**CZECH TECHNICAL UNIVERSITY IN PRAGUE**
**FACULTY OF CIVIL ENGINEERING**
**Department of Steel and Timber Structures**

**BACHELOR THESIS**

# Metal Work Used in Timber Engineering

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**2010**

**Supervisor: Doc. Ing. Petr Kuklík, CSc.**

## Honesty Declaration

I declare that this bachelor thesis has been carried out by me and only with the use of materials that are stated in the literature sources.

Prague, 14 May 2010 Michael Somr

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## Acknowledgement

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## Abstract

It is commonly stated that “a structure is an assembly of joints separated by members” and in timber engineering the joint is generally the critical factor in the design of the structure.

The strength of the structure is normally determined by the strength of the connections. Their stiffness greatly influences the displacement behaviour. Member sizes are often determined by the number and physical characteristics of the type of connector which is used, rather than by the strength requirements of the member material.

This thesis can be perceived as a guide through the world of the design of steel-to-timber connections using dowel type fasteners, with a short overview of other possible ways how to make joints in timber structures.

Since timber structures have become very popular, the knowledge of a fast and convenient jointing by means of the metal connectors is very important. The author believes that this work can help those who are interested in this issue to understand it.



**Keywords:** steel-to-timber connection, dowel type fastener, timber joint, shear resistance, axial withdrawal resistance



## Abstrakt

Běžně se uvádí, že „konstrukce je soustava styčníků oddělených prvky“ a v dřevařském inženýrství je styčník obecně kritickým místem v návrhu konstrukce.

Pevnost konstrukce je dána pevností spojů a jejich tuhost významně ovlivňuje deformace. Velikosti prvků jsou často určeny spíše počtem a fyzikálními charakteristikami spojovacího prostředku, než pevnostními požadavky na materiál prvku.

Tato práce může být vnímána jako průvodce světem navrhování spojů dřevo-ocel pomocí spojovacích prostředků kolíkového typu, spolu s krátkým přehledem dalších způsobů spojování dřevěných prvků.

Vzhledem k tomu, že se dřevěné konstrukce staly velmi populárními, je znalost rychlého a pohodlného stykování s využitím kovových spojek velmi důležitá. Autor věří, že tato práce může pomoci porozumět této problematice těm, kteří se o ní zajímají.



**Klíčová slova:** spoj dřevo-ocel, spojovací prostředek kolíkového typu, dřevěný spoj, únosnost ve střihu, únosnost na vytažení

# Contents

# 1 Introduction

In the recent years, we have been able to observe a progress of timber structures in the Czech Republic which was induced by a new European trend of the sustainable development and good resources of the material (among the European countries, the Czech Republic is the fourth one in the forest coverage per hectare and the sixth one in the annual increment per hectare). Other incentives are also efforts of investors to speed up an erection together with a minimization of wet processes on a building site.

Fig. 1.1: Olympic oval, Richmond, Canada

Timber, as a building material, has been used for ages and not only for its good mechanical properties, ease of a processing and renewability of its natural resources. The huge evolution of timber technologies brought also a development of other branches of an industry. For example new adhesives, which enable a production of timber based materials, were evolved and there are also drying kilns and sorting machines operated by computers. The new timber based materials and sorted or treated timber become a perfect material even for the most complex building structures like churches, theatres, exhibition halls, etc. An example is shown in Figure 1.1.

The largest area of a utilization of timber is in a housing construction, where it is possible to use it either for whole load-bearing structure or at least for roof structure. Recently, there have been obvious tendencies to use wood based panels instead of solid timber even for load-bearing elements. The usage of these elements with small thicknesses is limited by their connection possibilities. Traditional carpentry joints are not suitable for the wood based panels, because notches and mortices weaken the connected elements.

An alternative to the carpentry joints which can be used even in cases of thin elements are metal connectors. These steel elements are an analogy to punched metal plate fasteners (also known as gang-nail). It is very easy to use them just in-situ and moreover, they enable a connection of timber elements to steel or concrete structures. A production of metal connectors started in the Czech Republic in the 1990's.

A number of structures using timber either as main load-bearing material or material for other part of structure were steeply increasing. Even though the last few years were affected by financial crisis there is still a huge amount of money in this branch of an industry. (A development of a production of prefabricated timber structures and their parts is illustrated in Figure 1.2.) Therefore, there is a space for a development of new structural elements and connectors or new wood based materials. In nowadays, it is possible to see many structures of a bold design made of relatively slender elements which look elegant and at the same time they satisfy all structural criteria. A load-bearing structure can work well if and only if it behaves as a complex. Therefore, a design of joints is important for the modern and slender structures. These structures are possible to build mainly from the reason that new types of

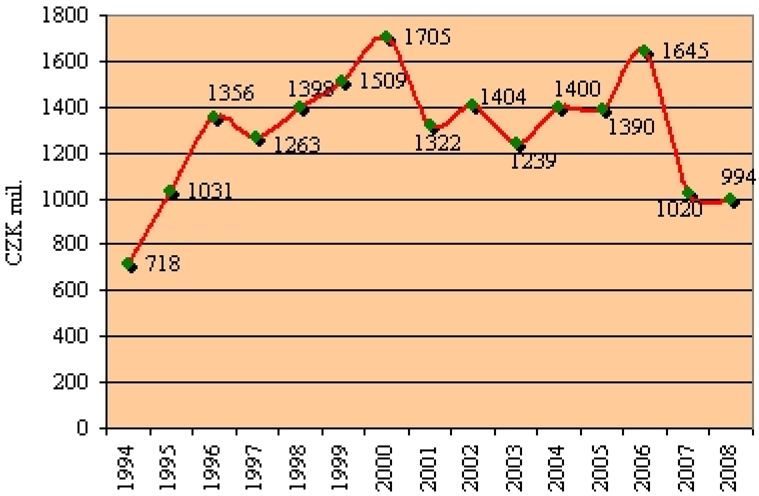



*Fig. 1.2: Development of a production of prefabricated timber structures and their parts*

connectors originated and they replace present connectors which are intended mainly for massive elements.

If we do not take into account current technological possibilities during a wood processing and sawn timber production, then wood as a raw material is, with its physico-mechanical properties, actually unchanged for centuries. In the past, a limiting factor for using timber structures were connections of individual structural elements. These limits were gradually diminished and in nowadays, civil engineers can design a wide variety of joints which are elegant and have high load-bearing capacity.

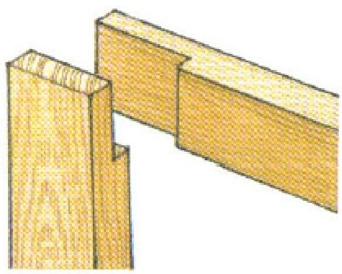
# 2 Present Ways of Connecting

Timber structures are formed by connecting timber elements so that they create a basic load-bearing structure. Strength and durability of timber structures is directly dependent on the strength of the connections. Joints must therefore be made accurately and using appropriate fasteners. Timber jointing is dependent on a mutual position of elements, their size, size of stresses and the way the stresses are acting on the joint. For a quick connection of two or more timber elements, mechanical fasteners such as nails, screws and dowels are used. Timber elements can be also connected by various adhesives. The traditional way of connecting timber elements are carpentry joints.

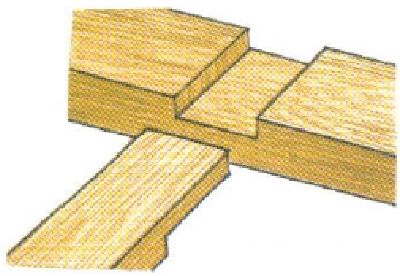
In the following part of this chapter will be described the basic ways of connecting timber elements, which are presently used, in more details.

## 2.1 Carpentry Joints

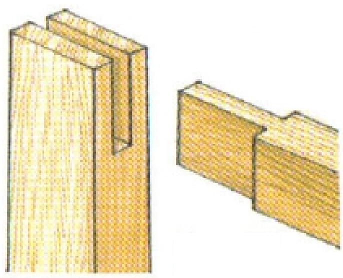
This kind of connections undoubtedly belongs among the oldest types of joints which we can see on timber structures. They are used since the beginning of timber utilization. The carpentry joints are usually designed as contact ones, it means that internal forces in the structure are transferred by means of a compression of frontal areas of the connected elements, eventually by their friction. It is advisable to complete these joints by dowel type fasteners, either steel or wooden ones, which fixate the joint and stop the borne element from falling out. It is also possible to combine the carpentry joints with the metal connectors. There are many types of the carpentry joints and we can divide them into several subgroups: butt joint, lapping and overlapping, cogging, mortise-and-tenon, bird’s-mouth, embedment. Some of them are illustrated in Figure 2.1.



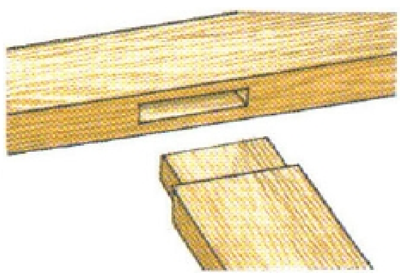
*(a) End lap*



*(b) Centre lap*



*(c) Open slot mortise-and-tenon*



*(d) Mortise-and-tenon*

*Fig. 2.1: Examples of carpentry joints*

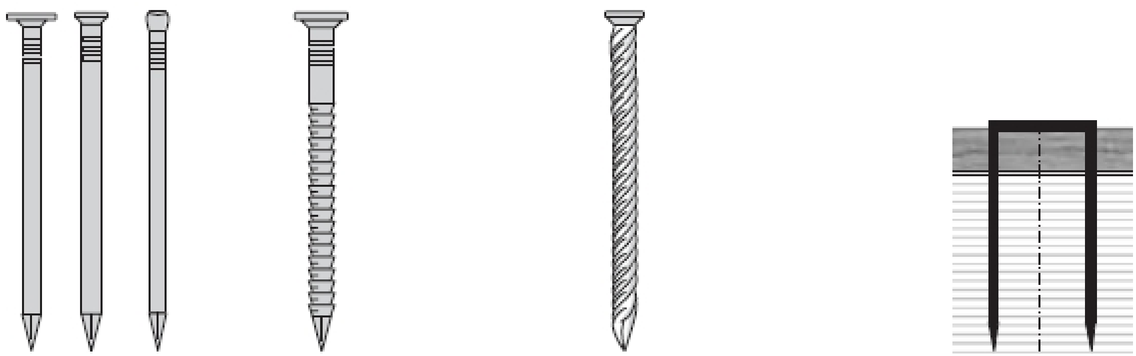
Structures connected by the carpentry joints look compact and elegant. A design and making of carpentry joints have optimized to a satisfactory level during centuries when they were the only way how to connect timber roof trusses or floor structures. Formerly, sizes of tenons and depths of mortises (but also whole structures) were designed empirically. This experience was used during a creation of standards which already contain design procedures for most of the carpentry joints. A main disadvantage of the carpentry joints is that they weaken connected elements whereby they significantly reduce a load-bearing capacity of the elements. The load-bearing capacity is lowered firstly by reduction of a cross-section which resists internal forces and secondly by a possibility of a stress concentration in notches. Another disadvantage is a high labour consumption and consequently high costs of a carpentry joints production. Most of these joints are used during reconstructions and refurbishments and not in new buildings. Since wood is an organic material it has quite significant volumetric changes. Because of timber shrinkage, insufficiently fixated joints can loosen and therefore, lose not only their appearance but also their function.

# 2.2 Dowel Type Fasteners

A transfer of forces among connected elements is realized by means of a shank eventually a head of a connector. There are bending of a shank, bearing of timber below a shank and shear stress acting on timber along a shank due to a loading. These fasteners are very frequently used in common structures in nowadays. Main groups are described thereinafter.

## 2.2.1 Nails and Staples

Nails are the most frequently used and the oldest fasteners. There are many types of nails produced presently, simple smooth annular or square nails made of a wire but also nails with special shanks with a higher withdrawal resistance e.g. annular ring nails, helical thread nails or nails with a cement coating. Pneumatic pistols have been used for a hammering of nails recently; these pistols can hammer a nail 100 mm deep. Alternative types of nails are shown in Figure 2.2 (a)-(c).



*(a) Smooth nails* *(b) Annular ring nail* *(c) Helical thread nail* *(d) Staple*

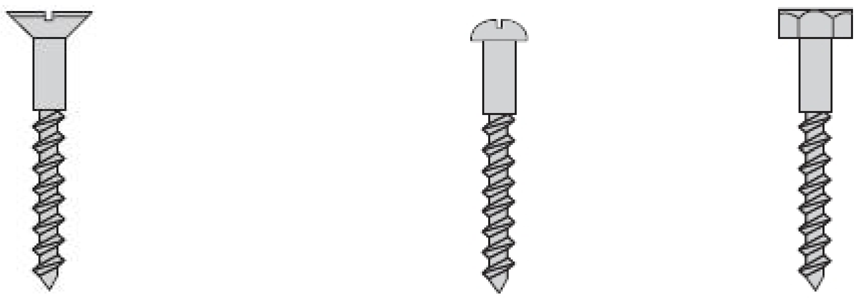
*Fig. 2.2: Common types of nails and staple*

A development of pneumatic pistols has brought a new kind of a fastener – staples. The staple is a U-shaped wire usually used for connecting wood based panel to timber. Such an example is illustrated in Figure 2.2 (d).

Nails are often combined with other types of connectors especially with metal connectors. However, it is necessary to take into account a higher compliancy of nails and other problems like a dependency of a timber grain direction with respect to a loading and a nails location during a design.

### 2.2.2 Screws

Wood screws are used in place of nails in applications requiring higher capacities, in particular in situations where a greater withdrawal capacity is required. They can be used for timber-to-timber joints but are especially suitable for steel-to-timber joints. Screws should always be fixed by a screwing into the timber, not by a hammering into position. Where screws are used in softwood connections and the smooth shank diameter of the screw is 6 mm or less, pre-drilling is not required. Where the diameter is greater than 6 mm in softwood connections and for screws of any diameter in hardwood connections, pre-drilling must be used. Screws can be more complicated than nails from a technological point of view and therefore, they are often replaced by special nails with a profiled shank. The most common types of wood screws are shown in Figure 2.3.



*(a) Countersunk head screw* *(b) Round head screw* *(c) Coach screw*

*Fig. 2.3: Common types of wood screws*

### 2.2.3 Bolts and Dowels

Bolts are easy to use and have a relatively high shear bearing capacity. However, this capacity is affected by a high compliancy of the bolt. That is because the bolts are always put into pre-drilled holes with bigger diameter than diameter of the bolts by 1 mm and because the bolts are cutting through timber (a big load acts on a relatively small area). When the bolts are used, washers are required under the bolt head and under the nut to distribute the loads, and when tightened, a minimum of one complete thread on the bolt should protrude from the nut. Bolts can transfer shear and axial load. Typical bolts are presented in Figure 2.4.

Bolts are sometimes substituted by dowels (mainly from aesthetic reasons). Dowels are steel rods with circular cross-section which are hammered into pre-drilled holes smaller than a diameter of the dowel. Such joints have a lower compliancy but they cannot transfer an axial load. Usual types of dowels are in Figure 2.5.

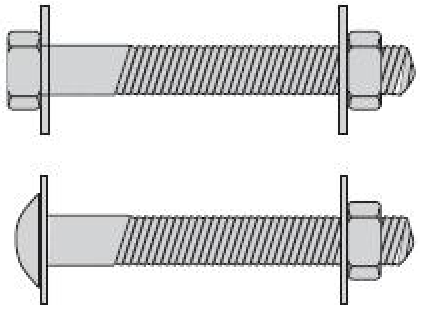


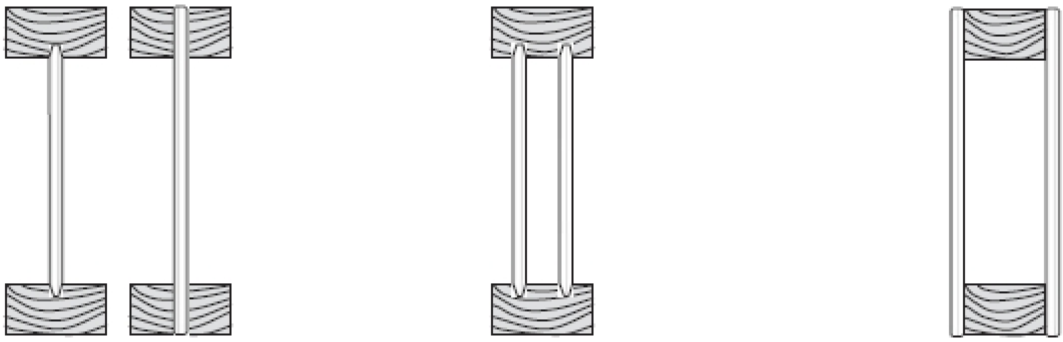
Fig. 2.4: Bolts

Fig. 2.5: Dowels

# 2.3 Gluing

## 2.3.1 Glued Connections

Glued connections are used for longitudinal connections of long-span double-tapered beams and arches or for I, U, box etc. sections. Some of these section, you can see in Figure 2.6. Gradually, adhesives have developed to such level that it is possible to make glued joints. These glued connections are more technologically demanding but they are not as compliant as other connections of timber structures.



(a) I-beams (b) Double I-beam (c) Box beam

Fig. 2.6: Examples of composite timber and wood-based sections

## 2.3.2 Glued-in-Rods

Glued-in-rods are placed especially to glulam structures, where volumetric changes and drought cracks are minimized. These rods are used for a strengthening of apexes of double-tapered beams or places close to notches and openings, where they eliminate a tension perpendicular to a grain. It is also possible to use them as an anchoring of columns to foundations. Such joints are advisable from a point of view of a fire safety because rods are protected by a relatively thick layer of timber. An example of a threaded rod is in Figure 2.7.

They are also an appropriate fastener-system for the connection of CLT (=Cross Laminated Timber) elements. They enable a connection of the narrow sides, in particular for the transfer of higher loads and when loads occur transversal and longitudinal to their axis. Such an application is shown in Figure 2.8.

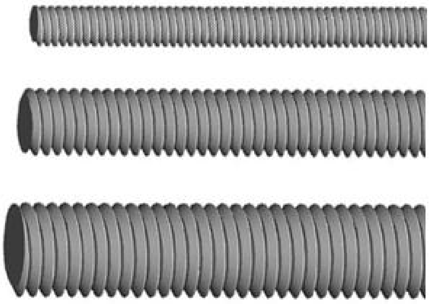


Fig. 2.7: Threaded type of glued-in-rod

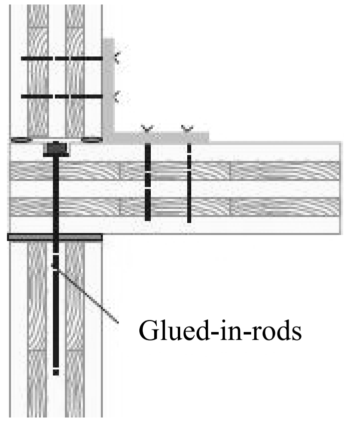



Fig. 2.8: Glued-in-rod in a CLT joint

## 2.4 Metal Connectors

Connections done by means of metal connectors are an alternative to traditional carpentry joints. They work as a contact connection; it means that internal forces in a joint are transferred by the contact between timber and a steel element. This contact is ensured by dowel type fasteners (nails, bolts) and that is the reason why these joints can resist to a tension. If connecting timber elements by the metal connectors, it is not necessary to weaken them by notches and mortices. By means of the metal connectors, even very complicated spatial joints can be created, as shown in Figure 2.9.

