

Large Glulam Arch Bridges - A Feasibility Study

Kolbein BELL

Professor

NTNU

Trondheim, Norway

Kolbein Bell, born 1939, received his civil engineering degree from the Norwegian Institute of Technology (NTH) in 1962, and his dr.ing degree from the same university in 1968. He has been professor of structural mechanics at NTH/NTNU since 1981.



Espen KARLSRUD

Structural engineer

Spenncon Trøndelag

Trondheim, Norway

Espen Karlsrud, born 1976, received his civil engineering degree from the Norwegian University of Science and Technology (NTNU) in 1999.



Summary

A new 3-span timber bridge, to be built at Tynset, Norway, spans the main span of 70 m by two parallel glulam truss arches. Each of these arches supports 12 vertical steel hangers. This paper reports on an investigation (carried out as a student diploma work) of an alternative design using inclined hangers. It is shown that with inclined hangers it is feasible to replace each truss arch by a single massive glulam arch. The pros and cons of the alternative design are discussed.

Keywords: Timber bridge, network arch, glulam arch, inclined hangers.

1. Introduction

During 2001 a new major timber bridge for ordinary road traffic will be built at Tynset, Norway. The support structure consists of three circular arches, two small ones of massive glulam with a span of about $26,5\text{ m}$ and one large arch with a span of 70 m , see Figure 1¹. The main arch, which is a glulam truss arch, supports 12 vertical steel hangers, each of which carries a secondary steel beam supporting a stress laminated timber deck. The bridge has two lanes for ordinary road traffic and one lane for light (pedestrian/cycle) traffic. The total width (between the parallel arches) is $11,7\text{ m}$.

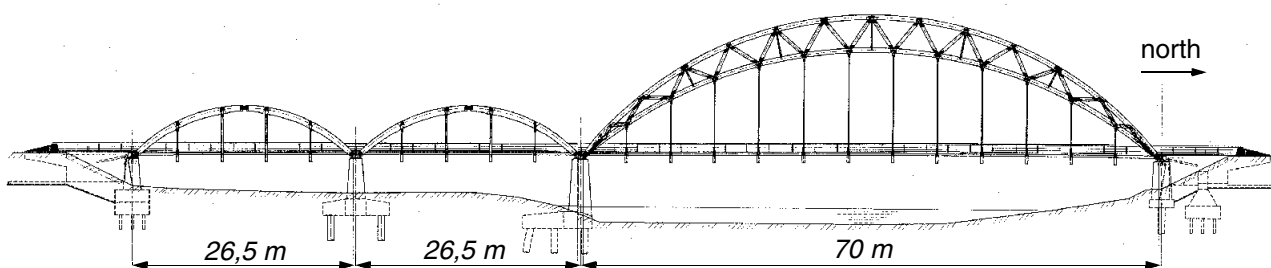


Figure 1 The new Tynset road bridge

In this paper we focus on the main (70 m span) arch. As designed the top and bottom chords of the glulam truss arch have a massive cross section of $710 \times 600\text{ mm}$ and $710 \times 560\text{ mm}$, respectively. Each diagonal is made up of two glulam beams each having a cross section of $240 \times 400\text{ mm}$. Over the lowest 10 m the width of the top chord is increased from 710 to 1100 mm at the base. The sideways stiffness is provided by a glulam wind truss between the top chords of the arch trusses; the bottom chord is stiffened by a total of 5 inclined stiffeners between the chord and the transversals of the wind

1. Reproduced with permission of the Norwegian Public Roads Administration.

truss. The free height is 6800 mm . The vertical hangers are solid steel bars with 70 mm diameter. The total amount of glulam in one truss arch is approximately 85 m^3 . The horizontal thrust from the arch, which can be in the order of 3100 kN , is taken by the concrete foundation/pillar.

2. The network arch

It is well-known that for a bridge, which is subjected to large concentrated (axle) loads, inclined hangers between arch and deck structure reduce the bending moments in the arch, and thus lead to more favourable stresses in the arch than do vertical hangers. Per Tveit has been a warm spokesman for the *network arch* in Norway since before he took his doctorate on this subject in the early sixties. He has also designed two network arch bridges built in Norway (in 1963 and 64), both with steel arches (with spans of the order of 80 m) and concrete decks. These bridges are arguably the most slender arch bridges in our country, and they have both functioned, and are still functioning, very well. Anyone interested in this type of bridge should consult Tveit's most recent work on the subject [1] which is available on the Internet and which has a very extensive list of references.

