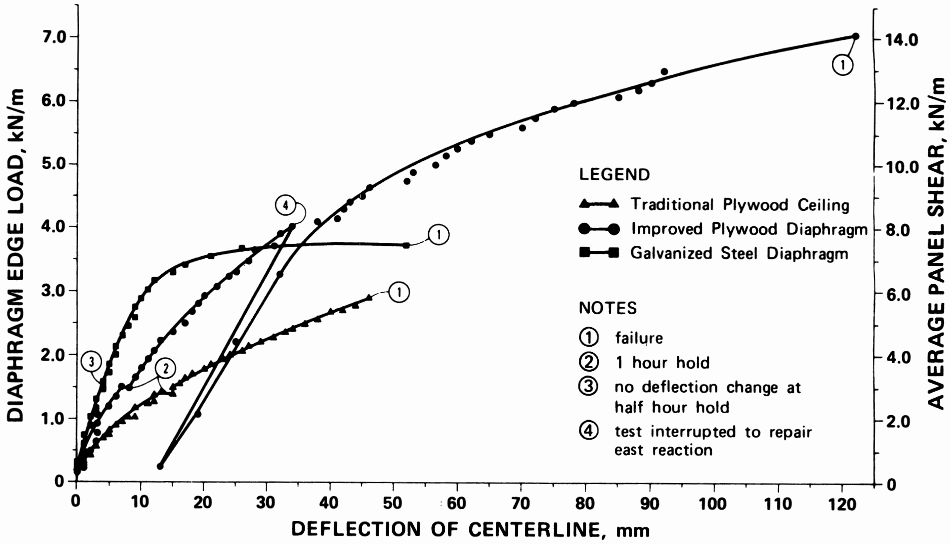

Figure 6. Load/deflection diagrams for 4.8 × 19.2-m diaphragm ceilings.

quarter-point along each ceiling length. The maximum deflections (at ceiling center-point) were used in plotting Fig. 6. In the improved plywood diaphragm and the screwed steel diaphragm, edge-to-edge panel longitudinal slips were also recorded near the ends of the ceilings where shear stress would be at maximum.

RESULTS AND DISCUSSION

Table I and Fig. 6 summarize the results of the load tests. In Table I, 12 mm deflection was chosen for the first comparison of ceiling types, since 12 mm corresponds to an obvious 'break' in the load/deflection curve for the steel ceiling, corresponding approximately to the onset of steel buckling (Fig. 8). This break was not obvious in the curves for the plywood ceilings; unlike steel, the loads for plywood continued to increase well after diagonal buckling started to occur. A periodic check was made of the overall stretch of the downwind plate beam members ((4), Fig. 3 for example) using a steel tape; see Table I for the stretch recorded at each maximum load.

The screwed steel ceiling was 2.4 times stiffer than the plywood ceiling and 1.6 times stiffer than the improved plywood ceiling up to 12 mm deflection. Above the break in the load/deflection curve for steel, diagonal buckling of the steel (at one end in particular) caused the steel performance to go abruptly from 'stiff' to 'soft', a failure mode quite unlike that of plywood.

Some screws also popped out near the ends of the steel ceiling where shear stresses were greatest. The screws that popped may have been over-driven (stripping threads in the wood framing) or they may have simply had the random misfortune of being located at a point where the crest of a buckling 'wave' would cause maximum uplift force under the screw head. To prevent over-driving and stripping the screws, the use of a screwgun with an adjustable slip-clutch is highly recommended.

The conventional plywood ceiling was neither stiff nor strong as compared with the other two tests. The plyclips appeared to provide longitudinal edge-to-edge support for the plywood sheets; combined with the 'trusses', the plyclips provided an edge support spacing of 300 mm. It was observed that when plywood buck-

TABLE I. TEST LOADING RESULTS FOR DIAPHRAGM CEILINGS

	Conventional plywood ceiling	Improved plywood diaphragm	Screwed sheet steel diaphragm
Maximum stretch of downwind plate (mm) @ deflection (mm)	1.0 @ 46	3.5 @ 117	2.0 @ 11
Load (kN/m) @ deflection (mm)	1.29 @ 12	2.08 @ 12	3.15 @ 12
Load (kN/m) @ deflection (mm)	2.83 @ 46	4.62 @ 46	3.71 @ 46
Max. ceiling load (kNm) @ deflection (mm)	2.83 @ 46	7.03 @ 117	3.71 @ 52
Design transverse unit shear load, *S* (kN/m)	2.54	4.18	3.77

ling commenced the plyclips were contributing a great deal of edge support; from this, one might suppose that buckling would have occurred at a much lower load without the plyclips. It was also noted that as the conventional plywood ceiling test progressed, some of the plyclips became wedged more tightly between adjacent sheets, and they probably contributed increasing resistance to longitudinal ceiling shear.

