

# MODELING OF METAL-PLATE-CONNECTED WOOD TRUSS JOINTS: PART II—APPLICATION TO OVERALL TRUSS MODEL

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**ABSTRACT.** A previously developed, three-dimensional (3-D) finite-element (FE) joint model and several two-dimensional (2-D), beam-element based FE models with different joint-stiffness assumptions (pinned, rigid, and semi-rigid) were used to analyze the full-scale behavior of a metal-plate-connected (MPC) wood scissors truss. The truss displacement and force results were investigated for sensitivity with respect to the modeling approach.

To validate the assumption that the overall truss displacement predictions from different models were realistic, the results were compared to and shown to be within 10% of the experimental results. Greater variability existed between the truss member force and moment results, as predicted by various models, depending on the modeling approach used. The 3-D model's predictions of moments at the heel joint were significantly higher than the predictions of 2-D models.

The differences in results demonstrated the importance of selecting a modeling approach capable of accurately predicting member force and moment distributions. The 3-D model, developed primarily for detailed analyses of individual MPC joints, was successful at predicting selected displacements of the test truss. Its use demonstrated a potential for future application of such models in the analysis and design of MPC trusses.

**Keywords.** Truss systems, Models, Metal-plate-connected trusses.

Today, metal-plate-connected (MPC) trusses in service are typically designed and analyzed using specialized computer software. These programs, generally relying on the finite-element (FE) method to analyze and predict the global characteristics of MPC trusses under service loads, are often developed by the truss-plate manufacturers. One commercially available program, the Purdue Plane Structures Analyzer (PPSA) (Suddarth, 1972) was specifically developed for the analysis of plane frame structures, and the first to be largely accepted by the MPC truss industry.

Foschi (1979) developed 'Structural Analysis of Trusses' (SAT), a truss analysis computer program. Triche and Suddarth (1988) developed a new truss analysis program (PPSAFT) by modifying and extending the SAT and PPSA programs, so that it would provide design information for the lumber members as well as the plate connectors. They developed additional subroutines to evaluate tooth forces throughout the plates in truss joints, taking into consideration plate size and shape. Cramer et al. (1993) presented an alternate stiffness-calculation method for the SAT model, as well as developed automated means

of computing the geometric characteristics of each plate-wood contact surface and member forces in MPC wood trusses.

Lum and Varoglu (1988) compared the displacement results of full-scale parallel chord truss tests with the predictions of SAT, and the results matched reasonably well. Gupta et al. (1992) investigated the effects of including joint semi-rigidity in the analysis of MPC trusses, through modifying fixed-end forces and wood-member element stiffness matrices. They found that maximum truss deflection of a semi-rigid truss was decreased by 34%, when compared to a pinned truss analog, and maximum member bending moment reduced by 13%, when compared with a rigid truss. Riley et al. (1993) compared the results from truss analyses assuming different joint fixity conditions: pinned, rigid, semi-rigid (matrix method), fictitious members, and the methods described by Truss Plate Institute (ANSI/TPI, 1995) specification for truss analysis. The maximum truss deflection and maximum member moment for trusses with fictitious members were found to fall between the results for trusses with pinned and rigid joint connection assumptions, and were less than those for the trusses where the matrix method was used to model semi-rigidity.

Finite-element-based computer programs have helped design thousands of MPC structures that have performed well in service over the past few decades. However, considering the constant development and growth of the computer industry, more comprehensive models in the future may become simpler to use, faster and more approachable, and yet at the same time provide for more extensive, optimized and enhanced investigation. This study introduces a possible predecessor to this next generation of truss models.

A three-dimensional (3-D) FE model capable of representing and analyzing the behavior of MPC joints in

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service was developed in the first part of this study (Vatovec, 1996b). The model was relatively simple to define, yet it incorporated the major factors that influence the performance of MPC joints: wood and steel plate properties; wood-to-plate load-slip interaction that depends on plate-to-grain-orientation; wood-to-wood contact; load eccentricity; and gaps between the members.

The basic aspects of a joint model were defined and validated for splice joints with various plate-grain orientations and loading conditions. Certain model simplifications were investigated as well.

The second stage of the project, described in this article, uses the previously defined joint modeling approach to model an entire MPC wood scissors truss. The 3-D truss model, if it performed well, would demonstrate a potential for future application of more comprehensive models in the analysis and design of MPC trusses.

The main focus of the article, however, is the investigation of the sensitivity of overall truss analysis results to modeling-approach variations. The overall truss displacement and force results from the 3-D truss model were compared to the results from several two-dimensional (2-D) beam-element based finite-element truss models: pinned, rigid, and semi-rigid (joint stiffnesses based on the scissors-truss-joint tests from Vatovec et al., 1996a). Differences in model results, if found, would demonstrate the importance of selecting a modeling approach capable of accurately assessing truss force and moment distributions, which are the basis for MPC truss design.

## METHODS

All models in this study were developed to simulate the behavior of a 12.3 m span MPC wood scissors truss, because test results for this truss were available from the plate manufacturer (Alpine Engineered Products, Inc., 1993). The deflection measurements from that test are used to compare the overall truss displacement predictions from different models. The truss and the joint nomenclature are shown in figure 1.

