

On development of network arch bridges in timber

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Summary

Inclined hangers have been used in bridges for nearly hundred years. The network arch bridge was developed in the early sixties based on steel arches, concrete decks and tension ties. The most important feature is the use of inclined and crossing hangers, which typically cross each other twice or more. The use of inclined hangers dramatically lowers the moment in the arch and makes the bridges much stiffer. By introducing this concept for timber bridges very interesting perspectives are opened, as it becomes easier to obtain the necessary stiffness and also simplified connections for the hangers may be introduced. The paper describes current work on a network arch bridge where both the deck and arch are based on timber. The chosen concept for this study is represented by a scaled experimental bridge model in the laboratory, which is used in parallel to numerical models.

Keywords: timber bridge, arch, inclined hangers, network arch, stress laminated, timber deck, experimental, numerical, scaled model

1. Introduction

1.1 Arch bridges

Many of the arch bridges built in Norway in the recent years have the arch above the deck and use vertical hangers to transfer the deck loading to the arches. Bridges having monolithic cross-section in the arches, are typically based on the three hinge statically determined lay-out. In cases with longer spans the arches are made by the use of truss work which enhances the bending capacity of the arch without much increase in material use. These design concepts are well expressed at Tynset bridge (70 m span), see Fig. 1. The connections used in the truss work consist usually of slotted-in steel plates combined with a plurality of steel dowels. For the truss work type of arches it may be more convenient to apply the two hinge arch concepts.

The size of a prefabricated bridge part depends on the curvature and length due to limitations in chemical treatment facilities as well as transport. Typically, parts exceeding the length of 30-35 m cause problems. For the truss-work type of arches this size handling problem may be overcome by

using the truss connections also as mounting connections. This can readily be done as the connections in a truss are exposed solely to axial forces. However, the splicing of arches having massive cross sections is much harder since these have to be spliced also for quite large moments.



Fig. 1 Tynset bridge, Norway (photo: K. Bell)

In bridge lay-out with vertical hangers the moment actions in the arch will easily become large due to traffic loads, introducing concentrated high forces as point loads in the arch. Furthermore, by introducing the loads into the arch by few vertical hangers, the combination of shear stress and tension normal to grain might become challenging, see more about this e.g. in [1].

1.2 Sideway stability

Usually the arches are made so slender that there is a need for sideway support to increase the stability. Having arches unsupported along the span necessitates clamping at the support in the transverse direction. In accordance to St. Venant's principle, the effect of the clamping will diminish as the arches are made longer and more slender. For long span bridges the height of the arches might be sufficient to design a frame action for sideway forces. The rigid corner of the moment carrying frame are then formed by the wind bracing at the top of the arches, connecting two arches by e.g. a K-shaped truss, carrying all forces to the supports. This has been used for Tynset bridge, see Fig. 1. In this case the hangers are vertical and hinged at both ends transferring only vertical forces and hence, no horizontal forces from the arches are transferred to the deck.

For smaller spans a simpler design has been utilized where the transverse actions have been transferred to the timber deck. This is achieved through the use of transverse U-shaped steel frames. Here, the hangers are replaced by rigid beams, increasing the necessary transverse stability properties of the arches. The pre-stressed timber decks have high stiffness and acts as a horizontal beam, transferring the transverse actions to the supports, see Fig. 1.

1.3 Durability issues

The durability requirements have mainly been met by use of chemical treatments like CU and creosote impregnation, in combination with cover of details and connections. This is the approach for durability used on Tynset bridge, see Fig. 1. It is expected that in the coming years more restrictions will be put forward on the use of environmental unfriendly chemicals and for long durability of timber bridges this can easily lead to increased costs. Considering Fig. 1, it is quite obvious that "clean" designs like the two small side span of the Tynset bridge are much easier to cover in some way than the complicated 3-D truss of the main span. An example of cover used on a simple bow-string bridge is Fretheim bridge, Norway, see Fig. 2. Here, copper cladding is used on the top faces, while the side faces is cover by ventilated venetian blinds .



Fig. 2 Fretheim bridge, Norway (Photo by SWECO, [2])

The basic principle to obtain durable timber structures is simple, it is about to keep the water out of the wooden material in order to maintain a moisture content below 18 to 20 % (in the Scandinavian climate). Susceptible points for entrance of water are everywhere, where water can be collected in liquid form like on horizontal upward surfaces, in cracks on horizontal and vertical surfaces, around details like fish-plates, screw discs, bearing shoes and in connection to drilled holes and cut-outs which are not pointing downwards. It is also to be kept in mind that water is very rapidly transported into the wooden material if end-grains are brought in contact with liquid water. More on recent findings related to durability can be found in [3] and [4].

Some countries like Switzerland and USA have several fully covered bridges showing good performance for centuries. However, this approach has mainly been used for pedestrian bridges and it is more questionable for road bridges. Heavy goods vehicles will on a rainy day transport considerable spray of water into the bridges and the cover may prevent effective ventilation and drying up. The result may be high levels of moisture content in the wood. Furthermore, the wind exposed area of fully covered bridges might be considerable, leading to strong requirements for sideway stiffness.

