A Comparative Study of the Analysis Model for Timber Structures: Addressing Nonlinearities and Connection Behaviour

Huseyin Kursat Celik,[a,b,]* Gokhan Sakar,[b] and Haytham F. Isleem,[c]

Timber has been studied as a material of construction from many perspectives, including strength and durability. Despite many studies showing a good correlation between material models, connection behaviour, and structural modelling, it is still not clear which approach is suitable under what constraints. This study was performed to clarify the problem. The basis for the analysis of timber structures is emphasized in this work in terms of problem dimension, material constitution, and geometrical nonlinearities. The modelling and the idealisation methods of structures are categorised into five different groups and briefly explained. An experiment available in the literature is used as a reference to illustrate modelling capabilities of different techniques, and models from five different groups is used to perform analysis. By comparing the analysis results and the experiment, key notes on the analysis proficiency are highlighted, including failure load, maximum displacement, and failure mode. The results show that most of the errors occurred in the displacements. Furthermore, the divergence between test and analysis results are investigated, and an approximate method for calculating actual displacements is proposed.

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Contact information: a: Department of Civil Engineering, Izmir University of Economics, Izmir, Türkiye; b: Department of Civil Engineering, Dokuz Eylul University, Izmir, Türkiye; c: Department of Computer Science, University of York, York YO10 5DD, United Kingdom;
** Corresponding author: kursat.celik@ieu.edu.tr*

INTRODUCTION

Timber has been a choice to construct buildings, bridges, and other civil structures throughout history. It has high strength relating to density. It can be easily obtained and shaped into desired forms. It also is highly regarded when considering the environmental impacts.

Although timber has many favourable properties, some disadvantages should be considered. Timber is a heterogeneous, anisotropic material, and it exhibits brittle failure under tension. It shows different load-deformation behaviour depending on the loading direction (Fig. 1). The strength parameters of timber are significantly influenced by climatic conditions. Timber joints generally have lower load-bearing capacity compared to connected structural elements. In addition, it is difficult to obtain a large cross-section from the natural size of a tree. Today, many disadvantages of timber can be overcome by technological advances such as engineered wood products, preservation techniques, *etc*. Any shape and section can be achieved with glued laminated timber. With advances in wood treatment, it has become possible to achieve favourable results for the strength and durability of timber members, relative to their use in construction In this way, timber being a traditional material has become a modern constructional material. Timber has been used to construct bridges, buildings, *etc*., since the industrialization of timber began in 1900s with the invention of glued laminated timber. Today's modern timber structures are generally constructed by using engineered wood products and steel connectors. Challenging projects have been installed all around the world. Two of the best known examples are Mjöstarnet and Mistissini bridge (Lefebvre and Richard 2014; Abrahamsen 2017).

With the industrialization of timber construction, studies on the timber have also gained more attention for understanding the efficient use of timber in construction. Many studies have been proposed to improve strength and durability of timber. New construction methods have been improved such as mass-timber and timber-concrete composites. Similarly, mechanics of timber has also been studied by researchers (Bazan 1980; Buchanan *et al.* 2008), since it is important to study the behaviour of a civil structure under loads. In this scope, one of the major areas of research is the analysis of timber structures.

To analyse a structure, a material model, joint behaviour, and idealisation of the structure are required (Çankal *et al*. 2023). Literature knowledge on material modelling started with the linear-elastic assumption of the timber. The linear-elastic assumption states that timber shows totally elastic behaviour throughout the load history, with linear correlation between stress and deformations (Buchanan *et al*. 2008). Due to its simplicity, the elastic model is still widely used in design practice; however, it does not fully reflect reality, as timber shows plastic behaviour under compression. Thus, many studies have been proposed to model timber's plasticity. A well-known example of this is the elastic-perfectly plastic model (Eurocode 5 2004). In this model, the stress-strain relationship is modelled in two parts. First, timber shows linear elastic behaviour up to specific stress. Beyond this point, the material is unable to withstand further stress and deforms severely under the same stress. Strain hardening or softening effects can also be included, especially where local behaviour is important, such as timber-steel or timber-concrete joints (Dias *et al*. 2007; Awaludin *et al*. 2012).

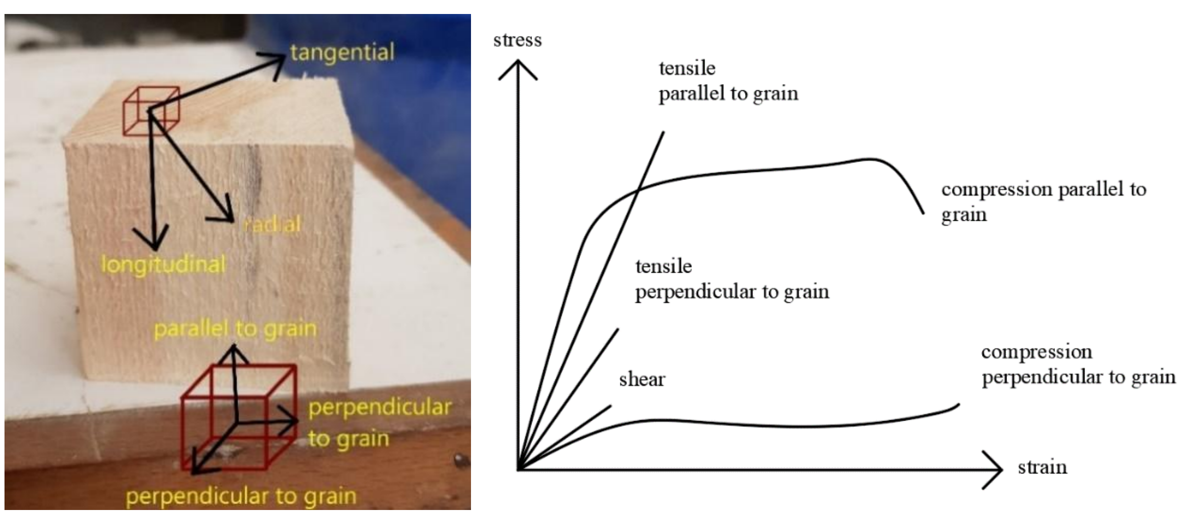


Fig. 1. Load-deformation behaviour of timber in different load directions and the symmetric orthotropic idealization of timber

