Chapter 4: Test Results, Analysis and Interpretations

4.1 Introduction

Objectives of the current research were to perform necessary testing and analyses to allow timber frame building designers to make use of the in-plane strength and stiffness of SIPs to resist lateral loads utilizing diaphragm action. It was therefore necessary to utilize data obtained from previously discussed testing and extract that which was required to make use of ASAE 484.2 (ANSI/ASAE, 1999) diaphragm design procedures and IBC 2000 (ICC, 2000) seismic design procedures. In this chapter are described results of tests conducted on five roof assemblies, analyses of strength and stiffness parameters acquired from testing, and recommendations for interpreting obtained test data to establish methods for future testing and design of timber frame structures incorporating diaphragm action. Test results include descriptions of each test, including failure modes and testing details, and monotonic strength for completed timber frame and SIP roof assemblies. Analyses were performed utilizing assembly load and deflection data from monotonic failure tests to calculate ultimate and allowable panel shear strength, test diaphragm shear stiffness and building diaphragm stiffness. Cyclic test load and deformation data for configurations described in Chapter 3 were used to determine cyclic stiffness, equivalent viscous damping, damped hysteretic energy and strain energy. Methods were recommended, based on test results and analyses, for incorporating roof assembly testing data into design procedures for timber frame and SIP structures as discussed in Chapter 5, along with design examples utilizing obtained data.

4.2 General Roof Assembly Testing Behavior

4.2.1 Assembly 1

Assembly 1 was 8 ft (2.44 m) long and 24 ft (7.32 m) wide, constructed as described in Chapter 3. During final construction of Assembly 1, just prior to installation of edge boards, there was a failure of the hydraulic controlling device, which converted computer input into commands for the hydraulic pump and valves. Failure occurred during the second week of January 2002 and the system was not functional again until the first week of June 2002. Every effort was made to keep the partially constructed
assembly dry by covering it with polyethylene and keeping rain and snow off the covering during the winter months. Unfortunately, moisture beneath the covering could not be fully avoided and the assembly was partially wetted 8 to 10 times over the 6-month duration. Slight puckering of the OSB around several of the SIP screws on top of Assembly 1 was the only observable damage prior to final testing. Results indicated that stiffness of the assembly was potentially compromised due to repeated drying and wetting cycles of timbers and SIPs. Weathering appeared to have much less impact on assembly strength, as further discussed in Section 4.3.1.

After replacement of the hydraulic controller, the regime of cyclic stiffness tests (Table 3.3) were completed and no damage was observed as a result of these tests. Assembly 1 was pulled toward the hydraulic actuator at a rate of 0.17 in./minute (4.3 mm/minute) to failure. It was assumed that the strapped joint at the top of the center rafter (Figure 3.7) would fail in tension before the splined timber joint at the bottom of the center rafter (Figure 3.11) would fail in tension due to chord forces. Therefore pulling the assembly to failure would presumably have resulted in a lower strength and provided the worst possible scenario for roof panel assembly strength. After some consideration, it was noted that typically there is a timber member (ridge purlin for example) at the roof ridge to which SIPs from both sides of the roof are fastened. A common member at the ridge results in the chord forces being offset by one another at the ridge to SIP connections, assuming the slope lengths of both sides are equal. Specifically, tension chord forces from the windward roof slope are counterbalanced by compression chord forces from the leeward roof slope, resulting in a negation of chord forces at the ridge. Assuming a common member at the ridge indicated that pushing remaining assemblies to failure would be a more conservative method of assessing roof assembly strength since chord forces would be greater at the eave line in an actual building than at the ridge.

A plot showing load versus displacement for Assembly 1 is shown in Figure 4.1. Displacements were acquired using an LVDT inside the hydraulic actuator and by a string potentiometer mounted to the concrete wall, with the string attached to the top of the center rafter. String potentiometers were also mounted to the concrete wall and attached to the tops of each outer rafter to determine displacements of the simple beam
Figure 4.1. Load versus displacement for Assembly 1 test to failure. Ram displacements were acquired utilizing internal LVDT on hydraulic actuator, global displacements were acquired utilizing string potentiometer attached to concrete wall and center rafter and adjusted global displacements included compensation for rigid body motion by subtracting off the average of the movement at the outer rafter supports.
supports. Adjusted global displacements were calculated by subtracting the average of the support displacements from center rafter displacement recorded using the string potentiometer. Bending of the steel channel connected to the hydraulic ram and center rafter was observed during testing, resulting in slight differences between the two recorded displacements. Adjusted global displacements were used for all monotonic analyses, regardless of observed deflections of the steel channel. Global displacement data was omitted from remaining assembly plots for clarity. Maximum attained load for Assembly 1 was 10,600 lbs (47,200 N) and occurred just prior to a rapid decline in the applied load at a displacement of 1.41 in. (36 mm).

Failure of Assembly 1 was initiated by consecutive failures of screws attaching SIPs to the timber frame. As applied load approached approximately 50% of ultimate load, screws could be heard breaking and continued to do so at random intervals up to assembly failure. Condition and location of screws extracted after testing are illustrated in Figure 4.2, depending on whether screws were bent, broken off, or not damaged. Remaining assembly SIP screw condition and location diagrams are presented in Appendix B. As a result of pulling Assembly 1 to failure, both steel straps at the top of the center rafter and the steel strap holding intermediate purlins to the center rafter were broken. Figure 4.3 illustrates separation of the SIPs and broken straps at the top of the center rafter. No significant damage was observed to either SIPs or timber frame for Assembly 1, and upon dismantling, all pegs were easily withdrawn from timber frame joints with no visible wear or compression set. After removal of SIPs from the timber frame, it was noted that some screws attaching the SIPs to the timber frame along the plates had missed the OSB spacer, which resulted in 19/32 in. (15 mm) of the screw threads being exposed. It was surmised that diaphragm strength and stiffness would be compromised by exposed screws and subsequent installation of SIPs for remaining assemblies utilized a wider OSB strip along the top of the plates to ensure that screws were fully encased within OSB spacers.

4.2.2 Assembly 2

Assembly 2 was 8 ft (2.44 m) long and 24 ft (7.32 m) wide, constructed as described in Chapter 3. After considering results from Assembly 1 testing, an explorative approach was pursued regarding Assembly 2 tests in order to determine potential effects
Figure 4.2. Condition and location of screws attaching SIPs to timbers following failure of Assembly 1. (BR indicates broken screws, B indicates bent screws, ND indicates no damage to screws and NA indicates screws that could not be assessed or withdrawn)
Figure 4.3. Separation of SIPs and broken strap at the top of center rafter following failure of Assembly 1.
of chord forces on behavior of roof test assemblies. After the regime of cyclic stiffness tests (Table 3.4) were conducted, none of which caused any damage, Assembly 2 was pulled toward the hydraulic actuator at a rate of 0.17 in./minute (4.3 mm/minute) to a load of approximately 4,000 lbs (17,800 N), which coincided with a displacement of approximately 0.25 in. (6.4 mm). Based on Assembly 1 testing, 4,000 lbs (17,800 N) was estimated to be approximately 40% of ultimate failure load, or design load. It was assumed that this test would result in a failure of some of the SIP screws or steel straps although no damage was observed nor sounds indicating screw breakage were heard during the test. A cyclic test was then conducted utilizing a scaled version of the previously utilized waveform with a maximum displacement of 0.25 in. (6.4 mm). An example of this waveform is presented in Appendix A. No degradation was observed during the cyclic test, which indicated that assembly stiffness could be considered linear for displacements up to 0.25 in. (6.4 mm). Remaining cyclic stiffness test regimes included a cyclic stiffness test with a maximum displacement of 0.25 in. (6.4 mm) for the bare timber frame and for the completed assembly. Failure was induced in Assembly 2 by pushing the assembly away from the hydraulic actuator at a rate of 0.17 in./minute (4.3 mm/minute). Excessive downward bending of the steel channel bolted to the hydraulic actuator and center was observed as the test neared completion. Breaking SIP screws were heard and significant flattening of the load-deflection plot was noted, leading to termination of the Assembly 2 test, which was considered failed with a maximum load of 11,600 lbs (51,600 N) at a displacement of 0.789 in. (20 mm).

