System behaviour of wood truss assemblies

Rakesh Gupta
Oregon State University, USA

Summary
Wood truss assemblies are widely used in light frame construction all over the world. Although the volume of literature on single trusses and joints is relatively huge, the system behaviour of truss assemblies has received only limited attention. The main approach to take into account the system behaviour of an assembly is to use a system factor in the design of single trusses. However, technical advances now make it possible to analyse and design these complex assemblies as systems using three-dimensional structural analysis programs, thereby including the system effect directly. This paper presents an overview of previous research on system behaviour of truss assemblies and recommends a system design approach to designing truss assemblies.

Key words: load sharing; system effects; repetitive-member behaviour; wood structural systems; load distribution; wood truss assembly design

Introduction
Wood truss systems, both for roofs and floors, offer one of the best ways to resist vertical loads in many structures. They have been used as one of the main structural components in residential and light commercial structures in many parts of the world. They have a long history of good performance in most structures. Lately, however, configurations of roof truss assemblies are becoming increasingly complex (Fig. 1). Therefore, it is important to study the system behaviour of these complex truss assemblies to understand how the loads are resisted, transferred, and distributed in order to design them safely and economically.

Most wood trusses are pre-engineered components fabricated from dimension lumber and connected with metal connector plates. The pre-fabricated wood trusses can be erected faster and with less skilled labor than necessary for other types of roofs and are therefore more economical to produce and use. In addition, light frame wood trusses offer outstanding benefits from both an architectural and engineering perspective. Thus, light frame wood trusses have become increasingly popular in residential construction in North America and other parts of the world.

To construct a light frame wood truss assembly, the trusses are erected and spaced typically 0.61 m on centre. The sheathing (plywood, oriented strand board or other structural panels) is then nailed to the truss top chord members. Not only is the roof sheathing used to cover the facility and carry imposed loads, but it also serves as a load-distributing element among trusses in the assembly. Moreover, there are other structural components, such as purlins and bracing, connecting the trusses together. These construction characteristics make the truss assembly act as a system.

This paper discusses only load sharing in sheathed truss assemblies (i.e. truss plus structural wood panel-type sheathing). Load sharing in other types of assemblies (e.g. heavy tiles over battens spanning between trusses used in the UK, parts of Australia and New Zealand) will differ, depending upon the type of lateral elements (sheathing or battens) spanning between trusses, and is not discussed in this paper.

It is well known that there is load sharing among trusses within a truss assembly. The previous research on load sharing in truss assemblies is based on only one fact, i.e. statistical variation in member stiffness properties. This is generally true for regularly shaped assemblies containing identical trusses. However,
most real truss assemblies are much more complex (Fig. 1) having irregular shapes and contain tens of different types of trusses (including stiff girder trusses, multi-ply trusses, very stiff gable end trusses, etc.). This author believes that the behaviour of real truss assemblies is effected much more by system geometry and the types of trusses used in the assemblies than by the variation in the member properties. Several studied at Oregon State University have shown this to be true and are discussed later in the paper. However, this may or may not be true for floors containing trusses, depending on how regular or irregular the floor geometry is.

Still, truss assemblies have been traditionally analysed and designed on a single-truss basis, which assumes that each truss in the assembly carries loads based on its tributary area. This overall approach is known as the conventional design procedure (CDP). CDP is relatively simple and is assumed to be conservative. Although CDP for truss assemblies has a good track record, it does not take into account all system effects encountered in realistic assemblies, which may or may not be conservative in actual assembly behaviour.

A system design procedure (SDP), proposed by researchers at Oregon State University[1], that analyzes assemblies as a system (three-dimensional analysis) could consider the system effects directly. SDP can provide a more realistic description of the behaviour of trusses in an assembly and may lead to: (a) improved truss system design by including system behaviour directly; (b) increased safety through improved analysis; and (c) potential reduction in truss fabrication costs. Technological advances and high-speed computers now make three-dimensional structural analysis of real truss assemblies that include actual geometry and construction conditions possible. Additionally, in light of emerging performance-based design concepts, a system approach to analysing and designing wood structures would be required[2]. Mtenga[3] also suggested several benefits of considering system effects in wood truss assembly design.