### 3-D FINITE-ELEMENT MODEL FOR MPC TRUSS

**Truss Model.** All modeling in this study was performed using a 486-generation personal computer and the finite-element software, ANSYS® (1992). The geometry, constraints and loading conditions of the modeled truss simulated those of the actual truss. However, only one-fourth of the truss was modeled, taking advantage of the geometric symmetry about the plane between the faces of the truss (x-y plane, fig. 1), and about the plane that divides the truss in half at the midspan (y-z plane, fig. 1). Proper boundary conditions were imposed on the model: nodes at symmetry planes were restrained from translating in the direction perpendicular to the planes of symmetry, and from rotation about the axes in the planes of symmetry.

To examine effects of the material asymmetry, the two "quarters" from opposite sides of the truss' midspan were modeled separately with appropriate material properties (table 1), and no significant differences in the displacement results were found. Therefore, in the following paragraphs, only the results based on the "left" quarter of the truss will be discussed.

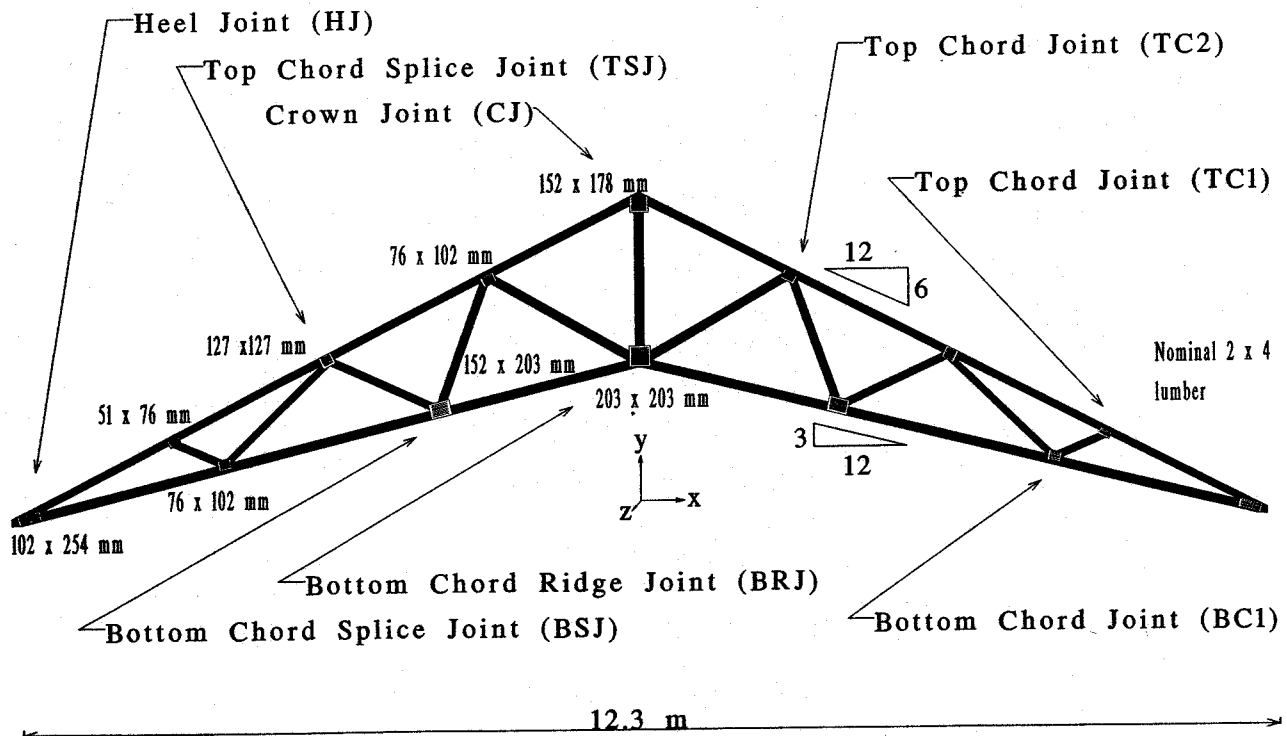


Figure 1—Scissors truss.

**Table 1. Truss lumber properties (Alpine Engineered Products, 1993)**

Piece ID*	MOE (MPa)	Moisture Content (%)
TC-1	11 500	11.0
TC-2	11 700	14.0
TC-3	11 600	15.0
TC-4	12 000	11.0
BC-1	14 200	16.0
BC-2	12 100	11.0
BC-3	12 100	11.0
BC-4	15 000	17.0
W-1	10 500	14.0
W-2	9 300	11.0
W-3	9 900	15.0
W-4	9 900	15.0
W-5	9 900	9.0
W-6	9 300	11.0
W-7	10 500	14.0
W-8	10 000	12.0
W-9	10 500	13.0
W-10	9 900	9.0
W-11	10 000	12.0

\* The truss chord and web members are numbered from left to right in the truss.

NOTE: TC = top chord; BC = bottom chord; and W = webs.

