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Findings and points of interest of the Nordic Timber Bridge Projects

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1. Introduction

During the years of 1994 – 1996, 1997 – 1999 and 1999 –2001 three Nordic Timber Bridge Projects were carried out. The partners involved came from Denmark, Finland, Norway and Sweden. The projects were part of a large wood project called Nordic Wood that was coordinated and financed by the Nordic Industrial Fund. The total budget of the three Timber Bridge Projects was about 2,7 M€, from which the share of the Nordic Industrial Fund was 30 %. Additional governmental money, i.e. 20 %, came from national research funds. The remaining half came from national road and bridge authorities and private wood industry enterprises. The common interest and main object of the projects was to increase the competitiveness of timber bridges compared to other materials like steel and concrete and to develop and activate the Nordic wood industry and timber bridge construction.

For coordination, a steering committee of 6 – 8 persons was established for each project. These persons represented the main industrial financers and national road administrations. Each participant country was equally represented. Additionally, a representative of the Nordic Wood, an observer from Estonia and the leaders of the four project groups attended the meetings. The project groups, which were responsible for the research and practical work, were

Dansk Teknologisk Institut, Denmark
Laboratory of Bridge Engineering of Helsinki University of Technology, Finland
Norsk Treteknisk Institutt, Norway and
Trätek, Sweden.

The results of the projects were reported in 33 printed reports and several national and three international conferences. Additionally two booklets presenting Nordic timber bridges built during the projects were published. Some of the results had contribution to the ongoing work with Eurocodes. The main result, however, was that many new timber bridges were built and that a considerably increased interest in timber bridges in the Nordic countries was created. As a consequence, an IABSE conference on innovated wooden structures and bridges with 100 papers and 280 participants was held in August 2001 in Lahti, Finland.

The purpose of this paper is to present the results and achievements of the three projects mentioned above.

2. Development of wood-concrete composite bridges

One of the main interests of the current research was to investigate the possibility to use wood in road bridges. Although timber bridges were always built especially in Finland, it was felt that new solutions were needed. Such solution was found in wood-concrete composite construction that became quite popular in Finland. So far this kind of bridges have not been built in the other Nordic countries.

The three major benefits achieved by using wooden beams and a concrete deck as a composite structure in road bridges are

- the strength and stiffness properties of the two materials can be utilized better,
- the concrete deck makes it possible to use similar wearing surfaces as in bridges made from other materials and
- the durability of the timber beams is increased when they are protected against harsh weather conditions under the concrete deck.

The essential parts in a composite structure are the connectors placed between the two materials to make them to act together. To be effective and competitive, the connectors must be stiff, durable and easy to manufacture. After analyzing and testing several alternatives it was found out that simple and effective shear connectors can be constructed by using steel bars embedded in wood by glueing them into inclined predrilled holes (Fig. 1).

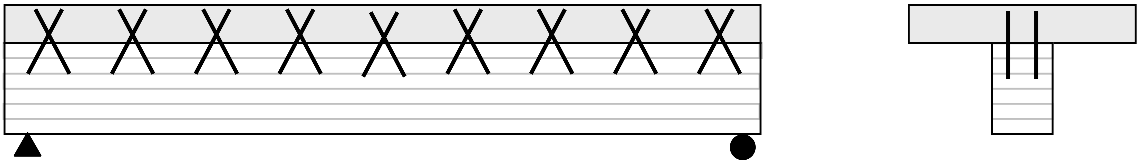


Fig. 1. Principle of joining a timber beam and a concrete deck by using glued-in steel bars.

After the decision to use wood-concrete composite concrete in the 42-meter span Vihantasalmi Bridge (Fig. 15) was made, several experimental tests were performed to ensure the reliability of the shear joints under service and ultimate loading [13]. To study the effect of different parameters on stiffness, strength and durability of the joint, four specimens were prepared and tested in the laboratory by using a one-splice shear test as shown in Fig. 2 and Fig. 3, respectively [5].

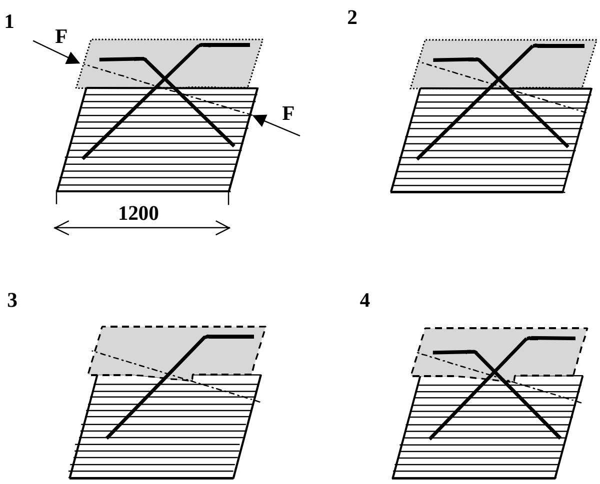


Fig. 2. Four shear specimens tested in the laboratory.

