An Introduction to the Network Arch

Lectures at NTNU Trondheim on August 15th 2006

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These lectures were given at a summer course on bridges for third and fourth year students of civil engineering. Page numbers in *italics* refer to "The Network Arch", which can be found on my home page http://pchome.grm.hia.no/~ptveit/ Most references are given like [Name year]. These lectures will end up on my home page as an introduction to network arches.

Contents

Key words: Bridges, network arches, arch bridges, railway bridges, road bridges, hanger arrangements, Nielsen-Lohse bridges, concrete ties, economy, aesthetics, lightness, slenderness, steel weight, erection.

SUMMARY containing a bit more than is mentioned in the next 27 pages.

Compared with conventional bridges, the network arch, where the tie is a concrete slab, usually saves more than half the steel weight. The details are simple and highly repetitive. Thus the cost per tonne is not very high.

Network arches are arch bridges with inclined hangers where some hangers cross other hangers at least twice. In its optimal form the tie is a concrete slab with partial longitudinal prestress. The transverse bending in the slab is usually much greater than the longitudinal bending. The partial prestress reduces the cracks in the tie. This is part of the reason why the two Norwegian network arches are in good shape after over 40 years.

The arches should be universal columns or American wide flange beams less than 18 m apart. They are attractive slender bridges that do not hide the landscape behind them. *P. 7.* A network arch bridge is likely to remain the world's most slender arch bridge. The slender tie leads to short ramps and makes it easier to branch out roads at the end of the bridge. Like any tied arch the network arch can be seen as a beam with a compression and a tension zone. An increased rise in the arch will give smaller axial forces in the chords and lower steel weights. It is mainly aesthetic considerations that limit the rise of the arches. Most of the shear force is taken by the vertical component of the arch force. Much of the variation in the shear force is taken by the hangers. They act like a light web.

For load cases that relax none or only very few of the hangers, network arches act very much like many trusses on top of one another. They have little bending in the tie and the arches. To avoid relaxation of many hangers, the hangers should not be inclined too steeply. Small inclination of hangers will increase the bending moments due to concentrated loads. Therefore a compromise must be sought. All hangers should have the same cross-section and nearly the same decisive load. Their upper nodes should be placed equidistantly along the arch.

Because there is little slenderness between the nodal points of the arch, and tension is predominant in the rest of the structure, this type of bridge makes good use of high strength steel. [IABSE 2005]. Network arches are very stiff. This is very important when the network arch is used for railway bridges, especially in bridges for high speed railways.

The local conditions will influence the type of erection. Sometimes the tie can be cast on a temporary scaffold. *P. 7a.* After the arch and hangers have been erected, the hangers can be tensioned till they carry the tie. The arch and hangers supplemented by a light temporary lower chord can be moved when lifted at both ends. *P. 12.* This steel skeleton can be erected on side-spans or on ice between the abutments. *P. 30b*. It can also be lifted in place by pontoons and floating cranes. When the span is in place, this steel skeleton has enough strength and stiffness to support the casting of the concrete tie. Finished network arches spanning 200 m or more can be moved to the pillars by means of pontoons or big floating cranes. This is more likely to take place in coastal areas.

The fact that the optimal network arch uses so little materials makes it environmentally friendly in a broader sense. Unemployment is a problem in most countries. A high percentage of the cost of network arches is wages. Thus the network arches would make possible more bridges and more employment from the same limited funds. The building of optimal network arches can bring great savings. Considering the great poverty in the world, it would be morally wrong not to use network arches at suitable sites. General conservatism is probably the main obstacle to the use of this very promising structure.

PREAMBLE

When I was a student over 50 years ago, I got the idea of the network arch. It is an arch bridge with inclined hangers. Some of them cross other hangers at least two times. - I look forward to giving these lectures. I have a lot to tell you. I can think of no better audience. If you were all professors, Max Planck's statement would have been relevant. "Professors do not alter their opinions, they die out". You are more likely to absorb my ideas.

Fig. 1. Traffic on bridge

In the next six pictures the author will try to explain why the network arch is so efficient. The purpose of a bridge is to take traffic over an obstacle. The traffic can be on a road as in this slide. Often there is little room for members under the traffic.

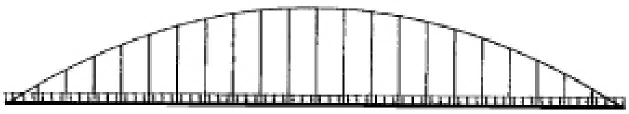


Fig. 2. Arch for an evenly distributed load.

For an evenly distributed load an arch with vertical hangers as shown in fig. 2 is a good solution. All members have mainly axial forces. In concrete arches the effect of creep must be counteracted by curvatures near to second degree parabolas. When the arch is made of steel, it should have curvature more like a circle.

For uneven and changing loads it is best to use crossing hangers like in the network arch in fig. 3. Here too the arch can be part of a circle. [Brunn and Schanack 2003] have explored a more advanced shape of the arch. Near the wind portal they use a reduced curvature of the arch. See chapter 6 of their Master's thesis. It can be found at http://fag.grm.hia.no/fagstoff/~ptveit/ In a network arch all loads are transferred to the arches in such a way that there is very little bending in the chords. The bending in the members is usually less than in trusses.

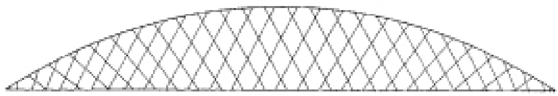


Fig. 3 show skeleton lines for a network arch from [Tveit 1980]. P. 8 and 59-72.

Fig. 4. Shape of lower chord.

The simplest tie would be a concrete slab like in fig. 4 spanning between the arches. The tensile force in the tie is best taken by prestressing cables in the edge beams. When there is little or moderate load on the span, the compressive force gives a beneficial compressive stress in the tie. This leads to less cracking and less maintenance for the tie.

Fig. 5 shows the necessary thickness of a concrete slab between arches. [Teich and Wendelin 01] p. 109. The bending in the middle of the slab is normally bigger than the longitudinal bending in the tie. *P. 13 and 14.* Thus there is normally no need for longitudinal steel beams in the tie. For distances over 10 m between the arches transversal prestress should be considered.

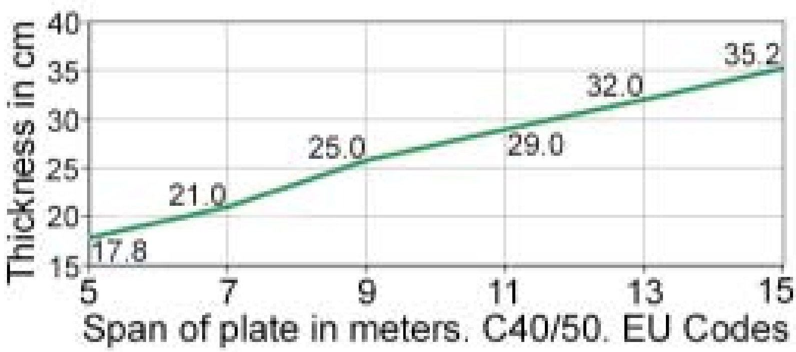


Fig. 5. Concrete slab between the arches.

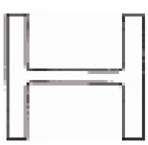


Fig. 6. Arch

The hangers give the arch good support in the plane of the arch. Universal columns, as shown in fig. 6 give very slender arches. The universal columns in the arches should have vertical flanges. Still the buckling strength can be about the same in the plane of the arch and out of the plane of the arch.

THE FIRST NETWORK ARCH

Fig. 7. Bridge at Steinkjer, Norway, built 1963-1964

Fig. 7 shows the author's first network arch. [Tveit 1964] *P. 5a to 6a*. We are going to visit it tomorrow. It was built because it was less costly than a competing alternative. It is a mistake that there are no rails between the traffic and the hangers. Still the bridge is in good shape after more than 40 years.

The arch is part of a circle. The hangers are placed equidistantly along the arch. They all have the same cross-section and nearly the same maximum tension.

The railing is placed at the outer edge of the footpath, but the wide handrail gives the pedestrians a feeling of security. *P. 25.* The tie is a simple concrete slab with small edge beams. The prestressing cables that take the axial force between the arches are placed centrally to reduce the stress variation that can cause fatigue.

Due to shrinkage and creep the bridge was expected to become a little bit shorter over the years. This has not happened, so the planned shortening of the handrail has not been necessary.

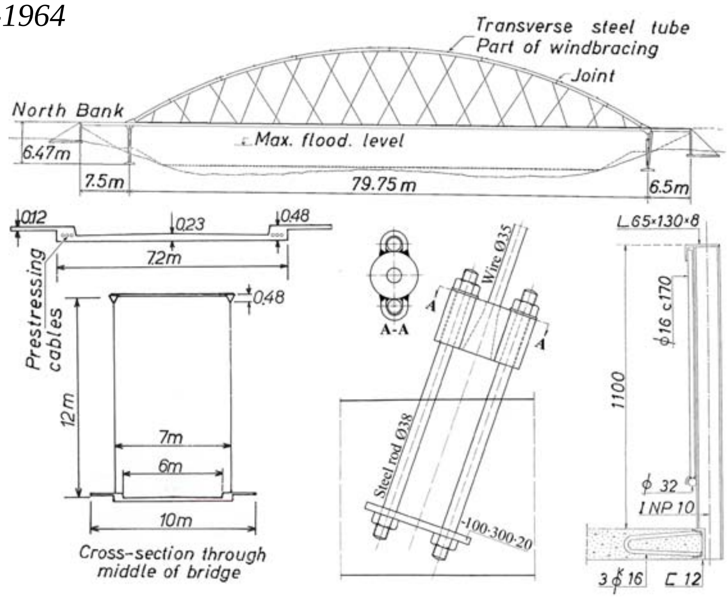


Fig. 8. The network arch at Steinkjer.

Fig. 9 shows a hanger

The lack of shortening in the tie might have many causes. For one thing 450 kg of cement per cubic m of concrete were used. In the moist, cold climate the delayed hardening of the cement might have led to a slight expansion of the concrete.

It was the author's luck that Terje Moe, a very able young architect, advised me when I designed the Steinkjer Bridge. He said: "Let your design show the flow of the forces in the bridge." He later went on to become a professor of architecture.

In a private conversation Man-Chung Tang, chairman of T. Y. Lin International once said: "We do not dress just to cover and keep ourselves warm. The same applies to our housing. Why then should our bridges be the cheapest structures that can get us from A to B?" He certainly had a point.

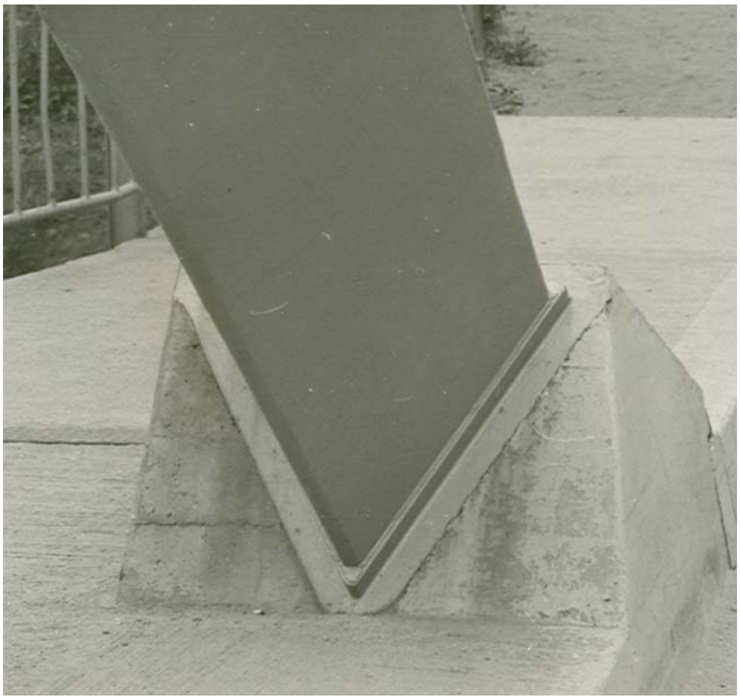


Fig. 10 shows an end of the arch at Steinkjer.

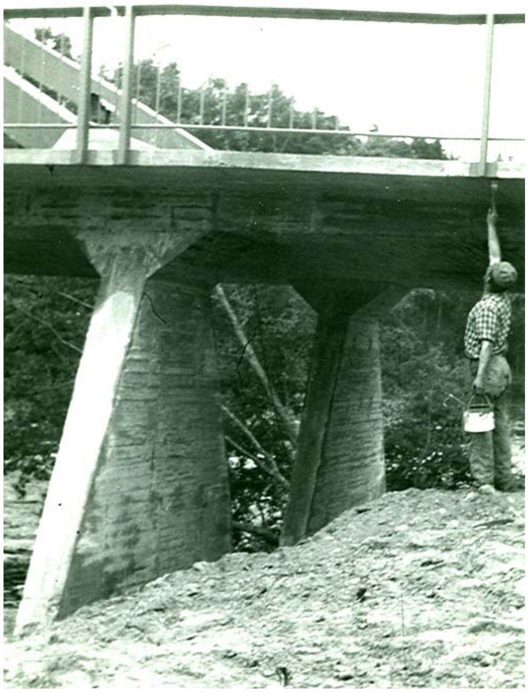


Fig. 11. Joint between arch and side span.

The author's experience is very limited, but he would like to warn against letting architects design bridges. That could be very costly. Architects' advice should be sought, but the engineer should have the final say.

At the northern end of Steinkjer Bridge a side span is joined to the main span. The column under the joint is shown in fig. 11. The first 6.7 m of the triangular arches are filled with concrete to increase the resistance to collapse due to colliding lorries.

