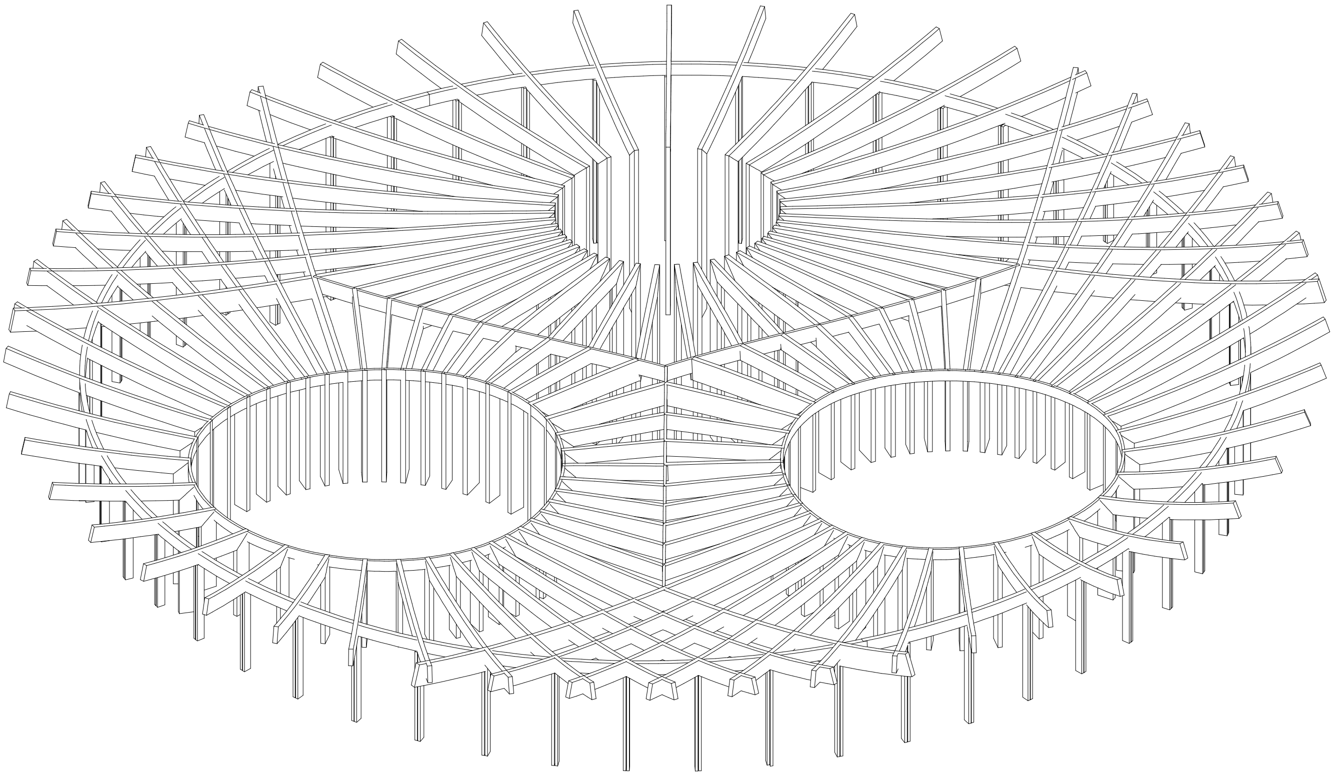

Connections in Large Timber Beams in Free-form Structures

Master of Science Thesis in the Master's Programme Structural Engineering and Building Technology

AMER BITAR

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Department of Architecture and Civil Engineering
Division of Structural Engineering
Group of light-weight structures
CHALMERS UNIVERSITY OF TECHNOLOGY

Master's Thesis ACEX30
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Cover:
Overview of the free-form roof timber structure analysed in the case study.

Department of Architecture and Civil Engineering
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ABSTRACT

Wood has long been used as a construction material, but the properties of solid wood in relation to strength, maximum length, and natural defects have always been a limiting factor in the construction of more ambitious projects. The development of knowledge in the field of concrete and steel has enabled the construction industry to shape modern society, but it has also brought with it an eminent and gradually increasing abuse of the environment. The rescue of construction using wood represents a conscious step towards sustainable development. Progress in the manufacture of wooden laminated panels and the production of synthetic glues has given greater scope for advances in the fields of engineered wood products. More and more research has focused on improving manufacturing techniques, making glued laminated wood a standardized product available in various shapes and sizes. This product has made possible the execution of works that previously only concrete or steel was capable of, such as bridges, high buildings, and large roof structures.

The computer revolution that took place in the 1990s had an immense influence on the construction industry, constructions that were previously mostly constituted by planar elements began to have freely curved building shapes. From this moment on, structural designers and architects have been able to explore fascinating architectural shapes, including the free-form structures. In timber free-form structures, connections are necessary to join single timber elements, which are necessary due to limitations in transportation and production. When using large beams these connections may need to transfer considerable high forces and moments to ensure an efficient structure. Besides the resistance also the stiffness is of high importance for a satisfactory structural behavior and to correspond to the assumptions made in the global model. The aim of this master's thesis is to gain a deeper understanding of timber connections in large timber beams and point out the connection's influence on the global behavior of the structure. The study has been applied to a free-form timber roof structure in development at the Danish consultant company COWI. The work presents different design concepts of connections and how these concepts could be optimized.

Keywords: sustainable development, timber free-form structures, large timber beams, timber connections, COWI.

Contents

Preface

After six months of working on this master thesis, we have reached now the final result. It is time to thank everyone that helped us to reach this point, starting from our teachers at Chalmers University and especially our examiner Robert Jockwer and our supervisors Oscar Pagrotsky and Andrea Alexandersson. Also, to our beloved family and friends.

We take here a personal space to allow us to write in our languages to show appreciation to our family members.

To my dear parents, thanks are not enough for every moment of love and care and support despite the distance and longing between us. To my brother, who is my family far from my home, thank you very much. To my homeland friends, or rather, my brothers and sisters, thanks for your support and being with me.

إلى والدي العزيزين ، الشكر ليس كافيا لكل الحب والدعم على الرغم من المسافة والشوق . إنها خطوة على أمل لقاء قريب. إلى أخي ، الذي هو عائلتي في الغربة ، شكرا جزيلا. لأصدقاء العمر، أو بالأحرى إخوتي وأخواتي ، شكرا لمحبتكم و تشجيعكم حتى لو كانت الظروف تفرقنا.

Amer / عامر

Minha querida família, escrevo aqui as minhas últimas linhas destes 2 anos de mestrado na Suécia. Me faltam palavras para descrever meus sentimentos neste momento, as janelas abertas, sem blusa e sentindo o calor vindo de fora. Escuto “Apesar de você” por coincidência. “Amanhã há de ser outro dia”. Me lembro dos momentos em que vocês me deram força e me disseram para sempre acreditarem no meu potencial. Obrigado pai, mãe e Nana, foi uma linda jornada começar minha jornada profissional. Foi mágico, aproveitei cada momento, sofri para conquistar, e aprendi em todos os segundos. Isso aqui é para vocês com todo meu amor do mundo.

Quero também agradecer a minha grande companheira que me deu um suporte do tamanho do mundo, me agraciou com seu charme e positividade. Um grande beijo com muito carinho, isso aqui é para você Titirinka.

Pedro

1 Introduction

Wood as a raw material has been used as a building material since the early stages of humanity. In recent decades deeper research has been carried out aiming to be able to explore the possibilities of the usage of timber in more ambitious design projects that involves more challenges to the structure arising from height, length of spans, etc. This effort on the development of new techniques on timber engineering production has been based on the advantages of the material compared to concrete or steel. Timber has a strength to weight ratio higher than conventional building materials. This means that timber can resist a certain load as concrete and steel with less self-weight and in large structures, longer spans are achieved since the dead load is less. Furthermore, the increased concern about reducing the environmental impact caused by the construction business settles the importance of further developing techniques using timber as a structural material.

The advances in the manufacture of more robust laminated timber members have allowed building long spans and higher structures. Due to limitations on transportation and production of such long members, engineers have been facing the challenge to propose and design appropriate connections that withstand very demanding stresses. The usage of deep beams in large structures demands that the connections must transfer considerable high forces and moments in order to ensure structural efficiency. Especially in free - form structures, connections that resist a multitude of different combinations of forces and moments are beneficial in order not to design every single connection individually. Besides the resistance, the connection stiffness is also of high importance to control the deformation and the stability of a structure.

1.1 Aims and objectives

The main aim of this project is to provide an effective study about connections in timber structures and especially in large free-form structures. The objective is to develop a study on timber beam-to-column, beam-to-beam connections based on a free-form roof timber structure which is in the design process at COWI. The work gives focus on a certain number of connections considered to be the most critical ones in terms of stresses, complexity and number of intersections. These connections are tested in order to obtain the most optimized design solution. The criteria are:

- Type of connection,
- adequate stiffness and strength,
- dimensions,
- geometry,
- aesthetics,
- effect of the stiffness of a joint on a global level.

1.2 Method

Seeking to achieve the aims of this master's thesis the following procedures will be adopted:

1) Review of the state of the art of existing large structures and free-form structures built in practice and the existing connections in timber structures.

2) Literature study of relevant standards, guidelines, and previous knowledge on the design and structural behaviour of connections locally and globally. Present a study of different types of connections and the relevant failure modes. In addition, present a review of the importance of the connection's stiffness.

3) Analytical calculations for different types of connections under bending moment, shear, and tensile forces. This step supports the evaluation of the connection types based on strength, stiffness, and advantages and disadvantages of each type. In addition, present recommendations, and conclusions to determine the design of connections in large timber structures.

4) The final step is to adopt the gained knowledge to understand and analyse the main object of this Master Thesis, designing certain connections in the free-form roof structure project from COWI. The connections are optimized in terms of resistance, stiffness, and ductility.

1.3 Case study

The information and knowledge presented in this master thesis will be applied to a project of the company COWI. Therefore, the case study will be based on the design proposal of some of the connections of the free-form roof timber structure presented in Figure 1.1.

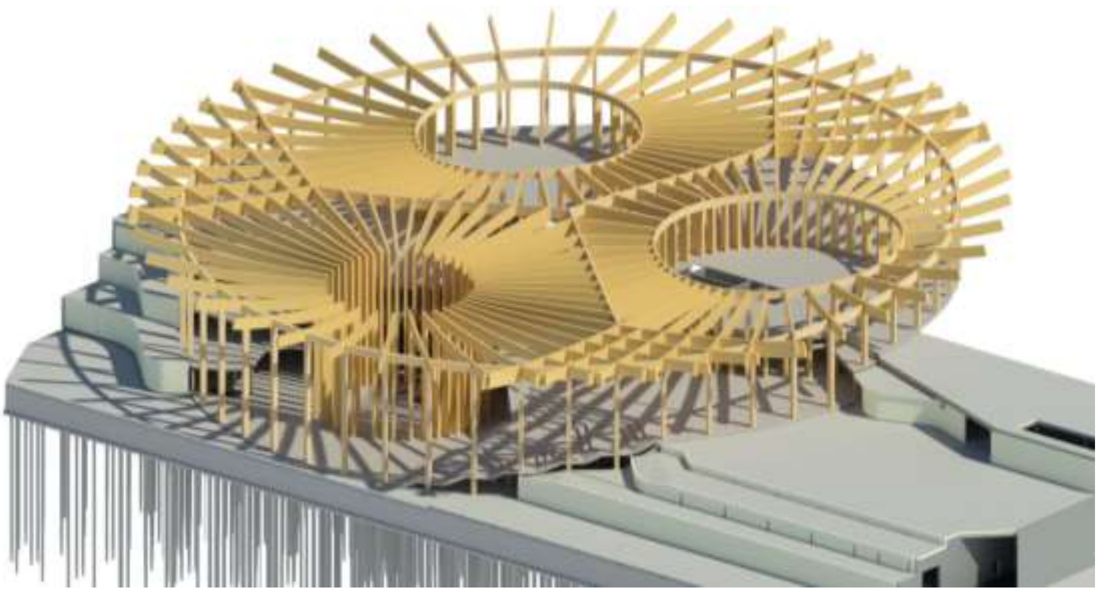


Figure 1.1 Free-form roof timber structure of the case study.

1.4 Limitations

The case study has limitations in which must be considered in the development of the proposals for the design of the connections.

The glulam timber sections and its geometry must comply with the design from COWI, therefore the study is adapted to the limitations in dimension and loads from the case study. The study was limited with three connections of the case study that were chosen for further investigation.

The evaluation focus of the timber connections is based mainly on the load-bearing capacity, stiffness, and controlling of deformations. Another criterion that is very important is that the design of the connections is realistic in a construction assembly situation, in terms of dimensions and productivity. Fire and weather risk and appearance are mentioned briefly.

The analytical calculations are based on EC5 (CEN, 2004) as a reference. Since some investigated connections are not covered by EC5, other books and papers are consulted. For glued-in rods connections, the book “Limträhandbok” (Svenskt trä, 2016) is used as the main reference combined with some articles provided by the supervisors. Multi-planes metal plates are also not explicitly detailed in EC5, therefore the book Limträhandbok and the book “Design of Connections in Timber Structures” (COST, 2018) are followed as the main reference.

The FEM software “FEM-Design” from StruSoft is the tool used to extract the forces and stresses in the studied project and to check how the designed connections affect the global behaviour of the structure (deformations and sectional forces).

2 Background

For a better understanding of the studied subject of this thesis, an overview of free-form timber structures and connections is given in this chapter. The fundamental structural concept that is the reference for various free-form design concepts is presented and which forces are generated in its structural members. The second part gives a comprehensive view about types of connections in timber structures and gives an introduction about the theoretical approach to fasteners type connections and the design of timber joints.

2.1 Free-form timber structures

2.1.1 Introduction

The development in the timber industry has led to enhancement in mechanical properties and the strength of manufactured members. Timber is an anisotropic material and its strength and mechanical properties vary in different planes, being the parallel to the grains plane the strongest one. The carried-out research and the development in timber products manufacturing have been focusing on increasing the general strength and decrease the existing gap between different directions properties. Glued laminated timber also called *glulam*, is an engineered wooden product that is a result of this progressive development. Today it represents a widely used solution of timber on the erection of large-scale buildings and constructions. It is produced of lamellas with a maximum thickness of approximately 50 mm which are connected by means of finger joints and gluing, Figure 2.1 shows an example of a *glulam* member. These continuous sections are cut into laminations of the required length (Blaauwendraad & Hoefakker, 2013).

Figure 2.1 Glulam timber cross-section, source: www.johnsbuildingsupplies.com.au.

Further development of glulam was to produce curved beams in one direction and later in two directions as shown in *Figure 2.2*. The doubled curved beams are bent in one main direction and twisted or bent in the other direction. With this vast possibility of shapes and sizes, free-form timber structures started to be feasible to erect. This kind of structure has attracted a lot of focus on structural design, due to their complexity in shapes and assembly. The production requirement for curved glulam is referred to EN 14080 (Jacob et al., 2018).

Figure 2.2 Bent and twist glulam beam, source www.designtoproduction.ch.

A further design step is the computational and modelling ability which has given the power to provide the needed tools to support the designers to move from common typical framing-type constructions to complex-shaped structures. (Blaauwendraad & Hoefakker, 2013).

2.1.2 State of art of free-form timber structures

Free-form shape structures are structures that cannot be defined by a regular or uniform shape and there is no mathematical clear expression that represents the structure's shape. A way to understand and define the free shaped form is by defining it as a double-curved structure in two directions. Each curve is measured by its curvature which is the inverse of the curve radius $K = \frac{1}{R}$ at every point. Principle curvatures of a surface are defined by points with maximum or minimum curvature. According to Carl Friedrich Gauss, the Gaussian curvature is determined by the product of the minimum and the maximum curvature for a certain surface:

$$K = K_1 \times K_2 = \frac{1}{R_1} \times \frac{1}{R_2} \tag{1}$$

The direction of the curvature can be captured by the sign of the Gaussian curvature as shown in Figure 2.3 (Blaauwendraad & Hoefakker, 2014).

K>0, The two curvatures pointing in the same direction (synclastic surface).
K<0, The two curvatures pointing in a different direction (anticlastic surface).
K=0, One or both curvatures are flat or zero (monoclastic surface or eroclastic surface).

Figure 2.3 a) K>0, b) K<0, c) K=0.

From structural aspects, the positive sign of the Gaussian curvature means that all the points in the structure are subjected to pure compression or tension. While the negative sign means points in the surface are subjected to compression or tension.

The structural behaviour of free-form timber structures is related to the magnitude of the curvature which is varied in different points and areas. On areas with high curvature, the structure acts as a membrane, in other words, in-plane stresses of pure compression or pure tension are generated. While the flatter part with low curvature has a plate's behaviour, and it is subjected to bending out of a plane that generates moments and shear forces. In the transition areas between the two parts, both behaviours are combined as in shell structures. The complex nature of these structures demands numerical modelling that can be performed by finite element software.

2.1.3 Grid shells

Grid shells are based on the geometry of shell structures. The difference is that grid shells are structures made of double-curved beam grids instead of a solid surface. A timber free-form structure is a grid shell of timber elements that work as beams and columns in the direction of the grains, which have the best mechanical properties of timber. *Figure 2.4* shows an example of grid shells.

Figure 2.4 Haesley Nine Bridges, South Korea, view from the interior and top view, source www.shigerubanarchitects.com.

2.1.4 Type of connections in free-form timber structures

Different kinds of stresses in members can arise based on the geometry, the spans, and boundary conditions of such a structure. In order to be able to build a stable structure that sustains equilibrium when subjected to loads, each grid shell member must be adequately connected. The conditions of each individual joint may vary a lot, which makes the design very complex, in general, several or even hundreds of types of connections can be found in these structures. Briefly, connections on free-form timber structures transfer in-plane tension or compression, bending moments, shear forces, and torsion. In some cases, these can act simultaneously.

A connection becomes complex when it joins several horizontal members in different directions, especially when it needs to transfer moments. Also, in horizontal members connected to vertical supporting members, high bending moments and shear forces must be transferred as bending and axial forces.

2.2 Connections in timber structures

Connections in general are categorized according to the forces they resist. An overview of the main type of connections according to shear-resisting connection, moment-resisting connection and combined connection are presented.

2.2.1 Shear resistant connections

Shear resistant connections transfer shear forces between the connected members. Although some bending stresses can also be transferred, this contribution is very small, therefore it is neglected. This kind of connection corresponds to a boundary condition that prevents translation in the direction of the forces but still allowing rotation. Generally, it is called a simple connection or a hinged connection.

Connections that rely on transferring shear forces have been used broadly in the past due to the timber's good resistance to shear. The advantage of these connections is the simplicity of being built on-site. There are several applications to these connections, on timber building construction, as in beam-to-beam connections or column-to beam-connections. Figure 2.5 shows shear connections which are used in a non-moment resistant frame.

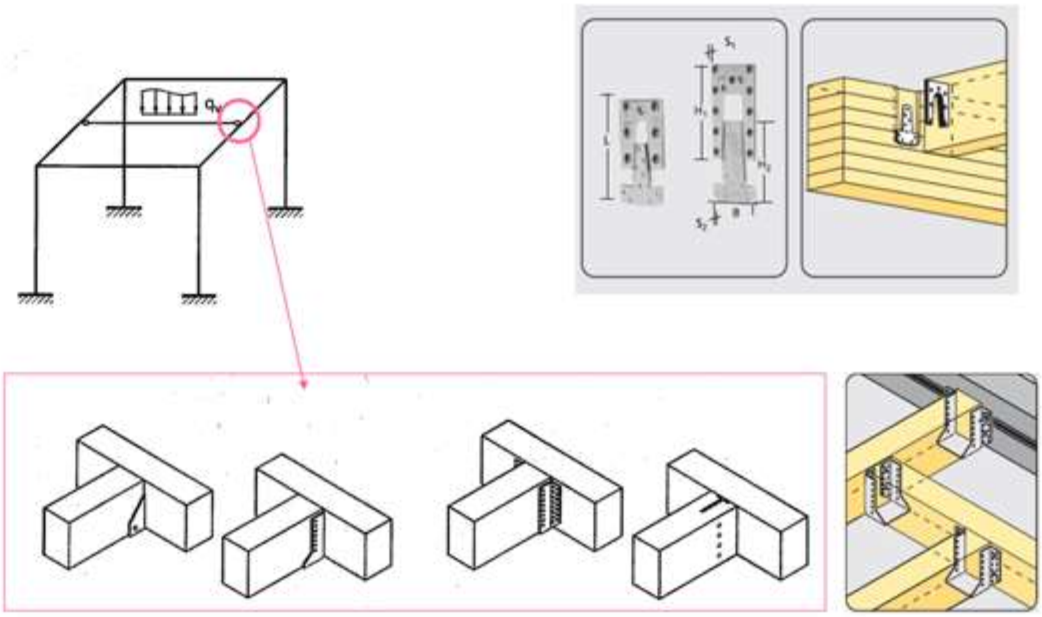


Figure 2.5 Shear-resisting connections in timber structure (VSM 196 – MPSEB).

In a beam to beam connection, when a secondary beam stands on a primary beam, transversal loads will be transferred as showed in Figure 2.6. Also, in structures with large spans where it is necessary to connect a beam on its length, the usage of splice joints is considered a practical solution, i.e. Gerber hinges. Gerber hinges allow extending a horizontal member to overcome the span length in regions that have almost zero bending moment and high shear forces values, Gerber joints are demonstrated in Figure 2.7.

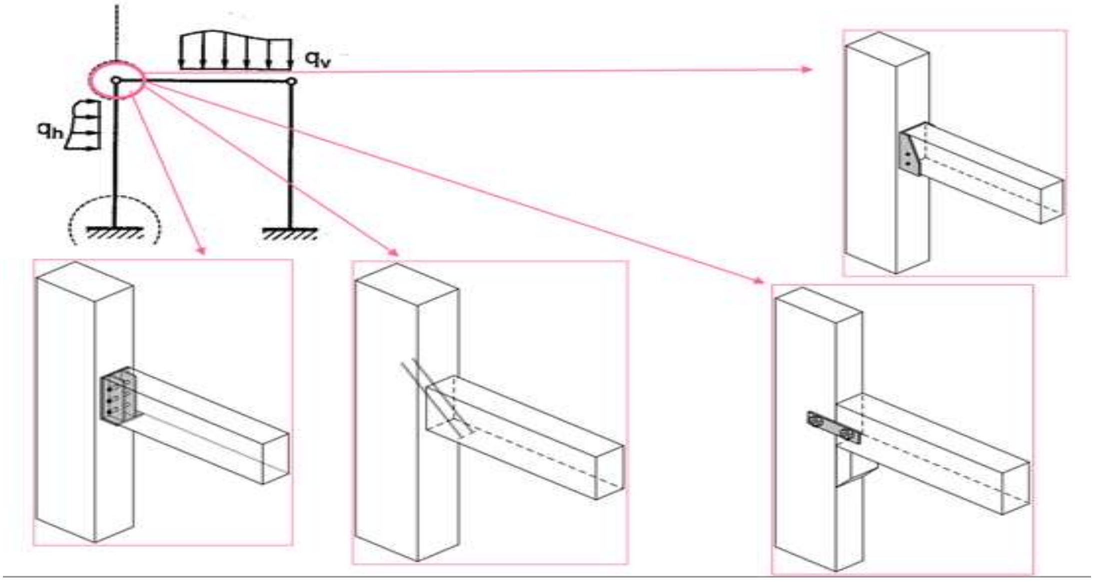


Figure 2.6 Shear-resisting connections in a non-moment resisting frame. (VSM 196 – MPSEB).

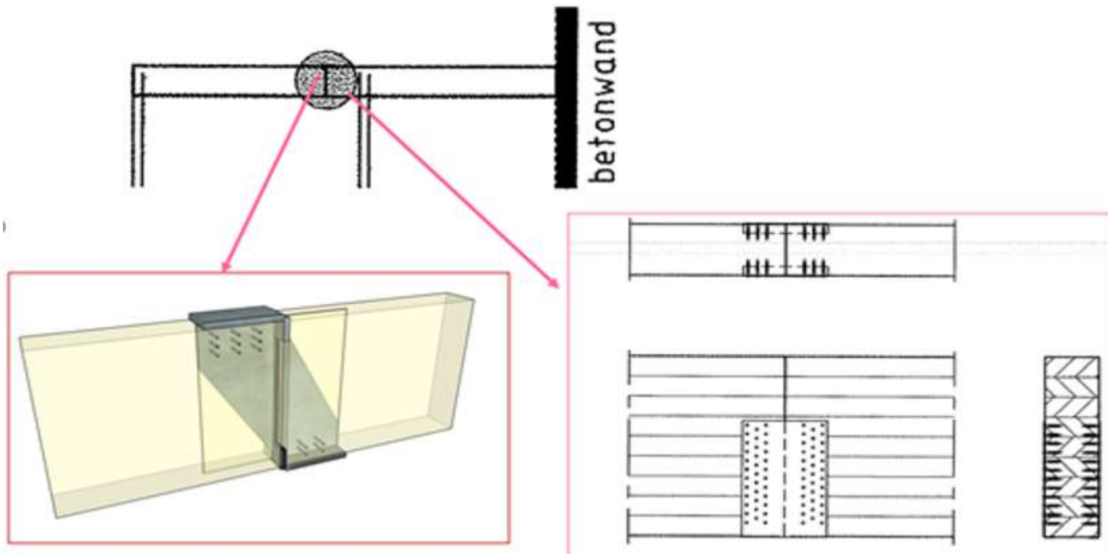


Figure 2.7 Gerber hinge (VSM 196 – MPSEB).

Shear connections can be executed like a traditional carpentry joint, glued joints, and mechanical connections which are the most common in structural timber design, i.e. single-plate shear connections. Some more types of shear resistance connections are later investigated in Chapter 5.

Hinged connections, however, have the disadvantage of originating longitudinal shear forces parallel to the grain and tension forces perpendicular to the grain These forces cause a brittle failure which should be avoided. Chapter 5 will give a deeper explanation of this issue.

2.2.2 Moment resisting connections

In some structural cases, a member must be restrained against rotation in relation to other elements, for these purposes moment resisting connections are necessary. These connections are fundamental where no in-plane bracing mechanisms are utilized. In order to restraint a frame and provide enough lateral stability, the connections between the elements must be moment resistant. Portal frames are broadly designed for buildings and it is a structural solution that provides wide clearance and area for the intended usage and enough slope for roofs. It has the advantage of standardizing the structural design, such consistency eases the construction and it is beneficial for the result in terms of overall quality. It makes construction faster and reduces costs (TRADA, 2016).

In engineering terms, moment-resisting connections are boundary conditions that are treated as a fixed connection, where rotation and vertical and horizontal translations are restraint. Realistically, due to the soft characteristics of timber, it is not suitable to assume full rigidity as on concrete or steel. Therefore, semi-rigid connections are assumed when designing (Aicher et al., 2012).

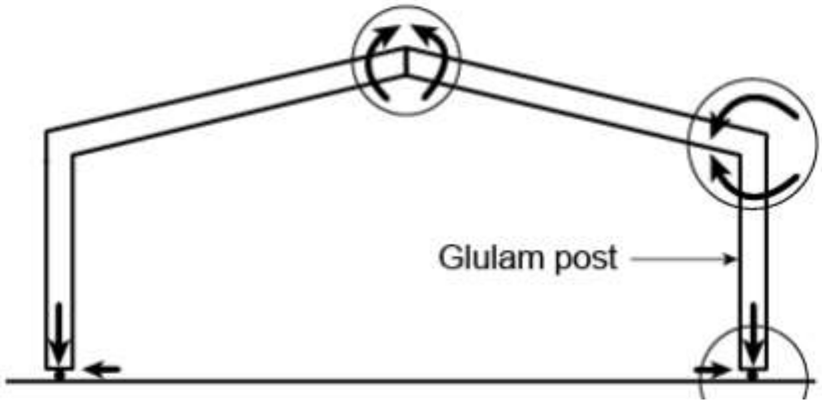


Figure 2.8 2-pin portal frame (TRADA, 2016).

Designers have alternatives when planning frame structures. Figure 2.8 shows a typical 2-pin portal frame, this is the most common form of timber portal frame, which is stable by relying on moment-resisting connections on its knee and apex. Figure 2.9 demonstrates some alternatives to knee connections.

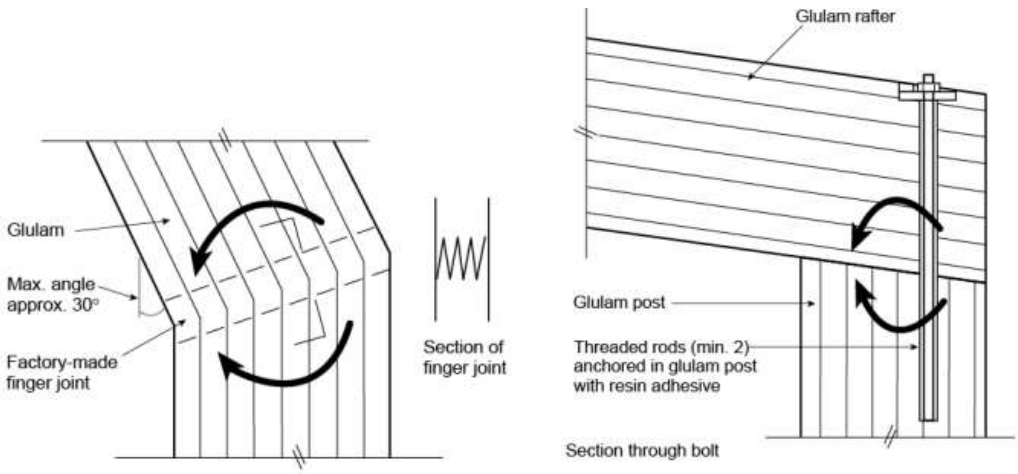


Figure 2.9 Knee connections (TRADA, 2016).

Figure 2.10 and Figure 2.11demonstrate some alternatives to apex connections.

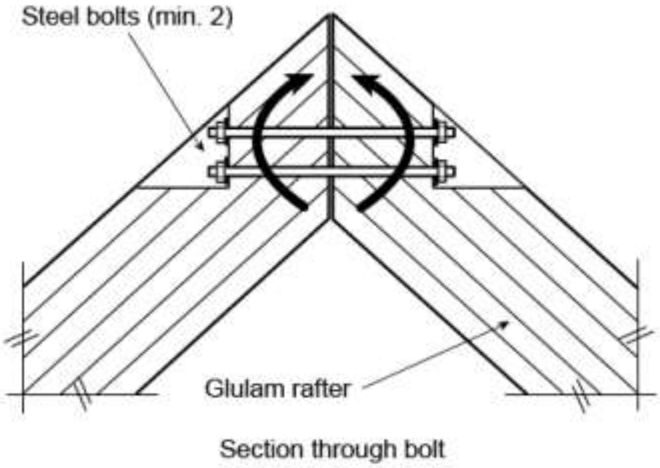


Figure 2.10 Moment resisting apex connection with bolts (TRADA, 2016).

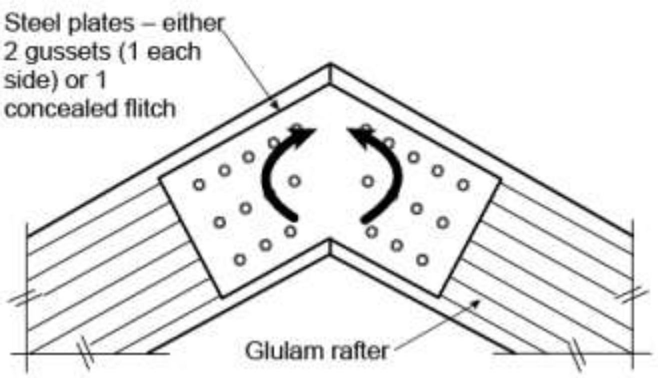


Figure 2.11 Moment resisting apex connection with steel plates (TRADA, 2016).

Sometimes in order to facilitate the construction, the apex can also be designed to be a pinned connection; this configuration is called a 3-pin portal frame and is shown in Figure 2.12. In this case, the knee connection must withstand more stresses and therefore members must have larger dimensions.

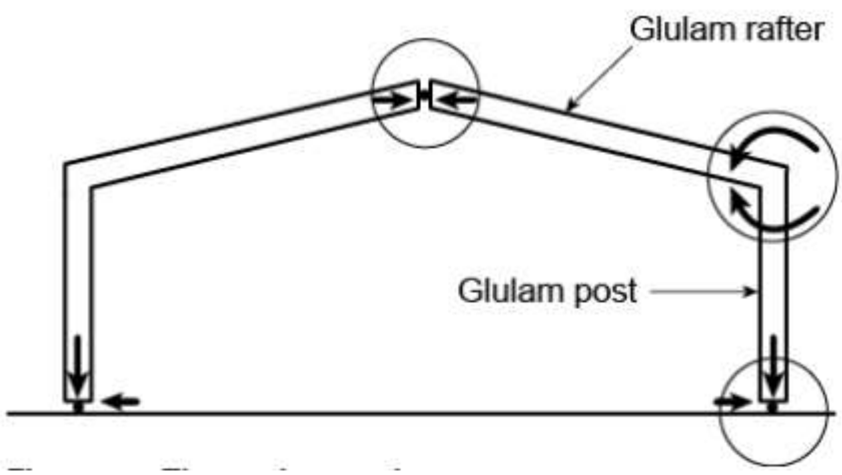


Figure 2.12 3-pin portal frame (TRADA, 2016).

When members are getting too large dimension the base column must be fixed to the foundation, Figure 2.13 shows an example of this kind of connection.

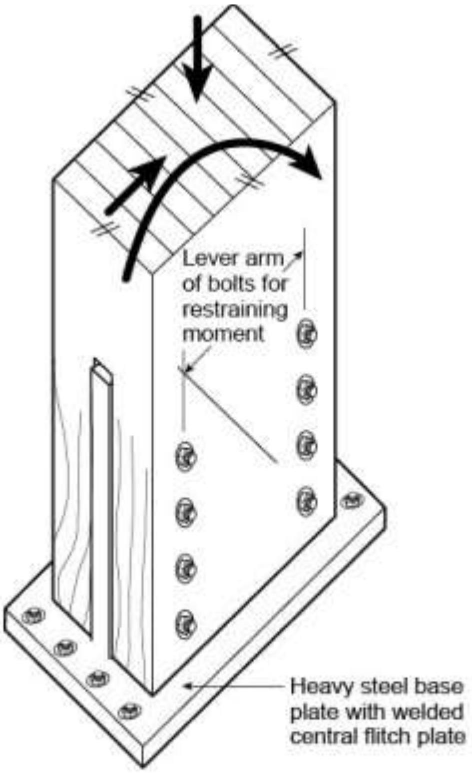


Figure 2.13 Fixed column (TRADA, 2016).

2.2.2.1 A simple calculation of the force distribution from an applied moment

When a joint is designed on a moment with or without shear, the dowels' contribution in resisting differs according to the dowel's location from the centre of gravity. The dowel would be subjected to different forces in different angles to the grains (Lidlöw, 2015).

Fundamentally, the dowel will still resist the moments by their shear capacity but in a different direction. Although it should be checked how this force also acts on the timber and in which direction since it could generate tension or compression perpendicular to the grain.

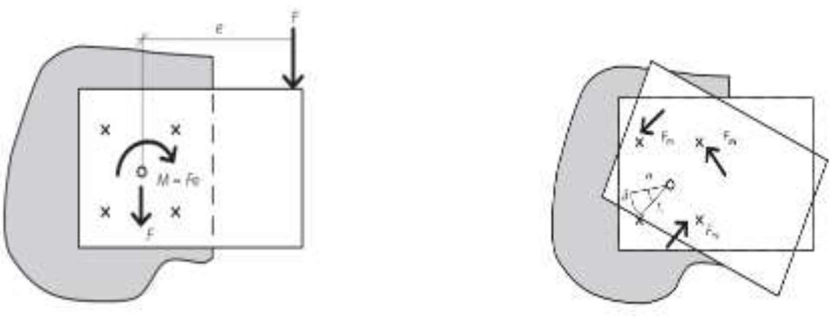


Figure 2.14 Elastic moment distribution (Borgström, 2016).

- Elastic load distribution.

  The force acting on every dowel is calculated as follow:

  1. Determining the centre of gravity.
  2. Divide the shear forces acting on the centre of gravity to the fastener.

     $F_y = nF_{y_i} \quad F_x = nF_{x_i}$

  3. Translate the moment from the centre of the gravity to forces acting on the fastener.
     From moment equilibrium: the external moment should equal the sum of the internal moment.

     $M = F.e = \Sigma F_{mi} \cdot r_i$

     $F_{m_i} = k_\delta \quad I_P = \sum r_i^2 = \sum(x_i^2 + y_i^2) \quad F_{m_i} = Kar_i = \frac{M \times r_i}{I_P}$

k - Slip modulus.
δ - Displacement due to the rotation in the joint.
α - Rotation angle.
I_p - the polar moment of inertia of the system.

  4. The total load on each fastener is calculated as follow and the single dowels should be checked on its capacity.

     $F_i = \sqrt{(F_{xi} + F_{mxi})^2 + \left(F_{yi} + F_{myi}\right)^2}$

- Plastic load distribution.

Dowelled joints are designed to show a plastic behaviour. The plasticizing starts from the outermost to the closet fasteners from the centre of gravity, it is important to ensure that sufficient plastic deformation can be achieved before brittle failure occurs (Borgström & Karlsson, 2016). In this case, a plastic distribution could be determined by the lower or upper boundary. More information is found in the book "Design of timber structure page 145" (Borgström & Karlsson, 2016).

2.2.3 Moment + shear connections

In most of the structural arrangements, joints are designed to resist bending moment in combination with normal and shear forces. Usually, several components are included in this type of joints and each component contributes to load carrying capacity, each of them must be checked to its related failure mode to achieve a reliable design.

Combining the mechanism from Section 2.2.2 and Section 2.2.1 is a way to design a connection resisting both bending moment and shear forces. An example could be a connection working by the contact interface between the timber parts, mechanical fasteners in different locations and glued-in rods, etc. These members could be used simultaneously to form this connection.

2.3 Theoretical approach for design timber connections

Some principles should be considered during the design of timber connections. These principles are explained in detail in Chapter 3.

1. All possible failure modes in timber connections should be checked:

As most timber connections contain metal parts (dowels or plates) it is preferred that a ductile failure in steel is the failure mode Since timber show brittle behaviour in tension perpendicular to the grain and shear parallel to the grain.

2. Timber connections shows semi-rigid behaviour:

This behaviour should be considered during the design and how that reflects in deformation control and designing the members in the ultimate-limit-state in the structural members.

3 Connection mechanisms

Timber’s anisotropic properties induce the risk of splitting of the connected members caused by high forces perpendicular to the grain. Often, the stress transfer mechanism involving the connection type is complex. Moreover, the reduced timber’s cross-section net area on the region of the connection can lead to other brittle failure modes. Still, there is a lack of knowledge in the design process of connections and on manufacturing and assembling. All these aspects in combination add complexity to the structural design, therefore connections are considered critical points on timber structures. (Frühwald & Thelandersson, 2008). A study carried out in 2011 on typical causes of structural failure on timber structures, revealed that 23% of structural failures were directly linked to inaccurate design specifications and that 57% of the cases were in dowel-type connections. (Frühwald, 2011). Dowel-type connections will be further discussed in Section 3.4.

The aim of Chapter 3 is to summarize the existing connection mechanisms and type of fasteners of which Eurocode 5 covers. Furthermore, other solutions that have been a topic of study over the years are discussed.

3.1 Eurocode 5

Connections are covered in chapter 8 of Eurocode 5. Approximately 20% of the content presented is dedicated to connections, representing a long part of the current version of EC5. (Stepinac et al., 2018). Since it was first published in 2004, revisions were done in 2006, 2008, and 2014, most of the sections have been reviewed by the European Committee for Standardization – Technical Committee 250, CEN TC 250 (Jacob et al., 2018).

The 8th chapter of EC5 gives guidance to the design of metal dowel-type fasteners based on Johansen's theory and sub-sections detail the design of laterally and axially loaded different types of fasteners, applied for timber-to-timber, panel-to-timber, and steel-to-timber connections. Additionally, design procedures for connections with connectors are also covered. (TRADA, 2017).

The connections types covered by Eurocode 5 (CEN, 2004) are the following. Glued joint is presented without a detailed guideline:

- Glued joints,
- Stapled connections,
- Nailed connections,
- Bolted connections,
- Dowelled connections,
- Screwed connections,
- Punched metal plate connectors,
- Split ring and shear plate connectors,
- Toothed-plate connectors.

3.1.1 Questionnaire about the connection chapter

Although the connections chapter covers a substantial portion of the current version of Eurocode 5, only the most common connection types are presented in detail. (Stepinac et al., 2018).

Taking into account the shortcomings of the current version of the code, the European Cooperation in Science & Technology (COST) conducted a collective study to coordinate available scientific studies in other to standardize information that can be applied in practical design situations, the study has been named "Action FP1402". The main objective is to provide the basis for the elaboration of a new version of the code, by improving current design methods and deriving new design methods of developed wooden products that have been proven in matters of reliability and performance. (COST, 2018). Working Group 3 (WG3) of the project has been assigned the assessment of timber structural connections. Among other studies presented in the group's state-of-the-art report, a questionnaire was carried out in other to identify flaws that the current connections chapter of Eurocode 5 present and propose improvements for identified issues. The study was named "Results from a questionnaire for practitioners about the connections chapter of Eurocode 5". (Stepinac et al., 2018). The questionnaire was filled out by 412 engineering practitioners, manufacturers, and academia from 28 European countries and 5 non-European countries, the given feedback about the issues faced when working with the connections section of the code is given in Table 3.1.

Table 3.1 Main problems with the current version of Chapter 8 (N_{all}=410 , $N_{experts}$=184). Multiple responses were possible. (Stepinac et al., 2018).

Problem	Number of Responses - All	Number of Responses - Experts
Difficulties in navigating	223 (54%)	123 (67%)
Confusing statements	156 (38%)	89 (48%)
Lack of information	143 (35%)	64 (35%)
Poor presentation of technical content	134 (33%)	71 (39%)
Dependency on other standards	87 (21%)	46 (25%)
Lack of consistency	68 (17%)	41 (22%)
Other	43 (10%)	21 (11%)
No problem	34 (8%)	3 (1.6%)

The most agreed issue regarding missing information has been the lack of design guidance and detailing of glued-in rods, carpentry joints, reinforced connections, and self-tapping screws with large diameters and fasteners subjected to axial compression. Moreover, it was also noticed responses regarding lack of information on moment resistance connections and modern screws, new rules that improve the effective number of fasteners, combined effects of lateral and tensile loads, more brittle failure modes, and improved explanation on the achievement of ductility in connections (Stepinac et al., 2018).

Regarding troubles on technical interpretation, 55% considered spacing rules difficult to understand. Punched metal plate connectors and geometrical requirements for multiple shear connections were also included as confusing sections that need better clarification. The definition of loaded and unloaded edges on spacing requirements was mentioned by almost 50% as not clear to understand, same for the definition of rope effect and the calculation of double-shear fasteners capacities (Stepinac et al., 2018).

3.2 Research and proposals

The development of longer EWP has led to the need of providing strong connections that can transfer large bending moment, shear and torsional forces. This has become a great challenge for researchers and companies in the structural timber engineering field. The last published version of the Eurocode 5 is dated 2004. Since then, several solutions have been proposed by researchers, who dedicate their effort to the formulation of connector systems that have properties that promote suitable situations to cope with the modern demands used in more complex projects. Such elaborate structures consist of remarkably robust members and long spans, therefore the evident need for special type connections is fundamental. The principal aim of research is to provide sufficiently strong and stiff connections that present a ductile-type failure behaviour when reinforcing the member against forces perpendicular to the grain direction. Working groups from the European Committee for Standardization (CEN) are actively developing a "second generation" Eurocode 5, which is estimate to be completed in 2025 (Jacob et al., 2018).

These are some of the solutions proposed by academic researchers in recent years:

- Glued-in rods,
- Threaded rods,
- Self-tapping-screws.

3.3 Glued Joints

Glued joints are only briefly presented in Section of 10.3 EC5 (CEN, 2004). Gluing is used on a large scale in the production of glued laminated timber, plywood and composite members as for instance I-sections and stressed-skin-panels. In the case of long laminated members, the individual veneers are spliced with glue. The specific type of glue influences aspects related to mechanical properties, fire and moisture resistance, curing time, and workability (TRADA, 2007). Currently, in Sweden, Melamine-Urea-Formaldehyde is the prevailing type of glue adopted in the production of glulam (Borgström & Karlsson, 2016). Glued connections can reach very high stiffness, moreover, it presents a better fire resistance than steel fasteners. Since one element is glued to the other, the connection is hidden. The key disadvantages are that meticulous quality control is demanded, it is unsuitable in conditions of fluctuating moisture content and present a brittle failure-type behaviour. The failure occurs in the glue-line or close to it and does not exhibit plasticity, so it is instantaneous (TRADA, 2007). Figure 3.1 shows examples of different types of glued connections.

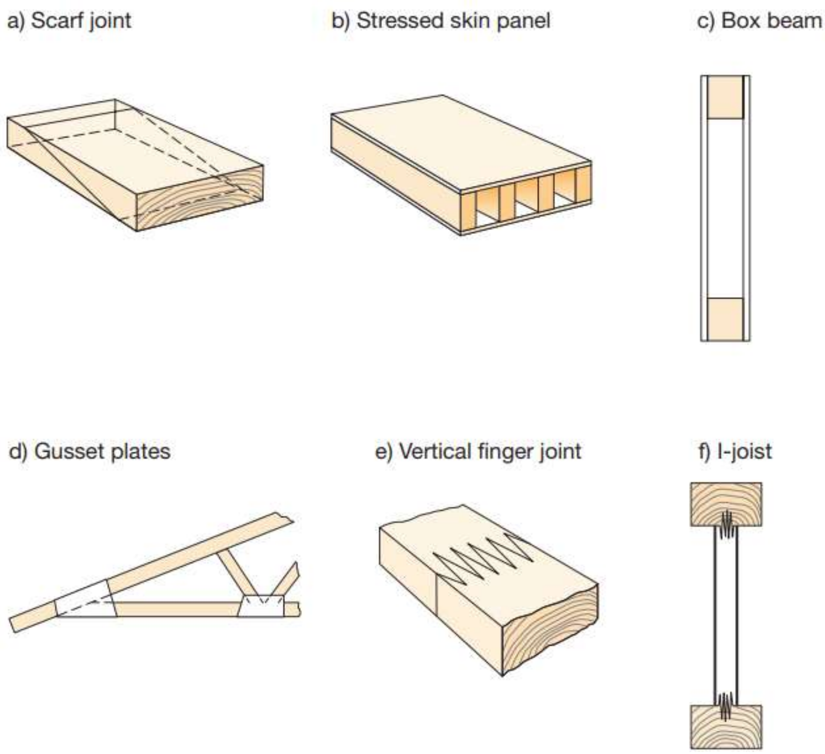


Figure 3.1 Examples of glued joints (TRADA, 2007).

