Test on Diaphragm Behavior of Dry-Floor System with Steel-Edged Gypsum Planks

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THE U. S. Steel Corporation and the U. S. Gypsum Company have cooperated in testing a lightweight "dry" steel-edged gypsum-plank floor system for steel-framed apartment buildings. Less dead weight, reduced construction time, and all-weather installation are the chief advantages of the dry floor.

In actual service, floors, in addition to meeting the performance criteria with regard to vertical loading, sound transmission, and fire resistance, can also act as diaphragms distributing lateral loads from unbraced areas into braced frames. Although the diaphragm behavior of monolithic and uniform floors is relatively easily predicted by analysis, the gypsum-plank floor, being a complex composite system, cannot be easily analyzed. Therefore, a series of diaphragm tests was conducted on a full-scale floor system, and the result is reported herein.

Although specific details had to be selected for the steel frame and plank floor specimen, it is recognized that other details may be more advantageous in particular building applications. The degree to which the test results apply to floors using such other details is a matter of judgement and can not be ascertained in a general manner.

TEST SPECIMEN

A full-scale test specimen representing a two-bay portion of floor area between the braced column lines in a typical apartment building was erected at the U. S. Steel Applied Research Laboratory. The floor plan of the building is one of conventional center-corridor apartment building construction. See Fig. 1. The framing plan has 24-ft column spacing, with every other column line braced to resist the wind forces. The floor-to-floor height of the building is assumed to be 9 ft-6 in. The diaphragm action of the floor is to transfer the wind loads, which are transmitted by the exterior wall to the edge of the floor to the braced column lines.

The frame of the specimen is shown in Fig. 2. The beams were connected to 10WF72 stub columns by connections composed of seat angles and high-strength bolts. Intermediate floor supports were provided by 14H4 open-web joists spaced 4 ft c to c in the transverse direction.

The gypsum planks are precast units 2 in. thick, 15 in. wide, and 10 ft long; they weigh approximately 12 lbs per sq ft (psf), 140 lbs each. The plank is reinforced with 22 ga. galvanized-steel tongue-and-groove edges to form

