Chapter 2: Literature Review

2.1 Historic Timber Frame Structures

The notion of connecting large timbers together to create shelter from the elements is one that has been with humankind from the earliest use of tools that could work wood into useable forms. All over the world, different cultures, climates and geographic conditions resulted in various methods of joining wood members to create a legacy of structures that have lasted for centuries and been in forms ranging from the humble log homes of Norway to the elaborate temples of Japan. As early as 200 B. C., structures were being constructed throughout Europe and Asia that made use of the fundamentals of joinery that are still used in timber frame structures today (Benson, 1980). Several authors (Benson, 1980 and 1997, Sobon, 1994, and Knox, 1997) have discussed the variations among different timber frame structural typologies throughout the world and the historic periods during which they were created.

Relevant characteristics of these ancient buildings that defined them as timber frames were large timber columns, beams, rafters and purlins joined together using wooden joints, forming a skeletal frame that served as the fundamental structure for the building. While some definitions of timber frame include construction methods such as balloon and platform wood framing, current research is focused on this more venerable building type, although with some modern accommodations as explained in the following sections.

As immigrants from Europe landed upon the eastern shores of America, they brought the techniques of timber framing from their native lands and utilized the untouched forests of this country to create homes, barns and other timber frame structures. An amalgam of English, French, German, Dutch and Scandinavian timber framing resulted in a uniquely American style that was suited to the climates and geography of this country (Benson, 1980). Timber framing remained the mainstay of building construction in this country until the early 19th century. As America embraced industrialization, the availability of milled lumber in combination with mass production of metal fasteners led to the proliferation of balloon and platform framing and the decline of craft intensive timber frames (Ehringer, 1995 and Price, 1996). Other parts of the world continued to build utilizing large timbers. In Japan, it is still common for single-family residences to be constructed utilizing mortise and tenon joinery for the framing members (Lam et al., 2001).

Throughout the 19th century and for most of the 20th century, the art of timber framing lay dormant in the United States. The 1970s, however, ushered in new ways of thinking about the natural, as well as the built environment. Energy efficiency (Price, 1996), the importance of the processes involved in building (Sobon, 1994), durability (Benson, 1980) and structural and architectural honesty (Knox, 1997) of timber frames attributed to the resurgence of interest in timber frame buildings as an alternative to more common, mass-produced structures. Several timber frame companies sprang up as a result of this interest, and as of 1999, it was estimated that 216 timber frame manufacturing companies were operating successfully in the United States and Canada (O'Connell and Smith, 1999).

2.2 Contemporary Timber Frame Structures

Timber frames that have been designed and built in the United States since the revival of the 1970s have been modified from their predecessors in several ways. The most significant of these differences are methods of manufacturing, erecting and cladding timber frames. While these variations sound as though modern timber frames would be vastly different from older examples, the wooden joints, concepts and reasons for constructing in this manner are much the same as they were hundreds of years ago.

The manner in which modern timber frames are fabricated and erected have changed drastically over the past 30 years. Some framers still rely on hand hewn timbers and joints cut only with hand tools, but the majority of individuals and companies who manufacture timber frames take advantage of specialized equipment that facilitates cutting of mortises, tenons and dovetails at the appropriate scale for structural applications. Beemer (2001) discussed this equipment and provided examples of its use. The most advanced of these machines is The Hundegger, an "automated timber cutting machine" (Brungraber, 1999) that cuts joints and drills holes for pegs using computer precision. Timbers used are typically milled and planed by machine before any joints are cut and may come from distant states rather than locally felled and hewn timbers as used in the past. Timbers are also being obtained utilizing recycled timbers from older buildings that are being torn down (Falk et al., 2000). With regard to the erection of timber frames, modern erection is primarily done using a crane or boom truck, whereas in the past, timber frames were raised by hand using pike poles, block and tackle and lots of man power.

The most significant difference between traditional timber frames and contemporary ones is the manner of cladding the wooden skeletons. Traditional structures enclosed or filled in between the timber frames with a myriad of materials including logs, clay, brick or clapboards over studs (Benson, 1980). Some of these systems also served to insulate the structures whereas others needed additional insulating materials. The majority of contemporary structures integrate enclosure of the frame with insulation by using SIPs. Straw bales and stud walls are also used with modern timber frames, but not nearly as frequently as SIPs.

SIPs consist of a layer of stiff insulating foam varying from 3-1/2 in. (124 mm) to 11-1/4 in. (286 mm) thick with a layer of 3/8 in. (9.5 mm) to 5/8 in. (15.9 mm) thick oriented strand board (OSB) on one side and oriented strand board, gypsum drywall, or some other interior finish, such as tongue and groove paneling, on the other side. The insulating foam for SIPs can be polyurethane, polyisocyanurates, expanded polystyrene or extruded polystyrene, with each having different R-values ranging from 15 to 25 for a 3.5 in. (89 mm) thick panel core (Price, 1996). Although manufacturing techniques vary among companies, SIPs are typically assembled utilizing structural-grade adhesive to bond the pre-formed foam core to the outer skins, or by filling the space between the outer skins with uncured foam. Both manufacturing processes utilize pressure to ensure adequate bonding between the foam core and the outer skins. SIPs are connected together and attached to the frames by various methods and these will be discussed in the following sections.