A plot illustrating load versus displacement behavior for Assembly 2 is shown in Figure 4.4. Excessive bending of the steel channel connected to the hydraulic ram and center rafter was observed during testing, resulting in significant differences between the global and actuator displacements. After all tests were conducted on Assembly 2, a section of 3 x 5 x 3/8 in. (76 x 13 x 9.5 mm) steel tube, 9.5 ft (2.90 m) long, was welded, in the weak direction, onto the bottom of the steel channel to stiffen it for remaining tests. The end of the steel tube began 2.12 ft (0.65 m) from the steel plate that bolted to the hydraulic actuator. A 1/4 in. (6.4 mm) thick steel plate was welded between the flanges of the steel channel between the end of the reinforcing tube and steel plate that bolted to the hydraulic actuator to further stiffen the section while allowing it to pass between the
Figure 4.4. Load versus displacement for Assembly 2 test to failure. Ram displacements were acquired utilizing internal LVDT on hydraulic actuator. Adjusted global displacements were acquired utilizing string potentiometers attached to concrete wall and rafters and include compensation for rigid body motion by subtracting off the average of the movement at the outer rafter supports.
Chapter 4: Test Results, Analysis and Interpretations

4.2 Restraint Testing

4.2.2 Assembly 2

While Assembly 2 was considered failed due to fastener breakage after the monotonic test, a final cyclic failure test was conducted on Assembly 2 utilizing failure data from Assembly 1 to obtain a reference deformation of 1.02 in. (26 mm) for use with the Krawinkler et al. (2000) waveform described and illustrated in Appendix A. Continued bending of the steel channel previously discussed was responsible for termination of this cyclic test. Condition and location of screws extracted after testing are illustrated in Appendix B. After the final cyclic test, it was observed that both steel straps at the top of the center rafter and the steel strap holding the intermediate purlins to the center rafter were broken. The strap that was visible with the SIPs attached was not broken before the final cyclic test. No significant damage was observed to either the SIPs or timber frame components of Assembly 2, and upon dismantling, all pegs were easily withdrawn from timber frame joints with no visible wear or compression set.

4.2.3 Assembly 3

Assembly 3 was 8 ft (2.44 m) long and 24 ft (7.32 m) wide, constructed as described in Chapter 3. After the updated regime of cyclic stiffness tests (Table 3.5) were conducted, which included tests on the bare frame and completed assembly with maximum displacements of 0.25 in. (6.4 mm), none of which caused any damage, Assembly 3 was pushed away from the hydraulic actuator at a rate of 0.17 in./minute (4.3 mm/minute) to failure. A plot showing load versus displacement for Assembly 3 is shown in Figure 4.5. Maximum attained load for Assembly 3 was 12,600 lbs (56,100 N), and occurred at an adjusted global displacement of 0.899 in. (23 mm).

Failure of Assembly 3 was initiated by consecutive failures of screws attaching the SIPs to the timber frame. As the load approached approximately 50% of ultimate load, screws could be heard breaking and continued to do so at random intervals up to assembly failure. Condition and location of SIP screws extracted after testing are illustrated in Appendix B. Only the steel strap holding the intermediate purlins to the center rafter was broken during the test. Decrease in load with continued deflection was attributed to SIP screw failure, whereas the large drop in the load-deflection plot at about 12,000 lbs (53.4 kN) in Figure 4.5 was due to fracture of the post stub on the A side of the timber frame into three sections, as shown in Figure 4.6. Drying checks along growth
Figure 4.5. Load versus displacement for Assembly 3 test to failure. Ram displacements were acquired utilizing internal LVDT on hydraulic actuator. Adjusted global displacements were acquired utilizing string potentiometers attached to concrete wall and rafters and include compensation for rigid body motion by subtracting off the average of the movement at the outer rafter supports.
Figure 4.6. Fractured post stub from A side of Assembly 3 test after failure. Cracks were observed in the post stub prior to testing.
rings of the post stub were noted prior to testing. In a timber frame building, post stubs would typically be full-length members at least 8 ft (2.44 m) long and would likely have more integrity than stubs utilized for testing purposes, therefore this fracture was not considered the primary failure mode for Assembly 3. No significant damage was observed to the SIPs for Assembly 3 and upon dismantling, pegs were easily withdrawn with no apparent damage.

4.2.4 Assembly 4

Assembly 4 was 20 ft (6.10 m) long and 24 ft (7.32 m) wide, constructed as described in Chapter 3. The regime of cyclic stiffness tests (Table 3.6) were conducted, none of which caused any damage, and Assembly 4 was pushed away from the hydraulic actuator at a rate of 0.17 in./minute (4.3 mm/minute) to a maximum load of 36,000 lbs (160,000 N), at which point the steel channel connecting the hydraulic actuator to the center rafter buckled. Buckling occurred where the reinforcing steel tube on the underside of the channel ended and the 1/4 in. (6.4 cm) thick steel plate on the underside began. Although screw breakage was heard during the test, the decision was made to return Assembly 4 to zero displacement, further reinforce the steel channel, and push the assembly to complete failure. Considering the damage observed during the initial test, it was elected to calculate stiffness for Assembly 4 from the first test and strength of Assembly 4 from the second test to complete failure. Modifications were performed to the steel channel and the restraining rollers as seen in Figure 4.7. A cyclic stiffness test that utilized a maximum displacement of 0.25 in. (6.4 mm) was conducted before the final monotonic failure test and indicated that Assembly 4 stiffness had not been reduced by the previous failure test. Assembly 4 was taken to failure by being pushed away from the hydraulic actuator at a rate of 0.17 in./minute (4.3 mm/minute) to a maximum load of 38,300 lbs (170,000 N), at a displacement of 1.33 in. (34 mm).

A plot illustrating load versus displacement behavior for both monotonic tests on Assembly 4 is shown in Figure 4.8. Assembly 4 was considered failed due to fastener breakage, as with previous assemblies. Data acquired and shown in Figure 4.8 for test 4B, the test to final failure, were limited by the time allotment within data acquisition software. Assembly 4 continued to be pushed until the post stub on the C side of the frame fractured, causing the plate on that side of the assembly to fall out of the stub, as
Figure 4.7. Shown is reinforcement welded onto steel channel to eliminate compression buckling prior to failure test of Assembly 4. Modifications were made to roller assembly to accommodate reinforcement. Steel tubing seen beneath timbers provides bearing for underside of channel and resists downward movement of channel.
Figure 4.8. Load versus displacement for Assembly 4 tests. Test 4A was the initial test during which the steel channel buckled. Test 4B pushed the assembly to complete failure. Ram displacements were omitted for clarity. Adjusted global displacements were acquired utilizing string potentiometers attached to concrete wall and rafters and included compensation for rigid body motion by subtracting off the average of the movement at the outer rafter supports.
shown in Figure 4.9. Figure 4.10 shows the fractured post stub after dismantling the frame.

Condition and location of screws extracted after testing Assembly 4 are illustrated in Appendix B. It appeared that screws were failing in a single shear Mode IV failure, as described in NDS (AF&PA, 1997) Appendix I. An example of an extracted screw is shown in Figure 4.11. After failure and dismantling, it was observed that steel straps at all intermediate purlin connections along the center rafter were broken and all other straps remained undamaged. Figure 4.12 provides an example of strap damage at the joint where intermediate purlins closest to the eave plates (farthest from the hydraulic actuator) connected to the center rafter. No significant damage was inflicted on the SIPs for Assembly 4, as noted upon dismantling. Pegs holding timber frame joints together were damaged where outer rafter on the C side of the frame connected to the post stub as seen in Figure 4.13. Wear and slight crushing damage were observed in pegs in the spline joint at bottom of the center rafter (Figure 4.14) and tenon joints where plates connected to post stubs on both sides of the frame. All other pegs were easily withdrawn from timber frame joints with no visible wear or compression set.

4.2.5 Assembly 5

Assembly 5 was 20 ft (6.10 m) long and 24 ft (7.32 m) wide, constructed as described in Chapter 3. After the full regime of cyclic stiffness tests (Table 3.7) were conducted which included tests on the bare frame and completed assembly with maximum displacements of 0.25 in. (6.4 mm), none of which caused any damage, Assembly 5 was pushed away from the hydraulic actuator at a rate of 0.17 in./minute (4.3 mm/minute) to failure. A plot showing load versus displacement for Assembly 5 is shown in Figure 4.15. Maximum attained load for Assembly 5 was 37,600 lbs (167,000 N), and occurred at an adjusted global displacement of 1.60 in. (41 mm).