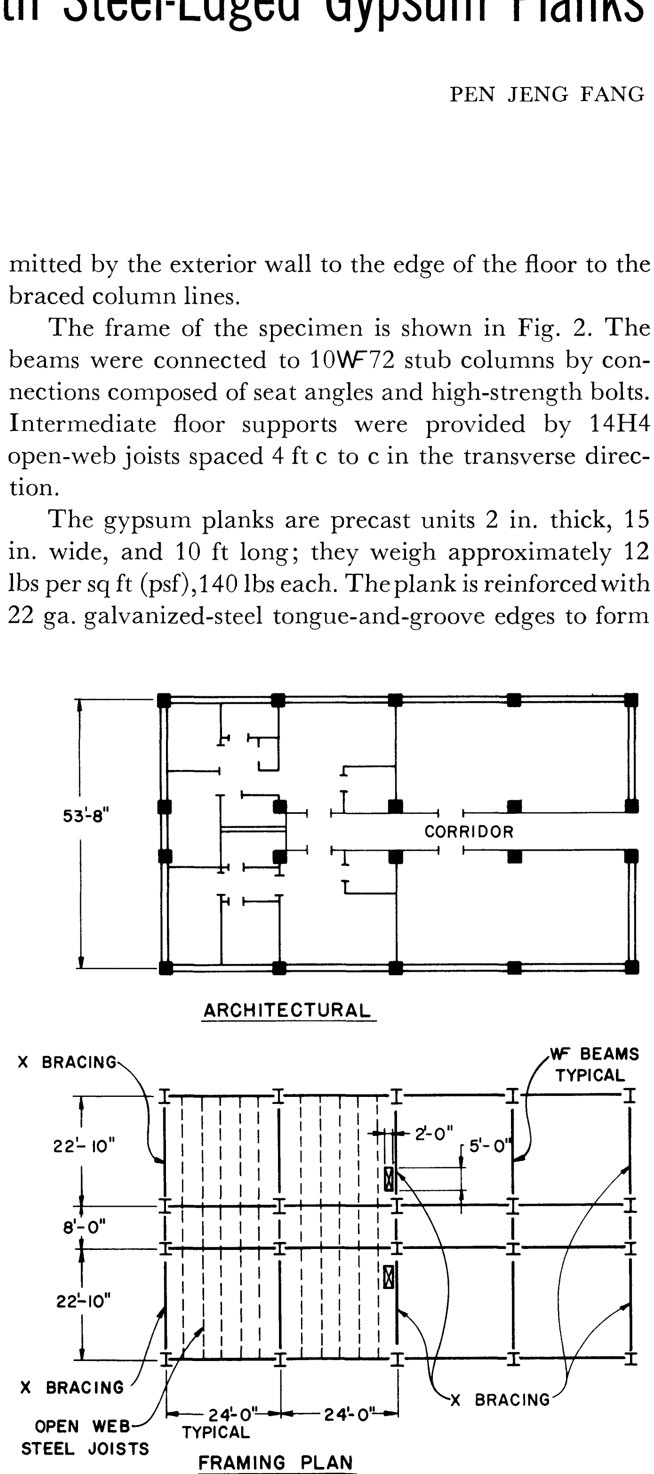

Fig. 1 Partial plan of proposed floor framing

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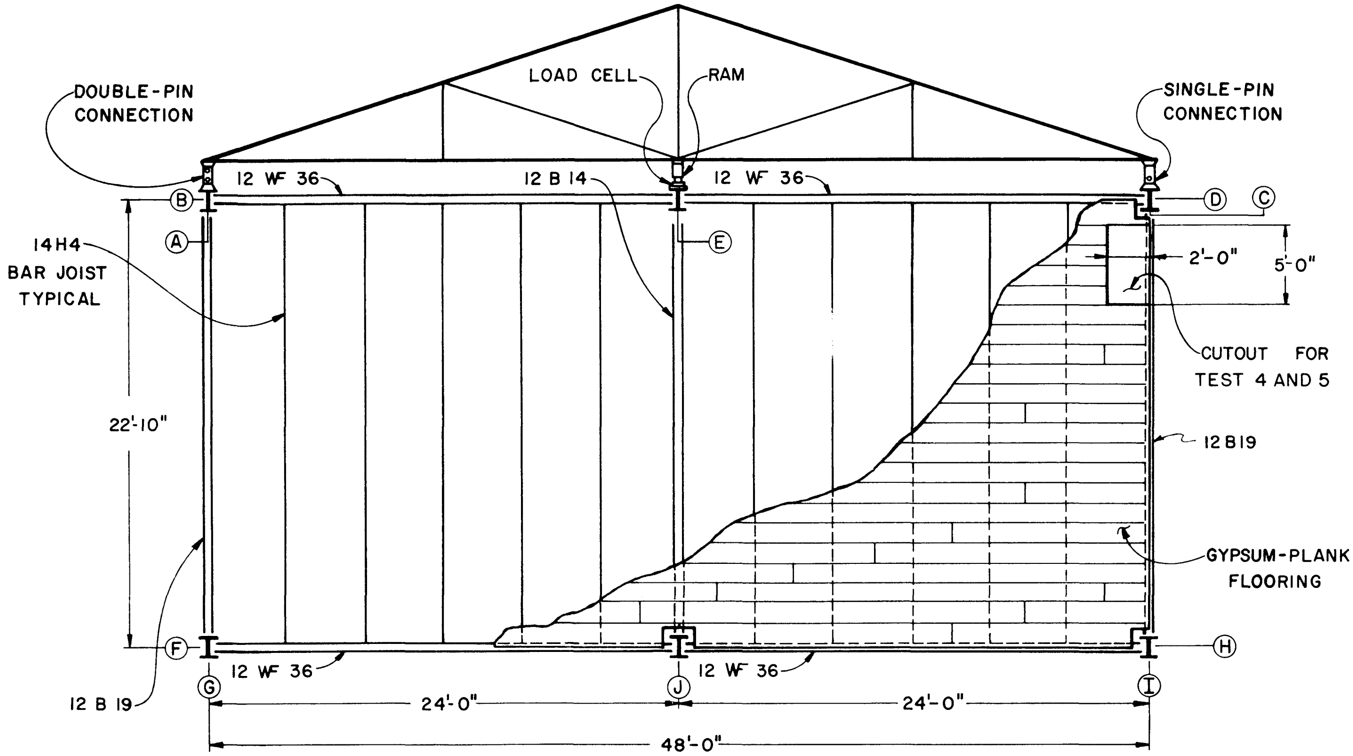


Fig. 2. Plan of test setup

mating joints and with a 16 ga. galvanized-steel-wire mat (Fig. 3).

