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Fatigue Performance of Adhesive Joints in Timber Wind Turbine Towers

A study to evaluate the fatigue performance of adhesive joints in timber connections

Master's thesis in Master Program Structural engineering and building technology

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Abstract

Fatigue is a process which weakens a material over time under variable load. For some materials, the fatigue process is known and studied and thus the damage accumulation can be predicted and taken well into consideration when designing construction elements. For materials like steel, these properties are well known and checked whenever a steel construction is designed. For timber however, it is a rather unexplored subject and one which this thesis focuses on.

The reason that the fatigue process has not been researched for timber is that it is rarely the deciding factor for the capacity of timber structures with bridges as an exception. However, for wind turbine towers, the amount of load cycles is extensive. During their lifetime, they will experience a high number of load cycles with low stress levels. This is called high cycle fatigue. This raises the question of the fatigue properties which will have a high demand regarding these load conditions.

This thesis is focused on the application of glued in rod connection, which also have a lacking design regulation, even for static loading. For this connection, some research has been concluded, where some reliable design regimes have been put forward along with some information about the connections. The information available was gathered and put forward in order to present a reliable design recommendation. The available design regimes were tested using general coding software and then compared with other available lab tests done previously by others. In conjunction to this, an finite element model made in Abaqus was produced in order to provide further understanding and general knowledge into these connection types under cyclic as well as static loading.

The conclusion was finally made that an adhesive layer of $2mm$ would be optimal for this type of connection, if fatigue as well as a static performance is to be retained. The adhesive itself should be restricted to either polyurethane or epoxy of the 2-components type. These two adhesives gave similar performance properties with small differences that could be neglected. The recommended edge and rod distance should be kept to no less than $2.5d$ as well as $4d$ respectively. The use of a laminated veneer lumber would simplify the connection with regard to load angles toward grains but glulam is a viable option, especially glulam made out of spruce or other coniferous lumber. The pullout strength was generally improved when increasing the anchorage length of the rod. Anyhow, the fatigue performance did not benefit of this to the same extent.

Keywords: Fatigue, Adhesive connections, Timber structures, Wind turbine towers.

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1

Introduction

The following chapter explains the background, purpose and aim of the thesis project. The limitations are also introduced to create a good understanding of whats to come. The chapter introduces the reader with achieved information from previous tests and research studies from literature.

1.1 Background

All structural elements and materials are subjected to variable and permanent loads. The variable loads are inducing so called fatigue. This is a process that causes permanent changes in the material that experience variable stress levels. Fatigue damage occur when initial cracks widen into larger cracks that in worst cases can result in failure.

Long term loading resistance or so-called fatigue in wooden structure and in wooden connections are a rather unexplored topic which is not included in existing building regulations such as Eurocode. This is mainly the case because there has not been any immediate need to evaluate timber connections in fatigue. This work is performed at Chalmers.

There are therefore several unknown factors about the fatigue performance of timber connections, what parameters that are of importance such as type of loading, fasteners, adhesives, timber materials that all have different effects on the fatigue. Through gathering of literature about all aspects research and analysis will be performed to achieve enough results to draw credible conclusions.

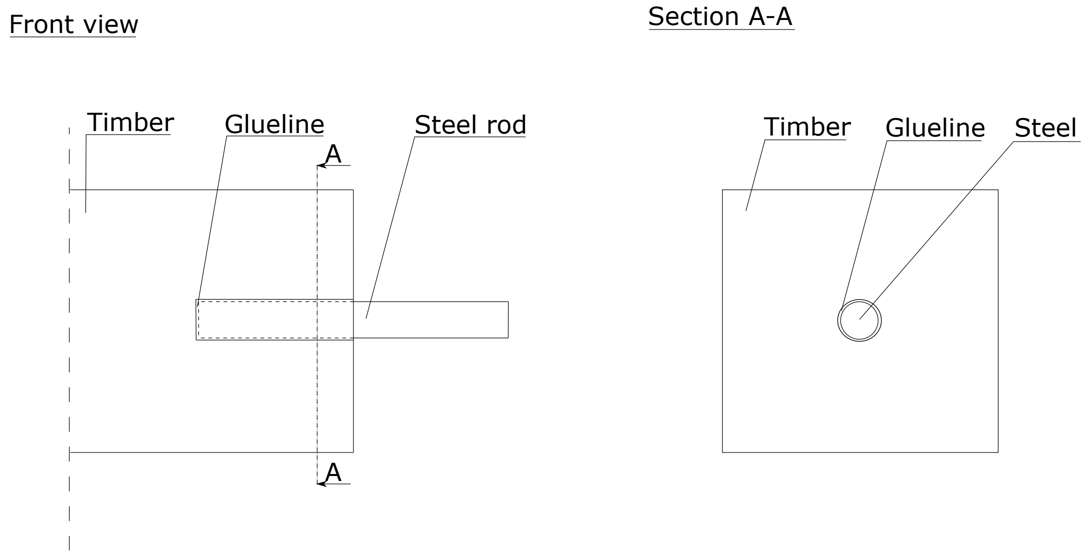


Figure 1.1: Sketch of glued-in rod

1.2 Purpose

The purpose of this project is to gather missing information of the described problem as well as provide results based on various tests that have been performed. These test will include calculations based on stated assumptions and limitations of the problem at hand. Numerical analysis will also be performed using finite element applications such as Abaqus in order to achieve more reliable result and see how the results deviate.

1.3 Objective and Aim

' The aim of this thesis is to provide information about timber connections specified for mainly glued in steel rods into softwood timber structures. To provide a research discovering the unexplored fields of fatigue performance of timber connections and serve as a basis for further research.

The study conducted in this thesis is expected to provide useful information regarding structural behavior and performancee of glued in rod adhesive joints under fatigue cyclic loading, with application to the future generation of timber wind turbine towers developed by Modvion.

The objective of this thesis is to gather and provide information appropriate to the designer and constructor of timber wind turbine towers, the following research questions should be answered:

- What is the dominating cracking modes in timber connections due to fatigue?
- What type of cyclic loading is mostly critical?

- How will different connections handle different loading conditions and variations?
- What type of timber and adhesive materials in an adhesive joint are most resilient to fatigue?
- What type of adhesive connections is most efficient, considering both static and long term, fatigue loading conditions?
- How will different geometrical and material properties in a glued in rod joint influence the performance in fatigue?
- How does the glue line configuration influence fatigue performance?
- What type of fatigue loading is the worst?
- How does the height of the turbine towers matters?
- How should the materials be protected?
- Which are the critical stress positions for a glued in rod?

The main goal is therefore to finalize a good suggestion with recommendations for a construction of the glued in rod type and to establish the important factors in order to achieve a strong, resistant and long term efficient connection. The thesis also aims to contribute to further research in this area.

1.4 Limitations

In order to get a manageable task to analyze, some initial limitations has been made. The type of engineered wood product (EWP) is limited to glulam and laminated veneer lumber and will be the timber material of choice for this report. However, other materials will be tested and further analyzed in order to provide a broader perspective on the EWP usage and its influence.

The timber type to be considered in this study will mostly be limited to the performance of glued laminated glulam softwood since softwood is the mostly used timber type now a days. However, hardwood will be considered and investigated but not into as deep of a consideration as softwood. Since the climate change might have an impact on the use of softwood [27], this thesis will also look at the differences and performance fluctuations of hardwood.

The rod type to be considered in this report will mainly be focusing on the steel material due to the broad fatigue knowledge. A steel of a mild strength type for improved fatigue performance is the initial focus of this report. However, Fiber reinforce plastic (FRP) rods have been proven through many investigations and reports to be more efficient in fire resistance and strength performance and are widely present in the available information to gather. The FRP rods is anyhow not to be considered due to its brittle failure mechanism.

Adhesives will mainly be focused on the two components glues, Epoxy and Polyurethane. Information about the differences in performance due to the different glue types will be gathered and an analysis about this subject will be carried out.

1.5 Method

A literature study was performed in the early stages to obtain a broad knowledge around the subject. The methods for further information gathering are mainly focused on hand calculations and computer software. The finite element program ABAQUS based on simplified assumptions and mathematical tools like Matlab is broadly used in order to find answers for subjects that lack scientific information.

2

Problems in wind turbine towers

In the following section, theory from investigated articles, studies and related literature will be specified and summerized.

2.1 Wind turbine towers

The wind turbine towers of today is mainly created out of steel or most commonly fiber glass. To efficiently remake these towers in the material of timber as well as glued in steel rod connections takes a lot of considerations to say the least.

The loads and fatigue performance expectancy's would require a thorough investigation. Myslicki et al. states that the loads typically ranges between a frequency of about 0.01 to about $1.0Hz$. A thorough reading of Ajaei and Soyoz article [10] composing about the effects of preload deficiency's on fatigue demands of wind turbine tower bolts gives a rough understanding of the need in bolt strengths for the kind of connections used in these constructions. Ajaei and Soyoz [10] composes about the bolt designs and the requirements of each bolt. Each bolt in the construction investigated are of the $M36$ bolts kind. This name reefers to the diameter of the bolt being $36mm$ as shown in the figure 2.1 below.

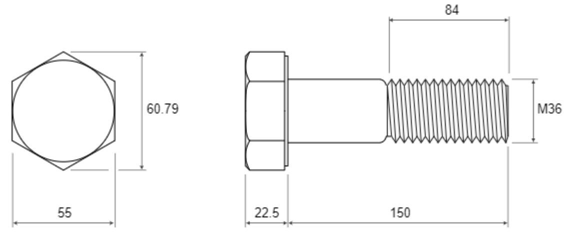


Figure 2.1: The geometry of a M36 bolt [6]

The bolts in these investigations are designed to be withstanding a preload force of $510kN$ [10]. The bolts were placed in the connection as showed in the fig. (2.2) and (2.3) below. These fasteners were designed to be placed in the cylindrical tower formations with 88 bolts in total covering the whole cylinder. Further, bolts and connections were modelled in a finite element program and the following conclusions were made. Ajaei and soyoz [10] clearly witnessed that the bending stresses of the connection had a severe role in the fatigue performance and should be included in the fatigue design process. Due to the fatigue mode being a very common failure mode for wind turbine towers, the fatigue performance of the connections is recommended to be further analysed and methods of this application should be improved within the industry [10].

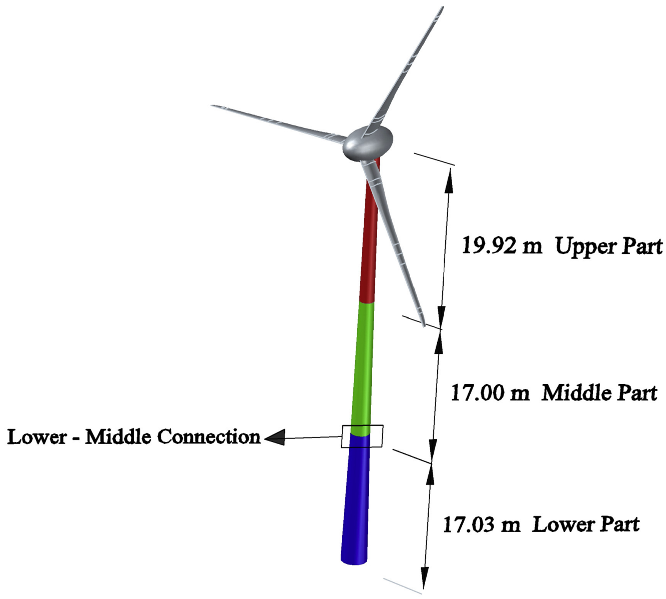


Figure 2.2: Wind turbine tower connection [10].

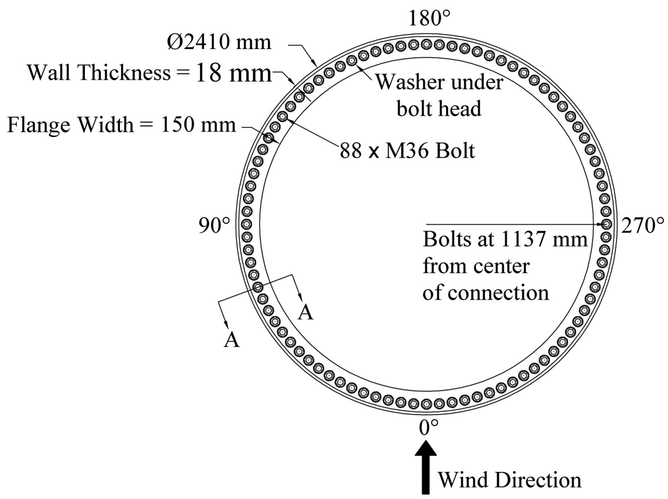


Figure 2.3: Wind turbine tower bolt placement [10].

Further on, taking into consideration that the tower to be investigated in this thesis case is made out of a timber-steel construction material combination makes for an even more complicated connection. Unlike steel, timber is generally a construction material which the adequate knowledge has not formulated for fatigue. Therefore, an investigation of the timber fatigue performance is thoroughly going to be done in this thesis.

2.2 Fatigue

This section is to be explaining the general terms and basic information around fatigue. When discussing fatigue, a couple of introductory facts needs to be stated. The fatigue performance of a connection is generally the ability for the connection to withstand longtime cyclic and static loading. The most conventional load type for fatigue design is considered to be of constant amplitude. This load type is the load type which is mostly used within fatigue testing [22]. The ratio between the minimum and maximum stresses to be considered is known as the R value presented in eq. (2.1) below [23].

$$R = \frac{\sigma_{d,min}}{\sigma_{d,max}} \text{ where } -1 \leq R \leq 1 \tag{2.1}$$

R Ratio between minimum and maximum stress
$\sigma_{d,min}$ Minimum stress
$\sigma_{d,max}$ Maxmimum stress

The fatigue life of a structure is mostly numerated through the number of cycles experienced at certain stress levels before failure [22]. The way to display the fatigue life performance is the way of using S-N curves. These curves consists of a good display of the number of cycles N for different stress levels s, visualized in a log-log graph. The S-N curve is divided into a low cycle fatigue and a high cycle fatigue. The low cycle fatigue is known as a part of the S-N curve where the loads are considerably high and the cycles withstanded considerably low [22]. After a certain numbers of cycles and a low enough stress level, the fatigue performance is considered to be unlimited.

