Systematic Thesis on Network Arches   *2014*   *By Per Tveit, dr. ing. UiA, Norway*

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The author prefers network arches with concrete ties and H profiles in the arch. A network arch of this type normally saves around ⅔ of the steel weight needed for other steel bridges. If there are steel beams in the tie, around 1/3 of the steel weight is likely to be saved. Network arches look nice. Network arches have been designed in many different ways. See chapter H: Network arches built or planned.

My main publication: "The Network Arch" can be found on my homepage: http://home.uia.no/pert/ under the button "The Network Arch". It was started in 2000 and is the result of an organic growth. Thus readers will often have a hard time if they are seeking information on specific items. The present publication should be more readable and should lead interested readers more directly to what they are looking for. This publication can only be found on the internet. It will be updated at least till 2014. There are many references in the text. They are supposed to lead to more information on the same subject. **Thus the system of references at the bottom of the page is important.**

Contents

Chapter A is a general chapter. The network arch is an arch bridge where some hangers cross each other at least twice. The hangers give the arches efficient support and high buckling strength. The chords have little bending. Tie and hangers have tension. Thus the network arch makes good use of high strength steels.

Chapter AA. Materials needed for two lane network arch road bridges spanning between 60 and 200 m.

Chapter AAA. Materials needed for two track network arch railway bridges spanning between 60 and 150 m.

Chapter B is on arches. In bridges of small or moderate length arches, universal columns and American wide flange beams make very economical arches. They can come from the steel works bent to the right curvature. If arches slope towards each other, it makes the transversal beams in the tie longer. Thus the steel weight goes up.

Chapter C is on lower chords. The tensile force between the two ends of the arches is best taken by prestressing cables. These cables are placed in small edge beams and give partial prestress in the tie. For everyday loads there will be full longitudinal prestress. If the distance between the arches is more than 20 m, transversal steel beams are likely to be needed. The longitudinal bending moment is normally smaller than the bending moment in the middle of the slab between the two arches.

Chapter D is on hangers. The inclined hangers make the network arch work much like trusses as long as only a few hangers relax. Suitably small angles between arch and hangers give the hangers suitably high resistance to relaxation and to tensile force and fatigue.

Chapter E is on erection. Economy of network arches is very dependent on efficient methods of erection. A temporary lower chord combined with arches and hangers can carry the casting of a concrete tie. The arches can be steel tubes that are filled with concrete after the steel skeleton has been lifted to the pillars. In long bridges over navigable water finished high strength concrete spans can be lifted in place by big floating cranes. In arctic regions the steel skeleton can be erected on ice and be lifted to the pillars before the ice breaks up in the spring.

Chapter F is on cost. Here network arches are compared to other bridge types. Comments on various methods of erection are given.

Chapter G is a short chapter on strengthening of network arches already built.

Chapter H gives examples of network arches built or being built.

Chapter I contains references.

Chapter J is a history of network arches as experienced by Per Tveit.

On references:

References to pages in this publication are in the form A-1 and B-1. References to drawings in this publication are in the form fig. C1 and fig. I2. References to the list of literature in this publication are like (Leonhardt 1991) and Herzog (1975 p. 281). References to "The Network Arch" in my homepage are for (Tveit 2013). It will gradually be updated as long as the author feel like doing so, but the reference will remain the same. This also applies to the present publication.

Contents

AA. Amount of materials needed for two-lane road bridges spanning between 60 and 200 m

In bridge design in Aalborg University Center in the nineteen eighties the students were taught to make at least three sketchy designs before choosing the final alternative. This can be time consuming. The diagram in fig. AA2 has been made to save time for designers that consider network arches an alternative for a road bridge. It gives a rough estimate of the materials needed for double track road bridges spanning between 60 and 200 m.

The network arches might be especially advantageous for these spans. In longer spans erection is more difficult. It is stresses due to traffic loads that are reduced by the crossing hangers. For longer spans the deadweight becomes more important and the crossing hangers do not necessarily reduce the stresses due to deadweight.

The diagram of steel weights has been made starting out with the materials needed for the bridge in fig. C1. The cross-section of the bridge is shown in fig. AA3. It is from the master's thesis of (Teich and Wendelin 2001). Their calculations can be found at http://home.uia.no/pert/ under the button "Masters Theses".
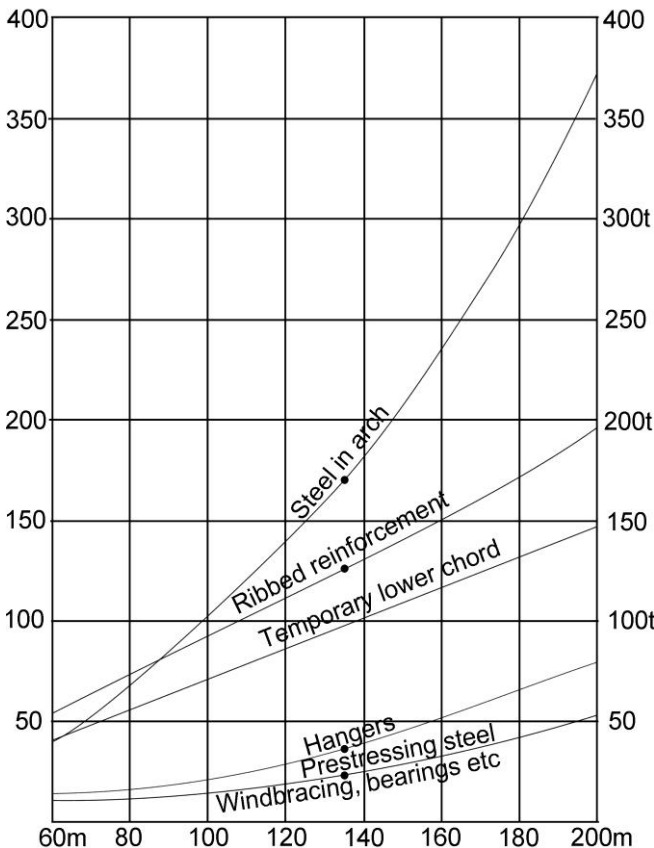

Fig. AA2. Steel needed for various two-lane spans

Teich and Wendelin used a wind load v_{ref} = 32 m/s. The other loads were the standard EU-loads used in Germany in 2001. The dot on curves indicates amounts of steel taken directly from Teich and Wendelin's thesis. The diagram is more reliable near these dots.

The variation in weight due to increasing spans has been arrived at by rough estimates. Thus the result is a preliminary estimate of materials needed, but good enough to decide if a network arch would be a relevant alternative for a two lane road bridge. It could also be helpful for educated guesses of the materials needed for other network arches.

Teich and Wendelin assumed concrete with cube strength 50 MPa. To increase the durability of the tie, higher strength concrete could be used. This would make lighter bridges and would lead to more ribbed reinforcement and less steel of other qualities. Higher concrete strengths could also be used for making wider bridges, but that could lead to more problems with controlling the deflection of the slab between the arches. See also p. C-7.

In Norway many bridges have only one footpath. This is to facilitate the removal of snow. Then the room for pedestrians and bicycles must be extra wide. Thus little steel is saved by having only one footpath. The amount of structural steel needed can be reduced by increasing the rise of the arch. Teich and Wendelin used a rise of the arch that is 15% of the span. This looks good. In the US a rise around 20% of the span is sometimes used because it saves a lot of steel.
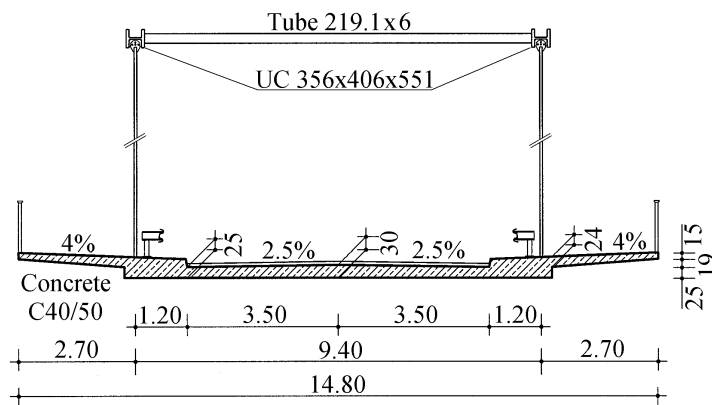

Fig. AA3. Cross-section of the two lane spans in fig. AA2

For spans near 200 m buckling of the arch might lead to very big cross-sections made from American wide flange beams. If that is the case, box shaped arches might be a better alternative.

The weight of the temporary lower chord in fig. AA2 is bigger than necessary for the bridge in fig. AA3. That is because the author thinks that temporary lower chords should be designed to be used for various network arches.

The author would very much like to receive feedback from designers who suggests alterations to the diagram in Fig. AA2.

AAA. Amount of materials needed for TWO-TRACK RAILWAY bridges spanning between 60 and 150m
Frank Schanack and Benjamin Brunn.

Undoubtedly, network arches are adequate for railway bridges. As one of the first network arch bridges, the German Fehmarn Sound Bridge from 1963 with a span of 248 m carries not only two lanes of road traffic, but also one railway track. Especially during the last couple of years, the number of new network arch railway bridges has increased remarkably, for example the new German railway bridges over Mittellandkanal (2009, one track, 132.6 m, 1,000 t of steel), over the Oder River (2008, two tracks, 104 m, 1,100 t of steel) and over the valley Rosenbachtal (2008, one track, 89 m, 670 t of steel, Fig..AAA1) (Graße, Tveit 2007). From a financial and an aesthetic point of view, Geißler, Steimann and Graße (2008) recommends network arch railway bridges for spans between 80 m and 300 m.

All of the above mentioned bridges have a steel deck with longitudinal tie girders. Based on the three most recently built bridges among them, the amount of necessary structural steel is about 7.5 t/m for single track bridges and about 10.5 t/m for double track bridges and spans of around 100 m.

In network arch bridges axial forces are predominant, favouring the use of high strength materials. Consequently, the longitudinal tie girders should be replaced by prestressing cables, which have much higher yield strength. Furthermore, single and double track railway bridges have distances between the arches of about 6 m and 10 m between the arches, respectively. Such relatively short distances lie within the application range of economical concrete slabs. Hence, railway network arch bridges go well together with a longitudinally prestressed concrete deck.

Fig. AAA1. View of railway network arch bridge over Rosenbachtal, Germany (courtesy of Stahl- und Brückenbau Niesky GmbH)

A double track network arch railway bridge with a concrete tie was the subject of (Brunn & Schanack 2003). It spans 100 m and was assessed for Eurocode load model 71 (α=1.0). A front view of the bridge is shown in Fig. AAA2. This bridge needs the following amounts of structural steel: 37 t for the steel bar hangers, 188 t for the arches and their connections and 15 t for the wind bracing. Furthermore, 71 t of prestressing cables and 65 t of reinforcement steel are necessary for the deck.

In this material-optimized design the bridge needs a total amount of about 376 t of structural steel, reinforcement and prestressing cables. This value corresponds to 3.8 t/m for a double track bridge of 100 m span. In practice, the structure would probably be built with bigger safety margins, so that a value of 4.5 t/m is more realistic.

In longer network arches the transverse reinforcement of the deck and the structural steel of the wind bracing and hangers ≈20% of the total steel weight.

The steel for the arches and the longitudinal prestressing cables ≈80% of total steel weight.

In order to get a rough estimate of the amount of steel needed for a two-track network arch railway bridge with a length L you can use this unreliable formula:
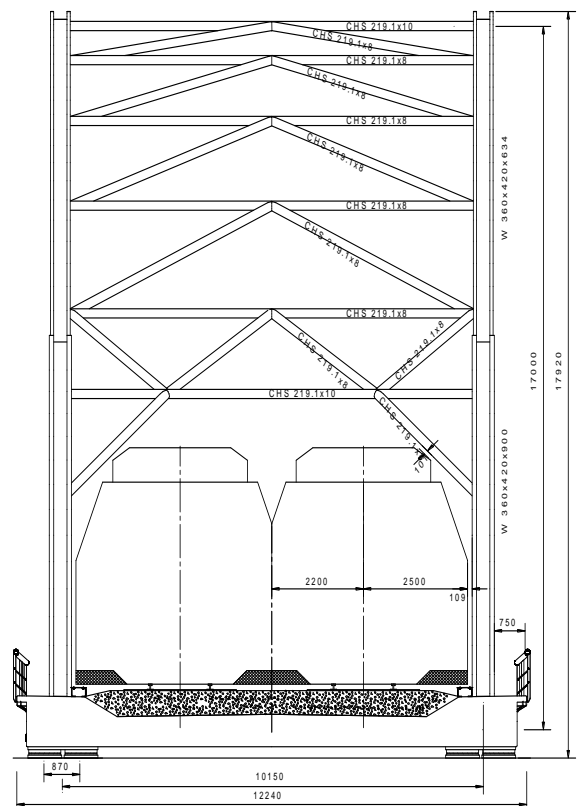
Weight=375x(L/100)^2

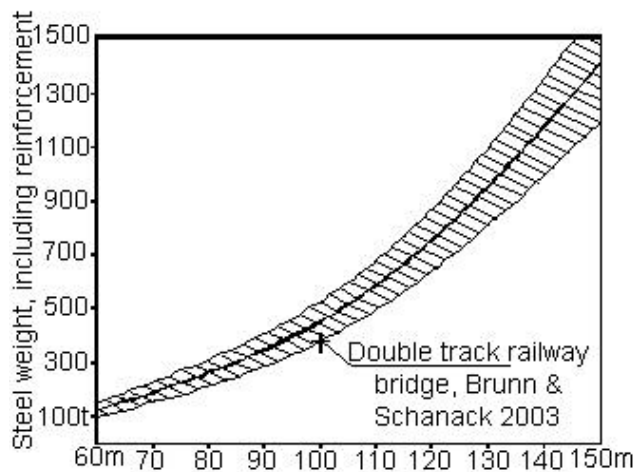Fig. AAA2. Front view of the railway bridge designed in (Brunn & Schanack 2003)

Fig. AAA3. Steel weight in two-track railway arches with concrete deck (UIC 71, α= 1.0)

As the network arch is a simply supported beam, maximum design forces for uniform (maximum) load increase by roughly the square of the span increment. Consequently, a 60 m long two track railway network arch bridge with a concrete deck will need about 4.5 t/m · ((0.6)^2 · 80% + 0.6 · 20%) · 60 m ≈ 110 t of total steel. A 150 m long two track railway network arch bridge with concrete deck will need about 4.5 t/m · ((1.5)^2 · 80% + 1.5 · 20%) · 150 m ≈ 1418 t of total steel (Fig. AAA3).

These values have to be adapted to the load classification factor α of specific projects. This factor applies to live load only, which was about 50% of the total load in ultimate limit state of the bridge mentioned above (Brunn & Schanack 2003).

Railway traffic is characterised by very high axle loads. In network arches, a high live load to dead load ratio increases the hanger's tendency to relaxation and special care is required when designing the hanger arrangement. For the hanger net design good results have been achieved with the use of the radial hanger arrangement (Schanack & Brunn 2009a). See also D-4.

The network arch is a very stiff structure that shows very small static deflections under traffic load. See Fig. 7 on page H-24. This is important in order to assure the safety of railway traffic. Dynamic analyses of railway network arch bridges with a concrete deck have shown that the vertical acceleration limits are not exceeded for railway traffic speeds of up to 200 km/h (Schanack 2008). However, as for all bridge types, high speed railway traffic requires a stiffening of the deck in order to reduce acceleration and resonance vibration. Economically, additional steel girders are more effective than concrete members, therefore high speed railway network arch bridges need more steel than the values given above.

The Deutsche Bahn (DB – German railway authority) advisory board for bridge design, founded in 2007 and promoting design and construction innovations as well as aesthetic design quality of bridges, has published guidelines for the design of railway bridges (Schlaich et al. 2008). In these guidelines the network arch is proposed as an innovative alternative to the classic tied arch bridges (Fig. AAA4).

The authors of that publication stress the economical advantages of network arch railway bridges compared to standard tied arch bridges with vertical hangers, especially for long spans. The guidelines point out the equality in structural and fatigue behaviour, despite the possibility of remarkably lighter structures for network arches. Considerable advantages of the network arch over its classic contender are also seen in the serviceability limit state due to larger stiffness and lower self-weight and therefore smaller deflections. Wind-induced and rain-wind-induced vibrations are also less significant when sticking to the hanger arrangement of network arches. The members of the DB advisory board do not regard the possible high slenderness of network arches as a drawback in terms of robustness.
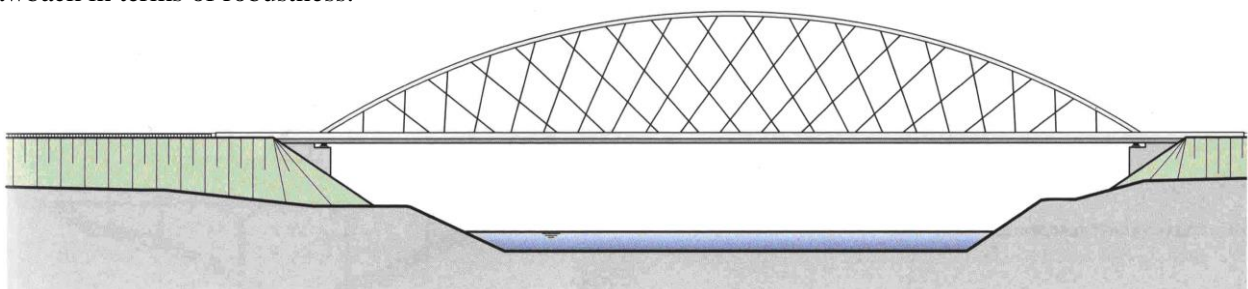
Fig. AAA4. The network arch as an innovative alternative railway bridge according to (Schlaich et al. 2008)

Frequently a matter of discussion, the costs for corrosion protection measures are also regarded as being equal to both network arches and tied arches with vertical hangers, whereas network arches become more advantageous taking benefit of the possibility of using a pre-stressed concrete tie.

In all aspects, the DB advisory board's comparison between railway network arches and railway tied arch bridges with vertical hangers turns out equal or in favour of network arches. Special attention should be paid to a connection design that implies little fatigue strain and an appropriate hanger arrangement. Guidelines for good hanger arrangements are given in (Schanack, Brunn 2009a).

B. On arches.

Main points:

In the author's opinion most of the arch should normally be part of a circle, but a smaller radius could be used at both ends of the arch. The rise of the arch should normally be between 0.15 to 0.17 times the span. In America the rise of the arch can be up to 0.2 times the span.

The Nielsen bridges are the forerunners of the network arches. See Fig. J1, (Nielsen 1929) and (Tveit 2013 pp. 54 and 55). Their arches were usually made of concrete. If the formwork and the erection are not too costly, this is a good idea. It might be an extra good idea for bridges over long stretches of navigable waters where it might be practical to produce the spans on land. (Tveit 2013 pp. 47-55).

These days most arches should be made of steel. An economical alternative would be arches made from universal columns or American wide flange beams. p. C-1. The profiles could be bent to the right curvature at the steel works. (Tveit 2013 p. 93). If they have vertical flanges, attaching hangers and windbracing would be simple. See fig. C1. The distribution of stiffness would be good, because the arch is more likely to buckle out of the plane of the arch than in the plane of the arch.

Box sections should be considered for longer spans and when extra stiffness is needed for the erection.

When the arches in arch bridges have vertical hangers, it is important that creep does not change the shape of the arch over the years. Therefore the shape of the arch should be near to a second degree parabola. (Tveit 2013 p. 14). When the arch is made of steel, a constant curvature is better.

Compared to the parabolic shape the constant curvature gives more even force in the middle of the arch and shorter wind portals. (Tveit 2013 p. 14). It also gives ease of production. Other curvatures of the arch can only give moderate savings.

If a smaller radius of curvature is used near the ends of the arches, the wind portal would be even shorter and the force in the middle half of the arch would be smaller and more even. It would also lead to more even maximum bending in the chords. (Brunn & Schanack 2001). The same effect can be achieved by elliptic arches (Teich 2012).

If the last members in the tie are made a little shorter, then the bending and rotation at the end of the bridge is reduced and the tension in the shortest hanger will be more equal to the tension in the longer hangers. This has been done in the Steinkjer and the Bolstadstraumen network arches. (Tveit 2008 p. 5b.). This has made the main spans 0.3 m shorter than the influence lines.

If there are no transversal beams in the tie, then the distance between the nodal points in the arch should normally be constant. The arch members at the end of the span could be 1.5 times longer than the other member of the arch.

Arches made of steel tubes look good and attract less force from wind. Steel tubes were chosen in the very slender arches of the Brandanger Bridge in western Norway. Steel tubes were chosen for the arches partly in order to reduce the wind pressure.

Steel tubes can be used in the arches of network arches to make the steel skeleton lighter. Then smaller cranes are needed to move the steel skeleton to the bridge site. The steel skeleton consists of arch hangers and a temporary lower chord. This skeleton becomes much lighter if the arch is a steel tube that is filled with concrete shortly after the steel skeleton has been moved to its permanent position.

There are two ways of filling the steel tube with concrete. The concrete can be pumped in at the bottom of the tube or it can be pumped through a flexible pipe inserted at the top of the arch. The latter method gives smaller stresses in the steel tube. Furthermore there will be less problems if the concrete starts to harden before the casting is finished. Pumping in the concrete at the bottom of the arch is probably best, provided everything goes according to plan.

In the Waikato River network arch in New Zealand the rectangular arches are filled with concrete. See fig. H-13.
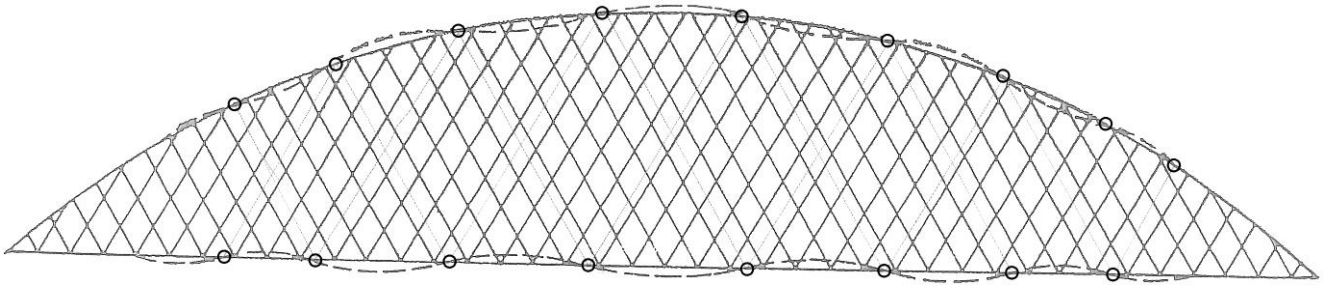
Fig B1. Possible form of buckling in the plane of a network arch

In (Tveit 66) the author suggested the possible form of buckling seen in fig. B1. The rings and the dotted lines have been added later. Often fig. B1 can convince us that bucking out of the plane arches is decisive.
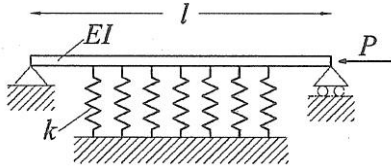

Fig. B2. Strait compressive member with elastic support

In a very good article (Schanack 2009) shows how buckling in network arches can be calculated by the formula for the member in fig. B2. The article is in German, but introduction and text for the drawings are in English. The same material is presented in Schanacks doctoral thesis (Schanack 2008).

For an exact calculation of buckling we can assume that the tie follows the arch with a much bigger radius of curvature.

Frank Schanack (Schanack 2009) rightly pointed out that sufficiently accurate results are found if we assume that the tie remains straight. Then the arch buckles like a column supported by flexible springs and the problem becomes much simpler.

$$P_{cr} = \min\left(\left(\frac{n \cdot \pi}{\ell}\right)^2 \cdot EI + \left(\frac{\ell}{n \cdot \pi}\right)^2 \cdot k\right)$$

P_{cr} = buckling load
n = number of waves
ℓ = Length of member
EI = stiffness of member
k = stiffness of support

For the maximum load on the whole bridge (Schanack 2008) shows that the buckling load is:

$$N_{cr} = \min\left(\left[\left(\frac{n \cdot \pi}{\alpha_0}\right)^2 - 1\right]\frac{EI_y}{R^2} + \frac{\dfrac{EA_p \cdot n_p \cdot R}{f \cdot 2 \cdot \alpha_0}\sin^3 \alpha}{\left(\dfrac{n \cdot \pi}{\alpha_0}\right)^2 - 1}\right)$$

$$R = \frac{f^2 + \dfrac{s^2}{4}}{2 \cdot f} \qquad \alpha_0 = \arcsin\left(\frac{s}{2 \cdot R}\right)$$

f = Rise of arch
EA_p = Stiffness of a hanger
EI_y = Stiffness of arch
s = Span
α = Angle between arch and hangers
n = Number of waves
n_p = number of hangers per arch

Schanack have stated that the formula is not exact, but is most unlikely to deviate more than 5% from the exact value.

In network arches with moderate length and width it is often best to avoid structural steel in ties of network arches. If the tie is a concrete slab on top of elongating steel beams, a lot of reinforcement must be used to keep the crack width down. The next two figures illustrate this point. They come from (Tveit 2013 pp. 93 to 93b).
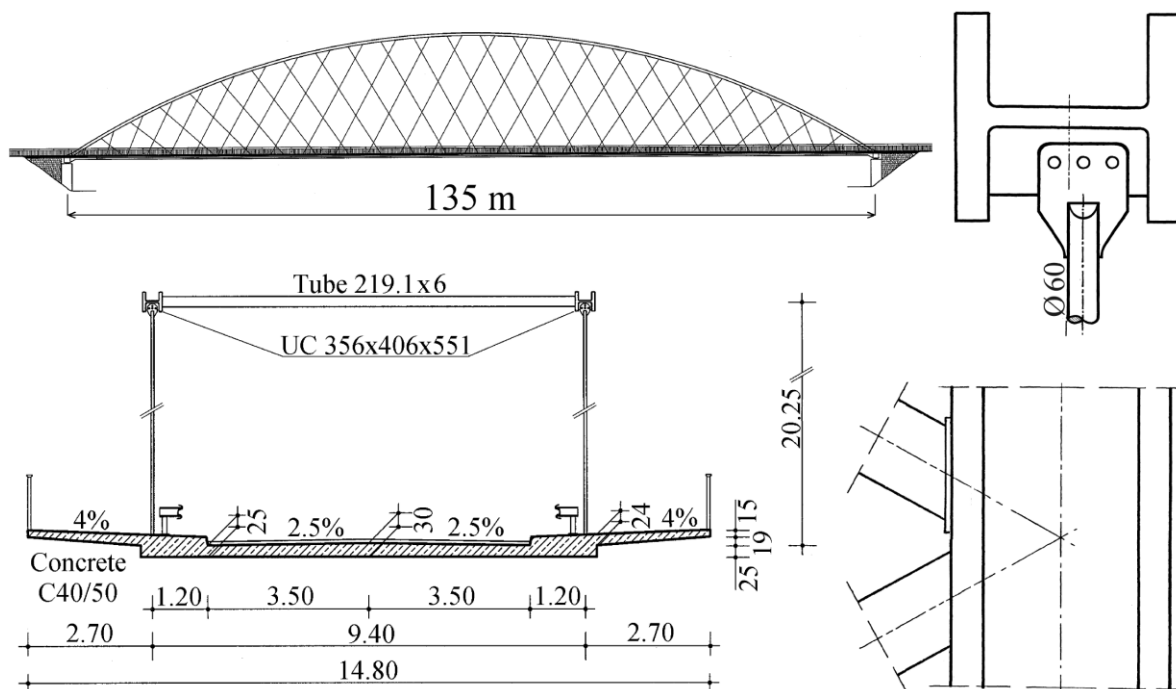
The Åkvik Sound Bridge

Fig. C1 shows the Åkvik Sound network arch designed according to EU codes in 2001

Fig. C1 shows a bridge between two islands in northern Norway. Teich and Wendelin designed this bridge when doing their master's thesis in Grimstad, Norway. (Teich & Wendelin 2001). Their calculations can be found at http://home.uia.no/pert/ under the button "Master's Theses". Two ways of fastening the diagonals between the arches are shown. See also (Tveit 2013, pp. 9-12). This bridge is used for predicting materials needed for two lane network arches spanning between 60 m and 200 m. See chapter AA.

Fig. C2 compares the steel weight of the network arch in fig. C1 to the steel weights of German arch bridges with vertical hangers. N indicates that there are no members between the arches. S indicates that the arches slope towards each other. The year when the bridges were built is also indicated. (Tveit 2013, p. 93-93a).

The bridges with vertical hangers use more reinforcement than the network arch. This is remarkable because the bridges with vertical hangers have steel beams under the concrete slab in the tie. The longitudinal steel beams have tension. Therefore the slab on top needs a lot of reinforcement to keep the cracks in the slab small. - The network arch uses only 35% of the structural steel in the Calbe Bridge (Fiedler & Ziemann 1997) and only 23% of the structural steel in the Jerusalem Bridge in Magdeburg. (Fiedler 2005)
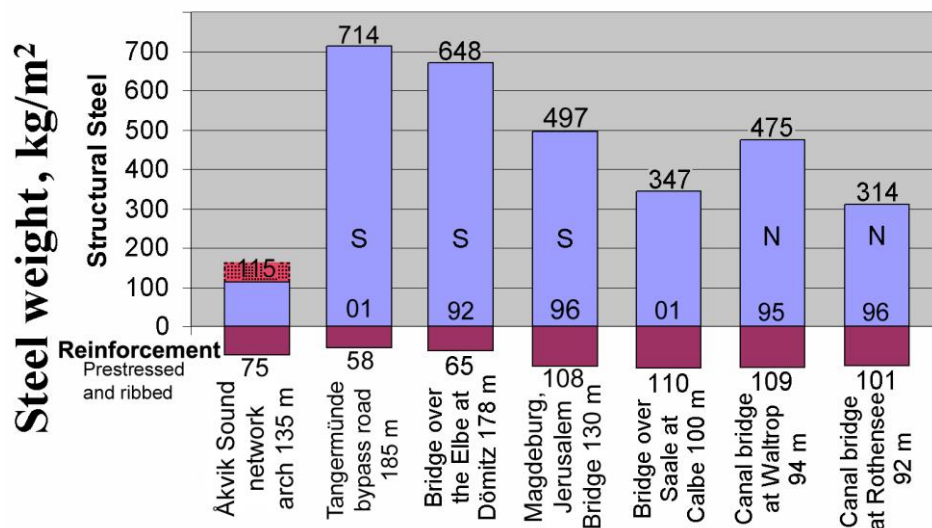
Fig. C2. Steel weights per square metre in various arch bridges

In fig. C2, the dotted area on top of the Åkvik Sound steel weight indicates the extra steel weight necessary if a temporary lower chord is used for the erection of the bridge in p. C-1.

Nevertheless the network arch uses much less steel than arch bridges with vertical hangers.

The steel weight in the temporary lower chord costs less than the rest of the steel, partly because it needs little to no corrosion protection.

THE STRAUBING BRIDGE COMPARED TO A NETWORK ARCH

This and the next page are from page 13 and 14 in (Tveit 2013).

The next two pages compare two tied arch bridges spanning 200 metres. The bridges are shown in fig. C3. The bridge with vertical hangers was built over the Danube in Bavaria in 1977. (Kahman & Beisel 1979) The author designed the network arch for the IABSE congress in Vienna in 1980. J-12, (Tveit 1980) and (Tveit 2008, pp. 59 to 72). The pattern of hangers is 200A, which is shown to the left in fig. 61 in (Tveit 2013).

The main dimensions of the two bridges are surprisingly similar. The rise of the arch at Straubing is about 7% higher. For the network arch the concentrated loads are bigger, but the total payload for the two bridges is about the same.

The tie of the network arch should usually have a concrete slab because the weight of the tie restrains the relaxation of hangers. The lower chord of the arch at Straubing is an edge beam and an orthotropic plate. The two arches have roughly the same cross-section and stiffness. The stiffness of the tie in the network arch is just under half the stiffness of the lower chord in the bridge with vertical hangers.

The steel weight for the bridge with vertical hangers is only twice the steel weight of the network arch. This impresses the author because the Straubing Bridge uses no concrete in the deck.
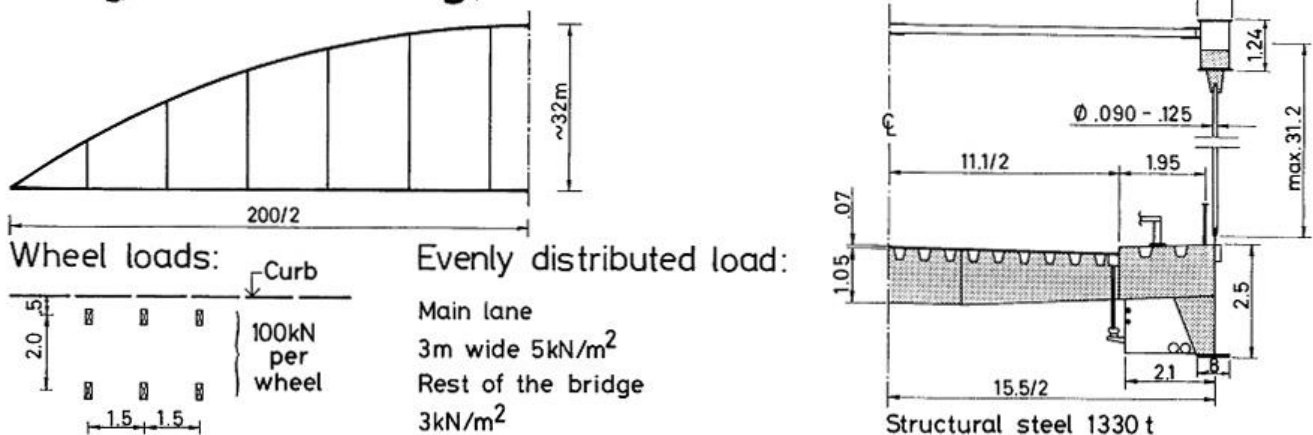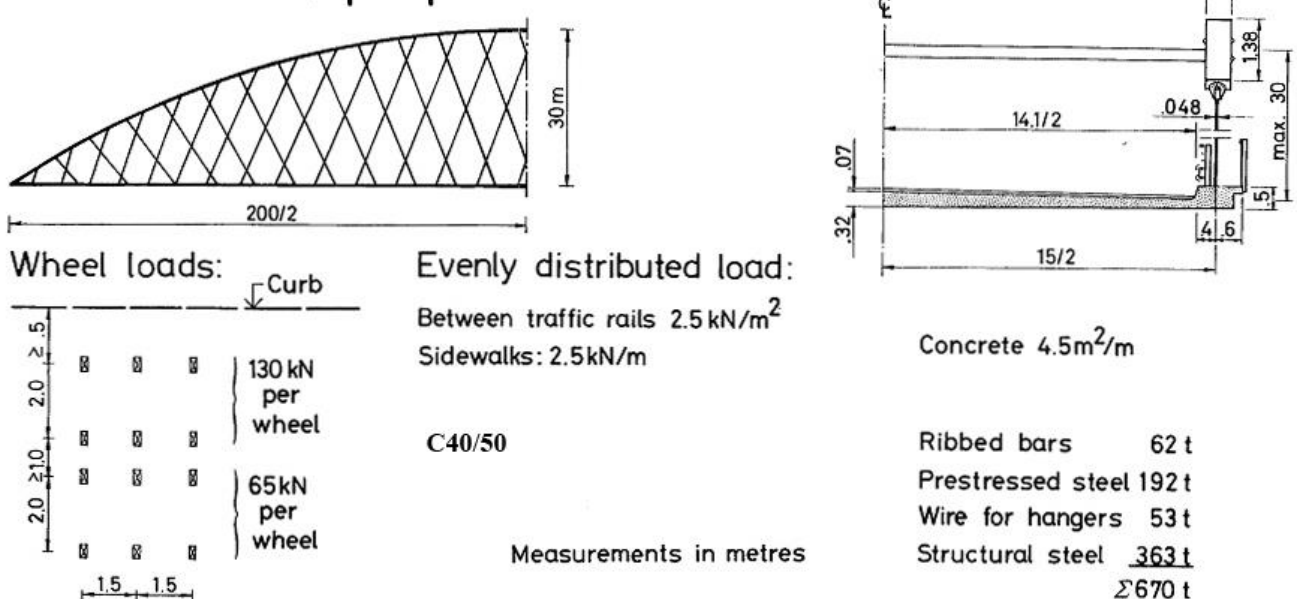
Fig. C3. Geometry, loads and quantities of two tied arches

The lower part of fig. C4 shows a comparison between the influence lines for bending moments in the lower chord of the two spans shown in fig. C3. Please note that the maximum influence ordinate in the lower chord of the network arch is the same as for a simply supported beam spanning 5.6 m. The distance between the arches is 15 m. Thus it is obvious that the bending in the tie is normally much smaller than the maximum bending found in the middle of the slab.

In long narrow bridges, however, the longitudinal bending might become decisive mainly because much of the strength of the concrete is needed for taking the variation of the axial force in the tie. In which case a bit of extra longitudinal ribbed reinforcement is all that is needed to put things right.
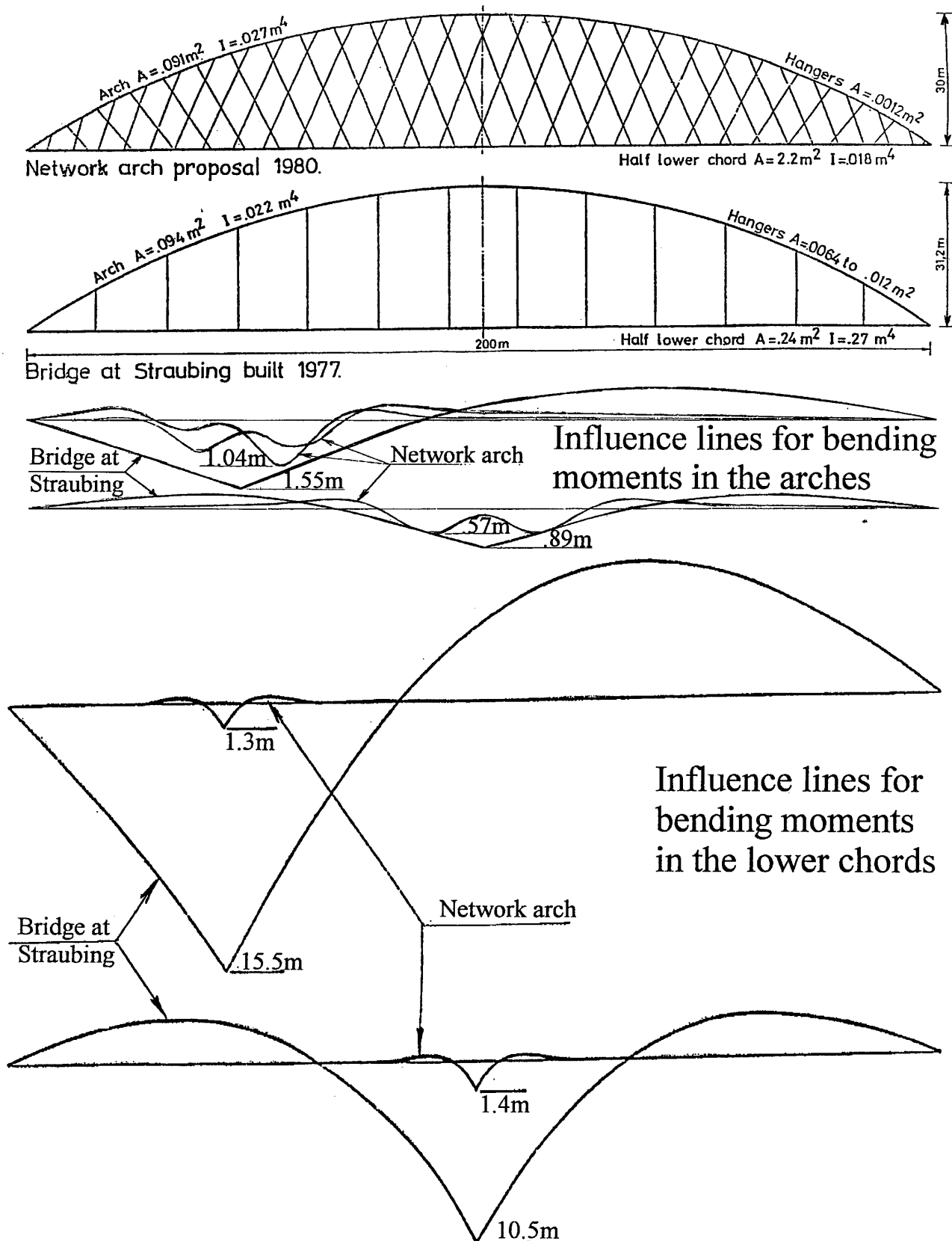
Fig. C4. Areas, stiffness and influence lines for the lower and upper chord of two tied arches

Network arches with all hangers in tension act as trusses and have little bending in the chords. Hangers can, however, be made to relax by live load on one side of the span. Fig. C4 shows how the network arch labelled 200A (See fig. D8) reacts to a very big load on one side of the span. The span is also pictured in fig. C3. The left 54% of the span carries an unlikely live load equal to the dead load on the lane.

The dotted hangers are relaxed due to live load. They are numbered according to the sequence in which they relax. The segments of the chords marked "a" belong to parts of the arch which act like a truss, i.e. where all hangers are in tension.

The segments of the arch marked "b" are attached by hangers in tension to a section of the span acting like a truss.

The chords marked "c" are connected to each other by one set of hangers in tension. This part of the bridge functions a bit like a tied arch with one set of hangers.

The equilibrium of zone "c" is dependent on shear and bending in the chords. Zone "c" can have large bending moments. Zones "a" and "b" are more firmly held in place than zone "c".

Relaxation of hangers causes a significant increase in bending moments in the chords only after a zone "c" exists, and even then bending moments do not increase as fast as the moments in a tied arch with vertical hangers.

This is because the sloping hangers restrain the horizontal displacement of the arch and because parts of the network arch work like a truss.

Even if some hangers relax, moderate live load on part of the span might give smaller maximum stresses in the arch than the same live load on the whole span. This is because the partial live load gives smaller axial force in the arch.

Fig. C5a. Forces and deflections due to an extremely skewed load on 200A

Fig. C5. Development of maximum stress in the arch member numbered 114 in fig. C5a

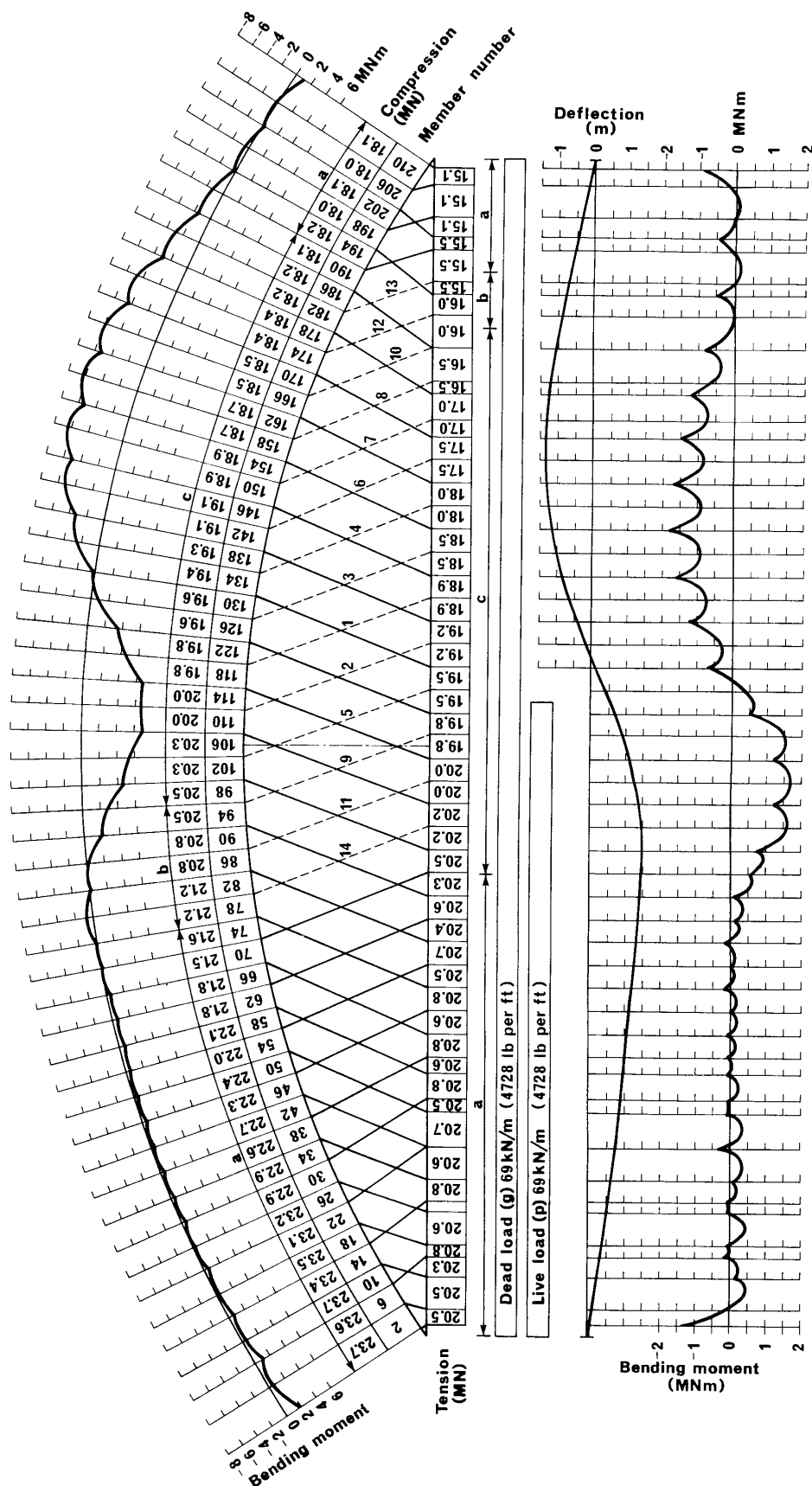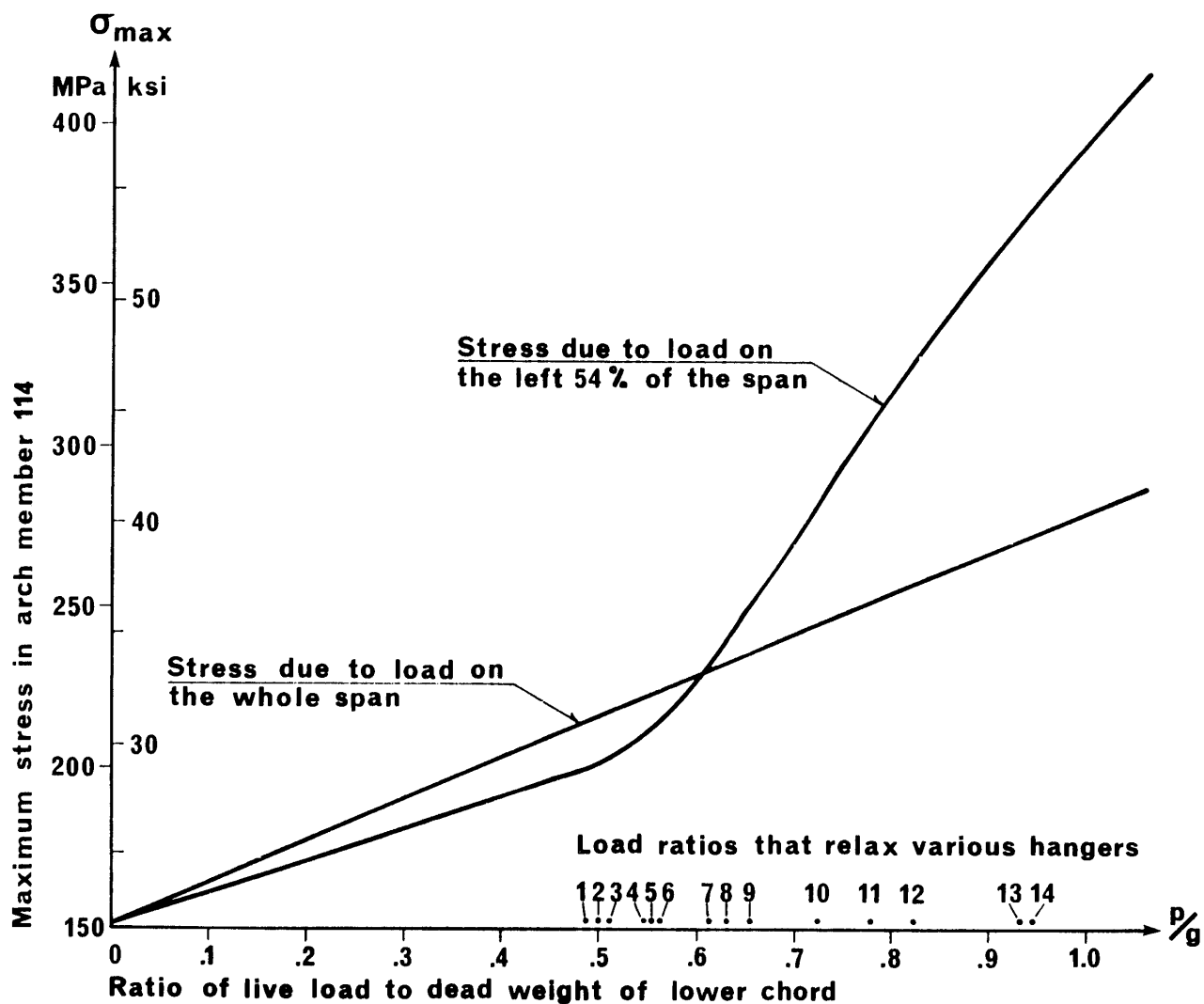Fig. C5 applies to bridge 200A which has the arch shown in fig. C5a. In bridges with more slender chords, i.e. universal columns, the bending moments will increase more abruptly after some hangers have relaxed. This is because slender chords have less ability to take the increased bending moments that occur after some hangers have relaxed. There is more on this in (Tveit 2013, pp. 60 to 68).

