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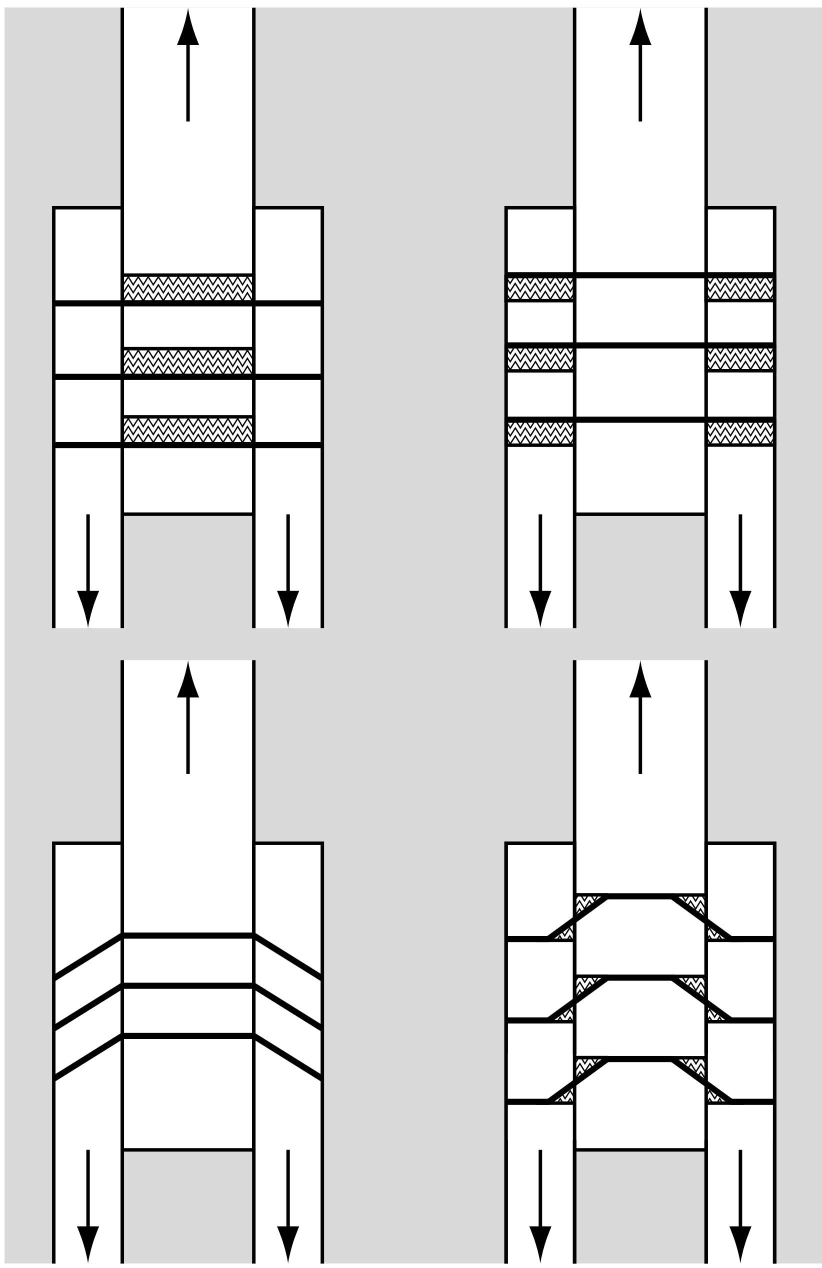
Multiple-Bolted Joints in Wood Members

A Literature Review

Peter James Moss



Abstract

This study reviewed the literature on experimental and analytical research for the connection of wood members using multiple laterally loaded bolts. From this, the influence of geometric factors were ascertained, such as staggered and aligned fasteners, optimum fastener configurations, row factors and length-to-diameter bolt ratios, spacing, end and edge distances, and the effect of mixed types and sizes of fasteners and eccentric loading. Areas of additional research needed on multiple-bolted joints in wood members are identified.

Acknowledgments

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Multiple-Bolted Joints in Wood Members

A Literature Review

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Introduction

To determine safe, allowable loads for specification in design codes and standards, a considerable amount of research has been conducted on single-fastener joints in timber members. This research has primarily covered nails, bolts, and shear plates. In practice, few single-fastener joints are used. Multiple fasteners are required to provide a joint whose strength is matched to the strength of the members being joined and the forces carried by them.

It has been known for some time that the end fasteners in a row of fasteners resist more load than the inner fasteners. The actual load distribution is affected by the relative stiffness of the timber members being joined and the bolts used, the fabrication, and variations in individual load–slip curves of each fastener. This unequal load distribution means that the load-carrying capacity of a row of fasteners is less than that calculated by multiplying the load capacity of a single fastener by the number of fasteners. As a result, a row modification factor has been introduced in several timber building codes to determine the strength of a row of fasteners. The use of a row modification factor has been primarily applied to joints with bolts and shear plate connectors, but in some instances, the factor has also been applied to nailed joints.

Until recently, design recommendations for bolted timber joints were based on research by Trayer (1932). The recent revision of the *National Design Specification for Wood Construction* (NFPA 1991) utilized the European Yield Method (Johansen 1949), which also forms the basis for timber joint design in Europe. This shift in the design approach, from expected behavior at the proportional limit to behavior at the yield, calls for a re-evaluation of some code recommendations. Examples include the geometry of a multiple-fastener joint in terms of (a) the end and edge distances and fastener spacing, both parallel and perpendicular to the grain, (b) whether staggered fasteners can give a greater and more reliable ultimate load, and (c) whether there is a way to optimize the performance of a joint. With an emphasis on the reliability of structures, it would be useful to predict the load–slip relationship for a joint to allow the displacements to be determined at the ultimate and serviceability loads.

In looking at the reliability of structures and joints, we need to look not only at the strength of the fasteners but also at the strength of the timber being joined and the interaction between fasteners and timber. This approach was followed in recent experimental and analytical joint research for glulam rivets (mainly used in Canada) and composite fiber-reinforced plastics.

This report reviews past analytical and experimental research on multiple-bolted joints in timber to determine our knowledge base about their performance and behavior under load. From this, future research areas are suggested.

Experimental Research

Trayer (1932) presented design formulas for bolted joints using sideplates to connect axially loaded softwood main members. His formulas were based on compressive test data. He noted that the interaction of the bending of the bolt and the crushing of the wood affected the performance of a joint. Trayer's load–deformation curves demonstrated a nonlinear, monotonically increasing rate of deformation beyond the proportional limit.

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Trayer's research was the basis for the allowable bolt strength values used in design prior to 1991. His primary research was conducted on steel side-plated joints with 6.5- to 25-mm- (0.25- to 1-in.-) diameter bolts for parallel-to-grain loads and 12.5-mm- (0.5-in.-) diameter bolts for perpendicular-to-grain loads. He developed an empirical curve fit to data for three softwood species parallel to grain and five species (three softwoods, two hardwoods) perpendicular to grain.

Trayer then extended the work to include wood sideplates. He limited this research to 12.5-mm- (0.5-in.-) diameter bolts. He further limited the parallel-to-grain work to two softwoods: Sitka spruce and southern yellow pine (Fig. 1). Perpendicular-to-grain results (Fig. 2) were for five species with steel sideplates; Trayer stated that steel or wood sideplates give equal results for perpendicular-to-grain loading. These tests were carried out using bolts having a stress yield of 310 MPa (45,000 lb/in^2). Trayer also looked at bolts with 860 MPa (125,000 lb/in^2) stress yield and found that his empirical curve shifted to the right.

Doyle and Scholten (1963) compressively tested three-member bolted joints with one and four bolts (the latter arranged in a pattern of two by two bolts). The joints were made with 19.0-mm (0.75-in.) bolts in lumber at three stages of seasoning, and with 12.5-, 19-, and 25-mm (0.5-, 0.75-, and 1-in.) bolts in standard 69-mm- (nominal 3-in.-) thick Douglas Fir dry lumber. Sideplates were either wood or steel; bolt holes were either the same size as the bolt or 1.5 mm (0.0625 in.) oversize. The joints were tested with the load applied either parallel or perpendicular to the grain of the central member. They concluded that the bolt-bearing strength per bolt parallel to the grain was about 90% that of a single-bolt joint for joints with four bolts bearing.

The following are specific conclusions from Doyle and Scholten (Fig. 3):

Bearing parallel to the grain:

- The bolt-bearing proportional limit stress of joints made from dry material with steel sideplates was about 150% that of joints with wood sideplates, but the maximum load of the joints with steel sideplates was only about 110% that of joints with wood sideplates.
- The bolt-bearing stress of green lumber joints was about 60% that of dry lumber joints.
- The bolt-bearing stress of joints constructed of green lumber and loaded after seasoning was about the same as that of joints constructed of dry lumber when all bolt holes were oversized. However, the loads at specific slips below the proportional limit were considerably less.

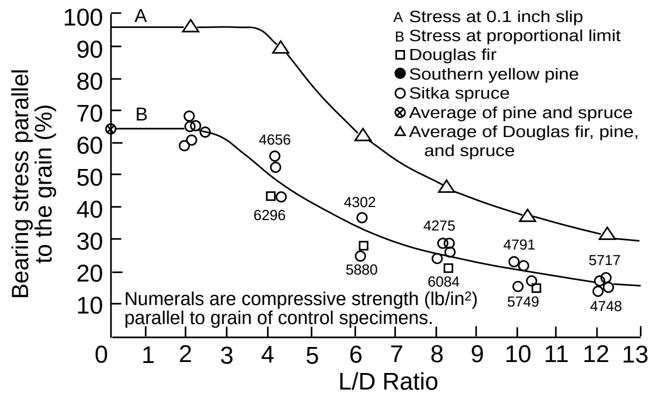

Figure 1—Relationship between the average bolt-bearing stress parallel to grain and the ratio of the length of bearing (*L*) in the main member to the bolt diameter (*D*) (Trayer 1932).

