In-plane mechanical properties of birch plywood and its performance in adhesively bonded connections

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Stockholm, Sweden 2024
Abstract
Birch (*Betula* spp.) is a hardwood species with a wide natural distribution on the Eurasian continent, especially in northern Europe. Compared with conventional plywood made from softwood, birch plywood has favorable mechanical properties that could be used in new types of efficient connections for timber structures, and thus enable a substitution of current systems using slotted-in steel plates. Such new connections could result in significant advantages in terms of environmental impact and economy as well as ease of prefabrication and assembly. However, birch plywood has rarely been utilized in connections, and therefore, there is a lack of knowledge necessary to design safe timber structures applying such connection systems. In particular, there is a need for increased knowledge of the mechanical properties of birch plywood and its structural performance under various loading conditions. Current connections in timber structures usually also involve mechanical fasteners, e.g., steel screws and dowels, but there is limited use of adhesively bonded (glued) connections.

The aim of this thesis is to gain new knowledge required for the development of adhesively bonded connections using birch plywood as gusset plates in structural applications. Examples of such structural applications are timber trusses and portal frames. In this context it is necessary, first, to characterize the in-plane mechanical properties of birch plywood, and second, to investigate its performance in adhesively bonded connections.

The results of the mechanical testing show that birch plywood possesses the highest and lowest tensile, compressive, and bending strength and elastic modulus at 0° (parallel) and 45°, respectively, to the face grain (the fiber direction of the face veneers). The opposite findings were noticed for the shear strength and the shear modulus. All these strength values are similar to or higher than the corresponding strength values of common softwood structural timber in its longitudinal direction. Moreover, a size effect on the in-plane bending strength property was observed at 0° and 90° to the face grain but not at other angles, which is attributed to different failure mechanisms. Based on the experimental work, both analytical and
numerical models to predict the in-plane mechanical properties of birch plywood are proposed.

Three different adhesives systems were used in the studies: melamine-urea-formaldehyde (MUF), phenol-resorcinol-formaldehyde (PRF), and a two-component polyurethane (2C PUR). All adhesives used show adequate bonding strength between birch plywood and spruce glulam. However, the use of the adhesive systems should be further investigated in the future. The different manual pressing methods investigated show no significant influence on the bonding strength. Moreover, the bonding strength changes within a relatively small range when the loading direction is varied from 0° to 90°, which is beneficial for the design of birch plywood in adhesively bonded connections. A clear correlation exists between the bonding strength and the shear strength of the weakest wood adherend.

In addition, the moment capacity and bending stiffness of adhesively bonded connections using birch plywood were determined experimentally as well as by analytical and numerical models with a satisfactory agreement.

In timber connections, especially those that are prevalently loaded in tension and/or compression (e.g., in timber trusses), the contribution of the plywood width on the load-bearing capacity needs to be quantified. The results show that the tensile strength of birch plywood within the bonded area shows very low angle-dependence. This is possibly due to the restricted crack propagation at 22.5° and 45° when the gap between the bonded regions is small. The tensile capacity of birch plywood loaded at 0°, 22.5°, and 45° reaches a plateau at certain widths of the gusset plate, which can be well predicted and explained by the spreading angle theories proposed in this study.

In the future, more studies are required for the further development of the adhesively bonded connections with birch plywood. Some preliminary studies serving this purpose have been presented in the thesis as on-going work.

**Keywords**
Birch plywood, in-plane mechanical properties, adhesively bonded connections, gusset plates, load-to-face grain angle, spreading angle.
**Sammanfattning**


Syftet med denna avhandling är att ta fram ny kunskap nödvändig för en utveckling av limmade förband med skivor av björkplywood i bärande träkonstruktioner. Exempel på sådana träkonstruktioner är fackverk och portalramar. I detta sammanhang är det nödvändigt att dels karaktärisera de mekaniska egenskaperna i planet hos björkplywood, dels att undersöka dess prestanda i limmade förband.

Resultaten av de mekaniska testerna visar att björkplywood har den högsta och lägsta drag-, tryck- och böjhållfastheten och elasticitetsmodulen vid 0° (parallellt med) respektive 45° relativt ytfanérens fiberriktning. De motsatta resultaten noterades för skjuvhållfasthet och skjuvmodulen. Alla dessa hållfasthetsvärden är liknande eller högre än de motsvarande hållfasthetsvärdena för vanligt konstruktionsvirke av barrträ i dess längdriktning. Dessutom observerades en storlekseffekt för böjhållfastheten i planet vid 0° och 90° relativt ytfibrerna men inte vid andra vinklar, vilket tillskrivs olika brottbeteenden. Utifrån det experimentella arbetet föreslås både analytiska och numeriska modeller för att förutsäga de mekaniska egenskaperna i planet hos björkplywood.
Tre olika limtyper användes i studierna: melamin-urea-formaldehyd (MUF), fenol-resorcinol-formaldehyd (PRF) och tvåkomponentens polyuretan (2C PUR). Alla använda limtyper uppväxer tillräcklig styrka mellan björkplywood och limträ av gran. Dock bör användningen av limtyperna utredas vidare i framtida studier. De olika manuella limpressmetoderna som undersöks visar ingen signifikant inverkan på limfogens styrka. Dessutom ändras limfogens styrka marginellt när belastningvinkeln i förhållande till plywoodens yta varieras från 0° till 90°, vilket är fördelaktigt vid utformandet av björkplywood i limmade förband. Det finns en tydlig korrelation mellan limfogens styrka och skjuv hållfastheten hos det svagaste träsubstratet.

Vidare bestämdes momentkapacitet och böjstyrhet hos limförband med björkplywood experimentellt samt genom analytiska och numeriska modeller med tillfredsställande överensstämmelse.

I träförband, särskilt de som belastas i drag och/eller tryck (exempelvis i träfackverk), måste plywoodens bredd i förhållande till bär förmågan beräknas. Resultaten visar att draghållfastheten hos björkplywood inom det limmade området inte påverkas av ytfanerens fiberriktning i förhållande dragriktningen. Detta beror möjligen på den begränsade sprickutbredningen vid 22,5° och 45° då mellanrummet mellan de limmade områdena är litet. Draghållfastheten hos björkplywood belastad vid 0°, 22,5° och 45° når en platå vid vissa bredder hos plywoodskivan, vilket förutsägs väl och förklaras genom de teorier för spridningsvinkeln som föreslås i denna studie.

I framtiden krävs fler studier för vidareutveckling av limmade förband med björkplywood. Vissa preliminära studier som tjänar detta syfte har också presenterats som pågående försök i avhandlingen.

**Nyckelord**

Björkplywood, mekaniska egenskaper i planet, limmade förband, belastningsvinkel (relativt ytfanérens fiberriktning), spridningsvinkel
Preface

The work presented in this doctoral thesis was carried out at the Division of Building Materials, Department of Civil and Architectural Engineering, KTH Royal Institute of Technology in Stockholm, Sweden. The China Scholarship Council and Svenskt Trä are gratefully acknowledged for the financial support. The work has also been supported by the Vinnova project 2017-02712 “Bärande utomhusträ” within the BioInnovation program as well as the Kamprad Family Foundation (reference number: 20200013) and from Produktion2030, a strategic innovation program supported by Vinnova [reference number: 2021-03681], Swedish Energy Agency, Formas including the industry partners.

Firstly, I would like to express my deepest gratitude to my supervisors Prof. Roberto Crocetti and Prof. Magnus Wålinder. Both of you are the main supervisors in my mind. I started to learn timber by taking your course Building Materials in late 2017. Timber is warm and beautiful, and I was attracted to learning more about it. Thank you for taking me on board and supervising my master thesis in 2019 and then my doctoral studies since 2020. I would like to thank Roberto for your invaluable guidance and insightful advice. Much of your thinking inspired me deeply and guided my research work. Thanks Magnus for introducing me to the field of wood science. I gleaned some knowledge about wood from time to time, attended wood conferences (mainly WSE), and published one article in a wood journal, thanks to your supervision and encouragement.

I would like to extend my thanks to Dr. Lars Blomqvist, at RISE Research Institutes of Sweden. You have been highly involved in the second half of my doctoral studies. And you are always willing to share your expertise and experience with me.

Now, it is time to give special thanks to my colleague and friend Yue Wang. In this small plywood research group, we worked very close to each other, spent a lot of time together taking courses, conducting experiments, and addressing small and big problems. We always learn something from each other. I also want to thank another new colleague, Mattia Debertolis, for your help and communication about the work. I wish you all the best in future research and orienteering.
Proceeding to the industry partners, Koskisen is sincerely acknowledged for supplying birch plywood materials from the beginning of the project. Moelven is deeply thanked for supplying the glulam beams and offering help to perform the tests in the factory. Dynea must be thanked for supplying the melamine-urea-formaldehyde and phenol-resorcinol-formaldehyde adhesives. I need to sincerely thank the contact person, Ronny Bredesen, for your guidance and expertise in bonding techniques. Thanks to Rubner Holzbau and the CNR Institute of Bioeconomy for the help with the project. Thanks Rothoblaas for the technical communication.

Doctoral studies involve a large number of laboratory tests. A big thank you to the lab manager, Viktor Brolund, for your effective assistance. Thanks to Gürsel Hakan Taylan and CBI Concrete Institute for access to the equipment. Thank you to Dr. Hassan Abdulazim Fadil Mohammed for your guidance on the testing machine.

I have been involved in some diploma thesis work related to the birch plywood studies. Many thanks to the students I have worked with: Pontus Wedin, Patrik Hedlund, Jonatan Ringaby, Agnė Laurinavičiūtė, Farid Mobin, and Ibrahim Abd Ullah Alhamo.

Moreover, I wish to sincerely thank Dr. Michael Schweigler, Lecturer at Linnaeus University, who co-supervised my master thesis and taught me research skills. Now you also follow the progress of my doctoral studies.

I wish to express my gratitude to Dr. Bert Norlin for the review of the licentiate thesis and the doctoral thesis.

The people that have helped me are far more than the ones mentioned. Thanks you all for the help.

Finally, I would like to thank my family and friends in China and Sweden, and most importantly, my wife Chui Ping Lai, for their endless support, especially during the pandemic period of this long journey.

Stockholm, January 2024
Tianxiang Wang
汪天享
List of appended papers

This thesis is based upon the following scientific articles referred to in the text by their roman numbers:


IV. Wang, T., Wang, Y., Debertolis, M., Crocetti, R., Wålinder, M., Blomqvist, L. (2024). Bonding strength between spruce glulam and birch plywood at different load-to-plywood face grain angles. Submitted manuscript.


VI. Wang, T., Wang, Y., Debertolis, M., Crocetti, R., Wålinder, M., Blomqvist, L. (2024). Spreading angle analysis on the tensile capacities of birch plywood plates in adhesively bonded timber connections. Submitted manuscript.

In the appended papers, the first author planned and performed the majority of the experiments, analyzed the analytical and numerical models, and wrote the manuscript with the help of the co-authors.
Summary of papers

Paper I
This paper presents new experimental data that could serve as the input in the analytical or numerical models for the application of birch plywood under various loading conditions. Specifically, in-plane mechanical properties of birch plywood were investigated under five different angles to the face grain, i.e., from 0° (parallel) to 90° (perpendicular) to the face grain, with an interval of 22.5°. The stress-strain relationships, failure modes, strength, and elastic properties of birch plywood are highly dependent on the load-to-face-grain angle. The strength and the elastic properties were also predicted by various analytical and empirical models.

Paper II
Birch plywood exhibits outstanding in-plane mechanical properties with regard to tensile, compressive, and shear behaviors according to the findings in Paper I. However, the edgewise bending strength and stiffness, which are often critical for the design of gusset plates in moment-resisting structures, had not yet been investigated thoroughly. Moreover, in engineering applications, the size and moisture content of the plywood plate are in general very different from those adopted in laboratory testing according to current standards. This paper investigates the influence of face grain angle, size, and moisture content on the edgewise bending strength and stiffness of birch plywood. Analytical and numerical models, both taking non-linear elasto-plastic compressive behaviors into account, were developed for the prediction of the ultimate moment capacity based on different failure definitions. Moreover, clear relationships between the bending strength and elastic modulus were observed.

Paper III
In the paper, a proper workflow of adhesive bonding birch plywood with other timber parts, e.g., spruce glulam, that results in adequate bonding strength in timber connections was investigated. Tensile shear tests were conducted on produced bonded joints to evaluate the bonding strength.
Three different adhesive systems were used in the studies: melamine-urea-formaldehyde (MUF), phenol-resorcinol-formaldehyde (PRF), and two-component polyurethane (2C PUR), including tests in both dry and moist conditions. The influence of three pressing methods, i.e., (a) screw-gluing, (b) clamping by means of clamps, and (c) clamping by application of weight loads, on the bonding strength was investigated. The bonding strength was thereafter compared to the shear strength of spruce glulam. The wood failure percentage was also examined in this study.

**Paper IV**

When employing birch plywood in structural applications such as trusses and frame corners, stresses from different directions need to be transmitted by the plywood gusset plate. However, it is still uncertain how the bonding strength is affected by different loading angles to the face grain. This research question, specifically concerning the bonding strength between birch plywood and spruce glulam, was studied in this paper. It was found that the bonding strength varies within a relatively small range when the load-to-plywood face grain angle varies from 0° to 90°, which is promising for the development of adhesively bonded joints. Failure mainly occurred in glulam at 0° and 15°, while at other angles, a mixture of failure in glulam and plywood face veneer was dominant. The weak angle-dependence of the bonding strength can be explained by further checking the shear strength of the weakest wood adherend, i.e., a comparison of the shear strength between the spruce glulam and the birch plywood face veneer. A clear correlation was observed between the bonding strength and the shear strength of the weakest wood adherend.

**Paper V**

In this paper, two separate glulam beams were connected by birch plywood plates at mid-span and then loaded in four-point bending. Four test series with two different bonding areas and birch plywood face grain orientations were carried out. The bonded region was designed as the weakest part to investigate the failure modes, moment capacity, bending stiffness, and moment-rotation angle relationships. Furthermore, numerical models
were developed to predict the structural behaviors in the linear-elastic stage, while an analytical model was proposed to predict the moment-carrying capacity. Both the numerical and analytical models display satisfactory agreement with the test results.

