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Analysis and design of metal-plate-connected joints subjected to buckling loads

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Abstract. A comprehensive analytic study has been conducted to investigate the instability problems of metal-plate-connected (MPC) joints in light frame trusses. The primary objective in this study is to determine the governing factors that constitute the buckling of the metal connectors and their effects on the structural response of joints. Another objective is to recommend design curves for the daily structural design of these joints. The numeric data presented in this paper has emerged from a broad base that was founded on over 350 advanced computer simulations, and was supported by available experimental results obtained by others. This basic-to-applied research includes practical engineering parameters such as size of gaps, shear lengths, gauge (plate thickness) of connectors, size of un-braced areas, failure modes, and progressive disintegration of joints. Square-end members have been emphasized though the results cover the custom-made fitted joints. The results indicate that chord shears cause and dominate the buckling of MPC joints, and the shear length has a more pronounced effect than the size of gaps. Further, large gauges and small un-braced areas improve the buckling response. Several practical recommendations have been suggested throughout the paper such as keeping the ratio of gap/shear length below 3/4 for improving the buckling strength. The study reveals that multi-area joints should not be simplified as single web-to-chord MPC joints such as keeping the ratio of gap/shear length below 3/4 for improving the buckling strength, even where one web is in tension and the other in compression. Finally, the results obtained from this study favorably agree with experimental data by others, and the classic buckling theories for other structural components.

Key words: buckling; compression; connections; connectors; critical loads; finite elements; gauge; joints; light frame; metal plate; MPC joints; shear; stability; standards; trusses; wood.

1. Introduction

In light frame trusses, perforated metal connectors are traditionally pressed into a number of structural fitted or square-end members that are merging at a common point, thus forming an integrated connection. The former application is the most common in practice whereas the latter is emerging as a needed design. By this fastening mechanism, the interaction forces are transmitted from the wood members through the teeth into the connector plates. While the wood-to-wood bearing is still developing, un-braced areas become the principal media for interaction of forces in the connected members for the equilibrium of joints. These areas could become unstable and thus buckle once transmitted forces exceed critical values.

On the other hand, the increasing economic and environmental concerns in today's global competitive market have stimulated industrial needs for new, sustainable and cost-effective structural

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design concepts. Square-end web members in light frame constructions have thus emerged with some of the desired practical advantages compared to the traditional custom-cut members. This new concept offers great promises for flexibility in the design of diverse and complex architectural layouts; large mass production through easy manufacturing and erection processes; cost effectiveness through lowering the costs of fabrication, material handling and using shorter web members (Yasutoshi *et al.* 1990). However, the utilization of standard members for square-end webs creates new structural problems including higher member and joint forces. In addition, unequal standard length of jointed members cause gaps between their ends. Generally, gaps enclose un-braced portions of metal connectors and thus expose them to buckling. Further, changes to standard practices may have significant consumer and political reactions (Grossthanner 1993). This is an important issue in the construction industry because its market values at approximately \$150 million for metal connectors and \$2.5 billion for trusses, and more than 90% of all residential buildings use light frame trusses.

Currently, the literature incorporates some experimental data for the diverse commercial connectors (ANSI 1995, ASTM 1982, CSA 1980, WTCA 1997, Yasutoshi 1990), and empirical design recommendations (ANSI 1995, CSA 1984, TPI 1987). Certainly, the buckling of MPC joints was not rigorously addressed by research institutes and organizations, and efforts to improve design procedures for MPC joints are very limited. The metal plate industry has already expressed its concerns and called for research to validate current practice methods (Grossthanner 1993).

In MPC joints, confined triangular areas of the metal connectors continue freely beyond the surfaces of wood members and lack adequate lateral supports. These areas are fully responsible and dominate the buckling behavior of MPC joints under excessive joint forces. The structural behavior of MPC joints is very complex because of the extended number of related engineering variables (ASCE 1996). Several factors contribute to the engineering complexity of this phenomenon.

First, national and international standard experimental methods do not incorporate buckling of MPC joints. Until today, research efforts have not described a testing procedure for solving this dilemma. The principles established by the Truss Plate Institute (ANSI 1995), the Canadian Standards Association (1980), and the International Organization for Standardization (1990) are well recognized in global practice. However, non of these standards simulate MPC joints with un-braced areas.

Secondly, current buckling-related recommendations are empirically determined (Grossthanner 1993). The American National Standard Institute (ANSI 1995) recommends maximum unsupported plate areas between 1.0 to 1.5 in^2 for resisting shear stresses, though relevant experimental programs have considered much larger areas (Yasutoshi *et al.* 1990). In addition, the empirical values under chord shears published in the Metal Plate connected Wood Truss Handbook (1997) appear unusual. The Canadian Standards Association (1984) even considers connectors to be ineffective in transferring interaction wood-connector forces at the joints.

Thirdly, current standards and previous studies have recognized the failure modes of MPC joints as pure tension, pure shear, and lateral or withdrawal resistance. The buckling phenomenon on the other hand, has not received any exhaustive attention and is surrounded with high subjectivity. For example, in single web-to-chord joints, chord compressive and shear-compression forces are regarded as the sole cause of buckling (Yasutoshi *et al.* 1990). In double web-to-chord joints with one web in tension and the other in compression, the joint is considered less likely to buckle under compressive forces than the single web-to-chord compression joints. The analytic data established in this study has discovered other critical facets that may contribute to the answer of many unanswered questions in this field.

