The Network Arch.

Bits of Manuscript after Lectures in 44 Countries

Internet edition in September 2007

Summary

This publication contains advice on how to design network arches. In the future, revised editions of this publication can be found at http://pchome.grm.hia.no/~ptveit/

Network arches are arch bridges with inclined hangers where some hangers cross other hangers at least twice. When the arches are less than 18 m apart, the tie should be a concrete slab with partial longitudinal prestress. The arches should be universal columns or American wide flanges. Network arches are best suited for spans between 80 m and 170 m. This leads to attractive bridges that do not hide the landscape behind them. A network arch bridge is likely to remain the world's most slender arch bridge.

The transverse bending in the slab is usually much greater than the longitudinal bending. Thus the main purpose of the edge beam is to accommodate the hanger forces and the longitudinal prestressing cables. 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.

For load cases that relax none or only very few 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 extensive relaxation of hangers, the hangers should not be inclined too steeply. Small inclination of hangers will increase the bending moments due to concentrated loads. A compromise should be sought. All hangers might have the same cross-section and nearly the same decisive load. Their upper nodes should be placed equidistantly along the arch.

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.

Because there is little slenderness between the nodal points of the arch, and tension is predominant in the rest of the network arch, this type of bridge makes good use of high strength steel. All members in an optimal network arch efficiently carry forces that can not be avoided in any simply supported beam. 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.

Compared with conventional bridges, the network arch, where the tie is a concrete slab, usually saves over ⅔ of the steel weight. See fig. 98. The details are simple and highly repetitive. Thus the cost per tonne is not very high. The slender tie leads to short ramps. This makes it simpler to attach roads at the ends of the bridge.

The network arch on page 93 is one of the most competitive network arches ever designed. The details are simple and the exposed surface is small. The steel weight is low, but not minimal. The arch and hangers supplemented by a light temporary lower chord can be moved when lifted at both ends. This steel skeleton can be erected on side-spans or on ice between the abutments. It can also be lifted in place by floating cranes. When the span is in place, this steel skeleton has enough strength and stiffness to support the concrete tie while it is being cast.

For wide bridges, three or four parallel arches could be used to keep down the span of the concrete slab between the arches, see figs 30 to 32. For long bridges, where many spans are needed, the network arches could be made exclusively from prestressed high strength concrete. The spans should be cast on shore before being floated to the site on pontoons or by big floating cranes. See pp. 24k-30a, 47–50a, 94-94a.

The local conditions will influence the type of erection. Sometimes the tie can be cast on a timber structure in the river bed. After the arch and hangers have been erected the hangers can be tensioned till they carry the 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, more employment and more practical training of the workforce from the same limited funds.

The building of optimal network arches can bring great savings. However steel firms are usually not interested in using very little steel. Considering the great poverty in the world, it would be morally wrong not to use network arches at suitable sites. The introduction of the network arch would create extra work for bridge authorities, but it is up to them to promote it. General conservatism is probably the main obstacle to the use of this very promising structure. To some civil engineers the author's claims may seem exaggerated, but it would be stupid to exaggerate when the bare facts seem like an exaggeration.

Key words: Network arches, arch bridges, road bridges, railway bridges, steel weight, light bridges, erection, economy.

THE NETWORK ARCH

Manuscript after Lectures in 44 Countries

CONTENTS Page

Preamble TO THE INTERNET EDITION SEPTEMBER 2007.

This edition is a picture of the state of the manuscript before my fourth journey around the globe. There is no index on the last page and some number of the pages and the drawings might be misleading. The text will be updated again at the beginning of 2008. Some information from Teich and Schanak's doctoral thesis will then be included.

In 1955 the author considered patenting the network arch. He then visited the German patent office in Munich. There he found an expired patent of a cable-stayed bridge. He understood that the cable-stayed bridge was stiffer and carried the forces more directly than the suspension bridge. On second thoughts he reckoned that conservatism and problems of erection would stop the bridge from being built. History has proved him wrong.

Many engineers who have read about the network arch bridge might have reached a similar conclusion. This is easy to understand because the network arch is much more complicated than the cable-stayed bridge. On the other hand, recent advances in computer technology have made it easier to calculate the network arch. The two Norwegian network arches were built because they were cheaper than competing alternatives. If the arches are universal columns, the network arches will be even more economical. Nevertheless conservatism and lack of experience with this structure will work against the network arch.

In February and March of 2000 the author made a lecture tour of 12 European countries to talk about network arches. It was a great pleasure to give 21 lectures, at which the average number of listeners was over 35. A list containing most of the organisers of the lectures is found on page 99 of this manuscript.

The author would like to express his sincere thanks to Professor Dr. Günter Ramberger of TU-Wien for recommending the lecture to all colleagues. This was done on the strength of Tveit 92. A high point of the lecture tour was a dinner with Professor Dr. Drs. h. c. Jörg Schlaich after the lecture in Stuttgart. Later Professor Schlaich wrote to the author that he was on the lookout for an opportunity to build a network arch. The author hopes that he will succeed.

This publication is published on the Internet, because this saves time and makes updating easy. More literature on network arches can also be found on the author's homepages. http://pchome.grm.hia.no/~ptveit/ and http://fag.grm.hia.no/fagstoff/ptveit/ This includes for instance graduation theses completed in Grimstad by six students from TU-Dresden, Germany.

The author looks forward to continually updating his homepage during the rest of his professional life. In future editions of this publication there will be more on: The deflection and thickness of concrete slabs and lifting of finished spans by means of big floating cranes. Readers' suggestions are very welcome. Very few have come since this publication was first made available on the Internet in 2000.

The author aims to inspire readers to construct their own network arches. The author's home page will help designers to achieve the maximum advantages of the structure. The author recommends the optimal network arch with H-beams in the arches and a concrete slab tie. He aims at giving the readers a chance to become familiar with his ideas before they start to design something preferably better or possibly worse.

The author's 34 publications found in the list of references contain many ideas that can be found in this publication. On some items the author has changed his mind, and he is glad to set the record straight in this edition. - The author wishes to help everybody that considers building a network arch. His pension is so high that his services would be extremely moderately priced.

The author would like to thank the colleagues mentioned on page 94b for suggestions and corrections to this edition. The author would strongly encourage more suggestions, comments and corrections. He is a dyslectic who appreciates help. The author apologises for repeating himself over and over again. His poor excuse is: "Repetitio est mater studiorum". A translation into English might be: "Repetition is the mother of studies".

Recently network arches of the type preferred by the author have been built or are being built in the Czech Republic, Germany and the USA. It has also been decided to build a network arch in Peru. There is design work in progress on network arches Germany and Norway. The author is delighted about this. There might also be other designs of network arches that the author does not know about.

Grimstad, Norway, September 5th 2007 per.tveit@hia.no

Per Tveit

AN EXPLANATION OF THE EFFICIENCY OF NETWORK ARCHES

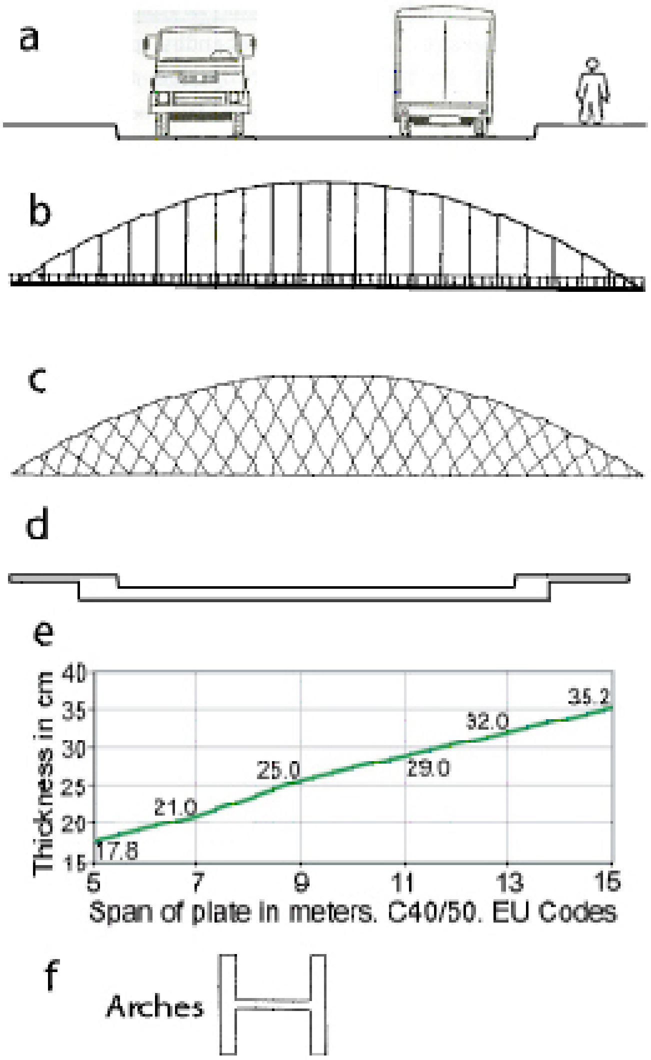


Fig 1. Stages in the design of network arches

The purpose of a bridge is to take traffic over an obstacle. The traffic can be on a road as in fig. 1a. Often there is no room for members under the bridge. For an evenly distributed load a parabolic arch with vertical hangers like in fig. 1b is very good. The forces in all members are axial forces.

For uneven loads it is best to use crossing hangers like in the network arch in fig. 1c. Then all loads are led to the arches in such a way that there is very little bending in the chords. See fig. 85 to 87. In trusses secondary stresses and load between the nodes give bigger stresses than in a well designed network arch.

The simplest tie would be a concrete slab spanning between the arches. See fig. 1d. The slab transfers the single loads to the edge beam. The edge beams distribute any load between many hangers.

The necessary thickness of a slab is shown in fig. 1e. For distances between the arches over 10 m transversal prestress should be considered. The bending in the middle of the slab is normally bigger than the longitudinal bending in the tie. See p. 13. Thus there is no need for steel beams in the tie.

The tensile force in the tie is best taken by prestressing cables in the edge beam. 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 maintenance of the tie.

The hangers give the arch good support in the plane of the arch. Universal columns like in fig. 1f can be used. They give very slender and good-looking bridges with low upkeep. The universal columns in the arches should be turned like in fig. 1f. Very often the buckling strength can be higher in the plane of the arch than and out of the plane of the arch. High strength steel can be well utilised. Fig. 98 shows that much steel can be saved by using optimal network arches instead of arch bridges with vertical hangers. The network arch is equally well suited for rail and road bridges.

At suitable sites the arch bridge indicated on this page gives a very light and economical bridge if economical methods of erection can be found. The network arch can be built on a scaffold. See fig. 7a. Sometimes a temporary lower chord can be used. Combined with arches and hangers it can make a steel skeleton that can be moved if lifted near the ends. See p. 29k-30a, 50-55 and 94-94a. The steel skeleton can support the casting of the concrete tie. In coastal areas big floating cranes can be used to lift steel skeletons or finished spans from the shore to the pillars.

On references: A reference is given by author and year. For instance: Nielsen 1930 indicates that Nielsen published something on the subject in 1930. The author finds that this method gives valuable information without forcing the reader to look at the end of the publication. The usual (1) is much less effective and harder on the author.

When referring to information on the internet one should often give page numbers or numbers of figures. That makes it easier to check statements. It is much more difficult to get hold of books and publications. Thus it is not so important to give numbers of pages and figures.

On tons: In this publication tons mean metric tons, which are 1000 kilos.

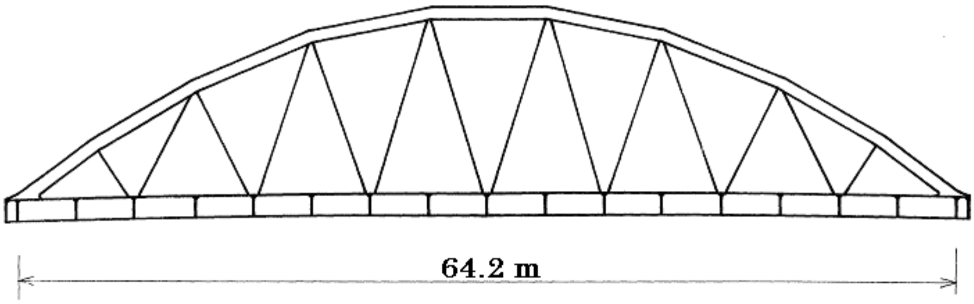


Fig. 2. Tied arch over Mänam Pasak, Aynthia, Thailand

To explain the advantages of network arches, the author starts with an arch bridge built in Thailand in 1942. Krück 1946. See fig. 2. In this concrete bridge the hangers were steel rods that could not take compression. For the loads and materials used between the two world wars this was an efficient structure. As long as the steel rods have tension, the bridge acts like a truss with little bending in the chords.

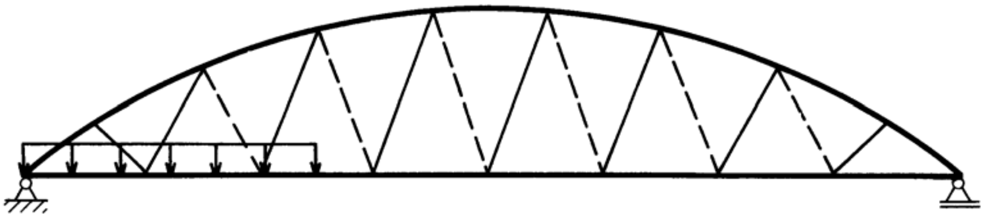


Fig. 2a. One-sided live load on the span might make the dotted hangers relax

With today's loads and materials many hangers may relax due to load on one side of the span as indicated in fig. 2a. The hangers' tendency to relax might be counteracted by increasing the distance between the nodal points. See fig. 3. This, however, would lead to increased bending in the lower chord and less buckling strength of the arch.

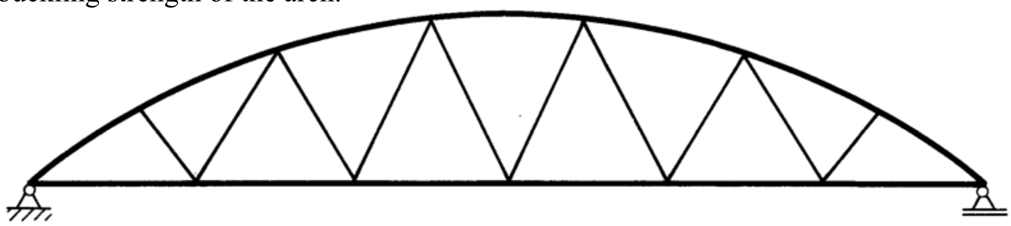


Fig. 3. Increased distance between the nodal points gives decreased tendency for the hangers to relax

Between the nodes the arch has a tendency to move upward and the lower chord has a tendency to move downward. Thus it would be an advantage to put in an extra set of hangers between the chords as shown in fig. 4a and fig. 10.

Two sets of hangers give smaller buckling lengths in the arch and less bending in the chords. This effect is even greater if we put in three sets of hangers. See fig. 4 b. This structure has three trusses on top of one another. The author's definition of a network arch bridge is an arch bridge where some hangers cross other hangers at least twice

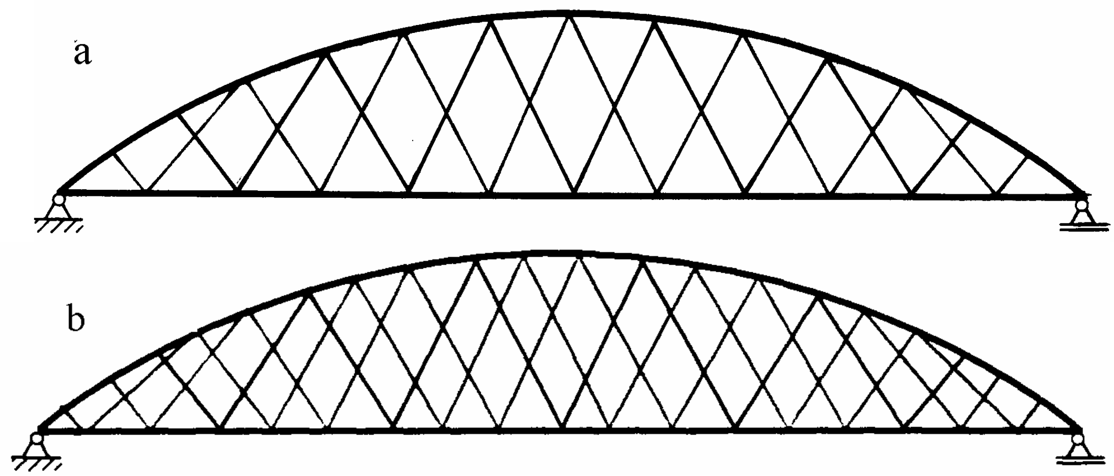


Fig. 4. Tied arches with two and three sets of hangers respectively

The network arch can be seen as a simply supported beam with a tensile and a compressive flange. The hangers are the web. Most of the shear force is carried to the supports by the vertical component of the force in the arch. Much of the variation in the shear force is taken by variation in the hanger forces. The hangers distribute the load between the chords in such a way that the chords have little bending.

The axial forces in the tensile and the compressive flanges are inversely proportional to the distance between them. In tied arches, aesthetic reasons limit the distance between the arch and the tie. In most Japanese network arches the rise of the arch is between 14 % and 17 % of the span. Nakai 1995. Saving of materials depends mainly on whether or not a design gives light chords and a light web.

The arch could be part of a circle. This gives evenly distributed bending moments in the chords. A reduced radius of curvature near the ends of the arch can give less bending in the wind portal and a constant axial force in a longer portion of the arch.

The upper nodes of the hangers should be placed equidistantly along the arch. The members between the last node and the end of the bridge can be a little longer than the other members. If universal columns are used in the arches, the lower half of the last member in the wind portal should have a steel plate on top of the arch. The cavity under the steel plate could be filled with concrete to make it more collision resistant.

All hangers should have the same cross-section. With careful choice of the slope of the hangers, the force in the hangers can become surprisingly even. See chapter on optimal arrangement of hangers p. 26. The tie has a concrete slab spanning between the planes of the arches. The footpaths are outside the arches. The biggest bending moment in the tie is found in the middle of the slab between the arches. The longitudinal bending is usually smaller. If high-strength concrete is used, the distance between the arches can be more than 16 m. For distances more than 12 m transverse prestressing should be considered.

The small edge beams in the planes of the arches must be able to carry the hanger forces. They must also have room for the prestressing cables that take the axial forces between the ends of the arches. These prestressing cables give the tie a moderate longitudinal prestress. The longitudinal prestress has probably contributed to preventing problems at the lower end of hangers that go directly into the concrete.

From the hangers the arch gets good support in the plane of the arch. Thus it has little tendency for buckling in the plane of the arch. All the members in an optimal network arch efficiently carry forces that can not be avoided in any simply supported beam. Therefore it is a most efficient structure. The network arch uses any kind of steel efficiently, but it will be even more competitive in combination with future steels with high yield strengths. There is valuable information on high strength steels in IABSE (2005).

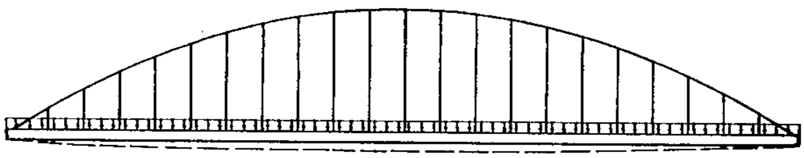


Fig. 5. Arch bridge with vertical hangers and even load

An arch bridge with vertical hangers like in fig. 5 works well for evenly distributed loads. There are only axial forces in every part of the structure.

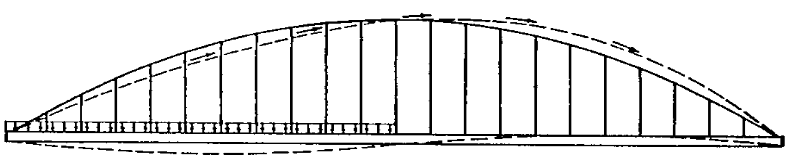


Fig. 5a. Load on half the bridge in fig. 5

For live loads only on the half of the span the axial force in the arch is reduced, but the arch moves to the right, and the deflections and the bending moments increase.

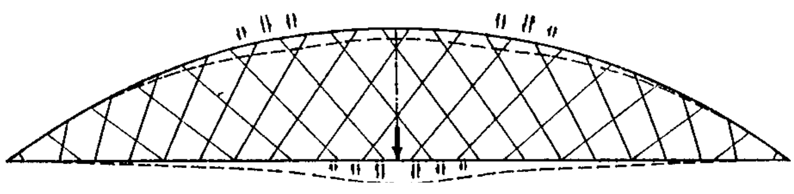


Fig 5b. Network arch

The type of bridge in fig. 5 is less efficient for increased modern live loads and modern materials. That is because the stronger materials lead to reduced cross-sections of the chords and thus to reduced bending capacity in the chords.

Fig. 5b shows a network arch. Sloping hangers reduce the bending moments in the chords. It can be seen as a simply supported beam consisting of many trusses on top of each other. A concentrated load gives some bending, but it does not have far to go to distribute itself to all the trusses. The distribution is indicated by double arrows with opposite directions. Thus the bending moments are very small.

One can normally get a good estimate of the axial force in the chords with the help of the influence lines in figs 63, 64, 65, 81 and 82. Tveit 66 p. 251 has formulas for calculating the axial forces in the chords. Normally it is enough to calculate the axial forces in the middle of the span before more precise values are calculated by means of a computer program. The force in the arch varies little before we get down to the wind portal. The bending moment is biggest and the prestressing force is smallest in the middle of the span. The formulas are simple.

The formula for the compressive force in the arch in the middle of the span is:

$N= ql^2/8f+fq/2\ \tan^2 v_h$

The formula for the tensile force in the tie in the middle of the span is: $N=ql^2/8f-fq/2\ \tan^2 v_h$

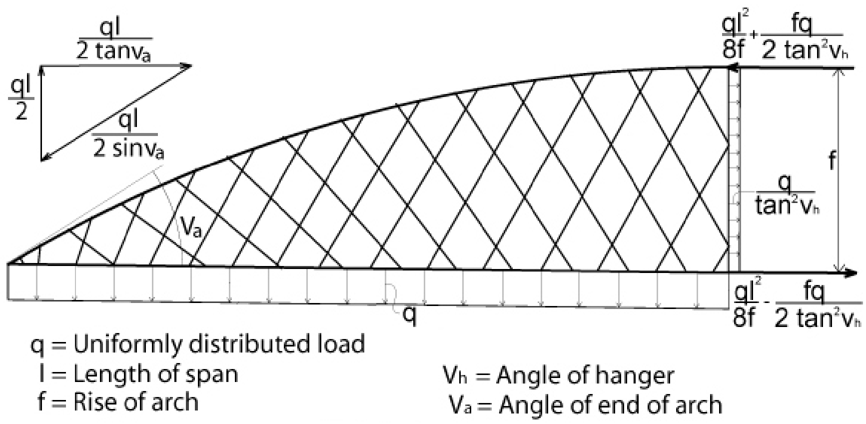


Fig. 5c. Axial forces in the middle and at the end of the chords

Fig. 5c gives the axial forces due to evenly distributed loads in the middle and at the ends of the chords. The formulas for the axial force in the chords can be found in Tveit 2000 p. 5 and in Tveit 1966 p. 251.

The influence lines for axial forces in the chords are ~triangular. See fig. 63–65, 77, 81 and 82.

For calculation the axial force at mid span due to a concentrated load, the load can be replaced by a uniformly distributed load equal to the concentrated load divided by half the span. This is because the maximum ordinate of the influence line is roughly twice as big as the average ordinate. This makes it easy to find the axial force due to concentrated loads. Preliminary hanger forces can be found by looking at influence lines on pp. 57, 58, 60 and 72.

Now a few words on how the hangers should be placed. It is usually best to place the nodes equidistantly along the arch. See fig. 8. The nodes in the lower chord should be placed to obtain nearly the same maximum force in all hangers and to give all hangers a suitable resistance against possible relaxation. This has been achieved in the bridges in fig. 8. There will be more on this for instance on pp. 26-29j.

When some hangers relax, the bending moment in the chords increases and the buckling strength of the arch is reduced because the support of the arch is less stiff. This is normally compensated for by the inevitable reduction of the normal force in the arch. There is more on this on pages 67 and 68. In the ultimate limit state it is all right to let a few hangers relax due to load on part of the span.

More relaxation of hangers is allowed if the arch is a box section like in the bridges for Vienna, pp. 59 to 72. Fewer hangers should be allowed to relax when the arches are universal columns like in Teich and Wendelin 2001. See fig. 97. This is because the arch in the bridges for Vienna has considerable capacity for taking the increased bending due to the relaxation of hangers. In long bridges with universal columns in the arch and slender ties it might not be advisable to let any hangers relax in the ultimate limit state. Here an increased skew load might quickly give too much bending in the arches. More on this on pages 27, 29i, 67 and 68.

In the serviceability limit state it might be best if all hangers are in tension. This is a must if the arches are universal columns or American wide flange beams. In any case that would simplify the calculation of the hangers. The arch should have a continuous curvature. This simplifies production. If the radius of the curvature is smaller in the wind portal, the wind portal becomes shorter and the axial force in the arch have less variation. Brunn and Schanack 2003 chapter 6.

NORWEGIAN NETWORK ARCHES ETC.

This chapter has many subheadings that are not mentioned in the index.

THE NETWORK ARCH AT STEINKJER

After eight years of study the author was given the opportunity to build two network arches. These network arches are still in good shape. Steinkjer Municipality 85 kilometres northeast of Trondheim, Norway, built the first one. It was built because it was more economical than an arch bridge with vertical hangers. It was finished in the spring of 1963. See pp. 5b to 6c.

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". Later on Terje Moe became a professor. At that time the computers could hardly calculate the effect of hangers relaxing. To simplify calculations, the slopes of the hangers prevent the relaxation of any hanger for the loads and codes used at the time.

Details of arch and windbracing are shown on page 56. The joints in the arch are simple flanges because there is hardly any bending in the chords. Still the details are costly to produce compared to the details of the network arch in fig. 97.

For data and influence lines for the Steinkjer Bridge see pp. 56-57. The span of the Steinkjer Bridge is 0.25 m shorter than the influence lines. The shortening takes place only at the end nodes. This shortening gives less bending at the lower end of the arches. It also gives better utilization of the shortest hanger and smaller bending moments at the end of the attached span at the north bank.

The prestressing cables that take the axial force between the arches are placed centrally to reduce the stress variation that can cause fatigue. A drawing of the railing outside the footpath is shown in fig. 34 on page 24.

Materials pr m^2 of the main span. Concrete with the cube strength of 35 MPa 0.22 m^3. Structural steel with a yield stress of 355 MPa 60 kg. Reinforcement 40 kg. Pre-stressing steel 7 kg.

Fig. 6. Bridge at Steinkjer, Norway. Tveit 64 and 66.

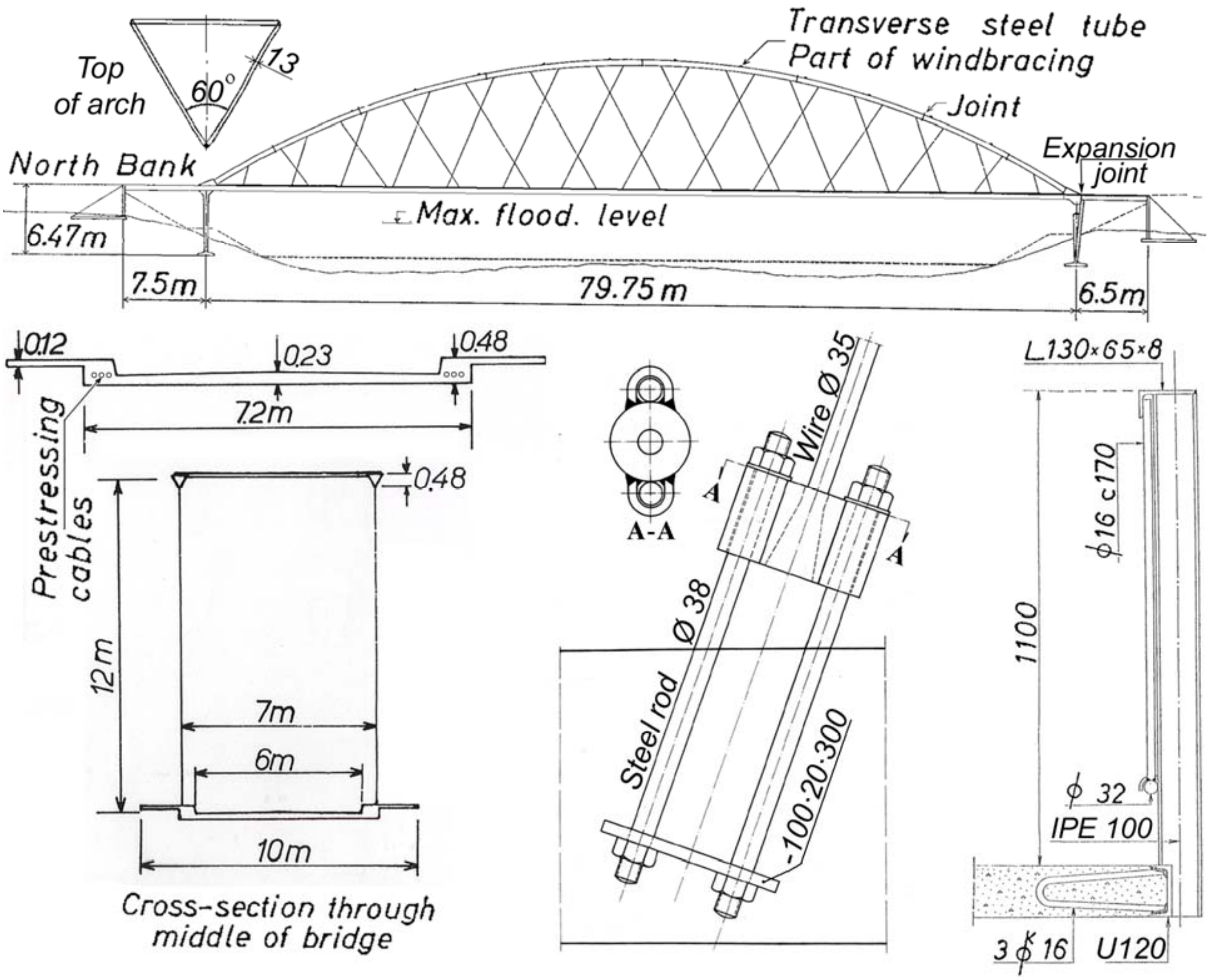


Fig. 6a. Network arch at Steinkjer opened 1964.

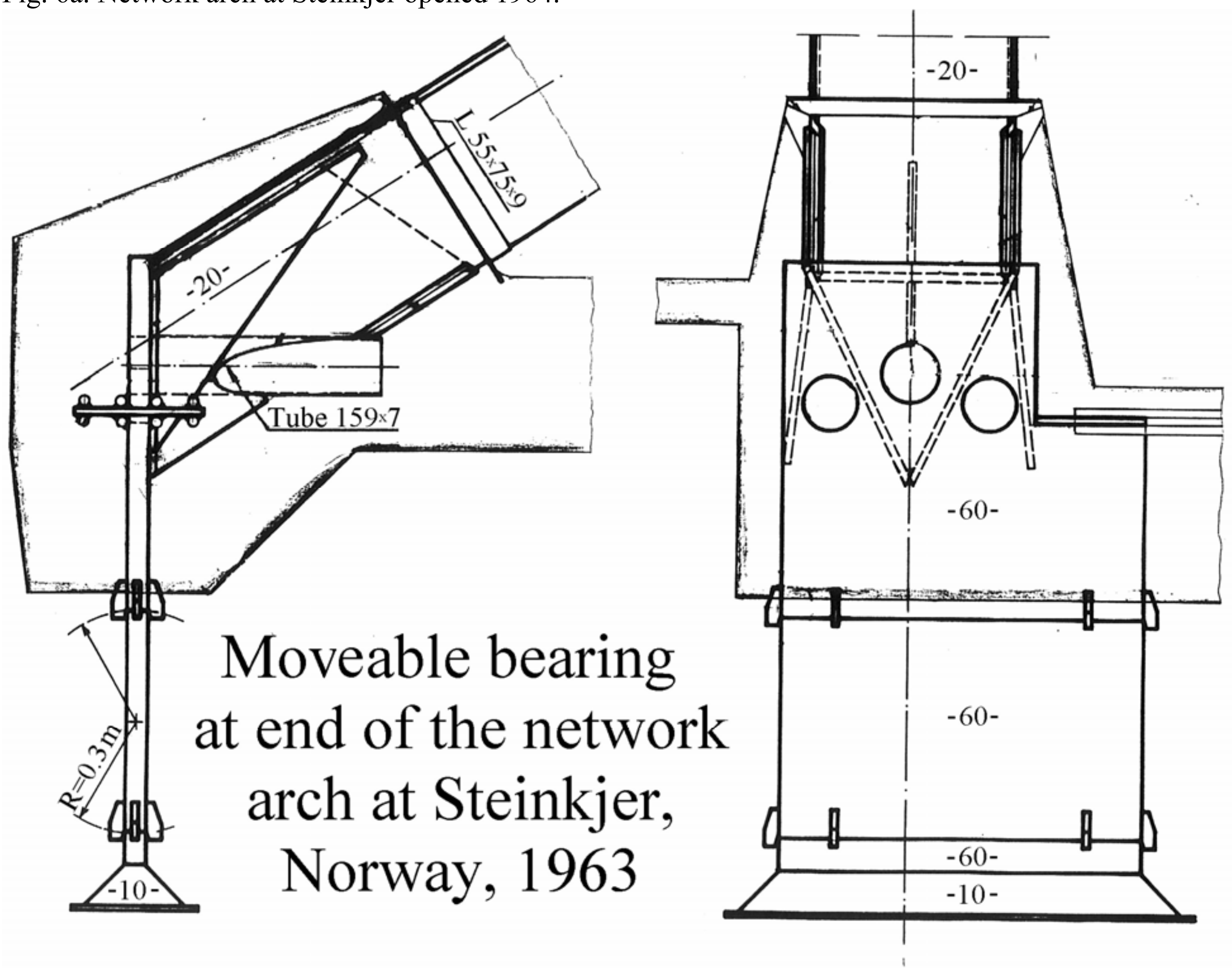
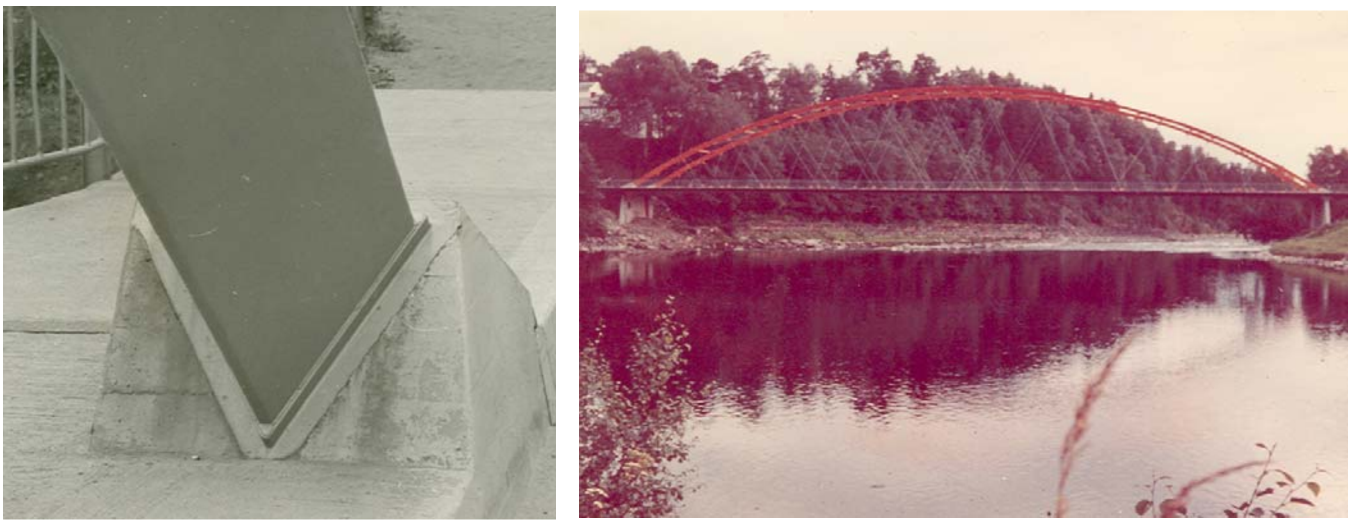


Fig. 6b. Moveable bearing of the network arch in fig. 6a

Fig. 6c. Photos of the network arch at Steinkjer



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.