Fig. C5 shows how maximum stresses in member 114 increase with increasing evenly distributed load. The loaded length of 54% of the span and member 114 has been chosen because it is the member that gives equal maximum stress due to partial load and full load for the lowest live load intensity. The curved line shows how stresses increase with increasing load intensity on the left of the span. The straight line shows how stresses increase due to live load on the whole span.

Considerable bending moments due to relaxation of hangers are present when the two load cases in fig. C5 lead to equal maximum stresses.

For fig. C5 the span in fig. C5a has been calculated by non-linear calculation in the deflected state assuming constant modulus of elasticity. Still stresses due to partial load are almost linear until the first hanger relaxes. When hangers 1 and 2 have relaxed, the maximum stress in member 114 increases equally fast due to partial load as due to load of equal intensity on the whole span. For a live load of 61% of dead load, partial load and full load give equally high maximum stress in member 114. Hangers 1 to 6 are now relaxed.

Broadly speaking, hangers relax because of horizontal displacement of the arch due to partial live load on the lane. If we give the hangers a smaller angle with the lower chord, their tendency to relax is reduced, and bending due to relaxation is reduced. The smaller angle with the chords will normally lead to an increase in the bending moments due to concentrated loads. Clearly a compromise must be found.

Since it is more complicated to calculate spans where hangers relax, it saves considerable time if the hangers have slopes that make it relatively easy to prove that it is the load on the whole span that decides the dimensions of the chords. Fatigue in hangers also speaks for hangers that do not relax. Look also at the lower 40% of page 5a in (Tveit 2013).
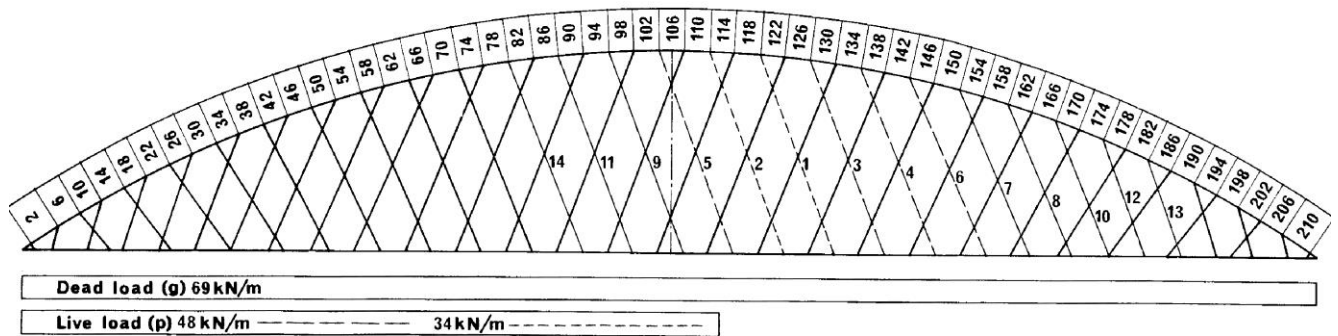
Fig. C6 relaxation of hangers in 200A when the live load is 0.7 and 0.5 times the dead load

TIES WITH LONGITUDINAL CONCRETE BEAMS AND TRANSVERSAL STEEL BEAMS
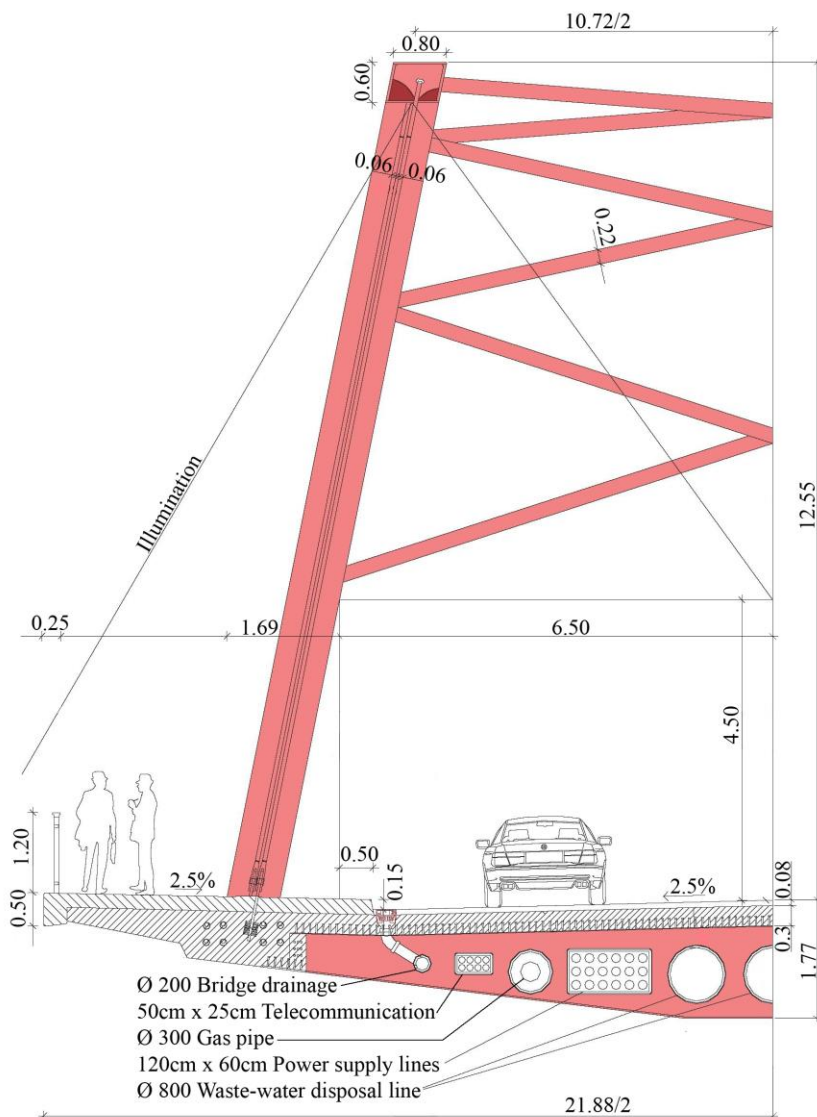
Longitudinal steel beams in the tie can be avoided even if there are transversal steel beams. It has been achieved in the Schulenburg Bridge. Span 90 m. See fig. C7. It is presented in a master's thesis by Wolfgang Beyer (Beyer 2006). Frank Schanack was adviser. He used a combination of transversal steel beams and longitudinal concrete beams. In this way the prestressing cables do not give compression in the steel beams in the tie.

Fig. C7 shows a network arch suggested in the master's thesis of W. Beyer in 2006. Span 90 m

Fig. C8 shows that the Schulenburg Bridge does not look clumsy in spite of the deep beams, but it would have looked even better if the tie had been 0.5 m deep.

Fig. C8. Picture of the Schulenburg Bridge in fig. C6

Thickness of simple concrete slabs between planes of arches
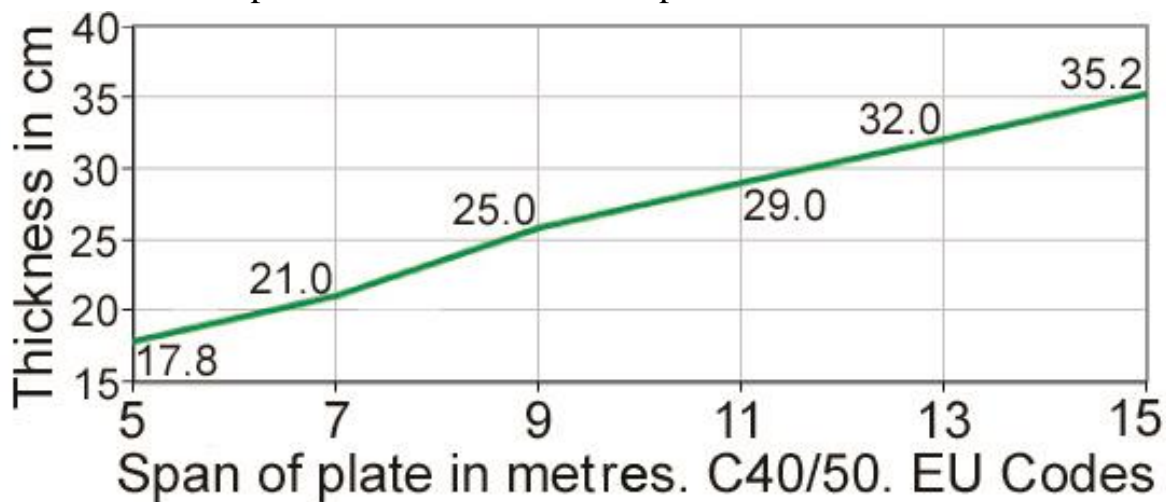
Fig. C9. Necessary thickness of concrete slabs

The diagram in fig. C9 comes from (Teich & Wendelin, 2001). In bridges where the arches are more than 10 m apart the deflection will influence the necessary thickness of the concrete slabs. This problem of deflection becomes more important with increasing strength of concrete. Suitable camber can counteract deflection due to stresses, creep and shrinkage.

Normally it does not pay to use transversal prestressing in the tie, especially when high concrete strengths are used. This is because the prestressing cables will not be low in the cross-section. In this case the prestressing does not contribute much to reducing the compression in the top of the cross-section.

The longitudinal prestress will give a reduced deflection in the slab between the edge beams. Because of Poisson's ratio, the upper fibres in the middle of the slab will have more compression than the lower fibres. This will lead to a reduction in the deflection.

If the deflection becomes too big due to more creep and shrinkage than expected, the deflection can be counteracted by spanning fibre-reinforced polymer cables transversally under the concrete slab. See "Strengthening of network arches already built" p. G-1. Wedges can be put in between the slab and the cables till the deflection is sufficiently reduced. Since this can be done, thinner slabs can be used in the hope that the deflections will remain suitably small. See also "Strengthening of network arches already built" p. G-1.

Where the transverse prestressing reinforcement can be changed, its stress can also be altered to control the transverse deflection. In this case distances up to 20 m between the arches can be used.

Should the arches be inside or outside the pedestrians?

The biggest bending in the lower chord is usually halfway between arches. The longitudinal bending in the lower chord is usually smaller.

In 2012 some engineers from the public roads in Norway suggested that the arches should be outside the footpaths. This would not be economical, because it would lead to much longer spans between the arches. Thus the lower chord would be much thicker and heavier.

For the pedestrians it is nicer to have the hangers and the arches between them and the heavy traffic. The railings can be lighter if they are between the hangers and the outside of the lower chord. See fig. AA3 and fig. E11. The space under the arches can be used when groups of pedestrians pass each other, and when cyclists pass small groups of pedestrians.

Should the lower chord from the erection of a network arch become part of the permanent structure?

The author has often been asked: Since you have got the temporary lower chord anyway, why do you not make it part of a permanent combination of steel and concrete? Then you do not have to remove it. Below is a list of reasons why the temporary lower chord should not become part of the permanent tie. The author would like to state that the full potential of most network arches can be achieved only if the tie is a concrete slab.

1. A permanent longitudinal temporary steel chord leads to a deeper lower chord. This is unfavourable from the aesthetic point of view and leads to longer ramps at some bridge sites.

2. If the tie is a concrete slab instead of a steel structure, there is a smaller distance between the traffic under the tie.

3. If the traffic under the bridge bumps into the tie, there is a bigger chance of damage if the tie is a steel structure.

4. A network arch with a permanent steel tie might be simpler to erect, but it would have a much higher steel weight. The comparison is lopsided, but in fig. 14 on p. 10 the weight of the lower chord and the transverse beams is 264 tons. The temporary lower chord for the Åkvik Sound Bridge weighs 24 tons. See page 12.

5. The longitudinal bending in the lower chord in the finished span is so small that there is no need for a longitudinal beam of structural steel to take the longitudinal bending.

6. The concentrated wheel loads always cause a lot of bending in the slab. In narrow bridges only moderate amounts of extra reinforcement are needed for the slab to span between the arches.

7. If you use transversal beams, the loads on the slab are concentrated before they reach the edge beam. This gives more bending in the edge beams and in the arches. On page 66 it is shown how the wheel loads for the network arch in fig. 23 on p.13 are distributed when they reach the edge beam.

8. A permanent tie of structural steel in tension causes cracks in a concrete slab above it. This reduces the durability of the concrete slab.

9. Transverse beams in the permanent lower chord would make the reinforcement in the slab more complicated.

10. The temporary lower chord is joined together by high strength bolts. It needs no corrosion protection and can be produced on site. Thus the cost of fabrication per tonne is not high.

11. The edge beam is cast before the slab. When the slab is cast the longitudinal bending in the tie is taken mainly by the edge beam. Thus the distances between the temporary transverse beams can be constant.

12. A temporary lower chord can be used again and again in bridges of varying widths and lengths. One just has to make some new holes and maybe cut or weld some beams and windbracing. The wood on the temporary lower chord can be Doka beams that can be reused.

13. The transverse beams in the temporary lower chord can be chosen freely as long as they have sufficient strength. Any old beam might be used.

14. A permanent lower chord would be shop welded and would have a corrosion protection that has to be maintained forever.

15. The great longitudinal tensile forces in the tie are best taken by prestressing cables because of the high strength-to-cost ratio.

16. Prestressing cables prestressed against concrete will take fatigue well.

17. The longitudinally partially prestressed slab in the lower chord of a network arch bridge is favourable as far as maintenance is concerned.

18. The temporary lower chord is simple to remove and erect. See "Erection of the steel skeleton of a network arch on the side span of the Skodje Bridge," page 50a, and "Removing the temporary lower chord of the Skodje Bridge", page 52.

19. If a permanent lower chord is used, the formwork might still have to be removed. If the formwork becomes part of the permanent structure, it is likely to be relatively costly.

The validity of these statements might depend on the quality of sand and gravel and the available concrete technology in the country in question.

Steel or concrete in the tie of network arches?

The authors first two network arches, built in 1966, had concrete ties. At the time concrete ties were used in Norwegian suspension bridges.

During the ice ages most organic material was removed from Norway. Thus Norwegian sand and stone are suitable for making concrete. In the sixties concrete was much more important than steel in the building industry in Norway.

The author thought that it was important that the weight of the concrete counteracted the relaxation of hangers. In page C-4 and C-5 it is shown that the relaxation of hangers is less important than the author thought in the beginning.

If there is concrete on top of the steel beams in the tie, there will be many transversal cracks in the tie. Prestressing cables in concrete ties are an efficient way of taking the tension in the ties. They give a beneficial compression in the tie and counteract cracks.

Most network arches that have been built have steel ties. Those ties are lighter and are usually finished before the main spans are lifted to the pillars. In every design of a network arch it must be decided whether the tie in the main span should be made of steel or concrete.

The sloping hangers make network arches different from other bridges. Thus the chapter on hangers will be long. It discusses design of hangers, slope of hangers and effect of the possibility of breaking of hangers.

1. *Design of hangers*

A. Wire hangers with adjustable length

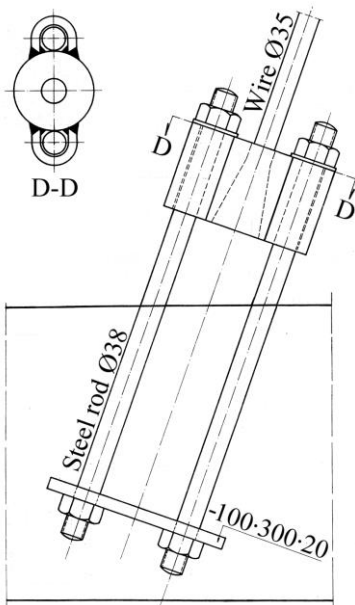The hangers in the first two network arches.

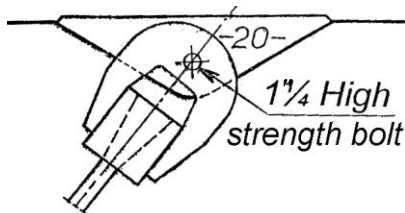Fig. D1. Lower end of a hanger in the network arch in Steinkjer

Fig. D2. The upper end of a hanger in the bridge at Steinkjer

The cables in the Steinkjer Bridge are shown in figs D1 and D2. (Tveit 2013) pp. 5b-6c. H2 and H3 and J-7 and J-8). The lower end of the hanger is copied from a bridge at Luleå in Sweden (Kungliga Väg- och vattenbygnadsstyrelsen (1954)). An advantage of this design is that the length of the hanger can be adjusted by one man turning the nuts at the end of the steel rods. If there had been only one rod at the lower end of the hangers, altering the length of a hanger would not have been so easy. The upper end of the hangers in the bridge in Luleå is shown in fig. D3.

The hangers at Steinkjer were made of cables that consisted of 37 galvanized compact round strands. Locked coil ropes were not used because the author was afraid that they could be bent and opened during transport and erection. In the Steinkjer Bridge the diameter of the hangers was 35 mm. The ends of the wires are anchored in conic holes. The threads at the ends of the wire must be spread out well before the conic holes are filled with molten zinc. The mining industry has experience with fastening wire ropes in conical holes with molten zinc and so an expert on wires in mines supervised the process.

The hangers of the Bolstadstraumen Bridge were very similar, but in the Bolstadstraumen Bridge the supervision was not so good and antimony was added to the zinc to lower the melting point. There the wire ropes had a tendency to slip out of the conical hole.

Rods set in the concrete cannot be changed. Thus they must have good corrosion protection. Olav Grindland who works with upkeep in the Norwegian Public Roads Bridge Department, suggests corrosion protection of the steel rods 60 mm into the concrete. The level of concrete around the rods should be 5-10 mm higher than the surrounding concrete. In this way the de-icing salt will be washed away from the rods by the rain. (Tveit 1964) shows putty on top of the concrete between the steel rods and the concrete.
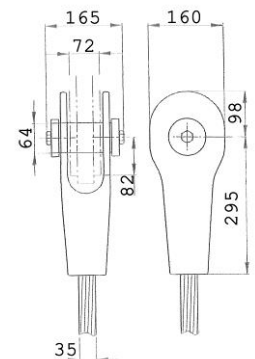
Fig. D3 Upper end of hangers

Where the hangers in the Steinkjer and Bolstadstraumen bridges cross each other, one of the hangers is covered by a split open plastic tube. See fig. D4. Thus they will not damage each other if they bang or rub against each other. The ends of most hangers were in the same plane. Thus the hangers had to be pushed sideways when they crossed each other. This is alright because the resulting stresses are small. Furthermore, when hangers have maximum load the crossing hangers have moderate loads. This reduces the sideways bending in the hangers when they carry the maximum load.

The author did not quite trust the strength of the bolts at the upper ends of the hangers in fig. D2. Therefor a weld was introduced at the upper end of the hanger. It would be better if the bolts and the plates to which they were fastened were so thick that welding could be avoided. In the Bolstadstraumen Bridge there were two bolts through the plate at the upper end of the hanger. This was more complicated.

The force in a hanger was measured by applying a transversal load to it and measuring the deflection. See eight pages on "Adjustment of hangers in network arches" on the author's home page http://home.uia.no/pert/ under the button "Supplementary Information". The force in a hanger can also be found by measuring the frequency of the vibrations in the hanger.

There are many different designs of hangers in H-7 to H-37.

Fig. D4. Crossing hangers

B. Hangers with replaceable rods through the edge beam

The Swedish bridge office of the public roads in Borlänge has been dissatisfied with rods that come directly out of the concrete, as these cannot be changed. They would like to avoid this solution. A lot of experience has brought them to that conclusion.

The fact that this arrangement has worked well in the Steinkjer network arch (See pp. H-2 and H-3) has little significance in Sweden. The fact that the rods coming out of the concrete have given no problems for 50 years might be due to the longitudinal prestress in the ties of the Steinkjer and Bolstadstraumen Bridges. (Tveit 2013 pp. 6-6c.). In the Brandanger Bridge in western Norway the lower ends of the hangers can be changed. They are fastened to bolts through the lower chord.

When the steel rods at the lower ends of the hangers in the Steinkjer network arch were designed (See fig. D1), Professor Arne Selberg mentioned that maybe the rods could be twisted out of the concrete. That is why there is a nut at the ends of the rods.

In fig. D5 there is supposed to be a compound glued to the vertical steel rod that makes it possible to twist them out of the concrete. Removal of some of that compound should make it possible to put in new rods to replace the old ones.

Maybe the triangular plate on top of the edge beam should have dowels underneath. It could be about a centimetre above the concrete at the start of the casting. Afterwards it should be lowered and maybe hammered into the soft concrete.

The lower end of the wire in the hanger in fig. D5 can be used to adjust the hanger length. The details have been copied from the home page of the Italian firm Redaelli. A tensioning device must be applied to the lower end of the hanger. Otherwise the tensioning rod cannot be turned round.

The author admits that he prefers the lower ends of the hangers in the network arch at Steinkjer. See fig. D1. In French: "On revient toujour à ses premiers amours."
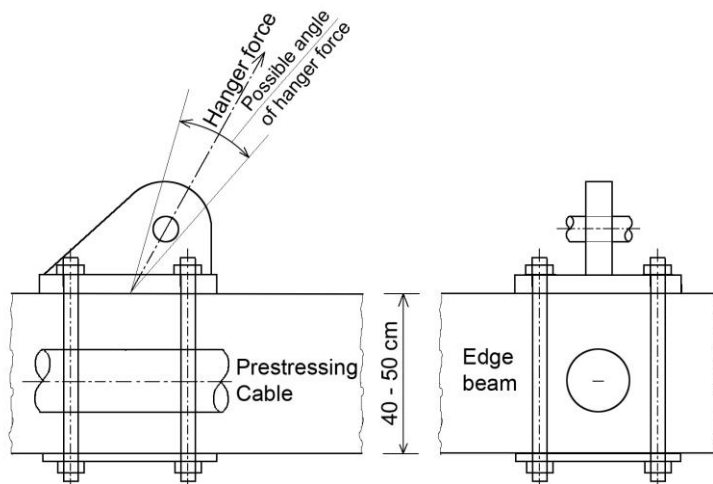
Rough sketch of lower end of hanger

Fig. D5. Exchangeable lower ends of hanger
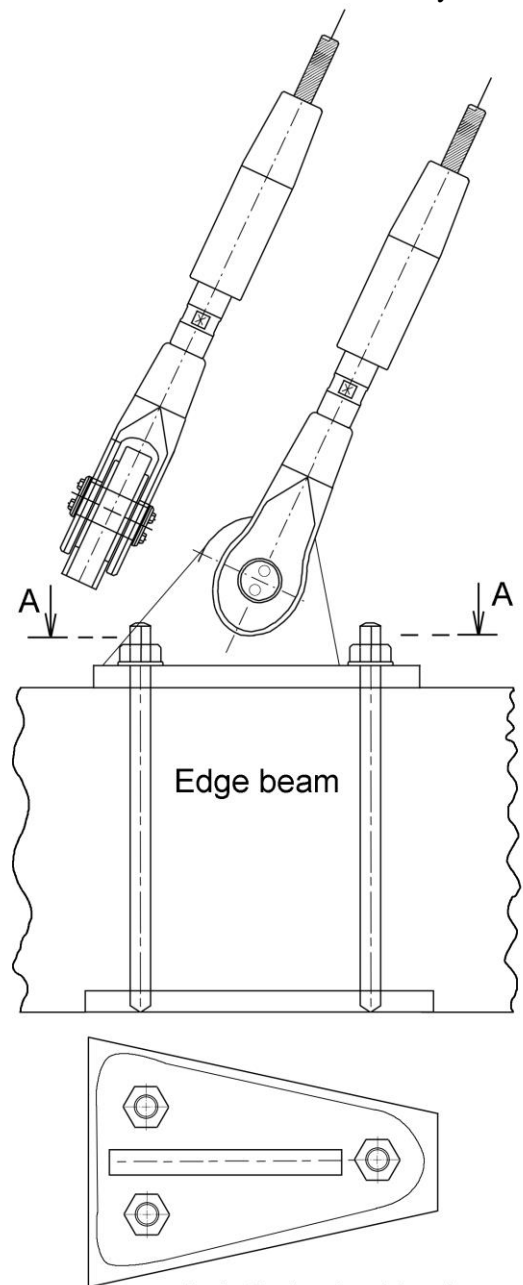
A-A Only steel is shown
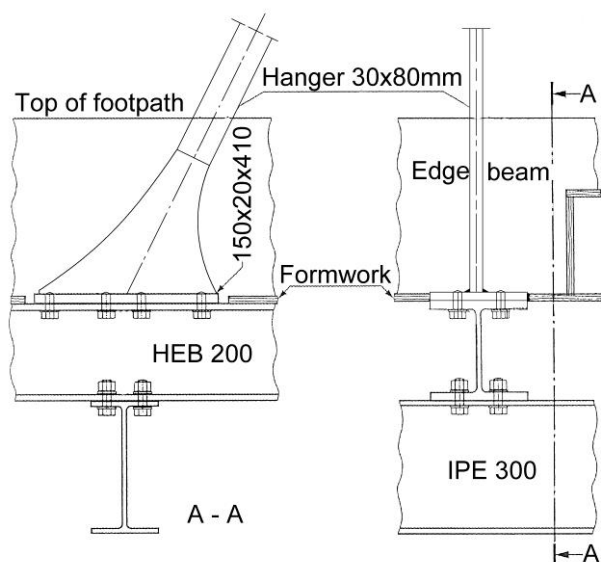
Fig. D6. Joint between a flat hanger and a
temporary lower chord

C. Hangers made of steel rods

a. Round hangers

Such hangers can be found in arch bridges with vertical hangers. See (Günther et al. 2000) and (Bast 2008). In this publication hangers for network arches are mentioned on page 14. In his doctoral thesis Stephan Teich (2012, chapter 2) has done extensive research on this subject. An interesting kind of hanger can be found on page H-10.

b. Hangers made of flat steel

Welds in hangers made of flat steel have high fatigue strength. See fig. 9.26 in (Graße and Tveit 2007). The width of the hanger can easily adjust to the hanger force.

Here there might be extra trouble where the hanger comes out of the concrete. Maybe putty should be used between the concrete and the flat steel. Round hangers tend to look better than flat hangers. In Germany flat hangers in railway bridges have had unwanted vibrations.

Fig. D6 indicates how hangers made of flat steel can be fastened to a temporary lower chord. The upper end of the hanger can be fastened to the flanges of universal columns or American wide flange beams as indicated in fig. 97 in (Tveit 2013). Hangers for various network arches are shown on pp. H-5 to H-37.

D. Slope of hangers
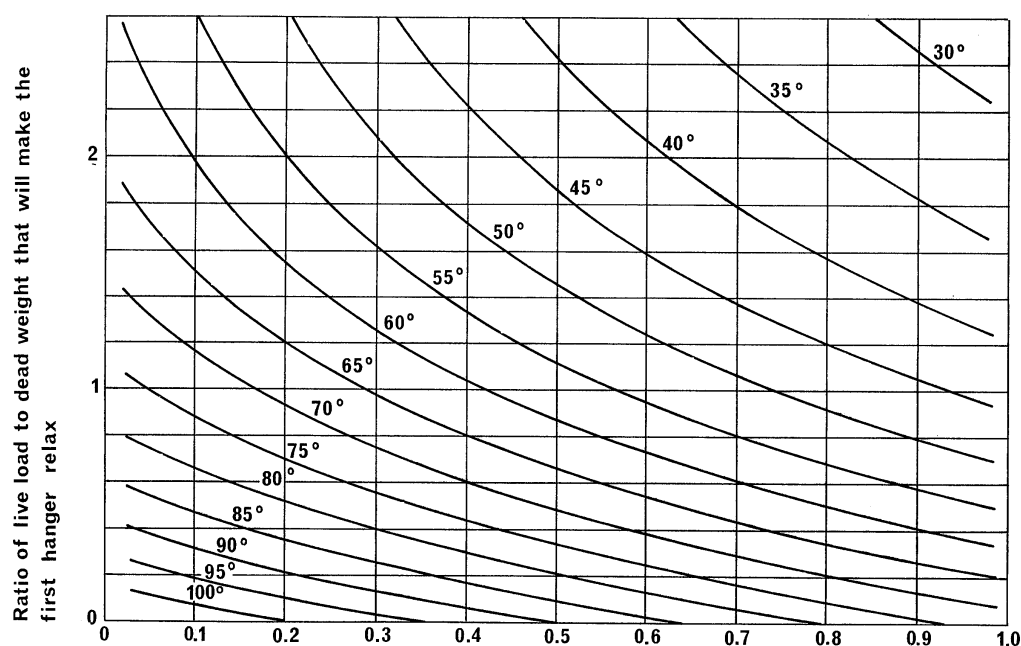
Fig. D7. Diagram that indicates when the first hanger will relax due to
a evenly distributed live load on the left side of the span

The slope of the hangers influences their resistance to relaxation, their maximum force and their fatigue load. Back in 1963 the author made many diagrams like the one shown in fig. D7 (Tveit 1966). It indicates the hangers' tendency to relax in a network arch where the rise is 0.15 times the span. The left of the span has an evenly distributed load. There is a lot on finding the ideal slope of hangers in (Tveit 2013, pp. 26 to 29j).

Back in 1980 the author (Tveit 1980b) suggested the hanger arrangement in fig. D8 for bridges spanning 200 m. The hanger arrangement was found by trial and error. The co-ordinates of fig. D8 can be found on (Tveit 2013 p. 62). The hanger arrangement on the right hand side of fig. D8 has been reduced by a factor 0.675 and used in (Teich and Wendelin 2001). The coordinates can be found in (Tveit 2013 p. 76).
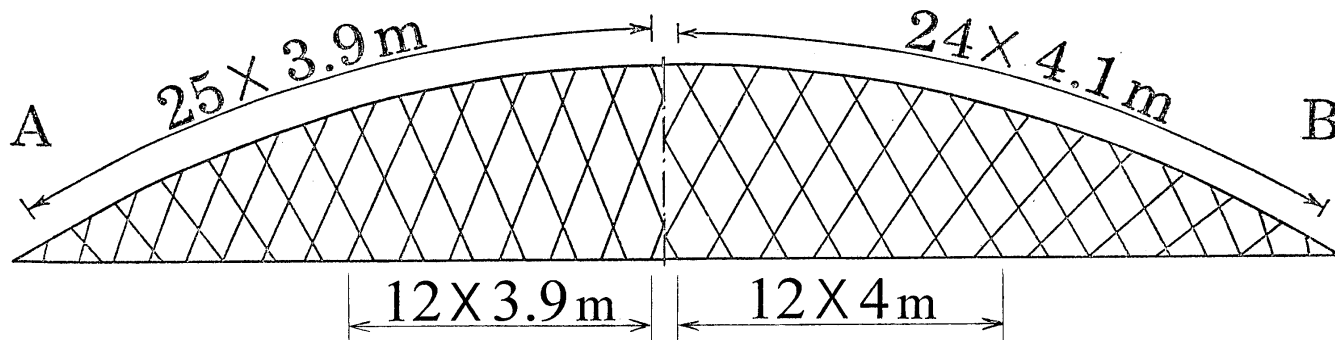
Fig. D8. Suggested hanger arrangement for two network arches spanning 200 m

A. Slope of hangers when the tie is a simple slab.

When there are no transverse beams in the tie, the upper ends of the hangers should be evenly distributed along the arch. Evenly distributed hangers tend to give a more even maximum force in the hangers. If there are transverse beams in the tie, the hangers would often be placed at the end of those beams.

The distance between the end of the arch and the first node in the arch should be around 1.5 times the distance between the nodes in the arch. In the network arch in Steinkjer the span was shortened by 0.25 m to get a better stress distribution in the hangers. This shortening can be seen by comparing the span in fig. 6a in (Tveit 2013) to the influence lines on (Tveit 2013 p. 57).

In the first two network arches, Steinkjer and Bolstadstraumen, the author used a small constant change of slope in adjoining hangers. See (Tveit 2013 p. 57 and p. 58). In their master's thesis (Brunn and Schanack 2003) suggested that the angle between the arch and the hangers should be constant in most of the arch. In section 6 in a revised edition of their master's thesis http://home.uia.no/pert/ under the button "Masters Theses" there is a lot on optimisation of hanger arrangements.

In his doctoral thesis, chapter 3 (Teich 2012) Teich has examined many hanger arrangements. He found that hanger arrangements with a constant slope between arch and hangers and hanger arrangements with a small constant change of slope in adjoining hangers are the best. In a recent article (Schanack and Brunn 2009a) go in for a constant angle between arch and hangers, but find that a constant change of angle between adjoining hangers is almost equally good. The constant angle between the arch and hangers gives simpler details.

B. Slope of hangers at the end of transversal beams

When the network arch has transversal beams in the tie the lower ends of hangers are normally fastened to the ends of the transversal beams. This is the case for the network arches in pp. H-8 to H-10 and H-23 to H-25. They all have a constant slope of the hangers. In these wide bridges the arch is so stiff that the uneven distances along the arch do not lead to much additional stresses due to bending.

Sometimes there are constant distances between the nodes along the arch and along the tie. See pp. H-17 to H-22. This looks good. The author does not know how uneven the maximum hanger forces are. In any case the amount of steel in the hangers is a small percentage of the steel used for the whole network arch.

3. BREAKING OF HANGERS

The hangers should be well protected by guard rails. If hangers break nevertheless, many hangers will have to be broken at the lower chord before the bridge is endangered in the collapse limit state. This is because hangers that have their lower ends near to each other at the lower chord have their upper ends well spread out at the upper chord. Thus the arch will not collapse until many hangers are broken. Near the end of the arch the hangers are not so well spaced, but here the arches are stronger.

Tension in the prestressing cables will prevent a rupture in the lower chord till a lot of hangers are broken. Bending capacity and tension in the deflected state will delay the collapse of the lower chord. Collision between lorries and the superstructure is a problem whenever structural members are above the lane. However, this problem is not overly serious in well designed optimal network arches.

Learned friends have told the author that network arches are robust structures.

Zoli and Woodward (2005) came to the same conclusion after having done a thorough dynamic examination of the effects of the breaking of hangers. Zoli has done a lot of work on the effect of the breaking hangers in network arches. Zoli concludes that in a well designed network arch hangers can be changed without the traffic being reduced. It is easier to do this exchange of hangers without interrupting the traffic if the arches do not slope towards each other. Furthermore a well designed network arch will not collapse if one or two hangers are broken. Because of the high redundancy, network arches are less likely to collapse than trusses and most other bridges.

For the author's dissertation (Tveit 1959) he built a model of a road bridge spanning 100 m. See J-4. He did some tests removing some hangers and concluded: "Only when a very high number of hangers were missing or when the load was extremely big, would we get buckling in the arch and collapse of the whole structure."
The very slender Bolstadstraumen Bridge (p. H-4) has a rise of 18% of the span. Two hangers going down from the top of the bridge were removed while the author was on top of the bridge measuring deflection in the arch. He concluded that the maximum stress in the arch <u>might</u> have reached yield.

The hangers of around 60 Nielsen arch bridges built in Sweden were steel rods. See (Tveit 2013 p. 55) and (Ostenfeld 1976 p. 124). They were meant to relax due to one-sided loads. Nielsen 1929, 1932 and 1936. The author once asked a very experienced engineer at the bridge office of the public roads in Sweden: "How do you repair hangers in the Nielsen bridges if they break?" He seemed surprised by the question and said that he had never heard of the hangers in the Nielsen bridges breaking. This is very interesting, because to the best of the author's knowledge many of the Nielsen bridges had no railings between traffic and hangers.

When he was is Sweden in the early sixties the author heard of a hanger in a Nielsen bridge breaking. He does not know if this was due to fatigue or other reasons. If a steel rod hanger in a network arch of the Åkvik Sound type broke, the author would have elongated the hanger by thermostatically controlled heating in a metal casing around part of the hanger. Then the hanger would have been welded in such a way that the original length was maintained.

(Bast 2008) is about hangers in arch bridges. Page 14 says that the vibration tendency in the hangers of network arches is reduced when the hangers are tied to each other where they cross. In the Steinkjer and the Bolstadstraumen network arches, split open plastic tubes were attached to one of the hangers where they crossed each other. See fig. D4. This arrangement has worked well for over fifty years. If the hangers can move away from each other, they can be tied to each other with plastic bands.

(Zoli and Woodward (2005)) have written an interesting article on cable loss in long span bridges. Their practical example was the Blennerhassett Bridge. See pp. H-23 to H-25. Their results are not frightening, especially when the hanger failures are gradual. That will be the case when the rupture is in cables and not in rods or in the ends of cables.

E. On erection

When designing network arches, it is also necessary to suggest at least one method of erection. This chapter suggests various methods of erection. Other interesting methods for erecting network arches are suggested for the Mangamahu Bridge H-8, the Waikato River Bridge H-11, the Bechyne Bridge (Šašek 2005 and 2006), the Blenderhasset Bridge (Wollmann et al. 2007), the Providence Bridge H-5 (Steere et al. 2008) and the pedestrian bridge in Happy Hollow Park and Zoo in San Jose, USA, H-26.

ERECTION ON A SCAFFOLD

Erection on a scaffold is the most straightforward way of erecting a network arch with a concrete tie. It can be used where temporary scaffolding under a bridge can be tolerated for a limited period to be removed when the bridge is finished. If thick ice and soft soils make the pillars extra costly, that speaks against short spans in rivers. For narrow bridges that cross over roads a scaffold might be tolerated when the network arch is built. This might also be the case when a network arch is built over a road that is going to be made wider.

In Peru some rivers carry no water about half the year. The dry sand in the river bed is sometimes shifted around by strong currents. Thus the durable foundation for the pillars will be costly. This speaks for relatively long spans. Extensive scaffolding can be tolerated in the dry season.

The scaffolding used for the Bolstadstraumen network arch in Norway is shown fig. E1. In the longitudinal direction the piles are ~3.25 m apart. There the hangers were tensioned maintaining the right shape of the arch. When the hangers carried the tie, the scaffold was removed. See also fig. J8. On the adjustment of hangers in network arches (8 pages) see the author's homepage: http://home.uia.no/pert/ under the button "Supplementary Information". A simpler method was used for the Brandanger Bridge.

Fig. E1. Scaffold for the tie of the Bolstadstraumen network arch
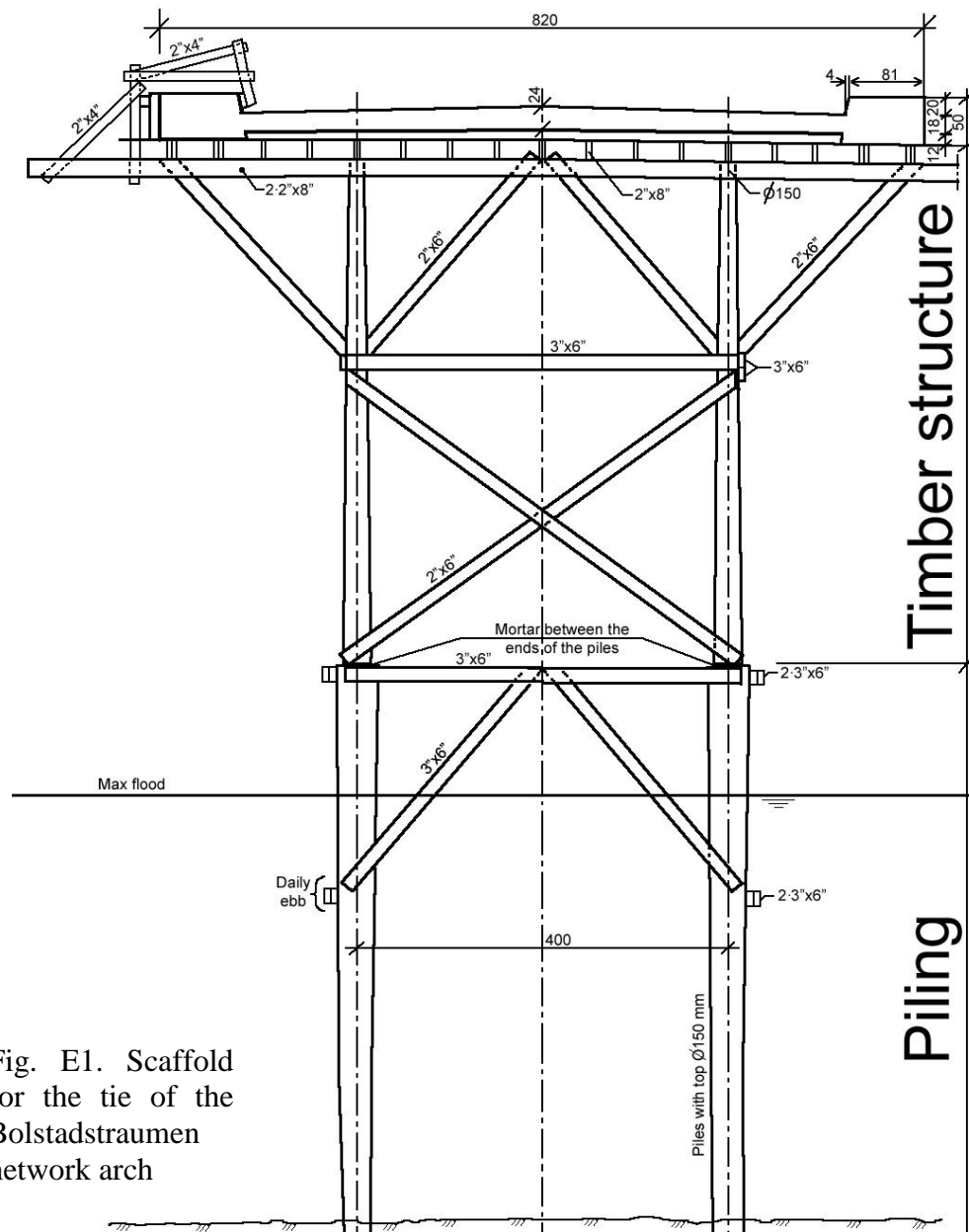
CASTING THE CONCRETE TIE ON A TEMPORARY LOWER CHORD

Arch, hangers and a temporary lower chord can make a stiff steel skeleton that can be moved when lifted near the ends or by cables pointing towards the ends of the arches. See fig. E10. It can be moved by various cranes and/or pontoons. In arctic regions it can be erected on the ice on the rivers. See p. E-8.

Local conditions will decide how much formwork and reinforcement should be put in before the steel skeleton is moved to its final position. When the steel skeleton is in place, the concrete is first cast at the ends of the tie and around the curved parts of the prestressing cables. After that a slight prestress can reduce the stress in the longitudinal beams in the lower chord. Then the edge beams are cast.