The full potential of the network arch can only be realized with many hangers crossing each other, and it is therefore best suited for slab type decks that do not rely on transverse, secondary beams. With a stress laminated timber deck, as in our case, transverse secondary (steel) beams cannot be avoided. Since the dimensions of the secondary beams are determined by the axle loads, economical arguments suggest that we keep the number of such beams to a minimum, which again puts restrictions on the number of hangers. Nevertheless, the network concept is an interesting one, and preliminary analyses of a truss arch, similar to the one in Figure 1, with inclined hangers, prompted us to try a single, massive arch.

3. Alternative design for the main span of Tynset bridge

3.1 Basis

The alternative design assumes the same deck structure and traffic lane arrangement as the selected design. Hence all dead load (deck, asphalt, barriers and secondary beams) is the same, except for one minor difference: we use one more secondary beam. The traffic loading, which is defined by the Public Roads Administration's specifications for standard road bridges, is also the same. The vehicle lanes are next to each other on one side of the bridge. We consider the highest loaded arch closest to the vehicle lanes.

3.2 Hanger patterns

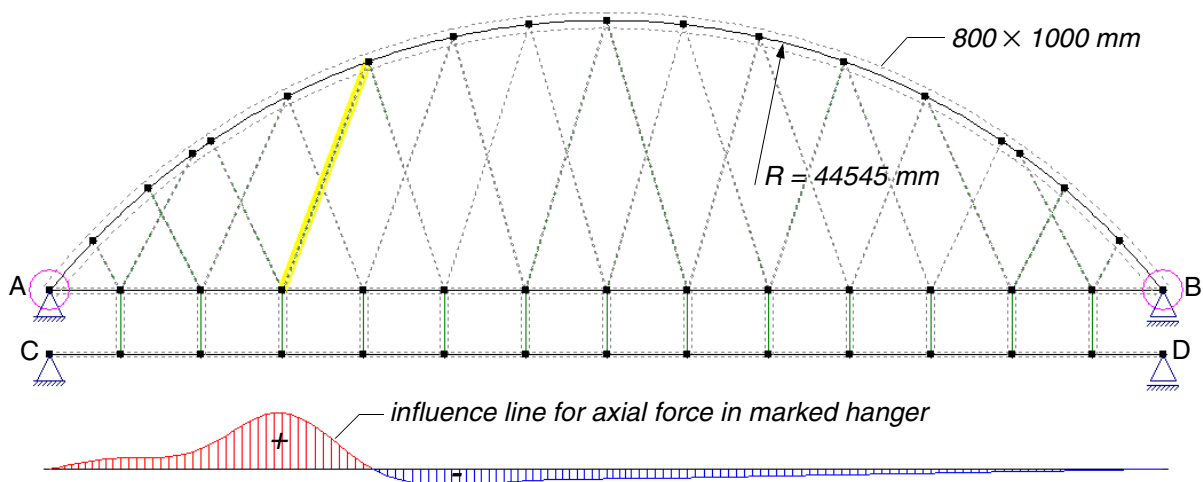


Figure 2 Arch with steeply inclined hangers

Figure 2 shows the 2D computational model of a preliminary design of a network arch. The arch has the same height as the top chord of the truss arch in Figure 1. The straight 'element' from A to B is a 'tension or tie rod' (in the shape of a standard hot rolled HEA or HEB section). The deck itself, symbolized by the straight 'element' from C to D, is, for reasons of clarity, suspended an arbitrary

distance from the tie rod by very stiff and weightless bars (one for each secondary beam). The inclined hangers are fastened to the HEB tie rod at the point of support of the secondary beam. Two hangers per secondary beam clearly limits the number of hangers. The thirteen secondary beams are spaced $5,1\text{ m}$ apart except the first and the last which are $4,4\text{ m}$ from the supports. In Figure 1 the maximum distance between the secondary beams is $5,4\text{ m}$. The designs in Figures 1 and 2 differ not only with respect to the arch and the hangers, but also in the way in which the horizontal thrust is controlled. While the selected design uses the foundation to absorb these significant forces, the alternative design uses a tie rod.