Numerous advantages are derived from using SIPs for enclosing timber frame structures. Andrews (1992) noted the speed of using SIPs, superior energy performance and the strength of SIPs as three specific benefits to using them. Speed of erection was important because it reduced labor costs, reduced interest costs on financing, and reduced exposure and theft on the job site (Carusone, 1992). The superior energy performance of

SIPs was realized not only in the high R-values, but also in the more tightly sealed envelop SIPs created around a building (Andrews, 1992) and long term energy savings (Pierquet et al., 1998). At present, the major disadvantage of using SIPs is the lack of data on their structural behavior when used as roof diaphragms or shear walls.

2.3 Timber Frame Research

Resurgence in interest in timber frame structures as an alternative to more typical wood construction methods has brought with it a moderate amount of research regarding the behavior of timber frame buildings. Published research on the modern conception of timber frame construction typically addresses either the strength of joints and members within a frame, or structural modeling of the timber frame structure, but only as independent frames, not including cladding as a load resisting element.

2.3.1 Timber Frame Joint Engineering

A portion of recent literature on timber frame engineering has concentrated on the design and capacity of the members and joints within the structure. Early work by Magee and Brungraber (1985 and 1986) sought to compare calculated joint capacity values with tests on joints in tension. Capacities of tested joints were much higher than predicted by engineering calculations and the authors made recommendations for future testing. Brungraber (1992) and Chappell (1994) discussed the mechanics and provided examples of tension joinery within timber frames. In a separate article, Chappell (1985a) provided a layman's description of the manner in which wind and gravity forces were distributed within a typical timber frame and how various members resisted these forces. Simple mortise and tenon joints were tested in tension by Kessel and Augustin (1995) in order to obtain data to be used for design of timber frame structures in Germany. An additional 110 oak and spruce mortise and tenon joints were tested in tension (Kessel and Augustin, 1996) and recommended allowable loads were stipulated. While these two testing programs provided some direction for American research, methods of joinery were somewhat different from current American timber framing practices and should not necessarily be used directly for design purposes.

Schmidt and MacKay (1997) and Schmidt and Daniels (1999) wrote on the behavior and design of traditional timber connections and performed experiments to identify failure mechanisms of these types of wooden connections. Schmidt and MacKay (1997) concluded that current NDS-97 (AF&PA, 1997) yield model equations were applicable to some mortise and tenon joint failures, but additional modes would be needed to fully represent the scope of timber frame joint failures. Schmidt and Daniels (1999) developed several parameters for behavior of pegged joints in tension and made recommendations as to the design of these types of joints. Several wood species were tested by Schmidt and Erikson (2000) for use as 1-inch diameter pegs subject to tension, resulting in a yield stress and modulus of rupture for each test performed. Seasoning and load duration effects on joints in tension were studied by Schmidt and Scholl (2000), who concluded, "design of mortise and tenon joints for long-term loading is a serviceability concern rather than a strength issue." and proposed minimum joint detailing distances that included considerations for wood moisture content and drawbore effects.

Burnett et al. (2002) investigated the effects of end distance on joint stiffness, proportional limit, 5% offset yield load and ultimate load for joints using 1-inch diameter pegs. While stiffness of the joints was not affected by end distance, they strongly recommended further research be conducted on end distances between 100% and 40% of NDS-1997 (AF&PA, 1997) requirements, especially for wood species with low tensile strength.

Church and Tew (1998) conducted experiments on bearing strength of white oak pegs in red oak and Douglas fir timbers and elucidated the importance of grain orientation and species when determining the strength of joints where bearing is a critical failure mode. A finite element analysis was also conducted to help verify their simplified experiments.

Daniels et al. (1999) performed full-sized tests of wood-pegged joints using recycled Douglas fir, southern yellow pine and red oak, and are currently developing recommendations for the design of these types of joints in tension. Brungraber (1993) developed several charts based on the NDS-1991 (AF&PA, 1991) that provided some of the parameters needed to design mortise and tenon joints for different wood species. Sandberg et al. (1996) tested four different types of timber frame joints under gravity and lateral loads and made suggestions for modeling these joints based on test results. More recently, Sandberg et al. (2000) tested 72 pegged joints and developed a model for estimating joint stiffness and strength based on member specific gravity. The pegs were red oak, which were considered typical for contemporary timber framing.

This body of research has provided valuable information for designers of timber frames as well as for researchers who have attempted to analyze and model the behavior of timber frames.

2.3.2 Timber Frame Analysis Techniques

Research that has been conducted on strength and other parameters that were critical for effective design of timber frame joints and members has provided a piece of the puzzle of designing timber frame structures. An additional puzzle piece was the creation of a method of analyzing and modeling these structures that accurately represented their complex structural behavior.

Most research focused on timber frame structural behavior has utilized finite element analysis, which models the structure as an assembly of discrete elements, each having specific degrees of freedom (Holzer, 1985). Levin (1993) performed a finite element analysis on a timber frame structure using symmetric and asymmetric combinations of dead, live, snow and wind loads. This analysis provided data on stresses in the members of the modeled timber frame, which was created under the assumption that, “the frame must stand on its own without any help from the skin” (Levin, 1993). The initial model, which was considered to be typical of current residential design, had to undergo significant modifications to be within design limitations. These modifications included a 25% increase in frame material and resulted in an impractical design where some of the joints were still loaded beyond design capacity. A similar finite element analysis was performed by Levin (1995, 1996) on a slightly different structure and calculated stresses were used to exemplify design procedures for some of the joints in the analyzed timber frame.