Usually the casting must be done from both sides to avoid relaxations of hangers that can lead to big deflections. The load that leads to limits of deformations are indicated by (Tveit 2013 fig. 37) and the accompanying text. After the edge beams have been cast, they take most of the longitudinal bending in the tie. Then the concrete slab is cast. During the casting the prestressing cables must gradually be stressed to reduce the tension in the longitudinal beams in the temporary tie.

Unlike a permanent lower chord the temporary lower chord needs no corrosion protection. It can be produced on site using high strength bolts. In the author's opinion there are sites where the full potential of the narrow network arch can not be utilised without using a temporary lower chord.

Fig. E3 shows a cross-section of a wagon for removing the formwork (Tveit 2013 p. 52 to 53a). It rolls along the edge of the finished tie.

The floor in the middle of the removal wagon has been a part of the formwork for the casting of the tie. It has two extra long transverse beams. The floor has been lowered after the casting was finished. The longitudinal beams in the platform have been parts of the longitudinal beams in the temporary lower chord.

Jacks fastened to the lower end of the hangers can move the platform.

In wider bridges the workers can stand on the platform while they prestress and inject the transverse prestressing rods or cables.

With slight alterations a temporary lower chord can be reused in many different network arches. If the width of the lower chord is not increased, only new diagonals and new holes might be needed.

Cross-section of formwork and half the permanent and temporary lower chord. (Camber of the transversal beams is not shown)

Steel in temporary lower chord seen from above

Fig. E2. A temporary lower chord

Fig. E3. Wagon for removing a temporary lower chord

In wide bridges transversal steel beams are likely to be used. Then longitudinal steel beams in the tie can simplify the erection. Fig. E-5 shows the railway bridge across the Oder-Havel Channel at Henningsdorf. (Geißler et al. 2008). The span is 97.4 m. A pontoon will be used for floating the steel skeleton across the channel. See fig. E6.

The inner and outer hangers under each arch are sloping in different directions. This gives a torsion that can easily be taken in the windbracing and in the transverse beams in the tie. There is a footpath between the hangers. The footpath must be outside the arches at the ends of the arches. The hangers are flat steel. They give high resistance to fatigue, but their vibrations are considerable.

The nodes are placed equidistantly along the arch. In the irregular variation of hangers an optimum has been sought. There are elastomer dampeners between the longer hangers to reduce wind induced vibrations. Other erections that use pontoons can be found in (Tveit 2012 pp. 15, 20, 34, 47 and 54).
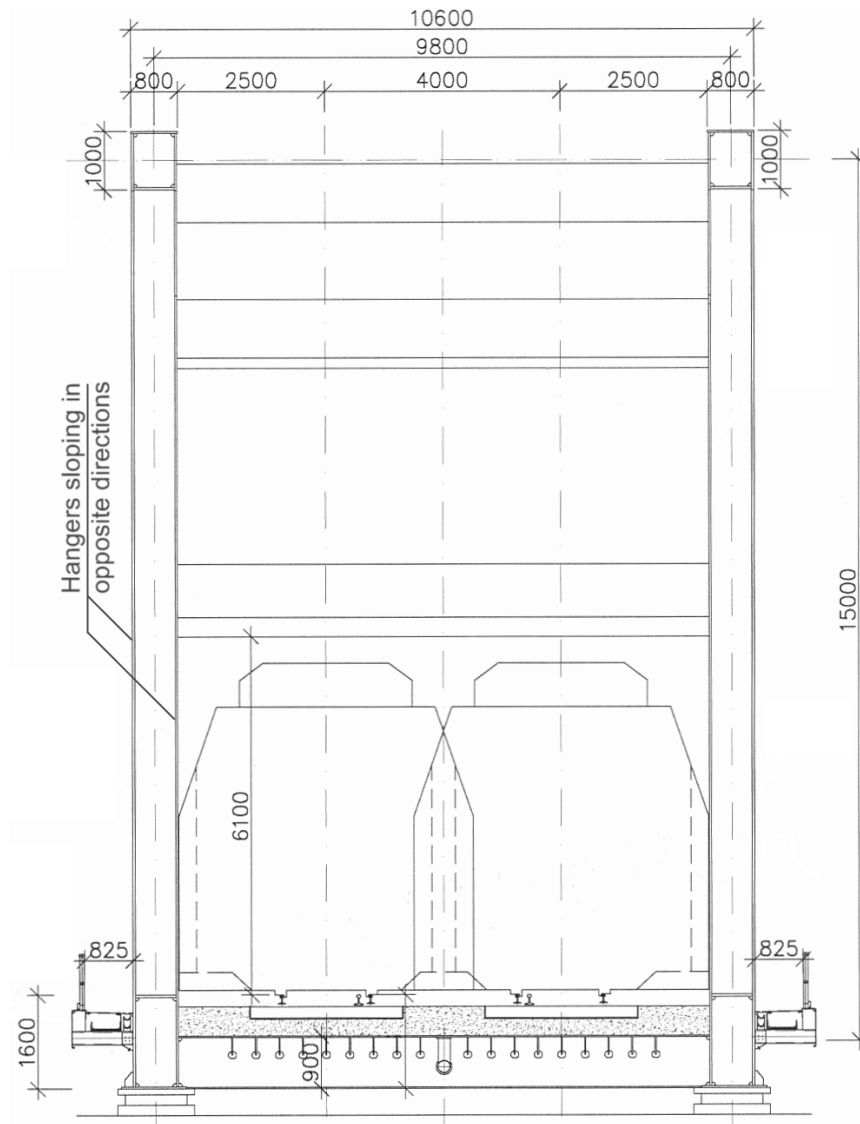
Fig. E5. Network arch railway bridge over the Oder-Havel Channel
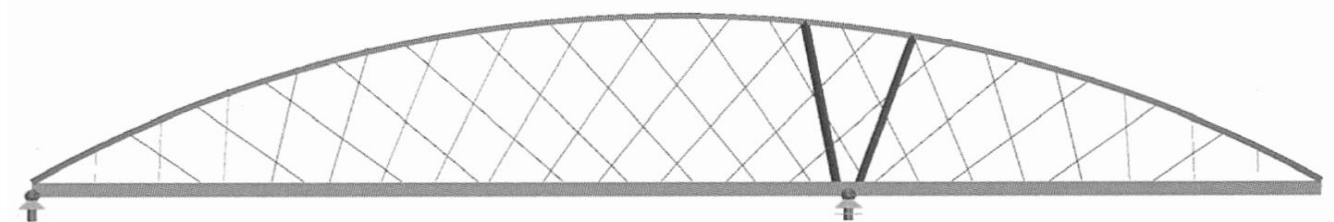
Fig. E6. V-shaped strengthening of the steel skeleton above the pontoon when it is moved over the Oder-Havel Channel

CRANES FOR ERECTION

Cranes for lifting parts of arches in place will be used in the erection of practically all network arches. Here the cranes for lifting steel skeletons or whole main spans will be examined.

MOBILE CRANES

Ladislav Šašek (2005 and 2006) suggested using a mobile crane for erecting the steel skeleton of the Bechyne Bridge built in the Czech Republic in 2005. (Tveit 2013 pp. 92a to 92c. and H-14 to H-16) See fig. E7. The span is 41 m. However, the contractor preferred another method. (Tveit 2013 pp. 92c)
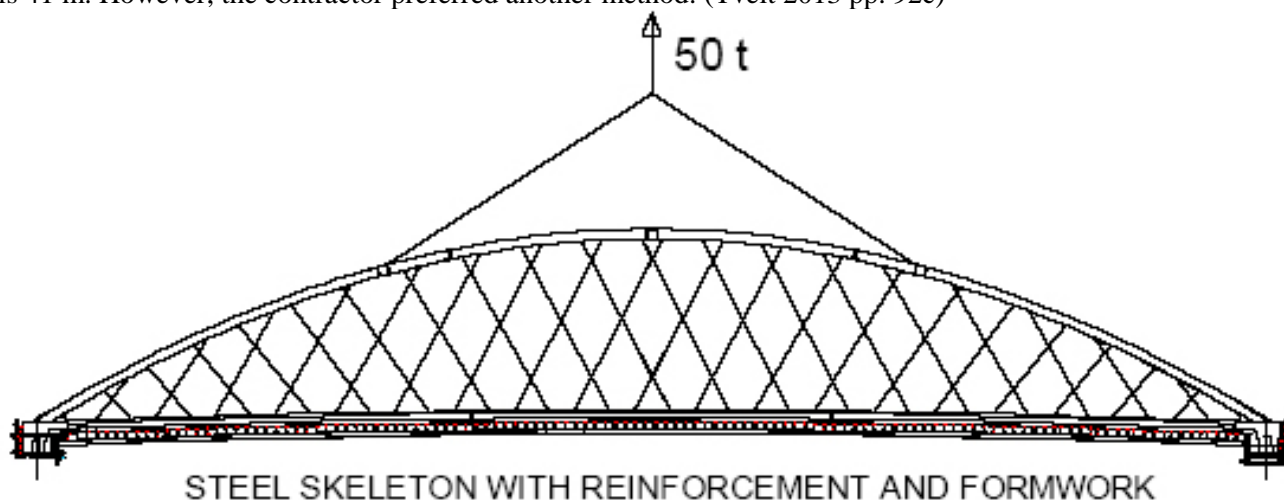
Fig. E7. Proposed method for erecting the Bechyne network arch

Fig. E8. Method for lifting the steel skeleton of the Mangamahu Bridge in place. Span 86 m. Built 2008.

Fig. E8 illustrates the erection of the Mangamahu Bridge in New Zealand. The temporary steel tie was erected on two temporary supports. See fog 3 on page H-9. Later concrete was put on top the steel tie. Two halves of the arch were erected by the two red mobile cranes. The yellow crane in Fig. E8, carried the two men that joined the arches at the top.

Afterwards the hangers were put in. The concrete was put on top of the temporary lower chord. If we define the slenderness of an arch bridge as the span divided by the combined heights of the chords, this was the world's most slender tied arch bridge when built. See H-8 to H-10.
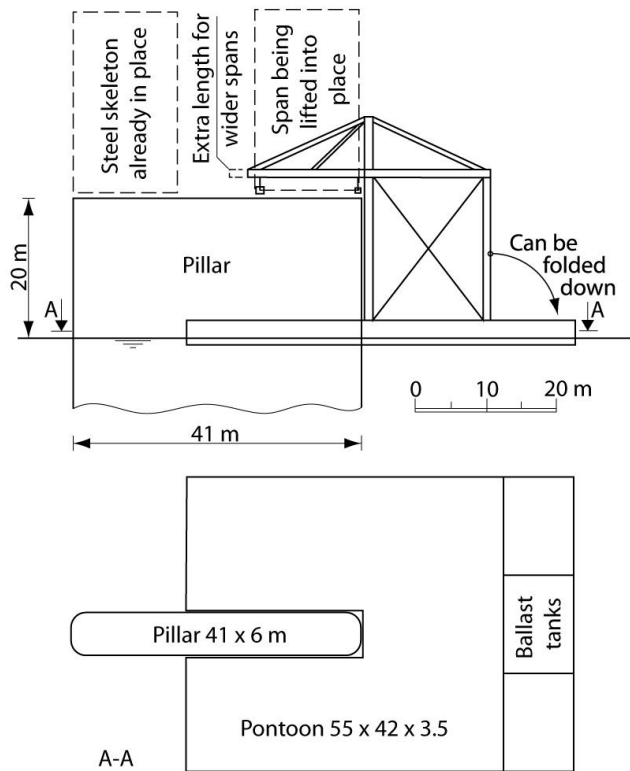
Fig. E9. Crane for erection of network arches in wide rivers

In big rivers and coastal areas network arches or steel skeletons of network arches can be lifted in place by big floating cranes. See also (Tveit 2013 p. 29k).

The crane shown in fig. E9 can be built to erect network arches in wide rivers. It can bring steel skeletons from the shore to the pillars. The crane can be folded down so that it can pass under existing bridges. It needs less than 1 m of water under the pontoon.

The crane in fig. E9 is shown erecting one half of the steel skeleton for a combined road and rail bridge with spans of 160 m. (Räck 2003). The bridge has four arches. Each of the two steel skeletons shown in fig. E9 weighs ~1000 metric tons. (Tveit 2013 p. 54).

When both steel skeletons in fig. E9 are in place, they can be joined together and the concrete tie can be cast. At the top of the pillars there must be room for prestressing the longitudinal cables.

The crane can be used for road bridges that are longer and/or wider because road bridges use less steel than railway bridges. For tall pillars, pontoons and cranes on top of the pillars would sometimes be a better idea.

In coastal areas whole spans of network arches or steel skeletons of network arches can be lifted in place by big floating cranes. See fig. E10. The crane can move in the middle of the sound and place the span on pillars on the shores.

Combined with arch and hangers, the temporary lower chord makes a stiff steel skeleton. This steel skeleton can be moved when lifted near the ends. It has enough strength and stiffness to carry the concrete tie while it is cast.

If the temporary lower chord has longitudinal beams between the ends of the arches, then transverse beams in the lower chord are placed equidistantly except near the ends of the arches.

Fig. E10. Lifting the temporary steel skeleton of the bridge in Fig. C1

Longitudinal wooden beams on top of transverse beams carry the wooden form. This gives a convenient platform for placing the reinforcement and the prestressing cables. Local conditions will decide how much formwork and reinforcement should be put in before the steel skeleton is moved to the final position. Fig. E10 shows how the steel skeleton of the Åkvik Sound Bridge in fig. C1 is lifted in place by Norway's biggest floating crane Uglen. The lifting capacity is 600t. The steel skeleton weighs ~230t, but it might be practical to put in so much wood and reinforcement that 410t is lifted.

According to Wikipedia floating cranes that can lift over 10000 metric tons are available. The limited room under the hooks decides how high over the sea level the steel skeleton can be lifted. Normally it is better to use one crane at each end of the span to be lifted.

In 2010 the 220 m finished main span of the Brandanger Bridge in Norway (Tveit 2013 p. 93c) was built on a flat area where rock for covering pipelines in the North Sea had been taken out. The main span weighed 1862 tonnes. It was lifted by two big Dutch floating cranes. See p. 35 in http://elearning-iabse.org/l20

HOW TO ERECT THE LOFTESNES NETWORK ARCH. Span 135m

An examination of possibilities of erection of a network arch.

The plan is to see if a steel skeleton for the Loftesnes Bridge can be erected on a quay to be lifted to the bridge site by floating cranes. The bridge is much like the Åkvik Sound Bridge calculated by Teich and Wendelin (T&W) in 2001. See http://home.uia.no/pert under the button "Master thesis". More on network arches can be found under the button "The Network arch" (TNA).

The bridge has a span of 135m. The width of T&W's bridge is 14.8m. The Loftesnes Bridge might be less wide and have a slightly longer distance between the planes of the arches. The Norwegian loads on the Loftesnes Bridge will be slightly smaller than the loads on T&W's bridge. In these calculations the data from T&W's bridge are used.

Calculation of the temporary lower chord of the steel skeleton.

To make the bridge lighter, 85 ^{N}/mm^2 will be the cube strength of concrete used. The cylindrical strength of the concrete will be increased from 40 to 74 ^{N}/mm^2. According to T&W p.109 the strength asks for a necessary plate thickness of ~27 cm. Because the concrete strength is increased, the thickness of the slab between the arches can be reduced to 24 cm. If the deflection of the concrete slab becomes too big, it can be counteracted by putting fibres of reinforced polymer ropes under the slab. See TNA p. 50.

The transverse beams in the steel skeleton are placed 3 m apart. The thickness of the wood in the temporary lower chord is 3 cm. Try using a steel beam HE 240A. The max load on the transverse beams is:

Concrete 0.24m x 25kN/m^3 3m	=	18.00 kN/m
Wood 0.03m x 7kN/m^3. 3m	=	0.63 kN/m
Steel beam 240 A 60.3 kg/m 9.81/1000	=	0.06 kN/m
		18.69 kN/m

Max stress in a steel beam HE 240 A:

$$M/W=((ql^2/8)/W)=((18.69 \times 9.4^2/8)/675.1)= .3058 \text{ kN/m}^2 = 305.8 \text{ }^N/\text{mm}^2 \sim 420/1.4 \text{ }^N/\text{mm}^2$$

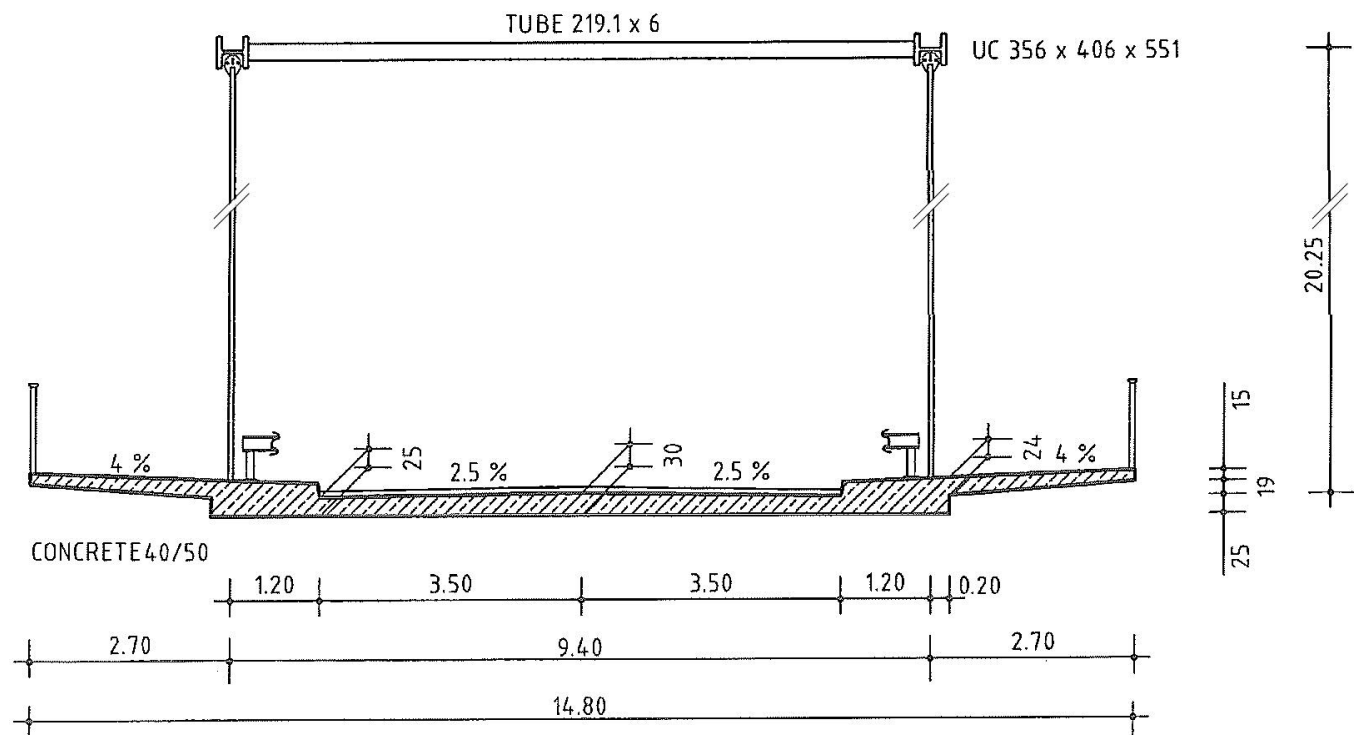This would be all right for steel S 420 M which has a yield strength of 420 ^{N}/mm^2

Fig. E11. Cross-section of T&W's network arch. Span 135 m. The Loftesnes Bridge might have only one footpath.

The longitudinal steel ties in the temporary lower chord must have a height of around 200 mm because that is the height of the longitudinal wooden beams that carry the form for the concrete. See figs 21 and 56 in Tveit 2011. Use HE 200 A which has a height of 190mm. The weight is 60.3 kg/m. If we use this profile, the temporary lower chord can carry the form and reinforcement when the steel skeleton is lifted. The temporary lower chord can be reused in spans of different length and widths.

Assume that the steel tube in the arch is Dxt = 610x12.5mm. A=23464mm^2.
Assume steel S 460 ML with a yield strength f_y=460 N/mm^2 Yield force 460 23464/ 100000=10.8 MN.
Assume that the cube strength of the concrete that is filled into the tube is 85 N/mm^2 f_{cd}=40 N/mm^2

Force in the concrete: ((0.5(610-2x 12.5)^2)x3.14x40)/1000000=10.75MN

Assume a rise of the arch: 135x0.17=23m
In Teich and Wendelin the maximum force in the arch is in the collapse limit state 17.05 MN. The rise of the arch has been increased from 0.15L to 0.17L, the concrete slab is thinner and the Norwegian loads are smaller than the German loads. Thus the load in the collapse limit state is more like 14 MN. The strength of the concrete in the tube can be increased if necessary. Therefore the chosen dimensions are about right.

Buckling perpendicular to the plane of the arch is assumed to be decisive. Buckling has been examined.

Weight of the steel skeleton, with form and reinforcement, when it is moved from the quay to the bridge site.

Wood 0.03 m^3/m^2 Width 14.8 m 710 kg/m^3 (135+2)m	43.2 t
Temporary transverse steel beams in the tie 60.3 kg/m 16m 48 beams	46.3 t
Temporary longitudinal beams in the tie 42.3 kg/m x135m x2	11.4.t
Reinforcement from Teich and Wendelin p. 84. 150 t + 10t extra because the slab is thinner	165 t
Steel in tubes in arch 184kg/m 2x135m +20% extra steel at the lower ends of the arches	59.6 t
Hangers, windbracing, bearings and other details. Teich and Wendelin p. 84.	60.0 t
Load to be lifted to the bridge site	385.5 t

Norway's biggest floating crane "Uglen" can lift 600 t up to 60m above sea level. See (Tveit 2013 fig. 39). We can also use one crane at each end of the steel skeleton. If we do not install most of the formwork and the reinforcement before the steel skeleton is lifted, a crane that can lift 190 t can be used. Two cranes that can lift 100 t each can also be used. Several floating cranes in Norway can lift that much.

When the steel skeleton is in place the concrete is pumped into the steel tubes in the arches. The pumping can be done from the bottom of the steel tubes. When the concrete is sufficiently hardened, the edge beams are cast. To avoid the relaxation of hangers the edge beams might have to be cast from both ends. The concrete slab under the lanes is cast last. More on the casting of the tie can be found at (Tveit 2013 p. 12).
Materials needed for the main span of the Loftesnes Bridge:

Permanent structural steel 120 t.
Concrete 1300 t.
Reinforcement 165 t.
Formwork 2200 m^2.
Temporary lower chord 100 t.
Information on removing the temporary tie can be found at Tveit 2013 pp. 52 to 53a.
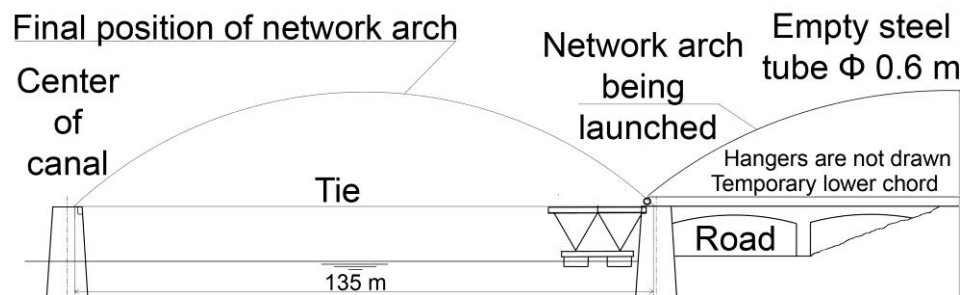
Fig. E12. Erection of a network arch over a canal in China

Conclusion: The main span of the Loftesnes Bridge could be a network arch with steel tubes in the arches. Span 135 m. A steel skeleton consisting of arches, hangers and a temporary lower chord can be assembled on a quay. It can be lifted to the bridge site before the steel tube is filled with concrete. Then the concrete tie is cast. The temporary lower chord can be reused in network arches of different lengths and widths. The bridge needs very little materials.

In Russia, Canada and northern Scandinavia the steel skeleton of network arches can be erected on ice in the winter and be lifted onto the pillars. (Tveit 2013 p. 30b). This method of erection could be competitive and would contribute to reducing winter unemployment.

To make the span look good, the lower chord should have an upward camber of at least 1% of the span. The surface of the ice is flat. The creep, shrinkage and elastic compression in the tie will give the span an upward camber. The compression in the arch works the other way. To achieve a suitable camber in the lower chord, blocks of wood of varying heights can be put on the ice under the transverse beams in the temporary lower chord.

For this type of erection preventing water from seeping onto the surface of the ice near the steel skeleton is desirable. If this is achieved, the strength of the ice would be ample. 600 to 800 mm might be sufficient for carrying the 1.2 t per metre that the steel skeleton for the Åkvik Sound (See fig. C1) would weigh for the two weeks that the erection of the steel skeleton would take. The Alberta Occupational Health and Safety gives this general guidance: For clear blue ice 690 mm thick, the permissible point load for working on river ice is 8 t.

Sufficient thickness of the ice can be produced on cold days by pumping water onto the ice. Spraying water in the air above the ice can accelerate the process. In Arctic areas this might not be necessary since it is usually easy to achieve one metre thick ice. Reinforcement of ice with wood is treated by (Cederwall and Fransson 1979). If such reinforcement is used to reduce the creep in the ice, it could be placed on 100 to 150 mm thick ice before water is pumped onto the ice. (Tveit 2013 p. 30b).

The snow must be removed from the ice near the bridge site in order to avoid layers of snow on the ice. Such intermediate layers of snow would reduce the strength of the ice cover. Another reason for removing the snow is that the insulating effect of the snow could make the ice melt from below. This might be important if very thick ice has not been achieved.

When there is only a slight movement in the water, it is enough to put the snow in longish heaps in the vicinity of the bridge. If there is a slight unidirectional current at the bridge site, only the snow on the upstream side of the bridge should be put together in longish heaps.

When the ice is thick enough, it will be an almost ideal platform for erecting the steel skeleton. The steel skeleton consists of the arches and the hangers plus a temporary lower chord. See fig. E2. The skeleton can be put up with the help of steel scaffolding. A mobile crane can be used for erecting the steel skeleton.

Stepwise building of bridges using network arches

The author likes network arches with concrete ties. Network arches are most competitive for spans that are between 70 and 170 m. Such spans are usually not built unless the traffic needs quite wide bridges. This speaks against concrete slabs in the tie. Stepwise building should be considered.

Where there is little traffic and more traffic can be expected in the future, we can build wide pillars at once. For the present traffic two or three lane concrete slabs can be sufficient. When more lanes are needed, another string of spans can be added. This is very relevant in countries like China, India and Russia where there are big rivers and a significant increase in traffic can be expected.

It might be an extra good alternative to build wide pillars from the start where the dimensions of the pillars are greatly influenced by horizontal load such as collision with ships, current and ice. The second string of spans can be designed to carry bigger loads than the first string of spans. Later the spans in the first string can be replaced by wider spans designed to carry heavier traffic.

F. On cost of bridges. This chapter can also be found on pp. 93a – 93c in (Tveit 2013)

In his work with network arches the author has presented influence lines and quantities to make it easy for fellow engineers to check his claims concerning savings of materials. Optimal network arches have little welding and simple details that repeat themselves many times. The cost per tonne will be fairly low if efficient methods of erection can be found. The author has been reluctant to specify savings in US dollars or sterling because such savings are much more difficult to defend.

It deserves a mention that two network arches in the USA were built because they were less costly than the alternatives first suggested. One was the Blennerhassett Bridge (Wollmann and Zoli 2008). See also page H-23 to H-25. Another was a pedestrian bridge in Happy Hollow Park and Zoo in San Jose in California H-26 to H-28. There two equal network arches replaced a cable stayed bridge and over 50% of the cost of the superstructure was saved.

The reduction in cost resulting from the use of network arch bridges is of great interest. Therefore a network arch with a span of 150 m will be compared to an arch bridge with vertical hangers spanning 100 m built over the river Saale near Calbe in Germany, (Fiedler and Ziemann 1997). A comparison of steel weight of various steel bridges has been made by Max. Herzog (1975) and by Tveit (1999). Fig. F1 shows that arch bridges do not tend to use more steel than other steel bridges.

At similar sites network arches should normally have longer spans than other bridge types. This is because the steel weight of the network arch is smaller and it increases more slowly with increasing spans. This is an extra advantage if the size of the pillar depends mainly on the forces due to collision with ships or forces from breaking of ice in the spring.

The data for the network arch are based on the network arch designed by (Teich and Wendelin 2001). See fig. C1 on p. C-1. A slightly revised version of their work can be found on the author's home page, at http://home.uia.no/pert/ under the button "Masters Theses".

Fig. F1. Steel weights of various bridges
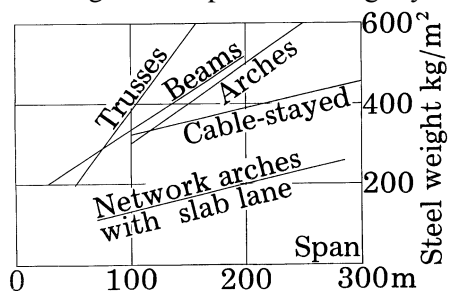
The cost per m^2 of bridge between the railings is compared. The average width between the railings is 13.9 m for the Calbe Bridge and 14.8 m for the network arch. Both bridges are assumed to have many equal spans.

It is shown that the network arch with a span of 150 m will need about the same size of supports as the 100 m arch bridge with vertical hangers. The loads and codes of the EU are used for both spans. Factors that influence the cost of both spans are presented. (Tveit 2013 pp. 93a-93c).

Material and cost of the Calbe Bridge compared to that of a network arch
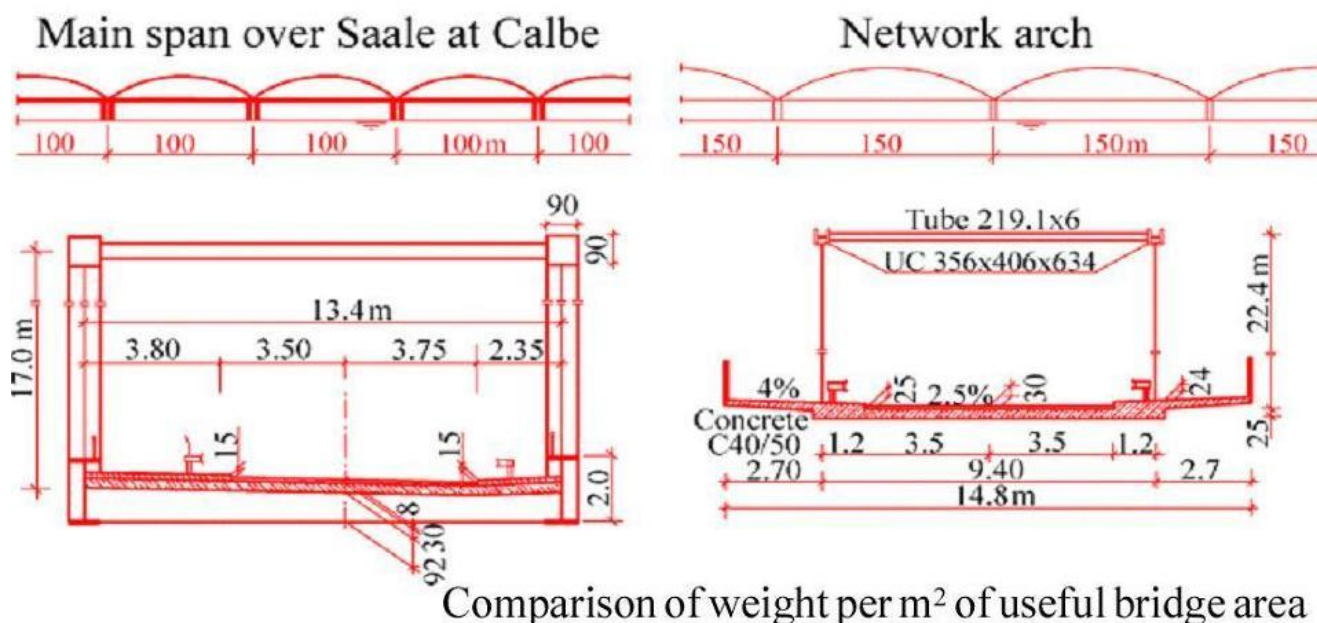
60% of the structural steel is saved.   34% of the reinforcement is saved.   24% of the concrete is saved.
The pillars are the same for both bridges.   Less weight needs to be moved during erection.
The saving on cost is probably 35 to 45% per m^2 of useful bridge area.

Fig. F2. Comparison between the Calbe Bridge and a network arch spanning 150 m

Data from the comparison

Permanent load per span: Calbe (100m)   Network arch (150m)

	Calbe (100m)		Network arch (150m)
Structural steel	530 t	$255.1 (150/135)^2=$	315 t
Railings 200kg/m	20 t		30 t
Reinforcement	151 t	$126.2 (150/135) =$	140 t
Concrete	1463 t	$1358 (150/135) =$	1509 t
Asphalt, etc. 80mm	136 t		197 t
	2300 t		2191 t

Live load on a support:

Calbe, area: $1390 \text{ m}^2 : ((9.0-2.5) \cdot 3 \cdot 100 + 1390 \cdot 2.5)0.981/10 = 532$ t
Network arch: $2205 \text{ m}^2 : ((9.0-2.5) \cdot 3 \cdot 100 + 2205 \cdot 2.5)0.981/10 = 828$ t

The load on a support due to concentrated live load is about the same for both bridges.

The live load on each support is added to the permanent load on the support after it has been multiplied by the relevant partial safety factors γ_Q / γ_G:

Calbe: $2300+532(1.5/1.35)=2891$ t   Network arch: $2191+828(1.5/1.35)=3111$ t

Area exposed to wind:

	Arches and tie	Hangers	Railings	Traffic	
Calbe:	$(0.9 \cdot 2 + 2)100$	$0.12 \cdot 207$[m]	$1 \cdot 100$	$2 \cdot 100$	Σ 701 m²
Network arch:	$(0.424 \cdot 2 + 0.6)150$	$0.06 \cdot 1528$[m]	$1 \cdot 150$	$2 \cdot 150$	Σ 759 m²

The vertical load on each pillar is about 7% smaller for the Calbe Bridge. The area exposed to wind on each pillar is approximately 8% smaller for the Calbe Bridge. Since the span of the Calbe Bridge is 33% smaller, the saving in the pillars when using the network arch is likely to be between 25% and 32%.

Comparison of the superstructure of the Calbe Bridge with a span of 100 m and a useful area of 1390 m² to a network arch with a span of 150 m and a useful area of 2205 m².

	Calbe	Network arch	Reduction per m² of useful bridge area
Structural steel	530 t	315 t	60%
Reinforcement bars	151 t	140 t	42%
Concrete	1463 t	1509 t	35%
Weight of steel skeleton during erection	530 t	~400 t	24%

All comparisons will be lopsided. The additional facts in fig. F3 should be taken into consideration. The network arch makes better use of high strength steels. The yield strength of the steel in the Calbe Bridge is 345 MPa compared with 430 MPa in the network arch. See fig. C1 and fig. F3 for comparisons between network arches and arch bridges with vertical hangers.

The rise of the arch is 17% of the span in the Calbe Bridge and only 15% of the span in the network arch. In the network arch, the arch and the hangers protrude from the bridge area making the bridge area less useful. This is partly compensated for by widening the network arches up to 1.2 m at the end of the span. This widening is not included in the useful bridge area mentioned above.

Points of Importance	Other steel arch road bridges compared to optimal network arches
Aesthetics	Bulkier bridges
Adaptability	2 to 8 times deeper lower chords
Materials	2 to 4 times the steel weight
	15 to 30 times longer welds
Fabrication	More complicated details
Corrosion protection	3 to 7 times more surface to protect
Maintenance	Other concrete parts need much more maintenance than concrete slabs with a slight prestress.
Erection • On sidespans • Floating into place • Erection on ice	Erection is more expensive with 2 to 4 times more steel.

The author thinks that using the network arch can save between 40% and 50% of the cost of the superstructure. The author also thinks that using the network arch instead of the arch with vertical hangers can save between 35% and 45% of the cost per m².

If anybody makes a careful comparison of the cost of an optimal network arch bridge spanning more than 100 m with other types of bridges, the author would like to know the results.

Fig. F3. Comparison of optimal network arches
      to arch bridges with vertical hangers

FA. Earthquake

The network arch has good resistance to earthquakes because it has a high strength to weight ratio. It is therefore ideally suited for earthquake prone regions. Since most of the reinforcement is in the edge beams, the sideways strength in the tie is good. Good stoppers must be used to prevent the ends of the network arch from sliding sideways.

Professor Dr. Semih Tezcan of Boğaziçi University in Istanbul has kindly given general recommendations on the design of network arches where strong earthquakes are likely to occur. These recommendations were given to Eduardo Marianyi in connection with the design of a network arch over the Carbon River in Peru. Now the recommendations can be found at the author's home page **http://home.uia.no/pert/** under the button "Supplementary Information".

FB. Facts that speaks against the building of network arches

The bridge type is new and people who make decisions are often reluctant to use new bridge types unknown to them. This might be very important in India where the opinion of senior personnel carries more weight than in Europe and the USA.

Spans of network arches are straight and are not able to accommodate roads with horizontal curvature.

If the design of a network arch is delayed, the design office gets the blame. They are less likely to get blamed if the bridge becomes 20% more costly than necessary. In USA, a design might not be used if the cost is much above budget. In Germany an alternative design is likely to be built if it cost less and looks good. Many network arches has been accepted in these countries.

Not many engineers can be trusted with a design firm's first network arch and these engineers have lots of intriguing problems to attend to. Steel firms have moderate interest in bridges that uses little steel. The concrete firms would like to see more concrete. General conservatism might prevent network arches from being used at suitable sites.

FC. Network arches compared to continuous beams built from both sides of pillars

It is interesting to compare network arches to continuous beam bridges. They both have to carry the same loads. In the continuous beam however, the bending moments have their maximum values near the supports. Here the beams can have extra height and strength. In the middle of the spans the bending moments are smaller and the height and strength of the cross-section is smaller. Thus the beam can have a curved underside. This looks good.

In the network arches the bending moments at the supports are zero. Thus the biggest bending moments are near the middle of the span. Here the height of the arch is maximum.

Cost of the continuous beams is moderate if the span can be built from both sides of the pillars. So is the cost of erection of a network arch if a light steel skeleton can be lifted in place. The bending moment diagrams illustrate the situation:
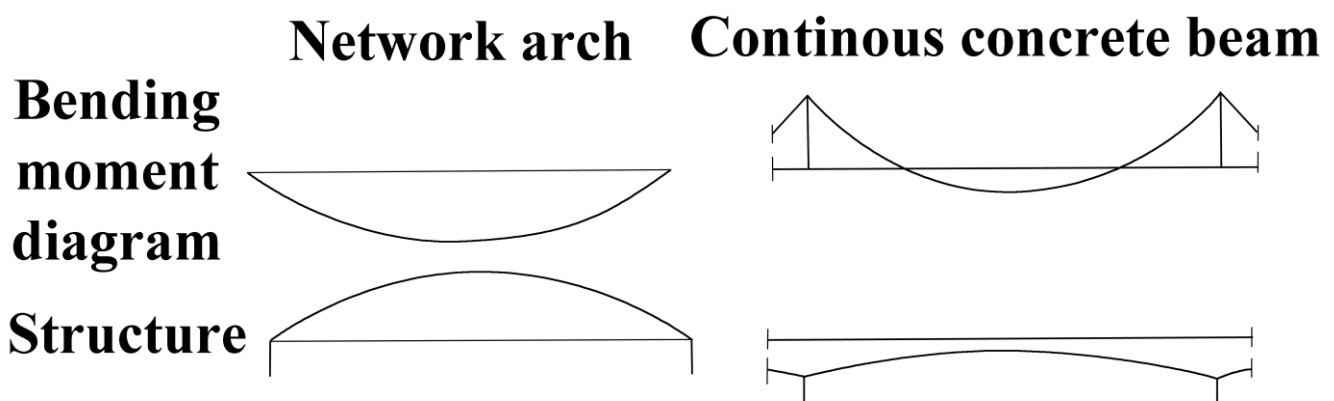
It seems to the author that the network arch will be lighter and the axial forces in the beam bridge will be bigger, but the beam bridge has no structural steel. The continuous concrete beam is less competitive when there is one main span and two much shorter side-spans like in the Brandanger Bridge in fig. J12a.

G. Strengthening of network arches already built

It is never easy to strengthen an existing bridge. The network arch is no exception, but in some cases it can be done. If the lower chord is a concrete slab, it can be strengthened by transverse tension members under the slab. This speaks for using the concrete slab between the arches. If the deflections of the slender slabs become too big, this can be counteracted by tensioning the transverse fibre reinforced polymer treads under the tie. See fig. G1.

Fig. G1. Schematic design of a device glued to both sides of the tie.

The tension members can be fastened to anchors glued to the lower outside corner of the slab. (See fig. G1). They can be stressed through tensioning and by later putting a kind of wedge between the 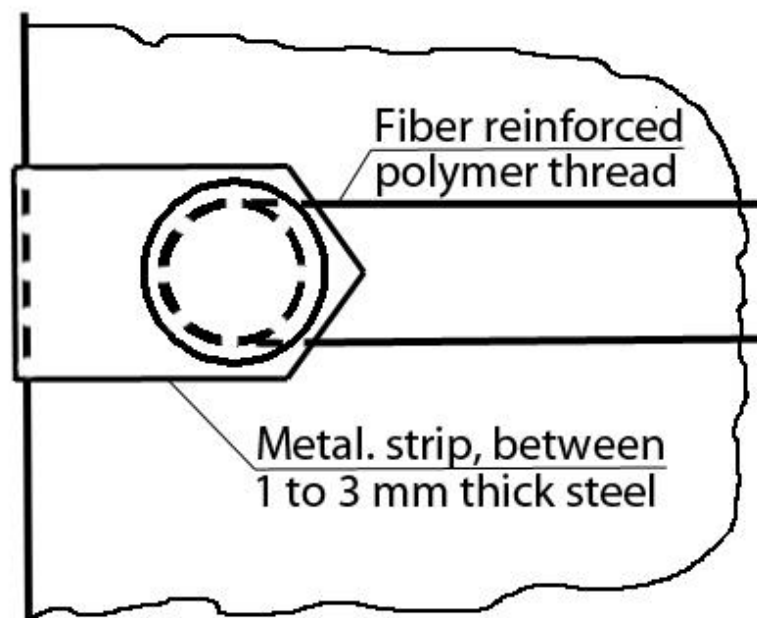slab and the tension member. The wedges should make the tension in the transverse rope counteract some of the deflection in the slab due to dead load. The transverse tension members can strengthen the slab so that bigger vehicles can pass over the bridge.

If the transverse fibre reinforced strands have not been used to strengthen the bridge, one does not have to worry much about the strength of the strands. If a few of them break it does not matter much. The fibre reinforced strands under the bridge will not be very noticeable because the sun will never shine on them. Builders should hope that the deflection of the tie does not become too big. Then the strengthening in fig. G1 will not be necessary. The transverse ropes can be shortened by rolling them around the wheels in fig. G1. The transverse ropes can be tensioned from above and below by a device not shown.

Normally it does not pay to use transversal prestress in the tie of a network arch. That is because the prestressing members cannot be placed so deep in the cross-section that the prestressing force will reduce the transversal compression in the top of the tie. In two track railway bridges the tie is deeper. Here the prestressing force might give suitable tension in the top of the tie.

Since we can counteract unwanted deflection, we can use thinner concrete slabs between the arches. Normally we would hope that the deflection will not become too big.

If the arch is a universal column or a wide flange beam, it can be strengthened by welding a steel plate on top of the arch. Then the arch can take a bigger axial force. The plate will increase the bending capacity and the torsion stiffness of the arch. The longitudinal bending capacity in the lower chord will also increase, but not much. The cavity under the plate can be filled with concrete. This is more likely to be done near the ends of the arches if the arch ends in concrete.

In the longitudinal direction an outside tensile member can strengthen the lower chord. If such a member has not been taken into account when designing the bridge, it will be difficult to fasten the tensile member at the ends of the span.

MORE ON COUNTERACTING TO BIG DEFLECTION OF THIN PLATES BETWEEN ARCHES

The lower cord of a network arch can be a concrete plate spanning 15 m or more between the arches. In the unlikely case of this plate getting too much deflection, ropes with carbon fibres under the plate can give the plate a suitable upwards movement. The fibres can be fastened to the lower ends of the hangers where the longitudinal beams of the temporary lower chord have been fastened during the erection.

These fibres can be tensioned in many ways. The upwards movement op the plate can be increased by putting stools between the concrete plates and the fibres under them. The fibres can be designed to taka some of the bending, but it is probably more economical that the fibres only counteract too big deflections that we do not want..

Steel or concrete in the lower chord of network arches?

The author has advocated concrete in the tie of network arches since 1950, but more often steel is being used. Here the author tries to state impartially the pros and cons of the two alternatives. Engineers that do not agree with my explanations are invited to contact him by email.