**Paper VI**

This paper focuses on a particular application of birch plywood in adhesively bonded connections, namely, a node of a timber truss. The aim of this paper is to study the influence of the plywood width and the load-to-face grain angles on its load-bearing capacity in uniaxial tension. Specifically, the plywood width was increased step by step at three different load-to-face grain angles, 0°, 22.5°, and 45°, until the tensile capacity of plywood reached a plateau. It was found that the tensile strength of birch plywood within the bonded area shows very low angle-dependence. This is possibly due to the restricted crack propagation at 22.5° and 45° when the gap between the bonded regions is small, further resulting in different tensile strengths within and outside of the bonded area. The maximum tensile capacities at 22.5° and 45° are only slightly lower than that at 0° but the tensile capacity at different load-to-face grain angles reaches a plateau at different widths, which can be well predicted and explained by the spreading angle theories with the introduced concept of effective width and spreading angle.
# Nomenclature

## Abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>ANOVA</td>
<td>Analysis of variance</td>
</tr>
<tr>
<td>COV</td>
<td>Coefficient of variation</td>
</tr>
<tr>
<td>EWP</td>
<td>Engineered wood product</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear variable differential transformer</td>
</tr>
<tr>
<td>MC</td>
<td>Moisture content</td>
</tr>
<tr>
<td>MUF</td>
<td>Melamine-urea-formaldehyde</td>
</tr>
<tr>
<td>PRF</td>
<td>Phenol-resorcinol-formaldehyde</td>
</tr>
<tr>
<td>RH</td>
<td>Relative humidity</td>
</tr>
<tr>
<td>RMSE</td>
<td>Root mean square error</td>
</tr>
<tr>
<td>T</td>
<td>Temperature</td>
</tr>
<tr>
<td>WFP</td>
<td>Wood failure percentage</td>
</tr>
<tr>
<td>2C PUR</td>
<td>Two-component polyurethane</td>
</tr>
</tbody>
</table>

## Latin Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>Cross-sectional area [m$^2$]</td>
</tr>
<tr>
<td>$A_1$</td>
<td>Ratio between $E_y$ and $E_x$ [1]</td>
</tr>
<tr>
<td>$b$</td>
<td>Width of birch plywood in edgewise bending [m]</td>
</tr>
<tr>
<td>$b_0$</td>
<td>Width of glulam [m]</td>
</tr>
<tr>
<td>$b_p$</td>
<td>Width of birch plywood in a bonded connection [m]</td>
</tr>
<tr>
<td>$d$</td>
<td>Depth of birch plywood [m]</td>
</tr>
<tr>
<td>$E_i$</td>
<td>On-axis elastic modulus in i direction [N m$^{-2}$]</td>
</tr>
<tr>
<td>$E_{t(c)}$</td>
<td>Tensile or compressive elastic modulus [N m$^{-2}$]</td>
</tr>
<tr>
<td>$E_x$</td>
<td>On-axis elastic modulus in x direction [N m$^{-2}$]</td>
</tr>
<tr>
<td>$E_y$</td>
<td>On-axis elastic modulus in y direction [N m$^{-2}$]</td>
</tr>
<tr>
<td>$E_z$</td>
<td>On-axis elastic modulus in z direction [N m$^{-2}$]</td>
</tr>
<tr>
<td>$E_\theta$</td>
<td>Elastic modulus at $\theta$ to the face grain [N m$^{-2}$]</td>
</tr>
<tr>
<td>$E I_{exp}$</td>
<td>Experimental bending stiffness [N m$^2$]</td>
</tr>
<tr>
<td>$E I_{num}$</td>
<td>Numerical bending stiffness [N m$^2$]</td>
</tr>
<tr>
<td>$f_b$</td>
<td>Edgewise bending strength [N m$^{-2}$]</td>
</tr>
<tr>
<td>$f_t$</td>
<td>Tensile strength [N m$^{-2}$]</td>
</tr>
<tr>
<td>$f_{t,b_0}$</td>
<td>Tensile strength with a small gap [N m$^{-2}$]</td>
</tr>
<tr>
<td>$f_{t,b_0,l}$</td>
<td>Tensile strength with a longer gap [N m$^{-2}$]</td>
</tr>
<tr>
<td>$f_{t,dumbbell}$</td>
<td>Tensile strength (dumbbell-shaped) [N m$^{-2}$]</td>
</tr>
<tr>
<td>$f_v$</td>
<td>Bonding strength [N m$^{-2}$]</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$f_v(0-0)$</td>
<td>Bonding strength (glulam $0^\circ$, plywood $0^\circ$)</td>
</tr>
<tr>
<td>$f_v(0-22.5)$</td>
<td>Bonding strength (glulam $0^\circ$, plywood $22.5^\circ$)</td>
</tr>
<tr>
<td>$f_x$</td>
<td>On-axis normal strength</td>
</tr>
<tr>
<td>$f_{xc}$</td>
<td>On-axis normal strength in compression</td>
</tr>
<tr>
<td>$f_{xt}$</td>
<td>On-axis normal strength in tension</td>
</tr>
<tr>
<td>$f_{xy}$</td>
<td>On-axis shear strength</td>
</tr>
<tr>
<td>$f_y$</td>
<td>On-axis normal strength</td>
</tr>
<tr>
<td>$f_{yc}$</td>
<td>On-axis normal strength in compression</td>
</tr>
<tr>
<td>$f_{yt}$</td>
<td>On-axis normal strength in tension</td>
</tr>
<tr>
<td>$F_{12}$</td>
<td>Interaction coefficient</td>
</tr>
<tr>
<td>$F_c$</td>
<td>Resultant force in the compressive zone</td>
</tr>
<tr>
<td>$F_{max}$</td>
<td>Failure load</td>
</tr>
<tr>
<td>$F_N$</td>
<td>The imposed normal force</td>
</tr>
<tr>
<td>$F_t$</td>
<td>Resultant force in the tensile zone</td>
</tr>
<tr>
<td>$F_{t(c)}$</td>
<td>Resultant force in the tensile or compressive zone</td>
</tr>
<tr>
<td>$F_{t,bp}$</td>
<td>Tensile capacity of birch plywood</td>
</tr>
<tr>
<td>$F_{t,\theta,bp,adv}$</td>
<td>Tensile capacity predicted by the advanced model</td>
</tr>
<tr>
<td>$F_{t,\theta,bp,sim}$</td>
<td>Tensile capacity predicted by the simple model</td>
</tr>
<tr>
<td>$\Delta F$</td>
<td>Load increment</td>
</tr>
<tr>
<td>$g$</td>
<td>Gap between the bonded connections</td>
</tr>
<tr>
<td>$g_1$</td>
<td>Longer gap between the bonded connections</td>
</tr>
<tr>
<td>$G_{\text{destructive}}$</td>
<td>Shear modulus from destructive tests</td>
</tr>
<tr>
<td>$G_{ij}$</td>
<td>On-axis shear modulus in i-j plane</td>
</tr>
<tr>
<td>$G_{\text{modified}}$</td>
<td>Modified shear modulus</td>
</tr>
<tr>
<td>$G_{xy}$</td>
<td>On-axis shear modulus in x-y plane</td>
</tr>
<tr>
<td>$G_{xz}$</td>
<td>On-axis shear modulus in x-z plane</td>
</tr>
<tr>
<td>$G_{yz}$</td>
<td>On-axis shear modulus in y-z plane</td>
</tr>
<tr>
<td>$G_\theta$</td>
<td>Shear modulus at $\theta$ to the face grain</td>
</tr>
<tr>
<td>$h$</td>
<td>Height of glulam</td>
</tr>
<tr>
<td>$I$</td>
<td>Second moment of area</td>
</tr>
<tr>
<td>$k$</td>
<td>Shear correction coefficient</td>
</tr>
<tr>
<td>$l$</td>
<td>Length of the bonded area</td>
</tr>
<tr>
<td>$l_1$</td>
<td>Original length of strain gauge</td>
</tr>
<tr>
<td>$l_r$</td>
<td>Reduced length</td>
</tr>
<tr>
<td>$L$</td>
<td>Span of birch plywood</td>
</tr>
<tr>
<td>$m$</td>
<td>Cosine function of an angle $\theta$</td>
</tr>
<tr>
<td>$M$</td>
<td>The imposed bending moment</td>
</tr>
<tr>
<td>$M_{u,\text{anl}}$</td>
<td>Analytical ultimate moment capacity</td>
</tr>
<tr>
<td>$M_{u,\text{exp}}$</td>
<td>Experimental ultimate moment capacity</td>
</tr>
<tr>
<td>$M_{u,\text{num}}$</td>
<td>Numerical ultimate moment capacity</td>
</tr>
<tr>
<td>$MOE_b$</td>
<td>Elastic bending modulus</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>( n )</td>
<td>Sine function of an angle ( \theta )</td>
</tr>
<tr>
<td>( R_M )</td>
<td>The moment capacity</td>
</tr>
<tr>
<td>( R_N )</td>
<td>The normal capacity</td>
</tr>
<tr>
<td>( R_V )</td>
<td>The shear capacity</td>
</tr>
<tr>
<td>( t )</td>
<td>Thickness of birch plywood</td>
</tr>
<tr>
<td>( \Delta u )</td>
<td>Deformation increment</td>
</tr>
<tr>
<td>( \Delta u_b )</td>
<td>Bending deformation increment</td>
</tr>
<tr>
<td>( \Delta u_{LVDT} )</td>
<td>LVDT deformation increment</td>
</tr>
<tr>
<td>( \Delta u_s )</td>
<td>Shear deformation increment</td>
</tr>
<tr>
<td>( V )</td>
<td>The imposed shear force</td>
</tr>
<tr>
<td>( w_{eff} )</td>
<td>Effective width in the simple model</td>
</tr>
<tr>
<td>( w_{eff,adv} )</td>
<td>Effective width in the advanced model</td>
</tr>
<tr>
<td>( W )</td>
<td>Elastic section modulus</td>
</tr>
<tr>
<td>( y_t )</td>
<td>Depth of the tensile zone</td>
</tr>
<tr>
<td>( y_c )</td>
<td>Depth of the compressive zone</td>
</tr>
<tr>
<td>( z )</td>
<td>Lever arm</td>
</tr>
</tbody>
</table>

**Greek Symbols**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \alpha )</td>
<td>Spreading angle in the simple model</td>
<td>[°]</td>
</tr>
<tr>
<td>( \alpha_{adv} )</td>
<td>Spreading angle in the advanced model</td>
<td>[°]</td>
</tr>
<tr>
<td>( \gamma )</td>
<td>Shear strain</td>
<td>[1]</td>
</tr>
<tr>
<td>( \gamma_{correction} )</td>
<td>Correction factor</td>
<td>[1]</td>
</tr>
<tr>
<td>( \varepsilon_c )</td>
<td>Compressive strain</td>
<td>[1]</td>
</tr>
<tr>
<td>( \varepsilon_{t(c)} )</td>
<td>Tensile or compressive strain</td>
<td>[1]</td>
</tr>
<tr>
<td>( \varepsilon_{tu} )</td>
<td>Tensile strain at failure</td>
<td>[1]</td>
</tr>
<tr>
<td>( \theta )</td>
<td>Face grain angle to the loading or beam axis</td>
<td>[°]</td>
</tr>
<tr>
<td>( \nu_{ij} )</td>
<td>Poisson’s ratio in i-j plane</td>
<td>[1]</td>
</tr>
<tr>
<td>( \nu_{xy} )</td>
<td>Poisson’s ratio in x-y plane</td>
<td>[1]</td>
</tr>
<tr>
<td>( \nu_{xz} )</td>
<td>Poisson’s ratio in x-z plane</td>
<td>[1]</td>
</tr>
<tr>
<td>( \nu_{yz} )</td>
<td>Poisson’s ratio in y-z plane</td>
<td>[1]</td>
</tr>
<tr>
<td>( \rho )</td>
<td>Density</td>
<td>[kg m⁻³]</td>
</tr>
<tr>
<td>( \sigma_1 )</td>
<td>Off-axis normal stress in the 1-2 system</td>
<td>[N m⁻²]</td>
</tr>
<tr>
<td>( \sigma_2 )</td>
<td>Off-axis normal stress in the 1-2 system</td>
<td>[N m⁻²]</td>
</tr>
<tr>
<td>( \sigma_c )</td>
<td>The compressive stress</td>
<td>[N m⁻²]</td>
</tr>
<tr>
<td>( \sigma_{t(c)} )</td>
<td>Tensile or compressive stress</td>
<td>[N m⁻²]</td>
</tr>
<tr>
<td>( \sigma_x )</td>
<td>On-axis normal stress in the x-y system</td>
<td>[N m⁻²]</td>
</tr>
<tr>
<td>( \sigma_y )</td>
<td>On-axis normal stress in the x-y system</td>
<td>[N m⁻²]</td>
</tr>
<tr>
<td>( \tau )</td>
<td>Shear stress</td>
<td>[N m⁻²]</td>
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<tr>
<td>( \tau_{12} )</td>
<td>Off-axis shear stress in the 1-2 system</td>
<td>[N m⁻²]</td>
</tr>
<tr>
<td>( \tau_{xy} )</td>
<td>On-axis shear stress in the x-y system</td>
<td>[N m⁻²]</td>
</tr>
</tbody>
</table>
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1. Introduction

1.1. Plywood

Plywood is one of the earliest produced so-called engineered wood products (EWPs). It is composed of an uneven number of veneers (also called plies), normally with a thickness of less than one millimeter to several millimeters. The veneers are mostly produced by a rotary lathe where a log is peeled in green state after being heated and softened by hot-water immersion. The veneers are thereafter dried, sorted, and bonded with an adhesive in a hot press with the grain direction of adjacent veneers perpendicular to one another [1]. The cross-lamination configuration of plywood is schematically shown in Figure 1.

![Figure 1: The cross-lamination of plywood [2].](image)

The important features of plywood compared to sawn timber are improved dimensional stability, the possibility to overcome the dimensional limitation caused by the normal size of the trees, and the redistribution of natural defects [3, 4].

In terms of mechanical properties, it is well-known that timber is an anisotropic material, with high strength and stiffness in the direction
Parallel to the grain but much lower values in the radial and tangential directions. Plywood, on the other hand, due to the cross-lamination configuration, shows values of strength and stiffness much more balanced in different directions of the plane.

Regarding the number of veneers, the commonly produced plywood has at least 3 veneers and up to 35 veneers [5]. It is straightforward that the increased number of veneers leads to a lower degree of anisotropy provided that the veneer thickness is constant [6]. The configuration of plywood also enhances its resistance to splitting because there is no line of cleavage; thus fasteners can be inserted at closer spacing or nearer to the edges than in unidirectional timber products [7].

1.2. Structural applications of plywood

Due to the aforementioned advantages, plywood exhibits versatile applications for structural purposes. Strength, stiffness and the durability of the wood-adhesive bond are important for structural plywood while the appearance of the face veneer may or may not be of significance. Typical structural applications of plywood are listed below [1, 8, 9]:

- building construction systems, e.g., sub-flooring, decking, sheathing, bracing, roof, shear wall, concrete formwork, etc.;
- connection systems, e.g., gusset plates in truss system, frame corner, beam-to-column connection, etc.;
- beam systems, e.g., webs in I-beam or box beam systems;
- some structural parts of aircrafts.

