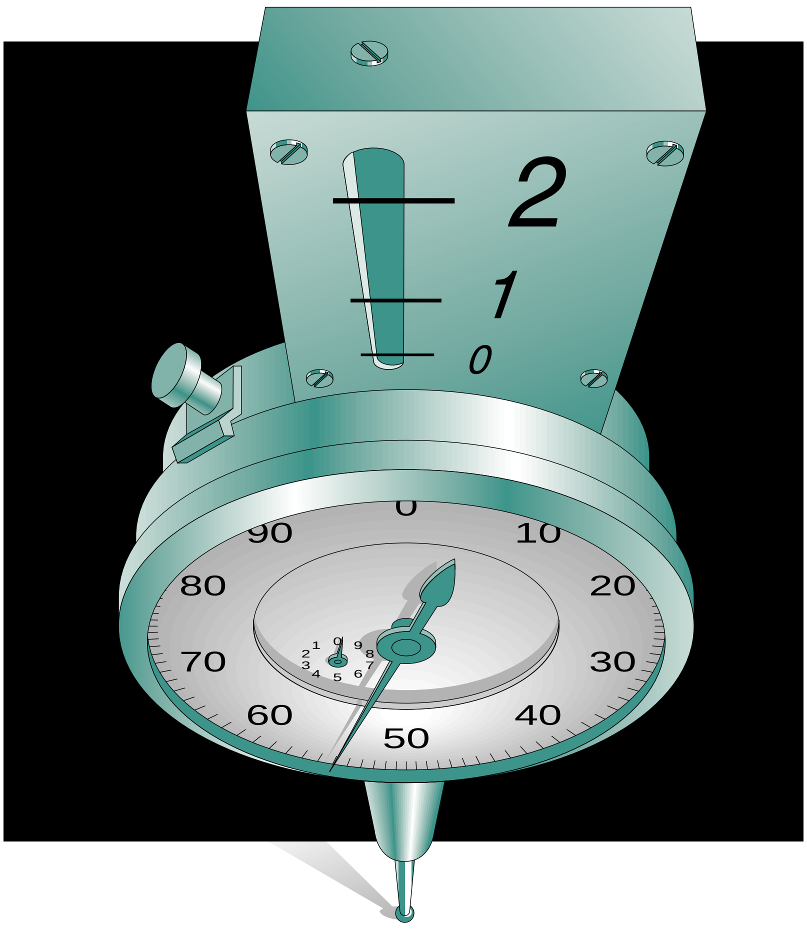
Report 158

Preliminary Testing of Wood Structural Panel Shear Walls Under Cyclic (Reversed) Loading

by John D. Rose • March, 1998
Technical Services Division



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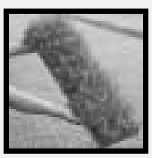

- ***We're growing more wood every day.*** American landowners plant more than two billion trees every year. In addition, millions of trees seed naturally. The forest products industry, which comprises about 15 percent of forestland ownership, is responsible for 41 percent of replanted forest acreage. That works out to more than one billion trees a year, or about three million trees planted every day. This high rate of replanting accounts for the fact that each year, 27 percent more timber is grown than is harvested. Canada's replanting record shows a fourfold increase in the number of trees planted between 1975 and 1990.

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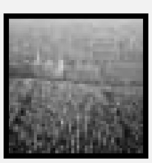
Material	Percent of Production	Percent of Energy Use
Wood	47	4
Steel	23	48
Aluminum	2	8



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ABSTRACT

Damage to wood-framed residential buildings in the January, 1994 earthquake at Northridge, California raised questions about performance of shear wall assemblies when subjected to cyclic (reversed) loading. Factors such as the effect of monotonic (one-directional) loading versus cyclic loading on shear wall stiffness and strength; contribution of dissimilar materials to stiffness and strength of the wall; shear wall displacement limits, to minimize cosmetic and structural building damage during an earthquake; ductility; fatigue resistance of fasteners; and slip and deformation of shear wall hold-down connectors are considerations when walls are subjected to cyclic loading from seismic conditions.

Evaluation of shear wall performance historically has been based on monotonic testing, which is used to develop provisions for prescriptive wall bracing and engineered shear walls for incorporation in model building codes. There are presently no recognized U.S. standards for conducting cyclic load testing of assemblies. However, a modification of a sequential phased displacement test procedure – developed cooperatively in 1987 by U.S. and Japanese engineers and scientists participating on the Joint Technical Coordinating Committee on Masonry Research (TCCMAR) – has been used as a basis for developing a cyclic load test method for evaluating performance of structural assemblies.

Eight 8-foot x 8-foot shear walls with wood structural panel sheathing were tested using the modified TCCMAR procedure. Sheathing materials and fastener schedules were varied to obtain information on the load-displacement characteristics and load capacity of typical constructions. Gypsum wallboard was included as a sheathing material in one test, to evaluate the contribution of dissimilar sheathing materials to wall stiffness and strength. Shear wall performance, as determined by a simplified cyclic load test procedure used for evaluating components for steel structures, also was investigated for comparison with results obtained by the modified TCCMAR procedure. This information has been used to assist in developing a standardized test procedure for cyclic (reversed) loading of structural assemblies, and in understanding the performance of wood-framed shear walls under cyclic loading, and the factors which govern their performance.

Reported by:

John D. Rose
Senior Engineer
Technical Services Division

Reviewed by:

William A. Baker, P.E.
Manager, Market Support Services
Technical Services Division

Laboratory Accreditation

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This report contains data generated through testing of engineered wood products according to various test methods. Many test methods conducted by APA are accredited by the organizations listed above. A list of accredited methods is available upon request. Any test data in this report that is derived from test methods which deviate from accepted accredited procedure are noted. Accreditation does not constitute endorsement of this report by the accrediting agency or government.

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CONTENTS

INTRODUCTION

In the United States, model building code recognition of allowable design shear loads for shear walls is based on results of tests using "monotonic" loading; e.g., shear loads are applied in the plane of the wall, acting in one direction only.[1,2] Observations of building performance after hurricanes and earthquakes indicates that this method of evaluating shear wall performance generally provides a conservative basis for building design, when wood structural panels are used for wall sheathing or siding.

In high wind conditions, building walls are subjected to in-plane shear loads (as well as normal uniform loads) which act primarily in one direction, with fluctuations in load due to the response of the structure to wind gusts. In hurricane or tornado conditions, reversal of shear loads from the opposite direction may occur if the building happens to be located in or near the "eye" of the storm.

In contrast, building walls are subjected to in-plane cyclic (reversed) loads in earthquake/seismic conditions. For a number of years, structural engineers and building officials have requested information on the performance of shear walls under cyclic loading, to evaluate whether code-recognized values are appropriate under such conditions. However, response to these requests has been hampered by the lack of consensus on a test protocol for conducting cyclic load tests. Furthermore, until recently, no commercial or industry testing laboratories in the United States had equipment for conducting such tests, except on a rudimentary basis. Because of the expense of the test equipment, only a few university laboratories are equipped for and capable of conducting cyclic load tests on shear walls.

After the Northridge, California earthquake in January, 1994, evaluation of shear wall performance by cyclic load tests was encouraged by a local building code change adopted by the City of Los Angeles, Department of Building and Safety, in response to recommendations from the Structural Engineers Association of Southern California (SEAOSC). This change reduced the allowable design shear loads by 25%, for shear walls fabricated with wood structural panels, until cyclic load tests are conducted to confirm previously recognized values, or another appropriate reduction factor is proposed.

SEAOSC formed an Ad Hoc Committee on Testing Standards for Structural Systems and Components, to study appropriate test protocols and evaluation criteria for cyclic load tests on assemblies such as shear walls, hold-down connectors, fasteners, and anchor connectors into concrete and masonry elements. Their deliberations have focused on a cyclic load test protocol suggested for masonry construction during earthquake research discussions in 1987 between the United States and Japan.

To address these concerns, and provide preliminary information on shear wall performance under cyclic load conditions, *APA – The Engineered Wood Association* sponsored a limited series of cyclic load tests on seven 8-foot x 8-foot shear walls, fabricated with wood structural panel sheathing attached to wood framing with hand-driven nails. One additional shear wall test, on a matched construction but with wall sheathing attached with pneumatically driven nails, was sponsored by a fastener manufacturer and results are included in this report. The tests were conducted at the Structural Laboratory of the Department of Civil and Environmental Engineering at the University of California - Irvine (Irvine, California), and used the cyclic load test method developed by SEAOSC.

OBJECTIVE

The objectives of these shear wall tests are to:

1. Obtain preliminary data on the load-displacement characteristics and load capacity of typical wood-framed walls with wood structural panel sheathing, under cyclic load conditions. Wood structural panel sheathing included both plywood and oriented strand board (OSB) in three thicknesses and two panel grades.

2. Study two different cyclic load-displacement test protocols, in tests using identical wall constructions, to determine whether results adequately characterize the performance of shear wall assemblies. One protocol, on which a SEAOSC recommended test method is based, is referred to as the TCCMAR sequential phased displacement procedure, which was proposed in 1987 by the Joint Technical Coordinating Committee on Masonry Research for the United States - Japan Coordinated Earthquake Research Program. This protocol was used in seven of eight tests in the series. The other protocol, used in one test, consists of "ramped" cyclic displacement increments without subsequent "decay" cycles used in the TCCMAR procedure.

3. Evaluate whether gypsum wallboard, installed on one side of the wall, contributes to the stiffness and strength of shear walls sheathed with wood structural panels on the other side of the wall, under cyclic load conditions.

4. Evaluate shear wall performance when wood structural panel sheathing is fastened with pneumatically driven common "short" (diaphragm) nails, popularly used as an alternate to hand driven common nails.

TEST SPECIMENS AND MATERIALS

Shear wall test specimens were fabricated using an 8-foot x 8-foot frame with construction as shown in Figure 1 and Table 1. The wall construction complies with recent local building code changes adopted by the City of Los Angeles, for shear walls when allowable design shear load exceeds 300 pounds per foot. The allowable design shear loads for the shear wall test specimens are listed in Table 1, and are based on recognized values listed in Table 23-II-I-1 of the 1997 Uniform Building Code, and in National Evaluation Service, Inc. Report NER-108.

Framing

Lumber used for framing in all tests was green Douglas-fir. At the time of test, the lumber moisture content was in the range of 12 - 18%, as determined by a resistance-type electric moisture meter. The wall sheathing was fastened to the framing within 48 hours of testing, which minimized any effects of moisture seasoning (drying) on the performance of the wall.

Intermediate wall studs consisted of 2x4s spaced 16 inches on center, and double 2x4s were used for top plates. End studs (or posts) consisted of 4x4s. To conform with emergency changes adopted in 1994 to local building code requirements for the City of Los Angeles, 3x4s were used for the bottom plate, and for the center stud where the wall sheathing panels butt together, both of which are required when the allowable design shear load for the wall is higher than 300 pounds per foot. A similar requirement has been incorporated in the 1997 Uniform Building Code, when allowable shear values exceed 350 pounds per foot.[a] The increased lumber size for the bottom plate was deemed necessary to increase bending capacity of the bottom plate when the wall is subjected to racking shear and uplift forces from sheathing fasteners, and to increase resistance to splitting along lines of foundation and hold-down bolts. The bottom plate must resist both cross-grain bending and tension perpendicular-to-grain forces, developed by sheathing fasteners. For the center stud, the 3x4 member is required to provide sufficient lumber edge distance for fasteners when the sheathing edge distance for fasteners is increased from 3/8 inch to 1/2 inch, which was another building code change adopted by the City of Los Angeles when the allowable design shear load for the wall is higher than 300 pounds per foot.

The above changes to framing requirements were based on shear wall failure modes observed in buildings damaged in the Northridge, California earthquake. In many cases, the bottom plate of shear walls was notched, or enlarged holes were cut for installing foundation bolts,

[a]Table 23-II-I-1, Footnote 3

FIGURE 1

Cyclic load shear wall test specimen

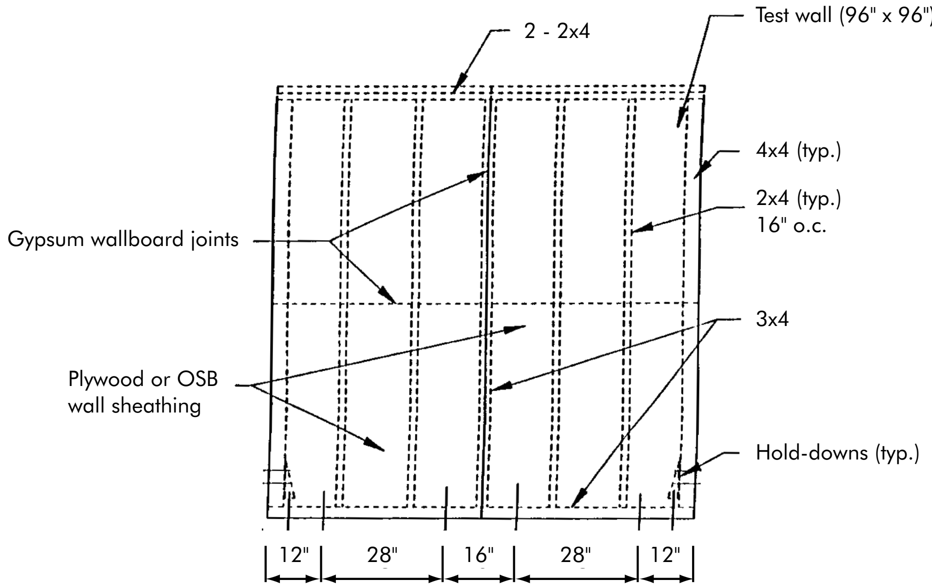


TABLE 1

Description of APA cyclic load shear wall test specimens (tests conducted at University of California - Irvine, April 1995)

Test No.	Wall Sheathing		Sheathing Nail Schedule(1)		Shear Wall Design Load, lb/ft(2)
	Ext.	Int.	Ext.	Int.	
1A	15/32" Plywood (Structural I)	–	10d Com. @ 4" oc	–	510
2B	15/32" Plywood (Structural I)	–	10 Com. @ 4" oc	–	510
3A	15/32" Plywood (Structural I)	1/2" GWB	10d Com. @ 4" oc	5d GWB 7" oc	510(4)
4A	15/32" OSB#1 (Structural I)	–	10d Com. @ 4" oc	–	510
5A	15/32" OSB#2 (Structural I)	–	10d Com. @4" oc	–	510
6A	3/8" Plywood (Structural I)	–	8d Com. @ 3" oc	–	550
7A	7/16" OSB#3	–	8d Com. @ 3" oc	–	490
8A	15/32" OSB#1 (Structural I)	–	10d Com., Short(3) @ 4" oc	–	510

(1) 10d Com: 0.148" x 3" (head dia. – 0.312") – Tests 1A, 2B, 3A, 4A, 5A
8d Com: 0.131 x 2-1/2" (head dia. – 0.281") – Tests 6A, 7A
5d GWB (wallboard-smooth shank): 0.092" x 1-5/8" (head dia. – 0.297") – Test 3A

(2) Table 23-II-I-1 of 1997 Uniform Building Code (UBC)
and Sec. 2302 – wood structural panel definition, and Sec. 2315.5.3. Also NER-108, Table 2.

(3) 10d Com. "short": 0.148" x 2-1/8" (head dia. – 0.279") – Test 8A

(4) Does not include shear contribution from gypsum wallboard (GWB) per Sec. 2513.1 of 1997 UBC

wiring conduit or plumbing. These field practices severely reduce the capacity of the bottom plate to resist uplift/bending forces that develop through the wall sheathing fasteners, when the wall sheathing panels rotate in response to lateral shear forces on the wall. Further, it was not uncommon to find that sheathing fasteners at panel edges missed the framing members, or were driven too close to the edges (or ends) of the panels, which diminished the shear resistance of the wall and "pull-through" resistance of fasteners near panel edges. These deficiencies can be detected by inspection of shear walls, and should be corrected during construction.

Wall Sheathing

Plywood and oriented strand board (OSB) wood structural panels were used for wall sheathing in the eight shear wall tests. For six of the tests, 15/32-inch APA Structural I Rated Sheathing 32/16, Exposure 1 was used (5-ply plywood and OSB – three tests each), while for one test 3/8-inch APA Structural I Rated Sheathing 24/0, Exposure 1 plywood (3-ply) was used. In another test, 7/16-inch APA Rated Sheathing 24/16, Exposure 1 OSB was used.

Plywood panels were manufactured in accordance with U.S. Product Standard PS 1-83[b], *Construction and Industrial Plywood*. The OSB panels were manufactured to comply with the performance requirements of APA Standard PRP-108, *Performance Standards and Policies for Structural-Use Panels*[c].

In all tests, wood structural panels were applied vertically, with their face grain or strength axis parallel to studs. Panel edges were spaced 1/8 inch apart along the vertical joint at the center stud, to conform with APA recommendations for panel installation, which provides space for panels to expand if subjected to moisture under on-site field conditions.

In one test, 1/2-inch gypsum wallboard was installed on one side of the wall, opposite the wood structural panel sheathing. The gypsum wallboard was installed horizontally, as is typical in practice, with an unblocked horizontal joint at mid-height of the wall. The upper panels were installed in two 4-foot x 4-foot panels with a vertical joint located at the center stud, while the lower panel consisted of a single 4-foot x 8-foot panel. The joints between panels were butted to a light contact. No tape or joint compound was used over gypsum wallboard joints, to minimize the assembly and testing schedule.

Fasteners

Wood structural panel sheathing was fastened to framing with hand driven, U.S.-made 8d and 10d common steel nails for all but one of the tests. Samples of fasteners were measured and compared (Table 2) with standard dimensions in Federal Specification FF-N-105, *Nails, Brads, Staples and Spikes: Wire, Cut and Wrought*[d].

In one test, pneumatically driven 10d common "short" (diaphragm) nails were used. When used to fasten 15/32-inch wood structural panel sheathing, these nominal 0.148-inch-diameter x 2-1/8-inch-long nails provide penetration into framing of 1.659 inch, or slightly more than 1-5/8-inch minimum penetration prescribed in Table 23-II-I-1 of the 1997 Uniform Building Code. The Uniform Building Code requirement is based on minimum fastener penetration into framing of approximately 11 times the nail shank diameter (= 11 x 0.148 inch = 1.628 inch penetration). The pneumatic nailing tool for driving these nails was equipped with an adjustable device to control the depth to which the fastener is installed, so that the top of the nail heads were set flush with the surface of the panel. For diaphragm (and shear wall) applications, countersinking of fasteners below the surface of the panel is not permitted by Section 2315.1 of the 1997 Uniform Building Code.

