Perforated Shear Walls with Conventional and Innovative Base Restraint Connections

By

Jay Crandell, P.E.[1]
William Freeborne[2]
Shawn McKee[1]

ABSTRACT

Ideally, the residential construction industry wants to build stronger, safer buildings that can withstand hurricane and earthquake loads while at the same time using material and labor resources more efficiently. Shear walls are a primary lateral force resisting assembly in conventionally wood-framed construction. This paper reviews the current status of shear wall design using the perforated shear wall method. Past perforated shear wall research is discussed. The findings from this phase of research support the use of the perforated shear wall method for 2 ft. (0.61 m) narrow wall segments, 6 ft. (1.83 m) on center anchor bolt spacing, and nailed sole plate anchorage.

KEYWORDS: anchorage, design, energy dissipation, force-displacement, perforated shear wall method, stiffness, uplift

1. INTRODUCTION

Between one and two million new homes are built in the United States each year, predominantly with wood framing. For this reason, efficient utilization of our lumber supply is important. Ideally, the residential construction industry wants to build stronger, safer buildings that can withstand hurricane and earthquake loads while at the same time using material and labor resources more efficiently. In order to accomplish this goal, the actual performance of these structures must be better understood from an engineering standpoint.

Shear walls are a primary lateral force resisting assembly in conventionally wood-framed construction. Traditional shear wall design requires individual sheathed wall sections to be restrained against overturning. Design of exterior shear walls containing openings, for windows and doors, involves the use of multiple shear wall segments and is required to be fully sheathed and have overturning restraint supplied by mechanical anchors. The design capacity of shear walls is assumed to be equal to the sum of the capacities for each full height shear wall segment. Sheathing above and below openings is typically not considered to contribute to the overall performance of the wall.

The traditional method of design described above is significantly different than wall bracing methods used historically in conventional residential construction in the United States. It is also more expensive than conventional construction while providing greater strength. However, there are significant opportunities to optimize this design process so that both safety and economy are achieved through more accurate design approaches. This report is a continuation of an effort to develop, confirm, and enhance such an approach. The ultimate goal is to provide optimum value in both safety and economy for housing construction in all wind and seismic areas.

An alternate empirical-based approach to the design of shear walls with openings is the perforated shear wall method that appears in the *Standard Building Code* [1] and the *Wood*

[1] Structures and Materials Division, NAHB Research Center, Inc., Upper Marlboro, MD, USA, 20774
[2] Division of Affordable Housing Research and Technology, U.S. Department of Housing and Urban Development, Washington, DC, USA, 20464

Frame Construction Manual for One and Two Family Dwellings [2]. The perforated shear wall method consists of a series of simple empirical equations used for the design of shear walls containing openings. When designing for a given load, shear walls resulting from this method will generally have a reduced number of overturning restraints than a similar shear wall constructed with multiple traditional shear wall segments. The inferred performance will be achieved due to the accuracy of the method. Only when strength demands exceed the capabilities of the perforated shear wall method will the more traditional engineering approach be more cost effective and desirable.

2. BACKGROUND

Yasumura and Sugiyama conducted tests studying one-third scale monotonic racking tests of wood stud, plywood sheathed shear walls with openings [3][4]. The loads required to displace the wall at a shear deformation angle of 1/60, 1/75, 1/100, 1/150, and 1/300 were monitored. The shear deformation angle is defined as displacement of the top of the wall minus the bottom of the wall divided by the total height.

Sugiyama defined *r*, the sheathing area ratio, in order to classify walls based on the amount of openings a wall contains. This value is determined by the ratio of the area of openings to the area of the wall with full height sheathing to the total length of the wall. The sheathing area ratio, *r*, is defined as

$$r = \frac{1}{1 + \frac{A_o}{H \Sigma L_i}} \qquad \textbf{(Eq. 1)}$$

where:

A_o = total area of openings
H = height of the wall
L_i = length of the full height wall segment

Sugiyama and Matsumoto determined an empirical equation to relate shear capacity and sheathing area ratio, based on the scaled tests. According to Sugiyama and Matsumoto the following empirical equation is applicable for the apparent shear deformation angle of 1/100 radians and for ultimate capacity:

$$F = r/(3-2r) \qquad \textbf{(Eq. 2)}$$

This equation relates the ratio, *F*, of the shear load for a wall with openings to the shear load of a fully sheathed wall at a particular shear deformation angle.

A significant number of full scale monotonic and cyclic tests have provided verification of the perforated shear wall method [5] [6] [7] [8] [9]. The first verification studies included full-scale monotonic tests with 4 ft. (1.22 m) wall segments [5] [6] [7]. A series of tests investigated the use of corners as end restraints instead of mechanical hold-down devices [8]. An additional series of tests confirmed the performance of full scale tests with 2 ft. (0.61 m) wall segments, reduced base restraint, and the use of alternative framing practices that further optimize the perforated shear wall performance [9]. The use of an alternative empirical equation:

$$F = r/(2-r) \qquad \textbf{(Eq. 3)}$$

resulted in a more accurate prediction of capacity on average than did Eq. 2 [9]. The following study provides additional information about the performance of full scale tests with 2 ft. (0.61 m) wall segments and various conventional and innovative methods of providing base restraint.

3. EXPERIMENTAL PROGRAM

3.1 Wall Specimens

A total of 8 shear wall specimens were tested in this investigation (Table 1). Two hold-down anchors were used on specimens Wall 1 through Wall 7 applied to each end of the wall specimens. Wall 8 utilized an 18 gauge strap to attach each stud to the foundation using four 10d bright common nails on each side of the connection. In addition to the hold-downs, the bottom plate of the specimens were anchored with either 5/8 in. (15.9 mm) diameter bolts 2 ft. (0.61 m) on center, 5/8 in. (15.9 mm) diameter bolts 6 ft. (1.83 m) on center, or 2 – 16d pneumatic nails (3 in. x 0.131 in. diameter) at 16 in. (0.41 m) on center.

Wall 1, Wall 2, and Wall 3 were fully sheathed and served as the control from which shear ratios were derived for walls with openings having variations in base restraint. Also, 1-5/8 in. (41.3 mm) diameter flat washers were used with the 5/8 in. (15.9 mm) diameter anchor bolts throughout the testing program. Previous tests utilized a 3 x 3 x 1/4 in. (76.2 x 76.2 x 6.4 mm) steel plate washer on each anchor bolt [5][6][7][8].

All specimens were constructed with Spruce-Pine-Fir Stud grade lumber. Studs were spaced 16 in. (406.4 mm) on center for Wall 1, Wall 2, Wall 3, Wall 4, Wall 5, and Wall 8. Stud spacing was increased to 24 in. (609.6 mm) on center for Wall 6 and Wall 7 to coincide with the investigation of 2 ft. (0.61 m) wall segments. Headers and window sills were constructed to span openings, and a king and jack stud were used on either side of openings. Exterior sheathing consisted of 7/16 in. (11.1 mm) OSB, oriented vertically. The OSB was attached using 8d pneumatic nails (2-3/8 in. long x 0.113 in. diameter) spaced 6 in. (152.4 mm) along the perimeter and 12 in. (304.8 mm) in the field of the panels. Interior sheathing consisted of 1/2 in. (12.7 mm) GWB, oriented vertically. The GWB was attached with #6 screws spaced 7 in. (177.8 mm) along the perimeter and 10 in. (254.0 mm) in the field. Both interior and exterior sheathing were cut in separate pieces to fit above and below the doors and windows. A summary of the wall materials and construction data can be found in Table 2 and Table 3.

