CHAPTER 4

4. Monotonic Shear Wall Test Results

4.1 Introduction

Results of the monotonic loading tests are presented in this chapter. In all, 12 monotonic tests on 8 ft by 8 ft walls were performed. Two different types of monotonic loading, discussed in Chapter 3, were used to test the walls. The load-deflection equivalent elastic-plastic curves and the qualitative behavior of the walls are discussed in this chapter. Load resistance and elastic stiffness values of the walls are reported. An equivalent elastic-plastic curve analysis is used to determine the yield point and the ductility of the walls. The overall behavior and failure modes of the walls are also discussed.

4.2 Results

Results were obtained from analyzing the load-displacement curves of the monotonic tests performed. Curves for the static one-directional monotonic tests were derived from data sampled continuously while a displacement was applied at a constant rate. Curves for the ASTM E564 tests, however, appear in the form of envelope curves. Data for the ASTM E564 tests is presented in this manner because of the requirement of the specification that each incremental load in the test be held for a period of one minute before a data point is taken. These one-minute points were extracted from the continuously recorded data obtained during testing.

Values presented in this chapter represent averages of the values obtained from each wall configuration. Values obtained from each individual test can be found in Appendix A. Load-deflection curves for the four different wall configurations are presented in Figures 4.1-4.4. Lines presented in these figures represent the static one-directional test data and plotted points represent the envelope values from the E564 tests.

As can be seen from Figures 4.1-4.4, the two tests are equivalent for determining strength and stiffness parameters.
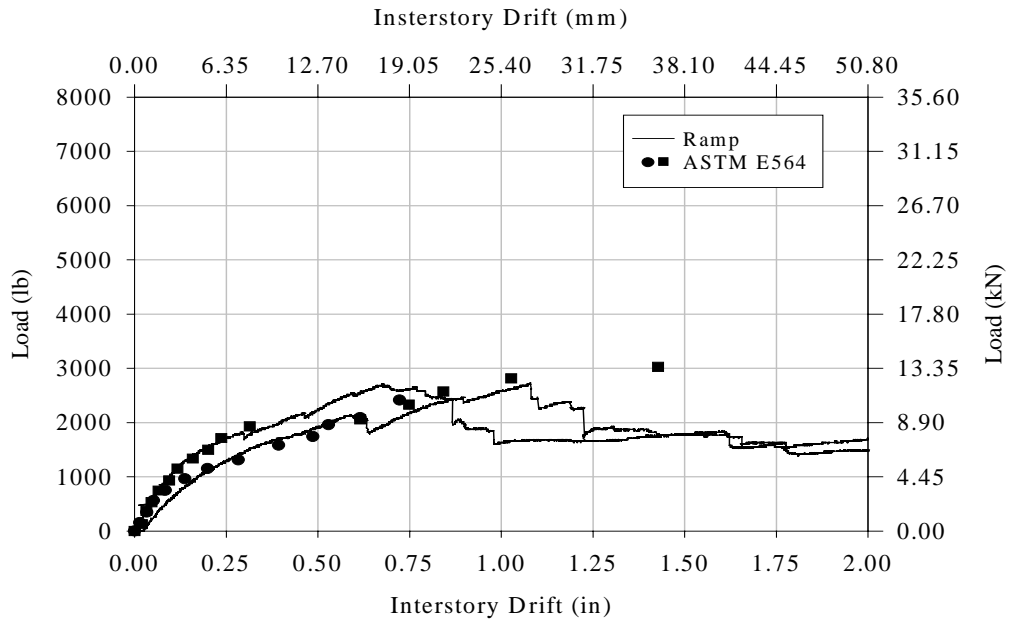

