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Experimental Investigation of Steel Stud Shear Wall Diaphragms

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EXPERIMENTAL INVESTIGATION OF STEEL STUD SHEAR WALL DIAPHRAGMS

by Cynthia S. McCreless* and Thomas S. Tarpy, Jr.**

Summary

This paper presents the results of an experimental full scale test program for determining design information for shear wall diaphragms constructed of steel studs and gypsum wallboard with different aspect ratios. Wall construction used is representative of the type of construction commonly used for interior wall partitions. Testing is performed in accordance with ASTM E 564 - 76.

Introduction

The shear resistant capabilities of steel stud wall panels can be of great advantage to the structural engineer in designing buildings to resist forces caused by wind, seismic action and other lateral loads. Their lateral stiffening effect to a building has long been known, however in the past steel stud wall panels have primarily been used as elements of enclosure and were designed only for the transfer of the normal components of surface loads in the structural framework. As such, the shear resistance of the panels was not utilized because of the lack of generally available and accepted

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design information. The availability of such data could permit the effective use of wall panels as main shear resisting elements in building design.

For many applications steel stud panels are perhaps the most economical and most quickly erected system for interior and exterior walls. The studs are pre-cut and pre-punched or are easily modified to allow the passage of pipes and wires and are ready to install upon delivery. The studs are connected to a runner track on both top and bottom by either welding, self-drilling screws, or friction. The wall diaphragm material can then be easily attached to the studs with self-drilling screws. From a structural viewpoint, steel studs have a high strength-to-weight ratio, leading to economical and efficient designs. For example, the framing weight for a typical wall with steel studs is considerably less than the same wall framed in wood with fewer members required.

While the advantages of steel studs are numerous, very little design information is available on the shear strength and stiffness of the panels. The shear strength and stiffness are best determined experimentally due to the many parameters (fastener spacing, wall sheathing, construction details, etc.). The experimentation to date (1977) has been limited and much remains to be done before any design data and procedures can be established.

The earliest known research project involving a full-scale diaphragm test installation was initiated at Cornell University in the mid-fifties under the direction of Winter and Nilson (10). Their research focused on light gage steel diaphragm action of floor and roof
systems. Results of this investigation demonstrated that shear
diaphragms constructed of light gage steel panels with proper
welding were capable of resisting large horizontal loads to the extent
that the need for horizontal bracing systems in many buildings could
be eliminated.

This initial investigation was followed by an extensive research
effort over the next several years of both an experimental and
analytical nature to study the behavior of both floor and wall sheet
steel diaphragms (1, 6, 15) as well as limited studies on steel stud
wall diaphragms (11). The effort culminated in the American Iron
and Steel Institute publications "Design of Light Gage Steel Diaphragms"
(5) and the "Specification for the Design of Cold-Formed Steel Structural
Members." (12)

URS/John A. Blume and Associates, beginning in the mid-sixties
as part of a structural response research program for the Nevada
Operations Office of the U.S. Energy Research and Development Administra-
tion, developed and conducted a testing program for wall panels subjected
to racking loads (2, 3, 7, 8). Fifty-four 8'-0" H x 8'-0" W wall panels
with both wood and drywall studs were tested. The majority of the
panels were constructed of gypsum wallboard, but plaster, plywood,
concrete block and combination plywood and gypsum wallboards were also
tested. Pop-rivets and friction connections were used to attach the steel
studs to the track. Also, many of the panels had architectural windows
and door openings to determine their effect on the overall wall
behavior. Both static and dynamic loading were used in testing.

While the research to date has provided many valuable results on
the behavior of light gage wall systems very little structural design
information is available to assist the structural engineer in the
construction of the wall panels for possible usage to resist lateral
loads. The purpose of this paper is to present the results of a
test program aimed at establishing preliminary design information for
typical interior light gage steel wall systems commonly encountered in
building construction.

The objective of the test program is two-fold. The first is to
determine the effect of various aspect ratios (height/width) on the
shear strength and shear resistance of steel stud wall systems.
The second is to determine the degree of panel distortion possible
before major wall panel damage is obtained. Also a single test to
determine if the shear capacity of the wall system could be increased
by the addition of a single horizontal stiffener located at mid-height
in the plane of the wall is considered.

