Chapter No. 15

TIMBER BRIDGES

1 Introduction

The last two decades have seen a growing interest in timber bridges in many European countries. There are several reasons for this. The growing interest in environmental questions and sustainability has definitely paved the road for more use of structural timber, but also new and innovative use of timber, such as the stress laminated timber deck and better connections, have played an important role. The fact that reinforced concrete did not turn out to be an everlasting material - many countries have experienced serious problems with concrete bridges built in the 1960’s and 70’s - is another factor. Last, but not least, the enthusiasm of individuals should also be acknowledged.

We start this brief account by a short historical review before we indicate the most common types and sizes of timber bridges. We then move on to the main structural systems, for both the support structure and the bridge deck. Connections and details are very important for the success of timber bridges, perhaps more so than for other types of timber structures. In general, dynamic excitation is not considered a serious problem for most timber bridges, but some aspects need to be addressed. We also include some notes on erection and economy of timber bridges. The most crucial challenge for a timber bridge is perhaps the question of longevity which brings in protection and durability as major concerns.

We round off the account by some comments on the Nordic Timber Bridge Project which has proved to be of vital importance for the revival of timber bridges in the north of Europe.

2 Brief historical note

Before the advent of, first cast iron (the Iron Bridge near Coalbrookdale on the river Severn, opened in 1781), then steel (one of the first major steel bridges, the King Albert Bridge over the Elbe, was opened in 1893), and finally reinforced concrete, stone and timber were the only available materials for bridge building. While we have many fine examples of very old stone bridges, this is, for obvious reasons, not the case for timber bridges. Although some covered timber bridges have survived for a remarkably long time, the most famous example being the Kapellbrücke in Lucerne, built in the period between 1300 and 1333, which is still standing (restored after a fire in 1993), most of the timber bridges of the past have disappeared.

Caesar’s bridge over the Rhine, built around 50 B.C., is quoted as one of the first major timber bridges in Europe. Andrea Palladio (1508-80), an Italian architect, is often mentioned in connection with timber bridges of the past, both for his introduction of the timber truss bridge design, and for some famous bridges, like the Ponte degli Alpini (1567) at Bassano which is still there. Another “Rheinbrucke”, at Schaffenhausen in Switzerland, built by Ulrich and Grubenmann in 1755-58, was designed with one span of 119 m, but the town authorities demanded a pillar put in at mid-span. The “Colossus”, an arch like timber truss spanning 104 m, built in 1812 by Louis Wernwag, is described as both an architectural and engineering masterpiece.

In the United States the building of the railroads in the latter part of the 1800's produced many, quite large timber bridges. One of them, the Cascade Bridge, built by Thomson Brown as early as 1845, was a truss-arch-truss bridge with a span of 90 meters and quoted by a visiting Swiss engineer to be one of the finest timber structures in the US.

However, with steel and reinforced concrete on the scene, road and railway bridges made of timber more or less disappeared in the 20th century, except for very short spans. In the latter (short span) category, a very large number of timber bridges exist in both North America and Australia, but most of them are fairly insignificant structures and hardly considered to be bridges by the traveling public. Most structures considered to be timber bridges, built in the 20th century, are footbridges. Around 1990 there is a change, and we see a gradual increase in the number of road bridges made of timber.

3 Types and sizes

Bridges are naturally divided into two major groups: footbridges and road bridges (for vehicle traffic). We will focus on the latter, but we start with a short review of timber footbridges.

3.1 Footbridges

The term footbridge also includes the bridges for combined pedestrian and cycle traffic. Such bridges come in all shapes and sizes. Most of them are simple beam-type bridges, either with massive glued laminated timber (*glulam*) sections or as truss beams, and typical span lengths are in the 15 to 30 meter range. However, we also find a large number of innovative and spectacular designs, such as the 192 m long bridge at Essing in Germany (with a maximum free span of 73m), over the Rhein-Main-Donau-Kanal, built in 1992 and shown in Fig. 1.

Does anyone have a decent picture of this bridge that I can ("legally") use here ?

Fig. 1 Footbridge over the Rhein-Main-Donau-Kanal at Essing, Germany

Another interesting footbridge is the so-called Leonardo bridge. Inspired by Leonardo da Vinci's sketch of a stone bridge over the "golden horn" (from Istanbul to Pera), Norwegian artist Vebjørn Sand managed to create sufficient interest (and money) to build a rather spectacular glulam bridge across a major road (E-18) at Ås, south of Oslo. The bridge, built in 2001 and shown in Fig. 2, is about 120 m long with a main span of about 40 m. It should be noted that the bridge replaced one of the ugliest footbridges in the country.

Figure 3 shows another recent Norwegian footbridge, built at Lardal in 2001. A creosote impregnated glulam bridge with a steel cable reinforcement in the mid-section, it has a free span of 92 m and a total length of about 130 m. This bridge has some dynamic problems which we shall return to in a later section.

Fig. 2 The Leonardo footbridge at Ås, south of Oslo, Norway (photo: Moelven Limtre AS)

Fig. 3 Footbridge at Lardal, Norway (photo: A. Rönnquist)

3.2 Road bridges

The vast majority of timber bridges for ordinary road traffic is short span (5 to 20 m) slab and beam type bridges, often made as wood-concrete composite structures. Some 20-25 years ago timber became an interesting material also for longer bridge spans, both in Europe and North America, and we now find a fair number of medium size timber bridges, even on major roads. Figures 4, 5 and 6 show some typical examples of modern timber road bridges.

