

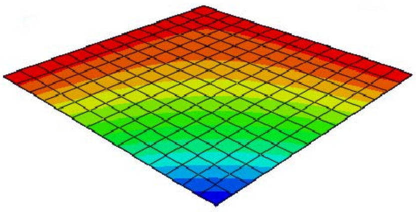
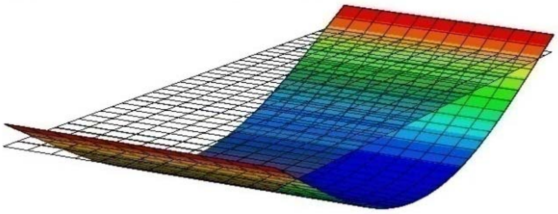

Influence of Butt Joints on Stress Laminated Timber Bridge Decks

Theoretical and Experimental Study

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

ANDERS CARLSSON
MÓNICA LUCAS ROMERO

Department of Civil and Environmental Engineering
Division of Structural Engineering
Steel and Timber Structures
CHALMERS UNIVERSITY OF TECHNOLOGY
Göteborg, Sweden 2010
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Cover:
Stress-laminated timber bridge decks during tests and deflections from FE models performed before tests. (Photo: Anders Carlsson, Mónica Lucas Romero, 2010)

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ABSTRACT

Stress laminated timber decks (SLTD) are relatively common in design of bridges today. Timber is a light weight material which is very strong in comparison to its weight. It is also a relatively cheap and renewable material. The bridge decks are made of solid timber or glulam laminations that are pre-stressed with steel bars in transversal direction to act as a solid timber slab. This gives the deck an orthotropic plate behaviour.

The timber decks made of laminations often have a longer span than the included laminations and therefore the beams have to be joined. These butt joints reduce the strength and stiffness in the SLTD. This project deals with the problems caused by butt joints. The influence of the joints on the deck's structural behaviour was evaluated by both calculations and laboratory tests of timber decks in reduced scale.

The laboratory tests were made on bridge decks both without butt joints and with butt joints arranged in different patterns. Because timber is a natural material with a large variation in behaviour between different specimens the same materials were used in all tests, when possible. Initially a deck was tested and then the individual laminations were cut in order to introduce butt joints in the timber deck. This procedure was made two times to study the difference between different densities of joints.

Test results show that there is an influence of butt joints on the structural behaviour. The behaviour also varies with different pre-stress levels. In a deck with butt joints the pre-stress level has a more significant influence than for a deck without butt joints. From the tests, observations were made that butt joints had an influence on the longitudinal stiffness. The influence of butt joints was more significant at a lower pre-stress value. From the results of shear tests, no difference in behaviour related to butt joints was observed.

Key words: timber bridge, stress-laminated timber bridge deck, butt joints, orthotropic plate, pre-stress, friction, moisture content, bending stiffness, shear modulus

Stumskarvars inverkan på tvärspända träbroar
Teoretisk och experimentiell utvärdering

Master of Science Thesis in the Master's Programme Structural Engineering and Building Performance Design
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SAMMANFATTNING

Tvärspända träbroar är vanliga inom byggnation av broar idag. Trä är ett lätt material med hög hållfasthet i förhållande till vikten. Trä är också ett relativt billigt material samtidigt som det är förnyelsebart. Tvärspända träbrobaneplattor är tillverkade av trälameller som spänns ihop med stålstag och får ett beteende som en solid platta. Denna träplatta kan antas ha ortotropa materialegenskaper.

Vid långa spännvidder behövs plattor som är längre än de individuella lamellerna i plattorna. Därför skarvas lamellerna med stumskarvar som reducerar hållfastheten och styvheten för bron. I det här projektet har stumskarvars inverkan på träplattans uppträdande undersökts och både beräkningar och prover av plattor i mindre skala har utförts.

Provningen av träplattor gjordes både med och utan stumskarvar. Eftersom trä är ett naturligt material med mycket stor variation mellan olika provbitar har samma material använts vid de olika testerna. Först testades en träbroplatta för att sedan plockas isär och lamellerna kapades för att göra skarvar i plattan. Sedan gjordes nya tester på plattan. Denna procedur upprepades ytterligare en gång för att undersöka skillnaden mellan två olika tätheter mellan skarvarna.

Resultaten från testerna visar att stumskarvar påverkar egenskaperna för tvärspända träbroplattor. Egenskaperna påverkas också av olika förspänningsnivåer. I en träbro med stumskarvar har förspänningsnivån en större inverkan än i en träbro utan skarvar. Under testerna observerades att stumskarvar påverkar böjstyvheten. Vid skjuvning i planet har inte stumskarvar lika stor påverkan.

Nyckelord: träbro, tvärspänd, stumskarv, ortotropisk platta, förspänning, friktion, fukthalt, böjstyvhet, skjuvmodul

Influencia de las Juntas de Tope en los Tableros de los Puentes de Madera Pretensada
Estudio Teórico y Experimental

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

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RESUMEN

El uso de la madera pretensada en los tableros de los puentes se está convirtiendo hoy en día en una práctica común. La madera es un material muy ligero, que además es muy resistente en relación a su peso, y además es un recurso natural renovable. Los tableros de los puentes están hechos de madera tratada o madera laminada encolada que por medio del proceso de pretensado en el que el acero se tensa en sentido transversal a la dirección de las láminas, conforman un sólido bloque. Esto, es lo que le proporciona al tablero su comportamiento como placa ortotrópica. Los tableros de los puentes de madera laminada suelen tener mayor vano que el inicial de las vigas y por lo tanto, éstas deben ser unidas mediante juntas de tope. Estas uniones lo que hacen es reducir el esfuerzo o tensión que soporta el tablero del puente y esto conlleva una pérdida de la rigidez del mismo. La influencia de tales uniones en los tableros de estos puentes pretensados ha sido el objetivo de control y cálculo en el laboratorio durante la realización de esta tesis.

Las pruebas del laboratorio se hicieron en tableros de puentes sin juntas de tope, así como en algunos tableros modificando la disposicion del número de juntas. Debido a que la madera es un material natural con un variado comportamiento en función de la naturaleza de las vigas que se usen, se trató de emplear el mismo material para todas las pruebas, en la medida de lo posible. En primer lugar, se creó para los ensayos un tablero de puente de madera laminada y se cortaron las láminas para crear el número de juntas deseadas.

Durante los ensayos se observó la influencia de las juntas en la reduccion de la rigidez del tablero. Dicha influencia fue más significativa cuanto menor era la fuerza de pretensado en el módulo de Young. De los resultados del ensayo de torsión, no se obtuvo ninguna conclusión en relacion con las juntas de tope por la obtención de resultados inesperados.

Palabras clave: puente de madera laminada, tablero pretensados en puntes de madera laminada, juntas de tope, pretensado, fricción, contenido de humedad

Contents

Preface

In this study, an investigation of influence of butt joints on stress laminated timber bridge decks (SLTD) has been performed. Studies of theory with calculations were added to the experimental lab tests of the structural behaviour. The study has been made from January 2010 to June 2010. The project was carried out at the Department of Structural Engineering, Steel and Timber Structures, at Chalmers University of Technology, Sweden. The project was in close co-operation to a research project called Competitive Bridges and the material for the laboratory tests had been financed by the glulam manufacturer Moelven, Töreboda.

This thesis work was carried out with PhD student Kristoffer Karlsson as supervisor and Professor Robert Kliger as examiner. All tests were made in the laboratory of the Department of Structural Engineering at Chalmers University of Technology.

The laboratory tests during the project could never been conducted without the sense of high quality and professionalism of the laboratory staff. We want to thank Lars Wahlström for his cooperation and help during the work. We also want to thank Thomas Kruglowa, PhD student at Chalmers, for his help during the lab tests and his help and support during the project.

Special thanks are also directed to our colleagues Fernando Serrano Toledano and Jasim Muhsun Naser, for their feedback and cooperation during the project. We also want to thank them for the help with our FE models which had been much more time consuming without their support. Thanks are also sent to our opponent group Albert Giró Piracés, Núria Fabregat Bolant and Enming Chen for their help and feedback to our work.

We also want to thank our friends and families for their support during the project.

Göteborg June 2010

Anders Carlsson
Mónica Lucas Romero

Notations

Roman upper case letters

A	Area of cross section
C_b	Butt joints factor
$C_{bj}, C_{bj\min}$	Butt joints factor for each lamination
D_t	Effective distribution width (Crew´s)
D_w, D_{wi}	Effective distribution width (Ritter´s)
E, E_x, E_{ts}	Longitudinal modulus of elasticity
E_y	Transversal modulus of elasticity
F	Applied load
F_1, F_2	Applied loads for bending test
F_k	Applied load in relation with kinetic friction coefficient
$F_{\max}$	Maximum applied load needed to move a block
F_s	Applied load in relation with static friction coefficient
G_{xy}, G_{ts}	In-plane shear modulus in x-y direction
I	Second moment of inertia for deck without butt joints
I_e	Effective moment of inertia
L	Length of lamination
M	Bending moment
N	Vertical load that holds block to ground
N_i	Number of butt joints
P	Applied load for static bending test
V	Shear force

Roman lower case letters

a	Distance between applied loads
b	Width of lamination
d	Distance between pre-stressed bars
f	Frequency
f_1	Natural frequency
h	Height of lamination
k	Stiffness reduction factor
l	Distance between butt joints
l_1	Gauge length (distance between LVDT´s)
$\ell_{,1}$	Minimum spacing between butt joints
t	Width of lamination (EC)
w_1, w_2	Deflection from bending test
$w_z(0,0)$	Deflection in the middle of the deck
x, y, z	Main directions in a cartesian coordinate system

Signs and mathematical symbols

%	Percent
Σ	Summatorium

Greek letters

α	Factor for determination of effective distribution width D_w
θ	Factor for determination of effective distribution width D_w
λ	Factor of influence of development length with distance between supports
μ	Coefficient of friction
μ_k	Kinetic coefficient of friction
μ_{max}	Maximum coefficient of friction
μ_s	Static coefficient of friction
ν	Poisson´s ratio
ρ	Density
σ	Bending stress
σ_{max}	Maximum bending stress
τ	Tangential stress
τ_{max}	Maximum tangential stress
Δ	Development length for deck with butt joints
Δ_{max}	Deflection at the critical section for deck with butt joints

Abbreviations

A.D	Anno Domini
AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
B.C	Before Christ
EC (5)	Eurocode
EN (408)	European Norm
FE	Finite elements
G	Giga-
Glulam	Glued-laminated timber
k	kilo-
LVDT	Linear Variable Differential Transducer
M	Mega-
m	meter
m	milli-
MOE	Modulus of elasticity
MOI	Moment of inertia
N	Newton
OHBDC	Ontario Highway Bridge Design Code
Pa	Pascal
PhD	Philosophy Doctor
SLS	Serviceability Limit State
Spb	Spacing per bar
ULS	Ultimate Limit State

1 Introduction

This thesis deals with the influence of butt joints on stress-laminated timber decks. The background, aim, limitations and methods used are described in this Chapter, as well as the outline followed in the whole work.

1.1 Background

Stress laminated timber decks (SLTD) can be used as bridge decks. In Sweden they often used in pedestrian bridges. Timber is used since it is a light weight material which is very strong in comparison to the weight. Furthermore, timber is also relatively cheap and a renewable material.

Timber decks made of laminations often have a larger span than the included laminations and therefore the laminations have to be joined. These butt joints reduce the strength and stiffness of the plate. From the manufacturer's point of view there is an interest to be able to increase the number of butt joints, since the laminations would be cheaper in shorter dimensions. There are also logistic reasons to use shorter laminations, because of the cost and the need of use relatively small transport vehicles due to the road regulations.

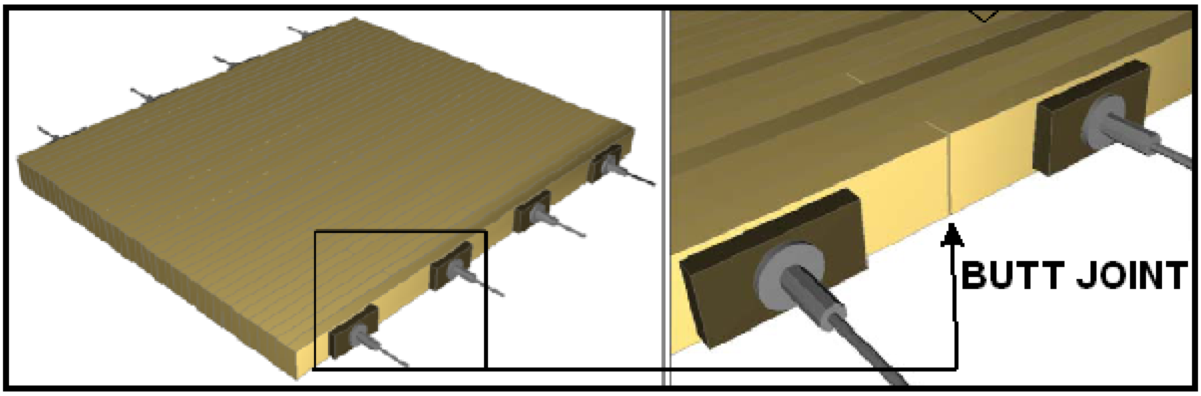


Figure 1.1 The principle of butt joints in SLTD. When a bridge deck is longer than the included laminations the laminations have to be jointed with butt joints.

Some previous researchers studied butt joints during the last decades. Michael A. Ritter (1990) wrote a design guide where he dealed with the problems of butt joints. He introduced a butt joint factor that described the influence from different densities of the joints. He also introduced some guidelines for the limitations of distance between the joints. Another researcher who worked with this type of problems was Keith Ian Crews (2002), who wrote his PhD thesis about cellular decks, a similar type of timber decks used in Australia. In his research he introduced some formulas about how to deal with butt joints in the design of SLTD. There were both agreements and contradiction between Ritter's and Crew's work. They were for example not really in agreement about how much influence butt joints have on the behaviour of the bridge deck. But there should be ways to improve the behaviour of the bridge deck when butt joints are used.

This project studied the problems that come up when butt joints are used in SLTD. The influence of joints on the decks structural behaviour was investigated by both experimental and theoretical studies.

1.2 Aim of the thesis

The aim of this thesis was to better understand how to deal with butt joints in design of SLTD. The idea was to understand the difference between a bridge deck with or without butt joints, and to study the differences in behaviour between different butt joint patterns. There was also a wish to find out why there were differences in the conclusions from older research work in this area.

1.3 Methods

A literature study was made with reference to SLTD and especially about the influence of butt joints. Different evaluation methods for lab tests were analyzed in order to find the best test methods to reach the aims of the thesis. The objective was to try to improve the butt joints behaviour under loading.

For the lab tests all materials and test equipments were designed with traditional calculation and design methods. In order to verify the dimensions of materials and sizes of loads also FE analyses of SLTDs used in the lab were conducted.

The lab tests were performed in the laboratory of the Department of Structural Engineering at Chalmers University of Technology. The longitudinal MOE and the in-plane shear modulus were the main parameters that were investigate. In order to verify the results from the bending test, also a dynamic test was made for each lamination in order to obtain the longitudinal MOE.

The influence of butt joints on the decks structural behaviour was investigated and some improvements of design were presented. This was made by literature studies and with laboratory tests of bridge decks in scale 1:2. The results from the lab tests were compared to the results from previous calculations on SLTD following EC 5 (2004).

1.4 Limitations

This work was made during a 20 week semester and the amount of work has been conformed to be able to finish within this time. Due to the time limit the relation between moisture content and the behaviour of SLTD was not evaluated, but the moisture content in the material was measured in order to control that the values seemed reasonable. Also, a comparison with a detailed FE model would have been desirable in this thesis but that is something that can be made in further research.

The bending test was made on one simply supported bridge deck with one type of timber. It was of quality C24 and not GL32 which is the ordinary used in SLTD. This gave the timber decks a lower strength and a larger deflection than for an ordinary SLTD.

During the tests the pre-stress levels were measured with strain gauges on the bars. Since the level of pre-stress were calculated from the strain the desirable level was hard to control. Therefore the stress levels between different tests varied a bit, and this variation might have some influence on the results.

1.5 Outline

This report is divided into 11 Chapters. The first chapter is the introduction part.

Second chapter gives an introduction to timber bridges. The first part of this chapter describes the history of timber bridges which is followed by a description of different types of timber bridges which have been used and that are used today.

In Chapter 3 the behaviour of SLTD is described including the influence of butt joints. This chapter describes friction, pre-stress systems and assembling of SLTD. There is also a description of butt joints and how they are dealt with in design of timber bridges.

Chapter 4 describes different testing methods for mechanical properties. Both dynamic and static test methods are described for the longitudinal modulus of elasticity. A pure twisting test, in order to investigate the in-plane shear modulus, is also described.

Chapter 5-7 is about the lab tests. In this part the materials used in the lab are described. The difference between the specimens and how they were assembled are described here. There are also descriptions of the equipment used in the tests. Chapter 5 describes the preparation for tests while Chapter 6 and Chapter 7 describe the bending and twisting test respectively.

The results are presented without any analysis or conclusions in Chapter 8. Both graphs and tables with the results are presented.

An analysis of the lab results is made in Chapter 9-10. A discussion about the results including the conclusions from the results is made in Chapter 10. Some suggestions for further research are also presented.

After the report, Appendices consisting of deeper information about the lab results can be found. Also additional materials, for example applied pre-stress levels, can be found in the Appendix part.

2 Timber bridges

This Chapter gives a short review of the history of timber bridges. It also describes some common types of timber bridges. A brief description of different types of timber bridge decks is also given.

2.1 Short introduction to the history of timber bridges

The wood has been an early used and common material during the history of bridge construction. Its history and development can be divided into four main periods, since timber bridges have been built according to the industrial situation for each period. Therefore timber bridge construction can be divided into these four periods *(AITIM, 2010)*:

- From the Prehistory until the Middle Ages (1000 A.D.)
- From the Middle Ages to the 18th century (1000 – 1800)
- The 19th century (1800 – 1900)
- The 20th and 21th century

The first timber bridges were probably fallen trees over a water-stream. The first human built bridges were probably made of timber logs. The idea of suspended bridges arose in the subtropical regions, when using climbing root and small shaft dimensions. Later on the use of ropes was initiated and the design became more sophisticated. Since 800 B.C many timber bridges were built by the Persians, Babylonians, Greeks and other civilisations from that time *(CSCAE, 2010).*

The Romans further developed the technique of timber bridge construction around 2000 years ago. A new type of bridge were made of beams and inclined piers with notches and the bridge could easily be erected and removed in order to be built in another place. This type of bridge is called “Ceasars Bridge” from the Roman Emperor of this time *(Ritter, 1990)*

Construction methods for timber bridges were not much developed between the Romans and the 16th century. The Architect Palladio suggested during the 1550s some new methods of timber bridge design. He introduced both the arch timber bridge and the truss bridge. With an arch it was possible to built spans up to 30m and with a truss the possible span could be up to 20m. But the interests in this kind of constructions were very small and not much further development was made before the 1850s. *(Ritter, 1990)*

The oldest bridges still standing in Europe were built from the 14th to 16th century and are covered bridges. Because they were covered the roof of the bridges provided better preservation of the structural parts. Many covered bridges are still in good conditions and in use (some of them are in use also for heavy traffic).

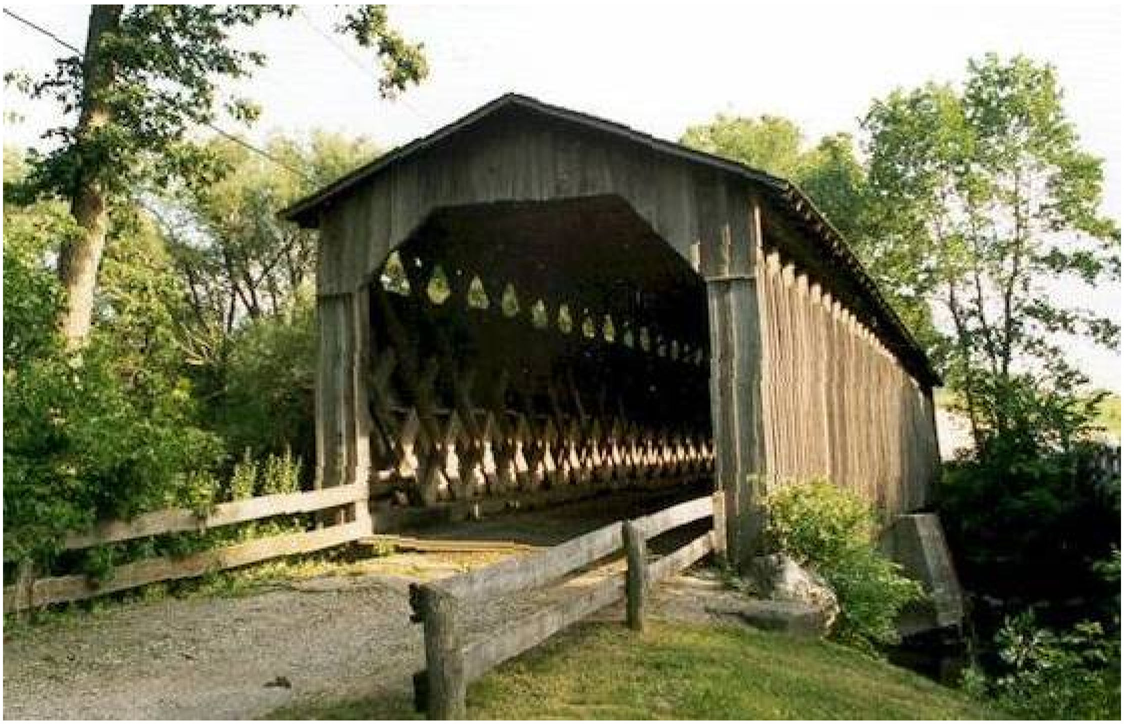


Figure 2.1 Covered bridge built by the Town of Cedarburg, Wisconsin, United States. It was built in 1876 and retired in 1962. Figure courtesy of American Society of Civil Engineers. (Ozaukee County, 2010)

In the 18th century the production development of timber bridges increased rapidly. More advanced techniques were developed in both Europe and the United States. French engineers developed a technique to build bridge arches of clamped timber planks and these bridges could be built in spans from 20 up to 50 m. In Germany a timber bridge with a single span of 120 m was built at Wittingen during this time. Timber bridges in United States constructed during the early 18th century were made of timber beams placed between timber piers. *(Ritter, 1990)*

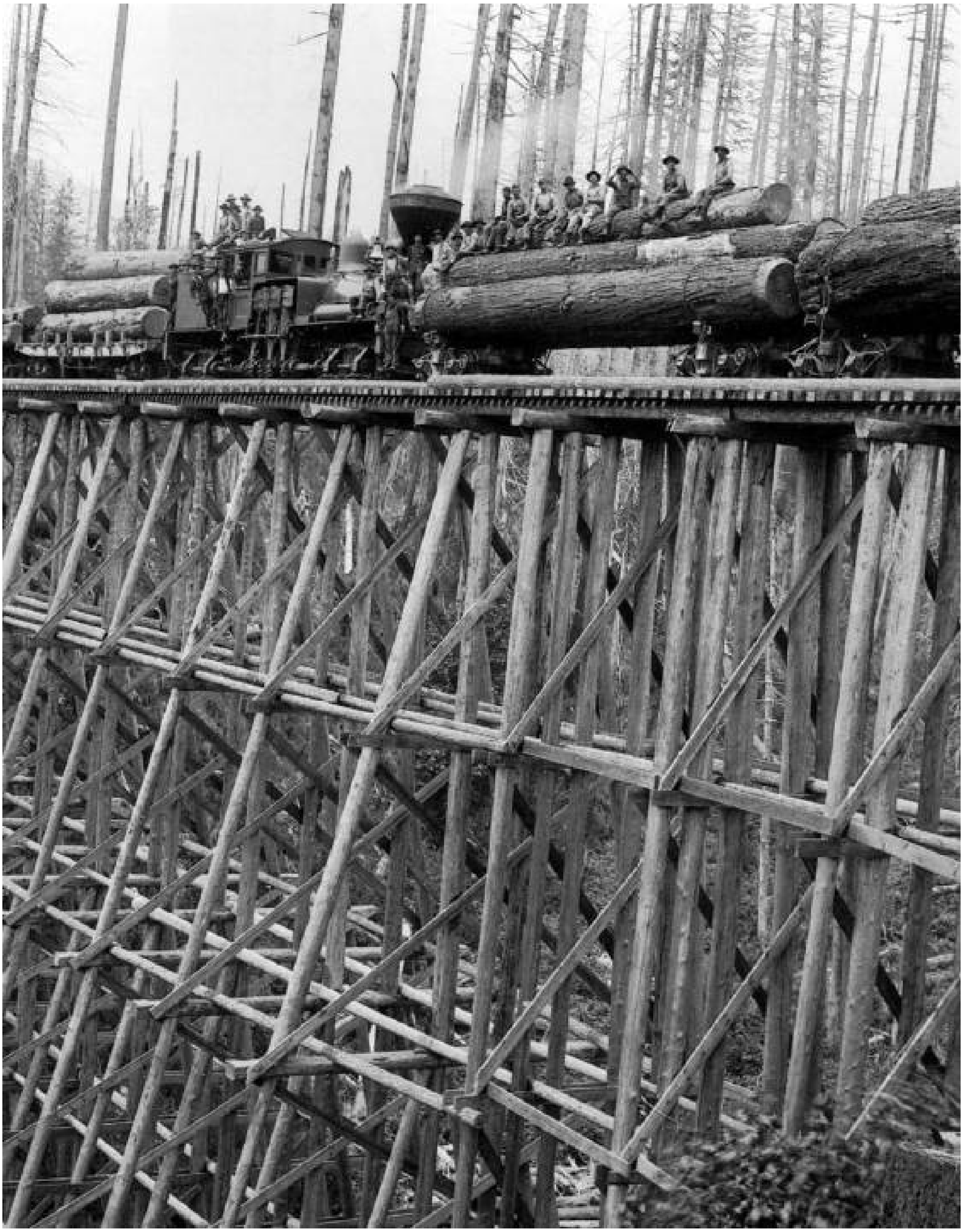


Figure 2.2 Forrestal Bridge in Seattle, United States (CSCAE, 2010)

During the 19th century, the industrialisation brought a major change in knowledge of bridge structures. New types of metal fasteners (bolts, connectors, tips, etc.) and overlapping joints instead of bonds were some of the innovations from this time. *(CSCAE, 2010)*

In Europe during 1802-1807, the Bavarian engineer Wiebeking, developed the horizontal lamination for the construction of bridges and in 1809 built the first glued laminated timber bridge in Altenmarkt.

In North America, the economical development led to an increased number of wood bridges with long spans. The engineers competed with each other which led to more advanced structures. At this time also the protection aspects became more important, and therefore the number of covered bridges increased. The first of those was the "Waterford Bridge" (1804) across Hudson River in New York. It was built by Theodore Burr with pine wood. It was covered in 1814 and is still in use today. It consists of four arches with different span lengths of 47, 49, 53 and 55 m.

The suspension bridge is an old bridge type but first known in Europe from the nineteenth century thanks to photos taken in China by the Scotsman Forrest. The first designs for long spans were obtained thanks to the development of steel cables that attached the wood deck. One of the oldest bridges of this type, which is in use today, is the Ojuela pedestrian bridge, in Mexico. It was built in 1892 with a span of 278 m.

During the 20th and 21th century trusses were introduced in covered bridges in North America. Some progresses were made in the field of wood preservation and the autoclave treatment systems. New and improved adhesives were developed, which later led to the use of glulam.

Figure 2.3 Leonardo da Vinci pedestrian bridge in Norway raised over the highway E-18 close to the city of Oslo. This bridge was built in 2001. (CSCAE, 2010).

Since then, the development of timber bridges has had a continuous development that has intensified in recent years after an interval of 40-50 years that were completely dominated by steel and concrete. Among the “new” types of timber bridges stress laminated timber bridge decks can be mentioned.

Nowadays, glulam is the most widely used material in timber bridges. It is made by sawn lumber laminations bonded together with some adhesives. It provides higher strengths than sawn lumber and allows to manufacture glulam members with a wide field of sizes. It means that the possible depth, width, and length are very variable. Thanks to the technological advances in the last years and the wish to build more and more renewable structures, the number of timber bridges and its applications are increasing.

Figure 2.4 "The Dragon's Tail" in the Thuringia region in Germany. The undulating bridge, with 240m length, spans a 25m deep valley close to the town of Ronneburg. (CSCAE, 2010).

2.2 Different types of timber bridges

There are a lot of different types of timber bridges. The span ranges can vary from simple beam bridges to composite bridges and SLTD. For long spans, there are also cable stayed bridges and arch bridges.

Figure 2.5 shows some of the used timber bridge types today and in which spans they can be built. There are a lot of different types of timber bridges and different types are used for different purposes and in different lengths. For longer spans, the bridge deck can be supported by an arch or cables connected to pylons

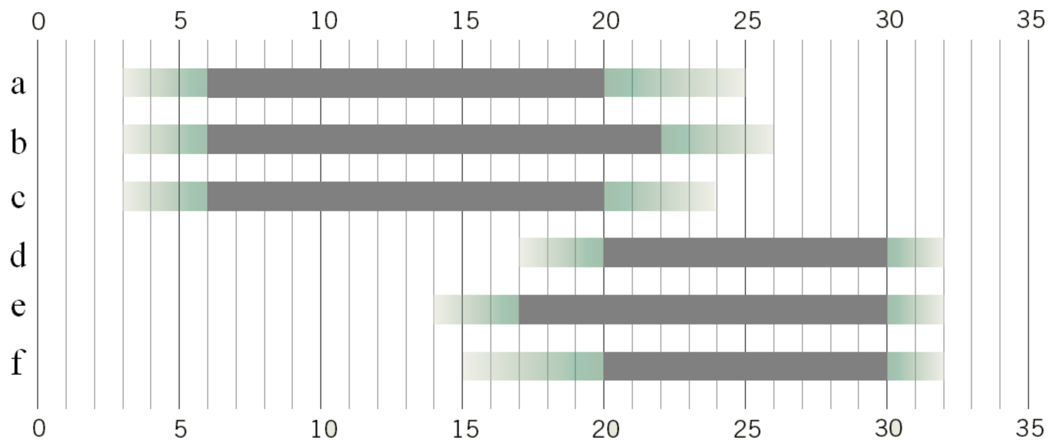


Figure 2.5 *Recommendations for different span lengths in different types of simply supported timber bridge decks. (Martinsons, 2010)*

The bridge types in *Figure 2.5*

a- Pedestrian beam timber bridge

b- Stress laminated timber road bridge

c- Stress laminated timber pedestrian bridge

d- Box-beam timber pedestrian bridge

e- Timber truss pedestrian bridge

f- T-beam timber pedestrian bridge

2.2.1 Timber beam bridges

Timber beam bridges are carried by timber beams with the bridge deck placed on the beams. The beams are often made of glulam. Transversal beams can be included between the longitudinal beams in order to make in larger widths of the bridge.

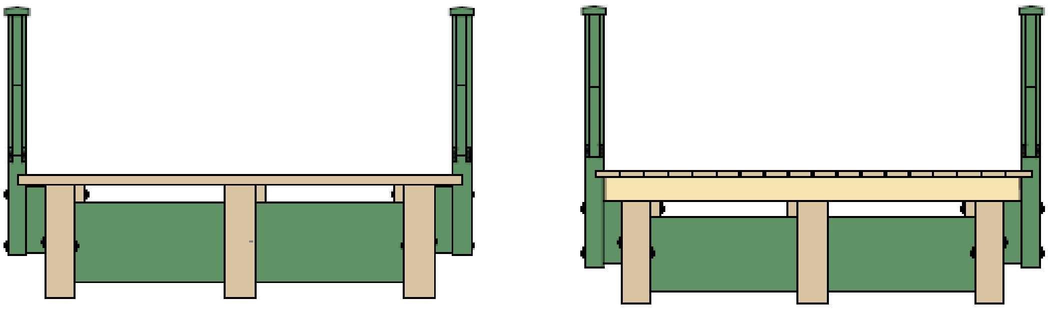


Figure 2.6 Timber beam bridge cross sections. The left figure is a bridge without transversal beams and in the right one is transversal beams included to carry the bridge deck, (Martinsons, 2010).

2.2.2 Timber truss bridges

A timber truss bridge usually consists of trusses and a floor system between them. In many cases the trusses also act as parapets. Timber trusses consist of straight members acting in either compression or tension.

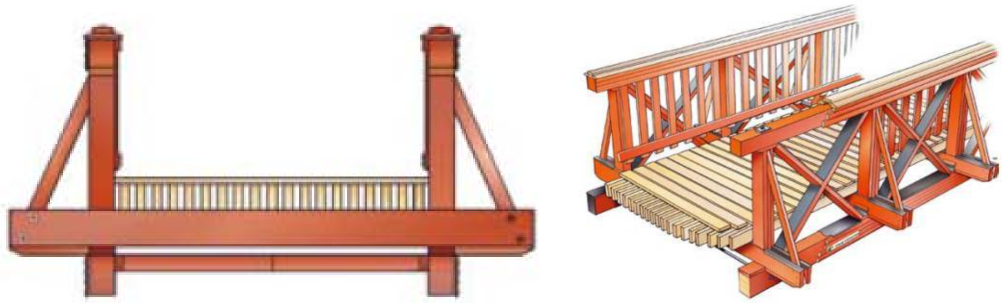


Figure 2.7 Timber truss bridge. Figure from Träbroguiden, Martinsons (2010).

Trusses are light weight structures but also expensive, since they consist of many parts and have many connections. The maintenance is difficult to obtain and also very expensive for the same reason. Nowadays, most truss bridges are built for aesthetical reasons. *(Ritter, 1990)*

2.2.3 Timber arch bridges

In arch bridges the bridge deck is supported by an arch made of glulam. The arch is prefabricated and transported to the site. The arch bridges are divided into two different types; with two hinge arches or with three hinge arches. The two hinge arch bridges are used for short spans up to 24m but for long spans the three hinge arches are used. The main difference between the two and three hinge arches is that a three hinge arch also has a hinge on the apex. The three hinge arches is a more convenient way to build longer spans since they can be transported in two parts to the site *(Ritter, 1990).*

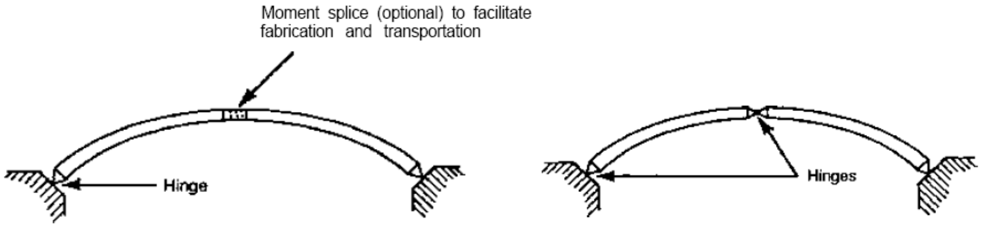


Figure 2.8 *Two different types of arch bridges. The two hinge arch to the left (hinges at the supports) and the three hinge arch to the right. The three hinge arch also has a hinge on apex, in addition to the ones at the supports (Ritter, 1990).*

Figure 2.9 *Three hinge arch bridge in Branäs in northern Sweden. This bridge is made for skiing. (Moelven, 2010).*

2.2.4 Cable stayed timber bridges

Cable stayed timber bridges can be built with relatively long spans. Since timber has a poor dynamic behaviour, the cable stayed timber bridges are used only for pedestrian bridges. In cable stayed timber bridges the bridge deck is supported by cables which are connected to the pylons on each side of the bridge span. The bridge deck can be carried on beams, or be made as stress laminated bridge decks, or T- and box beam cross section.

Figure 2.10 Cable stayed timber bridge in Sweden. (Moelven, 2010)

2.3 Different types of timber bridge decks

There are a number of different types of timber bridge decks. This part of the report describes some of the types used during the last 100 years. Some of the types are still common in use and some are not used any more.

2.3.1 Composite bridge decks

Composite bridge decks, with timber included, are built up with timber and concrete rigidly fixed to each other, in order to function as one unit. In most cases a concrete slab is casted on top of timber beams or a timber slab. For single span bridges, the concrete resists compression while the timber resists tension. But in continuous spans, the materials behave in the opposite way over the supports.

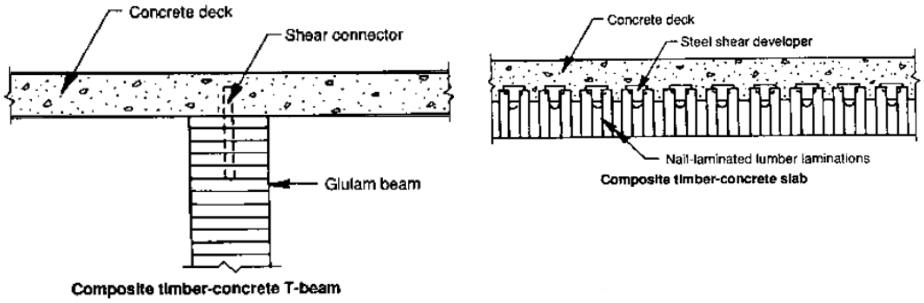


Figure 2.11 Composite bridge cross sections made of timber and concrete. The left figure shows a glulam beam with a concrete slab casted on it. Plane composite bridge decks are made of a timber deck with concrete cast on the top, shown in the right figure. (Ritter, 1990)

Composite bridge decks of timber and concrete were first used during the 1930´s. This type of deck was common during the following decades but it is very unusual today. One of the main disadvantages is the price because of the cast on site concrete (*Ritter, 1990*).

2.3.2 Nail- and dowel laminated timber deck

Nail laminated timber decks consist of timber laminations that are nailed together. Often the nail laminated deck is placed with the laminations in transverse direction and is carried by beams of steel or structural timber. This type was common in bridge construction from the 1920s to the 1960s, but after introduction of the technique of glulam, the use of nail laminated timber has decreased significantly. The maintenance of nail laminated timber decks is complicated since the laminations are nailed together and this makes it difficult to replace a single lamination. (*Ritter, 1990*)

2.3.3 Stress-laminated-timber bridge decks (SLTD)

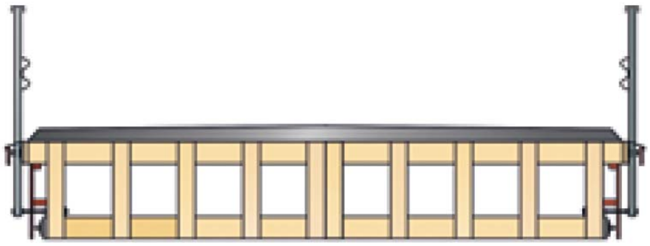
Stress laminated timber bridge decks (SLTD) consist of beams (laminations) connected with transversal steel bars. The deck behaves like an orthotropic plate, this means with different behaviour in different directions. Load can therefore also be carried in the transversal direction. When there are joints in the laminations the load can be redistributed to the surrounding laminations in the slab, because of the friction between laminations. SLTD is very commonly used in bridges today, especially in pedestrian bridges.

Figure 2.12 SLTD in a bridge construction, (Martinsons, 2010).

2.3.4 T- and box-beam bridge decks

A SLTD has a good structural behaviour in many cases but since it is a flat deck with a low height the MOI is relatively small. In order to increase the moment capacity the two types T-and box beam bridges were developed. These types were developed in United States during the 1980s. A T-beam bridge deck is similar to an ordinary SLTD, with the difference that webs made of glulam are added under the deck. A box-beam deck is similar to a T-beam but with a pre-stressed deck added also in the lower part of the webs to form a lower flange. *(Crews, 2002)*

a)



b)

Figure 2.13 Timber T- and box-beam bridge cross sections. a) T-beam bridge deck. b) Box-beam bridge deck. (Martinsons, 2010)

The T- and box beam decks have a higher MOI than an ordinary SLTD. Therefore they are suitable in longer spans than ordinary timber decks. One of the disadvantages is a higher construction height that can be an issue in some cases, for example when there is traffic under the bridge.

3 Behaviour of Stress Laminated Timber Bridge Decks

A SLTD consists of laminations that are transversally pre-stressed with steel bars in order to behave like a plate. This gives a better resistance to concentrated loads, since the load can be distributed between the laminations. SLTD is also an answer to the need to build longitudinal bridge systems with a continuous deck with longer span than the included laminations. The first standards for design and the construction procedure for SLTDs were made by Ritter (1990) and are included in the AASHTO standards.

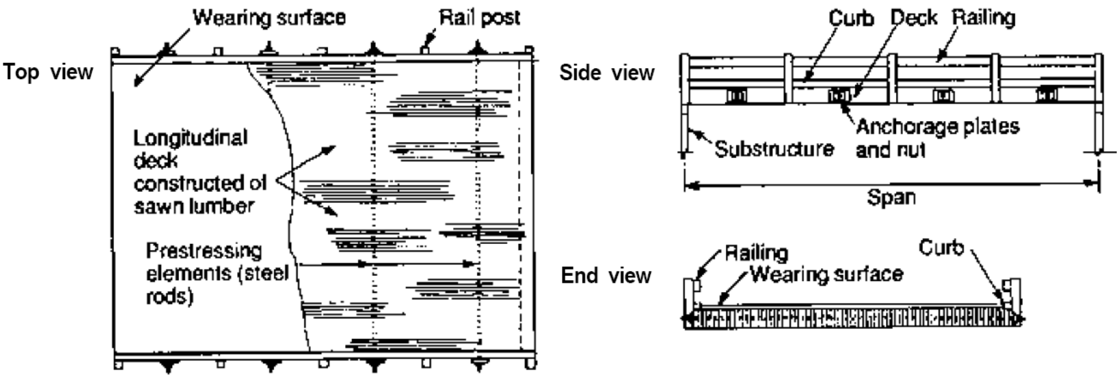
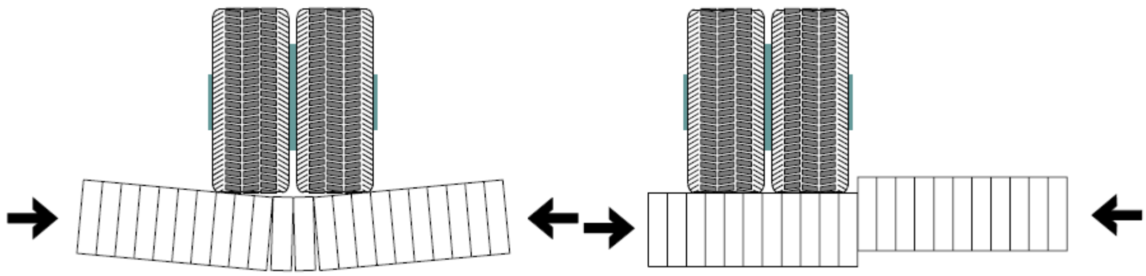


Figure 3.1 Normal configuration of a SLTD (Ritter, 1990).

SLTDs behave like a beam in the longitudinal direction (the direction parallel to laminations). If the applied load is uniformly distributed over the whole deck width then it will behave as a beam, but if the deck is subjected to a concentrated load the stress will be redistributed to the surrounding laminations. In the transversal direction the behaviour is a bit complicated. The load is transferred by friction caused by the pre-stressing force. Friction between the laminations is the most important function to resist shear stress between laminations.



a) *b)*

Figure 3.2 Effect of moment and shear applied in the transversal direction in a SLTD. a) bending moment is acting on the deck. b) effect of shear. The arrows in the figures show the pre-stressing force (Crews, 2002)

A transversal bending moment in a SLTD is resisted by the pre-stressing steel in tension and the timber laminations in compression. Transversal shear is resisted by the friction between laminations. *(Crews, 2002)*

3.1 Pre-stressing system

The pre-stress is acting in transversal direction to the laminations in a timber deck. The idea of pre-stressing is to help the laminations to act as one unit instead of acting like single beams. Pre-stressing will also help the laminations to redistribute the load if there are joints or any irregularities in the laminations.

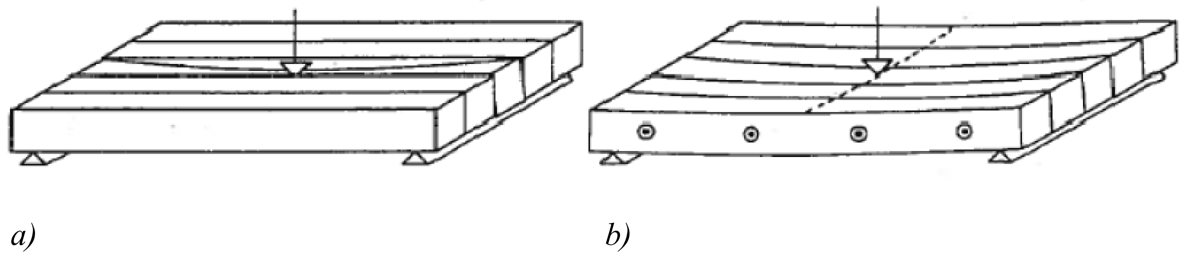


Figure 3.3 Behaviour of a pre-stressed slab compared to a slab without pre-stressing. a) a number of beams put together but with no connection between them. b) A deck with a pre-stressing system helping the beams to behave like an orthotropic plate. (Kalbitzer, 1999)

Pre-stress gives a redistribution of the loads in transversal direction, in addition to the ordinary longitudinal beam behaviour. If there is a concentrated load in the middle of the deck, then the middle lamination is resisting most of the load, but some load is redistributed to the other laminations. The result of this type of behaviour is shown in the right figure in *Figure 3.3*

3.1.1 Different pre-stressing systems

There are mainly two different ways to apply the pre-stress in SLTD. One method is to place the bars outside the timber laminations, with one bar on the top and one bar under them. The other solution, the mostly used today, is to use one pre-stressing bar in the cross section which is mounted in holes in the laminations. This type requires a lower number of bars, but there is some loss in shear capacity because of a decreased cross section area.