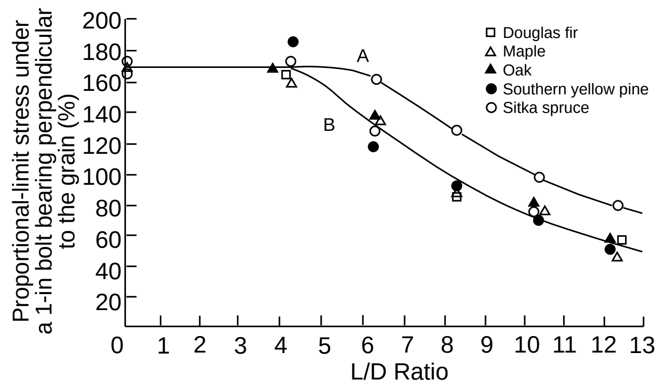

Figure 2—Relationship between proportional-limit bolt-bearing stress perpendicular to grain and the *L/D* ratio of the bolt: (A) species of low strength; (B) species of high strength (Trayer 1932).

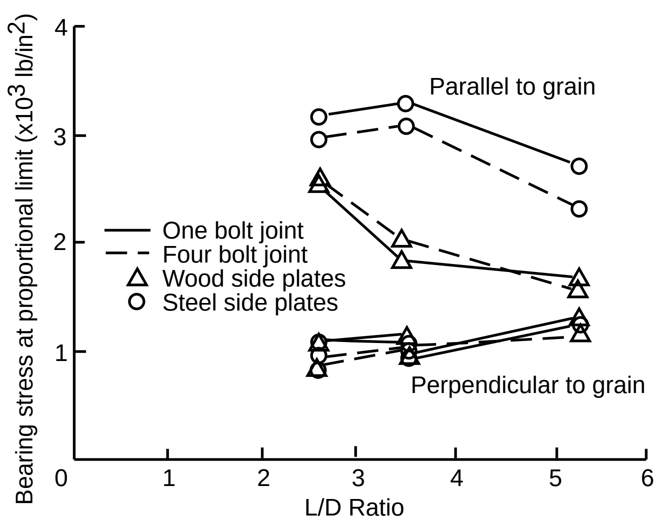

Figure 3—Relationship of the bolt-bearing stress at the proportional limit in air-dry Douglas-fir and the ratio of bearing length to bolt diameter (*L/D*) (Doyle and Scholten 1963).

- The bolt-bearing stress of joints with 12.5-mm (0.5-in.) bolts in standard 69-mm- (nominal 3-in.-) thick lumber was about 80%, and with 25-mm (1-in.) bolts, about 115% that of joints with 19-mm (0.75-in.) bolts. That is, for bolts with *L/D* (thickness of the main member to the bolt diameter) ratios of 2.7, 3.6, and 5.4, the bearing strength showed a ratio of 1.15:1.00:0.80.
- The bolt-bearing stress of joints with bolt-size holes constructed of green lumber and loaded after seasoning was about 120% that of similar joints with 1.5-mm (0.0625 in.) oversize holes when wood sideplates were used, but only 60% that of joints with oversize holes when steel sideplates were used.

Bearing perpendicular to the grain:

- The bolt-bearing stress of joints of dry material with steel sideplates was about the same as that of joints with wood sideplates.
- The bolt-bearing strength per bolt of joints with four bolts was about the same as that of single-bolt joints.
- The bolt-bearing stress of joints constructed of green lumber was about 70% that of joints of dry lumber.
- The bolt-bearing stress of joints constructed of green lumber and loaded after seasoning was about 40% that of joints constructed of dry lumber.
- The bolt-bearing strength of joints with 12.5-mm (0.5-in.) bolts in standard 69-mm- (nominal 3-in.-) lumber was about 130%, and with 25-mm (1-in.) bolts, it was about 95% that of joints with 19-mm (0.75-in.) bolts. That is, for bolts with *L/D* ratios of 2.7, 3.6, and 5.4, the ratios of strength were 1.30:1.00:0.95.
- The bolt-bearing strength of joints with bolts in bolt-size holes was slightly greater than joints with bolts in 1.5-mm (0.0625-in.) oversized holes.

Doyle (1964) tested a series of joints fabricated with eight bolts. The joints were three-member assemblies consisting of a three 6- by 190-mm (0.25 by 7.5 in.) laminated Douglas Fir member, two steel sideplates with two rows of four bolts acting in double shear parallel to the grain of the wood, and 12.5- and 19-mm (0.5- and 0.75-in.) bolts. Single-bolted joints were tested for comparison. Bolt spacings were either 75 or 114 mm (3 or 4.5 in.).

One result from Doyle's research was that the ultimate stress per bolt in the eight-bolt connected joint was between 60% and 80% that of the single-bolt joints using 19-mm (0.75-in.) bolts. This lower ultimate bearing stress per bolt was attributed to factors such as tension parallel and perpendicular to the grain, splitting, shear along the grain, and non-uniform bearing of the bolts. The test results showed that the load per bolt when plotted against the joint slip for the multiple-connected joint was not nearly proportional to the strength of a single-bolt connection.

Specifically, Doyle concluded the following:

- The bearing stress at the proportional limit for joints with two rows of four 19-mm (0.75-in.) bolts in laminated Douglas Fir members was about the same as for similar joints with one bolt, but the ultimate bearing stress was about a third less (Fig. 4). With 12.5-mm (0.50-in.) bolts, the bearing stress at the proportional limit and the ultimate bearing stress was about 15% less (Fig. 5).
- Joints with the eight bolts under tensile load slip had two to three times more slip at the proportional limit than did joints with a single bolt. However, joints with a single bolt had twice as much slip at ultimate load as did joints with eight bolts.

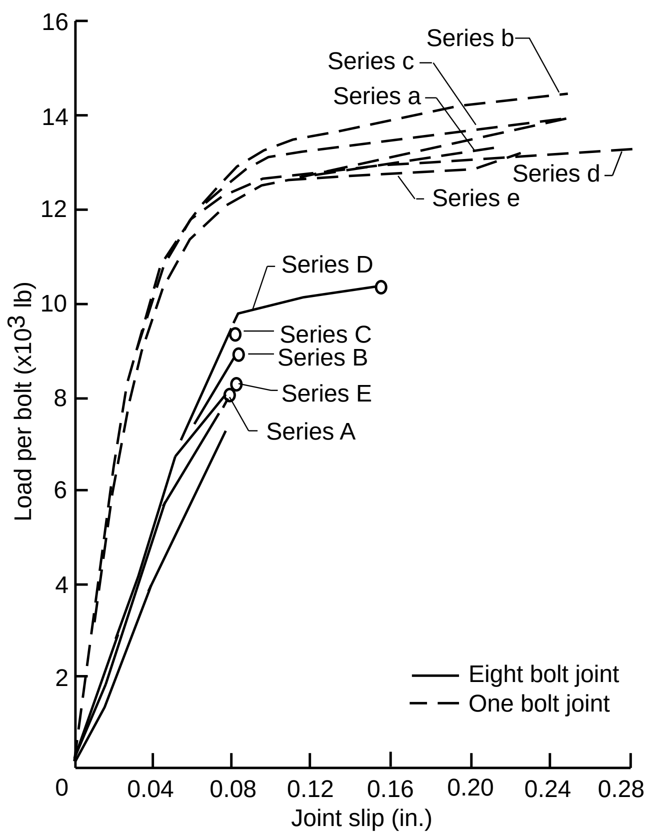

Figure 4—Load–slip curves for five series of joints with eight 19-mm (0.75-in.) bolts in laminated Douglas Fir members (series A through E) and their matching single-bolt control joints (series a through e). The curves are composite load–slip curves derived from tests of three specimens of a kind for the eight-bolt joints and six specimens of a kind for the one-bolt joints: Series A, 3 in. (75 mm) bolt spacing; B, 114 mm (4.50 in.) bolt spacing; C, stitch bolts; D, difference in densities; and E, tapered end (Doyle 1964).

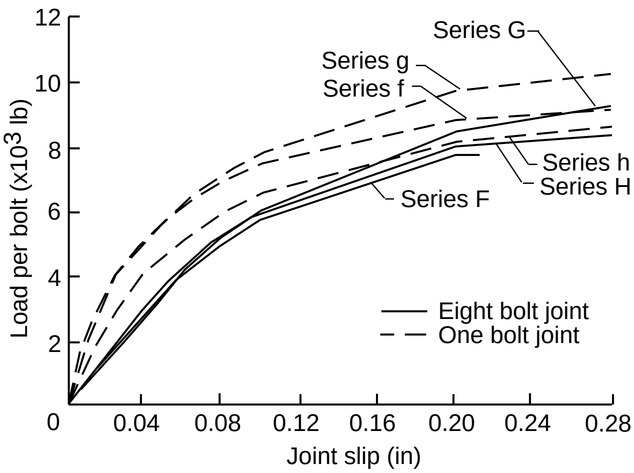

Figure 5—Load–slip curves for three series of joints with eight 12.5-mm (0.50-in.) bolts in laminated Douglas Fir members (series F through H) and their matching single-bolt control joints (series f through h). The curves are composite load–slip curves derived from tests of three specimens of a kind for the eight-bolt joints and six specimens of a kind for the one-bolt joints: Series F, 75-mm (3-in.) spacing; G, 114-mm (4.50 in.) bolt spacing; and H, difference in densities (Doyle 1964).