Analyses have also been performed that specifically sought to model behavior of joints and members within a timber frame, rather than the entire frame. Bulleit et al. (1996) conducted matrix analyses and portal frame analyses on joints that were experimentally tested by Sandberg et al. (1996). Results varied in accuracy and the authors concluded that more work was needed before these techniques could be used for design purposes. A method of analyzing timber frames that accounts for connection behavior was developed by Bulliet and Drewek (1999) and incorporated characteristic stiffness of the pegs and contact between the wood members within the analysis. The authors recommended comparing the results to full-scale frame tests to ensure the validity of the structural analogs.

Although there were numerous publications on joint and member design and timber frame building analysis, none of the papers described methods for including the structural contribution of materials used to clad the timber frames. Not to say that the previously mentioned authors were not aware of structural contributions of the cladding, only that the current state of timber frame design and analysis does not account for it. Levin (1993) noted that, "Buried in our analysis in the assumption that the frame must stand on its own without any help from the skin." with regard to the finite element analysis he conducted. When engineers and timber frame designers are able to quantify the contribution of cladding elements when determining stresses within individual frame components, more efficient timber frame structures will be possible and it will also be possible to demonstrate by calculations that the designs are code conforming.

2.4 SIP Research

Although SIPs have been utilized since the 1940s, only recently have their value for use in conventional building types been realized. As SIP usage has increased, research has been conducted and articles written that discussed the physical properties, marketing, and manufacturing of these products. Design methods for assessing the capacity of SIPs have been included in several building codes, but none of these considered the capability of SIPs as diaphragms for use with timber frame structures.

2.4.1 Physical Properties and Marketing of SIPs

In order to take advantage of SIPs as diaphragms within the timber frame structural assembly, specific strength and stiffness values for these assemblies must be determined. While there have been some investigations into these assemblies, research performed was proprietary and available only to the companies who performed them for their particular products. It should be pointed out that certain SIPs can be used as structural elements without timber frame skeletons. At the present time there are no published research data available that specifically address timber frames with SIPs as cladding, but there has been some research conducted and articles written regarding SIPs themselves and how they behave under various loading conditions, as well as the market for SIPs and how they are used in modern construction.

Current research on SIPs has focused on the structural and insulating capacity of the panels. Taylor et al. (1997) experimentally evaluated creep behavior of OSB faced SIPs with expanded polystyrene and urethane foam cores as a means of understanding the viscoelastic behavior of SIPs. Experimental results were compared to several finite element models and the authors made recommendations as to the appropriateness of different analysis tools. Pierquet et al. (1998) compared 11 different wall systems, one of which was made of extruded polystyrene SIPs, with regard to thermal performance and embodied energy of each system. The SIP wall maintained a superior long-term energy performance over many of the other systems, but had a higher embodied energy value, due to amount of non-renewable resources that were expended during the manufacture of the rigid insulating foam. Monotonic and cyclic tests were performed by Jamison (1997) on 23, 8 ft (2.4 m) by 8 ft (2.4 m) shear walls constructed from SIPs. This research emphasized the importance of the connections of panels to the bottom plate and the use of tie-down anchors for determining strength and stiffness of SIP building assemblies subjected to lateral loading.

Other important considerations for designers and manufactures of SIPs are how they compare with other building systems, light-frame construction in particular, how they are marketed to builders and consumers, and how to properly detail SIP construction. Gagnon and Adams (1999) conducted a marketing profile of the SIP industry and concluded that although SIPs hold only a small share of the wood products industry, there is potential for considerable growth, especially if more research is done regarding mechanical properties of SIPs. Two authors (Andrews, 1992 and Carusone, 1992) have written on the benefits of SIP construction relative to light-frame construction and both found panels superior with regard to building erection and energy efficiency. Chappell (1990) discussed various aspects of SIPs such as the insulating foams typically utilized, the tighter building envelope created, the recyclibility of SIPs and costs in relation to R-values, and aimed the article toward installers and consumers interested in using SIPs to enclose timber frames. Use of SIP roof systems was addressed by Kondor and Chappell (1996), who made recommendations for construction details specifically for use with SIP roofs in order to avoid moisture infiltration problems. A more recent article by Tracy (2000) discussed the increase in SIP usage in recent years and attributed the increase to shrinking labor markets, increased energy usage awareness and the overall easier constructibility, resulting in fewer call-backs when using SIPs. Also mentioned was the superior performance that buildings constructed using SIPs exemplified during extreme wind, seismic and snow load events.

Articles have also been written in order to clarify the structural and terminology differences among SIPS and other construction methods. Baker (2000) discussed terms related to SIP construction and eliminated some of the ambiguity surrounding SIP nomenclature. Carradine et al. (2001) provided contrasts among the load carrying resistance of conventional, stressed-skin and SIP construction, and related this to the importance of quality control measures with regard to SIPs.

