# Chapter 3: Experimental Methods

## 3.1 Research Objectives and Overview

The purpose of this research was to derive a methodology for designing lateral load resisting components within a timber frame building that included the contribution of diaphragm action of the SIPs used to enclose and insulate the building. It should be noted that the scope of this research was limited to only roof assemblies.

Based on recommendations made by Carradine et al. (2000), one of the most important elements currently lacking in order to apply ASAE EP484.2 procedures for diaphragm design was test data on the strength and stiffness of SIP roof assemblies used in conjunction with timber frames. Tests were conducted on three test panel assemblies and two full-scale sized roof panel assemblies in order to determine stiffness and strength values as well as how predictions should be made from test panel assemblies to full roofs. The construction of all tested assemblies was based on the most typically utilized timber frame joinery and SIP installation methods. Quasi-static cyclic tests were also conducted on the roof assemblies to assess their behavior under seismic and high wind loading.

Once all of the tests were conducted and the data collected, comparisons were made between post-frame test results and the timber frame test data. Timber frame and SIP assemblies behaved very effectively as diaphragms and it is discussed in later chapters what modifications are needed to ASAE EP484.2 to create procedures for design of timber frame structures utilizing diaphragm action.

## 3.2 Roof Panel Test Assembly Fabrication

### *3.2.1 Timber Frame Fabrication*

Fabrication of the timber frame portions for the roof assemblies consisted of labeling, laying out, rough cutting, finish cutting, strap installation and pre-assembly for each of the five frames. The timbers used to fabricate the timber frame portions were No. 2 and Better Southern Pine, a commonly utilized species for timber frame construction. The splines and pegs were made from mixed oak. Upon arrival at the Brooks Forest Products Center, timbers were labeled according to which member they were within the frame and how they would be oriented if they were used in an actual building. Labels used for Assembly One are shown in Figure 3.1.

Any crown in timbers was oriented towards the outside of the building. Timbers were laid out for cutting utilizing lines drawn and indications on how deep saw cuts should be made. Several joint configurations required the size of members to be mapped to ensure tight fitting joints. Rough cutting was performed with power saws and drills and involved making all of the end cuts, making shoulder and cheek cuts for the tenons and hogging out wood for the mortises with a drill. Chisels, hand planes and some power tools were utilized to do finish cutting, where imperfections left from rough cutting were “cleaned up” and pockets for the purlins and ridge purlins were cut. The final steps of finish cutting were to label each timber, indicate which timbers would be connected to each timber and then apply a coat of end sealing wax to the cut joints.

A router was used to create shallow troughs for each of the metal straps so that the SIPs would fit snugly against the timbers or the oriented strand board (OSB). The metal straps were installed to hold the purlins and ridge purlins in place. Before attaching the SIPs, each timber frame was pre-assembled on the test bed and squared, then peg holes were drilled and straps, pegs and splines installed. These steps concluded the timber frame fabrication.

### *3.2.2 SIP Installation*

Installation of the SIPs included attachment of OSB onto rafters and purlins, attachment of SIPs to timbers, inserting splines in SIPs, installing perimeter edge boards and foaming the splined joints between SIPs. A strip of 19/32 in. (15 mm) by 5 in. (127 mm) OSB was centered along the top of each post stub, rafter, and it was attached with 1-1/2 in. (38 mm) long deck screws [shank diameter equal 0.138 in. (3.5 mm)] at 12 in. (31 mm) on- center (o. c.) down the middle of the strip. On purlins this strip was 3-1/2 in. (89 mm) wide and centered. On plates the strips were 2 in. (51 mm) wide and held flush to the eave line for Assembly 1, but were widened to 2-1/2 in. (64 mm) for the remaining assemblies to ensure that the SIP screws would remain inside the OSB strip. In practice, this OSB creates a ledge where drywall can be fitted without having the drywall placed between the timber and the SIPs, which would result in a soft layer, compromising the stiffness and strength of the roof diaphragm. Figure 3.2

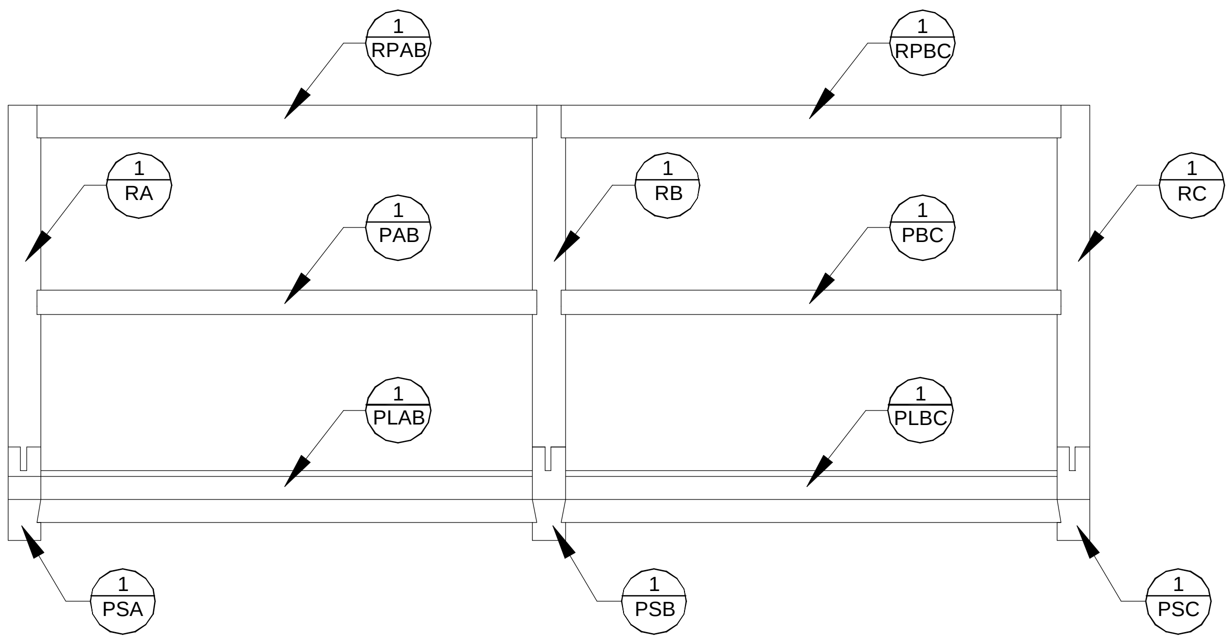


**Figure 3.1. Example of basic test panel (8 ft x 24 ft (2.4 m x7.3 m)) assembly including member labels. Members shown include rafters (R), ridge purlins (RP), purlins (P), plates (PL) and post stubs (PS). Numbers indicate assembly and letters following member designations (i.e. RAB) indicate between which bents (A, B and C) members are located.**

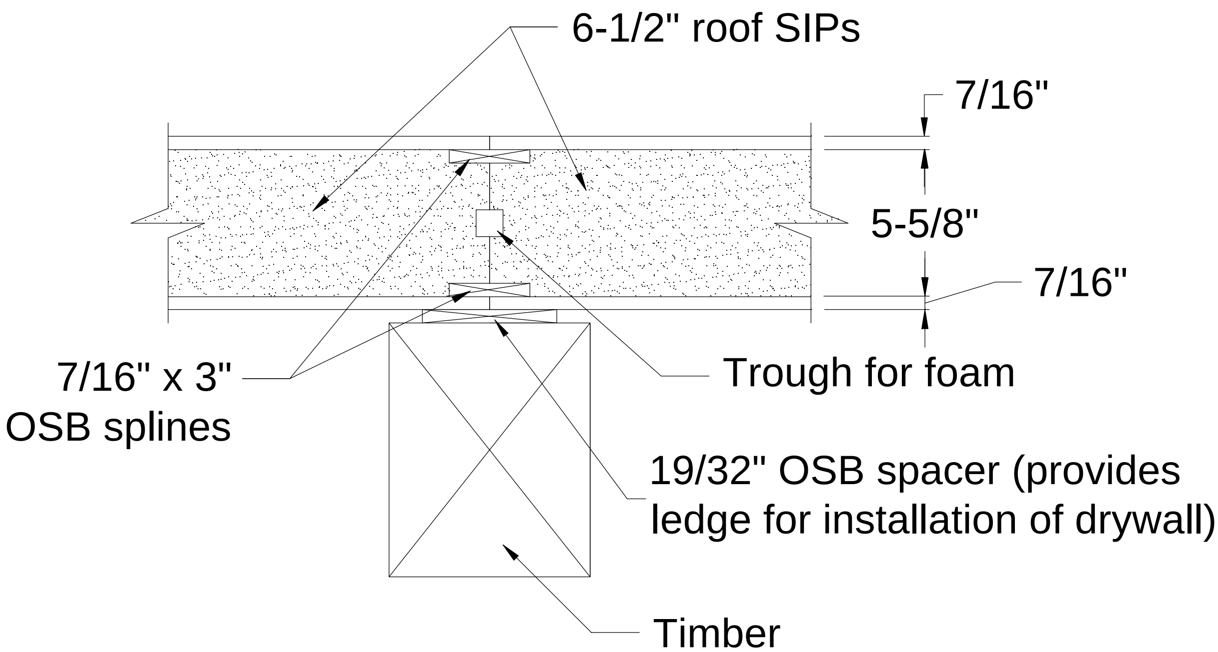


**Figure 3.2. Example of interface between timber, OSB spacer strips and SIPs. (Screws that attach SIPs to timbers omitted for clarity.)**

illustrates this interface between the timbers, OSB strips and SIPs. Directly on top of the OSB strips were placed the 6-1/2 in. (17 mm) thick SIPs, comprised of a 5-5/8 in. (143 mm) thick expanded polystyrene foam core layer with 7/16 in. (11 mm) OSB adhered to both sides. The SIPs were fixed to the timbers utilizing 9 in. (229 mm) long screws at 12 in. (31 mm) o. c., with a shank diameter of 0.190 in. (4.8 mm). The thread length was 3 in. (76 mm), the root diameter was 0.17 in. (4.3 mm) and the screws were manufactured by Olympic Manufacturing Group, Agawam, MA. Presently, nails ranging from 8 to 11 in. (203 to 279 mm) are also used to attach SIPs to timber frames, but screws were chosen for this study due to ease of installation, especially on roofs, their ability to be withdrawn when needed, and a general consensus among professionals that were contacted who recommended their use. As the SIPs were brought together for attachment, 7/16 in. (11 mm) by 3 in. (76 mm) wide OSB splines were placed into grooves routed just beneath the outer skin of the SIPs. These splines were attached to the SIP skin using 8d annular ring shank nails (2.5 in. (64 mm) x 0.113 in. (2.9 mm)) at 8 in. (203 mm) o. c. on both sides of the joint with a pneumatic nail gun. Edge boards made from 2x6 (38 mm x 140 mm) No. 2 Spruce-Pine-Fir (SPF) were let into a groove routed into the SIPs around the perimeter of each assembly as shown in Figure 3.3. The grooves for splines and edge boards were routed during the manufacture of the SIPs. No edge boards were installed along the edge of the assemblies simulating the ridge, as per current construction practices. Edge boards were fixed with 8d annular ring shank nails [2.5 in. (64 mm) x 0.113 in. (2.9 mm)] on 8 in. (203 mm) centers using a pneumatic nail gun. In practice, the edge boards create a nailing surface for exterior trim and assist in keeping SIPs lined up with one another. Foam application consisted of drilling 1/4 in. (6.4 mm) holes through the outer skin of the SIPs and the OSB splines at 12 in. (305 mm) o. c. along all of the splined panel seams and then spraying insulating foam though the holes to fill the spaces between the SIPs and the splines. In actual construction, this foam, a two component slow-rise polyurethane (Handi-Foam SR), manufactured by Fomo Products, Inc., is used to ensure a tightly sealed exterior envelope and avoid a break in insulation at the SIP interfaces. Some stiffness testing was performed before the edge boards were installed and prior to the complete attachment of the SIPs to the timbers, as presented in Section 3.4.