Among these applications, connections are of the most interest in this thesis. Plywood can be applied as the gusset plates in truss systems, where the axial forces in different directions are transferred. Plywood plates could also be utilized in moment-resisting connections, e.g., beam-to-column and portal frame haunches, etc., where a bending moment is induced as well. Figure 2 shows the veneer-based products in timber connections [10, 11].
However, in high-rise timber buildings as well as long-span timber applications, slotted-in steel plates are today the dominant connection technique [12]. For instance, such connections were used in the 18-story timber building “Mjøstårnet”, located in Brumunddal, Norway, which is one of the world’s tallest timber buildings with a height of around 85 m.
The glulam elements in this building are connected by using 8 mm or 12 mm thick slotted-in steel plates and dowels with a diameter of 12 mm [13]. Comprehensive analyses of timber connections with slotted-in steel plates have been carried out during the past few decades [14-16]. The design model of the slotted-in steel plates is better developed than that of the plywood gusset plates. There are some uncertainties concerning the load-bearing capacity of the plywood plate mainly since plywood is weaker than steel, and is an anisotropic material. Knowledge regarding the mechanical properties of the plywood plate and its structural behavior in timber connections should be improved. The plywood gusset plate should be properly designed in such a manner that it is not the weakest link in the connection system, thereby holding the potential of reaching a similar capacity to the ones with slotted-in steel plates and dowels.

Plywood plates, compared with the slotted-in steel plates, are more environmentally friendly, cost-effective, and less prefabrication demanding, with better workability and fire resistance. The “better workability” refers to that, the technique using slotted-in steel plates requires high precision during the prefabrication process to avoid the misalignment between the predrilled holes of the steel plates and glulam elements for smooth insertion of the fasteners during the assembly process. This is less of an issue for gusset plates made of plywood.

More attention has been paid recently to plywood made of birch (Betula spp.) [17, 18]. Birch has a wide natural distribution area in Europe and Asia, especially in the Nordic and Baltic countries [19]. Some physical and mechanical properties of common species in Scandinavia are listed in Table 1 [20, 21]. It is obvious that birch in general possesses superior mechanical properties compared with the listed softwood species.
Table 1: Some physical and mechanical properties of common species in Scandinavia [20, 21].

<table>
<thead>
<tr>
<th>Species</th>
<th>Density (dry) (kg/m³)</th>
<th>Tensile strength in fiber direction (MPa)</th>
<th>Compressive strength in fiber direction (MPa)</th>
<th>Elastic modulus in fiber direction (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft wood</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pine (Pinus silvestris)</td>
<td>450-500</td>
<td>102</td>
<td>45-47</td>
<td>10000-12000</td>
</tr>
<tr>
<td>Spruce (Picea abies)</td>
<td>370-440</td>
<td>88</td>
<td>35-44</td>
<td>8300-13000</td>
</tr>
<tr>
<td>Larch (Larix decidua)</td>
<td>520-600</td>
<td>105</td>
<td>47-54</td>
<td>9900-13500</td>
</tr>
<tr>
<td>Hard wood</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Birch (Betula pendula)</td>
<td>580-620</td>
<td>137</td>
<td>54-60</td>
<td>13000-15000</td>
</tr>
<tr>
<td>Oak (Quercus robur)</td>
<td>650-720</td>
<td>90</td>
<td>53-65</td>
<td>10000-13000</td>
</tr>
<tr>
<td>Beech (Fagus sylvatica)</td>
<td>640-680</td>
<td>135</td>
<td>52-56</td>
<td>10000-16000</td>
</tr>
</tbody>
</table>

Note: Strength and elastic properties are mean values determined for clear wood samples at 12% moisture content (MC).

1.3. Mechanical and adhesively bonded connections

The top photo in Figure 2 shows a mockup of a truss node. On the left side of this truss node, the timber elements were connected using birch plywood gusset plates and mechanical connectors, i.e., steel dowels, while on the right side, the corresponding connection was adhesively bonded (glued). This model connection illustrates that there are two possibilities to connect
the glulam beams and the plywood plates, namely, mechanical connection and adhesively bonded connection.

It is likely that adhesively bonded connections using plywood plates could be cost-effective with nearly no consumption of steel materials. In terms of the structural behavior, the adhesively bonded connection is typically stiffer than the mechanical one [22, 23]. For both types of connections, if properly designed, the ultimate failure should take place in the timber parts. In this context, the adhesively bonded one shows higher load-bearing capacity since the timber material is not weakened by the drilled holes. The holes in the mechanical connection also cause stress concentration, increased risk of moisture penetration, and increased creep deformation with time [24]. Despite the potential of adhesively bonded connections, the bonding technique and bonding performance are currently lacking research, particularly in terms of the hybrid bonding between different EWPs and different species. One relevant investigation on the adhesively bonded joints in a timber truss with the spruce glulam as a diagonal member and the beech laminated veneer lumber as lower chords has been reported by Stimpfle et al. [25].

On the other hand, the mechanically connected joint enables that separate structural components can be transported and assembled on-site considering that the prefabricated components in large formats are often limited by both manufacture and transportation capacity [26]. The strength of adhesively bonded connections is strongly dependent on the quality of the bond line, which means that strict production quality controls are typically needed. The bonding performance is, in fact, sensitive to the process-related parameters and the surrounding environment (temperature and relative humidity) during assembly [27-30]. Hence, the gluing on-site technique, in particular between timber elements and birch plywood plates, is not recommended at the moment, unless special precautions are used. Currently, one potential proposal is to adhesively connect as many timber components as possible in the factory and mechanically connect these prefabricated modules on-site, providing a highly prefabricated solution with shortened assembly time on-site.
1.4. Aim and objectives

The aim of this thesis is to gain new knowledge necessary for the development of adhesively bonded connections using birch plywood gusset plates in timber structures. The application of such adhesively bonded connections includes but is not limited to timber trusses and portal frames, providing a potential alternative to the slotted-in steel plates typically adopted in timber structures. In this context it is necessary, first, to gain new knowledge regarding the in-plane mechanical properties of birch plywood, and second, to investigate its performance in adhesively bonded connections.

The specific objectives are:

- Establish a comprehensive experimental database of the in-plane tensile, compressive, shear, and bending strength and stiffness at a few loading angles to the face grain between 0° and 90° and predict these properties at any angle to the face grain by means of analytical and numerical models.
- Study the influence of size and moisture content on the edgewise bending strength and stiffness of birch plywood.
- Evaluate the adhesive bonding performance between birch plywood and spruce glulam by conducting tensile shear tests. The investigated factors are the adhesive types, pressing methods, moisture, and load-to-plywood face grain angles.
- Investigate the moment-resisting bonding performance.
- Determine how the plywood width and load-to-face grain angle influence the tensile capacity of plywood gusset plates in adhesively bonded connections.

1.5. Limitations

This study focuses on the potential structural use of birch plywood in indoor applications and does not include any durability test on birch plywood.
Moreover, the types of adhesive systems are discussed mainly based on their bonding performance. Some other key aspects, e.g., the approval of the adhesive systems in timber connections, gap-filling property, fire resistance, and formaldehyde emissions, etc., should also be taken into consideration.

1.6. On-going studies

The thesis will also present some results obtained from the on-going studies which are not reported in the appended papers.

The on-going studies comprise:

- Testing of an innovative “wheel/geared” connection system.
- Proposing a design formula to predict the load-bearing capacity of birch plywood plates in adhesively bonded frame corners.
- Comparing the structural behaviors of full-scale portal frames mechanically or adhesively connected by birch plywood gusset plates.

1.7. Thesis structure

This thesis begins with an introductory chapter, in which the background, motivation, aim, objectives, limitations, and on-going studies are introduced. Chapter 2 contains a description of the investigated materials and methods. Chapter 3 presents and discusses the experimental and predicted results in the appended papers, which involve the in-plane mechanical properties of birch plywood (Papers I and II) [31], the adhesive bonding performance between birch plywood and spruce glulam (Papers III-V), and the tensile capacity of birch plywood in adhesively bonded connections (Paper VI). Chapter 4 comprises some results from the on-going studies that are not reported in the appended papers. Finally, the conclusions drawn in this thesis and the suggested future work are presented in Chapter 5. A flowchart of the work performed in the appended papers and the on-going studies is shown in Figure 3.
Figure 3: A flowchart of the work performed in the appended papers and the on-going studies.
2. Materials and methods

2.1. Materials

2.1.1. Birch plywood

The birch plywood panels tested in this thesis have different thicknesses, i.e., 6.5 mm (5 veneers), 12 mm (9 veneers), 15 mm (11 veneers), 21 mm (15 veneers), 30 mm (21 veneers). All the birch plywood materials were produced and supplied by Koskisen Oy (Järvelä, Finland). Phenol formaldehyde resin was used as adhesive between each veneer. Given that the birch plywood panels are commercial products, details of the manufacturing processes, including the pressing schedule and the characteristics of the phenol formaldehyde resin, etc., are not described in the thesis. Figure 4 shows four different birch plywood plates with different thicknesses.

![Figure 4: Birch plywood plates with different thicknesses.](image)

The inner veneers of the tested plywood have an identical thickness of 1.4-1.5 mm while the face veneers are thinner (approximately half of the thickness of the inner veneers) since the surfaces of the plywood are
typically sanded in the production line in order to control the total thickness of the plate.

### 2.1.2. Glulam

Spruce glulam beams GL28cs [32] used for the experiments in Papers III-VI were produced by Moelven (Töreboda, Sweden). The beams had cross-sections of 42 mm × 180 mm and 56 mm × 225 mm.

### 2.1.3. Adhesives

Three types of thermosetting adhesives, i.e., melamine-urea-formaldehyde (MUF), phenol-resorcinol-formaldehyde (PRF), and two-component polyurethane (2C PUR), were selected and utilized in Papers III-VI. Information of the adhesives with associated hardeners is listed below:

- MUF (Prefere 4546/5022) (Dynea, Lillestrøm, Norway)
- PRF (Aerodux 185/HRP 155) (Dynea, Lillestrøm, Norway)
- 2C PUR (Aro-Bond 925) (Ureka, Bristol, UK)

### 2.2. In-plane tensile, compressive, and shear properties (Paper I)

#### 2.2.1. Experiments

For each type of test (tension, compression, and shear), birch plywood specimens with a nominal thickness of 21 mm were tested at five load-to-face grain angles, from 0° (parallel) to 90° (perpendicular), with an interval of 22.5°. Each test series had 12 replicates, resulting in 180 specimens in total. Prior to the tests, the specimens were conditioned in a climate chamber at a temperature (T) of 20 °C and a relative humidity (RH) of 65% until the mass did not change more than 0.1% at an interval of six hours. The density was then measured on compressive specimens as they have a regular shape, thus the volume can be easily determined. The moisture content (MC) was evaluated on 12 additional replicates with a similar mass compared to the tested specimens based on the oven-dry method [33]. The mean density and MC were approx. 693 kg/m³ and 12%, respectively.
The test setup and the configuration of birch plywood specimens were adapted from the testing standard ASTM D3500 [34] for tensile tests, ASTM D3501 [35] for compressive tests, and EN 789, ASTM D1037, ASTM D2719 [36-38] for the shear tests. Two 38 mm-long strain gauges were installed for the measurement of the elastic properties. Details regarding the test procedure have been introduced in Paper I. See Figure 5 for the test setup and the information about the specimens.

![Test Setup](image_url)

**Figure 5:** Test setup: (a) tension, (b) compression, and (c) shear. (d) Illustration of the loading angle to the face grain (unit of dimensions: mm).

It is noticed in Figure 5b that there is one spherical seat platen connected to the loading head. This is to ensure full contact with the top surface of the specimen during compressive loading.

Tensile and compressive strengths are defined as the measured failure load divided by the cross-sectional area. The tensile and compressive
elastic moduli are determined as the slope of the linear portion of the stress-strain curve, as expressed in Eq. 1.

\[ E_{t(c)} = \frac{\sigma_{t(c)}}{\varepsilon_{t(c)}} = \frac{\Delta F/A}{\Delta u/l_1} \]  

(1)

where \( E_{t(c)} \) is the tensile or compressive elastic modulus; \( \sigma_{t(c)} \) is the tensile or compressive stress; \( \varepsilon_{t(c)} \) is the tensile or compressive strain; \( \Delta F \) is the increment of the load between 15% to 35% of the failure load; \( \Delta u \) is the increment of deformation measured from the strain gauges corresponding to \( \Delta F \); \( l_1 \) is the original length of the strain gauge; and \( A \) is the cross-sectional area. The chosen range, i.e., 15-35%, fulfills the requirement in ASTM D3501 [35] that at least 20% of the ultimate load is covered.

As shown in Figure 5c, the configuration of the shear specimen in the destructive test was designed with a reduced length in the middle to lower the capacity. The shear strength can be well characterized by the destructive tests and is defined as the failure load divided by the cross-sectional area between the vertical slots. However, the shear modulus determined from the destructive tests should be modified by a correction factor based on the non-destructive test results. This is due to that, in the destructive tests, shear deformation was concentrated along the middle line but the strain gauges measured the average deformation between the attached points (38 mm long), resulting in an overestimation of the shear modulus. In contrast, the shear strain in the non-destructive test was nearly uniform within the area where the strain gauges were attached. Therefore, non-destructive tests were performed on specimens without slits at 0°, 45°, and 90°, with 5 replicates for each angle.

The shear modulus obtained from the destructive tests is expressed in Eq. 2.

\[ G_{destructive} = \frac{\tau}{\gamma} = \frac{\Delta F/(l_r \cdot t)}{2\Delta u \cdot /l_1} \]  

(2)

where \( G_{destructive} \) is the shear modulus derived from the destructive tests; \( \tau \) is the shear stress; \( \gamma \) is the shear strain; \( \Delta F \) is the increment of the load between 15% and 35% of the failure load; \( l_r \) is the reduced length in the
middle; \( t \) is the thickness of the specimen; \( \Delta u \) is the increment of deformation measured from the strain gauges corresponding to \( \Delta F \); and \( l_1 \) is the length of the strain gauge.

The modified shear modulus is shown in Eq. 3.

\[
G_{\text{modified}} = \gamma_{\text{correction}} \cdot G_{\text{destructive}},
\]

where \( G_{\text{modified}} \) is the modified shear modulus derived from the non-destructive tests; and \( \gamma_{\text{correction}} \) is the correction factor, which is the ratio of the shear modulus measured from the specimens without slits (non-destructive tests) and with slits (destructive tests). This ratio is determined to be 0.33, 0.35, and 0.32, at 0°, 45°, and 90° to the face grain, respectively, leading to a mean correction factor (\( \gamma_{\text{correction}} \)) of 0.33. It is noted that the shear modulus results presented in the chapters below are the modified ones (\( G_{\text{modified}} \)).

2.2.2. Predictions of off-axis strength

In order to predict the off-axis strength in tension, compression, and shear, the first step is to transform the uniaxial stress in the 1-2 (parallel–perpendicular to the loading axis) coordinate system to the x-y (parallel–perpendicular to the face grain of the birch plywood) coordinate system.