According to Tveit [1] the arch in Figure 2 is not a true network arch since each hanger only crosses another hanger *once*. The hangers form a moderate angle with the vertical, and the result is that for certain positions of the traffic loading many hangers become 'slack' - they *relax*. The influence line in Figure 2 shows that with traffic loading only on the right-hand 2/3 of the deck, the marked hanger will take compression (which it is permitted to do in this particular model). With a proper model, in which the hangers can only take tension, the result is that significant bending moments are introduced in the arch, and its overall performance is not all that good. In fact, the proposed cross section of $800\times 1000\text{ mm}$ will not have sufficient capacity, even with a continuous two-hinge arch as shown.

Trial and error with a number of different models, all of which had the arch geometry of Figure 2, led to the design shown in Figure 3. This is a true network arch, as most hangers cross three other hangers.

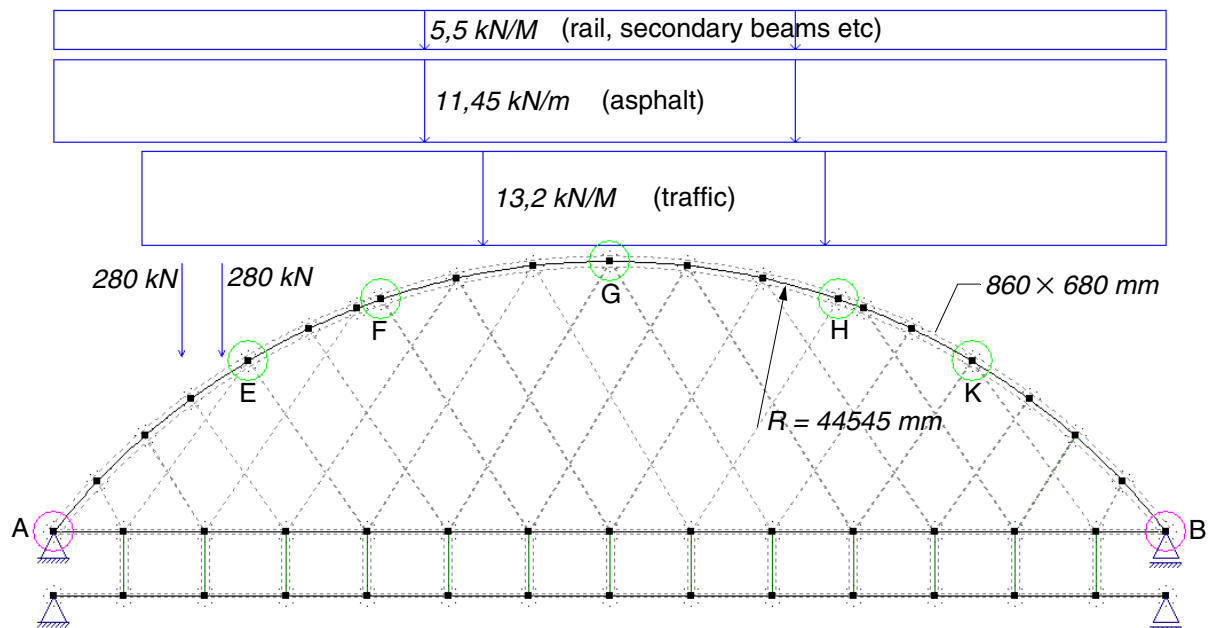


Figure 3 The 'best' network arch alternative for Tynset bridge

The arch in Figure 3 is drawn to scale, and it should be noted that the cross section is only slightly larger than the cross section of the upper chord of the truss arch in Figure 1! This is clearly a very slender timber structure. But will it perform satisfactorily under traffic loading? We believe it will.

3.3 Analysis and design

The main characteristics of the structure in Figure 3 are: Arch of glulam (GL36c) with cross section of $860\times 680\text{ mm}$, $\text{Ø}50\text{ mm}$ steel hangers (of standard steel quality), and a 'tie rod' made of a standard hot rolled steel section of type HEB300 (or HEA360). The secondary steel beams are of the same type as those used in the chosen design (in Figure 1), and so is the stress laminated deck (made of 48 by 223 mm creosote impregnated planks).