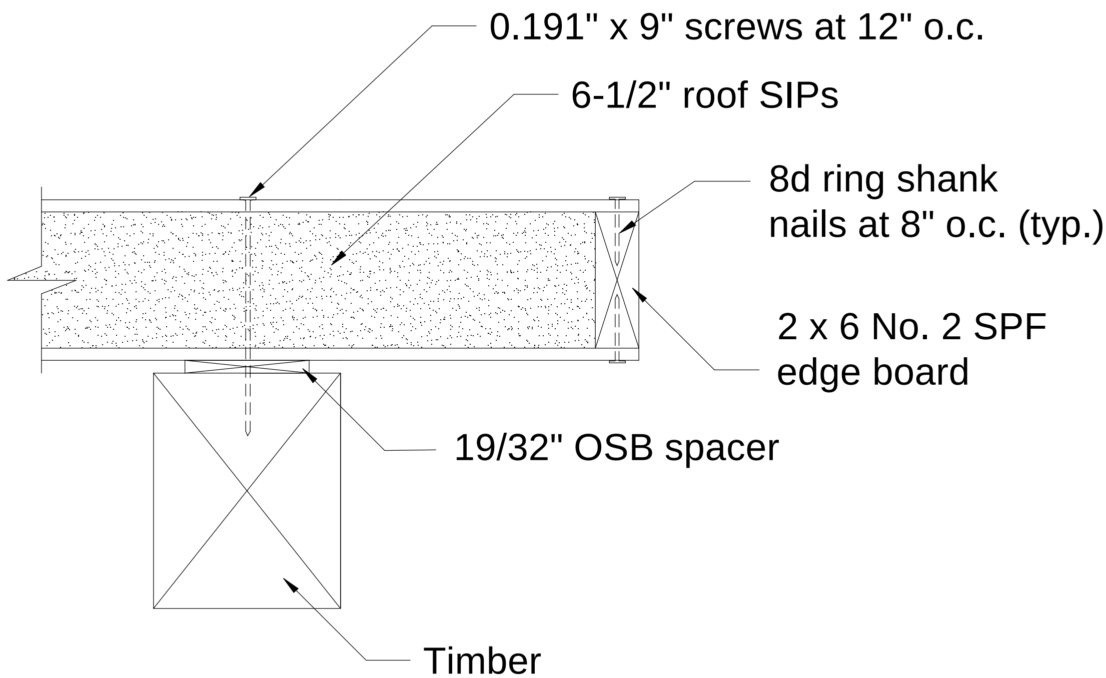


**Figure 3.3. Example of edge board installation around perimeter of SIPs. (Nails were placed only through the top of SIPs at locations where edge boards were on top of timbers.)**

### *3.2.3 List of Materials*

Presented below in Table 3.1 are the materials used for fabrication of the timber frame and SIP test specimens. The SIP screws were donated by the manufacturer, the Timber Frame Business Council provided reimbursement for the timbers, Blue Ridge Timberwrights donated the pegs, Dreaming Creek Timber Frames donated the oak splines and Great Lakes Insulspan donated a portion of the SIPs.

## 3.3 Roof Panel Assembly Testing

### *3.3.1 Strength and Stiffness Test Protocols*

Use of ASAE 484.2 (ASAE, 1999) to design structures utilizing diaphragm action was limited due to its dependency upon specific test values. The scope was limited to single-story, metal-clad wood-frame buildings of rectangular shape. The single-story limit for post-frame was introduced because two-story buildings of this type are uncommon and authors did not want to address the more complicated case of a building with roof and floor diaphragms. While most timber frame buildings are rectangular in plan, they often contain intermediate floor diaphragms. As a conservative approach, it was assumed, as in the case with post-frames, that the roof system was the only horizontal element in the building that transferred the lateral loads to the end walls.

Methods prescribed by ASAE EP484.2 (ASAE, 1999) required the use of either a cantilever test or a simple beam test in order to obtain diaphragm strength and stiffness. Both of these tests provided diaphragm shear stiffness, c, which was used to determine specific design parameters used with ASAE diaphragm design procedures, as discussed in following chapters. Typical timber frame construction lends itself most easily to the use of a version of the simple beam test utilizing a single load applied to the center rafter of an assembly having three rather than four rafters. Woeste and Townsend (1991) identified rafter buckling, out-of-plane panel movement and load transfer into the panel as concerns with this methodology. Due to large wooden member sizes and fastening techniques used with timber frame construction, rafter buckling and transfer of load into the panels were not an issue. Out-of-plane movement was addressed in more recent ASAE guidelines and was taken into consideration during the testing as addressed in Section 3.3.3. Anderson (1990) compared EP484 (ASAE, 1990) with a similar testing

**Table 3.1. List of Materials Utilized for the Fabrication of the Roof Diaphragm Assemblies**

| Building Component | Base Material | Dimensions | Manufacturer/Supplier |
|---|---|---|---|
| Timbers | Southern Pine No. 2 or Better | Varied | Dreaming Creek Timber Frames |
| SIPs | OSB and Expanded Polystyrene | 4 ft x 12 ft x 6-1/2 in. | Great Lakes Insulspan[1] |
| OSB Splines | OSB | 7/16 in. x 3 in. x various lengths | Supplied by Great Lakes Insulspan |
| Edge Boards | 2x6 No. 2 Spruce-Pine-Fir | 1-1/2 in. x 5-1/2 in. x various lengths | Various Suppliers and Manufacturers |
| Foam | Two-Component Slow-Rise Polyurethane Foam (Handi FOAM) | N/A | Fomo Products, Inc.[2] |
| SIP Screws | Hardened Steel (Grade 1022) | 0.190 in. dia. shank x 9 in. | Olympic Manufacturing Group |
| 8d Threaded (Annular) Nails | Hardened Steel | 0.113 in. dia. shank x 2-1/2 in. | Spotnails |
| OSB Strips | OSB | 19/32 in. x various widths and lengths | Georgia Pacific |
| Timber Frame Splines | Red Oak | 6 in. x 23-1/2 in. x 1-1/2 in. | Dreaming Creek Timber Frames |
| Pegs | Red Oak | 1 in. diameter x 12 in. | Supplied by Blue Ridge Timberwrights |
| Straps | Steel (20 ga) | 1-1/4 in. x 12 and 21 in. | United Steel Products Company (USP) |
| 16d Sinker Nails | Steel | 0.148 in. dia. shank x 3-1/4 in. | Grip Rite |

1. P. O. Box 38, 9012 E. US 223, Blissfield, MI 49228
2. P. O. Box 1078, 2775 Barber Road, Norton OH 44203

regime for roof assemblies, ASTM E455 Static Load Testing of Framed Floor or Roof Diaphragm Constructions for Buildings (ASTM, 2000) and pointed out that ASAE EP484 provided test results that were more easily utilized for the diaphragm analysis using ASAE techniques. Unfortunately, assemblies for timber frame and SIP construction were significantly different from post-frame roof assemblies to require a combination of testing procedures from ASAE EP484 (ASAE, 1991), ASAE EP558 (ASAE, 1999) and ASTM E 455 (ASTM, 2000). Therefore, stiffness and strength values for assemblies of timber framing with SIP panels as cladding were determined utilizing a hybridization of methods prescribed in both codes and are described in detail below. Quasi-static cyclic testing is discussed in later sections.

Testing performed has been specified according to test apparatus, testing procedures, the test report, and results. Beyond these criteria, ASAE EP484 (ASAE, 1990) stated that the test assembly was assumed to be "functionally equivalent to that used in the building being designed." Fabrication methods used were based on examples of timber frame structures and queries to members of the Timber Framers Guild, the Timber Frame Business Council and several timber frame and SIP manufacturing and installation companies, with the intention of being as similar to typical roof sections as possible.

The basic roof panel test assembly is illustrated in Figure 3.4 and the full-scale roof test assembly is shown in Figure 3.5. Timber frame joint details for both assemblies are shown in Figures 3.6 – 3.11. Dimensions for test assemblies were partially based on 4 ft by 12 ft (1.22 m by 3.66 m) SIP sizes, a commonly utilized and readily available size for SIPs. More important in selecting the sizes for the test assemblies were considerations regarding connection slip and future testing of timber frame and SIP roof assemblies.

The objective of selecting overall dimensions of the basic test panel assembly was to propose a small enough specimen so that future researchers would only have to test the basic test panel assembly to determine roof diaphragm stiffness, while being large enough to include all the potential slip conditions. Because stiffness was directly affected by slip among the various components, it was critical to include all the connections that could experience slip during load testing. The 8 ft (2.44 m) by 24 ft (7.32 m) roof

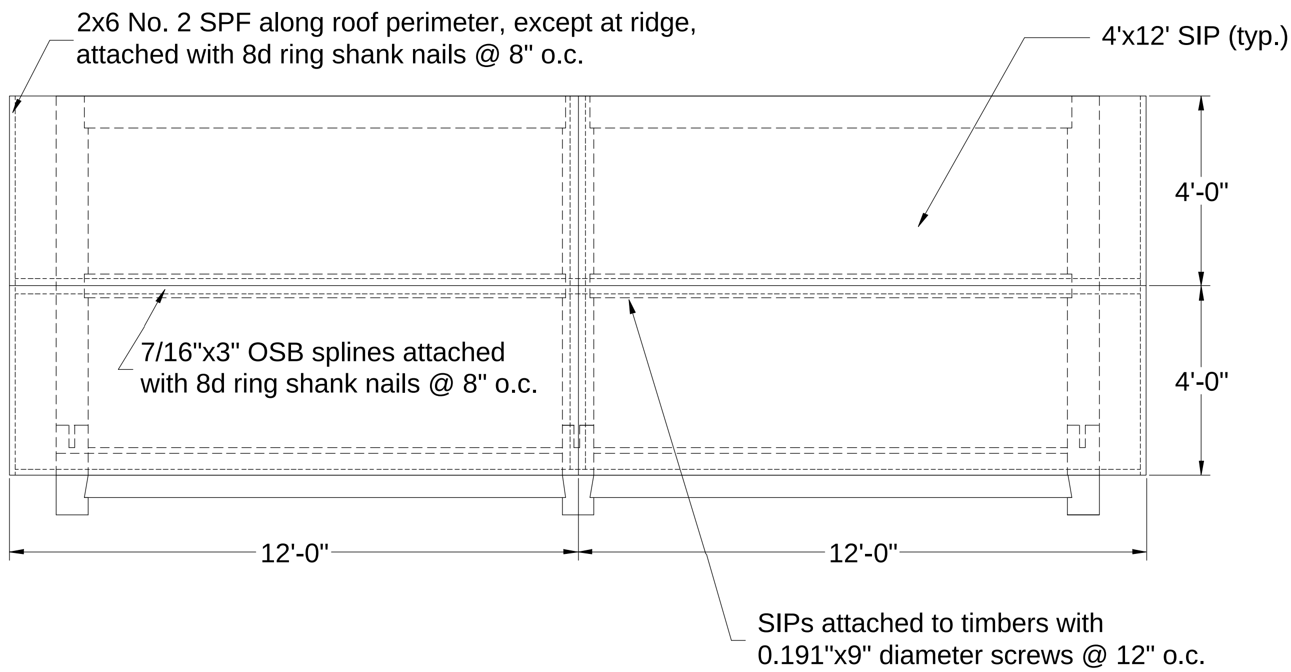


**Figure 3.4. Example of 8 ft x 24 ft (2.44 m x 7.32 m) basic test panel assembly. OSB spacer strips, nails and screws omitted for clarity.**