This paper summarizes the previous research on the system behavior of wood truss assemblies. It also highlights the recent research efforts at Oregon State University to understand the system behavior of realistic (actual) truss assemblies and proposes...
a practical solution to directly include all system effects in the design of these assemblies.

**Behavior of truss assemblies**

A vast amount of literature has been accumulated on single trusses and metal-plate-connected (MPC) joints,[4] but the system behaviour of wood truss assemblies has been studied by only a few researchers. In the past few decades, a number of investigators have studied the structural behaviour of wood truss assemblies, using both experimental testing and computer modelling. Experimental testing of truss assemblies is expensive and therefore only simple truss assemblies have been tested. Such tests do provide convincing results; however, these results are mostly applicable to the system tested. Many researchers have relied on computer modelling, especially for parametric studies; however, most actual assemblies are much more complex than the simple assemblies used to verify these models. Therefore, to predict the structural behaviour of actual truss assemblies, actual truss assemblies should be used either for testing or for modelling.

**Full-scale testing**

Research on full-scale testing of wood truss assemblies has been sporadic over the last several decades. A few studies have been conducted on different types of assemblies, mainly highlighting load sharing among various components of an assembly. Wolfe & McCarthy[5] provided an excellent review of the literature on full-scale testing of roof assemblies conducted until the early 1980s. Their conclusion was that most of the studies suggested load sharing and assembly interaction, but failed to quantify it. In two major studies, Wolfe & McCarthy[5] and Wolfe & LaBissoniere[6] tested four full-scale roof systems to improve design methods for light frame roof systems. Their goal was to use the results of the tests in the development and evaluation of analytical models capable of predicting roof system stiffness and load capacity.

In the first study, Wolfe & McCarthy[5] investigated the structural performance of light frame roof assemblies with high truss stiffness variability by testing full-scale, nine-truss assemblies. All nine trusses in the assembly were Fink trusses with 0.61 m spacing and 8.5 m span, and two roof assemblies were tested to provide the data for the evaluation of assembly models. They pointed out that: (1) stiffer trusses carry a greater share of the load in the assembly; (2) the load–deflection behaviour in the assembly was approximately linear; (3) in the range of design load, the average truss deflection within the assembly was 50% lower than that of an individual truss outside the assembly; and (4) the average failure loads increased 20% inside the assembly, compared with individual trusses outside the assembly. They finally concluded that component interactions could improve the structural system performance of light frame roof truss assembly by increasing stiffness and strength.

In the second study, Wolfe & LaBissoniere[6] tested two more nine-truss assemblies, and showed that the interactions within assemblies controlled the deflections of members because repetitive-member roof assemblies behaved like parallel systems. One other significant conclusion was that when the top chord of an individual truss is loaded to the design load, the loaded truss in the assembly carries only 30–60% of the applied load and distributes 70–40% of the load to the adjacent unloaded trusses through the plywood sheathing.

In both studies, all four nine-truss assemblies had nine identical and symmetrical trusses and no gable-end trusses. Actual truss assemblies generally have many types of trusses (symmetrical and asymmetrical) including gable-end trusses, all of which can significantly influence the load distribution in the assembly. Therefore, the results of the experimental tests of nine-truss assemblies may not be directly applicable to more complex, actual truss assemblies.

LaFave & Itani[7] also studied the load-sharing capacity of nine-truss roof assemblies. They showed that truss stiffness nearly doubles and becomes less variable due to load sharing in a sheathed assembly. Their test results also showed that the very stiff gable-end trusses greatly influence load distribution in the assembly. Additionally, they concluded that the distribution of load throughout the roof assembly did not change with the level of loading. This suggests that a linear analysis can be used to predict load distribution in truss assemblies. However, roof truss assemblies used in residential buildings are usually much more complex than what has been tested (i.e. nine-truss assemblies). Thus, the tested assemblies may not include all of the system effects that may exist in more complex roof geometries.