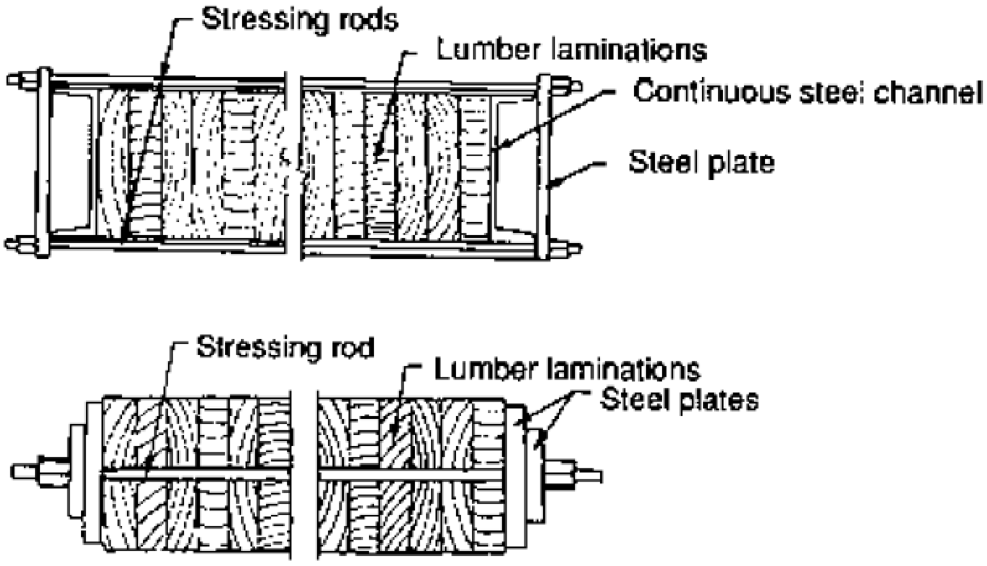


Figure 3.4 Different rod configurations for pre-stressing bars in SLTD. In the upper deck the pre-stressing bars are external and in the deck below the stressing bars are internal and placed in holes in the timber deck. Picture from Ritter (1990)

The anchorage regions of the bars have a large influence on the behaviour of the bridge deck. Most efficient way to anchor the bars is to put a continuous steel channel along the whole deck length with holes for the bars. This solution makes the deck stiffer at the edges and improves the resistance to deflection. This solution also gives a continuous distribution of pre-stress along the whole deck length. Transportation of the channel could also be complicated if it is very long and for logistical reasons it is sometimes preferable to use several small plates instead.

The most common anchorage solution for the pre-stressing bars is to put a rectangular bearing plate of steel or high density wood at the ends of each pre-stressing bar. This is a less expensive solution than the continuous channel because of the need of less material. The assembling of the deck can also be simplified since there is only one plate at each bar end, and not a long channel along the whole bridge.

When using load bearing plates to the bars it is very important to consider the stress distribution in the lamination close to the edge. The stress is distributed with an angle of 45 degrees into the timber and in the joint between the first and second lamination the stress should be fully distributed, in order to develop an appropriate pre-stress effect.

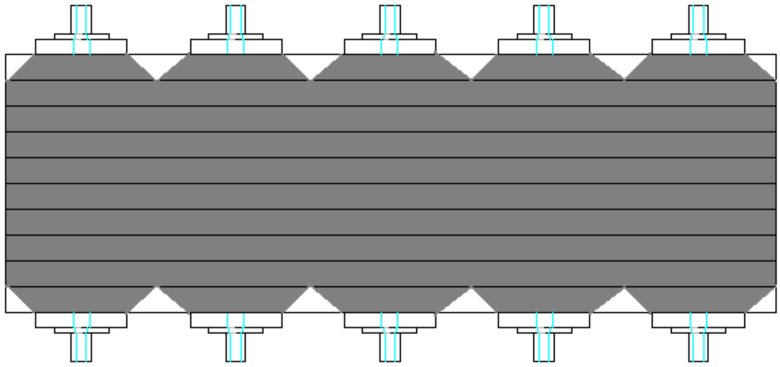


Figure 3.5 *SLTD with fully developed pre-stress between the two laminations that are closest to the edge. If a longer distance had been used between the pre-stressing bars the pre-stress had not been fully distributed between the two edge laminations.*

3.1.2 Friction from pre-stressing

Load distribution between laminations is influenced by the friction between them. If friction is low, then no or a very small part of the load is distributed to the adjacent laminations, but if there is a higher friction, the load distribution is more favourable. Friction is controlled by surface conditions for the laminations and the pre-stressing force.

The surface condition due to friction is controlled by the coefficient of friction μ. This coefficient can be defined from this formula:

$$\mu = \frac{F}{N} \tag{1}$$

Where F is the applied load that tries to move the block and N is the vertical load that holds the block and the ground together.

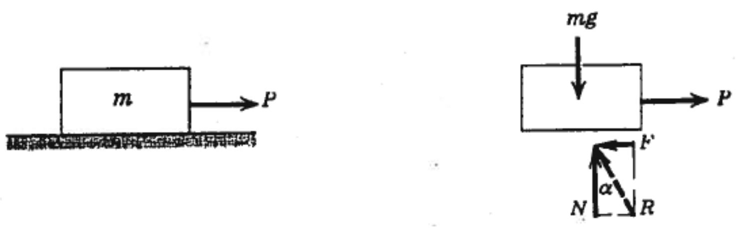


Figure 3.6 *Behaviour of a block subjected to a horizontal load and its own self weight.*

When a block is subjected to a horizontal load (P in *Figure 3.6*) that tries to move it on a surface, the self weight is acting in order to keep the block on the ground. This gives a force of friction that resists the load. With a heavier self weight, the load P has to increase in order to move the block. N is the normal force and F is the force caused by the friction. In a SLTD the same behaviour appears, but pre-stressing force acts in transversal direction instead of self weight. *(Kalbitzer, 1999)*

If the load is so large that it cannot be resisted by the friction, the block starts to slip on the surface. There are two different types of friction, static friction and kinetic friction. Static friction is the friction that occurs when applied load is not enough to move the block. When the block moves it is subjected to kinetic friction. The kinetic friction is smaller than the static friction *(Kalbitzer, 1999).*

The static coefficient of friction is given by the following formula (compare to *Equation 1*)

$$\mu_s = \frac{F_{\max}}{N} \tag{2}$$

The static coefficient of friction is given by the maximum applied force that is not enough to move the specimen.

The kinetic coefficient of friction is given by *Equation 3*:

$$\mu_k = \frac{F_k}{N} \tag{3}$$

A smaller force is needed to keep a movement than to start it. The same relationship exists between the coefficients of friction:

$$F_s < F_{\max} \tag{4}$$

$$\mu_s < \mu_{\max} \tag{5}$$

3.1.3 Relationship between material parameters and friction

Softwood has higher friction than hardwood. Since the surface of softwood is deformed and the contact area is larger than for hardwood, it has a larger contact area which gives a higher coefficient of friction.

3.1.4 Relationship between moisture content and friction

Friction of wood is strongly related to the moisture content. A totally dry wood specimen has a low coefficient of friction. If the moisture content increases, then also the coefficient of friction increases. Up to around 50% moisture content there is a strong relation between moisture content and friction. Above this value the influence of moisture is lower. At fibre saturation point the friction reaches its maximum value. A wet piece of wood is a bit softer than a dry one and therefore the real contact area for a wet piece is larger. This causes the increase of the friction when the moisture content increases (*N. Gnan, 1983)*.

If a surface is subjected to free water then the water will cause the reduction of the coefficient of friction since free water can work as a lubricant between surfaces.

3.2 Butt joints

Butt joints are introduced in a SLTD when the deck has to be longer than the included laminations. For SLTD with longer spans, butt joints reduce longitudinal stiffness and increase the deflection. The influence of butt joints is dependent of spacing between joints and level of pre-stress. This influence is also depending on the frequency of number of discontinuities across a deck.

According to Ritter´s guide (1990), the design values for butt joints are accepted as minimum pattern of 1 joint in each 4th adjacent lamination in a cross section (based in OHBCD (1991) requirements). Around this "weak area" which is caused by the butt joint, satisfactory shear redistribution has to be fulfilled from friction between the laminations.

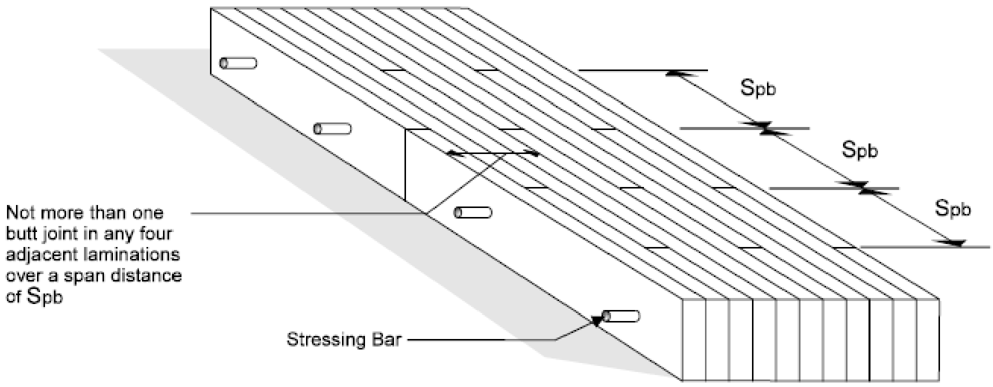


Figure 3.7 Distribution of butt joints in a standard pattern according to Ritter (1990). Here is a 1 in 4 recurrence in a SLTD with 4 steel bars. The distance between butt joints is denominated as Spb in the picture. (Crews, 2002)

3.2.1 Differences in design methods for butt joints in SLTD

Butt joints reduce the longitudinal stiffness and this have to be compensated during the design process. According to Ritter´s (1990) limitations in United States (OHBDC, 1991, and AASHTO, 1991), the minimum requirement for number of butt joints in SLTD is that there cannot be more than one butt joint in any four adjacent laminations. The Ritter´s guide (1990) proposes to design the SLTD as a beam with a width of D_w which is dependent of the traffic lines. This value is taken because it assumes that the load can be modelled as a longitudinal band of the deck. For the load distribution, two factors have to be computed, α and θ, who depend of the longitudinal MOE, the shear modulus, dimensions of the deck and the butt joint factor, C_b.

$$\alpha = \frac{2G_{ts}}{\sqrt{E(G_b)(E_{ts})}} \tag{6}$$

$$\theta = \frac{b}{2L} \frac{E(C_b)^{0.25}}{E_{ts}} \tag{7}$$

Ritter (1990) proposes a table with different values for the butt joint factor for SLTD, in function of the butt joints recurrence.

Table 3.1 Recommended values for C_B, butt joint factor according to Ritter (1990).

Butt joint frequency	C_B
1 in 4	0.80
1 in 5	0.85
1 in 6	0.88
1 in 7	0.90
1 in 8	0.93
1 in 9	0.93
1 in 10	0.94
No butt joints	1.00

Table 3.1 shows that if the butt joint frequency is 1 in 4, the coefficient of reduction is 80%. Therefore, the maximum reduction factor could be possible when following Ritter's criterion for butt joints is down to 80%, compared to a solid deck without butt joints. For timber decks without butt joints, the reduction factor would be equal to 1 (no reduction).

Crews´ (2002) limitations in Australia, propose some differences to the Ritter´s method for design of the decks with butt joints. The predicted distribution width is by Crew´s called *Dt,* and depends on the transverse section as well as of the value of deflection at the same critical section.

$$D_t = \frac{A}{\Delta_{\max}} \tag{8}$$

The formulas proposed by Crews (2002) assume a pre-stress level of 1000kPa, but if the design pre-stress level is less than that value, the distribution widths have to be reduced from 5% to 10%. The formula for equivalent beam width depends on the value of distribution width in function of the traffic lines, D_{wi} and the butt joint factors, C_{bj} and $C_{bj.min}$. This last reduction factor varies from 0.75 to 0.85 for hardwood and softwood decks respectively.

$$D_w = D_{wi}\left[\left(\frac{C_{bj}}{C_{bj.\min}} - 1\right)^2 + 1\right] \tag{9}$$

Shear transfer between laminations can be developed because of frictional contact between them which is induced by pre-stress. Crews (2002) described the development length concept which includes the distance between butt joints in analysis of stress redistribution between laminations. Development length, Δ, is the length that is necessary between two butt joints in order to get a full redistribution of stress between laminations. This length is dependent of friction and surface conditions are one of the influencing properties. For instance the development length for softwood is shorter than for hardwood. Stiffness of the deck has to be decreased if distance between butt joints is smaller than Δ. Development length is influenced by lamination thickness, b, maximum stress, σ_{max}, and shear stress, τ_{max}.

$$\Delta = \frac{b\sigma_{\max}}{2\tau_{\max}} \tag{10}$$

When considering a case when the distance between joints is equal to the development length, stress in the lamination with a butt joint is redistributed to the adjacent laminations and the longitudinal stiffness of the timber deck is the same as for gross cross section.

In order to be able to calculate the loss in stiffness because of butt joints, a stiffness reduction factor has to be introduced. The stiffness reduction factor, k, can be identified depending on the number of laminations and one variable called λ, which is the quotient of development length divided by the distance between laminations *(Crews, 2002).*

$$k = \frac{N - \lambda}{N} \tag{11}$$

$$\lambda = \frac{\Delta}{l} \tag{12}$$

In most cases, when following the design recommendations, the value for λ can be assumed to 1.0, which gives a bit conservative value of the stiffness reduction factor.

$$k = \frac{N - 1}{N} \tag{13}$$

With a known value of k, MOI and the reduction from influence of butt joints can be identified. Effective MOI is expressed as a function of the original MOI (considering no butt joints in the deck) times the number of laminations in the deck assembly and the reduction factor. This variable, I_e represents the remaining stiffness of the cross section.

$$I_e = kNI \tag{14}$$

With 1 in 4 butt joint recurrence: assuming λ value as 1 or different from 1, stiffness is higher for the first case. The difference between the two assumptions is not significant, but the first assumption can result in a lower reduction of stiffness. In the case of 1 in 8 recurrence, the values that can be identified are very close to each other because of the low reduction in stiffness.

One of the differences between the two design guides (Ritter and Crews) is allowed minimum distance between butt joints in two adjacent laminations. In United States a bridge deck can have a minimum spacing of 600mm (Ritter, 1990) while in Australia it is not permitted to use a spacing of less than 900mm for softwood and 1000mm for hardwood *(Crews, 2002)*.

Following the limitations in Eurocode (CEN,2004) no more than one butt joint is permitted in four adjacent laminations. This is the same pattern that Ritter´s guide (1990) recommended, but the allowed distance between butt joints according to Eurocode limitations have a different standard.

$$\ell_{,1} = \min\begin{cases} 2d \\ 30t \\ 1.2m \end{cases} \tag{15}$$

According to Eurocode (CEN, 2004), the minimum spacing between joints has to be the lowest value of three different requirements. The first requirement is two times the distance from pre-stressed bar to adjacent pre-stressed bar, second is thirty times the thickness of each laminations in direction of pre-stressing or the third one which is 1.2 meters. Furthermore, the section has to be reduced in relation to number of butt joints within a distance of four times the thickness of each lamination in the direction of pre-stress to calculate the stress strength.

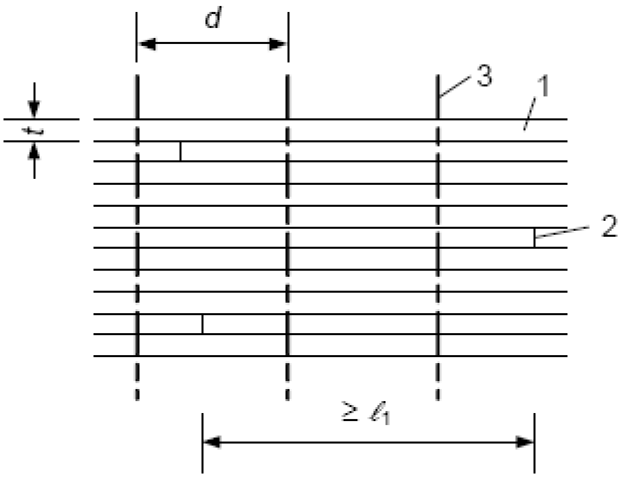


Figure 3.8 Recommendations distribution of butt joints according to Eurocode. Number 1 refers to lamination, number 2 the butt joint, and number 3 identifies the pre-stress steel bar. (CEN, 2004)

3.3 Assemble of SLTD

Following Ritter´s construction practices (1996) for assembly of SLTDs, there are three common ways. First method is to assemble it directly on the abutments. Second method is to assemble it close to the abutments and then lift it to the supports. The third and last method is prefabrication of SLTD. The bridge deck is in that case assembled in a factory and then transported to the site.

3.3.1 On site assembly

There are two methods to assemble a SLTD on site. One method is to assemble the laminations to one unit directly on the abutments. When using this method the laminations are put on the abutments and then pre-stressed together. This requires no butt joints in the deck, since the laminations have to be able to stand by themselves on the supports. However, butt joints can be possible to add if temporary supports are used during construction. This method is most usable when the span is short.

There is also a second method to assemble the bridge deck on site. The deck is then assembled temporarily on supports close to the abutments and intermediate supports, and the deck unit is lifted to and mounted on the supports. This method requires a very large and flat area close to the supports. Temporary supports can also be used in order to make a camber on the bridge, which improves the performance of the bridge, since the final deflection will be reduced. This method to assemble a bridge deck is very convenient when the bridge crosses a road and it has to be closed for a short time while the bridge deck is put in place. A timber bridge deck is very light weight compared to the size of it and therefore this method can also be used for relatively long bridges. *(Ritter, 1996)*

3.3.2 Prefabrication

Prefabrication in a factory is a common construction method for short bridge decks. A bridge deck is assembled in a factory and then transported to the site with a truck. Due to the road regulations, this method can only be used for relatively short bridge decks. One advantage of this method is that time needed on site is very short. The bridge deck can be put in place immediately when it arrives to the site. Therefore this method is convenient when a very short construction time is required. *(Ritter, 1996)*

3.3.3 Pre-stressing procedure

Pre-stress in SLTDs is applied in transversal steel bars. This presses the laminations together to form a deck unit. Pre-stress force is applied to the bars with a hydraulic jack and a common pre-stress practice is as follows.

A hydraulic jack is used to apply compression to the laminations in transversal direction. A back nut is put on each edge of the pre-stressing bar. Then the pre-stressing bar and the bearing plate are pulled away from each other with the jack. After that, the nut is tightened and the jack is released. Because of the nut, the pre-stress now remains in the laminations.

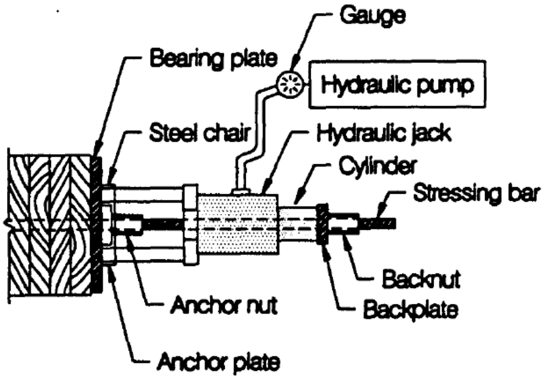


Figure 3.9 Equipment for bar tensioning. (Ritter, 1996)

Pre-stress is added from one side of the bridge deck. A backing plate and a nut are placed on one of the bar edges before the pre-stressing procedure starts on the other end. On this edge, the bar only has to be approximately 25 mm longer than the nut end, but on the pre-stressing side the bar has to be at least 250 mm longer in order to be able to fasten the jack to the pre-stressing system. The pre-stressing procedure of one bar could be divided in five steps:

- Start by adding the jack to the bar.
- Place the back plate and the back nut on the bar end
- Add a pressure with the hydraulic jack. The pressure should be 5-10% higher than the designed stress because of loss of the nut when it is tightened.
- The nut is tightened to the back plate.
- Release the hydraulic pressure from the jack and remove the jack from the bar.

With one single jack only one bar can be pre-stressed at each time. Since the deck includes a number of bars they have to be stressed to the same level. However, pre-stressing one bar will deform the wood and influence the stress in other bars. When one bar is stressed, the wood is shrinking, and the force in the surrounding bars decrease. Therefore the stresses in bars have to be controlled during the pre-stressing procedure of the other bars. Bars should be tensioned several times until all bars are fully tensioned up to the design level. If one bar is immediately tensioned to the design value, then the wood in laminations might be distorted. Therefore the bars should be pre-stressed several times during the process with an increased force each time.

The pre-stressing starts with one bar in one end of the deck and the bars are sequentially pre-stressed several times. The pre-stressing of the bars can be made in the following order:

- Apply 25% of design level
- Apply 50% of design level
- Apply 100% of design level
- The third step is repeated once more because of decreasing stresses in the surrounding bars.
- The fourth step is to control the force in several bars, and the bars that has lower than 10% below the design level should be re-tensioned.
- If stress level is enough, the third step does not have to be repeated again. But in some cases it has to be repeated a couple of times.

Due to time related stress loss in timber, pre-stressing procedure should be repeated three times. First time is immediately after assembling of deck, second time is 1-2 weeks after first time and then a third time which is 4-6 weeks after second time. The stress should also be controlled during the service life of the bridge. First control should be performed at two years after bridge construction and the following checks should be made at each 1-3 years. *(Ritter, 1996)*

3.4 Design procedure of stress laminated timber bridge decks

A short and easy to follow description of design is found in the report of Kalbitzer (1999). Design of SLTDs are performed in following order and the following steps are taken directly from his report:

- Define deck geometric requirements and design loads

- Select species and grade of lamination and compute allowable design values

- Determine preliminary lamination layout

- Compute the transverse modulus for the stress-laminated system

- Compute maximum vehicle live load moment

- Compute wheel load distribution width

- Estimate deck thickness and compute effective deck-section properties

- Compute deck dead load and dead load moment

- Compute bending stress

- Check live load deflection

- Compute dead load deflection and camber

- Determine required pre-stress level

- Determine spacing and size of pre-stress bars and the required pre-stress force in the bars

- Design of the anchorage system (plates of steel or hard density wood, avoid local compression failure in timber)

- Determine support configuration and check bending stress

Criteria and limitations can be found in EC 5, timber structures, and supplement to Eurocode about timber bridges.

4 Test methods for mechanical properties

This Chapter describes different testing methods to decide the mechanical properties in SLTD and is based on literature studies. A brief study of different testing methods was needed to perform necessary calculations and tests. The three methods which were decided to be used in this project are described in this chapter. The reasons why they were chosen are based, in addition to advantages in comparison to other test methods, on the possibility to carry out the tests in the available lab. One of the considered aspects was the laboratory test equipment available to use in the lab at Chalmers.

Since timber is a natural material with large variations between different individual specimens the same material was used in several tests, in order to get comparable results. Therefore the tests had to be of non-destructive character. Timber structures are usually designed due to deflection in serviceability limit state and the chosen limitations are therefore in agreement with design requirements.

The two elastic variables that were in interest in this project were longitudinal modulus of elasticity (MOE) and transversal shear modulus. To decide longitudinal MOE two testing methods was used and to decide shear modulus one method was performed.

4.1 Test of longitudinal modulus of elasticity

It was decided to use two methods to test the longitudinal MOE. The first test was a dynamic test where the eigenfrequency for every single lamination used in the timber decks was measured. From the eigenfrequency and the geometrical properties the longitudinal MOI for each lamination could be derived.

In order to test the longitudinal MOI for a SLTD a static bending test was made. This test was made according to EN 408 (2007) which is a standard method for four point bending test of simply supported beams. With this test it was also possible to investigate the influence of butt joints. Static and dynamic results could then be compared and evaluated.

4.1.1 Dynamic test of longitudinal MOE

A dynamic test was made to investigate the longitudinal MOE for every single lamination. This test was performed before the laminations were assembled to a SLTD. The test was made by measuring the eigenfrequency of each lamination. All laminations were also weighed and measured in order to obtain the geometrical properties that were necessary to derive MOI.

The lamination was placed between two supports close to the ends of the lamination. In one of the ends, by using a hammer, the beam was excited in fibre direction while at the same time a computer programme registered the frequency of the sound by using a microphone. The microphone was placed close to the other end of the beam. This was made twice, in order to see that the two measured values were almost equal. In the analysis the mean value of the two measured frequencies was used. By using a computer program, it was possible to find the longitudinal MOE.

The test method was based on fundamental theory of beams and has been widely used in earlier research. According to Hearmon (1952) approximate method that took into account the shear and MOI, the longitudinal MOE could be calculated as:

$$E = 4\rho f_1^{\,2} L^2 \tag{16}$$

Where, ρ is the density of the timber, f_1 is the natural frequency identified by the computer program, and L is the length of the laminations that in this case was 3000mm and 1350mm respectively.

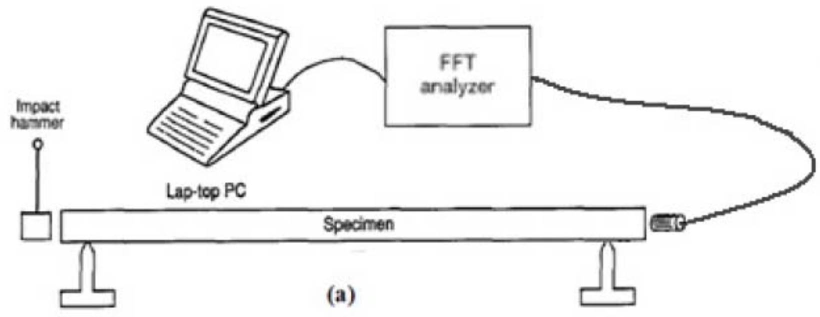


Figure 4.1 Equipment used in the dynamic test of longitudinal MOE. The lamination was excited by a hammer and a computer calculated the MOE from the measured eigenfrequency. (Jónsson and Kruglowa, 2009)

4.1.2 Static test of longitudinal MOE

A static test of the longitudinal MOE was chosen to follow the standard method EN 408 (2007). This is a method developed in order to evaluate the longitudinal MOE of simply a supported beam which is subjected to two concentrated loads. In this case, when investigating the behaviour of decks, this method could be used by applying two transversally applied line loads instead of concentrated loads.

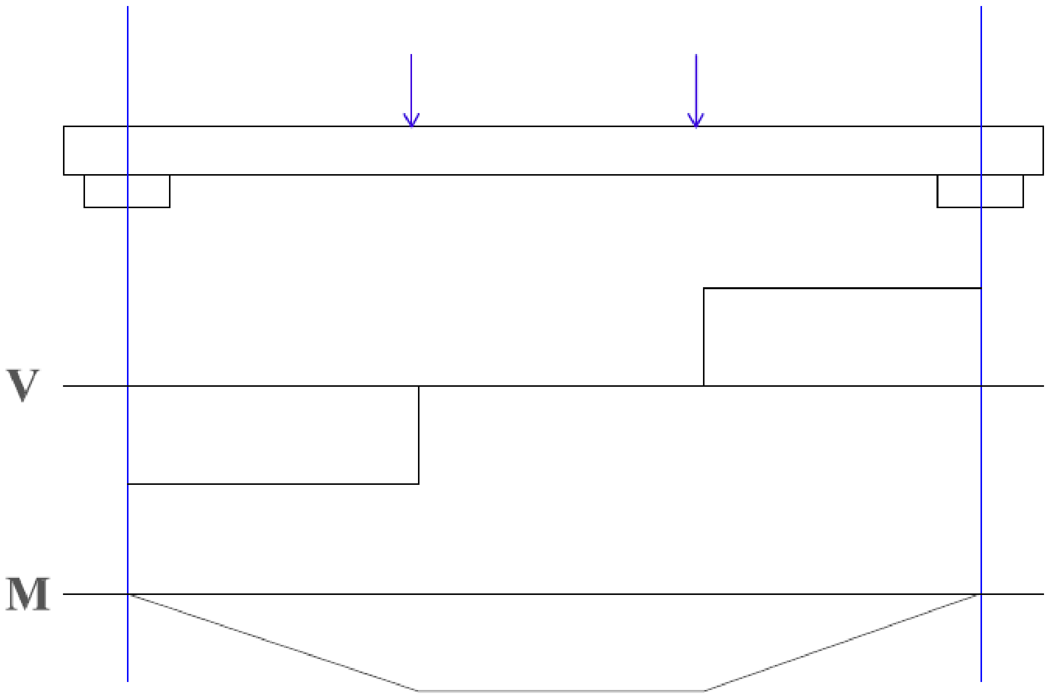


Figure 4.2 The recommendations of EN 408 (2007) were used to identify the longitudinal MOE. The deck behaved like a beam with two concentrated loads in the span. The deflection was calculated between the two loads.

Since a simply supported deck with two symmetrically applied line loads was used, the shear distribution was constant between the support and the closest load. But between the two loads the shear was zero. And the deck was subjected to maximum moment in the span between the applied loads.

In the first test a deck without butt joints were tested. This test was followed by two tests with butt joints introduced in the deck. Same laminations were used in all three tests, but before the two last tests they were cut in shorter to make butt joints. Because of the moment distribution, it was not necessary to create butt joints between the loads and the supports. In the tests with butt joints, the joints were placed only in the span between the loads.

From the applied load, the geometrical properties and deflection, the longitudinal MOE could be calculated according to this formula:

$$E_{m.l} = \frac{a l_1^{\,2}(F_2 - F_1)}{16I(w_2 - w_1)} \qquad (17)$$

Where l_1 is the distance between the measuring points close to the loads, *a* is the distance from support to load and *I* is moment of inertia for gross cross section. *F* is applied load, the recommendation of EN 408 (2007) was to measure load and corresponding deflection between 10% and 40% of the maximum allowed load. The deflection, *w*, was measured between the loads.

The length of the laminations used for this test was 3000mm and the distance between the supports was 2610 mm. A distance between supports of 18 times the height was the recommended value due to EN 408 (2007).

According to EN 408 (2007), the load and deflection used in the evaluation of MOE should be between 10% and 40% of the maximum ultimate load. In the recommendations, the use of values with a maximum deterioration of 1% has to be accounted. This means that the correlation factor cannot be less than 99%.

The load on the deck was applied by using a hydraulic jack acting on a load rig which distributed the load to two line loads. The maximum values of the applied loads were chosen according to calculations of the maximum deflections in the middle of the span in serviceability limit state. Therefore the applied load in the tests passed the 40% limit that was suggested. After the tests it was realized that the load passed 60% of the maximum allowable ultimate load. With a higher load the influence of butt joints were more significant.

4.1.3 Test of shear modulus

For test of transversal shear modulus a method designed by Tsai (1965) was chosen. This method was a combination between the theories of orthotropic plates and bending. In an earlier Master thesis work at Chalmers, Jónsson and Kruglowa (2009), the same experimental method was used. Therefore, the test equipments for the test were available at Chalmers, including special supports to obtain free rotation in the corners of the deck.

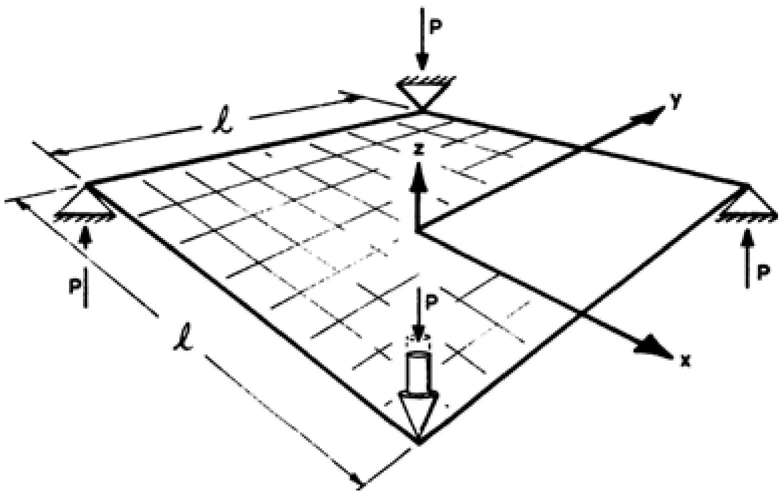


Figure 4.3 Principle for the pure twisting bending test developed by Tsai (1965) for plates.

In this pure twisting test, the load is applied in one corner of the deck while the other three corners are supported with free rotation in the corners. *Equation 18* gives the shear modulus, G_{xy}, from the results of the twisting test. This equation is obtained from a combination of orthotropic plate theory and bending equations.

$$G_{xy} = \frac{3Pl^2}{4h^3 w(0,0)} \tag{18}$$

The variable P is the applied load and $w(0,0)$ the deflection in the centre of the deck. This deflection is around four times smaller than the deflection in the corner with the applied load. h is the height and l is the length of the deck. The width is equal to the length in this case. Same measurement equipment, rig and supports were used as in Jónsson and Kruglowa´s Master Thesis project (2009).

Two tests were made, one with and one without butt joints. Same material was used in both tests and before the second test some laminations were cut to introduce butt joints. When studying pure shear, butt joints should not have any influence on the results, and the idea of this test was to see if there is any influence with butt joints. Therefore only two tests were made, with and without butt joints, while in the bending test two different butt joint patterns were used in different test runs. In bending butt joints should have a more clear influence on the results.

5 Preparation of specimens

The steps in construction of SLTDs are described in this Chapter. Measurement of geometry and moisture content, design of loads to be applied on the SLTD, drilling and assembly the laminations and experimental dynamic test performed, are described in this Chapter.

5.1 Geometry and density

The laminations were measured before they were used in the tests. Length, height, width and weight were measured for laminations of 3000mm and 1350mm length. All of them were numbered from 1 to 70, and from all values, the mean values for each dimension were calculated. The measured laminations were produced with a height of 145mm, thickness of 45mm, and length of 3000mm. After measuring all laminations, the mean geometry of them was found to be 145.14mm height, 44.86mm width and 2999.20mm length for the longest laminations. For the shorter laminations, with a length of 1350mm, the measured length was 1349.91mm. With the dimensions and weight of each lamination, the density of the timber could be calculated. The values of resulting density are shown in *Table B.1* and *Table B.2* that are found in *Appendix B.*

5.2 Design of applied loads

5.2.1 Hand calculations for bending test

The calculations were performed in the deck as a simply supported beam with two concentrated loads which were symmetrically placed. The loads were applied in order to avoid shear failure in the region of the supports and to get the maximum moments and deflections in the area between the applied loads. The test was performed following the EN 408 (2007) recommendations, but the applied load was designed according to the service limit state according to the maximum allowed deflection. For an applied load of 80kN, the calculated deflection was 9.64mm.

5.2.2 FE model for bending test

The SLTD was designed according to EC 5 (2004). Characteristic values for timber were taken from tables. The loads were applied according the test method EN 408 (2007). First the maximum allowed load in the test was decided. With this load the deflection in middle of the deck was calculated. This deflection was compared to the results for the real deck, first without butt joints and thereafter with butt joints in different patterns.

In order to verify the loads from the hand calculations a simple FE model was performed. The FE model was conducted with the software Abaqus. Loads and material parameters in the model were the same as in the hand calculations. The load was modelled as two line loads in the transversal direction of the deck. The deck was modelled with shell elements and the geometrical properties were the same as in the hand calculations. The E_x was 11GPa and found in tables for C24 timber.

Results from the FE model were in very good comparison with hand calculations. The calculated deflections had almost same value; the calculated value was 9.64mm while the result from the FE model was 9.67mm, with an applied load of 80kN.

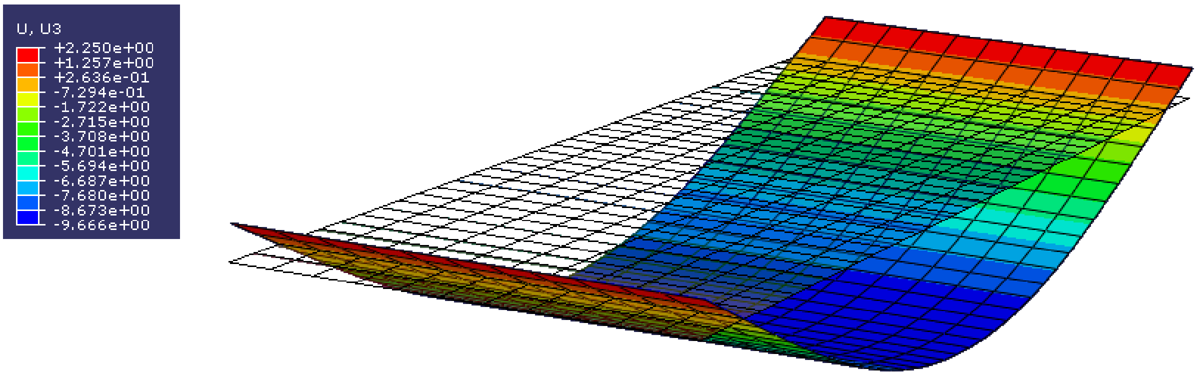


Figure 5.1 Result from the FE model made in Abaqus. The maximum deflection was 9.67 mm.

5.2.3 FE model for shear test

To check the materials and loads for the shear test a FE model was performed in Abaqus in order to design the loads applied in the tests. This model was made by using shell elements as timber deck and it was modelled as an orthotropic deck. It was a square SLTD simply supported in three corners and a load applied on the fourth corner. The material parameters for the timber were taken from a table for timber quality C24. E_x in C24 timber was equal to 11GPa and according to Ritter´s design guide (1990), the values of E_y and G_{xy} were assumed to be 1.3% and 3% of the E_x value.

The FE model was performed to design the applied load in one of the corners of the deck. In this case a value of the load was taken to check how the deck behaved. Then, the stresses in the deck were analysed to see that the plate did not fail. With an applied load of 3kN the deflection in the loaded corner was around 16 mm. The deflection is slightly more than 10% of the height of the deck which is in good comparison to the thesis of Jónsson and Kruglowa (2009) were a deflection of 27mm was used in a similar model but with a deck thickness of 315mm. The principal stresses in the deck were also controlled, to ensure that the timber will not fail. The maximum stress in the deck was 0.9MPa

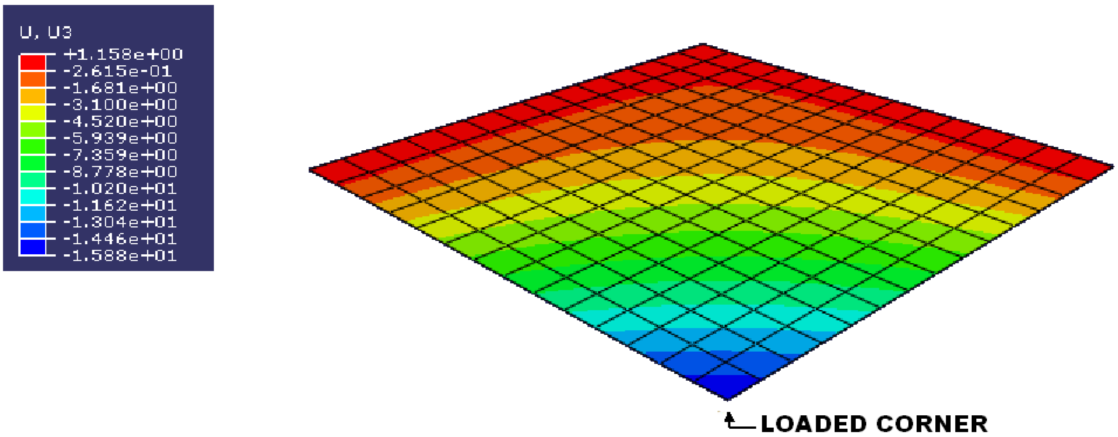


Figure 5.2 Resulting deflection from the FE analysis of the square deck. With a load of 3kN in one corner the deflection was 16 mm in the loaded corner.

5.3 Holes in laminations

According to recommendations from lab staff at Chalmers and studied literature, a decision was made to drill the beams after the first dynamic test. It was assumed that the possibility to obtain different values for mechanical properties before and after drilling was not so probable. This was observed in tests performed previous in a thesis at Chalmers (Jónsson and Kruglowa, 2009). A diameter of 16mm was chosen for the steel bars, and 29mm for the holes in laminations (this value was chosen according to the safety limitation in the code, which suggest a maximum diameter of holes of 20% of deck height). The diameter 29mm was also used for the holes in the hardwood plates used to pre-stress the laminations.

5.3.1 Laminations of 3000mm

Before the assembly of the steel bars, holes had to be drilled in the laminations. The holes of 29mm were located with a spacing of 300mm between each one. The drilling of laminations was performed after the first dynamic test. This decision was based on the assumption that no significant difference between the results in dynamic test before and after drilling the beams should appear. Therefore a dynamic test for the longitudinal MOE after the laminations were drilled, was neglected.

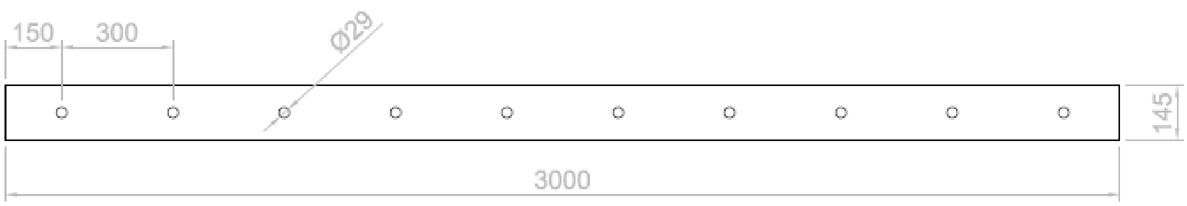


Figure 5.3 The dimension of each lamination was 3000x45x145mm. 10 steel bars were used with a space of 300mm between them.

5.3.2 Laminations of 1350mm

Diameter of the used bars was the same for 1350mm laminations as for 3000mm laminations, as well as the dimensions for the drilled holes. The holes were located with a spacing of 270mm between each one, and the distance to the edges was 135mm as is shown in the figure below. Another distance between the bars, than for 3000mm laminations, was necessary in this case, since the distance from hole to end has to be half of the distance between two holes.

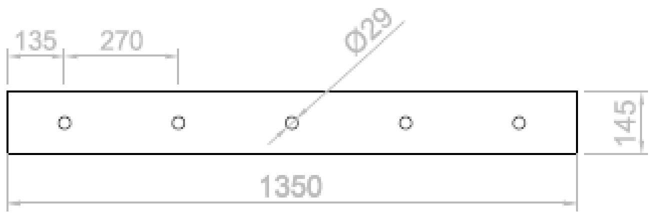


Figure 5.4 The number of steel bars used in this test was 5. The dimension of each lamination was 1350x45x145mm.

Figure 5.5 Laminations of 1350mm length before they were assembled to one deck unit.

5.4 Moisture content in laminations

5.4.1 Laminations of 3000mm length

The control of moisture content in laminations was made after holes had been drilled. The moisture content was measured by using a timber moisture meter. The timber moisture meter had several parts. At one end there were two sharp nails which were penetrated into the laminations. In the other end there was a cable which ended in an instrument where the percentage of moisture content in wood could be obtained by pressing the button corresponding for the type of wood. In this case, timber from Norway spruce was used.

Figure 5.6 The timber moisture meter used to measure the moisture content during lab tests.

To obtain the average value of the moisture content in the deck after assembling, moisture content was measured in four points in ten randomly chosen laminations. It was assumed that the main value of the moisture content was the average moisture content of the random content obtained in each lamination. Moisture content was measured in three points for each lamination, one in the middle and two closer to the ends.

The moisture content was measured in 20% of the total number of laminations and the results from this test with more details are shown in *Appendix B, Table B.3*. The value expected when the wood comes from a factory is between 12% and 16%, according to Jónsson and Kruglowa (2009), and in this case, the value for the moisture content varied from 15% up to 18%. It was a bit higher value than what could be expected with these circumstances, but it was close to the expectations. Moisture content was only a representative value of the moisture in timber used in the test and was not used for further test neither for any calculation. It was measured in order to get a hint about how the real moisture content varied from the expectations. Due to the dry lab environment the moisture content in the timber should decrease and therefore it was measured both before and after testing.

5.4.2 Laminations of 1350mm length

The test of moisture content for the square SLTD was checked using the same equipment used for the laminations of 3000mm length. The measuring procedure was the same as for the earlier test. The moisture content was measured in 6 laminations of a total number of 40. The points measured were in the edge of each lamination of 1350mm, in the middle and one point on the top. It was assumed that the mean value of moisture content was the average moisture of the moisture content in each lamination as well as in the previous measurements. The results obtained from this test with more detail are shown in *Appendix B, Table B.4.* The moisture contest varied from 13% to 17%, and it was reasonable due to the standards and recommendations for timber.

5.5 Experimental determination of longitudinal MOE for each individual lamination

5.5.1 Dimensions and materials

The determination of the longitudinal MOE was tested in laminations with two different sizes. First tests were performed with laminations of 3000mm length, and secondly tests were made for laminations of 1350mm length.

When testing the 3000mm laminations 40 solid pieces were used without any cut or drilled holes. They were numbered from 1 to 40 in the order they were tested to be able to know the properties for each lamination.

The timber used in this case has been explained before and the dimensions of the laminations were 3000x45x145mm. Soft pads were used between the lamination and the support in order to simulate a free-free support.

Secondly, a square deck of 30 laminations were composed with the same wood (C24) used for the SLTD described before. As in previous dynamic test of the 3000mm length, the test after drilling was neglected. The timber deck was composed by 30 laminations numbered from number 41 to 70 following the order started with previous laminations. Each lamination had a dimension of 1350x45x145mm. The detailed drawings of the SLTD are shown in *Appendix A*.

5.5.2 Dynamic test

A dynamic test was performed to measure the longitudinal MOE in each single lamination. The chosen method was explained in *Chapter 4.1.1*. The measurement devices used in the test can be identified in the following list:

- Measurement equipment to check dimensions of each lamination
- 40 laminations of 3000mm (before being drilled)
- 30 laminations of 1350mm (before being drilled)
- Computer program “Picoscope”
- Two supports
- Soft pads on each support to simulate free-free supports
- One hammer
- One microphone (with the support placed in a position with the same height as the measured lamination)

All equipment was available in the laboratory at Chalmers. Two supports were used for the dynamic test of each lamination and soft pads were placed on each support to simulate free-free supports.

For the dynamic test each lamination was placed between the two supports and it was tested by using a hammer to excite it in one end in longitudinal direction. A computer program measured the frequency with a microphone which was placed close to the opposite edge of the lamination and connected with one cable to the computer that measured the frequency of the sound. From this frequency it was possible to get the longitudinal MOE by the computer program Picoscope. At the end of this test, the values were compared with the expected values calculated by using the formulas and theories previously explained. Some differences obtained after the comparison are shown below.
During the test of laminations 1 to 40 there was no problems to measure the frequency with the computer program, however in the case of laminations 41 to 70, the frequency values in the computer varied depending of the force used to hit with the hammer.
The distance between the two supports was not important for the result for the laminations of 3000mm length. It meant that during the dynamic test with 1350mm laminations the importance of the length of each lamination in relation with its thickness was observed.
Results from dynamic test can be found in *Appendix B.1*.

5.6 Assembly of timber deck

For the laminations of 3000mm, ten steel bars from Dywidag were used to pre-stress the deck. The nominal diameter of the steel bars was 16mm and the length of them was 1500mm.

The square timber deck was pre-stressed by using five steel bars separated with a distance of 270mm between them. The steel bars were of the same type in both decks and therefore a lower pre-stressing force had to be applied. The distance between the last bar of the edges and the border of the SLTD was 135mm (the half distance for the edge bars was chosen in order to have a continuous stress distribution in the whole deck). The dimensions for SLTD used in the test were 1350x1350x145mm.

