

Three-Dimensional Nonlinear Finite-Element Analysis of Wood–Steel Bolted Joints Subjected to Large Deformations

A. R. Kaliyanda¹; D. R. Rammer, M.ASCE²; and R. E. Rowlands³

Abstract: This paper numerically models the behavior of double-shear, single-bolted joints in wood-steel structures when subjected to very large deformations and compares results with test information. A three-dimensional finite-element model is developed of the main Douglas-fir wood member, steel side plates, bolt, washers, and nut. The model accounts for friction, bolt clearance, progressive damage in the wood, nonlinear and inelastic behavior in the steel bolts and side plates, and complete (linear and nonlinear) compressive constitutive response parallel to the grain in the wood. Hashin's 3-D failure criteria are used to predict the onset and type of damage. Once failure is detected, and its mode identified at a particular location, material properties there are degraded to simulate the loss of load carrying capacity. The predicted load versus displacement results correlate with experiment. The present numerically determined displacements exceed by seven times those previously reported for bolted wood joints. DOI: [10.1061/\(ASCE\)ST.1943-541X.0002036](https://doi.org/10.1061/(ASCE)ST.1943-541X.0002036). © 2019 American Society of Civil Engineers.

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Introduction

General Comments

Bolted connections are extremely common in construction and machine components. Advantages of bolted joints include the relative ease with which they are assembled and can be dismantled readily for inspections or repairs. The design and analysis of bolted joints in wood have been important research topics and considerable relevant literature exists. However, understanding of the failure process is limited and clarification of the process would improve wood building design for extreme loading levels, such as earthquake or high wind loading. Bolted joints can be more complex to analyze in wood than in metals. Wood has lower compression yield capacity and limited tension strength perpendicular to grain in comparison to metals and its orthotropic properties result in complicated failure modes and limited ability to redistribute tensile stresses. Species variability, the influence of moisture, and grain variation add to the complexity of bolted wood joints.

The design of wood joints had been based on extensive empirical data, but in 1990 the National Design Specification (NDS) transitioned from an empirical methodology and adopted the yield theory for calculating design values for laterally loaded dowel-type connections (AWC 2001). At its essence, the yield theory computes the lateral connection strength assuming the wood under the dowel

fastener has yielded and the bolt has developed plastic hinges. Fastener spacing requirements are presumed to be adequate to develop the controlling yield mode. Results of multiple bolted wood connection tests raised questions about this assumption. The NDS code was revised in 2001 to provide specific wood failure modes. This gave rise to an increased impetus to understand the failure modes using numerical models that include the nonlinear and/or inelastic zone behavior of the wood. Among other considerations, a greater understanding of the nonlinear behavior of wood can be extremely beneficial when designing wood structures to withstand extreme loading events.

As shown in Figs. 1 and 2, bolted wood–steel joints are frequently described by bolt diameter, D , wood end distance, e , wood edge distance, a , main member thickness, L , steel side member thickness, t , and slenderness ratio, L/D . Although multiple-bolted joints are used, this paper focuses on a single-bolted, double-shear lap joint assembly having steel side members and subjected to an applied load, P . The entire connection is symmetrical about the horizontal plane through the middle of the wood member and about the longitudinal plane through the bolt. The load is transferred to the wood here primarily by bearing load rather than friction. While the bearing strength of a bolted joint often controls design strength, this aspect tends to be the least understood and most difficult to analyze at large deformations and at connection failure.

Motivation and Objectives

Notwithstanding its practical importance, relatively little prior research has focused on predicting load versus displacement behavior in bolted wood connections up to failure. Despite accounting for material nonlinearity, few studies have included material degradation or large deformations. An inherent complexity that occurs when accounting numerically for large deformations is the highly nonconvergent nature of the solution due to extensive distortion of the finite-element mesh. Numerical challenges associated with extremely large distortion are particularly evident in the 127-mm (5-in.) thick wood member analyzed here. As elaborated upon in the results that follow, after a certain load level, situations such as

¹Mechanical Design Engineer, Interior Cockpit Dept., Tesla Motors, 45500 Fremont Blvd., Fremont, CA 94538; formerly, Univ. of Wisconsin–Madison, Madison, WI 53706.

²Research General Engineer, USDA Forest Products Laboratory, Gifford Pinchot Dr., Madison, WI 53726.

³Professor Emeritus, Dept. of Mechanical Engineering, Univ. of Wisconsin–Madison, Madison, WI 53706 (corresponding author). Email: rowlands@engr.wisc.edu

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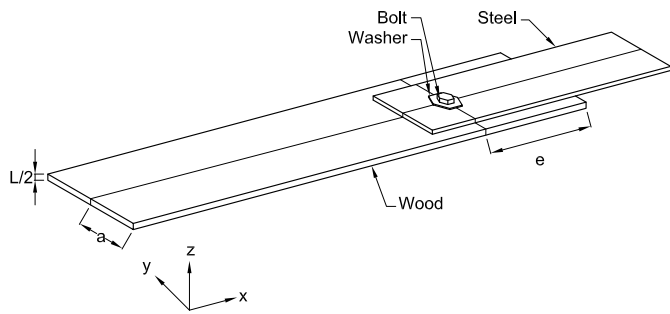


Fig. 1. View of top half of bolted joint assembly.

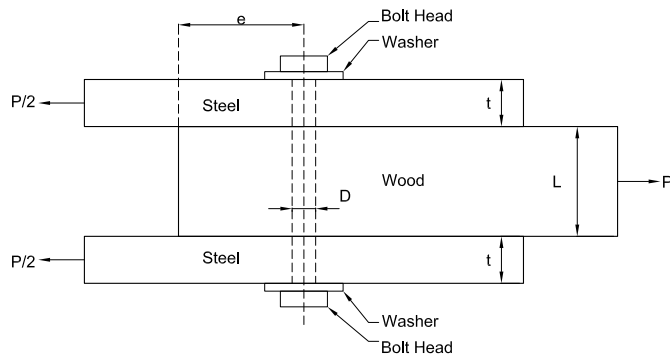


Fig. 2. Side view of double-shear single-bolted joint: D is bolt diameter, e is end distance, L is main member thickness, t is side member thickness, and P is applied tensile load.

extreme finite-element model (FEM) distortions and convergence challenges rendered it extremely difficult to obtain a solution. These 127-mm-thick wood members show extensive local damage where the bolt bears against the wood, especially at the edges of the wood member. A permanently yielded bolt is another indication of the extremely nonlinear response and highly developed compressive stresses that occur.