Maximum steel shear strength, as limited by buckling, was derived from Fig. 6. The onset of buckling (which corresponds to the 'steel' curve break at 12 mm deflection) occurred at a ceiling load of 3.15 kN/m. This becomes steel shear force per unit of span across the ends of the ceiling (see right-hand vertical scale, Fig. 6), as follows:

$$3.5 \text{ kN/m} \times 19.2 \text{ m}/(2 \times 4.8 \text{ m}) = 6.3 \text{ kN/m}.$$

This value is somewhat lower than the mean buckling shear of 6.91 kN/m (± 0.24 standard error) determined by Massé et al. (1981). Several factors might explain the value obtained here; test duration was 2.2 h whereas Massé's tests were completed in about 0.5 h, and his shorter load duration could increase the apparent resistance of his test panels. Other factors were the screw popping (mentioned previously) which may have contributed to the onset of buckling, and the size effect of the larger specimen wherein the distribution of shear across the ceiling width would not be quite uniform (as was assumed in the above calculation).

Measurements of joint slip were also recorded between longitudinal edges of the 'screwed sheet steel' diaphragm. These results are plotted against diaphragm distributed loads (Fig. 7).

Figure 7 is also scaled for shear load per screw. The break in the steel-to-steel panel slip versus shear load per screw curve occurred quite sharply at 1.01 kN/screw, giving an approximate value for the maximum strength of this type of longitudinal lapped steel-to-steel connection with blocking. Massé et al. (1981) found a corresponding mean value of 1.3 kN/screw, a somewhat higher value probably due to use of smaller test specimens.

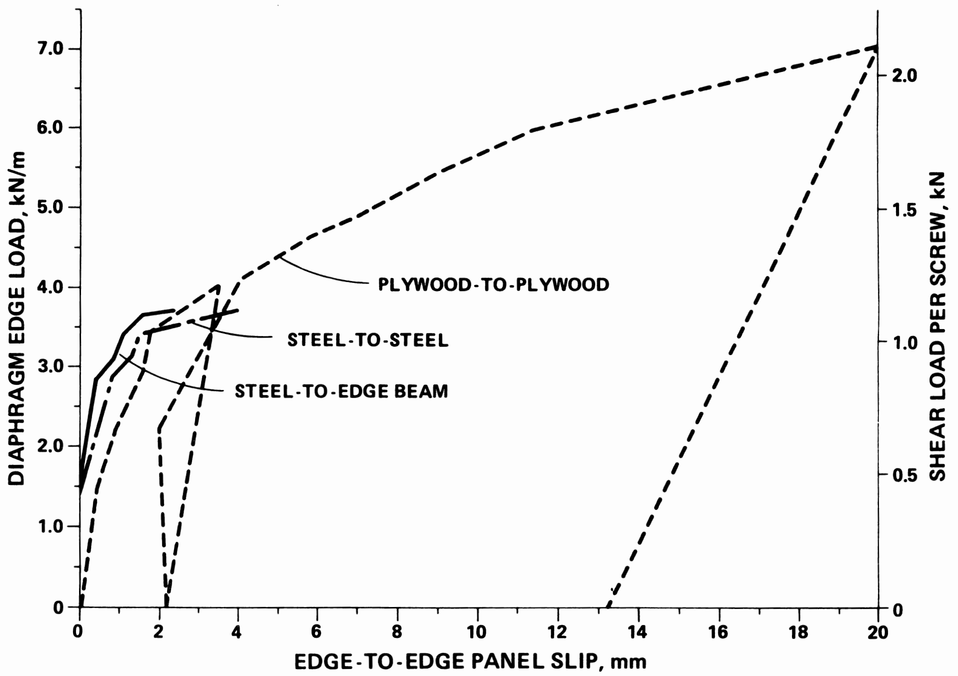

Figure 7. Load/slip diagrams for panel-to-panel edge slip for the improved plywood diaphragm and the screwed sheet steel diaphragm.