2.4.2 Building Code Provisions for SIPs

As SIP usage has become more prolific, several code writing agencies have included methods for determining the structural capacity of SIPs. The International Building Code (ICC, 2000) and Standard Building Code (SBCCI, 1997) referred to the Plywood Design Specification Supplement 4 – Design & Fabrication of Plywood Sandwich Panels (APA, 1990) as an allowable standard for structural analysis and construction for use with sandwich panels, another commonly used term for SIPs. Supplement Four (APA, 1990) contains formulae for determining shear, bending and buckling stresses within sandwich panels along with general information on fabrication and installation of these products, as well as a design example. The Uniform Building Code (ICBO, 1997) provides the same formulae as Supplement Four (APA, 1990) for the design of sandwich panels. The International Residential Code (ICC, 2000) defines SIPs, but does not provide any recommendations for their design or use. The BOCA National Building Code (BOCA International, 1999) mentions sandwich panels and emphasizes the importance of quality control, as discussed in the following section.

2.4.3 Quality Control Procedures for SIP manufacturing

Due to the manner in which SIPs resist loads, the adhesive bond between the outer skins and the rigid foam core is critical in ensuring the structural integrity of these products (Carradine et al., 2001). It is therefore important for the building designer to specify SIPs that bear the mark of a third party inspection agency. The stamp ensures that the SIP was manufactured in accordance with quality provisions of Plywood Design Specification Supplement 4 – Design & Fabrication of Plywood Sandwich Panels (APA, 1990). The BOCA National Building Code (BOCA International, 1999) Sections 2313.2 and 1704.0 provides guidelines for third party testing and labeling of SIPs and their components in order for these products to be code conforming.

Clearly there exists literature regarding the structural and insulating capacities of SIPs. Unfortunately, there remains a lack of published material with respect to structural assemblies consisting of timber frame skeletons with SIPs as the cladding diaphragm elements. A dearth of applicable building codes that address timber frame and SIP structures also exists, as pointed by Chappell (1985b), who encountered considerable resistance from code officials in Alaska regarding erection of a timber frame house there. Research currently underway at the University of Wyoming, under the direction of Dr. R. J. Schmidt, seeks to quantify the structural capacity of timber frame walls clad with SIPs. Erikson and Schmidt (2001) discussed the lateral loading of five single bay, single story frames, which provided stiffness values for frames of different wood species. Erikson and Schmidt (2002a) also conducted lateral load testing on four two bay, two story frames and determined that, "an unsheathed wood-pegged timber frame may experience unacceptable deformations due to lateral drift". Most recently, Erikson and Schmidt (2002b) conducted lateral load testing on four timber frame walls with SIPs attached in an effort to characterize the behavior of the shear walls. They concluded that installation of SIPs brought the wall stiffness within required limits for safety and serviceability. Additionally, they recommended attaching SIPs to timber frame members wherever possible and noted that even walls with large (6 ft (1.83 m) wide by 7 ft (2.13 m) high) openings maintained acceptable stiffness levels. This research provided information that can be utilized in conjunction with test data on roof stiffness in order to determine building stiffness and further indicates a need to quantify the contribution made by SIPs in resisting lateral loads in timber frame structures.

2.5 Diaphragm Action

Structural diaphragms are building assemblies that act as large, deep beams within the context of overall building behavior. Examples of structural diaphragms include metal sheeting roof and wall systems, composite steel and concrete floor systems and wooden sheathing over light-frame wall and floor systems. In a typical metal sheeting and wood frame roof system, the purlins and edge members act as beam flanges and carry the axial bending forces while the metal sheeting acts as the beam web and carries shear forces (Davies and Bryan, 1982). Although diaphragm action is utilized in numerous structural applications, this literature review is limited to diaphragms as they pertain to post-frame buildings with metal cladding, as this literature is most useful to apply to the case of timber frame buildings with SIP cladding.

2.6 Post-Frame Structural Design Utilizing Diaphragm Action

A post-frame building consists of a series of frames made up of trusses that are commonly assumed to be pinned to the tops of two wall columns. The columns are embedded in the ground or attached to a concrete slab. Wall girts span between posts and purlins span between upper chords of the roof trusses. Metal cladding is typically attached to girts and purlins to create wall and roof diaphragms, respectively as depicted in Figure 1.4. These diaphragms not only serve to enclose the structure, but contribute significantly to the stiffness and strength of the structure.

Extensive research has been conducted in order to quantify the contribution to building stiffness and strength that roof and end wall diaphragms provided within post-frame structures. This body of research included work regarding the nature of diaphragm action and strength in post-frame buildings, studies of the parameters that affect diaphragm strength of these structures and tests of full-scale buildings. Analysis and design procedures that include diaphragm action within post-frame buildings have also been the subject of numerous research projects and literature.

Post-frame buildings employ diaphragm action to resist lateral loads in that the roof system acts as deep beam that is supported on the ends by the end wall assemblies, which transfer roof loads to the ground (Anderson and Pohl, 1990). Utilizing diaphragm action in order to eliminate the need for knee braces and uneconomically large members in post-frame buildings was initially limited by a lack of experimental data on the strength and stiffness of these types of building assemblies (Hausmann and Esmay, 1977). This dearth of information prompted researchers to analyze differences in construction methods that would result in more effective and efficient post-frame structures.