In view of the above, the primary objective of this research was to develop a numerical model to predict the load versus displacement behavior until failure in bolted wood–steel connections and compare it with experimentally determined information. The numerical model involved utilizing a reliable failure theory that mimics the physical behavior. Associated experimental tests were conducted on single bolted three-member connections with steel side plates and a glued-laminated main wood member. Slenderness ratios of 3 and 10 were considered. Measured information was obtained from a separate and broader test program that investigated various configurations of bolt diameter, main member (wood) thickness, and end distance. Tests were designed to experimentally evaluate the effect of aspect ratio and end distance spacing on the incident of wood failure mechanisms.

Background

Commensurate with the practical importance and challenges, a rich collection of literature exists on the topic of bolted connections, including those in wood. Only those publications pertaining directly to the present research are included here. Kaliyanda (2011) includes an extensive list of references on mechanical joints. Patton-Mallory et al. (1997) numerically modeled nonlinear load

versus displacements in wood joints up to 0.761 mm but which was less than the failure of the joint assembly. Patton-Mallory's model is not suitable for investigating the behavior of the joint assembly all the way to failure. For small deformations, the complexity of the model is minimal and a fairly accurate representation of the physical behavior is obtainable. However, when large deformations were considered, the numerical complexity increased significantly in that interactions occur among the various material parameters (Young's moduli, E , shear moduli G , and Poisson ratios) as the material deforms. Accounting for bolt friction and yielding, Moses and Prion (2003) correlated FEM-predicted and measured load versus displacements in single-bolted wood joints up to 1.8 mm. Since the analysis did not account for large deformations or material degradation, their prediction also did not continue to failure (which occurred at 7.62 mm). Kharouf et al. (2005) studied single-bolted and double-bolted timber connections. As with previous analyses, their FEM model did not account for large deformations or material degradation. While they measured displacements up to 1.75 mm, their model load versus displacement curves were only up to 0.25 mm. Employing Hashin's failure criterion (Hashin 1980), de Castpo Camanho (1999) developed progressive damage models to predict failure in mechanically fastened carbon-fiber reinforced composites. Using Hashin's theory, McCarthy et al. (2005) developed, and compared the results with experiment, for a progressive damage model of bolted composite laminates, including bolt-hole clearance and finger-tightened titanium bolts. Tserpes et al. (2001), Huhne et al. (2010), Olman and Santiuste (2012), Wang et al. (2012), and Veisi et al. (2014) also applied progressive damage models to predict the behavior of aerospace composites. However, the authors are unaware of previous applications of 3D FEM to bolted wood connections that include nonlinearity (material and geometry) coupled with a progressive damage model. Treating wood as a natural composite consisting of cellulose fibers embedded in a lignin matrix, the present approach shows good correlation at the larger deformations between the measured and FEM-predicted curves.

Experimental Research

General Comments

The bolted joints were tested physically in accordance with ASTM D5652-95 (ASTM 2006b). This involves plotting load versus the relative displacement in the direction of loading between the main wood member and the side steel plates. The resulting load versus displacement data provide information on parameters such as ultimate strength, stiffness, and yield load, and are useful for the development of design loads.

Test Specimens

Figs. 1 and 2 illustrate the single-bolt, three-member bolted connection, including the end loading arrangements, individual steel side plates, and main wood member considered in accordance with ASTM D5652-95 (ASTM 2006b). ASTM D5625-95 (ASTM 2006b) provides a basis for determining comparable performance of different types and sizes of fasteners. Wood thicknesses of $L = 38.1$ and 127 mm were tested and analyzed. The relatively large wood thickness (127 mm) was chosen to emphasize the 3D effects and be representative of joints occurring in primary structures.

Both of the 38.1- and 127-mm-thick main wood members were fabricated from 222-mm-wide by 190-mm-deep by 5.5-m-long No. 3 axial combination of glued-laminated Douglas-fir as

specified by AITC 177 (2004) source beams. These beams were cross-cut to the final specimen length and then ripped lengthwise to produce the required main member thicknesses.

The ASTM A572 (ASTM 2003) Grade 50 steel side plates shown in Figs. 1 and 2 were 304.8 mm long, 152.4 mm wide, and $t = 9.27$ mm thick. Their end distance was $e = 50.8$ mm and edge distance $a = 76.2$ mm. The 495.3-mm-long by 190.5-mm-wide main wood members had an end distance of $e = 7D = 88.9$ mm and edge distance $a = 95.25$ mm. These dimensions were identical for both thickness configurations. The washers (ASTM A572 Grade 50 steel) between the bolt head/nut and the steel plates had an inner diameter of 13.67 mm, outer diameter of 26.82 mm, and were 2.65 mm thick. The bolt [ASTM 307 (ASTM 2014) Grade B low/medium carbon steel, similar to SAE Grade 2 but without cold work] diameter was $D = 12.7$ mm and its head was 7.87 mm thick. The bolts were inserted into 1.58-mm oversized holes according to standard practice (NDS 2015). Specimen ends were configured to the maximum design joint load.

Testing

The connections were loaded monotonically in tension in an Instron 5589 universal testing machine, as shown in Fig. 3. The physically loaded specimen and the testing hardware are visible in Fig. 3(a), whereas Fig. 3(b) illustrates how the relative displacement between the wood and steel side plates was measured. The upper attachment in Fig. 3(a) consisted of 22.2-mm (0.875-in.) diameter SAE 4140 quenched and hardened steel pins; the total edge and end distances in the main wood member were 50.8 and 254 mm (2 and 11 in.), respectively. An initial dead load was applied experimentally to ensure that all the parts involved were initially in contact, and this was replicated numerically by bringing all the components involved in contact before starting the simulation. A constant displacement rate was employed to achieve maximum load within 5–20 min while trying to limit the first occurrence of any cracking within the first 5 min. Connection

displacement was measured by four linear variable differential transformers (LVDTs) attached between the wood and side steel plates [visible at the lower section of specimen in Fig. 3(a)]. All displacement measurements were referenced to the centerline of the hole machined in the steel side plates. During testing, a computer acquired the load and joint deformation readings at a rate of 10 Hz. A test was concluded when the load dropped by more than 25% of its maximum.

The 12.7-mm-diameter steel bolts (ASTM A307 Grade B) were all finger-tightened such that the main wood member and steel side plates were in contact at the beginning of the test. However, as the loading progressed, bolt bending occurred, which introduced normal forces between the wood and steel side plates. Friction between the steel and the central wood plates was accounted for numerically.

