Analysis and Testing of a Ready-to-Assemble Wood Framing System

by

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(ABSTRACT)

The concept of a ready-to-assemble kit fabricated in a factory and delivered to the customer is well known and commonly used by the furniture industry. In wood construction, the lack of a simple and reliable method of assembling the frame members creates a barrier to wide acceptance of prefabricated kit structures. This thesis focuses on a novel technology of assembling structural components of a wood frame using a metal nail plate connector (NPC). This technology was referred to as a ready-to-assemble (RTA) wood framing system. The RTA system simplifies the framing process and allows for rapid erection of a wood structural frame by a small nonprofessional crew.

A 16 x 24 foot RTA building was constructed to demonstrate the feasibility of the RTA system concept. An effective assembly sequence was proposed and successfully implemented.

The design procedure for the RTA buildings was presented. The lateral load path for the RTA building includes diaphragms and shear walls. The contribution of the RTA frame can be ignored from the lateral load analysis. This conclusion was validated for the diaphragms with aspect ratios up to 4:1. The finite element method was used to model the RTA structures. The models incorporated semi-rigid behavior of the NPC.

An analytical model was developed to predict the nonlinear moment-rotation relationship of the NPC. The proposed model showed a good agreement ($R^2=0.98$) with the experimental data.

Tests were conducted to measure the load-drift response of the RTA shear walls.
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Chapter 1

Introduction

1.1 Problem Overview

The concept of a ready-to-assemble kit fabricated in a factory and delivered to the customer is well known and commonly used by the furniture industry. In wood construction, the lack of a simple and reliable method of assembling the frame members creates a barrier to wide acceptance of the prefabricated kit structures. Traditional wood connections: pegs, mortise and tenon, bolts, nails, screws, metal truss plates, and others require professional skills and experience for construction of structurally sound joints. Moreover, the fabrication of these connections can involve time consuming operations and special tools and equipment.

This thesis focuses on a novel technology of assembling structural components of a wood frame using a metal nail plate connector (NPC). This technology was introduced by Platt (1998) and referred to as a ready-to-assemble (RTA) wood construction system. The RTA system was an extension of work done by Piskunov (1995). The RTA system simplifies the framing process and allows for rapid erection of a wood structural frame by a small nonprofessional crew. Although a variety of structures can be constructed using the RTA technology and the NPC, this project mainly focuses on development of small structures for the do-it-yourself market. These structures include residential utility buildings such as detached home offices, garages, workshops, storage, and are generally less than 16 foot clear span.

The RTA system incorporates a set of technological steps: fabrication of the NPC, fabrication of the RTA frame members, packaging of the RTA kit, transportation of the package to the construction site, assembly and bracing of the RTA frame (Figure 1). The effectiveness and efficiency of each individual step are essential to the overall success of the RTA system. The first three steps are performed in a controlled factory environment and can be optimized and automated for mass production and packaging of the RTA kits. The frame
members can be stacked into a solid package to increase transportation savings for shipments to remote locations. The on-site construction of the structural frame is simplified and accelerated through using the prefabricated ready-to-assemble frame members.

![Diagram](image)

**Figure 1. The RTA technology.**

The NPC (Figure 2) is integral to the RTA framing system (Platt 1998). The NPC replaces the traditional wood-to-wood or wood-to-metal connections with simple and versatile bolted metal-to-metal connections that are easily assembled on site using bolts, nuts, and wrenches.
The NPC allows for creating the moment resistant connections that eliminate the need for extensive temporarily bracing of the frame during construction. Moreover, these connections produce a wood frame which is readily dismantled for relocation to another site.

Throughout this thesis the RTA framing system is compared with the light-frame and post-frame wood construction systems that are referred to as the “traditional” systems. The RTA system is positioned relative to these traditional systems in respect to the construction practices and design methodologies.

The public adoption of the RTA framing system will require a comprehensive design procedure for structural analysis of the RTA buildings. The design procedure should incorporate methods for analysis of both vertical and lateral load resistance. The RTA buildings include a number of attributes that distinguish them from the traditional timber framed structures. Therefore, either the effect of these attributes should be addressed by the design procedure, or one should show that their contribution does not change the load path and the failure modes when compared to the structures constructed using the traditional methods. In the latter case, the design procedures for the traditional wood construction systems are directly applicable.

A RTA building consists of a series of frames connected with girts and purlins and braced with structural sheathing panels. A variety of frame spacings is possible; the RTA buildings discussed in this thesis use 4 foot on-center spacing. The primary function of
the frames is to carry gravity loads into the foundation. Because the RTA frames are assembled with the moment resistant connections, the NPCs are the subject to moment forces. The design of moment resistant connections is complicated by the lack of an analytical methodology for predicting the rotational strength and stiffness of the multiple dowel wood connections. Moreover, structural analysis of the frames with moment resistant connections should incorporate the rotational stiffnesses of these connections in order to accurately capture the force distribution between and within frame members and connections.

Lateral resistance of a RTA building is provided by diaphragms and shear walls. The RTA shear wall consists of columns spaced 48 inches on-center connected with girts spaced 24 inches on-center. The RTA diaphragm consists of rafters spaced 48 inches on-center connected with purlins spaced 24 inches on-center. Shear walls and diaphragm are constructed with plywood or oriented strand board (OSB) sheathing panels. This type of construction is different from the traditional construction practices. Light-frame construction uses studs and rafters or trusses spaced 16 or 24 inches on-center. In contrast, a typical post-frame building consists of a series of frames spaced 96 inch on-center and braced with metal cladding. Therefore, the experimental data available on the traditional shear walls and diaphragms is not directly applicable to the RTA structures. Because the strength and stiffness of shear walls and diaphragms are necessary input for lateral design procedures, the resistance of the RTA shear walls and diaphragms should be measured.

Lateral design methods for the light-frame and post-frame construction systems are well developed and proven satisfactory. Applicability of these design procedures for the RTA structures is a subject of this investigation. To answer this question, the contribution of the frames to the total lateral resistance of the RTA buildings should be determined. Post-frame design procedures can be used to establish the lateral load path for the RTA buildings.

Using the Innovation Development Decision Model, Platt (1998) showed that the RTA framing system can be successful. He also conducted a cost analysis which demonstrated that the RTA system can be economical. Both studies were based on a series of theoretical assumptions. Therefore, a feasibility study was needed to further demonstrate
the viability of the RTA framing system concept. This feasibility study should include construction of a full-size building using the RTA technology.

1.2 Objectives

This study is a part of a research project titled “Development of a Ready-to-Assemble Construction System – Phase II” sponsored by the United States Department of Agriculture Small Business Innovation and Research Program. The objective of the Phase II project is to introduce the RTA system into the commercial market.

Research presented in this thesis contributes to satisfying the engineering part of the overall objective of the Phase II project. Specific thesis objectives are:

1) conduct a feasibility study that includes construction of a full-size building using the RTA framing technology. Sub-objective of this feasibility study is to develop an effective assembly sequence.

2) present a design procedure for structural analysis of the RTA buildings. The design procedure should incorporate methods of analysis for both vertical and lateral load resistance. The sub-objectives necessary to meet objective 2 are:

2.1) analyze vertical and lateral load resistance of the RTA frames. The analysis should incorporate moment resistant semi-rigid behavior of the NPC. The lateral frame stiffness should be computed for use with sub-objective 2.8;

2.2) derive a model for predicting the load-deformation relationship of the NPC under moment loading. The model can be used to design a variety of the NPCs and analyze the RTA frames (sub-objective 2.1);

2.3) conduct a series of tests to quantify the moment resistance of the NPC. The experimental data is used to validate the results of analytical modeling from sub-objective 2.2;

2.4) conduct a series of tests to measure the bearing strength and stiffness of the plate on the end grain of the RTA frame member. Results of this testing are used as input for sub-objective 2.2;
2.5) model the lateral resistance of the RTA shear walls. Results of the modeling are used with sub-objective 2.8;

2.6) conduct a series of full-scale tests to measure the monotonic response of the RTA shear walls. Results are used to design the RTA buildings, validate the modeling from sub-objective 2.5, and perform analysis from sub-objective 2.8;

2.7) conduct a series of tests to measure the load-deformation relationship of the fasteners used to attach sheathing to the RTA frame. Results of this testing are used as input for sub-objective 2.5;

2.8) define the lateral load path for the RTA structures.

1.3 Scope

This section is intended to narrow the scope of the proposed objectives. The section is presented in two parts to describe each objective, respectively.

1) a 16 x 24 foot building was constructed. A set of instructions for the assembly of the building was devised including bracing recommendations. Each step of the fabrication and erection of the building was documented and recommendations for improvements were specified;

2) the design procedure was demonstrated by the analysis of a 16 x 24 foot RTA building;

2.1) the RTA frames were modeled using SAP2000 Structural Analysis Program v. 7.10. (Computers and Structures, Inc., 1999). The modeling was limited to linear performance. The model was used to determine the forces in the frame members and connections. The sensitivity of the frame model to the system variables was quantified. The model was also used to compute the frame stiffness under lateral loading;

2.2) an analytical model for predicting the moment-rotation relationship of the NPC was developed using the energy conservation principle and a
closed form solution was derived. The model covers linear and non-linear performance. The model requires input of the plate bearing on the end grain of the frame member and the lateral load-slip relationship for a single nail. The model was validated using the experimental data of the NPC in TimberStrand® subjected to moment loading;

2.3) The experiments on the evaluation of the moment resistance of the NPC involved testing specimens made of solid sawn Spruce-Pine-Fir (SPF) lumber, Parallam®¹, and TimberStrand®². The sample size of fifteen replications for each group was based on the statistical analysis of the data acquired during the Phase I project (Platt 1998) and calculated using Equation [1].

\[
n = \frac{Z^{2} \sigma^{2}}{E^{2}} = \frac{1.96^{2} \sigma^{2}}{0.5 \sigma^{2}} = 15.3
\]  

[1]

where:

\[ n = \text{sample size}; \]
\[ \alpha = 0.05 \text{ significance level for 95\% confidence interval}; \]
\[ \sigma = \text{population variance}; \]
\[ E = 0.5\sigma \text{ – half interval width of } \sigma; \text{ according to the previous data it will provide a confidence interval width of (5-10)\% the mean value of the estimated parameter}; \]
\[ Z_{(1-\alpha/2)} = 1.96 \text{ standard normal variate}. \]

This sample size was also in compliance with the provisions of the ASTM standard 1761 “Standard Test Methods for Mechanical

¹ Parallam® is a registered trademark used by Trus Joist MacMillan
Fasteners in Wood” (ASTM 1998), which required a minimum sample size of ten specimens. Forty five specimens were tested;

2.4) the load-deformation relationship for the plate bearing on the end grain of the frame member was measured for TimberStrand®. The sample size of 10 was adopted from the ASTM standard D 1761 (ASTM 1998).

2.5) linear and non-linear analyses of the RTA shear wall were performed using SAP2000 Structural Analysis Program Results were validated with the experimental data and the contribution of the moment resistant connections to the total lateral resistance of the RTA shear wall was estimated;

2.6) three RTA shear walls with the girts recessed flush against the frame members and three RTA shear walls with the girts on top of the frame members were tested. A total of six full-scale tests were conducted. The sample size of three was adopted from the ASTM standard E564 “Standard Method of Static Load Test for Shear Resistance of Framed Walls for Buildings” (ASTM 1990);

2.7) a series of tests to measure the load-slip relationship of the screws used in the RTA shear walls was conducted. Based on the siding thickness, grain orientation of the frame member, and frame members material the specimens were organized into twelve groups. The sample size of 10 was adopted from the ASTM standard D 1761 (ASTM 1998). A total of 120 specimens was tested;

2.8) the contribution of the frame to the lateral resistance of the RTA building was estimated using the post-frame design procedure as outlined in ASAE EP484.2 standard “Diaphragm Design of Metal-Clad, Post-Frame Rectangular Buildings” (ASAE Standards 1999). Based on the findings, recommendations were given regarding the

\[\text{TimberStrand® is a registered trademark used by Trus Joist MacMillan}\]
lateral design of the RTA building. The analysis was based on a series of conservative assumptions and data derived from the shear wall tests. Diaphragm testing was beyond the scope of this project.
1.4 Definitions

This section is intended to clarify the terminology used throughout the thesis.

Bent. A transverse two-dimensional frame assembly.

Frame. One or a series of bents.

Girt. A horizontal element that connects the frames and provides nailing surface for the wall sheathing panels. A girt is a part of wall assembly.

Nail plate connector or NPC. A metal plate with a set of double pointed nails. (Figure 2).

Purlin. A horizontal element that connects the frames and provides nailing surface for the roof sheathing panels. A purlin is a part of roof assembly.

Rabbet. A groove on a face of a piece of wood that allows for accommodation of the NPC.

RTA building. A completed building constructed using the RTA technology.

RTA diaphragm. A structural roof assembly constructed using the RTA technology.

RTA frame member. A built-up member comprised of two or more wood components connected face-to-face with the NPC.

RTA framing system. A construction system used to assemble the prefabricated frame members into the final structure.

RTA shear wall. A structural wall assembly constructed using the RTA technology.

RTA system. Same as RTA framing system.

RTA technology. A set technological steps that may involve fabrication of the RTA kit and construction of the RTA building, and all other supplementary practices associated with them.
Chapter 2

Background

2.1 General

Both the buildability and structural behavior of wood buildings are governed by the performance of the connections. Connections in wood structures are the subject of continuous research and development. Because metal dowel connections are the most common type of connections used in the residential construction, there is a voluminous body of experimental and analytical knowledge on the behavior of these connections and their effect on the response of the structures. Some work in the field of dowel connections in wood structures, which is directly applicable to the subject matter, is highlighted in this chapter.

Most wood buildings in North America are constructed using either light-frame or post-frame constriction systems. A significant amount of work was done by researchers to quantify and model the response of structures built using these construction systems. Moreover, detailed structural design methods were developed and standardized. Both construction systems are briefly presented in this chapter. Applicability of the established design methods is discussed with respect to the structures built using the RTA technology.

The chapter starts with the history of the development of the RTA technology and NPC. Then, single and multiple dowel connections are discussed and the NPC is compared to traditional dowel connections. Next, the analytical and experimental methods used to measure performance of moment resistant connections are presented. The design methods for structures with the moment resistant connections are examined. The discussion of wood construction systems concludes this chapter.

2.2 Development of a ready-to-assemble wood framing system

Platt (1998) introduced the RTA framing system. He implemented an Innovation Development Decision Model to the development of the RTA framing system concept.
Using the model he showed that the performance of the individual NPCs and the RTA structures are critical to the acceptance of the RTA framing system. He further demonstrated the need for a design procedure capable of analyzing a variety of the RTA structures. He also compared the cost effectiveness of the RTA framing system relative to the conventional construction methods. Based on the results of the cost analysis, he concluded the economical competitiveness of the RTA system.

Platt tested the individual NPCs in shear parallel- and perpendicular-to-the-grain in solid wood of SPF and wood-based composites TimberStrand®, Parallam®, and Laminated Veneer Lumber (LVL). The yield theory was used to predict the strength of the NPC. Experimental data fell within ± 6.3 % of the connector strength predicted by the yield theory.

### 2.3 Development of the nail plate connector

The work done by Platt (1998) was an extension of a long-term project initiated in the Center for Research Engineering Material Building Structures (CREMBS) at Kirov Polytechnic Institute in Kirov, Russia in the 1970’s. The objective of this project was to improve the utilization of Russian wood resources in commercial and industrial construction. Analysis of the softwood timber resources available in the Western part of Russia showed that the major barrier for wide acceptance of wood structures was the lack of big diameter timbers. Available timber could not be economically manufactured into the lumber with the dimensions sufficient to carry the design loads. The result of the work done by the CREMBS was the NPC designed to mechanically laminate the lumber with small cross sections into the built-up elements with the increased moment of inertia. Furthermore, the NPC was used to simplify and accelerate the on-site assembly of wood structures. The concept was introduced, well developed, and successfully implemented in the commercial and industrial construction.

Piskunov (1993) established a criterion that can be used to evaluate performance of a connector:
1) functional criteria; this criteria evaluates the resistance of the connector under different failure modes. This criteria defines versatility of the connector and becomes important for ready-to-assemble systems, which are governed by the joint simplicity and uniformity;

2) constructive criteria; the connector strength and stiffness and its interaction with the connected members govern this criteria;

3) technological criteria; this criteria determines the labor and cost required to build a structure using the connector. This criteria covers the fabrication process from manufacturing of the connector through the construction and in-service expenses for maintaining the structure;

4) special features; the resistance of the connector to the special in-service conditions such as high humidity, high or low temperature, high concentration of chemicals, fire hazard, etc.

Piskunov (1995) showed that the NPC satisfies these criteria and can be used to construct economical wood structures.

A variety of the NPCs were designed by the CREMBS engineers (Figure 3). These connectors were used to fabricate built-up members from round or sawn timber and to simplify the assembly process.
Chapter 2  
Background

Figure 3. NPC designs.

A. Nails welded to the plate

B. Nails welded to the wire

C. Combined type with extension for on-site assembly

D. Twin type with extension to accommodate truss web

Figure 3. NPC designs.
The NPCs used a set of nails 5-8 mm (1/5-1/3 inches) in diameter, welded to a metal plate or a wire. The connector was made from readily available low-carbon steel, unless another steel type was required by the special service conditions. Piskunov (1995) also indicated that other materials and connector designs could be used to meet the specific objectives of a particular project. The following options can be considered when designing a NPC:

i) base type: soft (wood, plastic, cement), stiff (metal, plastic), flexible (wire), removable template which is used only for positioning and holding the nails during the initial embedment;

ii) nail type: point, pointless, combined (included combination of both types);

iii) nail material: metal, structural plastic;

iv) nail-base connection: nails welded to the plate, nails inserted into predrilled or pre-punched holes, nails punched into a soft plate;

v) plate extension: with or without extensions;

vi) nail orientation: one-sided, two-sided, three-sided connectors (Figure 4)

One-sided       Two-sided      Three-sided

Figure 4. Nail plate connector with different nail orientations.

The CREMBS researches developed two design methods to model the lateral resistance of the NPC. The first method incorporated the theory of an elastic-plastic beam resting on a non-elastic foundation. This method allowed for modeling the connector strength and stiffness. The second method was based on the yield limit theory and predicted connector strength. The next two paragraphs elaborate on these two methods, respectively.

The first method required input of the nail and the wood foundation properties. The nail stiffness and foundation stiffness were measured experimentally. The foundation behavior was modeled as a function of load, nail diameter, grain orientation, and time. A
three-parameter viscoelastic element (Figure 5) provided a qualitative representation of the foundation behavior. The foundation model was formulated with Equation [2].

\[
\delta (t) = \frac{(1+S_t)\sigma (1+\psi_i t/k)}{C_E}
\]

where:
- \(\delta (t)\) = deformation of the foundation as a function of time;
- \(S_t\) = proportion between viscoelastic and visco-plastic deformations at a given time increment;
- \(s\) = stress;
- \(\psi_i\) = viscosity parameter;
- \(t\) = time;
- \(k\) = time dependent parameter;
- \(C_E\) = viscoelastic parameter.

\[\text{Figure 5. Three-parameter viscoelastic element.}\]
The equilibrium of the nail-foundation system was defined by a nonlinear equation (Equation [3]).

\[
\left( E^* I_N \right) y^4 + K_\sigma y = q \tag{3}
\]

where:

\( (E^* I_N) \) = stiffness characteristic of the nail, which is a function of stress \( \sigma \);

\( K_\sigma \) = foundation stiffness coefficient, which is a function of stress \( \sigma \);

\( q \) = external load;

\( y \) = deformation.

The equation was solved using the principal of minimum potential energy of the nail-foundation system, which was expressed in the matrix (Equation [4]):

\[
[K](D) - R = 0 \tag{4}
\]

where:

\( K \) = general stiffness matrix of the nail-foundation system;

\( D \) = matrix with deformation vectors;

\( R \) = matrix of load vectors.

A closed form solution of Equation [4] was derived.

The second method employed the yield theory concept. A linear stress distribution along the nail shank was assumed (Figure 6).
a. Effective length of the nail.  
b. Stress distribution along the nail.

*Figure 6. Yield theory assumptions made by Piskunov.*

Imposing the conditions of equilibrium and assuming pure plastic behavior of the nail-foundation system, the maximum yield force was derived. Three failure modes were considered: 1) two plastic hinges at points C and D; 2) one plastic hinge at point D; 3) no plastic hinges, wood yielded along the nail length. These modes correspond to the modes IV, III\textsubscript{s}, and I\textsubscript{s} respectively of the connection yield theory as used in the 1997 National Design Specifications for Wood Construction (NDS) (AF&PA 1997).

Based on the yield theory approach, a design method for the NPC was developed (Appendix A). The method was further extended for the analysis of the built-up members fabricated with the NPC under axial and combined axial and bending loading (CSRICS 1988).

The nail spacing parallel to the grain and the minimum end distance sufficient to prevent wood splitting were determined experimentally. Deformation of wood fibers during the nail embedding into the wood were measured by the strain gauges located at the following positions: 1) near the nail hole, 2) 6 diameters and 3) 12 diameters away from the hole in the direction parallel to the grain. The specimens were fabricated of 12 percent moisture content pine lumber. Three groups of nails were investigated (Table 1). Nails from
the first and second groups were installed without predrilling, whereas the nails from the third group were installed into predrilled holes.

Table 1. Nail geometries.

<table>
<thead>
<tr>
<th>Group #</th>
<th>Nail diameter, in/mm</th>
<th>Nail length, in/mm</th>
<th>Nail point angle, deg</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.20 / 5</td>
<td>2.36 / 60</td>
<td>70, 63</td>
</tr>
<tr>
<td>2</td>
<td>0.32 / 8</td>
<td>3.54 / 90</td>
<td>63, 56</td>
</tr>
<tr>
<td>3</td>
<td>0.47 / 12</td>
<td>4.72 / 120</td>
<td>56, 50</td>
</tr>
</tbody>
</table>

The study concluded that the deformations of the fibers were strongly correlated with the nail diameter and the nail point angle, and less correlated with the depth of penetration. Deformation along the grain decreased to zero at 6d and 5d from the penetration point for 5 mm nails and 8, 12 mm nails, respectively. Therefore, the nail spacing of at least 12d parallel to the grain must be used to prevent wood splitting. Piskunov also recommended a row spacing of 3d and edge distance of 3.5d for the perpendicular to the grain direction. The smaller nail point angles were associated with less deformation. However, the excessive wood crushing around the hole decreased the stiffness of the connection. The recommended nail point angles were 63° and 56° for 5 mm and 8 mm nails, respectively. The placement requirements were valid for the nails with the shank length of 0.3-0.7 of the side member thickness.

Piskunov (1995) also indicated that a deviation of a nail from the vertical alignment must not exceed five percent, otherwise the design value should be adjusted with a strength reduction factor. The force required to install one nail into pine wood was measured 1.2 kN (270 lb) for 5 mm nails and 2 kN (450 lb) for 8 mm nails. The pressing rate of 0.5-2 cm/min (0.2-0.8 in/min) was recommended to minimize splitting.

The research conducted by the CREMBS showed that the most efficient utilization of the NPC was in the members subjected to compression and/or bending forces. Some examples are ceiling structures, rafter systems, folded roofs, framed-panel wall systems, top cord and web truss elements, columns, and railroad ties. CREMBS successfully
implemented these structures in the commercial and industrial buildings in the Kirov region of Russia. The trusses with wood built-up top chords and steel bottom chords were designed and constructed to span up to 24 meters (80 ft). In 1980, 24 meter trusses were used to cover a garage. The structure was monitored for five years. Results of the monitoring demonstrated an excellent performance of the structure. Since then a number of other buildings were constructed using the structures with the NPC.

2.4 Dowel connections

2.4.1 Single dowel connections

A single dowel connector is a metal pin or wooden stick which is used to transfer primarily shear forces from one element to another. Nails, bolts, screws, and spikes are the most common dowel connections. This chapter highlights the most important research done in the field of single dowel connectors. The information is used to position the NPC relative to the traditional connections.

Trayer (1932) conducted a series of experiments on bolted connections. The results of this work formed basis for the first edition of the NDS. The connection strength at the proportional limit was determined from empirical equations established based on experimental data. Tests were limited to the specific joint geometries and connector designs. Therefore, the connections beyond the scope of the NDS provisions required a code approval. This practice impeded development of new connectors. The empirical method was used in the NDS until 1991.

Johansen (1949) applied the yield limit theory to wood connections. Using the principle of static equilibrium and the yield limit theory assumptions, he formulated and solved a set closed-form equations that predicted the strength of wood connections. The following assumptions and limitations are inherent to the yield theory:

1) both bending of the dowel and wood crushing under the dowel are ideally elastic-plastic;

2) connector is a subject to lateral loading only;
3) yield strength of a single dowel is determined;
4) any additional boundary conditions are ignored;
5) friction is ignored;
6) brittle failure modes are beyond the model scope;
7) load – slip relationship is beyond the model scope.

The yield theory showed good agreement with the experimental results. According to the yield theory, the connector resists the applied load until the compressive yield strength of wood under the dowel is exceeded and/or plastic hinges develop in the dowel. There are four possible yield modes to be analyzed. The actual yield limit is the smallest load from all four yield modes. Dowel connections of any type and geometry can be modeled with the yield theory. Moreover, the yield theory provides the engineer with a sense of the mechanics involved in the connection response.

The yield theory was extended by Meyer (1957), who applied the theory to the nail joints in single and double shear, investigating effect of different bearing strength of the members and effect of asymmetry. A number of other studies were conducted over the last fifty years enabling world-wide adoption of the yield theory. The 1991 NDS edition incorporated the yield theory concept as the basis for establishing allowable strength for the dowel type connections. The NDS defines the yield strength with a 5% diameter offset rule, which locates the yield load between the proportional limit and the ultimate strength. The yield strength of the connector is achieved through appropriate spacing, end and edge distances.

The NPC is a dowel type connector. Therefore, the principles of the yield theory are applicable to the NPC. However, the NPC is beyond the scope of the 1997 NDS provisions. The NDS establishes the minimum nail penetration into the main member of twelve diameters for the full design value and at least six diameters for the reduced design value. Nails of the NPC penetrate the main member by 3.5 diameter of the nail. However, the analysis of the test results obtained by Platt (1998) did not reveal a reduction in the NPC strength as compared with the strength properties computed using the NDS methodology without applying the penetration factor. Moreover, the NDS “… does not preclude the use of connections where it can be demonstrated by analysis based on generally recognized theory,
full-scale or prototype loading tests, studies of model analogues or extensive experience in use that the connections can perform satisfactory in their intended end uses” (Section 7.1.1.4 of the 1997 NDS, AF&PA, 1997). Because the satisfactory performance of the NPC was demonstrated by the previous studies (Pishkunov 1995, Platt 1998) and it will be further proven satisfactory in this study, the NPC can be used for construction of wood buildings.

The yield theory requires input of the dowel bearing strength of wood. The dowel bearing strength of wood is a function of the angle between load and grain orientation, specific gravity of wood, and diameter of fastener. The 1997 NDS provisions recognizes that the bolt bearing strength depends on the wood grain orientation. For the small diameter fasteners such as nails and screws, it is assumed that the bearing strength on wood is independent of the grain orientation. Some studies (Foshi 1974; Mack 1960) indicated 15-20% difference for perpendicular-to-grain and parallel-to-grain nail bearing strength. McLain (1975) found that for small (≤ 0.01 in) deformations the nail bearing strength was independent of the grain angle, whereas at higher deformations the nail bearing strength was a function of the grain direction. Soltis and others (1987) determined the dowel diameter at which the bearing strength is identical for perpendicular- and parallel-to-grain directions. According to this study for 0.25 inch diameter dowel at 0.1 inch deformation, the critical specific gravity was around 0.45, at ultimate deformation it was around 0.35. Whale and Smith (1986) found that the nail dowel bearing strength was different for parallel and perpendicular-to-grain directions at the ultimate load deformations. Wilkinson (1992) showed that for 5% offset load the nail bearing strength was unaffected by the grain direction. He found that the reason behind different behavior of a nail and the same diameter bolt was dissimilar bearing conditions. The predrilled bolt hole provided smooth bearing surface, whereas the nail is driven into solid wood producing a different type of bearing surface. For parallel-to-grain loading, the nail bearing strength is about 80% that for the same diameter bolt. For perpendicular-to-grain loading, ¼ inch nail and bolt yielded approximately same bearing strength over most of the specific gravity range.

