Steien Network Arch Bridge

Johannes Veie
Senior Principle Engineer
Norwegian Public Roads Administration
Hamar, Norway
johannes.veie@vegvesen.no

M.sc. in structural engineering from NTH Trondheim 1996. Worked with steel and timber bridge planning since 2000 at the Norwegian Public Roads Administration.

Rune B Abrahamsen
Structural engineer Sweco Norge Lillehammer, Norway rune@sweco.no

M.sc. in structural engineering from NTH Trondheim 1995. Worked for Sweco since 1998. Group leader for bridges in Sweco.
Involved in the design of 60 timber bridges since 1996

Summary

The Steien bridge is situated in the county of Hedmark, Norway. A new network arch bridge will replace an old steel beam road bridge as well as a pedestrian bridge. The new bridge will have a single span length of 88 meters and arches that rise 15 meters. The bridge is located at Highway 3, which is one of the country's main roads. The bridge crosses Norway's largest river Glomma and will be a new landmark for the village Alvdal.

The new bridge will consist of laminated timber truss arches, network cable system and a reinforced prestressed concrete slab that also works as the tie for the arches. The network cable system allows for a significant reduction in bending moments in deck and arch, which leads to a structure of great slenderness and transparency. The new Eurocodes give the basis for the design, leading to comprehensive increase of load effects compared to earlier design codes. The structure will be assembled over the river, starting with casting of the reinforced prestressed slab and to be completed with the assembly of the timber arches and the wind truss. The construction on site is planned to start in the autumn of 2014 and finished within the year of 2015.

Keywords: Timber network arch bridge, concrete slab.

1. Introduction

Steien bridge is situated in Alvdal municipality. Alvdal is located in Hedmark County, 300 km north of the capital Oslo. The bridge is located on highway 3, which is one of the main roads connecting the east and mid-Norway. Daily traffic is about four thousand vehicles. The bridge, ref. figure 1, crosses Norway's largest river Glomma and will be a new landmark for the village Alvdal.

The new bridge replaces the old Steien road bridge built in 1953 as well as a parallel pedestrian bridge built in 1983. The bridge will be designed with traffic loads according to EN-1991-2 and with a 100-year design-life.

Fig. 1 The existing Steien bridge

The network arch bridge will be built with a record breaking span reaching 88 meters. The slab is made of reinforced prestressed concrete that also works as the tie for the arches. The slab carries the tension load which is in balance with the arch compression force. The vertical loads working on the slab are distributed to the arch through a total of 68 cables in the network cable system. One of the main benefits of using this type of bridge in this specific location is addressed to the slenderness of the deck structure. A normal beam bridge would have raised the road level considerably to overcome a 200 years flooding. The adjusted level would correspond to the height of the girder. The bridge carries two traffic lanes and two 3 meter wide pedestrian walkways.

2. Structural performance of the network arch bridge

A key feature of the network bridge is that it almost does not experience bending moments in the arch. This is the case as long as the majority of the hangers are in tension. The buckling strength in the arch is high, so is the stiffness regarding vertical deflection. The network system with inclined hangers allows for an efficient structural response which leads to very homogeneous hanger dimensioning. The cross section for the hangers is the same along the bridge. Figures 2 and 3 show the results from a comparison between the network arch concept for Steien bridge and a traditional 3-hinged arch bridge regarding moment distribution and vertical deflection due to asymmetric loading combined with self-weight of the structure.