As of today and despite all of the published data, a comprehensive analytic study capable of determining the primary structural buckling characteristics of MPC joints hasn't been developed. If instituted, it would become a valuable instrument in the process of engineering design of MPC joints, amplifies confidence in truss performance and new engineering ideas, such as the square-end MPC joints. This study meets this challenge to determine the governing factors that constitute the buckling of metal connectors and their effects on the structural response of joints.

This study has evolved through two phases. First, MPC joints have been represented by single web-to-chord MPC joints, and used to investigate the cause of buckling. This phase has covered the effects of size of gaps, shear lengths, plate gauges, and wood-connector interaction forces on critical buckling loads. The idea of solid equivalent connectors was also introduced for daily practice. Obtained numeric results were compared with and supported by available experimental data. Secondly, multi-area joints were used to investigate the failure modes, progressive disintegration of joints, and for a parametric analysis.

2. Equivalent metal connectors

Truss metal connectors are commonly 20 gauge, 18 gauge, and 16 gauge steel plates manufactured in three predominant configurations. Alignment, spacing, orientation, size, and shape of slots in a connector are but a few diverse geometric details. These details present great obstacles in analytic and experimental investigations of MPC joints. Though the finite element method (FEM) is capable of investigating complex connectors, the search for a unified, adaptable and simple approach was essential for practical applications. The significance of this new thought is that advanced engineering problems that have usually been beyond the ability of the average practitioner should then be comprehended and easily solved.

This paper institutes a fresh thought to unify and simplify the engineering approach to MPC joints. Accordingly, equipollent solid (un-punched) plates are developed to analytically alternate with the real complex punched connectors. This equivalency is a genuine key feature that has great potential and greatly restores the confidence of practitioners in dealing with this engineering dilemma. This is an adaptable approach that covers variou slot patterns and plate gauges.

Available theories (Hussein 1989, Lucas 1970, Szilard 1973) transform a continuum to an equivalent grid framework of line elements. They have been well established and validated over the years, employed and confirmed to be very reliable. This paper suggests that these techniques be carried out on metal connectors but in the reverse order of their customary procedures. One should first idealize a specified connector as a grid framework, and thereupon determine the characteristics of the equivalent solid continuum, primarily the effective gauge, using the available mathematical relationships. In this way, variables such as material properties and geometric details of connectors can mathematically be dealt with according to choices defined by users.

The concept of "effectiveness resistance ratio" endorsed by the American National Standard Institute (1995) and the American Society for Testing and Materials (1982); or the "correction factor" by the Canadian Standards Association (1980) is another sound approach. Theses references define the "effectiveness resistance ratio" as the ratio of the ultimate strength of the metal connector to the ultimate strength of the matched solid metal control specimen. They also define "correction factor" as the ratio of the minimum ultimate strength of the specified plate material to the actual ultimate strength of the plate material. These ratios (punched-to-unpunched) are used to calculate the design strengths. This paper further develops this international standard practice. To find an equipollent connector, interchangeable plates must have identical deformations and strengths, planar dimensions (width and length), and material properties. The transformation is accomplished by changing only the gauge of specified real connectors, and thus the effective gauge is obtained.

From the point of view of engineering mechanics, the authors recommend that the equivalency be exercised along various orientations of connectors using the minimum net cross-section (most critical) of uncoated plates. The most effective gauge or, probably an average value should afterward be used. Based upon other non-mechanics justification, designers may prefer adopting other gauges than computed ones. This procedure is sufficient to solve for an effective gauge, but performing ASTM (1982) or CSA (1980) standard experiments on real and transformed connectors can, if necessary, verify the legitimacy and the accuracy of the design gauges.

From a mathematical point of view, the standards' concept of effectiveness ratio is easier than the framework method because of the relatively large number of structural properties involved in the latter. Both procedures however provide adequate methodologies for all practical purposes.

3. Single web-to-chord MPC joints

Because of the inherent and diverse complexity of the geometric details of joints, materials, loads, and boundary conditions, it was vital to begin our analysis with a basic prototype, upon which the rest of investigation can evolve. Fig. 1 shows a model that represents single web-to-chord MPC joints. It comprises central wood blocks sandwiched between two metal plates. The unsupported area of each connector is characterized by its surface area a, the gap x, and the shear length d. This simple joint provides an opportunity to focus on the fundamentals of the buckling problem under consideration. This is a nucleus for all other larger joints, and its analysis contributes to the understanding of the buckling behavior of real MPC joints.

The analytic strategy in this phase is based upon the development of finite element (FE) models that closely simulate MPC joints' buckling behavior. Several researchers have developed specific computer codes for MPC trusses such as Crovella and Gebremedhin (1980), Gebremedhin and Crovella (1990). These codes are totally restricted by self-developed specific analytic techniques.

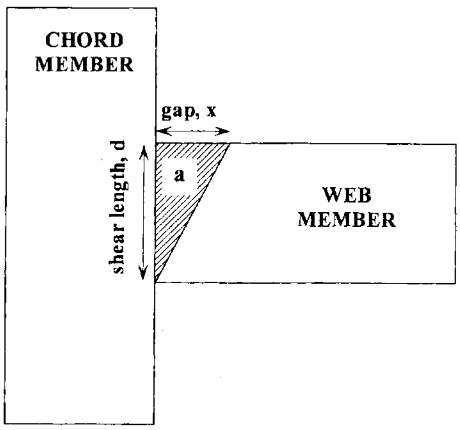


Fig. 1 Single web-to-chord MPC joint

Our search was directed towards general-purpose commercial FE packages; including ANSYS, COSMOS and STAAD, from which a sophisticated tool capable of performing the analysis of diverse and complex structural problems has been selected.