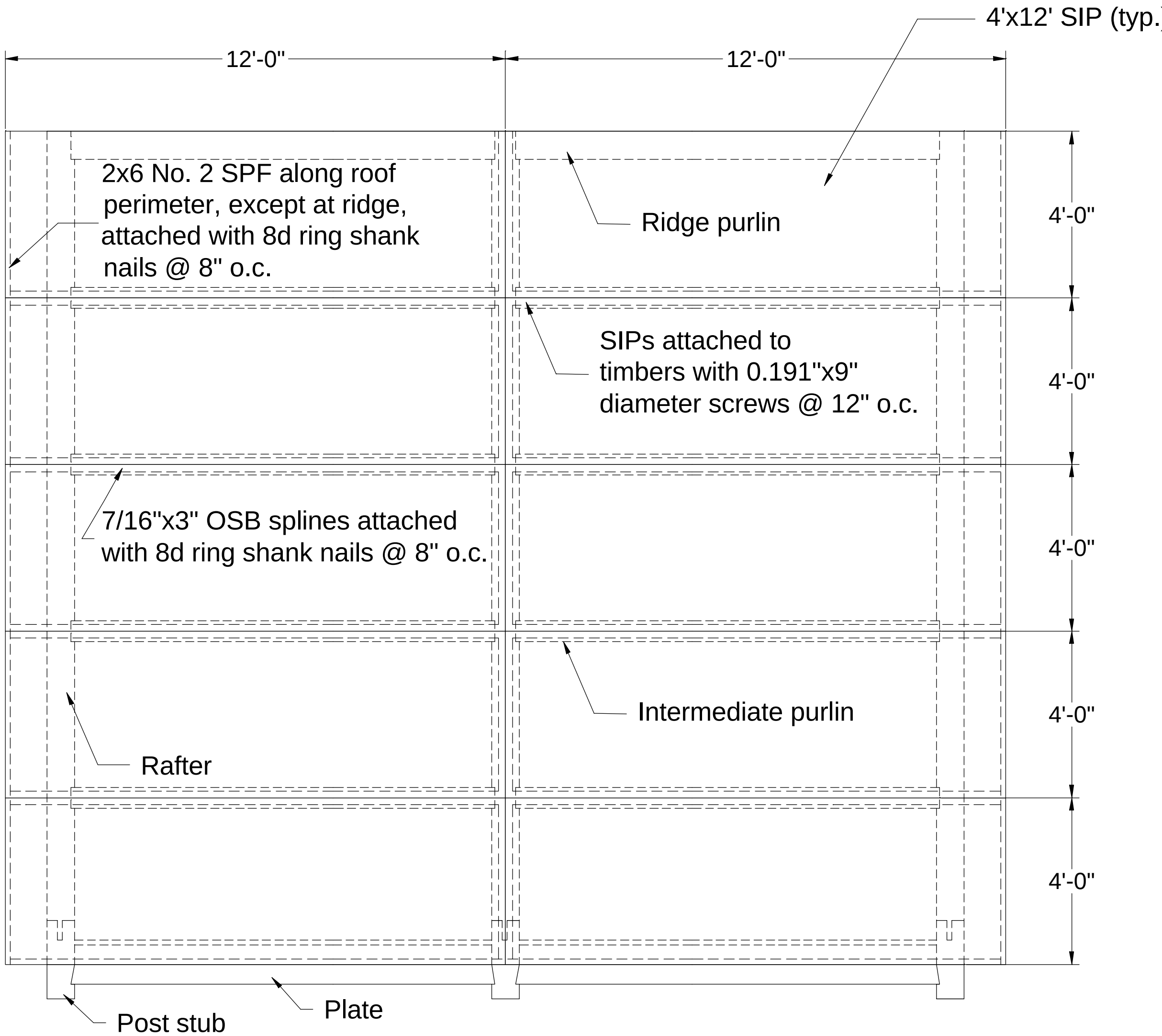


**Figure 3.5. Example of 20 ft x 24 ft (6.10 m x 7.32 m) full-scale test panel assembly. OSB spacer strips, nails and screws omitted for clarity**

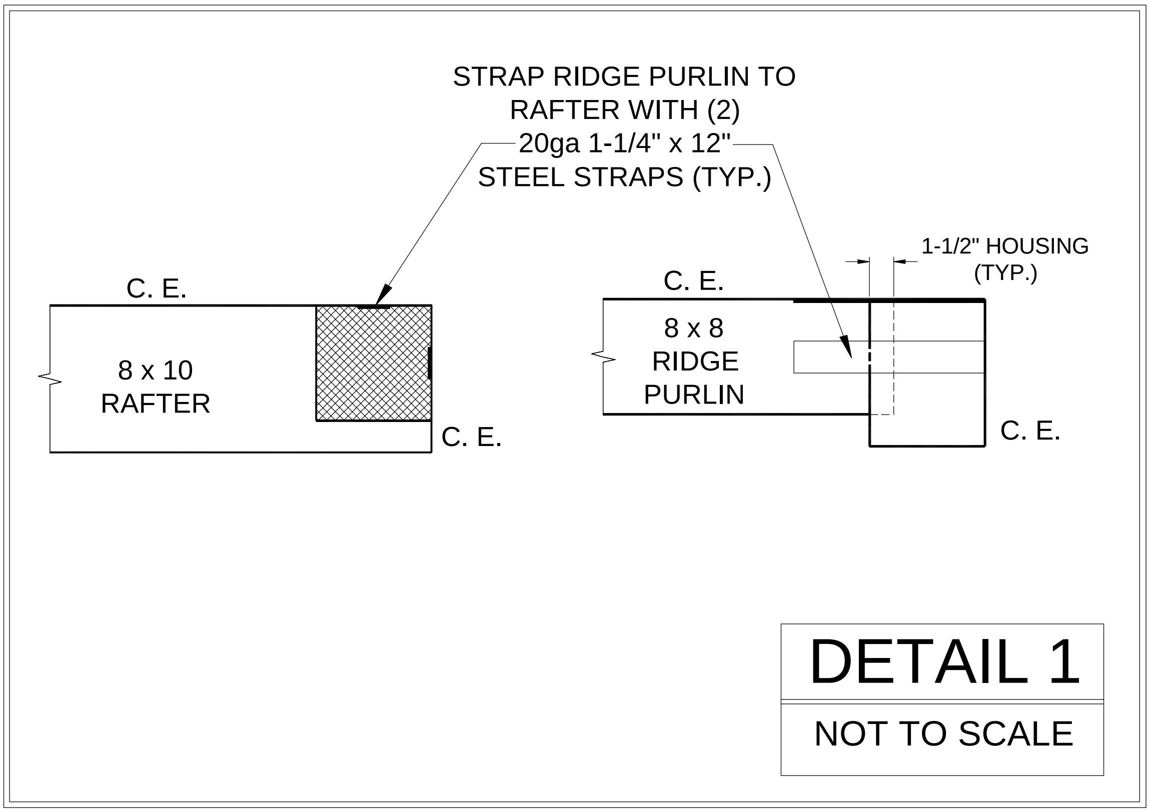


**Figure 3.6. Example of typical end rafter to ridge purlin joint consisting of a housing cut into the rafter into which the ridge purlin fits and is held in place with two steel straps. (C. E. indicates critical edge, a layout reference edge.)**

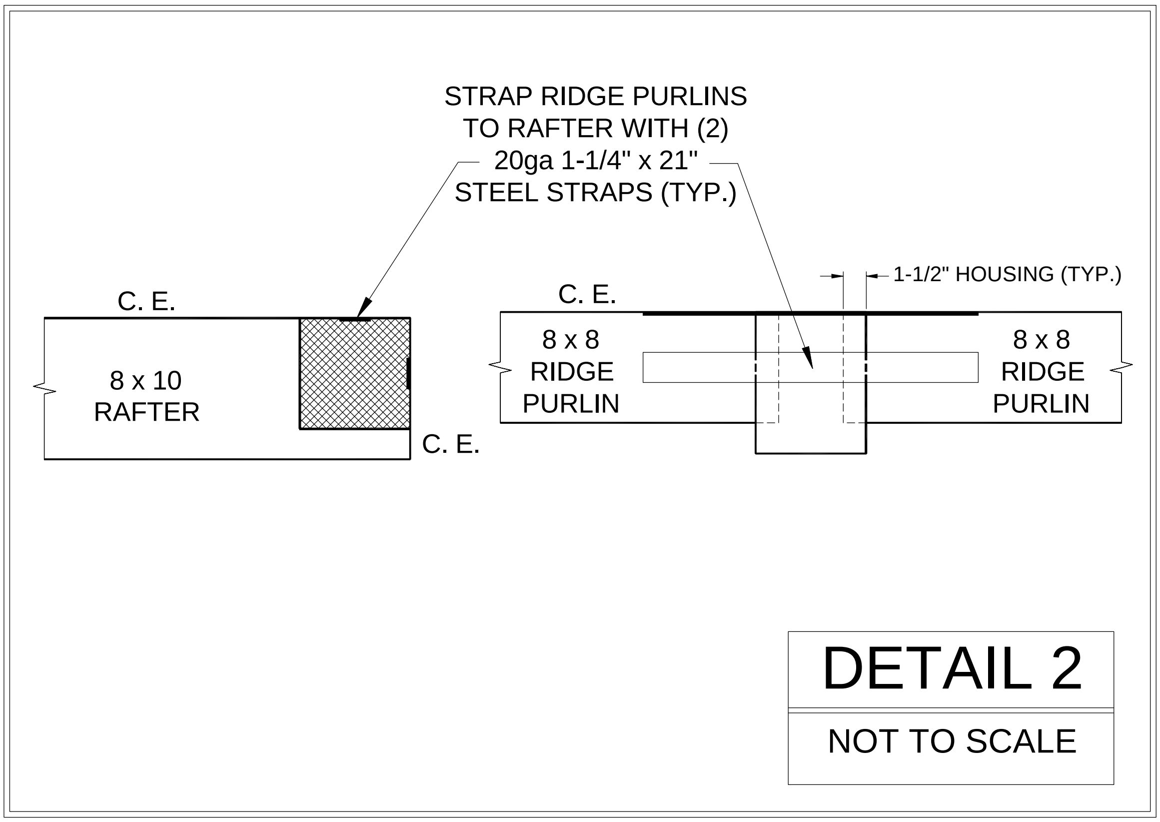


**Figure 3.7. Example of typical center rafter to ridge purlin joint consisting of two housings cut into the rafter into which the ridge purlins fit and are held in place with two steel straps.**

