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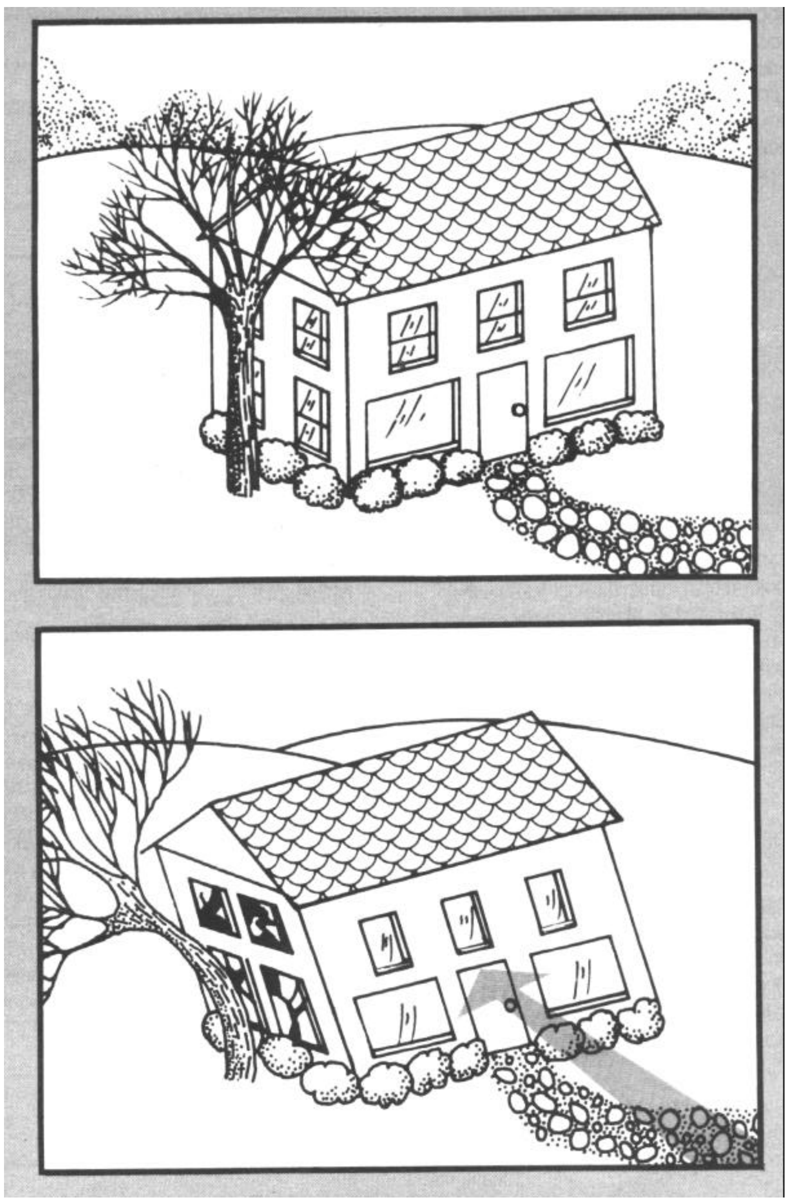
Contribution of Gypsum Wallboard to Racking Resistance of Light-Frame Walls

Ronald W. Wolfe



Abstract

Gypsum wallboard is the most commonly used interior wall sheathing material. Evidence suggests that it contributes to the shear performance of light-frame walls; however, it has received little recognition as a structural material. A better understanding of the structural behavior of gypsum wallboard could contribute to more efficient light-frame construction.

Thirty light-frame walls were evaluated to characterize the gypsum wallboard contribution to shear wall racking performance. Variables studied included windbracing, wall length, and wallboard orientation. Wallboard and windbracing were found to interact as parallel elements. The relationship between racking resistance and wall length was nonlinear for continuous wallboard diaphragms and varied with deformation level. Wallboard orientation had a significant effect on strength and stiffness. Results of this study provide a basis for engineers and code authorities to judge the contribution of gypsum wallboard to the shear resistance of walls under windloads and seismic loads. Results will also be useful in planning future research for light-frame construction.

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Ronald W. Wolfe, Research General Engineer

Introduction

Gypsum wallboard is the most common interior wall sheathing material used in residential construction. Due to the brittle nature of its core material and its low stiffness and strength relative to that of wood-base panel materials, however, gypsum wallboard is rarely recognized for any structural contribution to the integrity of light-frame buildings. This study was conducted to characterize the response of gypsum-sheathed walls to racking loads in order to provide a basis on which to judge its contribution to the wind- and seismic-load resistance of light-frame structures.

Light-frame walls perform three distinct structural functions: (a) transfer upper floor or roof loads to the foundation, (b) resist normal windloading and transfer this load to either the foundation, floor, roof diaphragm, or to a perpendicular wall, and (c) act as a shear diaphragm in transmitting lateral loads to the foundation. This study is concerned only with the wall's performance as a shear diaphragm. Shear or racking forces result from windloads or seismic loads. These loads induce shear stresses in the sheathing material, lateral loads on the fasteners connecting the sheathing to the framing members, and axial loads on diagonal braces used to improve shear wall stiffness and strength.

The most popular material for interior wall sheathing is gypsum wallboard. Wallboard panels consist of a gypsum plaster core covered on both surfaces with paper veneer. Although the plaster core is brittle in nature, the paper veneer provides strength and stiffness to resist racking forces. Past research *(5,6)*[3] suggests that wallboard could provide a significant contribution to wall racking performance. However, insufficient data exist regarding the effects of construction details on wallboard performance under shear loads. Structural analysis of light-frame wall systems has traditionally been conservative (see appendix A). Under racking loads, induced by horizontal wind and seismic forces, a wall is assumed to act alone rather than as part of a multi-member repetitive system. The Department of Housing and Urban Development (HUD) light-frame wall requirements *(25)* specify a braced section (plywood diaphragm, diagonal braces, etc.) to resist racking loads, and ignore structural contributions beyond this specified section.

The objective of this study was to determine the significance of gypsum wallboard contribution to wall racking resistance. Such information may lead to more precise analysis and design of shear walls. Thirty walls were evaluated at FPL to determine the influence of wallboard/frame interaction, panel orientation, and wall length.

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[1] Maintained at Madison, Wis., in cooperation with the University of Wisconsin.

[2] Research conducted in cooperation with the Department of Housing and Urban Development.

[3] Italicized numbers in parentheses refer to literature cited at the end of this report.

Materials and Methods

Wall constructions were selected to represent minimum allowable wood use. Tests were conducted following American Society for Testing and Materials (ASTM) Standard E 564-76 (1). This standard was developed to test shear resistance of framed walls, and specifies framing materials and anchorage connections simulating those used in actual construction.

Wall Configurations

Light-frame walls consist of four basic components: the frame, bracing, surface diaphragms, and fasteners. For this study, the basic frame designs described in the following section conformed to recommendations of the National Association of Home Builders (NAHB) OVE guidelines *(14).* Windbracing, when used, was applied at a 45° angle to the wall length; when used in conjunction with gypsum wallboard, it was applied to the opposite side of the frame. Gypsum wallboard was the only diaphragm material considered, and nails were the only fasteners used.

All walls were constructed using 2 by 4 studs spaced 24 inches on center (O.C.), end-nailed to single top and bottom plates using two 16d Common nails at each connection. Single end studs were also used. Twenty-two of the thirty walls tested had a 1/2-inch gypsum wallboard diaphragm attached to one side of the frame, and used 1-1/4-inch drywall nails spaced at 8-inch intervals along all framing members. For each of these walls, the common joint between adjacent panels was taped and spackled following procedures recommended by the United States Gypsum Company (USG) *(26).* The other eight walls had diagonal bracing, but no wallboard.

The wall sample consisted of 13 different wall configurations including the following test variable categories: windbracing, wall length, or panel orientation. Table 1 summarizes variable categories and wall configurations.

Control Walls

Configuration No. 1, table 1, is the control wall (walls 1-3). It is referenced as a basis for judging the effects of wall length and windbracing on racking performance. The control wall consists of an 8- by 8-foot wood frame and a wallboard diaphragm. Two 4- by 8-foot wallboard panels were applied parallel to the wall height dimension.

Windbracing

Four windbracing conditions were tested with the 8-foot-long walls. These included no diagonal bracing (walls 1-3) steel strap tension braces (walls 9-11 and 19-22) and let-in wood braces stressed in both compression (walls 4-5 and 12-15) and tension (walls 6-9 and 16-18). The steel strap brace was nailed to each stud with two 8d Common nails, bent around the top and bottom plates, and nailed to the wide surface using two 8d nails. The wood braces were cut into each stud and plate at the contact area, and nailed with two 8d Common nails at each intersection. Measurement of the effect of windbracing on long walls was confined to the use of the wood compression brace.

Table 1.—Wall test configurations

Configuration	Bracing	Length	Panel attachment	Number of tests
		Ft		
1	None	8	None	3
2	Wd Comp[1]	8	None	2
3	Wd Tens[2]	8	None	3
4	Mtl Strp[3]	8	None	3
5	Wd Comp	8	Vertical	4
6	Wd Tens	8	Vertical	3
7	Mtl Strp	8	Vertical	4
8	None	16	Vertical	2
9	Wd Comp	16	Vertical	2
10	Wd Comp	24	Vertical	1
11	None	24	Vertical	1
12	Wd Comp	24	Horiz	1
13	None	24	Horiz	1

[1] Wd Comp—1 by 4 wood brace cut into the studs and plates along the compression diagonal of the wall frame.
[2] Wd Tens—same as Wd Comp except placed along the tension diagonal.
[3] Mtl Strp—2-inch-wide metal strap placed along the tension diagonal and nailed to plates and studs.

Wall Length

Three lengths, 8, 16, and 24 feet, were selected as the minimum necessary to observe nonlinear relationship between length and racking performance. The use of diagonal wood compression braces with each wall length also enabled an evaluation of the interactive effects of bracing and wall length on the racking performance of walls.

Wallboard Orientation

In addition to windbracing and wall-length effects, 24-foot-long walls were also used to evaluate the effects of wallboard panel orientation on racking performance. Two of the 24-foot-long walls were tested with 12-foot-long panels applied parallel to the wall length (walls 29 and 30) which are referred to as horizontal application. Two others were tested using 8-foot long panels applied parallel to the wall height (walls 25 and 28) which are referred to as vertical application.