The experimental test program consisted of testing sixteen full
size wall panels of varying aspect ratios. Displacements were meas­
ured at critical locations on the wall for varying load levels and
load displacement curves plotted. Shear strength and shear stiffness
are calculated from the test results. General observations are made
from both a construction and behavioral viewpoint on the ability of
the wall panel to effectively function as a shear resisting element
in building design.

**Experimental Test Program and Procedure**

The shear strength and stiffness of the panels are determined by
racking a panel from a rectangle to a parallelogram. This is accomplished
by fixing the base of the wall and applying a force along the top of the wall parallel to the plane of the wall. The forces required to rack the wall and the corresponding displacements at increasing load intervals are measured. The shear strength and stiffness of the panels are then calculated from the load-deflection curves.

Testing is performed in accordance with ASTM E 564 - 76 (4). This method is a static load procedure designed to evaluate the shear resistance of framed walls for buildings and is not intended to be a means for evaluating the effects of cyclic load reversals. The recommended test assembly is shown in Figure 1. Specifications are not made regarding the type of connection system used except to duplicate as nearly as possible the system intended for use in actual building construction. The wall may be tested vertically or horizontally and the panel size should not be less than 8 ft. high by 8 ft. wide.

The test method requires that at least two specimens of a given construction be tested, but if the results differ by more than ten percent a third test must be performed. This requirement is satisfied in the case that a series of tests with varying parameters are performed. The loads are applied to the wall panel so that the design load level will not be reached in under ten minutes and at least ten deflection readings recorded. The time lapse between load applications should be sufficient enough to record deformation and at load levels of one-third and two-third ultimate load the loads should be fully released and the deflection recorded after a five minute recovery period.

The shear strength and shear stiffness are obtained from the test results. The ultimate shear strength (lb/ft) is determined by dividing the ultimate load (i.e. the last load that gage deflections were
Figure 1. Racking Load Assembly ASTM E 564 - 76.
recorded) by the length of the wall panel parallel to the application of the load. The shear stiffness (lb/in) is determined as one-third the ultimate load divided by the total deflection including shear deflection and that contributed by anchorage slippage at that load level times the aspect ratio of the wall panel.

For this investigation a series of interior wall panels with various aspect ratios as shown in Table 1 were tested. The panels were tested horizontally in a steel load frame assembly designed especially for the series of tests performed. The connections used to fix the wall panel to the frame were clip angles located on 48" o.c. One face of the angle was bolted to the stud and the other face of the angle was bolted through the track to the load frame. A load bearing block and 7 1/4" 18 gage structural steel joist were attached along the top of the panel to uniformly distribute the load over the full length of the wall. A digital strain indicator in combination with a linear load cell was used to apply the load.

Each wall panel was constructed of 3 1/2", 20 gage Super C studs spaced 24" o.c. The studs were attached to 3 5/8" web by 1 1/2" flange, 20 gage structural track with #10 x 1/2" low profile head screws. To avoid skewed wall panels each stud was installed using a carpenter's square. Care was taken to avoid gaps between the studs and the track. The studs were attached by screws to both flanges of the track.

Gypsum wallboard, 1/2" thick was attached to both sides of the stud assembly with #6 x 1" Bugle Head screws spaced 12" o.c. over the entire face of the panel along both studs and runner tracks. The gypsum wallboard seams were then taped and caulked to complete the
Table 1

TEST CONFIGURATION

<table>
<thead>
<tr>
<th>TEST TYPE</th>
<th>WALL HEIGHT</th>
<th>WALL WIDTH</th>
<th>TYPE WALLBOARD</th>
<th>STUD SPACING</th>
<th>WALL FASTENER SPACING</th>
<th>STUD ATTACHMENT</th>
<th>WALLBOARD ATTACHMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>12'-0</td>
<td>12'-0</td>
<td>1/2&quot; TYPE X GYPSUM EACH FACE</td>
<td>24&quot;-O.C.</td>
<td>12&quot;-O.C.</td>
<td>#10x1/2&quot; LOW-PROFILE HEAD SCREWS</td>
<td>#6x1&quot; BUGLE HEAD SCREWS</td>
</tr>
<tr>
<td>A*</td>
<td>12'-0</td>
<td>12'-0</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
</tr>
<tr>
<td>B</td>
<td>12'-0</td>
<td>16'-0</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
</tr>
<tr>
<td>C</td>
<td>12'-0</td>
<td>24'-0</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
</tr>
<tr>
<td>D</td>
<td>10'-0</td>
<td>12'-0</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
</tr>
<tr>
<td>E</td>
<td>10'-0</td>
<td>16'-0</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
</tr>
<tr>
<td>F</td>
<td>10'-0</td>
<td>24'-0</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
</tr>
<tr>
<td>G</td>
<td>8'-0</td>
<td>8'-0</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
</tr>
<tr>
<td>H</td>
<td>8'-0</td>
<td>12'-0</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
</tr>
<tr>
<td>I</td>
<td>8'-0</td>
<td>16'-0</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
</tr>
<tr>
<td>J</td>
<td>8'-0</td>
<td>24'-0</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
<td>''</td>
</tr>
</tbody>
</table>