Fig. 2.9: Connection detail at Portcullis house, Westminster, London

### 2.4.1 Punched Metal Plate Fasteners

This is a very widespread fastener used especially in a production of prefabricated trusses. It is suitable for connecting two or more timber elements with the same thicknesses. There are always two plates pressed from opposite sites. They are produced from a zinc-coated metal sheet with thickness 0.9-2.5 mm. Since these joints are technologically demanding, their production and design are matter of specialized companies in nowadays, and they deliver already completed trusses. Common types of these fasteners are illustrated in Figure 2.10.

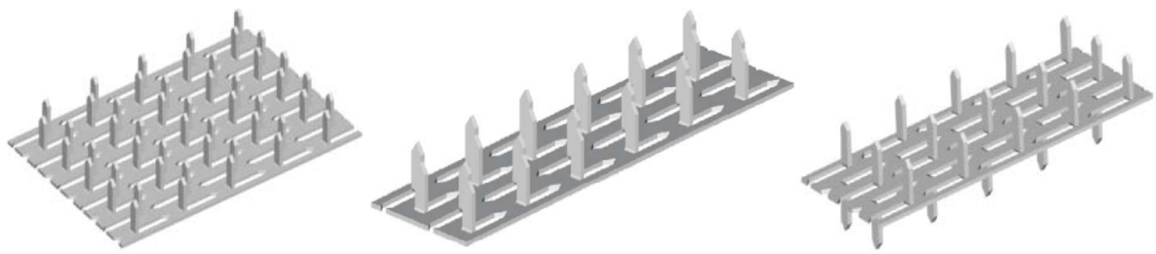


*Fig. 2.10: Typical punched metal plate fasteners*

### 2.4.2 Connecting Plates

These are thin metal sheets with a thickness 1-4 mm made of zinc-coated steel, with perforated holes for dowel type fasteners. Their usage is very similar to the punched metal plate fasteners but it is very easy to use them in-situ. Some of them are shown in Figure 2.11.

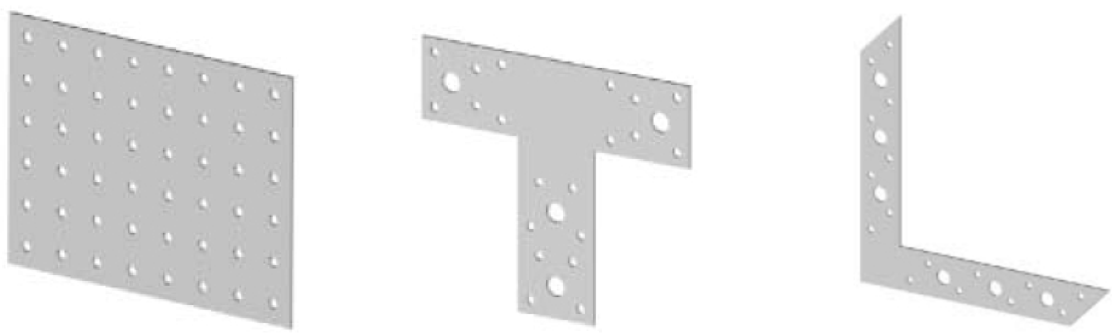
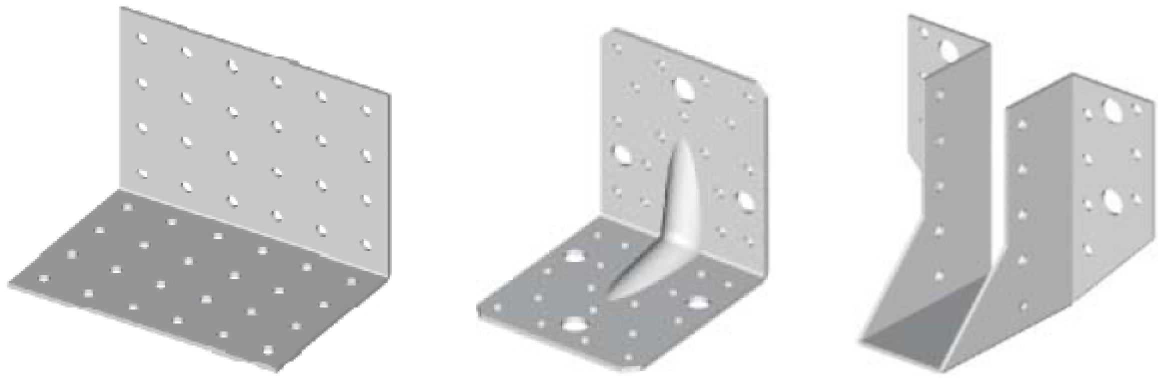


*Fig.2.11: Examples of connecting plates*

### 2.4.3 Shaped Elements

This subgroup includes angles, hangers, etc. They can be used for the same purposes as the previous subgroups, but also for connecting elements with different thicknesses. Shaped elements are made by pressing and punching of zinc-coated metal sheet with thickness 1-3 mm. Legs of the angles are perforated with holes of different diameters either for nails or bolts. Nails with special shanks with high withdrawal resistance are used for a fixation of the elements to timber. The shanks of the nails are conically widened just below the head to ensure a tighter connection between the nails and the steel element. These nails do not need pre-drilled holes. It is also possible to use bolts either to obtain joints with higher load-bearing capacity or for a connection with steel or concrete structures. From the technological point of view these joints are not demanding. Common types of the shaped elements are in Figure 2.12.



*(a) Angle* *(b) Angle with stiffening rib* *(c) Hanger*

*Fig. 2.12: Common types of angles and hangers*

# 3 Goals

The goal of this bachelor thesis is to describe a way how to analyze timber joints made by means of metal work connectors. It is particularly focused on:

- background of formulas used in EN 1995-1-1 and their explanation
- analysis of shear resistance of dowel type fasteners
- analysis of axial withdrawal resistance of dowel type fasteners and its combination with shear resistance
- verification of load-bearing capacity of a chosen metal work connector using the FEM
- programming of an Excel application for an evaluation of load-bearing capacity of joints
- comparison of spacings, and edge and end distances according to Eurocode and ČSN
- evaluation of load-bearing capacities of joints, with respect to timber grades and types of connectors and fasteners

# 4 Evaluation Methods

It is necessary to take into account a big compliancy of the metal connectors which causes big plastic deformations of used elements, before it exceeds its resistance. In the most of cases, a failure of the metal connector is not decisive. It arises from a limited number of openings which are punched in advance. Force acting on the element through the fasteners (nails, screws, bolts etc.) is therefore limited as well. A load-bearing capacity of the joints is influenced not only by strength of the fasteners, but also by a capacity of the weaker timber element. Especially tension perpendicular to the grain, caused by the distribution of the loading through the dowel type fasteners, can be troublesome. This tension very often leads to the failure of the joint before exceeding the load-bearing capacity of the fasteners or the metal connector.

The formulas from Eurocode 5 were mainly used for the following procedures. Nevertheless, the papers and the experiments of several researchers were also used, because EC5 knows only the smooth nails and the other nails. But behaviour of the annular ring nails is sometimes different, and it is necessary to take into account, that for the metal connectors the annular ring nails are, together with screws and bolts, the best choice, thanks to their several times higher withdrawal resistance in a comparison with the smooth nails. The screws can be more work demanding than the nails, especially when the pre-drilling is needed. Therefore, this work and the following procedures will be dealing in particular with smooth and annular ring nails (as the fasteners with a smaller diameter) and the bolts (as the fastener with a greater diameter).

## 4.1 Failure Theory and Strength Equations for Laterally Loaded Connections

When there is a lateral loading, a connection made with help of the dowel type fasteners may fail either in a brittle or a ductile mode. Design rules in EC5 have been developed to ensure that failure will be in a ductile rather than a brittle manner.

The minimum spacings, edge and end distances given in EC5 have been derived to prevent splitting failure. There is also given a procedure, ensuring that the splitting resistance of any member in a connection subjected to a design force at an angle to the grain will exceed the

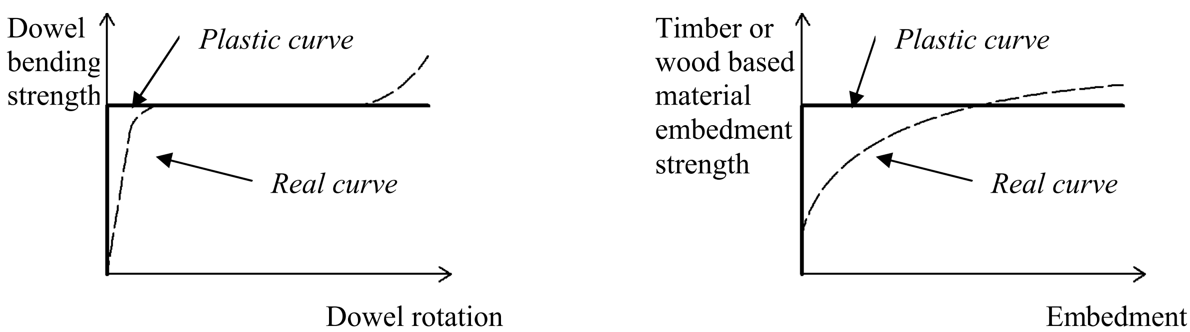



*Fig. 4.1: Strength-strain diagrams used for dowel type fasteners connections*

component of the design tension force in the member perpendicular to the grain.

The ductile failure theory used for connections formed with dowel type fasteners is that the connection of the fastener and the timber or wood-based material will behave as rigid plastic materials in accordance with the strength-displacement relationships shown in Figure 4.1.

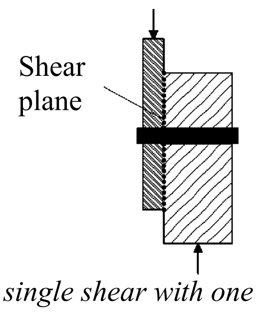
This assumption considerably simplifies the analysis and using such relationships, based on the possible alternative ductile failure modes that can occur in a connection, Johansen derived the strength equations for connections formed using metal dowel type fastener in timber. When using such fasteners, the possible failure modes that can arise in timber-to-steel connections are shown in Figures 4.3-4.7. There are also failure modes in timber-to-timber or wood panel to timber connections of course, but those are not important for this work. The associated connection strength equations are dependent on the geometry of the connection, the embedment strength of the timber or wood-based material, the bending strength of the fastener and that the fastener will not withdraw from the connection.

Regarding timber-to-steel connections, where the steel plate thickness is less than or equal to 0,5 times the dowel diameter $d$, the plate is classified as a thin plate and when it is equal to or grater than $d$ and the tolerance allowance for the dowel type fastener is less than 0,1$d$, it is classified as a thick plate. Strength equations have been derived for connections using each type of plate. For those formed using steel plates with a thickness between these limits, the strength is obtained by linear interpolation between the limiting values based on thin and thick plate arrangements. In a case of the thick plate, the nail is partially clamped and a plastic joint can occur next to the steel plate. On the contrary, in a case of the thin plate, it is supposed that the nail can rotate freely in the steel plate.

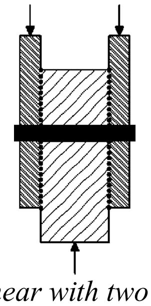
The thickness of the plate, from which the metal connectors are made of, is usually from 1 to 5 mm, the most often used nails have a diameter 4 mm. In the paper of Ehlbeck and Görlacher, it is said that in a case of the 2 mm thick plate and diameter of the nail 4mm (it means that it should be classified as the thin plate), it is advisable to use the formulas for the thick plate because the shank of the nail is conically widen below the head and therefore, it is actually clamped in the steel plate.

Johansen's equations have been slightly modified and added to equations made by other researchers to enhance the connection strength since they were derived. The final equations are the equations (4.1)-(4.12). The equations given for double shear connections (4.6)-(4.12) can be applied only to symmetrical assemblies and if non-symmetrical arrangements are used, new equations have to be developed or approximate solutions can be used.

Connections can be formed with fasteners in single or double shear. In the single shear connection, there is one shear plane per fastener and in the double shear connection there are two shear planes per fastener. It is important to note that the equations (4.1)-(4.12) refer to the characteristic load-bearing capacity of a fastener per shear plane.




*(a) single shear with one shear plane per fastener*



*(b) double shear with two shear planes per fastener*

*Fig. 4.2: Single and double shear connections*

For connections in single shear, the characteristic load-bearing capacity per shear plane per fastener, $F_{v,Rk}$, will be the minimum value from the equations (4.1), (4.2) or (4.3)-(4.5) depending on the used plate. Because there is only one shear plane this value will also be equal to the load-bearing capacity per fastener in the connection and the failure mode will be the mode associated with the equation with minimal value.

For symmetrical connections in double shear, the characteristic load-bearing capacity per shear plane per fastener, $F_{v,Rk}$, will be the minimum value from equations (4.6)-(4.8) or (4.9), (4.10) or (4.11), (4.12) depending on the used plate and its position. The failure mode will be the mode associated with that equation. However, because there are two shear planes per fastener, the characteristic load-bearing capacity of a fastener in double shear is 2 x $F_{v,Rk}$.

$$F_{v,Rk} = 0{,}4 f_{h,k} \cdot t_1 \cdot d \qquad \text{mode (a)} \qquad (4.1)$$

$$F_{v,Rk} = 1{,}15\sqrt{2M_{y,Rk} \cdot f_{h,k} \cdot d} + \frac{F_{ax,Rk}}{4} \qquad \text{mode (b)} \qquad (4.2)$$

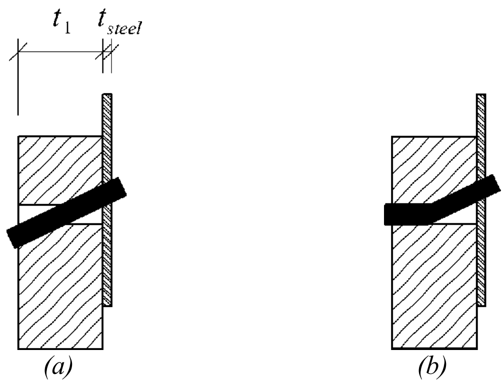



*Fig. 4.3: Failure modes in single shear with thin plates*

$$F_{v,Rk} = f_{h,k} \cdot t_1 \cdot d\left[\sqrt{2 + \frac{4M_{y,Rk}}{f_{h,k} \cdot t_1^{\,2} \cdot d}} - 1\right] + \frac{F_{ax,Rk}}{4} \qquad \text{mode (c)} \qquad (4.3)$$

$$F_{v,Rk} = 2{,}3\sqrt{M_{y,Rk} \cdot f_{h,k} \cdot d} + \frac{F_{ax,Rk}}{4} \qquad \text{mode (d)} \qquad (4.4)$$

$$F_{v,Rk} = f_{h,k} \cdot t_1 \cdot d \qquad \text{mode (e)} \qquad (4.5)$$

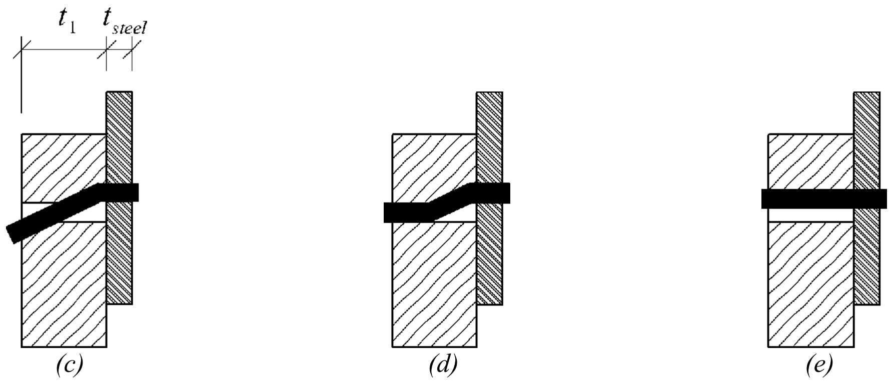



*Fig. 4.4: Failure modes in single shear with thick plates*

$$F_{v,Rk} = f_{h,1,k} \cdot t_1 \cdot d \qquad \text{mode (f)} \qquad (4.6)$$

$$F_{v,Rk} = f_{h,1,k} \cdot t_1 \cdot d \left[ \sqrt{2 + \frac{4M_{y,Rk}}{f_{h,1,k} \cdot t_1^2 \cdot d}} - 1 \right] + \frac{F_{ax,Rk}}{4} \qquad \text{mode (g)} \qquad (4.7)$$

$$F_{v,Rk} = 2{,}3\sqrt{M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \frac{F_{ax,Rk}}{4} \qquad \text{mode (h)} \qquad (4.8)$$

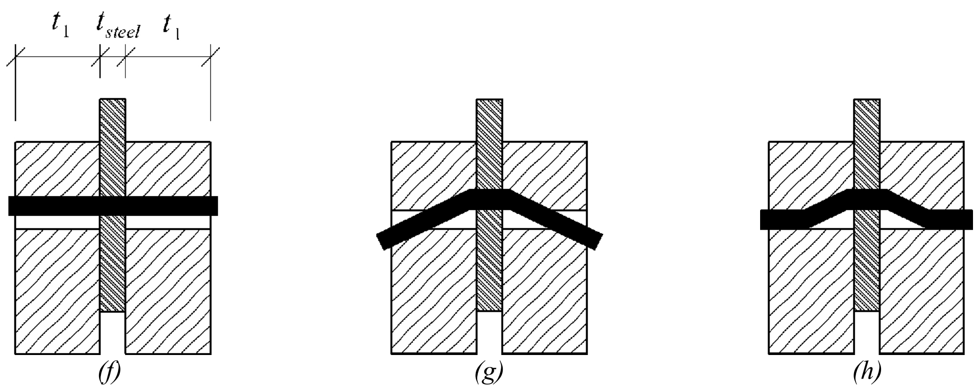



*Fig. 4.5: Failure modes in double shear with any thickness of plate as central member*

$$F_{v,Rk} = 0{,}5 f_{h,2,k} \cdot t_2 \cdot d \qquad \text{mode (j)} \qquad (4.9)$$

$$F_{v,Rk} = 1{,}15\sqrt{2M_{y,Rk} \cdot f_{h,2,k} \cdot d} + \frac{F_{ax,Rk}}{4} \qquad \text{mode (k)} \qquad (4.10)$$

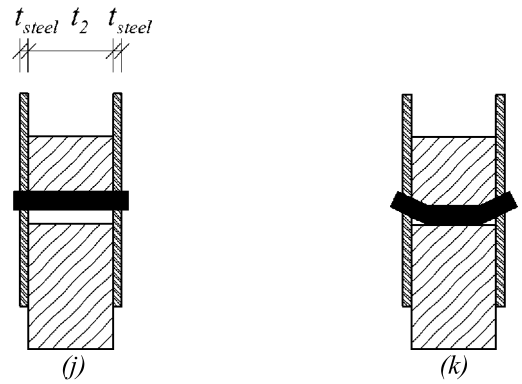



*Fig. 4.6: Failure modes in double shear with thin plates as outer members*

$$F_{v,Rk} = 0{,}5 f_{h,2,k} \cdot t_2 \cdot d \qquad \text{mode (l)} \qquad (4.11)$$

$$F_{v,Rk} = 2{,}3\sqrt{M_{y,Rk} \cdot f_{h,2,k} \cdot d} + \frac{F_{ax,Rk}}{4} \qquad \text{mode (m)} \qquad (4.12)$$

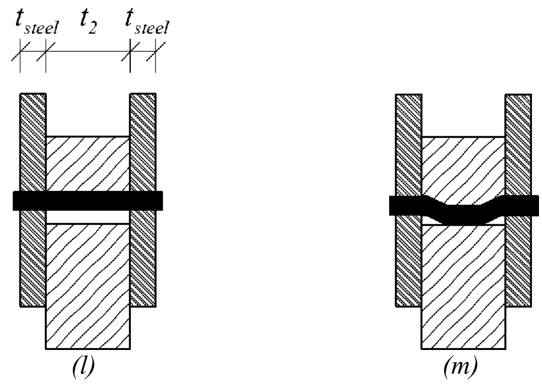



*Fig. 4.7: Failure modes in double shear with thick plates as outer members*

Where $d$ is the diameter of the fastener, $M_{y,Rk}$ is the characteristic fastener yield moment, $f_{h,i,k}$ is characteristic embedment strength of the $i^{zh}$ connected member, $t_i$ is thickness of the $i^{zh}$ connected member and $F_{ax,Rk}$ is the characteristic withdrawal resistance of the fastener. All the characteristics will be more described thereinafter.

