Case Study of the Longest Single Span Timber Bridge for Highway Loads in Sweden

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Summary

A 47.4 m long glulam arch bridge was built over the Nissan River when Highway 27 was upgraded in Sweden. The bridge consisted of two individual arches spaced 13.9 m apart which were unconnected at the top due to architectural requirements. The lateral stability of the bridge was obtained through stiff vertical hangers which were bolted to a stiff transverse girder underneath the road level. Extensive analyses were made to ensure the stability and capacity of the arches against accidental, wind and vehicle loads. The deck was constructed as a continuous stress-laminated-timber deck using glulam. A total of approximately 230 000 kg of untreated Swedish glulam was used in combination with approximately 100 000 kg of steel. The bridge could, to a large extent, be pre-assembled in the factory before being transported to the construction site. The entire assembly of the bridge took approximately two months.

Keywords: Glulam, arch bridge, stress-laminated-timber deck, FE analyses, wind analyses, buckling

1. Introduction

A new stretch of Highway 27 has been constructed to primarily reduce the amount of traffic through the town of Gislaved,Sweden, but also to increase the safety of the highway. The largest bridge on this particular new stretch of highway is the one crossing of Nissan River. The new highway was constructed with a width of either 9 or 14 m. The 9 m highway width had two lanes (one in each direction) while the 14 m wide one had three lanes, which is a common solution in Sweden including road shifts between two lanes in one direction and one in the opposite direction.

The Swedish Transport Administration (Trafikverket) decided early on that they would like a timber-arch bridge crossing the river. The required clear span of the bridge was 45 m. There were several architectural requirements related to the appearance of the arches that had to be fulfilled. Firstly, a rather small radius was required for the arches and they also needed to resemble an old steel-arch bridge in the area. A second requirement was that the two arches were not allowed to be connected above the road level. In essence, the architects aspired for the drivers crossing the bridge to have an unobstructed view unlike that provided by a tunnel.. The combination of a small radius for the arches and the lack of cross bracing in the top posted a challenge for the design of bridge. A stress-laminated-timber (SLT) deck made out of glulam beams was used for the bridge deck. Drawings of the bridge are shown in Figure 1.

The contract for the construction of the highway stretch went to Svevia (a Swedish contactor) which ordered the timber bridge from the manufacturer Moelven Töreboda (Moelven). Moelven contracted the consulting company WSP Civils Sweden (WSP) for the analysis and design of the bridge. The bridge was assembled during May and June of 2013.

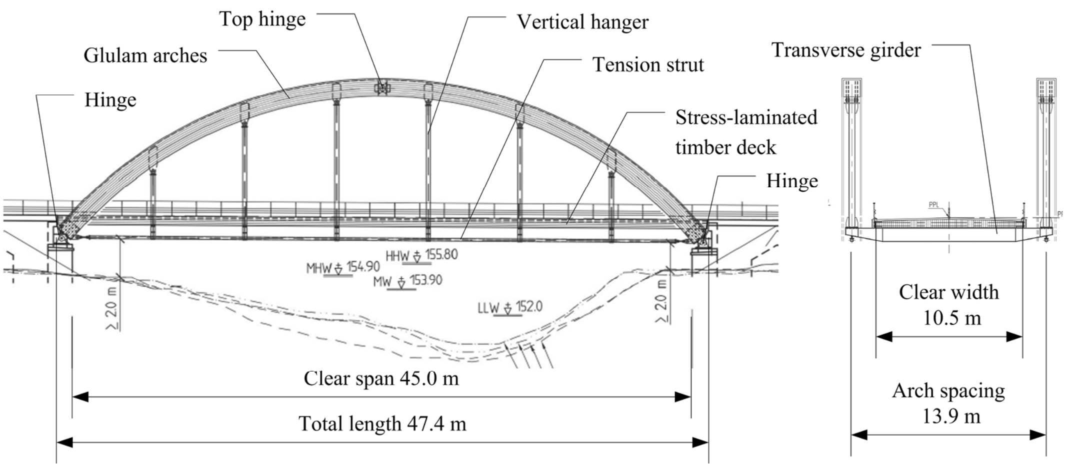

Fig. 1 Elevation and section drawing of the bridge.