- The strength and behavior of joints with eight 19-mm (0.75-in.) bolts were not appreciably affected by such modifications in construction as were bolt spacings of 114 mm (4.5 in.), stitch bolts, tapered end cut, or bolts in laminations of low and high density.
- The bearing stress at the proportional limit of bolted joints was about 35% that of the maximum crushing strength of the wood when 19-mm (0.75-in.) bolts were used, and about 30% when 12.5-mm (0.5-in.) bolts were used.

Kunesh and Johnson (1968) investigated the strength of multiple-bolted joints as a function of spacing, seasoning, and types of loading by testing three-member joints. These joints compared members standard 38 mm (nominal 2 in.) thick, bolted together by 19-mm (0.75-in.) mild steel machine bolts. Bolt holes were drilled 1.5 mm (0.0625 in.) oversize. All joints had a bolt length to bolt diameter ratio of 2.2 and a projected area of 1,420 mm^2 (2.2 in^2) for each bolt. The joints tested are shown in Figure 6.

Kunesh and Johnson point out that discrepancies in the bolt loads at the proportional limit exist when the strength of specimens consisting of assorted bolt spacings and patterns was compared with that of a single-bolt connection. In a two-bolt connection, a higher mean proportional load is reached if the bolts are spaced perpendicular to the applied direction of loading (i.e., spaced transversely), than if they are aligned parallel to the direction of loading. Furthermore, the

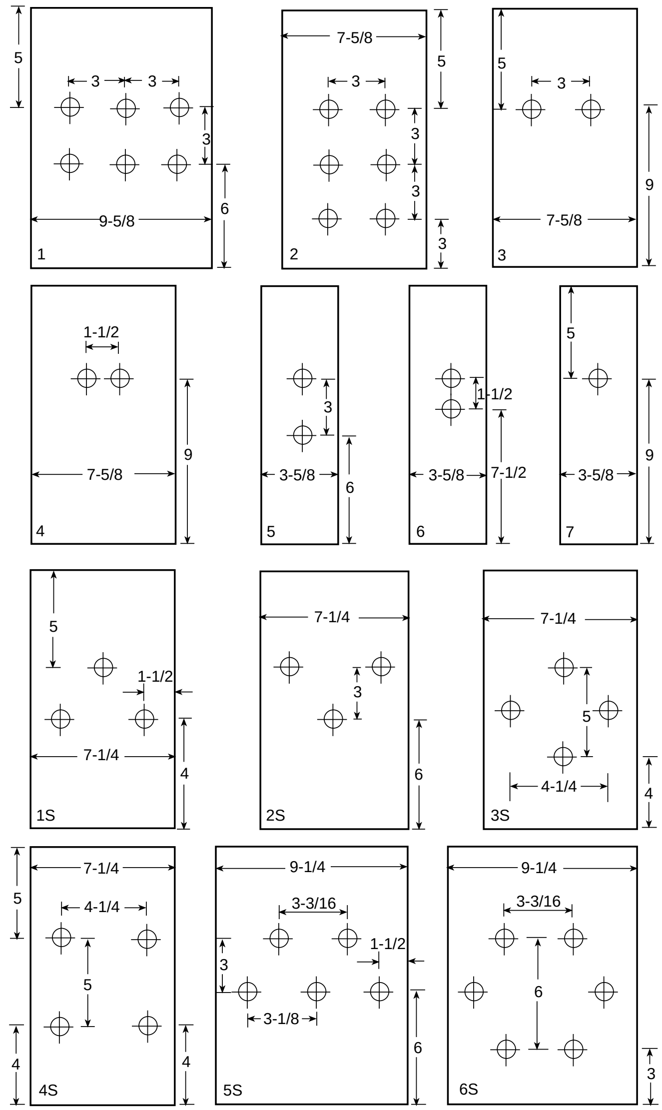

Figure 6—Spacing of bolts in regular and staggered patterns. All assembled joints were 355 mm (14 in.) long, but width was in inches. Pattern 4S was not staggered, but was included for comparison with pattern 3S (Kunesh and Johnson 1968).

proportional limit is sensitive to seasoning and loading type, both of which have little influence on the ultimate capacity of the joint. Their conclusion that staggered bolt patterns are an efficient means of minimizing the effect of shear contradicts Trayer's (1932) recommendations.

From the experimental evidence, the following was concluded by Kunesh and Johnson:

- Joints fabricated with unseasoned material, but tested dry, had significantly greater bearing capacity at proportional limit load than those fabricated with seasoned material. Seasoning was not a significant factor in determining maximum strength of joints.

- The pattern of bolts was a significant factor in determining bearing capacity at proportional limit and maximum load. In general, load-carrying capacity decreased as the number of bolts in a joint increased and appeared to be a function of the number of double-shear areas parallel to grain between rows of bolts.
- Cyclic loading of joints to simulate in-service conditions resulted in greater loads at proportional limit than did joints loaded continuously. Type of loading did not affect the maximum strength of joints.
- Proportional limit load of joints was sensitive to several variables that had little influence on the maximum load-carrying capacity. Therefore, proportional limit load of joints may be a poor datum to establish design values. A new approach to standards for design should be considered, possibly one that relates working values to minimum values expected for maximum load.
- Two methods of transmitting load to joint assemblies did not influence strength at either proportional limit load or maximum load, and differences between joints were caused mainly by the pattern of bolts.
- No significant difference was found between staggered bolt joints and single-bolt controls, either at proportional limit or maximum load. Staggered bolt joints, as a group, had greater load-carrying ability than did regular-pattern joints.

In the late 1960s, it was noted that load sharing of a multiple-connected joint in timber was non-uniform. Cramer (1967) carried out an experimental study and a theoretical analysis. Considerable care was taken in drilling and aligning the bolt holes. The bolts used were 19 mm (0.75 in.), and the central timber member was 38 mm (1.5 in.) thick with 6.5-mm- (0.25-in.-) thick aluminum sideplates. The drawback of this careful control was the inability to model the type of members used in the field.

The innermost bolts of a connection are the least stressed, and the end bolts are the most stressed. The addition of more than six bolts connected in a line produces a stress reduction on the already least-stressed inner bolts. This in turn, results in more load on the end bolts.

Increasing the bolt spacing has the same effect as increasing the number of bolts does on the ultimate strength of a member, but not as dramatic as increasing the number of bolts in a row. Cramer noted that minor misalignment of bolt holes can cause drastic changes in the bolt loads. Also, at the ultimate load, some redistribution of load occurs from the highly stressed end bolts to the less heavily stressed inner bolts. This is typical of a bearing-type failure.

Masse and others (1988) also tested multiple-connected bolted joints with a timber (glulam) center member and steel sideplates. The timber was 130 mm (5.1 in.) thick, and the steel plates were 6.25 mm (0.25 in.) thick. The bolts had a 19-mm (0.75-in.) diameter and a *L/D* 6.8. The bolt holes were drilled 1.6 mm oversize. The following summarizes their conclusions:

- The loaded end distance has an important effect on the bolt mean lateral strength. When the end distance was increased from 7*D* to 10*D* (an increase of 1.42), the bolt mean lateral strength increased by 1.35 and 1.24 for Douglas Fir and Spruce glulam, respectively.
- Bolt spacing in a row parallel to the grain had an important effect on bolt capacity. Increasing the bolt spacing from 4*D* to 6.7*D* caused the bolt mean lateral strength to increase by factors of 1.44 and 1.28 for Douglas Fir and Spruce glulam members, respectively.
- Increasing the number of rows of bolts from one to two decreased the bolt mean lateral strength by factors of 0.71 and 0.6 for Douglas Fir and Spruce glulam members, respectively.
- The group effect was more critical for Douglas Fir glulam members than for Spruce. When the number of bolts in a row was increased from one to four, the bolt mean lateral strength was decreased by a factor of 0.48 for the Douglas Fir glulam and 0.75 for the Spruce-Pine-Fir glulam members.
- Increasing the spacing from 2*D* to 3*D* between rows did not alter the mean lateral strength of the bolt.
- There was greater initial slip on the threaded end of the bolt.
- In the single-bolted joint, failure was by wood crushing. For the multiple-bolted joints, failure was generally by shear in the case of Douglas Fir glulam and a mixture of shear and splitting in the case of Spruce glulam.

Yasamura and others (1987) investigated the influence of end distance, spacing, and number of bolts on the ultimate properties of bolted joints in glued-laminated timber. The timber members were glued-laminated Spruce (average specific gravity 0.44) and Douglas Fir (average specific gravity 0.54). The ratio of the thickness of the main member to the bolt diameter (*L/D*) was 2, 4, and 8. The end and edge distances were 7 and 2.5 bolt diameters, respectively. When steel sideplates were used, the end distance varied from 2.5*D* to 10*D*. The bolt sizes used were 16 mm (0.625 in.) and 20 mm (0.75 in.). The predrilled holes were the same diameter as that of the bolt for the timber and a millimeter greater for steel.

The embedding strength of the Spruce and Douglas Fir was 36,951,840 Pa (5,360 lb/in^2) and 53,635,320 Pa (7,780 lb/in^2), and the bolt yield point was 490,300,000 Pa (71,120 lb/in^2).

The relationship between the end distance and the ultimate load of a single-bolted joint composed of a Spruce glulam member and steel sideplates loaded parallel to the grain is shown in Figure 7. For each *L/D* ratio, there is an end distance (measured in terms of the bolt diameter) above which the load is limited by the embedding strength of the bolt and consequently remains constant. The dashed lines indicate general behavior; the different end distances at which the limiting bearing strength values are reached explain the differences in the multiple-joint behavior for different end distances, as described in the following.