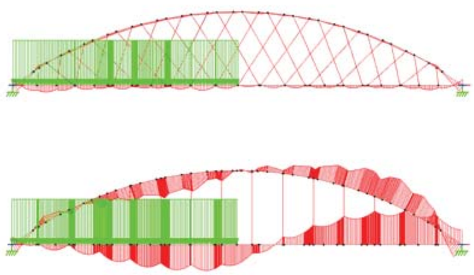

Fig. 2: Bending moment

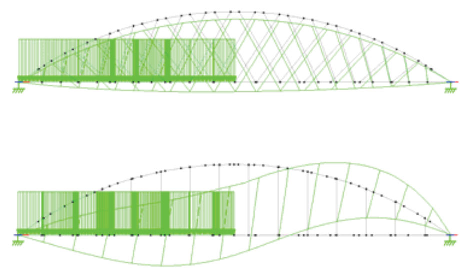

Fig. 3: Vertical deflection

For those who are interested in the general performance of network bridges, reference is made to the work done by dr. ing. Docent emeritus Per Tveit, found at http://home.uia.no/pert [1]. Reference is also made to work done by professor Kolbein Bell presented at IABSE conference in Lahti in Finland in 2001[2] addressing a design for a 70 meter span for a timber network arch bridge and another work presented at the ICTB2010 [3] conference in Lillehammer, Norway, which includes a proposal for a timber network arch bridge spanning 100 meter using a concrete slab.

After 50 years of development Tveit has come up with some suggestions to design the optimal network bridge:
The deck should be made out of concrete to give the necessary dead load to reduce the risk of relaxing hangers. The tie should be imbedded in the deck in the form of prestressing cables. This will decrease the tendency for the concrete to crack and thus increase the durability.
The arch should be made as part of a circle. This will give a more evenly distributed bending moment in the cords and will make the production easier.
The upper node of the hangers should be placed equidistantly along the arch and the hangers should not be merged in the nodal points. This gives a better support for the in plane buckling and reduces the bending moments.

3. Design for the new Steien bridge

Preliminary studies for a replacement of the existing Steien bridge showed that a network arch bridge is competitive with other relevant alternatives. Two other network arch bridges are built in Norway the last four years. Brandangersundet bridge with a main span of 220 meters and Aasnes bridge [4] with a main span of 111 meters. Before this, two network arch bridges were built in 1963 and 1964, both of them designed by dr. ing. Docent emeritus Per Tveit. All of them are made in steel and concrete. Brandangersundet bridge, the most slender arch bridge in the world, was of course a given example to compare against in terms of design. Brandangersundet bridge has the same type of slab working as a tie for the arch.
An experience from Brandangersundet bridge project was that although it was generally substantial savings in steel quantity, it was on the other hand, high installation costs. Although the bridge sites are not directly comparable, we chose a design philosophy that does not utilize the materials to their maximum in the operational phase, but rather provides some flexibility in the construction phase to reduce installation costs. Compared to arch bridges with vertical hangers the slenderness of the arches and tie is still extreme accounting for the use of a concrete slab instead of a much lighter stress laminated glulam deck.

Fig. 4: Steien bridge

Fig. 5: Steien bridge from the driver's view

Fig. 6: Steien bridge, 3D illustration

The new bridge showed in figures 4 to 6, will have a single span length of 88 meters and arches that rise 15 meters. The bridge carries two traffic lanes with a total width of 8 meters and two 3 meter wide pedestrian walkways. The total width of the slab is 19.8 meter.
The bridge is defined by arches made of rectangular glulam sections, slab made of prestressed concrete, wind bracings made of timber in combination with compression steel bars, and crossing hangers made of steel rods. The arches are inclined 7 degrees inwards.
The slab is made of in situ casted concrete with prestressing. The slab is casted in one stage. The hangers will be made of steel rods according to the ASDO tension rod system or likewise. The rods made of S550 steel are 48 millimeters in diameter and are delivered with adjustable fittings. The breaking load capacity is 795 kN.