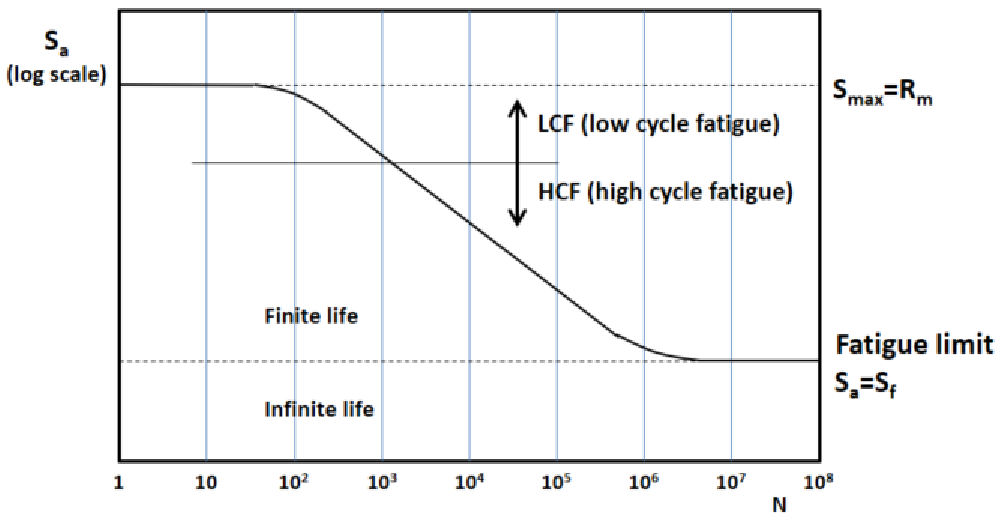
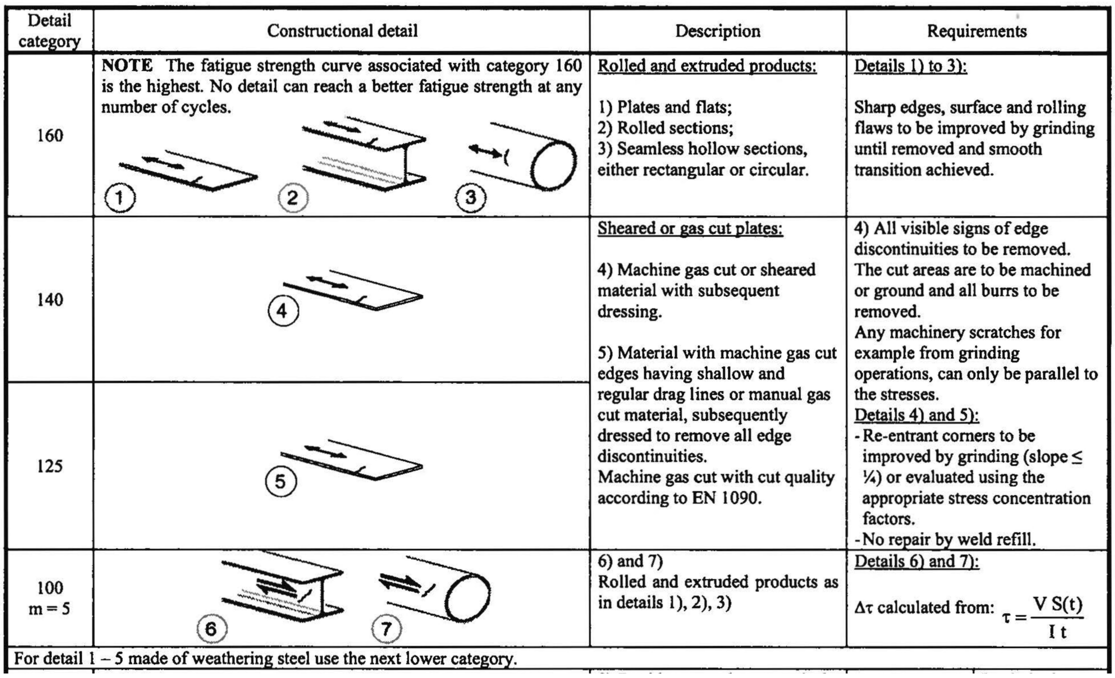


Figure 2.4: Estimation of fatigue life with regard to number of cycles

When designing a steel detail in terms of fatigue, an S-N curve is constructed using the results from multiple tests done to the detail [22]. In order to make a reliable representation of the fatigue performance of the detail roughly at least a minimum of 5 tests on each stress level as well as a minimum of 2 stress levels needs to be tested [22]. This is considered to be required in order to make reliability for the linearity of the fatigue strength. In Eurocode, a fatigue strengths of steel details is displayed thoroughly in Eurocode EN 1993-1-9. However, in our case, the fatigue performance for a glued in rod connection is missing and especially not for the material combinations we are considering. An example of the fatigue strength is displayed in fig. (2.5) below.



Detail category	Constructional detail	Description	Requirements
160	NOTE The fatigue strength curve associated with category 160 is the highest. No detail can reach a better fatigue strength at any number of cycles. (1) (2) (3)	Rolled and extruded products: 1) Plates and flats; 2) Rolled sections; 3) Seamless hollow sections, either rectangular or circular.	Details 1) to 3): Sharp edges, surface and rolling flaws to be improved by grinding until removed and smooth transition achieved.
140	(4)	Sheared or gas cut plates: 4) Machine gas cut or sheared material with subsequent dressing. 5) Material with machine gas cut edges having shallow and regular drag lines or manual gas cut material, subsequently dressed to remove all edge discontinuities. Machine gas cut with cut quality according to EN 1090.	4) All visible signs of edge discontinuities to be removed. The cut areas are to be machined or ground and all burrs to be removed. Any machinery scratches for example from grinding operations, can only be parallel to the stresses. Details 4) and 5): - Re-entrant corners to be improved by grinding (slope ≤ ¼) or evaluated using the appropriate stress concentration factors. - No repair by weld refill.
125	(5)		
100 m = 5	(6) (7)	6) and 7) Rolled and extruded products as in details 1), 2), 3)	Details 6) and 7): Δτ calculated from: $\tau = \frac{V\,S(t)}{I\,t}$
For detail 1 – 5 made of weathering steel use the next lower category.			

Figure 2.5: Fatigue strength for details, extractions from eurocode EN 1993-1-9.[3]

Many of the S-N curves considered in Eurocode have similar slopes and this is used in order to categories the different details. The two used categories for these are the slopes 1 : 3 as well as 1 : 5. In certain scenarios of the different slopes are used in different load cycle parts of the S-N curve.

The damage accumulated in the detail after a certain number of cycles n can be calculated. This is generally calculated to be the ratio between the cycles withstanded n and the number of cycles before failure N. This is presented in eq. (2.2). However, often the detail is exposed to multiple stress levels on different load cycles. To calculate the total damage done in this case, the ratios for each case is to be summarized. This is presented in eq. (2.3).

$$D = \frac{n}{N} \tag{2.2}$$

$$D = \sum_i \frac{n_i}{N_i} \tag{2.3}$$

While designing a detail, welds and stress concentrated areas are generally the dimensioning situation. In our case however, for glued-in steel rods, the steel itself to be fatigue designed is consisting of a circular steel rod. The connection consists of a timber structure with stress concentrated areas, connecting to the rod via adhesive bonding.

2.3 Fatigue for wind turbines

When it comes to fatigue and fatigue damage for wind turbine towers it can occur both in small areas and in larger areas, so called "Widespread fatigue damage" or WFD for short. WFD causes failure when an initial micro cracks grows into a larger cracks which eventually lead to structural failure. Since wind turbin towers is subjected to a high number of load cycles, WFD is likely to happen. Knowing this, precautions should be taken to minimize the risks. Failure will occur if one of the two following conditions are reached [4].

1. The net section stress, accounting for the loss of section caused by damage, exceeds the ultimate strength of the material.

2. A critical crack forms by accumulation of damage.

In order to somehow estimate the damage and thus the fatigue life of a structural element, the following formulas were used by Ragheb to first observe the load ratio.

$$R = \text{Load Ratio} = \frac{\text{Maximum applied cyclic load}}{\text{material tensile strength}} \tag{2.4}$$

After that the damage for the element are calculated as:

$$\frac{dD}{dN} = f(\sigma, R, D) \tag{2.5}$$

The fatigue life is later defined as the number of cycles it takes to achieve a critical crack. The picture below illustrate a simplified S-N curve where one can observe when the fatigue life is being exceeded.

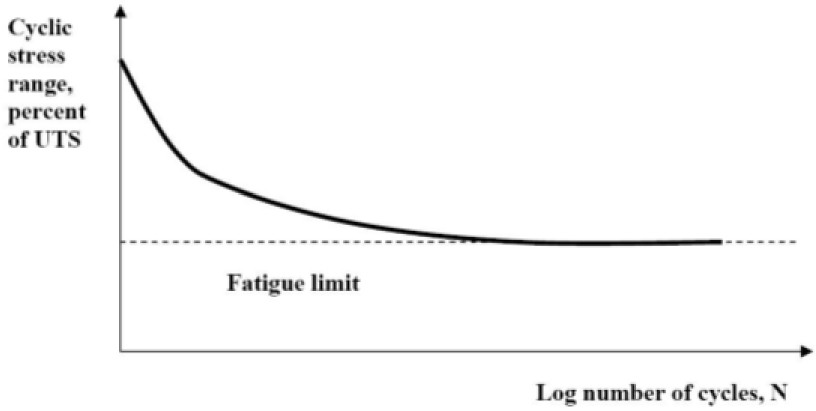


Figure 2.6: Fatigue life estimation [4]

Raghheb however states that theses S-N curves does not take into consideration the complex, various and relevant forces that comes into question when talking about the fatigue for wind turbine towers. These forces result from other types of forces like wind forces, the mass of the rotor blades. The gravitational forces caused by the mass of the rotor blade creates both tension and compression in each load cycle. Ragheb states that research shows that low frequency and high amplitude wind forces contributes highly to fatigue damage.

It is important insure that the construction and the structural elements are protected against environmental exposure, which can negatively effect the fatigue resistance. These factor are for example the design of the rotor blades, corrosive pits should be avoided. These leads otherwise to stress concentration which will arise from rain and wind fragments. Since the further away from the wind mill itself the blade will have a higher speed than the blade parts closer to the mill. This will lead it to be subjected to higher forces and thus have a higher exposure to fatigue damage.

3

GiR adhesive joints in wind turbine towers

3.1 Glued-in steel rod connections

The glued-in rod connections are typically considered to be a visually appealing connection type. It is also considered to be a very powerful and moment-resisting type of connection as well as a proven, highly efficient way to transfer high amounts of axial forces according to a thesis made by Ogrizovic [7] and an article made by Xu, Guo and Bouchaïr [32].

However, the fact that a connection of this type is created with generally three different building materials, adhesive and steel, complicates things. One need to understand how each of these materials handle fatigue loads to know which of these . Therefore this kind of connection is typically pretty hard to develop a calculation regime for [29].

Using the steel as of the rod type would benefit the connection in the manner of good fire resistance and ductility. Using a lower steel strength would also benefit the connection in the terms of fatigue. The connection of this type is commonly used within column-base, beam-beam and beam-column relations [32]. Therefore, the glued-in steel rod connection type is the topic of the investigation in this thesis.

To create optimal timber connections, proper application needs to be followed out. The drilled hole for the rod needs to be fully cleaned away from any lose timber chips. Further on, the glued should be applied into the hole before the rod is dipped. When the rod later on is inserted into the hole, a vibration needs to be applicated in order to make sure that the glue covers all holes and nooks. The correct application that therefore needs to be strictly followed are as follows:

1. Hole predrilling
2. Blow and clean hole
3. Apply glue into the hole
4. Insert the rod while at the same time applying vibration and rotation

Another way of application would be to drill injection holes along the side of the hole. In this was, the rod can be inserted first and then an injection of the adhesive can be made.

3.2 GiR joint's failure modes

When it comes to failure modes in the timber connection, it is typically considered to be common to appear in either the timber, the glueline or the steel itself [20]. The glue line can have failure modes appearing in either the connection between the glue and the steel or the glue and the timber [20]. Failure modes are shown and visually presented in the figs. (3.1) below.

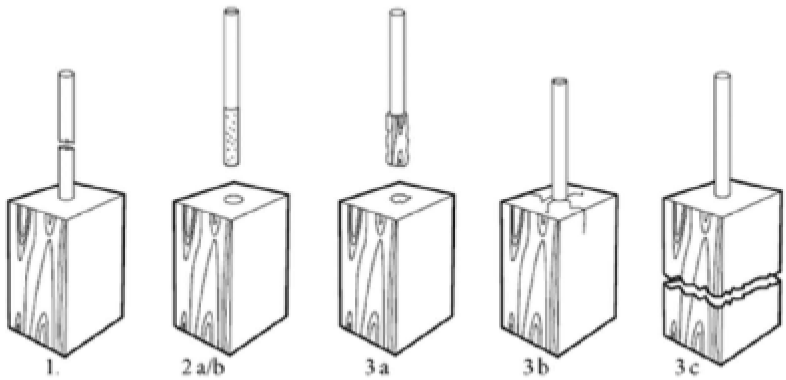


Figure 3.1: Different possible failure modes in Glued in rod joints; the picture is reproduced from [23]

Due to the complexity of these connection types a brittle failure is not to be preferred. To create a reliable connection, a ductile failure mechanism is recommended. The best way to achieve this in a reliable way would be to make the steel rod the critical part of the connection. The steel is a thoroughly investigated material which have clear design directions. It is also clearly discovered when it comes to the fatigue performance and if these properties could be utilized, a reliable connection would be achieved [29].

3.3 Geometry and load conditions in GiRs

Depending on the geometry and the load condition of the connection, different kinds of strength conditions could be suspected. Often two different beams are connected via a steel joint. The steel joint itself is then in some way attached to both beams. Using glued-in rods in this case is not very uncommon [29].

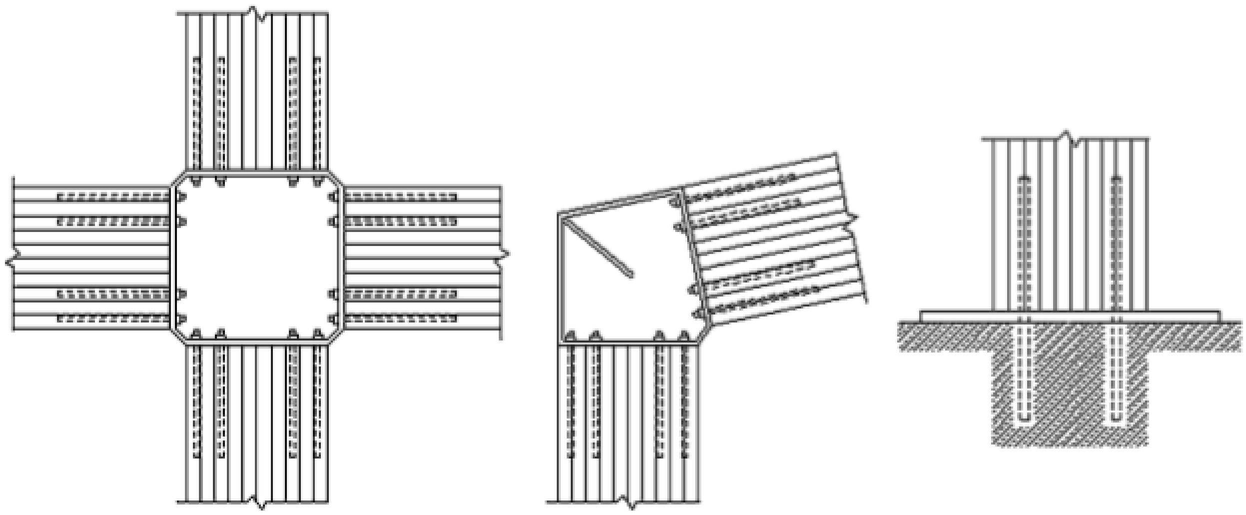


Figure 3.2: Example of connection types, reproduced from [29]

3.3.1 Anchorage length

The necessary anchorage length and rod length is generally very dependant on glue type and the resulting bondline strength. The glue types to be considered in this thesis is mainly 2-components PUR and EPOXY. It is stated in [29, 28] that the pull-out strength typically increases with the glued-in length but that at the same time a decrease of the shear strength happens, due to the shear stress peeks on the edges of the connections. These shear stress peeks are presented in fig. (3.3) below.