Auxiliary Tests

Three-point bending tests with bolts having the longest L/D ratio determined the bolt bending yield strength, while the traditional dowel-bearing strength was determined using matched samples from each connection test according to ASTM D5764-97 (ASTM 2006a) full-hole method. Fig. 4 shows the measured load-deformation response of three separate identical 12.7-mm-diameter ASTM A307 Grade B bolts subjected to three-point bending over the 127-mm (5-in.) span, while Fig. 5 shows the measured tensile stress-strain behavior of three of the 12.7-mm-diameter bolts up to yielding. The average modulus and yield point are 206 GPa and 515 MPa, respectively.

In order to determine the postpeak parallel to grain compression response, twenty 50.8- by 50.8- by 203-mm Douglas-fir specimens were tested. The specimens were necked down to a 25.4- by 25.4-mm cross-section at midheight to localize the compression failure away from the end conditions. An extensometer was employed to measure the deformation response in the necked down section of the specimen. Figs. 6 and 7 are photographs of such specimens being tested, including the extensometer.

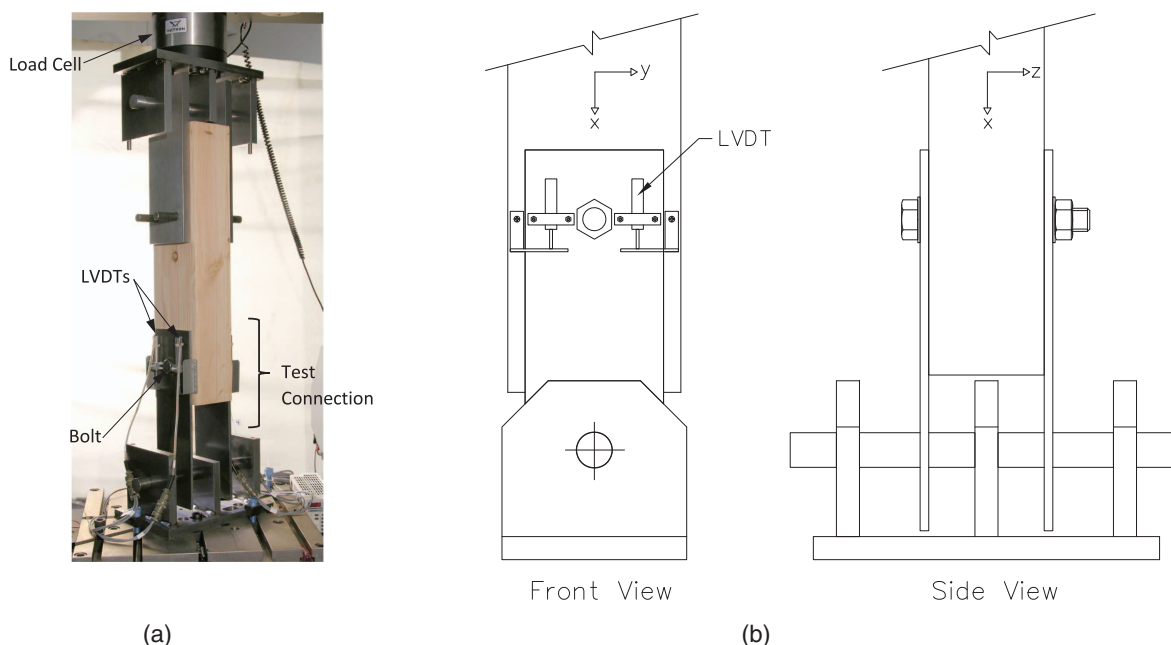


Fig. 3. (Color) (a) Tensile testing of bolted joint; and (b) details of measuring relative displacement between the wood and steel side plates.

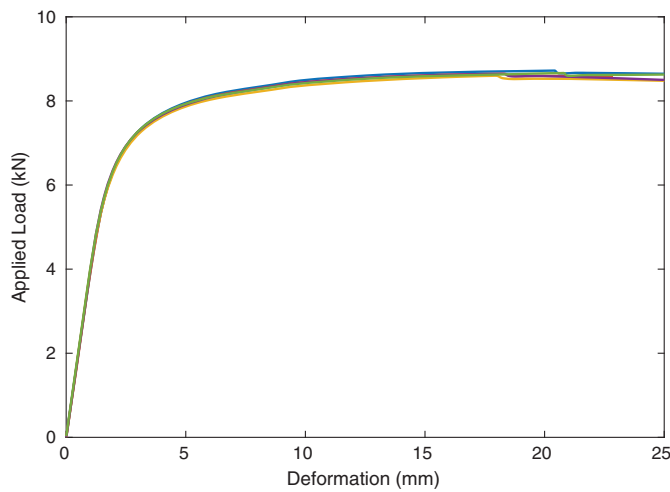


Fig. 4. (Color) Load-deformation response of three individual ASTM A307 Grade B bolts subjected to three-point bending.

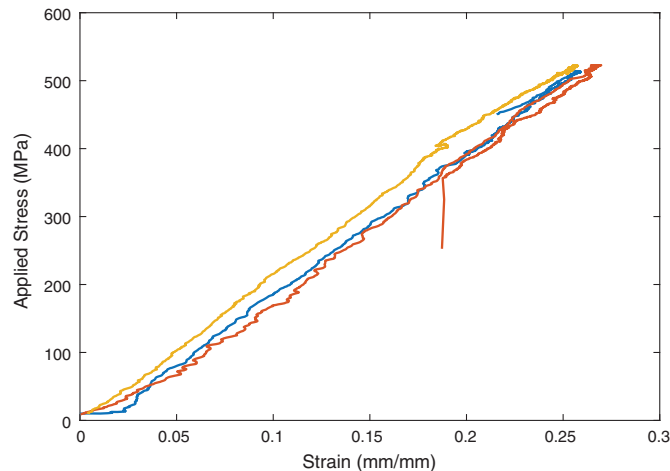


Fig. 5. (Color) Stress-strain response of three individual ASTM A307 Grade B bolts.