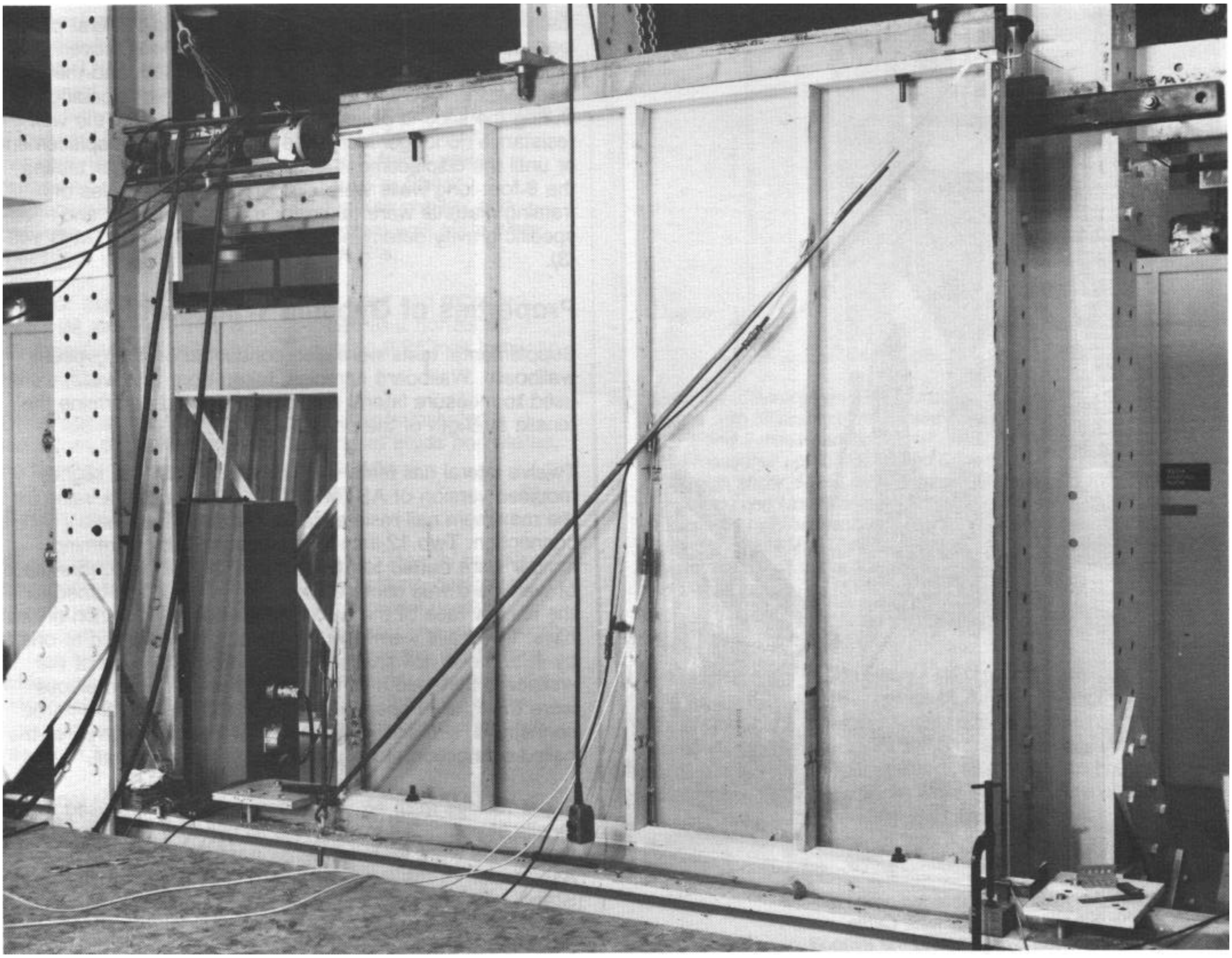

Figure 1.—Typical test setup showing an 8 by 8 wall with 2 by 4 studs spaced 24 inches O.C. with the cable holddown at the lower left and a hydraulic loading cylinder at the upper left corner. (M 147997)

Materials

Framing lumber, gypsum wallboard, and nails were obtained "off the shelf" from a Madison, Wis., lumberyard to simulate actual construction. The lumber was construction-grade spruce-pine-fir (SPF), 2 by 4 wall plates, stud-grade SPF precut studs, and No. 2 SPF 1 by 4 wood windbracing. The wallboard was 1/2 inch thick and labeled as conforming to ASTM Standards C 36, C 79, and C 588.

HUD-approved flat metal strap windbracing had to be purchased separately. The 2-inch-wide metal strap windbracing had nail holes at l-inch intervals along its length. The straps were longer than the diagonal of an 8- by 8-foot wall section so the strap ends could be bent around and nailed to the top and bottom plates.

Experimental Methods

All walls were tested in a vertical orientation following ASTM E 564 *(1)* with the bottom plate bolted to the base of the test frame as shown in figure 1. A kick plate was also fastened to the base at the end of the bottom plate to help restrain lateral movement of the wall. The 8-foot walls had a wood 4 by 4 and a steel channel bolted to the top plate to aid load distribution, and provide a hard surface for the roller guides used to maintain wall alignment. The steel channel and 4 by 4 were not used along the top plate for the longer walls. Additional roller guides were added to restrain lateral movement of the top plate in these cases.

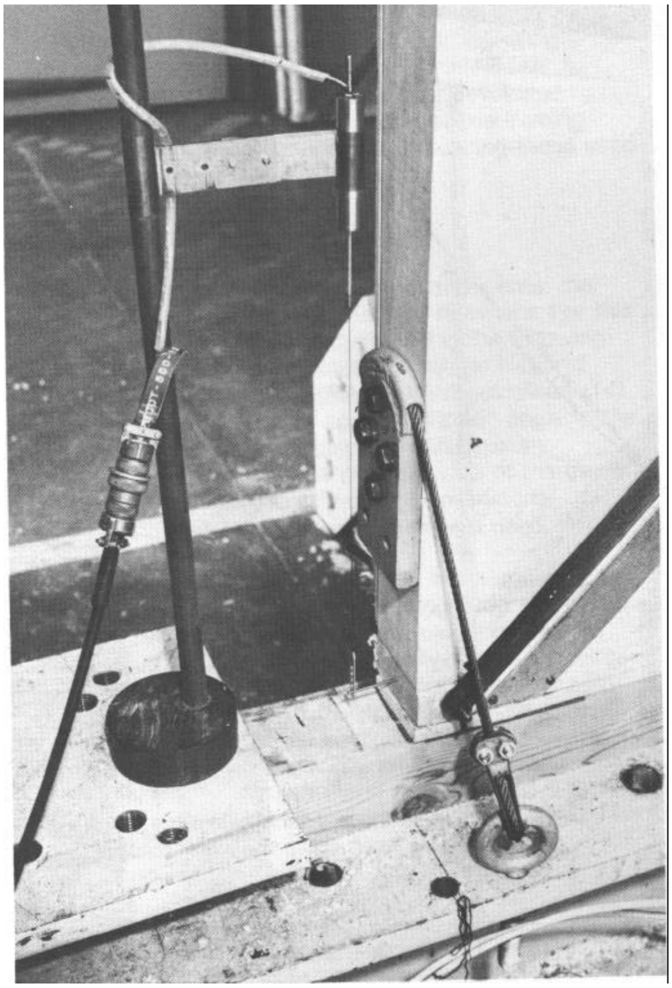

Figure 2.—Cable holddown fastened on base beam flange either side of test wall. Cable tension was adjusted by tightening the eyebolt connection through base beam flange. The bottom p/ate was also bolted to the base. (M150021-3)

Load was applied to an upper corner of each wall in a direction parallel to the wall length. A load rate set to give a constant displacement rate of 0.1 in./min was used for all tests.

The loaded end of the wall was held down by a 1/4-inch, 6 by 37 carbon steel fiber-core cable *(28)* as shown in figure 2. The effective modulus of elasticity of this cable was given as 11 X 10^6 Ib/in.^{2} and its elastic limit as 3,300 pounds. The cable was looped through a bracket mounted 1 foot from bottom of the end stud and fastened to the test frame base on either side of the test wall. Cable connections to the base of the test frame were positioned so the cable was angled to pass through a point close to the end stud axis of rotation. Rotational resistance was minimized by avoiding a moment arm between the cable and the reaction force at the end stud point of rotation.

Each wall was tested in two phases. During the first phase, load was applied until the top plate of the wall moved 0.25 inch horizontally. The load was then released, and the wall was given a 5-minute recovery period before reloading. During the second phase, load was applied until the wall resistance no longer increased with increasing displacement or until the displacement exceeded 2 inches. After testing the B-foot-long walls were dismantled, and samples of framing material were taken for moisture content and specific gravity determination as described in ASTM D 143 *(3).*

Properties of Gypsum Wallboard

Supplemental tests were also conducted on the gypsum wallboard. Wallboard samples, taken from test walls, were used to measure lateral nail resistance and determine the tensile strength of the paper facings.

Twelve lateral nail tests were conducted using a slightly modified version of ASTM D 1761 *(4)* to obtain a value for the maximum nail resistance for gypsum to frame connection. Two 12-inch-long pieces of 2 by 4 framing lumber were butted together, and a 3-1/2-inch square piece of wallboard was centered over the joint and fastened to the narrow face of the wood pieces using l-l/Cinch drywall nails. Two nails were used to fasten the wallboard to one 2 by 4, and one nail spaced 3/4 inch from the edge of the wallboard fastened it to the other piece. The two pieces were then pulled apart placing a lateral load on the nailed connection, similar to the connector loading incurred at the nailed connection along the bottom plate of a wall.

Paper facings from six wallboard samples were used to test tensile strength. After removing all gypsum core material, the facings were tested in tension according to Technical Association of the Pulp and Paper Industry (TAPPI) Standard T-404 *(22).* Tests were conducted on both front and back facings, and in directions parallel (machine direction) and perpendicular (cross direction) to the length dimension of the wallboard.

Data Collection

All data were collected using electronic monitoring devices. These included a 10,000-pound load cell with an accuracy of ± 1 percent of full range, and linear variable differential transducers (LVDT’s) with an accuracy of ± 0.5 per cent of their full range. Recording devices included a 2-channel x-y recorder and a 56-channel scanner which digitized the output signals and recorded them on magnetic tape and a teletype printer. All digitized data were in units of millivolts and converted to engineering units by computer program.

For most cases, six LVDT's were used to measure the wall response to racking load. However, in several tests a seventh LVDT was added. Figure 3 shows LVDT locations. Horizontal movement of the top plate and diagonal elongation were measured using LVDT's with a full range of 6 inches. Uplift at the loaded end and horizontal slip between the bottom plate and the test frame were measured using 3-inch full-range LVDT's. Shear displacements between adjacent gypsum panels, and between the gypsum panels and bottom wall plate were measured using 2-inch full-range LVDT's.

Each of the deformations was measured and recorded every 24 seconds. This corresponded to a horizontal displacement of the load head equal to 0.04 inch between readings.

Other observations made during the tests included indicators of distress such as bowing of studs and plates, nailhead protrusion, and popping sounds.

Analysis Methods

The analysis of results was intended to identify relationships between configuration variables and racking performance of gypsum-sheathed walls. Hypothetical model predictions were compared to measured test results. The small number of test repetitions for each variable limits the confidence which may be placed in derived constants; however, relationships discussed provide a basis for judging the importance of configuration variables and planning future research.

Diagonal elongation measurements were included in these tests to provide a more direct measure of shear displacement. The horizontal displacement is affected to varying degrees by uplift, stud bending, and movement of the bottom plate. The diagonal elongation, however, is not affected by boundary conditions. For purposes of data analysis, all horizontal shear displacements reported are based on the diagonal elongation.

To test the hypothesis that individual elements of a wall act as parallel springs in the composite system, contributing elements were tested separately and compared to composite wall performance. This hypothesis was tested for both the interaction of windbracing and diaphragm, and the effect of wall length. For the case of windbracing, individual stiffness values were added and compared to composite stiffness at incremental deformations. For wall length, plots of wall performance versus length were checked for linearity.