The multiple-bolted joints tested (Fig. 8) were composed of a Spruce glulam main member with 12-mm- (0.5-in.-) thick steel sideplates and 16-mm- (0.625-in.-) diameter bolts. The number of bolts varied from two to twelve, and the number of rows varied from one to three. The *L/D* ratios used were 4, 6, and 8; bolt spacings were four and seven times the bolt diameter.

Figure 9 shows the relationship between the number of bolts and the ultimate load per bolt for *L/D* 4, 6, and 8. Two trends are apparent.. The first is that as the *L/D* ratio increases, the ultimate load per bolt increases roughly in proportion to the increase in bolt length. The second trend is that as the number of bolts increases, the ultimate load per bolt decreases. For *L/D* 4, the load was not greatly reduced when the bolt spacing was reduced from 7*D* to 4*D*. The reduction in load with the reduction in spacing was much greater for the case of *L/D* 8 and was somewhere in between for the case of *L/D* 6. Values of the “rate of decrease,” or modification factor, for the number of bolts are shown in Figure 10 for both bolt spacings. Although the trend lines (dashed) may not be accurate, they nevertheless show that for the larger bolt spacing of 7*D*, the longer bolts (*L/D* = 8), which would exhibit more bending, showed the smallest reduction in load/bolt as the number of bolts increased. In the case of the 4*D* bolt spacing, the trends for each *L/D* ratio are more difficult to determine, and the single trend line is probably accurate.

The influence of the number of rows of bolts parallel to the loading is shown in Figure 11. Increasing the number of rows of bolts caused the greatest reduction in load for bolts of *L/D* = 8.

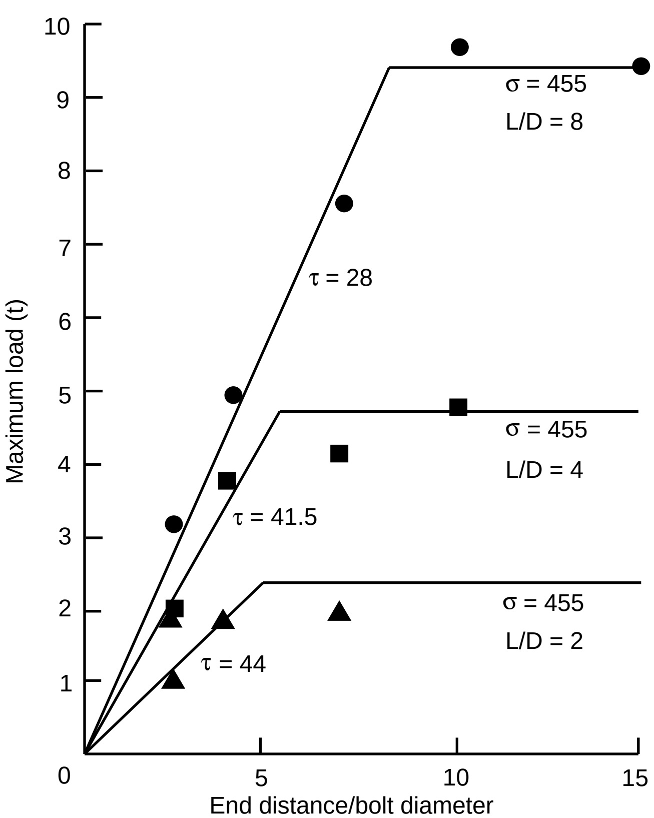

Figure 7—Relationship between maximum load and end distance (Yasamura and others 1987). (Reprinted with permission.)

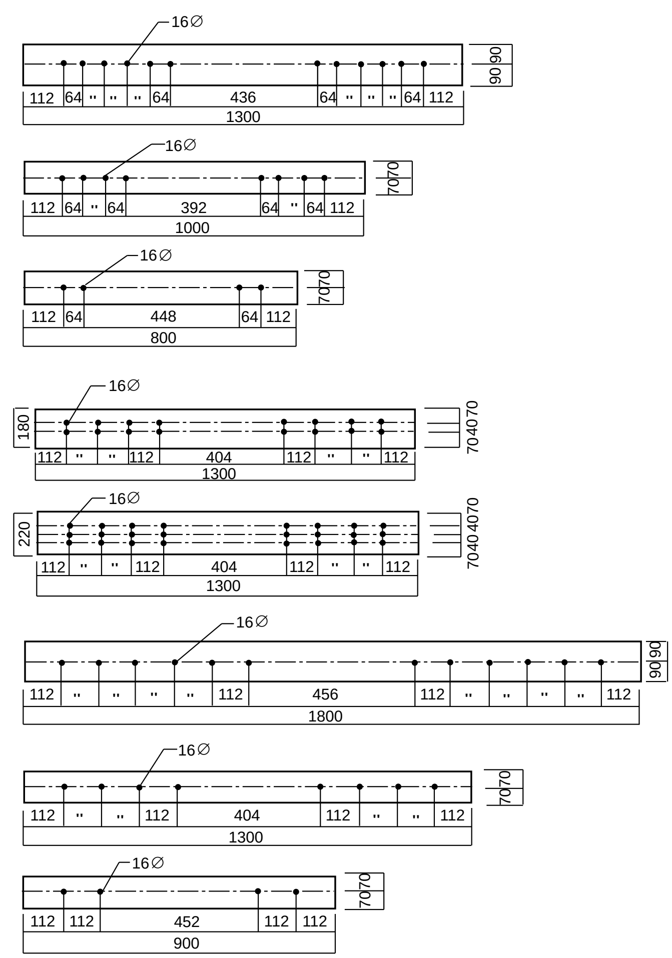

Figure 8—Multiple-bolted joints tested (Yasamura and others 1987). (Reprinted with permission.)

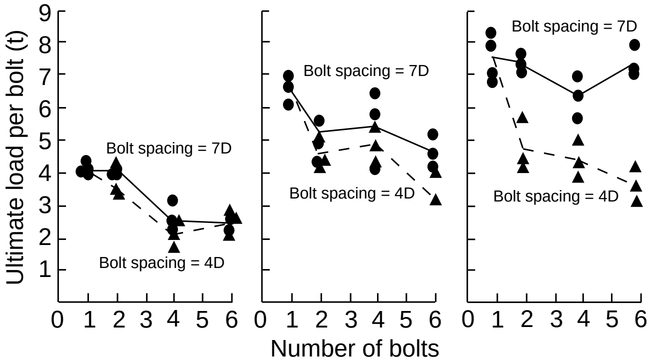

Figure 9—Relationship between the ultimate load per bolt and the number of bolts for (left) *L/D* = 4; (center) *L/D* = 6; and (right) *L/D* = 8 (Yasamura and others 1987). (Reprinted with permission.)

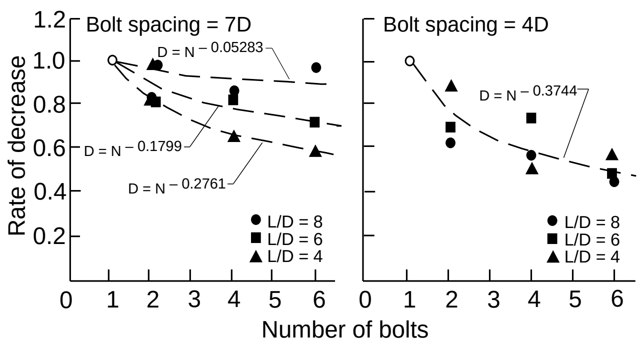

Figure 10—Relationship between the rate of decrease of the bolt load to the number of bolts for (left) bolt spacings of 7*D*, and (right) bolt spacings of 4*D* (Yasamura and others 1987). (Reprinted with permission.)

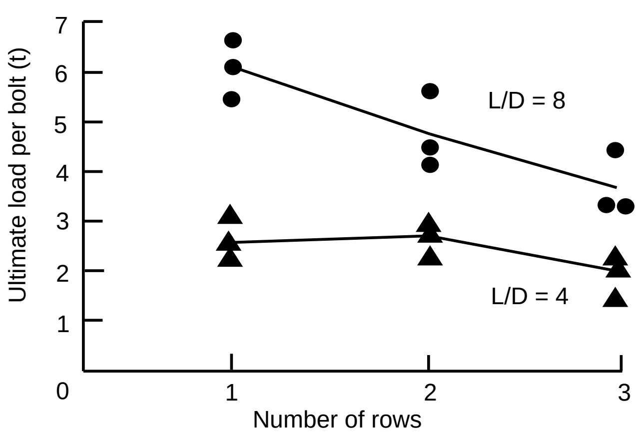

Figure 11—Relationship between the ultimate load per bolt and the number of rows of bolts (with four bolts per row) (Yasamura and others 1987). (Reprinted with permission.)

Tests were also carried out on single-bolted joints loaded perpendicular to the grain, using 16- and 20-mm- (0.625- and 0.80-in.-) diameter bolts having *L/D* 4, 8, and 10. The end distances varied from 4 to 25 times the bolt diameter, and the edge distances varied from 2.5 to 10 times the bolt diameter. Loads perpendicular to the grain of the glulam were applied either by direct tension or loading a beam member supported at one end by a bolt passing through a hole in the beam and a roller support at the other end. Multiple-bolted joints were not tested using perpendicular-to-the-grain loading.

Mettem and Page (1992) reported on tests conducted on both single- and multiple-bolted joints. The objective was to investigate how the load on a multiple-fastener bolted joint with steel sideplates and a glulam central member would be distributed between the individual bolts. The tests were designed so that failure would occur in a pure embedment mode in the case of both single- and multiple-fastener joints.

