A New Bridge Proposal – Road Bridge with a Cross-Laminated Timber Slab

Hannes Behrens and Per Benner

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A New Bridge Proposal  –  Road Bridge with a Cross-Laminated Timber Slab

Ett nytt broförslag – vägbro med en brobana av korslimmat massivträ

Hannes Behrens och Per Benner

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Abstract

In this thesis, a new bridge type is proposed. It is a steel bridge with a slab of cross-laminated timber elements. The bridge is intended for road traffic, and it has a span length of 26 m. It aims to be competitive when a short construction time is important. Long construction time could be a critical issue when launching a bridge over a railway where disruption of traffic flow is associated with high costs.

The design of the new bridge type was based on an existing steel-concrete composite bridge built in Ulricehamn in Sweden. The design was limited to the main structural elements of its superstructure. The biggest difference to the reference bridge is that the new bridge type do not have composite action between its girders and slab. Since steel and timber have very different moduli of elasticities, composite action is not believed to be effective.

For the design of the cross-laminated timber slab, rolling shear in cross-layers due to large concentrated loads was decisive. Whereas for the design of the steel structure, deflections in the serviceability limit state was governing. The cross-laminated timber slab did not provide sufficient torsional stiffness to the main girders. Thus, the deflections of the bridge became too large. Therefore, planar and cross-bracing spaced every 3.25 m were required. The bracing created a virtual box-section with large torsional stiffness.

The construction cost of the superstructure was estimated for the proposed bridge and the reference bridge. The cost of the cross-laminated timber slab and the concrete slab were comparable. However, the steel structure of the new bridge type was more expensive. Therefore, the cost of the superstructure of the new bridge type was higher.

To construct the proposed bridge, it was estimated that it would take two weeks to launch the main girders and to mount the cross-laminated timber elements. Furthermore, it could take up to three months to erect the superstructure of the reference bridge. Consequently, the higher cost of the superstructure of the new bridge type could be compensated by the fact that it is faster to build. Therefore, it may be competitive for projects where short construction time is necessary.

Keywords: Cross-laminated timber, CLT, bridge design, short construction time, steel-concrete composite bridge, steel bridge, timber bridge
Sammanfattning

I det här examensarbetet föreslås en ny brotyp. Det är en 26 m lång vägbro av stål med en broplatte av korslimmat massivträ. Tanken är att bron skall vara ett konkurrenskraftigt alternativ när kort lanseringstid önskas. Kortare lanseringstid kan exempelvis vara en prioritet vid byggen över järnvägar där trafikstörningar är kostsamma.

Utformningen av den nya brotypen begränsades till brobanan och baserades på en befintlig kompositbro i stål och betong som bygglats i Ulricehamn. Den största skillnaden mot referensbron är att den nya brotypens huvudbalkar och broplatte inte samverkar. Anledningen är att elasticitetsmodulerna för stål och trä skiljer sig åt för mycket för att kompositverkan ska vara effektivt.

Rullskjuvning i tvärarter på grund av stora koncentrerade laster var dimensionerande för den korslimmade massivträplattan medan nedböjningar var begränsande för stålkonstruktionen. Eftersom den korslimmade massivträplattan inte gav tillräcklig vridstyvhet till huvudbalkarna blev deformationerna för stora. För att minska deformationerna krävdes tvärskott och planstabilisering som tillsammans med huvudbalkarna skapade ett slutet tvärsnitt med hög vridstyvhet.


Det uppskattades att det skulle ta två veckor att lansera den nya brotypens stålkonstruktion och montera massivträelementen medan det skulle ta nästan tre månader att bygga referensbron. Följaktligen torde den höga kostnaden för den nya brotypens brobana kunna kompenseras av att den går snabbt att bygga. Slutsatsen är att den nya brotypen kan vara lämplig för projekt där kort lanseringstid är avgörande.

Nyckelord: Korslimmat massivträ, brobyggnad, kort byggtid, kompositbro, stålbro, träbro
Preface

We would like to thank our supervisor, Prof. Roberto Crocetti, who sparked the idea of the new bridge type and supported us throughout the project. We would also like to thank Ph.D Henrik Danielson, who patiently guided us through the jungle of cross-laminated timber design methods. Furthermore, we would like to thank Assistant Prof. Eva Frühwald for her constructive feedback.

Ola Bengtsson at Centerlöf & Holmberg provided us with the reference bridge and answered our many questions on bridge design. For this, we are very grateful. We also want to thank Oskar Bruneby at Peab, who helped us with the cost estimate and gave us an insight in how bridges are built. Moreover, we would like to thank Dlubal who thoroughly answered all of our questions on finite element modelling of cross-laminated timber with RFEM.

A warm thanks to KLH for their invaluable help with the design of the cross-laminated timber slab. Especially, we would like to thank Johannes Habenbacher for all the help and for the great tour of KLH’s production facilities close to Graz and the many timber bridges in the area. We would also like to thank Dipl. Engineer Andreas Ringhofer who gave us a great display of the cutting-edge research on cross-laminated timber that is carried out at the Technical University of Graz. The study trip to Austria would not have been possible without the funding from Brosamverkan.

Finally, we are thankful to all of our friends and family that supported us. We are happy that you never gave up on us though we barely left the university during the long final weeks of the project.

Hannes Behrens & Per Benner, June 2015
Symbols

Roman Uppercase Letters

\(A\) ............... Cross-sectional area
\(A_{\text{eff}}\) ............... Effective cross-sectional area
\(A_{\text{net}}\) ............... Net cross-sectional area
\(C_i\) ............... Cross-sectional layer \(i\)
\(D\) ............... Stiffness matrix
\(D_{ij}\) ............... Element included in stiffness matrix
\(E\) ............... Modulus of elasticity
\(G\) ............... Shear modulus
\(G_k\) ............... Characteristic value of permanent action
\(I\) ............... Moment of inertia
\(I_{\text{net}}\) ............... Net moment of inertia
\(I_t\) ............... St. Venants torsion constant
\(I_w\) ............... Warping constant
\(L\) ............... Length
\(L_i\) ............... Longitudinal layer \(i\)
\(M\) ............... Bending moment
\(M_{cr}\) ............... Critical moment
\(N\) ............... Normal force
\(Q\) ............... Point load
\(Q_k\) ............... Characteristic value of a single variable action
\(S\) ............... First moment of area
\(S_{\text{net}}\) ............... Net first moment of area
\(V\) ............... Shear force
\(W_{\text{el}}\) ............... Elastic section modulus

Roman Lowercase Letters

\(b\) ............... Width
\(h\) ............... Height
\(t\) ............... Thickness
\(f_c\) ............... Compressive strength
\(f_m\) ............... Bending strength
\(f_v\) ............... Shear strength
\(f_y\) ............... Yield strength
\(k_{\text{def}}\) ............... Deformation factor
\(k_{\text{mod}}\) ............... Modification factor
\(k_{\text{sys}}\) ............... System strength factor
\(q\) ............... Distributed load
\(u, w\) ............... Deflection
**Greek Letters**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha$</td>
<td>Coefficient of thermal expansion</td>
</tr>
<tr>
<td>$\gamma_d$</td>
<td>Safety partial factor</td>
</tr>
<tr>
<td>$\gamma_M$</td>
<td>Partial factor with respect to material properties</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>Slenderness factor</td>
</tr>
<tr>
<td>$\chi$</td>
<td>Reduction factor considering buckling of a compressed element</td>
</tr>
<tr>
<td>$\chi_w$</td>
<td>Reduction factor considering shear buckling of web</td>
</tr>
<tr>
<td>$\sigma_c$</td>
<td>Compressive stress</td>
</tr>
<tr>
<td>$\sigma_m$</td>
<td>Bending stress</td>
</tr>
<tr>
<td>$\tau$</td>
<td>Shear stress</td>
</tr>
<tr>
<td>$\psi_1$</td>
<td>Factor for frequent value of a variable action</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Poisson’s ratio</td>
</tr>
</tbody>
</table>
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1 Introduction

The aim of this thesis is to develop a new type of bridge and to evaluate if it is an economically viable design. The bridge consists of a deck of cross-laminated timber elements supported on steel girders and is intended to be a cost-effective alternative when it is important with a short erection time.

1.1 Background

The idea to design a bridge with a cross-laminated timber (CLT) deck originates from the need of bridges with short construction time. This could be a key issue where site conditions are difficult, e.g. when launching a bridge over a stream or working at large heights. Another example of when a short construction time could be favourable is when a new bridge is launched over a railway where disruption of traffic flow is associated with high costs (Isaksen, 2005).

Currently, these conditions are often addressed by using a beam bridge with steel girders acting in composite with an in-situ cast concrete deck. The compressive strength of the concrete slab combined with the high tensile strength of the steel girders composes a composite cross-section with favourable material properties (El Sarraf et al., 2013). During the service life of the bridge, the concrete slab stabilizes the steel girders against instability phenomena such as lateral torsional buckling and local buckling of the upper flange. However, the stabilizing properties of the concrete deck develop as the concrete hardens. Thus, the steel girders are vulnerable to buckling during the casting and hardening processes. The risk for instability failure during the construction phase can be decisive for the structural design of the steel girders, and it can be required to use temporary bracing (Lebet and Hirt, 2013). Furthermore, the falsework and casting procedure is both labour intensive and time consuming which could lead to considerable costs.

For the new bridge type, the in-situ cast concrete slab is exchanged for a prefabricated cross-laminated timber slab. Currently, CLT elements have only been used scarcely for bridges. However, CLT elements are interesting to use as bridge slabs due to the possibility to produce large prefabricated elements. If the structural elements of new bridge type are prefabricated to a great extent, a significant reduction of construction time could be possible. KLH in Austria is one of the largest manufacturers of CLT products in the world, and they provide comprehensive information for structural design. Therefore, the design of the CLT slab of the new bridge type is based on the specifications in KLH’s production.

Due to that steel and wood have different thermal expansion coefficients, the CLT elements are not supposed to work in composite action with the steel girders. Composite action is not thought to increase the bridge’s moment capacity enough to compensate for the risk of cracks in the wood as well as the extra work with shear connectors. Furthermore, it can be shown that a considerable portion of the moment capacity of a steel-concrete composite cross-section is consumed by the moment that is caused by the large self-weight of the concrete. Since the weight of wood is only a fifth of that of concrete, it is believed that a non-composite bridge with a CLT deck would not require much larger main girders.
than a steel-concrete composite bridge.

As mentioned earlier, the compressed flange of the steel girders of a composite bridge is prone to buckling before the concrete has hardened. A lighter bridge deck would not subject the girders to such high loads. Thus, the risk of local buckling of the compressed flange during construction is decreased. Another advantage of a light bridge deck is that the load on the foundation is reduced. Therefore, smaller and cheaper abutments should be required. Also, since the falsework and casting process is eliminated a shorter overall construction time of the bridge is facilitated. The aforementioned properties of the new bridge type are assumed to make it a viable alternative for road bridges when a short construction time is necessary.

1.2 Aim and Scope

The aim of the thesis is to develop a new type of road bridge, consisting of a deck of cross-laminated timber elements with underlying steel girders, and to evaluate if it is an economically viable design. The bridge is intended to be a cost-effective alternative when a short bridge erection time is important. Furthermore, the study wants to contribute to broadening the areas of application for cross-laminated timber products by introducing it as a feasible lightweight decking material for road bridges.

The scope of the literature study of cross-laminated timber is the application for slabs, the design specifications stated in KLH’s ETA-06/0138 (2012) and the most commonly known design methods. Moreover, the scope of the design of the proposed bridge type is the structural behaviour of the main structural elements of its superstructure. The durability of the bridge and the design of details and connection are not within the scope of this thesis. Finally, the scope of the cost estimate of the bridge is the construction cost of the main structural elements of the bridge’s superstructure.

1.3 Hypothesis

A single span beam bridge consisting of a deck of cross-laminated timber elements with underlying steel girders is an economically viable design for short span road bridges when short construction time is necessary.

1.4 Method

There are several different design methods available for cross-laminated timber elements. Therefore, the most widely spread methods will be presented. Thereafter, the method that was found to be most suitable will be used for the design of the new bridge type.

To facilitate the evaluation of the new bridge type, a suitable existing road bridge, constructed with an in-situ cast concrete deck and steel girders in composite action, is used as reference. The new bridge type will be designed according to the span length, traffic volume and width of the reference bridge. The new bridge type will be considered feasible
if its cost of material and construction is lower or equal to that of the in-situ cast concrete alternative. The reference bridge is selected with the help of an experienced bridge engineer.

The design and evaluation of the new bridge type are carried out according to the following steps:

- Preliminary design according to the specification of the reference bridge.
- Structural analysis and design of the CLT slab according to a suitable design method. The analysis aims at resulting in a suitable cross-section.
- Design of the bridge according to ultimate limit state and serviceability limit state requirements in Eurocode. The scope of the design is the main components of the bridge, i.e. the main girders, the CLT deck and if necessary bracing.
- Estimation of the cost of material and construction of the new bridge type.
- Economic comparison between the new bridge type and the reference bridge.

1.5 Limitations

The design of the bridge is limited to its main components, i.e. the CLT deck, the main girders and if necessary its bracing system. Abutments and details are not covered within the scope of this thesis. However, some general concerns and suggestions regarding the connection between the CLT elements and their connection to the steel girders will be addressed briefly.

The design of the cross-laminated timber deck will be based on the specifications given in the European technical approval (ETA) of KLH’s CLT products. Furthermore, the design of the deck cross-section is limited by the maximum thickness of 500 mm allowed in KLH’s production.

The design of the main girders is limited to traditional welded I-girders. Climate-induced actions on the CLT elements, such as creep and shrinkage, will not be investigated. The main focus of the study is on the structural behaviour of the new bridge type, and not its building physical properties.

The cost estimate of the new bridge type is based on the initial cost of its superstructure. Life cycle costs, such as maintenance, and demolishing and recycling of the bridge, will not be investigated.

1.6 Outline of the Report

The thesis begins with a brief introduction to cross-laminated timber in chapter 2. Furthermore, the chapter includes a presentation of the different design methods that are available for the product. Thereafter, some aspects of bridge design are addressed in chapter 3. To get a better understanding of the reference bridge, the chapter begins with
a presentation of the main features of a steel-concrete composite bridge. Afterwards, some concerns regarding timber bridges and cross-laminated timber bridges are raised. Finally, the chapter ends with a short presentation of the economical facets of a bridge construction project, i.e. the cost of its main components, the costs of different bridge types and the different methods that can be used to compare the cost of bridges.

The reference bridge and the new bridge type are presented in chapter 4 and 5 respectively. Moreover, the characteristic actions according to Eurocode that were governing for the design are shown in chapter 6. To facilitate the analysis of the bridge, a finite element model was made. The modelling procedure is presented in chapter 7. Furthermore, the ultimate limit state and the serviceability limit state verifications of the new bridge type are given in chapter 8 and 9 respectively. After that, the cost estimate and the cost comparison of the new bridge type and the reference bridge are shown in chapter 10.

The behaviour and the feasibility of the new bridge are discussed in chapter 11. Thereafter the main conclusions of the projects are stated in chapter 12. Finally, some suggestions for further research are given in chapter 13.
2 Cross-Laminated Timber

2.1 Brief Introduction

The first research related to cross-laminated timber was made in Switzerland in the 1990s. Further development was mainly done in Austria, and the product has gained popularity in recent years (WoodSolutions, 2014). Cross-laminated timber is a versatile building material, wherefore it is used in several different types of structural systems. Elements might be load-bearing or non-load-bearing, and the product can be found in ordinary apartments, industrial buildings and bridges. Besides good structural properties, prefabricated elements often reduce the erection time at building sites (KLH Massivholz, 2013). Moreover, the sustainable characteristics of timber make it a competitive alternative to concrete. Low weight eases transportation, and with correct treatment, the durability of the material can be increased (Mettem, 2011).

Cross-laminated timber consists of several, at least three, board layers that form a solid wood panel. As the product name implies, adjacent layers may be placed in orthogonal directions with respect to the individual fibre direction of the boards. As long as the production criterion is fulfilled, desired element properties are obtained by adjusting the amount of layers and their arrangement.

In Europe, spruce is the most common board material. In order to get the desired length of boards, they are finger-jointed according to Figure 2.1. A solid slab element is then assembled by stacking boards on top and next to each other. The boards are in some cases, depending on the manufacturer, glued edgewise and different board layers are bonded together face-to-face using adhesives. By adding pressure to the stacked layers, often by using a hydraulic press device, they are formed into one solid element. After that, precise CNC machines are used to cut the part into its final shape (WoodSolutions, 2014). Figure 2.3 illustrates how a three-layered cross-laminated timber element might look like.

Because the product is relatively new, there is not any standardised design approach in Eurocode. Therefore, designers may be referred to technical approvals related to the particular product. In countries part of the European Union, technical approvals for construction products are issued by members of the European Organisation for Technical Assessment, EOTA. The definition of a European Technical Assessment is formulated in the Regulation (EU) No 305/2011 of the European Parliament and of the Council of 9 March 2011 as follows:
"European Technical Assessment means the documented assessment of the performance of a construction product, in relation to its essential characteristics, in accordance with the respective European Assessment Document."

2.2 Rolling Shear

The shear strength and shear modulus perpendicular to the boards are of particular importance when designing cross-laminated timber. Fellmoser and Blaß (2004, p. 1) define rolling shear as "shear stress leading to shear strains in a plane perpendicular to the grain direction". Therefore, the low shear strength might be the cause of considerable shear deformations in an element when the element is being subjected to loads perpendicular to its plane. Since shear deformations should be taken into account due to shear strains, the Euler-Bernoulli beam theory should be avoided when designing cross-laminated timber elements with small span-to-depth ratios (Ottosen and Petersson, 1992). Figure 2.2 illustrates the structural behaviour of an element with respect to rolling shear. Part A of the figure shows an exaggerated deformation of a part of a beam due to shear stresses, while B illustrates the failure within a cross-layer of the beam being equally exposed.

Figure 2.2: Beam subjected to an external load, P
2.3 Element Restrictions According to KLH

The width and thickness of a single board are limited to 44 - 290 mm and 10 - 45 mm respectively. Furthermore, the recommended slab thickness is limited to 57 - 300 mm while the width and length should not exceed 2.98 and 16.5 m respectively. The number of board layers in a solid element should not be less than 3 and no more than 16. The geometry specifications for a single board and a solid slab according to ETA-06/0138 (2012) are shown in Table 2.1.

<table>
<thead>
<tr>
<th></th>
<th>Single board</th>
<th>Solid slab</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness</td>
<td>10 - 45 mm</td>
<td>57 - 300 mm</td>
</tr>
<tr>
<td>Width</td>
<td>44 - 298 mm</td>
<td>≤ 2.98 m</td>
</tr>
<tr>
<td>Length</td>
<td>-</td>
<td>≤ 16.5 m</td>
</tr>
</tbody>
</table>

The length and width of slab elements are restricted due to geometrical limitations of the CNC machines. However, the thickness of an element might be increased by turning it around during the cutting procedure. Since two-way cutting requires an extra step throughout the production, thicker elements are more expensive to manufacture compared to standard elements. KLH produces elements with thicknesses up to 500 mm. Finally, the standard production allows widths of 2.40, 2.50, 2.72 and 2.95 m (KLH Massivholz, 2013).

Desired element properties, such as load-bearing capacity and stiffness, are obtained by choosing a suitable element design. This is done by selecting an appropriate cross-section that fulfils the structural requirements. Since the production of CLT allows different layer arrangements, there are a wide range of possible cross-sections. The outermost layers of a load-bearing element usually have their fibres running in the same direction as the main load-bearing direction of the three-layered element in Figure 2.3.

According to ETA-06/0138 (2012), the quality of boards used in cross-laminated timber elements are in most cases graded C24. Because of local discrepancies of strength quality among the boards, reduction factors have been used when evaluating mechanical properties of solid slabs. Furthermore, every single layer should consist of at least 90% timber with a board quality equal to C24. The quality of a possible remaining part should be at least C16.
2.4 Product Characteristics According to ETA-06/0138

Material properties related to actions perpendicular to and in plane of a solid wood slab manufactured by KLH are shown in Tables 2.2 and 2.3.

Table 2.2: Material properties related to actions perpendicular to a solid wood slab (* denotes properties related to rolling shear)

<table>
<thead>
<tr>
<th>Material property</th>
<th>Value related to the material property</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_{0,mean}$</td>
<td>12000 MPa</td>
</tr>
<tr>
<td>$E_{90,mean}$</td>
<td>370 MPa</td>
</tr>
<tr>
<td>$G_{0,mean}$</td>
<td>690 MPa</td>
</tr>
<tr>
<td>$G_{90,mean}$*</td>
<td>50 MPa</td>
</tr>
<tr>
<td>$f_{m,k}$</td>
<td>24 MPa</td>
</tr>
<tr>
<td>$f_{t,90,k}$</td>
<td>0.12 MPa</td>
</tr>
<tr>
<td>$f_{c,90,k}$</td>
<td>2.7 MPa</td>
</tr>
<tr>
<td>$f_{v,k}$</td>
<td>2.7 MPa</td>
</tr>
<tr>
<td>$f_{v,R,k}$*</td>
<td>0.8 - 1.2 MPa</td>
</tr>
</tbody>
</table>

Table 2.3: Material properties related to actions in plane of a solid wood slab

<table>
<thead>
<tr>
<th>Material property</th>
<th>Value related to the material property</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_{0,mean}$</td>
<td>12000 MPa</td>
</tr>
<tr>
<td>$G_{0,mean}$</td>
<td>500 MPa</td>
</tr>
<tr>
<td>$f_{m,k}$</td>
<td>24 MPa</td>
</tr>
<tr>
<td>$f_{t,0,k}$</td>
<td>16.5 MPa</td>
</tr>
<tr>
<td>$f_{c,0,k}$</td>
<td>24 MPa</td>
</tr>
<tr>
<td>$f_{v,k}$</td>
<td>3.9 - 8.4 MPa</td>
</tr>
</tbody>
</table>

As seen in Table 2.2, the characteristic rolling shear strength, i.e. $f_{v,R,k}$, varies. The value depends on the geometric properties of the cross-layers according to Table 2.4. When considering coupled cross-layers, the thickness $t_q$ equals the total height of the joined lamellae (Riebenbauer, 2013).

Table 2.4: Strength values regarding rolling shear (after ETA-06/0138 (2012, Table 4))

<table>
<thead>
<tr>
<th>Thickness of cross-layer(s) [mm]</th>
<th>Width of board/ Thickness of board [-]</th>
<th>Characteristic rolling shear strength [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t_q \leq 45$ mm</td>
<td>$\geq 2.3 : 1$</td>
<td>$f_{v,R,k} = 1.2$</td>
</tr>
<tr>
<td>$t_q &gt; 45$ mm</td>
<td>$\geq 2.3 : 1$</td>
<td>$f_{v,R,k} = 0.8$</td>
</tr>
</tbody>
</table>
Moreover, the characteristic rolling shear strength for layers equal to or thinner than 45 mm should in some cases be reduced from 1.2 MPa. Lower values are chosen for single and multiple spanned slabs that are subjected to concentrated loads close to their midspans. The strength reduction depends on the location of the concentrated load according to Figure 2.4. However, it is only the shear stress caused by the concentrated load that should be verified against the lower rolling shear strength (ETA-06/0138, 2012).

![Figure 2.4: Variation of the rolling shear strength due to the position of a concentrated load (after ETA-06/0138 (2012, p.60))](image)

The recommended partial factor, $\gamma_M$, for the material is 1.25. Furthermore, the modification and deformation factors, $k_{mod}$ and $k_{def}$, of cross-laminated timber correspond to the factors of glue-laminated timber according to Tables 2.5 and 2.6. The modification factor is used when calculating design material strengths and it depends on the service class and load duration. Moreover, the deformation factor is used when determining deformations in serviceability limit state. It should be noted that CLT may only be used for service class 1 or 2 (Riebenbauer, 2013).