3.4 Dowel-type connections

The use of mechanical fasteners enables members to be connected with superior efficiency because of the steel's resistance properties, defined by its ultimate strength, f_u and yield strength, f_y. Unlike timber, steel has a ductile failure behaviour when designed for this propose. The forces between two members are transferred through shear, the fasteners work as a bridge when mounted at an angle to the force direction. This type of connection is the most used in Sweden and the rest of the world. There are several sorts of fasteners available in the market, the most substantial factors that differentiate them are the diameters, type of steel, and recommended use (Borgström & Karlsson, 2016).

Connections made with nails, screws, metal dowels, staples, and bolts behave in a similar way and therefore fall into the category of dowel-type connections (Ozelton & Baird, 2002). In order to obtain CE Marking, which is obligatory throughout the EU, dowel-type fasteners manufacturers in Sweden must comply with the rules given by "SS-EN 14592: 2008 + A1: 2012 Timber structures - Dowel-type fasteners – Requirements" (SIS, 2012).

In principle, the ultimate limit state of the connection may be reached by a lack of resistance of the timber or of the connecting element. In the structural design of connections, the resistance of the timber element in tension, compression, and embedding strength is considered, and the resistance to shear and bending of the fastener also requires to be checked. The performance and safety requirements of fasteners shall be checked according to EC5 (CEN, 2004). Each of these mechanical properties will be adequately explained in Chapter 4. To calculate the strength of a joint one needs:

- Joint geometry;
- Fastener yield moment;
- Embedding strength (Borgström & Karlsson, 2016).

3.4.1 Staples

Staples are mainly applied in lightly loaded structures, on panel-to-solid timber connections and to provide better adhesion during curing of glued joints (Jacob et al., 2018). The application is made with pneumatic tools, the staples are manufactured in collated strips, as shown in Figure 3.2, between each staple leg resin is applied not only to hold the elements together but also to increase the withdrawal resistance of the staples (TRADA, 2012). The shoulders are areas with high concentrated stresses and when combined with stresses emerging from induced corrosion can lead to fracture, culminating the fastener to perform as two slender pins. Therefore it is not recommendable to use on environments with induced corrosion risk (Jacob et al., 2018).

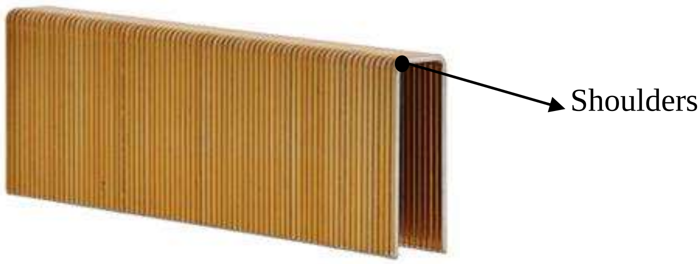


Figure 3.2 Strip of staples, source www.wuerth.de.

The production rules and specifications for the sizing of the elements are based on the nominal diameter of the wire. The crow's length must be at least 6d and the leg length must not exceed 65d. The cross-sectional area must be between 1.7 mm^2 and 3.2 mm^2 (TRADA, 2012).

3.4.2 Nails

Nails are metal cylindric connectors elements with an anchor head and are driven in timber by manual impact or portable machines with pneumatic control. Nails can be driven to timber with or without predrilling when inserted to the elements, nails driven with no predrilling will cut through the timber fibres, which weakens the element. Predrilled connections the fibres are separated resulting in a stiffer system, the downside is that the procedure is more time-consuming and more costly. It is recommended to pre-drill timber with characteristic density higher than 500 kg/m^3. Nails are manufactured from steel wire coil possessing a minimum tensile strength of 600 MPa (TRADA, 2007). It is available in an extensive variety of sizes lengths and diameters, having maximum diameter value as 8 mm and are categorized by two main types (CEN, 2004):

- Smooth nails.

The shank of these types of nail has no profiling and it is smooth, as seen in Figure 3.3. The cross-section geometry of the shank can be for instance round, squared, or grooved. The suitable selection of the shank geometry depends on the timber type and the required withdrawal capacity of the connection (Borgström & Karlsson, 2016). The withdraw capacity will be deeper explained in Section 4.3.1.

Figure 3.3 Smooth nail, source www.winzer.com.

- Other than smooth nails.

Shank profiling results in better withdrawal capacity, these types of nails have better anchorage capacity due to the shank's shape, see Figure 3.4 (Borgström & Karlsson, 2016). The profiling characteristics vary from type to type based on its usage.

Figure 3.4 Shank profiling nail, source www.winzer.com.

3.4.3 Screws

Screws are threaded cylinders with an anchor head and have more withdrawal capacity than nails due to the helicoidal shape of the threads that anchor the timber element, generating a locking system between the wood and the screw (Ozelton & Baird, 2002). Screws can be pre-drilled or not pre-drilled, this is dependent on the diameter of the member. According to Eurocode 5, for diameters less than 6 mm pre-drilling is not necessary, while for diameters above this value it is recommended. The main application of the type of connections is on steel-to-timber and panel-to-timber joints (TRADA, 2007). Figure 3.5 shows an example of coach screws.

Figure 3.5 Coach screws, source www.timco.co.uk.

3.4.4 Dowels

Dowels are smooth or striated cylinders without a head made of steel that are inserted through pre-drilling holes with no clearance between the dowel and the drilled aperture. The diameter ranges from 6 mm to 30 mm. This type of connection performs very well under high laterally loading as nails and screws of equal core diameter, but dowels are available in larger diameter and in a way lower price. The main application of dowels is on a multi-shear setup. (TRADA, 2007). Figure 3.6 shows a metallic dowel.

Figure 3.6 Metallic dowel, source www.rs-online.com.

3.4.5 Bolts

Bolts are manufactured like squared headed screws having as additional parts a washer or/and a nut and have diameters that range from 6 mm to 30 mm. Screws are placed in pre-drilled holes; the hole diameter must be adjusted to the screw so that the clearance is the smallest possible. EC5 recommends a maximum clearance of 1 mm. The diameter of the pre-bore contributes to the bond strength. Bolts can withstand axial and lateral forces (TRADA, 2007). Figure 3.7 shows an example of a bolt and washer.

Figure 3.7 Bolt, source www.wuerth.de.

3.4.6 Glued-in rods

Glued-in rods for connections or reinforcement have been used in use in existing and new-built timber structures for almost 50 years. Although the lack of a standardized guideline various projects and constructions have successfully adopted this fastener. Glued-in rods also referred to as bonded-in rods, or in short version GIR, will be covered in the newer version of EC5. (CEN, 2019).

GIR's present good strength and are embedded into the wood, a characteristic that represents an appealing way of connecting members since the connection is invisible, this also contributes to fire-resistance since the rod will be protected by the surrounding timber material, providing longer resistance than exposed members. Rods are commonly manufactured in diameters from 12 mm to 30 mm and have a length:diameter ratio between 10 and 20 mm (Trada, 2009).

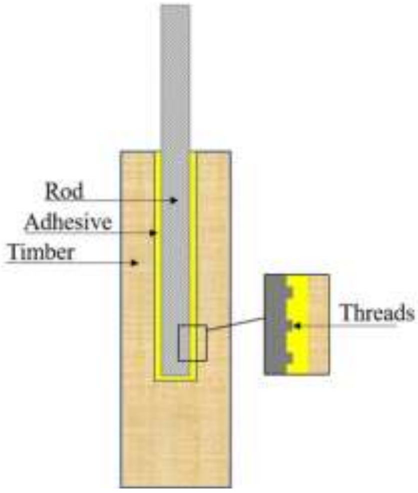


Figure 3.8 Glued-in rods mechanism (Feldt & Thelin, 2018).

The function mechanism relies on the adhesion interaction between the steel rod, the timber material, and the adhesive, see Figure 3.8. These three integrating members provide stiffness to the connection or reinforcement relying on the bond strength between them (CEN, 2019). The steel rods are positioned into pre-drilled holes containing the adhesive substance, the threads in the rods provide mechanical locking with the adhesive. The most common adhesive types used on the manufacture of GIR's are 2-part thixotropic epoxy resin, PRF resin, and 2-part polyurethane adhesive. Each adhesive possesses its own efficiency and advantages, based on the bored-hole diameter and service classes (Trada, 2009). The threaded rods are commonly manufactured on steel grade 380, although high strength steel grades can also be used in order to reduce the bar size. (Buchnan, 2007).

3.4.7 Long threaded rods

Long threaded rods are manufactured by rolling or forged wired rod, the process results in increased strength of the product, manufacture diameters range from 16 mm and 20 mm. Its main usage is as reinforcement of glued laminated beams against forces perpendicular to grain direction due to its high withdrawal capacity on axial forces. Threaded rods also perform well when lateral and axial loads are combined. Recent studies have set optimal inclination rod angles to the grain. (Cepelka et al., 2018). Figure 3.9 shows an example of a long-threaded rod.

Figure 3.9 Long-threaded rod, source www.sfs.ch.

3.4.8 Self-tapping screws

The development of self-tapping screws allowed overcoming the load-carrying capacity of axially loaded screws. In contrary to traditional screws, in which the threaded part is turned down from the original rod diameter, the threads of self-tapping screws are manufactured by rolling a wire around the shank, resulting in a smaller shank diameter in comparison to the outer cross-sectional threaded diameter. The continuous thread throughout its length provides a better transfer between the screw and the timber member, which increases the axial loading capacity. During production, the thread is hardened, resulting in an enhanced bending and torsional capacity, a down point is that this leads to a more brittle failure mechanism. The production ranges to diameters up to 20 mm and lengths up to 3000 mm. The fastener needs a pre-drilled hole to be inserted, moreover, some lubricant is recommended to reduce friction stress in installation. Depending on the length of the self-tapping screw, the axial load capacity may be limited by the tensile capacity of the steel or the buckling capacity in compression (Dietsch & Brandner, 2015). Figure 3.10 shows an example of a self-tapping screw.

Figure 3.10 Self-tapping screw, source www.wuerth.de.

3.5 Toothed-plate connectors

Toothed-plate connectors are metal elements that are pressed into the connecting timbers members. Diameters are available from 38 mm to 165 mm, but it is possible to find larger diameters to be used in glued-laminated members. Most of the connectors have a circular shape, but also oval shapes can be found for special applications. The joints are held together by bolts, nuts, and washers. Figure 3.11 shows an example of a tooth plate connector.

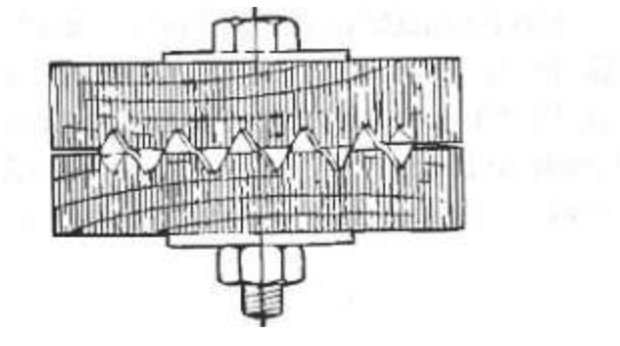


Figure 3.11 Tooth plate connector (TRADA, 2007).

3.6 Split ring and shear plate connectors

Split ring and shear plates connectors are metallic parts placed in pre-cut circular holes on the timber interfaces and held in position bolts. The diameters are available from 60 mm to 260 mm. Split ring connectors are used in laterally loaded timber-to-timber connections, while shear plate connector is utilized in steel-to-timber connections. The connections stiffness is influenced by the diameter of the connector and the timber density. Figure 3.12 shows the split ring geometry.

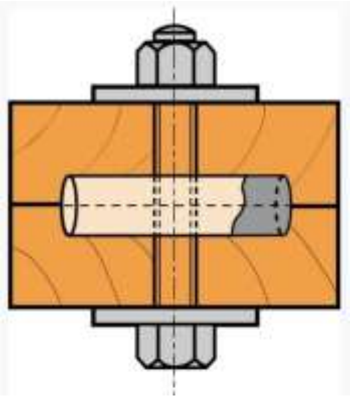


Figure 3.12 Illustration of a split ring, source www.strongtie.de.

3.7 Nail plates

Nail plates, as shown in Figure 3.13, are planar plates of a typical thickness of approximately 1-2 mm. They are commonly manufactured of light-gauge steel or stainless steel with predrilled nail holes with adequate spacing to avoid slip failure and are designed for an efficient number of fasteners. They are mainly fabricated in rectangular shapes and form joints between timber members by transferring load in shear at the surface of each component and are widely used on framing work. For prefabricated timber trusses so-called “punch-metal plates” are commonly used (TRADA, 2007), Figure 3.14 shows an example of a punch-metal nail plate.

Figure 3.13 Nail plate, source www.bovnail.com.

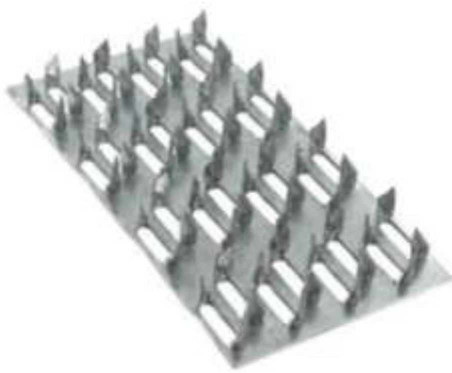


Figure 3.14 Punch-metal plate, source www.bovnail.com.

3.8 Connection examples

In order to present and provide a more comprehensive understanding and design guidelines about different types of timber joints, some solutions are summarized in the following Tables. The tables contain a number of timber joints for each type of force situation. The joints are adapted to different connection geometries and the resisted forces are compression, tension, shear, and bending moment. Pros and cons for every kind of joint in the tables are mentioned according to the following criteria:

- Fire resisting,
- Corrosion possibility,
- Neat Appearance,
- Workshop manufacture requirement.

The load-carrying capacity and stiffness of the joints are evaluated in the analysis carried out in Chapter 5.

- Fire resisting

One of the negative effects of a rise in temperature is an embedding strength loss in fastener type connections, which results in elongation of the fastener’s hole leading to a change of the failure mode in the timber connection. This effect can be treated by an insulating material and by increasing the thickness of the timber members.

When using external metal plates or some other metal part that is exposed to fire, the heat transfer rate to the timber member increases and the steel’s yielding strength is reduced. Due to this, exposed metal components are more susceptible to fire exposure and must be properly insulated. By using slotted in plates the fire risk is reduced due to the surrounding timber mass (Maraveas et al., 2015).

- Corrosion risk

Steel fasteners and plates are sensible to corrosion for several reasons.

  - Timber parts with PH less than 4,0 is detrimental and induce corrosion if there is direct contact with the metal segment.

  - Gaps filled with oxygen between fasteners and timber members increase the possibility of corrosion of metal.

  - Timber members with a moisture content of 18% or more, significantly increase the risk of corrosion of the metal parts of the connection. Rainfall and humidity are factors that influence the timber´s moisture content. These influences are expressed by service classes in EC5 (L. Zelinka, 2014).

In general timber joints with outer metal parts are more vulnerable to corrosion possibility.

- Neat Appearance

In modern timber structures, sealing the metal parts is preferable from an architectural point of view. Based on this trend, the neat appearance evaluation, which is related to a sense of design, is not evaluated in terms of the personal opinion of the designers.

- Workshop manufacture requirement

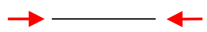
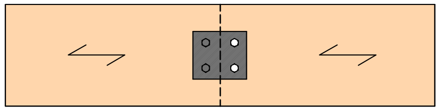
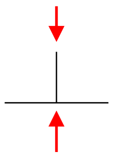
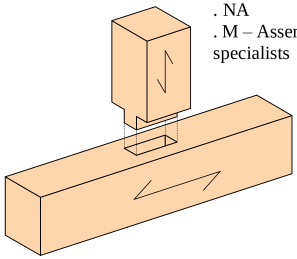
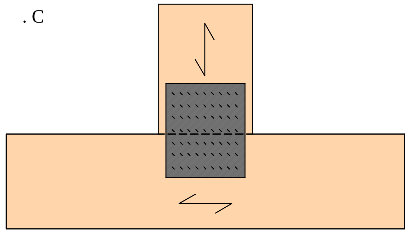
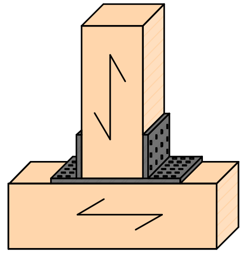
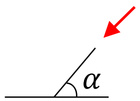
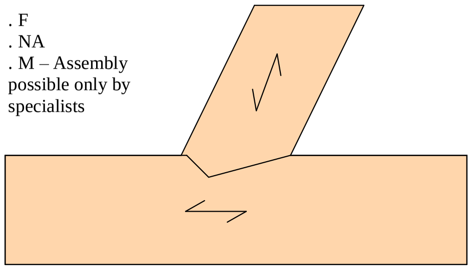
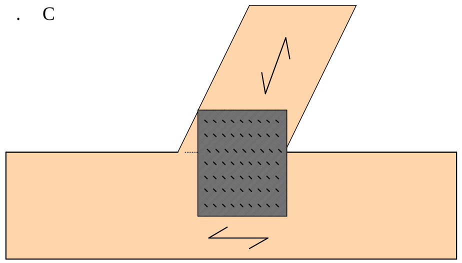
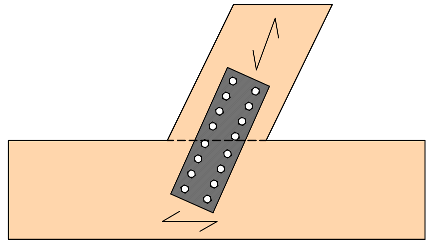
During the manufacturing and assembling of a timber joint, different processes are demanded, which determines if joints can be assembled on-site. Each type of connection has its special condition which is mentioned in the following Tables.

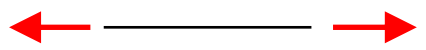
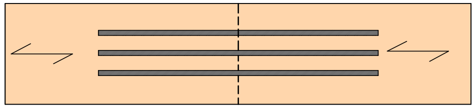
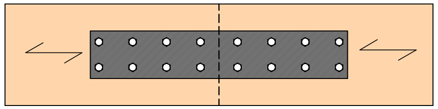
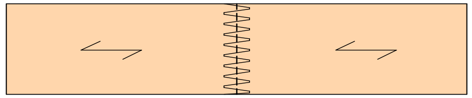
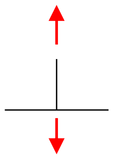
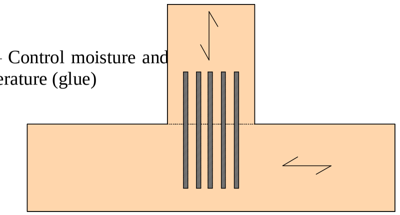
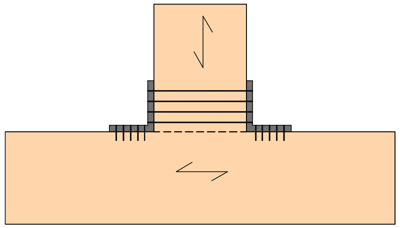
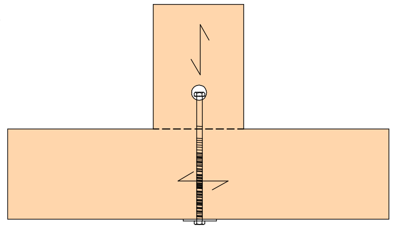
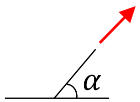
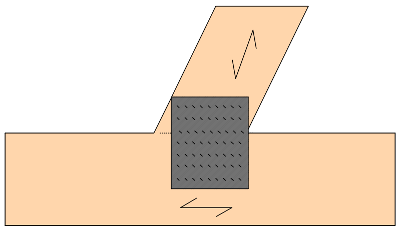
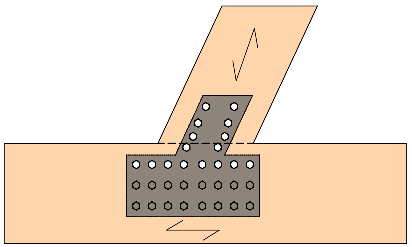
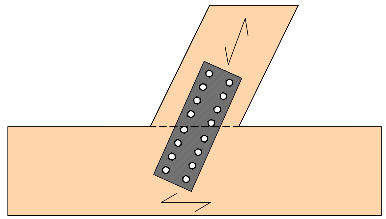
3.8.1 Example of timber connections

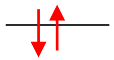
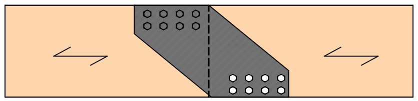
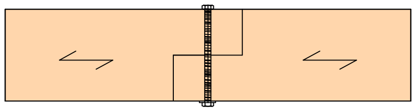
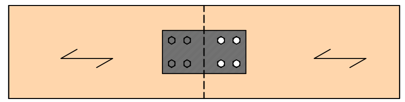
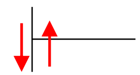
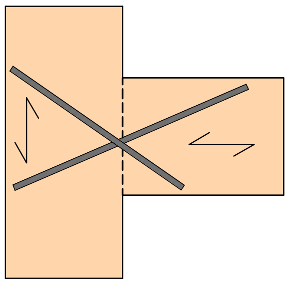
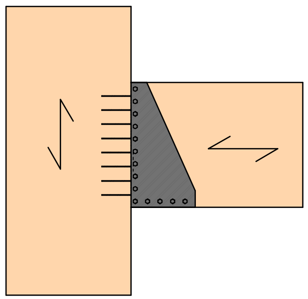
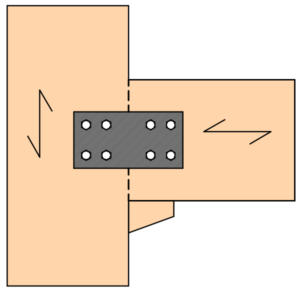
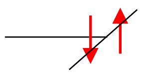
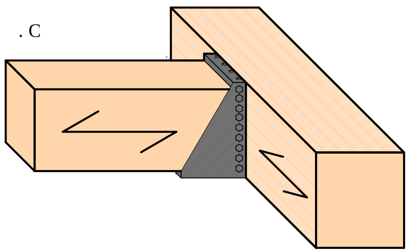
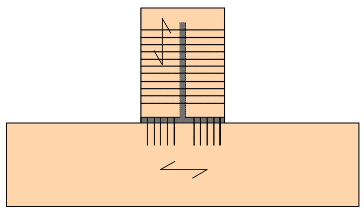
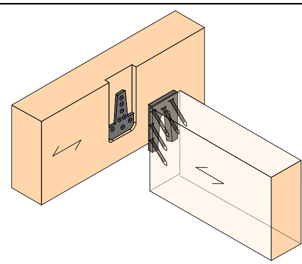
The following tables illustrate different types of connections.

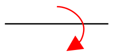
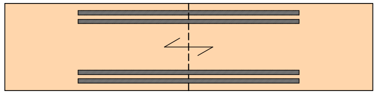
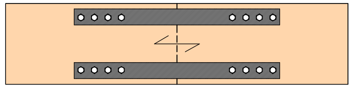
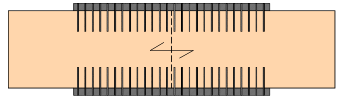
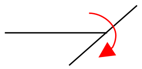
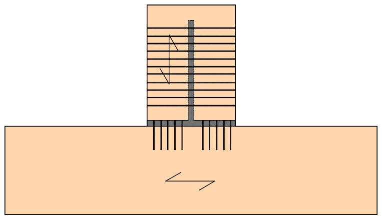
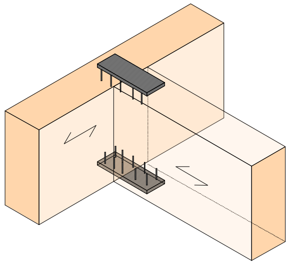
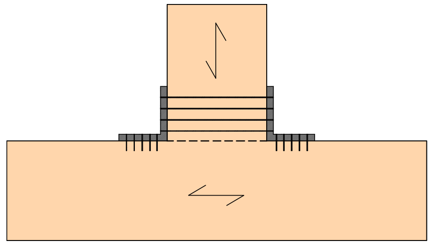
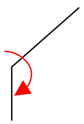
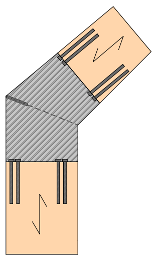
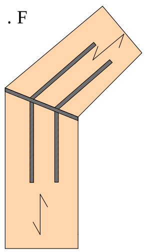
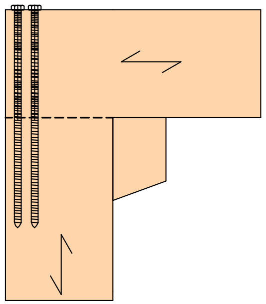
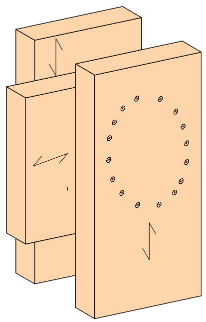
Compression

F – Fire Resistant
C - Corrosion possibility
NA - Neat Appearance
M – Workshop manufacture required

Splice Joints 	 **External metal plate with fasteners** installed for position control. The compressive force is transferred by the contact surface of the timber members which is sufficient.		
90° angle 	. F . NA . M – Assembly possible only by specialists  **Mortise and tenon joint**. Prevents lateral displacement of compression members.	. C  **Punched metal plate with nails.** Mainly used for prefabricated trusses. Members' thickness is limited to small values.	 **Angle metal plates with fasteners**. The bottom plate increases the contact surface area and spreads the compressive stress.
Inclined joint 	. F . NA . M – Assembly possible only by specialists  **Head notch joint**. Capacity depends on the skew angle α, the notch depth, and the end length.	. C  **Punched metal plate with nails.** Assure adequate plate width in the horizontal member to avoid large compressive stresses perpendicular to the grain.	. C  **Steel metal plate with fasteners.** Penetrated by nails screws and bolts.

Tension		F – Fire Resistant C - Corrosion possibility	NA - Neat Appearance M – Workshop manufacture required
Splice joints 	 . F . NA . M – Control moisture and temperature (glue)	 . C . NA	 . F . NA . M – Control moisture and temperature (glue)
	Glued in rods or threaded rods.	**Slotted in steel plate with fasteners.** Penetrated by dowels**.**	**Finger Joint.** The tension force is resisted by the shear capacity of the timber member on the glued surface.
90° angle 	. F . NA . M – Control moisture and temperature (glue) 	. C 	. F (Washer and nut must be protected.) . NA 
	Glued in rods or threaded rods. Hard-to-assemble solution. Rods perpendicular to grain must penetrate 70% of the member thickness.	**Double L steel plates with fasteners**	**Bolted beam to beam connection.** Washer area needs to be wide enough to avoid high compressive stress perpendicular to the grain, arising from anchoring effect from the tensioner bolt.
Inclined joint  	. C 	. F . NA 	. C 
	Punched metal plate with nails. Assure adequate plate width in the horizontal member to avoid large compressive stresses perpendicular to the grain.	**Slotted in-plate with dowels.**	**Steel metal plate with fasteners.** Penetrated by nails, screws, or bolts.

Shear		F – Fire Resistant C - Corrosion possibility	NA - Neat Appearance M – Workshop manufacture required
Splice joints 	 C . NA (if slotted-in)	 . C . NA	 . C
	Gerber joint. Used in large spans and strategically positioned where the bending moment approaches zero.	**Straight lap joint reinforced with a bolt.**	**External double shear plates.** Plate length has to be controlled to avoid eccentrically moments.
Column-to-beam 	. F . NA 	. C 	. C 
	Glued in rods or threaded rods. Preferred inclination close to 45° to benefit from the high axial capacity of threaded members.	**Metal shoe with fasteners.**	**Notched beam and external metal plate.**
Beam-to-beam 	. C 	. F . NA 	. F . NA 
	Metal shoe with fasteners	**Concealed T-plate with dowels and fasteners.**	**Concealed beam hanger.** The first part is pocked in the header timber and fixed with nails, while the second is fitted to the end of the beam with screws.

Bending moment		F – Fire Resistant C - Corrosion possibility	NA - Neat Appearance M – Workshop manufacture required
Splice 	 . F . NA . M – Control moisture and temperature (glue). Insertion cavities are drilled on-site in cases of long connectors.	 . C . NA – Only if slotted-in.	 . C
	Glued in rods or threaded rods. Compressive forces can be transferred by the contact surface of the timber members or by GIR's.	**External top and bottom metal plates with fasteners.**	**Top and bottom steel to timber joints with (nails or screws).**
Beam-to-beam 	. F . NA 	. C 	. C 
	Concealed T-plate with dowels and fasteners.	**Top and bottom steel to timber joints with (nails or screws).**	**Double L steel plates with (bolts, nails, screws)**
Knee joint 	. NA . M – Control moisture and temperature (glue).  . F 	. F & C (if head is protected). . NA . M – Control moisture and temperature (glue). 	. F . NA 
	Gusset plate/Glued in Rods	**Screwed and notched connections**	**Forked connection with fasteners.** Ring steel dowels secure the connection.

4 Mechanical properties and failure modes in timber connections

Connections in timber structures represent a critical part with their local and global effect on the structure. Designers must have a good understanding of their behaviour and their response during the lifetime of the structure while they are subjected to different loads. One of the most important parts is to properly understand how the failure of the connection happens and what are the expected failure cases. In general, the failure modes, from a structural point of view can either be brittle or ductile. It is always recommended to design a joint so that it shows a ductile failure behaviour. In this case, it will be easier to predict when a collapse will occur. In other words, a brittle failure should be prevented and checked during the design process. In this section, the principal modes of failure in timber connections are prescribed to help the reader to understand the behaviour and the expected critical aspect during the design. Eurocode 5 is based on Johansen (1949), which is explained in the following sections.

In this chapter, different kinds of failure for different connections will be introduced and the mechanical properties for mechanical fasteners. The next part of this chapter will give an overview of joints behaviour and introduction to slip modulus and connections stiffness. Also, a simplified way to evaluate different kinds of stiffness.

4.1 Johansen theory – fasteners in timber-timber joints and panel-timber joints

4.1.1 Timber-timber joints

According to Johansen's theory, several failure modes could be distinguished when a dowel joint is subjected to shear forces. The assumption of Johansen's Theory is summarized that the connector and the timber connected members are acting in rigid plastic behaviour (Borgström & Karlsson, 2016). According to these assumption failure modes in Johansen Theory are considered ductile.

- Rigid-plastic behaviour of the fastener in bending.
- Rigid-plastic behaviour of the timber in embedding.

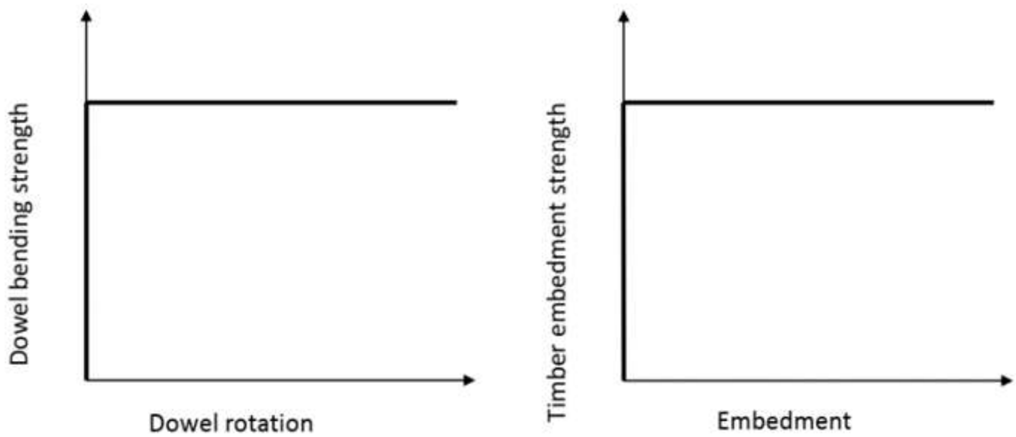


Figure 4.1 Strength-strain relationship for dowel connections (Borgström & Karlsson, 2016).

Based on Johansen's theory, several factors are considered decisive for the dominant failure modes. These factors are:

- Embedding strength of the timber member,
- Yielding moment of the dowels,
- The thickness of timber members.

(Borgström & Karlsson, 2016)

Figure 4.2 shows possible failure modes in timber-to-timber joints, single shear, and double shear. In timber to timber joint single shear, there are three various types of failure modes.

1. The failure occurs in the timber when the timber member crushes after reaching its embedding strength as in mode a, b. Or a rotational mode where crushing occurs in both timber members at the same time as in c.

2. The failure happens after forming a plastic hinge in the inserted dowels into the timber members if the timber member has enough thickness as in d, e, f. The plastic hinge is formed when the dowel bends and reaches the yielding strength.

3. In timber to timber joint double shear, the same failure modes are formed either by timber crushing or forming a plastic hinge as in modes g, h, j, k.

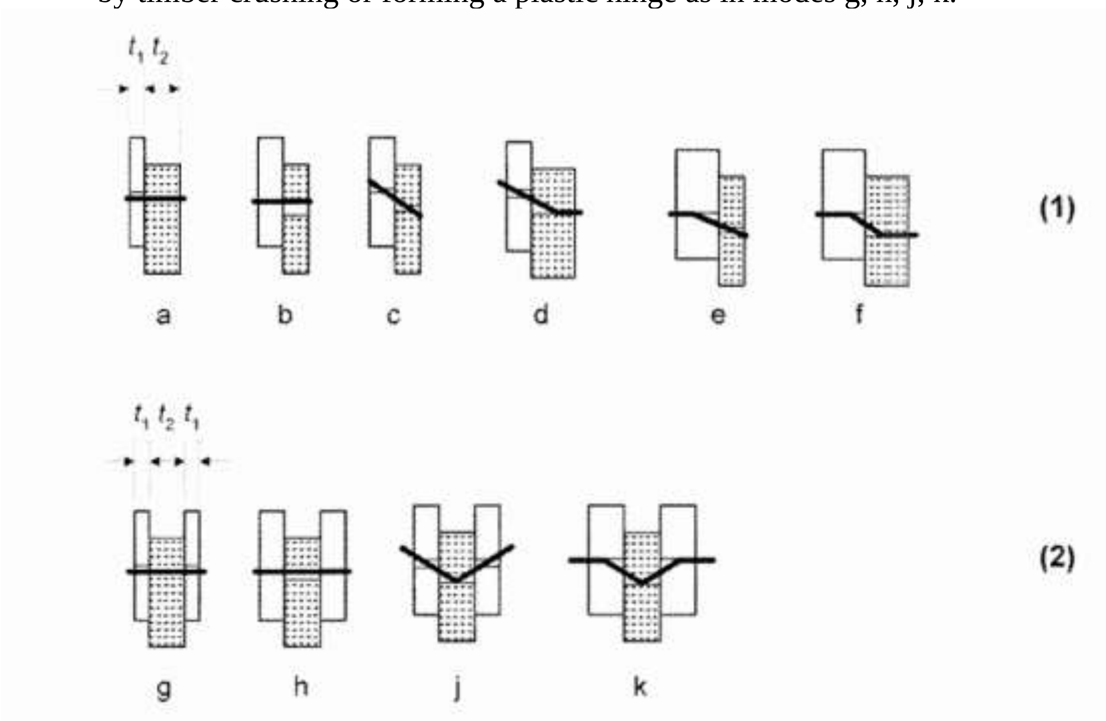


Figure 4.2 Possible failure modes in timber-to-timber joints EC5 (CEN, 2004).

4.1.2 Steel-to-timber joints

Another alternative is to use a steel plate as one of the members. One advantage of substituting is getting more stiffness and gaining more capacity. A plastic hinge will form in the interface between the steel and timber member. The steel plate works as a support to the dowel and the type of support depends on steel plate thickness (Borgström & Karlsson, 2016):

1. Thick steel plates work as fixed support which leads to forming a plastic hinge in the dowel under the steel plate. Modes c, d, e.

$$t_{steel} \geq d$$

2. Thin steel plate works as pinned support. In this case, no plastic hinge could be formed, instead, a rotation could be achieved. Modes a, b.

$$t_{steel} \leq 0{,}5.\,d$$

The failure modes on steel-to-timber joints are presented in Figure 4.3.

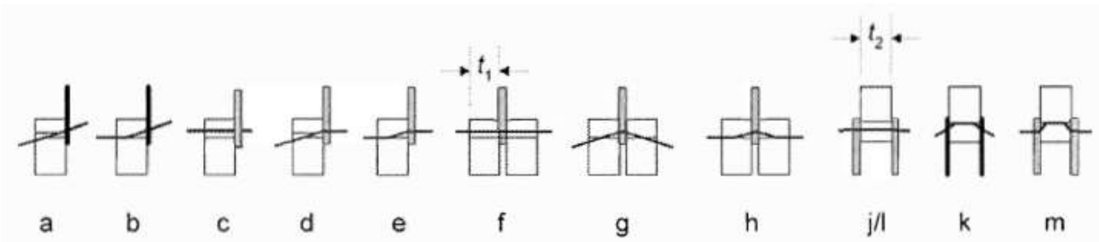


Figure 4.3 Possible failure modes for steel-to-timber connections.EC5 (CEN, 2004).

- Slotted-in steel plates:

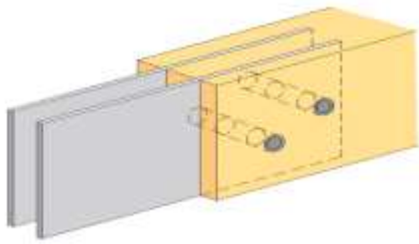


Figure 4.4 Two slotted-in steel plates connection (Borgström & Karlsson, 2016).

Despite the considerable capacity provided and the vast possibilities provided by using steel plates in more robust timber structures, the influence of the damage caused by the fire is considered as a major risk. A solution is to slot and hide in the steel plate inside the timber members. This kind of joints also has the advantage to avoid fire and weather exposure since the connection will be concealed.

The failure modes for slotted-in steel plates joints are based on the same manner described previously for steel to timber joints. Additionally, the plastic hinge is not related to the plate thickness (Borgström & Karlsson, 2016).

- Double shear steel-to-timber joints:

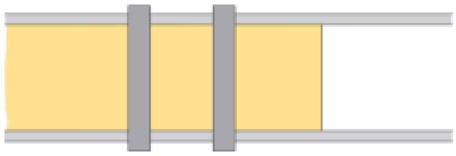


Figure 4.5 Double shear steel to timber joint (Borgström & Karlsson, 2016).

This type of steel-to-timber connection consists of steel plates located on the outer sides of the timber member. If the used fasteners do not fully penetrate the member's thickness, the connection is considered as a single shear timber-to-steel joint from both sides. While if there is a full penetration from bolts or dowels, the connection is a double shear timber-to-steel joint. This type of connection presents the same type of behaviour as in slotted-in-steel plates (Borgström & Karlsson, 2016).

4.1.3 Rope effect

When rotation takes place in dowels due to the formation of a plastic hinge or due to rotational failure, a new force arises in the form of tension f_{tens}. This force is generated from the withdrawal capacity of the dowels. The added value to the capacity from Johansen theory is $\frac{F_{ax.Rk}}{4}$. Table 4.1 shows that screws have the most contribution from rope effect which enhanced by the interlocking from the threaded part. Figure 4.6 demonstrates the mechanism of the rope effect.

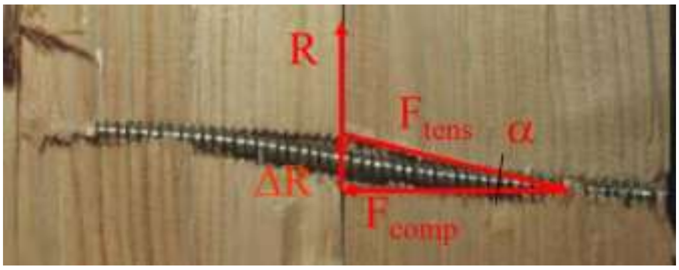


Figure 4.6 Forces acting on the fastener after yielding (VSM 196 – MPSEB).

Table 4.1 Rope effect contribution to dowel-type fastener on shear capacity.

Fastener type	Percentage
Round nails	15%
Square and grooved nails	25%
Other nails	50%
Screws	100%
Bolts	25%
Dowels	0%

4.1.4 Design situation

All the previous failure modes are presented in the EC5 8.2.2 and 8.2.3 (CEN, 2004) and in (Borgström & Karlsson, 2016) with figures.

4.2 Brittle failure

4.2.1 Group and wedge effect

Very often a group of fasteners is organised in the connection and several fasteners are in a row parallel to the direction of the grain. Then a group effect emerges that could affect the global capacity of the joint. If the distance of these fasteners is too narrow, the wedge effect formed around every fastener accumulates and a high-stress perpendicular to the grain occurs. In this case, a brittle failure occurs in corresponding to this stress

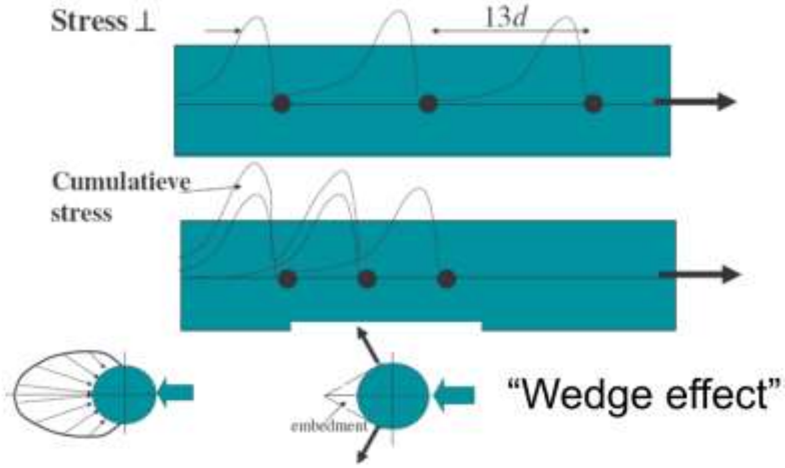


Figure 4.7 The wedge effect and group effect in a row of fasteners (VSM 196 – MPSEB*).*

In addition, the stress distribution in a row of fasteners is varied due to several factors as:

- Variation of timber strength,
- Hole size,
- Misalignment,
- Uneven load transfer.

Eurocode 5 treats the group effect depending on the type of fastener and gives limits to the spacing between the fasteners to be used in designing as tables. By applying these tables during the design, the splitting and the row shear could be avoided for a certain level. Also, EC5 gives formulas to count the effective number of fasteners in a row parallel to the grain.

$$n_{ef} = n^{k_{ef}} \quad screws\ d \leq 6mm, nails\ and\ staples \tag{2}$$

$$n_{ef} = min \begin{cases} n \\ n^{0,9} \sqrt[4]{\dfrac{a_1}{13 \cdot d}} \end{cases} \quad screws\ d > 6mm, bolts\ and\ dowels \tag{3}$$

a_1 - The distance between fasteners parallel to the grain.
n – Number of fasteners in a row parallel to the grain.

4.2.2 Brittle failure modes in dowelled joints

The brittle failure modes are the most critical modes and should be avoided in all the joints. When the dimension of the connection does not provide enough spacing between the fasteners, a brittle failure occurs due to group effect. Also, load perpendicular to the grains especially the tension could cause brittle failure. Shrinkage in certain conditions could also cause brittle failure.

The brittle failure modes that should be checked are row shear, block shear, plug shear, and splitting. Eurocode 5 gives in Appendix A a method to check the block and plug shear while there is no clear way to check the splitting and the row shear.

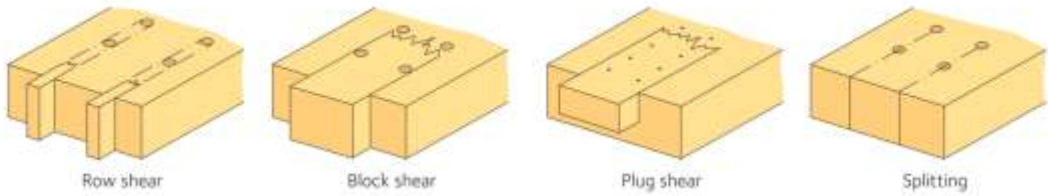


Figure 4.8 Types of brittle failure modes(Borgström & Karlsson, 2016).

In a row of fasteners, block/plug shear or row shear failure might occur. Also, in nailed joints, row shear never appears, and block shear is unlikely to happen, and it only could be checked on plug shear.

EC5 Annex A(CEN, 2004) gives guidelines to calculate the block and plug shear but it does not cover cases where multi shear planes as multi slotted-in metal plates are used.

1. Block shear failure:

In block shear failure, two components are contributing to the resistance; tension of the end face and the shear of the side face.

$$F_{bs.Rd} = max \begin{cases} 1.5 \cdot A_{net.t} \cdot f_{t.o.d} \\ 0.7 \cdot A_{net.v} \cdot f_{v..d} \end{cases} \quad (connectors) \qquad (4)$$

$A_{net.t}$ *- Net area at the end of the plug.*
$A_{net.v}$ *- Net area of the sides of the block.*

- 1.5 is based on the high local tensile strength at a joint.
- 0.7 is based on the volume effect that affects the shear strength when the loaded area is large.

2. Plug shear failure:

This type of failure mode is similar to block shear and the same concept used to calculate block shear capacity is used in plug shear. The difference is in $A_{net.v}$ which has an extra area at the bottom of the split part.

3. Multi-shear planes:

EC5 does not present a method to calculate the block shear where multi shear planes are used. The following is going to be used in the design of connections presented in Chapters 5 and 6 (COST, 2018).

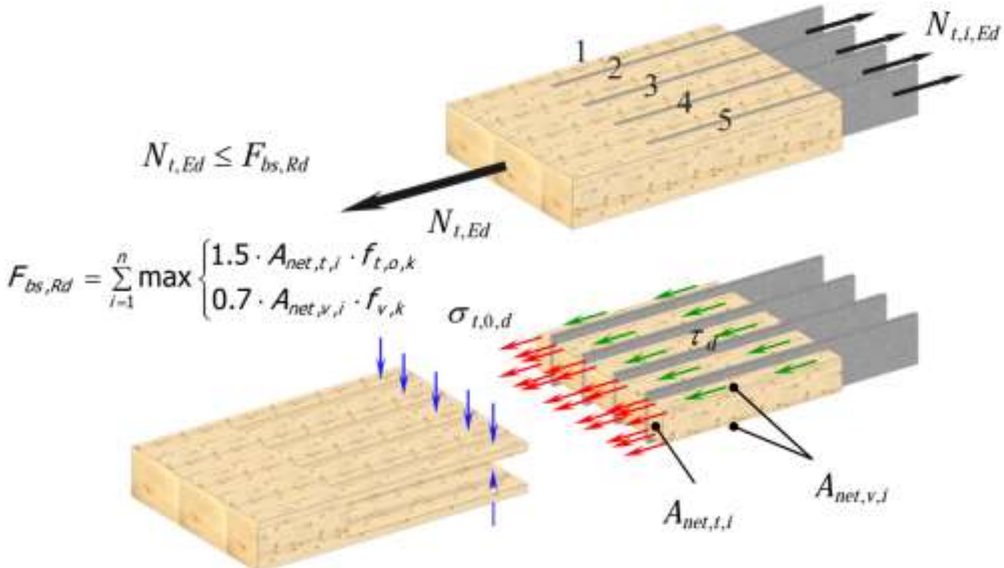


Figure 4.9 Block shear failure in multi- shear plane connections on tension (COST, 2018).