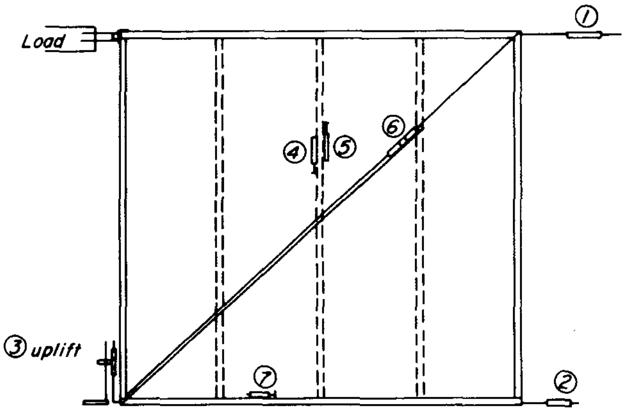

Figure 3.—Displacements were monitored using LVDT's with different displacement ranges. LVDT Nos. 1 and 2 measured horizontal displacement, No. 3 measured uplift, Nos. 4 and 5 measured diaphragm shear, No. 6 measured diagonal elongation, and No. 7 measured horizontal displacement of the diaphragm with respect to the bottom plate. LVDT Nos. 7 and 6 had 6-inch ranges and No. 2 had a 3-inch range. All others had 2-inch ranges. (M 151726)

The Tuomi and McCutcheon *(24)* strength model was derived on the basis of results of ASTM E 72 *(2)* tests of walls containing two wood-base panels which rotate independently with respect to the frame under racking loads. To test the applicability of this model to predict the ultimate strength of gypsum walls, values were predicted for 8-, 16-, and 24-foot lengths and compared to measured values. Wallboard panel orientation effects were also evaluated.

Results

Results of these tests pointed out several performance features of gypsum walls that are independent of wall configuration and variations in construction details. These include performance of taped joints, cyclic load-displacement characteristics, and failure mechanism.

The ability of taped joints to transfer load enables individual wallboard panels to act together as a continuous diaphragm. Of the 30 tests conducted, 22 contained gypsum wallboard diaphragms with taped and spackled joints. None of these wall tests indicated any sign of weakness along the taped joints.

All walls displayed increased stiffness for the second load application. The second load-displacement plot was almost linear from the point of residual displacement at zero load to a curve that formed a natural extension to the original load-displacement curve (fig. 4).

Finally, the failure mechanism common to all wall tests with continuous wallboard diaphragms was that of nails bending and tearing through the paper surface. This failure mechanism usually occurred along a cut edge where the gypsum core cracked and fell away due to the lack of a confining paper edge.

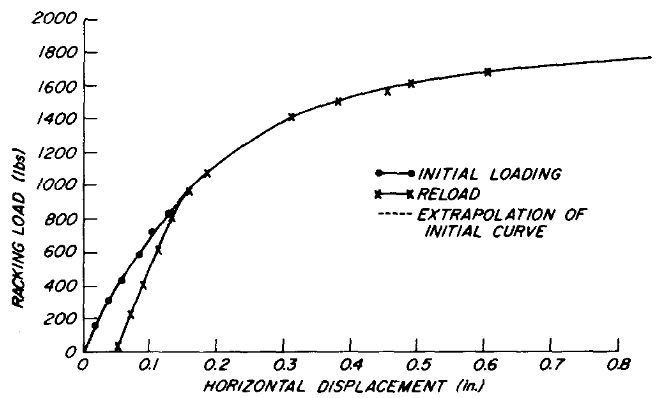

Figure 4.—Typical load-displacement curve showing increased stiffness for reload to the initial load-displacement curve. (M151736)

Unbraced Gypsum Walls

Results of three tests on unbraced gypsum walls are given in table 2. Walls 2 and 3 demonstrated failure modes significantly different from those expected on the basis of results of previous studies involving plywood and reconstituted wood composite panels. These walls exhibited complete nail failure~~when nails were~~ distributed along either the top or bottom plates rather than concentrated at the corners, and decreased in severity toward midheight and midwidth of each panel. Wall 1 failed initially in the tension corners (lower corner of the loaded edge and upper corner of the free edge).

Braced Frames and Walls

Results of 8 tests conducted on braced frames without gypsum wallboard sheathing, and 11 tests on braced frames with gypsum wallboard sheathing are given in table 2. Within each of these groups, results for three bracing types are given: wood let-in compression, wood let-in tension, and a metal strap. The performance of each wall type is discussed in the following section.

Braced Frames without Gypsum

Average racking load displacement curves for three types of braced frames without gypsum are given in figure 5.

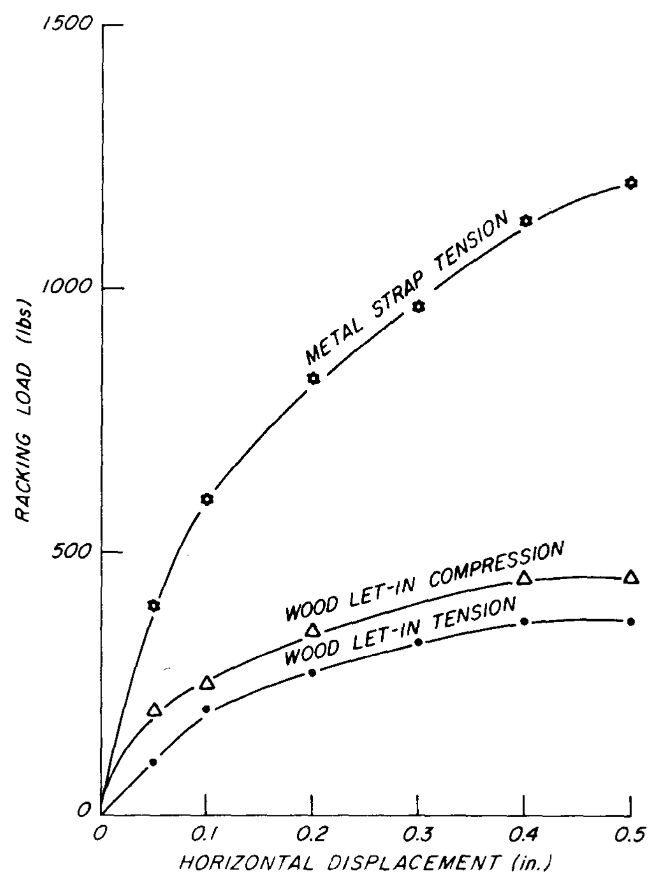

Figure 5.—Average load-displacement curves for braced *frames* without *gypsum. Three types of diagonal windbraces were tested; wood /et-in tension, compression, and* metal *strap tension. (M151737)*

Table 2.—Racking performance of 8-foot-long walls tested to determine the effects of bracing, gypsum wallboard, and taped joints

Identification	Brace	Racking resistance at displacements (in.)						Maximum load	Average specific gravity		Average moisture content	
		0.05	0.10	0.20	0.30	0.40	0.50		Plates	Studs	Plates	Studs
		Lb									Pct	
		UNBRACED GYPSUM WALLS										
1	None	400	600	800	1,000	1,100	1,100	1,100	0.36	0.36	9.8	11.8
2	None	400	600	800	900	1,000	1,000	1,100	.41	.40	9.2	9.6
3	None	400	600	900	1,000	1,100	1,200	1,300	.35	.38	9.8	10.0
		BRACED FRAMES WITHOUT GYPSUM										
4	Wood compression	200	300	400	400	500	500	600	.39	.48	9.3	9.4
5	Wood compression	200	200	300	300	400	400	600	.45	.46	9.5	9.6
6	Wood tension	100	200	300	400	400	400	600	.36	.45	8.8	9.4
7		100	200	200	300	300	300	500	.34	.42	9.1	9.5
8	Wood tension	100	200	300	300	400	400	600	.42	.41	9.3	9.6
9		400	600	900	1,000	1,200	1,300	1,600	.39	.37	9.3	9.1
10	Metal strap	400	500	800	900	1,100	1,100	1,300	.39	.43	9.2	9.3
11	Metal strap	400	600	800	1,000	1,100	1,200	1,500	.36	.42	9.3	9.4
		BRACED FRAMES WITH GYPSUM										
12	Wood compression	500	800	1.100	1.300	1.500	1.600	1,900	.37	.37	9.7	9.8
13	Wood compression	700	1,000	1,300	1,500	1,700	1,800	1,900	.38	.37	9.6	9.7
14	Wood compression	500	800	1,200	1,400	1,600	1,700	1,800	.34	.38	9.1	9.2
15	Wood compression	600	800	1,100	1.300	1.500	1.600	1,700	.38	.38	9.8	9.7
16	Wood tension	400	1,100	1,200	1,300	1,400	1,400	1,500	.37	.41	12.1	11.5
17	Wood tension	500	700	1,100	1,300	1,500	1,500	1,600	.35	.43	9.4	9.8
18	Wood tension	600	800	1,200	1,400	1,500	1,600	1,800	.36	.37	9.4	9.8
19	Metal strap	600	1,200	1,700	2,000	2,100	2,300	2,400	.36	.36	9.9	9.9
20	Metal strap	700	1,100	1,500	1,800	1,900	2,100	2,300	.38	.39	9.5	9.9
21[1]	Metal strap	500	700	1,500	1,900	2,200	2,300	2,500	.43	.38	10.2	10.4
22	Metal strap	500	1,000	1,700	2,100	2,300	2,500	2,500	.38	.41	9.5	9.6

[1] Wallboard damaged prior to test.

The wood compression brace, end stud, and bottom plate formed a rigid triangle that rotated about the brace connection to the bottom plate for walls 4 and 5. As the load increased, the top plate separated from the first stud at the loaded end, and the second stud pulled away from the bottom plate. The loaded end stud deformed as a cantilever, suggesting a rigid connection at the bottom plate. Buckling distortion of the top plate in the wall plane was also apparent. Due to the uplift at the loaded end, the final failure could not be strictly classified as a shear-type failure. Stud connections to the top plate remained square and the top plate did not move parallel to the bottom plate.

Steel strap and wood tension braces displayed similar failure modes, but the metal strap was significantly stronger. Failure occurred due to nail slip at both ends of these braces. The metal strap brace, wrapped around the plates, caused distortion of the bottom plate, whereas lateral load on the wood tension brace/bottom-plate connection resulted in nail withdrawal from the plate.

Gypsum with Wood Compression Brace

The average racking performance of walls 12-15 is given in figure 6. Wall 14 was damaged during setup when a 6- by 9-inch hole was accidentally punched through the wallboard about 4 inches from the first stud on the loaded end. The panel damage did not appear to have any effect on racking performance, therefore it was included in the evaluation of average performance.

During loading, the wallboard diaphragm appeared to move horizontally as the frame racked, causing nailheads to tilt. Failure for walls 12, 14, and 15 resulted from nails pulling through the gypsum along the bottom plate and the lower half of the loaded end stud. In these cases, little visible damage occurred to the wallboard along the top plate. For wall 13, nails pulled through along the top plate and the upper 16 inches of the last stud.

The performance of the diagonal brace was the same for all wood compression-braced walls. At approximately two-thirds of ultimate load (~ 1,200 lb), the brace began to slip noticeably at the top and bottom plate connections. In one instance the diagonal brace butted against the 4 by 4 on top of the wall, but there was no sign of buckling or bending of the brace throughout the test. Racking resistance continued to increase up to about 0.7-inch displacement and then dropped off rapidly.

Gypsum with Wood Tension Brace

The average load-displacement curve for walls 16-18 (fig. 6) displayed behavior similar to walls 12-15 up to a load of 1,200 pounds. Beyond this load level, these tension-braced walls rapidly approached their maximum load. At horizontal displacements beyond 0.5 inch, nailed connections of the brace at the top and bottom plates began to slip noticeably. This caused the load-displacement curve to flatten out. As loading continued, nailheads began to tilt along the plates and end studs. Bottom-plate connections then suddenly gave way, and the wallboard separated from the bottom plate. The nailheads had pulled through the wallboard along the bottom plate and 16 inches up the two end studs.

Gypsum with Metal Strap Brace

Walls 19-22 were constructed using a metal strap tension brace with a 1/2-inch gypsum wallboard diaphragm. Wall 21 had two holes accidentally punched through the wallboard during setup. One hole, at midheight between the first two studs from the loaded end, was the equivalent of a g-inch-diameter circle. The other hole, located below midheight between the last two studs, was about 6 inches in diameter. Test results of the damaged wall showed it was less stiff on initial loading to 0.25-inch displacement. However, after the initial load had been released and the panel reloaded, its performance was comparable to the other three walls and this test was included in the analysis of average performance.