Material used to apply the pre-stress force were hexagonal nuts of 50mm length, steel washers with a hole of 18mm and 35mm of external diameter (for the hexagonal shape). In order to distribute the applied stress in the laminations a plate of hardwood, with a dimension of 160x140x40mm, were placed between washer and lamination.

Some detailed drawings with all dimensions and annotations of the material and the set of nuts and fasteners, can be seen in *Appendix A*.

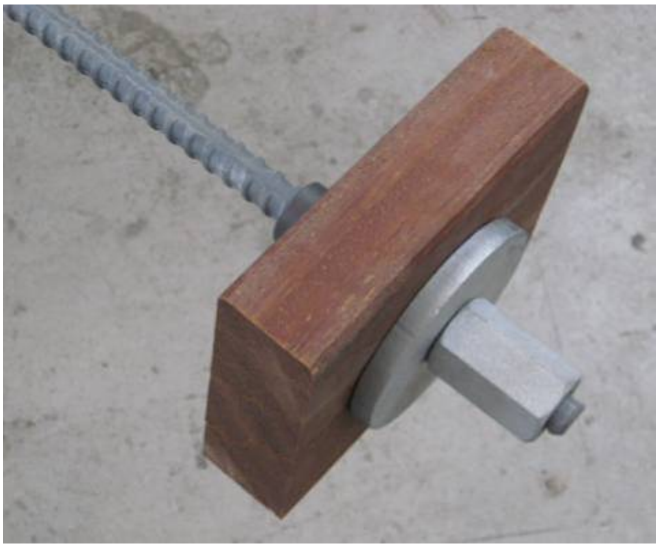


Figure 5.7 Steel bar with diameter of 16mm used to pre-stress the timber decks. The steel bars were from Dywidag, and the pre-stress was applied with a nut.

Furthermore, properties of the steel bars used in lab tests are described in *Appendix C*, including the expected value of tension per each bar. The properties and steel grade of the nuts was the same as for the steel bars used. Once the holes were drilled in the correct position, next step was to assemble the steel bars in the SLTD. This process had to be made carefully in order to get the correct location of the holes. To ensure that the steel bars did not come in contact with the timber, plastic cylinders were used to ensure that the bars were centred in the holes. These pieces were put into the laminations by using a hammer. The timber deck was assembled by using metal fasteners.

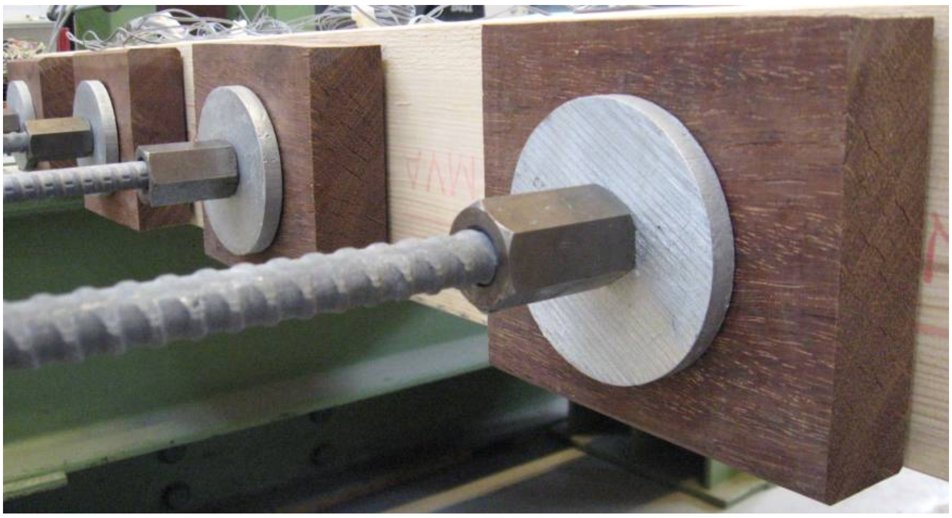
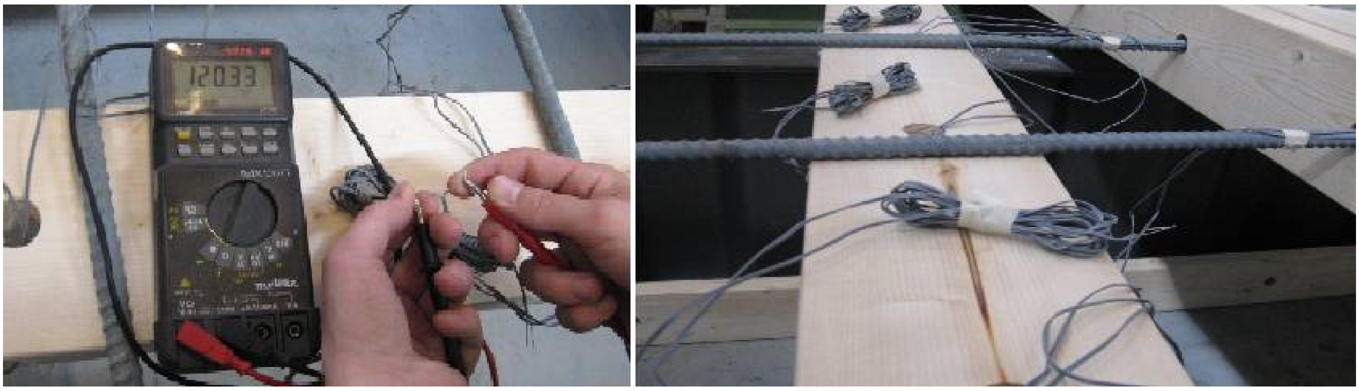


Figure 5.8 Assembled timber deck with steel bar, nut, washer and high density timber plate at the edge lamination.

Before laminations were assembled, the strain gauges were put on the steel bars. Two gauges were used on each bar. This made it possible to measure the pre-stress also in the case if one gauge failed. The connections with each strain gauge were threaded through small holes which were drilled from the top of the laminations to the holes for bars. When two strain gauges were used, it was assumed that the mean value of them corresponds to the real strain in the bar.



a) *b)*

Figure 5.9 Assemble of strain gauges in the timber deck. a) Control of strain gauges before pre-stress was applied in the bars. b) Strain gauges connections to the steel bar before assembly of all laminations.

6 Steps to determine longitudinal modulus of elasticity

A SLTD was constructed without butt joints and with a dimension of 3000x900x145mm, consisting of 20 laminations. In this deck several cuts were made to create three different butt joint patterns. The first one, without butt joints, the second with a pattern of one in eight butt joints (one butt joint in each eight adjacent lamination), and a third one where every fourth adjacent lamination had one butt joint. Discussions with supervisor and literature studies were used to choose the different patterns.

Same laminations were used in the three tests to avoid differences between different tests, because of different material properties of different timber specimens. The 20 laminations selected for the bending tests were selected because of its longitudinal MOE measured during the dynamic tests. A decision was made to have stiffer laminations close to the edges and weaker laminations in the middle of the deck.

During the tests, the decks were placed on supports that were in direct contact with the floor in the laboratory. The supports were made of steel and in order to compensate for initial imperfections in the deck, a fibre board was put on each support to redistribute the support loads. The load was applied from one single jack, and was distributed to two line loads with a load rig made of steel. Between the steel rig and the deck, fibre board shims were also used to compensate for imperfections in the deck.

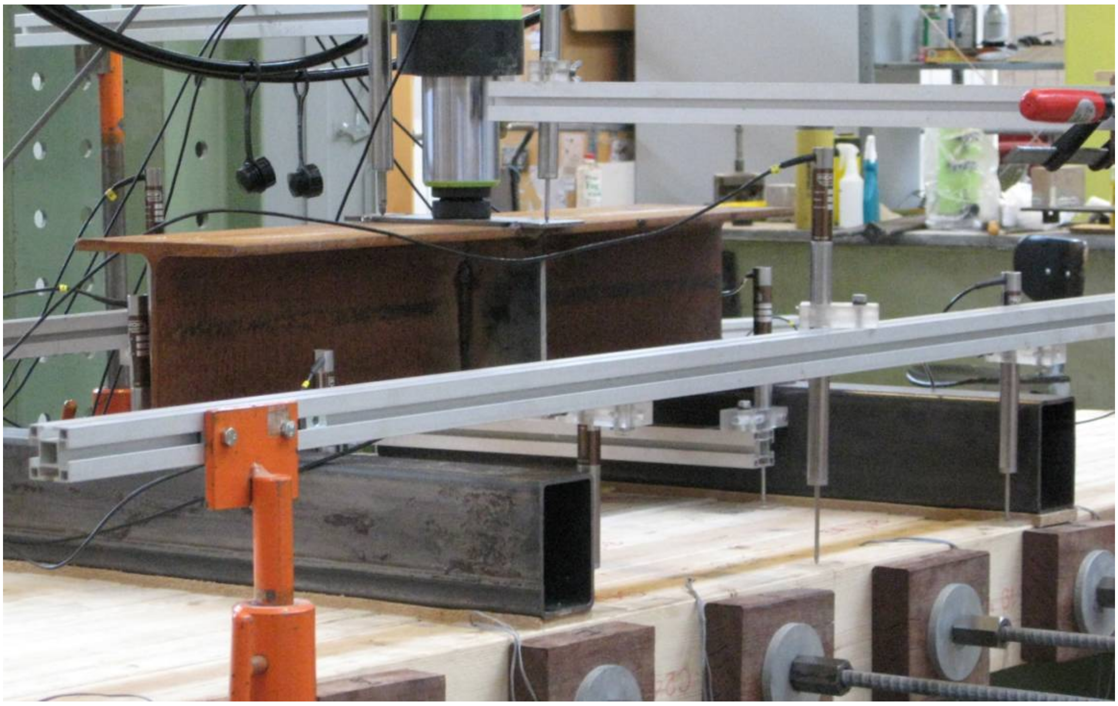


Figure 6.1 Timber deck with load rig and measurement devices during the bending test.

6.1 Equipment used in bending test

The pre-stress was controlled by strain gauges on the steel bars. Two gauges were used on each bar. From the strain in the bars the pre-stress between the laminations were calculated. LVDTs were used to measure the deflection in some points of the deck. Each LVDT was numbered in order to arrange the distribution and measurement of the deflection of the beam. To check which number corresponds with the different LVDT´s, see *Appendix A, Figure A.7*. One load cell was applied on the jack in order to control the applied load. One jack was used in the test and a load rig redistributed the load to two line loads.

Table 6.1 A list of measurement devices used in bending tests both with and without butt joints. For different LVDT position, see Appendix A.

Device name	Name in RAW data	Purpose	Range
		Strain in the steel bars	
Strain gauge, TML	TTG	Measure the strain in the 10 steel bars	1-2%
		Deflection in the deck	
LVDT, LDC 200A	LVDT 1	Measure the deflection in the position number 1	±5mm
LVDT, LDC 200A	LVDT 2	Measure the deflection in the position number 2	±5mm
LVDT, LDC 200A	LVDT 3-6744	Measure the deflection in the position number 3	±5mm
LVDT, LDC 200A	LVDT 4-48727	Measure the deflection in the position number 4	±5mm
LVDT, LDC 1000A	LVDT 5-2878	Measure the deflection in the position number 5	±25mm
LVDT, LDC 1000A	LVDT 6-2882	Measure the deflection in the position number 6	±25mm
LVDT, LDC 500A	LVDT 7-2874	Measure the deflection in the position number 7	±12.5mm
LVDT, LDC 500A	LVDT 8-2873	Measure the deflection in the position number 8	±12.5mm
LVDT, LDC 1000A	LVDT 9-2885	Measure the deflection in the position number 9	±25mm
LVDT, LDC 1000A	LVDT 10-2883	Measure the deflection in the position number 10	±25mm
LVDT, LDC 1000A	LVDT 11-2879	Measure the deflection in the position number 11	±25mm

LVDT, LDC 1000A	LVDT 12-2884	Measure the deflection in the position number 12	±25mm
LVDT, LDC 2000A	LVDT 13-1932	Measure the deflection in the position number 13	±50mm
LVDT, LDC 2000A	LVDT 14-1933	Measure the deflection in the position number 14	±50mm
LVDT,HBM W50TS	DEF 15	Measure the deflection at the loaded plate. Position number 15	±50mm
LVDT, LDC 2000A	LVDT 16-2878	Measure the deflection in the position number 1	±50mm
Load in the middle of the deck			
L & W 1220 AE	Last sensoric - 200kN	Measure the force from the applied load	200kN

6.2 Loading

The load on the deck was applied as presented in *Figure 6.2*. Two line loads were applied symmetrically, and with a spacing of 870mm between load and support. The loads were applied in this way in order to avoid shear deformation in the region of the supports and to get the maximum moments and deflections in the area between the applied loads. This kind of test is usually made at approximately 40% of ultimate load according to the characteristic bending strength. This is a recommendation in EN 408 (2007). After discussion with the supervisor, a decision to apply a higher load was taken (up to about 60%). A hydraulic jack with maximum capacity of 200kN was used to apply the load.

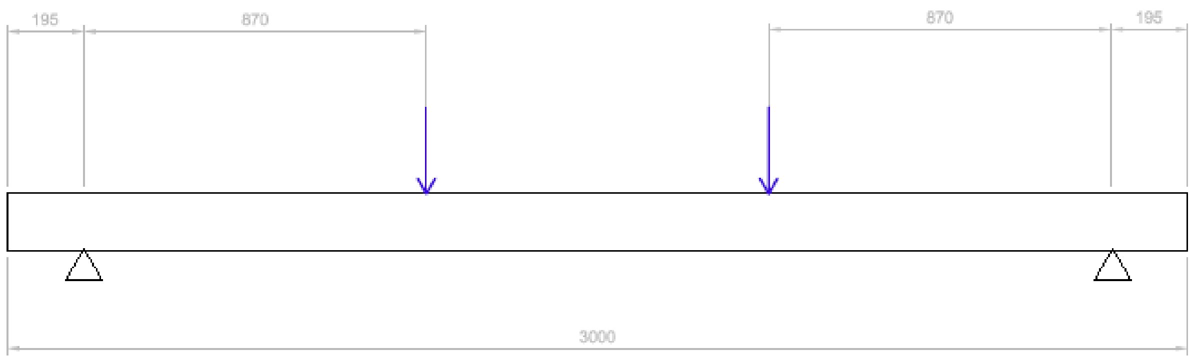


Figure 6.2 Timber deck with two applied loads. The loads are marked with two arrows. The distance between the two loads was 870mm.The view is a section of the SLTD of 3000mm pre-stressed with 10 steel bars.

The load was applied from one single jack. A load rig was used in order to distribute the load to two line loads. A HEA 200 steel beam was used as main beam and two VKR 120X80X5 as spreader beams. *Figure 6.3* shows the deck with the rig applied.

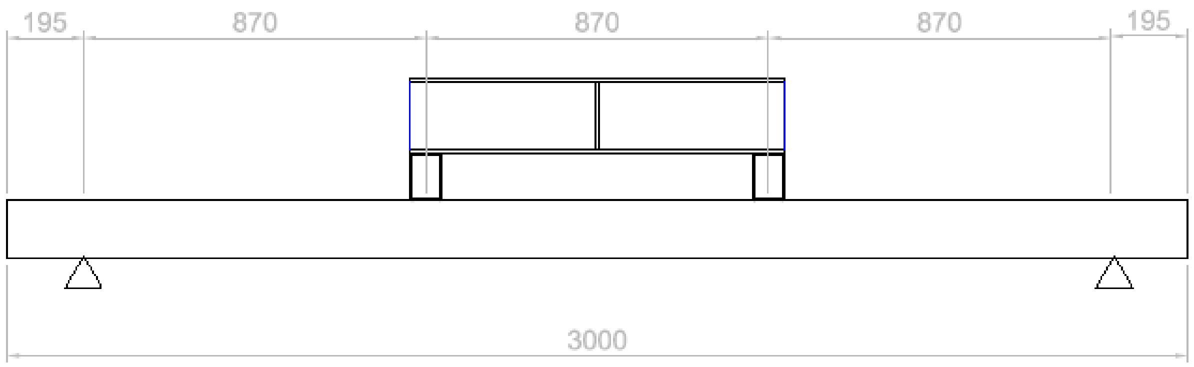


Figure 6.3 Load rig used to apply the load on the SLTD in the tests.

To avoid possible buckling or shear failure in the main beam a transversal stiffener was added in the middle of this beam. Detailed drawings of the load application rig are shown in *Appendix A*.

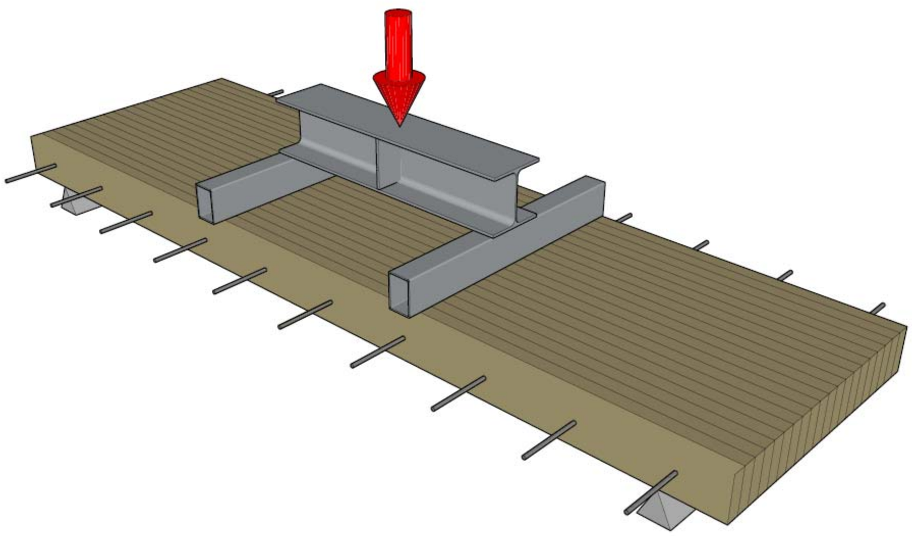


Figure 6.4 Distribution of the concentrated load applied in the middle of the SLTD. The concentrated load is redistributed to two line loads.

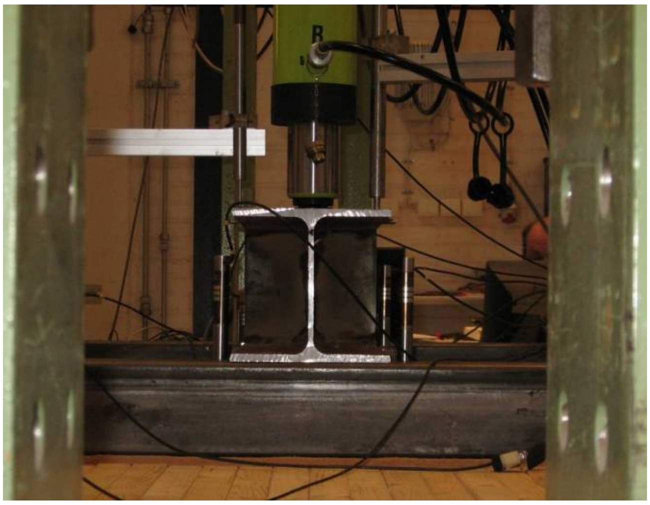


Figure 6.5 Load rig used during lab tests.

6.3 Test of SLTD without butt joints

6.3.1 Dimensions and materials

The dimensions of the deck were 3000x900x145mm and it consisted of 20 laminations. Each lamination had different longitudinal MOE (it was measured previously and known from the dynamic test). The stiffest laminations were placed close to the edges of the deck. The order of the laminations is shown with drawings in *Appendix A*, and values of each MOE appear in *Appendix B*, *Table B.1.* The deck consisted of solid laminations, without butt joints, and ten steel bars were used to apply the pre-stress. Same laminations that composed this deck were later sawn to get two different cases with butt joints, one with butt joints with 1 in 8 recurrence, and the other one with 1 in 4 pattern. Pre-stressing bars from Dywidag were used and distance between the centre of each bar was 300mm, and 150mm between the last bar and the end of the beam.

During the test, the distance between the spreaders beams was measured to 873mm, instead of 870mm from the design, and this was later taken into account during the evaluation of results. The supports were to be applied at 195mm from the edges of the deck, but due to the conditions in the lab this distance was changed to 190.5mm. The supports had a height of 730mm, and a width of 50mm. Positions of supports, load rig, and LVDTs were controlled both before, during and after test runs. Also, the strain gauges were checked several times during the preparation for the bending test.

Three different pre-stress levels were applied in each deck: 300kPa, 600kPa and 900kPa. This test was performed by loading the SLTD in order to understand the butt joints behaviour with different pre-stress levels. The first applied pres-stress level in each test was 900kPa, and it was lowered to 600kPa and then 300kPa between the test runs. Pre-stressing procedure was similar to the procedure described in *Chapter 3.3*. First the bars in the middle of the deck were stressed to half of the desired pre-stress level. All bars were then successively pre-stressed from the middle and toward the ends of the deck. A second pre-stress procedure was then made for all the bars, but now with the full pre-stress level. After this the strains in all bars were controlled and bars with a pre-stress that differed from the desired value were adjusted. This procedure was followed for the pre-stress level of 900kPa. After the test the pre-stress were successively loosened in the bars from the middle and towards the edges. Also here the last check and adjustment of the strain in bars were made.

Figure 6.6 Timber deck used to obtain the longitudinal MOE. Spacing between the supports was measured to 2611mm during the preparation for the bending test.

6.3.2 Summary of test

After the test, the maximum applied load and the maximum deflection for each applied pre-stress level were collected in the following table. For each case, the date of the test when the SLTD was loaded can be identified. However, the evaluation and relation between the different results are interpreted in *Chapter 9* and *10*.

Table 6.2 Summary of the test runs of the deck without butt joints (tests runned 2010-05-26)

Pre-stress levels (kPa)	Applied load//deflection (kN/mm)
910	95.60
630	92.59
300	92.55

For the SLTD without butt joints a loss of pre-stress was observed 20 hours after first pre-stress. This time span was due to preparation of measurement devices for the test. The loss of pre-stress was similar in all bars. Around 30% of pre-stress had been lost in all bars except of bar number 2, which only lost 18% of the first applied pre-stress. The loss of pre-stress was to a great extent caused by creep in timber. Before test, the pre-stress were adjusted to reach the required value for the test.

During the tests, an almost linear behaviour was observed for all three pre-stress levels. The load was decided to be increased up to 100kN. The originally applied load was 76kN. This decision was made by taking into account the total deflection of the deck (including compression of the fibre board at supports). The total deflection was decided to be 15mm (including compression of fibre board at the supports) and this was reached at a level close to 100kN.

6.4 Test of SLTD with a 1 in 8 butt joint pattern

6.4.1 Dimensions and materials

Same laminations as in previous bending test were used, but they were cut to create butt joints in the deck. The total number of laminations were 20 and 7 of them were cut. The laminations were cut with one cut in each eight adjacent lamination (1 in 8 butt joint pattern). Since the moment is at a constant maximum value between the loads it was decided to neglect butt joints outside the area between the loads. Therefore the number of cuts was reduced from 22 in the whole deck to 7 between the loads. The analysed deck and the simplified butt joint distribution used in the tests can be seen in *Figure 6.7* and *6.8* respectively.

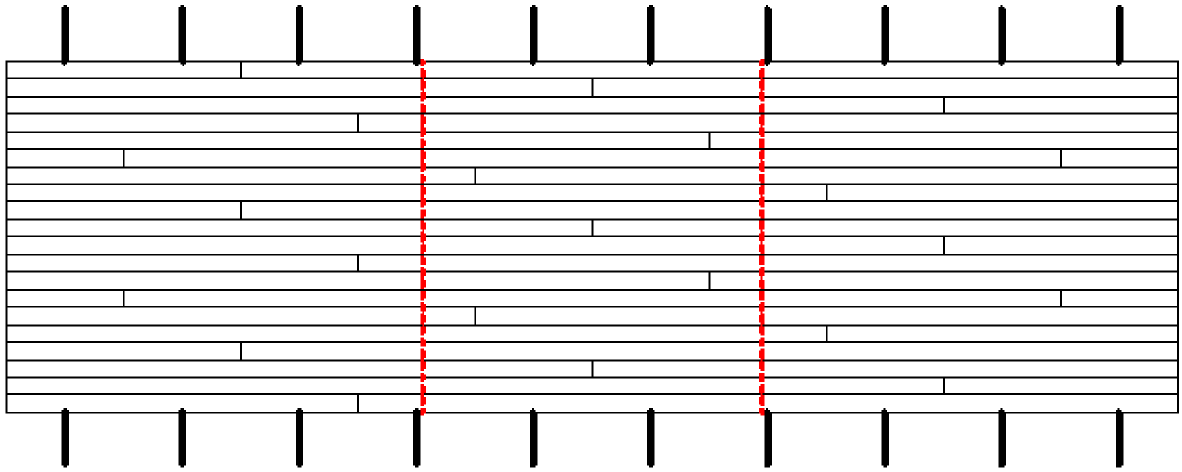


Figure 6.7 Deck with a butt joint pattern of 1 in 8. This means that there is one butt joint in every eight adjacent lamination.

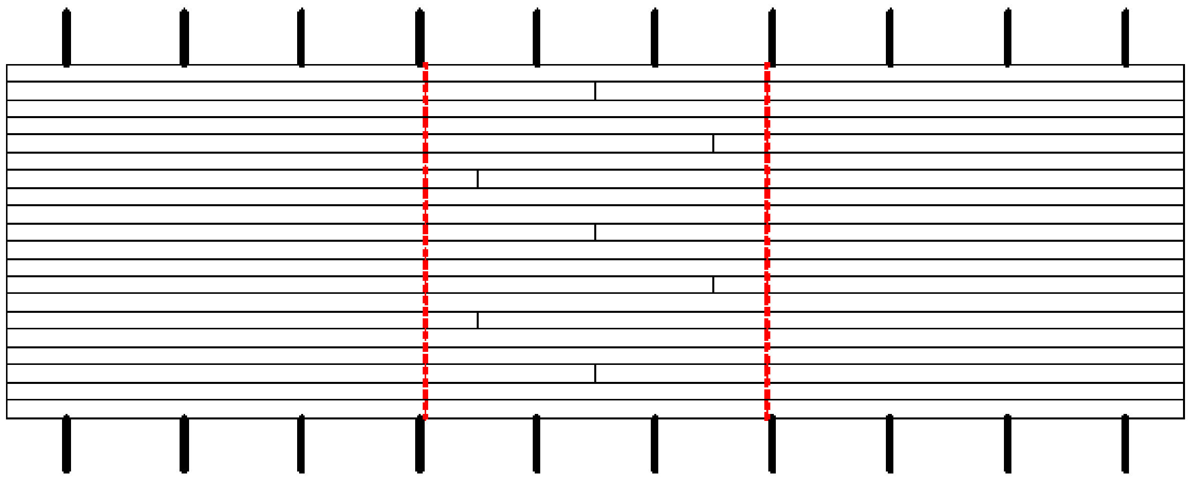


Figure 6.8 Deck with a butt joint pattern of 1 in 8 used in the test. The dotted lines are the load application lines.

After cutting the laminations, the number of laminations with 3000mm of length decreased to 13. There were 6 laminations of 1500mm, 4 of 1800mm and 4 with 1400mm length. The number of steel bars used in this test was the same as used in the previous one, 10 steel bars of Ø16mm.

According to recommendations in Eurocode (CEN, 2004), the distance between butt joints has to be longer than two times the distance from bar to bar and thirty times the thickness of lamination or 1.2m. With these regulations it is not permitted with more than one butt joint in each four adjacent laminations. Due to a scale of 1 in 2 in the tests, the required distance between butt joints could be decrased to the same scale. According to Eurocode this butt joint pattern is not acceptable since the limitation in Eurocode regarding distance between joints is 1200mm. Since the deck is in scale 1 in 2 this requirement could be decreased to 600mm. The pattern in test had to combine the design code limitations and the aim of this thesis. A 1 in 8 pattern with the reduction factor from Ritter´s (1990) and Crews´ standards (1995) was used to reduce stiffness in relation to the number of butt joints. The development length that Crew´s guide (2002) recommended was taken into account when designing the butt joint pattern.

6.4.2 Summary of test

For the SLTD with 1 in 8 butt joint pattern, the relation between maximum load and deflection was checked for each pre-stress level. The summary of the results are shown in *Table 6.3*.

Table 6.3 Summary of test runs of the deck with 1 in 8 pattern (tests runned 2010-05-27)

Pre-stress levels (kPa)	Applied load//deflection (kN/mm)
880	71.95
610	68.53
310	64.26

After bending test for pre-stress level 900kPa a remaining deflection of 2mm was measured. With 600kPa pre-stress level the corresponding value was only 0.6mm. For the third pre-stress level, it was decreased to 0.4mm. This difference is caused by the fibre boards between the deck and supports. During the first test it was compressed and during the two following tests it already had an initial compression. This gave a decreased value of remaining deflection for every test run. The same pieces of fibre board were used for three different pre-stress levels, but when the laminations were cut also the fibre board was changed and this gave a larger deflection in the first run in every case.

6.5 Test of SLTD with a 1 in 4 butt joint pattern

6.5.1 Dimensions and materials

For this test, the same SLTD was tested with the same laminations but with more cuts in order to create a 1 in 4 butt joint pattern. The laminations that were cut are described with more details in the *Appendix A*. Since butt joints between loads and supports do not have any large influence for the expected results, it was decided to make cuts only in the area between the two applied loads. Therefore the number of cuts was decreased from 45 to 15. In this test only eight laminations had to be cut since seven already was cut before previous test. The analysed deck and the simplified butt joint distribution used in the tests can be seen in *Figure 6.9* and *6.10* respectively.

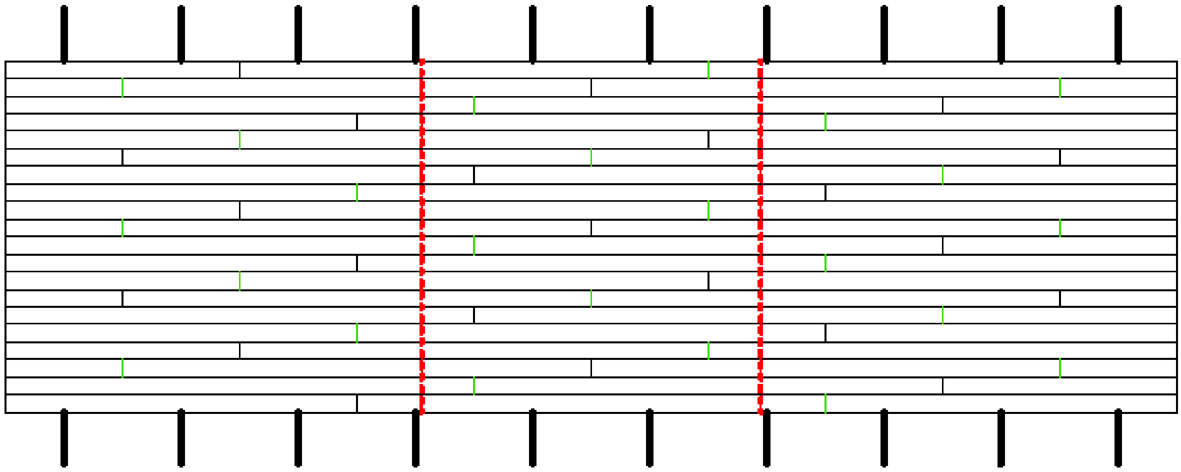


Figure 6.9 Deck with a butt joint pattern of 1 in 4. This means that there is one butt joint in every fourth adjacent lamination.

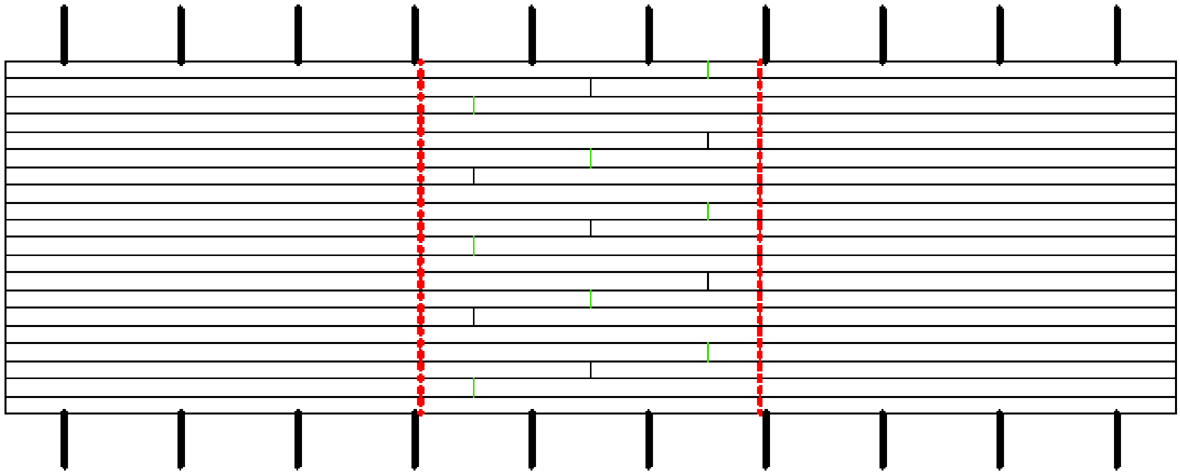


Figure 6.10 Deck with a butt joint pattern of 1 in 4 used in the test. The dotted lines are the load application lines.

Figure 6.11 Detail of the butt joints cut during the lab tests

In both *Figure 6.9* and *6.10* the cuts from the previous test are shown, and the new introduced lines are the second cuts performed to create a 1 in 4 butt joint pattern.

The butt joint pattern was based on Ritter´s recommendations (1990), with a limitation of one butt joint in every four adjacent laminations when there is one cut between each bar. According to Eurocode this butt joint pattern is not acceptable since the limitation in Eurocode 5 (CEN, 2004) regarding distance between joints is 600mm. (since the deck is in scale 1 in 2). In the deck in the test this distance was only 300mm, which is in agreement with Ritter (1990) recommendations.

6.5.2 Summary of test

The results from test of the SLTD with 1 in 4 butt joint pattern with maximum applied load in relation to maximum deflection for the three pre-stress levels are shown in *Table 6.4*.

Table 6.4 Summary of the test runs of the deck with 1 in 4 pattern (tests runned 2010-05-27)

Pre-stress levels (kPa)	Applied load//deflection (kN/mm)
860	74.29
600	65.33
310	54.77

7 Steps to determine in-plane shear modulus

With the results from a pure-twisting test of a square SLTD, the in-plane shear modulus could be calculated. The used materials, pre-stressing procedure and loading of SLTD are described in this Chapter. Tests were made with one solid deck and with one deck with butt joints in order to see the influence in shear due to butt joints.

7.1 Dimensions and materials

A square deck composed by 30 laminations was used in the test. It was the same type of timber, C24, which was used in previous tests. Each lamination had a dimension of 1350x145x45mm. The assembled deck had a size of 1350mm along each side and a thickness of 145mm. Details about the deck are found in *Appendix A, Figure A.22*. These supports were used in an earlier thesis work at Chalmers (*Jónsson and Kruglowa, 2009*) and also dimensions of the deck were similar to the previous tests, but now with a plate thickness of 145mm instead of 315mm. The deck was simply supported in three corners and a vertical load was applied in the fourth corner.

The ball bearing supports was composed by two steel plates and a steel ball, see *Figure 7.1*. A load cell was added to the support to be able to see the reaction force in the corner, see *Figure 7.2*. Under the support there was a Teflon layer to give the support a free lateral movement. The ball bearing supports and steel plates with dimensions are specified in *Appendix A, Figures A.19-A.21*.

Figure 7.1 Ball bearing support used under two corners (called west and east corners).

The support used in the corner opposite to the load were similar to the supports under the deck, but it was put upon the deck, and it also included bolts to adjust the height in order to correct the position of the deck.

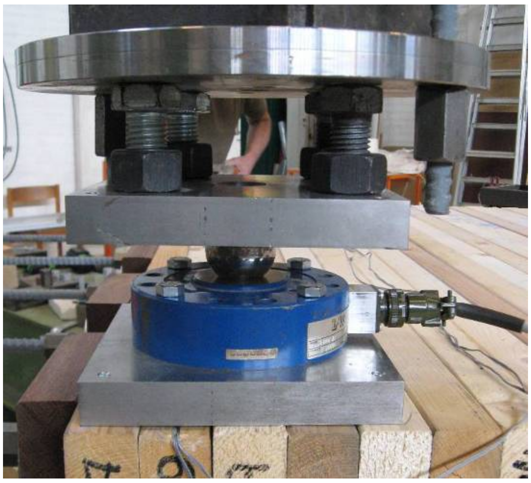


Figure 7.2 *Ball bearing support used in the corner on the opposite side of the loaded corner.*

7.2 Pre-stress procedure

The pre-stress was applied at three different levels in the following order: 300kPa, 600kPa, 900kPa. The lowest pre-stress level of 300kPa was first applied to avoid local compression failure in the timber. Then the pre-stress was increased to 600kPa and 900kPa for the following tests. It was necessary to pre-stress the deck before it was lifted into the test rig, in order to avoid slip between laminations before testing.

Pre-stress was applied by using a hydraulic jack to tension the pre-stressing bar. First bar to pre-stress was the bar in the middle and then the bars were successively pre-stressed toward the both ends of the deck. The pre-stressing procedure was repeated for each bar until expected pre-stress level was reached. Before starting the pre-stress procedure, the strain gauges were connected to the measurement devices.

Figure 7.3 *Hydraulic jack used to pre-stress each steel bar. In the photo are also the steel plates and nuts needed to pre-stress the bars.*

The pre-stress in each bar were calculated from the measured strain in that bar. These bars were pre-stressed several times with an increased force each time. Values obtained from the pre-stress are shown in *Appendix D.* The moisture content in timber was checked at the same time as the pre-stress in bars were applied. The value of measured moisture content was around 13-14%, which is reasonable according to the standards.

During the last pre-stress for 900kPa a gap between some of the laminations could be found. This indicates that they were not fully interacted to each other and the contact surfaces between them were smaller than expected. Therefore the resistance due to shear could be decreased, since friction between laminations could not be fully developed.

7.3 Application of load

The load on the square SLTD was a point load applied in one corner by using a hydraulic jack. The jack was able to produce a maximum force of 200kN and this was more than necessary, since the applied load was not larger than 5kN. The deflection at the loaded point was measured by LVDT's in each corner. In the three supported corners, LVDT's were used in order to control deformations over the supports. As corner support the ball bearing support explained in *Chapter 7.1* were used in order to allow lateral movements and to create free rotation in each corner. The ball bearing support was composed by two steel plates with a steel ball between the plates, and each plate was fixed to the deck with four screws.

The process to control deflections started after the pre-stress procedure. When the deck was stressed and the strain gauges were connected to the computer equipment, the LVDT´s were placed to the timber deck. The LVDT´s were positioned in the middle of the deck and over three of the four corners.

In the loaded corner, two LVDT´s were placed, one over the deck, so close to the applied the load as possible, and another one under the deck in the same position as the centre of the load. Deflection in the corner was measured with LVDT´s of 25mm; since the expected deflection modelled with FE for 3kN was around 16mm. The deflection in the middle of the SLTD was measured in order to be able to derive the transverse shear modulus, however the maximum deflection was expected in the corner where the load was applied. Measurement devices used in the tests are shown in *Table 7.1*.

Table 7.1 Measurement devices for the in plane shear test without butt joints

Device name	Name in RAW data	Purpose	Range
		Strain in the steel bars	
Strain gauge, TML	TTG	Measure the strain in the 10 steel bars	1-2%
		Deflection in the deck	
LVDT,HBM W50TS	LVDT Styrgivare	Measure the deflection at the loaded corner	±25mm
LVDT, LDC 500A	LVDT Mitt 2875	Measure the deflection in the middle of the deck	±5mm
LVDT, LDC 500A	LVDT Väst 2868	Measure the deflection in west corner of the deck	±5mm
LVDT, LDC 500A	LVDT Öster 2872	Measure the deflection in east corner of the deck	±5mm
LVDT, LDC 500A	LVDT Ulast 1935	Measure the deflection under the loaded corner	±12.5mm
		Load in the middle of the deck	
L & W 1220 AE	Last sens norr	Measure the force from the applied load	200kN
Sensoric 100kN	Last väst	Measure the force in the west support	100kN
Sensoric 100kN	Last öster	Measure the force in the east support	100kN
Sensoric 200kN	Last söder	Measure the force in the south support	200kN

It was necessary to pre-stress the deck before it was lifted into the testing rig, in order to avoid slip between laminations before testing.

The load application was deflection controlled and the test was finished when the hydraulic jack had produced a deflection in the loaded corner of one tenth of the deck height that was 145mm. The data from measurement devices was registered every 2 seconds into a logger made by Schulumberger Solatronic, model 35951 A and B. From the computer program the resulting values were available as CSV-files, and these were imported into Excel. The results from the tests are interpreted in *Chapter 9* and *10*.

The results from each pre-stress level are presented in the graphs in *Appendices B.3.1-B.3.3*.

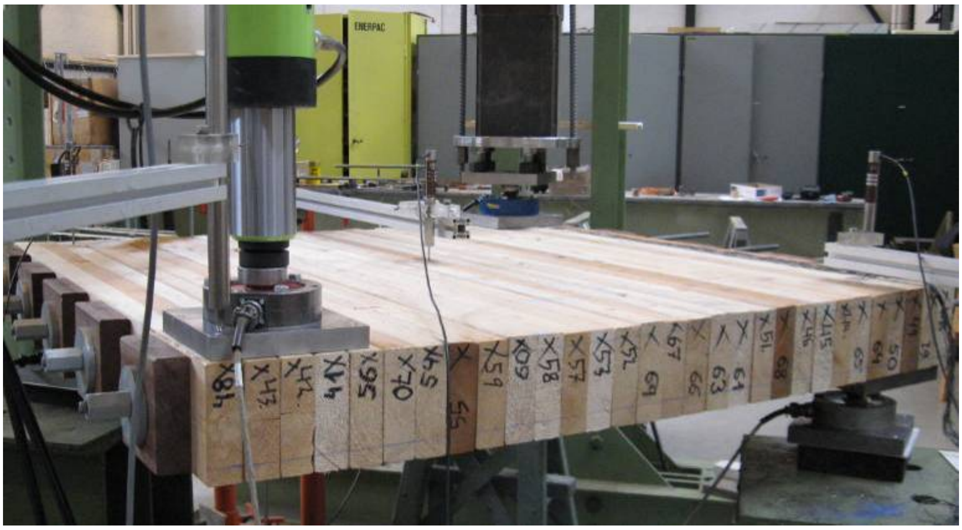


Figure 7.4 Deformed shape of the square SLTD during pure-twisting test

7.3.1 Summary of test

A summary of the test runs for the SLTD without butt joints are shown in *Table 7.2*. The results are shown in *Chapter 8* and evaluated in *Chapter 9*.

Table 7.2 Summary of test runs of square SLTD without butt joints (tests runned 2010-05-16).

Summary of the test runs in the SLTD without butt joints		
Pre-stress levels (kPa)	Maximum load (kN)	Maximum deflection in the middle of the deck (mm)
260	1.93	3.74
570	3.29	4.46
900	4.75	4.81

7.4 Determination of influence of butt joints

7.4.1 Dimensions and materials

Same laminations as in the test without butt joints were used, but now cut to make a 1 in 8 butt joint pattern. In total, 15 cuts were made in 15 of 30 laminations. After those cuts were made, lengths of laminations varied and new laminations had a length of 270mm, 540mm, 810mm and 1080mm respectively. More details about the laminations are described in *Appendices A.24* and *A.25*.

Between the butt joints thin shims of low density fibre board (masonite) were used in order to make a gap in each butt joint and these pieces were taken away before test. The thickness of those pieces was 4mm.

Figure 7.5 Butt joint pattern in the square deck was a 1 in 8 pattern. In this photo thin fibre board plates are used while assembling the deck, in order to make a gap between the two lamination ends.

Same pre-stress devices were used as in the previous shear test. In the test with butt joints the highest pre-stress level was applied first. The geometry of the deck was controlled before and after pre-stressing procedure. Supports, prestressing bars, measurement devices and all equipment for the test were same as used in previous test without butt joints.

7.4.2 Pre-stress procedure

In this case of deck with butt joints, the same three pre-stress levels were applied: 300kPa, 600kPa, 900kPa but in the opposite order than followed before. The reason of this order was based on the possibility of slip between laminations under lower pre-stressed level. Therefore the test started with 900kPa pre-stress level. For the first pre-stressed level, the steel bars were pre-stressed several times with an increased force for each time. In the second and third level of pre-stress, each bar was unstressed to achieve lower values. The data collected into the log in the computer program are presented in *Appendix C.*

7.4.3 Application of load

Same loading procedure that was explained in *Chapter 7.3* was followed for this deck. The loading of the square timber deck was by applying the force in one corner using a hydraulic jack while the deflection in that point was measured by one LVDT connected to a computer.

It was necessary to change one of the LVDTs in this test, because of the deflections in the loaded corner were supposed to pass 12.5mm. A LVDT with ±25mm was chosen in this case instead of LDC 500A with range ±12.5mm, which were used in the test without butt joints. All the measurement devices are presented in *Table 7.3*.

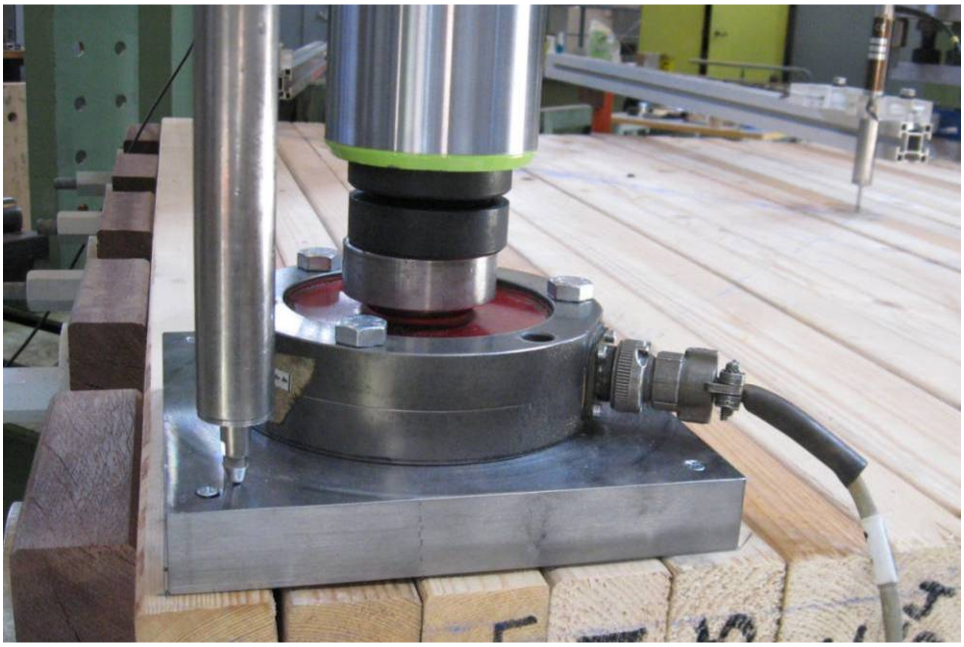


Figure 7.6 Detail of loaded corner with LVDT, HBM W50TS on the steel plate close to the hydraulic jack that applied the load.

