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Performance of Stress-Laminated-Timber Bridge Decks

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Cover: Cutaway illustration of a two lane stress-laminated-timber bridge deck used for highway traffic in Sweden

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Abstract

Stress-laminated-timber (SLT) bridge decks are a satisfactory alternative to conventional short-span bridges in terms of cost and performance. SLT decks are made from a number of timber or glulam beams positioned side by side and stressed together using high-strength steel bars. A concentrated load can therefore be distributed from the loaded beams onto adjacent beams due to the resisting friction caused by the stressing.

A project has been conducted in three parts in order to fill some of the knowledge gaps relating to SLT decks constructed in Sweden. The first part of the project focused on determining the ultimate-load capacity of SLT decks, as well as studying their behaviour when subjected to non-destructive loads. A full-scale test of a 270 mm deep SLT deck showed that interlaminar slip already occurs at load levels equivalent to serviceability limit state (SLS) loads. The interlaminar slip resulted in non-linear load-deflection behaviour in the deck, making linear design models insufficient. The ultimate load capacity of the tested SLT deck was 4.5 times higher than the SLS load. The deck showed a great redundancy when reloaded after failure.

The second part of the project aimed to study the cause and effects of interlaminar slip. Non-linear finite element (FE) models, which successfully simulated the interlaminar slip between the deck beams, were developed. Non-linear FE models can produce valuable knowledge about the stress redistribution that occurs in the deck and cannot be measured during tests. Even though the magnitude of the interlaminar slip is very small, it has a major influence on the load redistribution in SLT decks. Significant variations in the normal stress in the SLT deck were observed when the laboratory-tested full-scale test was simulated. The variations in normal stress have a significant influence on the amount of interlaminar slip in SLT decks, since the critical shear stress is dependent on the magnitude of normal stress.

The final part of the project focused on studying timber bridges in service and summarising the existing literature on the durability of SLT decks. A reduction in prestressing has been a major durability concern for SLT decks constructed using wet timber beams which shrunk with time, causing a prestressing reduction. Field inspections have shown that the prestressing loss in Swedish SLT decks made from dry glulam is much smaller than that reported in the literature.

Key words: stress-laminated timber, timber bridges, laboratory tests, linear modelling, non-linear modelling, FE modelling, butt joints, interlaminar slip, prestressing, durability

Tvärspända träplattors prestanda
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Sammanfattning

Tvärspända träplattor är ett fullgott alternativ gentemot broar byggda med konventionella material när det gäller kostnad och prestanda. Tvärspända träplattor består av ett flertal träbalkar placerade sida vid sida vilket spänns samman av höghållfasta stålstänger. Koncentrerade laster kan spridas från de direkt belastade balkarna till omkringliggande balkarna tack vare den mothållande friktion som uppstår av förspänningen.

Ett tredelat projekt har genomförts för att fylla en del av kunskapsluckorna om tvärspända träplattor konstruerade i Sverige. Den första delen av projektet fokuserade på att fastställa brottlastkapaciteten för tvärspända träplattor samt att studera plattornas beteende vid icke förstörande lastnivåer. Ett fullskaleförsök på en 270 mm tjock tvärspända träplatta visade att glidningar mellan balkarna uppstår redan vid lastnivåer som motsvarar laster i bruksgränstillstånd. Glidningarna mellan balkarna resulterade i ett ickelinjärt samband mellan last och deformationer vilket gjorde linjära beräkningsmodeller obrukbara. Brottlasten var 4.5 gånger högre en brukslast. Försöket visade på en enastående redundans när en last appliserades efter brottet utan att några nya brott uppstod.

Den andra delen av projektet hade som målsättning att studera orsak och verkan av glidningar mellan balkarna. Ickelinjära finita element (FE) modeller togs fram som simulerade glidningarna mellan balkarna framgångrikt. De ickelinjära FE modellerna ger värdefull information om de spänningsomfördelningar som sker i plattorna vilket är svårt att mäta under försöken. Även om storleken på den glidningen mellan balkarna är mycket liten så har den en stor inverkan på lastomfördelningen i tvärspända träplattor. Stora skillnader av normalspänningen observerades när fullskaleförsöket simulerades. Normalspänningsvariationerna har en signifikant inverkan på mängden glidning mellan balkarna då de kritiska skjuvspänningsnivåerna är baserad på normalspänningen.

Den sista delen av projektet fokuserade på att studera träbroar som är i bruk samt att summera framtagna kunskaper om tvärspännda träplattors hållbarhet. En förspänningsreduktion har observerats i plattor konstruerade med blötta träbalkar som krympt. Fältmätningar har visat att förspänningsförlusterna för svenska tvärspända limträplattor var mycket mindre än vad som tidigare rapporterats i litteraturen.

Nyckelord: tvärspända träplattor, träbroar, laboratorieförsök, linjär modellering, olinjär modellering, FE modellering, stumskarvar, förspänning, hållbarhet

List of publications

This thesis is based on the work contained in the following papers, referred to by Roman numerals in the text.

I. Ekholm, K., Crocetti, R., Kliger, R. (2012) Full-Scale Ultimate-Load Test of a Stress-Laminated-Timber Bridge Deck. *ASCE Journal of Bridge Engineering*, 17 (4): 691-699.

II. Ekholm, K., Crocetti, R., Kliger, R. (2013) Stress-Laminated Timber Decks Subjected to Eccentric Loads in the Ultimate-Limit State. *ASCE Journal of Bridge Engineering*, 18 (5): 409-416.

III. Ekholm, K., Ekevad, M. (2013) Modelling of Slip in Stress-Laminated-Timber Bridges: Comparison of Two FEM Approaches and Test Values. Submitted to *ASCE Journal of Structural Engineering*

IV. Ekholm, K., Kliger, R. (2013) Effect of Vertical Interlaminar Shear Slip and Butt Joints in Stress-Laminated-Timber Bridge Decks. Submitted to *Engineering Structures*

V. Ekholm, K., Wacker, P. J. (2013) Design of Stress-Laminated-Timber Bridge Decks in the USA and Sweden. Accepted for publication as a *Forest Products Laboratory (FPL) General Technical Report*

Specification of author's contribution to the included papers

I. The author was responsible for all parts of the paper

II. The author was responsible for all parts of the paper

III. The author was responsible for all parts of the paper except the elastic-plastic FE model

IV. The author was responsible for all parts of the paper

V. The author was responsible for all parts of the paper

Additional publications by the author

Licentiate thesis

Ekholm, K. (2011) *Stress-Laminated-Timber Bridge Decks Subjected to Ultimate Loads: Experimental Tests, Analytical and Numerical Modelling.* Licentiate thesis. Department of Civil and Environmental Engineering, Division of Structural Engineering, Steel and Timber Structures, Chalmers University of Technology, Publication no. Lic. 2011:4, Gothenburg, Sweden

Conference Papers

Karlsson K., Crocetti R., and Kliger R. (2009) Mechanical Properties of Stress-Laminated-Timber Decks – Experimental Study. *CIB-W18: Proceedings of meeting forty-two*, August 23-27 2009, Dübendorf, Switzerland

Karlsson K., Crocetti R., and Kliger R. (2010) Ultimate Limit State Load Test of Stress-Laminated-Timber Deck. *World Conference on Timber Engineering (WCTE) 2010 Proceedings*, June 20-24 2010, Riva del Garda, Italy

Ekholm K., Kliger R. and Crocetti R., (2012) Non-Linear Analysis of a Stress-Laminated-Timber Bridge Loaded to Failure. *International association for bridge maintenance and safety (IABMAS) 2012 Proceedings*, July 8-12 2012, Stresa, Italy

Kliger R., Ekholm K. and Crocetti R., (2012) Timber Bridges in Sweden – Ongoing Research and Steadily Expanding Market. *International association for bridge maintenance and safety (IABMAS) 2012 Proceedings*, July 8-12 2012, Stresa, Italy

Ekholm K., Nilsson P., Johansson E. (2013) Case Study of the Longest Single Span Timber Bridge for Highway Loads in Sweden. *International conference on Timber Bridges (ICTB) 2013 Proceedings*, September 30 - October 2 2013, Las Vegas, USA

Reports

Ekholm, K. (2012). *Vertikalt glidbrott och stumskarvars inverkan på tvärspännda träplattor: Provningsrapport* (Effects of vertical interlaminar slip and butt joint on stress-laminated-timber decks: Test report - In Swedish), Report 2013:06, Chalmers Tekniska högskola, Institutionen för Bygg- och Miljöteknik, Gothenburg, Sweden

Contents

Preface

The work in this thesis was carried out between October 2008 and September 2013 in the department of Civil and Environmental Engineering, Division of Structural Engineering, Steel and Timber Structures at Chalmers University of Technology. The work between October 2008 and June 2012 was a part of the project "Competitive Bridges" which was financed by VINNOVA together with multiple industrial partners (Martinsons träbroar, Moelven Töreboda, Vectura consulting, Tyréns, Reinersten Sverige, COWI Sverige, Ramböll Sverige and ELU). My research from June 2012 has been financed by WSP Sverige and Moelven Töreboda. For everyone's financing, support and other contribution, I am ever so grateful. During the period between September 2012 and June 2013 the work was carried out at the US Forest Service, Forest Products Laboratory (FPL) in Madison, Wisconsin. The scholarship I received from the Sweden-America foundation made this visit possible.

I would like to thank my supervisor Professor Robert Kliger who have supported my through the entire project. I would also like to thank the following persons: Professor Roberto Crocetti for his supervision during the first three years, Roland Olsson, for supporting me from the industry side, Lars Wahlström for the help in the lab, James P. Wacker for giving me the opportunity to come and work with him in Madison. I would also like to extend my gratitude to Mrs Jeanette Kliger for her professional language review of my publications.

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Kristoffer Ekholm

Gothenburg, September 2013

1 Introduction

1.1 Background

Wood is one of the oldest known building materials. In the building of bridges, wood has also been used successfully throughout history. There are bridges built as early as the 14th century that are still in use today. During the past couple of decades, the demand for sustainable bridges has increased significantly. Sustainability in terms of ecological impact, social adaptation and economy are key factors in the development of infrastructure for modern society. Timber bridges are an excellent alternative to conventional concrete and steel bridges –as an alternative not only for pedestrian bridges but also for regular short- and medium-span road bridges. Much research has been conducted over the past 40 years within the field of timber brides. The requirements imposed on bridges have changed throughout history, from being a simple structure for humans to cross streams and creeks to highway bridges several hundred metres long.

The amount of traffic on the roads today has increased significantly. Together with heavier vehicles, this poses a new challenge for timber bridges. Researchers and engineers have new design codes and new types of engineered wood products available. At the time of the birth of the modern timber bridge in the 1980s, most design codes around the world were written in an allowable stress design (ASD) format, where the main goal was to prevent the failure of the structure. Today, design codes have been rewritten in a load and resistance factored design (LRFD). The safety of people and vehicles crossing the bridge is still the main priority. Using LRFD, the function criteria of the structure can be defined. A function criterion can be to limit the deflection of a bridge in order to prevent damage to the asphalt layer, or to limit vibrations which would cause discomfort for the people crossing. Early timber bridges were constructed using large-dimension sawn timber beams. Most bridges built in Sweden are constructed using glue laminated timber (glulam) beams. Glulam is an engineered wood product in which short laminations are cleared of large imperfections (such as knots) and then glued together to form a larger section using multiple laminations. Glulam comes in much larger geometries than sawn timber and, in many cases, it has better controlled material properties.

Most research on the modern timber bridge was conducted in the USA in the 1990s in two federally funded research programmes. The first research programme focused on building bridges using local underutilised wood species. Many demonstration bridges were built during this programme, as it partly funded these bridges. The later of the two projects aimed at promoting conventional timber bridge systems that were effective, economical and long-lasting (Duwadi and Wood 1996).

As most demonstration bridges were constructed on rural roads, most research and tests have been performed on bridges subjected to lower loads which would be more representative of the assumed load during the service life of these bridges. The lack of specific knowledge, extensive experimental data and recorded long-term behaviour may result in the low utilisation of the material. Experimental tests have shown that timber bridges are capable of withstanding loads much greater than those suggested in the design codes. Recent studies have shown that deflection values in stress-laminated-timber decks are non-linear when larger loads are applied. This underlines the need better to understand the behaviour of timber bridges in order to develop new design models that are better suited to modern timber bridges.

One major concern for any type of bridge is its service life. Sustainable bridges need to be able to perform for many years with limited disruptions in function. Timber bridges face the same challenge as any type of new structure. The lack of old existing bridges makes it more difficult to learn and improve from existing knowledge. There are many old timber bridges around the world that are available for extensive studies. However, different wood species, treatment methods and structure types make the direct translation of knowledge difficult.

1.2 Research objectives

The principle objective of this study was to address the knowledge gaps relating to the structural behaviour of modern timber bridge decks subjected to concentrated loads of great magnitude. Most of the timber bridges built in Sweden are constructed using a stress-lamination technique. For this reason, the main focus has shifted to studying the behaviour of stress-laminated-timber (SLT) decks. Other types of timber bridge can be further studied using the knowledge acquired from studying SLT decks. The project has been divided into three parts.

Part I focuses on studying the applicability and structural behaviour of existing design models for SLT decks loaded in the serviceability limit state (SLS) and ultimate limit state (ULS). The sparse amount of data available in the literature comes from tests conducted on SLT decks using sawn timber instead of glulam beams which are used in Sweden. There is uncertainty about how applicable design codes are to SLT decks. Several parts of the existing design codes used today were developed for different types of SLT deck, for wood species other than those available in Scandinavia and using a different design philosophy. Within the scope of ***Part I,*** the objectives were to:

- Study how and when failure occurred in SLT decks constructed using glulam beams subjected to very high concentrated loads (ULS)
- Study the behaviour of SLT decks subjected to non-destructive loads (SLS)
- Investigate the material properties of SLT decks that can be used for design in Sweden

Part II of the project deals with understanding and modelling the sources of non-linear behaviour in SLT decks. Non-linear deflection behaviour was observed in several full-scale tests of SLT decks in the literature. Within the scope of ***Part II,*** the objectives were to:

- Develop and validate a numerical non-linear model for studying interlaminar slip in SLT decks
- Perform further testing of stress-laminated elements in order to isolate the parameters governing the non-linear slip

Part III of the project focuses on studying old existing bridges to learn about their durability and long-term performance. In North America, many old timber bridges are available for inspection. SLT decks are heavily dependent on the amount of prestress that is applied. There is an expected reduction in terms of prestressing level, but the amount of long-term prestress loss has been very difficult to predict. As a material, wood degrades when it is exposed to high levels of moisture. It is desirable to highlight the preventive measures that exist to reduce the degradation of wood in timber bridges. Within the scope of ***Part III,*** the objectives were to:

- Try to understand the factors governing the long-term performance of timber bridges by summarising the literature and inspecting existing bridges in the USA

1.3 Scope and scientific approach

The three parts of the project have mainly followed a chronological order, with some parts overlapping in time. ***Part I*** and ***Part II*** are connected to one another in multiple ways, as shown in Figure 1.1.

Part I focuses on studying previous research on the subject and applying that knowledge to our local conditions and it took place in 2008-2011. A literature study of other international timber bridge projects was conducted in order to obtain some technology transfer. The literature study resulted in the mapping of areas in which new or complementary knowledge was needed. Stress-laminated-timber decks in Sweden are internationally regarded as a second generation of SLT decks because of the use of engineered-wood products instead of timber. These decks are also subjected to heavier vehicles and an increased volume of traffic. Finite element analysis tools for the linear analysis of SLT decks were tailored to match the needs of the project. These tools were used to design both full-scale tests of bridges and also medium-scale tests of material properties. Medium-scale tests were conducted in an attempt to develop linear material properties for SLT decks in Sweden. The results demonstrated the complexity of SLT decks and indicated that full-scale tests were required in order to evaluate and understand the performance of SLT decks. Full-scale tests of SLT decks for bridge applications were constructed, tested and analysed. The results revealed the non-linear behaviour of SLT decks, which highlighted some shortcomings in available design models. The need for improved and more complex analysis methods for studying the non-linear behaviour of SLT decks was demonstrated.

Little was described in the literature about the mechanism causing non-linear slip in SLT decks and even less knowledge was available on how to model this slip. ***Part II*** of the project was carried out between 2010 and 2013. A literature study showed that one approach to modelling slip in SLT decks was tests using a non-linear material formulation for a solid deck. This model lacked the feature of modelling local discontinuities in a deck, such as butt joints. Another disadvantage was that the non-linear material formulation was not available in any commercial FE software. A new approach to modelling the global behaviour of SLT decks using available commercial software packages was applied. A model using a linear material formulation together with multiple contact formulations was developed. Good agreement was observed when this model was used for modelling the tests performed in Part I of the project. The contact formulations required input parameters for the friction behaviour of glulam beams in contact with other glulam beams. Test series of small-scale specimens were conducted to establish accurate values for the coefficient of friction for glulam beams. Further tests were required in order fully to understand the parameters governing the interlaminar slip in SLT decks. Multiple medium-scale stress-laminated elements were tested with varying parameters of prestress, geometry, load configuration and butt-joint configuration. All these tests were analyses using the non-linear FE model with individual contact formulations. The numerical models were again validated and could be used to further study test configurations, as well as stress behaviour in the tested elements.

