

Performance of Cold-Formed Steel-Framed Shear Walls: Alternative Configurations

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PREFACE

This report presents the results of twenty shear wall tests that were conducted to evaluate the performance of wall configurations not permitted in the building codes in 2002. In this test program, four areas of performance were addressed, as follows:

- Reversed cyclic performance of 7/16-in. OSB shear walls sheathed one side and framed with 54- and 68-mil steel.
- Reversed cyclic performance of 7/16-in. OSB shear walls sheathed each side and framed with 54- and 68-mil steel.
- Reversed cyclic performance of 27-mil sheet steel shear walls (sheathed one side) with simple lap shear connections at the adjoining edges of the sheet steel panels (adjoining edge perpendicular to framing).
- Monotonic performance of ½-in. gypsum sheathed shear walls (sheathed one side) with alternative (to the current codes) blocking configurations and fastener schedules.

The findings provided a basis for the AISI Committee on Framing Standards to establish design options for shear walls in the AISI *Standard for Cold-Formed Steel Framing – Lateral Design*.

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Performance of Cold-Formed Steel-Framed Shear Walls: Alternative Configurations

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An Applied Research Project

ABSTRACT

This report presents the results of twenty shear wall tests that were conducted to evaluate the performance of wall configurations not currently (2002) permitted in the building codes. Ten (10) walls were evaluated under reversed cyclic loading and the other ten (10) under monotonic loading. Brief descriptions of the test program and results are presented in the following paragraphs.

The reversed cyclic load tests comprised 4 ft. x 8 ft. 54- and 68-mil framed walls with 7/16-in. OSB rated sheathing on one or both sides of the wall and 33-mil framed walls with 27-mil sheet steel. The sheet steel wall incorporated a horizontal lap shear joint at the wall mid-height. The monotonic tests focused exclusively on 8 ft. x 8 ft. gypsum sheathed shear walls with an unblocked configuration, except for two tests.

Overall, the OSB tests showed that the No. 8 screws in 54-mil framing and No. 10 screws on 68-mil framing permitted a ductile mode of failure at the connection. In the doubled-sided (sheathing each side) wall tests, the load demands on the 54-mil chord studs exceeded the capacity of studs and the load demands at the holdown attachment to the 68-mil chords studs exceeded the capacity of the screws. As a result, the capacity of the double-sided wall was less than twice the capacity of the single-sided wall. In the sheet steel walls, shear buckling accompanied by diagonal tension resulted in high demands on a few screws at the mid-height joint which caused the panel to unzip prematurely along the joint. Failure in the GWB monotonic tests was characterized by breaking of the wallboard at the location of the fasteners along the “un-papered” edges and screw pull-through along the “papered” edges of the wallboard.

Keywords: Shear, walls, OSB, sheet, steel, cyclic, monotonic, cold-formed, gypsum, wallboard, sheathing.

Table of Contents

List of Tables	iii
List of Figures	iv
BACKGROUND	1
OBJECTIVE	2
SCOPE	3
TEST PROGRAM	3
TEST PROCEDURE	5
TEST RESULTS	8
DISCUSSION OF TEST RESULTS	12
RECOMMENDATIONS FOR FURTHER WORK	16
SUMMARY and CONCLUSIONS	17
APPENDIX A	18
APPENDIX B	29

List of Tables

Table 1. Reversed cyclic performance of 7/16-in. OSB single-sided shear walls (sheathed one side) framed with 54- and 68-mil steel	4
Table 2. Reversed cyclic performance of 7/16-in. OSB double-sided shear walls (sheathed each side) framed with 54-mil and 68-mil steel	4
Table 3. Reversed cyclic performance of 27-mil sheet steel shear walls with simple lap shear connections at the adjoining panel edges	4
Table 4. Monotonic performance of ½-in. gypsum wallboard (GWB) shear walls	5
Table 5. Reversed cyclic test procedure	7
Table 6. Observed modes of failure	8
Table 7. Measured material strength and thickness	12
Table 8. Measured performance of 7/16-in. OSB shear walls, sheathed one side, framed with 54- and 68-mil steel	14
Table 9. Measured performance of 7/16-in. OSB shear walls, sheathed each side, framed with 54-mil and 68-mil steel	15
Table 10. Measured performance of 27-mil sheet steel shear walls with simple lap shear connections at the adjoining panel edges	15
Table 11. Measured performance of ½-in. gypsum wallboard (GWB) shear walls	16

List of Figures

Figure 1. Position of displacement transducers	6
Figure 2. Observed behavior in Tests 1 and 2	8
Figure 3. Observed behavior in Tests 3 and 4	9
Figure 4. Observed behavior in Tests 6 and 7	9
Figure 5. Observed behavior in Tests 8 and 9	9
Figure 6. Observed behavior in Tests 10 and 11	10
Figure 7. Observed behavior in Tests 12, 13, 14, 15, 16, 17, 20 and 21	10
Figure 8. Observed behavior in Tests 18 and 19	11
Figure 9. Second cyclic peak load envelope	13
Figure 10. Second cycle peak loads	14
Figure A1. Response curve for Test 1	19
Figure A2. Response curve for Test 2	19
Figure A3. Response curve for Test 3	20
Figure A4. Response curve for Test 4	20
Figure A5. Response curve for Test 6	21
Figure A6. Response curve for Test 7	21
Figure A7. Response curve for Test 8	22
Figure A8. Response curve for Test 9	22
Figure A9. Response curve for Test 10	23
Figure A10. Response curve for Test 11	23
Figure A11. Response curve for Test 12	24
Figure A12. Response curve for Test 13	24
Figure A13. Response curve for Test 14	25
Figure A14. Response curve for Test 15	25
Figure A15. Response curve for Test 16	26
Figure A16. Response curve for Test 17	26

Figure A17. Response curve for Test 18	27
Figure A18. Response curve for Test 19	27
Figure A29. Response curve for Test 20	28
Figure A20. Response curve for Test 21	28
Figure B1. Resisting load time history Test 1	30
Figure B2. Resisting load time history Test 2	30
Figure B3. Resisting load time history Test 3	31
Figure B4. Resisting load time history Test 4	31
Figure B5. Resisting load time history Test 6	32
Figure B6. Resisting load time history Test 7	32
Figure B7. Resisting load time history Test 8	33
Figure B8. Resisting load time history Test 9	33
Figure B9. Resisting load time history Test 10	34
Figure B10. Resisting load time history Test 11	34

BACKGROUND

The 1997 UBC provisions for cold-formed (light-gauge) steel-framed braced walls limits the lateral force resisting system to 15/32-in. Structural I plywood, 7/16-in. OSB rated sheathing, ½-in. gypsum wallboard/sheathing or diagonal bracing (tension and/or compression members). Testing for development of the UBC design values for sheathed walls included both monotonic (“static”) and reversed cyclic loadings for wind and seismic conditions, respectively. Subsequent to publication of the 1997 UBC, additional wall testing was completed (including sheet steel shear walls) and the results of this latter work were used to update and expand the design data in the 1997 UBC. The current state-of-the-art for cold-formed steel-framed lateral force design is contained in the 2000 International Building Code (IBC).

In both the 1997 UBC and 2000 IBC, limitations were justifiably imposed on the range of applicability of the published design data for the simple reason that the data is based on physical testing. The primary code limitations are summarized below:

- For seismic design, the maximum uncoated framing thickness is 0.043 in. or 0.048 in. (depending on whether the UBC or IBC, respectively, is referenced), and the minimum uncoated framing thickness is 0.033 in.
- Studs are a minimum 1-5/8 in. (flange) x 3-1/2 in. (web) with a 3/8 in. flange return (lip).
- Tracks are a minimum 1-1/4 in. (flange) x 3-1/2 in. (web).
- A minimum of two back-to-back studs are required at the ends (boundaries) of a shear wall.
- Minimum of No. 8 screws are required for attachment of plywood and OSB sheathing.

- Minimum of No. 6 screws for attachment of gypsum wallboard/gypsum sheathing.
- Aspects ratios per the code (2:1 maximum in the UBC; up to 4:1 in the IBC for some applications).
- No increase in strength is permitted for walls sheathed both sides with the same material (except as provided for gypsum wallboard/sheathing).
- Gypsum wallboard/sheathing must be applied perpendicular to framing with minimum prescribed strapping at abutting joints.

Given the nature of the residential market today, designers/engineers are finding that the limitations imposed by the codes can potentially impede their ability to provide designs that are responsive to market needs and competitive with alternative light framing materials. Knowing that the code limitations are based strictly on a limited scope of testing, designers/engineers have extrapolated the existing design data based on their interpretation of basic engineering principles. In some instances, demonstrating that these extrapolations comply with the intent of the building code may be difficult.

OBJECTIVE

The objective of the research reported in this document was to develop performance data for cold-formed steel-framed shear wall systems not directly addressed or permitted in current building codes. An ancillary objective was to help focus the efforts of the industry on possible new design needs. In the following section, the scope of the test program is described.

SCOPE

In this test program, four areas of performance were addressed. These four areas included:

- Reversed cyclic performance of 7/16-in. OSB shear walls sheathed one side and framed with 54- and 68-mil steel.
- Reversed cyclic performance of 7/16-in. OSB shear walls sheathed each side and framed with 54- and 68-mil steel.
- Reversed cyclic performance of 27-mil sheet steel shear walls (sheathed one side) with simple lap shear connections at the adjoining edges of the sheet steel panels (adjoining edge perpendicular to framing).
- Monotonic performance of ½-in. gypsum sheathed shear walls (sheathed one side) with alternative (to the current codes) blocking configurations and fastener schedules.

Additional details of the test program are provided in the following section.

TEST PROGRAM

For each of the four performance areas identified in the previous section, a series of tests, as indicated in Tables 1 through 4, were conducted. As shown in the tables, for each wall configuration, two identical tests were completed.

The overall dimensions (out-to-out) of the walls for the reversed cyclic load tests were 4 ft. (wide) x 8 ft. (tall) while the wall dimensions for the monotonic tests (GWB) were 8 ft. x 8 ft. Stud spacing in all tests was 24 in. on center and all boundary members were back-to-back studs (same thickness at track and interior studs). Other wall details are given in Tables 1 through 4.

Table 1. Reversed cyclic performance of 7/16-in. OSB single-sided shear walls
(**sheathed one side**) framed with 54- and 68-mil steel

Test No.	Framing ¹	Sheathing and Attachment			Test protocol ⁴	Comments ⁵
		Type ²	Screw spacing ³	Screw size		
1 and 2	350S162-54 studs 350T125-54 track	7/16-in. OSB rated sheathing	2"/12"	No. 8 self-drilling screws	Reversed cyclic load	- Sheathing one face of the wall 4 ft. x 8 ft. wall (1) 4 ft x 8 ft. sheet
3 and 4	350S162-68 studs 350T125-68 track	7/16-in. OSB rated sheathing	2"/12"	No. 10 self-drilling screws	Reversed cyclic load	- Sheathing one face of the wall 4 ft. x 8 ft. wall (1) 4 ft x 8 ft. sheet

¹ Stud and track Grade 50 steel ASTM A653 or A792 or A875. Framing fasteners: No. 10 pancake head self-drilling screws.
² Per DOC PS1 or PS 2 exterior use.
³ a" / b"—a inches at the supported panel edges and b inches in the panel field. Fastener panel edge distance = 3/8 in.
⁴ See "TEST PROCEDURE" in main text
⁵ Simpson S/HD15 used at each chord (attached with 48 No. 10 screws)

Table 2. Reversed cyclic performance of 7/16-in. OSB double-sided shear walls
(**sheathed each side**) framed with 54-mil and 68-mil steel

Test No.	Framing ¹	Sheathing			Test protocol ⁴	Comments ⁵
		Type ²	Screw spacing ³	Screw size		
6 and 7	350S162-54 studs 350T125-54 track	7/16-in. OSB rated sheathing	2"/12"	No. 8 self-drilling screws	Reversed cyclic load	- Sheathing both faces of the wall 4 ft. x 8ft. wall (2) 4 ft. x 8 ft. sheets
8 and 9	350S162-68 studs 350T125-68 track	7/16-in. OSB rated sheathing	2"/12"	No. 10 self-drilling screws	Reversed cyclic load	- Sheathing both faces of the wall 4 ft. x 8ft. wall (2) 4 ft. x 8 ft. sheets

¹ Stud and track Grade 50 steel ASTM A653 or A792 or A875. Framing fasteners: No. 10 pancake head self-drilling screws.
² Per DOC PS1 or PS 2 exterior use.
³ a" / b"—a inches at the supported panel edges and b inches in the panel field. Fastener panel edge distance = 3/8 in.
⁴ See "TEST PROCEDURE" in main text.
⁵ Simpson S/HD15 used at each chord (attached with 48 No. 10 screws)

Table 3. Reversed cyclic performance of 27-mil sheet steel shear walls with simple lap shear connections at the adjoining panel edges

Test No.	Framing ¹	Sheathing			Test protocol ⁴	Comments ⁵
		Type ²	Screw spacing ³	Screw size		
10 and 11	350S162-33 studs 350T125-33 track	27-mil 33 ksi sheet steel	2"/12"	No. 8 self-drilling screws	Reversed cyclic load	- Sheathing one face of the wall 4 ft. x 8ft. wall (2) 4 ft. x 4 ft.-3/4 in. sheets 1.5 in. lap joint at wall mid-height w/single line of fasteners