### 4.1.1 Fastener Diameter

The diameter depends on the type the used fastener. For both, annular ring nail and bolt, the diameter of the unthreaded part of the shank is taken into account.

### 4.1.2 Characteristic Fastener Yield Moment

The yield moment was taken as the moment at the elastic limit of the fastener and was derived from the yield strength in Johansen's original equations. It gave a lower strength and in the next development of his theory by other researchers the elasto-plastic strength has been used. This takes into account amount of rotation at the failure state for different types of fasteners and the tensile strength of the fastener. The tensile strength of the fastener includes the effect of strain hardening.[1] The characteristic yield moments for different types of metal fasteners given in EC5 are thereinafter in equations (4.13) and (4.14).

- bolts:

$$M_{y,Rk} = 0{,}3 f_{u,k} \cdot d^{2.6} \qquad \text{[Nmm]} \qquad (4.13)$$

- smooth round nails:

$$M_{y,Rk} = 0{,}3 f_{u} \cdot d^{2.6} \qquad \text{[Nmm]} \qquad (4.14)$$

Where $d$ is the diameter of the nail or bolt (in mm), $f_u$ is tensile strength of the nail wire (in N/mm$^2$) and $f_{u,k}$ is the characteristic tensile strength of the bolt (in N/mm$^2$).

There is no formula or number for $M_{y,Rk}$ for the annular ring nails given in EC5. The annular ring nails are made of higher quality steel than the smooth nails, but their bending resistance is strongly affected by the shaping of the shank. The annular rings are places of a concentration of stresses when loaded transversally.

Werner and Siebert worked on this task. They made experiments with annular ring nails from four different producers and from their work results the following value.

- annular ring nails with $d$ = 4,0 mm:

$$M_{y,Rk} = 6370 \text{ Nmm} \qquad (4.15)$$

### 4.1.3 Characteristic Embedment Strength

The embedment strength of timber or a wood-based product is the average compressive strength of the timber or wood-based product under the action of a stiff straight metal fastener loaded as shown in Figure 4.8.

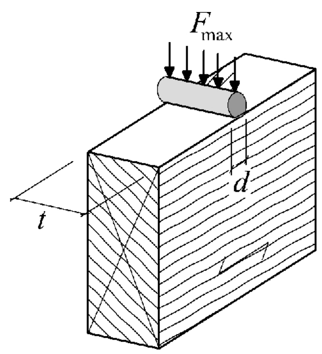



*Fig. 4.8: Embedment strength of timber or wood-based material*

[1] Strain hardening, also known as work hardening, is the strengthening of a metal by plastic deformation. This strengthening occurs because of dislocation movements within the crystal structure of the material. Any material with a reasonably high melting point such as metals and alloys can be strengthened in this manner.

For a piece of timber $t$ (mm) thick, loaded with a nail $d$ (mm) in diameter, under the maximum load able to transfer $F_{max}$ (N), the embedment strength is defined as:

$$f_h = \frac{F_{\max}}{d \cdot t} \tag{4.16}$$

The embedment is the distance the fastener depresses the timber or wood-based product. Because of the complex cellular nature of timber and wood-based products, the embedment strength is not a material property. It is a system property dependent on several factors, including the type of the used fastener. From the results of investigations by various researchers, the values derived for the characteristic embedment strength of timber and wood products when using different types of metal dowel type fasteners are written thereinafter.

#### 4.1.3.1 Nails with Diameter ≤ 8 mm

The embedment strength varies depending on the diameter of the used nail, the types of material used in the connection, and whether or not pre-drilling is done.

- nails up to 8 mm in diameter for timber connections without pre-drilled holes:

$$f_{h,k} = 0{,}082\rho_k \cdot d^{-0.3} \qquad [\text{N/mm}^2] \tag{4.17}$$

- nails up to 8 mm in diameter for timber connections with pre-drilled holes:

$$f_{h,k} = 0{,}082(1 - 0{,}01d)\rho_k \qquad [\text{N/mm}^2] \tag{4.18}$$

Where $d$ is the diameter of the nail or bolt (in mm), $\rho_k$ is the characteristic density of the timber (in kg/m$^3$).

#### 4.1.3.2 Bolts and Nails with Diameter > 8 mm

When using timber in a connection, the embedment strength of bolts and nails (with a diameter greater than 8 mm) is dependent on the direction of the applied load relative to the grain and for such fasteners the embedment strength is determined by Hankinson's equation. An arrangement of such loading is in Figure 4.9.

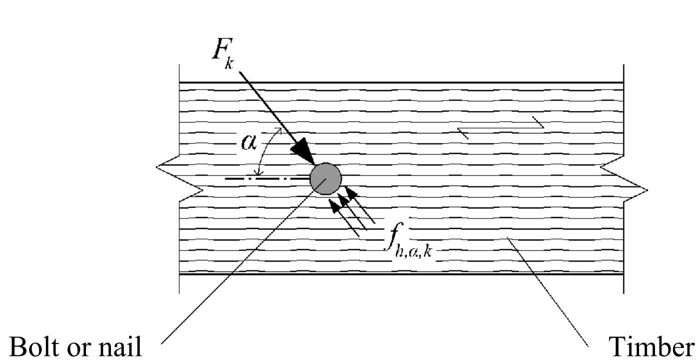



*Fig. 4.9: Embedment strength of a bolt or nail (d > 8 mm) loaded at an angle α to the grain*

Hankinson's equation for determining of the characteristic embedment strength, $f_{h,\alpha,k}$, when the fastener is loaded at an angle $\alpha$ to the grain is written as:

$$f_{h,\alpha,k} = \frac{f_{h,0,k} \cdot f_{h,90,k}}{f_{h,0,k} \cdot \sin^2 \alpha + f_{h,90,k} \cdot \cos^2 \alpha} \tag{4.19}$$

where $f_{h,\alpha,k}$ is the characteristic embedment strength at an angle $\alpha$ to the grain, $f_{h,0,k}$ is the characteristic embedment strength parallel to the grain and $f_{h,90,k}$ is the characteristic embedment strength perpendicular to the grain.

In Eurocode 5 the Hankinson's equation (4.19) is reduced to the equation for timber loaded at an angle $\alpha$ to the grain by bolts up to 30 mm in diameter and nails greater than 8 mm:

$$f_{h,\alpha,k} = \frac{f_{h,0,k}}{k_{90} \cdot \sin^2 \alpha + \cos^2 \alpha} \qquad [\text{N/mm}^2] \tag{4.20}$$

- and the equation for timber loaded parallel to the grain by bolts up to 30 mm in diameter and nails greater than 8 mm is:

$$f_{h,0,k} = 0{,}082(1 - 0{,}01d)\rho_k \qquad [\text{N/mm}^2] \tag{4.21}$$

where $d$ is the diameter of the fastener (in mm), $\rho_k$ is the characteristic density of the timber (in kg/m$^3$), $\alpha$ is the angle of the load in the fastener relative to the grain, $k_{90} = f_{h,0,k} / f_{h,90,k}$ and the values it can have for different timber are:

- for softwood[2]:

$$k_{90} = (1{,}35 + 0{,}015d) \tag{4.22}$$

- for hardwood[3]:

$$k_{90} = (0{,}9 + 0{,}015d) \tag{4.23}$$

## 4.1.4 Member Thickness

In a connection the members are classified as member 1 and member 2 as shown in Figures 4.3 - 4.7. Individual members are defined below.

### 4.1.4.1 Nailed Connection

$t_1$ is the nail pointside[4] penetration where the connection is in single shear, and the minimum of the nail headside[5] material thickness and the nail pointside penetration in a double shear connection.

$t_2$ is the central member thickness for a connection in double shear.

[2] The term softwood is used to describe wood from conifers. It may also be described as trees, which tend to be evergreen, exception is the larch.

[3] Hardwood is wood from angiosperm trees. These trees are mostly deciduous, but in tropics and subtropics mostly evergreen. Hardwoods are not necessarily harder than softwoods.

[4] Nail pointside thickness is the distance the pointed end of the nail penetrates into a member.

[5] Nail headside material thickness is the thickness of the member containing the nail head.

#### 4.1.4.2 Bolted Connection

$t_1$ is the bolt threaded end member thickness when the connection is in single shear and the thickness of one of the outer members in double shear.

$t_2$ is the central member thickness when in double shear.

### 4.1.5 Friction Effects and Axial Withdrawal Resistance of Fastener

The basic Johansen's yield equation for each failure mode for connections in single or double shear can be derived by the use of a static analysis or by the virtual work approach commonly used in the plastic analysis of steel structures. In deriving these equations, friction forces between the members of the connection are ignored as well as the withdrawal resistance of the fasteners. In EC5 the Johansen's yield equations form the basis of the strength equations. However, for those failure modes that involve yielding of the fastener, the equations have been modified to include friction and withdrawal effects.

There are two types of friction effects that can occur in a connection. One will develop if the members are in contact after assembly and the other will occur when the fasteners yield and pull the members together when the fasteners deform under lateral load. The first type of the friction will be eliminated if there is shrinkage of the timber or wood products when it is in service and because of this, this type of the friction is not included in the EC5 strength equations. The latter type of the friction will always occur in the failure modes that involve yielding of the fasteners and this has been included in the EC5 equations related to such modes.

Consider, for example, a single shear connection formed with a steel plate and a timber member connected by a single dowel type fastener as shown in Figure 4.10. Assume that

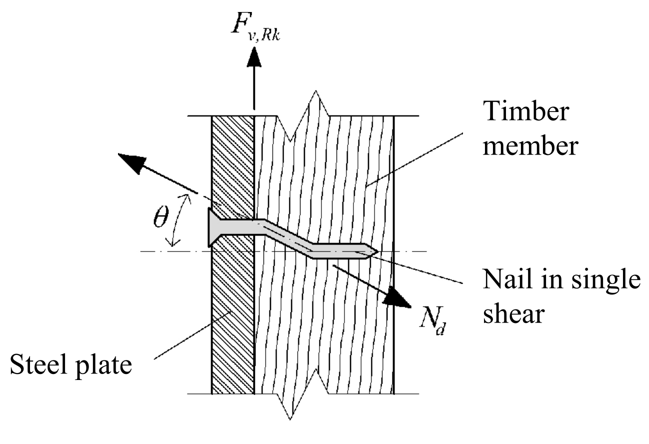



*Fig. 4.10: Connection with dowel type fastener in single shear at the failure condition*

under the lateral shear force on the joint the fastener yields next to the steel plate and in the timber member allowing it to rotate by an angle $\theta$ and that the friction coefficient between the plate and the timber is $\mu$. In addition to a bending, the fastener will be subjected to a tension force $N_d$ due to the withdrawal effect during loading. Force $N_d$ will have a vertical component, $N_d \cdot \sin\theta$, and a horizontal component, $N_d \cdot \cos\theta$. The latter force compresses the plate onto the timber and induces an additional vertical resisting force $\mu \cdot N_d \cdot \cos\theta$, due to the friction. The force in the fastener, $F_{v,Rk}$, is equal to the sum of the all vertical forces in the connection as follows:

$$F_{v,Rk} = N_d \cdot (\sin\theta + \mu \cdot \cos\theta) + \text{Johansen's yield load for the joint } (F_{y,Rk})$$

At failure condition, $N_d$ will be the withdrawal resistance of the fastener and in EC5 the component $N_d \cdot \sin\theta$ is taken to be $F_{ax,Rk}/4$, where $F_{ax,Rk}$ is the characteristic withdrawal resistance of the fastener, discussed in 4.4, and component $\mu \cdot N_d \cdot \cos\theta$ is taken as a percentage of the Johansen's yield load $F_{y,Rk}$. Taking these effects into account, the characteristic lateral load-bearing capacity of a fastener, $F_{v,Rk}$, is in EC5 written as:

$$F_{v,Rk} = \text{friction factor} \times \text{Johansen's yield load} + (\text{withdrawal resistance})/4$$

In Eurocode 5 the values used for the friction are 5% where the fastener partially yields (used in timber-to-timber connections) and 15% where the fastener fully yields. The use of these factors can be seen in equations (4.2)-(4.4), (4.7), (4.8), (4.10) and (4.12). In equations (4.4), (4.8) and (4.12) the numerical coefficient incorporates a factor 1,15 for this effect as a double value.

To discriminate between the Johansen's yield load and the combined withdrawal and friction forces in a connection, the latter are commonly referred to as the rope effect forces, however in Eurocode 5 reference is only made to the term $F_{ax,Rk}/4$ as the contribution from this effect..

In EC5 an upper limit is also set for the value of $F_{ax,Rk}/4$. It is taken to be a percentage of the first term of the relevant strength equations (i.e. a percentage of the Johansen's yield load, $F_{y,Rk}$, enhanced by the friction factor associated with the rope effect). For the round nails, the rope effect must be less or equal to 15% of the first term and for the bolts, the rope effect must be less or equal to 25% of the first term.

As previously stated, $F_{ax,Rk}$ is the characteristic axial withdrawal resistance of the fastener. For those fasteners that are potentially susceptible to withdrawal, the minimum permitted penetration in timber is specified.

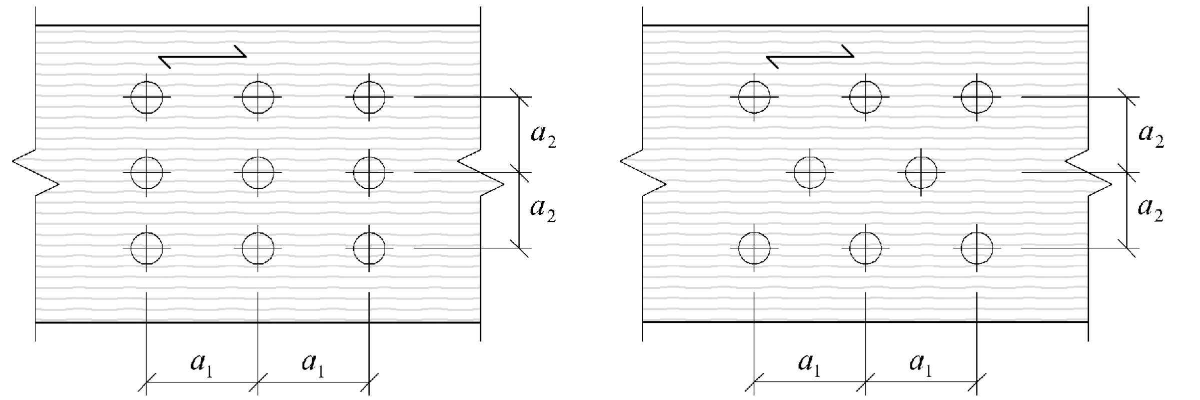
#### 4.1.5.1 Minimum Penetration when Using Nails

For smooth nails, the minimum pointside penetration (i.e. the penetration of the pointed end of the nail into the timber) is $8d$, however at this value the pointside withdrawal resistance of the nail is taken to be zero. Where the pointside penetration is at least $12d$, the full characteristic value of the withdrawal strength given in EC5 can be used and between $8d$ and $12d$ the withdrawal strength should be multiplied by $((t_{pen} / 4d)$-2), where $t_{pen}$ is the pointside penetration length, as discussed in 4.4.1.

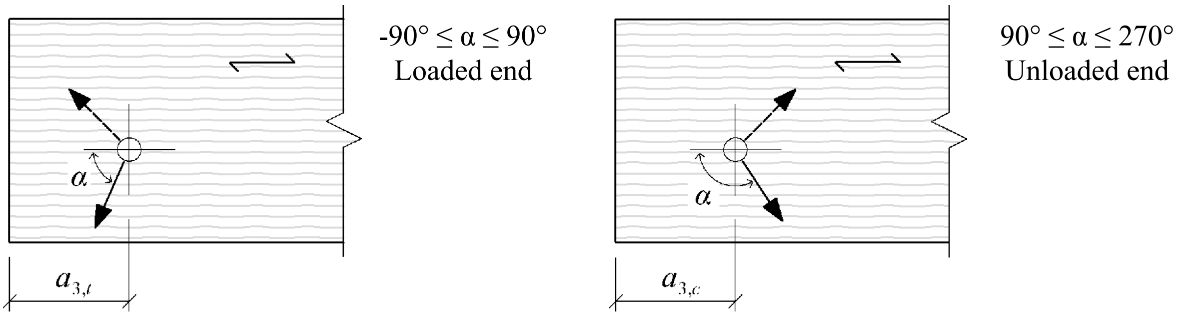
For other nails, the minimum pointside penetration is $6d$ and at this value the pointside withdrawal resistance of the nail is taken to be zero. When the pointside penetration is at least $8d$ the full characteristic value of the withdrawal strength can be used and between $6d$ and $8d$ the withdrawal strength should be multiplied by $((t_{pen} / 2d)$-3), which is discussed in 4.4.1.

### 4.1.6 Brittle Failure

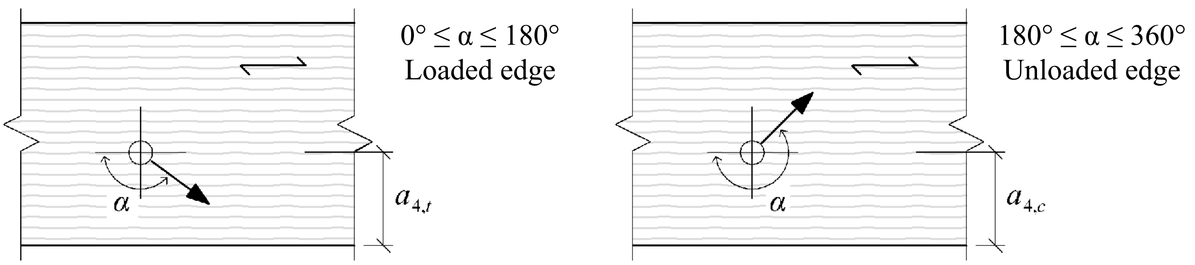
The EC5 strength equations, (4.1)-(4.12), are only valid if there is no premature splitting or shearing of the timber resulting in a brittle-type failure. To eliminate the risk of such failures, minimum edge, end and spacing criteria for connections with dowel type fasteners have been developed from testing and the requirements for nails and bolts are given in Table 4.1 and Table 4.2. The spacings and distances used in the table are illustrated in Figure 4.11. Also, to prevent splitting in the timber when using nails greater than 6 mm in diameter, pre-drilling must be used. The pre-drilling may also be necessary for nails driven in the timber with a characteristic density greater than 500 kg/m$^3$ to allow fixings to be formed.




*(a) Spacing parallel and perpendicular to grain*




*(b) End distances*




*(c) Edge distances*

*Fig 4.11: Fastener spacings and distances*

Where multiple dowel type fastener connections near the end of the timber member in a steel-to-timber connection are loaded parallel to the grain, there is a risk of a brittle-type failure due to block shear[6] and plug shear[7].

[6] Block shear is type of failure when a block, with the same thickness as a thickness of a timber member, connected with a metal work connector is torn out of the timber member due to a combination of tension and shear on the failure path.
[7] Plug Shear is similar to the block shear failure but a shear on the bottom face is present and therefore, the torn-out block has smaller thickness than a timber member; just a "plug" is torn out.