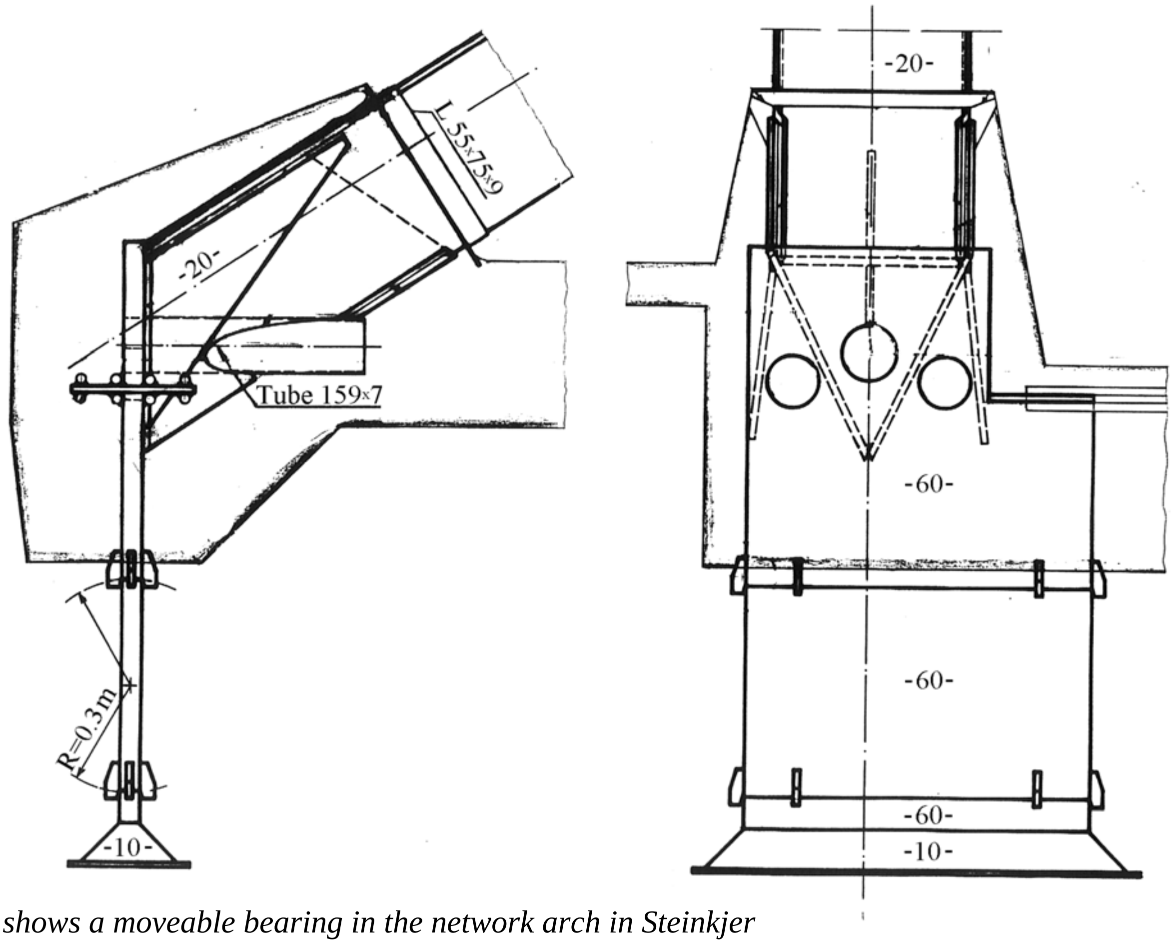


Fig. 12 shows a moveable bearing in the network arch in Steinkjer

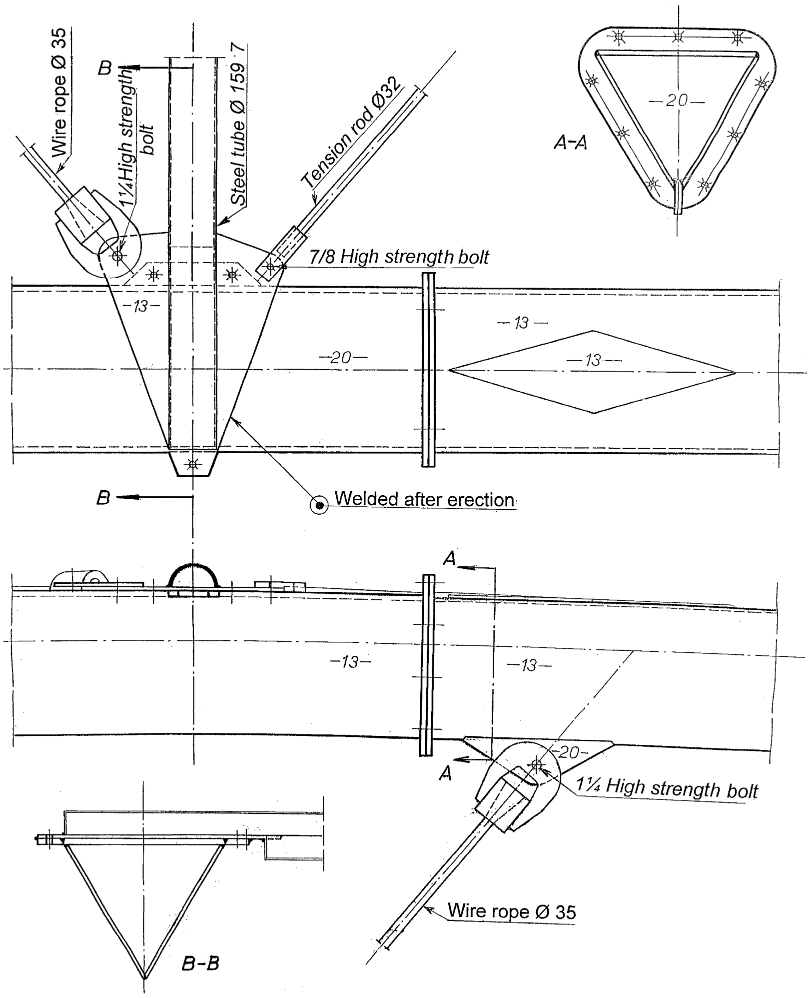


Fig. 13. Details in arch at Steinkjer.

Fig. 14. Prior to the opening.

Fig. 13 shows details around the second tube of the wind bracing of the Steinkjer network arch. The first diagonal in the wind bracing is like a hanger. The other diagonals are steel rods. The joints in the arch are simple flanges because pressure is predominant in the arch.

Fig. 14 shows the Steinkjer Bridge prior to the opening. The architect suggested that the red colour was adopted, but the author lacked the courage to paint the bridge red. After seeing red bridges in China he has changed his mind.

ARCHES MADE FROM H-PROFILES OR BOX SECTIONS?

The Steinkjer Bridge would have been even more competitive if the arch had been a universal column or an American wide flange beam. Figs 15 and 16 show how simple it is to fasten hangers and diagonals to that cross-section. Two ways of fastening the diagonals to the arches are shown in fig. 16.

Fig. 17 shows a joint in the arch. There is no tension in the arch. The bolts in the flanges are needed only during erection, but there is no need to take them away afterwards. There is a hole to drain away rainwater. When there is just a drizzle, the rain-water will run along the lower edge of the flange and not run down along the hangers.

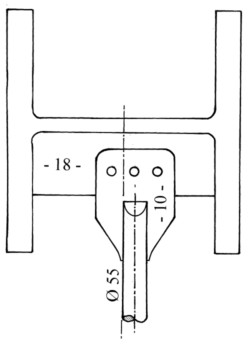


Fig. 15. Fastening of a hanger to the arch.

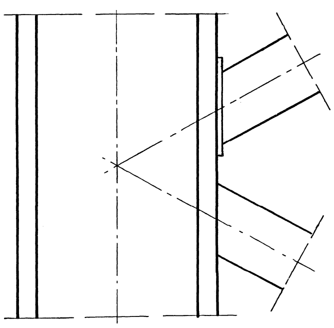


Fig. 16. Joint in wind-bracing.

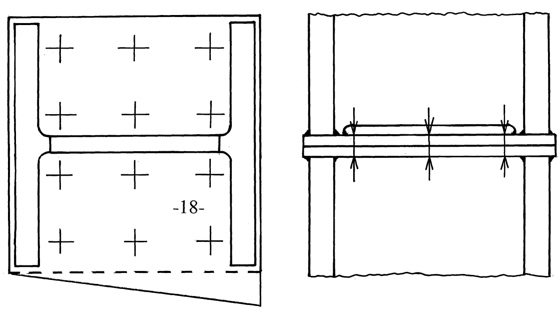


Fig. 17. Joint in arch.

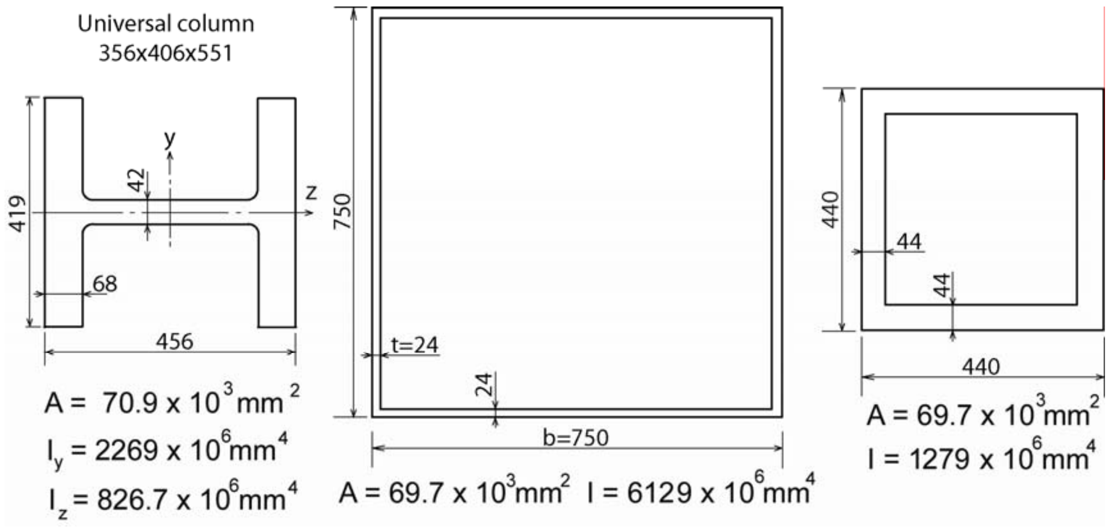


Fig. 18. Three cross-sections that have the same area.

Fig. 18 compares the three cross-sections with the same area. *p. 24*. The box section in the middle looks less slender than the universal column. The two other cross-sections have the same diagonals. The universal columns have a good distribution of stiffness, because in the plane of the arch the support of the arch is better than the support out of the plane of the arch. [Tveit 73] p. 8-12. Fig. 19 shows how simple it is to attach the end of the arch to the end of the tie.

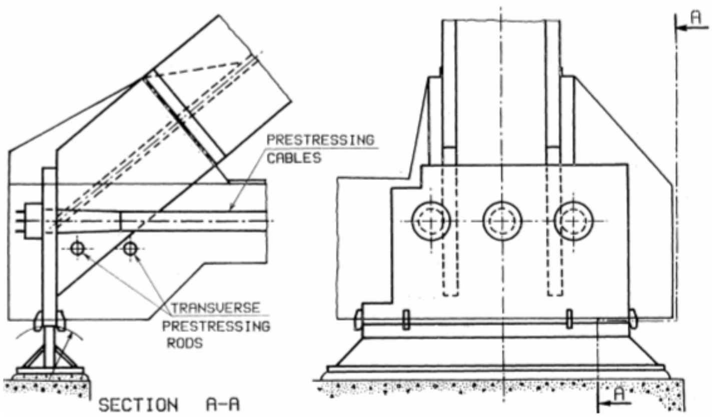


Fig. 19. An end of a network arch.

Fig. 20 shows the forces in the middle and at the ends of the chords due to an evenly distributed load. More formulas for the axial force in the chords can be found in [Tveit 1966] p. 251.

Due to the triangular shape of the influence lines, the axial force due to a concentrated load in the middle of the span is roughly twice as big as the axial forces due to an equally big, evenly distributed load. Preliminary hanger forces can be found by looking at examples on *pp. 57, 58, 60 and 72*.

ON THE STATICS OF NETWORK ARCHES

The calculation of network arches is simplified by the fact that the axial forces are dominant. It is simple to find the axial forces in the chords. Calculation of hanger forces is more complicated.

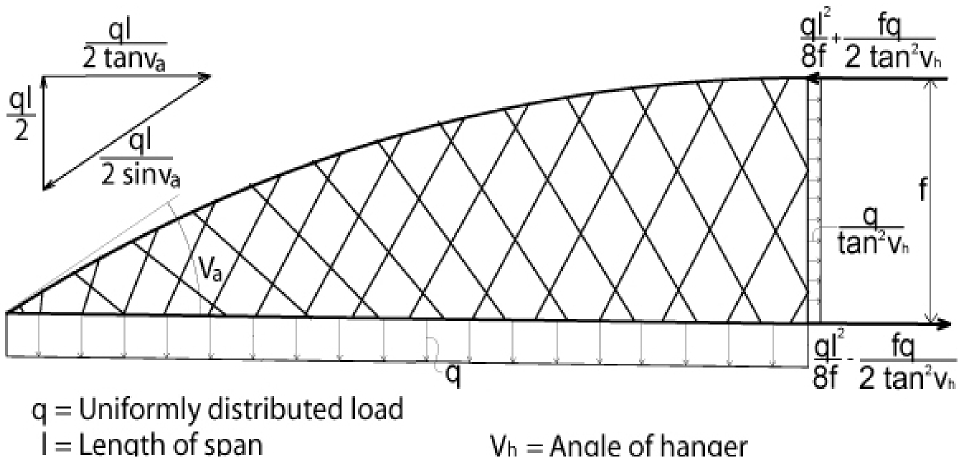


Fig. 20. Axial forces in the middle and at the end of the chords.

The forces in the arch increase only a little as we go further down from the top of the arch. This can be seen from the influence lines on *pp. 57, 58, 60, 72 and 78.* The influence lines can help designers who want to design network arches. When transferring values from one span to another, general model laws apply. *See p. 56.*

If the network arches mentioned in the paragraph above do not have suitable resistance to relaxation and/or suitable distance between the hangers look up the chapter on optimal arrangement of hangers. *See P 29j and 26 to 29i*. The pages are hard to read and will be rewritten reasonably soon.

The bending in the chords is strongly influenced by the stiffness of the chords. The bending in the lower chord influences the moderate longitudinal reinforcement. There is little bending in the arches. Thus exact information on bending in the chords is not important prior to the computer calculation.

Fig. 21 is a comparison between a network arch and an arch bridge with vertical hangers. [Tveit 1980 a and b], [Kahman and Beisel 1979]. The two bridges have nearly the same width. They both span 200 m. The network arch in fig. 21 was designed by the author for an IABSE conference in Vienna in 1980. *See pp. 59 to 62*. The bridge with vertical hangers uses only twice as much steel. The author finds this most impressive.

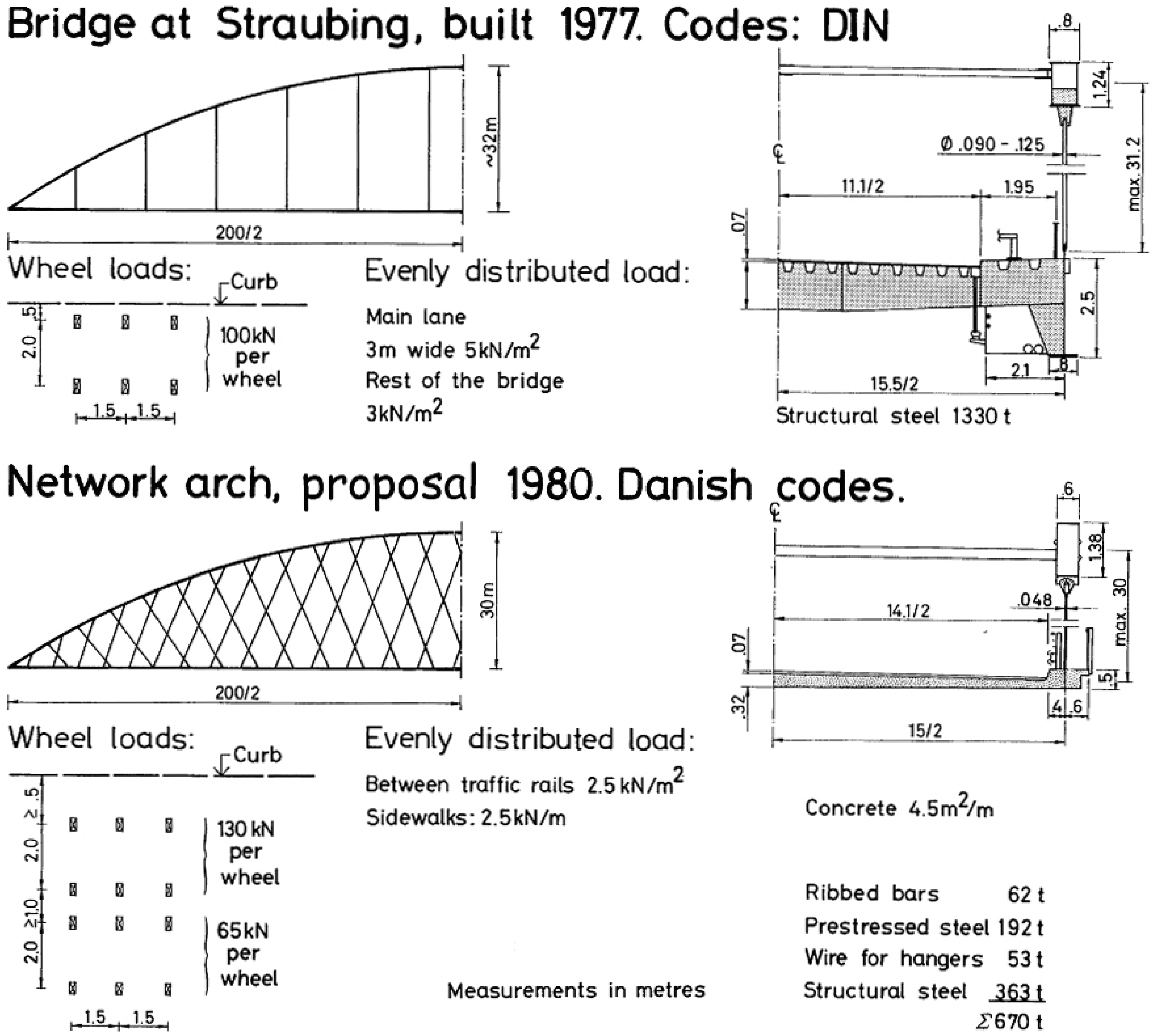


Fig. 21. Data for two bridges.