The NPC exhibited lower strength and stiffness for the perpendicular-to-the-grain direction as opposed to parallel-to-the-grain direction of loading (Platt 1998). After scrutinizing the results of the NPC tests, the author came to the conclusion that the
difference in the NPC strength in wood-based composites between the two grain directions was mainly associated with the test setups rather than the nail bearing strength. In the case of solid wood, the dowel bearing strength was also an important factor. The test setups were designed in a manner so that perpendicular-to-the-grain loading induced a tension perpendicular to the grain failure mode, whereas parallel-to-the-grain loading resulted in the dowel yield failure mode. The results of the dowel bearing tests of the 0.25 inch nails conducted by Platt further contribute to this conclusion. The maximum nail bearing strength for the parallel-to-the-grain loading direction was greater as opposed to the perpendicular-to-the-grain loading by 0.4%, 8.7%, and 69.5% for Parallam®, TimberStrand®, and SPF, respectively. For both wood-based composites, the difference was not statistically significant at $\alpha=0.05$. These findings were consistent with the data for TimberStrand from Johnson and Woeste (1999). Johnson and Woeste reported that the same value of the dowel bearing strength for nails in TimberStrand® should be used for the connection design regardless the grain direction. Therefore, the grain orientation effect on the strength of the NPC in Parallam and TimberStrand® can be ignored for most practical purposes. The grain orientation effect in solid wood is more pronounced and should be considered in the design of the NPC.

Aune and Patton-Mallory (1986) discussed the yield theory in regard to the nail connections. They showed that for a three-member joint with a steel plate in the center two yield modes could develop (Figure 7 and 8).
Figure 7. Yield mode with two nail yield hinges.

Thickness condition:
\[
\sqrt{2} < \frac{t_1}{\sqrt{\gamma}} < 4 \quad [5]
\]

Yield load (lb):
\[
F_u = 2fe \sqrt{2\sqrt{t_1^2 + 2\gamma t_1}} \quad [6]
\]

Figure 8. Yield mode with four nail yield hinges.

Thickness condition:
\[
\frac{t_1}{\sqrt{\gamma}} \geq 4 \quad [7]
\]

Yield load (lb):
\[
F_u = 4 \sqrt{f_e M_y} \quad [8]
\]
The current NPC design (Figure 63) corresponds to the first case (Figure 7):

\[ M_y = f_y \cdot d^3 \cdot \frac{1}{6} = 130,000 \times 0.25^3 \times \frac{1}{6} = 338.54 \text{ lb-in} \]

\[ f_e = F_e \cdot d = 4650 \times 0.25 = 1162.5 \text{ lb-in} \]

\[ \gamma = \frac{M_y}{f_e} = \frac{338.54}{1162.5} = 0.291 \]

\[ \sqrt{2} < \frac{t_1}{\gamma \sqrt{0.291}} = 1.62 < 4 \]

where:

- \( d \) = nail diameter, in;
- \( t_1 = 0.875 \) = nail length, in;
- \( M_y \) = nail yield moment, lb-in;
- \( f_y = 130,000 \) = nail yield strength, lb/in², obtained experimentally by Platt (Platt 1998);
- \( f_e \) = wood embedding strength, lb/in;
- \( F_e = 4,650 \) = dowel bearing strength for TimberStrand, lb/in², using Table 12A of the 1997 NDS and the effective specific gravity method (Johnson and Woeste 1999);
- \( \gamma = M_y/f_e \) – coefficient.

To generate the second yield mode the nail length, \( t_1 \), should be 2.15 inch. The dowel connections with a metal plate have higher strength than all-wood connections due to the effect of nailhead fixity. The researchers also found that because little friction was present in the connections with a steel member, the yield strength was well predicted by the yield theory. Moreover, the connections with a steel member showed a considerable increase in stiffness when compared to all-wood connections. For small deformations, the connections with a steel plate were up to three times stiffer than corresponding all-wood connections.
Based on experimental data on the bolted connections with steel side members, Wilkinson (1992) concluded that thickness of the metal plate had only a slight effect on the connector strength.

In light of the above discussion, the lateral resistance of the NPC in TimberStrand® or Parallam® can be estimated using the 1997 NPC methodology for bolted wood-to-metal connections in double shear (Part VIII: Bolts, 1997 NDS, AF&PA 1997) or for nailed wood-to-metal connections in double shear (Part XII: Nails and Spikes, 1997 NDS, AF&PA 1997) with the penetration factor equal to unity. For the NPC with 0.25 inch nails in yield mode III, the bolt method will result in a more conservative allowable design values than the nail method. The reason is that the bolt method uses the reduction term of \(3.2 \times (1 + 0.25(\theta/90))\), whereas the nail method uses the reduction term \(K_D = 3.0\) (Technical Report 12, AF&PA, 1999). The engineer should use his/her judgment and experience based on the particular applications of the NPC in the structure. The author recommends using the nail method because the NPC is a nail connector. Because 0.25 inch nail bearing strength in SPF was significantly different for two the grain direction, the lateral resistance of the NPC in solid wood should always be estimated using the bolt method. The bolt method will allow one to account for the grain orientation effect.

2.4.2 Multiple dowel connections

The lateral force can be distributed nonuniformly between the dowels in multiple dowel connections. Therefore, the strength of a dowel connection is not directly proportional to the number of the fasteners. Cramer (1968) developed an analytical method for predicting the force distribution between the bolts in a multiple-bolted connection. The method was based on the assumptions of elastic stiffness of the connected elements, nonuniform stress distribution in the members, lateral loading, negligible friction, and a constant value of load-slip modulus for all bolts. He concluded that about 50% of the total load was resisted by the two end bolts in a six-bolt connection. Lantos (1969) developed a similar model based on the assumption of uniform stress distribution in the members. His model showed a good agreement with the Cramer’s method.
Cramer’s work was extended by Wilkinson (1980, 1986). Wilkinson assumed unique load-slip behavior for each bolt in the multiple bolt joint, and he accounted for variable spacing of the bolts in a row. Wilkinson’s numerical model incorporated fabrication tolerances and variability in the single-bolt load-slip relationship.

Thomas and Malhotra (1985) used the finite element method to model the behavior of a multiple nail connection with up to eight nails in a row. The model was validated against the experimental results of the connection fabricated using 0.127 inch (3.23 mm) nails. They concluded that the load was not uniformly distributed between the nails. They proposed to apply a modification factor of 0.9 for connections with more than three nails for the deformations within the proportional limit. The model was developed and verified for one row connections.

Blass (1994) investigated the variation in the load-slip behavior of the nails arranged parallel to the grain within a multiple nail joint. His tests demonstrated that there is about 20% increase in the maximum load and the elastic modulus for the nails driven into a knot, whereas the initial modulus was independent of the knot presence. The study also showed weak correlation between the nail spacing and the joint strength and stiffness for the perpendicular-to-the-grain loading. Furthermore, Blass (1994) developed a stochastic model to predict the ultimate strength of a nail connection. The model incorporated individual load-slip relationship for every nail in a multiple nail connection. He verified the model with the results from the previous investigations. He also indicated that the load-slip relationship was well described by a parametric model with the following parameters: initial modulus of elasticity, final modulus of elasticity, intercept of final modulus of elasticity line with load axis, and maximum load. Blass found strong correlation between the maximum load and the final modulus of elasticity. The results of the stochastic simulation demonstrated a lack of correlation between the number of nails per row and the ultimate strength of the connection for parallel-to-the-grain loading.

Although Lantos’s model does not differentiate between bolts and nails, it was mostly supported by experimental studies on bolted connections. The NDS does not specify a group action factor for multiple nail connections. This practice is based on the assumption that the forces are redistributed between small dowels due to their ductile behavior. The
NPC, which has nails ¼ inch in diameter and additional restraint due to the metal plate, should exhibit uneven load distribution among the nails in the same row. In the current NPC design (Figure 63) each nail is positioned in a separate row. If the design changes, and three or more nails in a row are used, the group action factor should be applied unless it is proven unnecessary by tests. A study on the group action factor effect on the NPC response is beyond the scope of this project.

2.5 Moment resistant connections

An important attribute of the RTA frames is the moment resistance of the connections. The use of these connections simplifies the assembly process by eliminating the need for temporary bracing in the plane of the frame. Modeling of the moment resistant nail connections in wood structures is a complex problem because the response of each nail is a function of the moment arm of this nail. Nail moment arm is the distance between the centroid of the connection and the center of the nail. While the most remote nails can be at the capacity level, the nails close to the centroid can be within the elastic performance level. Therefore, these connections exhibit semi-rigid performance. A number of theoretical and analytical studies were conducted to quantify the rotational strength and stiffness of the moment resistant connections in wood structures. Some of them are summarized in this section.

Bouchair and others (1996) modeled multiple dowel connections using the finite element method. A two-dimensional analysis was performed. Nails were modeled as a set of linear springs. The circle dowel arrangements were studied for L- and T-shaped connections. They concluded that the ratio of the frame member stiffness to the stiffness of the connection influenced the load distribution between the dowels. If the frame members had high stiffness relative to the connection stiffness, the load was more uniformly distributed between the dowels. However, the connections with a steel side member had higher stiffness and the load was distributed less uniformly between the dowels. The external circle of dowels always experienced higher loads than the internal circle.
Ohashi and Sakamoto (1989) investigated behavior of the moment resistant multiple bolted connections with additional shear fasteners inserted between the connected elements. The following shear fasteners were studied: bulldog dowel, split ring, and shear plate. The study included testing of the connection under cyclic loading and numerical modeling of the connection response. The numerical model was limited to linear behavior. Tests showed high stiffness and large ductility for all connections except the connections with the shear plates.

Bainbridge and Mettem (1998) reviewed concealed moment resistant connections. They indicated that the concealed prefabricated wood connections have attractive appearance and high strength. Moreover, the prefabricated systems improved quality control over the on-site constructed connections. They categorized these connections into five categories:

a) concealed bonded-in rods;

b) concealed bonded-in plates;

c) adhesive bonded surface contact joints;

d) timber connectors within lapped joints;

e) dowel joints.

They concluded that both traditional and novel moment resistant connections were beyond the scope of design specifications.

Haller (1998) discussed pros and cons of different methods for modeling semi-rigid wood connections. Table 2 summarizes the conclusions of this study.

<table>
<thead>
<tr>
<th>Method</th>
<th>Effort</th>
<th>Flexibility</th>
<th>Comprehension</th>
<th>Abstraction</th>
<th>Handling</th>
</tr>
</thead>
<tbody>
<tr>
<td>Functional</td>
<td>Medium</td>
<td>Very low</td>
<td>Medium</td>
<td>Low</td>
<td>Difficult</td>
</tr>
<tr>
<td>Spring-Dashpot</td>
<td>Medium</td>
<td>Medium</td>
<td>Medium</td>
<td>High</td>
<td>Very easy</td>
</tr>
<tr>
<td>Analytical</td>
<td>High</td>
<td>High</td>
<td>Very high</td>
<td>High</td>
<td>Easy</td>
</tr>
<tr>
<td>FE-method</td>
<td>High</td>
<td>Very high</td>
<td>Low</td>
<td>Low</td>
<td>Medium</td>
</tr>
</tbody>
</table>

Table 2. Assessments of the modeling techniques for semi-rigid wood connections.
Experimental methods include testing of the connections and statistical analysis of the results. This approach requires testing for every new connection configuration.

Spring-dashpot models can be also used to describe the load-deformation relationship of the semi-rigid connections. This approach fails to predict the failure modes and creates highly abstractive models.

Analytical modeling of semi-rigidity involves modeling of the beam-foundation interactions which can be very complex. Solutions for this models often obtained using numerical methods.

The finite element method can be successfully implemented for modeling wood connections. Both two- and three-dimensional models can be formulated. The disadvantage of this method is that every joint configuration requires generation of a unique mesh. Moreover, the three-dimensional models need input of the material properties for every independent axis, which are difficult to measure experimentally.

Guan and Rodd (1996) used the finite element method to model moment resistant connections in heavy glulams. A three-dimensional mesh was generated (Figure 9). The dowels were made from hollow steel tubes to decrease the stiffness of the joint and to increase its ductility. Additionally, the joints were reinforced with densified veneer wood (DVW) plates glued to the sides of the joints. The dowel and the reinforcing plates were modeled as an elastic-plastic material, whereas the wood was modeled as orthotropic elastic material except the areas of dowel embedment, where the perfect plastic response was assumed. The model enabled the researches to predict moment-rotational relationship of the connection. Connections with 4 and 8 dowels were modeled. Results of the modeling showed good correlation with the experimental data.
Leijten and Virdi (1996) studied behavior of a moment resistant timber joint with dowel type fasteners reinforced with DVW. To eliminate the gap between the dowels and wood members the pins were made of expandable tubes. After the installation the pins were expanded in diameter resulting in the tight fit and, therefore, increased initial stiffness due to an instantaneous bearing of the dowel on the surrounded wood. Moreover, the dowel expansion prestressed the connection further increasing its overall stiffness. Tubes with 18 and 35 mm diameter were used in this study. The moment-rotation relationship of the joint was measured from four-point bending test and tension test.

Jensen and Larsen (1998) modeled the semi-rigid behavior of the dowel wood connections with steel gusset plates using the finite element method. The model was based on the response of a single dowel and was capable of predicting the load-deformation relationship in the range between zero and the first peak load. A new element was developed to model a two-dimensional group of dowels. The element incorporated material and geometrical nonlinearities. The authors also indicated disadvantages of modeling the

![Figure 9. FE mesh for a multiple dowel moment-resistant connection.](image)
semi-rigid connections with a set of independent spring elements due to the limited applicability of the principle of superposition beyond the proportional limit. They concluded that the coupling effect should be incorporated in the analysis.

Pellicane and others (1991) analyzed moment resistance of the connections with two bolts. The researches developed a nonlinear model for predicting the moment-rotation relationship of a two-bolt connection loaded at an arbitrary angle. The model incorporated the nonlinear translational springs in the parallel and perpendicular-to-the-grain direction. The springs were assigned the lateral load-slip relationships of the bolt in the corresponding directions. Results of the modeling were validated with the experimental data for the linear and the initial nonlinear performance of the connections loaded at $0^\circ$, $60^\circ$, $75^\circ$, and $90^\circ$ to the grain. The model ignored the spring coupling effect. Comparison of the analytical and test results showed the maximum error of 5% for the wood-to-wood connections and 15% for the wood-to-metal connections.

Masse and Salinas (1986) modeled the nonlinear moment-rotation relationship of the multi-laminated nailed connections using Equation [9]. Formulation of this equation was based on the modified torsion formula introduced by Perkins (1962) and the estimated load-slip curve for a single nail. Grain orientation effect was ignored. The model was validated using the results of two point bending tests of beams fabricated with the multi-laminated nailed connections. Theoretical and experimental results were in good agreement.

\[
M = \sum_{i=1}^{n} m_i
\]

\[
m_i = \begin{cases} 
4223 \left(2 \theta R_i\right)^{0.507} R_i & \text{for } \theta R_i < 0.5 \\
4152 \left(2 \theta R_i\right)^{0.349} R_i & \text{for } \theta R_i > 0.5
\end{cases}
\]

[9]

where:

- $M$ = total resisting moment of the joint, N-mm;
- $m_i$ = moment resisted by a single nail;
- $\theta$ = rotation, rad;
- $R_i$ = distance from the centroid to the nail, mm.
Frenette (1997) studied the dynamic response of a timber frame with dowel moment resistant connection under cyclic loading. The analysis incorporated semi-rigid behavior of these joints. A cyclic response of the joints was modeled using a finite element model. Dowels were modeled as an elastic-plastic beam supported by a nonlinear wood foundation. The principle of virtual work was used to formulate the equilibrium of the dowel. The model was calibrated using the results from a cyclic test on a single dowel connection. A computer program was developed to obtain solution for the model.

2.6 Buildability of wood structures

Engstrom (1996) introduced a concept of “connection design for buildability” (CDFB). The concept focused on the development of the simplified assembly methods for timber-framed structures that would also meet the strength and serviceability criterion. He indicated that a successful timber-framed structure should be optimized with respect to cost of the material and cost of the assembly process and, at the same time, perform its structural functions. According to Engstrom, the buildability of the connections is the key to the economical timber-framed structures. Moreover, to meet the requirements of the CDFB concept the connections should be “cheap and uncomplicated”.

2.7 Analysis of structures with semi-rigid joints

The traditional methods of structural analysis model connections as pinned or fixed. However, neither the first nor the second assumption realistically represents the response of the moment resistant wood connections. The assumption of the pinned connections results in conservative designs of members, liberal designs of connections, and additional bracing requirements. The assumption of fixed connections may lead to unsafe designs of the members and over designed connections. The assumption of pinned connections means that the rotational stiffness is zero, whereas the assumption of rigid connections means that the rotational stiffness is infinite. Wood connections have finite stiffness and exhibit nonlinear behavior at very small deformations. Therefore, the structural
analysis of wood buildings should incorporate the semi-rigid behavior of the connections in order to accurately estimate the distribution of the forces between the members and to determine the forces acting on the connections. Researches developed a number of methods that enable structural analysis of wood buildings with semi-rigid connections. This section summarizes some of these methods.

Monforton and Wu (1963) discussed matrix analysis of the frames with semi-rigid connections. The semi-rigid connections were incorporated in the analysis by introducing the initial member force, J. Then, the node forces, R, were determined using Equation [10].

\[
R = F - a^t J \tag{10}
\]

where:

\begin{align*}
F &= \text{external forces}; \\
 a^t &= \text{transformation matrix from the node forces to the member forces}; \\
 J &= \text{initial member force}.
\end{align*}

Therefore, resulting node displacements, r, can be computed with Equation [11].

\[
r = K^{-1} (F - a^t J) \tag{11}
\]

Resulting member forces can be computed with Equation [12].

\[
S = k a r + J \tag{12}
\]

where:

\begin{align*}
K &= \text{stiffness matrix of the structure; this matrix relates the node and the node displacements}; \\
k &= \text{block-diagonal matrix; this matrix included the members stiffness matrixes}.
\end{align*}

A computer subroutine was developed to solve the formulation.

Gupta and others (1992) analyzed MPC wood trusses using the matrix method. The model incorporated semi-rigid behavior of the connections. They also analyzed the same trusses modeling the connections as pinned and rigid. The contribution of the semi-rigid joints reduced the truss deflection by 34% as opposed to the trusses with pinned connections and the maximum moment by 13% as opposed to the trusses with rigid
connections. They concluded that the assumption of the semi-rigid behavior of the truss connection allowed for improved modeling of the MPC truss response.

Poutanen (1982) discussed analysis of MPC wood trusses. He indicated that there were two sources of moment forces in the connections: moment resistance of the truss plate and geometric eccentricity of the connection. He concluded that the moment resistance should be incorporated in the design procedures. He further discussed two methods of analysis of the MPC trusses. The first method was to analyze the truss without considering the eccentricity to compute the element forces, and considering eccentricity when designing the individual connections. The second method was to account for the eccentricity directly in the truss model.

Riley and others (1993) analyzed structural performance of a 28 ft span MPC wood truss with 5/12 slope. The semi-rigid behavior of the truss metal plates was modeled with fictitious members. The fictitious members were assigned with stiffnesses obtained from the previous experimental studies. Connections were also modeled as pinned and rigid to compare the analysis to classical methods. The authors concluded that the maximum deflection of the truss was 23% lower and 71% higher compared to the pinned and rigid analyses, respectively. Maximum moment in the truss was 17% and 16% lower compared to the pinned and rigid analyses, respectively. The sensitivity study showed that modeling the truss using the mean value of the modulus of elasticity was adequate; the model was insensitive to changes of the axial plate stiffness; changes in the rotational stiffness affected the response differently for different locations of the connector. Increase of the stiffness in the heel joint by a factor of four lead to the decrease of the deflection by 6%; whereas an increase in stiffness of the tension-splice joint by a factor of 1.65 resulted in a decrease of the deflection by 7%.

Masse and Salinas (1986) analyzed heavy duty wood trusses with the multi-laminated nailed connections. The analysis incorporated fictitious elements, which were assigned with connection stiffness. The global stiffness matrix was a function of the external load. The analysis showed different force distribution in the truss members as opposed to traditional analyses and a shift in the failure mode. Full-scale trusses were tested to validate the analysis.
Foschi (1977) proposed a method for modeling the MPC wood trusses without employing the fictitious or spring elements. He approximated the load-slip relationship of a single tooth with an exponential curve, calculated virtual work done by the tooth through its displacement, and integrated over the plate area to compute the non-linear response of the entire plate. This response was incorporated into the system model of the truss. After separating the linear and non-linear components, the final system model was expressed with Equation [13].

\[
[K] \{x\} = R_0 + R_1 (\{x\})
\]  

[13]

where:

- \([K]\) = stiffness matrix from linear part;
- \(\{x\}\) = global vector of the unknown displacements;
- \(R_0\) = load vector;
- \(R_1\) = vector from nonlinear component, function of \(\{x\}\).

The load-slip relationship for a single tooth was obtained from four basic tests. The proposed method modeled the contribution of the metal truss plate connection without using the analog models.

Maraghechi and Itani (1984) analyzed the MPC wood trusses. They modeled the connections with dimensionless elements composed of three independent linear springs. The axial, shear, and rotational springs were assigned with stiffnesses of the truss plate in the respective direction. Stiffness characteristics of the truss plate were measured experimentally. Tests of the full-scale MPC wood structures showed good agreement with the model. Moreover, the results of the analysis were compared with the method proposed by Reardon (1971) for analysis of the frames with semi-rigid connections. A good correlation was obtained between the two methods. Sensitivity studies showed little effect of the shear spring on distribution of the member forces, whereas the structure was sensitive to the properties of the axial and rotational springs.

Gesualdo and Riskwoski (1996) analyzed behavior of a wood beam with semi-rigid dowel connection. The non-linear load-slip relationship was formulated using a power function statistically fit to experimental data (Equation [14]).
\[ P = k \Delta^c \]  \hspace{1cm} [14]

where:
- \( P \) = load;
- \( k \) = constant statistically determined from the experimental data;
- \( \Delta \) = slip;
- \( c \) = exponent from the experimental data.

The final stiffness matrix incorporated the effect of the semi-rigid connections and the gap between members. The system model was formulated using Equation [15].

\[
\left( [K_f] + [C_1] \right) \{u\} = \{F\} - \left( [K_c(u)] - [C_1] \right) \{u\} \]  \hspace{1cm} [15]

where:
- \([K_f]\) = linear stiffness matrix for the frame members;
- \([C_1]\) = fixed connector matrix;
- \(\{u\}\) = nodal displacement matrix;
- \(\{F\}\) = nodal force matrix;
- \([K_c(u)]\) = non-linear connector matrix.

This formulation was incorporated into a computer program which was used to analyze structures with semi-rigid connections. Results of this analysis showed good agreement with the an in-service computer program.

2.8 Wood construction systems

2.8.1 General

This section briefly describes light-frame and post-frame wood construction systems. The systems are compared in respect to the methods of structural analysis used to design the buildings. The lateral and vertical load paths are specified for both systems. The RTA construction system is positioned relative to these traditional construction systems. The applicability of the available design methods to the RTA system is discussed and the required research topics are identified.
2.8.2  

**Wood frame construction**

The light-frame wood construction method is a building technology which uses small closely spaced members. Typical spacing of the frame members are 16 or 24 inch on-center. Frame members are made of dimension lumber with 2 or 3 inch nominal thickness. Light-frame buildings resist gravity loads with a system of beams, trusses, and columns. Every frame member is designed individually for gravity and wind loads. Although an individual frame member can be designed for wind load, it is assumed that this individual frame member does not contribute to the overall lateral resistance of the building. To resist the lateral load the frame members are braced with structural panel sheathing. This produces diaphragms and shear walls. Therefore, the lateral load is transferred through the roof and floor diaphragms to the shear wall and into the foundation. Diaphragms and shear walls are designed as complete structures to resist the unit shear. Lateral design methods ignore combined action between the frame and the system of diaphragms and shear walls. More detailed description of the light-frame construction system can be found in “Wood Building Technology” textbook (Canadian Wood Council, 1993). The design methods are described by Breyer (1993).

A post-frame building consists of a series of frames typically spaced at 96 inch on-center. The frame includes two columns called posts and a truss. Posts are embedded into the foundation or attached onto a concrete slab. Frames are connected with a system of girts and purlins typically spaced 24 inch on-center and nailed to the frame members. The building is enclosed with metal cladding nailed to the girts and purlins. This produces diaphragms and shear walls. Design of the post-frame buildings is more than for light-frame buildings. Because the frame can be sufficiently stiff to resist a portion of the lateral force, the combined action between the frame and the envelope should be considered in the analysis. Therefore, the frame is designed to resist gravity loads and a portion of the lateral load. The value of the lateral force acting on the frame is a function of the ratios of diaphragm stiffness to lateral frame stiffness and shear walls stiffness to lateral frame stiffness. The higher the ratios, the smaller the lateral force acting on the frame and the greater the contribution of the envelope. Alternatively, the smaller the ratios, the lesser the relative contribution of the diaphragm and shear walls and the greater the lateral frame force.
The design procedure for post-frame buildings is outlined in the ASAE Standard EP484.2 “Diaphragm Design of Metal-Clad, Post-Frame Rectangular Buildings” (ASAE Standards 1999). This procedure involves laboratory testing of diaphragm and shear wall panels. Results of the testing are extrapolated to the actual building geometry. The use of validated structural models and equivalent assemblies is also allowed for computing the diaphragm and shear wall stiffnesses. Frame stiffness can be determined using the principles of mechanics. Calculation of the frame forces involves the combined action of the frame and envelope. This analysis is complex and requires computerized structural design software. Symmetric buildings without intermediate walls and with uniform stiffness parameters can be analyzed using tabulated coefficients.

Bender et al. (1991) presented a simplified method for calculating maximum roof shear in post-frame buildings. Although the method was an approximation based on the assumption of infinite diaphragm stiffness, it eliminated the need for structural computer analysis software and resulted in a good agreement with the more sophisticated design methods. Comparison of the presented method with the ASAE EP481.1 (ASAE Standards 1990) method showed the conservative maximum error of 2.7%.

Skaggs et al (1993) extended the rigid diaphragm design method and developed a simplified analysis procedure for calculating forces in the frame posts. This method also did not require structural computer analysis software. The method was validated for post-frame buildings with heights of 10-20 feet and diaphragm aspect ratios up to 3.07. Analysis was limited to post-frame buildings with posts fixed at the foundation and pinned at the eave.

An extensive description of the post-frame construction system, design methods, and a design example can be found in the short course notes “Design of Commercial Post-Frame Buildings” (Bender and Woeste, 1999).

This thesis compares the RTA and traditional construction systems and investigates the applicability of the available design methods. Current design of the RTA building uses frames spaced 48 inch on-center. Frame connections are semi-rigid, moment-resistant connections with relatively low rotational stiffness. Columns are pinned at the foundation. Frames are connected with recessed girts and purlins spaced 24 inch on-center. Walls and roof are braced with wood-based sheathing products. This study investigates in
details the performance of the RTA shear walls and RTA frames. A series of conservative assumptions is made to evaluate the combined action between the frame and the diaphragm.

2.9 Chapter summary

The chapter highlights history of the development of the RTA system and the NPC. Single and multiple dowel connections are discussed and the NPC is compared to traditional dowel connections. The chapter presents the analytical and experimental techniques for measuring the performance of the moment resistant connections. Design methods for analysis of the structures constructed with the moment resistant connections are presented. Finally, the traditional and the RTA construction systems are compared with respect to the building and design practices.
Chapter 3

Materials and Methods

3.1 Introduction

This chapter describes the materials and methods used to meet the objectives of the study. The following experimental and analytical methods are covered: moment rotation test of the NPC, testing of the plate bearing on the end grain of the frame member, RTA shear wall test, frame-sheathing connection test, and FE modeling of the RTA structures.

3.2 Moment-rotation test of the nail plate connector

3.2.1 General

The objective of this test was to measure the moment-rotation relationship of the NPC. A four point beam bending test was used to apply moment force on the NPC (Figure 10).

![Figure 10. Beam under four point bending. Force distribution in the beam.](image)

Load was applied at two points spaced at equal distances. This test setup resulted in a constant moment and zero shear force throughout the middle span of the beam. The
specimens were assembled with the NPC so that the connection was located in the middle of the span. Therefore, the deformation of the NPC was caused only by the moment force.

3.2.2 Test setup and data acquisition system

Figures 11 and 12 show a drawing and a photograph the test setup, respectively. The drawing details the dimensions and specifies the components of the setup. The load was applied with a 20,000 lb capacity MTS universal testing machine at a constant rate of 0.2 inch/min. This loading rate was chosen according to ASTM standard D 5652 “Standard Test Methods for Bolted Connections in Wood and Wood-Based Products” (ASTM 1998) which required that the maximum load should be reached in approximately

Figure 11. Test setup for the moment-rotation test of the NPC. Detailed drawing.
ten minutes. Preliminary tests were conducted to determine the loading rate. The displacement controlled protocol was used. Load was measured with a 22,000 lb capacity electronic load cell calibrated in the 4,000 lb load range. Displacement of the load cell was measured by a linear variable differential transducer (LVDT) built into the hydraulic actuator.