At maximum load, three of the walls continued to deform with little or no loss in strength for up to a 2-inch displacement when the test was stopped. The damaged wall, No. 21, reached a maximum load slightly greater than the average of the other three at 0.7-inch displacement after which the load dropped off rapidly.

Failure of these walls occurred first at the gypsum corners, and then at the nailed connections along the end studs and bottom plate. This was accompanied by nail slip at the lower plate/strap connection and distortion of the plate due to compressive loads imposed by the strap connection. Little nail slip was apparent along the upper plate.

Long Walls

Data collected from tests of 16- and 24-foot long walls are given in table 3. Racking loads at six incremental levels of horizontal displacement and maximum strength provided some basis for evaluating the effect of wall length on strength and stiffness, as well as the changing influence of the diagonal let-in wood compression windbrace with wall length (figs. 7 and 8).

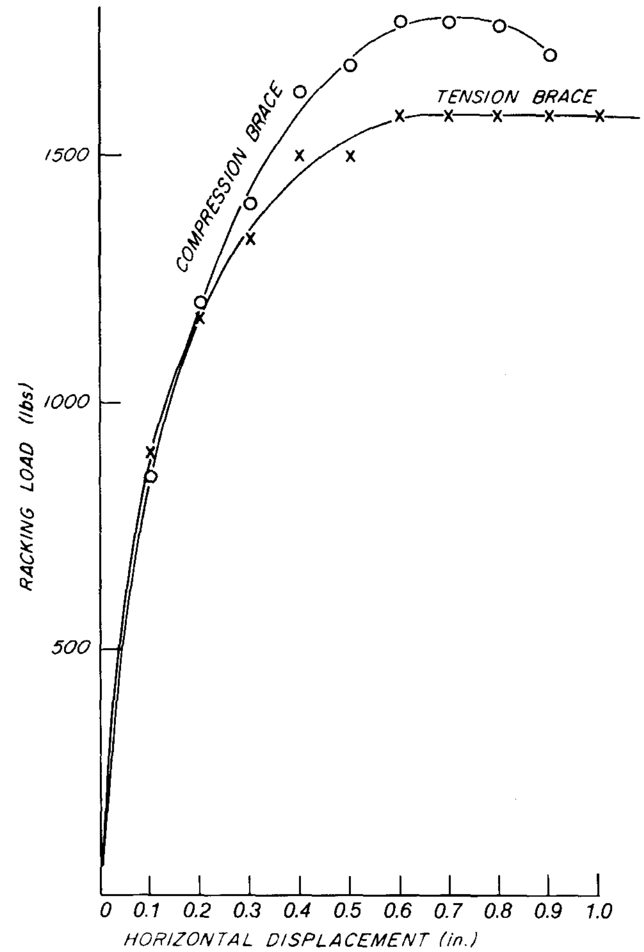

Figure 6.—Comparison of the racking performance of walls with compression and tension braces. The 8- by 8-foot walls contained a 1/2-inch gypsum wallboard diaphragm and wood diagonal windbracing. (M151738)

The most obvious effect of wall length on racking performance was a shift in the failure pattern. The longer walls showed less tendency for rotation of the wallboard diaphragm with respect to the frame, and failure appeared to be more confined to nail connections at the bottom plate. As wall length and stiffness increased, deflection at ultimate load and rotation of the end stud with respect to the diaphragm decreased. As the load approached the ultimate capacity for the 16- and 24-foot walls, nails bent at the lower corners and along the bottom plate, and failure occurred suddenly as if all bottom-plate nails gave way at the same time. Movement of the wallboard, measured with respect to the bottom plate, was fairly uniform with distance from the loaded end for the 16- and 24-foot walls. This

Table 3.—Racking performance of 16- and 24-foot-long walls tested to determine effects of wall length, and panel orientation

Identification	Length	Racking resistance at displacement levels (in.)						Maximum load
		0.05	0.10	0.20	0.30	0.40	0.50	
	Ft	Lb						
UNBRACED GYPSUM WALLS								
23	16	900	1,600	2,100	2,400	2,600	2,600	2,600
24	16	700	1,200	2,000	2,300	2,300	—	2,300
25	24	1,900	2,600	3,600	3,900	4,100	—	4,100
BRACED FRAMES WITH GYPSUM[1]								
26	16	800	1,500	2,300	2,500	2,700	2,900	3,000
27	16	1,100	1,800	2,400	2,600	2,700	2,800	2,800
28	24	2,500	2,900	3,700	4,000	—	—	4,200
HORIZONTAL PANELS ON UNBRACED FRAME								
29	24	3,100	3,900	4,900	5,500	5,900	—	6,000
HORIZONTAL PANELS ON BRACED FRAME[1]								
30	24	2,800	4,300	5,600	6,200	6,600	—	6,600

[1] Wood let-in compression brace.

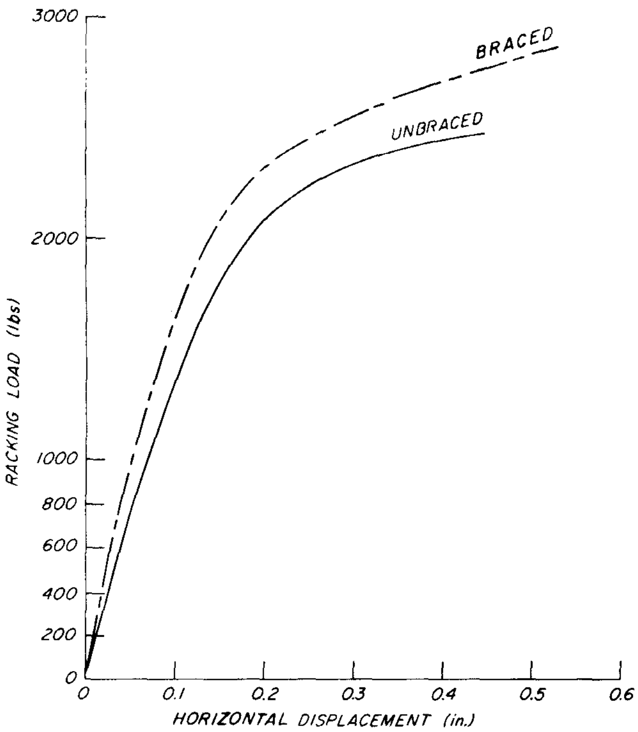

Figure 7.—Comparison of racking performance for braced and unbraced 16-foot walls with vertically applied 1/2-inch gypsum wallboard. (M151739)

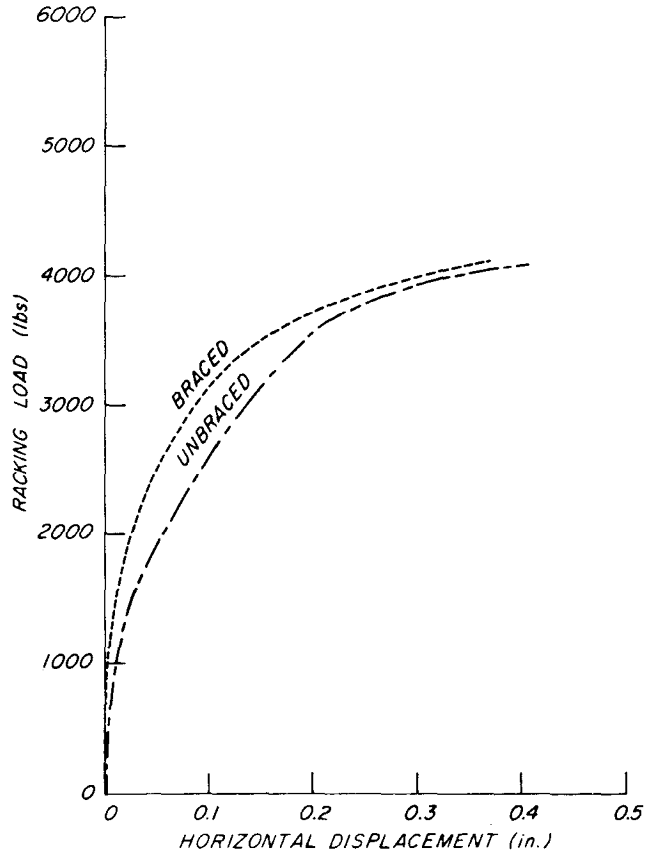

Figure 8.—Comparison of racking performance for braced versus unbraced 24-foot-long walls with vertically applied 1/2-inch gypsum wallboard. (M151740)

suggests that all bottom-plate nails were equally stressed. Inspection of the failures showed that connections along the top plate were still intact, while nails along the bottom plate and close to the bottom of the studs had pulled through the wallboard. Nail failure along the studs extended past midheight of the 16-foot walls while nail failure for the 24-foot walls extended about 2 feet above the bottom plate.

With the addition of the let-in windbrace the failure mode changed slightly, and the effect on racking resistance appeared to be a constant for all wall lengths. Both 16- and 24-foot walls reacted in a manner similar to that observed for 8-foot walls with a wood compression brace. A gap appeared between the top plate and the loaded end stud, while other studs intersecting the brace lifted off the bottom plate as racking deformation increased.

Analysis and Discussion

Panel Orientation

Results of tests for 24-foot walls, with and without windbracing (fig. 9), showed a 50 percent increase in ultimate strength for horizontal versus vertical panel orientation. For the vertically oriented panels, nail loads along the bottom edge caused the gypsum core to crack and fall away from the panel. Thus, as the support of the nailheads diminished, the wallboard-plate connection began to slip. The manufactured paper edge on the horizontally oriented wallboard confined the gypsum core, and therefore the compressive support for the nailhead was maintained to higher nail loads. In both cases, failure occurred when the nailhead began to tilt and cut into the paper surface.

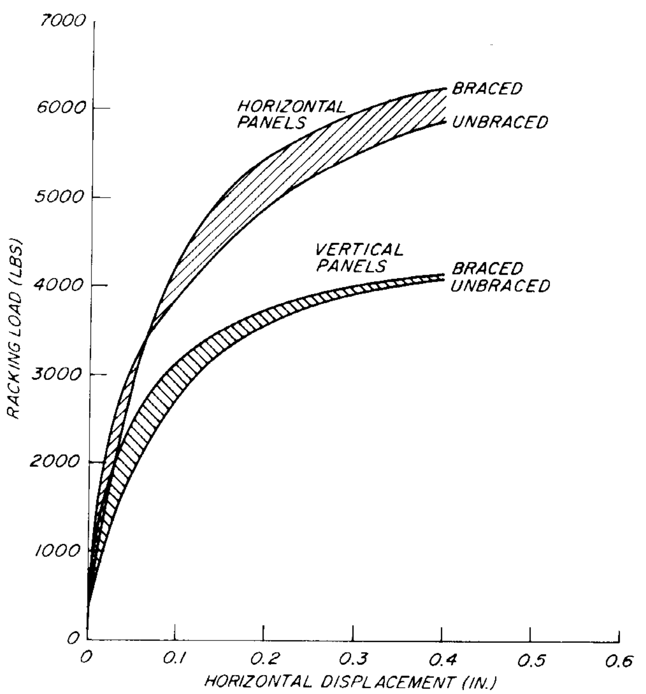

Figure 9.—Effects of wallboard orientation and diagonal windbracing on racking performance of 8- by 24-foot walls. (M151741)

In order to apply the results of tests conducted in this study, mathematical models were developed which characterize effects of the various test variables on racking strength and stiffness. Although the accuracy of the constants derived for the selected models was limited by the number of tests and the resolution of measuring devices, the relationships they represent do provide a basis for judging the importance of wall configuration.