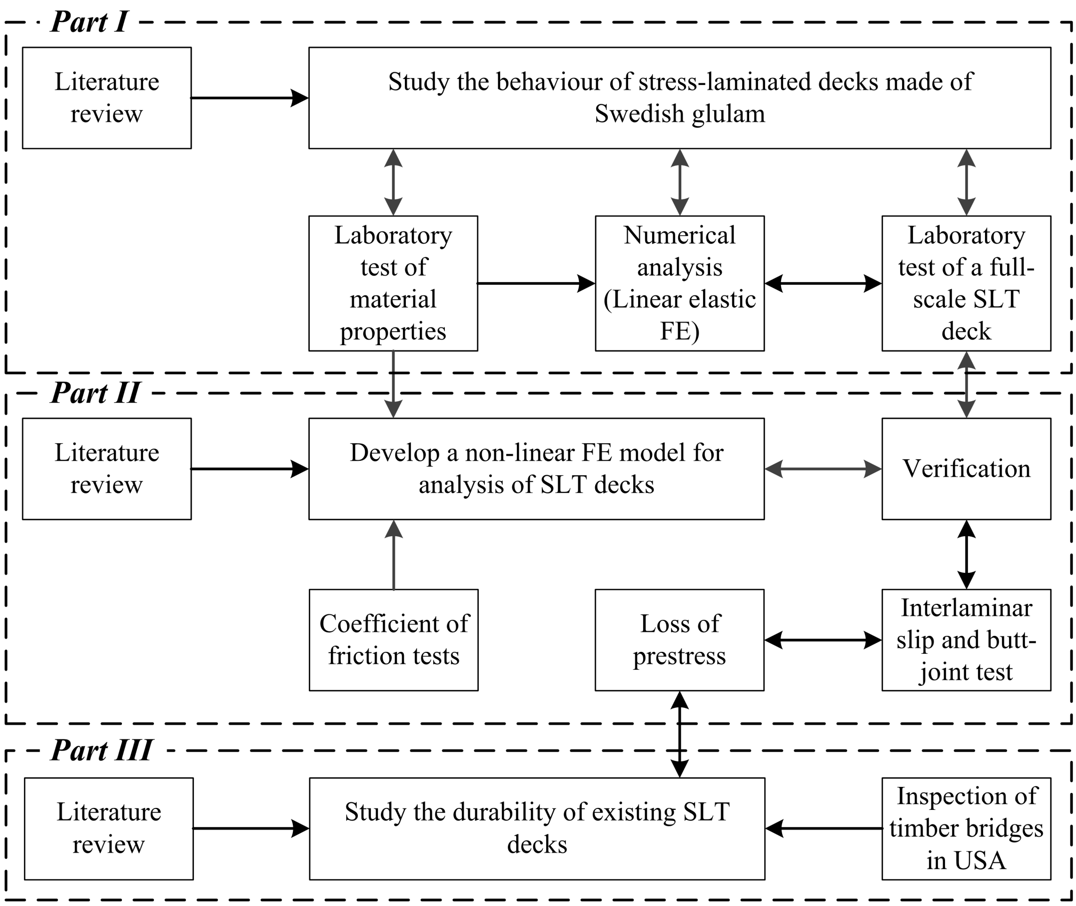


Figure 1.1 Scientific approach of the project

Durability is of great importance for any type of bridge. Like any other type of bridge, timber bridges require regular maintenance and inspections to ensure their safety and long-term performance. ***Part III*** of the research (between 2012 and 2013) focused on studying the durability of timber bridges and the factors influencing their long-term performance. Most of the timber bridges in Sweden are not of an age significant enough to have experienced sufficient long-term degradation. There are many old timber bridges in the USA and they were studied in the field and in the literature. However, most bridges in the USA are not constructed in a similar manner to the second-generation SLT decks constructed in Sweden. Comparisons of practised inspection and maintenance routines were made in order to try and apply existing knowledge to the timber bridges in Sweden.

1.4 Limitations

Several timber bridge types are based on the principle of stress lamination. Stress-laminated box bridges and T bridges are elaborations of the stress-laminated timber deck. However, only SLT decks have been studied experimentally in tests and in documented literature.

Timber bridges are built using several different kinds of wood species around the world. However, in Sweden, almost all timber bridges are constructed using conifer wood and spruce in particular. All test specimens have been constructed using conifer wood. Most of the numerical modelling has been conducted using the assumption of European softwood.

SLT decks are highly dependent on the prestress between laminations. Generally, this stress is generated by stressing high-strength steel bars, which are anchored by a system of hardwood and/or metal plates. Different researchers have studied some alternative materials for prestressed rods and anchorage systems. This thesis does not cover other types of prestressed tendon and/or other prestressed anchorage configurations.

Stress-laminated-timber decks in Sweden are generally designed for a service life of 40 or 80 years, depending on factors such as the weather protection, geographical location and amount of traffic. Several long-term effects such as stress relaxation, creep and mechano-sorptive creep in timber are very difficult to model. Long-term effects have not been studied in any tests. For this reason, the time-dependent long-term effects have also been ignored in the numerical modelling.

The tests in the project have not been extensive enough to produce statistically significant results that can be directly implemented in the current design codes. Most tests, together with the numerical models, can give strong indications of the behaviour of SLT decks.

2 Stress-laminated-timber decks

2.1 History and development around the world

Stress lamination for bridges began when engineers started to put steel bars over and under nail-laminated decks to prevent beams separating from one another in such a manner that the bridge became unusable. The post-tensioning of the nail-laminated timber decks not only restored the original capacity, it also increased the load distribution between the beams. The first reported research on the post-tensioning of laminate-timber decks was conducted in the form of a collaborative project between the Ontario Ministry of Transportation and Communication and Queen's University in Canada. The research programme resulted in the first design specifications for stress-laminated-timber (SLT) decks included in the Ontario Highway Bridge Design Code (OHBDC 1983).

Many researchers have studied the suitability of timber bridges as a part of modern infrastructure. The United States of America was the centre of timber-bridge research during the last two decades of the 20th century. Gutkowski and Williamson (1983) performed a state-of-the-art study in which several timber-bridge types and preservation techniques were studied. Gutkowski and McCutcheon (1987) studied the performance of 18 timber bridges that were built during the 1960s and 1970s. The results revealed that the timber decking used at the time was generally very wet but had protected the underlying stringers from getting wet. As a result, calculations for dry timber could be used for timber stringer bridges but not for stress-laminated timber decks.

Oliva and Dimakis (1988) performed a study on a part of a full-scale SLT deck designed as a highway bridge. Constant demands for replacement bridges on rural roads initiated this experiment. They studied the benefits of using timber bridges as a highway structure. Further steps forward were made when the Forest Products Laboratory (FPL) presented a report on recent developments in the area of SLT bridge decks, written by Olivia, Dimakis et al. (1990). This report presented several changes and updated material values compared with what was written in the OHBDC (1983). Sarisley (1990) studied the construction methods and costs of stress-laminated-timber decks. He identified several aspects in the construction phase that give timber bridges an advantage over conventional construction material. These aspects included short construction and assembly times, low weight and low cost.

In 1990, Michael Ritter and the US Forest Service published a comprehensive guideline on the design, construction, inspection and maintenance of timber bridges. This was the first guideline to include all aspects relating to timber bridges. The publication contains information about basic timber knowledge, general bridge concepts and detailed descriptions of how different types of timber bridge can be designed and maintained. The US Forest Service, Forest Products Laboratory (FPL) and the Federal Highway Administration (FHWA) set up a joint co-operative research programme between 1991 and 1996 based on a national assessment of research needs for timber bridges. The project resulted in advances within many areas, such as system development, design properties, preservatives, inspection and rehabilitation, to mention just a few (Ritter and Duwadi 1998).

Crews began the Australian research on timber bridges in the early 1990s. During the course of approximately a decade, he developed techniques for SLT decks and stress-laminated cellular timber bridge decks adapted for Australian materials and conditions. Crews and Walter (1996) presented a review of timber bridge development in Australia. During the five years included in the review, 32 timber bridges of different types were planned and/or constructed. Two full-scale ultimate-load tests were performed, with the conclusion that SLT decks still carry a significant load after failure. During the following five years, much research focused on cellular timber bridge decks made from LVL beams as webs and timber laminations in the flanges (Crews 2001; Crews 2002). Crews collaborated with researchers from the USA on the transition from an allowable stress design (ASD) code to a load and resistance factored design (LRFD) code for timber bridges (Crews and Ritter 1996).

The Nordic Timber Bridge Project was a collaborative project between the four Scandinavian countries, Sweden, Denmark, Norway and Finland. The project was active between 1996 and 2002 and started with the extensive transfer of knowledge from the United States. During this period, no fewer than 34 reports were produced on different aspects of timber bridge construction. Some of the aspects covered in the project were a market survey, design rules, stress-laminated bridges, truss bridges, joints, service life and information. Several reports on stress-laminated bridge decks were produced in Sweden within this project. These reports cover aspects such as climate influence, bar force loss, field measurements and design values, to mention just a few (Marklund 1997a; Marklund 1997b; Pousette 1997; Pousette 2001; Pousette et al. 2002).

The state of São Paulo in Brazil initiated a timber bridge programme in 2001. As in many parts of the world, São Paulo has an ever-growing need to rehabilitate its road network and timber bridges could be a possible solution. Eleven timber bridges of different types were built during a period of five years (Calil Jr 2008). New bridge concepts, as well as adaptations of existing timber bridge technologies to local wood species, were studied during this project (Cheung and Calil Jr 2004; Fonte and Calil Jr 2004; Cheung et al. 2006; Calil Jr 2008).

In Norway, Dahl, Bovim et al. (2006) performed a full-scale test on a stress-laminated-timber bridge deck subjected to failure loads. They concluded that delamination occurred in the deck at moderate loads, which made linear models insufficient. Norway has had several successful research projects working with timber bridge aspects such as bridge aesthetics, connections and environmental impact (Hammervold 2010; Malo and Ellingsbø 2010; Stensby et al. 2010; Svanæs 2010). The Norwegian Transport Administration (Vegvesen) has encouraged and financed the construction of some of the most innovative timber bridges in the world, built to this day.

In Switzerland, a stress-laminated-timber deck was built using only wood and fibre-reinforced polymers (FRP) (Brönnimann and Widmann 2010). The dynamic damping characteristics of cable-stayed timber deck bridges made from a cross-laminated timber (CLT) deck and an SLT deck have also been studied (Schubert et al. 2010).

2.2 Load redistribution

One of the main purposes of stressing beams together is to create a deck with stiffness in the transverse direction. Concentrated loads are distributed across several beams instead of just the one in direct contact with the load. In Figure 2.1 [A], the load only deforms the loaded beam, as there is no connection between the beams. In Figure 2.1 [B], the load is transferred to several beams due to the contact forces in the interlaminar surface between the beams. Friction forces redistribute the load from one to multiple adjacent beams. Very little prestressing force is required for the beams to start acting as a deck plate. Many researchers (Batchelor et al. 1981; Oliva and Dimakis 1988; Sarisley and Accorsi 1990; Dahl et al. 2006) have studied how a prestressing force creates a deck plate with orthotropic material behaviour.

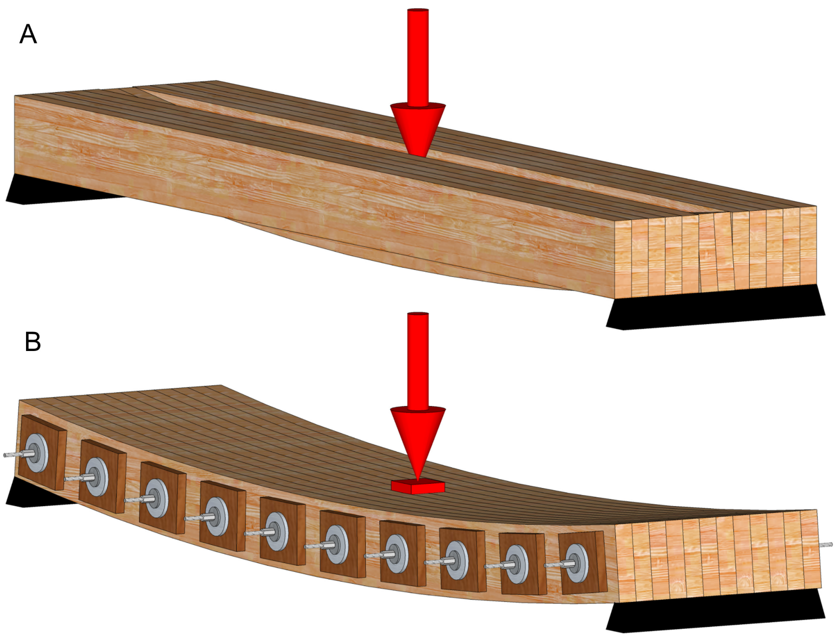


Figure 2.1 *Load redistribution between beams as an effect of prestressing. [A] No prestressing applied, only directly loaded beams deflect. [B] When prestressing is applied, the load is distributed to several beams which deflect.*

2.3 Interlaminar slip and gap

The concept of stress-laminated-timber decks is highly dependent on the prestressing between the beams. Today, prestressing is generated by means of high-strength steel bars positioned through pre-drilled holes in the deck. Normally, the holes in the deck are twice the size of the prestressed bars. Contact between the beams and the prestressed bars is undesirable. The prestressed bars are loaded to a stress level close to the yielding strength of the bar. Any surface damage to the bars could result in stress concentrations and, in the worst case, rupture in the bar.

In the past, two failure modes (transverse gaps and vertical slip) have been prevented from occurring in SLT decks. However, in this project, a third and a fourth distortion mode are suggested. The most significant slip mode has not been considered in the literature.

The prestressing force needs to be of such a magnitude that no gaps are allowed to form due to the transverse bending moment in the plate, cf. Figure 2.2 [A]. Gaps between beams in the deck would reduce the interlaminar contact surface area. The risk of moisture and debris entering the deck increases significantly when gaps are formed.

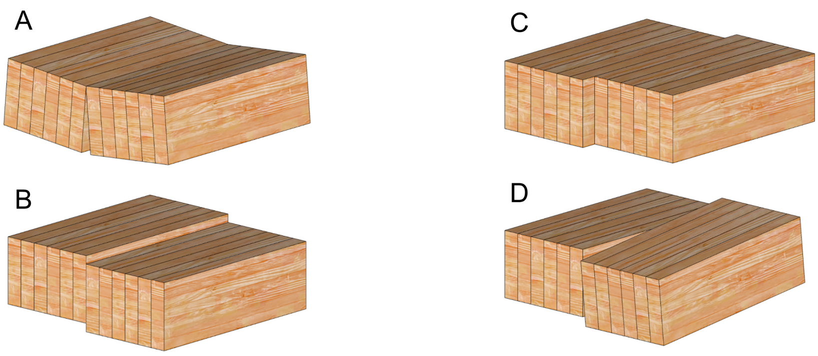


Figure 2.2 Interlaminar distortion modes for stress-laminated decks.
[A] Gaps forming due to transverse bending
[B] Vertical slip between beams due to transverse vertical shear
[C] Horizontal slip between beams due to transverse horizontal shear
[D] Combination of vertical and horizontal slip due to deck twisting

The prestressing, together with the coefficient of friction, should generate a resistance to vertical interlaminar slip. Figure 2.2 [B] illustrates slip between beams due to high concentrated loads. Vertical interlaminar slip is a major cause of stress redistribution in the deck and could also damage the moisture sealing or the asphalt layer. Vertical interlaminar slip may occur at high loads, but it is almost always preceded by twisting slip (***Paper III***).

There is a mode of distortion which is generally not regarded as a failure mode in the literature. Horizontal interlaminar slip, cf. Figure 2.2 [C], may occur if the horizontal loads become too large. Uniform horizontal slip is very uncommon and most horizontal slip actually comes from the twisting distortion.

The last distortion mode is twisting of the beams, cf. Figure 2.2 [D]. This is basically a combination of horizontal and vertical slip. Interlaminar slip occurs when the twisting moment in the plate is too large. Most slip of this type occurs as horizontal slip at the top and bottom of the beam. It has been shown that this distortion mode occurs at fairly low load levels and is the main cause of the global non-linear behaviour in SLT decks (***Paper I**, **Paper III** and **Paper IV***).

2.4 Intentional imperfections

Longitudinal lamination joints (commonly referred to as butt joints) can be regarded as imperfections in the deck, although they are intentional and should not be confused with natural imperfections, such as knots or grain deviation in laminations.

The use of butt joints enables the stress-laminated-timber deck to span distances greater than the maximum length of the individual beams. This is a great advantage when SLT decks are made from sawn timber beams, which generally have a maximum length of approximately six metres. Even though the use of glulam beams as deck laminations increases the possible maximum length of the laminations, other factors such as transportation may limit the maximum length of the deck beams. The beams are unable to transfer any bending moment or longitudinal shear force over a butt joint. The load has to be transferred to the adjacent beams past the butt joint.

In his design guide, which in the case of butt joints is based on recommendations in the OHBDC (1983), Ritter (1990) stated that not more than one in four (1 in 4) adjacent laminations should be butt jointed within a distance along a span of four feet (approximately 1.2 m). If a one-in-four pattern is used (as shown in Figure 2.3), the reduction in the effective deck width is approximately 20%. If a one-in-six (1 in 6) or one-in-eight (1 in 8) pattern is used instead, the reduction is 12% and 7% respectively.

