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INFLUENCE OVER THE STRESSES OF THE SIZE AND POSITION OF THE PUNCHED METAL PLATES STRUCTURAL TIMBER JOINTS

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ABSTRACT

The Eurocode 5 gives the rules for calculating the minimum size of the punched metal plates used in timber structural joints, taking into account the strength of the plates and the anchorage capacity between the plates and the timber. The geometric parameters considered by the standard are basically the plate area and its length measured along the connection line between the timber members. This paper presents a further study of this kind of joints, analysing the relationships between the position and dimensions of the punched metal plates of a joint of a light frame structure and the stresses that appear in the plates and the timber elements, through the application of the finite element method.

Keywords: timber joints, finite elements, punched metal plates

INTRODUCTION

The use of steel nail-plates to join timber members together in light structures, such as frames and trusses, began in the late 1950s (Karadelis, 2000). These connectors, made of light-gauge galvanized mild steel, have nails or teeth that have been punched out to one side of the plate by a stamping process. The joints are built placing a pair of nail plates on opposite faces of the timber elements. This procedure is used for connecting members of framed timber structural components at any angle within the same plane, and has some advantages over other connection methods. For example, with this kind of connection, the size of the timber members does not depend on the size of the joint. Also, costly procedures needed in other classes of joints, which imply drilling or hand nailing, are no longer necessary. In addition, these joints are manufactured under factory conditions, where press or roller equipment is used to embed the projected nails into the timber (USDA, 1998), eliminating the errors and the lack of precision involved in field work.

In Europe, the design of this class of joints is regulated by the Eurocode 5 (CEN, 2004). This standard essentially prescribes the checking of the anchorage capacity between the plates and the timber, and the strength capacity of the plates. For every plate type, the geometric data that appear in these verifications are basically the plate area and its length measured along the connection line between the timber pieces (Argüelles, 2003).

The aim of the present work was to gain further insight into the influence of the size and position of the plate, considering its length and height independently, as well as its horizontal and vertical position relative to the joint. More specifically, the purpose was to determine the correlation between the parameters indicated and the level of stresses that appear in the joint. The study used the software ABAQUS to design several parametric finite element models, which allowed a detailed analysis of the stress state in the plates and timber elements.

RULES FOR DESIGNING PUNCHED METAL PLATE FASTENERS

The connections made with punched metal plate fasteners can fail in two situations. The first happens when the anchorage or adherence capacity between the plates and the timber members is exceeded due to the forces transmitted between the teeth and the timber. If these forces provoke static stress concentrations under the teeth that surpass the timber strength, the nail-plate tends to pull away, sliding from the timber as shown in the left part of Fig. 1 (Stehn, 2004). The withdrawal of the nails is a consequence of the splitting in the timber (Ellegaard, 2002). In this case, the solution is to increase the transmission area between the timber and the plate and to establish an adequate number of teeth, with suitable separations between them.

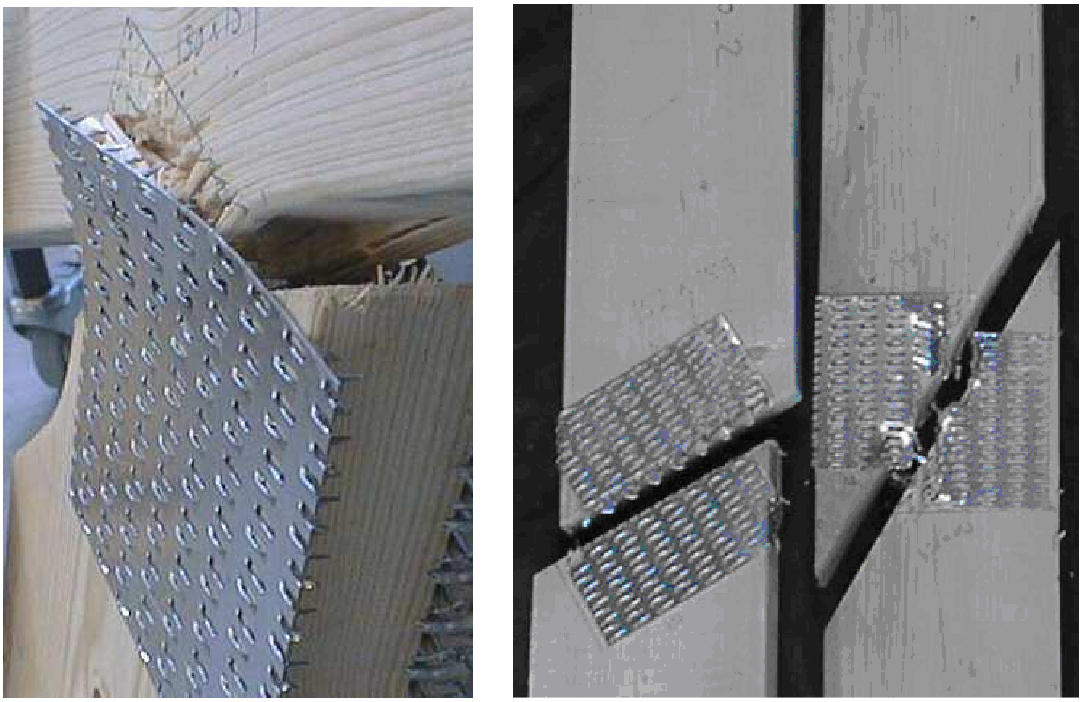


Fig. 1 Cases of failure of punched metal plate connections

The second failure possibility is related to the plates. It occurs when the forces transmitted through them surpass their strength properties. Depending on the kind of force working on the plates, it could be a compression, a tension or a shear failure. The former implies the buckling of the plate and the others its breakage. The right part of the Fig. 1 shows an example of the last failure mechanisms, where breakage appears due to the combination of shear and tension stresses. In these cases, the solution is to adequately modify the geometry, in order to enlarge the length of the plate along the connection line.