**Table 2.5: Modification factors, $k_{mod}$ (SS-EN 1995-1-1 - Design of timber structures part 1-1: General - Common rules and rules for buildings)**

<table>
<thead>
<tr>
<th>Service class</th>
<th>Load-duration class</th>
<th>Permanent action</th>
<th>Long term action</th>
<th>Medium term action</th>
<th>Short term action</th>
<th>Instantaneous action</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>0.50</td>
<td>0.55</td>
<td>0.65</td>
<td>0.70</td>
<td>0.90</td>
<td></td>
</tr>
</tbody>
</table>

**Table 2.6: Deformation factors, $k_{def}$ (SS-EN 1995-1-1 - Design of timber structures part 1-1: General - Common rules and rules for buildings)**

<table>
<thead>
<tr>
<th>Service class</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.60</td>
<td>0.80</td>
<td>2.00</td>
</tr>
</tbody>
</table>
Moreover, the load bearing capacity of an element may be increased with a system strength factor, $k_{sys}$, that depends on the geometry of the element. Table 2.7 shows how to determine its value.

Table 2.7: System strength factors, $k_{sys}$, for cross-laminated timber elements manufactured by KLH (ETA-06/0138, 2012)

<table>
<thead>
<tr>
<th>Loading perpendicular to the solid wood slab</th>
<th>Loading in plane of the solid wood slab</th>
<th>System strength factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Member width (b)</td>
<td>Number of layers (n)</td>
<td>$k_{sys}$</td>
</tr>
<tr>
<td>$b \leq 20$ cm</td>
<td>$n = 1$</td>
<td>0.90</td>
</tr>
<tr>
<td>$20$ cm $&lt; b \leq 100$ cm</td>
<td>$2 \leq n &lt; 5$</td>
<td>1.00</td>
</tr>
<tr>
<td>$100$ cm $&lt; b \leq 160$ cm</td>
<td>$5 \leq n &lt; 8$</td>
<td>1.05</td>
</tr>
<tr>
<td>$b &gt; 160$ cm</td>
<td>$n \geq 8$</td>
<td>1.10</td>
</tr>
</tbody>
</table>
2.5 Design Methods

Different approaches might be used for the design of cross-laminated timber slabs. Thiel (2014) mentions the $\gamma$-method, the shear analogy method and a method based on Timoshenko beam theory. Furthermore, Blaß and Fellmoser (2004) present a theory where effective values are calculated, called the composite method. These methods are described in sections 2.5.1 - 2.5.4. Analyses can additionally be made by using a beam structure program or a finite element software. In this chapter, it is explained how stiffness values and stresses as a result of out-of-plane loads are determined by commonly used methods.

As seen in Figure 2.5, calculated deflections according to Timoshenko theory (TIMO), the shear analogy method (SAV) and the $\gamma$-method (GAMMA) agree well with the exact solution, $w/w_{exact}$, for larger span-to-depth ratios. The figure also shows the inaccuracy associated with Euler-Bernoulli beam theory (EULER BERN.) due to that it neglects shear deformations.

![Figure 2.5: Deflections, $w$, of CLT elements calculated with different methods. $l$ and $t_{CLT}$ denote the lengths and thicknesses of the elements (Thiel, 2014, p. 79)](image)

2.5.1 $\gamma$-method

The $\gamma$-method, or the mechanically jointed beams method, is used when designing beams where the separate parts are joined with mechanical fasteners. In order to design a cross-laminated timber element according to the theory, the method has to be adapted to match new conditions (Gagnon and Pirvu, 2011). The design procedure with respect to mechanical joints is explained in Annex B: Eurocode 5 (SS-EN 1995-1-1 - Design of timber structures part 1-1: General - Common rules and rules for buildings).
Applying the modified theory for cross-laminated timber, a net cross-section is used in order to obtain stresses. Furthermore, an effective cross-section is calculated using $\gamma$-factors for the determination of bending stiffness. In both cases, the modulus of elasticity is assumed to be zero in layers perpendicular to the load-bearing direction of the slab according to Figure 2.6. In order to get a closed form solution, the method requires a sinusoidal load distribution (Wallner-Novak et al., 2013).

![Figure 2.6: Modulus of elasticity associated with different layers of a cross-laminated timber slab. The arrows denote the load-bearing directions of the element.](image)

When calculating bending stresses, the moment of inertia, $I_{0,\text{net}}$, is determined for longitudinal layers only. Due to the equilibrium of forces between layers, this also applies when calculating the first moment of area, $S_{0,\text{net}}$. Because of the theoretical absence of bending stresses in cross-layers, shear stresses are constant in these regions. In case of different moduli of elasticity in longitudinal layers, a reference modulus is determined (Wallner-Novak et al., 2013). Figure 2.7 shows an element exposed to a uniformly distributed load where $L$ denotes longitudinal layers and $C$ cross-layers. Furthermore, $b$ denotes the width of the cross-section, $t_{Li}$ the thickness of longitudinal layer $i$, $t_{Ci}$ the thickness of cross-layer $i$ and $a_{Li}$ the distance between the center of gravity of the longitudinal layers.

![Figure 2.7: Cross-laminated element exposed to a uniformly distributed load](image)
Moreover, Figure 2.8 illustrates stress distribution as a result of bending moments. As seen in the figure, bending stresses in B are nonexistent in cross-layer C1 and C2 where the moduli of elasticities are assumed to be zero. Furthermore, C clarifies why shear stresses are constant over cross-layers. No matter where cross-layer C1 is cut, horizontal force equilibriums will generate the same shear stress values since there are no bending stresses acting on C1. The force equilibrium when considering layer \(ji\) is shown in equation 2.1.

\[
\rightarrow : \int_{t_{ji}} \sigma \cdot dt + \int_{b} \tau \cdot db = \int_{t_{ji}} (\sigma + d\sigma) \cdot dt
\]  

(2.1)

Equations 2.2 - 2.5 show how to determine cross-sectional properties according to Wallner-Novak et al. (2013). Assuming equal moduli of elasticity in layers, the net area of the cross-section is calculated as

\[
A_{0,\text{net}} = \sum_{i=1}^{n} b \cdot t_{Li}
\]  

(2.2)

where \(n\) denotes the number of longitudinal layers, \(b\) the width of the cross-section and \(t_{Li}\) the thickness of longitudinal layer \(i\). Furthermore, corresponding moment of inertia and section modulus for a cross-section are determined by

\[
I_{0,\text{net}} = \sum_{i=1}^{n} \frac{b \cdot t_{Li}^3}{12} + \sum_{i=1}^{n} b \cdot t_{Li} \cdot a_{Li}^2
\]  

(2.3)

\[
W_{0,\text{net}} = \frac{I_{0,\text{net}}}{z_{\text{max}}}
\]  

(2.4)

where \(a_{Li}\) is the distance between the center of gravity for layer \(Li\) and the center of gravity for the entire cross-section. Moreover, \(z_{\text{max}}\) is the maximum distance from the center of gravity of the entire cross-section to top or bottom edge of the outermost longitudinal layer.
Due to the low rolling shear stiffness of cross-laminated timber, it is of interest to calculate the shear stresses that effect cross-layers. Since shear stresses are constant in these areas, it is sufficient to calculate the first moment of area at locations where the grain changes direction.

With notations according to Figure 2.9, the first moment of area at location $x_j$ for a symmetrical cross-section with equal moduli of elasticities of longitudinal layers is determined by

$$S_{0,\text{net}}(x_j) = \sum_{i=1}^{L_n} b \cdot t_{Li} \cdot a_{Li} \quad (2.5)$$

where $L_n$ represents longitudinal layers above $x_j$ where the first moment of area is being calculated and $a_{Li}$ denotes the distance between the center of gravity of the parts.

![Figure 2.9: Locations $x_i$ where grain changes direction](image)

The maximum shear stress occurs in the middle of the cross-section, i.e. between $x_2$ and $x_3$. The corresponding first moment of area is calculated as

$$S_{0,\text{net,max}} = b \cdot t_{L1} \cdot a_{L1} + b \cdot t_{L2} \cdot t_{L2}/2. \quad (2.6)$$

With sectional properties known, bending stresses at a distance $z$ from the neutral layer and shear stresses at location $x_i$ can be calculated according to equations 2.7 and 2.8 (Wallner-Novak et al., 2013).

$$\sigma = \frac{M}{I_{0,\text{net}}} \cdot \frac{z}{b} \quad (2.7)$$

$$\tau = \frac{V \cdot S_{0,\text{net}}}{I_{0,\text{net}}} \cdot \frac{1}{b} \quad (2.8)$$
The corresponding stress distribution for a slab that carries weight in two directions is shown in Figure 2.10.

![Figure 2.10: Stresses acting on a five-layered slab](image)

In order to take shear deformations because of rolling shear into account, the modified $\gamma$-method is used. The new cross-sectional properties are determined by reducing the moment of inertia of the longitudinal layers with $\gamma$-factors. These factors depend on the shear stiffness of the boards. Thus, shear deformations that occur in cross-layers are considered by reducing the bending stiffness of longitudinal layers (Wallner-Novak et al., 2013).

For a three or five layer thick element, the $\gamma$-approach according to Annex B of SS-EN 1995-1-1 - Design of timber structures part 1-1: General - Common rules and rules for buildings is applicable with only a slight modification. By using equations 2.9 - 2.15 and notations according to Figure 2.11, the effective bending stiffness $(EI)_{ef}$ can be calculated for a five-layered element. In the below equations, $A_{Li}$ denotes the area of the specific layer while $l_{ref}$ is a reference length that depends on the boundary conditions of the element. Furthermore, $E_c$ is the so-called reference modulus (Wallner-Novak et al., 2013).

$$\gamma_{L1} = \frac{1}{1 + \pi^2 \cdot \frac{E_{0,L1} \cdot A_{L1}}{l_{ref}^2} \cdot \frac{t_{C1}}{w \cdot G_{90,C1}}}$$  \hspace{1cm} (2.9)

$$\gamma_{L2} = 1,0$$  \hspace{1cm} (2.10)

$$\gamma_{L3} = \frac{1}{1 + \pi^2 \cdot \frac{E_{0,L3} \cdot A_{L3}}{l_{ref}^2} \cdot \frac{t_{C2}}{w \cdot G_{90,C2}}}$$  \hspace{1cm} (2.11)
\[ a_{L2} = \gamma_{L1} \cdot \frac{E_{0,L1}}{E_c} \cdot w \cdot t_{L1} \cdot \left( \frac{t_{L1}}{2} + t_{C1} + t_{L2} \right) - \gamma_{L3} \cdot \frac{E_{0,L3}}{E_c} \cdot w \cdot t_{L3} \cdot \left( \frac{t_{L3}}{2} + t_{C2} + t_{L4} \right) \]

\[ \sum_{i=1}^{3} \gamma_{Li} \cdot \frac{E_{0, Li}}{E_c} \cdot w \cdot t_{Li} \]

\[ (2.12) \]

\[ a_{L1} = \left( \frac{t_{L1}}{2} + t_{C1} + \frac{t_{L2}}{2} \right) - a_{L2} \]

\[ (2.13) \]

\[ a_{L3} = \left( \frac{t_{L2}}{2} + t_{C2} + \frac{t_{L3}}{2} \right) + a_{L2} \]

\[ (2.14) \]

\[ I_{0,ef} = \sum_{i=1}^{3} \frac{E_{0,Li}}{E_c} \cdot \frac{w \cdot t_{Li}^3}{12} + \sum_{i=1}^{3} \gamma_{Li} \cdot \frac{E_{0, Li}}{E_c} \cdot w \cdot t_{Li} \cdot a_{Li}^2 \]

\[ (2.15) \]

With equal moduli of elasticity in the longitudinal layers, \( \frac{E_{0,Li}}{E_c} = 1 \). Finally, the effective bending stiffness of the cross-section can be obtained by

\[ (EI)_{ef} = \sum (E_{0,Li} \cdot I_{Li} + \gamma_{Li} \cdot E_{0, Li} \cdot A_{Li} \cdot a_{Li}^2) \]

\[ (2.16) \]

Figure 2.11: Cross-sectional properties

2.5.2 Composite Theory

When designing cross-laminated timber elements according to the composite theory, effective values concerning strength and stiffness related to compression, tension and bending are calculated. The new properties are determined with the use of composition factors that depend on how the load is acting. With effective stiffness values known, bending stresses can also be determined. The method is based on Euler-Bernoulli beam theory wherefore it does not consider shear deformations.

The composite theory is only applicable to elements with large span-to-depth ratios as a result of neglecting shear deformations. When considering loads acting perpendicular to the element, the span-to-depth ratio should be larger than 30 (Blaß and Fellmoser, 2004).
According to Blaß and Fellmoser (2004), the ratio between the moduli of elasticity parallel and perpendicular to the grain of the boards may be assumed according to 2.17. The composition factors for load cases according to Figure 2.12 are calculated with equations 2.18 and 2.19.

\[
\frac{E_0}{E_{90}} = 30 \quad (2.17)
\]

\[
k_1 = 1 - \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{a^3_{m-2} - a^3_{m-4} + \cdots \pm a^3_1}{a^3_m} \quad (2.18)
\]

\[
k_2 = \frac{E_{90}}{E_0} + \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{a^3_{m-2} - a^3_{m-4} + \cdots \pm a^3_1}{a^3_m} \quad (2.19)
\]

Effective stiffness values of an element with respect to bending perpendicular to its plane are determined with 2.20 - 2.23. $0_{ef}$ and $90_{ef}$ denote whether the orientation of outer grains is parallel with or perpendicular to the bending direction of the element when calculating effective values (Blaß and Fellmoser, 2004).

\[
E_{m,0,ef} = E_0 \cdot k_1 \quad (2.20)
\]

\[
E_{m,90,ef} = E_0 \cdot k_2 \quad (2.21)
\]

\[
(EI)_{0,ef} = E_{m,0,ef} \cdot \frac{b \cdot a^3_m}{12} = E_0 \cdot \frac{b \cdot a^3_m}{12} \cdot k_1 \quad (2.22)
\]

\[
(EI)_{90,ef} = E_{m,90,ef} \cdot \frac{b \cdot a^3_m}{12} = E_0 \cdot \frac{b \cdot a^3_m}{12} \cdot k_2 \quad (2.23)
\]
Finally, the corresponding bending stresses in outermost longitudinal layers are given by

\[
\sigma_{m,0} = \frac{M}{(EI)_{0,ef}} \cdot E_0 \cdot \frac{a_m}{2} \quad (2.24)
\]

\[
\sigma_{m,90} = \frac{M}{(EI)_{90,ef}} \cdot E_0 \cdot \frac{a_{m-2}}{2} \quad (2.25)
\]

### 2.5.3 Shear Analogy Method

The shear analogy method takes shear deformations into account. Therefore, the method can be applied for elements with smaller span-to-depth ratios. When using the theory, the original element is divided into two virtual beams with different mechanical properties, but cross-sections equal to the cross-section of the original one according to Figure 2.13. The two virtual beams, \(A\) and \(B\), are assumed to be infinitely rigid coupled to each other, wherefore their deflections are the same. The bending stiffness of beam \(A\) corresponds to the sum of the individual bending stiffness values of the layers while \(B\) collects the remaining Steiner parts. Moreover, shear deformations are only assumed to occur in beam \(B\) (Gagnon and Pirvu, 2011). Since the virtual beams have different stiffness values and equal deflections, they will experience different moments and shear forces.

**Figure 2.13: Original beam divided into beam A and B**

Bending and shear stiffness related to beam \(A\) are obtained by

\[
(EI)_A = B_A = \sum_{i=1}^{n} E_i \cdot I_i = \sum_{i=1}^{n} E_i \cdot b_i \cdot \frac{h_i^3}{12} \quad (2.26)
\]

\[
(GA)_A = S_A = \infty \quad (2.27)
\]

where \(E_i\) corresponds to \(E_0\) for longitudinal layers and \(E_{90} = E_0/30\) for cross-layers. Furthermore, \(b_i\) and \(h_i\) are the width and height of each specific layer (Gagnon and Pirvu, 2011).
The moments (2.28), shear forces (2.29) and corresponding bending and shear stresses (2.30) and (2.31) that effect the different layers of beam A are calculated as

\[ M_{A,i} = \frac{E_i \cdot I_i}{B_A} \cdot M_A \]  

(2.28)

\[ V_{A,i} = \frac{E_i \cdot I_i}{B_A} \cdot V_A \]  

(2.29)

\[ \sigma_{A,i} = \pm \frac{M_{A,i}}{I_i} \cdot \frac{h_i}{2} \]  

(2.30)

\[ \tau_{A,i} = \frac{E_i \cdot I_i}{B_A} \cdot 1.5 \cdot \frac{V_A}{b \cdot h_i} \]  

(2.31)

where \( I_i \) is the moment of inertia of each individual layer.

Figure 2.14 illustrates how the stress distributions might look like according to 2.30 and 2.31.

According to Gagnon and Pirvu (2011), bending and shear stiffness associated with beam B are obtained by

\[ (EI)_B = B_B = \sum_{i=1}^{n} E_i \cdot A_i \cdot z_i^2 \]  

(2.32)

\[ \frac{1}{(GA)_B} = \frac{1}{S_B} = \frac{1}{a^2} \cdot \left[ \sum_{i=1}^{n-1} \frac{1}{k_i} + \frac{h_1}{2 \cdot G_1 \cdot b_1} + \sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i} + \frac{h_n}{2 \cdot G_n \cdot b_n} \right] \]  

(2.33)

where \( A_i \) is the area of each layer and \( z_i \) the distance between the center of gravity of each individual layer and the neutral axis of the entire element according to Figure 2.13. Moreover, \( G_i \) corresponds to the shear modulus, \( G_0 \), for longitudinal layers and the rolling shear modulus, \( G_R \), for cross-layers. Since no mechanical fasteners are used in glued cross-laminated timber, the slip factor \( k_i \) does not need to be accounted for. Finally, \( h_i \) denotes the height of layer \( i \) and \( a \) is determined by

\[ a = h_{tot} - \frac{h_1}{2} - \frac{h_n}{2} \]  

(2.34)
The normal forces and the corresponding normal stresses acting on the layers of beam $B$ are calculated according to 2.35 and 2.36. Shear stresses acting between the different layers of the beam are determined with 2.37 (Gagnon and Pirvu, 2011).

\[
N_{B,i} = \frac{E_i \cdot A_i \cdot z_i}{B_B} \cdot M_B \quad (2.35)
\]

\[
\sigma_{B,i} = \frac{N_{B,i}}{b_i \cdot h_i} = \frac{E_i \cdot z_i}{B_B} \cdot M_B \quad (2.36)
\]

\[
\tau_{B,i,i+1} = \frac{V_B}{B_B} \cdot \sum_{j=i+1}^{n} E_j \cdot A_j \cdot z_j \quad (2.37)
\]

Figure 2.15 shows how the stress distribution according to equations 2.36 and 2.37 might look like.

![Figure 2.15: Stresses related to beam B (after Gagnon and Pirvu (2011, p.17 of chapter 3))](image)

By superposition of the stresses calculated for beam $A$ and $B$ according to Figure 2.16, the final stress distribution for the element is obtained.

![Figure 2.16: Superposition of stresses acting on beam A and B (after Gagnon and Pirvu (2011, p.17 of chapter 3))](image)

In order to calculate deflections, the effective shear stiffness is determined according to

\[
(GA)_{eff} = \frac{a^2}{\left[ \frac{h_1}{2 \cdot G_1 \cdot b_1} + \sum_{i=2}^{n-1} \frac{h_i}{G_i \cdot b_i} + \frac{h_n}{2 \cdot G_n \cdot b_n} \right]} \quad (2.38)
\]
With effective bending and shear stiffness values known, the deflections can be determined. Equation 2.39 applies to an element being exposed to a uniformly distributed load \( q \), while Equation 2.40 is valid for an element with a concentrated force \( Q \) located at midspan. \( \kappa \) is the shear coefficient form factor.

\[
w_{\text{max}} = \frac{5}{384} \cdot \frac{q \cdot L^4}{(EI)_{\text{eff}}} + \frac{1}{8} \cdot \frac{q \cdot L^2 \cdot \kappa}{(GA)_{\text{eff}}} \tag{2.39}
\]

\[
w_{\text{max}} = \frac{1}{48} \cdot \frac{Q \cdot L^3}{(EI)_{\text{eff}}} + \frac{1}{4} \cdot \frac{Q \cdot L}{(GA)_{\text{eff}}/\kappa} \tag{2.40}
\]

where \( \kappa = 1.2 \) according to Timoshenko beam theory (Gagnon and Pirvu, 2011).

### 2.5.4 Timoshenko Beam Theory

At Graz University of Technology, a software based on the Timoshenko beam theory called CLTdesigner has been developed (Thiel, 2014). Since the theory considers shear deformations, elements with smaller span-to-depth ratios can be designed with the software. The software calculates the bending stiffness as

\[
K_{\text{CLT}} = \sum (E_i \cdot I_i) + \sum (E_i \cdot A_i \cdot e_i^2) \tag{2.41}
\]

where \( E_i \) is \( E_{0,i} \) for longitudinal layers and \( E_{90,i} = 0 \) for cross-layers. Furthermore, \( I_i \) is the moment of inertia and \( A_i \) is the cross-sectional area of layer \( i \). Finally, \( e_i \) is the distance between the center of gravity of layer \( i \) and the center of gravity when considering the entire element (Thiel, 2014).

Moreover, the shear stiffness of an element, \( S_{\text{CLT}} \), is determined by

\[
S_{\text{CLT}} = S_{\text{tot}} \cdot \kappa \tag{2.42}
\]

\[
S_{\text{tot}} = \sum (G_i \cdot b_i \cdot t_i) = \sum (G_i \cdot A_i) \tag{2.43}
\]

where \( G_i \) is the shear modulus of layer \( i \). For cross-layers, the shear modulus equals the rolling shear modulus, \( G_{r,i} \). Moreover, \( b_i \) and \( t_i \) are the width and thickness of layer \( i \) and \( \kappa \) is obtained by

\[
\kappa = \frac{1}{S_{\text{tot}} \cdot K_{\text{CLT}}^2 \cdot \int_{l_{\text{CLT}}} S^2(z, E(z)) \frac{1}{G(z) \cdot b(z)}} \tag{2.44}
\]

where \( b(z) \) is the width of the layer at location \( z \). Furthermore, \( S(z, E(z)) \) and \( G(z) \) are the first moment of area, with regard to the modulus of elasticity, and the shear modulus at \( z \) (Thiel, 2014).
With stiffness values known, bending and shear stresses are calculated as

\[
\sigma(z) = \frac{M}{K_{CLT}} \cdot z \cdot E(z) \tag{2.45}
\]

\[
\tau(z_0) = \frac{V \cdot \int_{A_0} E(z) \cdot z \cdot dA}{K_{CLT} \cdot b(z_0)} \tag{2.46}
\]

where \(z_0\) is the location where the shear stress is being calculated. Moreover, \(b(z_0)\) is the width of the cross-section at \(z_0\) and \(A_0\) is the sheared area. Finally, deflections because of loads perpendicular to the plane of the element are calculated according to the principle of virtual work as

\[
w_{tot} = \frac{1}{K_{CLT}} \int (M \cdot \ddot{M}) \cdot dx + \frac{1}{S_{CLT}} \int (V \cdot \ddot{V}) \cdot dx \tag{2.47}
\]

The maximum deflection for a simply supported beam exposed to a uniformly distributed load \(q\) is determined by (Thiel, 2014)

\[
w_{max} = \frac{5 \cdot q \cdot L^4}{384 \cdot K_{CLT}} + \frac{q \cdot L^2}{8 \cdot S_{CLT}} \tag{2.48}
\]

### 2.5.5 Finite Element Method

The structural behaviour of plates can be described with differential equations. By the use of the finite element method, approximate solutions can be found for these equations. When applying the method, the region of the structure is divided into several smaller elements that are connected through nodal points. Generally the elements are associated with unknown displacements, i.e. degrees of freedom. By considering known boundary conditions associated with the specific problem, numerical approximations can be made over the elements in order to determine unknown displacements as a result of external forces. With displacements, mechanical properties and geometric data of the element known, strains and stresses can be calculated (Ottosen and Petersson, 1992).