The transformation equation between the off-axis stresses and the on-axis stresses is shown in Eq. 4.

\[
\begin{bmatrix}
\sigma_x \\
\sigma_y \\
\tau_{xy}
\end{bmatrix} = \begin{bmatrix}
m^2 & n^2 & 2mn \\
2mn & m^2 & -2mn \\
-mn & mn & m^2 - n^2
\end{bmatrix} \begin{bmatrix}
\sigma_1 \\
\sigma_2 \\
\tau_{12}
\end{bmatrix},
\]

where \( m = \cos(\theta) \); \( n = \sin(\theta) \); \( \theta \) is the angle between the load direction and the face grain; \( \sigma_1, \sigma_2, \) and \( \tau_{12} \) are the off-axis stresses in the 1-2 system; and \( \sigma_x, \sigma_y, \) and \( \tau_{xy} \) are the on-axis stresses in the x-y system. \( \sigma_1, \sigma_2, \sigma_x, \) and \( \sigma_y \) are defined as positive in tension and negative in compression while \( \tau_{12} \) and \( \tau_{xy} \) are positive with the direction shown in Figure 6b and 6c. There is only \( \sigma_1 (\sigma_2 = \tau_{12} = 0) \) in tensile and compressive tests and only \( \tau_{12} (\sigma_1 = \sigma_2 = 0) \) in panel shear tests.
After transforming the stresses from \(1-2\) to \(x-y\) coordinate system, the second step is to employ a number of failure criteria to predict the off-axis strength based on the on-axis strength values. Several failure criteria (listed in Table 2) that are widely used for composite materials were examined in this thesis for their applicability to birch plywood.
Table 2: Failure criteria for off-axis strength predictions.

<table>
<thead>
<tr>
<th>Failure criteria</th>
<th>Formula</th>
<th>Eq.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hankinson [39]</td>
<td>$\frac{\sigma_x}{f_x} + \frac{\sigma_y}{f_y} = 1$</td>
<td>(5)</td>
</tr>
<tr>
<td>Linear criterion with shear effect</td>
<td>$</td>
<td>\frac{\sigma_x}{f_x}</td>
</tr>
<tr>
<td>Empirical Norris [40]</td>
<td>$\left(\frac{\sigma_x}{f_x}\right)^2 + \left(\frac{\sigma_y}{f_y}\right)^2 + \left(\frac{\tau_{xy}}{f_{xy}}\right)^2 = 1$</td>
<td>(7)</td>
</tr>
<tr>
<td>Theoretical Norris [40]</td>
<td>$\left(\frac{\sigma_x}{f_x}\right)^2 - \frac{\sigma_x \sigma_y}{f_x f_y} + \left(\frac{\sigma_y}{f_y}\right)^2 + \left(\frac{\tau_{xy}}{f_{xy}}\right)^2 = 1$ or $\left(\frac{\sigma_x}{f_x}\right)^2 = 1$ or $\left(\frac{\sigma_y}{f_y}\right)^2 = 1$</td>
<td>(8)</td>
</tr>
<tr>
<td>Tsai-Hill [41, 42]</td>
<td>$\frac{\sigma_x^2}{f_X^2} - \frac{\sigma_x \sigma_y}{f_X f_Y} + \frac{\sigma_y^2}{f_Y^2} + \frac{\tau_{xy}^2}{f_{xy}^2} = 1$</td>
<td>(9)</td>
</tr>
<tr>
<td>Hoffman [43]</td>
<td>$\frac{\sigma_x^2}{f_X f_Y} - \frac{\sigma_x \sigma_y}{f_Y f_X} + \frac{\sigma_y^2}{f_Y f_X} + \frac{f_X - f_Y}{f_Y f_X} \sigma_x + \frac{\tau_{xy}^2}{f_{xy}^2} = 1$</td>
<td>(10)</td>
</tr>
<tr>
<td>Tsai-Wu [44]</td>
<td>$\frac{\sigma_x^2}{f_X f_Y} + \frac{\sigma_y^2}{f_Y f_X} + 2F_{12} \sigma_x \sigma_y + \frac{f_X - f_Y}{f_Y f_X} \sigma_x + \frac{f_Y - f_X}{f_Y f_X} \sigma_y + \left(\frac{\tau_{xy}}{f_{xy}}\right)^2 = 1$</td>
<td>(11)</td>
</tr>
</tbody>
</table>

$f_x$, $f_y$, and $f_{xy}$ are the on-axis normal and shear strengths of birch plywood. The mean values obtained from the experiments are used as input for these strength properties. It is worth noting that the first two failure criteria, namely, Hankinson and linear criteria with shear effect are only capable for the prediction of the off-axis tensile and compressive strengths but not the shear strengths because the Hankinson’s criterion neglects the shear contribution and, for the linear criterion with shear effect, the shear term is only in the first order. Hoffman and Tsai-Wu failure criteria distinguish the on-axis normal strengths in tension and compression, which are
specified by \( f_{xt}, f_{xc}, f_{yt}, \) and \( f_{yc} \). In addition, the interaction coefficient \( F_{12} \) in Tsai-Wu failure criterion is unknown due to the complexity of experimentally determining this value. However, \( F_{12} \) is constrained by certain stability conditions. See the inequality in Eq. 12.

\[
-\sqrt{\frac{1}{f_{xc}f_{xt}f_{yc}f_{yt}}} \leq F_{12} \leq \sqrt{\frac{1}{f_{xc}f_{xt}f_{yc}f_{yt}}},
\] (12)

\( F_{12} \) was first assumed as zero to compare with the experimental data and other failure criteria. Hereafter, the influence of this parameter on the predicted strength was studied by varying \( F_{12} \) from its lower limit to the upper limit.

### 2.2.3. Predictions of off-axis elastic properties

Both elastic modulus and shear modulus were predicted. Three models were applied for elastic modulus predictions (see Table 3) and two models were applied for shear modulus predictions (see Table 4).

Table 3: Off-axis elastic modulus prediction models.

<table>
<thead>
<tr>
<th>Elastic modulus prediction models</th>
<th>Formula</th>
<th>Eq.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hankinson</td>
<td>( E_\theta = \frac{E_x \cdot E_y}{E_y \cdot \cos^2(\theta) + E_x \cdot \sin^2(\theta)} )</td>
<td>(13)</td>
</tr>
<tr>
<td>Transformation model (elastic modulus) [45, 46]</td>
<td>( \frac{1}{E_\theta} = \frac{1}{E_x} \cos^4(\theta) + \left( -2 \frac{v_{xy}}{E_x} + \frac{1}{G_{xy}} \right) \cos^2(\theta)\sin^2(\theta) + \frac{1}{E_y} \sin^4(\theta) )</td>
<td>(14)</td>
</tr>
<tr>
<td>Saliklis and Falk [47]</td>
<td>( \frac{1}{E_\theta} = \frac{1}{E_x} \cos^4(\theta) + \left( \frac{1}{A_{1}^{2A_{1}G_{xy}}} \right) \cos^2(\theta)\sin^2(\theta) + \frac{1}{E_y} \sin^4(\theta) )</td>
<td>(15)</td>
</tr>
</tbody>
</table>
Table 4: Off-axis shear modulus prediction models.

<table>
<thead>
<tr>
<th>Shear modulus prediction models</th>
<th>Formula</th>
<th>Eq.</th>
</tr>
</thead>
</table>
| Transformation model (shear modulus) | \[
\frac{1}{G_\theta} = 4\left(\frac{1}{E_x} + \frac{1}{E_y} - 2\frac{\nu_{xy}}{E_x}\right)\cos^2(\theta)\sin^2(\theta) \\
+ \frac{1}{G_{xy}}\left(\cos^2(\theta) - \sin^2(\theta)\right)^2
\] | (16) |
| Modified transformation model (shear modulus) | \[
G_\theta = \frac{G_{xy} \cdot G_{45}}{G_{xy} \cdot \sin^2(2\theta) + G_{45} \cdot \cos^2(2\theta)}
\] | (17) |

$E_\theta$ and $G_\theta$ are the off-axis elastic and shear moduli at an angle $\theta$ to the face grain; $E_x$ and $E_y$ are the elastic moduli parallel and perpendicular to the face grain determined from the tensile tests; $G_{xy}$ is the on-axis shear modulus determined from the shear tests; $\nu_{xy}$ is the Poisson’s ratio in x-y plane derived based on the mechanical properties of solid birch and the cross-sectional configuration of the plywood [6, 48]; and the parameter $A_1$ in Eq. 15 is the ratio between $E_y$ and $E_x$.

### 2.3. Edgewise bending properties (Paper II)

#### 2.3.1. Experiments

The influence of three factors, i.e., face grain angle, size, and moisture content, on the edgewise bending properties was investigated by testing 288 birch plywood specimens (24 test series with 12 replicates) in three-point bending. The test setup is shown in Figure 7.
The studied variables are listed in Figure 7. The birch plywood specimens tested in edgewise bending have the same nominal width \( (b) \) of 21 mm as the ones tested in tension, compression and shear. To study the size effect, the span \( (L) \)–to-depth \( (d) \) ratio was kept constant in all the tests.

Three properties were determined from the tests, i.e., ultimate moment capacity \( (M_{u,exp}) \), bending strength \( (f_b) \), and elastic bending modulus \( (MOE_b) \). The experimental ultimate moment capacity is defined as the maximum bending moment at mid-span (see Eq. 18).

\[
M_{u,exp} = \frac{F_{\text{max}} \cdot L}{4}, \quad (18)
\]

where \( F_{\text{max}} \) is the failure load. The edgewise bending strength is defined as \( M_{u,exp} \) over the elastic section modulus \( W \). The elastic bending modulus is calculated from the linear portion of the load-displacement curves (Eq. 19).

\[
MOE_b = \frac{\Delta F \cdot L^3}{48 \cdot I \cdot \Delta u_b} = \frac{\Delta F \cdot L^3}{4 \cdot b \cdot d^3 \cdot (\Delta u_{\text{LVDT}} - \Delta u_s)}, \quad (19)
\]

where \( \Delta F \) is the force increment between 15% to 35% of the failure load; \( I \) is the second moment of area; \( \Delta u_b \) is the increment of the bending deformation at mid-span corresponding to \( \Delta F \); \( \Delta u_{\text{LVDT}} \) is the average displacement increment measured from two linear variable differential transformers (LVDTs) corresponding to \( \Delta F \); and \( \Delta u_s \) is the increment of shear deformation at mid-span corresponding to \( \Delta F \) (see Eq. 20).
where \( k \) is the shear correction coefficient. \( k \) is assumed to be 5/6, for orthotropic laminate materials [49]. \( G_\theta \) is the mean experimental shear modulus at an angle \( \theta \) to the face grain determined in Paper I. The local bearing deformation at the supports is negligible for birch plywood due to the cross-lamination configuration.

### 2.3.2. Prediction models

It is also of interest to develop the prediction models to predict the angle-dependency of the edgewise bending properties. When the beam specimens are loaded in edgewise positive bending, the upper part is subjected to compressive stresses while the lower part is in tension. Thus, the stress-strain relationships in uniaxial tension and compression are required as input in the prediction model, which were obtained from the tensile and compressive tests in this study.

#### Analytical model

In the analytical prediction model, the strain and stress distributions along the depth of the birch plywood beam at mid-span could be derived based on the force equilibrium in the compressive and tensile zone. See Figure 8 for the schematic representation of strain and stress distributions at the ultimate failure state. Failure is first defined when the maximum tensile stress at the bottom of the beam reaches the tensile strength \( f_t \).

\[
\Delta u_s = \frac{\Delta F \cdot L}{4 \cdot k \cdot G_\theta \cdot A'} 
\]  

(20)
Figure 8: Strain and stress distributions along the depth at mid-span when the maximum tensile stress reaches the tensile strength.

The relationships between the tensile and compressive zones are illustrated in Eq. 21 and 22.

\[
\frac{y_t}{y_c} = \frac{\varepsilon_{tu}}{\varepsilon_c}, \quad (21)
\]

\[
y_t + y_c = d, \quad (22)
\]

where \(y_t\) and \(y_c\) are the depth of the tensile and compressive zones respectively; \(\varepsilon_{tu}\) is the tensile strain at failure; and \(\varepsilon_c\) is the compressive strain at the top of the beam when the failure occurs. The unknowns, i.e., \(y_t, y_c, \varepsilon_c, \sigma_c, F_t, F_c\) and \(z\), can be calculated based on the force equilibrium in the compressive and tensile zone (see Eq. 23).

\[
\sum F_i = F_t - F_c = 0, \quad (23)
\]

where \(F_t\) and \(F_c\) are the resultant forces in the tensile and compressive zones; \(\sigma_c\) is the compressive stress at the top of the beam when the failure occurs; \(z\) is the lever arm between the resultant forces. The analytical ultimate moment capacity \(M_{u,\text{u}}\) is therefore calculated in Eq. 24.
where $F_t(c)$ is the resultant force in the tensile or compressive zone.

**Numerical model**

3D solid models were created via the commercial FEM package Abaqus/Standard (Simulia, USA). In the numerical model, apart from the compressive elasto-plastic stress-strain relationships, some engineering constants should also be provided, as listed in Table 5. They consist of the density $\rho$, elastic modulus $E_i$, the Poisson’s ratio $v_{ij}$, and the shear modulus $G_{ij}$ ($i, j = x, y, z$). Axes x and y are parallel and perpendicular to the face grain while axis z is through the plywood thickness.

*Table 5: Engineering constants of birch plywood for numerical analysis*

<table>
<thead>
<tr>
<th>$\rho$ (kg/m$^3$)</th>
<th>$E_x$ (MPa)</th>
<th>$E_y$ (MPa)</th>
<th>$E_z$ (MPa)</th>
<th>$v_{xy}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>703</td>
<td>9400</td>
<td>6700</td>
<td>1110</td>
<td>0.036</td>
</tr>
<tr>
<td>$v_{xz}$</td>
<td>$v_{yz}$</td>
<td>$G_{xy}$ (MPa)</td>
<td>$G_{xz}$ (MPa)</td>
<td>$G_{yz}$ (MPa)</td>
</tr>
<tr>
<td>0.443</td>
<td>0.427</td>
<td>600</td>
<td>206</td>
<td>186</td>
</tr>
</tbody>
</table>

Note: $\rho$ is the mean density of the specimens. $E_x$, $E_y$, and $G_{xy}$ are the mean values presented in Figure 22 (tensile tests) and Figure 23 in Section 3.1.1. The rest of the engineering constants were derived or found in the literature [5, 6, 48], which have been explained in detail in Paper II.

The on-axis stresses at the bottom line of the model at mid-span were checked. Failure is defined when the empirical Norris failure criterion is reached. The reason to employ this failure criterion is explained in Section 3.1.2. The numerical ultimate moment capacity is referred to $M_{u,num}$. The mesh element type was chosen to be the 8-node hexahedral brick element with reduced integration (C3D8R in ABAQUS/CAE 6.14). The mesh size was 2.5 mm.
2.4. Adhesive bonding performance in single lap shear (Papers III and IV)

Fundamental knowledge regarding the bonding technique and bonding performance should be gained. It is of vital importance, but is currently in lack of research, particularly in terms of the hybrid bonding between different EWPs (plywood and glulam) and different species (birch and spruce).

In Papers III and IV, the bonding performance between birch plywood and spruce glulam was evaluated by conducting tensile shear tests. The bonding strength and the wood failure percentage (WFP) were examined. Moreover, the shear strengths of the wood adherends, i.e., spruce glulam and birch veneer, were also experimentally determined, so as to analyze the relation between the bonding strength and the wood shear strength. The investigated factors comprise the adhesive types, pressing methods, moisture, and load-to-plywood face grain angles.