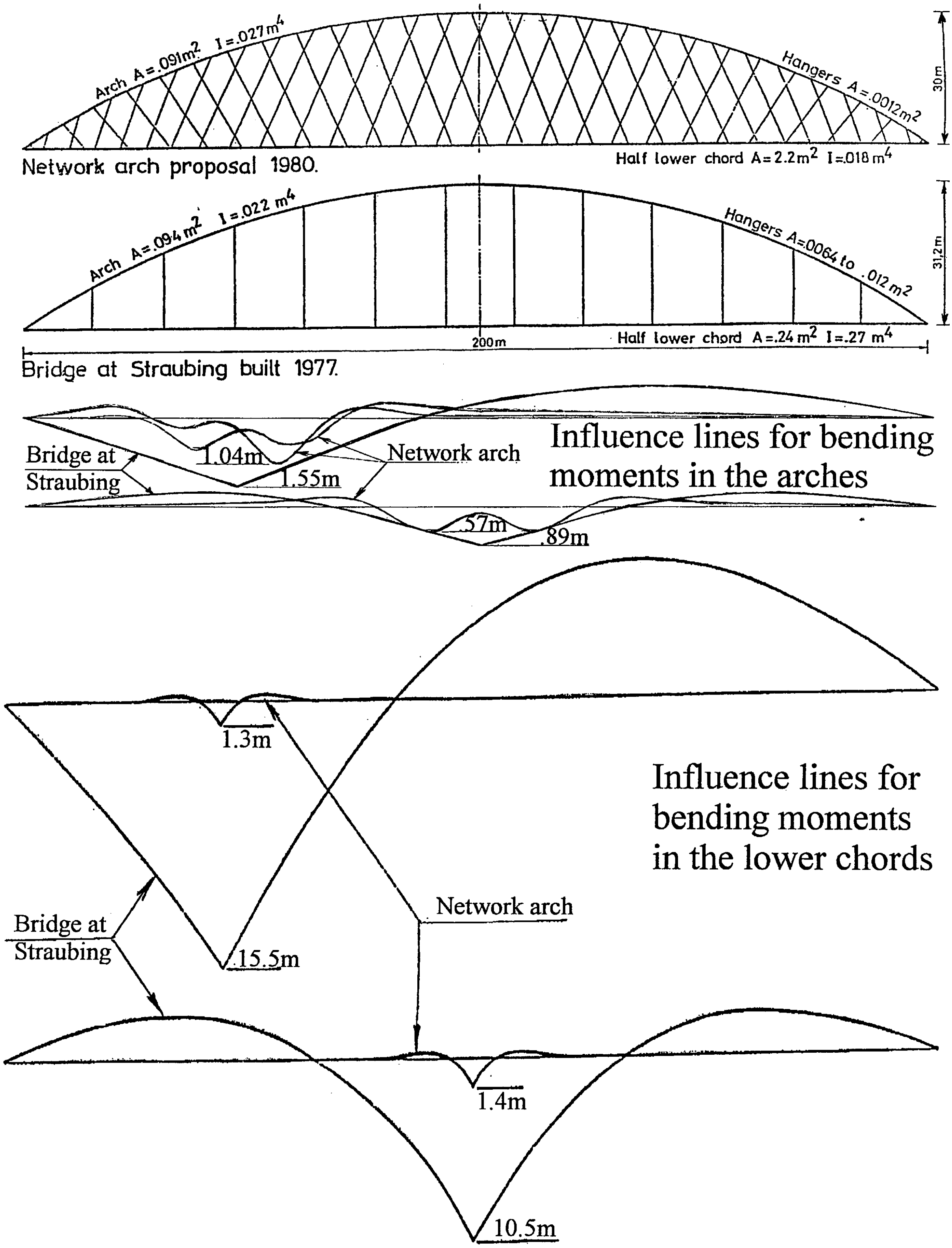


Fig. 22. Influence lines for two arch bridges.

Fig. 22 shows influence lines for the bending moments in the chords of the bridges in fig. 21 on the previous page. The influence lines show much bigger bending moments in the bridge with vertical hangers. The biggest influence ordinate for the tie in the network arch is 1.4 m. That is the same as for a simply supported beam spanning 5.6 m. Thus the biggest bending moment in the tie is normally in the middle of the slab spanning between the arches. The longitudinal bending moment in the tie is smaller. Furthermore the bending capacity of the edge beam is big.

In long narrow network arches much of the axial strength of the concrete can be needed to take the variation in the axial force. Here longitudinal bending can become decisive, especially if the tie has full longitudinal prestress.

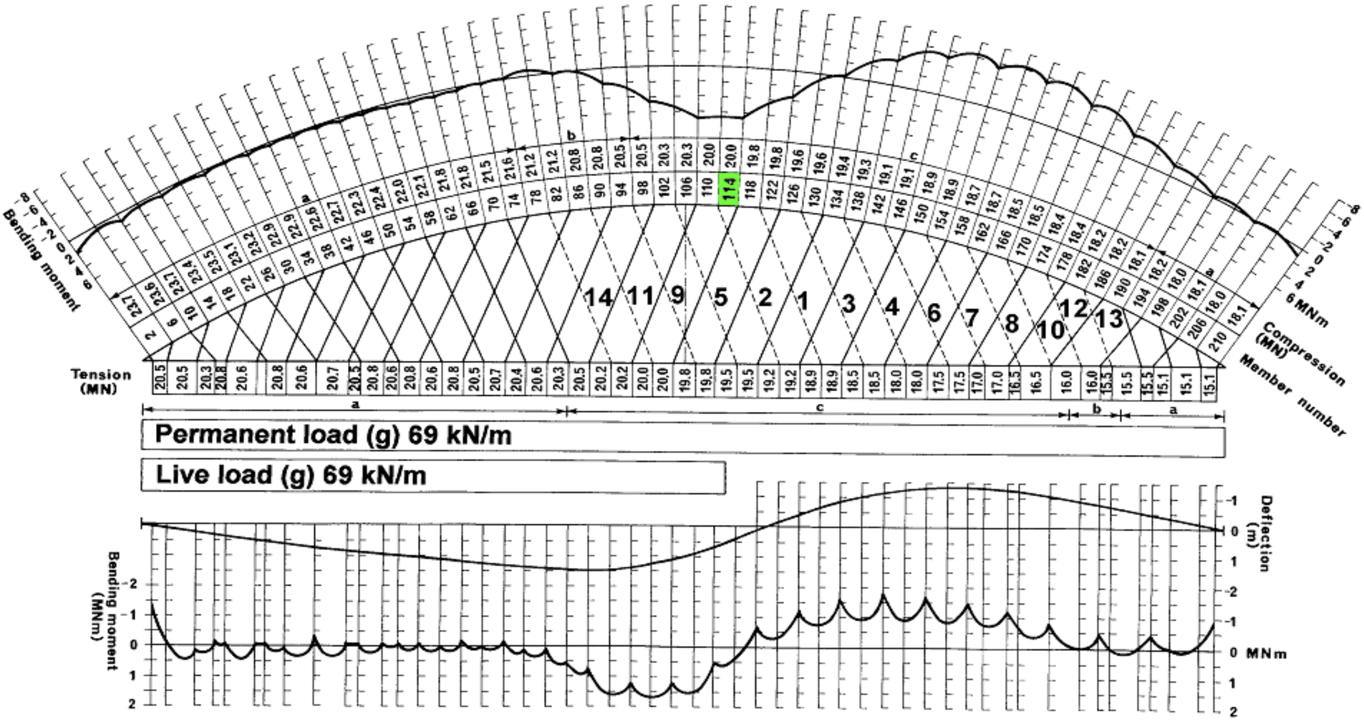


Fig. 23. Forces and deflection due to extreme skew live load on a bridge spanning 200 m.

In fig. 23 the partial live load is as big as the permanent load. [Tveit 1987] That is not likely to occur. The extreme live load causes the relaxation of many hangers. Numbers indicate the sequence of relaxation. The relaxation of hangers leads to increased deflection and considerable bending in the chords. *P. 67*. In the area where there is no relaxation of hangers, the bending moments are small. Arch member 114 is the first member where an increasing skew load gives the same stress as the load on the whole span.

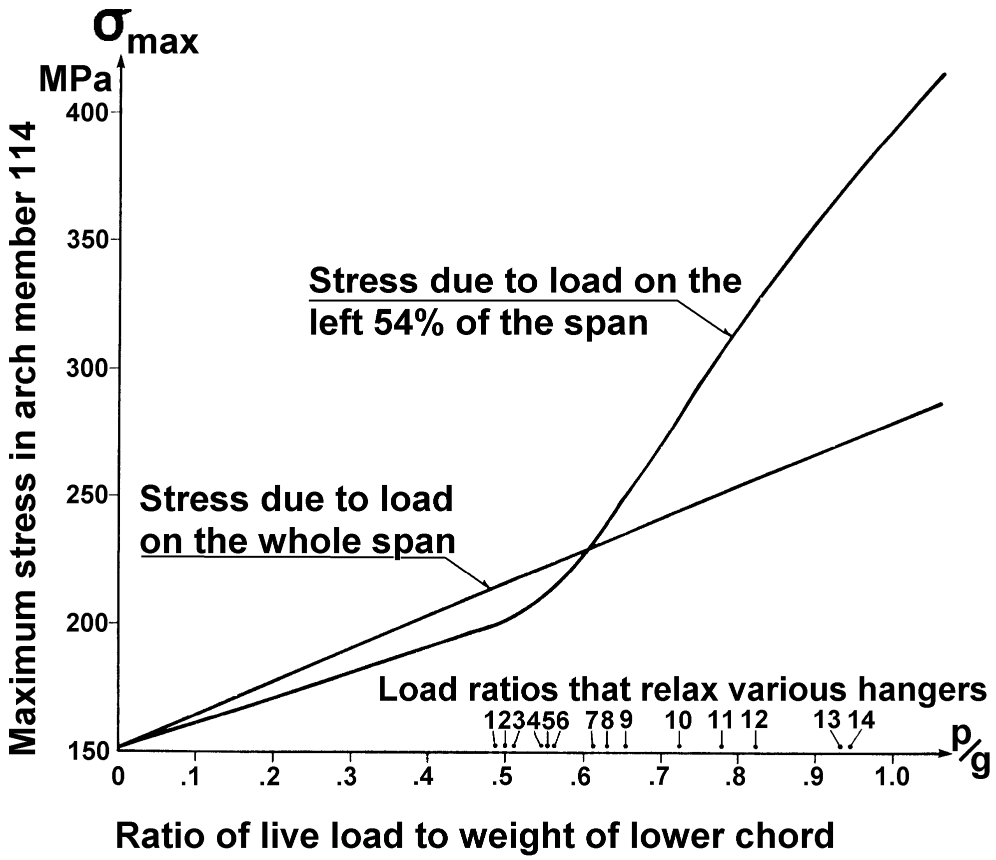


Fig. 24. Development of stresses in member 114.

Fig. 24 shows the development of maximum stress in member 114. For moderate live loads, live loads on the whole span are decisive.

Shortly after six hangers have relaxed, partial load and live load on the whole span give the same stress.

Partial live load is decisive when the partial live load is over 60 % of the permanent load. Please notice the big increase in stress when partial live load and the dead load become equally big.

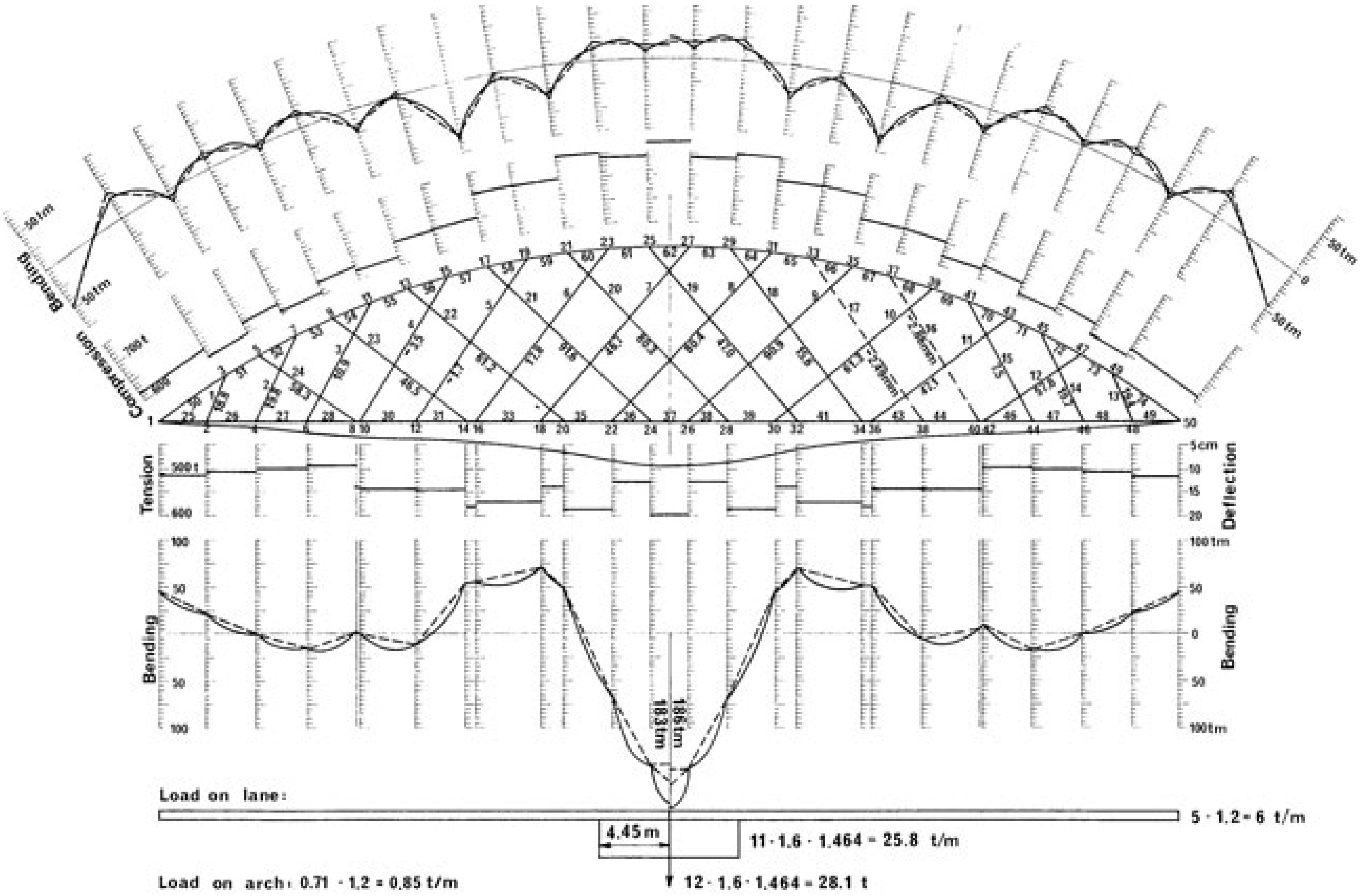


Fig. 25. Forces and deflections in a railway bridge spanning 65 m.

Fig. 25 shows forces due to a concentrated load in a railway bridge spanning 65 m. [Tveit 73] and *p.37*. On the left is the result if the hangers could take compression. On the right the hangers that relax have been removed. Where the hangers have been removed, the decrease in the distance between the nodes is given in mm.

It can be seen that the maximal forces in the hangers are very similar on both sides of the drawing. The forces are very similar as long as only two hangers relax. Thus it can be concluded that it would be a small mistake on the safe side, to calculate the hanger forces assuming that the hangers can take compression. Utilizing this assumption can save a lot of work when calculating maximum hanger forces. With some very advanced computer programs this advantage is of no use.

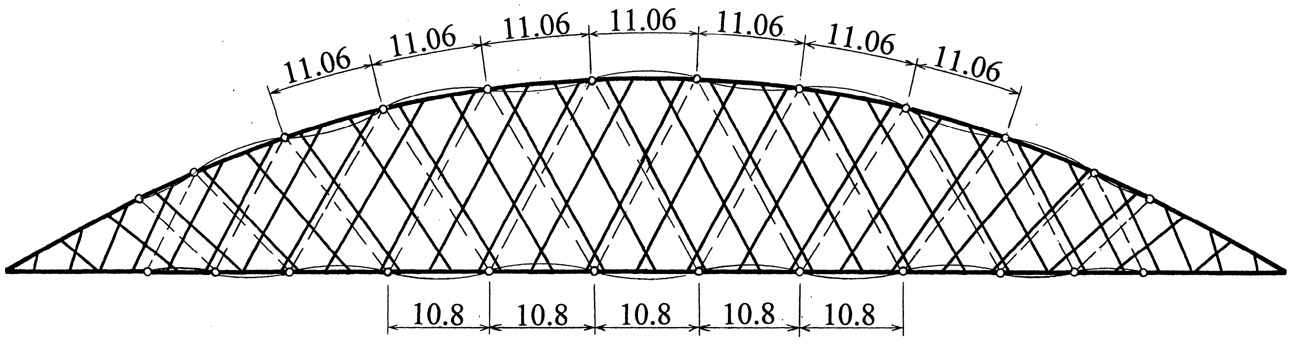


Fig. 26. Buckling in the plane of the arch.