Numerical Analysis

Finite-Element Modeling

ABAQUS version 6.81/CAE (Dassault Systèmes Simulia, Johnston, Rhode Island) and Intel Visual FORTRAN version 10.01 were used for the finite-element analyses. ABAQUS/Standard was employed since the problem was modeled as a static problem. Disadvantages of this method are the significant computing times that occur due to the recalculation of the tangent stiffness matrix at each iteration using the Newton-Raphson method and the difficulty of defining this matrix in some cases. However, the Newton-Raphson method is the only procedure that can provide quadratic convergence. The fact that the ABAQUS/Standard utilizes an unconditionally stable stiffness-based solution technique enhances reliability. Eight-node brick elements with reduced integration formulation, C3D8R, were utilized everywhere except at the geometric transition in the bolt from the hexagonal head to the circular shank, where C3D6 elements i.e., six-node linear triangular prism elements were employed.



Fig. 6. (Color) Compression testing of Douglas-fir.

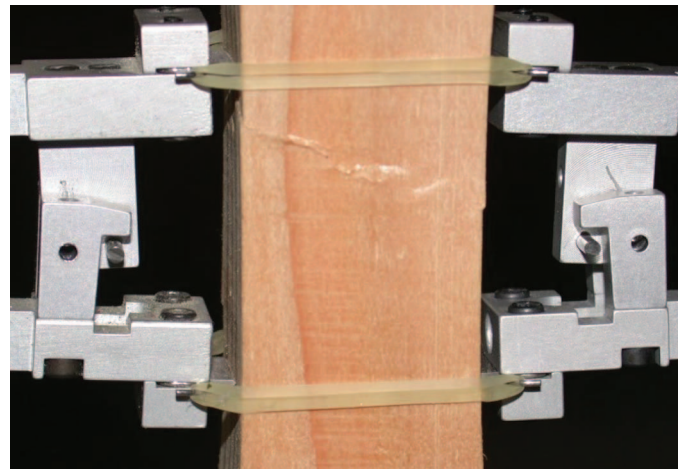


Fig. 7. (Color) Extensometers utilized to evaluate compressive strain in Douglas-fir.

The automatic mesh generator built into the ABAQUS version 6.81 graphical user interface was utilized. A very fine mesh was used near the hole in the steel side plates and the wood main member, while a slightly coarser mesh was employed in these members away from the hole. Element size was 2.25 mm close to the hole in the steel plates and wood, increasing to 4.0 mm away from the hole. This format prevails through the thickness of the steel and wood members. Bolt elements were 2.85 mm in size with 10 elements through the length of its shank, and the washers were modeled by two rows of 2.0-mm elements through their thickness. Both elastic and plastic strains accumulate as the metal deforms in the postyield region and these were accounted for when numerically modeling the material properties of the steel and the wood (see subsection “Constitutive Responses”).

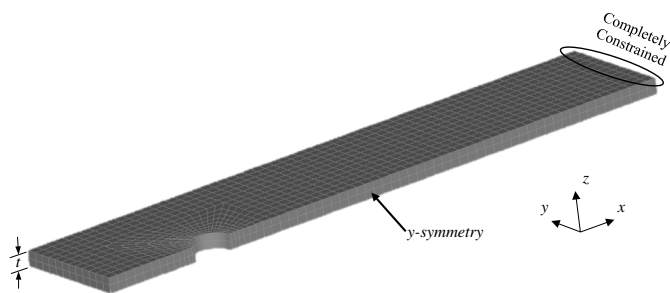


Fig. 8. Boundary conditions applied to the steel plate.

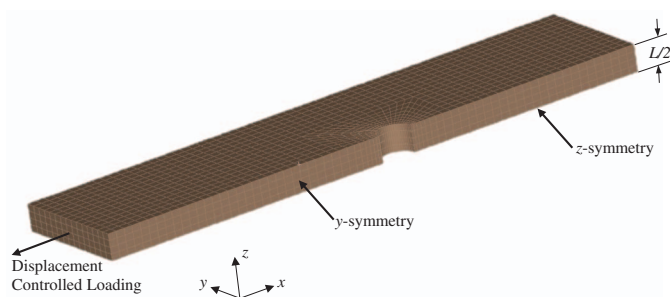


Fig. 9. (Color) Boundary conditions and displacement controlled loading applied to wood main member.

Geometry

Each wood main member of the connectors was evaluated at $L = 38.1$ mm and $L = 127$ mm. All ratios were in accordance with the standard (i.e., $L/D = 3$, $L/D = 10$, $a/D = 7.5$, and $e/D = 7$ for $D = 12.7$ mm) and were designed to promote a bearing-type failure. Detailed dimensions of the individual members are identified earlier.

Boundary Conditions and Loading

Due to symmetry, only one-fourth of the assembly was modeled and symmetrical boundary conditions were applied (Figs. 8 and 9). This same approach was adopted for the thin ($L = 38.1$ mm) or thick ($L = 127$ mm) wood member.

The load was introduced numerically by applying a prescribed displacement in the x -direction to the rightmost end of the wood main member to mimic a quasi-static displacement-controlled loading in the experiments (Fig. 9). The nodes on the bolt and washer, which are potential contact nodes with other components, were excluded from this boundary condition. Contact nodes were not given boundary conditions since ABAQUS applies tying boundary conditions automatically when contact is detected. Since the bolt-nut were only finger-tightened, no bolt prestress was applied. However, upon connector loading, the plate became loaded, and bolt bending introduced normal forces between the wood plate and the steel side plates. Friction then resulted between these plates and this was accounted for numerically. The complete FEM of the assembly is shown in Fig. 10. Kaliyanda (2011) contains additional FEM details.

Constitutive Responses

The nine elastic properties for straight-grained Douglas fir (interior north; 12% moisture content) of Table 1 represent the initial linear

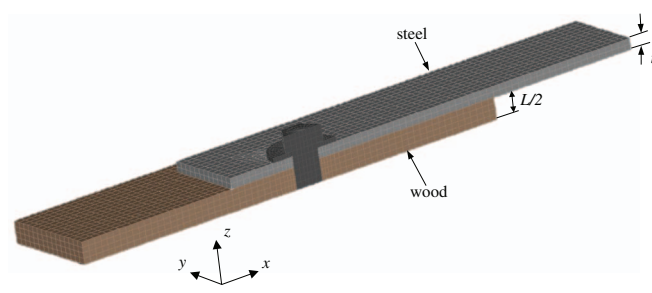


Fig. 10. (Color) Finite-element quarter-model of the physical joint.