2.4.1. Bonding strength

Investigated factors

i. Adhesive types

The research question about which type of adhesive should be used, ought to be studied in the first place. Based on a literature study done in Paper III, three types of adhesives, i.e., melamine-urea-formaldehyde (MUF), phenol-resorcinol-formaldehyde (PRF), and two-component polyurethane (2C PUR), were selected.

MUF and PRF take a considerable hold in glulam industries now and before while the last 2C PUR adhesive system has not been specifically approved for load-bearing constructions [50]. Nevertheless, this 2C PUR system exhibited satisfying performance in the preliminary test campaign. It is noted that this study focuses on research. Therefore, possible approvals of the adhesives are not considered.
Table 6 illustrates the adhesive and process-related parameters based on the technical data sheet and the results from a preliminary test campaign. The parameters optimized after performing the preliminary test series are underlined in Table 6.

Table 6: Adhesive and process-related parameters.

<table>
<thead>
<tr>
<th>Adhesives</th>
<th>MUF</th>
<th>PRF</th>
<th>2C PUR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mixing ratio (adhesive: hardener) (pbw)</td>
<td>1:1</td>
<td>5:1</td>
<td>5.7:1</td>
</tr>
<tr>
<td>Solid content (%)</td>
<td>63-64</td>
<td>55-61</td>
<td>100</td>
</tr>
<tr>
<td>Application amount (g/m²)</td>
<td>400</td>
<td>400</td>
<td>400</td>
</tr>
<tr>
<td>Open assembly time (min)</td>
<td>&lt;2</td>
<td>&lt;2</td>
<td>&lt;2</td>
</tr>
<tr>
<td>Close assembly time (min)</td>
<td>20</td>
<td>50</td>
<td>0</td>
</tr>
<tr>
<td>Pressing temperature (°C)</td>
<td>20</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>Pressing time for the two clamping methods (hour)</td>
<td>24</td>
<td>24</td>
<td>24</td>
</tr>
</tbody>
</table>

**ii. Pressing methods**

For bonded joints in timber connections, especially for large-scale timber structures, it is often practical to apply the pressure manually. In order to investigate how the pressing methods influence the bonding quality, three manual pressing methods, namely, screw-gluing, clamping by means of clamps and application of loads, were adopted. Although pressures from screw-gluing and clamping were not straightforwardly quantified, it is expected that the load application pressing method created the lowest pressure (around 0.01 MPa). Details regarding these pressing methods have been explained in Paper III.

**iii. Moisture**

Each type of adhesive was tested in both dry and moist conditions after the gluing was completed in the dry state. All the specimens were conditioned
in a climate room (T=20°C and RH=65%) for two weeks. The specimens tested in this condition were denoted as “dry condition” specimens. As a comparison, the “moist condition” specimens were further conditioned in another room (T=20°C and RH=100%) for five days before testing. MC measured on “dry condition” specimens was 11.8% for spruce glulam and 10.5% for birch plywood. MC of the “moist” specimens was 18.0% for both glulam and plywood.

It is worth noting that the pre-treatment method utilized in this study to examine the adhesive performance in the moist condition corresponds to service class 2 in EN 1995-1-1 [51] where the average MC of most softwoods will not exceed 20%. It is considered to be adequate as the scope of the thesis is limited to the indoor applications of birch plywood gusset plates, where the bond line would not be soaked by water but is likely to be exposed to high humidity.

iv. Load-to-plywood face grain angles

When employing birch plywood in timber structure applications such as trusses, stresses from different directions need to be transmitted by the plywood gusset plate. How the bonding strength is affected by different loading angles to the face grain should be addressed. In the experimental study, the glulam grain direction was kept parallel to the loading axis while the face grain direction of birch plywood varied from 0° (parallel) to 90° (perpendicular) with an interval of 15°.

Test series

To investigate the influence of the aforementioned factors on the bonding performance, tensile shear bond line tests were conducted on single lap joints. The information with regard to the test series is shown in Table 7.
Table 7: Information regarding the tensile shear bond line test series.

<table>
<thead>
<tr>
<th>Adhesives</th>
<th>Condition</th>
<th>Pressing methods</th>
<th>Load-to-plywood face grain angle (degree)</th>
<th>Replicates</th>
</tr>
</thead>
<tbody>
<tr>
<td>MUF</td>
<td>Dry</td>
<td>Screw-gluing</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Clamping</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Steel block</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Moist</td>
<td>Screw-gluing</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>PRF</td>
<td>Dry</td>
<td>Screw-gluing</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Clamping</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Steel block</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Moist</td>
<td>Screw-gluing</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>2C PUR</td>
<td>Dry</td>
<td>Screw-gluing</td>
<td>15</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>30</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>45</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>60</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Moist</td>
<td>Screw-gluing</td>
<td>75</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>90</td>
<td>6</td>
</tr>
</tbody>
</table>

Note: The "steel block" pressing method indicates that the bond line was pressed by applying weight loads, which created the lowest pressure (0.01 MPa).
Test methods

Several standards, e.g., EN 302-1, EN 14080, ASTM D4680, and ASTM D905 [32, 52-54], etc., describe the test method for determining the strength properties of adhesive bonds in shear. A single lap joint is suggested in these standards with the bonding area varying from 200 mm² to approximately 2000 mm². In this study, a bonding area of 1600 mm² (40 mm × 40 mm) was chosen. See Figure 9 for the adhesive test setup and configuration.

The bonding strength is defined as the maximum force during the load divided by the bonding area. In order to evaluate the influence of the investigated factors on the bonding strength, a two-way analysis of variance (ANOVA) was employed. The significance level was set at 0.05. If the influence is statistically significant, the Tukey post-hoc test could be further conducted to specify which group is significantly different from other groups.
2.4.2. **Wood failure percentage**

Wood failure percentage (WFP) was assessed for each specimen through visual inspection to the nearest 10% according to EN 314-1 [55].

2.4.3. **Wood shear strength**

The shear strengths of the wood adherends, i.e., spruce glulam and birch veneer, were also tested, with the same cross-section as the bonded ones (40 mm × 40 mm).

*Shear strength of spruce glulam*

Glulam specimens were tested in both dry condition and moist condition. Table 8 and Figure 10 display the information regarding the glulam shear test.

Table 8: Information regarding the glulam shear tests.

<table>
<thead>
<tr>
<th>Timber</th>
<th>Condition</th>
<th>Load-to-grain angle (degree)</th>
<th>Replicates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spruce glulam</td>
<td>Dry</td>
<td>0</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>Moist</td>
<td>0</td>
<td>8</td>
</tr>
</tbody>
</table>

Figure 10: Glulam shear test setup and configuration (unit: mm).
Shear strength of birch veneer

Seven test series were carried out to test the shear strength of birch veneer when its grain direction was oriented at different angles to the loading axis. The test series are summarized in Table 9. The test set-up and specimen configuration are displayed in Figure 11. From the side view, it can be seen that there is a saw cut from each side of the specimen, which is extended to the middle veneer. The birch plywood has a nominal thickness of 30 mm and is composed of 21 veneers. It is noticed that the middle veneer has the same grain direction as the face veneer.

Table 9: Information regarding the birch veneer shear tests.

<table>
<thead>
<tr>
<th>Timber</th>
<th>Condition</th>
<th>Load-to-plywood face grain angle (degree)</th>
<th>Replicates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Birch veneer</td>
<td>Dry</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>15</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>30</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>45</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>60</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>75</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>90</td>
<td>5</td>
</tr>
</tbody>
</table>

Figure 11: Birch veneer shear test setup and configuration (unit: mm).
2.5. Moment-resisting bonding performance (Paper V)

2.5.1. Experiments

Two glulam beams were adhesively connected by birch plywood plates at mid-span and then loaded in four-point bending. Four test series with two different bonding areas (100 mm × 100 mm and 140 mm × 140 mm) and birch plywood face grain orientations (0° and 22.5° to the beam length direction) were carried out. Each test series had three replicates and was labeled following the format: bonding area (AS (adhesive small) or AL (adhesive large)) – face grain orientation (0 or 22.5). The bonded region was designed as the weakest part to investigate the failure modes, moment capacity, bending stiffness, etc. See Figure 12 for the geometrical configuration of the tested specimens.

Figure 12: (a) Geometrical configuration of the test specimen, (b) two different bonding areas and face grain directions resulting in four test series, and (c) schematic illustration of the LVDT installation (unit: mm).
2.5.2. Analytical models

Analytical analyses were carried out to predict the moment capacity of the adhesively bonded timber-to-timber connection. One simple theoretical model that can be adopted is illustrated in “Model 1” in Figure 13 [56-58]. However, there are several limitations when applying this theoretical model to predict the moment capacity. Firstly, the theoretical model assumes that the bonded surface would fail when the maximum shear stress at the corners reaches the bonding strength, which might be conservative. Secondly, the shear stress at the corners has different angles to the glulam grain direction and also to the birch plywood face grain direction when the shape of the bonded region changes. Therefore, a large number of tests are required in order to characterize the bonding strength at any angles to the glulam grain direction.

Considering these limitations, a few modifications were made on “Model 1”. The stresses in “Model 1” were simplified into two groups, i.e., one parallel to the glulam grain direction (red arrows) and the other one perpendicular (blue arrows). The contribution from the blue arrows could be neglected as the rolling shear modulus of glulam is much lower than the longitudinal shear modulus of glulam (approximately one-tenth) [59]. After this simplification, the required input bonding strength is always parallel to the glulam grain direction. Moreover, the failure definition is adjusted in “Modified model 3” such that the maximum shear stress is uniformly distributed in a certain area close to the edge. By comparing the analytically derived moment capacity to the experimental result, the area with the uniform shear stress can be determined.
2.5.3. Small-scale supplementary tests

It is noticed in Figure 13d that the analytical prediction of the moment-capacity based on “Modified model 3” requires the bonding strength values as input. By assuming that only a small area close to the edge would reach
the bonding strength ($f_v$), small-scale supplementary tests were conducted with a bonding area of 200 mm² (20 mm x 10 mm), following the standard EN 302-1 [52]. Two test series were carried out with 10 replicates for each. As shown in Figure 14, the glulam grain direction is always parallel to the loading axis while the birch plywood face grain direction is 0° (parallel) or 22.5° to the loading axis, which is in line with the moment-resisting tests presented in Section 2.5.1. The bonding strengths tested from these two test series are referred to as $f_v(0-0)$ and $f_v(0-22.5)$, respectively.

![Figure 14: The setup and the specimen configuration of small-scale supplementary tests (unit: mm).](image)

2.5.4. Numerical models

3D numerical models were developed to predict the structural performance of the moment-resisting bonded connections within the linear-elastic stage. A surface-to-surface cohesive contact modeling method was adopted with the elastic contact behaviors assigned for the bonded regions. One of the input properties is the shear stiffness of the adhesive (2C PUR). A wide range of stiffness data can be found in the
literature from 586 MPa to 1919 MPa [60, 61]. In order to investigate how this value would influence the bending stiffness of the global structure, a parametric analysis was carried out by varying the shear stiffness value within the range from 400 MPa to 2200 MPa with an interval value of 200 MPa. To quantify the closeness of the numerically predicted bending stiffness \( (E_I_{\text{num}}) \) to the experimental ones \( (E_I_{\text{exp}}) \), the ratio \( (E_I_{\text{num}}/E_I_{\text{exp}}) \) was calculated for each individual test.

Once the input adhesive stiffness is determined, it is possible to investigate how the bonded area affects the elastic response of the global structure. Systematic analysis was conducted numerically by changing the bonded area from 10000 mm\(^2\) \((100 \times 100 \text{ mm}^2)\) to 99000 mm\(^2\) \((225 \times 440 \text{ mm}^2)\) gradually. It is noted that \(225 \times 440 \text{ mm}^2\) is the maximum area that can be bonded in this study.

### 2.6. Tensile capacity of birch plywood in adhesively bonded connections (Paper VI)

As can be noticed in Figure 2, plywood plates are usually wider than the truss elements. The additional width of the plywood could also help resist the transferred axial force. The contribution of the additional width on the load-bearing capacity of the plywood gusset plates should be quantified and formulated for better design.

#### 2.6.1. Reference tensile strength

Prior to the studies of the additional width, the case when the plywood plate had the same width as glulam was first evaluated in order to obtain reference tensile strength values for further analysis.

Each specimen consists of four glulam elements and one thin birch plywood plate. The glulam element has a height \((h)\) of 440 mm, a width \((b_0)\) of 110 mm, and a thickness of 56 mm, respectively. Four glulam elements were bonded to one thin plywood plate, with a nominal thickness \((t)\) of 6.5 mm, and then loaded in tension. Tests were carried out at three different load-to-plywood face grain angles, i.e., \(0^\circ\), \(22.5^\circ\), and \(45^\circ\), and the gap \((g)\) between glulam elements was kept as 10 mm. It is interesting to
find out that the derived tensile strength shows a notable difference compared to the tensile strength previously tested from the dumbbell-shaped specimens in Section 2.2.1, especially when the load-to-plywood face grain angles, are 22.5° and 45°. The specific difference concerning the tensile strengths is displayed in Section 3.5.1. In order to explain the observed difference, specimens with longer gaps ($g_i$) were further tested. The test setup is shown in Figure 15.

Figure 15: Uniaxial tensile test of birch plywood when bonded to glulam (a) reference tensile test and (b) reference tensile test with longer gap.
2.6.2. Experimental studies on the plywood additional width

After conducting the tests in Figure 15, the width of plywood was increased step by step until the tensile capacity reached a plateau (see Figure 16). These studies with the increased width were performed with a constant gap of 10 mm, to mimic the real situation. Each test series had three replicates.
2.6.3. Spreading angle analysis

An effective-width method was firstly introduced by Whitmore for the design of the steel gusset plate with dowel-type fasteners [62]. The definition of the “Whitmore effective width” is described in Figure 17b. However, birch plywood, as an anisotropic material, exhibits different mechanical properties at varying angles to the face grain. Therefore, the 30-degree spread angle theory proposed by Whitmore is not valid for birch plywood gusset plates and should be further investigated.

![Figure 17](image-url)

Figure 17: (a) Steel gusset plate and (b) definition of the “Whitmore effective width”.

Two analytical models were developed herein with the concept of effective width and spreading angle to predict the tensile capacity of birch plywood in adhesively bonded connections. These two analytical models are referred to as simple and advanced spreading angle models.

The simple spreading angle model

The simple model with a new spreading angle, $\alpha$, is illustrated in Figure 18. It is assumed that the force would transfer from the outermost line of the bonded area, leading to a uniform tensile stress distribution.

As aforementioned, the gap between the glulam elements would affect the reference tensile strengths at 22.5° and 45°. As a result, different tensile
strengths were employed for the plywood within and outside of the bonded area at $22.5^\circ$ and $45^\circ$ in the analytical models.

![Diagram of the simple spreading angle model](image.png)

Figure 18: The simple spreading angle model.