*Table 4.1: Minimum spacings and edge and end distances using nails of diameter d for timber-to-timber connections*

| Spacing or distance and $\alpha$ | Nails | | |
|---|---|---|---|
| | Without pre-drilled holes | | With pre-drilled holes |
| | $\rho_k \leq 420$ kg/m$^3$ | 420 kg/m$^3$ $< \rho_k \leq 500$ kg/m$^3$ | $d > 6$ mm and/or $\rho_k > 500$ kg/m3 |
| Spacing parallel to the grain: $a_1$<br>$0° \leq \alpha \leq 360°$ | $d < 5$ mm: $(5 + 5\,\lvert\cos\alpha\rvert)\,d$<br>$d \geq 5$ mm: $(5 + 7\,\lvert\cos\alpha\rvert)\,d$ | $(7 + 8\,\lvert\cos\alpha\rvert)\,d$ | $(4 + \lvert\cos\alpha\rvert)\,d$ |
| Spacing perpendicular to the grain: $a_2$<br>$0° \leq \alpha \leq 360°$ | $5d$ | $7d$ | $(3 + \lvert\sin\alpha\rvert)\,d$ |
| Loaded end distance: $a_{3,t}$<br>$-90° \leq \alpha \leq 90°$ | $(10 + 5\cos\alpha)\,d$ | $(15 + 5\cos\alpha)\,d$ | $(7 + 5\cos\alpha)\,d$ |
| Unloaded end distance: $a_{3,c}$<br>$90° \leq \alpha \leq 270°$ | $10d$ | $15d$ | $7d$ |
| Loaded edge distance: $a_{4,t}$<br>$0° \leq \alpha \leq 180°$ | $d < 5$ mm: $(5 + 2\sin\alpha)\,d$<br>$d \geq 5$ mm: $(5 + 5\sin\alpha)\,d$ | $d < 5$ mm: $(7 + 2\sin\alpha)\,d$<br>$d \geq 5$ mm: $(7 + 5\sin\alpha)\,d$ | $d < 5$ mm: $(3 + 2\sin\alpha)\,d$<br>$d \geq 5$ mm: $(3 + 4\sin\alpha)\,d$ |
| Unloaded edge distance: $a_{4,c}$<br>$180° \leq \alpha \leq 360°$ | $5d$ | $7d$ | $3d$ |

NOTE: for steel-to-timber connections, the minimum nail spacings are those given in Table 4.1 multiplied by a factor 0,7, the minimum edge and end distances remain unchanged

*Table 4.2: Minimum spacings, and edge and end distances for bolts in timber-to-timber and steel-to-timber connections*

| Spacing or distance and $\alpha$ | **Bolts** |
|---|---|
| Spacing parallel to the grain: $a_1$<br>$0° \leq \alpha \leq 360°$ | $(4 + \lvert\cos \alpha\rvert)\, d$ |
| Spacing perpendicular to the grain: $a_2$<br>$0° \leq \alpha \leq 360°$ | $4d$ |
| Loaded end distance: $a_{3,t}$<br>$-90° \leq \alpha \leq 90°$ | max $(7d;\ 80\ \mathrm{mm})$ |
| Unloaded end distance: $a_{3,c}$<br>$90° \leq \alpha \leq 150°$<br>$150° \leq \alpha \leq 210°$<br>$210° \leq \alpha \leq 270°$ | <br>max $[(1 + 6\sin \alpha)\, d;\ 4d]$<br>$4d$<br>max $[(1 + 6\sin \alpha)\, d;\ 4d]$ |
| Loaded end distance: $a_{4,t}$<br>$0° \leq \alpha \leq 180°$ | max $[(2 + 2\sin \alpha)\, d;\ 3d]$ |
| Unloaded end distance: $a_{4,c}$<br>$180° \leq \alpha \leq 360°$ | $3d$ |

Conditions for taking these phenomena (block shear and plug shear) into account are:

- there are ten or more metal dowel type fasteners with a diameter less or equal to 6 mm in a row parallel to the grain with a stagger less or equal to diameter *d*
- there are five or more metal dowel type fasteners with a diameter bigger than 6 mm in a row parallel to the grain

A Czech producer of the metal work connectors, Bova-nail company, does not have any product with neither ten or more holes for fasteners with a diameter less or equal to 6 mm in a row, nor five or more holes for fasteners with a diameter bigger than 6 mm in a row. And if so, they have bigger stagger than one *d*. The only exception is bracing strips. Anyway, with those brittle-type failures will not be dealt anymore.

#### 4.1.6.1 Brittle Failure due to Connection Forces at Angle to Grain

This form of the brittle failure can arise when fasteners apply a force at an angle to the timber grain resulting in the possibility of splitting caused by the force component perpendicular to the grain, as shown in Figure 4.12. This failure can occur at a load less than the design resistance of the fasteners.

Preliminary issues of Eurocode 5 were written on the presumption that by using the minimum spacings, end and edge distances given in the code, splitting would be prevented and joint failure would always be by ductile failure of the fasteners. The strength of the timber was based solely on a check of the shear strength of the member. Before the final publication of EC5 this was revised to incorporate a design check on the splitting resistance of the connected members where the forces in the fasteners were able to induce a tension component perpendicular to the grain in the connected members.

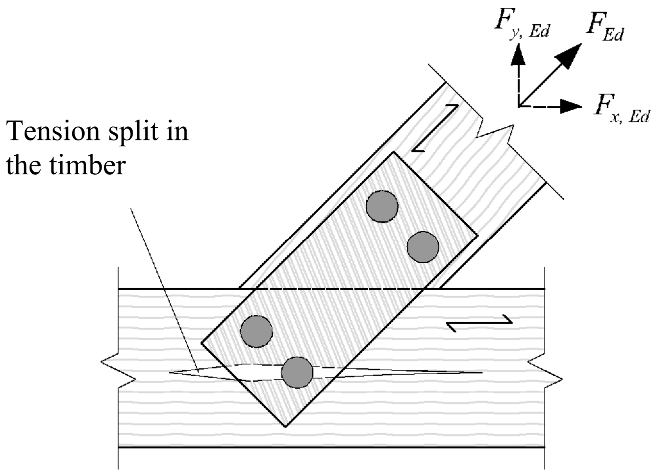



*Fig. 4.12: Member loaded in tension at an angle to the grain.*

The strength equation in EC5 is developed from the application of linear elastic fracture mechanics, and the requirements are as follows.

At the ULS[8],

$$F_{v,Ed} \le F_{90,Rd} \tag{4.24}$$

$$F_{v,Ed} = \max\begin{Bmatrix} F_{v,Ed,1} \\ F_{v,Ed,2} \end{Bmatrix} \tag{4.25}$$

where $F_{v,Ed,1}$ and $F_{v,Ed,2}$ are the design shear force on each side of the connection as shown in Figure 4.13 (both in N) and $F_{90,Rd}$ is the design splitting capacity calculated from the characteristic splitting resistance $F_{90,Rk}$ (in N) according to equation (4.40). The characteristic splitting resistance $F_{90,Rk}$ for softwood is:

$$F_{90,Rk} = 14b \cdot w \cdot \sqrt{\frac{h_e}{\left[1-(h_e/h)\right]}} \tag{4.26}$$

where $w$ is modification factor and $w = 1$ for all connectors but punched metal plate fasteners, $h_e$ is the distance from the most distant connector (in mm), $h$ is the member height (in mm), and $b$ is the member thickness (in mm).

[8] Ultimate limit state (ULS) is a limit state associated with collapse or equivalent forms of failure.

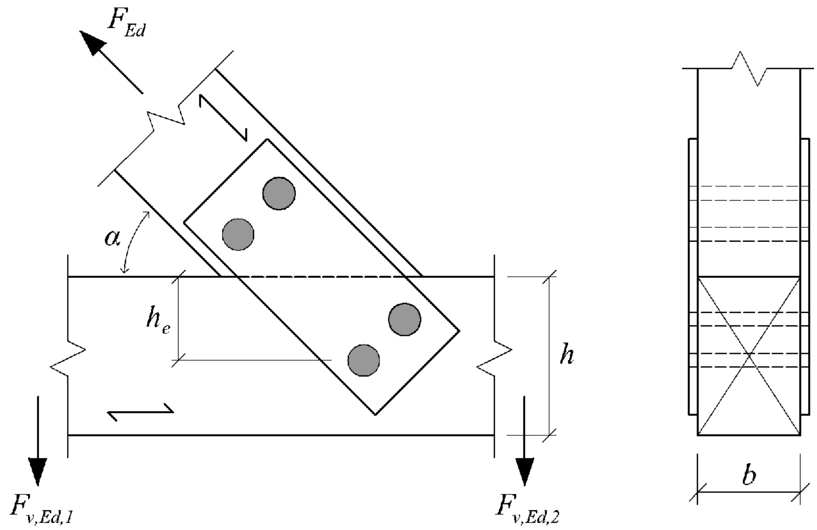



*Fig. 4.13: Inclined force transmitted by a connection*

## 4.2 Multiple Dowel Fasteners Loaded Laterally

In EC5, a number of fasteners lying along a line running parallel to the grain direction, as illustrated in Figure 4.14, are referred to as a row of fasteners parallel to the grain.

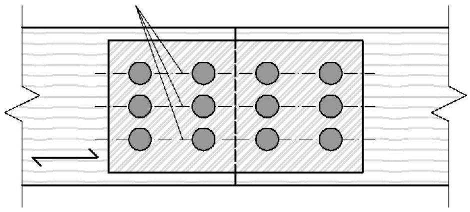



*Fig. 4.14: Rows of fasteners*

Where there is only a single fastener in the row, the design strength of the row per shear plane will be the design lateral load-bearing capacity of the fastener per shear plane. And where there are $r$ such rows, the design strength of the connection parallel to the grain per shear plane will be the design lateral resistance of the fastener per shear plane multiplied by number of the rows $r$.

Where there is more than one fastener per row parallel to the grain, the strength of the row depends on the stiffness of the fastener and the strength of the bedding material, and in general the stiffer the fastener the greater the design strength of the row. Several researchers have investigated this effect, and the effective characteristic load-bearing capacity of a row of fasteners parallel to the grain, $F_{v,ef,Rk}$ , is:

$$F_{v,ef,Rk} = n_{ef} \cdot F_{v,Rk} \tag{4.27}$$

where $F_{v,ef,Rk}$ is the effective characteristic lateral load-bearing capacity per shear plane of one row of fasteners parallel to the grain, $n_{ef}$ is the effective number of fasteners per shear plane in the row parallel to the grain, and $F_{v,Rk}$ is the characteristic lateral load-bearing capacity per shear plane of the fastener type used.

### 4.2.1 The Effective Number of Fasteners

The effective number of fasteners in a connection is dependent on the type of fastener and the direction of loading relative to the grain.

#### 4.2.1.1 Nails

*(a) Loaded parallel to the grain.* Where nails are staggered in a row by less than a nail diameter perpendicular to the grain, as shown in Figure 4.15, they will all form part of the row, and if they are staggered by a distance greater than the nail diameter, two separate rows will be formed.

- single nails in single or double shear:

$$n_{ef} = n^{k_{ef}} \tag{4.28}$$

- overlapping nails:

$$n_{ef} = n_p^{k_{ef}} \tag{4.29}$$

Where $n_{ef}$ is the effective number of nails in a row parallel to the grain, $n$ is the number of nails in a row parallel to the grain (for single nails), $n_p$ is the number of overlapping nails[9] in the row parallel to the grain, $k_{ef}$ is an exponent that is dependent on the nail spacing and whether or not pre-drilling is used, and is given in Table 4.3.

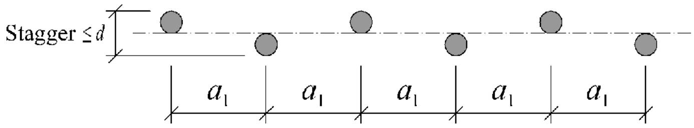



*Fig. 4.15: A row formed using single nails*

*(b) Loaded perpendicular to the grain.* When loading nails perpendicular to the grain in a single or double shear connection, as shown in Figure 4.16, the effective number of nails, $n_{ef}$, in each line of nails should be taken the same as the actual number of nails, $n$, when using single nails or the number of overlapping nails when using overlapping nails. As explained in 4.1.6.1, when loaded this way there is the risk of a splitting of the connection material and the connection resistance will be the lesser of the splitting resistance of the member subjected to the tensile force and the connection strength derived from the summation of the strength of the fasteners.

[9] An overlapping nail is formed from two nails.

*Table 4.3: Values for exponent $k_{ef}$ in equations (4.28) and (4.29)*

| Spacing | Pre-drilled | Not pre-drilled |
|---|---|---|
| $a_1 = 14d$ | 1,0 | 1,0 |
| $a_1 = 12d$ | 0,925 | 0,925 |
| $a_1 = 10d$ | 0,85 | 0,85 |
| $a_1 = 9d$ | 0,8 | 0,8 |
| $a_1 = 8d$ | 0,75 | 0,75 |
| $a_1 = 7d$ | 0,7 | 0,7 |
| $a_1 = 4d$ | 0,5 | - |

NOTE: linear interpolation of $k_{ef}$ is permitted for $a_1$ between the stated values

*(c) Loaded at an angle to the grain.* When the nails in a single or double shear connection are laterally loaded at an angle to the grain, the force components parallel and perpendicular to the grain have to be derived. The component of the design force acting parallel to the grain must not exceed the load-bearing capacity based on the use of the effective number of nails per row in the connection as defined in 4.2.1.1 (a). The component of the design force acting perpendicular to the grain must not exceed the load-bearing capacity as defined in 4.2.1.1 (b).

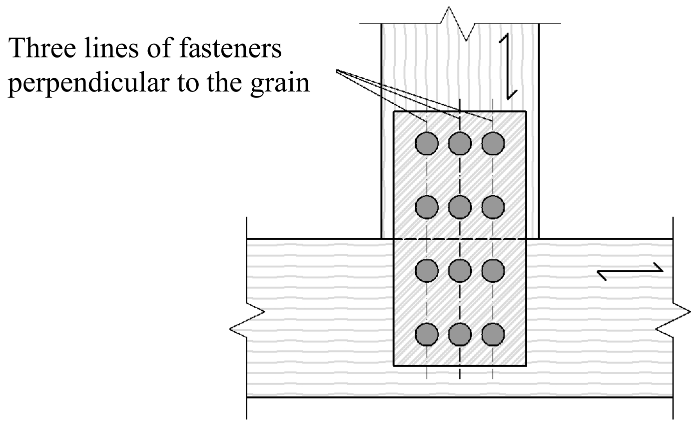



*Fig. 4.16: Lines of fasteners perpendicular to the grain*

#### 4.2.1.2 Bolts

*(a) Loaded parallel to the grain.* As bolts are stiffer than nails, for connections in single or double shear, the reduction in row resistance parallel to the grain is less than with nails:

$$n_{ef} = \min\left\{ \begin{array}{c} n \\ n^{0,9} \cdot \sqrt[4]{\dfrac{a_1}{13d}} \end{array} \right\} \tag{4.30}$$

where $n_{ef}$ is the effective number of bolts in a row parallel to the grain, $a_1$ is the bolt spacing in the grain direction, $d$ is the diameter of the bolt, and $n$ is the number of bolts in the row.

*(b) Loaded perpendicular to the grain.* For loading perpendicular to the grain, in a single or double shear connection there is no reduction:

$$n_{ef} = n \tag{4.31}$$

*(c) Loaded at an angle to the grain.* The load-bearing capacity parallel or perpendicular to the grain should be determined in the same way as for nails, as described in 4.2.1.1 (c). For angles $0° < \alpha < 90°$, $n_{ef}$ may be determined by linear interpolation between equations (4.30) and (4.31).

# 4.3 Design Strength of Laterally Loaded Metal Dowel Type Connection

The strength equations given in the following sub-sections assume that in the connection the design shear strength of the fasteners will always exceed the design resistance derived from timber strength equations for the relevant type of fastener.

## 4.3.1 Design Values of Material or Product Properties

The design value $X_d$ of material or product property is used for the ULS and it is obtained from:

$$X_d = \eta \cdot \frac{X_k}{\gamma_M} \tag{4.32}$$

where $X_k$ is the characteristic value of the property, $\eta$ is the mean value of a conversion factor that takes into account volume and scale effects, the effects of moisture and temperature and any other relevant parameters, and $\gamma_M$ is a partial factor for a material property at ULS, given in Table 4.4, taking into account uncertainty in the resistance model used for design together with the adverse effects of geometric deviations.

*Table 4.4: Partial factors for material properties and resistances, $\gamma_M$*

| Type of material | $\gamma_M$ |
|---|---|
| Solid timber - untreated or treated with a preservative | 1,3 |
| Glued-laminated timber | 1,25 |

In EC5, $\eta$ covers the effects of duration of load and variation in moisture content on the properties of the timber and wood products and is referred to as the modification factor $k_{mod}$. Where the connection comprises two timber elements $k_{mod,1}$ and $k_{mod,2}$ the value used in the equation will be $k_{\mathrm{mod}} = \sqrt{k_{\mathrm{mod},1} \cdot k_{\mathrm{mod},2}}$ . Factors covering scale and volume effects are considered separately in Eurocode 5. The modification factor is extremely important in timber design and a brief overview of how load duration and moisture content effects are taken into account is given thereinafter.

#### 4.3.1.1 Load Duration Classes

When subjected to loading, the strength properties of members reduce and the longer the duration of the load the greater the reduction will be. In order to establish a common basis for design, load duration classes have been defined to cover the range of durations likely to arise in practice and the duration associated with each class is given in Table 4.5.

*Table 4.5: Load-duration class definitions*

| Class | Period of time | Examples of loading |
|---|---|---|
| Permanent | > 10 years | Self-weight |
| Long term | 6 months to 10 years | Storage loading, water tanks |
| Medium term | 1 week to 6 months | Imposed floor loading, snow |
| Short term | < 1 week | Snow, maintenance or man loading on roofs, residual structure after an accidental event |
| Instantaneous | Instantaneous | Wind, impact loading, explosion |

#### 4.3.1.2 Service Classes

Because the strength (and creep behaviour) of timber and wood-based products is affected by the moisture content of the material, these properties are dependent on the temperature and relative humidity conditions the materials are subjected to over the design life of the structure. When the moisture content is low, the strength property will be at its maximum and as the moisture content increases the strength is reduced and will reach a minimum value at the fibre saturation point.

To take this effect into account in design, three service classes have been defined in EC5, covering the typical environmental conditions that timber structures will function under. These are as follows:

*Service class 1 – where the average moisture content in most softwoods will not exceed 12%.*

This corresponds to a temperature of 20°C and a relative humidity of the surrounding air exceeding 65% for only a few weeks per year.

*Service class 2 - where the average moisture content in most softwoods will not exceed 20%.*

This corresponds to a temperature of 20°C and a relative humidity of the surrounding air exceeding 85% for only a few weeks per year.

*Service class 3 - where the average moisture content in most softwoods will exceeds 20%.*

This corresponds to climatic conditions leading to higher moisture contents than service class 2.

The highest values of timber strength will be obtained when structures function in service class 1 conditions and the lowest when they function in service class 3 conditions. Values for

$k_{mod}$ based on the load duration referred to in 4.3.1.1 and the above service classes are summarised in Table 4.6.