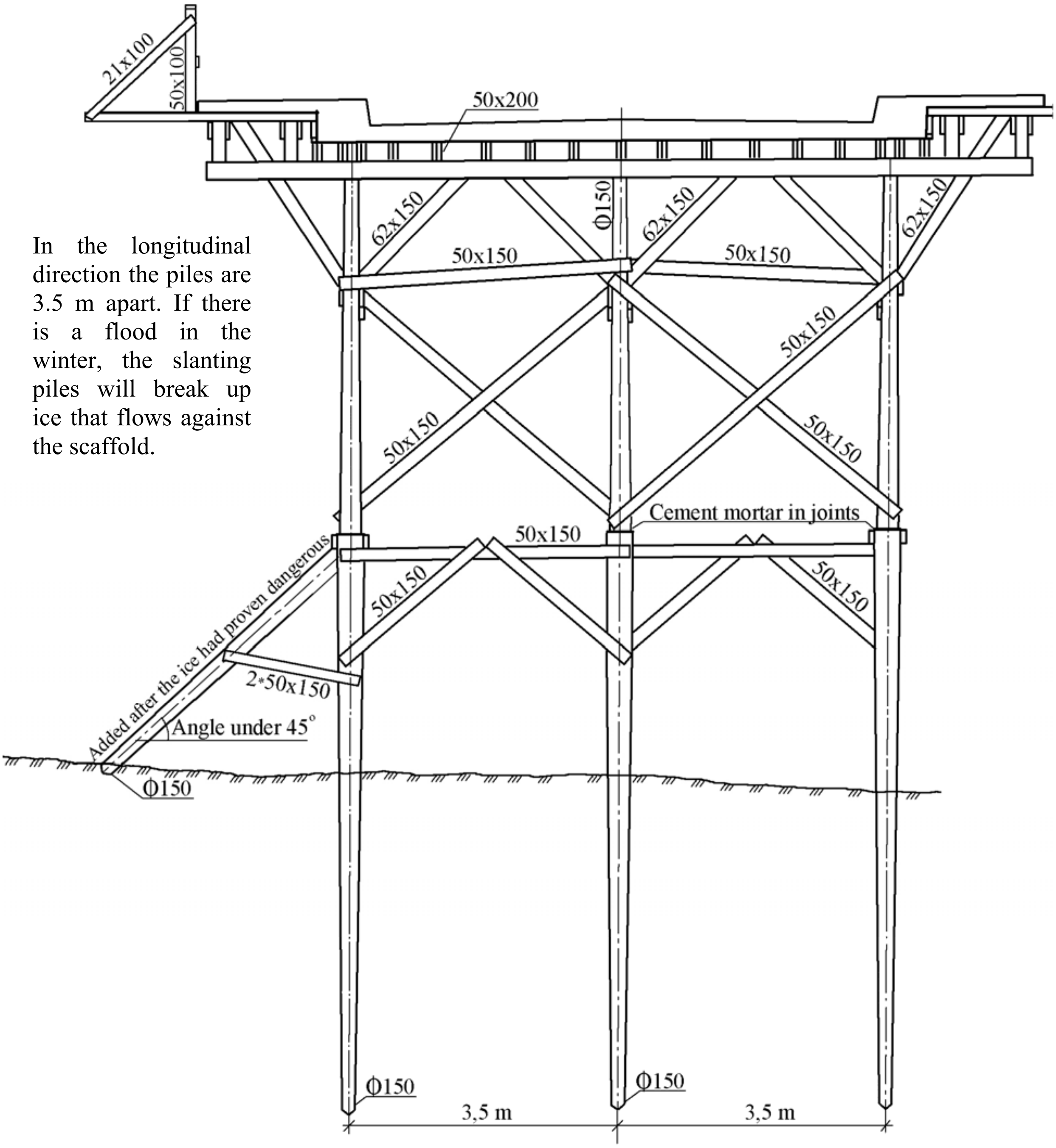


Fig. 6d shows the scaffold for the tie of the network arch at Steinkjer

Experience from the first 44 years of the Steinkjer network arch.

The network arch at Steinkjer is in good shape. Given good maintenance it is likely to last at least another 40 years. More on this can be found in the author's contribution to the report from ARCH'07 in Madeira.

The tie was cast on a scaffold on piles in the river bed. See fig. 6d. The cube strength of the concrete was well over the prescribed 35 MPa. The content of the cement was 450 kg/m^3. 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. A flood came after the ice was around 0.15 m thick and swept away over 17 m of the support under the tie. Due to this the tie was sagging 0.2 m and developed cracks over 2 mm wide.

Damage due to the flood with winter ice has been overcome. The scaffolding was repaired and strengthened in a makeshift way. Twelve piles in the river could not be replaced. When the steel had been erected, the tie was straightened by the tensioning of the hangers. Prestress closed the cracks. Now they are hard to find.

Damage due to lack of railings to protect the hangers

There are no railings between the traffic and the hangers. See fig. 5. That was a mistake. As a result four or five lower ends of hangers have been bent by vehicles bumping into them. The stresses are small where the steel rods enter the concrete. The concrete around the hangers has not been damaged. There is no point in bending the steel rods back because that might damage the concrete. Generally the concrete around the lower ends of the hangers seems to be in good shape. This might partly be due to the longitudinal prestress of the tie. For possible troubles with hangers coming out of the concrete see p. 55a.

Railing outside the footpath

The posts, IPE 100, of the railings outside the footpath, see fig. 6, 6a and 34, 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.

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. Tveit (2007) fig. 34. Since the IPE10 posts at the ends of the main span are still 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. Maybe this is due to delayed hardening of the cement.

The state of the concrete

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 contracted 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.

Damage to the steel

A vehicle has bumped into a tube above the lane at one end of the bridge. See fig. 6. There was extra bending capacity in the tube to take the resulting bending. The diagonals in the windbracing are tension rods, but the diagonals at each end of the span are the same as the hangers. See fig. 62. The steel structure is in good shape after 44 years. Good maintenance of the paintwork has contributed greatly to this.

Fig. 7. Bolstadstraumen Bridge. The main span is 84 metres.

The second network arch was built over Bolstadstraumen 50 km northeast of Bergen. In the two Norwegian network arches the arches had triangular cross-sections that were costly to produce. See fig. 62. Still both bridges were built because they cost less than competing designs. Network arches with arches made of Universal Columns or American Wide Flange Beams will be more competitive. See figs 16, 17, 19, 20 and 97. Tveit 64 and 66.

The Bolstadstraumen Bridge was opened at the end of 1963. Influence lines are found on page 58. In the network arches at Steinkjer and Bolstadstraumen no hangers relax for ordinary load cases. This precaution was used partly because there was no computer capacity for calculating the effect of hangers relaxing.

The Bolstadstraumen network arch used 44 tonnes of structural steel and 7 tonnes of prestressed steel. The rise of the arch was 18% of the span. A competing costlier design with vertical hangers would have needed 125 tonnes of structural steel. The rise of the arch was 21.5 % of the span. Both bridges had a concrete slab spanning between the edge beams.

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 has been the world's most slender arch bridge for more than 40 years.

It is about time that the old record was broken. It would not be very hard to break. For the network arch in fig. 23 and 8 and pages 59 to 62 the slenderness is 200/(1.24+0.5)=115. For the slimmer bridge in fig. 10 the slenderness is 135/(0.403+05)=150. For the bridge in fig. 97 the slenderness is 155. For the Brandanger Bridge in fig. 99 the final cross-section of the arch has not been decided, but the bridge 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 91 said: "…we recognise the need to integrate 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 can 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.

Normally the lower chord of a network arch bridge should be a simple slab. Where concrete with very high strength is available, prestressed slabs can be recommended for distances between arches up to 18 metres. Slabs are preferable to beams as far as upkeep is concerned. The slight prestress from the prestressing cables between the ends of the arches also reduces upkeep.

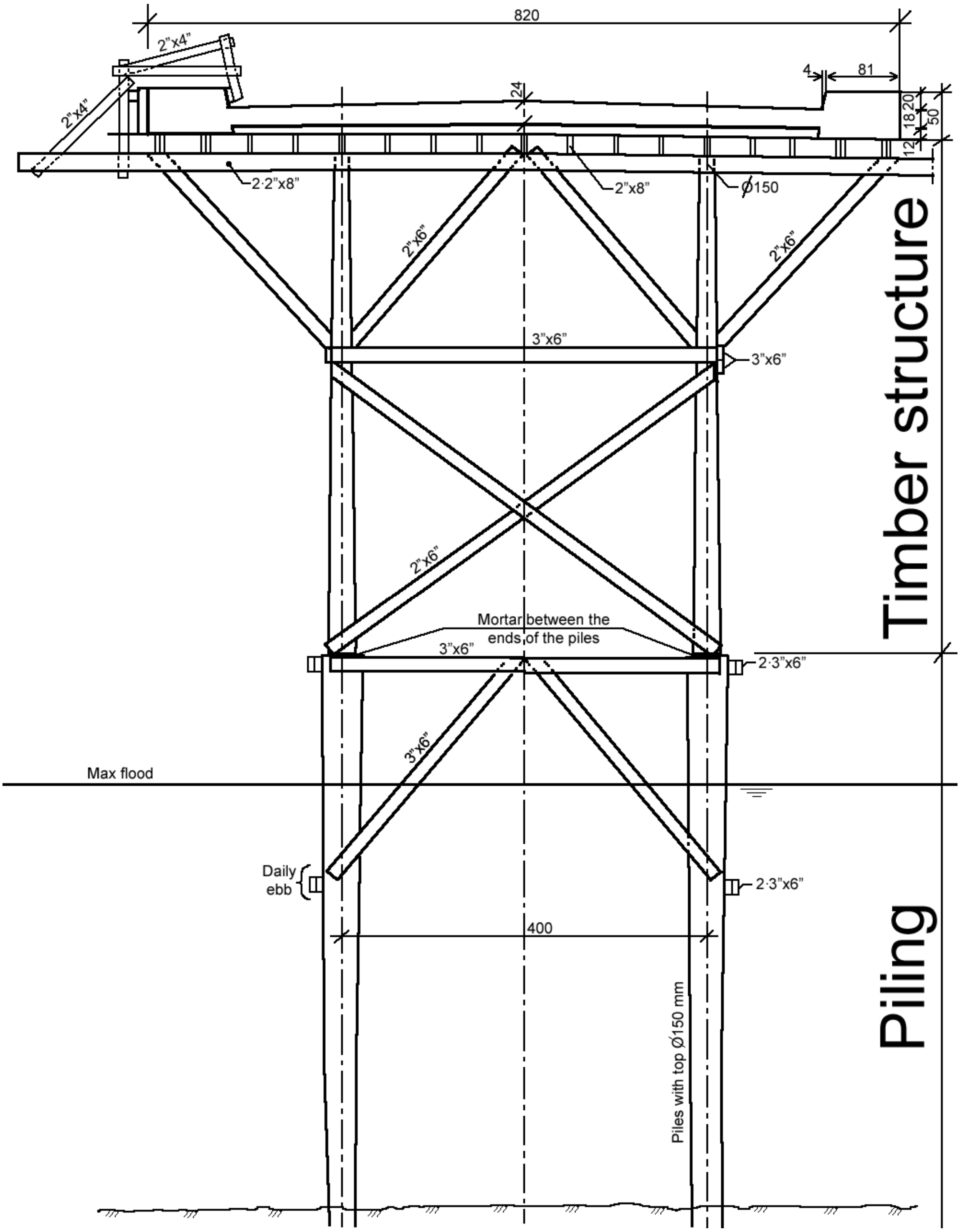


Fig. 7a. Scaffolding for erecting the concrete and steel of the network arch of the Bolstadstraumen Bridge

In the first two Norwegian network arch bridges the ties were cast on timber structures resting on piles in the river bed. See fig. 7a. When the arches and hangers had been erected, the hangers were tightened till they carried the concrete deck. Then the timber structure was removed. The form of the elements of the bridge was measured carefully prior to the erection of the steel. In order to control built-in stresses, great care was needed in the adjustment of the hangers.

The force in a hanger was found by measuring the deflection of the hanger due to a transverse force on the hanger. By comparing the actual forces in the hangers with the calculated ones, and the form of the loaded bridge with the intended forms of the arch and the deck, it was possible to determine which hangers needed adjusting.

A lecture on how to adjust hangers in network arches was a prerequisite for the author's PhD in 1964. A revised version of the manuscript can be found at the author's home page http://pchome.grm.hia.no/~ptveit/

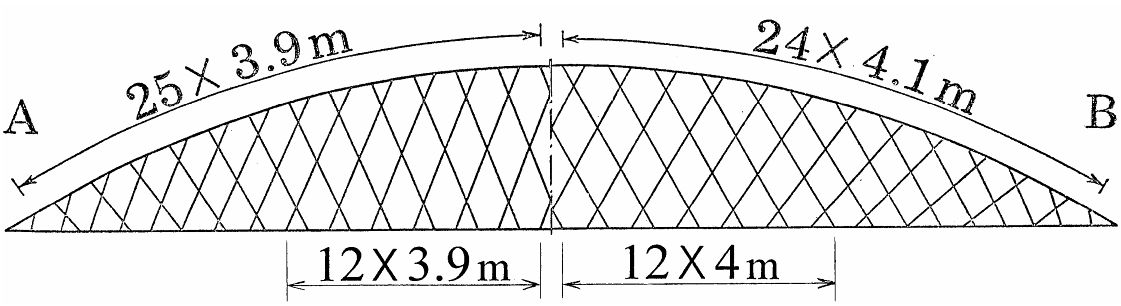


Fig 8. Arrangement of hangers for two bridges spanning 200 m

The hanger arrangement in fig. 8 was designed for the IABSE congress in Vienna 1980. These hanger arrangements are near optimal for the loads. pp. 59-72. Tveit 80a and 80b.

Some readers might be surprised that the author tries to avoid that two hangers meet at points in the chords. Only one hanger in each nodal point makes the nodal points simple, but other reasons are more important. In a normal network arch the decisive load cases are maximum load on the whole span. For these load cases equidistant nodes along the arch give the smallest buckling lengths in the arch and the smallest bending moments due to curvature of the arch.

The axle loads on the concrete slab between the arches spread out well before they reach the edge beam. See fig. 71. Therefore equal distances between the nodes along the lower chord give the smallest bending moments in the edge beam. This is not so important because the longitudinal bending in the lower chord is not great. Please note that the axle loads and the hanger force can be of the same magnitude.

Another group of load cases might also be decisive. This is live load on part of the span. Then some hangers relax. When many hangers relax, the network arches function in about the same way whether the hangers meet in the nodal points or not. There are two reasons why one should not let load on part of the span be decisive. 1: These load cases are complicated to calculate. 2: If the bridge has to carry bigger loads in the future, loads on part of the span that make some hangers relax will lead to a bigger increase in stresses than loads on the whole span. See figs 73 and 93.

For the two bridges shown in fig. 8 the nodal points in the lower chord are equally spaced along the middle of the tie. The hanger arrangement on the right makes the hangers less inclined to relax. Thus it is more likely to be optimal in the future when live loads increase and stronger concrete is used.

Fig. 9 shows predicted steel weights for various types of highway bridges. Max. Herzog, 1975, compiled most of the lines. The author compiled a line for network arches for the IABSE congress in Vienna in 1980. Tveit 80. The dot in the diagram corresponds to the network arch at the Åkvik Sound Bridge designed in 1998. See the following pages.

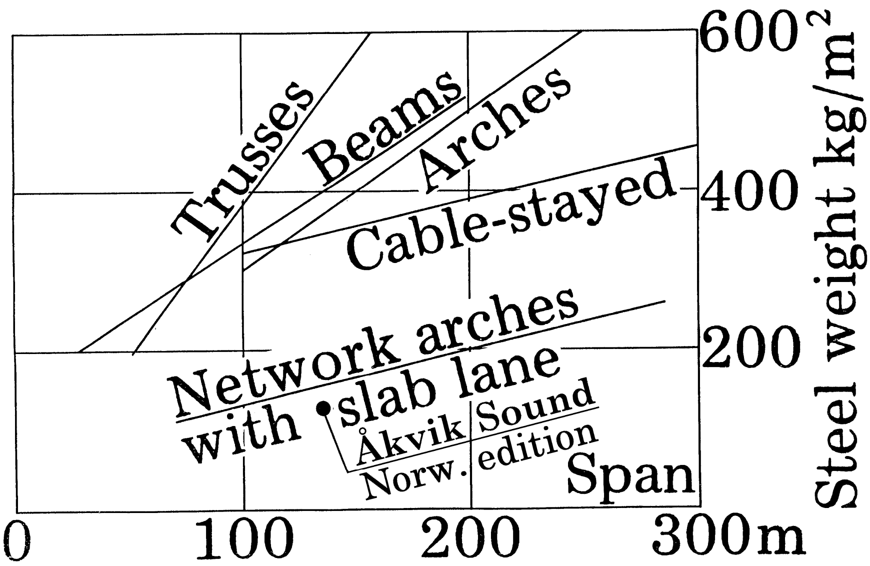


Fig. 9. Amount of steel in different types of highway bridges

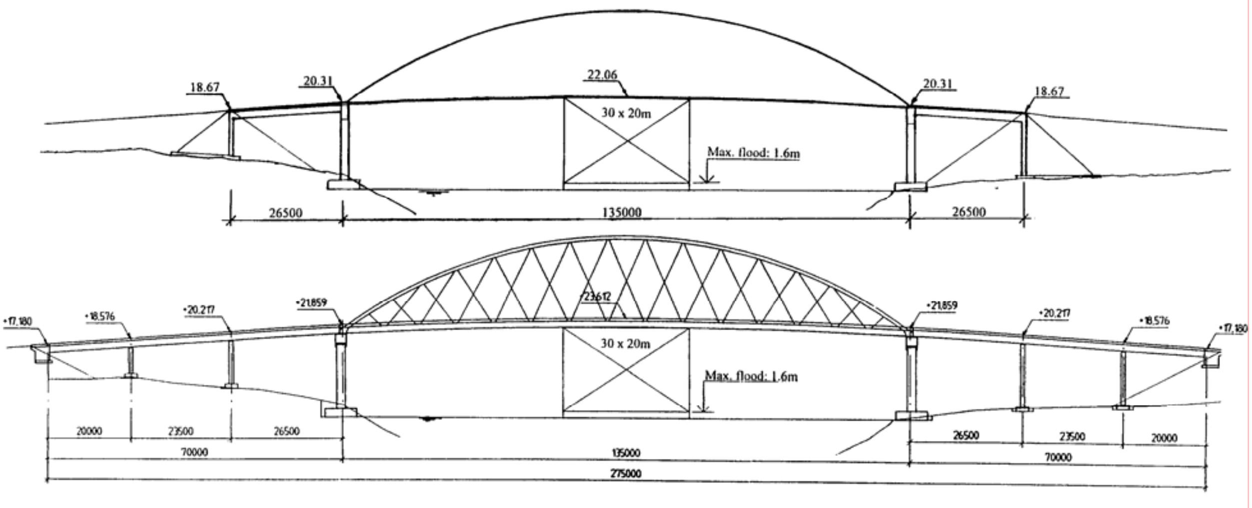


Fig. 10. Two bridges designed for the Åkvik Sound in Northern Norway. Tveit 1999 a, b, and c.

In 1997 to 1998 a group of steel users in Norway were looking for competitive bridge types for spans between 120 and 300 metres. The group initiated the design of the two bridges shown in fig. 10. The author designed the slim bridge. That bridge will be discussed in detail because it is a most competitive network arch bridge. Its steel weight is indicated by the dot in fig. 9.

A highly reputed Norwegian firm designed the heavier bridge. The lower chord of the heavier bridge is 1.5 metres deeper than the lower chord of the slimmer bridge. Therefore the ramps of the slimmer bridge can be made shorter.

In the bridge with many side-spans a beam is carried across the fjord by an arch. In the slim bridge the lower chord of the arch is much slimmer than the side-spans. Thus aesthetic reasons argue for only two side-spans. The ramps usually cost less per metre than the side-spans.

The real thickness in the slimmer bridge is meant to be shown in the drawing, but the arches are never seen behind each other and the railings are not shown. Thus the real bridge would seem less slim. The hangers of the slimmer bridge are not shown because they would be less than 0.03 mm thick in the drawing. As can be seen from fig. 7, they would hardly be seen even on a clear day. The skeleton lines of the slimmer bridge in fig. 10 are shown on the next page in fig. 12.

The lane loads used in the design of Norwegian highway bridges in fig. 10 are shown in fig. 11. On the footpath outside the hangers the maximum loads are 4 kN/m^2 and a single wheel load of 18 kN. When there is traffic load between the hangers, the load on the pavement outside the hangers is 2 kN/m^2. The skeleton lines of the slimmer bridge are shown in fig 12. See next page.

The main span of the Åkvik Sound network arch was designed again by Teick and Wendelin in 2001. They used German loads and codes. In this publication that bridge is called the Åkviksound network arch. See page 93.

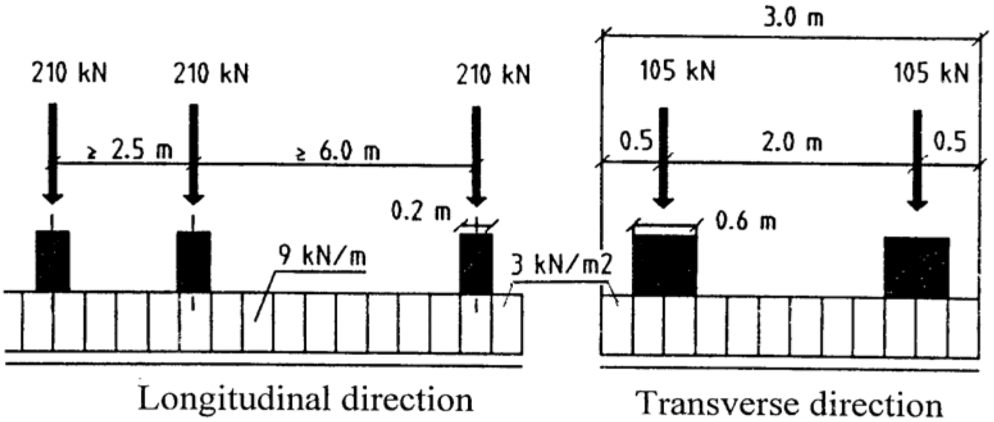


Fig. 11. Lane loads used for the design of the Åkvik Sound network arch in fig. 10

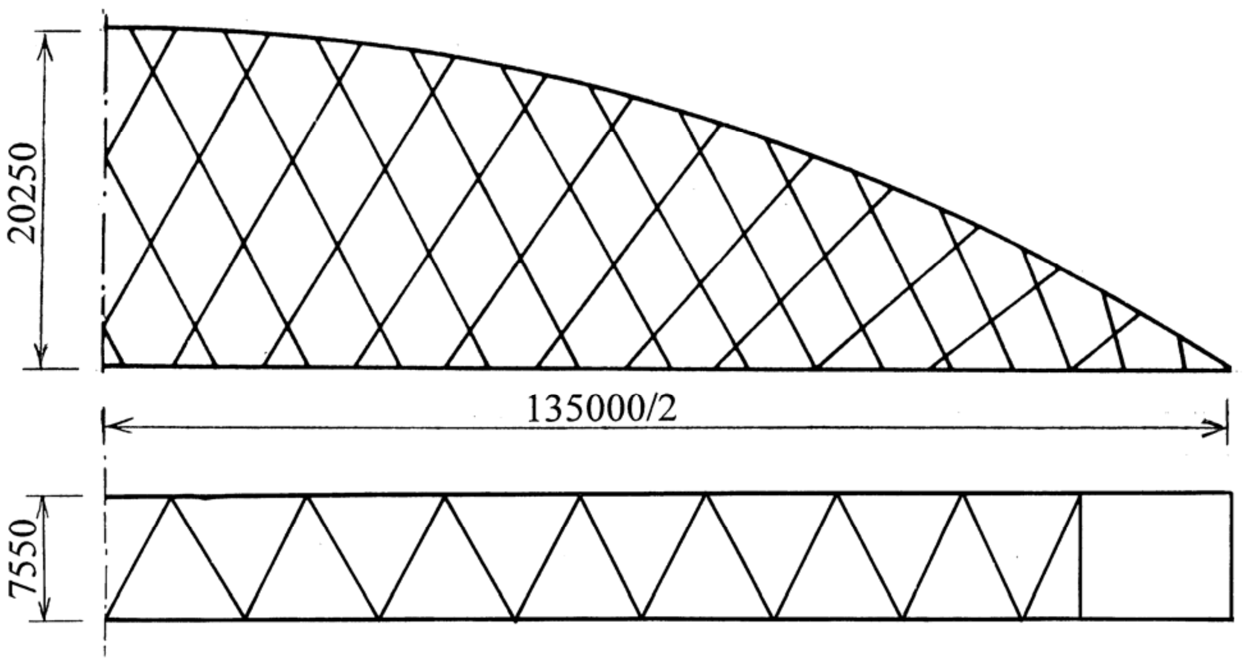


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Fig. 12. Skeleton lines for arch and windbracing of the slimmer bridge in fig. 10

The cross-section of the tie of the slimmer bridge is shown in fig. 13. It is made of concrete with a cube strength of 45 MPa. For the heavier bridge ~40% more concrete with a cube strength of 55 MPa has been used. If a higher cube strength had been used in the lighter span, the tie would have been lighter and the hangers may have been less steep. The number of hangers in the lighter span could have been reduced by 15% without any great increase in the steel weight.

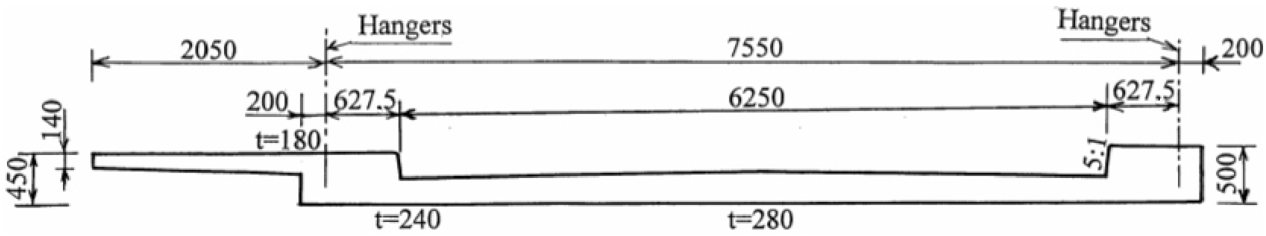


Fig. 13. Cross-section of the tie of the slimmer bridge

STEEL WEIGHTS OF THE LIGHTER SPAN

Arches:	88 tons
Windbracing:	5 tons
Hangers:	16 tons
Miscellaneous:	3 tons
Structural steel:	112 tons
Prestressing:	18 tons
Rebars:	65 tons
Total:	193 tons

STEEL WEIGHTS OF THE HEAVIER SPAN

Arches:	234 tons
Windbracing:	25 tons
Hangers:	42 tons
Transverse beams:	149 tons
Lower chord:	114 tons
Miscellaneous:	41 tons
Structural steel:	596 tons
Rebars:	83 tons
Total:	679 tons

Fig. 14. Steel needed for the two spans in fig. 10.

Erection costs will be small because there is little material to erect. For the lighter span, steel with a yield stress f_y=460 MPa is advantageous and has been selected. For the heavier span f_y=355 MPa and f_y=420 MPa have been chosen. The exposed steel surface that needs corrosion protection is 7 times bigger.

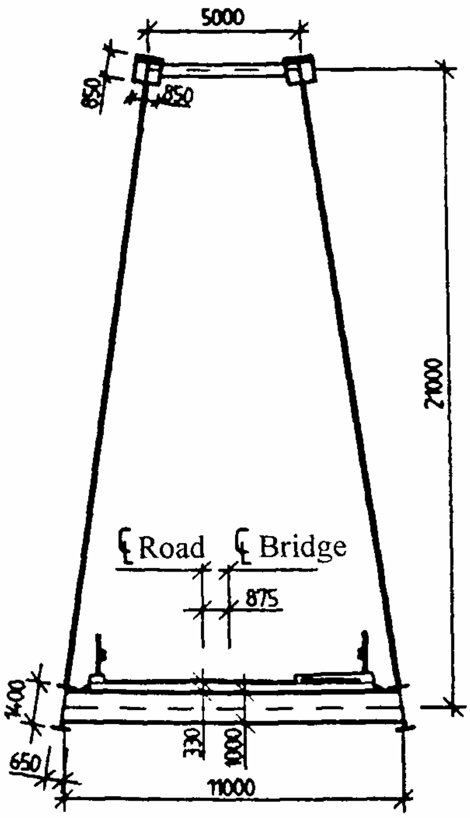


Fig. 15. Cross-section at midspan of the heavier bridge

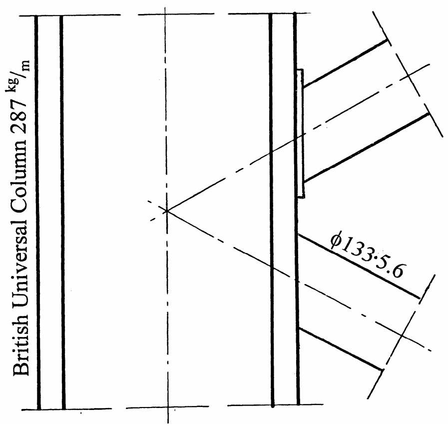


Fig. 16. Joint in the windbracing

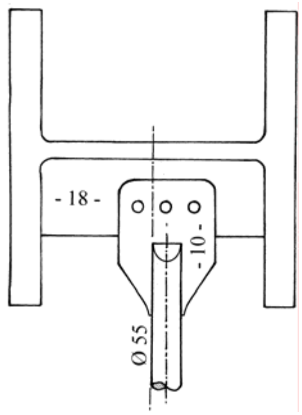


Fig. 17. Fastening of hanger to the stronger arch

For the heavier span the welds are 23 times as long as for the lighter span. The longer welds indicate that more parts need to be cut out and handled. Thus the price per ton might well be less for the lighter span. See also fig. 99. The main span in the lighter bridge uses only 28 % of the steel needed in the main span of the heavier bridge. Railings and expansion joints are not included in these weights.

The transverse beams in the heavier span weigh just as much as the structural steel in the lighter span. The transverse beams are heavy partly because the arches are outside the pavement and sloping towards each other. This increases the span of the transverse beams. Furthermore, there are no shear connections between the concrete lane and the steel of the transverse beams. This makes it easier to put the concrete elements in the lane in place.

To support the claim that the steel in the lighter bridge is not costly to fabricate, some structural details are shown in figs 16 to 20. Fig. 16 shows two ways of fastening the windbracing to the lighter arch. It can be done by welding or by flanges that are fastened to the tubes by means of high strength bolts.

In fig. 17 a hanger is fastened to the stronger arch by means of high strength bolts. The arches are Universal Columns. This profile is often used in high-rise buildings. The profile is ideally suited to the network arch where the buckling length is much smaller in the plane of the arch than out of the plane of the arch. The profile can be bent to the shape of the arch in the steel mill. This saves a lot of welding and other work.

Fig. 19 shows flange plates in a joint in the arch. The high strength bolts in the flanges are needed only for erection, because there is compression over the whole cross-section of the arch for all load cases. The joints in the arch can also be site welded. The steel structure at the end of a network arch is shown in fig. 18. For the steel weights in fig. 14 rubber bearings have been assumed.

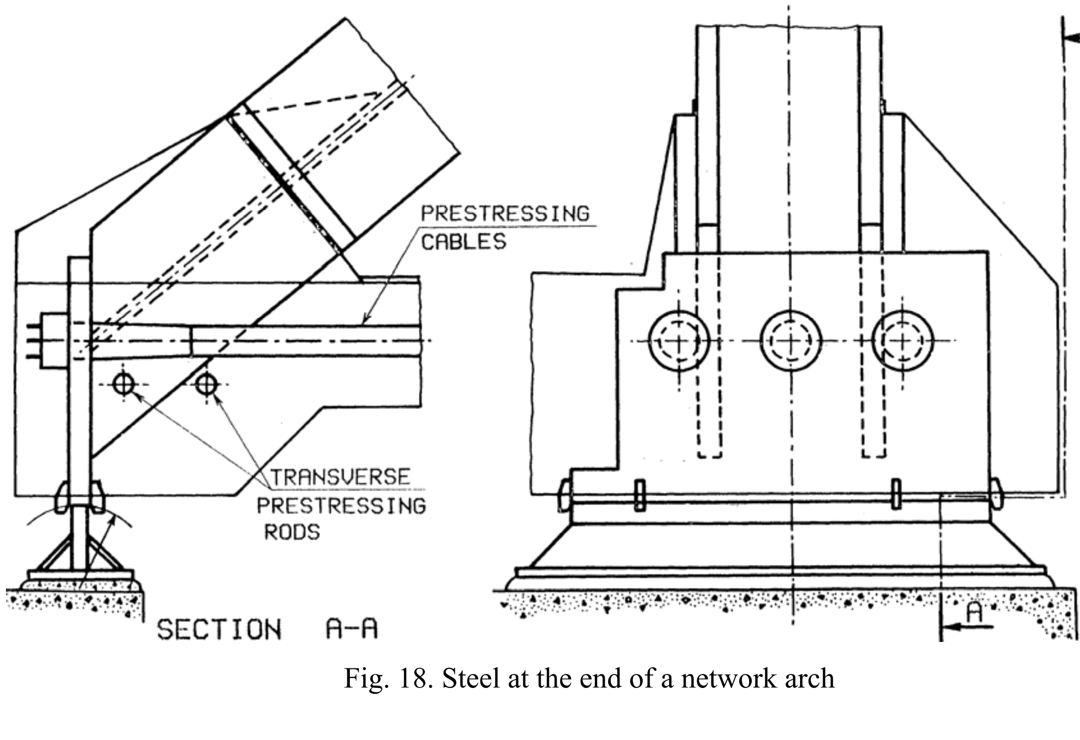


Fig. 18. Steel at the end of a network arch

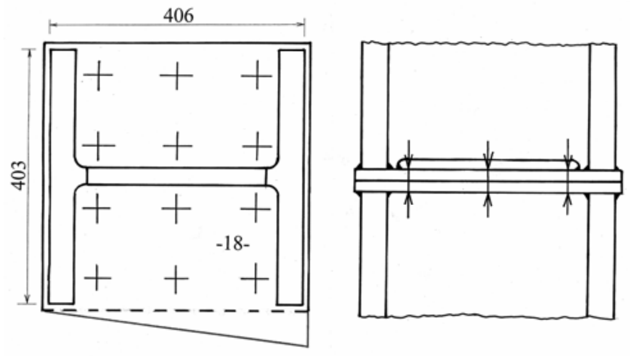


Fig. 19. Joint in the arch

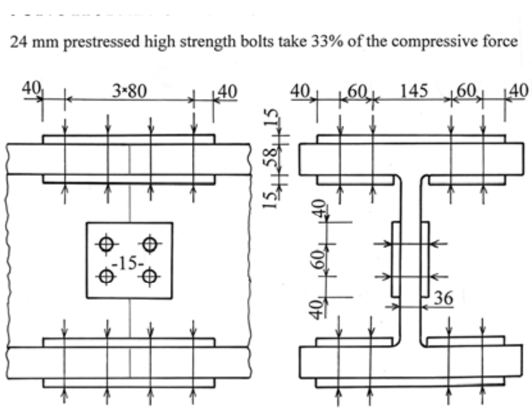


Fig. 20. Junction in UC 356x406x467

The steel and concrete for the network arch for the Åkvik Sound would not be costly. The economics also depend on the method of erection. The finished network arch, including the concrete slab, weighs ~1000 tons. The whole span can be finished on shore and lifted to its final position by two big floating cranes of the type used in offshore work. Using these cranes costs a lot.

The most versatile methods of erection utilise the fact that the structural steel, supplemented by a temporary lower chord, can form a steel skeleton with so much strength and stiffness that it can be moved. The steel for the temporary lower chord for the Åkvik Sound network arch weighs around 24 tons. It is shown in figs 21 and 22.

This temporary lower chord needs no corrosion protection. It can be produced on site using high strength bolts. When the steel skeleton and the reinforcement are in place, the lower chord can be cast. First the transverse beams at the ends of the arches are cast. Then the smallest edge beam is cast. See fig. 13.

To prevent bending moments due to hangers relaxing, the casting must start at both ends of the span and proceed to the middle. See page 87 fig. 90. Then the biggest edge beam is cast in the same way. When the edge beams have been cast, they will take most of the bending in the tie. The prestressing cables will take most of the axial force in the tie. Finally the deck is cast. See also pages 30 and 30a.

Later we shall see how the structural steel in a network arch bridge can be erected on the side-spans and be moved in place by means of pontoons and/or cranes. See pages 15, 20, 21, 35 and erection and removal of the temporary lower chord on pages 50a to 53a.

With slight alterations this temporary lower chord can be used in many different network arches. It can also be used in the unlikely event of replacement of the concrete in the concrete tie of an existing network arch. If the whole span is to be removed, a temporary lower chord can be used if the concrete is removed before the steel skeleton is removed.

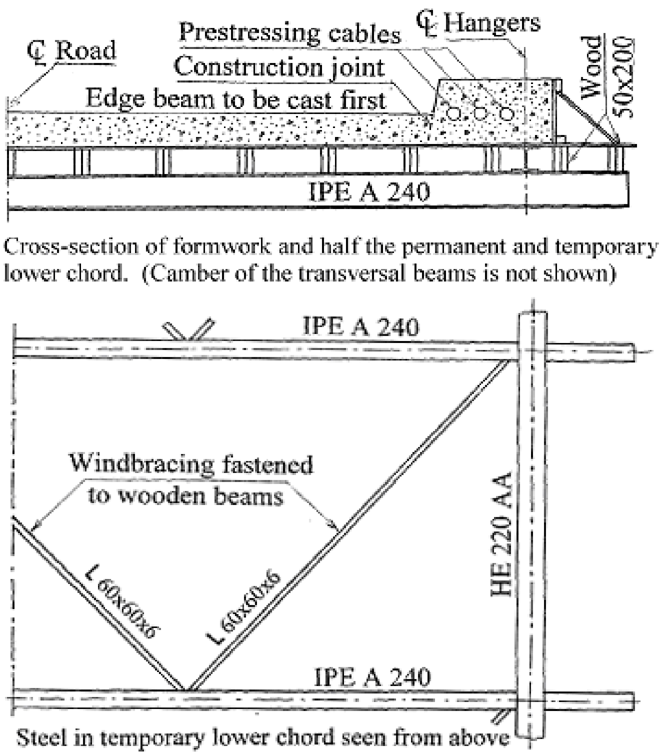


Fig. 21. Temporary lower chord
for the slim bridge in fig. 10

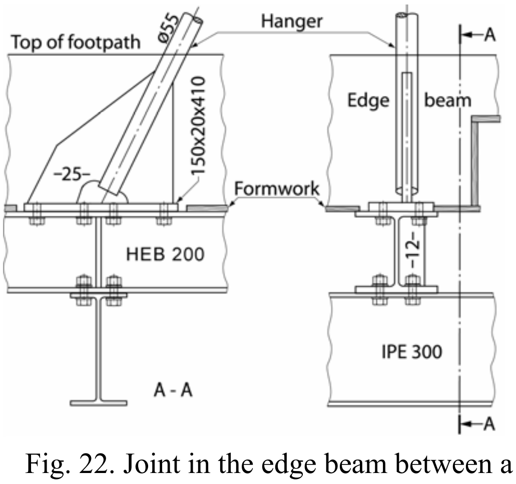


Fig. 22. Joint in the edge beam between a temporary lower chord and a hanger

Two tied-arch bridges spanning 200 metres are now going to be compared. The bridges are shown in fig. 23. The bridge with vertical hangers was built over the Danube in Bavaria in 1977. Kahman 1979. The author designed the network arch for the IABSE congress in Vienna in 1980. Tveit 1980 and pages 59 to 72. 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 be made of concrete 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 lower chord of the network arch is just under half the stiffness of the lower chord in the bridge with vertical hangers.

The upper part of fig. 24 on the next page shows a comparison between the influence lines for bending moments in the lower chord of the two spans. 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 can become decisive mainly because much of the strength of the concrete is needed for taking the variation of the axial force in the tie.