2. Massive Arch bridges

2.1 Inclined hangers

Many durable timber bridge designs exist, e.g. where the load carrying structures are located beneath a water tight deck and the end support details are designed so that they do not retain dirt and water. However, most building sites necessitate that the load-carrying structure is located above the deck. For traffic loading both the truss and the arch are effective structures in this case. The arch is preferred here since this structure can be more easily adopted to design for long durability.

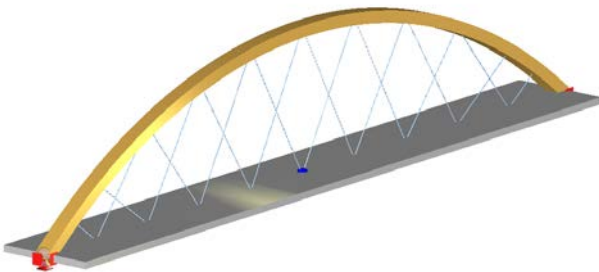


Fig. 3 Network arch, after [1]

For bridges exposed to traffic loading, the design case is often heavy loading located in a skew position. Using an arch with vertical hangers, this skew and relatively local loading will be transferred to the arch through the hangers as point loads. The result is large moment action in the arch. For the truss work arch this moment action is dealt with by a considerable increase of the internal moment arm, but it is also a possibility to reduce the moment action. O. F. Nielsen [5], nearly hundred years ago, appears to be a pioneer introducing inclined hangers in a V-configuration for bowstring arches transforming

the moment action into a more clean compressive arch bearing. This concept was further developed by P. Tveit [6] in the late fifties, letting the inclined hangers cross each other more than once, denoting it as “network arch bridge”. A summary of his design considerations is available from [7]. The effect of inclined hangers and network hanger design on bowstring arch bridges is clearly

demonstrated in [1], showing that the moment action in the arch might be reduced to roughly one quarter and a give reduced vertical displacement of the deck to nearly one sixth.

2.2 Stability of network arch

The in-plane stability of a network arch is dependent on the stiffness of the arch member as well as the deck and the distance between the hangers. In general it is forced to buckle in an inward fashion, leading to good in-plane stability.

For the out-of-plane stability, the inclined hangers do not make much difference compared to vertical hangers. For a two hinge network arch sideways clamped at the supports, the two lowermost buckling modes are visualized in Fig. 4. In fact, assuming that the load can be imposed on the arch in some way, the instability modes do not change much from an arch without hangers or a three hinged arch with the same geometry. For the three hinged arch the critical buckling loads will be only slightly smaller. Usually, the out-of-plane stability is assured by coupling two arches together with a bracing in-between. However, in the present work a different approach will be explored.

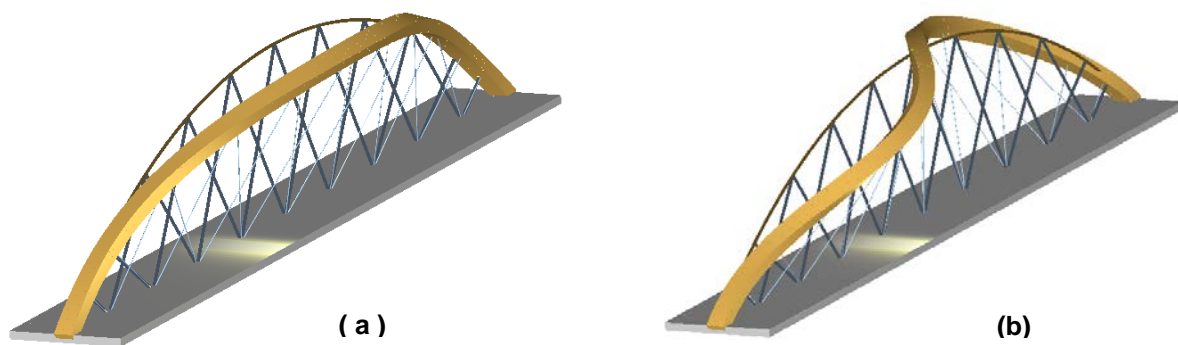


Fig. 4. The two lowermost buckling modes for an arch, after [1]

Consider Fig. 3, and assume that each of the hangers consists of two parallel rods. If the fastening points are displaced in the transverse direction in the deck for the two rods, the layout will be very similar to the familiar layout of the spokes in a bicycle wheel, see a visualization in Fig. 5.