*Table 4.6: Values of $k_{mod}$*

| Material | Service class | Load-duration class | | | | |
|---|---|---|---|---|---|---|
| | | Permanent action | Long term action | Medium term action | Short term action | Instantaneous action |
| Solid timber | 1 | 0,60 | 0,70 | 0,8 | 0,90 | 1,10 |
| | 2 | 0,60 | 0,70 | 0,8 | 0,90 | 1,10 |
| | 3 | 0,50 | 0,55 | 0,65 | 0,70 | 0,90 |
| Glued-laminated timber | 1 | 0,60 | 0,70 | 0,8 | 0,90 | 1,10 |
| | 2 | 0,60 | 0,70 | 0,8 | 0,90 | 1,10 |
| | 3 | 0,50 | 0,55 | 0,65 | 0,70 | 0,90 |

### 4.3.2 Design Resistance

When dealing with timber and wood-based product structures, the design value of a resistance is expressed in EC5 as:

$$R_d = k_{\text{mod}} \cdot \frac{R_k}{\gamma_M} \tag{4.33}$$

where $k_{mod}$ is a modification factor, given in Table 4.6, that takes into account the effect of load duration and moisture content, $\gamma_M$ is the partial factor for a material property at the ULS, and $R_k$ is the characteristic value of the load-bearing capacity at the ULS.

However, the resistance properties are defined in EC5 as functions of $F$ and the more representative expression for the design resistance for a timber or wood-based product is:

$$F_{Rd} = k_{\text{mod}} \cdot \frac{F_{Rk}}{\gamma_M} \tag{4.34}$$

### 4.3.3 Design Resistance of Fasteners Loaded Parallel to Grain

The design strength of a laterally loaded single fastener, $F_{v,Rd}$, is obtained from the characteristic load-bearing capacity of the laterally loaded fastener, according to the equation (4.34), as follows:

$$F_{v,Rd} = k_{\text{mod}} \cdot \frac{F_{v,Rk}}{\gamma_M} \tag{4.35}$$

where $k_{mod}$ is a modification factor given in Table 4.6, $\gamma_M$ is the partial factor for a material property at the ULS, and $F_{v,Rk}$ is the characteristic load-bearing capacity of the fastener per shear plane when loaded laterally.

For a connection containing $r_{pl}$ rows of fasteners laterally loaded parallel to the grain, with each row containing $n$ equally spaced fasteners of the same size, each with a design strength per shear plane, $F_{v,Rd}$, the effective lateral load design resistance of the connection parallel to the grain, $F_{v,ef,Rd}$, will be:

$$F_{v,ef,Rd} = n_{sp} \cdot r_{pl} \cdot n_{ef} \cdot F_{v,Rd} \tag{4.36}$$

where $n_{ef}$ is the effective number of fasteners in the connection in each row parallel to the grain and $n_{sp}$ is the number of shear planes in the connection.

### 4.3.4 Design Resistance of Fasteners Loaded Perpendicular to Grain

Where loads are imposed on the timber by fasteners loaded perpendicular to the grain, there are two possible forms of failure. Either by the timber splitting in tension and this is covered in 4.1.6.1 or by ductile yielding of the fastener and for this condition, where there are $r_{pr}$ lines of fasteners with each line containing $n$ fasteners, all of the same size, then:

$$F_{v,ef,Rd} = n_{sp} \cdot r_{pr} \cdot n \ \cdot F_{v,Rd} \tag{4.37}$$

where $F_{v,ef,Rd}$ is the effective design strength of the fastener per shear plane when loaded laterally and perpendicular to the grain, $n_{sp}$ is the number of shear planes in the connection, $n$ is the number of fasteners in each line of fasteners perpendicular to the grain (if overlapping nails are being used, $n$ will be the number of overlapping nails as defined in equation (4.29)), $F_{v,Rd}$ is the design load-bearing capacity of a laterally loaded single fastener per shear plane when loaded perpendicular to the grain.

For $F_{v,Rd}$, it is necessary to distinguish a size of the fastener. For nails ≤ 8 mm in diameter, the resistance will be the same as for the fastener loaded parallel to the grain. For bolts and nails > 8 mm in diameter, the resistance derived from the strength equations (4.1)-(4.12) will have to take into account the requirements of equation (4.20) where the characteristic embedment strength of the timber in the connection will become:

$$f_{h,90,k} = \frac{f_{h,0,k}}{k_{90}} \tag{4.38}$$

which is the equation (4.20) with $\theta = 90°$.

From the above, the design load-bearing capacity of a connection loaded perpendicular to the grain, $F_{Rd}$, will be:

$$F_{Rd} = \min \begin{Bmatrix} F_{90,Rd} \\ F_{v,ef,Rd} \end{Bmatrix} \tag{4.39}$$

where $F_{90,Rd}$ is the design splitting resistance of the timber obtained as:

$$F_{90,Rd} = k_{\text{mod}} \cdot \frac{F_{90,Rk}}{\gamma_M} \tag{4.40}$$

and $F_{90,Rk}$ is the characteristic design splitting resistance of timber calculated according to equation (4.26), $k_{mod}$ is a modification factor given in Table 4.6, $\gamma_M$ is the partial factor for a material property at the ULS.

# 4.4 Axial Loading of Metal Dowel Type Fasteners

The strength equations given in the following sub-sections assume that the tensile strength of fasteners will always exceed their withdrawal resistance from the connection. If, however, there is a need to evaluate the tensile strength of the fastener, it should be carried out according to the requirements of EN 1993-1-1 – Eurocode 3, Design of Steel Structures, General Rules and Rules for Buildings.

## 4.4.1 Axially Loaded Nails

The withdrawal resistance of nails loaded axially is dependent on the type of used nail. Smooth round wire nails give the poorest result and with threaded nails the resistance is greatly increased. However, no matter the type, nails are not considered capable of sustaining axial load in the end grain. A typical case of the axial loading is illustrated in Figure 4.17.

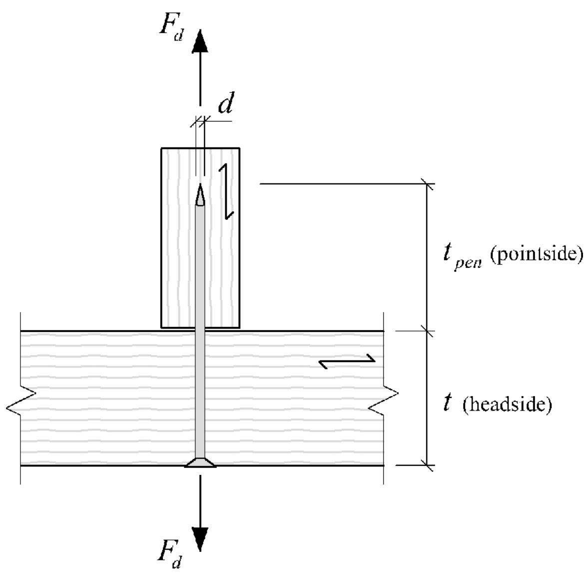



*Fig. 4.17: Nailing in tension*

Also, EC5 does not permit axially loaded smooth nails to be used in situations involving permanent or long-term loading. And where threaded nails are used, only the threaded part of the nail is taken as relevant for determining the nail strength.

Ignoring tensile failure of the nail, there are two possible failure modes when subjected to the axial loading, either pointside withdrawal of the nail or pull-through of the nail head.

However, in a case of the steel-to-timber connection is the head pull-through failure nearly impossible.

For threaded nails, only the threaded part of the nail is considered to be capable of transmitting axial load, consequently the headside resistance can only utilise the head pull-through resistance. With smooth nails, there will be withdrawal resistance for the pointside penetration, and the headside resistance takes both the headside pull-through strength and the shank friction resistance on the headside of the nail into account.

When a nail is subjected to an axial force, $F_d$, the following condition has to be satisfied:

$$F_d \leq F_{ax,Rd} \tag{4.41}$$

where $F_{ax,Rd}$ is the design withdrawal resistance of the nail.

The design withdrawal resistance of the nail is obtained from the characteristic withdrawal resistance, $F_{ax,Rk}$, according to equation (4.34) as follows:

$$F_{ax,Rd} = k_{\text{mod}} \cdot \frac{F_{ax,Rk}}{\gamma_M} \tag{4.42}$$

where $k_{mod}$ is a modification factor given in Table 4.6, $\gamma_M$ is the partial factor for a material property at the ULS, and $F_{ax,Rk}$ is the characteristic withdrawal resistance of a nail derived as follows:

- for nails, other than smooth wire nail:

$$F_{ax,Rk} = \min\begin{Bmatrix} f_{ax,k} \cdot d \cdot t_{pen} \\ f_{head,k} \cdot d_h^{\,2} \end{Bmatrix} \tag{4.43}$$

- for smooth wire nails:

$$F_{ax,Rk} = \min\begin{Bmatrix} f_{ax,k} \cdot d \cdot t_{pen} \\ f_{ax,k} \cdot d \cdot t + f_{head,k} \cdot d_h^{\,2} \end{Bmatrix} \tag{4.44}$$

where $f_{ax,Rk}$ is the characteristic pointside withdrawal strength, $f_{head,k}$ is the characteristic headside pull-through strength, $d$ is the nail diameter, $t_{pen}$ is the pointside penetration or the length of the threaded part in the pointside member, $t$ is the thickness of the headside member, $d_h$ is the diameter of the nail head. However, as it was said above, in a case of the steel-to-timber connection is the head pull-through failure nearly impossible. Therefore, the second parts of the equations (4.43) and (4.44) are not relevant.

Values for $f_{ax,Rk}$ can be determined by testing, and EC5 gives the following value for smooth nails with a pointside penetration of at least 12$d$:

$$f_{ax,k} = 20 \cdot 10^{-6} \rho_k^{\,2} \qquad [\text{N/mm}^2] \tag{4.45}$$

where $\rho_k$ is the characteristic timber density in kg/m$^3$. If the pointside nail penetration is less than 12$d$ the withdrawal resistance of the nail has to be linearly reduced by multiplying by the factor (($t_{pen}$ / 4$d$)-2). When the minimum nail penetration 8$d$ for smooth nails is used, the factor will be zero and there will be no axial withdrawal strength.

The annular ring nails have the value of $f_{ax,Rk}$ several times higher than the smooth nails. Therefore, it is also possible to use the equation (4.45) for them but the design of the joint would be very uneconomic. Werner and Siebert have done a set of experiments with annular ring nails from four different producers, and the have analyzed their behaviour during an axial loading. From their paper results, that for the characteristic pointside withdrawal strength of the annular ring nails the following equation should be used:

$$f_{ax,Rk} = 65 \cdot 10^{-6} \rho_k^{\ 2} \qquad [\text{N/mm}^2] \tag{4.46}$$

where $\rho_k$ is the characteristic timber density in kg/m$^3$. When the annular ring nails are used, the threaded length of the shank should be at least 6$d$ and the pointside penetration must also be at least 6$d$. For a threaded pointside nail penetration of 8$d$ the full value of the characteristic withdrawal strength can be used and for values less than this the strength is reduced linearly by multiplying by (($t_{pen}$ / 2$d$)-3).

Where structural timber has been designed to function under service class 1 or 2 conditions but will possibly be installed at or near the fibre saturation point[10], the values of $f_{ax,Rk}$ must be multiplied by 2/3 to take into account the reduction in the respective strength when drying out.

The spacings, end and edge distances for axially loaded nails are the same as those given in EC5 for laterally loaded nails.

### 4.4.2 Axially Loaded Bolts

In case of axially loaded bolts, the strength of the connection is dependent on the tensile strength of the bolt and the bearing strength of the material onto which the bolt washer beds. The tensile strength of the bolt is derived using the strength equations in EN 1993-1-1.

When bearing onto timber or wood-based products, the bearing capacity below the washer should be calculated assuming a 300% increase in the characteristic strength of the timber perpendicular to the grain over the contact area, it is $f_{c,k} = 3{,}0 \times f_{c,90,k}$ .

When using a steel plate, the bearing capacity per bolt should not exceed that of a circular washer.

## 4.5 Axial Loading due to Eccentricity

It can happen that there will be an additive loading of a fastener due to a spatial eccentricity of a shaped metal work connector. Than the applied axial force, $F_{ax,Ed}$, has to be increased by an additional axial force caused by a bending moment, $F_M$, which tends to withdraw the connectors. According to Figure 4.18, the most loaded fastener is the first one (i.e. the highest placed fastener) and the lower part of the connector is embedded into a timber element. If there is an elastic distribution of internal forces then a similarity of triangles is valid:

$$\frac{N_1}{r_1} = \frac{N_2}{r_2} = \frac{N_3}{r_3} = \ldots = \frac{N_n}{r_n} \tag{4.47}$$

where $N_i$ are axial forces caused by bending moment and $r_i$ are levers of the axial forces.

---

[10] Fibre saturation point is a term used to denote the point in the drying process at which only water bound in the cell walls remains - all other water, called free water, having been removed from the cell cavities. Further drying of the wood results in strengthening of the wood fibres, and is usually accompanied by shrinkage.

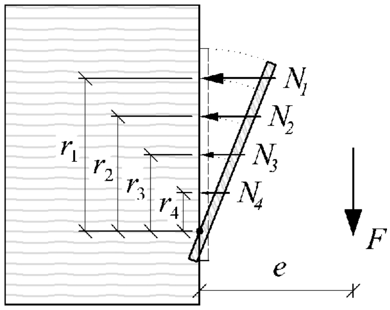



*Fig. 4.18: Linear distribution of axial forces caused by bending moment*

Values of the $r_i$ depend on a position of the centre of the rotation. After a simplification, the position of the centre of the rotation can be taken as the position of the lowest placed fastener. In real, the position of the centre of the rotation depends on geometry and stiffness of the connector. It is somewhere below the lowest placed fastener and therefore, the simplification is on the safe side. The bending moment, $M$, acting on the connector is:

$$M = F \cdot e \tag{4.48}$$

where $F$ is the force causing the bending moment and $e$ is the eccentricity.

It is obvious that the bending moment has to be equal to a sum of products of the axial forces caused by moment and the levers of the axial forces written as:

$$M = N_1 \cdot r_1 + N_2 \cdot r_2 + N_3 \cdot r_3 + ... + N_n \cdot r_n = \sum_{i=1}^{n} N_i \cdot r_i = F \cdot e \tag{4.49}$$

The equation (4.47) says that it is possible to express all the forces $N_i$ with help of the force $N_1$ as:

$$N_i = N_1 \cdot \frac{r_i}{r_1} \tag{4.50}$$

After substituting the equation (4.50) into the equation (4.49):

$$M = N_1 \cdot r_1 + N_1 \cdot \frac{r_2}{r_1} \cdot r_2 + N_1 \cdot \frac{r_3^{\,2}}{r_1} + ... + N_1 \cdot \frac{r_n^{\,2}}{r_1} = N_1 \cdot \sum_{i=1}^{n} \frac{r_i^{\,2}}{r_1} = F \cdot e \tag{4.51}$$

The greatest withdrawal force caused by the bending moment will be acting on the first fastener (i.e. the highest placed fastener in Figure 4.18) and the force can be calculated as follows:

$$F_M = N_1 = \frac{F \cdot e}{\sum_{i=1}^{n} \frac{r_i^{\,2}}{r_1}} \tag{4.52}$$

# 4.6 Combined Laterally and Axially Loaded Nailed Connections

When nailed connections are subjected to the combination of a lateral design load, $F_{v,Ed}$, and an axial design load, $F_{ax,Ed}$, they must comply with the interaction diagrams shown in Figure 4.19 and Figure 4.20.

$$\frac{F_{ax,Ed}}{F_{ax,Rd}} + \frac{F_{v,Ed}}{F_{v,Rd}} \leq 1 \tag{4.53}$$

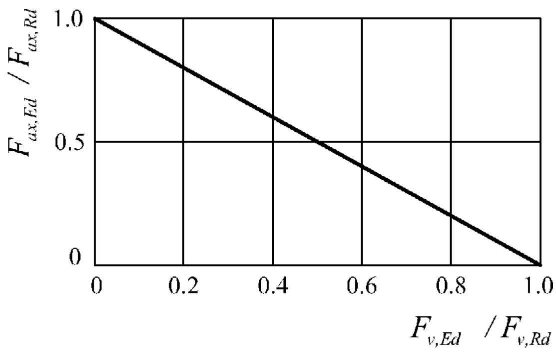



*Fig. 4.19: Interaction diagram for combined axially and laterally loaded smooth nails*

$$\left(\frac{F_{ax,Ed}}{F_{ax,Rd}}\right)^2 + \left(\frac{F_{v,Ed}}{F_{v,Rd}}\right)^2 \leq 1 \tag{4.54}$$

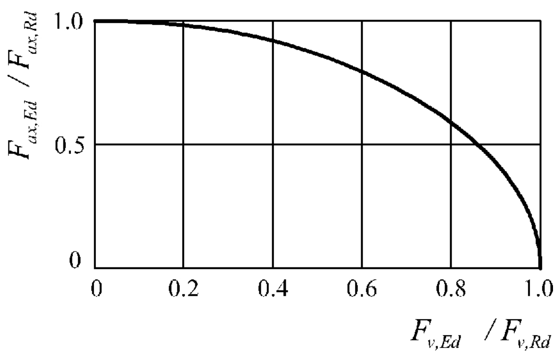



*Fig. 4.20: Interaction diagram for combined axially and laterally loaded other than smooth nails*

For smooth nails the combined design force to strength ratios must stay within the elastic range, resulting in a linear relationship in accordance with equation (4.53). The combined ratios for other nails type can extend beyond the elastic limit but must comply with the power function given in equation (4.54), where $F_{ax,Rd}$ is the design strength of the connection loaded axially, and $F_{v,Rd}$ is the design strength of the connection loaded laterally.

# 5 Evaluation of Joints

In the previous chapter, there were introduced a methods how to evaluate load-bearing capacities of the nails and the timber elements. This chapter is dedicated to the application of these methods and also to a numerical analysis of a steel element. In order to work with real products and numbers, the products of the Czech company Bova-nail are used.

## 5.1 Numerical Model of Steel Connector

The steel connecting plate BV/DS 03-32, illustrated in Figure 5.1, loaded by tension (in the same way as it is loaded in chapter 5.2.2.4) was used for this task.

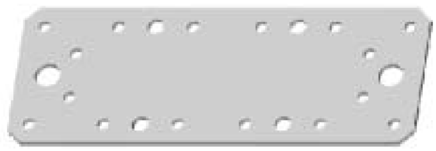


*Fig. 5.1: Connecting plate BV/DS 03-32*

To determine a load-bearing capacity of the plate the numerical model, using the finite element method, was created with help of the software ADINA Structures (Automatic Dynamic Incremental Nonlinear Analysis) version 8.5.0. The connector was made of cold worked steel and this kind of steel have no definite yield point, therefore, the values from the experiments done at the Faculty of Civil Engineering, Czech Technical University in Prague were taken. For yield strength the value 273,7 MPa and for ultimate limit strength the value 344,7 MPa were used.

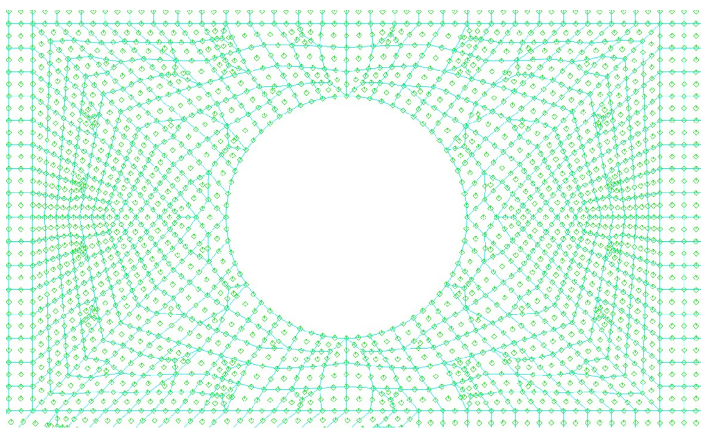


*Fig. 5.2: Detail of mesh with nodes around hole*

The plain stress behaviour and 2D model were used in this case. Very fine meshing with elements 0,5 x 0,5 mm and 9 nodes per element was used. Preferred shape of cells was quadrilateral shape as it gives better results, however around the holes the triangular shape was necessary (Figure 5.2).