Figure 4 shows the "Wennerbrücke" over the river Mur at Murau in Austria. Built in 1993, this is probably the first large timber bridge built in Europe to serve a major road. The main components of the support structure are four parallel, parabolic 3-hinge arches spanning 45 m. The glulam arches, together with straight glulam columns, support four massive glulam girders which in turn supports a pre-cast and post-stressed concrete deck. The total bridge length is 85 m. All (300 m^3 of) glulam is made of untreated larch wood, which was later surface treated with stain. An important feature of this bridge is the "roof effect" of the bridge deck.

In Fig. 5 is shown the Vihantasalmi bridge at Mäntyharju in Finland (some 180 km north of Helsinki). The main supprt system consists of three glulam king-post trusses, each with a free span of 42m. The total length of the bridge is 182 m; it was completed in 1999. The deck is a concrete-steel-glulam composite structure, in which glulam girders provide the longitudinal bearing, steel trusses the side-way stiffening and concrete the deck itself.

Fig. 4 “Wennerbrücke” over the river Mur at Murau, Austria (photo: Institut für Holzbau und Holztechnology, TU Graz)

Fig. 5 Vihantasalmi bridge at Mäntyharju in Finland (photo: K.Bell)

Fig. 6 Tynset bridge, Norway (photo: K.Bell)

Tynset bridge, shown in Fig. 6, is supported by three times two arches. The main arch, a 2-hinge glulam truss arch, has a free span of 70 m, whereas the two smaller arches are 3-hinge arches of massive glulam, each with a span of about 26,5 m. All glulam is creosote impregnated. The arches support cross beams of steel which in turn supports a 223 mm thick stress laminated timber deck made of creosote impregnated structural timber. The total length of the bridge, built in 2001, is 124 m.

4 Structural systems

The examples shown in the previous section are typical of modern timber bridge design, and they indicate the span range current technology can handle. Given the right conditions it is probably feasible to bridge a span of well over a 100 m with a timber structure, but the normal span for a timber bridge is in the range of 5 to 75 m.

4.1 Arches

The fact that half the bridges shown above are arch bridges is no coincidence. In the majority of modern timber road bridges one finds arches in one shape or another. For uniformly distributed load the arch will carry the load almost exclusively by axial compression and is thus a very economical shape.

4.1.1 Geometry, material and configurations

In principle we have three different arch designs, as shown in Fig. 7. The arch is normally in the shape of a parabola (Wennerbrücke) or a circle (Tynset). The ratio between span (*L*) and height or rise (*h*), that is *L/h*, is in the range from 4 to 8. The material is, for all practical purposes, glulam, and the cross section is normally a massive rectangle for the lower spans, up to about 50 m, whereas truss arches are the norm for the larger spans. Both production and transportation limit the size of curved glulam members with massive cross sections, and a consequence of this is that most such timber arches are made in the form of 3-hinge arches, as indicated by the hinge at the crown (C) in Figure 7b. This, of course, renders the arch statically determinate, which in itself can be an advantage in case of changes in moisture and temperature and settlements of the supports.

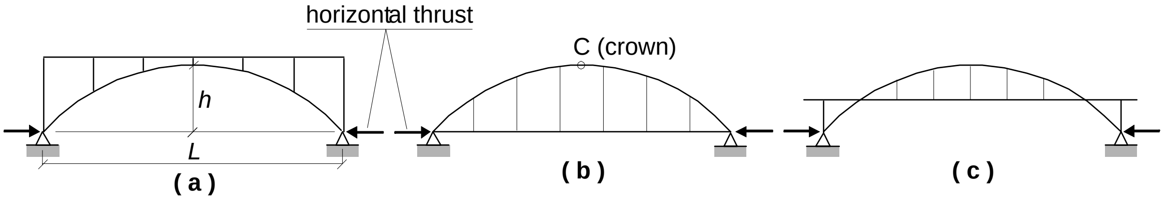


Fig. 7 Arch bridge designs

In Fig. 7a the arch is located under the deck, as for Wennerbrücke, and the deck is supported by the arch through compression (timber) members. In Fig. 7b the arch is placed above the deck which is suspended from the arch by (steel) tension members. In the last case, Fig. 7c, we have an intermediate solution. In all cases the vertical loading, dead load and traffic load, is transmitted to the arch, by (usually) vertical members, and the arch take the load to the supports, mainly as axial compression. This compressive force has a very significant horizontal component at the support, the horizontal *thrust*. In cases a) and c), this thrust must be absorbed by the support itself, which may or may not be a problem. In case b) we have two possibilities of neutralizing the thrust: by the supports (abutments) themselves, as in cases a) and c), or by a *tie rod* between the arch supports.

Having the support structure beneath the deck, as in Fig. 7a, has three distinct advantages: 1) the deck serves as a protective “roof” for the support structure, 2) no limitation as to the number of (parallel) arches, and 3) transverse stiffness of the support system is (normally) easily obtained.

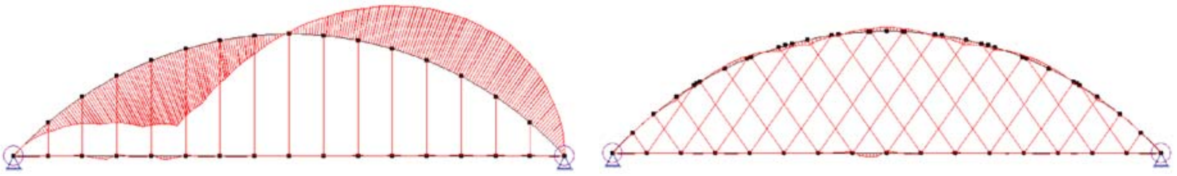
However, in many cases the bridge site does not lend itself to this solution, and the arches will have to be placed above the deck, as in Fig. 7b. With this design, and also the intermediate design in Fig. 7c, the number of arches, for a two-lane bridge, is normally two, one on each side of the bridge.