Some countries have the tradition the and expertise of using steel. Some countries, like Norway, use more concrete. The condeep oil platforms along with good sand and gravel have contributed to this.
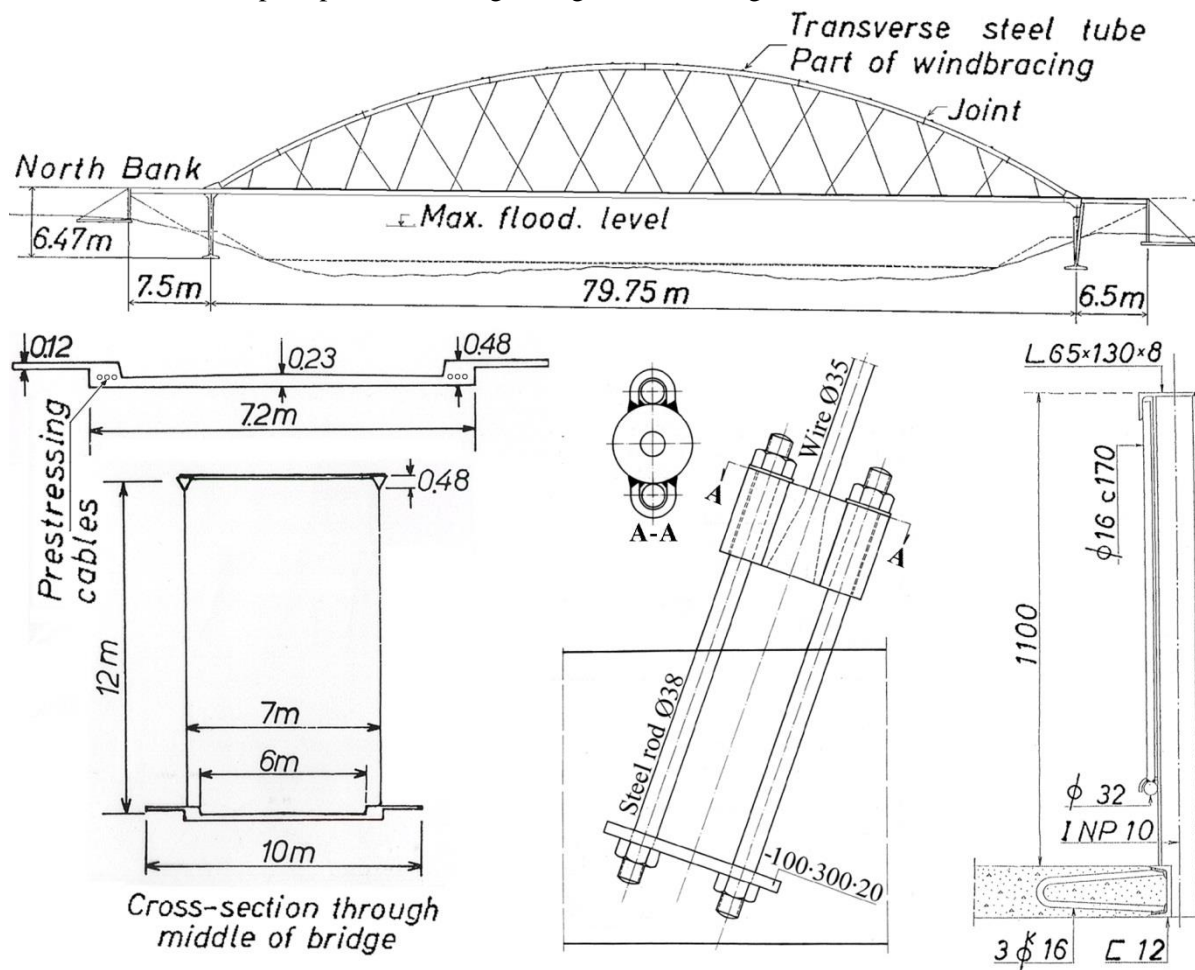
The first Norwegian network arch, built 1963. See (Tveit 2013) pp. 5b to 6c

Looks and design

The longitudinal bending in the tie is very small in network arches. The biggest transversal bending in the tie is usually half way between the arches. Thus we do not need high concrete edge beams in these ties. This leads to little distance between traffic under and over the tie. It also leads to shorter sidespans. The concrete edge beams must be able to take up the forces from the hangers and have room for the prestressing cables between the ends of the arches.

When there is steel in the tie the tensile forces between the ends of the arches can be taken in steel edge beams and in the steel plate between them. – The Americans in particular worry a lot about transversal cracks in the tie.

In arch bridges with vertical hangers a great deal of bending must be taken in the chords. This speaks for greater heights in the chords. In this type of bridge it is usual to use the same height of the beams i in the side spans and in the tie of the main span. See page H-23 to H-25.

In concrete network arches the side spans should normally have bigger height than the concrete ties. In steel ties transversal beams are used. This concentrate the load on the edge beam leading to bigger bending moments in the whole structure.

Durability and maintenance

Steel needs corrosion protection. Partially or fully prestressed concrete plates are very durable. Thus of the steel tie is the maintence likely to cost more.

Personnel for design and on-site control

Design is simpler if the network arch is an all steel structure. A wider range of knowledge is needed if the tie has various levels of prestress. For conrete ties more time should be used for control on site.

Erection

Erection of the steel tie is normally simpler than the casting of the concrete tie.

Conclusion

It is difficult to say whether steel or concrete should be used in the tie on network arches. – This differs from country to country and depends on what personnel are available in the design firm. Network arches with small spans are more likely to have concrete ties.

H. Network arches built or planned. This list will be extended.

Bridges built: Small print for unfinished contributions. Small print for authors are wishful thinking.

Bridges that are planned or being built:

Argentina

This page is not complete. It will be continually updated. Some of the authors that are indicated in small print have not been asked yet. Each bridge gets two or three pages. Normally the pages will contain: Name of the bridge. Name of the author. Where and when has been built. When it is opened or finished. Who in what firm designed it and which firm built it. Those who have written about a network arch bridge will be mentioned as co-authors on page A-1. The normal form of the bridge descriptions is indicated by the descriptions of the Providence River Bridge. The Steinkjer and the Bolstadstraumen network arches on pages H-2 and H-4 are extensively described elsewhere in the text.

THE NETWORK ARCH AT STEINKJER.
Steinkjer is a little town ~100 km north-east of Trondheim in Norway. Norway's first network arch was opened there in June 1963. It was ≈ designed by Per Tveit and built by K. Ellingsens mekaniske verksted (steel), and A/S Olav Sande (concrete and foundations). Fig. H1 to fig. H4 show the design. (Tveit 1964, 1966, 2007 and 2013)

Fig. H1. An overview with structural details of the network arch at Steinkjer
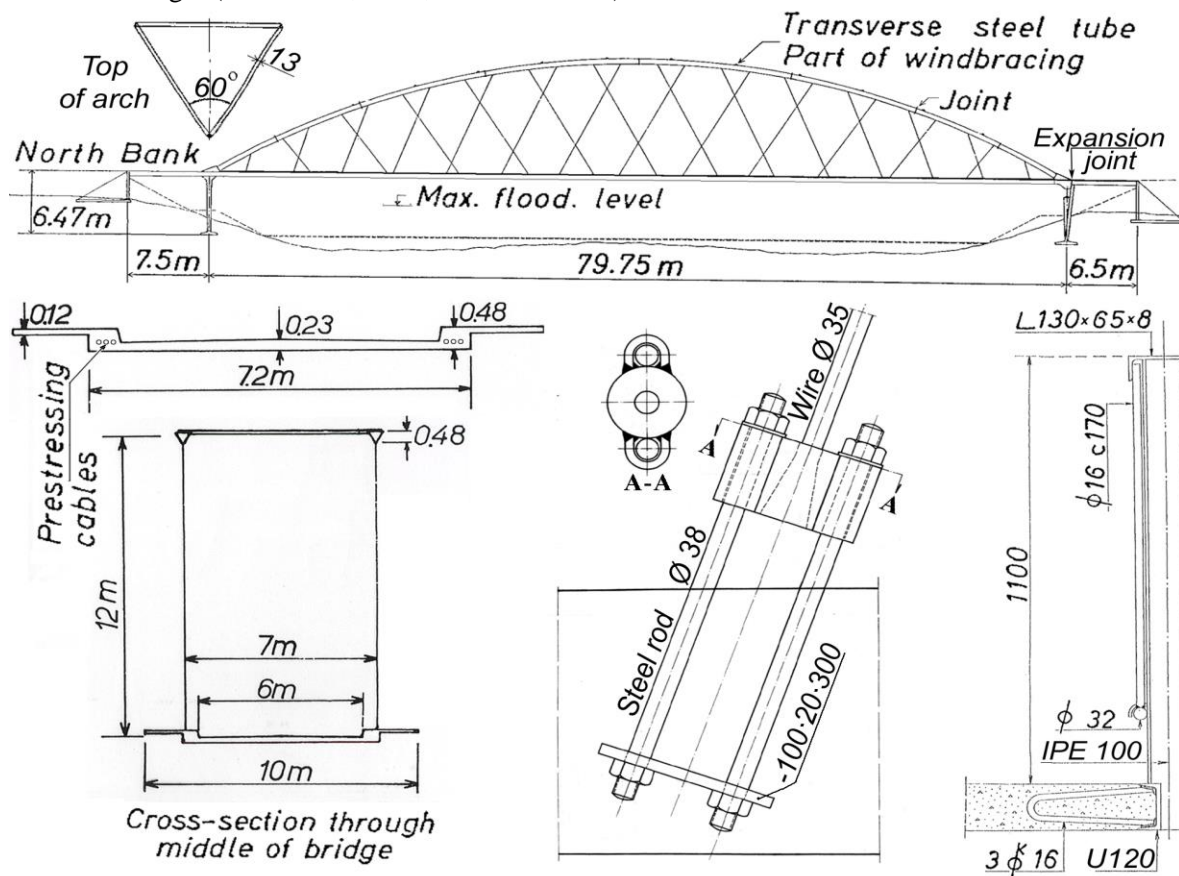
Fig. H2. Details in the arch at Steinkjer
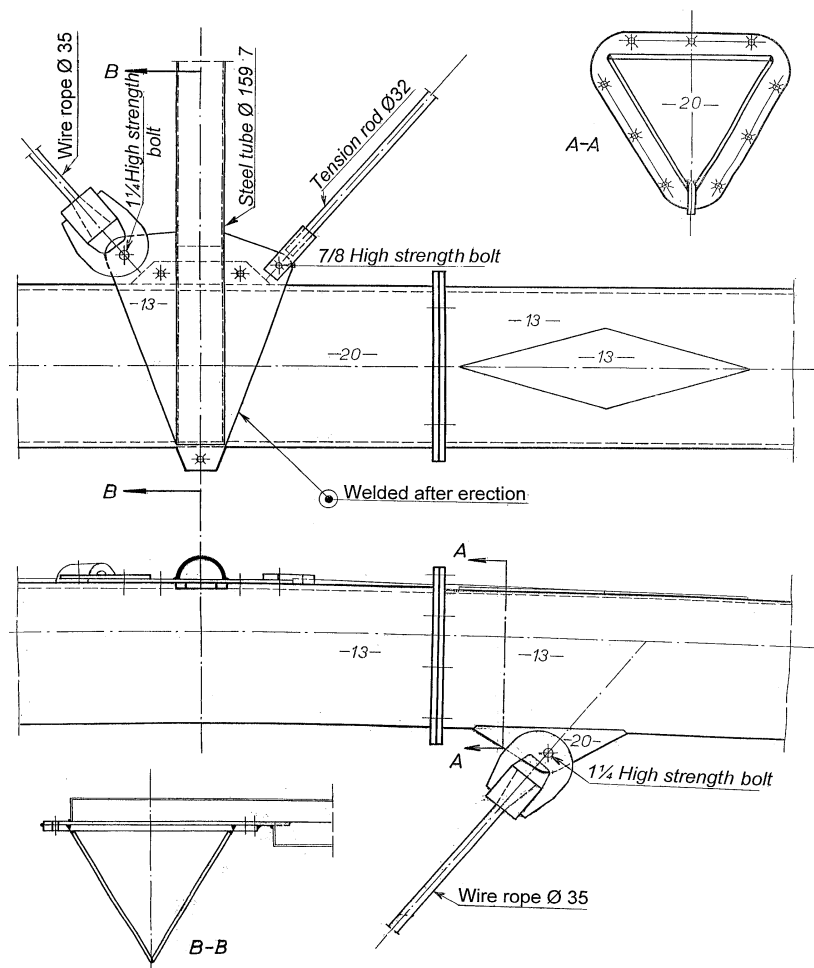
The tie on the scaffold on fig. H4 was cast first. Then the arch was erected and the hangers were put in. Then the hangers were tightened till they carried the lane and the scaffolding could be removed.

Allowable stresses without load factors were used in the calculation of the bridge. Comparison with modern loads and codes is complicated. The network arch at Steinkjer is built for a knife load of 177 kN in each of the two lanes plus an evenly distributed load that varies with the loaded length. For a loaded length of 9 m this load is 41 kN/m in each lane. The simultaneous snow load is 0.918 kN/m^2 over the whole bridge.

In the main span the materials needed per m^2 of area between the railings were: Concrete 0.22 m^3, structural steel 60 kg, reinforcement 40 kg, prestressing steel 7 kg.

$f/l \approx 0.15$

It was the author's luck that Terje Moe, a very able young architect, advised him when he designed the Steinkjer Bridge. He said: "Let your design show the flow of forces in the bridge". Terje Moe was strongly opposed to using universal columns in the arches. Later he became a professor of architecture.

Fig. H3 shows the bridge before it got the final coat of paint. Terje Mo recommended that the red colour should be used for the final coat. The author chose a more conventional silver grey. However, seeing the red colour of Chinese arch bridges convinced him that Terje Moe had been right.

The Steinkjer network arch is now 50 years old, and is still in good shape. (Tveit 2013) There are no railings between the traffic and the hangers. That was a mistake. Four or five lower ends of hangers have been bent by vehicles bumping into them. The cross-section of the lower end of hangers is ample just above the concrete. The concrete does not seem to be damaged.

A vehicle has bumped into a tube just above the lane. This tube had the extra capacity to take the shock and the resulting bending. See also (Tveit 2007) and J-7 to J-8.

Fig. H3. The Steinkjer network arch before the final coat of paint
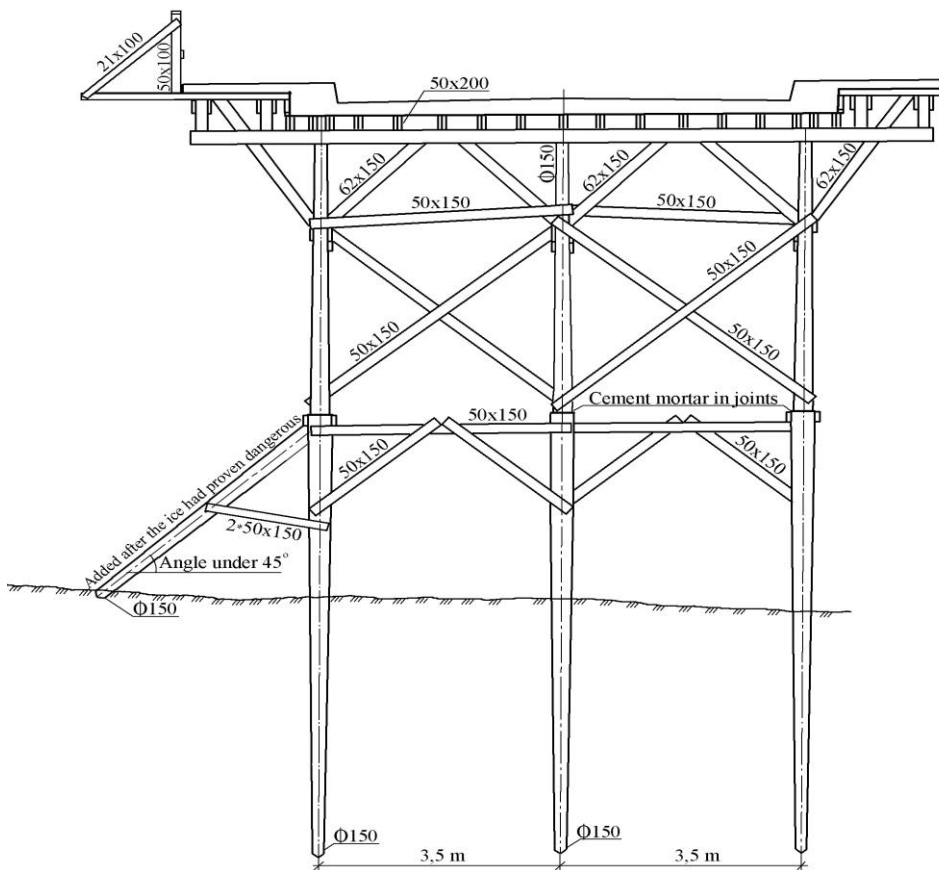
In the longitudinal direction the piles are 3.5 m apart. If there is a flood in the winter, the slanting piles will break up ice that flows against the scaffold.

Fig. H4. Scaffolding for the Steinkjer network arch

THE BOLSTADSTRAUMEN NETWORK ARCH

$$\frac{f}{l} = \frac{15.1}{84} = 0.18$$
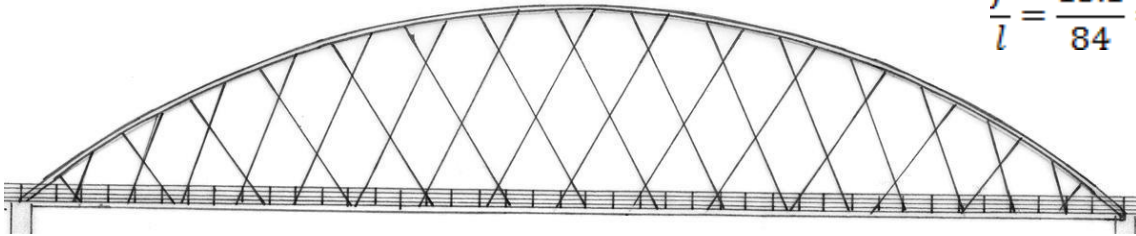
Fig. H4. The main span of the Bolstadstraumen Bridge is 84 m. It was opened at the end of 1963.

It was decided that there was to be a competition between the Norwegian bridge office of the Public Roads and the author on an arch bridge over the Bolstadstraum in western Norway. The terms were no cure, no pay. It did not worry the author much. He was fairly confident that he would win. See also J-9 and J-10 and (Tveit 1966 and 2013).

The author's network arch used 44 tonnes of structural steel and 7 tonnes of prestressed steel. The bridge office decided that the rise of the author's arch should be 18% of the span while the rise of their arch should be 21.5% of the span. Their design needed 125 tonnes of structural steel. Both bridges had a concrete slab spanning between the edge beams. The network arch was about 20% less costly.

The Bolstadstraumen Bridge influence lines are found on (Tveit 2013 p. 58). In the network arches at Steinkjer and Bolstadstraumen there was very little relaxation of hangers. This precaution was taken partly because there was no computer capacity for calculating the effect of hangers relaxing. In those days there was only one limit state in the norms. That made the task much easier.

The Bolstadstraumen Bridge was built over a tidal current in a fjord 45 km north-east of Bergen. It was opened in December 1963. It was built by Alfred Andersen (steel) and Stoltz Røtting (foundation and concrete).   The scaffolding for the main span is shown on fig. E1. Lately many houses have been built south of the bridge. Therefore the foot path has been made wider and the road has been narrowed down to one lane. See also E-1.

Fig. H5. The Bolstadstraumen Network Arch seen from the south

Design and Construction of the Providence River Bridge

PATRICIA D. STEERE, P.E., Bridge Department Manager, Maguire Group Inc

The Providence River Bridge is part of the $650-million "Iway" project being built by the Rhode Island Department of Transportation (RIDOT) with Federal Highway Administration (FHWA) supervision. The project involves the relocation and rebuilding of the intersection of I-95 and I-195 in downtown Providence, Rhode Island. In the center of the project is the Providence River Bridge with a network arch for the main span. Maguire Group Inc.[1] provided the engineering for the arch span and Cardi Corporation was the General Contractor.

The main span is a 400-foot-long (122 m) network arch bridge which was the first network arch built in the USA. The original concept for the Providence River Bridge was to create a signature bridge for the city including a 400-foot-long (122 m) arch bridge for the main span with vertical cables. Early on in the preliminary design phase, it became apparent that using inclined cables, rather than vertical cables could have a significant effect on the behavior and the weight of the structure. Preliminary two-dimensional analysis using vertical cables resulted in arch and tie depth and steel weights much greater than those required for the recently built Alsea Bay Bridge in Oregon. One of the differences in the bridges was the use of vertical cables for the Alsea Bay Bridge. Switching to inclined cables in the 2-D model of the Providence River Bridge resulted in a reduction of ten times in the deflections and a significant reduction in moments in the arch and tie.

Since he was a student, the Norwegian, Per Tveit, has done considerable research on the design of these bridges[2]. The idea has been used extensively in Japan[3]. Based on Tveit's research, it was decided to use a network arch cable arrangement to save a considerable amount of steel weight for the design of the arch span. The design of the Providence River Bridge began with a cable study to determine the optimal cable arrangement for the bridge. Initially, seven different arrangements were investigated.

Four options had varying cable angles, and three had constant cable angles with respect to the centerline of the tie. Preliminary design of the main members was performed. An estimate of the bridge cost was developed for each arrangement for dead plus live load cases where the live load was placed on varying portions of the span length. The floor beam spacing ranged from about 10 feet to 18 feet. In general, the further the floor beams were spaced, the lower the steel cost. When the floor beams were spaced at 17 feet or more, the deck thickness had to be increased. (A discussion of the cable arrangement study used for this bridge can be found in references 1)

In general, the biggest difference in results was in the bending moments at the end of the arch and tie, and the bending moments were smaller for the constant-angle cable arrangement. The bending moment stresses in all cases were small, however, compared to the axial stresses. The use of two cables per floor beam was not a cost-efficient option due to the large increase in the length of cable required. The chosen option had one cable per floor beam at a constant angle of 60 degrees from the horizontal and a floor beam spacing of 13'-9". See page H-7.

One unique feature of the Providence River Bridge is the use of three arch ribs rather than the normal two ribs. The bridge can carry five lanes of traffic in each direction and has a total width of 165 feet (50 m). Due to its large width, it was decided to use three arches and to use floor beams to span between the ties to carry the concrete deck. The center arch for this bridge carries over half the load due to the rigid connection of the floor beams to the ties. The floor beams were attached to the tie with moment connections based on the retrofit used on Wisconsin's Marquette-Jolliet Bridge in Prairie Du Chien, which connected the floor beam flanges to the tie to prevent fatigue cracking[4].

Another unique feature of the new bridge is the ten degree skew of the bridge. The preferred alignment of the bridge required that the bridge cross the river at a slight skew. The piers were aligned parallel to the river resulting in a skew to the bridge.

One of the main concerns of a tied arch bridge design is the fracture-critical nature of the tie member. At the beginning of the design process, it was decided to eliminate this problem by making the tie internally redundant. Several options were investigated for achieving redundancy, including using a welded tie member and adding either internal or external post-tensioning cables capable of carrying the tension load, and using a bolted tie section. The bolted section was comprised of four plates connected by bolting the four corners together using angles. This option was chosen and the section was designed so that any one plate or corner angle could crack, and the remaining section was capable of carrying the load.

Two other decisions were made to decrease the likelihood of problems with the tie in the Providence River Bridge. The first decision was to use high performance steel for the tie. This steel has greatly increased toughness, which increases the fatigue resistance of the steel. In addition, it was decided to metalize and paint the arch span. The added protection should enable the bridge to withstand corrosion longer than most bridges in this environment where large amounts of road salt are used in the winter months which cause significant damage to the bridges in the region.

The Contractor, Cardi Corporation[5], decided to build the bridge off site and to contract with Mammoet, a heavy lift firm, to float it from its construction site to the final site 12 miles (19 km) away. The steel erectors were Northeast Steel and Griffin Iron Works. The bridge site had difficult access for cranes and barges. In addition, the cost to build over water was a consideration, and the contract required coordination with the boat traffic moving under the bridge during steel lifts over the boating channel. One further advantage of building the bridge off site was that the construction of the supporting piers could be concurrent with the construction of the superstructure.

The 5.5 million pound (24,500 kN) steel skeleton of the bridge (See next page) was lifted to a height above the top of the pier bearing seat elevation and placed on barges using self-propelled modular transporters (SPMTs). Two large barges, which were linked to each other using crane booms to form a catamaran-type configuration, were used for the move. When the bridge was in the correct position, the outgoing tide helped to lower it onto the piers.

The bridge opened to traffic in November 2007. RIDOT won the AASHTO America's Transportation Award for Large Project – Innovative Management for the bridge float. The float was also featured on The History Channel's program, *Mega Movers*.

REFERENCES

1. Steere, Patricia D. and Wu, Yihui. "Design and Construction of the Providence River Bridge" *Official Proceedings of the 25th Annual International Bridge Conference June 2-4, 2008.* IBC 08-55.
2. Per Tveit "Design of Network Arches," The Structural Engineer, London, England 1966
3. Nakai, H. & al."Proposition of Methods for Checking the Ultimate Strength of Arch Ribs in Steel Nielsen-Lohse Bridges. " Staulbau 64 (1995) heft 5,pp. 129-137.
4. Fisher, John W. Fatigue and Fracture in Steel Bridges, Case Studies, Wiley, New York, 1984
5. Manual on Use of Self Propelled Modular Transporters to Remove and Replace Bridges, FHWA-HIF-07-022, June 2007

ELEVATION — ARCH SPAN

400'-0" (122m)

C BRG ARCH PIER 2
C ARCH SPAN
ARCH RIB
C BRG ARCH PIER 3

A   C   B

CROSS SECTION

167-0" (51m)

C NORTH ARCH   WIND BRACING   C PROVIDENCE RIVER BRIDGE & CENTER ARCH   C SOUTH ARCH

DIAGONAL SUPPORT CABLE   CABLE CONNECTION

82'-0" (25m)   82'-0" (25m)

DIAGONAL SUPPORT CABLE (TYP.)   CABLE CONNECTION (TYP)

80'-0" (24m)

2 BAR RAIL (TYP.)

PLATE GIRDER FLOOR BEAM   BOTTOM FLANGE IS LEVEL

CENTER ARCH

C ARCH
A
30°   6'
ANGLE PL
HANGER PL
C
CABLE

PLATE DETAIL (CENTER TIE)

CABLE
B
A490 BOLTS (TYP.)
C FLOORBEAM
C
TOP OF FLANGE PL
60°
C TIE

SECTION D

7'-2" (2m)
S.S. COTTER PIN
WASHER
6" (.15m)
3 1/4" (.08m)
5 3/4" (.14m)
10 9/16" (0.3m)
1'-7 1/4" (0.5m)

SECTION @ CENTER ARCH — A

C CENTER ARCH   C HANGER PL
3'-0" (0.9m)
FLANGE PL
4'-0" (1.2m)
WEB PL
DIAPHRAGM PL
HANGER PL

SECTION @ CENTER TIE — B

C CENTER TIE   C HANGER PL
GUSSET PL CONTINUOUS
A490 BOLTS (TYP.)
FLOOR BEAM   FLOOR BEAM
WEB PL
GUSSET PL CONTINUOUS
DIAPHRAGM PL
FLANGE PL

SECTION @ CENTER TIE — C

3'-0" (0.9m)
A325 BOLTS (TYP)
4'-0" (1.2m)
WEB PL
ANGLE PL (TYP)
FLANGE PL (TYP)

Design and construction of the Mangamahu River Bridge in New Zealand

Michael Chan, Holmes Consulting Group, PO Box 942, Wellington, New Zealand

The Mangamahu Road Bridge spans over the Mangawhero River northeast of Wanganui, on the north island of New Zealand. The previous three span bridge collapsed in a flood in 2006. The suggested replacement was a truss with two spans of 42.5m. Significant river protection works were required.

The network arch with a span of 85m was proposed by the contractor. See fig. 1. The network arch bridge eliminated the need for a central pier in the river. This removed the need for any substantial river protection works around a central pier in the river. The proposed network arch also has a shallow deck depth which permitted the existing road carriageway to remain at its current grade eliminating the need to build up approach roadways. These advantages and associated cost savings led the client to accept the network arch bridge as the replacement bridge design.
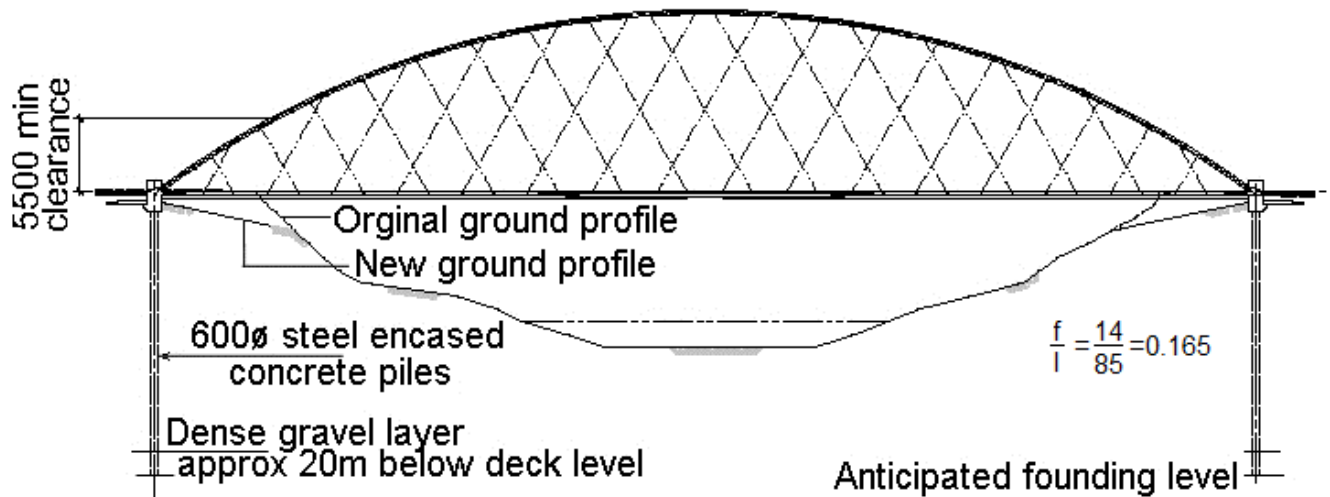
Figure 1 - The Mangamahu network arch. Span 85m. Built 2008.

The Mangamahu network arch bridge has a single 3.5-metre wide lane plus shoulders between side barriers. The approximate overall width is 6.6 metres. The arch has a rise of 14 metre and has a circular geometric profile for ease of fabrication. The steel encased concrete piles can carry the vertical loads from the empty span even if some of the earth around them is washed away in a 100 years flood.

The top chord member is a fabricated box section comprising a hot-rolled 310UC section arranged with its flanges oriented vertically, with extra top and bottom welded plates for the arch element. See fig. 2. The readily available 310UC section was curved into shape by being pressed in one metre increments. This minimized steel wastage and extra fabrication cost compared to cutting a curved web plate out of a steel sheet. Per Tveit who first suggested the network arches claims that a simple H-profile would have been able to carry the forces in the arches, but its lack of stiffness might have made the erection more difficult.

Figure 2 - Arch element comprising a curved 310UC with welded top and bottom plates
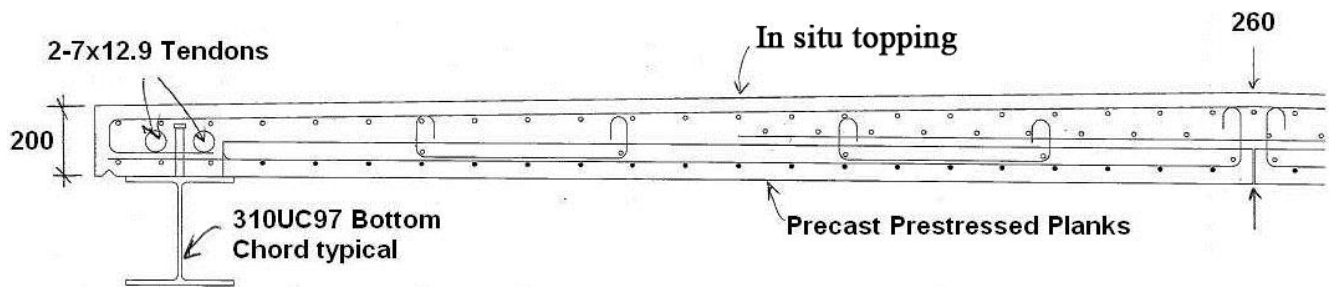
Figure 3 – Tie and deck slab of the Mangamahu network arch in New Zealand.

A ladder form of deck construction with hot-rolled 310UC longitudinal tie and transverse transoms at regular centers was utilized. The transoms support precast deck slabs made composite with the transoms and bottom tie with an in situ topping. This system offered an opportunity to complete much of the superstructure work over the river from a suspended, self-supporting platform. Longitudinal posttensioning is provided within the deck slab to provide a residual compressive stress and control the variable tensile stresses due to live loading.

The erection methodology was developed with the contractor, with the steel ladder deck supported on falsework at approximately 30-metre spans, while the arch was split into two halves, each assembled on opposite sides of the river. Each half of the arch, with its bracing elements attached, was lifted by an 85 tonnes capacity mobile crane and moved into position, then bolted into the knee-joint. When both halves of the arch were in position, two workers in a man-cage were lifted by a third mobile crane, allowing the arch crown splice to be bolted. See fig. 4.

The whole operation of erecting the arch took less than a day to complete. When all the hangers had been installed, the falsework supporting the ladder deck could then be removed as the structure was now capable of being self-supporting. Precast deck panels were lifted into place by a mobile crane, providing a permanent formwork for the in situ concrete topping. The whole construction process took less than three months to complete, proving the construction methodology to be very effective.

Figure 4 - Erection of the arches with mobile cranes

Typical hanger setout detail

Note: Epoxy grout bars into couplers, nuts & forks. Methodology and materials to be approved by the engineer. Minimum strength of resin and epoxy grout to be 50MPa at 28 days. Epoxy to complete coat treads between hangers, nuts, couplers, forks and seal these areas to water ingress.
Hangers to be fully tightened into couplers and fork fillings. Couplers are to be located clear of hanger crossing points.

Fig. 5 – Hangers for the Mangamahu network arch.

A parallel hanger arrangement of 32 hangers at 60 degrees measured to the bottom tie was adopted as this resulted in reasonable design actions on the arch, bottom tie and hangers. As the magnitude of actions is relatively small and fatigue not an issue for this bridge due to its low volume of traffic, Reid bars were utilized for the hanger elements, providing an economical, readily available and suitable hanger solution. See fig. 5. Purpose designed hanger fittings were also developed to improve fatigue performance compared to the proprietary fittings.

The total weight of structural steel for the network arch was approximately 80 tonnes. This is 150 kg/m^2 of bridge area. In addition to the numerous advantages the network arch bridge brought to this project, the resulting bridge is also an elegant structure which is welcomed by the local community. If we define the slenderness of a tied arch bridge as the span divided by the sum of the height of the arches then the Mangamahu network arch was the world's most slender arch bridge for less than two years. The bridge got a gold award of excellence from the Association of Consulting Engineers in New Zealand in 2009.

Reference:
Chan, m. & Romanes, M. (2008) "New Zealand's First Network Arch Bridge" Conference paper the 4th New Zealand's Industry Conference hosted by HERA in October 2008.

Design and construction of the Waikato River Network Arch

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A new crossing of the Waikato River was required as part of a proposed new road alignment bypassing the town of Taupo, New Zealand. The project involves 16 km of new road alignment and two major bridges, one across a geothermal power plant site and the other crossing the Waikato River.

The project to construct the new road was bid competitively between three major contractor and consultant design teams to develop tender designs for the overall project. The successful contractor was Fulton Hogan, lead consultants, Sinclair Knight Merz. Holmes Consulting Group were sub consultants for the project, responsible for the Waikato River Bridge design.

The network arch bridge concept was presented to the contractor who immediately saw the potential for this form of structure and potential savings in construction costs over a concrete bridge or other form of steel bridge. The Waikato River Bridge, upon completion in early 2011, will be only the second and the longest network arch bridge in New Zealand.

The first network arch bridge in New Zealand, the Mangamahu Bridge also designed by Holmes Consulting Group, was completed shortly before the Waikato River Bridge design commenced. The idea for using a network arch bridge on both of these projects was taken after a seminar presented by Per Tveit in New Zealand in 2007. Interestingly, one of the competing contractor/consultant teams also developed a network arch bridge design although their bid was not successful.

The Waikato River Bridge is 148 m long overall with a 100 m main network arch span over the river channel (refer Figure 1). The two end spans are steel ladder girder bridges. The bridge supports two traffic lanes, a footpath and provision to carry two geothermal steam pipes across the bridge. The arch has a rise of 20m and is 12.75m between arch chord centrelines to provide for the clearance envelopes required by the project (refer Figure 2).

Positioning of piers or any construction activity within the river waterway was not permitted by the conditions of the contract. The Maoris do not want the river spirits to be disturbed. This dictated the main span length and method of constructing the main arch span over the river.
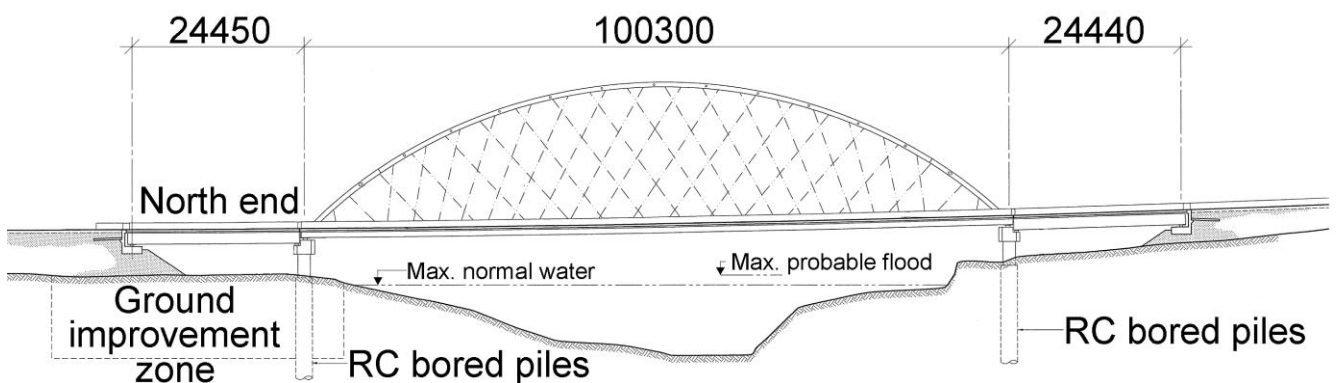
Figure 1. Waikato River Network Arch. Span 100 m. Built 2010.
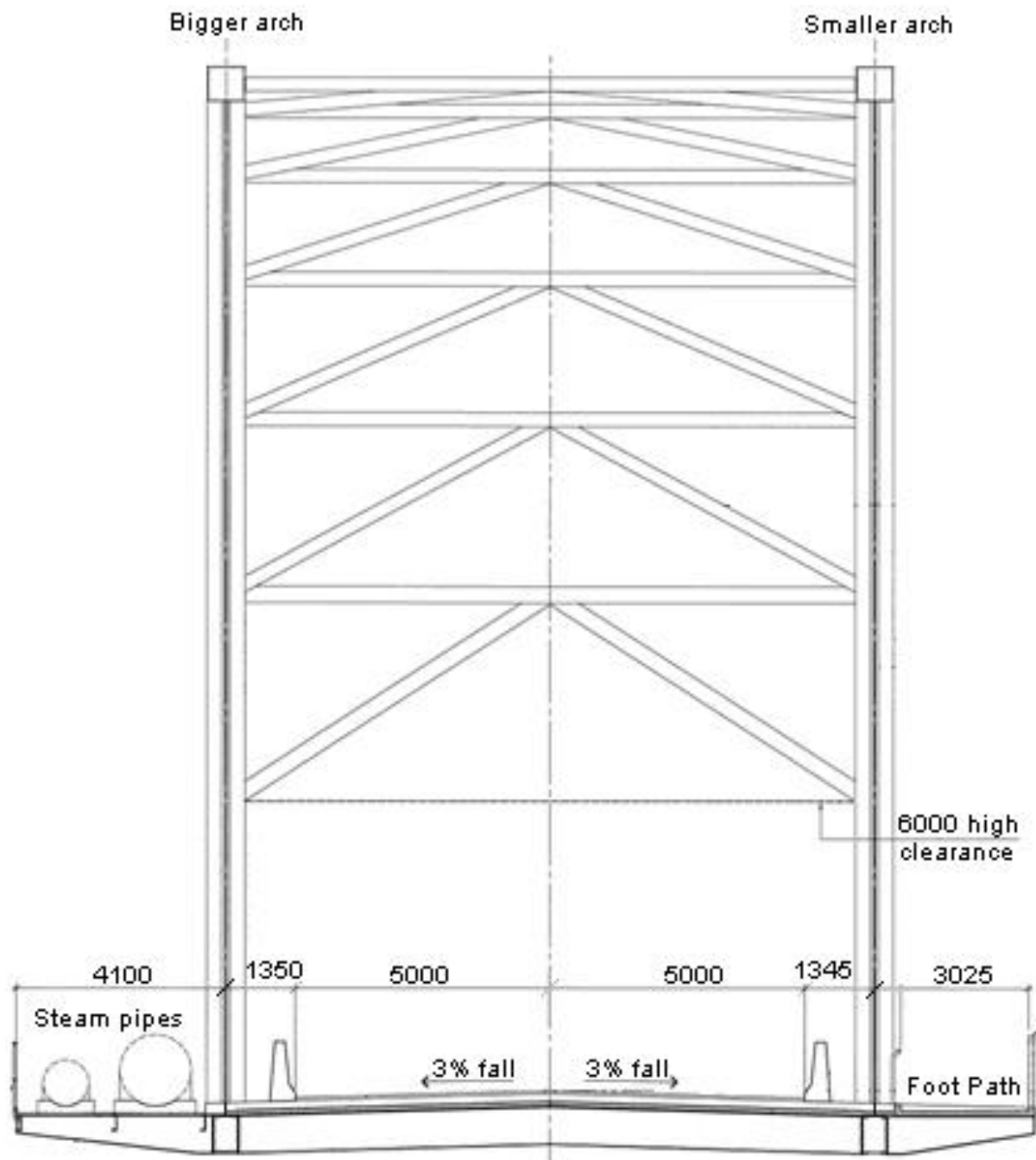
Figure 2. Waikato River Bridge cross section

The network arch structure uses a fabricated steel box section top chord, measuring 680mm deep and 600mm wide and is concrete filled to provide additional load carrying capacity (refer Figure 3). The project requirements for this bridge included additional design scenarios to consider the loss of hangers through fatigue or accident, as directed by the client. These load combinations proved to govern the size of the arch members because of larger bending moments developed in the arch top and bottom chords where the hangers had been removed.

The bottom tie also uses a fabricated steel open box section and forms a ladder deck arrangement with steel transom beams transverse to the bridge supporting a concrete deck slab. Post-tensioning tendons are provided in the bottom chord member to provide compression to the deck slab under service loadings and sufficient tension capacity to the bottom chord under ultimate limit (strength) conditions.

The arches for the Waikato River Bridge were filled using a self-compacting concrete mix, placed using a standard concrete pump connected to the base of each end of the arch. Concrete strength was normal strength (40 MPa compressive strength at 28 days). By pumping concrete from each end of the arch we could vent any air in the arch out of the ports along the length and at the crown of the arch.

A varying hanger arrangement was adopted for the bridge to optimise fatigue performance for the hangers. Hanger bars are 64mm diameter medium tensile solid bars with proprietary fittings (Refer Figure 4).

Bigger arch

Allow 200Ø hole for concreting located 500 clear of splices, stiffeners & bracings and specifically indicated. with 10mm cover plate

25mm PLATES (WEB)

Smaller arch

Allow 200ø hole for concreting located 500 clear of splices, stiffeners & bracings and specifically indicated with 10mm cover plate

20mm PLATES (WEB)
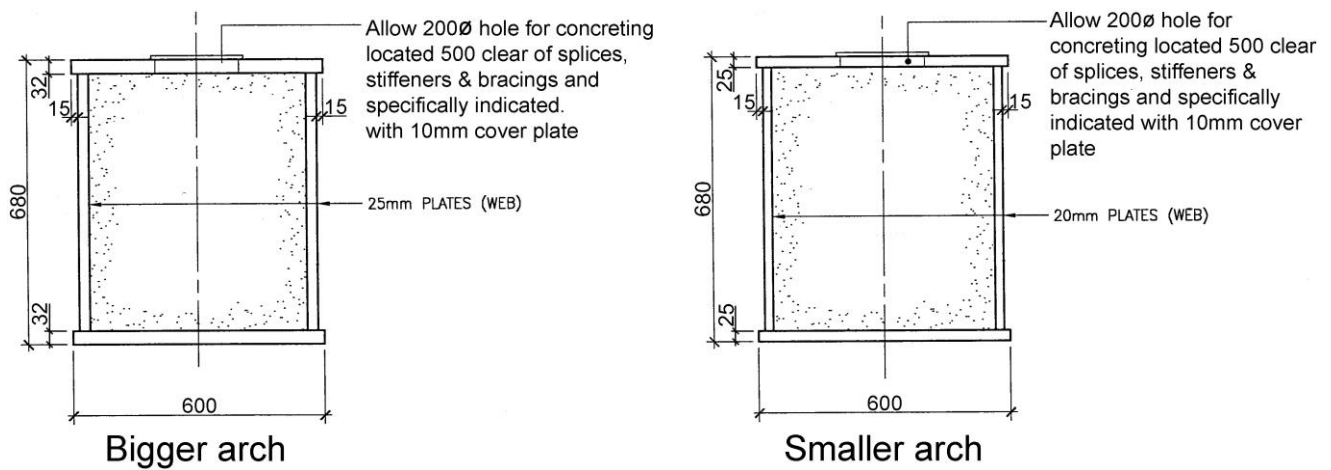
Figure 3. Waikato River Bridge Top Chord Sections

200x50mm HANGER CLEAT

M64 MACALLOY GRADE 460 STANDARD FORK ALLOW WASHERS/SHIMS TO EITHER SIDE OF CLEAT TO ENSURE HANGER FORK IS POSITIONED CENTRAL ON HANGER CLEAT

M64 MACALLOY GRADE 460 TURNBUCKLE

M64 MACALLOY GRADE 460 HANGER

M64 MACALLOY GRADE 460 STANDARD FORK ALLOW WASHERS/SHIMS TO EITHER SIDE OF CLEAT TO ENSURE HANGER FORK IS POSITIONED CENTRAL ON HANGER CLEAT

200x50mm HANGER CLEAT

APPROX. T.O SLAB

T.O FABRICATED STEEL BOTTOM CHORD

Figure 4. Waikato River Bridge typical hanger configuration

The network arch span provided a lightweight structural system – the central span utilises approximately 360 tonnes of structural steel (180 kg/m^2 on deck area). Construction of the arch span will include launching of the completed bottom chord across the river, supported on cantilevered temporary falsework so that no temporary piles are located in the waterway. The arch top chord will be craned into position in sections and site welded together to complete the arch. Installation of the hanger bars will complete the steelwork followed by completion of the concrete deck slab. See article on the Mangamahu bridge on page H-10.

Completion of the overall project was early in 2011.

Design and construction of the network arch over the Lužnice River

LADISLAV ŠAŠEK, Ph.D., MOTT MACDONALD, Prague, Czech Republic.

Fig. 1. Approach to the Bechyně Bridge

The Bechyne Bridge in southern Bohemia is the third network arch to have a prestressed-concrete deck. It has two steel arches with a rise of 0.147 times the span. The network arch was suggested by Per Tveit when he was a student in 1955. The two Norwegian arches designed by Per Tveit were finished in 1963. [1] pgs 5b to 7a. Both had concrete ties with longitudinal prestressing cables.