Because the strength parameters of timber are highly dependent on the source tree and the environment, empirical models are also favoured in the literature. Many studies have been reported such as those of Bazan (1980), Zakic (2003), and Moe (1961). Although it is the most realistic method, the availability of empirical models is limited by test data. Thus, when considering a wood material for which reliable data don't yet exist, a more specific model may be required. This can be achieved by incorporating failure criteria. Failure criteria can be defined as a stress state equation that describes the failure limit of a material. Because of the wood is anisotropic, anisotropic failure criteria such as Tsai and Wu (1971), Hashin (1981) and Hill (1998) are widely used. In general, the Hill criterion has shown good correlation in many studies (Xu *et al*. 2009).

Another major problem is how to idealise timber structures. Today, many structural elements can be constructed with timber, such as shear walls, trusses, columns or beams, braces, *etc*. In general, structures are idealised by their shape such as frame for beam-column, plane for slabs and walls, truss for tension or compression bars, *etc*., with respect to stress state of the considered problem. This is more important when the local behaviour is strongly influenced by the system behaviour. Khelifa *et al*. (2016) showed that complex material modelling, such as plastic models, is not sufficient for efficient analysis; finite element models are also extremely effective. Studies are now focusing more on local effects such as connections, grain distortion, knots, *etc*. (Bano *et al*. 2013).

Connections have a strong effect on structural behaviour. As well as being a load-bearing element, connection properties affect elemental and global load-displacement properties of a structure. In general, joints are assumed to be perfectly hinged or rigid in practice. Models based on hinged joints assume that the connection being considered will be rotate without showing any stiffness and thus cannot bear any moment. The rigid assumption states that connections will withstand moments without rotation. However, this is not the case in practice (Fig. 2).

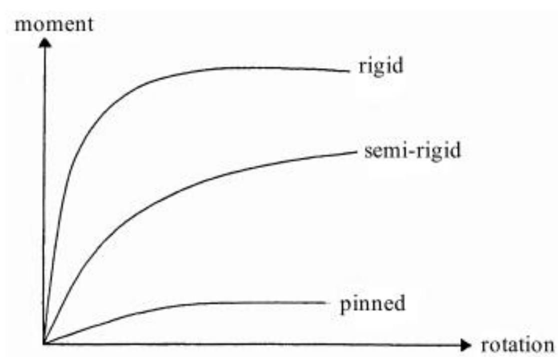


Fig. 2. Moment-rotation relationship of rigid, hinged and semi-rigid connections

In real applications, connections rotate under load while bearing moments. This is commonly named as semi-rigid and has been widely studied, particularly in steel construction (Celik and Sakar 2022). The behaviour of the connection should be considered in the analysis because the stiffness of the connection also affects the stiffness of the structure. Some studies have been reported on this subject. Connection behaviour is generally defined by the load-deformation relationship, which can be captured with empirical, analytical, or numerical approaches. Cao *et al*. (2019) used Bayesian method to model CLT wall-slab connectors. Nonlinear regression was employed by Hassanieh *et al*. (2017) for timber slab to steel beam connections. Wanninger and Frangi (2016) used compatibility equations to model pre-tensioned timber connections. A wide range of connections, from traditional to glued joints, were studied with the component method by Wald *et al*. (2000) and Yang *et al*. (2016).

To analyse a structure, it is not enough to determine the load-deformation relationship of a connection. It also depends on how the data is incorporated into the analysis. Generally, joint behaviour is incorporated into a model with rotational springs, as the end condition of a frame element. However, such a model is not always applicable. Consequently, the connection element method was developed by Li *et al*. (1995). In this method, the joint behaviour is idealised as a frame element. It should be noted that not much work has been done on this topic for timber structures.

Today, several techniques are used to connect timber elements, such as traditional joints or dowel-type. Traditional joints (also known as carpenter joint) can be explained as a mechanical process shaping timber elements to connect two or more timber elements. Several studies in the literature showed that traditional joints generally have a lower load-bearing capacity, and their stiffness is strongly influenced by the joint configuration, so that they can have zero stiffness (Chang *et al*. 2006; Pang *et al*. 2011; Moradei *et al*. 2018).

Another connection method is the dowelling of timber members with connectors made of steel, wood, plastic, *etc*. These types of joints generally transfer loads by bearing shear stress. Various connectors can be used as a dowel, such as nail, screw, bolt, *etc*. The main advantage of this technique is its ductility. Figure 3 shows a load-deformation curve of a typical dowel type connection. Due to its practicality and ductility, it has been widely studied. In general, dowel connections exhibit an initial deformation due to imperfect contact between connecting element and dowel. On loading, the contact surface becomes more uniform, and loads can be transferred effectively. Further loading results a sudden drop in the stiffness. Depending on the connection configuration and dowel material, a yield plateau can be seen. Yielding can occur by timber resin matrix or connector plasticity. After yielding, failure occurs (Dorn *et al*. 2013). The complex behaviour of dowel type connections has been extensively studied. Good correlations are generally obtained with nonlinear models with failure criteria, while the anisotropy of the timber idealised to symmetric orthotropy (Bouchair and Vergne 1995; Dias *et al*. 2007; Khelifa *et al*. 2016).