The effective width of birch plywood, $w_{\text{eff}}$, can be expressed as:

$$w_{\text{eff}} = b_0 + 2 \tan(\alpha) l, \quad (25)$$

where $l$ is the length of the bonded region. The tensile capacity of birch plywood predicted by the simple spreading angle model, $F_{t,\theta,b_p,\text{sim}}$, can be calculated as:

$$F_{t,\theta,b_p,\text{sim}} = f_{t,b_0} b_0 t + f_{t,b_0,l}(\min(w_{\text{eff}},b_p) - b_0) t, \quad (26)$$

where $f_{t,b_0}$ is the reference tensile strength of birch plywood with a small gap between the glulam elements; $f_{t,b_0,l}$ is the reference tensile strength of birch plywood with a longer gap between the glulam elements; and $b_p$ is the width of plywood.
The advanced spreading angle model

The advanced spreading angle theory assumes that the tensile force is uniformly taken by each part of the bonded area, leading to a more realistic non-uniform stress distribution outside of the bonded region in the plywood plate. The concept of the advanced spreading angle theory is shown in Figure 19.

![Figure 19: A modified spreading angle theory (a) concept illustration and (b) stress distribution.](image)

The effective width of birch plywood, \( w_{\text{eff,adv}} \), in the advanced model, can be expressed as:

\[
\begin{align*}
\text{(27)} \quad \text{where } & \alpha_{\text{adv}} \text{ is the advanced spreading angle. Detailed derivations have been presented in Paper VI. The solution of the tensile capacity of the birch plywood plate based on the advanced spreading angle theory is summarized in Eq. 28 and Eq. 29.}
\end{align*}
\]

\[w_{\text{eff,adv}} = b_0 + 2 \tan(\alpha_{\text{adv}}) l,\]
When \( b_p \geq w_{eff,adv} \),

\[
F_{t,\theta,b_p,adv} = (f_{t,b_0} - f_{t,b_0,t})b_0 t + \frac{2lt \tan(\alpha_{adv})f_{t,b_0,t}}{\ln(w_{eff,adv})-\ln(b_0)}
\]  \hspace{1cm} (28)

When \( b_p < w_{eff,adv} \),

\[
F_{t,\theta,b_p,adv} = f_{t,b_0}b_0 t + f_{t,b_0,t}t \left( b_p - b_0 - \frac{b_p \ln\left(\frac{b_p}{b_0}\right)-b_p+b_0}{\ln(w_{eff,adv})-\ln(b_0)} \right)
\]  \hspace{1cm} (29)

**Determination of the spreading angle**

Spreading angles were determined for each proposed model and for each investigated load-to-face grain angle, by comparing the analytically predicted tensile capacity with the test results. Least squares analysis was carried out to find the spreading angles with the highest R square value. Meanwhile, the predicted tensile capacity should also reach a plateau and the effective width should be no more than the maximum tested width.

**2.6.4. Stress distributions**

After the determination of the spreading angles, stress distributions of the birch plywood plate can be derived from the analytical models, which were further compared to the stress distributions obtained from the numerical simulations.

The simulated models comprise the specimen configurations with the maximum width at each load-to-face grain angle. The load was applied as surface tractions on the bonded surfaces. Failure was defined when the maximum tensile stress reached the tested reference tensile strength \( f_{t,b_0} \).

The mesh element type was the 4-node shell element with reduced integration (S4R in ABAQUS/CAE 6.14) and the mesh size was 2.5 mm. Similar to the numerical model described in Section 2.3.2, birch plywood is characterized as an orthotropic material. The investigated numerical model is depicted in Figure 20.
Figure 20: The numerical model for the analysis of stress distributions.
3. Results and discussion

3.1. In-plane tensile, compressive, and shear properties (Paper I)

3.1.1. Experimental results

All the stress-strain curves at five different angles in tension, compression, and shear are plotted in Figure 21 until reaching the failure load with the typical ones highlighted.

Figure 21: Stress-strain relationships of birch plywood specimens in (a) tension, (b) compression, and (c) shear. The size of the different specimens is shown in Figure 5.
It is readily seen in Figure 21a and 21b that, the stress-strain curves in tension are practically linear-elastic while those in compression exhibit elasto-plastic behavior. For the specimens tested in compression at 0° and 90°, a distinct plastic plateau can be observed after yielding, while for the ones at 22.5°, 45°, and 67.5°, hardening-type plasticity is noticed.

As shown in Figure 21c, the shear stress grows linearly with the increase of the shear strain, followed by a non-linear response before reaching the peak value. The stress transformation model illustrated in Eq. 4 and Figure 6 in Section 2.2.2 can help explain the nonlinear shear stress-strain relationships observed in Figure 21c. When shear specimens were loaded in off-axis directions (i.e., there is an angle \( \theta \) (0° < \( \theta \) < 90°) between the loading axis and the face grain angle), the off-axis shear stress (\( \tau_{12} \)) can be transformed to on-axis normal stresses (\( \sigma_x \) and \( \sigma_y \)). Following the specimen loading condition presented in Figure 6c, \( \sigma_x \) is in tension and \( \sigma_y \) is in compression. With the increase of the external off-axis shear force during loading, it is likely that \( \sigma_y \) would surpass the elastic limit. The elasto-plastic compressive behavior in the y direction could result in the nonlinearity in panel shear. It is also worth noting that the nonlinear shear stress-strain relationships have also been reported by other investigations performed on beech plywood and maritime pine clear wood [63, 64].

The tensile and compressive strength and elastic modulus in relation to the loading angle to the face grain are displayed in Figure 22.
Birch plywood exhibits the highest tensile strength at 0°, with mean tensile strength over 60 MPa. The tensile strength drops with the increase of the loading angle till 45°. Then, it increases gradually from 45° to 90°. The tensile strength at 45° is approximately 20 MPa, which is roughly one-third of that at 0°. The degree of angle-dependence of the compressive strength is lower than that of the tensile strength property. Both tensile and compressive strength and elastic modulus at 90° are close to those at 0°, which can be explained by the cross-lamination configuration of birch plywood.

The elastic modulus shows a similar tendency to the strength properties; but the angle-dependence is more noticeable. Moreover, it is observed that the elastic modulus derived from the compressive tests is slightly higher than that from the tensile tests. Theoretically, the tensile and compressive elastic modulus should be identical. However, it is rarely the case in the laboratory tests. In this study, the compressive specimens were partially constrained due to the contact between the steel plates and the end surfaces of birch plywood during the loading. It is likely that this constraint might overrate the elastic modulus. On the other hand, there was no constraint in the middle part of the tensile specimen with a constant cross-sectional area. Thus, the elastic modulus derived from the tensile tests is considered to be more representative.

As indicated in Figure 22a, birch plywood possesses higher tensile strength than compressive strength. This is in contrast with the mechanical properties of common timber products used in constructions, e.g., glulam or lumber, where – due to the presence of defects such as knots or grain deviation - compressive strength is typically higher than tensile strength. The mechanical properties of birch plywood reflect indeed those of clear wood or, more generally, those of the wood in a tree before cutting it down, where tensile strength is typically higher than compressive strength.

The shear strength and shear modulus in relation to the loading angle to the face grain are presented in Figure 23.
The angle dependence of the shear properties is the opposite of the tensile and compressive ones. The angle-dependent mechanical properties can be explained with the help of the stress transformation model illustrated in Section 2.2.2. When the tensile or compressive specimens are loaded in off-axis directions with a stress $\sigma_1$, the on-axis shear stress ($\tau_{xy}$) is activated. $|\tau_{xy}|$ is equal to $|\sigma_1| \cdot \cos(\theta) \cdot \sin(\theta)$; thereby it is the highest at $45^\circ$. The influence of $\tau_{xy}$ is dominant because the on-axis shear strength is much lower than the on-axis tensile or compressive strength (see Figure 22a and 23a), leading to the lowest off-axis tensile and compressive strengths at $45^\circ$.

Similarly, when the shear specimens are loaded in off-axis directions, the on-axis tensile and compressive stresses are involved and are the highest at $45^\circ$. The “negative” influence of the low on-axis shear strength is minimized at $45^\circ$, resulting in the highest off-axis shear strength at $45^\circ$.

It should be noted that, when comparing birch plywood with other commonly used structural timber elements, e.g., structural lumber or glulam, etc., the lowest tensile or compressive strength of birch plywood at $45^\circ$ is more or less the same as the highest tensile or compressive strength of these timber elements in their longitudinal direction. The lowest panel shear strength of birch plywood at $0^\circ$ and $90^\circ$, is remarkably higher than the shear strength of timber elements made of softwood [65].
outstanding mechanical properties of birch plywood make it promising for structural applications.

3.1.2. Predictions of off-axis strength

The off-axis strength prediction results are displayed in Figure 24. To quantify the closeness of the predicted strengths to the test data, statistical analyses were performed by comparing the root mean square error (RMSE) of each failure criterion. The failure criterion with the minimum RMSE is considered as the one that fits the best to the test data. The definition and the specific results of RMSE have been reported in Paper I.

As noticed in Figure 24a and 24b, the Hankinson’s failure curve varies between the strengths at 0° and 90°, considerably above the test data. On the contrary, the other employed linear failure criterion is rather conservative. Both of them are not applicable for predicting the strength of birch plywood. The quadratic failure criteria give more accurate
predictions than the linear criteria. In particular, the empirical Norris failure criterion gives the closest prediction to the tested tensile and compressive data, followed by Tsai-Hill and theoretical Norris failure criteria.

It is shown in Figure 24c that, all the criteria investigated herein underrate the panel shear strengths of birch plywood. Thus, one bilinear model is proposed by the author that linearly links the shear strengths at 0°, 45°, and 90°.

The interaction coefficient \( F_{12} \) in the Tsai-Wu failure criterion would affect the predicted results, to some extent. However, the Tsai-Wu failure criterion is neither competitive to the empirical Norris failure criterion in tension and compression, nor to the bilinear model in panel shear.

In conclusion, the empirical Norris failure criterion is recommended in applications where the birch plywood is mainly subjected to tensile or compressive stress. On the other hand, the bilinear model is suggested when the panel shear stress is dominant in the birch plywood plate.

3.1.3. Predictions of off-axis elastic properties

The elastic modulus prediction was compared to the experimental elastic modulus from the tensile tests since the results are more representative as aforementioned in Section 3.1.1.

The predicted results in comparison to the experimental data are presented in Figure 25.
It is evident that the transformation model predicts the elastic modulus quite well but overestimates the shear modulus. It is worth mentioning that, the Poisson’s ratio \( \nu_{xy} \), as one of the parameters in the transformation model, was derived instead of being tested. Sensitivity analysis has been performed in Paper I to study its influence on the elastic property prediction. It was found that its effect on the off-axis elastic modulus is negligible. Additionally, \( \nu_{xy} \) would influence the prediction of the shear modulus. However, the modified transformation model is consistently the superior choice.

### 3.2. Edgewise bending properties (Paper II)

#### 3.2.1. Factor I: face grain angle

*Experimental results*

The influence of the face grain angle on the edgewise bending properties of birch plywood can be revealed in Table 10. As expected, when the face grain angle varies from 0° to 90°, the edgewise bending properties exhibit a similar tendency to the tensile and compressive properties; birch plywood specimens possess the highest bending strength and elastic modulus at 0° and both the lowest at 45°.

Table 10: Experimental results of ultimate moment capacity, bending strength, and elastic modulus at five different load-to-face grain angles (\( d=50 \) mm, \( MC=11.9\% \)).

<table>
<thead>
<tr>
<th>( \theta ) (°)</th>
<th>( M_{u,exp} ) (Nm)</th>
<th>( f_b ) (MPa)</th>
<th>( MOE_b ) (GPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>559.4 (4.8)</td>
<td>65.1 (4.7)</td>
<td>10.6 (6.9)</td>
</tr>
<tr>
<td>22.5</td>
<td>354.4 (3.7)</td>
<td>41.5 (3.5)</td>
<td>4.4 (4.0)</td>
</tr>
<tr>
<td>45</td>
<td>264.0 (4.6)</td>
<td>31.0 (4.5)</td>
<td>2.4 (4.7)</td>
</tr>
<tr>
<td>67.5</td>
<td>311.0 (2.4)</td>
<td>36.6 (2.7)</td>
<td>3.4 (5.9)</td>
</tr>
<tr>
<td>90</td>
<td>417.3 (7.1)</td>
<td>48.7 (7.5)</td>
<td>7.3 (8.8)</td>
</tr>
</tbody>
</table>

Note: the numbers within parentheses indicate the coefficient of variation (COV) values.
Prediction results

Both analytical and numerical analyses were carried out to predict the influence of load-to-face grain angle on the ultimate moment capacity. The comparison between the analytical, numerical, and experimental results has been reported in Paper II. It was found that both analytical and numerical predictions underestimate the ultimate moment capacities at all the five angles to the face grain, which may be attributed to the conservative failure definition.

Tensile strength is determined by the uniaxial tensile test where the tensile stress is nearly uniform on the cross-section. The weakest fiber governs the tensile strength due to the weakest link theory [66]. In edgewise bending, only the extreme fiber in the bottom is subjected to the highest tensile stress. This smaller part has a higher probability to be defect-free, i.e., not the weakest link in the material. Hence, it may be reasonable to check the average stress of a certain part of the cross-sectional area from the bottom. Different failure definitions are illustrated in Figure 26.

![Cross section at mid-span](image)

**Different failure definitions**

Figure 26: Illustration of different failure definitions.

The influence of the defined area on the prediction of the ultimate moment capacities was studied by conducting parametric analysis. Results show that, when the defined failure area is around 20-25% of the cross-sectional area, the analytically and numerically predicted results are the closest to $M_{u,exp}$. 

3.2.2. Factor II: Size

In order to study the size effect on the edgewise bending properties, birch plywood beams with nominal depths of 20 mm and 50 mm were tested first. It was found that the size effect on the elastic modulus is not noticeable. Moreover, the size effect on the edgewise bending strength is noticeable on the specimens at 0° and 90° but nearly negligible at 22.5°, 45°, and 67.5° (see Figure 27). Therefore, specimens with nominal depths of 40 mm and 30 mm, were loaded at 0° and 90°, to further study the size effect between 20 mm and 50 mm.

![Figure 27: Edgewise bending strength at different load-to-face grain angles and different sizes. The error bars denote a 95% confidence interval, based on one-sample t-tests.](image)

The discrepancy mentioned above could possibly be explained by their different failure characteristics that for the birch plywood beams at 0° and 90°, force dropped dramatically once reaching the peak. While for the beams at 22.5°, 45°, and 67.5°, the post-peak behavior was significantly different; force decreased gradually instead. Consequently, at 0° and 90°, the birch plywood beams with the brittle failure mode are more dependent on the stressed volume.
3.2.3. Factor III: moisture content

Birch plywood beams were tested at three MC levels, i.e., 7.2%, 11.9%, and 21.8%, with the constant nominal depth of 20 mm. See Figure 28 for the test results.

Both the bending strength and elastic bending modulus decrease when the moisture content increases in the hygroscopic range. The linear regression models generalize the moisture effects fairly well with the lowest R square value of 0.94.
3.2.4. Bending strength–elastic bending modulus relationships

The bending strength and elastic bending modulus of each specimen from each test series are plotted in Figure 29.