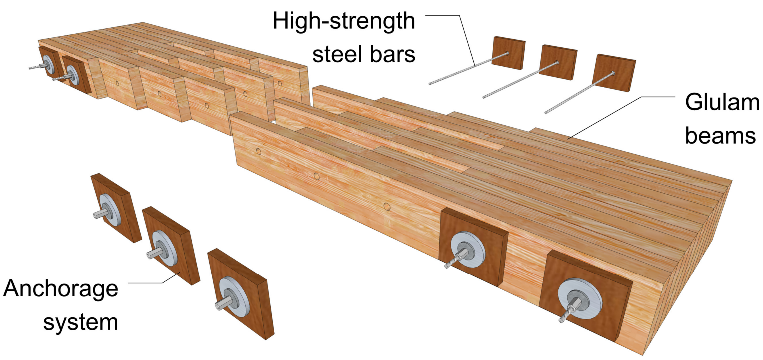


Figure 2.3 Butt joints in a stress-laminated-timber deck with a one-in-four pattern.

2.5 Prestress anchorage

The first SLT bridges were constructed with continuous steel profiles along the edges of the deck, cf. Figure 2.4 [A]. The steel profile enabled the prestress to be modelled as concentrated loads acting on a continuous steel beam supported by an elastic foundation, as described by Sarisley and Accorsi (1990). However, Oliva et al. (1990) showed both experimentally and analytically that the use of a continuous steel profile was not as favourable as previously thought.

AASHTO (2010) still recommends that SLT bridges constructed using softwood beams should use continuous steel profiles along the edges of the deck, cf. Figure 2.4 [B]. The two steel bars, over and under the deck, have been replaced by a single bar through the centre of the deck. If the deck is constructed using hardwood beams, or if at least the two beams closest to the edge are made of hardwood, the prestress can be anchored using a system of discrete steel plates instead of the continuous steel profile, cf. Figure 2.4 [C]. A system with discrete plates acting as several patch loads along the edge of the deck will cause the more uneven distribution of the prestress compared with the system with continuous steel profiles along the edge.

The SLT decks constructed in Sweden use a combined anchorage system of one or several wood, aluminium and steel plates to distribute the prestressing force to the beams. An example of a system of this kind is shown in Figure 2.4 [D], Figure 2.1 [B] and Figure 2.3. ***Paper IV*** shows the prestress distribution along the length of the edge beam for a stress-laminated deck. The numerical study showed that the width of the deck beams has an influence on the prestress distribution in the deck.

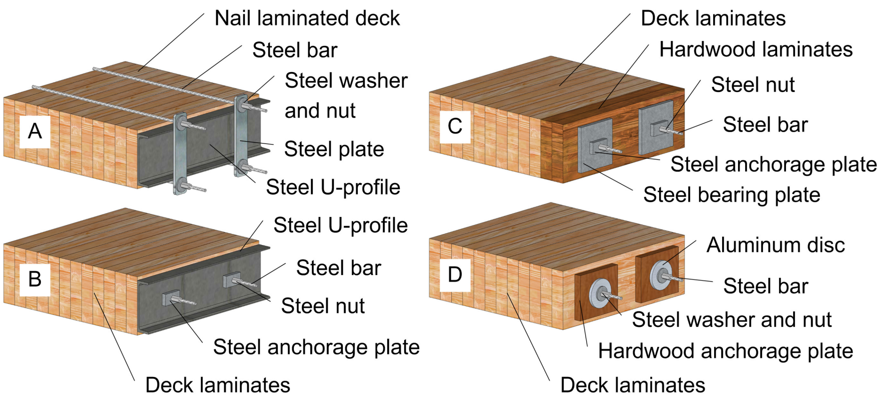


Figure 2.4 Illustration of prestress anchorage systems
[A] Measure of repair of nail-laminated decks
[B] Prestressing system with continuous steel profile used for softwood decks according to AASTHO
[C] Prestressing system with bearing plates used for hardwood decks according to AASTHO
[D] Prestressing system normally used in Sweden

2.6 Summary and key findings

- Research on timber bridges in general and SLT decks in particular has been conducted all over the world for the past forty years. There are many innovative bridges that have generated new knowledge.
- Concentrated loads can be successfully redistributed to several beams by stressing beams together. Little prestress between the beams is required for the beams to act as a deck plate.
- There are four interlaminar failure modes that need to be considered when designing SLT decks. However, only interlaminar gap and vertical slip have been of interest in the past. Horizontal slip and twisting have been identified during the current study.
- Beams with a limited length can be used successfully to create a deck with a span longer than the individual length of the beams when butt joints are used. A reduction in the strength and stiffness of the deck has to be considered.
- The anchorage system for SLT decks has evolved from a system that used continuous steel beams to a system comprising discrete steel or timber plates.

3 Experimental tests

Experimental tests have played a significant role in determining the behaviour of stress-laminated-timber (SLT) bridge decks. Studies of material properties related to the orthrotropic behaviour of decks and full-scale bridge decks have been conducted in order to acquire an understanding of deck behaviour. Many field measurements of SLT decks have been reported in the literature. Few of them have measured and inspected the function of SLT decks constructed using glulam beams instead of sawn timber beams.

The first reported SLT deck constructed using glulam beams was built in Wisconsin, USA, in 1989. Wacker and Ritter (1992) presented the monitoring observations for the first two years of the service life of the deck. It was load-tested twice during the first few years. Good correlation between the two tests and the FE model was observed.

A recent report published by (Pousette and Fjellström 2012) presented the load-deflection behaviour of five Swedish SLT decks constructed using glulam beams. One deck (Klockabergsbron) was inspected and load-tested for the second time 14 years after the first load test. No critical discrepancies between the two measurements were observed.

Bakht (1988) studied how the load is distributed in a laminated timber deck laid on steel girders. He presented an idea relating to how the load was distributed to a strip with a width that was equivalent to the contribution from the transverse stiffness of the deck. The equivalent strip could be calculated by integrating the entire deflection curve under a concentrated load. The deflected area could then be divided by the largest value for the deflection. A two-dimensional beam that generated the same maximum deflection as the deck could then replace the three-dimensional deck.

Davalos et al. (1996) studied the transverse modulus of elasticity (MOE), the in-plane shear modulus and the out-of-plane shear modulus. Tests were conducted on test specimens consisting of sawn timber that were either glued or stressed together. The influence of prestressing levels was studied. Several other researchers have conducted tests in order to develop material properties for orthotropic deck behaviour (Batchelor et al. 1981; Oliva et al. 1990; Crews 2002; Dahl et al. 2006).

Several full-scale or partially full-scale tests of SLT decks have been conducted over the years. Oliva and Dimakis (1988) tested a section of a full-scale deck. Davalos et al. (1993) also tested sections of a full-scale deck in order to study the influence of butt joints. Crews (2002) conducted full-scale tests on SLT decks and SLT boxes. Crews also conducted ultimate load tests to failure. Dahl et al. (2006) performed a full-scale test to study the behaviour of SLT decks constructed in Norway using sawn Norway spruce (*Picea abies*) timber beams.

To the best of the author's knowledge, the first reported ultimate load test to failure of a SLT deck constructed using glulam beams was reported by Ekevad et al. (2011). The deck showed a significant amount of interlaminar slip when loaded to failure.

3.1 Material properties of SLT decks

The longitudinal modulus of elasticity in stress-laminated-timber bridge decks has been thought to be independent of the prestressing level (Batchelor et al. 1981; Oliva et al. 1990). Several studies of the field performance of stress-laminated-timber bridge decks have concluded that the transverse stiffness was reduced due to long-term prestress loss (Hilbrich Lee et al. 1996; Ritter et al. 1996a; Ritter et al. 1996b).

Mid-scale tests to determine the material parameters of stress-laminated-timber decks were performed during the spring of 2009 by the author (Karlsson et al. 2009). The objective was to study the elastic material properties of SLT decks made of glulam beams made from Norway spruce (*Picea abies*). The method used for the test was first suggested by Tsai (1965) for thin plates and subsequently applied to SLT decks by (Oliva et al. 1990). The test method proposed that all four corners of a plate should be loaded with concentrated loads, as shown in Figure 3.1.

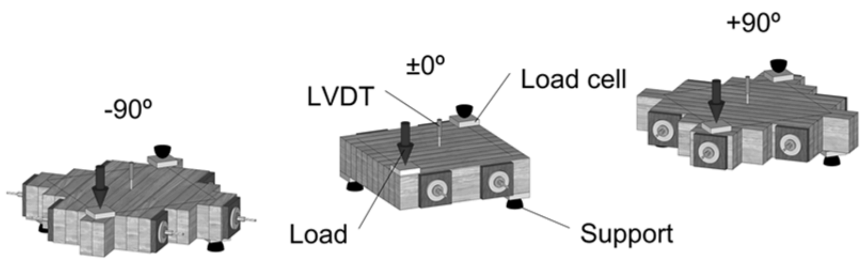


Figure 3.1 Test specimens used for mid-scale material tests. All the test specimens were loaded in one corner and had supports at the other three corners. The material was rotated both ± 90° in relation to the specimen in the centre.

By testing three SLT deck specimens with the beams oriented in three different directions in relation to the load and support configuration, the material parameters, E_T and G_{LT}, could be calculated by measuring the deflection in the centre of the specimen and by knowing the value for E_L. E_L and E_T are the longitudinal and transverse bending moduli of elasticity, while G_{LT} is the in-plane shear modulus. E_L was determined by measuring the dynamic modulus of elasticity of the beams.

The three test specimens had dimensions of 1.1 x 1.1 x 0.315 m³ between the supports and were made of glulam beams with a cross-section of 115 x 315 mm². Five different prestressing levels were tested to investigate the influence of the mechanical material parameters of the deck. The five prestressing levels were 290, 460, 650, 790 and 910 kPa. The beams used for the test specimens were very dry after having been stored in a dry, heated area for a long time. The moisture content of the timber was 10% or less. This influenced the results significantly, since the coefficient of friction is dependent on the moisture content.

The results of the tests revealed that the in-plane shear modulus, G_{LT}, varied between 0.7% and 3.0% of the E_L value, depending on the prestressing level. The test results were significantly influenced by the thickness of the test specimens. Local interlaminar slip was observed, instead of the desired global deflection. Oliva et al. (1990) performed a similar test using the same test procedure as described above. They observed similar behaviour where the depth of the deck influenced the results. Crews (2002) performed a similar test on SLT deck specimens which were three times larger, apart from the thickness, which was approximately the same (300 mm). This resulted in higher values for the in-plane shear modulus and the transverse MOE. The results of the tests are shown in Table 3.1, together with other reported material properties found in the literature.

The transverse bending modulus, E_T, varied between 0.6% and 1.7% of the E_L value. The test of the transverse bending modulus was also influenced by the thickness of the deck compared with the length of the sides. The results are slightly lower than those observed by Oliva et al. (1990). The high values suggested by OHBDC (1983) have been shown to be too high and have since been lowered.

Table 3.1 Studies of material properties of SLT decks. Transverse stiffness, E_T, and in-plane stiffness, G_L, are expressed as a percentage of longitudinal stiffness, E_L.

Study	**Material**	**E_T / E_L [%]**	**G_{LT} / E_L [%]**	**Reference prestressing level [kPa]**
OHBDC (1983)	Sawn timber	5.0	6.5	-
Oliva et al. (1990)	Sawn timber	0.9-1.7	1.0-1.7	69-689
Ritter (1990)	Sawn timber	1.3	3.0	> 280
Davalos et al. (1996)	Planed glulam	1.0-2.9	5.1-6.1	172-689
Crews (2002)	Planed timber	1.4-2.0	2.9	> 700
EN 1995-2 (2004)	Sawn timber/glulam	1.5	6.0	> 350
EN 1995-2 (2004)	Planed timber/glulam	2.0	6.0	> 350
Dahl et al. (2006)	Sawn timber	3.1	4.2	600
Values from test (Karlsson et al. 2009)	Planed glulam	0.6-1.7	0.7-3.0	290-910

The tested specimens could not be regarded as thin plates (a thickness less than 10% of the length of the plate) and the results were therefore significantly influenced by the depth of the glulam beams. The tests indicated that a full-scale test similar to that conducted by Dahl et al. (2006) was needed in order better to understand the behaviour of SLT decks without the impact of size effects.

3.2 Full-scale SLT deck test

Oliva and Dimakis (1988) tested a 14.6 m long, 2.6 m wide section of a stress-laminated-timber bridge in the laboratory. The SLT deck was constructed using butt joints with 1.2 m of longitudinal spacing. The results of the study concluded that the recommended material parameters describing the orthotropic behaviour of SLT decks in OHBDC (1983) should be lowered. Further studies by Oliva et al. (1990) showed how the material parameters of SLT decks are dependent on the prestressing level.

Crews (2002) conducted full-scale tests on both SLT decks and stress-laminated box bridges. The material properties of SLT decks made of Australian wood species were established. The majority of this study focused on stress-laminated box bridges and especially on slip between web and flange beams. Crews presented indications that SLT decks have remarkable redundancy, with the capability to carry substantial vehicle loads even after being loaded to failure.

A full-scale test was performed in Norway by Dahl et al. (2006) in order to investigate the behaviour of stress-laminated-timber decks made of wood grown in Norway (*Pinus sylvestris*). The material properties of the deck were obtained by optimising the numerical model compared with the test results. The study concluded that gaps and slip formed, even at non-destructive load levels.

The first full-scale test of an SLT deck made of Swedish glulam was presented in a paper by Ekevad et al. (2011). A 10.6 m long, 5.0 m wide deck was constructed from 495 mm deep glulam beams. One load position consisting of a single patch load was tested. The prestressing level in the deck was approximately 400 kPa. Interlaminar slip in the form of horizontal slip was observed during the initial loading sequences (Ekholm 2011).

From the studied literature, it became clear that there were still several questions regarding the behaviour of SLT decks that were unanswered. A full-scale test, presented in detail in ***Paper I***, was designed and performed in order to try and answer the following questions:

- What is the difference between a load axle close to the edge compared with a load axle in the centre of the deck?
- How much is the stiffness of the deck influenced by the prestressing level?
- How does the deck behave at loads close to the failure load?
- What is the failure load and failure mode of this specific SLT deck configuration?
- How does the SLT deck behave when loaded after it has failed?

A 5.4 m long, 8 m wide deck was constructed using 84 glulam beams with a cross-section of 95 x 270 mm (width, depth). The modulus of elasticity (MOE), density and moisture content were measured for each beam used in the deck. No butt joints were used in this test. However, the deck was subsequently rebuilt as a continuous deck that was 10.8 m long and 4 m wide, using butt joints. More information about the continuous deck can be found in Ekholm (2011).

Two load positions were tested. The load was first positioned in the centre of the span and in the centre of the width of the deck, cf. Figure 3.2 and Figure 3.3 [A]. The load was subsequently moved close to the edge of the deck, cf. Figure 3.3 [B]. A load configuration consisting of two patch loads was chosen to represent a vehicle axle. A 300 kN axle load was used for the non-destructive tests. The non-destructive test load was chosen to represent the characteristic load value of one of the vehicle axles from load model 1 in EN 1991-2 (2003). The ultimate load was applied until multiple beam failure in the deck was obtained.

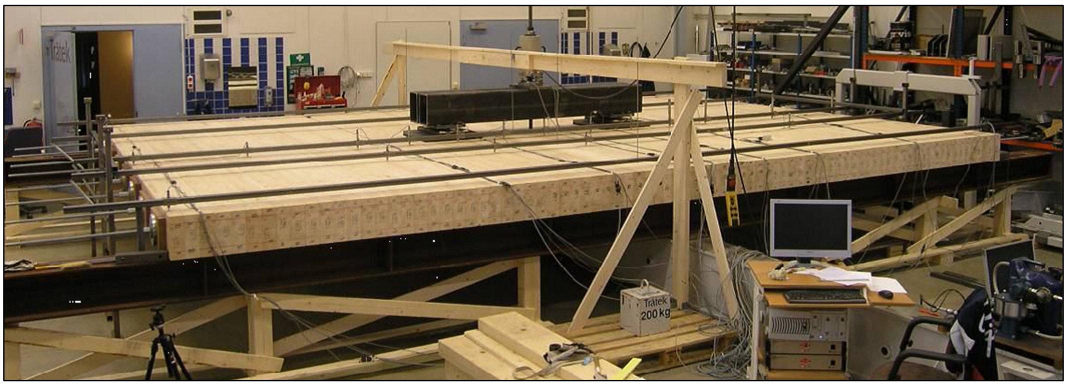
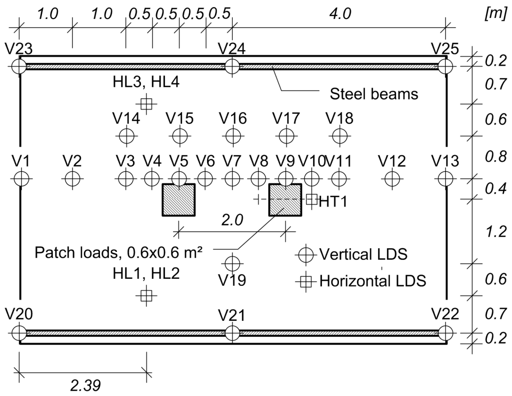


Figure 3.2 Test set-up for the 5.4 x 8.0 m deck with the load positioned in the centre of the deck.