*with horizontal stiffener @ mid-height
construction of the wall panel. The wall panel was then allowed to set for at least 24 hours before any movement to insure proper curing of the joint compound. Once the panel had cured it was positioned in the test frame with a wood spacer of the same length as the panel located between the test frame channel and the base of the panel. The completed wall assembly located in the load frame is shown in Figure 2. Typical construction details are shown in Figure 3.

Displacement indicating dials were located on the test frame assembly at points shown in Figure 4. The dial gage at the lower right measures the slippage of the panel in the test frame. The two vertical dial gages measure panel rotation and the dial at the upper right measures the same readings as the other dial gages plus deformation of the panel. Movement of the test frame was monitored by additional dial gages; two vertical dial gages, one at the right hand corner and one at the left hand corner and one gage in the direction of the load. The movement of the frame was negligible in the direction of the load therefore no readings were recorded. The other two frame gages were recorded and used for correction purposes in the calculations.

Prior to starting a test the ultimate load was estimated and loading increments determined. A preload of ten percent of the estimated ultimate load was initially applied to the wall panel for five minutes. The load was then removed and all the dial gages set to zero. The load was then applied incrementally to the wall and displacement measurements recorded after a two minute hold to allow the wall to stabilize. At load levels of one-third and two-thirds of the estimated ultimate the load was fully removed and the wall recovery recorded after a five minute duration. The load was then re-applied to the next increment
Figure 2. Completed Wall Assembly.
Figure 3. Construction Details of the Wall System.
above the back-off load. Loading continued in this manner until the panel was no longer capable of holding any additional load. The last load held for two minutes with displacement measurements recorded is defined as the ultimate load.

As discussed earlier information obtained from the test results are load-deflection curves, ultimate shear strength, ultimate shear stiffness, and damage threshold level. The load-deflection curves are plots of the applied load versus the corresponding wall deflection;
either net or total deflection. The ultimate shear strength is determined from the ultimate load and the ultimate shear stiffness is determined from the load-deflection curves. The damage threshold is defined as the level of loading which causes major damage to the wall panel; that is, the wall is no longer structurally effective.

Total deflection is a combination of shear deflection and bending deflection. Net deflection on the other hand is total deflection minus bending deflection and anchorage deflection. ASTM recommends that the total deflection be used in all computations. It is the writers opinion, however, that computations using net deflection provide more representative data to be used in design recommendations. For these reasons computations are shown both ways.

The total deflection of the wall panels is determined as follows:

\[ \Delta_{TOT} = \Delta_1 - \Delta_4 \text{ (in)} \] (1)

where \( \Delta_4 \) is the measured deflection at gage i. The net deflection of the wall panels is determined as follows:

\[ \Delta_{NET} = \Delta_1 - [\Delta_4 + \frac{a}{b}(\Delta_3 - \Delta_7 + \Delta_5 - \Delta_6)] \text{ (in)} \] (2)

where \( \Delta_4 \) is measured deflection at gage i, \( a \) is the height of the wall panel (ft.) and \( b \) is the length of the wall panel (ft.).

The ultimate shear strength of the wall panel is determined as follows:
where \( P_u \) (lbs) is the highest load level held long enough to record gage measurements and \( b \) is the length of the wall panel (ft.).