Percival & Comus[8] investigated the load distribution in a full-scale hip-roof system. The load tests showed that the hip girder in a terminal hip system carries much lower loads than generally assumed in the design of the girders. The folded plate action of the sheathing and three-dimensional interaction of the framing members are the primary reasons for the reduced loads. More recently, Waltz[9] described a series of three-dimensional finite element models used to define load paths and behaviour of a hip roof under uniform vertical loads. The model results were confirmed using test data from full-scale hip roof assembly tests. He showed that ‘many of the load paths utilize members and connections that are not always considered to share the burden.’ It was further concluded that ‘much of the disparity between historical performance and a component-based,
two-dimensional analysis can be attributed to the three-dimensional behavior of the roof that is not considered in the simplifying assumptions.‘ If the SDP is used for analysing and designing assemblies, both of the findings (concerns) of Waltz[9] may not be an issue.

In other related studies, Manui[10] tested a 1/8-scale model of a roof truss assembly to develop structural influence matrices. He concluded that, on average, the model results are representative of the typical distributions of truss end reactions observed from full-scale tests. Diaz & Schiff[11] investigated the influence of out-of-plane wall stiffness on the structural behaviour of wood roof assemblies. Gupta et al.[12] developed small-scale models of MPC wood truss joints and trusses in order to build and test truss assemblies to study their system behaviour.

**Modelling**

Varoglu & Barrett[13] made one of the first attempts to model roof truss systems by developing a structural analysis program for roof systems (SAR) at Forintek Canada Corp. Varoglu[14,15] later used the results of the tests conducted by Wolfe & McCarthy[5] and Wolfe & LaBissoniere[6] to verify the program. He found good agreement (within 5–6%) between the vertical deflection predicted by SAR and experimental results. Larger errors were observed in some trusses due to the interaction between the supporting walls and the side trusses. He finally concluded that system response is significantly better than individual truss performance.

Further verification of SAR was done by Lam & Varoglu[16], who generally found good agreement between experimental and model results, except for a few cases, where the model over-predicted experimental results. Later, Lam[17] used SAR to assess load-sharing behaviour of trusses in roof systems. He used parallel chord trusses with one configuration and evaluated the performance of a single truss inside and outside the roof assembly. He found an average system factor of 1.11–1.31 for tension members and 1.13–1.27 for compression members, using combined dead and snow loads at six locations in Canada. Based on this research, the Canadian Code added a 10% increase in tension, in addition to the 10% increase in compression and 15% increase in bending that already existed in the Canadian wood design code[18]. These increases are applied to the design stresses of individual members.

Most of the other system factors in the Canadian code[18] are based on the work of Foschi[9]. He studied floors and flat roofs in bending under uniformly distributed loads and showed that system factors strongly depend on the probability distribution of member properties and the loading and much less on variables that are considered in the design equation, such as joist depth and joist spacing. The average factor for systems studied was 1.63, with a coefficient of variation of 4.5%. For the more stringent performance requirement of no joist failure, the overall system factor was 1.43. Therefore, all system factors in the Canadian code are 1.4 or less.

Cramer & Wolfe[20] developed a roof-truss system model using the program, ROOFSYS, to study load-sharing effects in light frame wood roof assemblies. In the model, simple hinged connections were used. Additionally, composite action (T-beam action) and two-way action of the sheathing were also included. To represent roof sheathing in the direction perpendicular to the truss span, sheathing was modelled as a single continuous beam on each side of the ridge. The sheathing beam was rigidly connected to each truss. The strong and weak axes of bending of the sheathing beam were perpendicular and parallel to the truss slope, respectively. Results from this study showed that a loaded truss only carried approximately 50% of the load directly applied to it. This indicated that there was significant distribution of load occurring in the roof system.