The deck was stressed together using eight prestressed bars spaced 675 mm apart. Three prestressing levels of 900, 600 and 300 kPa (assuming uniform pressure) were used to study the influence of prestress reductions.

Deflections were measured using Linear Variable Differential Transformers (LVDT) positioned along and across the deck. Measurements were made of the longitudinal slip between beam and the gaps forming between beams from transverse bending. The layout of the deck, positions of sensors and load are shown in Figure 3.3.



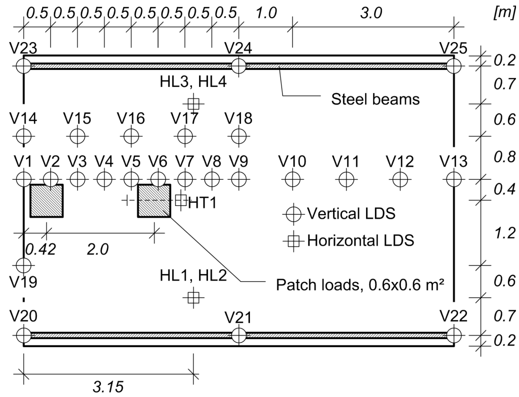


Figure 3.3 Loads in the mid-span, steel beam supports and LVDT positions (V1-V25, HL1-HL4 and HT1) for the simply supported 5.4 x 8.0 m deck.
[A] Concentric load position (tests NDT1-NDT3)
[B] Eccentric load position (test NDT4-NDT6 and ULT1)

The results of the non-destructive tests (NDT) revealed that the measured deflection values for the low prestressing level (300 kPa) were about 11-13% higher compared with the values measured for the high prestressing level (900 kPa). The difference in deflection values between the 900 kPa prestressing level and the 600 kPa prestressing level was about 3-6%. The results of the non-destructive tests are shown in Table 3.2.

Table 3.2 Summary of the full-scale test conducted on the 5.4 m by 8 m SLT deck. A total of six non-destructive tests (NDT1-NDT6) and one ultimate-load test (ULT1) were performed.

Test	**Load pos.**	**Prestress [kPa]**	**Max. load [kN]**	**Max. def. [mm]**
NDT1	Concentric	900	300	9.0
NDT2	Concentric	600	300	9.5
NDT3	Concentric	300	300	10.2
NDT4	Eccentric	900	300	18.2
NDT5	Eccentric	600	300	18.7
NDT6	Eccentric	300	300	20.2
ULT1	Eccentric	600	900	65.8 (before failure) ~115 (after failure)

Figure 3.4 shows the deflection profiles for the deck at a cross-section taken 400 mm away from the mid-span cross-section, cf. Figure 3.3. The deflection profiles for the three different prestressing levels are shown for 16.7%, 50% and 100% of the non-

destructive load. The deflection profiles verified how the transverse stiffness of the deck is dependent on the prestressing level, as shown by Oliva et al. (1990) and Dahl et al. (2006).

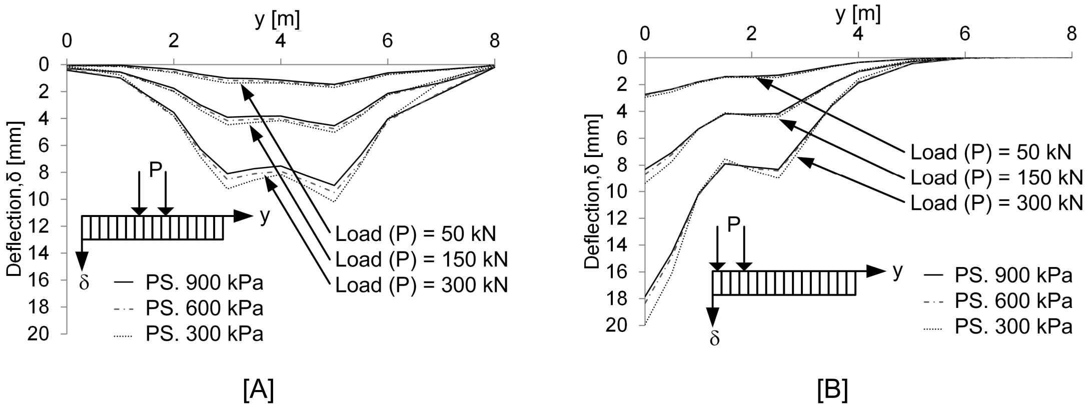


Figure 3.4 Deflection profiles across the SLT deck for vertical LDSs V1-V13 (close to mid-span) for three different prestressing levels (denoted P.S. in the figure) and at three different load levels.
[A] Profiles for concentric load position
[B] Profiles for eccentric load position

Horizontal interlaminar slip between beams in the deck (cf. Figure 3.5 [A]) was observed during the non-destructive tests. This confirmed the findings made by Ekevad et al. (2011) but for a different load configuration and a different deck geometry. The horizontal interlaminar slip initiated at load levels of approximately 150-200 kN. The full-scale tests also measured the transverse gaps over a distance of ten laminations. The sum of the gaps is shown in Figure 3.5 [B]. The largest gap between two beams was approximately 0.5 mm.

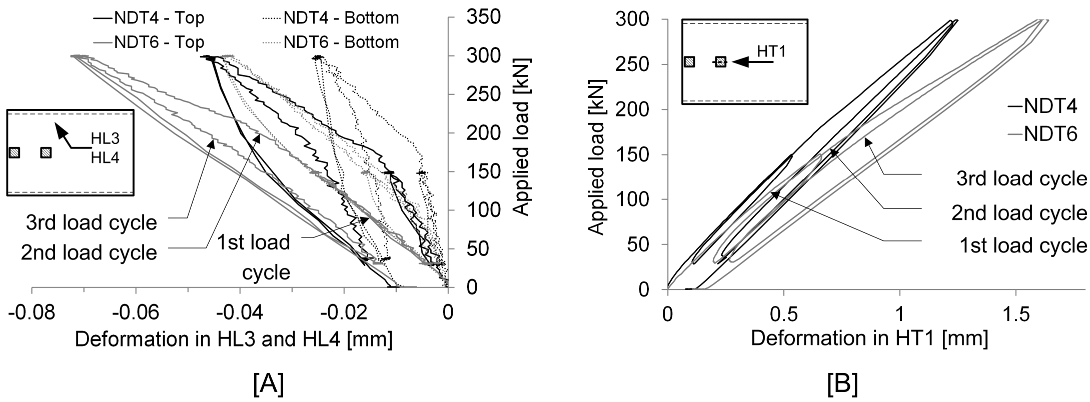


Figure 3.5 [A] Deformation in sensors HL3 and HL4, cf. Figure 3.3, showing the horizontal slip at the top and bottom surface between two beams in the deck during tests NDT4 and NDT6.
[B] Deformation in sensor HT1, cf. Figure 3.3, showing the size of the gaps forming between ten beams in the deck during tests NDT4 and NDT6.

The ultimate load capacity of the tested SLT deck was 900 kN on a single axle. The seven laminations closest to the loaded edge failed. The failure modes were bending failure, shear failure or a combination of both. Ekevad et al. (2011) reported bending failure in the beams at a load of 590 kN. The largest horizontal slip for the ultimate load test was approximately 0.31 mm. The sum of the horizontal gaps between ten laminations was 5.8 mm. The largest individual gap was measured as approximately 1.4 mm.

After the deck had failed and the load was removed completely, the deck was immediately reloaded up to a load level of 400 kN without any additional beams failing. Similar redundancy for a tested SLT box bridge was observed by Crews (2001).

3.3 Coefficient of friction tests

An extensive test series of 480 tests was conducted by Kalbitzer (1999). The aim was to study how the coefficient of friction (COF) varies depending on wood species, moisture content, surface roughness, wood grain direction and prestressing level. The findings from the study were that the direction of the motion relative to the wood grain and the surface roughness had the greatest influence on the COF for wood with a moisture content below 14%. For a higher moisture content, the prestressing level was most important.

In 2010, a test series comprising 48 tests was conducted on 30 different test specimens. The test was conducted by SP Technical Research Institute of Sweden (SP) in Skellefteå. The tested specimens were pieces cut from the glulam beams used in the full-scale SLT deck test (cf. Chapter 3.2). The specimens were planed glulam and were tested at three different prestressing levels of 300, 600 and 900 kPa. Tests were made to study the influence of the direction of the motion relative to the wood grain. The test set-up is shown in Figure 3.6. The results of the tests are shown in Table 3.3. Test data are available in a report from SP (2010).

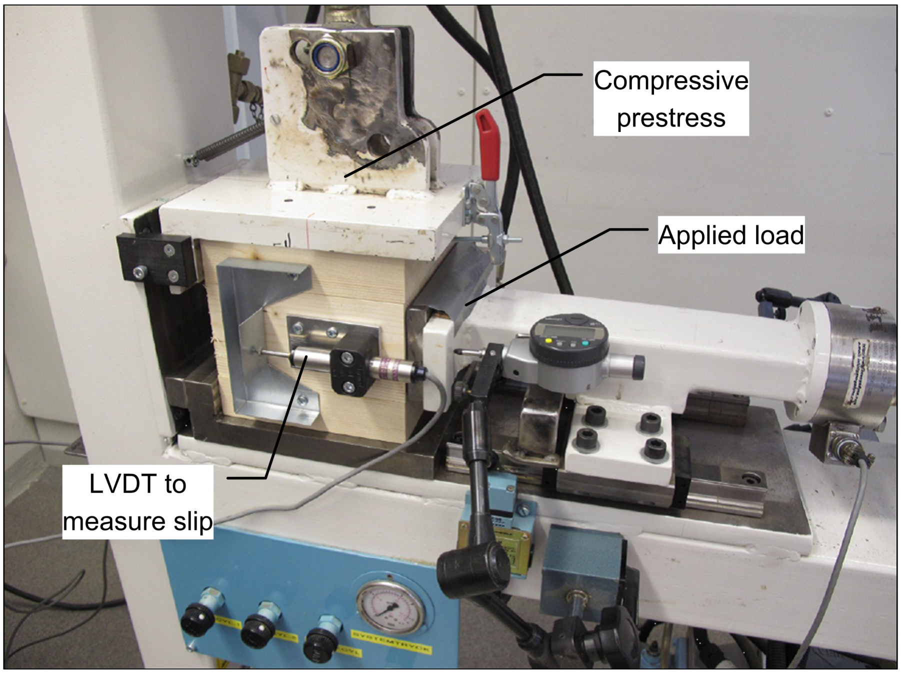


Figure 3.6 Test set-up for the COF tests 1 and 2. The prestress was applied vertically, while the load was applied horizontally. The amount of horizontal slip was measured by LVDTs mounted on both sides of the tested specimen. Photo taken from (SP 2012)

A second test series comprising 37 COF tests was conducted in 2012 at SP in Skellefteå. Every test was performed three times in order to investigate the repetitiveness of the COF. No significant change in the COF was observed between the three repetitions. The test specimens were cut from the butt joint and interlaminar slip tests (Chapter 3.4). Prestressing levels of 100, 300 and 600 kPa were studied for tests loaded both parallel and perpendicular to the wood grain. Two of the tests were made with thin metal strips with punched-out holes in order to enhance the friction, cf. Figure 3.7. The idea came from Nils Nilsson, a lab technician at Chalmers. This increased the amount of friction significantly. The same test machine that was used in the first COF test series was used for the second test series. Test data are available in a report from SP (2012).

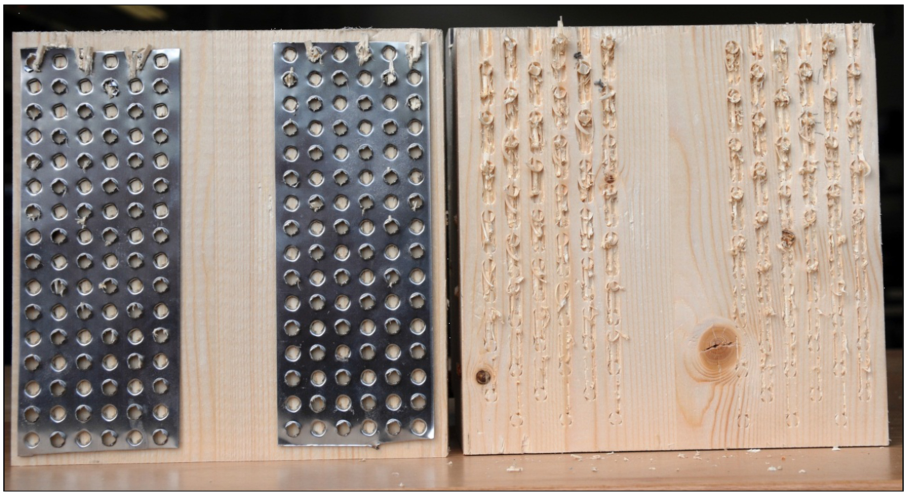


Figure 3.7 Test specimen with strips of punched-out holes before the test (left) and after the test (right). Photo taken from (SP 2012)

The results of the two test series (2010 and 2012) verified some of the results reported by Kalbitzer (1999). The moisture content of the tested specimens in test 1 and 2 was between 12.0-12.7%. The coefficient of friction varied significantly depending on the movement in relation to the direction of the wood grain. The COF for the test with the load parallel to the wood grain varied between 0.28-0.41. No correlation between prestressing level and COF could be observed. The COF values for loading parallel to the wood grain in EN 1995-2 (2004) is 0.17 for a moisture content of 12% and 0.3 for a moisture content of 16%, cf. Table 3.3.

The COF for a test specimen loaded perpendicular to the direction of the grain varied between 0.39-0.44. EN 1995-2 (2004) recommends a COF value of 0.2 for a moisture content of 12% and 0.4 for a moisture content of 16%, cf. Table 3.3.

The tested specimens with thin metal strips with punched-out holes increased the equivalent friction to a value of 1.53 to 2.49, when prestress of 300 kPa was used. No noticeable difference was observed between the test specimens loaded parallel and perpendicular to the grain.

Table 3.3 Specimen number, prestressing level, movement direction and result for all COF tests.

Movement direction	Prestressing level [kPa]	Moisture content [%]	Average COF [-]	Source [number of tests]
Parallel	100	12.2	0.35	Test 2 [3]
Parallel	300	12	0.41	Test 2 [3]
Parallel	300	12.7	0.29	Test 1 [5]
Parallel	600	12.2	0.33	Test 2 [3]
Parallel	600	12.6	0.28	Test 1 [5]
Parallel	900	12.3	0.3	Test 1 [5]
Parallel		12.3	0.33	Average of tests 1 and 2
Parallel		10	0.24	Kalbitzer (1999)
Parallel		12	0.17	EN 1995-2 (2004)
Parallel		14	0.3	Kalbitzer (1999)
Parallel		16	0.3	EN 1995-2 (2004)
Perpendicular	100	12.1	0.44	Test 2 [3]
Perpendicular	300	12.2	0.44	Test 2 [3]
Perpendicular	300	12.5	0.4	Test 1 [5]
Perpendicular	600	12.1	0.41	Test 2 [3]
Perpendicular	600	12.6	0.39	Test 1 [5]
Perpendicular	900	12.1	0.44	Test 1 [5]
Perpendicular		12.3	0.42	Average of tests 1 and 2
Perpendicular		10	0.28	Kalbitzer (1999)
Perpendicular		12	0.2	EN 1995-2 (2004)
Perpendicular		14	0.41	Kalbitzer (1999)
Perpendicular		16	0.4	EN 1995-2 (2004)

3.4 Butt joint and interlaminar slip tests

Butt joints have a weakening effect on stress-laminated decks. Research by Oliva et al. (1990) verified that the use of butt joints reduced the longitudinal stiffness of the deck. Butt joints also affect the effective deck width in terms of bending strength. The deck width reduction suggested by Ritter (1990) was subsequently verified by Davalos et al. (1993), who performed forty deflection tests on stress-laminated-timber decks with different butt-joint configurations. The decks were made of northern red oak and hickory. Three different butt-joint configurations and three different prestressing levels were investigated. The results of the tests were compared with existing theoretical predictions using linear regression analysis. The theoretical models predicted the bending deflection with good accuracy. Davalos et al. (1993) stated that the longitudinal stiffness of SLT decks with butt joints was prestress independent at prestressing levels above 172 kPa.

In Australia, several studies of the effects of butt joints were conducted during the 1990s (Crews 2001). The main difference compared with previous studies was the introduction of a variable called "development length". Since a butt joint is unable to transfer any bending moment (provided that the two beams are not in contact lengthwise), the load has to be transferred through friction between the butt-jointed beam and the adjacent beams. At a certain distance from the butt joint, the flexural capacity of the butt-jointed beam reaches its full capacity. This distance is defined as the "development length". Using this assumption as a minimum distance for design in Australia, every fourth lamella in an Australian softwood deck should be butt jointed with a minimum distance of 750-900 mm in the span direction. Crews (2001) also specifies a minimum prestressing level of 550 kPa, above which the butt-joint reduction factor is valid.

An initial study of the influence of butt joints was performed on an SLT deck constructed with a scale of 1:2 (Carlsson and Romero 2010). Planed 145 x 45 mm timber beams were used in a deck which was tested without butt joints and with butt joints with one-in-four and one-in-eight configurations. The decks did not follow the minimum longitudinal spacing recommended in the design codes. The test was constructed in order to generate the maximum influence of butt joints and interlaminar slip. The results of the initial study and the observations made in the literature resulted in the decision that further studies of butt-joint behaviour were needed.