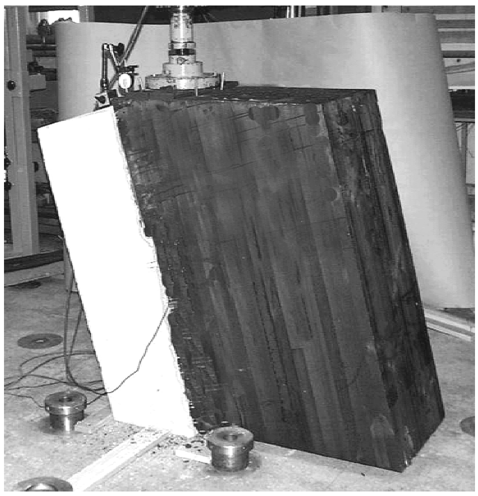


Fig. 3. One-splice shear test arrangement in the laboratory.

Two types of adhesives, polyurethane (Specimens 1, 2 and 4) and epoxy (Specimen 3) were used for glueing. Two specimens had a notch on the upper surface of the wooden web but different reinforcement (Specimens 3 and 4). Each specimen was subjected to a pulsating load with an amplitude of 160 kN for more than one million load cycles.

The behaviour of the connections during the fatigue tests was determined by measuring the relative displacement, i.e. the slip between wood and concrete, by using inductive transducers installed on both sides of each specimen. The results of the tests are shown in Fig. 4. The first curve on the left in each diagram represents the static force-slip response of the joint. The other curves show the static response during and after the fatigue tests, respectively.

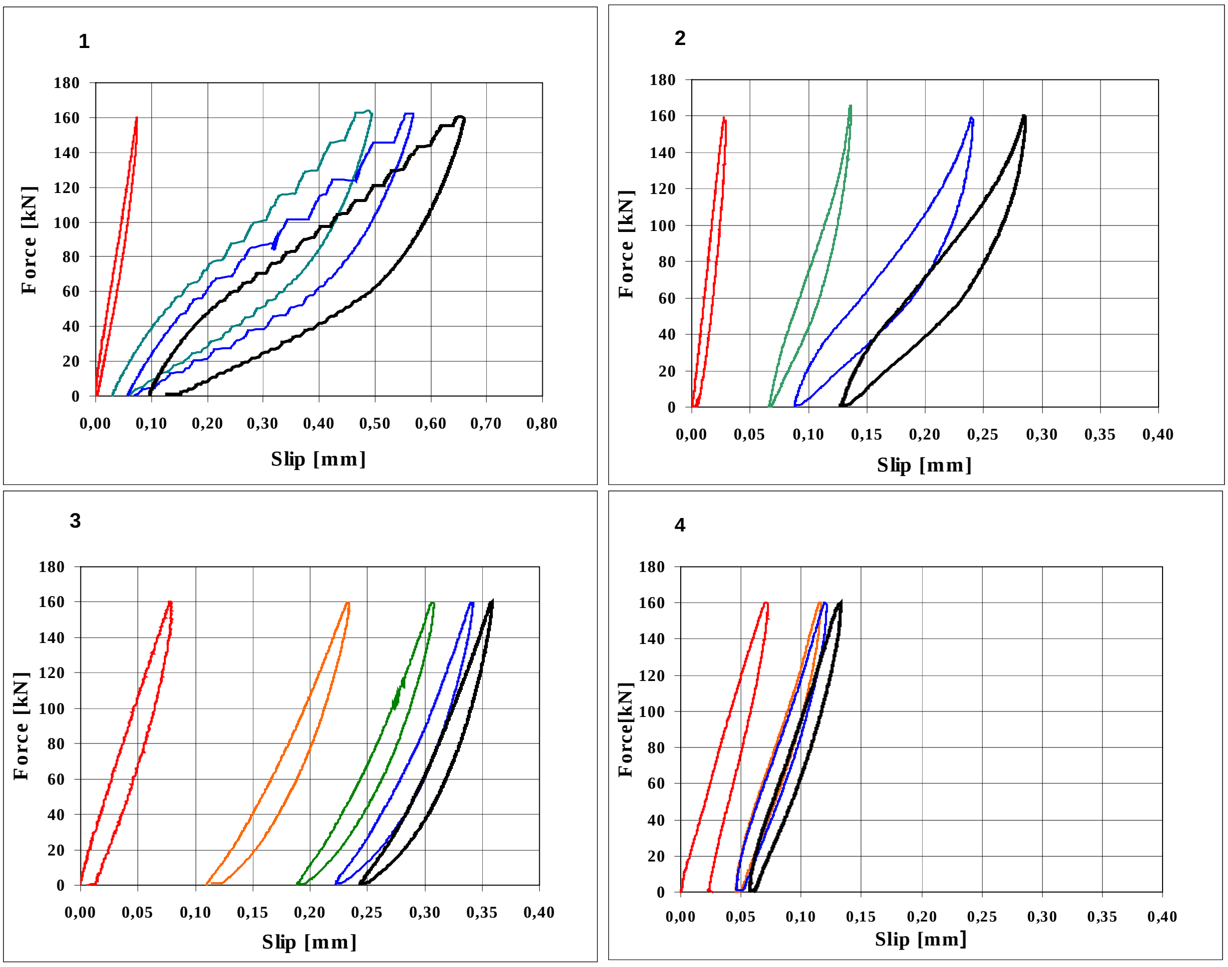


Fig. 4. Effect of fatigue loading on the slip of the shear joints tested. N is the maximum number of load cycles [5].