![Test setup for moment-rotation test of the NPC. Picture of the test.](image)

This LVDT was calibrated for 5 inches of maximum displacement. The hydraulic actuator had a maximum stroke of 5.5 inches. Load from the MTS was transferred to the specimen through a distribution beam, two sets of two sections of steel I-beams, and two wooden blocks. The purpose of the double steel I-beam was to create a space between the specimen and the distribution beam for positioning of the LVDTs. The wooden blocks had a radius of 8 inch and a width of 5 inch to minimize crushing of the specimen under the load. The radius of 8 inch was in conformance with ASTM Standard D 198 “Standard Test Methods of Static Tests of Lumber in Structural Sizes” (ASTM 1998) which requires the radius of curvature to be at least two times the depth of the specimen. The depth of the specimen was 3.5 inch. The two load points were at a distance from the reactions equal to one fourth of the span. A double layer of polyethylene with grease in the middle was located under the right
wooden block to minimize the friction and avoid axial forces in the specimen. The specimen rested on the reaction bearing plates (6 x 4 inch). This size of the plate met the requirements of the ASTM Standard D 198 (ASTM 1998). The reaction bearing plates were attached to the supports with a pinned connection that allowed for free rotation of the plates. The plates also could slide inside the span. The supports rested on a steel I-beam which was secured to the MTS table with steel plates and bolts. Although the supports overhang the MTS table, the results were unaffected because the NPC deformations were measured relative to the specimen.

Displacement of the NPC relative to the wood was measured using two LVDTs (Figure 13). LVDT1 had the maximum range of ± 0.2 inch, whereas LVDT2 had the maximum range of ± 0.5 inch. The LVDTs were attached to the specimen with an aluminum fixture that ensured the consistent distance between the LVDTs for each specimen. LVDT1 was located to the left and LVDT2 to the right from the centroid of the NPC. After transformation of the electric signal into the relative displacement, LVDTs readings were used with Equation [16] to calculate the angle of rotation of the NPC.

\[
\phi = \arctan\left(\frac{|\Delta_1| + |\Delta_2|}{L}\right)
\]

where:
- \(\phi\) = angle of rotation of the NPC;
- \(\arctan\) = arctangent function;
- \(\Delta_1\) = displacement measured by LVDT1;
- \(\Delta_2\) = displacement measured by LVDT2;
- \(L\) = distance between the LVDT’s, \(L = 4.75\) in.

Figures 13 and 14 show the parameters used in Equation [16] and describe the geometry involved in the transformation of displacements measured with the LVDTs into the rotational deformation of the NPC. This transformation is based on the assumption that the plate rotates as a rigid body. Therefore, the relative deformation of the edge of the NPC is identical to the relative deformation of the neutral axes of the NPC. This assumption is critical because the LVDTs measured the deformation of the edge of the NPC, whereas the
data was used to compute the deformation of the neutral axes. There were no adjustments made for a plate bending deformations and transformation was formulated using the small angle theory. It was assumed that the displacements, $\Delta_1$ and $\Delta_2$, were equal to their vertical projections.

![Figure 13. Positioning of the LVDTs on of the NPC. Initial position.](image)

![Figure 14. Deformed shape of the NPC.](image)
Each LVDT core was attached to a brass extension rod which had a round pin on the end. This pin minimized friction and allow for smooth sliding of the extension rod on the NPC.

The moment, M, acting on the NPC was computed using Equation [17].

\[ M = \frac{P \cdot l}{2} \]  

where:

- \( P \) = load from the load cell;
- \( l \) = distance from the center of the support to the center of the loading block, \( l = 19 \) inches.

LabTech® v. 8.04 (Laboratory Technologies Corporation©, 1994) data acquisition software package was used to record the data at a sampling rate of 2 Hz. The test was stopped when a major drop in the load occurred.

3.2.3 Specimens

Three groups of specimens were prepared from three types of wood materials: TimberStrand®, Parallam®, and SPF grade # 2 lumber. The first two materials were chosen based on the results of Phase I project (Platt 1998) as the most preferable for manufacturing the RTA frame members. SPF specimens were tested to compare performance of the NPC in wood-based composites and solid wood. The SPF specimens were cut so that only clear wood with no visual defects was found in the region where the NPC was installed. Fifteen specimens were tested for each group. The choice of the sample size is explained in the Scope section (Section 1.3). The specimens consisted of two 1.5 x 3.5 x 40 inch pieces connected face-to-face with the NPC and two nails (Figure 15). One of the pieces was rabbeted on one end to accommodate the NPC (Figure 16). The rabbet was produced with a router guided by a jig to ensure consistent rabbet dimensions between the specimens. The rabbet thickness was equal to the plate thickness so that there was minimal gap between the two wood pieces.
The NPCs were fabricated by MAZE Inc. of Salem, VA. Figure 17 shows a detailed drawing of the NPC used for the test. The plate of the NPC was made from hot rolled A-36 flat bar steel, the nails were made from ¼ inch cold rolled round 1018 stock. The technology used for manufacturing of the NPC and the reasoning behind this design of the NPC were described by Platt (1998). One edge of the NPC was ground to produce a smooth flat surface to minimize friction with the LVDT pins. Moreover, some grease was applied to this edge to further minimize friction between the LVDT pins and the plate.

Figure 15. Specimen for the moment-rotation test.
Figure 16. Specimen for the moment rotation test. Close-up view of the rabbet.

Figure 17. NPC for the moment-rotation test.
The wood specimens were conditioned in the Brooks Forest Products Center Wood Engineering Laboratory with equilibrium moisture content of around 6% prior to the installation of the NPCs. It was consistent with the methods used by Platt (1998) for direct comparison of the data.

The NPC was pressed into the wood specimens using a hydraulic press. First, the NPC was embedded into the rabbeted piece, and then into the unrabbeted piece. The press was equipped with a set patterns and stops to ensure consistent positioning of the NPCs. The hydraulics were manually operated and the amount of pressure used to embed the NPC was not monitored. Then, the wood specimens were nailed together with 8d penny nails as depicted in Figure 15 to prevent damage of the NPC before testing. After fabrication, the specimens were placed back in the Engineering Laboratory for a period of two weeks to allow for stress relaxation in the connection.

Before the test, the specimen was connected to a steel I-beam with two ½ inch bolts. A pneumatic wrench was used to tighten the bolts. The same steel I-beam was used in all tests. The use of the steel I-beam reduced the number of the specimens. If two specimens would be connected to each other and tested simultaneously, one of the NPC would start yielding first and eventually fail, while the second one would stay in the elastic range. Therefore, the data from only the first specimen would be relevant. A reuse of the second specimen would not be possible due to damage of the specimen.

After the test, the moisture content of the specimens were determined according to the primary oven-drying method (Method A) of ASTM standard D4442-92 (ASTM 1998). Specific gravity of the specimens was determined from the moisture content sections according to Method B (volume by water immersion) of ASTM standard D2395-93 (ASTM 1998). Two moisture content sections were cut from a specimen, one from each piece. The moisture content and specific gravity of the specimen were assumed to be average values of the two sections.
3.3 Testing of the plate bearing on the end grain of TimberStrand®

The objective of this test was to measure the load-deformation curve of the plate bearing on the end grain of the frame member. Results of this test were used as input for modeling of the moment resistance of the NPC as described in Section 4.4.

Specimens were made of TimberStrand® lumber. The width, thickness, and length were 1.5 x 3.5 x 3.5 inch, respectively. This length was chosen to prevent buckling of the specimen and to provide sufficient wood material between the plate and the MTS table. Figure 18 shows the test setup.

![Test setup for the plate bearing on the end grain.](image)

The description of the equipment used in this test and the data acquisition system can be found in Section 3.2.2. The load cell was operated in ±10,000 lb load range. The deformation was measured by the LVDT built into the hydraulic actuator. The LVDT was calibrated for ±2.50 inch of the maximum displacement. A deformation-controlled protocol
was used. Load was applied according to a ramp function at a rate of 0.03 inch/min. Using this rate the maximum load was reached in approximately 2 minutes. This rate was in compliance with the requirements of the ASTM Standard D 5764 (ASTM 1998).

Load was applied by a metal plate made of the same material as the plate used in the NPC. The cross section of the plate was 0.25 x 3.5 inch, whereas the cross section of the loading end of the plate was 0.25 x 1.75 inches. Therefore, the loading end of the plate had width equal to half-width of the wood specimen. The plate was positioned in the grip tightly against the top side of the grip to ensure that the load was transferred from the grip to the plate through the end plate bearing rather than friction. The grip was tightly attached to the load cell to minimize slack in the loading fixture.

The plate was positioned on the specimen at an angle of approximately of 30° to the long side of the cross section of the specimen. This prevented premature splitting of the specimen. During the preliminary testing, the specimens positioned parallel to the plate had a tendency to cleave in the plane of the plate. However, observation of the specimens from the moment-rotation test indicated wood crushing failure mode rather than splitting. Positioning the specimens at this angle minimized the splitting.

Ten specimens were tested. Before the test both ends of the specimen were greased to allow the specimen to expand in the perpendicular-to-the-grain direction due to the Poisson effect. This also contributed to the self-alignment of the specimen during the initial stage of loading. After the test, the moisture content of the specimens were determined according to the primary oven-drying method (Method A) of ASTM Standard D4442-92 (ASTM 1998). Specific gravity of the specimens was determined from the moisture content sections according to Method B (volume by water immersion) of ASTM Standard D2395-93 (ASTM 1998).
3.4  RTA shear wall test

3.4.1  General

The objective of this test was to derive the stiffness and strength of shear walls assembled using the RTA technology. The effect of two girt designs were investigated. The results of the testing were used to design shear walls in a RTA building (Appendix C). The test data was also used to validate the FE model of the RTA shear wall (Section 4.8.2).

3.4.2  Rationale

Although an voluminous body of knowledge on wood shear wall performance can be found in the literature, the RTA shear walls have a number of attributes that make them unique. Therefore, a series of full-scale tests was conducted to quantify the RTA shear wall response to monotonic loading. These unique attributes include:

- columns and top plates are built-up members;
- columns are spaced 48 inch on-center;
- girts are spaced 24 inch on-center;
- girts are recessed flush with the frame or installed on top of the frame;
- use of different materials for different frame members in the same shear wall: TimberStrand®, SPF, CCA treated Southern Pine (SP);
- orientation of the sheathing fasteners relative to the grain orientation of TimberStrand®;
- the frame is assembled with moment resistant connections;
- 24 inch on-center APA rated lap joint plywood siding with grooves is used as structural sheathing;
- sheathing is attached using either screws or nails.
3.4.3 Wall construction and instrumentation

Six 8 x 12 foot walls framed using the RTA technology were tested. Two wall constructions were investigated: (1) girts were recessed flush between frame members and (2) girts were nailed on top of frame members. Hereinafter, the first wall construction type is referred to as Type A (Figure 20), whereas the second as Type B (Figure 22). Type A walls corresponded to the construction methods used for the demonstration RTA building (Section 4.7). Type B walls were tested as an alternative for simplified construction. Three walls of each type were tested. Each wall included four columns, a top plate made of TimberStrand® lumber, and a sill plate made of CCA treated SP. The columns and top plate were built-up members consisting of two 1.5 x 3.5 inch pieces of lumber connected with the NPC. The ends of the columns without the NPC were nailed with 12d nails. The columns were attached to the top plate using the NPCs and 0.5 inch diameter steel bolts (Figure 21). The bolts were tightened with a pneumatic wrench. The corner columns of all six walls were secured to the test setup using Simpson PHD6 anchors (Simpson Strong-Tie Catalog, 1999) with 1/2 inch diameter anchor bolts. Moreover, the sill plate was secured to the test setup with three 1/2 inch diameter anchor bolts spaced 48 inch on-center. Three by three inch square washers were installed with every bolt to prevent a premature shear wall degradation due to perpendicular-to-the-grain failure of the sill plate.

All six walls were sheathed with 5/8 inch thick lap joint Reverse Board-and-Batten plywood siding with grooves. The grooves were ¼ inch deep, 1-½ inch wide, and spaced 12 inch on-center. This product is a 24 inch on-center APA rated plywood siding (APA Product Guide, 1988). At the lap joint, two plywood sheets were attached to the frame member independently (Figure 19). The girts were spaced at 24 inch on-center.
The recessed girts for Type A walls were made of TimberStrand® and attached to the frame using Simpson A23 angles (Simpson Strong-Tie Catalog, 1999). The angles were attached to the frame members and girts with #8 x 1 ¼ inch tapping screws (Simpson Strong-Tie Catalog, 1999). Four screws were used in each leg of the angle. For the Type A walls, the sheathing was attached with 9 gage (ga) 2 inch, full round, flat-head zinc-coated wood screws with a self drilling point installed into the predrilled holes. The diameter of the holes was about the same diameter as the diameter at the root of the screw thread as required by the 1997 NDS (AF&PA 1998) for the elements with specific gravity greater than 0.6. An electric screwdriver with an automatic clutch system was used to install the screws. The automatic clutch system ensured consistent positioning of the screws and prevented plywood crushing during the installation. The fastening schedule was 6 inch on the perimeter and 12 inch in the field. The perimeter screws were installed with a ¾ inch edge distance. The choice of the screws for the sheathing fasteners was governed by two factors. First, the marketing studies (Platt 1999) showed that do-it-yourselfers often prefer using screws as opposed to nails. Second, the nails were difficult to drive into dense wood composites without using special tools such as a pneumatic nail gun. The screws were easily installed with an electric screwdriver into predrilled holes.

The on-top girts (Type B walls) were made of 2 x 4 x 96 inch nominal Stud grade SPF lumber. These girts were nailed to the frame with 10d smooth-shank-cement-coated
nails. Four nails were used for every connection. If two girts met on one column, two nails were used for each girt. The sheathing was attached to the purlins with 10d hot dipped galvanized nails. The fastening schedule was 6 inch on the perimeter and 12 inch in the field. The sheathing was attached only to the purlins. All the nails were hand driven. The perimeter nails were installed with ¾ inch edge distance.
Figure 20. Type A wall construction.

Figure 21. Connection details of the RTA shear wall frame. A – corner column to collar-tie. B – intermediate column to collar-tie. These details are valid for both Type A and B walls.
Figure 22. Type B wall assembly.

Figure 23 shows the shear wall test setup and instrumentation. The walls were tested in a horizontal position. The top plate was attached to the distribution beam with 0.5 inch diameter bolts spaced 24 inch on-center. Load was applied to the shear wall by displacing the distribution beam with a 55,000 lb capacity hydraulic actuator with ±6 inch travel range. The distribution beam was equipped with two casters that allowed it to move with minimum friction resistance. Displacement was applied according to a ramp function at a rate\(^3\) of 0.1 inch/min. This rate was chosen based on the recommendations of ASTM Standard E564-76 (ASTM 1990). The walls were displaced by at least 6 inches. Deformations of the wall were measured with a set of LVDT’s and string potentiometers. Displacement of the distribution beam was measured with a LVDT built into the hydraulic actuator. Displacement of the top plate relative to the concrete wall was measured with a string potentiometer (Figure 23, # 4). Displacement of the top plate was used for the data analysis. Slip of the sill plate at the bottom of the wall relative to the concrete wall was 

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\(^3\) Type B wall # 1 was tested at a rate of 0.6 inch/min due to instrumentation error.
measured by another string pot (Figure 23, # 1). Displacement of the sill plate was subtracted from the displacement of the top plate to compute the displacement of the top of the wall relative to the bottom of the wall. This adjusted data was used to plot the load-drift relationship for the shear walls. Vertical deformations of the corner columns relative to the concrete wall were measured with two LVDTs (Figure 23, # 2 and 3). The LVDTs were attached to the metal beam with the cores positioned against metal flanges attached to the sides of the columns. The actual position of each cores relative to the corresponding column was measured for each test. This information was necessary to correlate the LVDT readings to the actual displacement of the column center. The deformation of the plywood panels relative to the distribution beam was measured with LVDTs (Figure 23, # 5, 6, 7). These measurements were made only for Type B walls. The LVDTs were attached to the distribution beam with the cores positioned against metal brackets attached to the panels. The actual position of each core was recorded in order to correlate the LVDTs readings to the displacement of the corner of the panel. For Type B walls, the slip of the girt attached to the top plate was measured with a LVDT (Figure 23, # 8). Figure 24 is a photograph of the Type A shear wall setup.

An acquisition program (LabTech® v. 8.04, Laboratory Technologies Corporation©, 1994) was used to record the data at a sampling rate of 2 Hz.
Figure 23. Wall instrumentation. Top view. 1,4 – string pot; 2-3 and 5-8 – LVDT.
The shear wall tests were conducted in an outdoor shear wall testing facility located at the Brooks Forest Products Research Center. Therefore, the walls were subjected to the ambient environment. Typically, a wall was tested the day after it was constructed. The walls were covered with plastic to avoid contact with rain water. After the test, the moisture content of the frame members, girts, and siding was determined according to the primary oven-drying method (Method A) of ASTM Standard D4442-92 (ASTM 1998). Specific gravity of the specimens was determined from the moisture content sections according Method B (volume by water immersion) of ASTM Standard D2395-93 (ASTM 1998).

3.4.4 Definition of the shear wall performance parameters

Figure 25 illustrates some of the parameters used to evaluate wall performance. Wall capacity, $F_{\text{max}}$, was defined as the maximum load from the load-drift curve. Failure load was defined as the point on the load-drift curve at 80% of the wall capacity. Other
parameters were determined based on the equivalent energy elastic-plastic (EEEP) response curve (Salenikovich 2000). Proportional limit was defined as the load equal 40% of the wall capacity. Yield load was determined from the EEEP response curve by equalizing the energy dissipated by the wall until failure with the energy of the ideal elastic-plastic model. Detailed description of this method can be found in Salenikovich (2000). Elastic stiffness was determined as the slope of a straight line passing through the point with 0.005 inch displacement and the proportional limit. Due to the setup imperfections and instrumentation noise, the data below 0.005 inch displacement had little value for analysis and resulted in overestimated stiffness values for some specimens and unrealistic stiffness variability.

Figure 25. Performance parameters of the shear walls.

Performance of the walls beyond the elastic region was assessed using the ductility ratios and toughness of failure index. These parameters show the ability of the wall to undergo significant deformations while resisting high loads. There were two ductility ratios calculated (Equations [18] and [19]). The first was based on the peak drift, whereas the second was based on the failure drift.
ductility ratio\(_1\) = \(\frac{\Delta_{\text{peak}}}{\Delta_{\text{yield}}}\) \[18\]

\[\text{ductility ratio}\_2 = \frac{\Delta_{\text{failure}}}{\Delta_{\text{yield}}}\] \[19\]

where:
\(\Delta_{\text{peak}}\) = drift at maximum load;
\(\Delta_{\text{yield}}\) = drift at yield load;
\(\Delta_{\text{failure}}\) = drift at failure load.

The toughness of failure index was computed using Equation [20].

\[\text{toughness of failure index} = \frac{\Delta_{\text{failure}}}{\Delta_{\text{peak}}}\] \[20\]

### 3.5 Frame-sheathing connection test

#### 3.5.1 General

The objective of this test was to measure the load-deformation relationship of the sheathing fasteners used to construct the RTA shear walls. This information was used in the RTA shear wall FE model (Section 4.8.2). The sheathing connections for the Type A shear walls were investigated.

#### 3.5.2 Specimen preparation and instrumentation

The specimens consisted of a main and a side member connected with a 9 ga 2 inch full round flat head zinc-coated wood screws with self drilling point. The main member was either TimberStrand® 1.5 x 3.5 inch lumber or CCA pressure treated SP nominal 2 x 4 inch lumber. Side members were made of the Reverse Board-and-Batten plywood siding used in the shear wall tests. Screws were installed into predrilled holes. The diameter of the
holes was about the same as the diameter at the root of the screw thread as required by the 1997 NDS (AF&PA 1997) for the elements with specific gravity greater than 0.6. Specimens were manufactured using a fixture to ensure the geometrical consistency.

Specimens were prepared and tested with a MC of approximately 6-7% because both frame members and siding had this MC during the shear wall tests. The specimens were conditioned for 14 days in the Engineering laboratory to allow for stress relaxation. The sample size of 10 was adopted from the ASTM Standard D 1761 (ASTM 1998).

The significance of the following parameters on the connection performance was investigated: siding thickness, siding end distance, material of the main member, and orientation of the main member. The siding thickness was investigated because the sheathing had grooves ¼ inch deep and 1 ½ inch wide. Therefore, approximately half of the screws were installed in the grooves, and the other half in the full-thickness material. Siding end distance was investigated to measure the effect of the fastener position relative to the edge of the sheathing panel. Two cases were examined: 2 inch and ¾ inch. The material of the main member was investigated because the sheathing panels were attached to the frame members and to the sill plate which were made of TimberStrand® and CCA treated SP, respectively.

The RTA shear wall framing includes girts and columns made of TimberStrand® lumber. TimberStrand® is an anisotropic material. Therefore, the properties of the fasteners installed into the face of a TimberStrand® element are expected to be different as opposed to the fasteners installed into the edge of the same element. Orientation of the girts relative to the sheathing is consistent between all the walls in the RTA building. Girts are always oriented edgewise so that the sheathing is attached to the edge of the frame member. The orientation of the columns relative to the sheathing varies between shear walls that are parallel to the plane of the RTA frames and shear walls that are perpendicular to the RTA frames. The former shear walls are constructed by attaching the sheathing to the face of the TimberStrand® columns, whereas the latter shear walls are constructed by attaching the sheathing to the edge of the TimberStrand® columns. Therefore, the load-deformation relationship of the screws was measured for the both cases: the sheathing attached to the
face and to the edge of TimberStrand®. The above discussion on the shear wall construction is detailed and illustrated with the drawings and pictures in Section 4.7.

Based on the variables described above, twelve groups of specimens were tested for a total of one hundred and twenty specimens. Table 3 summarizes the main member material and orientation, plywood thickness, and end distance for each group. Groups 1-4 correspond to columns, groups 5-8 correspond to the girts, and groups 8-12 correspond to the sill plate.

Table 3. Test groups.

<table>
<thead>
<tr>
<th>#</th>
<th>Main member material</th>
<th>Main member orientation/ dir. of loading</th>
<th>Plywood (groove or regular)</th>
<th>End Distance, inch</th>
<th>Sketch</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>TimberStrand®</td>
<td>Face /</td>
<td></td>
<td>to the grain</td>
<td>regular</td>
</tr>
<tr>
<td>2</td>
<td>TimberStrand®</td>
<td>Face /</td>
<td></td>
<td>to the grain</td>
<td>regular</td>
</tr>
<tr>
<td>3</td>
<td>TimberStrand®</td>
<td>Face /</td>
<td></td>
<td>to the grain</td>
<td>groove</td>
</tr>
<tr>
<td>#</td>
<td>Main member material</td>
<td>Main member orientation/dir. of loading</td>
<td>Plywood (groove or regular)</td>
<td>End Distance, inch</td>
<td>Sketch</td>
</tr>
<tr>
<td>----</td>
<td>----------------------</td>
<td>----------------------------------------</td>
<td>----------------------------</td>
<td>-------------------</td>
<td>--------</td>
</tr>
<tr>
<td>4</td>
<td>TimberStrand®</td>
<td>Face / $\parallel$ to the grain</td>
<td>groove</td>
<td>¾</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>TimberStrand®</td>
<td>Edge / $\perp$ to the grain</td>
<td>Regular</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>TimberStrand®</td>
<td>Edge / $\perp$ to the grain</td>
<td>Groove</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>TimberStrand®</td>
<td>Edge / $\parallel$ to the grain</td>
<td>Regular</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>TimberStrand®</td>
<td>Edge / $\parallel$ to the grain</td>
<td>Groove</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>#</td>
<td>Main member material</td>
<td>Main member orientation/ dir. of loading</td>
<td>Plywood (groove or regular)</td>
<td>End Distance, inch</td>
<td>Sketch</td>
</tr>
<tr>
<td>----</td>
<td>---------------------------</td>
<td>------------------------------------------</td>
<td>-----------------------------</td>
<td>--------------------</td>
<td>--------</td>
</tr>
<tr>
<td>9</td>
<td>CCA pressure treated Southern Pine</td>
<td>Edge /</td>
<td></td>
<td>to the grain</td>
<td>Regular</td>
</tr>
<tr>
<td>10</td>
<td>CCA pressure treated Southern Pine</td>
<td>Edge /</td>
<td></td>
<td>to the grain</td>
<td>Groove</td>
</tr>
<tr>
<td>11</td>
<td>CCA pressure treated Southern Pine</td>
<td>Edge / ⊥ to the grain</td>
<td>Regular</td>
<td>¾</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>CCA pressure treated Southern Pine</td>
<td>Edge / ⊥ to the grain</td>
<td>Groove</td>
<td>¾</td>
<td></td>
</tr>
</tbody>
</table>

Figure 26.a shows the test setup for groups 1-4 and 7-10. Figure 26.b shows the test setup for groups 5-6 and 11-12. The specimens were loaded in tension to avoid buckling of the slender side member. Load was transferred from the specimen onto the load cell through a friction-based gripping fixture. The bolts of the gripping fixture were tightened using a pneumatic wrench to prevent slip of the specimen relative to the grip. The application of moment forces on the load cell was minimized by a three-dimensional
moment release positioned between the gripping fixture and the load cell. The gripping device was designed so that the center line on the load cell was collinear with the shear plane of the specimen. The test fixture was equipped with a set of rollers to minimize the friction forces imposed on the side member. In the case of groups 5-6 and 11-12, a plywood spacer was inserted between the main member and the top plate of the test fixture. This was done to mitigate the reinforcement of the specimen by the top plate.

![Diagram of test setup](image)

*Figure 26. Frame-sheathing connection test setup.*

The load was applied with a 20,000 lb capacity MTS universal testing machine at a displacement rate of 0.03 inch/min according to a ramp function. This loading rate was chosen according to the ASTM standard D 5652 (ASTM 1998) which prescribes the maximum load to be reached in approximately ten minutes. Preliminary tests were conducted to estimate the required loading rate. A displacement controlled protocol was used. Load was measured with a 1,000 lb capacity electronic load cell. Displacement of the
side member relative to the main member was measured using a set of two LVDTs with the maximum range of ± 0.5 inch. The LVDTs were attached to the side member of the specimen with an aluminum fixture. The cores of the LVDTs rested on the aluminum flanges attached to the sides of the main member (Figure 27). This setup allowed the LVDTs to measure the displacement of the side member relative to the main member.

![Figure 27. Setup for the sheathing fasteners test.](image)

LabTech® (Laboratory Technologies Corporation®, 1994) data acquisition program was used to record the data at a sampling rate of 2 Hz. The test was run until a major drop in the load occurred. An average value between the readings of the two LVDTs was used for the data analysis.
After the test, moisture content of both the side member and main member of the specimen was determined according to the primary oven-drying method (Method A) of ASTM Standard D 4442 (ASTM 1998). Specific gravity of the specimens was determined from the moisture content sections according Method B (volume by water immersion) of ASTM Standard D 2395 (ASTM 1998).

3.6 Finite element modeling of the RTA structures.

3.6.1 General

The finite element (FE) method was chosen to model the RTA frames and shear walls. Because of its versatility and advanced graphical interface allowing the user to visualize the response of complex structures, SAP2000 structural analysis program was employed. Moreover, SAP2000 has options for both linear and non-linear analyses. The latter feature is valuable for the analysis of wood structures which exhibit non-linear behavior. The response of two dimensional structures was modeled. The linear static analysis was used to model the linear response of the RTA frames, the nonlinear dynamic analysis was used to model the nonlinear static response of the RTA shear walls. The nonlinear dynamic analysis option had to be used to model the nonlinear static response of the shear walls due to limitations of the pushover analysis in SAP2000.

3.6.2 Elements

The following elements were used in the analysis: frame element, membrane element, linear spring element, and the non-linear spring element. The frame element formulation covers biaxial bending, biaxial shear, torsion and axial deformations. The frame element was used to model columns, beams, and truss elements. The membrane element activates two translational in-plane stiffness components and one rotational in-plane stiffness component. The membrane element was used to model the frame sheathing. The spring element was used to model connections. The spring can be assigned either linear or
nonlinear properties. The detailed descriptions and formulations of all these elements can be found in SAP2000 analysis reference (Computers and Structures, Inc., 1996).

3.6.3 Material properties

Frame members were assigned the modulus of elasticity of TimberStrand, $E = 1,500,000$ psi (Trus Joist, 1995). Metal connector plates were assigned the modulus of elasticity of steel, $E = 29,000,000$ psi (Computers and Structures Inc., 1999). Shell elements were assigned the shear modulus of 45,470 psi. The shear modulus was determined from the panel shear through the thickness data (APA The Engineered Wood Association, 1999).

3.6.4 Linear static analysis of the RTA frames

As shown in Section 2.7, the connections in wood structures exhibit semi-rigid behavior which should be incorporated in the analysis procedure for accurate modeling of these structures. In this analysis, the NPCs were modeled as a set of zero-length independent springs. The number of springs required to model a connector is equal to the number of active degrees of freedom (DOF) of this connector. For the two-dimensional analysis, the NPC has three active DOF: two translational and one rotational. Therefore, three spring elements are required to model each NPC (Figure 28).
Figure 28. Set of three zero-length springs used to model the NPC.