3.2 Test Procedures

The shear walls were tested in a horizontal position according to ASTM E564 [13] using a monotonic loading. A hydraulic actuator, with a range of 12 in. (304.8 mm) and capacity of 115,000 lb. (511.5 kN), applied the load to the top right corner of each shear wall through a 4x4 structural steel tube at a rate of 0.3 in/min. A 1/2 in. (12.7 mm) thick steel plate was welded to the end of the tube to provide a uniform loading area for the actuator. A 50,000 lb. (222.4 kN) capacity load cell was attached to the end of the actuator to enable load recordings. The load cell was calibrated immediately prior to the tests using the NAHB Research Center's Universal Test Machine. Casters, which were attached to the tubing, and roller-plate assemblies were used to allow horizontal motion. The casters and the roller-plate assemblies were positioned parallel to the direction of loading.

Three linear variable differential transformers (LVDT) were used to measure the displacement of the specimens during the test. The LVDTs measured the horizontal displacement of the top of the wall, the horizontal displacement (or slip) of the bottom sole plate of the specimen, and the uplift of the end studs relative to the foundation. In addition, six "donut" shaped load cells were used to monitor the uplift forces in the sole plate anchor bolts. These load cells have a 1-5/8 in. (41.3 mm) outside diameter and were attached to the anchor bolts between the flat washer and the anchor bolt nut. A 1-5/8 in. (41.3 mm) flat washer was placed between the sole plate and the load cell. A hex-head bolt was then tightened directly onto the load cell. The hold-down anchor resisting uplift was tightened to a value of 500 lb. (222.4 N) and the remaining anchors were tightened to a value of 200 lb. (889.6 N). This was done to ensure consistency throughout the testing program. All readings were zeroed at the beginning of each test. Each load cell was calibrated immediately prior to the tests using the NAHB Research Center's Universal Test Machine.

All tests were one directional, displacing the top of the wall to a maximum of six inches over a twenty minute period. Data from the load cells and LVDTs were collected 2 times per second. Each of the seven wall configurations was tested once. Items of interest are ultimate load capacity, initial stiffness, energy dissipated, and uplift. Load-displacement curves were plotted for each of the wall specimens to better understand and compare the behavior of the walls during the test.

4. RESULTS

4.1 Force-Displacement Response

The response of the shear wall specimens to the loading history are shown in the force-displacement curves of Figures 1, 2, 3, 4, and 5. The initial stiffness was high, but the entire load-deflection behavior was essentially non-linear. The ultimate load, F_{max}, as well as the corresponding displacement, Δ_{Fmax}, was gathered directly from the data. Resistance at failure was determined as the capacity of the specimen immediately prior to a significant decrease in strength or when the load dropped to $0.8F_{max}$, whichever occurred first. These loads and displacements are presented in Table 4. Table 5 and Table 6 compare the predicted shear ratio to the actual shear ratio using Wall 2 and Wall 3, respectively.

Walls 1, 2, and 3 were fully sheathed with different sole plate anchorage and were used as the control specimens in this study. Each of these specimens had an ultimate capacity (F_{max}) of at least 20.8 kips (92.5 kN) with a mean F_{max} of 21.7 kips (96.5 kN) and a COV of 0.04. These consistent results indicate that the types of sole plate anchorage investigated do not significantly effect the ultimate capacity for the fully sheathed walls.

All specimens using 4 ft. (1.22m) and 2 ft. (0.61 m) wall segments performed in conservative agreement with the perforated shear wall predictions as shown in Figure 6. The type of sole plate anchorage used had a more noticeable effect on the specimens with openings. Walls 5 and 7 (nailed sole plate anchorage) provided 18% and 28% higher ultimate loads respectively to Walls 4 and 6 (anchor bolts 6ft. on center). These findings suggested that nailing the sole plate with 2-16d nails at 16 in. (0.41m) results in higher ultimate capacities than the anchor bolts 6 ft. (1.63 m) on center for specimens with openings. The use of straps on Wall 8 (with no hold-downs) increased the ultimate capacity of the specimen by 17% over Wall 5, which had no straps but used hold-downs at the ends only for overturning restraint. Additionally, the combination of straps with the nailed anchorage provided 38% higher capacity than a similar perforated wall using bolts at 6 ft. (1.63m) on center. However, it should be noted that all specimens (Walls 4, 5, 6, 7, and 8) reached ultimate capacities well above the predicted curve using the perforated shear wall method (see Figures 6, 7 and 8).

Predicted shear load ratios, F, were determined using Eq. 2 and are presented with the actual shear load ratios in Table 4, Table 5 and Table 6 using Wall 1, Wall 2 and Wall 3 as the reference in the calculation, respectively. The ratio of actual to predicted values is also presented in Table 4 through Table 6 where a ratio greater than 1.0 indicates a conservative prediction. Figures 6, 7, and 8 plot actual capacities and shear load ratios found from the testing. As shown in Figure 6, Eq. 2 conservatively estimates the capacity for all wall configurations in this investigation.

Due to the conservative predictions of Eq. 2, the following equation was used as an alternative to predict the shear load ratios [9]:

$$F = r/(2-r) \qquad \textbf{(Eq. 3)}$$

The additional predicted shear load ratios, F, were determined using Eq. 3 and are presented with the actual shear load ratios in Table 7 through Table 9 using Wall 1 through Wall 3 as the reference. The ratio of actual to predicted is also presented, where a ratio equal to 1.0 is an exact prediction and a greater ratio is a conservative prediction. Figure 9 plots actual capacities and shear load ratios found from the testing. As shown in Figure 9, Eq. 3 estimates the capacity for Walls 1 through 8 more accurately than that of Eq. 2. However, Eq. 3 slightly overpredicts the shear capacity of Wall 4, which had a 6 ft. (1.83m) on center anchor bolt spacing. Using the more similar Wall 3 as the reference for comparison produces a more accurate curve (see Figure 10 and Figure 11).

4.2 Initial Stiffness

The initial portion of the force-displacement curves were fit with a linear least-squares trend, the slope of which is taken as the initial stiffness. That portion of the curve for which the magnitude of the force did not exceed 40 percent of the peak load was used in the calculation. The initial stiffnesses are listed in Table 4.

In general, initial stiffness was proportional to the sheathing area ratio, hence as the sheathing area ratio decreased the initial stiffness also decreased as expected. Wall 1 experienced a larger initial stiffness than that of Walls 2 and 3. This indicates that the reduced sole plate anchorage allowed for a more flexible wall which can be advantageous in resisting seismic loads provided sufficient capacity is maintained. The nailed sole plate resulted in slightly larger initial stiffnesses than did the anchor bolts 6 ft. (1.83 m) on center. The strap reinforcement of Wall 8 significantly increased the initial stiffness.

4.3 Energy Dissipated

The toughness of a wall can be quantified by its ability to dissipate energy while deforming. Cumulative energy dissipation was obtained by calculating the area under each force-displacement curve up to $0.80F_{max}$ using Simpson's Method. These values are listed in Table 4.

The 2 ft. (0.61 m) narrow wall segments in Walls 6 and 7 did not adversely affect the energy dissipation capacity of the walls. Also, the performance of the nailed sole plate anchorage and the 6 ft. (1.83m) on center anchor bolt spacing were very similar for the specimens with openings. However, the nailed sole plate anchorage did have an adverse affect on the energy dissipation capacity of the fully sheathed specimen when compared to the 2ft. (0.61m) and 6ft. (1.83m) anchor bolt spacing. Thus, the fully sheathed wall would require greater nailing at the sole plate to transfer the total wall shear load more effectively so that the energy capacity of the fully sheathed wall is maximized. The addition of straps to Wall 8 provided some improvement in this area of performance as compared to Wall 5.