2. Structural analysis of the bridge

The structural analysis was performed using the finite element (FE) software Brigade/Plus 3.1-5 which is based on the solver from Abaqus. The structural analysis made by WPS was limited to the superstructure. The obtained reaction forces from the analyses were used for the design of the substructure.

The density of the SLT deck is larger than for the glulam arches in order to include the weight of the prestressing system and bridge railing. The stiffness values for glulam of grade CE L40c (Sweden glulam) was taken from the Swedish glulam manufacturers [1]. The orthotropic plate material was calculated using the defined ratios in EN 1995-2 [2].

Table. 1 Material property values used for the wood parts in the analysis

Material property	**Glulam arch**	**SLT deck**
E_1 [MPa]	13 000 [1]	13 000 [1]
E_2 [MPa]	410 [1]	0.02 E_1 = 260 [2]
E_3 [MPa]	410 [1]	0.02 E_1 = 260 [2]
G_{12} [MPa]	760 [1]	0.06 E_1 = 780 [2]
G_{13} [MPa]	760 [1]	0.06 E_1 = 780 [2]
G_{23} [MPa]	760 [1]	0.1 G_{12} = 78 [2]
ρ [kN/m^3]	4.2 [1]	4.5 [1)]

1) Combined weight of glulam, prestressing and bridge rail

2.1 Loads

The bridge was loaded by the self-weight of all steel and glulam components as described above in Table 1. Another dead load included was the surfacing of the deck, which was composed of a 110 mm thick asphalt with a density of 23 kN/m^3 [3].

Vehicle loads from Eurocode EN 1991-2 [4] were applied on the bridge using load combinations from EN 1990 [5]. In addition to the vehicle loads in Eurocode, fourteen classification vehicles should be used according to Trafikverket. These classification vehicles are often governing the design of bridges in Sweden. As well, both vertical and horizontal effects from the vehicle loads needed to be considered in design.

The geographical position of the bridge together with the geometry of the arches and hangers resulted in the need for extensive wind analyses. The wind loads on the bridge should conform with the requirements in TK Bro [3] and EN 1991-1-5 [6]. A quasi-static wind load which could be applied in the static analysis was calculated from the dynamic wind analyses. Special considerations had to be made to the stiffness of the vertical hangers in order to avoid severe fatigue problems from vortex shedding.

Temperature loads were applied to the bridge in the form of a uniform temperature load of +25ºC and a temperature difference of -43 ºC according to EN 1991-1-5 [7]. Loads from temperature gradients were also applied on the bridge.

An accidental load was applied with a force magnitude of 1000 kN in the direction of the bridge (direction 1) and 500 kN perpendicular to the direction of the bridge (direction 2). The load should be applied on the arches at a distance of 1.25 m above the top of the surfacing. The magnitude and the position of the load was taken from EN 1991-2 [4].

2.2 Arch

The two arches (spaced 13.9 m apart) had a total length of 46.6 m between the points of rotation in the bearings. The radius of the arches centerline was 29.9 m. The arches were modeled with hinges at the top and over the support. The tension strut ties the two halves of the arch together, cf. Figure 2.

The arches were modeled using beam elements with anisotropic material properties, cf. Table 1.

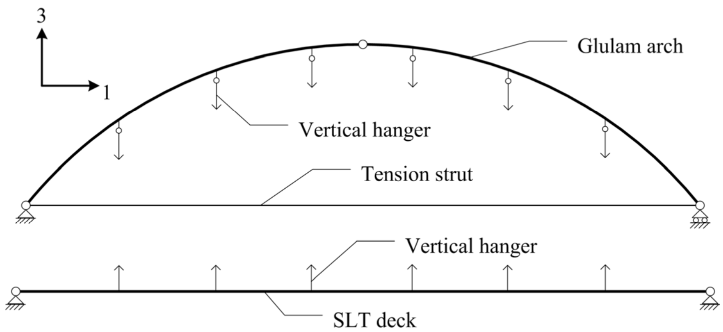

Fig. 2 Principe sketch of the structural behavior of the arches.