In order to avoid these possible failure mechanisms, different countries have standards to regulate the correct design of the joints. In Europe, the standards in charge of establishing a set of harmonised technical rules for the design of construction works, which would serve as an alternative to the national rules in the member states, are the Eurocodes, and for timber structures, specifically, the Eurocode 5 (CEN, 2004). This standard prescribes two verifications related to the failure mechanisms described above. On one hand, in relation to the anchorage capacity between plate and timber,

$$\left(\frac{\tau_{F,d}}{f_{a,\alpha,\beta,d}}\right)^2+\left(\frac{\tau_{M,d}}{f_{a,0,0,d}}\right)^2\le 1$$

Equation 1

and, on the other hand, related to the strength capacity of the plate under the transmission efforts,

$$\left(\frac{F_{x,d}}{R_{x,d}}\right)^2+\left(\frac{F_{y,d}}{R_{y,d}}\right)^2\le 1$$

Equation 2

The stresses $\tau_{F,d}$ and $\tau_{M,d}$, and the forces $F_{x,d}$ and $F_{y,d}$ are calculated from the design force $F_{A,d}$ and moment $M_{A,d}$ acting on each single plate. In particular, $\tau_{F,d} = F_{A,d}/A_{ef}$, where the effective plate area A_{ef} is the area of the total contact surface between the plate and the timber reduced by 5 mm on the perimeter of the plate, to take into account that the plate doesn't work properly at the limits. Additionally, the constants $f_{a,\alpha,\beta,d}$, $f_{a,0,0,d}$, $R_{x,d}$ and $R_{y,d}$, which characterize the plate strength properties, are obtained from tests carried out in accordance with the European Standard EN 1075 (CEN, 2004). The last two constants also depend on the length of the plate along the connection line. Accordingly, the geometric data that characterize the strength behaviour of the joint for every plate type are basically the last length indicated and the effective area A_{ef}.

The rules above only apply to punched metal plate fasteners with two orthogonal directions, the x-direction or main direction of the plate, and the y-direction perpendicular to it. The joint shall also comprise two plates of the same size and orientation, placed on each side of the timber members. Furthermore, Eurocode 5 establishes the conditions so that the joint can be considered to be a pinned one. Finally, the standard indicates that the contact pressure between timber members may be taken into account in order to reduce the value of F in compression, provided that the gap between the members has an average value that is not greater than 1.5 mm, and a maximum value of 3 mm. In such cases the connection should be designed for a minimum compressive design force of $F_{A,min,d/2}$.

FINITE ELEMENT MODELS

Due to the inherent variations in properties and the non-linearities which result from the constitutive laws governing timber and its interaction with other materials, experimental research procedures to study joints with punched metal plates become complex as well as extremely expensive, and the conclusions reached are not always very reliable. Thus, in these cases, it is advisable to take advantage of the new methodologies based on computer technology, especially those that use the finite element method (Mackerle, 2003). The application of this tool implies developing a model of the real system, which means generating the geometries, materials, loads and constraints in order to approximate to the real case as closely as possible.

In this study, several finite element models, which allowed a detailed analysis of the stress state in the different parts of the joints, were developed using the commercial software ABAQUS. These models were obtained from an initial one, modifying in each case the values of the variable in study within a specific range, through programmed routines written in PHYTON language (van Rossum, 2006). This made it possible to discover the influences of these variables.

Materials

In the real world, timber is an anisotropic material whose mechanical properties depend on the direction in which they are measured in relation to the grain direction (Thelandersson, 2003). In the procedures, codes and standards used to design timber structures, a simplification of its real behavior is adopted, considering the material as linear, elastic and orthotropic, with three principal directions: axial (L), radial (R) and tangential (T). With this assumption, according to Hooke’s law, nine constants define its behavior. Their values in the structure studied in this work, built of C18 timber class according to European Standard EN 338 (CEN, 2003), are indicated in Table 1. The material shows good strength properties in the axial or parallel-to-the-grain direction, and lower values in the other directions. In the finite element models, the timber’s mechanical behaviour was approximated with a 3D linear elastic orthotropic material, whereas the steel plates were approximated with a 2D linear elastic isotropic material (Stehn, 2004).

Table 1 Nine linear elastic orthotropic constants that define the behavior of the C18 timber (N/mm^2)

	EL	**ER**	**ET**	**υLR**	**υLT**	**υRT**	**GLR**	**GLT**	**GRT**
C18	9000	562	562	0.41	0.41	0.51	560	560	83.6

Geometry, Load and Constraints

In order to carry out the study, a representative case of a triangulated truss was proposed with realistic values for the constraints, geometries and loads that may appear in it (Herzog, 2003). The geometrical data of this structure was the following: span 6 m; slope 18.44º; section of the beam 89 mm in depth by 38 mm in width. Also, a Service Class 2 is assumed to take account the environmental conditions (CEN, 2004). To calculate the possible working combinations, the following loads were considered: a set of loads comprising the dead load, the imposed load, wind and snow. The values of each of these single loads were calculated following the Eurocode 1 (CEN, 2001). The structure was checked using the most unfavorable load combinations, verifying that it complies with all the Eurocode 5 requirements. With all these conditions, the structure can be considered as pin-joined, so there is no bending moment on the joints, and a force of 9357 N was determined to be acting on the heel joint.

The heel joint was built with a pair of steel plates of 79.36 mm by 71.4 mm, and 1.25 mm thick, on opposite faces of the timber elements and symmetrically arranged over the connection line. With the intention of studying only this joint, the model shown in Fig. 2 was considered. It was called the initial model, and constitutes the starting point for carrying out the analysis. In the following stages, variations in the dimensions of the plates and in their vertical and horizontal positions are considered.