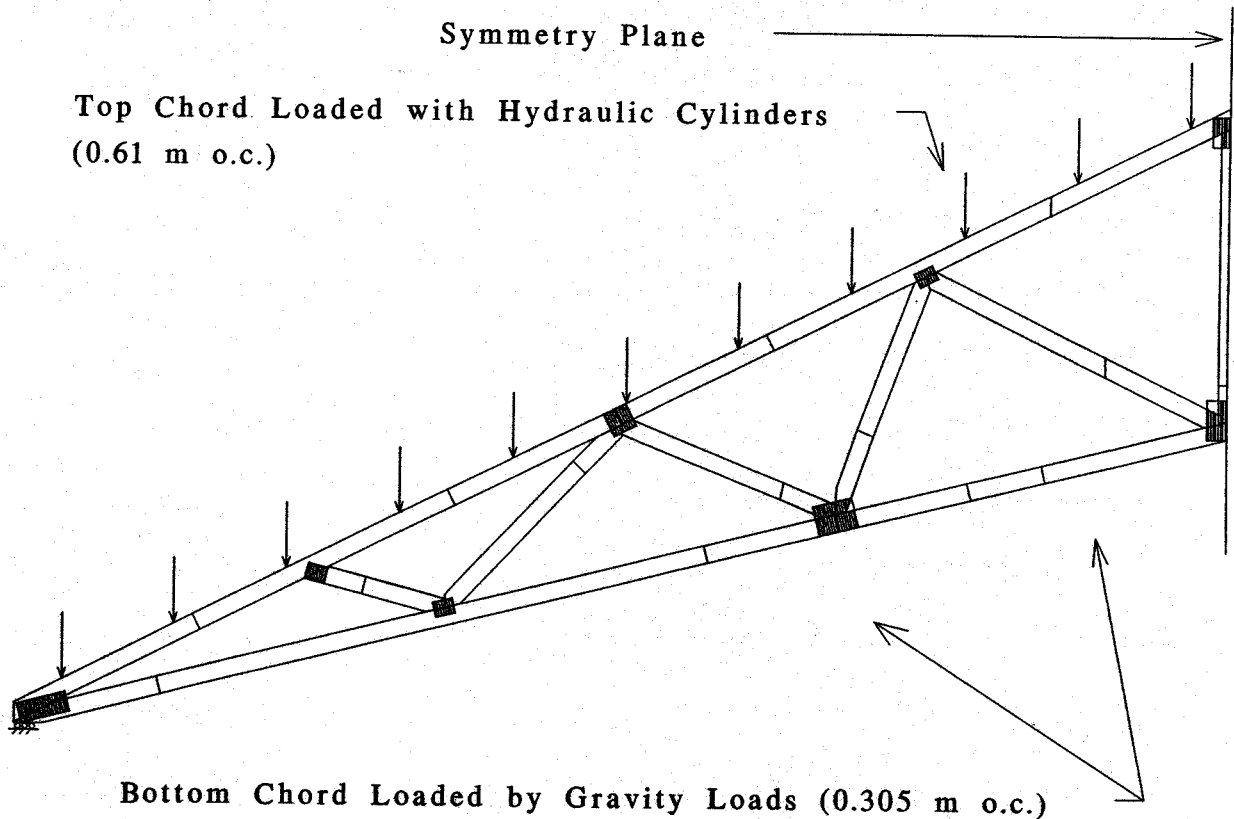
All elements forming wood members were placed in a single layer in the plane of the truss, and distributed to form the geometry of the modeled one-fourth of the truss. The steel plates were modeled as a single layer of elements forming the shape of the plate, without slots. Each tooth was represented as a single point on the surface between the wood and the plate (Vatovec et al., 1996b). Essentially,

only two layers of elements (with respect to the z-axis of the truss), one belonging to wood members, and one belonging to metal plates, were used to construct the entire model. Between those two groups of elements, wood-to-plate interface elements were inserted.

Due to the degree-of-freedom size limitations for ANSYS programs intended for educational purposes, the truss was divided into a number of substructures (ANSYS, 1992). Essentially, substructuring is a procedure that condenses a group of finite elements into one element. This single element is then called a *superelement*, and can be used in an analysis just as any other ANSYS element type, except that superelements need to be created prior to the actual model solution phase, by performing a substructure generation analysis.

The modeled one-fourth of the truss was divided into nine superelements. Each joint in the truss, with a portion of the appropriate wood members, was represented by one superelement. An additional superelement was added to facilitate the connection between the bottom chord splice joint (BSJ), and the bottom chord ridge joint (BRJ). The modeled truss, along with the boundaries forming the nine superelements, is shown in figure 2.

When using substructuring (superelements) in ANSYS, a compromise has to be made with respect to the capabilities of all nonlinear elements generated "inside" the superelements. The nonlinear nature of these elements is lost once they become an integral part of the larger, linear superelements. Therefore, the entire model of the truss, as it comprises nine superelements, becomes linear.



**Figure 2—Geometric layout of the 3-D model of the scissors truss.**

For this reason, it was decided to analyze the modeled truss at low load levels, while still in the elastic range. The following **loading conditions** were applied: a total of 5430 N applied uniformly at the top chord, and a total of 3650 N of load distributed uniformly over the bottom chord. This load was defined arbitrarily as the dead load condition, based on the horizontal projection of a typical load of approximately 720 Pa (15 lb/ft<sup>2</sup>) for TC, and 480 Pa (10 lb/ft<sup>2</sup>) for BC, with trusses 610 mm on center.