4.4 End Stud Uplift

Loading each wall resulted in uplift zones at the end of the walls due to end panel rotation, wall drift, and overall overturning forces. This condition is not dissimilar to the type or magnitude of restraint that may occur in actual conditions. The vertical displacement of the end stud was measured by an LVDT. The uplift displacement and load at the specimen's ultimate capacity is given in Table 4.

The maximum uplift load experienced at the hold-down remained relatively constant throughout this testing phase. The specimens with openings and 4 ft. (1.22 m) wall segments experienced the largest uplift loads while the specimens with 2 ft. (0.61 m) wall segments experienced slightly lower loads at the hold-downs. In every case, the hold-down load was well below the load that would be predicted theoretically by multiplying the unit shear capacity by the wall height (i.e. conventional overturning design analysis).

The vertical displacement of the end stud remained fairly constant for Wall 1, Wall 3, Wall 4, Wall 5 and Wall 8. Thus, increased anchor bolt spacing, using nails instead of bolts, and the alternative strap anchor approach had little effect on the uplift displacement at the end stud. The narrow wall segments, regardless of sole plate anchorage, developed 50% less displacement at the end stud in comparison to the fully sheathed and 4 ft. (1.22 m) wall segments.

4.5 Failure Modes

All walls tested had similar failure characteristics except for Wall 2. The initial loading was generally linear until the interior sheathing, GWB, began to pull through the screws. This failure resulted in a slight reduction in stiffness. As the load approached ultimate capacity, the OSB sheathing near the loaded end began to buckle, and bending of OSB and framing nails was observed elsewhere. Racking of full height OSB panels was observed, while the OSB above and below openings acted as a rigid body. After ultimate capacity, the nails tore through the edges of the OSB.

Wall 2 also was characterized by a generally linear initial loading. Failure of the specimen was not characterized by failure of the sheathing, but rather nail slippage (shear failure) at the sole plate to foundation was the mode of failure. This failure mode explains the lower energy dissipation levels in this specimen. However, the ultimate capacity was 93 percent of that for walls with anchor bolts 6 ft. (1.83 m) on center. The sole plate at the hold-down resisting the wall uplift experienced failure as shown in Photo 1. This failure mode may be prevented by the addition of sole plate nails in accordance with the total shear load on a wall line. The nailed sole plate connection in Wall 5, Wall 7, and Wall 8 did not fail because of the lower total shear load due to the perforations in the specimens.

Although the above failure mechanisms were consistent throughout the testing, some differences were observed which explain the results discussed above. Separation of the sole plate and foundation was more evident on walls using anchor bolts at 6 ft. (1.83 m) on center, such as Wall 3. Sensors on anchor bolts along the sole plate registered the largest loads for Wall 3. However, failure of the sheathing panel connection was the cause of ultimate failure of the specimen. Wall 5 also had slight nail withdrawal from the foundation, but this occurred after ultimate load had been realized. The alternative strap anchor method in Wall 8 experienced both tension and compression of the straps throughout the length of the specimen (See Photo 2). As expected, the first two straps experienced the greatest overturning load (See Photo 3).

Comparing Walls 5 and 7 (nailed sole plate) to Walls 4 and 6 (bolted anchors, 6 ft. (1.83 m) on center, nailed sole plates provided larger ultimate loads and initial stiffness over their bolted anchor counterparts. Once ultimate load was achieved, the two openings in Walls 4 and 5 were no longer square and the two end sheets of OSB had pulled through the nails causing a sudden decrease in load. However, the narrow wall segments in Walls 6 and 7 prevented the racking of the openings after ultimate load. This was evident from the tearing of the full height GWB along the top of the openings. The bottom of these intermediate sheets remained relatively stable while the rigid body motion of the sheathing above the openings caused tearing of the full height GWB at the top opening corners.

While the behavior of Walls 4 and 8 was very similar, the results were quite different. The alternative framing practices of Wall 8 (strap anchors) provided a 17 percent increase in ultimate capacity, a 16 percent increase in energy dissipated, and a 89 percent increase in initial stiffness even without hold-downs at the ends. The lack of hold-down restraints in Wall 8 is only evident in the vertical stud displacement at the end of the wall, which was very consistent with similar specimens with hold-down brackets instead of strapping (Wall 4 and Wall 5).

Increasing the anchor bolt spacing to 6 ft. (1.83 m) on center in Wall 3 or using the 2-16d nails as the sole plate anchorage in Wall 2 only resulted in a slight decrease in ultimate capacity. In general, the nailed anchor plates produced better results than the 6 ft. (1.83 m) on center bolt anchors.

This phase of testing gives additional promise to the use of the perforated shear wall method with reduced base restraint that is common to conventional framing practices. In addition, the use of straps in this study and truss plates in the previous study [9] provide an efficient enhancement to the perforated shear wall method while maintaining simplicity in construction detailing and assembly.

5. CONCLUSIONS

The perforated shear wall method was first developed by conducting tests on one-third scale monotonic racking tests of shear walls. Eq. 2 was developed to predict the shear load capacity for shear walls with openings [3][4]. The perforated shear wall method was confirmed to be a conservative design approach using full scale tests of 40 ft. (12.19 m) long shear walls with openings constructed with 4 ft. (1.22 m) wall segments [5]. Additional testing was conducted to determine the effect of overturning restraints. Again, it was concluded that the perforated shear wall method results in conservative design values for shear walls [6]. The next phase of testing quantified the effects of corners on uplift restraint. The 2 ft. (0.61m) and 4 ft. (1.22 m) corner returns provided sufficient end restraint to allow 85 percent and 90 percent, respectively, of the fully-restrained wall's tested unit shear to be realized [8]. Each of the aforementioned phases of research refined the perforated shear wall method resulting in a more efficient and economical shear wall design. Subsequent testing provided additional refinement to the perforated shear wall method [9]. The research presented in this report further evaluated the effectiveness of reduced base restraint, alternative overturning restraints (i.e. redundant strapping), and narrow wall segments.

The data presented provides additional verification of the perforated shear wall method using reduced base restraint and 2 ft. (0.61 m) wall segments. The calculated shear capacity using the empirical equation developed by Sugiyama and Matsumoto (Eq. 2) conservatively estimates the capacity of all specimens tested. The use of the alternative empirical equation (Eq. 3) resulted in a more accurate prediction of ultimate capacity. This research produced similar results to a previous preliminary test with regard to increasing anchor bolt-spacing [9]. The use of 6 ft. (1.83 m) on center bolt anchors slightly decreases the ultimate capacity, initial stiffness, and energy dissipated. Similar reductions were seen with the use of nailed sole plate anchorage in comparison to 2 ft. (0.61 m) on center bolt anchors. However, in general, nailed anchors faired better than 6 ft. (1.83 m) on center bolt anchors. Despite these decreases, the empirical equation (Eq. 2) developed by Sugiyama and Matsumoto conservatively estimates the capacity with either increased anchor bolt spacing or sole plate nails, even when using a fully sheathed wall with anchor bolts 2 ft. (0.61 m) on center as the reference. The amount of shear resistance (base anchorage) needed at the sole plate can be reasonably determined using Eq. 2 or Eq. 3 with perforated shear walls. The alternative framing practice (straps and nails with no hold-downs) investigated in this report shows promise for high-wind and high-seismic applications.