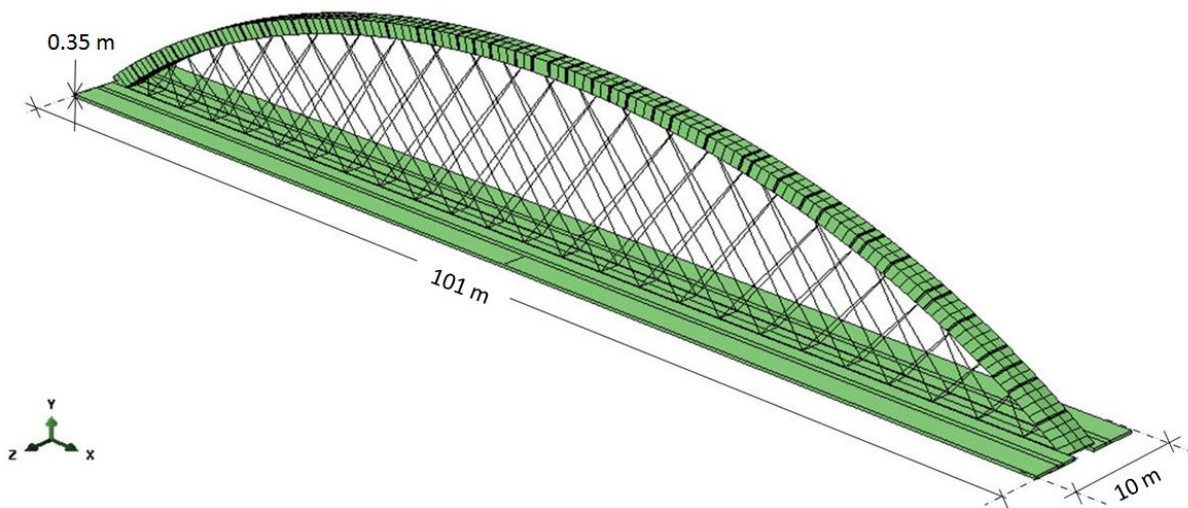


Fig. 5 Network arch with double hangers in spoked wheel configuration.

A simple model to demonstrate the possible effect on stability from this spoked wheel configuration can be made as follows. Consider Fig. 6 a), showing a situation with hangers only in the plane of the arch. If we assume that the arch tend to displace and rotate about the line between the support points, this rotation will be like a rigid body rotation and no lateral forces are activated to resist the

rotation. However, in the spoked wheel configuration, see Fig. 6 b, a lateral displacement or rotation about the support line will lead to an elongation of one of the hangers and thereby forces will develop to resist the lateral displacement.

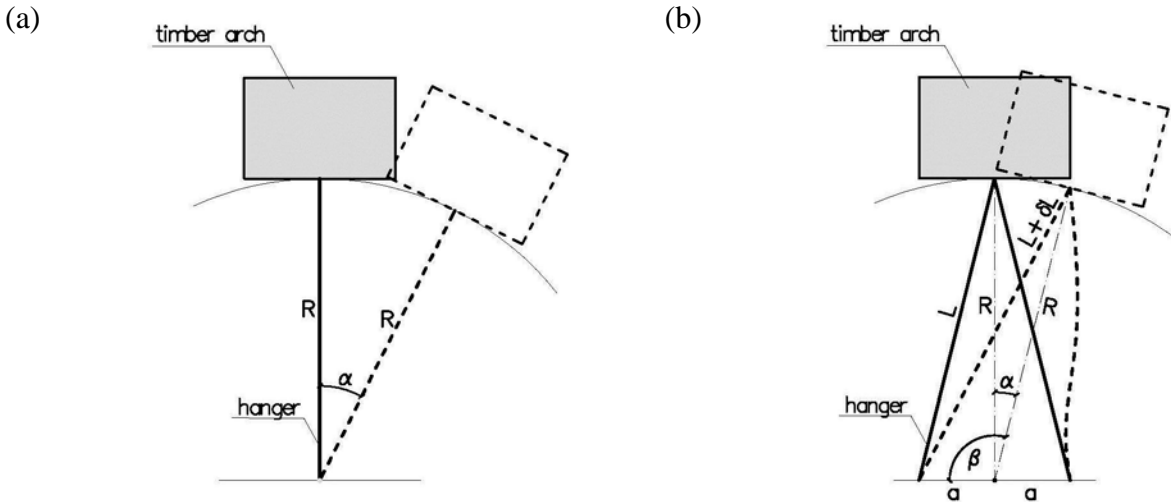


Fig. 6 Lateral stiffness from spoked wheel configuration.

A simple plane model reveals the governing parameters, consider Fig. 6 b. Assuming hangers of length L and a rigid body rotation α , the length of the tensile hanger in the displaced position is obtained by solving Eq. (1)

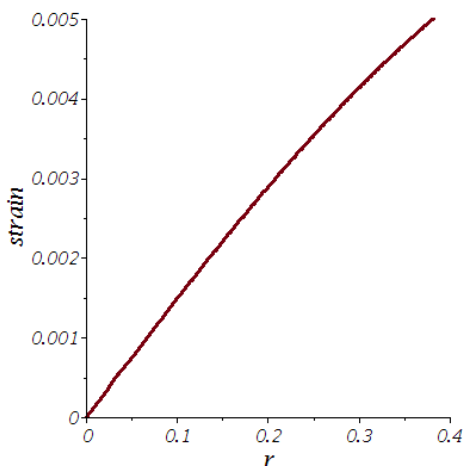
$$(L + \delta L)^2 = a^2 + R^2 + 2aR \sin(\alpha) \quad (1)$$

Here, the elongation is δL , a is the distance between the displaced fastening points of the hangers relative to the centreline of the arch, and R is the distance to the fastening point in the arch from the line of rotation. The strain in the tension hanger becomes:

$$\varepsilon = \sqrt{1 + \frac{2aR}{a^2 + R^2} \sin(\alpha)} - 1 \quad (2)$$

By introducing the geometric ratio $r = a/R$ and assuming small angles of α , a good approximation of the strain is

$$\varepsilon = \frac{r}{r^2 + 1} \alpha \quad (3)$$



A reasonable value of the rotation is about $\alpha = 0.015$ and Eq. (3) is plotted for this value in Fig. 7. Even for small values of $r = a/R$ there is considerable amount of strain in the tensile hangers with the spoked configuration. The actual values of a and R are of course dependent on the chosen design, but it is quite clear that arches having smaller curvature are beneficial from this point of view. The practical range of interest for the geometric ratio $r = a/R$ is may be up to the value of 0.4. Typically this value may be obtained close to the supports, but it becomes considerable less at mid-span. Therefore the sideways support effect will vary along the span in the same manner.