4.2.3 Shrinkage and swell

Timber responses differently in various grain directions (parallel, tangential, radial) to the change in moisture content. This response which is swelling or shrinkage is also dependent on the type of timber. Shrinkage is more resilient in the direction perpendicular to the grain than parallel to the grain. A significant reduction in moisture content generates concentrated tensions perpendicular to the grain if the shrinkage is prevented. In this case, the member will present a brittle failure, therefore this situation must be avoided. A horizontal timber member connected to a vertical timber or steel member can experience this failure mode. The section size of a timber member can be an effective factor in the formation of a collapse mechanism. If a large section is subjected to moisture fluctuations, higher stresses due to shrinkage appear, causing a negative effect on the shear resistance of the member. Figure 4.10 illustrates how splitting in timber fibres occurs when a timber member shrinks and is subjected to tension perpendicular to the grain (Ozelton & Baird, 2002).

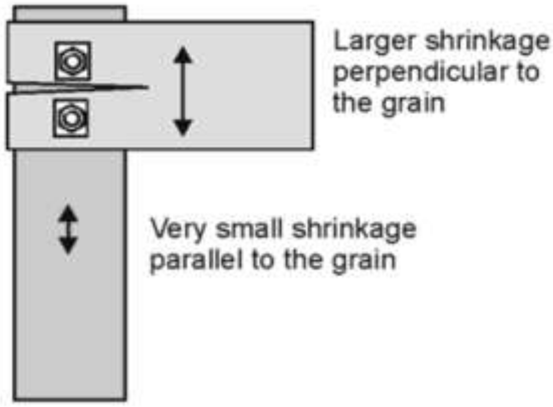


Figure 4.10 Shrinkage effect (VSM 196 – MPSEB*).*

4.3 Axial capacity and the related failure modes

Mostly, fasteners are subjected to an axial force in addition to shear forces. The dowel's ability to resist this force depends on the interface surface between the dowel and the joined members, and its anchorage capacity. In a situation where tensile forces are generated caused by rope effect or any other axial force, the dowel's tensile capacity must be enough to resist these forces (Borgström & Karlsson, 2016). These are the related failure modes:

1. Withdrawal caused by tensile force on the dowel and related to the shear capacity of the timber around the dowel.

    - Shear Failure 1: force parallel to the grain (extraction of a wooden cylinder).
    - Shear Failure 2: force perpendicular to the grain (cut of the grain).
    - Strength in 2 equals $\frac{2}{3}$ strength in 1 approximately.

2. Pushing in or buckling caused by compressive force.

3. Head pull through: It is related to the dimension of the washer of the dowel and the compressive strength of the spurned timber material.

4. Tensile failure of the dowels when the tensile strength of a dowel is exceeded.

4.3.1 Tensile capacity of single dowel according to EC5

The characteristic withdrawal capacity of dowels provides resistance to axial force. The resistance varies due to the type of profiling, threaded part, annular rings. Also, it depends on the washers and nuts, or any other anchorage mechanisms.

1. Nails:

It is related to the roughness of the surface along the threaded penetrated length of the nail and the anchorage capacity of the nails head. It equals the minimum of:

- Characteristic point-side withdrawal strength $f_{ax,k}$ which depends on the nail´s surface smoothness
- Characteristic head side pull-through strength $f_{head,k}$.

$$f_{ax.k} = 20.10^{-6} \cdot \rho_k^2 \qquad (5)$$

$$f_{head.k} = 70.10^{-6} \cdot \rho_k^2 \qquad (6)$$

ρ_k *- Characteristic timber density in kg/m³.*

2. Bolts:

In bolts, it depends on the anchorage capacity of the washer and nuts $F_{ax.washer,Rk}$, and the tensile capacity on the bolts itself.

$$F_{ax.washer.k} = 3. f_{c.90.k}. A_{washer} \qquad (7)$$

3. Screws:

The withdrawal capacity of the screws considers the most efficient and depends on the threading part. The tear-off and the pull-through failure of the screw head should be checked by determining the characteristic pull-through strength of the screw $f_{head,k}$.

$$f_{ax.k} = 0.52 \cdot d^{-0.5} \cdot l_{ef}^{-0.1} \cdot \rho_k^{0.8} \qquad (8)$$

- Limitations and designing equations are found in EC5 (CEN, 2004).

4.4 Shear capacity of single dowel

A dowel type connection is commonly used to resist lateral forces acting perpendicular to the dowel axes. In this case, the dowels behave like a beam subjected to forces along its length inside the connection. The behaviour of the dowel as a beam differs according to the thickness and the properties of the dowels and the timber members. If the dowel is stocky enough no bending in the dowel and no plastic hinge will be formed and the failure depends in this case on the embedding strength of the timber. While if the thickness of timber or the connected part is large enough no high stress acting on the timber and the dowel will bend and cause a ductile behaviour.

More specifically, certain parameters decide the shear capacity of the dowel joint (Ozelton & Baird, 2002).

- The embedding strength of the timber f_h.
- The yielding moment of the dowel M_y.
- The anchorage capacity enabling tensile action in the dowel f_{ax}.

4.4.1 Embedding strength

When the joint is subjected to lateral force, the dowel presses the surrounding timber and causes stress. This stress should not exceed a limit value called the embedding strength. The embedding strength is an important property of the timber joints and it is defined as the maximum stress applied on the timber around the dowel that could be resisted.

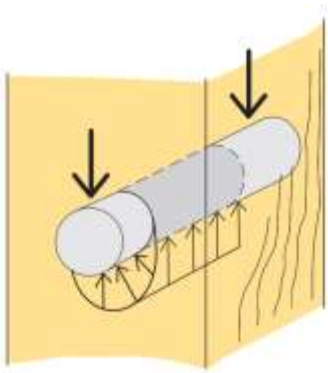


Figure 4.11 Embedding stresses on a dowel(Borgström & Karlsson, 2016).

According to (Borgström & Karlsson, 2016) the embedding strength is affected by several factors:

1. Density of timber ρ: proportionally increased.
2. Fastener diameter: the small diameter shows higher embedding stress.
3. The angle between grain and load direction α: the higher value is when the embedding stress acts as compression parallel to the grain and the lowest when it is compression perpendicular to the grain.
4. The friction between the dowel and timber members.
5. First type of friction appears if there is direct surface contact between the dowels and the connected timber members and it is highly affected if there is shrinkage or there is a change in moisture content in the timber members.
6. Second type appears when the fasteners bend, this type appears in failure modes when the fastener yields and the rope effect explain this type of friction.
7. Moisture content in the wood: in general, the moisture plays a negative role in the strength in timber so it is preferred that the moisture should below.
8. Pre-drilled hole or not.
9. Reinforcement perpendicular to the grain: it provides more resistance to the stresses perpendicular to the grain and so increasing in the embedding strength.

The characteristic embedding strength according to the EC5. 8 (CEN, 2004):

$$f_{h.0k} = 0{,}082 \cdot \rho_k \cdot d^{-0{,}3} \quad \textit{without pre-drilling, } d < 8mm \tag{9}$$

$$f_{h.0.k} = 0{,}082(1 - 0{,}01d) \cdot \rho_k \quad \textit{with pre-drilling, all } d \tag{10}$$

$f_{h.o.k}$ *- The characteristic embedding strength parallel to the grain* [N/mm ^{2}].

When the shear force acts with an angle between the loads and the grains:

$$f_{h.a.k} = \frac{f_{h.o.k}}{k_{90} \cdot sin^2 a + cos^2 a} \text{ [N/mm}^2\text{]} \quad (11)$$

Where:

k_{90} = 1,30+0,015 · d for softwood.

4.4.2 Yielding moment

When the dowel bends and the moment reach its yielding moment, a hinge will be formed somewhere in the dowel. The failure, in this case, is ductile and it is preferred to happen and could be achieved by having small dowel diameter. The yielding moment depends on the diameter of the dowel and the properties of dowel material.

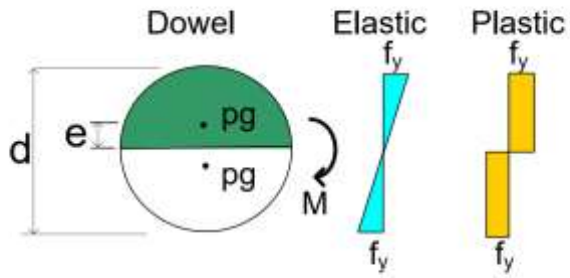


Figure 4.12 Plastic and elastic stress distribution on a fastener (VSM 196 – MPSEB).

- Limitations and designing equations are found in EC5 Chapter 8 (CEN, 2004).

4.5 Glued-in rods

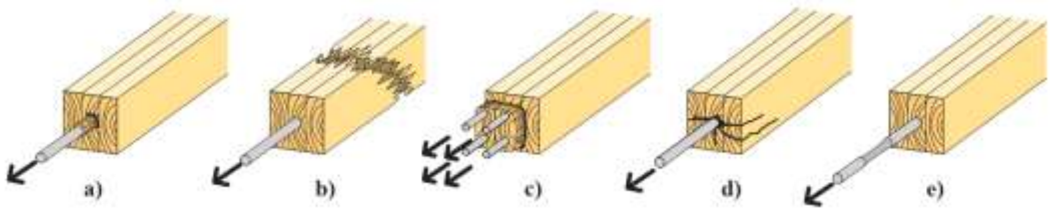


Figure 4.13 Failure modes for glued-in rods (Tlustochowicz et al.2010).

The modes of failure are related to the materials in the connections, their mechanical properties, and the properties of the bonds between them.

1. Pull -out failure (a) is dependent on the shear located at the interface between the adhesive and the timber and is characteristic for single rods in axial tension

and compression. It could happen for multiple glued-in rods systems when the timber volume around the rods is not enough (c).

2. Splitting caused by tensile stresses in the timber around single rods (b). It is more critical in rods glued-in perpendicular to the grain.

3. Splitting mode caused by tension perpendicular to the grain in (c) and (d). It occurs in the case of imperfect axial load or inclined force to the grain. Spacing recommendations are used to avoid this type of failure.

4. Failure is caused by the yielding of the rods and it is a preferable mode since it is a ductile failure. This could be achieved when the tensile failure in rods occurs before the other failure modes and therefore normally, not high strength steel is used in glued-in rods. Furthermore, a design where the steel part is the weakest link of the connection leads to robust structures that prove the possibility of dissipating energy under hazardous actions as i.e. earthquakes (Tlustochowicz et al.2010).

The current EC5 does not provide equations to calculate the axial and lateral capacity of GIR's. Many researchers have recommendations or equations. In this thesis, equations from Limträhandbok del 3, Table 13.2 *(Svenskt trä, 2016), and* "Design of bonded-in-rods" (CEN et al., 2019) have been used.

The design axial capacity of a single rod is the minimum value between the tensile capacity of the *r*od $F_{t.Rk.rod}$ and the timber pull-out capacity $F_{t.Rk.timber}$.

$$f_{ax.k} = 5.50\ MPa \tag{12}$$

$f_{ax.k}$- *Withdrawal capacity of the rod.*

$$F_{t.Rk.rod} = 0.6 f_{u.b} \cdot A_s \tag{13}$$

$F_{t.Rk.rod}$ - *Tensile capacity of the rod* [kN].
A_s - *Steel cross-sectional area.*

$$F_{t.Rk.timber} = \pi \cdot l_i \cdot f_{ax.k} \cdot k_1 \cdot \kappa_1 \tag{14}$$

$R_{t.k.timber}$ - *Timber pull-out capacity until 1-meter rod length.*
l_i - *Glued length of the rod.*
κ_1 - *Weather factor. Limträhandbok del 3, Table 13.23. (Svenskt trä, 2016).*
k_1 - *Reduction factor of the penetration factor. Limträhandbok del 3, Table 13.24. (Svenskt trä, 2016).*

The maximum effective length contributing to the timber's pull-out capacity is 1 meter and after this length, there is no increase in shear capacity around the rod. Also, the maximum length of the rods is 3 meters which could be used to strengthen the timber material in some locations. Following the spacing in Limträhandbok del 3, Table 13.23 will prevent splitting at the edges.

4.6 Combined loading

In many cases, the fastener is subjected to lateral and axial load at the same time. In this case and according to Eurocode 5, the interaction of the forces should be checked:
For smooth nails:

$$\frac{f_{ax.Ed}}{f_{ax.Rd}} + \frac{f_{v.Ed}}{f_{v.Rd}} \leq 1 \tag{15}$$

For another fastener:

$$\left(\frac{f_{ax.Ed}}{f_{ax.Rd}}\right)^2 + \left(\frac{f_{v.Ed}}{f_{v.Rd}}\right)^2 \leq 1 \tag{16}$$

4.7 Stiffness and slip modulus

For a reliable design of structures and beside the load-carrying capacity, the deformation as a result of stiffness and ductility of the connections is of importance. The stiffness, in general, is defined as the needed force or moment to cause a unit translation or rotation. In the joints, it depends on the slip modules related to the specific type of stiffness. In a more specific way, the stiffness depends on:

- Type of joint,
- Strength and quality of the parts,
- Grain direction,
- Type and magnitude of loads,
- Load duration and creep.

(TRADA, 2007)

The stiffness is important in the serviceability limit state as it should keep the deformation under a certain value allowed to assure comfortability and functionality of the structure during the lifetime. While in the ultimate limit state, the connection stiffness could affect the sectional forces distribution in the connected members if they are statically undetermined this will be further explained in Section 4.7.4.

The timber joint cannot be fully rigid, and it is in a more accurate behaviour a semi-rigid connection. That means it cannot prevent the translation or the rotation totally at the end of the connected structural elements. There is always a certain initial slip in curve deformation-load in timber connections. The relative stiffness caused by the semi-rigid behaviour causes a redistribution of stresses in the structure that would explain more in Section 4.7.4.

4.7.1 Load-deformation diagram of timber connection

Figure 4.14 shows the characteristic relation between the load and deformation in a timber joint and the curve consists of three distinguished stages (COST 2018).

- Initial displacement or initial slip: It happens at the beginning of load applying and it depends on the tolerance in the connections.

- Linear behaviour: it is used to determine the stiffness in the SLS as the constant slope.

- Non-linear behaviour: with load-increase the connection starts to show non-linear behaviour and the failure occurs. To determine the stiffness in ULS, the secant stiffness is used between two load points.

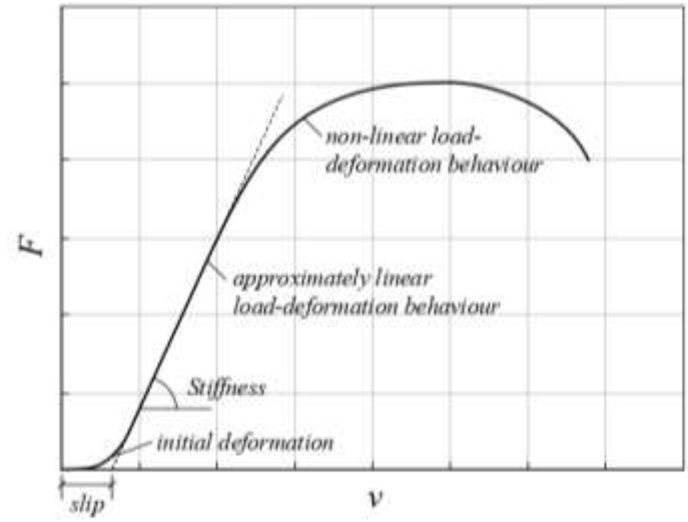


Figure 4.14 Load-deformation in a joint (COST 2018).

Experimental load-slip curves for joints in tension parallel to the grain (Racher,2001) show that the glued joint shows the highest stiffness but at the same time has a brittle behaviour. Moreover, the relation between the load and the deformation generally is non-linear.

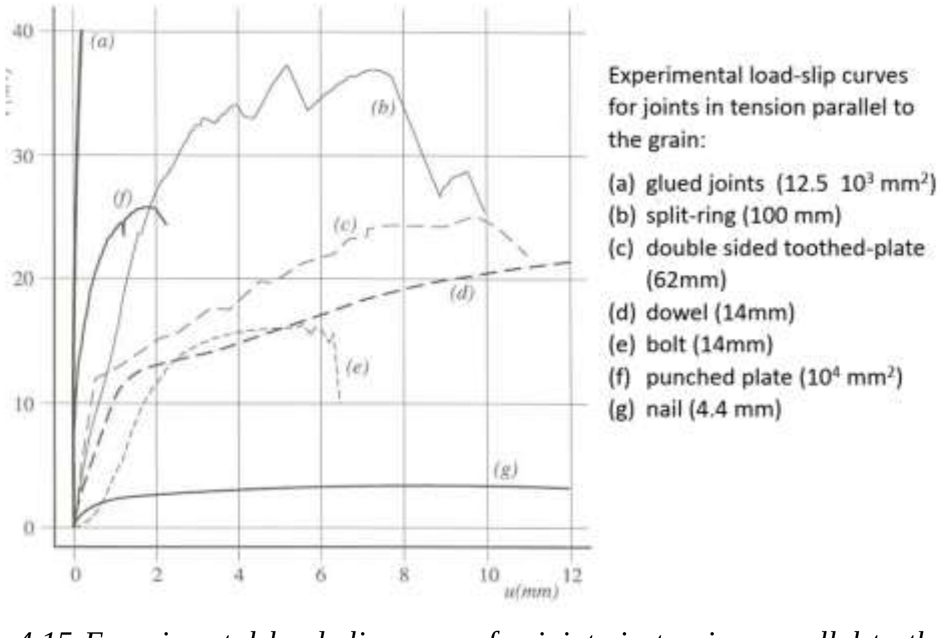


Figure 4.15 Experimental load-slip curves for joints in tension parallel to the grain (Racher,2001).

4.7.2 Eurocode 5

Eurocode 5 gives equations to calculate the stiffness in SLS to the single fastener in shear and is not covering deeply the more advanced or complicated joint cases. Also, Eurocode 5 gives a basic approach that the joint behaves in a linear way based on load-deformation relation, while the real behaviour according to tests is non-linear. According to that, Eurocode is satisfactory to design in the SLS to give enough limit for the deformation, but at the same time, there is a lack of deep understanding to get a more optimized design.

Eurocode provides the shear stiffness depending on the type of joints in Table 4.2 and it depends basically on timber density and fastener diameter.

The term slip modulus is defined in Eurocode 5 as the slope of the load-deformation curve. While in other books or references it refers to the deformation at a certain load level in the curve.

- The instantaneous stiffness in serviceability limit state design is taken as the secant modulus of the load-deformation curve at 40 percent of the maximum load (load-carrying capacity) of the joint according to EC5 and is calculated for fastener subjected to lateral force in one shear plane by the slip modulus in SLS $\mathrm{K_{ser}}$:

$$U_{inst} = F/k_{ser} \tag{17}$$

For steel-to-timber or concrete-to-timber connections, k_{ser} should be based on timber density and may be multiplied by 2.0. The background of the equations in Eurocode 5 comes from (Ehlbeck and Larsen, 1993).

- The instantaneous stiffness in ultimate limit state design is taken as the secant modulus of the load-deformation curve at a load level of approximately 60 to 70 percent of the maximum load. As an approximation Eurocode 5 gives the ultimate stiffness:

$$k_u = \frac{2}{3} \cdot k_{ser} \tag{18}$$

- The final stiffness after a long time is affected by the creep deformation and is presented in EC5 by $k_{def.joint}$ factor. Some recommendation about this factor when it is related to timber connections with different components and EC5 (2.3.2.1) states that the value of $k_{def.joint}$ should be doubled for mechanically fastened connections. The final deformation should be checked with the quasi-permanent load combination by considering the creep factor:

$$k_{ser.fin} = \frac{k_{ser}}{(1 + k_{def.joint})} \tag{19}$$

4.7.3 Stiffness joints in timber structures

In a three-dimensional structure, a joint has six degrees of freedoms, three translation ($\delta_x, \delta_y, \delta_z$) and three rotation ($\theta_x, \theta_y, \theta_z$). This is corresponding to three translational and three rotational spring stiffnesses

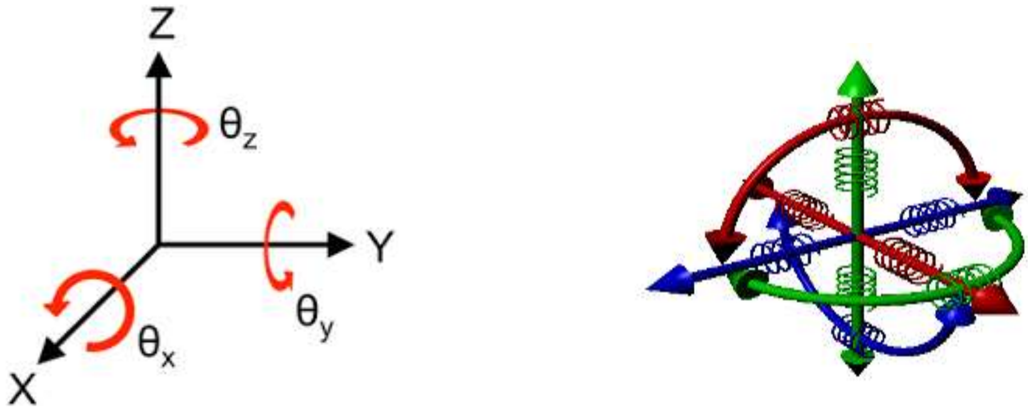


Figure 4.16 Translational and rotational springs.

Some simplified methods are going to be used in this project to determine different stiffnesses in a timber joint.

1. The translational stiffness responds to a fastener subjected to lateral force and is presented in EC5, Table 4.2 is a table extracted from EC5.

Table 4.2 Values of K_{ser} for fasteners and connectors in N/mm in timber-to-timber and wood-based panel-to-timber connections from EC5 (CEN, 2004).

Fastener type	K_{ser}
Dowels Bolts without clearance [a] Screws Nails (with pre-drilling)	$\rho_m^{1,5} \cdot d/23$
Nails (without pre-drilling)	$\rho_m^{1,5} \cdot d^{0,8}/30$
Staples	$\rho_m^{1,5} \cdot d^{0,8}/80$
Split-ring connectors type A according to EN 912 Shear-plate connectors type B according to EN 912	$\rho_m \cdot d_c/2$
Toothed-plate connectors: -Connectors types C1 to C9 according to EN 912 -Connectors types C10 to C11 according to EN 912	$1{,}5 \cdot \rho_m \cdot d_c/4$ $\rho_m \cdot d_c/2$
The clearance should be added separately to the deformation.	

2. Axial stiffness for GIR according to (CEN et al., 2019):

$$K_{ser.rod} = 0.004 \cdot d^{1.8} \cdot \rho_{mean}^{1.5} \tag{20}$$

3. According to Descamps et. al. (2006), the stiffness from the contribution of the contact surface of timber members could be calculated as follow:

- For components with lengths that are in the same order as the other dimension, equation (21) is used and referred to as member 1 in Figure 4.17.

$$k = \frac{EA}{L} \tag{21}$$

- For components with lengths that are far longer than the dimensions of the subjected area of the component equation (22) is used and referred to member 2 in Figure 4.17.

$$k = \frac{E\sqrt{A}}{0.85} \tag{22}$$

E -Young's modulus.
A - *Timber cross-section subjected to the force.*

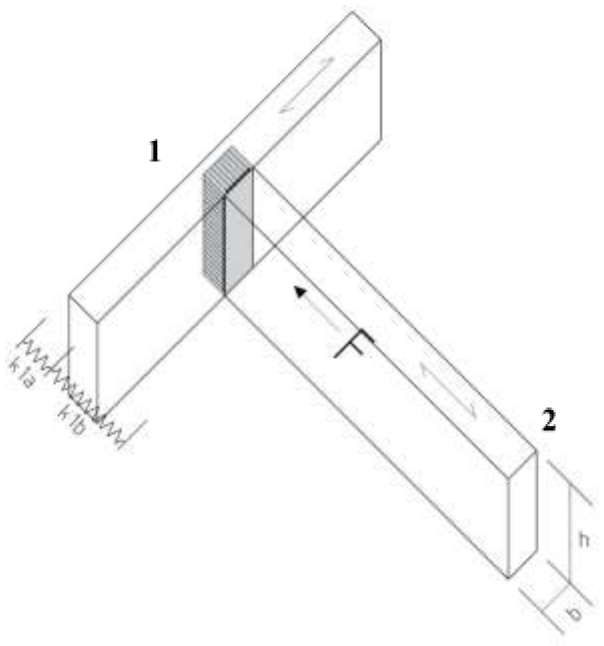


Figure 4.17 Diagram showing the concept of the stiffness contribution due to contact.

4. The rotational stiffness is based on equation (23). The analogous expressed in equation (24) helps to calculate the rotational stiffness in different joints:

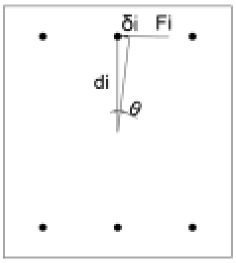


Figure 4.18 Rotational stiffness from the lateral force on one dowel.

$$\theta = \frac{M}{k_r} \tag{23}$$

θ *- Rotational angle.*

Assuming a small rotation, the deflection δ can be expressed as:

$\delta = sin(\theta) \cdot d$

d*- Distance from the natural axes.*

$$k_r = \frac{M}{\theta} = \frac{F_i.d_i}{\theta} = \frac{k_{ser}\,.\delta_i.d_i}{\theta} \approx \frac{k_{ser}\,.\theta.d_i.d_i}{\theta} = k_{ser}\,.\,d_i^2 \tag{24}$$

4. When several components (n) contribute to the joint's stiffness, they work in parallel or in series, see Figure 4.19. The Equations (25) and (26) show how the combined stiffness is calculated in those cases.

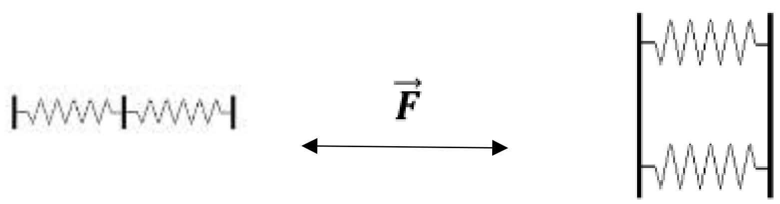


Figure 4.19 Springs in series. *Figure 4.20 Springs in parallel.*

$$k_{series} = \left(\sum_{i=1}^{n} \frac{1}{k_i}\right)^{-1} \tag{25}$$

$$k_{parallel} = \sum_{i=1}^{n} k_i \tag{26}$$

4.7.4 Joint stiffness in the global level

In a structural system, members are assumed to have even pinned or fixed boundary conditions. While in timber structures, this assumption is not valid, since timber connections show a semi-rigid behaviour (Aicher et al., 2012). Assuming a fully rigid joint is unrealistic and hard to achieve. While assuming a pinned member will lead to oversized members while in the real situation the member shows another behaviour between pinned and fixed depending on the real joint stiffness.

Many reasons lead to this behaviour. Timber is a soft material and has a relatively low modulus of elasticity comparing with steel material. Another reason is the low loose initial deformation modulus as depicted in Figure 4.14 which related to fasteners tolerance.

$$E_{GL30} = 13000MPa \ , \ E_{steel} = 210000MPa$$

The semi- rigidity could be represented in the six springs with a constant value between zero and infinity to give a more realistic approach during the design.

- Semi-rigid study:

Two simple studies on the effect of the semi-rigidity of timber joints on the global level have been investigated. One on the rotational stiffness and the second on the transversal stiffness. A semi-rigid beam model with a simply supported beam and fixed beam as the boundary condition will be derived to make a helpful comparison to understand the semi-rigid joint effect.

A one -span beam subjected to uniform load has been taken as an example. The displacement of the beam from Euler-Bernoulli theory:

$$-\omega''(x) = \frac{M(x)}{EI} \qquad (27)$$

Where:

ω''- *Curvature* [1/m].
$M(x)$ - *Bending moment at* x [N m].
E - *Young modulus* [N/m^2].
I – *Area moment of inertia* [m^4].

By substituting moment equation in Equation 27 :

$$EI\omega''(x) = Ax + M_A - \frac{qx^2}{2} \qquad (28)$$

By integration, the slope can (ω′) can be accessed:

$$EI\omega'(x) = \frac{Ax^2}{2} + M_A x - \frac{qx^3}{6} + C_1 \qquad (29)$$

Deflection (ω) can be accessed from slope equation integration:

$$EI\omega(x) = \frac{Ax^3}{6} + \frac{M_A x^2}{2} - \frac{qx^4}{24} + C_1 x + C_2 \tag{30}$$

A - Reaction [N].
q - Uniformly distributed load [N/m].
C_1 – Integration constant [Nm^2].
C_2 – Integration constant [Nm^2].

- Simply supported beam:

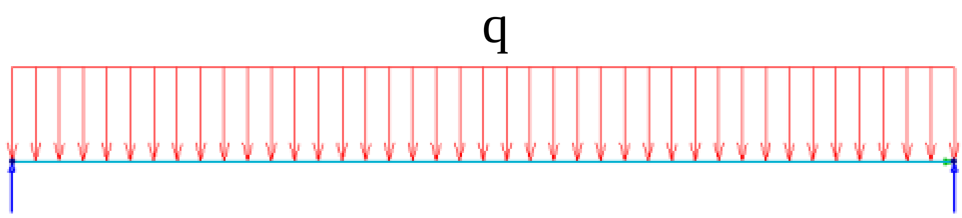


Figure 4.21 Beam with pinned supports.

By inserting the boundary condition. There is no transversal deformation in the supports and the pinned supports allow for the rotation that results in $\mathrm{M_{support}} = 0$:

$$\omega(x = 0) = 0 \Rightarrow C_2 = 0 \tag{31}$$

$$\omega(x = L) = 0 \Rightarrow C_1 = -\frac{AL^2}{6} + \frac{qL^3}{24} \tag{32}$$

$$A = \frac{qL}{2} [kN] \tag{33}$$

$$M_A = 0 \text{ [kNm]} \tag{34}$$

Which gives:

$$EI\omega(x) = \frac{9Lx^3}{12} - \frac{qx^4}{24} - \frac{qL^3 x}{12} + \frac{qL^3 x}{24} \tag{35}$$

The maximum deflection in the middle of the span (x = L/2) is:

$$EI\omega(x = L/2) = \frac{4qLx^4}{384} - \frac{qL^4}{384} - \frac{16qL^4}{384} + \frac{8qL^4}{384} \tag{36}$$

$$\delta = -\frac{5qL^4}{384EI} \tag{37}$$

δ – *Maximum deflection* [m].

- Fixed beam:

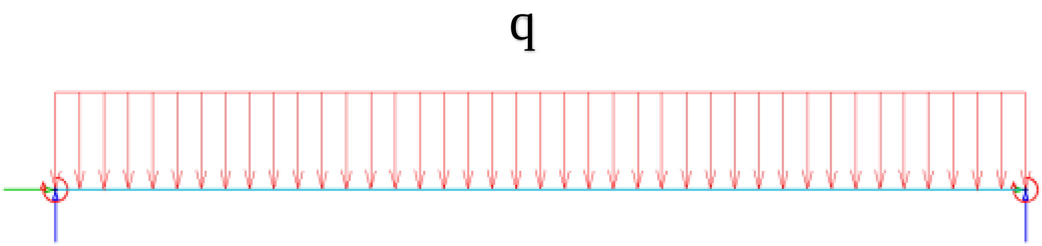


Figure 4.22 Beam with fixed supports.

Appling the boundary condition where $M_{support} = \frac{L^2 q}{12}$:

$$\omega(x = 0) = 0 \Rightarrow C_2 = 0 \tag{38}$$

$$\omega'(x = 0) = 0 \Rightarrow C_2 = 0 \tag{39}$$

$$A = \frac{qL}{2}\,[\text{kN}] \tag{40}$$

$$f_{ax.k} = 5.50\ MPa M_A = \frac{L^2 q}{12}\,[\text{kNm}] \tag{41}$$

Which gives:

$$EI\omega(x) = \frac{qLx^3}{12} - \frac{qL^2x^2}{24} - \frac{qx^2}{24} \tag{42}$$

The maximum deflection in the middle of the span (x = L/2) is:

$$EI\omega(x = L/2) = \frac{qL^4}{96} - \frac{qL^4}{96} - \frac{qL^4}{384} \tag{43}$$

$$EI\omega(x = L/2) = \frac{4qL^4}{384} - \frac{4qL^4}{384} - \frac{qL^4}{384} \tag{44}$$

$$\delta = -\frac{qL^4}{384EI} \tag{45}$$

- Semi-rigid beam:

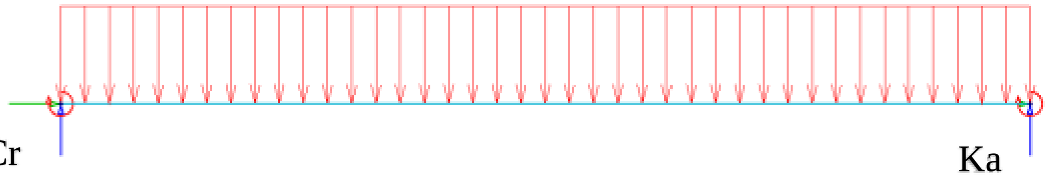


Figure 4.23 Beam with semi-rigid supports.

A semi-rigid connection as shown in Figure 4.23 is placed between the rigid and hinged connection. Appling the boundary condition for semi-rigid behaviour for the rotational stiffness:

$$\omega(x=0)=0 \Rightarrow C_2=0 \tag{46}$$

$$\omega(x=0)=0 \Rightarrow C_1=-\frac{AL^2}{6}-\frac{M_A L}{2}-\frac{qL^3}{24} \tag{47}$$

$$\omega'(x=0)=\theta=\frac{M_A}{c_r} \Rightarrow C_1=\frac{M_A EI}{c_r} \tag{48}$$

θ – *Rotation of the support* [rad].
M_A – *Support moment* [Nm].
c_r – *Rotational stiffness of spring* [Nm/rad].

Combining the boundary conditions:

$$\frac{M_A EI}{c_r}=-\frac{AL^2}{6}+\frac{M_A L}{2}+\frac{qL^3}{24} \tag{49}$$

$$M_A\left(\frac{1}{c_r}+\frac{L}{2EI}\right)=-\frac{qL^3}{24EI} \tag{50}$$

$$M_A\left(\frac{1}{c_r}+\frac{L}{2EI}\right)=-\frac{qL^3}{24EI} \tag{51}$$

Which can be written as:

$$M_A=M_{A,rigid}\,\frac{1}{\frac{2EI}{c_r L}+1} \tag{52}$$

Where:
$M_{A,rigid}=qL^2/12$ [Nm]

By substitution in Equation (52):

$$k = \frac{1}{\frac{2EI}{c_r L} + 1} \tag{53}$$

$$M_A = kM_{A,rigid} = kqL^2/12 \text{ [kN m]} \tag{54}$$

k is a factor defining the relation between the spring stiffness and the beam stiffness. Equation (52) with the reaction force is inserted in equation 30:

$$EI\omega(x) = \frac{qLx^3}{12} - k\frac{qL^2x^2}{24} - \frac{qx^4}{24} - \frac{qL^3x}{12} + k\frac{qL^3x}{24} - \frac{qL^3x}{24} \tag{55}$$

The maximum deflection is given:

$$EI\omega(x = L/2) = \frac{4qL^4}{384} - k\frac{4qL^4}{384} - \frac{qL^4}{384} - \frac{16qL^4}{384} + k\frac{8qL^4}{384} - \frac{8qL^4}{384} \tag{56}$$

$$\delta = -\frac{5qL^4}{384EI} + k\frac{4qL^4}{384EI} \tag{57}$$

Where:

$$k = \frac{1}{\frac{2EI}{c_r L} + 1} \tag{58}$$

The factor k is related to the rotational stiffness of the spring c_r. When $c_r$and k are 0 the deflection reaches the same deflection in simply supported beam Equation (30). While for k=1 and a high value for c_r the deflection converges to the deflection in the fixed beam Equation (45).

To verify the previous study with variable spring stiffness. A 10-meter-long beam (450 ×150 mm, GL28c) has been studied subjected to q=4 kN/m. According to k factor equation (53) and deflection equation (57) with semi-rigid behaviour.

$$k = \frac{1}{\frac{2EI}{c_r L} + 1} \qquad \delta = -\frac{5qL^4}{384EI} + k\frac{4qL^4}{384EI}$$

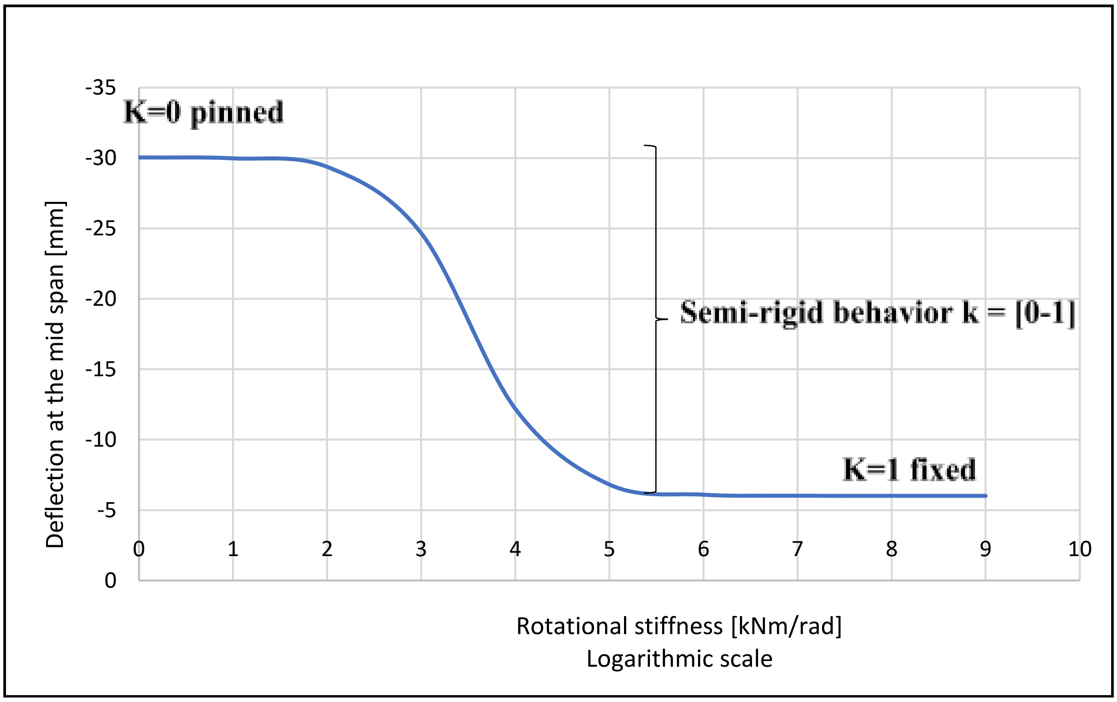


Figure 4.24 Relation between the rotational stiffness and the defalcation in mid-span.

To verify the result, a study in StruSoft (FEM-analysis) for the same beam has been done.

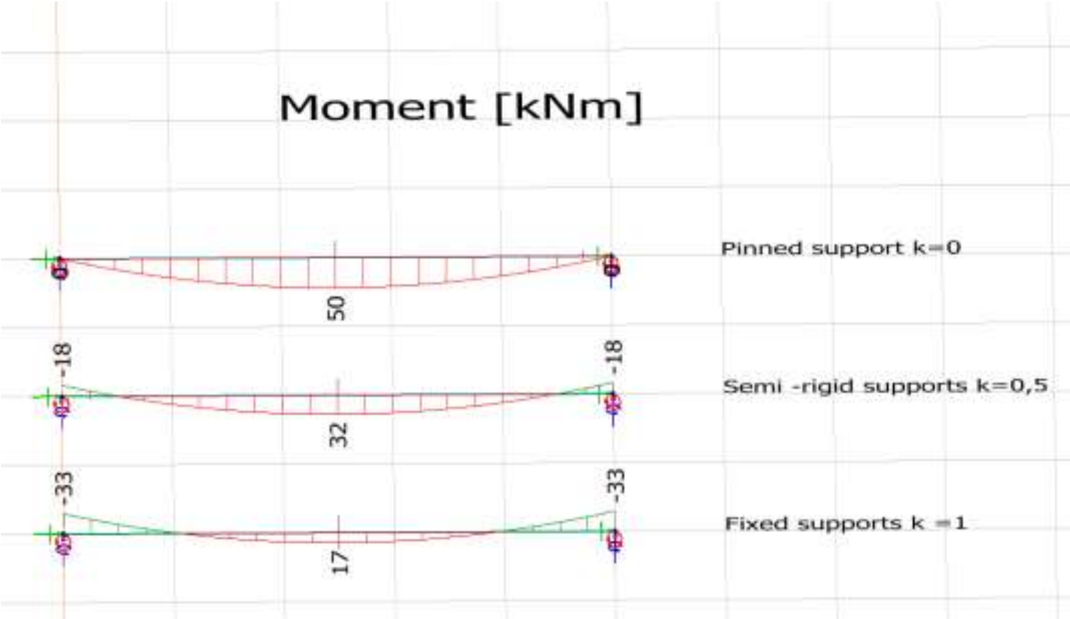


Figure 4.25 Moment diagram from FEM analysis with fixed, pinned, and semi-rigid supports.

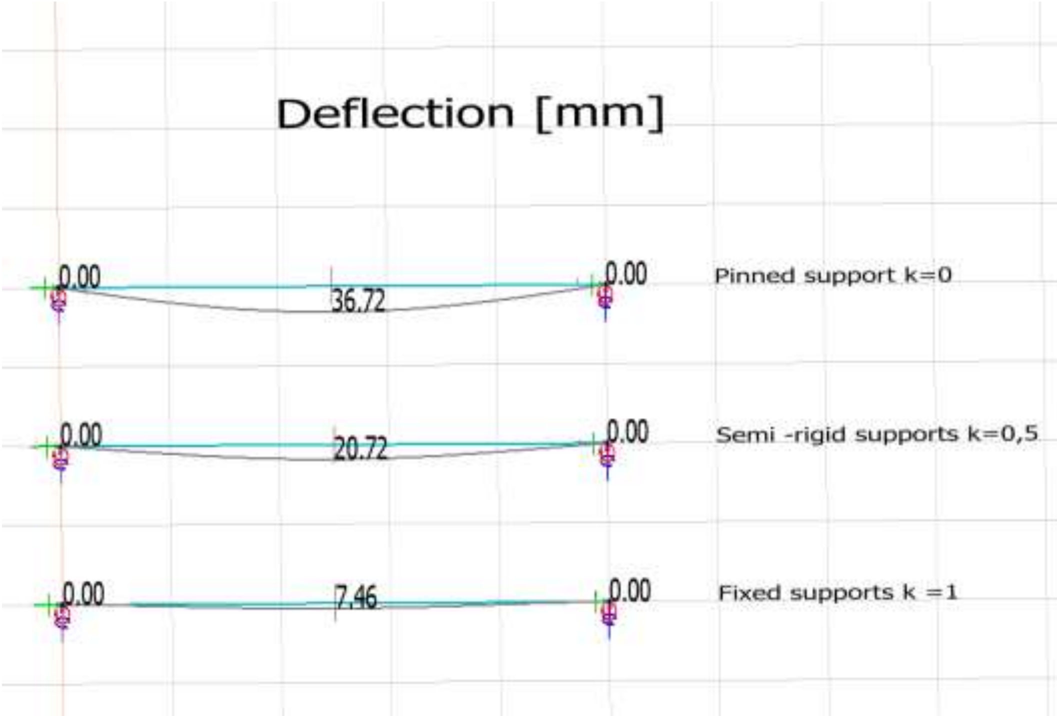


Figure 4.26 Deflection from FEM analysis with fixed, pinned, and semi-rigid supports.

- Semi-rigid transversal stiffness:

For a semi-rigid transversal stiffness with k_a and following the previous process for the same beam.

$$\omega(x=0) = 0 = -\frac{qL}{2k_a} \Rightarrow C_2 = -\frac{qL}{2k_a} \tag{59}$$

$$\delta = -\frac{5qL^4}{384EI} + k_a\frac{4qL^4}{384EI} - \frac{qL}{2k_a} \tag{60}$$

k_a - *Spring constant, translational stiffness* [N/m].

Also, the transversal stiffness in the support with value k_a has an influence on the deformation of the beam.

5 Investigation of connection types

Chapter 5 will assess splice joints firstly presented in Section 3.8 in terms of strength and stiffness. Each kind of connection is designed to withstand the cross-section capacity of a fictitious GL30c beam. The study aims to analyse the connections working in tensile stresses, bending moment, and shear forces. Members in compression are not included since the stress is mainly carried by the timber's contact surface. The pre-investigation represents the first design approach to the case study connections, the examples provide insight into the advantages and disadvantages of different settings. The main goal is to support decision making by narrowing the choices of connections types during the design process. The connections were calculated analytically using Excel spreadsheets created by the authors, which are available in the Digital Annex A. The design steps are demonstrated in Annex B.

The comparison between each proposal focusses mainly on the capacity and stiffness, and on the connection geometry and assembly feasibility. Fire resistance, weather exposure, and neat appearance are also factors considered but in a second level of importance. Besides the evaluation, tables, and information from other sources are presented to deliver broader recommendations to the design process.

5.1 Method of investigation

The investigation is carried out and the calculations are performed to a *glulam* beam-type GL30c with cross-section dimensions of 400x250 mm, demonstrated in Figure 5.1. Service class 2 is chosen.

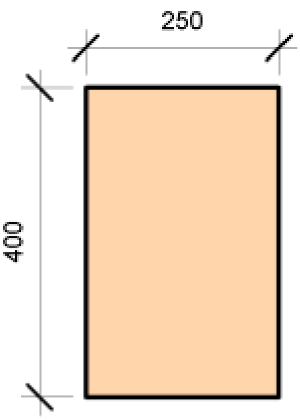


Figure 5.1 Analysed cross-section.