The methods used to define and assign properties to model constituent **elements** were elaborated in detail in the first stage of this study (Vatovec et al., 1996b) and they will only be briefly discussed here, except for the procedures used to assign properties to wood-to-plate interaction elements in the MPC joints. All wood members and steel plates in the truss were modeled with 3-D *Structural Solid* elements (ANSYS, 1992). The 3-D elements are standard quadrilateral elements with three translational degrees of freedom at each node. The wood members were assigned linear orthotropic material properties, including moduli of elasticity (MOE) in the three principal directions (longitudinal, radial, and tangential). The longitudinal MOE values (MOE<sub>l</sub>) for each member were assigned based on the MOE values for the experimentally investigated truss, provided by Alpine (1993), and shown in table 1. The values for the other two orientations were derived from MOE<sub>l</sub>, using the guidelines given in the *Wood Handbook* (Forest Products Laboratory, 1987): radial MOE<sub>r</sub> = 0.068 × MOE<sub>l</sub>; tangential MOE<sub>t</sub> = 0.050 × MOE<sub>l</sub>.

The plates were assigned bilinear elastic-plastic material properties. The modulus of elasticity for steel was 200 000 MPa, and the yield strength was 317 MPa (based on the test value provided by the manufacturer). However, the strength value was never used, since the material became linear once incorporated into a superelement.

**Background on Joint Model.** The wood-to-plate interaction for each tooth was modeled through a set of three *nonlinear spring elements* (ANSYS, 1992), each defined by a single load-displacement (L-D) curve, and each representing the wood-plate stiffness in one of the three main plate orientations. The three principal plate (teeth) orientations were recognized as: the major plate axis (x) that coincides with the direction of the plate slots, the minor plate axis (y) that is in the plane of the plate, but perpendicular to x, and the z axis that is perpendicular to the plate. The input stiffness L-D curves depended on the plate-to-grain orientation, and, in general, wood species, specific gravity, type of the plate, etc. Once the stiffness was assigned to joint spring elements, the overall MPC joint's wood-plate interaction stiffness was defined, and did not need to be adjusted for various loading conditions applied to the joint.

The per-tooth stiffness properties in the x and y directions for various truss joint models were based on, or derived from (for joints with intermediate wood-plate orientation angles) a set of MPC tensile-splice joint tests using Southern-Pine lumber. The results from Vatovec et al. (1996b) showed that the wood-plate stiffness in the z direction (perpendicular to the plane of the plate) had no influence on the behavior of modeled joints when loaded in the plane of the joint, and was assumed to be constant regardless of wood-plate orientation and species.

The same per-tooth z-direction load-displacement data obtained from an experimental study (Vatovec, 1996) were used for all joints in the truss.

The four basic tension tests, defined by the Canadian Standard Association (CSA, 1980), were performed to determine per-tooth stiffness properties for Southern Pine MPC joints. In this article these tests are designated as ST0, ST90, STP0, and STP90, where the numbers represent the angle between the main axis of the plate and the grain, 'ST' denotes *Southern Pine Tensile* test, and 'P' denotes that the applied load is perpendicular to the plate main axis. This nomenclature facilitated designating joints where the angle between the plate and grain was not 0 or 90° (Vatovec, 1996b). The metal plates used to fabricate all the joints were provided by Alpine Engineered Products, Inc., Pompano Beach, Florida. Six joints were tested for each of the four test configurations.

The average per-tooth L-D curve parameters for tests ST0, ST90, STP0, and STP90, using the customary Foschi notation (Foschi, 1979), along with wood member properties, are listed in table 2.

The data shown in table 2 represent the average per-tooth, in-plane, load-slip characteristics in the directions parallel and perpendicular to the plate main axis, which are to be assigned to truss joint models with plate-grain orientations of 0 and 90°. However, the joints in the modeled truss generally involved configurations where the plate main axes and the wood grain were neither parallel nor perpendicular to each other. Hence, a method to determine load-slip properties between wood and plates for those joints was needed.

Originally, it was intended to use Hankinson's formula to interpolate between the parameters that describe the per-tooth load-deflection curves (Foschi, 1979) for the four basic orientations. However, the results from an experimental validation involving Douglas-fir MPC joints (Vatovec, 1996) indicated that the Hankinson's formula approach did not accurately predict the overall in-plane, per-tooth stiffness parameters for intermediate plate-to-grain angles. Therefore, in this study, an alternative method was used.

Figure 3 shows the comparison between the average per-tooth load-deflection curves for the tested Southern Pine specimens, and similarly tested Douglas-fir specimens (Vatovec et al., 1996b), for the four basic orientations. It was observed that there was not any meaningful difference

Table 2. Southern Pine joint tensile test results

Test	6 Specimens per Test	m <sub>0</sub> (N)	k (N/mm)	m <sub>1</sub> (N/mm)	Predominant Failure Mode	SG	MOE (MPa)
ST0 (AA)*	Avg. COV	227.7 0.08	1212.0 0.08	80.3 0.19	Teeth Withdrawal	0.47 0.09	11 600 0.11
ST90 (AE)*	Avg. COV	284.7 0.34	311.8 0.12	-51.3 0.78	Teeth Withdrawal	0.53 0.11	11 900 0.12
STP0 (EE)*	Avg. COV	332.7 0.58	390.9 0.13	-34.2 2.75	Teeth Withdrawal	0.55 0.09	11 900 0.13
STP90 (EA)*	Avg. COV	181.0 0.27	1372.2 0.24	47.7 1.46	Teeth Withdrawal	0.54 0.13	11 900 0.12

\* These test orientations correspond to those used in Section 7 of ANSI/TPI 1-1995 *National Design Standard for Metal-Plate-Connected Wood Truss Connections*.