The gypsum planks were laid longitudinally on the joists by interlocking tongue-and-groove steel edges, which were welded at 4-ft spacings along the seams. The plank edges were also tack-welded to the joists and to the transverse floor beams. To secure shear transfer between the gypsum floor and the longitudinal marginal beams, 2-in. long pieces of 3 × 3 × 3/8-in. angles were welded to the beam flange and the steel edges of the planks at intervals of 4 ft. Also, 12-in. lengths of 10 [25 were welded to the beam flanges adjacent to the columns to support the plank at cutouts around the columns. These connections are shown in Figs. 4 and 5.

After two preliminary nondestructive tests on the complete floor structure were made, a 2 ft × 6 ft opening at one floor corner was made by cutting out the gypsum planks to simulate a pipe chase in the floor, as indicated in Fig. 2.

TEST SETUP

The test setup for determining the diaphragm behavior is shown in Fig. 2. The horizontal floor was end-connected to a horizontal reaction truss. The active load pushing the floor away from the truss was applied at the midspan by a hydraulic ram of 30-ton capacity in conjunction with a 50,000-lb load cell, which was inserted between the surface of the central column and the hydraulic ram.

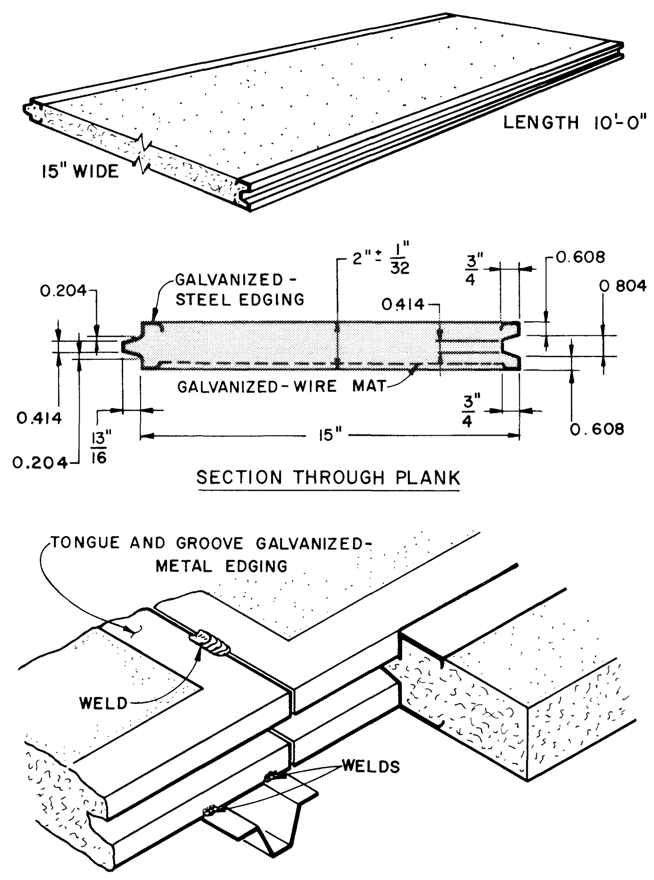


Fig. 3. Gypsum-plank details

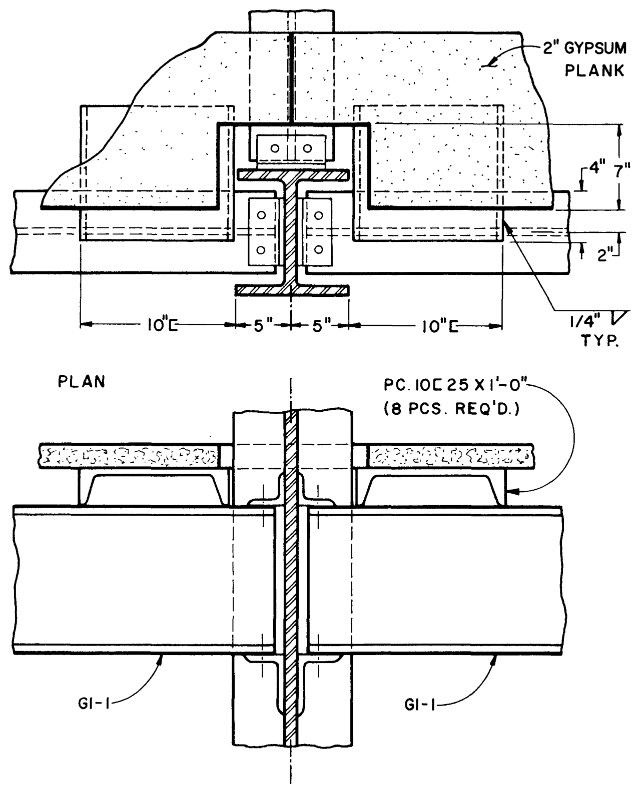


Fig. 4. Typical arrangement for plank support at columns

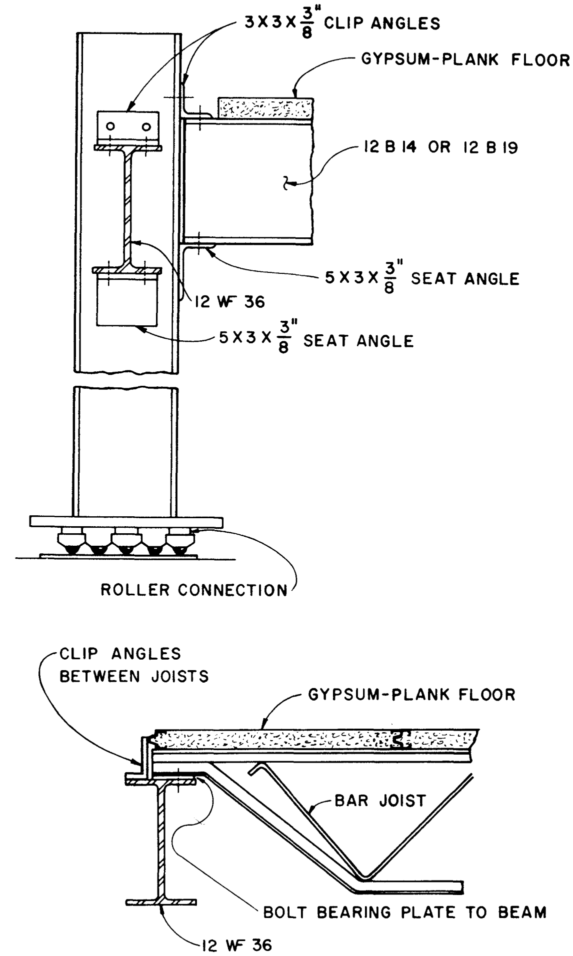


Fig. 5. Connections—beam-to-column and bar joists-to-beam

A device consisting of a group of rollers was attached to each column base plate to minimize restraint against horizontal displacements, as shown in Fig. 5. In-plane movements at right angles to the edge beams were measured by 0.001-in dial gages A through J mounted on independent stands, located as shown in Fig. 2.

Electrical-resistance strain gages were used only in the test on the base frame without the gypsum-planks. The purpose of the strain gages was to measure strains at critical points in the midspan beam-to-column connections and to check the stresses induced in the frame test.

DESCRIPTION OF TEST AND THE RESULT

According to the National Building Code, the wind pressure for the surface zone of building exterior walls 100 to 500 ft above the average level of the ground is 30 psf. Accordingly, if an average wind force of 30 psf is assumed to be on the exterior wall surface of a building, the story height of which is 9 ft-6 in., the wind forces are 285 lbs per lin ft along the exterior edge of the floor. Since the test specimen represented slightly less than one-half of the floor area between the braced frames, the maximum shear forces at the extreme ends of the 48-ft span would be less than 3420 lbs. These shear forces could be generated by applying a concentrated load of 6840 lbs at the midspan. For simplicity this was rounded off to 7000 lbs and was considered as the "working load."