The findings of this research support the use of the perforated shear wall method for 2 ft. (0.61 m) narrow wall segments, 6 ft. (1.83 m) on center anchor bolt spacing, and nailed sole plate anchorage. However, certain limits need to be placed on these conclusions such as the degree of sheathing nailing and the resulting unit shear value, as well as the amount of restraint provided by sheathing above and below openings. These results would not necessarily apply to the perforated shear walls with many large, full-height openings. In such cases, the segmented shear wall method may be more appropriate.

6. REFERENCES

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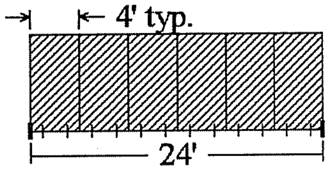
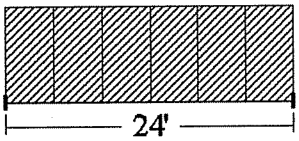
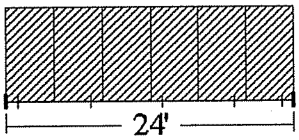
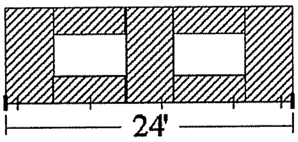
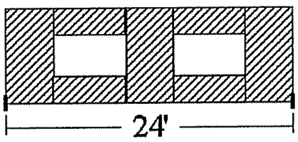
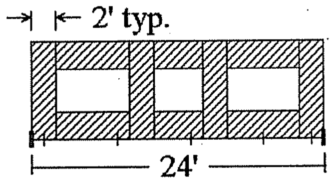
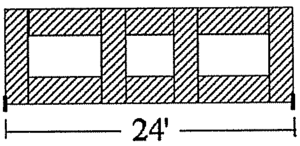
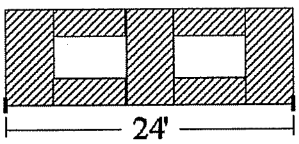
Table 1
Shear Wall Configurations

Specimen	Wall Configuration	Openings	Headers	Sheathing Area Ratio $(r)^1$	Sole Plate Anchorage	Hold-downs
Wall 1		None	None	1.0	5/8" dia. bolt @ 2' o.c.	Ends
Wall 2		None	None	1.0	2-16d nails @ 16" o.c.[3]	Ends
Wall 3		None	None	1.0	5/8" dia. bolt @ 6' o.c.	Ends
Wall 4		(2) - 6' x 4'	2 - 2x6	0.67	5/8" dia. bolt @ 6' o.c.	Ends
Wall 5		(2) - 6' x 4'	2 - 2x6	0.67	2-16d nails @ 16" o.c.[3]	Ends
Wall 6		(2) - 6'x 4' & (1) - 4' x 4'	2 - 2x6 & 2 - 2x4	0.50	5/8" dia. bolt @ 6' o.c.	Ends
Wall 7		(2) - 6'x 4' & (1) – 4' x 4'	2 - 2x6 & 2 - 2x4	0.50	2-16d nails @ 16" o.c.[3]	Ends
Wall 8		(2) - 6'x 4'	2 - 2x6	0.67	2-16d nails @ 16" o.c.[3]	None[2]

Notes: For SI: 1 ft. = 0.3048 m, 1 in. = 25.4 mm

1. The sheathing area ratio (r) is calculated in accordance with Equation 1.
2. 18g straps used to connect each stud to the base with 4 –.10d bright common nails on each end of the strap
3. Nails were 16d pneumatic nails (3 in. long x 0.131 in. diameter)

Table 2
Wall Materials and Construction Data

Component	Construction and Materials
Framing Members	Stud, Spruce-Pine-Fir, 2x4.
Sheathing	
Exterior	7/16 in. OSB, 8d common nails with 6 in. spacing on panel edges and 12 in. spacing in panel field (sheets installed vertically).
Interior	1/2 in. Gypsum Wallboard, #6 screws with 7 in. spacing on panel edge and 10 in. spacing in panel field (sheets installed vertically, joints taped).
Headers	
4'-0" opening	2-2x4 with an intermediate layer of 7/16 in. OSB. One jack stud and one king stud at each end.
6'-0" opening	2-2x6 with an intermediate layer of 7/16 in. OSB. One jack stud and one king stud at each end.
Structural Base Connections (Bottom of Wall)	
Hold-down	Simpson HTT 22, nailed to end studs with 32-16d sinker nails, 5/8 in. diameter tie rod to connect to reaction beam at the wall ends only (not used in Wall 8).
Anchor Bolts	5/8 in. diameter tie rods with 1-5/8 in. diameter flat washer.
Loading Tube Connections (Top of Wall)	
No Openings	1/2 in. diameter bolts with 1-5/8 in. flat washer @ 2 ft. on center.

For SI: 1 ft. = 0.3048 m, 1 in. = 25.4 mm

Table 3
Fastening Schedule

Connection Description	Type of Connector	Spacing
Framing:		
Top Plate to Top Plate (face-nailed)	16d pneumatic[1]	12 in. on center
Top/Bottom Plate to Stud (end-nailed)	2-16d pneumatic[1]	per connection
Stud to Stud (face-nailed)	2-16d pneumatic[1]	24 in. on center
Stud to Header (toe-nailed)	2-16d pneumatic[1]	per stud
Stud to Sill (end-nailed)	2-16d pneumatic[1]	per stud
Header to Header (face nailed)	2-16d pneumatic[1]	16in. on center
Hold-down (face nailed)	32-16d sinker	per hold-down
Sheathing:		
OSB	8d pneumatic[2]	6 in. edge/12 in. field
GWB	#6 screws	7 in. edge/10 in. field

Notes: For SI: 1in. = 25.4 mm
1. 16d pneumatic nails were 3 in. long and 0.131 in. diameter
2. 8d pneumatic nails were 2-3/8 in. long and 0.113 in. diameter.

Table 4
Force-Displacement Data from Testing

	1	2	3	4	5	6	7	8
Sole Plate Anchorage[a]	B–2ft	Nail	B–6ft	B–6ft	Nail	B–6ft	Nail	Strap
Sheathing Area Ratio	1.00	1.00	1.00	0.67	0.67	0.50	0.50	0.67
Predicted Shear Ratio[b]	1.00	1.00	1.00	0.40	0.40	0.25	0.25	0.40
Actual Shear Ratio	1.00	0.93	0.98	0.48	0.56	0.39	0.51	0.66
Actual/Predicted	1.00	0.93	0.98	1.18	1.40	1.58	2.02	1.64
F_{max} (kips)	22.5	20.8	22.0	10.7	12.7	8.8	11.3	14.8
Δ_{Fmax} (in.)[c]	1.842	1.466	2.117	2.305	2.327	1.790	2.382	2.160
$F_{failure}$ (kips)	18.0	16.6	17.6	8.6	11.5	7.1	9.1	11.8
$\Delta_{failure}$ (in.)	2.901	2.239	2.910	3.549	3.122	4.007	4.240	3.040
Initial Stiffness (kips/in)	54.7	36.0	35.1	10.8	16.8	10.7	11.9	20.4
Energy Dissipated (kips*in)	55.4	35.1	52.1	29.7	30.7	29.2	37.4	34.5
$F_{hold\text{-}down}$ (kips)[c]	3.7	4.6	4.1	5.9	5.6	4.0	4.5	NA
$F_{hold\text{-}down\text{-}max}$ (kips)	4.2	4.6	5.2	6.3	6.0	4.3	5.4	NA
$\Delta_{slippage}$ (in.)[c]	0.281	0.779	1.066	0.350	0.219	0.370	0.373	0.382
Δ_{uplift} (in.)[c]	0.440	0.136	0.376	0.468	0.455	0.240	0.234	0.487
Δ_{uplift} (in.)[d]	0.599	0.136	0.481	0.564	0.540	0.304	0.372	NA