[b]Now PS 1-95.

[c]U.S. Product Standard PS 2-92, *Performance Standard for Wood-Based Structural-Use Panels,* establishes comparable requirements.

[d]Superceded by ASTM Standard F1667, *Standard Specification for Driven Fasteners: Nails, Spikes, and Staples.*

TABLE 2

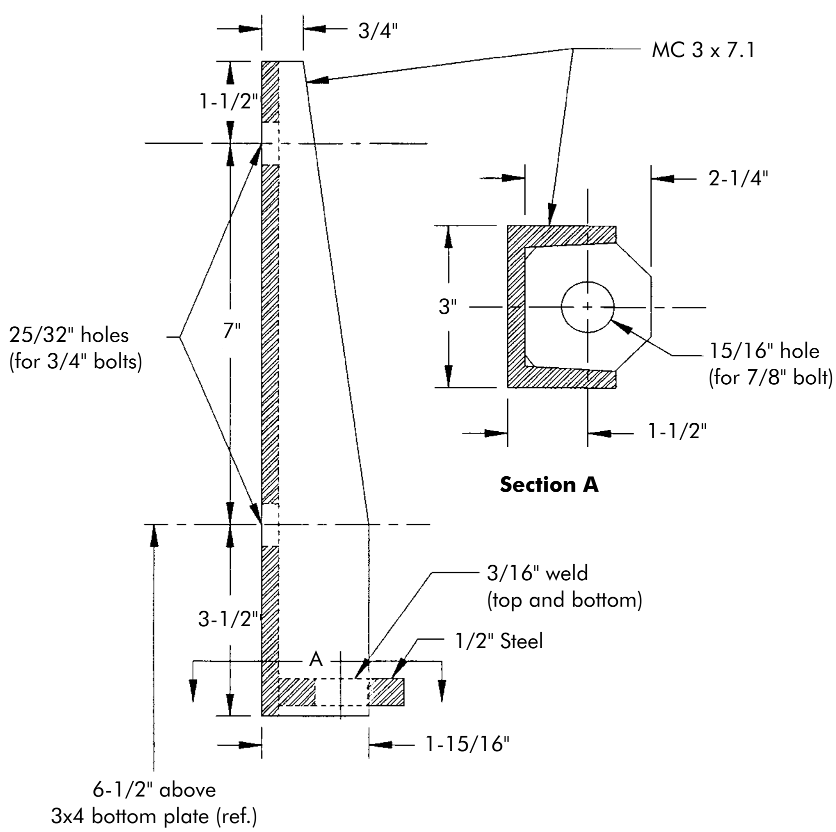
Description of nails used for attachment of wood structural panel sheathing and gypsum wallboard

	Dimensions, in.					
	Length		Shank Diameter		Head Diameter	
Nail Type	**Spec.[a]**	**Actual**	**Spec.[a]**	**Actual**	**Spec.[a]**	**Actual (avg.)**
8d common	2-1/2 (±1/16)	2-1/2	0.131 (±0.004)	0.140	0.281 (±10%)	0.267
10d common	3 (±3/32)	3	0.148 (±0.004)	0.150	0.312 (±10%)	0.292
10d common short (diaphragm)	–	2-1/8	–	0.147	–	0.279
5d gypsum wallboard (smooth)	1-3/4 (±1/16)	1-5/8	0.092 (±0.004)	0.092	0.375 (±10%)	0.301

(a) Federal Specification FF-N-105B (superceded by ASTM Standard F1667)

FIGURE 2

Shear wall hold-down connector details



Hold-Down Connectors

Specially designed welded steel hold-downs were used on both 4x4 end posts to prevent overturning and uplift of the wall when subjected to in-plane lateral forces (Figure 2). The hold-downs were designed by California structural engineer Ben L. Schmid, SE for another series of tests conducted on similar shear walls at the University of California - Irvine (see Appendix B), and were re-used in these tests. The hold-down design incorporates two 2-5/8-inch-diameter steel shear plate connectors, described in Part X of the *National Design Specification for Wood Construction* (NDS)[3]; and also in ASTM D5933, *Standard Specification for 2-5/8 in. and 4-in. Diameter Metal Shear Plates for Use in Wood Constructions.* The shear plates distribute bolt loads and minimize bolt deformation or slip in the end posts. Also, 1/4-inch-thick x 2-1/2-inch square steel plate washers were placed under nuts used with 3/4-inch-diameter hold-down bolts through the end posts, to distribute bolt loads without deforming the washer. If standard steel round cut washers had been used (instead of plate washers), field observations and experience indicates that they could deform and precipitate splitting failures in the end posts.

Hold-down deformation and bolt slip were minimized by using tight-fitting shear plate connectors, and careful drilling of bolt holes in the end posts using a template and drill guide. The diameter of the bolt holes was limited to a maximum of 1/16 inch oversize, also to minimize deformation, in accordance with NDS provisions. The groove and dap for the shear plate connector were accurately prebored in the end posts before the wall framing was assembled, using a template and drill guide to accurately locate the holes, and a special cutter head obtained from the manufacturer of the connector.

Deformation or slip of the hold-down connectors contributes directly to wall deflection, in proportion to the aspect (height/length) ratio of the wall. It also generates an added uplift component for lateral shear forces for sheathing fasteners along the bottom and top plates of the wall, which can contribute to splitting of the plates from cross-grain tension, and premature fastener fatigue failures under cyclic load conditions, thereby reducing the shear capacity of the wall.

TEST SET-UP AND PROCEDURE

The shear wall test specimens were attached to the base of the test fixture with four 5/8-inch-diameter anchor bolts, located as shown in Figure 1. The bottom plate of the shear wall was supported on a 3-inch-wide steel channel. The edges of the channel were inset from the nailing surfaces of the wall framing, to allow the sheathing panels to rotate when the shear wall is laterally loaded, without bearing on the test fixture at the bottom corners of the panels. Bolt holes in the 3x4 bottom plate were carefully located and predrilled using a template and drill guide, to minimize bolt deformation and slip when the shear wall is laterally loaded in the plane of the wall. The diameter of bolt holes in the bottom plate was limited to a maximum of 1/16 inch oversize, for reasons as noted above. Steel square plate washers, 1/4 inch thick x 2-1/2 inches square, were used under nuts for the bottom plate bolts.

Racking shear loads were applied horizontally to the top of the shear wall test specimens through a steel H-beam lag-screwed (with 3/8-inch-diameter lag screws spaced 16 inches on center) to the double top plate of the wall. The beam was placed on a 3/4-inch-thick x 3-1/4-inch-wide plywood spacer, sandwiched between the beam and the top plate, which allows the wall sheathing panels to rotate when the shear wall is laterally loaded, without bearing on the beam at the top corners of the panels. The beam was restrained laterally (normal to the plane of the wall) by pairs of Teflon® plastic pads fastened to the beam and to adjacent steel bracing frames, to allow in-plane displacement of the wall during the test, while preventing the top of the wall from twisting outside of the load application plane.

Racking shear loads were applied with a 55,000-pound-capacity programmable double-acting hydraulic actuator (MTS Systems Corporation) which was bolted to a stiff vertical cantilever column hinged at the base to act as a lever arm, as shown in Figure 3. The hinged cantilever column arrangement was necessary to “multiply” the actuator displacement, which was limited to ±3 inches in either direction, to achieve greater displacement at the top of the wall. The horizontal load at the top of the wall was measured with a hydraulic load cell in the horizontal plane of the top plate and steel H-beam.

FIGURE 3

Test setup for cyclic load shear wall tests (University of California-Irvine)

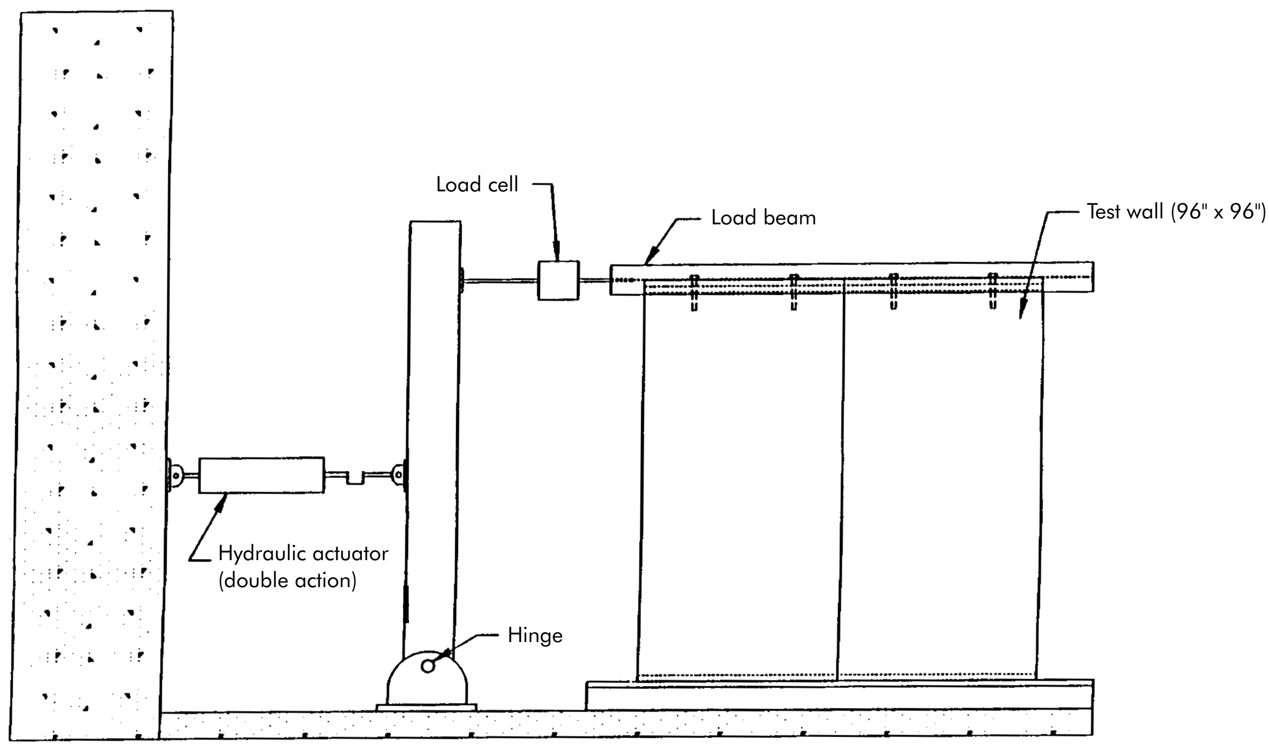


FIGURE 4

TCCMAR sequential phased displacement load-displacement procedure (Protocol A) for shear walls

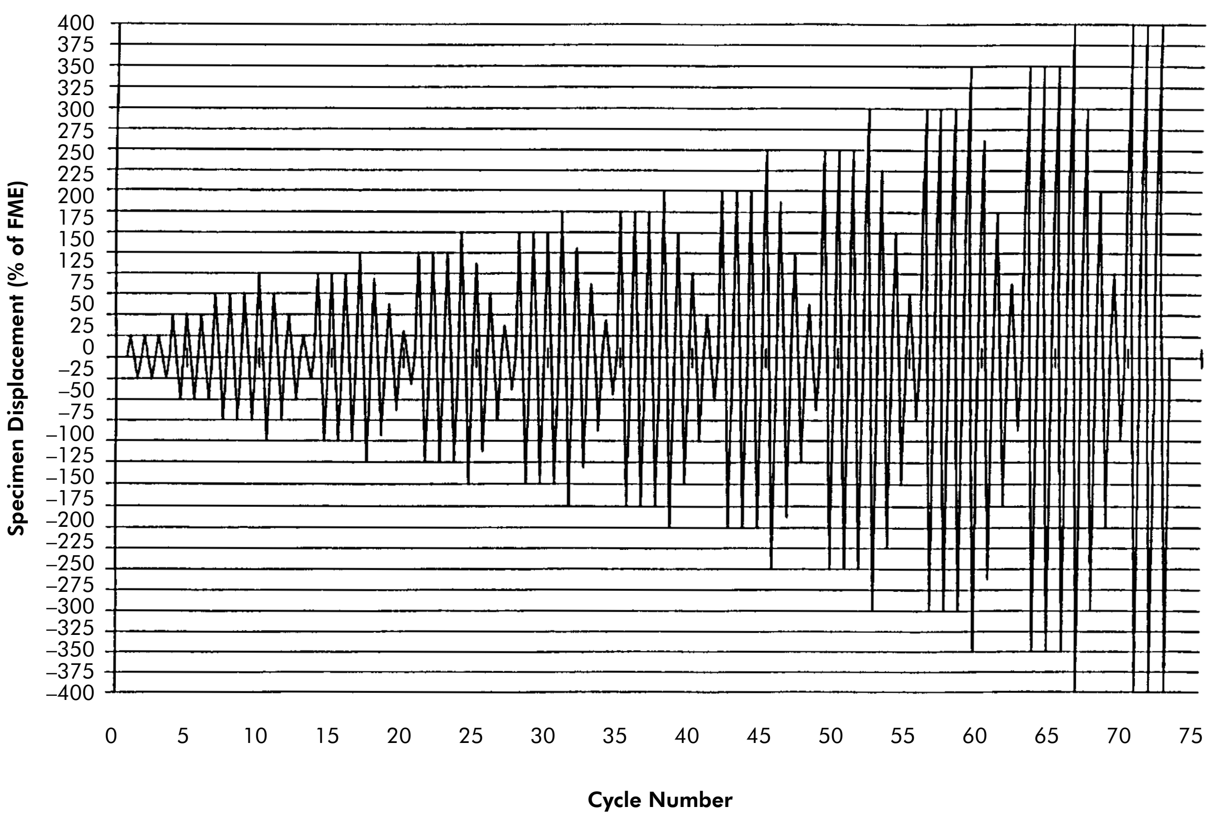


TABLE 3

TCCMAR sequential phased displacement load-displacement procedure (Protocol A) for shear walls

Cycle No.	% FME	Displ. (in.)
0	0	0
1	25	0.20
2	25	0.20
3	25	0.20
4	50	0.40
5	50	0.40
6	50	0.40
7	75	0.60
8	75	0.60
9	75	0.60
10	100	0.80
11	75	0.60
12	50	0.40
13	25	0.20
14	100	0.80
15	100	0.80
16	100	0.80
17	125	1.00
18	94	0.75
19	63	0.50
20	31	0.25
21	125	1.00
22	125	1.00
23	125	1.00
24	150	1.20
25	113	0.90
26	75	0.60
27	38	0.30
28	150	1.20
29	150	1.20
30	150	1.20
31	175	1.40
32	131	1.05
33	88	0.70
34	44	0.35
35	175	1.40
36	175	1.40
37	175	1.40
38	200	1.60
39	150	1.20
40	100	0.80
41	50	0.40
42	200	1.60
43	200	1.60
44	200	1.60
45	250	2.00
46	188	1.50
47	125	1.00
48	63	0.50
49	250	2.00
50	250	2.00
51	250	2.00
52	300	2.40
53	225	1.80
54	150	1.20
55	75	0.60
56	300	2.40
57	300	2.40
58	300	2.40
59	350	2.80
60	263	2.10
61	175	1.40
62	88	0.70
63	350	2.80
64	350	2.80
65	350	2.80
66	400	3.20
67	300	2.40
68	200	1.60
69	100	0.80
70	400	3.20
71	400	3.20
72	400	3.20

Seven of the eight shear walls (i.e., all tests except Test 2B) were subjected to fully reversing cyclic displacements following the TCCMAR sequential phased displacement procedure (Protocol A) described in Figure 4 and Table 3. This test procedure has been reviewed and refined over a 2-1/2-year period, and a finalized version was published in 1997 by the Structural Engineers Association of Southern California (SEAOSC) as *Standard Method of Cyclic (Reversed) Load Test for Shear Resistance of Framed Walls for Buildings.*[9]

The TCCMAR procedure introduces the concept of the First Major Event (FME), which is defined as the first significant limit state which occurs during the test. A limit state is an event which marks the demarcation between two behavior states. When a limit state occurs, some structural behavior of the element or system is altered. FME occurs when the load capacity of the wall, upon recycling of load to the same wall displacement increment, first drops noticeably from the original load and displacement. However, the effect of FME for wood-framed assemblies is not as pronounced as for rigid concrete or masonry assemblies, where cracking or bond failures around reinforcing steel may result in a substantial change in the stiffness or strength of the assembly. FME can be determined from preliminary cyclic load tests on an identical test specimen. For the wood-framed shear wall test specimens in this series, FME for all tests was estimated from prior cyclic load tests to occur at a displacement of about 0.8 inch. Subsequent analysis of test results for the eleven shear wall constructions described in this report revealed that FME (e.g., Yield Limit State) occurred at an average displacement of 0.48 inch (0.5% of height) (see discussion on pages 24-26).

The TCCMAR procedure consists of applying three cycles of fully reversing load (displacement controlled) at each wall displacement increment representing 25%, 50%, and 75% of FME. Then, wall displacement is increased for one load cycle to 100% of FME. Next, "decay" cycles of displacement for one cycle each at 75%, 50% and 25% of the maximum displacement (100% of FME) are applied, followed by three cycles of displacement at maximum displacement (100% of FME) to stabilize the load-displacement response of the wall. Then, the next increment of increased displacement (125% of FME) is applied, followed by similar decay and stabilization cycles of loading. This incremental cyclic load-displacement sequence is continued to 150%, 175%, 200%, 250%, 300%, 350% and 400% of FME, or until the wall exhibits excessive displacement (in these tests, at about 350% to 400% of FME), at which point shear load capacity is greatly diminished.

The other cyclic load procedure (Protocol B), used for Test 2B only, consists of applying three cycles of displacement, with increments of 0.2 inch for each cycle (see Figure 5 and Table 4); no decay cycles of displacement are applied. This procedure is similar to one developed by the Applied Technology Council (ATC) for seismic testing of steel structures.[7] The incremental cyclic load-displacement sequence is continued, with the same end point conditions as described above for the TCCMAR procedure (Protocol A). This simpler "ramped" loading procedure was included as an alternate to the TCCMAR procedure, to evaluate if wall performance is influenced by the test procedure.

The wall displacements were input to a computer which controlled the actuator displacement. A cyclic frequency of 0.5 Hz (e.g., one cycle per 2 seconds) was used to avoid inertial effects of the mass of the wall and test fixture hardware during cyclic loading, which could affect system response to cyclic loading. During later stages of the cyclic loading sequence, when wall displacement exceeded 1.5 inches, the cyclic frequency was slowed to a constant rate of displacement (about 0.25 Hz) to properly control the hydraulic system with the instrumented displacement input.