Table 1. Linear elastic Douglas-fir properties

Elastic property	Value
E_{xx} (MPa)	10,342
E_{yy} (MPa)	655
E_{zz} (MPa)	890.8
NU_{xy}	0.449
NU_{xz}	0.292
NU_{yz}	0.374
G_{xy} (MPa)	1,021.8
G_{xz} (MPa)	838.402
G_{yz} (MPa)	91.702

elastic orthotropic material behavior [Wood Handbook (US Department of Agriculture Forest Service, Forest Products Laboratory 2010)]. The respective Young's moduli (E_{xx} , E_{yy} , E_{zz}), shear moduli (G_{xy} , G_{xz} , G_{yz}), and Poisson's ratios (NU_{xy} , NU_{yz} , NU_{yz}) are with respect to the x - y - z axes shown in Figs. 1, 3(b), and 8–10. Among the various material data (i.e., compression or tension parallel or perpendicular to the grain), compression parallel to the grain plays a significant role when the bolt bears against the wood during tensile connector loading. The measured compression response parallel to the wood grain of Fig. 11 was used to mimic the yielding/flow behavior (under the bolt contact) during tensile loading of the bolted connector. Of the 20 different test curves shown in Fig. 11, the designated *selected curve* was utilized as being representative. Several individual points were selected along this selected curve and a polynomial function was used to smooth the curve. The resulting response was chosen to represent the nonlinear flow compressive behavior of the wood. The individual plots of Fig. 11 were obtained from the compressive testing illustrated in Figs. 6 and 7. The initial elastic portion of the selected curve of Fig. 11 is a linear regression of the data between 20% and 40% of the maximum load, while the degraded slope is a linear regression of the data between 90% and 100% of the maximum strain. The magnitude of the transition point between the initial elastic and subsequent degraded compression is the average compressive strength of the Douglas-fir parallel to the grain.

The data, including the selected curve, of Fig. 11 are given in terms of engineering stress–engineering strain. However, for plasticity/nonlinear constitutive responses, and because everything before the yield point can be defined by the elastic modulus, ABACUS uses the portion of the stress–strain curve beyond the yield point. A fourth-order polynomial was consequently fitted to the engineering stress–strain coordinates of the selected curve in Fig. 11. Since ABACUS needs the material response in terms of true stress–true strain, the fourth-order polynomial information was converted to true-stress/true-strain data and the resulting true-stress/true-strain plot of Fig. 12 was represented by a quadratic

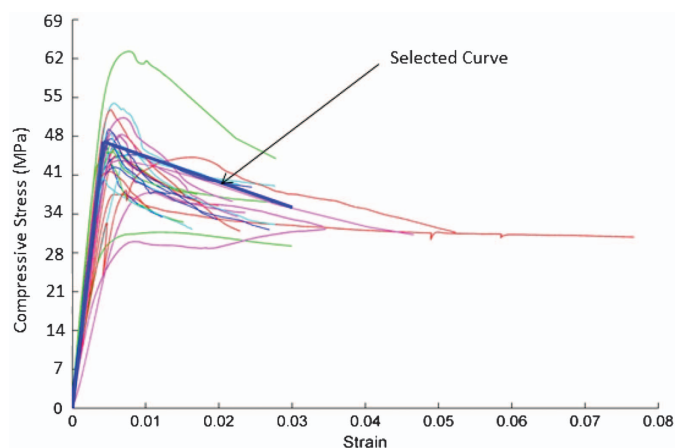


Fig. 11. (Color) Measured compression response parallel to the grain in Douglas-fir.

polynomial. If one does not use at least a quadratic or cubic equation to represent the postyield behavior, the ABACUS code can become very unstable due to convergence issues. Not unlike the just-described approach of handling the constitutive response of the wood, the engineering stress-strain behavior of the ASTM A572 Grade 50 steel used in the side plates was determined experimentally. Those data were subsequently converted to the true-stress/true-strain plot of Fig. 13, and which was represented by a quadratic polynomial for the ABAQUS software. The reader is directed to Kaliyanda (2011) for details on modeling the inelastic responses.

Progressive Damage Analysis

Progressive damage analysis was employed to predict the nonlinear failure response of the bolted connection. Little prior research appears to have been done on predicting the progressive failure of single-lap joints to failure. While progressive damage modeling is used in the aerospace industry to model degradation in composites, the authors are unaware of previous applications of 3D FEM to bolted wood connections that include nonlinearity (material and geometry) coupled with a progressive damage model. Motivated by previous contributions by de Castpo Camanho (1999) and McCarthy et al. (2005), who used this approach to model bolted joints in composites, this study employed a similar concept to model bolted joints in wood-metal structures, including damage in the wood. The scheme involved evaluating failure criteria at each Gauss or integration point in the model and if failure was detected, the appropriate material properties were reduced to some fraction of their original values. The connector load was then redistributed among the other Gauss points and the process was repeated. In this way, the damage can propagate throughout the component until final failure occurs. Hashin's (1980) 3D failure criterion was used to predict the onset and type of wood damage.

Based on the model by de Castpo Camanho (1999) and the contributions of Tan (1991), Tan and Perez (1993), Nuismer and Tan (1988), and Tan and Nuismer (1989), it was assumed that the effects of the different damage mechanisms on the elastic properties can be represented by a set of internal state variables that are functions of damage. It was further assumed that the presence of damage within an element has an effect on the constitutive properties of only that element. Thus, and as suggested by and Highsmith and Reifsnider (1982) and Reifsnider (1991), the stiffness reduction

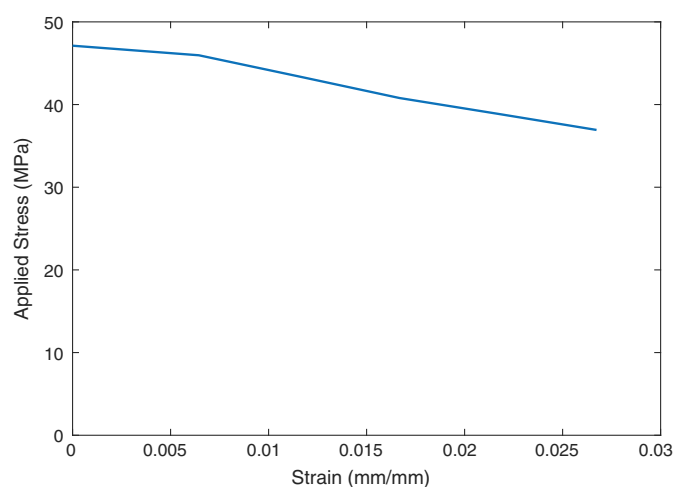


Fig. 12. (Color) Compressive true stress versus true strain response in Douglas-fir parallel to its grain.