Notes: For SI: 1 ft. = 0.3048 m, 1 in. = 25.4 mm, kip = 4.45kN.
a. Sole plate anchorage abbreviations: B-2ft = 5/8in. bolts 2ft. o.c. B-6ft = 5/8in. bolts 6ft. o.c. Nail = 2-16d nails 16in. o.c. Strap = 18g strap @ each stud
b. The predicted shear ratio is based on the empirical formula, F=r/(3-2r), developed by Sugiyama and Matsumoto for wood-framed shear walls. Wall 1 is the reference.
c. Measured at F_{max} (peak wall capacity)
d. Measured at $F_{hold\text{-}down\text{-}max}$

Table 5
Predicted Shear Load Ratios for
2-16d Nails at 16in. On Center Sole Plate Anchorage

	Wall Specimens			
	2	5	7	8
Sheathing Area Ratio	1.00	0.67	0.50	0.67
Predicted Shear Ratio	1.00	0.40	0.25	0.40
Actual Shear Ratio	1.00	0.61	0.55	0.71
Acual/Predicted	1.00	1.53	2.20	1.78

Note: The predicted shear ratio is based on the empirical formula, F=r/(3-2r), developed by Sugiyama and Matsumoto for wood-framed shear walls, Wall 2 is the reference.

Table 6
Predicted Shear Load Ratios for
5/8in. Diameter Bolts 6ft. On Center

	Wall Specimens		
	3	4	6
Sheathing Area Ratio	1.00	0.67	0.50
Predicted Shear Ratio	1.00	0.40	0.25
Actual Shear Ratio	1.00	0.49	0.40
Acual/Predicted	1.00	1.23	1.60

Note: The predicted shear ratio is based on the empirical formula, F=r/(3-2r), developed by Sugiyama and Matsumoto for wood-framed shear walls, Wall 3 is the reference.

Table 7
Predicted Shear Load Ratios, $F = r/(2-r)$, Wall 1 as Reference

	Wall Specimens					
	1	4	5	6	7	8
Sheathing Area Ratio	1.00	0.67	0.67	0.50	0.50	0.67
Predicted Shear Ratio	1.00	0.50	0.50	0.33	0.33	0.50
Actual Shear Ratio	1.00	0.48	0.56	0.39	0.51	0.66
Acual/Predicted	1.00	0.96	1.12	1.18	1.55	1.32

Note: The predicted shear ratio is based on the empirical formula, F=r/(2-r), using Wall 1 as the reference.

Table 8
Predicted Shear Load Ratios, $F = r/(2-r)$, Wall 2 as Reference

	Wall Specimens			
	2	5	7	8
Sheathing Area Ratio	1.00	0.67	0.50	0.67
Predicted Shear Ratio	1.00	0.50	0.33	0.50
Actual Shear Ratio	1.00	0.61	0.55	0.71
Acual/Predicted	1.00	1.22	1.67	1.42

Note: The predicted shear ratio is based on the empirical formula, F=r/(2-r), using Wall 1 as the reference.

Table 9
Predicted Shear Load Ratios, $F = r/(2-r)$, Wall 3 as Reference

	Wall Specimens		
	3	4	6
Sheathing Area Ratio	1.00	0.67	0.50
Predicted Shear Ratio	1.00	0.50	0.33
Actual Shear Ratio	1.00	0.49	0.40
Acual/Predicted	1.00	0.98	1.21

Note: The predicted shear ratio is based on the empirical formula, F=r/(2-r), using Wall 3 as the reference.

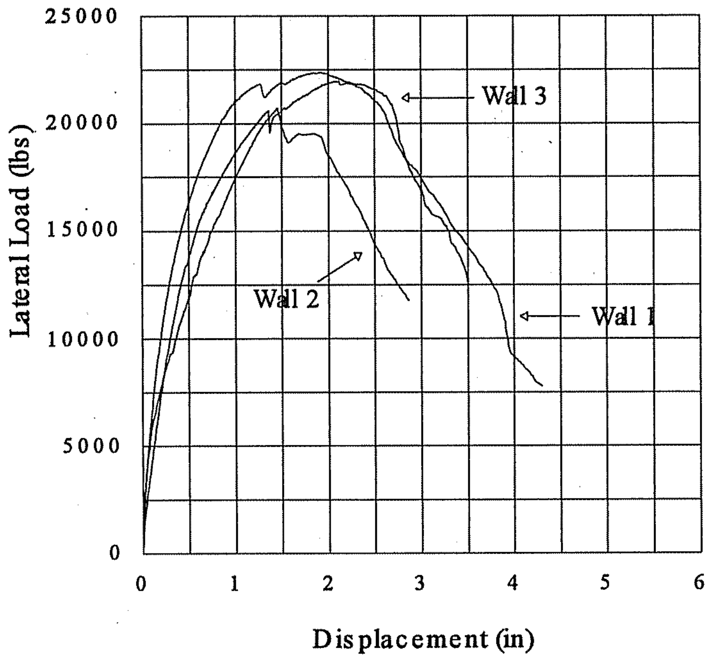


Figure 1[I]
Force-Displacement Response for Walls 1, 2, and 3 (r = 1.0)

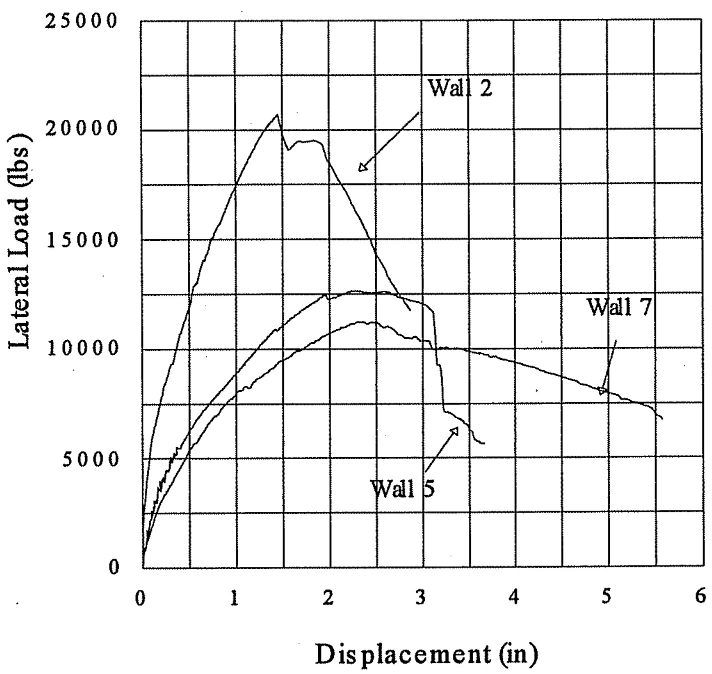


Figure 2[I]
Force-Displacement Response for Walls 2, 5 and 7
(Sole Plate Anchorage: 2 - 16d pneumatic nails @ 16in. o.c.)