Fig. 26 shows how the arch buckles in the plane of the arch. The buckling was first calculated manually in 1972 [Tveit 73] *See pp. 44-49*. Today it would normally be done by computer. The buckling strength is normally bigger in the plane of the arch than out of the plane of the arch. To find the ultimate strength of the bridge one can assume that the lack of precision in the building has the same shape as the buckling indicated in fig. 26. During building one has to make sure that the deviations in the shape are less than the deviations assumed.

Fig. 27. Bolstadstraumen Bridge. Span 80 m.

Fig. 27 shows the Bolstadstraumen Bridge. *P. 7, 7a and 58*. It was built in 1963. Some hangers cross other hangers three times. If we define slenderness of an arch bridge as span divided by the sum of the height of the chords, this has been is the world's most slender arch bridge for over 40 years.

It is a two lane bridge with a rise of 0.18 times the span. It used 44 tonnes of structural steel and 7 tonnes of prestressing steel. A competing tied arch bridge with vertical hangers needed 2.45 times as much structural steel. The rise of the arch was 0.205 times the span. Both bridges had a concrete slab between the arches. *P. 7*.

COMPARISON BETWEEN NETWORK ARCHES AND ARCH BRIDGES WITH VERTICAL HANGERS

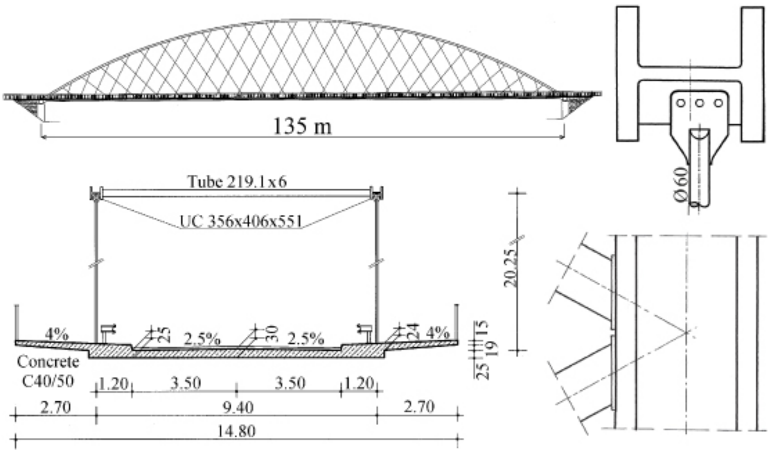


Fig. 28. Åkviksound Bridge. Designed in year 2001.

It would be relevant to compare network arch bridges to arch bridges with vertical hangers. The network arch in fig. 28 was designed by [Teich and Wendelin 2001] They were two students from TU-Dresden in Germany. They did their Master's thesis with the author in Norway. See http://fag.grm.hia.no/fagstoff/~ptveit/

In fig. 28 the arch in the network arch is an H-profile that would come cold bent from the steel mill. It has simple details. It is interesting to compare their network arch to German tied arch bridges with vertical hangers.

All the bridges in fig. 29 are designed according to German or EU loads and codes. *P. 93*. N shows that the bridges have no bracing between the arches. S shows that the arches are sloping towards each other. The German bridges use slightly more reinforcement than the network arch. That is because much reinforcement is needed to reduce the cracks on top of the elongating steel beams in the tie.

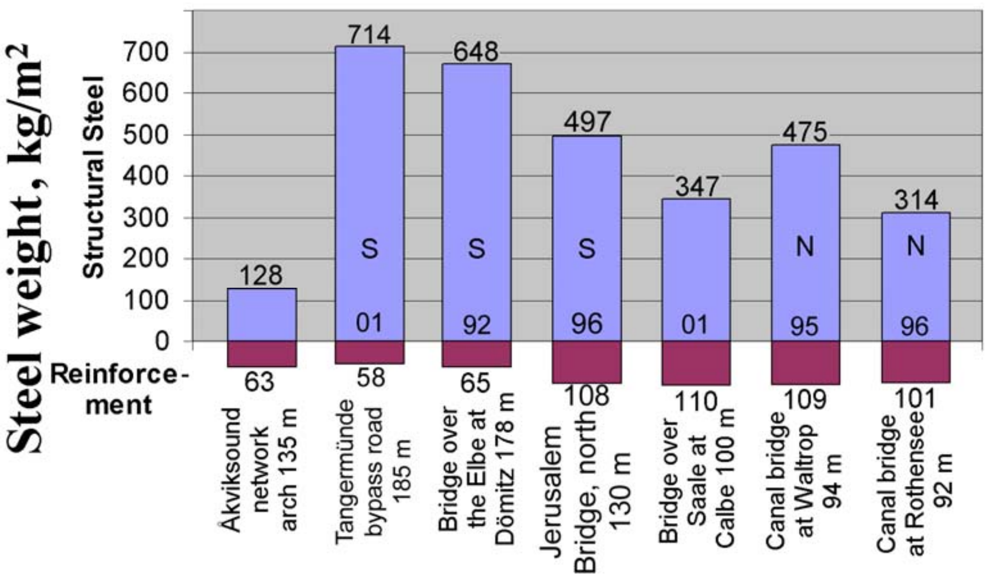


Fig. 29. Steel weight in various arch bridges.

The prestressing cables in the Åkvik Bridge are part of the structural steel. Still the Jerusalem Bridge in Magdeburg uses nearly 4 times as much structural steel per m^2 as the Åkvik Sound Bridge. The spans are nearly the same.

POINTS OF IMPORTANCE:

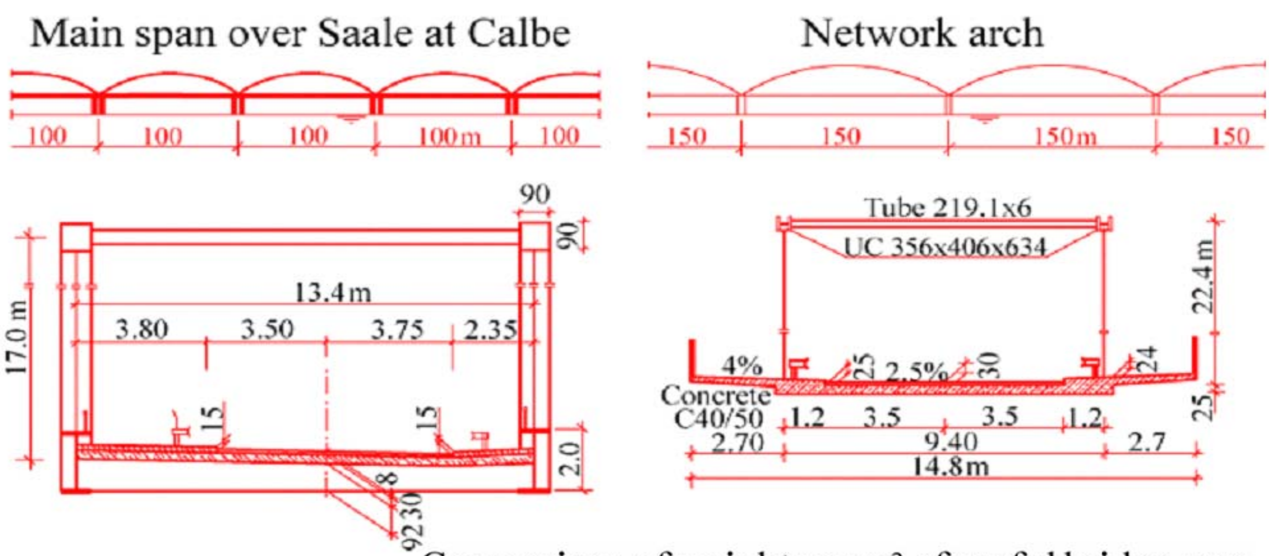
Aesthetics	Bulkier bridges
Adaptability	2 to 8 times deeper lower chords
Materials	2 to 4 times the steel weight 15 to 30 times longer welds
Fabrication	More complicated details
Corrosion protection	3 to 7 times more surface to protect
Maintenance	Other concrete parts need much more maintenance than 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.

Fig. 30. Other steel arch road bridges compared to optimal network arches.

As you all know, steel weight is not the only thing that matters. Let us look at other differences between arch bridges with vertical hangers and network arches of the Åkvik type.

Bridges with vertical hangers are less slim. They have 2 to 8 times deeper chords. They use 2 to 4 times more steel. Their welds are 15 to 30 times longer. Their details are more complicated. They have 3 to 7 times as much surface that must be painted. For the network arch it can be said that other concrete parts need much more maintenance than concrete slabs with a slight prestress. Erection is less expensive with half to a quarter of the steel weight to erect. The site will influence the method of erection. The method of erection is very important. See pp. 17-19.

In fig. 31 the materials needed for the Calbe Bridge [Fiedler 1997] are compared to the materials needed for a network arch of the Åkvik type. *P. 93.* Using a network arch we save 58 % of the structural steel. We save 34 % of the reinforcement. We save 24 % of the concrete. For the network arch the weight to be moved is smaller. The pillars are the same for the two bridges. The savings in costs are probably 35 – 45 % per m^2 of useful bridge area.



Comparison of weight per m^2 of useful bridge area

Structural steel incl. prestressing steel	100%	Reduction	58%
Reinforcement	100%	"	34%
Concrete	100%	"	24%
Min. weight to be moved during erection	100%	"	46%
Pillars are the same for both bridges	100%	"	33%

Savings in cost are probably 35 - 45% per m^2 of useful bridge area.

Fig. 31. Materials and cost of the Calbe Bridge compared to that of a network arch.

THE START OF THE NETWORK ARCH

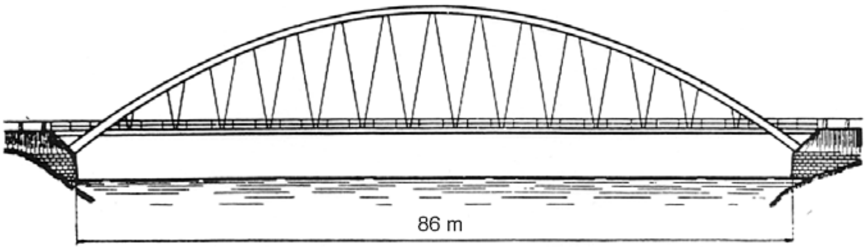


Fig 32. Bridge over Øster Daleelven in Sweden.

This second lecture starts by telling how it all started. The main part of the author's Master's thesis [Tveit 1955] was to show how Nielsen bridges could be calculated.

Fig. 32 shows the first concrete arch bridge with inclined hangers. It was built in the 1920s by Christiani & Nielsen. Design on the bridge in fig. S started in 1921.

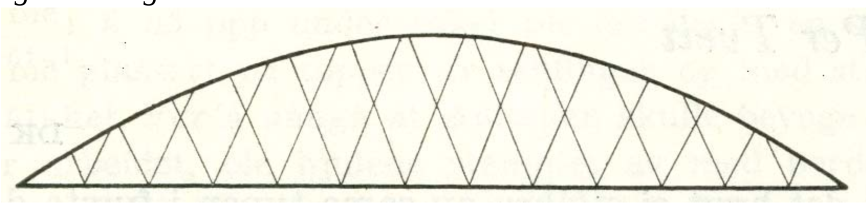


Fig. 33. Bridge from Nielsen's application for a patent in 1925.

The hangers had a constant slope because that made the calculations simpler. [Nielsen 1930, 1932 and 1936]. In the Nielsen bridges the hangers did not cross each other, but in fig. 33 which is taken from his patent application in 1925 Nielsen showed crossing hangers. In network arches some hangers cross other hangers at least twice.

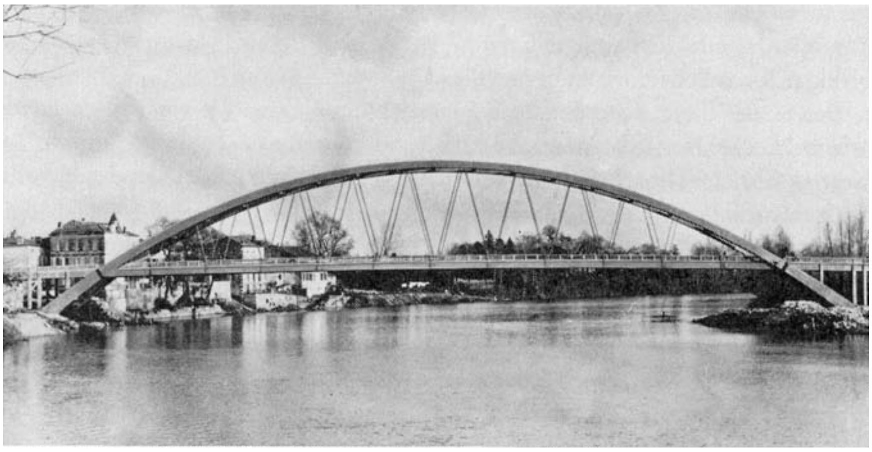
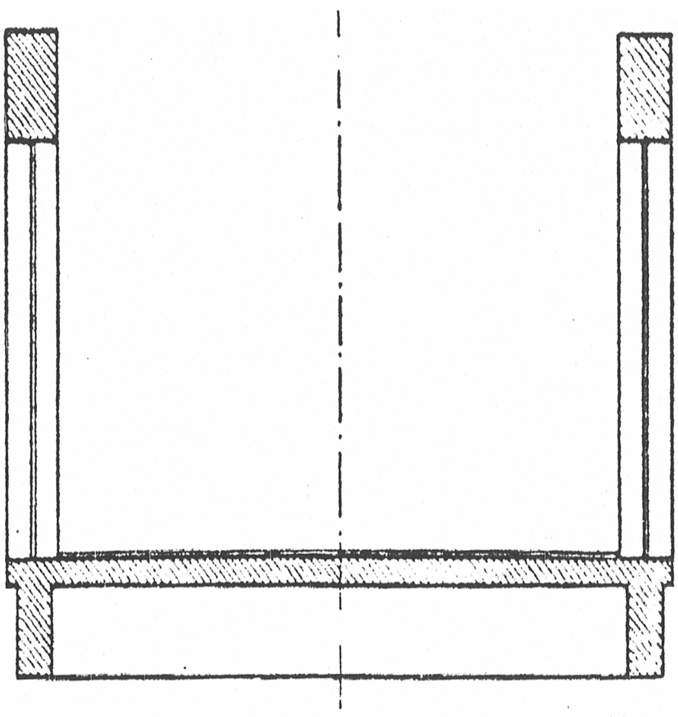


Fig. 34. The Castelmoron Bridge was built in France in 1933. Span 145. m

The bridge in fig. 34 is the longest Nielsen bridge. Around 60 Nielsen bridges were built in Sweden between the two world wars. [Ostenfeld 1976] The hangers were steel rods. Traffic loads were supposed to make them relax. In spite of a great increase in traffic loads only one or two hanger has broken in the last 70 years.

In his Master's thesis the author pointed out that bending in the chords could be reduced if the hangers crossed each other many times.



In his network arches the author has used very slim chords. Then it seems best not to let many hangers relax, because that could lead to a big increase in the bending in the chords. The author soon found that he could save 75% of the steel, but then he disregarded the reinforcement.

The author thought that many would be interested in building a bridge that saved so much steel. He certainly could not imagine that 50 years would go by before many would start building it. He underestimated the inertia and the conservatism in the bridge-building profession.

Fig. 35. Cross-section of a Nielsen bridge.

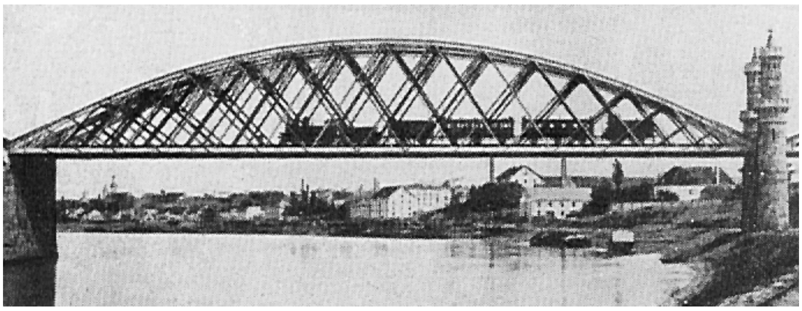


Fig. 36. Railway bridge at Rieza in Saxony, Germany.

The bridge in fig. 36 has a certain likeness to a network arch. It functions in much the same way, but the diagonals are stiff members that are tied to each other in the nodal points. The bridge was built in 1877/78. The calculations had only 56 pages. [Beger 1955]. Later trusses became very popular, partly because they could easily be calculated.