¹ Stud and track Grade 33 steel ASTM A653 or A792 or A875. Framing fasteners: No. 8 modified truss head self-drilling screws.
² Grade 33 ASTM A653 or A792 or A875.
³ a" / b"—a inches at the supported panel edges and b inches in the panel field. Fastener panel edge distance = 3/8 in.
⁴ See "TEST PROCEDURE" in main text.
⁵ Simpson S/HD10 used at each chord (attached with 33 No. 10 screws)

Table 4. Monotonic performance of ½-in. gypsum wallboard (GWB) single-sided shear walls

Test No.	Framing ¹	Sheathing			Test protocol ⁴	Comments ^{5, 6}
		Type ²	Screw spacing ³	Screw size		
12 and 13	350S162-33 studs 350T125-33 track	½-in. GWB	4"/4"	No. 6 self-drilling	Monotonic	• Sheathing one face of the wall • 8 ft. x 8 ft. wall • (2) 4 ft. x 8 ft. sheets (perpendicular to framing) • no blocking/strapping @ horizontal joint
14 and 15	350S162-33 studs 350T125-33 track	½-in. GWB	7"/7"	No. 6 self-drilling	Monotonic	• Sheathing one face of the wall • 8 ft. x 8 ft. wall • (2) 4 ft. x 8 ft. sheets (perpendicular to framing) • no blocking/strapping @ horizontal joint
16 and 17	350S162-33 studs 350T125-33 track	½-in. GWB	8"/12"	No. 6 self-drilling	Monotonic	• Sheathing one face of the wall • 8 ft. x 8 ft. wall • (2) 4 ft. x 8 ft. sheets (perpendicular to framing) • no blocking/strapping @ horizontal joint
18 and 19	350S162-33 studs 350T125-33 track	½-in. GWB	4"/12"	No. 6 self-drilling	Monotonic	• Sheathing one face of the wall • 8 ft. x 8 ft. wall • (2) 4 ft. x 8 ft. sheets (perpendicular to framing) • 2-in. 33 mil strap @ horizontal joint
20 and 21	350S162-33 studs 350T125-33 track	½-in. GWB	4"/12"	No. 6 self-drilling	Monotonic	• Sheathing one face of the wall • 8 ft. x 8 ft. wall • (2) 4 ft. x 8 ft. sheets (perpendicular to framing) • no blocking/strapping @ horizontal joint

¹ Stud and track Grade 33 steel ASTM A 653 or A 792 or A 875. Framing fasteners: No. 8 modified truss head self-drilling screws.
² Type X ASTM C 36
³ a" / b"—a inches at the supported panel edges and b inches in the panel field. Fastener panel edge distance = 3/8 in.
⁴ See "TEST PROCEDURE" in main text.
⁵ Simpson S/HD10 used at each chord (attached with 33 No. 10 screws)
⁶ Horizontal joint not taped and mudded.

TEST PROCEDURE

Each wall was tested in a horizontal position. Installation of the 4 ft. x 8 ft. walls was accomplished by attaching the bottom of the wall to a reaction beam. Between the bottom track and the reaction beam a 3-1/2 in. wide by ¾-in. thick plate was used to

facilitate movement of attached sheathing relative to the frame. The base attachment comprised holdowns at the boundary studs and two shear anchors between the holdown bolts (approximately 12 in. from each holdown bolt). Both the holdown bolts and shear anchors were 7/8 in. diameter high strength bolts. A 2 in. square washer was used in the track at each shear anchors. At the top of the wall, the wall track was separated from the loading beam by a 3/8-in. thick steel plate. The top track was attached through the 3/8-in. plate to a loading beam with (4) 7/8-in. high strength bolts. The same basic anchorage scheme was used for the 8 ft. x 8 ft. walls.

After a wall was installed in the test frame, displacement transducers were attached to monitor and record the wall performance. The transducers measured overturning uplift at bottom of the wall (at each holddown), slip at the bottom of the wall and lateral displacement at the top of the wall (see Figure 1). The resisting load was measured directly by a load cell in line with the loading beam.

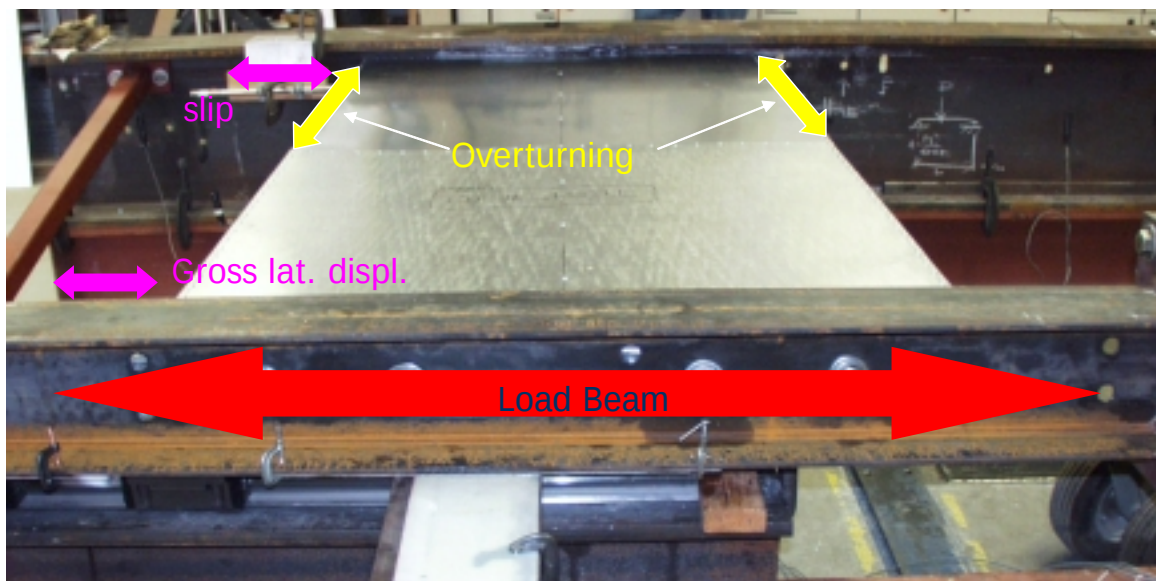


Figure 1. Position of displacement transducers

The reversed cyclic test procedure used in this program involved cycling the wall through a series of specified (per Table 5) increasing top of wall displacements (referred to as target displacements) up to 4 in. The cycling frequency was constant at 0.2 Hz (or 5 seconds per cycle). The monotonic tests were conducted by displacing the top of the wall to a maximum of 4 in. in one direction before returning the wall to its original state. The monotonic displacement was applied at a rate of 0.02 in/second.

Although the cyclic test procedure used in this project was similar to the procedure used to develop the 1997 UBC and 2000 IBC shear wall values (*Report No. LGSRG-1-97, "Additional Shear Wall Values for Light Weight Steel Framing," Santa Clara University, March 1997*), the following exceptions should be noted:

- There were no decreasing cycles following the first excursion at any target displacement.
- Only three cycles were executed at each target displacement.
- The cycling frequencies in previous tests were either 0.67 or 1.0 Hz (1.5 or 1.0 seconds per cycle, respectively).
- The maximum applied lateral displacement was 4 in. (66.7% more than the UBC/IBC prescribed inelastic drift limit of 2.5% for an 8 ft. wall height)

Table 5. Reversed cyclic test procedure

Target Displacement, in.	No. of Cycles
0.2	3
0.4	3
0.6	3
0.8	3
1.0	3
1.2	3
1.4	3
1.6	3

Target Displacement, in.	No. of Cycles
1.8	3
2.0	3
2.2	3
2.4	3
2.6	3
2.8	3
3.0	3
3.2	3

Target Displacement, in.	No. of Cycles
3.4	3
3.6	3
3.8	3
4.0	3

In both the reversed cyclic and monotonic tests, data was sampled and recorded at a minimum rate of 50 samples per seconds (i.e. one sample per 0.02 seconds).

TEST RESULTS

The behaviors of the tested walls are shown in Figures 2 through 8 and the modes of failure are described in Table 6.

Table 6. Observed modes of failure

Test Specimens	Mode of Failure
1, 2 (Figure 2)	Damage to OSB at the fasteners: fastener pulled through sheathing thickness and fractures panel edges.
3, 4 (Figure 3)	Damage to OSB at the fasteners: fastener pulled through sheathing thickness and fractures panel edges. In some instances, the fasteners were pulled out of the framing.
6, 7 (Figure 4)	Damage to OSB at the fasteners and local buckling in the chord stud at the web punchout. OSB damage from fastener pulling through sheathing thickness and fracturing panel edges.
8, 9 (Figure 5)	Damage to OSB at the fastener and shear failure of the holdown screws. OSB damage from fastener pulling through sheathing thickness and fracturing panel edges.
10, 11 (Figure 6)	Screws pulled out of the sheet steel along the horizontal joint.
12, 13, 14, 15, 16, 17, 20, 21 (Figure 7)	Fracture of GWB at the fasteners along the "un-papared" edges. With increased displacement, bearing of the panels at the horizontal joint was evident. Bearing at the horizontal joint resulted in panel buckling.
18, 19 (Figure 8)	Fracture of GWB at the fasteners along the "un-papared" and papared edges. There was also pull through of the screw heads at the papared edges (horizontal edges).