4.1.2 Some problems and challenges

If the arches are placed above the deck, sideway stiffening of the arches can be a problem. Given enough height, a “wind truss” between the arches, in which the arches form the chords, is the obvious choice (as indicated for the main arch of the Tynset bridge in Fig. 6). There will, however, always be a fairly long unsupported portion at the lower part of the arch, and this may call for additional measures.

For shorter spans, the height does not permit such a truss, and alternative methods must be found. For each of the two smaller arches of the Tynset bridge (Fig. 6), where L = 26,5 m and h = 5,8 m, there is not room for a truss. Instead, the two middle *hangers* are made of steel sections with significant bending stiffness, and these hangers are *rigidly* connected to the steel cross beams, thus forming two U-shaped frames which provide transverse stiffness to the arches.

With vertical hangers (and columns in case of arches below the deck) the large concentrated axel loads, from real or imaginary heavy trucks that all bridge load codes specifies, will cause significant bending moments and shear forces in the arches, in addition to the axial compression. The two bending moment diagrams in Fig. 8, taken from a feasibility study of a 2-hinge arch bridge with a span of 80 m, demonstrate this very clearly. Fig. 8a shows the standard design, with vertical hangers, whereas Fig. 8b shows an alternative design with inclined hangers, the so-called *network* arch.



a) M_{max} = 7295 kNm

b) M_{max} = 540 kNm

Fig. 8 Bending moments for two different cable arrangements, but roughly the same traffic loading

Both diagrams are drawn to the same scale, and the traffic loading is roughly the same, but placed such as to give the highest moments in the arch in both cases. The axial compression, which does not vary much along the arch, is quite similar for the two cases, although slightly lower for the network arch. We see that the network arch introduces the loading from the deck to the arch in a much more favorable way, particularly when the traffic loading is placed towards one of the supports. This is, however, not the complete story. Network arches have their problems too.

High bending moments and shear forces caused by the large concentrated forces introduced in the arch by concentrated traffic loading through the vertical hangers represent several problems. One in particular, associated with tension perpendicular to grain, should be mentioned. In Fig. 9 is shown the bending and shear force diagrams for one of the smaller arches of the Tynset bridge (Fig. 6), obtained by a 2D frame analysis with the concentrated traffic loads placed near the leftmost hanger. The bending moment on the left hand side of the arch, causing tension on the concave side, produces (moment induced) tension perpendicular to grain. At the point of maximum moment, and hence maximum tension perpendicular to grain ($\sigma_{t,90,d}$), we also have a large shear stress (τ_d), and satisfying the combined check of Eurocode 5 - EN 1995-1-1 (EC5-1), formula (6.53), may not be an easy task.

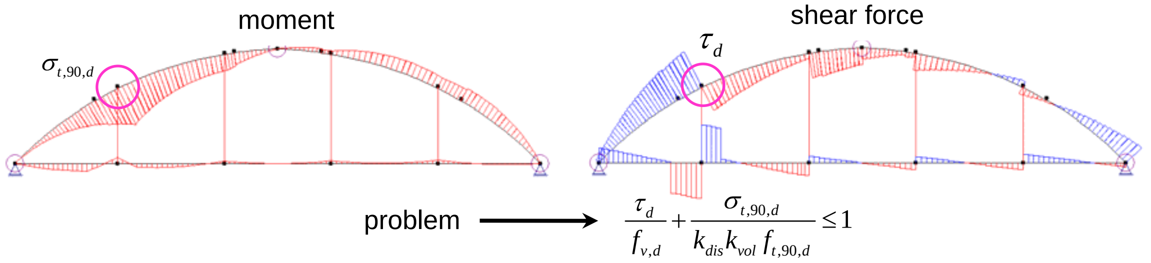


Fig. 9 Combined tension perpendicular to grain and shear stress – a problem for arches with vertical hangers

4.2 Trusses

The alternative to arches for road bridges with span over, say 20 m, is trusses in one form or another. Again we have the three configurations of Figure 7: the parallel trusses may be placed below the deck, as shown in Fig. 10a, which enables more than two trusses, they may be placed above the deck, as shown in Fig. 10b, or, more unusual, the deck may be placed between the top and bottom chord of the truss. For long spans, the lower chord will usually be curved for a truss below the deck (Fig. 10a), whereas it is the upper chord that may be curved if the truss is placed above the deck, as shown in Fig. 10b. The truss has an advantage over the arch in that it has no horizontal thrust at the supports. It also has fewer limitations with respect to production and transportation, since the chords may be assembled of several parts joined at nodal points. Connections represent a challenge, and transverse stiffening of the top chord, in the case of a design with the truss above the deck, is not necessarily a trivial problem.

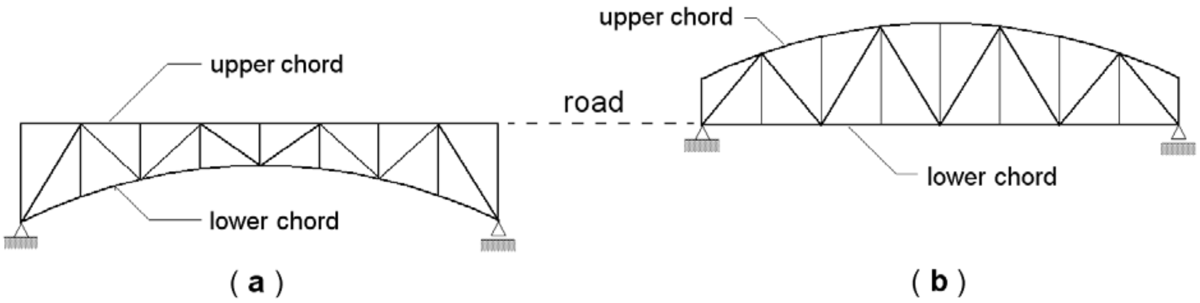


Fig. 10 Examples of trusses for road bridges, a) below the deck and b) above the deck

4.3 Other systems

For road bridges with a span over 20-25 m, arches and trusses dominate. For very short spans, up to about 10-15 m, timber *plates* of various makes are used as both load bearing system and bridge deck. Also timber *beams* are used as the main structural element for the shorter spans, often in connection (and interaction) with reinforced concrete slabs.