The design capacity of the section is calculated according to EC5 2.4.1.

$$X_d = k_{mod} \frac{X_k}{\gamma_M} \tag{61}$$

Where,

X_k *- Strength characteristic value of a strength property.*

$\gamma_M = 1.25$ *- Partial factor for a material property. Glued laminated timber, EC5 Table 2.3.*

$k_{mod} = 0.8$ - *Modification factor considering the effect of the duration of load and moisture content, EC5 Table 3.1 and for a medium load and service class 2.*

The studied connections are chosen based on simplicity, variety of components, and applicability to free-form structures with large glulam timber sections. The splice joints selected from the tables presented in Section 3.8 provide a satisfactory structural behaviour understanding of connection types, taking into account that the calculation procedure effort is lower when compared to other joint types. Additionally, good insight on failure modes is also obtained. The study provides good knowledge that can be applied in other joints conditions, i.e. beam-to-beam, column-to-beam, and knee joints.

Some connections presented in the tables from Section 3.8 were excluded from the analysis. Punched metal plates do not withstand large forces; thus, it is not suitable for large-sized members. In like manner, glued and carpentry joints are also not studied. Despite their large stiffness and capacity, limitation in section sizes and assembling in connections with several members make them also not a preferable choice (Batchelar & Mcintosh, 2012).

5.2 Connections in tension

The section capacity was calculated according to EC5 2.4.1 by calculating the design parallel tensile strength of the beam ($f_{t.0.d}$).

$$f_{t.0.d} = \frac{f_{t.0.k} \cdot k_{mod} \cdot k_{h.tension}}{\gamma_{m.GL}} \tag{62}$$

$$F_{GL30c.t.0.Rd} = f_{t.0.d} \cdot A_{cross\ section} \tag{63}$$

$$F_{GL30c.t.0.Rd} = 1372.80 \text{ kN}$$

Where:

$F_{GL30c.t.0.Rd}$ - *Section tensile resistance parallel to the grain.*

$f_{t.0.k}$ - *Characteristic tensile strength parallel to the grain.*

k_{mod} - *Modification factor for the duration of load and moisture content.*

$k_{h.tension}$ - *Depth factor (EC5 3.3(3)).*

$\gamma_{m.GL}$ - *Partial factor for glued laminated timber.*

5.2.1 Glued-in rods

The connection, as shown in Figure 5.2, is designed based on the equations given in "Limträhandbok del 3" page 74 (Svenskt trä, 2016).

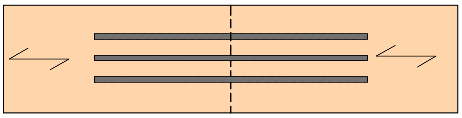


Figure 5.2 Glued-in rods in tension.

Rods specification:
Diameter: 10 mm; 14 mm; 16 mm; 20 mm.
Rods strength 4.8-5.8- 6.8- 8.8

The following steps have been followed during the design and are explained in Annex A.

- Design capacity of rods.
- Brittle failure - Net tension capacity of the timber cross-section $F_{t.net.Rk}$.
- Brittle failure – Single rod-end effect $F_{t.Aef.Rd}$.
- Stiffness K_{total}.
- Results.
- Conclusion

5.2.1.1 Results

The connection design is limited by:

- The minimum glued length l_i is $\max(0.5d^2, 10d)$ (CEN et al., 2019), while maximum l_i cannot be more than 1 meter.
- Axial tensile capacity of the rod.
- Timber tensile capacity parallel to the grain, in the net tension area, and at each rod end.
- Rods spacing to avoid splitting at the edges.

By testing different rods' strength with different diameters, two behaviors for rods were recognized pull-out failure in timber and tensile rods failure. The first is brittle and the second is ductile.

For high strength steel 8.8-6.8, the pull-out failure was the governing to decide the joint capacity. Figure 5.3 shows that 8,8 and 6,8 have the same joint capacity with different diameters and it depends on the pull-out capacity which is not related to the rods class but by the shear capacity of the timber as in Equation (14). All joint hasn't reached the section capacity $F_{t.t.Rd} = 1372.80$ kN and the maximum design capacity was achieved by d=10 mm which is $F_{t.Rd} = 1116.68$kN.

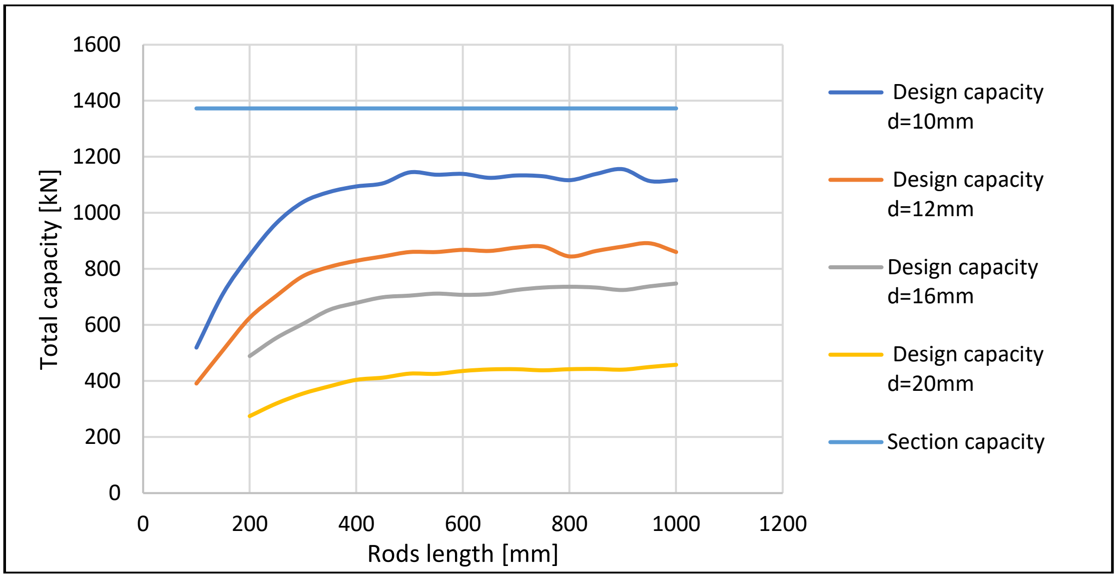


Figure 5.3 GIR joints capacity with 8.8 and 6.8 rods class.

The capacity between 800 mm and 1000 mm with d=12 mm and d=100 mm is not developed in a clear way. According to the analytical calculation, the reason for that is k_1 *- Reduction factor for shear strength as a function of the glued length (li) which is extracted from a graph presented in Table 13.24. (Svenskt trä, 2016).*

When using lower steel strengths, steel strengths 4.8 and 5.8 joints show a ductile behavior but with less capacity compared to higher rods class. Figure 5.4 and Figure 5.5 describe how rod class 5.8 with d=10 mm, and 4.8 with d=10 mm and d=12 mm, show a ductile behavior after a certain length.

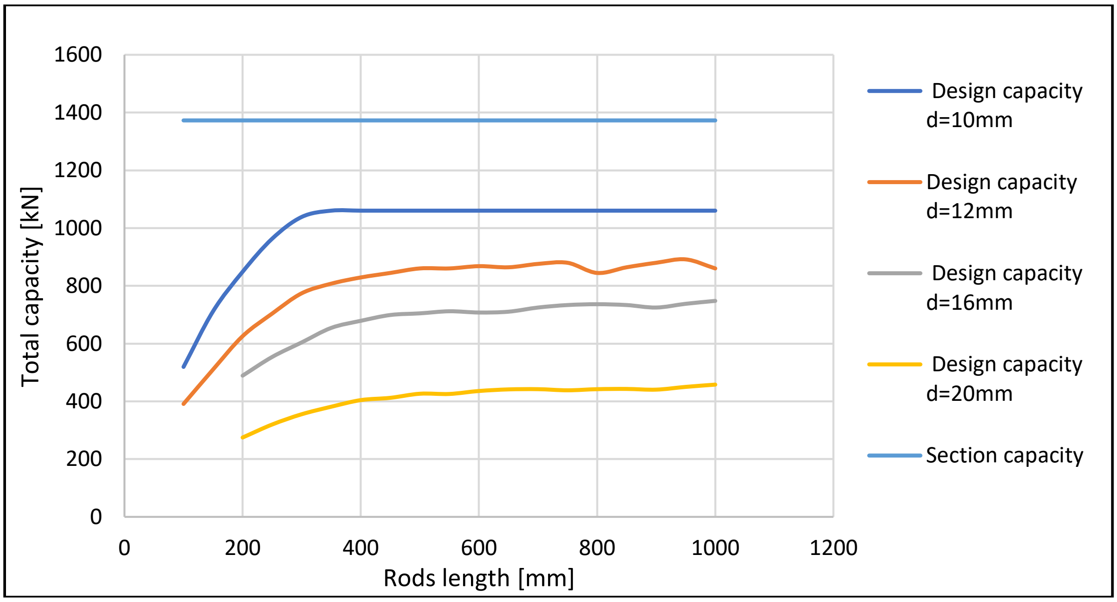


Figure 5.4 GIR joints capacity with 5.8 rods class.

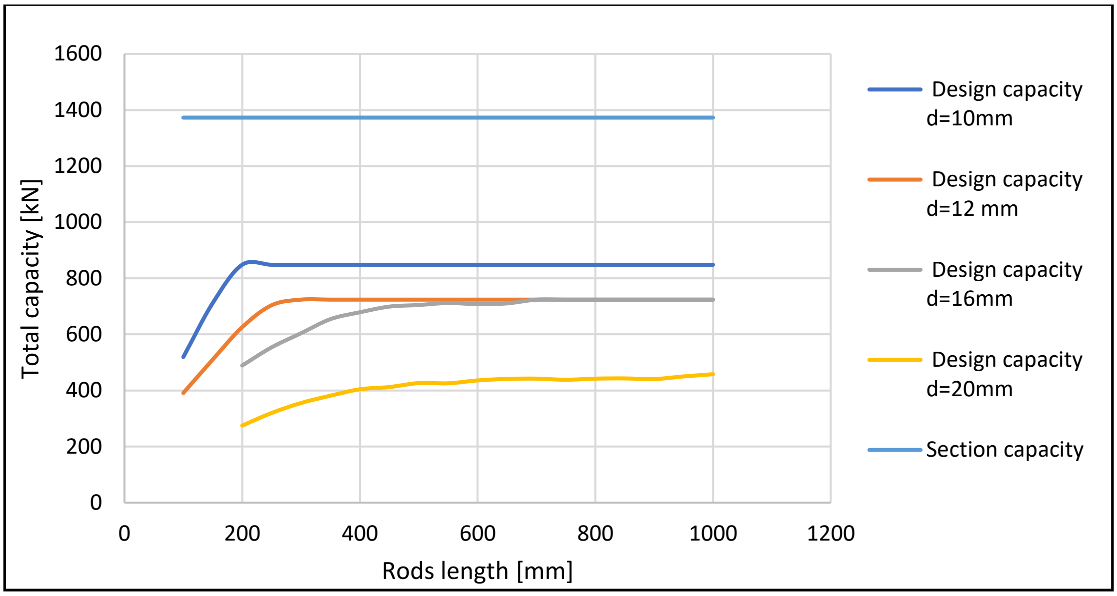


Figure 5.5 GIR joints capacity with 4.8 rods class.

Figure 5.6 illustrates how rods with d= 10 mm fails with different lengths and different strength classes.

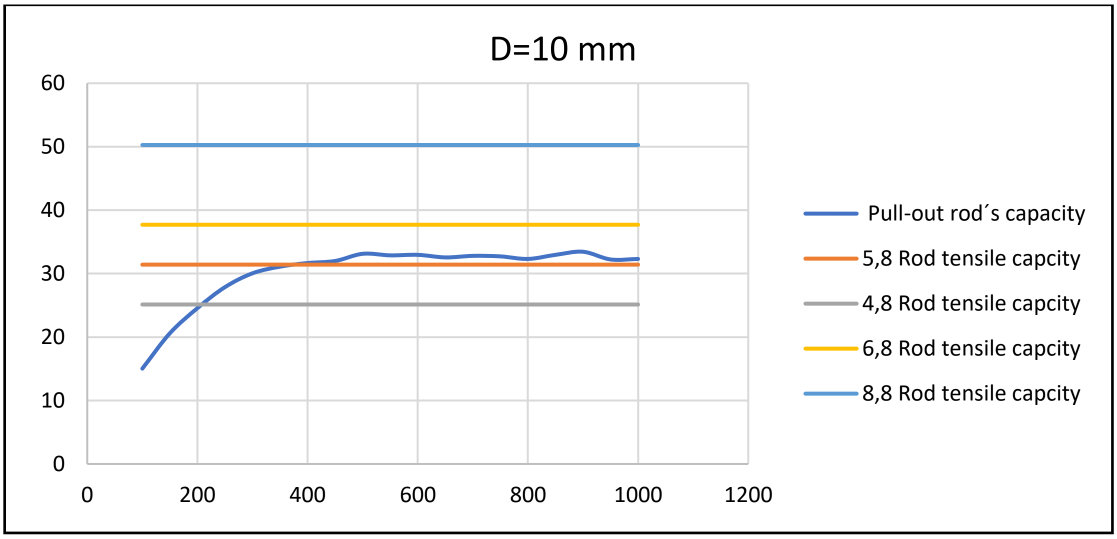


Figure 5.6 Rod d=10 mm with the axial capacity comparison.

Joints presenting ductile behaviour, d=10 mm (5.8), d=12 (4.8), and d=16(4.8) are presented with design values. Table 5.1 presents the maximum carrying capacity of the joints. The section's capacity of $F_{t.t.Rd} = 1372.80$ kN could not be reached by any rod diameter or length.

Table 5.1 Glued-in rods connection design on tensile force.

Diameter [mm]	Glued length [mm]	Rod number	Capacity [kN]	Stiffness [kN/m]	Net timber tension capacity utilization [%]
10 (5.8)	500	54	1117	121522	80
12 (4.8)	500	32	724	99986	54
16 (4.8)	750	18	724	94395	54

Comparison between joints with different diameter: using smaller diameters with a larger number of rods give more capacity caused by the enhanced load distribution and the reduction of stress concentration in timber based on experimental results (Xu et al., 2012).

The stiffness decreases with the increase of the diameter, as shown in Table 5.1. Smaller diameters allow a higher number of rods over the cross-sectional area, thus the stiffness, which is quantity dependent, also increases. This can be visualized in equation (25).

5.2.1.2 Conclusion

GIR's joints in tension show adequate capacity comparing with the reasonable size of the joints related to the rods glued length. They also provide considerable stiffness.

Joints with small diameters provide higher stiffness due to the increased number of rods and show a better stress distribution. It reduces the possibility of splitting at the edges and gives more capacity. Moreover, due to κ_1, the reduction factor to length penetration, the increase in l_i, the penetration length, does not necessarily mean an enhance in the capacity. The number of rods must be considered due to the feasibility and the ease of construction, more rods mean higher detailing level and difficulty in assembly. By increasing the cross-sectional area, it is more likely that a balance between the timber net area and rods diameter can be found.

In general, glued-in rods capacity is limited by the tensile capacity of the rods after a certain length and the increase in length does not result in an increase in the capacity. It is also preferable not to be using high tensile strength to guarantee a ductile failure.

5.2.2 External metal plate

The connection, shown in Figure 5.7. The design calculation is based on EC5 (CEN, 2004) and the stiffness calculation based on "Timber Structures: Rotational Stiffness of Carpentry Joints" (Descamps et al., 2006). Thick and thin plates were verified.

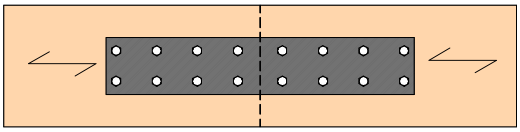


Figure 5.7 External metal plate in tension.

Bolt specification:
Diameter: 6 mm to 30 mm.
Bolt class: 4.80.
$f_{u.b}$ = 400 MPa.
$f_{y.b}$ = 320 MPa.
The following steps have been followed during the design and are explained in Annex A.

- Design capacity of the bolted plate $F_{t.Rd}$.
- Tensile capacity of the plate $F_{t.plate.Rd}$.
- Brittle failure – Block shear $F_{bs.Rd}$.
- Stiffness.
- Results.
- Conclusion.

5.2.2.1 Results

The connection design is limited by:

- Bolt shear capacity.
- Block shear of the section.
- Tensile capacity of the metal plate cross-section.

The connection is designed to reach the desired section design capacity. The metal plates extensions are considerably long, making it unpracticable in the construction site. Since the height of the metal plate is limited to the member's depth, 400 mm, a rise in the number of bolts is only possible by enlarging the plate in its length. Also, the number of bolts, especially in connections with a smaller diameter, is very high and is not an efficient way of transferring the load.

The metal plates are designed based on the applied tensile load with 8 mm. Table 5.2 presents the results.

Table 5.2 External metal plates connection design tensile capacity H_{plate} = 400 mm, t_{plate} = 8 mm.

	Diameter / Plate Size / Number of fasteners			Capacity	Block shear	Tension capacity	Stiffness
	d [mm]	L_p [mm]	n_{bolts} [-]	$F_{t.Rd}$ [kN]	Utilization [%]		K_{total} [kN/m]
Thick plate	6	975	448	1382	112	95	5317
	8	1000	264	1403	117	95	6769
Inter-polated	10	1150	180	1428	133	97	6978
	12	1200	126	1393	147	94	7237
Thin plate	16	1350	75	1435	174	95	8271
	20	1400	48	1414	180	93	10338
	24	1700	36	1479	157	95	9304
	30	2550	30	1451	100	90	6203

Block shear is determinant to the connection resistance and results in failure before the connection can reach the design capacity of the section. It is crucial to be aware if the design capacity along the fracture perimeter of the fasteners' area is resultant from the timber parallel tensile strength or from the timber shear strength.

In connections with shorter plate lengths the block shear capacity is led by the parallel tensile strength. Connections with longer plates the block shear capacity are led by the timber shear strength. Also, the spacing between fasteners in both directions influences the determination of shear and tension areas around the fracture perimeter. This pattern is clearly seen in Figure 5.8 together with analysing equation (4) in Section 4.2.2. Furthermore, Table 5.6 details the number of rows and fasteners in a row, and the required spacing according to EC5.

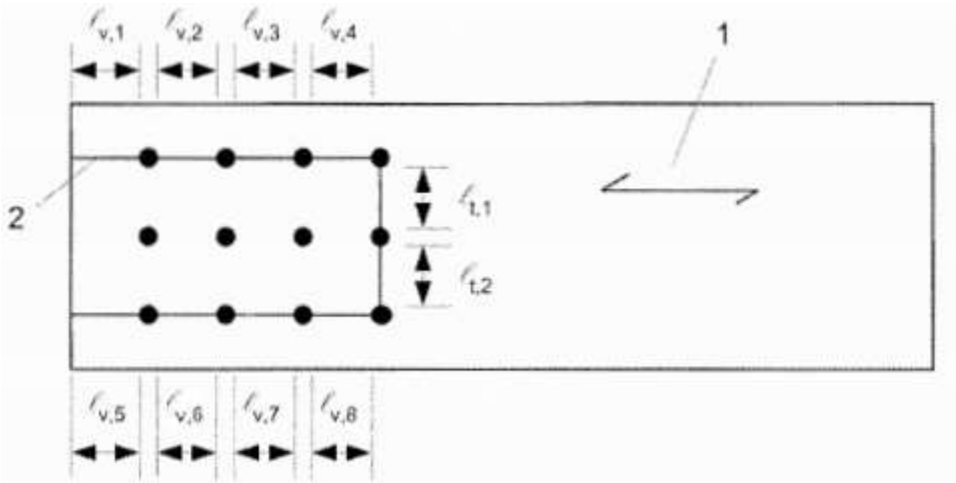


Figure 5.8 Fracture perimeter, where $L_{net.t} = \sum_i \ell_{t.i}$ and $L_{net.v} = \sum_i \ell_{v.i}$.
1 Grain direction 2 Fracture line.

Table 5.3 Influence of fastener direction to the governing block shear failure condition in external metal plate connections.

D [mm]	L_p [mm]	$n_{per.row}$ [-]	n_{rows} [-]	a_1 [mm]	a_2 [mm]	$F_{bs.Rd}$ (shear) [kN]	$F_{bs.Rd}$ (tension) [kN]
6	975	28	16	30	24	566	**1235**
8	1000	22	12	40	32	583	**1198**
10	1150	20	9	50	40	650	**1076**
12	1200	18	7	60	48	696	**954**
16	1350	15	5	80	64	777	**823**
20	1400	12	4	100	80	**784**	748
24	1700	12	3	120	96	**940**	561
30	2550	15	2	150	120	**1458**	280

The relation between the bolt's diameter to the number of fasteners and plate length can be better visualized by the graphs in Figure 5.9.

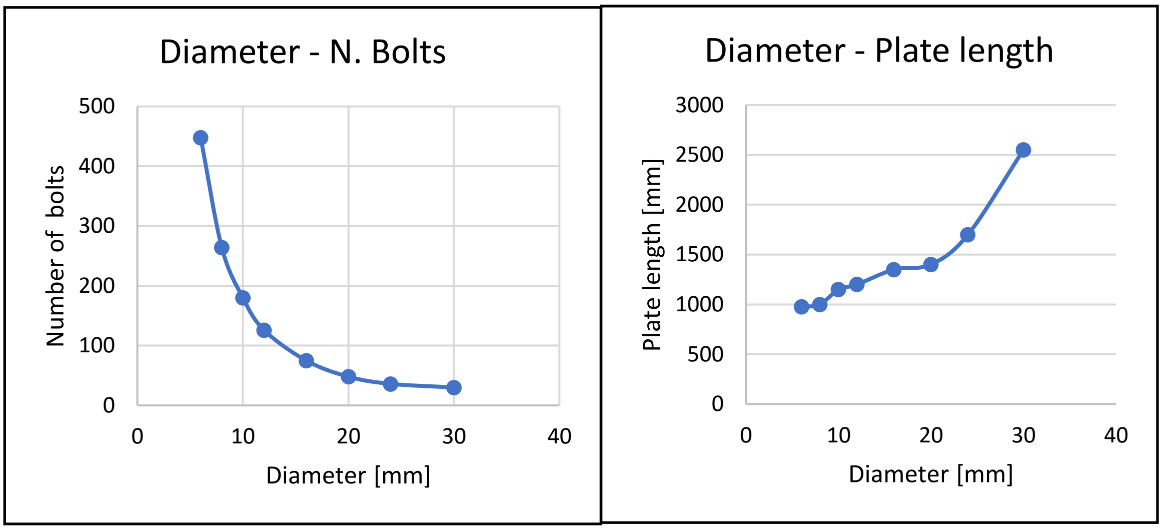


Figure 5.9 Bolt diameter to a) number of dowels b) plate length.

By increasing the diameter, the spacing between the bolts, according to EC5 8.5.1.1(3) Table 8.4, also rises to prevent splitting caused by the row effect. This explains why connections with larger diameters have a fewer number of dowels, despite the increasing length. Moreover, small diameters have lower embedding strength and failure in dowels will be more probable.

It has been observed that the single shear capacity of the bolts is governed by the failure of the bolts as shown in Table 5.4. This, of course, is preferable due to ductility, but the difference between (l/j) and (m/k) is large in magnitude and this explains the weakness of the connection and the need for extended plates.

The bolt class is a decisive factor to this concern, since $M_{y.Rk}$ is dependent on the ultimate tensile strength $f_{u.b}$, the value of $F_{v.Rk}$ can be increased. The connections are designed with bolt class 4.80; a stronger bolt class would result in shorter plates.

Table 5.4 Single shear capacity of bolts, the failure mode in brackets is dependent on the thickness of the plate.

Diameter	Plate Size	Single shear capacity – [kN]	
d [mm]	L_p [mm]	$F_{v.Rk}$ (l/j)	$F_{v.Rk}$ (m/k)
6	975	22	3
8	1000	29	6
10	1150	36	8
12	1200	42	12
16	1350	54	19
20	1400	64	28
24	1700	73	40
30	2000	84	51

5.2.2.2 Conclusion

When designing a connection, it is crucial to control the dimensions and number of fasteners considering the feasibility of the construction. Eurocode 5 does not provide a limitation guideline on the design shear plates dimensions; therefore, it is a duty of the designer to provide reasonable sizing.

External plates might be a good solution to smaller members or too low loaded structures. Additionally, by having the plates exposed, the risk of corrosion and damage by fire arises.

5.2.3 Single slotted-in metal plate

The connection, shown in Figure 5.10, is a steel-to-timber dowelled slotted-in metal plate working in double shear. The design and stiffness calculation is based on EC5 (CEN, 2004).

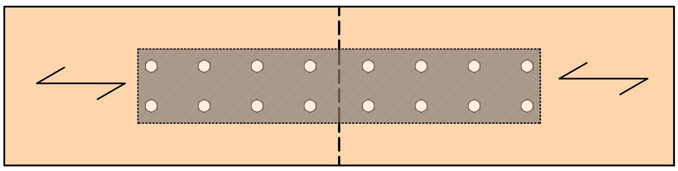


Figure 5.10 Single slotted-in metal plate in tension.

Dowel specification:
Plate thickness:20 mm.
Diameter: 6 mm to 30 mm.
Dowel class: 4.80.
$f_{u.b}$ = 400 MPa.
$f_{y.b}$ = 320 MPa.

The following steps have been followed during the design and are explained in Annex A

- Design capacity of the bolted plate $F_{t.Rd}$.
- Tensile capacity of the plate $F_{t.plate.Rd}$.
- Brittle failure – Block shear $F_{bs.Rd}$.
- Stiffness.
- Results.
- Conclusion.

5.2.3.1 Results

The connection design is limited by:

- Dowel shear capacity.
- Block shear of the section.
- Tensile capacity of the metal plate cross-section.

The connections are designed to reach the desired capacity, these are some remarks that are important to point out, Table 5.5 shows the results.

Plate lengths are very large, especially considering that the lengths showed in Table 5.5 represents half of the plate's final length. It is noticed that the only possible way to achieve the desired resistance is by increasing the length of the plates.

Table 5.5 Slotted-in metal plate connection design tensile capacity H_{plate}= 400 mm, t_{plate}=20 mm.

Diameter / Plate Size / Number of fasteners			Capacity	Block shear				Tension capacity	Stiffness
d [mm]	L_p [mm]	n_{dowels} [-]	$F_{t.Rd}$ [kN]	$F_{bs.Rd}$ [kN] (shear)	$F_{bs.Rd}$[kN] (tension)	Utilization [%]			K_{total} [kN/m]
8	1000	330	1409	536	**930**	152		85	8458
10	1050	216	1390	540	**904**	154		84	10338
12	1070	160	1431	571	**878**	163		86	11630
16	1290	98	1456	669	**757**	192		85	12405
20	1500	65	1412	**778**	602	181		80	11928
24	1900	56	1445	**1004**	516	144		80	10633
30	2600	45	1445	**1341**	387	108		79	9304

Block shear is governing the design, none of the connections could reach the desired design capacity. The concept that is observed in Section 5.2.2 on external metal plates is applicable to slotted-in metal plates regarding the block shear capacity, whether it is resultant from the timber parallel tensile strength or from the shear strength. However, due to the embedded plate, the net cross-sectional area is more reduced. Table 5.6 demonstrates the number of fasteners in each direction and the fasteners spacing.

Table 5.6 Influence of fastener direction to the governing block shear failure condition in slotted-in connections.

d [mm]	L_p [mm]	$n_{per.row}$ [-]	n_{rows} [-]	a_1 [mm]	a_2 [mm]	$F_{bs.Rd}$ (shear) [kN]	$F_{bs.Rd}$ (tension) [kN]
8	1000	22	15	40	24	536	**930**
10	1050	18	12	50	30	540	**904**
12	1070	16	10	60	36	571	**878**
16	1290	14	7	80	48	669	**757**
20	1500	13	5	100	60	**778**	602
24	1900	14	4	120	72	**1004**	516
30	2600	15	3	150	90	**1341**	387

Figure 5.11 shows the comparison between the dowel's capacity and the governing block shear capacity.

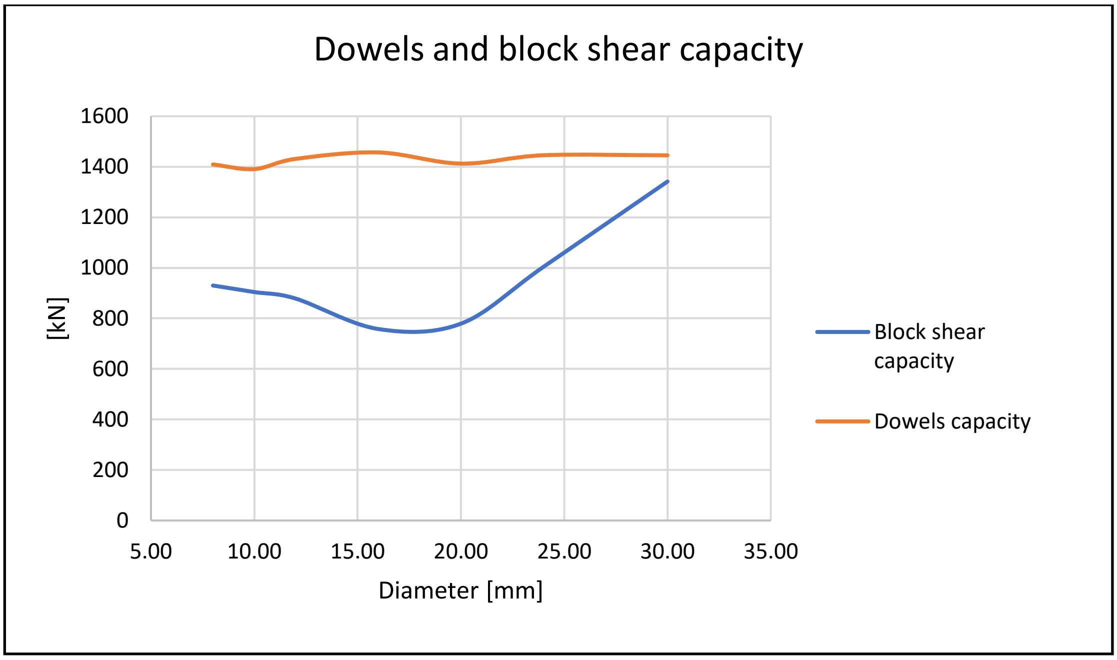


Figure 5.11 Comparison between the dowel's capacity and the block shear capacity.

The tension capacity of the slotted-in plate withstands the applied tensile force, reducing the plate's thickness is a possible optimization, since the utilization is not close to reaching 100%.

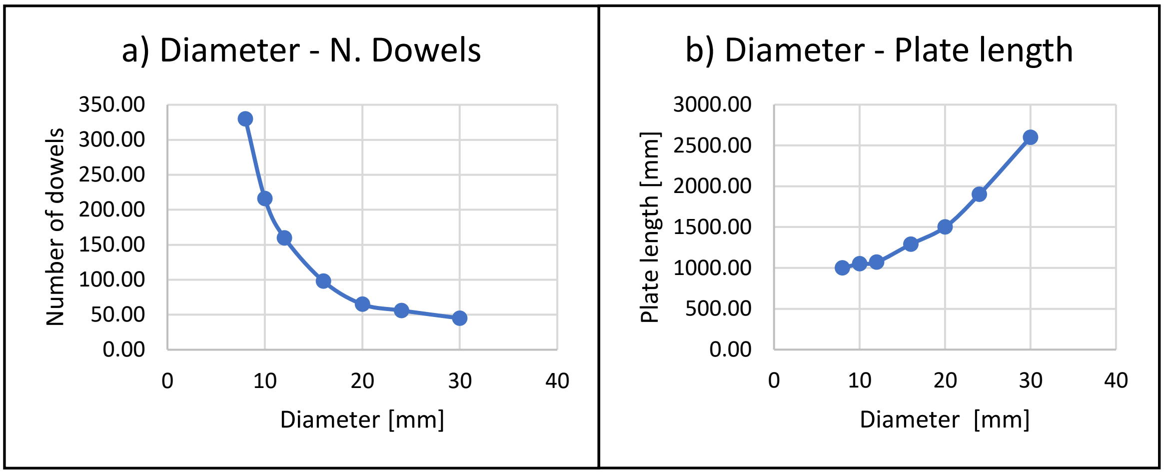


Figure 5.12 Dowel diameter to a) number of dowels b) plate length.

The same trend observed in external metal plates can be seen when analyzing Figure 5.12. The spacing increases according to the diameter to avoid splitting caused by the row effect.

Table 5.7 Single shear capacity of dowels the failure mode in brackets.

Diameter	Plate Size	Single shear capacity – [kN]		
d [mm]	L_p [mm]	$F_{v.Rk}$ (f)	$F_{v.Rk}$ (g)	$F_{v.Rk}$ (h)
8	1000	27	11	6
10	1050	33	14	9
12	1070	38	17	12
16	1290	49	22	19
20	1500	58	27	28
24	1900	67	33	38
30	2600	77	41	54

The single shear capacity of a dowel is governed by failure mode h explained in Section 4.1.2, as shown in Table 5.4. This failure mode occurs due to the formation of two plastic hinges in the dowel. The failure mode is ductile, which is an aim on design, however the ratio between $F_{v.Rk}$ (f) and $F_{v.Rk}$ (h) is too large. Using bolts with higher class is a solution to increase the capacity if the bolt failure is still governing.

5.2.3.2 Conclusion

Slotted-in metal plates are governed by failure due to block shear since the net cross-sectional area is reduced. An increase in the cross-section dimensions will provide a higher net cross-sectional area and net shear area, however, the sectional design capacity will also increase. Also, the number of dowels could be decreased to achieve a well-balanced ratio of dowel capacity, block shear capacity, and a moderately higher capacity of the net cross-section.

Single slotted in metal plate connections are also not suitable for large structures since large-sized connections are required to resist large forces. In a case where the timber sections are large enough more than one steel plate can be inserted, and the shear planes for every dowel will be increased, obviously, one must account for the dimension of the timber net cross-sectional area. More investigation is carried out in Section 5.2.4. An advantage point for this kind of joint that it is hidden and is not exposed to weather and fire risk.

5.2.4 Double slotted-in metal plate

The connection, shown in top view in Figure 5.13, is a double steel-to-timber dowelled slotted-in metal plate. The design calculation is based on the equations given in "Limträhandbok del 3" page 61 (Svenskt trä, 2016).

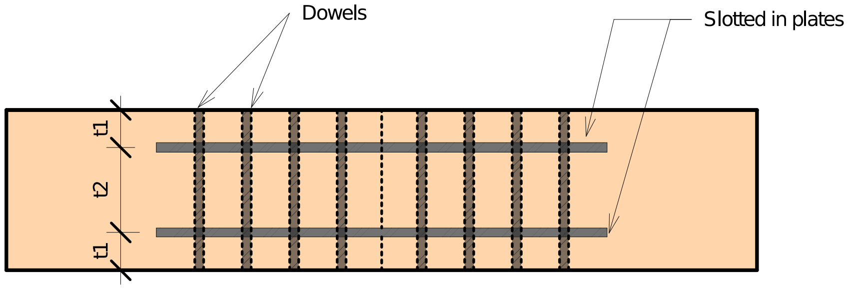


Figure 5.13 Double slotted-in metal plate in tension (Top view).

Dowel specification:
Plate thickness:20 mm.
Diameter: 6 mm to 30 mm.
Dowel class: 4.80.
$f_{u.b} = 400$ MPa.
$f_{y.b} = 320$ MPa.
The following steps have been followed during the design and are explained in Annex A.

- Design capacity of the bolted plate $F_{t.Rd}$.
- Tensile capacity of the plate $F_{t.plate.Rd}$.
- Brittle failure – Block shear $F_{bs.Rd}$.
- Stiffness.
- Results.
- Conclusion.

5.2.4.1 Results

The connection design is limited by:

- Dowel shear capacity.
- Block shear of the lateral and central parts.
- Tensile capacity of the metal plate cross-section.

Double plates slotted-in connections open new possibilities in terms of strength and stiffness due to the addition of shear planes, this can be observed in Table 5.8.

Table 5.8 Double slotted-in metal plate connection design tensile capacity H_{plate} *= 400 mm,* t_{plate} *=12 mm.*

Diameter / Plate Size / Number of fasteners			Capacity	Block shear		Tension capacity	Stiffness
d [mm]	L_p [mm]	n_{dowels} [-]	$F_{t.Rd}$ [kN]	$F_{bs.Rd}$ [kN]	Utilization [%]		K_{total} [kN/m]
8	650	195	1417	1292	110	71	28628
10	670	132	1442	1254	115	73	33834
12	690	90	1377	1191	116	69	41352
16	790	56	1421	1077	132	70	43420
20	980	40	1480	949	156	70	38768
24	1100	28	1421	847	168	66	42534
30	1350	21	1529	938	163	69	39875

The plate lengths are reduced by approximately half in comparison to a single slotted plate. The numbers of dowels are also reduced since each dowel accounts for four shear planes, it directly influences the resistance and the stiffness as it is mentioned. The tension capacity, being dependent on the thickness of the embedded plate, is not nearly reaching 100% of utilization meaning that optimization of the design is possible. The connection with 30 mm diameter dowels does not fulfil the cross-sectional width limitation of: ($B = 250\ mm$).

$$2t_{\mathrm{d.1}} + t_{\mathrm{d.2}} \leq B \tag{64}$$

Block shear is one more time the limiting factor in the design, the manner in which this failure mode influences the behaviour of multi-shear planes connections differs from what presented in the previous analysis of a single slotted-in plate. Therefore, controlling it involves the investigation of additional parameters. The connections are designed with $t_{\mathrm{d.1}}$kept as minimum value, allowing $t_{\mathrm{d.2}}$ to reach a maximum value. The results, presented in Table 5.9, show that the connections fail firstly due to block shear of the lateral block and with diameter increments also the centre block fails. It is important to consider a balance between the timber members' thickness.

Table 5.9 Governing block shear failure in double platted slotted-in connections.

	Laterals			Center		
D [mm]	$F_{bs.Rd}$ (shear)	$F_{bs.Rd}$ (tension)	Utilization [%]	$F_{bs.Rd}$ (shear)	$F_{bs.Rd}$ (tension)	Utilization [%]
8	**410**	129	131	311	**881**	100
10	**421**	149	30	312	**833**	107
12	**405**	168	129	294	**786**	108
16	**438**	184	123	330	**639**	138
20	**472**	178	119	388	**476**	192
24	**463**	179	117	**383**	381	229
30	**515**	168	119	**423**	252	217

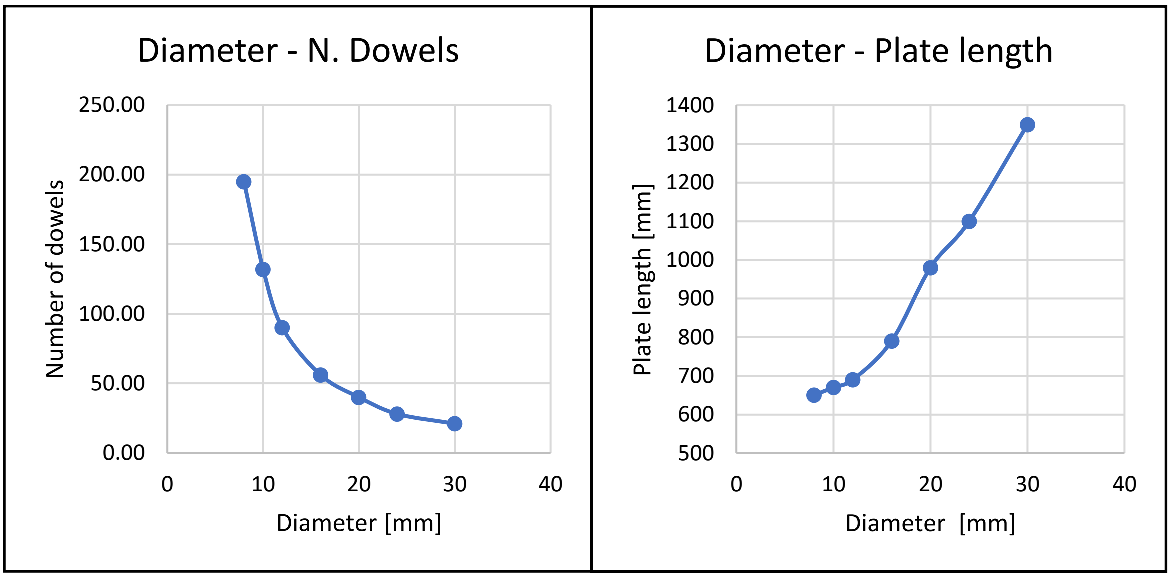


Figure 5.14 Dowel diameter to a) number of dowels b) plate length.

Figure 5.15 shows the comparison between the dowel's capacity and the governing block shear capacity.

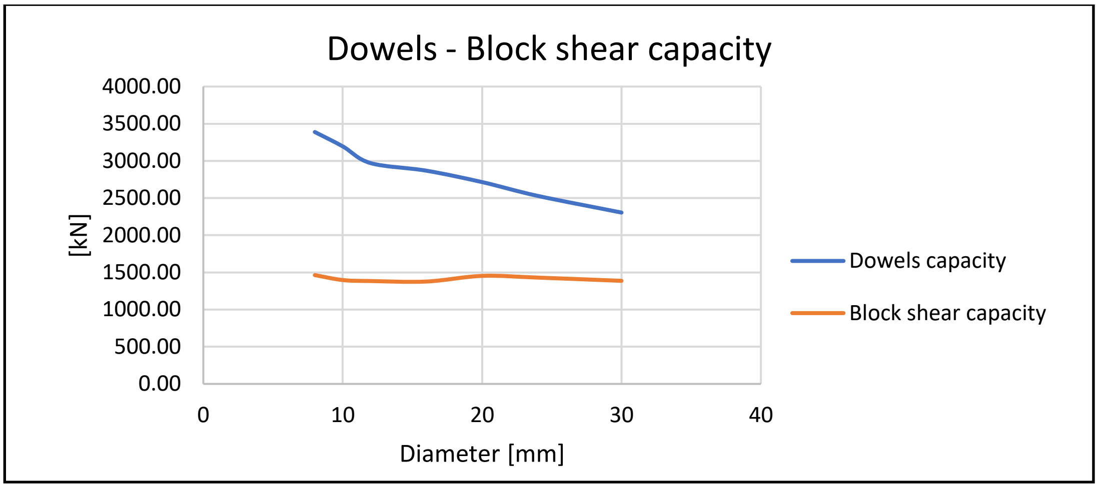


Figure 5.15 Dowels and block shear capacity double slotted-in plate connection.

5.2.4.2 Conclusion

Multi-planes slotted-in connections have enormous advantages in situations where high loads must be transferred. The block shear capacity is more critical than single slotted-in connections and is harder to control when designing since more parameters are included. Multi slotted-in steel plates perform well since they double the capacity and the stiffness. It has the disadvantage that in small sections the failure is mainly governed by block shear which is a brittle failure.

The studied section (400,250) is not as large as the section in the case study. Larger timber sections should have higher block shear capacity.

5.2.5 Comparison

A comparison between the previously designed connections on tension (force parallel to the grain) based on different parameters provides an overview and better understanding for the case study presented in Chapter 6. The comparison is limited by the size of the sections and the type of connections.

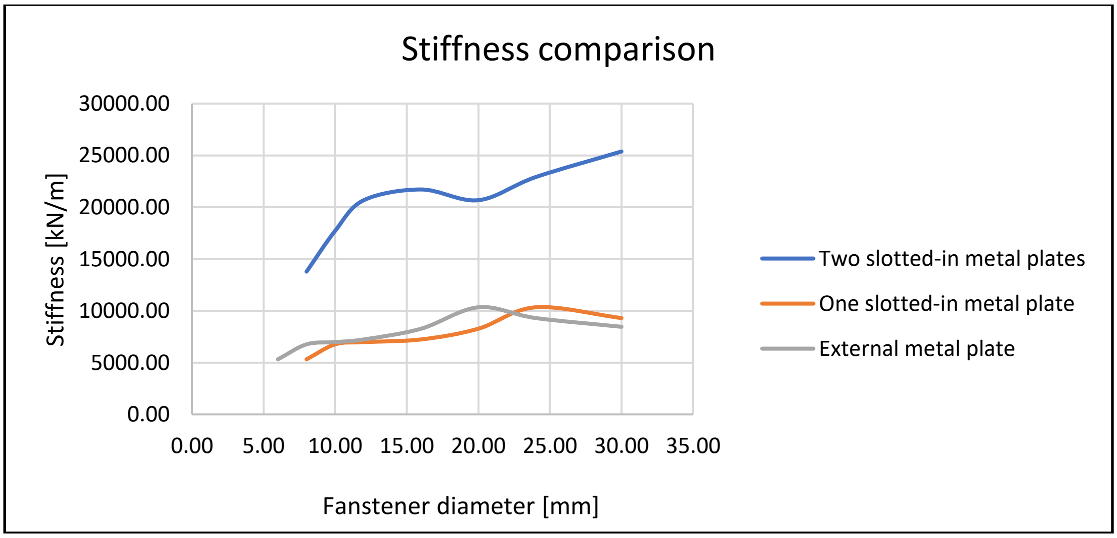


Figure 5.16 Stiffness comparison between splice joints on tension.

From Figure 5.16, the multi- shear planes connections (two in this study) shows advantages on one slotted-in metal plate and external metal plate in the stiffness. The reason for that is the increased number of shear planes per dowel.

The highest stiffness of the previous connections is k_{total} =25375 kN/m which is compared to the lowest stiffness from glued-in rods in tension k_{GIR} =94395kN/m.

$$\frac{k_{max}}{k_{GIR,min}} = 0.23$$

The difference is remarkably high, thus glued in rods has advantages related to the stiffness. A comparison between the connection's capacity is not fair evaluated since the dowel connections are designed to reach the section capacity, but with too large dimensions.

5.3 Connections in shear

The section capacity was calculated according to EC5 6.1.7 by calculating the shear strength of the beam ($f_{v.d}$).

$$f_{v.d} = \frac{f_{v.k} \cdot \ k_{mod}}{\gamma_{m.GL}} \tag{65}$$

The design shear capacity timber section:

$$F_{GL30c.v.Rd} = f_{v.d} \ \cdot \ A_e \cdot 2/3 \tag{66}$$
$$F_{GL3.c.v.Rd} = 100 \text{ [kN]}$$

5.3.1 Slotted-in metal plate

The connection, shown in Figure 5.17, is a steel-to-timber dowelled slotted-in metal plate working in double shear. The design and stiffness calculation is based on EC5 (CEN, 2004)

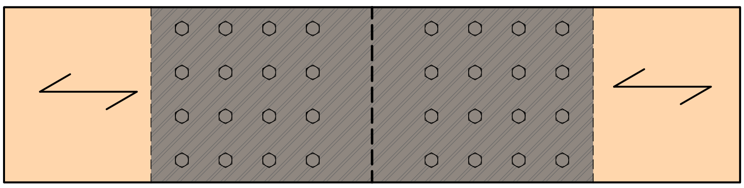


Figure 5.17 Slotted-in metal plate in shear.