LaFave & Itani[7] developed an analytical model to estimate the load-sharing effect and load distribution in a wood truss roof system. They modelled a nine-truss assembly with sheathing using three-dimensional frame elements for all members (truss and sheathing) in the assembly. The stiffness of the roof sheathing was either lumped in rows of sheathing elements connecting to the top-chord panel point of the trusses or distributed along the plane of the roof. The properties of the sheathing were based on actual panel widths and effective thickness. They concluded that the model developed could actually predict the distribution of loads obtained in the roof assembly tests. Since only one simple assembly was used to verify the model, its application to a wide range of truss assembly configurations is unknown.

Cramer et al.[21] and Mtenga et al.[22] developed the NARSYS program (Nonlinear Analysis of Roof System) for determining the strength of roof assemblies. The program included linear elastic three-dimensional frame elements to represent the wood truss members, nonlinear springs and rigid links to represent the joint connections, and deep beams to represent the roof sheathing. The sheathing beams were pin-connected to the trusses. The metal-plate load–slip properties followed Foschi’s[23] definitions. The study’s results showed that CDF of repetitive-use members in a roof assembly was conservative. The study also indicated that roof slope and other truss characteristics could cause significant changes in the system effects.

Cramer et al.[24] and Cramer & Kennedy[25] analysed truss assemblies using the Structural Analysis of Wood Frames and Trusses-Repetitive (SAWFTR) program. Translational and rotational springs were used in the model to represent the connections. Connections were assumed to have nonlinear
behaviour, and the truss members were assumed to behave linear-elasticly. Only similar trusses were modelled in each assembly, except that one assembly had seven geometrically dissimilar trusses. Six typical truss configurations were used for similar truss assemblies. Each type of assembly was modelled as a sheathed and as an unsheathed assembly. In the sheathed assembly, trusses were connected using a continuous beam element on each side of the ridge to represent the roof sheathing. The influence of partial composite action was not included. Both unsheathed and sheathed assemblies were analysed to study and quantify the effect of roof sheathing. The results showed that the current repetitive-member factor (for bending) of 1.15 in the U.S. wood design code is conservative. In addition to allowable bending stress, the results showed that both allowable tensile and compressive stresses could be adjusted by the current repetitive-member factor. The results also showed that stiffness, and relative stiffness and strength of members all had significant impacts on load-sharing behaviour. Both studies investigated only the system behaviour of simple truss assemblies, which do not involve some of the system effects (due to gable-end trusses, the hip system, etc.) that are almost always present in an actual complex roof assembly. Therefore, as noted above, the results may not be applicable to more complex, in-service truss assemblies.

Other researchers have quantified system factors or load-sharing factors for joist floor systems[26–29]. Gromala & Wheat[30] summarized load transfer mechanisms in light frame subassemblies until the early 1980s, which mainly focused on floors and walls. They concluded that no well-established analysis capability existed for roof assemblies and that research is needed not only to develop analytical models, but also for the testing of full-scale roof assemblies.

Various specialized, non-commercial computer programs for analysing roof truss assemblies have been developed. These programs have been verified with experimental results and generally show good agreement. However, because limited model verification was performed in these studies and the studies used simple assemblies, their application to a wide range of complex, actual assemblies is untested.

DESIGN

The general approach of various codes around the world is to use some kind of ‘system factor’ when designing truss assemblies. The factor is then used in CDP for designing individual trusses that qualify as a part of an assembly. In the USA, a repetitive-member factor (1.15) is used to modify the allowable bending stress for any member qualifying as a part of the repetitive-member assembly[31]. The system factor of 1.15 for bending in the U.S. wood design code[31] is based on some early work within ASTM Committee D7 in the 1960s[24,32,33] and on analysis of three parallel bending members[34]. The repetitive-member factor for bending has been in use since the 1960s, but there is no similar factor yet in practice for tensile and compressive design stresses in the U.S. wood design code. However, recently, TPI 1-2002[35] has recommended a repetitive-member factor (1.10) for allowable tensile and compressive stresses for MPC wood trusses when designed using CDP. The Truss Plate Institute of Canada[36] recommends 1.10 system factor for longitudinal shear as well, in addition to bending, tension, and compression.