For footbridges we find the same support systems as for road bridges, but in addition we also find a variety of mixed systems, often using steel cables/wires as additional structural elements (suspension type bridges and cable stayed variants).

5 Bridge decks

The vast majority of timber footbridges have decks made of timber, in one form or another, from simple timber boards or planks to crossed layers of boards and various types of laminated decks. For road bridges the situation is more complex. We find that the standard building materials, that is, reinforced concrete, steel and timber, are used in many different combinations, from “pure” concrete decks to almost “pure” timber decks. One might rightly ask, how much timber needs to be used in a bridge for it to be called a timber bridge? Usually the dominating material employed in the main support system will decide, and since the deck is, in most cases, a secondary bearing system it will not take precedence. Hence, the Wennerbrücke (Fig. 4) is definitely a timber bridge, although the deck is made of concrete.

For lack of space, we will concentrate on one particular type of deck made predominately of timber, the so-called *stress laminated* timber deck. The idea comes from Canada where it was first used in 1976 by the Ontario Ministry of Transportation and Communication for rehabilitating deteriorated nail-laminated lumber bridge decks. The method was successful and it was soon recognized that it offered a variety of possibilities also for the construction of new bridges. It was developed in Canada and the US, but soon found its way to Europe and Australia.

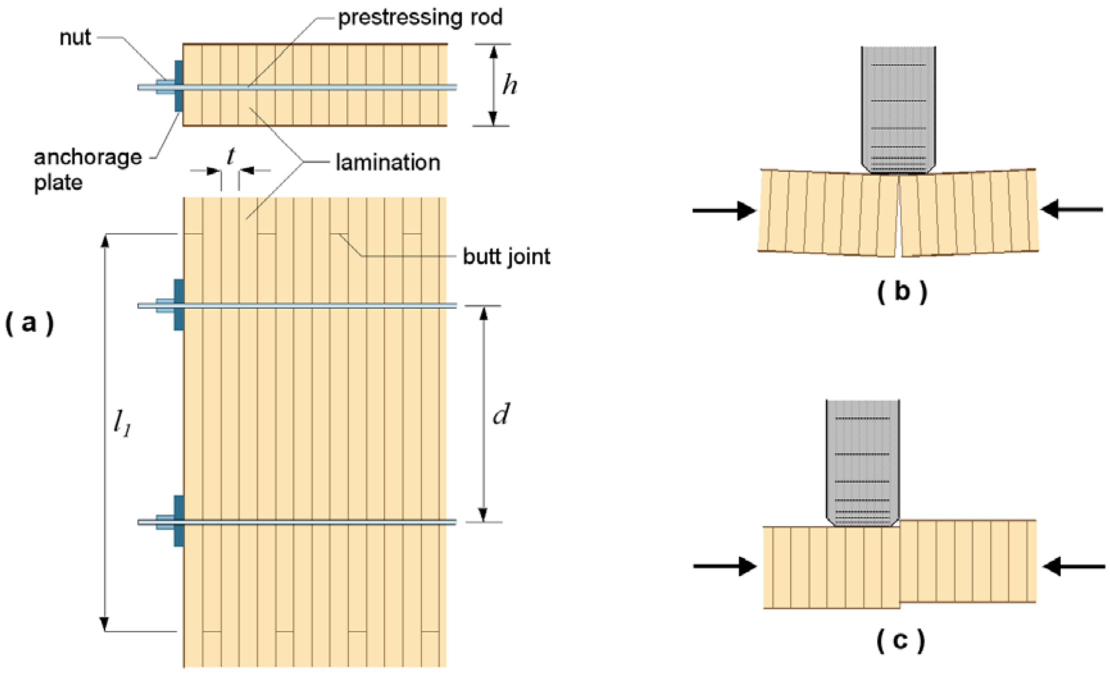


Fig. 11 Stress laminated timber deck - terms and principles

The basic idea is shown schematically in Fig. 11. Timber lamellas (planks of structural timber or glu-lam beams) are stacked, side by side, in the full width of the deck, and pre-stressing rods, through pre-drilled holes at regular intervals (*d*), will, when stressed, make the assembly of lamellas behave like an orthotropic plate. In Fig. 11b we see that transverse bending tends to cause an opening between the laminations on the underside of the deck, and in Fig. 11c transverse shear produces a tendency for laminations to slip vertically. Both these effects must be counteracted by the pre-stressing forces, and most codes, including Eurocode 5 - EN 1995-2 (EC5-2), require that the long term pre-stressing forces prevent all inter-laminar slip to occur.

Lamellas are of limited length, and they therefore need to be joined, butt to butt, as indicated in Fig. 11a. EC5-2 requires that not more than one butt joint shall occur in any *four* adjacent laminations within a length l_1 defined as the minimum value of

$$2d,\ \ 30t\ \text{ and }\ 1{,}2\text{ m}$$

where t is the thickness (width) of the lamination and d is the distance between the pre-stressing rods, see Fig.11.