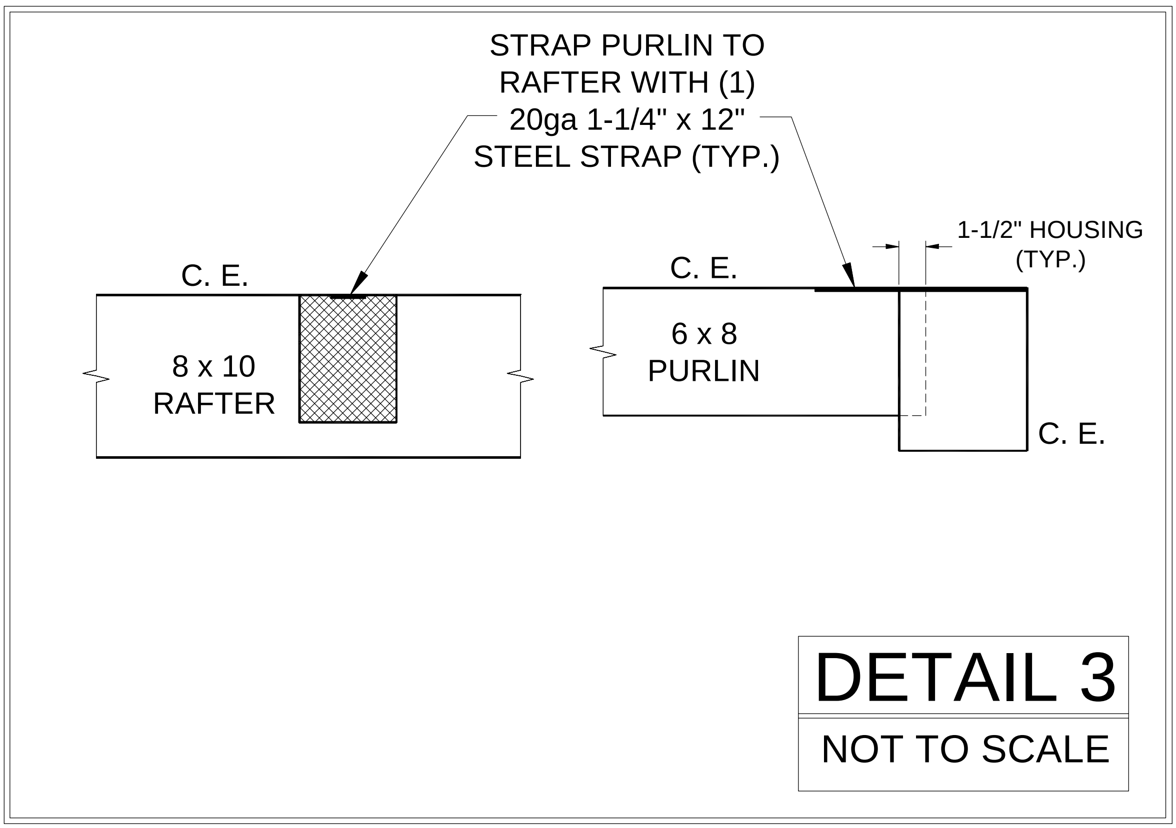


**Figure 3.8. Example of typical end rafter to purlin joint consisting of a housing cut into the rafter into which the purlin fits and is held in place with a steel strap.**

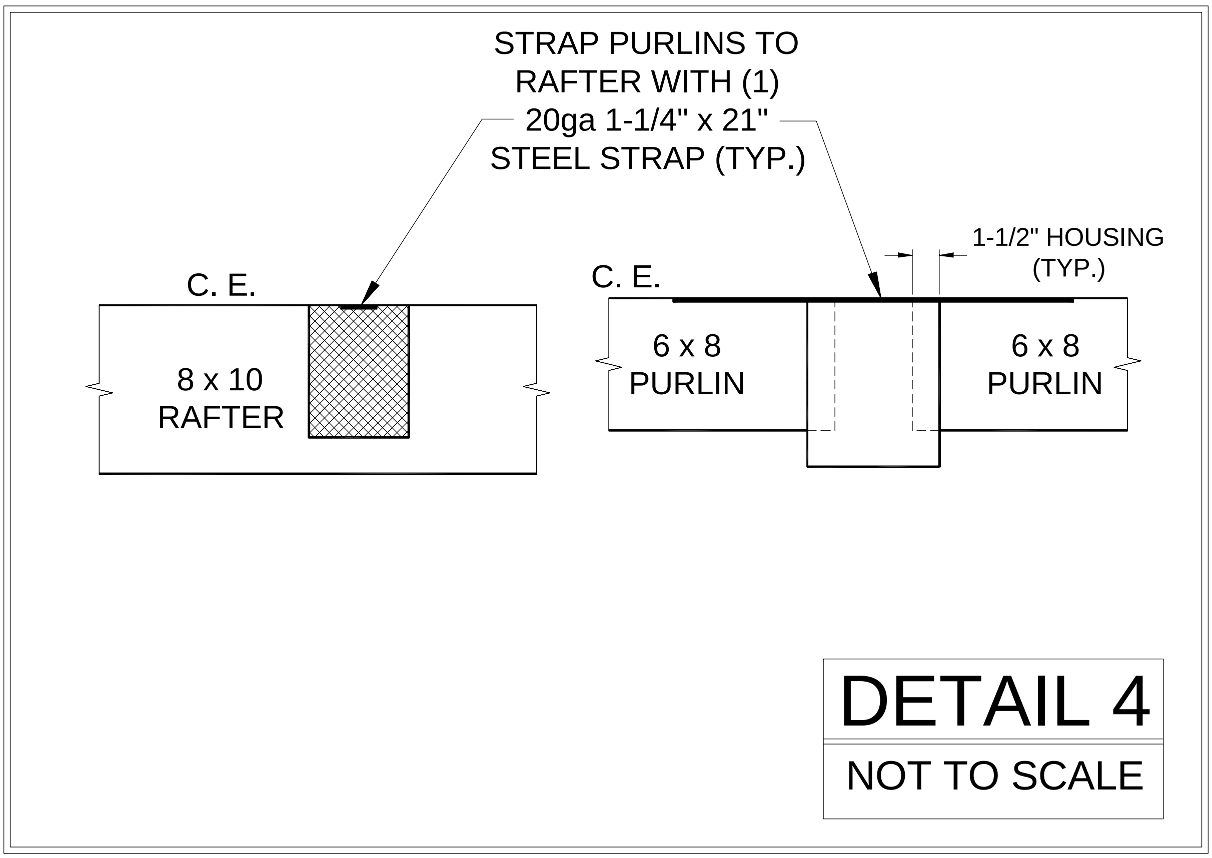


**Figure 3.9. Example of typical center rafter to purlin joint consisting of two housings cut into the rafter into which the purlins fit and are held in place with a steel strap.**

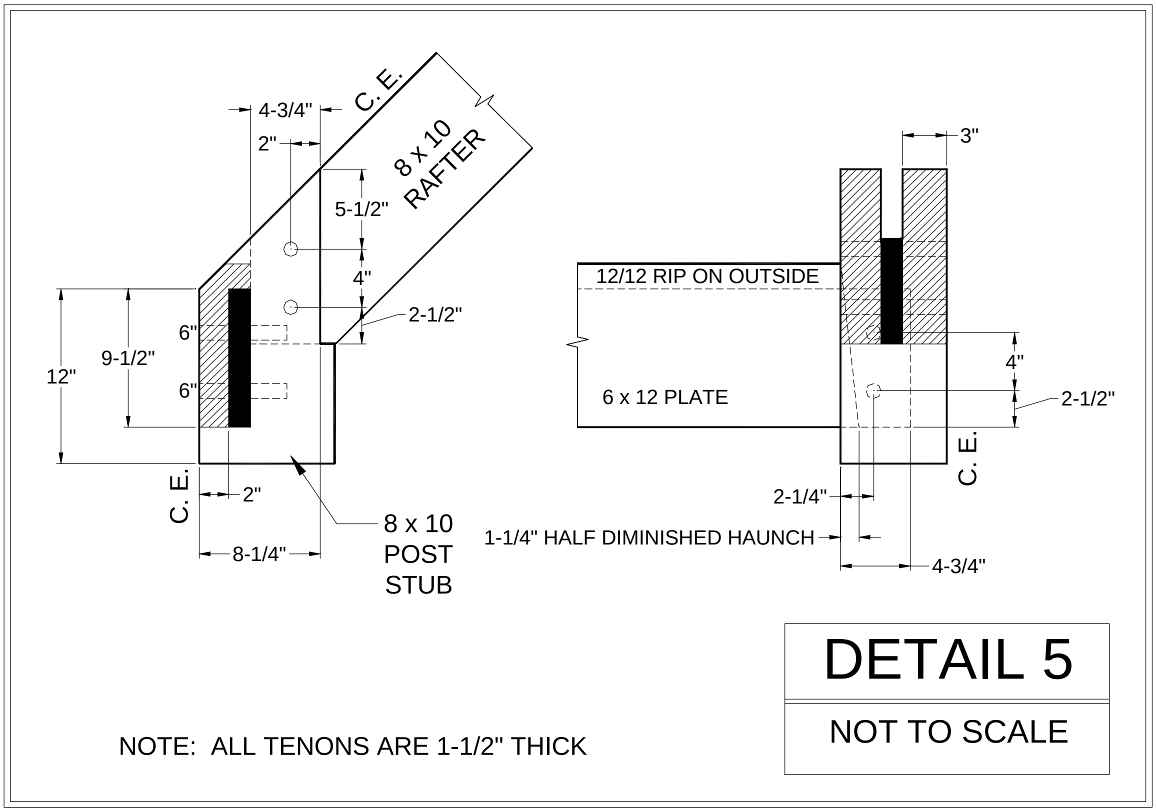


**Figure 3.10. Example of typical end rafter, post stub and plate joint consisting of a tenon cut into the rafter which fits into a mortise in the post stub and a half diminished haunch and tenon cut into the plate which fits into a mortise in the post stub. The joint is secured with 1 in. diameter red oak pegs. (The 6” demarcations refer to the length of the pegs.)**