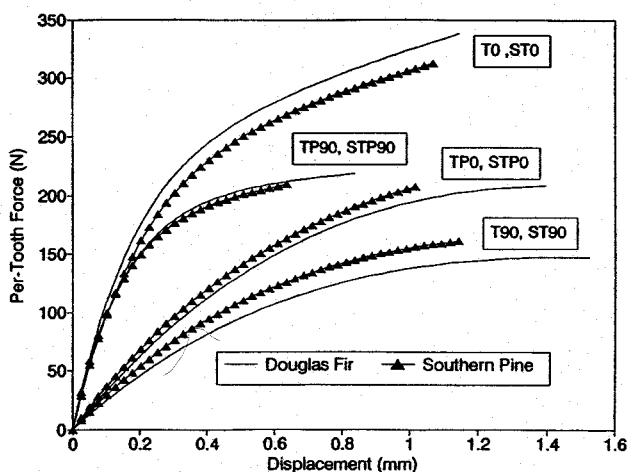


Figure 3—Average results for four basic tests on Douglas-fir and Southern Pine.

between the stiffness characteristics of the two species. This assessment was based on a visual observation of the curves, as well as the percentage differences in displacements at one-third of the ultimate load, two-thirds of the ultimate load, and at ultimate load, measured with respect to the Southern Pine curves.

Based on the observation that the stiffness characteristics for the two species were similar for 0 and 90° angle plate-grain orientations, it was decided to model the Southern Pine wood-plate characteristics for 30 and 60° based on existing stiffness data from Douglas-fir tests with wood-to-plate orientations of 30 and 60° (T30, T60, TP30, and TP60 test results from Vatovec et al., 1996b). The 0 and 90° angle stiffnesses, however, both for the orientation parallel and perpendicular to the plane main axis, were modeled based on the Southern-Pine per-tooth test stiffness data. For the angles that fell between the defined stiffness values for 0, 30, 60, and 90°, the joint per-tooth wood-plate stiffnesses were obtained using linear interpolation. This technique provided for a simple characterization of any L-D curve.

Even though specific non-linear L-D curves were assigned to all the joints, the spring elements lost their non-linearity once they were included into a superelement. Instead of being represented by a pre-defined non-linear load-deflection curve, they became linear, with stiffness represented by the initial slope of the original curve. Essentially, this provided added stiffness to all joints in the model at any loads above the initial, elastic range (therefore, the model was investigated at the dead load level).

Wood-to-wood interaction in the MPC joints was represented by the 3-D *Point-to-Point Contact Elements* (ANSYS element 52). The contact elements were placed between the nodes belonging to the surfaces of wood-member elements that may come in contact when the truss is loaded.

The properties of contact elements were assigned according to the recommendations given by ANSYS (1992). Each contact element was defined by two stiffnesses: the Coulomb coefficient of friction (0.4) for wood-to-wood contact governing stiffness properties in the plane of contact (Eshbach, 1990); and the stiffness in the direction perpendicular to the plane of contact of 87 N/mm/mm<sup>2</sup>, validated by a set of compression tests on

small clear wood specimens (Vatovec, 1996). When the surfaces do not touch, the contact elements do not influence the behavior of the model.

## TWO-DIMENSIONAL, BEAM-ELEMENT-BASED FINITE-ELEMENT TRUSS MODELS

The behaviors of three types of two-dimensional, linear-elastic, beam-element based finite-element truss models (analog) were investigated: with joints pinned, rigid, and semi-rigid. All 2-D models consisted of linear elastic beam elements representing wood truss members, connected at MPC joint locations with joint elements. The bottom and top chords were modeled as continuous members, except for the semi-rigid truss, where they were modeled as assemblages of two continuous members, connected at the Bottom-Chord Splice Joint, and Top-Chord Splice Joint, respectively. The truss model geometry showing the locations of all joints is shown in figure 4.

The analog followed the geometry of the truss (beams defined by locations of member center lines), except for the heel joints, where the two framing beam elements intersected directly above the supports (fig. 4).

The 2-D models of the truss were assigned simple support conditions (pin and roller), providing horizontal stability for the model. The models outputs were investigated at the same loading condition as used for the 3-D model.

Similar to the 3-D model, wood members in all 2-D models were assigned longitudinal MOE values according to the values given in table 1. Furthermore, beam elements were assigned cross-section properties for nominal two-by-four lumber (cross-sectional area of 3390 mm<sup>2</sup> and moment of inertia of 2 230 000 mm<sup>4</sup>).