Instrumentation included measurement of horizontal displacement of the wall at the top plate, vertical displacement at the bottom end of both end posts relative to test base (uplift and compression), horizontal displacement of the bottom plate relative to the test base (lateral in-plane sliding), and vertical displacement of the hold-down connectors relative to the end posts (displacement/bolt slip).

FIGURE 5

"Ramped" cyclic load-displacement procedure (Protocol B) for shear walls

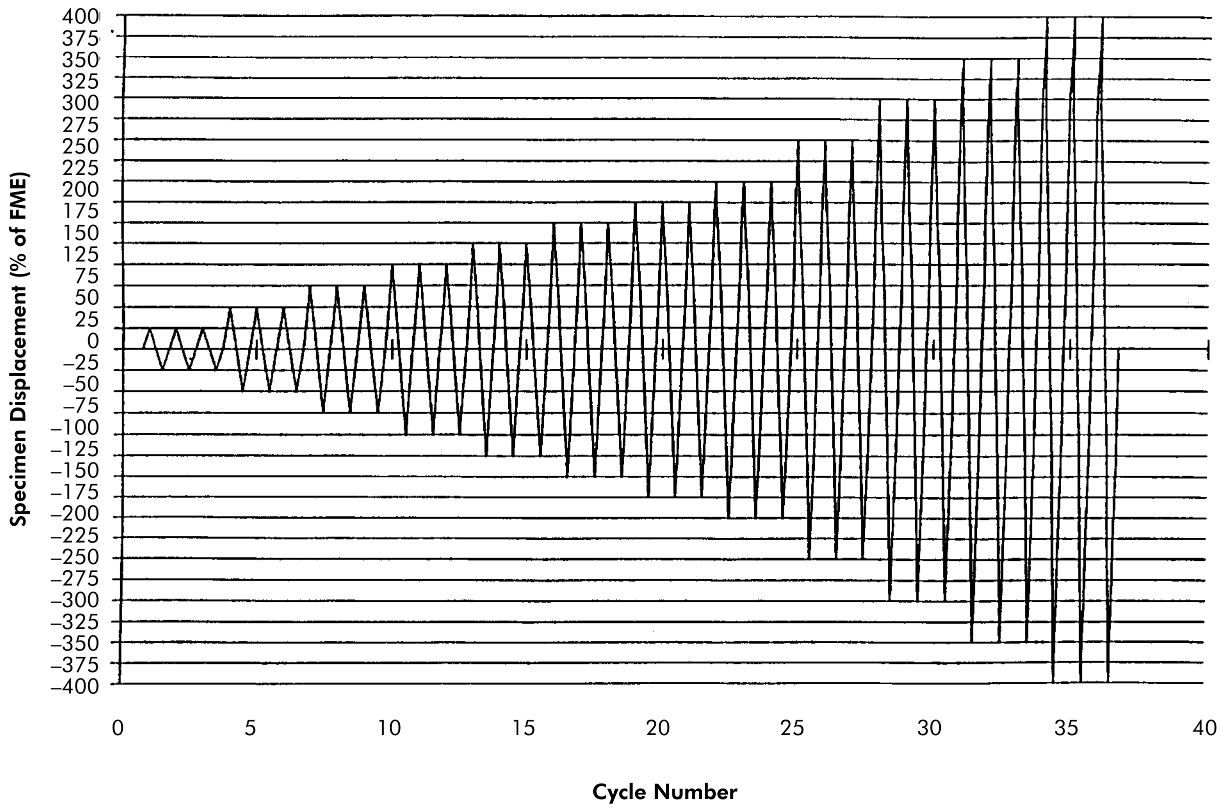


TABLE 4

"Ramped" cyclic load-displacement procedure (Protocol B) for shear walls

Cycle No.	% FME	Displ. (in.)	Cycle No.	% FME	Displ. (in.)
1	25	0.20	19	175	1.40
2	25	0.20	20	175	1.40
3	25	0.20	21	175	1.40
4	50	0.40	22	200	1.60
5	50	0.40	23	200	1.60
6	50	0.40	24	200	1.60
7	75	0.60	25	250	2.00
8	75	0.60	26	250	2.00
9	75	0.60	27	250	2.00
10	100	0.80	28	300	2.40
11	100	0.80	29	300	2.40
12	100	0.80	30	300	2.40
13	125	1.00	31	350	2.80
14	125	1.00	32	350	2.80
15	125	1.00	33	350	2.80
16	150	1.20	34	400	3.20
17	150	1.20	35	400	3.20
18	150	1.20	36	400	3.20

TABLE 5
Results of cyclic load shear wall tests (APA/UC-Irvine, 1995)

Test No.	Description	Design			At Δ = 0.48 in.[(a)]		At Δ = 0.96 in.[(a)]		At Δ = 1.44 in.[(a)]		Avg. Max. Load, lb (L.F.)
		lb/ft	lb	Δ in.	Load, lb	G_1[(b)]	Load, lb	G_2[(b)]	Load, lb	G_3[(b)]	
1A	15/32" Str. I Plywood 10d com. @ 4" o.c.	510	4,080	0.28	6,200	12.9	7,900	8.2	8,300	5.8	10,150 (2.5)
2B	15/32" Str. I Plywood 10d com. @ 4" o.c.	510	4,080	0.21	7,300	15.2	8,750	9.1	9,300	6.5	10,900 (2.7)
3A	15/32" Str. I Plywood 10d com. @ 4" o.c. 1/2" GWB 5d gwb @ 7" o.c.	510	4,080	0.20	7,800	16.3	9,000	9.4	9,100	6.3	10,600 (2.6)
4A	15/32" Str. I OSB #1 10d com. @ 4" o.c.	510	4,080	0.24	6,300	13.1	7,050	7.3	6,850	4.8	8,300 (2.0)
5A	15/32" Str. I OSB #2 10d com. @ 4" o.c.	510	4,080	0.28	6,150	12.8	7,150	7.4	7,450	5.2	8,750 (2.1)
6A	3/8" Str. I Plywood 8d com. @ 3" o.c.	550	4,400	0.29	7,150	14.9	8,750	9.1	8,800	6.1	10,300 (2.3)
7A	7/16" OSB #3 8d com. @ 3" o.c.	490	3,920	0.23	6,600	13.8	7,450	7.8	6,800	4.7	8,700 (2.2)
8A	15/32" Str. I OSB #1 10d com. Short @ 4" o.c.	510	4,080	0.32	5,300	11.0	6,000	6.3	6,350	4.4	7,700 (1.9)

(a) Tabulated data is average of ± cycles. Displacement (Δ) and shear loads – after fourth cycle of loading.
(b) lb/in. ÷ 1,000 (= kips/in.)

TEST RESULTS AND DISCUSSION

Test results are summarized in Table 5, and a typical load-displacement curve, for Test 1A, is shown in Figure 6. Load-displacement curves for the remaining tests are shown in Appendix A. Table 5 shows shear loads and shear modulus values for wall displacements of 0.48 inch (0.5% of wall height, or H/200), 0.96 inch (1% of wall height, H/100), and 1.44 inch (1.5% of wall height, or H/67), and are based on *cycled* test results, recorded in the fourth cycle of displacement. Evaluation could be based on displacements in other cycles of loading; further comments are provided in the discussion under "Design Analysis." Information on the slip (displacement) of the hold-down connectors during lateral loading of the shear walls is summarized in Table 6.

Results of cyclic load tests on three similarly constructed shear walls, conducted earlier at the University of California - Irvine for California structural engineer Ben Schmid, SE, are summarized for comparison in Appendix B.

In most tests with 10d common nails, the observed failure mode was fastener fatigue, with nails breaking within the lumber framing about 3/8 inch to 1/2 inch below the framing surface. Fastener fatigue failures started near the corners of the panels after near-maximum shear loads were reached (at wall displacements of about 2 inches), and progressed further along the top and bottom and vertical edges of the panels, away from the corners, as the load-displacement cycles continued.

When wall sheathing panels are oriented vertically, with their face grain or strength axis parallel to wall studs, the highest shear forces on fasteners occur near the corners of the panels, which are furthest removed from the centroid of rotation of the panel. The 2-inch displacement level was reached in Cycles 45-51 in the TCCMAR procedure (Protocol A), and in Cycles 25-27 in the ramped loading procedure (Protocol B).

For these tests, a 3/4-inch-thick plywood spacer was placed on the top plate of the wall, and the bottom plate of the wall was supported on a 3-inch-wide steel channel, so that panel sheathing panels were "free" to rotate without restraint (bearing) at their top and bottom corners. This condition duplicates test configurations used by APA when wall bracing and shear wall tests are conducted monotonically in accordance

FIGURE 6

Load-displacement curve for typical cyclic load shear wall test (Test 1A)

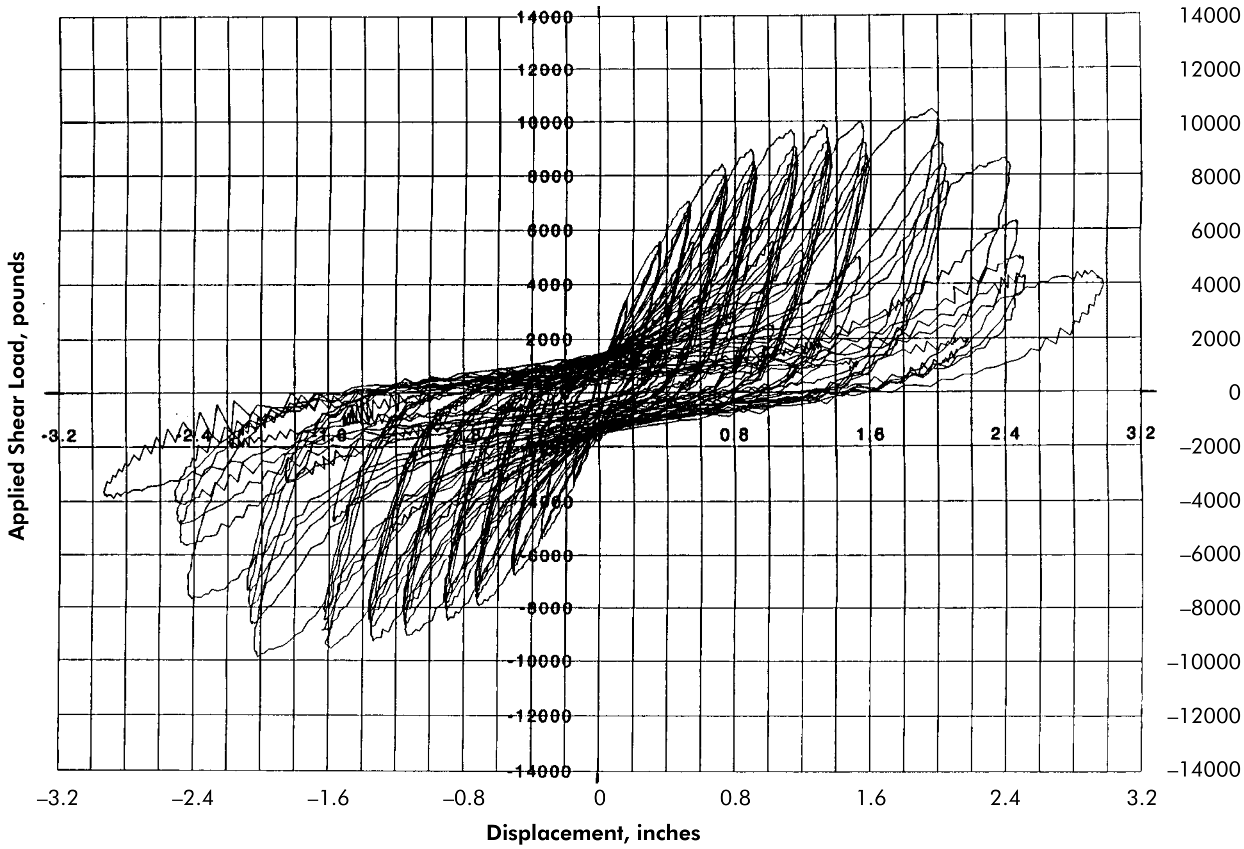


TABLE 6

Hold-down connector slip, Δ_h

Test No.	Shear Wall Design Load, lb[b]	Avg. Δ_h in.[a]	Avg. Δ_h in./1,000 lb	Shear Wall Load, lb[b] (Δ = 0.96 in.)	Avg. Δ_h in.[a]	Avg. Δ_h in./1,000 lb	Shear Wall Load, lb[b] (Δ = 1.44 in.)	Avg. Δ_h in.[a]	Avg. Δ_h in./1,000 lb
1A	4,080	0.020	0.005	8,000	0.049	0.006	8,600	0.063	0.007
2B	4,080	NR	–	8,900	NR	–	9,200	NR	–
3A	4,080	0.008	0.002	9,100	0.033	0.004	9,000	0.045	0.005
4A	4,080	0.006	0.001	7,100	0.017	0.002	7,000	0.030	0.004
5A	4,080	0.028	0.007	7,500	0.064	0.009	7,400	0.070	0.009
6A	4,400	0.013	0.003	8,900	0.041	0.005	8,900	0.052	0.006
7A	3,920	0.008	0.002	7,400	0.025	0.003	6,800	0.040	0.006
8A	4,080	0.026	0.006	6,200	0.047	0.008	6,400	0.057	0.009
		Avg.	0.004		Avg.	0.005		Avg.	0.007

NR = Not Recorded

(a) = Tabulated data is average of ± cycles. Slip (Δ_h) – after cycled loading

(b) = From prior cyclic load shear wall tests on similar test specimens, lateral (shear) load on hold-down connector is approx. 80% of shear wall load (see text) with ASTM E72.[1] Bearing of wall sheathing panels at the top and/or bottom ends can increase shear wall stiffness and strength,[4] since such bearing limits the displacement of the vertical panel edges relative to the framing at the end posts and at the center stud. This improves performance by reducing the displacement and resulting fatigue of the sheathing fasteners along vertical framing near the corners of the panels, and along top and bottom plates at the perimeter of the panel sheathing.

Two of the test specimens (Tests 1A and 2B) duplicated wall construction used in preliminary cyclic load shear wall tests conducted earlier at the University of California - Irvine for California structural engineer Ben Schmid, SE (Appendix B). For Schmid's tests, the steel loading beam on top of the wall was fastened directly to the top plate of the wall. The panel sheathing was permitted to bear on the steel loading beam at the top of the wall, but not on the test jig at the bottom of the wall. This condition restricted panel displacement along vertical panel edges at end posts and the center stud, as the panels rotated under applied lateral shear loading. This detail led to higher maximum strength and load factors (Table B1) than achieved in the series of cyclic load tests conducted for APA. The data from Schmid's tests is indicative of performance when the panel sheathing is partially restrained by bearing at one end of the panels. Even better wall stiffness and strength are anticipated if both the top and bottom ends of wall sheathing panels are permitted to bear on other panels, floor surfaces or ceiling/roof framing.

The cyclic load tests were conducted without axial loading on the walls. Past research in the United Kingdom has shown that the application of axial loading increases the shear load capacity of shear walls, since it reduces uplift on end posts and intermediate studs to which edges of sheathing panels are attached. This minimizes vertical components of shear forces on fasteners, particularly at the corners of panels. Section 6.1 of British Standards Institution Standard BS 5268: Part 6, *Code of Practice for Timber Frame Walls* includes provisions for increasing the allowable design shear load of walls based on the magnitude of axial loading that occurs. Axial loading would be especially effective for increasing shear load capacity of shear or bracing walls when hold-downs are not incorporated for overturning or racking resistance. However, axial loading also can increase shear capacity of walls where hold-downs are located at ends of the wall. In the latter case, axial loading provides for resistance against uplift of framing where hold-downs are omitted at intermediate locations, such as at vertical edges of adjacent sheathing panels, and at edges of wall openings such as windows or doors. The influence of axial loading on the performance of cyclically loaded shear walls deserves further study.

Comparison of Calculated vs. Measured Shear Wall Displacement

Displacement of shear walls can be calculated in accordance with the following formula in Section 23.223 of 1997 Uniform Building Code Standard 23-2, and associated tables therein:

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + \frac{h}{b}d_a$$

where:

Δ = calculated shear wall displacement, in.

v = maximum shear load due to design loads at the top of the wall, lb per ft

h = shear wall height, ft

E = modulus of elasticity of vertical boundary element (end post or studs at shear wall boundary), lb per sq. in.

A = area of cross section of vertical boundary element, sq. in.

b = shear wall length, ft

G = modulus of rigidity of wall sheathing, lb per sq. in.

t = effective shear thickness of wall sheathing, in. (Note: For plywood wall sheathing, values for G and t are tabulated in APA's Plywood Design Specification, Tables 1-3. For oriented strand board wall sheathing, use $G_v t_v$ values in APA Technical Note N375, Tables 2.4 and 2.4.1. Values for plywood wall sheathing also are tabulated in Technical Note N375.)

e_n = nail displacement, in. (1997 UBC Standard 23-2, Table 23-2-K)

d_a = shear wall hold-down displacement including fastener slip, in. (from tests or manufacturer's information)

For this report, shear wall displacements were calculated at two applied load levels (allowable design shear load, and 1.4 times allowable load) for the constructions tested, and compared with shear wall displacements measured in the tests.

One component of shear wall displacement is sheathing nail slip (displacement). Since sheathing nail slip values in Table 23-2-K are based on monotonic (one direction) fastener lateral load tests, shear wall displacements measured in the initial load-displacement application (e.g., prior to subsequent load-displacement cycling) were used for this comparison.

Table 7 summarizes calculated vs. measured shear wall displacement for the two load levels. Agreement between these values generally was within 1/8 in., and 13 of 22 values were within 1/16 in. Only 4 of 22 values differed more than 1/8 in. (the maximum difference was 0.15 in.). In two tests (Test Nos. 5A and 8A), the applied shear load of 1.4 times the allowable load, was approximately equal to or greater than the Yield Limit State (see discussion on page 24), yet the calculated and measured displacements were in close agreement. In the remaining nine tests, the applied shear load of 1.4 times the allowable load, was less than the Yield Limit State. These comparisons support the validity of the shear wall displacement formulas, for shear loads up to at least 40% greater than the allowable load.