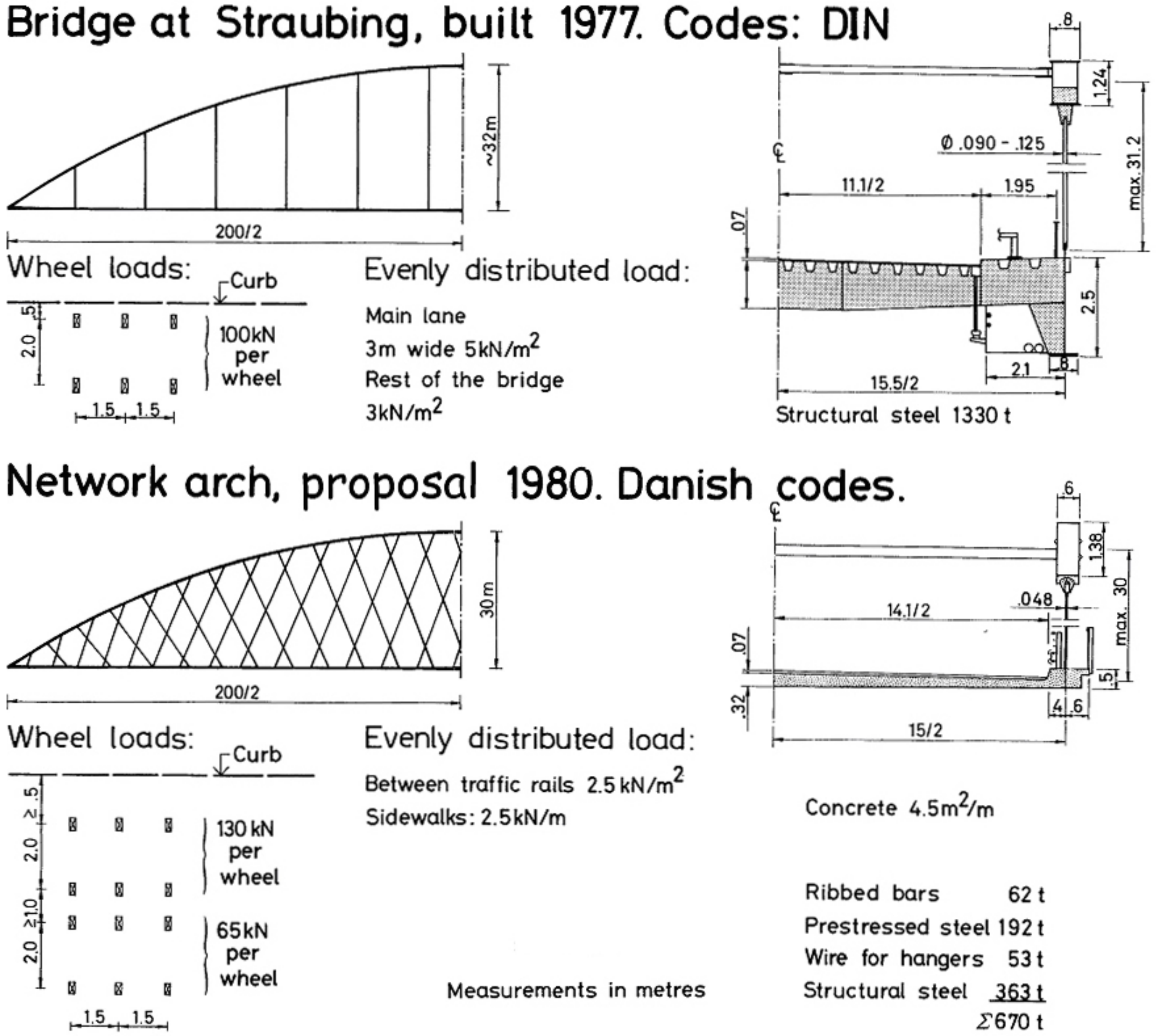


Fig. 23. Geometry, loads and quantities of the two tied arches

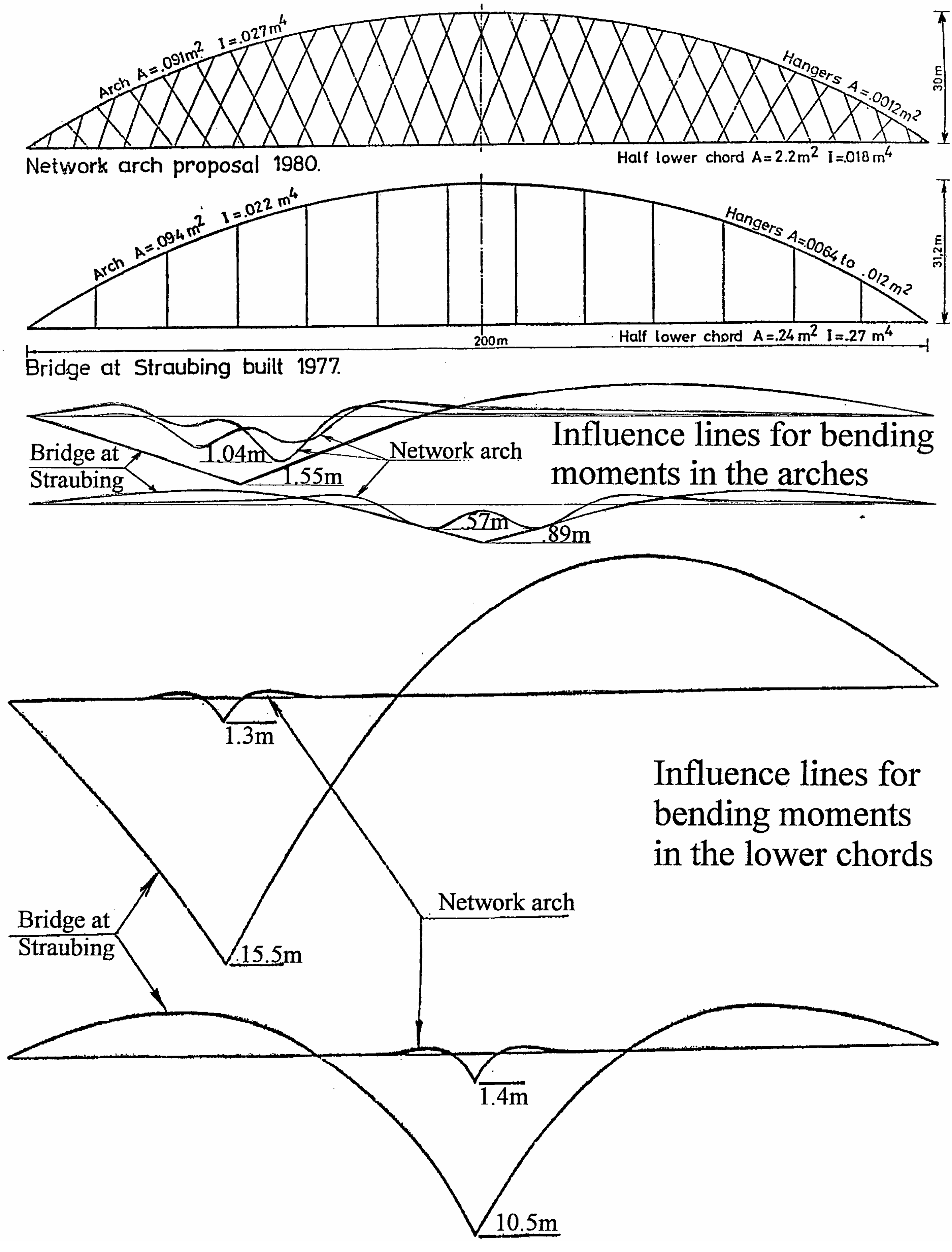


Fig. 24. Areas, stiffnesses and influence lines for the lower and upper chord of two tied arches

The bottom part in fig. 24 shows influence lines for bending moments in the arches. The network arch usually saves more than half the steel compared with other bridges, but the steel weight for the bridge with vertical hangers is in this case only twice the steel weight of the network arch. This impresses the author because the Straubing Bridge uses no concrete in the deck.

When the arches are made of concrete, 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. When the arch is made of steel it should have curvature more like a circle. That would lead to more even maximum bending in the chords.

If the shape of the arch of the Straubing Bridge had been more like a circle, the ordinates of the influence line for the bending moments in the chords could have been more equal. Now the ordinates in the middle of the span are smaller than in the quarter points.

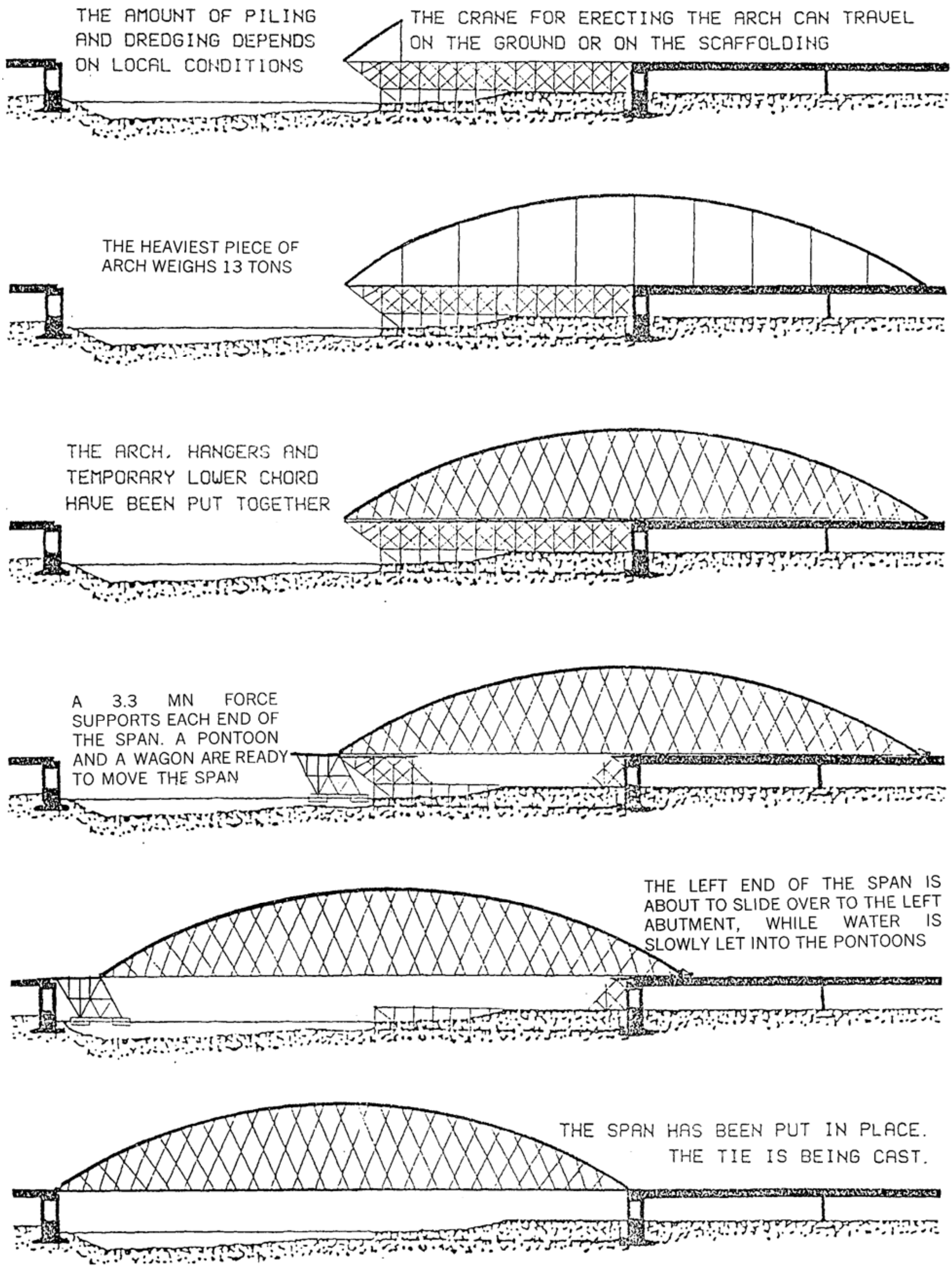


Fig. 25. Erection procedure for a network arch spanning 200 metres

Fig. 25 shows how a network arch bridge can be erected on the site at Straubing. The costly and complicated adjusting of hangers can be avoided if the arch, hangers and the temporary lower chord have the correct shape and no stresses when erected on the side-span. A wagon for moving the steel skeleton can roll along over the side spans. The side spans do not need strengthening to carry this wagon. After the span is in place, the steel crew has finished their job and the concrete workers can take over.

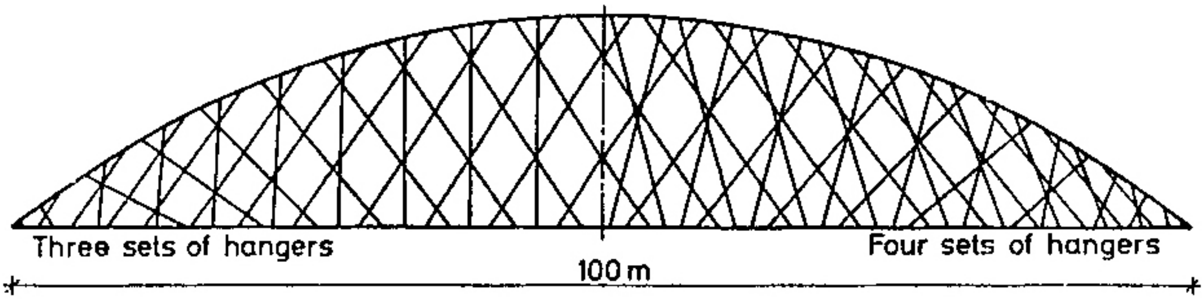


Fig. 26. Tied arches with various hanger arrangements

In special cases, for instance when there are very slim arches and/or big concentrated loads; many sets of hangers might be used. See fig. 26. Tveit 1984 b and c. If network arches are to support long-spanning flat roofs, hangers that go sideways out of the plane of the arch can ensure the sideways stability of the arch.

If three sets of hangers are used, the nearly vertical hangers have a tendency to attract too much force. To avoid this, the almost vertical set of hangers could be put in after the tie has been cast and before the asphalt layer has been laid. If four sets of hangers were used, it would probably be all right to put in half the hangers after the steel skeleton is in place.

The idea of making the arches from thin-walled steel tubes to be filled with concrete can be used in network arches spanning 250 m. These can be erected using two floating cranes with a combined lifting capacity of 250 tons. This can be suitable in rivers and in places where bigger cranes are not available.

Conservative preliminary calculations indicate that the steel skeleton of a network arch with a span of 150 m and a 10 m wide lane can weigh 114 tons. Two or three military helicopters can probably lift this, but the Russian and the American military keep the lifting capacity of their helicopters a secret. According to Guinness Book of World Records 1995 a Russian helicopter lifted 57 tons up 6560 feet.

To make the world's lightest arch bridge, one would have to use an orthotropic steel deck as in the Straubing Bridge in fig. 23. The low weight in the lower chord would lead to small slopes of hangers in order to get enough resistance to the relaxing of hangers. Three of four sets of hangers might be economical to keep the bending in the chords down.

THE FEHMARN SOUND BRIDGE AND SIMILAR DESIGNS

After his graduation the author got a grant to study in TH-Aachen in 1955-56. The author had already found that he could save about two thirds of the steel used in arch bridges with vertical hangers. Nobody in Aachen was interested in arch bridges with inclined hangers with multiple intersection. Finally Professor Philipp Stein took pity on the author and 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. 27. Stein and Wild 1965. 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 fig. 27 and 28. 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 Philipp 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.

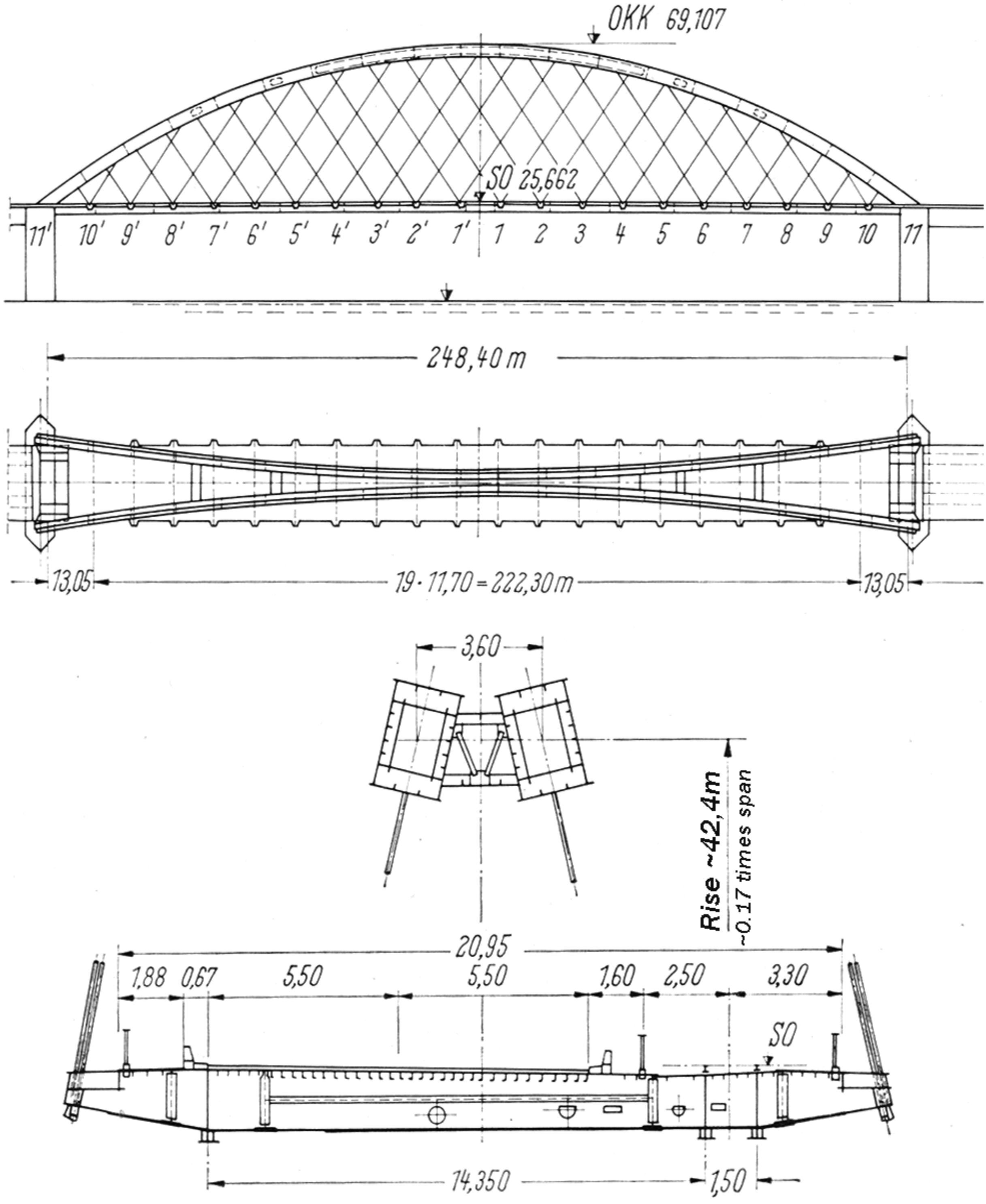


Fig. 27. The main span of the Fehmarn Sound Bridge. Opened in May 1963.

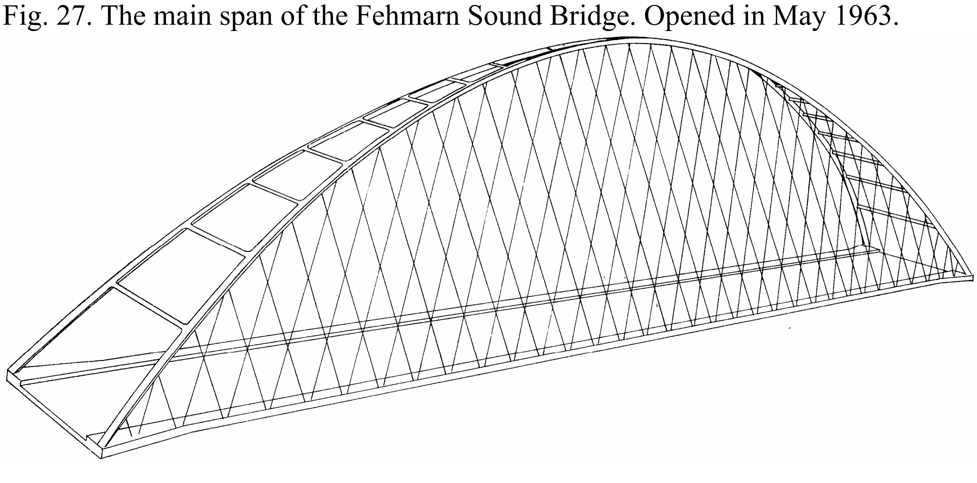


Fig. 28. Proposed network arch design from the author's PhD thesis. Tveit 1959.

In figs. 27-29 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. 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.

Professor Masao Naruoka saw model tests on the Fehmarn Sound Bridge in TH-Hannover in 1960. Naruoka took the idea to Japan where it has been flourishing. Naruoka 1977, Fujiono 1965, Hiroshi 1965 and Tagai 1970. More than 50 bridges of this type have been built. Kikuno 1973, Yoshikava 1993 and Nakai 1995. Many references to Japanese articles on network arches can be found at the end of Tveit 1999a.

The Japanese have a small country and it is very important for them to make it more beautiful by building beautiful bridges. Many of their bridges resemble the heaviest bridge for the Åkvik Sound. See fig. 10, page 9. About half of their network arches have parallel arches. Nakai 1995. Most Japanese articles on network arches tell about spans with arches sloping against each other. The Shinhamadera Bridge, fig. 29, is the longest Japanese network arch built so far. 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, 1932 and 1936.

In most Japanese network arches all hangers have the same slope. That was also the case in many of the bridges that Nielsen built in Sweden between the two world wars. In his bridges the hangers were supposed to relax due to live loads. The constant slopes of the hangers made the design of these bridges less complicated when Nielsen’s method of calculation was used. Nielsen 30. See also p. 55. With modern methods of calculation it is easy and more economical to vary the slope of the hangers. It is the author’s impression that the Japanese think that the constant slope of hangers looks best.

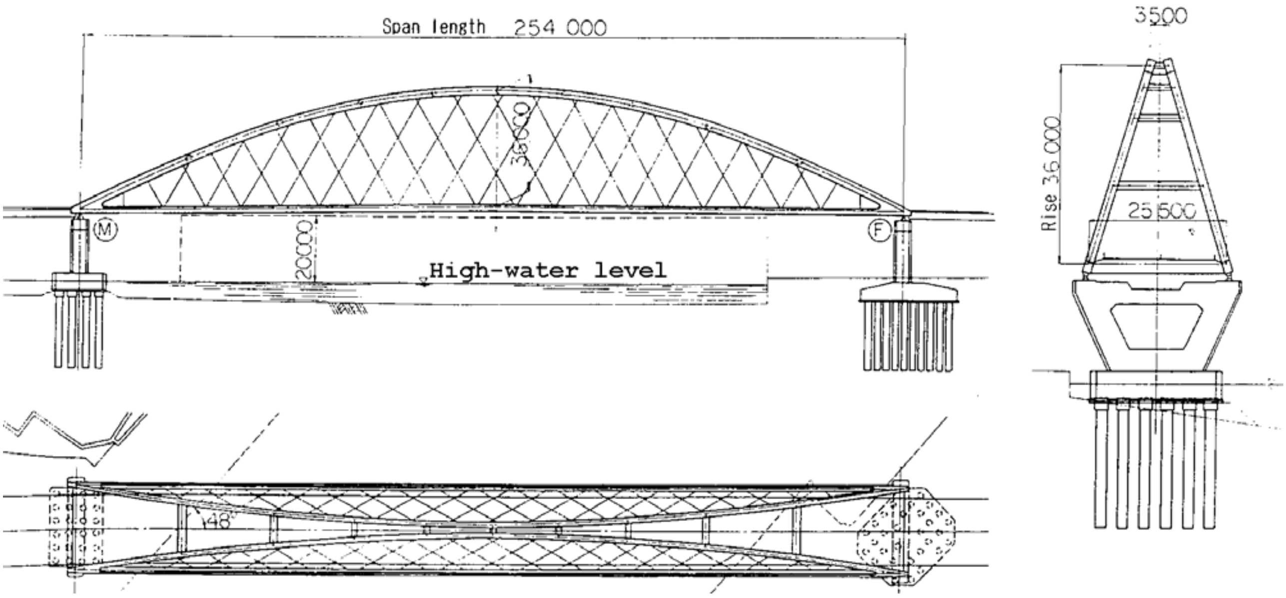


Fig. 29. Shinhamadera Bridge Built 1991. Yoshikawa 1993.

The arches in the bridges in figs. 27 to 29 slope towards each other. That looks nice and the forces in the windbracing and the wind portal are reduced. The cost of the spans increases considerably because the span and the steel weight of the transverse beams in the lower chord go up. - When the author was in Japan in 1998, he said to an engineer “It is all right for you in Mitsubishi Heavy Industries to make network arches that use three times as much steel as necessary as long as you have a client who is willing to pay for it”. He politely refrained from answering.

Gimsing 99 shows many structurally dishonest bridges and concludes: ”A bridge should be designed in such a way that structural function and efficiency are expressed in the form. Modern high strength materials should be used to make the bridge light and graceful, not to shape it without considering the structural function.” The author supports this point of view.

NETWORK ARCH IN PROVIDENCE ROHDE ISLAND

A network arch is being built in Providence, Rhode Island. See fig. 30. It will be opened in 2007. Fig. 31 gives an architect's impression of the bridge. The span is 122 metres. Three parallel arches are used because the distance between the two outer arches is 50 m. Because of the width of the bridge, the lower chord could not be a simple concrete slab. Instead transverse beams under the deck have been used. There is one hanger at the end of each transverse beam. The steel skeleton was built on the river bank. It was floated 25 km up the bay and put on its piers in the spring of 2006. Afterwards the concrete slab will be cast.

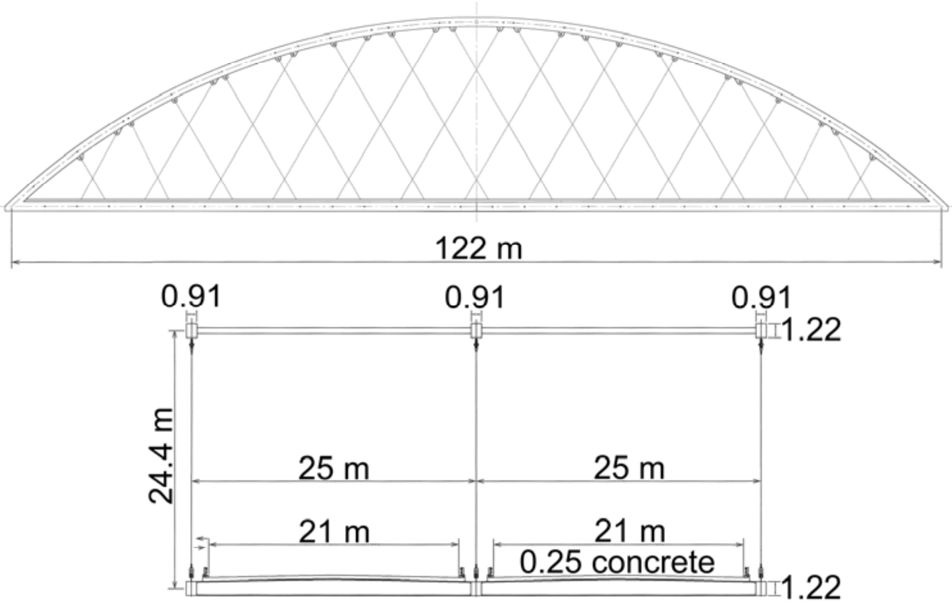


Fig. 30. Network arch in Providence Rhode Island, USA

Fig. 31. Network arch in Providence. Artist's impression.

Fig. 31a shows the steel skeleton of the network arch at Providence before it is floated to the site

The author would like to show two ideas for the design of network arches. The first one is illustrated in fig. 32. It shows the first stage of erection and transport of a skewed bridge across a canal.

The angle between the bridge and the canal is 45 degrees. The span is 100 metres. The width of the canal is nearly 70 metres. In order to reduce the thickness of the concrete slab the bridge has three arches.

The structural steel, supplemented by a temporary lower chord, is erected on the side-spans at one side of the canal. If the shape of the steel skeleton is right, then no adjustment of hangers is needed later.

While the beams on top of the pontoon are tied to the abutment, the steel skeleton is rolled to the middle of a pontoon. Then the pontoon is pulled across the canal. Finally the steel skeleton is rolled on to the abutments at both sides of the canal and the tie is cast.

Fig. 33 shows a suggestion for a bridge in Skodje in Norway. Very few ships will pass under the bridge, so the low parts of the arches could be allowed above the navigable waters in the fjord. Since there is little tension in the tie, it is possible, but probably not advisable, to omit the prestressing cables.

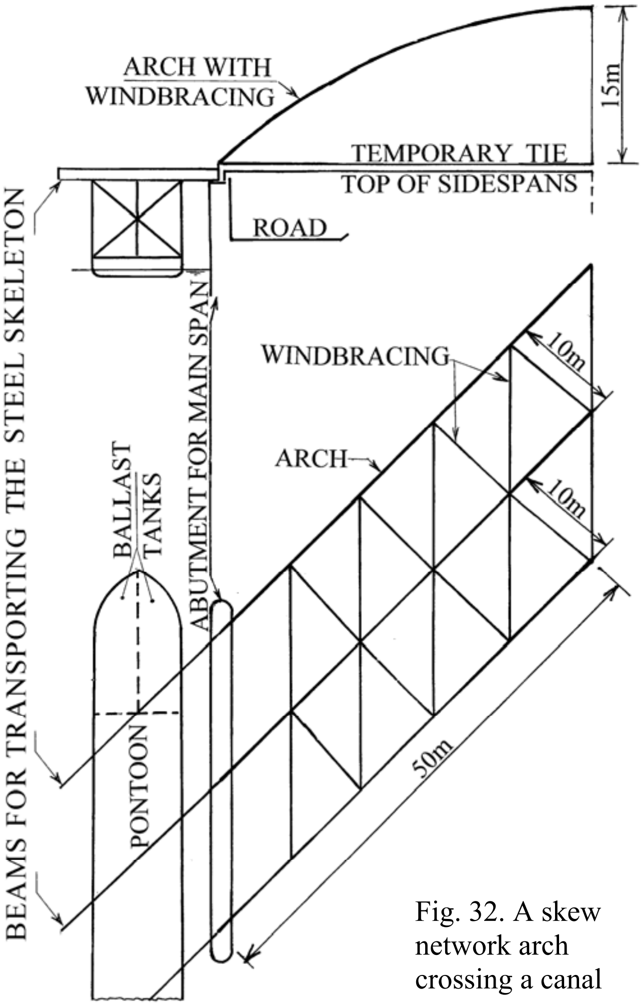


Fig. 32. A skew network arch crossing a canal

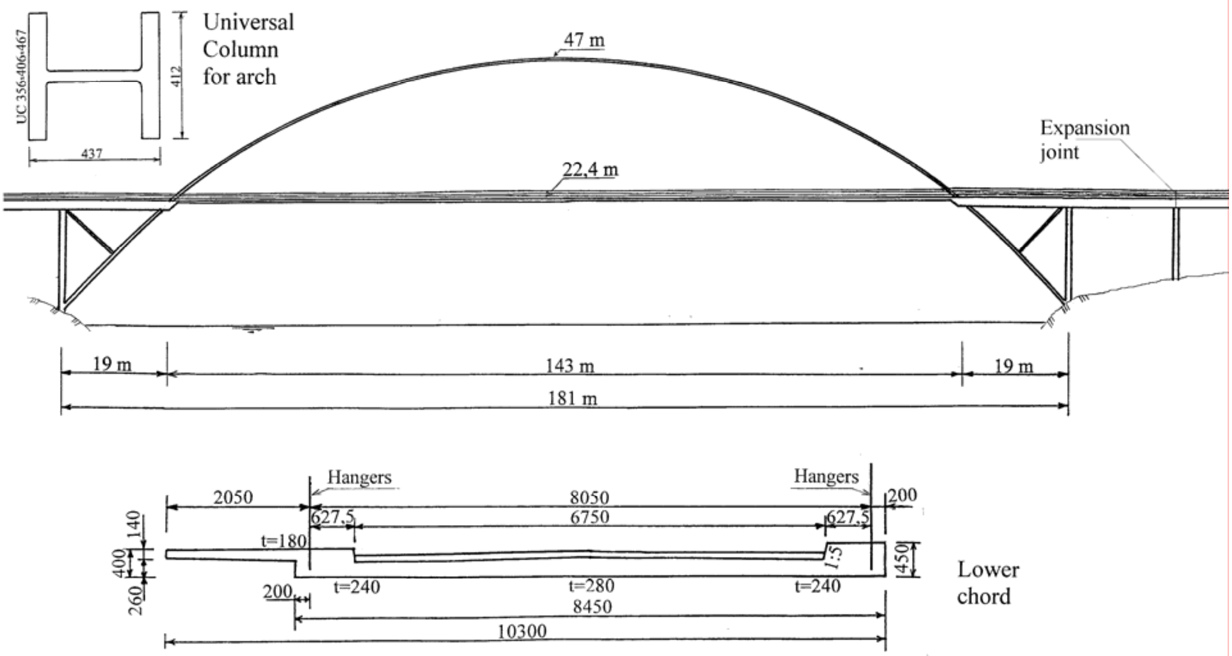


Fig. 33. A suggestion for a network arch bridge across Storestraumen in Skodje in Western Norway

Maybe the arch should be a box section to ensure enough buckling strength. The parts of the arches that are under the lane can be floated in or erected from the side-spans. The structural steel above the lane, supplemented by a temporary lower chord, see figs 21 and 22, can be erected on side-spans on one side of the fjord. It can then be floated across to the other side of the fjord by means of a floating crane or a pontoon.

Fig. 33a shows the reinforcement in the edge beam nearest the footpath. The capital letters indicate the sequence in which the reinforcement is to be put in place. The longitudinal prestress reduces the need for longitudinal reinforcement. In fact there will be no transverse cracks in the slab in the serviceability limit state.

The cracks in the slab in the longitudinal direction of the bridge will be nearer to each other because of the longitudinal prestress. This will lead to smaller cracks. This beneficial effect has not influenced the choice of transverse reinforcement in the slab.

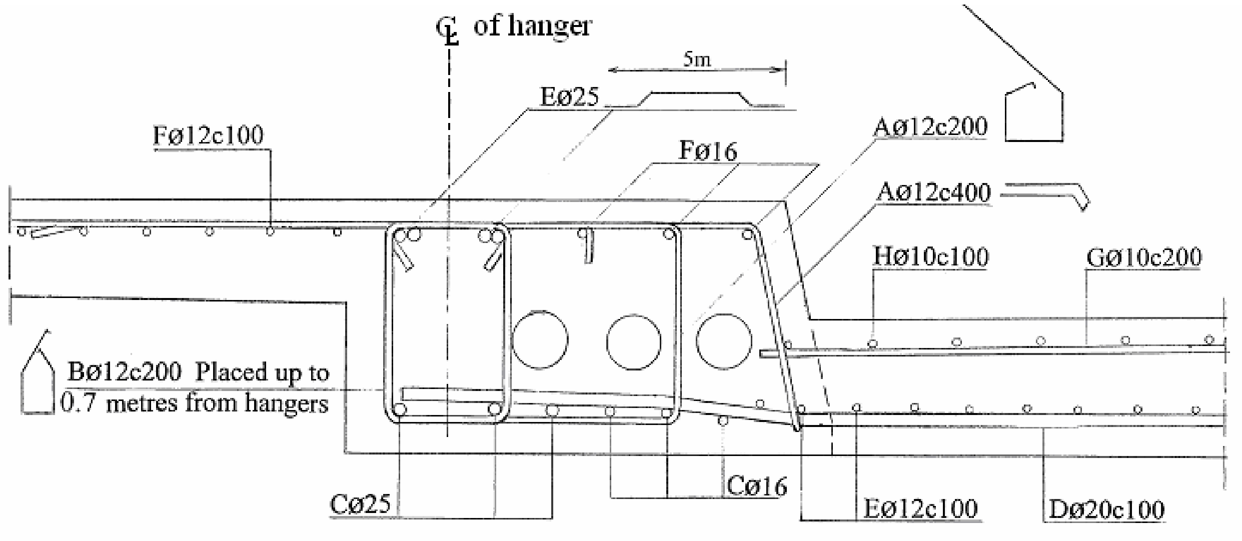


Fig.33 a. Reinforcement in the edge beam nearest to the footpath in the suggested Skodje Bridge.

There is more on the erection on the side-span and the removal of the temporary lower chord in: "Erection of the steel skeleton of the network arch on the side span of the Skodje Bridge" and in "Removing the temporary lower cord of the Skodje Bridge." See pages 50 to 53a.

In general the bridge authorities have a considerable influence on the type of bridge chosen. The author was very pleased when Professor Günter Ramberger after the lecture in Vienna in March 2000 said: "If design alternatives from firms have been invited and the network arch has the lowest price, it would be hard to avoid accepting it if it is technically in order. This applies to the whole of the EU." Norway is not a member of EU. A committee decided to build a bridge with the whole arch under the lane. See fig. 33b.

The author would very much like to work on preliminary designs of network arches in co-operation with bridge authorities and/or firms that take over the final design. His age and his pension are so high that his days for final designs are over.

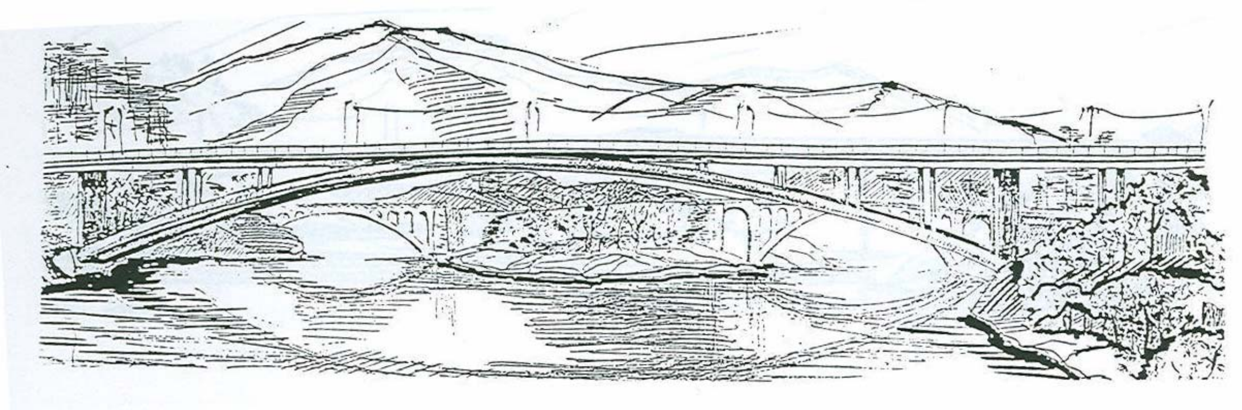


Fig. 33b. This arch bridge was built over Storestraumen in Skodje in 2000.