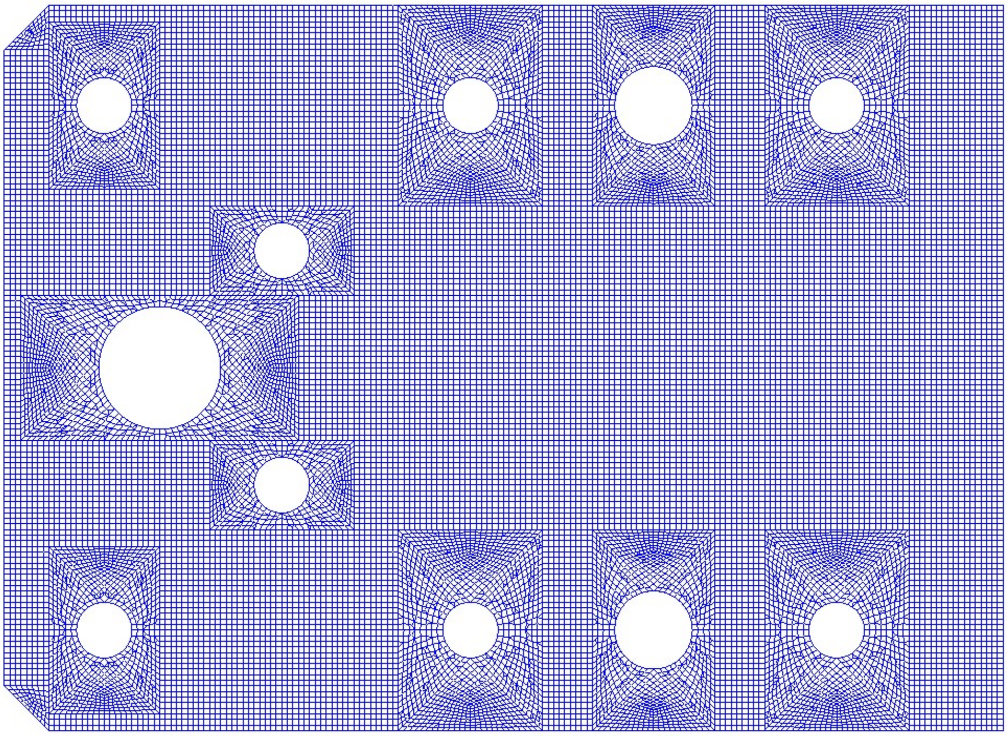


*Fig. 5.3: Original mesh of whole element*

For simplicity, the symmetry of the connecting plate was employed. To ensure a realistic behaviour of such a model, the boundary conditions had to be defined properly. The fixed joint was applied in the middle of an axis of symmetry whereas on the rest of the axis, the sliding pins were applied, as shown in Figure 5.4. This configuration enables the plate to contract in accordance with assumption.

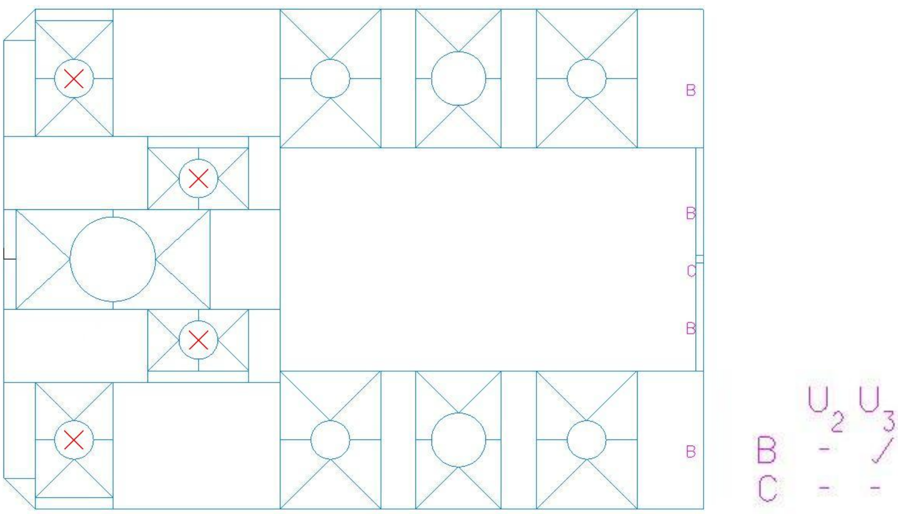



*Fig. 5.4: Symmetry and boundary conditions*

The loading was applied only to the four holes, for the nails with diameter 4 mm, marked by the red crosses, as you can see in Figure 5.4. This comes from the requirement of the EC5 on "loaded end distance" (for more information see chapter 4.1.6).

Since the wood as material was not a part of the model, the two possible scenarios had to be taken into account. All the holes were loaded either by the same forces (respectively pressures) or by the same displacements.

### 5.1.1 Holes Loaded by Equivalent Forces

This scenario can be used under the assumption that after loading by tensile force, the timber will behave as a compliant material and it will allow the nails to deform in the timber in such a way that the applied force will be spread equally among the nails.

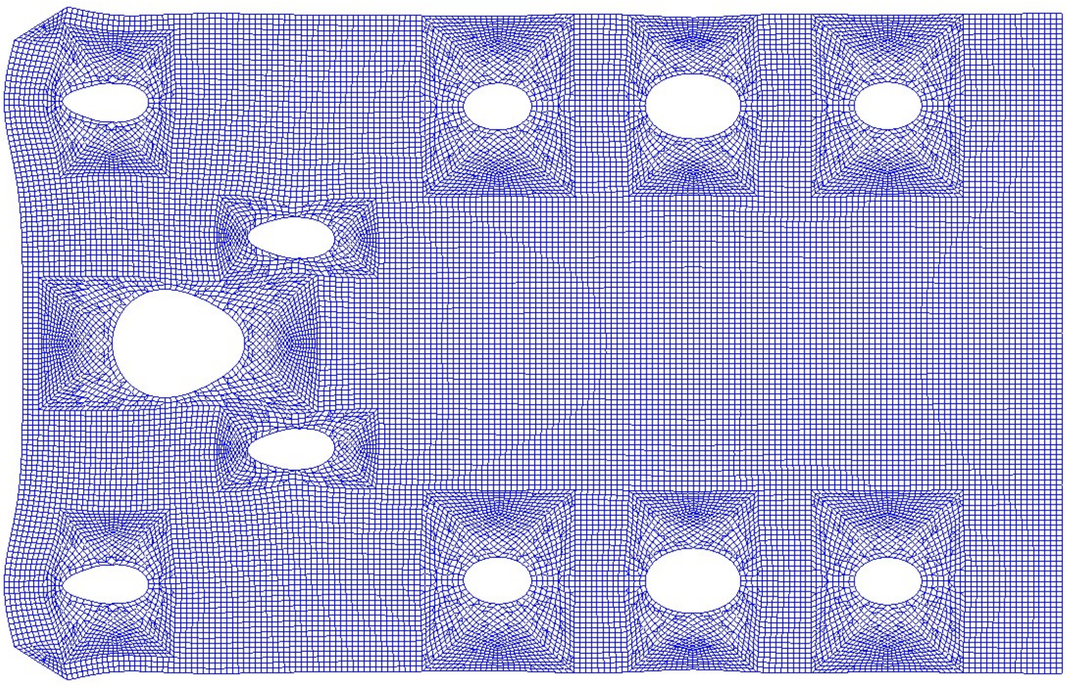


*Fig. 5.5: Deformed mesh (magnified app. 300x) of whole element during loading by equivalent forces*

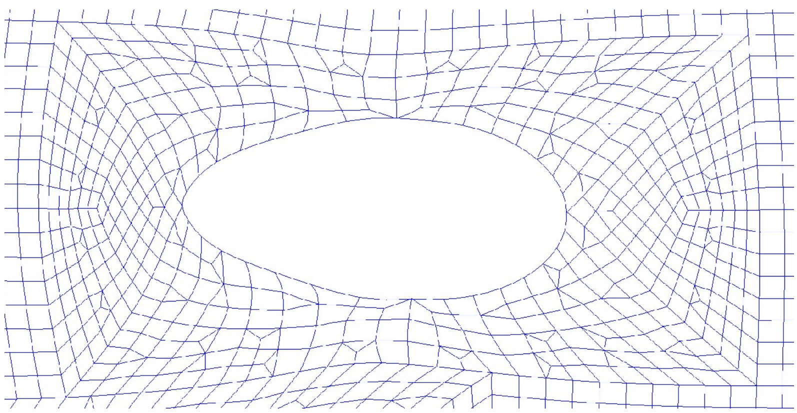


*Fig. 5.6: Detail of deformed mesh around hole during loading by equivalent forces*

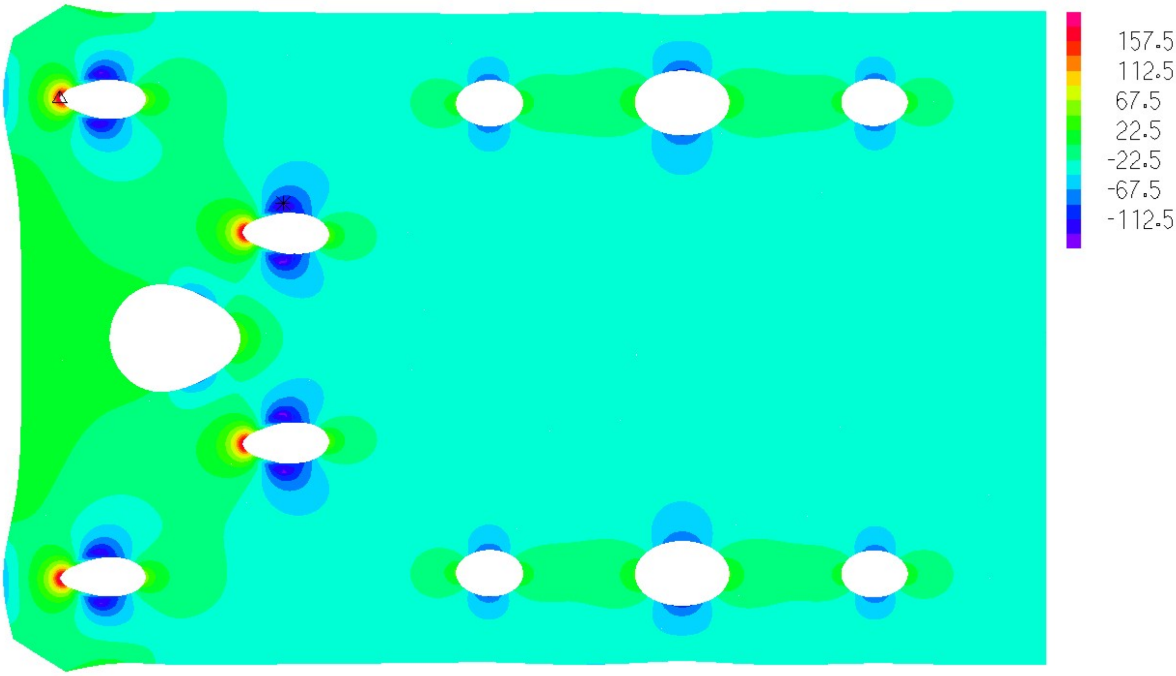



*Fig. 5.7: Band plot of whole element during loading by equivalent forces*

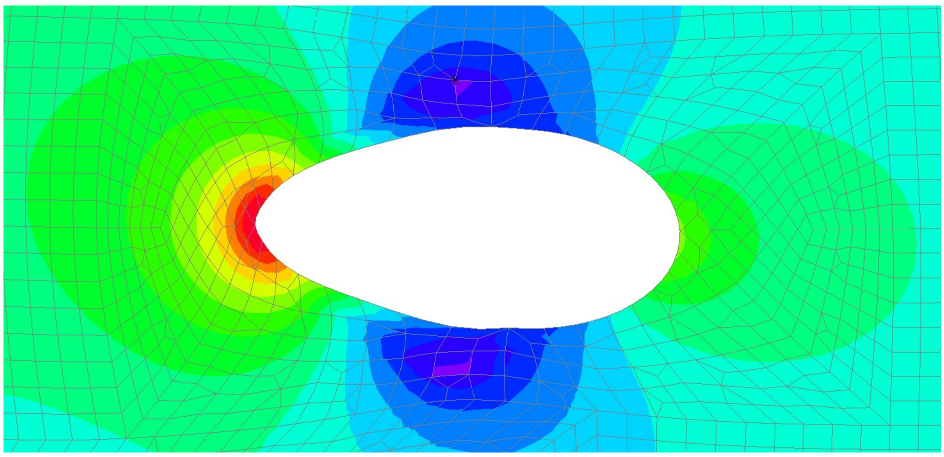


*Fig. 5.8: Band plot around hole during loading by equivalent forces*

It is possible to see that the stresses around all four loaded holes are very similar. The conclusion from this analysis is that the element loaded in the four holes by the four equivalent forces can withstand force 15,0 kN. It means that one most loaded hole (it is the one of the two holes which are closer to the left edge) can withstand force 3,75 kN. With respect to the fact that the maximum characteristic shear resistance of the annular ring nail is 1,91 kN, it is clear that strength of the steel plate will definitely not be decisive.

### 5.1.2 Holes Loaded by Equivalent Displacements

This scenario can be used under the assumption that after loading by tensile force, the timber will behave as a rigid material and it will not allow the nails to deform and therefore, there will be an equivalent displacement in all loaded holes.

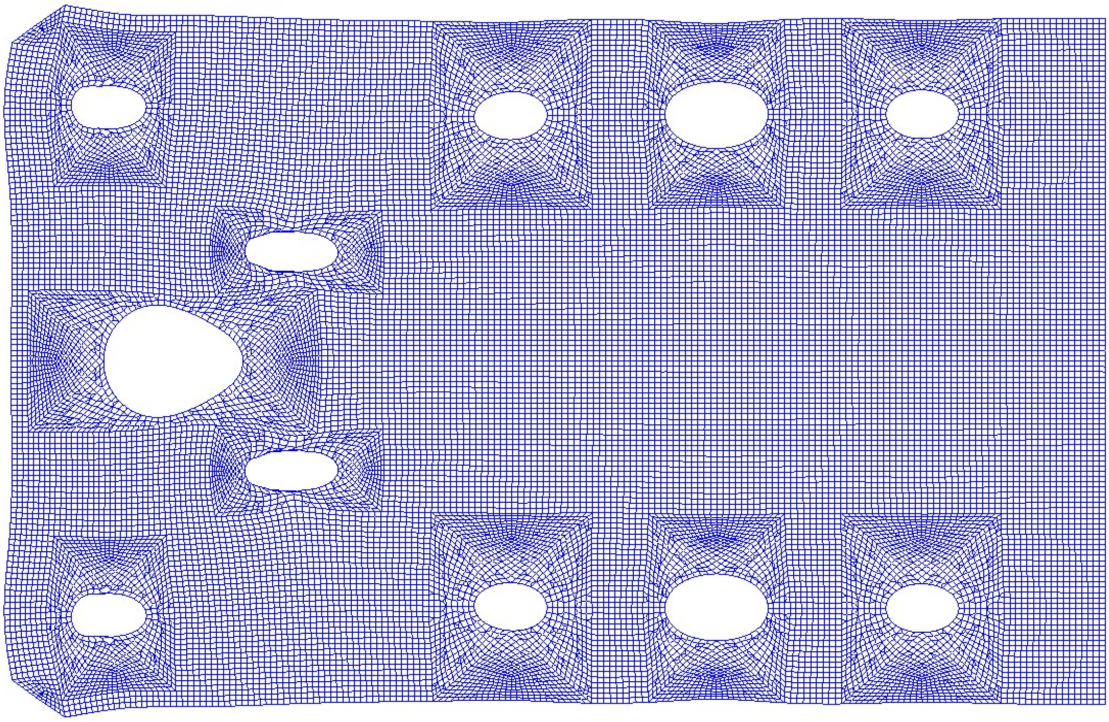


*Fig. 5.9: Deformed mesh (magnified app. 300x) of whole element during loading by equivalent displacements*

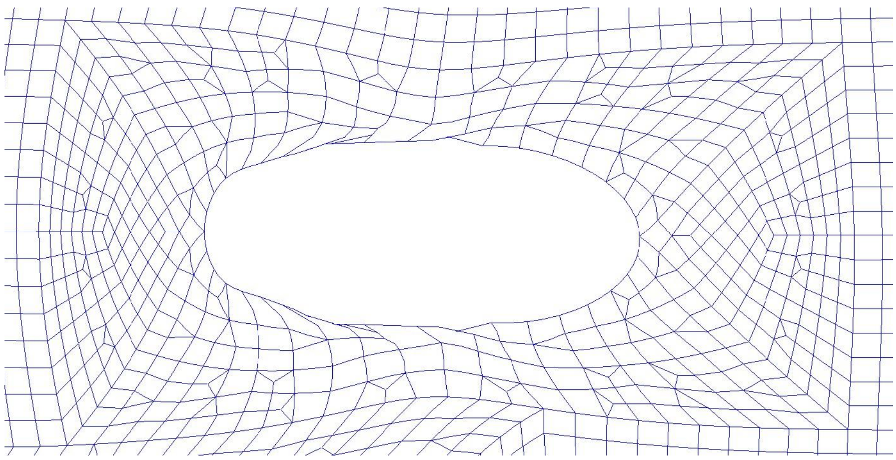


*Fig. 5.10: Detail of deformed mesh around hole during loading by equivalent displacements*

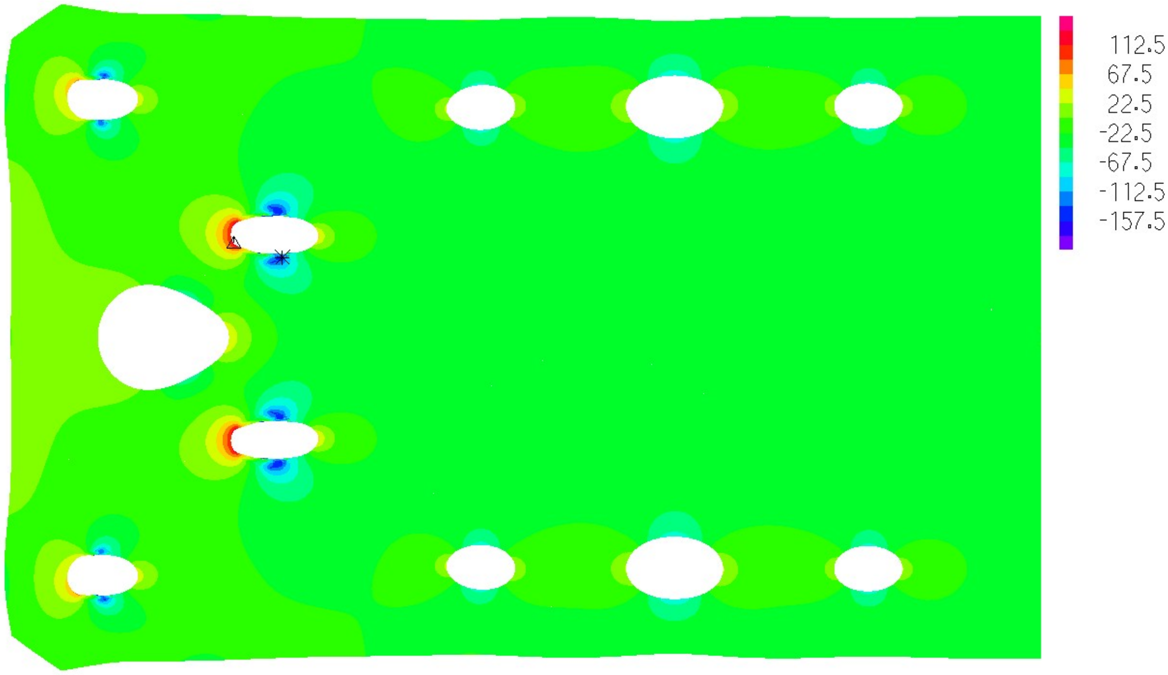



*Fig. 5.11: Band plot of whole element during loading by equivalent displacements*

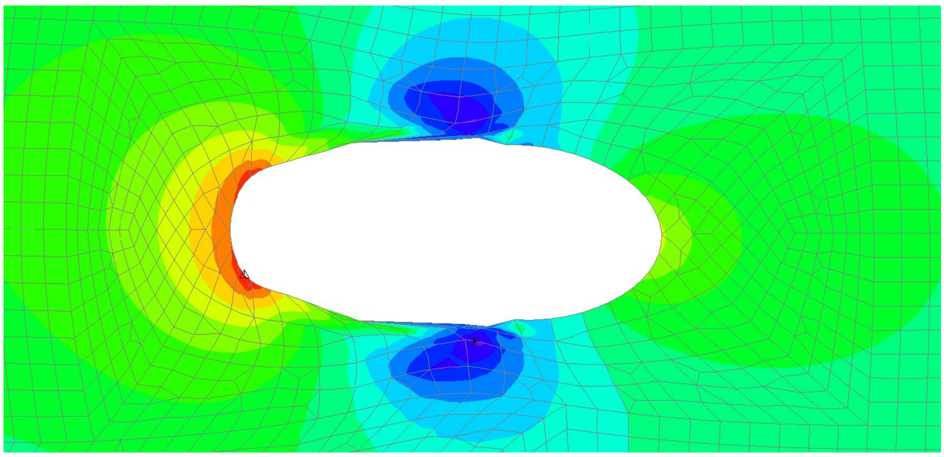


*Fig. 5.12: Band plot around hole during loading by equivalent displacements*

It is possible to see that the stresses around loaded holes are different. It is obvious that the two holes which are further from the left edge are more stressed than the two holes closer to the left edge. The reason why it is so is that there is more material in front of the more stressed holes; it means that they are stiffer and they therefore take higher stress. The conclusion from this analysis is that the element loaded in the four holes by the four equivalent displacements can withstand force 13,25 kN. It means that one most loaded hole (it is the one of the two holes which are further from the left edge) can withstand force 4,4 kN. With respect to the fact that the maximum characteristic shear resistance of the annular ring nail is 1,91 kN, it is clear that strength of the steel plate will definitely not be decisive.