Damage caused by fatigue could not be observed during the tests. Some softening of the shear connections, however, is evident as can be seen from the decreasing slope of the curves when the number of load cycles increases. The stiffness obtained when using epoxy glue (Specimen 2) was considerably higher than that obtained with polyurethane glue (Specimen 1). A notch (Specimens 3 and 4) increases the stiffness and decreases the slip, as could be expected. The inclined reinforcing bars for compression force in Specimen 4 kept the permanent slip low. In this specimen nearly linear and stabile response was obtained even after 1,7 million pulsating load cycles.

After the fatigue tests each specimen was loaded up to failure by using a monotonically increasing static load. The ultimate load varied between 588 kN and 921 kN. Notched specimens had considerably (approximately 35 %) higher shear capacity than the un-notched ones [13].

On the basis of the shear tests it is evident that by using reinforcing bars and notches it is possible to achieve a connection that is stiff, behaves nearly linearly, maintains its stiffness under repetitive loading at the service state level, and has high ultimate shear capacity. That is why the connection
of type 4 and epoxy glue was chosen for the composite deck structure of the Vihantasalmi Bridge (Fig. 5).

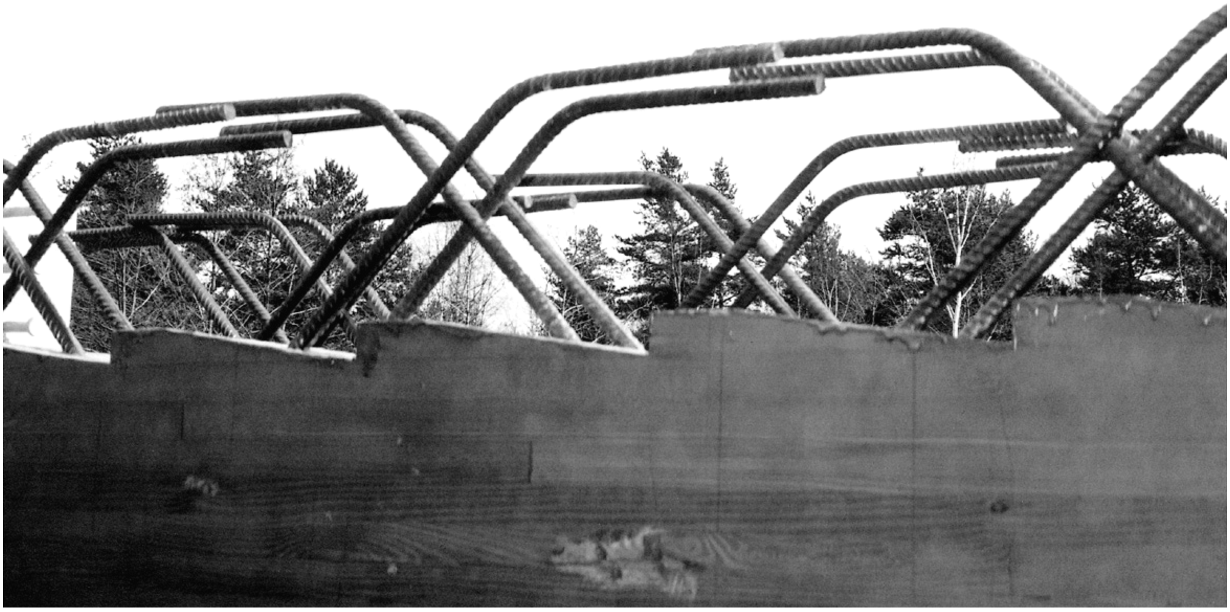


Fig. 5. Shear connectors of the main beams of the Vihantasalmi Bridge.

Additionally, environmental effect on the connections was tested by using two test beams. In the first phase, the beams were subjected to more than one million load cycles. After that the beams were stored two years outside uncovered and exposed to variable weather conditions and then tested again. The purpose was to find out, whether the storage time had reduced the stiffness or fatigue strength of the connections [3].

Comparison of the slip measurement results shows that the two years storage time has no influence. It means that the connections developed are not sensitive to environmental loading.

The subproject on wood-concrete composite bridges played a vital role in the development of connections between wood and concrete. As a result, a variety of new deck types were introduced. Moreover, a timber road bridge with 42 m span and thereby wood-concrete composite structures became a reality in bridge construction in Finland.