The shear stiffness is determined from the load-deflection curve. A reference load in the elastic range of the load-deflection curve at one-third ultimate is recommended by ASTM and that load and corresponding deflection used in the calculations. The shear stiffness \( G_v \) computed from the total deflection is determined as follows:

\[
G_v = \frac{a}{b} \left( \frac{P}{\Delta_{TOT}} \right) @ \frac{1}{3} P_u
\] (4)

The shear stiffness \( G_N \) computed from the net deflection of the wall panel is determined by the relations:

\[
G_N = \frac{a}{b} \left( \frac{P}{\Delta_s} \right) @ \frac{1}{3} P_u
\] (5)

where \( \Delta_s \), the shear deflection for the one-third ultimate load, can be defined as

\[
\Delta_s = \Delta_{NET} - \Delta_B
\] (6)

where \( \Delta_{NET} \) is the deflection obtained from the load-deflection curve for the load of one-third ultimate and \( \Delta_B \) is the computed bending deflection at the free end of the cantilever beam at the one-third load level. That is
STEEL STUD SHEAR WALL DIAPHRAGMS

\[ \Delta_B = \left( \frac{P_a^3}{3EI} \right) \] \( \text{at } 1/3 P_u \) \hspace{1cm} (7)

where \( P \) is one-third ultimate load, \( a \) is the height of the panel, \( E \) is the modulus of elasticity of steel (ksi) and \( I \) is the moment of inertia (in\(^4\)) considering all the stud members of the test assembly.

Summary of Results

Table 2 shows the damage thresholds observed during testing. The first noticeable damage is the point where the first hairline cracks in the wallboard material were observed. Major damage is defined as the point where the damage to the wall was extensive and unrepairable. Human judgement is a primary factor for the determination of these values and varies from one observer to another. As such, the values reported are based on the general observations of several individuals involved in the testing.

For all tests except the longer walls bending deformation dominated. For the longer walls shear deformation controlled. Where the deflection due to shear dominated the visible signs of yielding followed the same general pattern. The first sign was the screws along the edges of the walls beginning to rotate. This is the first noticeable diaphragm damage and generally occurred at about 1/4" of deflection. As the load increased the screws continued to rotate and would eventually twist through the wallboard in the direction of the load. This is considered real damage and generally occurred at between 1/4" and 1/2". The final failure was by the stud framing shearing through the gypsum wallboard along the top.
Table 2

GYPSUM DAMAGE THRESHOLDS

<table>
<thead>
<tr>
<th>TEST TYPE</th>
<th>WALL HEIGHT</th>
<th>WALL WIDTH</th>
<th>DISPLACEMENT (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>FIRST NOTICEABLE DAMAGE</td>
</tr>
<tr>
<td>A</td>
<td>12'-0</td>
<td>12'-0</td>
<td>0.50</td>
</tr>
<tr>
<td>A*</td>
<td>12'-0</td>
<td>12'-0</td>
<td>0.50</td>
</tr>
<tr>
<td>B</td>
<td>12'-0</td>
<td>16'-0</td>
<td>0.40</td>
</tr>
<tr>
<td>C</td>
<td>12'-0</td>
<td>24'-0</td>
<td>0.20</td>
</tr>
<tr>
<td>D</td>
<td>10'-0</td>
<td>12'-0</td>
<td>0.20</td>
</tr>
<tr>
<td>E</td>
<td>10'-0</td>
<td>16'-0</td>
<td>0.50</td>
</tr>
<tr>
<td>F</td>
<td>10'-0</td>
<td>24'-0</td>
<td>0.20</td>
</tr>
<tr>
<td>G</td>
<td>8'-0</td>
<td>8'-0</td>
<td>0.40</td>
</tr>
<tr>
<td>H</td>
<td>8'-0</td>
<td>12'-0</td>
<td>0.40</td>
</tr>
<tr>
<td>I</td>
<td>8'-0</td>
<td>16'-0</td>
<td>0.30</td>
</tr>
<tr>
<td>J</td>
<td>8'-0</td>
<td>24'-0</td>
<td>0.10</td>
</tr>
</tbody>
</table>

*with horizontal stiffener @ mid-height
For the walls tested when the deflection due to bending dominated, the visible signs of yielding followed the same general pattern. The first sign of panel damage was one of the wall base track deforming around the clip angle at the exterior corner tension anchorage point. As the load was increased the wallboard fastener in the lower left corner began to rotate. This was the first noticeable wallboard damage and occurred at about 1/4" to 1/2" total displacement. Continued loading resulted in increased deformation in the track and increased cracking separation of the wallboard. This resulted in real damage to the wall panel and occurred at about 1/2" to 3/4" total displacement. The final failure was yielding of the wall system due to excessive rotation. The general types of panel failure are shown in Figure 5.