2.6.1 Post-Frame Panel Assembly Testing

The majority of the research performed on post-frame diaphragms was done using test panels and aimed at quantifying the contributions made by various components of the diaphragm panel assemblies. An early study on 26 test panels of various sizes and configurations by Hausmann and Esmay (1977) concluded that, "Diaphragm strength varies appreciably with the framing system, type and number of fasteners, purlin and girt spacing, cover width of the metal panel, length, width and eave height of the building, and door openings.". Massé et al. (1983) tested four diaphragms, which were intended to simulate ceiling diaphragms, and found that although blocked diaphragms were stiffer than stitch-screwed diaphragms, it was thought to be more practical to add more screws than to require the labor intensive job of blocking. Four test panels were tested by Turnbull et al. (1986), who reported that the best diaphragm performance was obtained with flat roof purlins and extra stitch-screws (more than were specified by manufacturers at that time) along the roofing side-laps. Screw size and fastener group effects were investigated by Massé and Munroe (1989). After testing five small-scale (single fastener) specimens and two full-scale test panels, they concluded that screw size [0.165 in. (4.2 mm) or 0.189 in. (4.8 mm)] had nearly no effect on the load-deformation characteristics of the connections, multiple fasteners did not induce group action effects and that small scale tests could be used to adequately determine screw strength and stiffness values. Efforts to determine strength and stiffness values for 16 different panel configurations by Massé and Salinas (1990) led to testing of various metal profiles and thicknesses, and different fastener spacings. They recommended using more fasteners to increase panel stiffness and using thicker metal if higher panel strength was desired. Anderson and Bundy (1990) pointed out that diaphragm action could only be used for design purposes when the strength and stiffness values of the diaphragms were known, and that typically testing must be performed to obtain these values. In an effort to avoid the need to test every possible diaphragm assembly an engineer may want to utilize, they tested 15 panels to failure and varied purlin and rafter spacing, fastener patterns and the percentage of sheathing on the panels (67%-100%). Results indicated that the number and type of fasteners had the greatest effect on diaphragm stiffness and that fasteners placed near the edges of the metal sheets influenced diaphragm stiffness more than those in the middle of the sheets. Anderson and Bundy (1992a and 1992b) also made recommendations regarding appropriate testing methods and interpretation of those tests for determining diaphragm assembly strength and stiffness. While research on test panels previously discussed helped to identify the factors that must be addressed in order to effectively design and build adequate diaphragms, researchers were still concerned about how these panel assemblies behaved when they were acting as a sub-assembly of a completed building.

2.6.2 Full-Scale Post-Frame Testing

Testing has been done on full-scale buildings (Milner et al., 1998). Also, several researchers have performed full-scale research on post-frame diaphragm action, the most pertinent of which was summarized by Gebremedhin et al. (1992b). This summary included comparisons among the experimental work performed by Johnston and Curtis (1984), McFadden et al. (1991) and Gebremedhin et al. (1991). Johnston and Curtis (1984) concluded from their study that knee braces were ineffective in resisting horizontal loads, screws performed better than nails for use with roof diaphragms and endwall stiffness was fundamental when determining eave deflections. Work by McFadden et al. (1991) also indicated that endwall stiffness was an important consideration, as well as denoting the non-conservative assumption that the endwalls were rigid. In addition, McFadden et al. (1991) found that post movement at the floor level and purlin slip were negligible. All of the summarized research verified that the use of roof diaphragms significantly increased lateral force resistance capabilities of the structures under scrutiny. The investigation performed by Gebremedhin et al. (1991) was discussed in Gebremedhin et al. (1992b) and has led to numerous other research articles, which are discussed below.

The most prolific body of research regarding full-scale testing of post-frame structures utilizing diaphragm action has been done at Cornell University by Dr. Kifle

Gebremedhin, working in conjunction with numerous colleagues and graduate students. The thrust of this literature was based on the testing of a 40 ft (12.2 m) by 80 ft (24.4 m) post-frame building with an eave height of 16.25 ft (4.95 m) and an 8 ft (2.44 m) frame spacing, constructed in a manner characteristic of typical post-frame buildings by an experienced builder (Gebremedhin et al., 1991). Concentrated loads of equal magnitude were applied to each of the interior post-frames utilizing an electronically controlled series of hydraulic actuators in order to simulate horizontal wind loads on the structure at various stages of construction. These construction stages consisted of exclusively the wood framing with no steel sheathing, steel sheathing attached to the endwalls only, steel sheathing attached to the endwalls and sidewalls, steel sheathing attached to all walls and one side of the roof, steel sheathing attached to all walls and both sides of the roof with the ridge cap also fastened and then with the removal of the ridge cap fasteners on one side of the roof. Gebremedhin et al. (1991) cited several objectives for this study, the most relevant being the contribution of diaphragm action to the lateral deflection resistance of the structure. Figure 2.1 shows distinctly the reduction in eave deflection in the building progressing from only the wood framing as compared to the building with all of the walls and roof sheathing attached. Deflections were reduced from 6.1 in. (155 mm) to 0.44 in. (11.2 mm), a 93% reduction (Gebremedhin et al., 1992b). These findings have allowed designers to use smaller columns and shorter column embedment depths, both of which resulted in less expensive buildings.