[I] For SI: 1kip = 4.45kN, 1in = 25.4mm

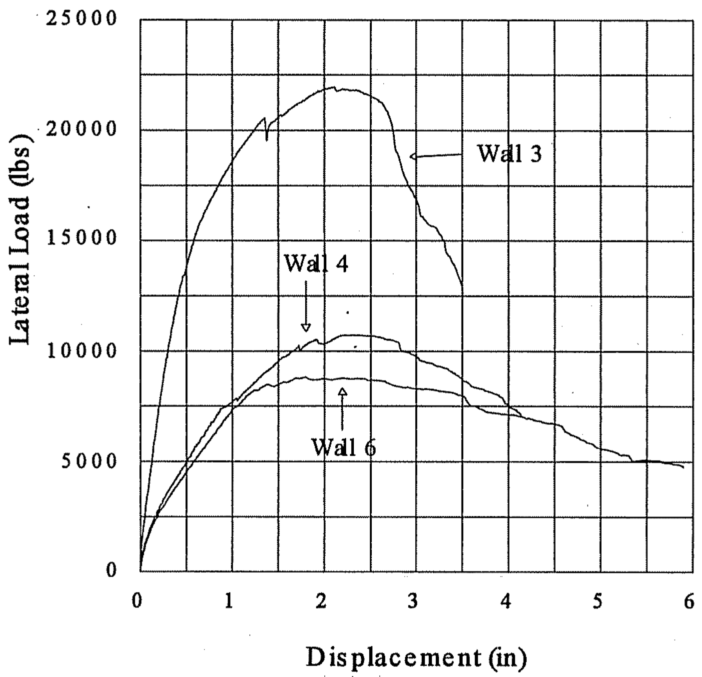


Figure 3[I]
Force-Displacement Response for Walls 3, 4 and 6
(Sole Plate Anchorage: 5/8in. diameter bolts 6ft. o.c.)

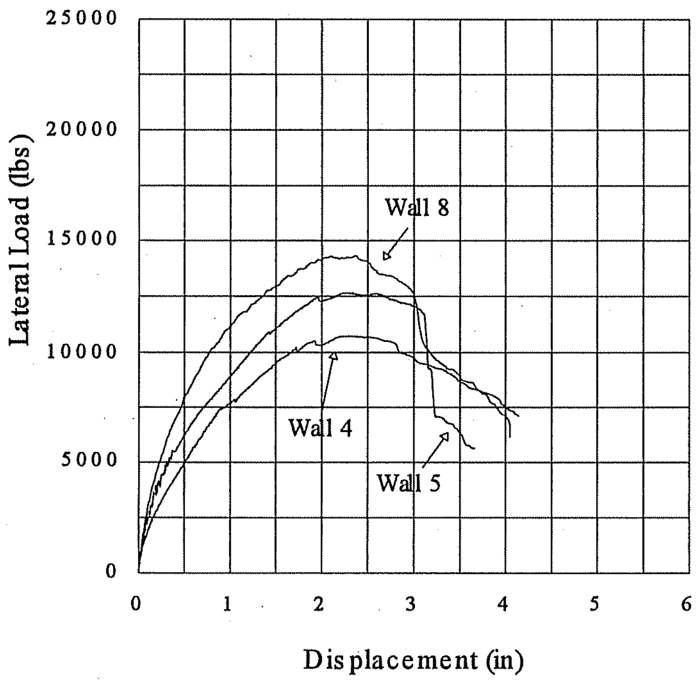


Figure 4[I]
Force-Displacement Response for Walls 4, 5 and 8 (r = 0.67)

[I] For SI: 1kip = 4.45kN, 1in = 25.4mm

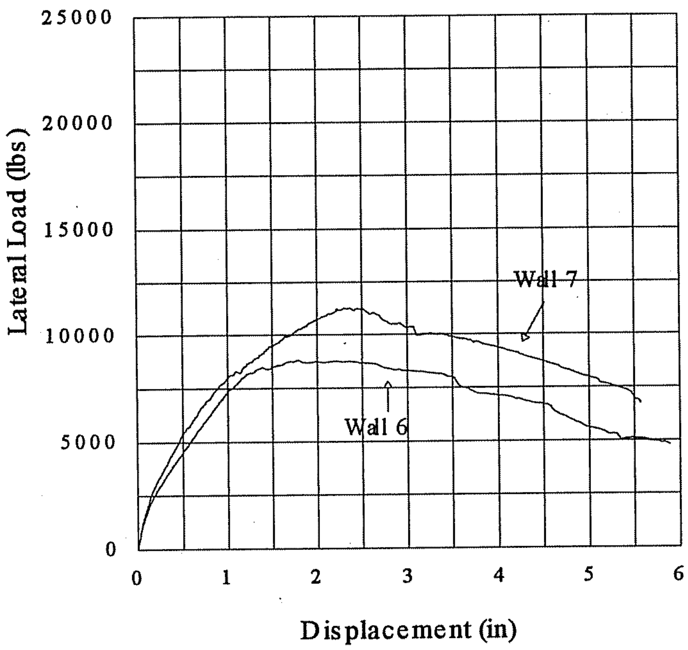


Figure 5[I]
Force-Displacement Response for Walls 6 and 7 (r = 0.50)