3. Joints in arch and truss bridges

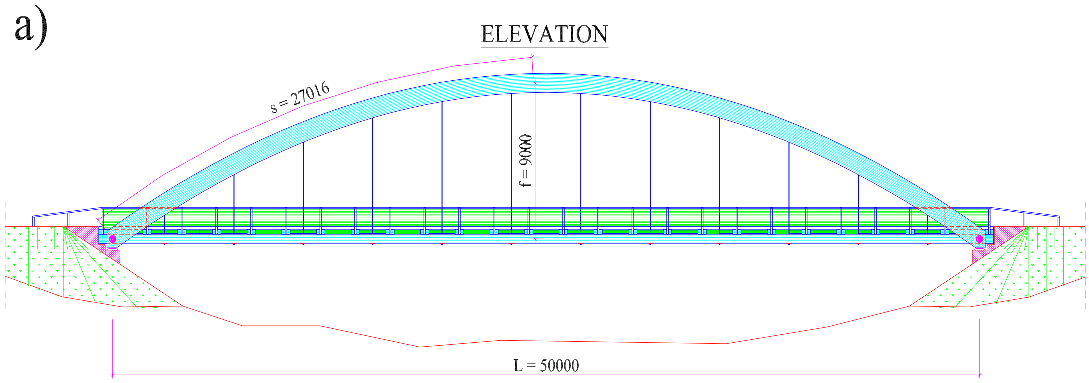
3.1 Types of joints developed

During the Nordic Timber Bridge Projects different kind of joints for arch and truss bridges were developed and partly also tested. The main concern was with the joints of large members on one hand and some details on the other. The joints of large members are needed for connecting two arch segments and at the arch footing. Smaller joints appear at hanger connections and in trusses.

For connecting large members in arches and trusses, a BSB-type connector (Blumer System Binder) was found to be most suitable, but also other types of connectors were developed. The difference with the ordinary BSB connectors is that bigger bolts and holes, 12 mm and 12,5 mm, respectively, and normally 8 mm thick steel plates were used.

As an example, a 50 m span pedestrian bridge designed is shown in Fig. 6. The arch itself consists of four 215 x 1100 mm^2 curved gluelam members arranged in pairs so that there is a gap for hangers between two members. With this arrangement the hanger connection becomes rather simple, as shown in Fig. 7.

a)



b)

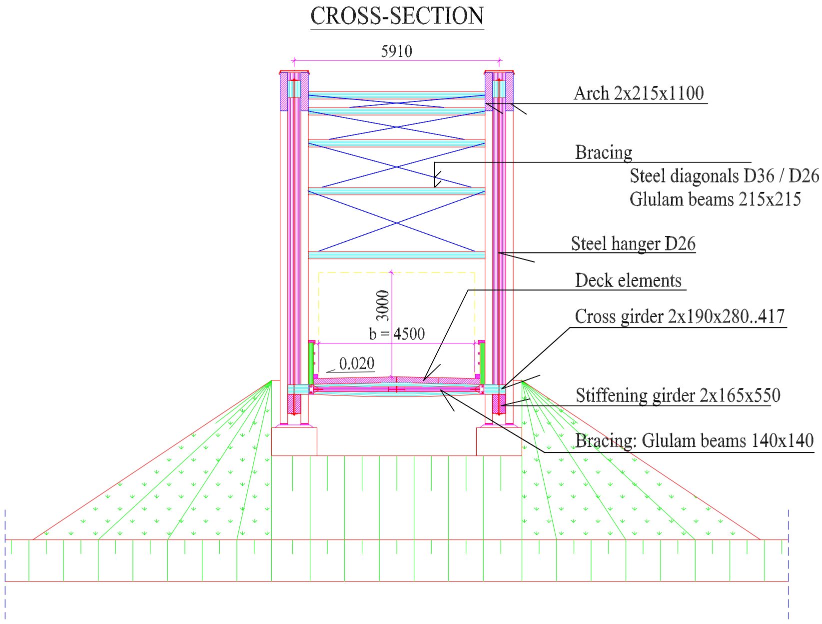


Fig. 6. Elevation and cross-section of a 50 m span pedestrian bridge designed. Dimensions in mm.

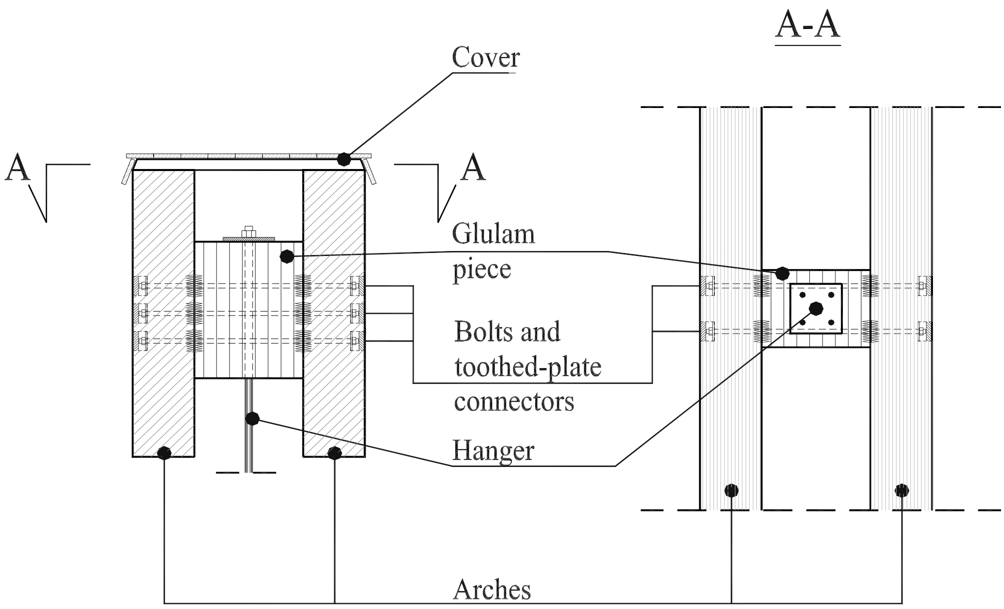


Fig. 7. Hanger connection between two adjacent gluelam arches. Dimensions in mm [12].