The thickness of the central glulam member was 2.75 times the diameter of the fastener (i.e., *L/D* = 2.75), and the thickness of the steel sideplates was 0.83 times the bolt diameter. The bolts used had a 12-mm (0.5-in.) diameter; the European whitewood glulam member was 33 mm (1.25 in.) thick with 10-mm- (0.375-in.-) thick steel sideplates. A special multiple embedment testing rig was used with strain-gauged sections to enable the load applied to each bolt to be measured. Tests were carried out for both tension and compression parallel to the grain, although only compression was carried out perpendicular to the grain. Bolts were spaced at 5*D* parallel or perpendicular to the grain. The end distances were 7*D* parallel to the grain and 4*D* perpendicular to the grain; the edge distances were 4*D* parallel to the grain and 2.1*D* perpendicular to the grain. These spacings and distances conformed with the Eurocode 5, April 1992, recommendations.

The test procedure was to drill the bolt holes immediately before each test to a diameter of 12.2 mm (0.5 in.), with the intention of eliminating possible fabrication effects, misfit, shrinkage, swelling between holes. The bolt holes were positioned to avoid gluelines in the laminated material.

For single-bolt specimens, loading was carried to failure after the elastic stiffness had been measured. For the four-bolt embedment tests, loading was not continued until failure because the intention was only to measure load distribution within the elastic range.

The mean values of the four-bolt embedment tests (Fig. 12) showed some variation from the value of 0.25 corresponding to the bolts sharing the load equally. These results relate only to elastic embedment and do not reflect the likely distribution at failure.

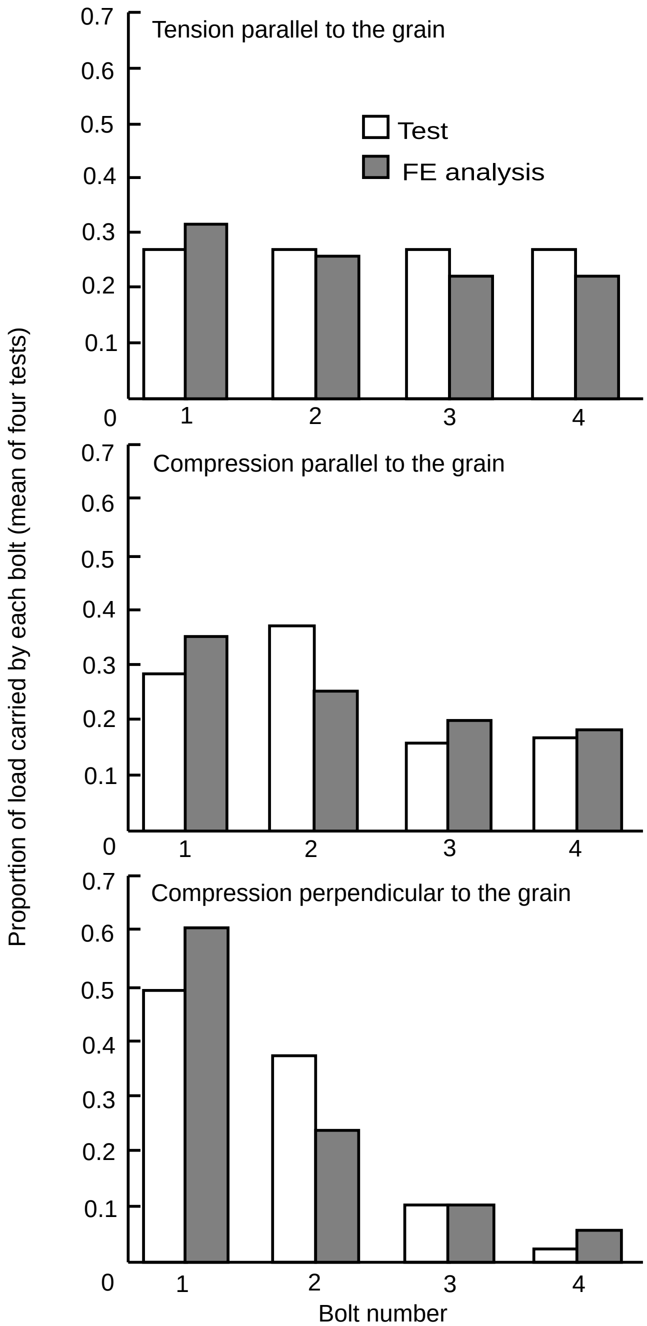

Figure 12—Proportion of load carried by each bolt in a four-bolt joint (Mettem and Page 1992).

A finite element analysis of the single- and multiple-bolted joints was also carried out. The embedment stiffness was modeled by introducing an additional spring element into the analysis at each bolt hole instead of using a finite element model where the variations in the bolt/wood contact area and the effects of friction and slipping need to be considered. The parameters for the embedment stiffness were taken from the single-bolt tests. The results are shown in Figure 12.

Table 1—Multiple-fastener joint reduction factors for different modes of loading in elastic embedment

Mode of loading	Reduction factor By test	Reduction factor By analysis
Compression parallel to grain	0.66	0.69
Tension parallel to grain	0.89	0.81
Compression perpendicular to grain	0.50	0.41

To compare these results with the work of other researchers, reduction factors were calculated for the multiple-fastener joints. This was done by assuming that the total load on the multiple-fastener joint might be such that the individual fastener carrying the greatest part of the load would only be loaded to the value permitted in a joint with a single fastener of the same type. It was assumed that with perfect load distribution, each bolt would carry 0.25 of the total load. Dividing this value by the proportion of load actually carried by the most heavily loaded bolt in the tests or in the finite element analyses provides the information given in Table 1.

An observation from these tests is that the load sharing for tension parallel to the grain is more uniform than is compression parallel to the grain. Another observation is that under compression perpendicular to the grain, the bolt nearest the loaded end carries most of the load and the bolt nearest the free end carries almost no load. Consequently, the reduction factor would appear to be as low as 0.4 to 0.5. Even though the values in Table 1 relate to elastic embedment rather than failure in a bolted joint, they suggest that the reduction values given in most codes need to be reassessed. This is especially true for compression perpendicular to the grain, which in the past, has not been separated from the case parallel to the grain when determining reduction factors.

Analytical Research

In the late 1960s, a series of analytical (and experimental) studies determined that the load sharing of connectors in a multiple-connected joint was non-uniform. Cramer (1967) proposed a model for a butt-type, multiple-connected timber joint loaded in the elastic range. He used the theory of elasticity that considered deformation of the timber member and the bending of the bolts. Dimensionless constants were used to represent the relative stiffness of the sideplates to the main member. His analysis included the properties of the members and bolts, the number of bolts in a row, and the bolt spacing. Cramer assumed that the bolts would deflect in bending, resulting in a non-uniform distribution of stress in the members.

Solutions were presented in the form of curves from which individual bolt loads could be obtained for joints having up to 10 bolts in a row. Specific results from Cramer's study include the following:

- The theoretical equations predicted the distribution of load among the bolts in a butt-type timber joint.
- The most uniform distribution of bolt loads occurred where the main member and bolt splice plates had the same stiffness.
- The addition of more than six bolts in a row did not substantially increase the elastic strength of a timber joint.
- Bolt spacing should be kept as small as possible, consistent with good design practice.
- A timber joint must be fabricated with high quality and accurate alignment of holes to have a predictable distribution of load among the bolts.

Lantos (1969) suggested that the allowable load on a group of connections should not be a linear function between the number of fasteners and a single fastener. Lantos's analysis is similar to Cramer's in so far as joint displacement compatibility and force equilibrium are concerned. However, instead of using a joint slip proportionality factor or flexibility coefficient as Cramer did, Lantos used a stiffness factor in the form of a joint slip modulus. His analysis treats a single line of bolts and assumes that the maximum bolt load will occur on one of the extreme ends. Expressions are derived for the loads acting on these two end bolts. Lantos concluded that factors influencing the load distribution on the bolts were the relative stiffness of the sideplates and the main member, the number of fasteners and their spacing, and the stiffness of the connection. As the relative stiffness of the joint members change, the more the load distribution among the bolts deviates from uniformity. Also, the percentage deviation from uniformity of load among the bolts increases as the number of bolts in a line increases.

Wilkinson (1980) reviewed the state of knowledge of modification factors for a row of bolts or timber connectors. He looked at both the analytic methods of analysis that had been proposed and the experimental work that had been undertaken. He concluded the following for fasteners placed in a row parallel to the grain and loaded parallel to the grain:

- Present (1980) methods of analysis appear to predict the proportional limit load for a row of fasteners. However, the actual proportional limit load can be difficult to determine experimentally.
- Present (1980) analytical methods overestimate the strength (failure load) of a row of fasteners as would be expected, because the methods do not consider the nonlinear load–slip behavior of a single fastener.
- Assumed values of joint variables used to arrive at modification factors are adequate to conservative. Fewer assumptions would be needed if tables of modification factors were based on stiffness rather than area.
- It may be desirable to have separate tables for bolts and timber connectors because of the large difference in single-fastener stiffness and the different procedures by which design loads are developed.
- It would be desirable to have an analytical method to predict single-fastener load–slip relationships to failure.
- It would be desirable to have an analytical method to predict the distribution of load among fasteners in a row; such a method must account for fabrication tolerances and nonlinear load–slip relationships for single fasteners.