Fig. 7 Strain in hanger.

3. Concept study of long network arch bridge with spoked hangers

The major part of this study has been performed through the Master Thesis' of MSc Runa Barli and MSc Idun Hakvåg [8]. Only parts of the study will be presented here.

3.1 Conceptual design

3.1.1 Structural models

The objectives of this study were to explore if the network arch concept using inclined hangers can be combined with a light-weight deck and to study the effects of the hangers in a spoked configuration i.e. a configuration where the hangers are inclined both in the plane of the arches as well as in the transverse direction. The approach in this study has been to use a full 3-D numerical model [9] for the bridge and in order to catch the real behaviour of materials and connections, a scaled experimental model has also been built. The experimental model has been made using geometric scaling 1:10. A few exceptions have been necessary due to limitations in availability of materials, and here similar stress level has been used as the governing condition. It is believed that this has negligible influence on the overall behaviour of the bridge. Finally, to bridge the gap between the scaled experimental model and the full scale model, an accurate numerical model of the experimental layout has also been created.

The design configurations have been made on the full-scale numerical model applying the European timber structure regulations [10] and [11] and the regulations for traffic loads [12] and wind [13].

3.1.2 Design requirements

The requirements for the conceptual design approach were the following; free span of 100 m, two lines of road traffic leading to width of deck of 10 m, glulam circular arch(es), inclined network hangers in spoked configuration, tension tie, no wind truss between arches and a timber stress laminated deck. Design lay-out which is known to cause large problems with respect to moisture trapping and durability, should be omitted [3].

3.2 Design considerations

3.2.1 Inclination of hangers in the plane of the arch

The objective of the use of inclined hangers or network arch design is to reduce the moment in the arch, letting it carry the loads by compressive stresses. This lead to a great reduction in the amount of material needed for the arch. The side-effect which might occur from the use of inclined hangers is that some of the hangers may become relaxed. This can lead to buckled hangers and a visual or practical problem. But also a stability problem might occur, since the in-plane buckling length is dependent on the distance between the hangers and relaxed hangers are ineffective in this regard.

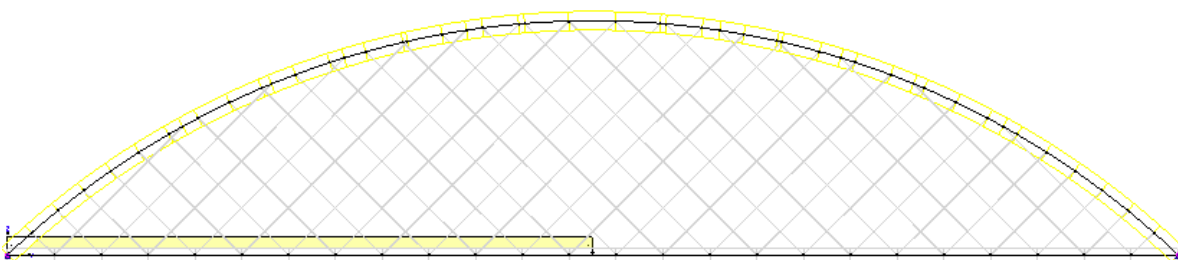


Fig. 8 Hanger layout with constant horizontal spacing and angle

The most straight-forward approach is to use a hanger layout with constant horizontal spacing and angle, see Fig. 8. From numerical simulations in the range of angles between 45 to 80 degrees (relative to the deck), it seems like the best choice is 45 degrees since this will lead to few relaxed hangers for skew loading. Imagine that the hangers occur in pairs from the fastening point at the deck and that they approximately have the same force, the resulting force in the pair of hangers will be vertical for all fastening points.

The idea behind the next lay-out is visualized in Fig. 9, to the left. To maximize the effect of arch bearing, it is best to impose the loading along radial rays with constant angle between adjacent rays. However, using only single rods along the rays when only one or a few of the rays carry loads, leads to large moment action in the arch and possible instability due to relaxed hangers. To spread out the force, pairs of hangers are introduced. In this lay-out the resultant of the forces in the pairs of hangers follow the direction of the radial rays.