When considering cross-laminated timber plates, it is generally of great importance to regard shear deformations. Therefore, the Mindlin-Reissner plate theory is applicable for CLT. As for Timoshenko beam theory, Mindlin-Reissner plate theory accounts for shear deformations (Ottosen and Petersson, 1992).

The mechanical properties and geometrical data of an element can be expressed by a stiffness matrix, \(D\). For an orthotropic plate, the global stiffness matrix is determined by
\[
D = \begin{bmatrix}
D_{11} & D_{12} & 0 & 0 & 0 & D_{16} & D_{17} & 0 \\
D_{21} & D_{22} & 0 & 0 & 0 & D_{26} & D_{27} & 0 \\
0 & 0 & D_{33} & 0 & 0 & 0 & 0 & D_{38} \\
0 & 0 & 0 & D_{44} & D_{45} & 0 & 0 & 0 \\
0 & 0 & 0 & D_{54} & D_{55} & 0 & 0 & 0 \\
D_{61} & D_{62} & 0 & 0 & 0 & D_{66} & D_{67} & 0 \\
D_{71} & D_{72} & 0 & 0 & 0 & D_{76} & D_{77} & 0 \\
0 & 0 & D_{83} & 0 & 0 & 0 & 0 & D_{88}
\end{bmatrix}
\]

(2.49)

where \( D_{12} = D_{21}, D_{16} = D_{61}, D_{27} = D_{72}, D_{45} = D_{54}, D_{38} = D_{83}, D_{67} = D_{76} \) and \( D_{17} = D_{71} = D_{26} = D_{62} \) because of symmetry (Dlubal, 2013).

The elements of the stiffness matrix are linked to various structural properties with respect to different directions as follows:

- **Bending stiffness (EI):**
  \[
  \begin{cases}
  D_{11} \\
  D_{12} \\
  D_{21} \\
  D_{22}
  \end{cases}
  \]

- **Shear stiffness (GA):**
  \[
  \begin{cases}
  D_{44} \\
  D_{45} \\
  D_{54} \\
  D_{55}
  \end{cases}
  \]

- **Axial stiffness (EA):**
  \[
  \begin{cases}
  D_{66} \\
  D_{67} \\
  D_{76} \\
  D_{77}
  \end{cases}
  \]

- **Eccentricity effects:**
  \[
  \begin{cases}
  D_{16} \\
  D_{26} \\
  D_{17} \\
  D_{27} \\
  D_{38} \\
  D_{61} \\
  D_{62} \\
  D_{71} \\
  D_{72} \\
  D_{83}
  \end{cases}
  \]

- **Torsional stiffness (GJ):** \( D_{33} \)

- **In-plane shear stiffness (GA):** \( D_{88} \)
The eccentricity effects depend on how the plate is connected to other structural elements. If no eccentricity effects are present and the local axes of the plate coincide with the global coordinate system, a reduction of the global stiffness matrix can be made according to

\[
D = \begin{bmatrix}
D_{11} & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & D_{22} & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & D_{33} & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & D_{44} & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & D_{55} & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & D_{66} & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & D_{77} & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & D_{88}
\end{bmatrix}
\]  

(2.50)

With curvatures and strains known, load effects with respect to different orientations can be determined with

\[
\begin{bmatrix}
m_x \\
m_y \\
m_{xy} \\
v_x \\
v_y \\
n_x \\
n_y \\
n_{xy}
\end{bmatrix} = \begin{bmatrix}
(EI)_x \\
0 & (EI)_y \\
0 & 0 & (GA)_x \\
0 & 0 & 0 & (GA)_y \\
0 & 0 & 0 & 0 & (EA)_x \\
0 & 0 & 0 & 0 & 0 & (EA)_y \\
0 & 0 & 0 & 0 & 0 & 0 & (GA)_{xy}
\end{bmatrix} \begin{bmatrix}
k_x \\
k_y \\
k_{xy} \\
k_{xz} \\
k_{yz} \\
\gamma_x \\
\gamma_y \\
\gamma_{xy}
\end{bmatrix}
\]  

(2.51)

where \(k, \gamma\) and \(\varepsilon\) represent curvatures, shear strains and axial strains respectively (Dlubal, 2013).

Figure 2.17 illustrates the relationship between load effects and stresses that act on a plate with thickness \(t\). Since the structural behaviour of a slab generally does not include membrane actions, axial forces and corresponding stresses are not included in the figure.

Figure 2.17: Normal and shear stresses acting on a plate (Dlubal, 2013, p. 35)
Normal stresses according to Figure 2.18 are calculated as

\[
\begin{align*}
\sigma_{x,+} &= \frac{n_x}{t} + \frac{6 \cdot m_x}{t^2} \\
\sigma_{x,-} &= \frac{n_x}{t} - \frac{6 \cdot m_x}{t^2} \\
\sigma_{y,+} &= \frac{n_y}{t} + \frac{6 \cdot m_y}{t^2} \\
\sigma_{y,-} &= \frac{n_y}{t} - \frac{6 \cdot m_y}{t^2}
\end{align*}
\] (2.52)

where \( n \) and \( m \) represent axial forces and moments respectively, \( t \) is the thickness of the studied layer and the sign depends on whether \( z \) is positive or not (Dlubal, 2013).

![Figure 2.18: Normal stresses in x- and y-direction respectively (Dlubal, 2013, p. 36)](image)

Shear stresses according to Figure 2.19 are obtained by

\[
\begin{align*}
\tau_{xy,+} &= \frac{n_{xy}}{t} + \frac{6 \cdot m_{xy}}{t^2} \\
\tau_{xy,-} &= \frac{n_{xy}}{t} - \frac{6 \cdot m_{xy}}{t^2} \\
\tau_{xz,max} &= \frac{3}{2} \cdot \frac{v_x}{t} \\
\tau_{yz,max} &= \frac{3}{2} \cdot \frac{v_y}{t}
\end{align*}
\] (2.54)

where \( v_x \) and \( v_y \) are shear forces while \( m_{xy} \) is the torsional moment (Dlubal, 2013).

![Figure 2.19: Shear stresses acting in different planes (Dlubal, 2013, p. 36)](image)

With stresses according to Figure 2.18 and 2.19 known, principal and equivalent stresses can be determined. By comparing the design values of the calculated stresses with the design values of the material strengths, a verification of the cross-section with regard to the ultimate limit state can be made.
2.5.6 Choice of Design Method

The methods described in sections 2.5.1 - 2.5.4 all have their limitations, but they are generally straightforward and easy to use. The $\gamma$-method is limited to single span structures subjected to uniformly distributed loads. Since large concentrated loads are present in load models related to bridge design, the $\gamma$-method is not appropriate to use. Moreover, the composite theory neglects shear deformations and is therefore not accurate since the span-to-depth ratio is less than 30. When considering the shear analogy method, shear deformations are taken into account. However, if the distribution of coupling points between the virtual beams is poorly chosen, the inaccuracy of the outcome may be significant (Riebenbauer, 2013).

When analysing the structure with regard to load effects, stresses and deflections, different load cases must be regarded. Since the variety of cases is fairly wide, the use of a finite element software has big advantages against other design methods. By the use of an appropriate finite element software, multiple load cases are quickly generated. Hence, design load cases and corresponding stresses are easily determined. Since the span-to-depth ratios of the cross-laminated timber slabs are moderate, the impact of shear deformations must be taken into account. By applying the Mindlin-Reissner plate theory, shear deformations are considered. Moreover, cross-laminated timber elements can be treated directly in some programs. By entering structural characteristics, i.e. geometric and material properties, a stiffness matrix is then automatically determined for the specific element.

However, one should be careful when using generated matrices since their stiffness values might differ from recommendations of manufacturers. By comparing with matrices from e.g. CLTdesigner or KLHdesigner, a software adapted for elements produced by KLH, an evaluation of the generated matrix can be made.
3 Bridges

3.1 Steel-Concrete Composite Bridges

The majority of the bridges in Sweden are concrete bridges (Sundquist, 2008). However, due to the extensive falsework and casting procedures required to build these bridges, they are very labour intensive and requires a long construction time. Therefore, when a shorter construction time is necessary, it is common to use a steel-concrete composite beam bridge. The cross-section of a composite bridge normally consists of an in-situ cast concrete deck that is supported on steel girders. Composite action between the elements is achieved through numerous shear connectors, e.g. headed studs, which are welded to the top of the flanges of the main girders (Sundquist, 2008). The shear connectors transmit shear stresses between the concrete slab and the steel girders and ensures that no slip occurs (Lebet and Hirt, 2013). The cross-section of a composite bridge is shown below in Figure 3.1.

![Figure 3.1: Steel and concrete composite cross-section](image)

When a composite bridge is constructed, the steel girders can be launched or lifted into place. After that, the falsework and the casting of the concrete slab can be made. Since the main girders of steel-concrete composite bridge can be erected in a short time, these bridge can be built faster than concrete bridges. However, a lot of time and work are still required for the falsework and casting of the deck. Therefore, the total construction time required to build a composite bridge can be significant.

The compressive strength of the concrete slab combined with the high tensile strength of the steel girders composes a cross-section with favourable material properties (El Sarraf et al., 2013). This generates a bridge cross-section with high bending moment capacity. During the service life of the bridge, the concrete slab stabilizes the steel girders against instability phenomena such as lateral torsional buckling and local buckling of the upper flange. However, the stabilizing properties of the concrete deck develop as the concrete hardens. Thus, the steel girders are vulnerable to buckling during the casting and hardening processes. The risk for instability failure during the construction phase can be decisive for the structural design of the steel girders, and temporary bracing can be required.
The main objectives of the concrete slab in the composite cross-section are listed below (Lebet and Hirt, 2013):

• To act as the carriageway and support the load from the traffic in the bridge’s transverse direction. It distributes the live load to the main girders.
• To restrain the upper flange of the main girders from lateral torsional buckling.
• To transfer transverse loads to the abutments by acting as planar bracing.
• To contribute to the load bearing capacity of the bridge in its longitudinal direction.

In Sweden, composite bridges normally have two main girders. The girders are spaced in order to equalize the bending moments acting on the slab, i.e. positive bending moments at midspan of the slab and negative bending moment at the girders. The main girder spacing is generally 50-55% of the total width of the slab (Lebet and Hirt, 2013). For very wide bridges, multiple main girders can be necessary. The main girders of a multi girder bridge are usually spaced evenly (Lebet and Hirt, 2013). A multi girder approach requires more steel for the main girders than a twin girder bridge. However, given an appropriate design of the bracing system, it can provide a better load distribution between the main girders than a twin girder bridge. Furthermore, multi girder bridges tend to have good redundancy against accidents. If one girder fails, and if the remaining girders have sufficient load bearing capacity, the accident is not necessarily catastrophic for the bridge’s structural integrity.

The main girders of a composite bridge are generally manufactured from hot rolled steel plates that are connected by welding (Lebet and Hirt, 2013). This allows for some flexibility in the design of the girders. The cross-section can be optimized for the application of the structure. When the bridge is subjected to bending moments, the concrete slab in the span sections of a composite bridge relieves the upper flanges of the girders from some of the compression stresses. Therefore, the upper flanges of the main girders can be made smaller than the lower ones. For multi-girder bridges, it can sometimes be economical to use standardised hot rolled beams instead of welded plate girders (Lebet and Hirt, 2013).

Bridge girders are normally manufactured from steel of grade S355 or S460, and quality N or M. N indicates that the steel has been normalised after hot rolling with temperature treatment. Temperature treatment generates a steel with refined grains and a regularised structure of the metal. The steel also gets good tensile strength and toughness. Steel quality M is a thermomechanical steel, and it is a relatively new product. It is produced in rolling machines that perform accelerated cooling of the material. Therefore, the steel does not require subsequent thermal treatment. Thermomechanical steel possesses the same material properties as normalised steel, but it has better weldability due to that it contains less carbon (Lebet and Hirt, 2013).

In order to carry the load of the carriageway in the transverse direction and to fit reinforcement, the concrete slab is generally at least 300 mm thick above the supports and 240 mm in the span sections (Lebet and Hirt, 2013). Since concrete has a high density, the slab tends to become very heavy. If the concrete slab did not act in composite with the main girders, the steel girders would have to carry the entire load of the bridge including the self-weight of the slab. However, the weight of the concrete slab is carried by
the composite cross-section, i.e. the concrete slab to some extent carries itself. However, it can be shown that a substantial part of the bending moment capacity of a composite bridge is consumed by the weight of the concrete slab. Hence, if a light bridge slab should carry the same live load as a composite bridge, its main girders would not have to be much larger.

To illustrate how much of the bending moment capacity of a composite bridge that is consumed by the weight of the concrete deck, the cross-sections of two different beam bridges are compared. The first is a steel-concrete composite bridge with two steel main girders acting in composite with a 350 mm thick concrete slab (see Figure 3.2). The second is a steel girder bridge with a 400 mm thick CLT slab without composite action (see Figure 3.3). Both bridges are simply supported with a span length of 26 m and a deck width of 10 m. The two bridges have identical steel girders of steel grade S355.

![Figure 3.2: Example steel-concrete composite bridge](image)

![Figure 3.3: Example steel bridge with a CLT slab without composite action](image)

The bridges total bending moment capacity, the bending moment that is caused by the weight of the slabs, and the bending moment capacity that can be utilized to carry live load are shown in Table 3.1. The bending moment capacity that can be used to carry live load was calculated by subtracting the bending moment that is caused by the weight of the slabs from the bridges’ total bending moment capacity. The calculations are shown in Appendix A. Furthermore, it should be noted that the steel girders top and bottom flanges have the same dimensions. As was stated earlier, this is usually not the case for composite bridges. The simplification was made in order to facilitate the comparison with the second bridge type.
Table 3.1: Bending moment capacities of the two bridge types

<table>
<thead>
<tr>
<th></th>
<th>Steel-concrete composite bridge [kNm]</th>
<th>Steel-timber non-composite bridge [kNm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending moment resistance of the bridge</td>
<td>12900</td>
<td>8500</td>
</tr>
<tr>
<td>Bending moment induced by the slab</td>
<td>4400</td>
<td>1000</td>
</tr>
<tr>
<td>Remaining bending moment capacity</td>
<td>8500</td>
<td>7500</td>
</tr>
</tbody>
</table>

The total bending moment capacity of the steel bridge with a CLT deck is only 66% of the capacity of the composite bridge. However, if the bending moments that are induced by the self-weight of the slabs are subtracted from the bridges’ total bending moment capacity, the difference is much smaller. Subsequently, the ratio between the bridges’ remaining bending moment capacity is 88%. Consequently, it can be assumed that the steel bridge with a CLT deck would require slightly larger main girders to carry the same live load as the composite bridge. However, it is possible that if the steel bridge with a CLT slab is faster to erect, it could compensate for the higher cost of its girders.

3.2 Timber Bridges

Timber has regained popularity as a bridge building material, and between 1994-2005 almost 500 timber bridges were constructed in Sweden (Troive, 2005). The breakthrough was largely spurred by the introduction of stress-laminated timber elements as bridge decks. Earlier, the heavy concentrated loads from vehicles had been problematic for timber bridges, but the invention enabled them to be used for road traffic.

Timber has many favourable characteristics that make it interesting as a material for bridges. It is light and has a high strength-to-weight ratio (Pousette, 2011). If compared to concrete, a timber structure puts less load on the transport of the structural elements. Furthermore, a light structure puts less load on the substructure. Furthermore, timber is interesting from a sustainability point of view. If the timber originates from a responsible forestry it can have a positive impact on the carbon balance due to that it binds carbon dioxide during its lifetime (Mettem, 2011).

However, timber is susceptible to damages caused by moisture. Fungus and bacteria can grow on the timber if it is moist, and endanger its durability. Hence, for timber bridges to function properly it is paramount to protect the timber against moisture. Traditionally moisture protection of timber has been made with either biological treatment, preservative treatment or structural weather protection (Mettem, 2011). In an attempt to increase the environmental friendliness of timber bridges, the Swedish road authority forbade the use of chemical treatment containing creosote, chrome and arsenic (Troive, 2005). Therefore, the need for well-designed weather protections are further emphasised. Mettem (2011) states that the weather protection of the timber elements of a bridge should be included in the design process from the very beginning. It should also be noted that according to the approval of KLH cross-laminated timber product, CLT is only allowed to be used in service class 1 or 2 (ETA-06/0138, 2012). Hence, it is not permitted for CLT elements to
be directly exposed to precipitation. It is recommended that a CLT bridge slab should be sealed against water seepage from the driving surface. For instance, the slab could be protected with a waterproof membrane. Furthermore, the driving surface should be paved on a welded insulation mat above the waterproof membrane (Mettem, 2011).

It is also advised that timber bridges are inspected periodically and that regular maintenance is performed when necessary (Mettem, 2011). Maintenance typically includes removal of debris, tightening of connectors and control of weather protections (Troive, 2005). If a timber bridge is designed carefully and maintenance is regularly performed, its life cycle cost could be very competitive (Mettem, 2011).

### 3.3 Cross-Laminated Timber Bridges

The main reason that a cross-laminated timber slab is used for the proposed bridge type is the possibility to utilize large pre-fabricated elements to facilitate a fast erection. If a cross-laminated timber slab is compared to a stress-laminated timber slab, the main difference is that the later contains post-tensioned steel rods. All of the boards of a stress-laminated timber slab are oriented in the same direction. Therefore, the post-tensioned steel rods are required to avoid slip between adjacent boards due to large shear forces. However, post-tensioning is labour intensive and more time is required to mount a stress-laminated timber slab than a CLT slab. Since a cross-laminated timber slab has cross-layers, board layers with different orientation reinforces the slab and prevents slip between boards. Hence, a CLT bridge slab does not require post-tensioning. Since the post-tensioning procedure is eliminated for a CLT slab, it should be faster to mount. A short construction time is the main purpose of the new bridge type. Therefore, a CLT slab will be used for the proposed bridge.

Several timber bridges have been constructed over the river Mur in Austria. Among these are a few bridges with cross-laminated timber slabs. Through means provided by a scholarship from the Swedish foundation *Brosamverkan*, a visit to some of these timber bridges was conducted. Notably, there were two similar arch bridges that had decks consisting of ribbed CLT elements. The bridges are used for road traffic, and one of these bridges is displayed in Figure 3.4.
The arch bridges had some differences in their structural weather protection. Therefore, the condition of the decks varied. The bridge that is shown in Figure 3.5 has a metal cladding that should protect the deck. However, it can be noted that the outermost brighter ribs have been replaced. The ribs had suffered from moisture damages that probably occurred due to that precipitation could penetrate beneath the metal cladding.

The deck of the second arch bridge was in better condition (see Figure 3.6). It could not be observed that any major maintenance of the deck had to be performed. A metal cladding protects the CLT elements, and a wooden board protects the ribs. The structural weather protection is shown in Figure 3.7.
Furthermore, a fully covered pedestrian bridge with a box cross-section was visited. Both the deck, the walls and the roof of the bridge were made out of cross-laminated timber elements. The bridge is shown in Figures 3.8 and 3.9.
At the entrance of the bridge, the wall elements had been damaged by precipitation. If Figure 3.10 is considered, it can be noted that the metal cladding covers the edges of the walls. The wood had been damaged in the area around the cladding. It is likely that the CLT elements had been in better condition if the cladding had been extended along the walls for a few more decimetres.
3.4 Cost of Bridges

The cost of a bridge can be calculated as either its total investment cost or as its life-cycle cost (Isaksen, 2005). The investment cost includes everything that is required to build the bridge and to put it into service, e.g. cost of labour, material, management and planning. Whereas, the life-cycle cost includes the initial investment and adds the cost related to the bridge’s lifecycle, e.g. maintenance during its service life, and later the cost of demolishing and recycling the bridge (Lebet and Hirt, 2013).

The choice of the design can substantially influence the cost of the bridge, and the biggest savings can be made during the early stages of the design process (Lebet and Hirt, 2013). The relative cost between labour and material influences whether it is more feasible to build a bridge that requires more work and less material or one that requires less work and more material. Moreover, the time required to construct the bridge can be decisive for its design. For instance, when launching a bridge over a railway it can be very expensive to disrupt the railway traffic for a prolonged time (Isaksen, 2005). Therefore, it could be economically feasible to use a bridge design that allows for quick erection even if its construction cost is higher than for a conventional bridge.

When replacing an old bridge, it can be possible to reuse the existing foundations if the new bridge is light enough. This was the case for the Flisa Bridge in Norway. A new concrete bridge would have required a new path and a longer bridge whereas a timber bridge could utilise the existing foundations. The expected cost of the concrete bridge
A study in Norway compared the unit price of steel, timber and concrete bridges (Isaksen, 2005). The price was calculated in NKr/m\(^2\). The area of the bridges were determined for their *free area*, i.e. the total length of the bridge multiplied with the clear width of the carriageway. For slab bridges, the timber bridges spanned up to 20 m and had an average cost of 12 000-14 000 NKr/m\(^2\). The cost depended on whether the deck was made out of sawn timber or glulam. Concrete slab bridges spanned up to 30 m and had an average unit cost of 11 000-12 000 NKr/m\(^2\). For girder bridges, it was found that timber bridges had an average cost of 14 000 NKr/m\(^2\). The timber girder bridges were mainly used for short spans on lightly trafficked roads. The average cost of a prefabricated concrete girder bridge and an in-situ cast concrete girder bridge were 11 000 NKr/m\(^2\) and 12 000 NKr/m\(^2\) respectively. The average cost of a built-up steel girder was 20 000 NKr/m\(^2\) and the cost of a steel box girder bridge was 15 000 NKr/m\(^2\) (Isaksen, 2005).

The cost of the main components of a steel-concrete composite bridge in relation to its total construction cost are shown in Table 3.2 (Lebet and Hirt, 2013). The considered bridge has average span lengths, and the cost of design and management is not included in the comparison.

**Table 3.2: The cost of the main components in relation to the construction cost of a steel-concrete composite bridge (Lebet and Hirt, 2013)**

<table>
<thead>
<tr>
<th>Component</th>
<th>Relative price</th>
</tr>
</thead>
<tbody>
<tr>
<td>Substructure</td>
<td>25-40%</td>
</tr>
<tr>
<td>Superstructure</td>
<td>40-60%</td>
</tr>
<tr>
<td>Site installations</td>
<td>6-8%</td>
</tr>
<tr>
<td>Other Components</td>
<td>10-15%</td>
</tr>
<tr>
<td>Annual maintenance</td>
<td>1.0-1.2%</td>
</tr>
</tbody>
</table>
4 The Reference Bridge

The design of the new bridge type is based on an existing steel-concrete composite bridge. The reference bridge was selected with the help of an experienced bridge engineer from the bridge engineering firm Centerlöf & Holmberg. The bridge was built over the river Ältran in Ulricehamn in Sweden. The elevation of the reference bridge is shown below in Figure 4.1. The bridge’s dimensions are shown in Table 4.1.

![Figure 4.1: Elevation of the reference bridge (Centerlöf och Holmberg)](image)

<table>
<thead>
<tr>
<th>Total length</th>
<th>Theoretical span length</th>
<th>Width of the slab</th>
<th>Free width</th>
</tr>
</thead>
<tbody>
<tr>
<td>35.9 m</td>
<td>26.0 m</td>
<td>10.4 m</td>
<td>9.75 m</td>
</tr>
</tbody>
</table>

The slab of the reference bridge has a slight curvature in plan and elevation. However, for the cost estimate of the reference bridge, the slab is assumed to be straight. Considering Figure 4.2, it can be noted that the thickness of the concrete slab varies along its transverse direction. To facilitate the cost estimate, the slab is assumed to have a constant thickness of 350 mm. It should also be noted that one of the outer lanes is reserved for pedestrian traffic. For the new bridge type, the pedestrian lane is not considered.