The glulam arch is made up of 4 arches, each having a cross section of $215\times 680\text{ mm}$. These arches are mechanically joined together, see Figure 6. The two outer beams are made in four segments, and joined at points E, G and K (Figure 3), whereas the two inner beams are made in three segments and joined at points F and H. We assume the combined arch section to behave as a massive section.

However, at points E, F, G, H and K, where two and two of the individual arch segments are joined, we introduce hinges with rotational springs. The stiffness of these springs is conservatively set to 10000 kNm/rad . A series of analyses, including linearized buckling analyses, show that the results are not very sensitive to this spring stiffness. Let us for instance examine the load combination shown in Figure 3 (the weight of the structure, including the deck, is automatically computed and included by the program). Based on a series of influence lines, we have found this to be the traffic load situation that will cause the highest combined bending and axial stresses in the arch (all the loads in Figure 3 are without load factors). Defining the capacity index (κ) equal to $1,0$ for a fully utilized cross section, we find that the highest value of this index for the arch (κ_{max}) and the buckling factor χ varies with the spring stiffness k_ϕ as follows:

k_ϕ [kNm/rad]	1000	10000	100000	∞
κ_{max}	0,744	0,740	0,729	0,724
χ	10,9	12,2	15,1	15,5

The buckling factor χ (determined through an eigenvalue analysis) times the entire load combination (including the dead load) yields the (1st) buckling load.

The capacity index for combined bending and axial compression, which we have not found to exceed $0,74$ for the arch and hanger pattern in Figure 3, is highly dependent on the assumed buckling lengths. For in-plane buckling the inclined hangers very effectively keep the buckling lengths low, and an estimate, based on the linearized buckling analyses, shows that 8000 mm is a fair (but conservative) value for the in-plane buckling length of the arch. In order to estimate the out-of-plane buckling lengths we need to consider the stiffness provided by the wind truss. The proposed wind truss is shown in Figure 4 as a plane structure, obtained by ‘straightening out’ the circular surface to become a plane. In other words, the length of the chords of the wind truss (which are assumed to be

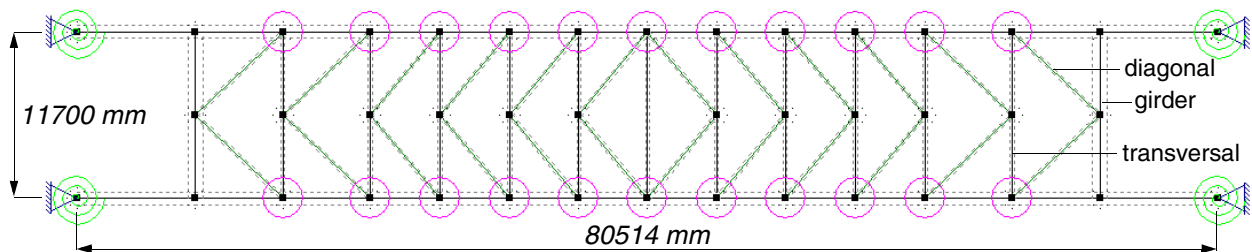


Figure 4 Plane (2D) model of the wind truss

continuous) are equal to the true length of the arches.

The first (and last) transversals, called girders in the Figure, are fairly solid glulam beams, with $220 \times 1000 \text{ mm}$ cross sections. These girders, which are assumed to be rigidly connected to the arches, are placed such as to give a free height for traffic of about $5,0 \text{ m}$. The remaining transversals (which are continuous) are modelled as beams with moment-free connections to the chords (arches), and the diagonals are modelled as bar elements. The glulam transversals and diagonals both have cross sections of $280 \times 320 \text{ mm}$. The arches are assumed to be semi-fixed at the supports, as indicated in Figure 4. The stiffness of the rotational springs must be provided by accurate fastening to fairly stiff transverse (steel) beams.

Subjected to a uniformly distributed transverse wind load of $1,4 \text{ kN/m}$ on each chord (arch) the maximum transverse displacement at mid-span is of the order of 50 mm , and there is no capacity problem. In fact, most transversals and diagonals have a very low capacity index, and, from a strength point of view, their cross sections could be reduced.