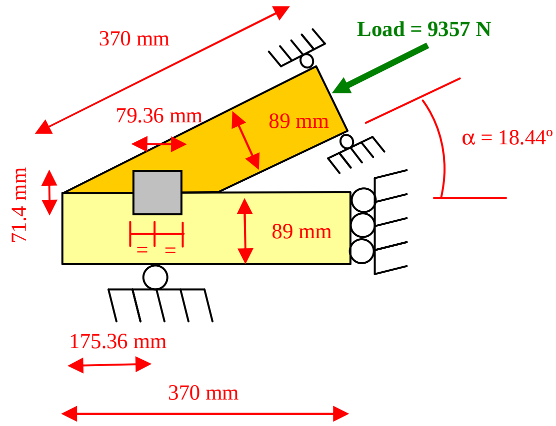


Fig. 2 Model of the joint selected for the study

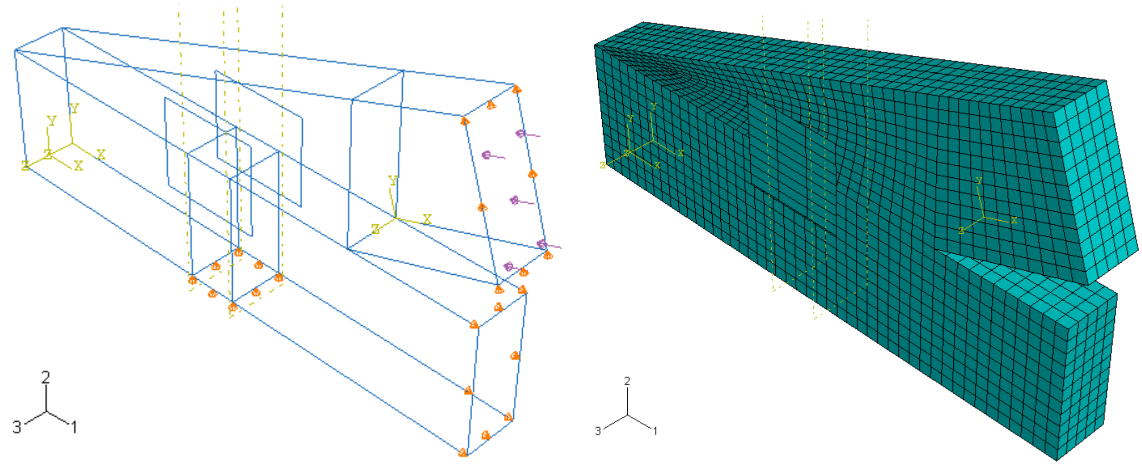


Fig. 3 Finite element model with parts, loads, constraints, and its mesh

Mesh

The joint was modeled using "C3D20" volume elements for the timber parts and "S4R" shell elements for the steel plates, from the ABAQUS element library (Abaqus, 2006). In the timber parts, the mesh was designed as uniformly as possible, and a convergence study was developed in order to establish which mesh size was accurate enough without seriously penalizing the computer possibilities. The decision was to take a mesh with 8 mm edge length elements. Additionally, in the plates, the elements were chosen with rectangular shape in order to obtain nodes in the positions where the nails of the real plate appear, and hence their size was also defined taking this issue into account. A correct definition of the directions that specify the orthotropic behavior of the material was essential. Accordingly, different local coordinate systems adapted to the direction of the fiber were described in each part of the model. Fig. 3 shows the model with its parts, loads, constraints and coordinate systems, together with the meshed model.

The effective thickness used for the shell elements was calculated from the nominal thickness of the plate multiplied by a ratio smaller than one, which takes into account the presence of the stamped holes in the plate. In the initial model, the plates of 1.25 mm nominal thickness are meshed with shell elements of 0.815 mm effective thickness. This thickness was calculated taking into account the thickness of a plate without holes, but with the same length, height and amount of material as the real plate. Also, following the Eurocode 5, it was considered to be an effective plate area, reducing the dimensions of the real one by 5 mm around the entire perimeter. Thus, in the initial model, the plates of 79.36 mm by 71.4 mm were modelled as having only 69.36 mm by 61.4 mm.

Finally, it should be noted that the model was programmed to take into account the possible direct contact between the timber members in the joint zone. If the nodes of one of the timber surfaces attempt to penetrate the other element surface, the transmission of contact pressure takes place.

RESULTS AND DISCUSSION

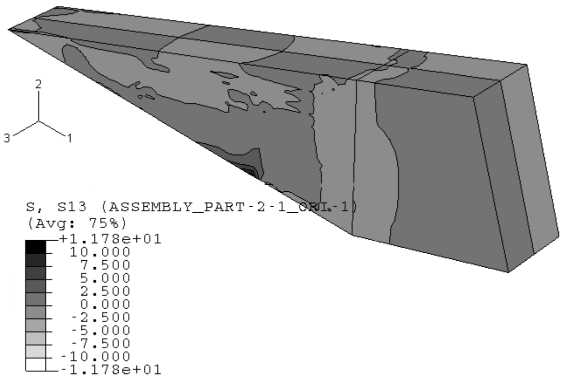
This section reports the results obtained from the studies carried out in this research. First of all, the stress distribution of the initial case represented in Fig. 2 will be displayed, then the results from the models with the modifications of the different parameters indicated in section "Geometry, Load and Constraints", studying how their variation affects the stress level in each case.

Initial case Stresses

As indicated in section "Rules for Designing Punched Metal Plate Fasteners", there are two principal kinds of failures in a joint made of metal punched plates, both types of failures are controlled by the levels of certain types of stresses. These stresses are, on one hand, the anchorage stresses S13 between the timber members and the plates, and, on the other hand, the capacity stresses S22 and S12 which appear in the plates. Fig. 4 shows their distributions for the initial case, and also indications of their maximum and average values. The normal stresses S22 and shear stresses S12 and S13 are referred to in the global axes represented in Fig. 3, and also indicated in the different images of Fig. 4.