Discussion of Results

The calculated shear strength, net deflection, total deflection and shear stiffness are summarized in Table 3 for the different wall panel sizes considered.

The calculated shear strength of the wall panels seems to indicate that the shear strength is essentially independent of aspect ratio. Both maximum total and maximum net deflections follow the same general trend as far as aspect ratios with the wall deflections basically larger for the taller panels. This is reasonable in that the wall behaves as a cantilever system with larger deflection for taller walls and smaller panel moment of inertia.

The shear stiffness computed from both net deflections and total deflections shows that the shear stiffness increases for the shorter
Figure 5. Types of Panel Failure

a) Rotation of stud and track exposing clip angle in lower left corner.

b) Yielding of wall system by excessive rotation

c) Shear of stud assembly through the wall assembly
<table>
<thead>
<tr>
<th>Test Type</th>
<th>Wall Height</th>
<th>Wall Width</th>
<th>Ul. Shear Strength (lb/ft)</th>
<th>Max. Net Deflection (in.)</th>
<th>Shear Stiffness (lb/in)</th>
<th>Max. Total Deflection (in.)</th>
<th>Shear Stiffness (lb/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>12'-0</td>
<td>12'-0</td>
<td>413</td>
<td>0.78</td>
<td>13,000</td>
<td>1.05</td>
<td>8,200</td>
</tr>
<tr>
<td>A*</td>
<td>12'-0</td>
<td>12'-0</td>
<td>350</td>
<td>0.69</td>
<td>11,800</td>
<td>0.98</td>
<td>10,800</td>
</tr>
<tr>
<td>B</td>
<td>12'-0</td>
<td>16'-0</td>
<td>394</td>
<td>0.80</td>
<td>10,100</td>
<td>1.00</td>
<td>8,200</td>
</tr>
<tr>
<td>C</td>
<td>12'-0</td>
<td>24'-0</td>
<td>363</td>
<td>0.65</td>
<td>12,400</td>
<td>0.75</td>
<td>12,500</td>
</tr>
<tr>
<td>D</td>
<td>10'-0</td>
<td>12'-0</td>
<td>375</td>
<td>0.78</td>
<td>10,300</td>
<td>0.89</td>
<td>8,900</td>
</tr>
<tr>
<td>E</td>
<td>10'-0</td>
<td>16'-0</td>
<td>356</td>
<td>0.44</td>
<td>14,500</td>
<td>1.01</td>
<td>5,200</td>
</tr>
<tr>
<td>F</td>
<td>10'-0</td>
<td>24'-0</td>
<td>388</td>
<td>0.16</td>
<td>24,400</td>
<td>0.33</td>
<td>17,500</td>
</tr>
<tr>
<td>G</td>
<td>8'-0</td>
<td>8'-0</td>
<td>400</td>
<td>0.83</td>
<td>11,800</td>
<td>0.88</td>
<td>9,200</td>
</tr>
<tr>
<td>H</td>
<td>8'-0</td>
<td>12'-0</td>
<td>400</td>
<td>0.60</td>
<td>15,600</td>
<td>0.66</td>
<td>10,800</td>
</tr>
<tr>
<td>I</td>
<td>8'-0</td>
<td>16'-0</td>
<td>469</td>
<td>0.51</td>
<td>12,000</td>
<td>0.95</td>
<td>12,100</td>
</tr>
<tr>
<td>J</td>
<td>8'-0</td>
<td>24'-0</td>
<td>388</td>
<td>0.10</td>
<td>47,200</td>
<td>0.16</td>
<td>30,200</td>
</tr>
</tbody>
</table>

* with horizontal stiffener @ mid-height
and wider walls. This behavior is reflected in the modes of failure of the wall as the failure pattern shifts from one of yielding due to excessive rotation (bending) to one of shear.

The trends mentioned above appear in all test results with the exception of test types E and I. These discrepancies were in single panel tests and can be explained by problems encountered in construction of the specific wall panel. For a detailed discussion of each test one should consult references (9) and (13).