Eurocode 5—Design of Timber Structures uses system strength factor of 1.10 for assemblies defined in the code[37,38]. It is largely justified[38] using the works of Foschi[19], Wolfe & McCarthy[5] and Wolfe & LaBissoniere[6]. The Canadian wood design code[38] uses different system factors for different systems, materials and properties. The factors range from 1.0 to 1.40, depending upon the type of system, material and property. These factors are largely based on a detailed study by Foschi[19].

The Australian wood design code[39] probably has the most detailed inclusion of system effects by incorporating strength-sharing factors. It uses a general approach for different types of parallel systems. It combines the number of laminations in each member, the number of members in the parallel system, the spacing between members, and the effective span of the parallel members into one equation for calculating the strength-sharing factor. The factor only applies to allowable bending stresses.

It is clear from these standards that there is no single ‘system factor’ that takes into account the ‘system effects’ in truss assemblies. If SDP is used as the general approach for analyzing and designing truss assemblies, however, there will be no need for these different system factors in different codes, hence unifying the approach for designing truss assemblies.

After almost a decade of debate, a consensus-based guide for evaluating system effects in repetitive-member wood assemblies[40] was approved in 2001 in the USA. The guide ‘identifies variables to consider when evaluating repetitive-member assembly performance for parallel framing systems[and]...discusses general approaches to quantifying an assembly adjustment including limitations of methods and materials when evaluating repetitive-member assembly performance.’ No studies are yet available, however, showing the use of this standard in evaluating system effects in repetitive-member wood assemblies.

Studies at Oregon State University

The premise of the research at Oregon State University (OSU) is that there are ‘system effects’ that
are not currently included in the truss assembly design using CDP, and that a three-dimensional structural analysis program can be used to analyze and design whole-truss assemblies (i.e. SDP) in order to include system effects directly. These system effects are largely induced by the irregularly shaped assemblies, connections between various subassemblies, and the type of trusses used in the assemblies. This is an alternate approach to solving a problem that has been discussed for decades. Thus far, only one approach has been suggested and researched for including system effects in assembly design. In the past, researchers tried to simplify the problem by developing system or load-sharing factors that can be used in CDP. The research to develop these factors included analyzing simple assemblies with no gable-end trusses. Since actual assemblies have several different types of trusses, different shapes (L-, T-, odd-shapes), and gable-end trusses, the use of these factors in more complex assemblies is still open to discussion.

SDP is a simple yet comprehensive approach to solving a rather complex problem. It is an integrated analysis and design approach, as suggested by the research needs in wood engineering[41]. The proposed approach is also consistent with current industry trends[42–45] and national initiatives[46,47].

The basic approach of SDP is to analyse actual truss assemblies, whose layouts are currently developed by most truss-plate manufacturers, using three-dimensional structural analysis software. Also, if three-dimensional assembly models are used to derive system factors for a two-dimensional analysis of a single truss, the same three-dimensional model of the assembly could be used to analyze and design the entire assembly without the system factors. Although the use of system factors is appropriate when designing a single truss, which is always a part of an assembly, the same factor may not be needed if the whole assembly is analyzed and designed as a system.

In the first study at OSU, Li et al.[48] used a commercially available, general purpose, three-dimensional, structural analysis program, ETABS[69], to investigate the system behaviour of wood truss assemblies. In this study, single trusses (Fink and parallel chord) and nine-truss Fink roof truss assemblies were modelled. The Fink roof truss assembly (Fig. 2) studied had an 8.5 m span, and plywood sheathing attached to the top of the trusses. MPC Fink trusses and truss systems were modelled with two main types of elements: beam elements and spring elements. Wood truss and plywood sheathing members were modelled using beam elements. Although metal plate connectors have nonlinear semi-rigid behaviour, for simplicity, only linear semi-rigid connections were used in his model. The heel joints and bottom-chord tension-splice joints were modelled using ETABS spring elements. The stiffness of a spring element was calculated from the plate–wood contact area following the approach of Foschi[23]. Additionally, to include the two-way action of the sheathing, three-dimensional beam elements having six degrees of freedom at each end were used for roof sheathing. The sheathing beams were pin-connected to the top-chord members of the truss. Three sheathing beams were modelled on each side of the ridge. Moreover, partial composite action was included in this study.