TABLE 7

Comparison of calculated vs. measured shear wall displacement

Test No.	Applied Shear Load		Δ Calc., in.	Avg. Δ Meas., in.[c]	Difference, in.
	lb/ft[a]	lb[b]			
1A	510 714	4,080 5,712	0.21 0.39	0.22 0.38	+0.01 –0.01
2B	510 714	4,080 5,712	0.21 0.39	0.15 0.25	–0.06 –0.14
3A[d]	510 714	4,080 5,712	0.21 0.39	0.14 0.24	–0.07 –0.15
4A	510 714	4,080 5,712	0.18 0.34	0.16 0.27	–0.02 –0.07
5A	510 714	4,080 5,712[f]	0.18 0.34	0.20 0.34	+0.02 0
6A	550 770	4,400 6,160	0.25 0.42	0.23 0.37	–0.02 –0.05
7A	490 686	3,920 5,488	0.15 0.26	0.15 0.27	0 +0.01
8A	510 714	4,080 5,712[g]	0.18 0.34	0.22 0.45	+0.04 +0.11
BLS.1A[e]	510 714	4,080 5,712	0.21 0.39	0.31 0.48	+0.10 +0.09
BLS.1B[e]	510 714	4,080 5,712	0.21 0.39	0.34 0.54	+0.13 +0.15
BLS.2A[e]	550 770	4,400 6,160	0.21 0.37	0.22 0.35	+0.01 –0.02

[a]First value listed for each test is code-recognized allowable design shear load for tested construction (1997 UBC Table 23–II–I–1). Second value is 1.4 times allowable load.

[b]Corresponding applied shear load for 8 ft. long wall segment as tested

[c]Displacement measured during initial load-displacement application; average of +/– cycles

[d]Gypsum wallboard installed on opposite side of wall

[e]See Appendix B for details and discussion

[f]Approximate Yield Limit State (YLS) observed in test (F_{YLS} = 5,700 lb @ Δ_{YLS} = 0.35 in.). See text for discussion.

[g]Exceeds Yield Limit State observed in test (F_{YLS} = 5,100 lb @ Δ_{YLS} = 0.36 in.).

Load-Displacement Relationships

Allowable design shear loads for the tested constructions are based on the 1997 Uniform Building Code and NER-108. At the allowable design shear load, measured wall displacement averaged 0.21 inch (range of 0.14 to 0.34 inch) for eleven test specimens (Table 7). Also, load-displacement curves were nearly linear, and hysteresis loops were repeatable for all test specimens after four cycles of repeated loading in this range (Figure 6 and Appendix A and B), demonstrating that the shear load capacity of the walls was undiminished. Within the allowable design shear load range, wall displacements were consistent with observed results in monotonic tests, in which vertical steel tiedown rods are used to resist overturning forces, when walls are tested in accordance with ASTM E72.[1,4]

At a wall displacement of 0.48 inch, equivalent to 0.5% of height (H/200) for the 8-foot wall height used in these tests, the load-displacement curves and hysteresis loops continued to be nearly linear and repeatable for all test specimens (Figure 6 and Appendix A). After three cycles of repeated loading to this displacement, the shear load capacity of the walls diminished only slightly. The shear loads (pounds per foot) at 0.48-inch displacement averaged 1.7 times the allowable design shear load listed in the code, for Tests 1A through 7A (range 1.5 to 1.9), and 1.9 for Test 3A with gypsum wallboard in combination with plywood sheathing. For Test 8A with pneumatically driven fasteners, the value was 1.3 times the allowable design shear load. Overall, the maximum shear strength of the walls averaged about 1.4 times the shear load at 0.48-inch displacement. In general, these comparisons confirm the conservative nature of the allowable design shear load.

The next wall displacement of interest is 0.96 inch, or 1% of height (H/100). At this displacement, the load-displacement curves are non-linear, but the hysteresis loops continued to be repeatable for all test specimens. However, the maximum *cycled* shear strength of the walls "flattened out" at nearly the same shear load reached at 0.96-inch displacement, when subjected to repeated cycles of loading up to a displacement of 1.6 inches. This "expected maximum shear strength" provides a basis for evaluating the performance of shear walls subjected to repeated cyclic loading, in terms of stiffness and strength.

Examination of the load-displacement curves for all test specimens (Appendix A) revealed that the hysteresis loops continued to be repeatable up to displacement in the range of 1.4-1.6 inches, at which point the maximum shear strength (e.g., strength limit state) was reached. Beyond this displacement, yielding and resulting fatigue failures of fasteners resulted in decreasing shear load capacity at each repeat cycle of displacement. The hysteresis loops were not repeatable (at least for three replications of loading), and the shear load was not stabilized upon repeated loadings at each displacement increment. It is uncertain whether additional replications of loading would stabilize the hysteresis loops, or instead cause earlier fatigue failures of fasteners due to the large displacements which occur at these stages.

The 1997 Uniform Building Code (Sections 1630.9.1 and 1630.9.2) provides a basis for establishing a maximum value for *inelastic* displacement of structural systems such as shear walls. Wall displacement is derived from calculated static force displacement, which is increased by factors related to overstrength and ductility capacity of the assembly as follows:

$$\Delta_M = 0.7\, R\, \Delta_S$$

where:

Δ_M = Maximum Inelastic Response Displacement that occurs when the structure is subjected to design ground motion.

R = Numerical coefficient representative of overstrength and ductility of lateral-force-resisting system (Table 16-N of the 1997 Uniform Building Code).

Δ_S = Design Level Response Displacement that occurs when the structure is subjected to design seismic forces; based on static, elastic analysis of the structural system.

The R factor is 5.5 for bearing wall systems with light-framed walls sheathed with wood structural panels, when building heights are three stories or less; or 4.5 for all other light-framed wall systems. Accordingly, maximum inelastic displacement calculated by the above formula is 3.15 to 3.85 times the wall displacement at allowable design load (static force). In the summary of test results in Table 7, calculated wall displacement at allowable design load averaged 0.20 in. (measured displacement averaged 0.21 in.). Accordingly, the maximum inelastic wall displacement would be 0.63 in. to 0.77 in. For comparison, shear wall displacement at Yield Limit State in the eleven tests was in the range of 0.34 in. to 0.76 in. (see Table 8).

Section 1630.10.2 of the 1997 Uniform Building Code limits maximum story drift (wall displacement) to 2% to 2.5% of height. For an 8-foot wall height as used in these tests, maximum story drift would be limited to 1.92 in. (2%) or 2.40 in. (2.5%). All walls described in this report attained maximum shear capacity (Strength Limit State) before reaching 2.5% story drift (see Table 8).

However, in discussions of the Structural Engineers Association of Southern California Ad Hoc Committee for Testing Standards for Structural Systems and Components, a maximum story drift limit of 1% of height (H/100) was suggested to limit structural damage.

For brittle exterior finishes such as stucco or brick veneer, or interior finishes such as gypsum wallboard, it is recognized that wall displacement, the magnitude of which is undetermined at present, may result in minor cosmetic cracking of the finish around window and door openings in shear wall segments, during an earthquake. When brittle wall finishes are used, a reduced story drift limit, such as 0.5% of height (H/200), may be necessary to limit damage and repair costs.

Shear Wall Load Capacity – Cyclic vs. Monotonic Tests

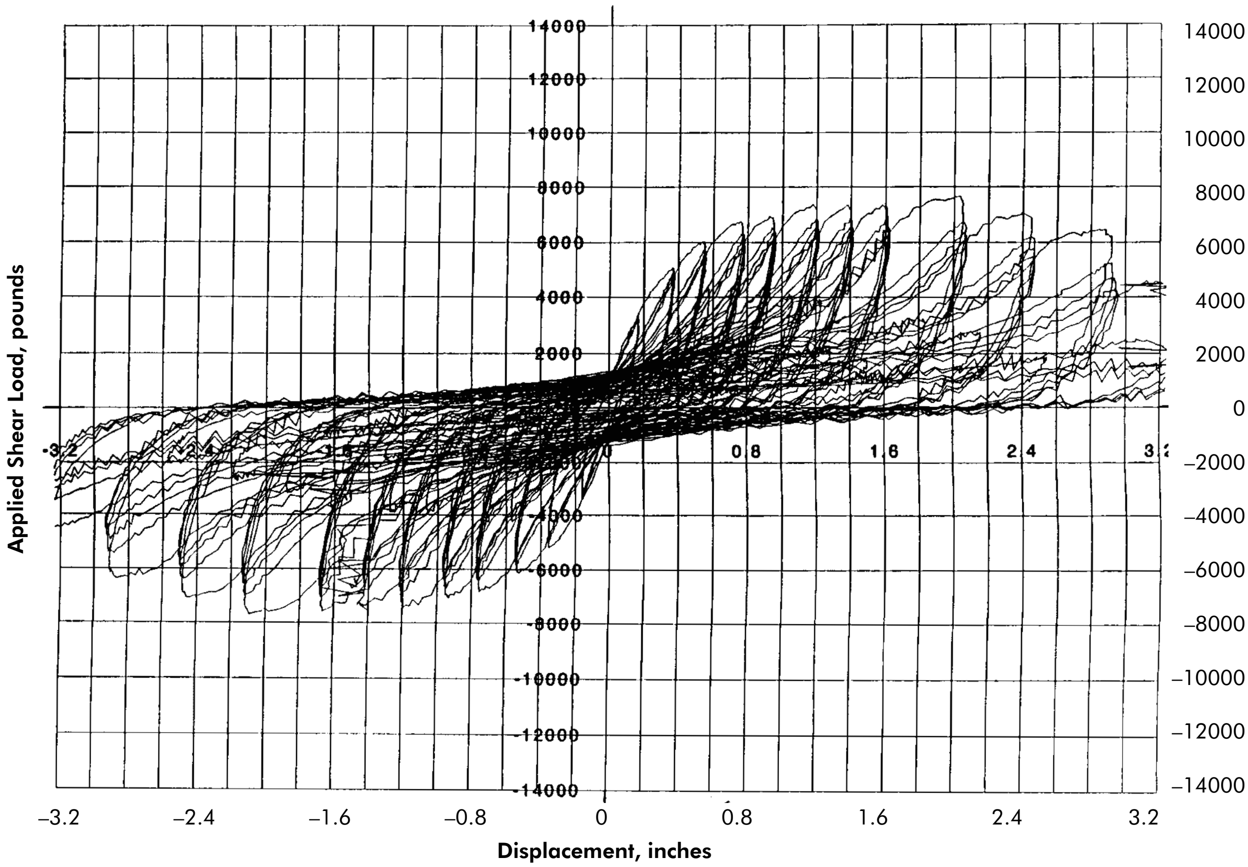
The shear strength of cyclically loaded walls, in these tests, was lower than typically obtained in monotonic tests.[5] The load factor (average maximum shear load/allowable design shear load) averaged 2.4 (range 2.0 to 2.7) for seven tests with plywood and OSB sheathing fastened with hand-driven 8d or 10d common nails. A load factor such as 2.5 may be appropriate when performance of seismic-resisting assemblies is evaluated on the basis of cyclic (reversed) load tests.

Since matching monotonic tests were not conducted on these shear wall constructions, it is not possible to evaluate the potential reduction in shear stiffness or maximum shear capacity that might occur under repeated cyclic (reversed) loading. However, when the maximum *cycled* shear loads (at 1.44 inches of displacement) are compared with the average maximum shear loads of the walls (Table 5), these preliminary results indicate that the shear strength for repeated cyclic loading averages about 18% lower than the maximum shear strength reached during these cyclic load tests.

In Yasumura's tests,[6] the construction details used in fabricating test specimens are unknown. However, Yasumura reported

FIGURE 7

Load-displacement curve for typical cyclic load shear wall test with pneumatically driven 10d common "short" diaphragm nails (Test 8A)



only a 10% reduction in shear wall stiffness and strength, when results of cyclic tests were compared to monotonic tests.

In 1994, the City of Los Angeles Department of Building and Safety implemented an emergency code change calling for 25% reduction in code-recognized allowable design shear loads for shear walls, since these values were based on monotonic tests. This emergency code change was later finalized in 1995. Based on the preliminary test results in this series, this interim measure appears to be conservative, for *seismic* design of buildings, until more comprehensive monotonic and cyclic load testing of matched test specimens is conducted to confirm or modify this adjustment.

The City of Los Angeles also implemented a code change to limit the allowable design shear load to 200 pounds per foot, for shear walls constructed with 3-ply plywood sheathing. This limitation applies to plywood of 5/8-inch thickness or less, when manufactured in accordance with provisions of U.S. Product Standard PS 1-95. Test 6A, with 3/8-inch Structural I plywood (3-ply) had an allowable design shear load of 550 pounds per foot. The load factor for Test 6A was 2.3, which was consistent with results of other cyclic load shear wall tests with 15/32-inch 5-ply plywood. Thus, the limitation on allowable design shear load for 3-ply plywood sheathing does not appear to be warranted, based on the results of this preliminary test. It was observed that a greater proportion of 8d common nails, used to fasten the 3/8-inch wall sheathing in Test 6A, pulled through the sheathing and remained in the framing. This contrasts with other tests with 15/32-inch sheathing fastened with 10d common nails, where fatigue failure of the nails was typical.

Plywood vs. OSB Wall Sheathing

The load factor for four walls tested with plywood sheathing averaged 2.5 (range 2.3 to 2.7). For three walls tested with OSB sheathing (excluding Test 8A with pneumatically driven nails), wall displacement within the design shear load range was consistent with values obtained in tests with plywood sheathing. However, the load factor at maximum shear load averaged 2.1 (range 2.0 to 2.2), or about 15% lower than plywood (2.1/2.5 = 0.84). There seemed to be earlier and more extensive fastener fatigue failures in the cyclic load shear wall tests with OSB than with plywood. This may be due to higher density and perhaps higher dowel bearing capacity for OSB than plywood. It is theorized that denser OSB sheathing deforms less along the nail shank in contact with the sheathing, creating a "fixed end" condition on the fastener. As a result, higher internal bending stresses in the nail shank may develop at locations below the surface of the wood framing, thus causing earlier fatigue failures of the fasteners under cyclic loading. Further study of this possibility is needed, however.

Contribution of Gypsum Wallboard

Two tests (Tests 1A and 3A) were identical except that 1/2-inch gypsum wallboard was installed on one side of the wall in Test 3A. The results were as anticipated; the gypsum wallboard added to early stiffness but not to shear strength of the wall. This would seem to be appropriate for reversed cyclic loading as might occur during an earthquake. For wind loading, where cyclic (reversed) loading is unlikely, the resistance of the two materials appears to be additive, as shown in other monotonic tests.[8]

Pneumatically Driven Nails

For Test 8A, 15/32-inch Structural I OSB sheathing was fastened with pneumatically driven 10d common short (diaphragm) nails. The nails provided slightly more than 1-5/8 in. penetration (about 11 diameters) into the framing. The load factor for this wall was 1.9, with a maximum shear load of about 5% less than a matching wall constructed with full-length, hand driven nails (Test 4A, load factor 2.0). A greater proportion of the shorter pneumatically driven nails withdrew from the framing, but only a few fractured in fatigue as was typical of hand driven nails. The load-displacement curve in Figure 7 shows that the wall displacement continued to increase up to 2 inches without a significant reduction in cycled shear strength. Thus, the energy dissipation of the shear wall with sheathing fastened with shorter nails was greater than in Test 4A (compare Figures A4 vs. A8 in Appendix A). At the possible expense of somewhat lower shear strength, a potential increase in energy dissipation might be an advantage under certain situations. However, consideration also would have to be given to the amount of wall displacement that can be tolerated without resulting in structural damage that would be uneconomical to repair. Further study is needed to determine whether minimum fastener penetration permitted by the NDS[3] has an effect on lateral load capacity of fasteners when loaded cyclically.

TCCMAR vs. "Ramped" Cyclic Loading Procedure

Seven of the eight tests (i.e., all tests except 2B) used the TCCMAR sequential phased displacement test procedure. This procedure has been used by SEAOSC for development of a cyclic load test method for shear walls, and also for a proposed ASTM fastener cyclic load test procedure. Incremental increases in load/displacement during cyclic loading are followed by a "decay" cycle in which cyclic loads are applied to reduced displacement levels before increasing load/displacement to the next higher increment.

For Test 2B, a "ramped" cyclic loading procedure was used, in which incremental increases in load/displacement are repeated for three cycles at each increment, before increasing load/displacement to the next higher increment; no decay cycle is used.[7] The results of Tests 1A (TCCMAR procedure) and 2B (ramped procedure), with matched test specimens, were in good agreement (compare Figures A1 vs. A2 in Appendix A). Thus, the ramped procedure appears to offer an alternate, simpler load/displacement procedure for conducting cyclic loading tests. However, some researchers or structural engineers may need complete load-displacement history information from the TCCMAR procedure, to permit refined dynamic or finite element analysis of buildings for earthquake analysis. Further, the TCCMAR procedure has established precedent for cyclic load testing of assemblies and fasteners. Since the results obtained by the two methods were similar, there appears to be no substantive reason at this time to suggest consideration of any cyclic load procedure other than the TCCMAR procedure.

Shear Wall Hold-Down Slip

As noted previously under "Hold-down Connectors," shear wall hold-down slip should be minimized. This minimizes displacement of the shear wall, and overstressing of fasteners attaching sheathing to framing, especially in the critical areas near the corners of the sheathing panels. In these tests, a special welded steel plate hold-down connector (Figure 2) was used in conjunction with shear plates, to minimize eccentricity of the hold-down bolt and distribute lateral shear forces from the bolts into the end posts with minimum slip.

Cyclic load shear wall tests conducted for Schmid indicated that the measured uplift force on the hold-down bolts was about 20% less than ordinarily calculated (corresponding hold-down slip was not recorded) – see Figure B4 in Appendix B. This is believed to be due to the additional overturning resistance provided by the fasteners which attach wood structural panel sheathing to the top and bottom plates of the wall; such resistance is not normally taken into account by the shear wall analysis procedure. The difference between the calculated and actual hold-down forces depends on the length of the shear wall, the distance between hold-down bolts, and the shear capacity of the wall as determined by the sheathing type, thickness and grade, framing species, and sheathing fastener schedule.

In the cyclic load shear wall tests conducted for APA, uplift force on the hold-down bolts was not recorded, but hold-down slip, relative to the end posts, was measured (data for Test 2B was not recorded due to a malfunction of the data acquisition system). The hold-down slip is shown in Table 6. At the allowable design shear load for the walls, hold-down slip averaged only 0.016 inch; and at the expected maximum shear strength of the walls (at 0.96-inch displacement), only 0.039 inch (range 0.017 to 0.064 inch) after load cycling.