The FE modeling in this paper has used Southern Pine for wood members and ASTM A 446 steel for metal connectors. Physical properties of these materials, such as the stiffness, were obtained from national standards. Species, grade, and other physical properties of the constituent materials were not regarded as variables in this basic study.

Fig. 2(a) shows a three-dimensional FE model for the single web-to-chord joint in Fig. 1. The model includes 660 solid-elements, 3580 nodes, and 10264 degrees-of-freedom. With the aid of this large model and exhaustive computer simulations, only the metal connectors were subsequently idealized using thin shell elements. The latter FE model is much simpler, yields fast results, and exhibits analogous structural response to the 3D model. Fig. 2(b) shows a typical buckling mode due to chord shears. The shell-based idealization was adopted throughout this phase under six autonomous loads applied on the joints: chord shear, web shear, axial web compression, web shear-compression, axial web tension, and web shear-tension.

As expected, non of the axial web tension and shear-tension loads have caused buckling in any of the solved models, and there was about 10% difference between critical loads caused by the axial web compression and shear-compression values. Figs. 3 to 6 show the effects of unsupported area and plate thickness on critical loads.

Fig. 3 compares the critical web axial compression and chord shear loads. It is noted that for unsupported areas from 0.5 in^2 to 2.0 in^2, critical shears are smaller than the corresponding compression values. The compression loads drop off nonlinearly by 80% as compared to 70% in shears. Within the same range, and regardless of loads, the shear length governs the buckling behavior. Under chord shears, the nonlinear decrease and the linear increase in buckling loads are 70% and 27%, respectively. In areas with gab length (x) equal to the shear length (d), the buckling response was not affected by these variables (x, d and a). Beyond this specific case, the buckling behavior reverses among loads as shown. Fig. 3 also shows the progressive failure modes. For areas smaller than 2.0 in^2, shear modes occur before compression ones. For larger areas than 2.0 in^2, compression modes occur within the chord shear buckling with constant x and the chord shear buckling with constant d. The figure finally reveals that the ratio of gap/shear length must be larger

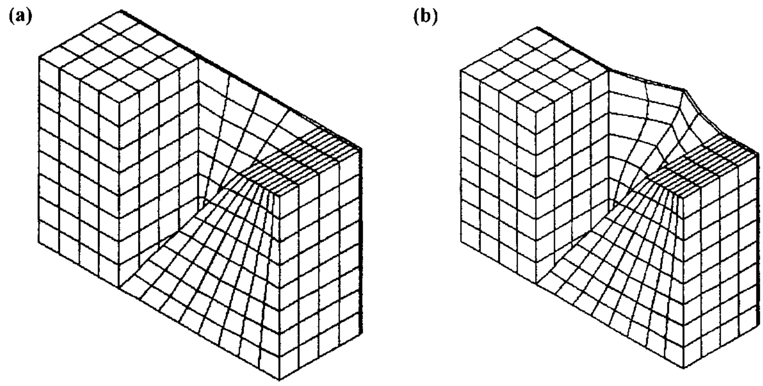


Fig. 2 (a) A three-dimensional finite element model for the single web-to-chord MPC joints, (b) A typical first buckling mode of single web-to-chord MPC joints due to chord shear

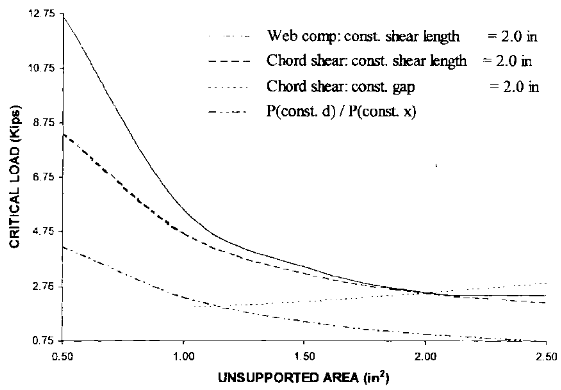


Fig. 3 Effects of gap and shear length on critical buckling loads of single web-to-chord MPC joints (thickness=0.030 in)

than one for length-based failures to precede gap-based modes. Nevertheless, the chord shears dominate the buckling throughout all sizes.

Fig. 4 shows the effects of the gauge on buckling loads of the standard area of 1.5 in². As seen, critical loads are proportional to thickness. This agrees with traditional buckling theories of linear structural components, where loads increase with increasing the cross-sectional property such as cross-section area (Timoshenko 1936). The figure in conjunction with Fig. 3 also reveals an important outcome for practical designs. For a given triangular area a, the buckling strength of MPC joints improves enormously by keeping the ratio of gap/shear length below 3/4 (this ratio is the reciprocal of 1.33 obtained from the last legend in Fig. 3). To elaborate, the gain in the buckling strength is about 50% by changing this ratio from 4/3 to 3/4. Also, the compression and shear buckling loads differ insignificantly.