Although the shear deflection of the structure under the concentrated load at the midspan would be different from that under the uniformly distributed wind load, the load application does test the capacity of the floor to sustain the shearing forces. One advantage of this arrangement is that since the shearing forces maintain constant magnitude along the span length, it is simpler to correlate the shearing forces and the resulting deflections.

The dial-gage readings were adjusted for the support movements and the bending effect to obtain the shear deflection, Δ_s, respectively, by

$$\Delta_0 = \tfrac{1}{2}[E+J-\tfrac{1}{2}(A+C+G+I)] \tag{1}$$

and

$$\Delta_s = \Delta_0 - \Delta_b \tag{2}$$

In the above expressions, the capital letters designate movements of the dial gages, Δ_0 denotes the average value of the deflection at the top and bottom points of the midspan under loading, and Δ_b is the bending deflection computed by

$$\Delta_b = PL^2/48EI_{eff} \qquad (3)$$

where L is the span length of the floor and E is Young's modulus. The gypsum planks can be transformed into an equivalent thin steel sheet; therefore, the contribution of planks and the intermediate support may be conservatively neglected in the I_{eff} computation, and the external moment is considered to be resisted by the axial forces in the marginal members. Hence, the effective moment of inertia, I_{eff}, is

$$I_{eff} = AW^2/2 \qquad (4)$$

where A is the cross-sectional area of the edge beams, and W is the width of the floor. Δ_s being evaluated, the shear stiffness, K_s, can be computed as

$$K_s = \frac{(P/2)/W}{\Delta_s/(L/2)} = \frac{PL}{4W\Delta_s} = \frac{\alpha P}{4\Delta_s} \qquad (5)$$

in which $\alpha = L/W$ is the aspect ratio.

Frame Test—The preliminary test on the framework without the gypsum planks showed that the frame had little stiffness. For only an 800-lb load, the midspan deflection was more than ½-in. (Fig. 6). Very small magnitude of deflections resulted in subsequent tests of the complete floor with the gypsum planks in place; consequently, it was apparent that virtually all the resistance to horizontal movement was provided by the gypsum deck and that very little shear was contributed by the stiffness of the frame.

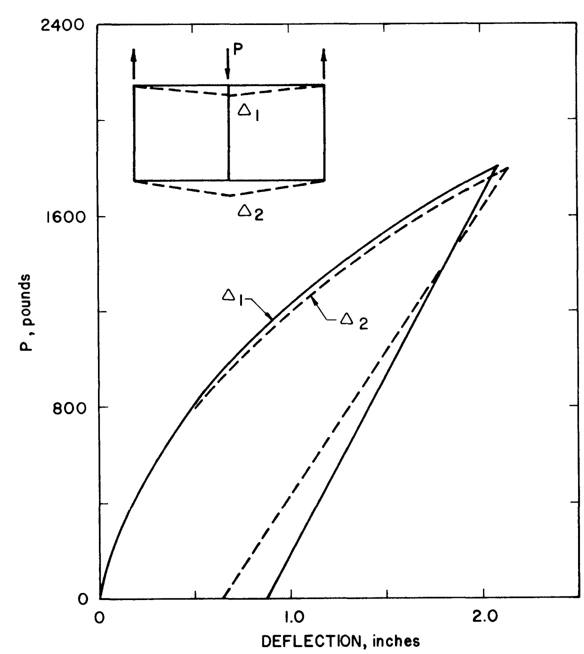


Fig. 6. Frame test load-deflection curve

Tests on the Completed Floor—Three nondestructive tests were conducted on the completed floor before the specimen was tested to failure. In these tests no cracks or any other sign of failure were detected. Results of these tests are given in Fig. 7.