Corresponding hold-down slip, based on applied lateral load (approximately equivalent to hold-down uplift force), averaged 0.004 in. per 1,000 lb at shear wall design load, and slightly greater at higher loads (see Table 6).

DESIGN AND CONSTRUCTION CONSIDERATIONS

When designing shear walls, a number of factors should be considered in addition to the typical specifications based on sheathing thickness and grade, framing species and size, and fastener type and schedule which determine the allowable design shear load for the wall.

If multiple studs are used in lieu of solid one-piece wood framing at panel edges, the studs must be fastened together to transfer the shear forces that the shear wall is designed for. This may call for closely spaced face nailing of studs with 10d or 16d common nails, or perhaps bolts or lag screws. When multiple studs are used, sheathing fastening should be apportioned between these members.

When walls are subjected to racking shear forces, the end posts act alternately as tension and compression "chords" of the vertical diaphragm (shear wall). When the end posts are in compression, the bearing capacity of the end posts on the top and bottom plates of the wall should be checked. This design check should consider not only axial design load from tributary dead and live loads on the structure, but also the additive component of chord forces at the maximum shear strength for the shear wall. The maximum shear strength could be as much as 2.5 or more times the allowable design shear load for the assembly. If the bearing stress is exceeded, larger or multiple end posts are needed to minimize or prevent crushing of the top and bottom plates at the expected maximum shear strength. If such crushing occurs, it can have an adverse effect on the displacement or expected maximum shear strength of the assembly.

In situations where the shear wall is supported on a concrete slab floor, it may be possible to detail the shear wall so that the bottom end of end posts bears directly on the concrete instead of on wood, to utilize higher lumber parallel-to-grain compression stresses and increase the bearing capacity of the end posts. However, for wood-framed walls, the upper end of end posts typically bears on the wood top plate of the wall. In this situation, the bearing capacity of the end posts would be governed by the allowable stress for compression perpendicular to grain for the end post bearing on the top plate.

The net section of the end post at the hold-down connector bolt holes (if applicable) should be checked to insure that the end posts have sufficient tension capacity to resist the uplift loads imposed at this location. If shear plate connectors (Part X of NDS[3]) are used in conjunction with bolts for attaching hold-down connectors to the end posts, the net section of the end post is further reduced by the dapped slot cut into the end post to accommodate the shear plate connector.

In situations where the hold-down bolt is embedded near the edge of a concrete slab floor or foundation wall, the amount of embedment and uplift capacity of the hold-down bolt in the concrete should be checked in accordance with building code design provisions. If the "shear cone" from the bolt intersects the side of the slab or foundation wall, the uplift resistance of the bolt is reduced. The mass of the tributary area of the slab, foundation, footing and overlying earth backfill (if applicable) must be sufficient to resist the uplift load acting on the hold-down bolts.

Shear wall displacement should be checked for conformance with model building code provisions for maximum inelastic displacement and story drift. Calculation of shear wall displacement is described on pages 17-18 of this report. In the future, it may be possible to use a shear modulus (G), as described in ASTM E564[2] and derived from cyclic load tests, for calculating shear wall displacement. For further information, see the following discussion under "Design Analysis."

During fabrication, hold-down deformation and fastener slip should be minimized, to avoid additional shear forces on sheathing fasteners, and to minimize the potential for splitting the horizontal top and bottom plates when sheathing panels rotate or uplift due to racking (shear) forces acting on the wall. When bolts are used to attach hold-down connectors to end posts, the bolt holes should be carefully located and drilled, preferably with a template and drill guide. A maximum of 1/16-inch oversize hole is specified by the NDS.[3] Installation of shear plate connectors for hold-down bolts in end posts, as mentioned previously, will reduce slip of these connections when subjected to high lateral (uplift) forces. Cyclic load shear wall tests (in progress by others) indicate that hold-down connectors that are fastened to end posts with nailed or lag screw connections can minimize wall displacement due to hold-down slip.

These tests also emphasized the need for carefully locating and installing hold-down bolts in foundations or slabs, or at floor intersections. Structural or architectural drawings should include details showing the type of hold-downs required and dimensions locating where hold-down connectors are to be installed. For on-site construction, jigs or templates are suggested for accurately locating and drilling holes for hold-down bolts, and for bolts to connect the bottom plate of shear walls to slabs and foundation walls.

Hold-downs may extend through or across floor intersections. Dry lumber, with a moisture content of 19% or less, should be specified for floor framing members located beneath shear walls, to minimize lumber shrinkage which could affect the effectiveness (e.g., displacement) of the hold-down. For wood in protected applications, 9% to 12% moisture content is the usual in-service range. Dry wood framing products for floor rim joists, such as APA Rim Board,® APA PRI® wood I-joists and blocking panels, or structural composite lumber, are now available from several engineered wood product manufacturers in the United States and Canada.

DESIGN ANALYSIS

Load-displacement curves for all test specimens have similar shape characteristics (see Appendix A). A potential method of analyzing shear wall performance in terms of stiffness and shear strength combines features of a design approach suggested by John Kariotis, SE (Kariotis & Associates, Sierra Madre, CA). It is based on the static force procedure for seismic design of structures in accordance with Section 1630.2 of the 1997 Uniform Building Code.

Fundamental to this method is the determination of a shear modulus (G') for the wall. The shear modulus for shear walls also is evaluated in ASTM E564.[2] In that standard, the shear modulus is determined at one shear load value (33% of the maximum shear strength of the wall). The shear modulus can be derived from the following formula:

$$\Delta = (F/G')(H/L) \qquad [1]$$

where Δ = Displacement, in.

F = Applied load, lb

G' = Shear modulus, lb/in.

H = Height of wall, ft

L = Length (width) of wall, ft

Re-arranging terms, $G' = (F/\Delta)(H/L)$, which is the slope of the load-displacement curve or wall stiffness, reported in terms of pounds per inch (or kips per inch, where 1 kip = 1,000 pounds) at any desired shear load. An average (F/Δ) value for *cyclic* stiffness can be determined for any desired shear load or displacement increment by determining the slope of a line drawn between the maximum positive and negative values of load or displacement on initial loading or stabilized hysteresis loops. Since the load-displacement curve is not linear, except at low shear loads, the wall stiffness changes continuously. Therefore, to accurately characterize the performance of the shear wall, G' must be determined for key wall displacements of interest.

From the hysteresis loops recorded during the cyclic load tests, an envelope or "backbone" curve can be constructed through the maximum shear load recorded at each displacement increment (Figure 8 for Test 1A). This curve can be used to check the initial shear stiffness and the maximum shear strength of the wall; the latter value also can be used to check the strength of the hold-down connections for the wall. Similarly, a "*cycled*" backbone curve can be constructed through the cycled shear load recorded for the stabilized hysteresis loops at each displacement increment (in the fourth cycle or any other desired cycle after the initial cycle), as shown in Figure 8. The cycled backbone curve represents the shear stiffness and expected maximum shear strength of the shear wall after having been subjected to repeated cyclic (reversed) displacements such as typically occur during an earthquake. It is emphasized that the many repeated cycles of load-displacement in the TCCMAR (SEAOSC) test method, especially beyond the allowable design load and Yield Limit State (FME), provide a very conservative basis for evaluating shear wall stiffness and maximum shear strength. In effect, this protocol evaluates a building's shear wall performance when subjected to numerous cycles of reversed loading, not only for "design level" but also higher magnitude earthquakes over the life of the building. An accurate representation of shear wall performance requires development of backbone curves for both positive and negative displacements, and for both initial and cycled backbone curves, using the average values from these curves for analysis.

The shape of the backbone curve can be approximated by bilinear segments, such as shown in Figure 9. Key wall displacements occur at Yield Limit State (YLS) and Strength Limit State (SLS). The resulting bilinear load-displacement curve represents a reasonable approximation of shear wall displacement and shear strength from initial loading to YLS, and from YLS to SLS.

The following terms have been proposed by SEAOSC to describe various characteristics of the bilinear load-displacement curve, which are important in evaluating design of shear walls.[9]

Yield Limit State (YLS): The point in the force-displacement relationship where the difference in forces between the first and fourth cycle, at the same displacement, does not exceed 5%.

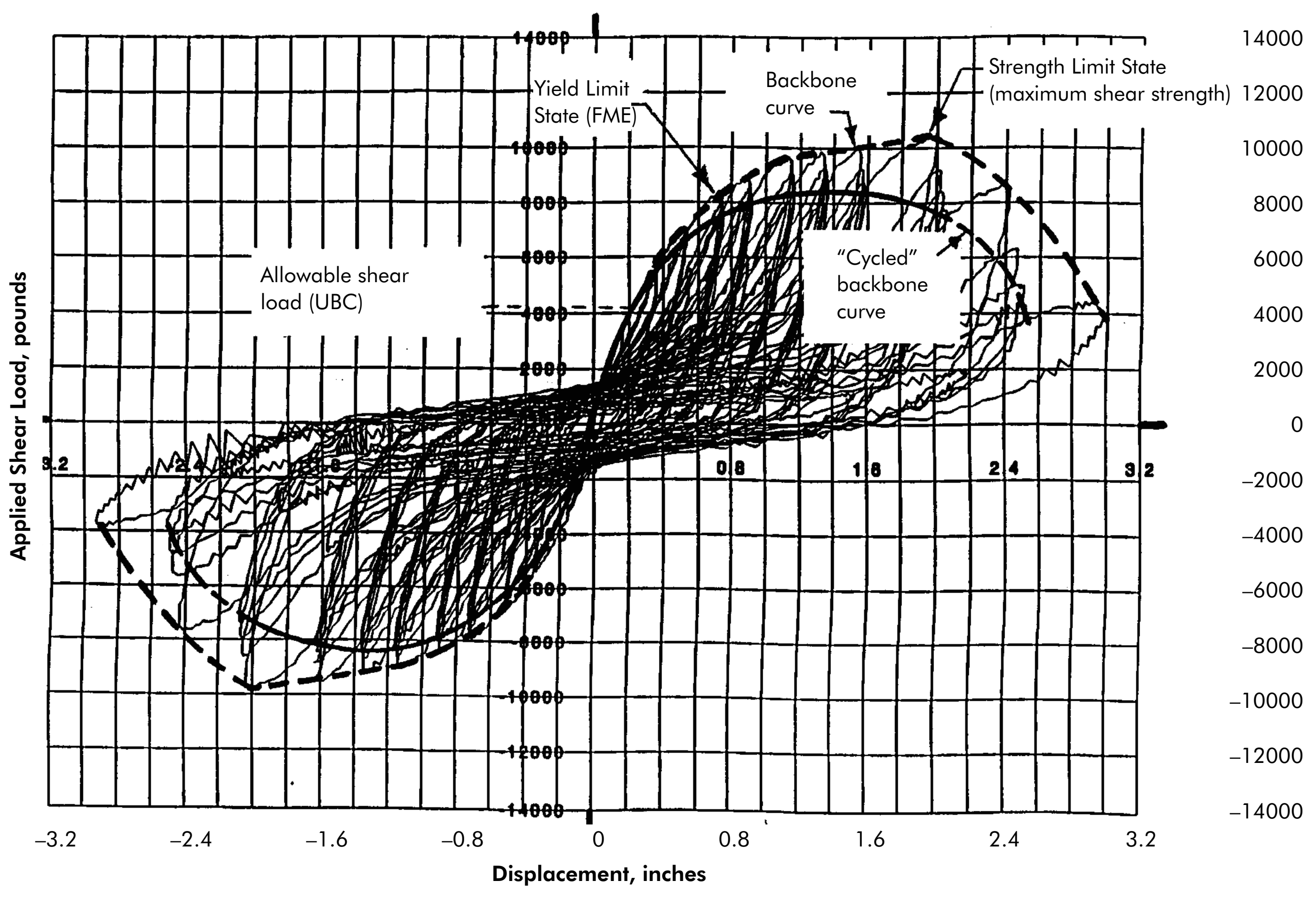
Strength Limit State (SLS): The point in the force-displacement relationship corresponding to maximum displacement for the peak force attained by the element or system.

Δ_{yls}, F_{yls}: Displacement (Δ_{yls}) and shear load (F_{yls}) at Yield Limit State (FME).

Δ_{sls}, F_{sls}: Displacement (Δ_{sls}) and shear load (F_{sls}) at Strength Limit State (maximum shear capacity).

FIGURE 8

"Backbone curve" and "cycled backbone curve" for cyclic load-displacement shear wall (Test 1A)



Overstrength Factor (R_o): $R_o = F_{sls}/F_{yls}$

Ductility Factor (R_d)[10,11]: $R_d = \mu = \Delta_{sls}/\Delta_{yls}$ (for $T>T_s$); or $(2\mu - 1)^{0.5}$ (for $T<T_s$)

R Factor: Numerical coefficient representative of the inherent overstrength and global ductility capacity of the lateral-force-resisting system (e.g., shear wall); per Section 1628 of the 1997 Uniform Building Code.

Data for these factors, calculated from eight tests conducted for APA and three tests conducted for Schmid (Appendix B), are summarized in Tables 8 and 9. Data for effective stiffnesses ($K_e = F/\Delta$) for the eleven walls also are shown in Table 9, for information. The effective stiffnesses are applicable to the tested walls which had an aspect (height/length) ratio of 1.0.

The following procedure has been suggested for determining the required length of shear wall(s), based on shear strength and displacement, in accordance with the Static Force Procedure described in Section 1630.2 of the 1997 Uniform Building Code (UBC). In the following discussion, all references to the UBC are based on the 1997 edition of the code. Total design base shear is determined from Formula (30-4) in Section 1630.2.1 of the UBC:

$V = (C_v/T)(I)(W) / R$ [2]

$= (2.5C_a)(I)(W) / R$; maximum V when $C_v/T = 2.5\ C_a$; or [2a]

$= (3.0C_a)\ (W) / R$ for simplified structures (for example, single-family dwellings) per Section 1630.2.3 of the UBC. [2b]

where:

V = Total design base shear.

C_v, C_a = Seismic Coefficients based on Seismic Zone Factor Z (= 0.075 to 0.4), Soil Profile Type S, and Near-Source Factor N (= 1.0 to 2.0).

I = Building importance factor (= 1.0 for single-family dwellings).

W = Total seismic dead load of structure, plus other applicable loads per Section 1630.1.1 of the UBC.

R = Coefficient representative of overstrength and ductility of structure. See discussion on page 26 and Table 16-N of the UBC for code-assigned values (= 4.5 to 5.5 for buildings with shear panels on light-frame walls). For additional background information, see References 10 and 11.

FIGURE 9

Bilinear load-displacement relationship for shear wall Test 1A

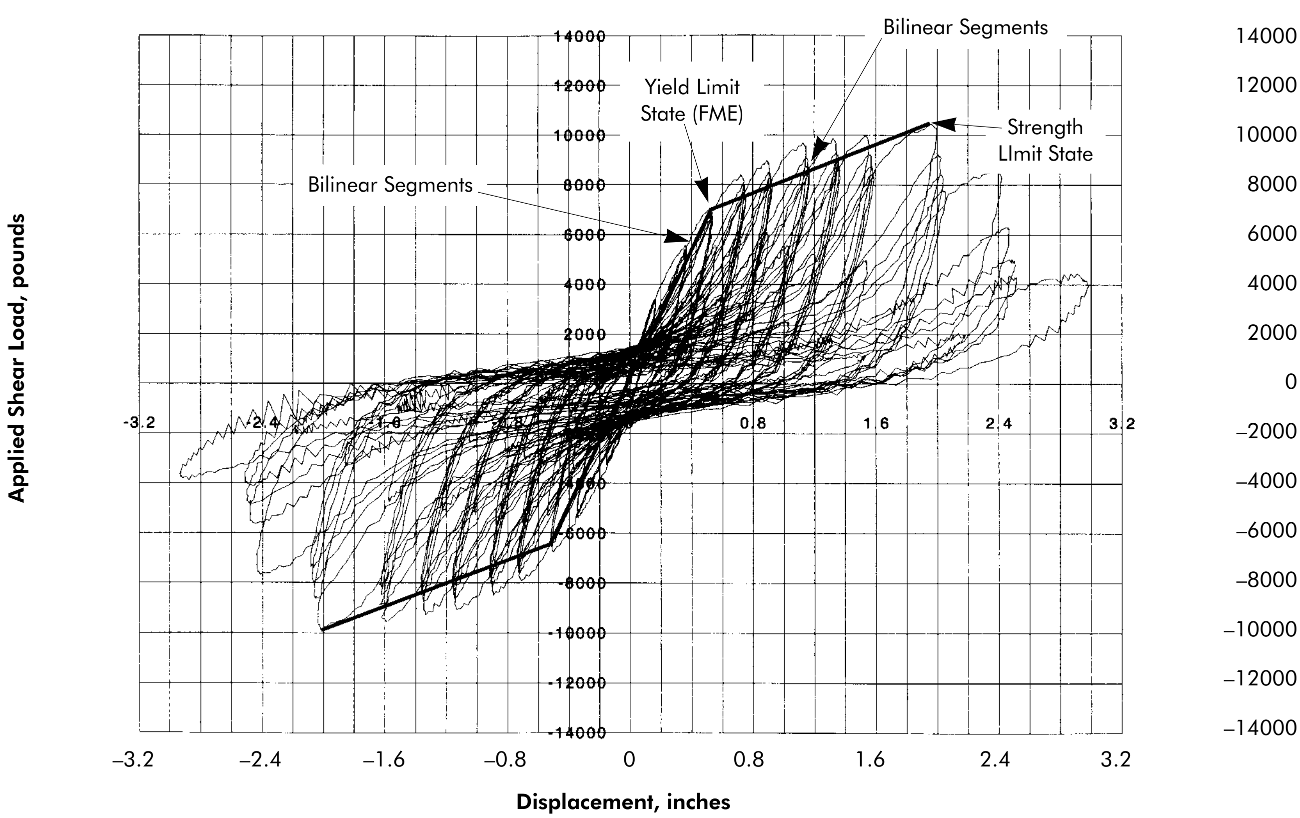


TABLE 8

Calculated R factors (R_o-overstrength and R_d-ductility) from wood structural panel shear wall tests[(a)]

Test No.	F_{yls} (lb)	Δ_{yls} (in.)	F_{sls} (lb)	Δ_{sls} (in.)	$R_o = \frac{Fsls}{Fyls}$	$R_{d-L} = \mu = \frac{\Delta sls}{\Delta yls}$	$R_{d-S} = (2\mu-1)^{0.5}$
1A	6,950	0.51	10,150	2.00	1.5	3.9	2.6
2B	7,900	0.47	10,900	1.99	1.4	4.2	2.7
3A	8,250	0.47	10,600	1.74	1.3	3.7	2.5
4A	6,200	0.34	8,300	1.16	1.3	3.4	2.4
5A	5,700	0.35	8,750	1.82	1.5	5.2	3.1
6A	7,700	0.51	10,300	2.02	1.3	4.0	2.6
7A	6,150	0.35	8,700	1.16	1.4	3.3	2.4
8A	5,100	0.36	7,700	2.06	1.5	5.7	3.2
BLS.1A	6,050	0.55	11,100	2.33	1.8	4.2	2.7
BLS.1B	7,150	0.76	10,900	2.32	1.5	3.1	2.3
BLS.2A	8,550	0.62	12,400	1.80	1.5	2.9	2.2

[(a)]Based on average bilinear load-displacement relationship determined from Figures A1-A8 and B1-B3. For description of wall test specimens, see Tables 1 and B1.