Fig. 5 depicts the importance of small triangular areas for improving the buckling response of MPC joints. In agreement with current standard empirical recommendations, smaller areas improve sharply the buckling strengths of joints. This is of a great industrial benefit. When the area a, shear length d, and gap x have been specified, then larger plate gauges will enhance the joint capacity. On

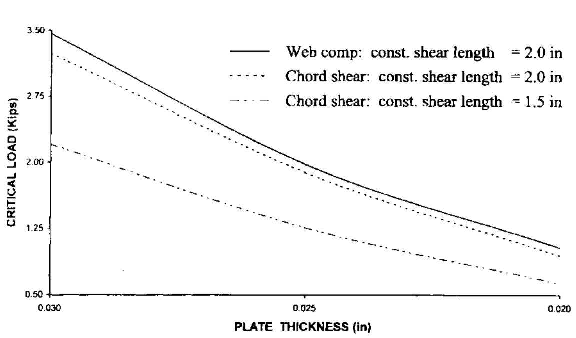


Fig. 4 Comparison between chord shear and web compression critical loads of single web-to-chord joints (unsupported area=1.5 in²)

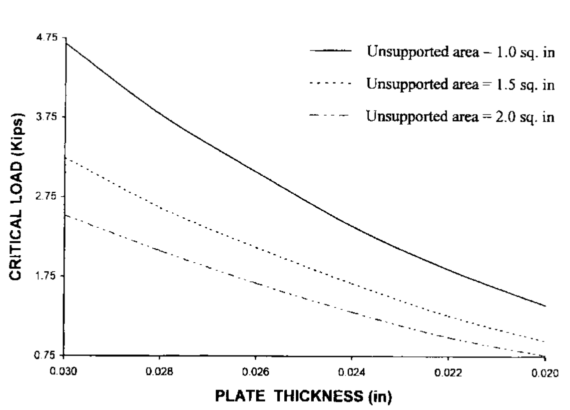


Fig. 5 Effects of connector thickness on critical chord shear loads of single web-to-chord MPC joints

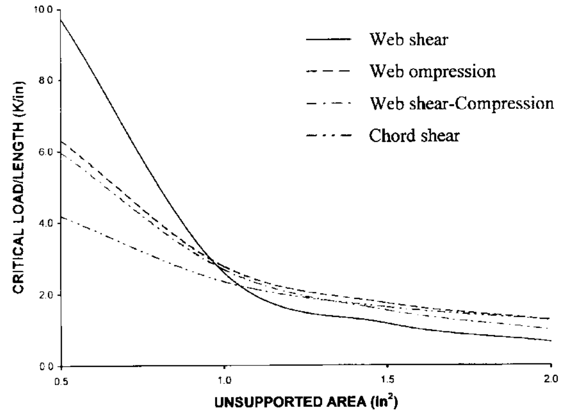


Fig. 6 Comparison between critical loads caused by various autonomous loads on single web-to-chord MPC joints (thickness=0.030 in)

the other hand, for a prescribed gauge, the designer should minimize the size of areas. This is clearly illustrated by observing the difference between the lower two curves as compared with the upper ones.

Fig. 6 compares the critical loads created by all applied autonomous loads. As stated earlier, web axial tension and shear-tension loads do not cause buckling. The load per unit length of relevant edge of the triangular area is used for consistency in our comparison. The figure shows that for areas $a < 1.0$ in^2, the chord shear is the most critical. For areas larger than 1.0 in^2, the differences in buckling loads may be ignored practically, thus the effectiveness of loads becomes meaningless. The slight difference between the responses under web compression and shear-compression is evident. Fig. 6 can also be used to characterize the effective shear length. For practical applications, shear areas of MPC joints should not have long edges unless the significant drop off in the buckling loads are considered.

The results shown in Figs. 3 to 6 may institute the shear-based design approach and modernize the current compression-based practice. Almost all-available experimental data (Yasutoshi and Takemura 1990) were obtained under compression loads. Our results however indicate that chord and web shears dominate the buckling of single web-to-chord MPC joints. In addition, the only reference that refers to chord shear lacks technical data for clarity (Wood Truss Council of America 1997). Contrary to all published works, it indicates that critical shears are linearly proportional to the sizes of the area a. This is a divergence from the inherent buckling characteristics (Timoshenko 1936). Further, Figs. 3 to 6 also provide an answer to what constitutes effective unsupported area. This issue was addressed empirically in current standards (American National Standard Institute 1995, Wood Truss Council of America 1997). It is observed that larger areas may be used as the connector gauge increases. An area of 1.0 in^2 would be conservative regardless of gauges, whereas caution is advised in using an area larger than 2.0 in^2. In this regard, designers must not underestimate the role of the type of applied loads. As depicted in our results, geometrical details, gauge, size of area, and type of loads concurrently determine the buckling performance of metal connectors.

Experimental validation of our approach was a necessary step before any further advancement towards larger and more complex MPC joints that have no experimental support. Wood Truss

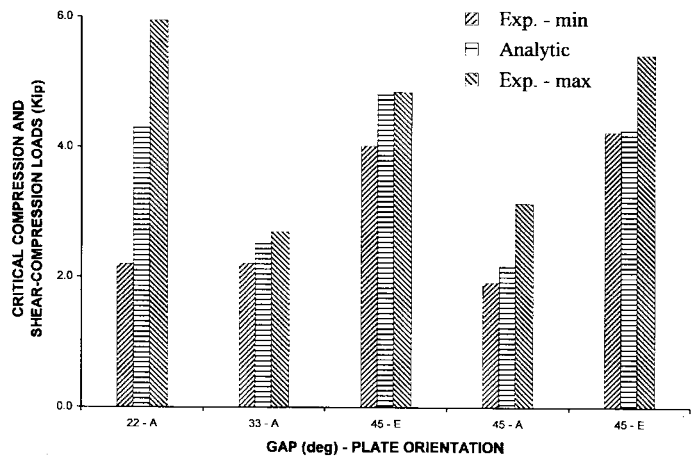


Fig. 7 Comparison between experiment and analytic values obtained for single web-to-chord MPC joints

Council of America (1997) has provided the only available experimental data on buckling of single triangular areas under chord shears. The overall pattern of the published experimental data (Wood Truss council of America 1997) exclusively agrees with the above results. Specific one-to-one numeric comparisons are not possible because of insufficient technical details of the tested joints. However, it was confirmed experimentally that critical shears decrease by increasing size of gaps, and increases by increasing the shear length. Also, experiments have illustrated that the difference between these effects is not constant. Further, the reversal in effects of gap and shear length, Fig. 3, has been observed in the experimental data.