Figure 2. Observed behavior in Tests 1 and 2

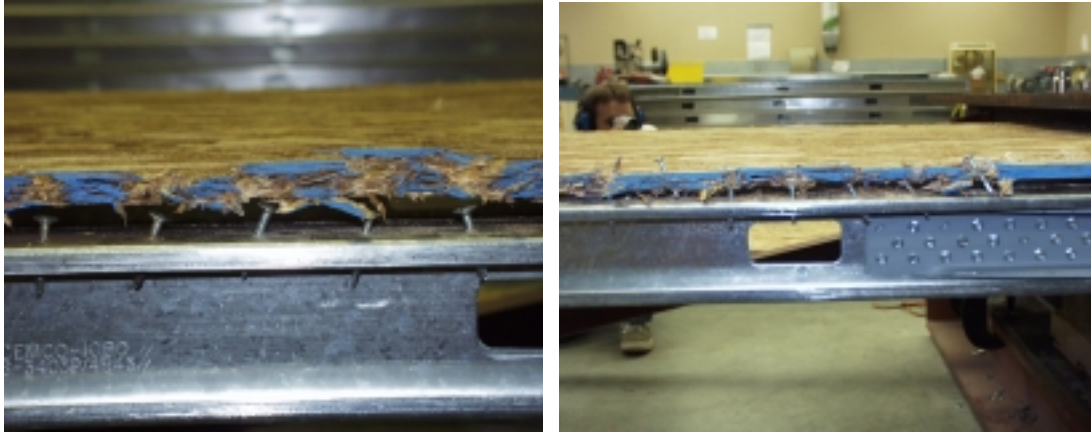


Figure 3. Observed behavior in Tests 3 and 4

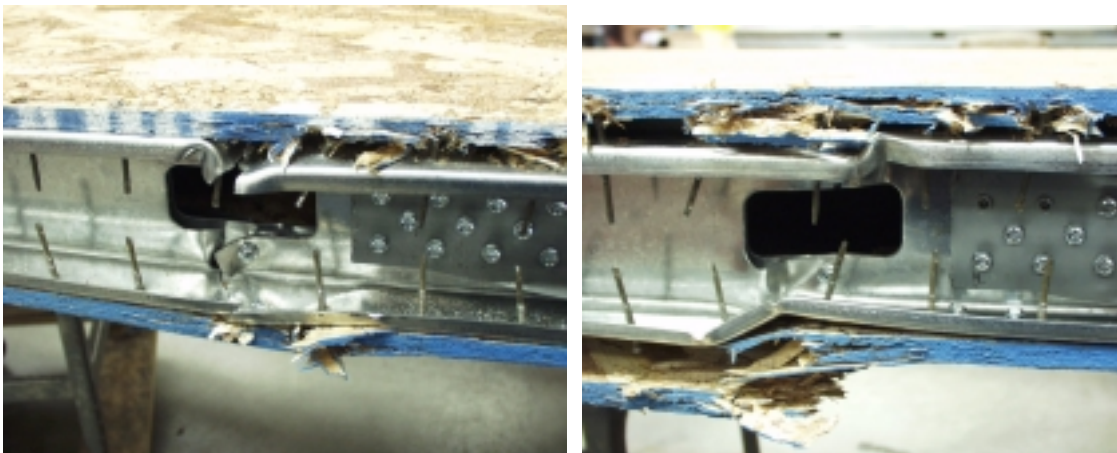


Figure 4. Observed behavior in Tests 6 and 7



Figure 5. Observed behavior in Tests 8 and 9

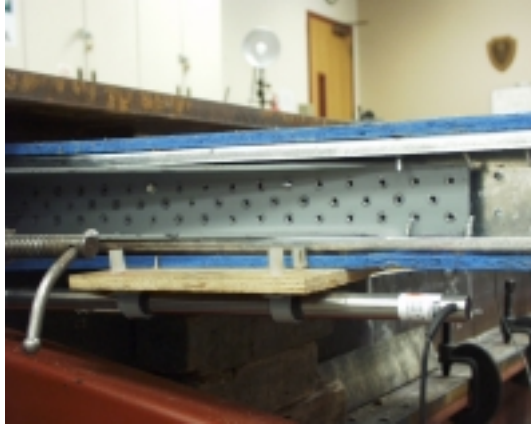


Figure 5 continued. Observed behavior in Tests 8 and 9

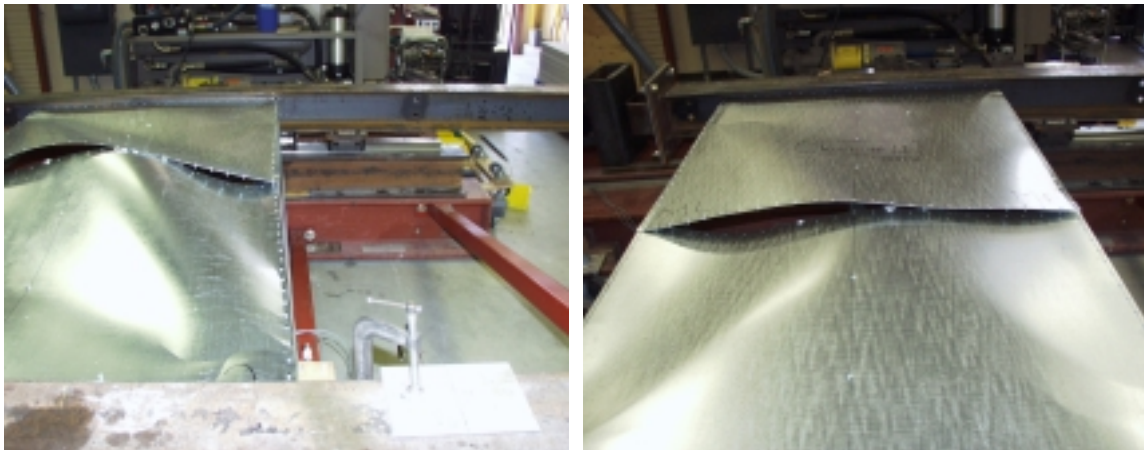


Figure 6. Observed behavior in Tests 10 and 11



Figure 7. Observed behavior in Tests 12, 13, 14, 15, 16, 17, 20 and 21



Figure 8. Observed behavior in Tests 18 and 19

The measured responses of all the tested walls are given in Appendix A. The response curves in Appendix A give relationships between the measured resisting lateral load and net lateral displacement for each wall. As used in this report, net lateral displacement is defined as follows:

$$\text{Net lateral displacement, } \Delta_{\text{net}} = \Delta_{\text{gross}} - \Delta_{\text{rotation}} - \Delta_{\text{base slip}}$$

Δ_{gross} = Gross lateral displacement

Δ_{rotation} = Overturning (rigid body) lateral displacement

$\Delta_{\text{base slip}}$ = Slip at the base of the wall

Evident in the hysteresis curves from the reversed cyclic tests (Figures A1-A10) is a degrading strength associated with consecutive cycles at a defined target displacement and a decreasing “initial” stiffness as the target displacement was increased. As an aid for visualization of the test data based on strength only, the resisting load time histories for the reversed cyclically tested walls are presented in Appendix B.

In Table 7, the measured material properties of the steel used in this project are reported. Coupons for the framing members were taken from the member web and tested. In all cases the measured yield and tensile strengths were greater than the

specified minimum strength values. The ratio of tensile to yield strengths and percent elongations (for a 2-in. gage length) are given in Table 7. The measured uncoated thicknesses were almost identical to the nominal mil thicknesses.

Table 7. Measured material strength¹ and thickness

Member/Component	Yield Strength, Ksi	Tensile/Yield Strength Ratio	Elongation, %	Uncoated Thickness, mils (= 1/1000 in.)
33 mil studs	46	1.13	36	33
54 mil studs	59	1.14	27	54
68 mil studs	56	1.18	24	68
33 mil track	48	1.17	32	32
54 mil track	59	1.14	24	52
68 mil track	66	1.12	23	68
27 mil sheet steel	46	1.20	36	28
¹ Per ASTM A 370				

DISCUSSION OF TEST RESULTS

Overall, the OSB wall tests demonstrated that No. 8 screws in 54-mil framing and No. 10 screws in 68-mil framing provided for a ductile mode of failure at the connection. For the doubled-sided (sheathed each side) walls, the demands on the chord studs exceeded the capacity of the studs in the 54-mil framed walls (Figure 4). When the double-sided walls were framed with 68-mil studs, the chord studs capacity was sufficiently high to prohibit stud failure but the demand on the screws attaching the holdown to the chords exceed the capacity of the screws (Figure 5). Premature failure in these elements prevents development of the sheathing capability and limits the efficiency and effectiveness of the wall.

In the sheet steel walls, diagonal shear buckling accompanied by diagonal tension resulted in high demands on a few screws at the center of the mid-height horizontal joint. These high demands caused the panel to unzip along the joint before the sheet steel could develop its “full strength.”

Failure in the GWB monotonic tests was characterized by breaking of the wallboard at the fasteners along the “un-papered” edges and screw pull-through along the “papered” edges of the wallboard. These modes of failure are consistent to what has been observed and recorded in previous AISI testing at Santa Clara University. The large displacement capacity and ductility of GWB shear walls appear to be a result of bearing of the panel edges at the horizontal joint.

Although, it was not the intent in this report to provide a detailed interpretation of the test data based on any specific acceptance criteria or code provisions, the following general interpretation is offered based on current building codes. In both the 1997 UBC and 2000 IBC, the tabulated design values for cold-formed steel framed shear walls were based on the 2nd cycle target displacement peak load envelope, as illustrated in Figure 9. The cycles and loads associated with this envelope are highlighted in Figure 10. A review of the response curves in Appendix A shows that 2.5 times the wall strength defined at ½-in. of net lateral displacement is always greater than the maximum resisting load. As such, the maximum strength values given in Tables 8 through 11 may be taken as the nominal strength of the walls.

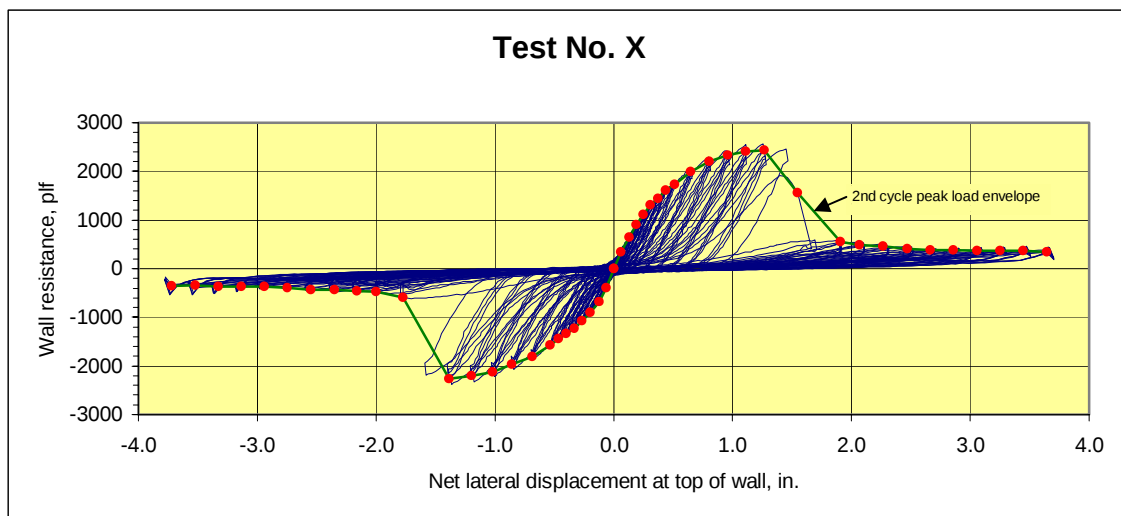


Figure 9. Second cyclic peak load envelope

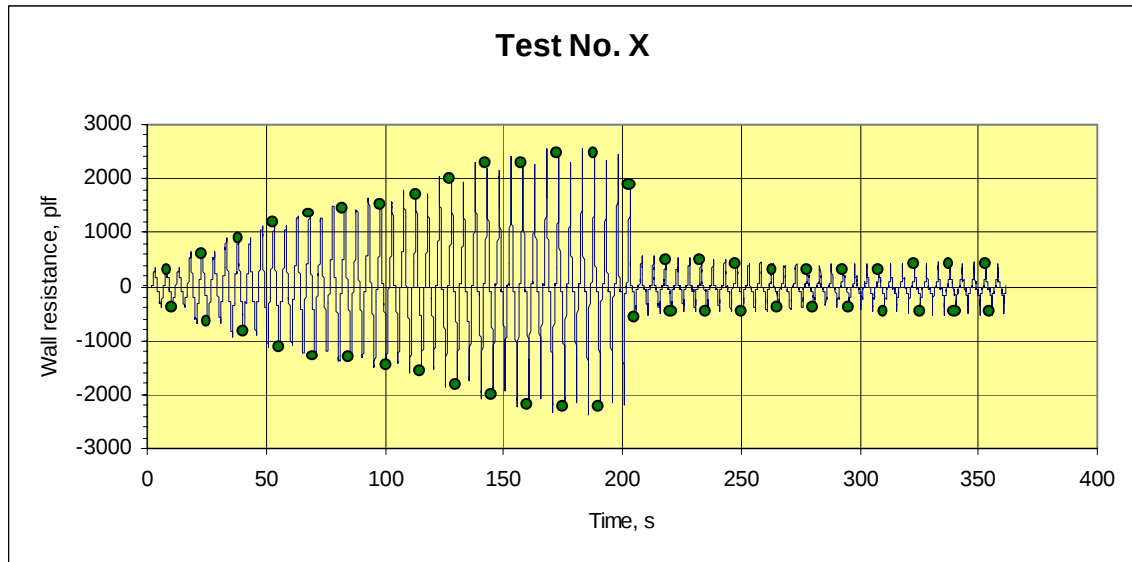


Figure 10. Second cycle +/- peak loads

Using the envelope curves (2nd cycle peak) for the reversed cyclic tests (as illustrated in Figure 9) and the resisting load versus net lateral displacement curves for the monotonic tests, the maximum resisting loads and the corresponding displacements for each wall are given in Tables 8 through 11. The drift ratio (story angle) given in these tables is the drift displacement divided by the wall height.

Table 8. Measured performance¹ of 7/16-in. OSB^{2,3} shear walls (**sheathed one side**) framed with 54- and 68-mil steel

Test No.	Framing ⁴	Screw Size	Resistance			Mode of Failure
			Max. Load ⁵ , Plf	Drift @ Max. Load, in.	Drift Ratio (Story Angle) x 10 ³ , rad	
1	350S162-54 studs	No. 8	2349	1.375	14.327	Fracture of OSB along panel edges at the fasteners
2	350T125-54 track		2356	1.397	14.552	
3			2961	1.290	13.436	
4	350S162-68 studs 350T125-68 track	No. 10	3201	1.283	13.370	Initiated with fracture along panel edges at the fasteners and screw pullout from framing

¹ Reversed cyclic loading

² Rated sheathing exposure 1.

³ Sheathing attached with screws at 2 inches on center at the supported panel edges and 12 on center inches in the panel field.

⁴ Stud and track were Grade 50 steel. Framing fasteners: No. 10 pancake head self-drilling screws.

⁵ Based on target displacement 2nd cycle envelope

Table 9. Measured performance¹ of 7/16-in. OSB^{2,3} shear walls (**sheathed each side**) framed with 54-mil and 68-mil steel

Test No.	Framing ⁴	Screw Size	Resistance			Mode of Failure
			Max. Load ⁵ , Plf	Drift @ Max. Load, in.	Drift Ratio (Story Angle) x 10 ³ , rad	
6	350S162-54 studs 350T125-54 track	No. 8	4173	0.798	8.313	Buckling of chord stud
7			4181	0.977	10.182	
8	350S162-68 studs 350T125-68 track	No. 10	5205	1.084	11.291	Fracture of OSB along panel edges at fasteners (there also failure of some of the holdown screws)
9			5283	0.862	8.978	Initiated with fracture along panel edges at fasteners and ended with failure of all screws in one holdown.

¹ Reversed cyclic loading

² Rated sheathing exposure 1.

³ Sheathing attached with screws at 2 inches on center at the supported panel edges and 12 on center inches in the panel field.

⁴ Stud and track were Grade 50 steel. Framing fasteners: No. 10 pancake head self-drilling screws.

⁵ Based on target displacement 2nd cycle envelope

Table 10. Measured performance¹ of 27-mil sheet steel^{2,3} shear walls with simple lap shear connections at the adjoining panel edges

Test No.	Framing ⁴	Screw Size	Resistance			Mode of Failure
			Max. Load ⁵ , Plf	Drift @ Max. Load, in.	Drift Ratio (Story Angle) x 10 ³ , rad	
10	350S162-33 studs 350T125-33 track	No. 8	825	1.064	11.080	Pullout of screws from the holding sheet in the lap (horizontal) joint.
11			749	0.648	6.745	

¹ Reversed cyclic loading

² Grade 33 steel.

³ Sheathing attached with screws at 2 inches on center at the supported panel edges (including lap joint) and 12 on center inches in the panel field.

⁴ Stud and track were Grade 33 steel. Framing fasteners: No. 8 modified truss head self-drilling screws.

⁵ Based on target displacement 2nd cycle envelope

Comparing the results in Tables 8 and 9, it can be seen that the strength of the double-sided shear walls was 70 to 75 percent more than that of the single-sided wall.