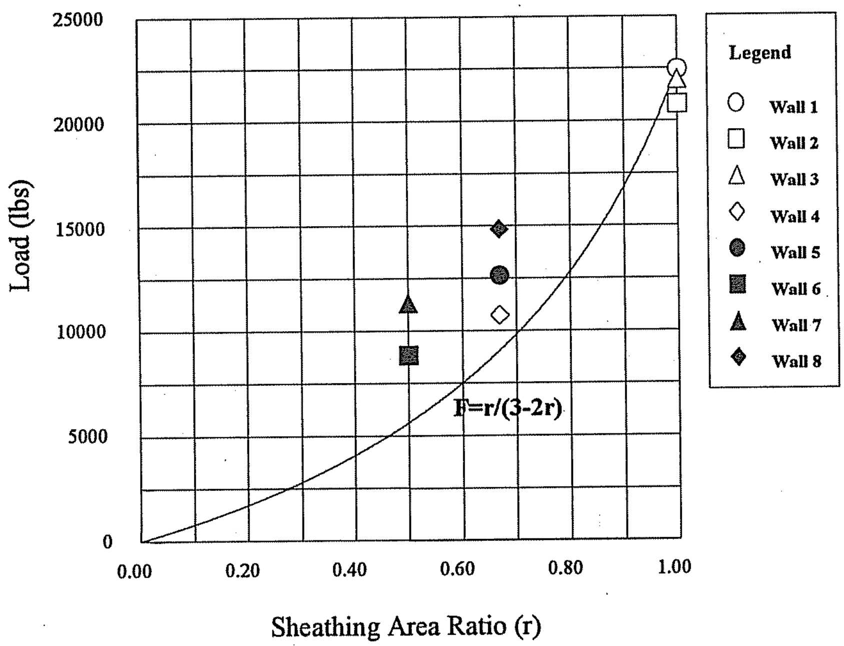


Figure 6[I]
Ultimate Capacity vs. Sheathing Area Ratio
Wall 1 as Reference

[I] For SI: 1kip = 4.45kN, 1in = 25.4mm

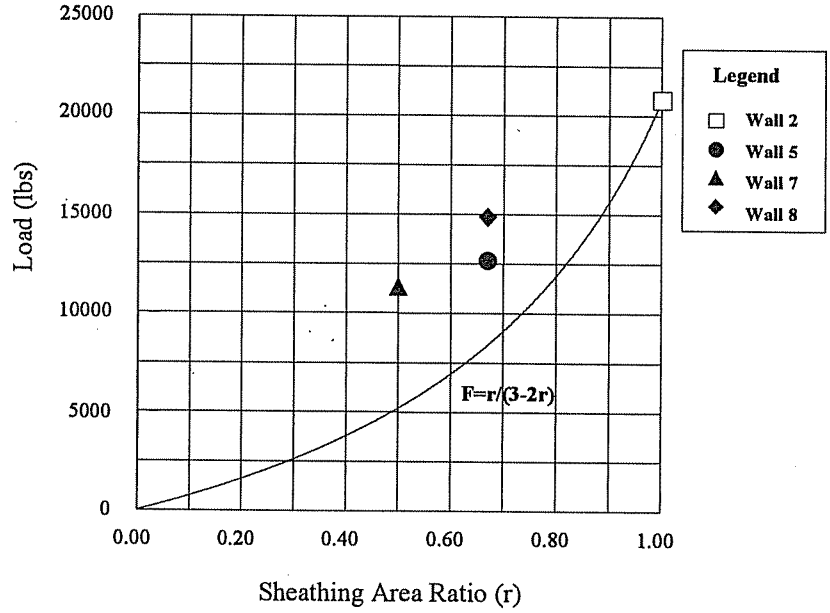


Figure 7[I]
Ultimate Capacity vs. Sheathing Area Ratio
Wall 2 as Reference

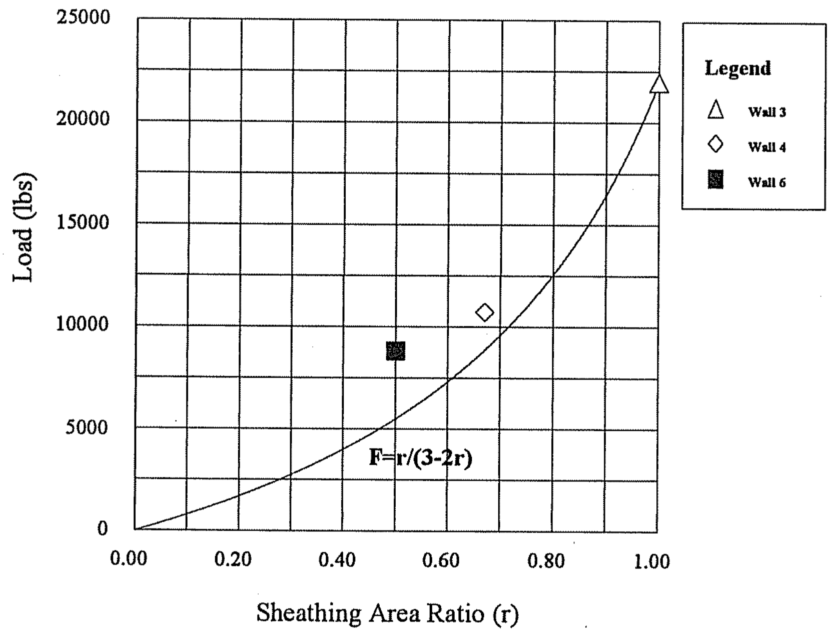


Figure 8[I]
Ultimate Capacity vs. Sheathing Area Ratio
Wall 3 as Reference

[I] For SI: 1kip = 4.45kN, 1in = 25.4mm

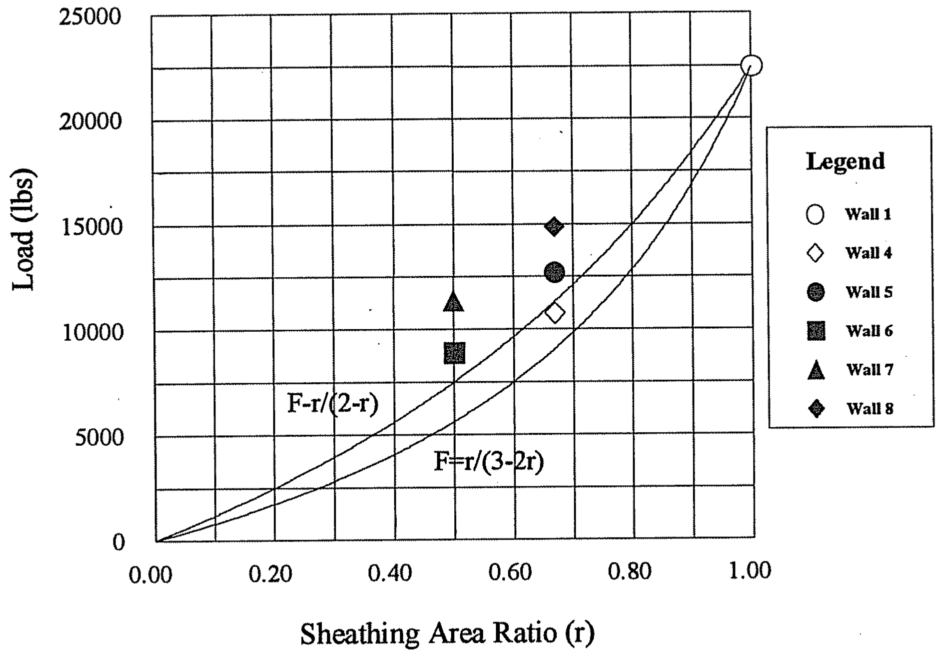


Figure 9[I]
Ultimate Capacity vs. Sheathing Area Ratio (Wall 1 as Reference)

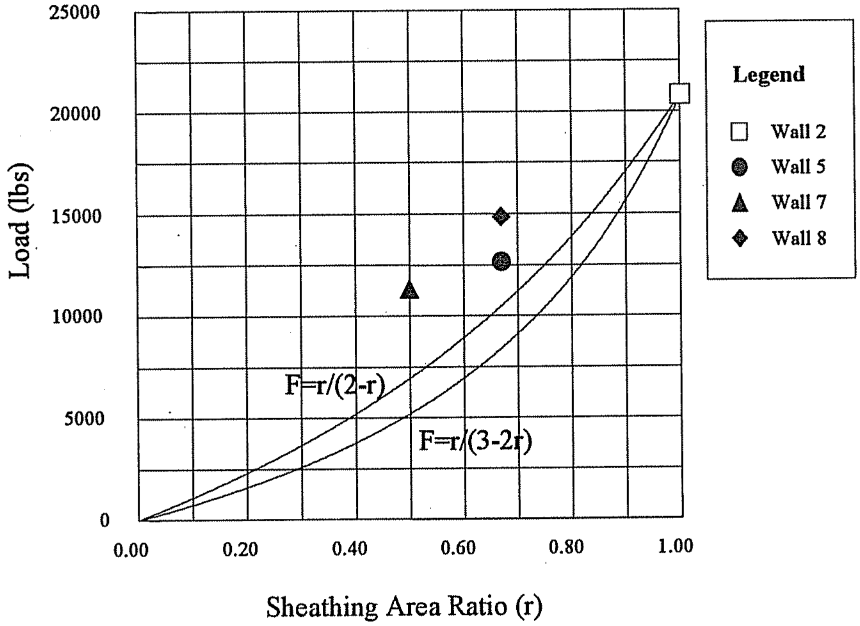


Figure 10[I]
Ultimate Capacity vs. Sheathing Area Ratio (Wall 2 as Reference)