## 5.2 Evaluation of Fasteners and Timber Elements

In this chapter, the Excel application was programmed using the procedures stated in the chapter 4. The following part is done in a form of a comparison between values evaluated with help of EC5 and values used in a catalogue (a second edition) of the Bova-nail company which uses ČSN.

### 5.2.1 Annular Ring Nails

The values of shear resistance and axial withdrawal resistance written in the catalogue of the Bova-nail company are calculated using ČSN and the timber grades are not taken into consideration here. For the comparison with the EC5 results, the three most frequented timber grades were used.

The compared annular ring nails have still the same diameters but the length changes. It is obvious that the length of nail shank has very small influence on shear resistance whereas it has significant influence on axial withdrawal resistance. The timber grade influences both resistances but the axial withdrawal resistance is influenced more. All the results of the comparison are summarized in Table 5.1.

*Table 5.1: Comparison of shear and axial withdrawal resistances according to EC5 and Bova-nail company*

| | EC5 | | | | | | BOVA | |
|---|---|---|---|---|---|---|---|---|
| | C16 | | C24 | | C30 | | | |
| Nail Dimensions | $F_{v,Rd}$ [kN] | $F_{ax,Rd}$ [kN] | $F_{v,Rd}$ [kN] | $F_{ax,Rd}$ [kN] | $F_{v,Rd}$ [kN] | $F_{ax,Rd}$ [kN] | $F_{v,Rd}$ [kN] | $F_{ax,Rd}$ [kN] |
| 4,0×40 | 0,80 | 0,43 | 0,85 | 0,55 | 0,88 | 0,65 | 0,71 | 0,38 |
| 4,0×50 | 0,80 | 0,55 | 0,85 | 0,70 | 0,88 | 0,82 | 0,71 | 0,51 |
| 4,0×60 | 0,80 | 0,66 | 0,85 | 0,85 | 0,88 | 1,00 | 0,71 | 0,64 |
| 4,0×70 | 0,80 | 0,78 | 0,85 | 0,99 | 0,88 | 1,17 | 0,71 | 0,75 |
| 4,0×80 | 0,80 | 0,89 | 0,85 | 1,14 | 0,88 | 1,34 | 0,71 | 0,88 |
| 4,0×90 | 0,80 | 1,01 | 0,85 | 1,29 | 0,88 | 1,52 | 0,71 | 0,96 |

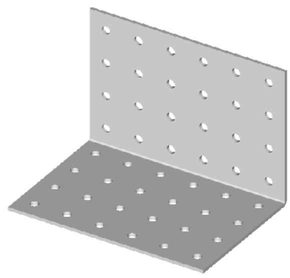
### 5.2.2 Joints

Three different products from Bova-nail company were chosen and analysed from the point of view of the position of the steel connector, number and type of the fasteners (sometimes also their position in the steel connector), timber grade, edge and end distances and spacings. For the analysis, the Excel application with EC5 formulas was used again. The spacings, and edge and end distances computed according to EC5 and ČSN were compared.

All the results were finally summarized in tables. There are two tables for each connector which are always followed by notes, emphasizing or explaining important facts.

### 5.2.2.1 Angle 05-01

diameter of nails: 4 mm
length of nails: 50 mm
angle of force acting on
the elements: 90° (resp. 270°)
height of timber elements: 160 mm
width of timber elements: 100 mm



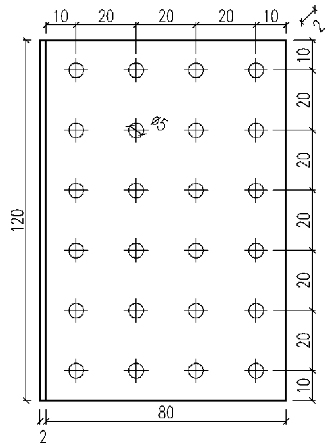



*Fig. 5.13: Dimensions and layout of the angle*

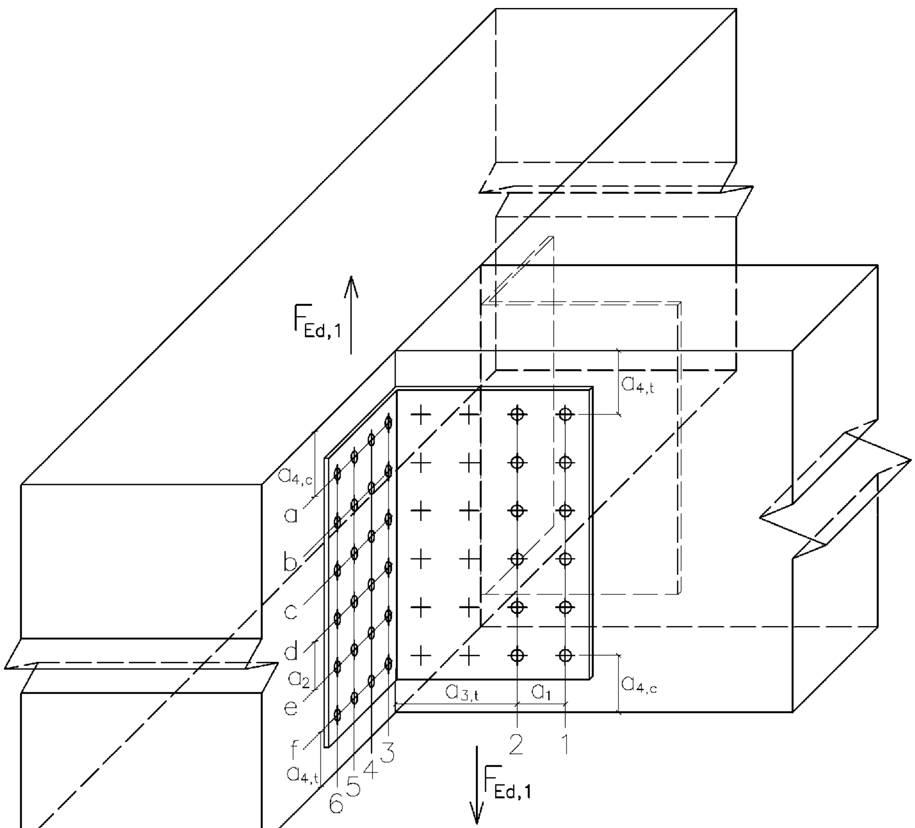



*Fig. 5.14: Model of the angles and their connection*

*Table 5.2: Maximum design values of force $F_{Ed,1}$ which can be applied on the angle 05-01 according to EC5*

| $F_{Ed,1,max}$ [kN] | | Smooth Nails | | | Annular Ring Nails | | |
|---|---|---|---|---|---|---|---|
| | | C16 | C24 | C30 | C16 | C24 | C30 |
| Fully Used Columns/ Columns | 3/2 | 2,25 | 2,62 | 2,90 | 4,55 | 4,91 | 5,15 |
| | 3,4/2 | 4,50 | 4,83 | 5,08 | 4,79 | 5,09 | 5,30 |
| | 3,4,5/2 | 4,50 | 4,83 | 5,08 | 4,79 | 5,09 | 5,30 |
| | 3,4,5,6/2 | 4,50 | 4,83 | 5,08 | 4,79 | 5,09 | 5,30 |
| | 3/1,2 | 2,04 | 2,40 | 2,67 | 4,46 | 4,83 | 5,09 |
| | 3,4/1,2 | 4,09 | 4,81 | 5,34 | 8,91 | 9,67 | 10,18 |
| | 3,4,5/1,2 | 6,13 | 7,21 | 8,01 | 9,58 | 10,18 | 10,60 |
| | 3,4,5,6/1,2 | 8,18 | 9,61 | 10,16 | 9,58 | 10,18 | 10,60 |
| Fully Used Rows/ Columns | a/2 | 0,94 | 1,13 | 1,28 | 2,59 | 2,91 | 3,12 |
| | a,b/2 | 2,56 | 3,02 | 3,37 | 4,79 | 5,09 | 5,30 |
| | a,b,c/2 | 4,28 | 4,83 | 5,08 | 4,79 | 5,09 | 5,30 |
| | a,b,c,d/2 | 4,50 | 4,83 | 5,08 | 4,79 | 5,09 | 5,30 |
| | a,b,c,d,e/2 | 4,50 | 4,83 | 5,08 | 4,79 | 5,09 | 5,30 |
| | a,b,c,d,e,f/2 | 4,50 | 4,83 | 5,08 | 4,79 | 5,09 | 5,30 |
| | a/1,2 | 0,82 | 1,00 | 1,13 | 2,41 | 2,75 | 2,98 |
| | a,b/1,2 | 2,30 | 2,73 | 3,06 | 5,65 | 6,21 | 6,59 |
| | a,b,c/1,2 | 3,88 | 4,58 | 5,10 | 8,79 | 9,57 | 10,10 |
| | a,b,c,d/1,2 | 5,40 | 6,35 | 7,06 | 9,58 | 10,18 | 10,60 |
| | a,b,c,d,e/1,2 | 6,81 | 8,01 | 8,90 | 9,58 | 10,18 | 10,60 |
| | a,b,c,d,e,f/1,2 | 8,18 | 9,61 | 10,16 | 9,58 | 10,18 | 10,60 |

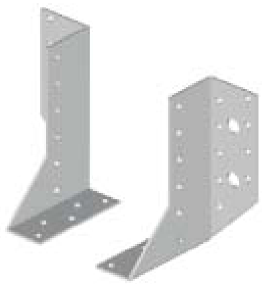
*Table 5.3: Comparison of spacings according to EC5 and ČSN 73 1701*

| **Spacing and Distance [mm]** | | Nails | | |
|---|---|---|---|---|
| | | EC5 | ČSN | Satisfactory |
| Spacings [mm] | $a_1$ | 14 | 60 | ✓ |
| | $a_2$ | 14 | 16 | ✓ |
| End and Edge Distances [mm] | $a_{3,t}$ | 40 | 60 | ✓ |
| | $a_{4,t}$ | 28 | 28 | ✓ |
| | $a_{4,c}$ | 20 | 16 | ✗ |

NOTES:
- the angle always has to be placed on the both sides of the connected elements
- only the holes marked by a cross and a circle can be used (see Figure 5.13)
- the numbers 3 to 6 in front of a slash sign columns of holes, in the left wing of the angle, which are fully occupied by nails and the letters "a" to "f " in front of a slash sign rows of holes, in the left wing of the angle, which are fully occupied by nails (it is illustrated in Figure 5.14)

- the numbers 1 and 2 behind the slash sign columns of holes, in the right wing of the angle, which are fully occupied by nails (as shown in Figure 5.14)
- in the right wing of the angle, there are always used only two columns of holes because the remaining two columns do not satisfy the recommendation of a "loaded end distance" (for more information see chapter 4.1.6)
- in this case, the nails used in the columns 3 to 6 and the rows "a" to "e" are subjected to both shear force and axial force because of a spatial eccentricity of the angle whereas the columns 1 and 2 are subjected only to shear force and therefore, the load-bearing capacities stated in Table 5.2 are results of comparison of combined loading resistance of nails in the left wing of the angle and the shear resistance of nails in the right wing of the angle, and the lower values are used
- resistances of the annular ring nails are approximately two times higher than in case of the smooth nails as stated in Table 5.2, it is due to significantly higher axial withdrawal resistance of the annular ring nails, but also due to the fact that for the annular ring nails the combined design force to strength ratios must stay within the plastic range, whereas for the smooth nails it must stay within the elastic range (for more information see chapter 4.6)
- the fact, in Table 5.2, that the resistances of the annular ring nails are approximately two times higher than in case of the smooth nails is valid only till the moment when the combined loading resistance of the nails in columns 3 to 6 and rows "a" to "f" exceeds the shear resistance of the nails in columns 1 and 2, then the shear resistance becomes decisive and since the shear resistances of the annular ring nails and smooth nails are very similar, also the load-bearing capacities of the whole joints, for both types of the nails, are very similar
- the angles of forces acting on the elements (see Figure 5.14) are 90° respectively 270°, it means that there is a danger of a "brittle failure" (for more information see chapter 4.1.6) and the forces therefore must not exceed the $F_{90,Rd}$ which is 17,01 kN

### 5.2.2.2 Hangers BV/T – P,L 11-20



| | |
|---|---|
| diameter of nails: | 4 mm |
| length of nails: | 50 mm |
| diameter of bolts: | 10 mm |
| angle of force acting on the element: | 90° |
| height of timber elements: | 150 mm |
| width of timber elements: | 100 mm |

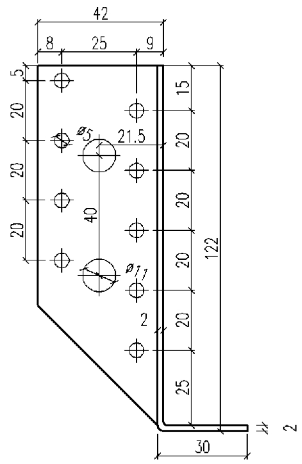



*Fig. 5.15: Dimensions and layout of the hanger BV/T – P,L 11-20*

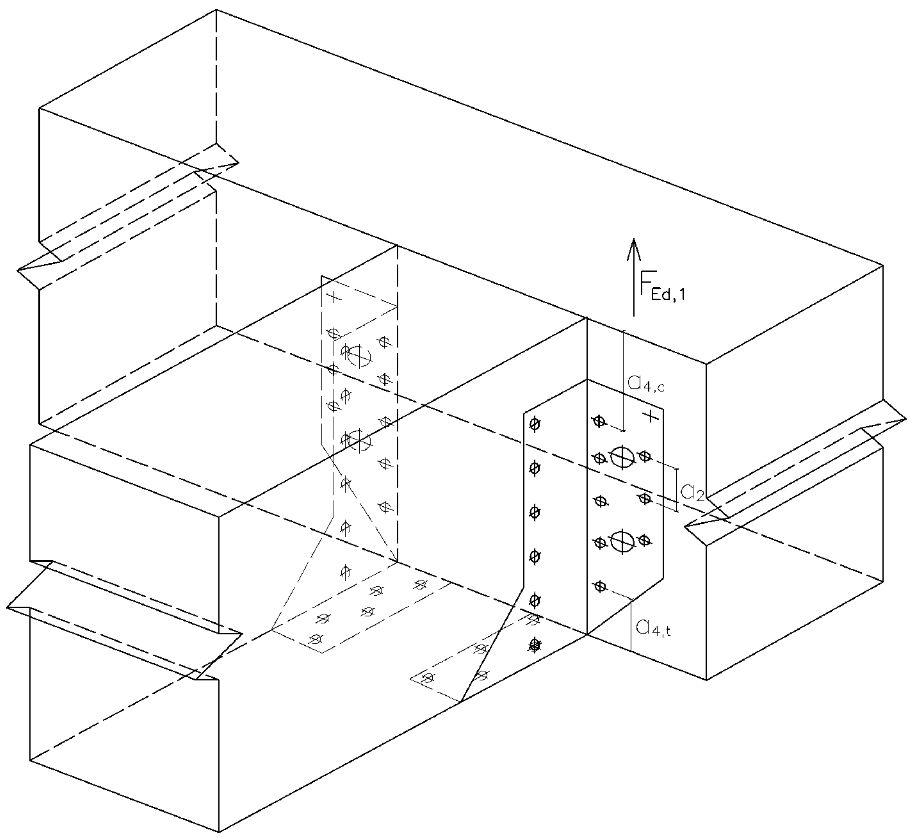



*Fig. 5.16: Model of the hangers and their connection*

*Table 5.4: Maximum design values of force $F_{Ed}$ which can be applied on the hanger BV/T – P,L 11-20 according to EC5*

| **$F_{Ed,1,max}$ [kN]** | | Smooth Nails | | | Annular Ring Nails | | | Bolts | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | C16 | C24 | C30 | C16 | C24 | C30 | C16 | C24 | C30 |
| Number of Connectors | 2 | 1,50 | 1,61 | 1,69 | 1,60 | 1,70 | 1,77 | 8,43 | 9,32 | *9,71* |
| | 3 | 2,25 | 2,42 | 2,54 | 2,39 | 2,54 | 2,65 | × | × | × |
| | 4 | 3,00 | 3,22 | 3,39 | 3,19 | 3,39 | 3,53 | × | × | × |
| | 5 | 3,75 | 4,03 | 4,23 | 3,99 | 4,24 | 4,42 | × | × | × |
| | 6 | 4,50 | 4,83 | 5,08 | 4,79 | 5,09 | 5,30 | × | × | × |
| | 7 | 5,25 | 5,64 | 5,93 | 5,59 | 5,94 | 6,18 | × | × | × |
| | 8 | 6,00 | 6,45 | 6,78 | 6,38 | 6,78 | 7,07 | × | × | × |