![Figure 29: Relationships between edgewise bending strength and elastic bending modulus.](image)

The relationship between strength and modulus can be well expressed by a linear function (see the black dashed lines in Figure 29). This linear relationship was also observed on birch sawn timber [67]. The clear relationships confirm the feasibility of assessing the edgewise bending strength of birch plywood by detecting its elastic modulus non-destructively.
3.3. Adhesive bonding performance in single lap shear (Papers III and IV)

When evaluating the bonding performance between birch plywood and spruce glulam in single lap shear, the investigated factors comprise the adhesive types, pressing methods, moisture, and load-to-plywood face grain angles.

3.3.1. Factor I: Adhesive types

Three types of adhesives, namely, MUF, PRF, and 2C PUR, were tested. Figure 30 shows the bonding strength results in the dry condition (T=20°C and RH=65%), with the shear strength of spruce glulam as a reference. The error bars denote the standard deviation. It is noted that, when analyzing factors I-III, both spruce glulam and birch plywood have the grain or face grain direction parallel to the loading axis. Major wood failure occurs in spruce glulam. Hence, the shear strength of birch veneer is not presented in Figure 30. A comparison of the wood shear strengths between spruce glulam and birch veneers is displayed in Section 3.3.4.

Figure 30: Bonding strengths in the dry condition with the grain direction of spruce glulam and face grain direction of birch plywood parallel to the loading axis.
It is noticed that the specimens bonded with 2C PUR exhibit the highest bonding strength, over 5 MPa, while the ones with MUF and PRF possess lower but similar bonding strength properties (4-5 MPa). All the bonding strengths are within or close to the pink region representing the mean shear strength of spruce glulam ± one standard deviation.

Two-way analysis of variance (ANOVA) reveals that the bonding strength has significant difference in terms of the adhesive types ($p<0.001$). The slightly higher bonding strength with 2C PUR could possibly be explained by a more uniform shear stress distribution along the bond line. A few researchers commented that PRF and MUF are stiffer than PUR (one component or two components) [22, 68-70]. The adhesives with high shear modulus tend to exhibit higher shear stress concentration at the end of the bond line; the shear stress distribution is, however, more homogenous for less stiff adhesives [68, 71]. The non-uniform stress distribution would result in the underestimation of the bonding strength [72], which might be the cause for the slightly lower bonding strength with MUF and PRF in this study. It is worth mentioning that the findings are only valid in this specific case with a relatively small bonded area. When scaling up the bonded area to the industry level, the influence of the adhesive types might be different.

### 3.3.2. Factor II: Pressing methods

Bonding strength results in relation to the manual pressing methods investigated in this study is also displayed in Figure 30. Two-way analysis of variance (ANOVA) indicates that the influence of the pressing methods ($p=0.353$) is not significant.

It is worth noting that the screw was not withdrawn before the tests. Supplementary tests have also been conducted on the specimens with the screw withdrawn before the tests. It was found that the bonding strength with or without the screw showed no significant difference at a significance level of 0.05. This might be attributed to: firstly, the slip modulus of a single screw with an outer diameter of 4 mm is much lower than the shear stiffness of a “large” bonded area of 1600 mm². The force in the loading direction was thus mostly taken by the bonded connection rather than by the screw; secondly, the peeling stress is usually high close to the edge and
low in the center area of the bond line [30, 72]. In this study, the screw was inserted at the center of the bonded area since it also served the purpose of applying pressure during assembly. Consequently, the peeling stress taken by the screw might be negligible. Therefore, the influence of the screw in the bonded area can be neglected in this context. Besides, the bonding strength of the specimens without the screw is also within the range of the glulam shear strength. It indicates that the interaction effect between the peeling stress and bonding shear stress is not pronounced in this test. These discussions regarding the influence of the screw have been mentioned in Paper III.

3.3.3. Factor III: Moisture

In Figure 31, the test results in the moist condition are compared to the bonding test results in the dry condition. The error bars denote the standard deviation.

![Graph showing bonding strength comparison](image)

Figure 31: Test results in the moist condition compared to the bonding test results in the dry condition.
The test results indicate that, although the mean shear strength of spruce glulam in the moist condition (3.8 MPa) is 30% lower than its shear strength in the dry condition (5.4 MPa), the bonding strengths in the moist condition only decrease 6.4%, 4.4%, and 6.2% for the MUF, PRF, and 2C PUR specimens, respectively. Satisfactory bonding performance is unveiled for all the tested adhesives. The slight reduction of the bonding strengths in the moist condition (MC = 18%) compared to the dry condition (MC = 10-12%) also implies the effectiveness of the adhesives to be applied in service class 2 in EN 1995-1-1 [51].

3.3.4. Factor IV: Load-to-plywood face grain angles

Bonding strengths with a comparison to the weaker wood shear strength at different load-to-plywood face grain angles are presented in Figure 32. The error bars denote the standard deviation.

![Figure 32: A comparison between the bonding strengths and the weaker wood shear strengths.](image)

The test results indicate that, when varying the load-to-plywood face grain angles from 0° to 90°, the bonding strength (black line) changes within a relatively small range, from 4.6 MPa to 6.5 MPa, although there are some statistically significant differences. The weak angle-dependence of the
bonding strength is beneficial for the design and the development of the bonded joints when utilizing birch plywood as gusset plates.

The shear strength of spruce glulam tested parallel to the grain (red line) was compared to the tested shear strength of birch veneer from 0° to 90° (blue line). It was found that the longitudinal shear strength of spruce glulam is lower than the birch veneer shear strength at 0° and 15° but higher than that at other angles. Although, at 90°, the birch veneer is subjected to rolling shear, the rolling shear strength of birch veneer (4.6 MPa) is only slightly lower than the longitudinal shear strength of spruce glulam (5.4 MPa). A clear correlation was observed between bonding strengths and shear strengths of the weaker wood adherend, which is in line with the results in the literature [73].

Furthermore, the wood failure percentage (WFP) of each specimen was examined through visual inspection to the nearest 10% according to EN 314-1 [55]. The total WFP was above 70% at all the load-to-plywood face grain angles, with the lowest WFP of 70% at 45° and the highest of 95% at 75°. Some typical failure modes of the bonded specimens are shown in Figure 33. At 0° and 15°, failure mainly occurred in the bulk glulam; while at other angles, a mixture of cohesive failure in glulam and plywood face veneer was dominant. Only a very small amount of wood failure took place in the second veneer of plywood and its influence can be considered negligible.
3.4. Moment-resisting bonding performance (Paper V)

3.4.1. Experimental results

All the test specimens failed in the bonded area as expected. The wood failure (either spruce glulam or birch plywood) took up most of the bonded surfaces, with an average WFP of more than 80% via visual inspections. The mean and COV values of the moment capacity and bending stiffness of each test series are listed in Table 11. Moreover, it is also interesting to compare the structural behaviors of the adhesively jointed beams with those of a single unjointed glulam beam with the same span and cross-section. Therefore, the analytically calculated moment capacity and bending stiffness of an unjointed glulam beam are also compared to the experimental results in Table 11.

Table 11: Experimental results of each test series and the analytical results of an unjointed beam.

<table>
<thead>
<tr>
<th>Beam</th>
<th>Moment capacity (kNm)</th>
<th>Bending stiffness (kNm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS-0</td>
<td>4.4 (7.3%)</td>
<td>353.5 (0.3%)</td>
</tr>
<tr>
<td>AS-22.5</td>
<td>4.7 (6.2%)</td>
<td>293.2 (2.5%)</td>
</tr>
<tr>
<td>AL-0</td>
<td>12.1 (8.3%)</td>
<td>517.4 (3.5%)</td>
</tr>
<tr>
<td>AL-22.5</td>
<td>10.7 (2.4%)</td>
<td>407.8 (2.7%)</td>
</tr>
<tr>
<td>Unjointed</td>
<td>16.9</td>
<td>664.5</td>
</tr>
</tbody>
</table>

Note: The numbers within parentheses indicate the COV values.

It was found that when the bonding area is relatively small (“AS” groups with the bonding area of 100 × 100 mm²), AS-0 exhibits slightly lower moment capacity than AS-22.5. This finding becomes different when the bonding area is relatively large (“AL” groups with the bonding area of 140 × 140 mm²). The moment capacity of AL-0 is around 13% higher than that of AL-22.5. As a result, the ratio of the moment capacity between AL-0 and AS-0 is 2.75, which is higher than the ratio between AL-22.5 and AS-22.5, i.e., 2.28. This might be attributed to the angle-dependent size effect. Moreover, it is shown in Table 11 that, the bending stiffness of the 0° test
series is higher than that of the 22.5° test series. This phenomenon is expected because it has been found in Paper II that birch plywood depicts the highest edgewise bending stiffness when its face grain is parallel to the beam length direction. In addition, according to the analytical calculations, the single unjointed glulam beam possesses both higher moment capacity and bending stiffness than the adhesively connected beams. However, as aforementioned, the bonded joint was designed as the weakest part in this study so as to investigate the structural behaviors of the adhesively bonded connection. Stronger and stiffer joints could have been achieved by simply increasing the bonded area.

3.4.2. Small-scale supplementary test results

In the analytical model proposed in Section 2.5.2, there is a certain area with the assumed uniformly distributed shear strength close to the edge. The bonding strengths tested from the small-scale supplementary tests serve as the input data in the proposed analytical model. The mean bonding strengths are 9.48 MPa for $f_v(0-0)$ and 8.76 MPa for $f_v(0-22.5)$.

3.4.3. Analytical predictions

In order to determine the proportion of this certain area to the bonded area, the analytically derived moment capacity was compared to the experimental results in the test series AL-0 and AL-22.5. It was found that when the proportion of this certain area to the bonded area is around 31% and 26% for AL-0 and AL-22.5, respectively, the analytical predictions are in good agreement with the experimental results.

With the determined bonding strengths from the small-scale tests and the determined proportions, the analytical model was then employed to predict the moment capacity for AS-0 and AS-22.5. A comparison between the prediction results and the experimental results is shown in Table 12.
Table 12: A comparison between the experimental and analytical moment capacities for AS-0 and AS-22.5.

<table>
<thead>
<tr>
<th>Test series</th>
<th>Moment capacity (kNm)</th>
<th>Experimental mean results</th>
<th>Analytical predictions</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS-0</td>
<td>4.4</td>
<td>4.4</td>
<td></td>
</tr>
<tr>
<td>AS-22.5</td>
<td>4.7</td>
<td>3.9</td>
<td></td>
</tr>
</tbody>
</table>

As displayed in Table 12, for the test series AS-0, the analytically predicted moment capacity agrees well with the experimental results. For the test series AS-22.5, the analytical prediction is nearly 20% lower than the tested moment capacity. Although it is, on one hand, favorable for the design purpose since the prediction leaves some safe margins, on the other hand, it implies that the input bonding strength employed for the prediction of AL-22.5 might be underestimated for AS-22.5, leading to the lower prediction value for AS-22.5.

3.4.4. Numerical predictions

A surface-to-surface cohesive contact modeling method was adopted for the bonded joints. It is noticed that the assigned adhesive stiffness would influence the predicted bending stiffness and the closeness to the experimental results. The numerically predicted bending stiffness reaches the closest value to the experimental results when the shear stiffness of the adhesive is around 900 MPa. The value 900 MPa is used in further numerical studies.

In the experiments, only two bonded areas were tested, i.e., 100 × 100 mm² and 140 × 140 mm². To investigate how the bonded area affects the elastic response of the global structure, systematic analysis was conducted numerically by changing the bonded area from 10000 mm² (100 × 100 mm²) to 99000 mm² (225 × 440 mm²) gradually. The bending stiffness in relation to the relative bonded area is presented in Figure 34. The relative bonded area is defined as the percentage of the bonded area to the maximum area that could be bonded. Each individual bending stiffness
value from the test series AS-0 and AL-0 is also presented in Figure 34 for comparison to the numerical predictions.

![Graph showing bending stiffness vs. bonded area](image)

**Figure 34:** The numerically predicted bending stiffness in relation to the bonded area.

It can be seen in Figure 34 that the bending stiffness of the adhesively connected beam increases nonlinearily with the increase of the bonded area. When the bonded area is over $180 \times 180 \text{ mm}^2$, the connected beam could be stiffer than an unjointed glulam beam with the same span and cross-sectional area.

### 3.5. Tensile capacity of birch plywood in adhesively bonded connections (Paper VI)

#### 3.5.1. Reference tensile strength of birch plywood

As introduced in Section 2.6, the first step in this study was to evaluate the case when the plywood plate had the same width as glulam in order to obtain the reference tensile strengths for further analysis. Figure 35 compares the tensile strengths of birch plywood obtained from the adhesively bonded specimens with a small gap ($f_{t,b_0}$) and a long gap ($f_{t,b_0,l}$),
and from the dumbbell-shaped specimens \( (f_{t, dumbbell}) \). The size of different specimens has been introduced in Section 2.2.1 and Section 2.6.1.

As a comparison to \( f_{t,dumbbell} \), \( f_{t, b0} \) at 0° (around 50 MPa) is slightly lower but the tensile strengths at 22.5° and 45° are much higher (slightly over 40 MPa), leading to a significantly lower angle-dependence. This is due to that the small gap between the bonded regions restricted the crack propagation of the 22.5° and 45° plywood but had nearly no influence on the failure mode of the 0° plywood. The higher tensile strengths at 22.5° and 45° and the lower degree of anisotropy of birch plywood in adhesively bonded connections are promising for the design of plywood gusset plates in glued applications. When the specimens were designed with a longer gap, \( f_{t,b0,l} \) at 22.5° and 45° are closer to the dumbbell-shaped test results.

### 3.5.2. Experimental and analytical results on the plywood additional width

The second step in this study was to increase the birch plywood width step by step until the tensile capacity \( (F_{t,bp}) \) reached a plateau. Two analytical models, namely, the simple and advanced spreading angle models, were
introduced in Section 2.6.3. Experimental results and analytical predictions on the tensile capacities of birch plywood in adhesively bonded connections in uniaxial tension are displayed in Figure 36.

The test results indicate that, when $F_{t,bp}$ approaches the maximum, $F_{t,bp}$ at $0^\circ$ (roughly 60 kN) is slightly higher than the other two at $22.5^\circ$ and $45^\circ$ (nearly 50 kN). However, the tensile capacities at different load-to-face...
grain angles reach plateaus at certain widths. In other words, when the load-to-plywood face grain angle is 45°, the activated width of plywood that resists the tensile force is the largest, the same width being the smallest at 0°.

As shown in Figure 36, the simple and advanced spreading angle models predict slightly different tensile capacities but both predictions are close to the experimentally determined tensile capacities. More specifically, the simple model tends to overestimate the tensile capacities while the advanced one tends to underrate the tensile capacities. The advanced spreading angle model might be more suitable for design purposes. The determined spreading angles with the corresponding effective widths are listed in Table 13.