Another type of a big arch-hanger connection is shown in Fig. 8. It is based on the use of dowels and parallel slotted-in steel plates. A similar connection was found to be useful in trusses when joining two diagonals with a chord (Fig. 8).

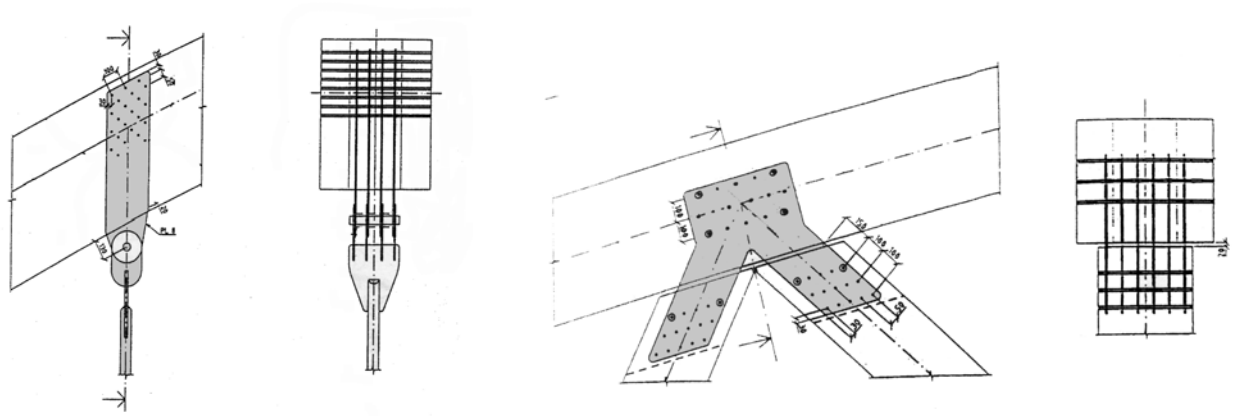


Fig. 8. Typical joint of a hanger in an arch and two diagonals in a truss, respectively [14].

3.2 Fatigue tests

Otherwise than buildings, bridges are subjected to variable stresses due to traffic. Although fatigue resistance of wood material generally is high, mechanical fasteners may cause local damage and lead to fatigue failure of the structure. Due to the limited information on fatigue of joints with slotted-in steel gusset plates combined with steel dowels, a series of tests was carried out to verify the present knowledge and code requirements. Therefore, 25 specimens shown in Fig. 9 were prepared to make altogether 3 static tests and 42 fatigue tests in a rig.