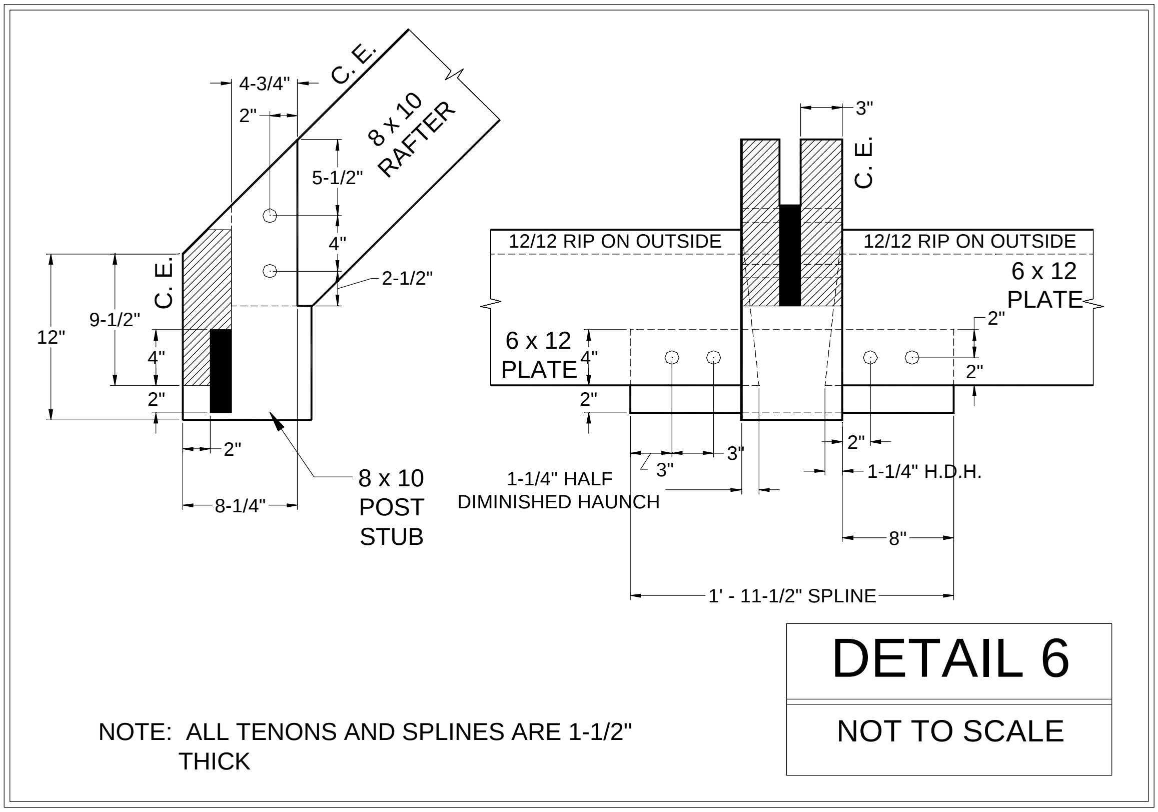


**Figure 3.11. Example of typical center rafter, post stub and plate joint consisting of a tenon cut into the rafter which fits into a mortise in the post stub and a half diminished haunch and slot cut into the plates and post stub into which fit an oak spline. The joint is secured with 1 in. diameter red oak pegs.**

assembly, as seen in Figure 3.1, had 2 bays, and each bay contained a ridge purlin, an intermediate purlin, a wall plate, 2 rafters and 2 post stubs. This assembly, when load tested permitted slip between ridge purlin and rafter, intermediate purlin and rafter, plate and post stub and post stub and rafter, within the timber framing. Four, 4 ft (1.22 m) by 12 ft (3.66m) SIPs were utilized for this assembly, which allowed slip between SIPs and the splines and foam holding the SIPs together along horizontal and vertical seams, as well as slip between SIPs and edge boards. Having a flexible timber frame connected to an extremely stiff SIP cladding system suggested that the critical design element would be the screws connecting the two systems, which also introduced a source of slip that would be permitted by the basic test panel assembly.

Future researchers also need to interpret data from basic test panel assemblies for use with larger roof diaphragms. Testing equipment at the Brooks Forest Products Center was limited by size of the test bed and the capacity of the hydraulic actuator, and the 20 ft (6.10 m) by 24 ft (7.32 m) roof assembly shown in Figure 3.5 was considered to be the largest full-scale diaphragm practical for the test apparatus. Testing of full-scale roof assemblies allowed for creation of a procedure, as discussed in Chapter 4, whereby test data obtained from basic panel assembly tests was utilized to estimate full-scale roof strength and stiffness values for design purposes.

Support and loading conditions for the basic test panel assembly are shown in Figure 3.12. Testing was performed at the Brooks Forest Products Center on the Virginia Tech University campus and utilized a servo-hydraulic computer controlled 55 kip (245 kN) actuator with ± 6 in. (152 mm) of travel that allowed for continuous monitoring of the loads up to failure of the specimen. Deflection measurements were taken using potentiometers and linear variable differential transformers (LVDT's) at the locations shown in Figure 3.13 and were accurate to the nearest 0.001 in. (0.025 mm). Data were collected at 10 observations per second. Devices for measuring deflections were attached directly to the timber frame portion of the assemblies since it was movement of the underlying framing that allowed the assembly stiffness to be calculated as needed for a diaphragm analysis and design. In order to record shear deformations, deflection measurements were recorded between points C and J, G and H, H and E, and J and D, as shown in Figure 3.13. In order to obtain these, 1-1/2 in. (38 mm) holes were drilled

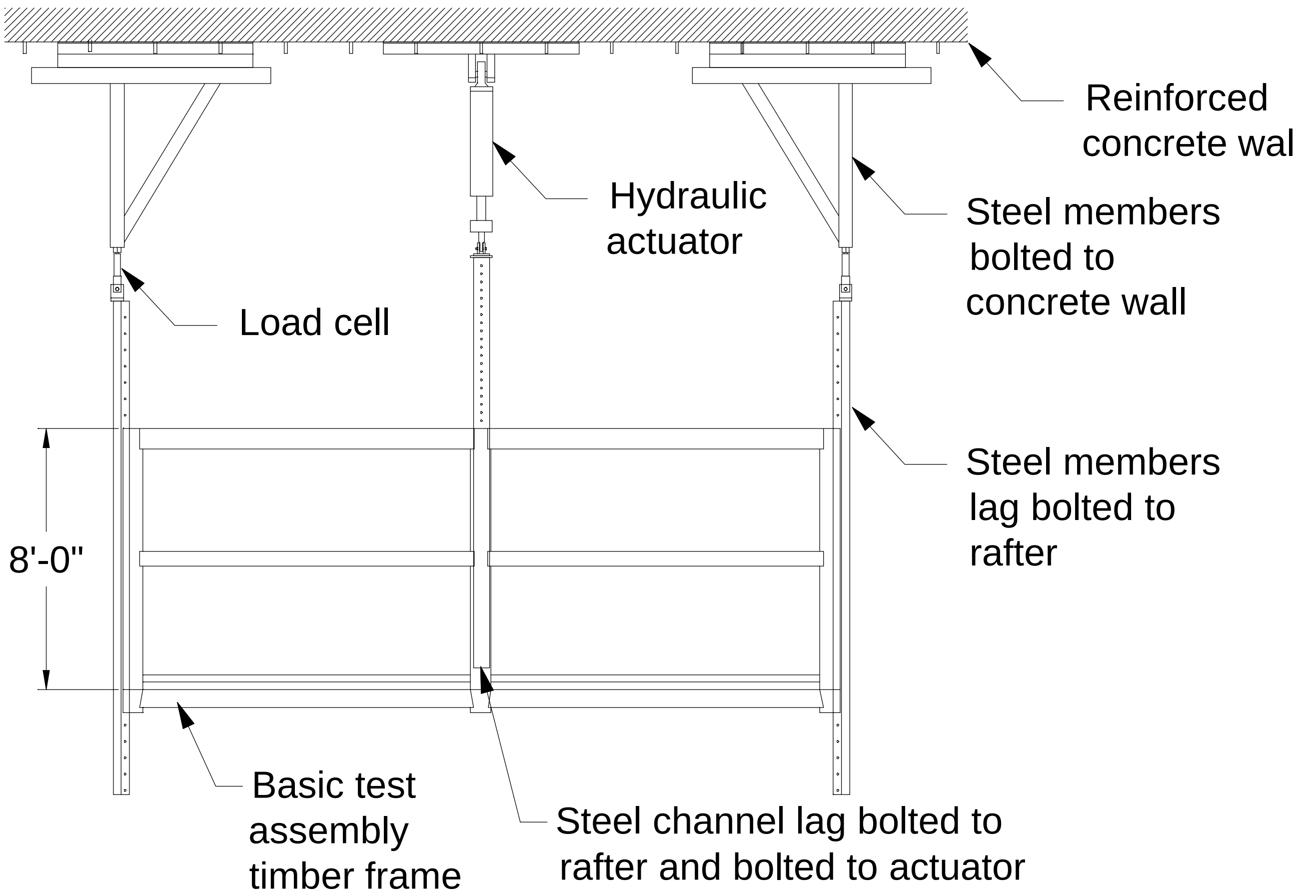


**Figure 3.12. Loading and support conditions for the basic test panel assembly (8 ft x 24 ft (2.44 m x 7.32 m)). Load is applied to the assembly utilizing the 55 kip hydraulic actuator to a maximum of 6 in. in either direction. (Only timber frame is shown; SIPs omitted for clarity.)**

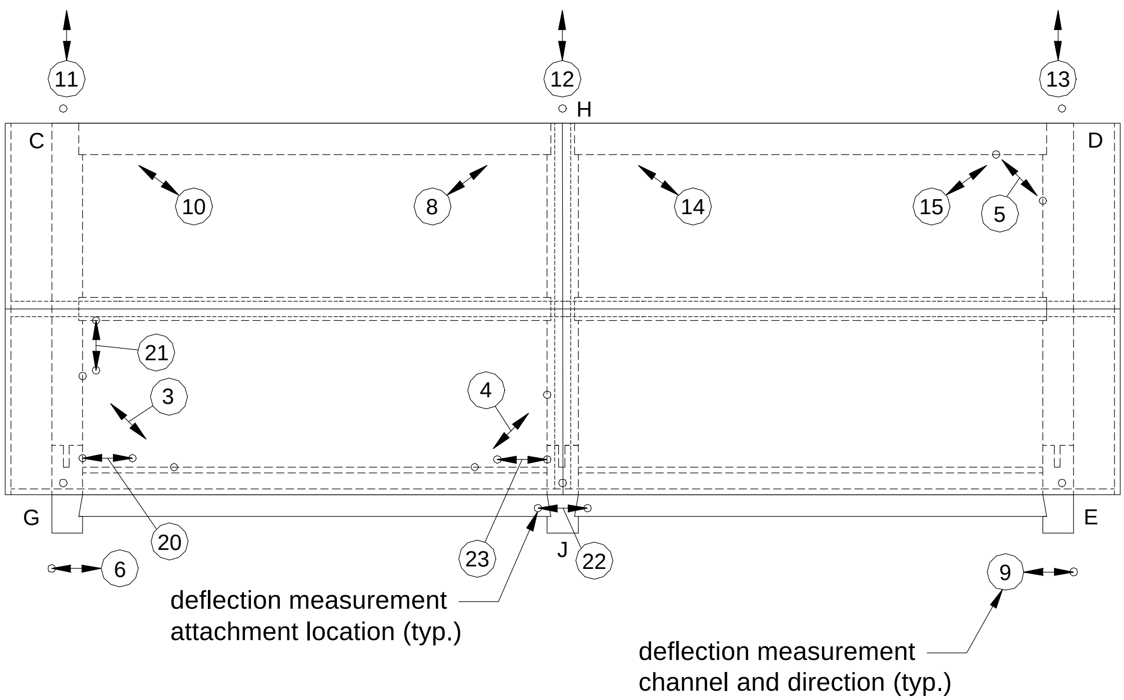


**Figure 3.13. Plan view of displacement measurement locations for the basic test panel assembly (8' x 24'). Channels 20 – 23 are LVDT's and all other channels are string potentiometers. (Not shown is a string potentiometer at the top of center rafter which monitors slip between the steel channel and the center rafter.)**