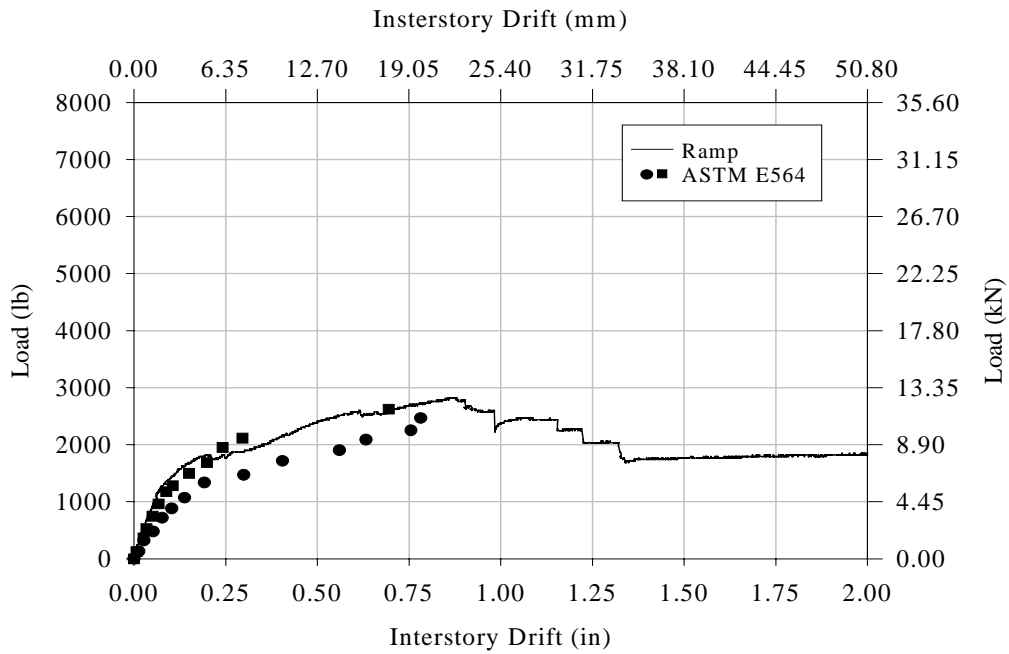**Figure 4.1 – Load-Drift Curves for Wall A.**

Insterstory Drift (mm)

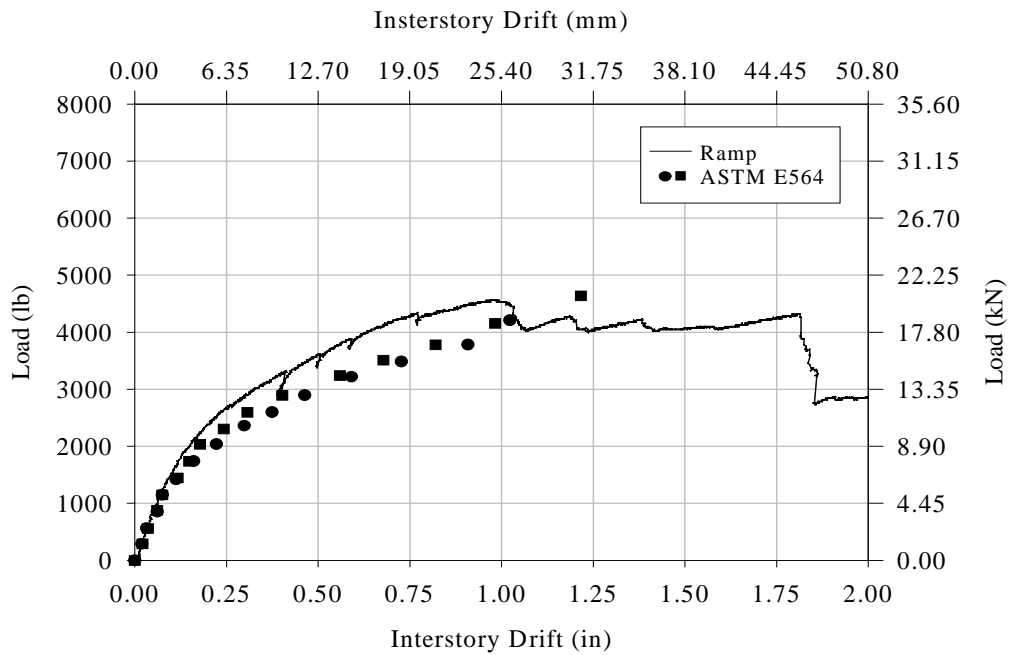**Figure 4.2 – Load-Drift Curves for Wall B.**