Racking Displacement

For racking displacement, the elongation of a 45° diagonal (ΔD) provided a more reliable measure than did the horizontal movement of the top wall plate. Thus, racking performance models were derived using the geometric relationship given by equation (1) to estimate racking displacement (ΔH).

$$\Delta H = 1.414 \Delta D \qquad (1)$$

A comparison of horizontal displacement, determined by this equation and the measured horizontal shear displacement, resulted in a discrepancy which increased in proportion to applied load. This discrepancy was attributed to partial rotation resulting from uplift and slight bending deformation observed in the loaded end stud due to end restraint imposed by the cable holddown.

Table 4 presents average results for all tests reported in tables 2 and 3, rounded to the nearest 10 pounds. In most instances, only these average values were used in the effects analysis of wall configuration.

Diagonal Windbracing

A parallel spring model was selected to characterize the combined effects of bracing and sheathing. Development of this model is based on the assumption that the stiffness of the wall is equal to the sum of the stiffnesses of each contributing element. Neglecting component interaction and contributions due to the frame, the stiffness of the braced frame plus that of an unbraced frame containing a shear diaphragm should equal the stiffness of the composite wall. For this analysis, stiffness was the secant modulus or the slope of a line extending from the origin to the point on the load-displacement curve corresponding to a given displacement.

This parallel spring model was evaluated for each of the three brace types along with diaphragm length variations. The results are presented in tables 5 and 6.

For walls with compression or tension wood braces, the sum of individual stiffness contributions averaged within 2 percent of the measured value for the 8-foot wall tests. Walls tested with the metal strap brace had an initial stiffness 38 percent less than predicted. Measured and predicted values did converge, however, with increased displacement.

Table 4.—Average performance of wall tests used to analyze the effects of bracing, length, and panel orientation

Variation[1]	Walls	Length	Racking resistance at deformation levels (in.) 0.05	0.10	0.20	0.30	0.40	0.50	Maximum load	Unit strength
		Ft	Lb						Lb	Lb/ft
			CONTROL GYPSUM WALLS							
N	1,2,3	8	400	600	830	970	1,070	1,100	1,170	150
			BRACED FRAMES WITHOUT GYPSUM							
C	4,5	8	200	250	350	350	450	450	600	70
T	6,7,8	8	100	200	270	330	370	370	570	70
M	9,10,11	8	400	570	830	970	1,130	1,200	1,470	180
			BRACED FRAMES WITH GYPSUM							
C	12,13,14,15	8	580	850	1,180	1,380	1,580	1,680	1,830	230
T	16,17,18	8	500	870	1,170	1,330	1,470	1,500	1,630	200
M	19,20,21,22	8	580	1,000	1,600	1,950	2,130	2,300	2,430	300
			LENGTH AND BRACE							
N	23,24	16	800	1,400	2,050	2,350	2,450	—	2,450	150
N	25	24	1,900	2,600	3,600	3,900	4,100	—	4,100	170
C	26,27	16	950	1,650	2,350	2,550	2,700	2,850	2,900	180
C	28	24	2,500	2,900	3,700	4,000	—	—	4,200	180
			HORIZONTAL PANEL ORIENTATION							
N	29	24	3,100	3,900	4,900	5,500	5,900	—	6,000	250
C	30	24	2,800	4,300	5,600	6,200	6,600	—	6,600	280

[1]N = no brace; C = wood let-in compression; T = wood let-in tension; and M = metal strap.

Table 5.—Parallel spring stiffness model for 8- by 8-foot walls. Comparison of composite wall stiffness to the sum of stiffness contributions for individual wall components at incremental displacements

Wall identification	Stiffness P/A at incremental displacements (in.) 0.05	0.10	0.20	0.30	0.40	0.50
	Lb/in.					
UNBRACED GYPSUM WALLS						
A = unbraced gypsum wall	8,000	6,000	4,150	3,230	2,670	2,200
WOOD COMPRESSION BRACE						
C = braced frame	4,000	2,500	1,750	1,170	1,130	900
A + C	12,000	8,500	5,900	4,400	3,800	3,300
GC = gypsum on braced frame	11,600	8,500	5,900	4,600	3,950	3,360
(A + C)/GC	1.03	1.00	1.00	.96	.96	.98
WOOD TENSION BRACE						
T = braced frame	2,000	2,000	1,350	1,100	930	740
A + T	10,000	8,000	5,500	4,330	3,600	2,940
GT = gypsum on braced frame	10,000	8,700	5,850	4,430	3,675	3,000
(A + T)/GT	1.00	.92	.94	.98	.98	.98
METAL STRAP BRACE						
M = braced frame	8,000	5,700	4,150	3,230	2,830	2,400
A + M	16,000	11,700	8,300	6,470	5,490	4,600
GM = gypsum on braced frame	11,600	10,000	8,000	6,500	5,330	4,600
(A + M)/GM	1.38	1.17	1.04	.99	1.03	1.00

Table 6.—Parallel spring stiffness model for 8- by 16- and 8- by 24-foot walls with let-in diagonal wood compression windbrace. Comparison of composite wall stiffness to the sum of stiffness contributions of individual wall components at incremental displacements

Wall identification[1]	Stiffness P/A at incremental displacement (in.) 0.05	0.10	0.20	0.30	0.40
	Lb/in.				
VERTICAL PANELS ON 16-FOOT WALLS					
AA = unbraced gypsum	16,000	14,000	10,250	7,830	6,130
AA + C	20,000	16,500	12,000	9,000	7,260
GAC	19,000	16,500	11,750	8,500	6,750
(AA + C)/GAC	1.05	1.00	1.02	1.06	1.08
VERTICAL PANELS ON 24-FOOT WALLS					
AAA = unbraced gypsum	38,000	26,000	18,000	13,000	10,250
AAA + C	42,000	28,500	19,750	14,170	11,380
GAAC	50,000	29,000	18,500	13,330	10,500
(AAA + C)/GAAC	.84	.98	1.07	1.06	1.08
HORIZONTAL PANELS ON 24-FOOT WALLS					
BBB = unbraced gypsum	62,000	39,000	24,500	18,330	14,750
BBB + C	66,000	41,500	26,250	19,500	15,880
GBBC	56,000	43,000	28,000	20,670	16,500
(BBB + C)/GBBC	1.18	.96	.94	.94	.96

[1] A designates an 8-foot unbraced frame section with gypsum sheathing oriented vertically; C designates an 8-foot braced frame section using a wood let-in compression brace; B designates an 8-foot unbraced frame section with gypsum sheathing oriented horizontally.

These results suggested that the parallel spring model gives acceptable estimates of composite wall performance, if load-displacement curves are available for component contributions. Such a model would take the form

$$R_i = \Delta_i(K_{1,i} + K_{2,i} + \ldots K_{n,i}) \qquad (2)$$

in which

R_i = composite racking resistance (lb) at displacement Δ_i

$K_{n,i}$ = secant modulus from the racking load-displacement curve of component n at deformation A,

For the 8-foot-long walls, the average contribution of the diagonal brace to racking resistance of the composite wall varied from 26 percent for the wood tension brace to 45 percent for the metal strap brace.

As wall length increased, the contribution of the diagonal brace decreased, resulting in wall strength being controlled by the gypsum contribution. Results given in table 4 show a 660-pound increase in average ultimate strength of 8-foot walls resulting from the use of diagonal let-in wood compression braces. This influence decreased to 450 pounds for the 16-foot, and 100 pounds for the 24-foot walls. Thus it appears that the length of the continuous diaphragm affects its interaction with the frame.

Wall Length Effects

The second parameter investigated was wall length. The wall length analysis considered both braced and unbraced walls. The initial hypothesis was that racking resistance is linearly proportional to length; and the diagonal wind-brace provides a constant increase for all wall lengths at a given level of displacement.

To test this hypothesis, wall racking resistance was plotted as individual points, one for each wall length (8, 16, and 24 ft), connected by straight line segments. Figure 10 shows these plots for both braced and unbraced walls for shear displacements of 0.05 and 0.30 inch. These plots do not support either hypothesis. Racking resistance was not linearly proportional to wall length at all displacements and the brace effect varied with wall length and displacement level.

The plots of figure 10 suggest that racking resistance is a nonlinear function of length at 0.05-inch displacement. A least squares regression was performed on the logarithms of racking resistance (lb) versus length (ft) at five displacements to model the nonlinearity. This gave estimates of parameters A and B for the expression

$$\text{Resistance} = B^*(\text{Length})^A \qquad (3)$$

These values are given in table 7. The value of A is inversely related to displacement. A fairly linear relationship between length and racking resistance at 0.30-inch horizontal displacement (fig. 10) suggests that A approaches 1.O as displacement increases. The value of B increases with displacement. Thus, if A does attain the value of 1.0, B would have the value of the ultimate unit strength of the wall (lb/unit length) (table 4).

Results given in table 4 and shown in figure 10 suggest that building code recommendations for allowable strength per unit length of gypsum walls do not impose equivalent displacement limits for all walls. Due to the nonlinear relation between stiffness and length, an allowable load based on an estimate of ultimate strength permits greater displacements in shorter walls. For example, interpolating from table 4 values shows that for unbraced gypsum walls, the 100-lb/ft value recommended by the Uniform Building Code (UBC) *(11)* would permit an average of 0.19-inch displacement for an 8-foot wall, 0.13 inch for a 16-foot wall, and 0.09 inch for a 24-foot wall.

As for the interaction of wall length and bracing, results in table 4 indicate the brace contribution increases with displacement level for the 8-foot walls, is constant for 16-foot walls, and decreases with displacement level for the 24-foot walls. For the 24-foot walls, however, only one test was conducted under each condition, and for the 16-foot walls, two tests were conducted for each condition. On the basis of this limited information and the previous discussion of the additive nature of brace and diaphragm contributions to wall strength, insufficient information exists to conclude that wall length has an effect on the contribution of windbracing.

Wallboard Installation Details

Variations in wallboard installation have a significant effect on wallboard contribution to racking resistance. Three contributing factors include panel orientation, taping of the wallboard joints, and the panel-frame connection.

Panel Orientation

Wallboard panel orientation had a significant effect on wall racking performance. Figure 9 shows that strength and stiffness were greater for panels oriented horizontally.

Table 7.—Constants for use in equation (3) to express wall racking resistance as a function of wall length at various displacements

(Resistance = B*(Length)^A)

Displacement	A	B
In.		
0.05	1.46	16
.10	1.36	33
.20	1.35	50
.30	1.28	66
.40	1.22	83
MAX	1.19	93

Note: Results based on tests of 8-, 16-, and 24-foot unbraced walls.

These plots show an average increase in ultimate strength of 50 percent, and 43 percent average increase in stiffness. Although the small sample size would not support the use of these factors in design, their magnitude suggests that the horizontal orientation of 12-foot-long sheets is structurally superior to the vertical orientation of &foot-long sheets.

Two explanations for the improved performance observed for horizontal panel orientation are, (1) the directional properties of the paper facing, and (2) edge differences. Tensile strength tests of paper facing samples taken from the tested walls (table 8) showed the strength of the paper was about four times greater in the machine direction (parallel to panel length) than in the cross direction for both front and back paper facings.

Edge differences include core confinement and a thinner section along the long edge. A continuous paper edge confines the gypsum core and provides improved support for the nailhead. The thinner section, due to edge taper, results in smaller nail bending moments. These two factors combine to provide increased nail holding ability along the long edge of the gypsum board.