Truss and truss system models were verified by comparing the predicted deflections, member internal forces, truss strengths, and load-sharing of four nine-truss roof systems with the experimental results from the literature[5,6,50,51]. The models were found to be in good agreement with the experimental results. Parametric study results showed that this model could be used to represent a three-dimensional truss assembly and accurately predict the system behaviour of a roof truss assembly. The study demonstrated the feasibility of using three-dimensional structural analysis software for investigating truss-assembly system behaviour. The study concluded that the truss system design may be improved by including system behavior directly, rather than using system modification factors to design single trusses.

In the second study[11], an actual MPC wood truss assembly was analysed as a system to investigate some of the behavioural issues that may be present in an actual, complex roof truss assembly. The assembly is in use in the United States, and the designs of all of its trusses were obtained from a truss-plate manufacturer (TPM). The TPM had designed the assembly using CDP. The assembly was also analysed as a system using SDP. A commercially available structural analysis computer program, SAP2000[52], was used to analyse the three-dimensional assembly.

The actual assembly is shown in Fig. 3(a) and the assembly model is shown in Fig. 3(b). The assembly consisted of fourteen different types of trusses plus two gable end trusses (Fig. 3b). The assembly has a total of 54 trusses, including more than one of most of the truss types. The three-dimensional SAP2000 model (Fig. 3b) consisted of sheathing beam model from Li et al.[53], no composite action, no vertical walls or foundation, and pin–pin boundary condition for all trusses.
The combined stress indices (CSI) of individual trusses from CDP were compared with the CSI obtained from SDP as shown in Table 1. Using SDP, the CSI for one member of a truss increased over 1.0 (indicating failure) in the assembly, demonstrating that redesigning the member (to bring CSI below 1.0) by using SDP will increase safety. The truss with the maximum CSI value of 0.99 (Truss type ASGR) based on CDP had CSI of only 0.62 based on SDP. The CSI values for most other trusses decreased (by as much as 43%), indicating potential truss fabrication cost reductions using SDP. Most individual trusses inside the assembly actually carried lower load (lower CSI values) than those outside the assembly because stiffer trusses (e.g. gable-end trusses, ASGR-1, BGR-1) attract a lot of load. The predictions of maximum CSI and its location using SDP differed from those based on CDP. The study concluded that a computer program capable of analysing three-dimensional structures can be employed as a practical analysis tool to design MPC truss assemblies.

The effect of variability of modulus of elasticity (MOE) on load distribution in subassemblies was also investigated. Truss types A1 and A2 (Fig. 3b) were chosen to illustrate this effect. The average MOE of the truss was used as a measure of its stiffness. Reactions at both supports of a truss were used as the load distributed to that truss. In general, a truss with higher MOE should attract more load in the assembly. However, there was no trend found to show that trusses with higher average MOE values attract more load. This is mainly due to the fact the difference between the stiffness of gable-end truss (AGB in Fig. 3b) and stiffness of A2 trusses is so large that variation in MOEs of A2 trusses has no or very little effect on load distributed among A2 trusses. The same is true for A1 trusses because their behaviour is affected more by the boundary condition (one end of all A1 trusses are supported by girder truss BGR-1) than by the variation in MOEs of A1 trusses. What this demonstrated is that variability in MOE values may be only one source of system effects. Our study showed that this effect is generally very small or not present at all. In actual truss assemblies there are other effects, related to geometry and configuration of assemblies which have much more pronounced effect on system behavior of truss assemblies than any effect due to variation in MOEs.