TABLE 9

Summary of unit shear resistance at yield limit state (v_{yls}), 1% story drift ($v_{0.01H}$) and strength limit state (v_{sls}), and corresponding effective stiffnesses (K_e=F/Δ) from wood structural panel shear wall tests[(a)]

	Yield Limit State (yls)		1% Story Drift (H/100)		Strength Limit State (sls)	
Test No.	v_{yls} (Kip/ft)	K_{e-yls} (Kip/in.)	$v_{0.01H}$ (Kip/ft)	$K_{e-0.01H}$ (Kip/in.)	v_{sls} (Kip/ft)	K_{e-sls} (Kip/in.)
1A	0.87	13.6	0.99	8.3	1.27	5.1
2B	0.99	16.8	1.11	9.3	1.36	5.5
3A	1.03	17.6	1.15	9.6	1.33	6.1
4A	0.78	18.2	0.98	8.2	1.04	7.2
5A	0.71	16.3	0.88	7.3	1.09	4.8
6A	0.96	15.1	1.06	8.9	1.29	5.1
7A	0.77	17.6	1.01	8.4	1.09	7.5
8A	0.64	14.2	0.75	6.3	0.96	3.7
BLS.1A	0.76	11.0	0.91	7.6	1.39	4.8
BLS.1B	0.89	9.4	0.96	8.0	1.36	4.7
BLS.2A	1.07	13.8	1.22	10.2	1.55	6.9

[(a)]Based on average bilinear load-displacement relationship determined from Figures A1-A8 and B1-B3, and Tables 5 and B1. For description of wall test specimens, see Tables 1 and B1.

T = Elastic fundamental period of vibration (seconds) of the structure.

To determine the required length of shear wall(s) based on shear **strength**, the first step is to calculate a fundamental period of vibration (T) for the structure, which can be estimated from Method A and Formula (30-8) in Section 1630.2.2.1 of the UBC:

$T = (0.020)(h_n)^{0.75}$, where h_n is the height (ft) above the base to level n. [3]

Next, a spectral response modification factor for peak ground acceleration (corresponding to C_v/T) can be determined per Figure 16-3 of the UBC, based on the estimated elastic fundamental period of vibration.[e]

The design shear load for the wall (lb or kips) is then calculated from Formula [2], by multiplying the assigned seismic load (lb or kips) by the spectral response modification factor, and dividing by R.

Finally, the length of shear wall(s) (ft) required for adequate **strength** is determined as follows. (a) Divide the design shear load (V) by the unit shear load at Yield Limit State (v_{yls}). (b) Divide the design shear load (V) by the product of the Resistance Factor ϕ (sometimes referred to as the Capacity Reduction Factor) for Load and Resistance Factor (LRFD) design, and the unit shear load at Strength Limit State. For shear walls and connections, $\phi = 0.65$ in accordance with ASCE 16-95.[11] The greater of the shear wall lengths determined by (a) or (b) determines the length of wall required for strength.

[e]The maximum value of the spectral response modification factor (C_v/T) is $2.5C_a$, per Figure 16-3 and Sec. 1630.2.1 of the 1997 Uniform Building Code. Therefore, it is conservative to use the maximum value of 2.5 for the spectral response modification factor. For example, in Seismic Zone 4 with $C_a = 0.4$, the spectral response modification factor would be $(2.5)(0.4) = 1.0$. For a more exact solution, use values of Cv, Ca as determined from Tables 16-Q, 16-R, 16-S and 16-T of the 1997 Uniform Building Code for applicable soil profile type.

Next, it is necessary to recheck the shear wall length based on the shear modulus (G') of the assembly. A refined value for the elastic fundamental period of vibration can be determined from Formula [5] below, which is based on Method B and Formula (30-10) described in Section 1630.2.2.2 of the UBC. The length of shear wall(s) determined above, and the shear modulus of the shear wall assembly, are used in this calculation. Since the shear wall length determined above is based on shear load at Yield Limit State (F_{yls}), the shear modulus corresponding to the shear wall displacement at this load [$G'_{yls} = (F_{yls} / \Delta_{yls})(H / L)$] is used in this analysis. Values for G'_{yls}, determined from these cyclic load shear wall tests, are summarized in Table 9 (note that $K_{e\text{-}yls} = G'_{yls}$ when $H / L = 1.0$).

$$T \approx (2\pi)\left[\frac{(W)(\Delta^2)}{(g)(F)(\Delta)}\right]^{0.5} \qquad [4]$$

For $\Delta = \frac{(F)(H)}{(G')(L)}$ and $g = 386.4$ in. per second^2 (= 32.2 ft per second^2), this formula reduces to:

$$T \approx (2\pi)\left[\frac{(W)(H)}{(386.4)(G')(L)}\right]^{0.5} \qquad [5]$$

After determining T per Formula [5], recheck the spectral response modification factor per Figure 16-3 of the UBC, and the design shear load for the wall(s). If this value has changed, repeat the process described above until closure (e.g., no change in the design shear load) results.

When the required length of shear wall has been determined, the shear wall **displacement** can be calculated per Formula [1].

Also, the process described above can be used to check the design for other serviceability limits, such as limited damage at $\Delta_{0.01H}$. In this case, the unit shear load and shear modulus at the applicable limit state (for example, see Table 9 for $v_{0.01H}$, $K_{e\text{-}0.01H}$) are substituted where appropriate in the calculations for determining T per Formula [5] to recheck the design shear load based on the refined spectral response modification factor, the shear capacity of walls required for strength, and wall displacement.

A design example to demonstrate the analysis procedures described above is included in Appendix C of this report.

The shear load should not exceed the "expected maximum shear strength" (page 24); and wall displacement should be limited to a value which can be tolerated without substantial damage to the structure or attached finishes. If necessary, the shear wall design (or length) is revised until a suitable solution is found.

CONCLUSIONS

The following conclusions are based on the results of a series of eight cyclic load shear wall tests conducted at the University of California - Irvine. Since only one test was conducted of each shear wall configuration, and no matching monotonic shear wall tests were conducted, the conclusions are considered ***preliminary*** until additional shear wall tests are conducted to confirm or modify these conclusions. More extensive cyclic and monotonic shear wall tests are in progress at the Research Center of *APA – The Engineered Wood Association* (Tacoma, WA), using recently acquired cyclic load test equipment and apparatus.

1. The load-displacement curves and hysteresis loops for the test walls were nearly linear and repeatable for all cycles of loading up to a story drift limit (displacement) of 0.48 inch, or 0.5% of wall height. The shear load at this displacement averaged 1.7 times the allowable design shear load recognized in the 1997 Uniform Building Code and National Evaluation Service, Inc. Report NER-108. The maximum shear strength of walls averaged about 1.4 times the shear load attained at this displacement level.

2. The load factor (maximum shear load/allowable design shear load) averaged 2.5 for four test walls with 3/8-inch or 15/32-inch plywood sheathing fastened with 8d or 10d common hand-driven nails, respectively. The tested performance appears to confirm the conservative nature of the allowable design shear loads for shear walls with plywood sheathing, which are based on monotonic tests and recognized in model building codes in the U.S.

3. The shear modulus of similarly fabricated shear walls with oriented strand board (OSB) or plywood sheathing appears to be equivalent. However, the maximum shear strength of shear walls with OSB sheathing appears to average about 15% less than walls with a comparable thickness and grade of plywood sheathing. This appeared to result from earlier fatigue failures of the fasteners used to attach higher density OSB sheathing, rather than to the sheathing itself. More cyclic load tests on matched shear wall constructions are in progress to confirm or modify this relationship.

4. Gypsum wallboard contributed to increased stiffness of a wall sheathed with wood structural panels in the range of allowable design shear loads, but did not increase the maximum shear strength of the wall, when subjected to cyclic (reversed) loads.

5. One shear wall with sheathing fastened with common short ("diaphragm") nails exhibited about 5% less maximum shear load than obtained in a matching test with full-length common nails. However, the energy dissipation of the wall with short nails was substantially more than obtained in the matching test with full-length nails.

6. The expected maximum shear strength, obtained after repeated (reversed) cycles of displacement, averaged about 18% less than the maximum shear strength reached in the initial displacement cycle. More cyclic (and monotonic) load tests on matched shear wall constructions are in progress to further study this relationship.

7. Results of other shear wall tests indicate that the shear stiffness and shear strength of shear walls can be increased when sheathing panel ends are restrained by bearing on the test fixture, simulating bearing on adjoining panels, floors, ceiling framing or foundations at the top and/or bottom of the wall. The building code in the United Kingdom also provides for increased allowable design shear load when shear walls are axially loaded.

8. In matching cyclic load shear wall tests, the "ramped" cyclic load test procedure provided results similar to the TCCMAR (Technical Coordinating Committee for Masonry Research) sequential phased displacement cyclic load test procedure, which incorporates decay cycles of load after ramped increments of increased cyclic load. The TCCMAR procedure has precedent for cyclic load testing of assemblies and fasteners, and is considered appropriate as a test protocol for conducting cyclic load tests.

9. Shear wall performance as represented by these tests is contingent on use of hold-down connectors which minimize slip and deformation. If commercially available hold-down connectors are used which permit more slip or deformation, the shear wall stiffness and shear strength will be compromised. In this case, resulting shear wall performance should be verified by cyclic load tests on appropriate wall assemblies.

10. A method is presented for analyzing the test results of cyclically loaded shear walls to determine shear wall stiffness (shear modulus) at Yield Limit State and maximum shear strength (Strength Limit State), and expected maximum shear strength (e.g., cycled shear strength). Use of this information in analyzing a structure is described.

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ACKNOWLEDGMENT

Robert Kazanjy, PE, President of PMR Enterprises, Inc. (P.O. Box 10934, Santa Ana, CA 92711) is thanked for providing input and close cooperation in arranging for and conducting these cyclic load shear wall tests for APA.

One of the cyclic load shear wall tests in this series of eight tests was sponsored by Halstead Enterprises/Stan-Tech Division of Stanley-Bostitch Inc. (Rancho Cucamonga, CA).[f] Their pneumatically driven 10d common short ("diaphragm") nails were used in attaching OSB sheathing to framing in Test 8A. A Halstead pneumatic nailing tool was used to drive the nails; it was equipped with a depth control adjustment device that limits the depth of fastener embedment, so that the nail head was driven flush with the surface of the panel. Code recognition for Halstead fasteners for shear wall and diaphragm applications was provided through ICBO Evaluation Service, Inc. Report No. 4296 (cancelled 1996).

Outside observers at the cyclic load shear wall tests conducted in April, 1995 at the University of California - Irvine included:

John Kariotis, SE/Kariotis & Associates (Sierra Madre, CA)

Ben Schmid, SE/Consulting Engineer (Balboa Island, CA)

Robert Harder/City of Los Angeles, Dept. of Building & Safety

(Los Angeles, CA)

Chuck Shubnell/Halstead Enterprises (Rancho Cucamonga, CA)

Prof. Donald Breyer/California State Polytechnic University (Pomona, CA)

Dr. Robin Shepherd, SE/Forensic Expert Advisers, Inc. (Santa Ana, CA)

Kurt Katsumata, SE/State of California, Div. of State Architect (Sacramento, CA)

[f] This plant is now closed. Current fastener production by Stanley-Bostitch Inc. (East Greenwich, RI). Code recognition per National Evaluation Service, Inc. Report NER-272.

APPENDIX A – LOAD-DISPLACEMENT CURVES (APA/UNIVERSITY OF CALIFORNIA – IRVINE TESTS)

The following pages present load-displacement curves for the eight shear walls tested at the University of California - Irvine and discussed in the body of this report.

FIGURE A1

Load-displacement curve for cyclic load shear wall test (Test 1A)

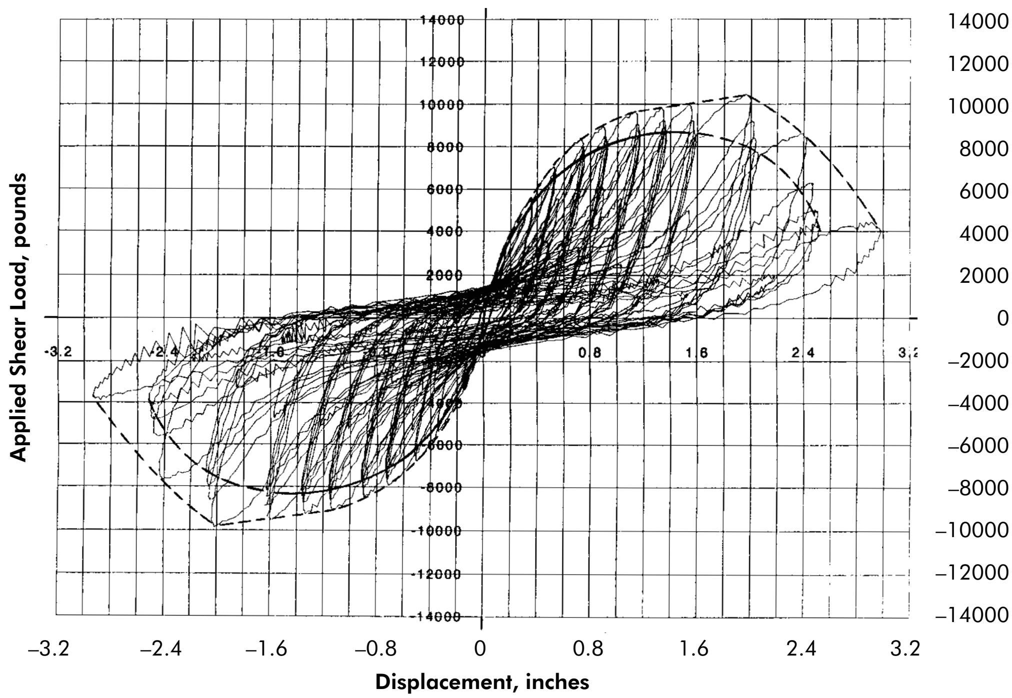


FIGURE A2

Load-displacement curve for cyclic load shear wall test (Test 2B)

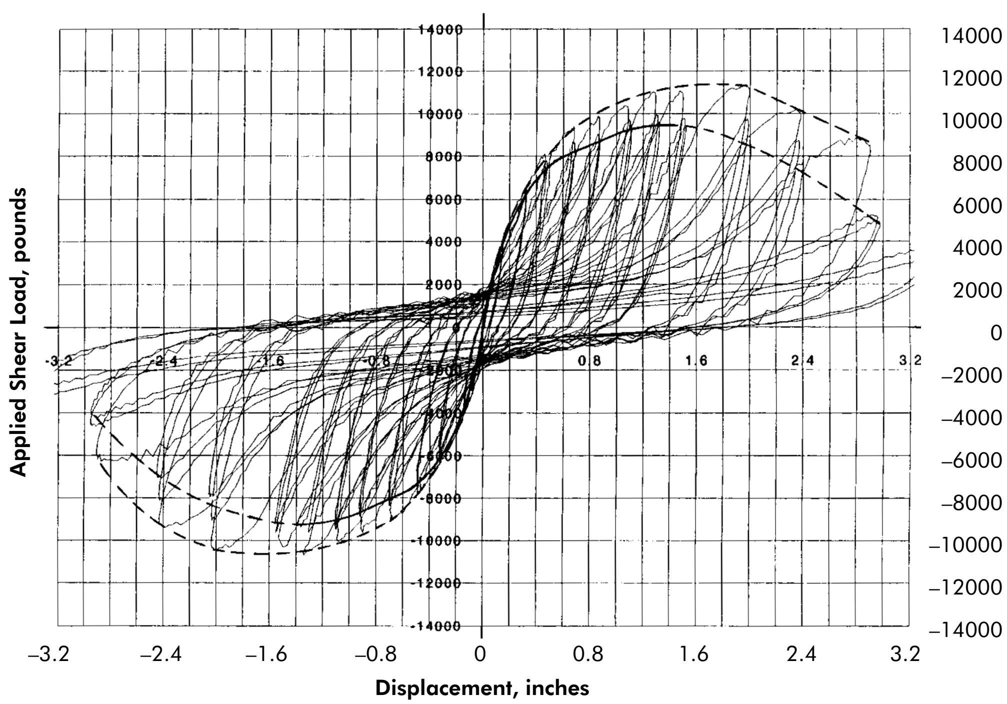


FIGURE A3

Load-displacement curve for cyclic load shear wall test (Test 3A)

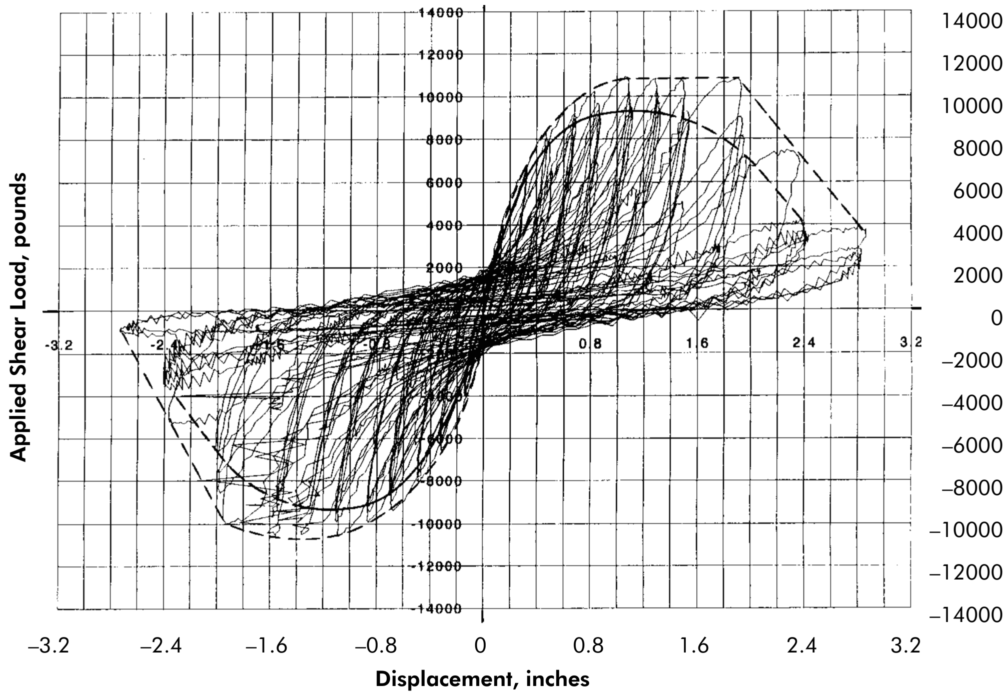


FIGURE A4

Load-displacement curve for cyclic load shear wall test (Test 4A)