through the SIPs and the OSB between them and the timbers, in order for a pin to be driven into the timbers (Figure 3.14), allowing readings to reflect movement of the timbers without interference from the SIPs. These measurements were used to determine assembly deformation that was attributed to shear. Utilizing shear contribution data and equations provided by Fischer et al. (2001), the assembly deformation attributed to bending was also determined. These shear and bending contribution estimates were utilized for determining effective mechanical properties of timber frame and SIP assemblies, and are further elaborated on in Chapter 4. All measurements were recorded in digital format, utilizing LabTech Version 11 data acquisition software. A listing of the data acquisition devices, labels and descriptions is provided for Assembly 1 in Table 3.2. The remaining four assemblies utilized the same devices, except that LVDT's on channels 20 and 21 were relocated to determine rotation between Rafter A and Purlin AB and rotation between Plates AB and BC, respectively. LVDT's on channels 22 and 23 were relocated to determine slip between the SIPs and timbers near the top of Rafter C and on the Plate AB near the connection to Post Stub A, respectively

Determination of the loads applied to assemblies to assess stiffness and strength values were based on estimated strength required of assemblies and application of the appropriate factor of safety. Carradine et al. (2000) found that an allowable roof diaphragm shear value of 157 lb/ft (2,290 N/m) was needed to resist lateral loads for a typical residential timber frame subjected to 80 mile per hour (129 km/hour) winds. An additional wind analysis was performed on a large winery building constructed by Blue Ridge Timberwrights, which required an allowable shear value of nearly 300 lb/ft (4,380 N/m) of resistance. Therefore, 300 lb/ft (4,380 N/m) was assumed to be adequate for a wide range of typical construction, and was chosen as the target allowable shear value for these assemblies.

Published factors of safety have traditionally differed depending on which code researchers applied when designing test procedures. The post-frame community has utilized a factor of safety of 2.5 for shear strength based on monotonic loads, as found in ASAE EP484.1 (ASAE, 1991), which was modeled after testing performed on steel diaphragms with steel framing. Anderson and Bundy (1992b) advocated this value based on testing they performed and reviews of other tests, where there was a lack of


**Figure 3.14. Example of pin driven into timber and 1-1/2 in. (38 mm) hole drilled through SIP. Pin was utilized for diagonal displacements for use with shear deformation determinations.**

**Table 3.2. Descriptions, labels and channels for the displacement measurements obtained during testing**

| Channel | Device | Label | Description |
|---|---|---|---|
| 0 | UTP displacement | UTPdisp | Displacement of hydraulic ram |
| 1 | UTP load cell | UTPload | Load observed between ram and RB |
| 3 | 2” string pot. | RArotPL | Rotation between RA and PLAB |
| 4 | 20” string pot. | RBrotPL | Rotation between RB and PLAB |
| 5 | 2” string pot. | RProtRC | Rotation between RPBC and RC |
| 6 | 2” string pot. | Ashrink | Lateral movement at bottom of RA |
| 7 | 5” string pot. | Bslip | Slip between steel channel and RB |
| 8 | 20” string pot. | RBdiagPSA | Diagonal from top of RB to PSA |
| 9 | 2” string pot. | Cshrink | Lateral movement at bottom of RC |
| 10 | 20” string pot. | RAdiagPSB | Diagonal from top of RA to PSB |
| 11 | 20” string pot. | WallA | Movement between wall and RA |
| 12 | 20” string pot. | WallB | Movement between wall and RB |
| 13 | 20” string pot. | WallC | Movement between wall and RC |
| 14 | 20” string pot. | RBdiagPSC | Diagonal from top of RB to PSC |
| 15 | 20” string pot. | RCdiagPSB | Diagonal from top of RC to PSB |
| 16 | Load Cell | Left Load | Load observed between wall and RA |
| 18 | Load Cell | Right Load | Load observed between wall and RC |
| 20 | 2” LVDT | PLrotPSA | Rotation between PLAB and PSA |
| 21 | 2” LVDT | RArotP | Rotation between PA and PAB |
| 22 | 1” LVDT | PLrotPL | Rotation between PLAB and PLBC |
| 23 | 1”LVDT | PLrotPSB | Rotation between PLAB and PSB |

consistency between construction details and failure modes. A senior engineer with the American Plywood Association, Dr. Tom Skaggs, P. E., suggested minimum safety factors of 2.0 and 2.5 for wind loads and seismic loads, respectively (Skaggs, 1999). The discrepancy between the two was based on the current state of knowledge regarding wind and earthquake loads, essentially pointing out that wind loading has a larger database upon which the loading is defined. Guidelines presented in the International Building Code (ICC, 2000) indicated that for methods and materials of construction that were unable to be designed by approved engineering analysis or for which there were not applicable testing standards, the allowable stress design load should be defined as the maximum average test load (based on deflection criteria) or failure test load divided by 2.5. Considering the present understanding of loads and the lack of design and test criteria for timber frame and SIP roof assemblies, a factor of safety of 2.5 was used in this dissertation and was recommended for determining wind and seismic allowable design values of a timber frame and SIP roof diaphragm.

Utilizing the conservative design value of 300 lb/ft (4,380 N/m) and safety factor of 2.5, the resulting target failure capacity of the proposed roof test assemblies was at least 750 lb/ft (10,950 N/m). The basic test panel assemblies had a depth of 8 ft (2.44 m), which indicated that the minimum ultimate load applied to this test configuration was at least 12,000 lbs (53,400 N). Full-scale test assemblies were 20 ft (6.10 m) deep with a target minimum ultimate load of 30,000 lbs (134,000 N).

ASAE EP484.2 (ASAE, 1999) dictated a number of parameters required for test procedures when determining strength and stiffness values for panel assemblies, most of which were based on ASTM E72-80, Method for Conducting Strength Tests of Panels for Building Construction (ASTM, 1984). Both of these standards recommended applying and removing loads in three stages in order to assess creep, relaxation and permanent set of the assemblies. Since these factors were not relevant to this study, monotonic loads were applied continuously until failure was achieved, such that failure load was not reached before 10 minutes had elapsed. Failure was defined as either, “permanent failure of the cladding, framing or fastenings which would be objectionable based on appearance or performance” (ASAE, 1991) or a deflection at the midspan of the deep beam exceeding one twenty-fourth of the width, which was 5.33 in. (135 mm) displacement of the center rafter. Load-deflection measurements were recorded sufficiently throughout loading and failure of the assemblies in order to establish accurate and smooth load-deformation curves for each test. Specific details of testing procedures are presented in Section 3.4, and include steps for determining stiffness and strength of the assemblies along with steps aimed at quantifying contributions that panel screw density and perimeter edge boards made toward assembly stiffness.

Data obtained from tests, as well as information regarding the specifications of each test set up, were consistently recorded in such a manner that the final results remained useable with the design specifications as intended. The data sheet utilized for each of the tests to failure is shown in Figure 3.15. Collection of data and calculation of shear stiffness and strength on this form and through the data acquisition system for the loading ram, string potentiometers and LVDT's were sufficient to determine the necessary design parameters for use with ASAE EP 484.2 (ASAE, 1999) diaphragm design methodology.

In accordance with ASTM E 455 (ASTM, 2000) and ASAE EP558 (ASAE, 1999) recommendations, two panel assembly tests were conducted for the 8 ft (2.44 m) by 24 ft (7.32 m) basic test panel assembly, hereafter referred to as Assembly A. Two tests were also conducted for the full-scale test assemblies that were 20 ft (6.10 m) deep and 24 ft (7.32 m) wide, hereafter referred to as Assembly B, with the remaining details of the assemblies unchanged. Utilizing two different test assembly diaphragm lengths provided data for the creation of equations to provide designers the ability to estimate stiffness of the entire roof system from stiffness obtained utilizing a proposed Assembly A test, as discussed in Chapter 4. Test procedures for the nearly full-scale Assembly B test assemblies were the same as for the Assembly A tests, and were conducted in order to determine the existence of any size effects on strength, and to define the impact of diaphragm length on stiffness. Ultimate strength (without a safety factor) of the different sized panel assemblies was the average strength from the three Assembly A tests and the lower of the two Assembly B tests. Design stiffness was the average of the calculated stiffness values for the tests for each assembly type. Ultimate shear load of the panel assembly, $V_{ult}$, was calculated as the maximum load, $P_{ult}$, at failure divided by two, since the load was applied at the center of the simple span. Determining the stiffness of the

| | | |
|---|---|---|
| Test ID: Assembly 1 | | Notes and Comments |
| Date: | | Example of data sheet |
| Overall Length: | 8'-0" | |
| Overall Width: | 24-0" | |
| Frame Bay Spacing: | 10'-8" | |
| Purlin Spacing: | 4'-0" | |
| SIP Size: | 4'-0"x12'-0" | |

| | Timbers | | | | |
|---|---|---|---|---|---|
| | Rafter | Purlin | Post Stub | Top Plate | Ridge Purlin |
| Number | 3 | 2 | 3 | 2 | 2 |
| Actual Size | 7.5"x9.5" | 5.5"x7.5" | 7.5"x9.5" | 5.5"x11.5" | 7.5"x7.5" |
| Species | Southern Pine | Southern Pine | Southern Pine | Southern Pine | Southern Pine |
| Grade | No. 2 | No. 2 | No. 2 | No. 2 | No. 2 |
| Stiffness | 1200000 psi | 1200000 psi | 1200000 psi | 1200000 psi | 1200000 psi |
| Specific Gravity | 0.55 | 0.55 | 0.55 | 0.55 | 0.55 |

| | Structural Insulated Panels |
|---|---|
| Manufacturer | Insulspan |
| Rigid Insulation | 5.5 in. thick polystyrene |
| Exterior Panel | 7/16 in. OSB |
| Interior Panel | 7/16 in. OSB |
| Splines | 7/16 in. x 3 in. OSB |
| OSB Spacer | 19/32 in. x various OSB |
| Edge Members | 2x6 No. 2 SPF |

| | Fasteners | | |
|---|---|---|---|
| | SIP/Timber | Splines | Edges |
| Manufacturer | Olympic | Spotnails | Spotnails |
| Type | screw | ring shank | ring shank |
| Diameter | 0.190" | 0.113" | 0.113" |
| Length | 9" | 2.5" | 2.5" |
| Base Metal | steel | steel | steel |

| | Results |
|---|---|
| Shear Strength | must be calculated |
| Shear Stiffness | must be calculated |

**Figure 3.15. Example of data collection sheet for use with timber frame and SIP roof diaphragm tests.**

assemblies was more complex and required the creation of load-deflection curves and performing calculations, which are discussed in Chapter 4.