The study demonstrated that although there potentially are various ‘system’-related issues that strongly influence the behaviour of an assembly, they are not considered in CDP. The interaction of subassemblies and boundary conditions are identified as some of the ‘system effects’ that are not recognized in CDP, but strongly influence the behaviour of an assembly. The interaction of subassemblies was identified as the interaction between subassemblies A and B and A and J (Fig. 3a). The study showed that the behaviour of trusses connecting these subassemblies (BGR-1 and ASGR-1) is very different when analysed as a part of the assembly (i.e. using SDP) than when analysed individually (i.e. using CDP). Boundary conditions in truss assemblies (i.e. gable-end trusses) significantly affect load distribution. The analysis showed that the trusses near gable-end trusses have member forces that are much different when analysed using SDP as compared to CDP.

In the most recent study, Limkatanyoo[53] analysed three actual assemblies (Fig. 4) using SDP. Three different three-dimensional truss assemblies, T-shaped, L-shaped, and complex assemblies, were composed of 4 (total of 37 trusses), 17 (total of 56 trusses), and 27 (total of 123 trusses) different types of
individual trusses, respectively. TPM software was used to lay out and design the assemblies; the program uses CDP for designing assemblies. SAP2000 was then utilized to model and analyse three-dimensional truss assemblies (i.e. SDP). The structural responses, including CSI, truss deflections, and reactions from both CDP and SDP, were compared and the ‘system effects’ were evaluated.

The study showed that there are three system effects observed by SDP that are not accounted for by CDP. These effects were observed in all three assemblies. The first is the reduced applied load effect. This effect occurs because CDP assumes 0.61 m spacing for every truss in an assembly, whereas SDP assumes the actual tributary area. In most cases, CDP overestimates the applied loads. For example, in the complex assembly model, the total applied load assumed by CDP was 690 kN (assuming 0.61 m spacing on center for every truss in the assembly), while the total load for SDP was only 632 kN (based on the actual overall roof loading area in the assembly). The second system effect is deflection compatibility. Unlike CDP, SDP provides deflection compatibility because it analyses the entire three-dimensional truss assembly as a whole. On the other hand, CDP analyses one truss at a time and it assumes that truss supports have zero deflection in the vertical direction. When trusses are supported by other trusses, the support at the connecting point will not have zero deflection. Thus, the CDP is not a good approach for examining this connection in the assembly. The last observation is the stiff-truss effect. As expected, stiffer trusses in the assembly attract load. This effect can be observed by an increase in CSI values for the stiffer trusses and by a decrease in CSI values for the adjacent trusses that are less stiff. Based on this investigation, the maximum CSI for most trusses in all three assemblies reduces by 6–60% because of ‘system effects’. The study shows that SDP can help to improve the analysis of truss assemblies by directly including ‘system effects’ that are not accounted for by the CDP.

These studies demonstrate changes in individual truss behavior due to system effects related to the geometry and configuration of the assemblies. SDP can be used to consider these system effects directly. It can also provide a fuller description of truss behaviour in the assembly compared with CDP. Software used by TPMs already has the capability to lay out complete three-dimensional assembly geometry, and it could easily be extended from current two-dimensional truss analysis to three-dimensional assembly analysis and design.