Table 11. Measured performance¹ of ½-in. gypsum wallboard (GWB) shear walls^{2, 3}

Test No. ⁴	Panel Attachment	Screw Spacing (Supported edge/field), in./in.	Resistance			Mode of Failure
			Max. Load, plf	Drift @ Max. Load, in.	Drift Ratio (Story Angle) x 10 ³ , rad	
12	Sheathing one face of the wall (2) 4 ft. x 8 ft. sheet no blocking @ horizontal joint	4/4	170	1.274	13.275	Fracture of the GWB along the "un-papered" edge
13			162	1.584	16.505	
14			118	2.279	23.735	
15		7/7	115	1.669	17.389	
16			99	1.756	18.292	
17			86	1.473	15.342	
18	Sheathing one face of the wall	4/12	299	1.065	11.095	Initiated with fracture along the "un-papered" edge; this was followed by screw heads pulling through thickness along the horizontal joint
19	(2) 4 ft. x 8 ft. sheet w/ 2-in. 33 mil strap @ horizontal joint		291	1.000	10.416	
20	Sheathing one face of the wall	4/12	103	1.837	19.140	Fracture of the GWB along the "un-papered" edge
21	(2) 4 ft. x 8 ft. sheet no blocking @ horizontal joint		81	1.698	17.686	

¹ Monotonic loading.
² ½-in. Type X GWB attached with No. 6 x bugle head screws
³ 8 ft. x 8 ft. walls.
⁴ Framing: 350S162-33 studs and 350T125-33 track Grade 33 steel. Framing fasteners: No. 8 x modified truss head self-drilling screws.

It is also evident that in all the tests, the inelastic drift displacements were less than the code permitted 2.5% of wall height (2.4 in.) inelastic drift for structures that utilize these types of lateral systems. Finally, as stated previously, the interpreted maximum load values given in Tables 8 through 11 are consistent with the interpretation used in development of the 1997 UBC and 2000 IBC nominal strength values for cold-formed steel-framed shear walls.

RECOMMEDATIONS FOR FURTHER WORK

1. Based in the modes of failure in tests 6,7, 8 and 9, it seems probable that if chord buckling and holdown failure are prevented, the capacity of the double-sided wall

may be closer to double the capacity of the single-sided wall. Additional testing is proposed to validate this conclusion.

2. Given the current state of building codes and in anticipation of probable future changes, a committee should be established to develop a consistent set of acceptance criteria for interpretation of the test data for design.

3. Given the usual limited number of test used to develop design data, a statistically justifiable method should be formulated for assignment of design values based on testing.

SUMMARY and CONCLUSIONS

A series of 20 full-scale tests were carried out to evaluate the capacity of cold-formed steel-framed shear wall with configurations different that those permitted in current building codes. The derived test data provides a basis for expanding the current design options and should allow for more efficient design in cold-formed steel.

APPENDIX A

[Resisting Load vs. Net Lateral Displacement Response Curves]