*Table 5.5: Comparison of spacings according to EC5 and ČSN 73 1701*

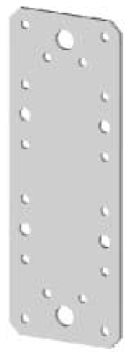
| **Spacing and Distance [mm]** | | Nails | | | Bolts | | |
|---|---|---|---|---|---|---|---|
| | | EC5 | ČSN | Satisfactory | EC5 | ČSN | Satisfactory |
| Spacings [mm] | $a_2$ | 14 | 16 | ✓ | 40 | 30 | × |
| End and Edge Distances [mm] | $a_{4,t}$ | 28 | 28 | ✓ | 40 | 25 | × |
| | $a_{4,c}$ | 20 | 16 | × | 30 | 25 | × |

NOTES:

- the hanger always has to be placed on the both sides of the connected elements
- only the holes marked by a cross and a circle can be used (see Figure 5.15)
- in this case, all the fasteners are subjected only to shear force
- the angle of force acting on the elements (see Figure 5.16) is 90°, it means that there is a danger of a "brittle failure" (for more information see chapter 4.1.6) and the forces therefore must not exceed the $F_{90,Rd}$ which is 14,34 kN for nailed connections and 9,69 kN for bolted connection
- the value written in italic (2 bolts and timber grade C30 in Table 5.4) is higher than $F_{90,Rd}$ for bolted connection, it means that the bolts would withstand the force 9,71 kN but timber element would not, therefore, the maximum force which can be applied is 9,69 kN

### 5.2.2.3 Connecting Plate BV/DS 03-32 – perpendicular elements

| | |
|---|---|
| diameter of nails: | 4 mm |
| length of nails: | 50 mm |
| diameter of bolts: | 10 mm |
| angle of force acting on the first element: | 0° |
| angle of force acting on the second element: | 90° |
| height of timber elements: | 100 mm |
| width of timber elements: | 100 mm |



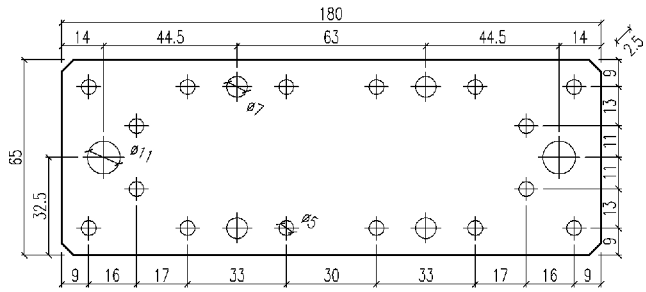



*Fig. 5.17: Dimensions and layout of the connecting plate*

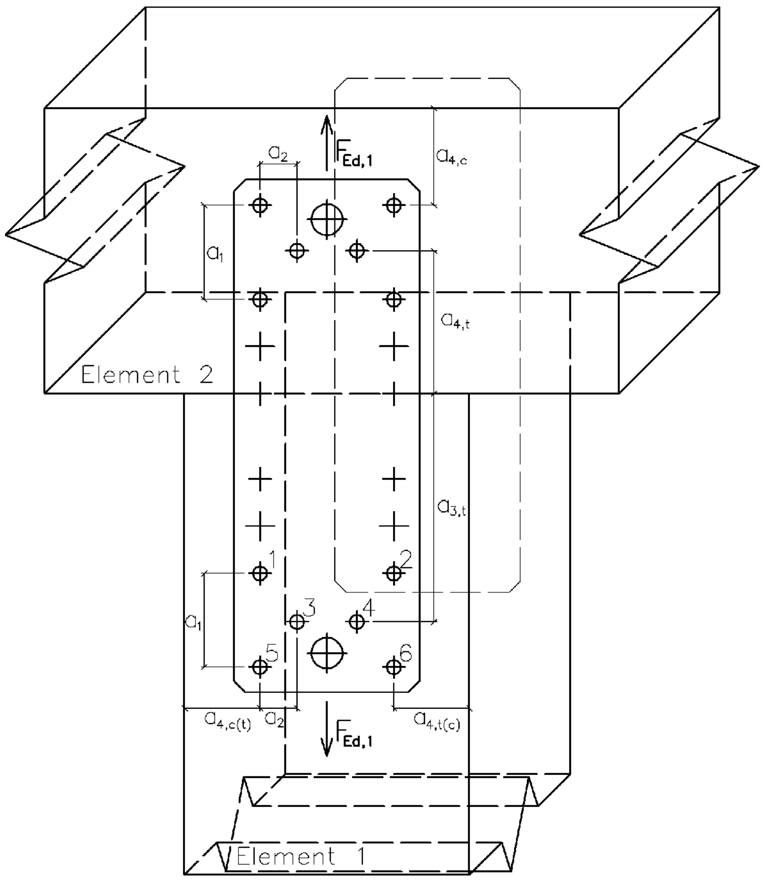



*Fig. 5.18: Model of the connecting plates and their connection*

*Table 5.6: Maximum design values of force $F_{Ed}$ which can be applied on the connecting plate BV/DS 03-32 according to EC5*

| $F_{Ed,1,max}$ [kN] | | Smooth Nails | | | Annular Ring Nails | | | Bolts | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Number of Element | Number of Connectors | C16 | C24 | C30 | C16 | C24 | C30 | C16 | C24 | C30 |
| 1 | 1 | × | × | × | × | × | × | 10,56 | 11,42 | 11,90 |
| | 2 | 1,50 | 1,61 | 1,69 | 1,60 | 1,70 | 1,77 | × | × | × |
| | 4 (1,2,3,4) | 3,00 | 3,22 | 3,39 | 3,19 | 3,39 | 3,53 | × | × | × |
| | 4 (1,2,5,6) | 2,70 | 2,90 | 3,05 | 2,88 | 3,06 | 3,19 | × | × | × |
| | 6 | 4,50 | 4,83 | 5,08 | 4,79 | 5,09 | 5,30 | × | × | × |
| 2 | 1 | × | × | × | × | × | × | 7,04 | 7,95 | 8,63 |
| | 2 | 1,50 | 1,61 | 1,69 | 1,60 | 1,70 | 1,77 | × | × | × |
| | 4 | 3,00 | 3,22 | 3,39 | 3,19 | 3,39 | 3,53 | × | × | × |
| | 6 | 4,50 | 4,83 | 5,08 | 4,79 | 5,09 | 5,30 | × | × | × |

*Table 5.7: Comparison of spacings according to EC5 and ČSN 73 1701*

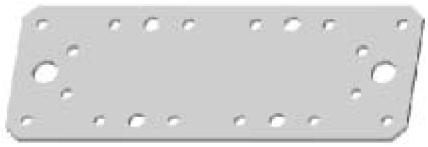
| **Spacing and Distance [mm]** | | | | Nails | | | Bolts | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | EC5 | ČSN | Satisfactory | EC5 | ČSN | Satisfactory |
| Number of Element | 1 | Spacings [mm] | $a_1$ | 28 | 60 | ✓ | 50 | 60 | ✓ |
| | | | $a_2$ | 14 | 16 | ✓ | 40 | 30 | ✗ |
| | | End and Edge Distances [mm] | $a_{3,t}$ | 60 | 60 | ✓ | 80 | 60 | ✗ |
| | | | $a_{4,t}$ | 20 | 16 | ✗ | 30 | 25 | ✗ |
| | | | $a_{4,c}$ | 20 | 16 | ✗ | 30 | 25 | ✗ |
| | 2 | Spacings [mm] | $a_1$ | 20 | 60 | ✓ | 40 | 60 | ✓ |
| | | | $a_2$ | 20 | 12 | ✗ | 40 | 30 | ✗ |
| | | End and Edge Distances [mm] | $a_{4,t}$ | 28 | 28 | ✓ | 40 | 25 | ✗ |
| | | | $a_{4,c}$ | 20 | 16 | ✗ | 30 | 25 | ✗ |

NOTES:

- the connecting plate always has to be placed on the both sides of the connected elements
- only the holes marked by a cross and a circle can be used (see Figure 5.17)
- in this case, all the fasteners are subjected only to shear force
- the numbers in parenthesis, used in Table 5.6, mean the number of hole used as marked in Figure 5.18 (e.g. 4 (1,2,3,4) means that four nails were used and placed into holes 1, 2, 3 and 4)
- the reason why the load-bearing capacities of 4 nails placed either into holes 1, 2, 3, 4 or 1, 2, 5, 6 (see the third and the fourth row in Table 5.6) differ is that there is a "row of fasteners" in the second case, and it means it has to be multiplied by $n_{ef}$ (for more information see chapter 4.2)
- the angle of force acting on the element number 2 (see Figure 5.18) is 90°, it means that there is a danger of a "brittle failure" (for more information see chapter 4.1.6) and the force in the second element therefore must not exceed the $F_{90,Rd}$ which is 7,01 kN for nailed connections and 9,56 kN for bolted connection

### 5.2.2.4 Connecting Plate BV/DS 03-32 – parallel elements

| | |
|---|---|
| diameter of nails: | 4 mm |
| length of nails: | 50 mm |
| diameter of bolts: | 10 mm |
| angle of force acting on the elements: | 0° |
| height of timber element: | 100 mm |
| width of timber element: | 100 mm |



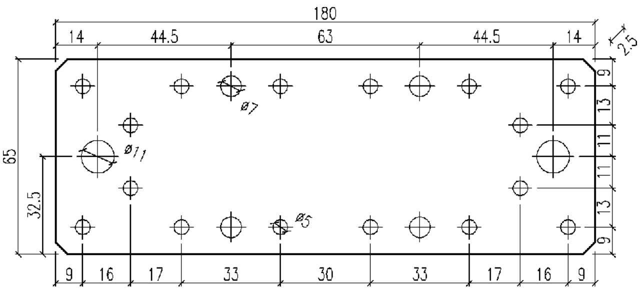



*Fig. 5.19: Dimensions and layout of the connecting plate*

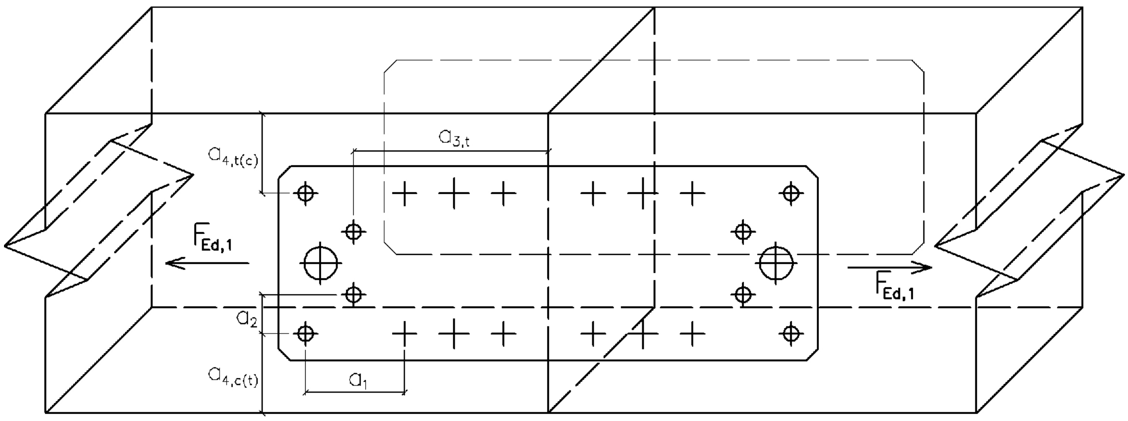



*Fig. 5.20: Model of the connecting plates and their connection*

*Table 5.8: Maximum design values of force $F_{Ed}$ which can be applied on the connecting plate BV/DS 03-32 according to EC5*

| $F_{Ed,1,max}$ [kN] | | Smooth Nails | | | Annular Ring Nails | | | Bolts | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | C16 | C24 | C30 | C16 | C24 | C30 | C16 | C24 | C30 |
| Number of Connectors | 1 | × | × | × | × | × | × | 10,56 | 11,42 | 11,90 |
| | 2 | 1,50 | 1,61 | 1,69 | 1,60 | 1,70 | 1,77 | × | × | × |
| | 4 | 3,00 | 3,22 | 3,39 | 3,19 | 3,39 | 3,53 | × | × | × |

*Table 5.9: Comparison of spacings according to EC5 and ČSN 73 1701*

| **Spacing and Distance [mm]** | | Nails | | | Bolts | | |
|---|---|---|---|---|---|---|---|
| | | EC5 | ČSN | Satisfactory | EC5 | ČSN | Satisfactory |
| Spacings [mm] | $a_1$ | 28 | 60 | ✓ | 50 | 60 | ✓ |
| | $a_2$ | 14 | 16 | ✓ | 40 | 30 | × |
| End and Edge Distances [mm] | $a_{3,t}$ | 60 | 60 | ✓ | 80 | 60 | × |
| | $a_{4,t}$ | 20 | 16 | × | 30 | 25 | × |
| | $a_{4,c}$ | 20 | 16 | × | 30 | 25 | × |

NOTES:
- the connecting plate always has to be placed on the both sides of the connected elements
- only the holes marked by a cross and a circle can be used (see Figure 5.20)
- in this case, all the fasteners are subjected only to shear force

# 6 Conclusion

Before writing this thesis I thought that a design of a timber joint with metal work connectors by means of the dowel type fasteners is relatively easy. Just to take appropriate Eurocode, use several formulas which are moreover mostly empirical and that is all.

To my surprise, it was not true at all. Eurocode 5 has a quite complex structure with many formulas and many conditions for their use. The formulas are empirical rather exceptionally. Most of them are based upon sophisticated theories, like Hankinson's equation or Johansen's (or European-based) yield theory, using fracture mechanics, virtual displacement method, etc.

The part dealing with the evaluation methods is very beneficial, because it was created in such a way that each sub-sections of the design of the steel-to-timber connection is always dedicated to all the types of the fasteners used in this thesis. Therefore, it is possible to see immediately what differences are among the dowel type fasteners in individual steps of the design. On the contrary, there is always the whole design procedure dedicated to the individual type of the fastener in Eurocode. This part was also amended by explanations of formulas and procedures in order to make it more enriching, interesting and readable.

Since the numerical model of the metal work connector does not include timber which is very important factor, and with no doubts influences the results, two extreme scenarios were considered. Firstly timber being totally rigid and secondly totally compliant. As expected, both scenarios gave different values. It is obvious that as the true behaviour of timber is somewhere in between these two scenarios, the true strength of the connector is somewhere in between as well. Nevertheless, the exact numbers are not important in this case, because the important finding is that the load-bearing capacity of the most critical hole in the connector is at least two times higher than shear resistance of any commonly produced nail. Therefore, the assumption that the strength of the metal work connectors will not be decisive was undoubtedly correct.

An application of the design procedures was done in a form of a comparison with a catalogue of the Bova-nail company, which uses ČSN. Interesting is that in some cases the engineers of the Bova-nail company do not take into consideration a spatial eccentricity of metal work connectors. This eccentricity introduces an axial loading of nails and it can be a crucial factor in a design of a joint especially when using smooth nails.

A check of spacings, and edge and end distances has been done. The result is that ČSN is in some cases stricter than EC5 but not always, there are also cases when it is vice versa. The differences in the spacings could be unpleasant for the Bova-nail company because it means that they would have to either redesign their connectors or change the rules of their usage.

It is very difficult, nearly impossible, to compare shear and axial withdrawal resistances evaluated by means of EC5 and the ones used by the Bova-nail company, because the Bova (respectively ČSN) does not distinguish the timber grades, whereas EC5 does. Anyway, the result is that resistances given by Eurocode, even for the very low timber grade (timber grade C16) are slightly higher than those given by ČSN. It is clear that the higher is the timber grade, the higher is the difference.

There are also issues I would like to deal with in the future. I would like to make more numerical models analyzing behaviour of different metal work connectors when loaded by different types of fasteners and afterwards, with complete knowledge of structural behaviour, make a new design which would be as effective as possible. Another task I would to do is to make an analysis and a comparison of parts of EC5 with appropriate Czech Technical Standards; this would be very valuable information for Czech companies which have not converted their products into EC5 yet.

# Notation

| | |
|---|---|
| $a_1$ | spacing, parallel to grain, of fasteners within one row |
| $a_2$ | spacing, perpendicular to grain, between rows of fasteners |
| $a_{3,c}$ | distance between fastener and unloaded end |
| $a_{3,t}$ | distance between fastener and loaded end |
| $a_{4,c}$ | distance between fastener and unloaded edge |
| $a_{4,t}$ | distance between fastener and loaded edge |
| $b$ | member thickness |
| $d$ | diameter of fastener |
| $F_{90,Rd}$ | design splitting resistance |
| $F_{90,Rk}$ | characteristic splitting resistance |
| $F_{ax,Ed}$ | design axial force |
| $F_{ax,Rd}$ | design withdrawal resistance of fastener |
| $f_{ax,Rk}$ | characteristic pointside withdrawal strength |
| $F_{ax,Rk}$ | characteristic withdrawal resistance of fastener |
| $f_{c,90,k}$ | characteristic compressive strength perpendicular to grain |
| $f_{h,0,k}$ | characteristic embedment strength parallel to grain |
| $f_{h,90,k}$ | characteristic embedment strength perpendicular to grain |
| $f_{h,i,k}$ | characteristic embedment strength of $i^{th}$ connected member |
| $f_{h,\alpha,k}$ | characteristic embedment strength at angle $\alpha$ to grain |
| $f_{head,k}$ | characteristic headside pull-through strength |
| $F_M$ | additional axial force caused by a bending moment |
| $F_{Rd}$ | design load-bearing capacity of connection |
| $f_u$ | tensile strength of nail wire |
| $f_{u,k}$ | characteristic tensile strength of bolt |
| $F_{v,Ed}$ | design shear force per shear plane of fastener |
| $F_{v,ef,Rd}$ | effective design lateral load-bearing capacity per shear plane of one row of fasteners |
| $F_{v,ef,Rk}$ | effective characteristic lateral load-bearing capacity per shear plane of one row of fasteners |
| $F_{v,Ek}$ | characteristic shear force per shear plane of fastener |
| $F_{v,Rd}$ | design load-bearing capacity per shear plane per fastener |
| $F_{v,Rk}$ | characteristic load-bearing capacity per shear plane per fastener |
| $F_{y,Rk}$ | Johansen's yield load |
| $h$ | member height |
| $h_e$ | distance from most distant connector |
| $k_{ef}$ | exponent dependent on nail spacing and whether or not pre-drilling is used |
| $k_{mod}$ | modification factor for duration of load and moisture content |
| $M_{y,Rk}$ | characteristic fastener yield moment |
| $n$ | number of fasteners per shear plane in row parallel to grain |
| $N_d$ | tension force due to the withdrawal effect |
| $n_{ef}$ | effective number of fasteners per shear plane in row parallel to grain |
| $N_i$ | axial force caused by bending moment in $i^{th}$ fastener |
| $n_p$ | number of overlapping nails |
| $n_{sp}$ | number of shear planes in connection |
| $r$ | number of rows of fasteners |
| $R_d$ | design value of load-bearing capacity |

| | |
|---|---|
| $r_i$ | lever of the axial force of $i^{th}$ fastener |
| $R_k$ | characteristic value of load-bearing capacity |
| $r_{pl}$ | number of rows of fasteners laterally loaded parallel to grain |
| $r_{pr}$ | number of rows of fasteners laterally loaded perpendicular to grain |
| $t_i$ | thickness of $i^{th}$ connected member |
| $t_{pen}$ | pointside penetration length |
| $w$ | modification factor |
| $X_d$ | design value of material or product property |
| $X_k$ | characteristic value of material or product property |
| $\alpha$ | angle of load in fastener relative to grain |
| $\gamma_M$ | partial factor for material property |
| $\eta$ | mean value of conversion factor |
| $\theta$ | angle of rotation |
| $\mu$ | friction coefficient between plate and the timber |
| $\rho_k$ | characteristic density of timber |

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