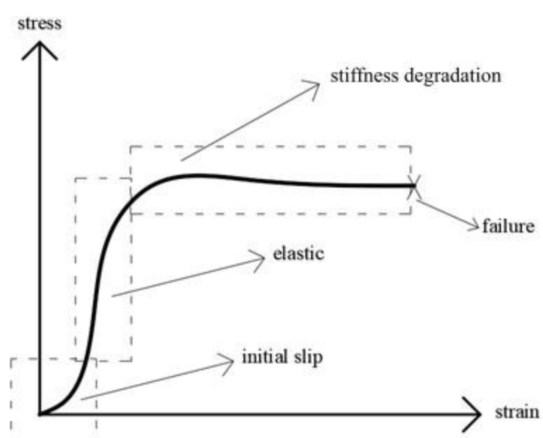


Fig. 3. Load-deformation relationship of dowel joints

Although many studies have been carried out on material modelling, numerical analysis, structural topologies and experiments, *etc*., both the behaviour and the selection of the correct method for analysis and design remain unclear. To illuminate uncertainties of the analysis of timber structures, the present study was performed. A full-scale experiment on a timber structure was adopted from the literature (Vallee *et al*. 2011); and five different methods were determined and classified for the job. Simulation of the experiment was performed with different methods and critiques were made.

EXPERIMENTAL

There are many methods available to analyse timber structures, as mentioned above. For instance, first order linear elastic frame analysis can be used to design a frame to resist vertical loads. Connection stiffness and material nonlinearity can be included where seismic loading is the case. Plane stress analysis can be used to investigate door and window openings a CLT panel while contact analysis used to study local behaviour of a connection. Similarly, methods can be used to model a structure for different aspects. First order elastic analysis can be used to study the elastic behaviour of trusses. If connectors are not fully pinned in reality, connection stiffness can be included and frame analysis can be performed. If nonlinear response is more important, shell elements can be used. To capture failure mode and exact behaviour up to failure, contact forces with nonlinear finite element analysis is better. From this perspective, from linear analysis to nonlinear finite element analysis, methods can be classified according to the indeterminacy and design requirements of a structure. Thus, five different modelling techniques have been generated by the authors in terms of model complexity.

Modelling Techniques

To analyse a structure, the relationship between the external forces and the stiffness of the structure and also the displacement of the structure, should be determined. Since external forces and boundary conditions are given, modelling issues are more about how to idealise the stiffness of the structure. So modelling means idealising the structure in a geometrical and material way such as frame, shell, or solid elements.

Five modelling types are examined within the scope of this article. Model 1 is defined as first order linear elastic analysis. According to Model 1, a mathematical model of a structure can be idealised as frame or truss elements, based on problem requirements. In this model, material is considered as linear elastic throughout the load history, with respect to anisotropy. Geometric nonlinearities are ignored; and first-order static analysis is performed. To include connection behaviour, element end conditions are assumed to be perfectly hinged or rigid. This model is generated for ease of modelling and structural behaviour under service loads. This model also represents the design practice even for seismic design of structure with specific limitations according to codes or regulations (Eurocode 5 2004).

In some cases, geometric nonlinearities cannot be totally ignored even in elastic range of load-history, such as the slenderness problem. To meet this requirements, Model 2 was proposed. In this model, material behaviour is assumed linear elastic. Static analysis is performed with regarding the P-Δ(load—displacement) effect. Connection stiffness is included into analysis model with the spring end condition of a frame element. Only the elastic stiffness of connection is considered with respect to material elasticity. To determine joint flexibility, any method can be used in the literature, such as empirical or analytical. With frame idealization of structure, global analysis is performed.

To investigate the inelastic behaviour of a structure, materials should be modelled as nonlinear. Nonlinear material models are especially important for extreme loads such as seismic action, *etc*. To achieve this, an elastic-perfectly plastic (Eurocode 5 2004) material assumption is used in Model 3. Geometric nonlinearities are incorporated with general second order static analysis.

Similar to the material model, joint flexibilities are modelled with fully inelastic without ignoring nonlinear parts of connection behaviour. Based on the problem and material, strain hardening in compression and strain softening in tension also can be included. The mathematical model of structure is idealized with frame elements. This model is developed with regarding to structural engineering design practices such as code-based performance evaluation of a structure. For this job, inelastic material modelling is suggested to illustrate real behaviour of a structure.

To this extent, general load arrangements were considered, including imposed, point, or a joint load. However, a structure can be subjected to a local load such as an impact, blast, *etc*. Moreover, local discontinuities of a structure cannot be studied with frame idealization. To overcome this drawback, Model 4 is proposed. In this model, the mathematical idealization of a structure is performed with plane finite elements. There are many types of planar finite elements available in literature, such as thick or thin shell, plane stress elements, composite shells, *etc*. In this model, based on the relevant material and problem, an appropriate plane element is used. The material is modelled as totally nonlinear.

Based on material properties, elastic-plastic idealisation with strain hardening/softening can be included. To accurately model the inelastic behaviour of a structure, fracture mechanics should also be included. Failure criterion is considered for this task. Depending on the problem, anisotropic or isotropic failure criteria can be used. Second-order static analysis should be performed with respect to the P-Δ effect. However, geometric nonlinearities such as contact between elements is ignored. Slip-like interactions between elements are not included in this model.