When the author studied in TH-Aachen in 1955-56, nobody else seemed interested in arch bridges with inclined hangers with multiple intersections. Professor Philipp Stein took pity on the author, helped him to build a simple model and discussed the network arch with him. When the author was building the bridge at Steinkjer, he heard about the Fehmarn Sound Bridge. See fig. 37. [Stein and Wild 1965].

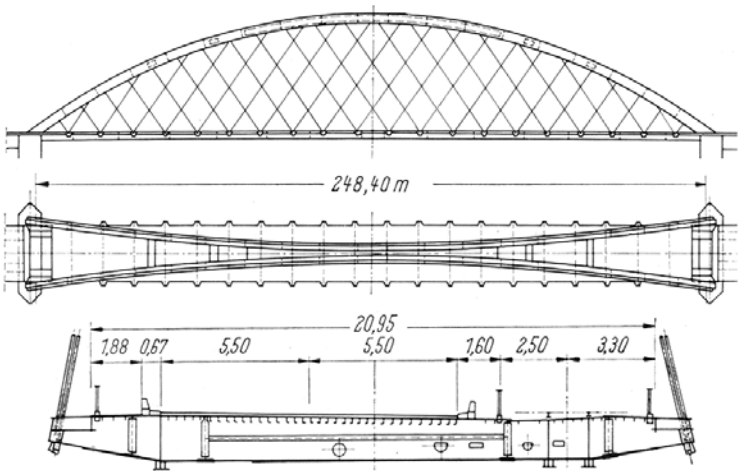


Fig. 37. The Fehmarn Sound Bridge, Germany. 1964.

At first the author thought that it was a coincidence that the Fehmarn Sound Bridge had inclined hangers with multiple intersections like the bridge in figs. 37 and 39. Ten years later he found that Professor Philipp Stein had been writing the 100 year history of Gutehofnungshütte at the time when the design of the Fehmarn Sound Bridge started.

The author wondered if his ideas of saving steel by using inclined hangers with multiple intersections could have been passed on to Gutehofnungshütte by Professor Stein. When asked about this in a letter, Professor Stein answered: “Dass ist durchaus möglich.” (That might very well have been the case). The author was quite happy about his ideas being applied to a bridge much more complicated than anything that he himself could have designed at the time.

JAPANESE STYLE NETWORK ARCHES

The Japanese 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].

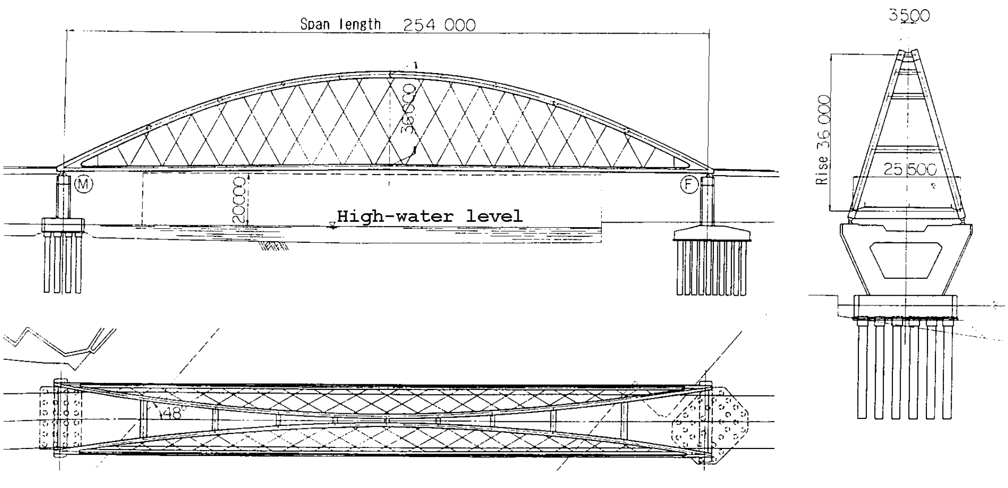


Fig. 38. The Shinhamadera Bridge built in Japan in 1991.

More than 50 bridges of this type have been built. [Hiroshi 1965], [Kikuno 1973], [Yoshikava 1993] and [Nakai 1995]. Many references to Japanese articles on network arches can be found at the end of [Tveit 1999].

The Japanese have a small country, and it is very important for them to make it more beautiful by building beautiful bridges. About half of their network arches have parallel arches. [Nakai 1995]. Most Japanese articles on network arches tell about spans with arches sloping towards each other. The Shinhamadera Bridge, fig. 38, is the longest Japanese network arch built so far. [Nakai 1995] The Japanese call these bridges "Nielsen-Lohse bridges". They might not know that O. F. Nielsen never crossed the hangers in the bridges that he built. [Nielsen 1930, and 1936].

In most Japanese network arches all hangers have the same slope. That was also the case in most of the bridges that Nielsen built in Sweden between the two world wars. It is the author's impression that the Japanese think that the constant slope of hangers looks best. See also *pp 17-19*.

Fig. 39. Lifting of the Shinhamadera Bridge.

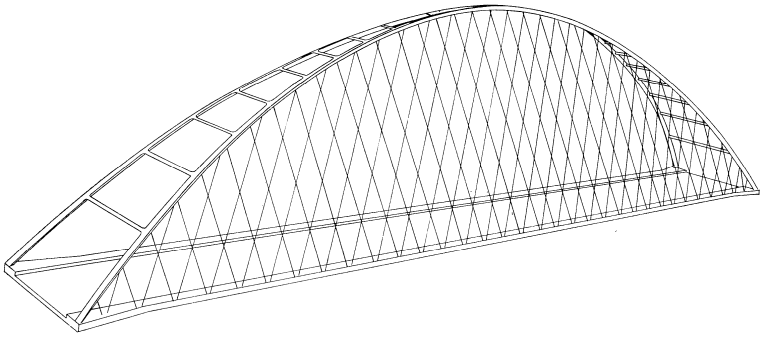


Fig. 40. Suggested bridge from the author's Ph.D. thesis.

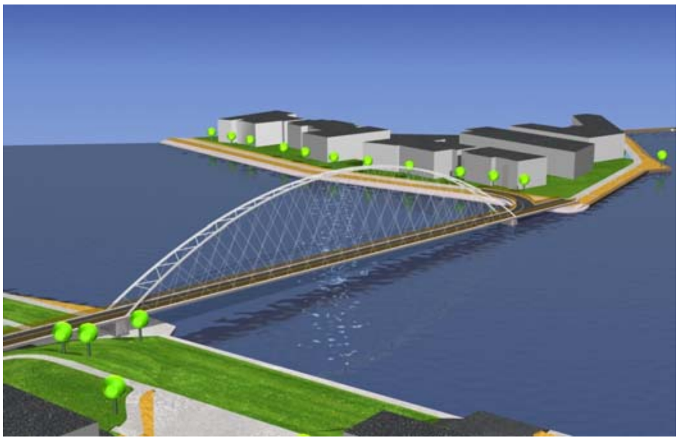


Fig. 41. Pedestrian bridge in Kristiansand, Norway.

The bridge in fig. 40 is from the author's doctoral thesis in 1959. [Tveit 1959 fig. 76] Here the arches slope toward each other. This reduces the bracing between the arches and makes the slab or transverse beams in the tie longer. This leads to increased steel weight. In the design in fig. 40 there is a high rise of the arch and the narrow footpaths. This keeps the increase of steel weight down. At the time the author still suggested constant slope of hangers and some nodes where two hangers meet. Now he would not even consider such arrangements.

Figs 41 and 42 shows a bridge for pedestrians and cyclists. It was designed in 2002 under the author's supervision by third year students at Agder University College according to the rules of a design competition in a neighbouring town. It is quite elegant. The span is 135 m. The cross-section is shown in fig. 42. The room for the pedestrians is 2 m wide. The room for the cyclists is 3 m wide. You see that the chords are very slim. Had it been built at the time it would have been the world's most slender arch bridge.

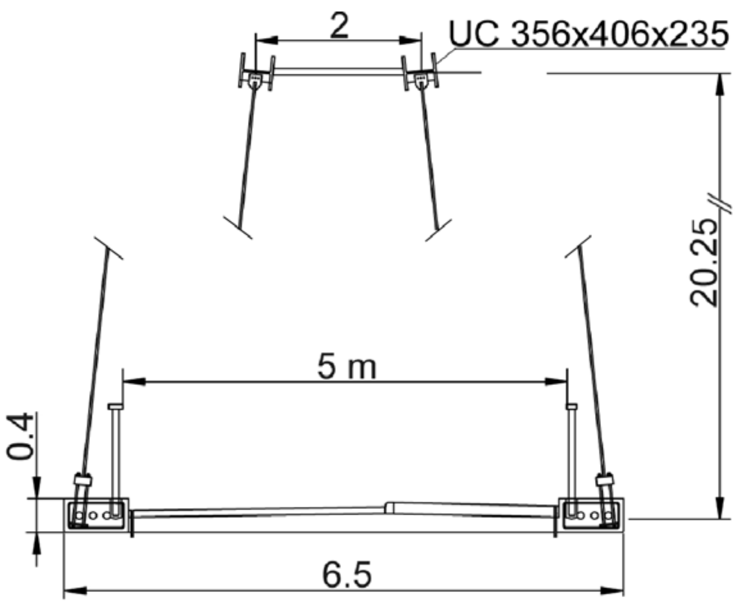
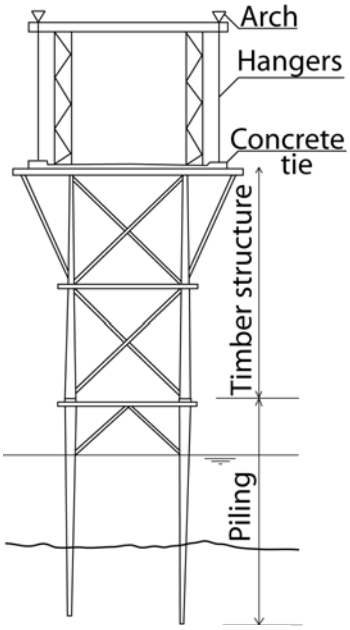


Fig. 42. Cross-section of the pedestrian bridge in fig. 41.



Erection can be done in many ways. The two Norwegian network arches were erected on a timber structure resting on piles in the river bed. [Tveit 2005] *P. 7a*. See fig. 43. First the concrete tie was cast. Then the arches were erected and the hangers were put in. Then the hangers were tightened until the arch carried the tie and the timber structure could be removed.

Fig. 43. Timber scaffold

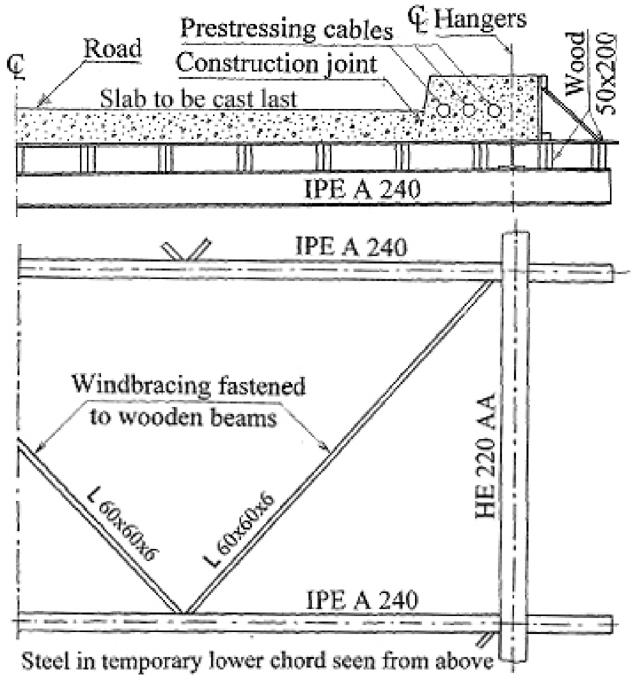


Fig.44. Temporary lower chord.

Some most promising methods of erection use a temporary lower chord. Combined with arches and hangers it makes a stiff steel skeleton. This skeleton can be moved when lifted near the ends. It has enough strength and stiffness to carry the concrete tie while it is cast. There is more on erection on pp. *12, 15, 20, 29k-30b, 40, 42, 50-55 and 92c.*

Fig. 45 shows a joint in the temporary lower chord. *P.12*. The transverse prestressing rods of the tie can be anchored to the plates at the lower end of the hangers. The hole under the end of the hangers has room for reinforcement.

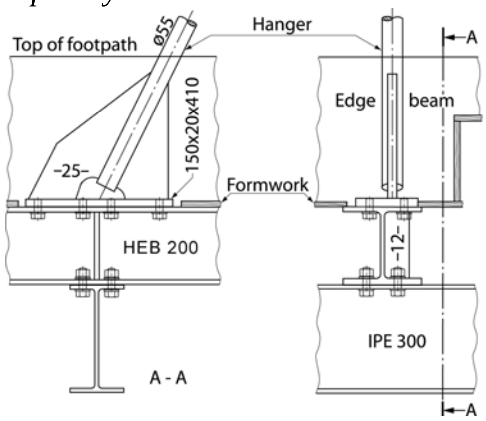


Fig. 45. Joint in temporary lower chord.

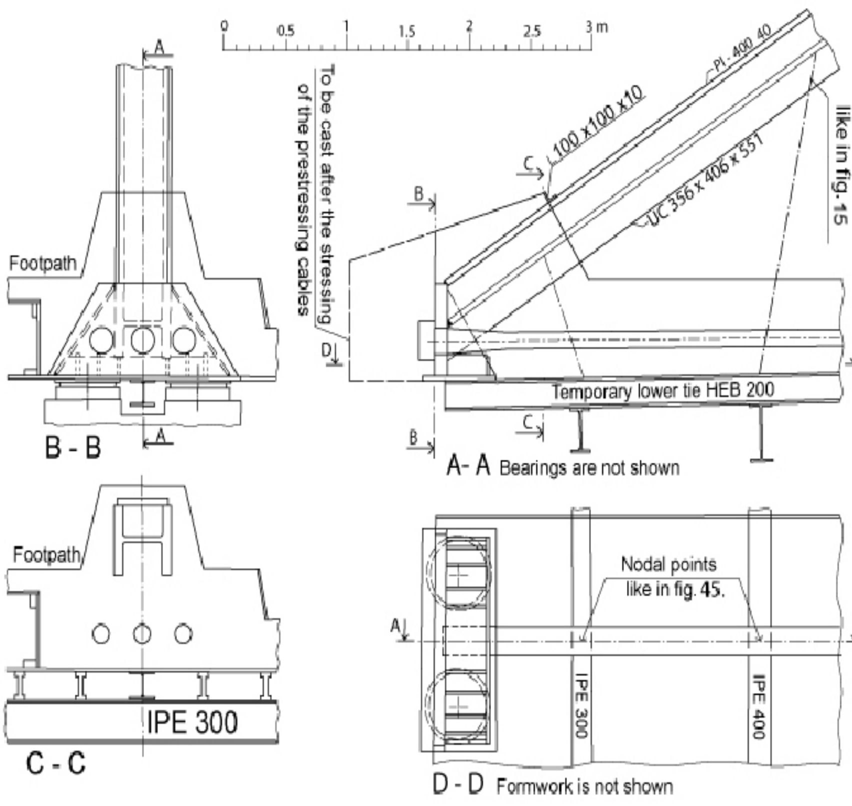


Fig. 46 indicates how the end of the span might look when there is no end beam in the tie. *P 30*. Instead the concrete slab is given extra thickness between the bearings at the end of the slab.

After the end of the temporary lower chord has been removed, the hole between the bearings has room for hydraulic presses if the bearings are to be changed.

The cavity under the plate on top of the H-profile is to be filled with concrete.

Fig. 46. End of span with a temporary lower chord.