Another related experimental investigation has been reported in Yasutoshi and Takemura (1990). Various single web-to-chord MPC joints were prepared using Southern Pine, tested, and their buckling loads obtained. In our study, some of these specimens have been analyzed according to our analytic approach. Fig. 7 compares both results. It is seen that the analytic results fall favorably within minimum and maximum values, and compare well with experimental data. Many other agreements have been observed. For example, deformations due to buckling were found to be more pronounced on the open side of joints. All aspects of the pattern of relative strengths observed experimentally (Yasutoshi and Takemura 1990) have also been confirmed in our analysis.

Finally, the evident agreements between this phase and the published experimental data (Wood Truss Council of America 1997, Yasutoshi and Takemura 1990) verify the analytic approach and establish the necessary confidence in its reliability as a foundation for the multi-area MPC joints presented next.

3.1. Multi-area MPC joints

The first phase in this study has furnished a solid launch pad from which the analysis has taken-off towards multi-area MPC joints, also known as double web-to-chord joints. These joints incorporate a large number of structural complex variables (American Society of Civil Engineers 1996), hence it was necessary to adopt a genuine joint for our investigation. The joint shown in Fig. 8 may exemplify many engineered MPC connections. It shares common details with bearing and interior web-to-chord joints. It is also very suitable for custom-cut and square-end wood members as well as

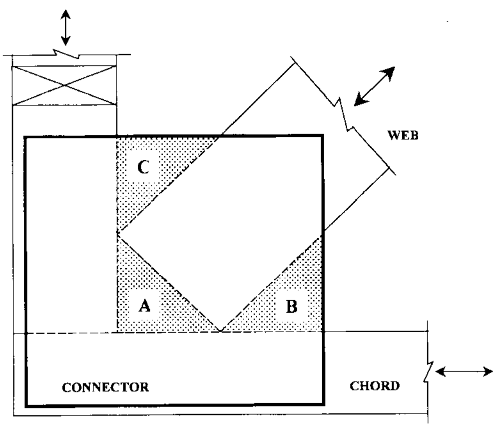


Fig. 8 A multi-area MPC joint

for joints under web and chord loads.

The joint consists of three wood members connected using metal connectors. Three equal metal triangular areas, A, B and C are enclosed between the wood members. Based upon our experience with the previous 3D FE models of Figs. 2 and obtained results, the buckling analysis was done on only the connectors while preserving the wood-plate interactions. The FE models have thus utilized thin shell elements with the appropriate set of global interacting boundaries. This phase was divided into two tasks. First, the distribution of the wood-connector interaction forces was determined. Second, FEM was used to ascertain the buckling performance.

3.2. Distribution of wood-connector interface forces

Reliable design of MPC joints in trusses requires an understanding of the structural effects of various factors such as the wood-connector interactions. Without such an insight, any joint engineering would be naturally incomplete.

Generally, the forces in wood members are difficult to measure (Yasutoshi and Takemura 1990) and current standard practices (ANSI/TPI 1-1995, ASTM E 489-81. 1982, ASTM E 767-80 1982, ASCE 1996, CSA 1984, CSA 1980) do not specify or recognize the contribution of the connector-wood interface forces in the design. However, the literature provides some results from previous studies to analyze the interaction distribution on a per tooth basis. Crovella and Gebremedhin (1990) have used analytical techniques to model the behavior of toothed metal-plate joints backed by laboratory testing for verification. In that study, they used beams (tooth) on elastic foundation (wood) in conjunction with the finite element method to determine the distribution along only the tooth array and to predict joint stiffness.

In this endeavor, a specific FE model was developed for determining the distribution of interaction forces, as shown in Fig. 9(a). In this specific model, The wood member was 2 by 4, Southern Pine with a modulus of elasticity (MOE) of 1.6×10^6 psi. The MOE of the steel plates was 25.5×10^6 psi. Wood and metal connectors of the MPC joints were modeled using 3D structural solid elements. Two entirely independent sets of nodes along the interfaces between the central and side members have defined wood and plate elements. The mesh of elements has represented the entire geometry of the joint, although only half of the joint could have been used by virtue of symmetry.

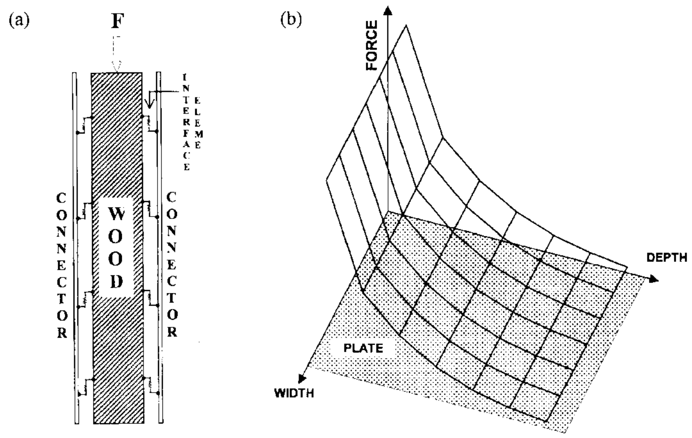


Fig. 9 (a) Finite element model for distribution of wood-connector interaction forces, (b) Typical distribution of wood-connector interaction forces

The interaction between wood members and metal connectors in this FE model was represented by the 3D linear spring element. This element assumed the role of the tooth on the connectors. This feature is probably what makes this FE simulation as realistic as it can be. The spring stiffness was determined from the results published in Crovella and Gebremedhin (1990). Although efforts were made to preserve characteristics of actual joints in the modeling process, only interaction-related aspects were considered. For example, tooth withdrawal between metal teeth and wood were not considered, though it could easily have been incorporated.