Assembly 5 was considered failed due to fastener breakage, as with previous assemblies. As the load approached approximately 50% of ultimate load, screws could be heard breaking and continued to do so constantly throughout the test up to assembly failure. Descriptions from observers included comments like “popcorn” and “fireworks” regarding the sounds of screws breaking during testing. Condition and location of screws extracted after testing Assembly 5 are illustrated in Appendix B. It appeared that screws
Figure 4.9. Collapse of eave plate BC after complete failure of Assembly 4. Collapse was initiated by failure of post stub on C side.
Figure 4.10. Fractured post stub from C side of Assembly 4 after failure. No cracks were observed in the post stub prior to testing.
Figure 4.11. Screw extracted from Assembly 4 after testing. The deformed “S” shape of the screw indicates single shear Mode IV failure of the fasteners (AF&PA, 1997) and was typical of fasteners removed from Assembly 4.
Figure 4.12. Example of strap breakage from Assembly 4 after testing. The strap shown was used to secure the intermediate purlins closest to the eave (farthest from the hydraulic actuator) to the center rafter.
Figure 4.13. Damaged pegs utilized to hold timber frame joint together where outer rafter on the C side of the frame connected to the post stub. Wood fibers are compressed on top side and fractured in tension on the bottom side.
Figure 4.14. Damaged Assembly 4 pegs utilized to hold timber frame joint together where eave plates connect to center post stub utilizing oak spline. Pegs show wear near the top (end without writing) due to bearing of the spline against them.
Figure 4.15. Load versus displacement for Assembly 5 test to failure. Ram displacements were acquired utilizing internal LVDT on hydraulic actuator, global displacements were acquired utilizing string potentiometer attached to concrete wall and center rafter and adjusted global displacements included compensation for rigid body motion by subtracting off the average of the movement at the outer rafter supports.
were failing in a single shear Mode IV failure, as described in NDS (AF&PA, 1997), Appendix I. Examples of extracted screws are shown in Figure 4.16. Global buckling, or bowing was noted near the bottom of the center rafter as shown in Figure 4.17, as maximum load was achieved and progressed for the remainder of the test. Dismantling Assembly 5 revealed that no steel straps were broken. Separation of SIPs and edge boards was observed, as shown in Figure 4.18, most notably where global buckling was observed during testing. No significant damage was observed on the SIPs or timber frame for Assembly 5 upon dismantling. Wear and slight crushing damage were observed in pegs in the spline joint at the bottom of the enter rafter (Figure 4.19) and tenon joint where plates connected to post stubs on C side of the frame. All other pegs were easily withdrawn from timber frame joints with no visible wear or compression set.

4.2.6 Roof Assembly Testing Comments

Monotonic and cyclic testing of five timber frame and SIP roof assemblies provided ample qualitative and quantitative information in order to accomplish previously discussed research objectives. Analyses of data are presented in following sections. Failure of all assemblies was a result of breaking of the screws attaching SIPs to the timber frames. Specifically, once enough screws were broken, the load required to deflect the assemblies declined until the test was terminated or until another breakage, such as the timber post stubs occurred. The assumptions that SIPs provided a very stiff plate and timbers were rigid members free to rotate at the joints were based on observed failures and were used to derive a relationship between cyclic testing analysis results and number of screws broken and bent as a result of assembly failure. Clearly the properties of SIP screws were critical in determining strength and stiffness of assemblies and should be taken into consideration for design purposes. Screws utilized were manufactured from hardened steel, which resulted in a non-ductile connection between the SIPs and timbers, observed as shearing rather than yielding of screws during failure tests. Nails utilized in timber frame and SIP construction are typically more ductile fasteners and would result in more ductile roof systems, although further quantification of different SIP to timber fasteners is left to future research, as discussed in Chapter 6.
Figure 4.16. Screws extracted from Assembly 5 after testing. The deformed “S” shape of the screws indicates single shear Mode IV failure of the fasteners (AF&PA, 1997) and was typical of fasteners removed from Assembly 5.
Figure 4.17. Global buckling observed near the bottom of the center rafter in Assembly 5 during testing.
Figure 4.18. Separation of SIPs and edge boards observed near the bottom of the center rafter in Assembly 5 during testing, near the location of global assembly buckling. Spaces between SIP skins and edge boards were tight at the outset of testing.
Figure 4.19. Damaged pegs utilized in Assembly 5 to hold timber frame joint together where eave plates connect to center post stub utilizing oak spline. Pegs show wear near the top (end without writing) due to bearing of the spline against them.
4.3 Roof Panel Testing Analyses

4.3.1 Monotonic Test Analyses

Shear strength and shear stiffness are currently undefined parameters for diaphragm design of timber frame and SIP roof assemblies. ASAE EP558 (ASAE, 1999) recommended use of a simple beam test that utilized two loads at the 1/3 points of the span rather than the single point load prescribed for this research and recommended under ASAE EP484.2 (ASAE, 1999). It was necessary to slightly modify the ASAE equations for determining shear stiffness, as described below to coincide with the use of two-bay assemblies utilizing a single point load applied to the center rafter.

Ultimate shear strength, \( V_{ult} \), was determined utilizing the following equation:

\[
V_{ult} = \frac{P_{ult}}{(2b)}
\]

where,

\( V_{ult} \) = assembly shear strength at maximum load, lbs/ft (N/m),
\( P_{ult} \) = maximum load at assembly failure, lbs (N), and
\( b \) = diaphragm length measured from ridge to eave, ft (m).

The appropriate factor of safety of 2.5 was established, as described in Section 3.3.1, and indicated allowable design shear strength of a test roof panel should be calculated from collected test data by multiplying test ultimate shear strength, \( V_{ult} \), by 0.4, which is the inverse of the factor of safety, 2.5. According to ASAE EP558 (ASAE, 1999), if lumber breakage or fastener failure within the wood were the cause of assembly failure, then allowable design shear strength was equal to \( 0.4(V_{ult}) \), divided by the appropriate load duration factor as determined using NDS-1997 (AF&PA, 1997). For all other failure cases, allowable design shear strength was equal to \( 0.4(V_{ult}) \). In this study, failure occurred in the fasteners and was not considered wood failure; therefore allowable design shear strength for test assemblies was calculated utilizing the following equation:

\[
V_a = 0.4(V_{ult})
\]

where,

\( V_a \) = allowable test assembly shear strength, lbs/ft (N/m).

Effective test diaphragm shear stiffness, \( c \) in lbs/in. (N/mm), was determined using the following equation:

\[
c = 0.4(P_{ult})/D_f
\]
where,

\[ D_T = \text{adjusted load-point deflection at } 0.4(P_{ult}), \text{ in. (mm)}. \]

Adjusted load-point deflection, \( D_T \), was calculated utilizing the equation:

\[ D_T = D_B - 0.5(D_A + D_C), \quad (4.4) \]

where,

\[ D_B = \text{displacement measured at top of center rafter at } 0.4(P_{ult}), \text{ in. (mm)}, \]
\[ D_A = \text{displacement measured at support on A side of test frame at } 0.4(P_{ult}), \text{ in. (mm)}, \]
\[ D_C = \text{displacement measured at support on C side of test frame at } 0.4(P_{ult}), \text{ in. (mm)}. \]

Deflection measurements \( D_A, D_B \) and \( D_C \), were recorded at locations utilizing channels 11, 12 and 13, respectively, as illustrated in Figure 3.14. Utilizing adjusted displacement, \( D_T \), eliminated rigid body movement due to the sinking of reaction supports at deflections 11 and 13 in Figure 3.14. Effective shear modulus, \( G \), was calculated utilizing the following equation:

\[ G = c(a/b), \quad (4.5) \]

where,

\[ G = \text{effective shear modulus, lbs/in. (N/mm), and} \]
\[ a = \text{frame spacing, ft (m).} \]

The “frame spacing” for the reported tests was 11 ft (3.35 m). A summary of results obtained from monotonic failure tests of Assemblies 1 through 5 is presented in Table 4.1 and discussed below. Comparative load versus deflection plots for the three for 8 ft x 24 ft (2.44 m x 7.32 m) wide roof assemblies, the two 20 ft x 24 ft (6.10 m x 7.32 m) wide roof assemblies and for all five tested assemblies are presented in Figure 4.20,4.21 and 4.22, respectively.