Panel-Frame Connection

The panel-to-frame connection influences both strength and stiffness of the wall. The importance of individual nail contribution and nailing pattern has been demonstrated by Tuomi and McCutcheon *(24).* They developed a model to predict ultimate strength using the principle of energy conservation. This model estimates the energy adsorbed by each nail on the basis of an assumed nail failure pattern, and a linear relation between lateral nail displacement and the energy adsorbed at maximum load.

For the nailing pattern used in this study, the energy model predicts racking strength (R) as a constant multiple (K) of individual nail strength (r) for each wall length.

$$R = K^*r \qquad (4)$$

The derived values of K for the 8-, 16-, and 24-foot-long unbraced walls are 14.49, 36.80, and 59.52, respectively.

A limited number of lateral nail tests conducted, using wallboard and framing lumber samples from the test walls, gave an estimate of nail strength (r) of 90 pounds. Dividing measured racking strengths (R) of unbraced 8-, 16-, and 24-foot-long walls by their respective K values gives estimates of effective lateral nail strengths of 80 pounds for the 8-foot-long walls and 70 pounds for the 16- and 24-foot walls. The slight discrepancy between measured and derived nail values may be due to the nail failure pattern. The derivation of the energy model assumes that as the wall frame distorts, the diaphragm maintains its rectangular shape and rotates slightly to accommodate a symmetrical distribution of nail forces along its perimeter. This rotation produces vertical nail force components which are proportional to the distance from the nail to the vertical centroidal axis of the panel. Racking tests of 8-by 8-foot walls with plywood diaphragms exhibited this behavior.

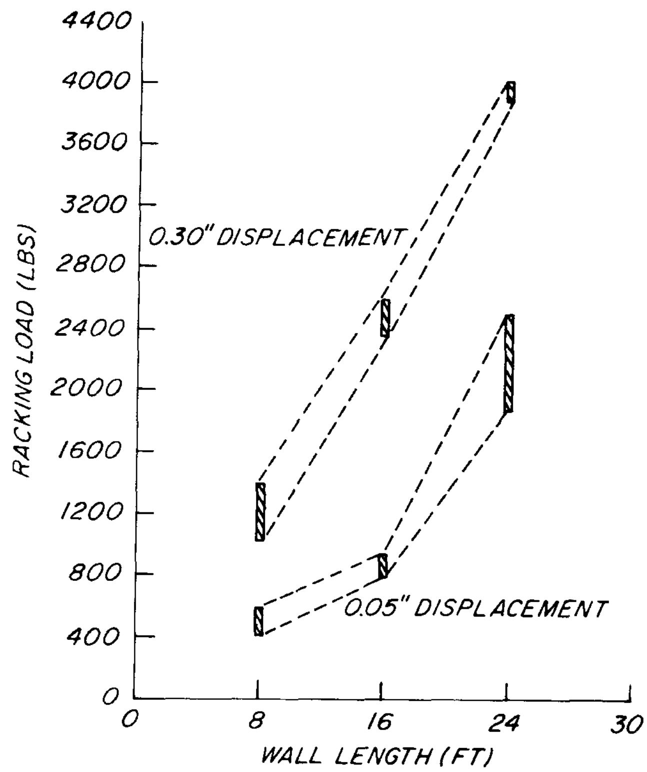

Figure 10.—Comparison racking performance at two displacement levels for three wall lengths. For both displacement levels, the top curve applies to a braced wall and the bottom curve applies to an unbraced wall. The racking strength versus length relationship appears to approach linearity at 0.3 inch displacement. (M151724)

Table 8.—Properties of wallboard paper facing

Sample	Face	Density	Tensile strength[1] Machine direction (MD)	Tensile strength[1] Cross direction (CD)	Ratio of MD/CD
		g/cc	– – – Lb/in. width	– – –	
1	Front	0.55	87.6	21.0	4.17
	Back	.67	70.2	18.3	3.84
2	Front	.56	89.7	20.6	4.35
	Back	.65	75.3	17.2	4.38
3	Front	.55	86.7	19.6	4.42
	Back	.66	75.3	19.0	3.96
4	Front	.55	78.5	20.7	3.79
	Back	.65	76.6	18.2	4.21
5	Front	.55	75.3	19.3	3.90
	Back	.66	75.0	19.9	3.77
6	Front	.53	87.9	20.6	4.27
	Back	.64	73.6	18.4	4.00

[1] According to TAPPI Standard T-404.

The nail failure pattern for the wallboard diaphragms, however, did not exhibit these vertical force components. The 8-foot walls did show some vertical distortion but the failure pattern was predominately horizontal and unsymmetric. Nailed connections along one plate remained intact as those along the other plate let go. Observations made of long wall failures indicated that the nails along the bottom plate were all bent parallel to the wall length with no failures occurring along the end studs. This suggests that the vertical force component was not significant for these walls.

Ignoring vertical displacement, Tuomi and McCutcheon's energy model (appendix B) simplifies to

$$R = r[n + m\Sigma y_1^2/h^2] \tag{5}$$

in which

R = ultimate racking resistance
r = ultimate lateral nail strength
n = number of nails along each horizontal plate
m = number of vertical studs
h = distance between top and bottom plates
y_i = the distance from nail i, along the vertical members, to the midheight of the wall

Figure 11 indicates a tendency for predictions based on this model, and a 90-pound nail value to converge to measured values as wall length increases.

As for wall stiffness, the Ramberg-Osgood model *(17)* (eq. (6)) for nonlinear load deformation curves gives a good fit to the measured curves

$$\varepsilon = \sigma/E + K(\sigma/E)^N \tag{6}$$

where

ε = the strain or displacement
σ = the stress or load
E = the modulus of elasticity or slope of the linear portion of the load displacement
K and N = constants

This model requires only three parameters in order to recreate that portion of the load displacement curve which is most critical for design. Appendix C provides a further discussion of this model, as well as a short program listing which may be used to estimate and test the model parameters.

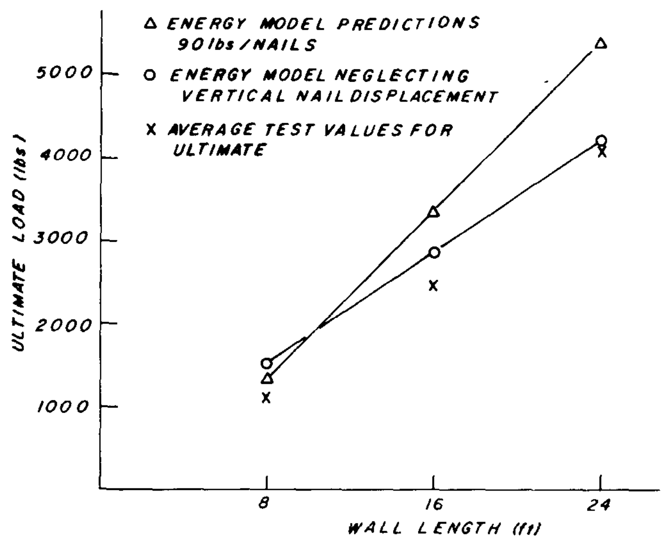

Figure 11.—An evaluation of the predictability of racking strength using two variations of an energy model. The energy model derived with no consideration for vertical nail deformation appears to converge to test results as wall length increases. Accounting for vertical nail displacement gives a better estimate of B-foot wall performance. (M151722)

Summary

Results of this study show that gypsum wallboard can provide a significant contribution to wall racking performance. This contribution does not appear to be affected by interactions with windbracing; however, it does vary with panel orientation and wall length.

The racking resistance of walls tested with a gypsum diaphragm and a diagonal windbrace appeared to be equal to the sum of contributions of these elements tested independently. This relationship held for all wall lengths tested.

Horizontal panel orientation appeared to offer a significant improvement over that obtained with vertical panel orientation. Walls tested with panels oriented horizontally were more than 40 percent stronger and stiffer than those with panels oriented vertically.

The relationship between ultimate shear strength and wall length was approximately linear. However, at low shear displacements, wall stiffness was a power function of length. This was attributed primarily to panel interaction resulting from the effects of joint taping. Thus, allowable strength contributions attributed to gypsum wallboard, based on tests of 8-foot-long walls, became conservative as wall length increased.

This study provides insight to the relationship between construction variables and performance of walls containing gypsum wallboard. However, more intensive study is required to develop models to quantify relationships found in this study and to relate individual wall contribution to whole-house performance.

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Appendix A

Literature Review

This section will review: (1) current design requirements for racking performance, (2) test procedures, (3) performance models, and (4) related research on the performance of walls covered with gypsum wallboard.

Design Requirements

Wall racking requirements of HUD are prescriptive in nature and require limited knowledge of loading conditions. HUD approves wall constructions on the basis of standard test results *(2,7)* used to compare the wall's performance to that of a base or calibration wall. The critical performance criterion for this test is a racking strength of 5,200 pounds. In actual construction, one 8-foot-long section on each exterior wall is required to carry this load. In earthquake-prone areas, an 8-foot-long braced section is required for each 25 feet of exterior wall and principal partition.

From the viewpoint of engineered design, the HUD approach has several shortcomings. First, the calibration wall was selected on the basis of a good performance record. The margin of safety is not quantified. Second, the approved wall constructions may be used anywhere, regardless of load conditions. Thus, in order for these wall constructions to be safe in areas where wind and seismic loads are critical, they will be overly conservative in areas where these loads are of little concern. Finally, the design ignores the effects of wall length. The requirement that anticipated loads be carried by one 8-foot section assumes no contribution from the rest of the wall.

Few guidelines are available for specifying wall racking stiffness. Some state building codes *(19)* limit lateral deformation in high-rise buildings to 0.50 percent of wall height and 0.25 percent of the head-to-sill height of glazed openings. A study conducted by Hirashima *(8)* indicated cracking of plaster-lath walls at shear displacements of 0.36 percent of wall height. Building codes *(11,18)* give allowable shear loads per unit length of common materials, but give no indication as to the corresponding displacements under those loads.

Test Procedures

The information and design tools available for the evaluation of wall racking performance are of limited value. The majority of available wall racking test data were generated using a standard test procedure published by ASTM (2). This test was established to evaluate the relative performance of sheathing materials. However, additional information is needed regarding effects of other construction variables as well as design limitations. Construction variables include framing, windbracing, door and window openings, wall length, and wall interaction with floor and ceiling diaphragms. Design limitations should include wall stiffness or deflection as well as ultimate strength.

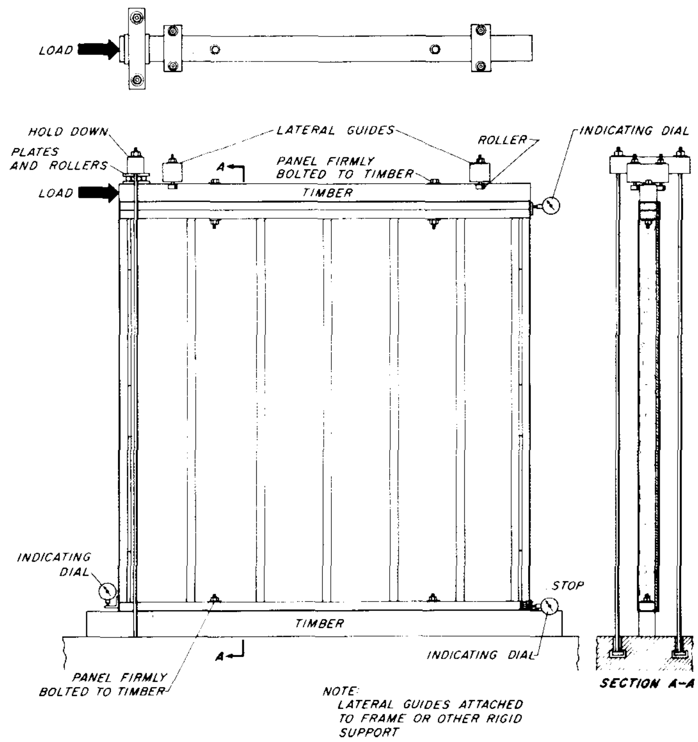

Figure A-1.—Standard ASTM E 72 test assembly. (M123922)

The test procedure used to evaluate these factors is an important consideration. Currently two ASTM standards describe test procedures for the racking resistance of light-frame walls; ASTM E 72-77 *(2)* and ASTM E 564-76 *(1).*

Standard E 564 is similar to E 72 except that it was intended for testing walls rather than evaluating panel performance. For this reason, it permits variation of wall frame configuration and boundary conditions to simulate construction practice. Standard E 72, however, specifies grade and species of framing lumber, as well as frame configuration and restraint conditions (fig. A-1).