Thus far, models were proposed to study a specific problem with some assumptions. Ignoring some interactions and inelasticity may lead to miscalculation. Therefore, Model 5 is proposed based on comprehensive finite element models without ignoring geometric and material nonlinearities. In this model, structural idealisation is realized with solid finite elements with inelastic material models. Fracture mechanics are also included to capture the total behaviour up to structural failure.

Example

In accordance with the model definitions, a timber structure was included in the study to investigate the efficiency of the models. The selected timber structure has nonlinearities, statically determinate and ductile joints, which are crucial to illustrate modelling aspects and study modelling efficiencies. The example used in this study was adopted from Vallee *et al*. (2011).

The structure under study is a timber truss with steel plates dowelled into truss members. The truss consists of seven nodes, and it is restrained at the span ends. Monotonic loading was applied at midspan up to failure. The truss geometry and the loading scheme are illustrated in Fig. 4.

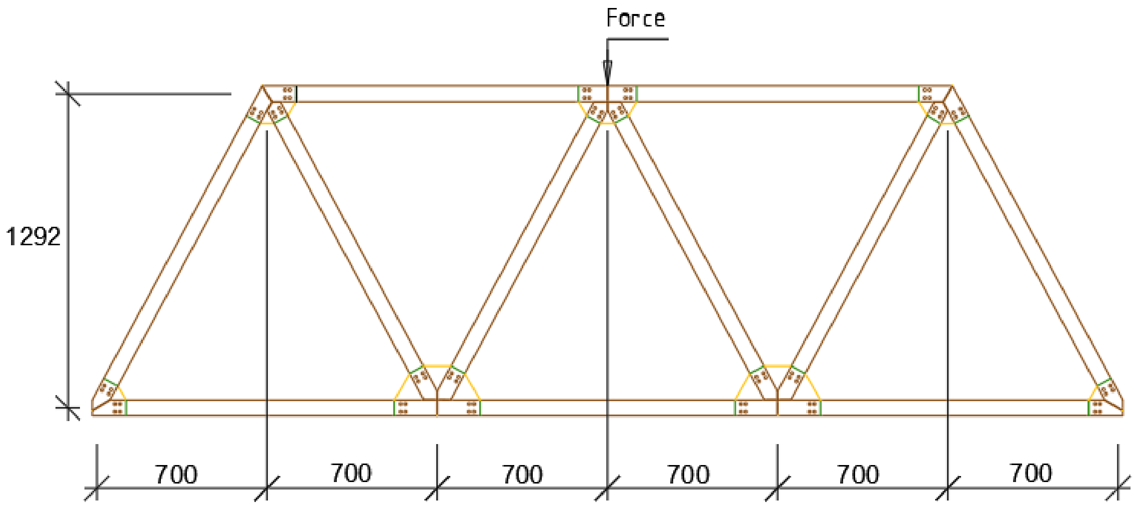
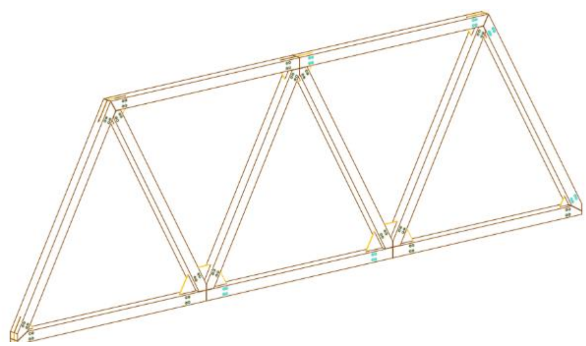
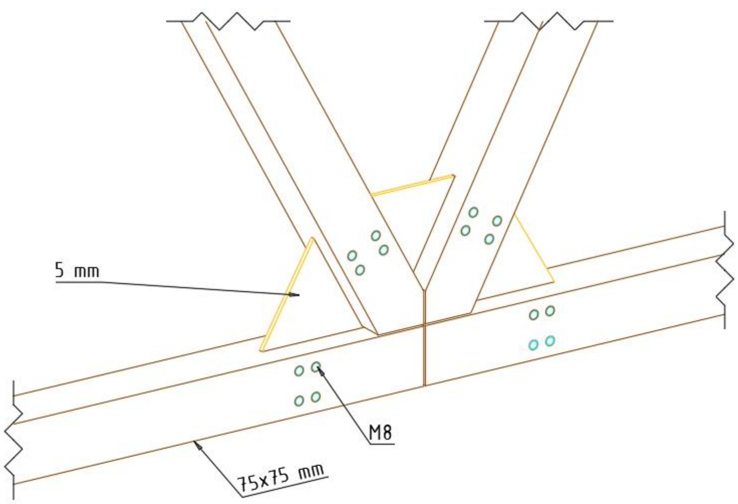


Fig. 4. Truss geometry



(a)



(b)

Fig. 5. (a) 3D view of the truss; (b) joint detail

The truss was constructed from *Picea abies*. The strength parameters were determined through testing and are given in Table 1. All timber and connection elements have the same properties. The timber elements were 75x75 mm square and the steel plates at the joint were 5 mm thickness. Timbers and plates were connected with four 8 mm steel dowels. Steel plates were socketed 120 mm inside to truss members. The truss and connection are shown in Fig. 5.