Insterstory Drift (mm)

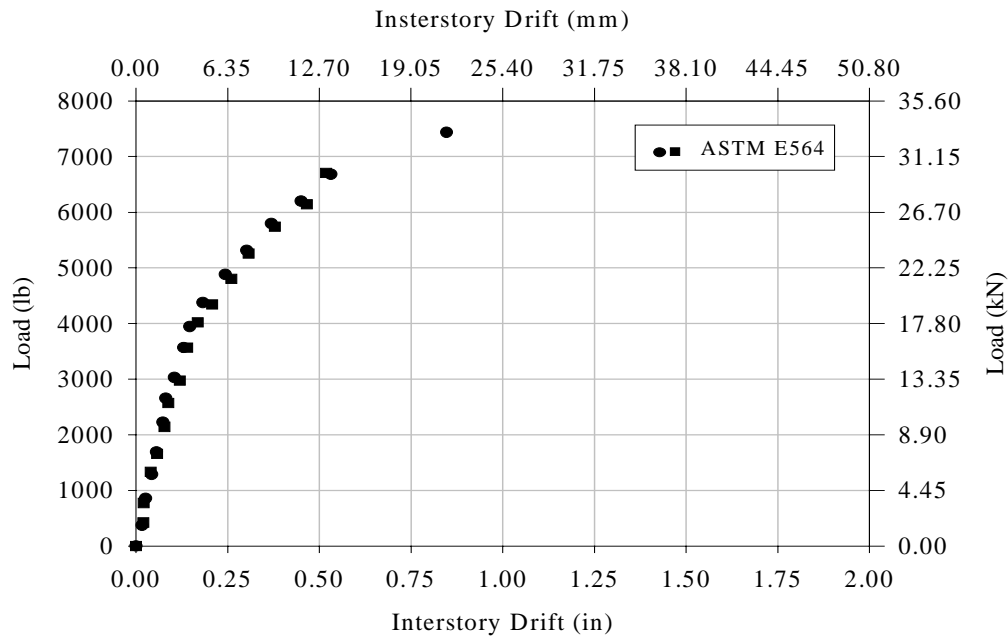**Figure 4.3 – Load-Drift Curves for Wall C.**

Figure 4.4 – Load-Drift Curves for Wall D.

Two ASTM E564 tests were performed for each wall configuration. The ASTM standard specifies that a third test be performed if the values of strength or stiffness did not agree within 15%. A third test was not performed either because it was not needed, as for Wall D, or the ramp monotonic tests provided the additional required data. No ramp monotonic tests were performed for Wall D.

4.2.1 Strength and Deflection

Values of strength and deflection at the point of yield, failure, and maximum load resistance are presented in Table 4.1. There may be variability between tests on the load resistance at a certain deflection for different reasons. The inherent variability of wood would cause some scatter in data. Also the carriage bolts used to attach the wall to the test frame sometimes crushed into the wood causing more uplift at the ends of the walls resulting in greater values of drift.

Table 4.1 – Monotonic Values for Load Resistance, Drift, Elastic Stiffness, and Ductility at Maximum, Failure, and Yield.