Direction 1 was designated for the arch defined as the tangent to the centerline of the arch. A simplified buckling analysis was initially made on the arches. The critical buckling load and mode shape showed that the hangers and especially the stiffness of the SLT deck needed to be included in the analysis. A second buckling analysis was made on the entire structure. The critical load for the structure was obtained through an analysis where a load was applied in one of the hangers. The critical load analysis was repeated for all of the hangers. Eight different combinations of loading the hangers were evaluated in order to find the critical buckling load for the entire structure.

2.3 Hangers and transverse girder

The vertical hangers and transverse girders were modeled using beam elements with isotropic material properties. The stiffness of the steel in the model was 210 000 MPa and the density was 78.5 kN/m^3 , taken from EN 1991-1-1 [8]. In the design, the hangers and the transverse girder are supposed to stabilize the arch for horizontal loads. The connection between these components was modeled as a stiff frame corner, cf. Figure 3. The connection between the hanger and the arch was modeled as pinned around axis 2, cf. Figure 2. The girders needed to be very stiff due to the stiff connection between the girder and the hanger. A small deformation of a weak girder would result in a large transversal displacement on the arch. Hence, the girders had to be very stiff, cf. Figure 3.

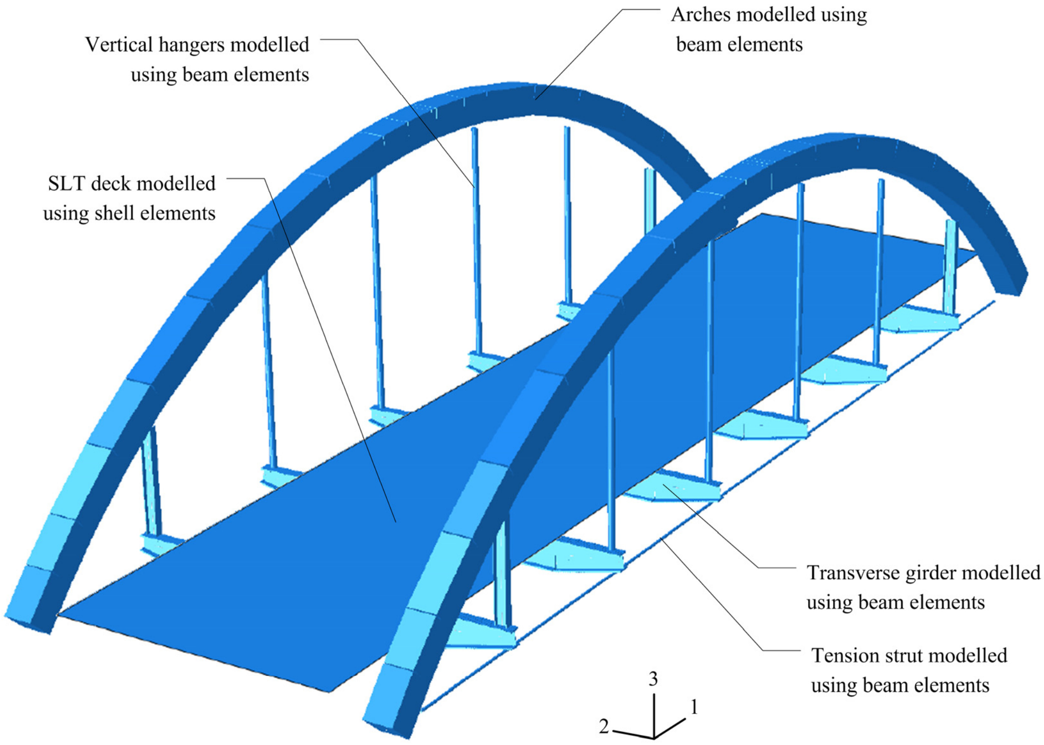

Fig. 3 Numerical mode from Bridge-Plus. The parts of the bridge were modeled using beam or shell elements. All beam elements in the figure are rendered to show their correct profile. The coordinate system used is also shown.

2.4 Deck

The SLT deck was modeled as a continuous slab with six intermediate supports (the transverse girders) and two end supports, cf. Figure 2. The total length of the deck was 47.4 m and the width was 10.5 m, cf. Figure 1 and Figure 3. The thickness of the deck varied between 450 mm and 580 mm with the deepest section in the middle and 450 mm along the longitudinal edges. The SLT deck was modeled using rectangular shell elements with orthotropic material properties, cf. Table 1. Connections between the deck and the girder were modeled with restrained movement in directions 1 and 3 in order to measure the uplifting force. The middle point of the girder restrained the movement of the deck in direction 2, thus simulating the slotted-in plate restraining translation the deck in the transverse direction.