In order to obtain information about the out-of-plane buckling lengths for the arches, the following reasoning is adopted: From the analysis of the problem in Figure 3 we find that the axial force in the arch is approximately 3300 kN for the lower part of the arch, and it is nowhere smaller than 2400 kN . Relaxing the horizontal constraint at the two right-hand supports in Figure 4, and applying axial loads of 3300 kN at each of the two supports (which will simulate a constant axial compression in the

arches), we obtain a reasonable model for the out-of-plane stiffness. A linearized buckling analysis of this problem gives the buckling shape shown in Figure 5. The corresponding buckling factor is

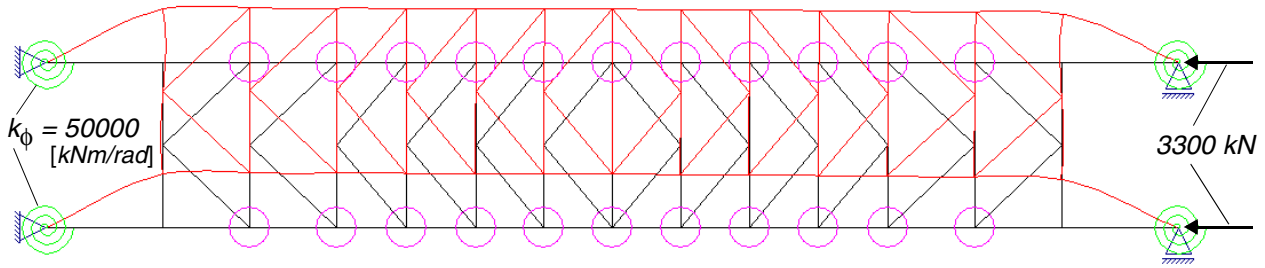


Figure 5 First buckling mode - buckling factor = 4,13

4,20, and from this we derive a critical axial force in the arch of $3300 \times 4,13 \approx 13600 \text{ kN}$. Introducing this as the Euler load in the classical formula we can estimate the buckling length to be:

$$P_{cr} = \pi^2 \frac{EI}{l_k^2} = 13600 \quad \Rightarrow \quad l_k = \pi \sqrt{\frac{EI}{P_{cr}}} \approx 15000 \text{ mm}$$

The capacity index of 0,74 quoted above is based on this out-of-plane buckling length for the lower part of the arch. Once we get 'inside' the wind truss the buckling lengths reduce to the length between the nodal points.

The buckling factor in Figure 5 clearly depends on the rotational stiffness at the supports (*i.e.* on the value of k_ϕ). If we disregard the rotational constraint, *i.e.* $k_\phi = 0$, we find a buckling factor of 2,10. This corresponds to a buckling length of approximately 21000 mm, which in turn results in a capacity index of 0,96. If we go to the other extreme and assume no rotation at all about the vertical axis of the base of the arches (*i.e.* $k_\phi = \infty$), we find a buckling factor of 7,70 and a corresponding buckling length of approximately 11000 mm. This will give a maximum capacity index for combined bending and axial compression of 0,64.

From the above discussion we conclude that the network arch in Figure 3 along with the wind truss in Figure 4 have sufficient capacity in the ultimate limit state. Hardly any hangers relax, regardless of how we position the variable traffic loading. We have found not more than one relaxing hanger for any given load combination. As a result the force distribution in the arch is very favourable. The largest design moment we have found does not exceed 600 kNm, and the highest and lowest compressive forces we have found in the arch (in any load combination involving traffic loading) are 3400 kN and 1600 kN, respectively. We have not found tension forces in any hanger in excess of 600 kN, nor have we found forces in the tie rod exceeding 2750 kN.

The network arch in Figure 3 also performs well in the serviceability limit state. The maximum vertical displacement of the deck, for dead load only, is about 60 mm, and for dead load plus traffic load it is about 115 mm. The latter (which is equivalent to approximately $L/600$) is no more than the corresponding displacement of the selected design in Figure 1.