	Wall A	Wall B	Wall C	Wall D
F_{max} (lb)	2700	2650	4450	7050
Max Base Shear (plf)	338	331	556	881
Δ_{max} (in)	0.98	0.79	1.08	0.68
$F_{failure}$ (lb)	2650	2650	4400	7050
$\Delta_{failure}$ (in)	1.02	0.79	1.36	0.68
F_{yield} (lb)	2200	2150	3700	5850
Δ_{yield} (in)	0.29	0.18	0.30	0.20
k_e (lb/in)	7990	13300	12300	28600
Ductility	3.7	4.9	4.6	3.3

Values of maximum load resistance, or capacity, ranged from 2650 lb to 7050 lb. Values of load resistance at catastrophic failure ranged from 2650 lb to 7050 lb. Values of load resistance at yield ranged from 2150 lb to 5850 lb. Values of load resistance at yield, $F_{yield,}$ ranged from 81% to 82% of capacity, $F_{max.}$ This is slightly higher than the minimum 80% required by the definition of F_{yield} in an equivalent elastic-plastic curve analysis.

It can be seen in Figure 4.5 that there is no difference in the capacity of Walls A and B. Load resistance at failure and yield also correspond very closely between Walls A and B. Wall A and Wall B have the same bottom and top plates with different vertical connection elements (either a 2x4 or a 5 in. OSB spline). Failure in these two wall configurations occurred along the bottom plate and not along the vertical connecting elements so these similarities should be expected.
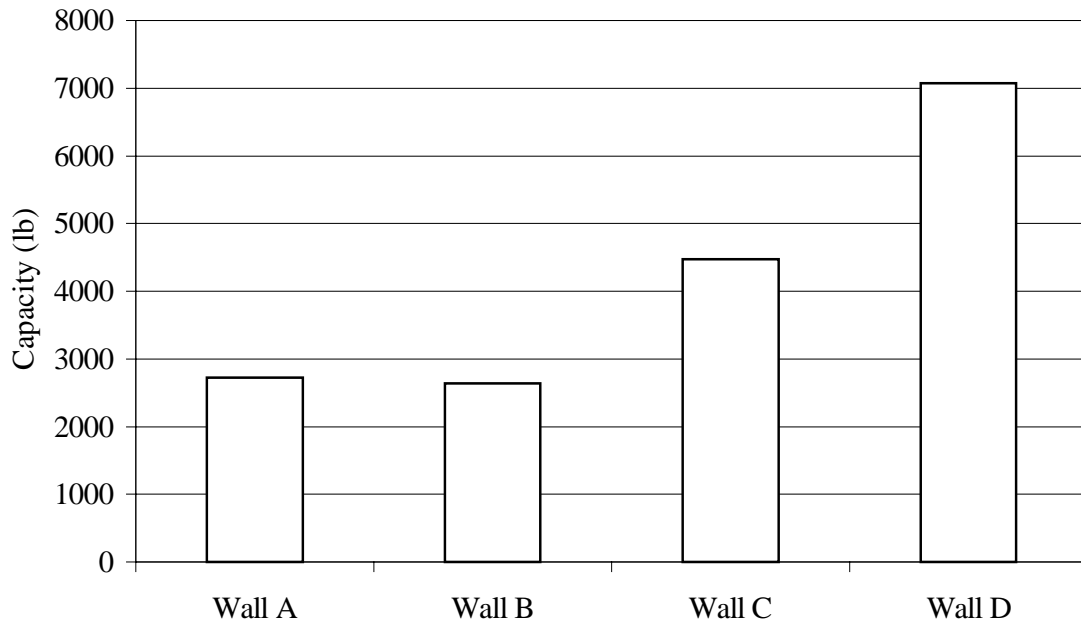
Figure 4.5 – Monotonic Capacities for the Four Different Wall Configurations.

Wall C added a second bottom plate to Wall configuration A to try to increase the strength of the critical bottom plate section. The addition of a second bottom plate increased the capacity of the system by 67% over wall configurations A and B. Failure loads were also increased by 67%. Load resistance at yield was increased by 70% with the addition of a second bottom plate.

The wall configuration with the highest capacity was Wall D. Wall configuration D, with the addition of tie-down anchors at the bottom of the wall, experienced failure away from the bottom of the wall. This is the reason that the capacity is higher. Overturning anchors force the failure mode to occur somewhere else in the shear wall. The addition of tie-down anchors increases capacity of the system by 164% when compared to Walls A and B. Load resistance at failure and yield are increased 168% by the addition of tie-down anchors.