Another test method that is often preferred for testing the shear capacity of a wall construction is the diagonal load test. Isenberg et al. *(12)* concluded that this test has the potential to give more uniform test results due to lack of need to resist panel rotation. However, he did note problems with panel buckling out of plane.

Japanese tests used walls with length-to-height ratios less than 1 and no rod holddown *(10,20,21).* These walls are subject to greater rotation and bending stresses; thus it is difficult to make direct comparison to results obtained using ASTM procedure.

Wall Performance Models

Models have been developed to predict ultimate racking strength as a function of the interaction between the diaphragm, frame, and connector. The model presented by Tuomi and McCutcheon *(24)* was derived as a summation of the energy absorbed by nail connections. Based on observation of 8- by 8-foot wall tests, nail distortion was modeled as a function of location with respect to the center of rotation of the wall panel, assuming a distortion pattern similar to that shown in figure A-2. This model has been applied with acceptable results for the prediction of ultimate shear load of 8- by 8-foot walls using a variety of diaphragm materials including gypsum wallboard *(6).*

The energy model may be used to explain the importance of nail placement on the racking strength. Figure A-3 shows how this model predicts ultimate wall strength to vary with nail pattern. These plots represent walls containing a continuous diaphragm fastened with a perimeter nail spacing (p), and interior or field nail spacing (f). Each curve is labeled (p/f) to represent the nailing pattern assumed. Comparison of ultimate strengths shows a strong correlation to perimeter nailing, and very little influence from the field nail spacing.

The energy model has some critical limitations for design applications. It was derived to estimate ultimate strength and assume a linear nail load/slip relationship. Under racking conditions wall stiffness, or ability to resist load without exceeding deformation limits, is usually of greater concern than ultimate strength. The nonlinear character of nail load/slip and racking load/displacement relationships results in questionable reliability of designs based on the assumption that load at an acceptable deformation is a constant fraction of the ultimate strength.

Design versatility requires wall racking performance models to consider the effects of wall configuration on load/deformation relationships. This includes the composite performance of several structural elements as well as the effects of wall length. The simplest assumption would be that of a parallel spring model. This implies that individual structural elements act independently; thus the racking resistance of a unit length wall would equal the sum of individual element contributions and full-wall performance would be linearly proportional to the number of unit lengths. Based on tests of several types of sheathing, lizuka *(10)* concluded that the parallel spring model does not apply to composite wall performance. However, a number of studies *(6,10,11,21,24)* suggest a linear relationship between strength and wall length. A method is needed to characterize the performance of composite wall construction.

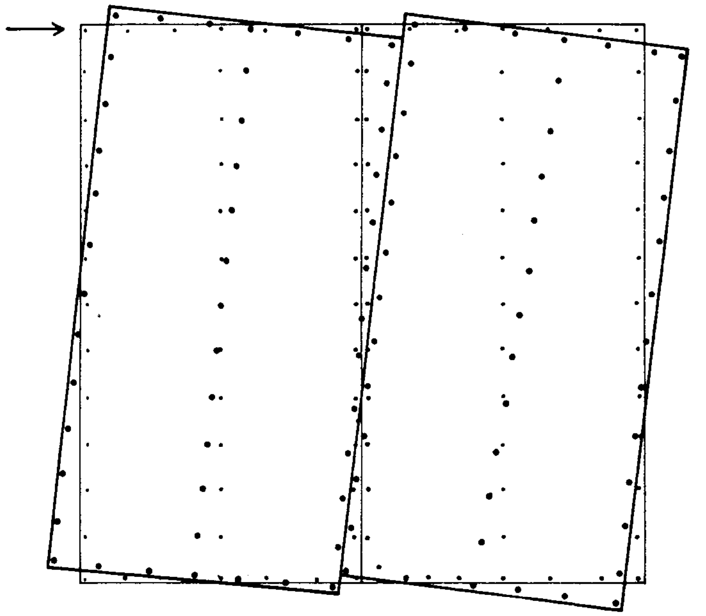

Figure A-2.—Failure mode for an 8- by 8-foot wall with untaped joint. Includes maximum frame-sheathing displacements at the four corners of each panel due to independent panel rotation. (M151721)

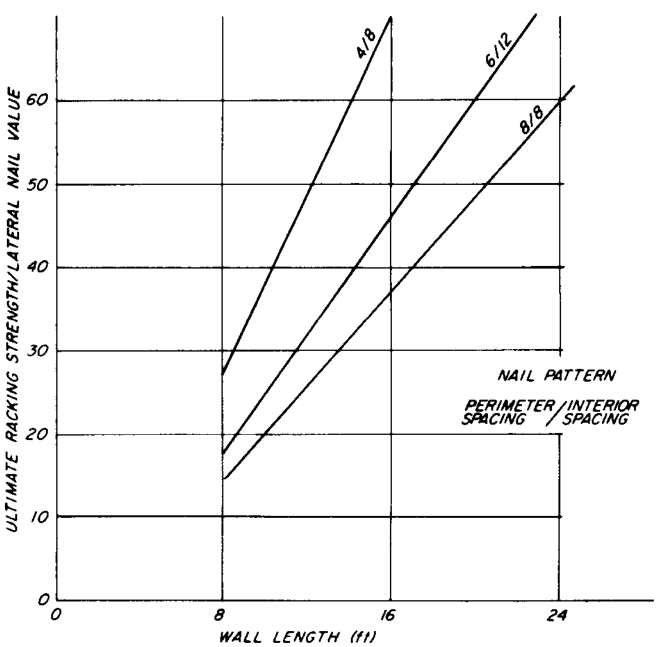

Figure A-3.—Dependence of racking strength on nailing pattern. Plots compare the strength versus length relationship for three nailing patterns predicted using the energy model for wall racking (23). (M151720)

Gypsum Wallboard Performance

Many different materials are currently used in wall construction. Among these, gypsum wallboard is the most common. Most data generated for walls containing gypsum diaphragms have been sponsored by the gypsum manufacturers. USG recommends a shear modulus of 1.05 X 10^5 lb/in.^{2} and a modulus of elasticity in bending of 2.45 X 10^5 lb/in.^{2} *(6).* Polensek *(16)* has also presented information on the mechanical properties of gypsum wallboard. Ultimate racking loads for gypsum shear wall tests sponsored by USG vary from less than 0.5 to over 1.25 times the HUD requirement of 5,200 pounds. These variations are apparently related to construction details as well as test conditions.

Recommended construction for gypsum walls without supplemental bracing takes advantage of the interaction between the wall sheathing and floor. USG *(26)* suggests installing sheathing with the bottom edge bearing on the subfloor and glue-nailing the panels to wall framing with nails spaced 12 inches O.C. The bottom edge bearing condition should be especially advantageous in tests of short wall sections ($\leq$ 8 ft) in which the diaphragm has a tendency to rotate with respect to the frame as the wall is racked. This may partially explain some of the variation in test values reported *(9,15,26).*

Studies sponsored by gypsum manufacturers and conducted by private testing laboratories have covered a range of 8- by 8-foot wall fastening details. These tests were conducted in accordance with ASTM Standard E 72 *(2).* Underwriters Laboratory tests of walls with 1/2-inch gypsum, glued both sides of 2 by 3 framing members, spaced 16 inches O.C., indicated a shear capacity of 880 Ib/ft (File MH 9733). Similar tests conducted by Pittsburgh Testing Laboratory using 2 by 4 framing showed average ultimate loads of 730 Ib/ft (75). Tests of 1/2-inch gypsum, nailed to one side of a 2 by 4 frame, conducted by IIT Research Institute (IITRI) gave an average of 660 Ib/ft *(9).* Assuming that nailing gypsum to both sides of the frame would double the ultimate load, the IITRI results suggest nailed shear wall capacities exceeding 1,300 Ib/ft. This exceeds test values obtained for walls with glued gypsum board. Comparison of such test results suggests a weakness in the E 72 test procedure, which makes the comparison of data collected from various laboratories confusing. Conclusions regarding the effects of variations in wall configuration should, therefore, not be drawn on the basis of results reported from different testing laboratories until a test procedure is developed which will give consistent results independent of the test location.

Most of the research on gypsum shear walls has been to determine its ability to meet code requirements for common construction. The International Conference of Building Officials (ICBO) *(11)* lists allowable shear loads of 75 Ib/ft for 1/2-inch gypsum fastened with 5d Common nails at 7-inch spacing, and 110 Ib/ft for 4-inch nail spacing. The Uniform, and Standard Building Codes (UBC *(11)* and SBC *(18))* cite values of 100 Ib/ft and 125 Ib/ft, respectively, for these same conditions. If the frame is blocked, UBC and SBC values are 125 Ib/ft and 150 Ib/ft, respectively. Some members of the building trades, such as mobile home manufacturers, base the design of shear walls on these values.

In comparing minimum results reported by testing labs to the highest value permitted by the building codes, it seems the code values have a factor of safety of at least 4.4 (660/150) for gypsum. However, this is not a valid conclusion. The E 72 test only provides a means of evaluating relative performance of various wall covering materials. Values derived from this test are not representative of the performance of walls used in actual building construction. This standard does not provide for testing effects of wall length or building component interactions. Tests are confined to one wall frame configuration. Species of framing and the method suggested for resisting uplift and rotation may not represent actual wall restraint.

The ASTM Standard E 72 specifies No. 1 Douglas-fir or southern pine 2 by 4 framing lumber, studs spaced 16 inches O.C. with double end studs, and a double top plate. The OVE design guide published by HUD *(13)* suggests that for many applications a 24-inch O.C. spacing is sufficient, and doubled end studs may be replaced by single members used in conjunction with corner clips. The use of No. 1 grade lumber for wall framing does not represent actual construction. This fact is important in that interactions between the wallboard and frame may play a major role in the response of gypsum-sheathed walls to racking loads. his interaction may vary with the quality of framing material.

Dishongh and Fowler *(6)* conducted a study of mobile home shear walls constructed with gypsum wallboard diaphragms, and containing door and window openings. Their results supported an ultimate shear strength of 325 Ib/ft and an allowable design value of 175 Ib/ft. Using ASTM E 564 test procedure *(1)* to simulate actual conditions, their walls were 13 feet 8 inches long by 7 feet 6 inches high and built using No. 2 southern pine studs. Two walls, tested simultaneously, were connected by a 4-foot-wide section of ceiling, side walls, and floor which served to prevent uplift. The wall diaphragms consisted of 5/16-inch gypsum wallboard stapled and glued to one side, and fastened only with staples on the other side. Eight tests were conducted. Three tests had continuous diaphragm walls, three had door openings, and two had window openings centered along the length. Allowable shear was estimated by dividing the ultimate load for each wall by its effective length, then estimating the lower 5 percent point of the distribution of ultimate shear load. In each case, effective length consisted only of those portions of the wall that were fully covered by gypsum from floor to ceiling.