The main girders are straight in plan. Furthermore, they have a pre-camber of about 100 mm. The main girder spacing is 5200 mm. Moreover, the main girders are not vertically aligned. There is a height difference of 60 mm between the main girders’ centre of gravities. Therefore, the concrete slab becomes tilted. However, a height difference of 60 mm over a distance of 5200 mm corresponds to an angle of 0.66°. Hence, the inclination is so small that it should have a negligible effect on the bridge’s structural behaviour.
The main girder dimensions vary along the length of the bridge. The girders are divided into three parts. The external parts are identical, and 6 m long. The middle part is 14 m long and has wider flanges (see Figure 4.3). The dimensions of the main girders are shown in Table 4.2 and the notations are shown in Figure 4.4. The steel of the main girders is of grade S460M.
Table 4.2: Dimensions of the reference bridge’s main girders

<table>
<thead>
<tr>
<th>Position</th>
<th>$h$ [mm]</th>
<th>$t_{f,\text{top}}$ [mm]</th>
<th>$b_{f,\text{top}}$ [mm]</th>
<th>$t_{w}$ [mm]</th>
<th>$h_{w}$ [mm]</th>
<th>$t_{f,\text{bot}}$ [mm]</th>
<th>$b_{f,\text{bot}}$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-6 000</td>
<td>1350</td>
<td>25</td>
<td>350</td>
<td>17</td>
<td>1295</td>
<td>30</td>
<td>500</td>
</tr>
<tr>
<td>6 000-20 000</td>
<td>1382</td>
<td>25</td>
<td>430</td>
<td>15</td>
<td>1312</td>
<td>45</td>
<td>620</td>
</tr>
<tr>
<td>20 000-26 000</td>
<td>1350</td>
<td>25</td>
<td>350</td>
<td>17</td>
<td>1295</td>
<td>30</td>
<td>500</td>
</tr>
</tbody>
</table>

It can be noted in Figure 4.3 above that the main girders are stabilized with crossbeams at five positions. At the abutments, the crossbeams are HEB 650 of steel grade S355. The three intermediate crossbeams are HEA 240 of steel grade S275. The bracing is shown in cross-section in Figure 4.5 and 4.6.

**Figure 4.5:** Cross-section of the main girders braced with an HEB 650 (Centerlöf och Holmberg)

**Figure 4.6:** Cross-section of the main girders braced with an HEA 240 (Centerlöf och Holmberg)
5 The Proposed Bridge Type

The new, proposed bridge type is a steel beam bridge with a slab of cross-laminated timber elements. The bridge is intended for road traffic. The idea to design a bridge with a CLT deck originates from the need of bridges that are fast to build. Since CLT can be produced as large prefabricated elements, it should be possible to mount the slab on the steel girders in a short time. Therefore, the new bridge type could be competitive when a short construction time is necessary.

The design of the new bridge type is limited to the structural elements of its superstructure, i.e. the main girders, the CLT slab and the bracing system. The geometry of the new bridge type was based on the reference bridge with some minor deviations. The most prominent difference is that the cross-laminated timber slab of the new bridge type is straight in plan and is not pre-cambered. Likewise, the main girders are not pre-cambered. The new bridge type is shown in Figure 5.1, and its basic geometry is shown in Table 5.1.

![Figure 5.1: The new bridge type](image)

<table>
<thead>
<tr>
<th>Span length [m]</th>
<th>Width [m]</th>
<th>Free width [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>26.0</td>
<td>10.0</td>
<td>9.0</td>
</tr>
</tbody>
</table>

The bridge has a parapet on both of its longitudinal sides. The parapet is supported on prefabricated concrete edge beams that are 500 mm wide and 200 mm thick. Therefore, 500 mm on both sides of the carriageway is closed for traffic. Unlike the reference bridge, the new bridge type does not have a pedestrian lane. Hence, the new bridge type is designed for vehicular traffic to occur over the bridge’s entire free width.
In contrast to the reference bridge, the thickness and the width of the CLT slab is constant along the bridge’s length and width. The same applies for the main girders, their dimensions do not vary along their length. Moreover, the spacing between the main girders is 5200 mm. This corresponds to 52% of the bridge’s total width. The cross-section of the new bridge type is shown in Figure 5.2. The dimensions of the structural elements of the bridge are given in section 5.1.

Since steel and timber have different moduli of elasticities, composite action is not believed to be effective for the new bridge type. Therefore, the steel girders do not act in composite with the cross-laminated timber slab. Steel has a much higher modulus of elasticity than wood. Hence, for a steel-CLT composite cross-section, the modular ratio becomes high. If \( E_s = 210 \text{ GPa} \) and \( E_{CLT} = 12 \text{ GPa} \) for short-term actions, then the modular ratio is \( \eta_0 = E_s/E_{CLT} = 17.5 \). In comparison, the modular ratio for short-term actions for a steel-concrete composite bridge where the concrete quality is C40, would be \( \eta_0 = E_s/E_c = 210/35 = 6.0 \). Even more, for long-term actions the difference between the modular ratios would be much greater. Therefore, it is not believed that composite action would increase the bridge’s bending moment capacity enough to compensate for the work required to ensure full shear connection.

Furthermore, to ensure full composite action between the steel girders and the CLT slab, a very stiff connection would be required. Changes in temperature can cause the steel and timber to expand or contract according to equation 5.1. If these movements are restrained, it will lead to stresses in the elements. If Table 5.2 is considered, it can be noted that the materials have different coefficients of thermal expansion. Therefore, the steel girders and the CLT slab will get different movements when exposed to temperature changes. Since the shear connection would restrain these movements, there would be stress concentrations in the timber in the area around the connectors. It is likely that the timber would crack due to these stresses. Similarly, timber has different coefficients of thermal expansion in the directions parallel and perpendicular to the grain. Therefore, layers with different orientation restrain each others movements. However, the restraint stresses are distributed over a large surface, i.e. the glued contact surface between layers. Thus, the stresses become small and do not pose a problem. Since concrete and steel have similar coefficients of thermal expansion, these materials work well in composite.
In the same manner as for temperature movements, changes in moisture content can cause the timber to swell or contract. Since steel is indifferent to moisture changes, this could also cause restraint stresses.

\[ \varepsilon = \alpha \cdot \Delta T \]  \hspace{1cm} (5.1)

<table>
<thead>
<tr>
<th>Table 5.2: Thermal expansion coefficients (Burström, 2007, p. 140)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spruce</td>
</tr>
<tr>
<td>[10^{-6}$/K$]</td>
</tr>
<tr>
<td>( \alpha )</td>
</tr>
</tbody>
</table>

To meet the requirements in the serviceability limit state, it was found necessary to stabilize the main girders with x-type planar and cross bracing spaced 3.25 m. The bracing system is shown in Figures 5.3 and 5.4. It can be noted that the bracing members are connected to the main girders via web stiffeners. The web stiffeners are shown in the figure for illustrative purposes. However, they are not included in the design of the new bridge type.

Figure 5.3: The bracing system that stabilizes the main girders of the new bridge type
In an attempt to reduce the amount of steel, it was tried to exchange the steel bracing members for glulam beams. It was found that the glulam beams would have to have the dimensions $215 \times 450$ mm$^2$. Therefore, the glulam beams were considered to be too large to be practical. The size of the bracing would reduce the space under the bridge and obstruct maintenance work. It is also likely that large dimensions of the bracing would make the appearance of the bridge look bulky. Therefore, the bracing made out of steel angle sections were considered more feasible.

A prototype of the new bridge type with three main girders was also developed. The bridge had a slightly thinner cross-laminated timber slab but required much more steel. Therefore, it was considered to become too expensive and was abandoned. The prototype bridge is shown in Figure 5.5.
5.1 Dimensions of the Structural Elements

5.1.1 The Cross-laminated Timber Slab

The design of the cross-laminated timber slab was based on the specifications given in the European technical approval of KLH’s CLT elements. Therefore, the cross-section is not allowed to be thicker than 500 mm. Furthermore, the layers cannot be thicker than 90 mm, and the thickness of the individual lamella is limited to 45 mm (ETA-06/0138, 2012). Moreover, the basic material of KLH’s CLT slabs is spruce of quality C24. The cross-section of the cross-laminated timber slab is shown in Figures 5.6 and 5.7.

From a structural design point of view, cross-laminated timber elements have a very attractive characteristic. The lamellae lay-up can be optimized to the specific load effects inherent to the application of the structure. Therefore, a CLT slab can be designed to resist the large concentrated loads from traffic. For the design of the CLT slab of the new bridge type, the rolling shear stress in the cross-layers were found to be decisive.

To resist the shear forces, it is necessary to avoid large rolling-shear stresses in the cross-layers. The shear stress in a section has a parabolic variation, and the maximum stress occurs at the neutral layer. For a symmetric cross-section, the neutral layer is positioned in the middle of the section. Hence, the cross-layers should be positioned as far away from the middle of the cross-section as possible. Consequently, it was found to be beneficial to have a central longitudinal layer constituent of two lamellae, and to make the four innermost lamellae 5 mm thicker than the residual lamellae.

Furthermore, for a given shear force, the magnitude of the shear stresses depends on the effective area of the cross-section. If the effective area is increased, then the magnitude of the shear stresses is reduced. Thus, to reduce the rolling shear stresses in the cross-layers, it can be advantageous to have a thick slab with many longitudinal layers. For the CLT slab of the new bridge type, a 500 mm thick cross-section constituent of eight longitudinal lamellae and four cross-oriented lamellae were used.

If the bending resistance of the CLT slab is considered, it is advantageous if the longitudinal layers have a large internal lever. Therefore, double longitudinal lamellae were positioned as the external layers.
Figure 5.6: Cross-section in load-bearing direction

Figure 5.7: Cross-section in non-load-bearing direction
5.1.2 The Main Girders

The new bridge type has two 26 m long main girders that are spaced 5.2 m apart from each other. The dimensions of the main girders of the new bridge type are shown in Figure 5.8 and Table 5.3. The main girders are made from steel of grade S355M. Since it was the design in the serviceability limit state that was found to be decisive for the design, a higher steel grade was not considered necessary.

![Figure 5.8: Notations with respect to main girder](image)

<table>
<thead>
<tr>
<th></th>
<th>$h$</th>
<th>$t_{f,\text{top}}$</th>
<th>$b_{f,\text{top}}$</th>
<th>$t_{w}$</th>
<th>$h_{w}$</th>
<th>$t_{f,\text{bot}}$</th>
<th>$b_{f,\text{bot}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>[mm]</td>
<td>1650</td>
<td>50</td>
<td>500</td>
<td>17</td>
<td>1550</td>
<td>50</td>
<td>500</td>
</tr>
</tbody>
</table>

5.1.3 The Bracing System

The top flanges of the main girders of the new bridge type are connected to the cross-laminated timber slab and restrained from lateral movements. Therefore, the main girders are protected against lateral torsional buckling, and the girders do not have to be stabilized with bracing in order to resist the loads in the ultimate limit state. However, when the bridge was controlled for deflections in the serviceability limit state, the bridge was unable to fulfil the requirements. If the traffic loads were positioned in the notional lanes so that
the girders were unevenly loaded, the most heavily loaded girder deflected much more than the other. This caused large transverse deflections of the bridge deck. To reduce the deflections, it was necessary to create a virtual box-section by introducing x-type planar and cross-bracing. Since a box-section has large torsional stiffness, the loads were distributed more evenly to the main girders, and it was possible to meet the requirements for the deflections.

The bracing consists of angle-sections of steel of grade S355M. The dimensions of the angle-sections are shown in Figure 5.9. The members are connected to the main girders at the positions where the top and bottom flanges connect to the web.

![Figure 5.9: Dimensions of bracing](image)

5.2 Conceptual Design of Connections

In this section, the conceptual designs of the two connections are discussed briefly. The considered details are the connection between the main girders and the cross laminated timber elements, and the interconnection between two adjacent cross-laminated timber elements.

The connection between the main girders and the CLT elements should be able to resist uplift forces due to wind loads. The connection should also be able to resist the horizontal forces from wind, and centrifugal and braking forces from traffic, so that the slab does not move horizontally. In this project, the CLT elements were modelled as simply supported on the main girders. Therefore, the connection should not restrain the CLT elements from rotating. In Figure 5.10, a conceptual sketch shows an example of how the main girders and the CLT slab could be connected. The top flange of the main girder is connected to the CLT slab by means of coach screws. To reduce the risk of damages due to moisture, it is better to make the holes from the underside of the slab. Since the elements are not modelled to act in composite with the main girders, the connection should allow some movement in the bridge’s longitudinal direction.
A conceptual sketch of the interconnection of two adjacent CLT elements is shown in Figure 5.11. In this project, the CLT slab was modelled without rotational release at the joint between the elements. Therefore, the connection should be bending stiff. The bending moments are transferred by the cover plates. The cover plate could be made out of a timber board, plywood or a steel plate that is screwed to the CLT element. The shear forces are transferred by the cross-screws inside the CLT element.
5.3 Weather Protection

The new bridge type should be designed for a service life of 100 years (SS-EN 1990 - Basis of structural design). Therefore, it is important to protect the bridge from damages that could jeopardize its durability. A major concern for both the steel structure and the cross-laminated timber slab is moisture, and they should be provided with suitable protection.

The display of bridges built with cross-laminated timber presented in section 3.3 clearly shows the importance of a carefully designed weather protection to ensure the durability of cross-laminated timber elements. Since it is not allowed to design CLT for service-class 3, the CLT elements should be protected from direct exposure to precipitation. Hence, a structural weather protection of the CLT elements is necessary. The top face of the elements should be covered with a water protective membrane (Mettem, 2011). Furthermore, the end grains of the timber are particularly susceptible to moisture induced damages. Therefore, the exposed edges of the elements should be protected with a structural cover. The cover could be either a metal cladding or a cover board made from a durable hardwood. If precipitation should penetrate the cladding, it is important to design the structure so that the water easily can drain off from the surface of the elements.

The steel girders and the bracing could be protected with an anti-corrosive paint. Furthermore, the steel structure should be designed to promote surface runoff.
6 Characteristic Actions on the New Bridge Type

The structural behaviour of the bridge was analysed for load effects due to permanent loads and standard traffic loads. The verification of the bridge’s resistance against accidental loads, braking forces, snow, wind loads and fatigue effects are not within the scope of this thesis. Furthermore, stresses due to restrained shrinkage, creep and temperature movements are neglected. However, if a complete design of the bridge was made, these verifications should be included.

6.1 Permanent Loads

The permanent loads acting on the CLT slab are the weight of the slab, parapets, edge beams and pavement. In addition to these loads, the self-weight of the main girder and the bracing system have to be included for the design of the main girders.

Since the design of the parapet is not included in this project, a standard steel parapet with a load of 1 kN/m is used (Lebet and Hirt, 2013). The edge beams are assumed to be prefabricated concrete elements that are 500 mm wide and 200 mm thick. The permanent load from the parapet and the edge beams are applied as line loads that act 250 mm from the edge of the bridge deck.

The pavement is assumed to be a 100 mm thick layer of asphalt concrete that covers the entire slab’s surface.

The permanent loads are listed below in Table 6.1. The specific weights are taken from Bro 2004 (2007). The cells marked with "n/a" correspond to data that is not applicable to that element.

<table>
<thead>
<tr>
<th>Element</th>
<th>t</th>
<th>b</th>
<th>A</th>
<th>γ</th>
<th>q₀</th>
<th>qₖ</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLT slab</td>
<td>500</td>
<td>n/a</td>
<td>n/a</td>
<td>6.0</td>
<td>3.0</td>
<td>n/a</td>
</tr>
<tr>
<td>Parapet</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>1.5</td>
</tr>
<tr>
<td>Edge beam</td>
<td>200</td>
<td>500</td>
<td>100</td>
<td>25</td>
<td>n/a</td>
<td>2.5</td>
</tr>
<tr>
<td>Pavement</td>
<td>100</td>
<td>n/a</td>
<td>n/a</td>
<td>23</td>
<td>2.3</td>
<td>n/a</td>
</tr>
<tr>
<td>Main girders</td>
<td>n/a</td>
<td>n/a</td>
<td>76.4</td>
<td>77</td>
<td>n/a</td>
<td>5.9</td>
</tr>
<tr>
<td>Bracing</td>
<td>n/a</td>
<td>n/a</td>
<td>5.54</td>
<td>77</td>
<td>n/a</td>
<td>0.43</td>
</tr>
</tbody>
</table>

6.2 Traffic Loads

Load model 1 (LM1) is used to simulate the load from an extremely severe traffic situation on a heavily trafficked road (SS-EN 1991-2 - Actions on structures part 2: Traffic loads on bridges). It is supposed to symbolize a traffic load with a probability of annual exceedance of 0.1 % (Lebet and Hirt, 2013).
The loads of LM1 are applied on the carriageway in notional lanes. The notional lanes are generated by dividing the carriageway according to Figure 6.1 and Table 6.2.

**Table 6.2: Number of notional lanes**

<table>
<thead>
<tr>
<th>Width of the carriageway</th>
<th>Number of notional lanes</th>
<th>Width of a notional lane</th>
<th>Width of the remaining area</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w$</td>
<td>$n_l$</td>
<td>$w_l$</td>
<td></td>
</tr>
<tr>
<td>$w &lt; 5.4 , m$</td>
<td>$n_l = 1$</td>
<td>$3 , m$</td>
<td>$w - 3 , m$</td>
</tr>
<tr>
<td>$5.4 , m \leq w &lt; 6 , m$</td>
<td>$n_l = 2$</td>
<td>$\frac{w}{2}$</td>
<td>$0$</td>
</tr>
<tr>
<td>$6 , m \leq w$</td>
<td>$n_l = \text{Int}(\frac{w}{3})$</td>
<td>$3 , m$</td>
<td>$w - 3 \cdot n_l$</td>
</tr>
</tbody>
</table>

![Diagram of notional lanes](image)

**Figure 6.1: Numbering of notional lanes (SS-EN 1991-2 - Actions on structures part 2: Traffic loads on bridges)**

Since the new bridge type is 10 m wide and has 500 mm wide edge beams on both side, its carriageway is 9 m wide. Moreover, the carriageway consist of 3 notional lanes with a width of 3 m each (see Table 6.3).

**Table 6.3: Notional lanes of the new bridge type**

<table>
<thead>
<tr>
<th>Carriageway width</th>
<th>Number of notional lanes</th>
<th>Width of a notional lane</th>
<th>Width of the remaining area</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w$</td>
<td>$n_l$</td>
<td>$w_l$</td>
<td></td>
</tr>
<tr>
<td>$9 , m$</td>
<td>3</td>
<td>$3 , m$</td>
<td>$0 , m$</td>
</tr>
</tbody>
</table>

The characteristic loads of LM1 consist of a tandem part and a uniformly distributed part. The tandem load represents the boogie-axle of a fictive heavy truck, and the uniformly distributed load represents the load from cars. As stated earlier, LM1 represents an extreme load on a bridge where very heavy traffic is likely to occur. Since most bridges are not likely to be subjected to such high loads, it is allowed to reduce the loads, according to the national annex of the respective country (SS-EN 1991-2 - Actions on structures part 2: Traffic loads on bridges). The characteristic loads of LM1 and their corresponding reduction factors according to the Swedish national annex (TRVFS2011:12, 2011) are shown in Table 6.4 and 6.5.
Table 6.4: Characteristic tandem loads of LM1 and their corresponding reduction factors according to the Swedish national annex (TRVFS2011:12, 2011)

<table>
<thead>
<tr>
<th>Lane number</th>
<th>Axle loads $Q_{0,ik}$ [kN]</th>
<th>Reduction factor $\alpha$</th>
<th>Reduced axle loads $Q_{ik}$ [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>300</td>
<td>0.9</td>
<td>270</td>
</tr>
<tr>
<td>2</td>
<td>200</td>
<td>0.9</td>
<td>180</td>
</tr>
<tr>
<td>3</td>
<td>100</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 6.5: Characteristic uniformly distributed loads (UDL) of LM1 and their corresponding reduction factors according to the Swedish national annex (TRVFS2011:12, 2011)

<table>
<thead>
<tr>
<th>Lane number</th>
<th>UDL $q_{0,ik}$ [kN/m$^2$]</th>
<th>Reduction factor $\alpha$</th>
<th>Reduced UDL $q_{ik}$ [kN/m$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9</td>
<td>0.7</td>
<td>6.3</td>
</tr>
<tr>
<td>2</td>
<td>2.5</td>
<td>1.0</td>
<td>2.5</td>
</tr>
<tr>
<td>3</td>
<td>2.5</td>
<td>1.0</td>
<td>2.5</td>
</tr>
</tbody>
</table>

The numbering of the characteristic loads corresponds to the notional lane that they belong to (see Figure 6.2). The notional lanes should be numbered to obtain the most adverse load effect on the bridge (SS-EN 1991-2 - Actions on structures part 2: Traffic loads on bridges). I.e. if it is found decisive for the design of the bridge, notional lane number 1 and its corresponding loads could very well be centred in the carriageway.

Figure 6.2: Application of load model 1 (SS-EN 1991-2 - Actions on structures part 2: Traffic loads on bridges)
The uniformly distributed load should be positioned in the area of the notional lane where it generates an unfavourable load effect. Correspondingly, it is allowed to move the tandem load within the notional lane. However, it is not allowed for tandem loads of adjacent notional lanes to be closer than 500 mm to each other (see Figure 6.3) (SS-EN 1991-2 - Actions on structures part 2: Traffic loads on bridges).

![Diagram showing application of tandem systems for local verification](image)

**Figure 6.3: Application of tandem systems for local verification (SS-EN 1991-2 - Actions on structures part 2: Traffic loads on bridges)**

If Figure 6.3 is considered, it can be noted that the contact surface of the wheels of the tandem loads is $400 \times 400$ mm$^2$. However, SS-EN 1995-2 - Design of timber structures part 2: Bridges states that concentrated “loads should be considered at a reference plane in the middle of the deck plate”. Therefore, the contact surface of the tandem load is increased to account for the load dispersion through the pavement and the CLT slab (see Figure 6.4).
The angle of dispersion is 45° for pavement and cross-laminated timber respectively (SS-EN 1995-2 - Design of timber structures part 2: Bridges). Given a 500 mm thick CLT slab with a 100 mm thick layer of pavement on top, the contact surface of the tandem load is obtained by

\[ b_{w,\text{middle}} = b_w + 2 \cdot \tan (\beta_{\text{pavement}}) \cdot t_{\text{pavement}} + 2 \cdot \tan (\beta_{\text{CLT}}) \cdot \frac{t_{\text{CLT}}}{2} \]  

(6.1)

\[ \Rightarrow \]

\[ b_{w,\text{middle}} = 400 + 2 \cdot \tan (45^\circ) \cdot 100 + 2 \cdot \tan (45^\circ) \cdot \frac{500}{2} = 1100 \text{ mm} \]

For local verification of compression perpendicular to the grain of the CLT slab, load model 2 (LM2) is used (see Figure 6.5). The load of LM2 consist of a single axle that can be positioned at any location of the carriageway (SS-EN 1991-2 - Actions on structures part 2: Traffic loads on bridges). When applicable it is allowed to use the load from just one of the wheels.
The load on the axle is $\beta Q \cdot Q_{ak}$ where $\beta = 0.9$ and $Q_{ak} = 400 \text{kN}$. If one wheel is used, half of the axle load is applied.

### 6.3 Governing Load Cases

The influence line method was used to determine how the traffic loads should be positioned in the notional lanes to induce the most severe load effects on the structural elements. The influence line method illustrates how the magnitude of a load effect, at a given position in the structure, is influenced by the position of the load. Influence lines can be used in either a qualitative or quantitative way (Hibbeler, 1990). In the quantitative method influence lines can be used to determine the exact magnitude of the load effect for a given load and position. Whereas, in the qualitative method influence lines can be used to indicate how the load should be positioned to induce the maximum load effect. It is the qualitative method that was used in this thesis. The influence lines were determined according to the Müller-Bresslau principle (Hibbeler, 1990). A unit load is introduced at the considered position on the structure, and then the constraint that gives rise to the considered load effect is released. The release of the constraint corresponds to either a unit deflection or a unit rotation. The deflection curve of the beam due to the unit load comprises the influence line. Moreover, the size of the deflection is to scale with the magnitude of the load effect (Hibbeler, 1990).

To determine the influence line for the bending moment at the midspan of a simply supported beam, a hinge with a unit rotation is created. The influence line constitutes of the deflection curve of the beam (see Figure 6.6).