Table 1. Material Properties of Truss

E_x	17900 N/mm^2
E_y	11000 N/mm^2
u_{xz}	0.4
y_{yz}	0.4
f_o	98.2 N/mm^2
f_{90}	4.5 N/mm^2
F_{xy}	16.5 N/mm^2
where, *E* is modulus of elasticity, *u* is Poisson ratio, *f* is strength, 0/90 load angle	

Model 1

To analyse the truss using Model 1, the structure was idealised with truss elements. In this idealisation, deformations that differ from axial strain are ignored. This means that only the axial stiffness of the elements is considered. The timber elements were modelled as a symmetric orthotropic material (Fig. 1). Symmetric orthotropy states that an anisotropic material can be determined with only two different axial behaviours due to the symmetry of two axes. Timber can be modelled as symmetric orthotropic because it has similar properties in the radial and tangential direction. Consequently, timber was modelled as symmetric orthotropic for ease of modelling in this study. First order linear analysis was performed to obtain the failure load. Connection properties were not considered realistically. Nonlinear behaviour of connections was ignored, and all joints were assumed to be perfectly hinged and idealised as truss nodes. The model and deformed shape of the model are shown in Fig. 6. The truss elements were defined along the timber element axis and the nodes were obtained by intersecting of the element axes. The analysis was performed with SAP2000 (2023). In Model 1, the failure load was determined by the external load value at a timber element or joint reaching the elastic stress limit.

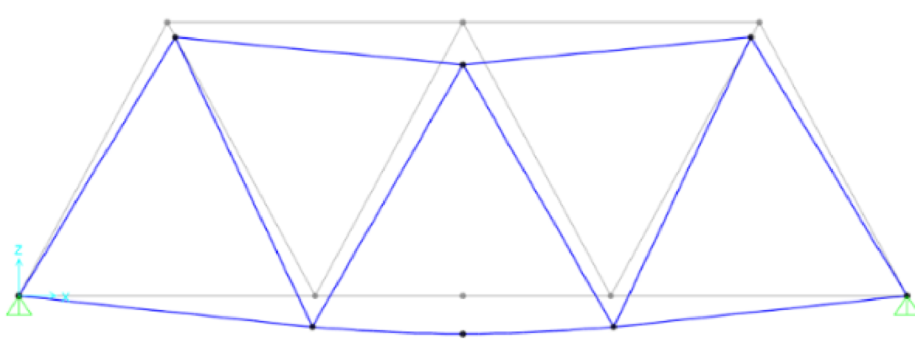


Fig. 6. Frame model of the truss

Model 2

According to Model 2, the truss was modelled without hinges at joints. The rotational stiffness of the elements was included as a semi-rigid joint. Rotational springs were used for this purpose. Because the behaviours of joints are not perfectly pinned in nature, truss element will be subjected to bending moment. Hence, the truss was idealised with a frame element. The material was modelled as symmetric orthotropic and linear elastic while second order static analysis was performed.

In order to perform Model 2 analysis, two points should be illustrated: how to define the stiffness properties of the connection and how to include them in the analysis model. Many methods such as empirical, test and finite element, *etc*., are available in the literature for this purpose. The authors preferred to use rotational springs. According to this approach, stiffness values are considered directly as the end stiffness of a frame element. The stiffness of the connections was determined from the test results given in Vallee *et al*. (2011). The stiffness was idealised as linear with respect to the linear elastic material model, ignoring the inelastic parts such as initial slip or strain softening. The idealisation of the stiffness is shown in Fig. 7. Analysis was performed with SAP2000 (2023). The failure load is determined as in Model 1. Due to the elastic analysis, the failure load is the load level that caused a frame or connection to reach the elastic stress limit.

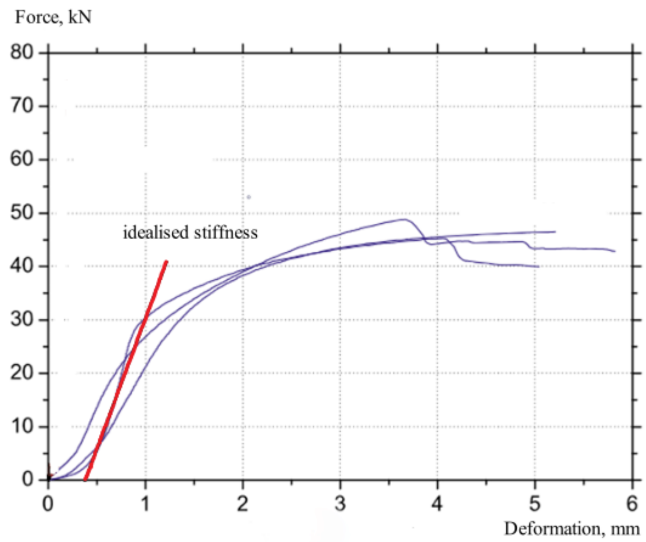


Fig. 7. Linear idealization of connection stiffness (adopted from Vallee *et al*. 2011).

Model 3

According to Model 3, the material should be modelled as nonlinear. Therefore, timber was modelled as nonlinear, and the material model was taken from Eurocode 5 (2004). According to Eurocode 5, timber is assumed to be elastic-perfectly plastic in compressive strength, while tensile strength is assumed to be linear-elastic with brittle failure. Second-order static analysis was carried out to failure while joint stiffness was modelled using the connected element method (Li *et al*. 1995). According to this method, the truss elements can be modelled as frames with rigid ends. The truss members were meshed into three finite elements as timber element and both end connections of the member to account for connection flexibility. Correspondingly, the end meshes were modelled to represent connection flexibility, but as a frame. To do this, it is sufficient to know the moment-rotation relationship of a connection. More details of the method were given in Li *et al*. (1995). In contrast to Model 2, the joint stiffness was modelled as tri-linear to include the initial slip and the elastic-plastic region of connection. The tri-linear idealisation of connection element is illustrated in Fig. 8. The analysis was performed with ABAQUS (2022).