QUESTIONS AND ANSWERS

During his lecture tours the author has been asked many good and relevant questions. Some are answered here and some have influenced the text in other parts of this publication. If somebody has questions that have not been answered, the author would be very happy to answer. The author would like to include the answers in the Internet edition of this publication. The address will be: http://pchome.grm.hia.no/~ptveit

Q: Why have more network arches not been built?
A: The short answer is: The author does not know. An obvious explanation might be that optimal network arches are not economical. The author is not willing to accept this explanation and will try to come up with other reasons.

As you can see from the list of literature on pages 98 to 98a, efforts have been made to draw attention to this very promising type of bridge, but for a long time interest was very limited. Part of the reason might be human nature. Back in the fifties a speaker in the student's society in Trondheim said something like this: "You might have heard about the agility and outstanding achievements of the human spirit. The truth is that the human spirit is slow, backward and utterly conservative."

All design offices have a shortage of engineers that can be trusted with the design of network arches, and these engineers have many other tasks that have high priority. For a firm it might previously not have been economical to find out all about network arches to design just one bridge. After the establishment of the EU this might have changed. See Professor Ramberger's statement on page 20a.

Network arches require co-operation between steel firms and concrete firms. The firms might have less motivation to co-operate because network arches use little steel and little concrete. Bridge authorities have better reasons for building network arches, but the introduction of the network arch would create extra work for them. If the bridge authorities do not promote the network arches nobody else can.

A man of great experience in bridge building Director Man-Chun Tang of T. Y Lin International was involved in the design of the Fehmarn Sound Bridge. About 40 years later he told the author that in his experience with long spanning arch bridges the dead-load/live-load ratio was so big that it does not pay to use inclined hangers. The problem of erection also speaks against the network arches.

In the author's opinion increasing loads, stronger building materials and suitable methods of erection will favour inclined hangers for long spans and even more so in the future. The network arch is more competitive for spans where the distance between the arches is less that 15 to 18 m.

In hindsight one might wonder why the author was allowed to build the first two network arches. The bridge at Steinkjer was built because the town engineer at Steinkjer thought that promising ideas should be helped along. He thought that it would be safe to build a network arch because Professor Arne Selberg supported the idea.

At that time Professor Selberg was the number one bridge expert in Norway. He thought that network arches should be built, but the bridge office of the Norwegian Public Roads Administration was sceptical. Then the author's mother went to Oslo to talk to her brother about her son's fascination with a very promising type of bridge. The author's uncle was permanent secretary to the minister of transport.

In his naivety the author never spoke to his uncle about the network arch, because he thought that everybody would support such a promising idea. The author does not know if his uncle ever spoke up for the network arch, but all of a sudden it was decided that the bridge over Bolstadstraumen might be a network arch. After his uncle retired, the author was not allowed to build any Norwegian network arches for over forty years.

The two Norwegian network arches on pages 5a to 7a were built because they were less costly than competing alternatives. Designs like the network arches on pp. 9 to 12 and fig. 97 would be even more cost efficient. If the low cost of maintenance is considered, the optimal network arch would seem even more cost efficient.

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 them at suitable sites. General conservatism is probably the main obstacle to the use of this very promising structure.

Q: How about the effect of the breaking of hangers?
A: 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 very 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 prevent the breaking of the lower chord. Collision between lorries and the superstructure is a problem whenever structural members are above the lane. It is not much more serious in optimal network arches. Zoli and Woodward 2005 come to the same conclusion after having done a thorough dynamic examination of the effects of the breaking of hangers.

For the author's dissertation in 1959 he built a model of a road bridge spanning 100 m. He did some tests removing hangers and concluded: "Only when a very high number of hangers were missing or the load was extremely big, would we get buckling in the arch and collapse of the whole structure."

The hangers of around 60 Nielsen arch bridges built in Sweden were steel rods. See p. 55. Ostenfeld 1976 page 124. They were meant to relax due to one-sided loads. Nielsen 1930, 1932 and 1936. Once the author 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.

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 due to fatigue, the author would elongate the hanger by thermostatically controlled heating in a metal casing around part of the hanger, and then weld the hanger in such a way that the original length was maintained.

Q: How do you prevent the hangers from vibrating in wind and rain, and how do you prevent them from banging against one another?
A: It is advantageous that the length of the hangers and the distance between the points where hangers cross each other are not constant. That gives many unequal periods of vibrations and more damping. In the two Norwegian network arches the hangers have been covered by slit-open plastic tubes where they cross each other. This prevents damage to the hangers if and when they bang against each other. This arrangement has worked well for over 40 years. Where the hangers cross, they could be tied together, for instance, by the kind of bands which plumbers use to hang up water tubes. This arrangement would also dampen the vibrations in the hangers.

Q: What about fatigue in hangers?
A: The Åkvik Sound Bridge was proposed for a sound between two islands that have populations of 1900 and 1600 respectively. Since the traffic between these two islands will be moderate, fatigue in the hangers is not a problem. Network arch bridges with more traffic will use more steel in the hangers. The steel weight of the hangers in the Åkvik Sound Bridge is 16 tons. This is 8% of the total steel weight. Even a very unlikely doubling of the steel weight in the hangers would not make the network arch considerably less competitive. Stephan Teich has been doing some thorough work on fatigue in hangers for his Dr.-Ing. in TU-Dresden. Teich 2004a.

Benjamin Brunn and Frank Shanack 2003 have done interesting work on fatigue in hangers and hanger arrangements that improve resistance to fatigue. Stephan Teich 2004, is doing his Dr.-Ing. in TU-Dresden. He goes deep into the field of hanger fatigue.

In his master's thesis Steimann 2002 pointed out that hangers made of flat steels were preferred by the German railway company. Graβe 2007 p.195 stresses that hangers made of flat steel plates are simpler to produce and have a higher fatigue strength. They might not look as good as round hangers.

Q: How does the author suggest putting the hangers in place?
A: The hangers are put in after the arches and the lower chord are in place. If the hangers are steel rods they would usually be put in resting in a long channel profile that is lifted by a moveable crane. During erection they would be resting on supports in the steel scaffolding. See page 51.

Q: What is the earthquake resistance of the network arch like?

A: Good, because of the high strength to weight ratio and ample reinforcement in the edge of the tie. Professor Semih S. Tezcan at Bogazici University in Istanbul adds: “Since it is very light, the network arch is ideally suitable for earthquake prone regions. There is practically no problem.”

Q: What spans do you recommend in future network arch bridges?

A: For highway bridges spans up to 300 metres are recommended. It would be very valuable soon to get experience with network arches with spans between 120 and 180 metres. For spans over 200 metres one would probably use box arches. For the Brandanger Bridge on page 94 a round tube has been suggested for a one lane span of 220 m. This is mainly because the bridge is so narrow that sideways buckling and tilting might be a problem. For railway bridges recommended spans would be smaller, maybe between 50% and 70% of the lengths of spans recommended for highway bridges.

Q: Are the network arches in danger of being subject to harmful vibrations due to wind?
A: The short answer is no. Arch bridges with vertical hangers are not known to have harmful vibrations of a whole span due to wind. Furthermore network arches are more like trusses. They are lighter and much stiffer than arch bridges with vertical hangers. Furthermore the slim lower and upper chords are aerodynamically more favourable than the usual chords in bridges with vertical hangers. Thus network arches are unlikely to have harmful vibrations.

Professor Erik Hjorth-Hansen of NTNU, Trondheim adds: If you have modest dimensions of arches and tie, there will be no dangerous organised vortex wakes and I see few and small dangers. On vibration of a network arch with a span of 220 m see page 94.

The author has been writing about this question in Tveit 1966 and Tveit 1992 and in other publications. He hopes that somebody who has been working on the vibrations of suspension bridges soon will examine vibrations in network arches. Yoshikawa, 1993, see page 18, found that the aerodynamic vibrations of the Shinhamadera Bridge were not dangerous. Vibrations in the bridge in the Brandanger Bridge, p. 94, will now be thoroughly examined because it will be built in 2008.

Q: Could a box profile be used for the arch instead of the universal column?
A: Yes, one would have to do so if the axial force in the arch is too big for the universal column or an American wide flange beam. See fig. 30. The hollow welded arch does cost more than arches made from universal columns. If a hollow arch were used, the joints between the arch and the windbracing (fig. 16) and the joints between the arch and the hangers (fig. 17) would also be more costly. Furthermore a box would be less slender and have a less favourable stiffness distribution in the horizontal direction. See also next page.

Q: Where does the optimal network arch compete best?

A: The network arch probably competes best for spans between 100 and 150 m. In railway bridges slightly shorter spans might be very competitive. The vertical reactions and the low weight are an advantage where soil conditions are difficult. With a high strength to weight ratio the optimal network arch is suitable for seismic regions. Because the network arch is related to trusses it is a very stiff structure. The partial longitudinal prestress in the lower cord adds to the stiffness. That is an advantage where small deflections are important, for instance in railway bridges.

The optimal network arch has advantages in flat terrain where there is little room for members under the lane. The slim tie leads to shorter ramps. This is more important in rail than in road bridges because the slope of rails is smaller than the slope of roads. This might be a reason why railway bridges are more likely to have structural members above the lane.

The network arch is advantageous over navigable water where cranes or pontoons are available for lifting the steel skeleton of the network arch in place. This is especially so in coastal areas where big floating cranes are available. See page 94.

Since the network arc uses little steel, it is very competitive where the price of man-hours is small compared to the price of steel. This only applies when suitable manpower with a sufficient technological know-how is available. If the bridge is a through arch with navigable water underneath, then the part that is above the lane can be a network arch. The steel skeleton for this part can be lifted in place from pontoons. Pages 15, 50a, 54 and 94.

Here comes a question that the author posed during his lecture tour in year 2000.

Q: When the author visited Bundesanstalt für Straßenwesen in Bergisch Gladbach near Köln, he asked if they were likely to accept the railing that is shown in fig. 34 outside the hangers of a network arch. The railing can also be seen in figs 6 and 6a of the network arch bridge at Steinkjer. Tveit 1964.

To give the pedestrians a feeling of safety since they were near to the edge of the bridge, the top of the railing is 130 mm wide. Where the railings are outside the main span, vehicles can not hit them. The railings are without expansion joints in the main span.

The posts of the railing are welded to U120-profiles cast into the outer edges of the footpath. The welding was done in small steps to avoid too much heating of the concrete.

To avoid harmful stresses due to temperature differences, bending and creep in the lower chord, the grid outside the footpath is fastened to the handrail, but not to the vertical posts. If a shortened tie gave much horizontal deflection in the posts, the handrail could also be shortened. There has been no shortening in the tie after the concrete was half a year old.

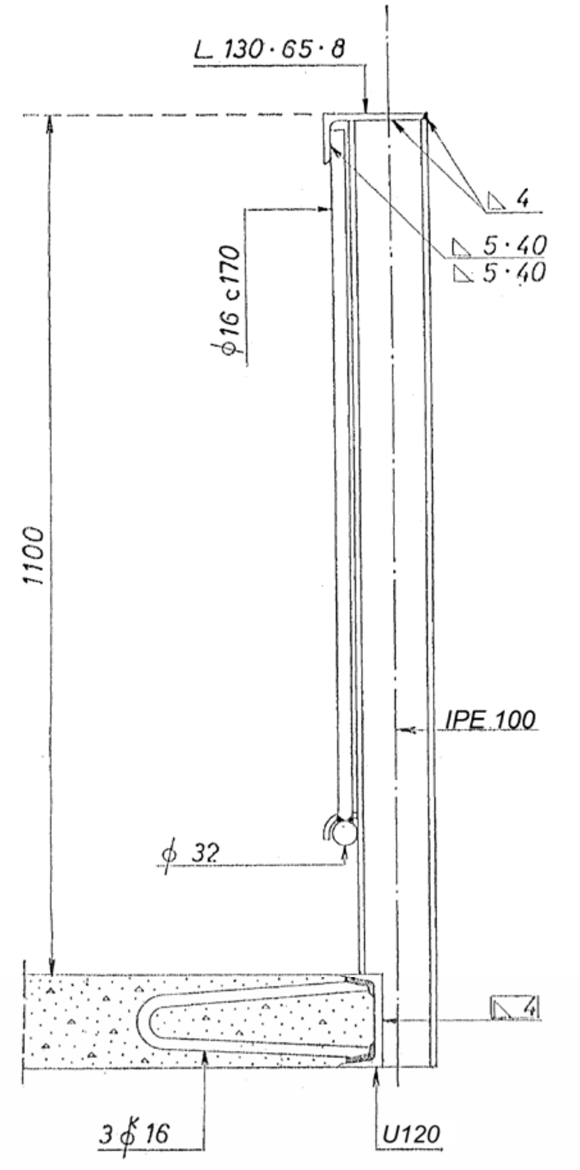


Fig. 34. Cross-section of the railings of the bridge in Steinkjer. Figs 5 to 6a.

A: It is unlikely that such a type of railing would be accepted. - This did not come as a shock to the author because the Norwegian bridge authorities have said the same. Since the railing of the Steinkjer Bridge has worked well since 1963, the author would like to suggest a safe way of testing the Steinkjer type of railing.

If a railing of the Steinkjer type does not work, it could be replaced by the concrete elements to which a usual railing is often fastened. The design would have to have extra space in the footpath for the usual concrete elements for fastening the standard railings and possibly extra strength to carry the weight of a heavier railing.

In 2003 a student from TU-Dresden in Germany, Frank Schanack, was doing his graduation thesis in Grimstad. He told the author that he had been working on a bridge for a motor road where the outer railing was fastened to screws that were jutting out of the concrete slab under the footpath. There was an inner railing between the traffic and the footpath so that the traffic could hit the outer railing. In the main span of the Steinkjer Bridge the hangers are between the traffic and the footpath. This indicates that the railing in fig. 34 might sometimes be accepted.

COMPARISON BETWEEN UNIVERSAL COLUMNS, TUBES AND BOX SECTIONS

Fig. 33a shows two box sections with the same areas as the universal column that could be used in the arch of the bridge like in fig. 97. The box section in the middle has b/t=31.25. If it had been used in the bridge in fig. 97 the bridge would not have been so slender. The lower end of the box could be filled with concrete to make it more resistant to shocks from vehicles.

The box to the right in fig. 33a has the same diagonal distance between the outer corners as the universal column that could be used in the arch of a bridge like in fig. 97. If had been used in the arch of a network arch, it would have looked nearly as slender as the arch in fig. 97. Fastening arch and hangers would have been less straightforward. The universal column has a bigger sideways stiffness. All this speaks for using universal columns and American wide flange beams in the arches of network arches. The higher wind force on the universal column speaks for using tubes in this very slender bridge.

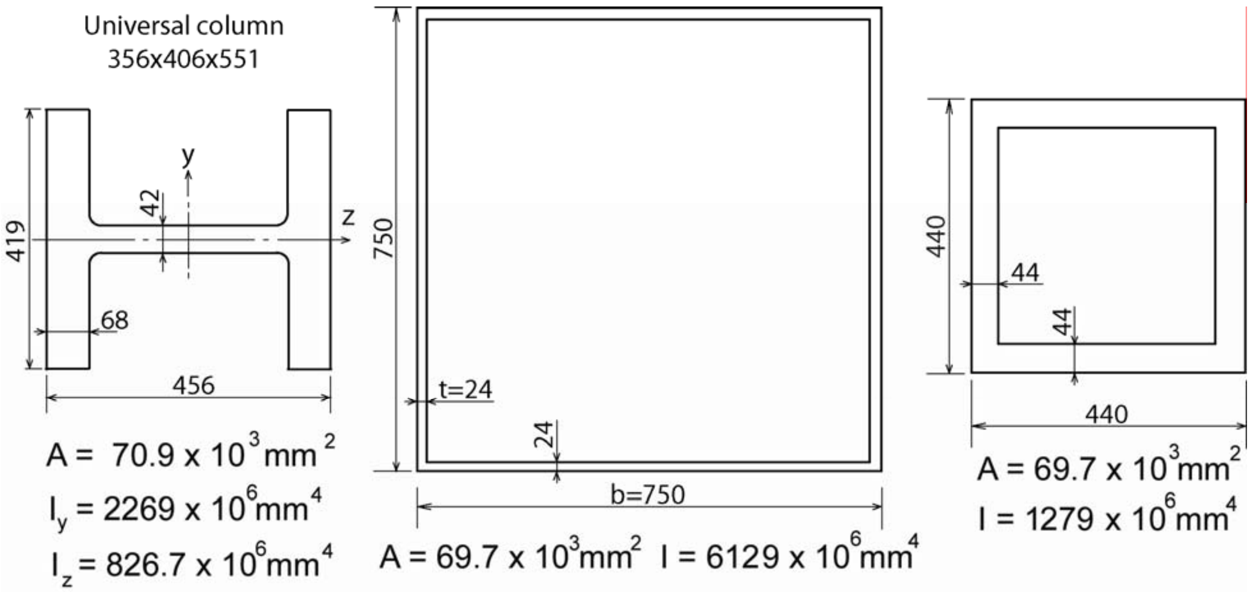


Fig. 33a. A universal column compared to two box sections with the same area

Fig. 34b shows two tubes with the same area as the universal column that could be used in the arch of a bridge like in fig. 97. Also the tube in the middle has a cross-section of class 1. The whole cross-section can have quite a lot of yield before local buckling occurs. The tube to the right in fig. 34b has the same diameter as the diagonal of the universal column to the left.

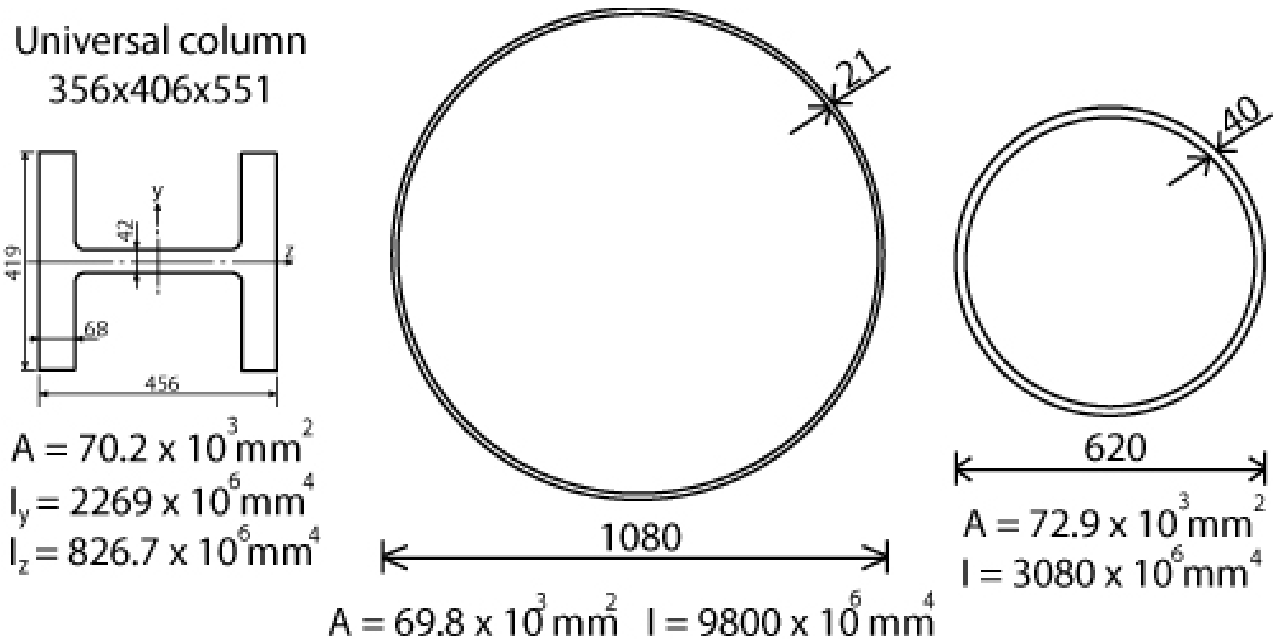


Fig. 33b. A universal column compared to tubes with nearly the same area

WHERE MIGHT A NETWORK ARCH WITH A CONCRETE TIE BE AN OPTIMAL SOLUTION ?

The slender network arches are beautiful and cost efficient bridges. They are especially cost efficient when the distance between the arches is less than 15 m. Then no transverse beams are needed in the tie. Longitudinal steel beams are not needed because the longitudinal bending in the tie is smaller than the transverse bending in the middle of the concrete slab between the arches.

When there are no transverse beams in the tie the tensile force is best taken by prestressing cables in edge beams under the arches. They give a beneficial compression in the concrete in the tie, especially when there is no load on the bridge. Longitudinal steel beams in the tie would attract compression. That would give higher stresses in the beams and increase the need for minimum longitudinal reinforcement in the tie.

Fig. 97 p. 93 shows that little steel is needed in the Åkviksound network arch compared to German arch bridges with vertical hangers. The reduction of the steel weight is much smaller when arch bridges with vertical hangers are replaced with network arches with steel ties. See the Flora Bridge in fig. 40i on page 35. Page 93a and 93b indicate that sometimes 35 to 45 % of the cost can be saved by using network arches with concrete ties.

The possibilities of erection play an important role in the choice of bridge type. It is relatively straightforward to use network arches when they can be built on a temporary scaffold. See p. 7a. But then short spans might be competitive. In Peru the current can move the sand away from around pillars in some rivers. There network arches can be a good solution in spite of the relatively long spans.

If temporary ties are used, they form a stiff skeleton with the arch and hangers. See pages 12 and 29k to 30a. The skeleton for small and medium spans can be lifted in place by mobile cranes. (Bechyne p. 92c). For bigger spans pontoons or barges can be used (p. 15 and p. 20).

In some coastal areas complete spans over 200 m can be finished on a quay and lifted in place by big cranes. See Brandanger Bridge p. 94. Cranes with a lifting capacity over 2000 tons are available. They are mainly used for oil exploration. The main span of the Brandanger Bridge is 220 m long. It has only one lane. The wind speed of up to 50 m/sec is an important factor in the design. We can assume that in bridges with two lanes the wind will seldom give severe problems for spans of up to 200 m.

When there is a small distance between the traffic over the bridge and the necessary room for the river below, a short network arch might be an alternative. That was the case for the network arch at Bechyne in the Czech Republic. See pp. 92a to 92c. It has a span of 42 m. The thin lower chord makes it easy to attach existing roads to the bridge.

Generally it can be said that network arches are an alternative when long spans are needed. That is the case in deep rivers or straits, especially when pillars will be relatively costly. Network arches can take relatively big settlement of the foundations due to weak soil. In rivers where thick ice can occur, the steel skeleton can be erected on ice and be lifted on to the pillars. See page 30b. Network arches are insensitive to seismic activities because the deadweight is small compared to the load carrying capacity. See p. 23.

On big roads there are often two lanes plus a stopping lane in each direction. The two directions are often metres apart. In such roads two parallel spans can carry the road over obstacles. The two parallel spans would be lighter and easier to erect than one. If universal columns were used the very slender arches would look good. The same temporary lower chord could be used for both spans.

In many sites much more traffic can be expected in the future. Then it might be economical to build one span sufficient for carrying the present traffic. Two parallel spans should be planned, and the second span should be added when the first span is congested.

Whether network arches are cost effective depends on the site and to some extent on the equipment and the technical level of the work force available. A certain technical level is needed for the design and building of network arches. Good computer skills are a necessity. A dedicated and trained work force is needed to obtain the good quality control necessary. It seems to the author that the network arch with a concrete tie <u>might</u> be the optimal solution for bridge spans between 40 m and up to and over 300 m.

MORE ON OPTIMAL ARRANGEMENT OF HANGERS ETC.

After the second page this chapter deals with the relaxation of hangers. Most of it was written in cooperation with the American student Gene O. Day at the University of Houston in 1978. The author hopes that this chapter will make it easier to find hanger arrangements that lead to near optimal network arches. Those who want a quick start at calculating a network arch can go to the articles on “preliminary design of a network arch road bridges” on the author’s home page http://pchome.grm.hia.no/~ptveit/

Before going deep into explanations about hanger arrangements, the author would like to sum up the characteristics of an optimal network arch:

1. Aesthetic appearance.
2. Small bending in the chords.
3. Equal cross-section and near maximum utilisation of tensile capacity in all hangers.
4. Little bending caused by long distances between nodes in the chords.
5. A distance between hangers that suits a light temporary lower chord for erection.
6. Hangers with slopes not causing too big bending moments in the chords due to relaxation of hangers.
7. No steel beams in the lower chord.

Since the network arch is the slimmest arch bridge possible, it is likely to look nice. The network arch normally looks best if the rise of the arch is not over 0.15 times the length of the span. Steel beams in the tie would make the tie less slim and thus detract from the good looks. Steel beams are likely to cost more than the simple concrete slab tie. The author does not think much of steel beams in the tie..

When there is less than 15 to18 metres distance between the arches, the tie should be a concrete slab with longitudinal partial prestress. The transverse bending in the slab is usually much bigger than the longitudinal bending. Thus the main purpose of the edge beam is to accommodate the hanger forces and the longitudinal prestressing cables.

For load cases that relax none or only very few hangers, network arches act very much like many trusses on top of one another. They have little bending in the tie and in the arches. How many hangers that could be allowed to relax depends on the stiffness and bending capacity of the chords. It is the extra bending capacity that can carry the bending moments that come when an increasing number of hangers relax. To avoid extensive relaxation of hangers, the hangers should not be inclined too steeply. Slight inclination of hangers will normally increase the bending moments due to concentrated loads. See fig. 26d.

Because there is little tendency for buckling in the arch, and tension is predominant in the rest of the network arch, the network arch makes good use of high strength steel. The network arch where the tie is a concrete slab usually saves much more than half the steel weight compared to conventional bridges. See page 93.

The network arches described on pages 5b to 12 and pages 59 to 93 are very competitive. The details are simple and the exposed surface is small. The steel weight is low, but not minimal. The arch and hangers supplemented by a light temporary lower chord can be moved. This steel skeleton can be erected on shore, on side-spans or on the ice between the abutments. When it is in place, this steel skeleton has enough strength and stiffness to support the casting of the concrete tie. In coastal regions network arches can be completely finished on shore and be lifted to the pillars by means of big floating cranes.

On pages 93 to 93b there is a chapter with a comparison between network arches and arch bridges with vertical hangers. Network arches spanning 150 m are compared to an arch bridge with spans of 100 m. In the network arch the cost per m² of bridge is between 35 % and 45 % less than in the arch bridge with vertical hangers.

For wide bridges, three or four parallel arches could be used to keep down the span of the concrete slab between the arches. See figs 32 and 33. For long bridges, where many spans are needed, the network arches could be made exclusively from prestressed high strength concrete. The spans could be made on shore and could be floated to the site on pontoons. See figs 50 and 51.

Stephan Teich has done some very interesting work on optimal hanger arrangement for his doctoral thesis. I will ask for his co-operation for the next edition of this chapter.

If only one hanger in a network arch relaxes, it has little influence on the bending in the chords. The span still works primarily like a truss, and the increase in bending moments is small. After part of the span starts to act like part of an arch with only one set of hangers, increasing partial live load can make stresses increase faster than the same increase in live load on the whole span. This load case is especially dangerous when the chords are very slender. It will normally take a substantial increase in partial live load after a hanger has relaxed before the partial live load becomes as severe as the same live load on the whole span. See pages 67 to 68.

The hanger arrangements in the two network arches on fig. 8 and on page 59 to 72 might be a help in finding optimal hanger arrangements. The ratio of live load to dead load decides which network arch is the best paradigm. Look also at the network arch on pages 73 to 93. Some very interesting work has been done in Brunn and Schanack 2003. Their work can be found at http://fag.grm.hia.no/fagstoff/ptveit/

In the wind portal Brunn and Schanack use a reduced curvature of the arch. Here the radius of curvature is 80% of the curvature in the rest of the arch. This leads to shorter wind portals with smaller bending moments and more even normal force in the arch. Brunn and Schanack's "Optimisation of the hanger arrangement" pp. 30 to 73 leads to recommendations in fig. 6.61 p. 69. Schanack plans to do his Ph.D on the subject. The author is looking forward to that, but here the author will explain his old ideas.

Hangers placed equidistantly along the arch give the smallest bending due to local curvature when the span is fully loaded. Two hangers at each nodal point would give more bending in the arch due to local curvature and less efficient support of the arch in buckling.

When hangers relax due to partial loading, it makes little difference whether hangers meet at the nodal points in the arch or not. When the nodal points are evenly distributed along the arch, the distance between some of the points of support doubles where some hangers relax. Then bending due to local curvature increases and the support of the arch becomes weaker. However these load cases do not necessarily give maximum stresses, because load cases that make hangers relax, give smaller maximum axial force in the arch. Because it saves design work and for many other reasons, it seems best that the load on the whole span decides the dimensions of every point in the arch.

The optimal number of hangers is a central question in the design of network arches. Many hangers will increase the amount of man-hours in design, workshop and erection. This increase will, however, be moderate since all hangers can have the same cross-section and the hanger details are all alike. Many repetitions tend to keep the labour costs down. Many hangers also give lighter hangers and lighter equipment for their erection. If universal columns and/or American wide flange beams are used instead of box cross-sections in the arches, more hangers are needed, but the welding in the arches is reduced.

More hangers could also lead to a lighter temporary lower chord for erecting the span. The edge beam is usually cast after the ends of the tie are cast and before the concrete slab is cast. This makes it easier to keep a constant distance between the transverse beams in the lower chord in spite of the uneven distance between the nodes in the tie.

Many hangers make it easier to replace defective hangers without interruption of traffic. This is because each hanger is lighter and the temporary removal of a hanger causes less extra stress. If there are many hangers, there is less chance that the breaking of one or more hangers caused by a vehicle will have catastrophic effects. See Zoli and Woodward 2005. The static effect of breaking of hangers is mentioned in Tveit 66. Fig. 96 deals with the breaking of a hanger in the Norwegian version of the Åkvik Sound network arch.

Adjacent hangers at the deck are well spaced at the arch. Thus the network arch is probably less sensitive to the breaking of hangers than the usual bowstring arch. The author is not aware that breaking of hangers is an important cause of accidents in bowstring arches. The hangers nearest to the ends of the arch tend to have smaller maximum forces than other hangers. This can be counteracted by increasing the distance between the hangers at the end of the span. When a temporary tie is used the first hanger in the arch should normally be sloping away from the end of the span as shown in figs 8, 12, 23, 39a, 40e, 41 and 96.

In the middle half of the span the nodes could be placed equidistantly along the lower chord. See fig. 8 and pages 59 to 93. In the rest of the lower chord hanger distances should be varied in order to obtain nearly the same maximum force in all hangers. It is an advantage to avoid long distances between nodal points because these tend to lead to the biggest bending moments in the tie. See influence lines in fig. 65. Since the longitudinal bending moments in the tie are small, this might not be a primary concern.

It deserves to be emphasised again that the bending moments due to concentrated loads can normally be reduced by making the hangers steeper. Too steep hangers lead to too much relaxation of hangers. See also page 16. For this reason a fair amount of this section will be devoted to the relaxation of hangers and a reasonable choice of slope of hangers.

The network arch where hangers relax is a discontinuous system, where each combination of relaxed hangers leads to new equations for calculating forces and deflections. This complicates calculations because influence lines can not be used. However, modern computer programs can easily calculate the effects of hangers relaxing.

Back in 1962 the influence lines for the two Norwegian network arches were calculated by means of the Sara digital computer of the Swedish Aircraft Corporatiuon (SAAB). Relaxation of hangers can be accounted for by removing hangers in compression and recalculating, until all hangers that were not in tension had been removed. When these bridges were calculated computer facilities were not available for recalculation.

For this reason it was necessary to avoid relaxation of any hangers. Thus a need was felt for an easy method for predicting the relaxation of hangers. The method might still be of interest, but casual readers should skip the rest of the chapter except for the last two pages. Others might come back to this chapter after having read most of this publication.

It can be seen that the influence lines for hangers on page 57 and 58 have nearly the same shape as influence lines for diagonals in trusses. If the network arch consists of three trusses on top of one another as in fig. 4b, then the ordinates of the influence lines can be expected to be approximately one third of what they are in a simple truss. This, however, does not influence the relative size of the positive and negative areas of the influence lines. Thus simple truss models could be used in order to decide approximately what loads will relax a hanger in a network arch.

The reasoning leading to the diagram in fig. 35 was slightly different. See fig. 36 next page.

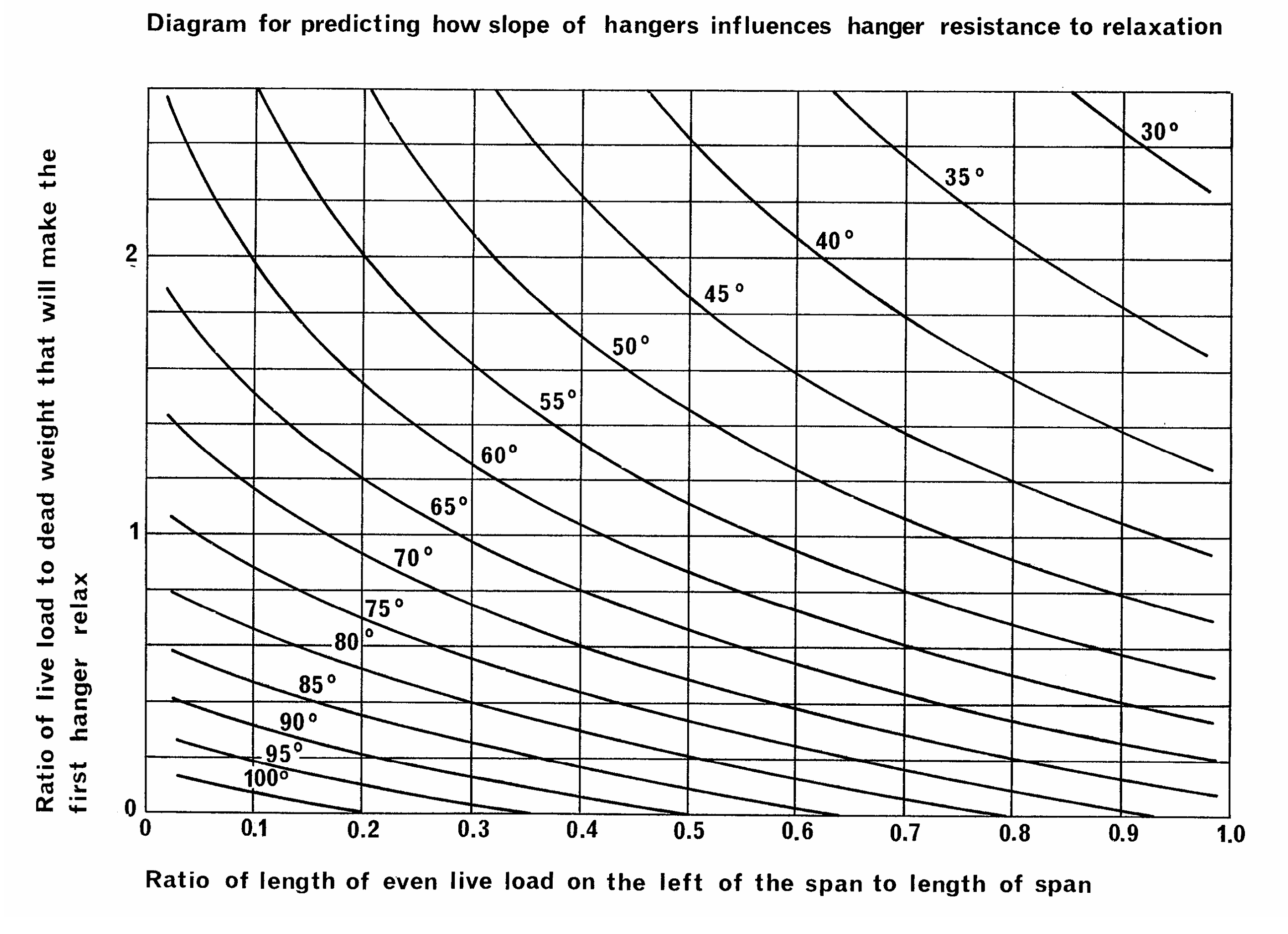


Fig. 35. Connection between slope of hangers and relaxation of the first hanger for a span with a rise/length ratio of 0.15

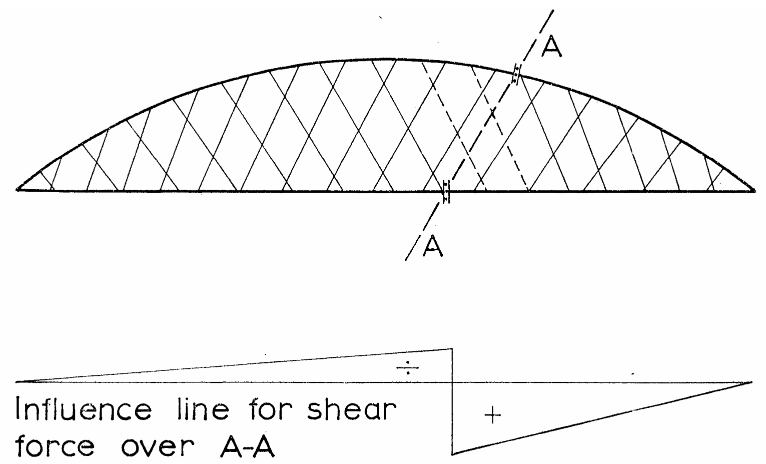


Fig. 36. Network arch with hypothetical releases for the calculation of hangers' resistance to relaxing

The line A-A in fig. 36 is parallel to adjacent hangers. The relaxation of hangers leads to considerable increase in bending moments only after all hangers across the line A-A have become relaxed. Thus it is of interest to calculate what loads will relax all hangers across a line A-A in fig. 36. Tveit 66.

For the dotted hangers in fig. 36, this can be done by finding the sign of the shearing force across in A-A. These joints can take bending, but no shearing force. Assuming no bending in the chords, the hangers will not be relaxed if the shearing force at A-A is positive. The dotted hangers' resistance against becoming relaxed will depend on the slope of A-A, i.e. the slope of the hangers adjacent to A-A.

The hangers' actual resistance against becoming relaxed is slightly larger than found by this method. This is mainly due to the presence of shear and bending in the chords. The above method has been used to prepare fig. 35 and figs 38 to 38f. These graphs indicate the slopes that will enable hangers to resist certain ratios of live to dead load.