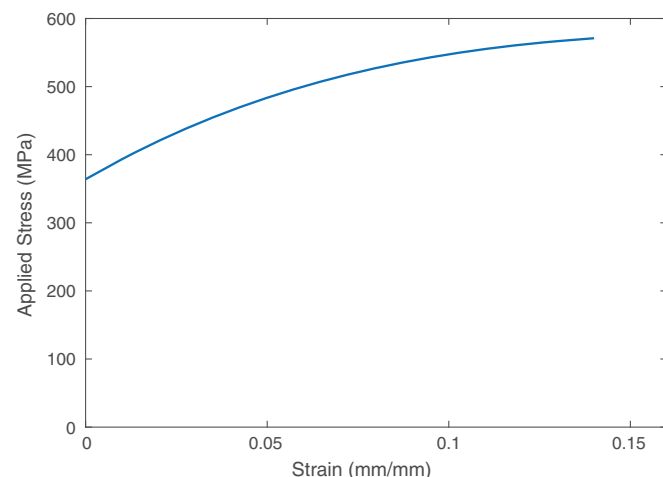


Fig. 13. (Color) True-stress versus true-strain response for steel.

associated with failure at a point was confined to that point and its neighborhood.

Property Degradation

While the steel was modeled as an elastic-plastic material (see subsection "Constitutive Responses"), the wood was assumed to behave as a composite material (i.e., unidirectional fibers in a matrix). Once a failure was detected at a particular material point (Gauss point) and the failure mode identified, the material properties were numerically degraded at that location to simulate the local loss of load carrying capacity. A property reduction scheme proposed by McCarthy et al. (2005) was employed to progressively degrade the wood stiffness in each orthotropic direction once the failure has been predicted (Table 2). This approach idealizes the material surrounding a given Gauss point as a single fiber embedded in a cube of matrix material. Depending on the failure mode, a flaw in the form of a matrix crack or a fiber break was introduced. The cube was then subjected to imaginary direct and shear loads and the response determined from the mechanical behavior of both the matrix and fiber yields how the material

properties change. A similar approach was taken by Gamble et al. (1995) to study damage initiation and growth in carbon-fiber composite materials. An X in Table 2 indicates that the material property is to be degraded. A dash (–) indicates that the material property is unaffected by that mode of failure. If failures were detected in more than one mode at a material point, which may or may not happen simultaneously, the material point was assumed to no longer be capable of sustaining load in any direction and all material properties were reduced at that location.

Decreasing a Young's or shear modulus to zero can cause numerical instabilities mainly due to the singularity of the material Jacobian that could not be inverted. To circumvent this situation, a modulus was reduced to approximately 10% of its initial value. However, just imposing the degradation process alone failed to mimic the observed experimental behavior. Only when the progressive damage model was combined with the compression parallel-to-grain stress-strain curve of Fig. 12 for the wood did the finite-element-predicted load-displacement curve closely follow the experimental curve. The compression parallel-to-the-grain response was a vital factor in mimicking the flow or yielding behavior exhibited by the physical bolted joint assemblies.

Damage Implementation in Wood

Whereas ABAQUS' built-in elastic-plastic model was used for the steel, the subroutine USDFLD (user-defined field variables, developed by and incorporated into ABAQUS) was used to implement the progressive failure criteria in the wood. Stresses were called into USDFLD by the GETVRM utility routine and used to evaluate the failure criteria. Upon predicting failure, the material properties were changed according to the degradation law imposed.

When material properties were degraded at a point, the load redistributes among other points, which could then cause failure there. It was, therefore, necessary to reiterate at the same load level when material properties changed to determine if other material points undergo failure. Equilibrium need not be re-established after the material properties change and one can omit re-establishing equilibrium by using very small load increments (Sleight 1999; McCarthy et al. 2005). The present analysis employed very small load steps, i.e., 0.01 in. = 0.25 mm (displacement controlled loading). This considerably increased the computer run time.

Contact

ABAQUS/Standard offers two contact discretization options: a traditional node-to-surface discretization and a true surface-to-surface discretization. While the increased number of nodes per constraint involved with a surface-to-surface discretization can increase run time and solution cost, this technique was used and it was found to significantly minimize contact penetrations. The extra cost associated with this formulation was justified due to

the severe contact interactions involved between the wood main member and the side steel plates for the thicker (i.e., $L/D = 10$) wood member.

Friction

Isotropic Coulomb friction was assumed [$\mu = 0.7$ for the 127-mm-thick (5.0-in.-thick) wood main member and $\mu = 0.2$ for the 38.1-mm-thick (1.5-in. thick) wood main member] between the contacting steel and wood surfaces. These values pertain to between the flat contacting steel and wood surfaces as well as to the contacting surfaces between the steel bolt and edge of the wood. On the other hand, $\mu = 0.2$ was used between steel and steel with both wood thicknesses, i.e., between the steel plates and the bolt head and nut, and the end portions of the steel bolt that contact the external steel plates. The higher steel-wood friction coefficient of $\mu = 0.7$ was chosen with the 127-mm-thick (5.0 in.) wood main member as the load transfer here is predominantly due to bearing of the steel bolt against the wood main member. It was observed that friction coefficients of $0.6 \leq \mu \leq 0.8$ between contacting wood and steel with the 127-mm-thick wood main member did not change the load-deformation curves significantly. Similarly, friction coefficients of $0.1 \leq \mu \leq 0.3$ between the wood and steel contacting surfaces for the $L = 38.1$ mm (1.5 in.) connector had little influence on the load-deformation curves. The magnitudes of the above utilized coefficients of friction were based on information from Juvinall and Marshek (2011), *Timber Bridges—Design, Construction, Inspection and Maintenance* (US Department of Agriculture Forest Service 1992), *The Physics Handbook* (Benenson et al. 2002), and *The Engineering Handbook* (Dorf 2004).

Computer Details and Run Times

A DELL XPS 420 Desktop computer with Intel Quad Core CPU T6000 @ 2.6 GHz and 16 GB RAM was used to perform the numerical simulations. Modeling the $L/D = 3$ connector with 34,484 elements and 56,656 nodes consumed 11.8 h of run time to numerically predict its load versus deformation plot. Using the same computer but modeling the $L/D = 10$ connector with 39,352 elements and 69,452 nodes took 26.6 h.