Results of Dishongh's study also indicate that gluing and stapling the panels to one side of the frame gives a 40 percent increase in ultimate strength over stapling alone. Comparison of their observed ultimate loads with those predicted using the energy model showed a fairly consistent ratio of approximately 1.4. However, it is not clear from their report *(6)* how the energy model was applied for walls with door and window openings. If the assumption is correct that contributions of the two sides of the wall are additive, their results suggest that gluing and nailing both sides would give an 80 percent increase over nailing alone. A study by lizuka *(10)* suggests that the stiffness of walls sheathed on both sides is controlled by the stiffer side and is not additive, and that ultimate strength of the composite wall is less than the sum of two single-sheathed walls.

Another variable affecting wall racking performance, and one that has received little attention from research and building code authorities, is gypsum wallboard orientation. Wallboard is applied in either a vertical or horizontal orientation. Most racking test data available for wallboard were obtained using 8- by 8-foot wall tests with the panels applied vertically; however, most professional applications involve 12-foot-long panels applied horizontally. Tests conducted by Wiss, Janney, Elstner, and Associates for USG *(27)* compared three 8- by 8-foot wall tests of each orientation and showed no significant difference. Ultimate strength for the walls with panels oriented vertically averaged about 4 percent higher than for walls with panels oriented horizontally. However, this difference did not appear significant due to the variation within each group of three tests.

The Canadian building codes *(14)* do not require that special windbracing be used. They assume that resistance to windloads and seismic racking loads is inherent in the standard building practice. Nationally recognized building codes in the United States *(11)* require bracing on all exterior walls but allow for bracing effects of common building materials such as fiberboard, gypsum, and plaster, many of which would not meet HUD's 5,200-pound requirement on an ASTM E 72 wall racking test. HUD's recommendations for windbracing are much more restrictive than those of recognized building codes in that they do not recognize the additive effects due to wall length.

Appendix B

Energy Model Derivation Neglecting Vertical Component of Nail Displacement

The basis of the energy model derivation is conservation of energy; the energy put into a system must equal the energy dissipated or stored by the system. In the case of wall racking, assuming a linear load-deformation function, energy input is

$$E = ½R\Delta \qquad \text{(B-1)}$$

in which
R = applied load
A = relative horizontal displacement between top and bottom wall plates

Assuming that lateral nail distortion is the dominating factor in energy absorption, a similar expression may be used to represent energy dissipated.

$$I = \Sigma ½ r_i d_i \qquad \text{(B-2)}$$

in which
r_i = force on nail i
d_i = displacement of nail i

Assuming a linear load distortion function for lateral nail displacement

$$r_i = Kd_i \qquad \text{(B-3)}$$

$$½R\Delta = \Sigma ½Kd_i^2 = ½K\Sigma d_i^2 \qquad \text{(B-4)}$$

in which K = constant slip modulus

Neglecting vertical displacements, the horizontal displacements of nails along the plates (fig. B-l) would be

$$d_{p1} = Y_1\,\Delta/h$$

and

$$d_{p2} = (h - Y_1)\Delta/h$$

in which
Y, = greater of two distances from either plate to the center of rotation of the frame with respect to the diaphragm
$d_{p1} \geq d_{p2}$ = nail displacements parallel to top and bottom plates
h = wall height

Assuming a symmetrical nailing pattern and that all nails along a plate deform the same, with no vertical components, the energy absorbed by plate nails would be

$$I_P = ½K\cdot n\cdot d_{p1}^2\cdot \left[1 + \left[\frac{h - Y_1}{Y_1}\right]^2\right] \qquad \text{(B-5)}$$

in which
I_P = internal energy adsorbed by the plate-nail interactions
n = the number of nails along each plate
h = the distance between the top and bottom plates

Assuming vertical framing members remain straight, the distortion of these nails is proportional to the distance (y_i) from the center of rotation

$$d_{vi} = y_i \cdot \frac{d_{p1}}{Y_1}$$

and their contribution to energy absorption is

$$I_v = ½\cdot K\cdot m\cdot \left(\frac{d_{p1}}{Y_1}\right)^2 \cdot \Sigma y_i^2 \qquad \text{(B-6)}$$

in which
I_v = internal energy taken by vertical member nail interactions
m = the number of vertical members

Combining expressions (B-5) and (B-6) to get total energy absorbed and equating it to energy input gives

$$½R\Delta = ½\cdot K\left(\frac{d_{p1}}{Y_1}\right)^2 \cdot [n(Y_1^2 + (h - Y_1)^2) + m\Sigma y_i^2]$$

but

$$\Delta = d_{p1}\cdot h/Y_1$$

$$r = K\cdot d_{p1}$$

$$R = \frac{r}{h\cdot Y_1}\cdot [n(Y_1^2 + (h - Y_1)^2) + m\Sigma y_i^2] \qquad \text{(B-7)}$$

in which
R = an estimate of maximum load for the wall
r = an estimate of maximum lateral nail load

Assuming nail failure is complete along one plate while nail connections
remain intact along the other plate, Y would equal h. This would simplify equation (B-7) to

$$R = r\cdot n + \frac{m}{h^2}\cdot \Sigma y_i^2 \qquad \text{(B-8)}$$

Applying this expression to the prediction of gypsum performance shows that as wall length increases, the prediction and test results converge. Lateral nail values obtained for frame and wallboard samples taken from the test walls indicated an 'r' of about 90 pounds per nail. For nails spaced 8 inches O.C. and a symmetric distribution of lateral nail strain about midheight of the walls, the value for Σy_i^2 for 96-inch height is 6,912. Using these values in equation (B-7) gives the predictions shown in table B-l for 8-, 16-, and 24-foot walls. Comparison with average test results shows that the prediction error decreases with wall length. This suggests that the simplified form of equation (B-7) could be useful in predicting performance of longer walls actually used in construction.

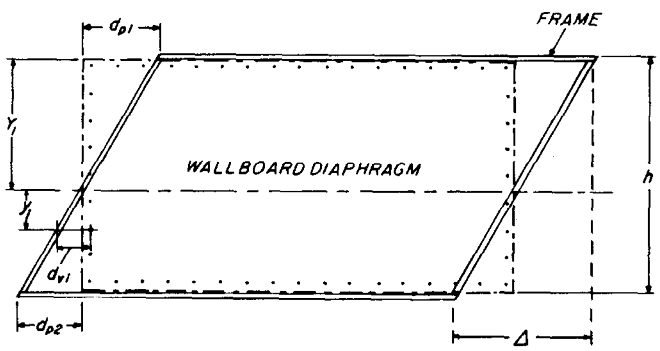

Figure B-1.—Wall racking distortion pattern ignoring vertical displacements due to diaphragm rotation. (ML835336)

Table B-1.—Simplified energy model prediction accuracy improves with wall length

Wall length	n	m	Predicted racking resistance	Measured racking resistance	Predicted / Measured
Ft					
8	13	5	1,507	1,138	1.32
16	25	9	2,857	2,465	1.16
24	37	13	4,207	4,075	1.03

Appendix C

Ramburg-Osgood model for nonlinear load deformation:

$$\epsilon = \sigma/E + K(\sigma/E)^N$$

This model assumes that the inelastic deformation (plastic + viscoelastic) can be represented as an exponential function of the linear strain (σ/E). Thus inelastic strain ($K(\sigma/E)^N$) is added to elastic strain to give total strain.

A program (fig. C-1) written for the Casio programable calculator FX 702P reads the strain "DISP" stress "LOAD" coordinates of the measured curve, estimates an initial linear slope (E), computes parameters K and N, and provides a routine to test stress given the strain. The initial E value is the slope from 0,0 to the first point put in. If the slope of the Pδ curve increases to the next point, the program recalculates E as the slope from 0,0 to the second point. Values of parameters K and N are estimated using a Ln-Ln linear regression of plastic versus elastic deformation

$$\mathrm{Ln}(\epsilon - \sigma/E) = \mathrm{Ln}\ K + N\ \mathrm{Ln}(\sigma/E)$$

Output includes the values for E, K, and N if a check is required, input "Y" EXE after the display "TEST??".

Table C-l lists model parameters derived for average load displacement curves given in table 5 of this report. Along with the parameters, there is also a listing of ratios of measured/predicted loads at several displacement levels.

Table C-1.—Ramberg-Osgood data storage model parameters and ratios of measured to predicted values

Wall		E	K	N	Measured/predicted strength					
					0.05	0.10	0.20	0.30	0.40	0.50
8 x 8	Unbraced	8,000	1,476	4.25	1.08	1.00	1.00	1.01	1.02	0.97
8 x 8	C	10,000	325	3.88	1.23	1.05	.99	.98	1.01	.99
8 x 8	T	10,000	21,700	5.85	1.02	1.01	.99	.99	1.02	.99
8 x 8	M	13,400	355	3.92	.92	.93	1.01	1.04	1.02	1.02
8 x 16	N	17,800	6,690	5.07	.94	.98	1.05	1.05	1.01	—
8 x 16	C	22,600	276,500	6.53	.87	.91	1.01	1.00	.99	1.00
8 x 24	N	38,000	8,070	4.67	1.11	.99	1.04	1.00	.97	—
8 x 24	C	50,000	61,890	5.00	1.21	.99	1.02	.99	—	—

```
LIST #1

 1 PRT "RMBRG-OSGD
   "
 5 VAC
 8 WAIT 25
10 PRT "INPUT"
12 INP "NUM PTS",N
15 FOR I=1 TO N
20 PRT "PT";I
21 INP "DISP",A(1,
   I)
22 INP "LOAD=",A(2
   ,I)
30 NEXT I
40 GOTO #2
```

```
LIST #2

10 PRT "E EST":I=1
15 IF I>N THEN 45
20 E=A(2,I)/A(1,I)
25 I=I+1
30 F=(A(2,I)-A(2,1
   ))/E+A(1,1)
40 IF F≥(A(1,I)-.0
   05) THEN 15
41 M=I
45 WAIT 50
50 PRT "E=";E
60 GOTO #3
```

```
LIST #3

 2 SAC
 5 WAIT 7
10 PRT "K,N:LN-LN
   REG"
15 FOR 1=M TO N
20 X=LN (A(2,I)/E)
25 Y=LN (A(1,I)-A(
   2,I)/E)
30 STAT X,Y
40 NEXT I
50 K=EXP LRA:F=LRB
52 WAIT 50
55 PRT ##.##↑;"K="
   ;K
56 PRT "N=";F
57 INP "TEST",T$
58 IF T$="Y" THEN
   #4
60 END
```

```
LIST #4

 10 N=F
 20 INP "STRN=",S:A
    =.8*S
 30 D=A+K*A↑N
 40 IF D<(S-.001) T
    HEN 75
 50 IF D>(S+.001) T
    HEN 75
 60 S=A*E
 70 GOTO 100
 75 A=EXP (LN A+(LN
    (S/D))/N)
 80 GOTO 30
100 PRT "STRS=";S
110 GOTO 20
120 END
```

Figure C-1.—Ramburg-Osgood load deformation model program written for Casio programable calculator FX 702P reads(load (P), displacement (d)) points along test curve and gives E, K, N parameters for model

Wolfe, R. W. Contribution of gypsum wallboard to racking resistance of light-frame walls. Res. Pap. FPL 439. Madison, Wis., U.S. Department of Agriculture, Forest Service, Forest Products Laboratory; 1983.

A study to investigate the structural contribution of gypsum wallboard showed that the wallboard contribution to racking resistance: (1) was significant, (2) added to the diagonal windbrace strength, (3) gave ultimate strength which varied linearly with length, and stiffness which increased as a power function of length, and (4) was greater for horizontal panel orientation than for vertical panel orientation.

Keywords: Walls, racking, gypsum wallboard, wind bracing, wall length, panel orientation.