Fig. 9(b) illustrates the general pattern that depicts the distribution of interaction forces between wood and metal connectors in MPC joints. The figure does not show the insignificant forces near free edges of plates where they gradually diminish. It is noted that interaction shears extend constantly across practically the full width of metal connectors. They also nonlinearly decrease gradually towards the free end of the wood member. This finding agrees with the concept of "development length" in concrete members, where a sufficient length is needed for each steel bar to fully develop the interaction bond force between it and surrounding concrete.

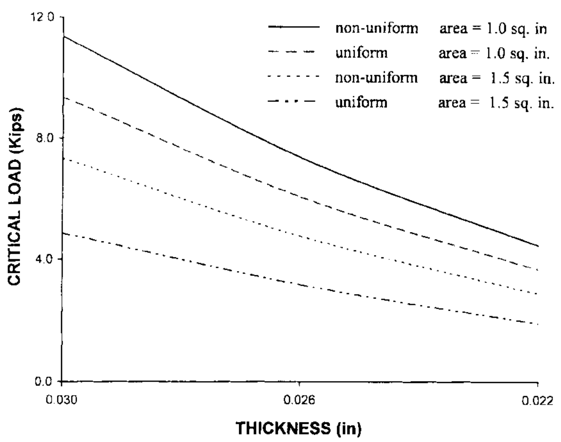


Fig. 10 Effects of wood-connector force distributions on critical web compression loads (area=A)

Non-uniform forces would certainly encumber the daily engineering of MPC joints. We thought that simpler yet reliable distributions should be found for practical purposes. At that point, autonomous non-uniform and uniform wood-connector interaction forces have been applied as joint loads in the FE models. Fig. 10 shows the effects of two distributions on the buckling response of the joint in Fig. 8. It is seen that non-uniform distributions exhibit larger buckling loads than uniform distributions for all gauges and triangular areas. It is also noted that the difference between corresponding critical loads is independent of the thickness, and is proportional to the size of the area. These are 18% and 34% for areas of 1.0 in^2 and 1.5 in^2, respectively.

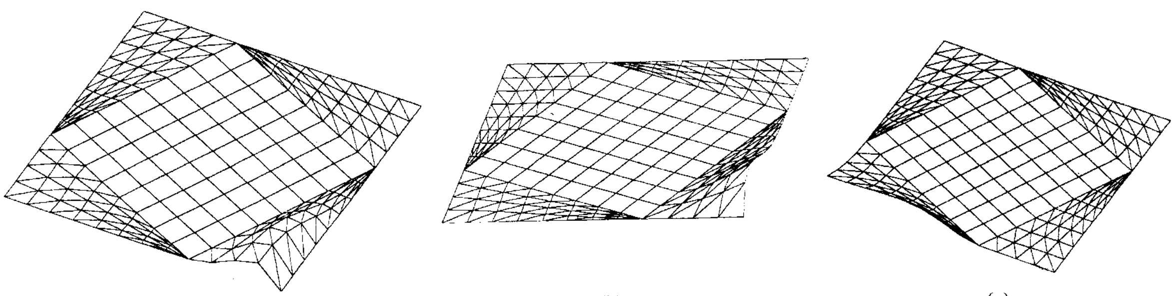
For everyday applications, uniform distributions should be sufficient and reasonably conservative for practical buckling analysis. Related to this, the merit and cost-effectiveness of precise interaction distributions would be a controversial issue among engineers. One may accept this fact by taking into account the natural characteristics of materials in light-frame constructions including wood, the diverse geometric details of commercial MPC joints, and the level of uncertainty in estimating design variables in this field. It is this believed that efforts for this purpose would render insignificant practical contributions, but may have some scientific interests.

3.3. Buckling performance of multi-area MPC joints

Truss joints are mainly subjected to member forces. In this study, autonomous web axial compression and chord shear loads were used. It should be noted that the web force compresses area A and shears areas B and C. In double web-to-chord connections with one web in tension and the other in compression, only compression is responsible for buckling. Tension loads have been used throughout but caused no buckling.

Figs. 11(a, b, c) show typical fundamental buckling modes of areas A, B, and C. As expected and observed in experiments by others (Yasutoshi 1990), deformations are more pronounced on the open side of all MPC joints. Also, metal connectors experience excessive deformations in higher buckling modes. Loads required to sustain these higher modes are much greater than the fundamental values. This post-buckling behavior has also been observed experimentally (Yasutoshi 1990). However, this reserve strength should not be relied upon since joints loose their structural integrity once they buckle.