The spans are assumed to be loaded from the left end, as this is the type of loading that is most likely to make some hangers relax. It has been found that for various reasons, including shear and bending moments in the chords, fig. 35 and figs 38 to 38f give a reasonable prediction of the relaxation of the first hanger. The prediction is best for long bridges with slender chords.

The design and use of the diagrams in figs. 35 and fig 38 to 38f are explained at length later in this chapter. The diagrams look alike, but the rise of the arch differs. In fig. 37 relaxed hangers are dotted. The bridge has begun to act as a part of an arch with one set of hangers along the line A-A. As mentioned before in this chapter, the bending moments will now start to increase faster because the nodes in the arch and tie are less firmly kept in place by the hangers.

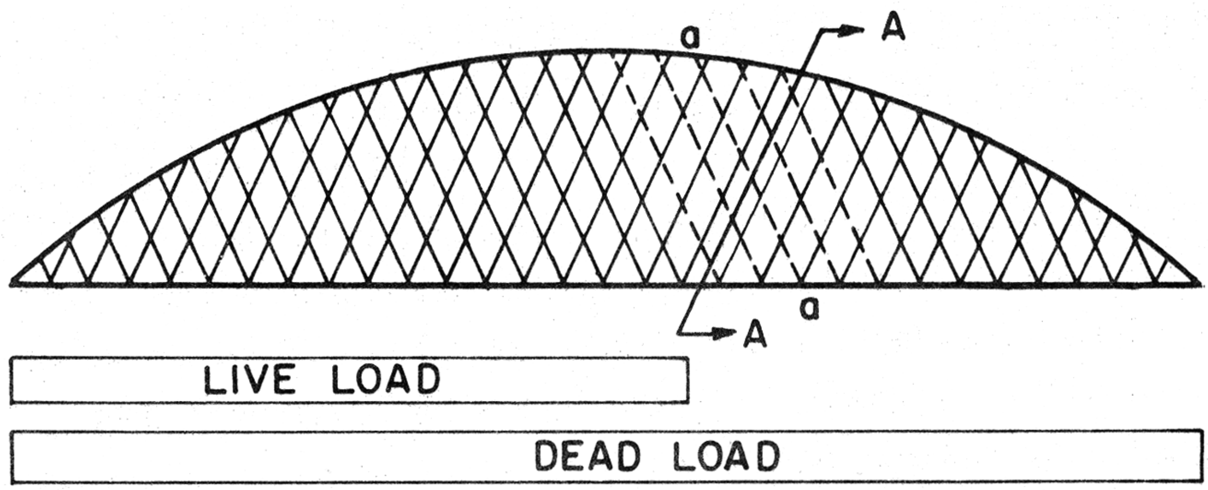


Fig. 37. Network arch with live load from one end of the span

Let us look at the stresses in the chords of the railway bridge in fig. 44 and fig. 44a. In fig. 44a there is no live load on the 28.05 m on the left of the 65 m span. Four hangers relax. Compared to the full live load on the bridge in fig. 44 the biggest increase in bending in the arch is in node 21 where the bending moment increases from ~35 tm to ~75 tm. At the same time the maximal axial force decreases from 1467 t to 1040 t. Maximum stress due to these forces will be:

For full load. (Fig. 44):

σ=N/F+M/W_{xu}=1440/690.6 + 3500/9059 = 2.47 t/cm^2

With no live load on the leftmost 28.05 m of the span. (Fig. 44a):

σ= N/F+M/W_{xu}=1040/690.6 + 7500/9059 = 2.33 t/cm^2

Excuse the old-fashioned terms. "t" means metric tons everywhere in this publication. The results come from of linear calculations. Hangers that had no tension were removed by hand before recalculation. The cross-sectional data were taken from fig. 42 . It can be seen that the reduction in the axial force more than made up for the increase in bending. This is a usual result.

How to predict when hangers will become relaxed?

When dead load and live load are fixed, the tendency for relaxation is primarily decided by the slope of the hangers and the form of the arch. The stiffness of the chords and the cross-sections of the hangers also influence the result, but to a lesser extent. Stiff chords and small cross-sectional area of the hangers reduce the tendency for relaxation.

After hangers have started to relax, the bending moments increase more quickly with increasing partial live load. When one set of hangers relax, the other set of hangers has to carry the entire load. Thus the maximum tension in hangers usually becomes smaller when the hangers' resistance against relaxation is increased.

This reasoning applies less well when the slope of the hangers is less than 60˚ because the loads on the tie are carried by the vertical component of the hanger force. The vertical components of the hanger forces decrease more quickly when the slope is less than 60˚.

Let us now turn our attention to computing the loads that will make a hanger relax. To calculate the ratios of live load to dead load which will make a hanger relax, it is necessary to compute the influence lines for the hanger by some computer program. First the relaxed hangers must be removed. Once the influence line of a hanger is obtained, it is easy to compute the ratio of live load to dead load that will give zero tension in the hanger.

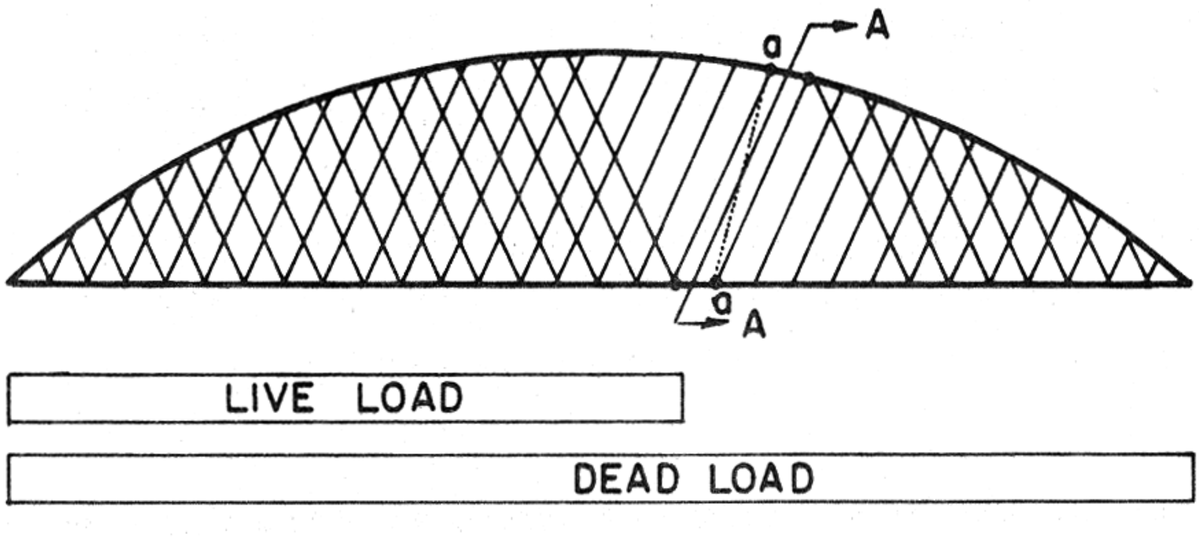


Fig. 37a. Bridge with live load on the left hand side of the span. Relaxed hangers are not shown.

Once the influence line is obtained, the ratio of live load to dead load which will give zero tension in a hanger can be computed as follows.

$-A_1(LL+DL) + A_2 \cdot DL = 0$ (e1) First equation

$LL/DL = A_2/A_1 - 1$ (e2)

Here LL is evenly distributed live load
DL is evenly distributed dead load
A_1 is the area under the negative part of the influence line
A_2 is the area under the positive part of the influence line

The ratio LL/DL may be approximated without having to compute the influence lines. If the bending moments and shear forces in the chords are disregarded, it is easy to find the ratio of live load to dead load that makes the hangers across the line A-A in fig. 37a relax.

This can be seen from drawing the influence line for the imaginary member a-a in fig. 37a. This is done in fig. 37b. See next page. Movements of the non-deformable parts I and IV that shorten and relax a-a will also shorten and relax the other hangers across A-A. The relative deformations of the parts I to IV can be found by a virtual displacement. This is done by a method thaught to undergraduates in the technical university in Trondheim in the middle of the last century. The method seems to have vanished.

The influence lines are drawn without stating the size of the ordinates. The factor K in the next equation can be determined by calculating one ordinate. However, calculation of this ordinate is not necessary in order to find the load that relaxes a-a.

Relaxation of the imaginary hanger a-a in fig. 37a occurs when the ratio of live load (LL) to dead load (DL) is given by the equation:
$LL/DL = (KA_2 - KA_1)/KA_1 = A_2/A_1 - 1$ (e3)

The assumption that bending moments and shear in the chords are equal to zero corresponds to assuming joints as shown in fig. 37a and 37b.

The result is more generally applicable if we assume that the distance between the hinges, λ, is very small compared to the length of the bridge. It is easily seen that the exactness in the prediction of increase of bending moments is not much influenced by this distance. Calculations become much simpler when the distance between the hinges is considered small.

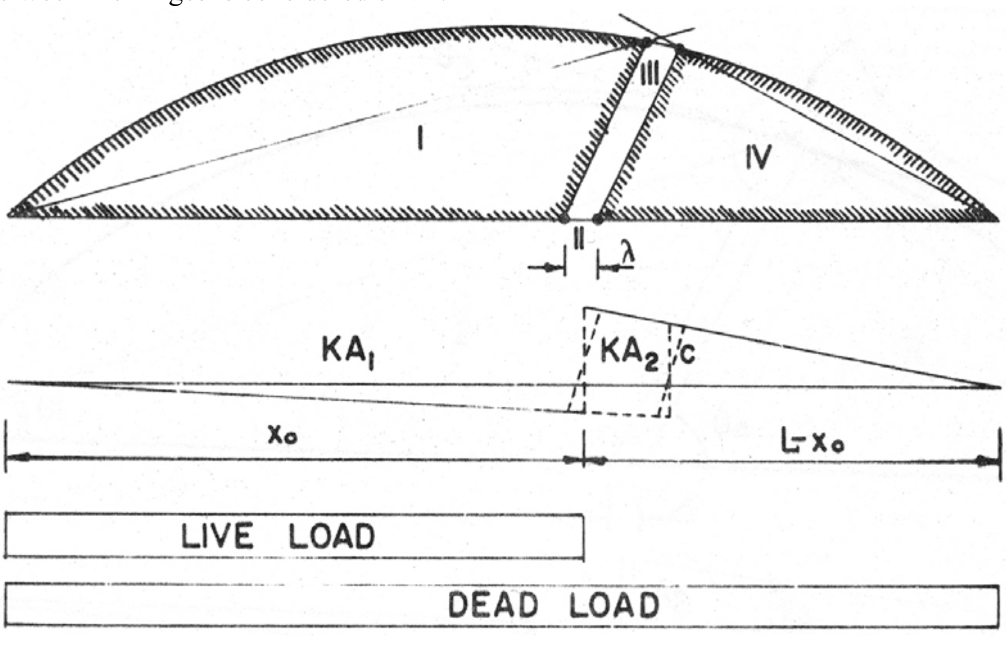


Fig. 37b. shows the parts of the bridge that will move as undeformable bodies when the influence line for a-a in fig. 38a is found by the kinematic method

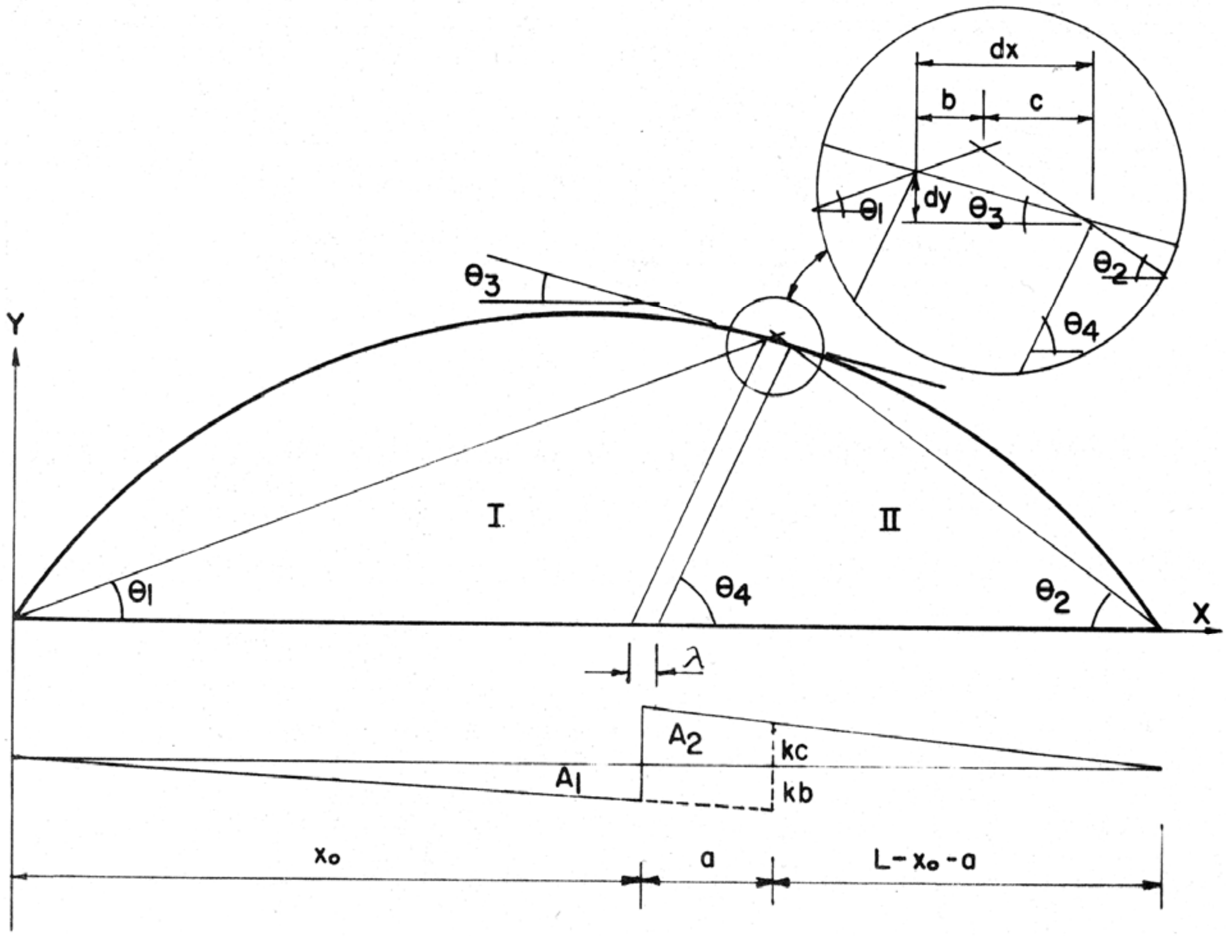


Fig. 37c is used for calculating hangers' resistance against relaxing

Calculations are made with symbols in fig 37c. Note that tanθ2, tanθ3 and dy are negative when they refere to lines that slope down to the right.

Figure 37c: is used for calculating hangers' resistance towards relaxing.

We have: $b + c = dx$ (e.4)

$\tan\theta_3 = dy/dx$ (e.5)

$b \times \tan\theta_1 + c \times \tan\theta_2 = dy$ (e.6)

$$\frac{A_2}{A_1} = \frac{c}{b} \frac{\frac{(L - X_0)^2}{L - X_0 - a}}{\frac{x_0^2}{x_0 + a}} \qquad \text{(e.7)}$$

The symbols used in (e.4) to (e.7) are defined in figures 37b and 37c.

Combining (e.4) and (e.7) and inserting them in (e.3) we get:

$$\frac{LL}{DL} = \frac{(\tan\theta_1 - \tan\theta_3)(x_0 + a)(L - x_0)^2}{(\tan\theta_3 - \tan\theta_2)(L - x_0 - a)(x_0)^2} \qquad \text{(e.8)}$$

Figure 37d shows the results of measurements on the model in Tveit 59. The disregarding of shear forces and bending moments leads to about 20% under-estimation of the live load that would cause all hangers crossing line A-A in fig. 37 to 37a to be relaxed.

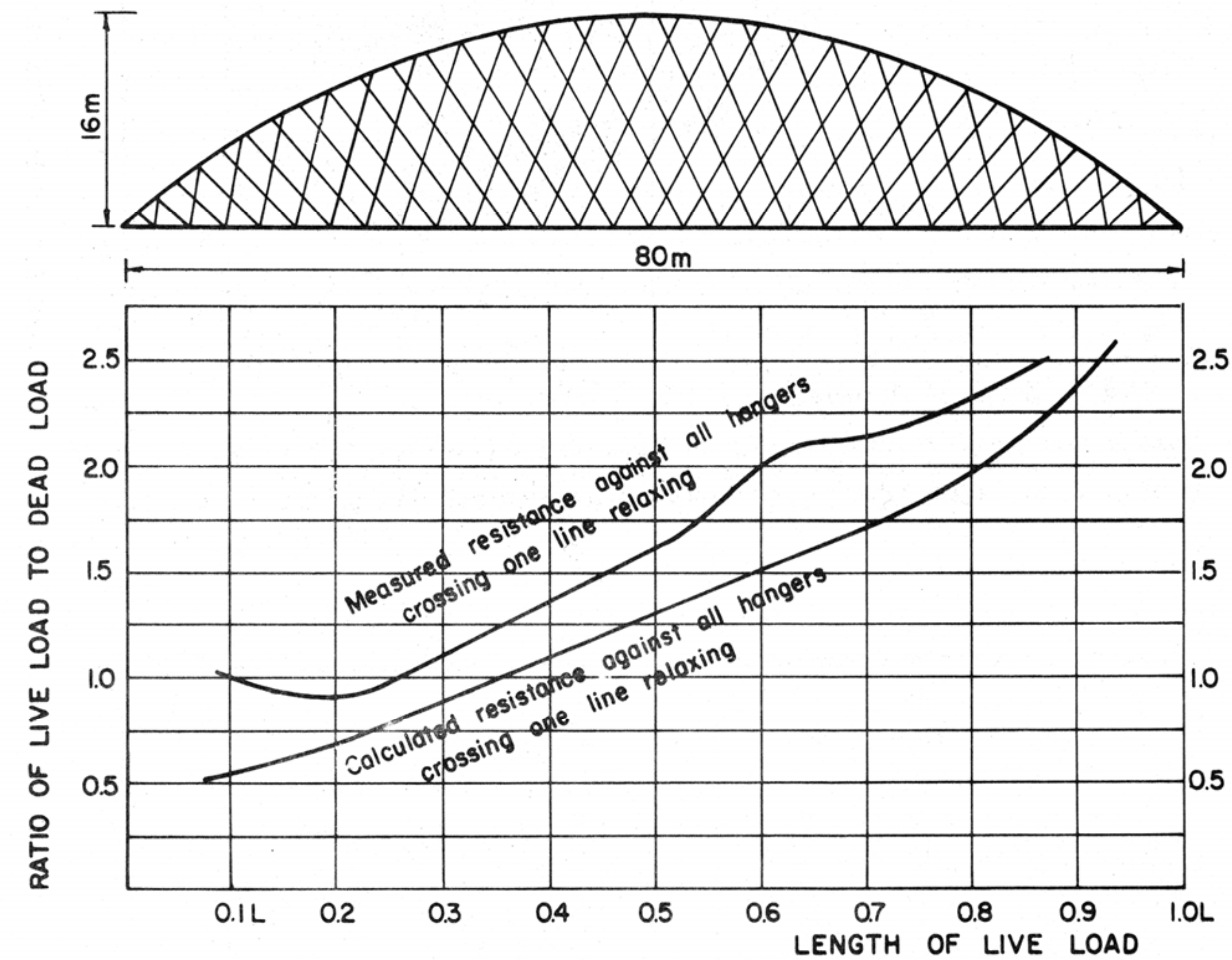


Fig. 37d illustrates the discrepancy between calculated and measured resistance against relaxation of all hangers crossing one line

As can be seen from fig. 37d the deviation of approximately 20% between calculated and observed live load that relaxes all hangers crossing a line, was observed in the middle 80% of the span. This deviation occurs because we have some bending moments and shear forces in the vicinity of A-A in figs 37 to 37a. Thus, the calculated theoretical value is a good enough indication as to what live load makes moments in chords start to increase due to relaxation of hangers.

Formula (K.8) has been used for calculating LL/DL values for hanger relaxation in 37d. It has also been used for calculating the diagrams in figs 38a to 38f. Some of these diagrams were first published in Tveit 66.

To sum up: The diagrams in figs 38a to 38f have been calculated to show what ratio of partial live load to dead load, (LL/DL), makes hangers relax in a section right in front of the live load, under the assumption that there is no bending moment or shear force in the chords.

Because of bending and shear that occur in the chords of the network arch, the diagrams in fig. 38 to 38f. underestimate the magnitude of the partial live load, that is necessary to make all the hangers in a section (like A-A in fig. 37a), in front of the partial live load, relax.

Experience so far shows that the diagrams in fig. 38a to 38f underestimate even the partial live load that will make the first hanger in front of the partial live load relax. Therefore, these diagrams can be used for finding ratios of partial live load to dead load that can be used without danger of having hangers relax. Fig. 37e illustrates how this is done.

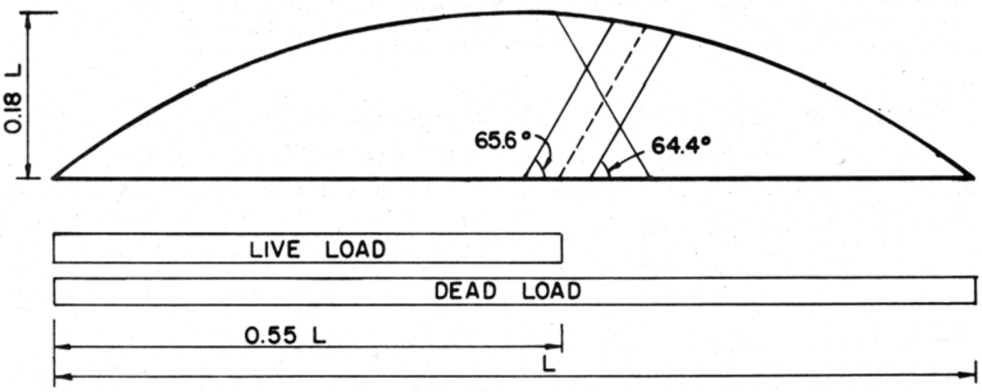


Figure 37e. Chords and some hangers in the Bolstadstraumen Bridge. Tveit 66.

In fig. 37e most of the hangers of the Bolstadstraumen Bridge are omitted. By means of fig. 38f the resistance against relaxation of the hanger sloping down to the right can be found. See dotted line in fig. 38f. The dotted line indicates that a partial live load of 0.7 times the dead load covering 0.55 x L from the left of the span will not relax the hanger sloping down to the right on the drawing. In fact no hanger crossing the dotted line on the drawing will relax as long as the partial live load is not greater than 0.7 times the dead load.

Figs 38 to 38f can be used to find hangers' resistance in hangers that are not too near to the ends of the arches. Usually the resistance against relaxation in the hangers that are near to the ends of network arches is no problem. See fig. 38g.

Figure 37e corresponds to the dotted line in figure 38f. For the network arches built in Norway the diagrams in fig. 35 and fig. 38f consistently give values of live load smaller than those necessary to make the first hangers relax. For the bridge at Steinkjer the diagram in 35 gives loads that were 15% or more too small. For the bridge at Bolstadstraumen the loads found by means of diagram 38f were 5% or more too small to make hangers relax. The bridge at Steinkjer had a stiffer lane, and that is probably the reason for the estimated loads here being more on the safe side.

How much concentrated loads influence hangers' tendency to become relaxed can best be judged by looking at influence lines of bridges that have already been calculated. See figs 63, 64, 65 77 and 84.

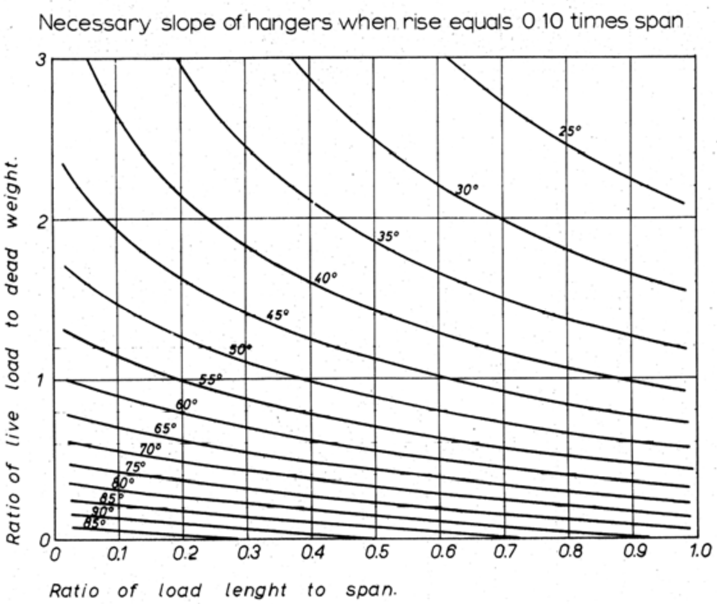


Fig. 38: Slope of hangers necessary for preventing the relaxation of hangers when bridges with f/L=0.1 carry uniform load from the left

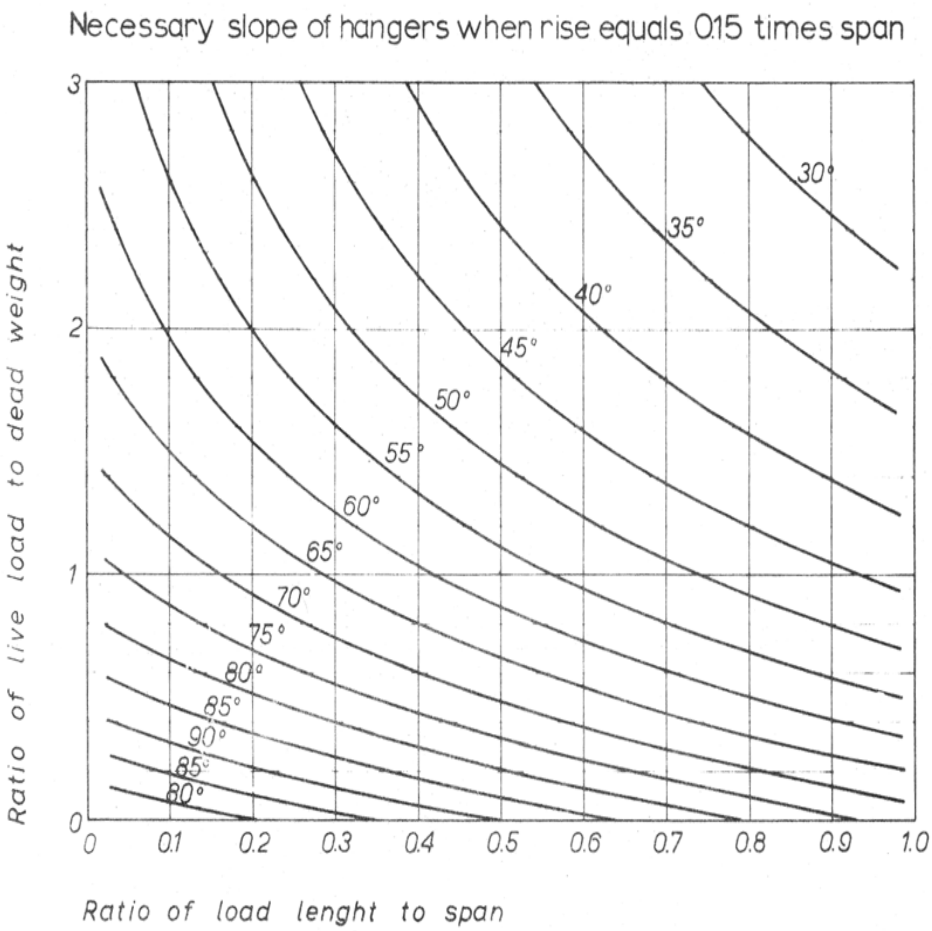


Fig. 38a. Slope of hangers necessary for preventing the relaxation of hangers when bridges with f/L= 0.12 carry uniform load from the left.

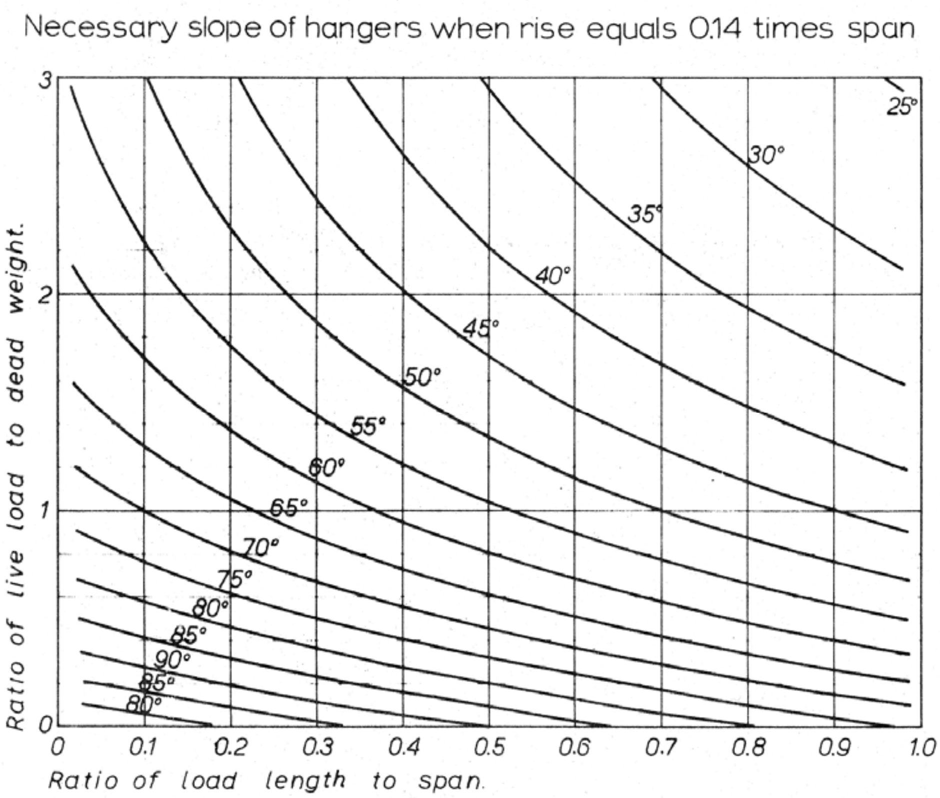


Fig. 38b. Slope of hangers necessary for preventing the relaxation of hangers when bridges with f/L=0.14 carry uniform load from the left

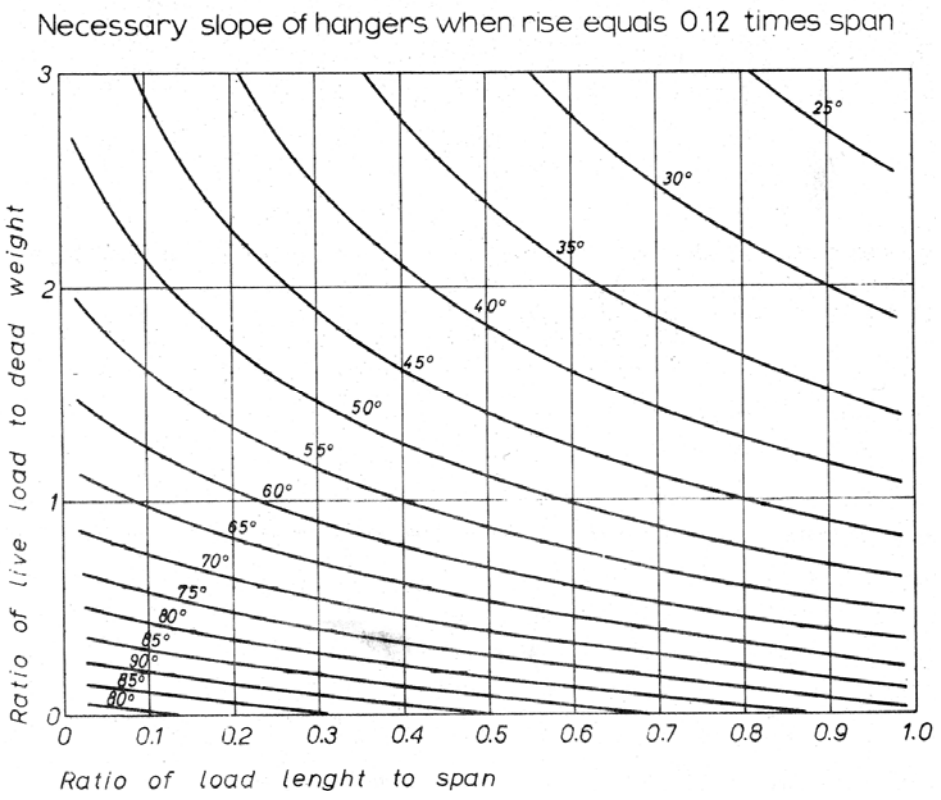


Fig. 38c. Slope of hangers necessary for preventing the relaxation of hangers when bridges with f/L=0.15 carry uniform load from the left

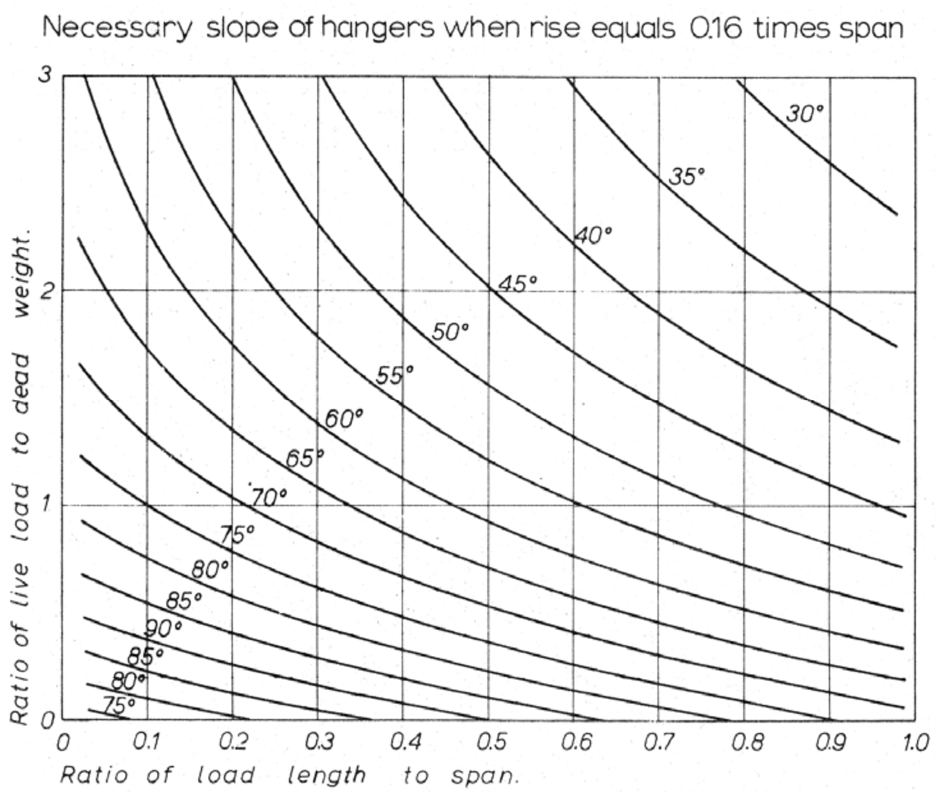


Fig. 38d. Slope of hangers necessary for preventing the relaxation of hangers when bridges with f/L=0.16 carry uniform load from the left

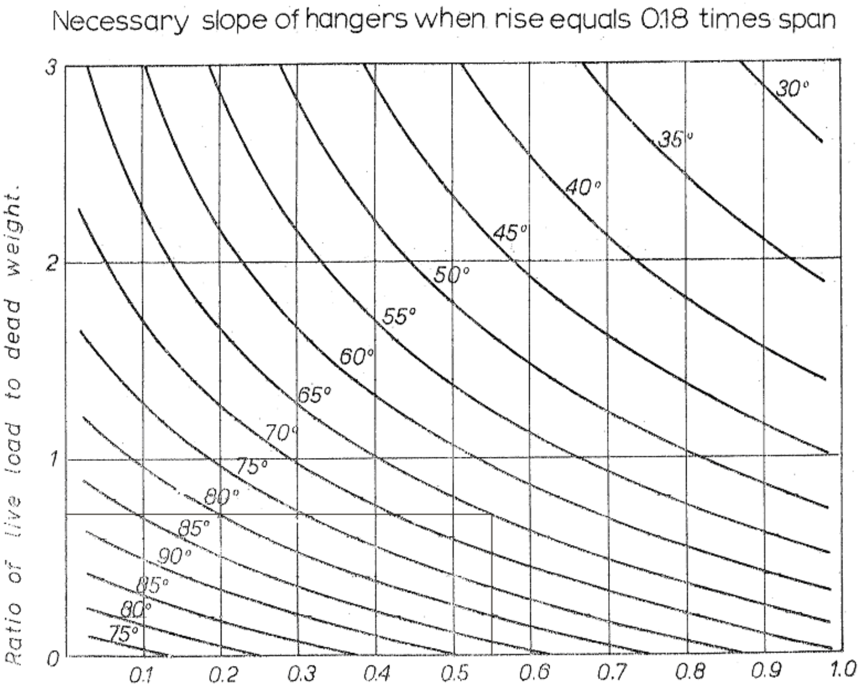


Fig. 38f. Slope of hangers necessary for preventing the relaxation of hangers when bridges with f/L=0.18 carry uniform load from the left

Bridge load usually consists of a combination of concentrated and evenly distributed loads. When using figs. 35 and 38a to 38f it seems a rough, but reasonable assumption that the loaded influence line is a triangle. Then the bridge load can be transformed into an equivalent evenly distributed load, which is equal to the evenly distributed bridge load, plus the concentrated load divided by half the loaded length.

Fig. 38g gives the result of applying the diagram in fig. 35 to the skeleton lines of the bridge on page 60. These results are compared to the tendencies for relaxation found from the calculated influence lines of the hangers. Tveit 80a. The correlation is surprisingly good in the middle two thirds of the span.