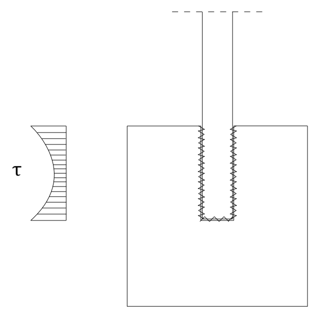


Figure 3.3: Visualization of the shear stress peeks in a glued in rod connection.

Articles and several tests have been tackling the optimization of the anchorage length. One article that has been tackling these issues further are Fueyor et al. [17]. In the article, several tests were made to test the change in several parameters within an finite element model made in the program Abaqus. One of these parameters were the rod length as could be seen in fig. (3.5). The rod lengths importance to the tensile strength perpendicular to the grain, could in all glulam classes to be optimized to be around $200 - 250mm$. [17] also shows how the rod diameter affects the tensile strength in these direction in fig. (3.4). When watching the tensile strength perpendicular to the grain while changing the rod diameter, one could see that the benefits of increasing rod diameter declined after reaching about 15mm of rod diameter.

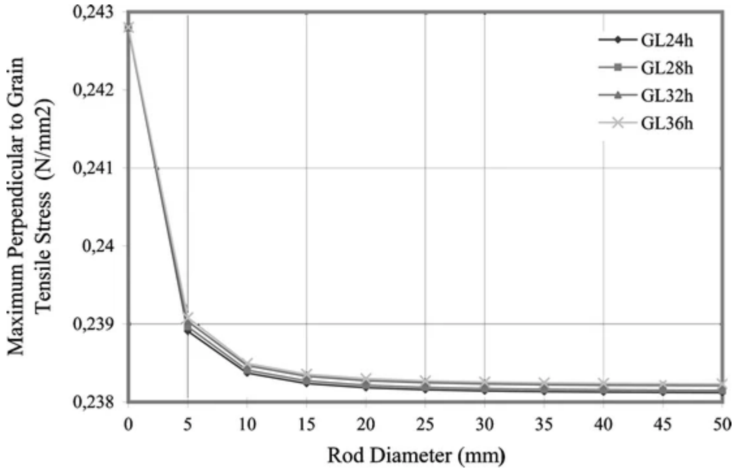


Figure 3.4: Rod diameter towards tensile strength for glulam classes, gathered from [17].

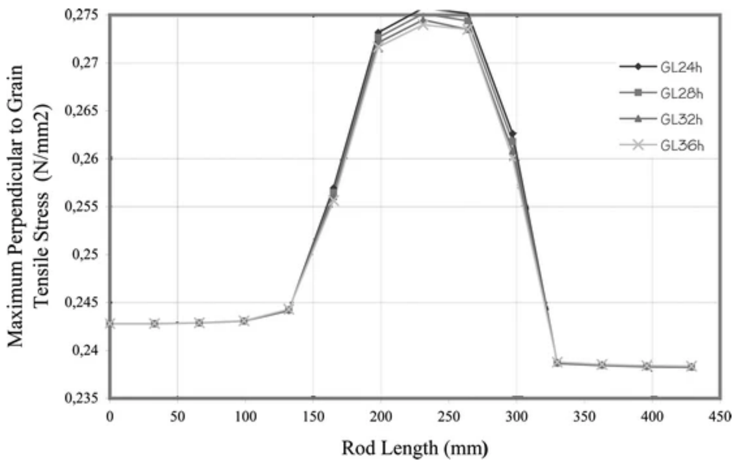


Figure 3.5: Rod length towards tensile strength perpendicular to grain for different glulam classes, gathered from [17].

3.3.2 Edge distance

During the 90s, recommendations for the lengths for glued in rods were starting to progress and be put forward in order to be able to design for the splitting failure mechanism of the connection. The recommendation was to keep the distance between each rod to at least 5 times the diameter of the rod itself. The distance from the edge to the closest rod should be kept at 2 times the diameter of the rod or more in order to be able to reliably assume no splitting to be possible. Anyway, if the edge distance is set to be smaller than 2.5 times the diameter, a reduction of the strength of the connection should be made because of inaccuracies [29]. These recommendations are based of laboratory tests and studies. In fig.(3.6) and the related table (3.1), least rod-to-rod distance and edge distance in glued in rod joints based on different existing standards and guidelines are compared.

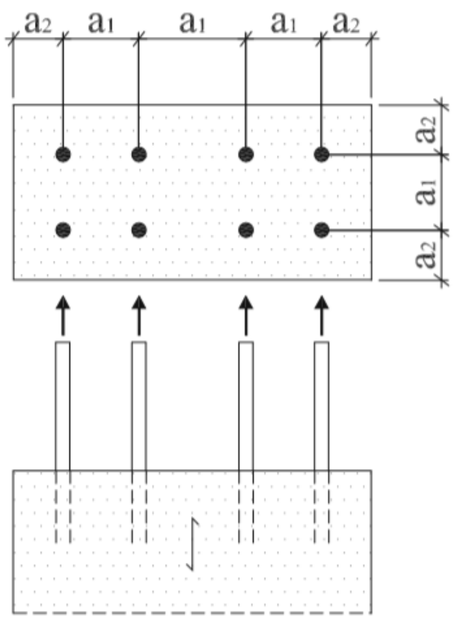


Figure 3.6: Comparisson regardning edge distance (parallel to grain), reproduced from [29]

Table 3.1: Center and edge distances according to recommendations [29, 16, 9, 5, 15]

Rods II to grain	prEN 1995:2001	DIN 1052:2004- 08	STEP1	French Professional Guide
a_1– Distance between the rods	$4d$	$5d$	$2d$	$3d$
a_2– Edge distance	$2.5d$	$2.5d$	$1.5d$	$2.5d$

The article [17] have previously been mentioned in this thesis, if we further analyse this article we can find some more interesting information. Fueyo also analyzed the rod distance, in the article he displayed the rod distance towards the tensile strength perpendicular to the grain for different rod amounts shown in fig. (3.7). The results here clearly show that the optimized rod distance change according to the amount of rods used.

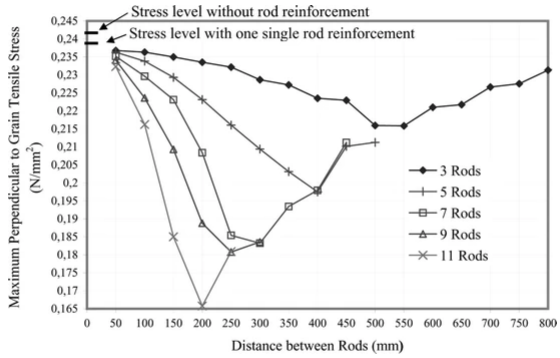


Figure 3.7: Rod distance towards tensile strength perpendicular to grain for different amounts of rods, gathered from [17].

3.3.3 Load-to-grain angle

The load-to-grain angle should not be higher than 10% , this is because several studies shows the decrease in strength properties that relates to the increase of load angle. This is because of the timber strength perpendicular to the grain is particularly weak in this direction.

3.3.4 Density of timber section

Smith states that studies have shown the the higher the density a timber material has the higher its fatigue performance and resistance will be. this density effect is more relevant with higher stress intensity (0.46-0.6) and not as much with lower stress levels (0.3-0.45)[27]. Below, fig. (3.8) shows the fatigue life versus density relation.

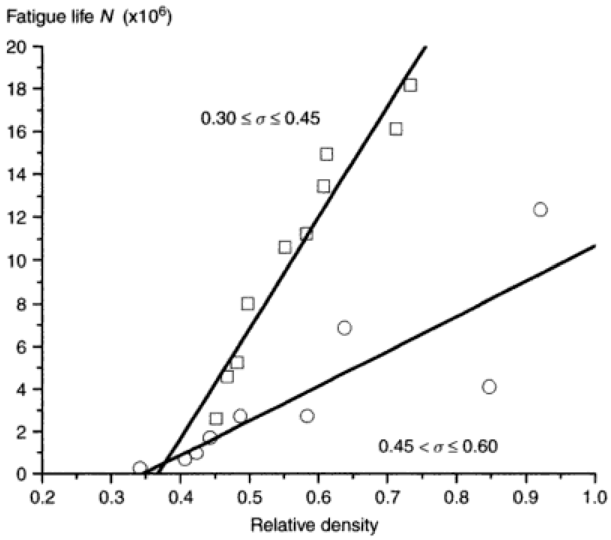


Figure 3.8: Fatigue life with regard to timber density, reproduced from [27]

This is something that will be described in details in section (3.4). Here, both [29] and [26] have been providing the construction community with design recommendations. One of the design recommendations are only related to one timber density configuration, meanwhile the other design recommendation has this set to be within a certain limitation span. Even though the first design choice is a more precise calculation it lacks the choice of timber density. Making the second, linear design choice more viable in our case of fatigue.

3.3.5 Moisture content

As will be discussed later on, the moisture content has a high effect on the fatigue performance on timber connection, especially when it comes to the timber material. An increase of moisture content over the fibre saturation point will lead to a decrease in fatigue strength. The figure below shows the response of different specimens with different moisture content that are subjected to the same stresses.

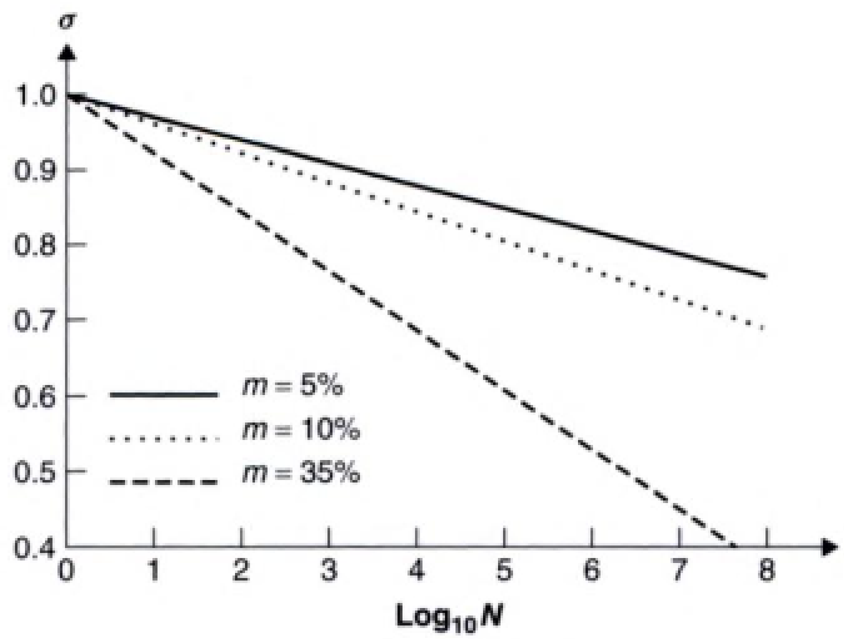


Figure 3.9: Influence of different moisture contents on the fatigue life [27]

3.4 GiR constituents and their fatigue performance

Pullout strength design guidelines has been revealed by Tlustochowicz [29]. The eqs. (3.1), (3.2) and (3.3) are referred to be used to design pullout strength of the glued in rod based on characteristic values with regard to the failure mechanism of bondline failure, regardless of the grain direction. This biliniar design model are based on the German standard DIN. Here d are known as the diameter of the rod and l_{ad} are the bonding length of the connection [29].

$$R_{ax,d0} = \frac{\pi d l_{ad}}{f_{k1,k}} \tag{3.1}$$

$$f_{k1,k} = 4.0\, if \;\; l_{ad} \leq 250mm \tag{3.2}$$

$$f_{k1,k} = 5.25 - 0.005 l_{ad}\, if \;\; 250mm < l_{ad} \leq 500mm \tag{3.3}$$

However, the design formulas for the design of axially loaded glued in rods of steel are shown below as eqs. (3.4), (3.5) and (3.6). These calculation models are purposed and written about in the articles [28] for parallel grain direction and [31] for perpendicular direction. For this model to be viable, one should design the connection in such a way that the failure happens within the rod itself. This is achieved by choosing the correct connection geometry. If the designer chooses a rod length not longer than $350mm$ one could assume steel plasticization. Here l is to be referred to as the glued length, d_h is the diameter of the drill-hole and k_1 aswell as k_2 being constants applied as 0.045 and 0.8. We can also notice the slenderness ratio λ of the rod being applied within equation 3.5 aswell as the density of the timber ρ. All distances in these equations are entered as mm and the area A_g is generally known as the surface area of the drill hole. The design model is below is published by Tlustochowicz [29]

$$F_{ak,mean} = f_{v,0,mean} \pi d_h l \tag{3.4}$$

$$f_{v,0,mean} = 7.8 \frac{N}{mm^2} \frac{\lambda}{10}^{-1/3} \frac{\rho}{480}^{0.6} \tag{3.5}$$

$$F_{ax,90mean} = k_1 A_g^{k_2} \; (kN) \tag{3.6}$$

These calculation recommendations are only reliable in a certain design span. However, one can argue that going inside these limitations anyway seems to result in a reliable design. Instead multiple rods would be applied in order to make sure of a reliable connection.

The limitations for these models are presented in the table (3.2) below, reproduced from [29].

Table 3.2: Table showing the limitations of the design recommendations for pullout strength [29]

Limiting ranges	Lower	Upper
λ	7.5	15
d	12 mm	20 mm
l_v	$5d$	$5d$
Edge distance	$2.3d$	∞
ρ	350	500

In order to be able to design the connection in terms of fatigue, some kind of design regime needs to be produced. Here is were the available information starts to lack, but some relatively good and necessary recommendations has been gathered.

Myslicki showed that the design of wooden bridges under fatigue related loads is regulated via the eqs. (3.7), (3.8) and (3.9) below, DIN EN 1995-2: 2010-12 [23, 21]. Here some coefficients (a and b) are left to be established by others to the lack of published work within the glued in rod margin. These equations can anyhow give a good understanding of how the fatigue performance is designed in regular well formed regimes for connections [23]. The limitations are described in table (3.5) below.

$$\sigma_{d,max} \leq f_{fat,d} \tag{3.7}$$

$$f_{fat,d} = k_{fat}\frac{f_k}{\gamma_{M,fat}} \tag{3.8}$$

$$k_{fat} = 1 - \frac{1-R}{a(b-R)}log(\beta N_{obs}t_L) \geq 0 \tag{3.9}$$

Table 3.3: Parameter explanation for fatigue

$f_{fat,d}$	Fatigue strength design value
k_{fat}	Strength reduction factor due to fatigue
f_k	Characteristic static load strength
$\gamma_{M,fat}$	Partial factor for fatigue, 1.0
a, b	Fatigue coefficients
N_{obs}	Number of annual load cycles (constant amplitude)
t_L	Time coefficient, lifetime in years
N	Number of load cycles (constant amplitude)
β	Coefficient for effect of damage to structural members
$\beta = 1$	Significant consequences
$\beta = 3$	Considerable consequences
R	Principle stress ratio, minimum and maximum

Further on however, [23] provides us with recommendations for these constants for most important configurations. The first is if the rod used, is chosen to be threaded and glued into hardwood, then the a and b values are to be chosen as $a = 6$ and $b = 1.35$. If the rod however is chosen to be a rebar of the type BST 500 s, one would be recommended to use the constants $a = 6$ and $b = 2.1$ for reliable results. These constants are chosen in such a way that the graphs coincide with the ones from provided laboratory tests. The constants can also be seen in table (3.4) below, for which the values are gathered from [23].