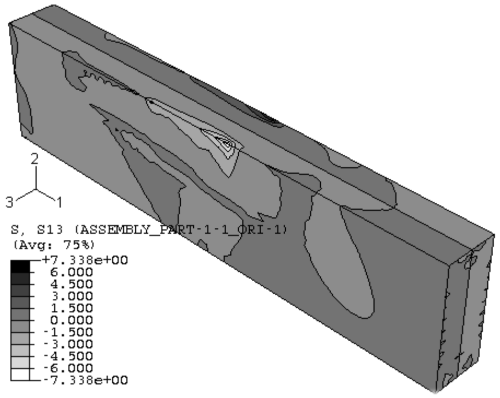
S13 STRESSES IN THE RAFTER

Maximum: 11.776 Average: 1.157



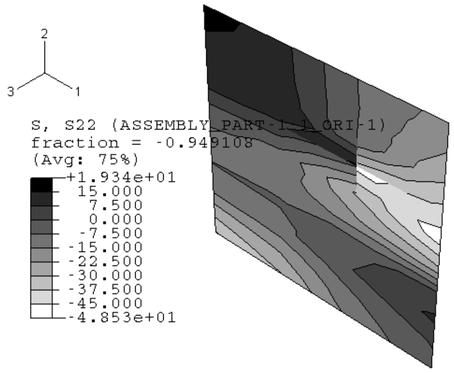
S13 STRESSES IN THE TIE

Maximum: -7.338 Average: -1.389



S22 STRESSES IN THE PLATE

Maximum: -48.53 Average: -11.798



S12 STRESSES IN THE PLATE

Maximum: -53.4 Average: -51.520

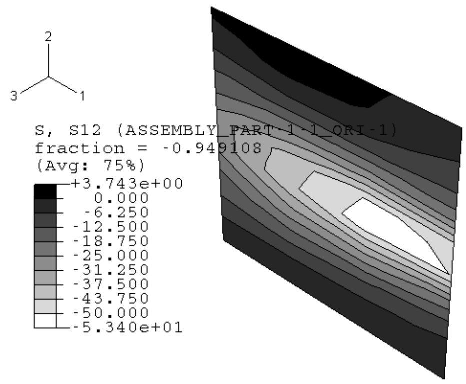


Fig. 4 Results of the stresses in the two timber members and in the plates (N/mm^2)

Influence of the X-Position of the Plate

In order to see how the variation of the x-position of the plate can affect the stresses, different models were prepared by changing the plate horizontal position from the initial case, in the range between $-\frac{3}{4}$ and $+\frac{4}{4}$ of its own effective length (69.36 mm), as can be seen in Fig. 5. Fig. 6 shows the variation of the average anchorage stresses in the tie and in the rafter when the x-position changes, whereas Fig. 7 represents the average plate stresses for the same variable modification.

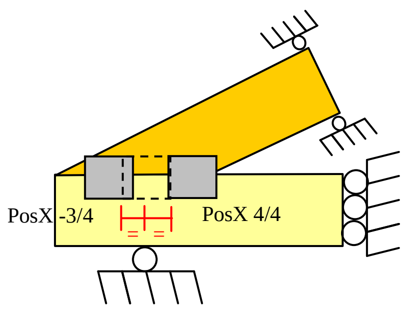


Fig. 5 The plate, changing its x-position between -3/4 and +4/4 of its length

By observing the anchorage stresses, it can be seen that the x-rearrangement of the plates hardly has a significant effect on the stress levels in the rafter, where there is a slight variation, always below 10%, but even this is erratic and without a clear trend. Additionally, the level of the stresses in the tie shows a maximum close to the position $+\frac{2}{4}$ of the effective plate length, and a minimum of approximately a 15% variation when the plate is distant from the initial case position. Regarding the plate stresses, Fig. 7 shows that S22 stresses increase when the plate moves to the right, whereas S12 stresses, which are the greatest and most important stresses in this case, hardly show variations with the x-rearrangement of the plate. To sum up, even when an x-position change can mean a remarkable variation in the level of the different stresses, the most important ones in this case, which seem to be the S12 stresses, do not show any special alteration.

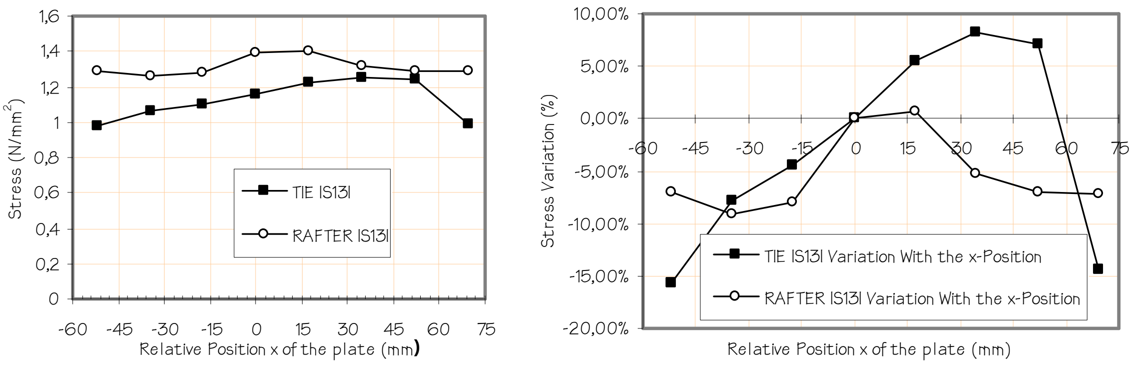


Fig. 6 Variation of the average anchorage stresses with the x-position of the plates