Wilkinson also suggested several areas for additional analytical and experimental research:

- Modify present methods of analysis to allow for different nonlinear load–slip relationships for each fastener. These modifications could also include fabrication tolerance effects by allowing for slip without load.
- Conduct a random simulation of non-linear load–slip relationships and fabrication tolerance for fasteners in a row to obtain statistical distribution of modification factors for design procedures.
- Verify modified methods of analysis with experimental tests in which the (a) amount of slip before contact between the fasteners is measured, (b) distribution of load among the fasteners is measured to failure, (c) slip of each fastener is measured, and (d) joints are evaluated over a range of member stiffness values and number of fasteners per row.
- Reassess the procedures for arriving at the values of design loads for single-fastener joints.
- Develop an analytical method of predicting the single-fastener load–slip relationship to failure.
- Investigate the load distribution in rows of fasteners where the loading is perpendicular to the grain. This should include how to define the joint area and the effects of shrinkage.

Wilkinson (1986) investigated some of the conclusions and suggestions given in his 1980 paper. In the later paper, Wilkinson described an analytical method for predicting the load distribution in a row of bolts. His method was based on that of Lantos but modified to allow for variable spacing of the bolts in a row and an independent, piecewise linear load–slip relationship for each fastener in a row. He also carried out experimental verification of his analytical results. His results showed that each row of bolts has a unique load distribution dependent on the variation in load–slip curves and fabrication. He showed that the analytical method was able to predict the load distribution if sufficient knowledge of the individual fastener load–slip relations is available. Wilkinson's results indicated that the design procedures for rows of bolts are not conservative.

The following summarizes Wilkinson's (1986) research:

- The load distribution for any particular row of bolts is unique. Any one bolt may be the major load carrier. Also, any bolt hole may be misdrilled, causing that bolt to transmit almost no load for a major portion of the joint loading.

- The analytical model can predict the load distribution among the bolts, providing that knowledge of the individual load–slip curves and fabrication effects are available.

- Current design procedures use a single value for the load–slip relationship of the bolt. However, comparison of load distributions, using a single curve for all bolts with results from random curves and fabrication effects, indicates that these present design procedures account for little of the actual load distribution among bolts in the row.

- To develop an adequate design procedure, more information is needed on the variability of individual fastener load–slip relationships within pieces of lumber. This information could be used to generate random load–slip curves for each bolt or fastener in the row. Additional statistical information is also needed on fabrication effects and their interaction with the load–slip curve.

- The analytical method used independent load–slip relationships for each fastener in a row. Therefore, the method provides a means of assessing the load-carrying capacity of rows of different size or type of fasteners if the load–slip relationship for each is known.

- Results indicate that present design procedures for rows of bolts are not conservative, because they do not account for the large effect of variability and fabrication on the load distribution. However, many joints are in use and are giving adequate service. This performance indicates that some other compensating occurrences are in the design procedure, such as low estimates of design loads for single fasteners or loads to which structures are subjected.

Steck (1984) presented his results for the effective number of dowels and nails in timber joints. It was assumed that the maximum loaded fastener would be either of the two end fasteners, hence the efficiency η of the joint could be derived from the requirements that

$$\begin{aligned} F_1 &\le F_{\text{allowable}} \\ F_n &\le F_{\text{allowable}} \\ \frac{N}{r \cdot n_{\text{ef}}} &\le F_{\text{allowable}} \end{aligned} \qquad (1)$$

where

N = axial force,
F_1, F_n = force in end fasteners,
r = number of rows of fasteners,
n_{ef} = effective number of fasteners, and
n = number of fasteners in a row.

Hence,

$$\eta = \frac{n_{\text{ef}}}{n} = \frac{1}{n(1-a_1)} = \frac{1}{na_{\text{n}-1})} \qquad (2)$$

with

$a_1 = F_1/N$

This solution depends on the number of fasteners in a row, the fastener spacing, the slip modulus, and the stiffness of the individual members being joined. By expressing some of these parameters in terms of the building code requirements, it was possible to put the solution from Lantos's equations in a form to compare it with the expressions in the CIB code. In the case of joints with dowels, the efficiency η was found to be a little less than the CIB code formula for n = 3 and 4, but greater for n > 4. Steck made suggestions to change the CIB code reduction factors for multiple-fastener joints.

In a subsequent paper, Smith and Steck (1985) re-examined the correctness of the CIB design code rules for the influence of the number of rows of fasteners on the ultimate capacity of axially loaded timber joints. Limited experimental results for the observed reduction factor (i.e., strength of the multiple-fastener joint compared with strength of a single-fastener joint) were compared with the CIB code, BS 5268: Part 2 design rule, and values from Lantos's theory for the cases where the (a) areas of the main and side members equaled the CIB recommended minimum values and (b) side members had half the cross-sectional area of the main members and their area equaled the CIB minimum. Recommendations were made to simplify the CIB design equations for nailed joints and split-ring and shear-plate connected joints. Smith and Steck concluded that the following topics still needed research:

- The need to account for the number of fasteners or connector units in ultimate limit states design calculations for joints with arbitrary combinations of thrust, moment, and shear forces

- The need to account for the number of fasteners or connector units in serviceability limit states design calculations for joints

Table 2—Geometric and material parameters for various studies of multiple-bolt connections (Soltis and Wilkinson 1987)[a]

	Main member		Side member						
Reference	Species	Specific gravity	Material	Specific gravity	Bolt diameter (in.)	Number of bolts	*L/D*	Bolt spacing (in.)	Bolt end distance (in.)
Doyle and Scholten (1963)	Douglas-fir	0.43	0.3125-in. steel and Douglas Fir	0.43	0.50, 0.75, 1.0	1, 4[b]	2.5 to 5	4.0	5.25
Longworth and McMullin (1963)	Douglas-fir	0.49 to 0.52	Douglas Fir	0.49 to 0.52	0.75	4	3.5	3.0	3.0
Doyle (1964)	Douglas-fir	0.47 to 0.52	0.50-in. steel	—	0.50, 0.75	1, 8[b]	44.3 to 6.5	3.0; 4.5	5.25
Kunesh and Johnson (1968)	Douglas-fir	0.42 to 0.54	Douglas Fir	0.42 to 0.54	0.75	1 to 6	2.2	1.5; 3.0	—
Potter (1982)	European redwood	0.47	European redwood	0.47	0.375	1 to 40	5.3	1.5	1.5
Hirai and Sawada (1982) and Hirai (1983)	Spruce	0.42 to 0.45	Steel	—	0.25 to 0.625	1	1.3 to 6.7	—	2.5 to 10.5 × diameter
Wilkinson (1986)	Douglas-fir	0.41 to 0.55	0.1875-in. steel	—	0.75	2 to 7	1.5	6.0	—

[a]1 in. = 25.4 mm.
[b]Total number of bolts in two rows.

Soltis and Wilkinson (1987) reported on the state of knowledge for the design of single- and multiple-bolted connections. The European Yield Theory (Johansen 1949) formed a common basis of comparison, although direct comparison of all previous experimental and analytical work was not always possible because the studies differed in more than one connection property. A summary of the various studies on multiple-bolt connections is given in Table 2.

Values of the modification factors determined from the studies on a two-row by two-column bolt pattern are summarized in Table 3, based on proportional limit and ultimate strength. Also included in Table 3 are the values given by the *National Design Specification for Wood Construction* (NFPA 1986). Table 4 gives the modification factor for other arrangements of multiple bolts. Some modifying factors have experimental values greater than unity, which is theoretically impossible, because of experimental variability. Soltis and Wilkinson also compared modification factors used in various conditions (Fig. 13).

Zahn (1991) suggested that Lantos's equations be reduced to a single equation to give the distribution of load among fasteners in a row. Current wood design codes include double entry tables constructed using the Lantos analysis, but for simplicity had to ignore certain effects and are limited to a range of design parameters. The single equation form of the Lantos analysis enables the (a) maximum possible capacity of a serial row of fasteners to be determined and (b) number of fasteners required to achieve a given row capacity to be determined. The single equation approach provides a suitable criterion for inclusion in a design specification for multiple-fastener joints. Zahn suggested suitable values for the load–slip constant in the Lantos analysis and the single equation approach based on available experimental data. The single equation for the Lantos analysis is

$$C = \frac{1-m}{1+r}\left[\frac{(1+rm^n)(1+m)-1+m^{2n}}{m(1-m^{2n})}\right]$$

Table 3—Values of row modification factor *K* for a two-row by two-column bolt pattern (Soltis and Wilkinson 1987)

Reference	Proportional limit	Ultimate strength	NDS[a]
Doyle and Scholten (1963)			
Wood sideplates, parallel to grain	0.65 to 0.95[b]	0.85 to 0.98[c]	0.97
Steel sideplates, parallel to grain	0.76 to 0.92[b]	0.84 to 0.94[c]	0.88
Wood sideplates, perpendicular to grain	0.53 to 0.82[b]	0.70 to 0.80[c]	0.97
Steel sideplates, perpendicular to grain	0.68 to 0.89[b]	0.63 to 0.82[c]	0.83
Kunesh and Johnson (1968)			
Wood sideplates, parallel to grain	0.91	0.99	0.92
Pyner and Mathews (1979)			
Laminated glass fiber-reinforced plastic		0.81	

[a]National Design Specification (NFPA 1991).
[b]Based on 20-mm (0.80-in.) slip; proportional limit values not given in original paper.
[c]Dependent on bolt diameter.

Table 4—Values of modifying factor *K* for various bolt patterns (Soltis and Wilkinson 1987)

	Bolts		Modifying factor K		
Reference	Number	Pattern	Proportional limit	Ultimate strength	NDS[a]
Doyle (1964)					
Steel sideplates, parallel to grain	8	4 rows by 2 columns	1.03 to 1.24[b]	0.61 to 0.87	0.87
Kunesh and Johnson (1968)					
Wood sideplates, parallel to grain	2	1 row by 2 columns	1.05	1.08	1.0
	2	2 rows by 2 columns	0.90	0.86	1.0
	6	3 rows by 2 columns	0.89	0.71	0.98
	6	2 rows by 2 columns	1.10	0.79	1.0
Potter (1982)					
Wood sideplates, parallel to grain	2	1 row by 2 columns	—	0.96	1.0
	2	2 rows by 2 columns	—	0.92	1.0
	6	3 rows by 2 columns	—	0.79	0.95

[a]National Design Specification (NFPA 1991).
[b]Dependent on bolt diameter and spacing.