Table 3.4: Table presenting a and b values for three different configurations.

Configuration	a	b
Threaded rods	6	1.35
BST 500 s	6	2.6
BST 500 B NR - Inoxripp 4486	6	2.8

The limitations and restrictions of the model is stated below in a bullet point scheduled reproduced from [23]. Which is very similar to the limitations set for the static strength models previously presented in table (3.2).

Table 3.5: Table showing the limitations of the design recommendations for pullout strength [23]

Limiting ranges	Lower	Upper
λ	7.5	15
d	12 mm	20 mm
l_v	$5d$	$5d$
Edge distance	$2.3d$	∞
ρ	350	500
Glulam made out of Norway spruce or other coniferous timber		

3.4.1 Adhesive

3.4.1.1 Glue type

The adhesives that this thesis will focus on are polyurethane and epoxy or variations of these types. These are the most common and most used adhesive types in glued-in timber connections. The glues thats to be inspected are 2-components adhesives which are folded and mixed in order to prevent air bubbles to be formed before application.

3.4.1.2 Embedment length

There are no clear regulations on how great the embedment length should be. Studies have revealed that an increase in embedment length increases the strength of the connection. However, an "optimal" length (where one could say that the change in strength is too small for optimization) is a topic that is relatively unclear.

R. Steiger, E. Gehri, and R. Widmann shows in their article [28], that the embedment length is a dominant parameter in terms of pull-out strength in a timber connection, both parallel to grain and perpendicular to grain.

Steiger et al. performed experiments on pull-out strength by varying parameters like hole dimension and anchorage length (embedment length). As previously mentioned, the anchorage length contributes with an increase of the ultimate load capacity. According to Steiger et al. [28], except its benefits, it also decreases the shear strengths of the connection. In fig. (3.10), we can see the anchoring length related to the nominal shear strengths in the connection for different thicknesses in the rod.

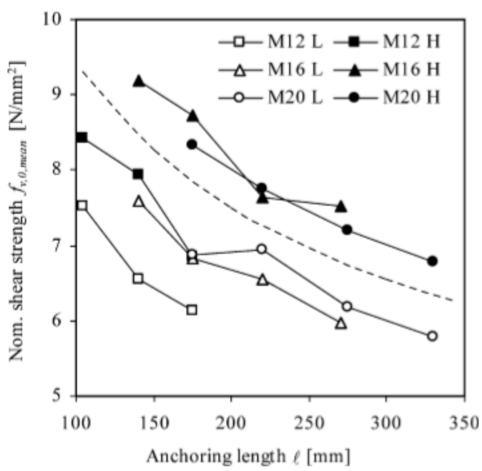


Figure 3.10: Increasing anchorage length related to shear strength [28]

However, the studies also showed that the hole dimension and anchor length had a close relation (slenderness), which, even if not proven, develop some regulations for the parameters. Other parameters that were also taken into consideration in this rapport where edge distance, density of the timber material and rod diameter. Some of the theoretical regulations can be seen above. As mentioned previously, timber is a anisotropic material, therefore similar tests were also done with pull-out capacity perpendicular to the grain as well.

3.4.1.3 Glue-line Thickness

While investigating the article made by Madhoushi and Ansell [20], a couple of understandings where made. Madhoushi and Ansell made several investigations on glued in rods. In this case, tests were made, both in labs as well as in finite element models. These tests were conducted with an initial control of the specimens gathered in order to make sure the moisture content were precise and correct [20]. Three configurations were investigated and two different glued in lengths were chosen for the specimens. The glue-line thickness was also shifted for the different configurations, 0.1mm, 1mm and 4.5mm were applied. Rotafix CB10TSS epoxy resin was then injected at $\frac{1}{3}$ of the length of the holes. CB10TSS is a 2-components epoxy which is blended thoroughly in order to prevent air bubbles before application [20]. All rods were grinded using sand paper and then cleaned using alcohol. The curing was followed out in 18-20 degrees temperature for 10 days [20].

The glueline thickness is generally decided in most articles to be around 2mm. This seems to be the optimal thickness for bonding when it comes to all types of adhesives. The glueline thicknesses of $0.5mm$, $2mm$ and $4mm$ were tested by Madhoushi et al. in 2004 [21] and a conclusion were made that a too large thickness results in earlier failure when the rate of loading is increased than the other specimens. Conducting these experiments made it possible to make the assumption that an increase from 2-4mm in glueline thickness could possibly result in higher static strengths but an increase may be vulnerable in fatigue performance.

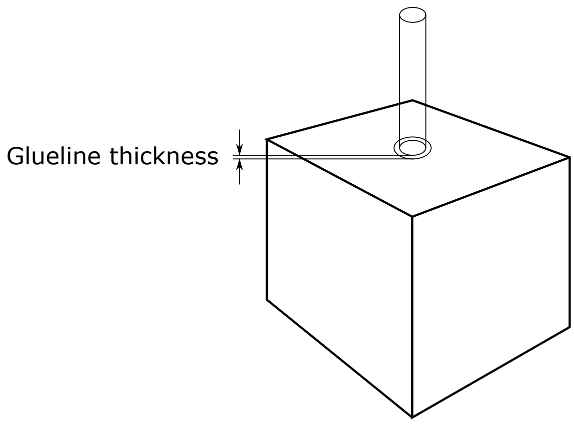


Figure 3.11: Sketch showing the glueline thickness of a glued-in rod connection.

3.4.1.4 Fatigue performance

Glue type has a significant influence on the fatigue performance, [11] has made laboratory tests stating that the different glue types used in those test has different performances, especially when it comes to fatigue performance. The different glue types used in these tests were PRF, PUR and EPOXY. The tests made were done using two different mildly performing steel rod thicknesses with different load cycles to make up a clearer picture of how the fatigue performance could be impacted by the glue type. For both glue line thicknesses of 16 and 8mm it was clear that for ½ cycle of load the Epoxy glue type had a better fatigue resistance. When using higher cycles of up to 10^7, this seemed to change.

For the smaller rods, the PUR glue seemed to perform slightly better and for the thicker rods it performed evenly as good as the epoxy did. This might tell us that the PUR glue type is more efficient for longer periods of fatigue loads. Considering timber wind turbine towers, that has to consist long term loads and wind cycles of higher values. If a glue type were to be chosen between these three types, a suggestion would be to go for a slightly higher fatigue performing PUR type for smaller rods and otherwise using an EPOXY resin. Something to keep in mind would be that a cost efficiency calculations has yet to be done for this. If the Epoxy glue is less expensive it might be worth it to consider slightly more repairs in exchange.

The glue line thickness is important when it comes to the strength and performance for connections. Too thin glue line might create a glue line failure mode with insufficient strength levels. A glue line thickness which is too thick will have a significant bad influence on the fatigue strength performance. This has been tested and stated in [21] as previously mentioned in section (3.4.1.3). In the studies made in this article, it was clear that an epoxy resin thickness, thicker than 2-4mm might create a more vulnerable fatigue performance [21].

Madhoushi et al. [23] states in the reports and articles that an investigation done [21] took the further conclusion done by measuring reactions from glued in rods with EPX glue that an increasing glue line thickness after around $2-3mm$ decreases the fatigue performance of the connection. The connections failure mode consisted of being timber or bonding related.

An article written by F. Hunger et al. shows the adhesive capabilities for Norway spruce and European beech [19]. Compression/pull-out tests were performed on glued-in rods. Rods diameter and anchor length were kept constant. Around 200 specimens were tested with different materials and 2 types of adhesives, epoxy and polyurethane. The results shows that shear failure of the rod were most likely to happen, this shows that thee characteristics of the adhesives had little influence. The tests also showed little differences between the 2 different adhesives epoxy and polyurethane. However the results shows the importance of the conditions for the adhesives, the shear stress distributions in the the wood material provides a big influence, the deformation of the adhesive itself, the deformation of the wood and the connection of the adhesive in the hole [19]. It is important to ensure that as little initial cracks are present as possible to provide full adhesive bonding.

It is also shown that the pullout strength of the timber material are underestimated especially for the Norway spruce. There is no immediate need to investigate these parameters further but it is important that research of this in the future are concluded.

In [1] a couple of tests were performed on three different kinds of adhesives in order to proceed with the ongoing GIROD project. Here [1] states that, due to bubbles forming in the PUR bonded samples, a more common timber-adhesive interface failure occured. The bubbles decreased the effective bonding area of the adhesive and made for perfect stress concentration points. This made such a unreliable bonding that a linear approximation of the behaviour could not be made without questioning the unreliability.

The best possible glueline thickness for fatigue performance while still maintaining the ultimate static strength is therefore adviced to be around the 2 mm spectrum. This is presented in fig. (3.12) below.

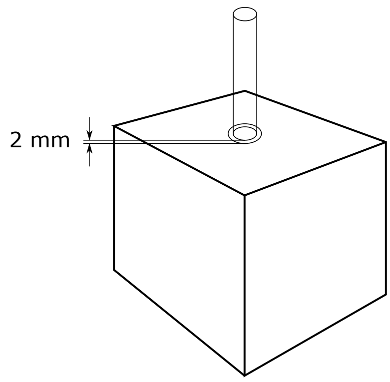


Figure 3.12: Sketch showing the recommended glueline thickness for optimal fatigue performance while still retrieving a high ultimate static strength.

In fig. (3.13), five different possible failure modes are presented from [23]. These failure modes are the modes interpreted by Myslicki, S et al. during the fatigue performance tests done in [23].

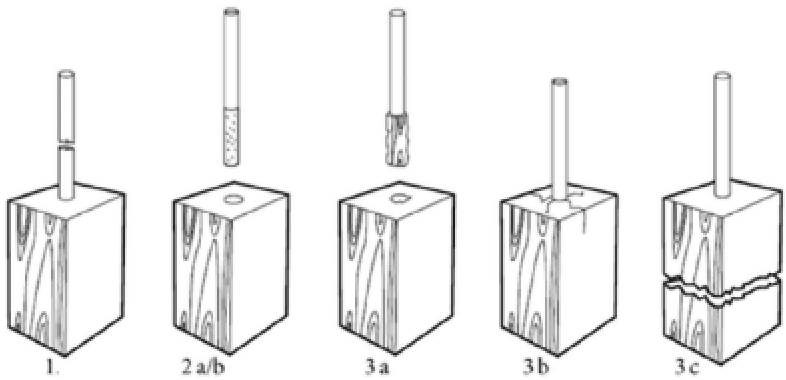


Figure 3.13: Figure showing the possible failure modes for fatigue performance in glued-in rods for hardwood products, reproduced from [23].

3.4.2 Steelrod

The rod itself will most likely be constructed by steel. Whereas the known and applied properties has a isotropic behaviour throughout the material [14]. The Elastic modulus of steel is generally known to be 190-210 GPa, as well as a applied Poisson's ratio of 0,27-0,30 [14].

3.4.2.1 Mechanical behaviour

To create a harder metal with better properties an alloy can be produced. Steel is produced heating up a ferrite, perlite, cementite and austenite combination to create an alloy [14]. The alloy gets created using a proper hematite and magnetite relation that's determined using a so called phase diagram. The production process consists of the melting process using a furnace, where iron ore and coke is inserted into the furnace to produce steel [14]. Oxygen is induced during the process and carbon monoxide and carbon dioxide is extruded.
Steel is a strong homogeneous material with same strength properties in all directions. The strength properties is displayed in the table 3.6 below. Steel is a relatively linear elastic material that follows Hooke's law until the yield point on which the plastic behavior happens until failure [14].

Table 3.6: Steel classes and its associated yield strengths, Elasticity modulus and Poisson ratio [14]

Steel class	s235	s275	s355	s420	s460
$F_y[MPa]$	235	275	355	420	460
$E[GPa]$	$190-210$	$190-210$	$190-210$	$190-210$	$190-210$
ν	$0.27-0.30$	$0.27-0.30$	$0.27-0.30$	$0.27-0.30$	$0.27-0.30$

Below in fig. (3.14), the steel behaviour of a rod in extension is presented. The initial elastic relation translates when overloaded into a plastic state. This plastic state then at a certain deformation begins to harden before the ending fracture happens.

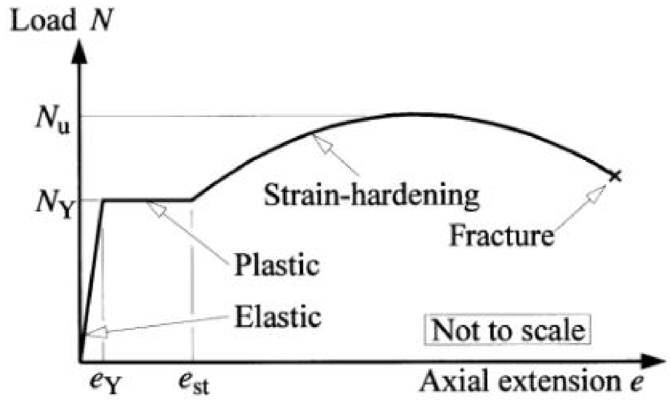


Figure 3.14: The axial extension behaviour of a steel rod, reproduced from [30]

3.4.2.2 Fatigue performance

The fatigue performance of steel is a subject that's initially very known and applied in the construction regimes such as Eurocode.

Other materials to be used for the rod itself has also been thought about. For example, using a fiber reinforced plastic (FRP) rod is something that is elaborated and written about. The article [20] handles the effectiveness of FRP as rod type. The clear conclusion were made during testings here that a solid laminated veneer lumber beam have a significantly higher static as well as fatigue strength performance than using FRP rods. It is also stated that the withstanding cycles of 10^6 are for LVL happening at stress levels of 50% and for FRP these cycles are being withstanded at as low of a stress level as 30% of the static strength. Something that Madhoushi [20] states though is the low dissipation of energy during cyclic loading that the laminated veneer lumber has.

Madhoushi and Ansell [20] states that glass fiber-reinforced plastic has been investigated by the university of Bath in 1996. These investigation states that GFRP rods has the advantage of being more resin and timber compatible. In general a better bonding performance and humid resistance. GFRP rods are also lighter in terms of weight than steel which gives it a benefit.

In fig. (3.15) below a picture aswell as a drawing from the article [20] is gathered in order to display the configuration of the tests. This gives us further notice of the credibility of these tests and what the information and conclusions tells us.