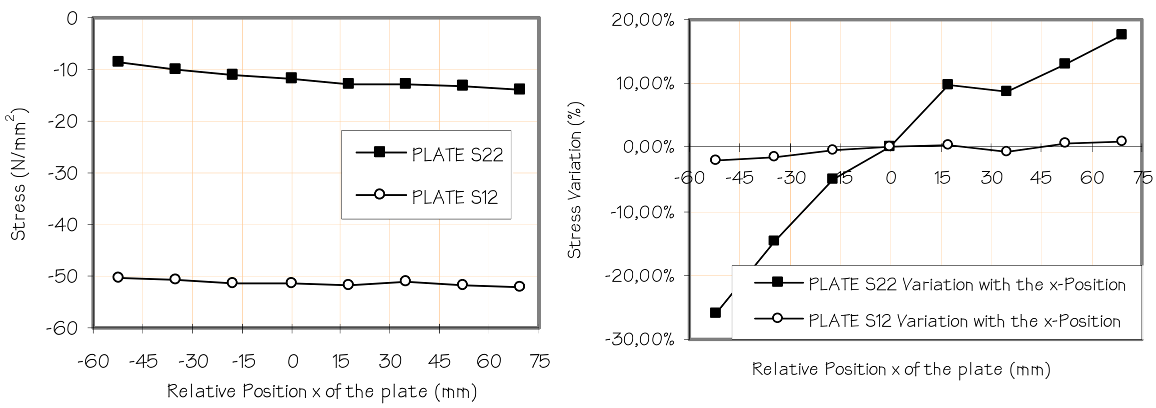


Fig. 7 Variation of the average plate stresses with the x-position of the plates

Influence of the Y-Position of the Plate

Taking into account that the plates are arranged more or less symmetrically over the contact line between the timber elements, there is no reason to consider great variations in the plate's y-position. However, a small vertical displacement can cause important stress variations, because the zone of the plate situated over the contact line (where the S22 and S12 stresses are maximum, as can be seen in Fig. 4, may be reduced from a continuous one (dashed line in Fig. 8) to the stretches between the holes (dotted line in Fig. 8). In the former case, the entire section of the plate with a nominal thickness of 1.25 mm is at work, whereas in the latter, reckoning the proportion between holes and material, will be equivalent to a supposed plate with a thickness of 0.361 mm. An intermediate case with the effective thickness defined for the initial model (0.815 mm) was also considered. The results of the average stresses in the tie, rafter and plates when the plate thickness varies in the range indicated are shown in Fig. 9 and Fig. 10.

In fact, because it was supposed that the y-rearrangement of the plate was simulated through a variation of the thickness of the plate in the contact line, the anchorage stresses should not play a role in this part of the study. However, since the results from these stresses were available, they have been represented in order to check that the thickness modification really has no effect over the anchorage stresses (Fig. 9). What is really of interest in this study are the stresses in the plate and their variation with the thickness. The stresses in the plate change notably with this variable. Both S22 and S12 stresses increase when the thickness decreases. However, because these last stresses are greater, their variation is the most important. In both cases the relationship is nearly linear.

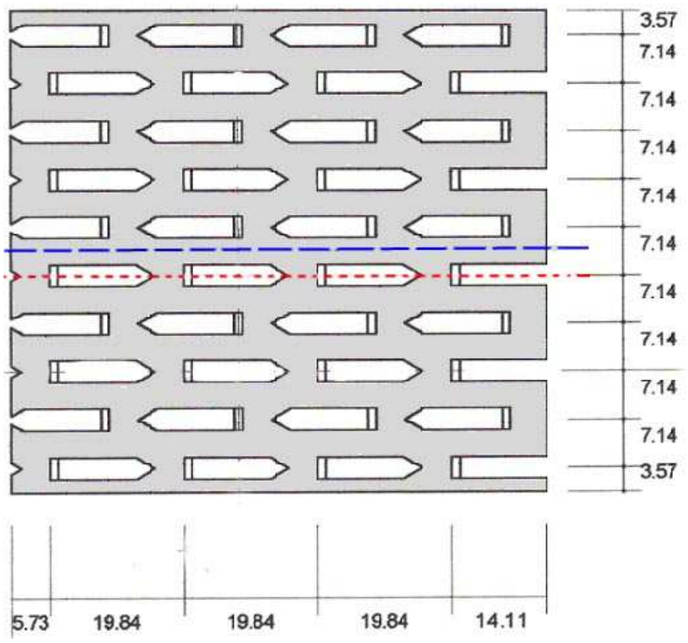


Fig. 8 Different contact lines when the y-position changes

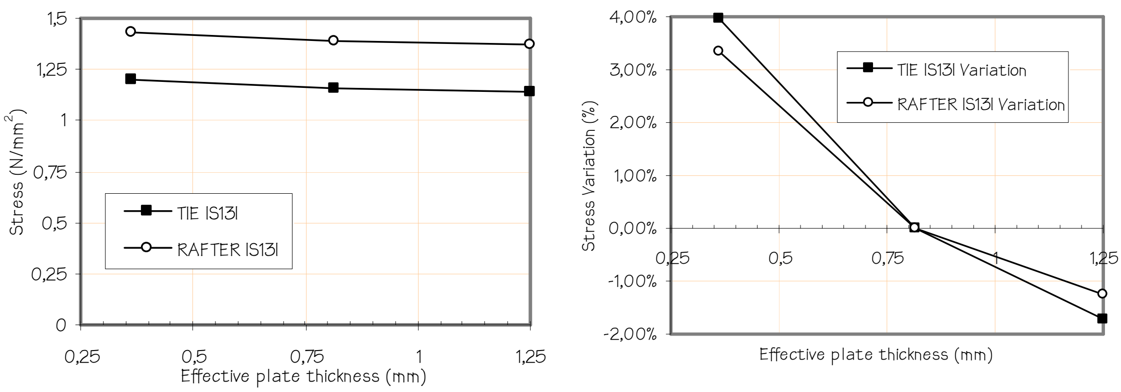


Fig. 9 Variation of the average anchorage stresses related to the y-position of the plates