Results and Discussion

Figs. 14 and 15 compare the numerically predicted and recorded load-displacement behavior of the three tested connectors for each of the two respective wood thicknesses. The measured displacements were obtained in each case from the average readings of the four LVDTs visible in Fig. 3. All experiments were conducted using a consistent species of wood (Douglas-fir), ASTM A307 Grade B steel bolts, and ASTM A572 Grade 50 steel side plates. One can therefore reasonably assume that the variability in the experimental load versus displacement response of the joint specimens having the same wood thickness (i.e., identical specimens) seen in these figures is due to inherent wood property variability, grain orientation and alignment, and small specimen or test geometry variations.

Following the numerical modeling strategy, the finite-element simulations were formulated using ABAQUS/Standard. The end spacing for both the configurations was 7 D (= 88.9 mm). The only difference between the two different cases of Figs. 14 and 15 is the thickness of the middle wood member. There is good agreement at higher deformation between the measured and finite-element-predicted curves for the progressive damage model mechanism adopted. The loss of physical stiffness due to crack initiation

Table 2. Degradation implemented scheme for wood

Failure mode	E_{xx}	E_{yy}	E_{zz}	G_{xy}	G_{yz}	G_{xz}	NU_{xy}	NU_{yz}	NU_{xz}
Tensile matrix	–	X	X	–	X	–	–	X	–
Tensile fiber	X	–	–	X	–	X	X	–	X
Compressive matrix	–	X	X	–	X	–	–	X	–
Compressive fiber	X	–	–	X	X	X	X	–	X
More than one failure mode	X	X	X	X	X	X	X	X	X

Source: Data from McCarthy et al. (2005).

Note: – = material property unaffected by failure mode; and X = material property affected by failure mode.

and subsequent crack growth was not modeled numerically and this could be a reason why the numerically predicted curves are stiffer than the measured ones. As the loading progresses, other complex mechanisms occurring within the wood could also soften it experimentally. All other factors were accounted for in the numerical model. A numerical analysis in wood connectors had previously been conducted for a displacement of up to 1.8 mm (Moses and Prion 2003). The present ability to numerically model the displacement in wood joints up to 15.24 mm, and to correlate with physical observations, particularly at the higher deformations (Figs. 14 and 15) is believed to be a significant advancement.

Since the experimental curves for different end distances, e , but the same main member thickness, L , followed the same general trend, it was not deemed worth the effort to numerically analyze cases of different end distance (Kaliyanda 2011). The only thing that changed for different end distances is the point at which failure occurred. Bolted wood connections having larger end distances can withstand an increased load due to a greater volume of wood now supporting the bearing load and which helps to prevent a shear-out plug-type failure.

The latter portion of the curves in Fig. 15 exhibits a series of kinks or disturbances. These are primarily due to the slippage between the steel side members and the main wood member of the bolted joint assembly for thick middle members (i.e., 127 mm). Since numerical simulations assume ideal contact conditions between the side and main members, this slipping phenomenon was

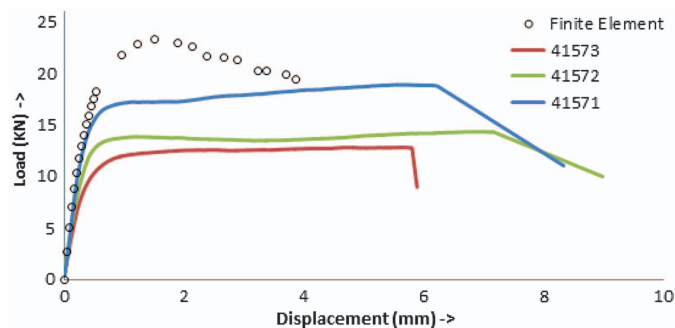


Fig. 14. (Color) Comparison of FEM-predicted and recorded load-displacement response of three individual but identical $L = 38.1$ -mm-thick main wood joints ($D = 12.7$ mm, $L/D = 3$, $e = 7D = 88.9$ mm, and $a = 95.25$ mm).

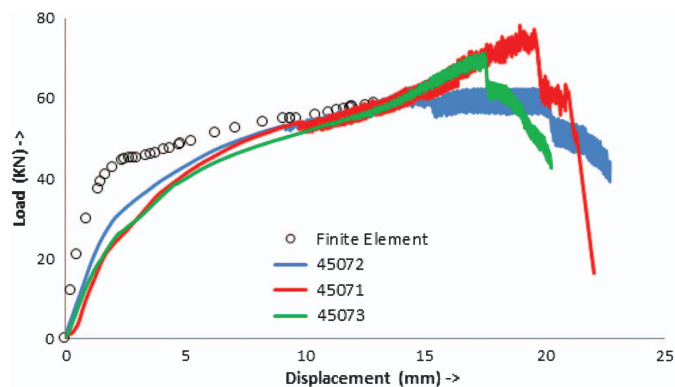


Fig. 15. (Color) Comparison of FEM-predicted and recorded load-displacement response of three individual but identical $L = 127$ -mm-thick main wood joints ($D = 12.7$ mm, $L/D = 10$, $e = 7D = 88.9$ mm, and $a = 95.25$ mm).

not accounted for or noticed in the finite-element results. However, severe distortions were noticed in the hexahedral elements used to model the wood. The numerical model tends to become highly non-convergent. Moreover, after about 75% of total load application, the numerical model for the thick middle wood connection virtually stalls with ABAQUS making too many cutbacks in the applied load increment.

Compatible with the responses illustrated in Fig. 15, Fig. 16 is an FEM image of a bent bolt and its head and washer in the 127-mm-thick middle wood member when loaded in the fixture of Fig. 3. Fig. 17, which is a cut-away photograph of the tested joint No. 45071 of Fig. 15 (127-mm-thick main wood member), illustrates the physical damage in the bolt-hole region of the middle wood member due to such a highly deformed bolt. Fig. 18 is an FEM image of the deformations of a bolt associated with the wood damage seen in Fig. 17, while the photograph of Fig. 19 shows the plastically deformed bolt from such as test. The photograph in Fig. 20 illustrates the mode of the wood failure in one of the $L/D = 3$ connectors.