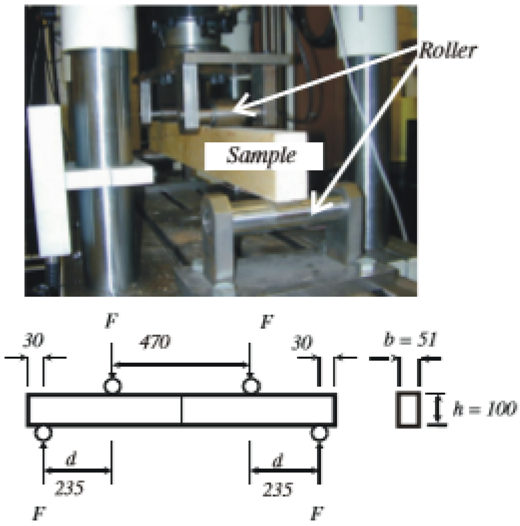


Figure 3.15: Loading configuration of the test specimens, picture gathered from [20]

Another aspect that needs to be investigated is the steel rod type used. Tests have been done to investigate this and [23] is a good example for this. The article cited investigates the fatigue performance of glued-in rods in engineered hardwood with the help of laboratory experiments and has been cited numerous times in this thesis before. One conclusion that was made for these experiments is that the fatigue performance was noticeably increased when using a normal re-bar instead of using a threaded steel bar. In this gathered graph below displayed in fig. (3.16) we can see a clear strength advantage of this type of rod usage and a numerical strength advantage increasing from $20kN$ for the threaded rod to $50kN$ for the re-bar.

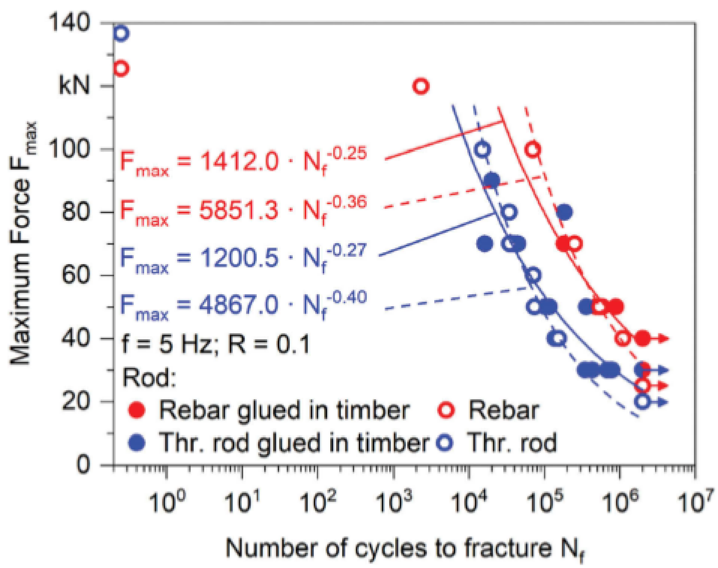


Figure 3.16: Number of cycles N_f withstanded for different forces during testing for rebars as well as threaded steel rods [24]

3.4.3 Timber

Timber is a rather untested material with regard to fatigue. However, some studies has been made regarding its fatigue properties. Timber is an an-isotropic material, which means that it has different properties in different grain directions.

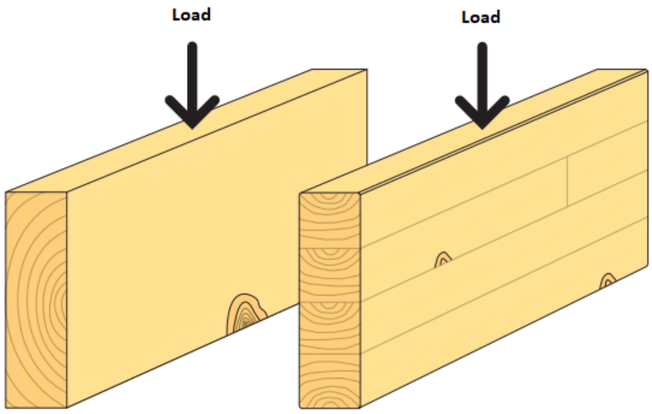


Figure 3.17: Grain direction relation, glulam and structural timber. Picture modified with English text and gathered from [13].

3.4.3.1 Harvest

As of in Sweden the law system is conducted in such a way that if you cut down a tree you are enlawd to plant a new one [8]. This is good to keep the timber supply high in Sweden since the Swedish welfare has timber selling as one of its main incomes. Having this statutory have been leading to the forests in Sweden mainly consisting of fir timber. This is mainly because of cost efficiency and production qualities of the fir timber. The statutory are however regulating the timber sorts planted. This is done by limiting the timber sort to the types best suited for the contemplated soil [8].

3.4.3.2 Anisotropic

Timber doesn't act as other materials. Timber has the unique ability of having different strength properties in different directions [14]. As shown in the picture below we can see that to describe the different strength properties, each direction is assigned name descriptions [14]. We have axial, radial and tangential direction. Axial direction being in the stock's directions, Radial being the perpendicular towards the annual rings and the grain direction, tangential direction being perpendicular towards the grain direction and parallel towards the annual rings [14]. Also described to as parallel and perpendicular to the grain for strength properties relation simplification [14]. Strength parameters for these different directions within in the timber is displayed in the table below.

- Axial direction – stocks direction
- Radial direction – Perpendicular towards the annual rings and the grain direction

- Tangential direction – Perpendicular towards the grain direction and parallel towards the annual rings.

Another good ability of timber is its minimalistic thermal movements. Since timber have less movements when it comes to temperature change, it's better under big temperature changes than for example steel and has very small chances of problem initiation from temperature changes [13].

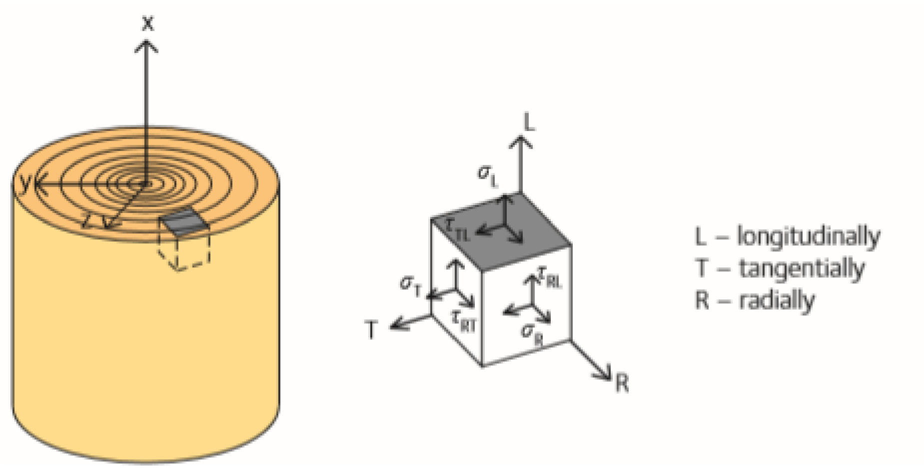


Figure 3.18: Reproduced picture visualizing the timber directions [13]

Different cutouts make different qualities. Depending on the direction of the grains in relation to the product to be cut out, makes a difference when it comes to shrinkage and expansion. When the grains get filled with vapor content it expand in the perpendicular directions. This is important when it comes to connection quality and the porosity makes a difference for the adhesive attachment.

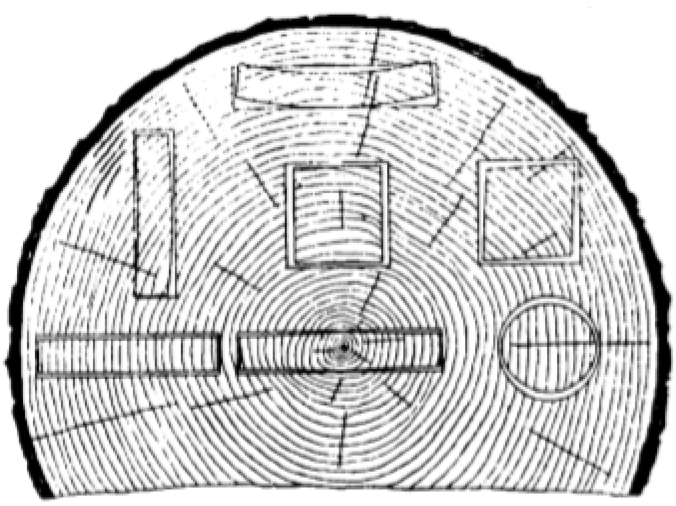


Figure 3.19: Reproduced picture showing the difference that cutouts make to the product, gathered from [14]

3.4.3.3 Engineering wood products

Depending on product to produce and timber to be used, different construction types are being developed. Mainly focused in this thesis will be glulam, lvl and fiber boards. In this section the main material properties and basic explanations for each material is going to be presented.

Glulam
Glued laminated timber is one of the strongest timber products when it comes to strength versus weight relation [13]. Glulam is constructed by gluing together multiple lumber pieces, which makes it stronger than other types of wood since it can have less failure modes. Weak spots can be avoided while at the same time sizes and lengths can be increased. A figure of the glulam structure is herein provided in fig. (3.20)

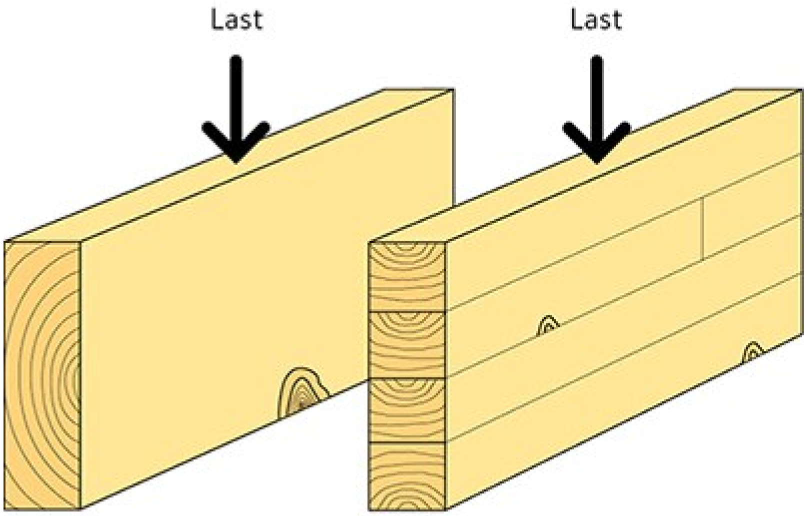


Figure 3.20: Figure visualizing the structure of the engineered wood product glulam (to the right), reproduced from [2]

Laminated veneer lumber
Lvl are constructed by multiple timber veneers together. Veneers are peeled thin layers of timber. One common way of constructing the laminated veneer lumber, for example for plywood, is to place each layer of veener crosswise. Since each layer of veneer is placed crosswise so that the grain direction is perpendicular to each layer [13], the laminated veneer lumber can achieve generally high strength properties. When it comes to the design process the size effect is though something that has to be taken considerably into account [13]. Fig. (3.21) presents the veneer layers production, and fig. (3.22 shows the laminated veneer lumber product itself.

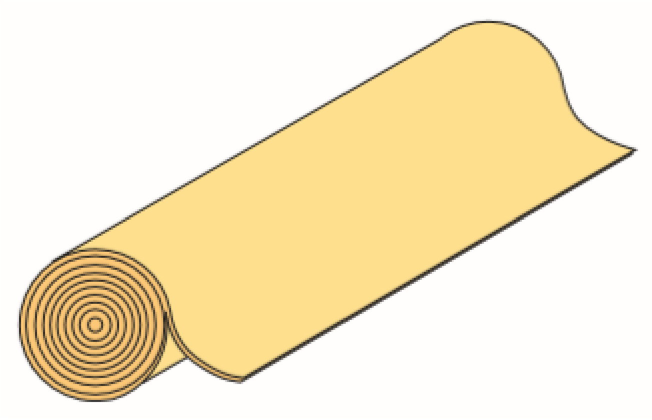


Figure 3.21: Reproduced picture showing how the lumber is devided into multiple veneer lumber layers [13].

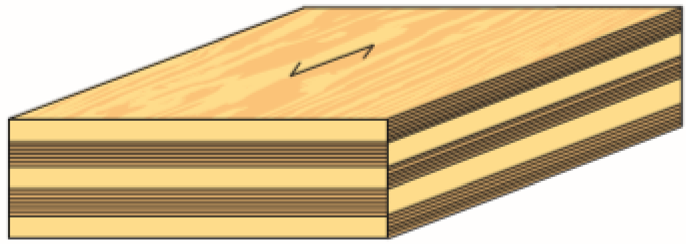


Figure 3.22: Visualization of the Laminated veneer lumber product, reproduced from [13].

3.4.3.4 Material Properties

Depending on Timber type as well as quality we can expect different material properties and different analytical basis. To get a good basis of this thesis, the swedish woods design of timber structure book collection as well as the glulam handbook collection are thoroughly studied [13, 12, 18, 25].

Timber is otherwise a well-known unisotropic material with varying properties in different grain directions. This will of course present further challenges when analysing the results.

The timber construction types mainly focused in this thesis are glue-laminated timber as well as laminated veneer lumber. Below, in table (3.8), the properties of glulam is presented. This could be compared to the laminated veneer lumber properties, table (3.7) presented below.

Table 3.7: LVL properties reproduced and retabled from [25].

Property (Characteristic values with unit [MPa])	Kerto-S Thickness 21-90 mm	Kerto-Q Thickness 21-24 mm	Kerto-Q Thickness 27-69 mm
Strength			
$f_{t,0,k}$	35	19	26
$f_{c,90,edge,k}$	0,8	6	6
$f_{c,90,flat,k}$			
$f_{0,k}$	35	19	26
$f_{c,90,edge,k}$	6	9	9
$f_{c,90,flat,k}$	1,8	2,2	2,2
Stiffness values for capacity analysis			
Elastic modulus			
Parallel to grain,along $E_{0,k}$	11600	8300	8800
Parallel to grain, across $E_{90,k}$		1000	1700
Edgewise, perpendicular to grain $E_{90,edge,k}$	350	2000	2000
Flatwise, perpendicular to grain $E_{90,flat,k}$	100	100	100
Shear modulus			
Edgewise $G_{0,edge,k}$	400	400	400
Flatwise, parallel to grain $G_{0,flat,k}$	400	50	100
Flatwise, perpendicular to grain $G_{90,flat,k}$		16	16
Stiffness values for deformation calculations, mean values			
Elastic modulus			
Parallel to grain,along $E_{0,mean}$	13800	10000	10500
Parallel to grain, across $E_{90,mean}$		1200	2000
Edgewise, perpendicular to grain $E_{90,edge,mean}$	430	2400	2400
Flatwise, perpendicular to grain $E_{90,flat,mean}$	130	130	130
Shear modulus			
Edgewise $G_{0,edge,mean}$	600	600	600
Flatwise, parallel to grain $G_{0,flat,mean}$	600	60	120
Flatwise, perpendicular to grain $G_{90,flat,mean}$		22	22
Density			
ρ_k	480	480	480

Table 3.8: Glulam properties reproduced and retabled from [18, 25]

Property (Characteristic values with unit [MPa])	GL24c	GL28c	GL30c	GL32c
Strength				
$f_{t,0,k}$	17	19,5	19,5	19,5
$f_{t,90,k}$	0,5	0,5	0,5	0,5
$f_{c,0,k}$	21,5	24	24,5	24,5
$f_{c,90,k}$	2,5	2,5	2,5	2,5
Stiffness values for capacity analysis				
$E_{0,05}$	9100	10400	10800	11200
$E_{90,05}$	250	250	250	250
G_{05}	540	540	540	540
Stiffness values for deformation calculations, mean values				
$E_{0,mean}$	11000	12500	13000	13500
$E_{90,mean}$	300	300	300	300
G_{mean}	650	650	650	650
Density				
ρ_k	365	390	390	400

The engineered wood product of laminated veneer lumber is also provided a table of material properties in [25]. These are reproduced and covered in table (3.7) for LVL.