The spring force is proportional to the spring deformation (Equation [21]). The spring is assigned with the stiffness of the connector in the corresponding DOF.

\[ f = k \cdot d \]  

[21]

where:

- \( f \) = spring force, units of force;
- \( k \) = spring constant, units of force/deformation;
- \( d \) = deformation, units of deformation.

A typical frame joint of a RTA structure includes two or three NPCs which are connected with bolts. This example shows the rafter-to-column joint with two NPCs. The model of this joint incorporates two sets of spring elements which connect four beam elements (Figure 29).
The spring elements are located at the centroid of the NPC. Each set of springs connect a beam element, which models a frame member, to a beam element which models an extension plate of a NPC. The extension plates are rigidly connected to each other. If the plates would be connected with a single bolt, the rigid joint should be changed to a pin. The length of the beam element which models the extension plate of the NPC is equal to the distance between the centroid of the NPC and the centroid of the bolted joint.

The static analysis involves solution of the system of linear equation (Equation [22]).

$$K \ u = r$$  \hfill [22]$

where:

- $K$ = stiffness matrix of the system;
- $r$ = vector of the applied loads;
- $u$ = vector of the resulting displacements.
3.6.5 **Linear and nonlinear analysis of the RTA shear walls**

The response of the shear walls is governed by two factors: the sheathing-to-frame connections and anchorage methods. Typically, the sheathing is connected to the frame with small diameter dowel fasteners such as nails, staples, or screws. Because these fasteners have a ductile component in response to load, and because the forces are distributed unevenly between the fasteners, the shear walls begins to exhibit nonlinear behavior at small deformations. Therefore, a shear wall model should incorporate the nonlinear response of sheathing connections. However, the linear analysis can also provide insights into shear wall performance within the initial range of deformations.

Type A RTA shear walls are assembled with screws. In the linear analysis, each screw is modeled as a set of two zero-length, independent linear translational springs. The spring element formulation is described with Equation [21]. The system model is formulated with Equation [22]. In the nonlinear analysis, each screw is modeled as a set of two zero-length, independent nonlinear translational springs. The springs are assigned with the nonlinear load-deformation relationship of the screw in the corresponding directions of loading. This nonlinear load-deformation relationship is formulated using the Wen model (Equations [23] and [24]).

\[
p = r \ k \ d + (1 - r) \ p_y \ z \tag{23}
\]

\[
d = \frac{z}{1 - |z|^n} \ \frac{p_y}{k} \tag{24}
\]

where:

- \(p\) = spring force;
- \(r\) = specified ratio of post-yield stiffness to elastic stiffness (k);
- \(k\) = elastic stiffness;
- \(d\) = deformation;
- \(p_y\) = yield force;
- \(z\) = internal hysteretic variable, it varies within a range of \(|z| \leq 1\);
  - \(z = 1\) corresponds to the yield line;
- \(n\) = exponent, which has a range of \(n \geq 1\) and influences smoothness of
the load-deformation curve (Figure 30).

Figure 30. Wen model.

The frame of the shear wall was modeled using the methods described in Section 3.6.4 with the exception of the nonlinear analysis when the NPCs were modeled using the formulation described with Equations [23] and [24]. The parameters of Equations [23] and [24] for the screws were determined from the experimental data (Section 4.6). The same parameters for the NPC were determined from the moment-rotation test (Section 4.1) and the load-translation test (Platt 1998). Due to the complexity of the Wen formulation, the commercially available statistical software packages (SAS®, TableCurve™) failed to fit Equations [23] and [24] to the experimental data. To accomplish this task, the parametric Foschi equation (Foschi, 1974) was fit to the data using TableCurve™ 2D v4 software package (SPSS Inc., 1996). Then, using a spreadsheet program the parameters of the Wen formulation were selected so that the line defined with the Foschi equation (Equation [25], Figure 31) coincided with the line defined with the Wen formulation. The computation of force, p, using the Wen formulation involved three steps: (1) range of z from 0 to 1 was specified, (2) using Equation [24] the deformations, d, were computed, and (3) the z values and the corresponding d values were used with Equation [23]. The accuracy of this method
was limited by the ability of the author to resolve two lines on the computer display (800 x 600 pixels), which should be sufficient to meet the engineering objectives of the modeling.

\[
P = (P_0 + K_1 \Delta) \left[ 1 - \exp \left( \frac{-K_0 \Delta}{P_0} \right) \right]
\]

where:

- \( P \) = force applied on the connector, \( P \leq P_{\text{max}} \);
- \( P_0 \) = force intercept of the line tangent to the curve at \( \Delta_{\text{peak}} \);
- \( K_0 \) = slope of the curve at the origin;
- \( K_1 \) = slope of the curve at \( \Delta_{\text{peak}} \);
- \( \Delta \) = deformation.

![Figure 31. Force-deformation relationship described by the Foschi equation.](image)

The nonlinear time-history analysis was used. This analysis involves solution of the system of the nonlinear dynamic equilibrium equations (Equation [26]).

\[
K_L \ddot{u}(t) + C u(t) + M \dot{u}(t) + r_N(t) = r(t)
\]

where:

- \( K_L \) = stiffness matrix for all linear elastic elements;
- \( C \) = proportional damping matrix;
\begin{align*}
M & = \text{diagonal mass matrix}; \\
r_N & = \text{vector of forces from the nonlinear spring elements}; \\
r & = \text{vector of the applied loads}; \\
u, \overset{.}{u}, \text{and } \overset{..}{u} & = \text{displacements, velocities, and accelerations}.
\end{align*}

The solution of Equation [26] was obtained using the mode superposition method. The modes were found using the Ritz-vectors method. The number of modes used for the analysis was equal to the number of the nonlinear degrees of freedom. This number of the modes was needed for the solution to converge. Because the inertia effect was not of interest the load was applied using a ramp function (Equation [27], Figure 32).

\[ r(t) = f(t) p \]  \hspace{1cm} [27]

where:

\begin{align*}
   r(t) & = \text{load vector at a given time}; \\
f(t) & = \text{linear time function}; \\
t & = \text{time}; \\
p & = \text{maximum load vector}.
\end{align*}
Chapter 3  Materials and Methods

Figure 32. Ramp function.

The model damping of 0.99 was used to further prevent the inertia effect from influencing the analysis.

Accurate modeling of the nonlinear shear wall behavior requires coupling the response of the independent springs. The principle of linear superposition becomes inapplicable beyond the linear performance. Because SAP2000 lacks the capability for direct coupling of the spring forces, an adjustment method is proposed for indirect modeling of the coupled springs. Figure 33 shows the geometry involved in this adjustment method. Axes x and y coincide with the axes of the horizontal and vertical springs, respectively. The fastener head travels along a line at angle $\alpha$ from the x-axis. Assume that the fastener starts yielding when it reaches deformation $\Delta$. $x_1$ and $y_1$ are the corresponding coordinates on x- and y-axes, respectively. However, in the model the springs do not yield until their deformations along the corresponding axes reaches the same value of $\Delta$. Therefore, the horizontal and vertical springs will yield when the projections of the actual fastener deformation on the corresponding axes will reach the values found with Equations [28] and [29], respectively.
Because the denominator is always less than unity, the model with independent springs yields conservative solutions. Therefore, the yield displacement of both springs should be adjusted. The degree of this adjustment depends on the angle $\alpha$ between the resultant vector and its horizontal projection. In the model with uncoupled springs, the error is maximum for the angle $\alpha = 45^\circ$, whereas the error is zero for the angle $\alpha = 0^\circ$. In a shear wall panel, the angle between the actual travel of the sheathing fasteners and its projection depends on the position of the fastener in the panel. Results of the FE modeling (Section 4.8.2) showed that the angle approaches $45^\circ$ at the corners of the panel and $0^\circ$ in the middle of the panel. Therefore, accurate modeling requires different adjustments of spring properties for each
fastener. The number of the fasteners and the limitations of the program input make this task difficult.

Because the corner fasteners undergo higher deformations as compared with the middle ones, the corner fasteners enter the plastic region first. Moreover, sensitivity studies showed that the yield load of the shear wall is sensitive to the presence and characteristics of the corner fasteners. Therefore, it is proposed to formulate the adjustment method based on the geometry of the corner fasteners. The angle $\alpha = 45^\circ$ was used. This assumption limited the analysis to the range of displacements from zero to the initial part of the nonlinear region. This model should allow for estimating the shear wall response within the linear and the initial nonlinear performance.

The adjustment method is exemplified in Figure 34. The original line has a yield load of 500 lb. The intersection between the initial slope line and the plastic slope line of the original curve is assumed to be the yield point and designated as 1. As it was shown previously, the position of this yield point should be adjusted. This adjustment is done by multiplying the displacement coordinate of the intersection point 1 by the factor of $\cos(45^\circ) = 0.707$. The point on the initial slope line with this new coordinate is designated as 1’. The new plastic slope line is defined as a line which intersects the initial slope line at point 1’ and is parallel to the original plastic slope line. Thus, the adjusted line is positioned below the original line and it has the yield load value of 355 lb.
Figure 34. Adjustment method for the spring load-deformation relationship.

This adjustment method introduces no error if the angle between the resultant vector of the sheathing fastener and its horizontal projection is 45°. The error between the actual force and the force found using this adjustment method increases as the angle approaches zero. Figure 35 shows the error introduced by the adjustment method. For the range of angles between 45° and 20° the error is within 7.8%. The error is maximum (23%) if the resultant vector is parallel to one of the axes (α = 0°). Therefore, the fasteners in the middle of the panel will enter the plastic region prematurely leading to decreased estimated shear wall strength and stiffness. In contrast, the fasteners support the load beyond their practical capacity because SAP2000 does not allow for a reduction of the spring force past the peak load. Instead, the spring element models the fastener as infinitely strong resulting in a greater estimated shear wall strength and stiffness.
Figure 35. Error introduced by the adjustment method for different angles.
Chapter 4

Results and Discussion

4.1 General

This chapter presents the results of the project including: moment-rotation tests of the NPC, bearing tests of the NPC plate on the end grain of TimberStrand®, nonlinear modeling of the moment-rotation relationship of the NPC, RTA shear wall tests, frame-sheathing connection tests, fabrication and construction of a demonstration RTA building, and FE analysis of the RTA structures. The results are discussed and conclusions are drawn in respect to the objectives of the study. A design example of a RTA building is presented in Appendix C.

4.2 Moment-rotation test of the NPC

Results of the moment-rotation test are summarized in Tables 4, 5, and 6, and graphically shown in Figures 36, 37, and 38 for TimberStrand®, Parallam®, and SPF groups, respectively. The parametric Foschi equation was fit to the data for every group. Ninety-five percent confidence intervals are also determined. Design values for the lateral resistance of the dowel connections, as established in the 1997 NDS, are based on 5% offset limit. This rule can not be applied directly to the moment-rotation relationship because the x-axis contains angles instead of displacements. It is proposed to apply the 5% offset rule to the nail which is the most remote from the centroid of the NPC. Therefore, the 5% offset angle for the NPC can be computed using Equation [30].

$$5\% \text{ offset angle} = \arctan \frac{0.05 \, d_n}{r_R} = \arctan \frac{0.05 \times 0.25}{1.16} = 0.011 \, \text{rad} \quad [30]$$

where:
arctan = arctangent function;

\( d_n = 0.25 \text{ inch} = \text{nail diameter}; \)

\( r_R = 1.16 \) = distance between the most remote nail and centroid of the NPC.

---

**Figure 36. Moment-rotation relationship for the NPC in TimberStrand®.**
Figure 37. Moment-rotation relationship for the NPC in Parallam®.

Figure 38. Moment-rotation relationship for the NPC in SPF.
Table 4. Data summary for the TimberStrand® group.

<table>
<thead>
<tr>
<th>#</th>
<th>Moisture Content, %</th>
<th>Specific Gravity</th>
<th>Max Moment, lb-in</th>
<th>( \varphi_{\text{peak}} ), rad</th>
<th>M_5%, lb-in</th>
<th>Linear Stiffness, lb-in/rad</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>7.29%</td>
<td>0.73</td>
<td>15,168</td>
<td>0.169</td>
<td>10,407</td>
<td>669,383</td>
</tr>
<tr>
<td>St. dev.</td>
<td>0.21%</td>
<td>0.04</td>
<td>700.9</td>
<td>0.013</td>
<td>747.5</td>
<td>104,731</td>
</tr>
<tr>
<td>COV, %</td>
<td>2.87%</td>
<td>5.30%</td>
<td>4.62%</td>
<td>7.89%</td>
<td>7.18%</td>
<td>15.65%</td>
</tr>
</tbody>
</table>

Table 5. Data summary for the Parallam® group.

<table>
<thead>
<tr>
<th>#</th>
<th>Moisture Content, %</th>
<th>Specific Gravity</th>
<th>Max Moment, lb-in</th>
<th>( \varphi_{\text{peak}} ), rad</th>
<th>M_5%, lb-in</th>
<th>Linear Stiffness, lb-in/rad</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>7.55%</td>
<td>0.68</td>
<td>11,905</td>
<td>0.149</td>
<td>8,270</td>
<td>515,709</td>
</tr>
<tr>
<td>St. dev.</td>
<td>0.41%</td>
<td>0.03</td>
<td>1,066.2</td>
<td>0.016</td>
<td>1,358</td>
<td>133,947</td>
</tr>
<tr>
<td>COV, %</td>
<td>5.38%</td>
<td>4.38%</td>
<td>8.96%</td>
<td>10.45%</td>
<td>16.42%</td>
<td>25.97%</td>
</tr>
</tbody>
</table>

Table 6. Data summary for the SPF group.

<table>
<thead>
<tr>
<th>#</th>
<th>Moisture Content, %</th>
<th>Specific Gravity</th>
<th>Max Moment, lb-in</th>
<th>( \varphi_{\text{peak}} ), rad</th>
<th>M_5%, lb-in</th>
<th>Linear Stiffness, lb-in/rad</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>8.83%</td>
<td>0.42</td>
<td>8,843</td>
<td>0.178</td>
<td>5,349</td>
<td>231,781</td>
</tr>
<tr>
<td>St. dev.</td>
<td>0.78%</td>
<td>0.04</td>
<td>736</td>
<td>0.037</td>
<td>623</td>
<td>42,573</td>
</tr>
<tr>
<td>COV, %</td>
<td>8.88%</td>
<td>9.08%</td>
<td>8.32%</td>
<td>20.95%</td>
<td>11.65%</td>
<td>18.37%</td>
</tr>
</tbody>
</table>

The Foschi equation was fit using TableCurve™ 2D v4 (SPSS Inc., 1996) software package. Because the Foschi equation is formulated to model the load-deformation relationship between zero and the maximum load, the data from 0 to \( \varphi_{\text{peak}} \) was used for the curve fitting. \( \varphi_{\text{peak}} \) was defined as a rotational angle at the maximum moment or at the maximum moment less 50 lb-in, whichever is greater. The reduction was done to account for the electronic noise associated with the data acquisition system. Linear stiffness was defined as a slope of a straight line which goes through the origin and through the point on the moment-rotation curve with the moment equal to 40% of the maximum moment.

Table 7 summarizes the parameters for the Foschi equation for the three groups. The correlation coefficient (\( R^2 \)) is specified for each group. The TimberStrand® group had
the highest initial stiffness, 880,000 lb-in/rad, the highest yield moment, 12,500 lb-in, and the second highest post-yield stiffness, 13,000 lb-in/rad. The TimberStrand® group had the lowest variability, $R^2 = 0.957$. The Parallam group had the second highest initial stiffness, 691,700 lb-in/rad, and yield moment, 9,900 lb-in, whereas it had the lowest post-yield stiffness, 12,500 lb-in/rad, and the highest variability, $R^2 = 0.864$. The SPF group showed the lowest initial stiffness, 326,450 lb-in/rad, and yield moment, 6,250 lb-in, whereas the post-yield stiffness had the highest value, 14,500 lb-in/rad. The variability of the SPF group fell between the other two groups, $R^2 = 0.936$.

Table 7. The parameters for the Foschi equation and $R^2$ parameter for each group.

<table>
<thead>
<tr>
<th>Group</th>
<th>$K_0$, lb-in/rad</th>
<th>$K_1$, lb-in/rad</th>
<th>$M_0$, lb-in</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>TimberStrand®</td>
<td>880,000</td>
<td>13,000</td>
<td>12,500</td>
<td>0.957</td>
</tr>
<tr>
<td>Parallam®</td>
<td>691,700</td>
<td>12,500</td>
<td>9,900</td>
<td>0.864</td>
</tr>
<tr>
<td>SPF</td>
<td>326,450</td>
<td>14,500</td>
<td>6,250</td>
<td>0.936</td>
</tr>
</tbody>
</table>

Parallam® specimens showed the highest variability among the three groups. One of the reasons was that some of the Parallam® specimens were made of two pieces with depth slightly varying along the length. The variation in the depth was measured as up to 1/16 inch. Initial load was applied to deepest piece making the nails that were in this piece yield first, whereas the nails in the second piece were within the linear performance range. When the gap between the wooden block and the narrow piece was closed, the two pieces deformed identically. Parallam® specimens had nonuniform depth because they were fabricated from 2 x 10 inch lumber. The 2 x 10 pieces were ripped using a table saw with manual feeding that did not provide adequate high accuracy. The increased variability of the results for the Parallam® group was also associated with inconsistent failure modes. Failure modes are discussed later in this section.

All three groups exhibited a similar moment-rotation relationship with a short elastic range and extensive plastic deformations. The plastic deformations were associated with the development of plastic hinges in the nails, wood crushing under the nails, and wood crushing under the plate bearing on the end grain (Figure 39). Rotation of the NPC also led
to the development of a gap between the two pieces. However, all three groups exhibited different failure modes. TimberStrand® specimens failed due to nail yielding and wood crushing. Parallam® specimens 3, 5, 6, 7, 8, 9, 11, 12, 13, 14, and 15 failed in the same mode as the TimberStrand® specimens, whereas specimens 1, 2, 4, and 10 failed in tension perpendicular-to-the-grain. The tension perpendicular-to-the-grain failure mode was evidenced by a crack on the end grain on the specimens. All SPF specimens failed in tension perpendicular-to-the-grain (Figure 40).

Figure 39. The failure mode of the NPC in TimberStrand®.
Figure 40. Brittle failure mode due tension perpendicular-to-the-grain observed in Parallam (left) and SPF (right).
4.3 Testing of the plate bearing on the end grain of TimberStrand®

Figure 41 shows the load-deformation curves for the plate bearing on the end grain of TimberStrand® for 10 specimens and a statistical model fit to the data. Table 8 summarizes the test results. Because the displacement was measured using the LVDT built into the hydraulic actuator, the original data was adjusted to exclude the error due to the initial slack in the setup (Figure 42). The adjustment was performed as follows: the data in the range between 1000 lb and the yield load was used to fit a straight line, the fitted equation was solved for $\Delta x$, an intersection point between the fitted line and the original data line was found, the data points below this intersection point were excluded from the analysis and replaced with the fitted line, all data points were shifted to the left by $\Delta x$. This modified data was used in the analysis.

![Load-displacement relationship for the plate bearing on the end grain of Timberstrand](image)

*Figure 41. Load-displacement relationship for the plate bearing on the end grain of TimberStrand®.*
Figure 42. Method used to adjust the data for the plate bearing on the end grain test.

Table 8. Test results for the plate bearing on the end grain of TimberStrand®.

<table>
<thead>
<tr>
<th></th>
<th>Moisture Content, %</th>
<th>Specific Gravity</th>
<th>Max Load, lb</th>
<th>Yield Load, lb</th>
<th>Displacement @ the Yield Load, inch</th>
<th>Linear Stiffness, lb/in</th>
<th>Post Yield Stiffness, lb/in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>7.02%</td>
<td>0.73</td>
<td>4,728</td>
<td>4,689</td>
<td>0.052</td>
<td>97,573</td>
<td>-4,833</td>
</tr>
<tr>
<td>St. dev.</td>
<td>0.33%</td>
<td>0.04</td>
<td>359.1</td>
<td>360.0</td>
<td>0.006</td>
<td>6,791</td>
<td>1,757</td>
</tr>
<tr>
<td>COV, %</td>
<td>4.77%</td>
<td>5.49%</td>
<td>7.60%</td>
<td>7.68%</td>
<td>10.64%</td>
<td>6.96%</td>
<td>36.37%</td>
</tr>
</tbody>
</table>

Maximum load was defined as a peak load value from the data set for each specimen. Yield load was defined as a load before the first drop on the load-deformation chart. For some specimens the maximum and yield load coincided, whereas for the other specimens the load recovered after the first drop and reached a higher value later in the plastic region.

The load-deformation relationship for all specimens consisted of two different regions. The first region covered the load range from zero to the yield load. The first region was well described by a straight line with a $R^2$ value for all specimens above 0.98. The slope of this line was referred to as the linear stiffness. The second region covered the load range from the yield load to the failure load. Failure load was defined as a load at the end of the test. The test was stopped if the displacement exceeded 0.30 inches or the specimen cleaved...
parallel-to-the-grain, whichever happened first. The second region was described by a straight line with a negative slope. This slope was referred to as the post-yield stiffness. The $R^2$ value of the straight line for the second region fell between 0.57 and 0.98. Eight specimens had had $R^2$ above 0.82, whereas the remaining two specimens had $R^2$ of 0.73 and 0.57, respectively. Although the specimen with $R^2$ of 0.57 showed weak linear trend, the maximum error between the linear model and experimental data was within 6.9%.

Although the experimental data showed a decrease in the plate bearing strength after the yield load was reached, this loss of resistance was associated with the splitting of the specimens due to tension perpendicular-to-the-grain failure mode. The effect of this failure mode was minimized but not eliminated by loading the specimens at an oblique angle on the end grain of the specimen (Section 3.3). Theoretically, the crushing strength should not significantly decrease after the yield load. Because visual observation of the specimens from the moment-rotation test showed no evidence of wood splitting under the connector plate, the elastic-plastic model (Figure 43) was chosen to describe the load-displacement relationship for the plate bearing on the end grain. The elastic-plastic response was modeled with a bilinear function (Equation [31]). This formulation resulted in a simple solution of Equation [32].

\[
\begin{align*}
    p_b &= k_L x & \text{if } x \leq x_1 \\
    p_b &= p_y & \text{if } x > x_1
\end{align*}
\]

[31]

where:

- $p_b$ = bearing force, lb;
- $x$ = displacement of the plate, inch;
- $k_L$ = the linear slope, lb/in;
- $p_y$ = the yield load, lb;
- $x_1$ = the $x$ coordinate of the yield load, inch.
The parameters of the elastic-plastic model were determined using TableCurve™ 2D v4 (SPSS Inc., 1996) software package. First, the data was sectioned into two regions. The first region covered the elastic part of the data and the second region covered the plastic part of the data. Straight lines were fit to both regions. The intersection point between the two lines was found. \( X_1 \) and \( P_y \) were assigned the values of abscissa and ordinate of the intersection point, respectively. The results of this statistical analysis are summarized in Table 9. Note that the value of the yield load from the elastic-plastic model, \( P_y \) (Table 9), and the average yield load value (Table 8) are different. They were determined using two different methods.

**Table 9. Parameters of the elastic-plastic model.**

<table>
<thead>
<tr>
<th>Function Parameters</th>
<th>Estimate</th>
</tr>
</thead>
<tbody>
<tr>
<td>( K_L ), lb/inch</td>
<td>97,573</td>
</tr>
<tr>
<td>( P_y ), lb</td>
<td>4,805</td>
</tr>
<tr>
<td>( X_1 ), inch</td>
<td>0.049</td>
</tr>
</tbody>
</table>
4.4 Non-linear modeling of the moment-rotation relationship of the nail plate connector

4.4.1 General

The multiple bolted joints used to connect the RTA frame members and the eccentric position of the NPC relative to the center of the frame joint introduce moment forces in the NPC (Figure 44). The structural design of the RTA frames requires input of the moment resistance for a variety of the NPCs.

![Image of eave joint](image)

*Figure 44. Eave joint of the RTA frame without collar-tie.*

An experimental study was conducted to quantify the response of the NPC with five nails under moment loading (Section 3.1). Results of the test showed the nonlinear behavior of the NPC. Therefore, there was no direct correlation between the number of nails and the NPC resistance to the rotational forces.

Structural analysis of a variety of the frames assembled using the NPCs with different number of nails and nails arrangements require an analytical model that can predict the rotational strength and stiffness of these connections. The model should incorporate both the resistance of the nails and bearing of the plate on the end grain of the frame member.
The objective of this modeling was to derive a nonlinear analytical model capable of predicting the NPC strength and stiffness under the rotational forces.

4.4.2 Scope and limitations

The model formulation is based on the assumption that the nonlinear load-displacement relationship of a single nail and the nonlinear load-deformation relationship of the plate bearing on the end grain of the wood member are known. The lateral response of the single nail is derived from the tests of the NPCs with five nails loaded parallel-to-the-grain (Platt 1998) by dividing the load values by a factor of five. It is assumed that the group action factor had no effect on this particular design of the NPC (Section 2.4.2). Furthermore, the grain orientation effect is ignored (Section 2.4.1). The response of the plate bearing on the end grain of the wood member was measured experimentally (Section 4.3). The model was validated for the response of the NPC in TimberStrand®.
4.4.3 Model formulation

Under pure moment loading the NPC deforms as depicted in Figure 45. The displacement of a single nail within the NPC detailed in Figure 46.

\[ \varphi = \text{rotation angle} \]
\[ \Delta = \text{horizontal projection of the lateral displacement of the corner of the NPC} \]
\[ M = \text{moment acting on the NPC} \]
\[ C = \text{centroid of the NPC} \]
\[ i = 1-5 = \text{nail number before deformation} \]
\[ j = 1'-5' = \text{nail number after deformation} \]
\[ h = \text{height of the bearing surface} \]

*Figure 45. The NPC subjected to pure moment loading.*
The model is formulated using the energy conservation principle: the work done by the external forces, $W_E$, is equal to the work done by the internal forces, $W_I$ (Equation [32]).

$$W_E = W_I$$

The external force acting on the NPC is moment, $M$, and the work done by this moment is a scalar product of the moment value and the rotation angle, $\varphi$. Because the moment is a nonlinear function of the rotation angle, the integral of this product should be used (Equation [33]).

$$W_E = \int_0^{\varphi_{\text{max}}} M \, d\varphi$$

There are two types of internal forces present in the NPC. The first is the force resisted by the nails, and the second is the force resisted by the plate bearing on the end grain of the wood member. Therefore, the internal work consists of the work done by the nails, $W_N$, and the work done the plate, $W_b$.

$$W_I = W_N + W_b$$

Solving Equation [33] for the moment force and substituting into Equation [34], the moment resisted by the NPC is found (Equation [35]).
The work done by the nails, $W_N$, is equal to the sum of the work done by each nail. Work done by a nail is a scalar product of the force, $p_t$, resisted by this nail and its displacement, $\delta$. Nail force is a nonlinear function of the displacement. Therefore, integral of this product should be used (Equation [36]).

$$W_N = \sum_{\text{nails}} \delta_{\max} \left( \int_0^\delta p_t \, d\delta \right)$$  \hspace{1cm} [36]

To enable this integration, the nail load-displacement relationship should be described with a mathematical function. Initially, three functions were considered: power function, logarithmic function, and the Foschi equation.

- **Power function:**
  $$p_t = A \delta^B$$  \hspace{1cm} [37]

- **Logarithmic function:**
  $$p_t = A \ln (1 + B \delta)$$  \hspace{1cm} [38]

- **Foschi equation:**
  $$p_t = (p_0 + k_1 \delta) \left[ 1 - \exp\left( -\frac{k_0 \delta}{p_0} \right) \right]$$  \hspace{1cm} [39]

Although power function is easy to integrate and would result in a simple final formulation, it significantly overestimates the initial stiffness. Because most of the static analyses are performed within the linear region, the power function was rejected. The logarithmic function proposed by McLain (1975) for modeling the nail slip up to 0.1 inch overestimates stiffness at higher deformations. The model based on this function would have limited application for the connectors with complex nail arrangements. The Foschi equation (Foschi 1974) fits the data well throughout the entire range of displacements. However, the complex formulation of this function leads to a very cumbersome solution of Equation [35]. A modified logarithmic function is proposed (Equation [40]).

$$p_t = A \ln (1 + B \delta) - C \delta$$  \hspace{1cm} [40]
This function predicts the load-slip relationship of the NPC well throughout the entire range of the displacements and yields a relatively simple solution of Equation [35] as compared to the Foschi equation. The modified logarithmic equation was fit to the data measured by Platt (1998) using TableCurve™ 2D v4 (SPSS Inc., 1996) software package. Table 10 summarizes the results.

Table 10. Estimates of the parameters of Equation [40].