The network arch has inclined hangers that cross each other at least two times. See fig.2. It works like many lattice girders on top of each other and has very little bending in the arches and in the longitudinal direction in the tie. Network arches are very stiff.

The Bechyne network arch was the third network arch with a concrete tie with longitudinal prestressing cables. [1] pgs 3a to 5a. Later network arches has been built in: Roudne, span 34m, 2008. Baliny, span 73,5, 2009. All Czech network arches have been designed by MOTT MACDONALD. They will all have longitudinal prestressing cables in the tie. The building of a fourth network arch with a span of 200m has started in Prague. It shall be finished in 2012. [3]

Aesthetic considerations were important in the choice of a network arch for the Bechyne Bridge [2]. It replaced a bridge from 1883 with insufficient load carrying capacity and not enough room for the 100-year flood. The old bridge could carry only three vehicles of 6 tonnes at the time. The network arch could carry three 26 tonnes vehicles or one vehicle of 50 tonnes.

The new bridge is built 1 m higher than the original structure. The new granite stones in the abutment are the same as in the original structure. The abutments are capped with reinforced concrete and a bearing that consists of rubber and steel plates. During a 100 year flood the bridge is expected to maintain a clearance of 1m. Only small alterations were needed at the roads on both sides of the bridge. The rise of the tie is 1% of the span. See fig. 2.

The hangers are distributed very tightly and provide a continuous support of the deck. This allows the deck to be very slender, creating an attractive structure with a delicate look. The cross section of the welded arches and the transverse members between them suggest an inverted U. They are designed to be open to allow easy access for inspections and maintenance. They all measure 320 by 320 mm. The transverse members are placed at the two quarter points and the halfway point. See fig. 2. Two additional transverse beams steel are located at each end of the arches just above the bearings. These crossbeams are buried beneath a layer of reinforced concrete.
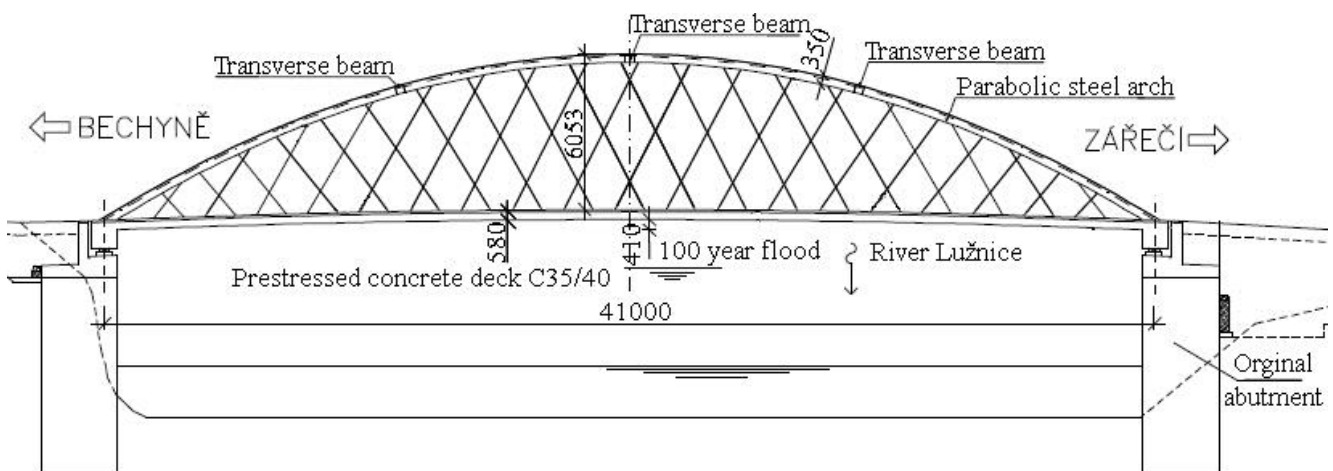

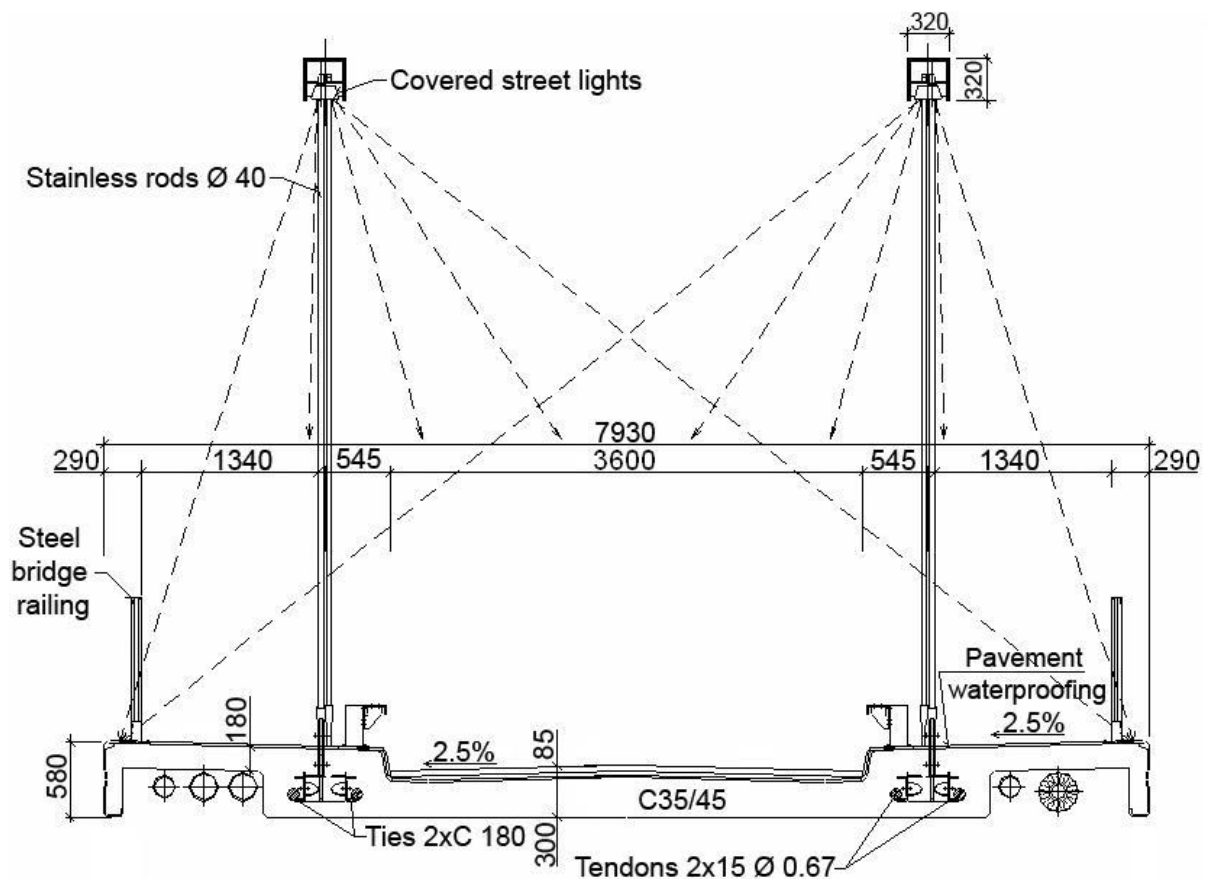Fig. 2. Longitudinal section of the Bechyne network arch

Fig. 3. Cross-section of the Bechyně Network Arch

The deck of the new bridge is only 38.5 cm thick. It has 30 cm of reinforced concrete topped by approximately 8.5 cm of asphalt road surface. See fig. 3. The comparable lower flange of the lattice in the original steel bridge was approximately 2 m thick. The new, thinner deck meant that the roadway could stay at the same elevation despite the increase in room under the bridge.

The space under the sidewalks contains the utilities. The traffic lane is 3.6 m wide and the sidewalks 1.3 m wide. The traffic lane is separated from the sidewalks by a 0.5 m high steel guardrail that also protects the hangers. Along the outer edges of the sidewalks there is a 1.1 m high steel railing. The deck is a 7.9 m wide slab of C35/45 concrete. Thickness is 18 cm under the sidewalks, 25 to 30 cm under the roadway and 56 cm just below the hangers. The deck is prestressed by four tendons, embedded in the concrete slab. Each tendon consists of 15 prestressing strands and has an outside diameter of 15.5.

The tendons are anchored not to the special anchors normally used in prestressing cables, but rather to the 30 mm thick vertical steel face plates at the ends of each arch. The tendons both carry the horizontal arch force and transport the prestress into the concrete slab. They eliminate longitudinal tensile stress in the concrete. In the transverse direction, the slab acts like reinforced concrete.

The edge beams are part of the bearing structure and feature a shell like coating of glass fibre and concrete that is sprayed on to a thickness of approximately 15 mm. The coating provides a smooth surface and enhances the structure's aesthetic appeal. The pavement on the roadway consists of a medium grained mastic asphalt carpet made of modified asphalt (40 mm), poured asphalt (40 mm), and waterproofing (3 mm). It has an overall thickness of approximately 85 mm. The bridge is protected with a multilayer waterproofing applied by spraying. See fig. 3.

Work on the Bechyne Bridge began in October 2002. The original bridge had to be moved approximately 10 m downstream and placed on temporary supports so that pedestrian and automobile traffic could continue. The steel structure for the new bridge was erected without welding with the aid of light support scaffolding. [1] p. 92c. The entire steel structure was divided into two arches, three central beams, and two transverse beams. Each arch was split into three parts. Consequently, there were 11 assembly elements altogether, plus elements of the lower tendon. Every assembly joint of the arch elements was designed as a friction joint with high-strength bolts.

Originally, MOTT MACDONALD had proposed using a crane to lift and place the steel skeleton as well as the concrete formwork. The steel skeleton would be built on land and was expected to support the form while the concrete deck was cast. [1] p. 92c. Instead of using the crane, however, the contractor chose a more traditional approach. The assembled steel structure with arches, crossbeams and hangers were assembled on-site using a steel lattice scaffolding that was erected on temporary supports made of steel micropiles set in the riverbed. [1] p. 92c. Then the network arch was lowered into place using hydraulic jacks.
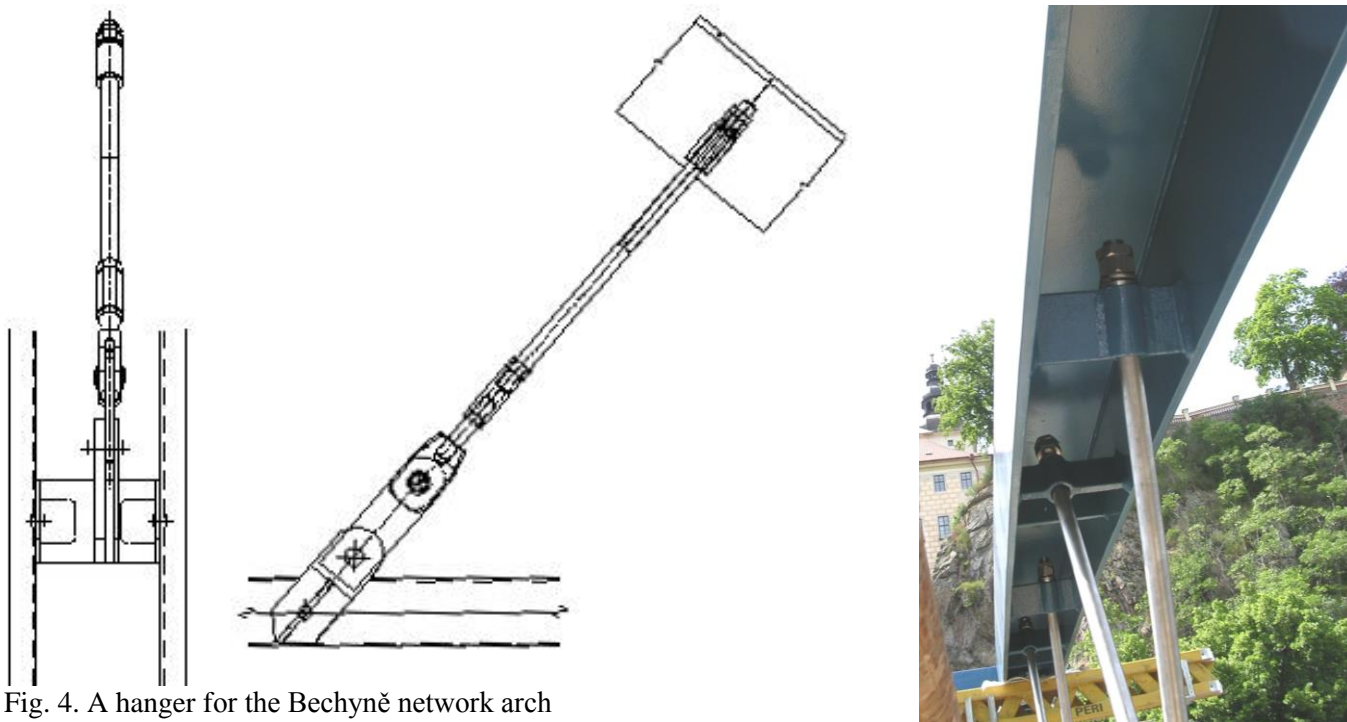
Fig. 4. A hanger for the Bechyně network arch

The hangers are stainless steel rods with an outside diameter of 40 mm. The hangers are fixed to both the arches and the slab by joints specially designed by MOTT MACDONALD for this project. See fig. 4. In the arch the hangers go through a hole and are fastened by a hexagonal nut. The connection to the deck features a hinged coupler with a hexagonal nut that can be rotated to the left or right to tighten the hangers. The joints are easy to assemble and adjust and can be replaced if necessary without causing accidental strains.

The stainless steel hangers were executed with professional care and give an exquisite appearance, shining impressively as they reflect the sun or the artificial light provided by four lamp poles on the abutments. There are 36 spotlights additionally located in the cross sections of the arches. They provide special lighting during festivals.

After the deck had been cast and cured for 7 days, all hangers were activated by tightening the hexagonal nuts of the joints. Prestressing the slab activated the structure from the static's point of view. The hangers were adjusted after the structure was lowered onto the abutments. To check the forces, tensiometer measurements were conducted on 50 percent of the hangers. In most hangers, the weight of the structure activated the hangers' tensile force, and there was no need for further adjustment. Some of the hangers near the abutments had to be tightened so that they would not have tension in the event of extreme accidental loads.

If the arch has the right shape and the hangers have no slack before the arch is lifted, then the hangers need no later adjustments.

Amount of materials used per m^2 is: Structural steel: 101 kg Concrete: 0.316 m^2

Reinforcement: 66 kg Prestressing steel: 9.3 kg

The bridge was finished in August 2004. At a total project cost of approximately $1 million, the Lužnice bridge should provide both an economical and an aesthetic argument in favour of the construction of more network arch bridges with prestressed-concrete decks.

References

[1] Per Tveit. Homepage http://home.uia.no/pert p. 92a refers to page 92a under the button "The Network Arch" p. E-4 refers to page E-4 under the button "Systematic theses". 2013a.

[2] Ladislav Šašek "Less Is More" Civil Engineering, April 2006 pp. 50-55.

[3] Ladislav Šašek and Jiří Petrák. "Design of Network Arch Bridges with Prestressed Concrete Deck". 3rd *fib* International Congress Washington in May 2010.

"Palma del Río" bowstring arch Bridge. Córdoba. Spain.

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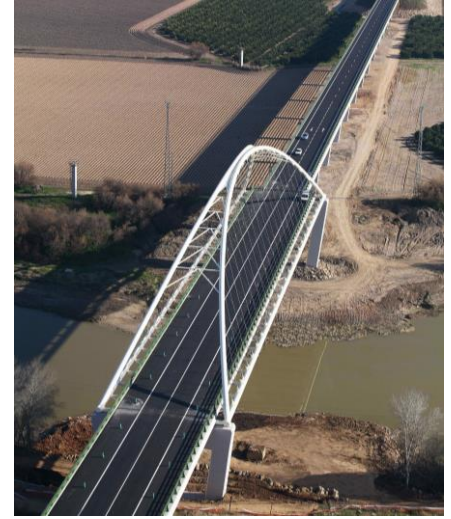
The bridge of Palma del Río[1], over Guadalquivir River, designed by IDEAM S.A., constitutes a remarkable development of the typology of Network bowstring arches, with a main span of 130 m, and two inclined arches that rise 25.0 m with a tubular steel cross section of 0.9 m of diameter and 50 mm of maximum thickness.

The deck consists of two simple lateral steel ties of 0.9 m of diameter 40 mm of maximum thickness, and one intermediate platform supported by transverse composite concrete-steel beams of variable height spaced every 5 m. The extreme slenderness of the arches and ties (H/L=1/144.4) is obtained thanks to closed cable network suspension system[2,3], which drastically minimizes the bending moments on the arches and ties, and reduces the buckling effects on the arches, as it has been described before.

The deck's platform is 16 m wide, and the deck consists of an upper slab of 0.25 m in thickness. This slab is materialized by the inferior precast slabs supported on the transverse steel beams of the deck placed every 5 m, and the upper reinforcement and cast-in-place concrete which completes the total thickness of the slab. The transverse beams that materialize the deck span 20.4 m between the tubular lateral ties of the bridge. The hanger's anchorages are located right where the transverse beams meet the lateral ties. This way the lateral ties aren't subject to the effect of the punctual loads that act on the deck.
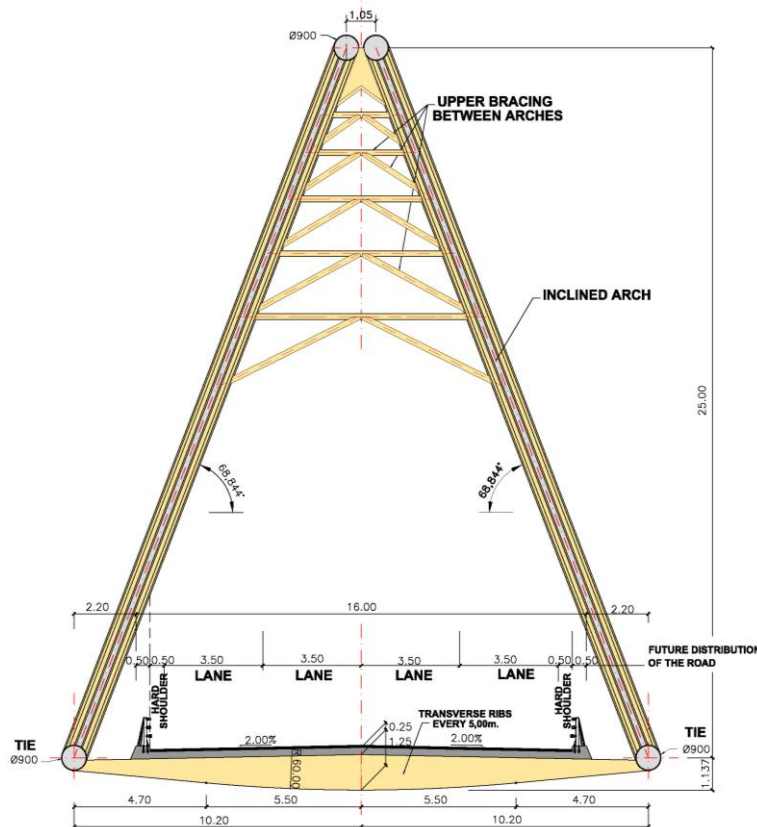
The deck's transverse composite steel-concrete beams have variable height, and the beam's lower plate follows a circular curve with 60 m of radius in the central part, and with a linear variation in the rest. The maximum height is 1.25 m, and the cross section is an "I" beam. The steel used in both arches, ties, and the deck is S-355-J2+N.

The aim of the lateral ties of the bridge, tubes of 0.9 mm of diameter, is mainly to equilibrate the horizontal component of the arches in the extremes, avoiding the transmission of the horizontal reaction to the foundations, as well as the previously mentioned lateral support for the transverse beams of the deck.

The hangers act as the linking element between the deck and the arches, and transmit the vertical loads from the former to the latter. The hangers were designed in an inclined network suspension system linking the ties to the arches, with spaces of 5 m between their extreme anchorages. Furthermore, as the number of hangers increases, it allows us to use small units that are easy to fit.

The main bearing structure of the bridge is the two arches, which are inclined 21.20º with respect to a vertical plane, and they rise from the extremes of the lower part of the deck converging with the lateral ties. Both arches meet at the crown of the bridge and rise 25.0 m, with a ratio rise-span 1/5.2. All along the arches, and every 5 m, are arranged the upper extreme anchorages of the hangers. Bracing the inclined arches, we designed a K truss with tubular elements, which reduces the out-of plane buckling length of the arches.

The distance between anchorages for the network suspension system is reduced to 5 m, fulfilling multiple objectives:

- Reducing the buckling length of the arches
- Reducing the bending moments of the deck.
- Simplifying the arches-hangers and deck-hangers anchorages, because of the use of small units of hangers.
- Achieving a great efficiency in the distribution of punctual loads on the deck.

The material used for the construction of the arches was steel. It could be thought that concrete would be the ideal material for the compressed arches, but analyzing the constructing process, steel has great advantages: the execution can be carried out by dividing the whole length of the arches in shorter pieces that are easily transportable and can be welded on site with simple auxiliary elements, reducing the construction duration. The use of steel as the predominant structural material also permits reducing the vertical reactions transmitted to the foundations, optimizing their cost.

The first stage of the construction was the execution of the piles, with 1.50 m of diameter, and the cap piles of the two piers of the arch bridge, and after that the two piers. The foundations of the temporary supports were also on deep piles so as to avoid troubles with eventual river flood. The assembly of the segments of the deck is done by welding on site the transverse beams to the lateral tubular ties until completing each modulus of the deck, leaving them prepared for lifting with the use of cranes.

Once the complete steel framing is finished propped in 4 temporary double supports can be erected the arches.

First of all, it is necessary to place over the deck 4 temporary double props to support the different segments of the arches. Meanwhile, in the three central segments, the bracing between arches was welded on site. These operations concluded, each modulus of the double arches was raised by using cranes in a similar way to that of the deck, leaving the arches propped.

Bottom figures show the arches' crown closing operation with the lifting and adjusting in place of the central segment. Finally all the welding between the different segments of the arches, and the connection of the arches to the lateral ties was completed. With the whole steel structure finished, the arches' temporary props can by removed causing the arches to start working.
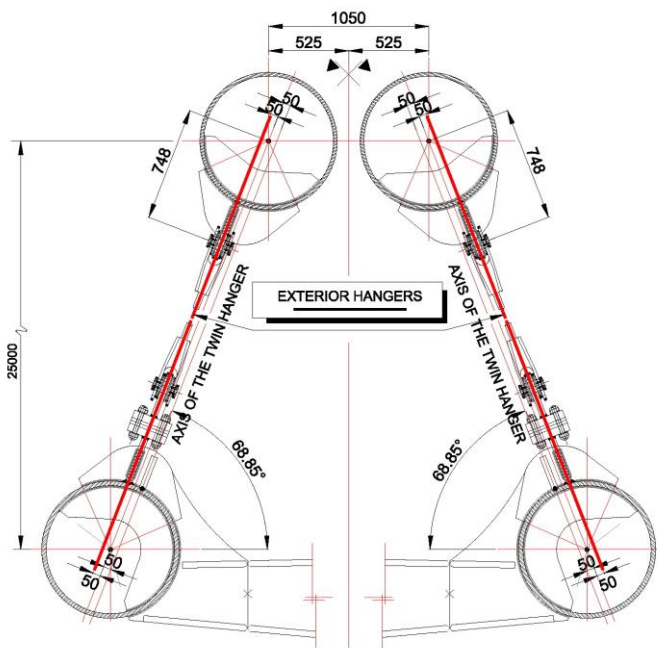
With the deck framing still propped, the hangers of the bridge were fitted and tensioned following the tension sequence designed in the project. This stage completed, the props of the deck were removed.

With the deck suspended by the hangers, the precast slabs on the upper platform were placed on site. These precast slabs span 5 m between the transverse beams of the deck.

Once all the precast slabs were positioned the deck was ready for placement of the upper reinforcement and the pouring of concrete on the slab.

Once the deck construction was completed the final adjustment of the tension of the hangers, the finishing tasks, and the load test were carried out.

The multiple crossings between hangers were resolved with an original device, which enables the hangers' arrangement and minimizes the visual impact of these crossings.

The hangers are closed cables with 45, 40, or 37 mm of diameter, which compose the network suspension system of each inclined arch, are located on two different parallel planes, each one of them with a slight eccentricity of 50 mm with respect to the arch's plane, that is the reason why the hangers cross but don't cut one another. At the place of the crossing hangers there is a device that allows the fastening of the cables as well as the free turn of one cable with respect the other.

REFERENCES

1. Millanes F., Ortega M., Carnerero A. "Palma del Río. Arch bridge, Córdoba, Spain". *Structural Engineering International, Vol. 20, Nr. 3*, published by IABSE, pg. 338-342, August 2010.
2. Millanes F., Ortega M., Martínez D., Solera P. "The use and development of the network suspension system for steel bowstring arches", *Proceedings of the IVth Congresso de Construçao Metálica e Mista* (Eds.: cmm-associaçao Portuguesa de Construçao Metálica e Mista), Lisbon. Portugal. Pp II-97/106, 2008.
3. Millanes F., Ortega M., Carnerero A. "Project of two metal arch bridges with tubular elements and network suspension system", *Proceedings of the 7th International Conference on Steel Bridges* (Eds.: ECCS-European Convention for construction SteelWork), Guimaraes. Portugal. Pp II-88/96, 2008.

"Deba" bowstring arch Bridge. Guipuzcoa. Spain.

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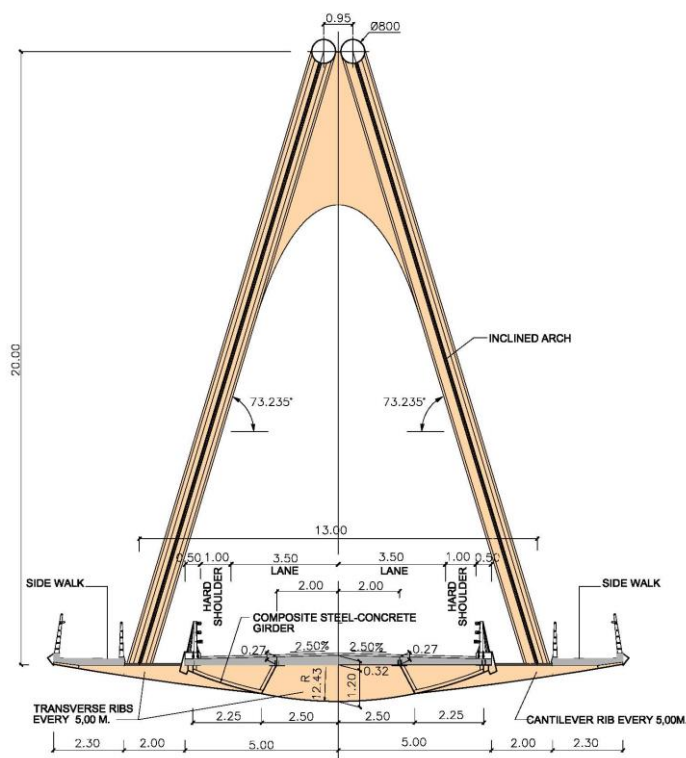
The bowstring arch bridge of Deba[1] in Gipuzkoa, located in the north of Spain, crosses the river Deba with a main span of 110 m. The bridge, designed by IDEAM S.A., is a remarkable example of the use of the network suspension system[2] in bowstring steel arches.

The main structure consists of two clearly defined zones:

- The access viaduct, located over the river's lowlands, a continuous steel and concrete composite hollow box girder with 20+30+30 m long spans.
- A 110 m long span bowstring arch bridge over the river.

Both the arch bridge and the access viaduct had lateral sidewalks at either side which allow the pedestrian passage, thus fulfilling the demanded requirements for the solution.

The bridge's cross-section is 18.6 m wide, of which 10 m constitute the central platform for the vehicles, with two 3.5 m wide lanes in opposite directions, 1 m wide lateral shoulders and 0.5 m at either side for the security railing.

In order to provide the bank promenade with continuity across the river, 2.3 m wide lateral sidewalks were placed at either side of the structure, 2 m away from the platform. The sidewalks consist of 0.20 m thick precast concrete slabs, supported on the transverse ribs located every 5 m. The void space between the sidewalks and the platform was covered by a bar-grated protective mesh.

The arch bridge spans 110 m and consisting of a double tubular arch linked to the deck by means of hangers arranged in a mesh pattern, which confer a series of peculiarities, both to its morphology and to its structural behaviour.

The arch bridge's deck has the same depth and the same bottom curve in its cross-section as the access viaduct. The deck's cross-section comprises 2 hollow box girders whose inner webs are 4 m apart at the top, in such a way that this separation defines the access spans' cross-section's geometry, since both deck's webs lie on the same plane.

The arch bridge's hollow box girders are 2.75 m wide each, and as deep as 0.95 m. Each girder's webs are very different. The outer one is vertical, barely 0.28 m deep, while the inner web, 1.04 m long, is inclined 26.8º.

Supported on the steel girders' top flanges are the precast concrete slabs with steel trusses on which the upper concrete slab is cast.

Every 5.0 m, just like in the access viaduct, transverse cantilever ribs are attached to the deck. The precast slabs which constitute the sidewalks are placed at the ribs' ends. The ribs act as transverse beams which take all loads coming from the sidewalks as well as from the deck and transfer them to the hangers through the anchorages located at the platform's edge, 6.5 m at each side of the deck's centre line.

The arch consists of two circular tubes 0.8 m in diameter, made of S-355-J2+N grade steel, 35 mm thick at the springing and 20 mm thick at the crown. Both arches lean inwards at an angle of 18º with the vertical plane. The arches' springing are 13 m apart, while at the crown the tubes are almost tangent to each other, with a minimal clearance of 0.15 m. Each arch's axis is a parabola with a rise of 20 m.

A 20 mm thick steel plate, ending in an elliptic edge, braces the two arches together. It is aimed at guaranteeing a joint response from both arches to transverse wind actions and limiting the arches' buckling length.

Lying on the arches' planes, the hangers are circular solid bars 56 mm in diameter made of S-460 grade steel. A network[2] arrangement was chosen leading to a latticed mesh with multiple crossings. Each plane of hangers contains two families, each of them parallel to one direction. With the anchorages 5.0 m apart both along the arch and the tie beams, and a 2.5 m offset between the upper and lower anchorage points, a mesh is created in which each hanger crosses two of the opposite family.

In order to solve the hanger crossing points, the contractor devised a special piece, in the shape of a needle eye which solved the hanger crossing. Besides, since the individual hangers were supplied in 12 m long units, the very piece was used as a coupler between bars to attain hangers as long as 21 m.

In order to prevent the hangers from colliding with one another due to transverse deflections caused by wind or by any other vibratory effect, an elastic disc-shaped piece was attached. It hampers direct contact between the metallic pieces and consists of two halves, fastened together by screws, which allows for easy assembly and adjustment right at the hanger crossing.

The hangers are anchored to the deck at the transverse ribs, between the deck and the sidewalks, 1.5 m from the former and 0.5 m from the latter, to be specific.

The anchorage eccentricity with respect to the longitudinal load-bearing elements, the sidewalks and the deck, would cause the ribs to transfer to them the load disequilibrium arising from the hangers meeting at each rib. Since the ribs were not designed for that purpose, a hot-rolled IPE-300 stringer was laid along the whole bridge, aimed at taking the stresses caused by hanger load disequilibrium at each anchorage. The disequilibrium forces were small and globally self-balanced. This profile was embedded within the sidewalk's inner edge and hidden by the railing's kerb's lower flap.
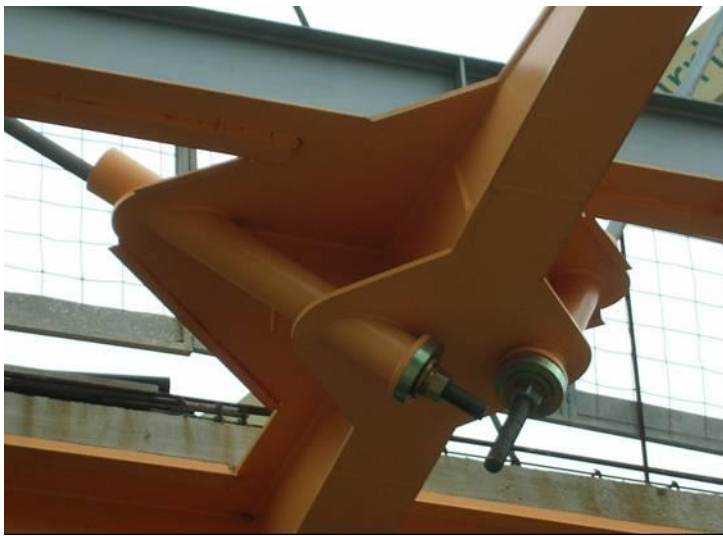
The hangers are anchored to the ribs by means of two cylindrical tubes, one at each side of the rib's web. They are welded to the rib with a plate parallel to their axis and two horizontal cap plates at the tubes' top and bottom ends securing them.

The hangers' stressing anchorage is the bottom one. It comprises a bearing plate on the tube's bottom face, a spherical hinge, a washer and two nuts fastening the bar. On the upper face of the anchorage tube a rubber ring is laid acting as a damper in order to reduce parasite bending moments originating at the hanger's anchorage.

REFERENCES

1. Millanes F., Ortega M., Carnerero A. "Project of two metal arch bridges with tubular elements and network suspension system", *Proceedings of the 7th International Conference on Steel Bridges* (Eds.: ECCS-European Convention for construction SteelWork), Guimaraes. Portugal. Pp II-88/96, 2008.
2. Millanes F., Ortega M., Martínez D., Solera P. "The use and development of the network suspension system for steel bowstring arches", *Proceedings of the IVth Congresso de Construçao Metálica e Mista* (Eds.: cmm-associaçao Portuguesa de Construçao Metálica e Mista), Lisbon. Portugal. Pp II-97/106, 2008.

Blennerhassett Island Bridge   Gregor Wollmann, HNTB, New York, USA

Introduction
The Blennerhassett Island Bridge is located near Parkersburg, West Virginia. Opened to traffic in June of 2008, it was the final link needed to complete an important regional transportation corridor. The 1214-m long bridge crosses two channels of the Ohio River and the island separating them. The main span across the navigational channel of the river is a 268-m long network tied arch (Figures 1 and 2). To the best of the author's knowledge this is currently the longest span for this type of structure in the world.

Fig. 1 – Blennerhassett Island Bridge
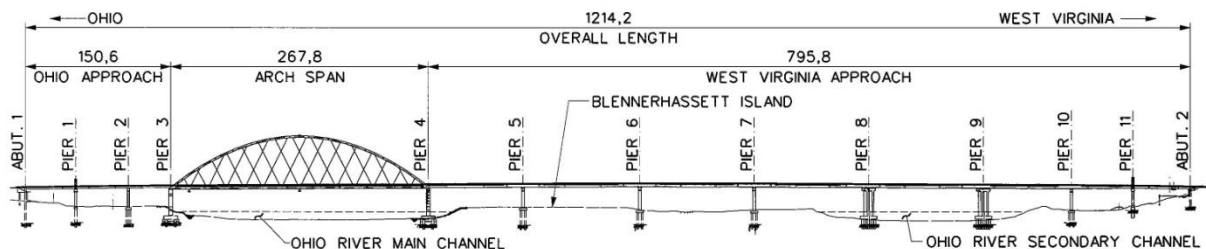
Fig. 2 – Span Arrangement

Tied Arch Description
Figures 3 and 4 show deck cross section and elevation of the tied arch span. The bridge accommodates three lanes of traffic in each direction with generous shoulders. The two vertical ribs are spaced at 32.6 m and rise 53.3 m above the deck. The ribs are laterally supported by an X-bracing system which visually complements the network hanger arrangement (Figure 5).
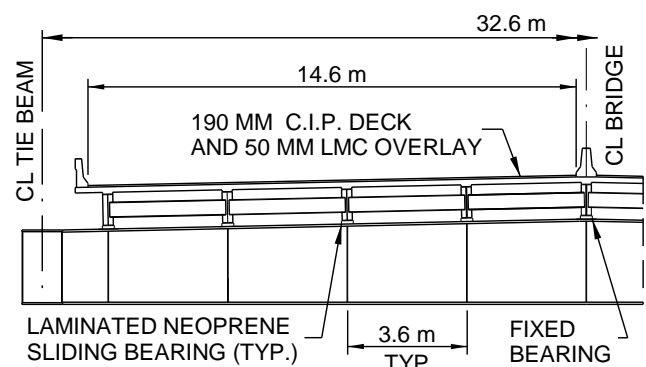
*Fig. 3 – **Typical** Section at Floorbeam*
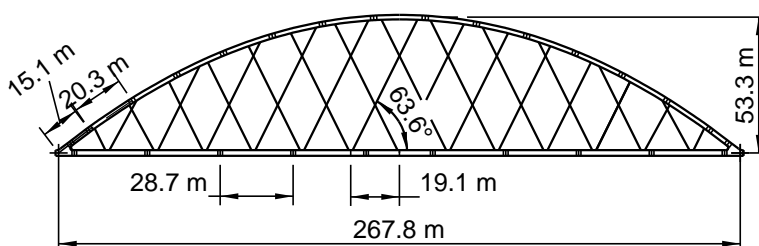
Fig. 4 – Network Tied Arch Elevation

Fig. 5 – Rib Bracing

H-23

The tie girders are steel box sections with inside dimensions of 1.2 m wide by 2.1 m tall. Each tie girder is composed of four individual plates bolted together along their edges to prevent crack propagation from one plate into adjacent plates. The arch ribs are welded boxes 1.2 m wide by 1.67 m tall. Floorbeams are spaced at 19.1 m. The hanger cables are anchored near the floorbeams and are arranged in two families of crossing cables to form the network system. All hangers in one family are parallel to each other and are inclined 63.6 degrees relative to the tie girder.

Each cable is composed of 27 individually sheathed and greased 15.7-mm seven-wire strands. The strand bundle is enclosed by a High Density Polyethylene (HDPE) duct. This system is very common in cable stayed bridges, but its application for the Blennerhassett Island Bridge was a first for an arch bridge in the United States. The cables are anchored inside tie girder and arch rib (Figure 6).

Fig. 6 – Cable Anchorage in Tie Girder

The deck system is composed of nine steel plate girder stringers composite with a concrete slab. The stringers are supported on longitudinal sliding bearings on top of the floorbeams, so that the deck is isolated from the tensile forces carried by the tie girders. Even with this floating stringer arrangement, the deck slab is subject to permanent longitudinal tensile stresses due to the staged concrete placement sequence: Local deflections and curvatures are largest during construction stages with partial concrete placement. Reversal of this curvature during subsequent deck placement stages introduces tension into the concrete slab. Therefore a nominal amount of longitudinal precompression (1.7 MPa) was applied by single-strand post-tensioning tendons embedded in the deck slab.

Advantages of Network Tied Arches

Initially the design envisioned a conventional tied arch with vertical hangers. However, such a system is sensitive to asymmetrical traffic loading, particularly in the case where one-half of the span length is being loaded. Given the great width of the deck and the correspondingly large live load it was not possible to meet the design criteria for deflection limits.

With the network hanger arrangement the structural system takes on truss-like characteristics, e.g. greater stiffness under non-uniform loads and smaller chord bending moments. For the case of the Blennerhassett Island Bridge comparative studies showed that with the network hanger arrangement live load deflections were reduced by a factor of nearly 11, compared to a system with exactly the same member sizes but using vertical hangers (Fig. 7). Bending moments in arch rib and tie girder were reduced by a factor of 4 and 5, respectively. This allowed selecting the depth of the tie girder as the minimum needed for hanger installation.

914 mm

84 mm
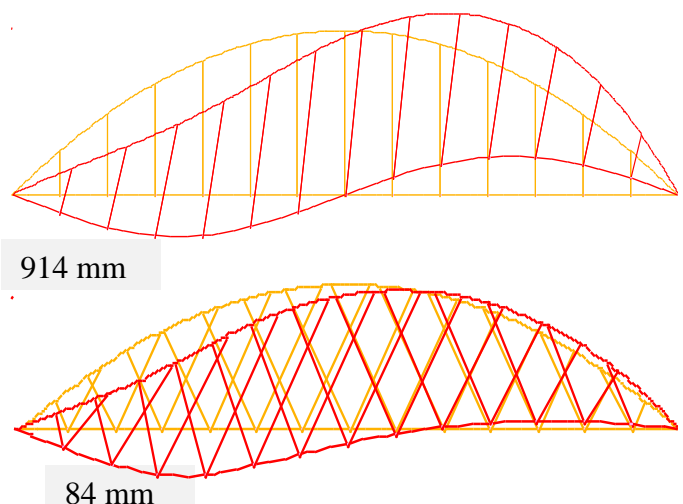
Fig. 7 – Live Load Deflections

Another advantage of the network tied arch system is its resiliency under accidental extreme load cases, such as the sudden loss of a cable or loss of the moment capacity of the tie girder due to partial fracture. With the inclined hanger arrangement cable spacing along the rib is smaller and non-uniform. Therefore loss of a cable has a less detrimental impact on bending moments in the arch rib including second order effects than with vertical hangers.

Also, in the event of the loss of the flexural capacity of the tie girder somewhere along its length, with vertical hangers the released tie girder moment is fully transferred to the arch rib. With the network hanger arrangement the released moment is simply absorbed by a small increase in rib and tie axial forces.

Construction

The bridge was constructed in two halves. The shoring system for each half comprised a pair of falsework beams supported on steel casings and inclined struts and ties to stabilize the partially erected arch (Figure 8). The struts and ties had jacking mechanisms that allowed length adjustments to within 3 mm. One half of the arch was initially erected 150 mm offset from its final position and was launched over Teflon sliding bearings for closure of the tie girders. The gap to fit the keystone arch rib segment was controlled by adjusting the length of the struts. After completion of hanger installation sand jacks supporting the falsework beams were drained to free the bridge. Subsequently, deck concrete was cast on stay-in-place metal forms. Due to the long span of the floorbeams and the use of high strength steel with 480 MPa yield strength, maximum floorbeam deflections during concrete placement were in the order of 50 mm. Special attention had to be paid to the concrete placement procedure to ensure uniform deck slab thickness and correct top of deck elevations.

Fig. 8 – Arch Erection

Hanger cables were installed one strand at a time, using a small winch and hand-held jacking equipment. Hanger forces were verified during installation using a load cell and by lift-off tests of individual strands in the completed structure. An interaction matrix and linear optimization program considering the significant interaction between cables during restressing was developed to determine the final cable force tuning plan.

Acknowledgements

At the time when the Blennerhassett Island Bridge was designed, not many network tied arches had been built and certainly not on this scale. The author was greatly encouraged by the work published by Dr. Tveit on http://home.uia.no/pert, and particularly by the examples of outstanding early network tied arches, such as the Bolstadstraumen Bridge in Norway and the Fehmarnsund Bridge in Germany. As is given evidence by the many projects described on his web site, the network tied arch system has gained much greater popularity in recent years. The author himself is currently involved in design or construction of three more network tied arch structures, including a pedestrian bridge, a highway bridge, and a heavy rail bridge.

The Blennerhassett Bridge got an award from the National Steel Bridge Alliance in 2009.

Structural Design of Arch: Michael Baker Jr. Inc./HNTB
Contractor: Walsh Construction Company

References

Wollmann, G., Zoli, T., *Bridge Across Ohio River and Blennerhassett Island*, Structural Engineering International, Vol. 18, No.1, Feb. 2008, p. 28 – 30.

Happy Hollow Park and Zoo Pedestrian Bridge

Gregor Wollmann, Robin Lee, Ted Zoli - HNTB, New York, USA
December 2011

Introduction

Opened in 1961 Happy Hollow Park and Zoo has been a landmark and local attraction in San Jose, California for the past 50 years. The park features family rides, play areas, a puppet theater, and a small zoo. As part of a major renovation and upgrade begun in 2008, the City of San Jose desired a signature bridge to serve as gateway to the new main entrance into the park. The bridge crosses Coyote Creek and an access road and is angled in plan view, giving it its name "Bent Bridge" (Figure 1). The owner envisioned a structure light and slender in appearance and seemingly floating through the existing tree canopy. No temporary or permanent impact to the Coyote Creek riparian corridor was permitted. An original design by others involving a cable stayed bridge concept was abandoned when contractors' bids for construction came in more than 2.5 times over budget. At this point our firm was retained to redesign a more cost effective bridge while still meeting the owner's requirements.

Network Tied Arches

The revised structural concept comprises twin network tied arches which are anchored at a pentagonal viewing platform where the two 82.3-m long spans meet. The arches are identical and were designed to maximize construction efficiency and thus to reduce cost. The ribs are circular with a constant radius of 68.6 m (Figure 2). With two doubly symmetric structures maximum repetition in fabrication could be achieved.

Figure 1 – Site Map

Figure 2 - Elevation
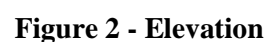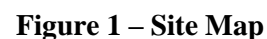
The steel members are all rolled Grade 350 MPa steel and only two different shapes were used: The arch ribs are fabricated from W360x162 (W14x109) I-sections. Tie girder and arch rib bracing both use WT230x56.5 (WT9x38) T-sections obtained by cutting in half the corresponding I-sections (i.e W18x76). In these shape designations the numbers indicate depth in mm (inches) and weight in kg/m (lbs/ft). All rolled steel members were made of weathering steel, therefore eliminating the need to paint the structure.

The ribs are arranged in a basket handle configuration with a transverse distance of 4 m at the spring line and 1.5 m at the crown (Figure 3). The bridge is designed to support pedestrian traffic and a light (9-tonne) maintenance vehicle. The deck is made of cast-in-place concrete varying in thickness from 203 mm to 229 mm and is supported on a stay-in-place corrugated metal deck (Figure 4). Four longitudinal tendons, each with four 15-mm diameter 1860-MPa seven wire strands keep the deck in compression under service loads and provide redundancy in case of tie girder fracture. At the knuckle joints a 22-mm thick load transfer plate with shear studs provides a load path between the thrust delivered from the arch ribs and the longitudinal tendon force (Figure 5).