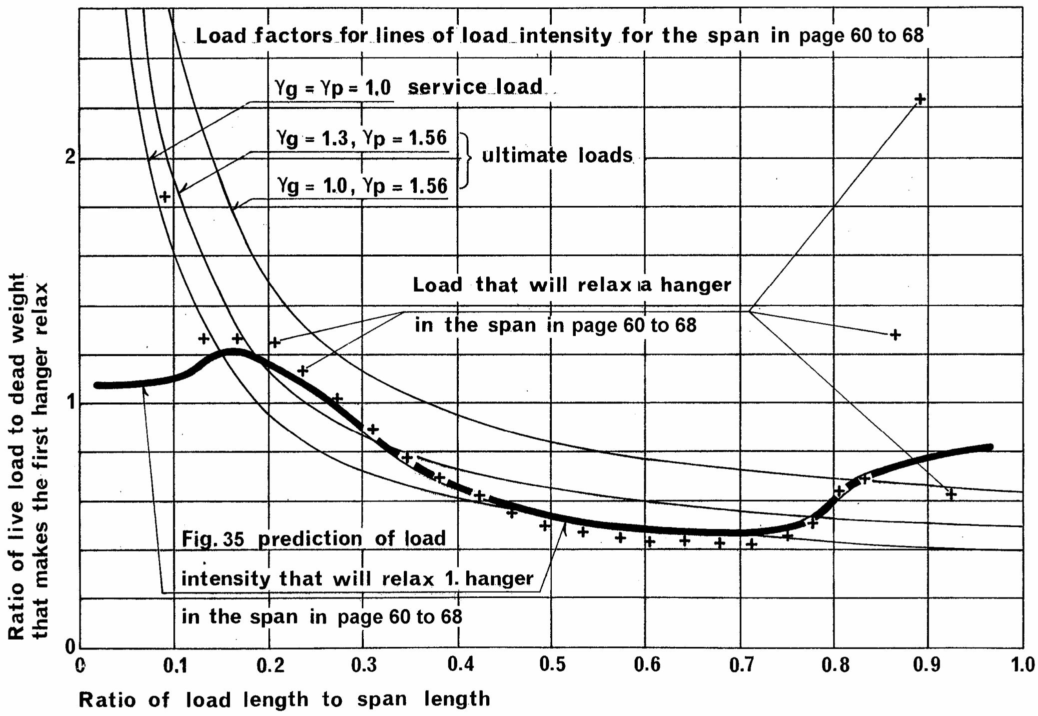


Fig. 38g. Relaxation of first hanger according to fig. 35 and according to the calculation of span on page 60

The prediction of a hanger's relaxation found by figs 35 and 38a to 38f is best for long bridges with slender chords. For stiffer and shorter spans the diagrams 35, 38a to 38f are likely to overestimate the hanger's tendencies to relax. This is because shorter spans and/or spans with stockier chords tend to accumulate more shear and bending in the chords prior to relaxation of hangers.

For Bolstadstraumen Bridge in figs 7 and 64 the overestimation of the hanger's tendency to relax was 5% or more for all hangers. That means that the hangers had a smaller tendency to relax than calculated by means of fig. 38f. For the Bridge at Steinkjer figs 6, 6a, 6c,and 63 the overestimation of the hangers tendency to relax by means of fig. 35 was nowhere under 15 %.

The thin curves in fig. 38g represent equivalent load intensity according to the Danish regulations for road bridge loads. See fig. 23 and Vejdirektoratet (74) for the span on page 60. It is interesting to note that full load on the whole span decides the dimensions of the span on page 60. This is so even though the equivalent load pr unit length for 0.61 times the length of the span was more than 75% greater than the load that made the first hanger relax. This is caused by the considerable stiffness of the arch. See figs 24, 70 and 76.

For full load on the left 50 % to 70% of the span some hangers may relax occasionally in the serviceability limit state. For these loads the chords in the ultimate limit state carry considerable bending moments. In network arches with more slender arches, for instance universal columns, one might choose to avoid the relaxation of hangers also in the collapse limit state.

This is because there is little bending capacity in the arch to take the bending moments that occur due to relaxation of hangers. Furthermore an increase in the live load over the years might be very dangerous. Another reason for avoiding the relaxation of hangers might be a desire to save work. See also pages 67, 68 and 90.

For the loads on the right quarter of fig. 38g, the hanger's resistance against relaxation might seem unnecessarily high. Here the slope of the hangers was decided by the desire to have a relatively even force in all hangers.

The slope of hangers cannot be chosen freely since it depends on the distance between nodal points in the chords and on the number of times a hanger in the middle of the span intersects with other hangers. Distance between nodes in the central part of the span and the number of intersections of a hanger in the middle of the bridge strongly influence the slope of all hangers.

Examples of choice of hanger arrangements can be found in two articles on preliminary design of network arch road bridges on the author's home page http://pchome.grm.hia.no/~ptveit/

Little is known about the optimal arrangement of three or more sets of hangers. See fig. 26e. It shows the hanger arrangement for a single-track railway bridge spanning 67 metres. The live load is a train according to Union International des Chemins de Fer 74. Since then the live load on a railway bridge has been increased by around 30 %.

To prevent relaxation in the serviceability limit state, the two most sloping hangers in the steepest sets of hangers have been given up to 0.4% extra length. In other words various degrees of lack of fit have been introduced. Furthermore, the steepest set of hangers, and some hangers at the end of the span have 44% smaller cross-section than the other hangers. These measurements give a very even utilisation of hangers and contribute to evenly distributed bending.

NETS OF HANGERS. SOME SUGGESTIONS.

The optimal net of hangers is dependent on many factors. Five will be mentioned here:

1. Ratio of live load to dead load.

2. Size of concentrated load compared to size of evenly distributed live load.

3. Length of concentrated live load.

4. Rise of arch.

5. Curvature of arch. See page 27, third paragraph.

Hanger arrangements that have been used:

1. The same slope for all hangers. This will not give an even load in all hangers, but it might be the arrangement that looks best. Since the hangers can hardly be seen from a distance this is probably not very important. If the hangers start from evenly distributed nodes in the lower chord, this speaks for thicker arches.

2. In the network at Steinkjer and in the Bolstadstraumen Bridge the slope of two adjacent nearly parallel hangers differs by 1.8˚ and 1.7˚. The steepest hangers are 74.4˚ and 73.8˚. See pp. 56-58 and Tveit 1966. Maximum hanger forces became very even.

3. In the two bridges designed for the IABSE congress in Vienna in 1980 (See pp 59-72) the distance between the nodes in the middle half of the tie is constant. For ViennaA that distance is the same as in the arch. For ViennaB the distance between the nodes in the middle half of the tie is 97.6 % of the distance between the nodes in the arch.

The nodes in the half of the tie nearest to the ends of the bridges are placed to obtain the same decisive force in all hangers. In this way the longitudinal bending moment in the middle half of the tie is minimum. See figs 65 and 77.

4. In the network arches suggested by Brunn and Schanack 2003 pp. 30-74 the angles between the arch and the hangers are constant. Towards the end of the arches one might give some of the hangers another slope to obtain more even hanger forces. This arrangement seems to produce some good results and might become very popular in the future.

Fig. 39. Lifting the temporary steel skeleton of the Åkviksound Bridge in fig. 97. Span 135 m.

DESIGN OF TEMPORARY LOWER CHORDS

Some very promising methods of erection use a temporary lower chord. See figs 21, 22, 56 and 57. 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.

The temporary lower chord has longitudinal beams between the ends of the arches. Transverse beams are placed equidistantly except near the ends of the arches. Longitudinal wooden beams on top of the 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. 39 shows how the steel skeleton of the Åkvik Sound Bridge in fig. 97 is lifted in place by Norway's biggest floating crane. The lifting capacity is 600 tons. The steel skeleton weighs around 230 tons, but it might be practical to put in so much wood and reinforcement that 410 ton is lifted. Floating cranes that can lift over 2000 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 steel skeleton.

When the reinforcement and the form are finished, the concrete tie can be cast. First, the ends of the tie with the curved parts of the prestressing cables must be cast. Then the edge beams are cast. The casting must be done from both ends of the span in order to avoid relaxation of hangers during the casting.

Fig. 90 shows forces and deflections when the smaller edge beam in the Åkviksound network arch is cast. For the longitudinal edge beams the decisive load occurs during the casting of the edge beam. After the edge beams are cast they take most of the bending in the tie. The prestressing cables take most of the axial force in the tie. Thus the transverse beams can be placed equidistantly even if this does not give minimum bending in the chords. Any transverse beam with enough length and strength can used. Finally the slab between the edge beams is cast. Afterwards the temporary lower chord can be removed and used again for erecting other network arches.

In fig. 39a, see next page, the temporary lower chord is joined to the ends of the arches just above the bearings. Reinforcement, dowels and diagonals in the lower chord are not shown. The temporary lower chord can be joined to the steel plate above the bearings. Some of the dimensions are taken from the bridge in fig. 97. Generally the dimensions are not the result of precise calculations. At the fixed end of the arches the main bearings can be of the types shown in figs 6b and 18.

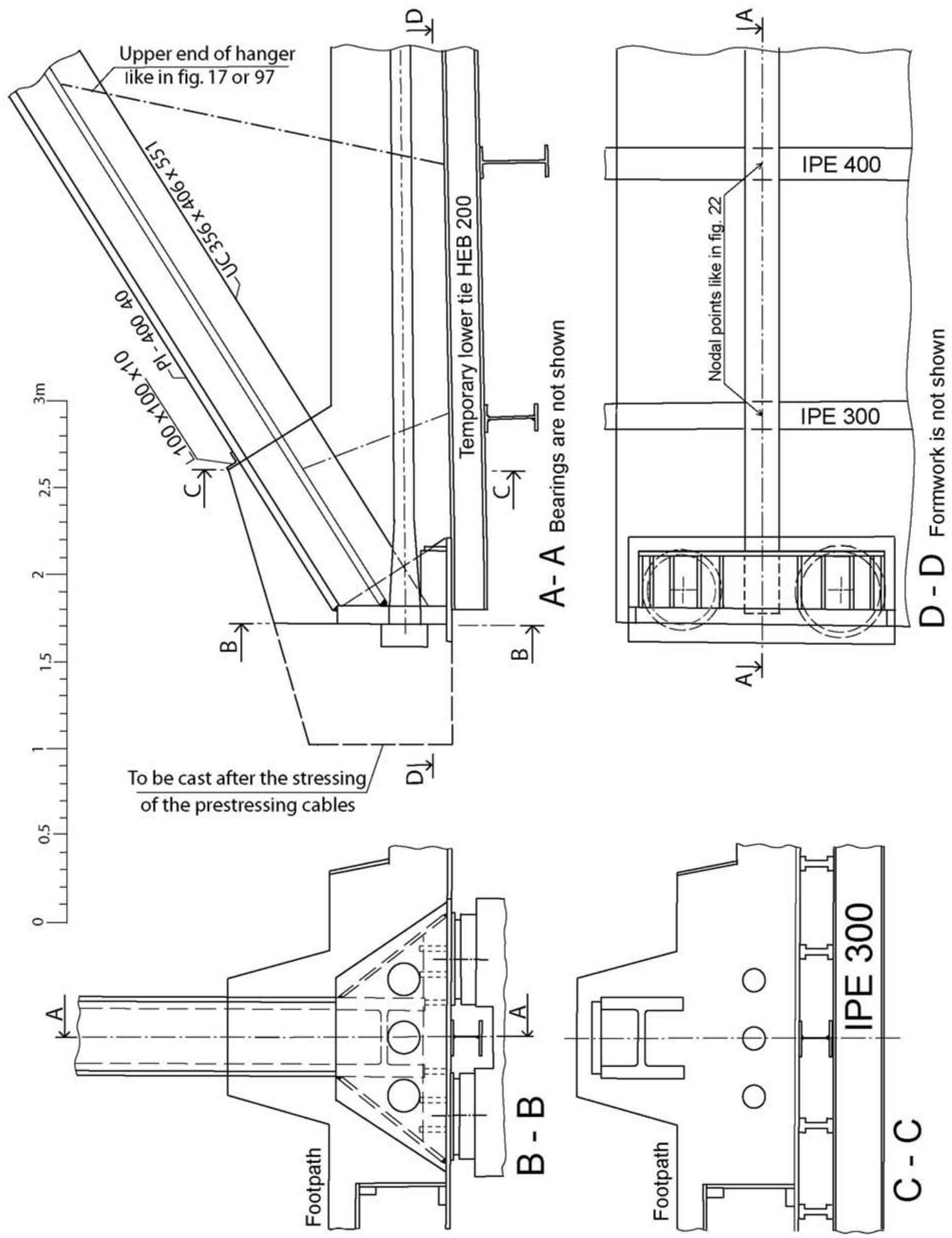


Fig. 39a. Ends of the temporary lower chord

Maybe the outer bearing under the ends of the arch in fig. 39a should be smaller than the inner bearing. Usually there will be an edge beam at the end of the concrete slab in the tie. In fig. 39a the end of the slab has been made thicker to avoid the edge beam. In order to reduce the necessary thickness of the end of the slab, one or two bearings can be placed on the bridge pier and under the ends of the slab. We have a discontinuous system if the slab touches the bearings at the end of the slab only due to big loads like trucks and buses.

Since the big loads are of short duration, load on the bearings last only a short while. Thus the bearings under the ends of the slab can be designed disregarding most of the relative movement between the bridge and the pier. Simple bearings can be used. After the temporary lower chord has been removed the hole for the temporary lower chord can be used for inserting hydraulic jacks when bearings need to be changed.

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. The author can list a lot of reasons why and would like to state that the full potential of a network arch 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. A network arch with a permanent steel tie might be simpler to erect, but would have a much higher weight than the arch and hangers of a network arch supplemented by a temporary tie. The comparison is lopsided, but in fig. 14 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.
3. 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.
4. 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.
5. 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 are distributed when they reach the edge beam.
6. A permanent tie of structural steel in tension causes cracks in a concrete slab above it. This reduces the durability of the concrete slab.
7. Transverse beams in the permanent lower chord would make the reinforcement in the slab more complicated.
8. The temporary lower chord is joined together by high strength bolts. It needs no corrosion protection and can be produced on site. Thus per tonne cost of fabrication is not high.
9. 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.
10. 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.
11. 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.
12. A permanent lower chord would be shop welded and would have a corrosion protection that has to be maintained forever.
13. The great longitudinal tensile forces in the tie are best taken by prestressing cables because of the high strength-to-cost ratio.
14. Prestressing cables prestressed against concrete will take fatigue well.
15. The longitudinally partially prestressed slab in the lower chord of a network arch bridge is favourable as far as maintenance is concerned.
16. 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.
17. 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.

NETWORK ARCHES ERECTED ON ICE

The author has designed a network arch with four sets of hangers. Tveit (84 b and c). See fig. 26. It was a two-lane bridge with a span of 100 metres designed for Arctic areas. The tie was to be cast and cured on a 2.5 metre thick reinforced layer of ice floating between piers.

All hangers have the same cross-section. Even before introducing lack of fit in the hangers, the average maximum hanger force was 93% of the maximum force in the hanger with the greatest maximum load. By introducing lack of fit in the hangers, maximum hanger force can be made more even and maximum bending in the chords can be reduced. In a network arch with only two sets of hangers little can be gained by introducing a lack of fit or prestressing some hangers.

When the concrete is cured, the structural steel is erected and the finished span is lifted up to its final position. The method of erection could be competitive and would contribute to reducing winter unemployment among building workers in Russia and Canada.

Another possibility is to erect the steel skeleton in fig. 57 on ice in the winter and lift it on to the pillars. 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 ice under the transverse beams in the temporary lower chord.

For this type of erection one would want to prevent water from seeping onto the surface of the ice near the steel skeleton. If this is achieved, the strength of the ice would be ample. 600 to 800 mm might be sufficient for carrying the 1.2 ton per metre that the steel skeleton for the Åkviksound 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 690mm thick the permissible load for working on river ice is 8 tons.

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 on to the ice.

Tekn. dr. Lennart Fransson suggests an experiment to find the necessary thickness of ice to support the erection of the steel skeleton. Sand weighing as much as the steel skeleton could be put out on ice of varying thickness to see how soon the water started seeping onto the surface of the ice.

The snow must be removed from the ice near the bridge site in order to avoid layers of snow in 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 figs 21, 22 and 39a. In the Norwegian Åkvik Sound Bridge (See figs 10 to 17.) the 18 pieces of the arch would weigh up to 6 tons. A mobile crane that can reach up to 23 metres would erect these. There would be around 200 hangers weighing up to ~250 kg. There would also be 260 profiles with a weight up to 300 kg, but more than half of these profiles would weigh less than 20 kg.

A standard steel scaffold could support the erection of the arch. See pages 50a to 51. When the scaffold is removed, the steel skeleton can be moved on to the bridge piers. In the spring the lower chord is cast. After the prestressing has been done, the room for the presses in the side spans can be finished.

RAILWAY BRIDGES

In 1964 when the author had finished his PhD thesis, he happened to come across the drawings of a bridge to be built over the Tinnelv river at Stormo in Norway. It was a single-track railway bridge spanning 67.8 m. 275 tons of steel were to be used for the welded truss. Through preliminary calculations the author found that only some 80 tons of steel would have been needed for a similar bridge using network arches.

It deserves to be mentioned that the author did not pay enough attention to fatigue in his preliminary design. Since the bridge was soon to be constructed, there was no time for a redesign. In the thirty years after the opening of the Tinnelv Bridge, no railway bridges with a span over 60 m (i.e. suitable for a network arch solution) have been built in Norway.

Since 1964 the author has been convinced that the network arch is suitable for railway bridges. It has a pleasing appearance, and the lower chord could very well be a concrete trough that carries the railway between the arches and at the same time provides the necessary horizontal tie needed. All lower chords made of concrete and especially the concrete trough with ballast give less noise than an all-steel solution. Ballast is gravel in which the railway track is placed. The thin lower chord gives shorter ramps. This is especially valuable in railway bridges in flat terrain because railways have smaller maximum gradients than roads.

Fig. 40 gives a comparison of the steel weights for different types of railway bridges. The lines for single-track railway bridges are found in Herzog 1975. The shaded areas were first presented in Tveit 1973. Note that the double-track network arch bridge tends to use less steel than a usual single-track railway bridge! The low steel weight of the network arch bridges is not only due to the optimal behaviour of the arches having inclined hangers. It is also due to the use of a concrete trough or slab in the lower chord instead of the more common solution, i.e. using floor-beams and stringers of steel.

Steel prestressed against the concrete in the tie is less susceptible to fatigue and can carry high stresses. This contributes to low steel weights. The prestress makes the lower chord less susceptible to fatigue and more resistant to corrosion. The stress in the hangers is much influenced by fatigue. This is not very important, because the hangers are a small proportion of the steel weight. Furthermore all diagonals in truss railway bridges are also influenced by fatigue.

Fatigue increases the cross-section of the hangers. This increases buckling strength in the arches. If fatigue decides the dimensions of the hangers and the arch, the weight of the concrete would hardly increase the steel weight because it gives constant stress in arch and hangers. There is little bending in a network arch. This is good because bending very often gives fatigue due to stress variation.

The cross in the diagram below indicates the result of the steel weight found by Brunn and Schanack 2003. Their work will be a great help to anyone who wants to design a network arch railway bridge. There is much more on railway bridges in Brunn and Schanack 2003 and Steimann 2004.

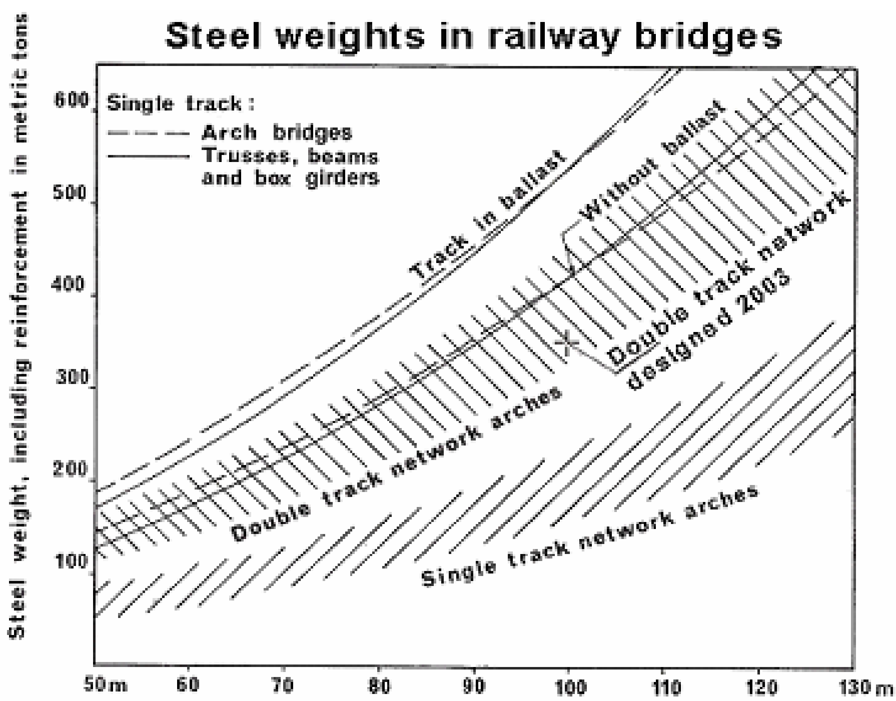


Fig. 40. Comparison of steel weights of various spans in various types of railway bridges

This part of the chapter is written partly by the designers of the various bridges. In 2002 and 2003 four students did their master's thesis in Grimstad, Norway. They were Uwe Steimann 2002, Benjamin Brunn and Frank Schanack 2003, and Mathias Räck 2003. Their gradiation theses can be found at http://fag.grm.hia.no/fagstoff/ptveit/ The professors Wolfgang Graβe and Steffen Marx have also contributed to this chapter.

Steimann designed a two-track network arch railway bridge spanning 100 m with a rise of 15 m. Ties of steel and concrete were examined. See figs 40a, 40b, 40c and 40d. The bridge with a concrete tie used half as much steel.

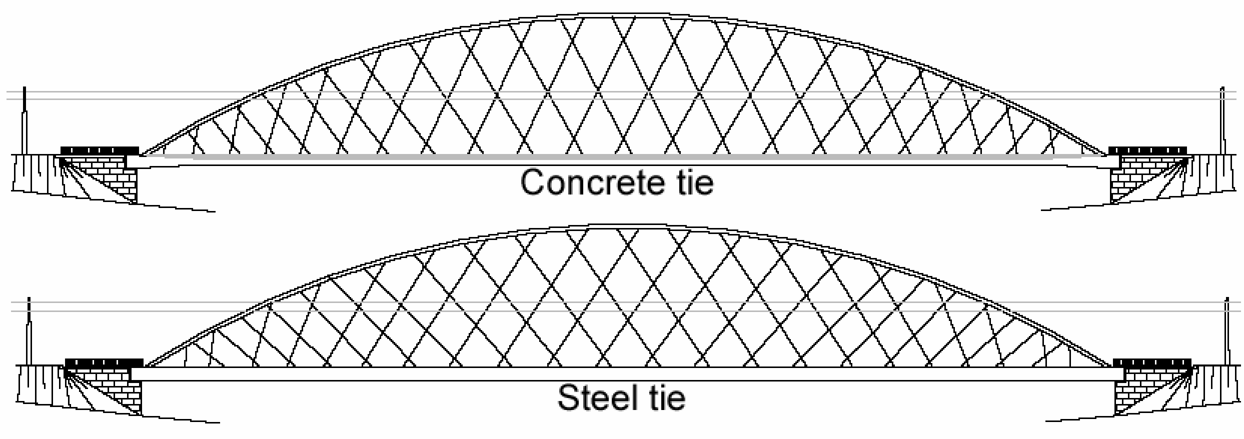


Fig. 40a Network arch railway bridges spanning 100m.

Steimann found that H-profiles could hardly be used in the windportal and recommended a box section in the arches. He also recommended flat hangers -80x50 mm. This is as recommended by the German railways. The hangers can be welded in a way that gives high fatigue strength.

Steimann has gone on to design network arches in the firm GMG-Ingenieurgesellschaft mbH, Dresden.

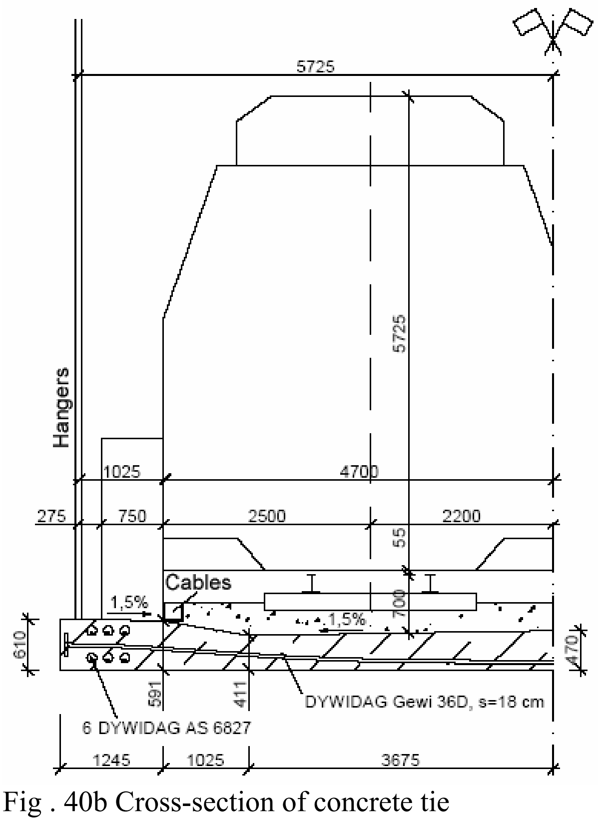


Fig . 40b Cross-section of concrete tie

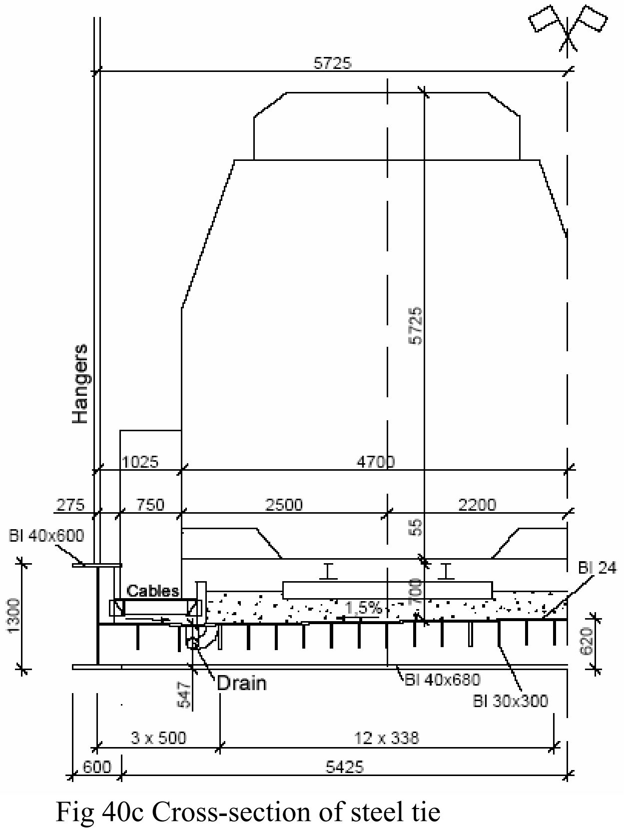


Fig 40c Cross-section of steel tie

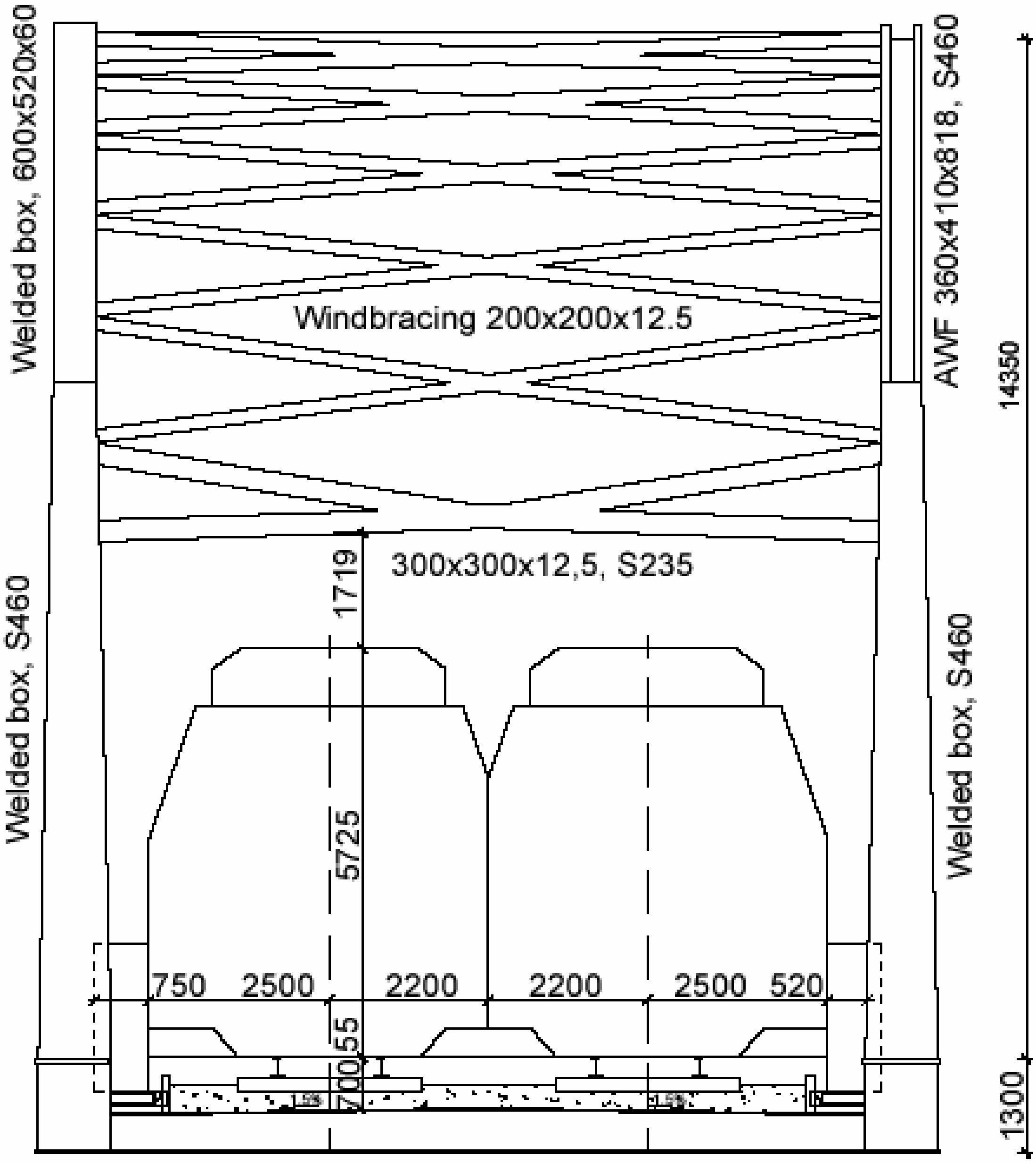


Fig. 40d. Front view with windbracing of steel alternative of Steimann's railway bridge.

Fig. 40e shows a two track railway bridge spanning 100 m designed by Brunn and Shanack

Benjamin Brunn and Frank Schanack did their master's thesis in Grimstad in the summer of 2003. They too designed a two track network arch railway bridge spanning 100 m. A colour picture is shown in fig. 40e. They used a rise of 17 m and a windbracing that is shown in fig. 40e. This windbracing has little room for electric lines above the train. The tie is made of concrete. One concrete cross-section is shown in fig. 40f. Their thesis is written in English. It can be found on http://fag.grm.hia.no/fagstoff/ptveit/

The distance between the arch planes measures 10.15 m. The 43 cm deep concrete slab that spans between the arch planes is transversely prestressed by threadbars with a diameter of 36 mm and at distances of 27 cm. The examination of a 47 cm deep reinforced concrete slab showed a reasonable alternative without prestressing. The bridge deck has extensions on either side providing an area for 75 cm wide service footpaths.

Each arch plane has 48 steel bar hangers with a diameter of 60 mm. The arrangement of the hangers, which essentially decides the forces and force variations in a network arch, was the subject of an extensive optimisation process. As a result Brunn & Schanack proposed the radial hanger arrangement as shown in Fig. 40e. In its simplest form it features constant distances of the upper hanger connections along the arch and a constant cross angle between arch and hangers.

The radial hanger arrangement has been applied successfully in many students' works, e.g. Skalda&Rohm (Awarded by the Saxonian Building Industry 2005), Drange Hole (Norway), Valenzuela (Chile) and Beyer (Awarded the Gottfried-Brendel-Prize 2006 of TU-Dresden). Furthermore it has been the object of scientific studies by Prof. Graße, Dipl.-Ing. Teich and J. Berthellemy. Flora Bridge, p 35, over Mittelland Canal at Haldersleben, Germany is the first network arch bridge that will be built with the radial hanger arrangement.

The existence of a wind bracing and the radial hanger arrangement leads to very small bending moments so that merely rolled American Wide Flange profiles can be applied for the whole arch length. That is W360x420x634 for the middle section and W 360x420x900 for the ends that also form the portal frame columns.

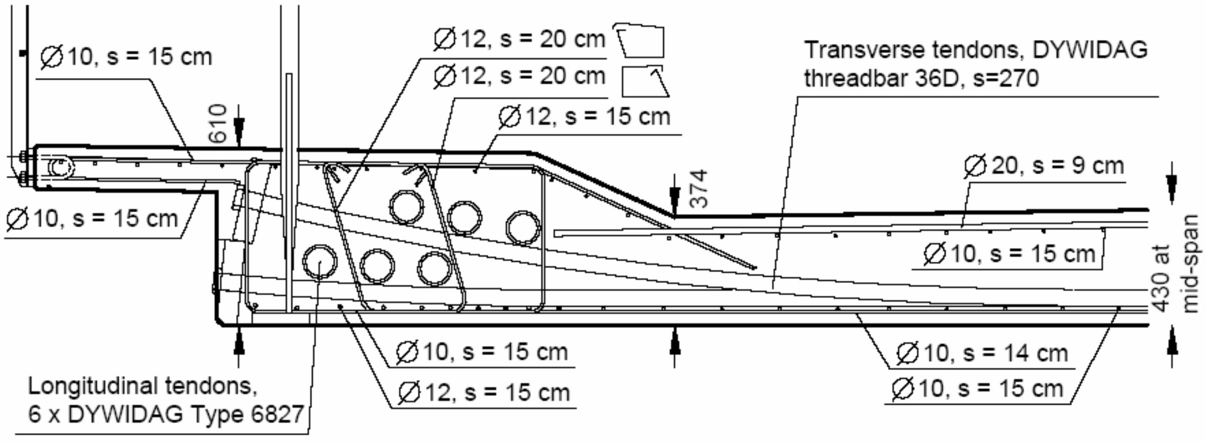


Fig. 40f. Reinforcement for a tie of a railway bridge designed by Brunn and Schanack, 2003.

The wind bracing is a K-truss of CHS 219.1x8 profiles. The bending length of the portal frame columns is reduced by diagonal struts as shown in the front view in figure 40e. The outline goes well with the railway clearance gauge.

The construction stage design is based on using a temporary lower steel chord. The bridge with this temporary lower chord weighs less than 400 t and can easily be transported. Resting on its final position the steel skeleton can support the formwork and fresh concrete for the deck. The casting sequence might be different from project to project to avoid hanger relaxation during the construction stages.

The double track railway network arch bridge has been designed according to the specifications of the Eurocode for a maximum train speed of 160 km/h. The total amount of steel is 448 t, which corresponds to 2.24 tons per track and meter of span. According to [Herzog 1975] the reference value for other steel railway bridges with a span of 100 m is about 5.5 tons per track, which is almost 150 % higher.

Brunn and Schanak, 2005, use a very advanced shape of arch and hanger arrangement that leads to very constant axial force in a long middle part of the span. The maximum hanger forces are very equal and near to optimal when it comes to fatigue. Their thesis is written in English. It can be found on http://fag.grm.hia.no/fagstoff/ptveit/

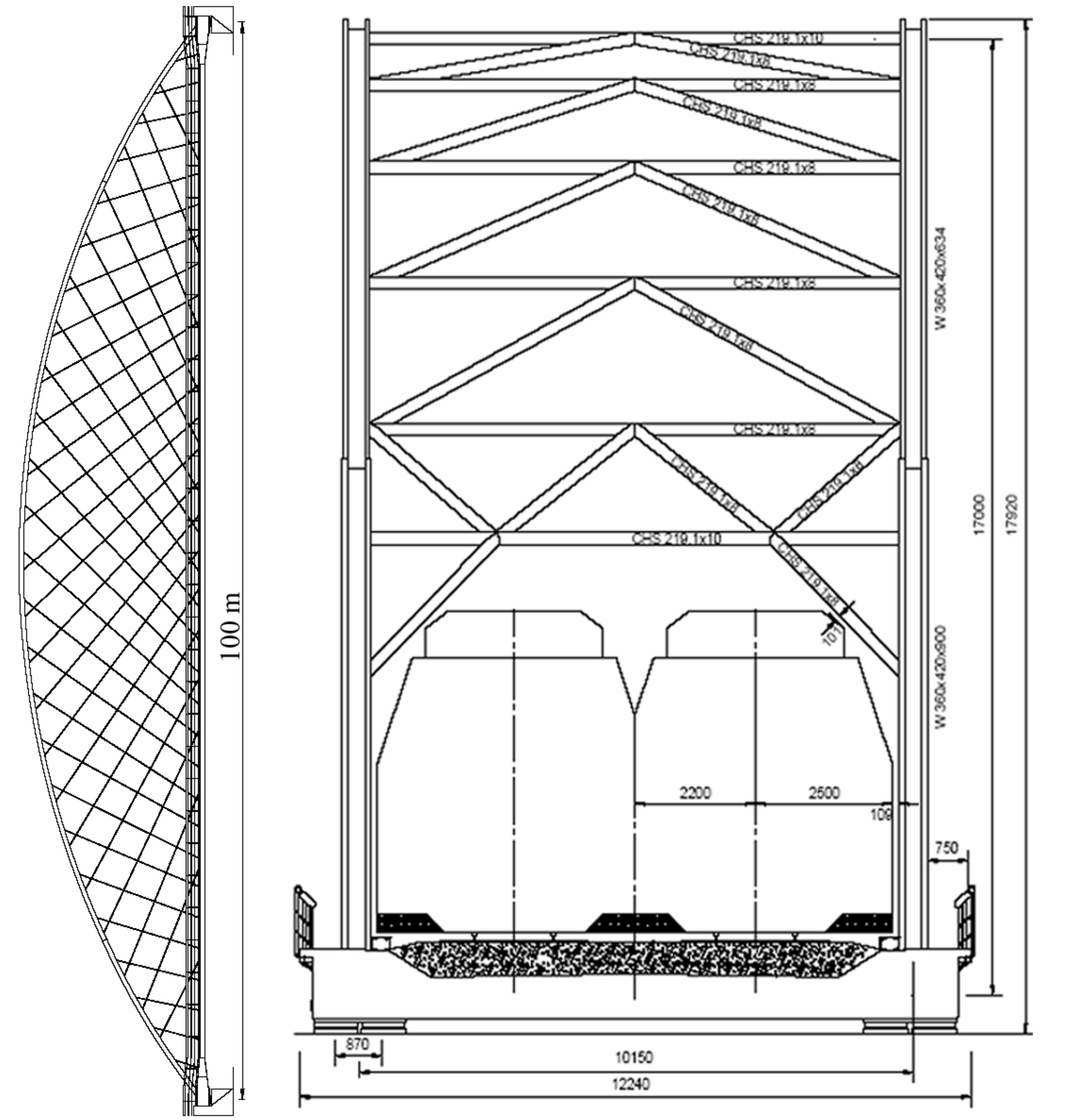


Fig. 40e. This network arch railway bridge was designed by Brunn and Schanack 2003. Their graduation thesis was written in English.