Addition of tie-down anchors is more effective for increasing capacity than the addition of a second bottom plate. Capacity of Wall D is 58% higher than capacity of Wall C. Load resistance at failure is 61% higher for Wall D than it is for Wall C, and load resistance at yield is 58% higher for Wall D than it is for Wall C.

Values of unit shear at capacity may be of more interest to designers. As is done for design, the shear is assumed to be distributed uniformly along the base of the wall. Values of unit shear at capacity are presented in Table 4.1. They range from 331 plf for Wall B to 881 plf for Wall D.

Values for drift at capacity, failure, and yield can be seen in Table 4.1. Average values of drift at each designated point of interest are smaller for Wall B than they are for Wall A. They are smaller on average by about 0.2 in. This would suggest that Wall B is stiffer. The stiffness values for Wall B have a large amount of scatter which may account for this effect. Individual tests among each wall configuration also differ for values of drift at capacity, failure, and yield. Stiffness is a better indication of behavior than these values of drift.

4.2.2 Elastic Stiffness

Values determined for elastic stiffness, k_e, are presented in Table 4.1. Average values of stiffness ranged from 7990 lb/in. for Wall A to 28600 lb/in for Wall D. It can be seen in Figure 4.6 that the stiffness of Wall D is significantly higher than other wall configurations. The addition of tie-down anchors to a wall greatly increases stiffness. This would be expected because the tie-down anchors restrain rigid body rotations of the wall.
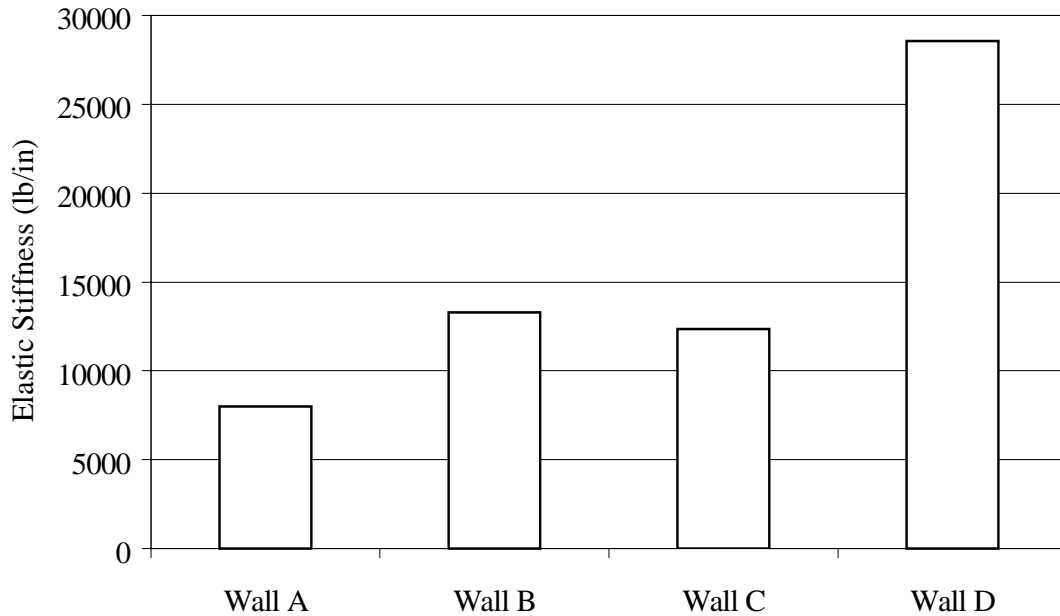
Figure 4.6 – Average values of Elastic Stiffness for Four Different Wall Configurations.