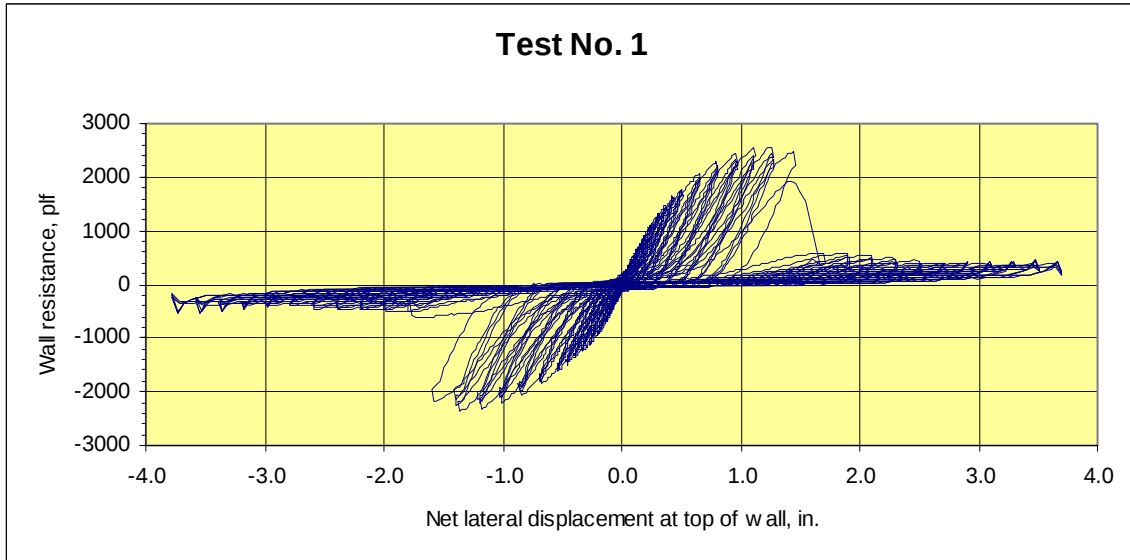


Figure A1. Response curve for Test 1

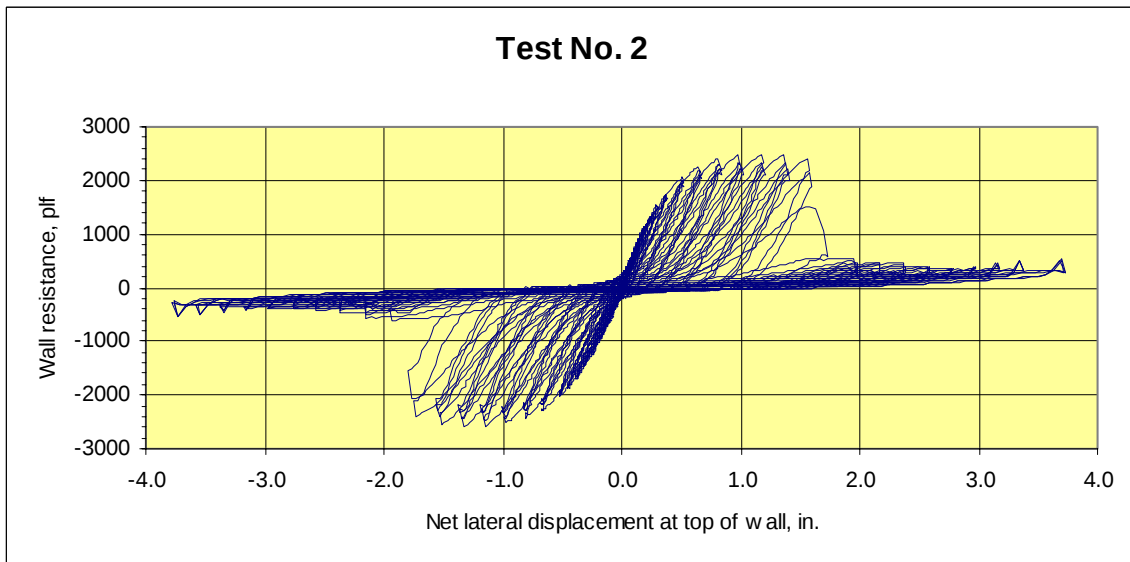


Figure A2. Response curve for Test 2

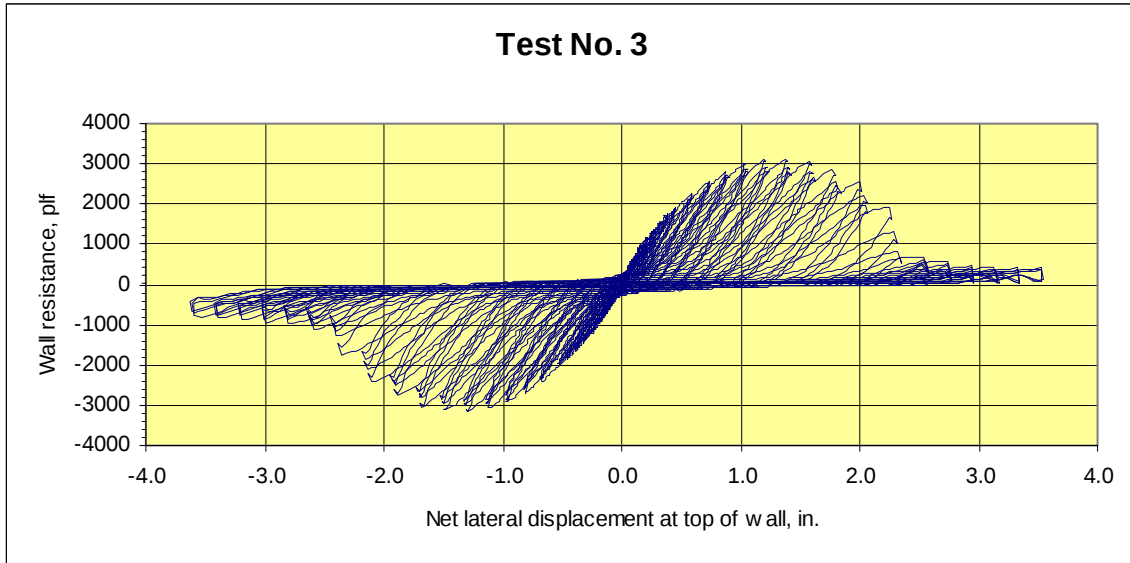


Figure A3. Response curve for Test 3

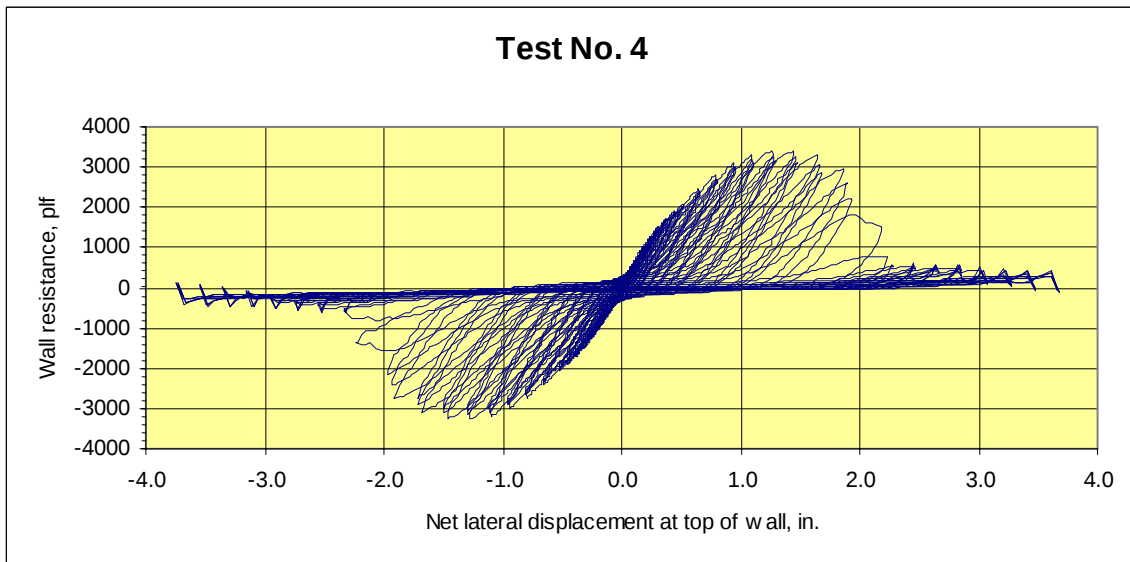


Figure A4. Response curve for Test 4

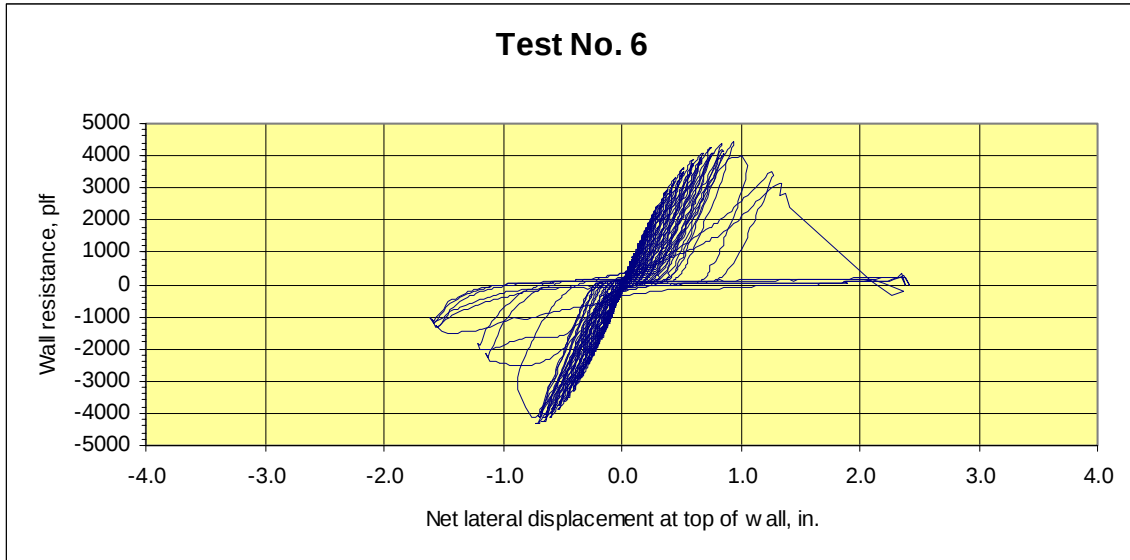


Figure A5. Response curve for Test 6

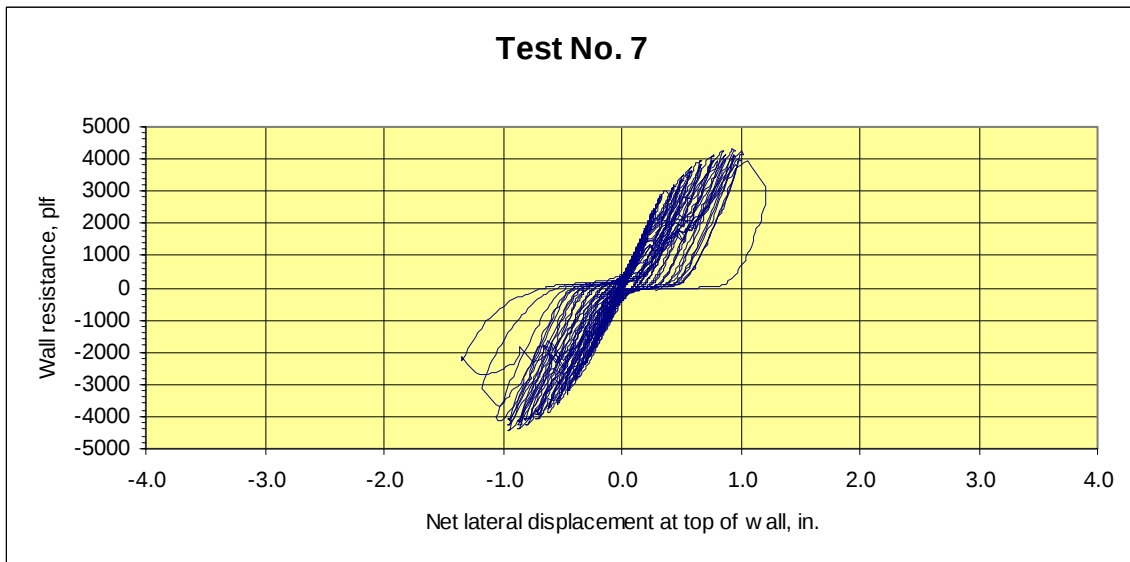


Figure A6. Response curve for Test 7

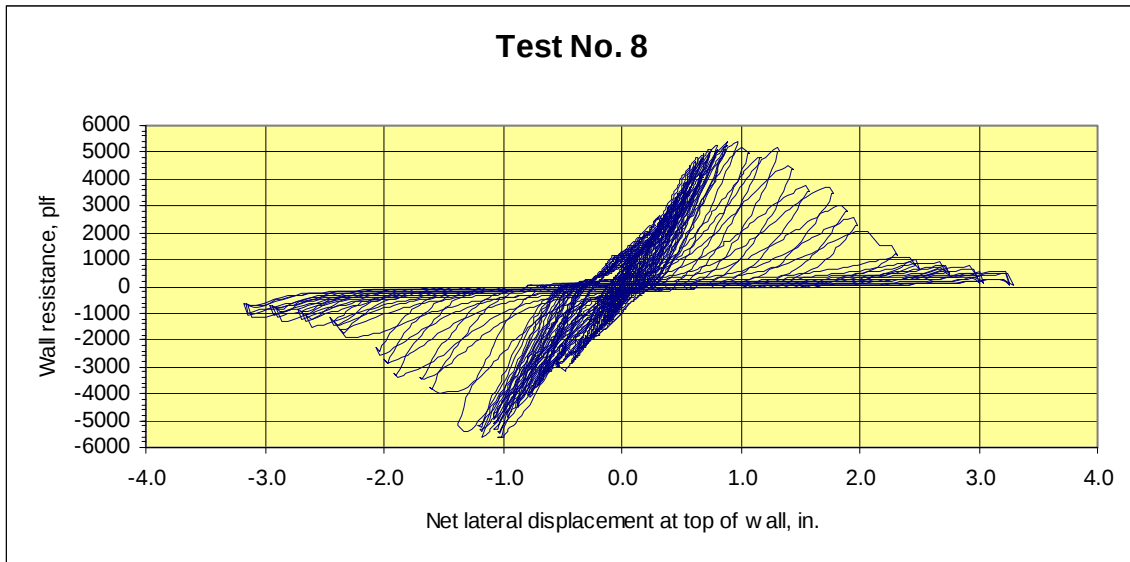


Figure A7. Response curve for Test 8