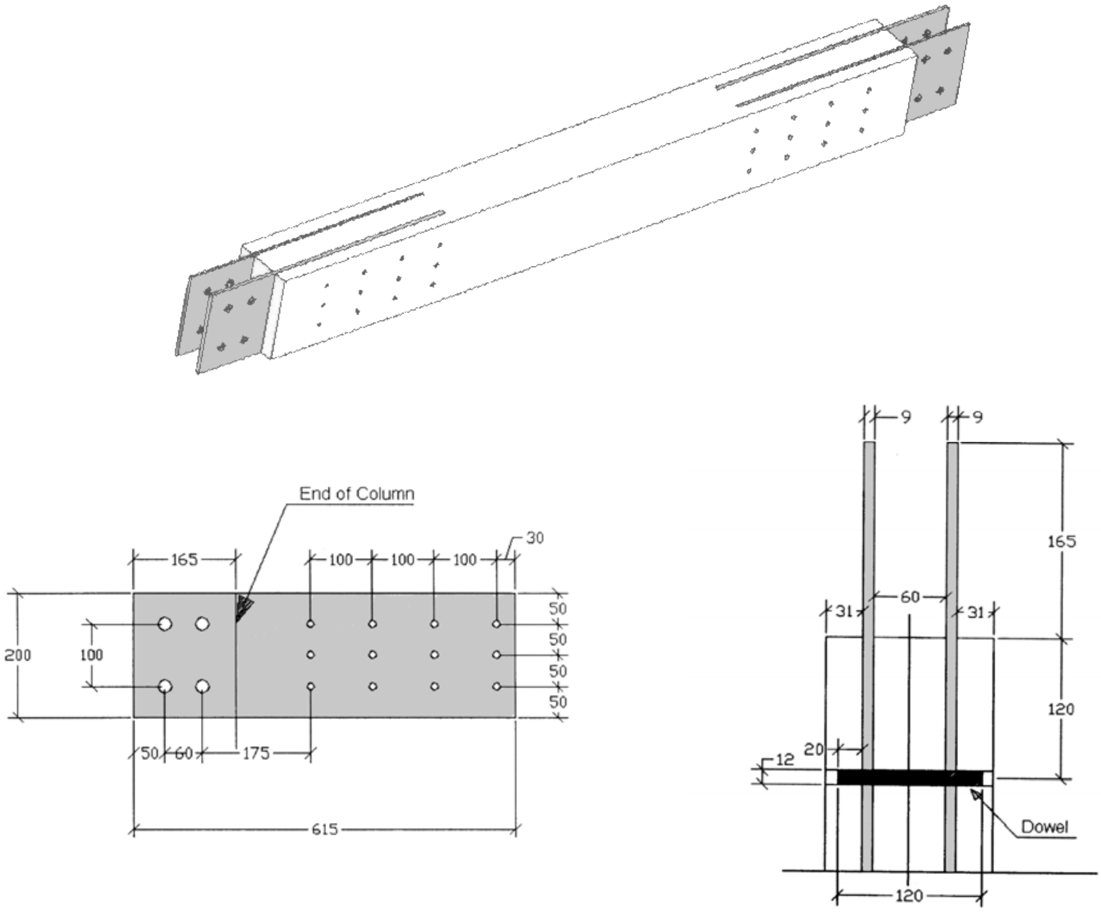


Fig. 9 Perspective view of a specimen with two similar ends and geometric data of the gusset plates, dowels and wooden parts, respectively. Dimensions in mm [6].

The conclusions drawn from these tests state that

- use of maximum stress as a fatigue strength parameter seems to be more reasonable than using the stress range,
- fatigue strength is dependent on the loading ratio, which is the ratio between the minimum and maximum stress and
- fatigue strength can be considered to be a linear decaying function of the logarithm of the number of cycles in accordance with Wöhler's classical work [6].

4. Lateral prestressing of timber decks

According to Professor *Ernst Gehri*, the origin of lateral prestressing of timber decks, or stress-lamination, as it is more commonly called, goes back to the 1950's, when in house construction the stress-laminated floors were first used. In Canada first use of bridge decks goes back to the 1970's [4]. In 1984 in the 12th Congress of IABSE in Vancouver, a poster on this topic was presented by *Bachelor*, *van Dalen* and *Taylor*. Inspired by this new technology, a diploma thesis study was carried out in 1986 at Helsinki University of Technology under the guidance of the first author. Its promising results were then presented to bridge authorities in order to implement such decks to timber bridge construction, but it turned out to be ten years too early. Today it is common practice in the Nordic countries to construct timber bridge decks by using lateral prestressing.

There are two important issues connected to laterally prestressed bridge decks of timber: time- dependent loss of prestressing force and friction between laminations. In order to tackle the former one, an attempt was made to find out new materials to be used in tendon design. In that sense the possibility to use synthetic fibre tendons was studied in one of the sub-projects in Phase 3 of the Nordic Timber Bridge Projects.

In Table 1, characteristic values of some synthetic tendon materials are shown. As a comparison, a Dywidag steel bar is also incorporated. The effect and importance of stiffness ratio E/σ_{adm}, i.e. the values in the last column, can clearly be seen in Fig. 10. Here tendons of different materials are supposed to be stressed to the maximum long-term stress level σ_{adm}, which corresponds to the desired compression stress of wood. After prestressing the time-related deformations of the wooden deck are developed and cause the shortening of tendons. According to *Hooke*'s law, shortening means loss in the prestressing force. When a constant strain development $\Delta\varepsilon$ is assumed, the remaining force in a steel tendon is only 66 % of the original one, whereas in synthetic tendons 87 – 91 % still remains (Fig. 10). This means savings in the number of restressing phases and consequently in costs, too.

Table 1. Characteristic values of some synthetic tendon materials compared to that of a steel tendon [9].

Tendon material	**E** [N/mm^2]	σ_u [N/mm^2]	σ_{adm} [N/mm^2]	E/σ_{adm}
E-glass-fibre	40000	1200	360	111
PAN carbon fibre	150000	2500	1650	91
Twaron aramid fibre	62500	1500	950	66
Dywidag 15 F 0105 bar steel	210000	1100	880	239

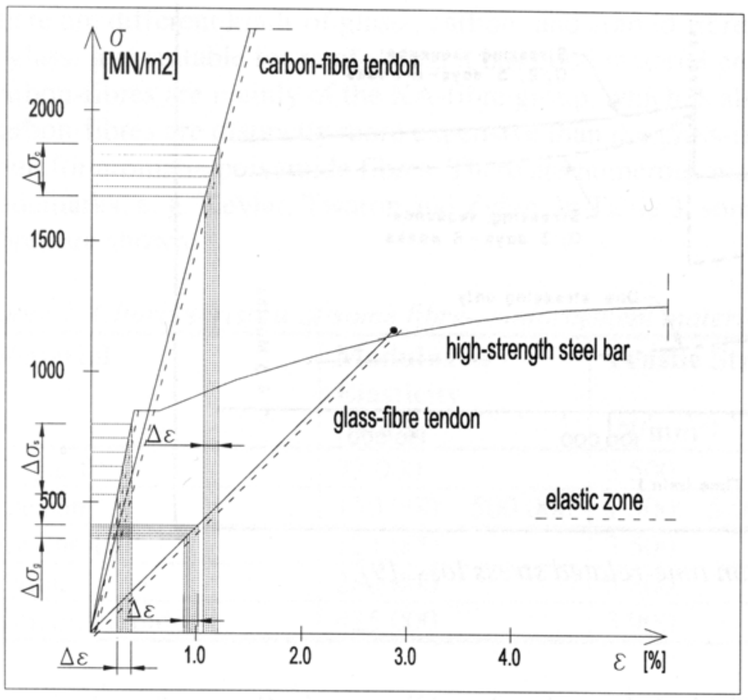


Fig. 10. Effect of stiffness ratio of tendons with constant strain [9].