According to ASAE EP558 (ASAE, 1999), when only two assemblies were tested, the lowest calculated allowable design shear strength should be utilized to determine allowable design shear strength for design purposes, and if three or more assemblies were tested, allowable design shear strength for design purposes should be calculated by averaging calculated allowable design shear strengths for all tests. As previously mentioned, there was reason to believe that data obtained from testing of
Table 4.1. Summary of monotonic failure test data for timber frame and SIP roof diaphragm assemblies. Assemblies 1, 2 and 3 were 8 ft x 24 ft (2.44 m x 7.32 m) and Assemblies 4 and 5 were 20 ft x 24 ft (6.10 m x 7.32 m).

<table>
<thead>
<tr>
<th>Assembly</th>
<th>Maximum Applied Load, $P_{ult}$</th>
<th>Ultimate Shear Strength, $V_{ult}$</th>
<th>Allowable Design Shear Strength, $V_a$</th>
<th>Effective Shear Stiffness, $c$</th>
<th>Effective Shear Modulus, $G$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1*</td>
<td>10,600 lbs (47.2 kN)</td>
<td>663 lbs/ft (9.68 kN/m)</td>
<td>265 lbs/ft (3.87 kN/m)</td>
<td>22,100 lbs/in. (3.86 kN/mm)</td>
<td>30,400 lbs/in. (5.31 kN/mm)</td>
</tr>
<tr>
<td>2</td>
<td>11,600 lbs (51.6 kN)</td>
<td>725 lbs/ft (10.6 kN/m)</td>
<td>290 lbs/ft (4.24 kN/m)</td>
<td>44,600 lbs/in. (7.92 kN/mm)</td>
<td>61,300 lbs/in. (10.9 kN/mm)</td>
</tr>
<tr>
<td>3</td>
<td>12,600 lbs (56.1 kN)</td>
<td>788 lbs/ft (11.5 kN/m)</td>
<td>315 lbs/ft (4.60 kN/m)</td>
<td>41,700 lbs/in. (7.23 kN/mm)</td>
<td>57,300 lbs/in. (9.94 kN/mm)</td>
</tr>
<tr>
<td>4</td>
<td>38,300 lbs (170 kN)</td>
<td>958 lbs/ft (13.9 kN/m)</td>
<td>383 lbs/ft (5.56 kN/m)</td>
<td>107,000 lbs/in. (18.9 kN/mm)</td>
<td>58,900 lbs/in. (10.4 kN/mm)</td>
</tr>
<tr>
<td>5</td>
<td>37,600 lbs (167 kN)</td>
<td>940 lbs/ft (13.7 kN/m)</td>
<td>376 lbs/ft (5.48 kN/m)</td>
<td>133,000 lbs/in. (23.0 kN/mm)</td>
<td>73,200 lbs/in. (12.7 kN/mm)</td>
</tr>
</tbody>
</table>

*Weathered assembly; not used for determination of shear strength and stiffness for 8 ft by 24 ft (2.44 m x 7.32 m) assemblies.
Figure 4.20. Load versus displacement for 8 ft x 24 ft (2.44 m x 7.32 m) Assemblies 1, 2 and 3 utilizing adjusted global displacements.
Figure 4.21. Load versus displacement for 20 ft x 24 ft (6.10 m x 7.32 m) Assemblies 4 and 5 utilizing adjusted global displacements. Results for Assembly 4 were those obtained from the initial test (4A), from which stiffness, not strength data was gathered.
Figure 4.22. Load versus displacement for 8 ft x 24 ft (2.44 m x 7.32 m) Assemblies 1, 2 and 3 and 20 ft x 24 ft (6.10 m x 7.32 m) Assemblies 4 and 5 utilizing adjusted global displacements. Results for Assembly 4 were those obtained from the initial test (4A), from which stiffness, not strength data was gathered.
Assembly 1 was compromised due to moisture cycling the assembly underwent during the 6 months (January 2002 through June 2002) that it was exposed while waiting for a hydraulic controller replacement. As seen in Table 4.1, the shear strength of Assembly 1 was 91% and 84% that of Assembly 2 and Assembly 3, respectively, but the shear stiffness was 50% and 53% of Assembly 2 and Assembly 3, respectively. Differences in stiffness between Assembly 1 and the other two similar assemblies are significant enough to warrant the use of only Assembly 2 and Assembly 3 for the determination of shear strength and stiffness for the 8 ft x 24 ft (2.44 m x 7.32 m) timber frame and SIP roof assemblies tested. Therefore, results from testing of timber frame and SIP roof assemblies provided allowable design shear strength of 290 lbs/ft (4.24 kN/m) for 8 ft x 24 ft (2.44 m x 7.32 m) wide roof assemblies and 376 lbs/ft (5.48 kN/m) for 20 ft x 24 ft (2.44 m x 7.32 m) wide roof assemblies. Provisions of ASAE EP558 (ASAE, 1999) dictated that regardless of the number of tests, effective shear stiffness for an assembly was calculated by averaging all test assembly results. Therefore, results from testing of timber frame and SIP roof assemblies provided effective shear stiffness of 43,200 lbs/in. (7.58 kN/mm) for 8 ft x 24 ft (2.44 m x 7.32 m) wide roof assemblies and 120,000 lbs/in. (21.0 kN/mm) for 20 ft x 24 ft (2.44 m x 7.32 m) wide roof assemblies.

Test procedures for diaphragm strength and stiffness tests in ASAE EP558 (ASAE, 1999) specified that if strength and stiffness test results were not within 10% of the lower values that a third specimen should be tested. Shear strength and stiffness values for Assemblies 2 and 3 were within 10% of lower values. Allowable strength values for 20 ft (2.44 m) long, 24 ft (7.32 m) wide roof assemblies were also with 10% of each other, while effective stiffness values varied by 24% of the lower value. According to an ASAE committee member, the 10% was not intended to apply to stiffness. Because stiffness variation of 25% plays only a minor role in the magnitude of frame forces in diaphragm analysis, additional testing in an attempt to reduce the stiffness variation was deemed unnecessary.

Discussion on interpretation of monotonic test data is presented in Section 4.4.1. Recommendations for incorporation of monotonic data from timber frame and SIP roof assembly tests for inclusion with ASAE diaphragm design methods are discussed in Chapter 5.
4.3.2 Quasi-Static Cyclic Test Analyses

In addition to the monotonic strength and stiffness test data needed for diaphragm design of buildings constructed of timber frame and SIPs, current building codes also lack information regarding cyclic performance of these structures that would allow designers to assess their resistance to seismic ground accelerations experienced during earthquakes. Prion and Filiatrault (1996) discussed performance of heavy timber and post-and-beam buildings during the Kobe earthquake of 1995 and concluded that the main causes of failure in these building types were lack or minimal use of effective lateral load resisting systems. These Japanese buildings were similar to timber frame structures under investigation in the current research. Tests conducted utilizing cyclic stiffness protocols aimed to evaluate the effectiveness of timber frame and SIP roof systems employing the quasi-static cyclic testing protocol are discussed in Chapter 3 and Appendix A.

According to the International Building Code 2000 (ICC, 1998), buildings must provide adequate strength, stiffness and energy dissipation through lateral and vertical force restraining systems to withstand ground accelerations based on geographic location. Determining cyclic stiffness and energy damping characteristics of timber frame and SIP roof systems was accomplished through cyclic load-deflection hysteresis analysis. These hysteresis analyses were derived from load-displacement curves resulting from the quasi-static cyclic tests.

Delineating dynamic parameters of timber frame and SIP roof framing systems was pursued in order to incorporate these parameters within existing building standards for seismic design of these buildings. Current IBC 2000 (ICC, 2000) seismic design procedures did not specifically accommodate timber frame and SIP construction as an option when selecting a basic seismic-force-resisting system. Designers were forced to use the “other systems” category for the basic seismic-force-resisting system, which resulted in overly conservative values for the response modification factor, system over-strength factor and deflection amplification factor, all of which were selected on the basis of seismic-force-resisting system. It was therefore intended, by conducting quasi-static cyclic testing of timber frame and SIP roof assemblies, to extract enough information
about their dynamic responses to make recommendations to the International Code Council regarding inclusion of these structural types within earthquake design provisions.