Due to the additional bottom plate, it would be expected that Wall C would have a higher stiffness than Walls A and B. However, this is not the case. Walls B and C have approximately equal stiffness, which is about 50% higher that Wall A. Wall B has a significantly higher stiffness (13300 lb/in) than does Wall A (7990 lb/in). The only difference between these walls is the vertical connecting elements. Wall B uses the 5 in. OSB spline connection while Wall A uses a 2x4 for the vertical connecting element. The higher stiffness may be explained by the fact that there is more surface area for adhesion of the glue in Wall B. However, there were no appreciable differences in capacity between Walls A and B; and the drifts should be the same since the bottom plate is critical to the performance of the wall. This difference in stiffness may have been due testing error or error involved in the construction of the walls. An adhesive was being used, and exact curing times and contact area probably varied. Variability in wood as a material may yield large variability of results. It was found in the cyclic tests that the values of stiffness for Walls A and B were closely related (see Chapter 5).

4.2.3 – Ductility

Values of ductility ratio for the four different wall configurations are presented in Table 4.1. Values of ductility for the individual wall configurations varied a great deal. No correlation could be found between the different wall configurations for ductility as can be seen in Figure 4.7. The definition of ductility tends to encourage results such as these. Ductility is based on the drift at yield and failure of the wall. The drift of the wall at failure is unpredictable and not consistent among tests because the behavior of the wall is inelastic. In addition, the value of ductility is a ratio of two values that vary themselves. Therefore, the values reported for ductility is a measure of relative performance between yield and failure and should not be used to judge performance on their own.
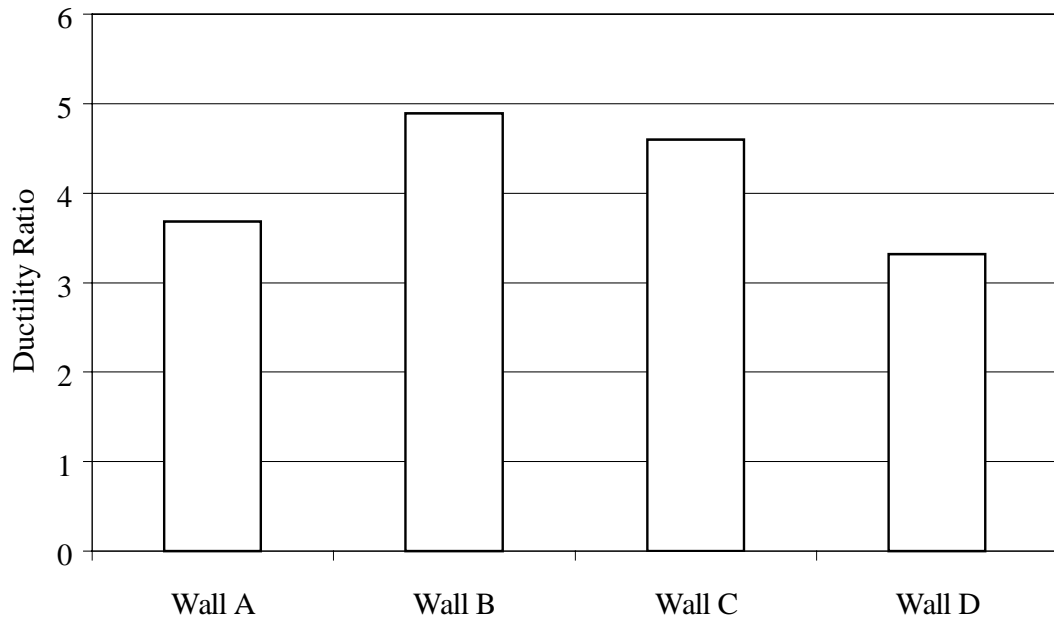

Figure 4.7 – Average Values of Ductility for the Four Different Wall Configurations.