2.5 Other components

The tension strut was modeled using beam elements and the same material as for the hangers and the transverse beam. A small weightless and infinite stiff element was used to couple the arch and the tension strut at the correct position, cf. Figure 3.

3. Design of components

The design of the arches, hangers, girders and the deck was an iterative process in combination with the structural analyses. All of the major components in the bridge had to be analyzed and designed simultaneously in order to achieve better material utilization as well as to avoid problems with wind induced fatigue.

All detailing of connections and other components could be made after the main structure was designed. Since the glulam is untreated, the design of the weather protection for the glulam components is crucial. The design of the bridge was made using the 3d-software Tekla Structures. Both 3d and 2d drawings were generated from the 3d model.

3.1 Arch

Each of the glulam arches were constructed using five parallel arched glulam beams with a cross section of 1170 x 215 mm (depth, width). The glulam beams were glued and screwed together forming a 1075 mm wide block. The cross-section of the arch was deigned to fulfill the criteria for compression, bending, combined compression and bending, shear, torsion and tension perpendicular to the grain. The shear capacity of the arch governed the depth of the cross-section, which was due to the crack factor $k_{cr} = 0.67$ for glulam in EN 1995-1-1 [9]. The magnitude of the crack factor has been widely debated in Sweden for a couple of years now, however, recent studies by SP Technical Research Institute of Sweden have shown that a smaller reduction of the capacity should be used [10].

A steel detail was created to connect the glulam arch to the bearing using doweled slotted-in steel plates, cf. Figure 4. The arch end detail also allowed for the tension strut to connect the two arch halves. The outer measurements of the steel detail are approximately 1.3 x 1.2 x 1.2 m (length, width and height).

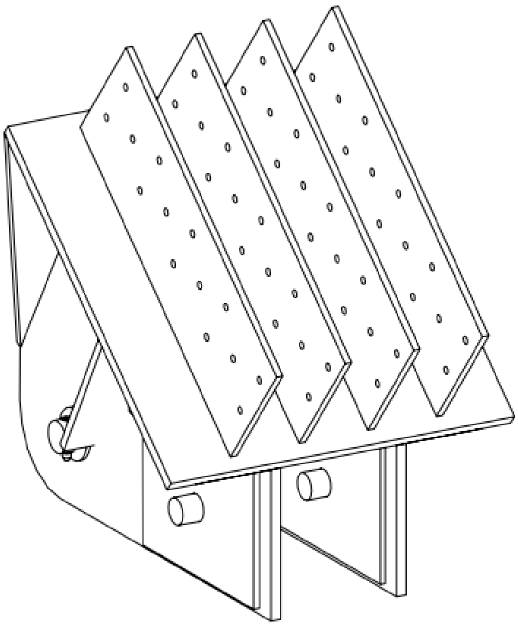

Fig. 4 Detail of the connection between the glulam arch and the bearing. The connection for the tension strut is also seen in the figure.

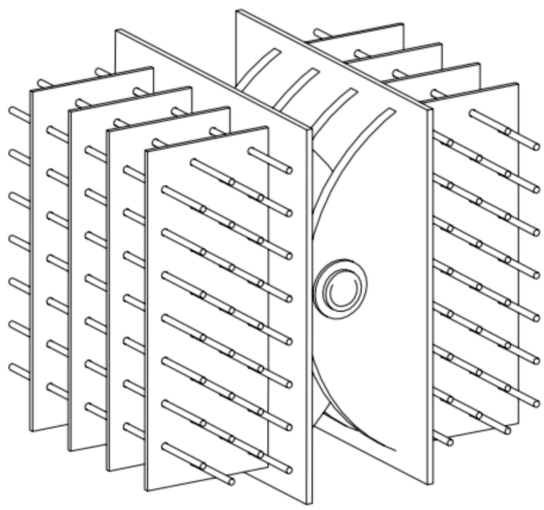

Fig. 5 Detail showing the hinge used for connection the two halves of the arch together. The steel dowels are shown in the figure.