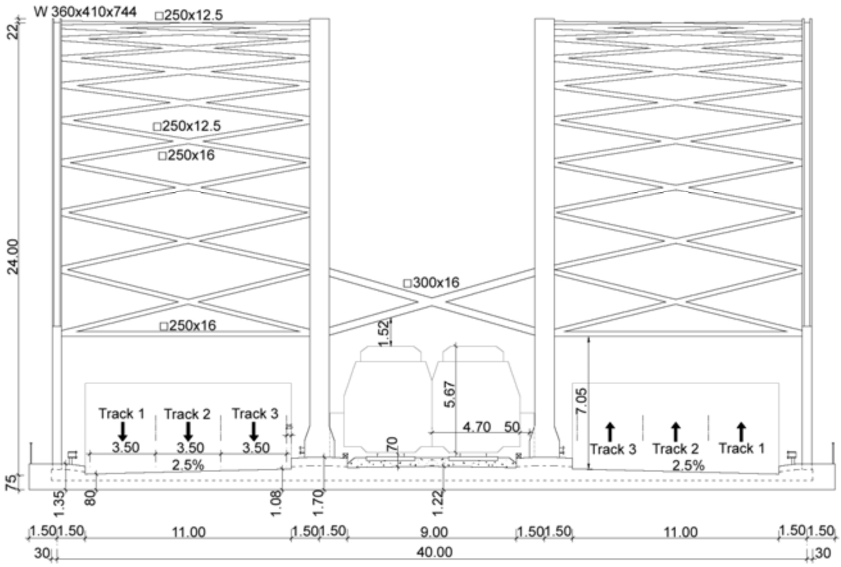


Fig. 31d. Cross-section of a combined road and railway bridge designed by Räck 2003.

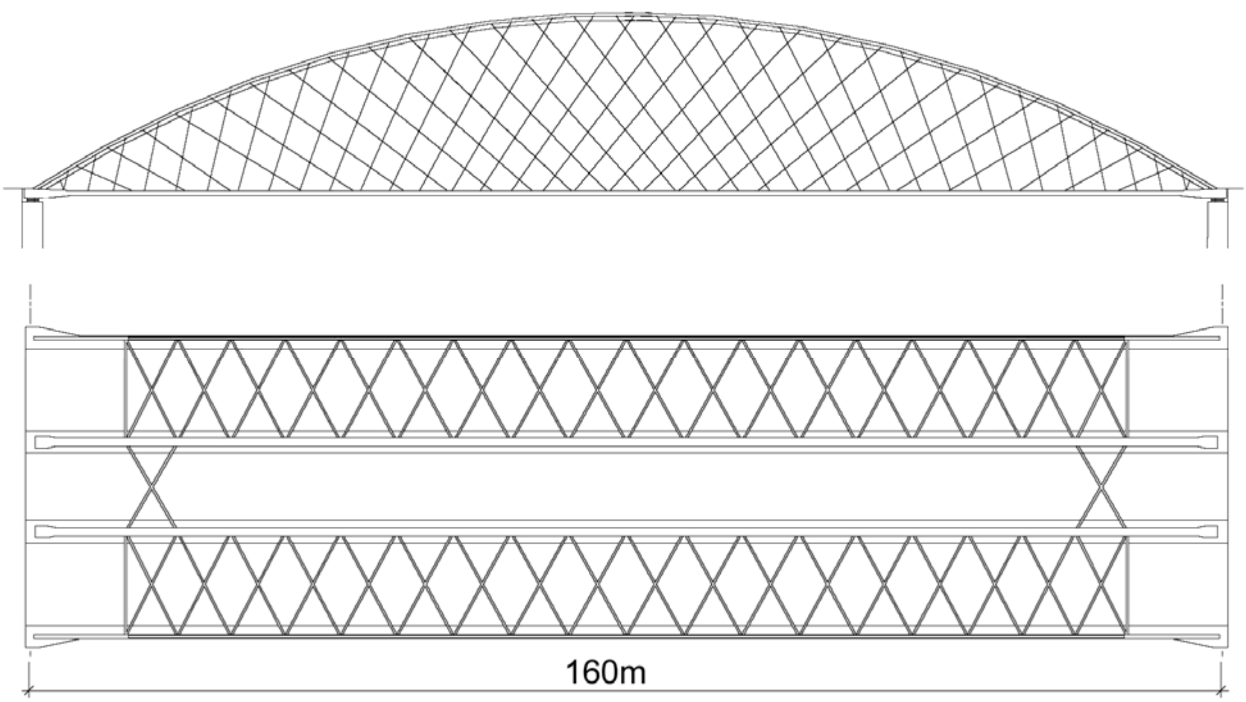


Fig. 31e. Two drawings of the bridge in fig. 31c. See also Räck http://fag.grm.hia.no/fagstoff/ptveit/.

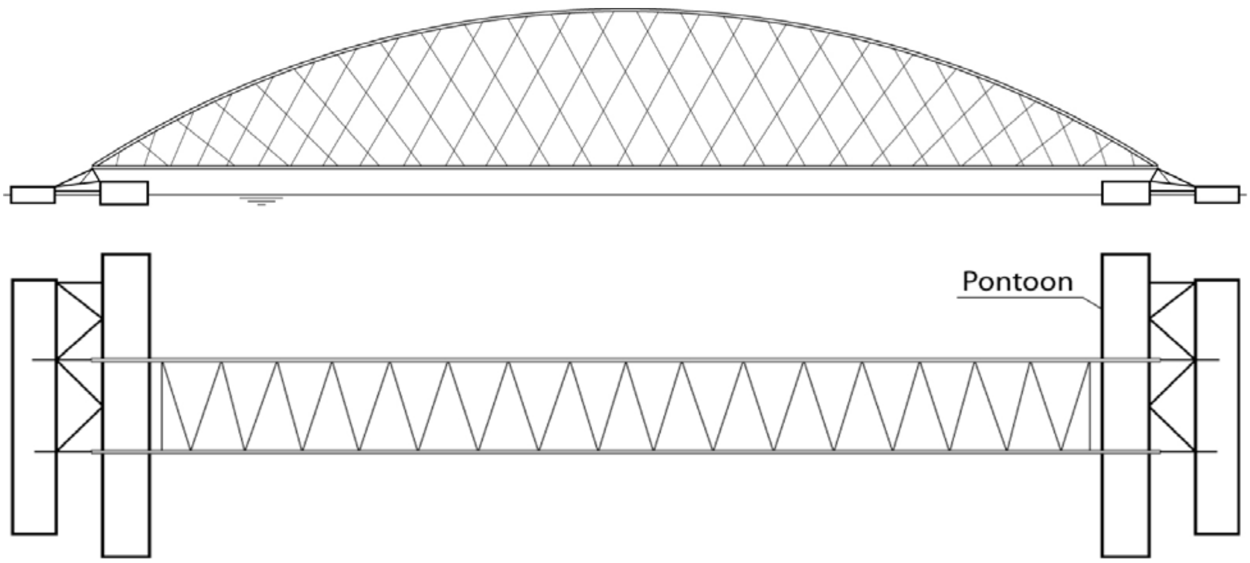
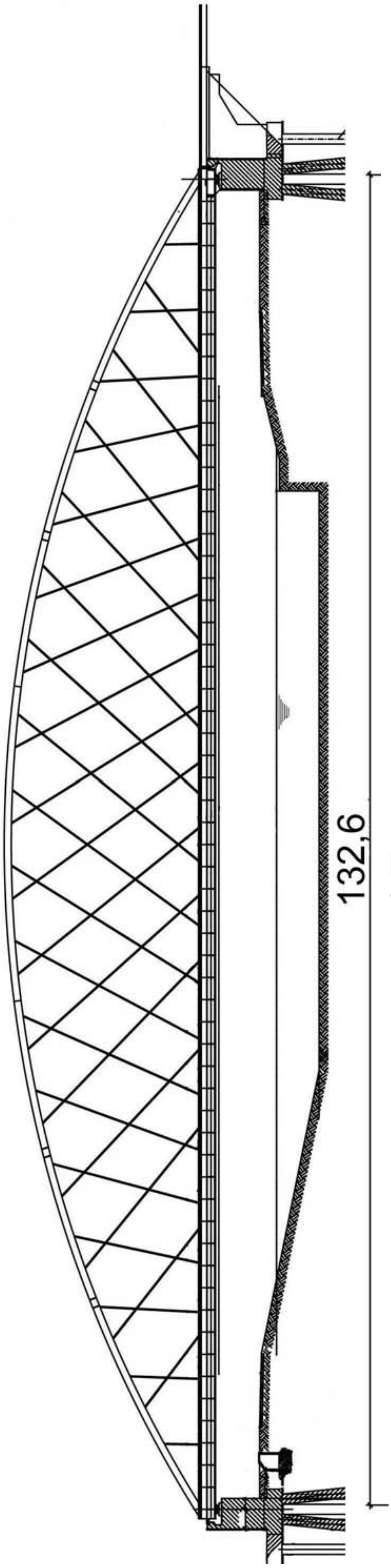


Fig. 31f. How the bridge in fig. 31d and 31e can be transported from the shore to the pillars.



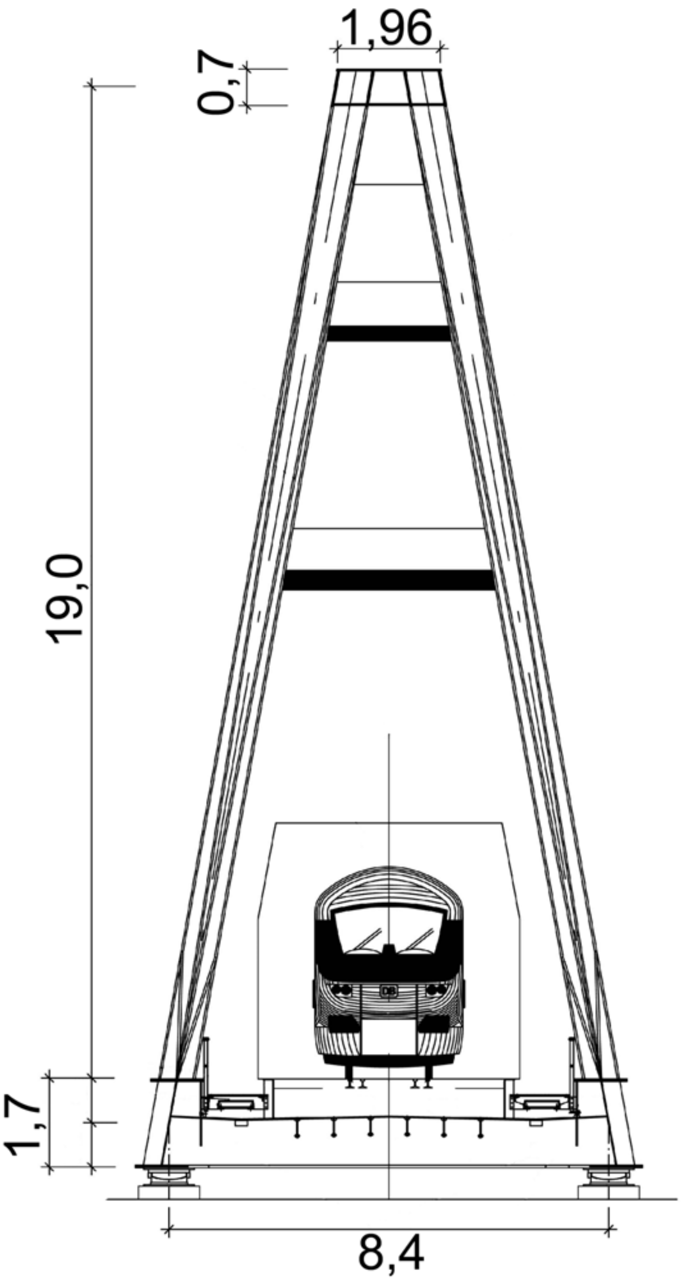


Fig. 40i. Flora Bridge over the Mittelland Canal in Germany.

This railway bridge with a span of 132.6 m was designed by GMG Ingenieurgesellschaft in Dresden. Compared with a similar arch bridge with vertical hangers under the same loading assumptions, clearance diagram, height of construction, arch rise and steel grade a reduction of steel mass from 1405 t to 1000 t could be achieved, see Graße and Tveit 2007.

The circular arches have a box cross-section of 700 x 700 mm and a rise of 19 m. The stiffening beams are relatively slender steel beams with one web and a depth of 1700 mm. The orthotropic steel deck connected with the stiffening beams and the 8.4 m span cross beams give the bridge the transverse stiffness necessary for driving dynamics.

The dominant feature of the bridge is the network of 34 inclined hangers per arch instead of the 12 vertical ones. The slope of the ascending hangers falls over the bridge length from 89° to 37°, the slope of the descending hangers increases from 37° to 89°.

This hanger arrangement shows no relaxation of hangers under service loads. The drastic reduction of bending moments in arches and stiffening beams is the main reason for the saving of steel. Since aesthetic aspects were not particularly significant for this railway bridge, hangers with a cross-section of 100x 40 mm were used. Therefore - compared with round hangers - a higher fatigue strength of the relatively simple butt welds was achieved.

The hangers are fastened to the inner and outer vertical plates of the arches alternately. So they have a distance at the crossing points, and devices for damping can be built in. The upper flanges of the stiffening beams are connected laterally to their web so that the hangers can be fastened directly to the web. The steel in the network arch has fewer joints than the arch bridge with vertical hangers.

The assembly of the bridge is planned to take place beside the railway track. Then the steel skeleton is moved transversely onto the track. Then comes a longitudinal launching across the canal by means of a pontoon that can carry 1250t.

Vertical planes for the arches and the footpaths outside the hangers would be more economical, but maybe this would not look so attractive. The sloping planes of the arches were adopted from a competing alternative with vertical hangers and can not be blamed on Ingenieurgeselshaft GMG in Dresden, Germany.

Professor Dr. Steffen Marx of TU-Dresden has written this German text on network arch railway bridges. Per Tveit has translated the text into English, making some insignificant alterations in the process.

Many network arches are being planned for the German railways. This is being done by a subsidiary company, DB Projekbau. It has been found that the network arch has the following advantages compared to conventional arch bridges with vertical hangers.

- Considerable savings of steel
- Greater stiffness and higher frequencies of vibrations. This leads to less chance of resonance caused by high speed trains.
- Smaller deflections that give greater passenger comfort
- Where hangers cross, they can support each other in such a way that this counteracts vibrations caused by wind and rain
- More slender chords that lead to a more pleasing appearance.

Network arches have the following drawbacks:

- The slender cords can only take small concentrated loads. Sometimes this makes the erection more complicated.
- The frequent change in the tension in the hangers results in a tendency to fatigue. This must be counteracted by the geometry of the net of hangers and by relatively high fatigue strength in the ends of the hangers.

Professor Marx describes of three railway bridges that are being designed:

The Rosenbachtal Bridge

The new bridge will replace 5 arch spans under the railway track because these spans are in poor shape. Compared to the rebuilding of the existing spans, the network arch means lower costs, better appearance and little damage to the valuable natural surroundings.

A single track bridge spanning 98 m is planned. It has an orthotropic steel tie. The superstructure needs 591t of steel. Compared to a conventional arch bridge with vertical hangers it saves 123 t of steel.

The network arch is erected in a field parallel to the railway track. Then it is moved sideways onto the track and rolled to the site. The railway traffic is stopped for three months while the existing bridge is dismantled and new abutments are built. The contract documents will be sent out in August of this year.

Railway bridge over the national highway B6 in Halle

A network arch is planned over the national highway B6 at the edge of Halle. See fig. 40j. It is on a track used by freight trains. Thus it will often carry heavy traffic loads. The arch has a span of 78 m. It will have a concrete track in the tie between two longitudinal steel beams. The slope of hangers varies from 83° to 52°. The slope in the middle is 62°.

The concrete tie provides many advantages compared to the steel tie.

- Considerable saving of steel
- Higher sideways stability
- Higher constant hanger force
- No corrosion protection required in the concrete tie

Additional advantages are the same as in other network arch railway bridges.

A simpler tie could be used if the concrete could resist the salt used on the track in the winter. Maybe the most exposed concrete could be covered by an epoxy membrane. The hangers and the windbracing are fastened to the arches in a simple way.

Some of Prof. Marx's comments are included in the description of the Flora Bridge

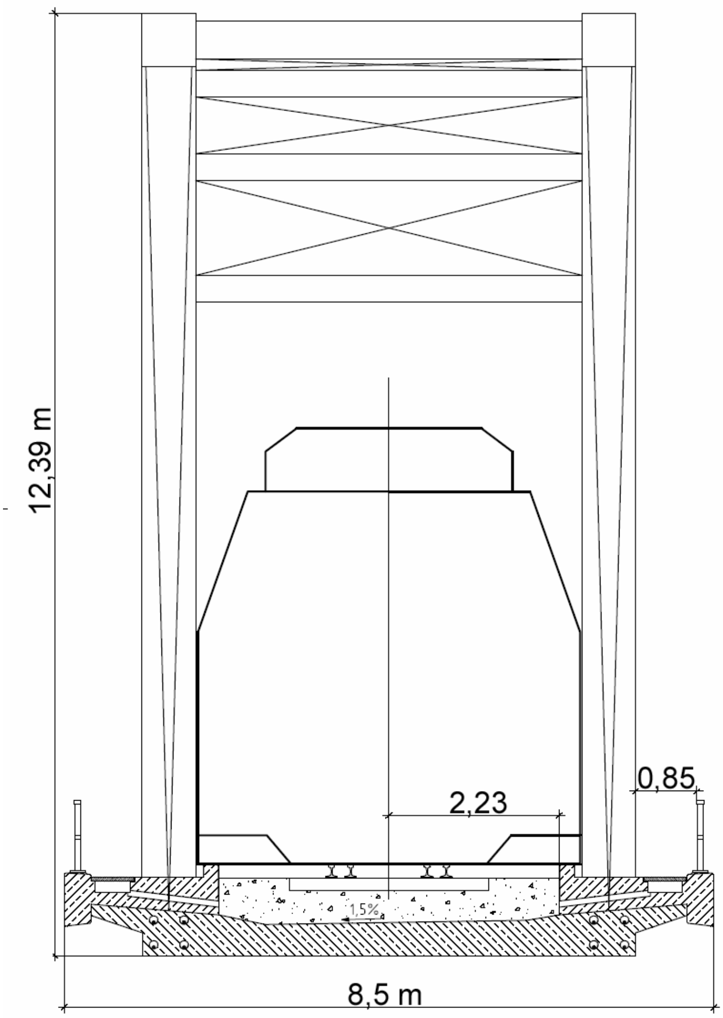


Fig. 40i. Shows an early suggestion by Prof. Marx for a cross-section of the railway bridge over B6 at Halle

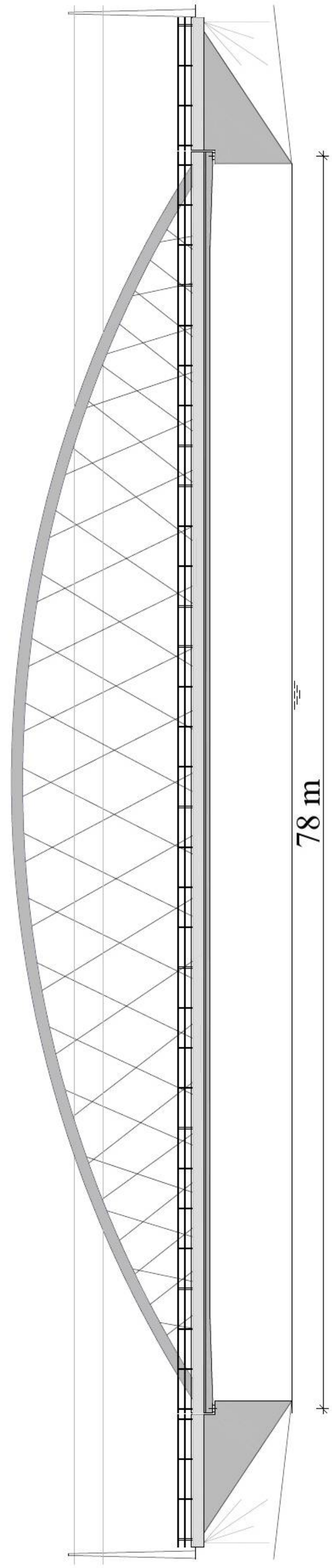


Fig. 40j. A planned network arch railway bridge over national highway B6

Some work on network arch railway bridges mainly before 1974.

In 1971 a group of Englishmen, Majid et al. 1971, published an article with the title "The Design of Inclined Tied Arch Railway Bridges over the M57". Among other things they concluded that inclined hangers were not as good as vertical hangers.

The author wrote a contribution to the discussion, Tveit 1972, where he presented influence lines and structural details, pointing out that the inclined hangers should have multiple intersections in order to be efficient. He also did show that, by using network arches, half the steel weight could have been saved for the two bridges presented in a paper by Gut 1971.

Majid et al. replied that complications in design and difficulties in the replacement of hangers were the reasons why inclined hangers had not been used. Needless to say the author does not agree with this reasoning.

The debate with Majid et al. subsequently inspired the author to write a more extensive report on a double track railway bridge spanning 65 metres. Tveit 1973. The drawings in the rest of this chapter are taken from this publication. They were exquisitely drawn by Henrik O. Madsen, a neighbour's son at the time. He is now head of Det Norske Veritas. The drawings can give valuable understanding of how network arches function.

For the design a structural steel with yield strength of 355 MPa and concrete cube strength of around 50 MPa were assumed. The steel quantities were 103 tons for the superstructure and 59 tons of reinforcement for the concrete tie. Most of the reinforcement was prestressing steel. The temporary lower chord for erection used 26 tons of structural steel.

All the data given are for half of the bridge. The weight of the concrete tie is 5 ton/m. The train carriages weigh 8 ton/m and the locomotive weighs 11 ton/m. The load factors in the ultimate limit state are for the dead load 1.2 and for the live load 1.6. A dynamic load factor for the live load has also been used.

The diagrams have been computed using linear methods. Hangers in compression have been removed. Loads and codes have changed considerably since the beginning of the 1970's, but still it is the author's hope that the diagrams will be of use to anybody who wants to design a network arch railway bridge.

The mechanism for adjustment of hangers shown in fig. 43 is probably not a good idea, but the way the prestressing cables are fastened to the lower end of the hangers should not be forgotten. These rods should be prestressed at an early stage. The author fears that it would be difficult to produce the arches in fig. 43. Instead of the suggested arch a single universal column could have been tried. See Brunn and Schanack 2003. They treat fatigue in chapter 7. Teich 2004 and Teich and Graβe 2004 treat fatigue in hangers.

In fig. 45 the load is symmetrical about the centre axis of the bridge. On the right hand side the hangers in compression have been removed before recalculation. For the sake of argument the hangers on the left-hand side have wrongly been supposed to take compression. On the right-hand side the moments are bigger.

On the left-hand side nearly all hanger forces are bigger than on the right. This is a useful general trend that makes it always on the safe side to use influence lines for calculating hanger forces, even though the influence lines have been calculated assuming that the hangers can take compression.

In a letter to the author Professor Niels J Gimsing of DTH Denmark points out that the change in the maximum load in the hangers of a railway bridge comes more abruptly than in a road bridge. This might give harmful effects if the hangers of railway bridges are allowed to relax in the serviceability limit state.

The author agrees that the changes in the load in the hangers of railway bridges come more abruptly than in road bridges. It will not cost much to avoid the hangers in railway bridges relaxing in the serviceability limit state. See Brunn and Schanack 2003. Such a practice is recommended. See also pgs 27, 29i, 67 and 68. Stephan Teich's doctoral thesis, 2008, will have valuable information on how the slopes of hangers influence their resistance to relaxation.

The hangers in the bridge in pgs 31a and 37 will not relax in the serviceability limit state. It is for future designers of railway bridges to calculate whether there will be harmful shocks in the hangers of their railway bridges. They have to consider how fast the load increases in hangers that have relaxed. Other factors are the frequencies of vibration in the hangers and in the whole span. The dampening in the joints between the hangers will also have to be considered.

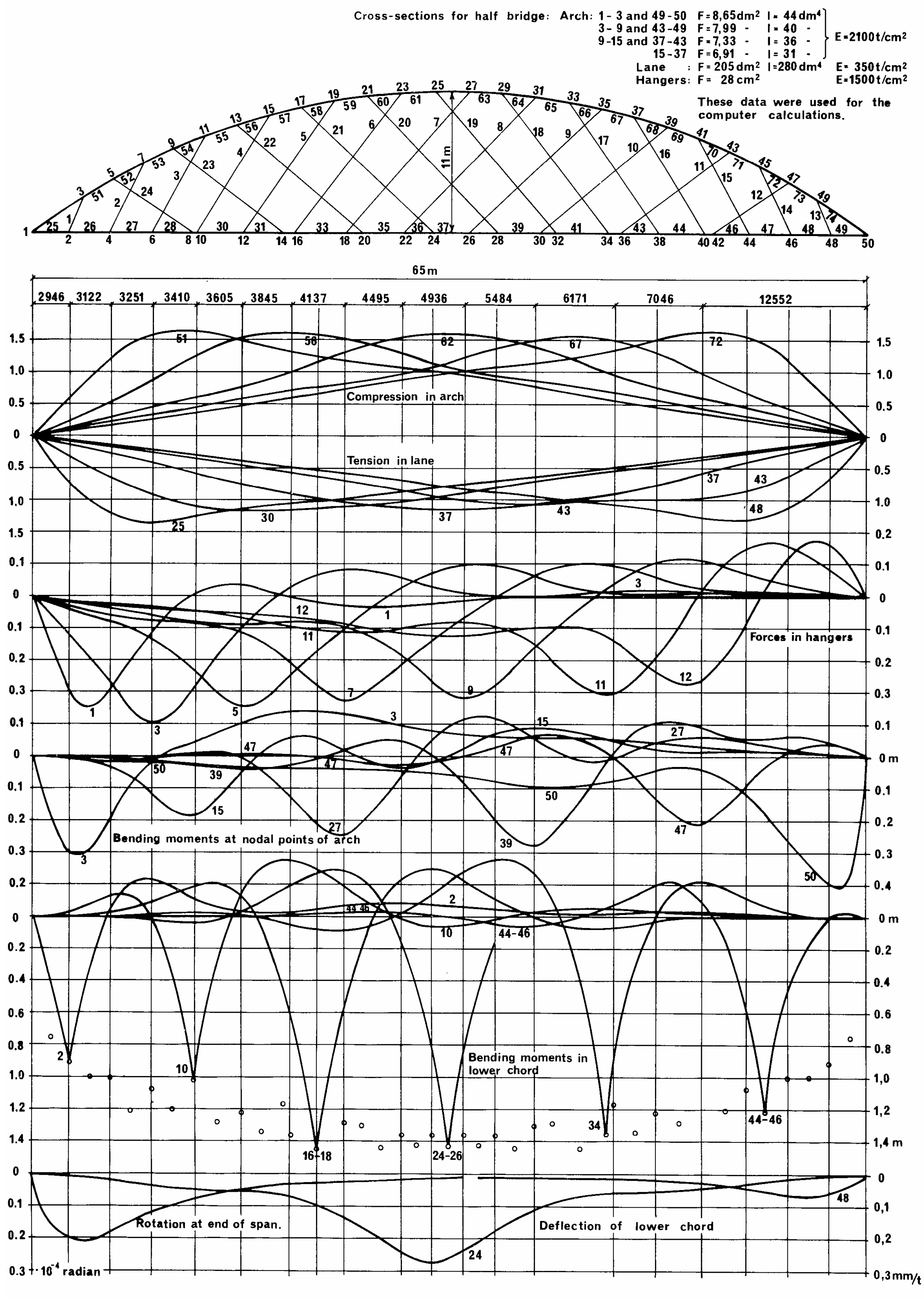


Fig. 41. Influence lines for half of a double track railway bridge. Tveit 1973.

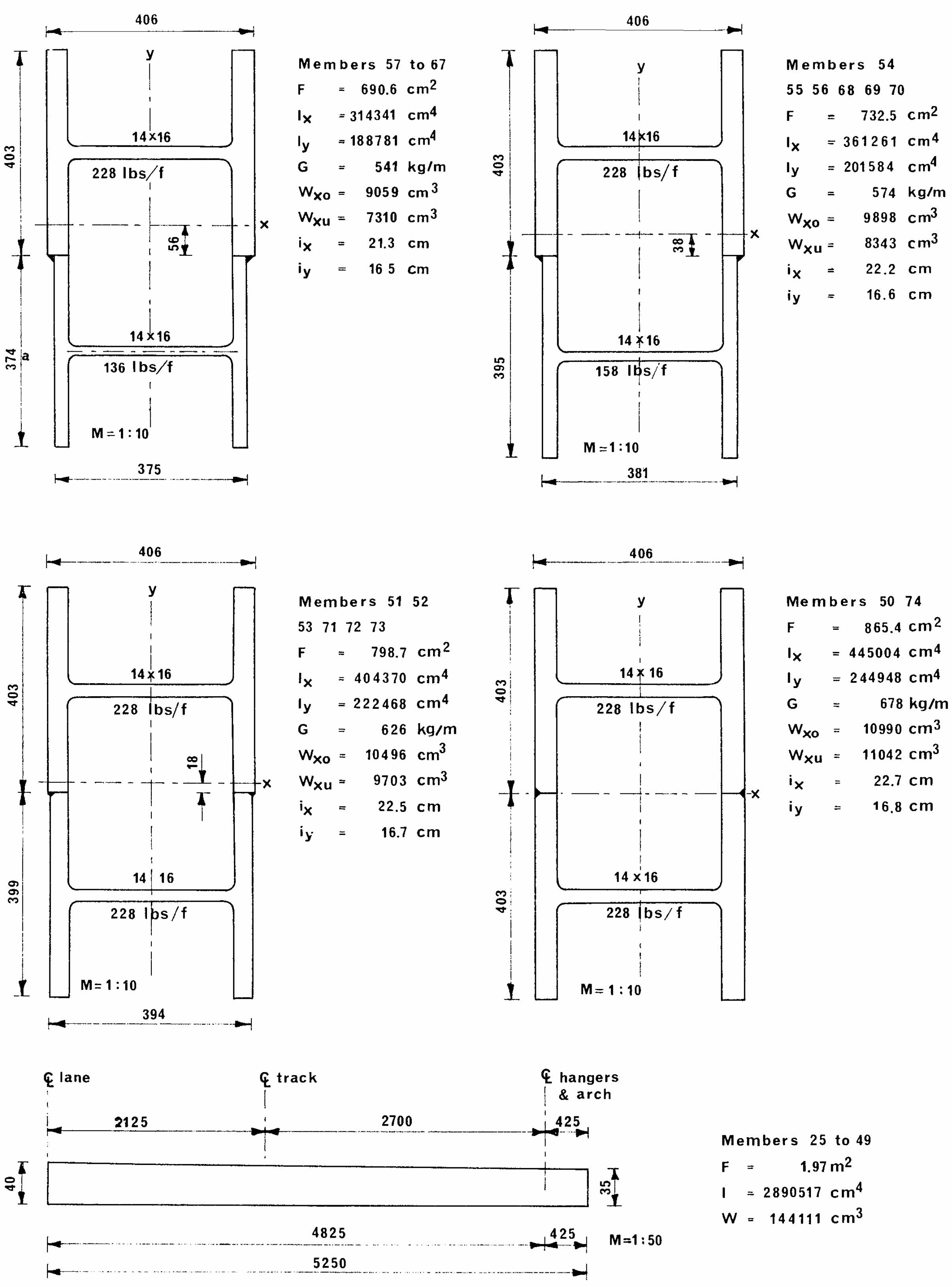


Fig. 42. Cross-section of arch and lane for suggested double track railway bridge. Tveit 73.

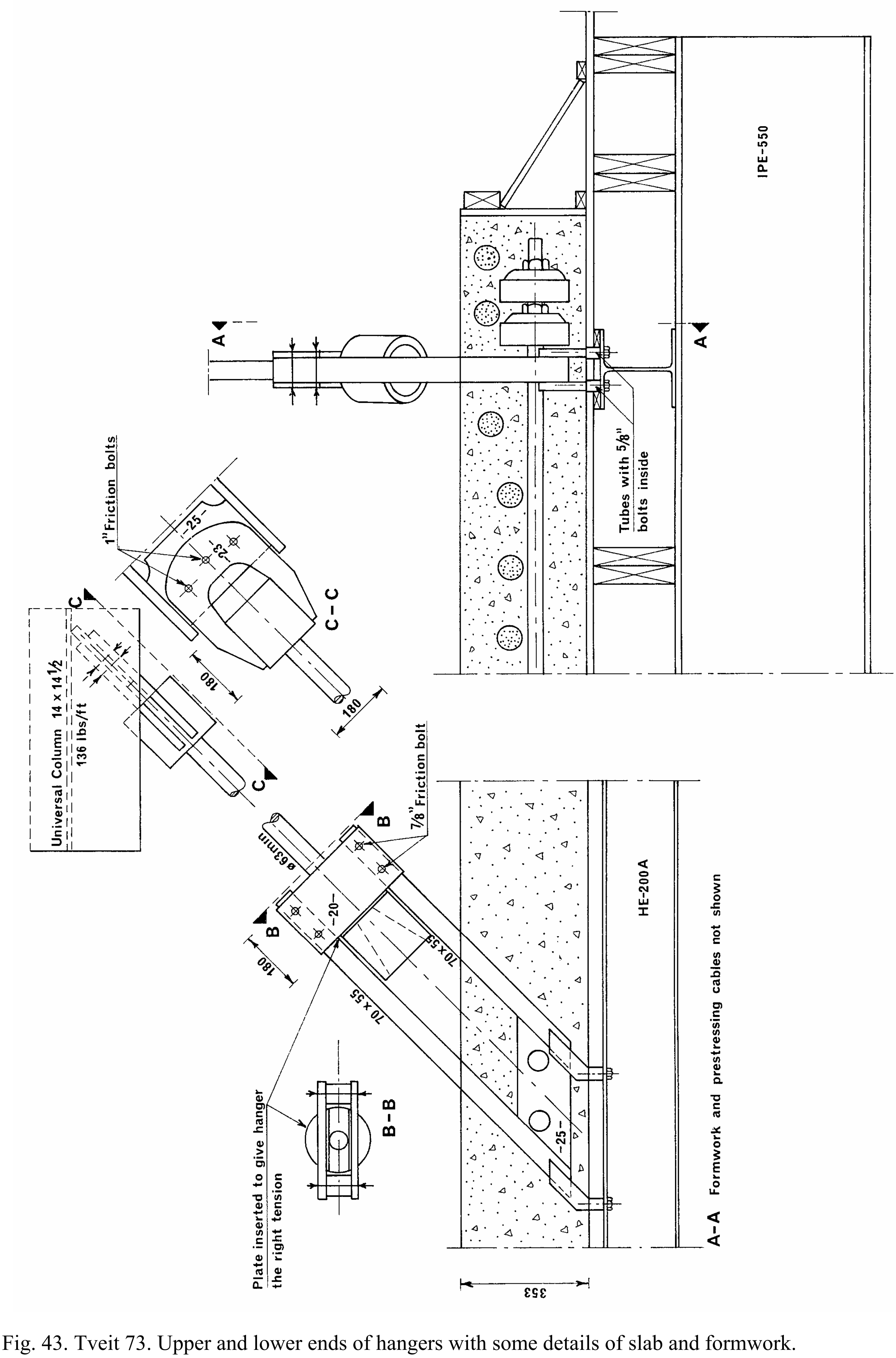


Fig. 43. Tveit 73. Upper and lower ends of hangers with some details of slab and formwork.

The shown arrangement for adjusting the tension in the hangers is not a good idea.