A series of tests were conducted in order to study the influence of butt joints and interlaminar slip in stress-laminated-timber bridge decks (***Paper IV***). The tested specimens were 5,400 mm long and consisted of either nine 90 x 270 mm (width, depth) glulam beams or nineteen 42 x 270 mm glulam beams. Three different butt-joint configurations were tested and compared with a test specimen without butt joints, cf. Figure 3.8. Test specimens without butt joints but with a depth of 180 mm were also tested. The test specimens without butt joints were also tested with a single patch load at mid-span instead of the two patch loads, as shown in Figure 3.8. Prestressing levels of 100, 200, 300 and 600 kPa were used for the tests. More information about the test and the results of all the tests can be found in Ekholm (2012).

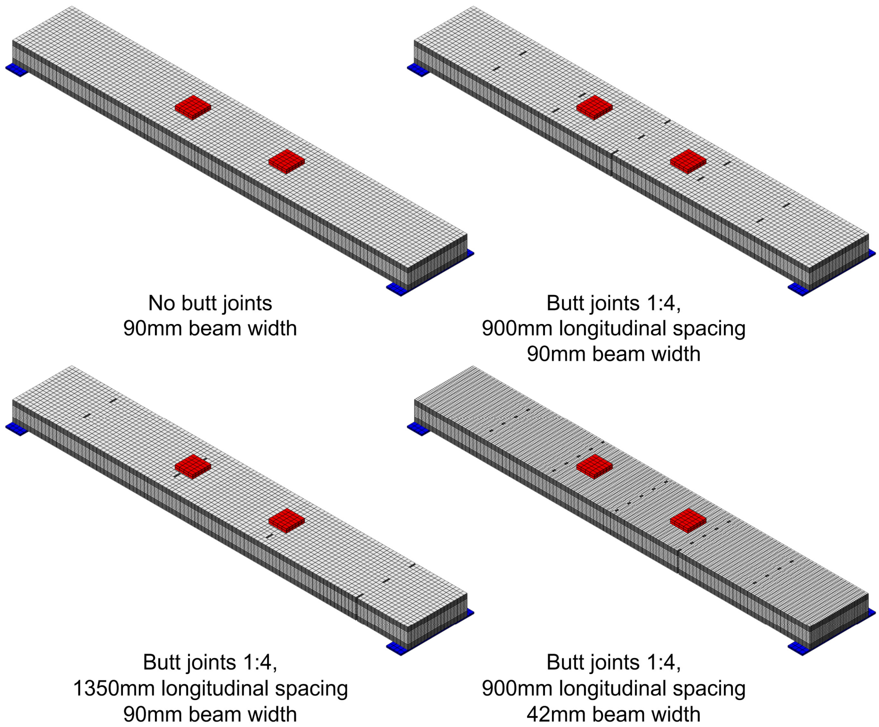


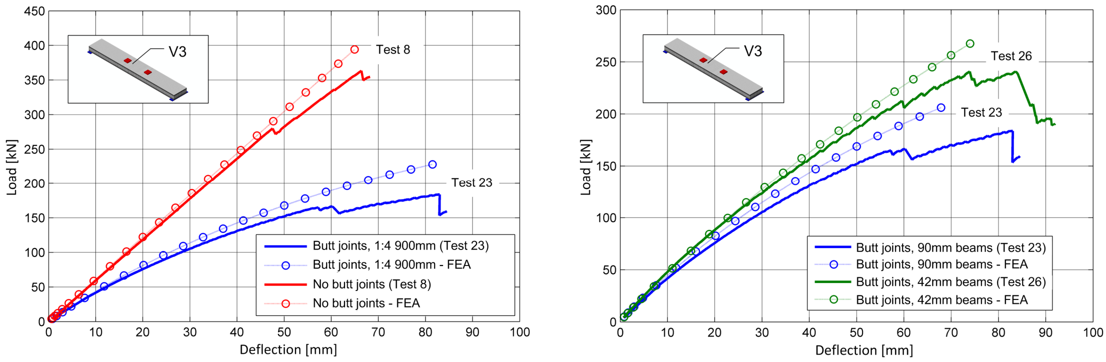
Figure 3.8 Test specimens with different butt-joint configurations.

The aim of the tests was to study the effects of the following variables:

- Span-to-depth ratio
- Single patch load versus two patch loads
- Elements with or without butt joints
- Width of beams
- Longitudinal spacing of butt joints
- Variation in the prestressing level

Little or no interlaminar slip was observed for the test specimens without butt joints, with a prestressing level of 300 kPa or higher. A significant stiffness reduction due to the butt joints could be observed, cf. Figure 3.9 [A]. A reduction in longitudinal stiffness due to butt joints has been reported in several previous studies (Oliva and Dimakis 1988; Oliva et al. 1990; Davalos et al. 1993; Crews 2002). However, the Eurocode (EN 1995-2 2004) does not include a stiffness reduction for SLT decks with butt joints.

Two test configurations with a one-in-four butt-joint frequency and longitudinal spacing of 900 mm were tested. The main difference between the two configurations was the beam width. One test specimen used 42 mm beams, while 90 mm beams were used for the other. The results of the two tests can be seen in Figure 3.9 [B]. The test specimen with 42 mm beams had a smaller reduction in the cross-sectional area (five of 19 beams = 26% reduction) due to butt joints compared with the test specimen with 90 mm beams (three of nine beams = 33% reduction).



[A] [B]

Figure 3.9 *Load-defection charts for butt-joint tests at a prestressing level of 600 kPa. [A] Comparison of specimens with and without butt joints. [B] Comparison of 42 mm and 90 mm beam width for specimens with one-in-four spaced beams butt jointed with a longitudinal joint spacing of 900 mm.*

3.5 Summary and key findings

- The material properties obtained from the tests showed that the transverse stiffness, E_T, and in-plane stiffness, G_{LT}, were dependent on the prestressing level.
- The obtained test values were lower than those reported in the literature. However, the size of the tested specimens had a significant influence on the results.

- There was a significant difference in deck behaviour when the load was positioned in the centre of the deck compared with when it was positioned close to the edge of the deck.
- The deflection values from the non-destructive tests were approximately twice as high for the eccentric load case compared with the concentric load case, which was also observed by Dahl et al. (2006).
- A prestressing level of 300 kPa generated 11-13% higher deflection values compared with a prestressing level of 900 kPa. Oliva et al. (1990) reported a very small change in the measured deflection values when the prestress was reduced from 690 to 345 kPa.
- Horizontal interlaminar slip was observed for tests in the deck with the eccentric load position. This verified the observations made by Ekevad et al.

(2011) but for several different prestressing levels. A larger amount of interlaminar slip was observed at lower prestressing levels. The slip did not increase between the second and third load cycles.

- The beams in the tested SLT deck failed at a load magnitude of 900 kN. Both bending and shear failure could be observed.
- No additional failure was observed when the deck was reloaded with 44% of the failure load after the deck had failed. The observation for SLT decks verified the results reported by Crews (2002), based on SLT box bridges.

- The coefficient of friction (COF) for planed glulam was independent of the prestressing level. The results strengthen the observations made by Kalbitzer (1999).
- The COF for a load parallel to the direction of the wood grain was between 0.29-0.41, with an average of 0.33. The COF for a load perpendicular to the direction of the wood grain was between 0.39-0.44, with an average of 0.42. The observed COF values for a moisture content of 12% were higher than the values recommended in EN 1995-2 (2004).

- Little or no interlaminar slip was observed in the tested specimens without butt joints for a prestressing level of 300 kPa or higher.
- A high span-to-depth ratio (shallower beams) generated less interlaminar slip compared with the test with a lower span-to-depth ratio (deeper beams).
- The stiffness of SLT decks is highly dependent on the number of butt joints, as previously stated by (Oliva et al. 1990) and (Davalos et al. 1993) but not considered in the Eurocode (EN 1995-2 2004).
- Much of the interlaminar slip occurred at fairly large deformation. Little interlaminar slip could be observed at deflection values equal to the span divided by 400.

4 Linear-elastic calculations

Oliva and Dimakis (1988) were two of the first people to perform the numerical modelling of stress-laminated-timber (SLT) decks. They used linear-elastic finite element (FE) analysis to predict the behaviour of an SLT deck with butt joints which was tested in a laboratory. Further studies by Oliva et al. (1990) verified the model and suggested updated material values for the orthotropic behaviour of SLT decks.

Ritter (1990) published a simplified method for designing SLT decks, using the assumption that the deck portion within the width of a vehicle lane was replaced by a equivalent beam in the direction of the traffic. The width of the equivalent beams was determined using equations which were established using the orthotropic FE model established by Oliva and Dimakis (1988). The transverse bending moment and shear forces of the orthotropic deck that normally require a plate analysis could be replaced by pre-calculated charts for the various parameters of deck geometry and vehicle loading.

Marklund (1997b) investigated the opportunity to calculate load distribution in the transverse direction of an SLT deck by modelling the cross-section of an SLT deck as a beam supported on an elastic foundation. The stiffness of the foundation was derived from the longitudinal stiffness of the deck.

A further development of the simplified analytical method using the equivalent beam assumption developed by Batchelor et al. (1981) was presented by Crews (2006). He described the orthotropic FE simulations as a research tool instead of a routine design aid. The design of SLT decks and SLT box bridges in Australia has primarily been performed using a simplified analysis with a few equations.

Dahl (2002) and subsequently (Dahl et al. 2006) performed linear FE plate analysis on full-scale and mid-scale tests performed on an SLT deck. Using an optimisation routine, material parameters could be calculated to fit the measured deflections. They concluded that a linear model was insufficient fully to describe deck behaviour, as delamination and slip were observed during the tests.

The limitations and applicability of using linear FE analysis was studied when linear-elastic FE analysis was used to analyse the full-scale test (***Paper II***). A comparison between simplified methods and linear-elastic FE analysis for SLT decks designed according to AASTHO and Eurocode was made in ***Paper V***.

4.1 Plate theory

There are several theories that are widely used and have been adopted for plate theory. The Kirchhoff plate theory is basically an expansion of the Euler-Bernoulli beam theory into three dimensions. The theory is based on the assumption that a straight line, normal to the surface in the centre of a plate, remains straight and perpendicular to the plate after deformation. The thickness of the plate also remains unchanged during deformation. Kirchhoff's plate theory should only be used for thin plates. A thin plate has a thickness of approximately 10% or less of the length of the plate. Kirchhoff's theory for thin plates is generally only accurate for small deflections. Deformations up to the size of one tenth of the thickness of the plate can be regarded as small. Membrane effects become too prominent for larger deformations (Timoshenko 1940).

When the thickness is between 10-20% of the length of the plate, the Mindlin-Reissner theory is normally used. Both Mindlin and Reissner developed theories which included shear deformations in the bending of plates. Kirchhoff's plate theory proved to be inaccurate for moderately thick plates. The main difference between the Kirchhoff theory and the Mindlin-Reissner theory is that the line normal to the centre surface of the plate remains straight but not necessarily perpendicular to the deformed surface. With this change, shear deformations could be included in plate analysis.

There are only a handful of exact solutions to plate problems. Plates with geometries, boundary conditions and load applications without an exact solution had to be solved using other techniques to approximate the results. One very powerful technique was to use Navier's approach of using double trigonometric series for solving rectangular plates subjected to bending (Timoshenko 1940). The deflection is expressed as a double sine series (also known as Fourier series). The load is also expanded into a double sine series. This technique provides rapid convergence for deflection calculations of plates with a distributed load. Only a few terms in the series have to be included for a good approximation. However, the solutions converge slowly for concentrated loads. As a result, new and improved methods had to be developed.

Most modern plate analyses are performed using the finite element method (FEM). However, alternatives, such as the finite difference method (FDM), grid and framework methods and finite strip method (FSM), can also be used. Szilard (2004) provides a good overview of analytical and numerical plate analyses.

4.2 Simplified hand-calculation analysis

A simplified hand-calculation method for the analysis of an SLT deck without using a computer was published by Ritter (1990). AASHTO (1991) published a guide specification in which a simplified design method was presented. The transverse bending moment and shear force could be calculated using a few equations. The equations were derived from the work by Batchelor et al. (1981) and Oliva and Dimakis (1988), among others. A further simplified equation for calculating the equivalent width distribution was established in Australia (Crews 2001; Crews 2002). EN 1995-2 (2004) also includes a simplified method for decks using the assumption of an equivalent beam. Further information about simplified hand-calculation methods can be found in ***Paper II***. The most recent version of the AASHTO US bridge design codealso includes a simplified design method AASHTO (2012). More information and a comparison with the simplified hand-calculation method can be found in ***Paper V***.

4.3 Linear-elastic analysis of the full-scale test

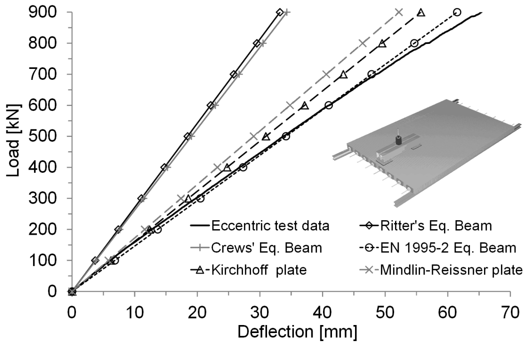
Linear-elastic FE analyses have been conducted on the full-scale deck presented in ***Paper I***. Two different load positions were used in the test, which produced very different deck behaviour. A full comparison of the application of linear-elastic FEM and simplified hand-calculation models can be found in ***Paper II***.

The FE analyses were made using ABAQUS 6.9-2 FE software (Abaqus 2009). Triangular shell elements with reduced integration were used to simulate the linear-elastic plate behaviour without the influence of shear deformations. This element is generally not recommended for this type of plate analysis (Bezine 2002). Rectangular shell elements with reduced integration were used to simulate the behaviour with the shear included. The element size was approximately 50 mm in all the numerical analyses.

The material properties assumed for the linear-elastic FE analysis were taken from EN 1995-2 (2004) and are shown in Table 3.1. The longitudinal modulus of elasticity (MOE) in the model was the same as the average of the measured value for the deck beams.

The tested concentric load position was used for both non-destructive tests (load up to 300 kN) and the ultimate load test (900 kN). Figure 4.1 [A] shows the results of the ultimate load test compared with three hand-calculation models, a Kirchhoff plate analysis and a Mindlin-Reissner plate analysis. The calculated deflection values for a load of 900 kN using the two numerical analyses were 55.7 and 52.4 mm. The measured deflection immediately before failure occurred in the deck was 65.4 mm. The linear-elastic plate models were unable to predict the increased deflection due to interlaminar slip between the beams. Two of the hand-calculation methods (Ritter (1990) and Crews (2002)) severely underestimate the deflection values. This is due to the fact that the vehicle load in these two design methods should not be positioned that close to the edge.

Both numerical models and two of the simplified design methods (Ritter (1990) and Crews (2002)) produced good predictions of the deflection value for the 300 kN load, as shown in Figure 4.1 [B]. The simplified design method in EN 1995-2 (2004) very significantly overestimates the deflection values.



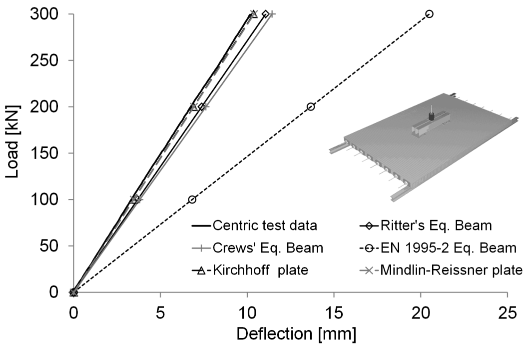


Figure 4.1 Load-deflection charts for the 5.4 x 8.0 m plate used in the full-scale test.
[A] Ultimate load test (ULT) with an eccentric load position
[B] Non-destructive test (NDT) with a concentric load position

Oliva et al. (1990) showed that an FE analysis assuming orthotropic plate behaviour produced fairly accurate results for the deflection of SLT decks. The tested deck was 2.64 m wide and had a span of 7.16 m. The small width of the deck influenced the transverse behaviour of the deck. The load in their tests was positioned at a distance of approximately 0.9 m from the deck edge.

The study by Dahl et al. (Dahl 2002; Dahl et al. 2006) showed that the maximum deflection values for a concentric load was 13.4 mm, while the eccentric load resulted in 27.1 mm. This relationship between the load and the deflection values is similar to the observations made for the full-scale test presented in Chapter 3.2. The study concluded that the simplified design method in EN 1995-2 (2004) was very conservative compared with Ritter (1990).

4.4 Linear FEM for design of SLT decks

Linear FEM for calculating the load effects in SLT decks is an alternative in some design codes, such as the Eurocode (EN 1995-2 2004) and AASTHO (AASHTO 2012).

Over the decades, a great deal of research has been devoted to the development of accurate material properties for the orthotropic plate behaviour of SLT decks, as presented in Chapter 3.1. Existing studies have compared measured deflection values with the results of numerical analyses. The material parameter which provided the fewest discrepancies in the deflection values could then be calculated.