<table>
<thead>
<tr>
<th>Load-to-face grain angle (degree)</th>
<th>Simple model</th>
<th>Advanced model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Spreading angle (degree)</td>
<td>Effective width (mm)</td>
</tr>
<tr>
<td>0</td>
<td>10</td>
<td>181</td>
</tr>
<tr>
<td>22.5</td>
<td>14</td>
<td>210</td>
</tr>
<tr>
<td>45</td>
<td>18</td>
<td>240</td>
</tr>
</tbody>
</table>

It is noticed in Table 13 that, the simple model results in both smaller spreading angles and effective widths than the advanced model at each load-to-face grain angle. It can also be seen from Figure 36 that the simple analytical prediction reaches a plateau prior to the experimental and advanced ones.

Although the spreading angles determined from the simple and advanced models are different, both increase gradually when the load-to-face grain angle changes from 0° to 45°.

The higher spreading angle at 45° compared to 22.5° and 0° has been explained in Paper VI. To summarize the explanation, when the glulam element is subjected to a tensile force, both the shear stresses along the sides of the bonded region and the tensile stresses within the width of
glulam (\(b_0\)) are activated in resisting this tensile force. Given the ratio of the shear strength to the tensile strength of birch plywood at 45° is much higher than that at 0°, it is reasonable to derive a higher spreading angle and observe more plywood resisting the external tensile force at 45°.

It is worth noting that the shear stresses could be considered equivalent to the tensile stresses in the outer region (\(w_{eff} - b_0\)) of the gusset plate. However, due to the eccentricity between the shear stress and tensile stress in the outer region, a bending moment is activated along the symmetry line and the length of the bonded area in the outer region of plywood. Hence, the shear capacity is not fully activated owing to the existence of the normal stresses in the direction perpendicular to the loading axis. See Figure 37 for better understanding.

![Figure 37: A discussion on the contribution from the shear stresses.](image)

### 3.5.3. Stress distributions

The tensile stress distributions obtained from the simple and advanced spreading angle models, and from the numerical models are compared in Figure 38.
Figure 38: Tensile stress distribution in birch plywood in adhesively bonded connections predicted by the simple and advanced spreading angle models and the numerical models.

The numerical stress distributions are, in general, more similar to the stress distributions derived from the advanced analytical model. However,
the numerical stress distributions tend to show peak value at the center of plywood and are thus not uniform within the width of glulam. Moreover, the numerical results at 45° indicate that both edges of the plywood plate are subjected to compressive stresses. This finding is believed to be reasonable as the tensile force from the glulam elements introduces a bending moment to the plywood plate in the region outside the bonded area along the symmetry line (see Figure 37). The compressive stress is not observed at 0° and is very limited at 22.5°, which might be because the investigated maximum widths at 0° and 22.5° are smaller than the investigated maximum width at 45°. However, the compressive stress at the plywood edge is not considered in the analytical models.
4. On-going studies

This chapter presents some preliminary results from the on-going studies which are not reported in the appended papers.

4.1. An innovative “wheel-geared” connection system

As aforementioned in Section 1.3, one of the limitations of the adhesively bonded joint is its difficulty to be assembled on-site. The gluing on-site technique, in particular between timber elements and birch plywood plates, requires more research to ensure adequate bonding strength, and is thus not recommended currently. One potential proposal is to adhesively connect as many timber components as possible in the factory and mechanically connect these prefabricated modules on-site, providing a highly prefabricated solution with shortened assembly time on-site.

Another connection system, namely, “wheel-geared” connection, is proposed here. This connection system enables not only the glue application in the factory but also an on-site adhesive-free and metal-free assembly. Figure 39 illustrates one of the possibilities using this “wheel-geared” connection system in timber frames.

The plywood plate can be cut, e.g., by water cut, into several pieces with a configuration similar to the one shown in Figure 39a. Then, two plywood plates are glued to each glulam element on both surfaces in the factory with a controlled indoor environment (Figure 39b). After curing, these prefabricated elements with the plywood plates adhesively attached are transported to the construction site (Figure 39c) and connected by the rest of the plywood pieces cut from the same plywood panel (Figure 39d).
4.1.1. Testing of the “wheel-geared” connection

It is of great interest to investigate the load-bearing capacity and the failure mode of this connection. One specimen with the same geometrical configuration as the one shown in Figure 39 was manufactured. The birch plywood utilized in the specimen has a nominal thickness of 21 mm. The glulam element has a cross-section of 90 mm × 360 mm. Each notch has a width and depth of 20 mm. A few screws were inserted in the plywood strips to keep them on the glulam elements. The frame corner specimen with a length of 1.4 m was laterally braced and then loaded in compression. The specimen was thus subjected to a combined bending moment, normal force, and shear force. See other detailed information regarding the test specimen and the test setup in Figure 40.

Figure 39: An innovative “wheel-geared” timber connection system.
Figure 40: (a) geometrical configuration of the tested specimen; (b) specimen before the test; and (c) specimen after the test with the failure mode highlighted (unit: mm).
In this “wheel-geared” connection system, bending moment and the forces can be transmitted via the notches to the plywood strips. The test results show that the maximum compressive piston load is approximately 41 kN. The specimen failed in the outer plywood strip in tension; therefore, the tested connection system has not utilized the full strength of the glulam elements.

In order to gain knowledge of the utilization ratio, i.e., the ratio between the capacity of the “wheel-geared” connection system and the capacity of the glulam element, it is worth reporting another on-going test result where the glulam element failed in this frame corner. See Figure 41 for the specimen that failed in the glulam.

As shown in Figure 41, the glulam elements were mechanically connected by the birch plywood plates and screws. The glulam element failed with the maximum piston load of around 104 kN. Consequently, the tested “wheel-geared” connection system utilized roughly 40% of the glulam strength, which might be satisfactory for applications where bending moment in the frame corner is not too big. Nevertheless, further studies are needed to improve the load-bearing capacity by e.g., changing the geometry of the notches or replacing some pieces of plywood with steel.
4.2. Load-bearing capacity of birch plywood in adhesively bonded frame corners

When the birch plywood is designed as a gusset plate in a frame corner, it is mainly subjected to bending moment. Apart from the dominant bending moment, it would also take the normal force and shear force from the column and rafter, leading to a complex stress distribution. Analytical calculation models should be proposed and validated.

Accordingly, one specimen was constructed with the whole plywood plate fully bonded to the glulam elements. Failure in the birch plywood was expected. The glulam elements have the same dimension as the ones in Figure 40. See the specimen in Figure 42.

![Figure 42: Laboratory test of a fully bonded frame corner: (a) before test and (b) after test.](image)

Test results show that the maximum compressive piston load is around 79 kN with failure in the plywood.

A quadratic analytical calculation model is proposed to predict the load-bearing capacity of plywood plate in frame corner:

\[
\left(\frac{F_N}{R_N}\right)^2 + \left(\frac{M}{R_M}\right)^2 + \left(\frac{V}{R_V}\right)^2 = 1, \quad (30)
\]
where $F_N$, $M$, and $V$ are the imposed normal force, bending moment, and shear force, respectively; $R_N$, $R_M$, and $R_V$ are the normal, moment, and shear capacity of birch plywood, respectively. It is evident that the prediction is dependent on the input strength values. The influence of the edgewise bending strength is dominant here. The size effect of the edgewise bending strength of birch plywood at $0^\circ$ is revealed in Section 3.2.2. It was found that the bending strength decreases from 69.6 MPa to 65.1 MPa when the depth increases from 20 mm to 50 mm. However, how the bending strength changes when the depth is over 50 mm is still uncertain. More size effect studies are needed in the future. In this frame corner test, the depth of birch plywood is 360 mm. It is noticed that, when assuming the input bending strength to be 55 MPa, the load-bearing capacity predicted by Eq. (30) is around 80 kN, similar to the test results.

4.3. Fully bonded connections versus mechanical connections in full-scale testing

There are mainly two possibilities to connect the timber elements and the birch plywood plates: (1) bonded by adhesives and (2) mechanically connected by fasteners such as dowels and screws. Although this thesis has a main focus on the adhesively bonded joints, it is of significance to compare the global structural behaviors by using these two different connection systems.

Experimental studies were thus conducted on two full-scale timber frames (one adhesively bonded and the other screw-connected). Tests were performed by using a hydraulic jack pulling a steel rod. The steel rod would then give a reaction force to the end of the glulam column and rafter. See the geometrical configuration in Figure 43b. Two sensors of the total six served to measure the displacement between the column and the rafter in the direction along the steel rod while the load was measured by a load cell.
Figure 43: (a) Portal frame connected by birch plywood, (b) half of the frame tested in full-scale (unit of dimensions: mm), (c) test preparation, (d) failure mode of the adhesively bonded frame, and (e) failure mode of the screw-connected frame.
The failure modes of the adhesively bonded and screw-connected frames are presented in Figure 43d and Figure 43e, respectively. The adhesively bonded frame failed in the glulam column. The cracks (highlighted in pink) were initiated from the connection region with high bending moment. Later, more cracks propagated to the end of the column along the grain direction. The failure mode indicates that the bond line and the birch plywood plates can be stronger than the glulam elements once properly designed. The screw-connected one failed in both column and rafter. The cracks were along the screws and more visible in the rafter.

The imposed load from the hydraulic jack in relation to the displacement between the column and the rafter is presented in Figure 44. Although both frames failed in the glulam elements, the adhesively bonded frame is 27% stronger than the mechanically connected frame, which might be due to that the glulam in the adhesively bonded frame was not weakened by the drilled holes and the tensile stress perpendicular to the grain around the mechanical fasteners. Moreover, by analyzing the slope of the curve between 10% and 40% of the maximum load, it can also be noticed that the adhesively bonded frame is 33% stiffer than the mechanical one.

Figure 44: A comparison of the load-displacement relationship between the adhesively and mechanically connected timber frames.
5. Conclusions and future work

The conclusions drawn in this thesis are subdivided into three sections summarized in Section 5.1-5.3 separately:

- In-plane mechanical properties of birch plywood (Papers I and II)
- Adhesive bonding performance between birch plywood and spruce glulam (Papers III-V)
- Tensile capacity of birch plywood in adhesively bonded connection (Paper VI)

The future work is suggested in Section 5.4.

5.1. In-plane mechanical properties of birch plywood

In this thesis, the in-plane mechanical properties of birch plywood at varying angles to the face grain were characterized experimentally and successfully predicted by analytical and numerical models.

The experimental results indicate that birch plywood possesses satisfying mechanical properties in all in-plane directions, thereby promising for structural uses. To be more specific:

- the lowest tensile and compressive strengths are achieved at 45° and they are both approximately 20 MPa;
- the lowest edgewise bending strength is achieved at 45° and it is approximately 30 MPa; and
- the lowest panel shear strength is achieved at 0° and 90° and it is approximately 12 MPa.

All these strength values are similar to or higher than the highest strengths of other commonly used strength graded softwood timber when loaded in the strongest direction, i.e., parallel to the grain.

Regarding the prediction models, the empirical Norris failure criterion is recommended for the tensile and compressive strength predictions while
the proposed bilinear model is the best for shear strength predictions. The edgewise bending strength prediction relies on the tensile and compressive stress-strain relationships as input and is highly dependent on the failure definitions in both analytical and numerical models. The analytical models that show the highest closeness to the tested elastic and shear modulus have also been put forward.

The influence of size and moisture content on the edgewise bending properties was investigated. A size effect is observable at 0° and 90° but not at other angles in edgewise bending, owing to the different failure mechanisms. Linear functions generalize the moisture effect fairly well. Clear relationships between edgewise bending strength and elastic modulus imply the feasibility to derive the angle-dependent bending strength by detecting its elastic modulus non-destructively.

5.2. Adhesive bonding performance

When developing the connection system using birch plywood as gusset plates in timber structures, particular interests are on the adhesively bonded connections where birch plywood was adhesively connected to other timber elements, e.g., spruce glulam. The bonding technique that resulted in adequate bonding strength was investigated. Single lap tensile shear tests have been conducted to evaluate the influence of some key factors on the bonding strength, i.e., the adhesive types, pressing methods, the moisture conditions, and the load-to-plywood face grain angles.

It was found that specimens bonded with two-component polyurethane (2C PUR) exhibit the highest bonding strength, over 5 MPa, while the ones with melamine-urea-formaldehyde (MUF) and phenol-resorcinol-formaldehyde (PRF), possess slightly lower but similar bonding strength properties (4-5 MPa), which might be due to the lower shear modulus of PUR adhesives which in turn results in less severe stress concentration in these specimens. The manual pressing methods investigated in this thesis show no significant influence on the bonding strength. Moreover, the bonding strengths in the moist condition (MC: around 18%) only decrease 6.4%, 4.4%, and 6.2% for the MUF, PRF, and 2C PUR specimens,
respectively, compared to the bonding strengths in the dry condition (MC: 10-12%). Satisfactory bonding performance in the moist condition also implies the effectiveness of the adhesives to be applied in service class 2 in EN 1995-1-1 [51]. In addition, when varying the load-to-plywood face grain angles from 0° to 90°, the bonding strength changes within a relatively small range, from 4.6 MPa to 6.5 MPa. The weak angle-dependence of the bonding strength is beneficial for the design and the development of the adhesively bonded joints when utilizing birch plywood as gusset plates. A clear correlation exists between the bonding strength and the shear strength of the weakest wood adherend.

The moment-resisting bonding performance has been studied. The bonded region was designed as the weakest part in the beam-to-beam connection. Numerical models were developed to predict the structural behaviors in the linear-elastic stage, while analytical models were proposed to predict the moment-carrying capacities. Both numerical and analytical models display satisfactory agreement with the test results.

5.3. Tensile capacity of birch plywood in adhesively bonded connection

In truss structures, the plywood plates are usually wider than the glulam elements. The contribution of the plywood additional width on its load-bearing capacity should be quantified and formulated for better design. Specifically, the plywood width was increased step by step at three different load-to-face grain angles, 0°, 22.5°, and 45°, until the tensile capacity of plywood reached a plateau. It was found that the tensile strengths of birch plywood within the bonded area show very low angle-dependence. This is possibly due to the restricted crack paths at 22.5° and 45° when the gap between the bonded regions is small, further resulting in different tensile strengths within and outside of the bonded area. When the tensile capacities approach the maximum, the tensile capacity at 0° is only slightly higher than the other two at 22.5° and 45° but the tensile capacity at different load-to-face grain angles reaches a plateau at different widths,
which can be well predicted and explained by the proposed spreading angle models.

### 5.4. Future work

Knowledge obtained from the thesis work is crucial for enabling the design of birch plywood plates in timber connections. However, not all crucial aspects have been addressed.

It should be noted that, the mechanical properties of birch plywood and the bonding strength were tested from relatively small specimens. When adopting these data for the structural design, size effect should not be ignored, in particular when it comes to the tensile, bending, and bonding strengths. It is worth mentioning that the size effect of the bonding strength may also depend on the shear stiffness of the adhesives, the thickness of the wood adherends, etc.

Moreover, the on-going studies presented in the thesis need further research work. The load-bearing capacity of the proposed “wheel-geared” connection system needs to be improved. The design formulas for the load-bearing capacities of birch plywood and glulam element in truss and frame should also be further validated by conducting more full-scale tests in the future.
References


[48] H. Yoshihara, Influence of the specimen depth to length ratio and lamination construction on Young’s modulus and in-plane shear modulus of plywood measured by flexural vibration, BioResources 7(1) (2012) 1337-51.