In Norway just about all timber road bridges built during the last decade have stress laminated timber decks, in which the laminations are 48 by 223 mm, creosote impregnated pine planks. The length of the laminations is normally around 6 m. The pre-stressing rods are of the same type as used for concrete (typically Dywidag 15 FW) usually placed at a distance (d) of around 600 mm. The initial pre-stress results in a normal stress between the lamellas of approximately 1 MPa. If stressed only once, most of the pre-stress (80 % or even more) will be lost due to creep effects and variation in wood moisture content. The normal procedure is to come back and re-stress the rods at least once, after 2 to 3 months, and after that the loss is quite moderate. Figure 12 shows the laying of the stress laminated deck on the Evenstad bridge (1996), and a detail of the pre-stressing rods of yet another Norwegian timber bridge (Måsør bridge, 2005). It should be mentioned that the laying of such a deck is a logistic challenge; more than 40 different kinds of pre-drilled lamellas were used in the deck at Evenstad (which is about 180 m long), and each lamella was painstakingly marked with its type number.

Fig. 12 Laying the deck of Evanstad bridge (photo: Moelven Limtre) and detail of Måsør bridge (photo: K. Bell)

The maximum height (h) of commercial sawn timber in Norway is 223 mm. With this height the span of a stress laminated timber deck for a road bridge is around 5m, a bit more for an inner span of a continuous deck and a little less for the end span. With glulam lamellas it is of course possible to increase the span length.

It is very important to prevent surface water from penetrating the asphalt wearing course, and some kind of impregnable membrane is therefore placed between the timber deck and the asphalt. Since this is a relatively new concept put to work in a structure designed for a long service life it is important to obtain reliable information about its long term performance. The Norwegian Public Roads Administration has therefore put in place quite extensive instrumentations on several of the recently built timber bridges, concentrating in particular on the stress laminated decks. The properties monitored are mainly moisture content in the bridge deck and loss of pre-stressing force in the steel rods. It is too early for strong statements, but some preliminary findings seem to indicate that the moisture content in bridge decks with watertight membranes stabilizes at a level of about 10%, independent of the ambient equilibrium moisture content. The loss of pre-stressing force is considerable in the period following the initial stressing, but after re-stressing and some more loss, the force seems to stabilize as the moisture content stabilizes. However, the pre-stressing force varies significantly with temperature changes.

As already mentioned, the stress laminated timber deck behaves as an orthotropic plate, and the code (EC5-2) suggests that it should be analyzed as such. However simplified methods, considering the “plate” as a grid, or even as one or several fictitious beams in the direction of the laminations, may also be used. The code specifies the material properties to be used, and it suggests an effective width of the fictitious beam. For the ultimate limit state the code specifies how to check the bending and shear strength, but it has an additional requirement on the shear force which involves the minimum long-term residual compressive stress due to pre-stressing ($\sigma_{p,\min}$) and the design value of the coefficient of friction μ_d .

6 Connections and details

Connections play an important role in all types of timber structures of some size, and timber bridges are no exception. If anything, their role is even more critical for these structures since we normally need to consider service class 3. The large road bridges in particular put heavy demands on the connections.

For lack of space and the experience of the author, we narrow the problem down to the “Norwegian solution”, which makes extensive use of slotted-in steel gusset plates in combination with steel dowels. This type of connection was, for very large timber structures, pioneered by Moelven Limtre AS in connection with the roof structures for three large halls built for the 1994 Olympic Games at Lillehamar. Taking this connection from a protected indoor environment to the rather harsh Norwegian outdoor climate required some serious considerations, but in the end it was thought to be feasible. Steel quality and dimensions as well as corrosion protection were major concerns in view of the long service life required (100 years). Depending on how well the connection can be protected against direct contact with water (from rain and/or splashing), even stainless steel have been used.

Figure 13 shows some typical examples, reproduced with the permission of Moelven Limtre AS who designed the connections in collaboration with Norconsult AS, the acting consulting engineers. Both examples are from the Tynset bridge, see Figs. 6 and 9. On the left-hand side, Fig. 13 shows how the leftmost hanger in Fig. 9 is connected to the arch (the connection marked with a circle). It should be noted that the massive cross section of the arch is in fact made up of four glulam arches, glued together along the side surface(s) to form a section that is 710 mm wide. The slots for the steel plates are sawn from both sides (indicated by the dotted circles), but it should be kept in mind that the arch has a copper “roof” that will protect the connection from rain. On the right-hand side, Fig. 13 shows the connection at a nodal point on the top chord of the major (truss) arch. Note that the chord itself is also joined at the nodal point, and in addition to the two diagonals a transversal of the bracing (wind) truss also joins on to this point. Note also the gap (of 20 mm) between the two parts of the chord member. This gap is injected with an expandable mortar once the arch has been assembled. This substance, when set, is capable of transmitting higher compressive stresses than the timber.

The durability of outdoor timber structures is very much a question of moisture. Rule number one is to keep the water out, and rule number two is to make sure that it can escape through adequate “ventilation” when it cannot be kept out (which it normally cannot). Proper detailing is extremely important in this respect. Inspection of a large number of wooden footbridges in Norway showed that most of the deterioration was caused by poor detailing, such as unprotected end wood, lack of space for the moisture to escape, once it was in, and vertical compression members resting directly, without any or proper protection, on surfaces that would regularly become wet. Details can also influence the