3.4.3.5 Fatigue in timber

Some factors has been concluded to be a part of the timbers fatigue properties. The amount of water that the material contain highly affects the fatigue performance. Smith states that a moisture content that is higher than 5 percent leads to a direct decrease for fatigue with the material will be prone to less load cycles until failure, this sensitivity is related to the quality of the material in question and will diminish with material of lower quality.

Further on, high density timber has proven to be more resistant to fatigue then that of lower density, this has been explained in earlier chapter. Another important factor is the load angle to the grain of the timber. Smith performed tests with different angles for the applied load to observe the difference in strength. The results showed that the straight load conditions towards grain direction had a fatigue strength of 60 percent of the tensile strength with decreasing strength with increased load angle. An general estimation is that the load angle should be below 10 degrees [27]. Different grain deviations can be found around impurities such as knots for example. These imperfections are of high relevance when it comes to fatigue performance, especially static fatigue where the members are subjected to constant loads. Here the imperfections in softwood can be loaded to critical level. Timber is also sensitive to different type of loading with the most damaging being fully reversed loading says Smith, (mean value for the variables loads are 0). This would be when the R value in equation 2.1 is 1. Studies also shows that the larger a timber specimen are the less prone it is to fail in fatigue than that of a smaller sample. Failure mechanism in clear wood is happening at a macro scale.

As is obvious here, there is a lot of parameters that need to be considered when it comes to fatigue for timber structures. In the design of timber structures regimes, no numerical directions is provided, even though a chapter for this is provided. In this chapter some sources of main information for this knowledge is however recommended. If a steel rod connection is to be made, information from previous investigations, lab tests and articles is to be followed to construct a reliable connection. The steel rod connection is however reliable and a commonly used connection.

$$\frac{t}{T} + (\frac{n}{N})^{0.02} = 1.0 \tag{3.10}$$

t Total loading to failure
T "pure" static fatigue lifetime
n Number of total load cycles to failure
N "Pure" cyclic fatigue

The equation above is the interaction between cyclic fatigue and static fatigue for loading frequencies of 0.1 to 1.0 Hz.[27]

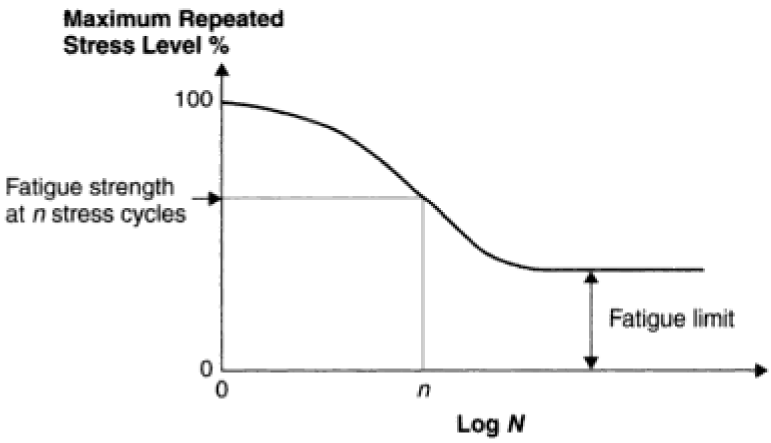


Figure 3.23: Stress level versus number of cycles to failure (S-N) curve

In our case the windmills will experience a high number of load cycles, which will most likely be in the millions. Which will result in high frequencies with decreasing service life of the tower itself if not taken into consideration. These load cycles will also have a relative low amount of force. The windmills will therefore experience so called High cycle fatigue (HCF). There are no clear numbers between low cycle fatigue (LCF) or high cycle fatigue, but in general you could say that high cycle fatigue has a minimum of ten thousand cycles. This phenomenon is highly relevant to our research for its application.

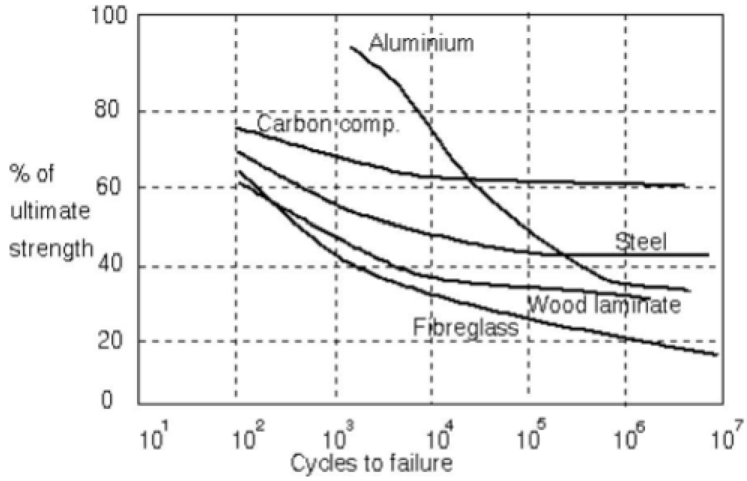


Figure 3.24: Fatigue life of wood compared to fatigue life of different metals

In the picture above we can observe how the fatigue life of wood can be compared to that of other materials, like steel for example whose fatigue properties are well known.

4

Fatigue analysis of GiRs joints

In the following chapter, the different methods used to further analyze and gather information which is hard to obtain due to a lack of studies in the subject. Numerical methods such as Matlab scripts has been put through.

4.1 Design models and recommendation for fatigue in GiRs

In order to further analyze reliability and viability of the design recommendations mentioned in this thesis, Matlab scripts were produced. The results from the script could in this way be compared to other results from previously made lab tests within the subject by others.

Using the design recommendations from [29] and also cited and showed previously in this thesis, scripts were produced. The scripts were created to make a convenient plot showing the strength dependency from changes done to the rod diameter and the length of the bonding. The equations of [29] is divided into a bonding failure and a steel rod failure for the pullout strength. Failure due to bonding was scripted using a for loop for most common rod diameters within the limits of the equations, this being 12, 16, 18 and 20mm. The equations used for the bonding failure is displayed below as eq. (4.1), (4.2) and (4.3) and the script itself is presented in A.

The connection failure relation can be explained using the equations presented in the eqs. (4.4), (4.5) and (4.6). This was also scripted using a for loop with elseif statements for the three different equations that's to be used depending on the bonding length used. The regular pullout strength is checked and scripted using if statements to check input values and then implemented into a for loop for each millimeter. In this case a for loop is made for both perpendicular and parallel cases, in the perpendicular case the bonding area is also included and calculated in each loop. Further on these calculations were plotted for each diameter of the rod where all plotted strength rates are characteristic. The scripts constructed are presented in the appendix (A).

$$R_{ax,d0} = \pi d l_{ad} f_{k1,d} \ \ (kN)[29] \tag{4.1}$$

$$f_{k1,k} = 4.0\, if\;\; l_{ad} \leq 250mm\, [29] \tag{4.2}$$

$$f_{k1,k} = 5.25 - 0.005 l_{ad}\, if\;\; 250mm < l_{ad} \leq 500mm\, [29] \tag{4.3}$$

Table 4.1: Parameter explanation for equation (4.1), (4.2) and (4.3).

l_{ad}	Bonding length, length of the adhesive bondline [mm]
d	Rod diameter [mm]

$$F_{ak,mean} = f_{v,0,mean} \pi d_h l\, [29] \tag{4.4}$$

$$f_{v,0,mean} = 7.8 \frac{N}{mm^2} \frac{\lambda}{10}^{-1/3} \frac{\rho}{480}^{0.6}\, [29] \tag{4.5}$$

$$F_{ax,90mean} = k_1 A_g^{k_2}\; (kN)\, [29] \tag{4.6}$$

Table 4.2: Parameter explanation for equation (4.4), (4.5) and (4.6).

d_h	Diameter of drill hole [mm]
l	Glued length [mm]
λ	Slenderness ratio
ρ	Density of the timber $[kg/m^3]$

Further explanations for the design recommendations are made in the subchapter (3.4).
Another analytical model made was one using the formulas from [23, 21]. The eqs. (4.7), (4.8) and (4.9) which are discussed in the previous design recommendation section (3.4). As mentioned in section (3.4), these design steps and equations are generally used within the wooden bridge construction and are presented within the Eurocode regimes of wood construction. However, the coefficients a and b are missing which gave Myslicki et al. the reason to find out a representative value when designing glued in rod connections in timber. Therefore three different values have been recommended and given which have been scripted and plotted for each and every configuration possible for these equations. The script itself are shown in appendix (B) and the graph, plotted for the stress ratio $R = 0$ is shown in fig. (4.5).

$$\sigma_{d,max} \leq f_{fat,d} \tag{4.7}$$

$$f_{fat,d} = k_{fat} \frac{f_k}{\gamma_{M,fat}} \tag{4.8}$$

$$k_{fat} = 1 - \frac{1-R}{a(b-R)} log(\beta N_{obs} t_L) \geq 0 \qquad (4.9)$$

4.2 Comparative analysis using Matlab

The script constructed in order to be able to compare and estimate the viability of this analytical model is presented in the appendix (B). All plotted values are characteristic and the k_{fat} factor is calculated according to eq. (4.9).

Table 4.3: Parameter explanation for fatigue

$f_{fat,d}$	Fatigue strength design value
k_{fat}	Strength reduction factor due to fatigue
f_k	Characteristic static load strength
$\gamma_{M,fat}$	Partial factor for fatigue, 1.0
a, b	Fatigue coefficients
N_{obs}	Number of annual load cycles (constant amplitude)
t_L	Time coefficient, lifetime in years
N	Number of load cycles (constant amplitude)
β	Coefficient for effect of damage to structural members
$\beta = 1$	Significant consequences
$\beta = 3$	Considerable consequences
R	Principle stress ratio, minimum and maximum

Due to the lack of a standard method to design a connection of the analysed type, A. Rossignon and B. Espion, among alot of other people have been researching the effect of certain parameters. In [26] Rossignon and Espion presents the results of 60 glued in rod connection specimens with different configurations. These were tested to find the more specific result of varying anchorage length as well as rod diameter. The tests were concluded using Norway spruce bonded in threaded steel bar parallel to the grain. The adhesive used to perform this bonding was a 2-component epoxy. As one can see, these conditions fulfill the demands for the design recommendations from section (3.4).

4.3 Results and discussion

We can clearly see that the pullout strength due to the bonding failure is modelled as an initial linear reaction leading into a somewhat unexpected reaction. The highest Pullout strength witnessed due to pullout failure is seen to be about 125kN while implementing 1 meter of bonding length and 20mm of rod diameter as shown in fig. (4.2). This can be compared to the approximate 500kN pullout strength while rod is placed parallel to the grain shown in fig. (4.2). The difference between 5.1 and 5.2 are the pull-out capacity and the bond-line strength. We can also see that placing the rod in a parallel direction to the grain instead of placing it perpendicular makes a huge difference in the pullout strength as shown in fig. (4.2).

Another notice that could be made from this is that the strength of the connection is declining quite quickly after 250mm bonding length. This means increasing the bonding length after 250mm has little to no impact on the strength of the bondline. When reaching a bonding length of 500mm this declination is increasing and a conclusion could be made that after the 1 meters of bonding length have been reach, even though a higher length than this can't be modelled here, the pullout strength is heavily declining. Therefore, instead of using longer bonding lengths than about the 350mm recommended, a suggestion could be made that using multiple rods. This would further increase the capacity of the connection in a more practical manner. One problem that could be realised though would be a problem of size and space within the connection itself.

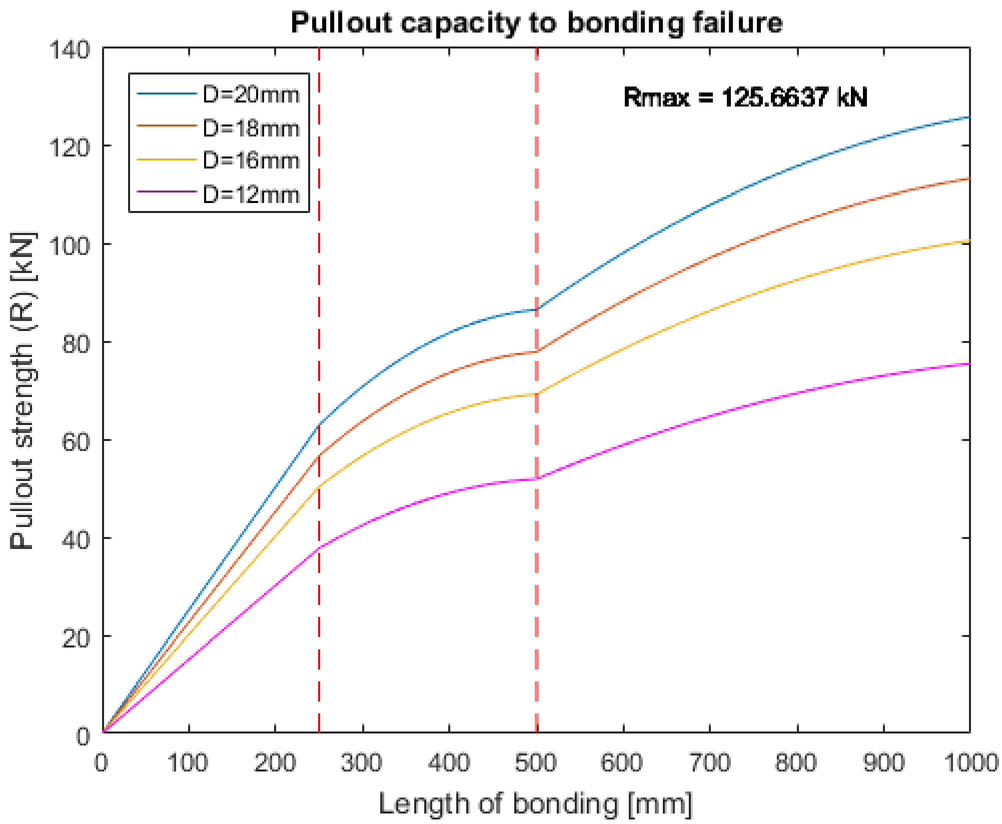


Figure 4.1: Plot of pullout capacity due to bonding failure

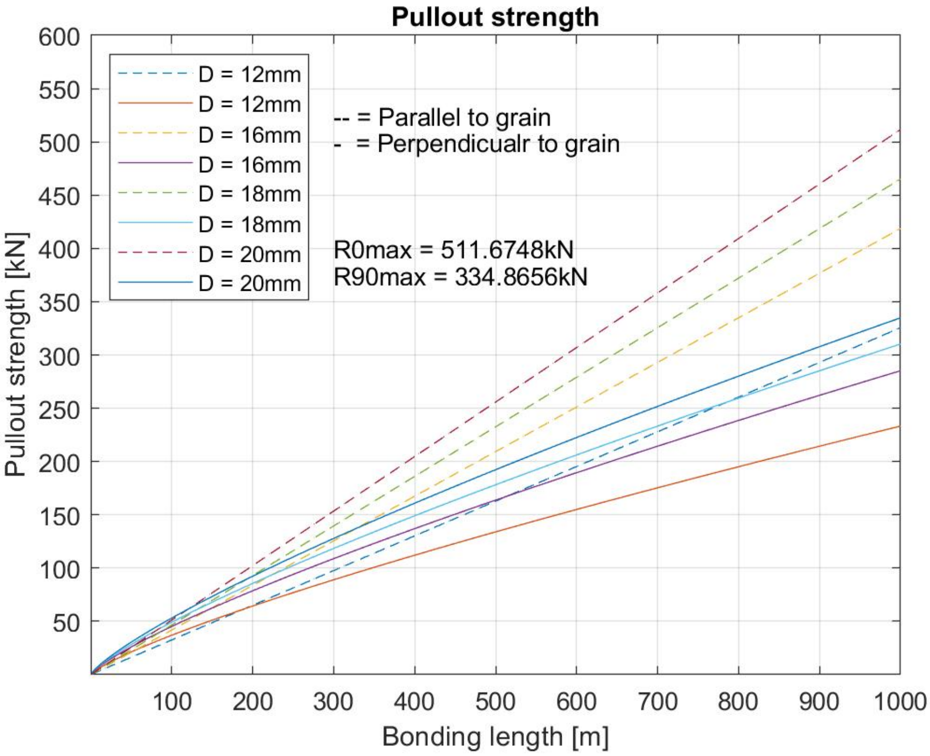


Figure 4.2: Plot of pullout strength.