Dowel specification:
Diameter: 6 mm to 30 mm.
Dowel class: 4.80.
$f_{u.b}$ = 400 MPa.
$f_{y.b}$ = 320 MPa.

The following steps have been followed during the design and are explained in Annex A.

- Design capacity of the bolted plate $F_{t.Rd}$.
- Check splitting perpendicular to the grain.
- Stiffness.
- Results.
- Conclusion.

5.3.1.1 Results

The connection reaches the section capacity $F_{GL30.c.v.Rd} = 100$ [kN]. See Table 5.10 for the results of connections utilizing different dowels diameters.

The connection design is limited by:

- Dowel shear capacity on force perpendicular to the grain.
- Splitting failure
- Variant steel plate length and fixed steel plate width H =400 mm.
- For simplicity, the moment form eccentricity is ignored.

Table 5.10 Slotted-in metal plate connection design shear capacity H_{plate}= 400 mm, t_{plate}=8 mm.

Diameter [mm]	Plate length [mm]	Number of fasteners	Capacity [kN]	Splitting [%]	Stiffness [kN/m]
8	160	15	99	31	827
10	160	12	117	41	1292
12	170	9	119	47	2067
16	230	7	151	69	3544
20	280	5	145	76	6202
24	340	4	137	80	9304
30	420	3	127	86	15507

The designed connections show a good load-carrying capacity, with practical member sizes and dowel numbers.

5.3.1.2 Conclusion

Both the length of the plate and the number of dowels have reasonable values; therefore, this kind of connection represents a suitable method to transfer shear forces in timber structures. In larger sections, a more effective solution would be to utilize more than one slotted-in metal plate ensuring more capacity and stiffness, in this case splitting effect due to forces perpendicular to the grain must be carefully checked.

5.3.2 External metal plate

The connection, shown in Figure 5.18 is a steel-to-timber bolted external metal plate working in double shear. The design calculation is based on EC5 (CEN, 2004). Thick and thin plates are verified.

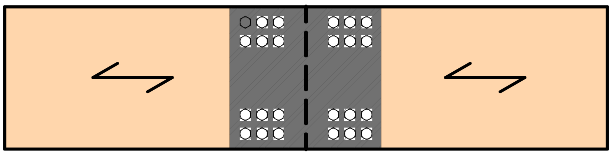


Figure 5.18 External metal plate in shear.

Bolt specification:
Diameter: 6 mm to 30 mm.
Bolt class: 4.80.
$f_{u.b}$ = 400 MPa.
$f_{y.b}$ = 320 MPa.
The following steps have been followed during the design and are explained in Annex A.

- Design capacity of the bolted plate $F_{t.Rd}$.
- Check splitting perpendicular to the grain.
- Stiffness.
- Results.
- Conclusion.

5.3.2.1 Results

The connection reaches the section capacity $F_{GL30.c.v.Rd}$ = 100 [kN]. See Table 5.11 for the results of connections utilizing different dowels diameters.

The connection design is limited by:

- Dowel shear capacity on force perpendicular to the grain.
- Splitting failure.
- Variant steel plate length.
- For simplicity, the moment from eccentricity is ignored.

Table 5.11 External metal plate connection design shear capacity H_{plate} = 300 mm.

Diameter [mm]	Plate length [mm]	Number of fasteners	Capacity [kN]	Splitting [%]	Stiffness [kN/m]
6	200	22	103	19	1691
8	200	16	126	25	3101
10	200	12	143	31	5169
12	220	10	167	33	7443
16	290	6	169	46	16541

20	360	4	147	50	31014
24	440	4	183	53	37217
30	590	2	105	50	10338

5.3.2.2 Conclusion

The designed connections show a good load-carrying capacity, with practical member sizes.

5.3.3 Comparison

In general, the section capacity on shear is less than the capacity on tension. That explains the difference in connections dimension between shear and tension. The length plates in the connections are similar to both of the solutions. The same is observed in the stiffness values. The number of dowels for external metal plates is slightly higher than in slotted-in metal plates.

5.4 Connections in bending

The section capacity was calculated according to EC5 2.4.1 by calculating the design parallel bending strength of the beam ($f_{m.0.d}$).

$$f_{m.0.d} = \frac{f_{m.0.k} \cdot k_{mod} \cdot k_{h.bending}}{\gamma_{m.GL}} \tag{67}$$

$$M_{GL30c.Rd} = f_{m.0.d} \cdot W_{el} \tag{68}$$

$$M_{GL30c.Rd} = 140.80\ kNm$$

Where:

$M_{GL30c.Rd}$ - *Section bending resistance.*
$f_{m.0.k}$ - *Characteristic bending strength parallel to the grain.*
k_{mod} - *Modification factor for the duration of load and moisture content.*
$k_{h.bending}$ - *Depth factor (EC5 3.3(3)).*
$\gamma_{m.GL}$ - *Partial factor for glued laminated timber.*

5.4.1 Glued-in rods

The connection shown in Figure 5.19 has glued-in rods working parallel to grain and the axial capacity of the rods resist the normal forces generated by the moment.

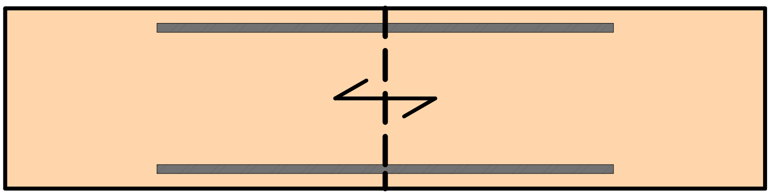


Figure 5.19 Glued-in rods in bending.

Rods specification:
Diameter: 10 mm; 12 mm; 16 mm; 20 mm.
Based on the study made in Section 5.2.1 a rod class 5.8 is used.
$f_{u.b} = 500$ MPa.
$f_{y.b} = 400$ MPa.
The following steps have been followed during the design and are explained in Annex A.

- Design capacity of rods and moment capacity, according to “Limträhandbok del 3” page 74 (Svenskt trä, 2016).
- Brittle failure - Net tension capacity of timber section $F_{t.net.Rk}$.
- Brittle failure – Single rod-end effect $F_{t.Aef.Rd}$.
- Stiffness $K_{r.total}$.
- Results
- Conclusion.

5.4.1.1 Result

The connection does not reach the section capacity of $M_{t.Rd} = 140.8\ kNm$ with any of the diameters because of the limited rods axial capacity shown in Figure 5.4. The results are presented in Table 5.12.

Limitations are:

- One row of rods in tension and compression.
- Rod's axial capacity based on the minimum of tensile and pull-out strength.
- Rods glued length 1 m.
- Only rods contribute to the resistance since a gap between the timber parts is assumed.

Table 5.12 Glued-in rods bending moment designed connections

Diameter [mm]	Moment Capacity [kN· m]	Rotational stiffness [kN· m /rad]	Rods length [mm]	Rods number [-]
10	41	827	500	6
12	37	722	1000	4
16	40	805	1000	3
20	34	705	1000	2

The maximum capacity was achieved by d=10 mm and with a rod length that is less than the other diameters. It has shown a ductile behaviour, after 500 mm of length the capacity is constant and as a result of rods tensile strength.

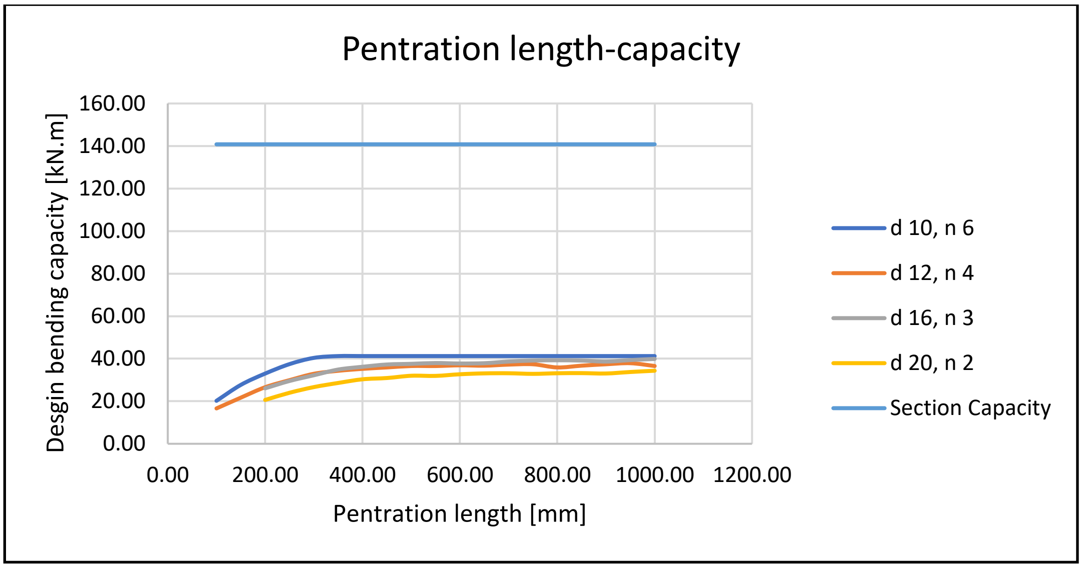


Figure 5.20 GIR moment capacity development.

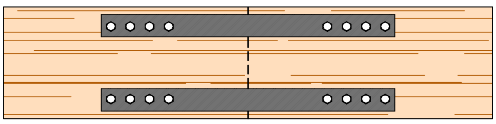
The axial capacity of rods explains the limitation on the bending moment resistance of the connections. The axial capacity after a certain length is constant since the pull-out capacity reaches its ultimate tensile capacity as observed in Figure 5.6 in Section 5.2.1.While other diameters the behaviour was governed by the pull-out capacity.

5.4.1.2 Conclusion

When using GIR to transfer bending moment the capacity is limited by the length of the rods if a high steel strength is used or by the rod's tensile capacity if the used rods have not high tensile strength.

The capacity can be increased by increasing the number of rods and arranging them in several layers. In this case, the capacity of the rod would be increased although the lever arm of the other layer would be smaller. There is not a large difference between different diameters, but d=10 mm is preferred since the behaviour is ductile. The rotational capacity will be later compared with other types of connections.

5.4.2 External metal plate with bolts



Bolt specification:
Diameter: 6 mm to 30 mm.
Bolt class: 4.80.
$f_{u.b}$ = 400 MPa.
$f_{y.b}$ = 320 MPa.

The following steps have been followed during the design and are explained in Annex A.

- Design capacity of the bolted plate and the design bending capacity.
- Top plate on compression and the bottom on tension check.
- Rotational stiffness K_r.
- Result.
- Conclusion

5.4.2.1 Results

The connections reach the section capacity with all diameters, but the length of the metal plates is too long and not reasonable. The results are shown in Table 5.13.

The design limitations are:

- Dowel capacity on force parallel to the grain.
- Maximum designed moment in GIR $M_{max,GIR,Rd}$.
- Block shear capacity.
- Tension and compression capacity of the metal plates.

Table 5.13 External metal plates connection design bending moment capacity t_{plate}= 24 mm.

Diameter / Plate Size / Number of fasteners				Moment Capacity	Block shear	Buckling	Tension	Rotational Stiffness
d [mm]	L_p [mm]	H_{plate} [mm]	n_{bolts} [-]	$F_{.Rd}$ [kN·m]	Utilization [%]			K_{total} [kN·m /rad]
6	4589	36	70	147	19	77	91	4
8	3608	48	43	141	22	60	71	9
10	3030	60	30	142	24	49	59	15
12	2638	72	22	142	27	42	51	24
16	2300	96	14	142	32	34	41	44
20	2079	120	10	140	37	29	35	68
24	1900	144	8	140	42	25	32	87
30	1938	180	6	142	47	22	30	102

For a reasonable evaluation, the maximum designed moment in the GIR's, $M_{max,GIR,Rd} = 41.23\ kNm$, is considered as a reference to compare with. See Table 5.14. Even in connections reaching the GIR's capacity, the plates are too long to be assembled.

Table 5.14 External metal plates connection design bending moment capacity compared to GIR's capacity. t_{plate}= 24 mm.

Diameter / Plate Size / Number of fasteners	Moment Capacity	Block shear	Buck-ling	Tension capacity	Rotational Stiffness

d [mm]	L_p [mm]	$\frac{H_{plate}}{[mm]}$	n_{bolts} [-]	$F_{.Rd}$ [kN·m]	Utilization [%]			K_{total} [kN·m /rad]
6	1300	36	21	42	15	77	52	7
8	1050	48	13	42	20	60	40	16
10	750	60	9	42	25	49	34	28
12	800	72	6	42	29	42	33	38
16	750	96	4	43	37	34	24	81
20	700	120	3	44	44	29	21	121
24	700	144	2	45	50	25	19	152
30	750	180	2	44	57	22	18	187

Due to the extended length of the plates, the block shear is controlled by the shear capacity along the fracture perimeter. The maximum utilization is around 57% on connections with bolt diameters of 30 mm. The 24 mm thick metal plates provide enough capacity on both tension and compression stresses.

The single shear capacity of a bolt is presented in Table 5.15. The difference between the failure modes in the dowel and in the timber member is too high. Increasing the steel quality of the bolts would increase the resistance of each fastener, and therefore the number of fasteners could be reduced.

Table 5.15 Single shear capacity of bolts.

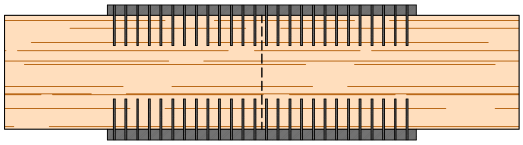
Diameter	Plate Size	Single shear capacity – [kN]	
d [mm]	L_p [mm]	$F_{v.Rk}$ (j/k)	$F_{v.Rk}$ (m/l)
6	2500	22	3
8	1969	29	6
10	1700	36	9
12	1700	42	12
16	1300	54	19
20	1200	64	28
24	1180	73	38
30	1171	84	54

It is recommended to use a large diameter to decrease the number of fasteners parallel to the grain and get less group effect if this size of diameter provides a failure in the fasteners not on the timber.

5.4.2.2 Conclusion

The studied connection is not suitable for large-sized members. To overcome the resulting stresses, very long metal plates are necessary. The design choice of having only one row results in the need of increasing the length of the metal plates.

5.4.3 Top and bottom steel nailed plates



Nails specification:
Diameter smooth nails: 4 mm to 6 mm.
Nail class: 4.80.
L=40 mm
$f_{u.b} = 400$ MPa.
$f_{y.b} = 320$ MPa.

The following steps have been followed during the design and are explained in Annex A.

- Design capacity of the nailed plate and the design bending capacity.
- Top plate on compression and the bottom on tension check.
- Plug shear of top and bottom plate.
- Rotational stiffness K_r.
- Result.
- Conclusion

5.4.3.1 Results

The connections reach the section's capacity of $M_{t.Rd} = 140{,}8\ kNm$ with all diameters. The length of the metal plates and the number of the nails and not reasonable in terms of practicality. The results are presented in Table 5.16. The high number of nails will result in reducing the efficiency to transfer the normal force.

The design limitations are:

- Dowel capacity on force parallel to the grain.
- The width of the metal plates or the joint is limited by the width of the beam 250 mm
- Maximum designed moment in GIR $M_{max,GIR,Rd}$.
- Plug shear capacity.
- Tension and compression capacity of the metal plates.

Table 5.16 Top and bottom nailed plate on bending with section capacity.

Diameter / Plate Size / Number of fasteners			Moment Capacity	Plug shear	Buckling	Tension	Rotational Stiffness
d [mm]	L_p [mm]	n_{nails} [-]	$F_{.Rd}$ [kN·m]	Utilization [%]			K_{total} [kN·m /rad]
4	1970	528	140	9	79	72	66
5	1850	405	139	9	79	72	68

6	2250	329	140	8	84	71	59

As made in the last example 5.4.2, for a reasonable evaluation the maximum designed moment in GIR $M_{max,GIR,Rd} = 77.21\ kNm$ is considered as a reference to compare with. Table 5.17 shows the results.

Table 5.17 Top and bottom nailed plates on bending with GIR capacity.

Diameter / Plate Size / Number of fasteners			Moment Capacity	Plug shear	Buckling	Tension	Rotational Stiffness
d [mm]	L_p [mm]	n_{nails} [-]	$F_{.Rd}$ [kN·m]	Utilization [%]			K_{total} [kN·m /rad]
4	500	132	43	1	24	22	132
5	500	99	42	1	24	22	141
6	600	84	44	1	24	22	116

5.4.3.2 Conclusion

The studied connection is not suitable for large-sized members. To overcome the resulting stresses, very long metal plates are necessary. The design choice of having only one row results in the need of increasing the length of the metal plates. Besides, the length and the high number of nails complicate the construction of this joint.

5.4.4 Comparison

The comparison is based on the GIR moment capacity, Table 5.18 presents the rotational capacity comparison.

Table 5.18 Rotation stiffness comparison for connections working on bending.

Rotational stiffness [kN·m /rad]	GIR	Top and bottom nailed plates	External metal plate
max	827	132	54
min	705	116	7.8

As in the studied connections in tension, the GIR´s has the higher stiffness also on bending. Regarding capacity, the GIR´s also provide limited but adequate capacity with reasonable dimensions not like what observed in other studied types of connections.

Both fasteners- type connections following the studied arrangements are not applicable in large sections where large bending moments are applied. A more practical solution is to use multi slotted-in plates to optimize the design.

The results and conclusion on bending are limited with the section and the simplicity of chosen connections. They will give an insight into these connections in type and geometry, but it will be not considered as a general reference. It will help to get more understanding more about how connections work in bending.

5.5 Conclusion

The focus of this simple study has been on the capacity, stiffness, and the dimensions of timber joints.

- Tension and bending

GIR's combine an adequate resistance to bending and tension and provide the system with high stiffness. Fire resistance and good appearance are added to their benefits also. They fit to use in large timber structures where capacity and stiffness have the same importance.

The slotted- in metal plates are also a possible solution for tension and bending. Generally, the applied bending and tension in a joint is related to the connected members capacity. The number of slotted-in plates could be increased and adapted to the section size. The possibility to get more capacity and stiffness makes them a possible solution in large timber structures i.e. trusses and free-form timber structures. Possibility for brittle failure occurrence can increase with multi-plates and must be avoided.

The studied external metal plates are limited in capacity and stiffness. Combining slotted-in plates and external metal plates is a way of increasing the capacity and the stiffness.

To overcome the large dimensions of these joints, decreasing the space between the dowels and providing reinforcement with self-tapping screws make the design more optimized since self-tapping screws inserted perpendicular to the grain are able to resist stresses perpendicular to the grain.

- Shear

Since the timber section capacity on shear is smaller compared to the tension and the bending capacity, the applied shear force is relatively small. From the studied connections on shear, both slotted-in and external metal plates provide a section capacity with applicable dimensions. For large timber structures, multi slotted-in plates could be an adapted solution.

Table 5.19 presents a ranked evaluation of the proposals. The ranking is done according to the following:

- 1 - Low capacity and stiffness.
- 2 - Enough capacity and low stiffness.
- 3 - Enough capacity and enough stiffness.

Table 5.19 Ranking of the joints.

Ranking	GIR	External bolted metal plates	Slotted-in metal plates with dowels
Tension	3	1	2

Shear	Not studied	2	3
Bending	3	1	2

6 Case study

Chapter 6 presents the design proposal for the timber connections in the roof structure of the project under development by the consulting firm COWI. The geometry of the structure is presented, as well as the cross-sections and materials of the structural members. The structure was modelled by the company COWI in the finite element software "FEM-Design 19 3D Structure" by StruSoft. Through this model the forces and stresses were extracted. The connections between members were calculated analytically using Excel spreadsheets created by the authors, which are available in the Digital Annex B. The design steps are demonstrated in Annex C. The method of choice of the connections was based on the study made in Chapter 5. The analytical calculations were made in accordance with Eurocode 5 and some academic publications that cover non-standardized concepts, such as glued-in rods and multi-planes slotted-in connections.

6.1 "The project"

The project is under development. COWI together with the client have agreed to authorize the use of this project provided that just a part of the building complex is presented by visual, informative, and/or technical means. Therefore, some parts are excluded from the scope of this Master's Thesis project.

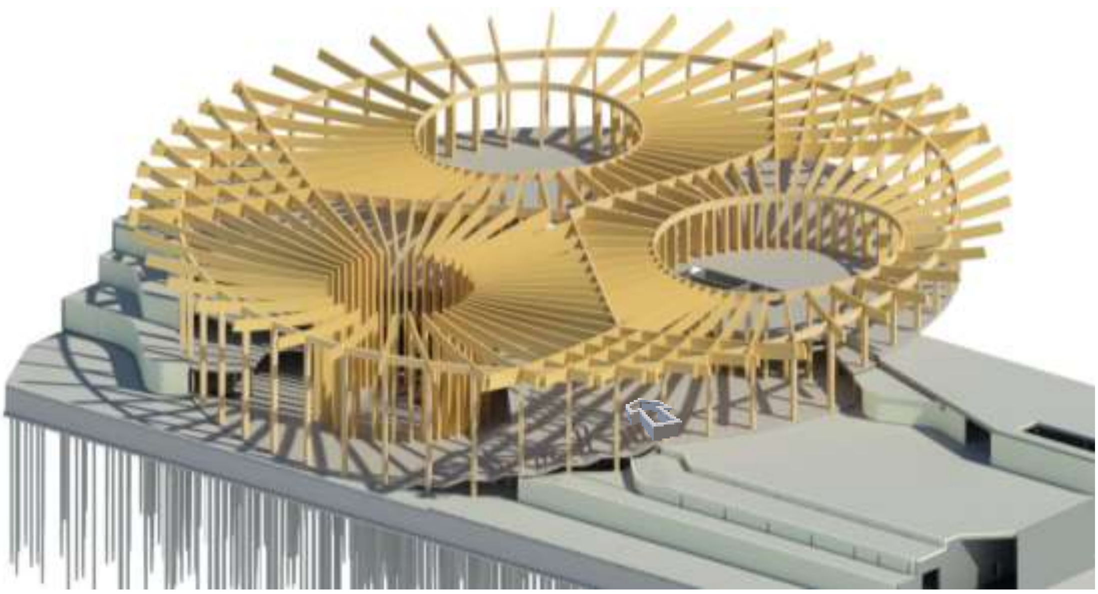


Figure 6.1 3-D model of the building complex, the grey arrow represents the viewpoint from Figure 6.2.

Figure 6.1 shows the proposed designed superstructure. The extensive timber structure supports a roof that covers approximately the entire perimeter of the building. The structure can be defined by 3 inner rings, two of which are similar with larger radius circles. In the perimeter of the structure, a large exterior circled beam generates the overall form of the structure. The beams that originate from the inner circles are supported either by columns located around the perimetrical circle by transversal beams in the internal regions of the building. A total of three transverse beams intersect each other in the center of the building. These are fundamental to the architectural intention of having a free span in the internal area of the building, Figure 6.2 shows the perspective view from the interior area of the building.

Figure 6.2 Interior perspective view.

The regions in which the angled beams point in an outward direction create intersections with beams coming from adjacent rings. The columns have different heights given that the building has floor plans in different elevations.

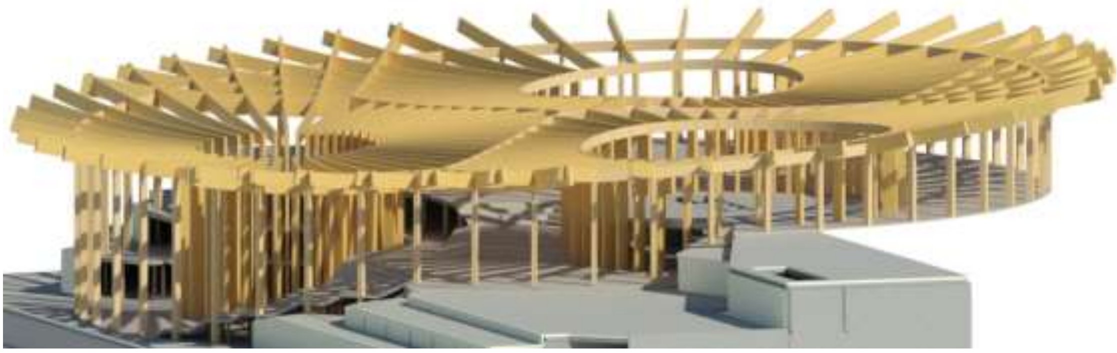


Figure 6.3 Side view of the timber roof structure

From this side view of the structure, illustrated in Figure 6.3, it is possible to observe the different slopes of the roof. Each beam and column have its own geometry, resulting in a complex structure, with different inclinations and angular members. The form-finding process was developed through parametric design using Rhino and Grasshopper and then imported to Revit and Tekla.

6.2 Structural aspects

Figure 6.4 shows the main dimensions of the timber free-form structure containing the diameters of the inner and outer circles, the length of the beams with the larger cross-section (in red), and the length of the transverse beams in the inner region of the building.

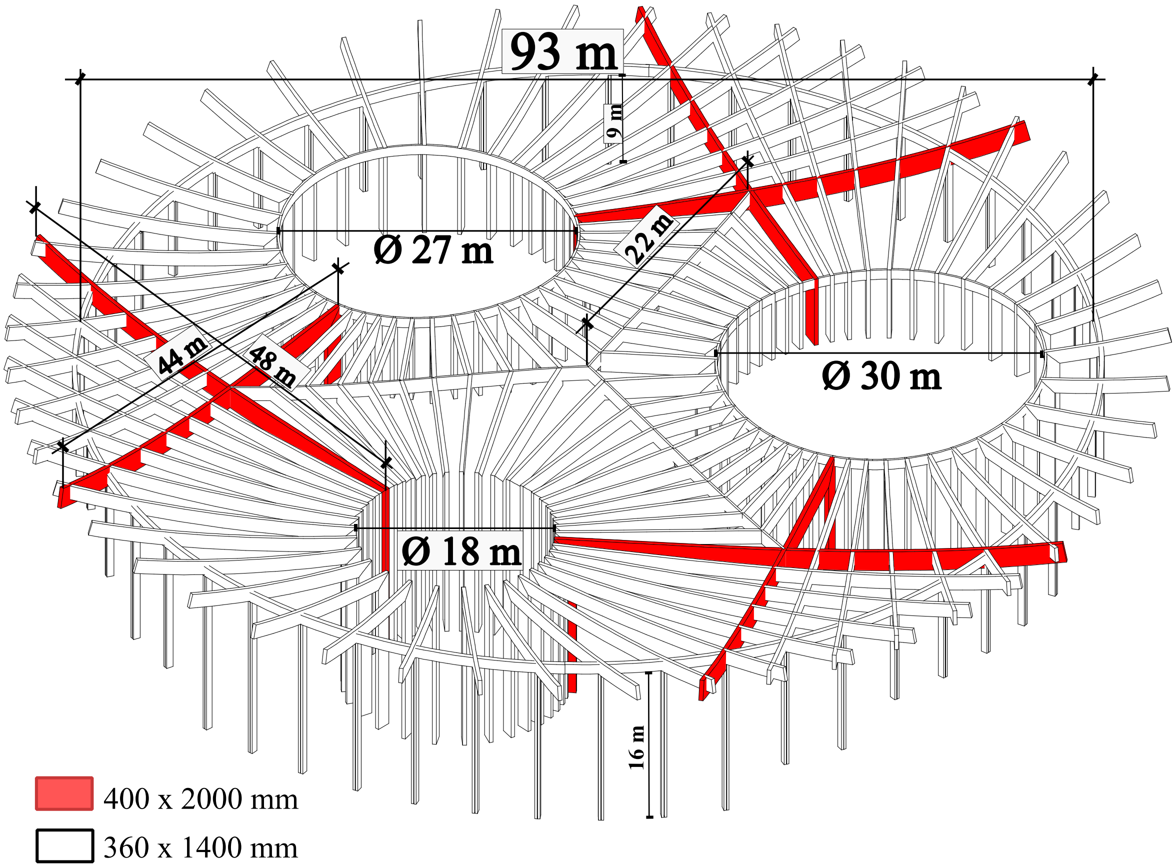


Figure 6.4 Main dimensions and cross-section of the members.

All the beams and columns supporting the 3 rings are made of glulam GL30 C with properties presented in Annex A.

6.2.1 Load cases and load combination

The loads and load duration according to EN 1995 1-1 are presented in Table 6.1 and the structure is assigned to service class 1 according to EC5 (CEN, 2004) Section 2.3.1.3.

Table 6.1 Type of loads acting on the roof structure.

Name	Duration
Beams self-weight	Permanent
Roof self-weight	Permanent
Self-weight of the above buildings	Permanent
Snow loads	Medium-term
Snow accumulation	Medium-term

The load combinations used in the analysis are according to Eurocode 1-1-3 (SIS, 2009b) In ultimate limit state two combinations have been checked, ULS-a and ULS-b according to equation (6.10) from EC1. In the serviceability limit state, the quasi parament load combination was used according to equation (6.10) from EC1. The coefficients are extracted from Table A1.1 and Table A1.2(A) in (SIS,2009a). The coefficient values are shown in Annex A – Material properties.

6.3 Proposals of the design

The structure was analyzed through the software "FEM-Design 19 3D Structure" in order to find areas with the greatest stresses. In this way, the design of the connections can be made for the nodes with higher stresses. The main idea in the design of connections in such a non-uniform structure is to aim to limit the number of design variations to facilitate the construction process. Taking this into consideration, 3 connections have been chosen to be studied and designed, seeking that they serve as the basis for other joints of the structure that differentiate in stresses and angle of intersection of members.

As mentioned, the process of defining the connections to be studied was based on the results provided by the numerical analysis made in FEM-Design. The internal forces in the structure being the effect of load combination in ULS-b, Figure 6.5 shows the bending moments and shear forces.

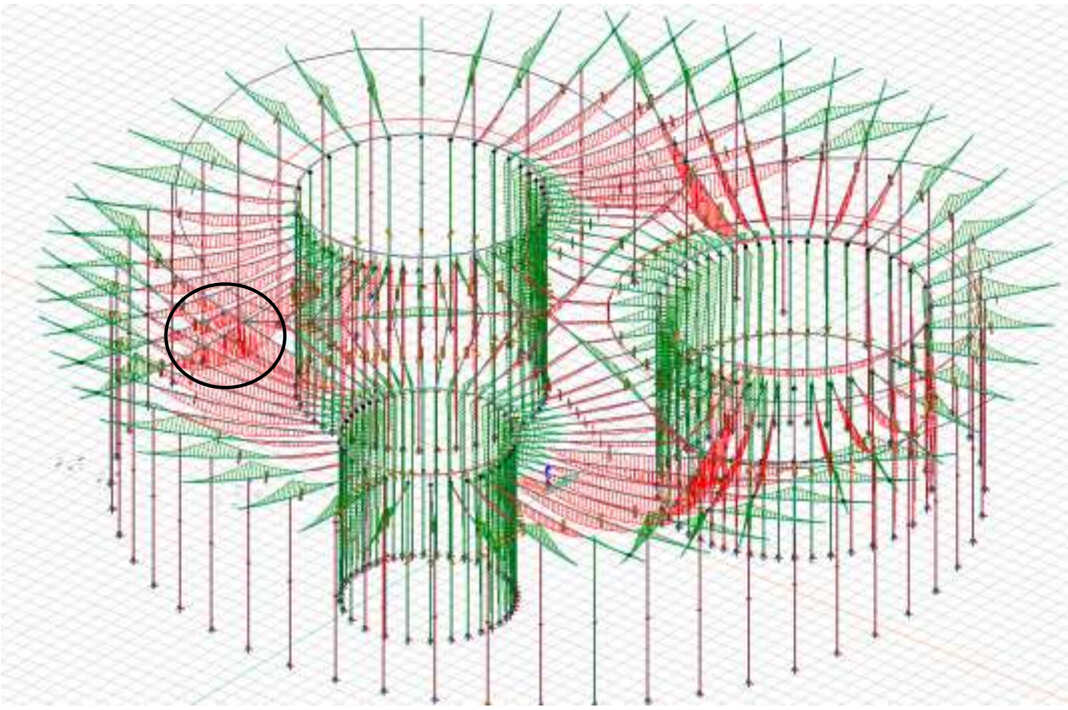


Figure 6.5 Bending moments around the strong axis – ULS-b, area with highest bending moments are highlighted by the black circle.

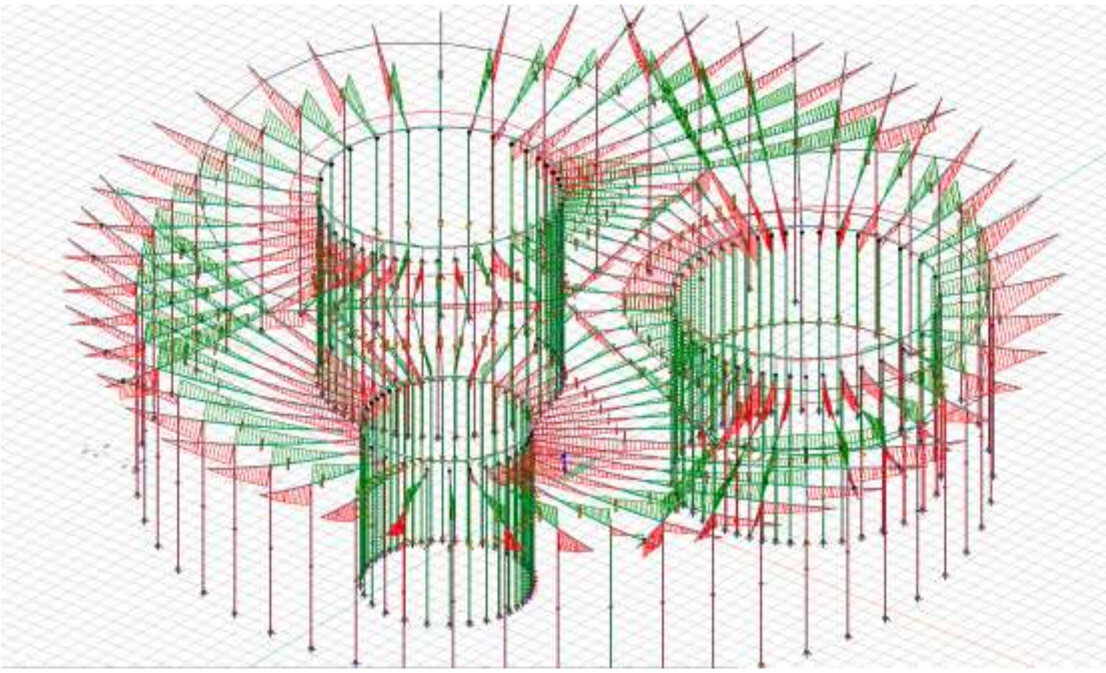


Figure 6.6 Shear forces – ULS-b.

As observed in Chapter 5, the connections studied showed that the proposed designs performed very well when subjected to shear forces, while connections transferring bending moments encounter greater difficulties. Therefore, areas with higher bending moments are more critical, thus leading to the nodes to be studied. Figure 6.5 shows with a circle where the largest bending moments occur. The geometry of the connection, in other words, the number of members that meet in the node is also a matter that influences which connections are more valid to be studied.

The resulting bending moments on the weak axis, the shear forces on the weak axis, and the torsional moments are neglected since these values are very small.

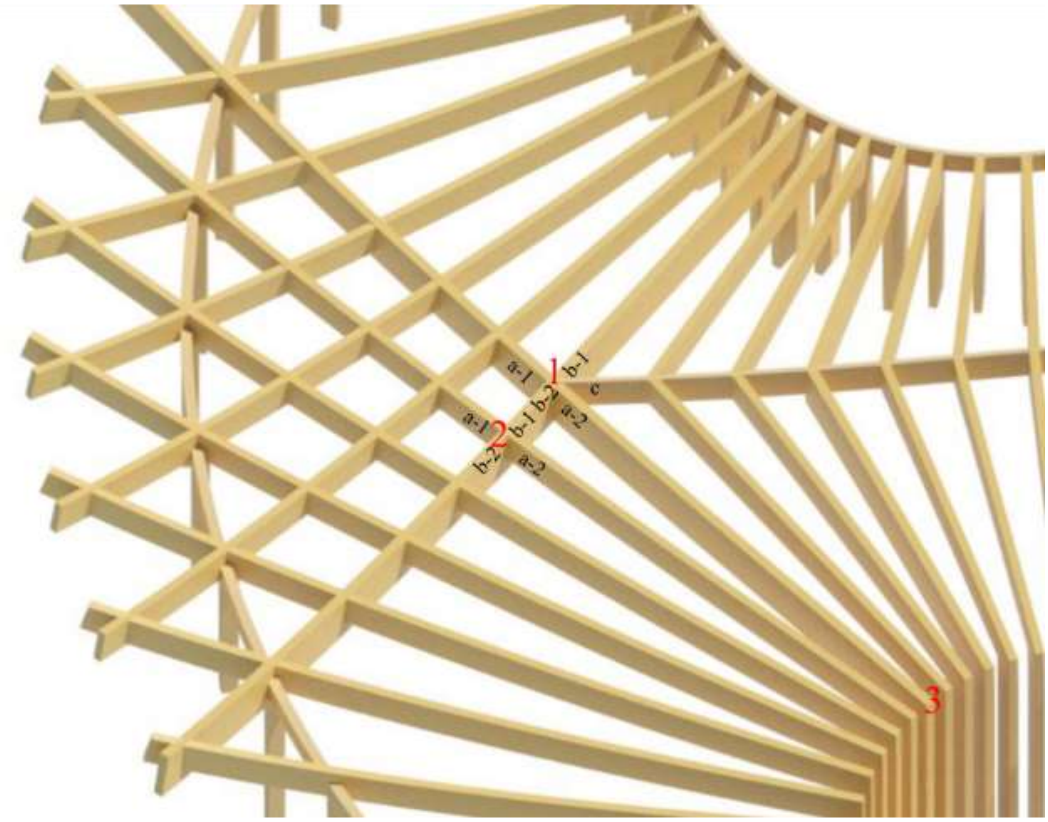


Figure 6.7 The chosen connections to be studied in this chapter.

- Connection 1 is connecting five members. Members A and B behave as continuous beams, while C is not continuous and has a smaller cross-section.
- Connection 2 consists of two intersected members both behaving as continuous beams. Member A has a larger cross-section.
- Connection 3 is a column to beam joint. Two different locations have been chosen to be studied since they have a different beam and column cross-sections. Both locations possess the maximum bending moment for their respective cross-sections.

6.3.1 Global behavior

The first step in the design of the connections is to define the behavior at the global level of the beams to be connected. The beams were defined into different parts according to Figure 6.8. The definition of the behavior of each beam is based on the cross-section of the member and the maximum possible length in transport-related matters. The longest length allowed for cargo transportation in Sweden without the need for special measures is up to 24 meters. Members up to 40 meters in length require special escort vehicles and signs (Borgström et al. 2018).

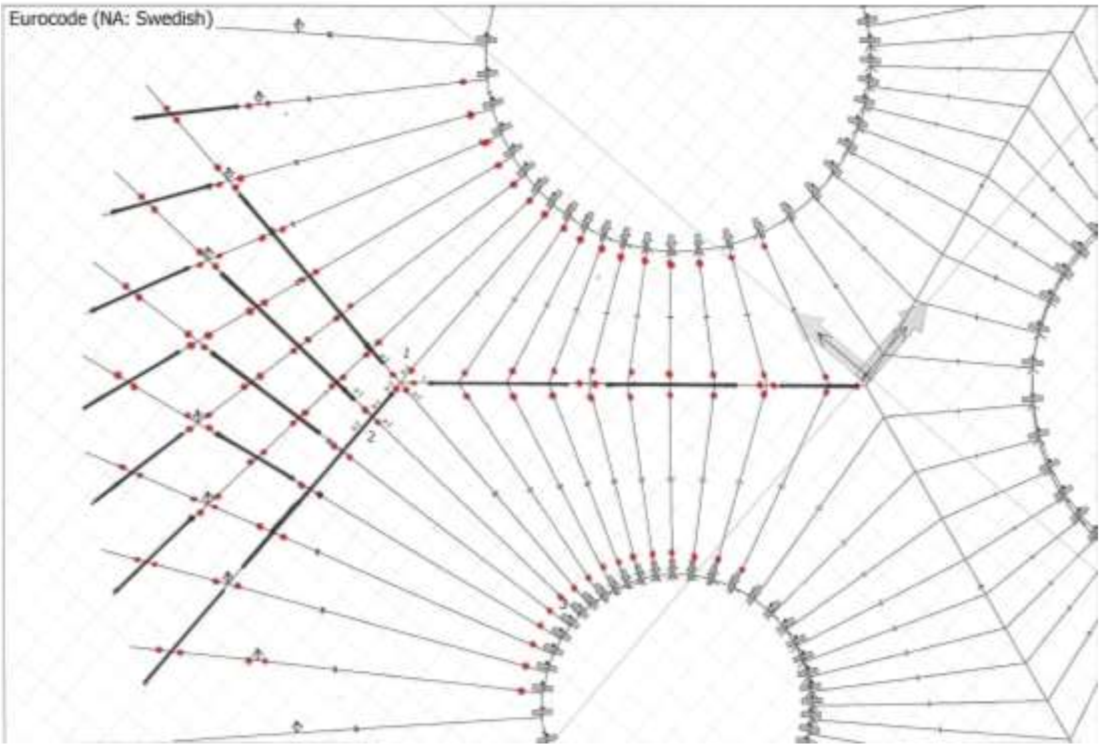


Figure 6.8 Distribution of the connections at the global level. The points represent connections points and the bold lines refer to continuous members without any splice or connections.

Some members in the connection are continuous, as in connection (2) (b-1) and (b-2), and the other intersected members should be spliced into two connections in each side of the continuous beam, as in connection 2 (a-1) and (a-2).

6.4 Connection 1

The connection consists of the intersection of five members in one location, Figure 6.9 shows the geometry of the connection, and Table 6.2 presents the forces in the connection.

Members (a-1) and (a-2) are parts of a continuous beam with section (2000×400) mm.
Members (b-1) and (b-2) are parts of a continuous beam with section (2000×400) mm.
Member (c) is a singular beam with section (1400×360) mm.

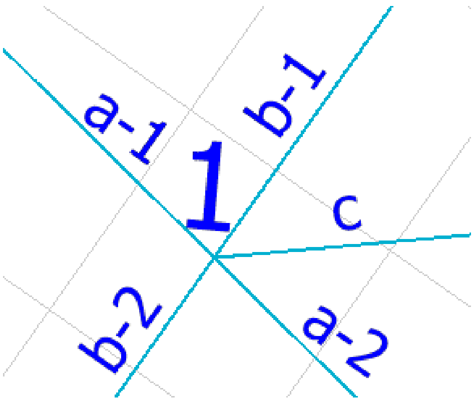


Figure 6.9 Connection 1.

Table 6.2 Forces in connection 1.

	M_y [kNm]	V_z [kN]	N [kN]	M_z [kNm]	V_y [kN]	M_t [kNm]
Member a-1	1740	-102	109	-70	-26	7
Member a-2	1951	-11	-60	3	0	-1
Member b-1	2319	47	-40	-4	0	0
Member b-2	2310	-51	147	85	49	-4
Member c	-264	203	278	-15	-5	-1

Two structural concepts are used to design connection 1:

1. Slotted-in metal plates.
2. Glued-in rods.

6.4.1 Slotted-in steel plates

Each member is connected with its respective angle to an octagonal metal plate. The forces in each member are transferred from slotted-in metal plates to the octagonal metal plate via welding. The forces are transferred to the successor members, i.e. a-1 to a-2, b-1 to b-2, through metallic connectors that have sufficient capacity to transfer normal forces, shear forces, and bending moments. In order to follow the architectural guideline, which aims to have hidden connections, non-structural timber plates can be inserted in the lower and upper parts. Figure 6.10 shows the geometry of connection 1.

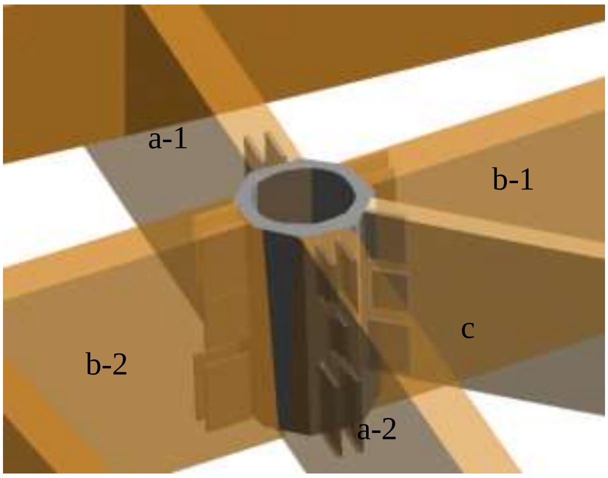


Figure 6.10 Geometry of connection 1, upper and lower plates working on bending, middle plate working on shear.

The main forces acting on the structure are transferred through the slotted-in plates. The upper and lower plates transfer the axial forces of compression and tension generated by the bending moment and the normal forces. These are double plates due to the great magnitude of these forces. Through the addition of shear planes, the connections are able to transfer forces in a ductile manner. The single central plate has the function to transfer shear forces.

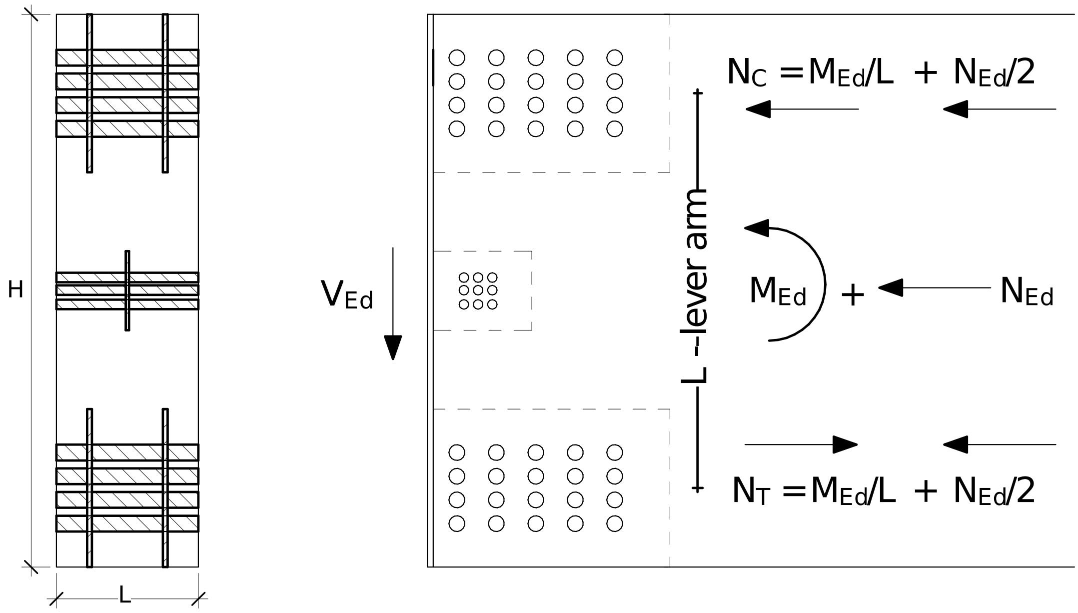


Figure 6.11 Structural concept of slotted-in plates.