[I] For SI: 1kip = 4.45kN, 1in = 25.4mm

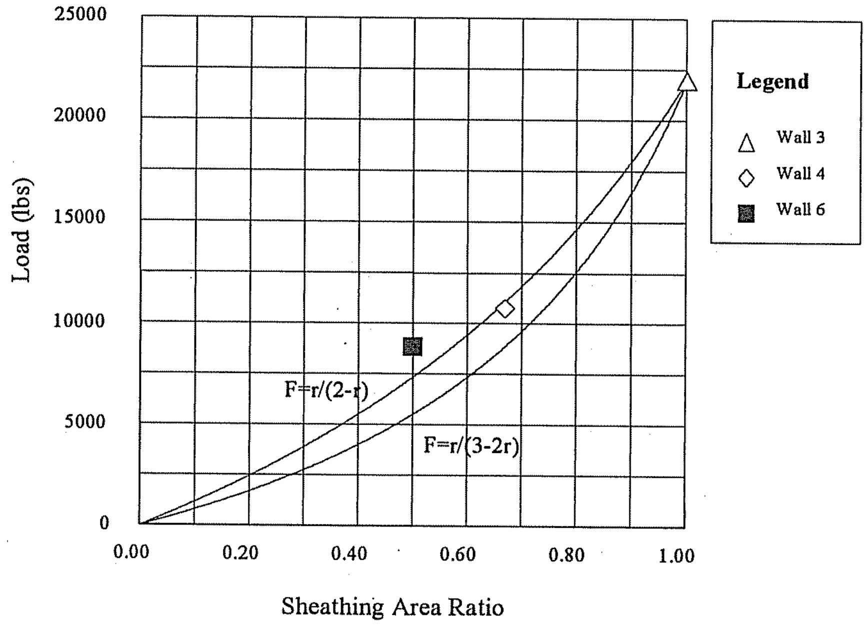


Figure 11[I]
Ultimate Capacity vs. Sheathing Area Ratio (Wall 3 as Reference)

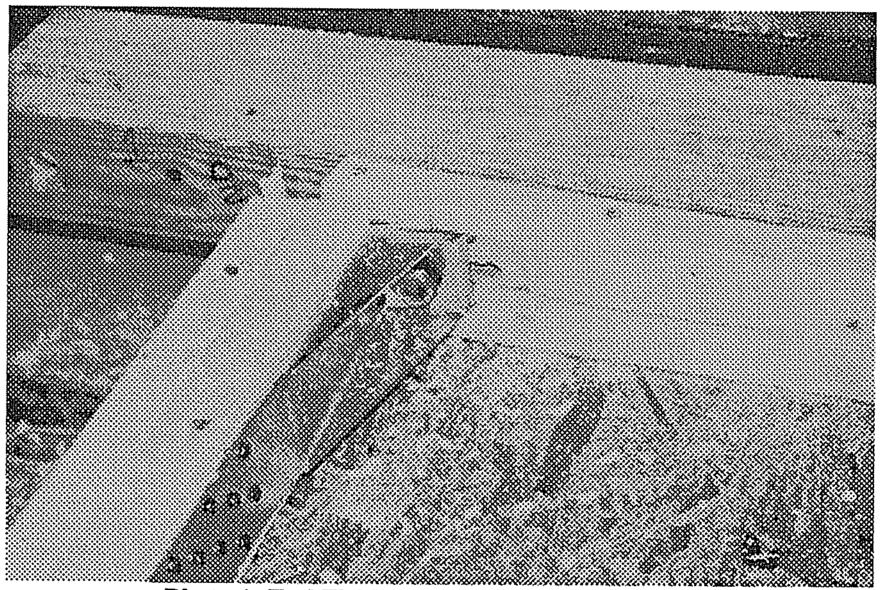


Photo 1: End Hold-down at Sole Plate (Wall 2)

[I] For SI: 1kip = 4.45kN, 1in = 25.4mm

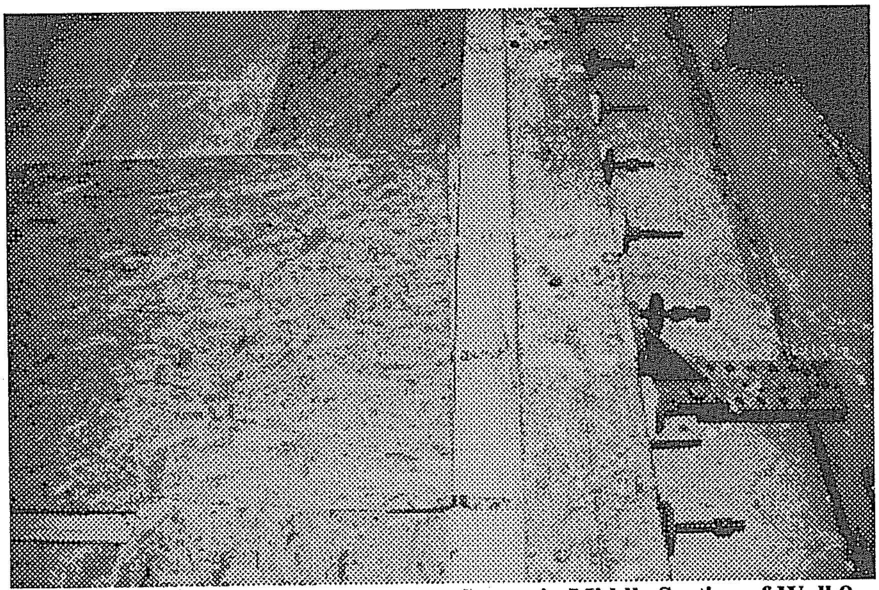


Photo 2: Compression and Tension on Straps in Middle Section of Wall 8

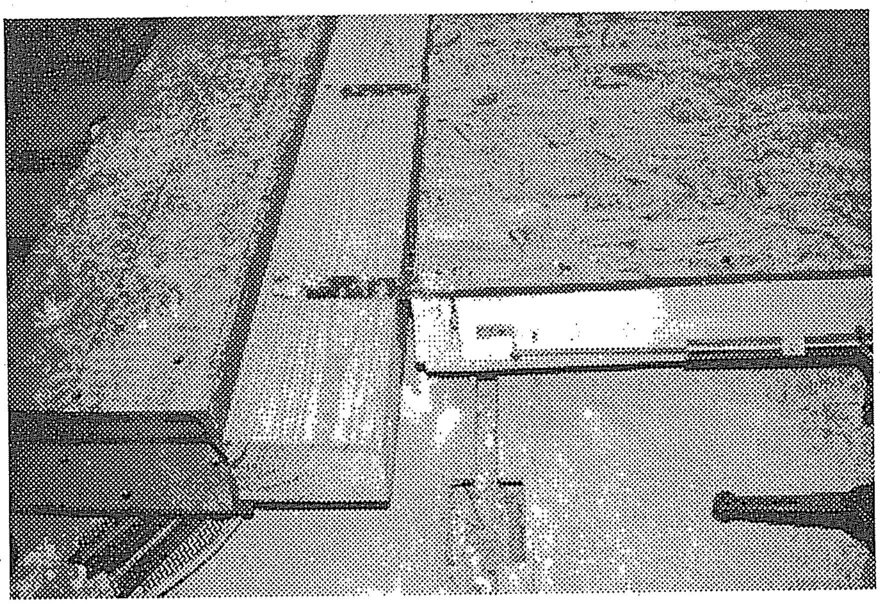


Photo 3: Load on First Two Straps of Wall 8