Figure 3 - Completed Bridge
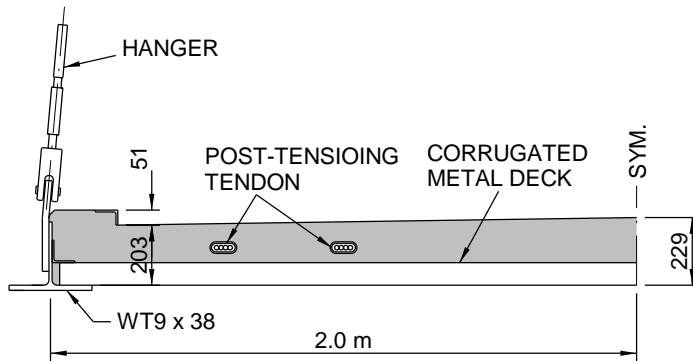
Figure 4 - Deck Cross Section

Hangers are galvanized 19-mm diameter bridge strand with simple, swaged pin connections at both ends. Each cable plane comprises 44 inclined hangers with the cable inclination gradually increasing from 57 degrees at one end to 71 degrees at the other end. The hanger spacing is constant along the arch rib at approximately 2 m. Connection plates are bolted to the stem of the WT tie girder at the lower end and they are welded between the flanges of the arch rib at the upper end. The lower anchorages are equipped with turnbuckles that provide +/- 50 mm of adjustability for cable force and geometry corrections (Figure 6).

Figure 5 - Knuckle Joint

Construction

The bridge was constructed on falsework that supported the tie girder and five preassembled arch units, each of which included two rib sections and the connecting bracing. Erection of the east span over Coyote Creek presented a particular challenge due to the requirement to remain outside the riparian corridor. A cable stayed falsework system was devised (Figures 7 and 8) that served to support the steel structure until arch closure and to brace the very slender arch rib during concrete placement. The network arch and the falsework system share the load of the wet concrete. However, the arch was much stiffer than the cable supported falsework and therefore picked up the majority of the superimposed load. The deck post-tensioning tendons were stressed prior to release of the falsework.

Due to the basket handle arrangement and the eccentricity of the support points for the stay-in-place metal deck, the tie girders have a

Figure 6 - Lower Hanger Connections

tendency to rotate under the weight of the fresh concrete. This was counteracted by attaching top and bottom flange of the stay-in-place forms to the tie girder to create a moment resisting connection. In addition tube steel was clamped to the flanges of the T-section for temporary additional rotational restraint.
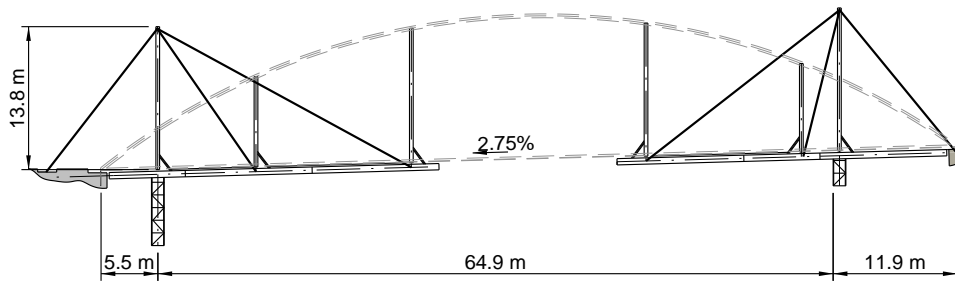
Figure 7 - Cable Supported Falsework

Final hanger adjustments to meet target cable forces and tie girder elevations were accomplished using light, hand-held tools. Cable forces ranged from 15 kN to 50 kN and were verified using a calibrated tensiometer. This instrument determines the axial force from the cable deflection over a given gauge length under a known lateral load. For forces below 30 kN, adjustments could be made by simply turning the turnbuckle with a pair of wrenches. For larger cable forces the turnbuckle had to be unloaded using a come-along before turning.

Figure 8 - East Arch Erection

The final hanger tuning schedule was developed using an influence matrix that relates unit length changes for each cable to the corresponding change in hanger forces and tie girder and arch rib elevations. The influence matrix approach presupposes that the structure behaves linearly. This condition is satisfied after the hangers have been sufficiently stressed under the weight of the concrete. An optimization program was used to find the hanger adjustments such that the error between predicted and target forces and elevations was minimized. With 2 x 44 cables and elevation control points at upper and lower cable terminals the influence matrix was quite large (88 x 264). However, this posed no particular problem and a solution could usually be found very quickly using a commercial spreadsheet program (Microsoft Excel). The length adjustment was translated into number of turns of the turnbuckle via the thread pitch. A complete 360-degree turn corresponded to a 3.6-mm hanger length change.

Conclusion and Acknowledgements

Dubbed "The Bridge To Adventure" it was opened in May of 2011, in time for the 50-year anniversary of the park (Figure 7). The bid price for the project was US$ 3.4 Million, significantly below the owners revised budget. With the network tied arch configuration it was possible to create a cost effective yet unique and attractive structure.

CONSTRUCTION OF THE TROJA BRIDGE IN PRAGUE

SUMMARY

The new Troja Bridge in Prague, has a main span of 200,4 m. The steel network arch with the bridge deck made of prestressed concrete represents an advanced bridge structure. The construction combines several technologies, including incremental launching and assembly of the arch from the bridge deck supported on temporary piers in the river.

1. INTRODUCTION

The new Troja Bridge is a part of the road system which is built within the Blanka Tunnel Complex. It carries 4 lanes of road, 2 tram tracks, which connect the city centre to the northern parts of the city, and also two wide pedestrian lanes used also by cyclists. The bridge crosses the Vltava River. The bridge has two spans; the main span crossing the river is 200.4 m long and the side span on the Troja riverside, which is 40.4 m long. The bridge is about 35 m wide. There are no supports located in the river.

In 2006, the architectural competition was organized by the client (City of Prague) and the winning project submitted by J. Petrak and L. Šašek (Mott MacDonald Prague) and by R. Koucky and L. Kabrt (R. Koucky arch. office, Prague). The bridge was open for traffic in October 2014.

2. DESCRIPTION OF THE BRIDGE

The main span and the side span are almost independent structures; there is an expansion joint over the pier no.2 between them. The basic theme of the elegance of the bridge lies in the ratio of the rise and span of the arch of 1/10. The slender network arch structure of the main span is tied arch (rise of the arch is only 20 m) with inclined hangers (Fig.1 and 2). The arch is made of steel – the central part is formed by a multiple box section (Fig.3), which is then divided into two legs on each side approaching the bridge deck at the ends of the span. The tie of the arch is located above the bridge deck and has a composite steel concrete cross-section. The longitudinal tensile force is carried mainly by 6 prestressing cables located inside the tie section. Each cable is composed of 37 strands 0.6" (15.7 mm). The bridge deck is completely made of prestressed concrete. The deck has transverse precast prestressed beams (C70/85) (Fig.4), which are suspended on the composite tie and which support a thin prestressed concrete slab (C50/60). The concrete deck carries the tram tracks and the road lanes, the pedestrian lanes are located on steel cantilevers.

The side span is completely made of cast in situ concrete (C50/60). The two longitudinal beams form the main load carrying element. The transversal beams have the same shape as those in the main span. The slab on the top is also very similar to that in the main span. The complete structure is prestressed longitudinally and transversely.

Fig. 1 Visualization of the complete bridge
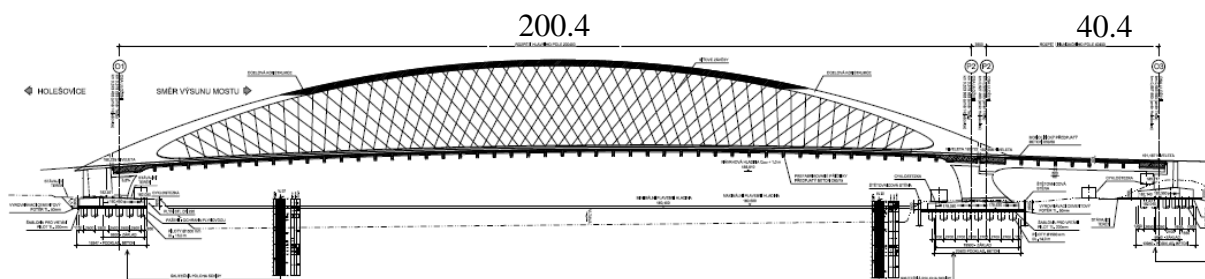
Fig. 2 Longitudinal section

3. CONSTRUCTION OF THE BRIDGE

The construction process of the main span was an object of many discussions. Initially the design assumed the process when first the arch with the tie and hangers would be built and then the bridge deck would be subsequently assembled and cast above the river. Due to the slenderness of the arch and very soft steel structure, the contractor saw many risks in such construction process. Also the economical evaluation was not favourable. Considering many alternatives, finally it was decided to build the deck first and then to assemble a steel arch using the deck as a fixed platform. A limited space on both sides of the river resulted in the following construction process.

The steel tie and the precast beams are assembled on the Holesovice riverside at the production yard. Since the bridge has no stiff longitudinal element, a temporary steel truss is used in order to work as a couple of stiff longitudinal beams. Top chord of the truss is definitive – the tie of the arch. The bottom chord and the diagonals are temporary and they will be removed after completion of the bridge. The 5 temporary piers were built in the river. The temporary truss with the precast transversal beams is assembled in the production yard
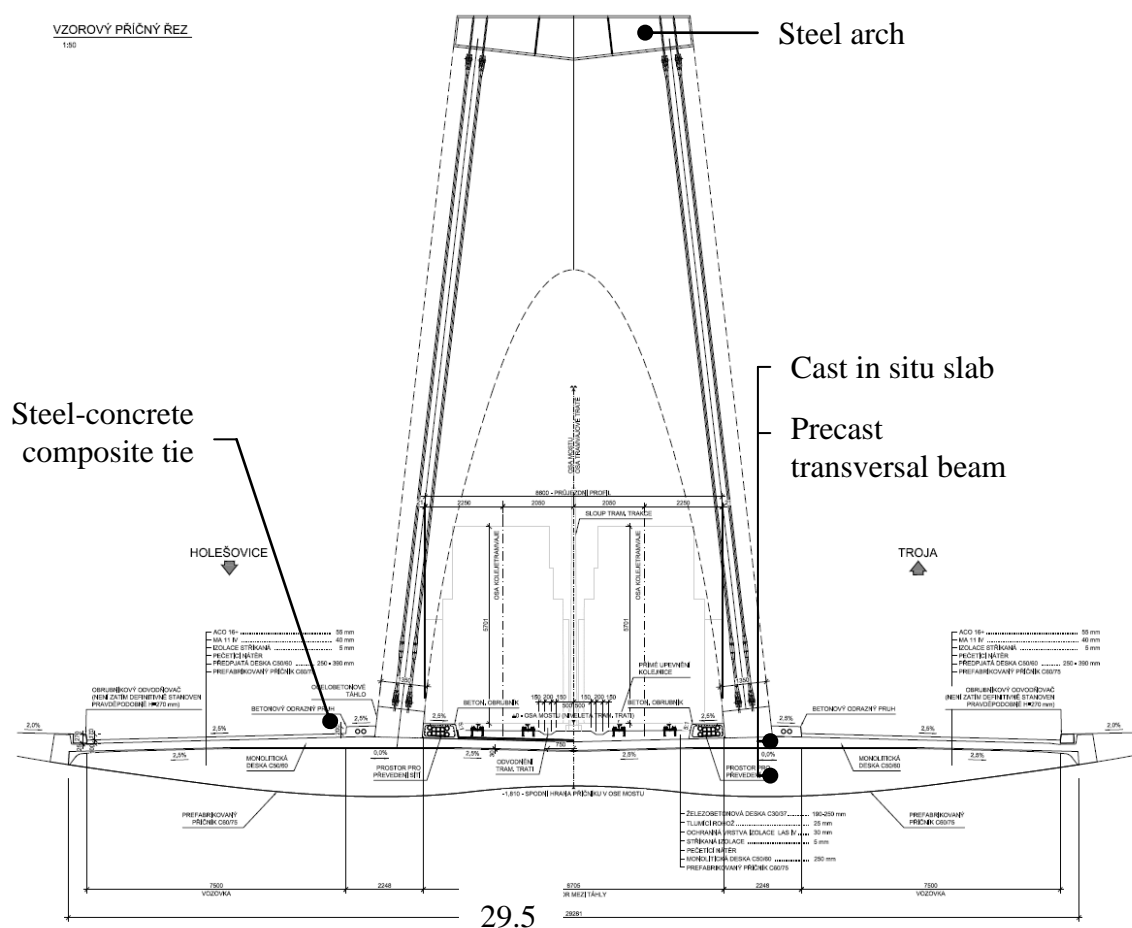
Fig. 3 Cross-section of the main span (without steel cantilevers)

Fig. 4 Precast transversal beams stored on the site prior to assembly

and then incrementally launched across the river in steps 16 m long using the temporary supports (Fig. 5). A short launching nose was used in front of the temporary steel truss. The hydraulic system with 6 cylinders with the capacity of 60 t each was used for moving the structures forwards. The stroke of cylinders was 250 mm. The structure was suspended on 8 steel bars. A special independent braking device was used when the steel bars where moved to the next position between individual launching stages. The structure was launched initially upwards to the slope of almost 7%, finally downwards due to the geometry of the bridge. The maximum capacity of 8 cylinders was used only for a short time. In the most of the launching process only 4 cylinders were able to push the structure. Very simple sliding bearings, where the bottom steel flange of the temporary truss moved directly on the teflon plates, were designed. No stainless steel sheets and inserted teflon plates were necessary. The friction coefficient varied between 2 and 4%.

After launching, the end elements (footings) of the arch will be connected to the tie and the end transversal beams will be cast in situ. The ends of the steel arch will be embedded into the end transversal beams. Their steel structure is extremely complex. The anchorage of 6 prestressing cables (each composed of 37 strands) represents a large force which has to be transferred to the steel arch. A number of stiffeners in the steel structure of the arch footing would be too high. Therefore the footing will be filled with high strength self-compacting concrete (C80/95). The self-compacting concrete will be pumped into the steel structure in several layers approximately 2 m thick. The concrete will distribute the forces from the anchorage into the entire steel section. The welded studs and steel reinforcement are used for connection of steel and concrete.

A complete deck slab will be subsequently cast from the ends to the centre of the main span in the sections again 16 m long. The formwork will be transported on ships, lifted and finally anchored to the transversal precast beams. The transversal prestressing will be activated immediately when the appropriate strength of concrete is achieved. When the slab is completed, approximately 1/3 of the longitudinal prestressing force will be activated. The slab supported on the temporary truss and on the 5 temporary supports in the river will form a platform with sufficient load carrying capacity for the assembly of the arch.

The steel parts of the arch will be delivered to the bridge deck and welded together into 3 parts of the arch. The temporary towers will be built on the deck. The three parts will be then lifted to the position and welded together in order to complete the arch (Fig.6). All the manipulations with steel elements on the arch will be carried out using hydraulic systems without any heavy crane. The hangers then may be installed and slightly prestressed to the stress level about 10% of their strength in order to avoid their excessive deflection. Very small deformations of the arch are expected in this stage which allows for a relatively precise specification of forces in hangers. All these activities will be executed in stable temperature conditions, preferably during the night. An extensive monitoring system will be used for detailed check of forces in individual hangers.
 Then it will be possible to activate the second level of the prestressing in the longitudinal direction, interrupt the bottom chord of the temporary truss and release the supports in the river. The bridge will deform and all the hangers become activated simultaneously. After that it will be possible to remove the temporary truss, finalize the prestressing (level 3) in the longitudinal direction and complete the works on the bridge (pavements, lighting, tram rails, etc.).

Fig. 5
Incremental launching of the steel truss and precast transversal beams

The side span will be cast in situ on the fixed scaffolding and then prestressed. The side span will be cast in three stages after the completion of the launching process and will be used for delivery of steel parts of the arch to the main span.
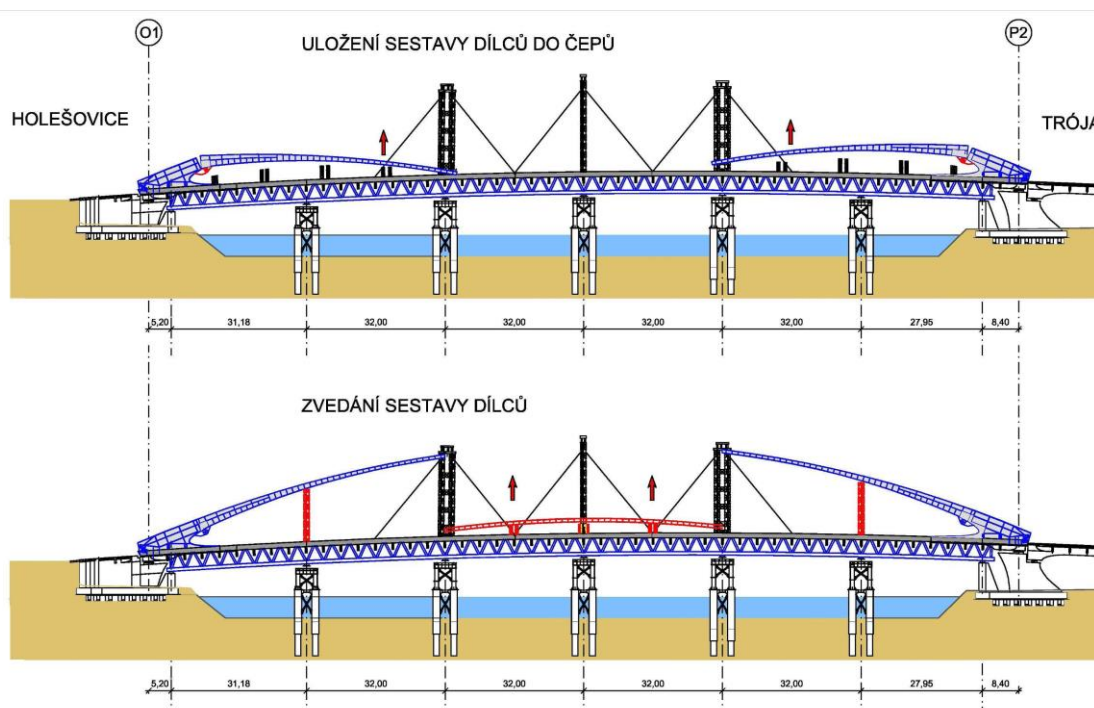
Fig. 6 Assembly of the arch

4. CONCLUSIONS

The structure of the new Troja bridge is rather complex. On the other hand the bridge is elegant and fits well into the environment of the recreation area of the city, which will be further developed and in some years it becomes a central areas. The city representatives decided to build this bridge because the arrangement and shaping of the bridge, with a span of 200.4 m, form an internationally unique structure. Taking into account the tradition of architectural development of the historical city of Prague it is certainly the right decision.

As of now (June 2011) the launching process has been completed. The steel elements of the arch are produced in two factories – in Metrostav and in MCE. The short construction time requires excellent organisation of all activities on the site and in factories. The structural performance is monitored using many strain and temperature gauges installed on the main parts of the structure. Additionally the detailed surveying provides information on deformations and movements of the structure. Execution of complex details is verified on models or trial structures. The executing team believes that it is able to finalize this sophisticated project on time and in adequate quality.

5. MAIN PARTICIPANTS OF THE CONSTRUCTION

Client:	City of Prague
Client represented by:	IDS – Engineering of transport structures.
General consulting office:	Mott MacDonald Prague, R.Koucky, arch. office
Consulting office – steel structure:	Excon, j.s.c.
Supervision and construction process analysis:	Novak and Partner, Ltd.
Contractor:	Metrostav j.s.c.
Steel production:	Metrostav j.s.c., MCE Czech branch in Slany
Concrete production:	TBG Metrostav, Ltd.
Other partners:	VSL-CZ, Ltd., SMP CZ, j.s.c., SM7, Ltd. MTEK, Ltd., CCE, Ltd. PONTEX, Ltd.

6. ACKNOWLEDGEMENTS

Amelia Earhart Bridge

Kansas City, MO, USA

Frank Blakemore and Natalie McCombs, HNTB,

Introduction

The states of Kansas and Missouri have constructed a replacement bridge for U.S. 59, crossing the Missouri River from Atchison, Kansas, to Winthrop, Missouri.

The location of the new structure parallels the existing Amelia Earhart Bridge and a railroad bridge just upstream from the existing bridge. The cross section of the bridge consists of four 3.66m traffic lanes and two 3m shoulders.
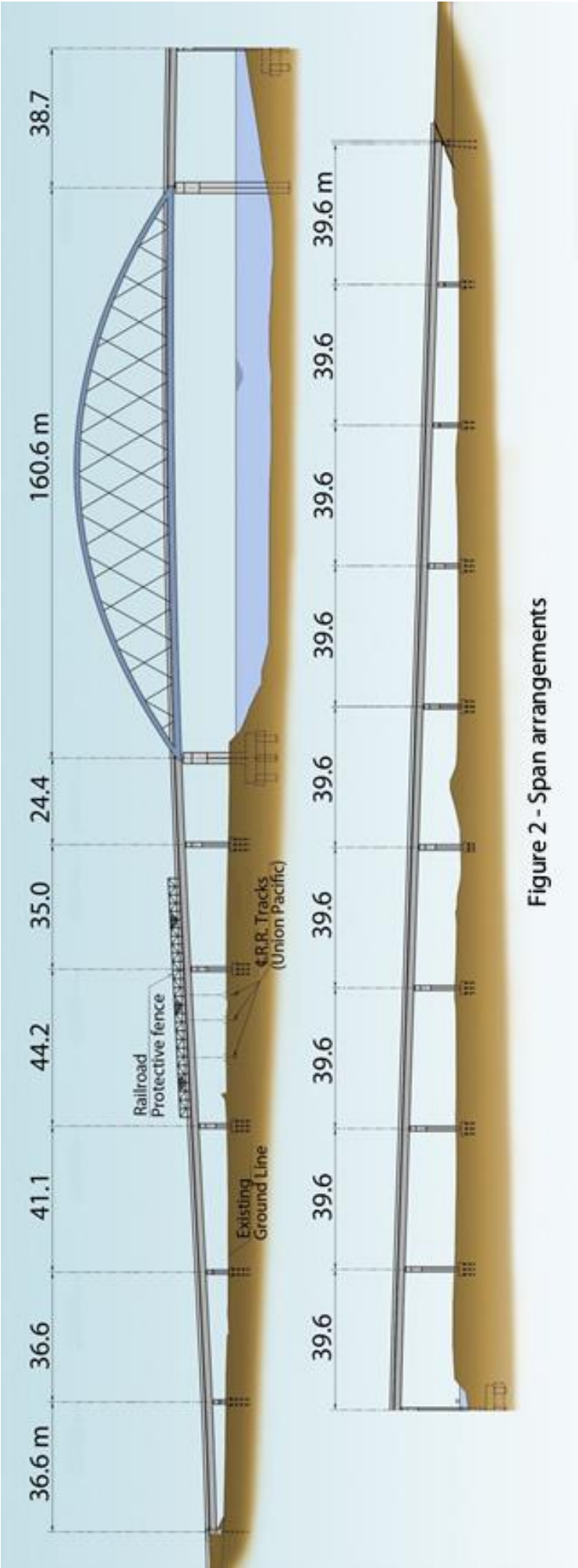

Figure 2 - Span arrangements

Figure 1 – Main Span of the Amelia Earhart Bridge

The new Amelia Earhart Bridge has an overall length of approximately 776m, which consists of 614m of 2m NU prestressed concrete I-girders spans for the approaches and a 160.6m steel network tied arch for the main span as shown in Figure 1 and Figure 2.

Tied Arch Description

The arch rib for the Amelia Earhart Bridge is rising 27.4m above the driving surface for a span-to-rise ratio of 5.83. The arch rib is a 1.22m wide by 1.372m deep welded box section and the tie girder is comprised of a 1.4m by 1.9m box section.

Both the arch rib and the tie girder sections are spaced 23.77m apart and are equipped with interior lighting, ventilation and access openings in the diaphragms for ease of inspection.

Figure 2 ARCH RIB SECTION
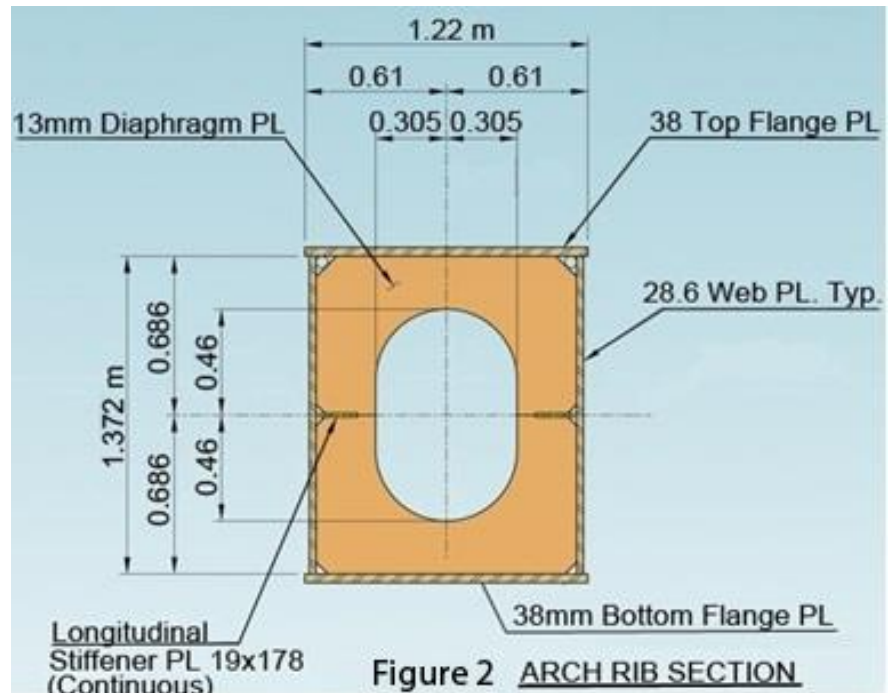
The tie girder is a tension member and therefore considered a fracture critical member. The design concern relating to the Federal Highway Administration (FHWA) advisory about tie girder redundancy was addressed by providing internal redundancy to the tie girder itself. Previous tie girders were welded built-up shapes. The welding of the members allowed crack propagation to spread through the entire section resulting in catastrophic failure.

For this structure, internal redundancy was created by bolting the top flanges to the webs, using L 200x200x25 angles as shown in Figure 3. If a crack were to occur in the flange or web plate, it would not continue through the adjacent plates for an entire loss of section. In addition, this structure was analyzed for stability with a loss of one of the flange plates.

The hangers used on the network arch are ASTM A586 bridge strand that have been pre-stretched and are spaced at 10.67m along the tie girder. The top hanger attachment is a standard open-ended socket attached to a fin plate. The fin plate is attached to a diaphragm plate that spans between the webs, which is then bolted to the arch rib web plates.

The bottom hanger castings are adjustable, open-ended sockets and were designed to allow for adjustments to the hanger lengths in the field. Both the top and bottom hanger castings are connected to the structure with stainless steel pins.

Figure 3 TIE GIRDER SECTION
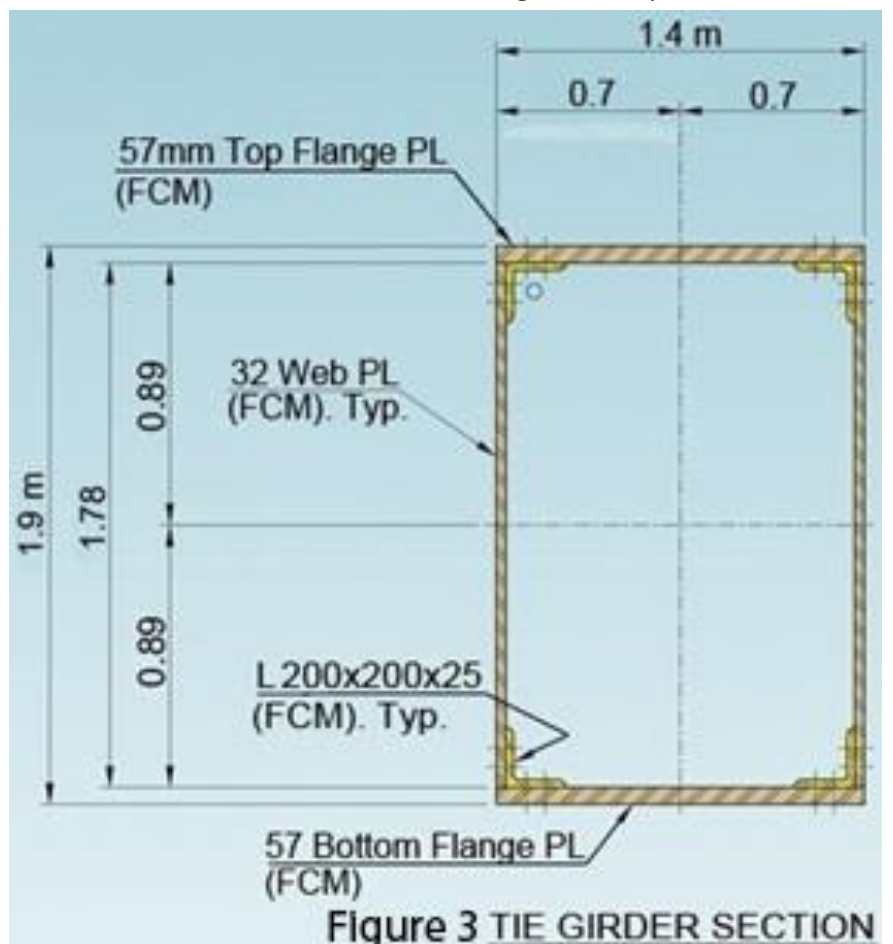
The floor system consists of floorbeams spaced at 10.7 spacing and corresponds to hanger lower anchorage location and the longitudinal members are stringers spaced at 2.5m apart. The intermediate floorbeams are 1.83m deep (minimum) I-beam sections with the top flange following the slope of the deck and the 24-inch rolled beam stringers are framed into the top flange. The end floorbeam is a 0.91m by 1.83m welded box section.

Typically, the stringers would sit on top of the floorbeams and movement would be allowed in the longitudinal direction. Due to vertical height restrictions, the stringers are instead framed into the floorbeams, where the top of the floorbeam and the top of the stringer are in line with each other as shown in Figure 4.

This framing method posed some additional challenges in the design and construction methods. The significant design concern involved allowing the elongation of the tie girder and floor system due to the arch rib thrust during construction.
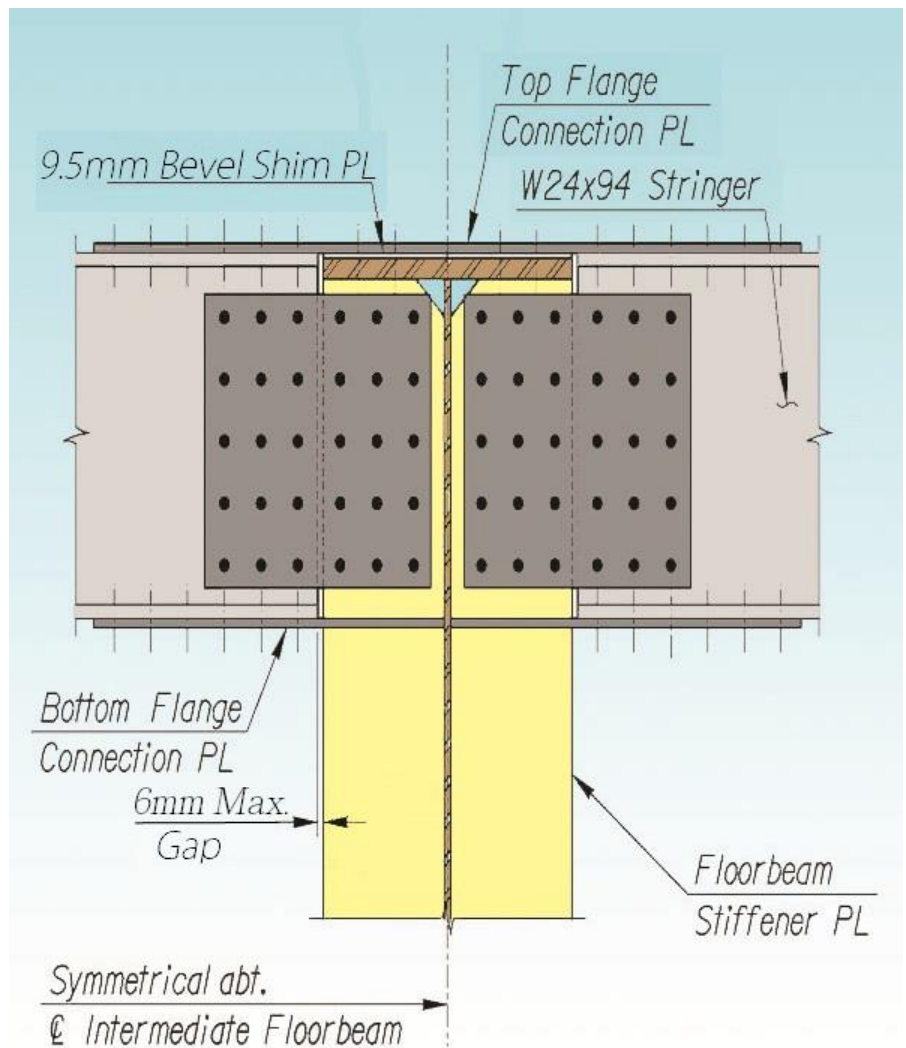
Figure 4 – Framed-In Stringer to Floorbeam Connection Detail

A majority of the elongation occurs during erection following the closure of the arch rib and tie girder. Additional dead load on the assembled arch rib and tie girder from the floorbeams, stringers and slab cause differential elongation in the tie girder and the stringers. To address this design issue, the stringer to floorbeam connections were detailed with slotted holes in either the connection plate or the web. The slotted holes occur at every other floorbeam and allowed the structure to elongate without inducing an axial load component into the stringers under dead load. The bolts in these connections are to be installed finger tight during the steel erection.

The slab is to be placed in approximately 15.2m sections with a closure pour opening over the slotted stringer connections. Once the entire deck was placed, with the exception of the closure pours, the stringer connection bolts will be fully tightened and the closure pours over the expansion connections will be poured. This method alleviated the elongation forces in the stringers

Advantage of Network Tied Arch Bridges
Historically, arch structures in the United States have consisted of arch ribs with vertical hanger systems. The downside to the conventional hanger arrangement is a slightly less redundant system, where the failure at one hanger location resulted in large tie girder local moments at the hanger location.

A network hanger system increases redundancy by having two hangers connect to the tie girder at the same location and both angle away from each other. A network hanger arrangement distributes the resulting hanger loss forces to the adjacent cable and still provides support to the tie girder at the hanger location. Similarly, the arch rib bending moment is reduced significantly from a vertical hanger system for hanger loss loads.

During preliminary design phase, a network hanger system was compared to a vertical hanger system and it was found that the network hanger system provides the following benefits: increased redundancy, improved safety to the public, and a cost savings of about 3% of the total project cost.

Construction

The entire project, including 655m of approach spans and associated roadway work, was bid at $59.4 million by Archer Western Contractors and construction began in June 2009.

The main arch span was built using erection towers at each end of the arch. The towers were anchored with back stays that connected to the adjacent pier and the fore stays attached to the arch rib segments. Tie girder segments were cantilevered out, and then the arch rib segment was placed and connected to the tie girder segment using the permanent hangers.

Erection continued until both sides of the arch and tie girder connected in the middle. The tie girders were connected prior to placement of the arch keystone piece, Figure 5.

The floorbeams and floor lateral bracing were connected as the tie girder segments were erected. The stringers were installed with the slotted connections finger-tight. As mentioned previously, the concrete slab was poured in stages to minimize stringer axial forces due to construction.

Figure 5 – Erection of the Arch Keystone Piece

The 18cm thick slab sections, shaded grey in Figure 6, were poured and the stringer connection bolts were fully tightened. The 18cm thick closure pours, shaded green in Figure 6, were poured and then a 38mm wearing surface was placed to allow for a smooth riding surface on the finished deck.

Hanger adjustments were made to meet the target hanger tensions designated in the erection analysis. Despite major flooding on the Missouri River during construction, the project opened to traffic in December 2012.

Figure 6 – Slab Pouring Sequence

Acknowledgements

Literature.

Andersen, G. (1979) "Brugerveiledning for FEMOPT. Program for optimering af bærende konstruksjoner." ("Users' manual for FEMOPT. Program for the optimising of load-carrying structures," in Danish.) Note no. 7902, Inst. Bldg. Tech. Struct. Engrg., Aalborg Univ. Centre, Aalborg, Denmark. 101 pages.

Backer, H., Pauw, B., Corte, W. and Bogaert, P. (2004) "Force Variation and Slackness in Tied Arch Bridges with Crossing Hangers." Arch Bridges IV, Barcelona, Nov. 2004. Advances in Assessment, Structural Design and Construction. P. Roca and C. Molins (Eds.). pp. 651-660. ISBN: 84-95999-63-3

Bast "Leitfaden zum Anhang II-H, Hänger von Stabbogenbrücken, des DIN-Fachberichtes 109 (Ausgabe 2008)" Instructions on hangers in tied arch bridges.

Berger, O. (1955) "Beurteilung des Tragvermögens einer Großen Eisenbahnbrücke auf Grund von Spannungsmessungen und Berechnung als hochgradig (38 fach) statish unbestimmtes System". (Evaluating the load carrying capacity of a big railway bridge and calculating it as a highly (38 times) indeterminate system. In German.) pp. 526-535.Bauplanung – Bautechnik H. 12 Des.1955. ISBN: 624.21 : 625. 183.3

Bretting, A.E. (1935). "Inclined Hangers Impart Slenderness to Ribs of 470-Ft. Concrete Arch". Engineering News-Record. April 25, 1935. pp. 577-580.

Bretting, A.E. (1936). "Über Bogentrträger mit schräg gestellten Hangestängen." ("On arches with inclined hangers," in German.) Internat. Vereiniung für Brückenbau und Hochbau. Zweiter Kongress, Schlussbericht 1936. pp. 514-515.

Brunn, B. and Schanack, F. (2003) "Calculation of a double track railway network arch bridge applying the European standards" Graduation thesis at TU-Dresden. August 2003. 320 pages. A revised version of this thesis can be found at http://home.uia.no/pert/ under the button „Masters Theses".

Brunn, B., Schanack, F., Steimann, U, (2004) "Network Arches for Railway Bridges." Arch Bridges IV, Barcelona, Nov. 2004. Advances in Assessment, Structural Design and Construction. P. Roca and C. Molins (Eds.). pp. 671-680. ISBN: 84-95999-63-3

Cederwall, K. & Fransson, L. (1979) "Förstärkning av ett istäckes bärförmåga med armering." (Strengthening of Sheets of Ice by Reinforcement." In Swedish.) Publ. 79:1 från avd før Konstrutionsteknik, Högskolan i Luleå. (Publ 79 from Department of Structural Design, Högskolan i Luleå.)

Chan, M., Presland, Rob., Romanes, Mike. (2008) "New Zealand's First Network Arch Bridge." Conference paper for the 4th New Zealand Metal Industry Conference, hosted by HERA in October 2008. All conference papers were published on a CD.

Cronholm, A. (1963) "Die versteiften Stabbogebrücken in Schweden." (The Stiffened Arch Bridges in Sweden. In German) Acier, January 1963 p. 6-9.

Fiedler, E. and Ziemann, J. (1997), "Die Bogenbrücke über die Saale bei Calbe – eine Brücke mit besonderer Bogenform", (The Arch Bridge over the Saale River at Calbe – a Bridge with an Unusual Arch Shape. In German.), Stahlbau, Vol. 66, No. 5, 1997, pp. 263-270, Dokumentation 1997, pp. 329-337, ISBN 3-927535-04-4.

Fiedler, E. (2005) "Der Stabbogen bei stählernen Straßenbrücken-Entwicklungstendenzen der letzen 50 Jahre – Teil I" Stahlbau 2005 Heft 2, pp. 96-107. Teil II Heft 5, pp. 281-294.

Geißler, K., Steimann, U., Graße, W. (2008) "Netzwerkbogenbrücken – Entwurf, Bemessung, Ausfürung" (Network Arch Bridges – Design and Execution.) Stahlbau H. 3, 2008 pp. 159-171.

Graße, W., Teich, W., Tveit, P., Wendelin, S. (2004) Arch Bridges IV, Barcelona, Nov. 2004. Advances in Assessment, Structural Design and Construction. P. Roca and C. Molins (Eds.). pp. 681-690. ISBN: 84-95999-63-3

Graβe, W. and Tveit, P. (2007) "Netzwerkbogenbrücken – Geschichte, Tragwerhalten und ausgeführte Beispiele" (Network Arches – History, Function and Built Examples. In German.) 17. Dresdner Brückenbausymposium. 13. March 2007. ISSN 1613-1169, ISBN 3-86005-542-9. pp. 183-198.

Günter, G., Hortmanns, M., Schwartzkopf, D., Sedlacek, G. and Bohmann, D. "Dauerhafte Ausfürung von Hängeranschlüssen an stahlerne Bogenbrücken". (Durable Attachment of Hangers in Steel Arch Bridges.) Stahlbau 69 (2000) Heft 11, pp. 894 – 908.

Herzog, Max. (1975). "Stahlgewichte moderner Eisenbahn- und Straßenbrücken." (Steel Weights of Modern Rail and Road Bridges.) Der Stahlbau 9/1975. pp. 280-282.

Kahman, R. and Beisel T. (1979) "Eine außergewöhnliche Montagemethode für die Bogenbrücke bei Straubing." Der Stahlbau 1979. Heft 4. Berlin. pp. 110-115.

Krück G. E. (1946) "Eisenbeton-Strassenbrücke über den Mänam Pasak bei Ayuthia, Siam." ("Concrete road bridge over the Mänam Pasak at Ayutia, Siam," in German.) Schweizerische Bauzeitung 1946. Vol. 127, pp. 139-146, Vol. 128, pp. 6-9, 15-19, 27-28.

Kungliga Väg- och vattenbygnadsstyrelsen. (1954) "Bro över Lulefjärden vid Luleå." ("Bridge over Lulefjärden vid Luleå. In Swedish. ") 76 pages.

Leonhardt, (1991) "Developing guidelines for aesthetic design." *Bridge aesthetics around the world*, M. P. Burke Jr. et al.,eds., Nat. Res. Counsil, Washington, D.C. pp. 32-57.

Nakai, H. et al. (1995) "Proposition of Methods for Checking the Ultimate Strength of Arch Ribs in Steel Nielsen-Lohse Bridges." Stahlbau 64 (1995) Heft 5, pp.129-137.

Naruoka, M., Itoh, K., and Matsukawa, A. (1977) "Nielsen System Bridges in Japan." Pfluger-Festschrift Hannover, Germany, 1977. pp. 193-202.

Nielsen, O. F. (1929) "Foranderlige Systemer med anvendelse på buer med skraatstillede Hængestenger." ("Discontinuous systems used on arches with inclined hangers", in Danish.) 121 pages. Gad Copenhagen. Ph.D. thesis.

Nielsen, O. F. (1932) "Bogenträger mit Schräg gestelten Hängestangen." ("Arches with inclined hangers," in German.) Internationale Vereiningung f. Brückenbau und Hochbau. Abhandlungen 1, 1932. pp. 355-363.

Nielsen, O. F. (1936) Same title and periodical as above. Abhandlungen 4, 1936, pp. 429-438.

Ostenfeld, C. (1976) "Christiani & Nielsen, Jernbetonens danske pionerer" ("Christiani & Nielsen, The Danish Pioneers of Reinforced Concrete," In Danish.) Polyteknisk Forlag, Lyngby, Denmark. 245 pages.

Räck, M. (2003) "Entwurf einer kombinierten Straßen-Eisenbahn-Netzwerkbogen-brücke" (Draft of a Network Arch for a Combined Road and Railway Bridge) Graduation thesis at TU-Dresden. August 2003. 237 pages. A revised version of this thesis can be found at http://home.uia.no/pert/ under the button "Masters Theses".

Šašek. L. (2005). "Getting on the Network. Innovation in arch design" BRIDGE Design § Engineering. v. 11, no 40, pp. 39-40

Šašek. L. (2006). "Less is More", Civil Engineering, The magazine of the American Society of Civil Engineers - ASCE, April 2006, pp. 50-55.

Schanack, F. (2008) "Puentes en Arco Tipo Network" (Network Arch Bridges) Doctoral Thesis. Departemento de Ingenieria Estrutural y Mecánica, Escuela Técnica de Caminos, Canales y Puertos, Universidad de Cantabria, Santander, Espñia, 2008. 370 pages.

Schanack, F. (2008a) " Puentes en Arco Tipo Network. Network Arch Bridges. Tesis doctoral. Departemento de Ingeneria Estructural y Mecánica, Escuela Técnica Superior de Ingeniería de Caminos, Canales y Puertos. Universidad de Cantabria, Santander 2008.

Schanack, F. and Brunn, B. (2009): "Analysis of the structural performance of network arch bridges" The Indian Concrete Journal 83(1), ACC limited, 2009. pp. 7-13.

Schanack, F. and Brunn, B. (2009a) "Netzgenerierung von Netzwerkbogenbrücken" ("Creation of hanger arrangements of Network arches" In German.) Stahlbau 2009. H7. pp. 477-483.

Schlaich J., Fackler T., Weißbach M., Schmitt V., Ommert C., Marx S., Krontal L. (2008) "Leitfaden Gestalten von Eisenbahnbrücken" (Guide for the Design of Railway Bridges. In German) Fischer Druck GMBH, Peine

Steere, P. and Wu, Y (2008) "Design and Construction of the Providence River Bridge" 25th Annual International Bridge Conference Engineers' Society of Western Pennsylvania Pittsburgh (PA). June 2-4, 2008. 8 Pages.

Steimann, U. (2002) "Berechnung und Konstruktion einer stählernen Eisenbahn-Stabbogenbrücke mit Netzwerkhängern" (Calculation and Design of Network Arch Railway Bridge of Steel. In German.) Graduation thesis at TU-Dresden. September 2002. 270 pages. A revised version of this thesis can be found at http://home.uia.no/pert/ under the button "Masters Theses".

Stein, Ph. (1951) "100 Jahre GHH" ("100 Years of GHH Bridge Building") Publisher: Gutehoffnungshütte Aktiengesellschaft, Oberhausen.