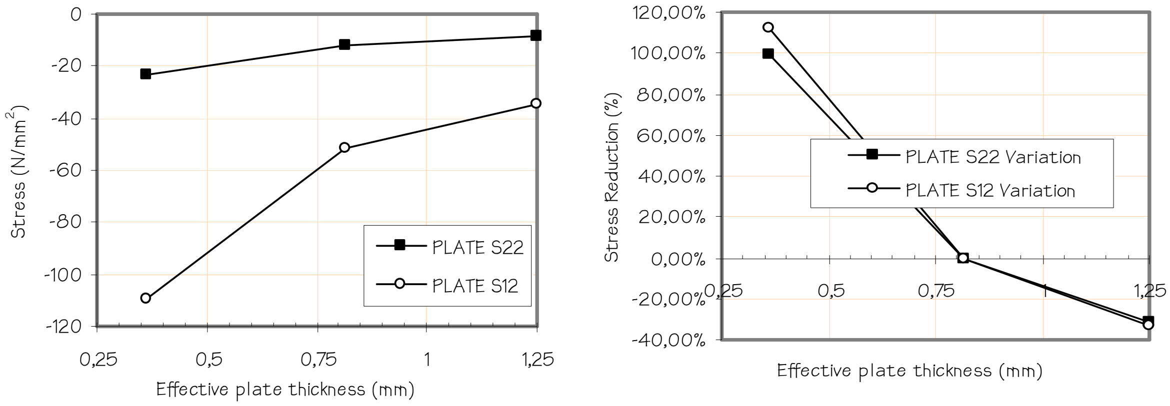


Fig. 10 Variation of the average plate stresses related to the y-position of the plates

Influence of the Plate Height

This third study tries to establish the relationship between the plate height and significant joint stresses. The joint was modeled several times, changing the height of the plate. This variable was calculated taking into account the number of holes in the vertical direction, as can be seen in Fig. 11, Fig. 12 and Fig. 13 show the results of the different stresses when the plate height changes.