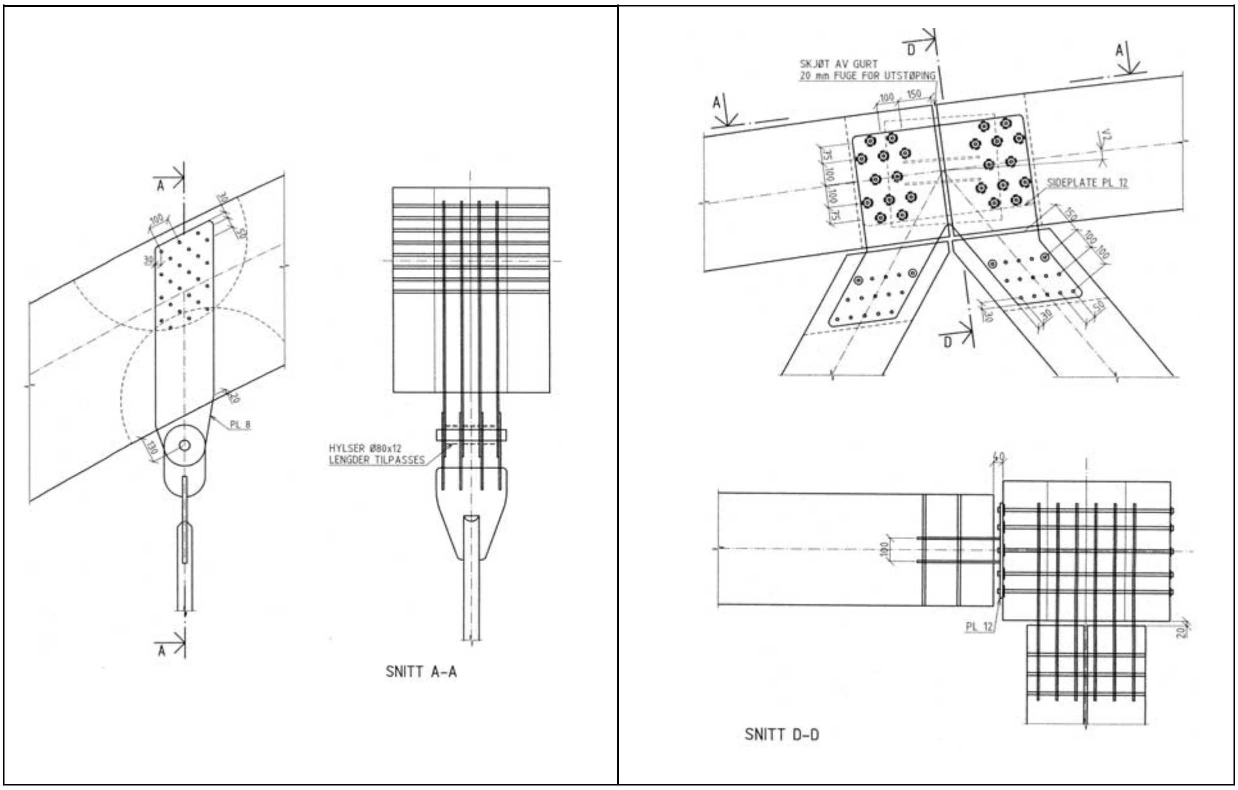


Fig. 13 Examples of connections in Tynset bridge (reproduce by permission of Moelven Limtre AS)

aesthetic expression of the bridge. Figure 14 shows two details of the Tynset bridge (Fig. 6), one of the hinge at the crown of one of the smaller massive arches, and the other of the base support of the two arches. Here the architect has obtained quite nice effects, through modest means.

Fig. 14 Details of the Tynset bridge (photo: K. Bell)

7 Dynamic effects

Wooden road bridges are normally not slender structures and hence not very susceptible to severe dynamic effects. Save for earthquake excitation, which is a problem for most structures in earthquake prone areas, *fatigue* is probably the one dynamic effect that most road bridges need to address. For a timber bridge the traffic loading is normally larger, in relation to the permanent loading, than for other types of bridges, and consequently the range of varying stresses can be significant. Fatigue is not considered to be much of a problem for the timber itself, but connections are a different matter. Tests carried out for the dowel type connection used extensively in Norwegian road bridges show that such connections can in fact fail in fatigue [1]. These results, and others, have been used to calibrate the current (informative) requirements of EC5-2 on this problem (Annex A).

While vibrations of timber road bridges are normally negligible, this may not be the case for foot-bridges, which can often be quite slender. Pedestrian induced vibrations, in particular, can be a problem, and the code (EC5-2) has a separate annex (B) devoted to this problem. As an example, let us go back to the Lardal footbridge in Fig. 3, a computer model of which is shown in Figure 15.

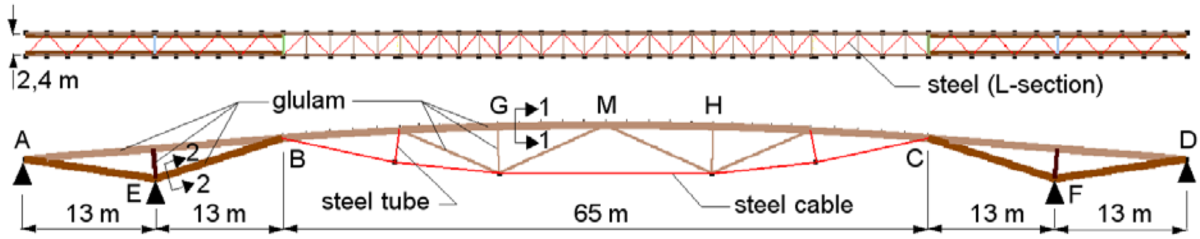


Fig. 15 Structural system of Lardal footbridge (see also Fig. 3)

On opening day a fair number of people attended which led to a dense flow of people across the bridge. And the London Millennium bridge syndrome repeated itself: very noticeable *lateral* vibrations were observed and experienced. Some people grabbed the handrails and verbally expressed concern about the behavior of the bridge. This came as somewhat of a surprise to the consulting engineers since this mode shape had not been detected by the dynamic analyses carried out during the design phase. The bridge became the subject of a PhD study [2], and the problem – which is still not resolved – is also summarized in a presentation at WCTE 2006 in Portland [3]. The mode shape causing the problems, which took a lot of model tuning to find, is shown in Fig.16.