Figs. 12 and 13 compare the effects of web compression and chord shear critical loads. Generally, it is noted that chord shears create and dominate the buckling phenomenon. This finding is not well known in the engineering design of MPCJ where only web compressions are accounted for buckling. Also, critical compressions and shears increase as the size of the unsupported area



(a) (b) (c)

Fig. 11 Typical first buckling mode of areas A, B, and C in multi-area MPC joints

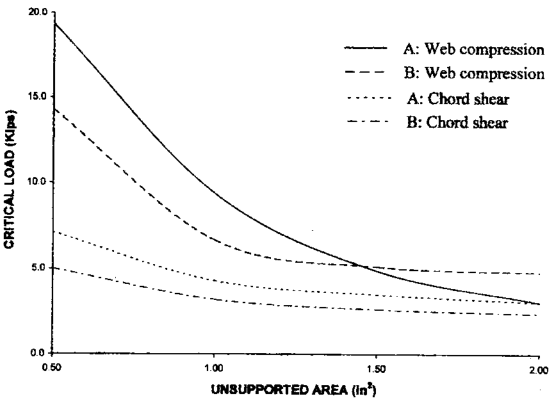


Fig. 12 Effects of unsupported areas on critical chord shears and web compressions (thickness= 0.030 in)

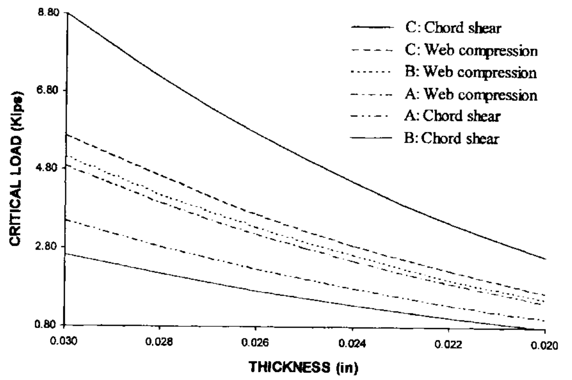


Fig. 13 Comparison between critical web compression and chord shear loads of areas A, B and C (area=1.5 in^2)

decreases, and the connector gauge increases. The curves portray the traditional buckling relations for other structural components (Timoshenko 1936).

Fig. 12 shows that area B always buckles before A under chord shears. This occurs only under web compressions for areas up to about 1.5 in^2, afterwards this sequence reverses. For area B, sizes of $a < 1.0$ in^2 have noticeable effects on critical loads, and have less changes for $a > 1.0$ in^2. To elaborate, in a range of a, from 0.5 in^2 to 1.0 in^2, the drop-off is 54% and 36% for critical compressions and shears respectively; 28% (compression) and 26% (shear) from $a = 1.0$ in^2 to $a =$ 2.0 in^2. For area A, the critical compressions rapidly diminish continuously for $a > 1$ in^2.

Fig. 13 strengthens our aforementioned findings that chord shears dominate the buckling of MPC joints. The plate gauge has no effects on the change in the critical loads. In all cases, the loads decrease by 70% as the thickness decreases from 0.03 in to 0.02 in. Based upon Figs. 12 and 13, and from a practical point of view, un-braced sizes less than 1.0 in^2 would generally improve the buckling performance of MPC joints, whereas larger areas significantly weakens area A, hence the entire joint. The critical values abruptly increase as the size, a, decreases. This is an indication that design values obtained from current standard tests, where a is very small, or other studies, where a is very large, should not be used for buckling loads. A combination such as large area and small thickness would of course be unwise design, whereas small areas and large gauges would be a waste of resources. In addition, the large increase in post-buckling loads among areas A, B and C in metal connectors indicates that MPC joints regain strength after the fundamental modes. However, this process is accompanied with sever distortions of the plate. A joint may be considered as having lost its structural integrity once the fundamental mode appeared in the system.

In this study and under buckling of single unsupported areas, the effects of size of gaps and shear lengths have been covered. These variables also affect the buckling of multi-area MPC joints. However, gaps and shear lengths have been concealed in the values of unsupported areas, a. Under web compressions, the shear lengths of areas B and C, and compression side of area A are all proportional to the values of a. In these cases, the perpendicular dimension of each area is what one may consider as gaps. Under chord shears, only areas A and B are directly sheared. Accordingly, our previous discussions on the effects of the unsupported areas on the buckling of connectors signify the gaps and shear lengths.

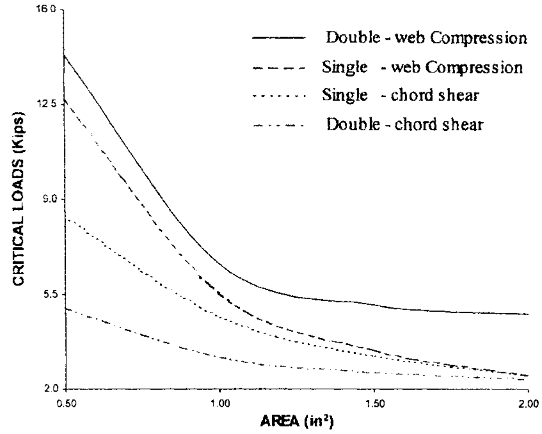


Fig. 14 Comparison between buckling of single and double MPC joints (area B and thickness=0.030 in)

Finally, the engineering analysis of single web-to-chord MPC joints should not be interpreted for multi-area joints, even where one web is in tension and the other in compression. Fig. 14 compares the buckling performance of the MPC joints in Figs. 1 and 8. It is evident that multi-area joints are less likely to buckle under web compression than single joints. The converse is true under chord shears. For area A, the joints exhibit this pattern up to sizes of about 1.25 in^2, after which only the chord shear buckling reverses.