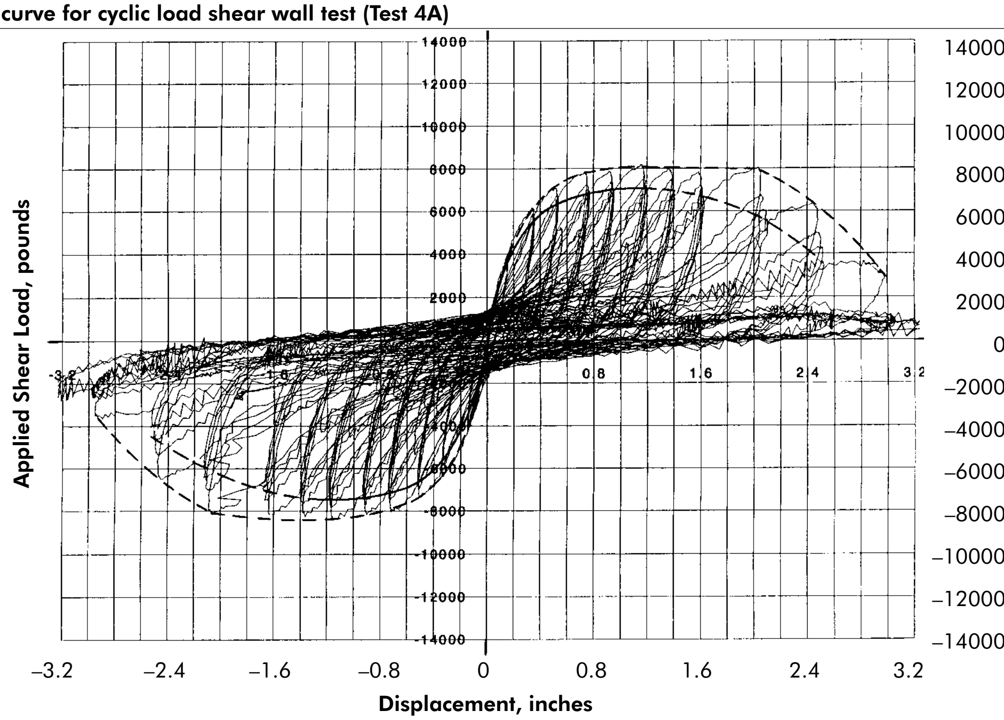


FIGURE A5

Load-displacement curve for cyclic load shear wall test (Test 5A)

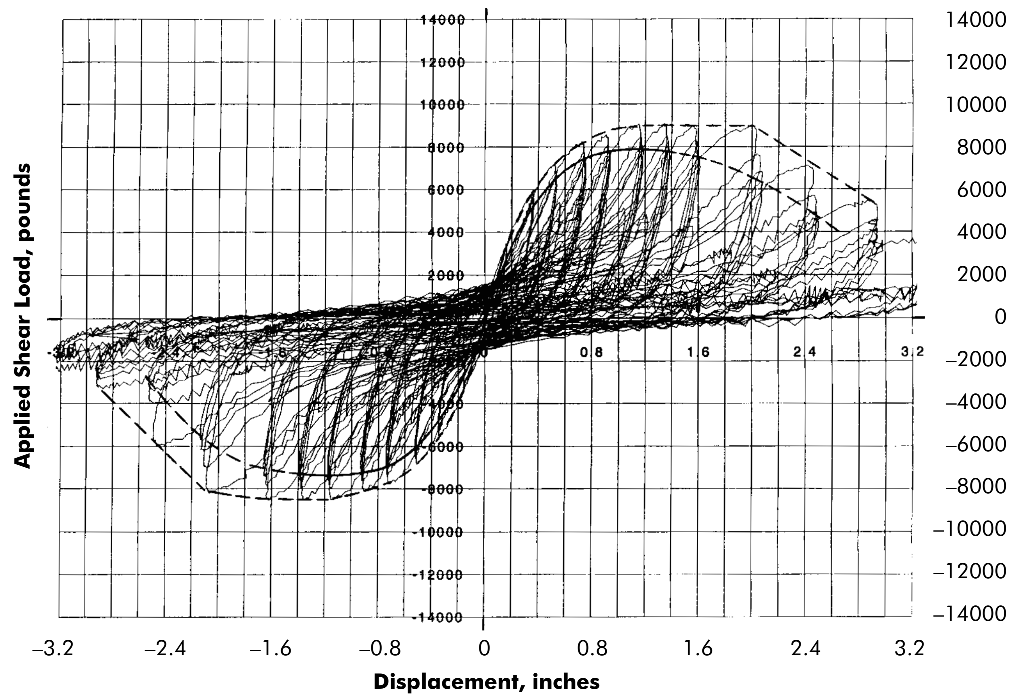


FIGURE A6

Load-displacement curve for cyclic load shear wall test (Test 6A)

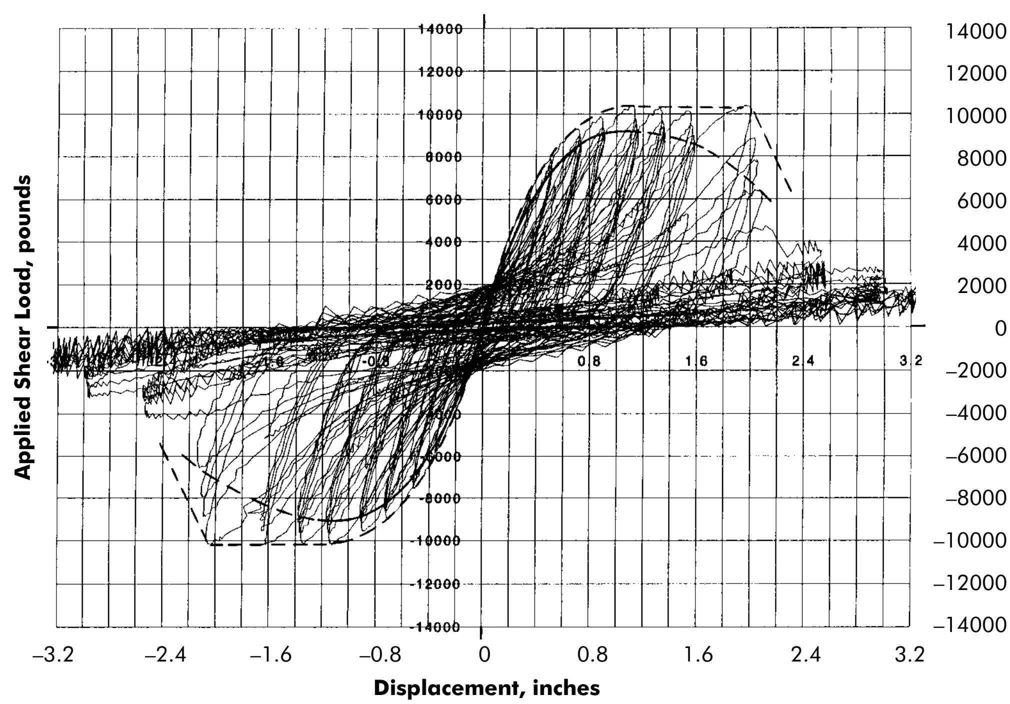


FIGURE A7

Load-displacement curve for cyclic load shear wall test (Test 7A)

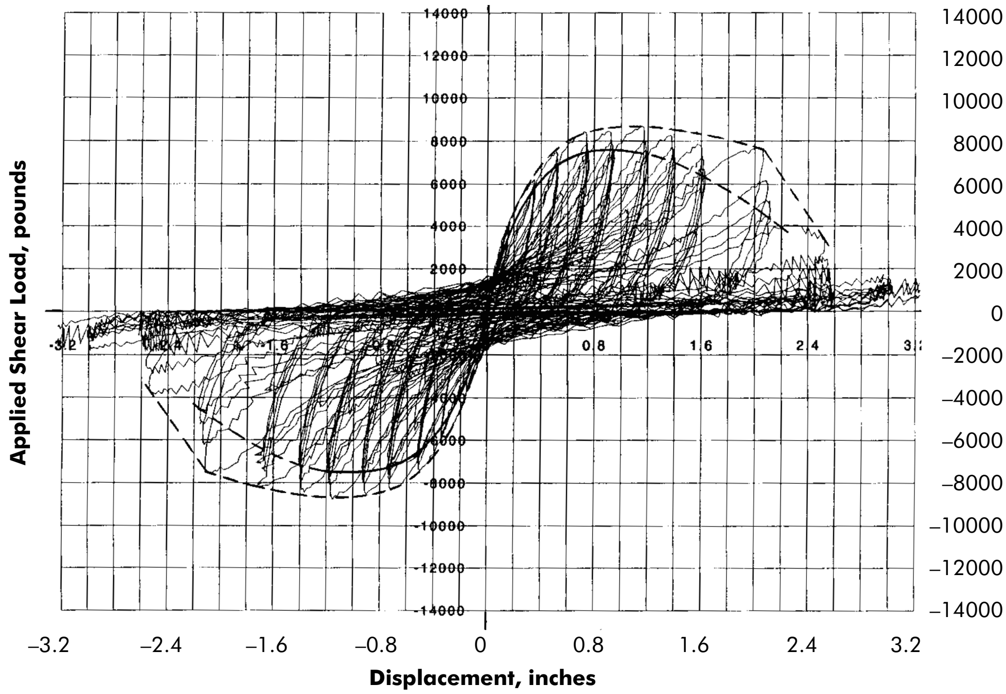
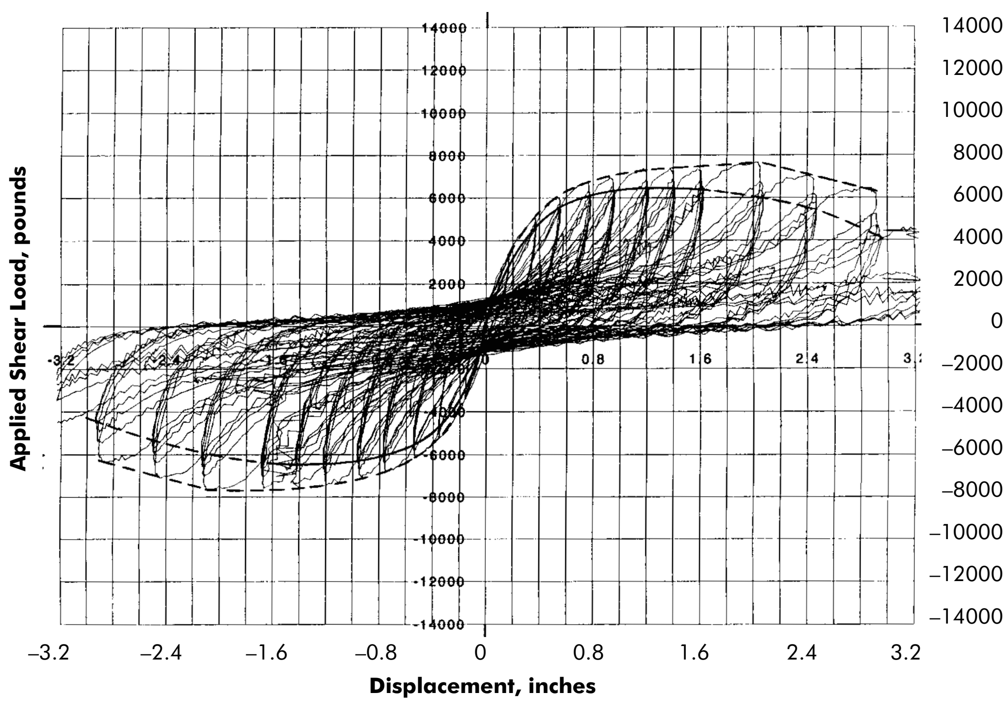


FIGURE A8

Load-displacement curve for cyclic load shear wall test (Test 8A)



APPENDIX B – TEST RESULTS AND LOAD-DISPLACEMENT CURVES (SCHMID/UNIVERSITY OF CALIFORNIA – IRVINE TESTS)

Three cyclic load tests on 8-foot x 8-foot shear walls were conducted in February and March, 1995 at the University of California - Irvine, for California structural engineer Ben L. Schmid, SE (Balboa Island, CA). Permission has been given to include results of these tests in this report.

The construction of the shear walls, test setup and TCCMAR sequential phased displacement cyclic load test procedure in these tests were similar to the tests conducted at the University of California - Irvine for APA, as described in the body of the report. All walls were framed with Douglas-fir lumber as described in the report, with only minor variations in fastening of studs and posts to the bottom plate of the wall. All walls were sheathed with 15/32-inch APA Structural I Rated Sheathing 32/16, Exposure 1 plywood (5-ply).

In two of the tests, the sheathing was fastened with 10d common, full-length hand-driven nails spaced 4 inches o.c. along panel ends and edges and 12 inches o.c. on intermediate studs. The allowable design shear load for these two walls is 510 pounds per foot in accordance with Table 23-II-I-1 of the 1997 Uniform Building Code; these walls were basically identical to Test 1A in the APA test series.

In the other test, the sheathing was fastened with 8d common, full-length hand-driven nails spaced 3 inches o.c. along panel ends and edges and 12 inches o.c. on intermediate studs. The allowable design shear load for this wall was 550 pounds per foot in accordance with the code. The results of the test on this wall can be compared to Test 6A in the APA test series. These walls were similar in construction except for the use of 15/32-inch Structural I plywood in Schmid's test, whereas 3/8-inch Structural I plywood (3-ply) was used in APA's test.

Table B1 provides a summary of shear wall deflection and shear modulus, and maximum shear strength for comparison with APA tests described in the body of the report. The load-displacement curves from these three cyclic load tests are shown in Figures B1 through B3. Figure B4 compares shear load applied to the wall versus the measured tension force in the bolt which attaches the hold-down connector to the rigid base of the test fixture.

Although results are similar, somewhat better shear wall performance was observed in Schmid's tests than in the APA tests. One factor that probably led to this difference is that the sheathing at the top of the wall came into bearing on the loading beam as the panels rotated during the cyclic load tests. As discussed in the body of this report, such bearing reduces the deformation of nails attaching the sheathing to the framing, leading to better wall stiffness and greater shear strength. This difference also has been noted in past monotonic load tests on walls.[4]

TABLE B1

Results of cyclic load shear wall tests (Schmid/UC-Irvine, 1995)

Test No.	Description	Design			At Δ = 0.48 in.(a)		At Δ = 0.96 in.(a)		At Δ = 1.44 in.(a)		Avg. Max. Load, lb (L.F.)
		lb/ft	lb	Δ in.(a)	Load, lb	G_1(b)	Load, lb	G_2(b)	Load, lb	G_3(b)	
BLS.1A	15/32" Plywood(c) 10d com. @ 4" o.c.	510	4,080	0.34	5,400	11.3	7,300	7.6	8,150	5.7	11,100 (2.7)
BLS.1B	15/32" Plywood(c) 10d com. @ 4" o.c.	510	4,080	0.36	5,000	10.4	7,450	7.8	8,300	5.8	10,900 (2.7)
BLS.2A	15/32" Plywood(c) 8d com. @ 3" o.c.	550	4,400	0.26	7,150	14.9	9,550	9.9	9,950(d)	6.9	12,400 (2.8)

(a) Tabulated data is average of ± cycles. Displacement (Δ) and shear loads – after fourth cycle of loading.

(b) lb/in. ÷ 1,000 (= kips/in.)

(c) APA Structural I Rated Sheathing 32/16 (5-ply plywood): Douglas-fir framing (studs spaced 16" o.c.).

(d) Expected maximum (cycled) shear strength: 10,500 lbs. at Δ = 1.33 in. (G = 7.9 kips/in.)