3.1 Details

Figures 7, 8 and 9 show some typical details for the bridge

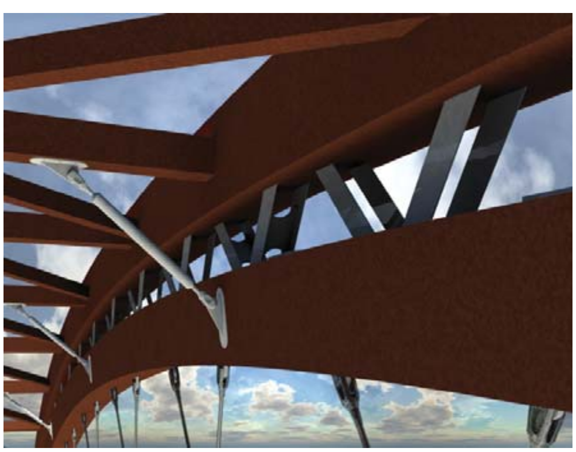

Fig. 7: Details of the truss arch

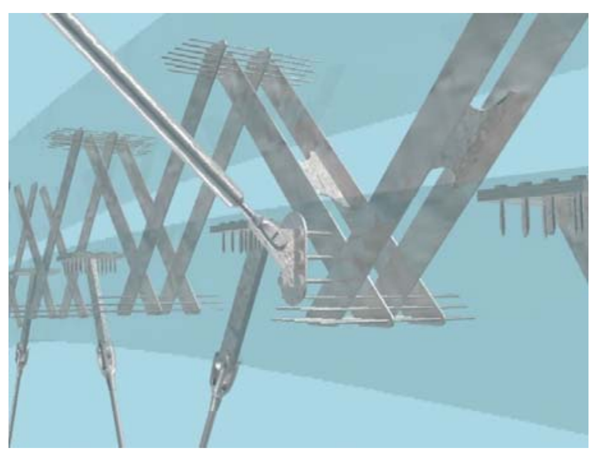

Fig. 8: Transparent view through the arch.

Fig. 9: Details of the hanger and slab connection

At the upper part the hangers are connected to steel plates that are going through the lower arch and are welded to a top steel plate that transfer the axial load from the hanger directly into the arch through compression perpendicular to the grain. In addition screws are used between the top plate and the arch in order to transfer shear loads between the steel and glulam. The diagonals in the truss arches are made of 10 mm thick stainless steel plates that are connected to the glulam with stainless D12 mm dowels. These steel plates have the same geometry throughout the arch structure.

At the lower part the hangers are anchored to the slab through a bolted connection as shown on figure 9.

4. Structural Analysis

The global analysis is performed with STAAD.pro. With the chosen cable layout it was found that there was an acceptable distribution of load distribution in the hangers without the need of individual stressing. In theory, if the overall geometry was in a perfect shape, the hangers could be produced with an initial unloaded length L0, which after installation did not have any need for adjustment. Since there are tolerances both for cable lengths and the arch assembly, there is a need to compensate for these deviations. For the ultimate limit state calculations the utilization of tension only elements in STAAD makes it possible to do a check of the impact of hangers going into compression. Some critical load combinations were selected for more comprehensive checking by non-linear analysis. For these calculations the initial geometry was given the shape of the first buckling modes, with the initial geometrical imperfection scaled to 0,0025L (but never less than 100 mm which is 3 times accumulated construction tolerances) in order to take residual stresses and geometrical imperfections into consideration. L is the length between the inflection points of the relevant buckling shape of the arch. In general for the code checking of the glulam sections a buckling length extracted from the first out of plane buckling mode was considered.

4.1 Traffic loading

The load combination cases that have been used are defined in NS-EN 1990.
Regarding global behavior due to traffic loading the only consideration has been taken to load model 1 according to NS-EN 1991-2, ref. fig. 10. Load model 1 comprises of two parts: one point-load part to simulate a heavy vehicle and one distributed part to simulate busy traffic.