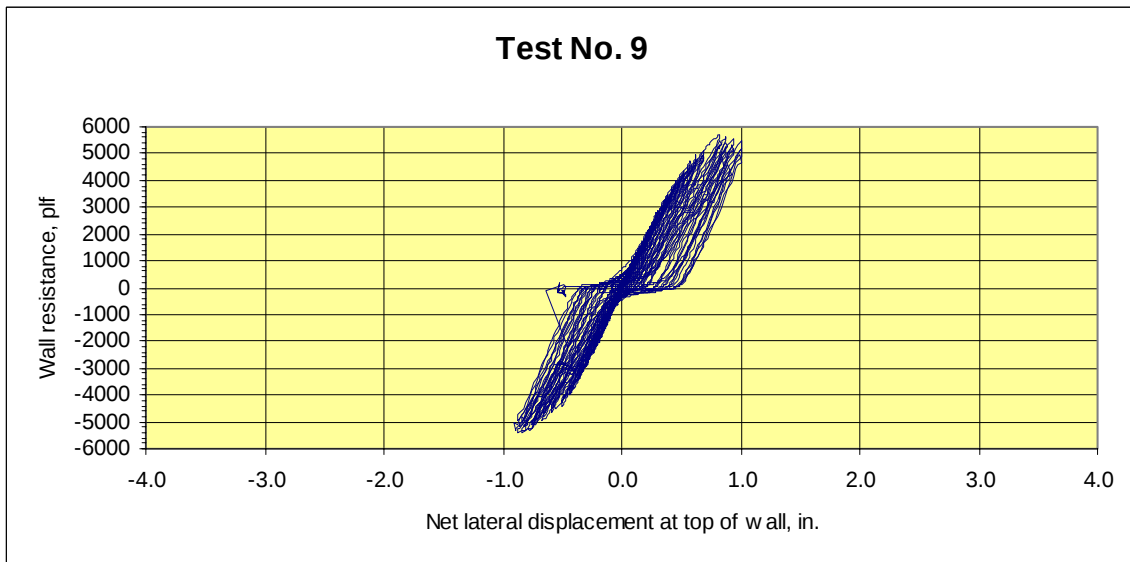


Figure A8. Response curve for Test 9

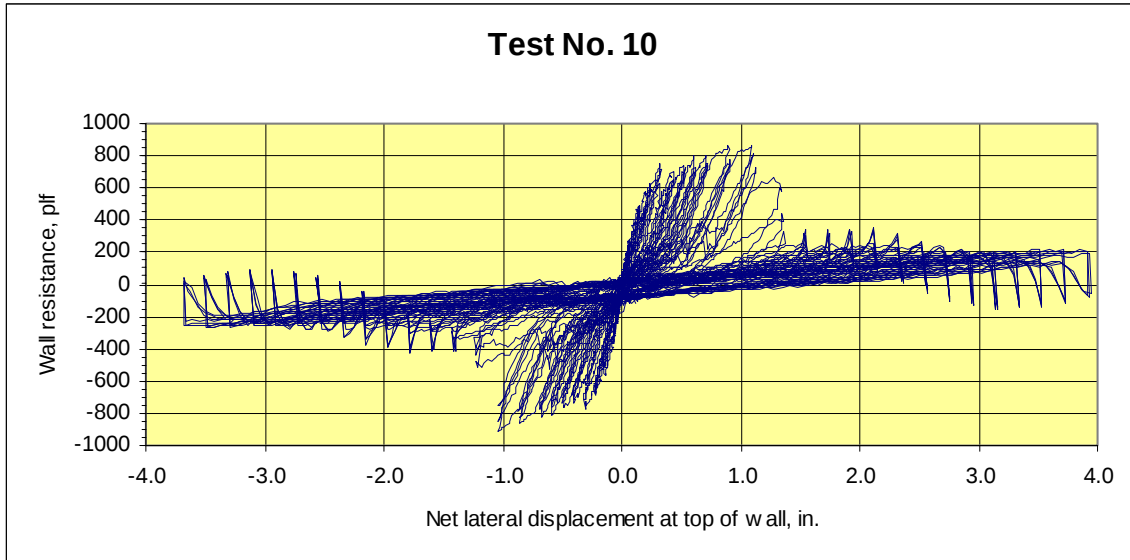


Figure A9. Response curve for Test 10

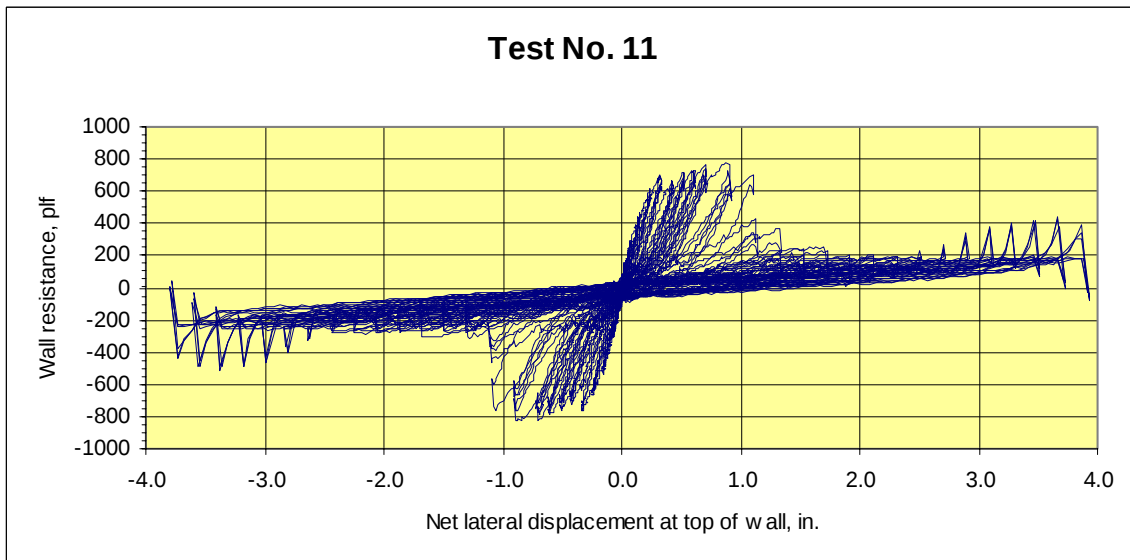


Figure A10. Response curve for Test 11

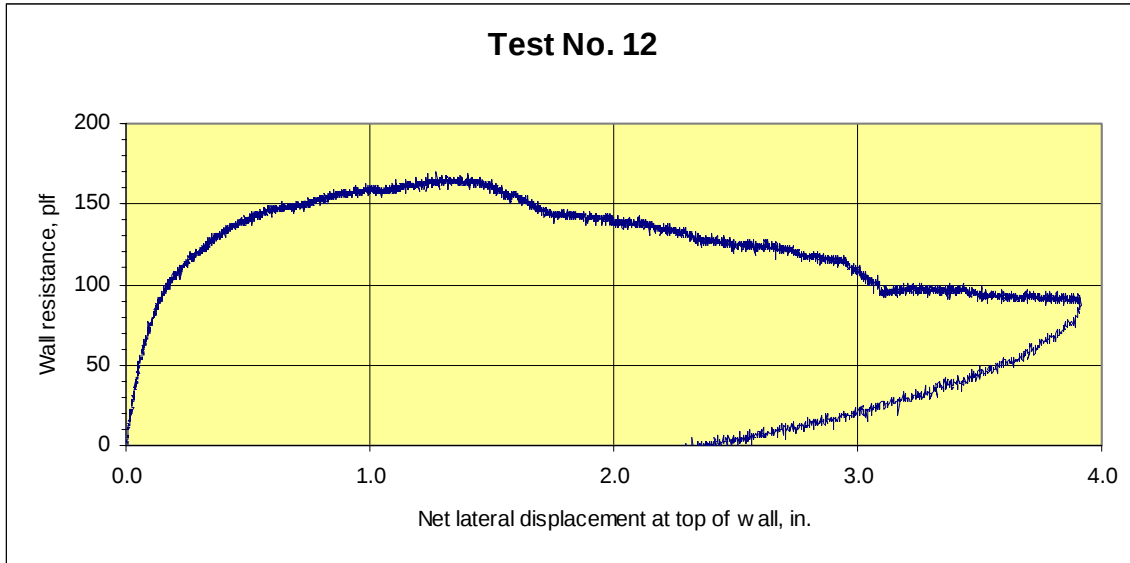


Figure A11. Response curve for Test 12

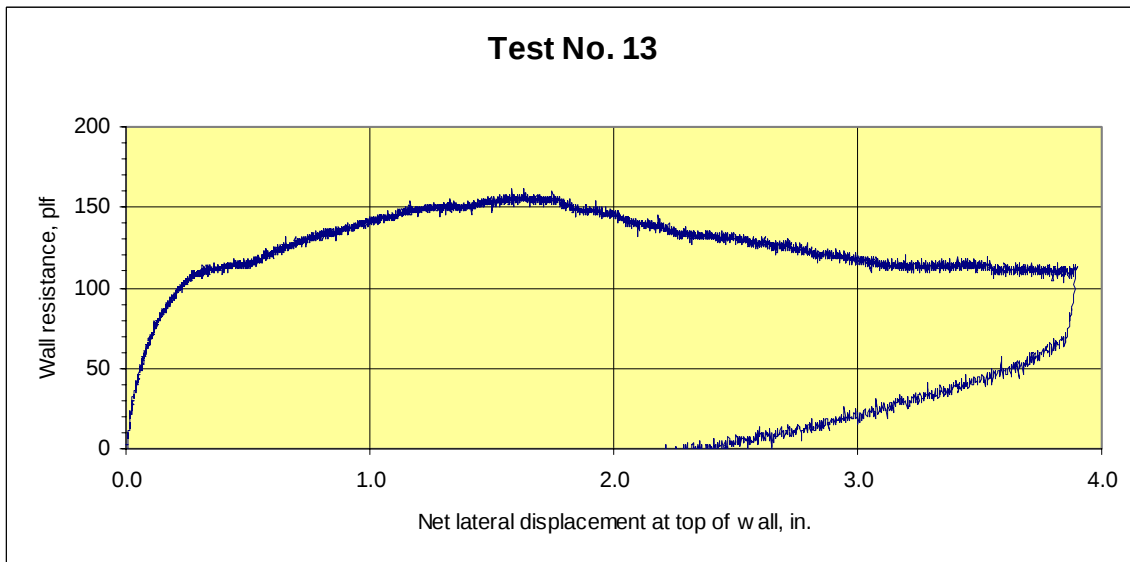


Figure A12. Response curve for Test 13

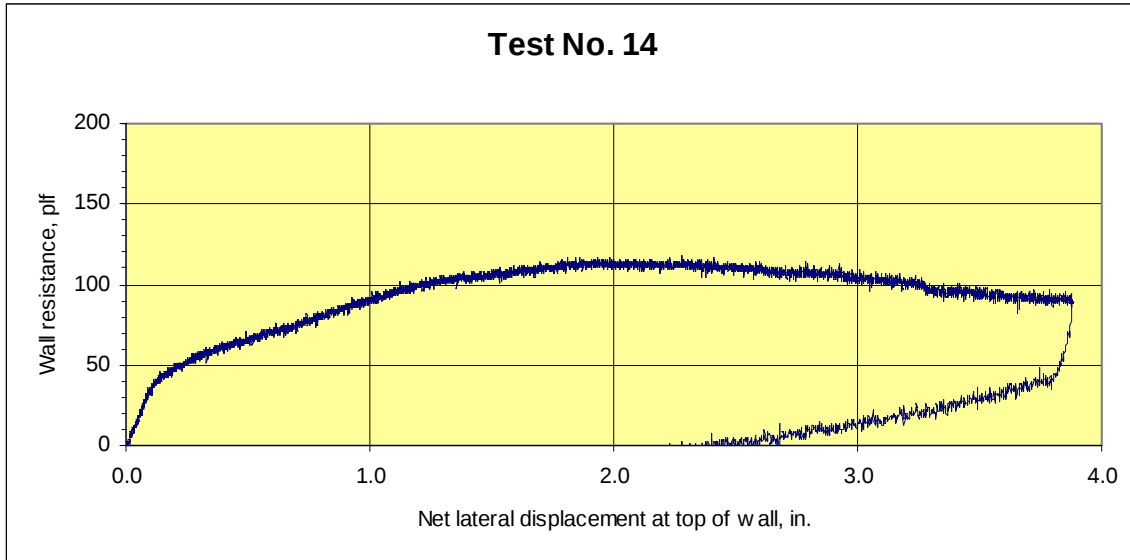


Figure A13. Response curve for Test 14

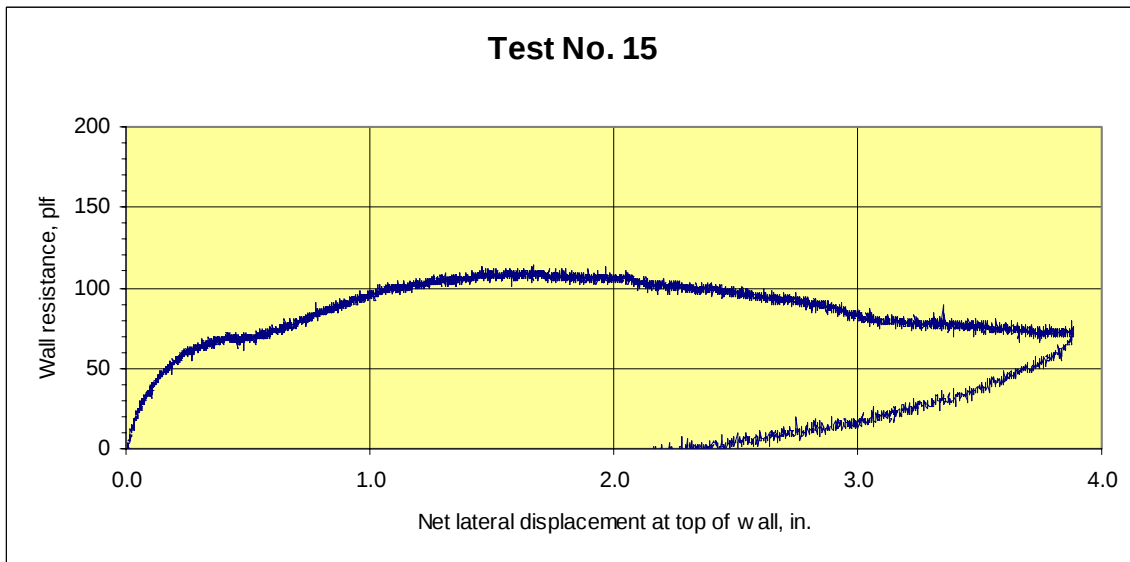


Figure A14. Response curve for Test 15