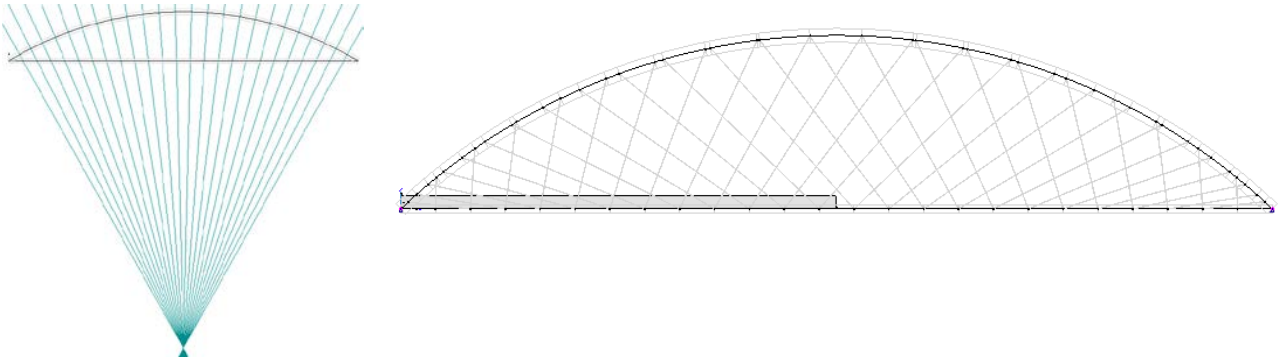


Fig. 9 Hanger layout with radial resultants of pair of hangers

Ideally the spacing along arc length should be kept constant, and this will lead to varying spacing of the fastening points in the deck. This might be a good choice if the deck has sufficient stiffness for the largest distance between the fastening points.

A concept which is close to the ideal situation is also gained by choosing equidistant fastening of the pair of hangers along the deck. In this case the fastening points along the arch will vary and be determined by the spreading angle of the pairs of hangers. This design concept is depicted to the right of Fig. 9. The angle between the intersecting pairs of hangers in this case is close to 90 degrees. Numerical simulation on this lay-out does not show any relaxed hangers, and this lay-out was chosen for further evaluations.

3.2.2 Stability

The height (depth) and width of the cross-sections together with the rise to span ratio and the out-of-the-plane support condition of the arch strongly influence the stability properties. The in-plane stability of a network arch is high and it may be reasonable to let the width of the cross-section be larger than the height in order to increase the out-of-plane stability. Furthermore, although the compressive forces in the arch will increase together with a slight increase in moment actions, it will be beneficial for the sideways stability to use an arch with quite low rise to span ratio. Suitable range for the rise (bow height) for arch bridges is about 0.1 to 0.25 of the span. In this case it was found that 0.14 is a reasonable good choice.

Moreover, to benefit from clamped conditions in transverse direction, It was chosen to let the width of the cross-section be 1.5 times the height, leading to an increase of the sideways stability properties of about 50%.

3.2.3 Transverse inclination of hangers

As observed from Eq. (3) as well as from Fig. 7, the transversal support from the transversely inclined hangers is almost linear with the ratio $r = a/R$. Therefore, the transverse distance between the fastening point of hangers ($2a$) should be made large and the rise of the arch small in order to improve sideways stability.

The area determined by the vertical projection of arches onto the deck cannot be used for traffic purposes since the entrance will be blocked by the arches. Therefore, this area can be used for inclined hangers and eventually guard railings. Without exceeding this area the transverse distance between the fastening point of spoked hangers ($2a$) is limited to the width of the cross-section.

3.3 Design for full scale 100 m bridge

The chosen design is not optimized, it is meant for conceptual study. A key point is the sideway support, and it is here chosen a relatively thick timber deck and weak transverse beams in order to perform further studies of the requirements for sideway stability.

The design for the full scale bridge leads to the following geometry and materials; The distance between the supports is 100 m and the rise of the circular arch is set to 14 m. The two hinge arches are in glulam, Scandinavian quality CEL40c (close to European GL 32c). The cross-section of the arches has width equal 1.8 m and height 1.2 m, constant along the span. A stress-laminated timber deck is used with width of 10 m and thickness of 1 m. Under the timber deck transverse steel beams, IPE 400, are used with spacing of 4 m. All hangers occur in pairs both in-plane and in the transverse direction, and are fixed to the top face of transverse beams and to the and the bottom face of the arches, see visualization from the numerical model in Fig. 10. All hangers are steel rods with diameter equal to 40 mm and they are fastened by axial load carrying screws to the wood pointing in the same direction as each pair of hangers

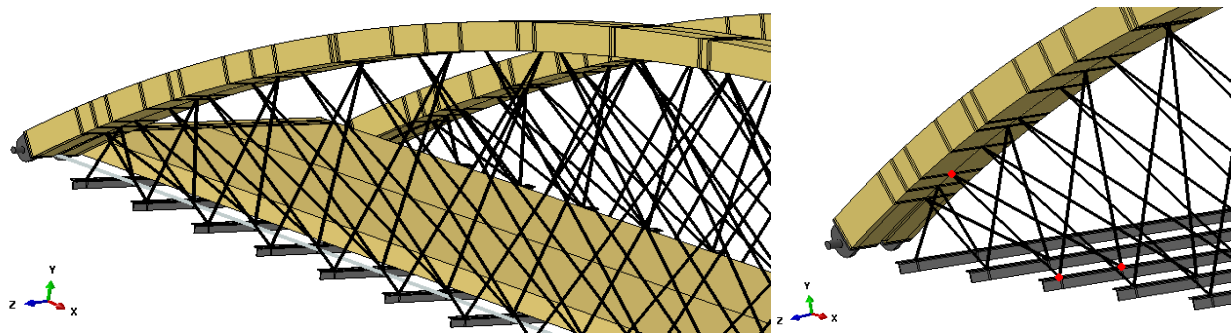


Fig. 10 Fastening of hangers to the transvers beams (numerical model)

The design was mainly performed by numerical non-linear 3-D simulations using Abaqus [9]. The investigations with respect to the stability were made by first imposing the dead weight and wind loading in the transverse direction. The deformation of the arches due to wind loading is then used as out-of-plane deformations for the consecutive stability analyses increasing the dead and live load until instability occurred. The analyses showed that the stability with respect to the load carrying capacity was good, the design load level could be increased by a factor of about 11 before instability occurred (exclusive load combination factors).