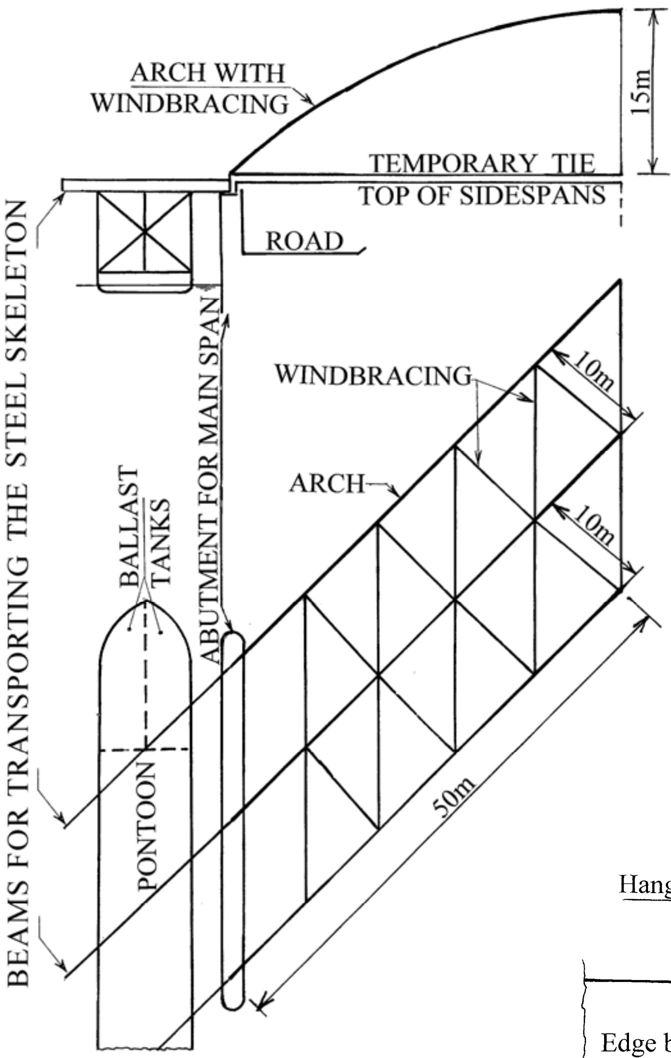


Fig. 47. A skew network arch crossing a canal.

Fig. 47 shows the first stage in the erection of a network arch crossing a canal at a 45° angle. *P. 20*. The span is 100 m. The steel skeleton is erected on the approach. If the shape of the steel skeleton is right, no later adjustment of the hanger's length is needed. To keep the thickness of the concrete tie down, three arches are used.

The steel skeleton is moved to its final position by means of a pontoon. First the beams on top of the pontoon are fastened to the abutment. Then one end of the steel skeleton rolls to the middle of the pontoon. Then the other end rolls on the side-spans.

The skeleton is so light that no strengthening of the side-spans is needed for moving the steel skeleton. Finally the pontoon is pulled to the other side of the canal, and the steel skeleton is rolled to its final position. Then the tie is cast.

Fig. 48 shows a wagon for removing a temporary lower chord. *P. 52*. The floor in the platform is a part of the form for the casting of the tie. It has been lowered and hangs in IPA profiles with wheels on top. The wheels can roll along the edge of the tie. Next to the IPA profiles are ladders for the workers.

Before the platform for removing the lower chord is taken away, the holes for the screws at the lower end of hangers must be filled with putty and scratches in the corrosion protection must be repaired.

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

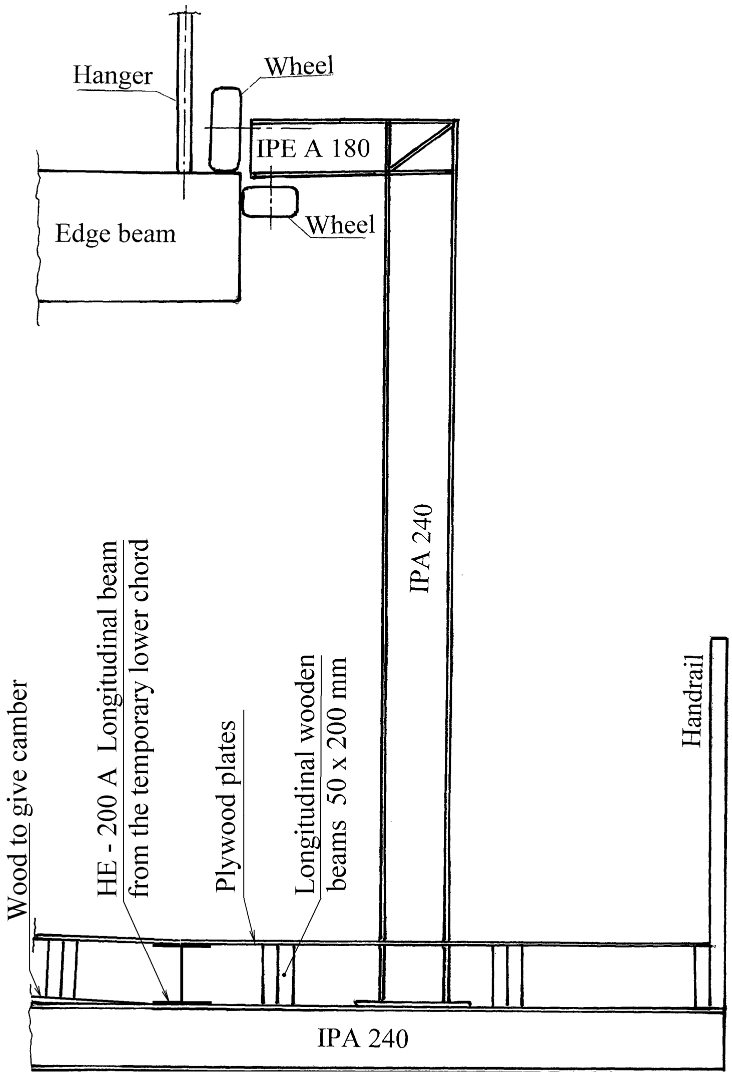


Fig. 48. Wagon for removing temporary lower chord.

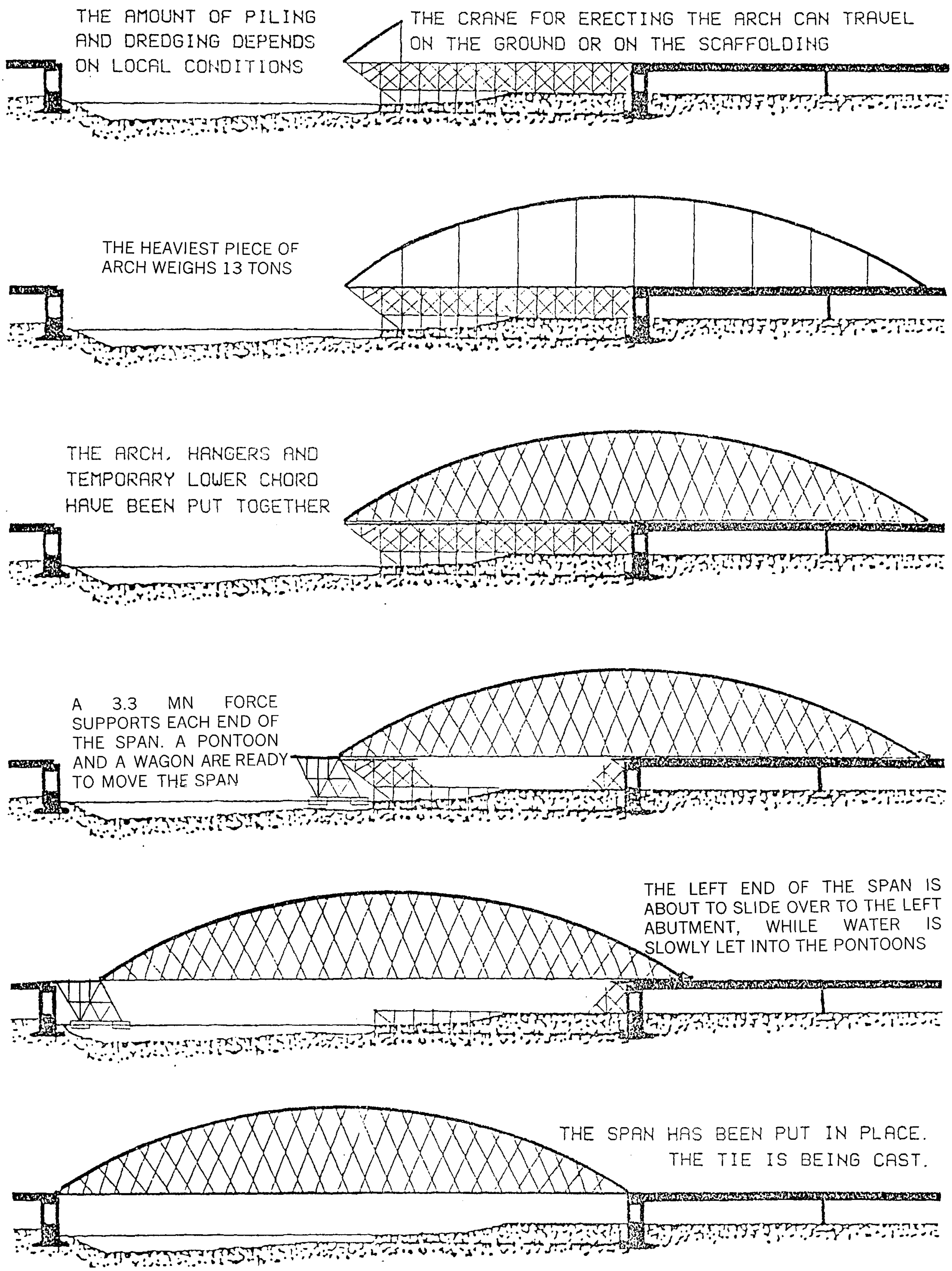


Fig. 49. Erection procedure for a network arch spanning 200 m.

Fig. 49 shows the erection of a network arch at the site of the bridge at Straubing. *P.15*. [Kahman and Beisel 1979]. The steel skeleton is erected on the approach and moved into place by means of a pontoon. Then the tie is cast. The steel skeleton is so light that no strengthening of the side spans is needed.

Fig 50. Two-track railway bridge spanning 100 m. [Brunn and Shanack 2003].

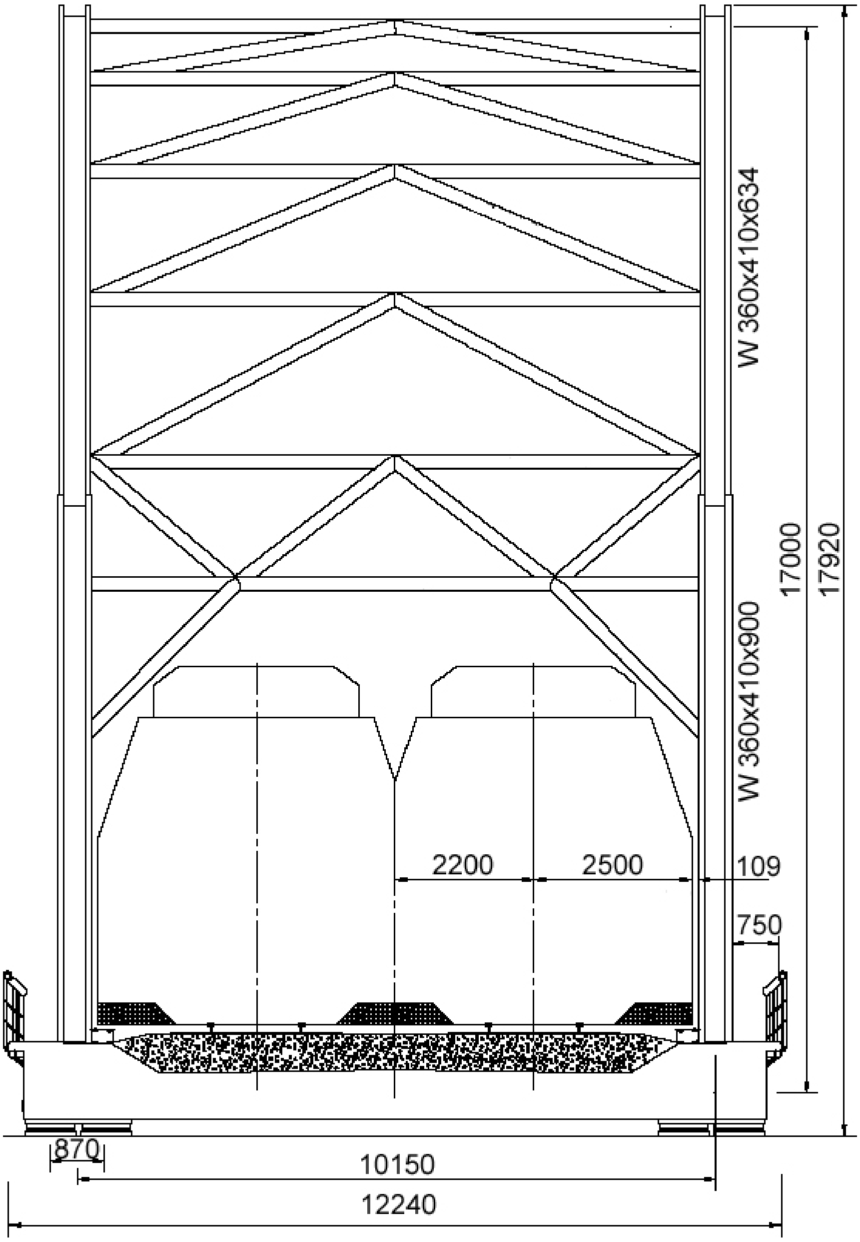


Fig. 51 shows the bridge in fig. 50

Savings in steel seem to be equally great when optimal network arches are used in rail as in road bridges. The network arch in figs 50, 51 and 54 comes from the graduation thesis of two students from TU-Dresden [Brunn and Schanack 2003]. They examined fatigue in great detail.

The thesis is written in English. Their work will be a great help to anyone who wants to design a network arch railway bridge. The EU codes were used in the design. The arches were made of American wide flange beams.

Since the network arch functions much like a truss, it is very stiff. The bridge is designed for a train speed of up to 160 km/hour. The deflection was only half the maximum allowable. Future engineers will have to decide if the network arch is suitable for high speed railways.

The concrete tie with ballast reduces the noise level. The thin lower chord contributes to shorter ramps. This is especially valuable in railway bridges in flat terrain.

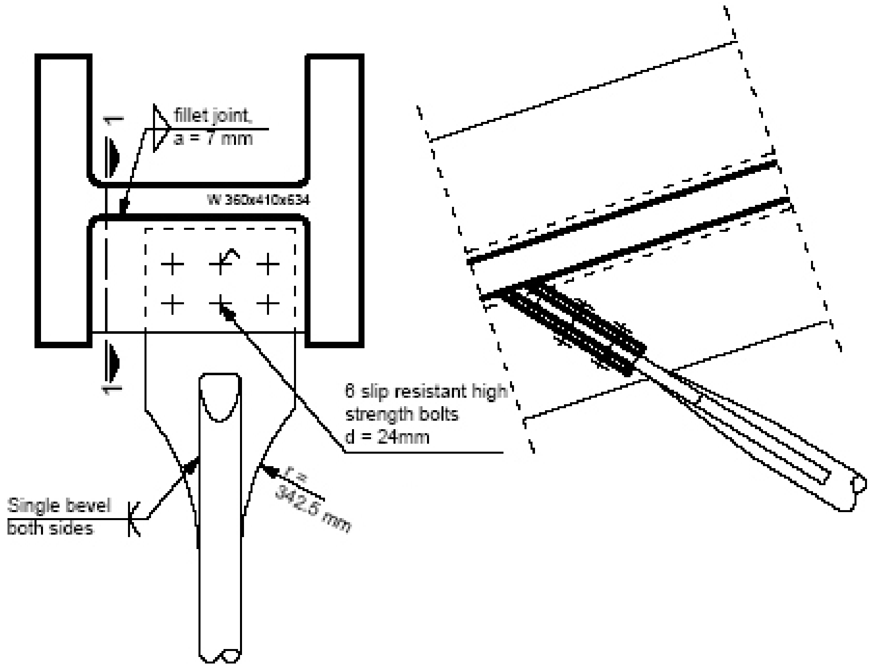


Fig. 52 shows an example of a joint between arch and hangers. Brunn and Schanack examined many examples. [Teich 2004] has examined the design of ends of hangers in great detail.

The hangers in opposite directions are fastened on different sides of the plane in the middle of the arch. The traffic might lead to torsion in the arch. This torsion is counteracted by bending in the diagonals between the arches.

Fig. 52. Joint between arch and hanger

Fig. 53 shows the curvatures recommended by [Brunn and Schanack 2003]. The span is 100 m. Compared to arches with constant curvature the curvature in fig. 53 leads to shorter wind portals and a more even maximum stress in the arch.
h is longer than the distance up to the first horizontal member between the arches. A constant cross-section can be used where R_1 describes the curvature. Fig. 54 shows the model used to calculate the railway bridge in figs 50 and 51.