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Estimate</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>691.8</td>
</tr>
<tr>
<td>B</td>
<td>393.7</td>
</tr>
<tr>
<td>C</td>
<td>3649.7</td>
</tr>
</tbody>
</table>

Substitution of Equation [40] into Equation [36] and integration of the latter for \( \delta \) results in the formulation of the work done by the nails in terms of the nail displacements (Equation [41]). Note, Mathematica®4 (Wolfram Research, Inc., 1999) software package was used for integration and differentiation of the mathematical equations.

\[
W_N = \sum_{nails} \delta_{\text{max}} \int_0^{\delta_{\text{max}}} (A \ln(1 + B \delta) - C \delta) \, d\delta =
\]

\[
= \sum_{nails} \{-A\delta - 0.5C\delta^2 + \frac{A}{B} \ln(1 + B\delta) + A\delta \ln(1 + b\delta)\}
\]

To use Equation [41] with Equation [35], \( \delta \) is expressed in terms of \( \phi \) (Equation [42]). The geometry involved in this formulation is depicted in Figure 46.

\[
\delta = r \cdot \frac{2 \tan \frac{\phi}{2}}{2}
\]

For small angles (\( \phi < 15^\circ \)), the tangent of an angle can be assumed equal to the value of this angle. This assumption results in the maximum error of 2.5% for 15\(^\circ\) angle. Thus, Equation [42] can be further modified to simplify the followup differentiation (Equation [43]).
\[ \delta = r \varphi \quad [43] \]

To compute the moment resisted by the nails, \( M_N \), Equation [41] is used with Equation [43] and differentiated by \( \varphi \).

\[
M_N = \frac{d}{d \varphi} \frac{W_N}{\varphi} = \frac{1}{\varphi} \sum_{\text{nails}} \left\{ -A r_i \varphi - 0.5 C r_i^2 \varphi^2 + \frac{A}{B} \ln(1 + B r_i \varphi) + A r_i \varphi \ln(1 + B r_i \varphi) \right\} = \\
\sum_{\text{nails}} \left\{ -A r_i - C r_i^2 \varphi + \frac{A r_i}{1 + B r_i \varphi} + \frac{A B r_i^2 \varphi}{1 + B r_i \varphi} + A r_i \ln(1 + B r_i \varphi) \right\} \quad [44]
\]

The work done by the plate bearing on the end grain, \( W_b \), is a scalar product of the resultant bearing force and the displacement of the point at which this resultant force is applied. Both the value of the resultant force and its position are nonlinear functions of the rotation angle of the plate. Figure 47 shows a diagram of the plate bearing on the end grain. To simplify the derivation, it is assumed that the plate rotates around the middle of its side (point C on the diagram). The bearing force, \( p_b \), changes from zero at point C to its maximum value at the corner of the plate according to an unknown function. The bearing surface is divided into \( n \) segments with a finite length \( \Delta Z \). It is further assumed that the bearing force within each segment is constant for any given rotation angle. The total work done the bearing force is equal to the sum of the work done by the bearing forces in each segment (Equation [45]). The work done by the bearing force within a segment is the product of the bearing force value and displacement, \( x \), of the point at which this force is applied. Because \( p_b \) is a nonlinear function of \( x \) (Equation [31]), the integral from zero to the maximum displacement, \( \Delta \), is used. \( p_b \) (Equation [45]) has units of force per length. Therefore, \( p_b \) is multiplied by \( \Delta Z \). The relationship between the bearing force, \( p_b \), and plate displacement, \( x \), was measured experimentally as described in Section 4.3.
Using the rules of integration, Equation [45] can be modified (Equation [46]).
After substitution of Equation [31] into Equation [46], the inner integral is computed as a sum of two integrals (Equation [47]). To avoid confusion with the other parameters used in this thesis, parameters $K_L$ and $P_y$ from Equation [31] are notated $D$ and $E$, respectively.

$$W_b = \sum_{i=1}^{n} \int_{0}^{\Delta} p_i \, dx \cdot \Delta Z = \int_{0}^{h} p \, dx \, dz$$  \hspace{1cm} [46]$$

$$\int_{0}^{\Delta} p_b \, dx = \int_{0}^{x_j} D \, x \, dx + \int_{x_1}^{\Delta} E \, dx = \frac{D x_1^2}{2} + E (\Delta - x_1)$$  \hspace{1cm} [47]$$

To compute the outer integral of Equation [46], two scenarios should be considered: (1) the rotation angle, $\phi$, is small so that only elastic deformations occur, and (2) both elastic and plastic deformations occur. In the first scenario, only the first term from Equation [47] is required to calculate $W_b$, whereas in the second scenario both terms are required. Figure 48 shows a diagram which divides the bearing area into two regions according to the deformations. The first region covers elastic deformations and the second region covers plastic deformations.

\[ 1 \text{ = region of elastic deformations} \]
\[ 2 \text{ = region of plastic elastic deformations} \]

\textit{Figure 48. Regions with different types of deformations.}
The first scenario is described mathematically with respect to the rotation angle with Equation [48].

\[ \varphi \leq \frac{x_1}{h} \]  \hspace{1cm} [48]

Therefore, region 1 (Figure 48) exists only. The internal energy, \( W_b \), for this scenario is computed using the first term of Equation [47].

\[ W_b = \int_0^h D \frac{x^2}{2} dz \]  \hspace{1cm} [49]

To enable integration of this equation, translational displacement, \( x \), is expressed in terms of rotational deformation, \( \varphi \), using the small angle theory.

\[ x = \varphi z \]  \hspace{1cm} [50]

\[ W_b = \int_0^h \frac{D(\varphi z)^2}{2} dz = \frac{Dh^3 \varphi^2}{6} \]  \hspace{1cm} [51]

The moment, \( M_b \), resisted by the bearing of the plate is computed with Equation [52].

\[ M_b = \frac{d}{d\varphi} W_b = \frac{d}{d\varphi} \left( \frac{Dh^3 \varphi^2}{6} \right) = \frac{Dh^3 \varphi}{3} \]  \hspace{1cm} [52]

The second scenario is described mathematically with respect to the rotation angle with Equation [53].

\[ \varphi \geq \frac{x_1}{h} \]  \hspace{1cm} [53]

In this scenario, both terms of Equation [47] should be included when determining \( M_b \). Elastic deformations are confined to region 1 and described by the first term. The length of region 1 is varied and equal to \( x_1/\varphi \). Plastic deformations occur in region
2 and are described by the both terms. Note, the first term in this case is constant because $x_1$ is constant. Therefore, the work done by the plate should be computed separately for regions 1 and 2 (Equation [54]).

\[
W_b = \int_0^{x_1' / \phi} \frac{D (\phi z)^2}{2} \, dz + \int_{x_1' / \phi}^{h} \left( \frac{D x_1^2}{2} + E (\phi z - x_1) \right) \, dz
\]

\[
= \frac{D x_1^3}{6\phi} + \frac{D x_1^2}{2} \left( h - \frac{x_1}{\phi} \right) + \frac{E\phi}{2} \left( h^2 - \frac{x_1^2}{\phi^2} \right) - Ex_1 \left( h - \frac{x_1}{\phi} \right)
\]  

[54]

The moment, $M_b$, resisted by the bearing of the plate is computed using Equation [55].

\[
M_b = \frac{d W_b}{d \phi} = \frac{D x_1^3}{3\phi^2} + \frac{E h^2}{2} - \frac{E x_1^2}{2\phi^2}
\]  

[55]

The method used to derive Equation [55] was validated numerically. A computer code (Appendix E) was developed in Visual Basic for Applications programming language to perform the numerical analysis. The error between the closed-form and numerical solutions was less than 0.1%.

Derivation of Equation [44] was based on the assumption that the NPC rotates around the geometrical center of the nail pattern, whereas the derivation of Equations [52] and [55] was based on the assumption that the NPC rotated around the middle of the plate side. In reality, the NPC neither rotates around the center of the rabbet nor around the geometrical center of the nail pattern. The centroid is moving as the NPC rotates and has to be determined using the vector analysis for every loading step. To simplify the analysis, it is assumed that the NPC rotates around geometrical center of the nail pattern. Because the nails contribute about seventy percent of the connector strength and stiffness, this assumption is justified. Moreover, the visual observations of the test specimens (Figure 39) support this assumption. Therefore, Equations [52] and [55] should be adjusted to account for this centroid position. Figure 49 shows the bearing surface of the NPC before rotation.
and after rotation to an angle $\varphi$. Initially, $\varphi=0$, the height of the bearing surface, $h$, equals to the half-depth of the member, $h_0$. As the rotation progresses, the height of the bearing surface decreases according to Equation [56].

![Figure 49. Adjustment of the bearing height.](image)

$$h = h_0 - a \tan \frac{\varphi}{2}$$  \hspace{1cm} [56]

where:

- $h_0$ = initial bearing height;
- $h$ = bearing height after rotation by angle $\varphi$;
- $a$ = distance from the centroid to the center of the plate side.

Because the small angle theory was used to derive the solution, Equation [56] can not be used directly with the equations for $M_b$. In general, if a distributed load is applied
to the element, the moment is a second power function of the element length. Therefore, the
equations for \( M_b \) can be adjusted by factor \( K_h \) (Equation [57]).

\[
k_h = \left( \frac{h}{h_0} \right)^2 \tag{57}
\]

Thus, the adjusted moment can be found using Equation [58].

\[
M_b^{adj} = k_h \cdot M_b \tag{58}
\]

The total moment resisted by the NPC is calculated using Equation [59].

\[
M = M_N + M_b^{adj} \tag{59}
\]

Or

\[
M = \sum_{\text{nails}} \left\{ -A r_i - C r_i^2 \varphi + S + B r_i \varphi S + A r_i \ln \left( \frac{A r_i}{S} \right) \right\} +
\]

\[
+ k_h \left( \frac{D x_1^3}{3\varphi^2} + \frac{E h^2}{2} - \frac{E x_1^2}{2\varphi^2} \right) \tag{60}
\]

for \( \varphi \leq \frac{x_1}{h} \) and \( x_1 = h\varphi \)

where:

\[
S = \frac{A r_i}{1 + B r_i \varphi} \tag{61}
\]

Equation [60] was validated using the experimental results of the moment-
rotation test (Section 4.1). Figure 50 compares the average curve fit to the experimental data and the curve computed using the model. Results of the model fell within the 95%
confidence interval for the experimental data.
Figure 50. Comparison of the model and experimental results.
The model overestimates the moment resistance of the NPC in a range of 0.3 – 1.0 inch with maximum deviation of 800 lb-in at 0.045 inch (Figure 51) deformation. The maximum relative error of the model as compared to the average experimental curve is 6.7% (Figure 52). Note, the relative error was computed as the absolute error at a given displacement divided by the experimental moment value at the same displacement.

A $R^2$ value was determined to measure the degree of the agreement between the model and the experimental results. Data points computed with the model were plotted
against data points from experimental data at the corresponding deformations. Using TableCurve™ 2D regression software package, a straight line with a slope of $45^0$ was forced to fit this curve. The regression analysis resulted in a $R^2$ value of 0.98 (Figure 53). If the model and the experimental data would be in a perfect agreement, the R2 value would be unity.

Figure 53. Determination of the $R^2$ value for the moment-rotation model.

Figure 54 divides the total resistance of the NPC into two components: resistance provided by the nails and resistance provided by the plate bearing on the end grain. The nails and plate contribute 75.7% and 24.3% to the maximum moment resistance, respectively.
Figure 54. The moment resistance of the NPC.

The model was used to investigate the moment-rotation relationship for a NPC with a modified nail pattern. As compared to the original nail pattern, nails 2 and 3 were shifted away from the rest of the nails by ½ inch. Therefore, each nail had a greater moment arm as opposed to the original nail pattern (Figure 55).
Figure 55. Nail patterns.

Figure 56 shows the moment-rotation relationships computed with the model for both nail patterns. The modified nail pattern increased the moment capacity by 15.1% or 2,280 lb-in, and the linear stiffness by 34.9% or 234,000 lb-in/rad. The increase in the moment arm had a more significant effect on the connection stiffness than strength. When using the model, the translational displacement of the most remote nail should not exceed 0.3 inch. Therefore, the limiting factor for the model is not the angle of rotation but the maximum nail translation. The larger the moment arm of the most remote nail, the smaller the allowable angle of rotation. For example, the most remote nail from the original nail pattern reached the critical 0.3 inch displacement at 0.252 rad rotation angle, whereas the most remote nail from the modified nail pattern reached the same displacement at 0.227 rad.
4.4.4 Conclusions and limitations

The proposed model showed a good agreement ($R^2=0.98$) with the experimental data, and therefore can be used to predict the moment response of a variety of the NPCs.

The model also can be used to investigate the effect of the gap between the NPC and the bearing surface of the rabbet on the connector performance. This gap can be the result of manufacturing imperfections.

The model ignores the development of the gap between the two wood pieces. The gap decreases the effective nail length and bearing surface for the plate. The model also disregards brittle failure modes.

When using the model, one should not allow the displacement of an individual nail to exceed 0.3 inch. Beyond this displacement, the load-deformation relationship for a single nail used in the derivation is inapplicable.
4.5 RTA shear wall tests

4.5.1 General

The objective of this test was to measure the strength and stiffness of the RTA shear walls. This information was used to design the RTA building and to validate the FE model.

4.5.2 Data analysis and discussion

Figures 59 and 60 show the load-drift curves for Type A and Type B walls, respectively. Tables 11 and 12 summarize the shear wall performance parameters for Type A and Type B walls, respectively.

Comparison of the results of the two wall types indicated that Type A walls exhibited higher strength and stiffness as compared to Type B walls. Average capacity for Type A walls was 11,342 lb versus 6,794 lb for Type B walls. Average elastic stiffness for Type A walls was 18,019 lb/in versus 9,314 for Type B walls.

However, Type A walls showed lower ductility and toughness of failure parameters as compared to Type B walls. Average ductility ratios 1 and 2 for Type A walls were 3.442 and 5.087, respectively. Average ductility ratios 1 and 2 for Type B walls were 6.028 and 11.302, respectively.

Because both wall types were secured to the foundation with the anchors, the walls failed mainly due to failure of the sheathing fasteners. Figure 57.a shows the rotation of the panels relative to the foundation for a Type A wall; the same pattern was observed for Type B walls. The difference in the response between Type A and Type B walls was associated with the different sheathing connection methods. The higher strength and stiffness of Type A walls was the result of greater number of the sheathing fasteners on the perimeter of each panel (44) as compared to Type B walls (24). Moreover, the screws of Type A walls were installed into TimberStrand® elements (SG ≈ 0.7, ESG = 0.5), whereas the nails of Type B walls were installed into SPF girts (SG ≈ 0.40). The use of screws versus
nails further contributed to the higher Type A wall stiffness values. The body of the screw was secured in the frame member with the thread minimizing the slack in the connection and preventing the screw from pulling out of the frame. Nails (Type B wall) installed without predrilling damaged the wood around the shank decreasing the connection stiffness.

The low ductility and toughness of Type A walls versus Type B walls was the results of different failure modes between the screws and nails, respectively. Two failure modes prevailed in Type A walls: (1) the shank of the screws broke due to combined shear and tension forces (Figure 57.b), and (2) the screws tore through the sheathing (Figure 57.c). Screws installed into the thick part of the siding always failed in the first mode, whereas the screws installed into the grooves failed either in the first or second mode depending on the end distance. These failure modes were also consistent with the results of the individual
screw tests reported in Section 4.6. Because both these failure modes were brittle in the nature, Type A walls exhibited low ductility response.

Two failure modes for sheathing connections were observed in Type B walls (Figure 58): (1) the nails pulled out of the frame members, and (2) the nails tore through the sheathing. The nails installed into the thick part of the siding always failed in the first mode, whereas the nails installed into the grooves failed either in the first or second mode depending on the end distance. Because there were no abrupt load drop associated with the first failure mode, Type B walls exhibited high ductility response. The Type B wall failure was accompanied by the splitting of the SPF girts.

Figure 58. Sheathing connection failure for Type B walls.
Table 11. Data summary for the shear wall test. Type A walls.

<table>
<thead>
<tr>
<th>Property, units</th>
<th>Wall #</th>
<th>Mean</th>
<th>St. Dev.</th>
<th>COV, %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Max Load, lb</td>
<td>11,360</td>
<td>11,467</td>
<td>11,199</td>
<td>11,342</td>
</tr>
<tr>
<td>Drift @ Peak Load, inch</td>
<td>2.071</td>
<td>1.479</td>
<td>1.784</td>
<td>1.778</td>
</tr>
<tr>
<td>Yield Load, lb</td>
<td>10,152</td>
<td>10,190</td>
<td>9,827</td>
<td>10,056</td>
</tr>
<tr>
<td>Drift @ Yield Load, inch</td>
<td>0.634</td>
<td>0.401</td>
<td>0.529</td>
<td>0.521</td>
</tr>
<tr>
<td>Proportional Limit, 0.4*F_{peak}, lb</td>
<td>4,544</td>
<td>4,587</td>
<td>4,480</td>
<td>4,537</td>
</tr>
<tr>
<td>Drift @ Prop. Limit, inch</td>
<td>0.284</td>
<td>0.181</td>
<td>0.241</td>
<td>0.235</td>
</tr>
<tr>
<td>Failure Load, 0.8 F_{peak}, lb</td>
<td>9,077</td>
<td>9,158</td>
<td>8,943</td>
<td>9,059</td>
</tr>
<tr>
<td>Drift @ Failure Load, inch</td>
<td>2.951</td>
<td>2.310</td>
<td>2.566</td>
<td>2.609</td>
</tr>
<tr>
<td>Elastic Stiffness, lb/in</td>
<td>15,233</td>
<td>21,553</td>
<td>17,272</td>
<td>18,019</td>
</tr>
<tr>
<td>Ductility1, Δ_{peak}/Δ_{yield}</td>
<td>3.269</td>
<td>3.684</td>
<td>3.372</td>
<td>3.442</td>
</tr>
<tr>
<td>Ductility2, Δ_{failure}/Δ_{yield}</td>
<td>4.658</td>
<td>5.754</td>
<td>4.849</td>
<td>5.087</td>
</tr>
<tr>
<td>Toughness of failure, Δ_{failure}/Δ_{peak}</td>
<td>1.425</td>
<td>1.562</td>
<td>1.438</td>
<td>1.475</td>
</tr>
</tbody>
</table>

Table 12. Data summary for the shear wall test. Type B walls.

<table>
<thead>
<tr>
<th>Property, units</th>
<th>Wall #</th>
<th>Mean</th>
<th>St. Dev.</th>
<th>COV, %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4</td>
<td>5</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>Max Load, lb</td>
<td>7,385</td>
<td>6,794</td>
<td>6,204</td>
<td>6,794</td>
</tr>
<tr>
<td>Drift @ Peak Load, inch</td>
<td>3.263</td>
<td>3.693</td>
<td>2.945</td>
<td>3.300</td>
</tr>
<tr>
<td>Yield Load, lb</td>
<td>6,375</td>
<td>6,079</td>
<td>5,473</td>
<td>5,976</td>
</tr>
<tr>
<td>Drift @ Yield Load, inch</td>
<td>0.72</td>
<td>0.504</td>
<td>0.473</td>
<td>0.566</td>
</tr>
<tr>
<td>Proportional Limit, 0.4*F_{peak}, lb</td>
<td>2.954</td>
<td>2.718</td>
<td>2.485</td>
<td>2.719</td>
</tr>
<tr>
<td>Drift @ Prop. Limit, inch</td>
<td>0.333</td>
<td>0.226</td>
<td>0.214</td>
<td>0.258</td>
</tr>
<tr>
<td>Failure Load, 0.8 F_{peak}, lb</td>
<td>5,881</td>
<td>5,425</td>
<td>4,942</td>
<td>5,416</td>
</tr>
<tr>
<td>Drift @ Failure Load, inch</td>
<td>5,731</td>
<td>6,668</td>
<td>6,017</td>
<td>6,139</td>
</tr>
<tr>
<td>Elastic Stiffness, lb/in</td>
<td>7,470</td>
<td>9,767</td>
<td>10,705</td>
<td>9,314</td>
</tr>
<tr>
<td>Ductility1, Δ_{peak}/Δ_{yield}</td>
<td>4.533</td>
<td>7.321</td>
<td>6.229</td>
<td>6.028</td>
</tr>
<tr>
<td>Ductility2, Δ_{failure}/Δ_{yield}</td>
<td>7.962</td>
<td>13.219</td>
<td>12.726</td>
<td>11.302</td>
</tr>
<tr>
<td>Toughness of failure, Δ_{failure}/Δ_{peak}</td>
<td>1.756</td>
<td>1.806</td>
<td>2.043</td>
<td>1.868</td>
</tr>
</tbody>
</table>
Figure 59. Load – drift relationship for Type A walls.

Figure 60. Load – drift relationship for Type B walls.
**Table 13. Moisture content and specific gravity of the wall members. Type A walls.**

<table>
<thead>
<tr>
<th>Property</th>
<th>Wall Element</th>
<th>Wall #</th>
<th>Mean</th>
<th>St. Dev.</th>
<th>COV, %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>MC, %</td>
<td>Frame members (TimberStrand®)</td>
<td>8.73</td>
<td>8.17</td>
<td>7.23</td>
<td>8.04</td>
</tr>
<tr>
<td></td>
<td>Sill Plate (SP)</td>
<td>22.21</td>
<td>20.05</td>
<td>9.04</td>
<td>17.10</td>
</tr>
<tr>
<td></td>
<td>Plywood</td>
<td>7.37</td>
<td>7.79</td>
<td>7.27</td>
<td>7.48</td>
</tr>
<tr>
<td>SG</td>
<td>Frame members (TimberStrand®)</td>
<td>0.72</td>
<td>0.71</td>
<td>0.69</td>
<td>0.71</td>
</tr>
<tr>
<td></td>
<td>Sill Plate (SP)</td>
<td>0.47</td>
<td>0.45</td>
<td>0.59</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>Plywood</td>
<td>0.60</td>
<td>0.61</td>
<td>0.59</td>
<td>0.60</td>
</tr>
</tbody>
</table>

**Table 14. Moisture content and specific gravity of the wall members. Type B walls.**

<table>
<thead>
<tr>
<th>Property</th>
<th>Wall Element</th>
<th>Wall #</th>
<th>Mean</th>
<th>St. Dev.</th>
<th>COV, %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>4</td>
<td>5</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>MC, %</td>
<td>Frame members (TimberStrand®)</td>
<td>7.96</td>
<td>6.9</td>
<td>6.41</td>
<td>7.09</td>
</tr>
<tr>
<td></td>
<td>Girts (SPF)</td>
<td>18.56</td>
<td>17.77</td>
<td>20.09</td>
<td>18.81</td>
</tr>
<tr>
<td></td>
<td>Sill Plate (SP)</td>
<td>25.82</td>
<td>8.16</td>
<td>15.53</td>
<td>16.50</td>
</tr>
<tr>
<td></td>
<td>Plywood</td>
<td>7.08</td>
<td>6.74</td>
<td>5.84</td>
<td>6.55</td>
</tr>
<tr>
<td>SG</td>
<td>Frame members (TimberStrand®)</td>
<td>0.68</td>
<td>0.72</td>
<td>0.73</td>
<td>0.71</td>
</tr>
<tr>
<td></td>
<td>Girts (SPF)</td>
<td>0.42</td>
<td>0.38</td>
<td>0.44</td>
<td>0.41</td>
</tr>
<tr>
<td></td>
<td>Sill Plate (SP)</td>
<td>0.53</td>
<td>0.54</td>
<td>0.43</td>
<td>0.50</td>
</tr>
<tr>
<td></td>
<td>Plywood</td>
<td>0.55</td>
<td>0.55</td>
<td>0.57</td>
<td>0.56</td>
</tr>
</tbody>
</table>
Chapter 4                                         Results and Discussion                                              118

4.6  Frame-sheathing connection test

4.6.1  General

The objective of this test was to measure the load-displacement relationship for the sheathing connections used in Type A shear walls (Section 3.4). This information was used as an input for the FE model of this shear wall.

4.6.2  Data analysis and discussion

Figure 61 shows the load-displacement curves for group 1 specimens and the Wen formula fit to the experimental data. Table 15 summarizes the performance parameters for group 1 specimens. Table 16 contains the parameters for the Wen formula. The similar diagrams and tables for the other eleven groups are summarized in Appendix B.

![Figure 61. Load-displacement curve for group 1 specimens.](image-url)
Table 15. Performance parameters for group 1 specimens.

<table>
<thead>
<tr>
<th>Property</th>
<th>Max Load, lb</th>
<th>Displ. @ Max Load, inch</th>
<th>Linear stiffness, lb/inch</th>
<th>Side Member</th>
<th>Main Member</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>MC, %</td>
<td>SG</td>
</tr>
<tr>
<td>Mean Value</td>
<td>646</td>
<td>0.229</td>
<td>11,375</td>
<td>4.95</td>
<td>0.62</td>
</tr>
<tr>
<td>St. Dev</td>
<td>65</td>
<td>0.056</td>
<td>2,913</td>
<td>0.42</td>
<td>0.03</td>
</tr>
<tr>
<td>COV, %</td>
<td>10.1</td>
<td>24.5</td>
<td>25.6</td>
<td>8.47</td>
<td>4.24</td>
</tr>
</tbody>
</table>

Table 16. Parameters for the Wen formula for group 1 specimens.

<table>
<thead>
<tr>
<th>K, lb/inch</th>
<th>P_y, lb</th>
<th>r</th>
<th>n</th>
</tr>
</thead>
<tbody>
<tr>
<td>15,000</td>
<td>485</td>
<td>0.065</td>
<td>2.5</td>
</tr>
</tbody>
</table>

Maximum load was defined as the largest load from the load-deformation curve. Linear stiffness was defined as the slope of the line fit into the data range between zero and forty percent of the maximum load.
4.7 Fabrication and construction of a demonstration RTA building

4.7.1 General

Although the development of the RTA framing system has a history of almost ten years, the only practical application of the theoretical concepts was the prototype building constructed by Platt (Platt 1998). A full-scale building was needed to further demonstrate the feasibility of the RTA technology including manufacturing of the NPCs, fabrication of the frame members, packaging and transportation of the kit, assembly and sheathing of the frame, and monitoring of the in-service building.

4.7.2 Design of the RTA building

A 16 x 24 foot RTA building was designed and constructed in Blacksburg, Va for use as a storage facility. The roof had a slope of 30°. The building height at the ridge was 13 feet 3 inches. The size of the building was selected so that a crew of three was capable of assembling the frame using a set of wrenches and a ladder. The design of the connections was governed by the simplicity of the assembly process. Figure 62 shows a drawing of this building with part of the frame exposed. The structural design procedure for this building is presented in Appendix C.
The building was assembled on a concrete slab which was constructed by a local contractor. The structural frame of the building consisted of seven bents connected with each other with recessed girts and purlins. The bents were spaced at 48 inch on-center. The girts and purlins were also spaced at 48 inch on-center. Two bent types were designed: intermediate bents and end-wall bents. Intermediate bents included two columns, two rafters, and a metal cable collar-tie. End wall bents had additional columns and a wood collar-tie to provide a nailing surface for the plywood sheathing. One of the end wall bents was designed to accommodate a 8 x 8 foot door. The specification of the components used in the demonstration RTA building is provided in Appendix D.

Frame members were fabricated from 2 x 4 inch TimberStrand® lumber. The columns and rafters were built-up elements which consisted of two pieces of lumber connected face-to-face with a NPC. Four designs of the NPC were used in this building for connecting the frame members. All four designs used the same five nail pattern, but the plate designs were modified to provide various element orientations. Half inch diameter bolts and
locking nuts were used with the NPC. NPC 1 (Figure 63) was located in the columns and the ends of the wood collar-ties, and was used to connect the columns to the rafters and collar-ties, and the collar-ties to the column-rafter connections. NPC 2 (Figure 64) was located in the rafters, and it was used to connect the rafters to the columns. NPC 3 (Figure 65) was located at the other end of the rafters and was used to connect a rafter to another rafter. NPC 4 (Figure 66) was located in the middle span of the wood collar-ties, and it was used to attach the collar-ties to the intermediate columns. Figure 67 shows the connection details of the end-wall bent. Figure 67.a shows the corner column connection to the rafter and the collar-tie. Because all three elements needed to be in the same plane to provide the nailing surface for the sheathing panels, an additional metal plate was used. Figure 67.b shows the intermediate column connection to the collar-tie. Figure 68.b shows the rafter-to-rafter connection which was the same for all bents.

Girts and purlins were recessed flush with the frame members to provide an additional nailing surface for the wall and roof sheathing. Girts and purlins were made of TimberStrand® lumber. They were bolted to the frame using 90° angle metal brackets and 5/16 inch bolts and locking nuts. Two bolts were used for each bracket leg. Figure 68 shows a purlin-to-rafter connection. Figure 69 shows a girt-to-the column connection. Frame members were predrilled to accommodate the bolts. Therefore, the entire frame was bolted together and can be disassembled without damage to the frame members for relocation to another site. Columns were attached to the sill plate using metal brackets. One leg of the bracket was bolted to a column with two 5/16 inch bolts and locking nuts, and the other leg was attached to the sill plate with two 3/8 inch lag screws. The top ends of the rafters were interconnected by a ridge beam with a cross section of five-side polygon to accommodate the roof sheathing panels (Figure 68.b).
Figure 63. NPC 1.