Figure 5.1 only considers bond-line failures and 5.2 is taking into consideration the whole connection.
As presented in fig. (4.3), the parallel bonding strengths does not really apply. This is due to the fact that shear strengths used in this case was set to $7.8\frac{N}{mm^2}$, which the function itself is preset to instead of $5.8\frac{N}{mm^2}$ used in [26]. The density of the timber is also decreased to $380\frac{kg}{m^3}$ from $440\frac{kg}{m^3}$ thus resulting in the differences. When these values are lined up to correspond to each other we can see that in fig. (4.4) related to fig. (4.3) the values for an anchorage/bonding length of $500mm$ approximately lines up. For the [26] we can see that a characteristic strength level of about $140-150kN$ is achieved while in the [29] design recommendations we get around $175kN$. When checking the anchorage length of $250mm$ for both cases, we get a strength of approximately $100kN$ for [26] and about $80kN$ for [29]. These comparisons are shown in table 4.4 below.

Table 4.4: Comparisons between models with characteristic strength values from Tlustochowicz et al [29] and Rossignon et al [26].

Anchorage length	Tlustochowicz, Serrano and Steiger	Rossignon and Espion
500 mm	175 kN	140 kN
250 mm	80 kN	100 kN

The slight difference here is due to the linear approximation done by Tlustochowicz, Serrano and Steiger [29]. The values that are compared are the mean values. The calculations used within the work of Rossignon and Espion [26] is focused on the eqs. (4.10) shown below.

$$F_{ax,mean} = (0.15\lambda^2 + 9.24\lambda)(\frac{d_r}{16})^{1.5}\ [kN]\ \rightarrow\ F_{ax,k} = 0.76F_{ax,mean} \tag{4.10}$$

These equations, which is more representative of the problem at hand, doesn't however take the density ρ or the direction to the grain into consideration. These equations are only applicable to the following scenarios, bullet point schedule gathered from [26].

- Slenderness ratio in the range of
- Rod diameter in the range of 12 - 14mm
- Bond line thickness t = 4mm
- Glulam beams of GL24 quality
- Service class 1
- Structures exposed to reasonable temperature(maximum temperature at 60'C and minimum temperature 40'C]
- Structures loaded statically or quasi statically
- Parallel to grain

Therefore, the design guidelines discovered by Tlustochowicz, Serrano and Steiger [29] could be seen as more applicable than the ones shown by Rossignon and Espion [26]. Even though [26] states that the linear approach is not suggested. With linear approach we assume a linear stress-strain relation for the materials. Something to have in mind though, is that the model made by Tlustochowicz et al. [29] are as previously mentioned assuming steel rod plasticity. Therefore, recommendations to keep the anchorage length of the rod not longer than 350mm should be followed.

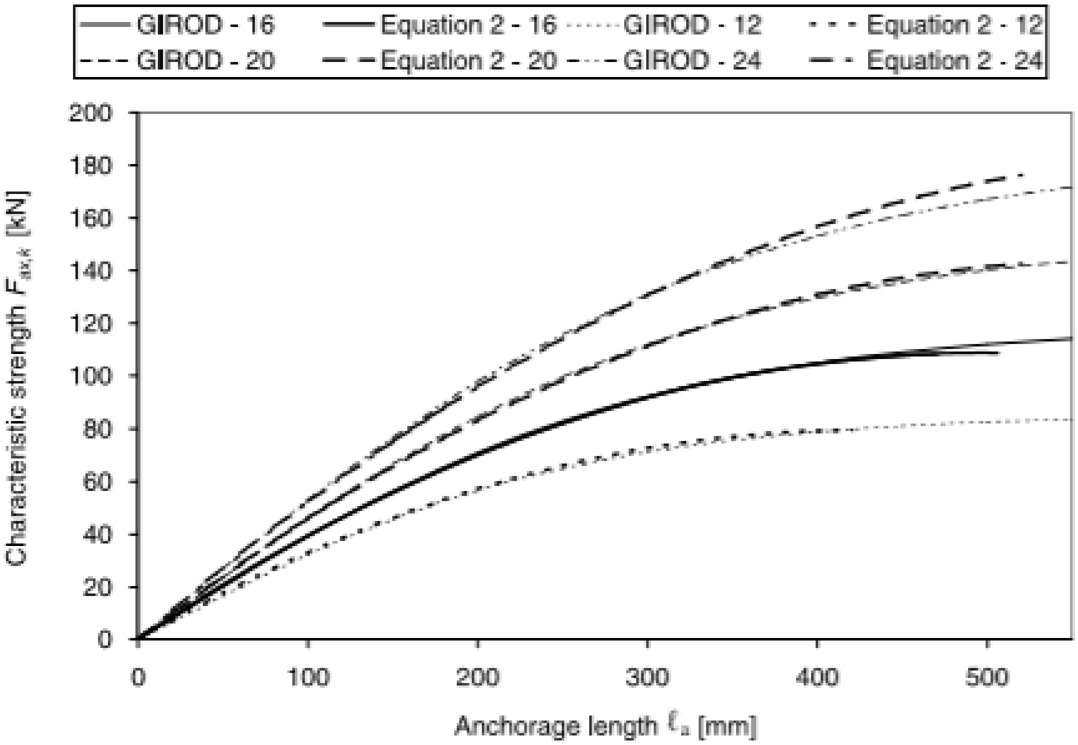


Figure 4.3: Figure showing the strength performance contra bonding length from [26]

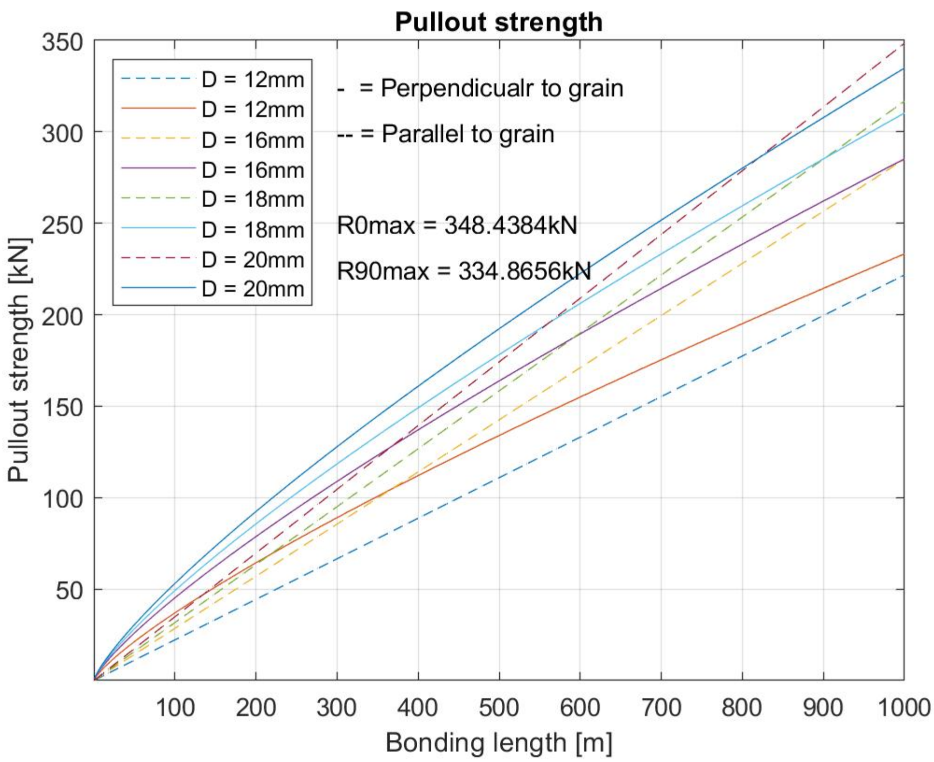


Figure 4.4: Matlab script with $\rho = 440\frac{kg}{m^3}$ and shear strength set to $5.8\frac{N}{mm^2}$

4.3.0.1 Fatigue

Myslicki et al. states in [23] that there is a considerable optimisation potential to the model. Its stated that more test results needs to be gathered in order to make of reliable modelling [23]. It was also seen that a failure mode within the wood and adhesive for low cycle analysis, while in the high cycle range a regular fracture ended up within the rod itself [23]. For example, [23] uses rod dimensions of $d = 16mm$ which makes it difficult to relate to any other diameter.

The initial plot constructed can be seen in fig. 4.5, this is later on custom to fit other test results.

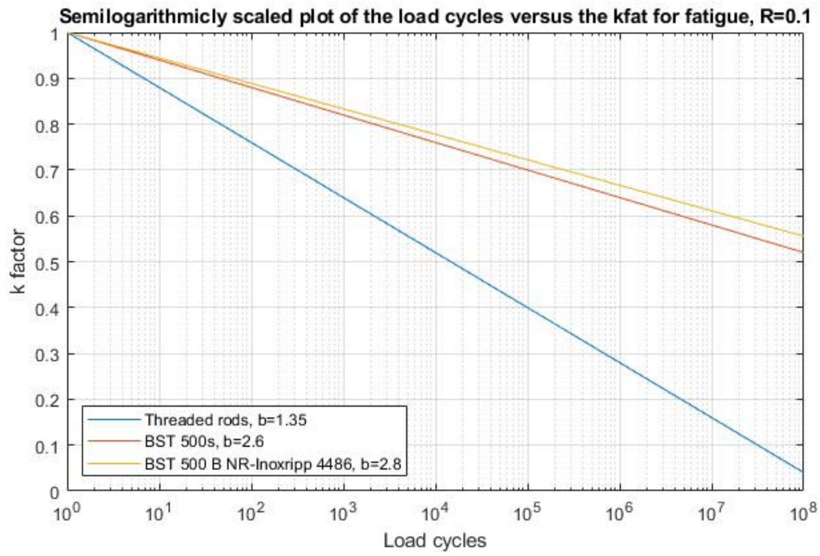


Figure 4.5: Semilogarithmicly scaled plot of the load cycles versus the k_fat factor for fatigue, using R=0.1.

To evaluate these models and their a and b values, we further on try to compare these test results with real life lab results to clearer see how good the representation actually are. One of a couple of lab tests results that has been retained are the test made by Bainbridge, Mattem from TRADA Technology as well as Harvey and Ansell from University of Bath [1]. The tests were focused on relating fatigue performance to adhesive bonding. There was a total amount of three adhesives tested which showed a couple of fatigue performances. The article shows graphs of the fatigue behaviour of the different glue types bonded with mild steel rods into glulam specimens of strength class C35. The graphs are hard to read and are therefore approximated and reproduced by ourselfs. That is why a reconstruction of the linear approximation was conducted via a matlab script, implementing the start and end values of the different graphs to plot three quality improved fig. (4.6), (4.7) and (4.8)

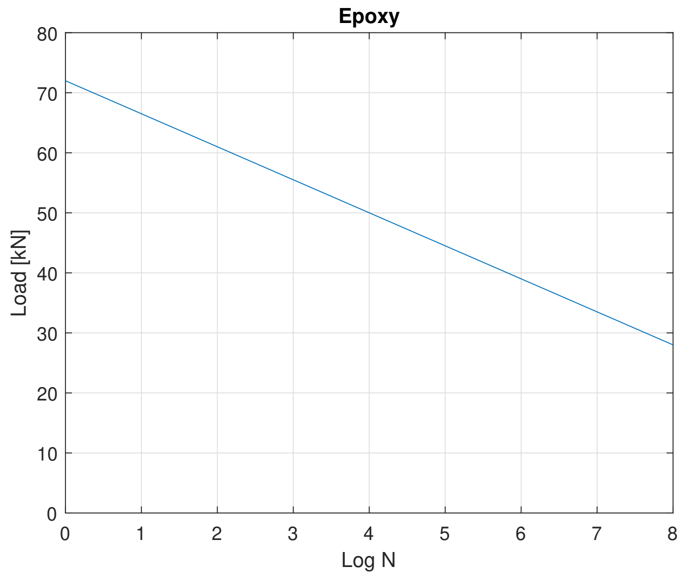


Figure 4.6: Reproduced load-LogN curve for 16mm EPX bonded mild steel rods at R=0.1 [1]

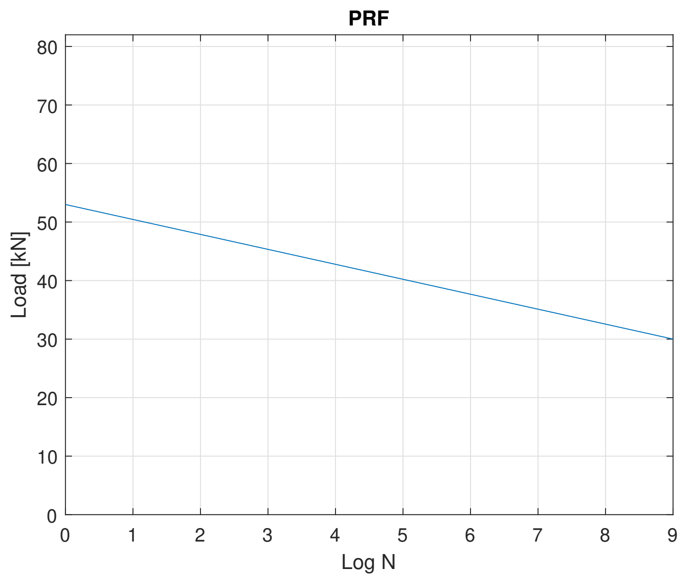


Figure 4.7: Reproduced load-LogN curve for 16mm PRF bonded mild steel rods at R=0.1 [1]

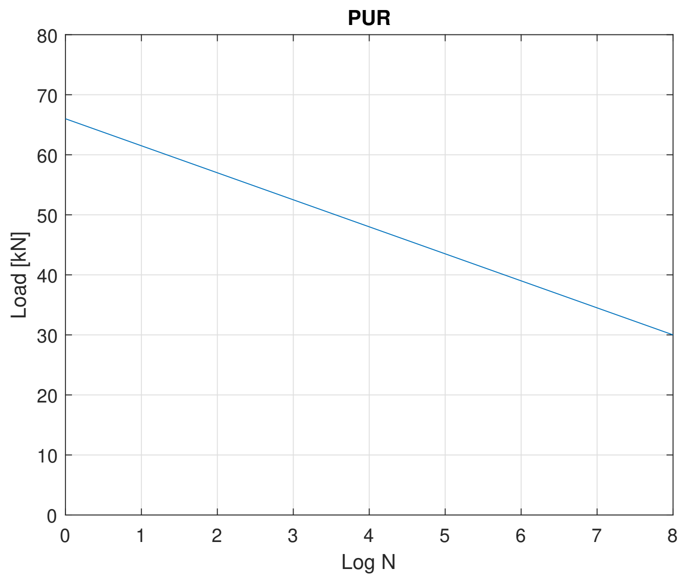


Figure 4.8: Reproduced load-LogN curve for 16mm PUR bonded mild steel rods at R=0.1 [1]

If results from [1] are compared to the results from the formulas and matlab scripts appended. We can, even though some variables are shifted for example glue, steel type and strength class of the timber, see a clear relation between the two graphs. An example is the graph gathered from the matlab scripts with the maximum force set to $72kN$ in order to as good as possible represent the Epoxy adhesive configuration. If we compare the results from the scripts in fig. (4.9) with the results gathered from the epoxy SN curve in fig. (4.6), we see that except some slight differences in terms of the results, we notice an otherwise excellent representation.