Results from cyclic regimes previously described provided load and deflection data for timber frame and SIP roof assemblies that were used to generate values for cyclic stiffness, strain energy, damped hysteretic energy and equivalent viscous damping for each of the tested assemblies. Load versus deflection plots for cyclic testing resulted in a series of hysteresis loops, rather than single lines obtained from monotonic tests. An example of a hysteresis plot for Assembly 2, after installation of edge board and prior to failure testing is shown in Figure 4.23. Figure 4.23 also illustrates the stiffness, or load, envelope curve which tracks the load extremities for hysteresis loops for the last five primary peak cycles for the test, as described in Appendix A. Cyclic envelope curves typically resemble load versus deflection plots for monotonic tests and are often used to compare monotonic and cyclic data (Heine and Dolan, 1998).

Each cyclic test for configurations described in Chapter 3 provided data for development of hysteresis plots from which were derived cyclic behavior parameters. A typical hysteresis loop plot is shown in Figure 4.24. Cyclic stiffness, like monotonic stiffness, represented a measure of load required to produce a specified displacement of the system being tested. Cyclic stiffness was obtained by calculating the slope of the line between points B and D in Figure 4.24 for hysteresis loops for the 7 primary peak cycles for each test configuration. Strain energy \( S_e \) was a measure of system potential energy, or the energy associated with the deformation of the assembly (Beer and Johnston, 1992) at a given displacement and was determined by totaling the area beneath triangles ABC and ADE in Figure 4.24 for each hysteresis loop. The area within the hysteresis loop represented the energy dissipated by the assembly for each cycle (Chopra, 1995) and was determined as hysteretic energy \( H_e \). Comparing the energy dissipated per cycle per radian \( H_e/2\pi \) with strain energy \( S_e \) provided equivalent viscous damping ratios \( \eta \) for each hysteresis loop utilizing the following:

\[
\eta = \frac{H_e}{2\pi S_e} \tag{4.6}
\]

where,

\[\eta = \text{equivalent viscous damping ratio, (decimal)},\]

\[H_e = \text{hysteretic energy, lbs-in. (N-mm)}, \text{and}\]
Figure 4.23. Example of hysteresis plot and stiffness envelope for Assembly 2 after completion of construction and prior to failure test.
Figure 4.24. Example of typical hysteresis loop plot.
\[ S_e = \text{strain energy, lbs-in. (N-mm)}. \]

Equivalent viscous damping refers to energy dissipated from a vibrating system and is often utilized to quantify dissipated energy of cyclically loaded wood structural systems (Heine and Dolan, 1998).

Data from cyclic testing provided load and displacement values from which cyclic stiffness, strain energy, hysteretic energy and equivalent viscous damping were calculated for all assemblies. Figures 4.25, 4.26, 4.27 and 4.28 illustrate comparisons among testing construction configurations for Assembly 2 for cyclic stiffness, strain energy, hysteretic energy and equivalent viscous damping, respectively. Table 4.2 provides numerical values for testing configurations for Assembly 2 for cyclic stiffness, strain energy, hysteretic energy and equivalent viscous damping at maximum displacements. Plots and tabulated data for remaining test assemblies are provided in Appendix C. Discussion on trends and interpretation of cyclic test data is presented in Section 4.4.2. Recommendations for potential incorporation of cyclic data from timber frame and SIP roof assembly tests for inclusion with seismic design methods are discussed in Chapter 5.

4.4 Interpreting Test Assembly Data

4.4.1 Monotonic Test Data Interpretations

Having calculated shear strength and stiffness for the four tested roof assemblies, there remains the issue of how to utilize these data for determination of building diaphragm shear stiffness, the critical stiffness parameter for use with ASAE EP484.2 (ASAE, 1999) design procedures. In order for designers to be able to make use of data obtained from testing 8 ft (2.44 m and 7.32 m) long assemblies, design rules need to be established for calculating diaphragm strength and stiffness for roofs with slope lengths other than 8 ft (2.44 m) and frame spacing other than 11 ft (3.4 m). Determination of test panel stiffness adjusted for length is discussed below and allows for comparisons with procedures utilized in Carradine et al. (2000) to design timber frame and SIP roof assemblies for use as structural diaphragms, as discussed in Chapter 5.

ASAE EP484.2 (ASAE, 1999) and ASTM E 455 (ASTM, 2000) discussed methods for extrapolating strength and stiffness of building diaphragms from smaller test panel diaphragms based on the assumption that strength and stiffness increased linearly
Figure 4.25. Cyclic stiffness as a function of displacement for Assembly 2 for configurations including the bare timber frame, timber frame with SIPs installed with screws at 24” (610 mm) o.c., timber frame with SIPs installed with screws at 12” (305 mm) o.c. and timber frame with SIPs installed with screws at 12” (305 mm) o.c. and edge boards installed. All tests performed with maximum actuator displacements of 0.1” (2.5 mm). Displacements shown were adjusted to coincide with wall mounted string potentiometer.
Figure 4.26. Strain energy as a function of displacement for Assembly 2 for configurations including the bare timber frame, timber frame with SIPs installed with screws at 24” (610 mm) o.c., timber frame with SIPs installed with screws at 12” (305 mm) o.c. and timber frame with SIPs installed with screws at 12” (305 mm) o.c. and edge boards installed. All tests performed with maximum actuator displacements of 0.1” (2.5 mm). Displacements shown were adjusted to coincide with wall mounted string potentiometer.
Figure 4.27. Hysteretic energy as a function of displacement for Assembly 2 for configurations including the bare timber frame, timber frame with SIPs installed with screws at 24” (610 mm) o.c., timber frame with SIPs installed with screws at 12” (305 mm) o.c. and timber frame with SIPs installed with screws at 12” (305 mm) o.c. and edge boards installed. All tests performed with maximum actuator displacements of 0.1” (2.5 mm). Displacements shown were adjusted to coincide with wall mounted string potentiometer.
Figure 4.28. Equivalent viscous damping as a function of displacement for Assembly 2 for configurations including the bare timber frame, timber frame with SIPs installed with screws at 24” (610 mm) o.c., timber frame with SIPs installed with screws at 12” (305 mm) o.c. and timber frame with SIPs installed with screws at 12” (305 mm) o.c. and edge boards installed. All tests performed with maximum actuator displacements of 0.1” (2.5 mm). Displacements shown were adjusted to coincide with wall mounted string potentiometer.
### Table 4.2. Cyclic test parameters for various testing configurations of Assembly 2 at maximum displacements.

<table>
<thead>
<tr>
<th>Test configuration</th>
<th>Maximum displacement</th>
<th>Cyclic stiffness</th>
<th>Strain energy</th>
<th>Hysteretic energy</th>
<th>Equivalent viscous damping</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bare timber frame</td>
<td>0.055 in. (1.4 mm)</td>
<td>2,430 lbs/in. (426 N/mm)</td>
<td>7.39 lbs-in. (835 N-mm)</td>
<td>11.5 lbs-in. (1.30 kN-mm)</td>
<td>0.25</td>
</tr>
<tr>
<td>SIPs installed; screws 24” o.c.</td>
<td>0.054 in. (1.4 mm)</td>
<td>26,600 lbs/in. (4.66 kN/mm)</td>
<td>73.1 lbs-in. (8.26 kN-mm)</td>
<td>33.5 lbs-in. (3.79 kN-mm)</td>
<td>0.073</td>
</tr>
<tr>
<td>SIPs installed; screws 12” o.c.</td>
<td>0.054 in. (1.4 mm)</td>
<td>29,400 lbs/in. (5.15 kN/mm)</td>
<td>82.2 lbs-in. (9.39 kN-mm)</td>
<td>29.4 lbs-in. (3.32 kN-mm)</td>
<td>0.057</td>
</tr>
<tr>
<td>SIPs installed; screws 12” o.c., edge boards installed</td>
<td>0.052 in. (1.3 mm)</td>
<td>32,700 lbs/in. (5.73 kN/mm)</td>
<td>85.6 lbs-in. (9.67 kN-mm)</td>
<td>29.4 lbs-in. (3.32 kN-mm)</td>
<td>0.055</td>
</tr>
<tr>
<td>SIPs installed; screws 12” o.c., edge boards installed</td>
<td>0.134 in. (3.4 mm)</td>
<td>32,500 lbs/in. (5.69 kN/mm)</td>
<td>567 lbs-in. (64.1 kN-mm)</td>
<td>258 lbs-in. (29.2 kN-mm)</td>
<td>0.072</td>
</tr>
</tbody>
</table>
Chapter 4: Test Results, Analysis and Interpretations

with respect to diaphragm length. Obtained allowable shear strength values of 290 lb/ft (4,240 N/m) and 376 lb/ft (5,480 N/m) for 8 ft (2.44 m) and 20 ft (7.32 m) long roof assemblies, respectively, indicated that for timber frame and SIP roof systems, shear strength per unit length was not constant with increasing diaphragm length. Similarly, obtained effective shear stiffness values of 43,200 lbs/in. (7,580 N/mm) for 8 ft (2.44 m) long roof assemblies and 120,000 lbs/in. (21,000 N/mm) for 20 ft (2.44 m) long roof assemblies, respectively, indicated that for timber frame and SIP roof systems, effective shear stiffness did not increase linearly with diaphragm length.