- (a-1) (a-2) and (b-1) (b-2)

As the values of the forces acting on these members are the same, the same connection design can be used in order to make the project simpler regarding the construction. Table 6.3 gives the design results of the middle single shear plate.

Table 6.3 Connection 1 design, a1-a2, and b1-b2 single middle plate (Shear forces).

Dowels Class 4.80		Plate S 355 N/NL			Shear Capacity	Transversal Stiffness
d [mm]	n_{dowels} [-]	L_p [mm]	H_p [mm]	t_s [mm]	$F_{V.Rd}$ [kN·m]	K_{total} [kN·m]
12	9	250	200	8	109	18609

The connection is designed to withstand the highest normal force, in this case, N_T= 1723,00 kN. Table 6.4 shows the design results of the top and bottom double plates working in bending.

Table 6.4 Connection 1 design, a1-a2, and b1-b2 top and bottom double plates (Bending moment). The number of fasteners is per double plate.

Dowels Class 6.80		Plate S 355 N/NL			Bending Capacity	Rotational Stiffness
d [mm]	n_{dowels} [-]	L_p [mm]	H_p [mm]	t_s [mm]	$F_{ax.Rd}$ [kN]	K_{total} [kN·m·rad^{-1}]
20	36	600	600	16	1815	136774

- C

The member c connection has the same design principle as the other connections. The distinction is the values of shear forces, bending moment, and normal forces. Table 6.5 gives the design results of the middle single shear plate.

Table 6.5 Connection 1 design, -c single middle plate (Shear forces).

Dowels Class 4.80		Plate S 355 N/NL			Shear Capacity	Transversal Stiffness
d [mm]	n_{dowels} [-]	L_p [mm]	H_p [mm]	t_s [mm]	$F_{V.Rd}$ [kN·m]	K_{total} [kN·m]
16	12	300	400	8	234	8271

The connection is designed to withstand the highest normal force, in this case N_T= 337,00 kN. Table 6.4 shows the design results of the top and bottom double plates working in bending.

Table 6.6 Connection 1 design, -c top and bottom double plates (Bending moment). The number of fasteners is per double plate.

Dowels Class 4.80		Plate S 355 N/NL			Bending Capacity	Rotational Stiffness
d [mm]	n_{dowels} [-]	L_p [mm]	H_p [mm]	t_s [mm]	$F_{ax.Rd}$ [kN]	$K_{r.total}$ [kN·m·rad^{-1}]
20	10	400	400	16	475	77536

6.4.2 Glued-in rods

The structural concept behind this joint is dependent on the axial capacity of steel rods. The bending moment and the normal force are transferred by the rods in compression and tension zones in different layers, Figure 6.12 illustrates the structural mechanism. The axial load of each zone is applied to the centre of rigidity of the compression rods and tension rods. The shear force is resisted by several rods arranged with a 45-degree inclination in the top and bottom of the timber section. The inclined rods work as inclined members in a truss system, where their axial capacity resists the shear forces. The bored holes have a diameter d+2mm with 1 mm thickness for the glue. The glue type can be one of these adhesives; 2-part thixotropic epoxy resin, PRF resin, or 2-part polyurethane adhesive.

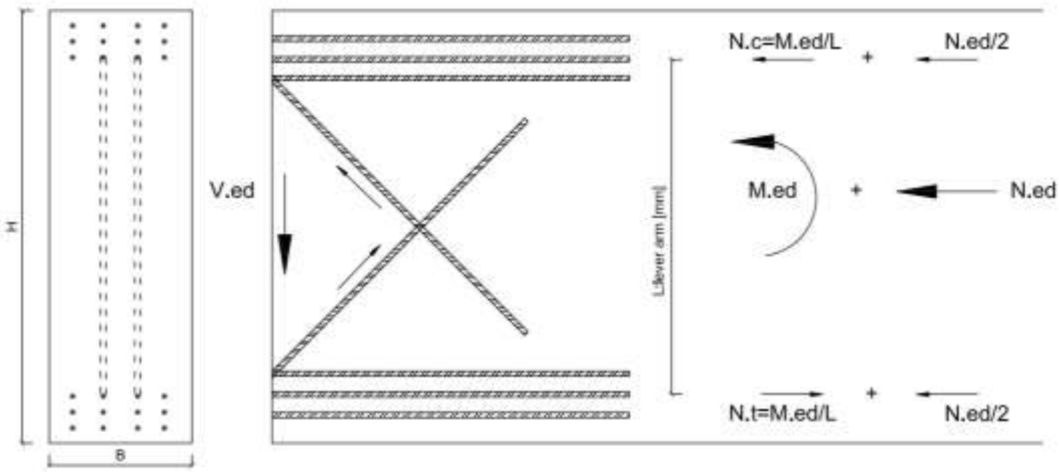


Figure 6.12 Structural concept of joint with GIR

- (b-1) (b-2) and (a-1) (a-2)

Since members (a) and (b) are subjected to the largest moment, 2310-1951 kNm., a design utilizing GIR's was difficult to achieve because the capacity is limited by even the tensile capacity of the rods or the pull-out capacity of the timber with a maximum rod's length of 1 m. The only way to achieve the capacity is by increasing the number of rods by adding rods in more than one layer. Because of the increasing number of layers, the lever arm will decrease and the normal force from the moment will increase. Calculation with five layers (4x20mm) in the compression zone and in the tension zone did not give the desired capacity. Arranging the rods in more than five layers is not a reasonable design to be considered because of the difficulty in the assembling. Also, a large number of rods, 40, does not provide an economical solution.

- C

As in the members (a) and (b), the member (c) did not reach the desirable capacity utilizing GIR's.

To achieve a consistent design in connection (1) the first alternative has been considered the most suitable design and is applicable for the five members in connection (1).

6.4.3 Evaluation

Because of the large bending moment, the design is based on the first concept, Figure 6.13 shows the five members connected by an orthogonal steel plate which is welded to slotted-in plates in every member. The rotational stiffness provided by the connection 1 in each member is compared with the rotational stiffness of a cantilever beam with length, L= 1 m, that is subjected to a moment M=1 kN.m at its end. Table 6.7 shows that the rotational stiffness is lower when compared to the section's rigidity.

Table 6.7 Connection 1 rotational stiffness evaluation.

Member	Rotational stiffness [kN.m. rad^{-1}]	Flexural stiffness of the section [kN.m/rad/m]	Utilization [%]
A, B	136774	3466667	3.9
C	77536	1070160	7.2

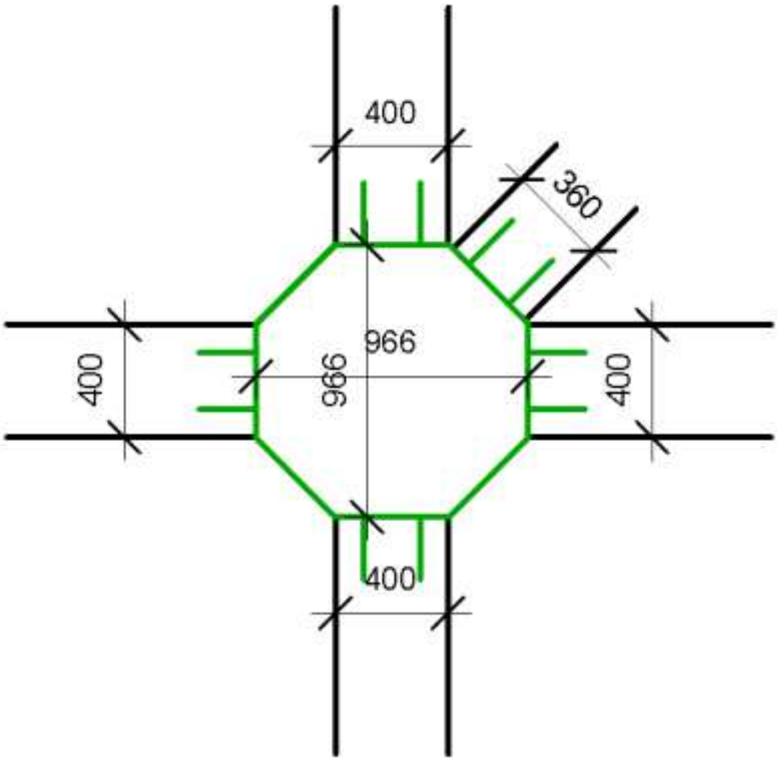


Figure 6.13 Geometry of connection 1.

6.5 Connection 2

The connection consists of the intersection of 4 members. The b-1 b-2 beams are continuous because it is the largest bending moment. The a-1 a-2 beams are connected through a connection that transfers shear forces, bending moments, and normal forces. Figure 6.9 shows the geometry of the connection and Table 6.8 presents the forces in the connection.

Members (a-1) and (a-2) are parts of a continuous beam with section (2000×400) mm. Members (b-1) and (b-2) are parts of a continuous beam with section (2000×400) mm.

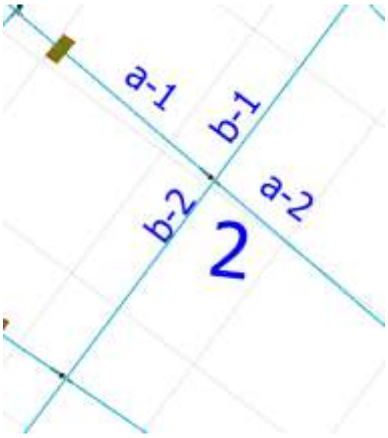


Figure 6.14 Connection 2.

Table 6.8 Forces in connection 2.

	M_y [kNm]	V_z [kN]	N [kN]	M_z [kNm]	V_y [kN]	M_t [kNm]
Member a-1	686	-21	41	-84	-43	3
Member a-2	686	-67	15	7	7	3
Member b-1	1904	-82	147	-65	49	-5
Member b-2	1904	-121	108	28	28	-5

Two structural concepts are used to design connection 2:

1. Slotted-in metal plates.
2. Glued-in rods.

6.5.1 Slotted-in metal plates

The upper and lower plates are responsible for transferring bending moments and normal forces, while the middle plate transfers shear forces. The plates are welded on a plate parallel to the side of the continuous beam. These plates are connected by metal bolts in the tension areas and by a circular profile in the compression area. The circular profile is also responsible for most of the transfer of the shear forces.

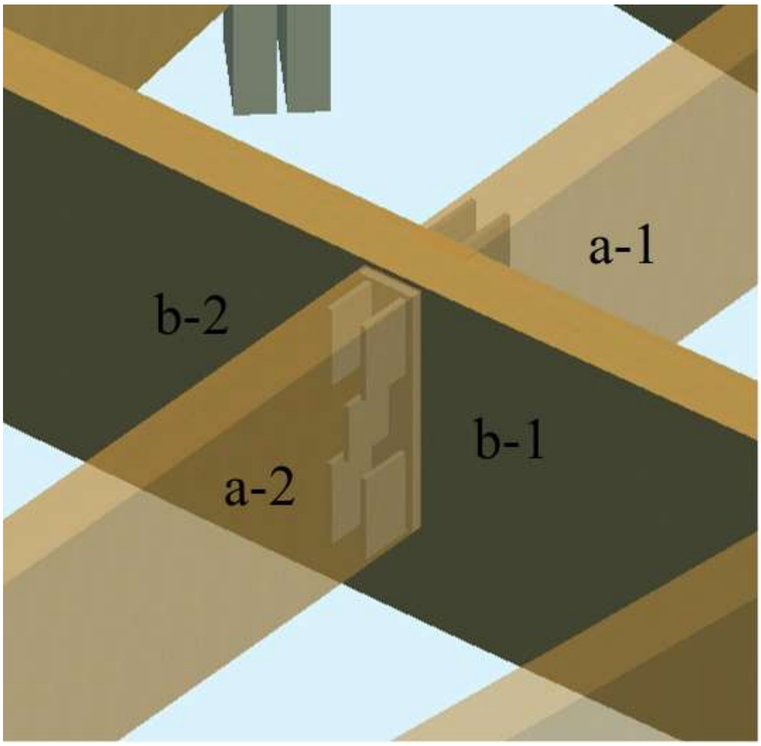


Figure 6.15 Geometry of connection 2, upper and lower plates working on bending, middle plate working in shear.

The structural concept of connection 2 is similar to the mechanism presented in Figure 6.11.

- (a-1) (a-2)

The failure mode on the dowels in the shear area is ductile. The failure mode according to Johansen's Theory is mode h. All failure modes in steel-to-timber connections are shown in Figure 4.3. Table 6.9 gives the design results of the middle single shear plate.

Table 6.9 Connection 2 design, a1- single middle plate (Shear forces).

Dowels Class 4.80		Plate S 355 N/NL			Shear Capacity	Transversal Stiffness
d [mm]	n_{dowels} [-]	L_p [mm]	H_p [mm]	t_s [mm]	$F_{V.Rd}$ [kN·m]	K_{total} [kN·m]
12	9	250	200	8	109	18609

In case of the double plates resisting the bending moment, the required thicknesses, t1 and t_2, are controlled to assure that the dowel will form 3 plastic hinges and the failure will not occur on the timber member. Figure 6.16 illustrates how the member is divided in case of a multi-plane connection.

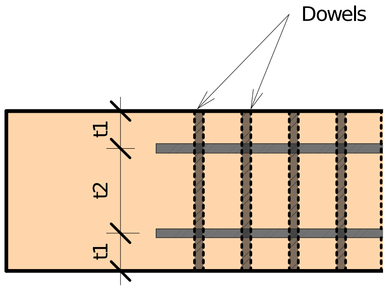


Figure 6.16 Required thickness is controlled in order to guarantee ductile failure.

The connection is designed to withstand the highest normal force, in this case N_T= 706.5 kN. Table 6.10 shows the design results of the top and bottom double plates working in bending.

Table 6.10 Connection 2 design, a1-a2 top, and bottom double plates (Bending moment). The number of fasteners is per double plate.

Dowels Class 4.80	Plate S 355 N/NL				Bending Capacity	Rotational Stiffness
d [mm]	n_{dowels} [-]	L_p [mm]	H_p [mm]	t_s [mm]	$F_{ax.Rd}$ [kN]	$K_{r.total}$ [kN·m·rad^{-1}]
20	20	600	400	12	886	38768

6.5.2 Glued-in rod

The same concept explained in Section 6.4.2 is applied to this connection.

- (a-1) (a-2)

Both parts of the member A have the same design. The rods are placed in the top and bottom zones in three layers as showed in Figure 6.16. The shear force is resisted by four inclined rods, two in the top, and two in the bottom.

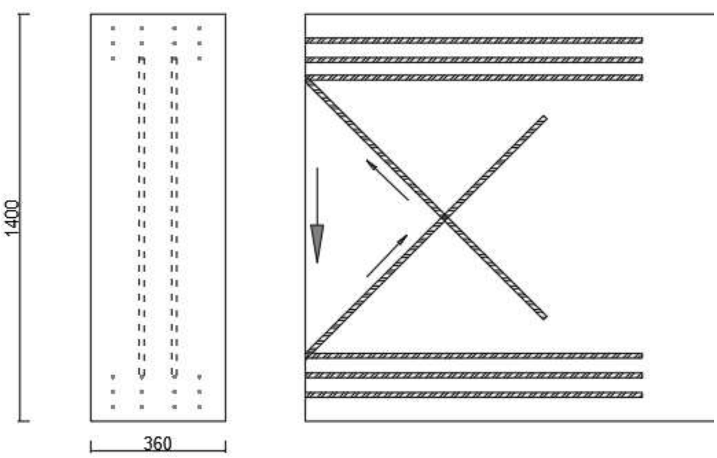


Figure 6.17 GIR connection 2 (A-1) (A-2)

The connection's performance with details is presented in Table 6.11 and in Table 6.12.

Table 6.11 Connection 2 design, a1-a2 horizontal GIR resisting normal forces.

Rods in compression and tension zones Class 4.80				Bending Capacity based on the axial capacity of the rods	Rotational Stiffness
Layer [-]	d [mm]	l_a [mm]	Number of rods [-]	$F_{ax.Rd}$ [kN]	$K_{r.total}$ [kN·m·rad^{-1}]
1	20	1000	4	673	54381
2	20	1000	4		
3	20	1000	2		

Table 6.12 Connection 2 design, a1-a2 inclined GIR resisting shear force.

Inclined rods with 45˚ in each zone Class 4.80				Shear capacity based on the axial capacity of the rods	Transversal Stiffness
Layer [-]	d [mm]	l_a [mm]	Number of rods [-]	$F_{v.Rd}$ [kN]	K_{total} [kN/m]
3	20	1000	2	190	22165

The horizontal and inclined rods with axial capacity $N_{ax.Rd} = 62.82\ [kN]$ are determined by the tensile strength of the rod.

The spacing of the rods is as follows:
$a_2 = 50\ [mm]$ spacing to the edge of the timber cross-section.
$a_1 = 80\ [mm]$ spacing between the rods.

The failure mode is governed by the tensile rods' capacity which is a ductile behaviour. The reason behind choosing a low strength class to the rods is to achieve a ductile behaviour and avoid failure in timber caused by pull -out stresses along the rods. The calculation is explained in Annex C and in the Digital Annex B.

Arranging some rods at an angle helps to overcome any stresses generated by shrinkage effects perpendicular and parallel to the grain.

The GIR´s rotational stiffness is underestimated using equation (20). Estimations based on FEM analysis of rods subjected to axial tension forces showed a better rigidity (Xu et al., 2012). The axial stiffness provided by equation (20) used in the calculation gave low values. More realistic values are given by (Xu et al., 2012), the following equation is used to calculate the axial stiffness.

$$k_{ser} = \frac{A_s \cdot E_s}{0{,}3 \cdot la} = 219911\ kN/m$$

This approach gives much higher axial stiffness, consequently more rotational stiffness according to equation (23).

6.5.3 Evaluation

- Connection between the intersected members (a) and (b):

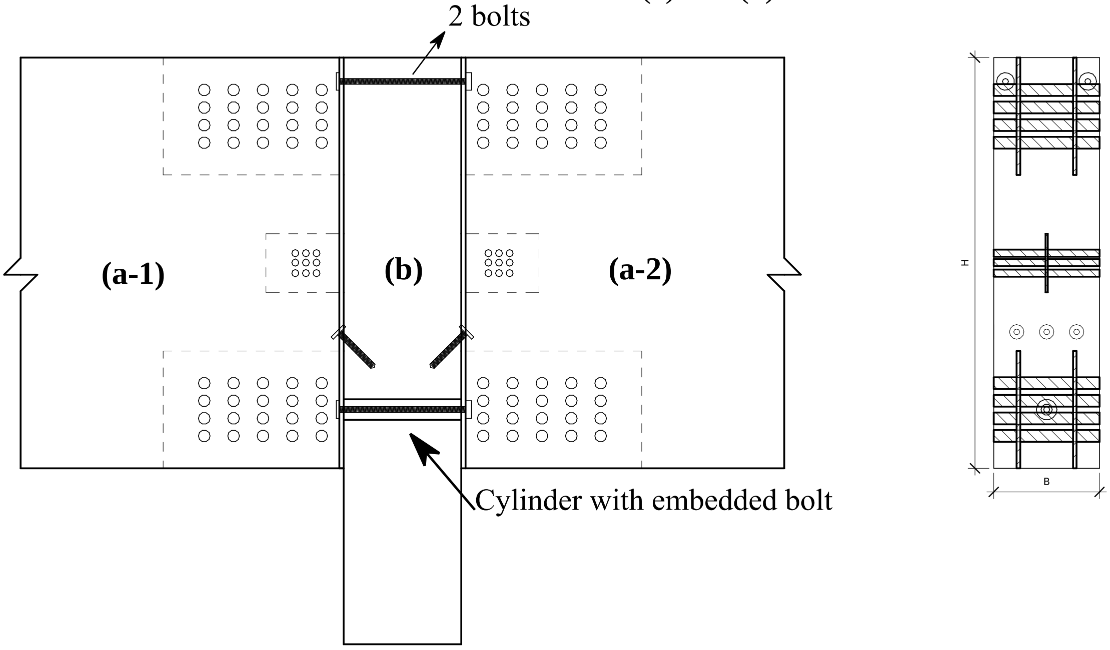


Figure 6.18 Connection between the intersected members (a)(b).

The connection between the intersected members (a) and (b) is a pinned connection that can transfer the shear forces and does not transfer the bending moment between the intersected members.

The connections in (a-1) and (a-2) are connected through member (b) by bolts, cylinder, and screws that are fixed to metal plates parallel to the side of the continuous beam (b) The bolts are placed at the top edge or in the tensioned zone to transfer the tension forces.

Bolts:

Class:10.8
d=20 mm
Number of bolts: 2

The cylinder is placed to transfer the compression and shear forces between the two parts. The void space inside the cylinder is designed to fit one bolt that has the function to fix the two metal plates in its compression zone.

Cylinder:

Class: S 460 NH/NLH
$d_{in} = 38\ mm$

$d_{ex} = 70\ mm$
$\mathrm{t} = 16\ mm$

To make sure that the shear force transferred from members (a) will not cause any high stresses perpendicular to the grain in member (b) several screws will be threaded in the two metal plates and inserted in the member (b) in 45 degrees angle.

- Comparison:

1. Both concepts provide reasonable dimensions and applicable design for manufacturing.
2. Fill the requirement of a hidden assembling.
3. From a structural behaviour point of view, both provide enough capacity with ductile behaviour.
4. The main difference is the influence on the global behaviour because they provide the global system with different rigidity which comes from their stiffness.

Table 6.13 Connection 2 stiffness evaluation.

Joint type	Transversal stiffness [kN/m]	Rotational stiffness [kN· m /rad]	Flexural stiffness of the section [kN.m/rad/m]	Utilization [%]
Slotted-in metal plates	18608	38768	1070160	3.6
GIR’s	5541	54381	1070160	5.1
GIR’s according to (Xu et al., 2012)	155501	1526097	1070160	142.6

6.5.4 Design pattern for different angled connections

The connection 2 in which 4 members intersect each other is the connection model most found in the structure, having as a differential factor the angle of intersection of the members. Therefore, the creation of a connection that can be used as a design basis is of utmost importance. Thinking about this, two models are briefly proposed. The first model, Figure 6.19, is made up of metal sheets in a triangular shape that adapts to the desired geometry, considering the angle of intersection. The advantage of this proposal is that the parts of the connection, slotted-in steel plates, or rods have the same length of insertion in the connected members, moreover, the beams do not need to be chamfered in their contact surface. As a disadvantage, the connections are apparent, breaking the architectonic proposal of hidden connections and these connecting members will be more susceptible to corrosion attacks and exposure to fire.

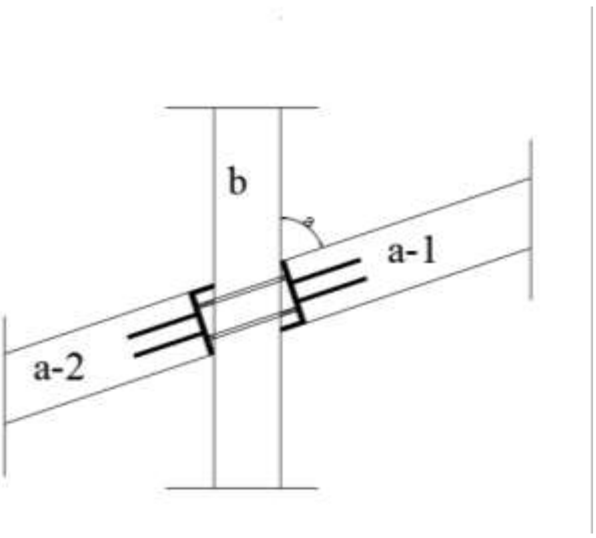


Figure 6.19 Model 1 for different angles.

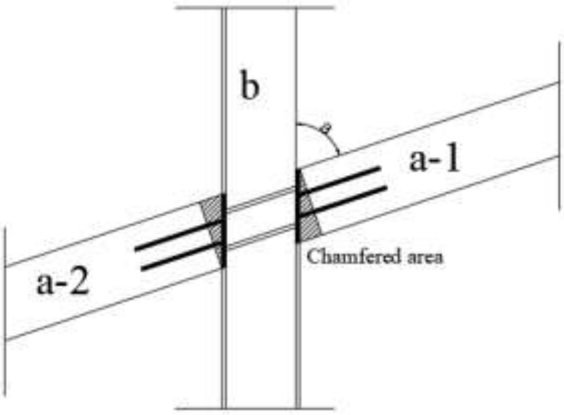


Figure 6.20 Model 2 for different angles.

The second proposal, Figure 6.20, is safer in relation to external agents, in contrast, it involves more detailing of each connection, as the beams should be adapted at their ends to the intersection angle and the internal connector members will have varying lengths, moreover, the anchorage of non-perpendicular bolts have to be deeper investigated. Another way is to locate the bolts and the cylinder perpendicular to element B, which makes it easier in production. Consequently, the bolts and the cylinder will be subjected to moment and torsion since the transferred forces are parallel to members A.

6.6 Connection 3

In the model two knee joints are critical and have been investigated. Columns C82 and C131 have different sections and stresses as shown in Table 6.14 and their geometry is shown in Figure 6.21.

C82 consists of a beam and column connection with section (1400 x 360) mm.
C131 consists of a beam and column connection with section (2000 x 400) mm.

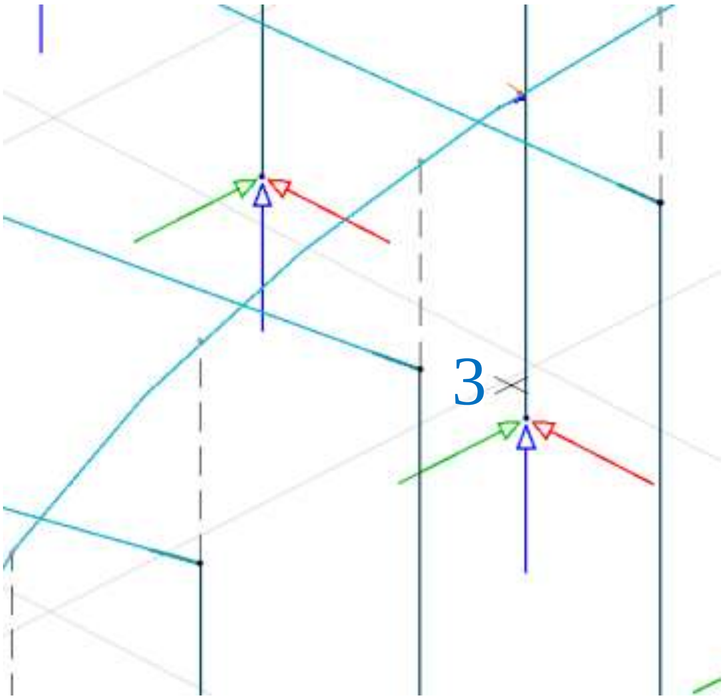


Figure 6.21 Connection 3.

Table 6.14 Forces in connection 3.

	M_y [kNm]	V_z [kN]	N [kN]	M_z [kNm]	V_y [kN]	M_t [kNm]
C82	-857	278	0	0	0	0
C131	1082	210	0	0	1	1

To choose a proper design two concepts have been calculated:

1. Fork joint with dowels.
2. Glued-in rods connected with metal plates.

6.6.1 Fork joint with dowels

The connection showed in Figure 6.22 is done by overlapping two external parts of the column and an internal part of the beam. They are connected by dowels type fasteners in two shear planes. The dowels are arranged in circles and subjected to lateral forces caused by the moment added to the shear force using elastic distribution as explained in Section 2.2.2.

- **C82:**

The shared overlapping area is 1400x 1400 mm with the thickness equal to the thickness of the beam, 360 mm, divided into three parts:

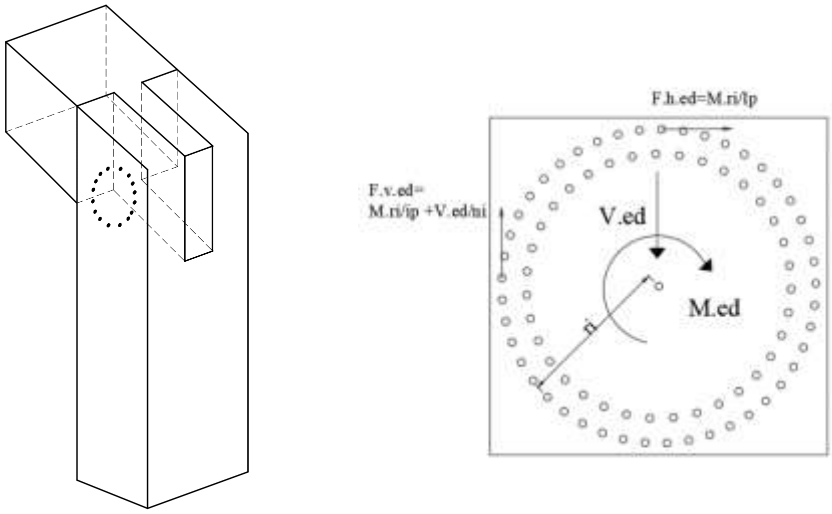


Figure 6.22 Fork joint with dowels.

Thickness of outer parts of the column $t_1 = 110\ [mm]$.
Thickness of the internal part of the beam $t_2 = 140\ [mm]$.

The internal section of the beam with the thickness t_2 has been checked to guarantee that it is able to transfer the moment and the shear force to the joint at the point where the beam is cut. The section (1400x140) mm has enough moment capacity with $M_{Rd} = 965.88\ kN.m$, but not enough shear capacity $V_{Rd} = 196kN$.

Total number of dowels n= 80

The dowels are in a circular arrangement, details in Table 6.15 and Figure 6.23.

Table 6.15 Dowels arrangement in C82

Class 6.8	D=20 [mm]
Radius of circular arrangement [mm]	Number of dowels [-]
650	40
540	40

Rotational Stiffness $K_{r.total}$ [kN·m·rad^{-1}]
447291

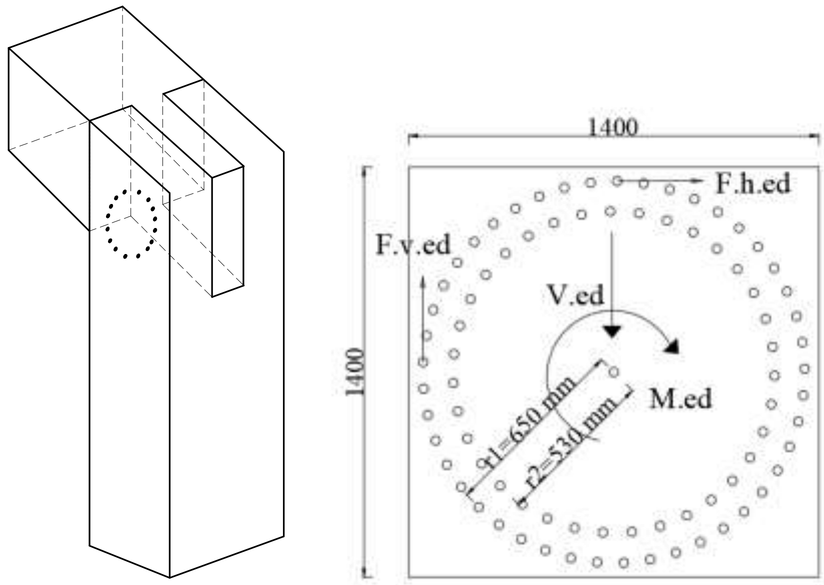


Figure 6.23 C82 Fork joint.

The choice of the diameter and the strength of dowels in addition to the thickness of timber parts are done to get the most efficient design according to EC5 and the calculation is explained in Annex C and in the Digital Annex B.

Table 6.16 C82 dowels capacity.

Lateral capacity of dowel [kN]	Type of failure mode	Utilization [%]
$F_{v.Rd} = 26.96$	k(ductile)	85
$F_{h.Rd} = 19.85$	J (ductile)	98.7

According to the results in Table 6.16, the dowel subjected to the horizontal load will plasticize first.

- **C131:**

The shared overlapping area is (2000 x 2000) mm with the thickness equal to the thickness of the beam, 400 mm, it is divided into three parts:

Thickness of outer parts of the column $t_1 = 120\ [mm]$.
Thickness of the internal part of the beam $t_2 = 160\ [mm]$.

Total number of dowels $n = 80$

The dowels are in a circular arrangement, details in Table 6.17, and in Figure 6.24.

Table 6.17 Dowels arrangement in C131.

Class 6,8	D=20 [mm]
Radius of circular arrangement [mm]	Number of dowels [-]
900	40
700	40

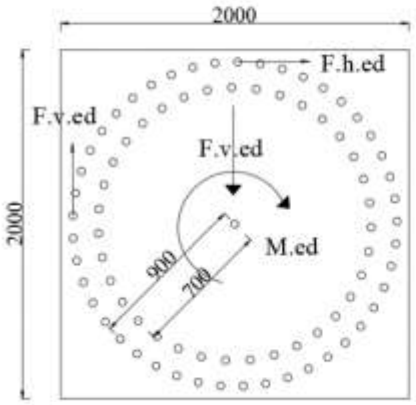


Figure 6.24 C131 Fork joint.

Rotational Stiffness $K_{r.total}$ [kN·m·rad-1]
877710

Table 6.18 C131 dowels capacity.

Lateral capacity of dowel [kN]	Type of failure mode	Utilization [%]
$F_{v.Rd} = 26.96$	K (ductile)	74
$F_{h.Rd} = 21{,}08$	J (ductile)	82

6.6.2 Glued-in rods with steel plates

The structural concept behind this joint is dependent on the axial capacity of steel rods. The bending moment is transferred by the rods in the compression and the tension zones in different layers. The shear force is resisted by the beam and column timber’s contact surface and checked as a compression force with an angle to the grain. Also, the shear forces cause a lateral force acting on the rods as explained in Figure 6.25.

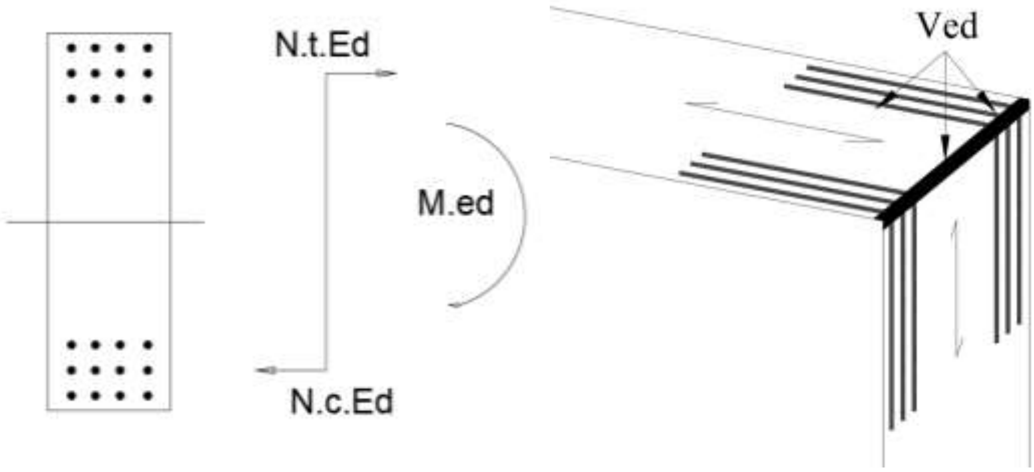


Figure 6.25 Connection 3 with GIR's.

The rods are inserted in predrilled holes with d+ 2 mm, in which 1 mm accounts for the thickness of the glued area. The glue injection points are drilled holes perpendicular to the rod. The rods are screwed onto the plates, by pre-drilled threaded bores, and the plates are attached together with of bolts.

- **C82:**

The connection C82 with GIR's is presented in Table 6.19, and the force distribution in the connection is presented in Figure 6.26.

Table 6.19 Connection 3 design, C82 GIR resisting normal forces

Rods in compression and tension zones – Class 4.80				Bending capacity	Rotational Stiffness
Layer [-]	d [mm]	l_a [mm]	Number of rods [-]	M_{Rd} [kN.m]	$K_{r.total}$ [kN·m·rad^{-1}]
1	20	1000	4	921	61908
2	20	1000	4		
3	20	1000	4		

The spacing of the rods is as follows:

$a_2 = 50\ [mm]$ spacing to the edge of the timber cross-section.
$a_1 = 80\ [mm]$ spacing between the rods.

The horizontal rods with axial capacity $N_{ax.Rd} = 67.32[kN]$ are determined by the tensile strength of the rod.

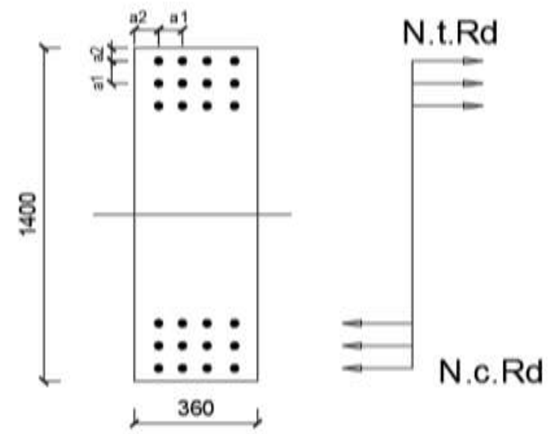


Figure 6.26 Force distribution in Beam C82.

The failure mode is governed by the tensile rods' capacity which gives a ductile failure behaviour. The reason to choose lower strength of the rods is to achieve a ductile behaviour and avoid failure in the timber member caused by pull -out stresses along the rods. The stress distribution in the section is plastic. This design is considered conservative since the contribution of the timber contact surface has been neglected and this affects largely the rotational stiffness. The reason to neglect the timber contact surface is to get homogenous stress distribution during the design. The calculation is explained in Annex C and in the digital attachment.

- **C131:**

The connection C131 with GIR's is presented in Table 6.20.

Table 6.20 Connection 3 design, C131 GIR resisting normal forces

Rods in compression and tension zones Class 4.80				Bending Capacity	Rotational Stiffness
Layer [-]	d [mm]	l_a [mm]	Number of rods [-]	$M_{.Rd}$ [kN.m]	$K_{r.total}$ [kN·m·rad-1]
1	20	1000	4	1406	123592
2	20	1000	4		
3	20	1000	4		

6.6.3 Evaluation

1. Both concepts provide reasonable dimensions and applicable design for manufacturing.
2. Both fill the requirement of a hidden assembling. In fork joints, many dowels make the assembling in the site harder and more time-consuming. Although the GIR's have also a big number of rods, it is still applicable, but expert workers are needed to assure an accurate assembling.

3. From a structural behaviour point of view, both provide enough capacity with ductile behaviour.
4. The main difference is the influence on the global behaviour because they provide the global system with different rigidity which comes from their stiffness.

For C82 and C131, the rotational stiffness for both types of connections is presented in Table 6.21 and in Table 6.22.

Table 6.21 C82 rotational stiffness with Fork and GIR.

Joint type	Rotational stiffness [kN· m /rad]	Flexural stiffness of the section [kN.m/rad/m]	Utilization [%]
Fork joint	447291	1070160	41.8
GIR	69433	1070160	6.5
GIR's according to (Xu et al., 2012)	1948503	1070160	182.1

Table 6.22 C131 rotational stiffness with Fork and GIR

Joint type	Rotational stiffness [kN· m /rad]	Flexural stiffness of the section [kN.m/rad/m]	Utilization [%]
Fork joint	707130	3466666	25.3
GIR	123592	3466666	3.6
GIR's according to (Xu et al., 2012)	3468355	3466666	100

As it was mentioned, the GIR´s rotational stiffness is underestimated. Especially in cases with large moments, where gaps between timber members are neglected. The rotational stiffness could be increased by considering the timber members' compression zones contribution (Xu et al., 2012).

Another reason is the axial stiffness for GIR in tension according to (Xu et al., 2012) also is underestimated. A closer value to a FEM analysis result is given by using the following equation to calculate the axial stiffness.

$$k_{ser} = \frac{A_s \cdot E_s}{0{,}3 \cdot la} = 219911.49\ kN/m$$

This gives a much higher rotational stiffness:

$$k_{r.C82} = 1737300.74\ kN.m/rad$$

Regarding the fork joint, the stiffness is higher and based on the number of dowels n=80, in which could be unpractical, in a connection with dowels with diameter d= 20 mm, 80 holes should be predrilled.

6.7 Optimized design

After investigating different types of connections in three locations in the model, the next step is to study the connection's influence on the global system. Glued-in rods connections are chosen for connection (2) and (3) and slotted-in type connections for connection (1).

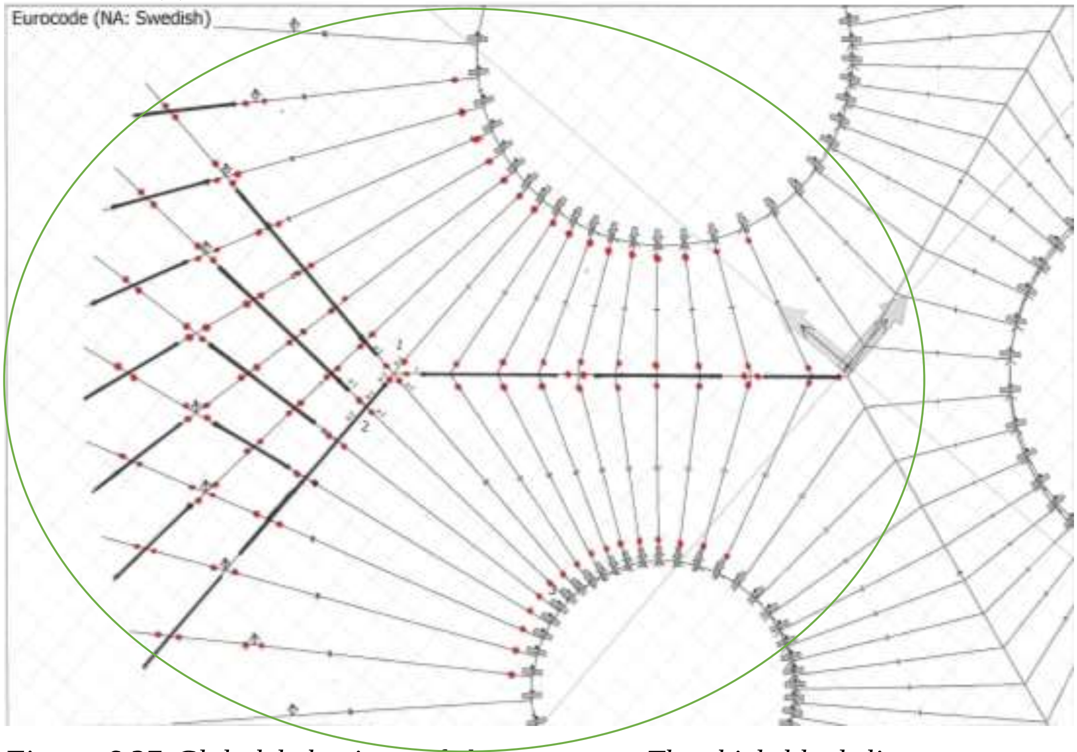


Figure 6.27 Global behaviour of the structure. The thick black lines represent the continuous beams and the red dots represent connection points. The green circle accounts for the area of study.

Since connection (2) and (3) are chosen in the joints with maximum stresses, it is applicable in several other points in the model. Based on the distribution of the connections shown in Figure 6.27, the connections chosen by their stiffnesses were applied to the studied part of the structural "FEM-Design 19 3D Structure" model. The FEM-analysis is again performed, this time with the modified stiffness of the joints.

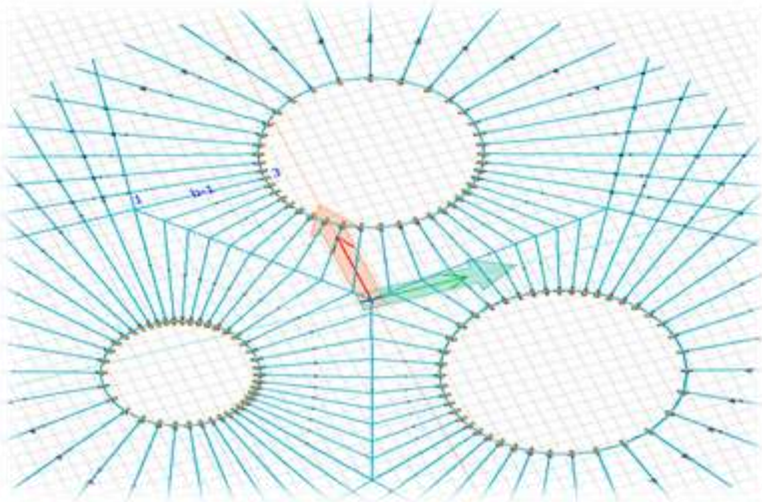


Figure 6.28 Member (b-1) and its connections in the model.

The internal forces in connection (1) from the updated modified analysis are presented in Table 6.24. The values are compared with the values from the original model in which all connections were modelled with full rigidity.

Table 6.23 Rotational stiffness provided in FEM model.

Joint type	Rotational stiffness [kN· m /rad]
Fork joint (3)	707130
GIR (3)	123592
Slotted-in a (1)	136774
GIR'(2)	54381
Default rotational stiffness provided by StruSoft	9.95E+09

Table 6.24 Internal forces in connection (1) after and before applying the connection's stiffnesses. After ☐ Before ■

Connection 1						
	M_y [kN.m]	V_z [kN]	N [kN]	M_z [kN.m]	V_y [kN]	M_t [kN.m]
Member a-1	1911	-97	-64	-296	-111	23
	1762	-102	109	-70	-26	7
Member a-2	1546	-22	-136	-7	0	-4
	1951	-11	-60	3	0	-1
Member b-1	1762	29	-72	-18	1	3
	2319	47	-40	-4	0	0
Member b-2	2132	72	147	257	95	-13
	2310	-51	147	85	49	-4
Member c	524	44	232	15	3	-6
	-264	203	278	-15	-5	-1

Figure 6.29 shows the moment distribution in the studied member after applying the connection's stiffness. Comparing the results with the original model, shown in Figure 6.31, it observed that the moment redistribution takes place between the beam and the connections. The moment in the column (knee joint) has increased, while the moment at the connection (21) has decreased. Since the provided stiffness at connections (1) and (3) are different, the one with higher stiffness will attract more forces. The difference is larger if fork joints are used in connection (3) since it provides more rotational stiffness than the connections using GIR's, as demonstrated in the calculation made in Section 6.6. The moment distribution adopting the fork joint instead of the knee joint is shown in Figure 6.30.