4. Design curves for the buckling load of MPC joints

The current literature in this area has no ready-to-use formulas or curves for the daily engineering routine. This paper also showed that the analytic investigation of the buckling problem of MPC joints is characterized by advanced and complex computer modeling. This may not be easily applied

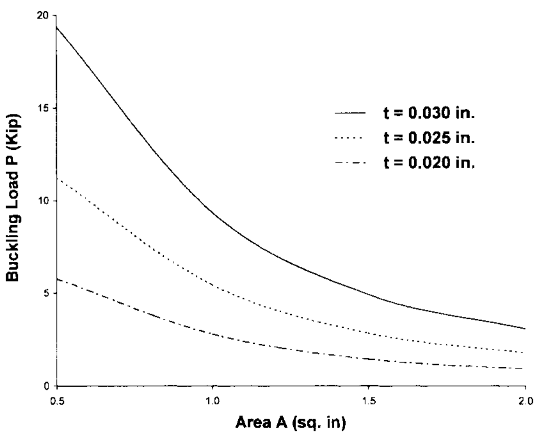


Fig. 15 Design loads for MPC-joints under web compression

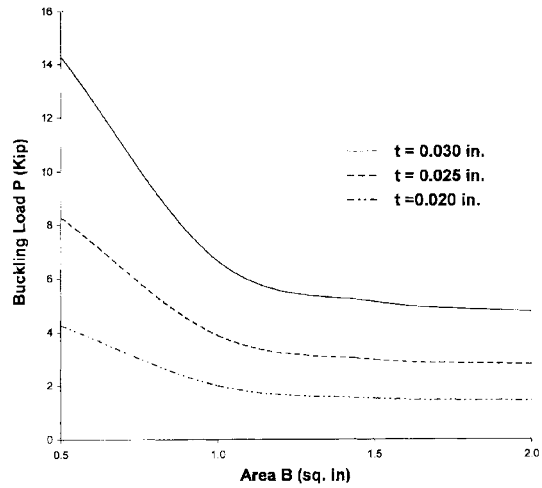


Fig. 16 Design loads for MPC-joints under web compression

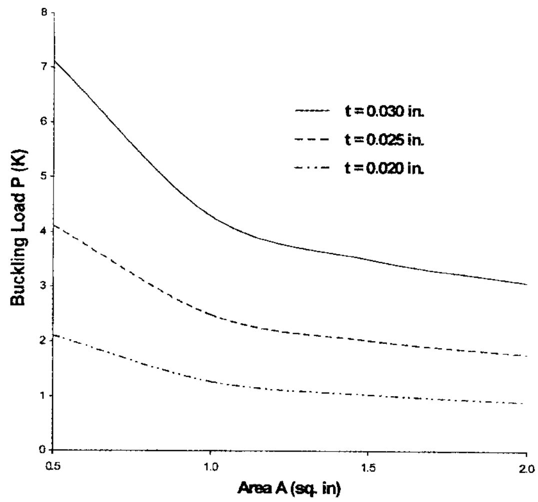


Fig. 17 Design loads for MPC-joints under chord shear

by ordinary practitioners and could limit the application of our discovery. To facilitate the application of our advanced analysis and reduce the amount of work involved in the buckling analysis MPC joints; graphical design aids are thus devised from which the critical load can be determined.

The numeric data previously generated in this paper for the buckling of multi-area MPC joints were comprehensively integrated and consequently the curves in Figs. 15 to 18 were prepared. Figs. 15 and 16 show the buckling loads of areas A and B of multi-area MPC joints under web compression. Figs. 17 and 18 show the buckling loads of areas A and B in joints under chord shear.

In a typical design routine, one may design the joints for static loads according to the preferred standards. The adequacy of this design for buckling loads could then be checked. Using the values of areas A and B from the static design in conjunction with Figs. 15 to 18, the buckling loads can be found and be compared with the applied forces. This comparison will determine the suitability of the designed plates. As in almost all other engineering designs, Figs. 15 and 18 are just tools for practical purpose. It is up to the practitioners to decide on how to use such tools in an on-going procedure.

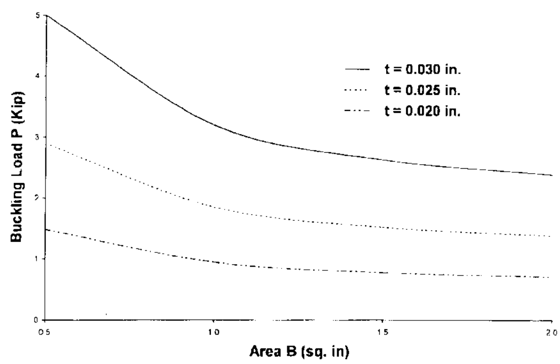


Fig. 18 Design loads for MPC-joints under chord shear

5. Conclusions

The buckling of single and multi web-to-chord un-braced areas metal-plate-connected joints has been investigated in this paper. Upon the results obtained, the following conclusions can be extracted:

1. Chord shears cause and dominate the buckling of MPC joints
2. The shear length has more pronounced effects than the size of gaps.
3. Large gauges and small un-braced areas improve the buckling response.
4. Non-uniform distributions of wood-connector forces exhibit larger buckling loads than uniform distributions.
5. Multi-area MPC joints should not be simplified as single web-to-chord joints, even where one web is in tension and the other in compression.
6. The available experimental data obtained by others advocate the analytic approach presented in this report. This has increased the confidence in its usefulness for practical applications. Accordingly, MPC joints with unsupported areas should be allowed in light-frame trusses provided their characteristics are considered.

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