Another steel detail related to the arch included the pinned connection between the two halves of the arch. Again, doweled slotted-in plates were used in this detail, cf. Figure 5.

The tension perpendicular to grain was larger than the capacity for glulam CE GL40c, as suchthe glulam beams were reinforced using fully-threaded screws with the dimensions 13 x 800 mm (diameter, length).

3.2 Deck

The deck was designed as a continuous deck using staggered butt joints. One in five adjacent beams was butt joined. The length of the glulam beams used for the deck varied between 12.3 m and 19.5 m. The width of the deck beams was 140 mm and the depth varied between 450 mm and 580 mm. The deck was designed to fulfill the criteria in ULS for bending, shear, interlaminar shear, compression perpendicular to grain, and compression parallel to grain for the break plates. The bending stresses in the deck governed the design of the deck depth. The smallest depth was 450 mm at the longitudinal edges of the deck. A fatigue verification in accordance to Appendix A in EN 1995-2 [2] had to be executed.

The deflection of the deck itself as well as the entire bridge had to be checked in SLS. The deflection of the deck in between the transverse girders was small. However, the global deflection of the deck was 50.5 mm, which corresponds to a deflection of $L/940$, where L is the span between the two end supports. The deflection criterion for bridges in Sweden which needs to be fulfilled is a deflection less than $L/400$.

Two rows of prestressing bars were used with a longitudinal spacing of 600 mm. Dywidag bars with a 20 mm diameter were used. The longitudinal edge of the deck including the glulam arch or a steel hanger had to be stressed using coupled 1 m long bar sections. The coupling of shorter bar segments was used in order to maintain the possible interchangeability of any part of the deck.

The deck was bolted to the transverse girder to prevent separation between the deck and the girder. Four M24 bolts were equally distributed over the width of the deck.

3.3 Hangers and transverse girders

The hangers were made out of HEB450 steel profiles. Eight out of twelve hangers had additional stiffeners welded onto the flanges to increase the stiffness.

The transverse girders were made out a welded steel I-profile. The 9.5 m profile in the middle of the girder had a depth of 1200 mm while the outer 2.66 m profiles on each side of the girder had a varying depth of 1200 – 600 mm. The width of the girder flanges was 440 mm. As previously mentioned, the transverse girders had to be very stiff in order to avoid generating larger transverse displacements on the arches.

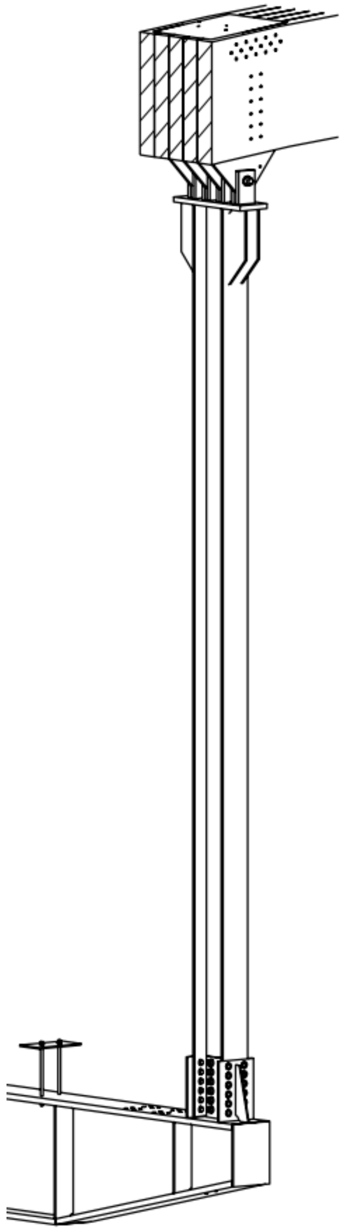

Fig. 6 Detail showing the hanger with connection to the arch and the transverse girder. The increasing depth of the girder is shown. The bolts preventing separation between the deck and the girder are also shown.

The connection between the hanger and the transverse girder was a bolted connection such that the hanger or girder could potentially be replaced. The bolted connection consisted of 36 M36 bolts, cf. Figure 6. The fatigue verification governed the design of the bolted connection.