Stein, Peter and Wild, H. (1965). "Das Bogentragwerk der Fehmarnsundbrücke." ("The arch of the Fehmarnsound Bridge" In German.). Der Stahlbau, 34(6) Berlin, B.R.D. pp. 71-186.

Teich, S. (2004) "Fatigue Optimization in Network Arches" Arch Bridges IV, Barcelona, Nov. 2004. Advances in Assessment, Structural Design and Construction. P. Roca and C. Molins (Eds.). pp. 691-700. ISBN: 84-95999-63-3.

Teich, S. and Graβe, W. (2004a) "Beitrag zur Optimierung von Hängeranschlüssen Stählerner Stabbogenbrücken" ("Contribution to the optimisation of ends of hangers in steel arch bridges" in German) Stahlbau, 2004, H. 12. pp. 1021-1030.

Teich, S. and Wendelin, S. (2001) „Vergleichsrechnung einer Netzwerkbogenbrücke unter Einsats des Europäishen Normenkonsepts." (In German). Graduation thesis at TU-Dresden. August 2001. 300 pages. A revised version of this thesis can be found at http://home.uia.no/pert/ under the button "Masters Theses".

Teich. S. (2012) "Beitrag zur Optimierung von Netzwerkbogenbrüken" (Contrbution to Optimizing Network Arch Bridges). PhD. thesis. ISSN 1613-6934. The whole text can be found at http://www.qucosa.de/fileadmin/data/qucosa/documents/8604/Dissertation_Teich.pdf

Tveit, P. (1959) "Bogebruer med skrå krysstilte hengestenger." ("Arch bridges with inclined intersecting hangers," in Norwegian.) Ph.D. thesis presented at the Tech. Univ. of Norway. 64 pages, 78 drawings.

Tveit, P. (1964) "Nettverkbogar, ein ny brutype". ("Network Arches, a New Type of Bridge.) Bygg, Vol. 12, May 1964, pp.105-113.

Tveit, P. (1966) "Design of Network Arches." Struct. Eng:, 44(7). London, England, pp. 247-259.

Tveit, P. (1973) "Network arch in double track railway bridge." Presented at Nordic Research. Days for Steel Struct., held in Oslo, Norway, Pre-print V713. (The publication is much the same as: "Report 7205." Ren og Anvendt Mekanik, Danmarks Ingeniørakademi, Bygningsafdelingen, Aalborg, Denmark. 35 pages.

Tveit, P. et al. (1978) "Network arches." 1st ed., Civil Engineering. Department., Univ. of Houston June 26 1978. 93 pages. Revised edition reprinted at inst. of Bldg. Techn. Struct. Engrg., Aalborg Univ. Centre Aalborg, Denmark, 1980. ISSN 0105-8185 NOTE NO. 8007. Including handout for the Poster Session of IABSE's 11. Congress, Vienna, September 1980. 45 pages.

Tveit, P. (1980a) "Network arches." Handout for poster session of IABSE's 11th Congress, held at Vienna, Publ. by inst. Tech. Struct. Engineering., Aalborg Univ. Centre, Aalborg, Denmark. 45 pages.

Tveit, P. (1980b) "Network Arches." 11th IABSE Congress, held in Vienna, Austria, *Final Report*, IABSE, ETH-Hönggerberg, CH-8039, Zürich, Switzerland, pp. 817-818.

Tveit, P. (1981) "25 år med nettverkbogar. Resultat og perspektiv." ("25 years with network arches. Results and perspectives," in Norwegian.) Guest lect. at Tech. Univ. of Norway, printed at Inst. of Bldg, Tech. Struct. Engineering., Aalborg University. Aalborg, Denmark. 40 pages. IFB/A 8101.

Tveit, P. (1984) "Economic design of network arches" Handout for the poster session of IABSE's 12th congress, Vancouver, September 1984. Published by Inst. Bldg. Tech. Struct. Engineering., Alborg Univ. Centre, Aalborg. 32 pages. ISSN 0105-7321 R8405.

Tveit, P. (1984a) "A network arch with four sets of hangers designed to be cast and erected on ice floating between piers." Handout for the poster session of IABSE's 12th Congress, held at Vancouver, Canada. Publ. by Inst. of Bldg. Tech. Struct. Engineering., Alborg Univ. Centre, Aalborg, Denmark. 106 pages.

Tveit, P. (1987) "Considerations for the Design of Network Arches." Journal of Structural Engineering, Vol. 1113, No.10, October, 1987. ©ASCE, ISSN 0733-9445/87/0010-21897 Paper No. 21892. pp. 2189-2207.

Tveit, P. (1992) "Network Arches in Perspective" Guest Lecture held at the technical Universities in Prague and Vienna, March 1986. Published by Agder College of Engineering, Norway. Report 2/92 ISSN 0801-6313, 36 pages

Tveit, P. (1999a) Comparison of Steel Weights in Narrow Arch Bridges with Medium Spans. A Note on the Optimal Design of Network Arches. Published by Agder University College. Publication Series No. 47. 8 pages. ISSN: 0806-5942 ISBN: 82-7117-378-2.

Tveit, P. (1999b) Netværksbuer – en interessant brotype. Foredrag i København , Gøteborg og NTNU. March 1999. (Network arches – an interesting type of bridge. Lectures in Copenhagen, Gothenburg and NTNU. March 1999). Published by Agder University College 1999. 30 pages. ISSN: 0806-5942, ISSN: 0806-5942, ISBN: 82-7117-394-4.

Tveit, P. (1999c) "Comparison of Steel Weights in Narrow Arch Bridges with Medium Spans" Stahlbau (1999) H9. pp. 753-757.

Tveit, P. (2000) A bridge suitable for India? ICI journal, Indian Concrete Institute. Vol. 1, April – June 2000 no.1, pp. 7-10.

Tveit, P. (2000a) "Ideas on Downward Arched and Other Underwater Concrete Tunnels." Tunneling and Underground Space Technology. Volume 15, Number 1, January-March 2000. ISSN 0886-7798 (799)

Tveit, P. (2001) "How, Why and Where for the Network Arch, the World's Most Slender Arch Bridge." Contribution to ARCH'01. Third international arch bridge conference. pp. 161-167. ISBN 2-85978-347-4.

Tveit, P. (2002) "Optimal design of network arches". Contribution to the IABSE Symposium in Melbourne 2002. 13 pages. ISBN 3-85748-107-2

Tveit, P. (2002a) "The Network Arch. Annual lecture of the ICI". Deliverered at the Annual Seminar of the Indian Concrete Institute in New Delhi in 2002. pp. 1-103.

Tveit, P. (2002c) "Konstruksjon av ein optimal nettverkboge med vekt på produksjons- og vedlikehaldskostnadar" (Design of an optimal network arch, stressing cost of production and maintenance. In Norwegian.) Brukonferansen 2002. Moderne stålbruer. Vika atrium I Oslo 30. – 31. October. Published by Statens vegvesen, Vegdirectoratet

Tveit, P. (2003) "Preliminary Design of Network Arch Bridges" The Bridge & Structural Engineer. Workshop on Innovation in Bridge Engineering, New Delhi, March 22-23, 2003. pp. I-1 to I-17

Tveit, P. (2003a) "Erection of Optimal Network Arches" Contribution to 8th International Conference on Innovation and Planning, Design and Construction Techniques in Bridge Engineering." Organised by : Indian Institution of Bridge Engineers. 26-27 April, 2003. Hyderabad, India. Vol.1 pp. 1-27

Tveit, P. (2003b) "Competitive Design of Network Arches". In: Structural Research. Anniversary volume honouring Péter Lenkei. Bársony, J. Ed. Pollack Mihály Faculty of Engineering. Pécs University, Hungary. pp. 225-233.

Tveit, P. (2004) "Introducing the Optimal Network Arch." Bridges in the Danube Basin. Procedings of the 5th International Conference on Bridges across the Danube 2004. Novi Sad/Serbia & Montenegro/ 24 – 26 June 2004. Bratislav Stipanic. Ed. Pub. by Euro Gardi Group, Rumenacka 17, Novi Sad. pp. 257-266

Tveit, P. (2004a) "Reduce Costs by Building Optimal Network Arches" Keynote lecture at Arch Bridges IV, Barcelona, Nov. 2004. Advances in Assessment, Structural Design and Construction. P. Roca and C. Molins (Eds.). pp. 26-37. ISBN: 84-95999-63-3

Tveit, P. (2004b) "Network arches for road bridges." Co-authors: Wolfgang Graβe, Stephan Teich, Stephan Wendelin. Arch Bridges IV, Barcelona, Nov. 2004. Advances in Assessment, Structural Design and Construction. P. Roca and C. Molins (Eds.). pp. 681-690. ISBN: 84-95999-63-3

Tveit, P. (2004c) "Erection of network arches." Co-authors: Mathias Räck, Frank Schanack. Arch Bridges IV, Barcelona, Nov. 2004. Advances in Assessment, Structural Design and Construction. P. Roca and C. Molins (Eds.). pp. 661-670. ISBN: 84-95999-63-3

Tveit, P. (2004d) "Optimal Network Arches for Rail and Road Bridges." 9th International Conference on Current Trends in. Aqueducts, Road, Rail and Marine Bridges, 20-22 December 2004.– Hyderabad, India. Organised by: Indian Institution of Bridge Engineers. pp.185-195.

Tveit, P. (2005) "Optimal Network Arches for Rail and Road Bridges" Contribution to 4th International Conference on New Dimensions in Bridges, 24-26 October 2005 in Fuzhou, China.

Tveit, P. (2007) "Visit to the Steinkjer network arch 44 years later". ARCH'07, 5th International Conference on Arch Bridges. Madeira, 12-14 September 2007. pp. 305-314, © University of Minho. Portugal. ISBN: 978-972-8692-31-5

Tveit, P. (2006) "Optimal Network Arches for Coastal Regions". International conference on bridges, Dubrovnik, May 2006. pp. 721-728. ISBN953-95428-0-4

Tveit, P. (2008a) "Concrete in the optimal network arch." Proceedings of the International *fib* Symposium 2008, Amsterdam, 19-21 May 2008. Editors: Walraven, J. and Stoelhorst, D. p. 164.

Tveit, P. (2008b)."The Network Arch and how to Erect it." Żabljak,03-07. Marta 2008. Univerzitet Crne Gore, ISBN 987-86-82707-14-1

Tveit, P. (2008c) "About The Network Arch". This is a lecture that IABSE has asked the author to give for a group of lectures that can be found at http://elearning-iabse/l20 Can also be found at the author's home page http://home.uia.no/pert/ under the button "My Publications".

Tveit, P. (2008d)."India Needs Network Arches" Contribution to national seminar on "Recent Advances and Future Trends in the Design and Construction of Bridges." 11-13 July, 2008. Organised by; National Bridge Research & Development Center, Hyderabad, Indian Institution of Bridge Engineers, Mumbai. Published by The Indian Institution of Bridge Engineers, 9,Gr. Fl.Greenfield CHS, Off Sitaladevi Tempel Road,Mahim, Mumbai-400 016, E-mail: iibe@vsnl.com. pp. 29-46.

Tveit, P. (2009a) "Submerged Floating Tunnels for Norwegian Fjords." Proceedings of the fifth Symposium on Strait Crossings, Trondheim, Norway June 21-24, 2009. ISBN 978-82-92506-69-1 pp. 515-520.

Tveit, P. (2009) "Efficient Utilisation of Network arches." Proceedings of the fifth Symposium on Strait Crossings, Trondheim, Norway June 21-24, 2009. ISBN 978-82-92506-69-1 pp. 509-514.

Tveit, P. (2009) "India Needs Network Arches". Civil Engineering & Construction Review. New Delhi, August 2009. Pp. 50-60.

Tveit, P. (2010) "Optimal network arches for road and rail bridges". ARCH'10. 6th International Conference on Arch Bridges. Fuzhou, Fujian, China. October 11-13, 2010. Editors Baochun Chen, Jiangang Wei. pp. 271 – 277.

Tveit, P. and Pipinato, A. *University of Padoa* (2011) II progretto di Ponti di tipo network-arch. (The network-arch bridge) In Italian. Costruzione Matalliche. Mar/Apr 2011, Pp. 47-56. ISSN nº 0010-9673.

Tveit, P. (2013) "The Network Arch. Bits of Manuscript in March 2014 after Lectures in 50 Countries." Can be found at http://home.uia.no/pert/ under the button "The Network Arch". This publication will be updated as long as the author is active. The same goes for the present publication "Systematic Thesis on Network Arches"(Tveit 2013a)

Tveit, P. (2014) "About the Network Arch" a revised version of Tveit, P. (2008c) It can be found at home.uia.no/pert/data/IABSE lecture revised.pdf.

Yoshikava, O. et al. (1993) "Construction of the Shinamadera Bridge" Stahlbau 63 (1993), Heft 5, pp.125-136.

Wollmann, G. et al. (2007) "Construction of Tied Arch Bridge Across Ohio River and Blennerhassett Island" Proceedings, 24th Annual International Bridge Conference, Pitsburgh, Pennsylvania, June 4-6

Wollmann, G., Zoli, T., (2008) "Bridge Across Ohio River and Blennerhassett Island". Structural Engineering International, Vol. 18, No.1, Feb. 2008, p. 28 – 30.

Zoli, T. and Woodward, R. (2005) "Design of Long Span Bridges for Cable Loss." IABSE Symposium, Structures and Extreme Events, September 14-17, Lisbon, Portugal.

J. A SHORT HISTORY OF NETWORK ARCHES AS EXPERIENCED BY PER TVEIT

My mother noticed that I was better at building with toy bricks than my brothers and sister. She encouraged me when I considered studying civil engineering.

In the summer of 1955 I worked on my master's thesis at NTH (Norway's Technical University) in Trondheim. I was supposed to study methods of calculating arch bridges with inclined hangers. See fig. J1 and J2. I managed to find the relevant literature. [Nielsen 1929], [Krück 1946], and [Bretting 1936 and 1935]

My professor, Arne Selberg, was one of four brothers who became professors. The three others became professors of mathematics. Arne Selberg was the number one bridge builder in Norway. He was also professor of statics, but later changed to steel. He thought that the bridge types in fig. J1 and J2 might some day be built in Norway.

At the time I did not know anything about the importance of being connected to people, but it deserves a mention that his father and my father came from the same district in Norway. Later our mothers became friends. Selberg and I never became good friends. See also J-3.
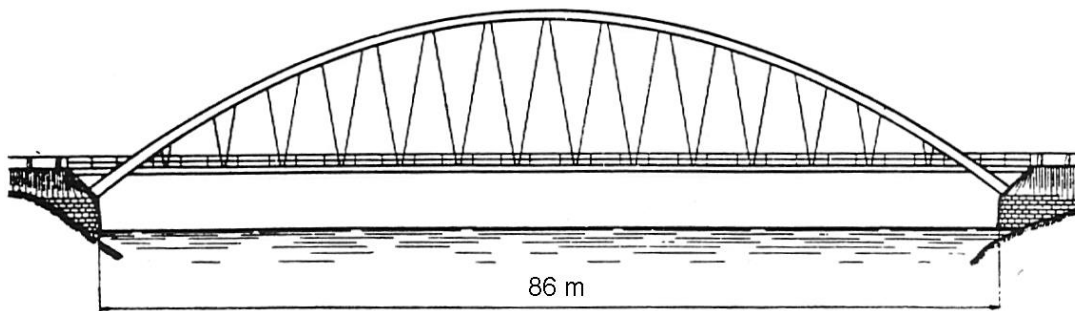

Fig. J1. Bridge over Øster-Dalelven in Sweden. Design of this bridge started in 1921. (Nielsen 1929)
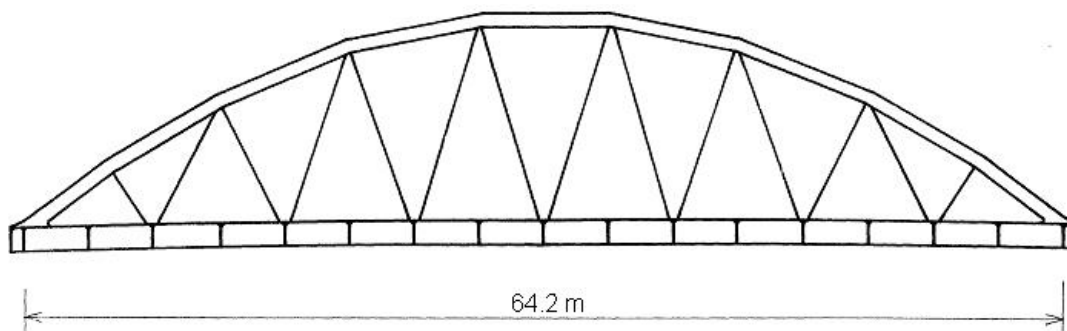
Fig. J2. Tied arch bridge over Mänam Pasak, Thailand. Designed by G. E. (Krück 1946). Finished 1942.

The spans function better if only a few hangers relax. The hanger's resistances to relaxation increase if the angle between tie and hanger is reduced. Since loads had become bigger and concrete had become stronger, it would be advantageous to use a smaller angle between hangers and tie. For the distance between the nodes in the arch and tie to remain small, the hangers must cross each other many times.

I still have a piece of paper where I outline the idea and say that it is not good. Later I accepted the idea and spent a part of my master's thesis on describing it. Professor Arne Selberg understood the idea at once, but he thought that I had devoted too much time to it in my thesis.

Here I would like to include parts of a chapter from my master's thesis. The heading is: "<u>A new type of bridge with inclined hangers and stiff arch and tie.</u> The chapter continues "While working with this task, I have come to think of a new type of bridge (see fig. J3, next page) which in my opinion has certain advantages over related bridge types. The bending moments in the chords are reduced towards zero….".

Later it says: "The hangers' tendency to relax can be counteracted by giving them a smaller angle with the tie. It is not the distance between the hangers, but their slope that decides how easily they relax. Proof: They relax because they can not counteract the tendency of the arch to move parallel to the tie."

Fig. J3. These network arches are fig. 30 in the author's master's thesis from 1955

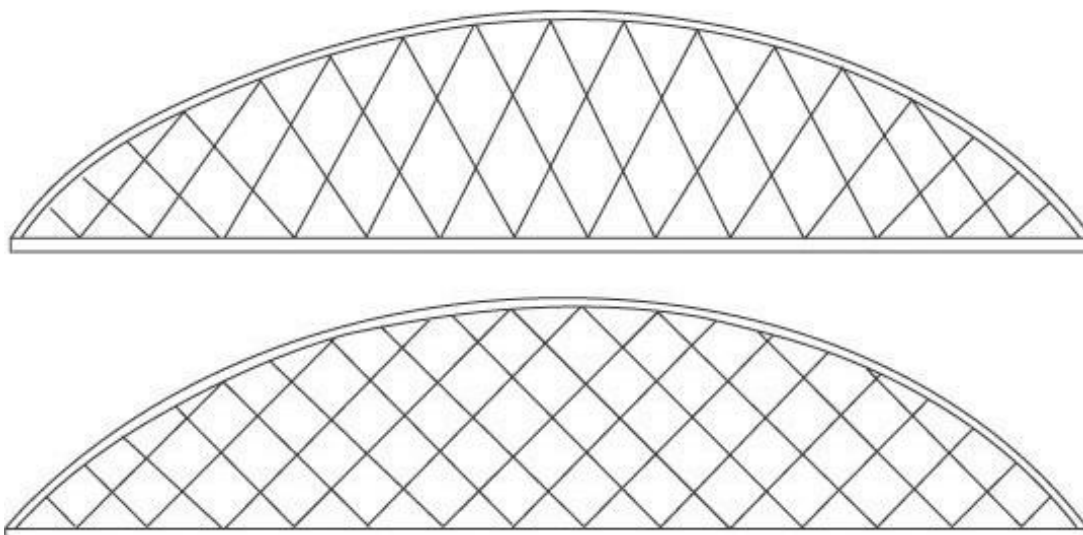I was 25 years old and had no great wish to get a job. I liked to study and got a scholarship to learn about prestressed concrete at the technical university in Aachen, Germany. When I got there, I presented myself at the office of the relevant professor, Kurt Hirshfeld. I did not know how deep you should bow to a professor in Germany. Professor Kurt Hirshfeld did not like me and threw me out.

The university library had not recovered after the Second World War. No books on prestressed concrete were available, so I returned to the arch bridges with inclined hangers. Professor Schultz-Grünow in Aachen was an old friend of my professor Gran-Olson in Trondheim. He was very kind to me and listened to my thoughts. I claimed that decreasing the slope of the hangers would increase the hangers' resistance to becoming relaxed. This was something that I could not prove.

Phillip Stein, professor of steel structures, was also very kind to me. Through him it was arranged that I was allowed to present my ideas to Professor Ferdinand Schleicher. He was the editor of the two volume "Taschenbuch für Bauingenieure". The fame and homage that surrounded him seemed almost unbelievable to a student from the egalitarian society of Norway.

A meeting was arranged between the professors Schleicher and Stein and me. Professor Schleicher listened to my explanation. Then he wrote down a simple formula that I did not understand and said that I could not stop the relaxation by altering the slope of hangers. Then he asked: "What do you want from me." Without thinking I said to Professor Stein: "Do you understand this". He said "Ich glaube ja". (I believe I do). What else could he say?

Then I thought that it was best to tread cautiously. I turned to Professor Schleicher and said something like: "I thank you for having shown me a new way of thinking. I will proceed from here." Then the big man left the room and I started to explain to Professor Stein what I did not understand. The next morning Professor Stein said: "Let us build a model".

A model was built in Professor Schultz-Grünow's laboratory. The chords of the model were strips of pine wood. The hangers were copper strands. The copper strands went through holes in the chords. Their position and length could be altered. I paid for the materials. A nearby chemist kindly lent me the bottles used for loads.

While the model was being built and tested, I sometimes met Professor Stein outside his office. Many times he said to me: "Experiments that do not give the expected results are also of value because they might show that something can not be done". Maybe he was worried that I might break down if the experiment gave the results predicted by Professor Schleicher.

To me it was a great strain that I seemed to disagree with everybody in the university on something that was so obvious to me. In fact nobody understood or was interested in arch bridges with inclined hangers. I had a sneaking feeling that I was on the way to going mad. I feared that some day I might start behaving so erratically that I would be locked up. It was a great comfort to me that Professor Arne Selberg in Trondheim had accepted my idea.
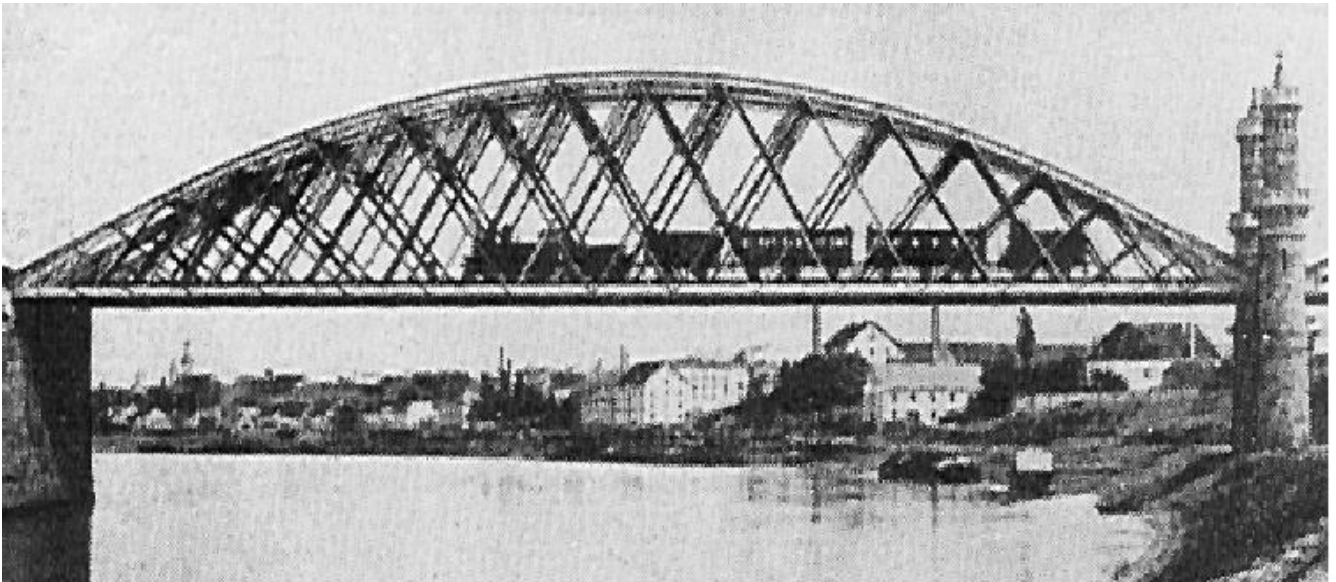
Fig. J4. One of the main spans of the Riesa Bridge over the Elbe built 1877/78. Span 97.4 m

An article on an arch bridge with crossing diagonals had been published in December of the same year. See fig. J4. (Berger 1955) It was a bridge over the Elbe River near Riesa. It was a truss with many stiff diagonals. Since very small forces are transferred between the diagonals, the bridge functions very much as if it had inclined hangers that could take compression. The bridge had been recalculated after the Second World War to examine the load carrying capacity. Nobody that I met in Germany in 1955 to 1956 had previously had any interest in arch bridges with inclined hangers.

Tests on my model showed the obvious that the slope of the hangers influenced their tendency to relaxation. Professor Stein wrote a letter of recommendation to the technical university in Trondheim. After my stay in Germany, I asked my father to support my work on arch bridges with inclined hangers. He declined to do so.

Then I went to my alma mater, the technical university in Trondheim. Nobody there had any money for my research into arch bridges with inclined hangers. Instead I was advised to apply for a scholarship to study for a licentiate degree. It was such a meagre sum of money that they were short of applicants.

In the autumn of 1956 I suggested to Professor Selberg that I wrote my licentiate thesis on arch bridges with inclined hangers. He said that the theme was not suitable. It did not worry me. When I came back I had written an explanation of my ideas covering less than 10 pages. From then onwards Professor Selberg supported my work on network arches for eight years.

I could discuss my ideas with Professor Selberg whenever I wanted to. He did not want us to knock at his door. We were expected to go right in and wait at the door till he looked up from his work. I appreciated his help, but was sometimes reluctant to give up my own ideas. He urged me to use arches that were part of a circle. It took a long time before I gave up my idea of using arches that were part of an ellipse. By the way, (Brunn & Schanack 2003) suggested that the curvature of the arch should be reduced at both ends of the arch. That would give a nearly elliptic curvature of the arch. In his doctoral thesis (Teich 2012) http://www.qucosa.de/fileadmin/data/qucosa/documents/8604/Dissertation_Teich.pdf found that theoretical elliptic arches were best. See also (Tveit 2013, p. 22).

Professor Arne Selberg helped me to build a 4 m long model of a network arch with money from a public fund. See fig. J5 on the next page. It was full of strain gauges. I had 400 hours of help with the reading of the results. A teacher from New Zealand named Tui did a lot. She later married one of our students and became Tui Sørensen. Selberg also paid for five students who helped me. They were Thor Hagen, Eilif Brodtkorb, Arne Vamnes, Stein Husebye and Martin Måseide. I never found out from where Professor Selberg got the money. Maybe I did not care.

In 1957 I wrote to my classmates from the University that I could save ⅔ of the steel compared to the formulas for prediction of steel weight that we used when we were students. I handed in my thesis in 1959. Professor Selberg was a bit disappointed in the result. I was not proud of it myself, but I still firmly believed in the idea of using crossing hangers in tied arch bridges.

Looking at the thesis over 50 years later, I am surprised how little my basic ideas have changed. My preliminary methods for calculating network arches show a good understanding of various effects, but they could not be used for final calculations. I still believed that model tests could be used to check a preliminary design. An outline of a method of calculating in plane buckling of the arch was there. It was published in (Tveit 1973). Then calculations by big frame programs had shown that it was right.
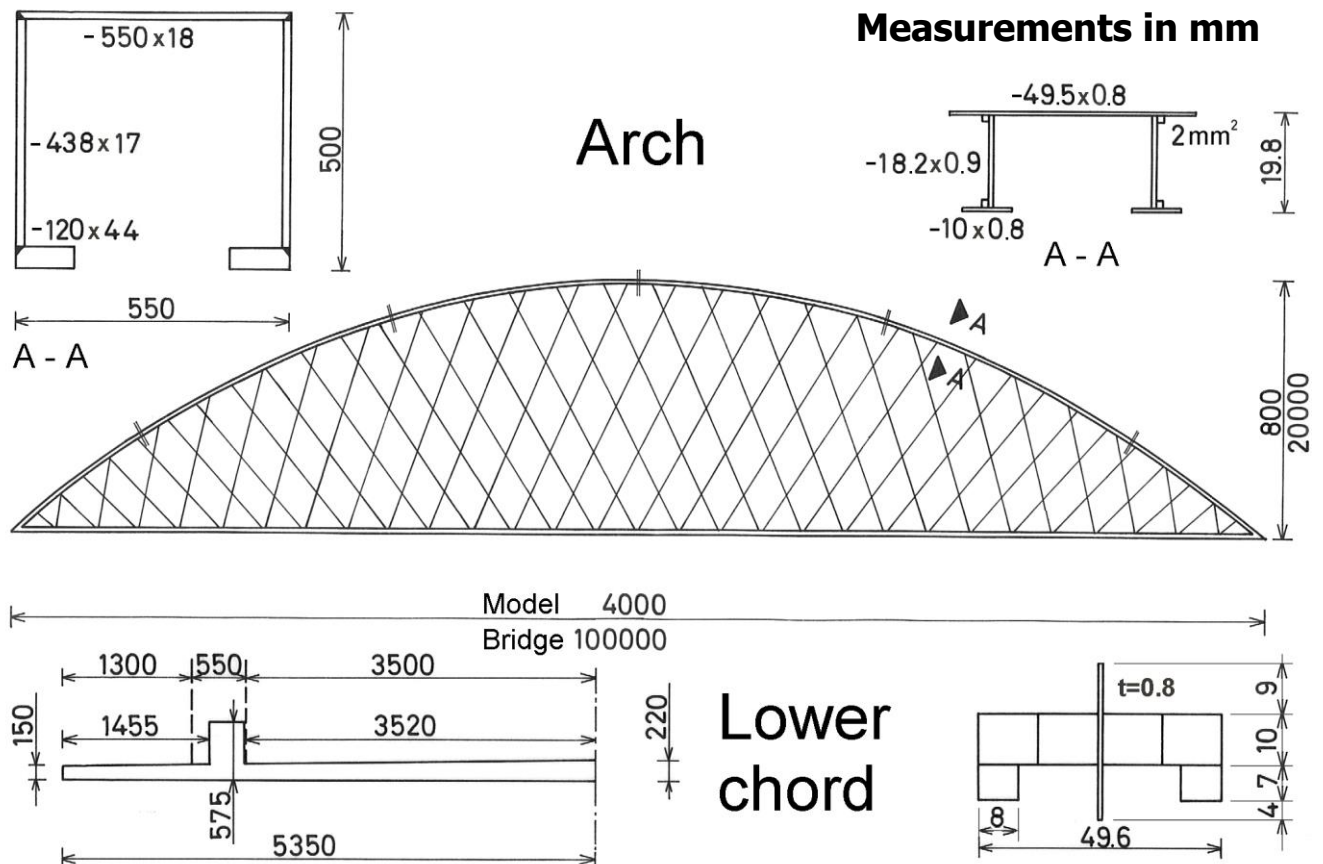

Fig. J5. Model of a network arch tested at the technical university in Trondheim, Norway, 1956 to 1959

I had given up the idea of patenting long ago. To prevent others from patenting the idea, I published an article on network arches in "Arbeider-Avisa" (The Worker's Paper) in Trondheim 10.10.59. The article contained many sketches. Many of the structural details were not good.

The article, which is in Norwegian, presents my central ideas on network arches:

 "The bridge is so light because it has little bending, well braced compression members and many tension members." – "Compared to most steel bridges built today over 50% of the steel weight can be saved".

"The tie is a simple slab of prestressed concrete". – "A bridge with a span of 100 m could need around 120 t of steel. For the same span, width and load estimate tables based on experience put the steel weight between 248 and 458 tonnes." – "The bridge type can of course be used everywhere, but it is extra competitive over soft soils. It will be very favourable when whole bridge spans can be transported to the pillars."

"Mainly two things prevented arch bridges with crossing hangers from being built:

1. Before the Second World War little could be gained by crossing hangers with the materials and traffic loads that could be foreseen.

 2. Computers and metering equipment suitable for calculating these bridges were not available. This has changed now".

"Today we have stronger steel and concrete, bigger traffic loads, better measuring equipment and electronic computers. Thus we should build arch bridges with hangers that cross each other. Where hangers cross, there could be plastic tubes on them to stop them from banging against each other".

"In bridges where many equal spans are needed, they could be finished on land and be transported to the pillars. There could be pretensioned rods in the longitudinal direction. On land it is easier to make extra strong concrete."

3.0 7.0 4.4 7.0 3.0

26.8m
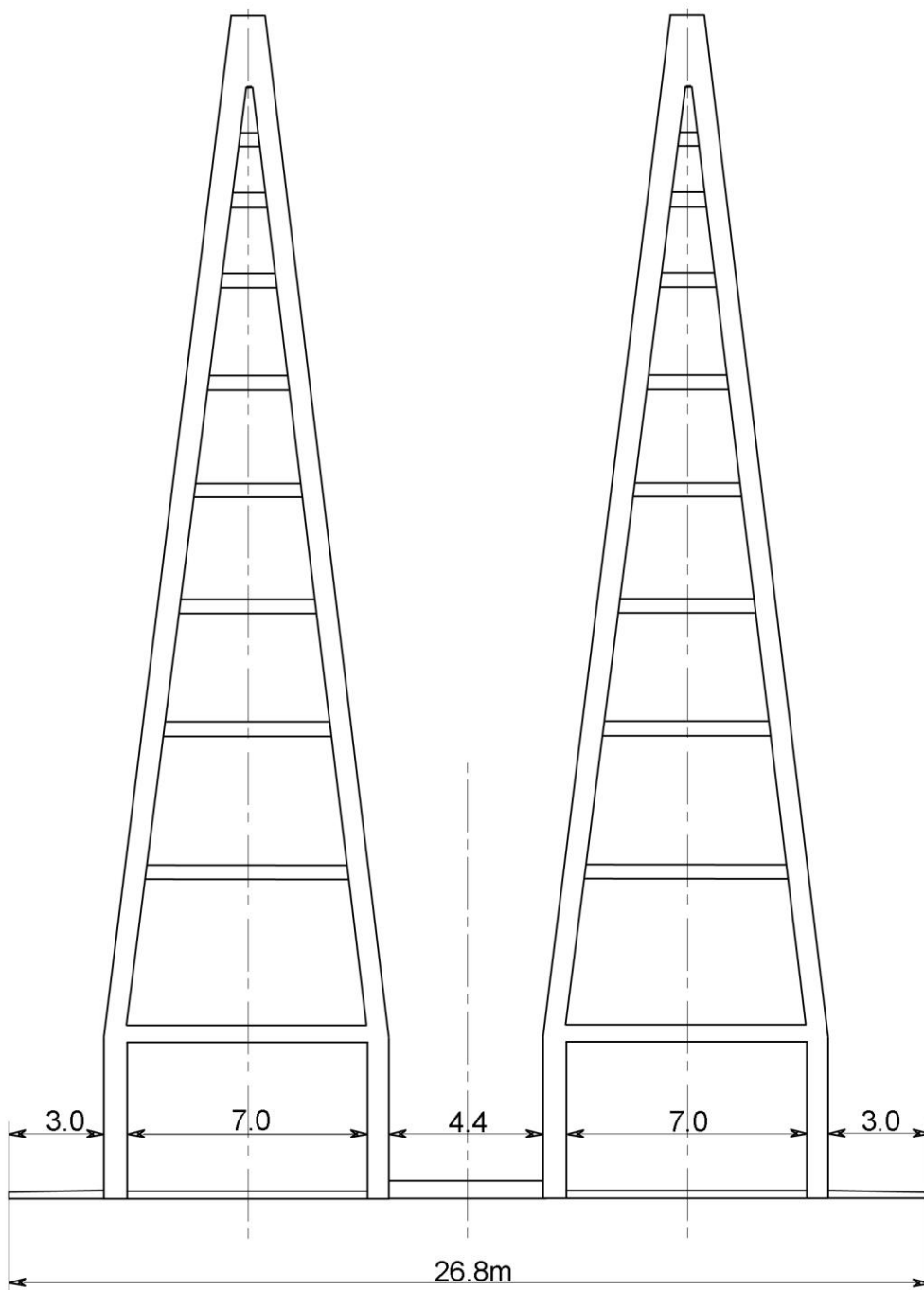
Bridge for road and railway. Span 125 m
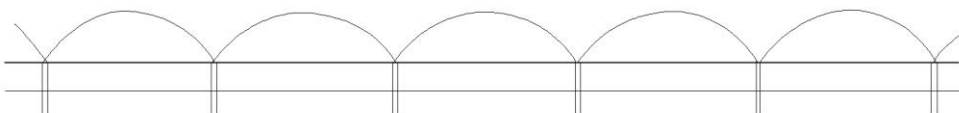
Fig. J6. Bridge for road and rail over a wide and shallow sound. (Tveit 1959)

"Where a railway and a motorway are needed over a broad sound, a bridge like in fig. J6 can be built. Each span should consist of two halves. Each half carries two lanes and a footpath outside an arch. The railway should rest on steel beams between the two halves."

One paragraph is almost prophetic. "Very many new ideas are a result of the technical progress. It is as if the ideas are in the air. Often people working independently of each other simultaneously get the same idea. It will now be interesting to see if others have found that arch bridges should have crossing hangers."

Somewhere in my thesis I wrote: "I will not write much about new designs, because I do not have the practical experience that makes it possible to distinguish between what is important and what is unimportant. Still I made a comparison between a bridge in Luleå in northern Sweden (Kungl. Väg- ock Vattenbygnadsstyrelsen 1954) and a network arch:

Comparison between two tied arch bridges:

Things similar:

Span 82 m.   Two tracks 3.5 m wide.   Two footpaths 3.0 m wide.   Swedish Norms.

Loads: 1st track: 1.2 t/m,   Axel load 14 t,   2nd track: 0.8 t/m,   Axel load 10.5 t   + Coefficient of impact
Footpath 0.3 t/m

Things different:
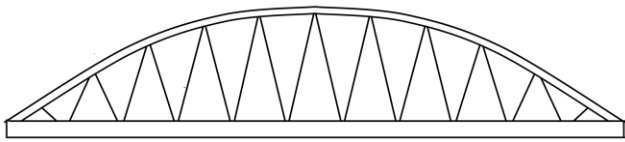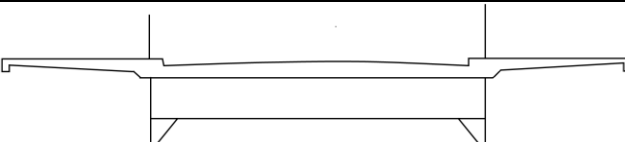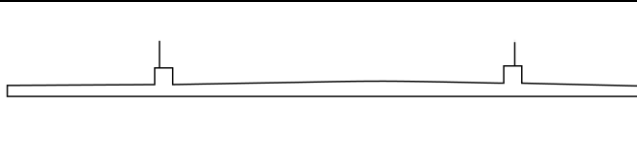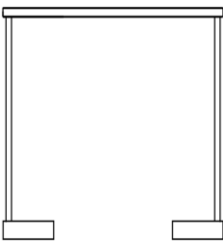
Bridge at Luleå. Finished 1954	Network arch designed 1957
The concrete slab has 80 mm asphalt on top. It rests on longitudinal and transversal steel beams. Ribbed reinforcement σ_y=420 MPa σ_{B28}= 40 MPa	Traffic directly on the prestressed concrete slab. The concrete slab is fastened directly to the hangers. Longitudinal reinforcement σ_y= 1600 MPa Transversal reinforcement σ_y=900 MPa, σ_{B28}=50 MPa
Weight of span 980 t	Weight of span 640 t
Steel needed 269 t	Steel needed ~110 t
Area subjected to wind ~6.5 m^2/m	Area subjected to wind ~2.5 m^2/m
The steel skeleton was erected on land and moved to the pillars on pontoons.	The steel skeleton was erected on land and moved to the pillars on pontoons. Then the tie was cast on a flat form.
88% St 44 12% St 42 The arch has a square cross-section 640×700 mm	37% St 900, Ø 26mm 13% St 1600, Ø 5mm 50% St 44 48% plates and 2% tubes The arch has a square cross-section 500×400 mm.

Fig. J7. A comparison between a bridge in Luleå in Sweden and a suggested network arch

By using inclined hangers the designers of the bridge in Luleå said that they saved 50% of the steel. That indicates that 80% of the steel needed for arch bridges with vertical hangers could be saved by using network arches.

THE TOWN OF STEINKJER LET ME BUILD MY FIRST NETWORK ARCH

The article in Arbeider-Avisa was read by town engineer Einar S. Balgaard at Steinkjer. Steinkjer is a town around 100 km north-east of Trondheim. The population was around four thousand. Balgaard thought that new promising ideas should be tried. He knew that Professor Selberg supported my idea. A classmate of mine, Asbjørn Auran, was one about Balgaard's assistant engineers. All this led to the building of the network arch at Steinkjer. See figs J8 to J10. More of that network arch can be found on p. H-2 and H-3, (Tveit 2008 p. 5b to 6c), (Tveit 2006), (Tveit 2008d), p. 22 and figs H2 and H3. The earliest publications about the first two Norwegian network arches are (Tveit 1964 and 1966).

For the design of the bridge at Steinkjer, I built a new model of an arch bridge where the pattern of the hangers and the length of the arch could be altered. In an alteration of the length of the arch there was a fault in the workmanship and the model collapsed.

About the same time the Swedish aircraft producer SAAB came to the technical university in Trondheim to sell us one of their computers. Their calculations made it possible to draw in the influence lines that were used in the design of the network arches in Steinkjer and Bolstadstraumen. See Tveit 2013 p. 57 and p. 58. To the best of my knowledge, the two network arches were the first Norwegian bridges to be designed with the help of computer programs.

Fig. J8. An overview with structural details of the network arch at Steinkjer

Bjørn Vik, another classmate of mine from the university worked at the bridge office of the Norwegian Public Roads Administration. He was sceptical. He said that a more conventional bridge would probably be less costly. So there was a competition. He suggested a concrete arch bridge with vertical hangers. Not very elegant, but that was not so important in Norway in those days. The network arch won, but Bjørn Vik remained doubtful of the advantages of the network arch for many years. If the arches at Steinkjer had been universal columns, the bridge would have been less costly, but it would also have been less good-looking.

THE BUILDING OF THE NETWORK ARCH AT STEINKJER

The steel arch and the hangers should have been carrying the tie before the winter set in, but the steel mill did not produce the steel as promised. When the ice was around 0.15 m thick, a flood came and swept the ice towards the scaffolding under the concrete tie. See fig. H4. The timber piles were broken under 17 m of the tie. Due to this the tie was sagging 0.2 m and developed cracks over 2 mm wide.

The scaffolding was repaired and strengthened in a makeshift way. Tragically, a man lost his life in the process because my instructions were ignored. Twelve piles in the river could not be replaced. When the steel had been erected, (See fig. J8), the hangers were shortened till they carried the tie. The tie was straightened in the process. Afterwards the scaffolding could be removed. Prestress closed the cracks in the tie. Now they are hard to find.

Fig. J9. Erection of arch and hangers at Steinkjer in the winter of 1962-1963

When I was building the network arch at Steinkjer, I heard about the Fehmarn Sound Bridge. Span 248 m. (Stein & Wild 1965). It was a network arch with arches that sloped towards each other. It was built by Gutehofnungshütte. See Tveit 2008 fig. 27.

Much later I found that my Professor Phillip Stein from Aachen had written the centenary history of Gutehofnüngshütte. (Stein 1951) I wondered if my ideas of saving steel by using hangers with multiple intersections could have been passed on to Gutehofnungshütte by Professor Phillip Stein.

When asked about this in a letter Professor Stein answered: "Dass ist durchaus möglich". (That might very well have been the case). Professor Masao Naruoka saw model tests for the Fehmarn Sound Bridge in TH-Hannover in 1960. He took the idea to Japan where it has been flourishing. (Naruoka 1977), (Yoshikava 1993), (Nakai 1995).

The posts, IPE 100, of the railings outside the footpath, see fig. J8, are welded to channels on the outer edge of the concrete. The welding was done slowly using little heat to avoid cracks between the channels and the concrete. To give the pedestrians a feeling of safety, the top of the railing is 130 mm wide. Outside the main span a vehicle has run into the vertical bars in the railing and has bent some of them. However, it was easy to straighten the bars by hand.

Fig. J10. The finished network arch at Steinkjer, Norway. (Photo Alvestrand)

There is a rod, Ø32, at the bottom of the grid inside the railing. In order to reduce stresses in the railing due to creep, shrinkage and bending in the concrete tie, this rod is not fastened to the vertical posts. See fig. H1.

Since the IPE 100 that are at the ends of the main span have remained vertical, we can conclude that creep and shrinkage have not made the main span shorter than it was about half a year after the tie was cast.

Little de-icing salt has been used on the bridge. Most of the concrete is in good shape. On the surface of the footpath porous pebbles have absorbed water that has frozen in the winter. This has broken the concrete cover over the pebbles and has led to some cavities about one cm deep. To compensate for this, an epoxy membrane has been glued to the footpath.

There are some very small dirty cracks all over the concrete, but there is no decay around these cracks. In a few places there is rust at the surface. Probably some reinforcement bars have come too near the surface. So far the rust has not yet caused concrete to fall off. (Zoli and Woodward 2005) have found that the breaking of a hanger is not likely to make a network arch collapse.

Fig. J11.The Bolstadstraumen network arch built in western Norway was opened at the end of 1963. Span 84 m

Professor Arne Selberg supported my wish that the Norwegian Public Roads Administration should build a network arch, but the bridge office of the Norwegian Public Roads Administration was sceptical. Then my mother had been to Oslo to talk to her brother about her son's fascination with a very promising type of bridge. Her brother was permanent secretary to the minister of transport.

All of a sudden it was decided that the bridge over Bolstadstraumen might be an arch bridge. See p. H-4 and E-1. The Norwegian bridge office designed an arch bridge with vertical hangers. I was allowed to design a competing network arch. See fig. J11. My terms in the competition were no cure, no pay. It did not worry me much because it was the best terms that I could get. Furthermore I was fairly confident that I would win.