![Figure 6.6: Influence line for the bending moment at midspan of a simply supported beam](image)
Correspondingly, the influence line for the shear force close to the support of a simply supported beam is derived by the introduction of a unit load at that position and then releasing the constraint one unit deflection (see Figure 6.7). The reaction force at the supports can be determined according to the same procedure.

![Figure 6.7: Influence line for the shear force close to the support of a simply supported beam](image)

### 6.3.1 Decisive Load Cases for the Design of the CLT Slab

The CLT slab of the new bridge type is simply supported on the two main girders, and it cantilevers 2.4 m outside of the girders. The following load effects were identified as decisive for the design of the CLT slab:

- The shear force on the slab close to the support on one of the main girders.
- The positive bending moment at midspan of the slab between the main girders.
- The negative bending moment on the slab at the position of the support on one of the main girders.

The influence lines for the load effects on the CLT slab are shown in Figures 6.8 and 6.9.

![Figure 6.8: Influence line for the shear force close to the support of the CLT slab](image)

![Figure 6.9: Influence lines for positive bending moments at midspan (left) and negative bending moments at support (right) of the CLT slab](image)

The tandem loads are distributed on a specified contact surface (see Figure 6.3), however, to facilitate the identification of possible load cases, the tandem loads were assumed to act as point loads. From the influence lines, seven different load cases were identified. All of these were investigated in the finite element model. Due to that the CLT slab only carries
load in one direction, the position of the tandem load in the bridge’s longitudinal direction
does not influence the magnitude of the load effects. Therefore, the tandem loads were
introduced at the midspan of the bridge, whereas the uniformly distributed traffic loads
were positioned along the bridge’s entire length. The decisive load cases for shear force
and bending moment on the CLT slab are shown in Figures 6.10 and 6.11. The negative
bending moment on the CLT slab at the support was greater than the positive bending
moment at midspan. Therefore, only the design load case for the support bending
moment is shown. Moreover, the design load cases for the cross-laminated timber slab
differs somewhat from what could be intuitively expected from the influence lines. The
reason is that the tandem loads are not point loads. They are concentrated loads with a
contact surface of 1.21 m² per wheel. Since the tandem loads are distributed over a fairly
large area, the design load cases do not correspond exactly to the influence lines.

Figure 6.10: Design load case for shear force on the CLT slab

Figure 6.11: Design load case for bending moment on the CLT slab
6.3.2 Governing Load Cases for the Design of the Main Girders

To determine the decisive load case for the design of the main girders, the traffic loads were positioned on the carriageway to get the largest possible support reaction on one girder. The influence line for the support reaction is shown in Figure 6.12.

![Influence line for the reaction force on one of the main girders](image)

Figure 6.12: Influence line for the reaction force on one of the main girders

For the main girders, only one load case was identified, and it is shown in Figure 6.13. To get the decisive shear force on the girder, the tandem loads are positioned close to one of the supports. Moreover, to get the decisive bending moment, the loads should be positioned at the midspan of the bridge. This load case is also decisive for deflections in the serviceability limit state. The load position for decisive shear force, bending moment and deflection is shown in Figure 6.14.

![Section of the design load case for the reaction force on the left main girder](image)

Figure 6.13: Section of the design load case for the reaction force on the left main girder

![Design load cases for shear force (left), bending moment (right) and deflections (right)](image)

Figure 6.14: Design load cases for shear force (left), bending moment (right) and deflections (right)
6.4 Load Combinations

The design load for verification in the ultimate limit state is obtained by combining the permanent and the live loads according to load combinations 6.10a or 6.10b (SS-EN 1990 - Basis of structural design). The combination that gives the most severe load situation is then used to calculate the decisive load effects on the bridge. Load combinations 6.10a and 6.10b are shown in equations 6.2 and 6.3 as follows:

\[
6.10a: \gamma_d \cdot 1.35 \cdot G_{kj,\text{sup}} + \gamma_d \cdot 1.5 \cdot \psi_{0,1} \cdot Q_{k,1} + \gamma_d \cdot 1.5 \cdot \psi_{0,i} \cdot Q_{k,i} 
\]

\[
6.10b: \gamma_d \cdot 0.89 \cdot 1.35 \cdot G_{kj,\text{sup}} + \gamma_d \cdot 1.5 \cdot Q_{k,1} + \gamma_d \cdot 1.5 \cdot \psi_{0,i} \cdot Q_{k,i} 
\]

where \( \gamma_d \) is the safety class factor, \( G \) is the permanent load, \( Q \) is the live load and \( \psi \) is a reduction factor for live loads.

The tandem loads and the uniformly distributed loads count as a single load type. Thus, only one type of live loads acts on the bridge. Equations 6.2 and 6.3 can therefore be reduced to

\[
6.10a: \gamma_d \cdot 1.35 \cdot G_{kj,\text{sup}} + \gamma_d \cdot 1.5 \cdot \psi_{0,1} \cdot Q_{k,1} 
\]

\[
6.10b: \gamma_d \cdot 0.89 \cdot 1.35 \cdot G_{kj,\text{sup}} + \gamma_d \cdot 1.5 \cdot Q_{k,1} 
\]

The bridge is designed for safety class 3, wherefore \( \gamma_d = 1.0 \). Since the live loads represent a substantial part of the total load on the bridge, load combination 6.10b is decisive.

The bridge is controlled in the serviceability limit state for characteristic and frequent loads. The characteristic load combination 6.14b (SS-EN 1990 - Basis of structural design) according to equation 6.6 is used to control that no permanent damages occur on the bridge’s structural elements.

\[
6.14b: 1.0 \cdot G_{kj,\text{sup}} + 1.0 \cdot Q_{k,1} 
\]

The main girders are in cross-section class 3 and the bracing members are in cross-section class 4 (see B.1 and B.2). Therefore, it is not allowed to exceed the yield strength of the steel of these members in the ultimate limit state. If the steel does not plasticize in the ultimate limit state, it is obvious that it will not plasticize in the serviceability limit state. Therefore, the characteristic load combination will not be used.

The deflections are controlled for the frequent load combination 6.15b (SS-EN 1990 - Basis of structural design) according to equation 6.7.

\[
6.15b: 1.0 \cdot G_{kj,\text{sup}} + \psi_{1,1} \cdot Q_{k,1} 
\]

According to the Swedish national annex, \( \psi_{1,1} \) is equal to 0.75 for tandem loads and 0.4 for uniformly distributed loads (TRVFS2011:12, 2011).

6.5 Design Loads on the New Bridge Type

The design loads on the bridge in the ultimate limit state according to load combination 6.10b (equation 6.3) are shown in Table 6.6. The tandem loads and the uniformly distributed traffic loads of LM1 are referred to as \( TS \) and \( UDL \) respectively.
The tandem load of LM2 is referred to as $Q_{ak}$. Furthermore, the value of the tandem loads are given in $kN$. The value of the load from uniformly distributed traffic loads, the pavement, and the CLT slab are listed as area loads in $kN/m^2$. The value of the load from the parapet, the edge beams and the main girders are listed as line loads in $kN/m$. The structural elements of the bridge are designed according to safety class 3. Thus the safety class factor, $\gamma_d$, is 1.0. It is not shown in the table.

Table 6.6: Design loads in the ultimate limit state according to load combination 6.10b

<table>
<thead>
<tr>
<th>Characteristic load</th>
<th>Load combination factor</th>
<th>Design load (6.10b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$kN$</td>
<td>$-$</td>
<td>$kN$</td>
</tr>
<tr>
<td>TS 1</td>
<td>270</td>
<td>1.5</td>
</tr>
<tr>
<td>TS 2</td>
<td>180</td>
<td>1.5</td>
</tr>
<tr>
<td>UDL 1</td>
<td>6.3</td>
<td>1.5</td>
</tr>
<tr>
<td>UDL 2</td>
<td>2.5</td>
<td>1.5</td>
</tr>
<tr>
<td>UDL 3</td>
<td>2.5</td>
<td>1.5</td>
</tr>
<tr>
<td>$Q_{ak}$</td>
<td>360</td>
<td>1.5</td>
</tr>
<tr>
<td>Pavement</td>
<td>2.3</td>
<td>1.2</td>
</tr>
<tr>
<td>Parapet</td>
<td>1.5</td>
<td>1.2</td>
</tr>
<tr>
<td>Edge beam</td>
<td>2.5</td>
<td>1.2</td>
</tr>
<tr>
<td>Parapet</td>
<td>1.5</td>
<td>1.2</td>
</tr>
<tr>
<td>CLT slab</td>
<td>3.0</td>
<td>1.2</td>
</tr>
<tr>
<td>Main girders</td>
<td>5.9</td>
<td>1.2</td>
</tr>
<tr>
<td>Bracing</td>
<td>0.43</td>
<td>1.2</td>
</tr>
</tbody>
</table>

The design loads on the bridge for the serviceability limit state according to load combination 6.15b (equation 6.7) are shown in Table 6.7. The tandem load of LM2 is not included in the table due to that it is not decisive for the serviceability limit state.

Table 6.7: Design loads in the serviceability limit state according to load combination 6.15b

<table>
<thead>
<tr>
<th>Characteristic load</th>
<th>$\psi_{1,1}$</th>
<th>Design load (6.15b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$kN$</td>
<td>$-$</td>
<td>$kN$</td>
</tr>
<tr>
<td>TS 1</td>
<td>270</td>
<td>0.75</td>
</tr>
<tr>
<td>TS 2</td>
<td>180</td>
<td>0.75</td>
</tr>
<tr>
<td>UDL 1</td>
<td>6.3</td>
<td>0.40</td>
</tr>
<tr>
<td>UDL 2</td>
<td>2.5</td>
<td>0.40</td>
</tr>
<tr>
<td>UDL 3</td>
<td>2.5</td>
<td>0.40</td>
</tr>
<tr>
<td>Pavement</td>
<td>2.3</td>
<td>1.0</td>
</tr>
<tr>
<td>Parapet</td>
<td>1.5</td>
<td>1.0</td>
</tr>
<tr>
<td>Edge beam</td>
<td>2.5</td>
<td>1.0</td>
</tr>
<tr>
<td>Parapet</td>
<td>1.5</td>
<td>1.0</td>
</tr>
<tr>
<td>CLT slab</td>
<td>3.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Main girders</td>
<td>5.9</td>
<td>1.0</td>
</tr>
<tr>
<td>Bracing</td>
<td>0.43</td>
<td>1.0</td>
</tr>
</tbody>
</table>
7 Finite Element Modelling of the New Bridge Type Using RFEM

The new bridge type was modelled in RFEM, a finite element software developed by Dlubal. The program has a module called RF-Laminate that facilitates the modelling of cross-laminated timber slabs (Dlubal, 2013).

Material and geometric characteristics of the cross-laminated timber slab were chosen in accordance with the ETA-06/0138 (2012). Thereafter, the properties were added to the RF-Laminate module. The program then generated a stiffness matrix according to an orthotropic plate that represented the cross-laminated timber slab. As stated in the ETA-06/0138 (2012), the torsional stiffness was reduced by 50%. The stiffness matrix that was used in the finite element model is shown in equation C.9 of Appendix C.

The main girders and the bracing were modelled as beam elements. Since the CLT slab does not act in composite with the main girders, the slab was modelled as simply supported on the main girders' neutral axes. The reason to that the slab is not positioned on the top flanges of the main girders is that this would have caused the elements to act in composite. By positioning the slab on the main girders neutral axes the risk of later torsional buckling of the girders is underestimated. However, the main girders are stabilised by planar and cross bracing. Therefore, the risk of lateral torsional buckling of the main girders should be negligible.

The bracing members were connected to the main girders with hinges and set not to interact with the slab. Therefore, the bracing members are only subjected to normal forces. The connection between the main girders and the bracing was positioned at the intersection between the main girder’s flanges and web.

The main girders were modelled as simply supported wherefore they were free to rotate around their major axes. Since one support was restrained to move in its local xy-plane while the others were allowed to move in at least one horizontal direction, unwanted stresses at the bearings were avoided. Figure 7.1 shows how the surface, members and the four different supports were assembled in the finite element model. The local coordinate axes x’, y’ and z’ of the supports correspond to the global coordinate system xyz of the model. Support 2 was free to move in the direction of its x’-axis while support 3 was allowed to move in the direction of its y’-axis. Furthermore, support 1 was fixed and support 4 could move in its x’y’-plane.
Figure 7.1: Finite element model of the bridge

The loads presented in section 6.5 were imposed in accordance with the load models given in *SS-EN 1991-2 - Actions on structures part 2: Traffic loads on bridges*. Since the cross-laminated timber slab is covered with a pavement layer, stresses caused by wheel pressures were set to spread through the pavement and half of the slab according to section 6.2. Moreover, the surface was divided into a mesh consisting of 0.2 m wide quadratic elements. The load effects were calculated according to second-order analysis in order to account for second-order effects.
8 Ultimate Limit State Verification of the New Bridge Type

8.1 Cross-Laminated Timber

The design value, \( f_d \), of a specific strength property for timber is obtained by

\[
f_d = k_{\text{mod}} \cdot \frac{f_k}{\gamma_M}
\]

where \( f_k \) is the characteristic strength value, \( \gamma_M \) a material dependent partial factor and \( k_{\text{mod}} \) a factor that considers moisture and load duration (SS-EN 1995-1-1 - Design of timber structures part 1-1: General - Common rules and rules for buildings).

Depending on what type of actions the material is exposed to, different verifications have to be fulfilled. When considering a cross-laminated timber plate that is loaded perpendicular to its plane, the requirements according to equations 8.2 - 8.4 must be fulfilled.

- bending perpendicular to the plane of the slab:
  \[
  \sigma_{m,d} \leq f_{m,d} \cdot k_{\text{sys}}
  \]
  (8.2)

- shear perpendicular to the plane of the slab:
  \[
  \tau_{v,R,d} \leq f_{v,R,d} \cdot k_v
  \]
  (8.3)

- compression perpendicular to the grain:
  \[
  \sigma_{c,90,d} \leq f_{c,90,d} \cdot k_{c,90}
  \]
  (8.4)

where \( k_{\text{sys}} \) is a system strength factor, \( k_v \) a factor that considers notches and defects and \( k_{c,90} \) a factor that depends on support and load conditions (ETA-06/0138, 2012).

When considering shear stresses close to point or line supports, the verification can be made at a distance \( e = \frac{t_{\text{CLT}}}{2} \) from the edges of the supports (ETA-06/0138, 2012). The CLT slab is supported on the top flanges of the main girders. Therefore, shear forces are controlled a distance

\[
e = \frac{t_{\text{CLT}}}{2} + \frac{b_{f,\text{top}}}{2}
\]

(8.5)

from the girders main axis. Figure 8.1 illustrates how the principle works for a \( t_{\text{CLT}} \) thick slab resting on two girders acting as line supports.
The decisive load effects for the ultimate limit state design of the cross-section were calculated with the finite element model. The cross-laminated timber deck was controlled for bending moments, shear forces and compression perpendicular to the slab. The stresses that the load effects induced in the cross-section were verified according to equations 8.2 - 8.4. Moreover, the traffic loads were regarded as short-term loads and the slab is designed for service class 2, wherefore $k_{mod} = 0.9$ (SS-EN 1995-2 - Design of timber structures part 2: Bridges). Since the elements are wider than 160 cm, the system strength factor, $k_{sys}$, is 1.1 according to Table 2.7.

The finite element model of the decisive load case for bending moments and its corresponding moment distribution with regard to the CLT slab are shown in Figure 8.2. The blue squares corresponds to the tandem loads of load model 1. A more detailed sketch of the design load case can be seen in Figure 6.11. The design moment occurs in section A and is shown in Figure 8.3.

---

Figure 8.1: Locations where shear stresses are be verified

Figure 8.2: Bending moment distribution in the slab (top view left and section right)

Figure 8.3: Bending moments [kNm/m] along section A of Figure 8.2
As seen in Figure 8.3, the design bending moment is

\[ M_{Ed} = 314 \text{ kNm} \]

for a 1 m wide slab strip. Moreover, the bending stresses are obtained by

\[ \sigma_{m,d} = \frac{M_{Ed}}{W_{x,net}} \]  \hspace{1cm} (8.6)

where \( W_{x,net} \) is the CLT slab’s net section modulus in the bridge’s transverse direction. The section modulus was determined to

\[ W_{x,net} = 32.7 \cdot 10^6 \text{ mm}^3 \]

in equation C.5 of Appendix C, wherefore

\[ \sigma_{m,d} = \frac{314 \cdot 10^3}{32.7 \cdot 10^{-3}} = 9.60 \text{ MPa}. \]  \hspace{1cm} (8.7)

Since

\[ f_{m,d,slab} = 19.0 \text{ MPa} \]

according to equation C.11 of Appendix C, equation 8.2 becomes

\[ \sigma_{m,d} = 9.60 \text{ MPa} \leq 19.0 \text{ MPa}. \]

The maximum shear force is obtained when the loads are placed as in Figure 8.4 (see Figure 6.10). Moreover, the design shear force occurs in section B as seen in Figure 8.5.

![Figure 8.4: Shear force distribution in the slab (top view left and section right)](image)

![Figure 8.5: Shear forces [kN/m] along section B of Figure 8.4](image)
The shear forces are most adverse close to the supports. However, the design value is determined at a distance

\[ e = \frac{500}{2} + \frac{500}{2} = 500 \text{ mm} \]  

from the flange according to equation 8.5. Therefore, the design shear force becomes

\[ V_{Ed} = 221 \text{ kN} \]

for a 1 m wide slab strip. Shear stresses over the cross-section are calculated according to

\[ \tau_{v,d} = \frac{V_{Ed} \cdot S_{x,net}}{I_{x,net} \cdot b} \]  

where \( b \) is the width of the cross-section. The moment of inertia and first moment of area, i.e. \( I_{x,net} \) and \( S_{x,net} \) respectively, are calculated according to equations C.3 and C.7 of Appendix C. Since rolling shear of cross-layers is decisive when considering shear of cross-laminated timber elements, only shear stresses over cross-layers are calculated. As the shear stresses are constant in these regions, it is sufficient to determine the design value at the boundary between \( C_2 \) and \( L_4 \) according to Figure 8.6.

![Figure 8.6: Cross-section of the CLT slab with layer notations](image)

With cross-sectional values according to Appendix C known, equation 8.9 becomes

\[ \tau_{v,R,d} = \frac{221 \cdot 10^3 \cdot 21.2 \cdot 10^{-3}}{8181 \cdot 10^{-6} \cdot 1.0} = 0.572 \text{ MPa}. \]  

(8.10)

Considering rolling shear stresses caused by concentrated loads acting in the mid-third of the slab’s span, the rolling shear strength is

\[ f_{v,R,d,slab} = 0.576 \text{ MPa} \]

according to equation C.15 of Appendix C. As shown in Figure 2.4, the rolling shear strength is somewhat higher for concentrated loads acting outside the mid-third of the span. However, the total rolling shear stress was verified against the lower strength value as a conservative assumption. Therefore, equation 8.3 is satisfied according to

\[ \tau_{v,R,d} = 0.572 \text{ MPa} \leq 0.576 \text{ MPa}. \]  

(8.11)
The slab also has to be verified for compression perpendicular to the grain. The control is performed for compressive stresses at the supports, i.e. for the contact stresses between the slab and the top flange of the main girders, as well as for wheel pressure. When determining compressive stresses at supports, the reaction force is obtained by calculating the difference between the shear forces on either side of the girder. The greatest reaction force occurs when the loads are placed as in Figure 8.7 (see also Figure 6.13) and the corresponding shear force distribution for section C is shown in Figure 8.8.

![Figure 8.7: Shear force distribution in the slab (top view left and section right)](image)

Therefore, the reaction force becomes

$$R_{c,Ed} = 252.98 + 161.49 = 414 \text{ kN/m}. \quad (8.12)$$

The force is assumed to act on an 1 meter long part of the flange, wherefore the contact area between the girder and the slab equals

$$A_{c,90} = 1 \cdot b_{f,top} = 0.5 \text{ m}^2. \quad (8.13)$$

The design compressive stress for a 1 m long part of the flange at the support equals

$$\sigma_{c,90,d} = \frac{R_{c,Ed}}{b_{f,top}} = \frac{414 \cdot 10^3}{0.5} = 0.83 \text{ MPa}. \quad (8.14)$$
Moreover, the design strength was determined to

\[ f_{c,90,d} = 1.94 \text{ MPa} \]

according to equation C.18 of Appendix C. Hence, equation 8.4 is fulfilled;

\[ \sigma_{c,90,d} = 0.83 \text{ MPa} \leq 1.94 \cdot 2.2 = 4.28 \text{ MPa} \]

(8.15)

where \( k_{c,90} = 2.2 \) since loads are acting close to the end of the element (ETA-06/0138, 2012, p.26).

When checking wheel pressure, load model 2 of *SS-EN 1991-2 - Actions on structures part 2: Traffic loads on bridges* is used. According to Table 6.6, the design value of the tandem load is equal to

\[ Q_{ad} = 540 \text{ kN}. \]

Furthermore, the contact area of one wheel is

\[ A_{LM2,1} = 0.35 \cdot 0.60 = 0.21 \text{ m}^2. \]

(8.16)

By considering the load dispersion through the 100 mm thick pavement layer, the contact area at the level of the cross-laminated timber slab becomes

\[ A_{LM2,2} = (0.6 + 2 \cdot 0.1) \cdot (0.35 + 2 \cdot 0.1) = 0.44 \text{ m}^2. \]

(8.17)

Since the self-weight of the pavement is negligible, the design wheel pressure is determined by

\[ \sigma_{c,LM2,90,d} = \frac{Q_{ad}/2}{A_{LM2,2}} = \frac{540 \cdot 10^3/2}{0.44} = 0.61 \text{ MPa} \]

(8.18)

wherefore equation 8.4 is satisfied according to

\[ \sigma_{c,LM2,90,d} = 0.61 \text{ MPa} \leq 4.28 \text{ MPa}. \]

(8.19)
8.2 Main Girders

The main girders are exposed to bending in their longitudinal direction and shear over their height. The design yield strengths related to different steel resistances are calculated according to

\[ f_{yd} = \frac{f_y}{\gamma_{Mi}} \]  

(8.20)

where \( \gamma_{Mi} \) is a partial factor, that corresponds to the specific resistance. The verification of the design shear force, \( V_{Ed} \), has to fulfill

\[ \frac{V_{Ed}}{V_{c,Rd}} \leq 1.0. \]  

(8.21)

In case of slender cross-sections, the shear buckling resistance, \( V_{b,Rd} \), must be considered wherefore

\[ \frac{V_{Ed}}{V_{b,Rd}} \leq 1.0. \]  

(8.22)

also must be satisfied. The shear buckling resistance is determined by

\[ V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \eta \cdot f_{yw} \cdot h_w \cdot t \frac{\sqrt{3}}{\gamma_{M1}} \]  

(8.23)

where \( V_{bw,Rd} \) and \( V_{bf,Rd} \) are the buckling resistances of the web and the flange respectively. If the web is very slender, web breathing must also be taken into account. Moreover, a reduction of the bending moment resistance is made if the design shear force exceeds 50% of the design resistance. According to SS-EN 1993-1-5 - Design of steel structures part 1-5: Plated structural elements, interaction between shear forces and bending moments should be considered if

\[ \tilde{\eta}_3 = \frac{V_{Ed}}{V_{bw,Rd}} \leq 0.5 \]  

(8.24)

for slender cross-sections. When considering bending, following requirement should be fulfilled:

\[ \frac{M_{Ed}}{M_{c,Rd}} \leq 1.0. \]  

(8.25)

If the beam is slender and lateral movements are not prevented, the bending resistance should be reduced by a factor, \( \chi_{LT} \), which takes lateral torsional buckling into account. In case of lateral torsional buckling, the bending capacity is limited by

\[ \frac{M_{Ed}}{M_{b,Rd}} \leq 1.0. \]  

(8.26)

where \( M_{b,Rd} \) is the lateral torsional buckling resistance. Before the CLT elements are connected to the flanges of the main girders, the main girders are not restrained from lateral movements. Therefore, the new bridge type should be controlled for lateral torsional buckling during the construction phase.
Design values with regard to the maximum shear force and bending moment are obtained when the loads are placed according to Figures 6.13 and 6.14. The load case that results in maximum shear force, as used in the finite element model, and the corresponding load effects are shown in Figures 8.9 and 8.10.