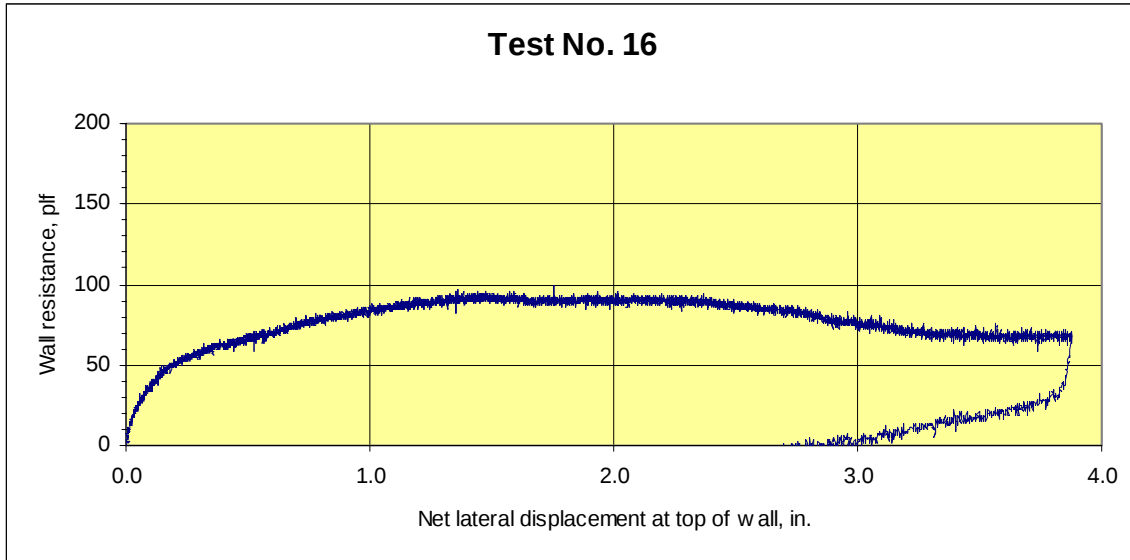


Figure A15. Response curve for Test 16

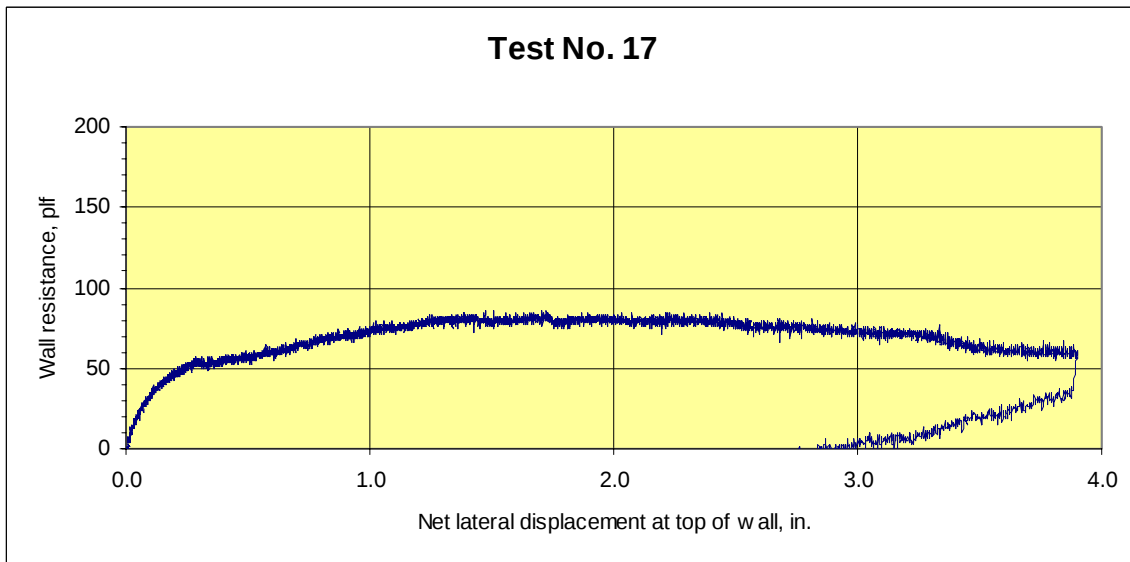


Figure A16. Response curve for Test 17

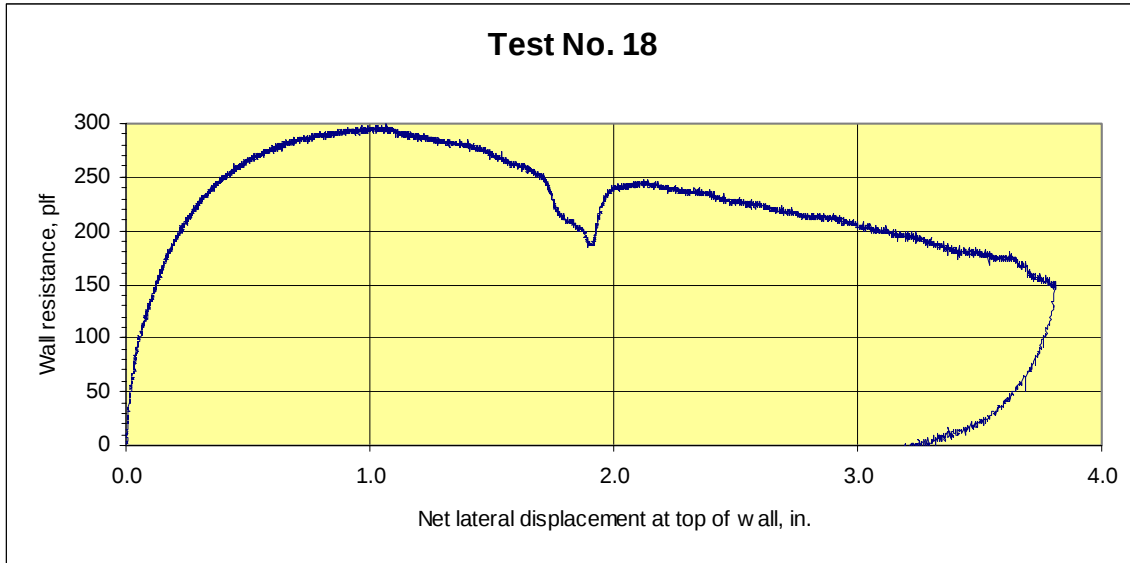


Figure A17. Response curve for Test 18

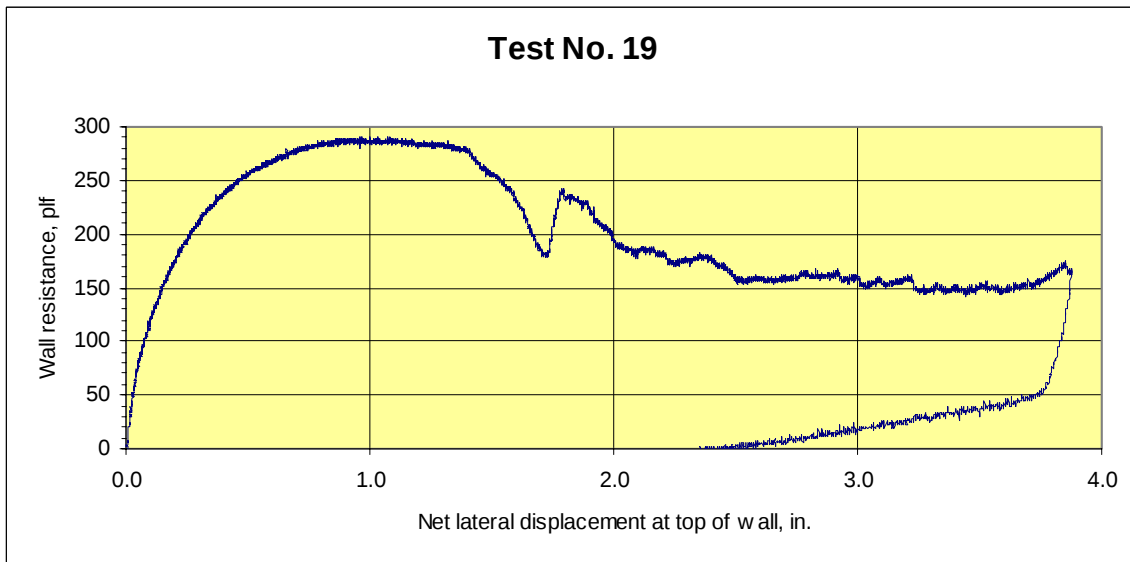


Figure A18. Response curve for Test 19

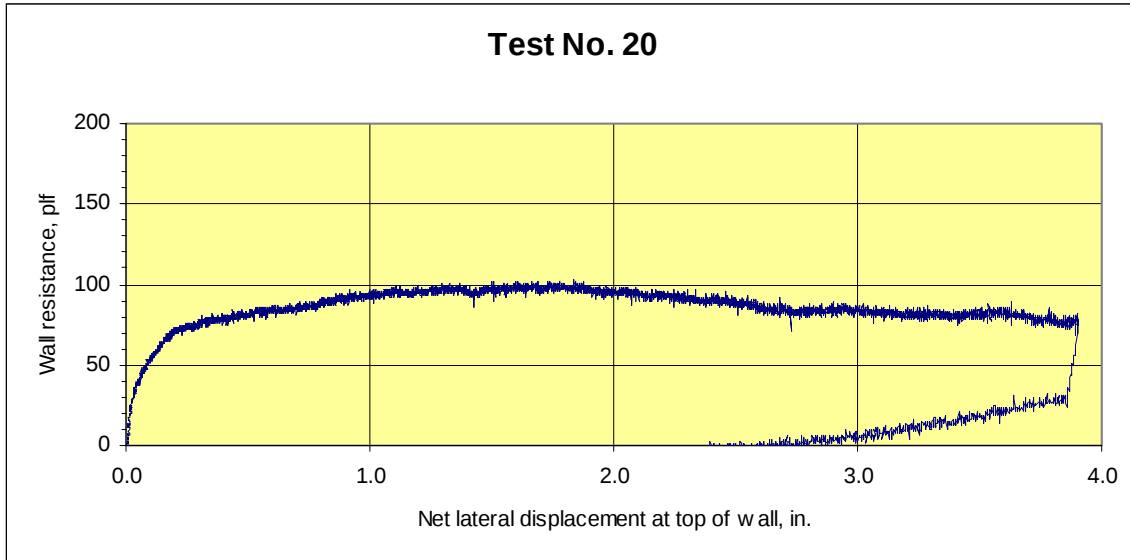


Figure A19. Response curve for Test 20

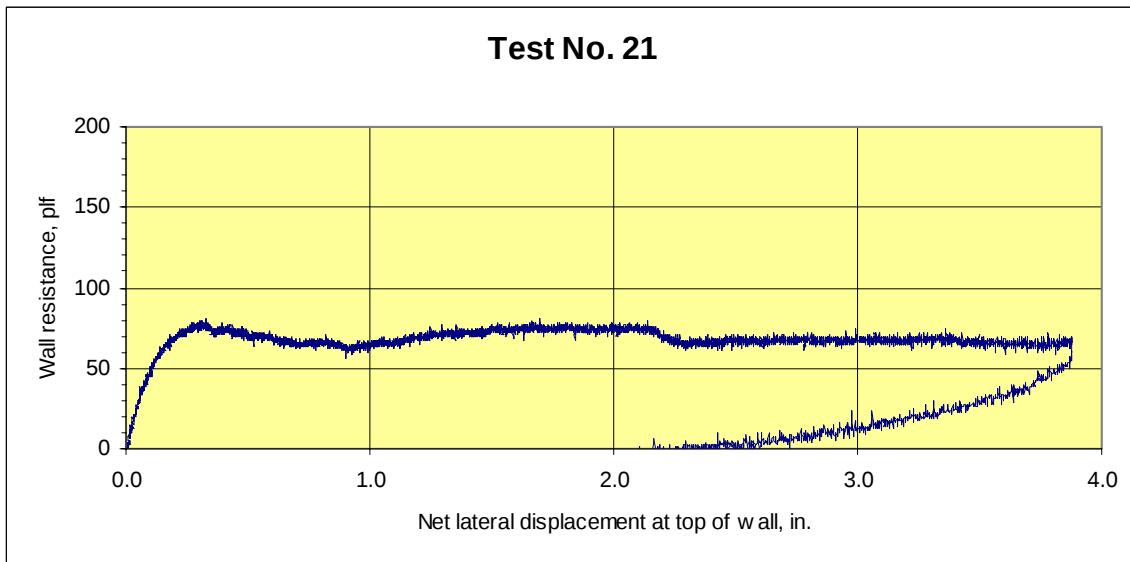


Figure A20. Response curve for Test 21

APPENDIX B

[Resisting Load Time History Curves]