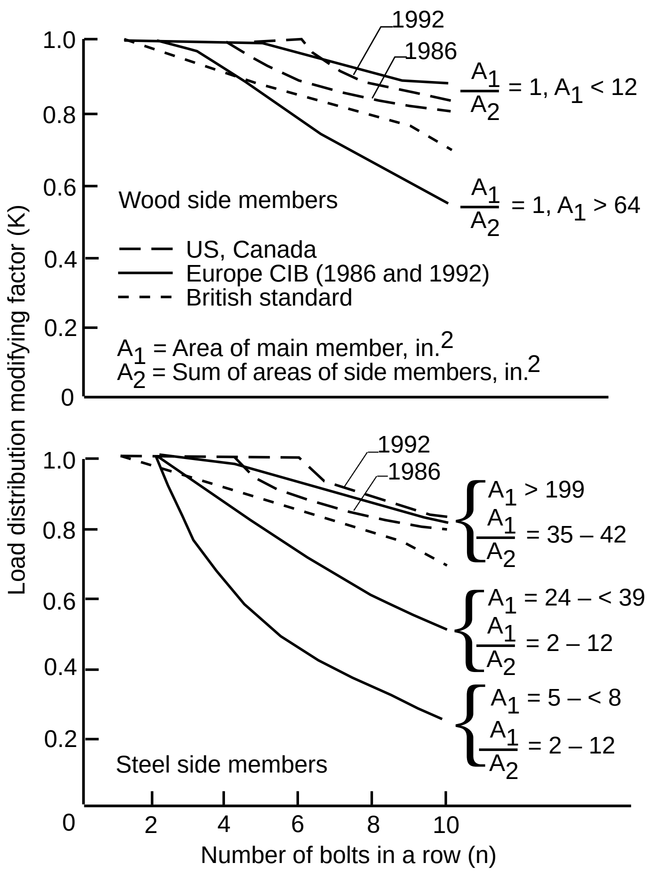

Figure 13—Load-distribution modifying factors defined by U.S. and Canadian, British and European standards for wood side members and steel side members (Soltis and Wilkinson 1987).

where

$$m = \tau - \sqrt{\tau^2 - 1}$$

$$\tau = 1 + \left[\frac{1}{(\mathrm{EA})_{\text{main}}} + \frac{1}{(\mathrm{EA})_{\text{sides}}}\right]\gamma\frac{s}{2}$$

γ = load–slip constant for a single fastener,
s = pitch spacing,
$(\mathrm{EA})_{\text{main}}$ = axial stiffness of main member,
$(\mathrm{EA})_{\text{sides}}$ = axial stiffness of side members, (ignoring boring or grooving), and
$r \le 1$ = ratio of smaller to larger axial stiffness value.

If we are interested in the effective number of fasteners a (which is the inverse of C), such that the row capacity is a times the capacity of a single fastener, then

$$a = \frac{1+r}{1-m}\left[\frac{m(1-m^{2n})}{(1+rm^n)(1+m)-1+m^{2n}}\right]$$

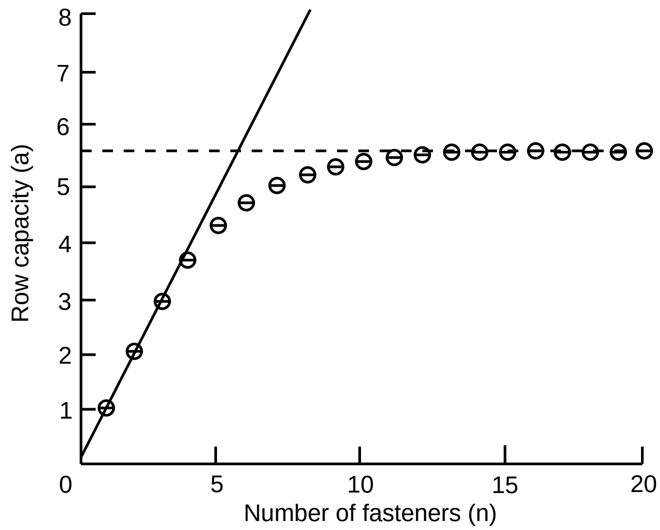

Figure 14—Increase of effective number of fasteners *a* with increase in number of fasteners *n*. If fasteners shared load equally, *a* would = *n*, as shown by solid line. Instead *a* approaches upper limit a_x (dashed line) (Zahn 1991).

As the number of fasteners $n \to \infty$, then $a \to a_x$ such that

$$a_x = \frac{1+r}{1-m}$$

where $a_x N$ is the solution to Lantos's equations. This is illustrated in Figure 14.

In terms of the maximum number of bolts required to achieve a given row capacity,

$$n = \frac{\ln[1-2Q+(rQ)^2-rQ]}{\ln m}$$

in which $Q = \dfrac{ma + a_x}{2(ma_x + a)}$ and $a < a_x$

For the load–slip parameter, γ, Zahn suggested the following values as being conservative:

bolts : 35 MN/m (2×10^5 lb/in)
small timber connectors : 70 MN/m (4×10^5 lb/in) ($D < 0.1$ m (4 in.))
large timber connectors : 88 MN/m (5×10^5 lb/in) ($D > 0.1$ m (4 in.))

These values are suitable for wood sideplates, and γ can be increased by a factor of 1.5 for steel sideplates for bolts but not timber connectors.

Zahn also points out that the work of Soltis and others (1986) showed that for Douglas Fir, γ is diameter d dependent, such that

$$\gamma = 180{,}000d^{1.5}$$

for bearing parallel to the grain. In the case of bearing perpendicular to the grain, γ should be reduced by a factor of 0.5.

In addition to the recommendations of Steck (1984) and Smith and Steck (1985), Steck (1986) summarized the design requirements of several national codes for multiple-bolted/doweled joints. The requirements of U.S. and Canadian codes are shown in Figure 15. Figure 16 shows Steck's comparison of the EC5 design rules compared with the most unfavorable (effectiveness η very low) and the most favorable ($\eta = 1.0$) design rules of the various national standards. The differences show the need for additional research.

Saba and others (1988) investigated the most economical (optimal) connections for a two-member, single-bolted connection. They sought to minimize the connection material cost while satisfying the yield load equations for the ultimate strength of the joint. The resulting nonlinear problem was solved using a sequential linear programming procedure. They determined the optimum combination of the main and side member thickness and bolt diameter, for a fixed value of the applied load, and evaluated the sensitivity of the connections to changes in the design parameters. However, this analysis did not include the effect of factors, such as an oversized hole, but merely sought to satisfy the yield load equations for the various joint failure modes. To date, this approach to optimization has not been applied to multiple-fastener joints. Even if it were applied, it would be an optimization based on code requirements rather than an optimization of the design parameters based on actual joint behavior.

In the section on fastenings in the *Wood Design Manual*, Turnbull (1989) reports that the research by Masse and others (1988) was unable to confirm the resistance values for joints having two or more bolts in a row. The single-bolted joints tested failed by a combination of ductile bending of the bolts and local crushing of the timber under the bolts. Whereas the multiple-bolted joints failed either by splitting along the bolt rows or by plug-shear around the perimeter of the bolt group. These latter failure modes were not considered in the development of the group modification factor previously applied to these types of fastenings. This was consistent with the hypothesis that when loading begins, only one bolt is properly aligned in its predrilled hole and able to start resisting load. This means that the initial slope of the load–slip curve is approximately that of one bolt. As loading and deformation continue, other bolts start to bear in their holes sequentially and the stiffness of the joint increases.

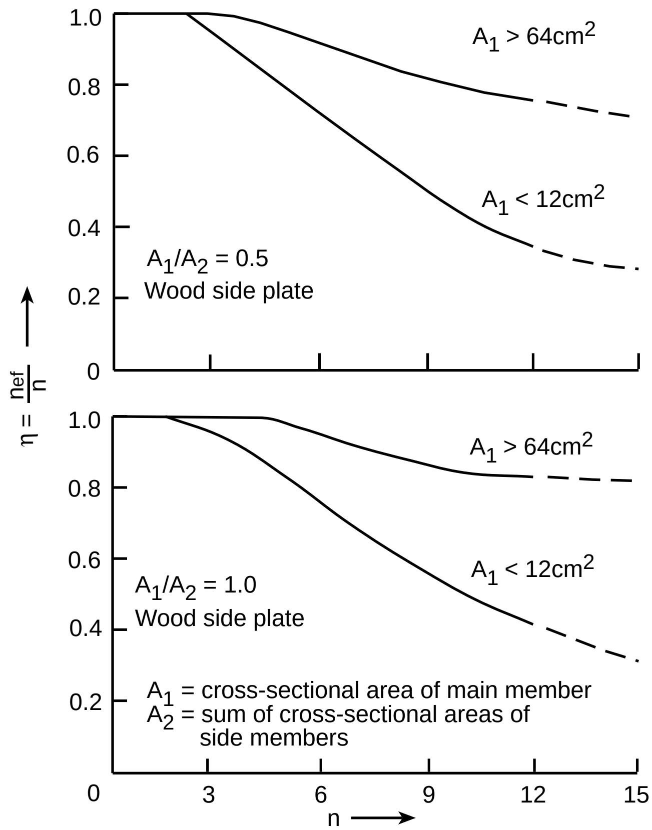

Figure 15—Effectiveness for bolts in Canadian and U.S. standards (Steck 1986). (Reprinted with permission.)