![Figure 8.9: Load placement with respect to the design shear force on the main girder (top view left and section right)](image)

As seen in Figure 8.10, the maximum design shear force is

\[ V_{Ed} = 1883 \text{ kN}. \]

Since the web is slender, a control is made to check whether shear buckling should be considered

\[ \frac{h_w}{t_w} > 72 \cdot \frac{\varepsilon}{\eta} \quad (8.27) \]

where

\[ \varepsilon = \sqrt{\frac{235}{f_{y,w}}} \quad f_{y,w} = 345 \text{ MPa for the web and } \eta = 1.2 \text{ for steel grades up to S460.} \]

Equation 8.27 results in

\[ \frac{1550}{17} = 91.2 > 72 \cdot \sqrt{\frac{235}{345}} = 49.5 \]

wherefore the shear buckling resistance will be decisive.
If shear resistances from flanges are neglected, equation 8.23 equals

\[ V_{b,Rd} = V_{bw,Rd} = 3686 \text{ kN} \]

according to equation B.13 of Appendix B. Therefore, the shear buckling resistance is sufficient in agreement with equation 8.22:

\[
\frac{V_{Ed}}{V_{b,Rd}} = \frac{1883}{3686} = 0.51 \leq 1.0. \tag{8.28}
\]

Moreover, interaction between shear forces and bending moments should be considered since

\[ \bar{\eta}_3 = \frac{1883}{3686} = 0.51 > 0.50 \tag{8.29} \]

according to equation 8.24. However, since the girder is simply supported, large shear forces coincide with small bending moments and vice versa. Therefore, the interaction between shear and bending can be disregarded.

Moreover, fatigue related to web breathing is checked according to

\[ \frac{h_w}{t_w} \leq \min [30 + 4L; 300] \tag{8.30} \]

where \( L \) corresponds to the total span length of the bridge (SS-EN 1993-2 - Design of steel structures part 2: Steel bridges). With \( L = 26 \text{ m} \), equation 8.30 becomes

\[
\frac{1550}{17} = 91.2 \leq \min [134; 300] = 134 , \tag{8.31}
\]

wherefore no consideration is needed for web breathing.

The maximum bending moment occurs when the loads are placed according to Figure 8.11 and the corresponding moment distribution is shown in Figure 8.12. This corresponds to the load case shown in Figures 6.13 and 6.14.

---

**Figure 8.11:** Load placement with respect to design bending moment of main girder (top view left and section right)
It can be noted that there are some cuts in the bending moment curve. The cuts in the bending moment curve occurs at the position of the cross bracing. It is believed that the cuts are caused by the increased stiffness of the main girders due to the bracing system.

The maximum design bending moment is

\[ M_{Ed} = 10875 \text{ kNm} \]

in accordance with Figure 8.12. Since the cross-laminated timber deck stabilizes the top flanges of the girders, there is no risk of lateral torsional buckling when the bridge is in service. Therefore, the bending resistance calculated according to equation B.10 of appendix B should not be reduced. The bending resistance during service life was determined to

\[ M_{c,Rd} = 15140 \text{ kNm} , \]

wherefore equation 8.25 is fulfilled according to

\[ \frac{M_{Ed}}{M_{c,Rd}} = \frac{10875}{15140} = 0.72 \leq 1.0. \] (8.32)

Moreover, a control should be made that considers the bridge during its construction phase. During construction, the deck does not prevent lateral torsional buckling of the girders. However, the bracing system provides lateral support every 3.25 m along the girders. The elastic critical moment for lateral torsional buckling and the corresponding slenderness factor, \( M_{cr} \) and \( \bar{\lambda}_{LT} \) respectively, are calculated in equations B.19 and B.20 of Appendix B. Because

\[ \frac{h_w}{b_f} = \frac{1550}{500} = 3.1 > 2 , \] (8.33)

buckling curve \( d \) should be used in Figure 6.4 of *SS-EN 1993-1-1 - Design of steel structures part 1-1: General rules and rules for buildings* when determining the reduction factor \( \chi_{LT} \) for a welded I-section. As \( \bar{\lambda}_{LT} = 0.0096 < 0.2 \), no reduction of the moment capacity has to be done. Since no major live loads are present during the construction of the bridge, the highest load effects will occur when the bridge is in service. Therefore, no verifications during the construction phase of the bridge are presented.
8.3 Steel Bracing

The bracing system consists of angle-sections according to section B.2 of Appendix B. Since the bracing members are connected to the main girders with hinges, they will not be subjected to any significant lateral loads. Consequently, only tensile and compressive forces are verified.

The design values when considering tension and compression should satisfy

\[
\frac{N_{t,Ed}}{N_{t,Rd}} \leq 1.0 \quad \text{and} \quad \frac{N_{c,Ed}}{N_{c,Rd}} \leq 1.0
\]

according to \textit{SS-EN 1993-1-1 - Design of steel structures part 1-1: General rules and rules for buildings}. In case of slender cross-sections, the design resistance with respect to compression should be reduced by a factor \( \chi \) in order to account for the risk of buckling. The verification with respect to buckling is

\[
\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0.
\]

Furthermore, since the angle-sections are in cross-section class 4 according to section B.2 of Annex B, an effective area should be used when evaluating their buckling resistance. However, as shown in equation B.22 of Appendix B, the effective area is equal to the entire area of the cross-section.

Both maximum tensile and compressive forces occur when the load is placed according to Figure 8.13. Furthermore, Figure 8.14 shows the heaviest loaded elements as a result of the load placement. Since only tensile and compressive forces are acting on the elements, the internal force distributions will be constant along their lengths.

---

Figure 8.13: Load placement with respect to the maximum tensile and compressive force in the angle-sections (top view left and section right)
The maximum tensile and compressive forces were determined to

\[ N_{t,Ed} = 271 \text{ kN} \quad ; \quad N_{c,Ed} = -270 \text{ kN}. \]

Correspondingly, the resistances of the members are

\[ N_{t,Rd} = 1967 \text{ kN} \quad ; \quad N_{c,Rd} = 1967 \text{ kN} \]

according to equations B.26 and B.27 of Annex B. Hence,

\[ \frac{N_{t,Ed}}{N_{t,Rd}} = \frac{271}{1967} = 0.14 \leq 1.0 \quad \text{and} \quad \frac{N_{c,Ed}}{N_{c,Rd}} = \frac{270}{1967} = 0.14 \leq 1.0. \quad (8.36) \]

When evaluating the buckling strength, a reduction factor is calculated according to equation B.29 of Annex B. The buckling strength was determined to

\[ N_{b,Rd} = 322 \text{ kN} \]

in equation B.28, wherefore the buckling criteria is satisfied;

\[ \frac{N_{Ed}}{N_{b,Rd}} = \frac{270}{322} = 0.84 \leq 1.0. \quad (8.37) \]
9 Serviceability Limit State Verification of the New Bridge Type

In the serviceability limit state, the bridge is controlled for deflections due to the frequent load combination 6.15b. The bridge needs to be verified for deflections in the bridge’s longitudinal and transverse direction. The requirements are that the deflections should be smaller than $l/400$ for simply supported structures and $l/200$ for cantilevered structures (TRVFS2011:12, 2011). The transverse deflections of the deck have to be verified for both local and global deflections relative to the deflected normal plane. The 2.4 m long side part of the slab is regarded as a cantilever and is thus verified for $l/200$. The notations when considering the deflections of the bridge are shown in Figure 9.1 while the maximum allowed deflections are shown in Table 9.1.

![Figure 9.1: Notations to the deflections of the deck](image)

<table>
<thead>
<tr>
<th>$l_{\text{long}}$ [mm]</th>
<th>$u_{d,\text{long}}$ [mm]</th>
<th>$l_{\text{edge}}$ [mm]</th>
<th>$u_{d,\text{edge}}$ [mm]</th>
<th>$l_{\text{span}}$ [mm]</th>
<th>$u_{d,\text{span}}$ [mm]</th>
<th>$l_{\text{tot}}$ [mm]</th>
<th>$u_{d,\text{tot}}$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>26 000</td>
<td>65</td>
<td>2400</td>
<td>12</td>
<td>5200</td>
<td>13</td>
<td>10 000</td>
<td>25</td>
</tr>
</tbody>
</table>

The decisive load case for the deflections of the bridge was shown in Figures 6.13 and 6.14. The bridge did not require any bracing in the ultimate limit state. However, if the main girders were not braced, the deflections became too big (see Figure 9.2 - 9.3 and Table 9.2).

![Figure 9.2: Finite element model of the deflections of the bridge without bracing](image)
Table 9.2: The deflections of the deck at the midspan of the bridge without bracing

<table>
<thead>
<tr>
<th>$u_{long}$ [mm]</th>
<th>$u_{edge}$ [mm]</th>
<th>$u_{span}$ [mm]</th>
<th>$u_{tot}$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>75.1</td>
<td>15.7</td>
<td>24.9</td>
<td>51.9</td>
</tr>
</tbody>
</table>

To reduce the deflections, it was tried to increase the height of the main girders. It was found that the girders would need to be almost 3.0 m high to fulfil the serviceability requirements. This would yield an impractically high beam. Instead, the original girder height of 1650 mm was maintained and an x-type cross-bracing was introduced. Three different cross-brace spacings were tried out; 13 m, 6.5 m and 3.25 m. The finite element model of the bridge with cross-bracing spaced 6.5 m are shown in Figure 9.4 and 9.5. The total transverse deflection $u_{long}$ and $u_{tot}$ as a function of the spacing of the cross-bracing are shown in Figure 9.6.

Figure 9.3: The deflections [mm] of the deck at midspan of the bridge without bracing

Table 9.2: The deflections of the deck at the midspan of the bridge without bracing

Figure 9.4: Finite element model of the bridge with cross-bracing spaced 6.5 m
Cross-bracing did not reduce the deflections enough. Furthermore, if Figure 9.5 is considered, it can be noted that there is some torsion of the deck. Planar-bracing at the top and bottom of the main girders were added to reduce the deflections and the torsion. The combination of planar- and cross-bracing creates a virtual box-section with large torsional stiffness (Lebet and Hirt, 2013). The finite element model of the bridge with cross-bracing spaced 3.25 m is shown in Figure 9.7. The total longitudinal and transverse deflections as a function of the planar and cross-brace spacing are shown in Figure 9.8.
It was found that a planar and cross-brace spacing of either 6.5 m or 3.25 m provided sufficient reduction of the deflections (see Figure 9.8). However, the planar bracing members became slender for a spacing of 6.5 m and were prone to buckle in the ultimate limit state. Therefore, it was considered more feasible to use a planar and cross-brace spacing of 3.25 m. The deflections of the bridge with planar and cross-bracing spaced 3.25 m are shown in Figures 9.9 - 9.10 and Table 9.3.
Timber bridges can be prone to vibrate, causing inconvenience for the users. To avoid resonance effects, it is interesting to determine the eigenfrequency of the first eigenmodes. The first bending mode occurs at 4.5 Hz and is shown in Figure 9.11. The first torsion mode occurs at 7.5 Hz and is shown in Figure 9.12.

Table 9.3: Deflections of the deck at the midspan of the bridge with planar and cross-bracing spaced 3.25 m

<table>
<thead>
<tr>
<th>$u_{long}$ [mm]</th>
<th>$u_{edge}$ [mm]</th>
<th>$u_{span}$ [mm]</th>
<th>$u_{tot}$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>61.7</td>
<td>8.7</td>
<td>8.9</td>
<td>20.5</td>
</tr>
</tbody>
</table>
There is no fixed requirement for resonance effects for road bridges in Eurocode. However, it is stated that the vibrations should be limited to avoid nuisance for the users ([SS-EN 1990 - Basis of structural design](https://www.wiley.com/en-us)). For pedestrian bridges, Eurocode states that if the first bending mode and the first transverse bending or torsion mode occurs at lower frequencies than 5 Hz and 2.5 Hz respectively, the comfort criteria should be verified ([SS-EN 1990 - Basis of structural design](https://www.wiley.com/en-us)). The first requirement were not met by the new bridge type. Since the eigenfrequency criteria do not apply for road bridges, no further verifications were made.
10 Cost Estimate of the New Bridge Type and the Reference Bridge

The design of the new bridge type was limited to the main structural elements of the superstructure. Therefore, the cost estimate of the new bridge type was also limited to the superstructure. The cost was appraised for the work and the material related to the construction of the main girders, the bracing, and the cross-laminated timber slab.

The cost of the CLT elements was based on a cost estimate provided by Habenbacher (2015). The cost of the elements include material, manufacturing, trimming and shipping to a construction site in Malmö of southern Sweden. The cost of mounting the CLT slab on the main steel girders were determined with the help of Oskar Bruneby at Peab. It was estimated that it would be necessary for three construction workers to work with the CLT slab for two full weeks. Furthermore, it was assumed that a mobile crane with a capacity of 7.5 tons and a range of 25 m would be required during the entire period. The cost of the mobile crane was assumed to 100 000 SEK. The cost of the CLT slab does not include the cost of the material for connections, weather protection and edge beams.

The cost of the steel structure of the new bridge type was based on a tender calculus of a 42 m long concrete composite bridge provided by Bruneby (2015). The price was based on the weight of the steel construction, and it includes the cost of material, assembling, painting and launching. Unfortunately, the cost of the steel structure also includes shear connectors. Shear connectors are not used for the new bridge type, but since the price was given for the total steel structure, the cost of the shear connectors could not be subtracted. However, these extra costs can to some extent be assumed to compensate for the costs of weather protection, edge beams and other details that were not included.

The cost of the superstructure of the reference bridge was based on the above-mentioned tender calculus. The cost of the steel structure was calculated according to the same procedure as for the new bridge type. The cost of the concrete slab was based on the volume of the concrete slab, and it includes the cost of casting, falsework, and edge beams.

The cost of the superstructure of the new bridge type and the reference bridge were compared for both their total cost and their unit cost. The unit cost was determined as the cost of the superstructure divided with the free area of the carriageway. The free area of the carriageway is the free width of the carriageway multiplied with the bridge’s theoretical span length.

The estimated cost of the superstructure of the new bridge type and the reference bridge are shown in Table 10.1. For a comprehensive presentation of the calculation of the costs, the reader is referred to Appendix D.
Table 10.1: Estimated cost of the superstructure of the new bridge type and the reference bridge

<table>
<thead>
<tr>
<th></th>
<th>Cost of the deck [SEK]</th>
<th>Cost of the steel structure [SEK]</th>
<th>Total cost [SEK]</th>
<th>Unit cost [SEK/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>New bridge type</td>
<td>800 000</td>
<td>1 240 000</td>
<td>2 040 000</td>
<td>8700</td>
</tr>
<tr>
<td>Reference bridge</td>
<td>680 000</td>
<td>700 000</td>
<td>1 380 000</td>
<td>5400</td>
</tr>
</tbody>
</table>

The cost of the superstructure of the reference bridge type is 68% of the cost of the superstructure of the new bridge type. The main reason for the new bridge type being more expensive is that much more steel was used. For instance, the weight of the bracing of the reference bridge is only 25% of the weight of the bracing of the new bridge type (see Table 10.2). Furthermore, the main girders of the new bridge type weigh almost 10 tons more than the main girders of the reference bridge.

Table 10.2: The weight of the substructure of the bridges. The weights are rounded to nearest whole numbers

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>New bridge type</td>
<td>31</td>
<td>13</td>
<td>44</td>
<td>62</td>
<td>106</td>
</tr>
<tr>
<td>Reference bridge</td>
<td>21</td>
<td>3</td>
<td>25</td>
<td>227</td>
<td>252</td>
</tr>
</tbody>
</table>
11 Analysis and Discussion of the Behaviour of the Proposed Bridge and its Feasibility

11.1 Design and Analysis of Cross-laminated Timber Slabs

The cross-laminated timber slab of the new bridge type was designed according to the ETA approval of KLH’s elements. KLH has produced a commented version of the ETA where different aspects of the design procedure are discussed. The commented ETA provides comprehensive information for how KLH’s elements should be used. When CLT slabs are used as floors for buildings and houses, the ETA is reasonably straightforward. However, when CLT elements are used for bridges, the load situation becomes more extreme. Large concentrated loads are present, and non-standard cross-sections could be required. For such applications, the ETA can be difficult to interpret. Especially, the formula that is suggested for the calculation of the shear modulus of a generic CLT slab is complicated to apply. Therefore, the elements in the stiffness matrix that corresponds to the shear stiffness of a slab are difficult to determine. Although, it should be noted that KLH’s ETA was originally written in German. However, for this project the translated English version was used.

The RF-LAMINATE module to the finite element software RFEM facilitated a user-friendly modelling of the cross-laminated timber slab. It was easy to model and control different cross-section lay-ups. However, since some of the exceptions and special cases that are stated in the ETA-06/0138 (2012) are not implemented in RF-LAMINATE, it is recommended that verification of stresses is done manually.

There are some uncertainties regarding the calculation of stiffness matrices of CLT elements. KLH provides a software, KLH-design, where the different stiffness matrices of their standard elements are presented. Furthermore, the technical university of Graz has developed a similar software, CLT-designer, which can calculate the stiffness matrix for CLT elements from several different manufacturers, and perform simple design calculations. If the stiffness matrix for a given cross-section was determined by RF-LAMINATE, KLH-design and CLT-designer, the elements corresponding to the shear stiffness varied slightly. There is a lack of transparency of how these elements should be calculated. The attempts to replicate the stiffness matrices of the different programs often resulted in deviations from the original. In most cases, the shear stiffness calculated by RF-LAMINATE and CLT-designer conformed. These values were somewhat lower than for KLH-design. For illustrative purposes, the value of the shear stiffness in the load-bearing direction of a KLH 120 slab element as calculated by all of the software are shown in Table 11.1. The total thickness of the element is 120 mm, and it consists of three 40 mm thick layers, and the central layer is a cross-layer. Unfortunately, the 500 mm thick element is not available in KLH-design and CLT-designer. Therefore, the much simpler KLH 120 element was chosen for the illustration instead.
Table 11.1: Comparison of the shear stiffness in the load-bearing direction of a KLH 120 slab

<table>
<thead>
<tr>
<th></th>
<th>KLH design [kN/m]</th>
<th>CLT designer [kN/m]</th>
<th>RF-LAMINATE [kN/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$G_{Ax}$</td>
<td>11 000</td>
<td>8800</td>
<td>8800</td>
</tr>
</tbody>
</table>

For this project, the stiffness matrix of the CLT slab, as determined by RF-LAMINATE, was used. This choice should be conservative.

11.2 The New Bridge Type

The new bridge type managed to fulfil the requirements for traffic loads in the ultimate limit state and the serviceability limit state. However, planar and cross-bracing were necessary to limit deflections. The bracing adds a considerable amount of steel to the bridge and increases its cost.

11.2.1 The Cross-laminated Timber Slab

The utilization rate of the cross layers’ rolling shear strength is 99%. However, even though the rolling shear capacity of the cross-section almost was reached, it may be possible to make the cross-section more slender. There is ongoing research that indicates that the rolling shear strength of cross-layers increase when the slab is subjected to compressive concentrated loads. Since concentrated loads are included in the load models of SS-EN 1991-2 - Actions on structures part 2: Traffic loads on bridges, the rolling shear strength of the cross-laminated timber cross-section may be undervalued.

Moreover, the rolling shear strength of the cross-layers is either 0.8 MPa or 1.2 MPa. The lower value of 0.80 MPa only has to be used for cross-layers that are thicker than 45 mm, and for the part of the shear stress that is caused by concentrated loads acting in the mid-third of the slab’s span. Otherwise, the higher characteristic rolling shear strength of 1.2 MPa may be used. If load dispersion through the slab is included, the concentrated loads of load model 1 acts on an area of 1.41 m² per wheel. Hence, in some cases parts of the shear force caused by the load from one wheel could be verified against different rolling shear strength values. The result was that it became very difficult to determine the most severe load case for rolling shear. Therefore, as a conservative assumption the lower strength of 0.8 MPa was used for the rolling shear verification for all loads.

If the bending resistance of the cross-laminated timber slab is considered, it has some excess capacity. The utilization rate of the bending moment capacity is 51%. The excess capacity is caused by the large height of the cross-section and the numerous longitudinal layers. However, this design was necessary to ensure the shear force resistance of the cross-section. Many other designs were tested but could not fulfil the ultimate limit state requirements.
Furthermore, the timber proved to have sufficient strength against compression perpendicular to the grain. The largest compression occurred at the support of the slab on the top flanges of the main girders. The utilization rate of the compressive strength is 19%.

11.2.2 The Main Girders

As expected, the lack of composite action in the new bridge type required that the main girders had to be bigger than in the reference bridge. The main girders of the new bridge type have a total weight of 31.2 tons whereas the main girders of the reference bridge have a total weight of 21.4 tons, a relative difference of 46%. However, it should be noted that the main girders of the reference bridge are optimised so that they are more slender at the outer sections. If the area of the cross-sections at midspan of the bridges are compared, the relative difference is only 31%. Hence, if the main girders of the new bridge type were optimised, it is likely that a considerable amount of material could be saved.

11.2.3 The Bracing System

The bracing system added 12.8 tons of steel to the new bridge type. Steel is expensive, and the extra bracing led to increased costs. Since the cross-beams of the reference bridge only weighs 3.25 tons together, its bracing system is much less expensive. Moreover, the bracing system of the new bridge needs more connections than the bracing system of the reference bridge, wherefore the cost differences might be even bigger.

It is believed that the reference bridge required less bracing because its superstructure is much heavier than the superstructure of the new bridge type (252 tons respectively 106 tons). The live load corresponds to a smaller part of the total load on a heavy bridge than for a light one. Therefore, an uneven loading of the girders, due to the live loads on the reference bridge, will not impact the transverse deflections of the deck as much as for the new bridge type. Furthermore, the concrete deck of a composite bridge facilitates some load distribution between the main girders. However, the load distribution is more pronounced for multi-girder bridges than for twin girder bridges.

It should also be noted that the reference bridge’s main girders have a pre-camber of 100 mm. A pre-camber could reduce some of the longitudinal deflections of the main girders of the new bridge type. However, it is the uneven deflections of the main girders that cause problems with transverse deflections. A pre-camber of the main girders would not solve the problem with the transverse deflections.

To save material, it could be possible to use the cross-laminated timber slab for planar stabilization, and eliminate the top planar steel bracing. If the CLT slab was used as top planar bracing, a stiff connection between the steel girders and the CLT slab would be necessary. However, in this thesis the CLT slab was modelled as simply supported on the steel girders. Therefore, the connection between the slab and the main girders should restrict transverse movements and allow rotations. Since the slab was not modelled for composite action with the girders, the connection should also allow some movements in the bridges longitudinal direction.
11.3 Cost Evaluation of the New Bridge Type

The cost of the structural elements of the superstructure of the new bridge type is almost 50% higher than for the reference bridge. The main reason is that much more steel is used for the new bridge type. Thus, if the bridges are evaluated solely for the cost of their superstructure, the steel-concrete composite bridge is the better option. However, the economic legitimacy of the new bridge type is not only inherent to the cost of its superstructure. That the new bridge would require more steel was expected. What the new bridge type facilitates, is the possibility to erect it in a short time. It was estimated that it would take two weeks to erect the superstructure of the new bridge type. In comparison, an experienced bridge constructor, Oskar Bruneby at Peab (personal communication, April 14, 2015), states that it could take up to three months to build the reference bridge. A prolonged obstruction of the traffic flow during the construction of a bridge over a road or railway could be related to high social costs. The gain in construction time of the new bridge type could in some cases probably outweigh the higher cost of its superstructure. Furthermore, a long construction time ties personnel for the contractor. If less time is needed for the erection of the bridge, more projects could be carried out during the same construction time that would have been required for the reference bridge. However, these conditions are very difficult to quantify. Especially, since they are dependent on many different variables, such as the current financial situation of the contractor and the client, the geographic location of the project, and the socio-economical importance of the specific project.