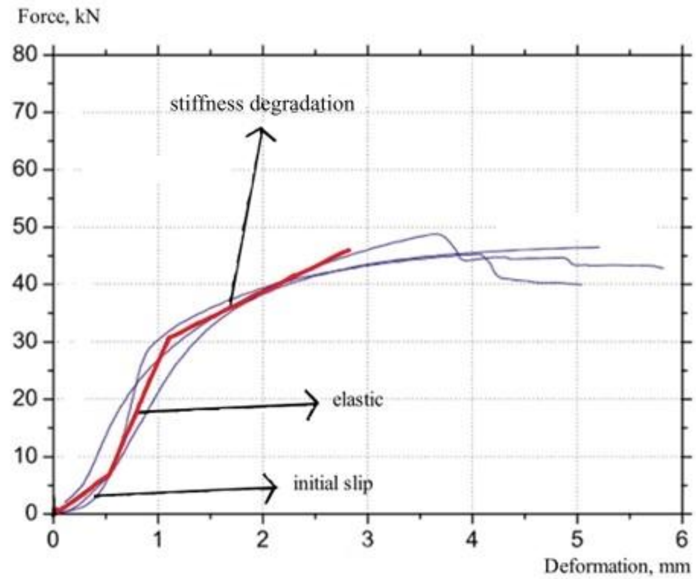


Fig. 8. Trilinear idealization of joint stiffness and connected element (adopted from Vallee *et al*. 2011).

Model 4

In Model 4, the timber truss was modelled with shell elements. The material was modelled as nonlinear with failure criterion. Although many studies have been proposed on failure criterion of timber, the Hill criterion (Hill 1998) was used in this study to model timber. Hill criterion is a well-known failure theory for anisotropic and composite materials that have different compressive and tensile strengths. Many studies reported using the Hill criterion to analyse timber structures showed good correlation. Thus, Hill criterion was chosen. According to the Hill criterion, six different yield criteria should be determined with respect to the anisotropy of the material. The truss was modelled with a thick shell and slip, and contact forces were ignored. A fine mesh was generated with four node quadrilateral shells. The structure was meshed by considering the magnitude of the stress distribution. The mesh was concentrated in the connection regions and loosened outside the joints. The minimum mesh size was 4 mm which correlates with the minimum dimension of the structure (dowel radius). The finite element model is given in Fig. 9. The analysis was performed with ABAQUS (2022).

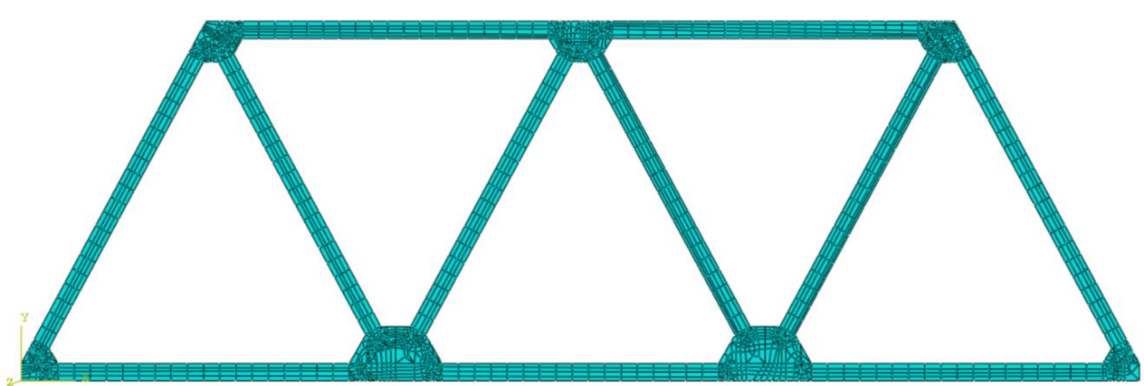


Fig. 9. Finite Element Model of Truss (Vallee *et al*. 2011).

Model 5

For Model 5, the results of the Vallee *et al*. (2011) were adopted and not modelled by the author. Vallee *et al*. (2011) modelled the truss with a solid element with nonlinear timber properties while the steel dowel was assumed to be linear. The Norris criterion (Norris 1962) was applied and contact forces were included.

RESULTS AND DISCUSSION

According to the results of the study, Model 1 showed that the failure occurred at the joints. The failure load was 22.5 kN and the corresponding deflection was 0.6 mm. At the failure load, the stress in the timber members was 1 to 2 MPa.

Analyses with Model 2 were performed step-by-step. Firstly, only P-Δ effects were included in the elastic analysis, and it showed similar results to Model 1. Deflections were increased but only by 0.1% while forces and stresses did not change. After that, analysis was repeated with semi-rigid joints. According to the analysis results, deflections were increased by 3% while the increases in forces and stresses were still small.

Model 3 analyses failed at 32.42 kN. The failure occurred at the joints. The deflection of the truss at the failure load was calculated as 14 mm.

The failure load calculated for Model 4 was 39.5 kN. Failure mode was rupture of timber at the joints. The deflection at the failure load was 16 mm. Finally, model 5 showed a failure load of 41.5 kN with a deflection of 10.45 mm. The deformed shapes and stress diagrams are shown in Fig. 10. The comparative analysis results are given in Table 2.