Figure 64. NPC 2.
Figure 65. NPC 3.

Figure 66. NPC 4.
Figure 67. Connection details of the end-wall bent. a. Corner column – rafter – collar-tie connection. b. Intermediate columns – collar-tie connection.

Figure 68. Connection details for roof purlins. a. Purlin – rafter connection. b. Ridge beam – rafters connection.
Walls were sheathed with 5/8 inch five-ply plywood. The plywood was attached to the columns and girts with 9ga 2 inch screws installed into the predrilled holes. The roof was sheathed with 23/32 inch OSB panels nailed to the rafters and purlins with 8d nails.

4.7.3 Construction of the RTA building

The fabrication process of the RTA building included: manufacturing of the NPC connectors, fabrication of the frame members components, fabrication of the RTA frame members, packaging and transportation of the RTA frame members to the construction site, assembly and sheathing of the frame.

The NPCs were fabricated by MAZE Inc. of Salem, VA. The technology used for manufacturing of the NPCs was described by Platt (1998).

The frame member components were fabricated at Brooks Forest Products Center, Blacksburg, VA. A component was a 1.5 x 3.5 inch piece of TimberStrand® cut to the required length. Two such pieces were used to manufacture a RTA frame member. To accommodate the NPC, one of the two pieces was routed to produce a rabbet. The rabbet had depth of 0.25 inch which was equal to the thickness of the NPC plate. Therefore, the rabbet eliminated the gap between the pieces and increased the NPC strength and stiffness in
compression and rotation by providing a bearing surface for the plate on the end grain of the member.

Fabrication of the RTA frame members involved installation of the NPCs between the frame member components. The installation was done with a hydraulic press (Figure 70) at a rate of approximately 5 inch/min. The rate was controlled manually by the operator. The press consisted of two steel I-beams. The base I-beam was stationary, while the top I-beam traveled vertically along steel guides. The base I-beam was equipped with a set of patterns for positioning the NPCs precisely at the required locations. The patterns were welded to the bottom plate of the press. The press was designed, built, and operated by Jeff Downs of Shawsville, VA.

![Hydraulic press used for manufacturing of the RTA frame members.](image)

**Figure 70.** *Hydraulic press used for manufacturing of the RTA frame members.*

A frame member was fabricated as follows:

1) the NPCs were positioned in the patterns on the base I-beam;
2) the rabbeted frame member component was attached to the top I-beam using clamps;
3) the top plate was lowered for the full penetration of the nails into the wood;
4) the top plate of the press was raised; the frame member component with the embedded NPCs was still attached to the top plate;
5) the second frame member component was placed on the base I-beam and clamped;
6) the top I-beam was lowered for the second time for the full penetration of the nails into the second component;
7) if the frame member had the NPC only on one end (ex.: column), the components were nailed together with 10d nails to prevent the frame member from mechanical degradation during handling, shipping, and installation.

Pressing concluded the factory stage of the process and produces prefabricated frame members ready for bolt-together assembly.

A set of photographs, presented in Figure 71, describes the assembly process of the RTA building. Figure 71.a shows a RTA frame kit which was shipped from the pressing facilities to the construction site on a pick-up truck. The kit includes: 19 columns, 14 rafters, 2 collar-ties, 30 girts, 36 purlins, and 2 ridge beams. If required, the long members can be fabricated as a set of shorter pieces for easy transportation. A crew of three assembled this RTA frame in one day using wrenches to tighten the joints. A pneumatic wrench was used to accelerate the assembly process. A ladder was needed to attach the roof purlins.

A simple frame assembly sequence was developed to increase the effectiveness of the construction process.

1. A level foundation ready for installation of the RTA frame should be prepared. In this project, a concrete slab and a CCA pressure treated sill plate were used. Alternatively, a wooden foundation can be designed.

2. Frame components for the first bent are set out on the slab and bolted together while in the horizontal position. The 90 degree angle between the columns and the collar-tie should be checked with a square. Metal brackets for the girts and purlins are also bolted. Then, the frame is tilted into the vertical position and temporarily braced to the sill plate (Figure 71. b). The vertical position of the first bent was checked with a level. The bent should be braced only in the out-of-plane direction. Because NPCs provided rigid joints, the
bent was stable in its plane during construction. The columns are attached to the sill plate with the metal brackets.

3. The second bent was assembled and raised into the vertical position in the same manner and was attached to the first bent with the girts and purlins. Because the first bent was plumb and braced, it served as a reference for the second bent. Therefore, additional bracing and leveling were unnecessary. Moreover, the girts and purlins spaced the second bent at the required distance (Figure 71. c).

4. The remaining bents are erected in the same manner as the second bent (Figure 71. d, e).

5. Because the walls and roof of the RTA building must resist the lateral loads, the shear walls and roof diaphragm were constructed using structural sheathing nailed or screwed to the frame members (Figure 71. f).

4.7.4 Conclusions

The construction of the demonstration building proved the feasibility of the RTA system concept. Moreover, the proposed assembly sequence was successfully validated. The frame for this 16 x 24 ft building was assembled in one day. Three people were used to install the end wall bents, whereas two people were able to install the intermediate bents.
Figure 71. Construction of a RTA building.

a. RTA frame kit.
b. The first bent is installed and braced.
c. The second bent is being installed.
d. The fifth bent is being installed.
e. The remaining bents are installed.
f. The RTA building in-service.
4.8  Finite element analysis of the RTA structures

4.8.1  Two-dimensional linear analysis of the RTA frames

4.8.1.1  General

The objective of this analysis was to compute forces acting on the members of the RTA frames. This information was used in the design example shown in Appendix C. Moreover, a series of sensitivity studies was conducted to evaluate the effect of design parameters on the frame performance. The method estimates relative changes in the response of the structure due to specified changes in the system variables. The method is effective for identifying the critical variables of the system. It facilitates designer efforts, and provides insight into the structural behavior of the system.

The general approach was to define realistic “base” material properties and connection stiffnesses for all elements and connections in the model. The structure was analyzed and its response was used as a baseline. Then, a variable (i.e. connection stiffness, material property or geometry of a frame) was changed, the structure was analyzed, and its response was compared to the baseline.

The NPCs were modeled as a set of three independent springs. The parallel-to-the-grain, perpendicular-to-the-grain, and rotational stiffnesses were assigned with values of 690,000 lb/in, 290,000 lb/in, 670,000 lb-in/rad. Translational stiffnesses were estimated from the data measured by Platt (1998). Rotational stiffness was estimated in Section 4.1.

4.8.1.2  Sensitivity studies

16 foot span frame with a collar-tie and 12 foot span frame without a collar-tie were investigated (Figure 72). The former frame was loaded with 8 lb/in gravity load (Appendix C), and the latter frame was loaded with 1.33 lb/in gravity load. The load of 1.33 lb/in was the dead load from the frame members. The load of this magnitude can be applied to the frame during construction if a collar-tie is attached after the erection of the frame. This practice was employed to construct the demonstration building. The load was applied along the length of the rafters. Columns were pinned to the foundation, and frame member connections were modeled as described in Section 3.6.2.
First, the sensitivity of the frame response to the connection stiffness was investigated. Only one spring stiffness was varied at a time, while the other two were held constant. The changes were applied simultaneously to all connections of the frame. A spring stiffness was changed by ±50%, +100%, and +200%. In addition, a spring was assigned with an infinitely high stiffness value (k*10^3) to model rigid connections. To model pinned connections between the RTA frame members, a moment release was assigned to the NPC-to-NPC connections. The maximum deflection at the ridge, maximum deflection of the middle span of a rafter, maximum value and location of the axial force, maximum value and location of the bending moment, the highest value of the moment acting on a NPC, and the location of this NPC were compared to the baseline case.

Figure 73 depicts the normal force, shear force, and moment force diagrams for the 16 foot span frame with collar-tie and baseline stiffness parameters. Figure 74 depicts the normal force, shear force, and moment force diagrams for the 12 foot span frame without collar-tie and baseline stiffness parameters.
a. Normal force diagram, lb.

b. Shear force diagram, lb.
Figure 73. Normal force, shear force, and moment force diagrams for 16 foot span frame with collar-tie.
a. Normal force, lb.

b. Shear force, lb.
The 16 foot span frame with collar-tie type showed little sensitivity to the changes in the axial spring stiffness, no sensitivity to the shear spring stiffness, and high sensitivity to the rotational spring. Results of the sensitivity studies for the axial and rotational springs are summarized in Tables 17 and 18, respectively. The results of the shear spring analysis are not shown because the baseline case values were constant regardless the shear spring stiffness values. Axial spring stiffness influenced the displacement of the ridge of the frame. For example, the increase in the axial stiffness of +200% resulted in a decrease of the ridge beam displacement by 26%. However, the rest of the parameters were not significantly influenced by the axial spring stiffness. The rotational spring stiffness had effect on the displacement parameters and the values and distribution of the moment forces. Figure 75 shows the moment diagrams for the rafter of the 16 foot span frame with collar-tie for five different stiffness scenarios. The connection stiffness is changed from pinned to

\[ c. \text{Moment force diagram, lb-in.} \]

\[ Figure \ 74. \ Normal \ force, \ shear \ force, \ and \ moment \ force \ diagrams \ for \ 12 \ foot \ span \ frame \ without \ collar-tie. \]
rigid. If the pinned connections are used, the NPCs are subjected to moment forces because the centroid of the NPC does not coincide with the location of the pin. Figure 75.e illustrates this case. The left and right NPCs are subjected to 2,054 and 2,686 lb-in moment forces, respectively. Note that the left NPC corresponds to the eave joint, and the right NPC corresponds to the ridge joint. As stiffness of the connection increases, the diagram moves up and the moments acting on the NPCs increase. The rigid analysis results in the moment forces 29.7% and 36.1% higher as opposed to the baseline case for the left and right NPCs, respectively. The first and second diagrams show that the left NPC is subjected to higher moment forces, whereas the third through fifth diagrams show that the right NPC is subjected to higher moments forces. Therefore, as the stiffness of the rotational spring changes, the location of the critical NPC changes. The NPC stiffness affects the moment forces acting on the connection and the member in the opposite manner. An increase in the connector stiffness results in an increase in the moment acting on the connector and a decrease in the moment acting on the rafter.

The frame without a collar-tie was only sensitive to the rotational spring stiffness (Table 19). Deflection of the ridge, deflection of the middle span of the rafter, moment force values and moment distribution were affected by the rotational stiffness. For example, the maximum deflection of the ridge decreased by 55.5% for the rigid analysis relative to the baseline case. The effect on the moment force distribution is illustrated in Figure 76. The diagram for the pinned case is not included because the structure becomes unstable. The distribution of the moment forces along the rafter is changed as opposed to the frame with a collar-tie. The moment diagram intersects the element only once. Thus, the moments acting at the ends have opposite signs. This moment distribution results in higher moment forces as opposed to the distribution for the frame with a collar-tie. As the rotational stiffness increases, the moment diagram shifts up relative to the neutral axes. In the first diagram, the highest moment is located on the right side of the rafter, and the right NPC is subjected to higher moment forces. In the next three cases, the highest moment is at the left end of the rafter, and the left NPC is subjected to higher moment forces.

The previous sensitivity studies covered the effect of the connection stiffness on the response of the frame. However, one should consider the ratio of the frame member
stiffness to the connection stiffness to capture all possible scenarios of the force distribution in the frame. The frame member stiffness is a function of three parameters: span, cross section geometry, and modulus of elasticity of the material. Another sensitivity study was conducted to evaluate the effect of increased stiffness of the rafter on the distribution of moment forces. The stiffness of the rafter was increased by using a 2 x 6 inch elements instead of 2 x 4 inch. The same 16 foot span frame with a collar-tie was analyzed to allow for the direct comparison with the previous case. The moment diagrams for the frame with rafters made of 2 x 6 inch elements are shown in Figure 77. The moment diagrams for the pinned case are identical. For all other cases, the moment diagram moved down as opposed to the case with more flexible rafters.

Another sensitivity study was conducted to evaluate the effect of the ratio of the connection stiffness to the frame member stiffness (Equation [62]). The parameter $K_{r}$ (Equation [63]) was used as a measure of the rafter stiffness.

$$ r = \frac{K_{3}}{K_{R}} $$  \[62\]

where:

$K_{3}$ = rotational stiffness of the NPC;

$K_{R}$ = see Equation [63].

$$ K_{R} = \frac{EI}{L^{3}} $$  \[63\]

where:

$E$ = modulus of elasticity;

$I$ = modulus of inertia;

$L$ = rafter span.

The same 16 foot span frame was analyzed. The rafter span was held constant. The rafter was made of 2 x 6 inch elements. Because the moment of inertia is a third power function of the member depth, it increased by a factor of 3.88 as compared to the case with 2 x 4 rafters. To offset this increase, the value of the modulus of elasticity was divided by the factor of 1.5, and the value of rotational stiffness was increased by a factor of 2.59. This resulted in
the same total factor of 3.88. Therefore, the ratio, \( r \), was held the same (Equation [64]). The modulus of elasticity and depth of the metal plates in the rafter-to-rater connection were adjusted in the same manner.

\[
    r = \frac{2.59 K_3}{3.88 K_R} = r
\]  

[64]

Figure 78 shows the moment diagram for this case. Comparison with the base line case (Figure 75.c) indicates that the diagrams are not identical. Moment values at the right end were practically the same (0.1% difference). However, the rest of the diagram fell lower the baseline case. The difference was 2.1% and 5.6% at the middle span and the left end, respectively. This fact can be explained with the influence of the rest of the structure on the distribution of moment forces in the rafters. Despite this small difference, the ratio, \( r \), can be used as a reference for estimating the redistribution of the moment forces due to changes in the stiffness on the frame components. If the ratio, \( r \), increases, the moment diagram moves up, whereas if the ratio decreases, the moment diagram moves down. In the former case, the connection experiences an increase in the moment forces. In the latter case, the rafter experiences an increase in the moment forces.

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Max deflection at the ridge, in</th>
<th>Max deflection of a rafter @ its middle span, in</th>
<th>Max axial force, lb</th>
<th>Location of max axial force</th>
<th>Max moment on a NPC, lb-in</th>
<th>Max moment of max moment</th>
<th>Location of NPC with max moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Baseline</td>
<td></td>
<td></td>
<td></td>
<td>Rafter at the eave</td>
<td>6402</td>
<td></td>
<td>Rafter, ridge</td>
</tr>
<tr>
<td>k₁ = 690000</td>
<td></td>
<td>0.019</td>
<td>0.348</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6402</td>
<td>4041</td>
</tr>
<tr>
<td>-50%</td>
<td></td>
<td>0.027</td>
<td>0.353</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6381</td>
<td>-2%</td>
</tr>
<tr>
<td>k₁ = 345000</td>
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<td>0.016</td>
<td>0.347</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6409</td>
<td>+1%</td>
</tr>
<tr>
<td>+50%</td>
<td></td>
<td>0.015</td>
<td>0.346</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6411</td>
<td>+2%</td>
</tr>
<tr>
<td>k₁ = 1380000</td>
<td></td>
<td>0.014</td>
<td>0.345</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6415</td>
<td>+3%</td>
</tr>
<tr>
<td>+100%</td>
<td></td>
<td>0.011</td>
<td>0.344</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6422</td>
<td>+5%</td>
</tr>
<tr>
<td>Rigid</td>
<td></td>
<td></td>
<td></td>
<td>Rafter at the eave</td>
<td>6422</td>
<td></td>
<td>Rafter, ridge</td>
</tr>
<tr>
<td>k₁*10³</td>
<td></td>
<td>0.019</td>
<td>0.348</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6402</td>
<td>4041</td>
</tr>
<tr>
<td>+50%</td>
<td></td>
<td>0.027</td>
<td>0.353</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6381</td>
<td>+2%</td>
</tr>
<tr>
<td>k₁ = 1380000</td>
<td></td>
<td>0.016</td>
<td>0.347</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6409</td>
<td>+1%</td>
</tr>
<tr>
<td>+100%</td>
<td></td>
<td>0.015</td>
<td>0.346</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6411</td>
<td>+2%</td>
</tr>
<tr>
<td>Rigid</td>
<td></td>
<td>0.011</td>
<td>0.344</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6422</td>
<td>+5%</td>
</tr>
<tr>
<td>Pinned</td>
<td></td>
<td>0.068</td>
<td>0.708</td>
<td>964</td>
<td>Rafter at the eave</td>
<td>9263</td>
<td>+4%</td>
</tr>
<tr>
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<td></td>
<td></td>
<td>Rafter at the eave</td>
<td>9263</td>
<td></td>
<td>Rafter, ridge</td>
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</tbody>
</table>


<table>
<thead>
<tr>
<th>Scenario</th>
<th>Max deflection at the ridge, in</th>
<th>Max deflection of a rafter @ its middle span, in</th>
<th>Max axial force, lb</th>
<th>Location of max axial force</th>
<th>Max moment on a NPC, lb-in</th>
<th>Max moment of max moment</th>
<th>Location of NPC with max moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Baseline</td>
<td></td>
<td></td>
<td></td>
<td>Rafter at the eave</td>
<td>6402</td>
<td></td>
<td>Rafter, ridge</td>
</tr>
<tr>
<td>k₁ = 670000</td>
<td></td>
<td>0.019</td>
<td>0.348</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6402</td>
<td>4041</td>
</tr>
<tr>
<td>-50%</td>
<td></td>
<td>0.028</td>
<td>0.420</td>
<td>1040</td>
<td>Rafter at the eave</td>
<td>6300</td>
<td>-7%</td>
</tr>
<tr>
<td>k₁ = 335000</td>
<td></td>
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<td>0.313</td>
<td>1050</td>
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<td>+8%</td>
</tr>
<tr>
<td>+50%</td>
<td></td>
<td>0.015</td>
<td>0.293</td>
<td>1052</td>
<td>Rafter at the eave</td>
<td>7223</td>
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</tr>
<tr>
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<td>0.270</td>
<td>1054</td>
<td>Rafter at the eave</td>
<td>7575</td>
<td>+18%</td>
</tr>
<tr>
<td>+100%</td>
<td></td>
<td>0.012</td>
<td>0.214</td>
<td>1061</td>
<td>Rafter at the eave</td>
<td>8456</td>
<td>+32%</td>
</tr>
<tr>
<td>Rigid</td>
<td></td>
<td></td>
<td></td>
<td>Rafter at the eave</td>
<td>8456</td>
<td></td>
<td>Rafter, ridge</td>
</tr>
<tr>
<td>k₃*10³</td>
<td></td>
<td>0.019</td>
<td>0.348</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6402</td>
<td>4041</td>
</tr>
<tr>
<td>+50%</td>
<td></td>
<td>0.027</td>
<td>0.353</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6381</td>
<td>+2%</td>
</tr>
<tr>
<td>k₁ = 1380000</td>
<td></td>
<td>0.016</td>
<td>0.347</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6409</td>
<td>+1%</td>
</tr>
<tr>
<td>+100%</td>
<td></td>
<td>0.015</td>
<td>0.346</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6411</td>
<td>+2%</td>
</tr>
<tr>
<td>Rigid</td>
<td></td>
<td>0.011</td>
<td>0.344</td>
<td>1047</td>
<td>Rafter at the eave</td>
<td>6422</td>
<td>+5%</td>
</tr>
<tr>
<td>Pinned</td>
<td></td>
<td>0.068</td>
<td>0.708</td>
<td>964</td>
<td>Rafter at the eave</td>
<td>9263</td>
<td>+4%</td>
</tr>
<tr>
<td>k₃ = 670000</td>
<td></td>
<td></td>
<td></td>
<td>Rafter at the eave</td>
<td>9263</td>
<td></td>
<td>Rafter, ridge</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Max deflection at the ridge, in</th>
<th>Max deflection of a rafter @ its middle span, in</th>
<th>Max axial force, lb</th>
<th>Location of max axial force</th>
<th>Max moment, lb-in</th>
<th>Locati on of max moment</th>
<th>Max moment on a NPC, lb-in</th>
<th>Location of NPC with max moment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Baseline</td>
<td>0.292</td>
<td>0.186</td>
<td>102</td>
<td>column</td>
<td>1454</td>
<td>Rafter, eave</td>
<td>1454</td>
<td>Rafter, eave</td>
</tr>
<tr>
<td>k = 670000</td>
<td>0.455</td>
<td>0.281</td>
<td>102</td>
<td>column</td>
<td>1441</td>
<td>Rafter, eave</td>
<td>1441</td>
<td>Rafter, ridge</td>
</tr>
<tr>
<td>-50%</td>
<td>k = 335000</td>
<td>0.238</td>
<td>-18.5%</td>
<td>column</td>
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<td>Rafter, eave</td>
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<td>Rafter, eave</td>
</tr>
<tr>
<td>+50%</td>
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<td>0.211</td>
<td>-27.7%</td>
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<td>1505</td>
<td>Rafter, eave</td>
<td>1505</td>
<td>Rafter, eave</td>
</tr>
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<td>+100%</td>
<td>k = 1340000</td>
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<td>-37.0%</td>
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<td>Rafter, eave</td>
<td>1528</td>
<td>Rafter, eave</td>
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<tr>
<td>Rigid K*10³</td>
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<td>0.089</td>
<td>102</td>
<td>column</td>
<td>1591</td>
<td>Rafter, eave</td>
<td>1591</td>
<td>Rafter, eave</td>
</tr>
<tr>
<td>Pinned, k = 670000</td>
<td>0.29</td>
<td>0.186</td>
<td>102</td>
<td>column</td>
<td>1454</td>
<td>Rafter, eave</td>
<td>1454</td>
<td>Rafter, eave</td>
</tr>
</tbody>
</table>

Note: structure is unstable
Figure 75. Moment diagrams for a rafter from 16 foot span RTA frame with collar-tie. Note, the rafter is rotated from its original position around the left end to the horizontal position. The diagram is drawn on the tension side.
Figure 76. Moment diagrams for a rafter from 16 foot span RTA frame without collar-tie. Note, the rafter is rotated from its original position around the left end to the horizontal position. The diagram is drawn on the tension side.
Figure 77. Moment diagrams for a rafter from 16 foot span RTA frame with collar-tie. Note, the rafter is rotated from its original position around the left end to the horizontal position. The diagram is drawn on the tension side.
Figure 78. Moment diagrams for a rafter from 16 foot span RTA frame with collar-tie. Note, the rafter is rotated from its original position around the left end to the horizontal position. The diagram is drawn on the tension side.
4.8.2 \textit{Two-dimensional linear and non-linear modeling of the RTA shear walls}

4.8.2.1 General

The objective of this analysis was to develop a model for predicting the racking resistance of the RTA shear walls. Because the RTA system is still under development, it is important to be able to estimate changes in the system performance due to changes in the design. The following variables of the RTA shear wall construction are the subject of possible changes: frame material, frame spacing, girts and purlins spacing and installation method, type of sheathing fasteners, and sheathing type. Two designs of the RTA shear walls were tested (Section 4.5). If in the future these designs will undergo changes, the model can be used to assess the appropriateness of the proposed alternations and minimize the scope of the future full-size tests.

Type A shear wall was modeled. The input for the modeling was the geometrical and mechanical properties of the components, and the load-deformation relationship of the screws. The modeling methods are described in detail in Section 3.6. The sheathing fastener stiffnesses are found in Section 4.6.

4.8.2.2 Limitations

The racking response of a shear wall is complex. Given the capabilities of SAP2000, the results of the model is the subject to limitations. Results of linear modeling are limited to the initial range of displacements. Results of nonlinear modeling are limited by the lack of a direct method for spring coupling in SAP2000. Deformations of a panel in the shear wall are restricted by the adjacent panels, but the model allowed the adjacent panels to overlap each other mathematically. Moreover, the springs fail to predict the reduced fastener strength beyond the maximum load. Instead, the springs modeled the fasteners as infinitely strong. Section 3.6.5 presents the indirect adjustment method used to mitigate the coupling effect and discusses these limitations in detail.
4.8.2.3 Results of the modeling

Figure 79 shows the FE model of the RTA shear wall. Load was applied at six
locations along the top plate. The bottom plate was pinned to the ground at five points. The
location of the pinned supports coincided with the anchor bolts of the test walls. The right
support was shifted to the left by 3.5 inch. This number was equal to the distance between
the center of the column and the anchor bolt. Three rollers were placed to the left from the
right pinned support to prevent bending of the bottom plate due to the column uplift.

Figure 80 depicts the deformed shape of the wall model.

Figure 82 shows the load-drift relationship computed using the analytical model
and experimental load-drift curves for three Type A walls. The comparison shows a good
agreement within the linear region for both linear and nonlinear analyses. The linear
stiffness of sheathing fasteners was defined as the slope of a line passing through the origin
and the point with $0.4P_{\text{max}}$ load. As the result of this assumption, the linear model
underestimates the initial stiffness of the wall. The slope of the analytical curve is 19,027
lb/in, which falls into the range of Type A wall stiffnesses computed from experimental data
reported in Table 11. Based on the visual examination of the analytical and experimental
data, the applicability of the linear analysis is limited to the displacement range of from 0 to
0.30 inch. Beyond this range the linear model considerably overestimates shear wall strength
and stiffness.
Figure 79. Shear wall model.
Figure 80. Shear wall model. Deformed shape.
The nonlinear analysis resulted in a slightly higher initial stiffness as compared to the linear analysis. The curves intersect each other at the point with 0.372 inch displacement and 7,200 lb load. Based on visual examination of the load-drift curve for the nonlinear analysis, the shear wall enters the region with nonlinear performance at approximately 6,000 lb load or 0.3 inch displacement. Beyond this point the model overestimates the strength of the shear wall. For 0.5-1.0 inch drift range the model overestimates the shear wall strength by 5.1% to 25.5%. The minimum error corresponds to wall #1, and the maximum error corresponds to wall #2. Note that the error is always calculated relative to the experimental data. The slope of the analytical curve in the post-yield region is within the range of the experimental data. The analytical post-yield stiffness in the drift range of 0.5 - 0.9 inch is 5,544 lb/in. Experimental stiffnesses for the same drift range are 4,082 lb/in, 5,822 lb/in, and 5,506 lb/in for walls 1, 2, and 3, respectively.

A sensitivity study was conducted to measure the contribution of the corner nails to the shear wall strength and stiffness. Results of this study contribute to the validation of adjustment method used to moderate the spring coupling effect beyond the elastic response. Figure 83 compares the model without the corner screws and the model with the corner screws. Overall 10 screws were removed: 3, 4, and 3 screws from the left, central, and right panels, respectively. The change of the wall stiffness in the displacement range below 0.25 inch was within 5.0%. However, the change of the wall post-yield stiffness in the displacement range of 0.5 – 1.0 inch was 11.1 %. Strength of the wall in the same region decreased by 1,000 lb (9.7%) and fell within the range of the experimental data. Given that the total number of screws used in the wall is 176, the ten removed screws account for 5.7% of the total number of screws. However, the removal of these 10 screws influences the post-yield performance of the shear wall by 9.7%.

Using the results of the linear analysis, the angle between the vector of displacement of a corner screw and its projection is calculated for the 10 corner screws (Table 20). The angle fell in the range between 35 and 43 degrees with the mean value of 39.3 degrees and the COV of 7.8%. Therefore, the assumptions involved in the adjustment method proposed in Section 3.6.5 with attempt to mitigate the spring coupling effect beyond the elastic response are justified. Figure 81 shows the results of the SAP2000 graphical
output for the left bottom corner of the left panel. It illustrates the deformation of the springs relative to the frame and the sheathing.

Table 20. Angle between the vector of displacement of a corner nail and either horizontal or vertical spring whichever is smaller.

<table>
<thead>
<tr>
<th>Panel</th>
<th>Corner screw position</th>
<th>Angle, deg</th>
</tr>
</thead>
<tbody>
<tr>
<td>Left</td>
<td>Left bottom corner</td>
<td>37</td>
</tr>
<tr>
<td></td>
<td>Right button corner</td>
<td>43</td>
</tr>
<tr>
<td></td>
<td>Right top corner</td>
<td>38</td>
</tr>
<tr>
<td>Central</td>
<td>Left bottom corner</td>
<td>42</td>
</tr>
<tr>
<td></td>
<td>Right button corner</td>
<td>43</td>
</tr>
<tr>
<td></td>
<td>Right top corner</td>
<td>36</td>
</tr>
<tr>
<td></td>
<td>Left top corner</td>
<td>35</td>
</tr>
<tr>
<td>Right</td>
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<td>42</td>
</tr>
<tr>
<td></td>
<td>Right button corner</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>Left top corner</td>
<td>37</td>
</tr>
<tr>
<td></td>
<td><strong>Mean</strong></td>
<td><strong>39.3</strong></td>
</tr>
<tr>
<td></td>
<td><strong>COV, %</strong></td>
<td><strong>7.8</strong></td>
</tr>
</tbody>
</table>
Another sensitivity study was conducted to measure the contribution of the moment resistant connections assembled with the NPCs to the total shear resistance of the RTA shear wall. The NPCs were removed from the model, and replaced with pinned connections between the columns and collar-tie. At the displacement of 0.25 inch the model with the NPC resisted a load of 5,126.4 lb, whereas the model without the NPCs resisted a load of 5,012.6 lb. Therefore, the contribution of the NPCs to the total resistance of the RTA shear wall at 0.25 inch drift is 2.3% percent and it can be ignored from the analysis for most practical purposes.
Figure 82. Load-drift relationships. Results of linear and nonlinear analyses are compared to the experimental data.