The connection between the vertical hanger and the glulam arch was a pinned connection around axis 2, but could transfer moment around axis 1 and 3. The connection consisted of four doweled slotted-in plates that were welded to a steel plate, cf. Figure 6. The purpose of the added platewas to reduce the amount of tension perpendicular to grain in the arch. A M48 bolt was used for transferring the forces from the arch to the hanger, cf. Figure 6.

The amount of steel used for the hangers and the transverse girders would have been significantly reduced if cross-bracings were allowed between the arches. However, the aesthetics of the bridge would be significantly changed.

3.4 Other components

The tension strut consisted of several joined M100 steel bars that were 6.3 m, 7.6 m or 10.2 m long. It was more economical to use joined bars instead of a high strength steel cable.

The railing was connected to the deck with a capacity equal to a load that is twice the size of the load which would cause the rail posts to plasticise. The two M20 bolts that will primarily be loaded in tension had to be anchored within the deck. A 450 x 150 x 15 mm (width, height, thickness) steel plate was used to anchor the force at a distance equal to 4 x 140 mm = 960 mm from the edge of the deck.

4. Durability

All wood parts of the bridge were constructed using untreated glulam made out of Norway spruce. The wood had to be protected from moisture in order to ensure the required durability. The deck was covered with a 5 mm sealing board to prevent moisture from penetrating the deck. A 110 mm asphalt layer with 2.5% slope covered the top of the deck. Eight inlets were used on the bridge together with a rail gutter on the side of the deck, cf. Figure 7. The gutter prevents some of the melted water from entering the river Nissan underneath the bridge.

A piece of bent sheet metal that runs along the entire length of the deck on both sides transports the water away from the deck, cf. Figure 8. A ventilated side panel is used to prevent the edge beams of the deck and the prestressing anchorage from rain and sun. The panel consists of painted 120 x 21 mm z-panel made out of wood. The panel is fastened to the deck through wood block spacers.

Fig. 7 Stainless steel inlet (before sealing board and asphalt are added)

Fig. 8 Bent sheet metal to divert water (before sealing board and asphalt are added)

Fig. 9 Ventilated wood panel and steel sheeting to protect the glulam arches.

The glulam arches were also covered with a ventilated wood panel. The top of the arches was covered with large painted steel sheet metal, cf. Figure 9.

The prestressing bars and nuts were made out of galvanized steel.

5. Bridge assembly

The bridge was fabricated in Moelven's facilities in Töreboda, Sweden. About six weeks was required for the manufacturing of the arches and the SLT deck. All steel connections were inserted into the glulam arches at the plant, cf. Figure 10. Most cuts in the bridge (i.e. slotted-in plates) were made by skilled labor. In total, approximately 230 000 kg of glulam and 100 000 kg of steel were used for the bridge.

The bridge was transported 180 km south from Töreboda to Gislaved in a meticulous order. One crane on each side of the bridge lifted the two halves of the arch in place and the pins could then be inserted, cf. Figure 11. Steel cables were used to stabilize the arches until a temporary HEA300 steel beam was mounted, cf. Figure 12. Pontoons were temporarily used for easy access to both sides of the construction site

The vertical hangers and transverse girders were assembled next to the construction site. The six frames were lifted into position using cranes, cf. Figure 13. The deck could be assembled as soon as the arches were stabilized by the hangers and girders.

Fig. 10 Insertion of dowels in the arch

Fig. 11 First half of the arch lifted into position using a crane.

Fig. 12 Both arches in place with temporary support beam. The pontoons are visible to the left in the figure.

The deck beams were transported to the construction site as pre-assembled blocks containing five glulam beams. The deck was constructed from the middle towards the sides. When all glulam beams were in place, a prestressing force of 1000 kPa was applied to the deck. An overview of the deck in place is shown in Figure 14.

The guard rail posts were mounted when the deck was stressed. The moisture barriers could be assembled after the guard rail was mounted. Some of the moisture barrier details are shown in Figure 7 and 8. The entire deck was covered with a sealing board. The bridge will be surfaced at the same time as the highway, cf. Figure 15.

Fig. 13 The second frame is lifted into position.

Fig. 14 SLT deck in position.

Fig. 15 View of the completed bridge without the asphalt surfacing.

6. References

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