Deflection and Serviceability

The Canadian Farm Building Code (Standing Committee on Farm Buildings 1977) stipulates that the deflection of a structural component (ceilings included) is not to be so excessive as to interfere with the operation of doors, windows or equipment. No numerical definition of this limit is given, for ceilings or other components.

If one applies the more restrictive National Building Code of Canada recommendation for nonfarm structures, then the maximum allowable deflection would be L/180, or 107 mm for the ceiling length tested. In all cases, failure of the diaphragm had occurred prior to reaching this deflection limit. In other words, the diaphragms tested here are all more than adequately 'stiff' when loaded to safe limits. The deflections measured were useful to indicate the onset of inelastic yielding and subsequent failure.

SUMMARY AND DESIGN IMPLICATIONS

The conventional plywood ceiling was outperformed by both the improved plywood diaphragm and the galvanized steel diaphragm. This is not to say that the conventional plywood ceiling is not adequate for wind-bracing some farm buildings, but rather that the improved plywood diaphragm and the galvanized steel diaphragm can be used to wind-brace taller buildings and buildings having greater length/width ratios.

Another implication is that where a stiffer diaphragm ceiling is required, the screwed steel ceiling (within its safe limit of resistance) will provide the greatest stiffness. This is due probably to the increased connection stiffness made possible by lap-screwing the full perimeter of each steel sheet.

A design shear stress for steel ceiling diaphragms as tested here can be determined from either the elastic limit shear strength of the test ceiling or the ultimate shear strength. For low-human-occupancy farm buildings the Canadian Farm Building Code (Standing Committee 1977) requires structural assemblies to withstand 2.0 times design load. Thus (from the ultimate strength of 7.42 kN/m, Fig. 6) the design shear load for 0.30-mm (30-gauge) steel becomes 7.42/2.0 = 3.71 kN/m. This value is well within the elastic performance range for the steel ceiling, and is lower than the 4.00 kN/m recommended previously for a similar thickness but a different steel profile (Turnbull and Guertin 1975).

It appeared that failure of the steel ceiling was a combination of sheet steel buckling and tearing at the screws, implying that a reasonably balanced design was achieved with panel perimeter screws spaced at 150 mm, a spacing corresponding to that of the major steel ribs.

As shown in Fig. 6, note (2), both plywood ceilings demonstrated some creep effects during a 'hold' period in the loading; this indicates that some adjustment for load duration is appropriate. Design loads for wind (assumed duration 24 h) were derived from test results taken from a much shorter loading period (1.38 and 1.9 h to design load, for conventional and