The first Assembly A test, Assembly 1, utilized low level cyclic loading to determine stiffness of solely the timber frame, the sheathed assembly with half of the recommended panel screws, the sheathed assembly with a full panel screw schedule, and the fully screwed and sheathed assembly without and with edge boards installed. Assembly 1 was then monotonically tested to failure for determination of ultimate failure load at a loading rate of 0.17 in./minute (4.3 mm/minute) applied as a pulling force (compression in the eave plate). Due to the asymmetry of the roof assemblies tested, all tests to failure were conducted monotonically rather than cyclically, although Assemblies 2 through 5 were pushed to failure (tension in the eave plate) rather than pulled, as in Assembly 1. Asymmetry was a result of different types of joints at the top and bottom of the center rafter. At the top of the rafter, where the hydraulic actuator was connected, the ridge purlins were housed 1-1/2 in. (38 mm) and held in place with two steel straps as illustrated in Figure 3.6. The joint at the bottom of the center rafter, where two plates and the rafter connected to the post stub, was a splined timber joint as shown in Figure 3.11, and was a much stiffer and stronger connection than the joint at the top of the center rafter.

The second and third Assembly A tests, Assembly 2 and Assembly 3, and the Assembly B tests, Assembly 4 and Assembly 5, also utilized low level cyclic loading to assess the same cyclic configuration stiffnesses as Assembly 1.

### *3.3.2 Quasi-Static Cyclic Test Protocols*

In the foreseeable future, timber frame structures will likely come under increased building code scrutiny with regard to their performance when subjected to seismic loads. It was therefore determined for this research to calculate the low level stiffness of all the assemblies utilizing the Basic Loading History quasi-static cyclic regime as described by Krawinkler et al. (2001). This regime was controlled by deformations and intended to represent a potentially damaging loading schedule from ground motions in the Los Angeles area of California. A reference deformation, $\Delta$, represented the assembly's deformation capacity, and was obtained from the monotonic test to failure conducted on Assembly 1. Monotonic deformation, $\Delta_m$, was defined as the deformation at which the applied load (during static testing) initially dropped to below 80% of the maximum applied load. Krawinkler et al. (2001) recommended using 60% of $\Delta_m$ for determination of reference deformation $\Delta$, which resulted in $\Delta = 0.6(\Delta_m)$. The specific loading history to be used is explained and shown graphically in Appendix A. Data sheets from static testing were also used to document cyclic tests. Additional information as specified by Krawinkler et al. (2001) was also documented. It was initially planned to cyclically induce failure in several of the test assemblies, but after attempting cyclic failure with Assembly 2, the previously mentioned asymmetry of the assemblies was noted and all the tests were conducted utilizing cyclic loading for stiffness determination and monotonic loading to reach failure load. The authors provided means of performing force controlled quasi-static cyclic and shake table tests, but for this study only the deformation controlled, quasi-static cyclic regime was utilized.

### *3.3.3 Test Apparatus*

In order to conduct the beam diaphragm test as recommended in ASAE EP558 (ASAE, 1999), it was necessary to fabricate a system of steel brackets that held the outer rafters secure while allowing for movement of the center rafter in a direction parallel to the longitudinal axes of the rafters. A plan view of the testing apparatus is shown in Figure 3.12. Due to the elevation of the hydraulic actuator, the bottoms of the rafters were located approximately 12-3/4 in. (324 mm) from the concrete test bed, which coincided with the centerline of the actuator.

Each of the outer rafters was held secure by attaching them to the reinforced concrete wall cast as part of the test bed, as seen in Figure 3.12. Hat-shaped sections of steel, 6 ft (1.83 m) long, welded from steel angles and channel, were bolted to the concrete wall on 7/8 in. (22 mm) diameter threaded studs embedded into the concrete. Welded to these hat-shaped members were 6 ft (1.83 m) long, 3 x 5 x 3/8 in. (46 x 127 x 9.5 mm) rectangular steel tubes with holes drilled in them in order for the triangular members to be attached. The triangular members were welded from 3 x 5 x 3/8 in. (46 x 127 x 9.5 mm) and 4 x 6 x 1/2 in. (102 x 152 x 13 mm) rectangular steel tubes. The purpose of the triangular members was to distribute the load induced by the hydraulic actuator to 6 threaded studs on each rafter. Threaded steel rod was threaded into the end of each triangular member and a load cell was threaded onto each side. Load cells were utilized at these locations as a means of monitoring the applied load as well as determining how much load induced by the actuator was lost due to friction. Rectangular steel tubes, 3 x 5 x 3/8 in. (46 x 127 x 9.5 mm), 20 ft. (6.10 m) long were pinned to each load cell utilizing a 1 in. (25 mm) thick plate of steel welded to a 5/8 in. (16 mm) thick plate of steel that was welded to the end of each tube. Along the full length of each tube, flush with the bottom, was welded a 3 x 3 x 1/4 in. (46 x 127 x 9.5 mm) steel angle. These angles provided a ledge for the outer rafters to be attached to for testing by having 5/8 in. (16 mm) diameter holes drilled at 6 in. (152 mm) on-center along the full length of each ledge. Outer rafters for the test panel assemblies were held in place using fifteen 5/8 in. (16 mm) diameter, 3 in. (76 mm) long lag screws in each rafter. NDS-97 (AF&PA, 1997), Section 9.1.2 stipulated that for lag screws, pilot holes should be drilled that were 60% - 75% of the shank diameter of the lag screws for wood species having specific gravities between 0.5 and 0.6. Published specific gravity for Southern Pine is 0.55 (AF&PA, 1997); therefore, 3/8 in. (9.5 mm) pilot holes were drilled into the bottom of each outer rafter for attachment to the steel ledges.

Upon loading of the test assemblies, there was potential for the ends of the outer rafters, farthest away from the hydraulic actuator, to lift off of the test bed due to the probable eccentricity between the plane in which SIPs resisted the lateral load and the level at which the actuator applied the load. In order to avoid this out-of-plane movement, two 5/8 in. (16 mm) diameter threaded rods were placed into inserts in the concrete test bed near the edge of the test bed. The rectangular steel tubing to which the steel angles that formed the ledge were welded to had 1 in. (25 mm) diameter holes drilled into them, through which the treaded rods were placed, as shown in Figure 3.16. Nuts were placed under the steel tube, which provided bearing for the tubes. Nuts were also placed on top of the tubes to restrain the rafter ends during testing.

Simulation of lateral loads was accomplished by moving the center rafter back and forth utilizing the hydraulic ram previously discussed. The center rafter was attached to the web of a 6 x 2 x 1/4 in. (152 x 51 x 6.4 mm) steel channel. During testing, this channel section proved inadequate for the applied loads and was reinforced, as discussed in Chapter 4. The same lag screws and pilot holes were used as for the outer rafters, except that the holes in the channel were 3/4 in. (19 mm) diameter and were drilled 3 in.

**Figure 3.16. Threaded rods in concrete utilized to support the outer rafter ends at the appropriate height while restraining the rafter ends from lifting off of the concrete test bed during testing.**

on-center, which allowed for 27 lag screws to be utilized for attachment for Assembly A tests and 41 lag screws to be utilized for Assembly B tests. Welded to the end of the steel channel was a 1 in. (25 mm) thick steel plate, which bolted to another 1 in. (25 mm) thick steel plate that was pinned directly to the hydraulic actuator.

During loading, as with the ends of the outer rafters, there was potential for lifting of the test apparatus at the top of the center rafter where it attached to the hydraulic actuator. A steel frame was welded from 1/4 in. (6.4 mm) thick steel plates and angles and bolted directly to the concrete test bed utilizing inserts in the concrete. A set of rollers was bolted on top of the frame, which allowed free movement of the steel channel while restraining it from lifting off of the test bed, as shown in Figure 3.17. The roller apparatus was also modified during the testing to accommodate reinforcement of the steel channel and is discussed in Chapter 4.

## 3.4 Testing Procedures

### *3.4.1 Instrumentation Calibration*

As previously discussed, numerous string potentiometers and LVDT's were utilized to gather the necessary displacement data for incorporation into design procedures for diaphragm analysis of timber frame and SIP structures. It was necessary to check the calibration of these instruments to ensure accurate data collection. String potentiometers were calibrated by pulling out the string and placing calibration blocks of known lengths between the collar at the end of the string and the body of the instrument. LVDT's were calibrated by placing calibration blocks between the plunger and the surface the plunger was bearing against. Data were acquired utilizing the blocks and then compared with the known lengths of the blocks. Scale factors within the data acquisition system for each instrument were checked and adjusted as necessary to bring accuracy to within 1%. All instruments were calibrated in December 2001 and again in May 2002 before the failure tests were conducted.

### *3.4.2 Assembly Displacement Level Determination*

Due to the lack of information regarding the behavior of timber frame and SIP roof assemblies, a series of initial tests were conducted on the bare timber frame in order to obtain a maxim displacement level for the frame and to gather stiffness data for later

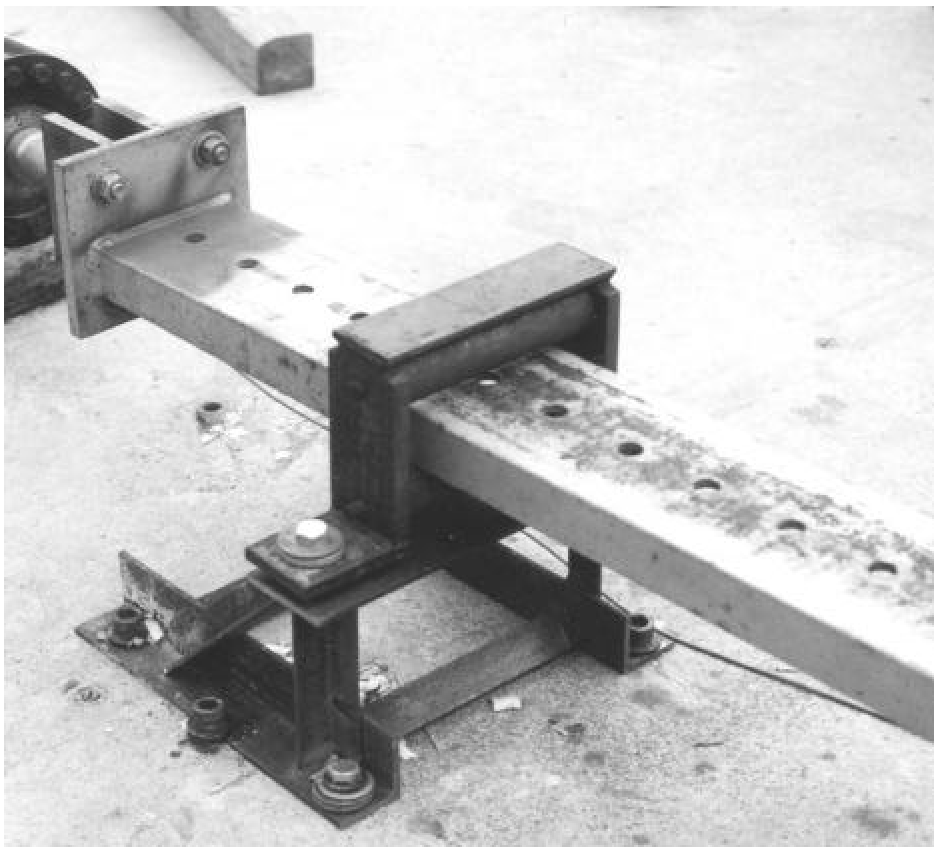

**Figure 3.17. Rollers and frame bolted to concrete utilized to restrain steel channel attached to center rafter from lifting off of the concrete test bed during testing.**

comparison with the sheathed assemblies.