A numerical study of the influence of the material parameters used for analyses of SLT decks was conducted. All the analysed materials used the same longitudinal MOE, E_L = 12000 MPa (average value of the measured deck beams). It has been shown that the longitudinal MOE has the greatest influence on the behaviour of SLT decks (Oliva and Dimakis 1988; Davalos et al. 1993; Dahl et al. 2006). The purpose of the study was to investigate how variations in the transverse MOE E_T and the in-plane shear modulus G_{LT} influence the following values:

1. Bending moments, M_L and M_T
2. Twisting moment, M_{LT}
3. Shear forces, V_L and V_T
4. Deflection value, Δ_{max}

The four different materials were compared using the same FE model consisting of three-dimensional shell elements. The FE model was the same one that was used to analyse the full-scale ultimate load test. The eccentric load position was used, together with a load level of 900 kN.

Material 1 used the recommended values for E_T and G_{LT} stated in EN 1995-2 (2004), which is the design code used for designing SLT decks in Sweden. Material 2 used the values suggested by Ritter (1990). Material 3 was based on the test results obtained from the material properties tested by the author and presented in Chapter 3.1. The values for the lowest prestressing level were used. Material 4 used the values for the OHBDC (1983), which was the first design code to include recommendations for designing SLT decks.

Table 4.1 Input values for the numerical FE analyses used to study the influence of material parameters.

Material parameter	**Material 1 EN 1995-2 (2004)**	**Material 2 Ritter (1990)**	**Material 3 Karlsson et al. (2009)**	**Material 4 OHBDC (1983)**
E_L [MPa]	12 000	12 000	12 000	12 000
E_T [MPa]	240 (0.02 E_L)	156 (0.013 E_L)	72 (0.006 E_L)	600 (0.05 E_L)
G_{LT} [MPa]	720 (0.06 E_L)	360 (0.03 E_L)	84 (0.007 E_L)	780 (0.065 E_L)

The results of the study showed that the longitudinal bending moment, M_L, increased for materials 2 and 3 but not for material 4, when compared with material 1, as shown in Table 4.2. Both materials 2 and 3 had lower values for E_T and G_{LT} compared with material 1, while material 4 had slightly increased values for E_T and G_{LT}.

Material 3 showed a significant reduction in the twisting moment, M_{LT}, compared with material 1. Material 3 had a significantly lower value for G_{LT} (84 MPa compared with 720 MPa). The maximum deflection value was also 86% higher for material 3 compared with material 1. However, the material parameters used for material 3 are not realistic for analyses of full-scale SLT decks, as the material test was significantly influenced by the size of the test specimens.

Table 4.2 Results of the numerical analysis of the influence of material parameters. Values within brackets compared the difference in per cent compared with material 1.

Load effect and deflection	Material 1 EN 1995-2 (2004)	Material 2 Ritter (1990)	Material 3 Karlsson et al. (2009)	Material 4 OHBDC (1983)
M_L [kNm/m]	466.3	510.4 (109%)	588.9 (126%)	457.0 (98%)
M_T [kNm/m]	15.7	16.2 (103%)	16.9 (108%)	21.3 (136%)
M_{LT} [kNm/m]	24.4	18.1 (74%)	8.1 (33%)	21.2 (87%)
V_L [kN/m]	458.7	426.1 (93%)	367.8 (80%)	457.2 (100%)
V_T [kN/m]	116.3	106.6 (92%)	94.1 (81%)	124.8 (107%)
Δ_{max} [mm]	52.4	62.0 (118%)	97.5 (186%)	51.0 (97%)

A second study was conducted with the aim of comparing the design of SLT decks constructed using glulam beams in Sweden and in the USA (***Paper V***). One part of the study compared the load effects from AASHTO (2012) and the Eurocode (EN 1995-2 2004). The study focused on comparing the design requirements from the codes, nationally unique design solutions and available materials. Both design codes make it possible to use a simplified design method or to use a more complex design method based on orthotropic plate analyses.

Wacker and Groenier (2010) conducted a comparison of the design codes used in Canada, the USA and Europe. They found that the bending moment from vehicle loads for a glulam stringer bridge was 1,900 kNm for load model 1 in EN 1991-2 (2003) compared with 600 kNm from load configuration HL 93 in AASHTO (2010).

The study verified that the vehicle loads in the Eurocode (EN 1991-2 2003) were significantly larger than those for the design vehicles in AASHTO (2012). However, the vehicle load effects for SLT decks were smaller compared with glulam stringer bridges. The required depth of the SLT deck differed significantly when compared for a two-lane bridge with a 10 m span. The required deck depth when designing according to AASTHO was 18 inches (457 mm). The required deck depth for design according to the Eurocode was 630 mm (38% larger than the AASTHO deck). The FE analysis for designs according to AASHTO used material parameters for the orthotropic plate behaviour specified in Ritter (1990). The Eurocode FE analysis used the material parameters recommended in EN 1995-2 (2004). The vehicle loads were positioned as close to the edge of the deck as required in the codes. The AASTHO vehicle was positioned 3 ft. (0.91 m) from the edge, while load model 1 in (EN 1991-2 2003) was positioned as close as 0.5 m from the deck edge. The load position on the deck had a significant effect on the results of an orthotropic plate analysis.

All the load effects were significantly larger for the Eurocode analyses compared with the AASHTO analyses. The deflection criteria governed the design of both the Eurocode (span/400) and the AASHTO (span/425) decks. The simplified design method in AASHTO (2012) underestimates the load effects, as well as the maximum deflection in the deck, when compared with the FE analysis. The simplified design analysis in EN 1995-2 (2004) resulted in a larger bending moment but lower longitudinal shear force and support compression compared with the FE analysis. The maximum deflection values for the Eurocode deck were the same using the simplified and the FE analyses.

Table 4.3 Combined load effects and deflection for both simplified and FE analysis using AASHTO and Eurocode.

	AASHTO		**Eurocode**	
Load effect and deflection	**Simplified**	**FE**	**Simplified**	**FE**
Longitudinal bending moment, M_L [kNm/m]	442.8	468.3	961.8	753.8
Transverse bending moment, M_T [kNm/m]	-	7.87	-	11.5
Longitudinal shear force, V_{LZ} [kN/m]	187.2	228.0	203.6	308.5
Transverse shear force, V_{TZ} [kN/m]	-	27.0	-	44.7
Support compression, F_Z [kN/m]	195.2	256.1	224.1	378.8
Deflection, u_Z [mm]	21.8	25.1	21.0	21.0

4.5 Summary and key findings

- All the linear-elastic calculations underestimate the deflection values compared with the measured deflection from the full-scale ultimate load test with an eccentric load position.
- The two linear-elastic FE analyses resulted in good agreement compared with the measured deflection for non-destructive tests (300 kN load).
- The simplified design methods developed by Ritter (1990) and Crews (2002) were not accurate for the analysis of SLT decks in the ultimate limit state with the load positioned close to the edge of the deck.
- The simplified design method in EN 1995-2 (2004) provided an acceptable deflection estimation for the eccentric load position for the tested full-scale deck. However, for the concentric load position, the simplified method severely overestimates the deflection values.

- The required deck depth for SLT decks in Sweden designed according to the Eurocode (EN 1991-2 2003; EN 1995-1-1 2004; EN 1995-2 2004) was approximately 38% greater than for decks designed in the USA according to AASHTO (2012).

- The simplified analysis in AASHTO (2012) underestimated the load effects and the deflection compared with the FE analysis. The simplified analysis in EN 1995-2 (2004) resulted in the same deflection value as the FE analysis.

5 Modelling interlaminar slip in SLT decks

Computer simulations of wood have been used in the research community for several decades. Mackerle (2005) provides an extensive bibliographic review of finite element analyses applied to wood. More than 300 papers and conference proceedings are included on topics such as material and mechanical properties, wood joining and fastening, fracture mechanics, drying processes and distortion, lumber, glulam and panels, trusses and frames, floors, roofs and bridges.

A great deal of work has been conducted on modelling the global deflection behaviour of SLT decks. As presented in Chapter 4, several researchers have used linear-elastic orthotropic plate analyses to simulate the behaviour of SLT decks. However, very little research has focused on simulating the interlaminar slip between individual beams in SLT decks. A non-linear computer program was developed in Canada to simulate the slip between transverse post-tensioned T-beam sections made of glulam (Shen 1992). The program modelled the frictional slip between glulam panels (the bridge deck) by introducing coupled elastic springs. The stiffness of the springs was dependent on the amount of post-tensioning. A model with springs can be effective when the slip interfaces are few in number and well defined. An SLT deck has significantly more slip interfaces, which makes this type of model difficult to use.

In his PhD thesis, Crews (2002) concluded that it was very difficult to model the frictional interface between beams due to variations in several key parameters. The anisotropic friction behaviour of wood, together with the large variations in normal stress between the beams, makes it almost impossible to predict where slip will initiate. When butt joints are added to the deck, the complexity of the problem becomes even greater.

Two different approaches to modelling the interlaminar slip in SLT decks were used in this study. The first approach was based on contact friction slip between the beams in the deck. The model is presented in ***Paper III***, ***Paper IV*** and in a paper by Ekholm et al. (2012). The second approach was to use an elastic-plastic material model to model the slip as plasticity in the material. The application of the elastic-plastic model for SLT decks is presented in ***Paper III*** and in a paper by Ekevad et al. (2011). The origin of the model can be found in Ekevad (2006).

5.1 Contact FEM

When the non-linear model for simulating interlaminar slip in SLT decks was developed, the industry was interested in examining the opportunity to use a widely available commercial FEM package. The most prominent non-linear behaviour in SLT decks is the interlaminar slip between the beams. The approach involved modelling each individual beam in an SLT deck using solid elements with linear-elastic material properties. A friction interface between the solid beams could then be formulated.

There are several commercial FE packages that could be used for this approach and the final choice fell on Abaqus, because of the built-in contact formulations.

The existing literature provided a foundation for the linear-elastic material properties of wood.

Anisotropic material properties were initially taken from Pousette (2001), which summarised several existing studies. Material properties for the numerical modelling of spruce were subsequently taken from Mirianon et al. (2008). A small sensitivity analysis was conducted on the way different material properties influenced the global deflection behaviour (Hellgren and Lundberg 2011). The results showed that Poisson's ratio had no influence on the global deflection behaviour of the deck. Another finding was that a variation in the coefficient of friction (COF) values had little effect. The non-linear analyses in ***Paper III*** and ***Paper IV*** used the material properties presented in Table 5.1.

Table 5.1 Anisotropic linear-elastic material properties of glulam. The co-ordinate system is defined with axis 1 in the transverse direction of the deck, axis 2 is the depth of the deck and axis 3 travels along the length of the beams.

Modulus of elasticity	**Shear modulus**	**Poisson's ratio**	**COF**
$E_1 = 0.02 \times E_3 = 240$ MPa	$G_{12} = 0.1 \times G_{23} = 72$ MPa	$v_{12} = 0$	$\mu_2 = 0.34$
$E_2 = 0.02 \times E_3 = 240$ MPa	$G_{13} = 0.06 \times E_3 = 720$ MPa	$v_{13} = 0$	$\mu_3 = 0.29$
$E_3 = 12\,000$ MPa	$G_{23} = 0.06 \times E_3 = 720$ MPa	$v_{23} = 0$	

The friction interface between the beams in an SLT deck would have to fulfil the following criteria, based on the observations made in the full-scale ultimate load test and the first coefficient of friction test.

1. Generate a pressure-dependent critical shear stress
2. Allow anisotropic friction
3. Allow the two beams to separate

All the interactions in the modelled deck could be simulated using the built-in interaction properties in Abaqus. However, simulating interaction with frictional behaviour for multiple surfaces is very computationally demanding. The above-mentioned criteria were fulfilled when using interaction behaviours, which in Abaqus are called a "penalty" friction model for the in-plane behaviour and a "hard" surface model in the normal direction of the surface (Ekholm et al. 2012).

The "penalty" friction is based on a slightly adjusted, idealised Coulomb friction model. A small "elastic slip" is added to the Coulomb friction, cf. Figure 5.1 [A]. True idealised Coulomb friction could be obtained if the elastic slip was set at zero. However, this would cause severe convergence problems and significantly increase the computational effort (Abaqus 2011). The size of the elastic slip used in the analyses was set at a small value (10^{-6} m) in order to avoid too much influence from the elastic slip.

No slip from friction occurred between the beams as long as the shear stresses were lower than the critical shear stress limits, cf. Figure 5.1 [B]. The critical shear vertical stress limit, $\tau_{12,crit}$, depended on both the COF perpendicular to the grain ($\mu_2 = \mu_\perp$) and the magnitude of the normal stress (σ_1) between the beams. The same criterion was used for the critical horizontal shear stress limit, except that the COF parallel to the grain ($\mu_3 = \mu_{||}$) was used instead (***Paper III***, ***Paper IV***). Slip occurs when a combination of the shear stresses is larger than 1, as shown in Equation 5.1.

The modelled interaction between the deck beams was able to transfer compressive normal stresses but not tensile stresses. As a result, the beams were free to separate from one another when the compressive normal stress became zero (***Paper III***, ***Paper IV***). The use of quadratic 20-node elements to analyse contact problems is strongly discouraged due to numerical instability (Abaqus 2011). Eight-node elements were used in the simulations.

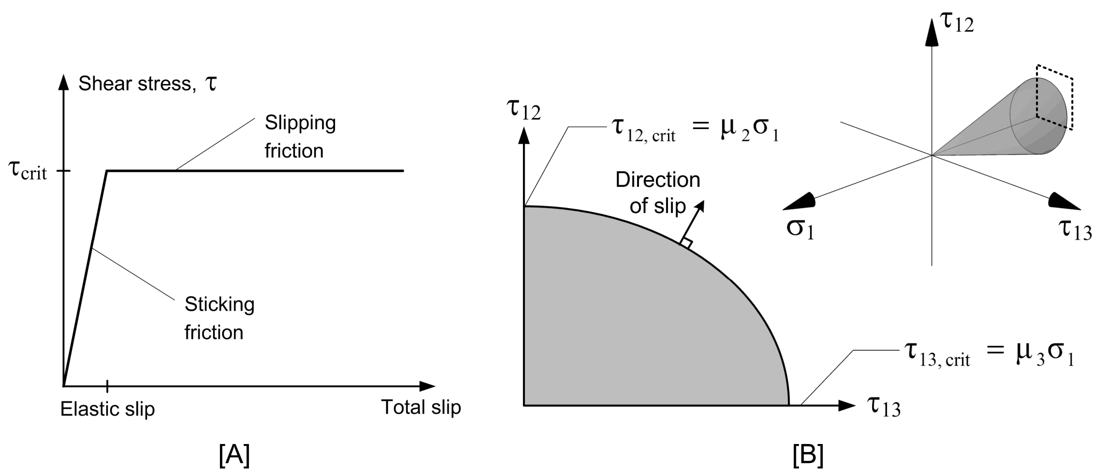


Figure 5.1 [A] Coulomb friction model with added elastic slip. Elastic slip will occur even when the shear stress, τ, is smaller than the critical shear stress, τ_{crit}.
[B] Illustration of the critical anisotropic shear stresses. The critical shear stress limit depends on both the coefficient of friction and the normal stress between the beams.

The prestressing was applied to the model either as uniform pressure along the entire length of the edge beam (***Paper III***) or as discrete patch loads simulating the pressure under the anchorage plates (***Paper IV***).

5.2 Elastic-plastic FEM

During the development of the contact FE model, the desire to compare the model with other non-linear FE models led to collaboration and the use of an elastic-plastic FE model. The elastic-plastic FE model originated from a wood-drying project in which the plasticity of the material was important (Ekevad 2006). The entire SLT deck was modelled as a single solid part in contrast to the contact FE model which modelled each beam as an individual part. The use of a single part simplified the meshing of the deck.

The elastic-plastic model used a linear-elastic and ideal plastic behaviour for the shear stresses, τ_{12} and τ_{13}. The yield surface criteria that were used are shown in Equation 5.***1***. The model was further developed in order to make it possible to simulate gaps between the beams. Gaps could be simulated in the continuous deck by locally reducing the values for E_1, G_{12} and G_{13} to almost zero when the normal stress (σ_1) became greater than 0 (tensile normal stress). The linear-elastic material properties used in the simulation were the same as for the contact FEM, cf. Table 5.1 (***Paper III***).

Equation 5.1 Yield surface criteria used for both the elastic-plastic FEM and the contact FEM. Yielding (interlaminar slip) occurs when f ≥ 1.

$$f = \frac{\tau_{12}^2}{\tau_{12,crit}^2} + \frac{\tau_{13}^2}{\tau_{13,crit}^2} = \frac{\tau_{12}^2}{(\mu_2\sigma_1)^2} + \frac{\tau_{13}^2}{(\mu_3\sigma_1)^2}$$

The elastic-plastic FE model required significantly fewer computer resources (***Paper III***). However, the greatest disadvantage of the elastic-plastic FE model was the lack of opportunity to model decks with discontinuities such as butt joints.

5.3 Application of the models to the full-scale SLT-deck test

A verification of the contact FEM (CFEM) and elastic-plastic FEM (EPFEM) models was made against the full-scale SLT-deck test in ***Paper III***. The FE models used the material properties shown in Table 5.1, as the tested deck had an average longitudinal MOE of 12,000 MPa. The tested deck is further presented in Chapter 3.2.