Lebet and Hirt (2013) state that the superstructure of a steel and concrete composite bridge represents 40-60% of the total construction cost of the bridge (see Table 3.2). Furthermore, the substructure of a composite bridge represents 25-40% of the total construction cost (Lebet and Hirt, 2013). If it is assumed that the cost of the superstructure and the substructure corresponds to 60% and 40% respectively of the total construction cost of the reference bridge, the combined cost of the superstructure and the substructure would be

$$1.38 + \frac{1.38}{0.6} \cdot 0.4 = 1.38 + 0.92 = 2.30 \text{ million SEK}.$$

The weight of the superstructure of the new bridge type is about 42% of the weight of the reference bridge’s superstructure. If it is further assumed that the cost of the substructure of a bridge is directly proportionate to the weight of the bridge’s superstructure, then the combined cost of the superstructure and the substructure of the new bridge type would be

$$2.0 + 0.92 \cdot 0.42 = 2.0 + 0.39 = 2.39 \text{ million SEK}.$$

According to the calculations above, the cost of the reference bridge and the new bridge type would be almost equal. However, the example does not presume to be scientifically accurate. It was only made to illustrate that the cost of the bridges can be comparable, based exclusively on the cost of their main components.

Moreover, the weight of the superstructure of a bridge affects the cost of some of its other components. For instance, the bearings of a bridge constitute one of its most expensive components. Since the new bridge type is lighter than the reference bridge,
the substructure of the new bridge will be less heavily loaded than the substructure of the reference bridge. Therefore, one may assume that the costs associated with the substructure of the new bridge will be lower than the corresponding costs of the reference bridge.

Moreover, considerable savings could be made for the new bridge type if the top planar bracing were eliminated, and the main girders were optimized along their length. It should also be noted that the main girders of the reference bridge and the bridge that the cost estimate was based on have shear connectors welded to their top flanges. Furthermore, their main girders were made out of steel of grade S460M. Whereas, the main girders of the new bridge type lack shear connectors and were made out of steel of grade S355M. Therefore, the cost of the main girders of the new bridge type may be overestimated.

The unit cost of the superstructure of the new bridge type and the reference bridge were 8700 $SEK/m^2$ and 5400 $SEK/m^2$ respectively. If the cost comparison of different bridge types in Norway, mentioned in section 3.4, is considered, the cost of the bridges was presented as the unit cost of the entire bridges in $NOK$. To facilitate a comparison with the new bridge type, the costs have to be converted to the cost of the superstructure in $SEK$. If it is assumed that the cost of the superstructure corresponds to 60% of the bridges’ total cost and that 1 $NOK = 1.1 SEK$ (Oanda, 2015a), then the unit cost of the superstructure of a built-up steel girder bridge and a steel box girder bridge are 13 200 $SEK/m^2$ and 9900 $SEK/m^2$ respectively. Moreover, the cost of the superstructure for a glulam timber slab bridge and a concrete slab bridge are 9240 $SEK/m^2$ and 7920 $SEK/m^2$ respectively. Hence, in comparison to the cost of the superstructure of the Norwegian bridges, the new bridge type is competitive. It should be noted that the relative cost of the superstructure was assumed to the higher value of 60%. However, even if the lower value of 40% was used, the unit cost of the bridges’ superstructures would still be comparable.

Finally, it should be noted that the new bridge type is 10 m wide and was designed with three notional lanes. It is possible that the new bridge type is more competitive for narrower bridges with fewer notional lanes.
12 Conclusions

It can be concluded that it is possible to build road bridges with a deck of cross-laminated timber supported on steel girders. Cross-laminated timber has the favourable characteristic that its cross-section can be designed to suit the requirements of a specific project. Therefore, a cross-laminated timber slab can be designed to resist the large shear forces and bending moments that are induced on the slab by the traffic loads.

The estimated cost of the cross-laminated timber slab of the new bridge type was comparable to the cost of the concrete deck of a composite bridge. The disadvantage of the new bridge type was that the main girders and the bracing required more steel than for a composite bridge. Therefore, the cost of the structural elements of the superstructure of the new bridge type were higher than for the reference bridge. However, the new bridge type is fast to erect, and given the right conditions; the new bridge type may be cost competitive.
13 Suggestions for Further Research

In this thesis, it was shown that it is possible to build bridges with a slab of cross-laminated timber. However, to ensure the durability of the structural components of the new bridge type it is essential to provide them with reliable weather protection. Therefore, further research should be focused on designing a suitable weather protection system. A good weather protection reduces the extent of required maintenance, and it can lower the life-cycle cost of the bridge.

The design of details, such as parapets, the connection of the cross-laminated timber elements to the main girders, and the interconnection between the CLT elements, were discussed briefly in this thesis. It is likely that the appeal of the new bridge type would increase if a rational and cost-effective design were provided for these details.

Moreover, there is ongoing research that indicates that the rolling shear strength of cross-laminated timber slabs could increase for concentrated compressive loads. If the research proves to be successful and the findings are implemented in the approval of the CLT products, it could be possible to make the CLT slab of the bridge more slender and less expensive.

Finally, the cost of the superstructure of the new bridge type became large due that a lot of steel was used for the main girders and the bracing. Therefore, the competitiveness of the new bridge type would improve if its steel structure were optimized.
14 References


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Appendices

A  Bending Moment Capacities of Composite and CLT Bridges

A.1  Composite Bridge

The example composite bridge is shown in Figure A.1. Moreover, the notations when considering the composite cross-section are shown in Figure A.2 and the dimensions of the main girders are shown in Table A.1. To facilitate calculations, the bending moment capacity is determined for half of the bridge’s width.

![Figure A.1: Cross-section of composite bridge](image1)

![Figure A.2: Notations with respect to the composite bridge](image2)
Table A.1: Dimensions of the main girders

<table>
<thead>
<tr>
<th>$h$ [mm]</th>
<th>$t_{f,top}$ [mm]</th>
<th>$b_{f,top}$ [mm]</th>
<th>$t_w$ [mm]</th>
<th>$h_w$ [mm]</th>
<th>$t_{f,bot}$ [mm]</th>
<th>$b_{f,bot}$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1350</td>
<td>30</td>
<td>500</td>
<td>17</td>
<td>1290</td>
<td>30</td>
<td>500</td>
</tr>
</tbody>
</table>

Material Properties According to Isaksson and Mårtensson (2010)

Elastic modulus of the steel: $E_s = 210$ GPa

Elastic modulus of the concrete: $E_c = 33$ GPa

Modular ratio for short term loading: $\eta_0 = \frac{E_s}{E_c} = 6.36$

Weight of concrete: $\gamma_c = 25$ kN/m$^3$

Design Strengths According to Isaksson and Mårtensson (2010)

Characteristic yield strength of the steel: $f_{yc} = 355$ MPa

Material partial factor for steel: $\gamma_{M0} = 1.0$

Design yield strength of the steel: $f_{yd} = \frac{f_{yc}}{\gamma_{M0}} = 355$ MPa

Characteristic compressive strength of the concrete: $f_{ck} = 30$ MPa

Material partial factor for concrete: $\gamma_C = 1.5$

Design compressive strength of the concrete: $f_{cd} = \frac{f_{ck}}{\gamma_C} = 20$ MPa

Cross-Sectional Properties According to Lebet and Hirt (2013)

Area of one main girder:

$$A_s = b_f \cdot h_{tot} - (b_f - t_w) \cdot h_w = 500 \cdot 1350 - (500 - 17) \cdot 1290 = (A.1)$$

$$= 52 \cdot 10^3 \text{ mm}^2$$
Area of half of the concrete slab:

\[ A_c = b_c \cdot t_c = 5000 \cdot 350 = 1750 \cdot 10^3 \text{ mm}^2 \]  
\[ (A.2) \]

Area of the composite cross-section:

\[ A_{sc} = A_s + \frac{A_c}{\eta_0} = 52 \cdot 10^3 + \frac{1750 \cdot 10^3}{6.36} = 327 \cdot 10^3 \text{ mm}^2 \]  
\[ (A.3) \]

Distance from the bottom of the main girder to its neutral layer:

\[ z_s = \frac{h}{2} = \frac{1350}{2} = 675 \text{ mm} \]  
\[ (A.4) \]

Distance between the neutral layers of the main girder and the concrete slab:

\[ (e_s + e_c) = \frac{h}{2} + \frac{t_c}{2} = \frac{1350}{2} + \frac{350}{2} = 850 \text{ mm} \]  
\[ (A.5) \]

Distance between the neutral layers of the composite cross-section and the concrete slab:

\[ e_c = \frac{A_s \cdot (e_s + e_c)}{A_{sc}} = \frac{52000 \cdot (850)}{327000} = 135 \text{ mm} \]  
\[ (A.6) \]

Distance between the neutral layers of the composite cross-section and the main girder:

\[ e_s = (e_s + e_c) - e_c = 850 - 135 = 715 \text{ mm} \]  
\[ (A.7) \]

Distance between the neutral layer of the composite cross-section and the bottom of the steel girder:

\[ z_{sc} = e_s + z_s = 675 + 715 = 1390 \text{ mm} \]  
\[ (A.8) \]

Moment of inertia of one main girder:

\[ I_s = \frac{b_f \cdot h_{tot}^3}{12} - 2 \cdot \frac{(b_f - t_w) \cdot 2 \cdot h_w^3}{12} = \frac{500 \cdot 1350^3}{12} - 2 \cdot \frac{(500 - 17) \cdot 2 \cdot 1290^3}{12} = 16 \cdot 100 \cdot 10^6 \text{ mm}^4 \]  
\[ (A.9) \]

Moment of inertia of the concrete slab:

\[ I_c = \frac{b_c \cdot t_c^3}{12} = \frac{5000 \cdot 350^3}{12} = 17900 \cdot 10^6 \text{ mm}^4 \]  
\[ (A.10) \]

Moment of inertia of the composite cross-section:

\[ I_{sc} = I_s + \frac{I_c}{n_0} + e_s \cdot e_c \cdot A_{sc} = \frac{17900 \cdot 10^6}{6.36} + 714 \cdot 135 \cdot 327000 = 50000 \cdot 10^6 \text{ mm}^4 \]  
\[ (A.11) \]
Bending Moment Capacity

The bending moment capacity of the composite cross-section is the bending moment for which either the compressive strength of the concrete slab or the tensile yield strength of the steel at the bottom of the lower flange is reached.

Moment capacity of the composite cross-section if the concrete is crushed:

\[
W_{sc,\text{top}} = \frac{I_{sc}}{h_s + h_c - z_{sc}} = \frac{50000 \cdot 10^6}{1350 + 350 - 1390} = 163 \cdot 10^6 \text{ mm}^3
\]  
(A.12)

\[
M_{\text{max, top}} = n_0 \cdot f_{cd} \cdot W_{sc,\text{top}} = 6.36 \cdot 20 \cdot 163 = 20700 \text{ kNm}
\]  
(A.13)

Moment capacity of the composite cross-section if the steel yields:

\[
W_{sc,\text{bot}} = \frac{I_{sc}}{z_{sc}} = \frac{50000 \cdot 10^6}{1390} = 36.3 \cdot 10^6 \text{ mm}^3
\]  
(A.14)

\[
M_{\text{max, bot}} = f_{yd} \cdot W_{sc,\text{bot}} = 355 \cdot 36.3 = 12900 \text{ kNm}
\]  
(A.15)

Bending moment capacity of the composite cross-section:

\[
M_{\text{max, top}} > M_{\text{max, bot}}
\]  
(A.16)

wherefore

\[
M_{0,sc,Rd} = M_{\text{max, bot}} = 12900 \text{ kNm}
\]

The design bending moment, caused by the self-weight of the concrete, on the composite cross-section at midspan:

\[
g_c = 1.2 \cdot \gamma_c \cdot A_c = 1.2 \cdot 25 \cdot 1750 \cdot 10^{-3} = 52.5 \text{ kN/m}
\]  
(A.17)

\[
M_{c,Ed} = \frac{g_c \cdot l^2}{8} = \frac{52.5 \cdot 26^2}{8} = 4440 \text{ kNm}
\]  
(A.18)

The bending moment capacity of the composite cross-section that can be utilized to carry live load:

\[
M_{sc,Rd} = M_{0,sc,Rd} - M_{c,Ed} = 12900 - 4440 = 8500 \text{ kNm}
\]  
(A.19)
A.2 CLT Bridge

The bending moment capacity of the CLT bridge is simply the bending moment capacity of its main girders. The CLT bridge has identical main girders as the composite bridge in section A.1 of Appendix A. The bending moment capacity of the CLT bridge is determined for half of the bridge’s width.

Material Properties According to Isaksson and Mårtensson (2010)

Elastic modulus of the steel: $E_s = 210$ GPa

Weight of CLT: $\gamma_{CLT} = 4.8$ kN/m$^3$

Design Strengths According to Isaksson and Mårtensson (2010)

Characteristic yield strength of the steel: $f_{yc} = 355$ MPa

Material partial factor for steel: $\gamma_{M0} = 1.0$

Design yield strength of the steel: $f_{yd} = \frac{f_{yc}}{\gamma_{M0}} = 355$ MPa
Cross-Sectional Properties According to Isaksson and Mårtensson (2010)

Area of one main girder:
\[ A_s = 52 \cdot 10^3 \text{ mm}^2 \]

Area of half of the CLT slab:
\[ A_{\text{CLT}} = b_{\text{CLT}} \cdot t_{\text{CLT}} = 5000 \cdot 400 = 2000 \cdot 10^3 \text{ mm}^2 \] (A.20)

Moment of inertia of one main girder:
\[ I_s = 16 100 \cdot 10^6 \text{ mm}^4 \]

Bending Moment Capacity

The bending moment capacity of the half of the CLT bridge is the moment capacity of one steel girder.

Moment capacity of the CLT bridge:
\[ W_s = \frac{I_s}{z_s} = \frac{16 100 \cdot 10^6}{675} = 23.8 \cdot 10^6 \text{ mm}^3 \] (A.21)

\[ M_{0,s,Rd} = f_{yd} \cdot W_s = 355 \cdot 23.8 = 8500 \text{ kNm} \] (A.22)

The design bending moment, caused by the self-weight of the CLT slab, on the CLT bridge at midspan:
\[ g_{\text{CLT}} = 1.2 \cdot \gamma_{\text{CLT}} \cdot A_{\text{CLT}} = 1.2 \cdot 4.8 \cdot 2000 \cdot 10^{-3} = 11.5 \text{ kN/m} \] (A.23)

\[ M_{\text{CLT,Ed}} = \frac{g_{\text{CLT}} \cdot l^2}{8} = \frac{11.5 \cdot 26^2}{8} = 970 \text{ kNm} \] (A.24)

Bending moment capacity of the CLT bridge that can be utilized to carry live load:
\[ M_{s,Rd} = M_{0,s,Rd} - M_{\text{CLT,Ed}} = 8500 - 972 = 7500 \text{ kNm} \] (A.25)
B Design Values Related to Steel

B.1 Steel Girders

Figure B.1: Cross-section of main girder

Table B.1: Dimensions of main girder

<table>
<thead>
<tr>
<th>$h_{tot}$ [mm]</th>
<th>$t_{f,top}$ [mm]</th>
<th>$b_{f,top}$ [mm]</th>
<th>$t_w$ [mm]</th>
<th>$h_w$ [mm]</th>
<th>$t_{f,bot}$ [mm]</th>
<th>$b_{f,bot}$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1650</td>
<td>50</td>
<td>500</td>
<td>17</td>
<td>1550</td>
<td>50</td>
<td>500</td>
</tr>
</tbody>
</table>
Material Properties According to Isaksson and Mårtensson (2010)

Elastic modulus: \( E = 210 \text{ GPa} \)

Shear modulus: \( G = 81 \text{ GPa} \)

Poisson’s ratio: \( \nu = 0.3 \)

Design Properties

Yield strength, web (EN10025-3): \( f_{y,w} = 345 \text{ MPa} \) for \( 16 < t_w \leq 40 \text{ mm} \)

Yield strength, flanges (EN10025-3): \( f_{y,f} = 335 \text{ MPa} \) for \( 40 < t_f \leq 16 \text{ mm} \)

Partial factors: \( \gamma_{M0} = 1.0 \)
\( \gamma_{M1} = 1.0 \)

Classification of Cross-Section

The equations used for the classification of the cross-section are taken from *SS-EN 1993-1-1 - Design of steel structures part 1-1: General rules and rules for buildings*.

Flanges:

Class 1: \( \frac{c_f}{t_f} = \frac{(b_f - t_w)/2}{t_f} \leq 9 \cdot \varepsilon \)  \hspace{1cm} (B.1)

where
\[ \varepsilon = \sqrt{\frac{235}{f_{y,f}}} \]  and  \( f_{y,f} = 335 \text{ MPa} \) for flanges.

The flanges are in cross-section class 1 according to B.1;
\[
\frac{c_f}{t_f} = \frac{(500 - 17)/2}{50} = 4.83 \leq 9 \cdot \sqrt{\frac{235}{335}} = 7.54
\]

Web:

Class 3: \( 83 \cdot \varepsilon \leq \frac{c_w}{t_w} = \frac{h_w}{t_w} \leq 124 \cdot \varepsilon \)  \hspace{1cm} (B.2)

where
\[ \varepsilon = \sqrt{\frac{235}{f_{y,w}}} \]  and  \( f_{y,w} = 345 \text{ MPa} \) for the web.
The web is in cross-section class 3 according to B.2;

\[ 83 \cdot \sqrt{\frac{235}{345}} = 68.5 \leq \frac{1550}{17} = 91.2 \leq 124 \cdot \sqrt{\frac{235}{345}} = 102 \]

Since the web is in cross-section class 3, the classification of the entire cross-section is also 3.

**Cross-Sectional Values**

**Moment of inertia about the y-axis**: 

\[ I_y = \frac{b_f \cdot h_{tot}^3}{12} - 2 \cdot \frac{(b_f - t_w)}{2} \cdot \frac{h_{w}^3}{12} = \]

\[ = \frac{500 \cdot 1650^3}{12} - 2 \cdot \frac{(500 - 17)}{2} \cdot \frac{1550^3}{12} = 37286 \cdot 10^6 \text{ mm}^4 \]

**Moment of inertia about the z-axis**: 

\[ I_z = \frac{t_{f,bot} \cdot b_{f,bot}^3}{12} + \frac{h_{w} \cdot t_{w}^3}{12} + \frac{t_{f,top} \cdot b_{f,top}^3}{12} = \]

\[ = \frac{50 \cdot 500^3}{12} + \frac{1550 \cdot 17^3}{12} + \frac{50 \cdot 500^3}{12} = 1042 \cdot 10^6 \text{ mm}^4 \]

**Elastic section modulus about the y-axis**: 

\[ W_{el,y} = \frac{I_y}{h_{tot}/2} = \frac{37286 \cdot 10^6}{1650/2} = 45.2 \cdot 10^6 \text{ mm}^3 \]  

**Elastic section modulus about the z-axis**: 

\[ W_{el,z} = \frac{I_z}{b_f/2} = \frac{1042 \cdot 10^6}{500/2} = 4.17 \cdot 10^6 \text{ mm}^3 \]

**First moment of area about the y-axis**: 

\[ S_x = t_w \cdot \frac{h_{w} \cdot h_{w}}{4} + b_{f,top} \cdot t_{f,top} \cdot \frac{(h_{tot} - t_{f,top})}{2} \]

- Middle of cross-section:

\[ S_x = 17 \cdot \frac{1550}{2} \cdot \frac{1550}{4} + 500 \cdot 50 \cdot \left(\frac{1650}{2} - \frac{50}{2}\right) = 25.1 \cdot 10^6 \text{ mm}^3 \]

**St. Venants torsion constant**: 

\[ I_t = \sum \frac{b_i h_i^3}{3} = \frac{500 \cdot 50^3 + 500 \cdot 50^3 + 1550 \cdot 17^3}{3} = 44.2 \cdot 10^6 \text{ mm}^4 \]

**Warping constant**: 

\[ I_w = \frac{b_f^3 \cdot h^2 \cdot t_f}{24} = \frac{500^3 \cdot 1600^2 \cdot 50}{24} = 66.7 \cdot 10^{13} \text{ mm}^6 \]

9
Design Resistances

Bending moment about y-axis according to *SS-EN 1993-1-1 - Design of steel structures part 1-1: General rules and rules for buildings*:

\[ M_{c,y,Rd} = \frac{W_{el,y} \cdot f_{y,f}}{\gamma_{M0}} = \frac{45.2 \cdot 10^{-3} \cdot 335 \cdot 10^6}{1.0} = 15140 \text{ kNm} \quad (B.10) \]

where

\[ f_{y,f} = 335 \text{ MPa for flanges and } \gamma_{M0} = 1.0. \]

Shear along z-axis according to *SS-EN 1993-1-1 - Design of steel structures part 1-1: General rules and rules for buildings* and *SS-EN 1993-1-5 - Design of steel structures part 1-5: Plated structural elements*:

The shear buckling resistance shall be taken into account if

\[ \frac{h_w}{t_w} > 72 \cdot \frac{\varepsilon}{\eta} \quad (B.11) \]

where

\[ \varepsilon = \sqrt{\frac{235}{f_{y,w}}} , \quad f_{y,w} = 345 \text{ MPa for the web and } \eta = 1.2 \text{ for steel grades up to S460.} \]

It gives

\[ \frac{1550}{17} = 91.2 > 72 \cdot \frac{\sqrt{235}}{345} \cdot \frac{1}{1.2} = 49.5 \]

For unstiffened or stiffened webs, the design resistance for shear is obtained by (*SS-EN 1993-1-5 - Design of steel structures part 1-5: Plated structural elements*)

\[ V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta \cdot f_{y,w} \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} \quad (B.12) \]

where

\[ V_{bw,Rd} = \frac{\chi_w \cdot f_{y,w} \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}}. \quad (B.13) \]

The reduction factor, \( \chi_w \), is calculated with B.14 - B.16.

\[ \chi_w = \frac{0.83}{\bar{\lambda}_w} \quad (B.14) \]

\[ \bar{\lambda}_w = 0.76 \cdot \sqrt{\frac{f_{y,w}}{\tau_{cr}}} \quad (B.15) \]

\[ \tau_{cr} = k_f \cdot \sigma_E \quad (B.16) \]
Since
\[
\frac{a}{h_w} = \frac{3.25}{1.55} \geq 1 ,
\]
where \( a \) is the distance between lateral restraints, \( k_r \) is obtained by
\[
k_r = 5.34 + 4.00 \left( \frac{h_w}{a} \right)^2 + k_{rsl}
\] \hspace{1cm} (B.17)
Moreover,
\[
k_{rsl} = 0
\]
when no longitudinal stiffeners are present. \( \sigma_E \) is then calculated as
\[
\sigma_E = \frac{\pi^2 \cdot E \cdot t^2}{12 \cdot (1 - \nu^2) \cdot b^2}
\] \hspace{1cm} (B.18)
Finally, following results are obtained:
\[
k_r = 5.34 + 4.00 \left( \frac{1550}{3250} \right)^2 \approx 6.25
\]
\[
\sigma_E = \frac{\pi^2 \cdot 210 \cdot 10^6 \cdot 17^2}{12 \cdot (1 - 0.3^2) \cdot 1550^2} \approx 22.8 \text{ MPa}
\]
wherefore
\[
\tau_{cr} = 6.25 \cdot 22.8 = 143 \text{ MPa}
\]
\[
\bar{\lambda}_w = 0.76 \cdot \sqrt{\frac{345}{143}} = 1.18 \geq 1.08
\]
\[
\chi_w = \frac{0.83}{1.18} = 0.70
\]
With \( \chi_w \) known, the shear buckling resistance is calculated as
\[
V_{bw,Rd} = \frac{0.70 \cdot 345 \cdot 10^6 \cdot 1.550 \cdot 0.017}{\sqrt{3} \cdot 1} = 3686 \text{ kN}.
\]
The last part of equation B.12 becomes
\[
\frac{\eta \cdot f_{g,w} \cdot h_w \cdot t_w}{\sqrt{3} \cdot \gamma_{M1}} = \frac{1.20 \cdot 345 \cdot 10^6 \cdot 1.55 \cdot 0.017}{\sqrt{3} \cdot 1} = 6290 \text{ kN},
\]
which is far greater than \( V_{bw,Rd} \).
Lateral torsional buckling according to SS-EN 1993-1-1 - Design of steel structures part 1-1: General rules and rules for buildings:

Depending on the elastic critical moment for lateral torsional buckling, $M_{cr}$, effects concerning the phenomena may be disregarded. Assuming a linear bending moment diagram between lateral restraints, i.e. where the bracing connects to the main girder, the critical moment can be determined with (Bureau, 2005)

$$M_{cr} = \frac{\pi^2 EI_z}{(kl)^2} \sqrt{\frac{I_w}{I_z} + \frac{GI_I((kl)^2)}{\pi^2 EI_z}}$$  \hspace{1cm} (B.19)

where

$$k = 1$$ and $l$ is the distance between lateral restraints, i.e. 3.25 m.