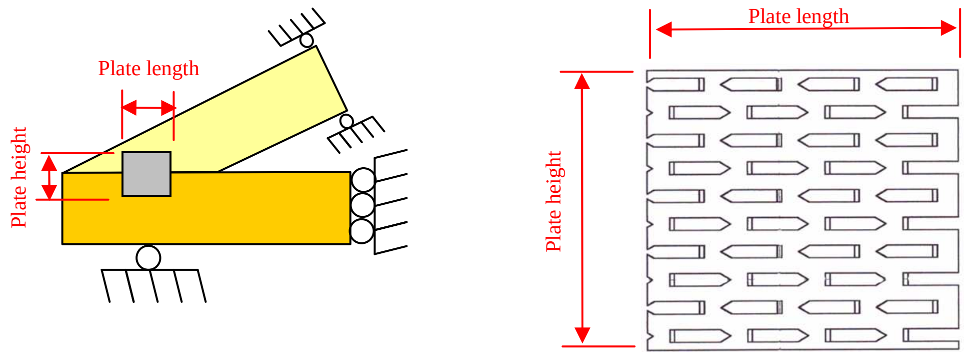


Fig. 11 Plate height and length as a function of the number of holes

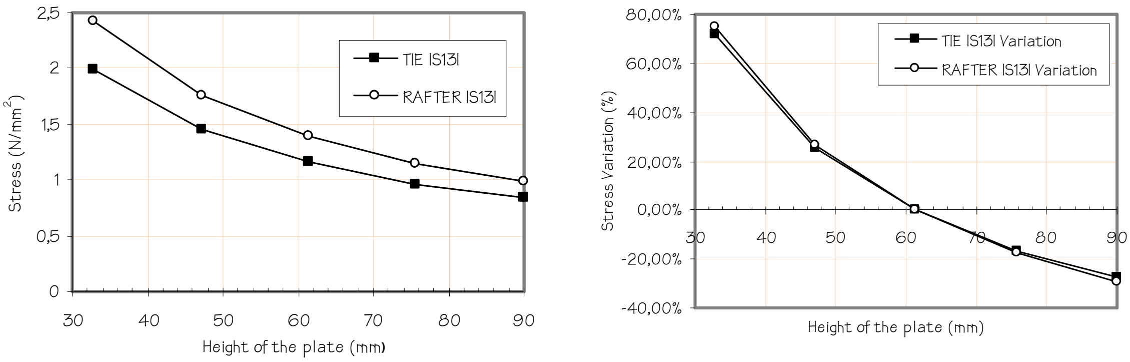


Fig. 12 Variation of the average anchorage stresses with the plate height

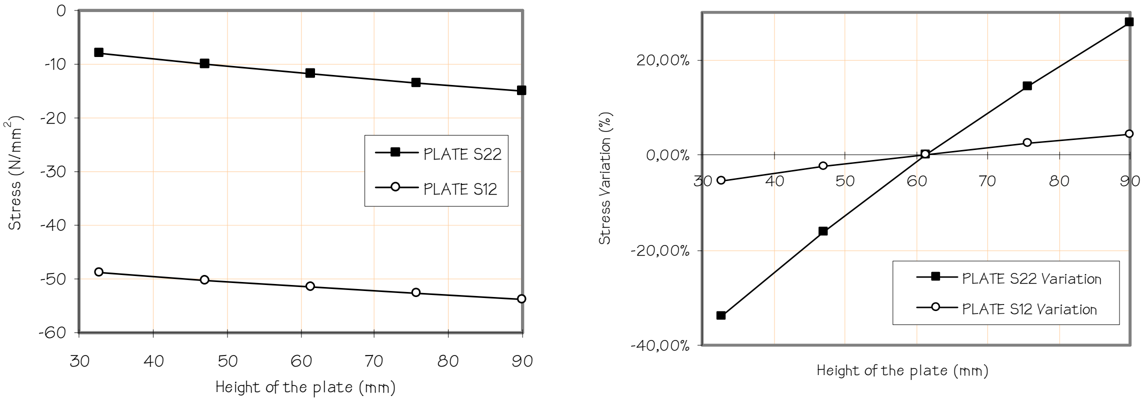


Fig. 13 Variation of the average plate stresses with the plate height

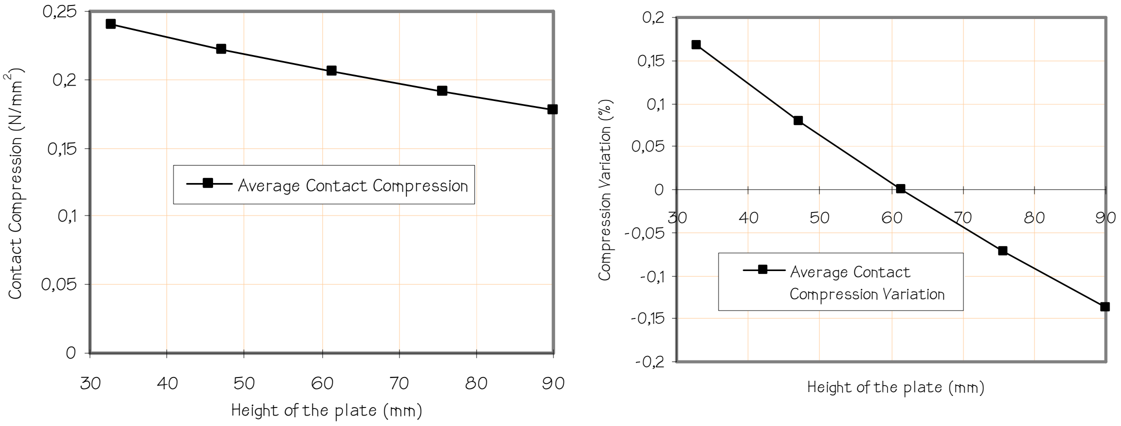


Fig. 14 Variation of the contact compression stresses with the plate height

The anchorage stresses increase when the height diminishes with almost the same trend and values as in the rafter and in the tie. This variation is not linear, and its slope increases dangerously for small heights. This effect is clearly related to the reduction of the area responsible for transmitting the forces between the timber and the plate. Additionally, the effect of the plate height variation on the plate stresses is very appreciable for the S22 stresses but hardly significant for the S12 ones (Fig. 13). Really, the absolute increment of the stresses is practically the same for both kinds of stresses but, because the S22 stresses are approximately five times smaller than the S12 ones, the relative increment of the S22 stresses seems to skyrocket compared to the relative increment of the S12 ones. The justification for this stress increment lies in the augmentation of the rigidity of the joint when the plate height increases (Larsen, 2000). Because the modulus of elasticity of the plate steel is greater than that of timber, the two timber pieces take contact with more difficulties when the height of the plate grows. This makes the transmission of the compression efforts through the direct contact between the timber elements diminish in favour of their transmission through the plates, as can be seen in Fig. 14.

Influence of the Plate Length

Finally, this last part of the work shows how the plate length affects the joint stresses. This time the joint was modelled several times, varying the length of the plate according to the number of horizontal holes represented in Fig. 11. Fig. 15 and Fig. 16 show the results of the different stresses when the plate length changes.