The 5.4 x 8.0 x 0.27 m (length, width, depth) deck was modelled as simply supported on 8.0 m long transverse steel beams spaced 5.0 m apart, cf. Figure 3.3. A contact surface was defined between the steel beam and the SLT deck. The deck was able to move longitudinally and transversally over the support area but was restrained by the friction between the beam and the deck.

The load was applied as two patch loads with a size and position exactly the same as those used in the test, cf. Figure 3.3. The load sequence in the FE models deviated from the test load sequence, as shown in Figure 5.2. A uniformly distributed gravity load was applied to the entire deck, equal to 5 kN/m^3. The prestressing levels in the model were the same as those used in the test (900, 600 and 300 kPa) and were applied as uniformly distributed pressure along the edge beams of the deck.

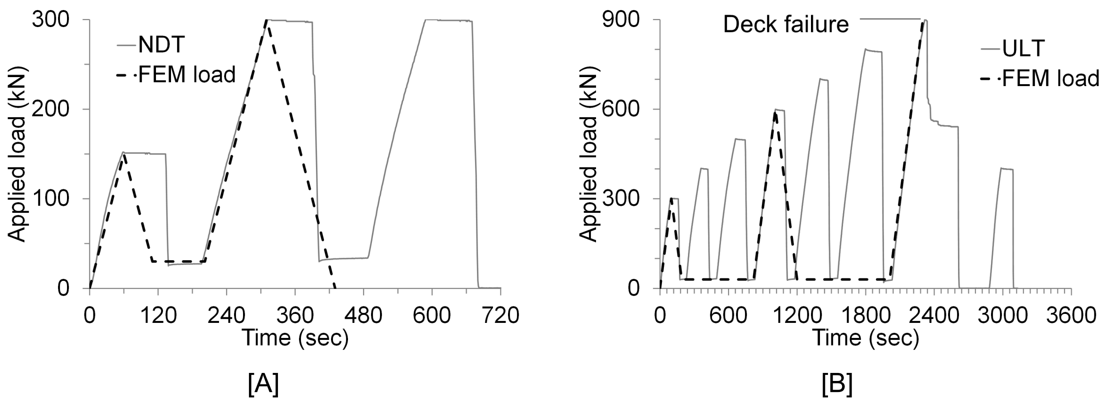
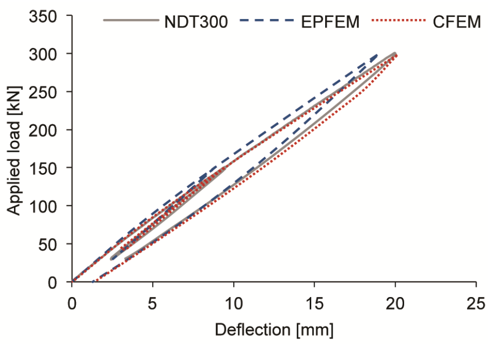


Figure 5.2 Comparison of the load application used in the full-scale SLT-deck test and the FE models. The FE models were not time sensitive, as no visco-elastic behaviour was modelled.
[A] Load sequence for the non-destructive test (NDT)
[B] Load sequence for the ultimate load test (ULT)

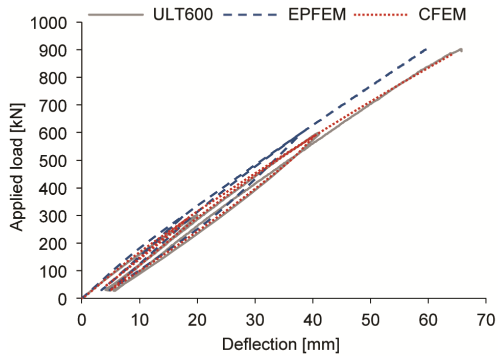
Both the EPFEM and the CFEM were able to simulate the interlaminar slip effectively in the non-destructive tests. The amount of irreversible interlaminar slip after unloading was the same (approximately 1.5 mm) as that seen after the load is removed in Figure 5.3 [A]. Both models simulated effectively the hysteresis of the loading and unloading phases.

The EPFEM was globally stiffer compared with the CFEM and the measured deflection. For the ultimate-load test (ULT), the higher stiffness in the EPFEM became even more distinct, cf. Figure 5.3 [B]. The CFEM was able to simulate the deflection values effectively.

The magnitude of the horizontal slip values during the ultimate-load test was much smaller than the magnitude of the vertical slip values, cf. Table 5.2. Both models predicted a higher maximum value for the horizontal slip compared with the measured value from the test. However, the measured value was taken at a slightly different position in the deck. The vertical slip was not measured during the loading. The EPFEM calculated a higher value for the maximum vertical slip compared with the CFEM. The CFEM was able to predict the size of the maximum transverse gap effectively.



[A]



[B]

Figure 5.3 Load-defection curves for the full-scale SLT-deck test (NDT and ULT), EPFEM and CFEM
[A]NDT with eccentric load position and a prestressing level of 300 kPa (NDT300 or NDT4 in Table 3.2)
[B] ULT with eccentric load position and a prestressing level of 600 kPa (ULT600 or ULT1 in Table 3.2)

Table 5.2 Calculated maximum deformation of the simply supported SLT deck from EPFEM and CFEM for the ULT.

Deformation [mm]	**EPFEM**	**CFEM**	**Measured values**
Max. horizontal slip	0.46	0.56	0.31
Max. vertical slip	3.40	2.71	
Max. gap	0.46	1.50	~1.4

No stresses were measured in the deck during the tests. The two FE models were therefore able to provide valuable knowledge about the stress state in the deck. The maximum and minimum stress values calculated using the EPFEM and the CFEM are presented in Table 5.3 for a load of 900 kN.

The calculated longitudinal bending stresses were 6-8% larger for the CFEM compared with the EPFEM. The difference between the two models was due to the fact that the maximum deflection calculated by the CFEM was about 12% greater than that calculated by the EPFEM.

The CFEM calculated a higher compressive bending stress in the transverse direction. The increase in normal stress between the beams influenced the shear stresses, as the critical shear stress limits were based on the normal stress. The normal stress (σ_1, S_{11}) increased significantly from -0.60 MPa (which was the applied prestressing level) to approximately -2.59 MPa. The variation in the normal stress in the deck (cf. Figure 5.4) highlights the complexity of calculating interlaminar slip in SLT decks. The gaps between the beams simulated by the CFEM are shown in Figure 5.4.

Table 5.3 Calculated maximum and minimum stresses in the mid-span of the simply supported SLT deck from EPFEM and CFEM. The total load was F=900 kN.

Stresses [MPa]	**EPFEM**		**CFEM**	
	Max.	**Min.**	**Max.**	**Min.**
Bending stress S_{33}	40.26	-42.66	43.45	-45.31
Bending stress S_{11}	0.36	-2.24	0.00	-2.59
Shear stress S_{12}	0.39	-0.32	0.37	-0.44
Shear stress S_{13}	0.45	-0.50	0.79	-0.81

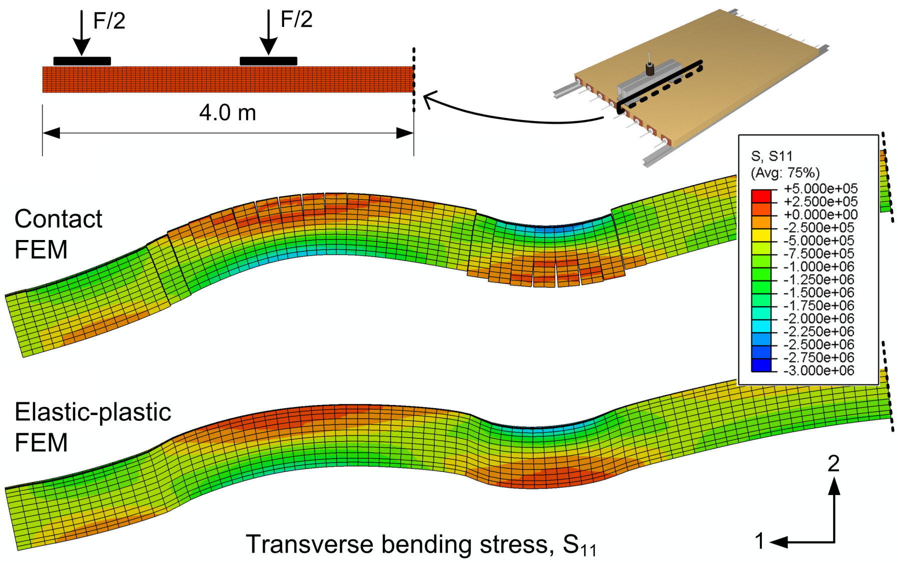


Figure 5.4 Comparison of the transverse bending stresses for the CFEM and the EPFEM at a load level of 900 kN. Cross-section taken at mid-span of a simply supported SLT deck.

5.4 Application of the CFEM on the butt joint and interlaminar slip test

Simulations of the butt joint and interlaminar slip tests were made using the contact FE model (CFEM). Comparisons of the calculated deflection values were made with the measured deflection values. The comparison is presented in ***Paper IV***. The butt joint and interlaminar slip test is presented in Chapter 3.4.

A common assumption that has been used in the design of SLT decks is that the prestressing level is uniformly distributed at a certain distance away from the edge. However, the distance at which a uniform distribution can be obtained has remained unknown. The application of the prestressing in the CFEM was changed to discrete patch loading compared with the uniformly distributed prestressing used for the simulations of the full-scale test. The prestressing level along the length of the edge beam in an SLT deck varies substantially, with high local peaks under the anchorage plates, as shown for beam 1 in Figure 5.5. The prestressing level varies between -400 and -800 kPa (target value was -600 kPa) in the interface between the second and third beams (counted from the edge), as shown for beam 3 in Figure 5.6. Not until the fifth beam can the prestressing level be assumed to be close to uniformly distributed. The prestressing distribution has a significant influence on the interlaminar slip behaviour, as the critical shear stress limits are dependent on the amount of normal stress.

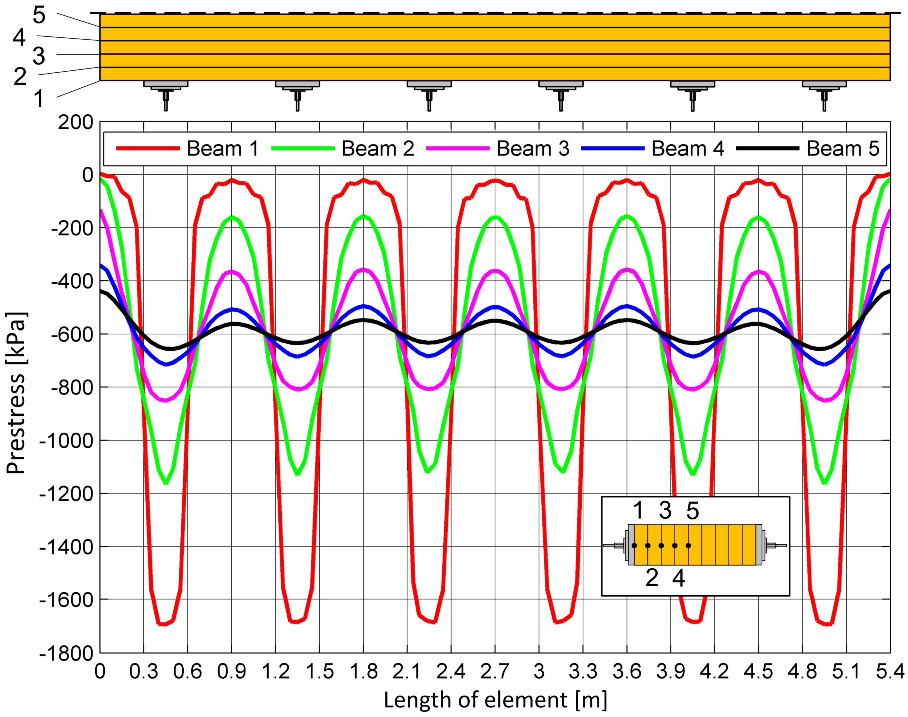
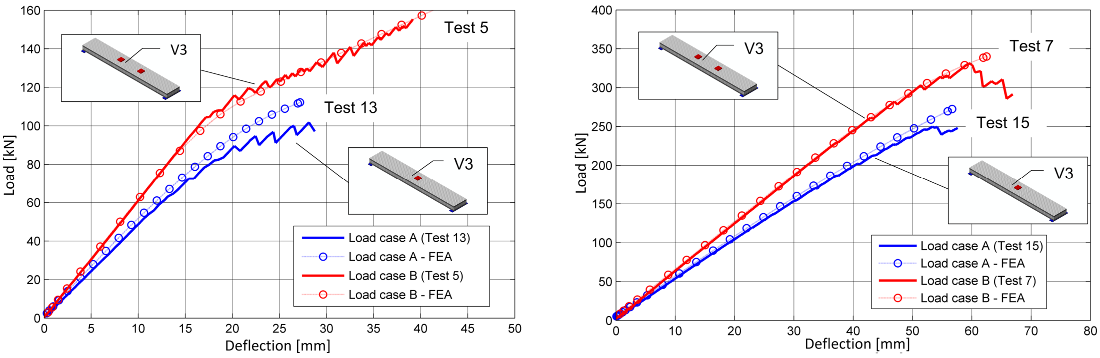


Figure 5.5 Calculated prestressing distribution in the contact FE model. The prestressing anchorage plates are attached to the edge beam of the deck (beam 1).

The linear-elastic material properties used in the simulations are shown in Table 5.1, except that the coefficient of friction values were 0.35 and 0.3 instead of 0.34 and 0.29.

The measured load-deflection behaviour of test elements without butt joints could be successfully simulated using the contact FE model. Both the initial linear-elastic behaviour of the test specimens and the post-slip behaviour showed a good correlation between the simulated values and the measured values, as shown in Figure 5.6. The test with a 100 kPa prestressing level (Figure 5.6 [A]) was non-destructive and was stopped at a predetermined load level. The test with a 300 kPa prestressing level (Figure 5.6 [B]) was loaded until failure occurred in the beams. Not much slip was observed in the test with a prestressing level of 300 kPa or higher. The simulations were performed up to a pre-set deflection value, as no failure criteria for the material were defined in the FE model.



[A] [B]

Figure 5.6 Load-deflection charts for the interlaminar slip tests for test elements without butt joints. A single load position is compared with a dual patch load application.
[A] Test conducted with a target prestressing level of 100 kPa
[B] Test conducted with a target prestressing level of 300 kPa

One of the most unique and important features of the CFEM is the opportunity to simulate the behaviour of the butt joints. SLT decks with butt joints have reduced stiffness due to the cross-sectional reduction. During the tests, observations were made of the local distortion of the butt joints. The model succeeded in predicting the butt joint that started to slip first and at which load level. An example of how a butt joint slipped is shown in Figure 5.7.

The simulation of the tests of elements with butt joints also produced encouraging results. Figure 3.9 [A] shows the simulation of specimens with and without butt joints at a prestressing level of 600 kPa. A similar correlation between the simulations and measured values was observed for the comparison of 42 mm and 90 mm beam widths, Figure 3.9 [B].

Figure 5.7 Butt joint slip at the deck edge, [A] photo from test and [B]FE model

5.5 Summary and key findings

- There is limited information and knowledge in the literature related to the FE modelling of interlaminar slip in SLT decks.
- Interlaminar slip was successfully simulated using numerical FE models with either a friction interface between beams or an elastic-plastic material model.

- Both FE models effectively simulated the load-deflection hysteresis behaviour of the loading and unloading in the full-scale test. Irreversible interlaminar slip was observed in both models, with a magnitude similar to the measured value in the test.
- The elastic-plastic FE model appeared to be slightly weaker for the global deflection behaviour compared with the contact FE model and the measured deflection values.
- The normal stress in the simulated deck increased from -0.6 MPa to -2.59 MPa when loaded.
- The contact FE model predicted larger gaps between the individual beams in the deck compared with the elastic-plastic FE model.

- The prestressing level varies significantly close to the edge of an SLT deck. After approximately five beams from the prestressing anchorage plates, the prestressing level could be assumed to be uniform.
- The contact FE model effectively simulated the amount of interlaminar slip depending on the prestressing level. The observation made from the test that little or no interlaminar slip occurred for decks without butt joints at prestressing levels higher than 300 kPa was verified.
- The stiffness reduction in decks with butt joints was successfully simulated.
- The contact FE model was able to predict the butt joint that would start to slip and at which load level.

6 Durability of SLT decks

The decline in the use of wood has resulted in many modern timber bridges ending up with a shorter life-span expectancy compared with some bridges that were built centuries ago. One possible source of the reduced service life of modern structures may be the loss of engineering skills and expertise in relation to wood as a material. There are some recent guides that are designed to help engineers design timber bridges with good durability (Ritter 1990; Sétra 2006; Pousette 2010). A good life span in a wooden bridge is the result of regular monitoring and maintenance to ensure good health in the bridge.

There are two main durability concerns when it comes to SLT decks. The first is to ensure that the prestressing level is sufficient for the SLT deck to continue to act as a solid timber deck. The second durability concern is to monitor and prevent the degeneration of the material in the deck, especially the wood.

6.1 Prestressing durability

The force in prestressing bars reduces with time, due to several factors that are generally difficult to determine exactly. For this reason, the phenomenon of the long-term performance of the prestressing has been widely studied by a number of researchers.