The ANSYS, (1992) joint element allowed for a direct input of joint stiffness values at wood member intersection locations. Three input element stiffnesses corresponding to rotations about the global x- and y-axes (axes in the plane of the truss), and translation along the axis perpendicular to the plane of the truss were kept constant for all model types and were assigned large stiffness values (four orders of magnitude larger than the adjacent wood member stiffnesses), simulating rigid connections in those directions. The other two stiffnesses, translational stiffness in the plane of the truss, and rotational stiffness about the

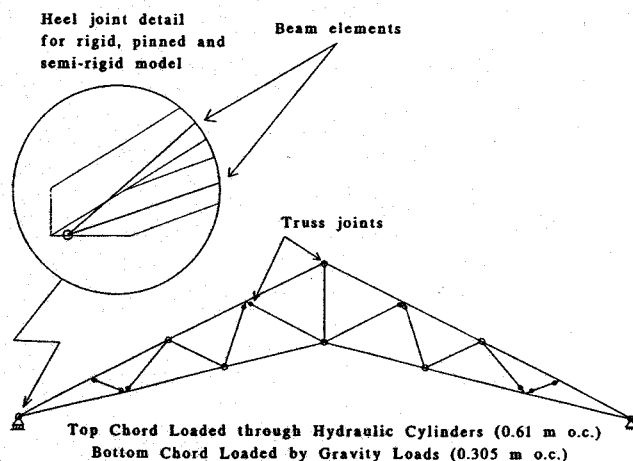


Figure 4—Geometric layout of a 2-D model of the scissors truss.

axis perpendicular to the plane of the truss, were varied depending on the model approach:

**Rigid Truss.** In the case of the rigid truss model, all joint elements were assigned large stiffness values (including web members), to simulate a rigid connection between wood members.

**Pinned Truss.** All joint elements were assigned large translational stiffness values, and zero rotational stiffnesses values. Exceptions were made for chord members which were continuous over the joint locations. The web members framing into those joints were assigned pinned-joint properties.

**Semi-Rigid Truss.** The joint elements connecting truss chord members (at BSJ and TSJ) were assigned translational and rotational stiffness values obtained through experimental testing (Vatovec et al., 1996a), as shown in table 3. The translational stiffness for joints connecting top chord members was not available through the testing (due to insignificant displacement readings of the members loaded in compression), and therefore they were assumed rigid in translation. The rotational stiffnesses for those joints were assigned based on test results. All web members were assumed pinned (large translational, zero rotational stiffness). Also, the chord joints whose stiffnesses were not obtained through testing, namely joints BC1, TC1 and TC2, were assigned pinned characteristics.

Table 3. Truss joint translational and rotational stiffness values

Joint Type	Translational Stiffness (N/mm)	Rotational Stiffness (N-mm/rad)
BSJ	61 700	51 250
HJ	29 200	49 800
CJ	N/A	33 000
BRJ	40 300	52 250
TSJ	N/A	43 100

### TRUSS TESTING

Alpine Engineered Products, Inc. (1993) performed a full-scale test on a scissors MPC truss, and the experimentally obtained values will be used in this study to validate the displacement results from the models.

The plates used to fabricate the truss were Alpine 20 gauge plates, with a steel yield stress of 317 MPa, and ultimate strength of 378 MPa (obtained from testing). The lumber properties and moisture contents were determined prior to truss assembly, and are shown in table 1. Note that the lumber was not conditioned to the equilibrium moisture content prior to testing.

The truss loading consisted of a constant bottom-chord dead load and linearly increasing with time (from zero-to-failure) top-chord load. Weights (89 N each) were attached 0.305 m on center along the bottom chord to simulate a 292 N/m (horizontal projection) distributed dead load. Uniformly spaced hydraulic cylinders (0.61 m on center) were used to apply load to the top chord.

The truss failed at the location of the left heel joint, by fracture of the metal plate, i.e., shearing along the gap of the heel joint.

## RESULTS AND DISCUSSION

### COMPARISON OF DISPLACEMENTS

The deflected shapes along with the original geometries of the 3-D truss model and of a 2-D truss model are shown in figures 5 and 6, respectively. Only one 2-D result is presented, since there were no noticeable differences in deflected shapes between any of the trusses investigated.

The comparison between the model displacement predictions for the dead load condition is shown in table 4. The only displacement results available from the experimental truss analysis were the mid-span vertical deflection, and the horizontal deflection of the supports in the plane of the truss. Since 2-D truss models were modeled with simple support conditions, the horizontal displacement of the roller support was divided by two, and reported as the horizontal displacement of the heel joint.

Each of the models used in this study predicted both the vertical displacement of the bottom-chord ridge joint and the horizontal displacement of the heel joint within 10% of the test results. The 3-D truss model provided the closest approximation to the actual displacement of any of the models. However, one must recognize that the experimental data was very limited (only one truss, and only two displacement measurements), and more

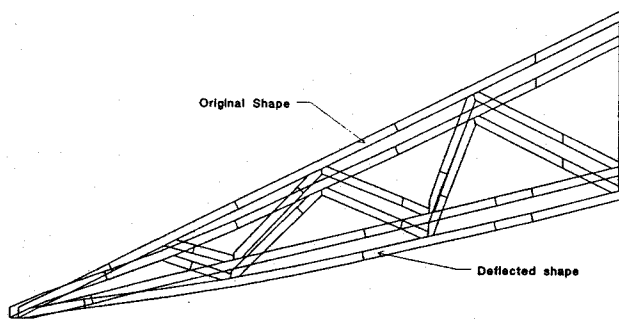


Figure 5—Deflected shape from the 3-D truss model.