4.3 – Wall Behavior

During each test, uplift of the end of each wall as well as the slip of the wall relative to the test frame was measured. Values obtained for slip were used to determine the interstory drift at each load increment. Values of wall end uplift, general wall behavior, and failure modes are discussed in this section

4.3.1 Uplift of Wall Ends

For purposes of discussion, the right side of the wall is defined as the side that was attached to the hydraulic actuator and the left side of the wall is defined as the opposite side. Average maximum values of uplift for each wall configuration recorded before catastrophic failure of the walls are presented in Table 4.2. A positive number indicates that the end of the wall was lifting away from the bottom plate or the test frame. In some cases, bending of the bottom plate caused initial uplift; but near failure, the drywall screws started to fail or tear through the Fiberboard sheathing, causing the end stud to separate from the base plate. Carriage bolts crushing the wood may have also caused some of the uplift of the wall ends. A negative number indicates that end of the wall was crushing the bottom plate of the wall. As expected, the right side of the wall experience separation and the left side of the wall experienced crushing because of the direction of load application.

Table 4.2 – Maximum Wall End Displacements Through Failure of Walls.

	Wall A	Wall B	Wall C	Wall D
Δ_{right} (in.)	0.63	0.41	0.72	0.18
Δ_{left} (in.)	-0.31	-0.31	-0.45	-0.10

These values of uplift are significant for Walls A, B, and C. This may be attributable to the fact that the walls may have experienced localized failures at the corners of the walls before total wall failure. The tie-down anchors present in Wall D kept the uplift of the end studs to a minimum. Uplift displacements of Wall D are

significantly lower than the values of uplift for Walls A, B, or C. Negative values, indicating crushing of the end stud into the base plate, are smaller than the positive values, indicating separation from the bottom of the test frame in the tension regions. This would be expected since compression forces are transferred through bearing while the adhesive and screws resist tension forces.

4.3.2 General Wall Behavior

The ASTM E564 test standard also requires that at 1/3 and 2/3 the anticipated ultimate load, the load be removed from the wall and wall allowed to relax for 5 minutes before measuring the drift, or set, of the wall. This gives an indication of how far beyond elastic the wall is at 1/3 and 2/3 its ultimate load. The 1/3 ultimate load point is of special interest because light-frame shear walls have an average factor of safety of about 3. This would mean that 1/3 the ultimate load would be the design load of the wall. It is important to understand what the "set" of the wall will be after it has experienced the design load. An average measure of how far beyond elastic a wall is would be to define a ratio (Set Ratio) of the set of the wall to the drift of the wall at a certain load as suggested in ASTM E564. A perfectly linear elastic system, for example, would have a set ratio of zero. The values of set after 5 minutes relaxation and the ratios of set to drift before load removal at 1/3 and 2/3 the ultimate load are presented in Table 4.3.

Table 4.3 – Values of Set and Set Ratio after ASTM E564 Load Removal.

	Wall A	Wall B	Wall C	Wall D
Set 1/3 F_{max} (in.)	0.02	0.01	0.01	0.01
Set 2/3 F_{max} (in.)	0.09	0.06	0.13	0.04
1/3 Set Ratio	0.23	0.10	0.10	0.12
2/3 Set Ratio	0.27	0.23	0.30	0.21

As would be expected, the values of set ratio increase as the load is increased from 1/3 to 2/3 ultimate load. This would indicate that the wall moves further into the inelastic deformation range as the amount of load increases. Values of 1/3 set ratio are about 10% except for Wall A which is a little higher. This would indicate that it is reasonable to assume linear elastic behavior for the structurally insulated panel shear walls within the design load range. Values of 2/3 set ratio range from about 20% to 30%. Shear wall would most likely not experience loads such as this during a normal life cycle. The wall with tie-down anchors, Wall D, has the lowest value of 2/3 ultimate load set ratio. This suggests that if the wall experienced very high loads, the wall with tie-down anchorage would perform the best with the least long-term ramifications.