Early studies, such as the ones conducted by Oliva and Dimakis (1988) and Oliva et al. (1990), showed that the prestressing loss in stress-laminated timber decks was as large as 60%. Their results verified several earlier studies that were undertaken in Canada during the 1980s. The suggested solution to the prestressing loss was to increase the initial prestressing level by a factor of 2.5 and accept the reduction due to long-term effects. The prestressing level could then be restored in the deck by re-stressing the bars whenever the level fell below 40% of the initial prestressing level. Crews (2002) concluded from his studies that maintaining higher prestressing in the deck was favourable in many respects when it came to the performance of the deck. He therefore suggested increasing the initial prestressing level at which the decks should be stressed at assembly. The initial prestressing level should be at least 1,000 kPa (instead of the approximately 700 kPa used in North America), regardless of whether softwood or hardwood species were used. If the initial prestressing is lower than this, the load distributing effect will be reduced. He also suggested that the deck should be stressed at least four times during the first six months. The prestressing level in the deck should then be monitored continuously.

Extensive field studies of SLT decks constructed in the USA between 1988 and 1994 reported observations made on several stress-laminated-timber bridges in service. A number of SLT decks were constructed using softwood or hardwood sawn timber. Some bridges were constructed using timber with a moisture content (MC) well above the fibre saturation point (FSP). During the first couple of years, the wood with a high

MC dried out, which in most cases caused the shrinkage of the wood, with a reduced prestressing level as a result (Ritter et al. 1995; Kainz et al. 1996; Ritter et al. 1996a; Ritter et al. 1996b; Hislop 1998; Dagher et al. 2001). Another observed cause of reduced prestressing level was the stress relaxation of wood loaded in compression perpendicular to the grain. Stress relaxation was more pronounced for wood with a high MC (Wacker and Ritter 1995; Hilbrich Lee et al. 1996; Hilbrich Lee and Ritter 1997; Wacker et al. 1998). One bridge was constructed using very dry wood (around 8% MC) in order to avoid excessive shrinkage from the very dry environment in which the bridge was constructed. However, the lack of a moisture barrier on top of the bridge resulted in the deck being subjected to large seasonal variations in moisture content (around 5% MC) (Hilbrich Lee and Lauderdale 1997). Large MC variations usually result in mechano-sorptive creep in the wood. The swelling of the deck beams could result in an increased prestressing level which, in the worst case, would cause the local crushing of the edge beam in the deck, cf. Figure 6.1. This was of special importance if discrete anchorage plates were used on softwood decks without a hardwood edge beam.

Figure 6.1 Local crushing of the timber perpendicular to the fibre direction of the edge beam. Photo taken from (Pousette and Fjellström 2004).

Field measurements of the prestressing levels in SLT decks made of glulam beams were conducted in Sweden by Pousette and Fjellström (2012). None of the inspected bridges had a prestressing level that was less than approximately 64% (compared with the 40% in North America) of the initial prestressing level. The oldest inspected bridge had been in service for 16 years.

Sarisley and Accorsi (1990) proposed an alternative using spring discs that could reduce the loss of prestressing due to creep in wood. The use of spring discs never became widely used for SLT decks. Dagher and Altimore (2005) suggested a possible solution to the large loss of prestressing force and the need to re-stress the bars using glass-fibre-reinforced polymer (GFRP) tendons instead of high-strength steel bars.

GFRP tendons have approximately one ninth the stiffness of steel. The resulting loss of prestressing force due to a given long-term transverse movement of the deck was therefore significantly less in GFRP tendons than in similar steel bars. A pilot bridge was constructed in the USA using this technique. The results indicated that, even after four and a half years, the prestressing in the bridge was still adequate.

Crocetti and Kliger (2010) presented an alternative solution to further reduce the prestressing loss. The conventional Swedish anchorage system with a hardwood plate and an aluminium disc could be replaced by a system consisting of a steel plate and fully threaded screws, cf. Figure 6.2. The 300 mm fully threaded screws with a diameter of 12 mm were inserted into the deck, through a number of beams from the edge of the deck, perpendicular to the grain. A 20 mm stiff steel plate was placed at the edge of the deck, on top of the heads of four screws. The system increased the ultimate capacity of compression perpendicular to the grain by up to 85%. An anchorage system of this kind could also significantly reduce the short- and long-term prestressing loss. Accelerated long-term tests in a climate chamber with varying humidity show that an anchorage system of this kind has a great advantage over the conventional anchorage system when it comes to long-term performance. Field measurements of bridges in Sweden constructed using the alternative prestressing anchorage have shown promising results. Some bridges were built using both the conventional and the alternative prestressing system for easy comparison, while some bridges were constructed using only the system with threaded screws. The measured prestressing levels in three SLT decks that were four to five years old showed that the decks had 70-76% of the initial prestressing left (Pousette and Fjellström 2012).

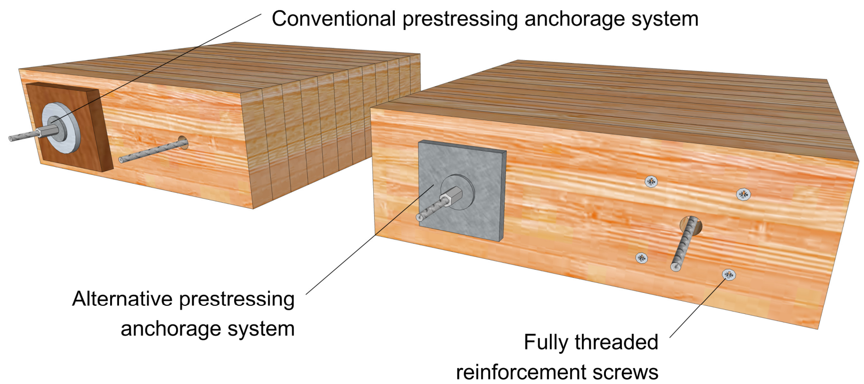


Figure 6.2 Conventional and alternative prestressing anchorage systems used in Sweden.

6.2 Degradation of wood in bridge applications

Dry wood has the ability to withstand natural degradation very well, as proven by timber structures built centuries ago. It is very important to ensure the durability of wood structures, like any other structures built from other materials. There are several causes of deterioration in wood(Sétra 2006). They include:

- Wood decay
- Mould
- Insects
- Marine borers
- Temperature
- Chemicals

Wood decay is the result of fungi growing in microscopic spores inside the wood. The fungi need access to nutrients (wood), air, moisture and favourable temperature. If any of the four is removed, the growth stops. Wood decay has a serious negative effect on the strength of a structure. If the moisture content is kept below approximately 20%, no fungal growth takes place. Water should be diverted away from the structure with proper detailing. The reported degradation of wood in existing SLT bridges in Sweden is mainly caused by the incorrect detailing of water diversion. The amount of detailing has evolved and improved over the years (Pousette and Fjellström 2004).

The purpose of treatment is to create a toxic environment in which no biological growth can occur. It is crucial to obtain sufficient penetration of the treatment and ensure that the treatment stays inside the wood. There are basically two main categories of treatment, oil-type and waterborne preservatives. The two most common oil-type preservatives are creosote and penta (pentachlorophenol). Most waterborne preservatives have some type of arsenic compound in a water solution. Some examples that have been used in the past are chromated copper arsenate (CCA), acid copper arsenate (ACA) and ammoniac copper zinc arsenate (ACZA). The treatment is applied using either non-pressure methods or pressure methods. Various species can be more or less complicated when it comes to applying treatment. A waterborne preservative is preferable if the wood is going to be painted. The use of wood treatments in Sweden is strictly regulated. However, the use of treated wood is a requirement for timber bridges in the USA. The effects on both humans and the environment of using chemicals as a wood treatment are very uncertain.

Mould or fungal stain may appear on the surface of wood structures. Mould will not affect the structural strength and can therefore be accepted in limited amounts. Keeping wood dry is usually sufficient to prevent mould or fungal stains. Termites have to be considered in several parts of the world (not in Sweden, however). Termites can be either subterranean or above ground. The best way to prevent subterranean termites is to limit the amount of wood in contact with the ground. No strength reduction takes place in wood from temperature, as long as the temperature is kept below 50ºC. Wood has superior qualities when it comes to handling chemicals compared with many other materials. Wood is not affected negatively by road salt in contrast to steel and reinforced concrete.

A timber bridge inspection report found that no serious discrepancies were found in 13 inspected timber bridges that were constructed between 1994 and 2001 (Pousette and Fjellström 2004). Several bridges showed signs of increased moisture levels in some more exposed parts, which indicates that the durability may be affected unless preventive measures are taken. The level of detailing to prevent the wood from getting wet has increased significantly in Sweden over the past two decades.

An on-going nationwide inspection study in the USA, which will include approximately 100 timber bridges that are exposed to different climates, will be published in 2014. Many of the inspected bridges have been in service for more than 50 years with sufficient structural performance but have been considered deficient for functional purposes. The expected outcome of the report will include a summary of inspection methods, a database of performance characteristics and tools for forecasting the service life of timber bridges. One main problem is how the bridge engineers perceive wood as a structural material.

6.3 Summary and key findings

- In the past, prestressing loss has been one of the major problems for SLT decks. Early studies showed that the prestressing reduction was more than 60% within a short period of time. The accepted solution to the prestressing loss has been to re-stress the deck.
- The main cause of prestressing loss in North American SLT decks has been the high initial moisture content of the deck beams. The shrinkage of drying deck beams resulted in significant prestressing reductions. The use of dry wood (such as glulam) beams prevents most of the prestressing loss.
- Another cause of prestressing reduction has been the lack of moisture barrier for SLT decks. Mechano-sorptive creep will occur when the moisture content fluctuates significantly. This can easily be prevented by covering the top surface and the sides of the deck, thus preventing rain and standing water on the deck from penetrating the deck beams. (***Paper V***)
- The use of reinforced anchorage zones for the prestressing has shown great potential both in the lab and in pilot bridges.
- Fungi will cause significant degradation in the wood if they are not prevented. The two simplest solutions for preventing fungal growth are to reduce the moisture content in the wood or to create a toxic environment where growth is impossible. The use of wood preservatives in timber bridges is strictly regulated and uncommon in Sweden, while it is required in the USA. (***Paper V***)
- Inspections have shown that very little wood decay has been found in Swedish timber bridges. Much less long-term prestressing reduction was also reported in comparison to the reductions observed in the USA.

7 Conclusions

7.1 General conclusions

Part I of the study focused on the behaviour of SLT decks constructed using Swedish glulam beams in the serviceability limit state (SLS) and the ultimate limit state (ULS). From ***Part I***, the following conclusions could be drawn.

- A bending failure occurred in the edge beam of the tested SLT deck at a load equal to 4.5 times larger than the SLS load (load which would cause the deflection to be equal to span/400). A total of seven beams failed during the ultimate load test to failure. The failure mode of the beams shifted from bending to shear failure when counting from the edge beam. (***Paper I***)
- A full-scale test to failure showed large-scale redundancy in SLT decks. Even after failure, the deck was able to carry a load approximately twice the SLS load. (***Paper I***)
- Increased deflection values were observed in the deflection profiles when the prestressing level was reduced from 900 kPa to 600 kPa for the non-destructive tests. The observed deflection values increased still further when the prestressing was further reduced to 300 kPa. (***Paper I***)
- A significant increase in the deflection values was observed when the load was moved in the transverse direction from the centre of the deck close to the edge of the deck. (***Paper I***, ***Paper II***)
- Non-linear load-deflection behaviour was observed in the tested deck with the load close to the edge. The interlaminar slip started at load magnitudes well below 300 kN, which is equivalent to SLS load magnitudes. (***Paper I***)
- The simplified analysis method in EN 1995-2 (2004) provided the best estimation of deck deflection when the load was positioned close to the edge of the deck. However, the deflection values were significantly conservative when the load was shifted away from the edge. The other equivalent beam methods (Ritter 1990; Crews 2002; AASHTO 2012) were not developed for vehicle loading close to the edge. (***Paper II***, ***Paper V***)
- The longitudinal modulus of elasticity (MOE) values had the greatest influence on the behaviour of SLT decks. The influence of the transverse MOE and the in-plane modulus was significantly smaller. The material properties recommended in EN 1995-2 (2004) produced acceptable results when used in the analysis of SLT decks. (***Paper II***)

Part II of the study comprised understanding and modelling the interlaminar slip in SLT decks. From ***Part II***, the following conclusions could be drawn.

- Two different non-linear finite element modelling approaches were used to simulate the interlaminar slip in SLT decks. The elastic-plastic FE model

(EPFEM) resulted in a slightly stiffer global response in the deck compared with the contact FE model (CFEM). (***Paper III***)

- Both models were able successfully to predict when interlaminar slip started in the tested deck. Both models also captured the hysteresis load-deflection behaviour. (***Paper III***)
- The critical shear stress limit is dependent on the normal stress between beams, as well as the coefficient of friction (COF). The coefficient of friction values differ between motion parallel and perpendicular to the wood grain. This results in anisotropic friction behaviour between beams in SLT decks.
- The magnitude of the normal stress in an SLT deck varies significantly, depending on the transverse load. An increase in the compressive stress in the deck from 0.6 MPa to 2.6 MPa was observed when a 900 kN axle load was applied to the deck. (***Paper III***) The magnitude of the normal stress varied significantly for the beams closest to the deck edge when discrete prestressing anchorage plates were used. (***Paper IV***)
- SLT decks without butt joints showed little or no interlaminar slip when the prestressing level was 300 kPa or higher. (***Paper IV***)
- A reduction in terms of deck stiffness occurs when butt joints are used in SLT decks. The global stiffness reduction depends on the amount of cross-sectional reduction. The behaviour both before and after initial slip in the deck is affected by the number of butt joints. (***Paper IV***)

Part III focused on studying both new and old existing bridges to learn about their durability and long-term performance. From ***Part III***, the following conclusions can be drawn.

- The prestressing reductions observed in North American bridges are caused by the use of wet deck laminations and the lack of moisture barriers. Wet deck beams caused a significant prestressing reduction due to shrinkage of the wood. When beams with a low moisture content were used, the reduction was much smaller, sometimes negligible.
- Wood preservatives are effective in preventing wood decay in North American timber bridges. However, the environmental and human effects of wood preservatives are still not fully understood and agreed upon. (***Paper V***)
- Swedish SLT decks which are constructed with dry glulam beams without the use of preservatives have performed better than most bridges reported in the literature. Smaller prestressing reductions have been observed and no severe wood decay has been found.
- Proper detailing of SLT decks, together with regular inspections and maintenance, is the key to ensuring good durability. (***Paper V***)

7.2 Suggestions for future work

It was concluded from this project that non-linear behaviour occurs in SLT decks that are subjected to high concentrated loads close to the edge of the deck. The interlaminar slip in SLT decks has been successfully simulated using non-linear FE models. Most of the laboratory work and numerical simulations involved single-span, simply supported decks. However, continuous SLT decks over several intermediate supports are very common in Sweden. More work on the difference in terms of structure behaviour between single-span decks and continuous decks is required. The non-linear models can be used in the future to determine how the dimensions of SLT decks and the boundary conditions influence the amount of interlaminar slip.

1. Do different span-to-depth ratios influence the amount of slip?
2. How does the deck behaviour change due to variations in transverse stresses (and thereby the critical shear stress limits)?
3. Is there a minimum deck depth that can be used in order completely to avoid interlaminar slip in SLT decks?

The butt-joint stiffness and strength reduction in SLT decks constructed using glulam beams have still not been accurately determined and quantified. A stiffness reduction should be implemented in the Eurocode (EN 1995-2 2004) as quickly as possible. The non-linear FE model can be used to determine the influence of various butt-joint configurations in SLT decks.

1. What minimum longitudinal butt-joint spacing is necessary to minimise the amount of interlaminar slip?
2. What is the reduction in deck strength and stiffness for different butt-joint configurations?
3. How can simple solutions such as friction enhancement and/or mechanical fasteners improve the function of decks with butt joints?

There are several other timber bridge types that utilise the principle of stress lamination. Both T-beam and box-beam bridges can be analysed using the non-linear models proposed in this project. However, other combinations of materials could also be considered. A combination of steel girders and a stress-laminated-timber deck could be possible. The shear stresses are usually critical for most types of stress lamination. They could all be analysed using the non-linear FE model with some minor adjustments.

Further monitoring of the SLT decks in service in Sweden is strongly recommended. A great deal of valuable knowledge can be acquired from studying how the performance of bridges in service changes with age. The alternative prestressing anchorage solution also needs further verification of its performance. More SLT decks constructed using screw-reinforced anchorage zones are needed.

In the author's opinion, the greatest problem for timber bridges in general is the common perception of wood as a structural material. Too many bridge engineers and authorities world-wide have a negative perception of wood as a structural material. This is perhaps due to a lack of knowledge, or the lack of well-documented good examples. More work to change the perception of wood is needed for timber bridges to be truly competitive.

New and improved tools for conducting life-cycle assessments (LCA) on bridges could help to change the perception of timber as a structural material for bridges. Timber bridges perform well in LCA and timber is a natural carbon material and is grown locally all over Sweden. The low weight and potential for prefabrication will simplify transportation. Recycling untreated timber is fairly easy and risk free in comparison with conventional materials. There are strong indications from past research that the cost of maintaining timber bridges is smaller than that of other bridge materials.

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