4. Scaled laboratory model

4.1 General

The 1:10 scale experimental model of the chosen bridge design is shown in Fig. 11 in finished condition. The experimental model will be further used as basis for future studies where different design options will be investigated. The arches, the timber deck, the support hinges for the arches and the tension ties will be kept as here, but different options for hangers and sideway support will be investigated. In the present study most emphasis has been put on stiffness and dynamic properties in order to spot the governing parts for possible improvements. The load capacity and safety of the bridge appears to be good, and therefore the main focus has been on the properties in the serviceability condition.

4.2 Arch support

The support of the bridge is collected on one square hollow section (HUP 120-120-10) beam in each end. The deck rests on a small square hollow section (HUP 50-50-5) and is fixed for sideway movements by adjustable steel support plates, visible on Fig. 11 and Fig. 12. The arches are mounted in the supports by installation of the pins.



Fig. 11 Experimental model in scale 1:10

Ideally, the arches should have been fully clamped with respect to sideways stability, but this is of course not achievable in reality. There will be some clearance around the pin in the connection and some flexibility in the fixing of the plane support plates against the mating compression surface of the wooden arches. However, it has been previously observed quite rigid behaviour using inclined threaded rods [14], so for this prototype self-tapping screws have been used experimentally model the behaviour of connections using threaded bars.

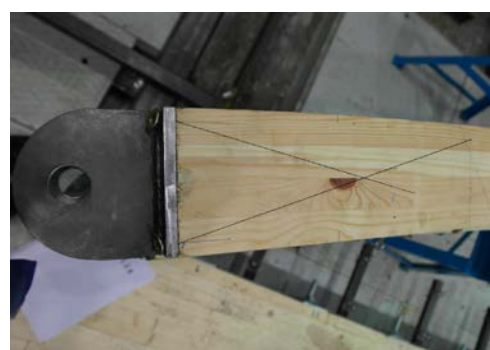
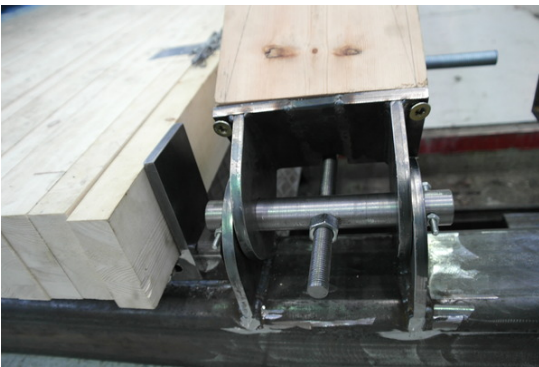


Fig. 12 Support conditions; hinged in the plane of the arch, transversely rigid

Four screws has been used, one in each corner of the supporting steel plate. The inclination of the screws is roughly 15 degrees in one plane and 8 degrees in the other. The inclination of the screws at the support detail is visualized by the drawn lines on the wood surface. Tensile rigidity is gained by the axial stiffness of the screws. The shear forces in the connection are addressed by the transverse force component of the inclined screws.

4.3 Hangers

A very simple fastening technique has been used to fix the hangers to the wooden arch. The hangers are mounted by threads in a small steel piece which in turn are fastened to the wooden arch by just using axial load carrying screws installed in the direction of pair of hangers; see the details on Fig. 13. The fastening of the hangers to the transverse beam is made using eye bolts and turnbuckles. The transverse beams are fastened on the bottom face of the deck by self-tapping screws.



Fig. 13 Fastening of hangers to the wooden arch.

5. Structural behaviour of bridge

The load bearing capacity and safety of the bridge appears to be sufficient, and it is deformations and vibrations which become the most challenging subjects. Bridges exceeding span length sixty to eighty meters tend to get so low fundamental frequencies that wind and traffic loading may induce dynamical effects. The interesting parameters for evaluation of the bridges than become stiffness and mass distribution, eigenfrequencies and vibrational modes, acceleration levels and damping characteristics.

5.1 Scaled models of the timber deck

The amount of wood material in the timber deck is roughly twice of that in the arches. The deck is an important part of the bridge and has a great influence on the structural behaviour of the bridge. The stress-laminated deck height in the scaled model is 98 mm and the deck was laid out on a horizontal floor and pre-stressed to nominal stress of 1.0 MPa. The evaluation here was done prior to mounting of the transverse beams. After pre stressing, the deck was put on two simple beam supports and the deformation due to the self-weight (measured to 560 kg) was compared to the simple beam formula. The measured deformation was only 4.4 % larger than the theoretical result for a beam having a massive cross-section.