It was first necessary to determine a maximum displacement of the center rafter that would take the slack out of the bare frame and begin to put load into the joints without causing any structural damage to the assembly. Maximum displacement was established by monotonically displacing the center rafter while visually monitoring the joints, and observing the load and displacement curve generated in real time by the data acquisition system. At the displacement where the joints visibly tightened and load applied to the frame began to increase for equivalent displacement increases, the test was terminated. A maximum displacement of 0.75 in. (19 mm) in either direction was determined to not cause damage to the bare timber frame.

It was assumed that the maximum displacement for the sheathed assembly would be considerably less than that of the bare timber frame, therefore it was necessary to conduct a series of stiffness tests with maximum displacements lower than 0.75 in. (19 mm) in order for comparisons to be made between tests at equal displacements. The waveforms utilized for the stiffness tests were scaled versions of the Krawinkler (2001) quasi-static cyclic regime and are shown in Appendix A. Scaling was accomplished utilizing a program developed by William P. Jacobs V in Excel that allowed the waveform to be adjusted based on the maximum desired displacement and the cycle time, and could be directly imported into the computer program that controlled the hydraulic actuator. Stiffness tests were conducted upon the bare timber frame for Assembly 1 with maximum displacements ranging from ± 0.08 in. (2.0 mm) to ± 0.75 in. (19 mm), at increments of approximately ± 0.075 in. (1.9 mm) per test.

After attaching the SIPs to Assembly 1, the same procedure was used to determine that the maximum allowable displacement was ± 0.10 in. (2.5 mm) for the sheathed assembly. Maximum displacement of the sheathed assemblies was established by examining plots of the applied loads versus the displacements of the center rafter. Maximum displacements for tests described above were increased until there was a slight decrease in incremental load for the incremental displacement, indicating that a further displacement would likely have damaged the specimens. During testing of Assembly 2 it was determined that sheathed assemblies could be displaced to ± 0.25 in. (6.4 mm) without inducing any damage. All stiffness tests were therefore conducted utilizing a maximum displacement of ± 0.10 in. (2.5 mm) and the bare frame and fully sheathed frame (including full SIP screw schedule and edge boards installed) stiffness tests were conducted utilizing a maximum displacement of ± 0.25 in. (6.4 mm). All specimens were loaded to failure as described in the following section.

### *3.4.3 Schedule of Testing*

In order to accomplish the research objectives discussed in previous sections, the testing schedules presented in Tables 3.3 through 3.7 were developed for each roof panel assembly. Test schedules were slightly different for the first two Assemblies as the details of testing were refined in response to results obtained. Specifics of testing are presented in Chapter 4, along with a rationale for changes in testing schedules.

## 3.5 Analysis of Testing Results

After completion of tests as described in previous sections, analyses were performed to assess appropriate applications of utilizing SIP and timber frame roof diaphragm assemblies as lateral load resisting elements for buildings. Monotonic loading tests provided enough information to allow for completion of the diaphragm design according to ASAE EP484.2 (ASAE, 1999), which indicated that tested roof assemblies adequately reduced the magnitude of lateral loads resisted by internal frames of the buildings, which were shown by Carradine et al. (2000) to be theoretically overstressed in certain members. A procedure was developed for adjusting test panel assembly stiffness for determination of roof assembly stiffness based on diaphragm length. Quasi-static cyclic test results were analyzed in order to determine how these types of assemblies could potentially be expected to perform under seismic loading. Additionally, data from quasi-static cyclic tests were used to make recommendations regarding seismic design to assess potential of these types of buildings to resist earthquake loads. All of these analyses are discussed in Chapters 4 and 5.

**Table 3.3. Testing protocol for Assembly 1**

| Assembly 1: Basic Test Panel, 8 ft Deep, 24 ft Wide | |
|---|---|
| Step 1 | Assembled timber frame only and subjected it to 6 sets of 5 cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine frame stiffness, $k_f$ |
| Step 2 | Installed OSB spacing strips and attached SIPs with screws at 24 in. on center then subjected it to 6 sets of 5 cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine assembly stiffness, $k_{a1}$ |
| Step 3 | Installed remaining screws at 12 in. on center then subjected it to 6 sets of 5 cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine assembly stiffness, $k_{a2}$ |
| Step 4 | Installed 2x6 edge boards then subjected it to 6 sets of cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine assembly stiffness, $k_{a3}$ |
| Step 5 | Displaced center rafter monotonically at a rate of 0.17 in./minute applied as a pulling force until assembly failed, to determine ultimate strength, P, and failure deformation, $\Delta_f$ |

**Table 3.4. Testing protocol for Assembly 2**

| Assembly 2: Basic Test Panel, 8 ft Deep, 24 ft Wide | |
|---|---|
| Step 1 | Assembled timber frame only and subjected it to 6 sets of 5 cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine frame stiffness, $k_f$ |
| Step 2 | Installed OSB spacing strips and attached SIPs with screws at 24 in. on center then subjected it to 6 sets of 5 cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine assembly stiffness, $k_{a1}$ |
| Step 3 | Installed remaining screws at 12 in. on center then subjected it to 6 sets of 5 cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine assembly stiffness, $k_{a2}$ |
| Step 4 | Installed 2x6 edge boards then subjected it to 6 sets of cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine assembly stiffness, $k_{a3}$ |
| Step 5 | Displaced center rafter monotonically at a rate of 0.17 in./minute applied as a pulling force up to a maximum load of 4 kips |
| Step 6 | Subjected assembly to 6 sets of 5 cycles each, beginning at 0.042 in. displacement per cycle, increasing by 0.042 in. up to 0.25 in. to determine assembly stiffness, $k_{a4}$ |
| Step 7 | Displaced center rafter monotonically at a rate of 0.17 in./minute applied as a pushing force until assembly failed, to determine ultimate strength, P |
| Step 8 | Utilizing the reference deformation, Δ, from Assembly 1 monotonic test, conducted the CUREE protocol (Krawinkler, et al., 2001) until excessive deformation of steel channel was noted and test was terminated |

**Table 3.5. Testing protocol for Assembly 3**

| Assembly 3: Basic Test Panel, 8 ft Deep, 24 ft Wide | |
|---|---|
| Step 1 | Assembled timber frame only and subjected it to 6 sets of 5 cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine frame stiffness, $k_{f1}$ |
| Step 2 | Subjected bare frame to 6 sets of 5 cycles each, beginning at 0.042 in. displacement per cycle, increasing by 0.042 in. up to 0.25 in. to determine frame stiffness, $k_{f2}$ |
| Step 3 | Installed OSB spacing strips and attached SIPs with screws at 24 in. on center then subjected it to 6 sets of 5 cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine assembly stiffness, $k_{a1}$ |
| Step 4 | Installed remaining screws at 12 in. on center then subjected it to 6 sets of 5 cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine assembly stiffness, $k_{a2}$ |
| Step 5 | Installed 2x6 edge boards then subjected it to 6 sets of cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine assembly stiffness, $k_{a3}$ |
| Step 6 | Subjected assembly to 6 sets of 5 cycles each, beginning at 0.042 in. displacement per cycle, increasing by 0.042 in. up to 0.25 in. to determine assembly stiffness, $k_{a4}$ |
| Step 7 | Displaced center rafter monotonically as a pushing force until assembly failed, to determine ultimate strength, P |

**Table 3.6. Testing protocol for Assembly 4**

| Assembly 4: Full-Scale Test Panel, 20 ft Deep, 24 ft Wide | |
|---|---|
| Step 1 | Assembled timber frame only and subjected it to 6 sets of 5 cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine frame stiffness, $k_{f1}$ |
| Step 2 | Subjected bare frame to 6 sets of 5 cycles each, beginning at 0.042 in. displacement per cycle, increasing by 0.042 in. up to 0.25 in. to determine frame stiffness, $k_{f2}$ |
| Step 3 | Installed OSB spacing strips and attached SIPs with screws at 24 in. on center then subjected it to 6 sets of 5 cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine assembly stiffness, $k_{a1}$ |
| Step 4 | Installed remaining screws at 12 in. on center then subjected it to 6 sets of 5 cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine assembly stiffness, $k_{a2}$ |
| Step 5 | Installed 2x6 edge boards then subjected it to 6 sets of cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine assembly stiffness, $k_{a3}$ |
| Step 6 | Subjected assembly to 6 sets of 5 cycles each, beginning at 0.042 in. displacement per cycle, increasing by 0.042 in. up to 0.25 in. to determine assembly stiffness, $k_{a4}$ |
| Step 7 | Displaced center rafter monotonically as a pushing force until assembly failed, to determine ultimate strength, P |

**Table 3.7. Testing protocol for Assembly 5**

| **Assembly 5: Full-Scale Test Panel, 20 ft Deep, 24 ft Wide** | |
|---|---|
| Step 1 | Assembled timber frame only and subjected it to 6 sets of 5 cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine frame stiffness, $k_{f1}$ |
| Step 2 | Subjected bare frame to 6 sets of 5 cycles each, beginning at 0.042 in. displacement per cycle, increasing by 0.042 in. up to 0.25 in. to determine frame stiffness, $k_{f2}$ |
| Step 3 | Installed OSB spacing strips and attached SIPs with screws at 24 in. on center then subjected it to 6 sets of 5 cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine assembly stiffness, $k_{a1}$ |
| Step 4 | Installed remaining screws at 12 in. on center then subjected it to 6 sets of 5 cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine assembly stiffness, $k_{a2}$ |
| Step 5 | Installed 2x6 edge boards then subjected it to 6 sets of cycles each, beginning at 0.017 in. displacement per cycle, increased by 0.017 in. up to 0.10 in. to determine assembly stiffness, $k_{a3}$ |
| Step 6 | Subjected assembly to 6 sets of 5 cycles each, beginning at 0.042 in. displacement per cycle, increasing by 0.042 in. up to 0.25 in. to determine assembly stiffness, $k_{a4}$ |
| Step 7 | Displaced center rafter monotonically as a pushing force until assembly failed, to determine ultimate strength, P |

### 3.6 Summary

Objectives of the previously discussed testing program were to obtain diaphragm test data and develop design procedures, whereby designers of timber frame buildings with SIPs could utilize diaphragm action of the roof system for lateral design of these structures. Test results, and subsequent analyses, provided necessary parameters, that were currently unavailable for these structural assemblies, thus allowing designers to use ASAE EP484.2 (ASAE, 1999) post-frame diaphragm design procedures for contemporary timber frame structures, as elaborated upon in following chapters.