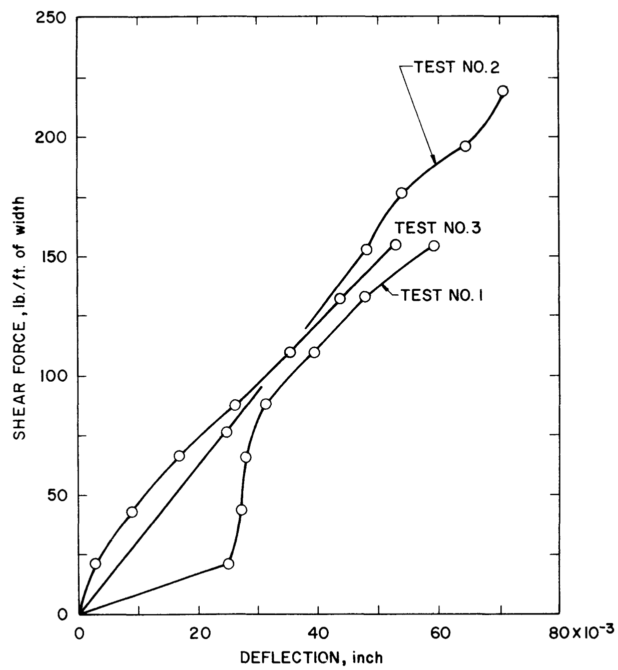


Fig. 7. Shear force per foot of web width vs. midspan shear deflection (non-destructive tests)

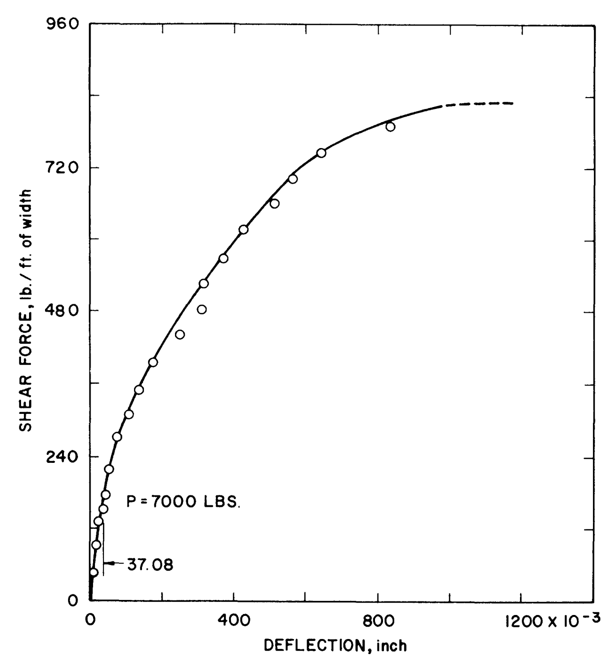


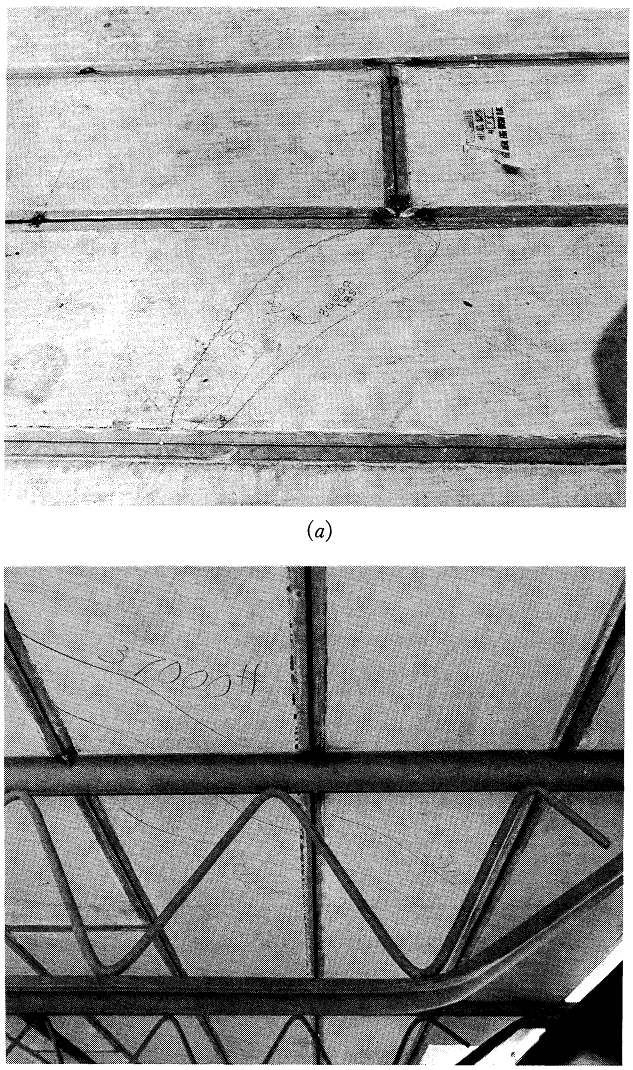
Fig. 8. Shear force per foot of web width vs. midspan shear deflection (test to failure)

In the first test, cyclic loads were applied in even increments of 1000 lbs up to 7000 lbs, and the floor was completely unloaded. The observed maximum deflection at 7000 lbs was approximately $\frac{1}{16}$-in., and the residual set upon the removal of the load was less than $\frac{1}{32}$-in.