Table 7.3 Measurement devices for the second in plane shear test for deck with butt joints.

Device name	Name in RAW data	Purpose	Range
Strain in the steel bars			
Strain gauge, TML	TTG	Measure the strain in the 10 steel bars	1-2%
Deflection in the deck			
LVDT,HBM W50TS	LVDT Styrgivare	Measure the deflection at the loaded corner	±25mm
LVDT, LDC 500A	LVDT Mitt 2875	Measure the deflection in the middle of the deck	±5mm
LVDT, LDC 500A	LVDT Väst 2868	Measure the deflection in west corner of the deck	±5mm
LVDT, LDC 500A	LVDT Öster 2872	Measure the deflection in east corner of the deck	±5mm
LVDT, LDC 500A	LVDT Ulast 1935	Measure the deflection under the loaded corner	±25mm
Load in the middle of the deck			
L & W 1220 AE	Last sens norr	Measure the force from the applied load	200kN
Sensoric 100kN	Last väst	Measure the force in the west support	100kN
Sensoric 100kN	Last öster	Measure the force in the east support	100kN
Sensoric 200kN	Last söder	Measure the force in the south support	200kN

7.4.4 Summary of test

As well as in SLTD without any butt joints, a summary of obtained results during the test is shown in *Table 7.4*. The pre-stress process in this case started with the highest value. Maximum deflection in the table refers to the deflection obtained for the value of maximum applied load.

Table 7.4 Summary of the test runs of deck with butt joints (tests runned 2010-05-20)

Pre-stress levels (kPa)	Maximum load (kN)	Maximum deflection in the middle of the deck (mm)
920	4.53	3.94
640	4.13	3.28
320	4.92	2.18

8 Results from laboratory tests

In this Chapter, the results from lab tests are presented without any additional comments. Two different tests were made, one bending test to see the longitudinal modulus of elasticity (MOE) and one torsion test to find the shear modulus. The tests were made both with and without butt joints in the decks. A dynamic test of the laminations was also made to find the individual longitudinal MOE for each lamination.

In the bending test, applied load and deflection between applied loads were information used to find the longitudinal MOE. In shear test, applied load and deflection in middle of the square deck were used to find the shear modulus. Other calculations and test results are strain in bars, pre-stress levels and reaction forces which also were measured and controlled to ensure reasonable results.

The results from tests are presented in tables and graphs in this chapter. From dynamic test the resulting MOE are presented for the laminations. For the bending and shear test the resulting deflection in the decks are presented. These values are used in *Chapter 9* to find the mechanical properties for the decks.

8.1 Results from dynamic test of longitudinal MOE

Here is a brief summary of the results from dynamic test presented. Tables and more information about the results for each lamination can be found in *Appendix B*. A description of the test procedure is found in *Chapter 4.1*.

Two different lamination types were used in the test, 40 laminations of length 3000mm and 30 laminations of length 1350mm. These laminations were later assembled to two timber decks of different lengths. The longitudinal MOE for laminations of 3000mm length varied within a span of 8000-15000MPa of MOE. The mean value from all those laminations was 12269MPa. The characteristic value of C24 timber is 11000MPa, and the result from the test were higher than the characteristic value. The resulting longitudinal MOE for laminations of 1350mm length was within same range as for the longer laminations (3000mm). The mean value was 11880MPa which was a bit lower than for the longer laminations, but still larger than the characteristic value (11000MPa). Variations in results were due to varying properties of different specimens. Since timber is a natural material the variations between specimens could be large, which can be seen in the table of results in *Appendix B*.

8.2 Results from bending test of longitudinal MOE

Three decks were used in the bending test of longitudinal MOE, one solid deck without butt joints, one with butt joint pattern 1 in 8 and one with butt joint patter 1 in 4. Three different pre-stress levels (900kPa, 600kPa and 300kPa) were used in all decks. Same materials were used in all decks, but the laminations were cut to shorter parts when butt joints were introduced.

Deflections of the decks were measured between loads and over supports. Since the load rig occupied the centre of deck between the loads, only the deflections along the edges were used in the analysis of results. But the deflection was also measured on the load rig in order to see how the deck and rig behaved in transversal direction.

In analysis of results only deflections between loads were used. The deflection in middle of the span was measured. The resulting deflection is here presented as the difference between the deflection in the middle of span and deflection close to the loads. By using these values, the deflection over supports can be neglected from analysis. This difference is later used to calculate the longitudinal MOE of the deck.

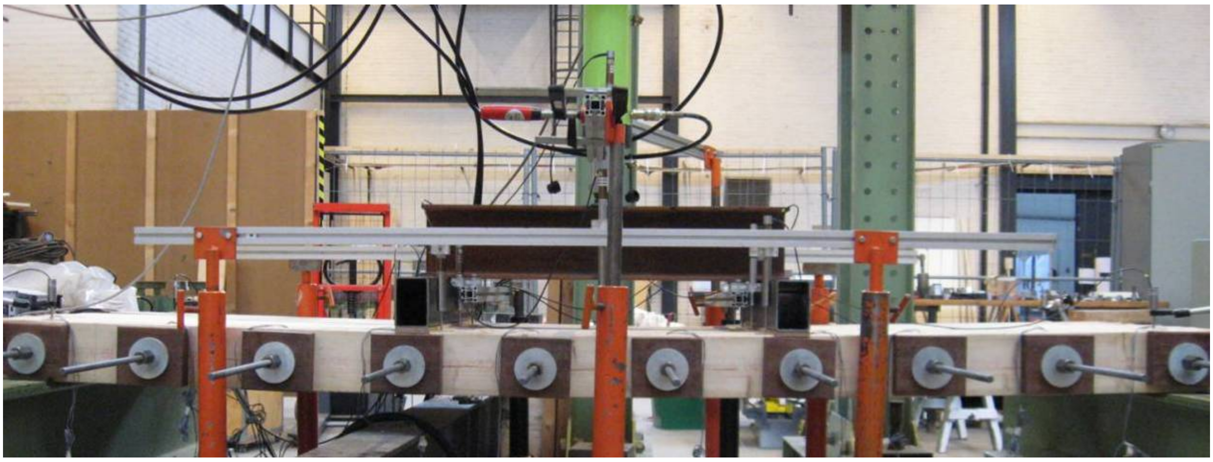


Figure 8.1 SLTD during bending test. Deflection was measured between the loads.

The relation between deflection and load for each deck with the three pre-stress levels are shows in following graphs. When looking on the trends of the curves, differences between behaviour of the deck with highest and medium pre-stress levels (900kPa and 600kPa) was not so pronounced, but the responses of the decks were more different with the medium and the lowest pre-stress levels.

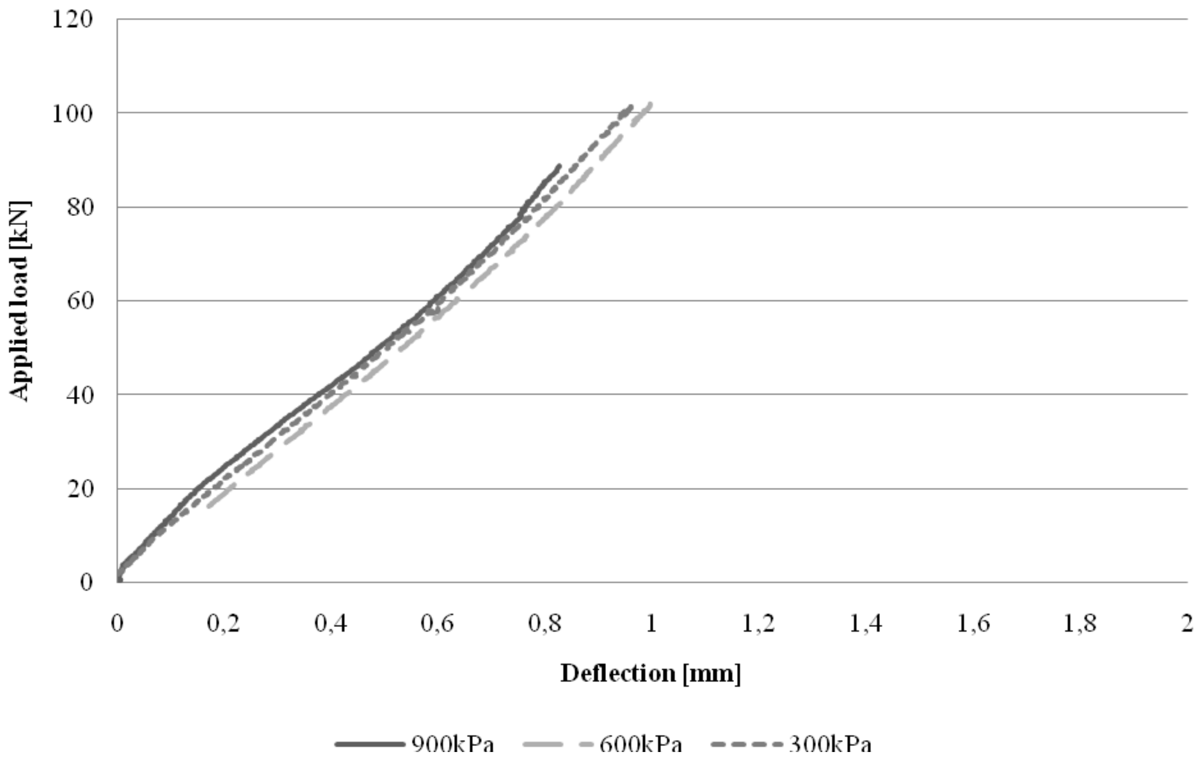


Figure 8.2 Deflection in the middle of the deck without butt joints for the three pre-stress levels. The different curves show different pre-stressing values, the value ranged from 300kPa to 900kPa between the timber laminations.

Applied load was plotted in relation to deflection. The slope of the three curves in *Figure 8.2* was almost the same during the whole loading procedure, and this table present results for the deck without butt joints. With 600kPa pre-stress level there was an error in one LVDT and it could not control the response in the curve up to 16kN. Anyway, the same trend could be identified for the three pre-stress levels, almost parallels during the complete bending test. The first test (900kPa) was loaded with a total load of 90kN. In the following tests it was decided to increase maximum load to 100kN in order to see a greater range of the deck behaviour. The curves indicates a bit stiffer deck at high load levels. This is not reasonable according to the expected behaviour of a loaded deck, but is probably caused by deformation of the supports.

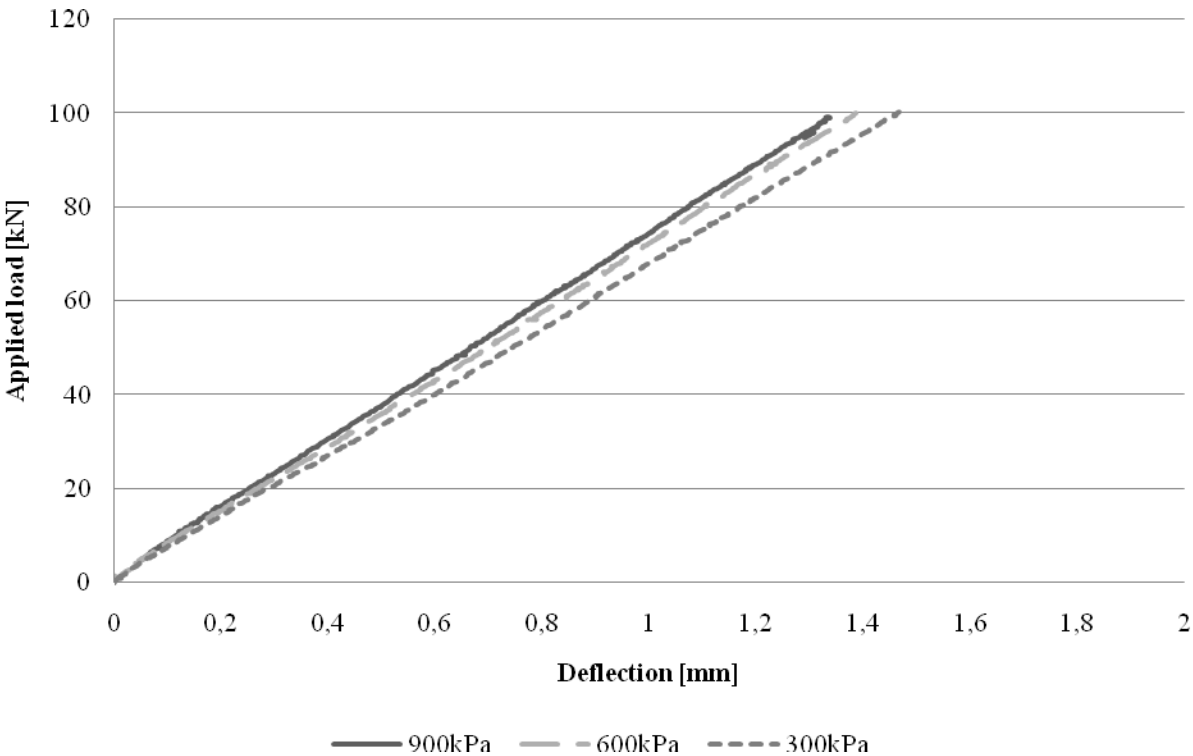


Figure 8.3 Deflection in the middle of the deck with 1 in 8 butt joints for the three pre-stress levels, varying between 300kPa and 900kPa.

In *Figure 8.3* a similar response is presented for the three pre-stress levels applied in the deck with 1 in 8 butt joint pattern. Under low values of applied loads, the decks behaved almost equal, having a very small difference of deflection between the different cases. Comparing with the deck without butt joints, the lines were not parallels in this case, the space between them increased with an increased applied load. It seemed that for 300kPa level, the difference with the 600kPa line was higher than the space between 600kPa and 900kPa. Maximum applied load in all three cases was 100kN.

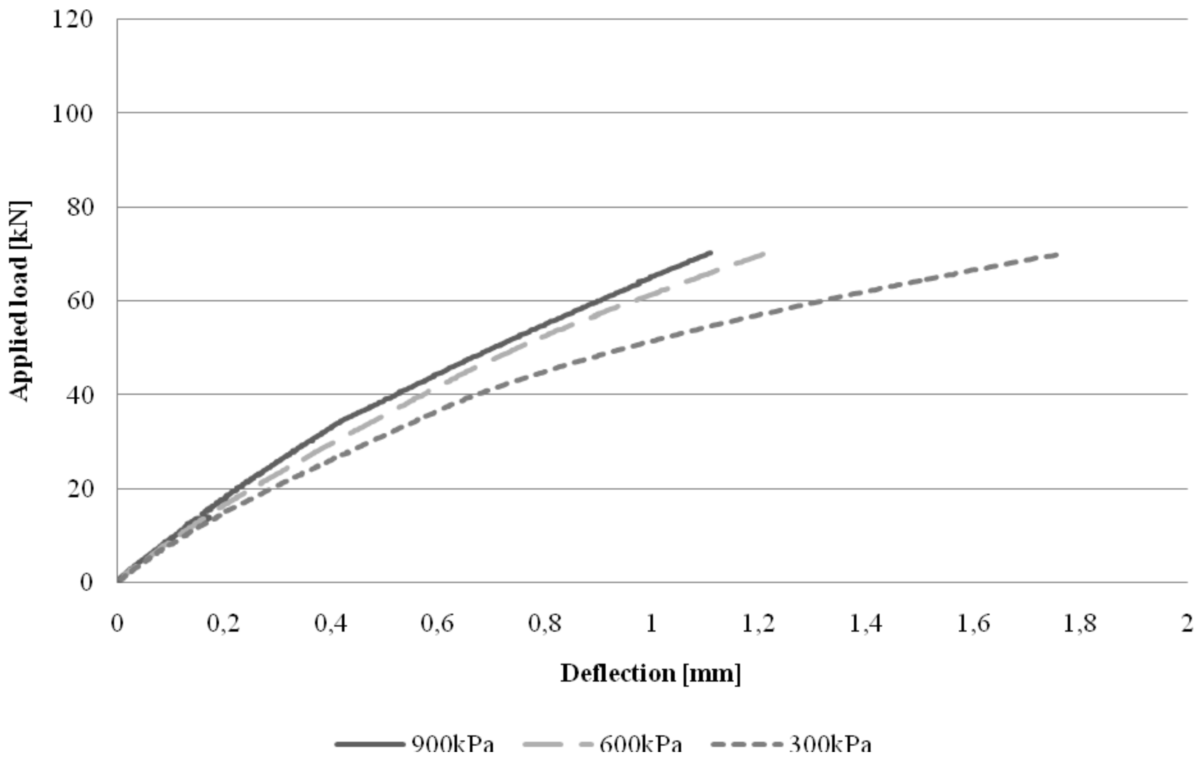


Figure 8.4 Deflection in the middle of the deck with 1 in 4 butt joints for the three pre-stress levels, varying between 300kPa and 900kPa.

Results from the deck with a butt joint pattern of 1 in 4 are presented in *Figure 8.4*. As well as for the other decks, with low values of applied loads, the response of the SLTD with 1 in 4 butt joints behaved almost equal for three pre-stress levels. But with an increasing load the difference with 1 in 4 butt joint pattern were obvious. The difference between 300kPa and 600kPa in the relation between applied load and deflection was much higher than for the other decks. The difference is also larger when compared to the relation between 600kPa and 900kPa. This indicates that when the number of butt joints increase and the pre-stress level decrease, the influence of butt joints should be more significant. In general, the longitudinal MOE value was higher for a deck without butt joints because of the relation between applied load and deflection was higher at the three pre-stress levels. Since the deck have lower maximum allowed load when butt joints is introduced it was decided to decrease the applied load on the deck with 1 in 4 butt joint pattern. The deck also showed a non-linear behaviour in this case, so the maximum load was decided to not pass 70kN.

In *Figure 8.5,* the relation between applied load and deflection are plotted with a 900kPa pre-stress level. These results were presented earlier, but here they are presented together in order to see the difference between the three decks. The different curves show the different decks: without butt joints, with 1 in 8 and 1 in 4 butt joint patterns. In the graphs in *Figure 8.6* and *8.7* the relation between applied load and deflection with both 600kPa and 300kPa pre-stress levels are presented respectively.

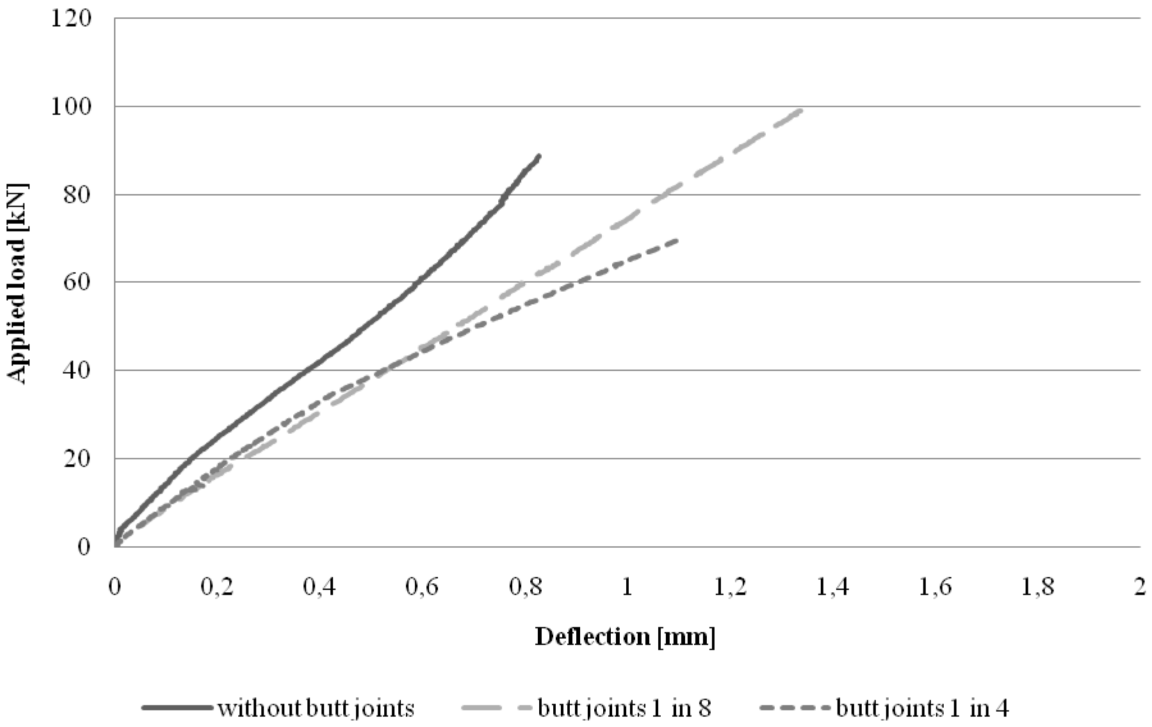


Figure 8.5 Relationship between applied load and deflection between loads for bending tests with a pre-stress level of 900kPa. Values for deck both without butt joints and two different butt joint patterns are plotted.

In the case with 900kPa pre-stress level the deck without butt joints has a significantly higher stiffness. The two curves with butt joints behave in a similar way up to 40kN load. After 40kN the deck with more butt joints has a more weak response and the curve changes direction. Before 40kN the deck with 1 in 4 butt joint pattern seems to have a more stiff behaviour than the deck with 1 in 8 butt joint pattern. This is not reasonable, but the difference is relatively small and probably caused by an inaccurate measurement procedure or a difference in pre-stress level. However, the curve for the deck without butt joints show a stiffer behaviour, which indicates that there is an influence of butt joints also with relatively small applied loads.

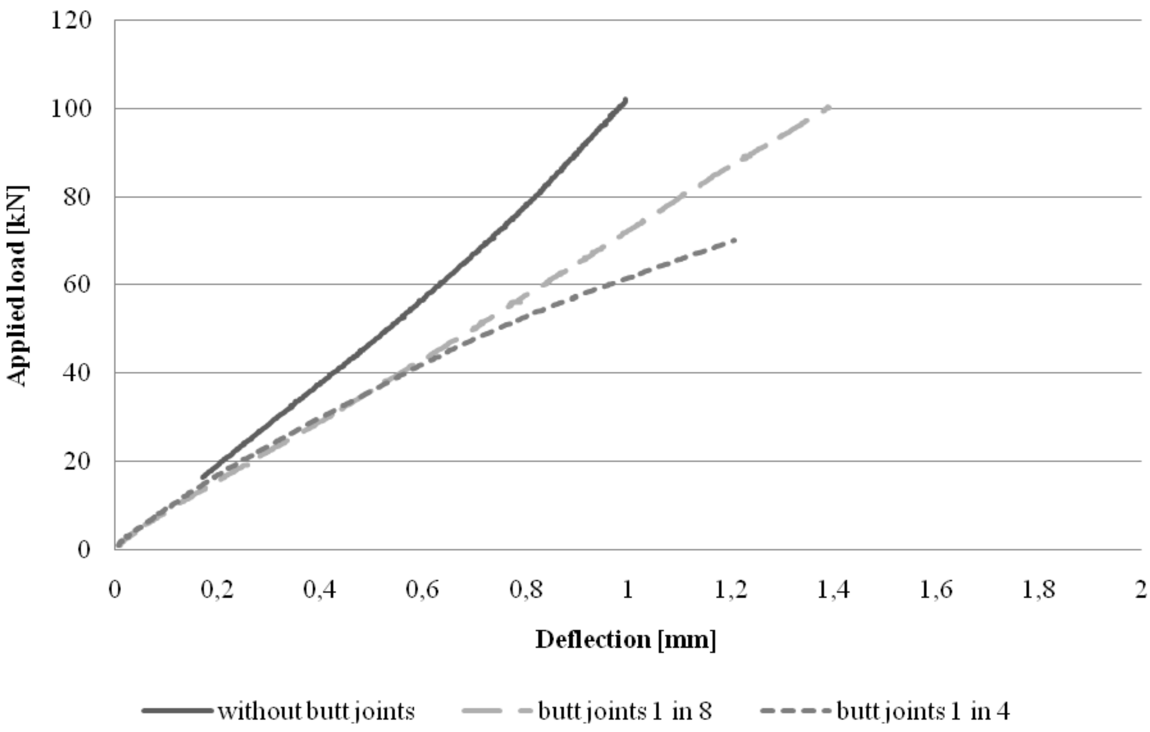


Figure 8.6 Relationship between applied load and deflection between loads for bending tests with a pre-stress level of 600kPa. Values for deck both without butt joints and two different butt joint patterns are plotted.

For the case with 600kPa pre-stress level the trend from the 900kPa measurements remains. Deflection of the two decks with butt joints is a bit larger but the deck without butt joints has almost the same behaviour as with a higher pre-stress level. In this case the curve for the deck without butt joints is more close to the other two curves at a low load level. During the test without butt joints one LVDT failed and there are no values of deflection at lower load than 16kN.

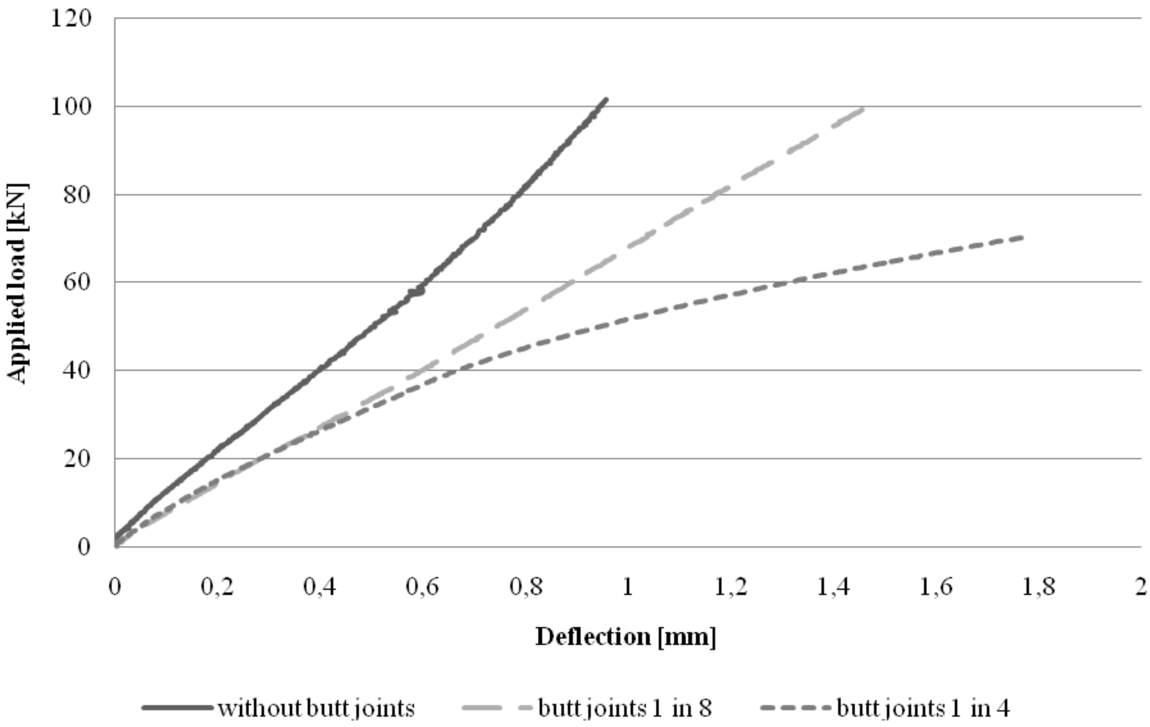


Figure 8.7 Relationship between applied load and deflection between loads for bending tests with a pre-stress level of 300kPa. Values for deck both without butt joints and two different butt joint patterns are plotted.

In the case with 300kPa pre-stress level, the influence of butt joints is more obvious than at the two higher pre-stress levels. Especially the deck with 1 in 4 butt joint pattern has a more significant deflection due to the load.

8.3 Results from torsion test of in-plane shear modulus

Two decks were used in the torsion test of a square deck, one solid deck without butt joints and one deck with butt joints. Three different pre-stress levels (900kPa, 600kPa and 300kPa) were applied in both decks. Same materials were used in the decks, but the laminations were cut to shorter pieces when butt joints were introduced.

Deflections of the decks were measured at supports, close to the load and in the middle of the deck. In the analysis of results the deflection in middle of the deck were used. This deflection was later used to calculate the transversal shear modulus of the deck.

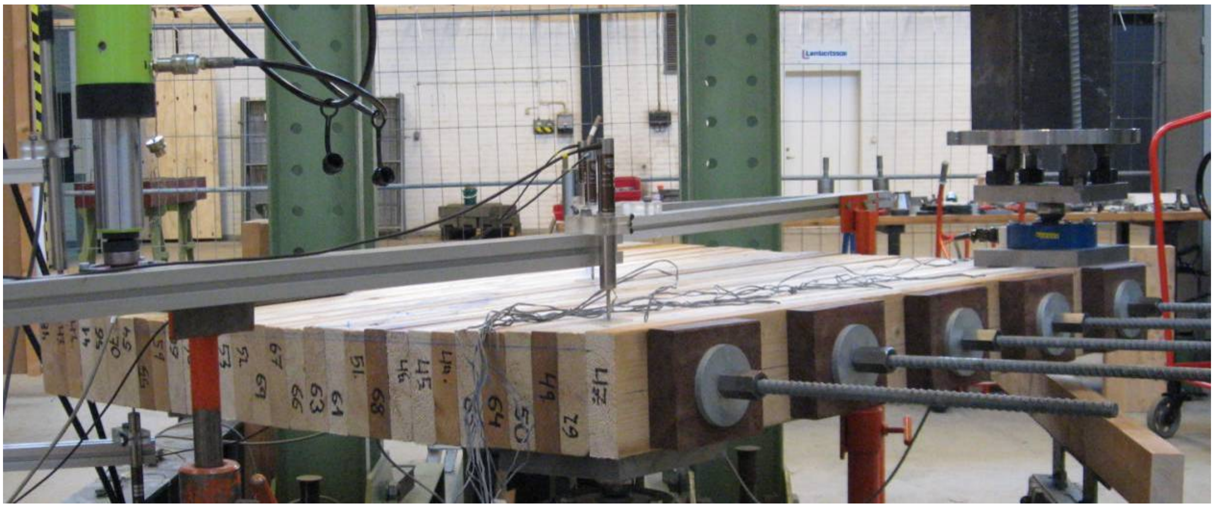


Figure 8.8 SLTD during pure-twisting test.

The relations of each SLTD, without and with butt joints, with the three different pre-stress levels are presented in *Figures 8.9 and 8.10*. The applied load is plotted in relation to the deflection.

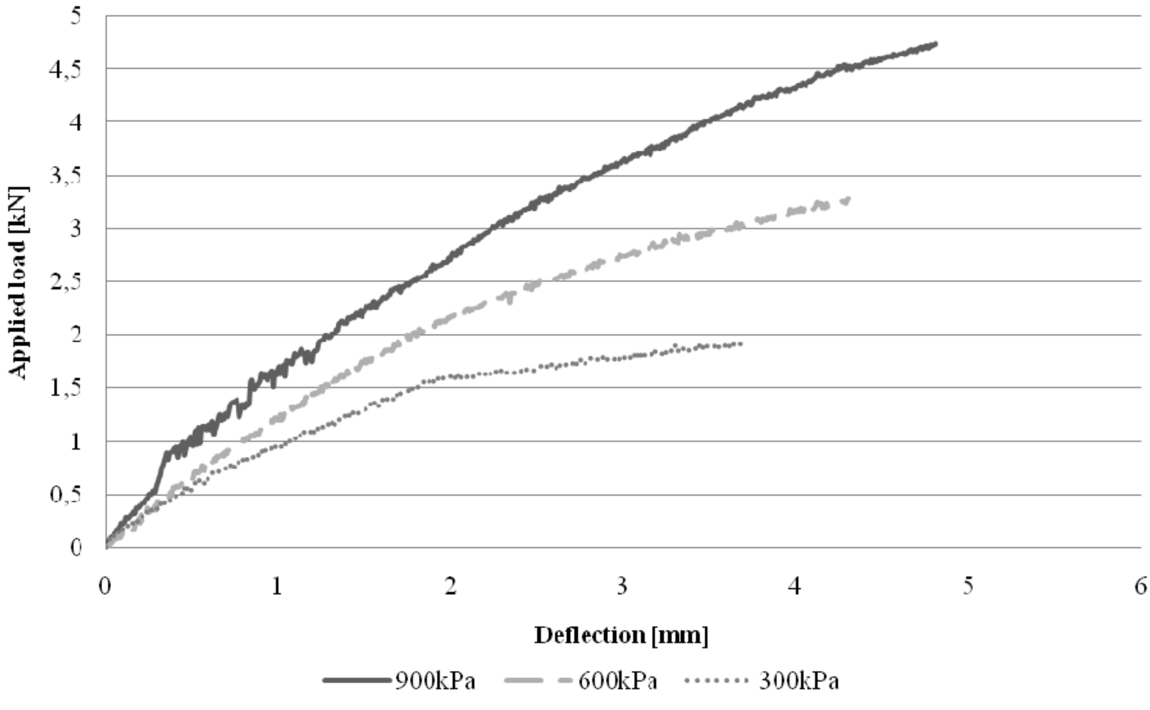


Figure 8.9 Relationship between applied load and deflection in the middle of deck without butt joints.

A lower prestress value gave a larger deflection in relation to the applied load. With the lowest prestress level of 300kPa the curve got a significant break with a higher deflection from the load after this break. The first part of the curve was probably due to deflection in timber, and the break of the curve was probably due to slip between laminations. This slip was related to friction between them. With a higher prestress level the surface between laminations were not so prone to slip, and therefore the curves of the two higher pre-stress levels were more straight.

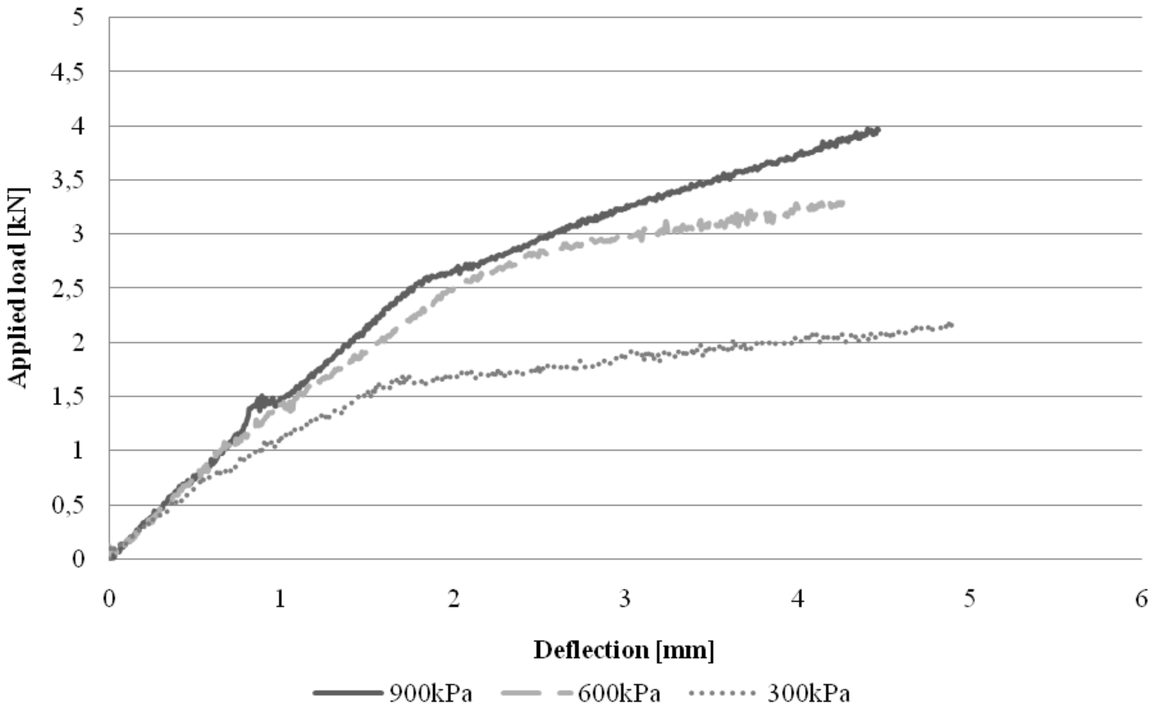


Figure 8.10 Deflection in the middle of the plate for the deck with butt joints. The different curves show different pre-stressing values, the value range from 300kPa to 900kPa between the timber laminations.

In *Figure 8.10,* the applied load is plotted in relation to deflection for the deck with butt joints. The behaviour with pre-stress level 300kPa seemed to be the same as for the deck without butt joints. With the two higher pre-stress levels there was a significant break on both curves, similar to the one for the 300kPa curve. With pre-stress level 600kPa and 900kPa, the load when the slip first occurred was a bit higher than for the lower pre-stress value. All three curves were similar in the beginning at low load levels, and the lowest pre-stress level deviated from the other two curves relatively early during loading.

Figures 8.11 , 8.12 and *8.13* show the relation between a deck without and with butt joints, with the plot of three different shapes depending on the pre-stress levels. For medium and lower pre-stress level, the end of the curves seemed to coincide at higher level. This means that for the highest applied loads, the deck with and without butt joints seems to be acting in a similar way. However, the response of the deck with the highest pre-stress level was completely different. While the two curves seemed to act with a similar behaviour while the load was increasing, the response of the two SLTDs was different. The deck without butt joints leaded to a higher applied load producing almost the same deflection as the deck with butt joints.

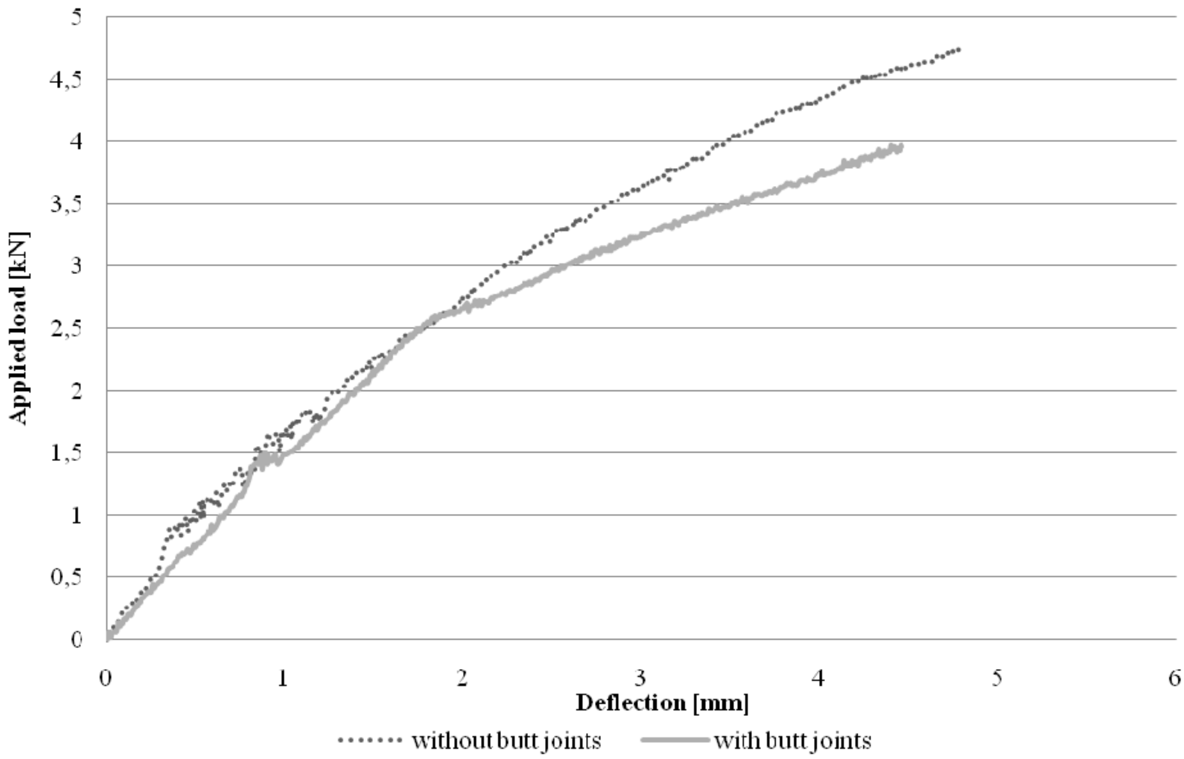


Figure 8.11 Relationship between applied load and deflection for the torsion tests with a pre-stress level of 900kPa. Values for both the deck with and the deck without butt joints are plotted.

For the highest pre-stress level, the two curves were similar up to a certain value where the butt jointed deck started to increase the deflection. This increase was caused by decreased friction between laminations, when slip occurred between them.

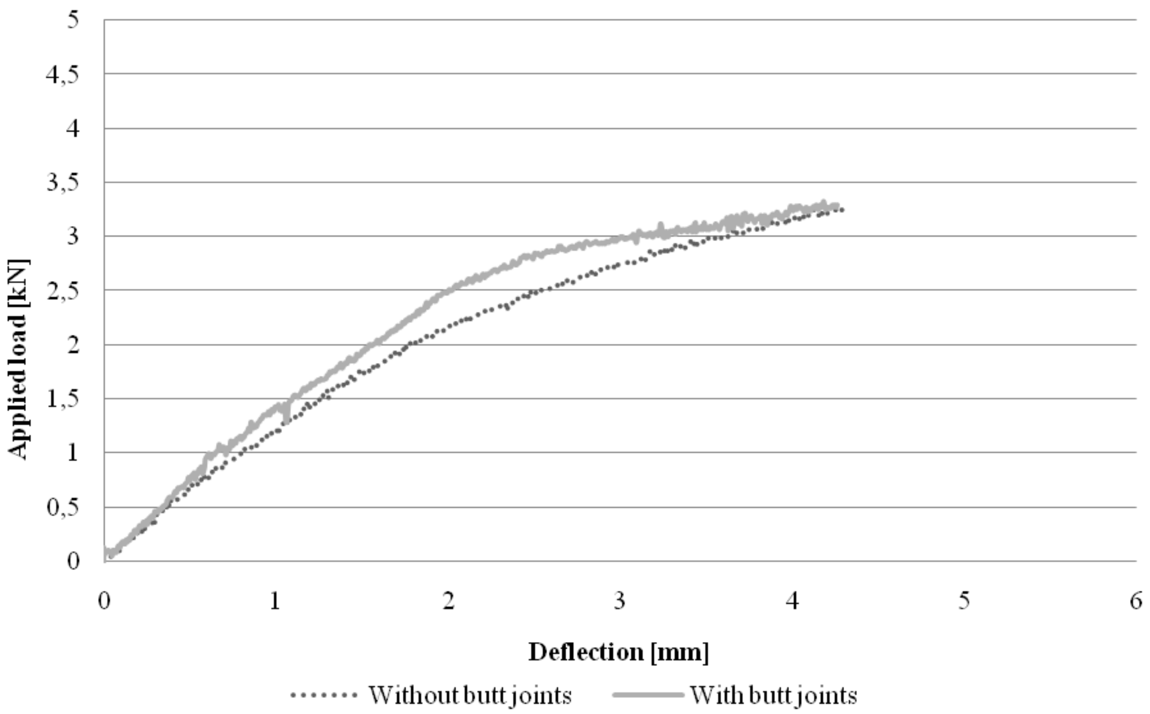


Figure 8.12 Relationship between applied load and deflection for the torsion tests with a pre-stress level of 600 kPa. Values for both the deck with and the deck without butt joints are plotted.

Figure 8.12 shows deflection in the middle of the SLTD with a 600kPa pre-stress level, both without and with butt joints. The two curves were similar to each other but with a more significant break on the curve with butt joints. One observation that could be made about these curves was that the deck with butt joints had a higher stiffness during load application. This was not expected and could be an error due to wrong pre-stress level in one of the tests. The pre-stress level was difficult to measure in a correct way during the tests, and this could be a reason for the difference.

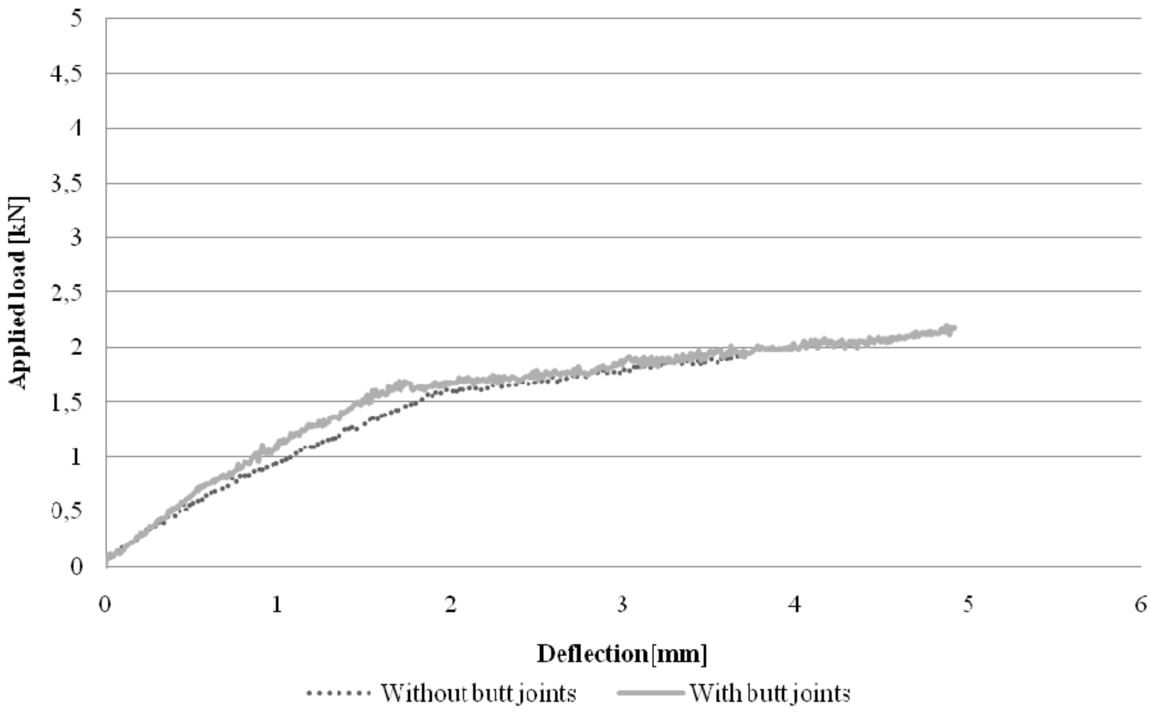


Figure 8.13 Relationship between applied load and deflection for the torsion tests with a pre-stress level of 300kPa. Values for both the deck with and the deck without butt joints are plotted. The behaviour of both deck is similar.