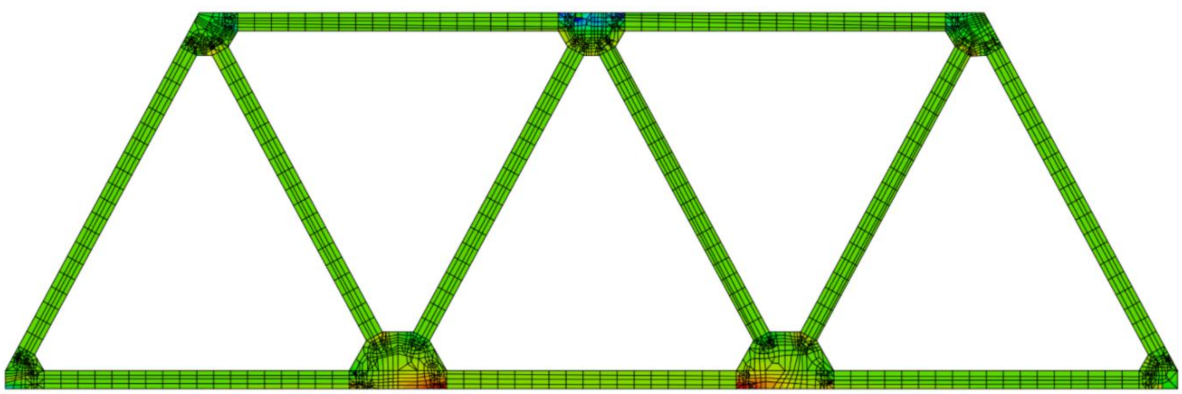


Fig. 10. Stress Diagram of Model 4

According to the study results, linear elastic models such as Model 1 and Model 2 showed failure loads that were not similar to the experiments. The displacements of Model 1 and Model 2 likely showed 95% deviation from the test results. It can be said that the difference between the elastic analyses and test results was mainly caused by the inelastic behaviour of the materials and the behaviour of the connections in the initial slip phase. Initial slip caused a large increase in the global displacement of the truss due to the statically determinate nature of the problem. Without modelling the real behaviour of the connection according to Model 1 and Model 2, the displacements cannot be correctly captured. However, even without modelling the connections, there was a correlation between the test and analysis results.

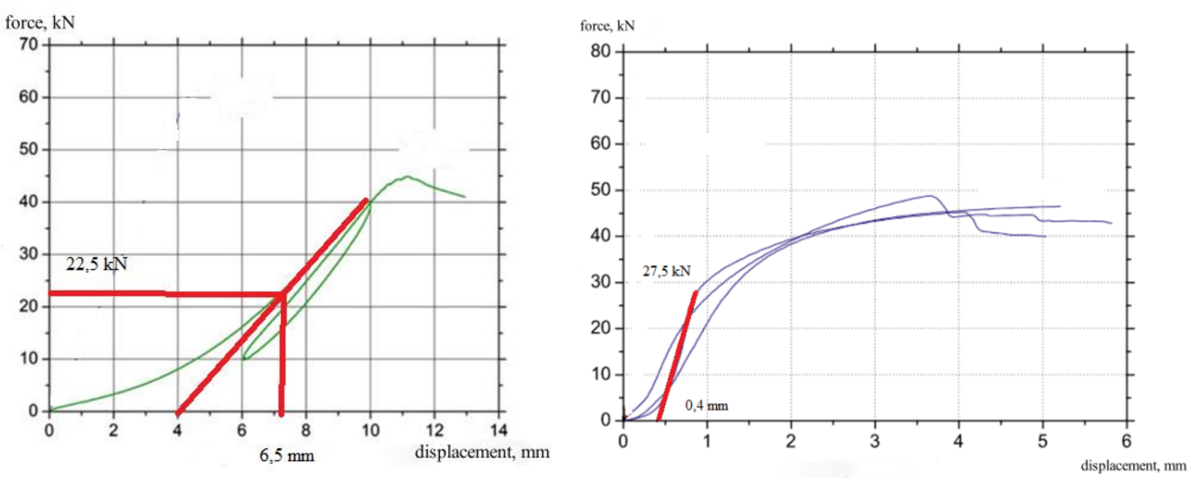
Table 2. Analysis Results of Models

Models	Model 1	Model 2	Model 3	Model 4	Model 5	Test
Failure Load (kN)	22.5	22.5	32.42	39.5	41.5	44.9
Model/Test for Failure	50%	50%	72%	88%	92%	-
Displacement (mm)	0.6	0.62	14	16	10.45	11
Model/Test for Displacement	5%	6%	127%	145%	95%	-
Failure Mode	Joint	Joint	Joint	Fracture in timber	Fracture in timber	Fracture in timber
Feasibility	Global	Global	Global	Local-Global	Local-Global	-
Complexity	Easy	Easy	Hard	Hard-extreme	Extreme	Extreme

Figure 11 shows the elastic failure loads from the study results. Most of the displacements were realized in the initial slip region. It can be assumed that until the initial slip is complete, the structure deforms similarly to an unstable structure. After the initial slip is complete, the system becomes statically determinate, and the load can be transferred between the elements. Based on these results, Equation 1 is proposed by the authors.

$$\sum_{x=i}^{j} \delta = \delta_x + \delta_{global,elastic} = \delta_{system} \quad (1)$$

where δ is displacement. According to this equation, the deflection of the global structural system is equal to the sum of all the initial slip deformation of joints and the deflection calculated from the global elastic structural analysis. Some assumptions have been made for the equation; initial slip does not cause any significant value of stress at the elements, initial slip can be ignored in the global analysis, and the superposition principle is valid for the final deflection of the system.



a) Truss **b)** Joint

Fig. 11. Load-displacement curves with indication of elastic limits (adopted from Vallee *et al.* 2011)