In the subsequent test, the loads were increased up to 10,000 lbs in a similar loading process. The maximum deflection was 0.0708 in.

In the third test, an open area 2 ft by 6 ft was saw-cut at the upper right-hand corner of the floor where it was close to the support (Fig. 2). The result indicated that the cutout, which represented only a very small reduction in the total surface area of the floor, had no significant effect up to the "working load."

The results of the final test, which was conducted to failure, are shown in Fig. 8. The first sign of local failure appeared at 19,000 lbs (average shear force being 415 lbs per ft of the web width) in the floor section containing the cutout. Cleavage separation of the edge metal and plank body along the specimen center and extensive cracks at both sides of the center line were observed at and after the 30,000-lb load. The failure load was 38,000 lbs.



(a)

(b)

Fig. 9. Typical cracks in gypsum-plank floor

Table 1. Diaphragm Strength

Wind Loads		Max Average Wind Shear, lb/ft of width	Test Shear at First Cracks vs Max Wind Shear	Failure Shear vs Max Wind Shear
Pressure, psf	Loads on Floor, lb/ft			
20	190.0	86.0	4.8	9.7
25	237.5	107.5	3.9	7.7
30	285.0	129.1	3.2	6.4
35	332.5	150.6	2.8	5.5
40	380.0	172.1	2.4	4.8

Except for the crack inclinations along the center line of the floor, the crack inclination was generally consistent; it ranged from 30 to 60 degrees with respect to the transverse direction (Fig. 9). Looking toward the load, one could note that cracks at the left side of the center line inclined from upper right to lower left, whereas those at the other side inclined from upper left to lower right. The crack pattern was characteristic of brittle failure.

Ratios of shear at first cracking and failure to the maximum shear forces that would develop in the floor under various wind pressures on the model building are given in Table 1. In the table the maximum wind shear in the floor is determined by dividing the distributed wind loads on the floor by the floor width of the model building.

CONCLUSIONS

Although the test program was not sufficiently broad to yield a general design rule, it is believed that the results obtained from tests on the full-scale floor system do provide certain definite information on the behavior on the steel-edged gypsum-plank floor system.

An absolute criterion for shear stiffness is difficult to establish, for the system is complex and, hence, is inherently nonlinear in load-deflection characteristics. In the specimen tested, differential movements and slips between various joints and connections were believed to be practically nonexistent because all gypsum-plank seams and plank-support connections were welded. This can also be observed from the generally consistent feature of the shear-deflection curves. However, if clips were used to hold the gypsum planks to the intermediate supports, as often occurs for this type of construction, deflections and residual sets would be greater.

The test indicated that the floor was very stiff. The deflection at 3500-lb shear force (153.3 lbs per ft of width, which corresponds roughly to the maximum shear intensity to be expected under the wind pressure of 30 psf) was very small and could be considered to be not more than $\frac{1}{16}$-in. An exact prediction of deflection for design wind load is not possible and perhaps of not much value at present. However, it can be concluded that under the usual assumption of static wind load, the deflection should be negligible. Repeated loading did not produce residual sets that might be considered unfavorable.

The first sign of failure occurred at the average shear intensity of 416 lbs per ft of floor width, when the visual diagonal cracks appeared in the gypsum planks. The specimen failed at the load of 38,000 lbs (832.24 lbs per ft of shear width), when extensive cracking occurred in the gypsum planks, greatly reducing the stiffness of the system.

The 2-ft by 6-ft cutout at one extreme corner did not appear to affect the behavior of the floor structure, at least at the practical wind shear. However, the test was not conclusive on the ultimate effect of the cutout.

The test result also indicated that the system had sufficient strength to resist the wind shear that would be expected to develop under usual static wind loads in practice. All welds appeared to be adequate.