For the lowest pre-stress level, the two curves were similar along the whole loading procedure. There was a small difference between the two curves but it was enough small to assume them as equal. The result indicates that with a low pre-stress value the influence of butt joints seemed to be very small.

9 Analyses of test results

In this Chapter the results from bending and shear tests are analyzed. The aim of this thesis was to investigate the influence of butt joints in SLTD used as bridge decks.

Three decks with different butt joint patterns were used in the bending test. Every deck was subjected to three different pre-stress levels. From the results of applied load and resulting deflection the longitudinal MOE was obtained in nine different cases. The longitudinal MOE was calculated from the inclination of the load deflection curve.

Two square decks were tested to check the influence of butt joints for the in plane shear modulus. From these tests the data shown in *Appendix B.4* were obtained. G_{xy} were found from these results by using the formulas explained in previous chapters. Three decks were tested to check the influence of butt joints in bending. This test was made with different butt joint patterns: without butt joints, with 1 in 8 and with 1 in 4. A summary of results from the analysis is shown in this Chapter.

9.1 Derivation of longitudinal MOE

According to the results in *Figures 8.2- 8.7* there seems to be an influence of butt joints on the longitudinal stiffness of a stress laminated timber deck. In the case without butt joints the pre-stress seems to have no or very small influence on the results. With a low pre-stress value the number of butt joints seems to have a higher influence than for higher pre-stress levels. With 300kPa pre-stress level the curves have a clearly non-linear behaviour. Probably this behaviour is because of slip between the laminations in this case.

The densities were identified by using the measured values from each lamination. 40 laminations of 3000mm and 30 of 1350mm were tested. The complete data of all results from dynamic test are shown in *Appendices B.1* and *B.2.* The dynamic test was performed to get the first natural frequency in Hz of each lamination. With the known values, the value of E_x dynamic was calculated.

Resulting longitudinal MOE from the dynamic test was used to be compared with the other values calculated after bending test, and the comparison between them is shown in *Table 9.1.*

Table 9.1 Summary of the results from the dynamic test for all the laminations

Number of laminations	Length of the lamination (mm)	Mean value Ex dynamic (MPa)	Standard deviation (MPa)
40	3000	12269	1748
30	1350	11880	1862

Longitudinal MOE was determined by static test. Resulting Ex from test were measured in a deck composed by laminations of 3m length, and results were calculated according to EN 408 (2007) using the formula below.

$$E_{m.l} = \frac{a l_1^2 (F_2 - F_1)}{16 I (w_2 - w_1)} \qquad (20)$$

Where,

a = distance between load and support (m)

l_1 = distance between LVDT´s (m)

I = MOI of gross cross section of the deck (mm^4)

F_2-F_1 = increment of applied load (N)

w_2-w_1 = increment of deflection under load (mm)

The value for longitudinal MOE was calculated for the three decks and with three different pre-stress levels in each case. As can be seen from the formula, the only unknown values were the increment of applied force and the increment of deflection. The values *a*, the distance between load and support, and *L*, the distance between LVDT´s, were known from the design of the deck and load rig. Longitudinal MOI was calculated by taking into account the height and width of the laminations.

Average distance between load and support was 870mm, and between LVDT´s the distance were 720mm for all three decks. To get the value for the relation between applied load and increment of deflection in kN/mm, a graph was plotted for the different SLTD with each pre-stress level.

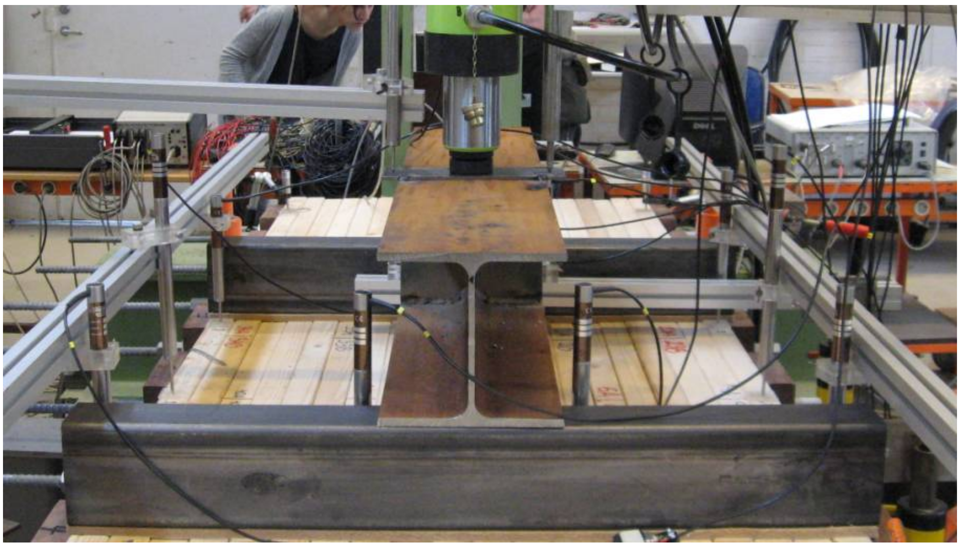


Figure 9.1 Load application during bending test.

One example of the trend line obtained for the case of SLTD without butt joints with the highest pre-stress level is presented in *Figure 9.2*. Inserting the known values in *Equation (20)* and the obtained value from the trend line plotted from the diagrams load-deflection, E_x was calculated.

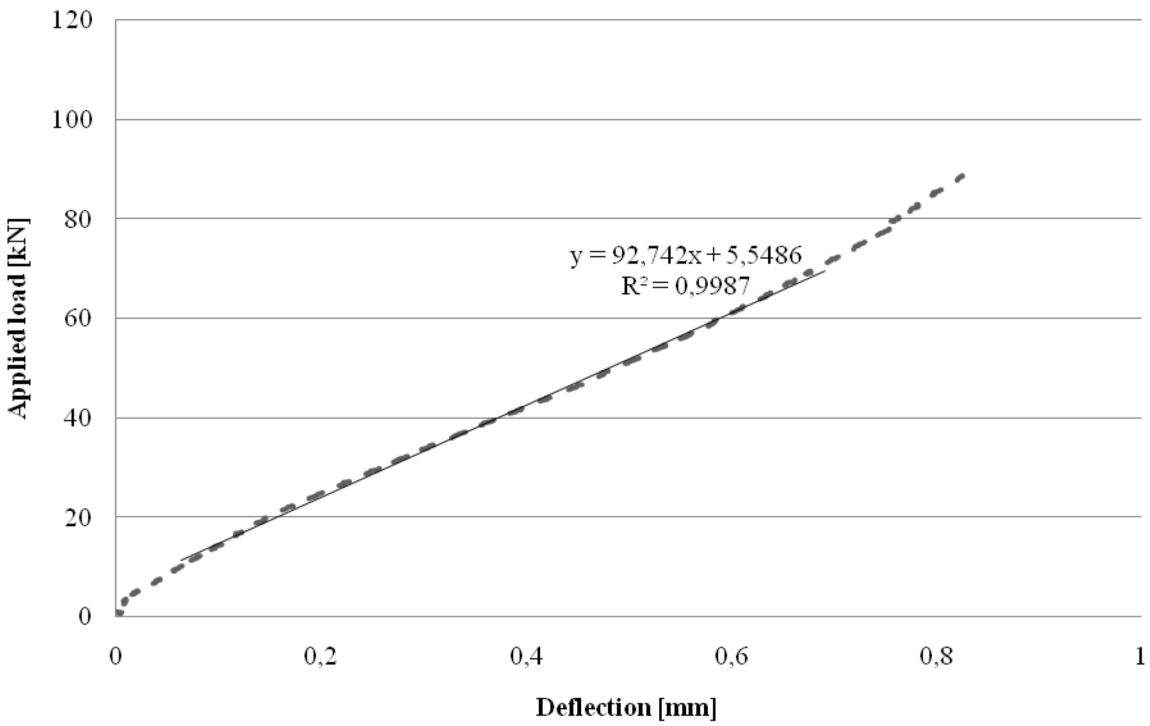


Figure 9.2 Deflection versus applied load for the deck without butt joints and a pre-stress level of 900 kPa. A trend line is used to find the slope of the curve.

Table 9.2 Resulting longitudinal MOE from bending test of deck without butt joints and a pre-stress value of 900kPa.

Relation increment applied load/increment deflection F_2-F_1/w_2-w_1	Longitudinal MOE Ex
(kN/mm)	(GPa)
92.742	12.739

The same procedure was followed for all the cases taking into account the different pre-stress levels obtained for laminations of 3000mm length. A summary of the results is shown in *Table 9.3*. These results can be compared to result from the dynamic test that gave a value of 12.636 GPa.

A summary of test results is shown in *Table 9.3*.

Table 9.3 Results from test of longitudinal MOE for deck without butt joints and with two different butt joint patterns.

Butt joints pattern	Pre-stress level (MPa)	Mean value Ex static (GPa)
Without	0.906	12.739
Without	0.633	13.074
Without	0.301	12.713
1 in 8	0.879	9.999
1 in 8	0.608	9.681
1 in 8	0.309	9.153

1 in 4	0.858	8.163
1 in 4	0.605	7.666
1 in 4	0.310	5.101

The graph in *Figure 9.3* shows the relationship between longitudinal MOE with pre-stress levels applied for the SLTD without butt joints, with 1 in 8 and 1 in 4 butt joint patterns.

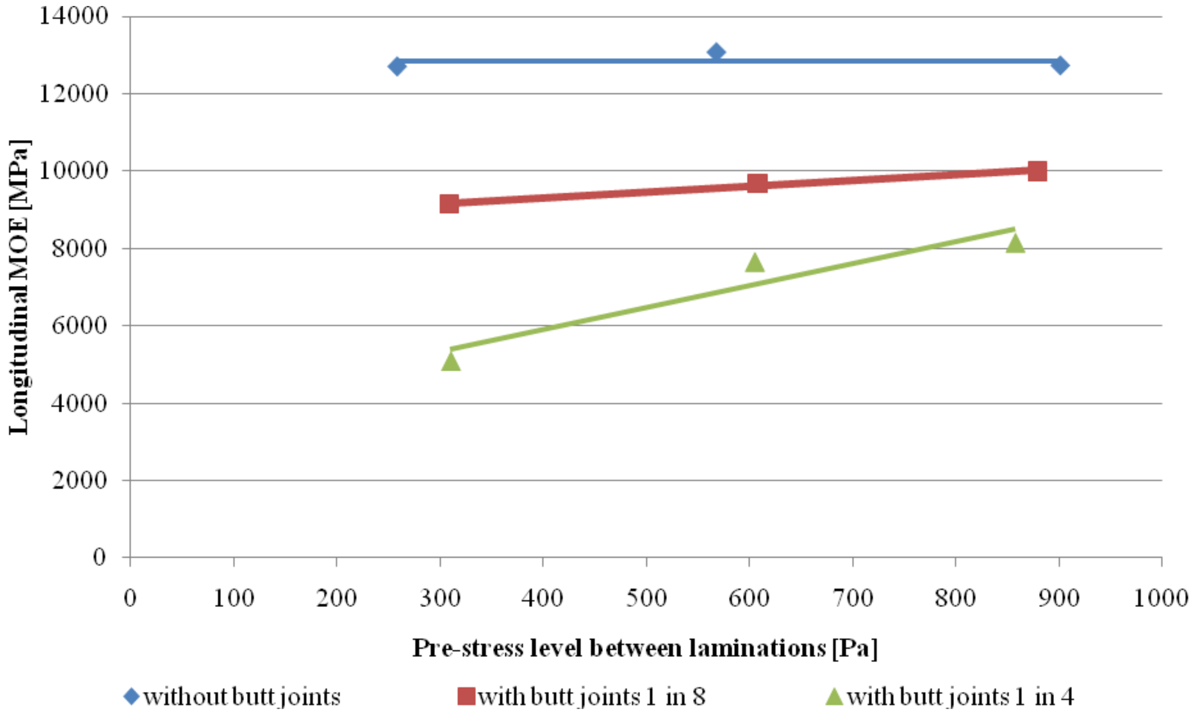


Figure 9.3 Relationship between longitudinal MOE and pre-stress levels with different butt joint patterns and without butt joints.

The results indicate that there is an influence of pre-stress when there are butt joints in a SLTD. Influence of pre-stress is more distinct when a more density butt joint pattern is used. The values for the deck without butt joints are almost the same as the measured value of each single lamination.

9.2 Derivation of in-plane shear modulus

The shear test was made with two different decks, one solid deck and another deck with butt joints. Same laminations were used in both decks but they were cut for the second test in order to make butt joints. Deflection of the SLTD was measured in the corners and in the middle of the deck. The deflection in the middle of the deck was about 25% of the deflection in the loaded corner. To calculate G_{xy} the relation between applied load and deflection in the middle of the deck were used.

The results were plotted with deflection in the middle of the deck on x-axis and applied force on y-axis. With the values from each test, a trend line was written to show a trend of the behaviour. The deflection at the loaded corner was also measured but used only for load control and to check if the deflections were as expected and not used in the derivation of shear modulus. The load deflection curve is not linear and therefore the curve is analysed in two different spans, one before and one after first slip had occurred. In cases where no obvious slip appeared (no sharp change in slope) two spans is taken within levels with different behaviour of the curve. Therefore two trend lines were plotted for each curve and the slopes of these lines were used in the analysis. Below is an example of how each curve is analysed. A total number of six curves were analysed, since one deck were without butt joints, one deck with butt joints and three pre-stress levels were applied for each deck. Trend lines and analyses of curves can be found in *Appendix B.4*.

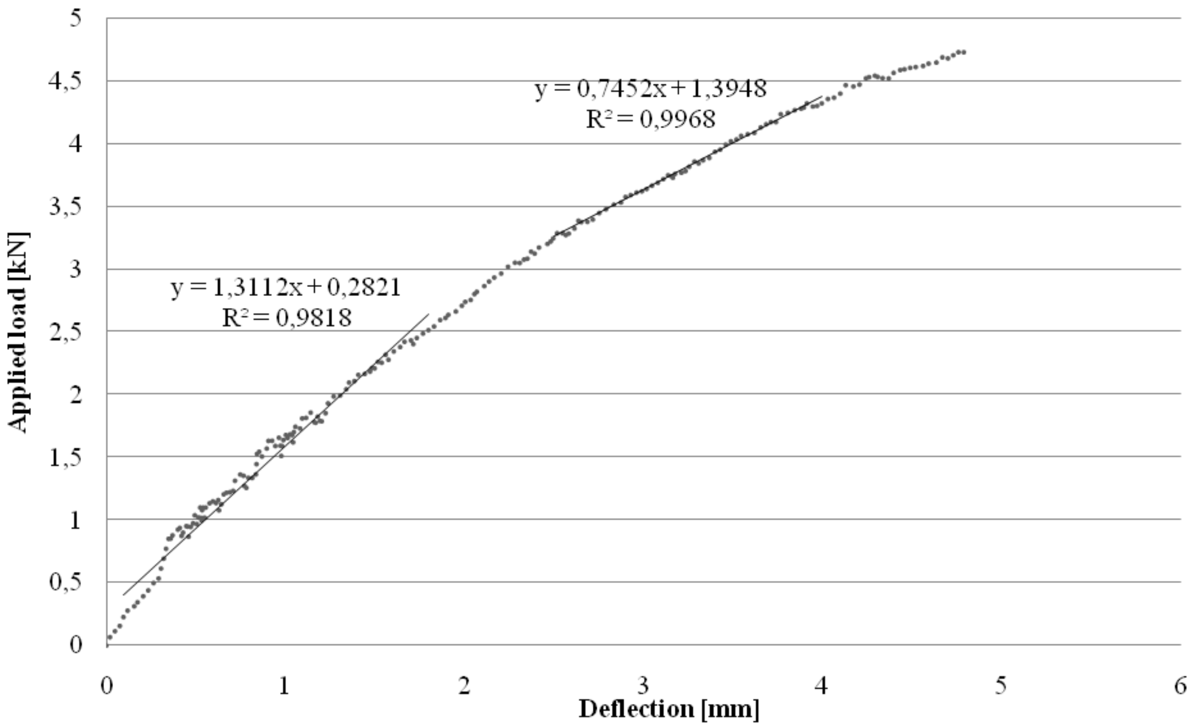


Figure 9.4 Deflection in the middle versus applied load for the square deck without butt joints and a prestress level of 900 kPa. Two trend lines are used to find the different slopes of the curve at different load levels.

Table 9.4 Resulting shear modulus G_{xy} for a deck without butt joints and with a pre-stress level of 900kPa

	Before first slip occurred		After first slip occurred	
Pre-stress level	Load/Deflection P/w	Shear modulus Gxy	Load/Deflection P/w	Shear modulus Gxy
(MPa)	(kN/mm)	(MPa)	(kN/mm)	(MPa)
0.90	1.311	413	0.745	235

By combining orthotropic plate theory and bending equations it was possible to find a formula for the in plane shear modulus (explained in Chapter 4.1.3). The shear modulus is given by the equation (21):

$$G_{xy} = \frac{3Pl^2}{4h^3 w(0,0)} \tag{21}$$

Where P is applied load, $w(0.0)$ is deflection in middle of deck, h is height of deck and l is distance between the supports. In this case the height was 145mm and the width 1350mm. P/w is in this case the same value as the slope of the curve.

The graph in *Figure 9.5* shows the relationship between shear modulus and pre-stress levels for the deck without butt joints in relation to the deck with butt joints. Values are presented before and after that slip occurred.

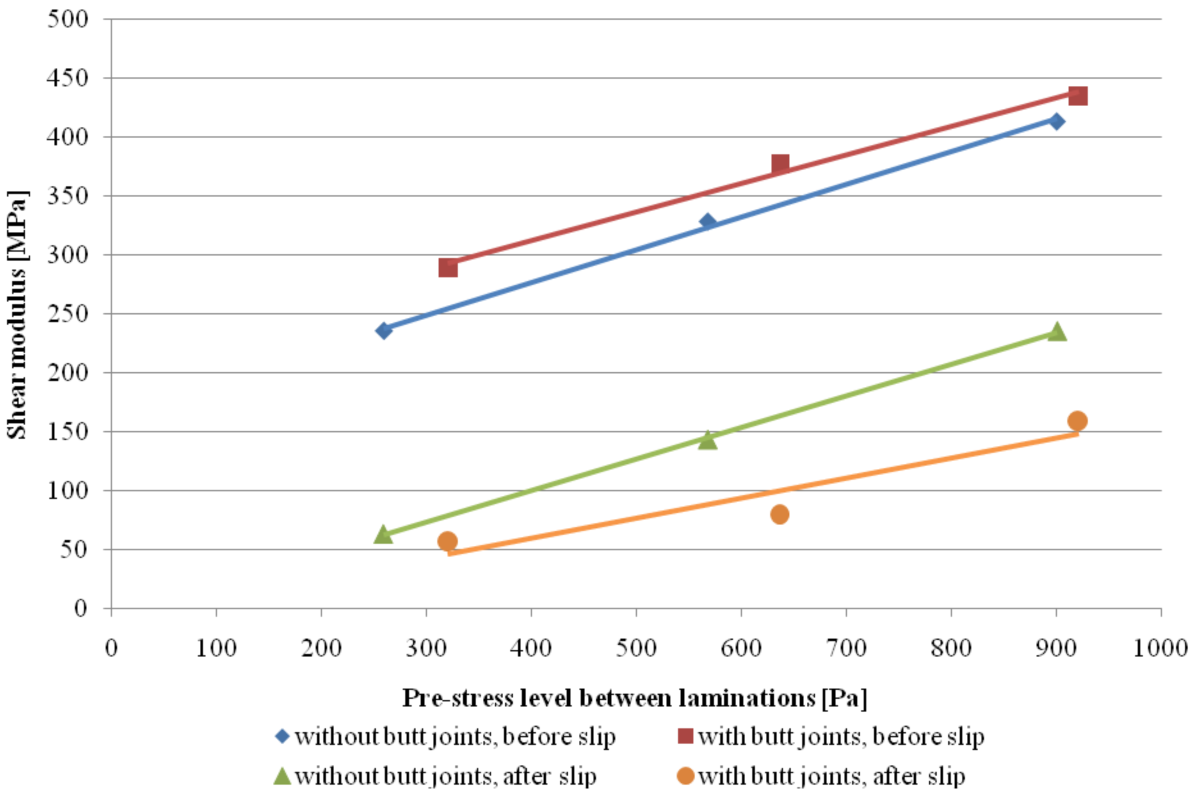


Figure 9.5 Relationship between shear modulus and pre-stress levels before and after slip had occurred.

The lines in *Figure 9.5* are linear trend lines of the values at the three pre-stress levels. Before slip the deck with butt joints seems to have a higher value of shear modulus than the deck without joints. But after first slip the shear modulus of the deck without butt joints had a higher value. The difference in shear modulus for the deck with butt joints is larger than without, which indicates that there should be an influence of butt joints in shear if the loads are at a high level (enough for slip to occur).

Table 9.5 Summary of test results for in plane shear modulus without butt joints. The results are also presented as a quotent between shear modulus and longitudinal MOE.

	Before first slip occurred		After first slip occurred	
Pre-stress level	Shear modulus Gxy	Relation Gxy/Ex.dynamic	Shear modulus Gxy	Relation Gxy/Ex.dynamic
(MPa)	(MPa)	(%)	(kN/mm)	(%)
0.90	413	3.48	235	1.98
0.58	328	2.76	143	1.20
0.26	235	1.98	63	0.53

Table 9.6 Summary of test results for in plane shear modulus with butt joints

	Before first slip occurred		After first slip occurred	
Pre-stress level	Shear modulus Gxy	Relation Gxy/Ex.dynamic	Shear modulus Gxy	Relation Gxy/Ex.dynamic
(MPa)	(MPa)	(%)	(kN/mm)	(%)
0.92	435	3.66	159	1.34
0.64	377	3.17	79	0.67
0.32	289	2.43	56	0.47

The recommended relationship between shear modulus and longitudinal MOE should be around 3% according to Ritter´s guide (1990). This can be compared to the values in *Table 9.5* and *9.6*.

9.3 Analysis of interlaminar slip during shear test

In Chapter 8, the data from the test was presented. Here is a description of a model to analyse the behaviour of laminations in a timber deck subjected to shear.

Following the simplifications of Jónsson and Kruglowa (2009) that proposed the use of the triangular areas (divisions for each beam) of the laminations to calculate the moment produced from the shear stresses it was possible to get theoretical forces needed to produce slip. The idea is that a load is applied close to one end of the lamination and this load is resisted by a moment caused by the friction between the laminations. How the moment is distributed can be seen in *Figure 9.6.*

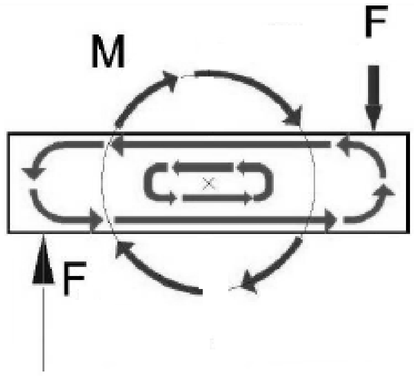


Figure 9.6 Twisting of lamination a deck due to the applied loads, F.

Slip occurred at different load levels depending on the butt joint pattern. As the length of the laminations was reduced because of butt joints, the motion to rotate the laminations should be lower.

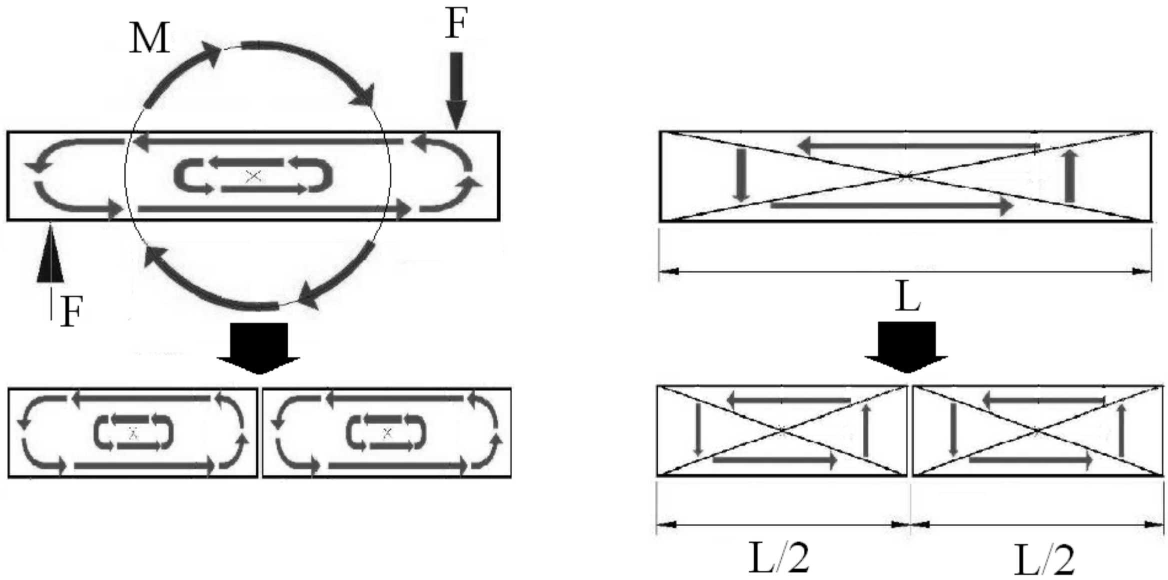


Figure 9.7 Difference of shear resistance when solid laminations were used in comparison to laminations with butt joints. A solid lamination is shown in the upper figure and a lamination with a butt joint in the figure below,

Figure 9.7 shows how a simplification could be performed to explain how friction between the laminations resisted a concentrated force. When a concentrated load is applied in one end of a lamination a moment is caused around its centre axis. This moment is resisted by the friction between the laminations. When a butt joint is introduced the lamination becomes shorter and the friction area between the laminations becomes smaller. This causes a decreased moment resistance of each lamination and therefore the maximum allowed load become smaller. From the information of different applied pre-stress levels in the two decks, inter laminar slip points were identified, and the force needed to produce slip could be observed also in some of the test results.

When analysing the friction the moisture content in timber is necessary. After the shear test, the main value of moisture content from the measurement of 8 different laminations was identified as 11.21%. For the values of the different moisture contents on the surfaces, see *Appendix B, Table B.4.* Taking into account the Kalbitzer (1999) standards, the value recommended for moisture content of 10.1% as coefficient of friction was μ=0.28 (considering force acting perpendicular to the grain in a planed beam).

Based on that, the interpolation to get the coefficient of friction leads to μ=0.311.

$$\mu_{11.21} = \frac{0.28 \times 11.21}{10.1} = 0.311 \qquad (22)$$

For each pre-stress level in the two timber decks the match point of moment from the applied forces in the tests and the moment from the shear produced in the laminations were searched. The values for loads needed to produce slip were defined, and the summaries of results from each SLTD are shown below.

Table 9.7 Summary of the load needed to produce a slip at different pre-stress levels in the deck without butt joints.

Pre-stress level (MPa)	Length of the lamination (mm)	Load needed to produce a slip (kN)
0.90	1350	10.787
0.58	1350	6.517
0.26	1350	3.034

For the SLTD with butt joints there were four different lamination lengths, and therefore it should had four different values of loads needed to produce slip, due to the variation of the contact area for each lamination.

Table 9.8 Summary of the load needed to produce a slip at different pre-stress levels in the deck with butt joints.

Pre-stress level (MPa)	Length of the lamination (mm)	Load needed to produce a slip (kN)
0.92	1080	8.562
0.92	270	2.901
0.92	810	6.675
0.92	540	4.788
0.64	1080	5.892
0.64	270	1.996
0.64	810	4.594
0.64	540	3.295
0.32	1080	3.038
0.32	270	1.029
0.32	810	2.369
0.32	540	1.699

Some comparisons between these results and the plots are shown in graphs taking into account that the maximum force applied in the test was 4.48kN.

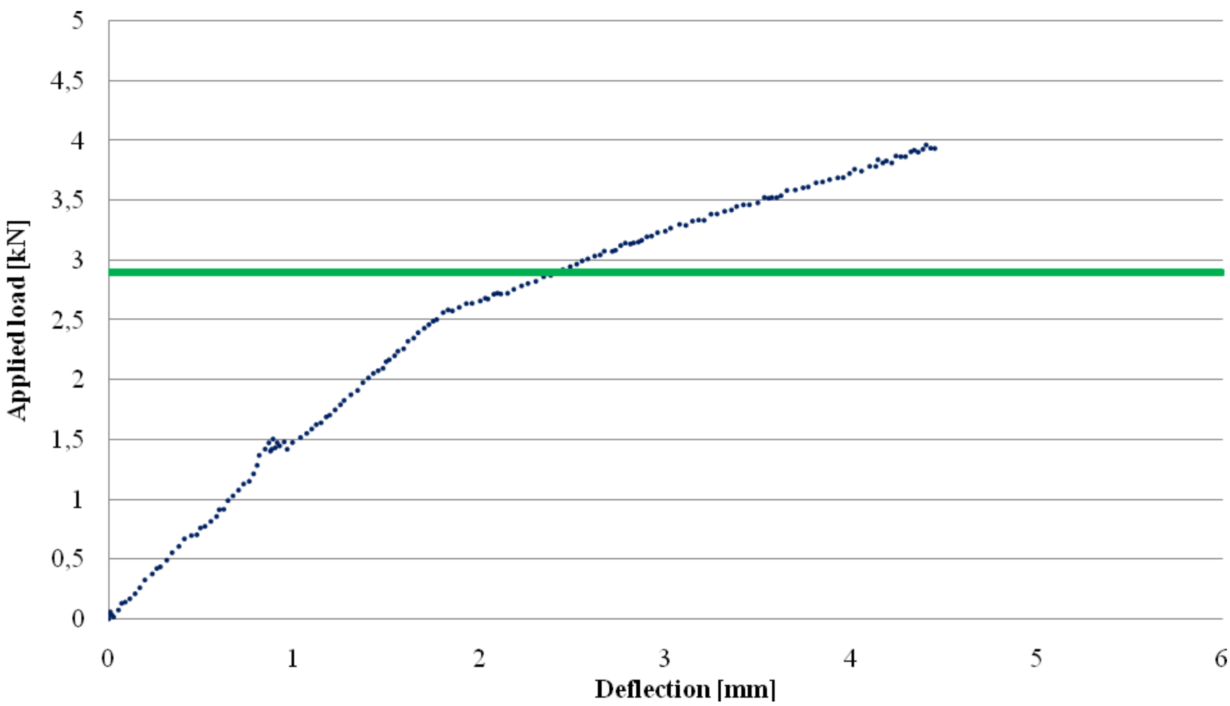


Figure 9.8 Deck with butt joints for 921kPa pre-stress level. The first load needed to produce slip is at 2.9kN.

The applied load to produce slip according to calculations is shown in the graph in *Figure 9.8* marked with a horizontal line at 2.9kN. This slip occurs in the contact surface for the laminations with 270mm length. The maximum load in the test was not enough to cause slip for the three other contact areas because the loads needed to create slip are higher than the maximum load applied during the test. In the graph one slip seemed to occur at 2.6kN and this was relatively close to the calculated result. There was also a disturbance on the curve at 1.4kN but since the line did not change slope after this value this was probably caused by the load application and not of the material in the deck. The difference between calculated and tested value is probably because of wrong pre-stress level during test.

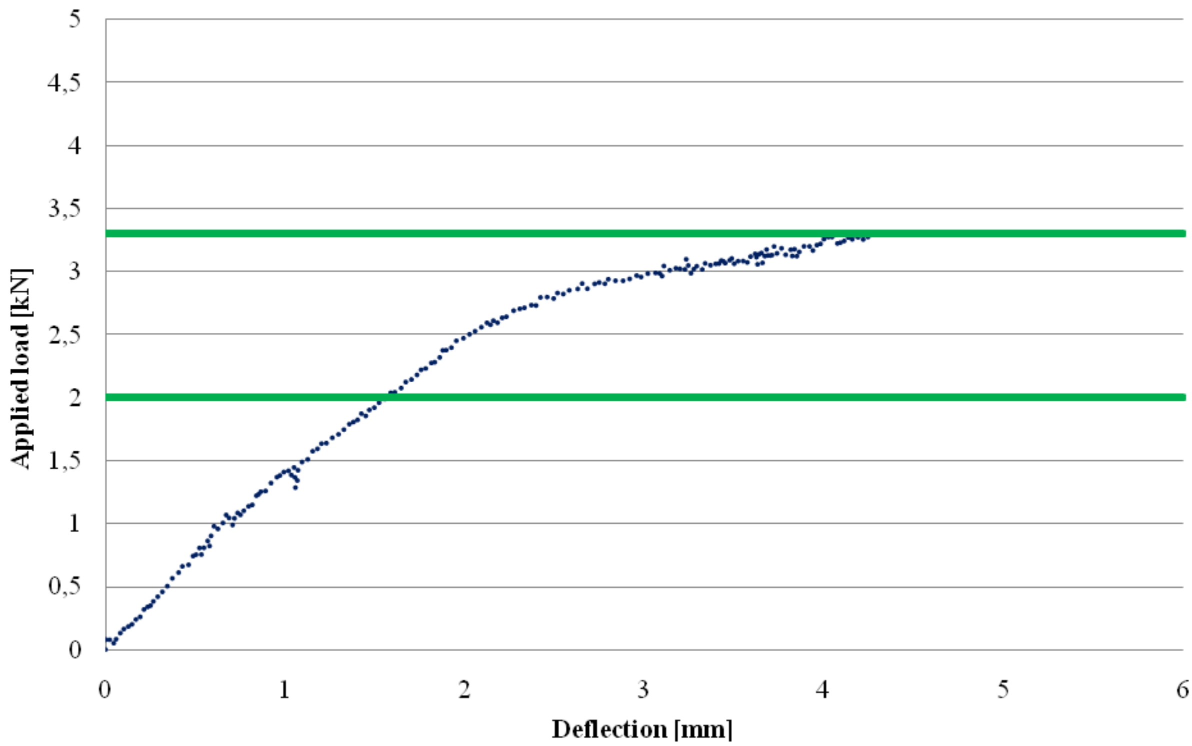


Figure 9.9 *Deck with butt joints for 637kPa pre-stress level. The loads needed to produce slip were at 2.0kN and 3.3kN.*

With the prestress level of 637kPa the applied load was enough to produce slip between two different laminations. The slip occured in the laminations of length 270mm and 540mm. Compared to result from the test, the calculated values for slip loads seemed a bit different from what was expected. In *Figure 9.9* a disturbance of the slope occured at a load level of 2.8kN. The reason of why a slip did not exist at the load 2.0kN could depend on that a slip was observed in the previous test with prestress level 921kPa.

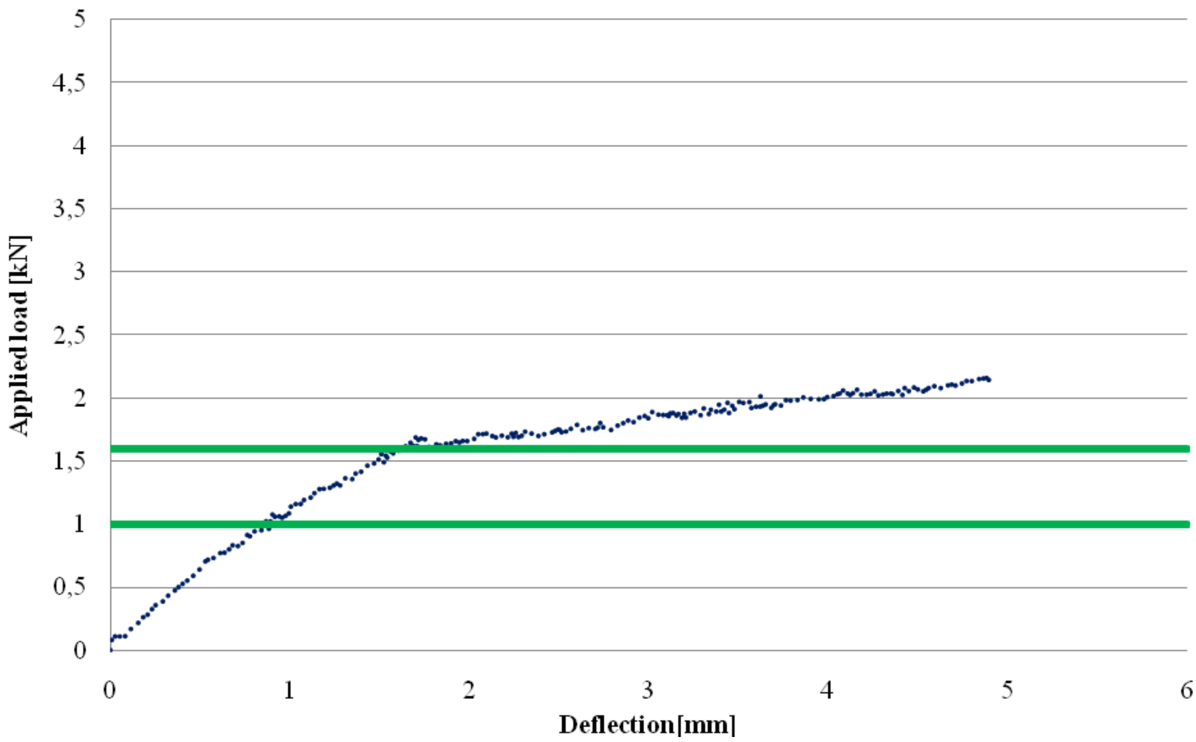


Figure 9.10 Deck with butt joints for 319kPa pre-stress level. The loads needed to produce slip were at 1.0kN and 1.6kN.

With the prestress level of 319kPa, the applied load was enough to produce slip between two different laminations. The slip occured in the laminations of length 270mm and 540mm, the same as with the prestress level of 637kN.

Compared to the result from the test the calculated values for slip loads seemed reasonable. In *Figure 9.10, a* significant disturbance of the slope occured at a load level of 1.6kN and this was the second slip according to the theory. Also in the load level of 1.0kN, a slip could be produced because of the slip in the previous pre-stress levels. Focusing on the curve before and after 1kN there seemed to be two different slopes of the curve that match each other at the expected load level.

9.4 Possible sources of error

The materials in the test were not of the quality originally expected. The timber used was ordinary construction timber of quality C24. Since it was solid timber and not glulam, the laminations were more prone to twist with losing moisture content in the dry laboratory area. Since the timber had a lower quality than desired, the influence of butt joints was not as big as expected. By using a better timber quality the weakening effect of the butt joints should have been more significant.

The measurement of pre-stress was made with strain gauges and the pre-stressing force was calculated from the strain in the steel bars. As the strain gauges sometimes gave strange results during the pre-stressing procedures, it could have caused wrong application of the pre-stressing force. A control calculation of the measured values of pre-stressing during the tests showed that the pre-stress levels in the deck with butt joints were a bit higher than in the deck without butt joints.

The results from the shear test with butt joints seemed to be stiffer than the deck without butt joints. This could be caused by mainly two reasons. The first, and more probable reason, was that the pre-stress level was a bit higher in the tests with butt joints. Because of a higher pre-stress level also the friction increased (that had an influence on the results). The second reason was due to geometrical properties of the timber laminations. Because of the loss of moisture in the timber some of the laminations became a bit twisted. This decreased the connection area between the laminations, and especially for lower pre-stress levels. When the laminations were saws into shorter parts, in order to make the butt joints, the connection surface between the laminations could be increased, since two short laminations were less distorted than the original longer lamination.

Figure 9.11 Detail of the slip between laminations after testing.

The moisture content and drying of timber in the dry laboratory environment might have had some influence on the results. There were two days passing between the test with and without butt joints which could cause a loss of moisture to the second test, and this gave a lower friction resistance between the laminations. The test with butt joints was made after the test without butt joints so this could cause a lower friction in the test with butt joints. The moisture content was measured in some randomly chosen laminations during and after the tests and no significant change of moisture was observed. But since only some laminations were measured there could be some variations in the other laminations. The measured moisture content in the laminations after the tests was 14.4% in the wood and 11.2% close to the surface in the wood. These values were higher than expected, in relation to the dry environment in the laboratory area.

Slip between the laminations closest to the corner was prevented by the square steel deck used as support. These steel plates covered several laminations and did not permit them to behave different from each other.

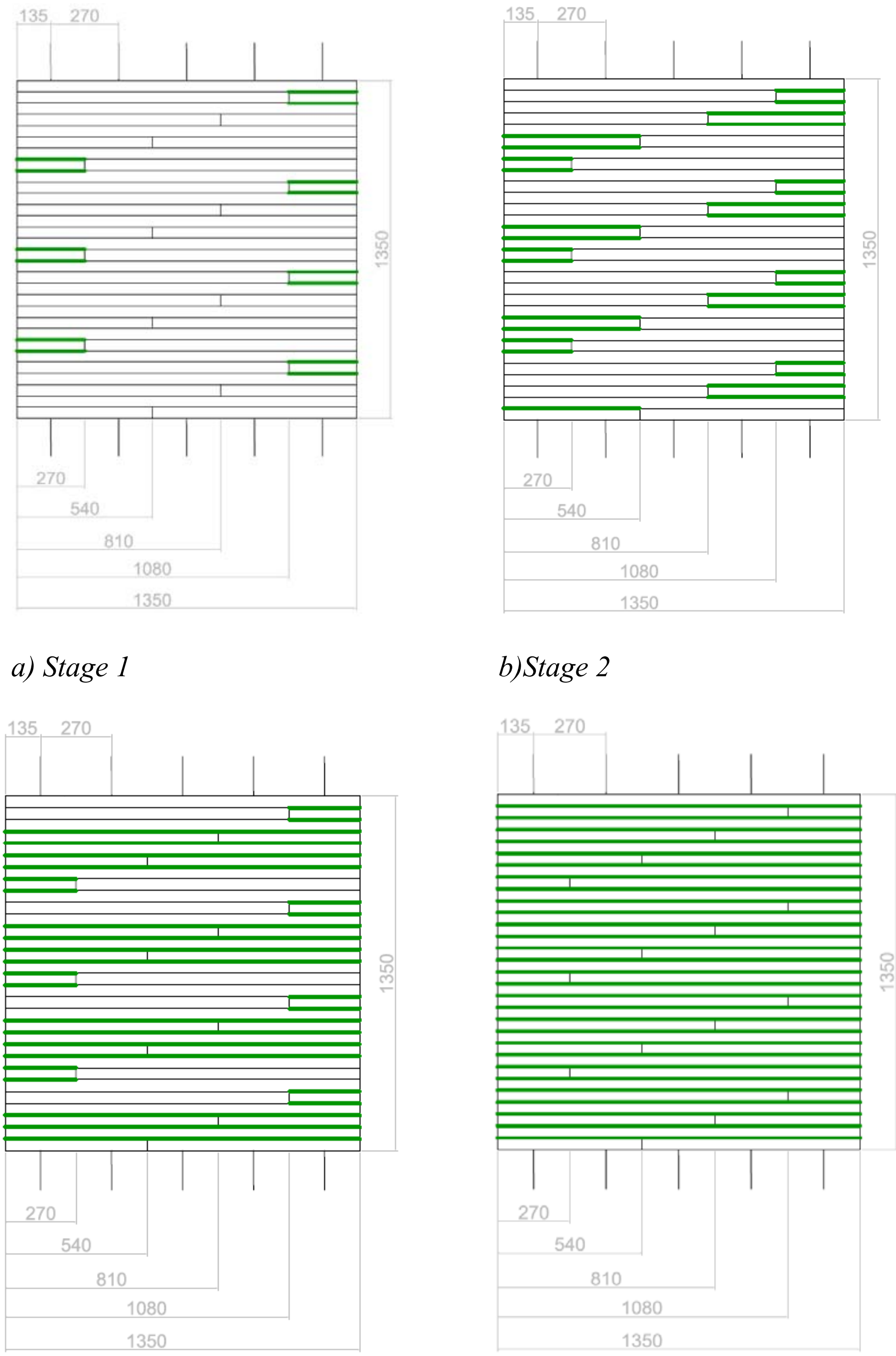
10 Discussions

This chapter show some of the evaluated results from the lab tests and the theories of why some results were obtained. The results are here evaluated and compared to other research within the area of SLTD.

10.1 Influence of butt joints on the shear modulus

As explained in *Chapter 9*, slip occurred depending on the butt joint pattern and when a butt joint caused a decreased moment resistance of each lamination. Therefore the maximum applied load had to decrease.

Since the laminations in the deck had different lengths the contact surfaces between them varied. A smaller contact surface should slip before a larger surface, since the friction is dependent of the size of the contact area between laminations. The deck used in the tests had four different sizes of the contact areas. Therefore it can be assumed to be four different stages of slip. When a small load is applied, deflection occurs due to elasticity in the material. With a higher load level, slip occurs between the smallest laminations and the surrounding laminations. This is *Stage 1* according to *Figure 10.1*. With an increased load the longer laminations successively starts to slip. In last stage, *Stage 4*, the contact surfaces between all laminations slip. This occurs at same load level as solid laminations, without butt joints, should start to slip, according to the theory.



a) Stage 1 b)Stage 2

c) Stage 3 d)Stage 4

Figure 10.1 Four stages of slip between laminations.

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Figure 10.2 shows the response of the shear modulus for the different pre-stress levels from this study together with other curves from previous studies, where other configurations of SLTDs were used. Material properties of timber are assumed to be the same in all studies, to be able to compare the results between them. The shear modulus obtained for each pre-stress level showed more or less a linear response in all studies.