Small-scale laboratory tests with 12 mm diameter glass-fibre rods simulating a tendon were also carried out. Anchoring by glueing was of special interest, but some material characteristics were studied as well. The results show that the values given in Table 1 are realistic. As expected, such tendons do not yield and rupture occurs suddenly and completely without any plastic deformation. From the viewpoint of stress-lamination this is not a problem, because the tendons reach their maximum load only when being prestressed. Anchoring by glueing, however, is problematic, because the outermost surface of a tendon tends to tear off during stressing, and as a result the full tendon capacity cannot be utilized. Solutions should be found for the anchoring of glass-fibre tendons before applying such tendons to prestressing of wooden bridge decks. Another obstacle is the high price.

5. Follow-up and monitoring of some bridges

The main purpose of the follow-up and monitoring of timber bridges is to gain knowledge of the behaviour of the bridge under service. The static or dynamic behaviour and the environmental influence on the properties of material and structures are the main parameters that can be followed either by field tests or continuous monitoring [2].

The static behaviour of a bridge can be most reliably studied by applying load tests. This is because the magnitude of mechanical loads and their positions are known and the corresponding response can be measured. By applying similar load test again after some year period, the effect of time, ageing and long-term loading on the static behaviour can be studied. This kind of static follow-up tests were carried out in 1995 and 1998 for the Uusisalmi Bridge, that is a multigirder wood-concrete composite bridge completed in May 1995 (Fig. 11) [7, 10].

Fig. 11. Static load test on the Uusisalmi Bridge.

The results of the measurements proved the capability of the composite bridge. It behaved statically as expected and no special differences were found when comparing the test results of 1995 and 1998. In 1998 also dynamic load tests were carried out by driving a fully loaded truck over the bridge using different speeds.

Five bridges altogether were monitored during the period 1998 – 2002. The Lusbäcken Bridge in Borlänge, Sweden, is a two-lane 20,5 m span bridge with a laterally prestressed glulam box beam [1]. The monitored parameters were temperature and humidity of wood and ambient air, force in prestressing tendons and support displacement, respectively.

Other bridges monitored are the pedestrian bridge in Årjäng, Sweden, and three road bridges, i.e. the Daleråsen (Fig. 19) and the Evenstad Bridge in Norway and the Vihantasalmi Bridge (Fig. 15) in Finland [3]. All these bridges were instrumented with measuring devices allowing continuous measuring. An example of the temperature measurement of the Vihantasalmi Bridge is seen in Fig. 12.

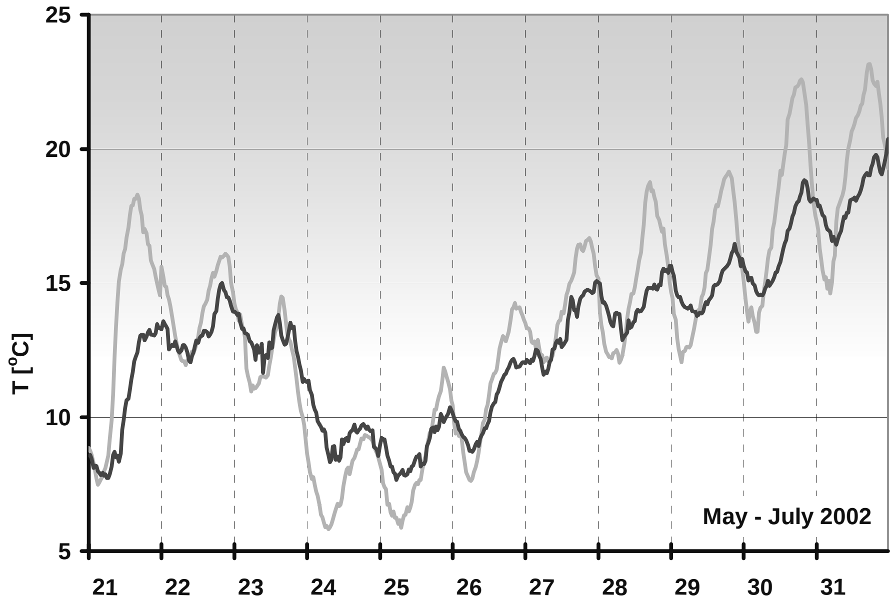


Fig 12. Ambient air temperature (orange) and wood temperature (red) variations in spring 2002 on the Vihantasalmi Bridge [11].

6. Examples of Nordic timber bridges

As a conclusion, examples of Nordic timber bridges constructed during or just after completition of the three projects described above are presented here. Due to the limited space, only a few representatives could be included. These examples, however, show that wood as a material gives excellent possibilities for engineers to construct innovative, technically reliable and aesthetically pleasing structures for the need and pleasure of people and society [8].

Fig. 13. Byholmen Footbridge, Dragsfjärd, Finland. Span 23 m, width 4 m. Completed 1997.

Fig. 14. Poukkasilta Ridebridge, Ypäjä, Finland. Span 32 m, width 3,5 m. Completed 2001.