This initial experimentation has been the subject of a considerable amount of literature in this field. Gebremedhin et al. (1992c) elaborated on the findings of the full-scale testing by addressing the design methods at that time (ASAE EP 484.1, 1990) and elucidating errors therein regarding endwall rigidity and transfer of shear from one side of the roof to another. In 1993, Gebremedhin and Jorgensen performed additional tests on the fully clad post-frame structure which focused on predicting endwall stiffness based on different sized door openings and various endwall reinforcement methods. They used the same loading procedures as Gebremedhin et al. (1991) but altered the endwalls. These alterations included a door that was 25% of the area of the endwall, a door that was 50% of the area of the endwall, metal strapping as reinforcement, endwall stitching as reinforcement and 15/32 in. (11.9 mm) thick plywood sheathing as reinforcement. The

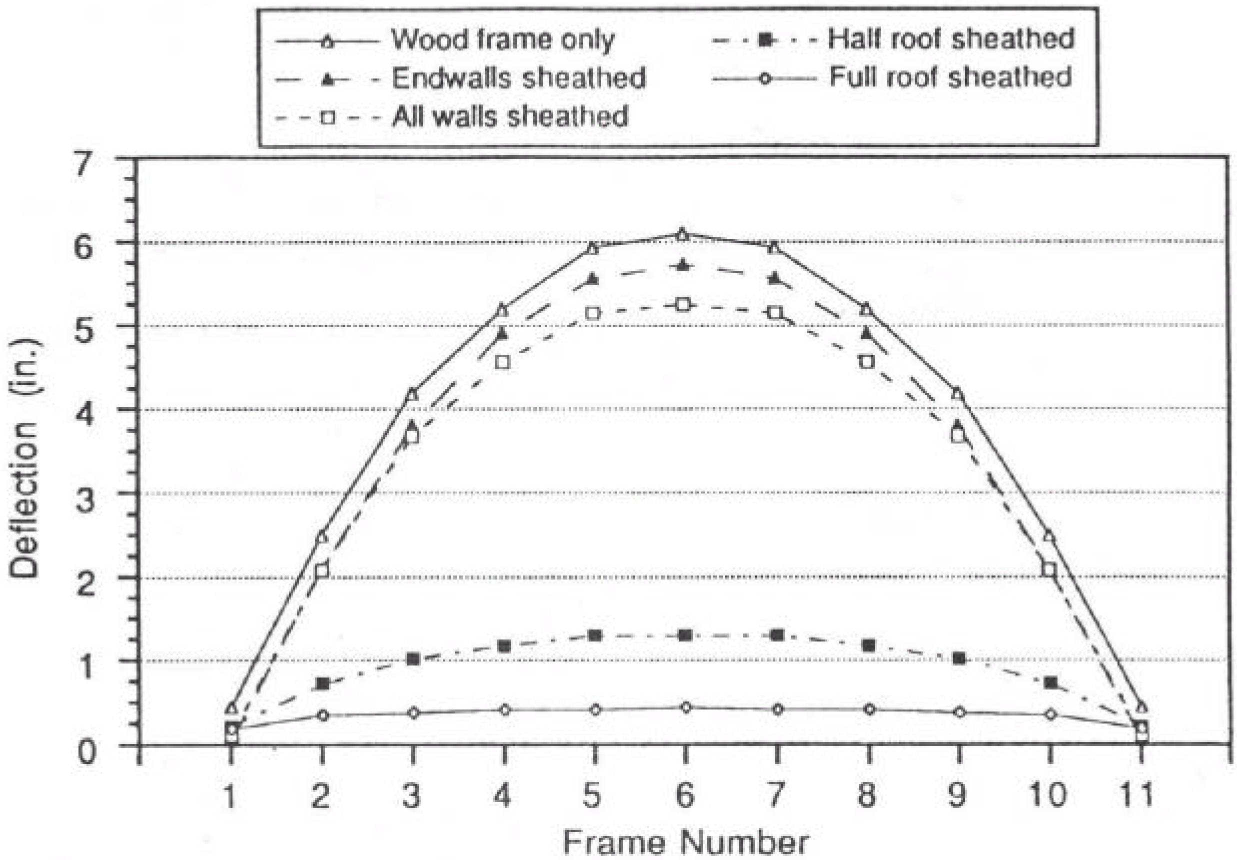

Figure 2.1. Eave deflections at 425 lb (1.89 kN) Load for all test configurations from Gebremedhin et al. (1992b), clearly indicating the reduction in deflections with the addition of metal sheathing. (Used with permission.)

authors concluded that while openings in endwalls significantly reduced stiffness, the reduction could be overcome using metal strapping or plywood sheathing. The full-scale post-frame building was tested again by Niu and Gebremedhin (1997a), who also conducted panel tests as a means of developing the instrumentation needed to measure roof sheathing and purlin strains. Additionally, they confirmed that stiffness behavior was directly affected by the number of overlap seam connectors. Interior purlin contributions should be considered when determining chord forces in edge purlins, and the two roof halves acted more as a single beam rather than a pair of beams. Based on previous panel assembly and full-scale experiments, Gebremedhin (1998) sought to compile a set of parameters affecting stiffness and strength of diaphragms in post-frame buildings. Roof stiffness was found to have the most significant effect on building stiffness for post-frame buildings, with endwall stiffness also playing an important role. Recommendations were also made in this article as to how to increase diaphragm stiffness using various fastener types and installation methods; in particular a stiffer diaphragm resulted when more fasteners were used. A more recent paper (Gebremedhin and Price, 1999) summarized previous research on full-scale testing of post-frame structures since the initial fabrication and testing of the building and concluded that repeated tests simulating long-term wind exposure had little effect on eave deflections of the loaded structure.