The dynamic behaviour of the deck alone was also characterized by experiments and numerical simulations. The experimental results were obtained by using an impact hammer and the roving hammer test (RHT) protocol. The input to the numerical simulation was made on the basis of measured stiffness properties of the timber deck material and on the measured total weight of the deck including the pre-stressing bars. The measured mode shapes is plotted in Fig. 14, while the results obtained from the numerical simulation is given in Fig. 15. The two figures should not be

directly compared as the experimental results are based on the experimental grid which does not fully cover the experimental model. However, as it may be observed, there is a high degree of similarity for the three lowermost modes. Only the lowest vibrational mode was measured in the transverse horizontal direction. The measured and numerically obtained results are given in Table 1. Note also that the damping can be characterized by these types of experiments.

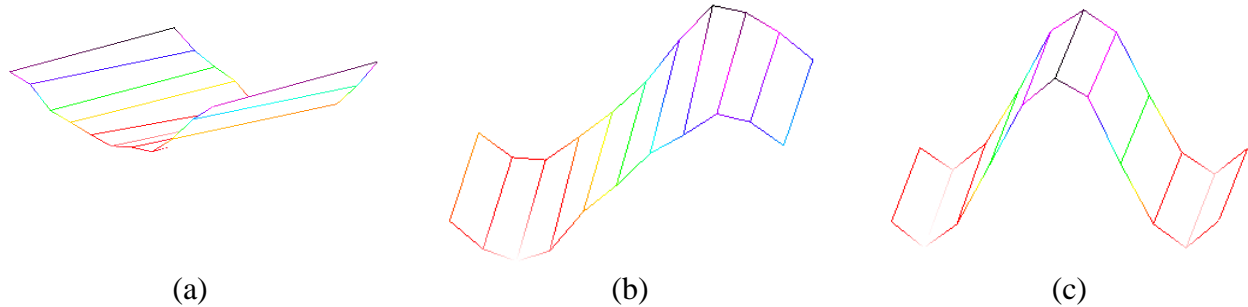


Fig. 14 Measured vibrational modes in vertical direction, experimental model of timber deck.

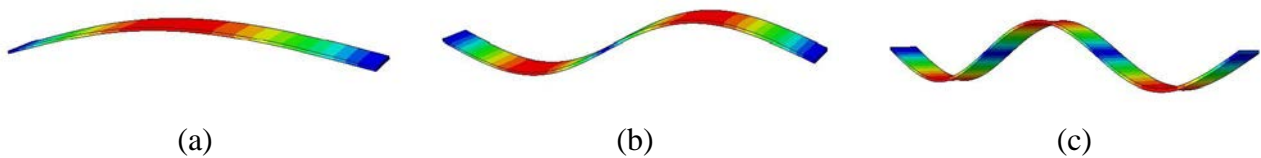


Fig. 15 Numerically obtained vibrational modes in vertical direction of timber deck

Table 1 Measured damping, modes and frequencies compared to numerically obtained frequencies

Mode	Measured damping [%]	Measured frequency [Hz]	Numerical frequency [Hz]	Deviation, frequency [%]
Vertical 1	4.2	3.0	3.2	5.8
Vertical 2	0.63	8.0	7.5	6.3
Vertical 3	0.95	17.5	16.9	3.3
Horizontal 1	2.4	17.8	18.2	2.2

From the above results it can be concluded that the stress-laminated deck behaves like a massive wooden block (with additional mass from the steel bars) with respect to dynamic properties, given that the pre-stressing is sufficient.

5.2 Scaled models of the bridge

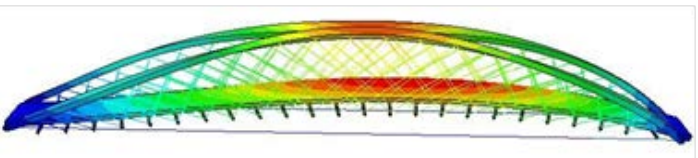

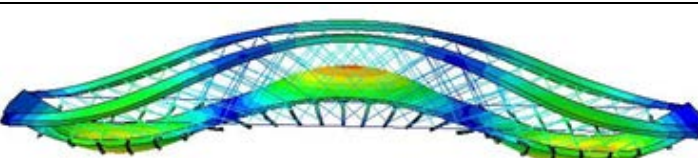
5.2.1 Vertical vibrations of the deck

After the deck was mounted and the hangers attached to the deck and arches, similar experiments as performed on the deck were also done here. In the impact hammer experiments only the deck were impacted using the same grid as for the deck alone. For the results presented in Table 2 the deck was impacted in vertical direction and only vertical deformations were registered. The evaluations of the measurements are based on an FFT transformation of the load and response, leading to frequency response functions. It should also be added that this measurement method is dependent on a certain minimum response and in this case the response corresponding to the lowest mode (mode 1) was almost absent. In the frequency range corresponding to the lowest vertical mode of the deck, the frequency response function was almost flat and hence no corresponding frequency could be extracted. Nevertheless, from the numerical models a mode 1 frequency could be determined, but more information is needed to evaluate the sensitivity of the structure to mode 1.

Axial vibrations are independent of scaling, while vibrations due to bending will be scaled with a factor inverse proportional to the geometric scaling factor. If the measured frequency in a 1:10 scaled model equals 1.0, only 1/10 of that frequency will occur in the full scale model for the bending case. As observed from the two rightmost columns of Table 2, the modes which to a large extent are dependent on bending of the deck are quite close to this. It should also be noted that the current full scale numerical model has nominal material data, while the scaled numerical model is based on measured material data from the experimental model. The influence from connections on scaling of model results is very uncertain, but considering the quite close resemblance between the experimental and the numerical scaled model, it seems like this effect is small here.