### Table 1: Comparisons of maximum CSI values from individual trusses and the truss assembly

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Truss type</th>
<th>Number of trusses</th>
<th>Maximum CSI value from individual trussa</th>
<th>Maximum CSI value in assembly</th>
<th>Decrease (increase) (%)</th>
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<tbody>
<tr>
<td>1</td>
<td>A</td>
<td>4</td>
<td>0.95</td>
<td>1.03</td>
<td>(8)</td>
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<tr>
<td>2</td>
<td>A1</td>
<td>8</td>
<td>0.98</td>
<td>0.82</td>
<td>16</td>
</tr>
<tr>
<td>3</td>
<td>A2</td>
<td>7</td>
<td>0.93</td>
<td>0.82</td>
<td>12</td>
</tr>
<tr>
<td>4</td>
<td>AS1</td>
<td>1</td>
<td>0.86</td>
<td>0.85</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>AS2</td>
<td>1</td>
<td>0.91</td>
<td>0.52</td>
<td>43</td>
</tr>
<tr>
<td>6</td>
<td>AS3</td>
<td>1</td>
<td>0.81</td>
<td>0.51</td>
<td>37</td>
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<tr>
<td>7</td>
<td>ASGR</td>
<td>1</td>
<td>0.99</td>
<td>0.62</td>
<td>37</td>
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<tr>
<td>8</td>
<td>B</td>
<td>7</td>
<td>0.65</td>
<td>0.39</td>
<td>9</td>
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<tr>
<td>9</td>
<td>BGR</td>
<td>1</td>
<td>0.90</td>
<td>0.58</td>
<td>36</td>
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<td>9</td>
<td>0.82</td>
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<td>4</td>
<td>0.44</td>
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<td>12</td>
<td>J2</td>
<td>4</td>
<td>0.18</td>
<td>0.19</td>
<td>– c</td>
</tr>
<tr>
<td>13</td>
<td>J3</td>
<td>4</td>
<td>0.04</td>
<td>0.16</td>
<td>– c</td>
</tr>
<tr>
<td>14</td>
<td>JGR</td>
<td>2</td>
<td>0.52</td>
<td>0.43</td>
<td>17</td>
</tr>
</tbody>
</table>

a CSI = combined stress indices (sum of ratios of applied to allowable stresses for applicable types of stress)
b Based on conventional design procedure provided by the truss plate manufacturer
c CSI values increased but were still well below 1.0

![Fig. 4](https://example.com/fig4.png)

**Fig. 4** Three actual assemblies: (a) T-shape assembly; (b) L-shape assembly; (c) complex assembly

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Conclusions

Wood trusses are one of the main structural components in light frame wood construction. They are pre-engineered components fabricated from structural lumber and connected mainly with metal plates. The trusses are typically spaced 0.61 m on centre and wood structural panel (e.g. plywood or oriented strand-board) is nailed to the truss top chords to form an assembly, which acts as a system under service loads. The behaviour of single truss and connection has been studied in some detail, but the system behaviour of these assemblies has received limited interest by researchers around the world. So far only one approach has been researched and used in including system effects in assembly design. In the past, researchers tried to simplify the problem by developing system or load-sharing factors, which are then used in the design of single trusses. The research to develop these factors has included too few members of an assembly or has analysed very simple assemblies (i.e. identical and rectangular trusses and no gable-end trusses). This reflected that computations were based on hand methods or simplified two-dimensional computer-based methods. However, since these days actual assemblies have many more trusses and several different types of trusses including gable-end trusses, assemblies have different shapes (L-, T-, odd-shapes), and computing is cheap and three-dimensional computer-based analysis models are widely available, the use of these factors in more complex assemblies may be open to discussion.

Researchers at Oregon State University have proposed a systems approach to including system effects in the design of these assemblies. The systems approach analyses and designs actual truss assemblies using a three-dimensional structural analysis program, hence including both positive and negative effects of system behaviour. Studies have demonstrated that various geometry and configuration related issues (e.g. interaction of subassemblies and boundary conditions) strongly influence the behavior of an assembly, yet are not considered in current design practice.

This paper has summarized previous research on the system behavior of wood truss assemblies and proposes a new system-based approach to be included in their design.

References

Recommended reading


Johnson JW. Lateral tests of a 20- by 60-foot roof section sheathed with plywood overlaid on decking. Report T-29, School of Forestry, Oregon State University, Corvallis, OR, USA, 1971.


Levine E. A Review of load-sharing in theory and practice. CIB-W18 paper No. 4-8-1. 1975.

Levine E. Load sharing. CIB-W18 paper No. 4-8-2. 1975.

Levine E. Load sharing—An investigation on the state of research and development of design criteria. CIB-W18 paper No. 3-8-1. 1974.


Rakesh Gupta
Associate Professor
Department of Wood Science and Engineering,
Oregon State University,
Corvallis, OR 97331, USA
E-mail: rakesh.gupta@oregonstate.edu