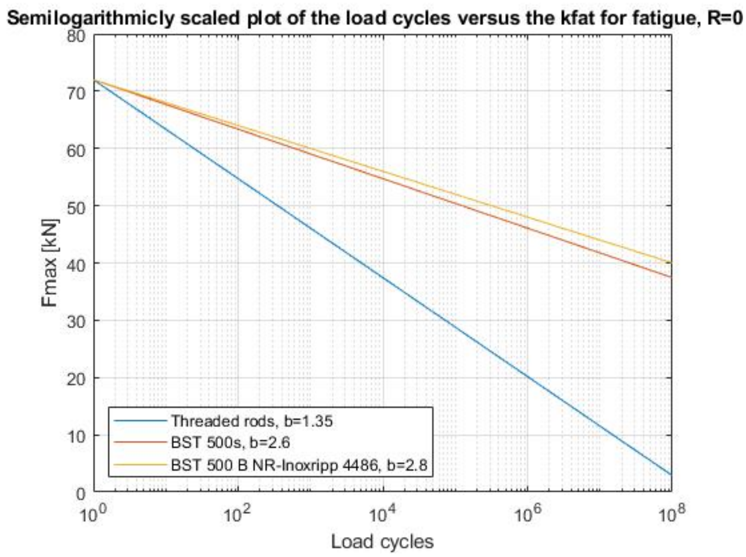


Figure 4.9: Load cycle versus k-factor with a maxmimum force set to $72kN$.

The same can be witnessed when comparing the PRF graph in fig. (4.7) with maximum force plot of $53kN$ in fig. (4.10). This can give us the interpretation that this calculation model can be a good representation of the glued in rod model as such. But as previously mentioned, more studies and numerical analysis needs to be established in order to find proper a and b values for all configurations.

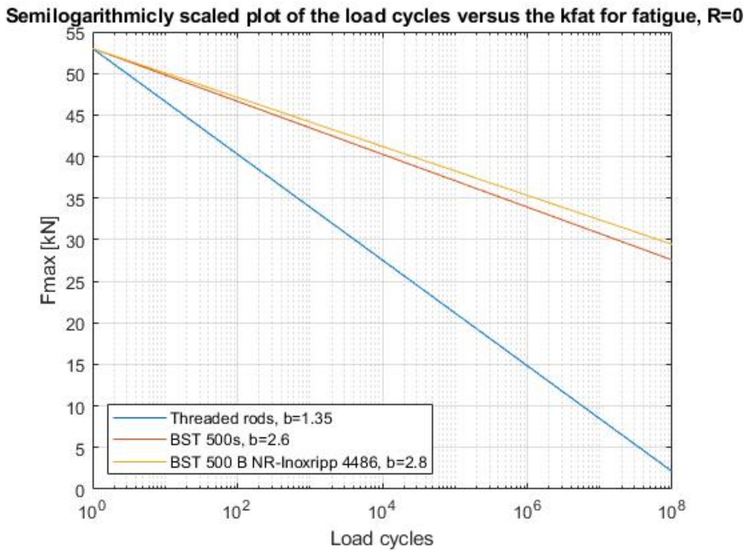


Figure 4.10: Load cycle versus k-factor with a maxmimum force set to $53kN$.

Figure 4.13 represent the connection failure in the connection as in general, expecting a non steel failure design.

5 Conclusion

From the various reports that have been read and summarized, we can draw some conclusions for the important parameters. From the mathlab test, similar results were received as from the relevant reports with a highest pullout load at 125kN, for a specific anchor length and rod diameter.

If possible the density of the timber material should be taken as high as possible since it has been shown that the density has an affect on the fatigue resistance, this is especially important for low level of stress intensity.

It is clear that different cases and parameters are relevant when talking about low cycle fatigue and high cycle fatigue that we have in this case. For HCF, there will be millions of load cycles which the construction will need to be able to manage. This puts the fatigue properties of the connection to be much more relevant than usual. When it comes to the timber material, it has to be treated or chosen in a way that minimize impurities in order to have as few stress concentration points for fatigue loading as possible.

To obtain an optimal connection some preparation should be made, in order to achieve full connection for the rod, adhesive and timber some pre-application that can be used are clean and pre-drill holes thus optimize the connection.

Further on after analyzing other material usage and geometry properties, a conclusion is herein made that the use of any other rod material than steel will have an influence on the performance.

The main problem with composite materials is the fact that these materials have a brittle failure mode which strictly needs to be avoided. For the material of which the rod is to be inserted in, one can make the conclusions that using laminated veneer lumber would benefit in terms of the problems glulam for example has with grain angle. The load to grain angle for Laminated veneer lumber is similar to glulam. Kerto-Q has a bit more of a homogeneous structure. The grain angle has a high impact on the fatigue properties. The grain angle should not exceed 10 degrees in any case.

Anyway, if the strength properties are studied we can clearly witness that the shear modulus for gl32h gluman has a mean value of $650 MPa$ while the laminated veneer lumber has a mean value of $600 MPa$. The question of using glulam or laminated veneer lumber remains unstated and is up to the designer to decide in the general subject, but for this particular case were fatigue performance matters the most, one need to consider the safety of using the laminated veneer lumber. When it comes to the decision of designing with hardwood and softwood respectively, we make the decision to recommend softwood in terms of fatigue performance in this case. Even though various reports investigated generally handle hardwood for static performance.

For the bond-line we can safely conclude that a thickness of $2mm$ is the optimal thickness out of a fatigue perspective, since it has been confirmed from several sources. Initially, the adhesive investigation witnessed a fatigue related bonding benefit in a certain adhesive type. Anyhow, after several reports have been investigated and compared, the decision to assume this fatigue related benefit to be result noise in a particular test were made. We finally came to the conclusion that using an 2-component EPOXY or PUR glue gave the best results both fatigue wise as well as statically. These adhesive types are commonly used in timber connection and seeing that they perform well in a fatigue perspective implies that the use of these will most likely increase even more. When using smaller rod diameters, the PUR resin has a slightly higher performance fatigue wise and when using higher rod diameters the two resins perform equally as well. The decision is therefore up to the designer.

The bonding length, after considering the fatigue and static design recommendations given in this thesis, should be kept at less than $350mm$. This is to keep the failure mode to happen in the steel rod. We need to remember that we want a plastic and a reliable connection which is achieved in this manner. After investigating several reports, one could also make the conclusion that an increase of the rod diameter beyond $16mm$ gives a very slight tensile stress reduction. this could be due to the fact that the steel rod area is increasing.

The configuration of the connection should be such that the distance to the nearest glued in rod connection is kept at 3 times the rod diameter d. The distance to the nearest edge should either exceed 2.5 times the rod distance d. This follows the directions of the french professional guidelines as almost all the articles and reports investigated maintains.

The investigated design recommendations makes for a good design basis. Using a k_{fat} factor which should be multiplied with the ultimate static design strength in this way is a good way to reliably represent the connection. However, the fatigue performance checks investigated has good arbitrary coefficients for a and b to make for a good representation, but the a and b values needs to be expanded further for more configurations. As of now the a and b values are only reliable for certain strict limitations which needs to be expanded for proper design usage.

5.1 Concluding remarks

This thesis should be finished with some remarks in which the results and conclusions unfolds into. The remarks and notices are presented with a bullet point schedule visualized below.

- Approximated pullout load capacity from $0kN$ to $125kN$ (Mathlab results).
- The density of the timber should be taken as high as possible.
- Pre-drilling, cleaning and evenly distributing the adhesive within the hole is essential to prevent stress concentrations as well as digestion and optimize adhesion.
- The main material focus of the thesis turned out to be Gl32h gluam.
- Fatigue performance recommendations
    1. Optimal bond-line thickness of 2mm.
    2. Recommended bonding length of 350mm.
    3. Recommended rod diameter of 16mm.
    4. Recommended adhesive, 2-components Epoxy or Polyurethane.
    5. A recommended fatigue performance design procedure, which also needs additional work in order to be able to relate to all possible configurations.

5.2 Suggestions for future work

For the future fatigue guidelines, the a and b coefficients should be thoroughly further investigated. If the design recommendation is to be viable, the limitations of the calculations needs to be widen. If more a and b coefficients would be produced, the fatigue design recommendation would be highly viable for the general design regimes. Further on, more basic knowledge about the fatigue in timber needs to be investigate to achieve an even better understand about the phenomenon.

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A
Appendix 1

```
%This script plots the pullout strength with regard to the bonding failure
%mechanism of glued-in rod connections. Rod made out of steel and timber
%out of glulam. The calculations further into this script,plotting the
%general pullout strength,uses the density of GL32c quality. Parameters
%gathered from Swedish timber and Tlustochovic's state-of-the-art review of
%timber connections with glued in rods.

close all;clc;clear all

d=20;                                                                     %↙
Diameter of the rod
for d=[12 16 18 20]
r=d/2;
i=0;                                                                      %Since↙
1 is adding in the beginning in each loop
x=linspace(1,1000,1000);                                                  %making↙
a values for simplifying plot, these could be done more nicely but is nothing we↙
but too much time into
for lad=1:1000                                                            %For↙
loop for the values interesting of lad
        i=i+1;                                                            %Adding↙
1 for upcoming loop
    if lad<=250 && lad>0                                                  %First↙
equation boundaries
        fkn=4;
    elseif lad<=500 && lad>250                                            %Second↙
equation boundaries
        fkn=5.25 - 0.005*lad;
    elseif lad<=1000 && lad>500                                           %Last↙
equation boundaries
        fkn=3.5  - 0.0015*lad;
    else
        error("Bonding length (lad) seems not be be within the limitations for the↙
calculations")
    end
    fk(i)=fkn;
end
clear i
for i=1:1000                                                              %for↙
loop to simplify the R calculation in vector form
R(i)=pi*d*x(i)*fk(i)*10^-3;
end
Rmax=max(R);
figure(1)
plot(x,R);
hold on

%% Parallel to grain
dh=d+2;                                                                   %↙
Diameter of hole, rebar + 2mm
lv=5*d;                                                                   %↙
Length, reduced cross-section
lambda=lv/r;                                                              %↙
```

```
slenderness (7.5-15)

if lambda>15                                                                    %Error↙
om slenderness är utanför tillåtet spann
    error("Slenderness extends the limitations for the calculations")
elseif lambda<7.5
    error("Slenderness extends the limitations for the calculations")
end

rho=440;                                                                        %↙
Density GL32c
fv0mean=    7.8*    ((lambda/10)^(-1/3))    *   ((rho/480)^0.6);                %↙
Equation for parallel to grain, tlustochowich

i=0;                                                                            %Since↙
1 is added in the beginning of each loop
for l=1:1000
    i=i+1;                                                                      %Adding↙
1 for upcoming loop
    faxmean(i)=    fv0mean*pi*dh*l;                                             %↙
Formula from tlustochowic
end
faxmean=faxmean*10^-3;                                                          %N->kN
f0max=max(faxmean);                                                             %Names↙
the maximum value
figure(2)                                                                       %New↙
figure
plot(x,faxmean,"--");                                                          %Plot
hold on
%% Perpendicular to the grain

k1=0.045;                                                                       %↙
Constants for perpendicular
k2=0.8;

i=0;
for l=1:1000
    i=i+1;
    Ag=l*pi*dh;                                                                 %↙
Formula perpendicular from tlustochovic
    fax90mean(i)=k1*Ag^(k2);
end
f90max=max(fax90mean);                                                          %Names↙
the maximum value
plot(x,fax90mean);                                                          %Plot

end
%% plots

figure(1)
xlabel("Length of bonding [mm]");ylabel("Pullout strength (R) [kN]");        %↙
Plotting
title("Pullout strengt due to bonding failure");grid on
```

```
plot([250 250],[0 140],"--r");plot([500 500],[0 140],"--r");                %Lines↙
for the boundaries of the equations
yticks([10 20 30 40 50 60 70 80 90 100 110 120 130 140]);
xticks([100 200 300 400 500 600 700 800 900 1000]);
text(400,130,"Rmax = " + Rmax + "kN")                                         %↙
Text stateing the maximum value of the pullout strength
legend("D = 12mm","D = 16mm","D = 18mm","D = 20mm","location","northwest")

figure(2)
yticks([50 100 150 200 250 300 350 400 450 500 550 600]);
xticks([100 200 300 400 500 600 700 800 900 1000]);
xlabel("Bonding length [m]");ylabel("Pullout strength [kN]");
grid on; title("Pullout strength")
legend("Parallel to grain","Perpendicular to grain");
text(300,400,"R0max = " + f0max + "kN")
text(300,375,"R90max = " + f90max + "kN")
text(300,525,"-- = Parallel to grain")
text(300,500,"-  = Perpendicualr to grain")
legend("D = 12mm","D = 12mm","D = 16mm","D = 16mm", "D = 18mm","D = 18mm","D =↙
20mm","D = 20mm","location","northwest")
```

B
Appendix 2

```
clc;clear all;close all

R=0.1;                                                                                          %stress↙
ratio of 1 indicates a static fatigue analysis,0 indicates loadings that fluctuat from↙
side to side
t=100;                                                                                        %Fatigue↙
life of 100 years
a=6;
b=[1.35
   2.6
   2.8];
beta=1;                                                                                       %↙
consequens of damage, 1=without, 3=considerable
N=linspace(1,10e7,10e5);                                                                      %Number of↙
cycles
Nobs=N/t;                                                                                     %Number of↙
cycles per year
for i=1:length(b)
    kfat=  ( (1- (     (1-R)    /   (a*(b(i)-R))     )   *    log10(beta*Nobs*t)));         %↙
K-factor formula from Myslicki
    figure(1)
    semilogx(N,kfat)                                                                          %Semi↙
logatrithmic plot
    hold on
end
axis manual
axis([0 10e7 0 1])
xlabel('Load cycles')
ylabel('k factor')
grid on
legend('Threaded rods, b=1.35','BST 500s, b=2.6','BST 500 B NR-Inoxripp 4486, b=2.↙
8','location','southwest')
title('Semilogarithmicly scaled plot of the load cycles versus the kfat for fatigue,↙
R=0')
```