Non-linear increases in stiffness were likely due to beam behavior in conjunction with the adhesive properties of the polyurethane foam used to fill voids and cavities between adjacent SIPs. Placing adhesive foam between SIPs effectively created a deep beam or plate, rather than a series of smaller rectangular segments, the SIPs, that would be able to shift with respect to one another. The stiffness of the resulting deep beam would potentially increase with diaphragm length (b), as beam stiffness increases with depth based on moment of inertia.

Sound engineering practice dictates that design methods provide conservative estimates of forces within structural members for safe construction. Although allowable shear strength increased with panel length, and there was a slight possibility that allowable shear strength could decrease with assembly length, a reasonable design assumption for determining building diaphragm strength per foot (m) would be that allowable shear strength obtained from testing 8 ft (2.44 m) long roof diaphragm assemblies should be utilized for all roof diaphragms (slope lengths) greater than 8 ft (2.44 m). Therefore, design examples considered in Chapter 5 will utilize allowable shear strength of 290 lb/ft (4,240 N/m). It is recommended that for future assessment of timber frame and SIP roof systems significantly different from those tested, researchers should assume that allowable shear strength per unit length is constant for different slope lengths, and that an 8 ft (2.44 m) long test panel assembly will provide an adequate measure of allowable shear strength for roof diaphragms at least 8 ft (2.44 m) long.

As expected, effective shear stiffness increased with panel length. ASAE EP484.2 (ASAE, 1999) diaphragm design methodology utilized effective shear stiffness within the roof and endwalls to reduce member forces in building frames. Greater
effective shear stiffness in the roof provided lower member forces when designing for lateral load resistance. It would therefore be conservative to establish methods for extrapolating effective shear stiffness from 8 ft (2.44 m) long test diaphragms that would provide lower shear stiffness than that which was observed during testing of 20 ft (6.10 m) long test diaphragms. Figure 4.29 illustrates possible means of determining shear stiffness for diaphragms with slope lengths greater than 8 ft (2.44 m) and data points from tests of 8 ft (2.44 m) and 20 ft (6.10 m) long diaphragms. Since it is unlikely that effective shear stiffness would decrease with increases in panel length, a reasonable design assumption for determining building diaphragm effective shear stiffness would be that effective shear stiffness obtained from testing 8 ft (2.44 m) long roof diaphragm assemblies should be assumed to increase linearly for all roof diaphragms greater than 8 ft (2.44 m) in length. Based on effective shear stiffness of 43,200 lb/in. (7,580 N/mm) for 8 ft (2.44 m) long roof assemblies, predicted effective shear stiffness for 20 ft (6.10) long roof assemblies would be 107,000 lb/in (19,600 N/mm) rather than experimentally obtained 120,000 lb/in. (21,000 N/mm) shear stiffness value. This results in conservative frame member forces when used with ASAE EP484.2 (ASAE, 1999) diaphragm design procedures and is consistent with sound engineering practice. Design examples considered in Chapter 5 will utilize 43,200 lb/in. (7,580 N/mm) for effective shear stiffness, which increases linearly beyond 8 ft (2.44 m) long diaphragms. This results in conservative frame member forces when used with ASAE EP484.2 (ASAE, 1999) diaphragm design procedures and is consistent with sound engineering practice.

It is recommended that for future assessment of timber frame and SIP roof construction similar to those tested for this dissertation, designers assume that effective shear stiffness of building diaphragms do increase linearly with diaphragm length, with a zero intercept. In order to maintain similarity to tested 8 ft x 20 ft (2.44 m x 6.10 m) specimens, future 8 ft x 20 ft (2.44 m x 6.10 m) assemblies should include four 4 ft x 12 ft (1.22 m x 3.66 m) SIPs with 7/16 in. (11 mm) thick OSB skins attached to timbers with 0.190 in. (4.8 mm) diameter, hardened steel screws at 12 in. (305 mm) on-center that penetrate timbers by at least seven times the shank diameter [1.33 in. (34 mm)], as per NDS-97 (AF&PA, 1997) Section11.3.3 screw penetration requirements for laterally loaded wood screws. Additionally, SIPs should be connected to one another utilizing
Diaphragm length, L (ft)

Shear stiffness, c (1000 lbs/in.)

\[ c = (5,400)L \]
(Recommended linear function with 0 intercept.)

Predicted c20

Possible but unlikely stiffness-length function.

Figure 4.29. Plot of effective shear stiffness, c, as a function of diaphragm length, L. Actual shear stiffness at slope length, \( L = 20 \) ft (6.10 m) obtained from testing is greater than the predicted shear stiffness at \( L = 20 \) ft (6.10 m) utilizing the linear function based on shear stiffness of tested 8 ft x 24 ft (2.44 m x 7.32 m) diaphragms.
7/16 in. x 3 in. (11 mm x 76 mm) OSB splines using 8d annular threaded (ring shank) nails at 8 in. (203 mm) on center and a two-component slow-rise polyurethane insulating foam should be placed inside all SIP-to-SIP seams. Timber frame roof systems should be constructed in a similar manner to those described in Chapter 3, including the use of a splined joint where the center post stub connects to the two eave plates as illustrated in Figure 3.11, to ensure chord forces are resisted along the eave line. As previously discussed, the tested roof assemblies were fabricated to include all potential slip conditions in an effort to simulate a worst case scenario with regard to slip. If future 8 ft x 24 ft (2.44 m x 6.10 m) timber frame and SIP roof assembly tests are not designed to include all connection slip possibilities as outlined above it is not recommended that extrapolations to longer roof slope lengths utilize a linear increase in shear stiffness without testing larger test assemblies to verify this assumption.

An 8 ft (2.44 m) long test panel assembly fabricated similarly to those tested for this dissertation will provide an adequate measure of effective shear stiffness for roof diaphragms because plate behavior of roof assemblies longer than 8 ft (2.44 m) results in a diaphragm whose effective shear stiffness increases as slope length increases. By assuming a linear increase in shear stiffness based on an 8 ft (2.44 m) long test panel assembly utilizing a zero intercept, designers will calculate a lower shear stiffness for longer diaphragms, resulting in higher, or conservative, forces in frame members according to ASAE EP484.2 (ASAE, 1999) diaphragm design procedures.

### 4.4.2 Quasi-Static Cyclic Test Data Interpretations

Having calculated cyclic stiffness, strain energy, hysteretic energy and equivalent viscous damping for cyclically tested timber frame and SIP roof assemblies, there remains the issue of how to interpret these data in order for IBC 2000 (ICC, 2000) seismic design procedures to be utilized for design of timber frame and SIP buildings. Discussed below are trends observed in cyclic test data analysis results for five assemblies and, interpretations of these trends relating cyclic parameter values to structural behavior during seismic loading. Additionally, cyclic data is utilized to separate diaphragm shear deformation from diaphragm flexural deformation based on diagonal racking deformation utilizing equations developed by Fischer et al. (2001).
Analysis of cyclic test results provided quantitative data for different construction configurations, as discussed in Chapter 3, for all five timber frame and SIP roof assemblies. Construction configurations included the bare timber frame, the timber frame with SIPs attached using SIP screws at 24 in. (610 mm) on-center (o. c.), the timber frame with SIPs attached utilizing SIP screws at 12 in. (305 mm) o. c. and the timber frame with SIPs attached utilizing SIP screws at 12 in. (305 mm) o. c. with 2 x 6 (38 mm x 140 mm) No. 2 SPF edge boards installed. All configurations with SIPs included OSB splines and polyurethane foam, as described previously.