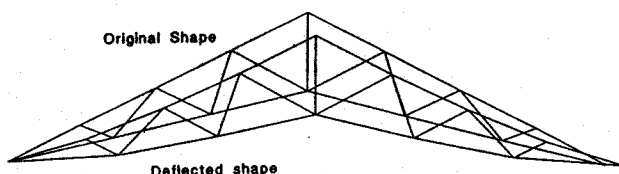


Figure 6—Deflected shape from 2-D truss models.

Table 4. Displacement results comparison for different truss models (mm)

Truss Joint*	Pinned	Rigid	Semi-Rigid	3-D Model	Test
T.C. web joint no. 1 (TC1)	9.78	9.42	11.81	5.94	N/A
B.C. web joint no. 1 (BC1)	12.12	11.71	14.27	12.90	N/A
T.C. splice joint (TSJ)	14.00	13.51	16.31	14.73	N/A
B.C. splice joint (BSJ)	15.67	15.11	18.16	16.46	N/A
T.C. web joint no. 2 (TC2)	15.95	15.42	18.36	16.69	N/A
B.C. ridge joint (BRJ)	16.43	15.90	18.90	17.22	17.02
Crown joint (CJ)	16.03	15.49	18.47	16.74	N/A
Heel joint (HJ): Horizontal displacement	5.55	5.49	6.27	6.04	5.97

\* Joints are numbered from left to right.

NOTE: B.C. = Bottom chord; T.C. = Top chord.

experimental validation is needed before more specific statements of model accuracy can be made.

For comparison, the predictions of the 3-D truss model were used as the base value to characterize the predictions of other models at all joint locations (since the only recorded test results were at mid-span and at the bearings). Using this approach, the pinned and rigid models underestimated truss displacements and could be characterized as unconservative. The displacement predictions for the semi-rigid model were somewhat larger than the rest of the models for all joints in the truss. At mid-span (BRJ), the semi-rigid model prediction was 9.7% conservative; whereas, the pinned and rigid approaches were 4.6, and 7.7% unconservative, respectively.

As stated above, the differences in the displacement results at the mid-span and at the bearings were less than 10% for all the models. However, elsewhere in the truss, it varied, and at certain joints for some models it was much higher (i.e., semi-rigid model prediction for TC1 was 99% conservative compared to the 3-D model).

In a previous study (Vatovec et al., 1993), the effect of assigned joint stiffnesses in 2-D truss models was investigated, and it was concluded that the translational stiffnesses of all joints influence vertical displacements of trusses; whereas, the rotational stiffnesses generally do not. And indeed, in this study, for all models where translation at the joint was fixed (pinned and rigid models), the overall displacements were smaller than for the models with partial translational fixity of joints. Based on the limited experimental data, the 3-D truss model predicted the overall displacement behavior of the truss very well in the elastic range. As the truss behavior at higher load levels was not investigated, no conclusions could be drawn with respect to the influence of superelement linearity, or the absence of plate slots (which caused the plates to become stiffer), on the overall truss deflections.

#### COMPARISON OF TRUSS MEMBER FORCES AND MOMENTS

Different truss models' predictions of internal force distributions among the truss members were compared. However, truss-member force data were not available from the experimental truss investigation. Table 5 shows the comparison of **axial member forces** for different models. Positive forces correspond to tension, and negative to compression. The force magnitudes were compared only for the chord members of five truss joints (BSJ, HJ, CJ, BRJ, TSJ).

The two-dimensional truss models consisted of linear beam-element members connected at joint locations. Therefore, single constant values for axial forces were

evaluated for each member. However, the magnitude of member axial forces in the 3-D model varied somewhat along the length of the members, and axial stresses varied over the cross-section as well. The wood truss members consisted of a number of elements, both lengthwise and widthwise, providing for a better insight into the force and moment distribution at the joints. The forces from the 3-D model listed in table 5 represent member axial forces observed in truss chords at the edges of the connector plates for the five truss joints. Except for the crown and ridge joints, which are symmetric, the forces were observed both at the left and at the right edge of the plate. These locations were chosen because, from an engineering standpoint, one is concerned with force concentrations at the joints. The reported member axial forces represent a summation of "element axial forces" across the width of truss members. The element axial forces were calculated from normal element stresses.

It was observed that axial forces at a particular location in the truss did not vary significantly among the different 2-D truss models. This observation agrees with the results of a previous study that evaluated truss member forces for two-dimensional models with different joint stiffness conditions (Vatovec et al., 1993). However, the magnitude of most member axial forces obtained from the 3-D model were significantly lower than the predictions from 2-D models (i.e., 37% difference for the bottom chord at the heel joint, and 40% difference for the crown joint). This difference could be attributed to the fact that the forces in the 3-D model were evaluated at the edges of the connector plates, where the web-member force components may have influenced the axial force magnitude in the chords (through the wood-plate interaction).