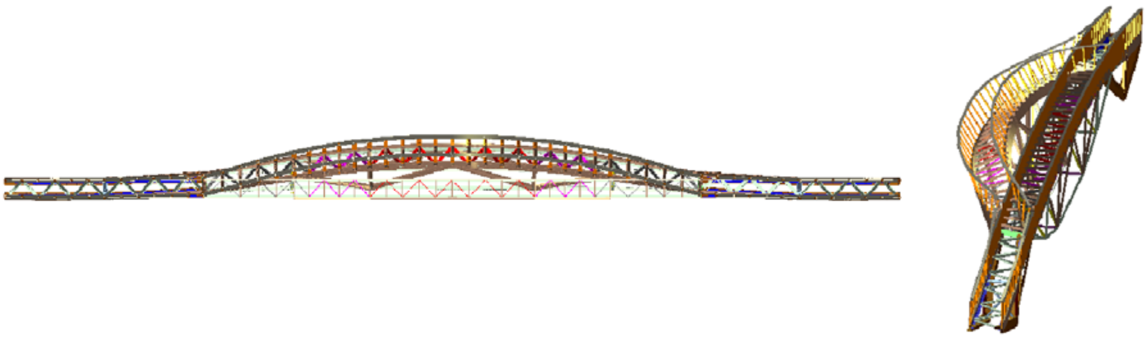


Fig. 16 First horizontal mode shape of Lardal bridge – eigenfrequency 0,83 Hz (the lowest eigenmode)

The computed eigenfrequency is the same as the value actually measured on the bridge. It should be mentioned, however, that it took quite a bit of model tuning before the measured frequency was attained. The first torsional and vertical modes, with measured eigenfrequencies of 1,12 Hz and 1,45 Hz, respectively, were not excited by ordinary pedestrian traffic.

In [?] various ways to stiffen the bridge sideways were looked into, but the tentative conclusion is that with the current ratio of width to span length, this bridge design is pushed to its limit, and perhaps beyond. After all, it is a very slender structure.

8 Protection and durability - maintenance

Bridge structures are normally designed for a long service life, from 60 to 100 years. In Norway the normal service life for a bridge is 100 years, also for timber bridges. Is this realistic without extensive and costly maintenance work? The answer to this question may differ, depending on your preferred material, and depending on where you live. Some countries are far more restrictive than others when it comes to the use of chemical treatment of the timber.

Modern timber bridges have not been around long enough for conclusive statements to be made. However, if we look at some of the timber bridges of the past, some of which has been, or were in service for more than 100 years, it seems quite feasible to build long lasting timber bridges. The main enemy is moisture. As already mentioned, rule number one is to keep the water out. That is not always possible, and even if we can make adequate cover for rain and snow, the timber will be subject to the moisture of the ambient air. It is therefore almost as important to make sure that the timber can dry out, through proper “ventilation”, as it is to keep the water out.

The resistance to moisture depends on the type and quality of the timber. Structural timber in the Nordic countries is, almost exclusively, limited to spruce and pine. Larch, which is believed to be a more durable species, is used, but not nearly as much as on the continent. Tests carried out in Norway seem to indicate that larch is not significantly more durable than heartwood of pine.

Chemical preservatives can significantly improve the durability of structural timber, in particular pressure impregnation. In the Nordic countries, only pine is a candidate for pressure impregnation, and the type of preservative is either salt, *e.g.* CCA (copper, chrome, arsenic), or creosote. Different countries have different rules for the use of preservatives. Norway and, to some extent, Finland, have up till now been quit liberal and allow both salt and creosote impregnated timber to be used in some infrastructure construction, such as bridges and power line masts and poles. Hence, the Norwegian practice up till now has been to use a “double dose” of chemical treatment of all critical components in timber bridges. This treatment consists of salt pressure impregnation of the lamellas before gluing, followed by pressure impregnation with creosote of the finished component, *e.g.* one half of a 3-hinge arch. In spite of this rather massive chemical treatment, most surfaces with a horizontal component are also “mechanically” protected, for instance by copper cladding. As an example, the top side of all arches of the Tynset bridge (Fig. 6) are covered by copper cladding, also the top side of the lower chord of the main truss arches.

Another example is shown in Fig. 17 of a more recent arch bridge, Fretheim bridge. The 3-hinge arch has a free span of about 40 m, and the picture gives a good impression of the copper “roof”. We also see how the arch support, which accommodates a (double) steel tie rod, is made to cover the entire base of the arch. Both sides of the arch are covered by a creosote Venetian blind type protection that will keep both rain and sun away from the arch, and at the same time provide adequate “ventilation” of the massive glulam arch. In this particular case the arch itself is *not* creosote impregnated, but the glulam is pressure salt impregnated. One might rightly ask if impregnation of the arch is at all necessary in this case.

Most countries have a much tougher attitude towards chemical preservatives than the current practice in Norway, and it is almost certain that Norway will soon have to impose restrictions similar to those adopted by most European countries. Fretheim bridge in Fig. 17 is an example of the kind of design that could do with very little, if any chemical protection. The glulam arch ought to survive nicely without any chemical treatment at all. The “Venetian blinds” on the sides of the arches can certainly be made without creosote impregnation; they would probably need some stain from time to time, but, if properly

Fig. 17 Fretheim bridge at Flåm in western Norway (photo: R. Abrahamsen, SWECO Gröner AS)

designed, this operation could be carried out on dismantled “panels of blinds”. If need be they could also be replaced once or twice during the life span of the bridge, without significant costs.