While it is not practical to perform full-scale structural tests for all building types, the Cornell tests provided a great deal of information about the contribution that diaphragm action makes towards the wind resisting capabilities of post-frame buildings.

2.6.3 Analysis of Post-Frame Buildings

Experimental research that has been performed to investigate diaphragm action as a means of bracing post-frame structures against lateral loads has made a great contribution to designers and engineers who work with these buildings. A great deal of work has also been done to refine methods of analysis that take into account the contribution of diaphragm action. Analyzing post-frame structures to include diaphragm action was a difficult task because the interaction between metal cladding and wood framing created an inherently complex set of relationships among distinctly differently behaving materials and fasteners. This resulted in researchers having to consider various aspects of post-frame structures in order to fully understand how to effectively analyze them. While research has been conducted utilizing numerical methods as well as computer modeling, nearly all of the analysis methods discussed below include some use of a computer.

While diaphragm action was difficult to quantify in post-frame buildings, the frames making up the structural assembly tended to be simple enough to apply basic numerical methods in order to obtain an estimate of their behavior under various loading conditions. Load sharing between wooden frames and metal diaphragms was addressed by Anderson et al. (1989), who derived a procedure using the force distribution method of structural analysis that could be solved using a computer and avoided the need to solve simultaneous equations by hand. Anderson and Bundy (1989) investigated definitions and applications of shear stiffness in roof diaphragms by performing several numerical analyses. Results of the study indicated the importance of considering panel length, slip among panels and independence of the two roof halves when analyzing post-frame structures. Bohnhoff (1992a) presented equations for frame stiffness and eave load estimations, and analyzed 27 frames to assess the validity of assuming infinitely stiff trusses for use with post-frame building analysis. Results illustrated that post connection rigidity was more critical than truss stiffness and additional equations were presented for determination of post shear and bending moments, and maximum shear load in roof diaphragms. In 1989, Boone and Manbeck modified an analytical model developed by Davies and Bryan (1982) for use with metal-framed diaphragms so that it could be used with wood-framed diaphragms. When compared to panel test data, the derived model was found to be inadequate for analysis purposes. Wright and Manbeck (1992) supported these conclusions based on an extensive literature review and recommended a comprehensive finite element model as a more accurate means of analyzing metal on wood-frame diaphragms.

In conjunction with a plane structures analyzer program, such as PPSA4 (Triche and Suddarth, 1993), analytical studies have provided valuable information on the salient issues surrounding diaphragm action within post-frame buildings. Gebremedhin and Woeste (1986) investigated the effects of diaphragm action, knee braces, knee brace slip, and building length and width using various combinations of numerical and computer analyses. This study elucidated the effectiveness of diaphragm action, and the relative ineffectiveness of knee braces for reducing post forces in post-frame buildings subject to lateral loads.

Several researchers have pursued the creation of computer programs and models that incorporate diaphragm action within the structural analysis. A BASIC computer program was formulated by McGuire (1991) that provided roof shears and lateral deflections using minimal input from the user. The program used springs to represent the diaphragms, as many other models do, but required verification using other programs to ensure accuracy. The computer program METCLAD (Gebremedhin, 1988) was developed for use by post-frame design engineers and for educational purposes, depending on what type of output the user needed. METCLAD utilized diaphragm action and provided results including structural responses of the building, member forces, moments and deflections, node displacements, support reactions, shear analyses, combined stress interaction analyses and locations of maximum stresses within members (Gebremedhin, 1988). Bohnhoff (1992b) constructed a computer program called DAFI (Diaphragm and Frame Interaction) for purposes of determining load distribution to the diaphragms and individual frames within post-frame structures. By inputting horizontal eave loads, and each diaphragm and frame stiffness, loads carried by various components were determined. This program was based on the design practices at the time of creation of the program.

Another technique for analyzing post-frame structures has been the creation of models using finite element analysis. Wright and Manbeck (1993) performed a finite element analysis using connection data from tests they conducted. The resulting model adequately predicted test panel stiffness and strength and provided further insight into the force-displacement responses of test panels, especially with regard to purlin continuity, chord forces and nonlinear connector behavior. Due to the time and cost of implementing the previously mentioned finite element model, Keener and Manbeck (1996) went on to create a simpler finite element model that represented a two-dimensional model of the test panel configurations under investigation. Results indicated that the simplified model predicted test panel behavior adequately at low loads (loads less than 5% of the ultimate panel load), but the lack of three-dimensional modeling provided inaccurate output at higher load levels. Niu and Gebremedhin (1997b) also developed a finite element model for purposes of extrapolating test panel data to full-scale structural behavior, predicting building eave displacements and determining upper-bound post-frame building strength. Resulting model analyses were compared to full-scale building experiments (Gebremedhin et al., 1991), with eave deflection prediction error ranging from 2% to 17% from the full-scale building tests.

Researchers developing analysis techniques for wood-framed, metal-clad diaphragms serving to resist lateral loads in post-frame structures have considered everything from single fastener connections up to full-scale building experiments. Numerical models have sought to incorporate this body of research into methodologies for quickly and accurately determining design parameters necessary for components of a post-frame building. From this myriad of analyses, design techniques were developed that allowed for simple and efficient design procedures, which are discussed below.