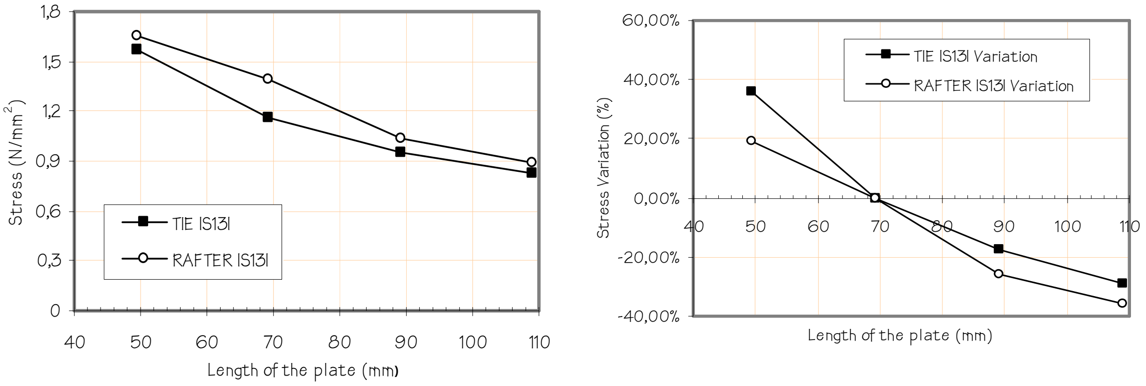


Fig. 15 Variation of the average anchorage stresses with the plate length

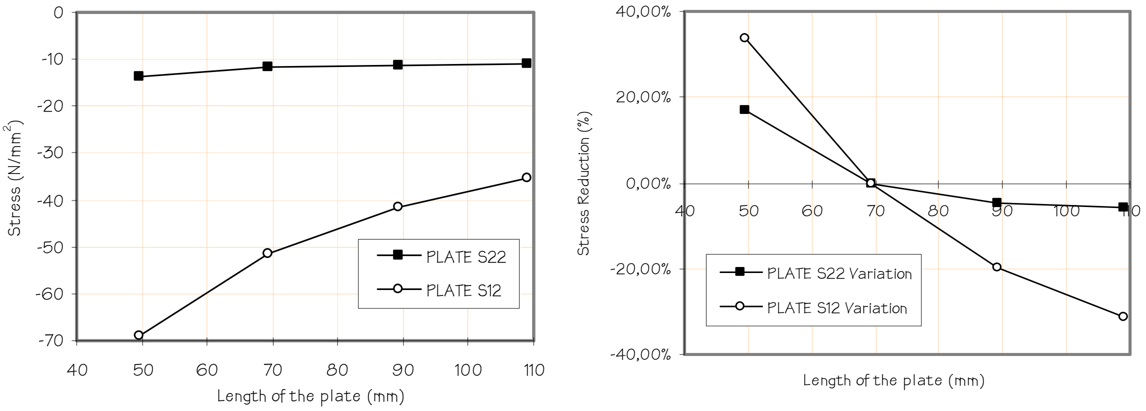


Fig. 16 Variation of the average plate stresses with the plate length

As in the case of height modification, the anchorage stresses increase when the plate length diminishes with the same trend in the rafter and in the tie, but this time with a little difference in the percentage. This effect is also related with the reduction of the area that transmits the forces between the timber and the plate. In the plate, the effect of the length variation is appreciable in both S12 and S22 stresses, but especially in the former (Fig. 16). The reason is that the value of these stresses is inversely related to the length of plate measured along the contact line between the timber pieces, which is precisely the variable being considered in this study. Furthermore, as in the case of height variation, the increase of the plate length makes the joint more rigid, diminishing the transmission of the forces through the compression contact and increasing the transmission of the forces through the plates, and consequently their associated stresses. But this effect is counteracted by the stress reduction due to the length increase indicated above.

CONCLUSIONS

The following conclusions can be derived from the present study:

Whereas a variation in the position of the plates hardly has a significant effect on the anchorage stress levels, the stresses change differently in the plate. With a modification in the x-position, only the S22 stresses are affected, whereas the S12 stresses hardly change. Regarding the y-position, if it changes, the effect is very significant in both S12 and S22 stresses. This effect is not really related to a redistribution of the forces due to the new position of the plate, but rather to the relative location of the contact line with respect to the holes of the plate. If the vertical position of the plates is not appropriate and the contact line between the two timber pieces meets the position of the holes, the area for transmitting the forces becomes extremely reduced and the stresses skyrocket. This point must be carefully taken into account when the joint is assembled.

On the contrary, regarding the position, if the plate size changes, its effect on the anchorage stresses is remarkable. This effect occurs with modifications in both the height and the length, and is related to the variation of the area which transmits the anchorage stresses between the timber and the plate.

With regard to the plate height variations, the plate stresses change practically in the same absolute values but, because the S22 stresses are sometimes smaller than the S12 ones, the relative increment of S22 stresses seems to skyrocket compared with the S12 ones. Moreover, when the plate height increases, the joint becomes more rigid, diminishing the transmission of the compression forces through the direct contact between the timber members in favour of their transmission through the plates, which means an increment of the plate stresses.

When the plate length increases, both S12 and S22 stresses decrease. The reason is that the value of these stresses is inversely related to the length of the plate measured along the contact line between the timber pieces. Also, in this case, the increase of the length makes the joint more rigid, diminishing the contact compression and increasing the stresses in the plates. But now this effect is counteracted by the stress decrease mentioned above.

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REFERENCES

ABAQUS, User’s manual, version 6.6 Documentation. Pawtucket (USA): Hibbitt, Karlsson & Sorensen, Inc., 2006.

Argüelles R. Arriaga F. Martınez J.J. Estructuras de Madera. Diseño y Cálculo. [Timber structures. Design and Calculus.] Asociación de Investigación Técnica de las Industrias de la Madera y Corcho (AITIM), Madrid, Spain, 2003.

CEN. Actions on Structures. EN 1991. European Committee for Standardization, Brussels, Belgium, 2001.

CEN. Structural Timber Strength. Classes. EN 338. Eurocode 1. European Committee for Standardization, Brussels, Belgium, 2003.

CEN. Design of Timber Structures. Part 1-1. EN 1995-1-1 Eurocode 5. European Committee for Standardization, Brussels, Belgium, 2004.

Ellegaard P. Analysis of timber joints with punched metal plate fasteners, PhD thesis. Aalborg University, Denmark, 2002.

Karadelis J. N., Brown P., Punched metal plate timber fasteners under fatigue loading. Construction and Building Materials, 2000, 14, p. 99-108.

Herzog T. Natterer J. Schweitzer R. Volz M. Winter W. Construire en Bois. Presses Polytechniques et Universitaires Romandes, Lausanne, Switzerland, 2003.

Larsen HJ. Jensen J.L., Influence of semi-rigidity of joints on the behaviour of timber structures. Progress In Structural Engineering And Materials 2000, 2, 267-277.

Mackerle J. Finite element analysis of fastening and joining: A bibliography (1990–2002). International Journal of Pressure Vessels and Piping, 2003, 80, p. 253-271.

Stehn L. Borjes K. The influence of nail ductility on the load capacity of a glulam truss structure. Engineering Structures, 2004, 26, p. 809–816.

Thelandersson S. Larsen HJ. Timber Engineering. Wiley & Sons Ed., 2003.

USDA Wood Handbook - Wood as an Engineering Material. USDA, Forest Service, Forest Products Laboratory, Madison, WI., 1998

van Rossum G. Python Tutorial. Python Software Foundation, 2006.