Conclusions and Recommendations

The results of the test program indicate that the steel stud wall panels as constructed could be used as a lateral load resisting element in building construction provided appropriate factors of safety and anchorage details are maintained. This conclusion is based on the ultimate shear strength of the panels as well as the level of loading at first cracking of the gypsum wallboard.

It should be noted that the test program conducted was a small statistical sample involving a particular manufacturer's products and one recommended installation procedure for interior wall partitions. Therefore, additional tests are required before precise conclusions and specific design recommendations can be made regarding the shear resistant capabilities of steel stud wall panels for the industry as a whole. Many observations and conclusions, however, can be inferred from the results.

From a construction viewpoint, much attention must be given to the workmanship and installation details due to their large effect on
the strength and deflection of the wall assembly. To avoid these difficulties, close supervision and field inspection during installation of the wall panels is recommended. The anchorage detail in the corners of the panel is a critical design consideration. This is due to the large tensile forces at the corners as the wall tends to pull away from the base runner track under lateral load. The angle clips used in the tests help resist this tendency at smaller loads but, as the load level increases, the track tends to deform around the clips due to the width of the clip angles. This problem could be alleviated by using special anchor clips in the corners for the full width of the track.

Another detail which must be considered is the method of attaching the studs to the runner track. Comparison with results of other test programs (8, 14) indicate that using fasteners to attach the stud to the runner track results in a stronger wall than using either resistance spot welds or friction connections. This observation is based on a one test comparison and is really too limited to accurately draw any general conclusions. Additional tests on panels of the same construction with both resistance spot welds and friction connections are necessary for a detailed comparison.

The addition of a horizontal stiffener at mid-height did not increase shear capacity; and, while one test is not necessarily enough to draw conclusions, it is felt that this approach is not feasible due to construction difficulties and cost of installation far outweighing any anticipated structural advantages.

The effect of varying the gypsum wallboard attachment points from
that tested to a smaller value around the perimeter should increase the wall shear strength and stiffness. However, the percent of increase is unknown. Also locating several screws in the corners of the wall should help increase the wall strength before first cracking of the gypsum.

The effect of varying the stud spacing to smaller centers is not felt to be a critical design parameter as the two foot spacing is a common spacing used for interior walls. On the other hand, the use of sixteen inch centers for exterior walls with gypsum on one face and sheathing on the other face where transverse wind controls should be investigated.

From a behavioral point of view the results clearly indicate that the shear strength is essentially independent of aspect ratio while the wall deflections are basically larger for the taller panels. This is reasonable in that the wall behaves essentially as a cantilever system with larger deflection for taller walls and smaller panel moments of inertia. The wall shear stiffness in turn increases for the shorter and wider walls as the deflection decreases. This behavior is reflected in the type of failure of the walls as the failure pattern shifts from one of yielding due to bending which is analogous to a cantilever beam to one of shear.

The test results reported herein provide a preliminary basis for the strength, deflection and damage thresholds of wall panels subjected to static loads. In order to accurately incorporate the structural capabilities of wall panels into certain design codes, the effects of a structure subjected to a succession of reversed loading cycles of
both a progressively increasing magnitude and large initial impact must be investigated. This loading is analogous to forces induced by wind gusts or earthquakes where a structure is subjected to a force of large magnitude and suddenly the force is removed. To obtain such data additional tests on wall panels need to be performed using a cyclic-loading test procedure. To date (1977) a recommended testing procedure by ASTM for cyclic loading is not available, and a procedure analogous to the static test standard must be used.

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Appendix--References


Appendix--Notations

\( a = \) Height of the wall panel (ft)
\( b = \) Length of the wall panel (ft)
\( E = \) Modulus of elasticity of steel (ksi)
\( G_N' = \) Shear stiffness based on net deflection (lb/in)
\( G_T' = \) Shear stiffness based on total deflection (lb/in)
\( I = \) Moment of inertia of steel frame (in\(^4\))
\( P_u = \) Ultimate load (lb)
\( S_u = \) Ultimate shear strength (lb/ft)

\( \Delta_B = \) Bending deflection (in)
\( \Delta_i = \) Deflection at gage \( i \) (in)
\( \Delta_{NET} = \) Net deflection (in)
\( \Delta_S = \) Shear deflection (in)
\( \Delta_{TOT} = \) Total deflection (in)