Trends in cyclic stiffness, strain energy, hysteretic energy and equivalent viscous damping for tested assemblies were observed to be consistent for tested assemblies as a function of varied construction configurations. The following discussion provides general trend information obtained from cyclic tests with maximum displacements of 0.1 in. (2.5 mm) for all assemblies. Assemblies 1, 2 and 3, which were 8 ft x 24 ft (2.44 m x 7.32 m) will be referred to as Assembly A specimens, while 20 ft x 24 ft (6.10 m x 7.32 m) Assemblies 4 and 5 will be referred to as Assembly B specimens.

Cyclic stiffness increased dramatically from the bare timber frame with the installation of SIPs using 24 in. (610 mm) o. c. screw spacing for all assemblies. Assembly A specimens averaged 11.5 times higher cyclic stiffness with installation of SIPs using 24 in. (610 mm) o. c. screw spacing, and cyclic stiffness for Assembly B specimens increased 18 times from the bare frame with the installation of SIPs using 24 in. (610 mm) o. c. screw spacing. Cyclic stiffness increased, on average, by 15% for Assembly A specimens and 12% for Assembly B specimens when SIP screw spacing was halved from 24 in. (610 mm) o. c. to 12 in. (3.5 mm) o. c. Edge board installation increased cyclic stiffness by an average 8% for Assembly A specimens and 5% for Assembly B specimens. Similar to monotonic stiffness previously discussed, cyclic stiffness of completed Assembly B specimens was approximately twice that of Assembly A. As peak cyclic displacement increased, cyclic stiffness also increased for assemblies with SIPs installed, indicating stiffening within the assemblies for increasing displacements. No degradation or decrease in stiffness was observed for completed assemblies subject to cyclic tests, even those with maximum displacements of 0.25 in. (6.4 mm).
Calculation of strain energy for varied construction configurations and assemblies provided similar trends to those observed for cyclic stiffness. Strain energy for assemblies with SIPs installed using screw spacing of 24 in. (610 mm) o. c. were 8 times greater and 16 times greater than bare timber frames for Assembly A and Assembly B specimens, respectively. Decreasing screw spacing from 24 in. (610 mm) o. c. to 12 in. (3.5 mm) o. c. increased strain energy values by 18% for Assembly A specimens and 14% for Assembly B specimens. Strain energy increased 8% and 3% for Assembly A and Assembly B specimens, respectively, as a result of installing edge boards. Completed Assembly B specimens dissipated average strain energy approximately 3 times that of completed Assembly A specimens at peak cyclic displacements of 0.1 in. (2.5 mm).

Similar to trends observed with cyclic stiffness and strain energy, hysteretic energy increased dramatically from the bare timber frame with installation of SIPs utilizing 24 in. (610 mm) o. c. screw spacing. Assembly A specimens dissipated 2.6 times as much hysteretic energy after initial SIP installation and Assembly B specimens dissipated 3.7 times as much hysteretic energy. Hysteretic energy calculations for increasing screw density and installing edge boards were not consistent enough to observe trends across configurations as some assemblies dissipated more energy and some less energy as screw density was increased and edge boards installed. Completed Assembly A specimens dissipated roughly half the hysteretic energy observed in completed Assembly B specimens at maximum cyclic displacements.

It was noted in Section 4.3.2 that equivalent viscous damping was a function of hysteretic energy and strain energy calculated utilizing an equation with strain energy in the denominator. Results from determining equivalent viscous damping for assembly configurations indicated that strain energy dominated the behavior of timber frame and SIP diaphragm assemblies as the trends for equivalent viscous damping were essentially the inverse of those for strain energy. Specifically, large decreases in equivalent viscous damping occurred after installation of SIPs using 24 in. (610 mm) o. c. screw spacing were observed for all assemblies, followed by smaller decreases in equivalent viscous damping with decreased screw spacing and edge board installation. Installation of SIPs using 24 in. (610 mm) o. c. screw spacing reduced equivalent viscous damping from that
of the bare timber frame by 72% for Assembly A specimens and 77% for Assembly B specimens. Increased screw spacing density to 12 in. (305 mm) o. c. reduced equivalent viscous damping 14% for Assembly A specimens and 18% for Assembly B specimens. Edge board installation reduced equivalent viscous damping by 5% for Assembly A specimens and 9% for Assembly B specimens. Equivalent viscous damping for completed Assembly B specimens was approximately 72% that of Assembly A specimens at maximum cyclic displacements.

Cyclic parameters described above for five timber frame and SIP roof assemblies follow trends that would be expected based on behavior of fasteners during tests to failure. Table 4.3 provides data regarding the condition of fasteners used to attach SIPs to the timber frames. As previously mentioned, the SIP screws dictated the behavior of the assemblies, which were comprised of a very stiff SIP plate fastened with SIP screws to a frame of rigid timbers, essentially pinned at the joints and free to rotate about those joints. Dramatic changes in all cyclic characteristics were expected upon initial SIP installation due to the flexible nature of the timber frame and rigidity of the SIPs.

Cyclic stiffness exhibited a large increase with installation of SIPs onto the timber frame and continued to increase slightly with increased screw density and installation of edge boards. Increased screw density distributed the load going from timbers into the SIPs to a larger number of screws, resulting in more load being required to move the assembly the same displacement, and consequently higher cyclic stiffness. Edge board installation stiffened the plate created by the SIPs slightly, which induced more load sharing into the screws and increased cyclic stiffness. Higher stiffness in Assembly B specimens over Assembly A specimens was attributed to the larger number of screws bent and broken, as seen in Table 4.3. Strain energy increased similarly with increases in cyclic stiffness for the same reasons discussed above. Because no damage was caused during cyclic testing and the assemblies remained within their elastic range, strain energy increased as additional load was required to attain the same displacements.

Large increases in hysteretic energy were observed with installation of SIPs utilizing 24 in. (610 mm) SIP screw spacing, but trends following decreased screw spacing and edge board installation were somewhat inconsistent. This would be expected in the elastic range, as all conducted cyclic tests were, because hysteresis loop formation
Table 4.3. Conditions of SIP screws following failure tests for all assemblies. Assembly 1, 2 and 3 are 8 ft x 24 ft (2.44 m x 7.32 m) and Assembly 4 and 5 are 20 ft x 24 ft (6.10 m x 7.32 m).

<table>
<thead>
<tr>
<th>Assembly</th>
<th>Total nails in assembly</th>
<th>Nails broken</th>
<th>Nails bent</th>
<th>Nails not damaged</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>120</td>
<td>43 (36%)</td>
<td>29 (24%)</td>
<td>48 (40%)</td>
</tr>
<tr>
<td>2</td>
<td>120</td>
<td>18 (15%)</td>
<td>11 (9%)</td>
<td>91 (76%)</td>
</tr>
<tr>
<td>3</td>
<td>120</td>
<td>23 (19%)</td>
<td>4 (3%)</td>
<td>93 (78%)</td>
</tr>
<tr>
<td>4</td>
<td>300</td>
<td>125 (42%)</td>
<td>31 (10%)</td>
<td>144 (48%)</td>
</tr>
<tr>
<td>5</td>
<td>300</td>
<td>193 (64%)</td>
<td>58 (19%)</td>
<td>49 (17%)</td>
</tr>
</tbody>
</table>
remains somewhat erratic until degradation of the system occurs and the loops become more distinct with increased displacements. Differences between hysteretic energy in Assembly A and Assembly B specimens were consistent with SIP screw data in that hysteretic energy dissipated by Assembly B specimens was approximately twice that of Assembly A specimens, and roughly twice the number of screws were damaged in Assembly B specimens as Assembly A specimens.

As previously discussed, equivalent viscous damping (Equation 4.6) was dominated by strain energy behavior, which resulted in equivalent viscous damping dropping significantly with installation of SIPS, and continuing to drop with decreased screw spacing and edge board installation, although at much smaller increments. Energy dissipation, or damping occurred as a result of friction among fasteners and components within the system as opposed to inducing damage of the materials comprising the assemblies. If cyclic displacements had been increased beyond the elastic range, damage would have occurred to the assemblies, reducing the cyclic stiffness and strain energy, and an increase in hysteretic energy would have resulted in increased values for equivalent viscous damping. Additionally, utilization of hardened steel screws to attach SIPS to timbers had the effect of increasing the cyclic stiffness and strain energy, but did not increase hysteretic energy because they did not have the ability to yield effectively. Use of fasteners with more ductility would have increased hysteretic energy and resulted in a more ductile assembly. Fasteners more ductile than the SIP screws utilized would likely have changed the effective dominance of the strain energy and resulted in higher equivalent viscous damping ratios.