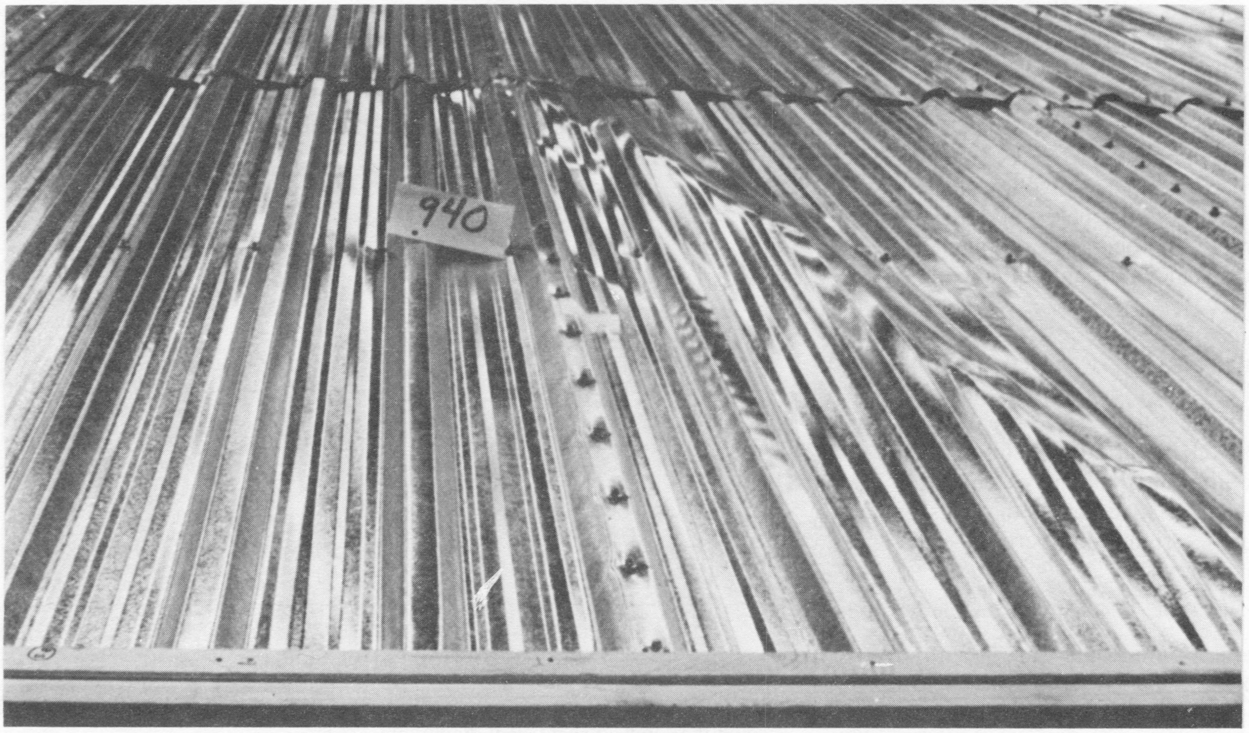

Figure 8. Screwed sheet steel diaphragm after onset of buckling failure at the end zone.

improved plywood ceilings, respectively). The steel ceiling showed no creep effect, indicating that load duration can be ignored in this case.

Using a graphic procedure by the Wood Research Laboratory at Purdue University (Anonymous 1973), it was estimated that the 'test' versus 'wind' load duration factor for the 'conventional' plywood ceiling would be 0.898; that for the improved plywood diaphragm would be 0.904 (slightly greater test time from zero up to design load).

Applying the above load duration multiplying factors for wood and a 2.0 safety factor to the test shear loads at 46-mm ceiling deformation, design unit shear loads S are calculated as follows:

conventional plywood, $5.66 \times 0.898/2 = 2.54$ kN/m
improved plywood, $9.24 \times 0.904/2 = 4.18$ kN/m
screwed steel, $7.42 \times 1.0/2 = 3.71$ kN/m

A design shear load $S = 4.18$ kN/m for the improved plywood diaphragm is well below the value of 11.5 kN/m recommended previously (Turnbull and Guertin 1975) for 7.5-mm Douglas fir plywood. In this test a nail spacing of 150 mm was inadequate to develop the full shear strength of the plywood, and a direct comparison, therefore, has little significance. In fact, a closer nail spacing would not only strengthen the inter-panel connections but could also raise the plywood shear stress at which buckling is first observed.

Using Eq. 1 and assuming a typical stud wall height $H = 2.4$ m and 1/10 hourly wind pressure $q = 0.64$ kN/m^2 (Lethbridge, Alberta), a conventional plywood ceiling as tested here could be safe for buildings with a L/W ratio up to 3.67, the improved plywood diaphragm would be safe for L/W up to 6.04 and the screwed steel ceiling would be safe for L/W up to 5.35.

The improved plywood diaphragm has ample reserve capacity to be further strengthened by reducing the nail spacing (from 150 to 75 mm, for example), but the problem here is that builders will not willingly drive this number of nails into a ceiling. Also the resistances of other components of the diaphragm system may become questionable (for example, ceiling-to-wall connections, and the walls themselves).

Traditional structural component deflection limitations quoted in codes would allow ceiling deflections which would be well beyond failure loads for the types of ceilings tested here. Therefore, traditional deflection limitations are apparently not applicable to structural diaphragm ceilings.

FUTURE DESIGN WORK

With minor modifications, the steel diaphragm principles described in this paper could be applied to roofing steel on buildings without ceilings. One problem to be resolved is the perimeter attachment of the roofing steel to the end and side walls. Another problem involves a continuous connection between adjacent roof-planes where they meet at the ridge.

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