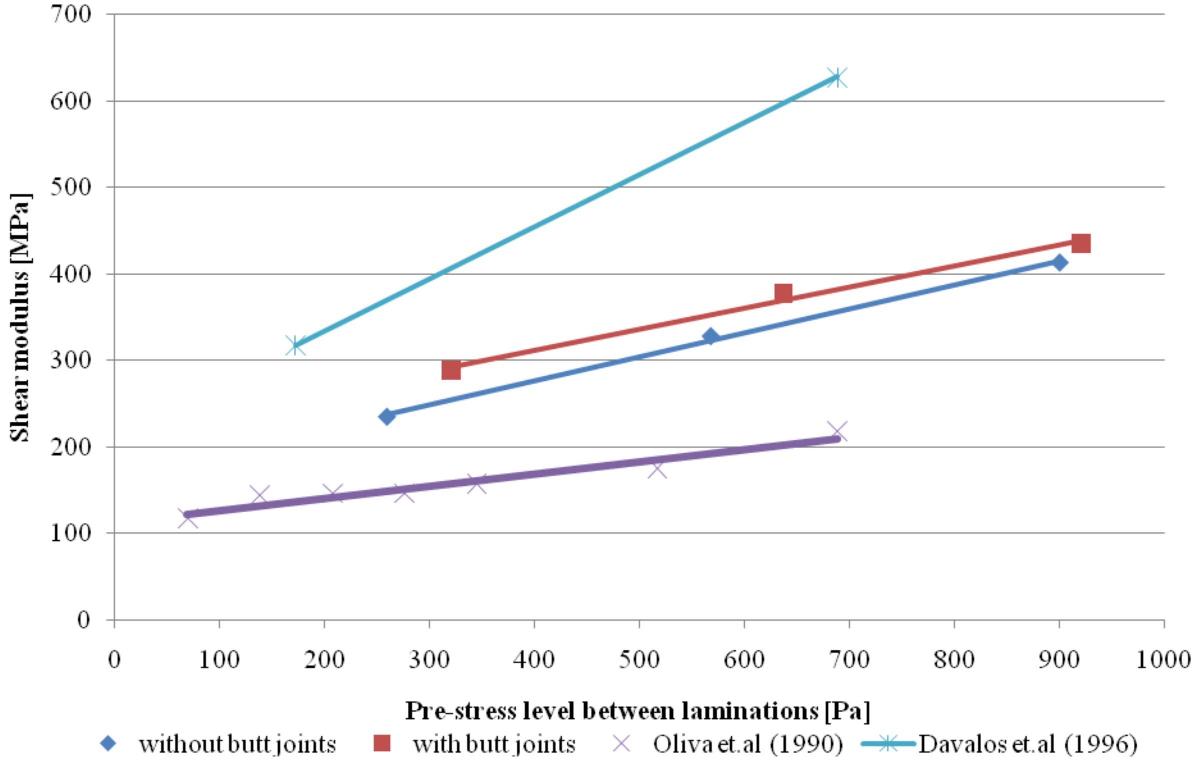

Figure 10.2 Relationship between shear modulus and pre-stress level between laminations for three different sources.

In *Figure 10.2* the linear response obtained from the shear test of this thesis can also be identified. Above are values before slip has occurred in the deck. The deck with butt joints has a higher shear modulus than the deck without butt joints

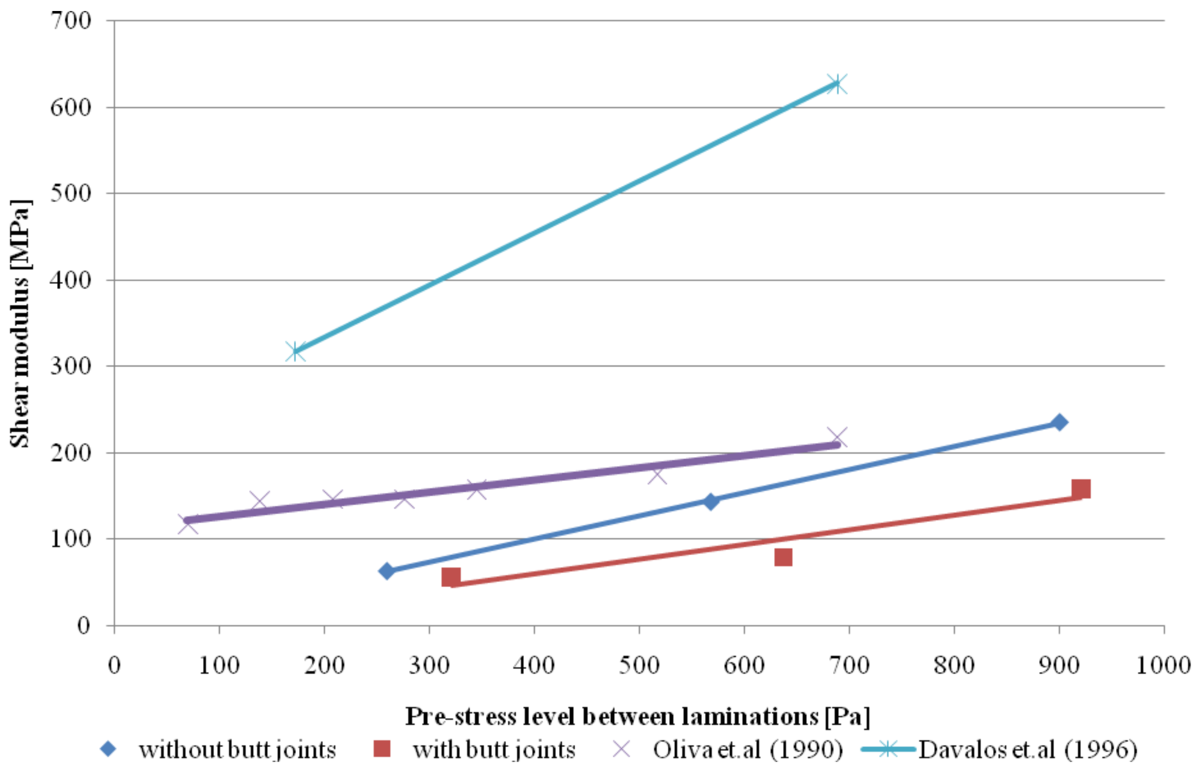

Figure 10.3 Relationship between shear modulus and pre-stress level between laminations for three different sources.

In *Figure 10.3* the linear response obtained from the shear test of this thesis can also be identified. Above are values after that slip has occurred in the deck. In this case, the shear modulus for the deck without butt joints is higher than the value for the deck with butt joints

In previous studies, different researches used different dimensions of the specimens and this could have an influence on the resulting shear modulus. Therefore a calculation of influence from height-length relation was done and the result is presented in *Table 10.1*.

Table 10.1 Relationship between height and length for different SLTD dimensions in order to check its influence on the shear modulus.

Influence of the SLTD dimensions in shear modulus in different sources					
This study		Davalos et al.		Oliva et al.	
Height	Length	Height	Length	Height	Lenght
145mm	1130mm	101.6mm	3660mm	286mm	1219mm
Height/length		Height/length		Height/length	
0.128		0.028		0.235	

The height/length ratio in the studies in this thesis is between values used in the two previous studies. The results obtained from the lab tests seemed reasonable because of the relation between height and length should be between the two other studies, which were indicated by the graph in *Figure 10.2.*

10.2 Influence of butt joints on the longitudinal MOE

Table 10.2 Summary of the results of E_x for each pre-stress level with the three SLTD.

Butt joints pattern	Pre-stress level (kPa)	Mean value E_x static (GPa)
Without	900	13.132
	600	9.883
	300	10.205
1 in 8	900	12.718
	600	9.414
	300	8.973
1 in 4	900	12.713
	600	8.827
	300	7.523

Table 10.3 Summary of the relationship for E_x.

	Without	With 1 in 8	With 1 in 4
Relationship E_x with 900-600kPa (%)	24.741	25.979	30.567
Relationship E_x with 600-300kPa (%)	-3.258	4.685	14.773

The increment of E_x under 900kPa and 600kPa was around 25% for the three patterns. It seemed clear that E_x increased a lot if the butt joint pattern also increased. The increment between the SLTD with 1 in 8 butt joint pattern and the other one with 1 in 4 was much higher than the difference between increment of E_x with 900kPa-600kPa. The increment in the 600kPa to 300kPa case was of 68.29% from the E_x value for 1 in 8 to E_x in SLTD with 1 in 4, while for 900kPa to 600kPa, the increment was only 15%.

An evaluation between the results obtained from the lab tests and the Ritter´s (1990) and Crews´ (2002) works was performed.

Figure 10.4 showed the mean value obtained from the dynamic test of each lamination, taking into account only the 20 laminations used (see *Appendix A Figure A.11, Figure A.13*) for the bending test. The mean value obtained was 12269MPa, and it is plotted in the graph with a line (same value for the three pre-stress levels).

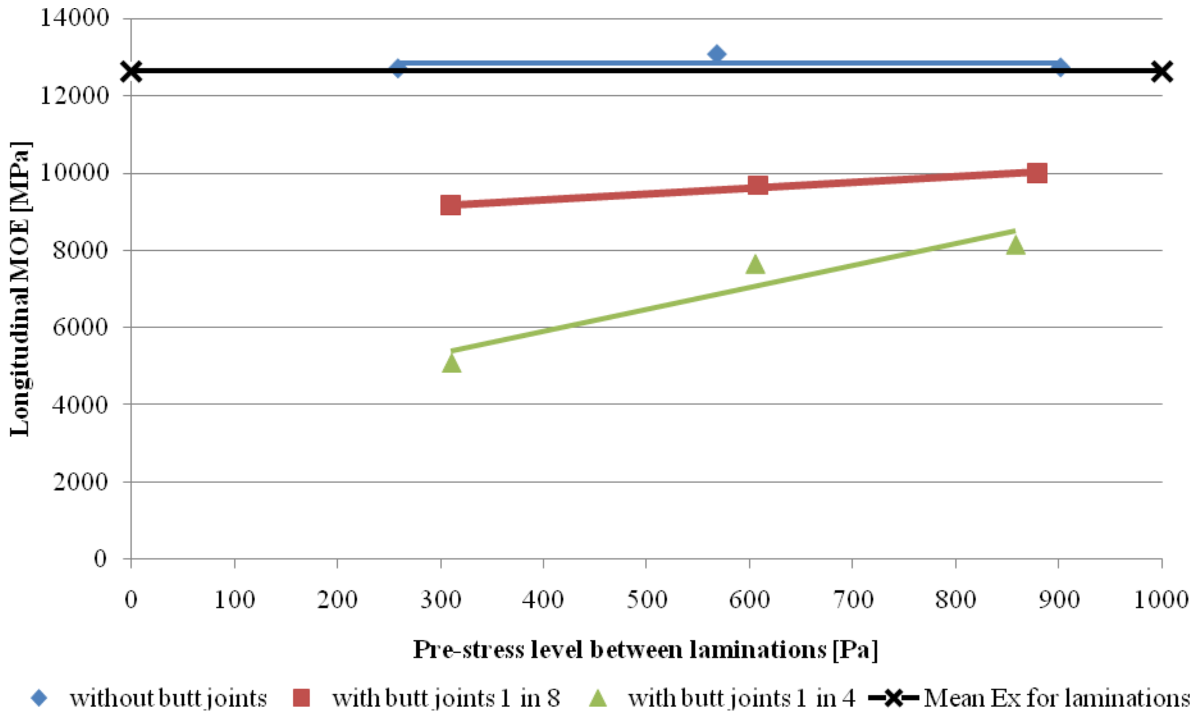

Figure 10.4 *Relationship between longitudinal MOE for different pre-stress levels in SLTD without butt joints, with 1 in 8 and 1 in 4 pattern, and mean MOE of each laminations of the deck.*

Figure 10.5 shows the relationship between longitudinal MOE and pre-stress for the SLTDs without butt joints and with 1 in 8 butt joint pattern, and also the response obtained following the recommendations of Ritter (1990) and Crews (2002) for the deck with 1 in 8 butt joint pattern. The response from the tests seemed to be linear for all cases, however the curve corresponded from test on SLTD with 1 in 8 pattern showed underestimated longitudinal MOE values.

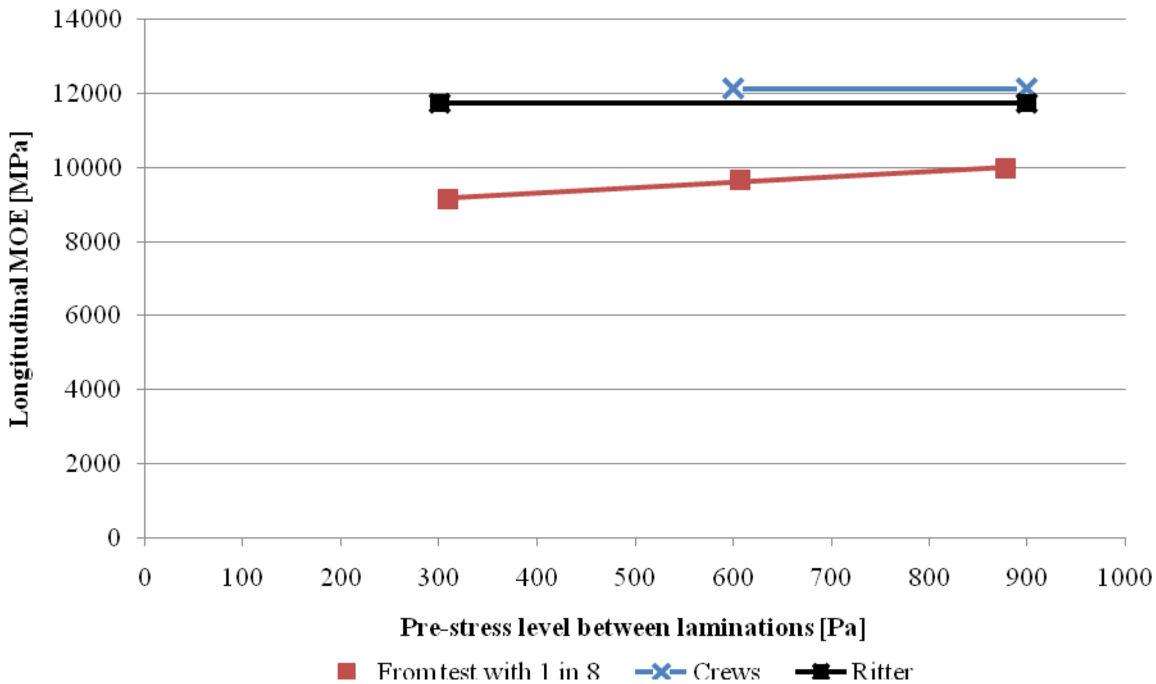

Figure 10.5 *Relationship between longitudinal MOE for different pre-stress levels in SLTD tested without butt joints and with 1 in 8 pattern, and the theoretical responses of the deck with 1 in 8 pattern, due to Ritter´s (1990) and Crews´ (2002) recommendations.*

Figure 10.6 shows the same kind of relation between longitudinal MOE and pre-stress for the SLTDs without butt joints and with the SLTD with butt joints, but in this case for the SLTD with 1 in 4 butt joint pattern. Also it was plotted the response obtained following the recommendations of Ritter (1990) and Crews (2002) for the deck with 1 in 4 butt joint pattern. The SLTD curves obtained with Crews and Ritter recommendations were close to the curve from test results under the highest pre-stress levels.

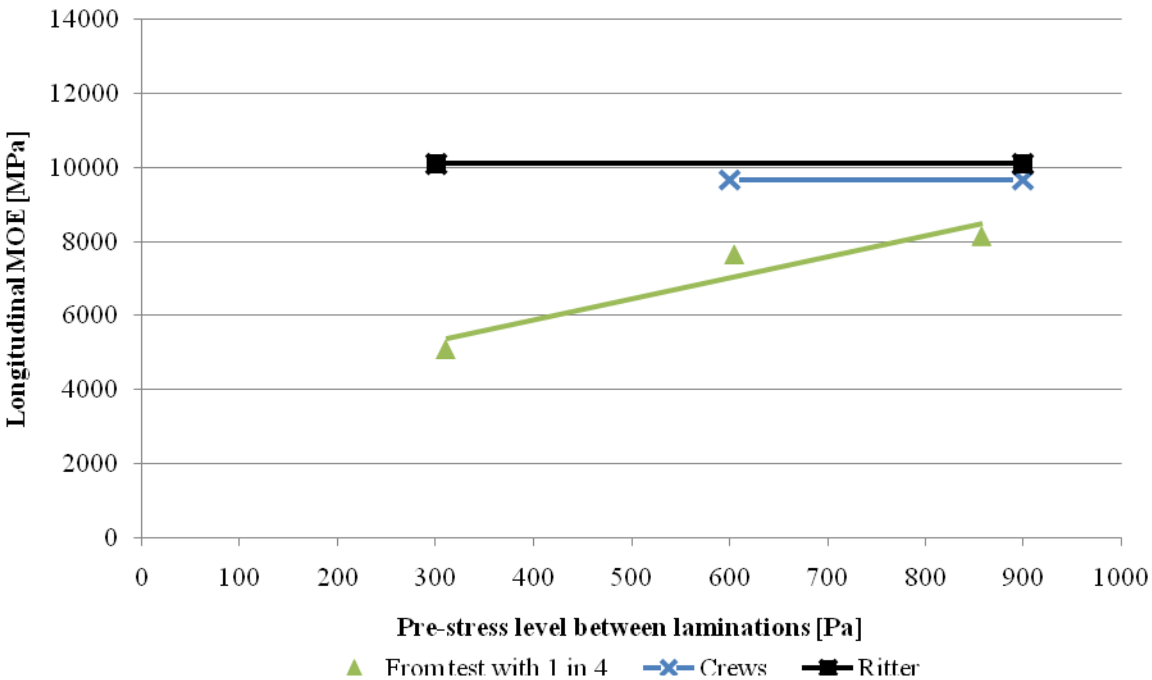

Figure 10.6 Relationship between longitudinal MOE for different pre-stress levels in SLTD tested without butt joints and with 1 in 4 pattern, and the theoretical responses of the deck of 1 in 4 pattern due to Ritter´s (1990) and Crews´ (2002) recommendations.

10.3 Relationship between pre-stress level and butt joints

Table 10.4 Comparison of shear modulus for a deck with and without butt joints for different pre-stress levels.

Shear modulus G_{xy} for SLTD without butt joints (MPa)	Shear modulus G_{xy} for SLTD with butt joints (MPa)
413	435
328	377
235	289

The value for G_{xy} varied depending of the butt joints pattern. A comparison of the values gotten for the two SLTDs tested showed that the shear modulus increased in a deck with butt joints. The pre-stress for the two SLTD was not the same, but anyway, the increment of G_{xy} value showed the highest increment for the case of the lowest pre-stress levels. So, the influence of the pre-stress level in the shear modulus depended also of the butt joints presence in the SLTD. For high pre-stress levels, the G_{xy} was not very different, so the influence of the butt joints was decreasing according with the increment of pre-stress.

10.4 Conclusions

From the analysis of the results, some conclusions about the influence of butt joints on SLTD were made.

The shear modulus did not seem to have any clear influence related to butt joints on SLTD. The results after shear tests showed that there were some differences between the deck without and with butt joints but nothing in relation with the introduction of butt joints in them. Initial imperfections and some effects from pre-stress of the steel bars could have an influence on the shear modulus results.

The longitudinal MOE decreased significantly with the increment number of butt joints. The response of the SLTD with most dense butt joints pattern behaved almost equal for different pre-stress levels. As the number of butt joints increased and the pres-stress level decreased, the influence of butt joints was more significant. In general, the longitudinal MOE value was higher for SLTD without butt joints because of the relationship between applied load and deflection was higher at the three pre-stress levels. The longitudinal MOE varied much more if the pre-stress levels in the SLTD were high, compared to lower pre-stress levels.

Comparing results obtained on SLTDs with 1 in 8 and 1 in 4 butt joint pattern following Ritter (1990) and Crews (2002) recommendations, the results were close to each other. However, the results on SLTDs from lab tests differed significantly from the two design methods, being that difference even higher at lowest pre-stress level. All the results of longitudinal MOE from the lab test were underestimated compared with the results following the recommendations, which seemed to be much more on the safety side.

The differencies in results between different butt joint patterns were more significant at higher load values. One question is if this also has an influence on a bridge in reality. A bridge is very seldom subjected to a load close to the maximum ultimate load. Therefore the influence should be smaller than what was indicated in this thesis. Most differencies in performance between different butt joint patterns in this thesis is due to slip between laminations. In a bridge slip does not usually appear, and therefore the influence in a real bridge due to butt joints seems to be very small.

10.5 Suggestions for further research

After experimental test, analyses of data, discussions and conclusions about the results, some suggestions for further research were done.

Influence of steel plates between butt joints. The influence of butt joints with different patterns in the same SLTD was checked, but the possibility of improve the butt joints introducing steel plates between laminations in the tension side is suggested to improve the SLT bridges decks.

Influence of the use of different timber species in SLTD with butt joints. The influence of the timber species of the laminations that compose the SLT bridge deck is suggested to improve the behaviour under load.

Finite Element model that analyses the influence of butt joints. Some of the results obtained from this thesis can be used to compare with a FE-model of the behaviour of SLTDs with butt joints. This model should take into account the moisture content and friction between the laminations. By a more correct analyse of the real behaviour of a SLTD could be made.

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Appendix A – Detailed sketches

In this Appendix are shown all the detailed draws with the dimensions specified for an easy comprehension of the test done. The values of the dimensions and loads applied in the entire test are included. The laboratory plates used for the test are reproduced in this Appendix.

A.1 Moisture tests

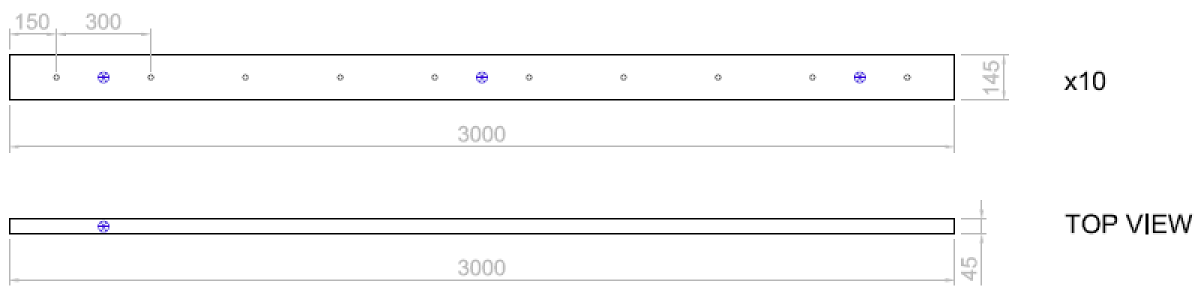


Figure A.1 Dimensions of the laminations used for the moisture test. The moisture was measured in 20% of the total number of laminations of 3000mm, and the number of the laminations used was: 1, 4, 8, 12, 16, 18, 26, 28, 34, 36.

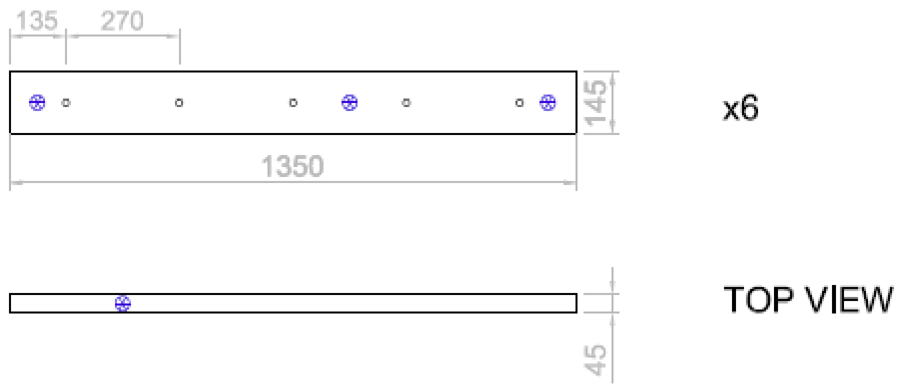


Figure A.2 *Dimensions of the laminations used for the moisture test. The moisture was measured in 6 of laminations of 1350mm length. The numbers of the laminations tested were: 41, 47, 50, 56, 62, 68.*

A.2 Bending test

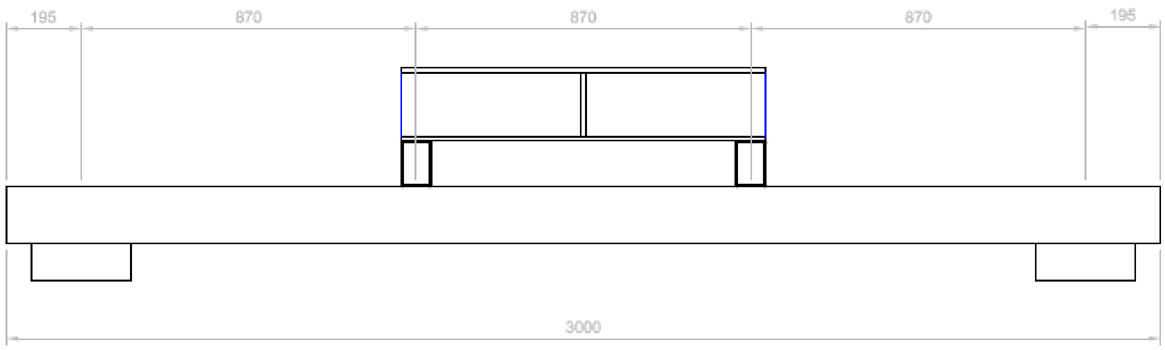


Figure A.3 *In this drawing the place and the dimension between each element used for the bending tests are presented. The distance between the two applied load lines was 870mm as well as for the distance between loads and supports. A load rig is used to distribute the punctual load applied in the middle of the span. In the following drawings a more detailed description of the load rigs used for the tests is given.*

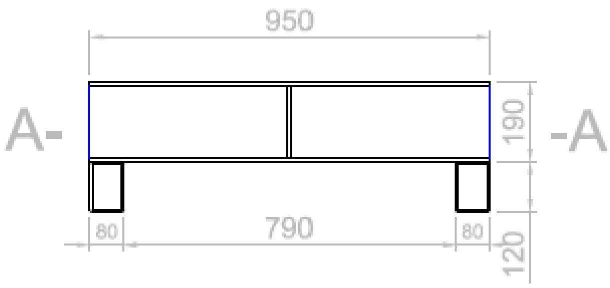


Figure A.4 *Frontal view of the load rigs used to distribute the punctual load applied in the middle of the span. The beam used as main beam was one HEA 200of 950mm length. Two VKR 120X80X5, of length 900mm, were used as spreader beams*

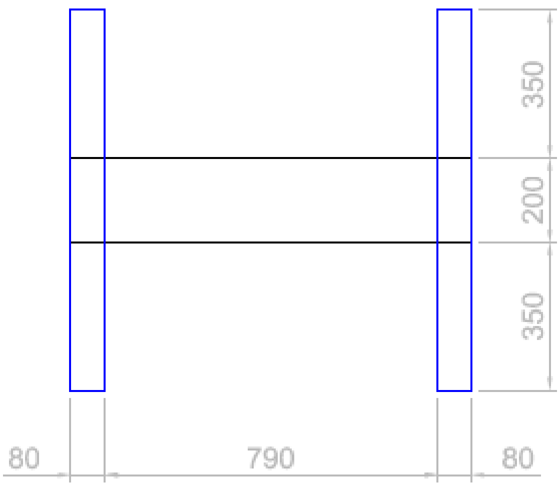


Figure A.5 *Plan view of the main beam and the two spreader beams placed symmetrically.*

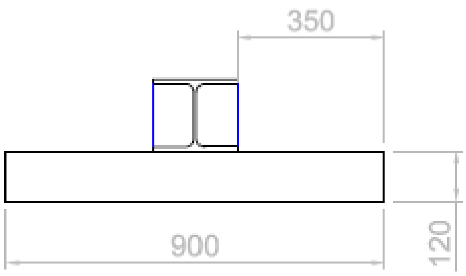


Figure A.6 *Side view of load rig.*

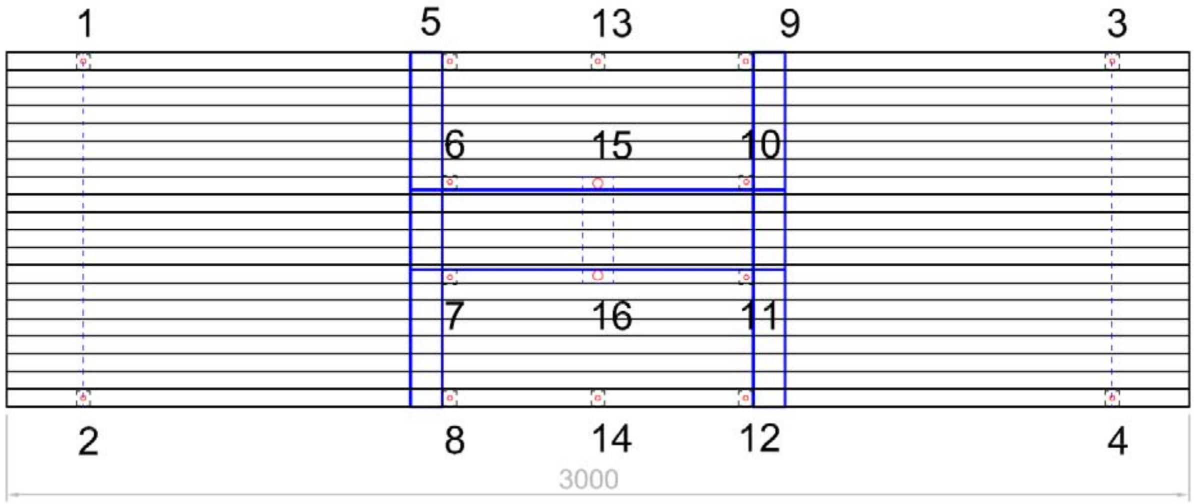


Figure A.7 *LVDT´s placed to control the deflections during the lab test. 16 LVDT´s were used, 4 of 5mm on the corners (1, 2, 3, 4), 2 of them of 12.5mm (7, 8), 6 of 25mm distributed symmetrically in the space between the spreader beams (5, 6, 9, 10, 11,12), and 4 LVDT´s of 50mm (13, 14, 15, 16).*

A.2.1 Beam without butt joints

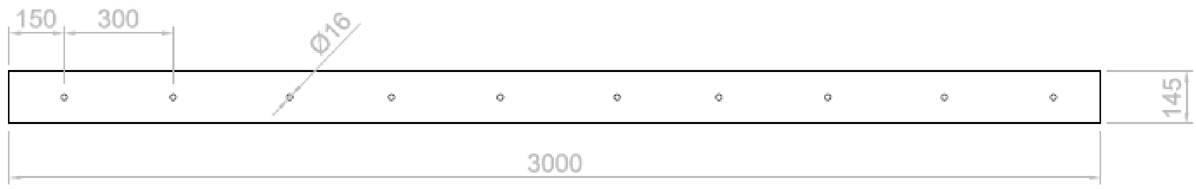


Figure A.8 *Plan view of the 20 laminations that composed the stress timber deck tested. The dimension of each lamination and the distances between the holes for steel bars are described also. The dimension of each lamination was 3000x45x145mm and the steel bars used were 10 of Ø16mm.*

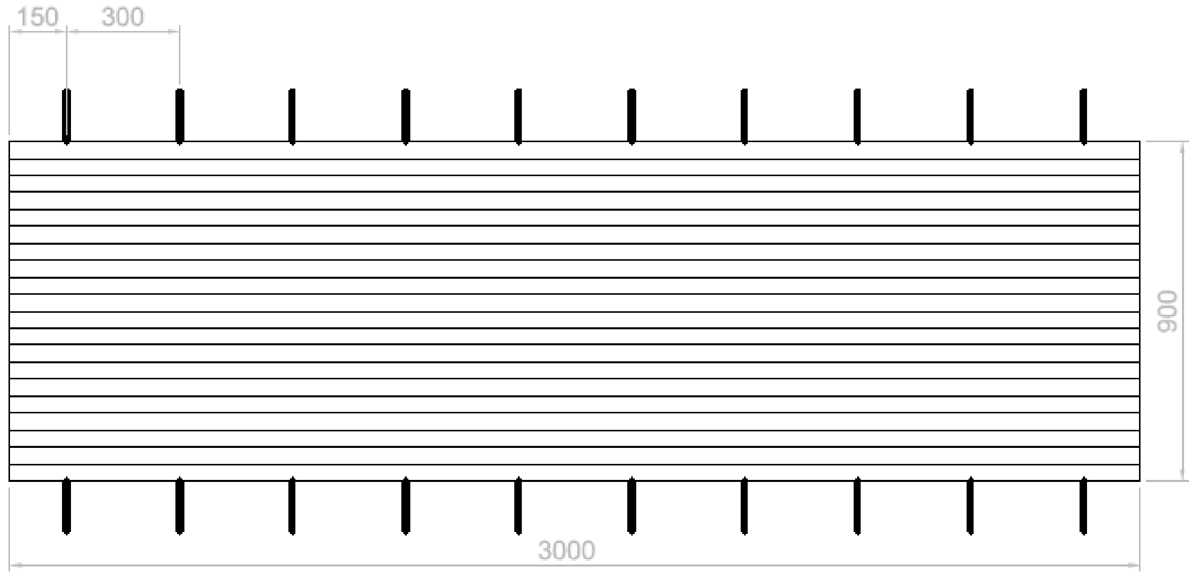


Figure A.9 Beam composed by 20 laminations of timber C24 with 3000mm of length and 10 steel bars to pre-stress them

A.2.2 Beam with a 1 in 8 butt joint pattern

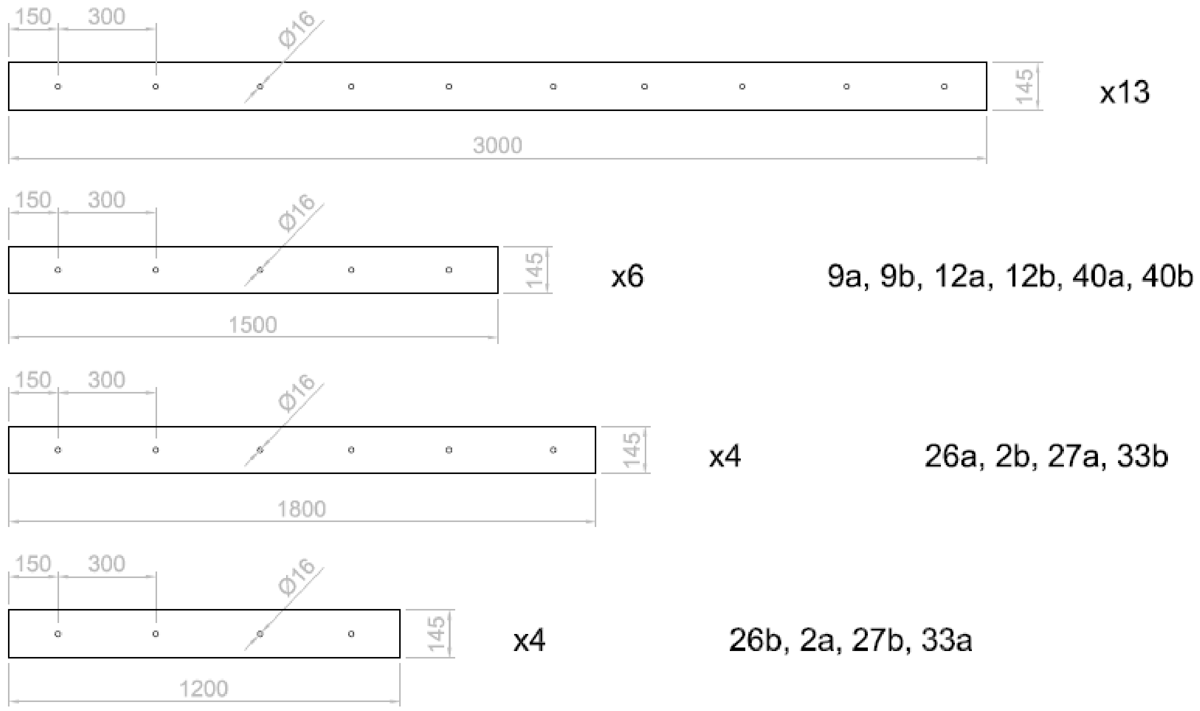


Figure A.10 Laminations used for the bending test of the stress laminated timber deck with cuts to get 1 in 8 butt joint pattern. The dimension of each lamination is shown with their respective holes diameter. The final beam in this test was composed by 13 laminations of 3000mm, 6 laminations of 1500mm, 4 laminations of 1800mm, 4 laminations of 1400mm and 10 steel bars of Ø16mm.

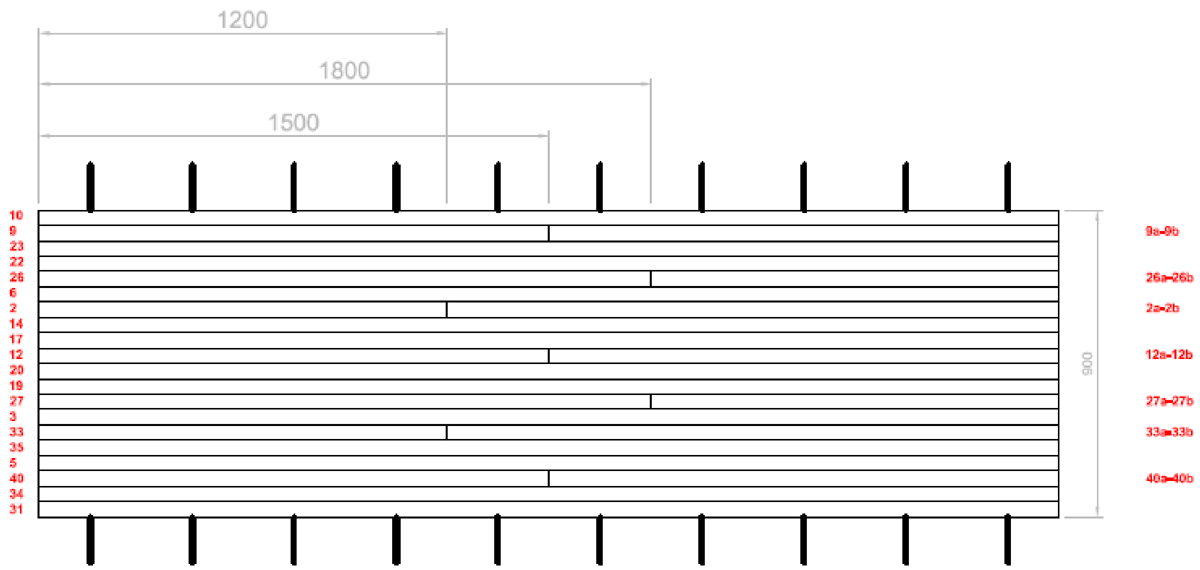


Figure A.11 Deck composed by 20 laminations with 7 cuts and their place in relation to the other ones, as well as the distance between the butt joints (1 in 8 in this case). The laminations are numbered from 1 to 20 in the left side. On the right part of the sketch are shown the number of laminations where there are cuts, with a letter. It means that if these cut laminations have two lengths, the left part is called a, and the right one is denominated b

A.2.3 Beam with a 1 in 4 butt joint pattern

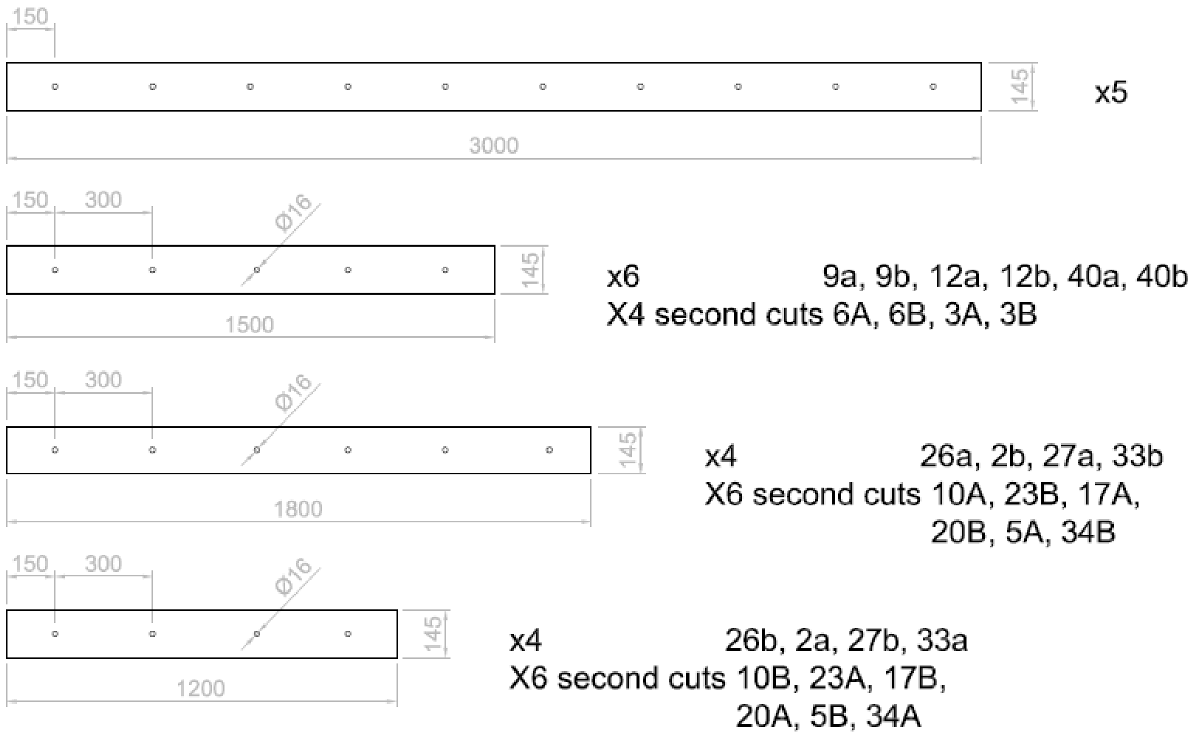


Figure A.12 Laminations that were used for the third bending test done increasing the numbers of cuts to get 1 in 4 butt joints pattern. The dimensions of each different laminations size are shown with their respective holes diameter in each one. The final deck in this test was composed by 5 laminations of 3000mm, 10 laminations of 1500mm, 10 laminations of 1800mm, 8 laminations of 1400mm and 10 steel bars of Ø16mm as well as in the previous bending test with 1 in 8 butt joints. Measurements are given in mm in the drawings.

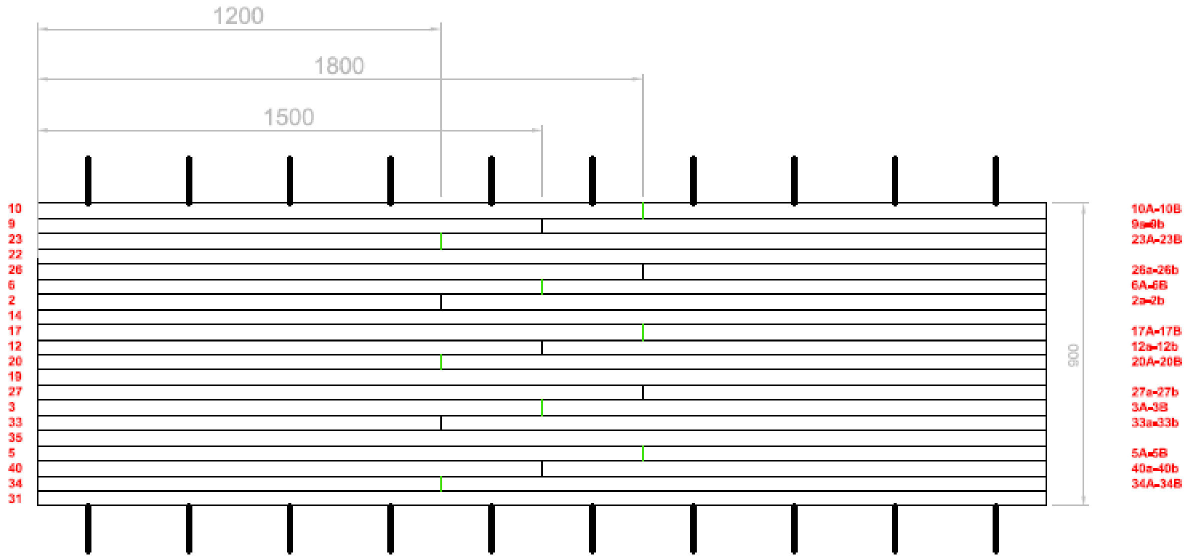


Figure A.13 *Deck composed by 20 laminations with 15 cuts and the distances between the butt joints (1 in 4 in this case). The green cuts shown in the figure are the second cuts made to get 1 in 4 pattern after the previous bending test done with the cuts represented with black colour. On the right the number of laminations are shown where there are cuts, with a letter, these laminations have two lengths, the left part is called a, and the right one is denominated b.*

A.3 Materials for assembly and pre-stress procedure

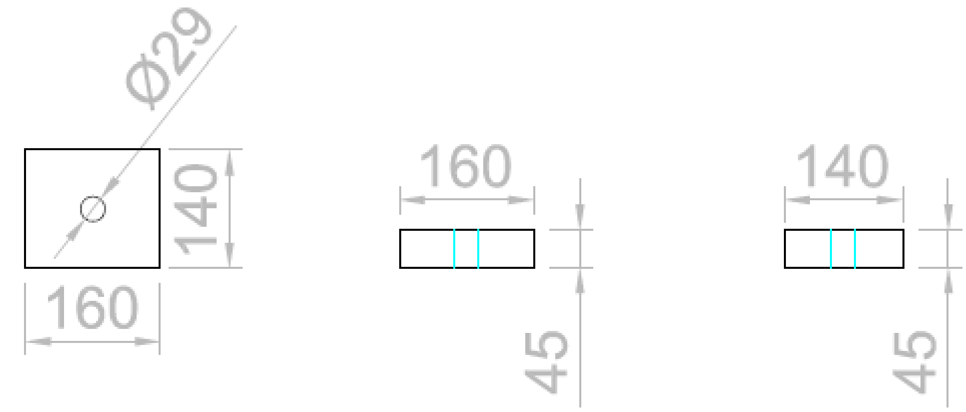


Figure A.14 Dimensions of the plan view as well as the front and sides view of the hardwood plate used between timber C24 of the decks (in the largest and in the shortest beams) and the steel bars used to pre-stress.

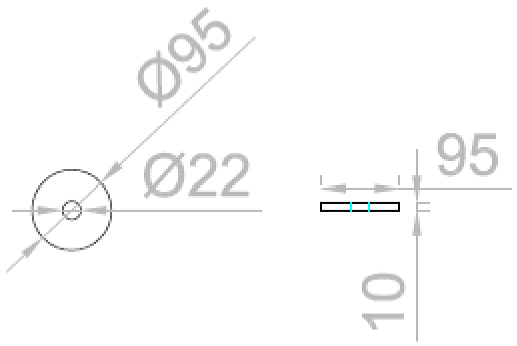


Figure A.15 Dimensions of steel washers used the deck. This washer was used between the steel nut and the plate of high density timber. The diameter of the steel washers was 95mm with a hole of 22mm. The thickness of the steel washers was 10mm as is represented in the right figure.