With values according to section B.1 of Appendix B, the elastic critical moment becomes

$$M_{cr} = \frac{\pi^2 \cdot 210 \cdot 10^9 \cdot 1042 \cdot 10^{-6}}{3.25^2} \sqrt{\frac{66.7 \cdot 10^{-5}}{1042 \cdot 10^{-6}} + \frac{81 \cdot 10^9 \cdot 44.2 \cdot 10^{-6} \cdot 3.25^2}{\pi^2 \cdot 210 \cdot 10^9 \cdot 1042 \cdot 10^{-6}}} =$$

$$= 165793 \text{ kNm}$$

The slenderness factor is determined by

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_{el,y} \cdot f_{y,f}}{M_{cr}}} = \sqrt{\frac{45.2 \cdot 10^{-6} \cdot 335 \cdot 10^6}{165793 \cdot 10^3}} = 0.0096.$$  \hspace{1cm} (B.20)

Since $\bar{\lambda}_{LT} \leq 0.2$, there is no risk lateral torsional buckling.
B.2  Steel Bracing

Figure B.2: Cross-section of bracing member (Dlubal, 2013)

<table>
<thead>
<tr>
<th>Table B.2: Dimensions of bracing member</th>
</tr>
</thead>
<tbody>
<tr>
<td>b</td>
</tr>
<tr>
<td>---</td>
</tr>
<tr>
<td>180</td>
</tr>
</tbody>
</table>

Material Properties According to Isaksson and Mårtensson (2010)

Elastic modulus: $E = 210$ GPa

Shear modulus: $G = 81$ GPa

Poisson’s ratio: $\nu = 0.3$

Design Properties

Yield strength: $f_y = 355$ MPa

Partial factors: $\gamma_{M0} = 1.0$

$\gamma_{M1} = 1.0$
Classification of Cross-Section

The equations used for the classification of the cross-section are taken from SS-EN 1993-1-1 - Design of steel structures part 1-1: General rules and rules for buildings.

Cross-section:

Class 3: \( \frac{h}{t} \leq 15\varepsilon \), \( \frac{b + h}{2 \cdot t} \leq 11.5\varepsilon \) \hspace{1cm} (B.21)

where

\[ \varepsilon = \sqrt{\frac{235}{f_y}} \quad \text{and} \quad f_y = 355 \text{ MPa}. \]

\[ \frac{180}{16} = 11.25 \leq 15 \cdot \sqrt{\frac{235}{355}} = 12.2: \quad \frac{180 + 180}{2 \cdot 16} = 11.25 > 11.5 \cdot \sqrt{\frac{235}{355}} = 9.36 \]

Since equation B.21 is not fulfilled, the cross-section belongs to class 4.

Cross-Sectional Values

The following values are taken from Isaksson and Mårtensson (2010):

Cross-sectional area: \( A = 5540 \text{ mm}^2 \)

Moment of inertia about the y-axis: \( I_y = 16.8 \cdot 10^6 \text{ mm}^4 \)

Moment of inertia about the z-axis: \( I_z = 16.8 \cdot 10^6 \text{ mm}^4 \)

Moment of inertia about the u-axis: \( I_u = 26.9 \cdot 10^6 \text{ mm}^4 \)

Moment of inertia about the v-axis: \( I_v = 6.79 \cdot 10^6 \text{ mm}^4 \)

Radius of gyration about the y-axis: \( i_y = 55.1 \text{ mm} \)

Radius of gyration about the z-axis: \( i_z = 55.1 \text{ mm} \)

Radius of gyration about the u-axis: \( i_u = 69.6 \text{ mm} \)

Radius of gyration about the v-axis: \( i_v = 35.0 \text{ mm} \)

Effective area according to SS-EN 1993-1-5 - Design of steel structures part 1-5: Plated structural elements:

Since the classification of the cross-section is 4, an effective area should be used for verifications that take compression into account. The effective area is obtained by

\[ A_{c,eff} = \rho \cdot A_c \] \hspace{1cm} (B.22)
where \( \rho \) is a reduction factor and \( A_c \) the area of the cross-section.

For outstanding compression elements, such as the legs of angles, the reduction factor equals

\[
\rho = \begin{cases} 
1.0 & \text{when } \bar{\lambda}_p \leq 0.748 \\
\frac{\bar{\lambda}_p - 0.188}{\lambda_p^2} & \leq 1.0 \quad \text{when } \bar{\lambda}_p > 0.748
\end{cases}
\]

(B.23)

where \( \bar{\lambda}_p \) is the plate slenderness factor. The factor is calculated as

\[
\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\bar{b}/t}{28.4 \cdot \varepsilon \cdot \sqrt{k_\sigma}}
\]

(B.24)

where \( \bar{b} \) corresponds to \( c \) in Figure B.2, \( t \) is the thickness of the element and

\[
\varepsilon = \sqrt{\frac{235}{f_y}}.
\]

(B.25)

Finally, the buckling factor \( k_\sigma \) depends on the stress distribution in the legs of the angles according to Figure B.3. For purely compressed elements, i.e. when \( \sigma_2/\sigma_1 = 1 \), the buckling factor equals 0.43.

![Figure B.3: Stress distribution on legs of angle sections (SS-EN 1993-1-5 - Design of steel structures part 1-5: Plated structural elements, p. 17)](image)

Hence, the slenderness factor becomes

\[
\bar{\lambda}_p = \frac{(180 - 50.2)/16}{28.4 \cdot \sqrt{\frac{235}{355}} \cdot \sqrt{0.43}} = 0.535 \leq 0.748
\]

wherefore

\[
A_{c,eff} = 1.0 \cdot A_c = A_c.
\]
Design Resistances

Tension according to *SS-EN 1993-1-1 - Design of steel structures part 1-1: General rules and rules for buildings*:

\[ N_{t,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} = \frac{5540 \cdot 10^{-6} \cdot 355 \cdot 10^6}{1.0} = 1967 \text{ kN} \] (B.26)

Compression according to *SS-EN 1993-1-1 - Design of steel structures part 1-1: General rules and rules for buildings*:

\[ N_{c,Rd} = \frac{A_{\text{eff}} \cdot f_y}{\gamma_{MO}} = \frac{5540 \cdot 10^{-6} \cdot 355 \cdot 10^6}{1.0} = 1967 \text{ kN} \] (B.27)

Buckling according to *SS-EN 1993-1-1 - Design of steel structures part 1-1: General rules and rules for buildings*:

The buckling resistance of a compressed member is determined by

\[ N_{b,Rd} = \frac{\chi \cdot A_{\text{eff}} \cdot f_y}{\gamma_{M1}} \] (B.28)

where the reduction factor, \( \chi \), is calculated as

\[ \chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \] (but \( \chi \leq 1.0 \)) (B.29)

Moreover, \( \Phi \), is obtained by

\[ \Phi = 0.5 \cdot \left[ 1 + \alpha \left( \bar{\lambda} - 0.2 \right) + \bar{\lambda}^2 \right] \] (B.30)

where

\[ \bar{\lambda} = \sqrt{\frac{A_{\text{eff}} \cdot f_y}{N_{cr}}} \text{ and } \alpha = 0.34 \text{ according to buckling curve b.} \] (B.31)

The slenderness factor, \( \bar{\lambda} \), may be calculated as

\[ \bar{\lambda} = \sqrt{\frac{A_{\text{eff}} \cdot f_y}{N_{cr}}} = \frac{L_{cr}}{i \cdot \pi} \cdot \sqrt{\frac{A_{\text{eff}} \cdot f_y}{A \cdot E}} \] (B.32)

for cross-sections in class 4.
The buckling length is determined by

\[ L_{cr} = \beta \cdot L \]  

(B.33)

where

\[ L = 6.132 \text{ m} \quad \text{and} \quad \beta = 1 \] for a simply supported member.

Since the smallest moment of inertia appears about the v-axis, corresponding buckling resistance is evaluated according to

\[
\tilde{\lambda}_v = \frac{L_{cr}}{i_v \cdot \pi} \cdot \sqrt{\frac{A_{eff} \cdot f_y}{A \cdot E}} 
\]

(B.34)

which gives

\[
\tilde{\lambda}_v = \frac{6.132}{35.0 \cdot 10^{-3} \cdot \pi} \cdot \sqrt{\frac{5540 \cdot 10^{-6} \cdot 355 \cdot 10^6}{5540 \cdot 10^{-6} \cdot 210 \cdot 10^9}} = 2.293 
\]

(B.35)

\[
\Phi_v = 0.5 \cdot [1 + 0.34 (2.293 - 0.2) + 2.293^2] = 3.485 
\]

(B.36)

\[
\chi_v = \frac{1}{3.485 + \sqrt{3.485^2 - 2.293^2}} = 0.164 
\]

(B.37)

\[
N_{b, Rd, v} = \frac{0.164 \cdot 5540 \cdot 10^{-6} \cdot 355 \cdot 10^6}{1.0} = 322 \text{ kN} 
\]

(B.38)
C Design Values Related to CLT

Material Properties according to ETA-06/0138 (2012)

Elastic modulus parallel to grain: $E_0 = 12000$ MPa
Elastic modulus perpendicular to grain: $E_{90} = 0$ MPa
Shear modulus parallel to grain: $G_0 = 690$ MPa
Shear modulus perpendicular to grain: $G_{90} = 50$ MPa

Design Properties according to ETA-06/0138 (2012)

Bending strength perpendicular to the plane of the slab: $f_{m,k} = 24$ MPa
Shear strength perpendicular to the plane of the slab: $f_{v,R,k} = 0.8 - 1.2$ MPa
Compressive strength perpendicular to the grain: $f_{c,90,k} = 2.7$ MPa
Modification factor: $k_{mod} = 0.9$
Partial factor: $\gamma_M = 1.25$

System strength factor: $k_{sys} = 1.10$

Reduction factor that regards notches: $k_v = 1.0$

Factor that depends on support and load conditions: $k_{c,90} = 2.2$

Cross-Sectional Values

The cross-sectional values are calculated in accordance with Wallner-Novak et al. (2013) and ETA-06/0138 (2012).

Net cross-sectional area in x-direction:

$$A_{x,net} = \sum_{i=1}^{n} b \cdot t_{Li} = 1000 \cdot (2 \cdot 45 + 6 \cdot 40) = 33 \cdot 10^4 \text{ mm}^2$$  \hspace{1cm} (C.1)

Net cross-sectional area in y-direction:

$$A_{y,net} = \sum_{i=1}^{n} b \cdot t_{Ci} = 1000 \cdot (2 \cdot 45 + 2 \cdot 40) = 17 \cdot 10^2 \text{ mm}^2$$  \hspace{1cm} (C.2)

Net moment of inertia in x-direction:

$$I_{x,net} = \sum_{i=1}^{n} \frac{b \cdot t_{Li}^3}{12} + \sum_{i=1}^{n} b \cdot t_{Li} \cdot a_{Li}^2 =$$

$$= 2 \cdot \frac{1000 \cdot 45^3}{12} + 6 \cdot \frac{1000 \cdot 40^3}{12} + 2 \cdot 1000 \cdot 45 \cdot 22.5^2 + 2 \cdot 1000 \cdot 40 \cdot 110^2 +$$

$$+ 2 \cdot 1000 \cdot 40 \cdot 190^2 + 2 \cdot 1000 \cdot 40 \cdot 230^2 = 8181 \cdot 10^6 \text{ mm}^4$$

Net moment of inertia in y-direction:

$$I_{y,net} = \sum_{i=1}^{n} \frac{b \cdot t_{Ci}^3}{12} + \sum_{i=1}^{n} b \cdot t_{Ci} \cdot a_{Ci}^2 =$$

$$= 2 \cdot \frac{1000 \cdot 45^3}{12} + 2 \cdot \frac{1000 \cdot 40^3}{12} + 2 \cdot 1000 \cdot 45 \cdot 67.5^2 + 2 \cdot 1000 \cdot 40 \cdot 150^2 =$$

$$= 2236 \cdot 10^6 \text{ mm}^4$$

Net elastic section modulus in x-direction:

$$W_{x,net} = \frac{I_{x,net}}{z_{L,max}} = \frac{8181 \cdot 10^6}{250} = 32.7 \cdot 10^6 \text{ mm}^3$$  \hspace{1cm} (C.5)
Net elastic section modulus in y-direction:

\[ W_{y,\text{net}} = \frac{I_{y,\text{net}}}{z_{C,\text{max}}} = \frac{2236 \cdot 10^6}{\left( \frac{500}{2} - 2 \cdot 40 \right)} = 13.2 \cdot 10^6 \text{ mm}^3 \]  

(C.6)

Net first moment of area about the y-axis:

\[ S_{x,\text{net}} = \sum_{i=1}^{L_i} b \cdot t_{Li} \cdot a_i \]  

(C.7)

- Where layer \( C_2 \) meets layer \( L_4 \):

\[ S_{x,\text{net}} = 1000 \cdot 40 \cdot (110 + 190 + 230) = 21.2 \cdot 10^6 \text{ mm}^3 \]

- Middle of cross-section:

\[ S_{x,\text{net}} = 1000 \cdot 45 \cdot 22.5 + 1000 \cdot 40 \cdot (110 + 190 + 230) = 22.2 \cdot 10^6 \text{ mm}^3 \]

Net first moment of area about the x-axis:

\[ S_{y,\text{net}} = \sum_{i=1}^{C_i} b \cdot t_{Ci} \cdot a_i \]  

(C.8)

- In the middle of the cross-section:

\[ S_{y,\text{net}} = 1000 \cdot 45 \cdot 67.5 + 1000 \cdot 40 \cdot 150 = 9.04 \cdot 10^6 \text{ mm}^3 \]

**Stiffness Matrix According to RFEM**

\[
D = \begin{bmatrix}
98290.2 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 26864.1 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 7187.5 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 48160.8 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 23370.8 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 3964890 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 2042519 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 86250.0
\end{bmatrix} \cdot 10^3 \quad (C.9)
\]

**Design Strengths**

Bending perpendicular to the grain of the boards according to **SS-EN 1995-1-1 - Design of timber structures part 1-1: General - Common rules and rules for buildings**:

\[ f_{m,d} = \frac{k_{mod} \cdot f_{m,k}}{\gamma_M} = \frac{0.9 \cdot 24}{1.25} = 17.3 \text{ MPa} \]  

(C.10)
Bending perpendicular to the plane of the solid slab according to ETA-06/0138 (2012, p.27):

\[ f_{m,d,\text{slab}} = f_{m,d} \cdot k_{sys} = 17.3 \cdot 1.10 = 19.0 \text{ MPa} \quad (\text{C.11}) \]

Shear perpendicular to the grain of the boards according to \textit{SS-EN 1995-1-1 - Design of timber structures part 1-1: General - Common rules and rules for buildings}:

\[ f_{v,R,d} = \frac{k_{mod} \cdot f_{v,R,k}}{\gamma_M} \quad (\text{C.12}) \]

\[ f_{v,R,k} = 0.8 \text{ MPa} \implies f_{v,R,d} = \frac{0.9 \cdot 0.8}{1.25} = 0.58 \text{ MPa} \quad (\text{C.13}) \]

\[ f_{v,R,k} = 1.2 \text{ MPa} \implies f_{v,R,d} = \frac{0.9 \cdot 1.2}{1.25} = 0.86 \text{ MPa} \quad (\text{C.14}) \]

Shear perpendicular to the plane of the solid wood slab (ETA-06/0138, 2012, p.27):

\[ f_{v,R,d,\text{slab}} = f_{v,R,d} \cdot k_v \quad (\text{C.15}) \]

\[ f_{v,R,k} = 0.8 \text{ MPa} \implies f_{v,R,d,\text{slab}} = 0.58 \cdot 1.0 = 0.58 \text{ MPa} \quad (\text{C.16}) \]

\[ f_{v,R,k} = 1.2 \text{ MPa} \implies f_{v,R,d,\text{slab}} = 0.86 \cdot 1.0 = 0.86 \text{ MPa} \quad (\text{C.17}) \]

Compression perpendicular to the grain of the boards according to \textit{SS-EN 1995-1-1 - Design of timber structures part 1-1: General - Common rules and rules for buildings}:

\[ f_{c,90,d} = \frac{k_{mod} \cdot f_{c,90,k}}{\gamma_M} = \frac{0.9 \cdot 2.7}{1.25} = 1.94 \text{ MPa} \quad (\text{C.18}) \]

Compression perpendicular to the slab according to ETA-06/0138 (2012, p.26):

\[ f_{c,90,d,\text{slab}} = f_{c,90,d} \cdot k_{c,90} = 1.94 \cdot 2.2 = 4.28 \text{ MPa} \quad (\text{C.19}) \]
D Cost Estimation

D.1 Basis of the Cost Estimation

A tender calculus for a 42 m long composite bridge was provided by Bruneby (2015). The cost estimation of the reference bridge and the new bridge type is based on this calculus.

Geometry of the bridge

Theoretical length: 42.0 m
Width: 12.16 m
Free width: 11.5 m
Bridge area: 511 m²
Free area of the carriageway: 483 m²

Cost of the steel girders

Steel weight: 66.7 ton
Cost of the steel girders including freight and assembly: 1 594 250 SEK (23 900 SEK/ton)
Cost of launching of the steel girders: 135 376 SEK (2030 SEK/ton)
Cost of painting the steel girders: 150 000 SEK (2250 SEK/ton)
Total cost of the steel girders: 1 879 626 SEK (28 180 SEK/ton)
Cost of the concrete deck

Volume concrete:
243 m³

Cost of the concrete deck including reinforcement and casting:
1 423 164 SEK (5857 SEK/m³)

Cost of the falsework:
316 253 SEK (1301 SEK/m³)

Total cost of the concrete deck:
1 739 417 SEK (7158 SEK/m³)

Total cost of the bridge

Total cost:
1 879 626 + 1 739 417 = 3 619 043 SEK

Unit cost:
\[
\frac{3 619 043}{483} = 7493 \text{ SEK/m}^2
\]
D.2 Cost Estimation of the Reference Bridge

Geometry of the reference bridge

Theoretical length:
26.0 m

Width:
10.4 m

Free width:
9.75 m

Bridge area:
270 m$^2$

Free area of the carriageway:
254 m$^2$

Cost of the steel girders of the reference bridge

Weight of the main girders:
\[ A_{0-6.0} = 0.0458 \, \text{m}^2 \]
\[ A_{6.0-20.0} = 0.0583 \, \text{m}^2 \]
\[ A_{20.0-26.0} = 0.0458 \, \text{m}^2 \]
\[ V = 4 \cdot 6.0 \cdot 0.0458 + 2 \cdot 14.0 \cdot 0.0583 = 2.73 \, \text{m}^3 \]
Weight: \[ 2.73 \cdot 7.850 = 21.4 \, \text{ton} \]

Weight of the bracing:
HEA 240:
Weight per meter: 0.060 ton/m
Length: \[ 3 \cdot 5.2 = 15.6 \, \text{m} \]
Weight: \[ 15.6 \cdot 0.060 = 0.91 \, \text{ton} \]

HEB 650:
Weight per meter: 0.225 ton/m
Length: \[ 2 \cdot 5.2 = 10.4 \, \text{m} \]
Weight: \[ 10.4 \cdot 0.225 = 2.34 \, \text{ton} \]

Total steel weight:
\[ 21.4 + 0.94 + 2.34 = 24.7 \, \text{ton} \]

Cost of the steel girders including freight and assembly:
\[ 23 \, 900 \cdot 24.7 = 590 \, 330 \, \text{SEK} \]

Cost of launching of the steel girders:
\[ 2030 \cdot 24.7 = 50 \, 141 \, \text{SEK} \]

Cost of painting the steel girders:
\[ 2250 \cdot 24.7 = 55 \, 575 \, \text{SEK} \]
Total cost of the steel girders:
28 180 · 24.7 = 696 046 SEK

Cost of the concrete deck of the reference bridge

Deck area:
10.4 · 26 = 270 m²

Volume:
270 · 0.35 = 94.5 m³

Cost of the concrete deck including reinforcement and casting:
5857 · 94.5 = 553 487 SEK

Cost of the falsework:
1301 · 94.5 = 122 945 SEK

Total cost of the concrete deck:
7158 · 94.5 = 676 431 SEK

Total cost of the reference bridge

Total cost:
696 046 + 676 431 = 1 372 477 SEK

Unit cost:
\[
\frac{1 372 477}{254} = 5403 \text{ SEK/m}^2
\]
D.3 Cost Estimation of the New Bridge Type

Geometry of the new bridge type

Theoretical length:
26.0 m

Width:
10.0 m

Free width:
9.0 m

Bridge area:
260 m$^2$

Free area of the carriageway:
234 m$^2$

Cost of the steel of the new bridge type

Weight of the main girders:
$A = 0.0764 \text{ m}^2$
$V = 2 \cdot 26.0 \cdot 0.0764 = 3.97 \text{ m}^3$
Weight: $3.97 \cdot 7.850 = 31.2 \text{ ton}$

Weight of the bracing:
Weight per meter: 0.0435 ton/m
Total length: 294 m
Weight: 12.8 ton

Total steel weight:
$31.2 + 12.8 = 44.0 \text{ ton}$

Cost of the steel including freight and assembly:
$23,900 \cdot 44.0 = 1,051,600 \text{ SEK}$

Cost of launching of the steel:
$2030 \cdot 44.0 = 89,320 \text{ SEK}$

Cost of painting the steel:
$2250 \cdot 44.0 = 99,000 \text{ SEK}$

Total cost of the steel:
$28,180 \cdot 44.0 = 1,239,920 \text{ SEK}$

Cost of the CLT deck of the new bridge type

Cost of the CLT deck including freight:
The cost of the CLT elements were provided by Habenbacher (2015).

\[ A = 10.0 \cdot 26.0 = 260 \text{ m}^2 \]

Cost of the elements: 175 €/m²
Trimming: 40 €/m²
Freight to Sweden: 9 000 €
Total cost:
\[ 260 \cdot (175 + 40) + 9000 = 64 900 \text{ €} \]
\[ 1 \text{ €} = 9.24 \text{ SEK} \text{ (Oanda, 2015b)} \]
\[ 64 900 \cdot 9.24 = 599 676 \text{ SEK} \]

**Cost of mounting the CLT deck on the main girders:**

It is estimated that in order to mount the CLT deck on the main girders, three carpenters will work with the deck for two weeks. The cost per hour is assumed to be 415 SEK/h (Bruneby, 2015). Furthermore, a mobile crane will be used, and the cost of the mobile crane was estimated to 100 000 SEK.

Time: \( 3 \cdot 2 \cdot 40 = 240 \text{ h} \)
Cost of labour: 415 SEK/h
Mobile crane: 100 000 SEK
Cost of mounting the CLT deck:
\[ 240 \cdot 415 + 100 000 = 199 600 \text{ SEK} \]

**Total cost of the CLT deck:**
\[ 599 676 + 199 600 = 799 276 \text{ SEK} \]

**Total cost of the new bridge type**

**Total cost:**
\[ 1 239 920 + 799 276 = 2 039 196 \text{ SEK} \]

**Unit cost:**
\[ \frac{2 039 196}{234} = 8715 \text{ SEK/m}^2 \]