Position	**Double axel load (BL)**	**Equal Distributed load**
	Axel load $\alpha_{Qi} \cdot Q_{ik}$ [kN]	**$\alpha_{qi} \cdot q_{ik}$ [kN/m^2]**
Loading area nr 1	1,0 · 300=300	0,6 · 9= 5,4
Loading area nr 2	1,0 · 200=200	1,0 · 2,5 = 2,5

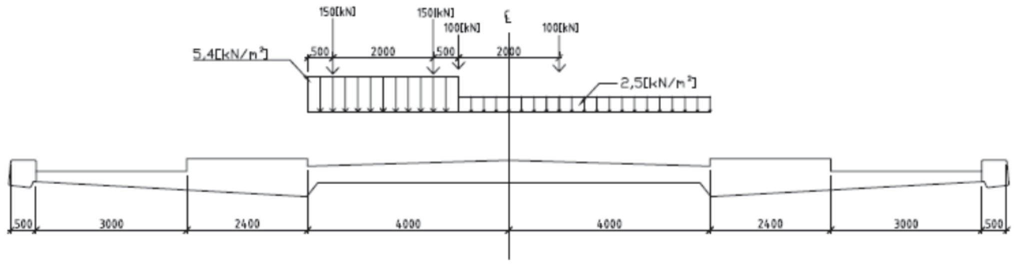

Fig. 10: Loadmodel 1 according t. NS-EN 1991-2:2003

Equally distributed load on the pedestrian carriageways are

$q=5{,}0\ kN/m^2$	without simultaneous traffic loading
$q=2{,}5\ kN/m^2$	with simultaneous traffic loading

4.2 Buckling analysis

The first buckling mode is shown on figure 11 and 12 for the case with self-weight. This mode gives basis for the non-linear analysis and also the basis for calculating the buckling factor, B_F. Three other buckling modes are also considered.

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

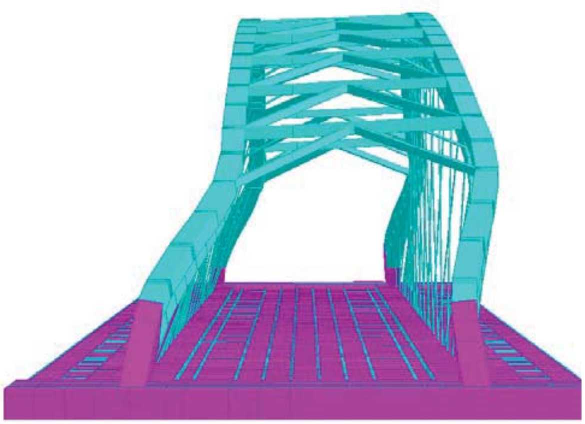

Fig. 11: Buckling mode 1

Introducing the classical formula;

$$P_{cr} = \pi^2 \frac{EI}{L_k^2}$$

for the Euler load we can estimate the buckling length to be:

Out of plane, $L_k = \sqrt{\frac{\pi^2 EI}{N_{max} B_F}} = 9.6$ meter

$E = 11100 N/mm2$
$I = 3.071E10\ mm4$
$B_F = 7.993$
$Nmax = 4576\ kN$

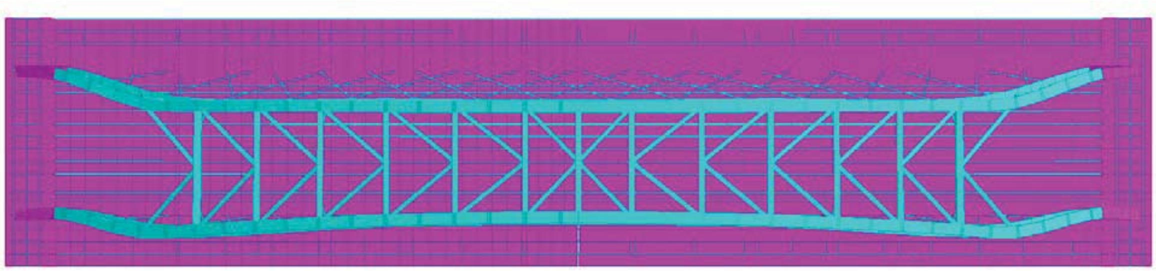

Fig.12: Buckling mode 1

Since the cross section for the glulam arches does not vary, the area around the wind portal will govern the design of the arches. This means that it is conservative to use L_k , derived from buckling mode 1, for all checking. This is also verified by the nonlinear analysis as described above.