3.4 Details, production and erection

The focus in this investigation has been on the capacity and performance of the network arch. Many important details have received only limited attention. One possible solution for the connection of the individual glulam arches and the fastening of the hangers to the arch is indicated in Figure 6. Note also the protective 'roof'. The hanger is fastened to the arch, via a system of welded steel plates (8 to 12 mm thick), by $\text{Ø}20 \text{ mm}$ bolts, each reinforced by six 4 inch shear plates. Each such combination of bolt and shear plates has a design capacity of approximately 200 kN parallel to the grain and approximately 145 kN perpendicular to the grain. This means that 4 bolts should be sufficient for all hanger connections, also for those with two hangers in the same connection. When one hanger has its maximum force, the force in the other is very moderate.

Very important details, for which we offer no definite solutions, are associated with the lower part of the arch, including the fastening of the arch base to the point where the 'tie rod' and the last (and first) secondary beam meet, as well as the fastening of the wind truss girder beam to the arch.

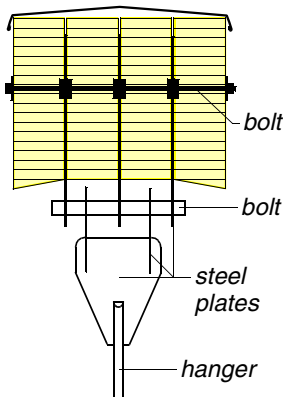


Figure 6 Fastening of hanger

The sideways stability of the arches requires a high out-of-plane bending stiffness of the lower part of the arches. It may be difficult to obtain this stiffness by four arches bolted together, and it may therefore be necessary to glue the lowest four sections together at the factory. This should be possible since we would suggest salt, but not creosote, impregnation of the arch, and hence no limitation due to the size of the creosote pressure tank. This would however, require some adjustment of the width of the individual arches, and would also necessitate slotting in the steel plates for the hanger fastening in this part of the arch.

We believe the alternative design may have some merit when it comes to erection. The following sequence is suggested for the main arch span, assuming the foundations (pillars) to be in place and that the terrain is sufficiently flat to the north of the bridge (see Figure 1), which in fact it is: 1) Each of the two arches are bolted together lying down on the ground to the north of the bridge. 2) The tie rods are put in place, and most (perhaps all) hangers are fastened to both arch and tie rod. 3) Each

network arch is lifted (rotated 90 degrees about the tie rod) to an upright position, and the wind truss is installed and fastened to the arches. 4) The first two secondary beams at each end of the span are joined (fastened) to the tie rods and temporarily stiffened by diagonal tie rods. The total weight of the main arch span as it now stands on dry land is about 1500 kN. Would it not be possible to lift and slide this structure in place, perhaps with the aid of a temporary support in the river at mid-span?

4. Pros and cons

It should be emphasized that no attempt has been made at optimizing the alternative design. The height and shape (circle versus parabola) of the arch should be looked into more closely, and it may be possible to improve the hanger pattern, although a deck requiring secondary beams leaves little room for manoeuvring. Even as it stands we believe the proposed network arch is a viable solution, and compared with the selected truss arch solution we think it has some advantages:

- Reduced amount of glulam. The network arch requires about 47 m³ of glulam which is about 55% of the glulam in the truss arch.
- Considerably reduced wind forces.
- The arch is well protected by a light 'roof', and creosote impregnation should not be necessary. For the truss arch, creosote is prescribed. Even so, the lower chord and its connections with the diagonals represent a durability problem (difficult to protect).
- The foundations can be made smaller (and cheaper) since they only need to support vertical forces.
- Easy assembly and simpler erection?

Clearly there are also disadvantages:

- More steel in hangers, 235 kN for the network arch vs 135 kN for the truss arch. Each tie rod requires from 70 to 80 kN of steel, and one more secondary beam adds about 35 kN.
- Some hangers may occasionally relax (become slack) which may be a problem. Tveit [1] is not overly concerned about this and he suggests means of alleviating the problem.
- Larger horizontal movements, also due to temperature expansion of the tie rods.

The aesthetic argument, which of course is a decisive one when it comes to selecting a bridge design, is open to discussion, and so we leave that aspect of the alternatives with the reader.

In closing it should be mentioned that letting the two network arches lean towards each other (so as to reduce the horizontal distance between them at the top to, say 5 m) will improve sideways stability noticeably (and simplify the wind truss).

5. Reference

- [1] Tveit, P., 'The Network arch. An extended manuscript from 21 lectures in 12 countries in year 2000, <http://pchome.grm.hia.no/~pert/>