Table 6 shows the comparison of **moments** computed from the different models at chord-member joint locations for five truss joints. The positive sign for moment corresponds to clockwise moment action; whereas, the negative sign corresponds to counter-clockwise action. The moments from all models were calculated at the same locations in the truss where the forces were observed in the 3-D model (at the edges of the connector plates). This explains the non-zero moments provided even for the pinned joint model. The magnitudes of moments along the truss members varied and were not evaluated at other locations in the truss. As in the case for member axial forces, the moments in the 3-D model were calculated from normal element stresses observed at the center of the wood members (depthwise). Therefore, the moment at a cross-section was calculated through a summation of the

Table 5. Model predictions of truss-member axial forces (kN)

Truss Member*	Pinned Truss Model	Rigid Truss Model	Semi-Rigid Truss Model	3-D Truss Model
HJ BC	13.43	13.48	13.48	8.50
HJ TC	-14.95	-14.81	-14.81	-10.41
BSJ L	12.68	12.68	12.68	8.94
BSJ R	10.94	10.94	10.94	8.63
CJ	-9.65	-9.65	-9.65	-5.74
BRJ	10.94	10.94	10.94	10.10
TSJ L	-14.28	-14.28	-14.19	-12.99
TSJ R	-12.45	-12.41	-12.46	-12.10

\* BC = Bottom chord, TC = Top chord, L = Left member, R = Right member.

Table 6. Model predictions of truss member moments at joint locations (N-m)

Truss Member*	Pinned Truss Model	Rigid Truss Model	Semi-Rigid Truss Model	3-D Truss Model
HJ BC	10.32	12.12	35.01	346.07
HJ TC	14.85	1.90	-1.93	211.51
BSJ L	-4.83	-7.86	-10.44	-5.99
BSJ R	8.31	18.93	6.15	-10.85
CJ	-1.31	16.12	13.77	-28.02
BRJ	-0.44	-10.43	-13.96	-10.06
TSJ L	-6.45	-9.48	-1.86	-75.93
TSJ R	3.93	14.42	-1.62	24.63

\* BC = Bottom chord, TC = Top chord, L = Left member, R = Right member.

products of element axial forces and the appropriate element distances from the member centerline.

As shown in table 6, the magnitude and orientation of moments throughout the truss varied depending on the modeling approach. Several general observations can be made. The obvious deficiency of pinned-type models was that they assume free rotation at joint locations, even though in the real structure that is not the case. Therefore, since the moments for the pinned-truss model were determined at locations very close to zero-moment locations (at the plate edges near the ends of members), they were generally underestimated.

It can be observed that the truss moment results were generally a function of the approach used to model the connections, but for most joints the differences in moments between models were relatively small, except for the heel joint, which will be discussed next.

The largest discrepancy in the results occurred at the heel joint location. Here, not only were the 3-D model moments considerably larger than the others, but perhaps due to different geometry constraint conditions, the plate-edge moments at heel joints for models where the heel joint was represented as the intersection of chord-member centerlines (rigid, semi-rigid and pinned) were generally of the opposite sign compared to the 3-D model where the eccentricity of the members was accounted for. Unfortunately, a complete explanation is still unavailable, without additional full-truss test validation.

By observing the deflected shape of the 3-D truss model (fig. 5), one can notice a distinct curvature of the top and bottom chord members about half way from the heel joint along the panel length. This behavior, along with the observed stress distributions across the members' widths (not reported here), indicated that both chord members near the heel joint were subjected to tensile stresses at the top of the chords' cross sections, and compressive stresses at the bottom (therefore the moments were of the same sign). In moving away from the heel joint, the chords start to bend upwards, indicating that the members' top fibers were loaded in compression, and the bottom in tension. Therefore, in moving away from the joint, both the magnitude and sign of the moments along the top and bottom chords were significantly changing.

After considering the differences in moment predictions from the various models, especially at the heel joint, commonly considered to be the weak point of the truss, it is critical to emphasize the importance of the capability of analysis methods to adequately estimate force distributions in trusses. However, the only reliable method to validate force predictions from analytical models is to observe the internal truss stresses during physical testing.

#### SUMMARY OF RESULTS

The results from the different analysis methods (3-D FE model and several 2-D models) showed (as observed by Cramer et al., 1993, as well) that there was a significant influence from the joint modeling approach on the overall behavior of the modeled truss. Although the displacement predictions at the center-span of the truss from all the models in this study were within 10% of the actual truss test values, the truss internal force distributions predicted by different models were notably different. This was specifically true at the heel joint, where, for example, the

moment results obtained from the 3-D model were significantly larger (up to an order of magnitude) than the 2-D models' predictions. However, further model validation by full-scale truss experiments such as those performed by Lum et al. (1996) is needed.

Moreover, besides the additional validation of model results with experimental results, future research should also include the development of possible model simplifications. Specifically, a smaller overall number of elements distributed in a simpler fashion (which would perhaps lead to a somewhat compromised level of accuracy, but would still preserve the main concepts of the method), along with standard procedures to more efficiently characterize the semi-rigid nature of the connections (reliable approximation of per-tooth, or per-plate load-slip properties for various plate-to-grain orientations, depending on wood species, plate size and type, wood contact, etc.) are two possible techniques that would significantly simplify the method.

#### CONCLUSIONS

Based on the limited experimental data and the analytical investigations from this study and the earlier phases (Vatovec, 1996; Vatovec et al., 1996b), the 3-D model developed for MPC joint analysis may be successfully applied to full-scale truss analysis. But, without additional full-truss experimental testing to determine the accuracy of the model results, validation of the assumptions used in the modeling process, and model simplification investigations, the 3-D model should be limited at present to analyses of individual MPC joints.

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