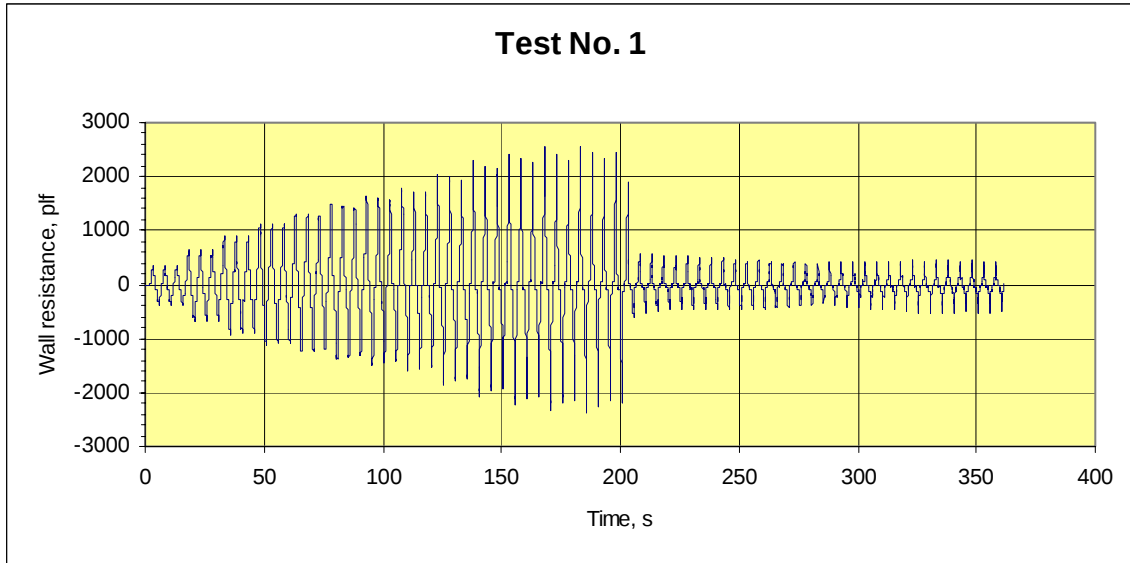


Figure B1. Resisting load history Test 1

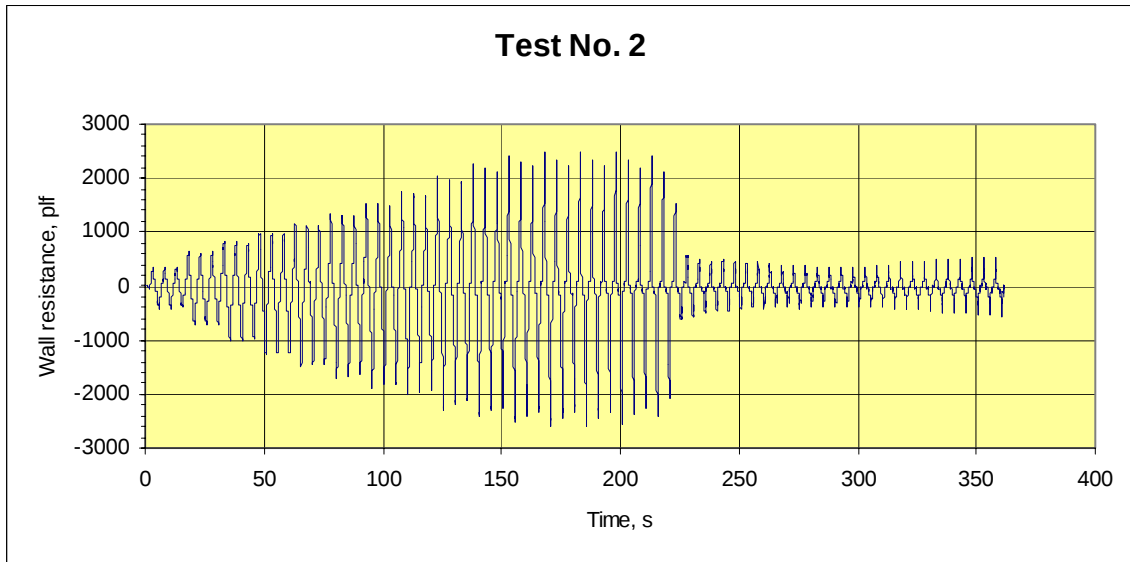


Figure B2. Resisting load history Test 2

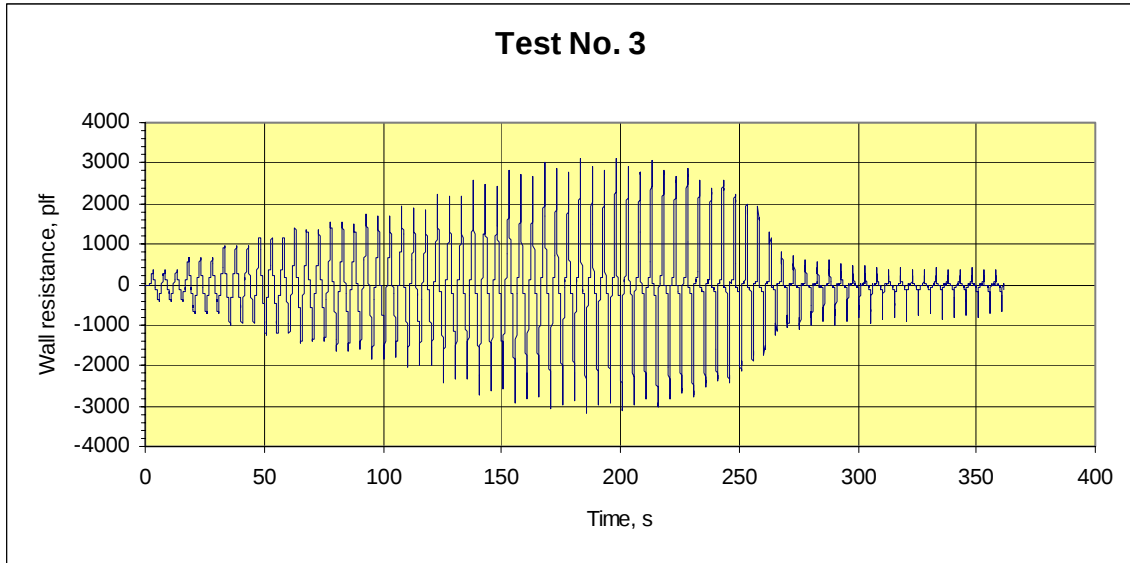


Figure B3. Resisting load history Test 3

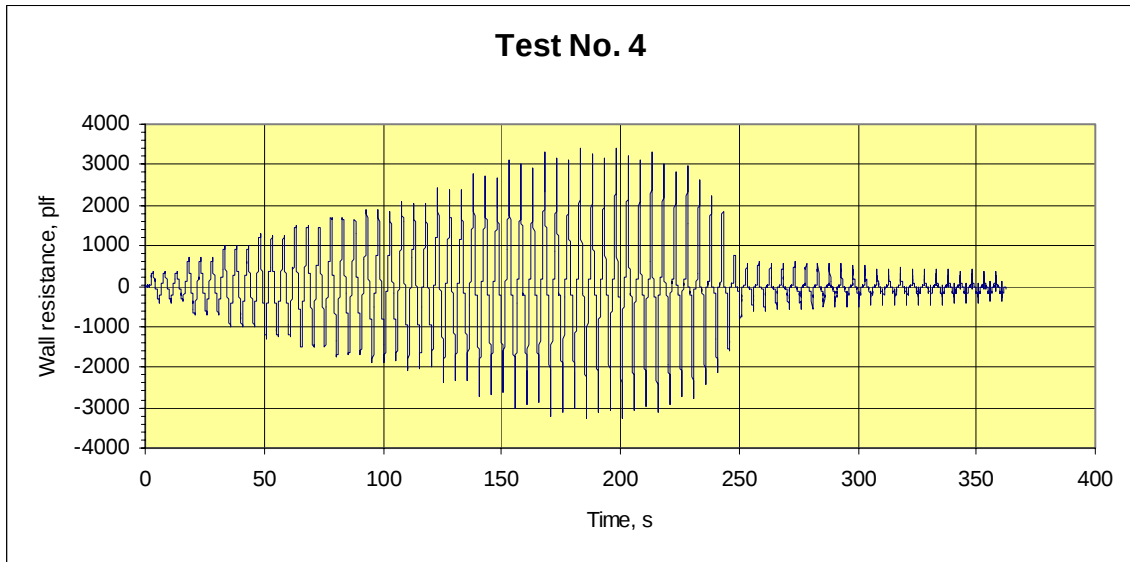


Figure B4. Resisting load history Test 4

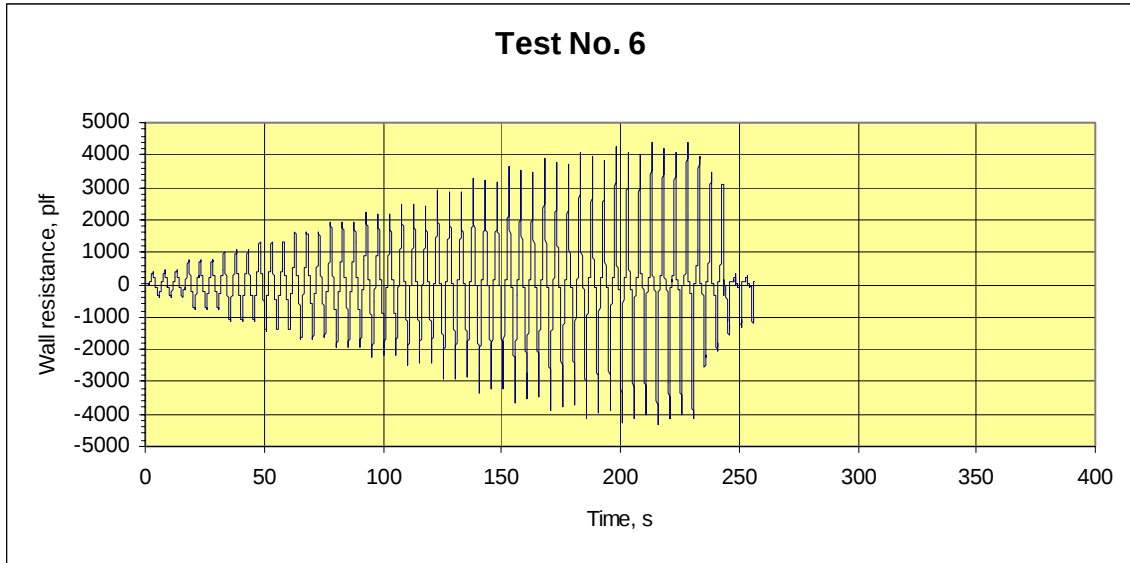


Figure B5. Resisting load history Test 6

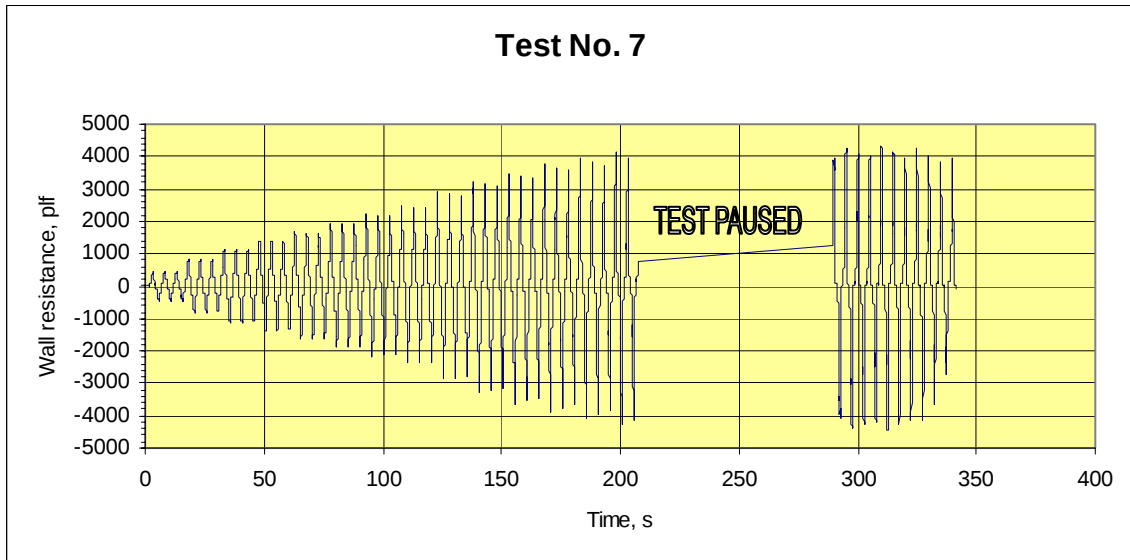


Figure B6. Resisting load history Test 7

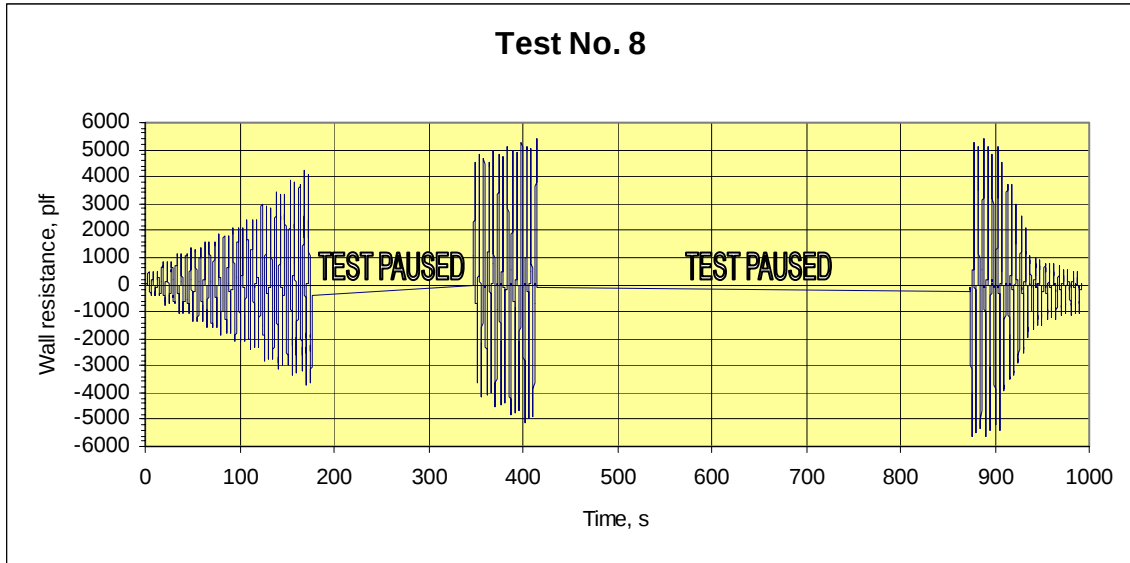


Figure B7. Resisting load history Test 8

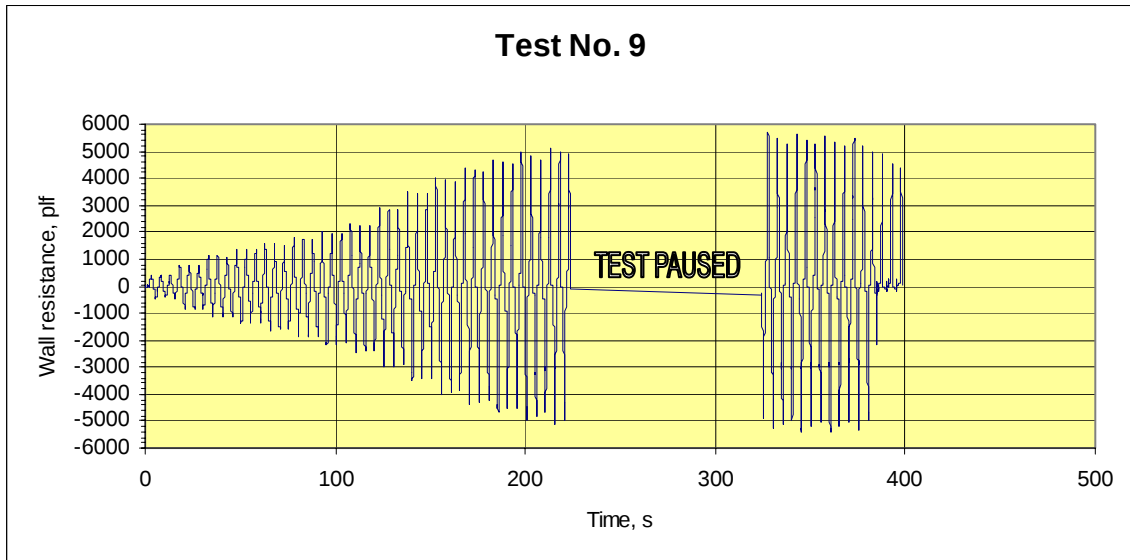


Figure B8. Resisting load history Test 9

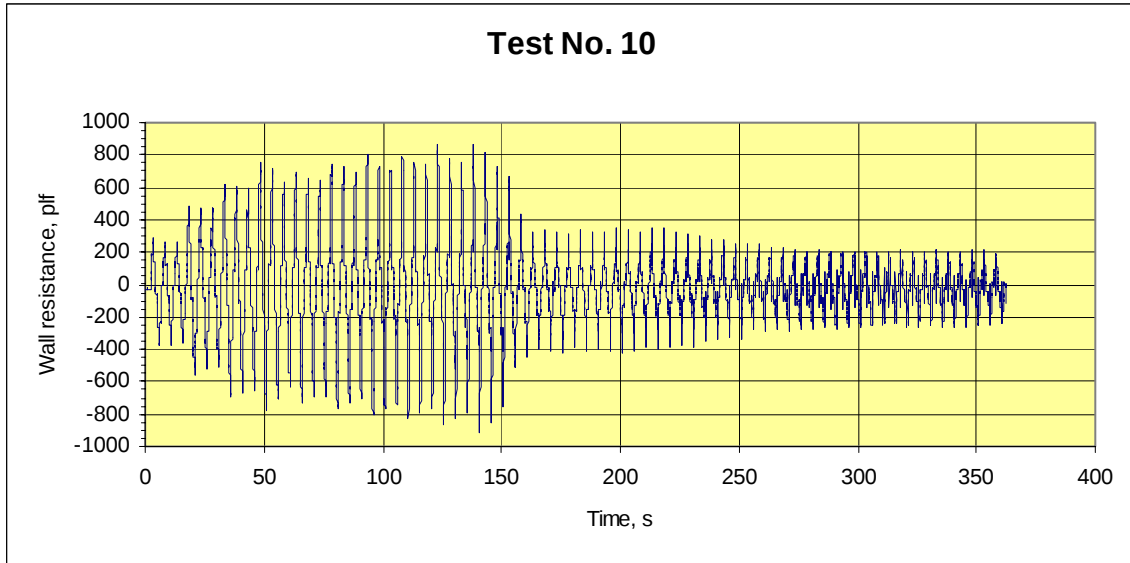


Figure B9. Resisting load history Test 10

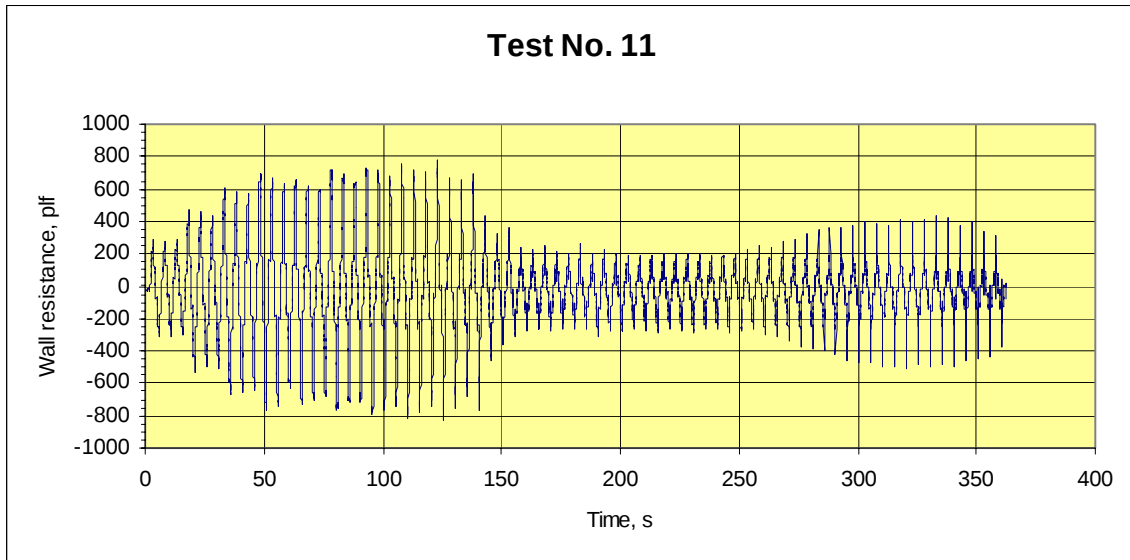


Figure B10. Resisting load history Test 11



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