Current seismic design procedures in the IBC 2000 (ICC, 2000) do not contain provisions for the use of heavy timber frame construction. Because system response factors for timber frame buildings are not provided, design of heavy timber frame construction would need to be performed utilizing “Alternative Means and Methods”, which require building designers to assume a value for Response Modification Coefficient, R. Designers must also justify use of the assumed R-value and obtain approval from governing building officials prior to building approval. Justification of an R-value is based on several embedded assumptions, primarily for vertical lateral load resisting systems within a structure, including displacement capacity, ductility, stiffness
and strength degradation, and hysteretic energy dissipation. While ample data is 
available for most seismic force resisting systems, there remains a lack of information 
regarding heavy timber frame and SIP construction vertical lateral load resisting 
elements. Testing conducted on timber frame and SIP roof systems, as discussed above, 
has provided some insight as to the behavior of these building types subject to seismic 
loads. Research conducted by Jamison (1997) on the cyclic and monotonic behavior of 
SIP shear walls indicated that SIP shear walls were extremely brittle when subjected to 
lateral loads, and were stiffer, less ductile and dissipated less energy than typical nailed 
wooden shear walls. Timber frame and SIP shear walls are currently being investigated 
at the University of Wyoming and available data indicated similar responses to lateral 
loads as roof assemblies tested for this dissertation. National Earthquake Hazards 
Reduction Program (NEHRP) provisions for designing shear walls utilizing adhesive 
attachment of wall sheathing require that an R-value of 1.5 be used for calculating 
seismic forces as opposed to an R-value of 7 that is typically used for nailed shear walls 
(Building Seismic Safety Council, 2001). Adhesive attachment of sheathing to shear 
walls is only permitted in low seismic regions (i.e., Seismic Design Categories A and B). 
Based on stiffness and lack of ductility inherent within SIP shear wall elements, and the 
assumption that they behave more like shear walls utilizing adhesives for sheathing 
attachment than nailed shear walls, it is recommended that designers utilize an R-value of 
1.5, the same as that for plain masonry shear walls and glued shear walls, which exhibit 
similar behavior when subject to lateral loads as SIP shear walls. Recommendations 
made with respect to IBC 2000 (ICC, 2000) seismic design procedures utilizing an R-
value of 1.5 provide higher seismic forces applied to the building, resulting in 
conservative estimates of forces acting on timber frame members and are consistent with 
sound engineering practice.

Fischer et al. (2001) provided a series of equations for separation of shear and 
flexural stiffness utilizing data from cyclic testing. String potentiometers on channels 8, 
10, 14 and 15 in Figure 3.14 provided diagonal deformation data which, in conjunction 
with global displacement of the center rafter and load data were utilized to calculate shear 
stiffness (GAs) and flexural stiffness (EI) for Assembly 2, 3, 4 and 5. Data was obtained 
from cyclic tests with maximum displacements of 0.25 in. (6.4 mm), which was
considered preferable for analysis purposes than maximum displacements of 0.1 in. (2.5 mm) utilized for previous analyses. Assembly 1 was omitted because no cyclic tests were conducted on that assembly using maximum displacements of 0.25 in. (6.4 mm). Calculations were performed utilizing data from bare timber frame tests from Assembly 2, 3, 4 and 5 and completed roof assemblies with 12 in. (305 mm) SIP screw spacing and edge boards installed for Assembly 3, 4 and 5. Table 4.4 provides results from calculations performed utilizing Fischer et al. (2001) equations for shear stiffness and flexural stiffness. Shear and flexural stiffness for the bare timber frames were drastically less than those of the sheathed assemblies. Shear stiffness of the bare frames was approximately 1% that of the completed assemblies. Flexural stiffness for the bare timber frame of Assembly 3 (8 ft x 24 ft (2.4 m x 7.32 m)) was around 4% that of the completed Assembly 3, and flexural stiffness for the bare frames of Assemblies 4 and 5 (20 ft x 24 ft (6.10 m x 7.32 m) were approximately 1.5% that of the fully sheathed assemblies. Shear and flexural stiffness comparisons utilizing Fischer et al. (2001) equations further reinforced the concept that for timber frame and SIP roof assemblies, the SIPs are resisting nearly all of the lateral loads, resulting in much lower forces in frame members. Lower frame member forces result in potentially smaller frame members and more economical and efficient designs that take advantage of the significant stiffness of SIPs for resisting lateral loads.

Numerical and graphical results for cyclic testing for all assemblies and configurations are provided in Appendix C. In general, timber frame and SIP roof assemblies proved to be extremely stiff structural systems that maintained their structural integrity up to cyclic displacements of 0.25 in. (6.4 mm), which coincided approximately with design loads, 0.4(P\text{ult}). Cyclic characteristics of SIP and timber frame roof systems supported the assumption that SIPs create a very stiff plate that is connected to the flexible timber frame utilizing only the SIP screws. It would certainly be possible to create a finite element model with a rigid plate and flexible frame, where the connections between them were modeled as non-linear springs with failure criteria based on SIP screw characteristics. Results from this dissertation could be utilized to validate and refine parameters of the finite element model. An example seismic design for a typical timber frame and SIP building incorporating cyclic behavior characteristics from testing
Table 4.4. Shear stiffness, $GA_s$, and flexural stiffness, $EI$, calculated utilizing Fischer et al. (2001) equations at 0.25 in. (6.4 mm) maximum displacements. Assembly 2 and 3 are 8 ft x 24 ft (2.44 m x 7.32 m) and Assembly 4 and 5 are 20 ft x 24 ft (6.10 m x 7.32 m).

<table>
<thead>
<tr>
<th>Test configuration</th>
<th>Assembly</th>
<th>Shear stiffness, $GA_s$ (1000 lbs)</th>
<th>Shear stiffness, $GA_s$ (kN)</th>
<th>Flexural stiffness, $EI$ (1000 lbs-in$^2$)</th>
<th>Flexural stiffness, $EI$ (kN-mm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SIPs installed; screws 12” o.c., edge boards installed</td>
<td>2</td>
<td>3,600</td>
<td>16,000</td>
<td>$3.5 \times 10^6$</td>
<td>240,000</td>
</tr>
<tr>
<td>Bare timber frame</td>
<td>3</td>
<td>46</td>
<td>200</td>
<td>$1.0 \times 10^6$</td>
<td>7,000</td>
</tr>
<tr>
<td>SIPs installed; screws 12” o.c., edge boards installed</td>
<td>3</td>
<td>4,300</td>
<td>19,000</td>
<td>$2.7 \times 10^6$</td>
<td>190,000</td>
</tr>
<tr>
<td>Bare timber frame</td>
<td>4</td>
<td>53</td>
<td>200</td>
<td>$1.0 \times 10^6$</td>
<td>7000</td>
</tr>
<tr>
<td>SIPs installed; screws 12” o.c., edge boards installed</td>
<td>4</td>
<td>7,700</td>
<td>34,000</td>
<td>$9.0 \times 10^6$</td>
<td>620,000</td>
</tr>
<tr>
<td>Bare timber frame</td>
<td>5</td>
<td>91</td>
<td>400</td>
<td>$1.7 \times 10^6$</td>
<td>12,000</td>
</tr>
<tr>
<td>SIPs installed; screws 12” o.c., edge boards installed</td>
<td>5</td>
<td>9,300</td>
<td>41,000</td>
<td>$11.2 \times 10^6$</td>
<td>780,000</td>
</tr>
</tbody>
</table>
is performed in Chapter 5, based on IBC 2000 (ICC, 2000) design calculations.

4.5 Conclusions

Monotonic and cyclic tests conducted on five timber frame and SIP roof assemblies were performed in order for timber frame building designers to make use of the in-plane strength and stiffness of SIPs to resist lateral loads utilizing diaphragm action. It was therefore necessary to analyze data obtained from previously discussed tests and extract that which was required to make use of ASAE 484.2 (ANSI/ASAE, 1999) diaphragm design procedures and IBC 2000 (ICC, 2000) seismic design procedures. Analyses discussed in this chapter were adequate for determining necessary parameters for diaphragm design of timber frame and SIP structures subject to either wind or seismic induced lateral loads, as discussed in the following chapter.