The individual load-displacement plots of the $L/D = 3$ connectors in Fig. 14 demonstrate considerably greater variability comparative with those for $L/D = 10$ in Fig. 15. Unlike with the essentially 2D behavior of the $L/D = 3$ joints, the $L/D = 10$ connectors exhibited appreciable 3D, bolt bending response (Figs. 16–19). This important bolt-bending role, and the uniform bending behavior of the commercial bolts seen in Fig. 4, might at least partially contribute to the fairly consistent load-displacement response of the $L/D = 10$ connectors up to a displacement of approximately 14 mm. On the other hand, the comparatively small, and perhaps somewhat variable, tensile strength of the wood transverse to its grain from connector to connector, could have contributed to the scatter in Fig. 14. The relatively low joint strengths and ultimate displacements of the $L/D = 3$ connectors of Fig. 14 compared with those for the $L/D = 10$ connectors of Fig. 15, is probably highly influenced by the role played by the low tensile strength transverse to the grain in the wood. The photograph of the failed specimen #41571 of Fig. 20, which displays relatively little compressed wood damage but transverse tensile failure (splitting) both below and above the loaded hole, had the largest ultimate stress of the three $L/D = 3$ specimens. Connector #41573 of Fig. 14 had a failure mode similar to that of #41571, but the smallest ultimate stress and displacement of the three $L/D = 3$ connectors. Interestingly, whereas the strength of connector #41572 was between those of the connectors #41571 and #41573, it had the largest maximum displacement and its vertical splitting occurred only below the bolt hole. These observations are probably associated with some mechanical variability of the wood.

Although ABAQUS' surface-to-surface discretization was utilized, Fig. 16 exhibits some penetration of the side steel plate into the bolt. This might have been reduced (or eliminated) by using more surface master nodes.

Summary and Conclusions

A 3D finite-element model was developed for double-shear, single-bolted wood-steel connectors subjected to very large deformations. The numerically predicted load-displacement response agrees particularly well with test information at the higher deformations.

Connector aspect ratios of $L/D = 3$ and $L/D = 10$ were analyzed. The numerical model accounts for a main middle Douglas-fir glued-laminated wood member; steel side plates; bolt, washers and nut; friction; bolt clearance; progressive damage in the wood;

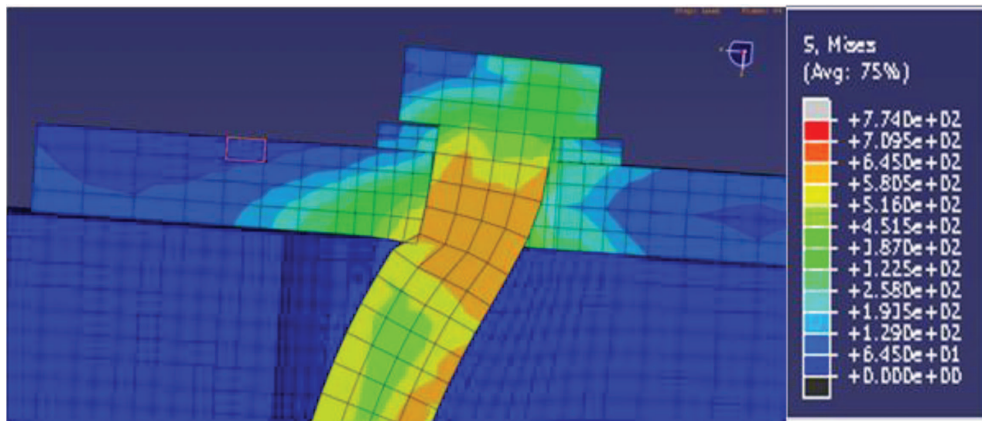


Fig. 16. (Color) FEM image showing the bolt bending for a 127-mm-thick wood ($L/D = 10$ connector) main member.



Fig. 17. (Color) Damaged bolt hole in 127-mm-thick wood ($L/D = 10$ connector) due to bolt bending.



Fig. 19. (Color) Significant bolt bending as observed in the $L/D = 10$ tests.



Fig. 20. (Color) Failed middle wood member of $L = 38.1$ mm ($L/D = 3$) connector.

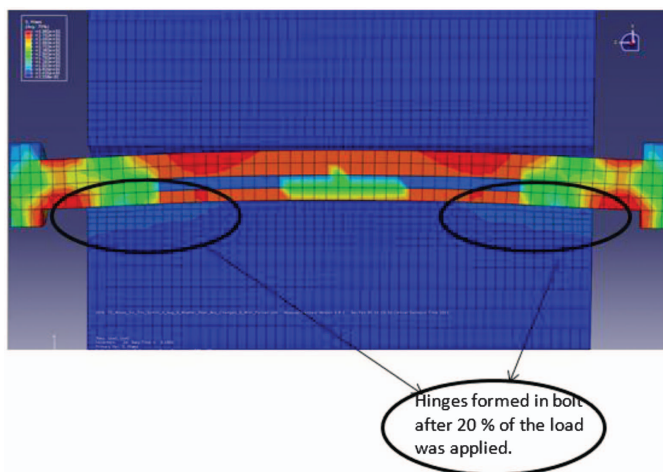


Fig. 18. (Color) Numerically predicted behavior in a 127-mm-thick wood connection due to bolt bending.

nonlinear and inelastic behavior in the steel; and complete (linear and nonlinear) compressive constitutive response parallel to the grain in the wood. Once failure was detected and its mode identified at a particular location, the material properties there were degraded to simulate the local loss of load carrying capacity.

The recorded displacement between the main wood member and steel side plates increased nonlinearly with applied load until failure. At significantly high deformations, a saw-tooth-type load versus displacement response can occur, and in some cases it

became difficult to obtain a numerical solution. The connections having the thicker wood member have significantly greater maximum loads/ductility and displacements to failure than do those with the thinner wood member. Unlike the later, the former connectors experience significant bolt bending and 3D responses associated with their ductile failure/plasticity of the wood and plastic hinges in the bolts. The specimens having the thinner wood members are more susceptible to brittle failure due to longitudinal splitting or cracking of the wood. The increased tendency for the plane-stressed connectors ($L/D = 3$) to experience wood splitting might contribute to their greater scatter than when there are appreciable bolt bending and 3D stresses ($L/D = 10$).

The authors are unaware of any previous publication of 3D FEM to bolted wood connections that couples nonlinear material and geometric behavior with a progressive damage model to predict the recorded load-displacement response to failure. The FEM-predicted bending and formation of plastic hinges in the steel bolt are also consistent with physical observations. The predicted load-displacement curves are generally stiffer than observed experimentally, something which Patton-Mallory et al. (1997) also observed.

The present numerically determined displacements exceed seven times those previously reported for bolted wood joints. It is hoped that the present information will benefit future engineering designs.

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