Eurocode 5 (2004) defines the slip modulus according to Eq. 2, per shear plane per fastener. So, in this example, connections have four shear planes, and the slip modulus can be calculated as 11,975 N/mm by considering the connector diameter and density of timber. Then, by using the failure load of the joint, which was found to be 27.5 kN in the elastic region and the slip modulus calculated from Eq. 2, the slip deformation could be calculated as 2.3 mm for a joint connecting four elements. Considering all the connections of the truss, the total initial slip of the structure could be calculated as 5.6 mm. The sum of the linear elastic and the initial slip displacement gives 5.6 + 0.6 = 6.2, which is very close to the deflection at the elastic limit of the truss, as shown below,

$$K_{ser} = \frac{{\rho_m}^{1.5} * d}{23} \tag{2}$$

where K is slip modulus, ρ is density of timber, and d is diameter of connector. Model 3 was successful in determining the displacement. This is due to the modelling of real joint behaviour. A non-linear analysis should be performed for this. A correlation was also found for Model 3. The initial slip can be taken as the flexibility of the joint. If the global elastic analysis is performed in two stages with two different joint stiffness, corresponding to initial slip and elastic stage, the final deflection of the system will be the sum of two corresponding results. The same assumptions have been made for this method. It is assumed that initial slip does not cause significant stress on the element and deformation due to initial slip can be ignored in the global elastic analysis. It can also be said that modelling the initial slip as a connection stiffness is able to capture the initial displacement of the truss.

Model 4 was successful in determining failure load and the failure mode but was slightly less accurate in determining deflection. This is due to ignoring second order effects such as contact. The failure mode of Model 4 and the test is given in Fig. 12. By including contact forces, more accurate results can be obtained as shown by the results of Model 5.

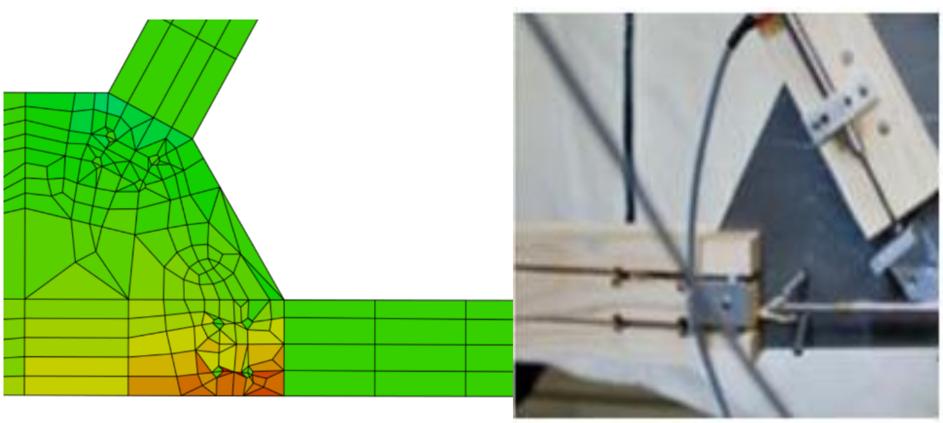


Fig. 12. Failure mode of Model 4 and test (adopted from Vallee *et al*. 2011)

This study reported a short summary of the literature on the analysis of timber structures. Modelling issues and the nonlinear complexities of the timber were highlighted. The analysis methods were categorised in terms of material and geometric nonlinearities including modelling aspects. A full-scale test from the literature was considered to investigate performances of models. Comparisons between modelling techniques were made in terms of failure mode, failure load, and deflection. According to the results of the study, the elastic model was the most erroneous approach due to deviation from all the criteria. It was also concluded that the connections stiffness was more important than the second order effects on the results of deflection and failure loads.

The overall behaviour of timber structures cannot be ignored in the analysis. From a design point of view, the stiffness of the joints and the degree of static determinacy are crucial to calculate deflections and cannot be ignored. Joint stiffness and the material nonlinearity are essential to determine failure load, and modelling techniques are very important to study failure modes.

CONCLUSIONS

1. The modelling of joint stiffness cannot be ignored to determine deflections correctly. The load-slip behaviour of dowel connections has a strong effect on deflections.
2. Finite element modelling and nonlinear shell elements showed good correlation with the test results. However, the finite element models (FEM) underestimated the truss deflections, and the shell model was more conservative than solid modelling.
3. The use of nonlinear frame analysis with semi-rigid connection (Model 3) and shell analysis is highly recommended, as comprehensive FEM models are generally not applicable in design practice. Shell elements were more successful than nonlinear frame element analysis in determining failure modes.
4. The elastic analysis gave the most inaccurate results, and it is often preferred in practice. Therefore, an approximate method to determine the real displacement with elastic analysis was proposed. The method is easy to use and can also be used together with Eurocode 5 (2004). Although Eurocode 5 ignores the initial deformation in elastic analysis, the results of the study showed otherwise. The large errors in the deflection results with elastic analysis can be attributed to the truss geometry, which is statically determined. This means that any failure in the structure will result in collapse. Thus, the initial slip leads to instability, and the deflection of the truss is suddenly and extremely increased, rather than being an elastic structure that shows strength to external forces.
5. As relative humidity has a strong effect on the mechanical properties of timber, initial slip deformations are also affected by it. Consequently, the effectiveness of the proposed method will depend on the treatment of service conditions in the calculations. Although Eq. 2 does not have a direct relationship with humidity, it can be modified with respect to Eurocode 5 (2004). Thus, the slip modulus should be adjusted with the relative humidity in service conditions to obtain more realistic results.

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