FIGURE B1

Load-displacement curve for cyclic load shear wall test (Test BLS.1A)

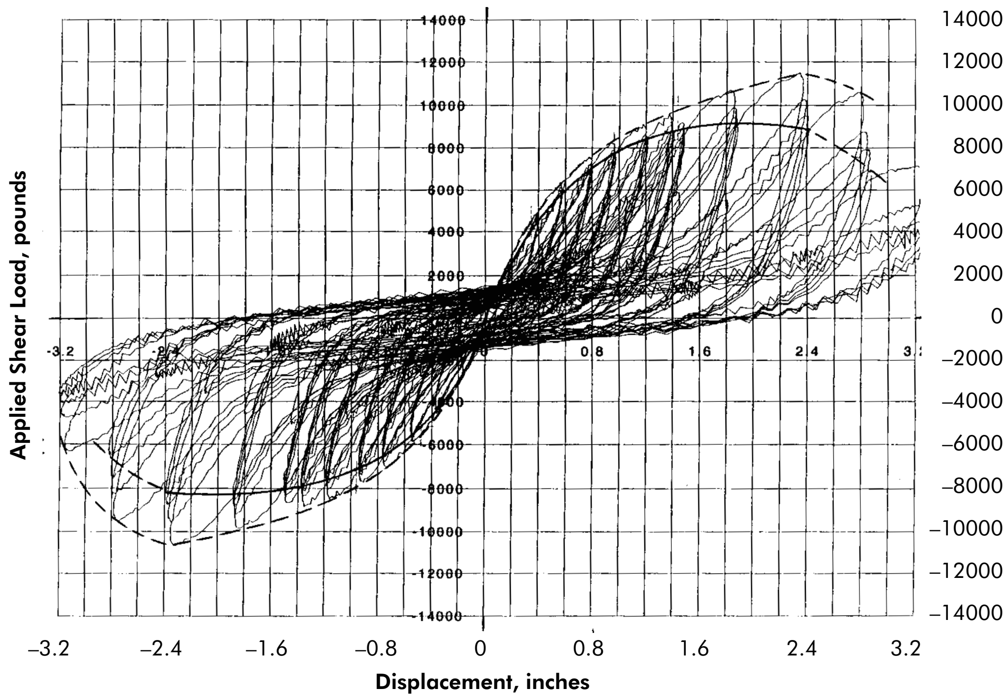


FIGURE B2

Load-displacement curve for cyclic load shear wall test (Test BLS.1B)

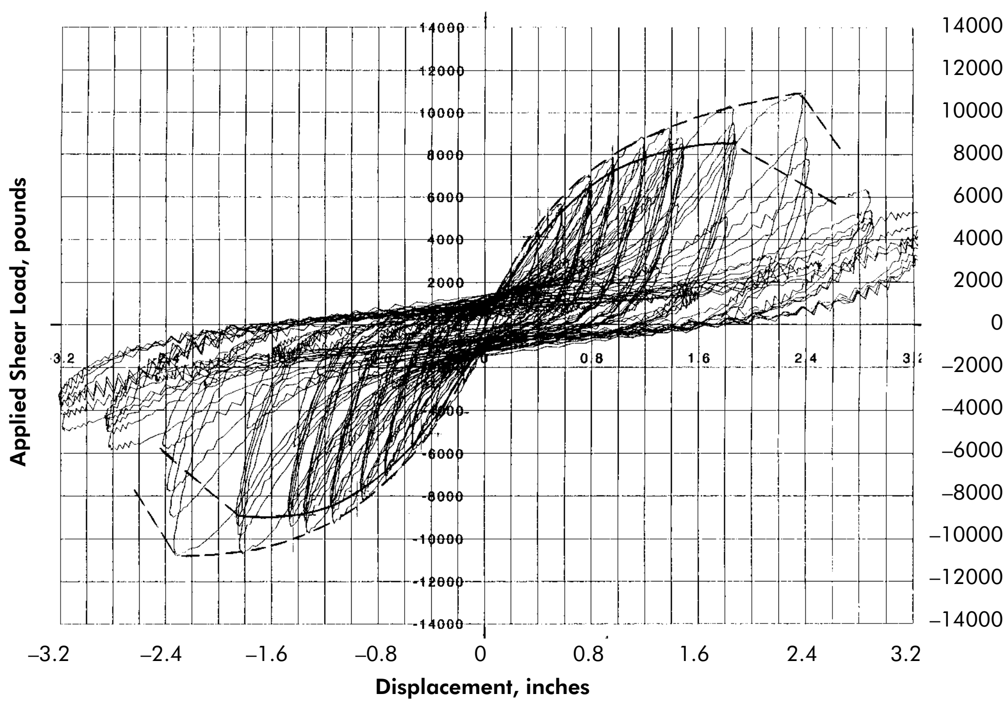


FIGURE B3

Load-displacement curve for cyclic load shear wall test (Test BLS.2A)

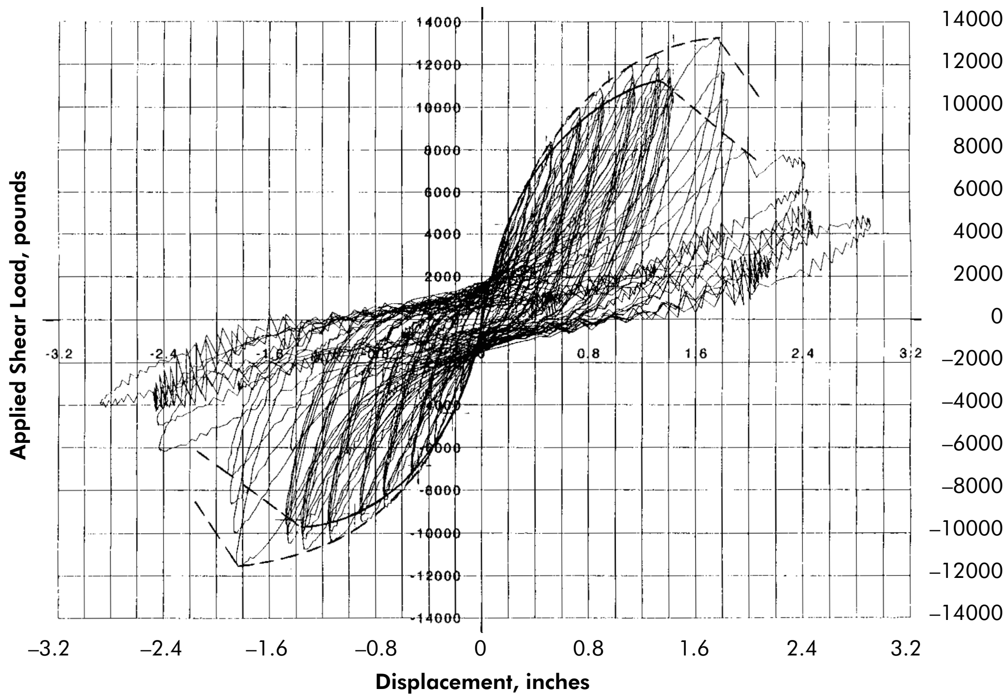
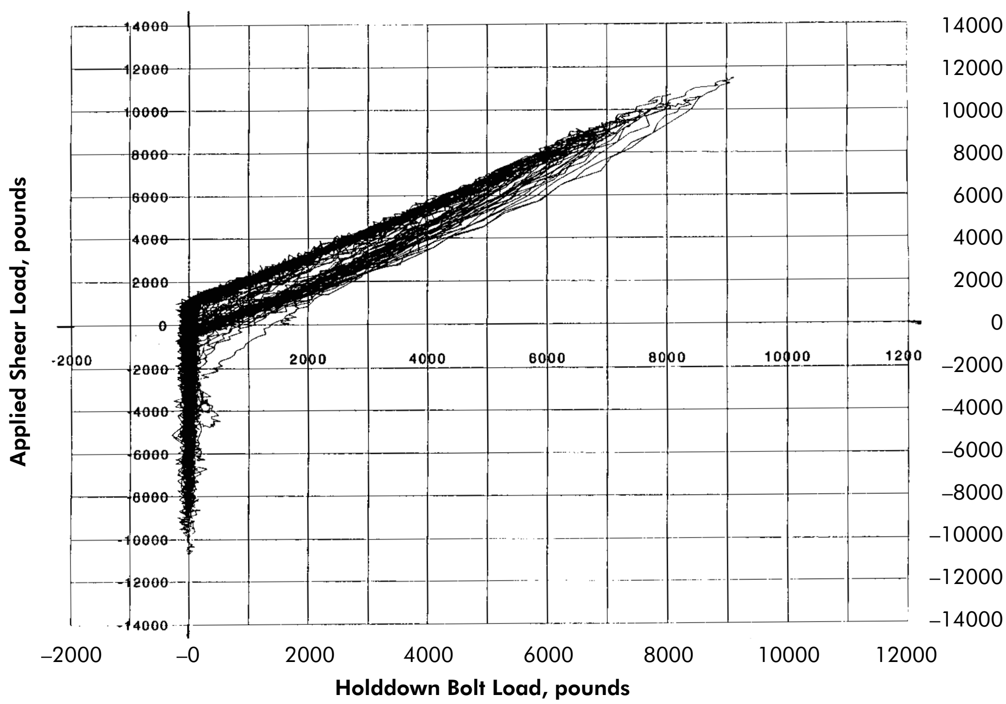


FIGURE B4

Typical hold-down load vs. shear wall load (Test BLS.1A)



APPENDIX C

Design Example for Shear Wall (Based on 1997 Uniform Building Code)

Problem: Calculate the length of shear wall (8 ft high) required to resist a total seismic dead load of 30 kips (Seismic Zone 4). Design shall be based on provisions of Chapters 16 and 23 in the 1997 Uniform Building Code (UBC).

The 1997 UBC introduces design of wood-framed structures using Load and Resistance Factor Design (LRFD) analysis, per UBC Section 1605.1. For wood, LRFD provisions[11] are referenced in UBC Section 2303.5 (Item 5.4). When analyzing a structure by LRFD, the full value of earthquake load (1.0 E) is used per UBC Sections 1605.1 and 1612.2.1.

Given:

a. Building design is assumed to comply with the criteria of UBC Section 1630.1.1 for assigning a Reliability/Redundancy Factor $\rho = 1.0$. For shear wall design using wood structural panels, based on LRFD analysis per ASCE Standard 16-95[11], the Time Effect Factor $\lambda = 1.0$ for earthquake (and wind) loads, and the Resistance Factor $\phi = 0.65$ (based on connections).

b. Shear wall consists of 15/32 in. APA Structural I Rated Sheathing 32/16, Exposure 1 (oriented strand board – OSB per U.S. Product Standard PS 2-92 or APA Standard PRP-108); fastened with 10d common nails spaced 4 in. o.c. at panel edges/12 in. o.c. to intermediate studs; framing of Douglas-fir, studs spaced 16 in. o.c. (4x end posts, 3x framing at common panel edges and bottom plate, 2x framing for intermediate studs and double top plate); see Figure 1 in report for details of 8 ft x 8 ft test wall. Allowable design shear load for wall = 510 lb/ft (0.51 kip/ft) (Ref: UBC Table 23-II-I-1).

c. Shear wall design is based on properties determined by cyclic load testing of walls (average of Tests 4A and 5A in APA Research Report 158). Average values from Tests 4A and 5A (Tables 8 and 9 in this report):

$G_{yls} = K_{e\text{-}yls}$ (all shear modulus values based on H/L = 1.0) = 17.3 kips/in.

$G_{0.01H} = K_{e\text{-}0.01H} = 7.8$ kips/in.

$G_{sls} = K_{e\text{-}sls} = 6.0$ kips/in.

$v_{yls} = 0.74$ kip/ft

$v_{0.01H} = 0.93$ kip/ft

$v_{sls} = 1.07$ kips/ft

$R_o = 1.4$

$R_{d\text{-}S} = 2.8$

1. Determine required length of shear wall based on shear strength.

a. Calculate estimated elastic fundamental period of vibration.

Ref: UBC Sec. 1630.2.2.1, Method A.

$T = (0.020)(h_n)^{0.75} = (0.020)(8)^{0.75} = 0.10$ sec.

b. Determine spectral acceleration coefficient (C_v / T).

Ref: Fig. 16-3 and Tables 16-Q (C_a), 16-R (C_v), 16-S (N_a) and 16-T (N_v) of UBC. Assumed: Seismic Zone 4, Soil Profile Type S_D, Near-Source Factors $N_a = 1.0$ and $N_v = 1.0$.

$T_s = C_v / 2.5C_a = (0.64)(1.0) / (2.5)(0.44)(1.0) = 0.58$ sec.

$T_o = 0.2\ T_s = (0.2)(0.58) = 0.12$ sec.

Proportioning from Fig. 16-3 with above values for T_s and T_o:

For T = 0.10 sec., $C_v / T = C_a + [(2.5C_a - C_a)(0.10) / 0.12] = 2.29\ C_a$

c. Calculate design base shear (LRFD).

Ref: UBC Sec. 1630.2.1.

$V_{lrfd} = (C_v / T)(I)(W) / R$; assume Importance Factor I = 1.0 and R = 5.5 (Ref: UBC Tables 16-K and 16-N). Note: See following Discussion for Allowable Stress Design (ASD) analysis. For ASD, earthquake load $E_{asd} = E_{lrfd} / 1.4$ per UBC Sections 1612.3.1 and 1612.3.2.

$V_{lrfd} = (2.29\ C_a)(W) / R = (2.29)(0.44)(30) / 5.5 = 5.5$ kips

d. Calculate length of shear wall required for shear strength at Yield Limit State.

$L_{lrfd} = V_{lrfd} / v_{yls} = 5.5 / 0.74 = 7.4$ ft

e. Check length of shear wall required based on shear strength at Strength Limit State.

Ref: Resistance Factor $\phi = 0.65$ (for connections; also applies to shear walls) per ASCE 16-95[11]. See following Discussion regarding comparison of wall length as determined by v_{lrfd} and v_{asd}.

$L_{lrfd} = V_{lrfd} / (\phi)(v_{sls}) = 5.5 / (0.65)(1.07) = 7.9$ ft (controls, > 7.4 ft)

f. Recheck elastic fundamental period of vibration and shear wall length.

Ref: UBC Sec. 1630.2.2.2, Method B.

$$T = (2)(\pi)\left[\frac{(W)(H)}{(386.4)(G')(L)}\right]^{0.5}$$

$$= (2)(3.14)\left[\frac{(30)(8)}{(386.4)(17.3)(7.9)}\right]^{0.5}$$

$$= 0.42 \text{ sec.}$$

[> (1.3)(T = 0.10 sec. per Method A; see 1.a above) = 0.13 sec. Therefore use T = 0.13 sec. per UBC Sec. 1630.2.2.2].

Note use of G_{yls} for G′. Length of shear wall is based on shear strength at Strength Limit State; and T is applicable for shear loads up to Yield Limit State.

From Fig. 16-3 for T = 0.13 sec., C_v / T = 2.5 C_a (see 1.b above).

$V_{lrfd} = (2.5C_a)(W) / R$
$= (2.5)(0.44)(30) / 5.5 = 6.0$ kips

$L_{lrfd} = V_{lrfd} / (\phi)(v_{sls}) = 6.0 / (0.65)(1.07)$
$= 8.6$ ft

Recheck T:

$$T = (2)(3.14)\left[\frac{(30)(8)}{(386.4)(17.3)(8.6)}\right]^{0.5}$$

$= 0.41$ sec.
[use T = 0.13 sec. per 1.f above]

From Fig. 16-3 for T = 0.13 sec.,
$C_v / T = 2.5C_a$

V = 6.0 kips (same as above)
(Closure – ok)

L_{lrfd} = 8.6 ft (based on shear strength at Strength Limit State)

Note: Shear capacity at Yield Limit State $= V_{yls} = (v_{yls})(L_{lrfd}) = (0.74)(8.6) = 6.4$ kips. This shear capacity corresponds to 6.4 / 6.0 = 1.1 times design base shear.

g. Check shear wall displacement at design base shear.

$\Delta = (V_{lrfd})(H) / (G_{yls})(L_{lrfd})$
$= (6.0)(8) / (17.3)(8.6) =$ **0.32 in.**
(corresponds to 0.003 H for 8-ft-high wall).

2. Determine shear capacity of shear wall based on limited structural damage (Δ_{max} = 0.01 H).

a. Length of shear wall required for shear strength.

$L_{lrfd} = 8.6$ ft (see 1.a - 1.f above)

b. Recheck elastic fundamental period of vibration.

$$T = (2)(3.14)\left[\frac{(30)(8)}{(386.4)(7.8)(8.6)}\right]^{0.5}$$

$= 0.60$ sec.
[use T = 0.13 sec. per 1.f above]

Note use of $G_{0.01H}$ for G′.

From Fig. 16-3 for T = 0.13 sec., $C_v / T = 2.5 C_a$ (see 1.b above)

V = 6.0 kips (same as 1.f above)
(Closure – ok)

c. Calculate shear capacity at Δ = 0.01H.

$V_{0.01H} = (v_{0.01H})(L_{lrfd}) = (0.93)(8.6)$
$= 8.0$ kips

Note: This shear capacity corresponds to 8.0 / 6.0 = 1.3 times design base shear. For limited damage, $R_o = v_{sls} / v_{0.01H} = 1.07 / 0.93 = 1.1$.

d. Check shear wall displacement.

$D = (V_{lrfd})(H) / (G_{0.01H})(L_{lrfd}) =$
$(8.0)(8) / (7.8)(8.6) =$ **0.95 in.**
(corresponds to 0.01H for 8-ft-high wall).

Discussion:

Allowable Stress Design (ASD)
A factor of 1.4 was introduced in the 1997 UBC for converting calculated earthquake loads based on LRFD to ASD. This can be seen by comparing earthquake load (E) in UBC Section 1612.2.1 for LRFD, with UBC Sections 1612.3.1 or 1612.3.2 for ASD. Note that the design base shear is calculated by using a Response Modification Factor R = 5.5 for light-framed shear walls with wood structural panel sheathing as specified in UBC Table 16-N (or 4.5 for light-framed walls sheathed with materials other than wood structural panels). These values apply to LRFD analysis and average about 1.4 times the design base shear determined with corresponding R_w factors (8 or 6) for ASD analysis in previous editions of the code.

When the above provisions are taken into account, a comparable shear wall design results when an ASD analysis method is used. Allowable design shear loads for shear walls analyzed by ASD are contained in UBC Table 23-II-I-1.

To compare the above shear wall design example using ASD instead of LRFD analysis per UBC provisions, consider the length of shear wall required to resist the calculated design base shear of 6.0 kips, applicable to LRFD analysis with maximum $V = (2.5C_a)(W) / R$ as noted in 1.f of the design example above. For this design example, shear wall length is controlled by required shear strength at Strength Limit State.

Shear wall length would be determined as follows according to UBC provisions noted above, and based on the ASD allowable design shear $v_{asd} = 0.51$ kip/ft:

$L_{asd} = (V_{lrfd} / (1.4)(v_{asd})$
$= (6.0) / (1.4)(0.51) = 8.4$ ft

The resulting shear wall length is approximately the same as in 1.f in the above analysis. It is noted that the ratio (v_{yls}/v_{asd}), which relates Yield Limit State to allowable design shear (ASD basis), has a value of 1.45 (=0.74/0.51) which is nearly equivalent to the 1.4 factor used in the UBC to adjust from ASD to LRFD values.

For comparison, a check was made on the required shear wall length determined using ASD design and the 1994 UBC requirements for equivalent conditions:

$V_{asd} = (Z)(C)(I)(W) / R_w =$
$(0.40)(2.75)(1.0)(30) / 8 = 4.1$ kips
[= 0.138W]

$L_{asd} = 4.1 / 0.51 = 8.1$ ft, or slightly less than determined above (shear wall length based on 1997 UBC is 4% longer).

The test results reported herein (Tables 8 and 9, on page 26 of this report) provide *preliminary* information on R_o and R_d factors obtained from cyclic load testing of shear walls with wood structural panels. More cyclic load shear wall testing is in process by APA to confirm these values.

APA RESEARCH AND TESTING

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7011 So. 19th St. ▪ P.O. Box 11700
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(253) 565-6600 ▪ Fax: (253) 565-7265

(International Offices: Bournemouth, United Kingdom; Hamburg, Germany; Mexico City, Mexico; Tokyo, Japan.)

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