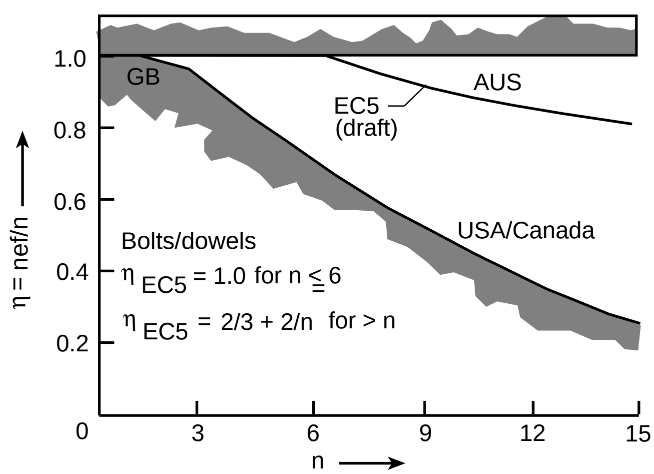

Figure 16—Effectiveness comparing Eurocode 5 (draft) to national design rules for bolted joints (Steck 1986). (Reprinted with permission.)

The tests of Yasamura and others (1987) served as a basis for deriving a new group factor of bolts for the 1989 revision of the Canadian Timber Design Code. The effects of bolt slenderness, bolt spacing, and the number of bolts in a row that were considered separately in the Japanese tests were combined in a single formula for the Canadian code.

The tests of Yasamura and others (1987) and Masse and others (1988) were considered when looking at the effect of the number of rows. It was decided to develop a number of row factors J_R from the comprehensive Japanese tests, so that $J_R = 1.0$ for one row, 0.8 for two rows, and 0.6 for three rows of bolts.

Turnbull noted that although bolted joints under loads perpendicular to the grain have not been studied recently, the J_R factors to be used were conservatively taken as the same as those used for bolt loads parallel to the grain.

Eccentric Loadings

Fantozzi and Humphrey (1995) discussed the effect of bending moments on the tensile strength of multiple-bolted timber connectors. The study described an experimental technique to model wood behavior in the plane perpendicular to the axes of multiple bolts in joint members that are subjected to simultaneous bending and tensile loads. The modeled joints consisted of 0.8-mm- (0.3-in.-) thick wood wafers sandwiched between clear plastic plates, with steel dowels representing bolts passing through them. Wafer thickness was 0.83 mm (0.3 in.), with a width of 90 mm (3.5 in.). The dowel had a 12.5-mm (0.5-in.) diameter, and the holes were drilled with a special edge-cutting drill bit that consistently produced smooth holes with a 13.1-mm (0.5-in.) diameter.

The results of tests with two dowel configurations—conventional three-in-a-row and modified triangular—suggest that relatively small bending moment can reduce the tensile strength of joints and that bolt configuration is an important factor affecting this susceptibility. The modeling method is suggested for developing joint designs with reduced susceptibility to bending moments and investigating the effects of variables, such as wood quality, growth ring orientation, and moisture content. The behavior of corresponding double-shear joints tested under a similar range of loading regimes and with wood of two grades will be reported in a companion paper (Humphrey and Fantozzi [in press]).

Conclusions

Staggered and Aligned Fastener Comparison

Trayer (1932) stated that staggered bolt patterns were to be avoided, because they impeded the stress flow in the tension field passing through the joint. In his view, any two staggered rows of bolts had to be considered as one row with half the fastener spacing, which means a reduction in load capacity for those bolts concerned.

In contrast, the analyses of Kunesh and Johnson (1968) showed no significant differences between staggered pattern joints and single-bolt joints at both the proportional limit load and the maximum load. They found that staggered bolt joints as a group had a greater load-carrying capacity than did regular pattern joints.

The *National Design Specifications for Wood Construction* (NFPA 1991) recommends that when fasteners in adjacent rows are staggered and the distance between adjacent rows is less than a fourth the distance between the closest fasteners in the adjacent rows, measured parallel to those rows, the adjacent rows shall be considered as one row for the purposes of determining group action factors (i.e., row modification factors). With an even number of rows, this principle is to be applied to each pair of rows; with an odd number of rows, the most conservative interpretation is to be taken. These recommendations seem reasonable, although no references were sighted in the literature to support them.

Optimum Configuration

Saba and others (1988) looked at the optimization of a two-member bolted timber joint to determine the thickness and bolt diameter of the member that are required for a minimum cost of material solution capable of carrying a specified load and subject to the requisite yield load conditions for the joint. Effects such as the relative bolt clearance were not included. The optimization was applied to a single-bolted joint and not to multiple-bolted joints.

Row Factors and *L/D* Ratio Comparison

Soltis and Wilkinson (1987) listed the bolt length to diameter ratio, *L/D*, used by previous researchers along with their determination of the row modification factor K and other details (Tables 3–4). Their results are not sufficiently extensive to draw conclusions.

The only test program that studied modification factors as a function of the bolt *L/D* ratio is that of Yasamura and others (1987), who conducted tests at *L/D* 2, 4, and 6. They found that the reduction in the load-carrying capacity with an increase in the number of bolts in a row was influenced by the *L/D* ratios of the bolts.

Spacing and End and Edge Distances

Spacing and end and edge distances are still largely used based on Trayer's (1932) recommendations. Doyle (1964) found that increasing the bolt spacing from 4*D* to 6*D* made no difference to the joint strength. According to Masse and others (1988), increasing the bolt spacing in a row parallel to the grain from 4*D* to 6.7*D* led to an increase in strength, but increasing the spacing between rows from 2*D* to 3*D* gave no change in joint strength. Yasamura and others (1987) showed that the ultimate load of a single-bolted joint was a function of both the end distance and the *L/D* ratio. No investigation has been carried out to determine the extent to which changes in fastener spacings and end and edge distances would make economical changes to joint design.

Type and Size of Connectors

Nothing appears to have been investigated for timber regarding type and size of connectors. In the case of steel joints, bolts with high levels of strength have been used several times to replace some rivets in the strengthening of riveted construction. However, it seems that no experimental work has been published relating to the strength and performance of such joints.

End fasteners in a row of fasteners are required to carry more load than the interior fasteners. Therefore, small-diameter fasteners could be used so that their reduced stiffness could allow greater deformation, thus permitting load redistribution to the interior fasteners. This should be better than using larger diameter fasteners or other types of fasteners of larger load-carrying capacity for the end fasteners, because their increased stiffness could lead to attracting even more load.

Eccentric Effects

Fantozzi and Humphrey (1995) showed that even a small eccentricity can considerably reduce the tension strength in tests using thin wafers of timber. Test results on full-size timber joints should be even more interesting (Humphrey and Fantozzi [in press]). The target moment was applied to the specimen before applying the axial tensile load, thus providing a constantly changing eccentricity as the load was applied. This leads to questions about the loading method.

Recommendations

Staggered and Aligned Fastener Comparison

Only a limited number of studies have been conducted on the effects of staggered and aligned fasteners, and these have contradictory results. Therefore, recommendation is for additional research in this area.

Optimum Configuration

An optimum fastener configuration would require a detailed study of the way that fastener spacing and end and edge distances affect the economics of joint design and the use of either staggered or aligned rows of fasteners. A study of the optimum configuration should also look at the effect of using either a large number of small-diameter fasteners or a small number of large-diameter fasteners. The latter would impose increased concentrated forces on the timber, with the possible failure of the group as a whole; the former would spread the load distribution over a large volume of timber.

Row Factors and *L/D* Ratio Comparison

A systematic study should be carried out to determine how the *L/D* ratio affects the row (or group) factor. This could then be compared with the results of Yasamura and others (1987) and other limited research carried out to date. Detailed research will be necessary for at least one bolt diameter and one species and size of timber member, but possibly only limited testing will be needed for other bolt sizes and timber species. Consistent end and edge distances and spacing would need to be used.

Spacing and End and Edge Distances

Research in spacing and end and edge distances needs to be conducted along the lines outlined for glulam nails, where failure modes relating to wood failure and nail yielding have been investigated.

The work of Mettem and Page (1992) with multiple dowels loaded perpendicular to the grain suggests that additional research needs to be done on multiple-bolted joints loaded perpendicular to the grain. This is because their work in the linear range of embedment shows that the modification factors for the doweled joints are less than when loaded parallel to the grain.

Type and Size of Connectors

Additional investigation is needed on the type and size of connectors to determine whether large-diameter fasteners or other types of fasteners are better able to resist the large forces that develop at the ends of a row of fasteners or whether the best approach is to use small-diameter fasteners that will deform more and shed load to the interior fasteners.

Eccentric Effects

The analytical method of Kamtekar and Wittrick (1984) could be investigated further for use with timber joints. Factors that need consideration are (a) the effects of differing strength levels parallel and perpendicular to the grain, (b) what happens when the fasteners have particular load–slip curves that are not elasto-plastic, and (c) how sloping grain affects the analysis. The author intends to investigate these factors in the near future. This will amplify the work of Humphrey and Fantozzi by showing the extent to which the low perpendicular-to-the-grain strength of timber affects the strength of an eccentrically loaded bolted joint when compared with that expected for a steel-bolted joint.

The performance of bolted joints where the bolts are placed in a circular ring (often done in Europe) should be studied in addition to joints using rectangular patterns of bolts.

Slip of Single- and Multiple-Fastened Joints

The slip of single- and multiple-fastened joints needs to be investigated to provide formulas for use in codes to predict the likely slip of joints under both serviceability and ultimate loads.

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