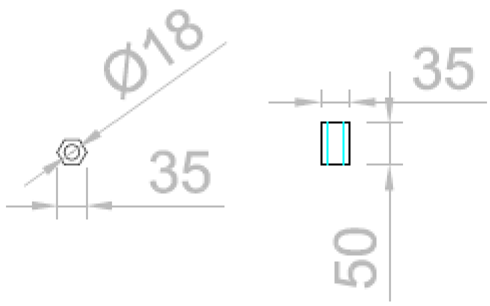


Figure A.16 *Dimensions for the steel nuts is presented. The diameter of these was 35mm but the hole had 18mm to introduce the steel bars of 16mm as diameter. The right sketch shows the front view of the nuts. The height can be identified to 50mm of each one.*

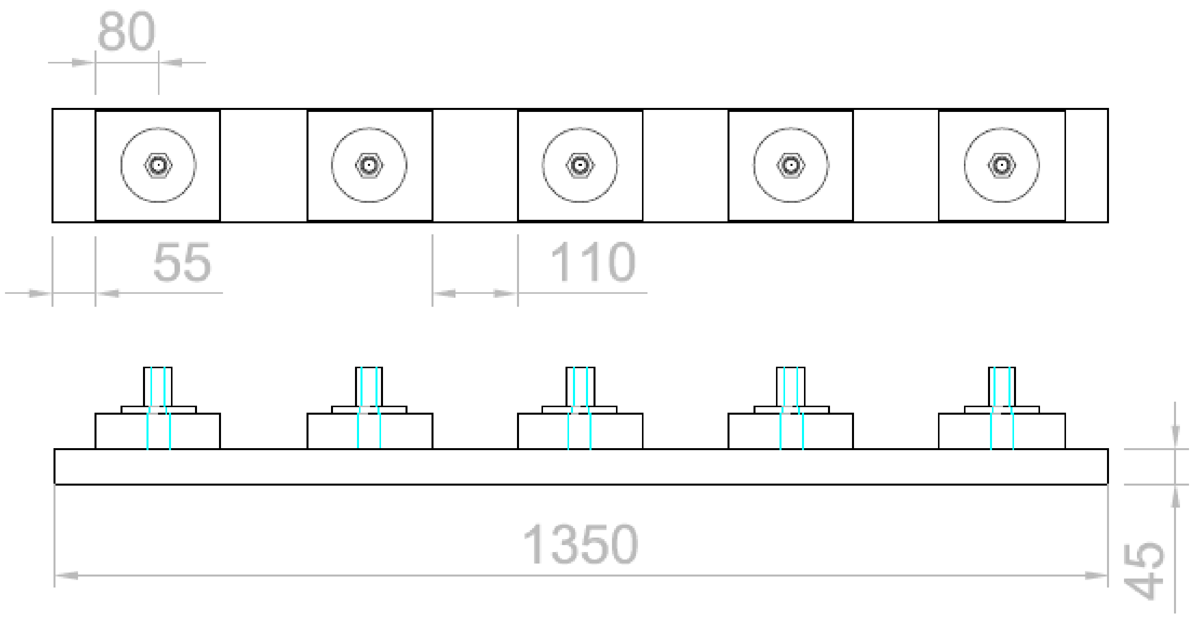


Figure A.17 *Front view and side view of one lamination used to construct the square deck with the set of fasteners needed. For the pre-stress procedure of that deck were 10 hardwood plates, 10 steel washers and 10 steel nuts necessary.*

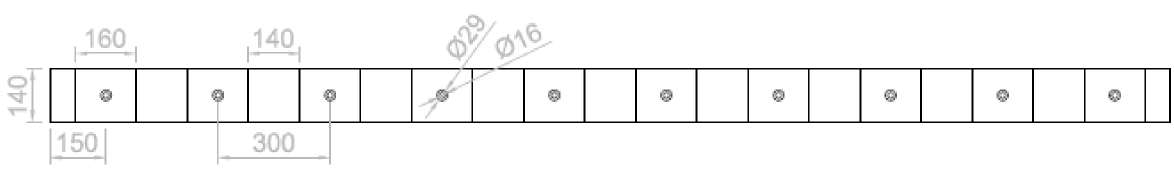


Figure A.18 Front view of the longer laminations with all the dimensions of the distance between hardwood plates, steel washers and hexagonal steel nuts used. To assembly and pre-stress the 20 laminations were 20 pieces of each fastener (one set for each steel bar of 16mm) used. Measurements are given in mm.

A.4 Shear test

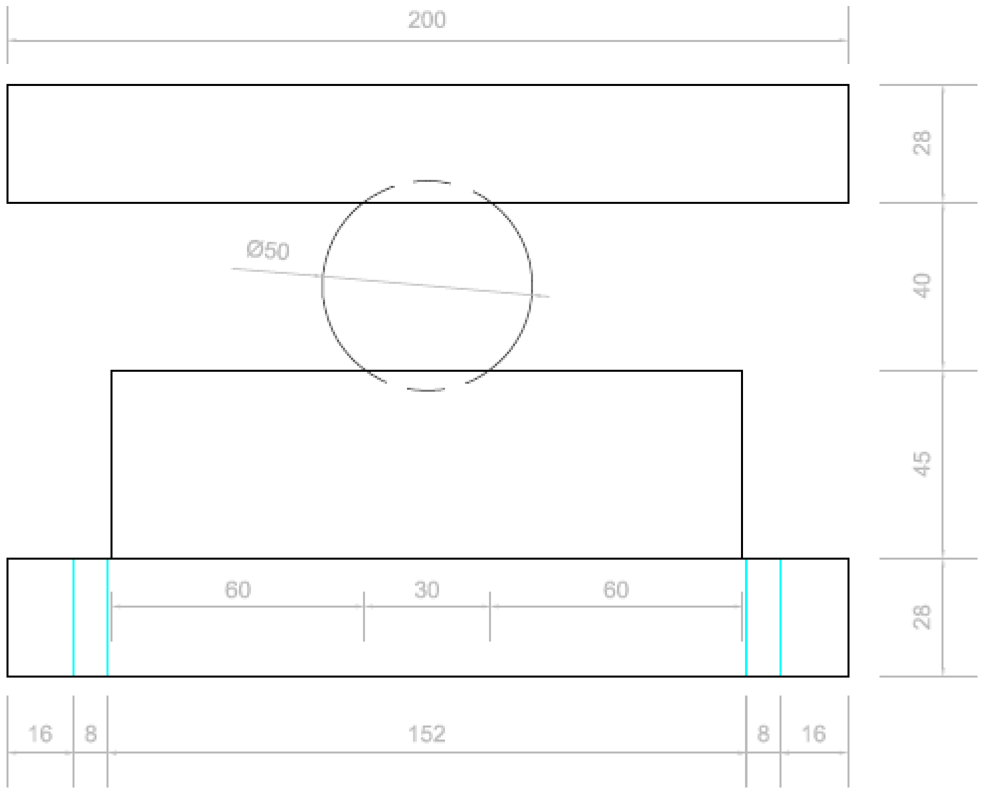


Figure A.19 Ball bearing support used during the shear test to achieve the required support condition. The detailing of the top and bottom plates is described in the following figures.

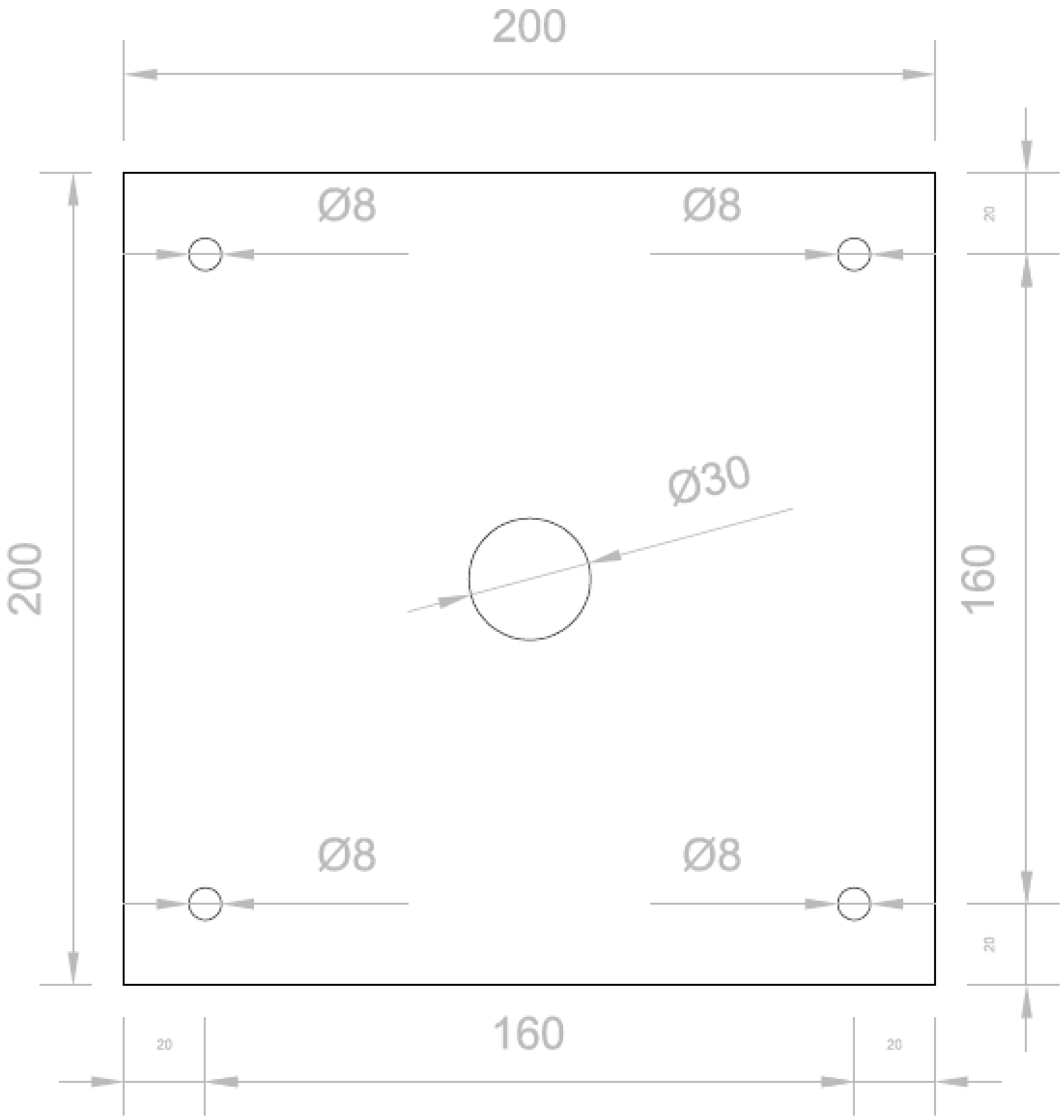


Figure A.20 Top plate detail used for the ball bearing support.

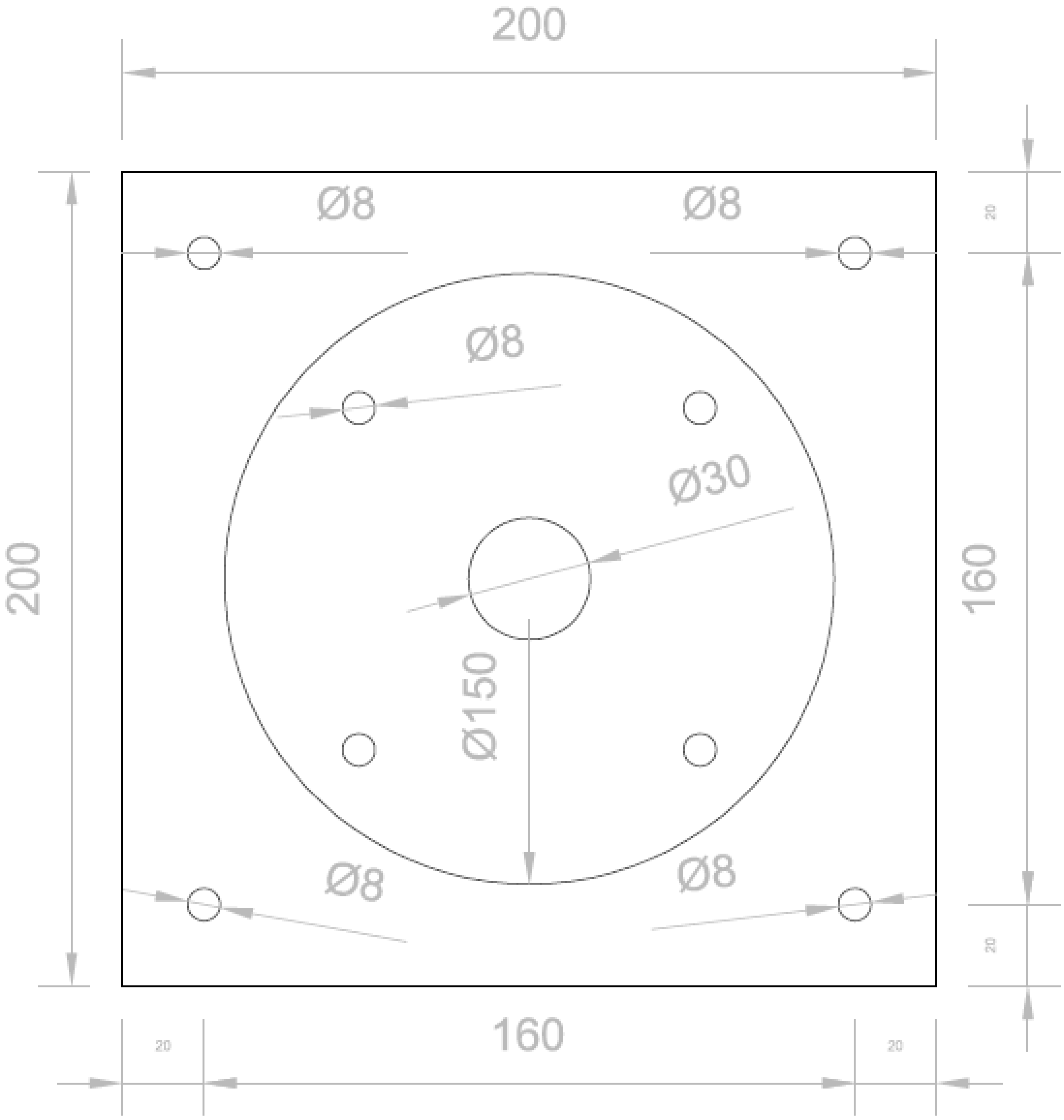


Figure A.21 Bottom plate of the ball bearing support

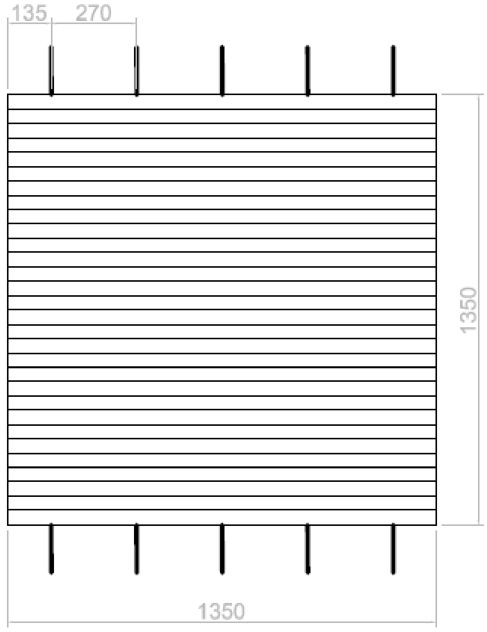


Figure A.22 The square timber deck used to determine the shear MOE was constructed with 30 laminations of 1350x45x145mm. It had 5 steel bars of 16mm separated with 270mm between them and 135mm from the edges

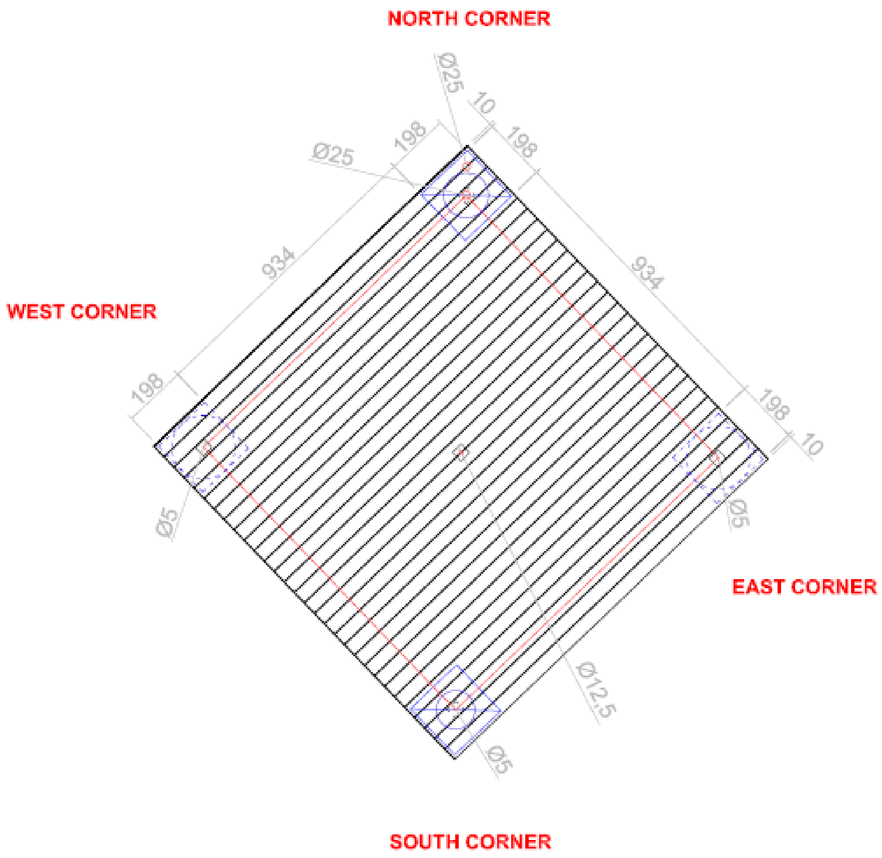


Figure A.23 The LVDT´s places used to measure the deflections during the lab tests. 3 LVDT´s of 5mm were used in the corners: west, east and south; 1 of 12.5mm was used in the middle of the square deck, and 1 of 25mm were used at the corner where the ball bearing support over the deck is situated, the north corner, as appear in the sketch. The east and west corners were supported by ball bearing supports under the deck

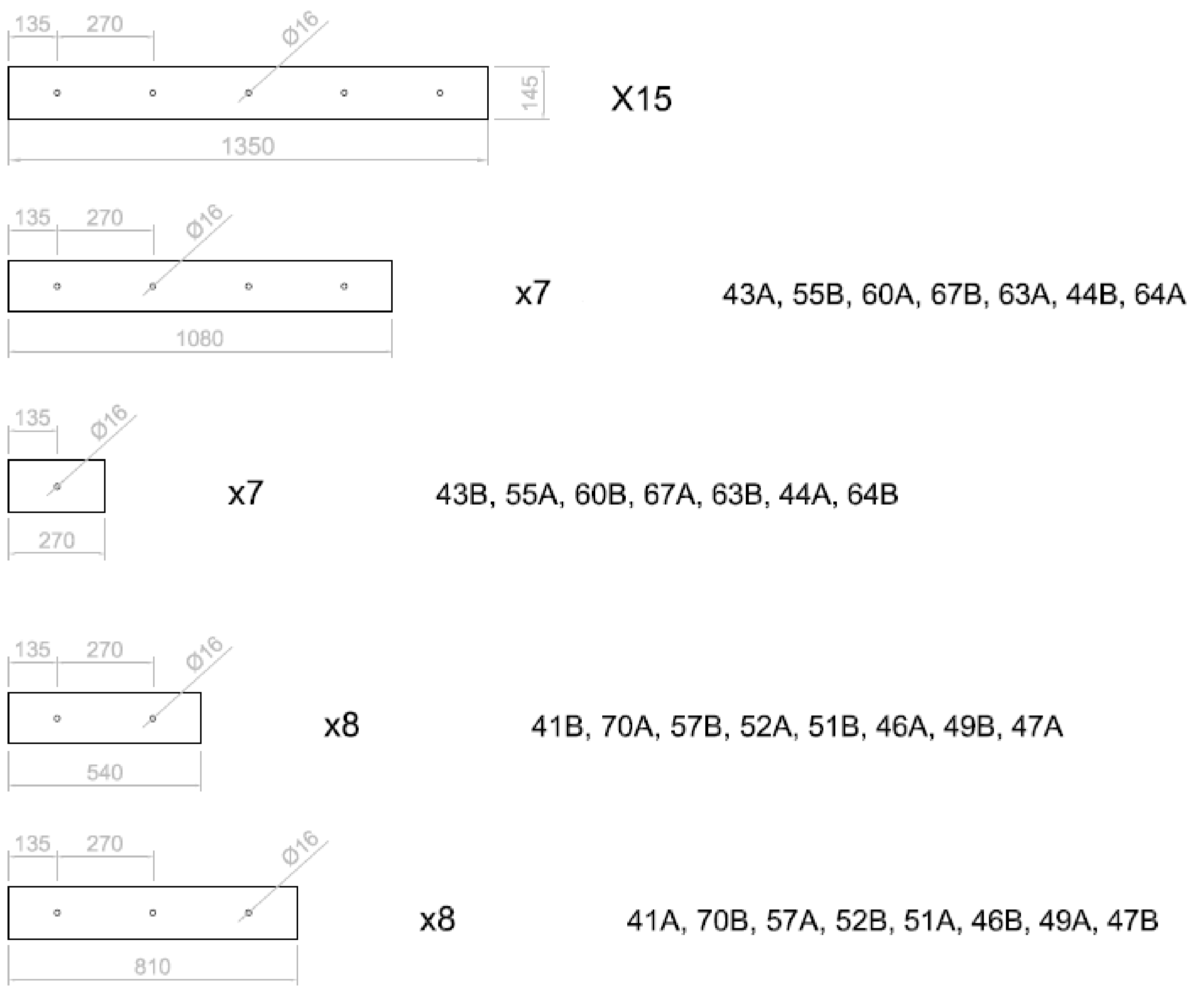


Figure A.24 Laminations used for the second in plane shear test made in the square SLTD. These is the cuts used to get a 1 in 8 butt joint pattern. The dimensions of each lamination used to compose the deck are showed. It means that the deck in this test was composed by 15 laminations of 1350mm, 7 laminations of 1080mm, 7 laminations of 270mm, 8 laminations of 540mm, 8 laminations of 810mm and 5 steel bars of Ø16mm

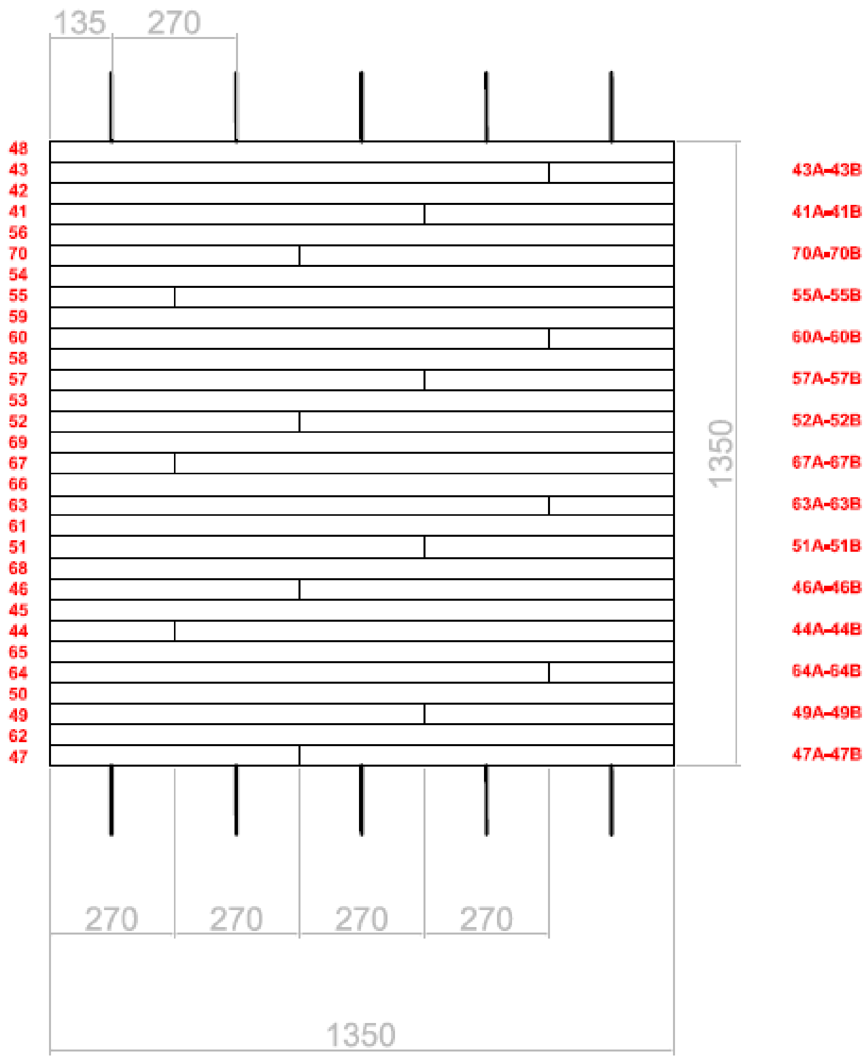


Figure A.25 *square SLTD composed by 30 laminations with 15 cuts and their place in relation to the others, as well as the distance between the butt joints. The laminations are numbered from 40 to 70, but they are not in the increased order. They were placed randomly in the deck in order to make a deck with behaviour as close to reality as possible. On the left each laminations number is shown, while in the right side are shown the number of the laminations with any cut with the letters A or B. With the letter A, are the left sides, and with the letter B are the right part of the same lamination.*

Appendix B – Results from tests

B.1 Results from dynamic test

B.1.1 Laminations of 3m length

Table B.1 *Results from dynamic test for laminations of length 3000mm. The results from the test were the eigenfrequency and from this value, the weight and the geometry longitudinal MOE was calculated for each lamination. The longitudinal MOE range varied within a span between 8000-15000MPa and the mean value was 12269MPa. The characteristic value of C24 timber is 11000MPa.*

Lamination number	Eigenfrequency 1 Hz	Eigenfrequency 2 Hz	Mean frequency Hz	Weight kg	Density kg/m3	E modulus MPa
1	880	880	880	9,3	477	13251
2	849	849	849	9,2	464	12049
3	834	834	834	9,8	503	12598
4	763	763	763	8,2	414	8676
5	824	824	824	10,4	534	13015
6	849	849	849	9,6	496	12863
7	911	911	911	7,4	376	11222
8	793	793	793	9,5	494	11193
9	937	937	937	9,2	477	15096
10	900	900	900	10,1	515	15021
11	778	778	778	7,8	401	8729
12	804	804	804	9,0	459	10663
13	885	885	885	9,5	486	13708
14	798	798	798	8,4	428	9798
15	788	788	788	11,3	584	13045
16	865	865	865	8,3	418	11271
17	804	804	804	9,0	457	10625
18	849	849	849	10,5	527	13695
19	788	788	788	8,9	454	10153
20	880	880	880	7,4	376	10479
21	829	829	829	10,8	552	13650
22	865	865	865	9,2	465	12515
23	875	875	875	10,4	525	14462
24	865	865	865	7,7	384	10340
25	849	849	849	8,1	411	10666
26	854	854	854	9,5	488	12802
27	768	768	768	10,0	509	10810
28	890	890	890	9,5	482	13741
29	804	804	804	9,8	501	11657
30	900	900	900	9,2	467	13614
31	922	922	922	10,1	509	15578
32	819	819	819	9,4	475	11448
33	880	880	880	8,4	429	11962
34	957	952	955	9,2	469	15373

35	860	860	860	9,0	458	12179
36	824	824	824	9,6	486	11876
37	849	849	849	8,8	449	11658
38	875	875	875	9,5	477	13151
39	809	809	809	9,4	485	11432
40	905	905	905	9,6	498	14686

B.1.2 Laminations of 1.35m length

Table B.2 *Results from dynamic test for laminations of length 1350mm. The resulting longitudinal MOE was in the same range as for the longer laminations (3000mm). The mean value was 11880MPa which was a bit lower than for the long laminations, but still larger than the characteristic value (11000MPa).The variation in the results was due to varying properties of different specimens. Since timber is a natural material the variations between the specimens could be large, which could be seen in the table of results.*

Lamination number	Eigenfrequency 1 [Hz]	Eigenfrequency 2 [Hz]	Mean frequency [Hz]	Weight [kg]	Density [kg/m^3]	E modulus [MPa]
41	1822	1822	1822	4,3	483	11670
42	1893	1893	1893	4,4	507	13269
43	1934	1934	1934	4,2	484	13197
44	1842	1842	1842	3,7	422	10440
45	1959	1959	1959	3,6	398	11132
46	1893	1893	1893	4,2	475	12383
47	1944	1944	1944	3,6	403	11113
48	2173	2173	2173	4,3	479	16498
49	1893	1893	1893	3,3	365	9530
50	1674	1674	1674	4,8	540	11014
51	1934	1939	1937	4,7	536	14662
52	1918	1918	1918	4,4	515	13786
53	2000	2000	2000	4,4	510	14850
54	1949	1949	1949	3,6	407	11258
55	1979	1979	1979	4,3	483	13770
56	1674	1669	1672	3,9	431	8779
57	1959	1959	1959	4,4	512	14311
58	1756	1751	1754	3,9	450	10089
59	1842	1842	1842	4,3	505	12493
60	1903	1903	1903	3,9	435	11497
61	1761	1761	1761	4,1	455	10277
62	1812	1812	1812	3,8	432	10346
63	1710	1771	1741	4,1	461	10173
64	1974	1974	1974	3,3	372	10548
65	1705	1705	1705	4,3	478	10107
66	1959	1959	1959	3,7	414	11558
67	1842	1847	1845	4,5	503	12486
68	1837	1837	1837	3,9	430	10567
69	1725	1725	1725	4,3	481	10423
70	2015	2015	2015	4,2	479	14182

B.2 Measured moisture content in timber before testing

The moisture content was measured in four points in each lamination for the deck of 3000mm long and in three points for the shortest laminations. The moisture was measured close to the both ends of the lamination, in the middle of the lamination, and on one of the narrow edges (top edge) of each one. The mean value of each lamination was a mean value of these values. For this measure, a number of laminations were randomly chosen to measure the moisture content. The moisture content was only controlled, and not included in the analyses of the results, so therefore we did not had to measure all laminations. The results seemed reasonable but were a bit higher than expected. The normal value of moisture content in construction timber was 12% and should not exceed 20%. The moisture was measured after the drilling of the timber but before the testing of the SLTD. The timber had been subjected to an indoor climate in a bit more than one week when the test was made.

Table B.3 Moisture content in the 3000mm long laminations. The moisture content varied from 15-18%. The mean value in the table was the average value from the four measure points in the beam. The laminations that were measured were randomly chosen and the results were supposed to give a rough but representative moisture content of all the laminations.

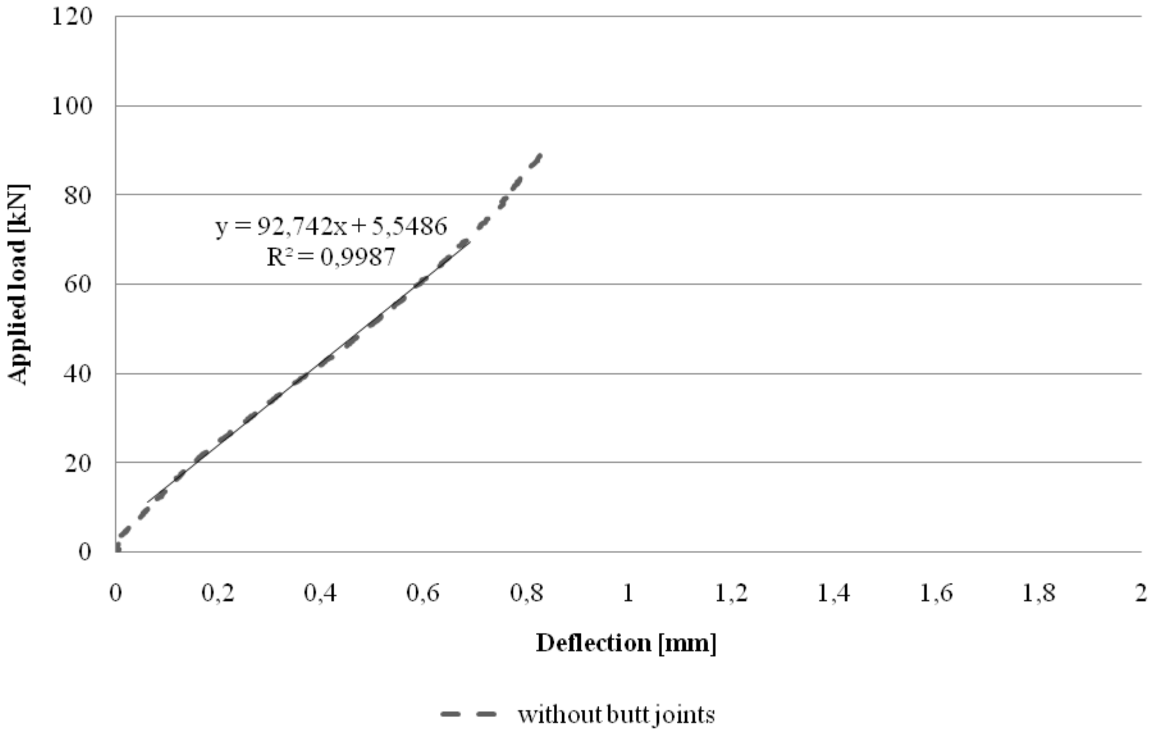
Lamination Number	Edge 1 %	Edge 2 %	Middle of lamination %	Top edge %	Mean value %
1	16,1	16,6	16,4	15,2	16,1
4	15,2	15	15,3	14,6	15,0
8	16,2	16	16	16,4	16,2
12	15,2	15,3	15,1	14,8	15,1
16	16,2	16,5	16,3	15,8	16,2
18	14,1	17,5	13	16	15,2
26	16,1	16,5	16,6	16	16,3
28	18,6	17,8	18,8	18,5	18,4
34	15,9	16,2	16	15,8	16,0
36	18,1	19,2	19,5	15,7	18,1

Table B.4 Moisture content in the short laminations. The moisture content varied from 13-17%. In some laminations there was lower moisture content in the end of the lamination than in the middle.

Lamination number	Edge 1 [%]	Edge 2 [%]	Middle of lamination [%]	Top edge [%]	Mean value [%]
41	16,2	16,2	16,7	16,7	16,5
47	13,3	13,6	15,3	15,3	14,4
50	16,3	14,6	18,7	16,8	16,6
56	13,5	14	14,2	13,8	13,9
62	13,4	14,7	16,5	15,6	15,1
68	16,7	15,2	17,9	16,6	16,6

B.3 Results from bending tests

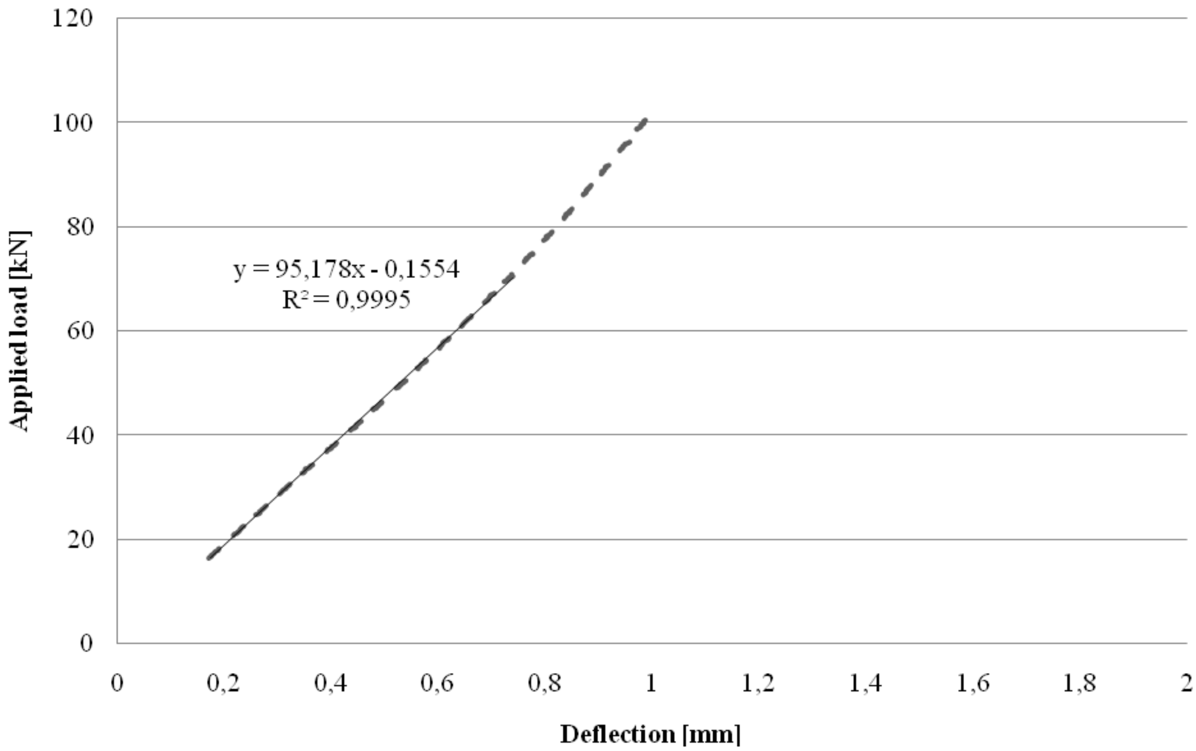
B.3.1 SLTD without butt joints and with 900kPa pre-stress level



Deck without butt joints					
Pre-stress level	Distance load-support	Distance between LVDT´s	MOI	Relation increment applied load/increment deflection	Longitudinal MOE
σ (MPa)	a (mm)	L (mm)	I (mm^4)	F_2-F_1/w_2-w_1 (kN/mm)	Ex (GPa)
0.91	870	720	2.286×10^8	92.742	12.739

Figure B.1 Relationship between load and deflection in the middle of the SLTD without butt joints when a pre-stress level of 900kPa was applied. Only a selected range of data was used to identify the trend line needed to find the load-deflection behaviour of the SLTD. The pre-stress value from the table is different than the aimed of 900kPa because of the real conditions in the lab.

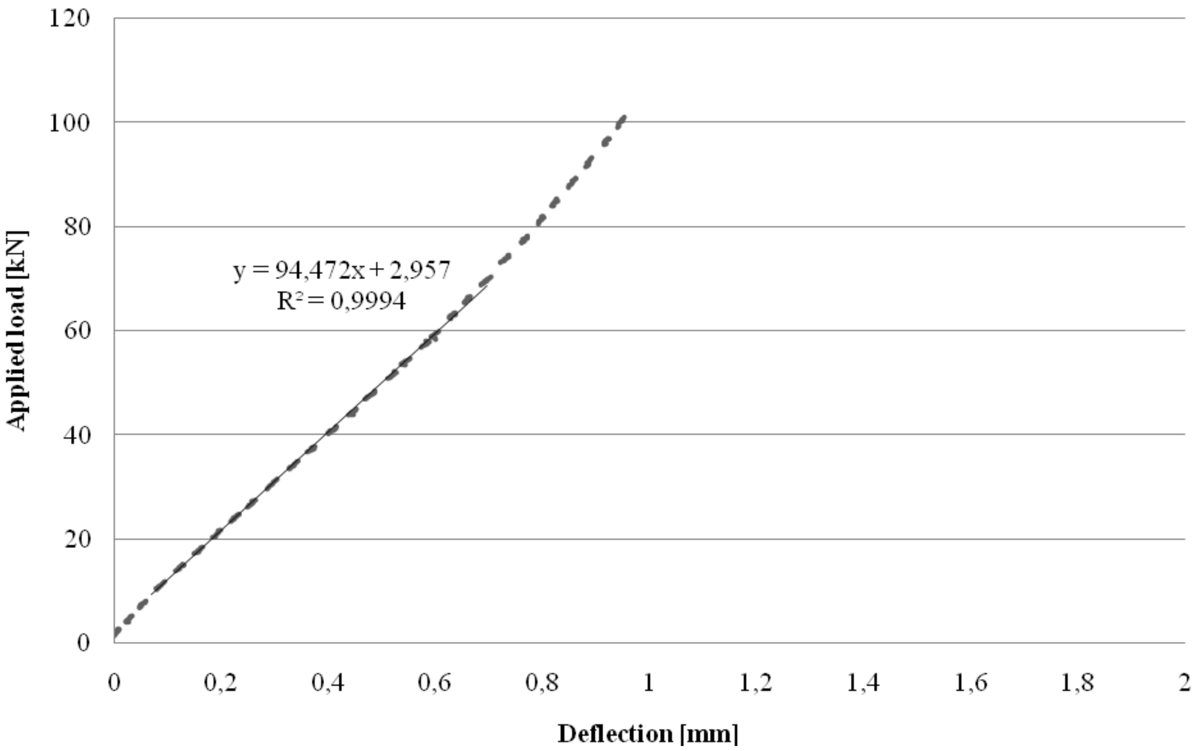
B.3.2 SLTD without butt joints and with 600kPa pre-stress level



Deck without butt joints					
Pre-stress level	Distance load-support	Distance between LVDT´s	MOI	Relation increment applied load/increment deflection	Longitudinal MOE
σ (MPa)	a (mm)	L (mm)	I (mm^4)	F_2-F_1/w_2-w_1 (kN/mm)	Ex (GPa)
0.63	870	720	$2.286x10^8$	95.178	13.074

Figure B.2 Relationship between load and deflection in the middle of the SLTD without butt joints when a pre-stress level of 600kPa was applied. Only a selected range of data was used to identify the trend line needed to find the load-deflection behaviour of the SLTD. The pre-stress value from the table is different than the aimed of 600kPa because of the real conditions in the lab. In this test one of the LVDTs were broken and did not show any deflection before an applied load of 16kN. This led to a strange value of deflection up to this load because it was only based on the deflection on one side.

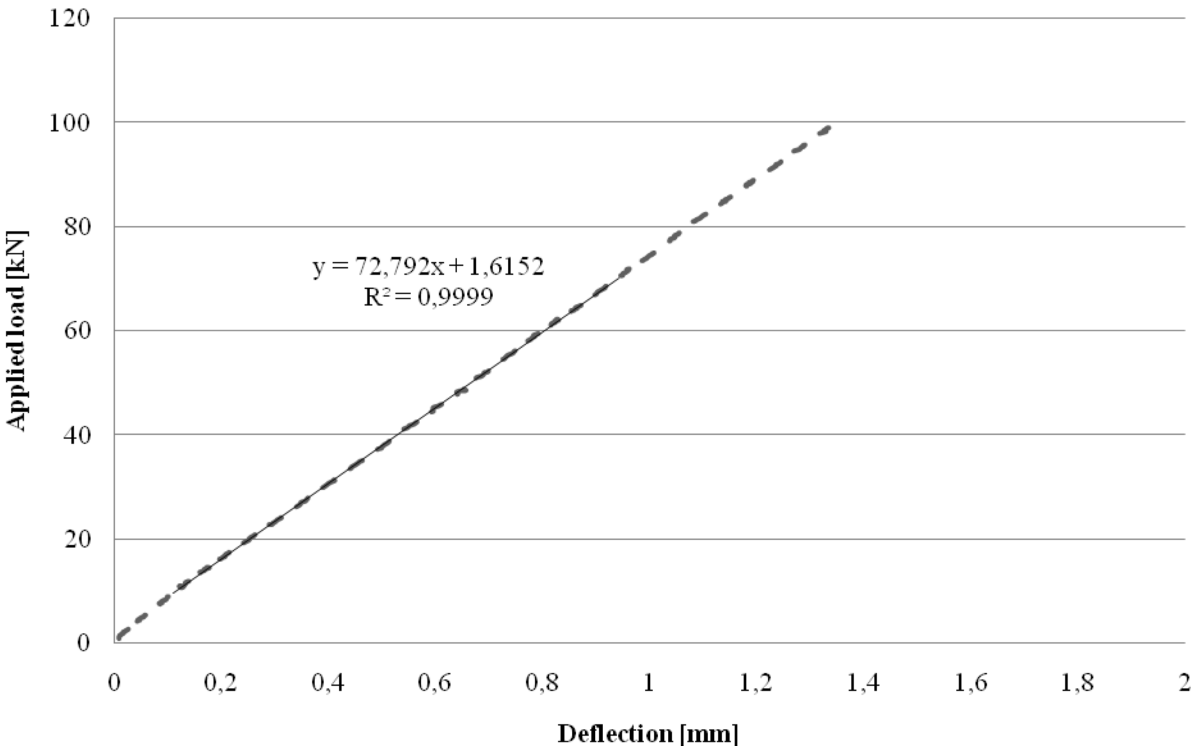
B.3.3 SLTD without butt joints and with 300kPa pre-stress level



Deck without butt joints					
Pre-stress level	Distance load-support	Distance between LVDT´s	MOI	Relation increment applied load/increment deflection	Longitudinal MOE
σ (MPa)	a (mm)	L (mm)	I (mm^4)	F_2-F_1/w_2-w_1 (kN/mm)	Ex (GPa)
0.30	870	720	2.286×10^8	94.472	12.713

Figure B.3 *Relationship between load and deflection in the middle of the SLTD without butt joints when a pre-stress level of 300kPa was applied. Only a selected range of data was used to identify the trend line needed to find the load-deflection behaviour of the SLTD.*

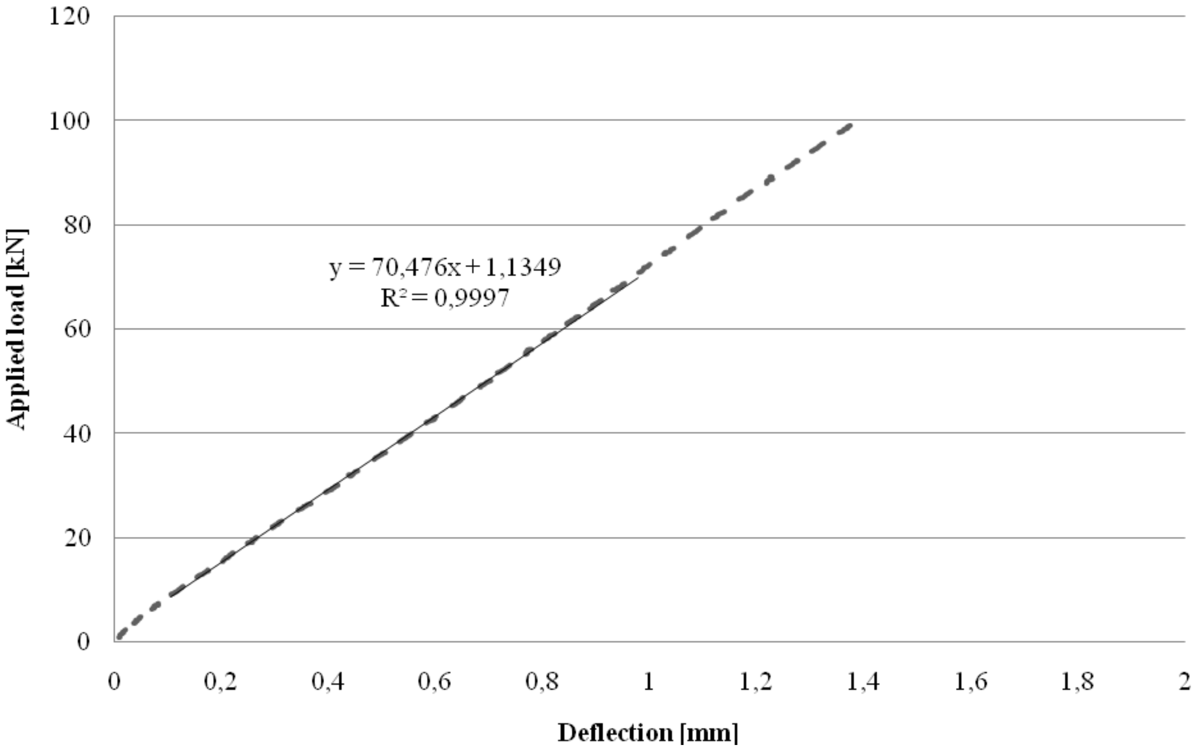
B.3.4 SLTD with a 1 in 8 butt joint pattern and 900kPa pre-stress level



Deck with 1 in 8 butt joints pattern					
Pre-stress level	Distance load-support	Distance between LVDT´s	MOI	Relation increment applied load/increment deflection	Longitudinal MOE
σ (MPa)	a (mm)	L (mm)	I (mm^4)	F_2-F_1/w_2-w_1 (kN/mm)	Ex (GPa)
0.86	870	720	$2.286x10^8$	72.792	9.999

Figure B.4 Relationship between load and deflection in the middle of the SLTD with1 in 8 butt joints when a pre-stress level of 900kPa was applied. Only a selected range of data was used to identify the trend line needed to find the load-deflection behaviour of the SLTD. The pre-stress value from the table is different than the aimed of 900kPa because of the real conditions in the lab.