4.3.3 Failure Modes

Failure modes experienced for the tests on Walls A and B are what lead to the development of the different wall configurations. Structurally insulated panels usually have only a single bottom plate with no tie-down anchors at the end of the walls (Walls A and B). The failure mechanism in this type of wall was noticed to be separation from the bottom plate. For this reason, an additional bottom plate was added for Wall C. The failure mechanism remained the separation from the bottom plate for Wall C, therefore, tie-down anchors were used at the wall ends moving the failure mechanism away from the bottom plate to other locations in the wall.

Wall configurations A, B, and C displayed similar modes of failure. The failure in these wall configurations occurred at the bottom plate of the wall. The walls appeared to rack as a single rigid body about a point along the bottom of the wall. The bottom corner of the wall on the side on which the load was applied experienced tension. The bottom corner of the other side of the wall experienced compression. Failure initiated on the tension side of the wall as the drywall screws tore through the Fiberboard and then the screws sheared along the interface of the bottom plate and the OSB. In some cases, the drywall screws failed in shear along the interface of the Fiberboard and bottom plate instead of tearing through the Fiberboard. This is probably due to local strong points in

the Fiberboard or a slightly larger distance from the edge of the panel to the screw. One of the tests of Wall C experienced a failure in the Fiberboard on the tension side of the wall. A break in the Fiberboard panel where the Fiberboard failed along the four foot length of a single panel occurred, leaving a small strip of Fiberboard screwed to the bottom plate.

On the compression side of the wall, the drywall screws usually failed in shear along the interface of both the Fiberboard and the OSB for wall configurations A, B, and C. This would occur until prohibited from continuing due to the resistance of the bottom plate to crushing from the end studs.

Wall D experienced a different mode of failure than other walls. The addition of the tie-down anchors moved the mode of failure from the bottom plate to the top plate and the middle connection of the walls. Two tests were performed monotonically on Wall D. One of the walls failed along the top plate. The top plate separated from the wall when the drywall screws failed in shear along the interface of both the OSB and the Fiberboard to the bottom plate. The second test failed along the middle connection of the wall as well as along the top plate of the wall. The 2x4 connection between the two 4x8 SIPS failed when the drywall screws tore through the Fiberboard and failed in shear along the OSB/2x4 interface. This allowed the wall to rack similar to a light-framed timber shear wall with tie-down anchorage. This racking resulted in a few of the screws along the corners of the individual panels at the bottom plate to fail in shear and the top plate failed by the shearing of the screws along both interface planes as happened in the previous test.

The failure modes of the different wall configurations suggest that Wall D behaves more in a racking and shear fashion and Walls A, B, and C behave more as rigid bodies.

4.4 Conclusions

The following conclusions can be drawn from the monotonic data presented and discussed in this chapter.

- Vertical connecting elements have no effect on the capacity of the structurally insulated panels.

- The bottom connection of SIPS is the critical connection of the wall to be considered when designing shear walls. Addition of tie-down anchors to a SIPS shear wall will move the failure away from the bottom plate.

- Addition of a second base plate to a SIPS shear wall system will increase capacity of the shear wall significantly.

- Addition of tie-down anchors to a SIPS shear wall system will greatly increase the capacity as well as stiffness of the shear wall. This results in the best system for monotonic loading because of the high capacity and ability to not deflect much under load. Addition of tie-down anchors will also decrease long term damage to the structure that experiences high peak loads.

- Addition of tie-down anchors to a wall allows the SIPS shear walls to behave more like a shear element than like a rigid body. A wall without tie-down anchors behaves more like a rigid body.

- There is little difference in the results obtained for capacity, stiffness, and ductility of SIPS tested under straight one-directional monotonic loading and the loading scheme prescribed by ASTM E564.

4.5 Summary

The load-deflection behavior of the four different wall configurations of structurally insulated panels tested under two types on monotonic loading is discussed in this chapter. Parameters of load resistance and drift at yield, maximum resistance, and failure were defined. The concept of an equivalent elastic plastic curve was utilized to define and present values of elastic stiffness and ductility. Behavior of the walls was quantified by defining the parameter of set ratio. Qualitative behavior of the four SIPS wall configurations was also presented.