4.3 Utilization

The wide slab generating high weight and combined with a reduced yield strength factor, k_{mod}, for the permanent load condition, the self-weight is governing for the design of the arches using normal concrete in the slab. In order to make the arches as slender as possible it is decided to use light weight aggregate in the concrete. This leads to that the utilization of the structure in the ULS condition with varying loads, corresponding to short time loading class and higher yield strength factor, is in the same magnitude as self-weight alone and permanent loading class.

5. Materials

The bridge consists of approximately 850 m^3 of concrete and 200 m^3 timber.
The cross section of one arch is designed with two massive 850 mm wide and 600 mm tall rectangular profiles forming a truss arch through steel diagonals.
The cross section of the slab is designed with a prestressed concrete slab with thickness varying from 800 to 250 mm.
The diagonals are designed as stainless steel profiles 200 x 10 mm connected to the timber with stainless steel dowels.
The cross sections of the wind truss are composed of square 400x400 and 300x300 glulam profiles and steel bars.
The inclined hangers are designed with D48 steel rods, S550 quality.

6. Durability

The Norwegian bridge design standards, issued by the Norwegian Public Roads Administration, require a service life of 100 years for all permanent bridges. Based on 20 years' experience on modern timber bridges we know were the possible weak points are. In general these are found in connection to stress laminated timber decks. For a typical timber bridge weight is critical, so stress laminated decks are normally preferred. In the case with network arches, the opposite is normally the issue. Because of this, it was natural to use concrete.
The protection of the arches is based on a combination of chemical treatment, metal cladding of exposed parts and good detailing. The wood treatment of the glulam members are carried out in a two-step process. First the individual lamellas are given a Cu treatment to a degree that corresponds to what is required for members in soil contact. The second step, a creosote treatment, is carried out after the glulam member is completely finished, including drilled holes and sawn slots. This allows for an optimum creosote penetration into the wood.

One millimeter thick copper sheeting will be used as metal cladding. Copper cladding of the top surface of the upper chord is quite easy, while covering the lower chord of the truss arch is not straightforward. The object is to cover horizontal surfaces as well as preventing water from running down the diagonals and into the slotted joints. In addition to wood treatment the slots will be sealed locally around the steel plates with a bitumen or epoxy layer. Horizontal plates will be connected to the lower part covering the slots for additional protection.

7. Fabrication and assembly

The brigde is planned to be assembled on site. A temporary bridge will handle the traffic after the demolition of the existing bridge. The abutments and slab are completed before the arch is mounted. The arch is planned delivered in 4 pre-assembled parts. The joints connecting the parts will not transfer bending moment. After the final assembly of the bridge the framework used for casting the deck is lowered and removed. Only a minor need for adjustments of the hangers should be necessary.
A significant challenge is how to ensure that the hangers are actually getting the defined tensile load and thus achieves the intended geometrical shape of the bridge. The behavior of the system is quite non-linear in terms of the effect of tensioning or de-tensioning of the hangers. Due to the system's non-linear behavior, a method involving tensioning of the cables in several stages would cause quite time-consuming analysis and complex tensioning procedures. A preferred solution is to produce the hangers in their final theoretical lengths in unstressed condition, and install them by jacking the slab up to a level where they could be installed in slack condition and then lower the slab again. Based on measurements to detect geometrical imperfections, the hangers are corrected before lowering the slab. The hangers are also checked after lowering to adjust any hangers if they are out of tolerances.

The tender for the construction works is due to be issued in the spring of 2014. The construction on site is planned to start in the autumn of 2014 and finished within the year of 2015.

8. References

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