It is interesting to note that plans are now being worked out for some additional protection of the arches and columns of the Wennerbrücke (Fig. 4), which has now been in operation for almost 15 years. It is only the outward facing surfaces of the arches and columns on both sides of the bridge that will be covered, to protect against both rain (during windy conditions) and sunshine. The cladding will be made of wood, but the design is not yet finalized.

If properly designed and protected, the cost of the maintenance work on a timber bridge is believed to be of the same order as for concrete and steel bridges. If the timber bridge has a stress laminated timber deck, the tension bars will probably need re-tensioning from time to time. Current belief in Norway is that the frequency of this work is about 15 years. However, this type of deck does not seem to require an elaborate expansion joint the end of the deck. Measurements show that the moisture content vary very little over the year, and with its very low coefficient of thermal expansion (in the grain direction), there is very little longitudinal movements in the deck. Evenstad bridge (Fig. 12), which has a 180 m long stress laminated timber deck, has in fact no expansion joints, and you cannot see any major cracking in the asphalt surface. Some minor cracks in the transverse direction, at intervals corresponding to the distance between the cross beams, can bee seen, but they are of no great concern.

9 Erection and economy

Perhaps one of the most convincing arguments in favor of a timber bridge is its quick and relatively easy erection. Sections may be assembled on site and, due to relatively low weight, hoisted in place by mobile cranes. Figure 18 shows how a section of the Evenstad bridge across the river Glomma in southern Norway is hoisted on to its concrete pillars in the river. This bridge, erected in 1996, consists of five equal truss sections, made of creosote impregnated glulam. The span of the section is about 36 m. Each section was assembled on site and transported on a temporary “road” built into the river (see the left-hand picture in Fig. 18), and then hoisted on to the pillars (right-hand picture). With all sections in place, the temporary road was removed (in parallel) with the laying of the stress laminated deck, see Fig. 12.

Fig. 18 Erection of Evenstad bridge in 1996 (photo: Moelven Limtre AS)

All timber road bridges built in Norway during the last decade, and we now have a fair number of such bridges, has been built after careful consideration of various criteria, economy being one of the most important. In most cases timber has competed favorably on price with steel and concrete, especially if a similar design (*e.g.* arch or truss) is considered. For the Tynset bridge (Fig. 6) the most economical design was a straightforward steel girder bridge. However, the people of the village of Tynset demanded a landmark structure to replace the old, one lane, suspension bridge. An arch bridge made of steel turned out to be more expensive to build than the chosen timber design.

For certain span lengths (5 – 50 m) timber bridges, or composite timber-concrete bridges, are economically attractive alternatives also in Sweden and Finland. The bridge site can be an important factor in the choice of bridge type and material, and if quick erection is of importance, with its economic implications, timber may well win the day.

10 The Nordic Timber Bridge Project

The revival of the timber bridge in the Nordic countries is a result of the Nordic Timber Bridge Project which was carried out in three phases from 1994 to 2001. The main objective of the program was simply to increase the competitiveness of timber as a bridge material compared with concrete and steel. The program was a joint effort by Finland, Norway and Sweden. Denmark did also participate in the two first phases of the project, and Estonia was an observer throughout the project period.

The total budget of about NOK 20 million was financed by timber industry and road/bridge authorities (50%), Nordic Industrial Fund and Nordic Wood (30%) and National research funds (20%). The total project was divided into about 20 sub-projects covering the whole area, from market research and economy to structural design and durability. Each sub-project produced its own report.

Three Nordic Timber Bridge Conferences were organized as well as a number of national workshops and seminars. A number of papers and articles were published at conferences, magazines, periodicals and newspapers. An important outcome of the project was the emergence of a limited number of dedicated enthusiasts who managed to overcome some deeply rooted skepticism and were able to complete a couple of successful pilot projects.

More details on the project can be obtained from the national contact persons listed in Ref. [4].

11 Concluding remarks

The strong Norwegian angle of this chapter does not in any way imply that Norway has a leading role in timber bridge design. Rather, it is the result of the author’s experience. Some solutions presented are particular to Norway, mainly due to our fairly liberal rules concerning chemical preservatives, but hopefully the chapter will give an overall picture of modern timber bridge design. This is, however, a fairly new area and there are still many challenges, particularly concerning durability in a future that will not tolerate much use of hazardous chemicals. Connections represent another challenge, and we will probably also see new or modified support structures with novel protection schemes that will secure a long service life with moderate maintenance costs.

In Norway our modern timber bridges have been well received by both those living near them and others who use them, and it seems fair to state that timber bridges are here to stay.

12 References

1. Malo, K.A., Holmestad, A and Larsen, P.K.:
Fatigue strength of dowel joints in timber structures,
9th World Conference on Timber Engineering, WCTE 2006, Portland, Oregon, August 2006

2. Rönnquist, A.:
Pedestrian induced lateral vibrations of slender footbridges,
Doctoral Theses 2005:102, Norwegian University of Science and Technology, Trondheim

3. Rönnquist, A., Wollebæk, L. and Bell, K.:
Dynamic behavior and analysis of a slender timber footbridge,
Conference on Timber Engineering, WCTE 2006, Portland, Oregon, August 2006

4. Nordic Timber Bridge Project – contact persons:
Finland: Aarne Jutila / aarne.jutila@hut.fi
Norway: Erik Aasheim / erik.aasheim@treteknisk.no
Sweden: Martin Gustafsson / martin.gustafsson@tratek.se

Chapter No. 15 has been prepared by Professor Kolbein Bell, NTNU, Norway