Based on these results it is concluded that no problems with vertical vibrations for this structure is expected as the fundamental frequency is quite high.

Table 2 Vibrational modes with vertical vibrations of the deck

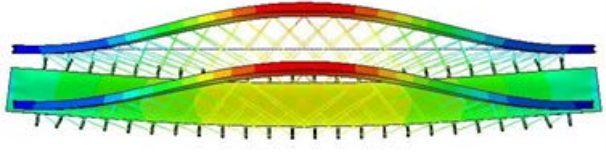
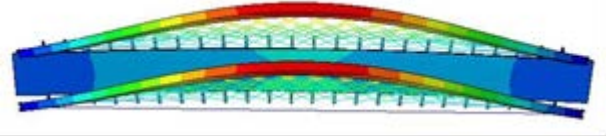
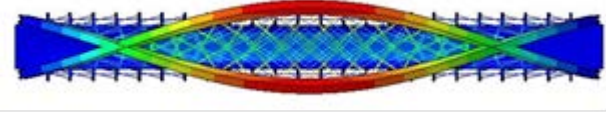
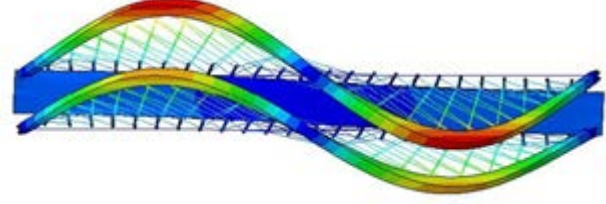
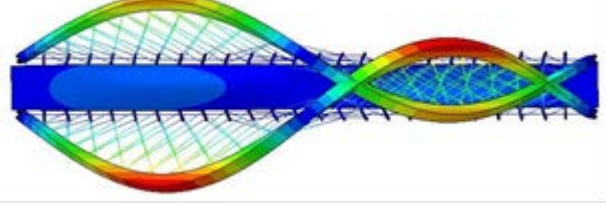
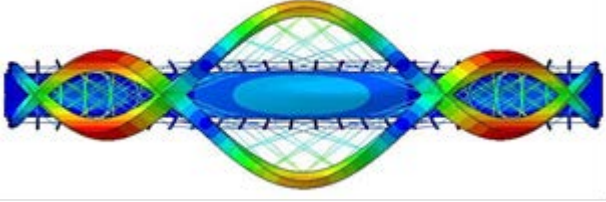

Mode shapes with deck vibrating in vertical direction	Mode	Measured experimental model 1:10 Frequency [Hz]	Numerical model scale 1:10 Frequency [Hz]	Numerical full scale(1:1) Frequency [Hz]
	1	none	26.5	2.95
	2	28.5	24.7	2.27
	3	43.5	42.2	3.99

5.2.2 Horizontal vibrations

A special evaluation of the deck was performed in order to evaluate the effect from the transverse beams, the hangers and their attachments, as well as the end support. Here, the deck was impacted on the side face in horizontal direction and the response was collected solely from the deck in the horizontal directions as well. Similar numerical simulations were also performed. The results are given as mode 1 in Table 3. More mass has been added and as expected the fundamental frequency for the horizontal mode of the deck has been reduced from about 18 to 16, and excellent agreement between the numerical scaled model and the test is achieved. From the full scale numerical simulation it is observed that the frequency decrease less for the higher modes than expected from bending response. Maybe some effects from the hangers come into play here.

Further evaluations of the mode shapes and eigenfrequencies have been based on numerical simulations. The results are visualized and quantified in Table 3. Although six modes of vibration, greatly exaggerated on the plots, are shown in Table 3, the difference between mode a and b is just dependent whether the arches are in-phase or out-of-phase relative to each other. All these three modes show a local bending vibration of the arches in a sideways manner, but almost no horizontal movements of the deck. It is therefore concluded that this vibration will not cause any problematic accelerations of the deck or the live load. However, as the frequencies are so low that wind-induced loading might give dynamic effects and this should be investigated further.

Table 3 Vibrational modes mainly in horizontal direction

Mode shapes with vibrations mainly in horizontal direction	Mode no	Measured experimental model 1:10 Frequency [Hz]	Numerical model scale 1:10 Frequency [Hz]	Numerical full scale(1:1) Frequency [Hz]
 <p>Horizontal deck impact</p>	1	15.9 – 16.4	15.9	1.98
	1a		8.66	.809
	1b		9.29	0.814
	2a		16.83	1.787
	2b		16.86	1.802
	3a		32.18	3.66
	3b		32.59	3.67

6. Conclusive remarks and acknowledgements

The network arch bridge is a very efficient bridge concept which may enhance the competitiveness of timber bridges. A network arch bridge becomes very stiff in the plane of the arches. It is also possible to use this concept to build long bridges without the need for truss-work for wind forces or stability, by using hangers in a spoked configuration, i.e. hangers which are inclined both in the plane of the arches and in the transvers direction. The paper has presented some work on a prototype of this concept with promising results. More work on the concept is underway.

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