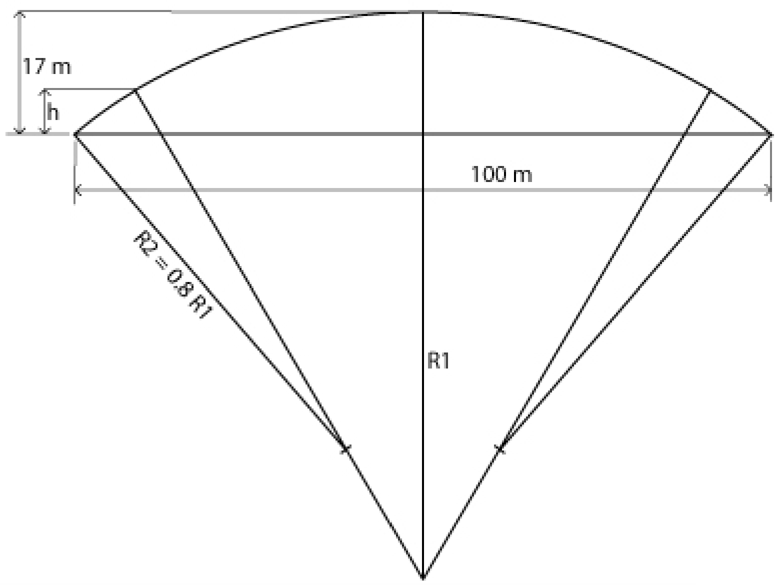


Fig. 53. Curvatures of the arch

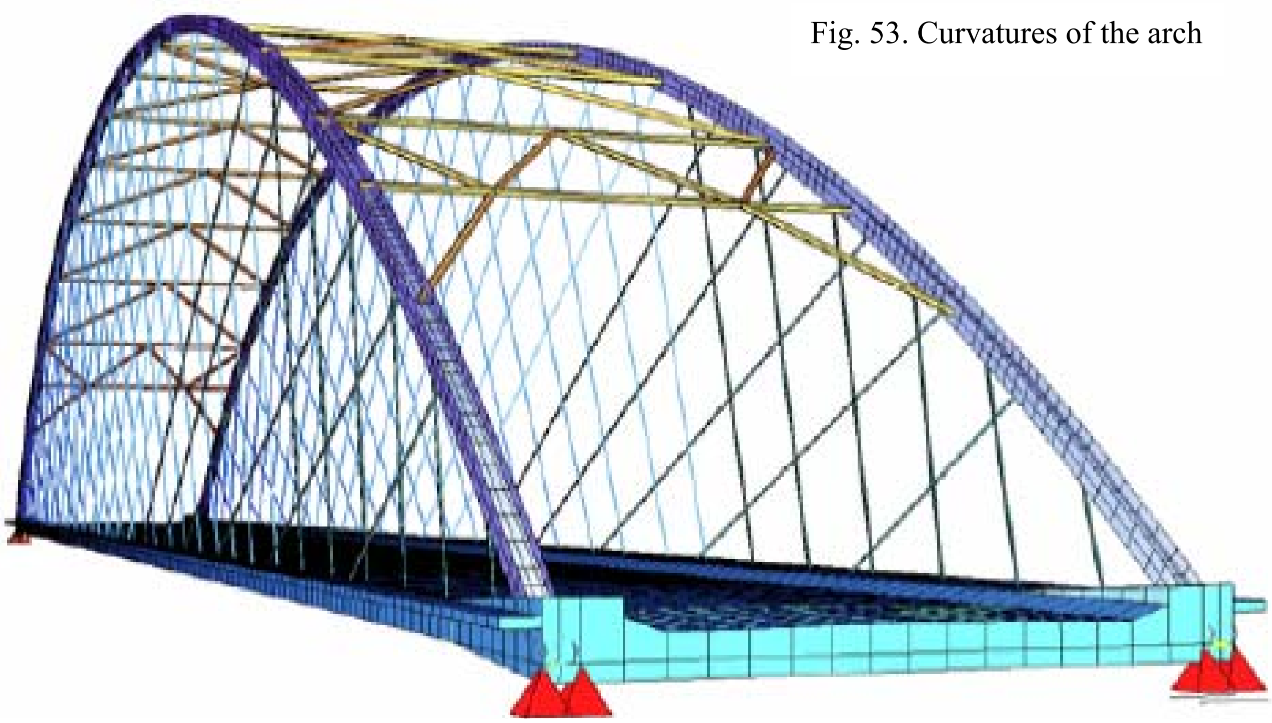
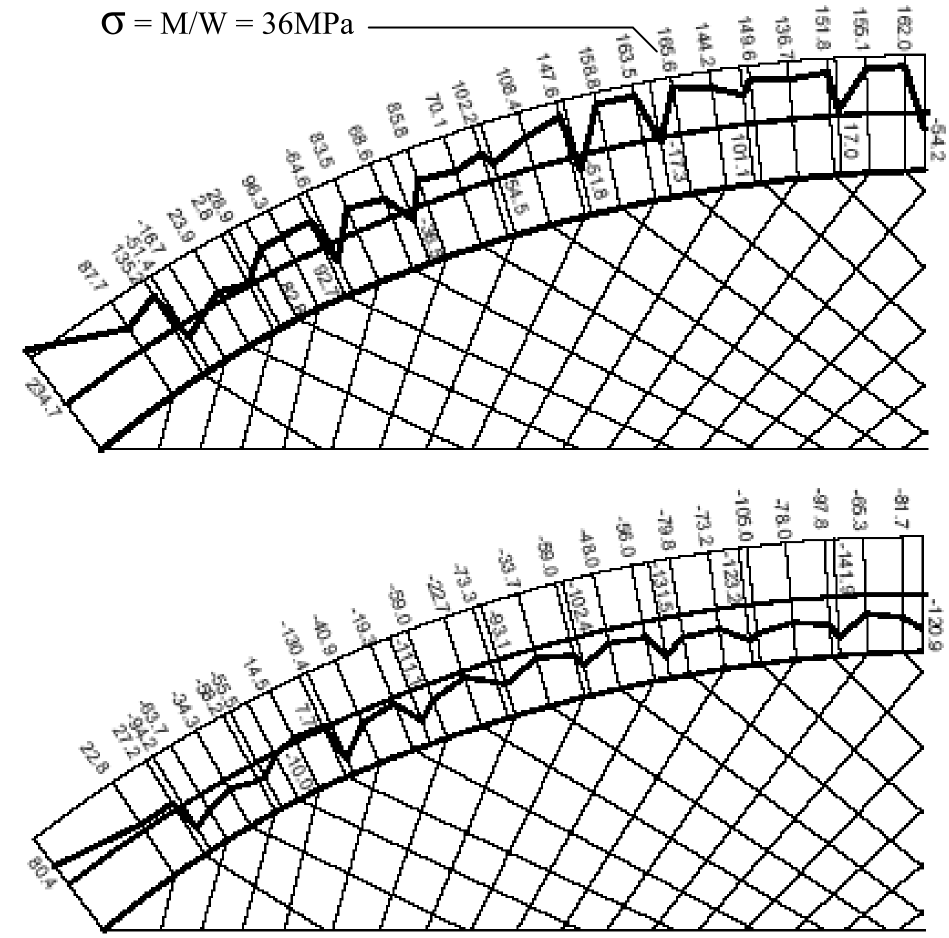


Fig. 54. View of the 3-D FEM model used for calculations by [Brunn and Schanack 2003].



	max N	min N	ΔN
1	174,8	128,1	46,7
2	912,4	420,7	491,7
3	666,8	390,1	276,7
4	829,2	383,8	445,4
5	779,9	396,3	383,6
6	774,1	389,8	384,3
7	826,9	405,9	421,0
8	790,1	423,3	366,8
9	813,1	406,9	406,2
10	857,8	458,0	399,8
11	814,9	402,6	412,3
12	903,4	453,6	449,8
13	835,3	380,8	454,5
14	943,6	446,5	497,1
15	903,5	370,5	533,0
16	949,4	432,8	516,6
17	944,3	347,7	596,6
18	917,0	381,8	535,2
19	945,2	330,7	614,5
20	871,1	353,4	517,7
21	931,9	336,4	595,5
22	871,9	346,9	525,0
23	924,2	349,1	575,1
24	889,8	343,2	546,6

Fig. 55. Maximum and minimum bending moments in the arch. *Fig. 56. Hanger forces*

Figs 55 and 56 give bending moments and hanger forces for one configuration of hangers in [Brunn and Schanack 2003]. It can be seen that the hanger forces are reasonably even and the bending moments give small stresses. Those who are very interested in this are recommended to look at Brunn and Schanack's Master's thesis.

Fig. 57 shows Brunn and Schanack's suggested reinforcement for a tie for the bridge in fig. 50. The author can not help wondering if the rail track could not be cast into the concrete. Then the ballast would not be necessary. This would increase the noise level, but there would be less noise than from an old fashioned railway bridge made of steel. To improve precision the top of the rail could be welded on to the top of the lower part of the rail after the concrete had hardened.

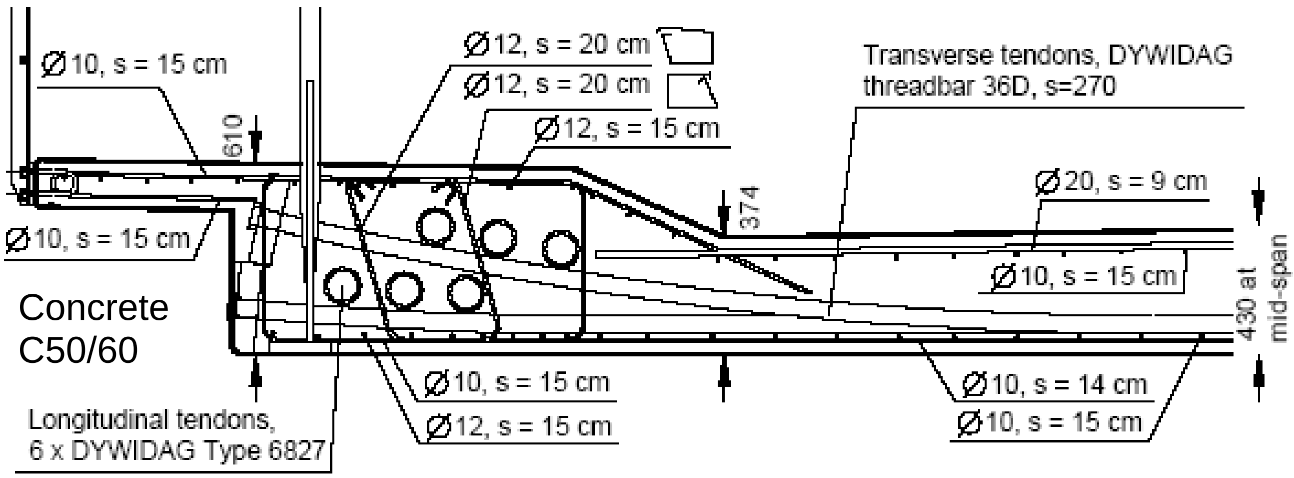


Fig. 57. Reinforcement of the tie of a railway bridge spanning 100 m.

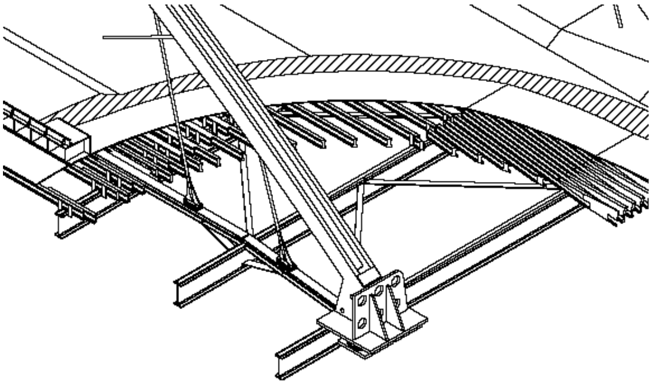


Fig. 58. Temporary lower chord at the end of the arch.

Fig. 58 shows how the temporary lower chord of the bridge in fig. 50 and 51 could be attached to the end of the arch. If the temporary lower chord is to be reused, it would normally have to be modified especially near the end of the span. If the width of the new bridge was different, the diagonals would have to be altered.

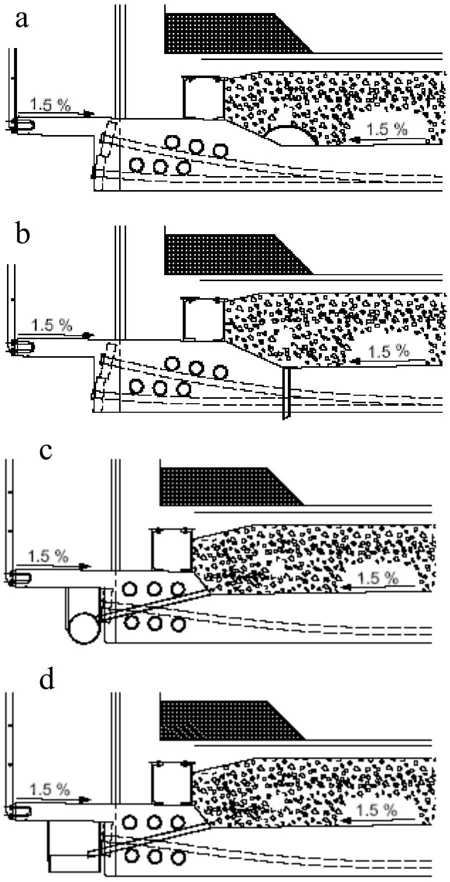


Fig. 59.Ways of leading the rain-water away from a railway bridge.

Fig. 59 shows different ways of leading the rain-water away from the bridge in figs 50 and 51. The rainwater becomes dirty from contact with the railway bridge.

The bridge is usually the place where it costs the most to get rid of the dirty rainwater. This speaks for the solution in fig. 59b. The money usually leads to more improvement of the environment if it is used on other parts of the railway.

Fig. 60 shows the cross-section of a combined road and railway bridge from the Master's thesis of [Räch 2003], [Tveit 2003]. The span is 160 m. It is meant to be for a bridge over a wide river where very big floating cranes are not available.

Fig. 60. Cross-section of combined road and rail bridge. Span 160 m.

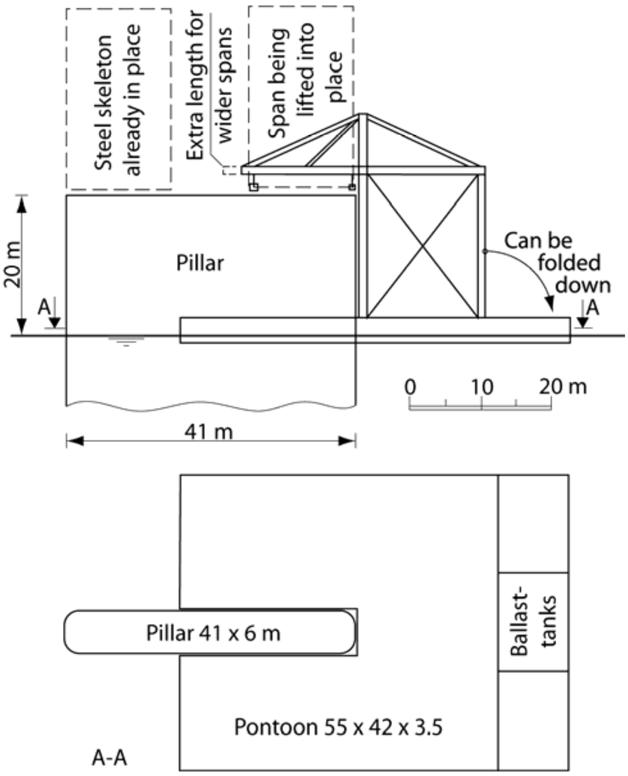


Fig. 61. Crane for lifting steel skeletons.

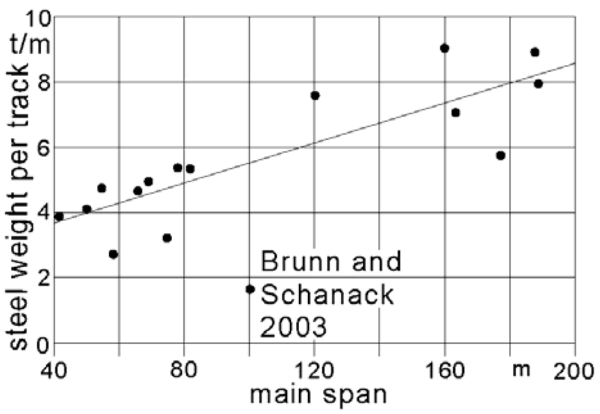


Fig. 62. Steel weight in railway bridges.