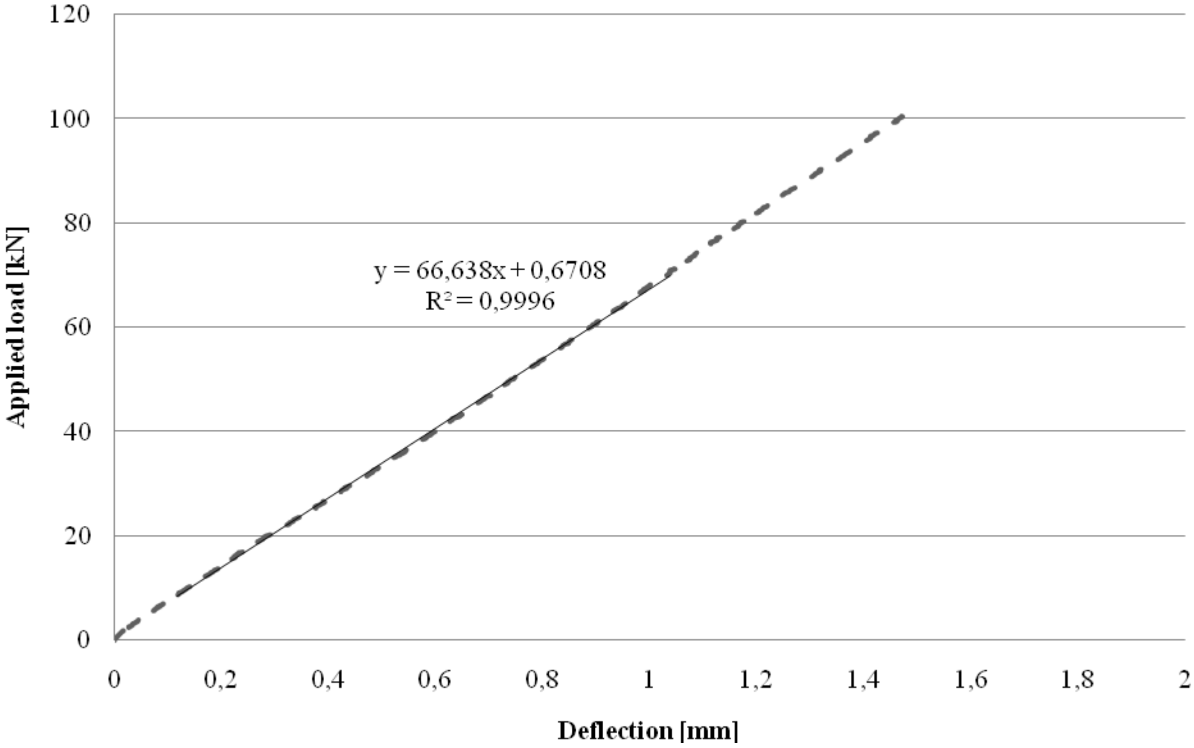
B.3.5 SLTD with a 1 in 8 butt joint pattern and 600kPa pre-stress level



Deck with 1 in 8 butt joints pattern					
Pre-stress level	Distance load-support	Distance between LVDT´s	MOI	Relation increment applied load/increment deflection	Longitudinal MOE
σ (MPa)	a (mm)	L (mm)	I (mm^4)	F_2-F_1/w_2-w_1 (kN/mm)	Ex (GPa)
0.60	870	720	2.286×10^8	70.476	9.681

Figure B.5 Relationship between load and deflection in the middle of the SLTD with 1 in 8 butt joints when a pre-stress level of 600kPa was applied. Only a selected range of data was used to identify the trend line needed to find the load-deflection behaviour of the SLTD.

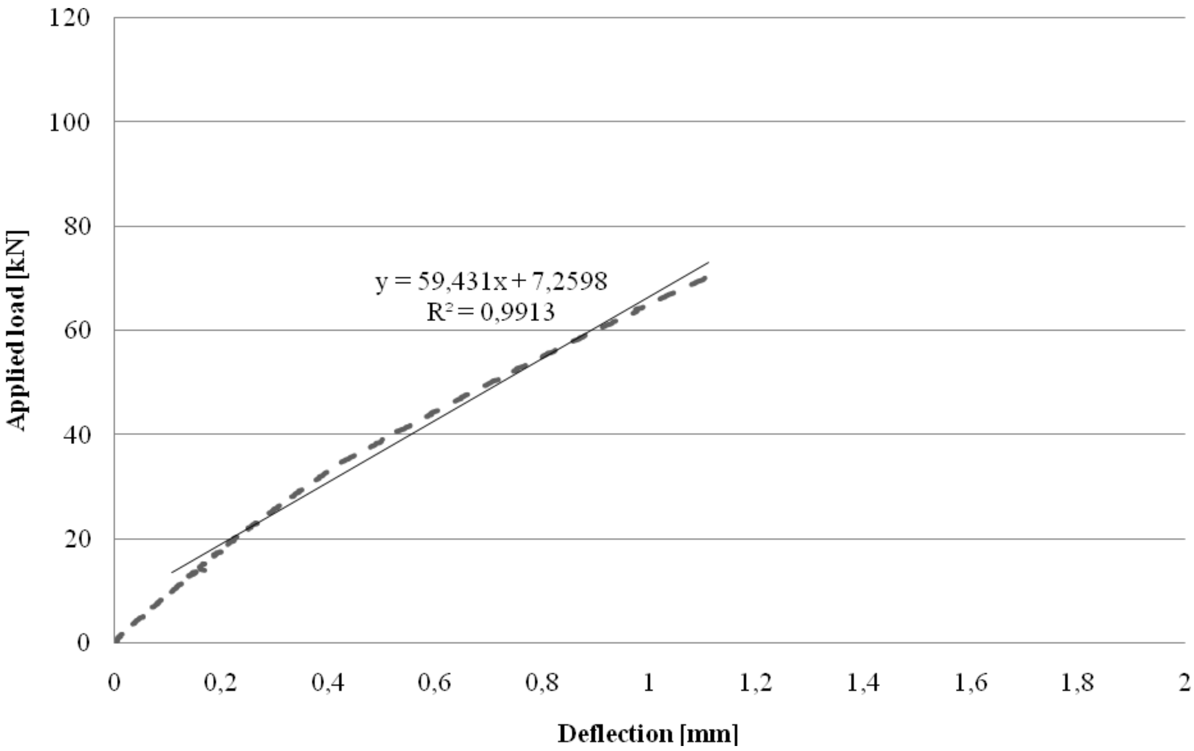
B.3.6 SLTD with a 1 in 8 butt joint pattern and 300kPa pre-stress level



Deck with 1 in 8 butt joints pattern					
Pre-stress level	Distance load-support	Distance between LVDT´s	MOI	Relation increment applied load/increment deflection	Longitudinal MOE
σ (MPa)	a (mm)	L (mm)	I (mm^4)	F_2-F_1/w_2-w_1 (kN/mm)	Ex (GPa)
0.31	870	720	2.286×10^8	64.638	9.153

Figure B.6 Relationship between load and deflection in the middle of the SLTD with 1 in 8 butt joints when a pre-stress level of 300kPa was applied. Only a selected range of data was used to identify the trend line needed to find the load-deflection behaviour of the SLTD. The pre-stress value from the table is different than the aimed of 300kPa because of the real conditions in the lab.

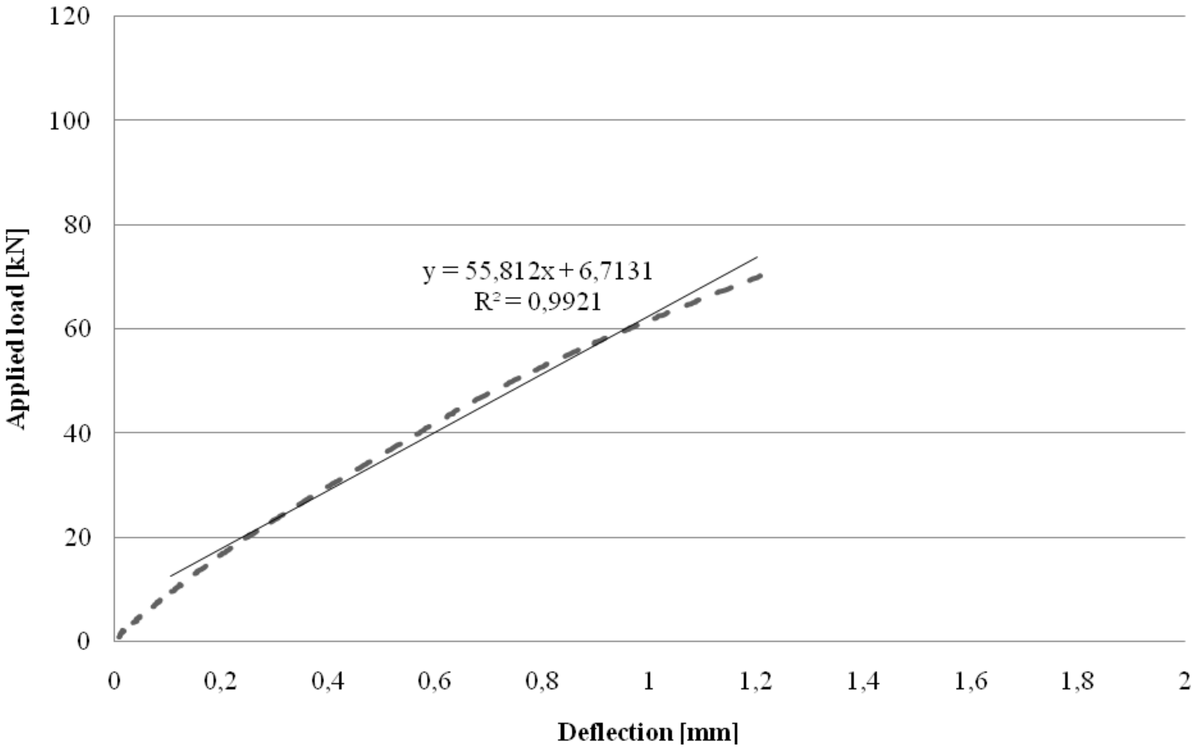
B.3.7 SLTD with a 1 in 4 butt joint pattern and 900kPa pre-stress level



Deck with 1 in 4 butt joints pattern					
Pre-stress level	Distance load-support	Distance between LVDT´s	MOI	Relation increment applied load/increment deflection	Longitudinal MOE
σ (MPa)	a (mm)	L (mm)	I (mm^4)	F_2-F_1/w_2-w_1 (kN/mm)	Ex (GPa)
0.88	870	720	$2.286x10^8$	59.431	8.163

Figure B.7 Relationship between load and deflection in the middle of the SLTD with 1 in 4 butt joints when a pre-stress level of 900kPa was applied. Only a selected range of data was used to identify the trend line needed to find the load-deflection behaviour of the SLTD. The pre-stress value from the table is different than the aimed of 900kPa because of the real conditions in the lab.

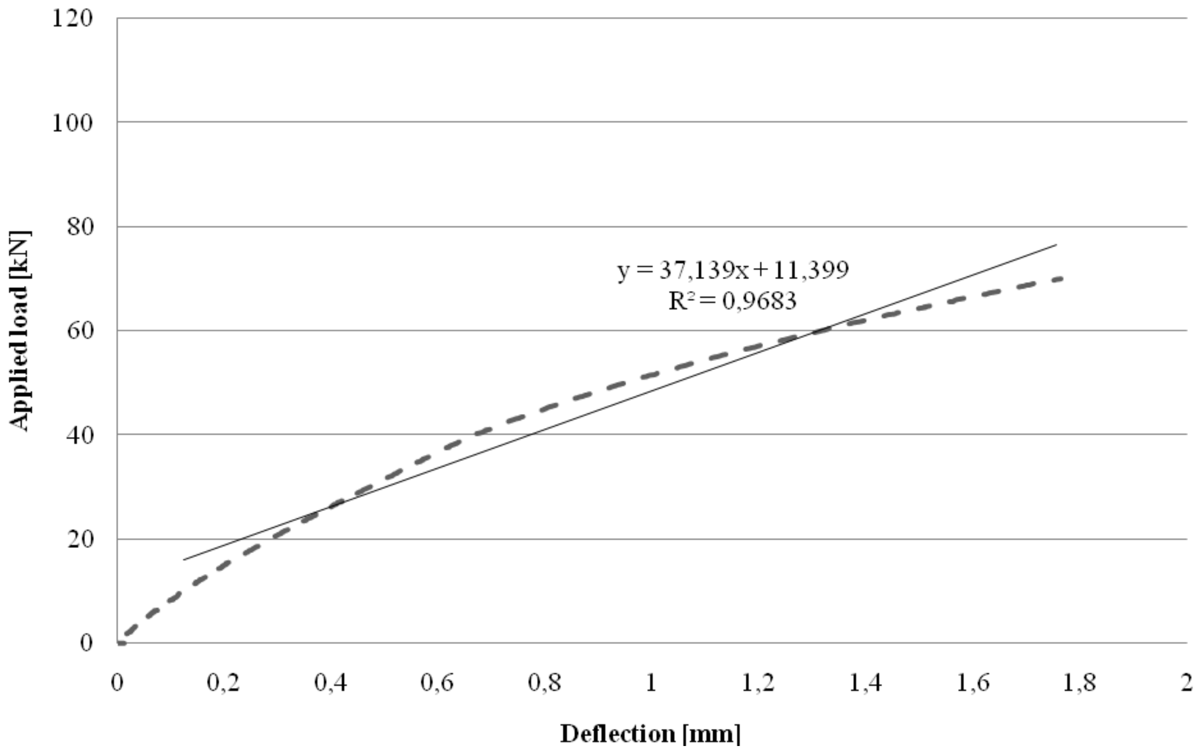
B.3.8 SLTD with a 1 in 4 butt joint pattern and 600kPa pre-stress level



Deck with1 in 4 butt joints pattern					
Pre-stress level	Distance load-support	Distance between LVDT´s	MOI	Relation increment applied load/increment deflection	Longitudinal MOE
σ (MPa)	a (mm)	L (mm)	I (mm^4)	F_2-F_1/w_2-w_1 (kN/mm)	Ex (GPa)
0.61	870	720	2.286×10^8	55.812	8.163

Figure B.8 Relationship between load and deflection in the middle of the SLTD with 1 in 4 butt joints when a pre-stress level of 600kPa was applied. Only a selected range of data was used to identify the trend line needed to find the load-deflection behaviour of the SLTD. The pre-stress value from the table is different than the aimed of 600kPa because of the real conditions in the lab.

B.3.9 SLTD with a 1 in 4 butt joint pattern and 300kPa pre-stress level

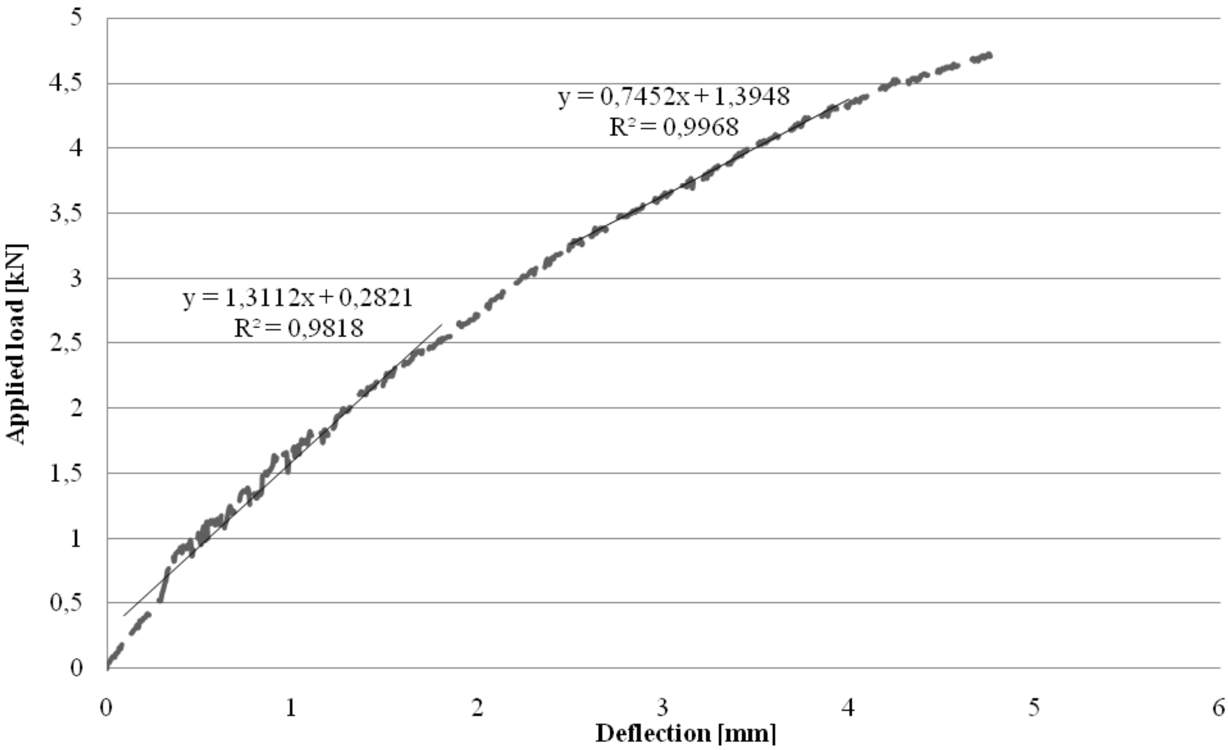


Deck with 1 in 4 butt joints pattern					
Pre-stress level	Distance load-support	Distance between LVDT´s	MOI	Relation increment applied load/increment deflection	Longitudinal MOE
σ (MPa)	a (mm)	L (mm)	I (mm^4)	F_2-F_1/w_2-w_1 (kN/mm)	Ex (GPa)
0.31	870	720	$2.286x10^8$	37.139	5.101

Figure B.9 Relationship between load and deflection in the middle of the SLTD with 1 in 4 butt joints when a pre-stress level of 300kPa was applied. Only a selected range of data was used to identify the trend line needed to find the load-deflection behaviour of the SLTD. The pre-stress value from the table is different than the aimed of 300kPa because of the real conditions in the lab.

B.4 Results from in plane shear tests

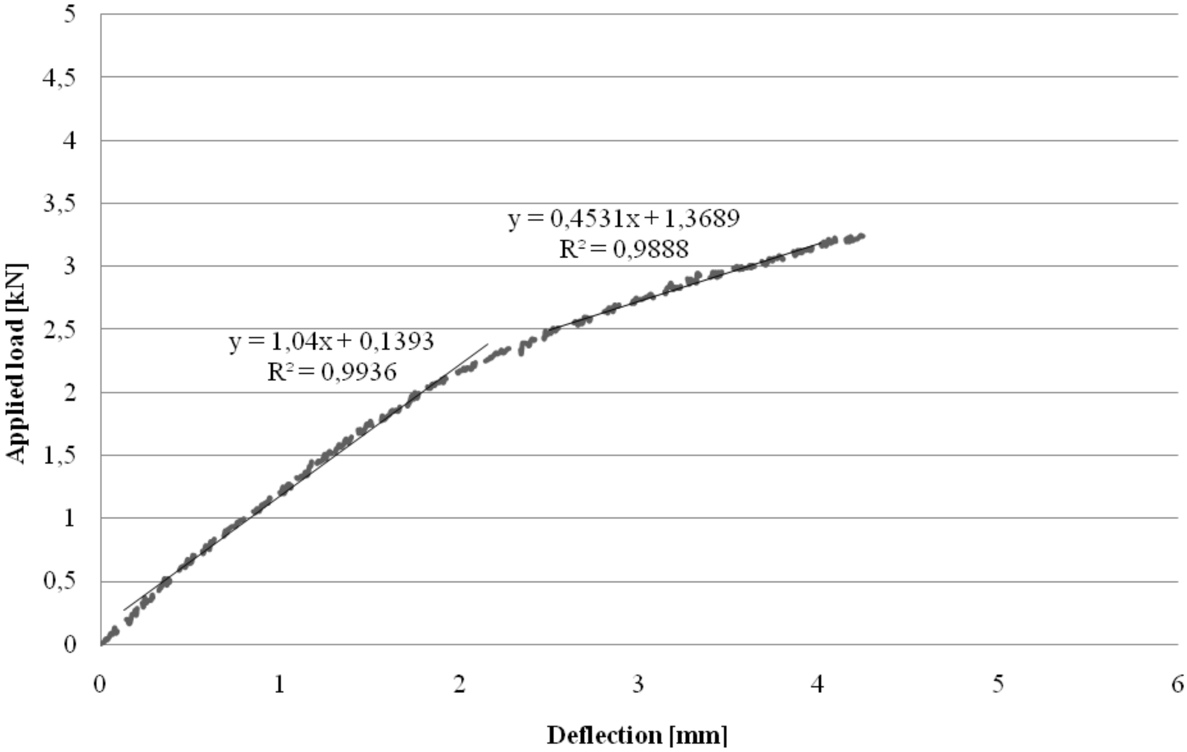
B.4.1 SLTD without butt joints and with 900kPa pre-stress level



Deck without butt joints				
Pre-stress level 900kPa	Dimensions		Results	
Timber stress	Length width	Height	Load/Deflection	Shear modulus
σ (MPa)	L (mm)	h (mm)	P/w (kN/mm)	Gxy (MPa)
0.90	1350	145	1.311	413

Figure B.10 Deflection in the middle of the SLTD without butt joints when a pre-stress level of 900kPa was applied. Only a selected range of data was used to identify the trend line needed to find the load-deflection relation taking into account the expected value for slip between laminations.

B.4.2 SLTD without butt joints and with 600kPa pre-stress level



Deck without butt joints				
Pre-stress level 600kPa	Dimensions		Results	
Timber stress	Length width	Height	Load/Deflection	Shear modulus
σ (MPa)	L (mm)	h (mm)	P/w (kN/mm)	Gxy (MPa)
0.57	1350	145	1.040	328

Figure B.11 Deflection in the middle of the SLTD without butt joints when a pre-stress level of 600kPa was applied. Only a selected range of data was used to identify the trend line needed to find the load-deflection relation taking into account the expected value for slip between laminations. The pre-stress value from the table is different than the aimed of 600kPa because of the real conditions for the SLTD.

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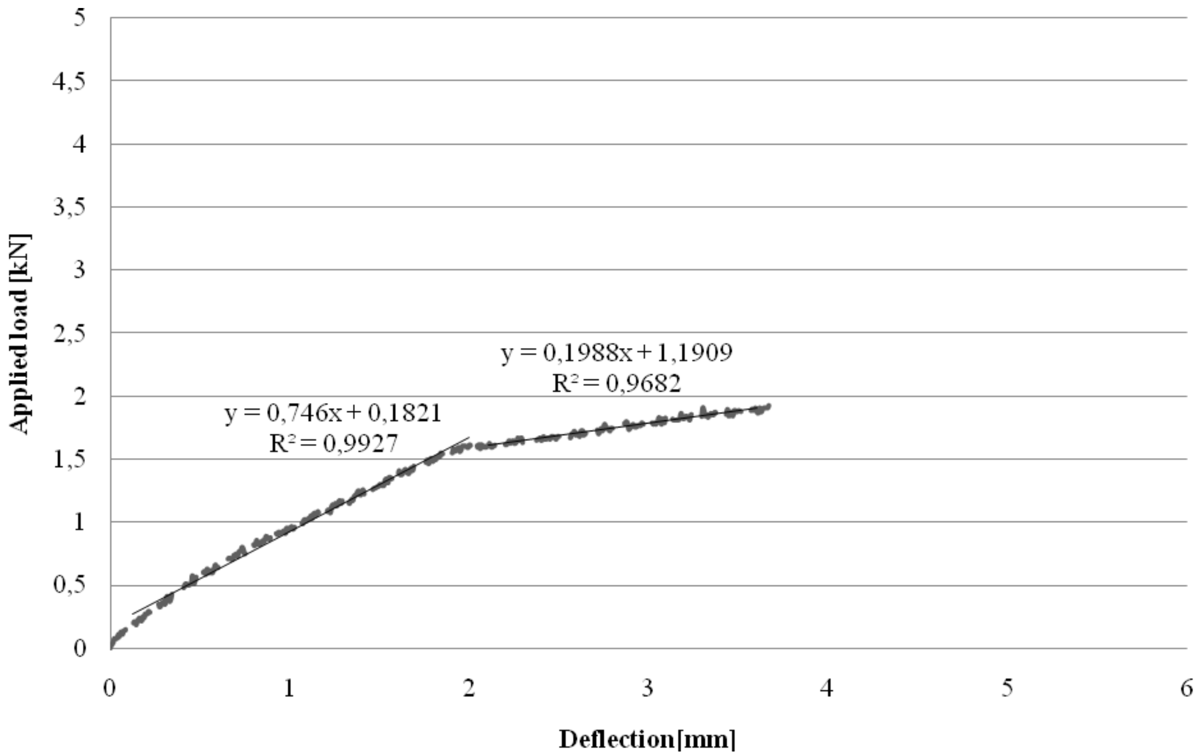
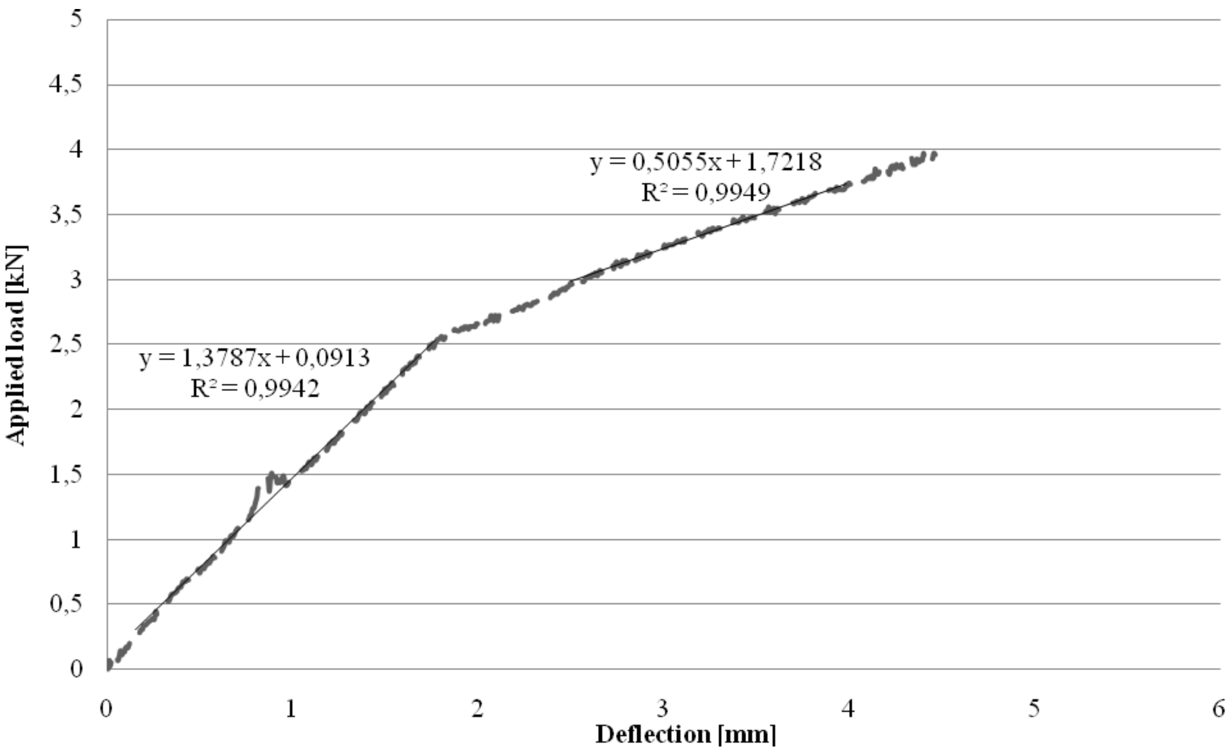
B.4.3 SLTD without butt joints and with 300kPa pre-stress level



Deck without butt joints				
Pre-stress level 300kPa	Dimensions		Results	
Timber stress	Length width	Height	Load/Deflection	Shear modulus
σ (MPa)	L (mm)	h (mm)	P/w (kN/mm)	Gxy (MPa)
0.26	1350	145	0.746	235

Figure B.12 Deflection in the middle of the SLTD without butt joints when a pre-stress level of 300kPa was applied. Only a selected range of data was used to identify the trend line needed to find the load-deflection relation taking into account the expected value for slip between laminations. The pre-stress value from the table is different than the aimed of 300kPa because of the real conditions for the SLTD.

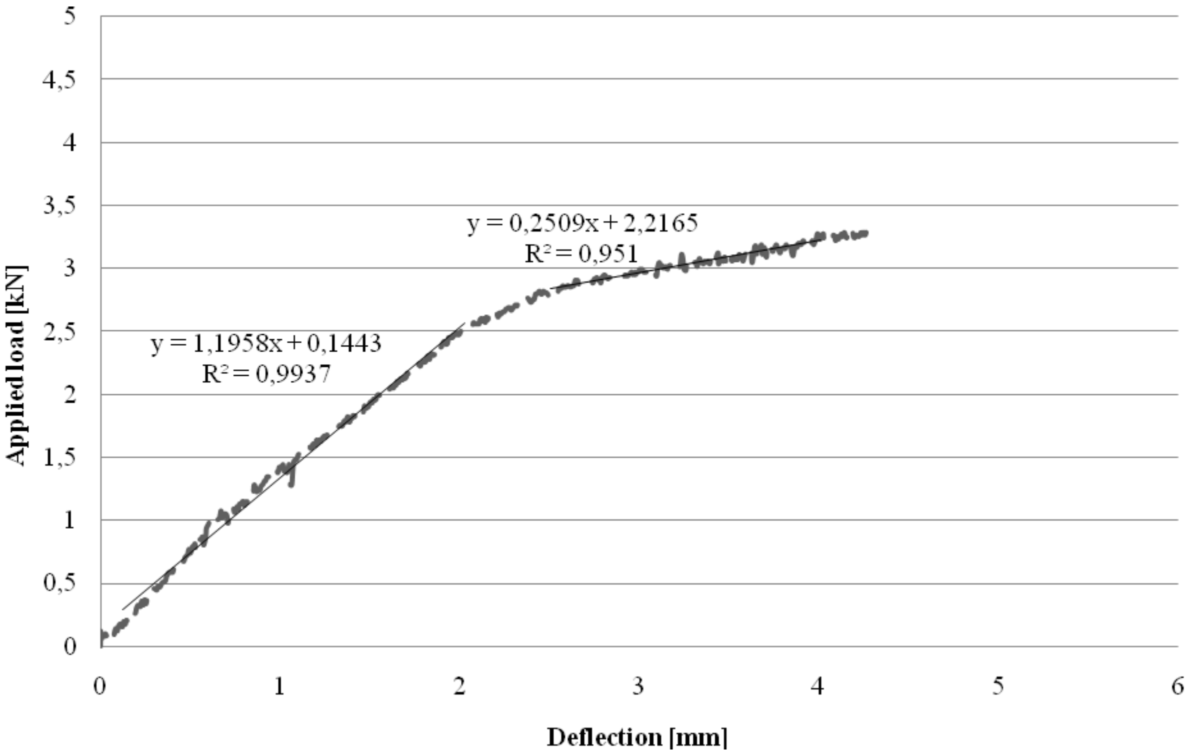
B.4.4 SLTD with butt joints and 900kPa pre-stress level



Deck with butt joints				
Pre-stress level 900kPa	Dimensions		Results	
Timber stress	Length width	Height	Load/Deflection	Shear modulus
σ (MPa)	L (mm)	h (mm)	P/w (kN/mm)	Gxy (MPa)
0.92	1350	145	1.379	435

Figure B.13 Deflection in the middle of the SLTD with butt joints when a pre-stress level of 900kPa was applied. Only a selected range of data was used to identify the trend line needed to find the load-deflection relation taking into account the expected value for slip between laminations. The pre-stress value from the table is different than the aimed of 900kPa because of the real conditions for the SLTD

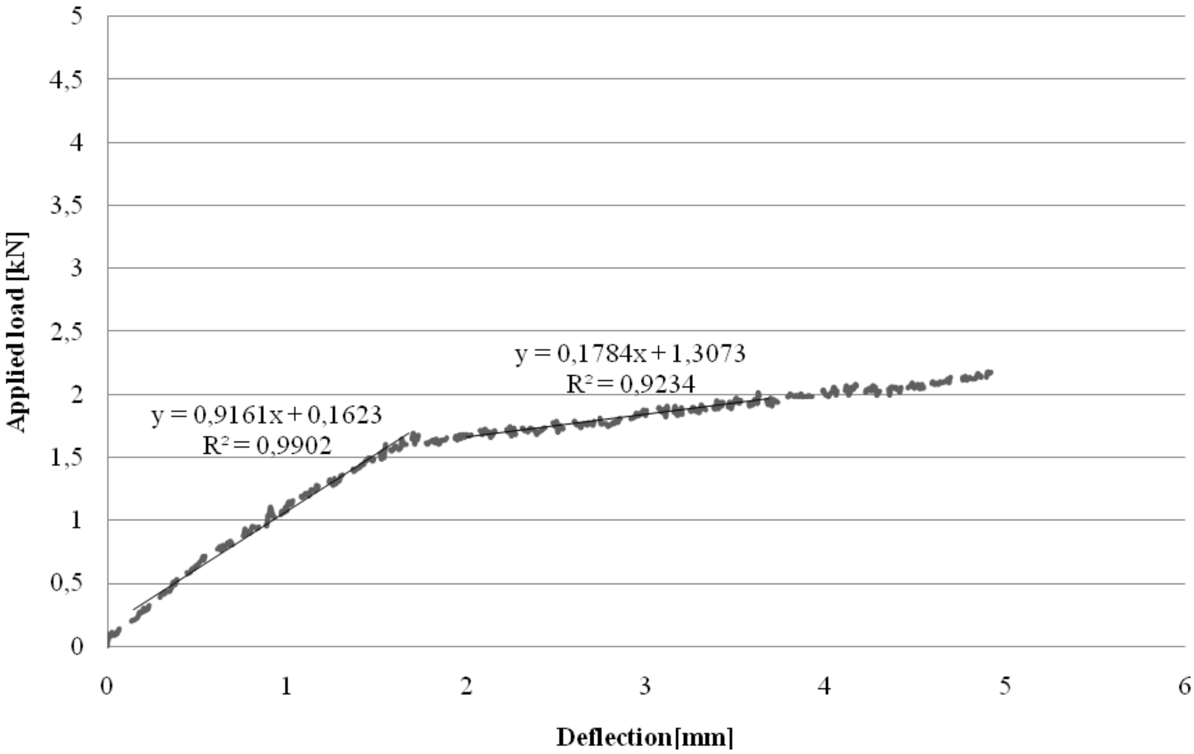
B.4.5 SLTD with butt joints and 600kPa pre-stress level



Deck with butt joints				
Pre-stress level 600kPa	Dimensions		Results	
Timber stress	Length width	Height	Load/Deflection	Shear modulus
σ (MPa)	L (mm)	h (mm)	P/w (kN/mm)	Gxy (MPa)
0.64	1350	145	1.196	377

Figure B.14 Deflection in the middle of the SLTD with butt joints when a pre-stress level of 600kPa was applied. Only a selected range of data was used to identify the trend line needed to find the load-deflection relation taking into account the expected value for slip between laminations. The pre-stress value from the table is different than the aimed of 600kPa because of the real conditions for the SLTD

B.4.6 SLTD with butt joints and 300kPa pre-stress level



Deck with butt joints				
Pre-stress level 300kPa	Dimensions		Results	
Timber stress	Length width	Height	Load/Deflection	Shear modulus
σ (MPa)	L (mm)	h (mm)	P/w (kN/mm)	Gxy (MPa)
0.32	1350	145	0.916	235

Figure B.15 Deflection in the middle of the SLTD with butt joints when a pre-stress level of 300kPa was applied. Only a selected range of data was used to identify the trend line needed to find the load-deflection relation taking into account the expected value for slip between laminations. The pre-stress value from the table is different than the aimed of 300kPa because of the real conditions for the SLTD.

B.5 Control of moisture content after in-plane shear test

Number of lamination	Moisture content in surface (%)
48	11.1
56	10.7
60	10.9
69	11.2
61	11.7
45	11.1
50	12.2
47	10.8
Mean value of the moisture content after the testing: **11.21%**	

Appendix C – Pre-stress levels from lab tests

Pre-stress values in bars were calculated from the strain in each bar. Two strain gauges were applied on each bar, and the strain in each bar was assumed to be the same as the intermediate value from the two strain gauges. The intermediate value of pre-stress in each deck was then calculated as the intermediate value of all bars in the deck.

BENDING TEST WITHOUT BUTT JOINTS

900kPa

	Bar 1		Bar 2		Bar 3		Bar 4		Bar 5		Bar 6		Bar 7		Bar 8		Bar 9		Bar 10	
Gauge number	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B	6A	6B	7A	7B	8A	8B	9A	9B	10A	10B
Strain value	0	1008	1126	1340	1052	1257	985	1147	1147	1000,7	975,3	997,2	1028	1074	1093	1152	1126	1212	1143	892,154
Strain in bar		1008		1233		1154		1066		1073,9		986,2		1051		1122		1169		1017,73
Intermediate value																				1088,17
Prestress between laminations [Pa]																				906227

600kPa

	Bar 1		Bar 2		Bar 3		Bar 4		Bar 5		Bar 6		Bar 7		Bar 8		Bar 9		Bar 10	
Gauge number	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B	6A	6B	7A	7B	8A	8B	9A	9B	10A	10B
Strain value	0	758,5	708,1	897,5	648,1	836,4	708,8	846,2	858,4	743,83	750,4	767,5	690,3	743,4	716,1	761	685,5	752,8	911,2	655,06
Strain in bar		758,5		802,8		742,2		777,5		801,11		758,9		716,9		738,5		719,1		783,143
Intermediate value																				759,879
Prestress between laminations [Pa]																				632827

300kPa

	Bar 1		Bar 2		Bar 3		Bar 4		Bar 5		Bar 6		Bar 7		Bar 8		Bar 9		Bar 10	
Gauge number	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B	6A	6B	7A	7B	8A	8B	9A	9B	10A	10B
Strain value	0	310,7	319,9	398,6	283,7	472	327,5	437,7	408,4	327,87	354,4	396	323,1	366,8	342,2	376,8	328	398,4	503,2	244,425
Strain in bar		310,7		359,2		377,8		382,6		368,14		375,2		345		359,5		363,2		373,837
Intermediate value																				361,533
Prestress between laminations [Pa]																				301085

BENDING TEST WITH 1 IN 8 BUTT JOINT PATTERN

900kPa

	Bar 1		Bar 2		Bar 3		Bar 4		Bar 5		Bar 6		Bar 7		Bar 8		Bar 9		Bar 10	
Gauge number	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B	6A	6B	7A	7B	8A	8B	9A	9B	10A	10B
Strain value	1082	1042	1011	1096	1039	1052	1012	1091	1085	995,32	1105	1066	1018	1072	956,2	1075	1009	1228	1080	1002,15
Strain in bar		1062		1053		1045		1052		1040,3		1086		1045		1015		1118		1040,97
Intermediate value																				1055,77
Prestress between laminations [Pa]																				879244

600kPa

	Bar 1		Bar 2		Bar 3		Bar 4		Bar 5		Bar 6		Bar 7		Bar 8		Bar 9		Bar 10	
Gauge number	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B	6A	6B	7A	7B	8A	8B	9A	9B	10A	10B
Strain value	781,4	704,7	696,3	785	723,5	745,9	662,7	743,8	774,8	709,2	738,2	687,2	724,2	783	687,2	780,9	634,1	799,7	757,3	691,725
Strain in bar		743		740,6		734,7		703,3		742,01		712,7		753,6		734,1		716,9		724,524
Intermediate value																				730,537
Prestress between laminations [Pa]																				608391

300kPa

	Bar 1		Bar 2		Bar 3		Bar 4		Bar 5		Bar 6		Bar 7		Bar 8		Bar 9		Bar 10	
Gauge number	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B	6A	6B	7A	7B	8A	8B	9A	9B	10A	10B
Strain value	356,5	222,7	283,4	391,3	234,6	283,7	391,3	460	398,4	398,62	400,9	354,6	364,6	440,4	342,4	394,1	364,8	501,3	437,3	393,526
Strain in bar		289,6		337,3		259,2		425,6		398,52		377,7		402,5		368,3		433		415,41
Intermediate value																				370,717
Prestress between laminations [Pa]																				308733

BENDING TEST WITH 1 IN 4 BUTT JOINT PATTERN

900kPa

	Bar 1		Bar 2		Bar 3		Bar 4		Bar 5		Bar 6		Bar 7		Bar 8		Bar 9		Bar 10	
Gauge number	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B	6A	6B	7A	7B	8A	8B	9A	9B	10A	10B
Strain value	1069	1079	1112	922	1009	1056	939,1	1020	1078	0	1139	1122	986,1	1045	909,7	1045	935,2	966	1087	1009,48
Strain in bar		1074		1017		1033		979,7		1077,9		1131		1015		977,5		950,6		1048,3
Intermediate value																				1030,42
Prestress between laminations [Pa]																				858131

600kPa

	Bar 1		Bar 2		Bar 3		Bar 4		Bar 5		Bar 6		Bar 7		Bar 8		Bar 9		Bar 10	
Gauge number	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B	6A	6B	7A	7B	8A	8B	9A	9B	10A	10B
Strain value	737,4	712	794	657,8	662,4	714,1	743,4	807,5	743	0	728,4	726,3	697,3	743,8	635,8	746,7	702,6	711,6	801,3	713,725
Strain in bar		724,7		725,9		688,2		775,4		743,05		727,3		720,6		691,2		707,1		757,513
Intermediate value																				726,116
Prestress between laminations [Pa]																				604710

300kPa

	Bar 1		Bar 2		Bar 3		Bar 4		Bar 5		Bar 6		Bar 7		Bar 8		Bar 9		Bar 10	
Gauge number	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B	6A	6B	7A	7B	8A	8B	9A	9B	10A	10B
Strain value	400,4	386,6	422,6	396,2	352	410,8	364,4	323	418	0	381,3	388,8	308,3	379,3	300,8	384,3	350,1	325,3	405,5	329,977
Strain in bar		393,5		409,4		381,4		343,7		417,97		385,1		343,8		342,6		337,7		367,756
Intermediate value																				372,282
Prestress between laminations [Pa]																				310036

SHEAR TEST WITHOUT BUTT JOINTS

300kPa

	Bar 1		Bar 2		Bar 3		Bar 4		Bar 5	
Gauge number	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B
Strain	354,8	274	437,1	295,1	105,1	363,8	146,7	290,6	341,5	195,24
Strain in bar		314,4		366,1		234,5		218,7		268,36
Intermediate value										280,4
Prestress between laminations [Pa]										259456

600kPa

	Bar 1		Bar 2		Bar 3		Bar 4		Bar 5	
Gauge number	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B
Strain	689,7	599,1	771,3	689,9	444,5	688,2	476,5	598,1	709,4	468,43
Strain in bar		644,4		730,6		566,4		537,3		588,93
Intermediate value										613,51
Prestress between laminations [Pa]										567685

900kPa

	Bar 1		Bar 2		Bar 3		Bar 4		Bar 5	
Gauge number	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B
Strain	1037	0	1147	989,6	735,1	1137	891,8	961,7	1060	736,75
Strain in bar		1037		1068		936,1		926,8		898,53
Intermediate value										973,27
Prestress between laminations [Pa]										900567

SHEAR TEST WITH BUTT JOINTS

300kPa

	Bar 1		Bar 2		Bar 3		Bar 4		Bar 5	
Gauge number	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B
Strain	516,6	330,6	96,35	233,3	206,6	193,6	469,4	668	464,8	273,9
Strain in bar		423,6		164,8		200,1		568,7		369,37
Intermediate value										345,32
Prestress between laminations [Pa]										319520

600kPa

	Bar 1		Bar 2		Bar 3		Bar 4		Bar 5	
Gauge number	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B
Strain	846,2	567,7	476,5	599,2	657,8	554,7	776,6	1042	803,5	561,73
Strain in bar		707		537,8		606,2		909,1		682,63
Intermediate value										688,54
Prestress between laminations [Pa]										637108

900kPa

	Bar 1		Bar 2		Bar 3		Bar 4		Bar 5	
Gauge number	1A	1B	2A	2B	3A	3B	4A	4B	5A	5B
Strain	1161	770,4	875,7	1057	1140	901	840	1176	1205	829,76
Strain in bar		965,7		966,5		1021		1008		1017,4
Intermediate value										995,6
Prestress between laminations [Pa]										921232