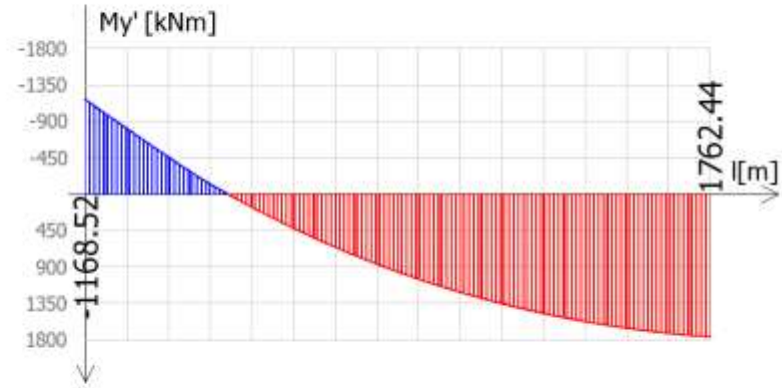


Figure 6.29 Moment distribution in (b-1) with GIR (knee joint) in connection (3).

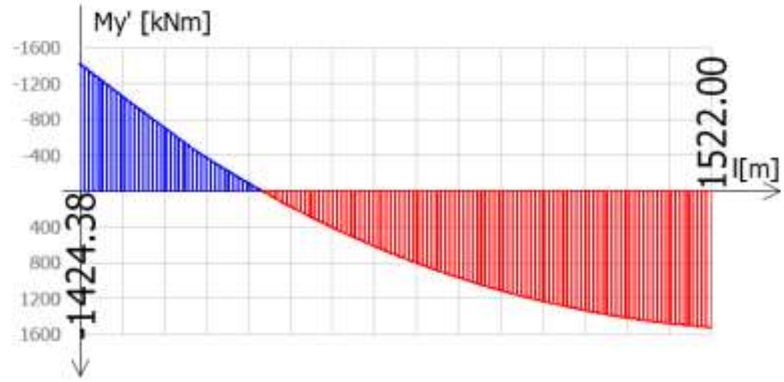


Figure 6.30 Moment distribution in (b-1) with fork joint in connection (3).

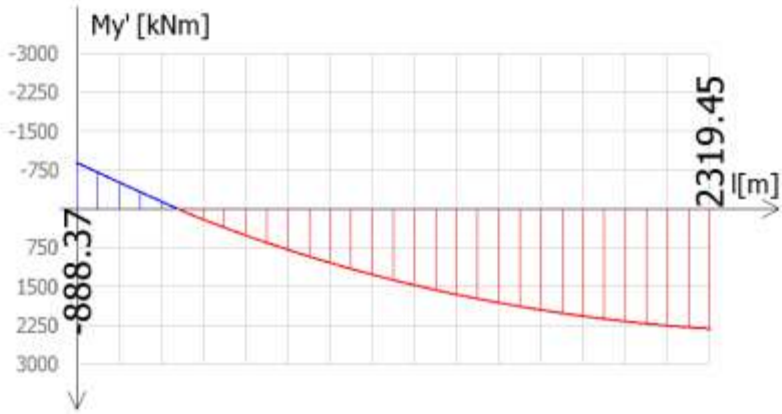


Figure 6.31Moment distribution in (b-1) in the original model.

The next step is to check the structural members and the connections according to their capacity to withstand the new forces. Also, the deflection in the serviceability limit state should be checked since the stiffness has a direct effect on the deflection in the system. If it is necessary to redesign any component of the system, the analysis should be repeated with the newly designed components and checked on ULS and SLS.

After getting the results from “FEM-Design 19 3D Structure” the design is an iterative process to get the most suitable and optimized design of the connections and of the global structure. The design process is shown in the flowchart presented in Figure 6.32.

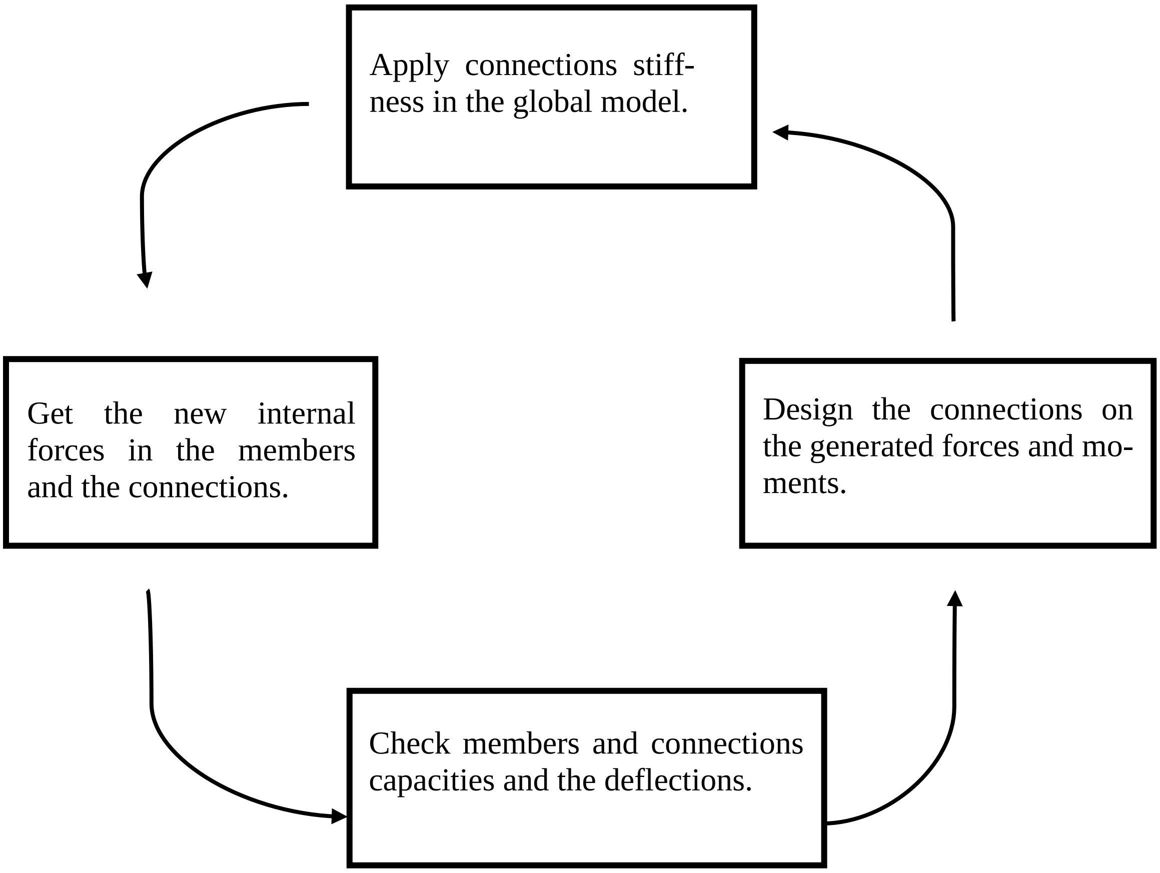


Figure 6.32 Design steps flowchart.

7 Discussion

This chapter focuses on discussing the results presented in the case study, as well as answering some questions raised in the aim and objectives of this master's thesis. From an evaluation of the two connection methods presented in the case study, namely, slotted in metal plates and glued-in rods connections, the main ideas on the understanding of connections of large wood beams are addressed.

- Slotted- in metal plates

The demands regarding capacity were met, and because they are hidden, the connection has effective protection against external agents such as fire exposure and corrosion. The connections using slotted-in metal plates could be designed using a higher strength class of dowels. The reason to use a lower strength class is to increase the number of dowels and consequently obtain higher stiffness. It is noted that the arrangement of dowels is of utmost importance in defining the stiffness of the connection, since increasing the number of dowels in a row parallel to the load, in which is the same to say, increasing the number of dowels in series, will result in a negative influence on the value of the rotational stiffness despite increasing the capacity of the connection. Analysing the Equation (24) one can notice that the final value of the stiffness will be smaller with the increase of dowels in the direction of the load. An increase in the number of dowels in the vertical direction will contribute a lot to a higher rotational stiffness but in return, the capacity of the connection will reduce, since the dowels will be closer to the centre of rotation of the connection. Figure 7.1 exemplifies the influence of the number of dowels per direction.

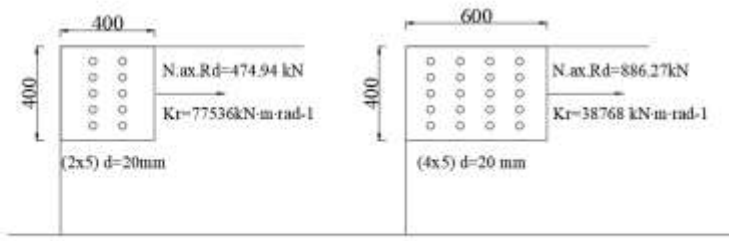


Figure 7.1 Comparison between connection (1) C and connection 2 (a) for bending resistance.

The same behaviour could be observed in relation to shear forces. The number of dowels in the direction of the applied force, in this case, 90 degrees, will influence the stiffness. Therefore, dowels in the vertical direction are not beneficial to achieve a high translational stiffness, although they contribute to the shear force capacity of the connection. An increase of dowels in the horizontal direction will result in higher stiffness, but in return, the dowels will have to withstand a higher moment generated by the eccentricity.

An influencing factor on the stiffness is the maximum number of dowels allowed when the rules for minimum spacing following EC5 are adopted. As mentioned, several times before in this report, the minimum spacing rules are defined in order to avoid splitting of the timber member that leads to instant collapse due to large stresses caused by the group effect. An alternative to decrease the minimum spacing required, increase the number of dowels and consequently increase the stiffness of a connection is to adopt reinforcement members, such as threaded rods or self-tapping screws. An example of this concept is presented in the article "Moment-resisting bolted timber connections"(Lam et al., 2010). The researchers were able to increase the capacity and stiffness of a column-to-beam connection by 40% through the usage of self-tapping screws. Therefore, a study taking this alternative into consideration should be considered.

The block shear effects that were determining factors of the connections presented in Chapter 5, could be effectively controlled, this is due to the size of the members. The areas of tensile and shear strength along the fracture perimeter are large enough to provide enough capacity.

The guide used for the design of the multi-plane slotted-in connections, “Limträhandbok del 3” (Svenskt trä, 2016), determines that the thickness of the part of the timber, between the metal plates, has the minimum value so that there will be no failure of the timber, instead the failure will occur in the dowels. This way, it was possible to guarantee a ductile failure mode.

Another proposal for slotted-in steel plates connections could be to use plates that would extend over the entire depth of the beam, instead of having separate plates distributed in the tension, compression, and shear areas. This way, it would provide easiness in questions related to the in-situ assembly of the structure. In contrast, more material would be used, the structure would be heavier and consequently result in an increase in the final price of the construction. In this case, cracking might occur when the beam shrinks as shown in Figure 4.10

- Glued-in rods

Based on the study carried out in Chapter 5 and the results from Chapter 6, glued-in rods provide satisfactory axial capacity combined with an adequate stiffness. In order to assure a ductile behaviour, the capacity must be limited by the tensile strength of the rods. The tensile capacity limitation of glued-in rods constrains its usage in cases where large bending moments capacity is necessary, this was noted on connection (1) of the study case. One approach to increase the bending moment capacity is to place more rods in more layers, however, as the rods approach the centre of rotation the influence on the capacity will be smaller and less efficient. In addition, by using a plastic stress distribution instead of an elastic distribution, will increase the capacity of the connection. The spacing between the rods is designed to prevent splitting at the edges and other brittle failure modes that occur due to a lack of parallel tensile capacity of the timber’s net section.

The lack of standards in glued-in rods design strongly affected the axial stiffness calculation of the case study. Two analytical methods were adopted, the first from “SC5.T5 Bonded-in-rods (first draft) Design of bonded-in-rods” (CEN et al., 2019), the second from the article “Analytical study and finite element modeling of timber connections with glued-in rods in bending” (Xu et al., 2012). The difference found in the analytical calculations were rather divergent. One more investigation subject that is also a discussion topic is the neglect of the timber’s compression contact surface which affects the rotational stiffness.

The designer should understand well the theoretical approach and the conditions behind this method in calculating the capacity and the stiffness. It is preferred to base the design on experimental curve fitting or on FEM analysis to find more reliable results.

- Fork joint

The capacity and stiffness of the dowels were calculated according to the elastic distribution of the bending moment and forces. This means that only the far most dowels would plasticize. Using the plastic distribution in the design, more capacity and stiffness could be obtained. Another important factor is to prioritize obtaining the largest number of dowels in the outermost layer of the circle, to achieve maximum strength and stiffness in the connection considering the longest possible lever arm. In this case, since the spacing can be less than the minimum according to EC5, reinforcement elements such as self-tapping screws should be introduced due to high stresses perpendicular to the grain. To overcome a large number of dowels, another alternative is to install hardwood plates between the beam part and the column parts. This would increase the dowel capacity by increasing the number of shear planes and the embedding strength.

In a design situation, the thickness of the fork joint's members should be defined based on whether the bending moment is more critical, or the shear force is more critical. This is explained by the larger capacity of the dowels when the force is applied parallel to the grain.

If the applied force on the dowel is generated mainly from the vertical shear force, the most critical check is the dowel's capacity on vertical load. The thickness of the parallel members (column parts) to the applied vertical force has a significant influence on the capacity of the connection, therefore it is important to allow a sufficient thickness for the parallel portions of the members. Figure 7.2 demonstrates the influence of the force in relation to the direction of the grains in a fork connection.

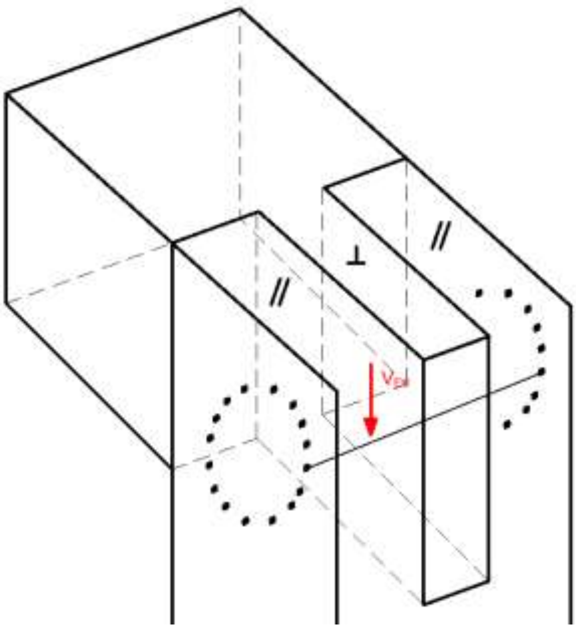


Figure 7.2 Direction of grains with an applied shear force.

Similarly, when the moment is larger enough to decide the force applied on the dowel the most critical to check the capacity of dowel subjected to horizonal force where the capacity is smaller which could be increased by increased the thickness of parts with directions parallel to the force the beam parts in this case since.
In cases where the bending moment and the shear force are equally contributing, it is necessary to consider a balance among the thickness of the contributions of each member.

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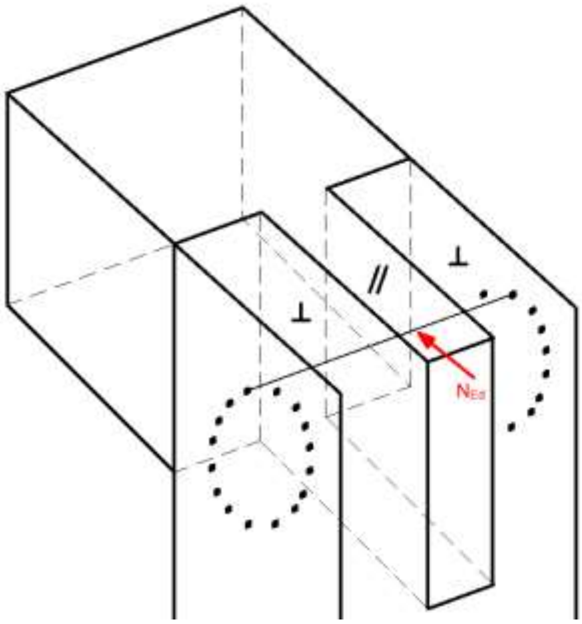

Figure 7.3 Direction of grains with an applied horizontal force generated from the bending moment.

It could be beneficial to "flip" the connection, so the beam has the two outer parts and the column the middle part. Since the moment capacity was sufficient and the shear force in the section of the splice was governing the beam, it would increase in the total width and by that would allow more shear capacity. The contact area of the compression force in the column will also increase by doing this.

- Global influence of connections stiffnesses:

The different connections designed in Section 6 provide different rotational and transversal stiffness. The following discussion is based on the connection stiffness and their application to the global system.

Preferably the design of moment resistance connections must attempt to achieve a behaviour close to a rigid connection to decrease the connection's influence on the sectional forces of the structural timber members.

When a member is connected with two connections with considerable difference in stiffness, there will be a large redistribution of internal forces in the connections and in the timber members. In this case, both considered critical and should be checked by their capacity and deflection. The connections with the larger value of stiffness will attract large forces compared with the other members.

While if the chosen connections provide the system approximately with the same stiffness, the effect will mostly be on the timber members since the rigidity of the connections is less than the timber section. As a result, the beams will attract more forces and will be more critical.

Even if the system contains connections with different structural concepts and components, it is important that the connections possess enough stiffness and that the stiffness value of the connections does not diverge considerably. In this way, the sectional forces distribution in the members will not change on a large scale. As an example, knee joints using glued-in rods were chosen over fork joints because their stiffness is closer to the stiffness of other connections in the model as in Section 6.7.

As a conclusion, not always a connection that has the highest rigidity is the most suitable for the final design. The designer must find a balance between the distribution of the internal forces in beams, columns, or connections so that there is no overload of one of these members.

8 Conclusion

- *General conclusion*

The study of connections in wooden structures has many variables, such as the size of the structure, the type of environment, the intention of use. In the case of simple structures, the design can be elaborated in a relatively simple way, because the demand in strength and rigidity is modest and many of the connectors available on the market are produced according to the sizes of the timber structural elements produced by the large producers. Available standards such as EC5, although in a very clear way, provides design guides for connections that cover up to a certain point of complexity. With the development of new techniques in the manufacture of members in Glulam and CLT, the need for standardization is evident. The role of the designer in this type of project is very influential because experience in connections and structural analysis is needed. It is not possible to design such advanced structures using methods that only apply to simple connections. Therefore, an understanding of glued-in rods, multi-plane slotted in plates is crucial, moreover, the reinforcement mechanisms must be meticulously understood, i.e. self-tapping screws and threaded rods.

- *Study on timber beam-to-column connections*

Connections of this kind can be considered simple in terms of geometry, considering that they consist of only two members. Nevertheless, the design of the case study 's connections varies as the connected beams have different inclinations in different areas of the structure.

Since both the beam and the column have wide sections, it allows the use of dowel type connections arranged in such a way as to provide sufficient capacity and rigidity, as in the fork joint concept. The intersecting area between the members facilitates the arrangement of the dowels far from the centre of the connections and withstands large moments.

The fork joints in the way that was proposed, in which the column and beam are cut in order to provide the fitting geometry, generates a negative impact in terms of ease in construction. Usually, in connections of this type, two individual columns are used which embrace the section of the beam. Connections using glued-in rods in this regard may be more effective, as only an angular cut off at the ends of the beams will be required.

To increase the shear capacity of the connection the addition of a part to the column as a notch could help. The inclusion of these notches can be easily implemented as the pillars are designed with a circular cladding between the beam and the column. This angular cladding has only an architectural function, taking advantage of this, the notches can be covered by these wooden panels.

Figure 8.1 Beam to column connection with the notched part.

- *Study on timber beam-to-beam connections*

Beam to beam connections are considered complex and require more detailing in the design. In the case study, the beams have 4 or 5-members intersections, with the members intersecting at varying angles and with different cross-sections.

As a design strategy, it is helpful to keep the members with the highest forces as continuous members. In this way, the highest internal forces will still be transferred by the timber members, while on intersecting members with smaller internal forces, they will have their forces transferred to their segment through connections. In this way, the connections, which are weak points in the structure, will have lower loading requests.

The most critical point of beam-to-beam connections is the determination of the connection method that crosses the continuous beams. The solution presented in the connection (2) was very convenient because it consists of small members that can be inserted in a hidden way, and fulfils the architectural limitation proposed for the project. On the other hand, the solution presented for the connection (1), in which an orthogonal metal plate was proposed, exceeded the dimensions of the beams. Another fact that should be mentioned is that due to the architectural proposal the beams are sideways inclined, this is a factor that for the final design of the connections is very challenging.

It is very important to understand how the forces are transferred between the members and under which assumption. The detail in the connections should follow these assumptions. In case the intersected member transfers only shear forces as a pinned connection, the mechanism in the connections should allow this behaviour and in the same time avoiding any stresses that could cause failure in the continuous beam.

- *Study of the most optimized design solution*

The design of the connections presented in Chapter 6 qualify for the preliminary design stage. Further research should be made concerning the optimization of the solutions aiming mainly at the assembly easiness in the construction. Other aspects related to stiffness and capacity should be improved through an on-going design process seeking to accomplish a final proposal. In other words, the determination of the connections can only be considered as the result when all connections, with their respective unique detailing, are added and analysed in the global structural model. It is noticeably that this process can be time-consuming.

After analysing different alternatives for connections of large timber members, as a conclusion the following alternatives proved to be the most effective and are worth being optimized in matters related to stiffness and uniformization between different connection points:

Beam to column connections: Glued in rods and dowel type fork joint.

Beam-to-beam connections: Glued-in rods and multi-plane slotted in metal plates.

- *Study of the influence of the stiffness on a global level*

The design at the global level is an important subject that needs to be further investigated. The process of studying the influence of the stiffness of connections on a global level is the last step of the design. It was not possible to perform the complete study in this thesis due to the time this process takes. As mentioned in the discussion, the major goal in the design is to provide the connections with an adequate and similar stiffness, so that there is a rational distribution of forces between the timber members and the connection.

The final design proposes presented in Chapter 6 can be considered conservative when following the stiffness calculation methods given in EC5. The great discrepancy observed in the values of the stiffness can be explained by considering that most of the methods to calculate the stiffness are based on the elastic portion of the force-deformation curve, which neglects the elastic-plastic portion.

8.1 Further studies

The sequencing of this study is essential to accomplish a final design proposal for the free-form timber roof structure connections in which the case study has addressed. Although the work has determined the type of connections that most appropriately perform in large timber members, the optimization of the proposed connections still remains to be achieved. Due to the lack of standardization regarding the more advanced methods of connecting timber structures, a more wide-reaching study may be carried out in order to identify solutions that might prove to be more efficient especially in terms of stiffness. For example, connections can be reinforced with self-tapping screws and threaded rods, and the spacing can be reduced and allowing more fasteners. This way the connection can be stiffer.

Different glued-in rods design methods are available in scientific publications, so there are still other ways to determine the capacities and limitations of connections using this type of mechanism, which have the potential to present themselves as more efficient. For example, it was observed during the study that the determination of the axial stiffness of the rods diverged from sources that determine it by means of probatory tests or by FEM-analysis. Another pertinent observation, which affects the rotational stiffness of the connections, is the fact that the contact surface of the timber in the compression area was not included in the calculations of this study. Therefore, further research on this topic may improve the design.

The slotted-in plates designed connections have individual elements that transfer the shear forces and bending moments, it might be interesting to study further how these elements interact if only one connector member is used instead.

The design proposals submitted in this master's thesis have a theoretical basis and can be followed in order to achieve the necessary efficiency in terms of serviceability limit state. The beam-to-beam connection also has plenty of room for other design interpretations and proposals in the continuous beam crossing region. Therefore, this is a topic worth further study and development. It is recommended that these parts are modelled in a FEM-analysis software for greater understanding and certainty of design efficiency. It is very important to have strict control of the influence of the intersecting connections on the behaviour of the continuous beam.

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Annex A – Material properties

The following tables demonstrate the properties of the materials that were used in the design of the connections presented in this master thesis.

Table A.1 Glulam 30c material properties according to according to SS-EN 14080:2013.

Strength values		
Bending parallel to grain $f_{m.k}$	30	[MPa]
Tension parallel to grain $f_{t.0.k}$	19.5	[MPa]
Tension perpendicular to grain $f_{t.90.k}$	0.5	[MPa]
Compression parallel to grain $f_{c.0.k}$	24.5	[MPa]
Compression perpendicular to grain $f_{c.90.k}$	2.5	[MPa]
Shear $f_{v.k}$ (shear and torsion)	3.5	[MPa]
Rolling shear $f_{r.k}$	1.2	[MPa]
Stiffness values for capacity analysis		
Elastic modulus $E_{0.05}$	10.800	[MPa]
Elastic modulus $E_{90.05}$	250	[MPa]
Shear modulus G_{05}	540	[MPa]
Stiffness values for deformation calculations. mean values		
Elastic modulus $E_{0.mean}$	13.000	[MPa]
Elastic modulus $E_{90.mean}$	300	[MPa]
Shear modulus G_{mean}	650	[MPa]
Density		
Density ρ_k	390	[kg/m3]
Density ρ_{mean}	430	[kg/m3]

Table A.2 Material factors utilized in the calculations, according to SS-EN 1995-1-1:2004.

Service class	2
Load duration class	Medium
$k_{.mod}$	0.80
γ_{MGL}	1.25
$k_{.h.tension}$	1.10
$k_{.cr}$	0.67
$k_{.h.bending}$	1.10

Table A.3 Yield strength and ultimate tensile strength of dowel type fasteners according to SS-EN 1993-1-8:2005.

Dowel Class	4.6	4.8	5.6	5.8	6.8	8.8	10.9
f_y (N/mm^2)	240	320	300	400	480	640	900
f_u (N/mm^2)	400	400	500	500	600	800	1000

Table A.4 Nominal values of yield strength and ultimate tensile strength for hot-rolled structural steel according to SS-EN 1993-1-8:2005.

Steel grade according to EN 10025-4	Nominal thickness of the element t [mm]			
	$t \leq 40$ mm		$40 \text{ mm} < t \leq 80$ mm	
	f_y [N/mm2]	f_u [N/mm2]	f_y [N/mm2]	f_u [N/mm2]
S 275 M/ML	275	370	255	360
S 355 M/ML	355	470	335	450
S 420 M/ML	420	520	390	500
S 460 M/ML	460	540	430	530

Table A.5 Load combination I ULS-a

Included loads	**Factor**
Self-weight	1.35
Roof self-weight	1.35

Table A.6 Load combination ULS-b

Included loads	**Factor**
Self-weight	1.20
Roof self-weight	1.20
Snow-as accumulated load	1.50
Snow	1.50

Table A.7 Load combination V SLS-quasi-permanent

Included loads	**Factor**
Self-weight	1.00
Roof self-weight	1.00
S-accumulated	0.10
Snow	0.10

Annex B - Chapter 5 connections

B.1 Connection on tension

B.1.1 Glued-in rods design steps

1. Design capacity of rods, according to “Limträhandbok del 3” page 74 (Svenskt trä, 2016).

The design capacity of a single rod is the minimum value between the tensile capacity of the rod $F_{t.Rk.rod}$ and the timber pull-out capacity $F_{t.Rk.timber}$.

$$f_{ax.k} = 5.50\ MPa \tag{69}$$

$f_{ax.k}$- *Withdrawal capacity of the rod.*

$$F_{t.Rk.rod} = 0.6 f_{u.b} \cdot A_s \tag{70}$$

$F_{t.Rk.rod}$ - *Tensile capacity of the rod* [kN].
A_s - *Steel cross-sectional area.*

$$F_{t.Rk.timber} = \pi \cdot l_i \cdot f_{ax.k} \cdot k_1 \cdot \kappa_1 \tag{71}$$

$R_{t.k.timber}$ - *Pull-out capacity.*

l_i - *Glued length of the rod (inlimmade längden).*

κ_1 - *Weather factor. Limträhandbok del 3, Table 13.23. (Svenskt trä, 2016).*

k_1 - *Reduction factor for shear strength as a function of the glued length (li) Limträhandbok del 3, Table 13.24. (Svenskt trä, 2016).*

The design rod capacity is the lowest value between $F_{t.Rk.rod}/\gamma_{Steel}$ (γ_{Steel}=1.2) and $F_{t.Rk.timber} \cdot k_{mod}/\gamma_{m.GL}$.

The number of rods is governed by the spacing of the rods, as shown in Figure B.1 and is calculated according to “Limträhandbok del 3”, Table 13.24 (Svenskt trä, 2016)

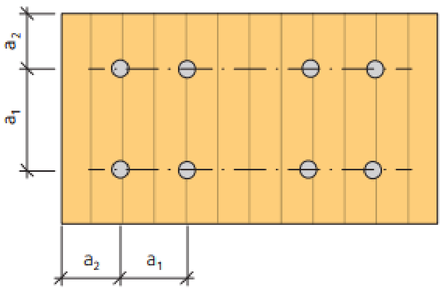


Figure B.1 Rods spacing (Svenskt trä, 2016).

The total resistance of the connection is:

$$F_{t.Rd} = n_{rods} \cdot \min\left(F_{t.Rd.rod}. F_{t.Rd.timber}\right) \tag{72}$$

2. Brittle failure - Net tension capacity of the timber cross-section $F_{t.net.Rk}$.

The tensile capacity parallel to the grain of the timber net cross-section must resist the applied tensile force on the connection.

$$F_{t.net.Rd} = f_{t.0.d}\, A_{timber.net} \tag{73}$$

The applied tensile force $F_{t.0.Ed}$ is compared to $F_{t.net.Rd}$ ($F_{t.0.Ed} = F_{t.Rd}$).

$$\frac{F_{t.0.Ed}}{F_{t.net.Rd}} \leq 1 \tag{74}$$

3. Brittle failure – Single rod-end effect $F_{t.Aef.Rd}$.

The affected timber area at the rod end must be checked according to "Design of bonded-in-rods" (CEN et al., 2019). The affected area A_{ef} is a square with each of its sides measuring 6d, where d is the rod's diameter, see Figure B.2.

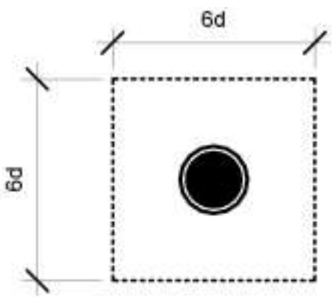


Figure B.2 Affected timber area at rod-end.

Acknowledging it, the resistance against the rod-end effect is given by:

$$\frac{F_{t.0.Ed}}{F_{t.Aef.Rd}} \leq 1 \tag{75}$$

Where,

$$F_{t.Aef.Rd} = \frac{f_{t.0.k}\, A_{ef}}{\gamma_{Steel}} \tag{76}$$

4. Stiffness K_{total}.

The stiffness of one single rod $K_{ser.rod}$ is given in Table 4.2 :

$$K_{ser.rod} = 0.004 \cdot d^{1.8} \cdot \rho_{mean}^{1.5} \tag{77}$$

Total stiffness as the rods work in parallel:

$$K_{total} = n_{rods} \cdot K_{ser.rod} \tag{78}$$

B.1.2 External metal plate design steps

1. Design capacity of the bolted plate $F_{t.Rd}$.

The single capacity of a bolt is calculated by the most critical failure mode according to "Johannsen's Theory" which is demonstrated in Section 4.1.2. Additionally, the effect of the Rope Effect is added., as in Section 4.1.3.

An important procedure is multiplying the final resistance by 2 because there are two shear planes in this case.

In the thin plate case, the failure modes are given in EC5 8.2.3(3) (CEN, 2004):

$$F_{v.Rk} = 2 \cdot min \begin{cases} 0.5\, f_{h.2.k}\, t_2\, d \\ 1.15\, \sqrt{2\, M_{y.Rk}\, f_{h.2.k}\, d} + \dfrac{F_{ax.Rk}}{4} \end{cases} \tag{79}$$

In the thick plate case, the failure modes are given in EC5 8.2.3(3) (CEN, 2004):

$$F_{v.Rk} = 2 \cdot min \begin{cases} 0.5\, f_{h.2.k}\, t_2\, d \\ 2.30\, \sqrt{2\, M_{y.Rk}\, f_{h.2.k}\, d} + \dfrac{F_{ax.Rk}}{4} \end{cases} \tag{80}$$

Where,

$f_{h.2.k}$ - *Embedment strength of the timber member.*
t_2 - *Thickness of the timber member.*
$M_{y.Rk}$ – *Fastener yielding moment.*

The number of bolts is governed by the spacing of the bolts, as shown in Figure B.3, and is calculated according to EC5 8.5.1.1(3) Table 8.4. The effective number (n_{ef}) of bolts in one row parallel to the grain is given by EC5 8.5.1.1(4) (CEN, 2004).

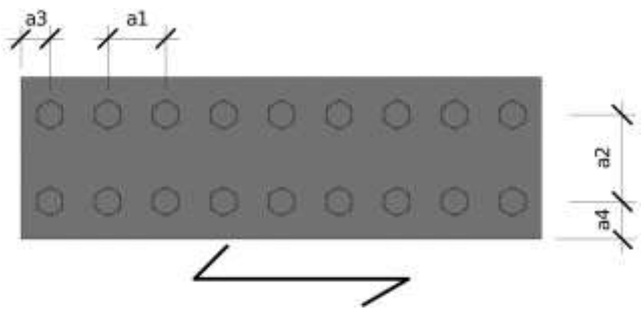


Figure B.3 Spacing of a bolted connection.

The design resistance of the connection is:

$$F_{t.Rd} = k_{mod} \cdot \frac{[n_{rows} \cdot n_{eff} \cdot \min(F_{v.Rk})]}{\gamma_{m.GL}} \text{ [kN]} \tag{81}$$

Where,

n_{rows}- *Number of rows*
n_{eff} – *Number of bolts in a row considering the group effect. EC5 Eq. (8.34).*
$M_{y.Rk}$ – *Fastener yielding moment.*

2. Tensile capacity of the plate $F_{t.plate.Rd}$:

Both thick and thin plates tensile capacity must be checked.

$$F_{t.plate.Rd} = \frac{f_{y.b} \cdot A_{net}}{\gamma_{steel}} \tag{82}$$

$$A_{net} = (H - n_{rows} \cdot d) \cdot t_{plate} \tag{83}$$

The applied tensile force per plate, $F_{GL30c.t.0.Rd}/2$, is compared to $F_{t.plate.Rd}$.

$$\frac{F_{GL30c.t.0.Rd}/2}{F_{t.plate.Rd}.} \leq 1 \tag{84}$$

3. Brittle failure – Block shear $F_{bs.Rd}$:

The block shear failure mode is calculated according to EC5 Annex A (CEN, 2004) and explained in Section 4.2.2. The calculations for block shear should be consistent with the relevant failure mode.

4. Stiffness:

The stiffness of one single bolt is calculated according to Table 4.2. For two shear planes, the value is multiplied by two, moreover, since it is a timber-to-steel connection it is multiplied by two one more time.

$$K_{ser} = (2 * 2) * \rho_m^{1,5}.d/23 \tag{85}$$

The stiffness in series is given by Equation (25) and the stiffness in parallel is given by Equation (26):

$$K_{total} = \left(\frac{n_{bolts.series}}{K_{ser}}\right)^{-1} \cdot n_{bolts.parallel} \tag{86}$$

B.1.3 Slotted-in metal plate design steps

1. Design capacity of the dowelled slotted-in plate $F_{t.Rd}$.

The single capacity of a dowel is calculated by the most critical failure mode according to "Johansson Theory" which is demonstrated in Section 4.1.2. Dowels have no contribution from rope effect since they do not have axial capacity as demonstrated in Section 3.4.4 and in Section 4.1.3.

An important procedure is multiplying the final resistance by 2 because there are two shear planes in this case.

The failure modes are given from EC5 8.2.3(3), (CEN, 2004):

$$F_{v.Rk} = 2 \cdot \min \begin{cases} f_{h.1.k}\, t_1\, d \\ f_{h.1.k}\, t_1\, d \left[\sqrt{2 + \dfrac{4M_{y.Rk}}{f_{h.1.k}\, d\, t_1^2}} - 1 \right] \\ 2.3 \sqrt{M_{y.Rk}\, f_{h.1.k}\, d} \end{cases} \tag{87}$$

Where,

$f_{h.1.k}$ - *Embedment strength of the timber member.*
t_1 - *Thickness of the smaller side timber member.*
t_2 - *Thickness of the timber middle member.*
$M_{y.Rk}$ – *Fastener yielding moment.*

The number of dowels is governed by the spacing of the dowels and is calculated according to EC5 8.5.1.1(3) Table 8.5. The effective number (n_{ef}) of bolts in one row parallel to the grain is given by EC5 8.5.1.1(4) (CEN, 2004).

The design resistance of the connection is:

$$F_{t.Rd} = \frac{[n_{rows} \cdot n_{ef} \cdot \min(F_{v.Rk})]}{\gamma_{m.GL}} \text{ [kN]} \tag{88}$$

2. Tensile capacity of the plate $F_{t.plate.Rd}$:

$$F_{t.plate.Rd} = \frac{f_{y.b} \cdot A_{net}}{\gamma_{steel}} \tag{89}$$

$$A_{net} = (H - n_{rows} \cdot d) \cdot t_{plate} \tag{90}$$

The applied tensile force per plate, $F_{t.t.Rd}$, is compared to $F_{t.plate.Rd}$.

$$\frac{F_{GL30c.t.0.Rd}}{F_{t.plate.Rd}.} \leq 1 \tag{91}$$

3. Brittle failure – Block shear $F_{bs.Rd}$

The block shear failure mode is calculated according to EC5 Annex A (CEN, 2004) and explained in Section 4.2.2. The calculations for block shear should be consistent with the relevant failure mode.

4. Stiffness K_{total}:

The stiffness of one single dowel is calculated according to Table 4.2.

$$K_{ser} = \rho_m^{1,5}.d/23 \tag{92}$$

The stiffness in series is given by equation (25) and the stiffness in parallel is given by equation (26):

$$K_{total} = \left(\frac{n_{dowel.series}}{K_{ser}}\right)^{-1} \cdot n_{dowel.parallel} \tag{93}$$

B.1.4 Double slotted-in metal plate design steps

1. Design capacity of each slotted-in plate $F_{t.Rd}$.

The calculation of the tensile resistance of the connection now possessing 4 shear planes, is dependent on the dowels shear resistance which is given by the resistance of the center block with a thickness t_2 and the lateral parts resistance with a thickness t_1. The center block has two shear planes and is analyzed as a thick plate double shear steel-to-timber joint. The control of the required thickness $t_{2.req} \geq 1.15 \cdot 4\sqrt{\frac{M_{y.Rk}}{f_{h.k}\,d}}$ assures that the dowel will form 3 plastic hinges and the failure will not occur on the timber member. The lateral parts will behavior as a central member of a double shear connection, that means that t_1 has also to be controlled to avoid failure of the timber block and to allow a plastic hinge to be formed in the dowel, $\sqrt{2}\sqrt{\frac{M_{y.Rk}}{f_{h.k}\,d}} < t_1 \leq 1.15 \cdot 4\sqrt{\frac{M_{y.Rk}}{f_{h.k}\,d}}$. The following equations give $F_{v.Rk}$.

$$F_{v.Rk.center} = 2 \cdot 1.15 \cdot \sqrt{2}\sqrt{2M_{y.Rk}\,f_{h.1.k}\,d} \tag{94}$$

$$F_{v.Rk.lateral} = 2 \cdot \min \begin{cases} f_{h.1.k}\,t_1\,d\left[\sqrt{2 + \frac{4M_{y.Rk}}{f_{h..k}\,d\,t_1^2}} - 1\right] \\ 1.15\sqrt{2}\sqrt{2M_{y.Rk}\,f_{h.1.k}\,d} \end{cases} \tag{95}$$

In both cases, the shear resistance is multiplied by 2 accounting the number of shear planes. Finally:

$$F_{v.Rk} = F_{v.Rk.center} + F_{v.Rk.lateral} \tag{96}$$

The number of dowels is governed by the spacing of the dowels and is calculated according to EC5 8.5.1.1(3) Table 8.5. The effective number (n_{ef}) of bolts in one row parallel to the grain is given by EC5 8.5.1.1(4) (CEN, 2004).

The design resistance of the connection is:

$$F_{t.Rd} = k_{mod.} \frac{[n_{row} \cdot n_{ef} \cdot F_{v.Rk}]}{\gamma_{m.GL}} \text{ [kN]} \tag{97}$$

2. Tensile capacity of the plate $F_{t.plate.Rd}$:

$$F_{t.plate.Rd} = \frac{f_{y.b} \cdot A_{net}}{\gamma_{steel}} \tag{98}$$

$$A_{net} = (H - n_{vertical} \cdot d) \cdot t_{plate} \tag{99}$$

The applied tensile force per plate, $F_{t.t.Rd}$, is compared to $F_{t.plate.Rd}$.

$$\frac{F_{GL30c.t.0.Rd}}{2F_{t.plate.Rd}} \leq 1 \tag{100}$$

3. Brittle failure – Block shear $F_{bs.Rd}$:

The block shear failure mode is calculated according to EC5 Annex A (CEN, 2004) and explained in Section 1. However, in the case of a double slotted-in plate, the block shear capacity must be calculated for the center block and for the lateral parts (Bocquet et al., 2018).

4. Stiffness K_{total}:

The stiffness of one single dowel is calculated according to Table 4.2 and multiply by 4 number of shear planes and 2 as the connection is between timber and metal plate.

$$K_{ser} = (4 * 2)\rho_m^{1,5}.d/23 \tag{101}$$

The stiffness in series is given by equation (25) and the stiffness in parallel is given by equation (26):

$$K_{total} = \left(\frac{n_{dowel.series}}{K_{ser}}\right)^{-1} \cdot n_{dowel.parallel} \tag{102}$$

B.2 Connections in shear

B.2.1 Slotted-in metal plate design steps

1. Design capacity of the bolted plate $F_{t.Rd}$:

The single capacity of a bolt is calculated by the most critical failure mode according to "Johannsen Theory" which is demonstrated in Section 4.1.2. Additionally, the effect of the Rope Effect, as in Section 4.1.3, is added.
The main difference from splice working on tension is calculating the embedding strength $f_{h.90.k}$ with α = 90 degrees from the grain as in Section 4.4.1.

An important procedure is multiplying the final resistance by 2 because there are two shear planes in this case.

The failure modes are given from EC5 8.2.3(3), (CEN, 2004):

$$F_{v.Rk} = 2 \cdot \min \begin{cases} f_{h.1.k}\, t_1\, d \\ f_{h.1.k}\, t_1\, d \left[\sqrt{2 + \dfrac{4M_{y.Rk}}{f_{h.1.k}\, d\, t_1^2}} - 1 \right] \\ 2.3 \sqrt{M_{y.Rk}\, f_{h.1.k}\, d} \end{cases} \quad (103)$$

Where,

$f_{h.1.k}$ - *Embedment strength of the timber member at 90 degrees from the grain.*
$M_{y.Rk}$ – *Fastener yielding moment.*

The number of bolts is governed by the spacing of the bolts, as shown in Figure B.4 below, and is calculated according to EC5 8.5.1.1(3) Table 8.5. Since the force acting perpendicular to the grain, no row effect is considered.

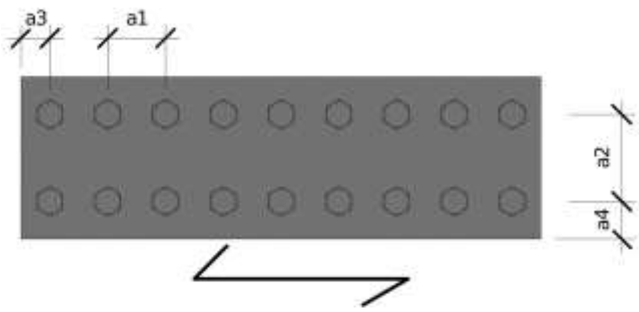


Figure B.4 Spacing of a bolted connection.

The design resistance of the connection is:

$$F_{t.Rd} = \frac{[n_{rows} \cdot n_{in\,row} \cdot \min(F_{v.Rk})]}{\gamma_{m.GL}} \text{ [kN]} \quad (104)$$

Where:
n_{rows}- *Number of rows.*
$n_{in\ row}$ – *Number of bolts in a row.*
$M_{y.Rk}$ – *Fastener yielding moment.*

2. Splitting failure:

As the force acting perpendicular to the grain, the possibility of splitting caused by tension perpendicular to the grain is checked as in EC5 8.1.4.

$$F_{90.Rk} = 14 \cdot b \cdot w \cdot \sqrt{\frac{h_c}{\left(1-\frac{h_c}{h}\right)}} \ \text{[kN]} \quad (105)$$

Where:
$F_{90.Rk}$ - *Characteristic splitting capacity.*
h_c - *Loaded edge distance to the center of the most distant fastener.*
h - *Timber member height.*
w - Modification factor =1.
b - Timber thickness

The characteristic splitting capacity should be larger than the joint characteristic capacity to avoid brittle failure.

$$F_{90.Rd} > F_{t.Rk} \quad (106)$$

3. Stiffness:

The stiffness of one single bolt is calculated according to Table 4.2. For two shear planes, the value is multiplied by two, moreover, since it is a timber-to-steel connection it is multiplied by two one more time.

$$K_{ser} = (2 * 2) * \rho_m^{1,5}.d/23 \quad (107)$$

The stiffness in series is given by Equation (25) and the stiffness in parallel is given by Equation (26):

$$K_{total} = \left(\frac{n_{bolts.series}}{K_{ser}}\right)^{-1} \cdot n_{bolts.parallel} \quad (108)$$

B.2.2 External metal plate design steps

1. Design capacity of the bolted plate $F_{t.Rd}$:

The single capacity of a bolt is calculated by the most critical failure mode according to "Johanssen's Theory" which is demonstrated in Section 4.1.2. Additionally, the effect of the Rope Effect, as in Section 4.1.3, is added. The main difference is calculating the embedding strength $f_{h.90.k}$ with α = 90 degree from the grain as in Section 4.4.1 .