Figure 83. Load-drift relationships. Results of nonlinear analyses compared to the experimental data.
4.9  Lateral design of the RTA structures

The objective of this section was to estimate the contribution of RTA frames to the overall lateral resistance of the structure. This information was needed to determine the magnitude of the lateral forces acting on RTA frames and define the lateral load path for RTA buildings.

The procedure outlined in ASAE EP484.2 “Diaphragm Design of Metal-Clad, Wood-Frame Rectangular Buildings” (ASAE Standards 1999) was used to answer the objectives of this section. The demonstration building (Section 4.7) was analyzed. The contribution of the frame to the lateral resistance of the building is a function of the ratios of the diaphragm stiffness to the frame stiffness, and the shear wall stiffness to the frame stiffness. At this time, there was no experimental data available on the shear resistance of the RTA diaphragms. Instead, the data from the shear wall test of Type A walls was used. Therefore, this discussion is limited to the demonstration of the approach and should be used as a reference point only. Because RTA diaphragms are assembled with OSB panels with thickness greater than the plywood siding used for the shear walls, and because the diaphragm panels are staggered as opposed to the shear wall panels, the use of the shear wall data should lead to a conservative solution. When the diagram stiffness is measured by others, the computations should be verified.

The effective shear modulus, $G$, of the RTA diaphragm was computed with Equation [65].

$$G = \frac{a}{b} = \frac{18,019 \text{ lb/in}}{12} = 1,502 \text{ lb/in}$$  [65]

where:

- $c = 18,019 \text{ lb/in} =$ effective shear stiffness from the cantilever test (Table 11);
- $a = 8 \text{ ft} =$ width of the test panel;
- $b = 12 \text{ ft} =$ length of the test panel.

The horizontal shear stiffness of the roof diaphragm including both slopes was determined with Equation [66].
\[ C_h = 2 G (\cos^2 \theta) \frac{b_h}{s} = 2 \times 12,013 \times (\cos^2 30^\circ) \frac{10.7}{4} = 48,202 \text{ lb/in} \]  

where:

- \( 2 \) = coefficient for roof with two slopes;
- \( G \) = see Equation [65];
- \( \theta \) = roof slope;
- \( b_h \) = horizontal span of diaphragm as measured parallel to rafters;
- \( s \) = frame spacing.

Frame stiffness was computed using SAP2000 FE model. A lateral load of 50 lb was applied to the eave joint of the 16 foot span frame with collar-tie. The imposed load resulted in the lateral displacement of the eave joint of 1.1 inch. Therefore, the lateral stiffness of the frame is \( k = 45.5 \text{ lb/in} \). The ratio of the diaphragm stiffness, \( C_h \), to the frame lateral stiffness, \( k \), is equal to 1059.

The front end wall of the demonstration RTA building accommodates an 8 x 8 foot opening for a garage door. The back end wall is fully sheathed. Therefore, these two walls have different stiffnesses. The analysis was performed for the shear wall with the opening. This assumption should lead to a conservative design. The stiffness of the shear wall with opening was computed using SAP2000 FE program by modifying the original model (Section 4.8.2). The middle panel was removed and the right panel was shifted by 48 inch to the right. Therefore, the modified model had two panels separated by 8 x 8 foot opening. An additional pinned restraint was imposed to the left column of the right panel to prevent its vertical movement due to the compression force. The right column of the left panel was free for uplift (Figure 84). The analysis resulted in the linear stiffness of 8,180 lb/in, which was 57% less than the wall without opening. Therefore, the ratio of the shear wall, \( k_e \), stiffness to the frame stiffness, \( k \), was 180.
Using Table 2 of ASAE EP484.2 (ASAE Standards 1999) with \( k_e/k = 180 \) and \( C_h/k = 1059 \) ratios and the number of frames of 7, the sideway restraining force factor \( mD \) is 0.97. Therefore, the envelope and the frame carry 97% and 3% of the lateral load, respectively.

The frame design should include a lateral force. The magnitude of the potential sideway restraining force was computed using SAP2000 FE model by applying the wind forces on the frame (see Appendix C for load calculations) and restraining the eave joint of the frame from the lateral movement. The reaction at the restrained joint was the maximum potential sideway restraining force, \( R \), of the diaphragm. The analysis yielded \( R = 306.6 \) lb. The roof diaphragm sideway restraining force, which was a product of \( R \) and \( mD \), was 297.4 lb. The roof diaphragm action was included by applying the sideway restraining force distributed as a horizontal uniform load along the rafters. The same frame was analyzed. The displacement of the eave joint was 0.003 inch. Using the frame stiffness of 45.5 lb/in, the lateral force acting on the frame at the eave of 0.13 lb was found. This load can be neglected from the analysis.

The same routine was repeated for the number of frames equal to 10, 12, and 16. The corresponding \( mD \) factors were 0.95, 0.93, and 0.91 respectively. The analysis yielded the lateral forces acting on the frame of 6.0 lb, 12.5 lb, and 18.9 lb, respectively. The frame was analyzed with these lateral forces applied to the eave joint and the moment forces were compared to the baseline case. Table 21 summarizes the results of the analysis. The moment force in the column was affected more significantly as compared to the rafter. The transition from 7 to 16 bents led to the increase of the column moment by 845 lb-in (164%) and the
rafter moment by 388 lb-in (10%). However, the rafter moment remained critical (4,249 lb-in vs 1,361 lb-in). Therefore, DL+SN load combination is critical for the frame analysis as opposed to DL+SN+½W or DL+½SN+W. The reason is that the load combinations containing wind load are used with the load duration factor, C_D, of 1.6, whereas the load combination without wind load is used with the load duration factor, C_D, of 1.15. The change in the load duration factor from 1.15 to 1.6 increases the allowable load by 39%, whereas the increase in the critical moment acting on the frame due to the addition of wind force is only 10%.

Table 21. RTA frame under combined gravity and lateral loading.

<table>
<thead>
<tr>
<th>Number of bents</th>
<th>Lateral force, lb</th>
<th>Lateral displacement at the eave joint, inch</th>
<th>Eave joint</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Max moment in the rafter, lb-in</td>
<td>Max moment in the column, lb-in</td>
</tr>
<tr>
<td>7</td>
<td>0</td>
<td>0</td>
<td>3,861</td>
</tr>
<tr>
<td>7</td>
<td>0.13</td>
<td>0.003</td>
<td>3,863</td>
</tr>
<tr>
<td>10</td>
<td>6.0</td>
<td>0.125</td>
<td>3,984</td>
</tr>
<tr>
<td>12</td>
<td>12.5</td>
<td>0.267</td>
<td>4,117</td>
</tr>
<tr>
<td>16</td>
<td>18.9</td>
<td>0.407</td>
<td>4,249</td>
</tr>
</tbody>
</table>

Results of this analysis indicate that the contribution of the frame to the lateral resistance of the RTA building can be excluded from engineering design for most practical purposes. The lateral load path for the RTA buildings includes diaphragms and shear walls. The composite action between the frame and the envelope can be ignored. These conclusions were validated for buildings with the aspect ratio up to 4.0. The RTA frame should be designed to resist gravity loads only. Note that because an individual frame member also is a part of shearwall or diaphragm, this frame member should be designed to resist the axial forces resulting from wind or seismic loading.
Chapter 5

Summary and Conclusions

This section is organized so that each conclusion answers the corresponding objective (Section 1.2). The conclusions are accompanied with a short summary to illustrate the subject matter.

1) A 16 x 24 foot RTA building was constructed. The project demonstrated the feasibility of the RTA system concept. An effective assembly sequence was proposed and successfully implemented. The RTA frame for the building was erected in one day by three people.

2) A 16 x 24 foot RTA building was designed. The design example covered vertical and lateral load resistance of the RTA building;

   2.1) RTA frames were analyzed using SAP2000 FE program. The FE model incorporated semi-rigid behavior of the NPC. Sensitivity studies showed that the frame model was sensitive to the rotational stiffness of the NPC;

   2.2) An analytical model was formulated to predict the nonlinear moment-rotation relationship of the NPC. A closed-form solution was derived. The proposed model showed a good agreement ($R^2=0.98$) with the experimental data. The maximum error between the analytical and experimental results was 6.7%. The model can be used to predict the moment-rotation relationship for a variety of the NPC geometries;

   2.3) A series of tests was conducted to measure the moment-rotation relationship of the NPC in wood-based composites and solid wood. The NPCs installed in TimberStrand® showed the highest strength and stiffness. Moreover, the TimberStrand® specimens failed due to nail yielding and wood crushing under the nails resulting in a ductile failure mode. NPCs installed in SPF and some Parallam® specimens failed in tension perpendicular-to-the-grain resulting in a brittle failure mode;
2.4) A series of tests was conducted to measure the load-displacement relationship of the plate bearing on the end grain of TimberStrand®. The elastic-plastic model was chosen to describe the load-displacement relationship;

2.5) RTA shear walls were analyzed using SAP2000 FE program. The analysis covered linear and non-linear performance. Both the linear and nonlinear models showed good agreement with the experimental data within the elastic region. The linear model was inapplicable and the nonlinear model showed limited applicability beyond the elastic range. The available analytical resources of SAP2000 did not allow the author to improve the accuracy of the nonlinear model. The contribution of the moment resistant connections to the total lateral resistance of RTA shear walls was estimated within 2.3%. This contribution can be ignored from the engineering analysis;

2.6) Tests were conducted to measure the load-drift response of RTA shear walls. Type A walls exhibited higher strength and stiffness, whereas Type B walls exhibited higher ductility;

2.7) Tests were conducted to measure the load-slip relationship of screws used to attach sheathing panel to the frame for Type A wall. The information was used to model the RTA shear wall response;

2.8) The lateral load path for the RTA building includes diaphragms and shear walls. The contribution of the RTA frame can be ignored from the lateral analysis. This conclusion was validated for the diaphragms with aspect ratios up to 4:1.
Recommendations for Future Research

This section is intended to identify future research projects that will provide new insights into the performance of RTA structures and contribute to successful commercialization of the RTA framing system.

1. The process of manufacturing of NPCs and RTA frame members needs to be refined for efficient mass production of the RTA kits. The project can involve time and motion studies. A production line should be designed and built to accomplish this task.

2. New sheathing products should be examined for the use with RTA frames. The preference should be given to the sheathing products that can span distances greater than 24 inches, because they can be attached to fewer frame members. The smaller the number of the structural frame members in the RTA kit, the easier the assembly process and the more economical the RTA building will be.

3. Performance of RTA diaphragms should be measured. Results of the tests will be used to design the RTA buildings to resist lateral loads. Two construction types can be investigated: the girts recessed flush between the rafters and the girts installed on top of the rafters. The diaphragms can be sheathed with structural wood-based panels or metal cladding.

4. The moment resistance of NPCs with other nail patterns should be measured and compared to the results of the analytical modeling. The effect of these nail patterns on failure modes should be investigated.

5. The effect of gap between the connector plate and the end grain of the frame member on the moment resistance of NPCs should be investigated. The critical gap beyond which the contribution of the plate bearing on the end grain can be ignored from the moment resistance of the NPC should be estimated. Based on results of this study, maximum allowable manufacturing tolerances can be specified.

6. Performance of NPCs subjected to combined lateral and moment loading should be investigated and an interaction equation should be developed.

7. The analytical model for moment resistance of NPCs should be revised to include brittle failure modes.
Bibliography


Appendix A

Design method for the nail plate connector subjected to lateral load


1. Allowable lateral resistance of a single nail with one shear plane:

\[ T = T_H K_a K_g \]  

\[ T_H = \] nominal lateral design value, provided nail length \( a = a_{\text{min}} \);
\[ K_a = \] coefficient for nail length exceeding \( a_{\text{min}} \);
\[ K_g = \] coefficient for out-of-plane deformations.

2. Minimum nail length required to provide yield mode IIIb:

\[ a_{\text{min}} = 0.44 d \left( \frac{R_{nb}}{R_b^\theta} \right)^{0.5} \]  

\( 0.44 = \) coefficient, derived to ensure yield mode IIIb.
\( d = \) nail diameter;
\( R_{nb} = \) allowable bending yield strength of nail;
\( R_b^\theta = \) nail bearing strength of wood.

3. Nominal lateral design value, provided nail length \( a = a_{\text{min}} \):

\[ T_H = 0.44 d^2 \left( R_{nb} R_b^\theta \right)^{0.5} \]  

4. Coefficient \( K_a \) is determined from Table A1.

Table A1. \( K_a \) values.

<table>
<thead>
<tr>
<th>a'</th>
<th>0.5</th>
<th>0.65</th>
<th>0.8</th>
<th>0.95</th>
<th>1.1</th>
<th>1.25</th>
<th>1.4</th>
</tr>
</thead>
<tbody>
<tr>
<td>( K_a )</td>
<td>1</td>
<td>1.03</td>
<td>1.08</td>
<td>1.15</td>
<td>1.23</td>
<td>1.32</td>
<td>1.41</td>
</tr>
</tbody>
</table>
Where:

\[ a' = \frac{a \left( \frac{R_a}{R_{nb}} \right)^{0.5}}{d} = 0.44 \frac{a}{a_{\min}} \]  \hspace{1cm} (4)

5. Coefficient \( K_g \) for connectors with out-of-plane deformations is determined from Table A2.

\textit{Table A2.} \( K_g \) values.

<table>
<thead>
<tr>
<th>( a' )</th>
<th>0.5</th>
<th>0.65</th>
<th>0.8</th>
<th>0.95</th>
<th>1.1</th>
<th>1.25</th>
<th>1.4</th>
</tr>
</thead>
<tbody>
<tr>
<td>( K_g )</td>
<td>0.8</td>
<td>0.82</td>
<td>0.84</td>
<td>0.86</td>
<td>0.89</td>
<td>0.92</td>
<td>0.95</td>
</tr>
</tbody>
</table>

6. Allowable bending yield strength of one nail in the NPC:

\[ R_{nb} = R_b K_H \]  \hspace{1cm} (5)

\( R_b \) = allowable bending yield strength of nail material;

\( K_H \) = coefficient accounting for the NPC type, \( K_H = 1.1 \) for nails inserted into predrilled holes and \( K_H = 1.2 \) for welded nails.

7. Nail bearing strength:

\[ R_b^\theta = R_c K_R K_\theta \]  \hspace{1cm} (6)

\( R_c \) = allowable load for compression parallel-to-grain strength of wood;

\( K_R \) = nail diameter coefficient (Table 3);

\( K_\theta \) = grain orientation coefficient (Table 3), for \( \theta = 0^\circ \) \( K_\theta =1 \).

\textit{Table 3.} \( K_R \) and \( K_\theta \) values.

<table>
<thead>
<tr>
<th>d, mm</th>
<th>5</th>
<th>6</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>( K_R )</td>
<td>0.80</td>
<td>0.77</td>
<td>0.70</td>
</tr>
<tr>
<td>( K_\theta, (\theta = 90^\circ) )</td>
<td>0.9</td>
<td>0.85</td>
<td>0.8</td>
</tr>
</tbody>
</table>

\( K_\theta \) given for nails embedded without predrilling.
8. Allowable lateral resistance of the NPC:

\[ T_c = T \, n_s \, n_N \, K_R \]  \hspace{1cm} (7)

\( T \) = allowable lateral resistance of one nail with one shear plane;
\( n_s \) = number of shear planes per nail;
\( n_N \) = number of nails;
\( K_R \) = group action factor, \( K_R = 1 \) for up to 7 nails in a row and \( K_R = 0.85 \) for 15 nails in a row. For a number of nails in a row between 7 and 15 \( K_R \) should be linearly interpolated.
Appendix B

Data summary for the frame-sheathing connection test

Figure B1. Load-deformation curve for group 2 specimens.

Table B 1. Performance parameters for group 2 specimens.

<table>
<thead>
<tr>
<th>Property</th>
<th>Side Member</th>
<th>Main Member</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MC, %</td>
<td>SG</td>
</tr>
<tr>
<td>Max Load, lb</td>
<td>4.77</td>
<td>0.62</td>
</tr>
<tr>
<td>Displ. @ Max Load, inch</td>
<td>0.251</td>
<td>9.317</td>
</tr>
<tr>
<td>Linear stiffness, lb/inch</td>
<td>16.4</td>
<td>5.36%</td>
</tr>
<tr>
<td>Mean Value</td>
<td>583</td>
<td>99</td>
</tr>
<tr>
<td>St. Dev</td>
<td>99</td>
<td>99</td>
</tr>
<tr>
<td>COV, %</td>
<td>16.9</td>
<td>16.9</td>
</tr>
</tbody>
</table>
Figure B 2. Load-deformation curve for group 3 specimens.

Table B 2. Performance parameters for group 3 specimens.

<table>
<thead>
<tr>
<th>Property</th>
<th>Max Load, lb</th>
<th>Displ. @ Max Load, inch</th>
<th>Linear stiffness, lb/inch</th>
<th>Side Member</th>
<th>Main Member</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>MC, %</td>
<td>SG</td>
</tr>
<tr>
<td>Mean Value</td>
<td>481</td>
<td>0.193</td>
<td>10,212</td>
<td>4.93</td>
<td>0.61</td>
</tr>
<tr>
<td>St. Dev</td>
<td>45</td>
<td>0.051</td>
<td>986</td>
<td>0.48</td>
<td>0.02</td>
</tr>
<tr>
<td>COV, %</td>
<td>9.3</td>
<td>26.4</td>
<td>9.7</td>
<td>9.74</td>
<td>2.88</td>
</tr>
</tbody>
</table>
Figure B 3. Load-displacement curve for group 4 specimens.

Table B 3. Performance parameters for group 4 specimens.

<table>
<thead>
<tr>
<th>Property</th>
<th>Max Load, lb</th>
<th>Displ. @ Max Load, inch</th>
<th>Linear stiffness, lb/inch</th>
<th>Side Member</th>
<th>Main Member</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>MC, %</td>
<td>SG</td>
</tr>
<tr>
<td>Mean Value</td>
<td>449</td>
<td>0.160</td>
<td>10,654</td>
<td>4.64%</td>
<td>0.62</td>
</tr>
<tr>
<td>St. Dev</td>
<td>47</td>
<td>0.047</td>
<td>2,428</td>
<td>0.78%</td>
<td>0.04</td>
</tr>
<tr>
<td>COV, %</td>
<td>10.6%</td>
<td>29.2%</td>
<td>22.8%</td>
<td>16.87%</td>
<td>6.79%</td>
</tr>
</tbody>
</table>
Figure B.4. Load-displacement curve for group 5 specimens.

Table B.4. Performance parameters for group 5 specimens.

<table>
<thead>
<tr>
<th>Property</th>
<th>Max Load, lb</th>
<th>Displ. @ Max Load, inch</th>
<th>Linear stiffness, lb/inch</th>
<th>Side Member</th>
<th>Main Member</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>MC, %</td>
<td>SG</td>
</tr>
<tr>
<td>Mean Value</td>
<td>773</td>
<td>0.325</td>
<td>6,085</td>
<td>5.27%</td>
<td>0.63</td>
</tr>
<tr>
<td>St. Dev</td>
<td>113</td>
<td>0.077</td>
<td>1,299</td>
<td>0.26%</td>
<td>0.02</td>
</tr>
<tr>
<td>COV, %</td>
<td>14.7%</td>
<td>23.8%</td>
<td>21.4%</td>
<td>5.00%</td>
<td>3.36%</td>
</tr>
</tbody>
</table>

- data for 10 specimens
- Wen formula
Figure B 5. Load-displacement curve for group 6 specimens

Table B 5. Performance parameters for group 6 specimens.

<table>
<thead>
<tr>
<th>Property</th>
<th>Max Load, lb</th>
<th>Displ. @ Max Load, inch</th>
<th>Linear stiffness, lb/inch</th>
<th>Side Member</th>
<th>Main Member</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>MC, %</td>
<td>SG</td>
</tr>
<tr>
<td>Mean Value</td>
<td>624</td>
<td>0.295</td>
<td>6,203</td>
<td>5.43</td>
<td>0.61</td>
</tr>
<tr>
<td>St. Dev</td>
<td>131</td>
<td>0.100</td>
<td>1,470</td>
<td>0.37</td>
<td>0.04</td>
</tr>
<tr>
<td>COV, %</td>
<td>21.0</td>
<td>33.9</td>
<td>23.7</td>
<td>6.90</td>
<td>6.62</td>
</tr>
</tbody>
</table>
Figure B 6. Load-displacement curve for group 7 specimens.

Table B 6. Performance parameters for group 7 specimens.

<table>
<thead>
<tr>
<th>Property</th>
<th>Max Load, lb</th>
<th>Displ. @ Max Load, inch</th>
<th>Linear stiffness, lb/inch</th>
<th>Side Member</th>
<th>Main Member</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>MC, %</td>
<td>SG, %</td>
</tr>
<tr>
<td>Mean Value</td>
<td>613</td>
<td>0.240</td>
<td>10,537</td>
<td>5.35%</td>
<td>0.62</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5.25%</td>
<td>0.75</td>
</tr>
<tr>
<td>St. Dev</td>
<td>88</td>
<td>0.068</td>
<td>2,402</td>
<td>0.41%</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.17%</td>
<td>0.05</td>
</tr>
<tr>
<td>COV, %</td>
<td>14.4%</td>
<td>28.2%</td>
<td>22.8%</td>
<td>7.57%</td>
<td>2.70%</td>
</tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>3.27%</td>
<td>6.71%</td>
</tr>
</tbody>
</table>
Figure B 7. Load-displacement curve for group 8 specimens.

Table B 7. Performance parameters for group 8 specimens.

<table>
<thead>
<tr>
<th>Property</th>
<th>Max Load, lb</th>
<th>Displ. @ Max Load, inch</th>
<th>Linear stiffness, lb/inch</th>
<th>Side Member MC, %</th>
<th>SG</th>
<th>Main Member MC, %</th>
<th>SG</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean Value</td>
<td>484</td>
<td>0.177</td>
<td>11,654</td>
<td>5.66%</td>
<td>0.63</td>
<td>5.27%</td>
<td>0.71</td>
</tr>
<tr>
<td>St. Dev</td>
<td>50</td>
<td>0.049</td>
<td>3,189</td>
<td>0.42%</td>
<td>0.03</td>
<td>0.61%</td>
<td>0.05</td>
</tr>
<tr>
<td>COV, %</td>
<td>10.4%</td>
<td>27.5%</td>
<td>27.4%</td>
<td>7.40%</td>
<td>4.99%</td>
<td>11.60%</td>
<td>6.39%</td>
</tr>
</tbody>
</table>
Figure B 8. Load-displacement curve for group 9 specimens.

Table B 8. Performance parameters for group 9 specimens.

<table>
<thead>
<tr>
<th>Property</th>
<th>Max Load, lb</th>
<th>Displ. @ Max Load, inch</th>
<th>Linear stiffness, lb/inch</th>
<th>Side Member</th>
<th>Main Member</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>MC, %</td>
<td>SG</td>
</tr>
<tr>
<td>Mean Value</td>
<td>631</td>
<td>0.335</td>
<td>9,022</td>
<td>5.68</td>
<td>0.62</td>
</tr>
<tr>
<td>St. Dev</td>
<td>49</td>
<td>0.113</td>
<td>508</td>
<td>0.24</td>
<td>0.03</td>
</tr>
<tr>
<td>COV, %</td>
<td>7.7</td>
<td>33.8</td>
<td>5.6</td>
<td>4.16</td>
<td>4.51</td>
</tr>
</tbody>
</table>
Figure B 9. Load-displacement curve for group 10 specimens.

Table B 9. Performance parameters for group 10 specimens.

<table>
<thead>
<tr>
<th>Property</th>
<th>Side Member</th>
<th>Main Member</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MC, %</td>
<td>SG</td>
</tr>
<tr>
<td>Max Load, lb</td>
<td>455</td>
<td>0.231</td>
</tr>
<tr>
<td>Displ. @ Max Load, inch</td>
<td>5.57</td>
<td>0.55</td>
</tr>
<tr>
<td>Linear stiffness, lb/inch</td>
<td>9.34</td>
<td>0.03</td>
</tr>
<tr>
<td>COV, %</td>
<td>15.2</td>
<td>26.0</td>
</tr>
</tbody>
</table>
Figure B 10. Load-displacement curve for group 11 specimens.

Table B 10. Performance parameters for group 11 specimens.

<table>
<thead>
<tr>
<th>Property</th>
<th>Max Load, lb</th>
<th>Displ. @ Max Load, inch</th>
<th>Linear stiffness, lb/inch</th>
<th>Side Member</th>
<th>Main Member</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>MC, %</td>
<td>SG</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>MC, %</td>
<td>SG</td>
</tr>
<tr>
<td>Mean Value</td>
<td>709</td>
<td>0.462</td>
<td>4,586</td>
<td>5.99</td>
<td>0.58</td>
</tr>
<tr>
<td>St. Dev</td>
<td>97</td>
<td>0.127</td>
<td>1,618</td>
<td>0.19</td>
<td>0.06</td>
</tr>
<tr>
<td>COV, %</td>
<td>13.7</td>
<td>27.6</td>
<td>35.3</td>
<td>3.15</td>
<td>9.86</td>
</tr>
</tbody>
</table>

- data for 10 specimens
- Wen formula
Figure B 11. Load-displacement curve for group 12 specimens.

Table B 11. Performance parameters for group 12 specimens.

<table>
<thead>
<tr>
<th>Property</th>
<th>Max Load, lb</th>
<th>Displ. @ Max Load, inch</th>
<th>Linear stiffness, lb/inch</th>
<th>Side Member</th>
<th>Main Member</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean Value</td>
<td>450</td>
<td>0.279</td>
<td>5,017</td>
<td>5.81</td>
<td>0.57</td>
</tr>
<tr>
<td>St. Dev</td>
<td>89</td>
<td>0.081</td>
<td>1,813</td>
<td>0.32</td>
<td>0.05</td>
</tr>
<tr>
<td>COV, %</td>
<td>19.9</td>
<td>29.0</td>
<td>36.1</td>
<td>5.56</td>
<td>8.67</td>
</tr>
</tbody>
</table>
Table B 12. Parameters for the Wen formula.

<table>
<thead>
<tr>
<th>Group #</th>
<th>K, lb/inch</th>
<th>P_y, lb</th>
<th>P_y adj, lb</th>
<th>r</th>
<th>n</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15,000</td>
<td>485</td>
<td>343</td>
<td>0.065</td>
<td>2.50</td>
</tr>
<tr>
<td>2</td>
<td>12,500</td>
<td>425</td>
<td>300</td>
<td>0.060</td>
<td>2.50</td>
</tr>
<tr>
<td>3</td>
<td>12,500</td>
<td>390</td>
<td>276</td>
<td>0.048</td>
<td>2.75</td>
</tr>
<tr>
<td>4</td>
<td>13,000</td>
<td>380</td>
<td>269</td>
<td>0.050</td>
<td>2.80</td>
</tr>
<tr>
<td>5</td>
<td>12,000</td>
<td>400</td>
<td>283</td>
<td>0.110</td>
<td>2.00</td>
</tr>
<tr>
<td>6</td>
<td>9,500</td>
<td>420</td>
<td>297</td>
<td>0.098</td>
<td>2.00</td>
</tr>
<tr>
<td>7</td>
<td>15,000</td>
<td>420</td>
<td>297</td>
<td>0.075</td>
<td>3.00</td>
</tr>
<tr>
<td>8</td>
<td>15,000</td>
<td>375</td>
<td>265</td>
<td>0.065</td>
<td>2.20</td>
</tr>
<tr>
<td>9</td>
<td>13,000</td>
<td>440</td>
<td>311</td>
<td>0.055</td>
<td>2.70</td>
</tr>
<tr>
<td>10</td>
<td>10,000</td>
<td>350</td>
<td>247</td>
<td>0.061</td>
<td>2.60</td>
</tr>
<tr>
<td>11</td>
<td>7,800</td>
<td>365</td>
<td>258</td>
<td>0.130</td>
<td>2.10</td>
</tr>
<tr>
<td>12</td>
<td>8,000</td>
<td>210</td>
<td>148</td>
<td>0.130</td>
<td>2.10</td>
</tr>
</tbody>
</table>
Appendix C

Structural Design of a RTA building

General

The design procedure for the RTA buildings is illustrated with this example. The example covers design of the frame, and design of the roof diaphragm and shear walls. A 16 x 24 foot building is analyzed. The building has a roof pitch of 30°. The frames are spaced at 4 feet on-center. One shear wall accommodates an 8 x 8 foot garage door. The design is performed according to 1997 UBC building code. The wind and snow loads were assumed.