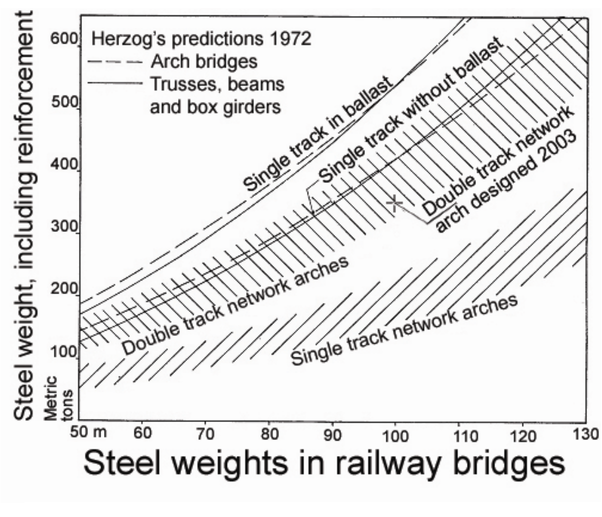


Fig. 63. Weight predictions from [Tveit 1973].

Fig. 61 shows a floating crane designed for the Ganges. [Tveit 2003]. It is lifting one steel skeleton for the bridge in fig. 60 in place. After two steel skeletons are in place, formwork can be built between and under the steel skeletons and the casting of the concrete tie can begin.

The pillars must be at least 6 m wide to make room for the prestressing. The crane can lift four lane road bridges because their steel skeletons tend to weigh less than the steel skeleton of the bridge in fig. 60.

The crane can be folded down so that it can pass under bridges already built, as long as there is 10 m room under the bridge. It needs only one metre of water under the crane.

For pillars over 20 m high the necessary size of the pontoon for the floating crane increases fast. Cranes on top of the pillars that can lift the steel skeletons from pontoon must be considered.

The steel weight of the bridge in figs 50 and 51 is 375 tonnes. In fig. 62 this steel weight is compared to the steel weight of railway bridges already built. [Brunn and Schanack 2003 p. 25] use about a third of the steel needed for the other bridges in fig. 62.

The steel in Brunn and Schanack's bridge is not likely to cost more per tonne. That is because 36 % of the steel is reinforcement and the details are simple. The fact that 55% of the steel is S 460 ML is not likely to increase the cost a lot.

The low steel weight of the railway bridge designed by Brunn and Schanack's came as no big surprise to the author. The predictions of steel needed for network arch railway bridges indicated by hatching in fig. 63 were first presented in [Tveit 1973].

Brunn and Schanack needed less steel than the author predicted. That is probably mainly because they used the steel S 460 ML instead of S 355 ML. Fig. 63 also shows Herzog's predictions of steel weights based on railway bridges built before 1972. [Herzog 1975].

The prediction that more than half the steel weight can be saved by using network arches may inspire somebody to build network arch railway bridges.

Fig. 64. A bridge in Bechyne, Czech Republic.

Fig. 65 shows the Bechyne bridge. Span 41 m.

The Bechyne network arch in the Czech Republic was built to replace an old bridge. *Pp. 92a - 92c.* See figs 64 and 65. At the same time the area under the bridge should be increased. Because the tie of the network arch is so thin, the road system on both sides of the bridge needed only minor alteration.

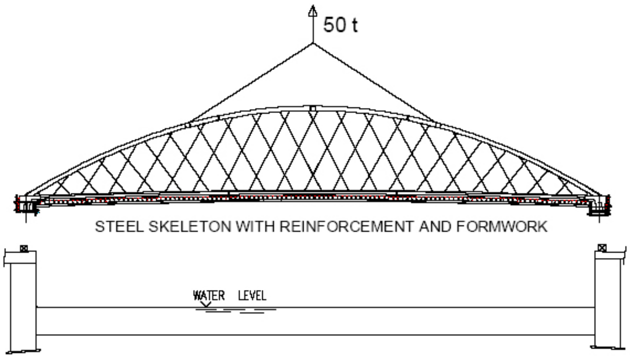


Fig. 66. An advanced method of erection.

The method of erection proposed in fig. 66 is very interesting. For the erection of long network arches in coastal areas one floating crane at each end of the spans is usually best.

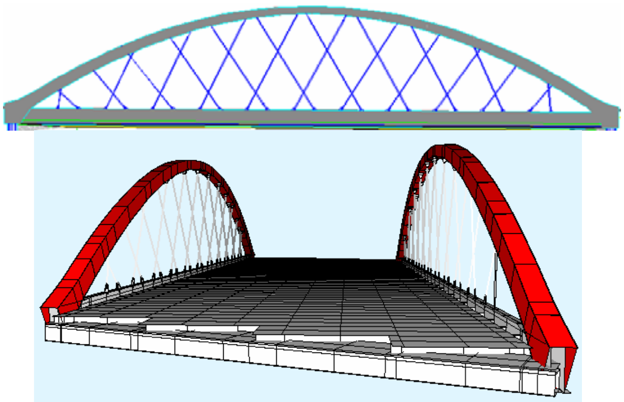


Fig. 67. Network arch for a motorway.

The bridge in fig. 67 was opened in Saxony in Germany this year. (2006) It was designed by Uwe Steimann. He works in the firm GMG-Ingenieurgesellschaft, Dresden. He did his Master's thesis with the author [Steimann 2002].

The span is 88 m. It is one of two spans for a motorway crossing another motorway. The tie has steel beams because the arches are 28 m apart. There is no bracing between the arches. One more bridge of this type is being built.

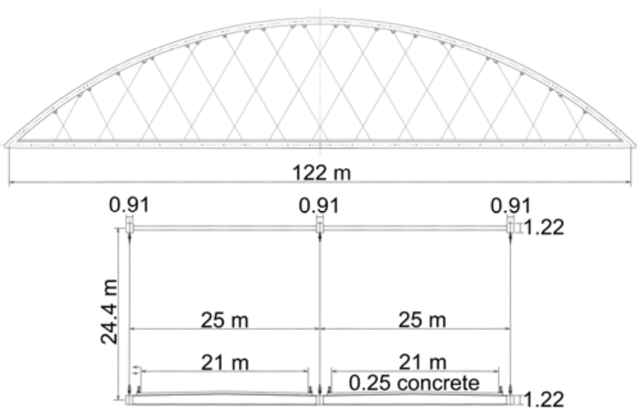


Fig. 68. Network arch with three arches.

The bridge in figs 68 and 69 will be finished next year in Providence, Rhode Island, USA. The transverse beams are continuous under the middle arch. Since the bridges in figs 67 and 68 are so wide, they have to have transverse steel beams in the tie.

The hangers are fastened to the ends of the steel beams and can not be placed equidistantly along the arch. All the hangers in the Rhode Island bridge have the same slope. Maybe that looks better. Steimann has varied the slope of the hangers. That saves materials.

Fig. 69. The steel skeleton of the network arch in fig. 68 before it is floated to the site.

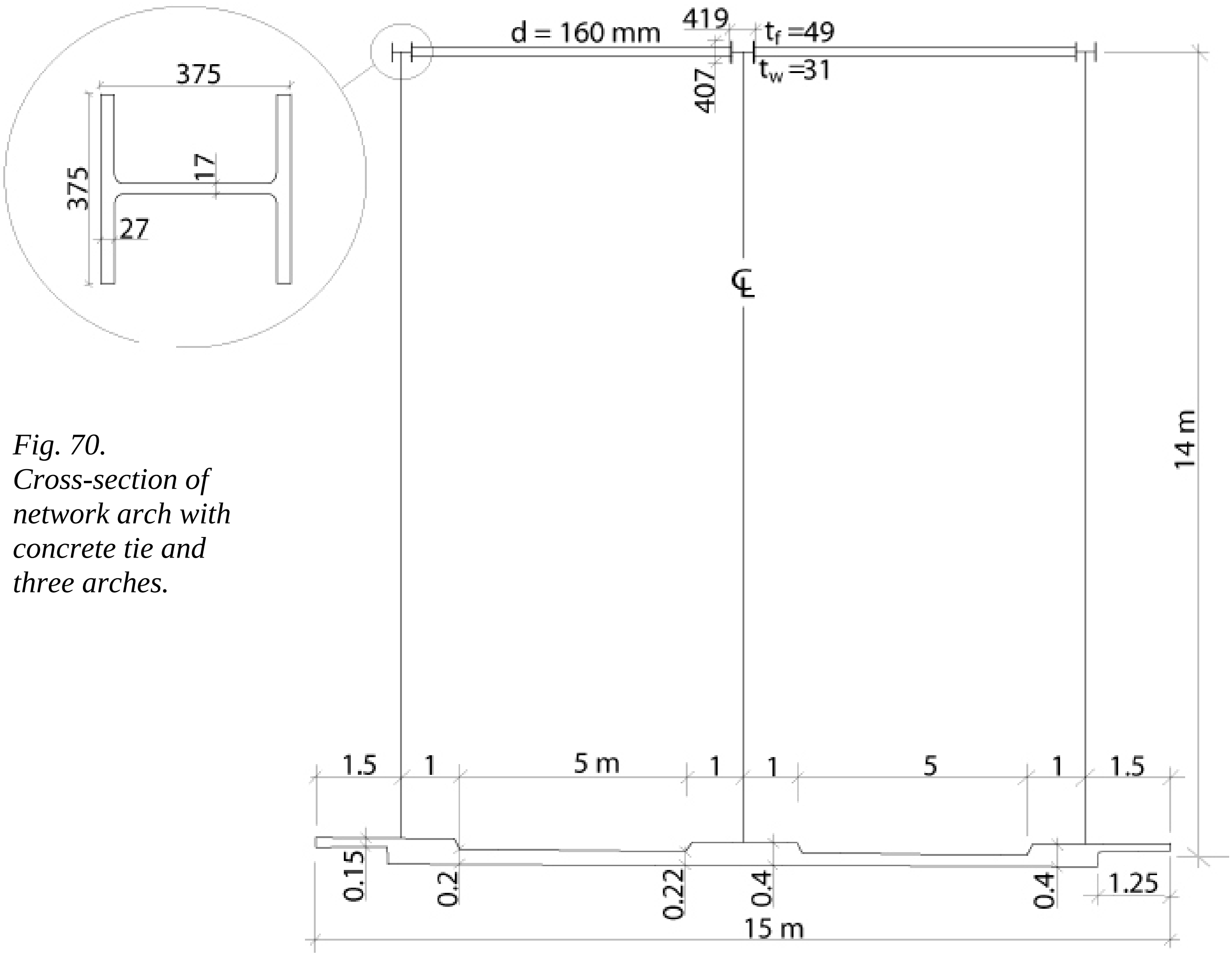


Fig. 70. Cross-section of network arch with concrete tie and three arches.

Fig. 70 gives an outline of a bridge with three arches. The bridge can be calculated in detail by somebody who would like to design the world's most slender arch bridge. It has a span of 88 m. The slenderness is 88/(.4+.407)=109. Till now the maximum slenderness of an arch bridge has been 91. The design of this bridge would be a suitable task for two very able students doing their Master's thesis. I would be most willing to assist their professor by email.

Maybe you would like to make the lanes wider and alter the span. The steel quality assumed is S 460 M. The yield stress of the middle arch is 430 MPa. Maybe thicker universal columns are needed, but in any case little steel is required. Much depends on the codes and the strength of materials used. It would be easier to introduce the network arch in a country where this bridge had already been designed.

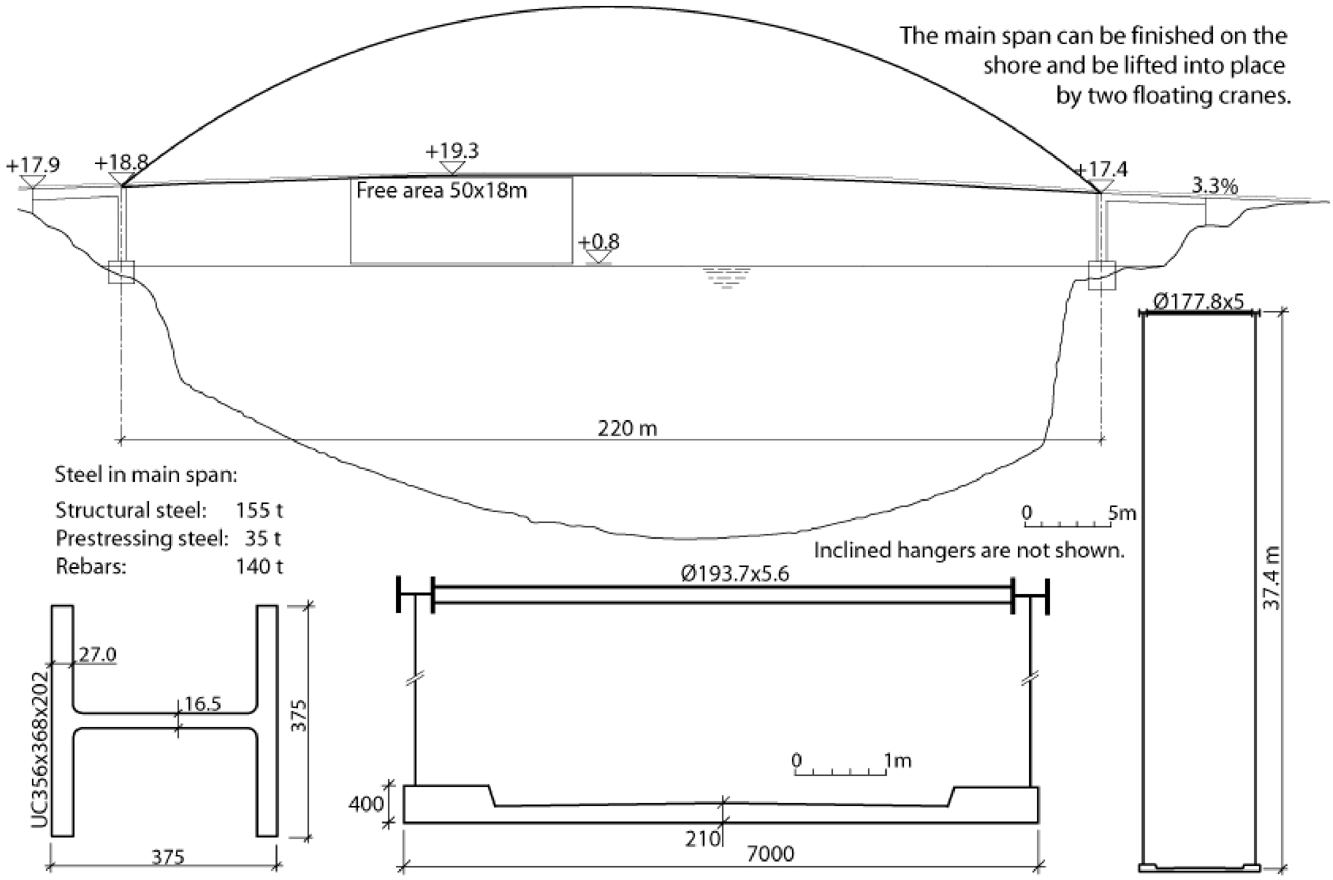


Fig. 71. The Brandanger Bridge in western Norway.

Let's look to the future. In Peru, Germany and Norway more network arches will be built. The Norwegian firm Aas Jacobsen is working on the final design of the bridge in fig. 71. The bridge will never have much traffic so only one lane of traffic is needed. The assumed load in this lane is shown on *p. 9.* and in [Tveit 1999a].

The bridge was the author's idea. [Tveit 2004] It has a main span of 220 m that can be finished on shore and be lifted in place by two big floating cranes. The simplified erection process should be able to pay for the cranes. This type of bridge will be very competitive where long spans are suitable and big cranes are available. In this slender span the bending in the middle of the tie due to wind is a problem. It might be overcome by a combination of higher concrete strength, deeper edge beams, a more aerodynamic lower chord or introducing continuity between the lower chord and the sidespans.

- Network arches are equally well suited for road and rail bridges. They use very little steel.
- An optimal network arch is likely to remain the world's most slender arch bridge. The slim chords are pleasing to the eye and do not hide the landscape or cityscape behind them.
- If the bridge is not too wide, the tie should be a concrete slab. Concrete ties with small edge beams can be used for up to 15 m between the arches.
- Efficient methods of erection are available.
- Since the network arch needs little materials, a high percentage of the cost will be employment.
- The poverty in some parts of the rest of the world is one more reason for using the network arch at suitable sites.
- Depending on the site the network arch can save up to 40% of the cost and 70 % of the steel.
- If the network arch had been a well known type of bridge, it would have been hard to argue convincingly for arch bridges with vertical hangers and many other bridge types.
- Conservatism is the main obstacle to the building of network arches.

You might wonder whether this very slim bridge will have unpleasant vibrations during strong winds. It will not, because it has the same length and width as a Norwegian suspension bridge from around 1940. Furthermore it is much stiffer and has an aerodynamically better tie.

The points to the left sum up the contents of this lecture. The author would be pleased to receive comments and questions regarding network arches.

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