My network arch used 44 tonnes of structural steel and 7 tonnes of prestressed steel. The bridge office decided that the rise of my arch should be 18% of the span while the rise their arch should be 21.5% of the span. Their design needed 125 tonnes of structural steel. Both bridges had a concrete slab spanning between the edge beams. The network arch was about 20% less costly. (Tveit 2013 pp. 7 and 7a)
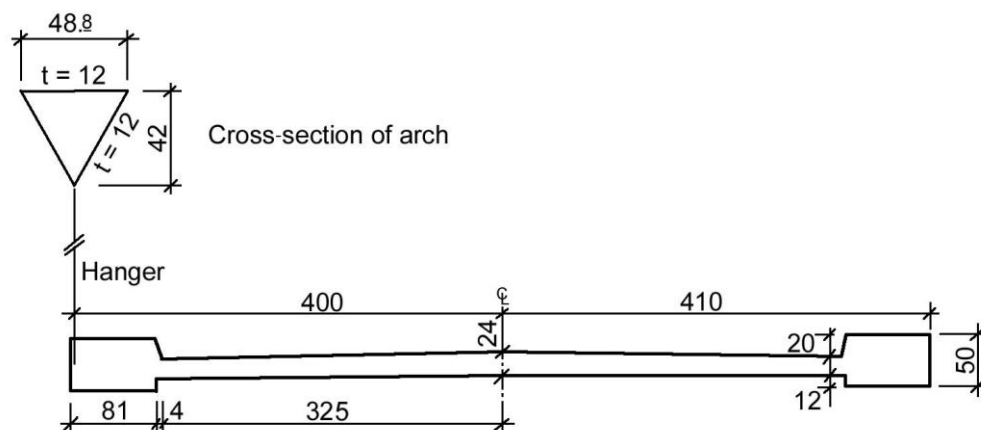
Fig. J12. Cross- sections in the Bolstadstraumen Bridge

I wanted to do the site control of the production of the Bolstadstraumen Bridge, but that was given to the engineer who had made the competing design. He did not do a good job. When the lane was cast he arrived at the scene 12 hours after the casting had started.

The casting of the tie was one of the earliest Norwegian examples of pumping of concrete. The concrete flowed fast and was not properly vibrated. In many places the concrete did not get down under the lower layer of reinforcement in the tie. Where this was the case, the cavities were filled with a rich mix of sand, lime and concrete.

The curvature of the arch members was too small, so the curvature of the arch was not constant. The drawing that showed how the length of lower end of the hangers should vary along the tie was not sent to the firm who cast the tie. They used their good sense when they put in the lower end of the hangers. This worked well, because the length of some of the hangers was not in accordance with the drawings.

Later the engineer who controlled the building of the Bolstadstraumen Bridge was put in charge of the section for steel bridges in the bridge department of the Norwegian public roads. Maybe he concluded that the network arch was too complicated for the bridge department of the Norwegian public road department.

I do not know if my uncle ever spoke up for the network arch, but after my uncle retired in 1959 with his St. Olav medal, first class, it took over 40 years before it was planned to build another network arch in Norway. That was the Brandanger Bridge. See Tveit 2013 figs 99 to 103. The initiative came from Aril Hanekamhaug. He asked me to do a preliminary design of a network arch for the Brandanger Bridge in western Norway. It was built by the county of Sogn og Fjordane (Tveit 2013 pp. 93c to 94a). See also fig. J13.

It seems reasonable to define the slenderness of an arch bridge as the span divided by the sum of the depth of the chords. By this definition the slenderness of the Bolstadstraumen Bridge is 91. It was the world's most slender tied arch bridge till 2008. Then the Mangamahu Bridge in New Zealand was built. P. H-8 to H-10. It is 4% slimmer. For the Brandanger Bridge in western Norway that was finished in 2010 the slenderness is 198. It will probably be the world's most slender arch bridge for many years to come.

The slenderness of the network arch is important from the aesthetic point of view. (Leonhardt 1991) said: "…we recognise the need to integrate a bridge into its environment, landscape or cityscape, particularly where the dimensional relationships and scale are concerned. Many mistakes have been made during the past decades by placing massive concrete blocks in the heart of older areas of a city…..Sometimes, long-span bridges with deep, heavy beams spoil lovely valley landscapes or towns." It is simple psychology of perception that when we look at a landscape with a network arch bridge, we automatically concentrate on seeing the bridge or on seeing the countryside. That is less straightforward if a bridge has high beams that cover up part of the countryside.

When the Bolstadstraumen Bridge was finished, I could not get more scholarships or more jobs as assistant at the technical university by saying that I was going to do my licentiate. Thus I had to do my final exam. Many years later that exam gave me the right to use the title dr. ing.

After my final exam in 1964 I applied for a beginner's job in the bridge office of the Norwegian Public Roads Administration. I wanted to continue my work on network arches. It was customary that engineers with a little experience started with one step above minimum pay. They would not promise me that much. I got the impression that they did not want me. When the head of the bridge office wrote a booklet about Norwegian bridge building from 1930 to 1980, he failed to mention the network arches. Professor Selberg suggested that I should ask him why. I was too proud to do so.

In June 1964 I designed a network arch in Sweden. It did not win. A beam bridge was chosen. Maybe the contractor underestimated the competition and put the price of the network arch too high. Maybe I had supported his optimism too much. Today I would have designed that Swedish network arch very differently.

In 1964 I happened to come across the drawings of a railway bridge to be built over the Tinnelv river at Stormo in Norway. It was a single-track bridge spanning 67.8 m. 275 t of steel were to be used for the welded truss. Through preliminary calculations only some 80 t of steel would have been needed for a similar railway network arch.

It deserves to be mentioned that I did not pay enough attention to fatigue in my preliminary design. Since the bridge was soon to be constructed, there was no time for a redesign. Furthermore the bridge office at Norwegian railways shoved no interest in my network arches. Hearing this, my mother said: "Per, now you must leave the country." That made sense to me.

Fig. J13. The Brandanger network arch in western Norway. Span 220 m. Opened 2010

Work with Christiani and Nielsen in Copenhagen

On September 1st 1964 I started to work for Christiani & Nielsen in Copenhagen. There I worked on underwater tunnels for three years. Most of my work was on the tunnel elements of the tunnel under the Limfjord in Ålborg in northern Jutland. That was the start of my interest in submerged floating tunnels. I have written over ten publications on submerged floating tunnels for Norwegian fjords. (Tveit 2009a)

When I was working with Christiani & Nielsen a colleague, Svein Fjeld, and I partook in a competition for a bridge over The Great Belt. Our network arch solution was not very good and did not win anything.

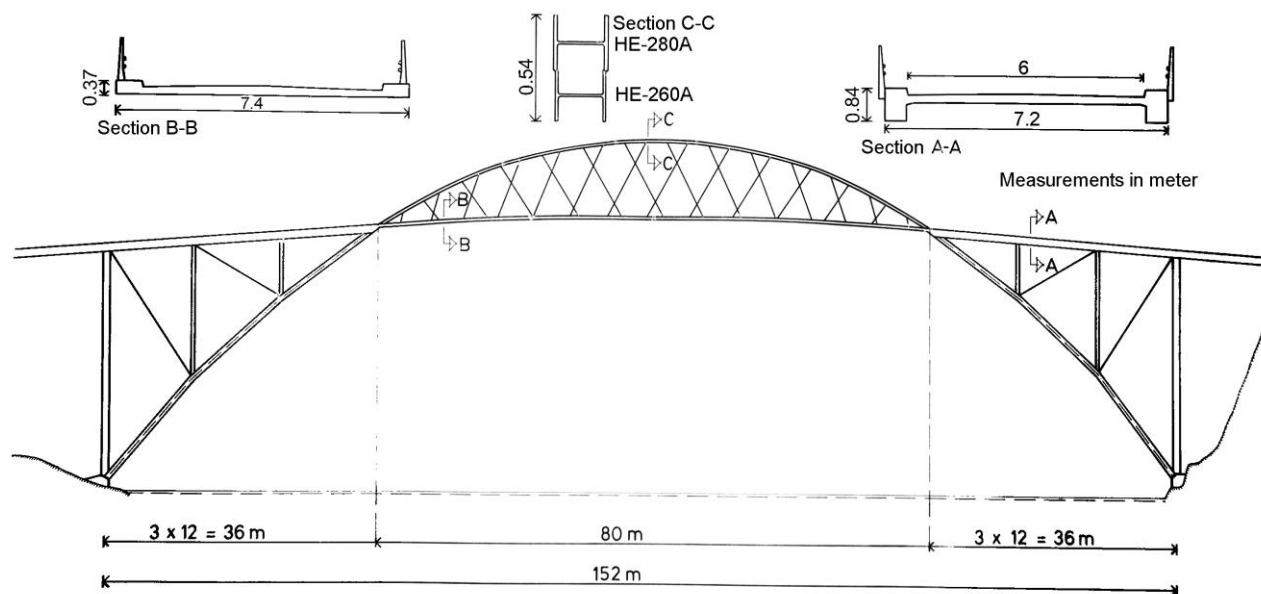# Work at an island on the west coast of Norway

Fig. J14. The Bridge over the Lukksund

In 1967 I became a partner in the firm Sørli and Tveit on Stord Island south of Bergen on the western coast of Norway. We took interest in roads and bridges in the district where we worked. I suggested a bridge over Lukksund. See fig. J14. The expansion joint is over the column to the right in the drawing. The big rise of the arch reduces the axial force in the arch. Thus the force in the tie is moderate. This gives an increased tendency for buckling in the arch. Still the profile of the arch above the lane is stiffer than necessary. A single universal column would have been sufficiently stiff and more economical.
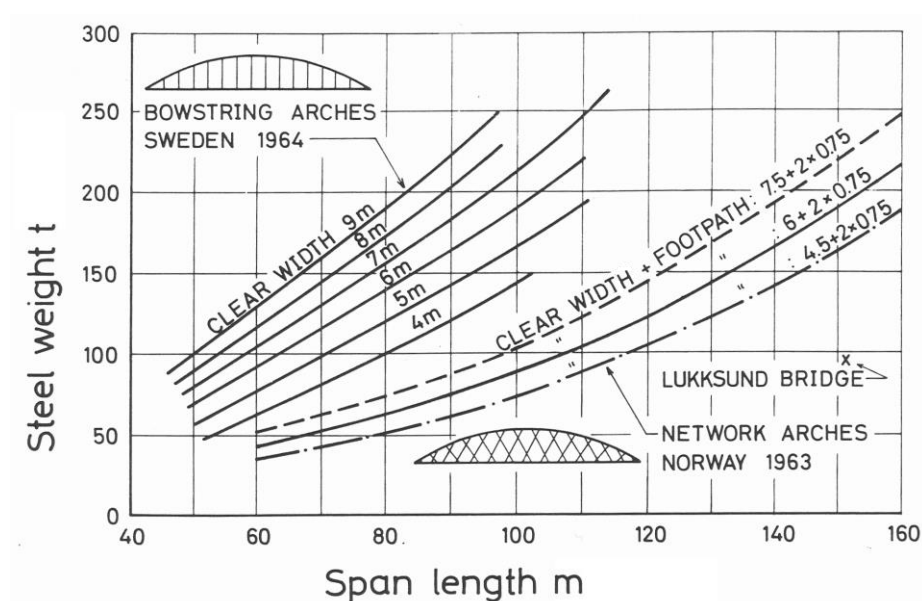
Fig. J14a. Steel weight of various arch bridges

The arches under the lane are meant to be steel tubes. They could be floated to the site and be swung up in place. Afterwards the arch over the lane could be put in place either from below or from a side span. The Lukksund Bridge has a very low steel weight per m^2. In fig. J14a the steel weight of the Lukksund Bridge is compared to the steel weight of other arch bridges. The weight of arch bridges with vertical hangers comes from (Cronholm 1963). The weight for network arches comes from (Tveit 1966). The bridge office of the Norwegian Public Roads was not interested. Another bridge was built.

I continued my work on the Lukksund Bridge after I got a job with Danmark's Ingeniørakademi in Aalborg, Denmark in 1969. There a computer was available for a lot of calculations on the Lukksund Bridge, but the calculations were never completely finished. Another bridge type was built. I stayed in Aalborg till 1985 and did a lot of research on network arches. Many students did valuable work on network arches. Some of them worked in groups. (Tveit 2013) has a list of literature with 11 publications on network arches from my Aalborg years.

A likely form of buckling of the arch of a network arch is shown in fig. 14 in (Tveit 66). The idea came back to me in 1972. It was presented in a publication on railway bridges (Tveit 1973). I assumed that the tie followed the ups and downs of the arch, but with smaller ordinates. Very slender arches with thin hangers could have smaller distances between points of inflections. In his doctoral thesis Frank Schanack (2008) pointed out that assuming that the tie was completely stiff gave sufficiently accurate results. This is a valuable simplification.
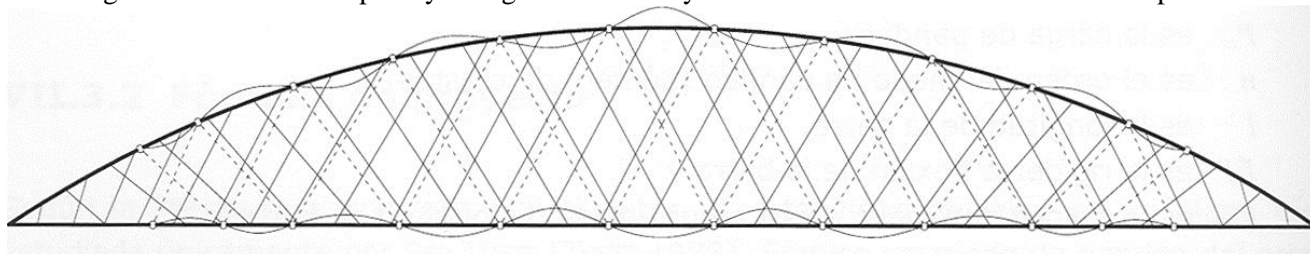

Fig. J15. Mode of buckling in a network arch (Schanack 2008)

In the academic year of 1977-1978 I was a guest lecturer at the University of Houston in Texas. In the spring semester 1978 a group of students did some very interesting work on network arches. Their names were John S. Cowen, I-Chun Ching, Gene O. Day, Augusto Duque, Ronald T. Grimes and Wah-kin Yan. We wrote a 93 page book on network arches. (Tveit et al. 1978) It contained a lot of advice on how to design network arches. Concrete ties were assumed.

I wanted to give a contribution on network arches to the IABSE congress in Vienna in 1980. For a long time there was no reaction to my abstract. Then I wrote to the committee stating that I was used to roaring silence when I wanted to speak about the network arches, but would they please give me an answer. I was allowed to display the poster in fig. J16. I also made a handout with 45 pages (Tveit 1980a). In the congress report (Tveit 1980 b) I suggested using concrete spans for long bridges over navigable water. The spans could be made on land and be floated to the pillars. That idea was used later in suggested side spans for a bridge over the Fehmarn Belt in (Tveit 2009).
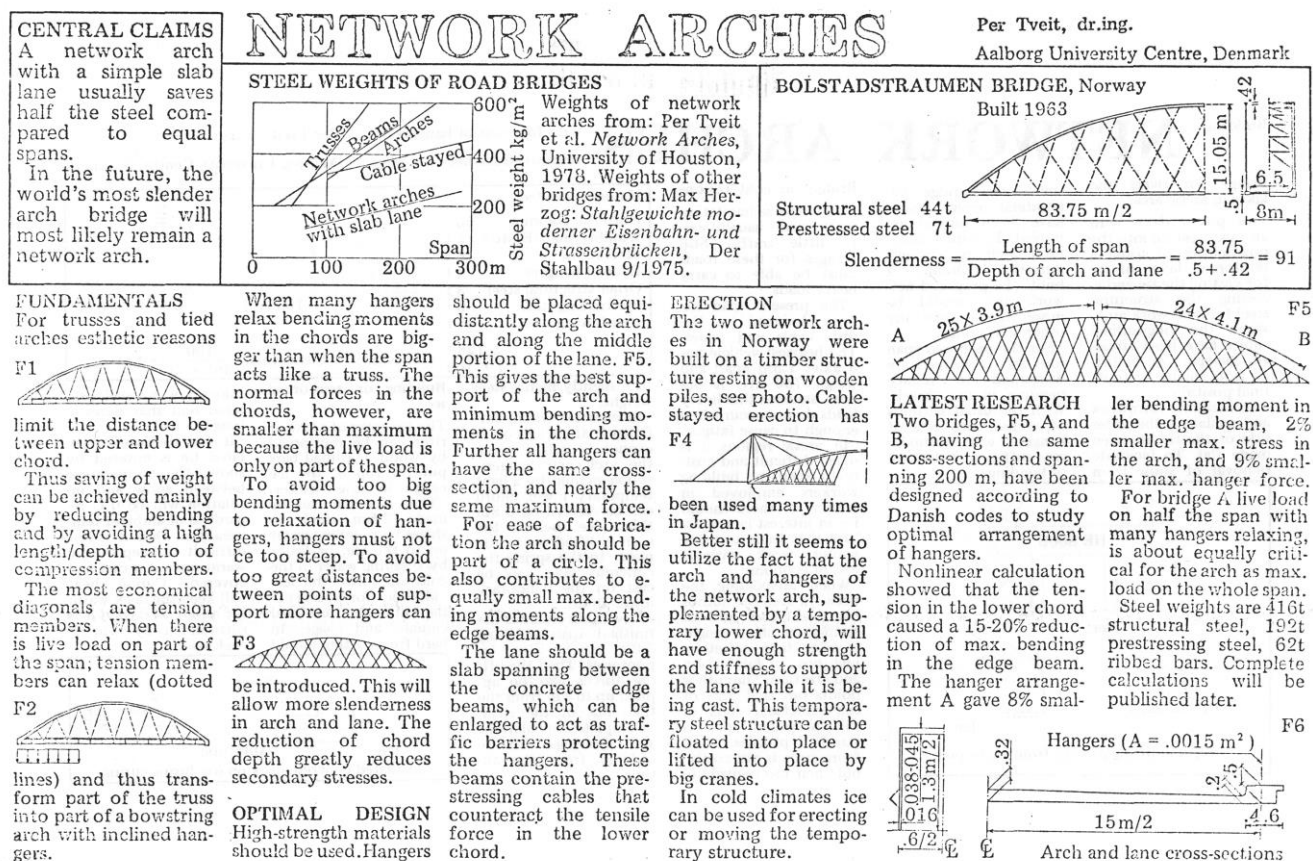

Fig. J16. Poster for the IABSE congress in Vienna in 1980

During my years in Ålborg, Danmark's Ingeniørakademi became part of Ålborg University Center in 1974 and we were supposed to do more research. First we formed groups that were supposed to do research. However very little research came out. Then it was decided that those who wanted to do research could apply for research time and get less teaching. They would be supposed to publish their results, or at least present their results in a lecture to colleagues once a year. Those who preferred to teach should concentrate on that, but they should have time for scientific updating. This worked much better.

I got time for research. I presented my ideas on network arches in a lecture in Trondheim in 1981. (Tveit 1981). Around 1980 there was a lot of unemployment amongst draft personnel in Denmark. Those who had been unemployed for more than two years were lent out free to firms for periods of 6 months to make sure that their skills were kept intact. The University Centre had an ample supply of these ladies. They drew many nice diagrams and influence lines for me. Nobody else had much work for them.

In 1984 I a had a poster for the IABSE congress in Vancouver. See fig. J17. (Tveit 1984) was a handout for the congress in Vancouver. 106 pages. The unemployed draft personell drew many diagrams for this publication.

The influence line in the longitudinal direction for the network arch in fig. J17 had the same maximum ordinate as a simply supported beam spanning 4.2 m. Since the distance between the planes of the arches was 7.6 m, it is obvious that the transversal bending moment is bigger than the longitudinal bending moment. The bending moment in the edge beam is kept down by a 1.5 m average distance between the nodal points in the edge beam.

It was not a good idea to cast the concrete plate on the ice. It would be a much better idea to erect a steel skeleton with a temporary tie on the ice and lift it on to the pillars. In the spring the tie could be cast. See (Tveit 2008 p. 30b). More on erecting network arches on ice is explained on page E-6.

In the early eighties there were too many teachers at the University Centre in Ålborg. Very sensibly we were offered leave of absence without pay for up to three years while we tried other jobs. In 1985 I was offered a job as reader in an institution of higher learning in Grimstad on the south coast of Norway. It later became part of Agder University.

I took the job partly because my Norwegian pension would depend on the number years of service and the pay in my final position. Partly I went to Norway because some colleagues in Aalborg were eager to tell me how I should do my research.

In Grimstad the computer science department was supposed to train computer engineers who after three years were able to work on structures in machines and civil engineering. I was elected head of the department. Administration took up a lot of my time. Gradually our students became more and more interested in computer sciences. In 1994 the structural engineering part of the curriculum was dropped and I stopped being head of the department.
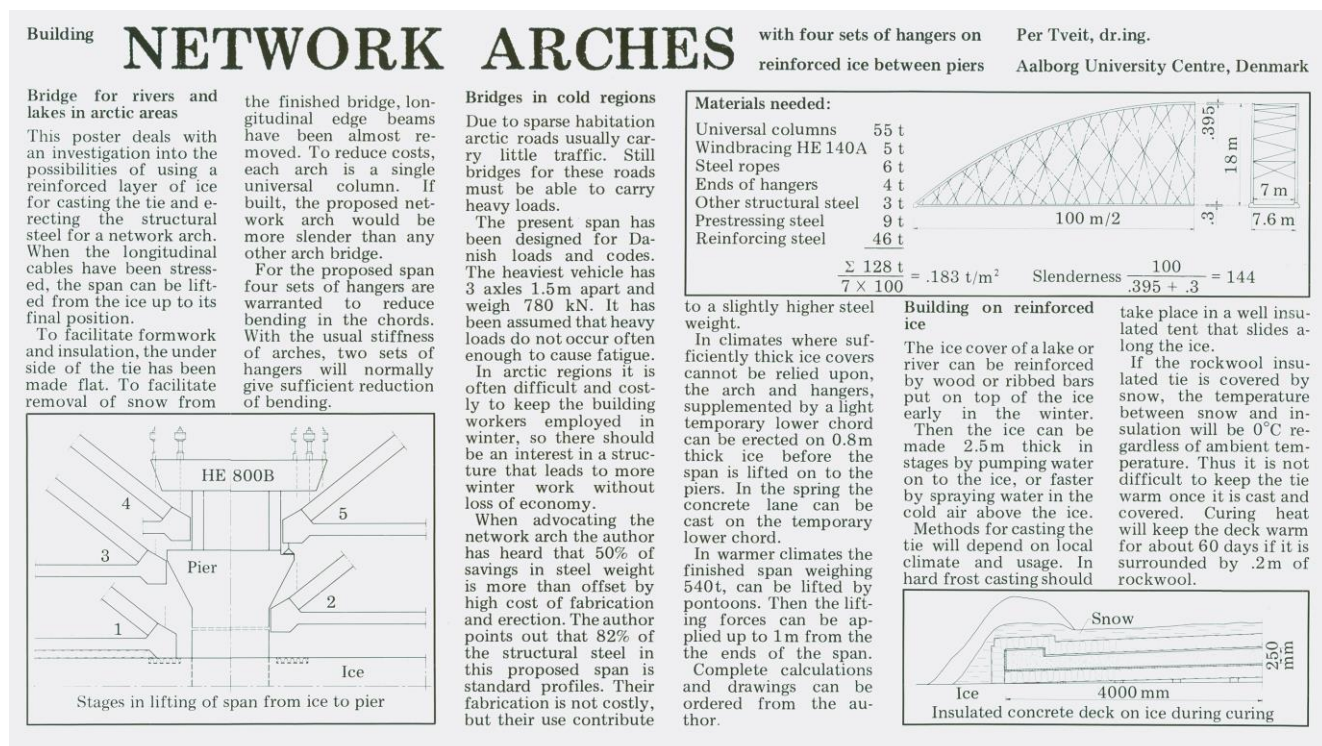

Fig. J17. Poster for the IABSE Congress in Vancouver in 1984

I retired in 1997. Till then most of my research in Grimstad was on submerged floating concrete tubes for road traffic, but the network arches were not quite forgotten. In 1986 I gave lectures on network arches in the technical universities in Prague and Vienna. In year 2000 Professor Günter Ramberger of TH-Vienna recommended my lectures on network arches to many colleagues on the strength of the lecture in 1986.
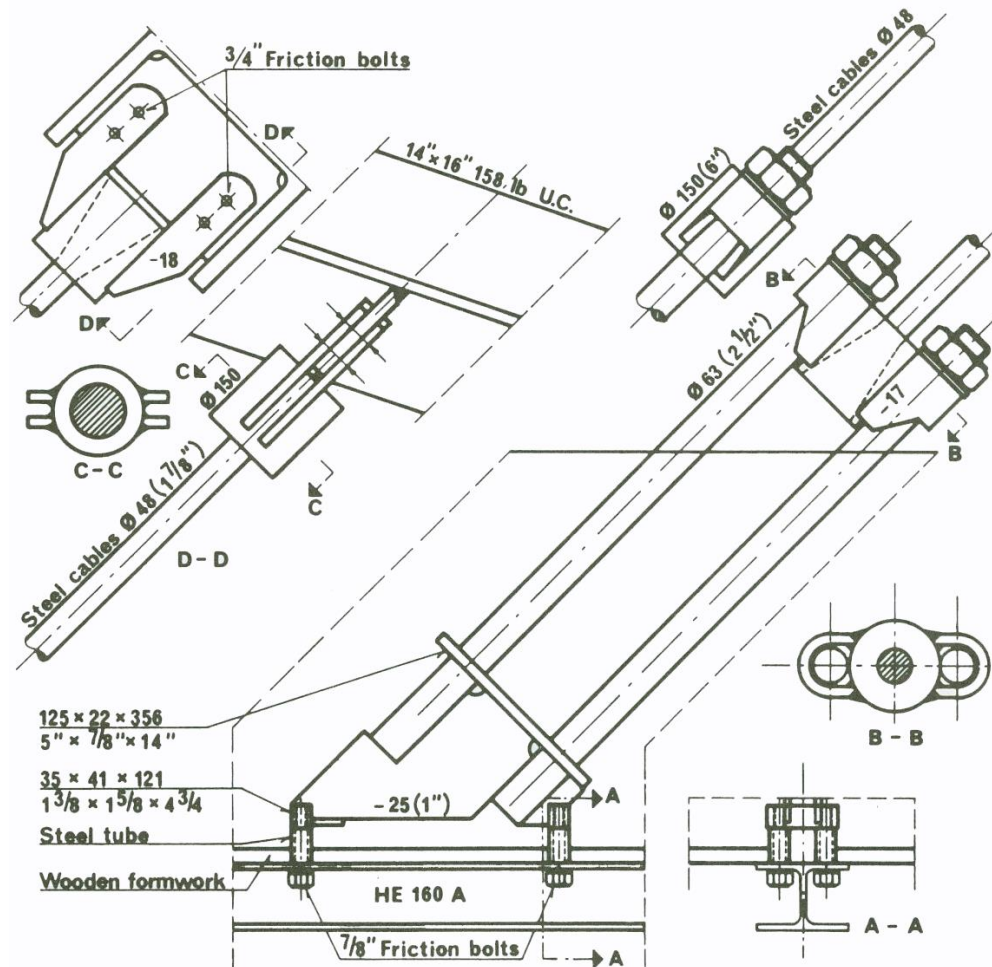
In 1987 I published a 16 page article on network arches in the American Society of Civil Engineering's Journal of Structural Engineering. (Tveit 1987). What was new was mainly the result of work done in Ålborg with a computer program developed by Georg Andersen in the late seventies. (Andersen 1979).

Fig. J18 is from this publication. It shows suggested hangers from a suggested single track railway bridge spanning 66 m. The lower ends of hangers are fastened to the longitudinal beam in a temporary lower chord. Otherwise the hangers are much like the hangers in the Steinkjer and Bolstadstraumen bridges.

Fig. J18. Adjustable hanger for a single track network arch railway bridge with a longitudinal temporary tie
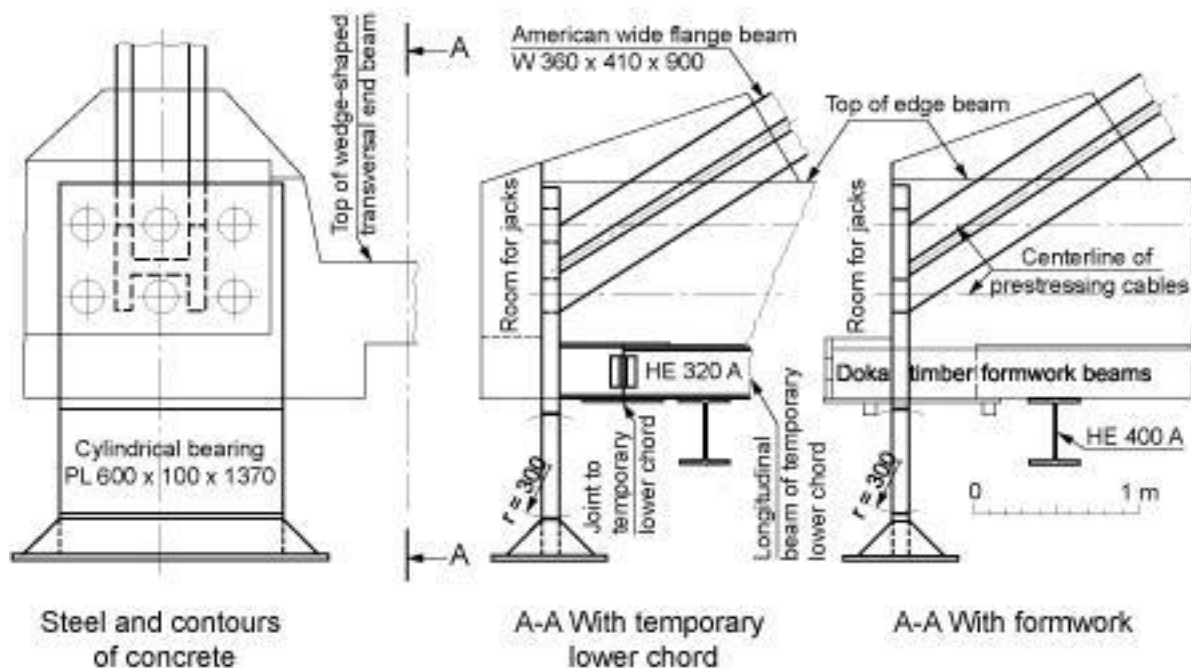

Fig. J19. Schematic drawing of the end of a 66 m long single track railway bridge with a temporary lower chord

Fig. J-14

When I retired in 1997, I decided to bring my research on submerged floating tunnels to an end and concentrate on network arches. (Tveit 2000a) was supposed to be my final publication on submerged floating tubes. Professor Geir Moe and I had a poster with the title "Submerged Floating Tubes with a free span of 4000 m" at the IABSE Symposium Kobe 1998. Since I was going to Kobe anyway, I made a trip around the globe giving lectures in Beijing, Osaka, Vancouver, Pittsburgh and Providence in Rhode Island.

In Pittsburg I met William Hess who had got the idea of the network arch before he discovered my work on the subject. He said: "I say that I can save 10 to 20% of the steel weight by using network arches. If I claimed to save more, I would sound unbelievable". I think that he had a point, but I have chosen to stick nearer to the exciting truth. The firm where he worked, Michael Baker Jr., later suggested the Blenderhasset Bridge. See (Wollmann 2007) pp. H-23 to H-25 and (Tveit 2013 p. 19a). In Providence they decided to build a network arch spanning 120 m. See (Steere and Wu 2008), (Tveit 2013 p. 19) and pp. H-5 to H-7, H-23 to H-25, H-5 to H-7.

In 1999 I also gave lectures in Copenhagen, Gothenburg and Trondheim (Tveit 1999). In the September number of Stahlbau I had an article on narrow network arches with medium spans. (Tveit 1999c). In February and March 2000 I went on a lecture tour to 12 European countries and gave 20 lectures. See (Tveit 2013 p. 99). During a three hour visit to TU-Dresden I met Professor, Dr.-Ing, habil, Wolfgang Graße. We agreed that some of his students should write their master's theses with me in Grimstad. The thesis would then be graded in TU-Dresden where they would graduate.

These students worked wonders for the network arches. Their average grade was 1.06. This arrangement worked very well till Professor Graße retired in 2004. Their theses were (Teich & Wendelin 2001), (Steimann 2002), (Brunn & Schanach 2003), (Räck 2003), and Rohm & Skalda 2004. The first four of these theses can be found on http://home.uia.no/pert/ under the button "Master's Theses". In the cellar of a hotel in Dresden we celebrated that Teich and Wendelin had defended their Master's thesis. Putin stayed in the same hotel, but on a higher level.

In 2000 I put the first edition of "The Network Arch" on my homepage. This publication has been updated between 12 and 15 times. The present edition is (Tveit 2013). See http://home.uia.no/pert/ under the button "The Network Arch". Updated versions will follow in coming years. It seemed to me that network arches would be suitable for India. I wrote an article presenting the idea. I asked Professor Dr. Jørg Schlaich if he knew somebody who might be willing to publish it. He sent it to India and it appeared in print two or three months later. (Tveit 2000).

From this point my story becomes even duller. Dear reader, you are very welcome to stop reading. In the autumn of 2000 I gave lectures in Bucharest, Dresden and the ministry of transport in Berlin. I gave seminars in the technical university in Gothenburg and in Borlänge in the bridge office of the Public Roads Administration in Sweden. 2000 was a busy year. I spent 100 nights away from home.

In 2001 I had a contribution at a conference on arch bridges in Paris. (Tveit 2001). The conference dinner was in the Eiffel Tower. I gave a lecture in Beograd. Then I travelled to Dresden where I attended Teich and Wendelin's defence of their thesis. Then I travelled to Warsaw, Minsk, Moscow, St. Petersburg and to Helsinki where I gave a lecture.

In 2002 I made my second tour around the globe. After I had attended Uve Steimann's defence of his master's thesis in TU-Dresden I visited Rio de Janeiro, Sao Paulo, Buenos Aires, Santiago de Chile, Lima, and Melbourne. There I gave a contribution to an IABSE conference. (Tveit 2002). India was a fairytale that lasted 15 days. I gave two days' courses on network arches in Kolkata and Hyderabad. I was one of the main speakers at the Indian Concrete Society (2002a) They thanked me by paying for my tour to the most fantastic Taj Mahal. I gave six other lectures in India. Being a guest in Indian homes for 4 nights was another very pleasant and interesting experience.

Because of the conflict between India and Pakistan, I had to visit Dubai two times to get to Pakistan. In Pakistan I gave lectures in Karachi and Lahore. I was very impressed with the beauty of Lahore. The last leg of the journey was Egypt. I gave a lecture in Cairo and saw the sights together with friends and family. At the end of October I gave a lecture on network arches at a conference on modern steel bridges organized by the Public Roads Administration of Norway. (Tveit 2002c).

Before Easter in 2003 I was invited to give one of the main lectures in a bridge conference organised by IABSE in New Dehli. (Tveit 2003). They paid the air fare. I also gave some lectures in Hyderabad and wrote a contribution for a conference there. (Tveit 2003a). After the defence of Brunn and Schanack's master's thesis in TU-Dresden I went to Luxemburg to ask some questions about steel and give a lecture there. There were only two people in the audience. Later that year I wrote a contribution for a volume honouring my old Hungarian friend Professor Peter Lenkei. (Tveit 2003b).

In 2003 much time was spent on the design of a network arch in Nybergsund. That is 350 m above sea level and as far inland as you can get in Norway. There the ice on the river would last long every winter. I wanted the steel skeleton of the network arch spanning 116 m to be erected on ice. (E-6) Then the steel skeleton should have been lifted onto the pillars. The concrete tie could have been cast in the spring. In the end the bridge office of the public roads decided to build a bridge with three much shorter spans. I was sorry about this because the method of erection suggested for the network arch in Nybergsund can be used in northern Sweden, Finland, Russia Alaska and Canada.

In June 2004 I gave a lecture at a conference in Novi Sad, Serbia, on bridges in the Danube basin. (Tveit 2004). In the autumn of 2004 I went on a lecture tour to Spain and Portugal. I lectured in San Sebastian and Lisbon. I was keynote speaker in a conference on network arches in Barcelona. (Tveit 2004). Professor Grasse and five of his former students gave contributions. (Brunn et al. 2004), (Graße et al. 2004), (Räck et al. 2004) and (Teich 2004). There was also a contribution on network arches from the Gent University in Belgium (Backer 2004)

In 2005 I found that a network arch spanning 41 m had been built in Bechyne in The Czech Republic. (Šašek 2005 and 2006). It was designed by Ladislav Šašek. He heard about the network arch from a colleague, looked at my homepage and designed the bridge that can be seen at (Tveit 2008 pp. 92a to 92c). Šašek has now won a competition for the building of a network arch in Prague. The main span will be 200 m long.

In 2005 I made my third tour around the globe. My wife and my brother joined me on a visit to Peking. We also saw the terracotta soldiers and the old capital Xi'an. After they went home, I gave lectures in Jiaotong University and Highway Research Institute of the Ministry of Communications of China in Beijing. Then I flew to Shanghai where I gave two lectures. After a very interesting train ride to Hangzhou, I gave a lecture in Zhejiang University.

The next day I gave a lecture in the Changsha University of Sciene and Technology and the Hunan University of Science and Tehnology. In the weekend I had a very interesting visit to Mao Zhendong's birthplace. Then there was a lecture in Guangzhou Municipal Engineering Institute and after that the Shenzhen Municipal Engineering Design Institute. I then went to visit Hong-Kong, Taipei, Bangkok, Hanoi, Ho Chi Min City and Kuala Lumpur.

I was surprised to find that there seemed to be no ill will towards the USA in Vietnam. My host Professor Dr. Tong Tran Tung told how he started his studies by working on rebuilding his university in the jungle. In Ho Chi Minh City I saw advanced drawings of a planned network arch for the main road to Hanoi.

Then I flew to Fuzhou where I gave a lecture at a university and did some sightseeing before I presented my contribution to The 4th International Conference on New Dimensions in Bridges, 24-26 October 2005. (Tveit 2005). After the congress I gave a lecture in Kowloon. The principal organiser of the congress was Professor Bao Chun Chen. He also arranged my lecture tour in China. It was a movable feast where I met many very interesting engineers.

From Vietnam I went to Manila, Kuala Lumpur and Korea where I gave a lecture at Seoul National University. An interesting young lady met me at the airport in Inchon. Many came to listen to my lecture. A limousine with a driver with white gloves drove me back to the airport. After ~7 km he was called back because I had forgotten something.

Before going home I gave lectures in San Francisco and Lima. On a shorter tour to a congress in New Dehli in India my brother, my wife, and a friend of my wife joined me. (Tveit 2005). We visited Agra, Jaipur, Hardiwar and Corbut Park.

In 2006 a German lady, Monica Stacha, did her master's thesis with me in Grimstad. She worked on the Brandanger Bridge. (Tveit 2008 pp. 93c to 94a). It is a one lane bridge with a main span of 220m. Monica Stacha now works at the bridge office of the Norwegian public roads. - In the middle of August I gave two lectures at the technical university in Trondheim. It was to a group of fourth year students from 20 European countries. The lecture can be found on my home page http://home.uia.no/pert/ under the button "My Publications".

In March 2007 Professor Graße and I gave a lecture on network arches in the 17th Bridge-building Symposium in Dresden. There were 1200 in the audience. (Graße & Tveit 2007). During a four day visit in April I gave a lecture in Moscow. I was treated like royalty. Engineers came all the way from St Petersburg to listen to me.

In the autumn of 2007 I made my fourth trip around the globe. First I gave a lecture at a conference on network arches in Madeira. (Tveit 2007). For the rest of the trip I used addresses from the IABSE member file. In New York I met Ted Zoli. He is director of the bridge department of the HNTB Corporation. He arranged that I gave lectures to classes in Columbia and Princeton Universities. He has done impressive work on network arches. (Zoli and Woodward 2005). The next lecture was at the Federal Highway Administration in Washington.

On the way to Peru, I gave a lecture in Queretaro in Mexico. In Lima I gave lectures at The Catholic University. In Chile there was great interest in the network arch. I was shown the plan for their first network arch before I left. In New Zealand I gave two long and two short lectures. A year after they sent me a picture of the erected steel skeleton of the Mangamahu Bridge. See fig. J-20 and pp. H-8 to H-10. Another network arch will be built over the Waikato river on the North Island. The Waikato River network arch was finished early 2011. See pp. H-11 to H-13

The steel skeleton of the Mangamahu Network arch in New Zealand was lifted in place by mobile cranes in 2008. See Fig. E8. It has a one lane span of 85 m.

Fig. J20. Steel skeleton of the Mangamahu network arch that was lifted in place by mobile cranes

I also gave a lecture in Sydney in Australia. In Jakarta I stayed in luxury with Professor Irvan Katili. He was the head of a civil engineering department in the University of Indonesia in Jakarta. I gave a lecture in his department. The next lecture was in Singapore. Singapore shows what the Chinese can achieve helped by peace and a global economy. In Dhaka in Bangladesh a lecture was arranged by I A Khan. There I slept under a mosquito net in his firm's guesthouse. I had never slept under a mosquito net before. A servant washed my clothes and cooked my meals. In Bangladesh they seemed very interested in network arches. They need many.

In Hyderabad I stayed once more with my good friend Professor Prakash Rao. He arranged that I gave six lectures in five days. He also helped me to write a contribution to a congress in Hyderabad in July 2008. (Tveit 2008c). After my stay the police asked questions about me. Maybe somebody has become suspicious about me because I use to say: "Like a good Muslim, which I am not, I do not drink any alcohol". Nobody wanted my lecture in Tehran so I flew straight home after 22 lectures and 77 days of travelling.

In March of 2008 I made a trip to the Balkans. I gave an invited lecture at a conference in Montenegro. (Tveit 2008b). Afterwards I gave lectures in four towns. Skopje in Macedonia was very interesting. The Greek boycott of the name Macedonia seemed very stupid to me. I told my hosts that the boycott was favourable for them. They would probably have the protection from NATO and most of the advantages of EU without the full cost of membership. Pristina in Kosovo was very quiet. Maybe it was because it was raining. Along the road to Macedonia there were burnt out Serbian villages. In Sarajevo there were few signs of the war. In Mostar we had dinner near the rebuilt old Turkish bridge.

Early in May of 2008 I was present in Santander in Spain when one of my former students, Frank Schanack, defended his doctoral thesis on network arches. (Schanack 2008 and 2008a). He got the highest grade possible and was then probably the man who knows most about network arches. Now he is a professor in Chile. Later in May I gave a lecture at a fip-congress in Amsterdam. (Tveit 2008a).

I was invited to be a keynote speaker at a conference in India in June 2008. I am sorry that I did not go, but I was short of money and time. In the autumn of 2008 IABSE let me put a lecture on network arches on their internet site. (Tveit 2008c). That pleased me no end. A revised version of that lecture can be found at (Tveit 2014) - In December 2008 German Rail published guidelines on the design of Railway Bridges (Schlaich et al 2009). See p. A-5. They were very favourable to network arches.

Fig. J21. Drawing of a railway bridge in New Jersey, USA

In May 2009 Director Ted Zoli of the NY branch of HNTB invited me to New York. He wanted me to convince Amtrack that a railway bridge in New Jersey should have network arches in the six main spans. I succeeded. In 2009 I spent a lot of time trying to prove that the 20 km bridge over the Fehmarn Belt between Germany and Denmark could be made of high strength concrete. Two very good students from TH-Dresden, Marcus Krug and Steffen Müller did some of the fundamental calculations that indicated that it could be done.

In June 2009 I had two contributions at "The Fifth Symposium on Strait Crossings" in Trondheim. (Tveit 2009a) was on submerged floating tunnels. (Tveit 2009b) was on "Efficient Utilisation of Network Arches". In November I was invited to lecture at "2009 World Steel Bridge Symposium" in San Antonio in Texas. I was the first speaker and allowed to speak for 40 minutes. My contribution and manuscripts can be found on my homepage under the button "My Publications".

On the way there Director Ted Zoli of HNTB and I spoke on network arches at the Manhattan College. On the way back I visited my Indian friend Professor Prakash Rao in The University of the West Indies in Trinidad. I gave two lectures there.

In August 2009 an eleven page article on network arches was published in New Delhi, in Civil Engineering & Construction Review (Tveit 2009b).

In October of 2010 I made a trip to China. In Fuzhou I gave a lecture on network arches at the 6th International Conference on Arch Bridges. ARCH'10 (Tveit 2010). A week later I gave a lecture in the First International Symposium on Archimedes Bridge in Hangzhou (Tveit 2010a). Between the two conferences, Professor Yiqiang Xiang organized a lecture on network arches to students of the Zhejiang University.

In November 2010 and March 2011 I was in Kiel in Germany to discuss the design of the Levensauer Bridge over the canal between the North Sea and the Baltic Sea. It was a very interesting design of a network arch. For the meeting in March 2011 Dipl.ing. Thomas Janßen also invited my former student Dipl.ing. Benjamin Brunn. In the winter of 2011 to 2012 I joined a team that came up with suggestions for how the Levensauer Bridge over the North Ostsee Kanal. Benjamin Brunn was the coordinator of the goup. Our suggestion did not win.

In 2010 I sent a picture of the Brandanger Bridge to Director Dr. Man-Chun Tang of T.Y. Lin. He worked on the Fehmarn Sound Bridge when he first came from China to Europe. After a doctorate in Germany he went to the USA. When he got the picture of Brandanger Bridge, he wrote back asking if it was a pedestrian bridge since it was so slim. When I said that it was a road bridge, he invited me to Chongquing in China to give a lecture in his firm.

When I was in Chongquing in 2012 I wanted to give a lecture in a conference so that my university could get some money. It was arranged that I gave a lecture in the 9th Asia Pacific Transportation Development Conference. In the program for the conference it said: Per Tveit, by invitation. The fact that I had invited myself was not mentioned.

On the way to China I had a 7 hour stop in Helsinki. Professor Dr. Aarne Jutila organized that I gave a lecture on network arches. My lecture was mentioned as interesting in the monthly internet bulletin from IABSE.

After my last trip to China I had help to write over 30 pages on network arches in English and Chinese. See my home page http://home.uia.no/pert under publications.