Load calculations

Snow load

\[ P_f = C_e I P_g \]  
(Eq. 40-1-1 1997 UBC)

where:
\[ P_f \] = minimum roof snow load, psf;
\[ C_e \] = snow exposure factor;
\[ I \] = importance factor for snow;
\[ P_g \] = ground snow load.

Ground snow load is assumed:
\[ P_g = 30 \text{ psf} \]

Reduction factor for snow load per degree of roof slope over 20 degrees:

\[ R_s = \frac{SL}{40} - \frac{1}{2} = \frac{30}{40} - \frac{1}{2} = 0.25 \text{ psf} \]  
(Eq. 14-1 1997 UBC)

The reduced snow load for 30 degree pitch:
\[ P_g = 30 - (30-20)*0.25 = 27.5 \text{ psf} \]

Because the building is neither in open terrain nor in the densely forested area, the snow exposure factor for “all other structures” category is used:
\[ C_e = 0.7 \]  
(Table A-16-A 1997 UBC)
The importance factor for “all others” category is used:
I = 1.0

Because SAP2000 applies gravity loads along the length of the rafter, snow load should be adjusted with a coefficient equal to a ratio of horizontal projection of the rafter to its actual length. For 30 degree angle, the coefficient is 0.87.

The design roof snow load is

\[ P_t = 0.7 \times 1.0 \times 27.5 \times 0.87 = 16.67 \text{ psf} \]

**Roof Live Load**

For 30° pitch roof and the tributary area less than 200 square feet:

\[ \text{RLL} = 16 \text{ psf} \]

**Roof Dead Load**

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight, psf</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roofing (5-ply)</td>
<td>2.5</td>
</tr>
<tr>
<td>Roll Roofing</td>
<td>1.0</td>
</tr>
<tr>
<td>23/32 inch OSB (3 psf * 0.75)</td>
<td>2.16</td>
</tr>
<tr>
<td>Framing (TimberStrand® SG = 0.7)</td>
<td>1.33</td>
</tr>
<tr>
<td>(estimated for the RTA diaphragm), all elements are 2 x 4’s</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>7.0</td>
</tr>
</tbody>
</table>
Wind Load

Design wind pressures for the main wind force resisting systems are computed as:

\[ P = C_e \cdot C_q \cdot q_s \cdot I_w \]  
(Eq. 20-1 1997 UBC)

where:
- \( P \) = design wind force;
- \( C_e \) = combined height, exposure and gust factor coefficient;
- \( C_q \) = pressure coefficient;
- \( q_s \) = wind stagnation pressure at the standard heights of 33 ft;
- \( I_w \) = importance factor for wind

Exposure B is assumed.

\( C_e = 0.62 \) for 0-15 ft above ground  
(Table 16-G 1997 UBC)

The minimum basic wind speed is assumed:
\( V = 70 \) mph  
(Figure 16-1 1997 UBC)

Therefore, wind stagnation pressure:
\( q_s = 12.6 \) psf  
(Table 16-F 1997 UBC)

Importance factor for “Standard Occupancy” is used:
\( I_w = 1.0 \)  
(Table 16-K 1997 UBC)

Wind loads on primary frames and systems

The pressure coefficients for walls and roofs using Method 1 (normal force method), UBC Table 16-H:

<table>
<thead>
<tr>
<th>Building Surface</th>
<th>Pressure coefficient, ( C_q )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward wall</td>
<td>0.8</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>-0.5</td>
</tr>
<tr>
<td>Windward roof</td>
<td>0.3 / -0.9</td>
</tr>
<tr>
<td>Leeward roof</td>
<td>-0.7</td>
</tr>
</tbody>
</table>

Note, negative signs indicate that the wind pressure is acting away from the surface.
The design wind pressures:

<table>
<thead>
<tr>
<th>Building Surface</th>
<th>Design wind pressure, psf</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windward wall</td>
<td>6.3</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>-3.9</td>
</tr>
<tr>
<td>Windward roof</td>
<td>2.3/-7.0</td>
</tr>
<tr>
<td>Leeward roof</td>
<td>-5.4</td>
</tr>
</tbody>
</table>

**Diaphragm design**

Using the equation discussed by Bender et al. (1991) the shear intensity on the diaphragm, I, is conservatively estimated:

\[
I = \frac{K (q_{WW} - q_{LW}) H_1 L_1 + (q_{WR} - q_{LR}) H_2 L_2}{2 W}
\]

where:

\[
\begin{align*}
I &= \text{roof shear intensity;} \\
K &= 0.5 \text{ for surface-mounted posts;} \\
q_{WW} &= \text{design windward wall pressure;} \\
q_{LW} &= \text{design leeward wall pressure;} \\
q_{WR} &= \text{design windward roof pressure;} \\
q_{LR} &= \text{design leeward roof pressure;} \\
H_1 &= \text{side-wall height;} \\
H_2 &= \text{roof height;} \\
W &= \text{building width;} \\
L_1 &= \text{wall length;} \\
L_2 &= \text{roof length.}
\end{align*}
\]

\[
\begin{align*}
q_{WW} &= 6.3 \text{ psf} \\
q_{LW} &= -3.9 \text{ psf} \\
q_{WR} &= 2.3 \text{ psf} \\
q_{LR} &= -5.4 \text{ psf} \\
L_1 &= 24 \text{ ft} \\
L_2 &= 26 \text{ ft} \\
W &= 16 \text{ ft} \\
H_1 &= 8.0 \text{ ft} \\
H_2 &= 4.6 \text{ ft}
\end{align*}
\]
This is the shear intensity at the two end-walls caused by the wind load. Because there is no data available on the RTA diaphragm resistance at this time, it is assumed that the diaphragm construction used in the demonstration building (Section 4.7) is adequate. The objective of the followup project will be to answer this question directly.

**Shear wall design**

The same shear intensity is applied to the end walls of the building. The wall with the 8 x 8 foot door will be critical for the design. Type A wall is proposed. The construction of this wall is described in Section 3.4.3. The anchors are installed at the corner columns. There are no additional anchors at the door.

The maximum allowable shear wall intensity is computed by adjusting the maximum shear intensity from the RTA shear wall tests. The mean value for the maximum load (Table 11) is 11,342 lb. Because the test walls were 12 feet long, the maximum shear intensity is 945 lb/ft. The allowable design value was obtained by dividing the maximum shear by a factor of 2.5. This factor is consistent with the findings reported by Rose (1998, APA Report 158). Thus, the allowable design load is 378 lb/ft. To account for the opening, the perforated shear wall design procedure is used (Proposal 7-17 (2000) Re-ballot to the NEHRP Provisions).

Percent full-height sheathing = (4 ft + 4ft)*100%/16 ft = 50 %

Maximum opening height ratio = 8 ft/8 ft = 1

Using Table 12.4.4-1 (Proposal 7-17(2000) Re-ballot), the shear resistance adjustment factor, $C_o = 0.5$.

Perforated shear wall resistance = 8 ft * 378 lb/ft * 0.5 = 1,512 lb

Total shear force from the diaphragm = 59.4 lb/ft * 16 ft = 950.1 lb < 1,512 lb

The wall is adequate.
Frame design for gravity and wind load

The first load combination from the 1997 UBC is used.

\[ \text{DL} + \text{Snow} \]

The total design loads applied along the rafters for 4 foot on-center spacing:

<table>
<thead>
<tr>
<th>Dead Load, lb/ft</th>
<th>Design Snow Load, lb/ft</th>
<th>Total Design Load, lb/ft</th>
<th>Total Design Load, lb/in</th>
</tr>
</thead>
<tbody>
<tr>
<td>28.0</td>
<td>67.0</td>
<td>95.0</td>
<td>8.0</td>
</tr>
</tbody>
</table>

Results of the analysis performed with SAP2000.

<table>
<thead>
<tr>
<th>Force(^1)/Location</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( M_{\text{max}} ), lb-in, middle span rafter</td>
<td>5,480</td>
</tr>
<tr>
<td>( N ), lb, middle span rafter</td>
<td>838</td>
</tr>
<tr>
<td>( M ), lb-in, NPC ridge joint</td>
<td>4,041</td>
</tr>
<tr>
<td>( N ), lb, NPC ridge joint</td>
<td>629</td>
</tr>
<tr>
<td>( V ), lb, NPC ridge joint</td>
<td>363</td>
</tr>
<tr>
<td>( M ), lb-in, NPC eave joint</td>
<td>3,861</td>
</tr>
<tr>
<td>( N ), lb, NPC eave joint</td>
<td>1,047</td>
</tr>
<tr>
<td>( V ), lb, NPC eave joint</td>
<td>360</td>
</tr>
<tr>
<td>Column, ( M_{\text{max}} ), lb-in</td>
<td>516</td>
</tr>
<tr>
<td>Column, ( N ), lb</td>
<td>835</td>
</tr>
<tr>
<td>( M ), lb-in, NPC column</td>
<td>516</td>
</tr>
<tr>
<td>( N ), lb, NPC column</td>
<td>835</td>
</tr>
<tr>
<td>( V ), lb, NPC column</td>
<td>6</td>
</tr>
<tr>
<td>Collar-tie, ( N_{\text{max}} ), lb</td>
<td>721</td>
</tr>
</tbody>
</table>

\(^1\)\( M \) = moment force;  
\( N \) = axial force;  
\( V \) = shear force.
Appendix C

Design of rafter for combined bending and compression loading according to Section 3.9.2 of the 1997 NDS.

Rafter dimensions: depth x width : 3.5 inch x 3 inch

TimberStrand® properties are found in Trus Joist Publication JM397/30M (Reorder #2105)

Compression

\[ F_c = 1,950 \text{ psi} \]

\[ F_c^* = F_c C_D C_M C_t C_F = 1,950 * 1.15 * 1.0 * 1.0 * 1.12 = 2,512 \text{ psi} \]

\[ C_F = (12 / 3.5)^{0.092} = 1.12 \]

\[ K_c = 1.0 \text{ (assume pin-pin end conditions)} \]

\[ l_0/d = K_c l_1/d_1 = 1.0 * 9.2 * 12 / 3.5 = 31.7 < 50 \]

\[ K_{cE} = 0.3 \]

\[ F_{cE} = (K_{cE} E')/(l_0/d)^2 = 0.3 * 1,500,000 / (31.7^2) = 452 \text{ psi} \]

\[ c = 0.8 \]

\[ C_p = 0.153 \]

\[ F_c^* = 2,512 * 0.153 = 384 \text{ psi} \]

Axial Force

\[ P = 838 \text{ lb} \]

\[ f_c = P/A = 838 \text{ lb}/10.5 \text{ in}^2 = 79.9 \text{ psi} \]

\[ F_c^* = 384 \text{ psi} > f_c = 79.9 \text{ psi} - \text{OK} \]

Bending

\[ F_b = 2,250 \text{ lb} \]

\[ F_b^* = F_b C_D C_M C_t C_F C_r = 2,250 * 1.15 * 1.0 * 1.0 * 1.12 * 1.0 = 2,898 \text{ psi} \]

\[ M_{\text{max}} = 5,480 \text{ lb-in} \]

\[ S = 6.121 \text{ in}^3 \]

\[ f_b = M_{\text{max}}/S = 5,480/6.121 = 895 \text{ psi} \]

\[ F_b^* = 2,898 \text{ psi} > f_b = 895 \text{ psi} - \text{OK} \]
**Combined loading**

\[(79.9/384)^2 + 895 / (2.898 * (1 - 79.9/452)) = 0.42 < 1 - OK\]

Design of column for combined bending and compression loading according to section 3.9.2 of the 1997 NDS.

Load combination:

**DL + SL + ½wind**

The bending moment is the result of wind force of 3.15 psf.

**Compression**

\[F_c = 1,950 \text{ lb}\]

\[F_c' = F_c C_D C_M C_t C_F = 1,950 * 1.15 * 1.0 * 1.0 * 1.106 = 2,480 \text{ psi}\]

\[C_F = (12 / 4)^{0.092} = 1.106\]

\[K_c = 1.0 \text{ (assume pin-pin end condition – conservative)}\]

\[l_d/d = K_e l_1/d_1 = 1.0 * 9.2 * 12 / 3.5 = 31.7 < 50\]

\[K_{CE} = 0.3\]

\[F_{CE} = (K_{CE} E')/(l_d/d)^2 = 0.3 * 1,500,000 / (31.7^2) = 452 \text{ psi}\]

\[c = 0.8\]

\[C_p = 0.153\]

\[F_c' = 2,480 * 0.153 = 379 \text{ psi}\]

**Axial Force**

\[P = 834 \text{ lb}\]

\[f_c = P/A = 834 \text{ lb/10.5 in}^2 = 79.4 \text{ psi}\]

**Bending**

\[F_b = 2,250 \text{ lb}\]

\[F_b' = F_b C_D C_M C_t C_F C_r = 2250 * 1.6 * 1.0 * 1.0 * 1.106 * 1.0 = 3,982 \text{ psi}\]
Appendix C

\[ M_{\text{max}} = 3.15 \times 4 \times 8^2 / 8 = 100.8 \text{ lb-ft} = 1,210 \text{ lb-in} \]
\[ S = 6.121 \text{ in}^3 \]
\[ f_b = \frac{M_{\text{max}}}{S} = \frac{1,210}{6.121} = 198 \text{ psi} \]

*Combined loading*
\[ (\frac{79.4}{379})^2 + 198 / (3,982 \times (1 - \frac{79.4}{452})) = 0.104 < 1 \text{ - OK} \]

**Design of connections**

*Ridge joint connection*
\[ M = 4,041 \text{ lb-in} \]
\[ N = 629 \text{ lb} \]
\[ V = 363 \text{ lb} \]

*Design for lateral forces*

The total lateral force acting on the NPC
\[ Q = \sqrt{N^2 + V^2} = 726 \text{ lb} \]

The allowable lateral resistance of the NPC is found according to the 1997 NDS Part XII Nails and Spikes. The penetration factor of unity is assumed. The applicability of this method is discussed in Section 2.4.1.

\[ t_s = 0.25 \text{ in} = \text{thickness of the plate} \]
\[ p = 0.85 \text{ in} = \text{penetration of the nail} \]
\[ F_{cs} = 58,000 \text{ psi} = \text{bearing strength of metal plate (1997 NDS)} \]
\[ F_{em} = 16,600 \times (\text{ESG})^{1.84} = 16,600 \times 0.5^{1.84} = 4637 \text{ psi} = \text{dowel bearing strength for dowels between 1/4 and 3/8 inch in diameter (Technical Report 12, AF&PA 1999)} \]
\[ \text{ESG} = 0.5 = \text{equivalent specific gravity for TimberStrand® (Johnson and Woeste, 1999)} \]
\[ F_{yb} = 130,000 \text{ psi} = \text{bending yield strength of the nails (Platt, 1998)} \]
D = 0.25 inch = nail diameter

$K_D = 3.0$ = reduction term for adjusting 5% offset values to nominal design values

(Technical Report 12, AF&PA 1999)

$Z_1 = 571$ lb = nominal design value for one nail in double shear

$Z = Z_1 * 5 = 571 * 5 = 2,472$ lb = same for five nails

$Z' = Z C_D C_M C_t C_g C_\Delta = 2,857 * 1.15 * 1.0 * 1.0 * 1.0 * 1.0 = 3,285$ lb $> 726$ lb

$Z' = 3,285$ lb $> Q = 726$ lb – OK

**Design for moment**

$M_{5\%} = 10,407$ lb-in (Table 4)

The reduction factor of $K_D = 3.0$ used in the lateral design is retained for the moment design.

Then nominal design moment resistance:

$m = M_{5\%}/3 = 10,407 / 3 = 3,469$ lb-in

$m' = Z C_D C_M C_t C_g C_\Delta = 3,469 * 1.15 = 3,989$ lb-in $< 4041$ lb-in – not adequate

The NPC is overloaded by 1.3%

**Design the NPC for combined lateral and moment loading**

The author is unaware of an established methodology for design of wood dowel connections under combined lateral and moment loading. The proposed method uses nonlinear interaction equation:

$$\left(\frac{Q}{Z}\right)^2 + \frac{M}{m} \leq 1$$

Because wood connections exhibit highly nonlinear behavior, the use of a linear interaction would result in a conservative, uneconomical design. The format of the proposed equation was adopted from the 1997 NDS provisions for combined bending and axial compression. The applicability of this equation should be verified by future studies.

$$\left(\frac{726}{3285}\right)^2 + \frac{4041}{3989} = 1.06 > 1$$ – not adequate
**Eave joint connection**

M = 3,861 lb-in

N = 1,047 lb

V = 360 lb

**Design for lateral forces**

The total lateral force acting on the NPC

\[ Q = \sqrt{N^2 + V^2} = 1,107 \text{ lb} < Z' = 3,285 \text{ lb} – \text{OK} \]

**Design for moment**

\[ m' = 3,989 \text{ lb-in} > M = 3,861 \text{ lb-in} – \text{OK} \]

**Design for combined lateral and moment loading**

\[ (1107/3285)^2 + 3861/3989 = 1.08 > 1 – \text{not adequate} \]

To pass the combined lateral-moment check the required allowable moment resistance of the NPC should be above 4,353 lb-in. The formulation derived in Section 4.4 can be used to design a NPC with the required resistance. However, an increase in the NPC strength entails an increase in its stiffness. Therefore, the frame with the new NPC rotational stiffness should be analyzed. Due to the stiffness increase, the NPC attracts higher moment force. Thus, the required allowable moment resistance is greater than 4,353 lb-in. This iteration should be continued until the NPC meets the design criteria.

A modified nail pattern is proposed. It consists of six nails organized in three rows. Because two nails are located in the same row, the minimum nail spacing of 12d = 3.0 inch is used.
Using the formulation derived in Section 4.4, the load-deformation relationship for modified nail pattern is calculated. The linear stiffness is estimated from the load deformation relationship as the slope of a straight line fit between the origin and 0.4$M_{\text{max}}$. The linear stiffness is 2,060,420 lb-in. The 5% offset moment load is computed according to the procedure discussed in Section 4.1.

$$5\% \text{ offset angle} = \arctan\frac{0.05 \, d_{n}}{r_{R}} = \arctan\frac{0.05 \times 0.25}{1.8} = 0.007 \text{ rad}$$

The 5% offset moment, $M_{5\%}$, is 17,774 lb-in. Because the model showed the nonconservative error of 6.7%, $M_{5\%}$ is reduced by the value of the error. The reduced $M_{5\%}$ is 16,583 lb/in. Then nominal design moment resistance:

$$m = 16,583/3 = 5,527 \text{ lb-in}$$

The allowable design moment resistance:

$$m' = 5,527 \times 1.15 = 6,365 \text{ lb-in}$$

The frame is analyzed with the new stiffness value for the rotational spring. The forces acting on the NPC at the ridge joint are:

- $M = 5,203 \text{ lb-in}$
- $N = 637 \text{ lb}$
- $V = 368 \text{ lb}$
**Design for lateral forces**

The total lateral force acting on the NPC

\[ Q = \sqrt{N^2 + V^2} = 736 \text{ lb} \]

\[ Z = 571 \text{ lb} = \text{nominal design value for one nail in double shear} \]

\[ Z = 3,426 \text{ lb} = \text{same for six nails} \]

\[ Z' = Z_C D_C M_C t_C g \Delta = 3,426 \times 1.15 \times 1.0 \times 1.0 \times 1.0 \times 1.0 = 3,940 \text{ lb} \]

\[ Z' = 3,940 \text{ lb} > Q = 736 \text{ lb} – \text{OK} \]

**Design for moment**

\[ m' = 6,365 \text{ lb-in} > M = 5,203 \text{ lb-in} – \text{OK} \]

**Design for combined lateral and moment loading**

\[ (736/3940)^2 + 5203/6365 = 0.85 < 1 – \text{OK} \]

The forces acting on the NPC at the eave joint are:

\[ M = 4,546 \text{ lb-in} \]

\[ N = 1,054 \text{ lb} \]

\[ V = 355 \text{ lb} \]

**Design for lateral forces**

The total lateral force acting on the NPC

\[ Q = \sqrt{N^2 + V^2} = 1,112 \text{ lb} < 3,940 \text{ lb} – \text{OK} \]

**Design for moment**

\[ m' = 6,365 \text{ lb-in} > M = 4,546 \text{ lb-in} – \text{OK} \]

**Design for combined lateral and moment loading**

\[ (1112/3940)^2 + 4546/6365 = 0.79 < 1 – \text{OK} \]
Because the demonstration building (Section 4.7) is used as a storage facility, it can be designed with the importance factor, I, for “other miscellaneous structures” (Table A-16-B, 1997 UBC):

I = 0.9

Then, the design roof snow load:

\( P_f = 15.0 \text{ psf} \)

The total design distributed load applied along the rafters:

7.4 lb/in

The frame is analyzed with the new load. The forces on the NPC at the eave joint are:

\( M = 3,571 \text{ lb-in} \)

\( N = 968 \text{ lb} \)

\( V = 333 \text{ lb} \)

Design for lateral forces

The total lateral force acting on the NPC

\[ Q = \sqrt{(N^2 + V^2)} = 1,023 \text{ lb} < Z' = 3,285 \text{ lb} - \text{OK} \]

Design for moment

\( m' = 3,989 \text{ lb-in} > M = 3,571 \text{ lb-in} - \text{OK} \)

Design for combined lateral and moment loading

\[ \left( \frac{1023}{3285} \right)^2 + \frac{3571}{3989} = 0.99 < 1 - \text{OK} \]
Appendix D

Specification of the components used in the demonstration RTA building

<table>
<thead>
<tr>
<th>Frame Elements</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column</td>
<td>14</td>
</tr>
<tr>
<td>Column (end wall)</td>
<td>5</td>
</tr>
<tr>
<td>Rafter</td>
<td>14</td>
</tr>
<tr>
<td>Purlin</td>
<td>36</td>
</tr>
<tr>
<td>Girts (top long wall)</td>
<td>12</td>
</tr>
<tr>
<td>Girts (middle long wall)</td>
<td>12</td>
</tr>
<tr>
<td>Girts (middle end wall)</td>
<td>6</td>
</tr>
<tr>
<td>Ridge Beam</td>
<td>2</td>
</tr>
<tr>
<td>Collar Tie</td>
<td>7</td>
</tr>
<tr>
<td>Gable Frame Vertical</td>
<td>6</td>
</tr>
<tr>
<td>Gable Frame Horizontal</td>
<td>2</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>116</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Envelope</th>
<th>Quantity</th>
<th>Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Sheathing</td>
<td>21</td>
<td>4 x 8 x 5/8</td>
</tr>
<tr>
<td>Roof Sheathing</td>
<td>8</td>
<td>4 x 10 x 23/32</td>
</tr>
<tr>
<td>Roof Sheathing</td>
<td>7</td>
<td>4 x 8 x 23/32</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>36</strong></td>
<td></td>
</tr>
</tbody>
</table>
## Connectors

<table>
<thead>
<tr>
<th>Type</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column – Collar-Tie/Rafter</td>
<td>NPC 1</td>
</tr>
<tr>
<td>Column – Collar-Tie (end wall)</td>
<td></td>
</tr>
<tr>
<td>Collar-Tie - Raft/column</td>
<td></td>
</tr>
<tr>
<td>Rafter - Column/Collar-Tie</td>
<td>NPC 2</td>
</tr>
<tr>
<td>Rafter – Rafter</td>
<td>NPC 3</td>
</tr>
<tr>
<td>Collar-Tie - Column (end wall)</td>
<td>NPC 4</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
</tr>
</tbody>
</table>

## Brackets

<table>
<thead>
<tr>
<th>Type</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girt-Column/Rafter</td>
<td>B 1</td>
</tr>
<tr>
<td>Girt - Column</td>
<td></td>
</tr>
<tr>
<td>Purlin - Rafter</td>
<td>B 2</td>
</tr>
<tr>
<td>Look-Out Purlin - Rafter</td>
<td></td>
</tr>
<tr>
<td>Gable Framing</td>
<td></td>
</tr>
<tr>
<td>Sill Plate - Column</td>
<td>B 3</td>
</tr>
<tr>
<td>Girt (end wall) - Column</td>
<td>B 4</td>
</tr>
<tr>
<td>Ridge Beam</td>
<td>B 5</td>
</tr>
<tr>
<td>Gable Framing</td>
<td>Flat plate</td>
</tr>
<tr>
<td><strong>Total</strong></td>
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</table>
### Connector Hardware

<table>
<thead>
<tr>
<th>Description</th>
<th>Type</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rafter - Rafter Bolts</td>
<td>7/16 x 1 1/2</td>
<td>14</td>
</tr>
<tr>
<td>Rafter - Rafter Nuts</td>
<td>7/16 nylock</td>
<td>14</td>
</tr>
<tr>
<td>Column – Collar - Tie - Rafter: Bolts</td>
<td>7/16 x 1 1/2</td>
<td>42</td>
</tr>
<tr>
<td>Column – Collar - Tie - Rafter: Nuts</td>
<td>7/16 nylock</td>
<td>42</td>
</tr>
<tr>
<td>Collar - Tie - Column: Bolts</td>
<td>7/16 x 1</td>
<td>10</td>
</tr>
<tr>
<td>Collar - Tie - Column: Nuts</td>
<td>7/16 nylock</td>
<td>10</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td><strong>132</strong></td>
</tr>
</tbody>
</table>

### Bracket and Panel Hardware

<table>
<thead>
<tr>
<th>Description</th>
<th>Type</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ridge Beam: Bolts</td>
<td>1/4 x 2</td>
<td>14</td>
</tr>
<tr>
<td>Ridge Beam: Nuts</td>
<td>1/4</td>
<td>14</td>
</tr>
<tr>
<td>Ridge Beam: Washers</td>
<td>1/4</td>
<td>14</td>
</tr>
<tr>
<td>Purlin and Look-Out Bolts (p)</td>
<td>1/4 x 2</td>
<td>336</td>
</tr>
<tr>
<td>Purlin and Look-Out Bolts (r)</td>
<td>1/4 x 4</td>
<td>168</td>
</tr>
<tr>
<td>Purlin and Look-Out Nuts</td>
<td>1/4</td>
<td>504</td>
</tr>
<tr>
<td>Purlin and Look-Out Washers</td>
<td>1/4</td>
<td>336</td>
</tr>
<tr>
<td>Girt-Column/Rafter</td>
<td>7/16 x 1 1/2</td>
<td>48</td>
</tr>
<tr>
<td>Girt-Column: Bolts (g)</td>
<td>1/4/x 2</td>
<td>72</td>
</tr>
<tr>
<td>Girt-Column: Bolts (c)</td>
<td>1/4 x 4</td>
<td>38</td>
</tr>
<tr>
<td>Girt - Column: Nuts</td>
<td>1/4</td>
<td>110</td>
</tr>
<tr>
<td>Girt - Column: Washers</td>
<td>1/4</td>
<td>72</td>
</tr>
<tr>
<td>Column - Sill Plate: Lag Screws</td>
<td>1/2 x 2</td>
<td>38</td>
</tr>
<tr>
<td>Siding Screws (square drive)</td>
<td>2 in. galvanized</td>
<td>1134</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td><strong>2898</strong></td>
</tr>
</tbody>
</table>
Appendix E

Visual Basic for Applications code used to validate derivation of Equation [55]

Public Sub Moment()

    For f = 0 To 0.2 Step 0.001 ' (Loop for angle)
        For z = 0 To 1.75 Step 0.00175 (loop for h)
            x = f * (z - 0.00175 / 2)
            If x <= 0.0492497 Then
                mom1 = 97573 * x * z * 0.00175 / 1.75
            End If
            if x > 0.0492497 Then
                mom2 = 4805 * z * 0.00175 / 1.75
            End If
            total1 = total1 + mom1 ' (Moment within the linear region)
            total2 = total2 + mom2 ' (Moment in the plastic region)
            mom1 = 0
            mom2 = 0
        Next z
    n = n + 1
    Range("c" & (3 + n)) = total1
    Range("b" & (3 + n)) = f
    Range("d" & (3 + n)) = total2
    total1 = 0
    total2 = 0
    Next f
End Sub
Vita

Vladimir G. Kochkin

The author was born on December 15, 1973 in Kirov, Russia. In 1991, he graduated from a local high school with honors for outstanding achievements in academic and extra-curriculum activities. He pursued his education in Vyatka State Technical University, which is one of the leading engineering schools in Russia, in the college of civil engineering. At the same time, he was the co-captain of the college intermural basketball team and helped to bring four championships to the college in five years. After his junior year, he participated in a 10-month co-op project with a general contractor. During his senior year, he was awarded with a scholarship sponsored by the USIA for one-year exchange program with Virginia Tech. He spent this year studying in the Department of Wood Science and Forest Products where he was involved in a number of projects related to the performance of wood in structures. After successful completion of this program, he returned to Russia to defend his diploma project and graduated with honors from Vyatka State Technical University in 1997. After graduation, the author worked in Kirov as a database engineer. Since August 1998, he has been working on his Master of Science degree in the Department of Wood Science and Forest Products at Virginia Tech with specialization in Timber Engineering. He accepted an engineering position with the National Association of Home Builders Research Center, where he is expected to join the Structures and Materials Group in September of 2000.