An important procedure is multiplying the final resistance by 2 because there are two shear planes in this case.
The embedding

In the thin plate case, the failure modes are given in EC5 8.2.3(3) (CEN, 2004):

$$F_{v.Rk} = 2 \cdot min \begin{cases} 0.5\, f_{h.2.k}\, t_2\, d \\ 1.15\, \sqrt{2\, M_{y.Rk}\, f_{h.2.k}\, d} + \dfrac{F_{ax.Rk}}{4} \end{cases} \tag{109}$$

In the thick plate case, the failure modes are given in EC5 8.2.3(3) (CEN, 2004):

$$F_{v.Rk} = 2 \cdot min \begin{cases} 0.5\, f_{h.2.k}\, t_2\, d \\ 2.30\, \sqrt{2\, M_{y.Rk}\, f_{h.2.k}\, d} + \dfrac{F_{ax.Rk}}{4} \end{cases} \tag{110}$$

Where,

$f_{h.2.k}$ - *Embedment strength of the timber member at 90 degrees.*
t_2 - *Thickness of the timber member.*
$M_{y.Rk}$ – *Fastener yielding moment.*

The number of bolts is governed by the spacing of the bolts, as shown in Figure B.5, and is calculated according to EC5 8.5.1.1(3) Table 8.4. Since the force acting parallel to the grain no row effect is considered.

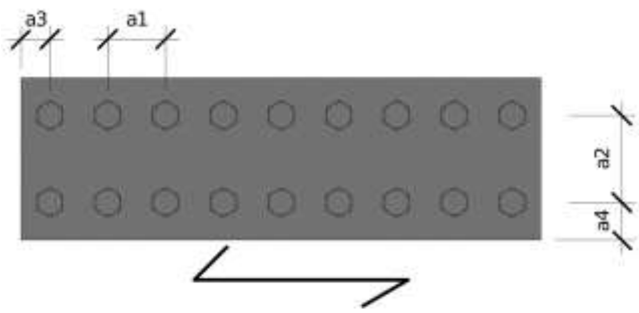


Figure B.5 Spacing of a bolted connection.

The design resistance of the connection is:

$$F_{t.Rd} = \frac{[n_{rows} \cdot n_{\,in\,row} \cdot \min{(F_{v.Rk})}]}{\gamma_{m.GL}}\ [\text{kN}] \tag{111}$$

Where:

n_{rows}- *Number of rows.*
$n_{in\ row}$ *– Number of bolts in a row.*
$M_{y.Rk}$ *– Fastener yielding moment.*

2. Splitting failure:

As the force acting perpendicular to the grain, the possibility of splitting caused by tension perpendicular to the grain is checked as in EC5 8.1.4.

$$F_{90.Rk} = 14 \cdot b \cdot w \cdot \sqrt{\frac{h_c}{\left(1-\frac{h_c}{h}\right)}} \quad [\text{kN}] \tag{112}$$

Where:
$F_{90.Rd}$ - *The characteristic splitting capacity.*
h_c - *Loaded edge distance to the center of the most distant fastener.*
h - *Timber member height.*
w - Modification factor =1.
b - *Timber thickness.*

The characteristic splitting capacity should be larger than the joint characteristic capacity to avoid brittle failure.

$$F_{90.Rk} > F_{t.Rk} \tag{113}$$

3. Stiffness:

The stiffness of one single bolt is calculated according to Table 4.2. For two shear planes, the value is multiplied by two, moreover, since it is a timber-to-steel connection it is multiplied by two one more time.

$$K_{ser} = (2 * 2) * \rho_m^{1,5}.d/23 \tag{114}$$

The stiffness in series is given by Equation (25) and the stiffness in parallel is given by Equation (26):

$$K_{total} = \left(\frac{n_{bolts.series}}{K_{ser}}\right)^{-1} \cdot\ n_{bolts.parallel} \tag{115}$$

B.3 Connection in bending

B.3.1 Glued-in rods design steps

1. Design capacity of rods, according to “Limträhandbok del 3” page 74 (Svenskt trä, 2016). And explained in B.1.1 Glued-in rods design steps

The number of rods is only one layer in the top and the bottom governed by the spacing is calculated according to “Limträhandbok del 3”, Table 13.24 (Svenskt trä, 2016)

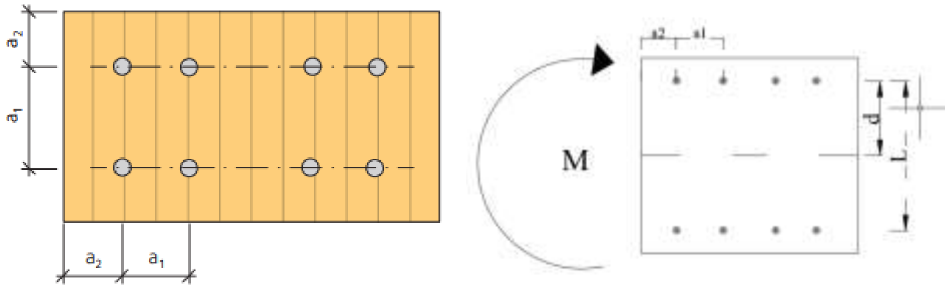


Figure B.1 Rods spacing (Svenskt trä, 2016).

The total moment resistance of the connection is:

$$M_{Rd} = n_{rods} \cdot \min(F_{t.Rd.rod}.F_{t.Rd.timber}) \cdot L \tag{116}$$

L – Lever arm between the rods in tension and compression

2. Brittle failure - Net tension capacity of the timber cross-section $F_{t.net.Rk}$.

The tensile capacity parallel to the grain of the timber net cross-section must resist the applied tensile force on the connection.

$$F_{t.net.Rd} = f_{t.0.d}\, A_{timber.net} \tag{117}$$

The applied tensile force $F_{t.0.Ed}$ is compared to $F_{t.net.Rd}$ ($F_{t.0.Ed} = F_{t.Rd}$).

$$\frac{F_{t.0.Ed}}{F_{t.net.Rd}} \leq 1 \tag{118}$$

3. Brittle failure – Single rod-end effect $F_{t.Aef.Rd}$.

The affected timber area at the rod end must be checked according to “Design of bonded-in-rods” (CEN et al., 2019). The affected area A_{ef} is a square with each of its sides measuring 6d, where d is the rod’s diameter, see Figure B.2.

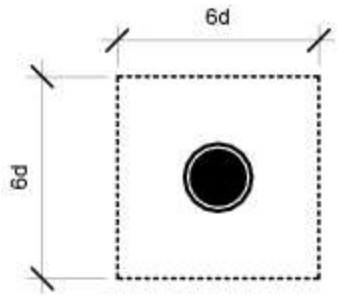


Figure B.2 Affected timber area at rod-end.

Acknowledging it, the resistance against the rod-end effect is given by:

$$\frac{F_{t.0.Ed}}{F_{t.Aef.Rd}} \leq 1 \tag{119}$$

Where,

$$F_{t.Aef.Rd} = \frac{f_{t.0.k}\, A_{ef}}{\gamma_{Steel}} \tag{120}$$

4. Rotational stiffness $K_{r.total}$.

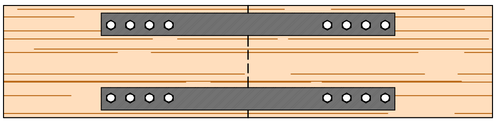
The axial stiffness of one single rod $K_{ser.rod}$ is given in:

$$K_{ser.rod} = 0.004 \cdot\ d^{1.8} \cdot\ \rho_{mean}^{1.5} \tag{121}$$

Rotational stiffness as the rods work in parallel:

$$K_{total} = n_{rods} \cdot\ K_{ser.rod} \cdot d^2 \tag{122}$$

B.3.2 External metal plates with bolts design steps



1. Design moment capacity of the bolted plate $F_{t.Rd}$.

The single capacity of a bolt is calculated by the most critical failure mode according to "Johannsen's Theory" which is demonstrated in Section 4.1.2. Additionally, the effect of the Rope Effect is added., as in Section 4.1.3.

An important procedure is multiplying the final resistance by 2 because there are two shear planes in this case.

In the thin plate case, the failure modes are given in EC5 8.2.3(3) (CEN, 2004):

$$F_{v.Rk} = 2 \cdot min\begin{cases} 0.5\, f_{h.2.k}\, t_2\, d \\ 1.15\, \sqrt{2\, M_{y.Rk}\, f_{h.2.k}\, d} + \dfrac{F_{ax.Rk}}{4} \end{cases} \tag{123}$$

In the thick plate case, the failure modes are given in EC5 8.2.3(3) (CEN, 2004):

$$F_{v.Rk} = 2 \cdot min\begin{cases} 0.5\, f_{h.2.k}\, t_2\, d \\ 2.30\, \sqrt{2\, M_{y.Rk}\, f_{h.2.k}\, d} + \dfrac{F_{ax.Rk}}{4} \end{cases} \tag{124}$$

Where,

$f_{h.2.k}$ - *Embedment strength of the timber member.*
t_2 - *Thickness of the timber member.*
$M_{y.Rk}$ – *Fastener yielding moment.*

The number of bolts in one row of the plate is governed by the spacing of the bolts a1 and a3, as shown in Figure B.3, and is calculated according to EC5 8.5.1.1(3) Table 8.4. The effective number (n_{ef}) of bolts in one row parallel to the grain is given by EC5 8.5.1.1(4) (CEN, 2004).

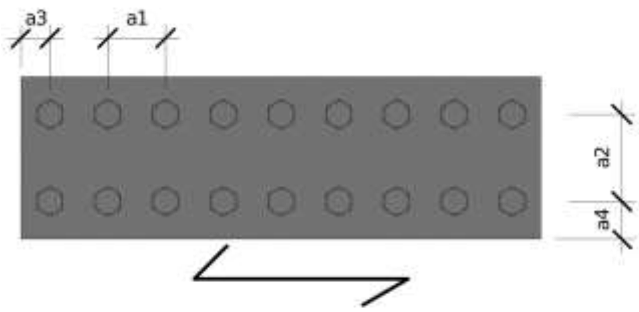


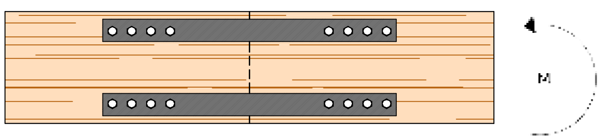
Figure B.3 Spacing of a bolted connection.

The axial design resistance of one plate subjected to compression or tension is:

$$F_{t.Rd} = k_{mod} \cdot \frac{n_{eff} \cdot \min(F_{v.Rk})}{\gamma_{m.GL}} \text{ [kN]} \tag{125}$$

The design moment capacity of the connection is the axial capacity of the plate multiply by the distance between the center of the compression and tension plates:

$$M_{Rd} = F_{t.Rd} \cdot L \text{ [kN]} \tag{126}$$



Tensile capacity and compression capacity of the plates are checked $F_{t.plate.Rd}$based on EC3

2. Brittle failure – Block shear $F_{bs.Rd}$:

The block shear failure mode is calculated according to EC5 Annex A (CEN, 2004) and explained in Section 4.2.2. The calculations for block shear should be consistent with the relevant failure mode.

3. Stiffness:

The stiffness of one single bolt is calculated according to Table 4.2. For two shear planes, the value is multiplied by two, moreover, since it is a timber-to-steel connection it is multiplied by two one more time.

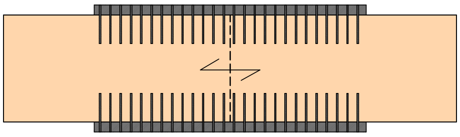
$$K_{ser} = (2 * 2) * \rho_m^{1,5}.d/23 \tag{127}$$

The stiffness in series is given by Equation (25):

$$K_{total} = \left(\frac{n_{bolts.series}}{K_{ser}}\right)^{-1} \cdot n_{bolts.parallel} \tag{128}$$

$$K_r = K_{total} \cdot 2 \cdot d^2 \text{ kN/rad} \tag{129}$$

B.3.3 Top and bottom steel nailed plates design steps



1. Design moment capacity of the nailed plate $F_{t.Rd}$.

The single capacity of a nail is calculated by the most critical failure mode according to "Johannsen's Theory" which is demonstrated in Section 4.1.2. Additionally, the effect of the Rope Effect is added., as in Section 4.1.3.

The plate thickness is 8 mm and the penetration length of the nails is 40 mm so the lateral capacity for nails is calculated for the thick plate.

In the thick plate case, the failure modes are given in EC5 8.2.3(3) (CEN, 2004):

$$F_{v.Rk} = 2 \cdot min \begin{cases} f_{h.k}\, t_1\, d \\ f_{h.k}\, t_1\, d \left[\sqrt{2 + \dfrac{2\, M_{y.Rk}}{f_{h.k}\ d t_1^2}} - 1\right] + \dfrac{F_{ax.Rk}}{4} \\ 2.30 \sqrt{2\, M_{y.Rk}\, f_{h.k}\, d} + \dfrac{F_{ax.Rk}}{4} \end{cases} \tag{130}$$

Where,

$f_{h.k}$ - Embedment strength of the timber member.
$M_{y.Rk}$ – Fastener yielding moment. EC5 8.14
$F_{ax.Rk}$ EC5 Eq (8.23a) EC5 8.15

The number of nails in one row of the plate is governed by the spacing of the nails a1 and a3, as shown in Figure B.3, and is calculated according to EC5 Table 8.. The effective number (n_{ef}) of nails in one row parallel to the grain is given by EC5 Equation EC5 8,17

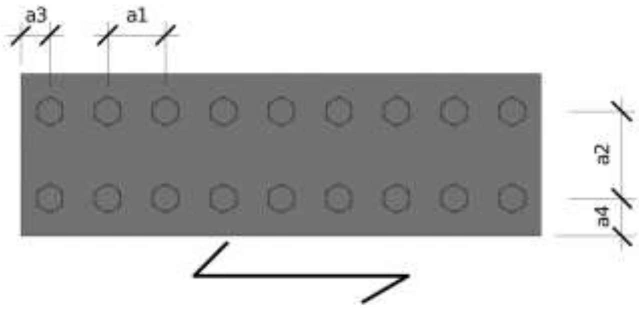


Figure B.3 Spacing of a nailed connection.

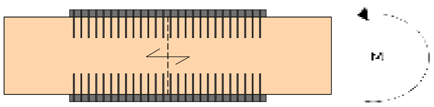
The axial design resistance of one plate subjected to compression or tension is:

$$F_{t.Rd} = k_{mod} \cdot \frac{n_{eff} \cdot \min(F_{v.Rk}) \cdot n_{row}}{\gamma_{m.GL}} \text{ [kN]} \quad (131)$$

The design moment capacity of the connection is the axial capacity of the plate multiply by the distance between the center of the compression and tension plates:

$$M_{Rd} = F_{t.Rd} \cdot \ L \text{ [kN]} \quad (132)$$

L- Height of the beam.



Tensile capacity and compression capacity of the plates are checked $F_{t.plate.Rd}$ based on EC3

2. Brittle failure – Plug shear $F_{bs.Rd}$:

The plug shear failure mode is checked according to EC5 Annex A (CEN, 2004) and explained in Section 4.2.2. The calculations for plug shear should be consistent with the relevant failure mode.

3. Rotational Stiffness:

The stiffness of one single bolt is calculated according to Table 4.2. For two shear planes, the value is multiplied by two, moreover, since it is a timber-to-steel connection it is multiplied by two one more time.

$$K_{ser} = (2) * \rho_m^{1,5}.d/23 \quad (133)$$

The stiffness in series is given by Equation (25) :

$$K_{total} = \left(\frac{n_{nail.series}}{K_{ser}}\right)^{-1} \cdot\ n_{bolts.parallel} \tag{134}$$

$$K_r = K_{total} \cdot 2\ \cdot d^2 \text{ kN/rad} \tag{135}$$

The plates are checked on tensile force and compression force according to EC3

Annex C – Chapter 6 connections

Detailed calculations covering all the structural concepts used in Section 6.

C.1 Connection 1 & 2

C.1.1 Slotted in metal plate

The calculation process presented in this section applies to connections 1 and 2 with slotted-in plates. To illustrate this design process, only connection 1 will be demonstrated to avoid repetition and ease the reading. For a complete analysis of the calculation process please refer to the Digital Annex B of this thesis. Figure C.1 shows the geometry of the connection.

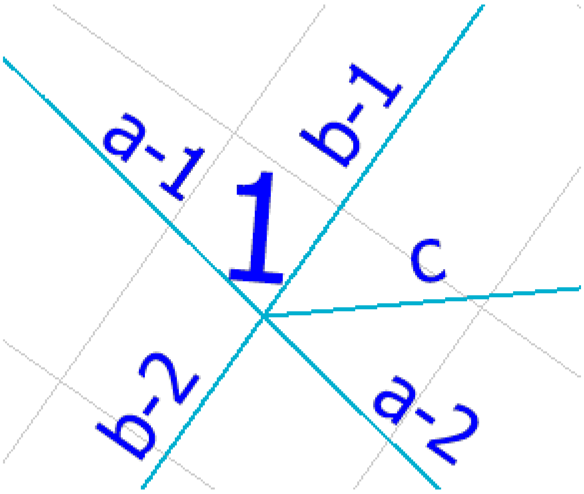


Figure C.1 Connection 1.

C.1.1.1 Connection 1 (A-1 to A-2) or (B-1 to B-2) - Shear

1) Forces acting on the connection

	V_z [kN]	
Forces	-51	[kN]
Load to grain α	90	[°]

2) Timber Cross Section

Width B	0,40	[m]
Height H	2,00	[m]
Area A	0,80	$[m^2]$

3) Fastener - Steel Dowel - 6 to 30 mm in diameter

Dowel class	4,80		
d	12,00	[mm]	
f_y	320,00	[mm]	Table 3.1 nominal value EN 1993-1-8

f_u	400,00	[MPa]	Table 3.1 nominal value EN 1993-1-8
Yielding moment M_{yk}	76745,42	[N*mm]	EC5 Eq. (8.30)
Embedment strength f_{h0k}	18,39	[MPa]	EC5 Eq. (8.31)
Thickness of timber part t_1	196,00	[mm]	

4) Plate

Steel grade	S 355 N/NL		EC3 Table 3.1
t_s	8,00	[mm]	
f_y	355,00	[MPa]	
f_u	490,00	[MPa]	
L	250,00	[mm]	
H	200,00	[mm]	

5) Single shear capacity

The single shear capacity is calculated according to equations 8.11 in EC5. The calculation as made for a steel plate of any thickness as the central member of a double shear connection.

$$F_{v.Rk} = \min \begin{cases} f_{h.1.k}\, t_1\, d & (f) \\ f_{h.1.k}\, t_1\, d \left[\sqrt{2 + \dfrac{4M_{y.Rk}}{f_{h.1.k}\, d\, t_1^2}} - 1 \right] & (g) \\ 2.3 \sqrt{M_{y.Rk}\, f_{h.1.k}\, d} & (h) \end{cases}$$

1 - (f)	43,26	[kN]
2 - (g)	18,47	[kN]
3 - (h)	9,47	[kN]

The minimum value is multiplied by 2 because there are two shear planes.

$F_{v.Rk} = 2 * 9{,}47$ = 18,93 kN

5.1) Spacing

The spacing is calculated according to Table 8.5 in EC5.

a_1	36,00	[mm]
a_2	36,00	[mm]
a_3	84,00	[mm]
a_4	48,00	[mm]

5.2) Group

n.row	3,28	3
Rows	3,89	3
Plate capacity	170,39	[kN]
Design plate capacity	109,05	[kN]

5.3) Number of fasteners

N parallel	3,00	[-]	
N_{eff}	3,00	[-]	EC5 Eq. (8.35)
Total number of fasteners	9,00	[-]	

6) Splitting due to tensile force in angle to the grain

As the force acting perpendicular to the grain, the possibility of splitting caused by tension perpendicular to the grain is checked as in EC5 8.1.4.

$$F_{90.Rk} = 14 \cdot b \cdot w \cdot \sqrt{\frac{h_c}{\left(1-\frac{h_c}{h}\right)}} \quad \text{[kN]}$$

Where:
$F_{90.Rd}$ - *The characteristic splitting capacity.*
h_c - *Loaded edge distance to the center of the most distant fastener.*
h - *Timber member height.*
w - Modification factor =1.
b - Timber thickness

w_{pl}	250,00	[mm]
h_e	1352,00	[mm]
h	1400,00	[mm]
b	392,00	[mm]
w	1,00	[-]
$F_{90.Rk}$	1089,80	[kN]
$F_{90.Rd}$	697,47	[kN]
Utilization ratio	7,82	[%]

The utilization ratio is calculated by:

$$\frac{F_{90.Rd}}{F_{V.Ed}/2}$$

$F_{V.Ed}$ *Design capacity of the connection.*

7) Stiffness

The stiffness of one single bolt is calculated according to Table 4.2. For two shear planes, the value is multiplied by two, moreover, since it is a timber-to-steel connection it is multiplied by two one more time.

$$K_{ser} = (2 * 2) * \rho_m^{1,5}.d/23 \tag{136}$$

The stiffness in series is given by Equation (25) and the stiffness in parallel is given by Equation (26):

$$K_{total} = \left(\frac{n_{bolts.series}}{K_{ser}}\right)^{-1} \cdot n_{bolts.parallel} \tag{137}$$

K_{ser}	18608.70	[N/mm]
N in series	3.00	[-]
N in parallel	3.00	[-]
K_{ser}Series	6202.90	[N/mm]
K_{total}	18608.70	[kN/m]

8) Generated moment from shear force

The calculation process follows Section 2.2.2.1.

Polar moment of inertia $I_P = \sum r_{i=}^2$ 26310 $[mm^2]$

The dowel subjected to the maximum force generated by the moment is the farthest away dowel, with local coordinates of (41 mm, 52 mm). The total applied force is 8.34 kN while the dowel resistance is 19,37 kN.

C.1.1.2 Connection 1 (A-1 to A-2) or (B-1 to B-2) – Bending

1) Forces acting on the connection

	M_y [kNm]	N [kN]
Forces	2319	-40

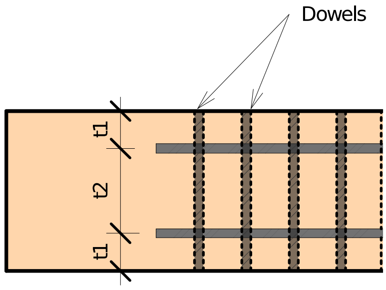
2) Timber Cross Section

Width B	0.40	[m]
Height H	2.00	[m]
Area A	0.80	$[m^2]$
Elastic sectional modulus W	0.27	$[m^3]$

3) Fastener - Steel Dowel - 6 to 30 mm in diameter

Dowel class	4.80		
d	20.00	[mm]	
f_y	480.00	[mm]	Table 3.1 nominal value EN 1993-1-8
f_u	600.00	[MPa]	Table 3.1 nominal value EN 1993-1-8
Yielding moment M_{yk}	434460.70	[N*mm]	EC5 Eq. (8.30)
Embedment strength f_{h0k}	25.58	[MPa]	EC5 Eq. (8.31)
Thickness of timber part $t_{1.min}$	41.21	[mm]	Limträhandbok 3 - P. 61- Table 13.8
Thickness of timber part $t_{2.req}$	134.04	[mm]	Limträhandbok 3 - P. 61- Table 13.8
Thickness control ($2\ t_{1.min} + t_{2.req} < B$)	216.46	[mm]	**Okay**

The control of the required thickness $t_{2.req} \geq 1.15 \cdot 4 \sqrt{\frac{M_{y.Rk}}{f_{h.k}\, d}}$ assures that the dowel will form 3 plastic hinges and the failure will not occur on the timber member, t_1 has also to be controlled to avoid failure of the timber block and to allow a plastic hinge to be formed in the dowel, $\sqrt{2}\sqrt{\frac{M_{y.Rk}}{f_{h.k}\, d}} < t_1 \leq 1.15 \cdot 4 \sqrt{\frac{M_{y.Rk}}{f_{h.k}\, d}}$.



4) Plate

Steel grade	S 355 N/NL	
t_s	16,00	[mm]
f_y	355,00	[MPa]
f_u	490,00	[MPa]
L	600,00	[mm]
H	600,00	[mm]

5) Single shear capacity

The dowel's shear resistance is given by the resistance of the center block with a thickness t_2 and the lateral parts resistance with a thickness t_1. The center block has two shear planes and is analyzed as a thick plate double shear steel-to-timber joint. The lateral parts will behavior as a central member of a double shear connection, the following equations give $F_{v.Rk}$.

$$F_{v.Rk.center} = 2 \cdot 1.15 \cdot \sqrt{2} \sqrt{2M_{y.Rk}\, f_{h.1.k}\, d}$$

$$F_{v.Rk.lateral} = 2 \cdot \min \begin{cases} f_{h.1.k}\, t_1\, d \left[\sqrt{2 + \dfrac{4M_{y.Rk}}{f_{h..k}\, d\, t_1^2}} - 1 \right] \\ 1.15\sqrt{2} \sqrt{2M_{y.Rk}\, f_{h.1.k}\, d} \end{cases}$$

In both cases, the shear resistance is multiplied by 2 accounting the number of shear planes. Finally:

$$F_{v.Rk} = F_{v.Rk.center} + F_{v.Rk.lateral}$$

$R._{k.center}$ (m)	68,59	[kN]
Design thickness $t_{d.1}$	72,00	[mm]
Design thickness $t_{d.2}$	256,00	[mm]
$R._{k.lateral}$ (g,h)	46,38	[kN]
$R._{k}$	114,97	[kN]

5.1) Spacing

The spacing is calculated according to Table 8.5 in EC5.

a_1	100,00	[mm]
a_2	60,00	[mm]
a_3	140,00	[mm]
a4	60,00	[mm]

5.2) Group

Dowels in a row - n	4,20	4	
N_{eff}	2,74		EC5 Eq. (8.34)
Number of rows $n_{.rows}$	9,00	9	
Plate capacity	2837,41	[kN]	
Design plate capacity	1815,94	[kN]	
Capacity in central part	1083,34	[kN]	
Capacity in one lateral part	366,30	[kN]	

6) Block shear

The block shear is checked for both the outer and central part. According to EC5 Annex 5. In multi-shear planes as in the case of this connection, the block shear capacity is calculated by the sum of the maximum value between the shear capacity and the tensile capacity per shear plane. Figure C.2 illustrates the design process.

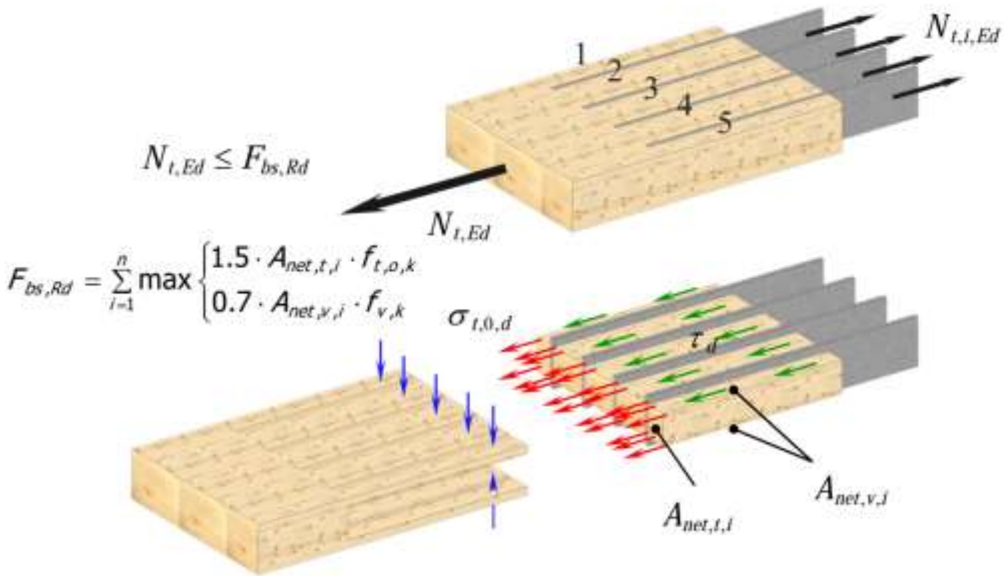


Figure C.2 Block shear capacity of multi-shear planes connection.

Block shear - both outer parts (g)		
$L_{net.t}$	300,00	[mm]
$A_{net.t}$	21600,00	[mm^2]
$L_{net.v}$	720,00	[mm]
t_{ef}	33,91	[mm]
$A_{net.v}$	132415,74	[mm^2]
$F_{bs.Rd}$ (shear)	415,26	[kN]
$F_{bs.Rd}$ (tension)	808,70	[kN]
$F_{bs.Rd}$	808,70	[kN]
Utilization	0,91	[-]

Block shear center part (m)		
$L_{net.t}$	300,00	[mm]
$A_{net.t}$	76800,00	[mm^2]
$L_{net.v}$	720,00	[mm]
$A_{net.v}$	184320,00	[mm^2]
$F_{bs.Rd}$ (shear)	1610,22	[kN]
$F_{bs.Rd}$ (tension)	1437,70	[kN]
$F_{bs.Rd}$	1610,22	[kN]
Utilization	0,67	[-]

$F_{bs.Rd.total}$	2418,92	[kN]
Utilization	0,75	[-]

7) Plate tension check

H_{net}	420,00	[mm]
A_{net}	6720,00	[mm^2]
Capacity	2385,60	[kN]
Design capacity	1988,00	[kN]
Utilization plate	0,46	[-]

$H_{.net}$ is the height of the plate disregarding the dowels.

8) Stiffness

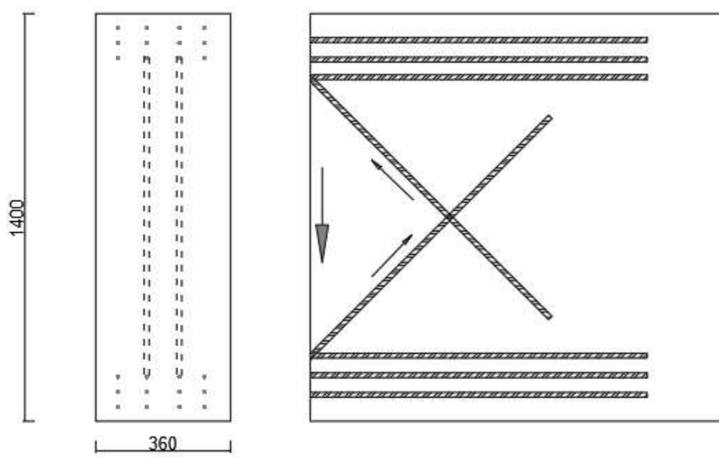
k_{ser}	62029,01	[N/mm]
N in series	4,00	[-]
N in parallel	9,00	[-]
k_{ser}. Series	15507,25	[N/mm]
k_{tot}	139565,27	[kN/m]
$k_{.rot}$	136773,96	[kN.m. rad^{-1}]

C.1.2 Glued-in rods

Connection 2 (A-1) or (A-2)

Force applied on the connection (ULS):

	M_y [kNm]	V_z [kN]	N [kN]
Forces	686	-67	41



The designed connection
The horizontal GIR arranged to resist the maximum from the tension and compression forces with spacing

- Moment and normal force capacity:

The rods' axial capacity is calculated according to "Limträhandbok del 3" page 74 (Svenskt trä, 2016).

Tensile capacity of the rod:

$$F_{ax.Rk.rod} = 0.6f_{u.b} \cdot A_{s=}0.6 \times 400MPa \times \frac{\left(\pi \times \frac{20^2}{4}\right)mm^2}{1000} = 75\ kN$$

$$F_{ax.Rd.rod} = \frac{F_{t.Rk.rod}}{\gamma_m} = \frac{75kN}{1,2} = 62,83kN$$

Withdrawal capacity of the rod:
Where:
$\kappa_1 = 0,29$ *Limträhandbok del 3, Table 13.23. (Svenskt trä, 2016).*

$$F_{ax.Rk.timber} = \pi \cdot l_{ef} \cdot f_{ax.k} \cdot k_1 \cdot \kappa_1 = \pi \times 1000\ mm \times 5,5MPa \times 1 \times 0,29$$
$$= 105,23 \times 1000\ N = 105,23kN$$

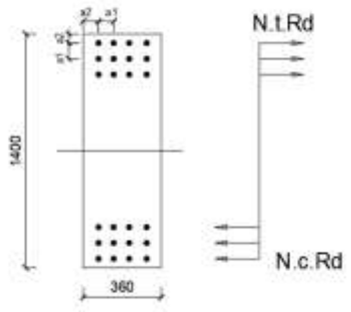
$$F_{ax.Rd.timber} = \frac{k_{mod} \times F_{t.Rk.timber}}{\gamma_m} = \frac{0,8 \times 105,23kN}{1,25} = 62,83kN$$

$$F_{ax.Rd} = \min(F_{ax.Rd.rod}.F_{ax.Rd.timber}) = 62,83kN$$

The capacity is determined by the tensile strength of the rods. All the rods assume to yield:

Rods in compression and tension zones Class 4.80				Bending Capacity based on the axial capacity of the rods	Rotational Stiffness
Layer [-]	d [mm]	l_a [mm]	Number of rods [-]	$F_{ax.Rd}$ [kN]	$K_{r.total}$ [kN·m·rad^{-1}]
1	20	1000	4	673	54381
2	20	1000	4		
3	20	1000	2		

$a_2\ edge\ distance\ = 50\ [mm]$
$a_1 between\ the\ rods = 80\ [mm]$



The axial designed capacity of each zone:

$$N_{Rd} = F_{ax.Rd} \times (n_1 + n_2 + n_3) = 673\ kN$$

According to the arrangement :
The lever arm or the distance between the compression zone and the tension zone :

center for tensile rods	1286.00	[mm]
center for tensile rods	114.00	[mm]
Lever arm	1172.00	[mm]

The moment and the normal force are decomposed to two forces acting on each group of the rods

$$N_{t \cdot Ed} = \frac{M_{Ed}}{L} + \frac{N}{2} = \frac{686}{1.172} + \frac{41}{2} = 605\ kN$$

$$N_{c \cdot Ed} = -\frac{M_{Ed}}{L} + \frac{N}{2} = \frac{686}{1.172} + \frac{41}{2} = 564\ kN$$

Utilization

$$U = \frac{\max\,(N_{t \cdot Ed}, N_{c \cdot Ed})}{N_{Rd}} = 0.9$$

- Inclined GIR resisting shear force.

Inclined rods with 45° in each zone Class 4.80				Shear capacity based on the axial capacity of the rods	Transversal Stiffness
Layer [-]	d [mm]	l_a [mm]	Number of rods [-]	$F_{v.Rd}$ [kN]	K_{total} [kN/m]
3	20	1000	2	190	22165

$F_{ax.Rd} = \min(F_{ax.Rd.rod}.F_{ax.Rd.timber}) = 62{,}83kN$

The capacity for two rods in the vertical direction:

$F_{v.Rd} = 2 \times F_{ax.Rd} \times \sin 45 = 95.45\ kN$

And by using 2 rods in each zone

$F_{tot.v.Rd} = 2 \times F_{v.Rd} = 190.48kN$

Utilization:

$$U = \frac{F_{v.Ed}}{F_{tot.v.Rd}} = 0.35$$

- Rotational stiffness:

According to Section 4.7.3 and Equation (24)

$$k_r = \Sigma k_{ser.rod} \cdot r_i^2$$

Where $K_{ser.rod} = 0.004 \cdot d^{1.8} \cdot \rho_{mean}^{1.5} = 7836.40\ kN/m$

$k_r = 54381.49\ kN.m/rad$

- Translational stiffness:

$k = 4 \times K_{ser.rod} \times \sin 45 = 22165\ kN/m.$

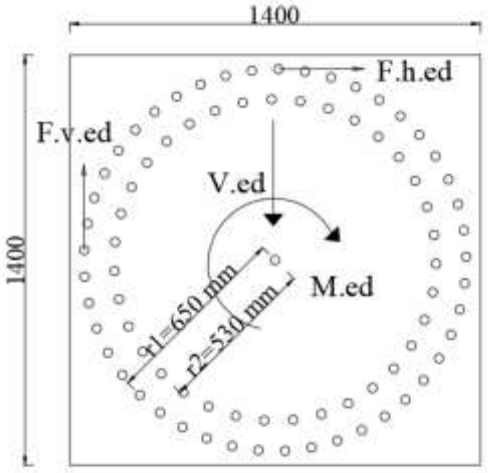
C.2 Connection 3

C.2.1 Fork joint calculation

C82:

	M_y [kNm]	V_z [kN]
Forces	857	278

Dowels ´Class 6,8
$D=20$ [mm].
$f_u = 600$ [MPa].
Number of dowels $n=80$.
Thickness of outer part of the column $t_1 = 110\ [mm]$.
Thickness of the internal part of the beam $t_2 = 140\ [mm]$.
The dowels are in a circular arrangement with detail in the next figure.



Radius of circles r_i [mm]	Number of dowels n_i [-]	r_i^2 [mm2]
650	20	8450000.00
550	20	6050000.00
450	16	3240000.00
350	16	1960000.00
250	10	625000.00

Polar moment of inertia $I_P = \sum r_{i=}^2 \ 29000000 \ [mm^2]$
Dowel subjected to the maximum force generated by the moment is with $r = 650 \ mm$:

$$F_{m_i} = \frac{M \cdot r_i}{I_P} = \frac{857\text{kN} \times 650mm}{20325000 \ mm^2 \times 1000} = 19{,}5 \ [kN]$$

Vertical force from shear on each dowel:

$$F_{vi} = \frac{F_v}{n} = \frac{278 \ kN}{80} = 3{,}48 \ [\text{kN}]$$

Three critical dowels with r=650mm horizontal $a = (90{,}0{,}90)$ and vertical $a = (0{,}90{,}0)$ from force to grain to t_1, t_2, t_1.
The applied shear force in these dowels are:

$$F_{v.v.Ed} = F_{m_i} + F_{vi} = 19{,}5 \ + 3{,}48 = 22.98\text{kN}$$
$$F_{v.h.Ed} = F_{m_i} = 19{,}5 \text{ kN}$$

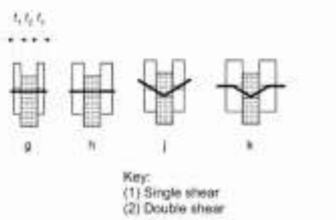
The dowel design capacity is calculated according to (EC5 (CEN, 2004) Equations (8,70) (8,8)).

– For fasteners in double shear:

$$F_{v,Rk} = \min \begin{cases} f_{h,1,k} t_1 d & \text{(g)}\\ 0,5 f_{h,2,k} t_2 d & \text{(h)}\\ 1,05 \frac{f_{h,1,k} t_1 d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} d\, t_1^2}}-\beta\right]+\frac{F_{ax,Rk}}{4} & \text{(j)}\\ 1,15\sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk} f_{h,1,k} d}+\frac{F_{ax,Rk}}{4} & \text{(k)}\end{cases} \quad (8.7)$$

with

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}} \quad (8.8)$$



Dowel subjected to vertical force has a characteristic load capacity $F_{v.Rk} = 42.13kN$ and response to failure mode h in which is a brittle failure.
Dowel subjected to vertical force has a characteristic load capacity $F_{h.Rk} = 31.02kN$ and response to failure mode j in which is a ductile failure.

$$F_{v.Rd} = \frac{F_{v.kd} \times k_{mod}}{\gamma_m} = 26.96\text{kN}$$

$$F_{h.Rd} = \frac{F_{h.kd} \times k_{mod}}{\gamma_m} = 19.85\text{kN}$$

Utilization:

$$U_v = \frac{F_{v.v.Ed}}{F_{v.Rd}} = \frac{22.98}{26.96} = 0.85 \qquad U_h = \frac{F_{v.h.Ed}}{F_{h.Rd}} = \frac{19,5}{19.85} = 0.98$$

Rotational Stiffness:
The stiffness of one single dowel is calculated according to Table 4.2.

$$K_{ser.i} = 2 \cdot \rho_m^{1,5} \cdot d/23 = 2 \times \frac{430^{1,5} kg}{m^3} \times \frac{20mm}{23} = 15507.25\text{kN/m}$$

Rotational stiffness is calculated according to Equation 0(24):

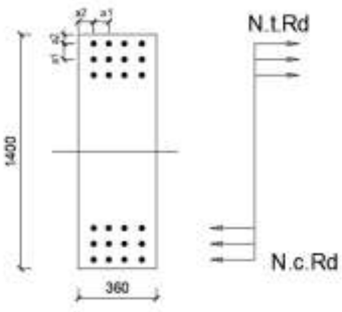
$$Kr = \sum K_{ser.i} \cdot r_i^2 = 449710.30 \text{ kN.m/rad} .$$

C.2.2 Glued-in rods with steel plate

C82:

	M_y [kNm]	V_z [kN]
Forces	857	278

Dowels Class 4.8
$f_u = 400$ [MPa].
$D = 20$ [mm].
$l_a = 1000\ [mm]$ with axial capacity $N_{ax.Rd} = 62{,}82\ [kN]$
Number of dowels: 4 x 3 in compression and tension zone
$a_2\ edge\ distance\ = 50\ [mm]$
$a_1 between\ the\ rods = 80\ [mm]$



- Moment capacity

The rods' axial capacity is calculated according to “Limträhandbok del 3” page 74 (Svenskt trä, 2016).

Tensile capacity of the rod:

$$F_{ax.Rk.rod} = 0.6 f_{u.b} \cdot A_{s=} 0.6 \times 400MPa \times \frac{\left(\pi \times \frac{20^2}{4}\right) mm^2}{1000} = 75\ kN$$

$$F_{ax.Rd.rod} = \frac{F_{t.Rk.rod}}{\gamma_m} = \frac{75kN}{1{,}2} = 62{,}83kN$$

Withdrawal capacity of the rod:
Where:
$\kappa_1 = 0{,}29$ *Limträhandbok del 3, Table 13.23. (Svenskt trä, 2016).*

$$F_{ax.Rk.timber} = \pi \cdot l_{ef} \cdot f_{ax.k} \cdot k_1 \cdot \kappa_1 = \pi \times 1000\ mm \times 5{,}5MPa \times 1 \times 0{,}29$$
$$= 105{,}23 \times 1000\ N = 105{,}23kN$$

$$F_{ax.Rd.timber} = \frac{k_{mod} \times F_{t.Rk.timber}}{\gamma_m} = \frac{0{,}8 \times 105{,}23kN}{1{,}25} = 62{,}83kN$$

$$F_{ax.Rd} = \min(F_{ax.Rd.rod}. F_{ax.Rd.timber}) = 62{,}83kN$$

The capacity is determined by the tensile strength of the rods. All the rods assume to yield to get the maximum resisted moment.

$M_{Rd} = F_{ax.Rd} \times (n_1 \times \ l_1 + n_2 \times \ l_2 + n_3 \times \ l_{31})$

Layer	n_i	$l_i[mm]$
1	4	1300
2	4	1140
3	4	980

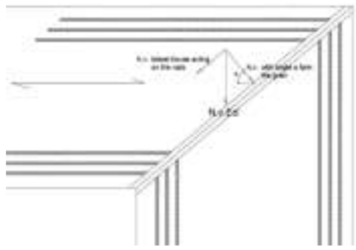
Where:
$l_i - lever\ arm\ for\ the\ \ i\ layer\ [mm]$
$n_i - number\ of\ rods\ in\ i\ layer$

$M_{Rd} = 859{,}93\ kN.m$

Utilization:

$U_{ax} = \frac{M_{Ed}}{M_{Rd}} = \frac{857}{859{,}93} = 0.997$

- Shear checking



The shear force transfer through the steel plates to the timber contact surface.

- Two checks should be done
- Compression at an angle with the grain.
- Lateral force acting on the rods.

1. Compression at an angle with the grain according to EC5 6.2.2.

$a = 45\ degree$
$F_{v.perp\ to\ plate} = F_{v.Ed} \times \sin(a) = 278\ kN \times \sin(45) = 196\ kN$

The stress resultant from $F_{v.perp\ to\ plate}$

$$\sigma_{c.a.Ed} = \frac{F_{v.perp\ to\ plate}}{b_{beam} \times \frac{h_{beam}}{\sin(a)}} = \frac{196\ kN}{0{,}36\ m \times \frac{1{,}4\ m}{\sin(45)}} = 0{,}55 Mpa$$

Compared to equation (EC5 (6.16))

$$\sigma_{c.a.Rd} = \frac{f_{c.o.d}}{\frac{f_{c.o.d}}{k_{c.90} \times f_{c.o.d}} \times \sin^2 a + \cos^2 a} = \frac{1{,}6}{\frac{1{,}6}{1{,}75 \times 1{,}6} \times \sin^2 45 + \cos^2 45}$$

$\sigma_{c.a.Rd} = 1{,}15 kN$

Utilization:

$U = \frac{\sigma_{c.a.Ed}}{\sigma_{c.a.Rd}} = \frac{0{,}55}{14{,}5} = 0{,}48$ ok

2. Lateral force acting on the rods:

$F_{v.//\ to\ plate} = F_{v.perp\ to\ plate} = 196\ kN$

Lateral force on one rod:

$$F_{l.Ed.rod} = F_{v.//\ to\ plate} \times \frac{\sin(45)}{\sum n_i} = \frac{139}{24} = 5{,}79kN$$

The lateral capacity of GIR is calculated according to (CEN et al., 2019) Section 7.3. The characteristic yield moment of the rod

$M_{y.Rk}$ =289640.46 EC5 Equation (8,30)

$f_{90.Rk}$ = 15.50545455 EC5 Equation (8,31. 8,32)

$f_{90.Rk.rod} = 1{,}25 \times f_{90.Rk} = 19{,}38\ MPa$

$f_{0.Rk.rod} = 0{,}1 \times f_{90.Rk.rod} = 1{,}94\ MPa$

$f_{90.Rk.rod}$ is the embedding strength of rod parallel to grain and equal 0,1 · $f_{90.Rk.rod.}$

$f_{0.Rk.rod}$ is embedding strength of rod perpendicular to grain equal 1,25 · $f_{90.Rk}$.

The failure mode associated with the design situation is a single plastic hinge in the fastener located in the timber member, giving the following relation

$$F_{l.Rd.rod} = 1{,}15 \times \sqrt{2 \times M_{y.Rk} \times f_{0.Rk.rod} \times d} + \frac{F_{ax.Rd}}{4} = 31kN$$

Utilization:

$U_l = \frac{F_{l.Ed.rod}}{F_{l.Rd.rod}} = \frac{5{,}79}{31} = 0{,}18$ ok

Interaction between bending and shear in each rod:

$\left(\frac{F_{l.Ed.rod}}{F_{l.Rd.rod}}\right)^2 + \left(\frac{F_{ax.Ed.rod}}{F_{ax.Rd.rod}}\right)^2$ =0.03+0,993=1

- Rotational stiffness:

According to Section 4.7.3 and Equation (24)

$$k_r = \Sigma k_{ser.rod} \cdot r_i^2$$

Where $K_{ser.rod} = 0.004 \cdot\ d^{1.8} \cdot\ \rho_{mean}^{1.5} = 7836.40\ kN/m$

$k_r = 61907.57\ kN.m/rad.$