Fig. 15. Vihantasalmi Bridge, Mäntyharju, Finland. Spans 21+3x42+21=168 m, width 11+3=14 m. Completed in November 1999.

Fig. 16. Munkedal Footbridge, Munkedal, Sweden. Span 60 m, width 3,5 m. Completed 1999.

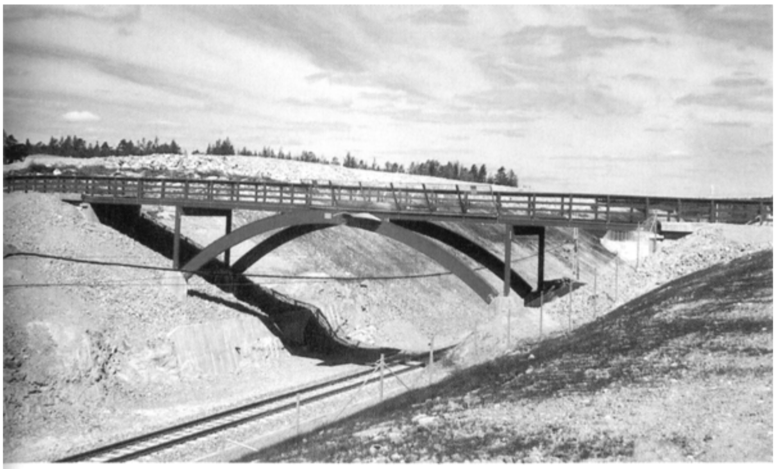


Fig. 17. Okb Footbridge, Söderhan, Sweden. Arch span 19 m, width 4 m. Completed 1998.

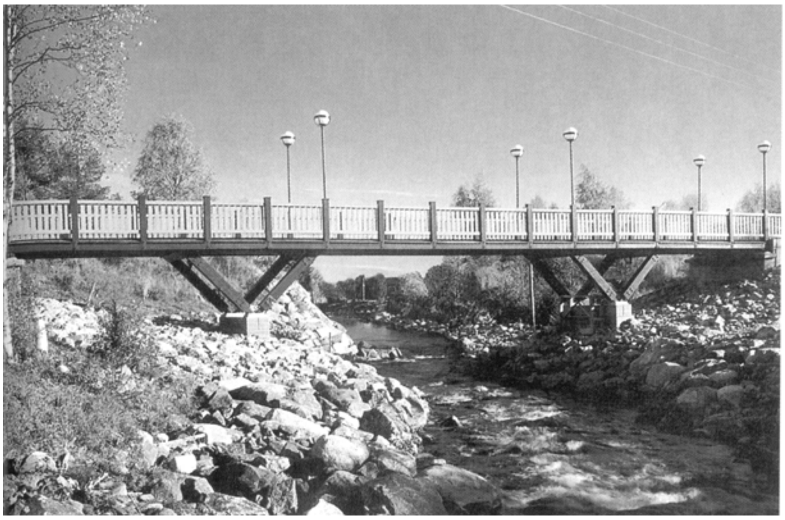


Fig. 18. Svanstein Footbridge, Svanstein, Sweden. Spans 6+12+6=24 m, width 3 m. Completed 1995.

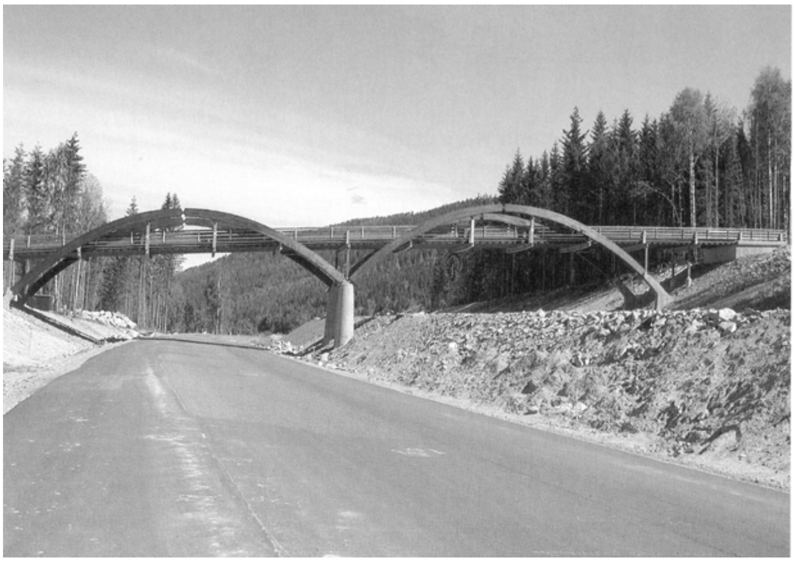


Fig. 19. Daleråsen Bridge, Mjøndalen, Norway. Spans 32,6 m and 27,4 m, width 5 m. Completed 2001.

Fig. 20. Leonardo da Vinci Bridge, Norway. Spans 45 – 55 m. Completed in Oktober 2001.

Fig. 21. Tynset Bridge,Tynset, Norway. Spans 2x27+70=124 m, width 7+3=10 m. Completed 2001.

Fig. 22. Merirahu Bridge, Tallinn, Estonia. Span 24 m, width 3 – 5,5 m. Completed 2000.

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