2.6.4 Design Procedures for Post-Frame Buildings

Sophisticated methods for analyzing post-frame buildings have led to the development of design methods using the large body of experimental and analytical research. Numerous components and systems comprise a post-frame building, therefore different authors have chosen various aspects of design to concentrate their efforts on with regard to the mechanisms involved in diaphragm action.

Design typically consisted of specifying components and construction practices. Components, such as fasteners or wooden posts and girts, were specified as to size, type or species and location. Construction practices described the exact manner in which components were to be assembled in order to create safe and effective structures. Research on the diaphragm design of post-frame buildings has addressed both of these design specifications. Turnbull (1983) derived some simple equations for determining shear forces and bending moments based on wind forces, building dimensions, and type of diaphragm sheathing material. In order to obtain smaller post sizes, Hoagland and Bundy (1983) created a design procedure using a two-dimensional frame analysis computer program that required the in-plane strength and stiffness data for the roof panels. Gebremedhin at al. (1986) simplified the Hoagland and Bundy (1983) procedure by creating the roof diaphragm lateral restraining force modifier (mD) and roofing shear force modifier (mS) tables for buildings with up to 30 frames. Versions of these tables continue to be used for the design of post-frame structures according to ASAE EP484.2 standards (ASAE, 1999).

In 1990, the American Society of Agricultural Engineers (ASAE) published a document that formalized the previously discussed research into a set of specific procedures for the design of metal-clad post-frame rectangular buildings. ASAE EP484 (ASAE, 1990) was summarized by Manbeck (1990), who also discussed the development of the document and provided suggestions on future directions for revising it. Revisions were made quickly and in 1991, ASAE EP484.1 (ASAE, 1991) was published. Gebremedhin (1992) reported that resulting eave deflections from full-scale testing of post-frame structures were nearly identical to those found using ASAE EP484.1 (ASAE, 1991). Gebremedhin and Manbeck (1992) discussed the procedures and expressed their confidence in them as a sound example of engineering practice.

Introduction of ASAE EP484.1 (ASAE, 1991) did not result in a cessation of research regarding the design and behavior of post-frame buildings. To the contrary, researchers continued to investigate problems surrounding diaphragm action in these buildings. Some issues considered were post-frame end walls, eave deflections, post embedment and roof designs. Wirt et al. (1992) investigated effects of openings in end walls and derived procedures for the design of corner post foundations and endwalls with large door openings. A process was created by McGuire (1998) for conservative determination of eave deflections in post-frame buildings using a single equation. Jackson et al. (1989) utilized post-frame diaphragm action to develop a set of steps for determining adequate post embedment using computerized structural methods. Regarding roof design, Bender et al. (1991) developed a simplified, yet accurate series of hand calculations based on ASAE EP484.1 (ASAE, 1991) and the assumption that the roof diaphragm acted as an infinitely stiff, deep beam. Additionally, Pollock et al. (1996) quantified the chord forces within a post-frame roof diaphragm and described the contribution of the purlins as components of the roof system.

ASAE EP484.1 has been considered a work in progress. While it was widely accepted as the official standard by which the design of post-frame structures were conducted, it has been the subject of continued discussion and refining. Anderson (1997) presented some of the topics of contention and discussed how they were to become part of the newest standard, ASAE EP484.2 (ASAE, 1999). This newest set of revisions is the most up-to-date document on the design of post-frame structures utilizing diaphragm action. The National Frame Builders Association (NFBA) published a Post-Frame Building Design Manual (NFBA, 2000) that was based on the same techniques found in ASAE EP484.2 (ASAE, 1999).

Clearly the research that has been performed over the past several decades regarding the contribution of diaphragm action to the analysis and design of post-frame structures is extensive. This work has granted designers, engineers and contractors the tools and techniques needed to analyze, design and build safer and more efficient structures. It is the intention of this research to contribute similarly to designers and builders of timber frame structures utilizing SIPs as the primary diaphragm elements.

2.7 Timber Frame Structural Design Utilizing Diaphragm Action

As mentioned previously, there are currently no engineering or building standards for utilizing diaphragm action when designing timber frame structures with SIPs. Benson has written about the installation of SIPs on timber frames (1992) and elaborated on how to effectively use SIPs when designing and building a timber frame structure (1997), but does not provide any means for including diaphragm action. Several papers discussed previously were taken from a compilation of articles (Christian, 1997) that provided a good deal of information about the design and history of timber frame structures, but also failed to include any design methodologies that included diaphragm contributions in the structural computations. Carradine et al. (2000) utilized methods for designing post-frame buildings to analyze a typical timber frame structure. While certain assumptions had to be made due to lack of test results for SIP and timber frame roof and end wall assemblies, the authors concluded that the post-frame procedures would be appropriate for use with timber frame structures, if the necessary test data were available.

2.8 Conclusions

Considering the dearth of research on timber frame structural design that accounts for the contribution of diaphragm action of the SIPs used to clad them, there is no question that more work needs to be done in this area. Specifically, tests need to be performed on diaphragm assemblies of timber framing and SIPs so that timber frame buildings can capitalize on the gains from the decades of work done on post-frame buildings. If the strength of the SIPs were included in the design of timber frame buildings, the potential is there for more architects and engineers, as well as building officials, to embrace this type of building. The structural benefits of diaphragm action may make it possible to reduce the sizes of timbers and minimize the use of steel reinforcement within these structures while preserving the aesthetic value that traditional timber frame buildings are famous for.