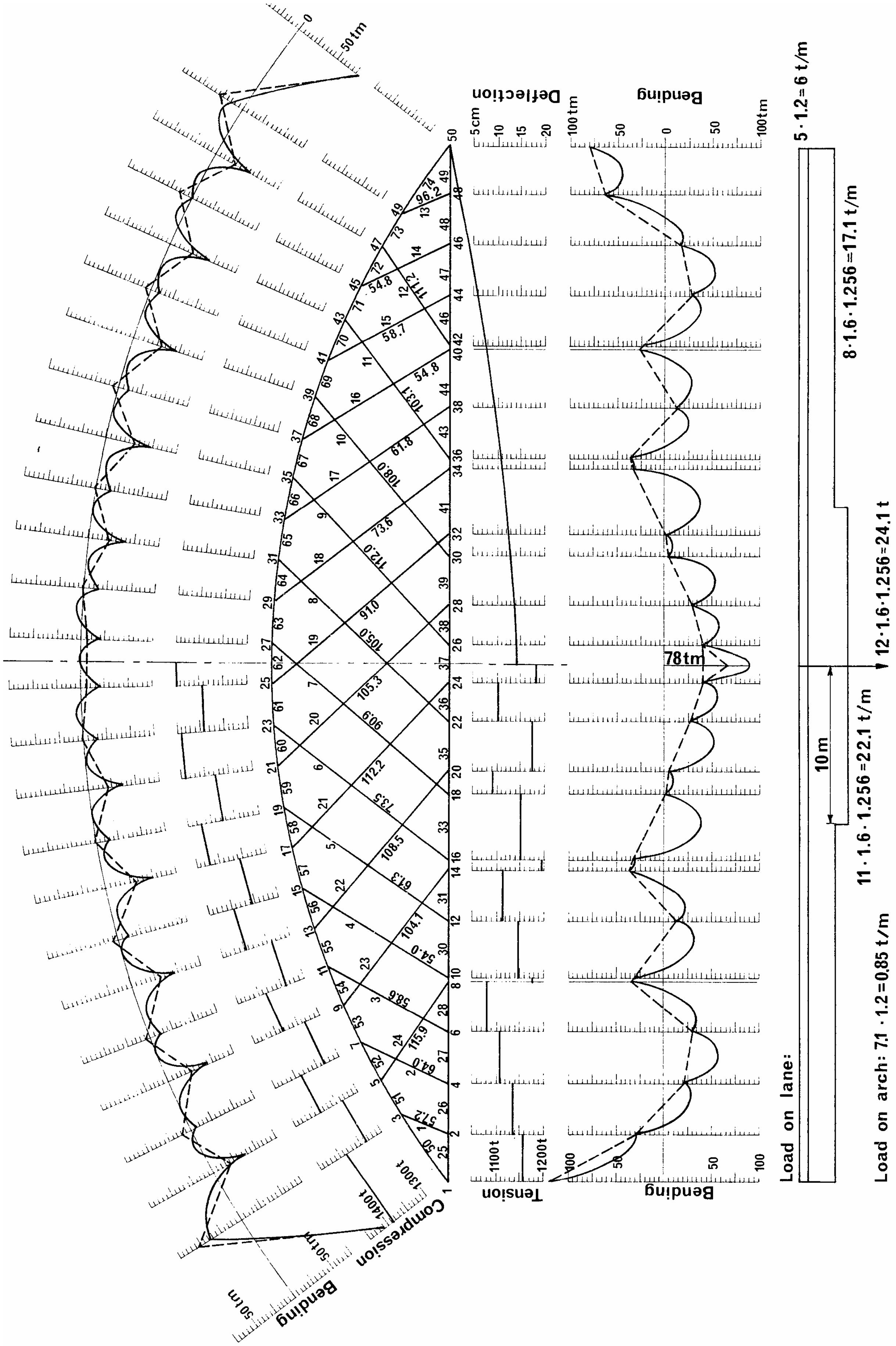


Fig. 44. Forces due to maximum symmetric load. On the right hand side of the diagram bending and hanger forces are also due to 5 mm shortening of hangers 1 and 13. Such a shortening of the last hanger is normally a good idea. It gives less bending in the chords at the ends of the span and a better utilization of the capacity in the hanger nearest to the end of the span.

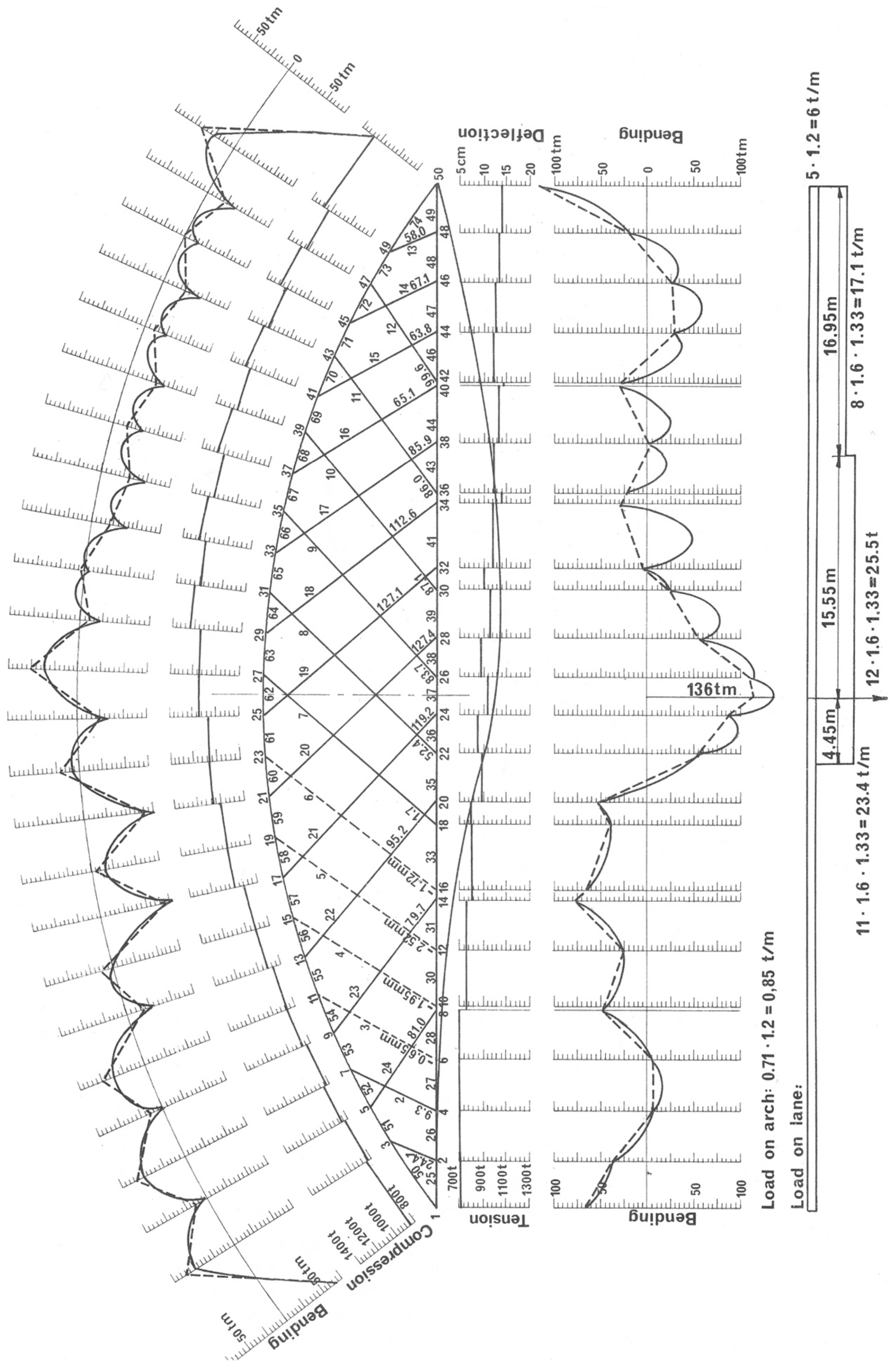


Fig 44a. Forces due to a skew load producing maximum bending in the middle of the bridge

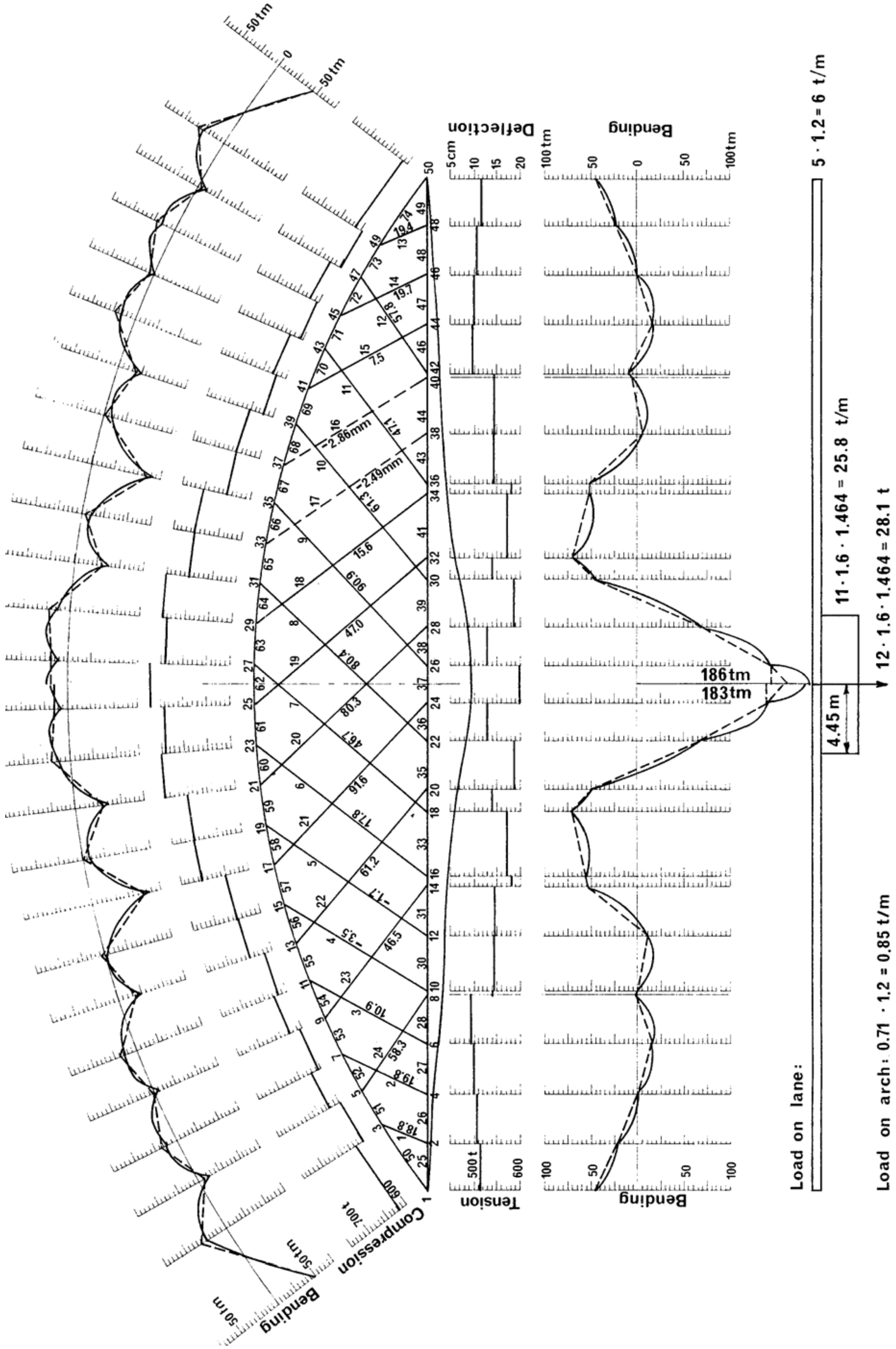


Fig 45. Forces due to the loading that gives maximum bending in member 37. The left side of the diagram gives the forces if the hangers were able to take compression. The right side gives the forces when hangers relax.

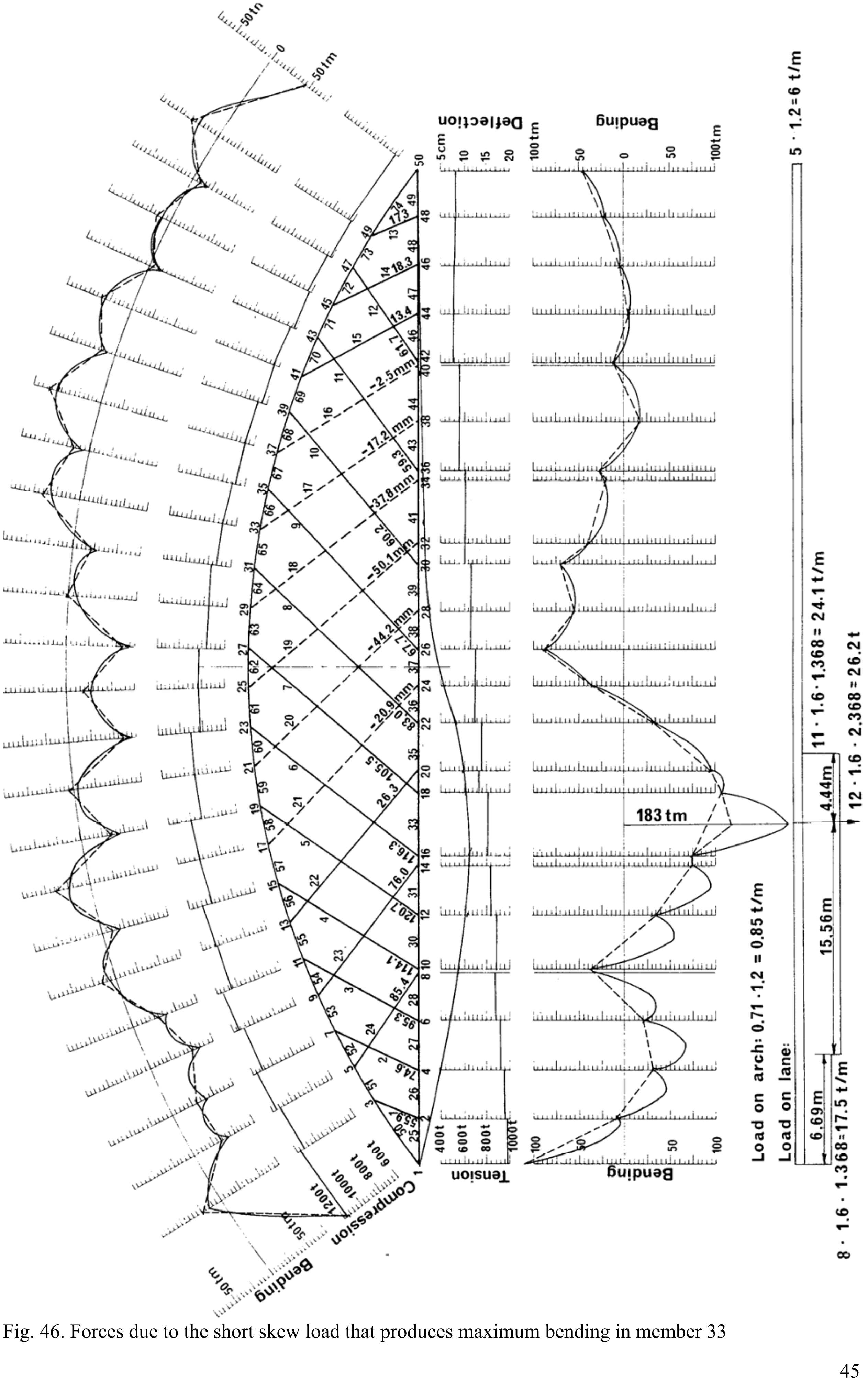


Fig. 46. Forces due to the short skew load that produces maximum bending in member 33

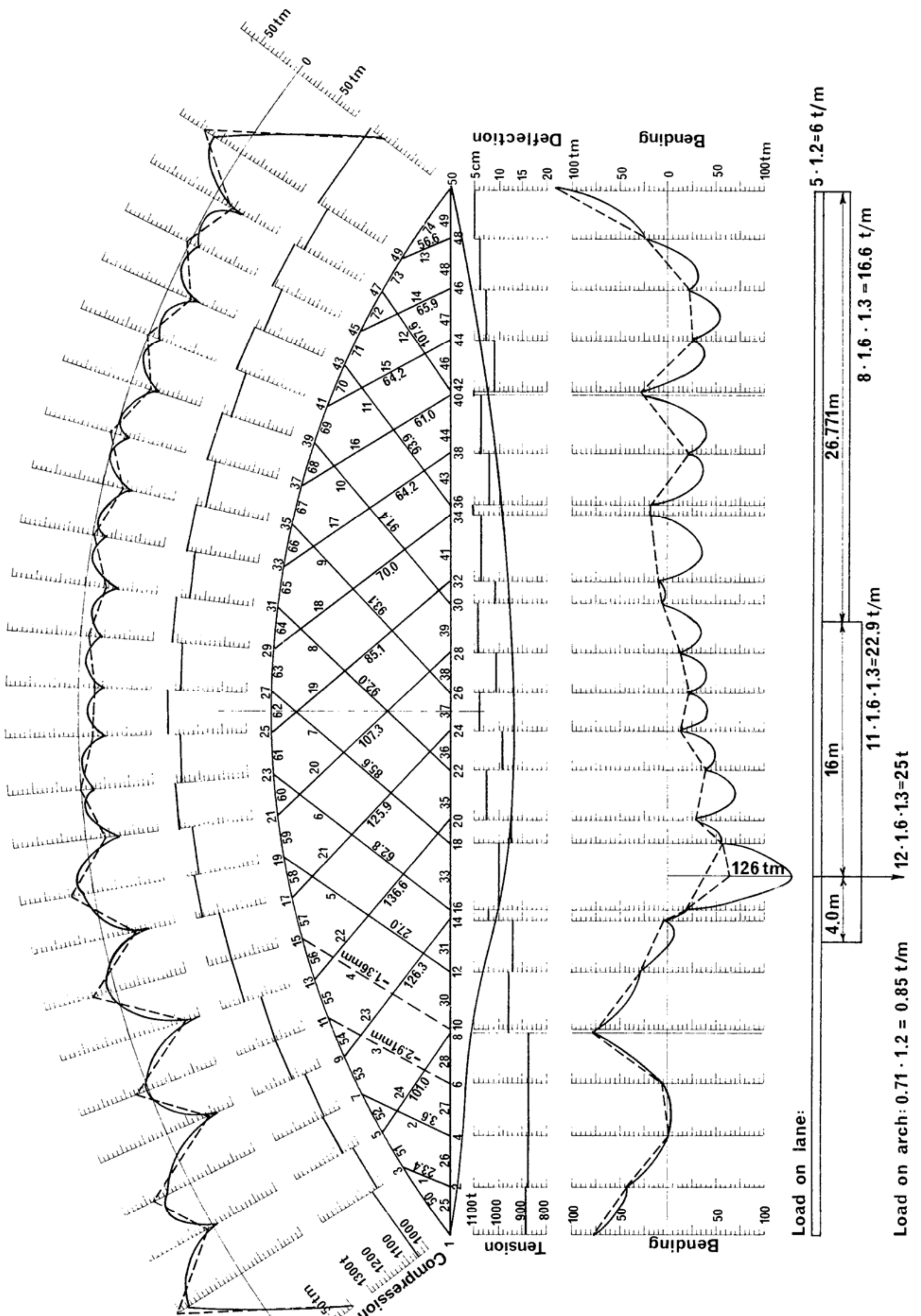


Fig. 47. Forces due to the long skew load producing maximum bending in member 33

NETWORK ARCHES MADE OF PRESTRESSED HIGH-STRENGTH CONCRETE

Mainly for the Rødby-Puttgarden Bridge between Denmark and Germany.

Until 1950 nearly all arch bridges with inclined hangers had concrete arches. This made sense because concrete is good at taking the compression that is predominant in the arch. Later most arches have been made from steel. This has kept the scaffolding cost down and has simplified erection.

Where many equal spans are needed, it might be economical to use concrete in the arches. High strength concrete is efficient at carrying big compressive forces in stocky members. To keep the cost of formwork down, the spans could be cast on land to be moved on to the piers. The drawings in fig. 48 are for a possible network arch spanning 100 metres. Tveit 1980b.

The slab lane and the traffic barriers are cast in one piece. The slab is reinforced in two directions by means of pre-tensioned wire. To cut scaffolding costs, it would probably be best to cast elements of arches with pre-tensioned windbracing on ground level.

The network arch would weigh ~1000 tons and could be moved to bridge piers by means of floating cranes or by two big pontoons. If the lane of the bridge is to be less than 10 metres above sea level, it seems feasible to slide the spans sideways from pontoons to piers. See fig. 2 in fig. 48.

During this sliding process, pontoon and pier must be fastened to each other. The buoyancy of the pontoon must be adjusted to compensate for the shifting of the weight of the span. Finally the hydraulic jacks that can be used for the changing of permanent bearings would be used when removing the steel rail and installing the permanent bearings.

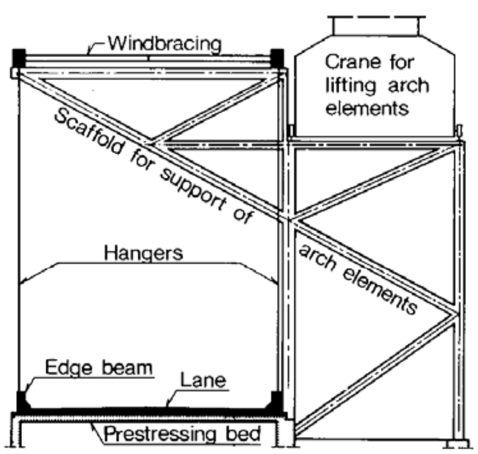


Fig. 1. Cross-sections of rig for casting of the lane, edge beam and joints in arches.

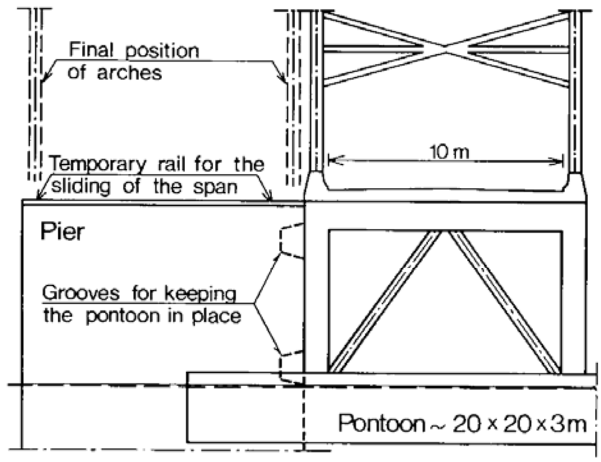


Fig. 2. Pontoon and pier with the span on the pontoon ready for transfer.

Fig. 48. Production site and transport of network arch made of concrete

In 1980 the author, Tveit 1980b, wrote: "For a long bridge the above arrangement would have these advantages: Low weight and a high degree of prefabrication, which would give low labour and material costs and good control of workmanship." Two developments favour this way of doing things. Over the last 25 years available concrete strength has nearly doubled and skills and equipment for moving heavy loads have increased.

The author has made some simple calculations for the bridge from Rødby to Puttgarden. See fig. 49. A similar cross-section has been used by Räck 2003. See page 31d. Calculations for Tveit 1996 have been updated. He does not think that his ideas will be used for this bridge, but he will publish his ideas anyway. Maybe some day somebody will have the good sense to use them in other projects.

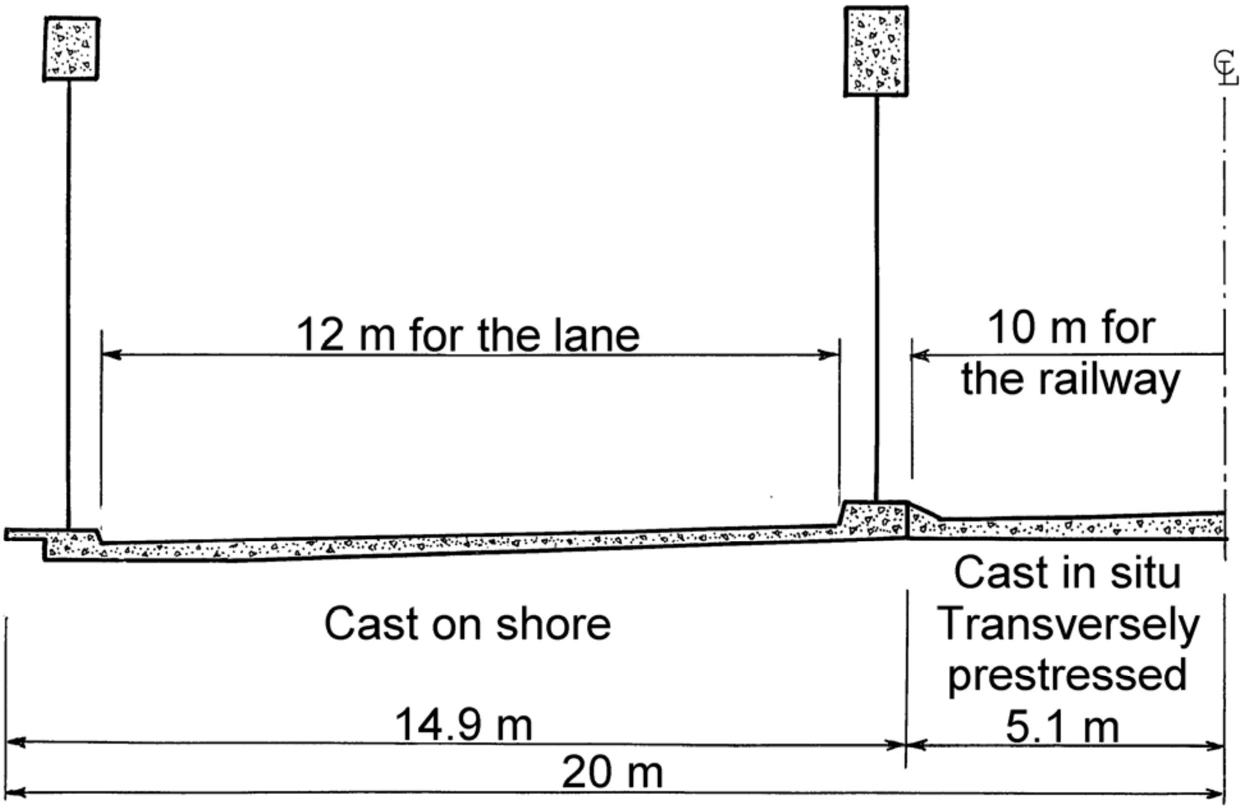


Fig. 49. Cross-section through half a network arch with a span of 215 metres. Tveit 1999.

The bridge will be around 19 km long. For the calculation the distance between the centres of the piers has been assumed to be 220 metres. For the deck a cube strength of 65 or 75 MPa has been assumed. With such concrete strengths the slabs could be made thinner than shown in fig. 49. For the arch cylinder strengths of 94 or 108 MPa have been assumed. Cylinder strengths of 108 MPa can be produced using good granite aggregate and advanced additives.

Two concrete arches with 12 metres of lanes between them are cast on shore. Pairs of arches 18 metres long with the windbracing between them could be cast on ground level. These pairs of arches are joined together by steel plates perpendicular to the axis of the arches. See fig. 19 page 12.

The prefabricated spans can be lifted onto the pillars by cranes or transported to the piers on the pontoons shown on fig. 51 on the next page. Temporarily they are put down on a ledge at the root of the bridge piers. See fig 50. Then they are lifted to the top of the piers up to 65 m above sea level and pulled 1.5 metres sideways on to the corbels for the bearings.

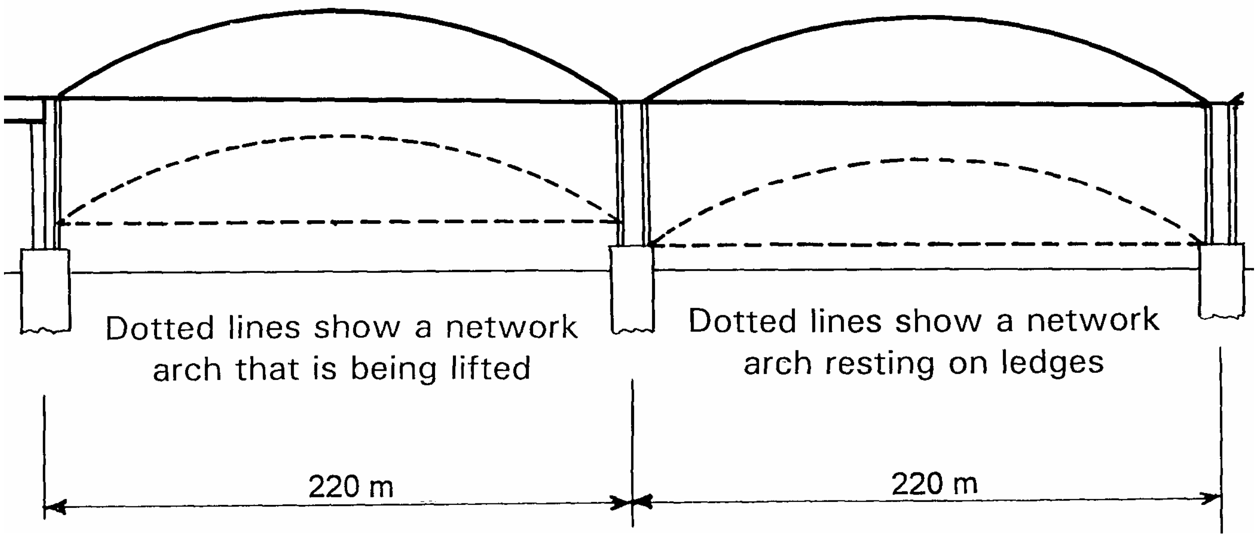


Fig. 50. Two network arches seen from the side. Tveit 1996.

To make it possible to lift the span past the outer corbel, there has to be a cut-out in the plate and the transverse beams at the ends of the spans. When the span is in place, the windbracing above the railway lines is installed. Simultaneously the cut-out in the transversal beams and at the end of the plate is cast and prestressed transversally to the axis of the span.

Then the plate under the railway is cast and prestressed. Finally a plate between the two spans is cast on the top of the pier. The structure under the railway and the windbracing above it could be made of steel. This would give a simpler and faster erection.

Fig. 51 shows various stages in the erection of the spans around a pillar. The first, fourth and a bit of the second quadrant show a pontoon for transporting the spans from the shore to the piers. Each of the pontoons can lift 2000 tons. The pontoons can be joined together for putting the substructure of the span in place. The substructure will probably have to be cast in a dry dock.

The third quadrant shows the contour of a span that rests on a ledge on the pier 5 to 10 metres above sea level. Note the cut-out for lifting the span past the corbel for the outer bearing of the span. The second and the third quadrant show the cross-section of the pillar above the ledge for the transport of the span. The forth quadrant indicates the contour of the crane for lifting the span in place. The first quadrant shows a completed span in place. Only the plate spanning the pier is missing.

The best solution might be to lift each span into place by two big floating cranes.

The Eurocode ENV 1991-3: 1995 gives no limits for deflection of railway bridges spanning more than 120 metres. Preliminary calculations of deflection for the 200 metre span made of prestressed concrete suggest a deflection compatible with very high train speeds. This is hardly surprising because the network arch is very stiff. See influence lines of deflection on pages 57, 58, 60, 72 and 86. Furthermore, the concrete arches are stiffer in the longitudinal direction than steel arches. The concrete tie gives less bending than steel ties because it has greater longitudinal stiffness. Most of the concrete in the tie is in compression all the time.

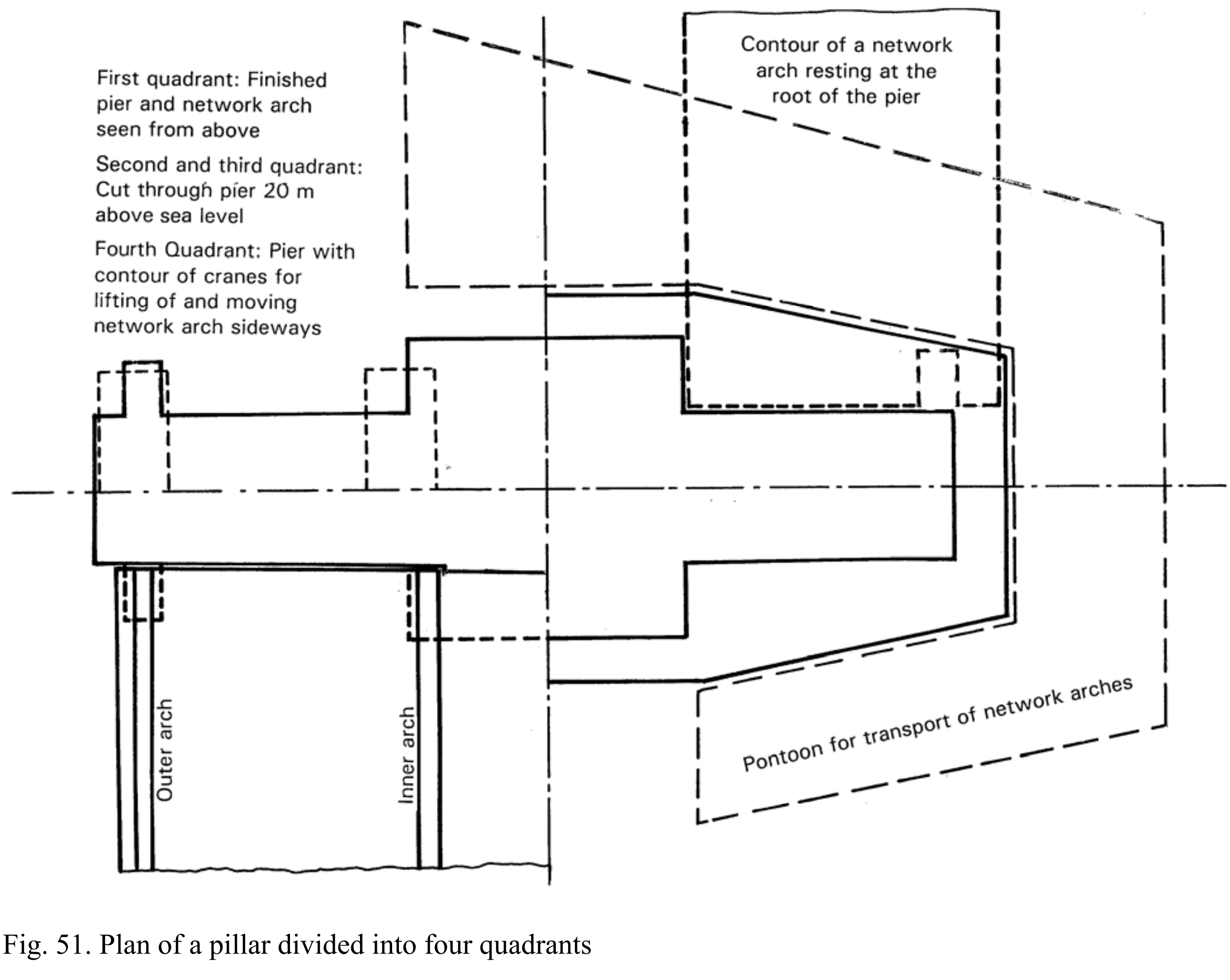


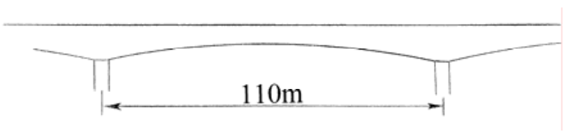
Fig. 51. Plan of a pillar divided into four quadrants

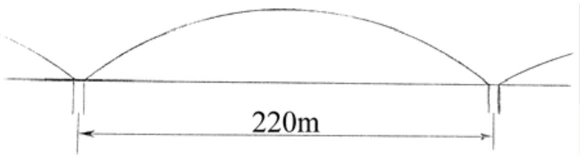
If terrorists want to damage the network arch, they will have to destroy many hangers before a span collapses. See page 22. The arch is so solid that it will be very hard to blow it up, but skilled terrorists with enough explosives can blow up most bridges. - Railings to protect the hangers could be made of steel and be part of the load-carrying structure that is not needed to carry accidental loads. These railings should be easy to repair.

Normally bridges without structural members above the deck are preferable because traffic can derail without damaging the structure. Because of possible savings, the network arch should also be considered for bridges where there is room for the structure under the deck. Possible savings are indicated by the table below where beam and network arch alternatives for the West Bridge in the Great Belt Link, Fries 1990, are compared.

COMPARISON BETWEEN THE BEAM AND NETWORK ARCH ALTERNATIVES

FOR THE WEST BRIDGE IN THE GREAT BELT LINK.





Max. weight of beam element:	5800t	Max. weight of arch to be moved	4000t
Weight of concrete per m:	100%	Weight of concrete per m:	45-55%
Weight of reinforcement per m:	100%	Weight of reinforcement per m:	50-75%
Bridge piers pr. m:	1/110	Bridge piers per m	1/220
Forces on bridge pillars per m:	100%	Forces on bridge pillars per m:	45-55%
Cylinder strength of concrete:	55MPa	Cylinder strength of concrete in tie:	55-65MPa
		Concrete strength in arches:	:80-90MPa
Price of substructure:	100%	Price of substructure:	50-75%
Price of superstructure:	100%	Price of superstructure:	60-80%

Fig. 51a

Savings in price: 25 to 30%

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.

The members can be fastened to anchors glued to the lower outside corner of the slab. They can be stressed through ordinary tensioning or by putting a kind of wedge between the slab and the tension member. The wedges should make the tension in the transverse member counteract some of the bending in the slab due to dead load. The tension members could be steel rods, wires or fibre reinforced polymers. The transverse tension members will strengthen the slab so that bigger vehicles can pass over the bridge.

If the arch is a universal column like in figs 40e and 97 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 in the arch. The bending capacity in the lower chord will also increase, but not so much.

An outside tensile member can strengthen the lower chord, but if such a member has not been taken into account in the design of the bridge it will be difficult to fasten the tensile member at the ends.

Too big deflections in slabs between the arches due to creep and shrinkage can be counteracted in this way: Bands of reinforced polymers can be put under the slab and fastened loosely to the lower ends of hangers. Then something can be put between the bands and the slab. This will increase the tension in the bands and create upward forces on the slab. This can reduce the deflection in the slab without introducing unfavourable stresses.

The joint in the bands could be a weak point. To counteract this the joints should be near the ends of the hangers. Here the tension in the bands is reduced because of friction due to curvature in the reinforced polymer bands. This curvature must not lead to considerable bending in the bands. Since we can counteract unwanted deflection, we can use thinner concrete slabs between the arches. Normally we would hope that the deflections would not become too big.

BUCKLING IN THE ARCHES OF NETWORK ARCHES

Today one would calculate stresses in the arch of the network arch up to the prescribed load using realistic modules of elasticity and usual factors of safety. Realistic deviations from the ideal shape of the arch should be assumed. This makes most of my work on buckling of network arches irrelevant. It is my impression that my designs and conclusions will probably still be roughly valid. Younger engineers can examine if that is so.

Before I leave this chapter I would like to mention that stiffness of the hangers has a great influence on the buckling load. This is a small factor in arguing against steel ropes. Buckling strength of Japanese network arches is examined by Nakai et al. 1995.

ERECTION OF THE STEEL SKELETON ON SIDE-SPANS OF THE SKODJE BRIDGE

Fig. 33 on page 20 shows the proposed Skodje Bridge. Fig. 57 shows a sketch of the steel skeleton for the Skodje Bridge erected on a side-span. The Cuplock scaffolding system has been applied. The scaffolding has intermediate supports that prevent the hangers from sagging. A mobile crane travelling on the side-span is used for the erection. The erection starts nearest to the fjord.

Erection is simpler if the side-spans and the approaching road have the same vertical curvature as the main span. While the steel skeleton is being erected, it might be practical simultaneously to put in the wood of the formwork. The side-span would be a convenient working platform. When the temporary lower chord and the arch and hangers have been erected, the Cuplock scaffolding is removed. Then gaps in the formwork for the concrete lane can be filled in.

The prestressing cables should normally take some of the tensile force in the tie during the transport of the steel skeleton. It is easier to put in some reinforcement before the prestressing cables are put in. How much reinforcement should be put in while the steel skeleton rests on the side-spans depends in part on the size of the pontoon which is to move the steel skeleton over the fjord. After the span has been lifted in place and the arch is made continuous across the fjord, an expansion joint is opened above a column at one end of the arch. See fig. 33 page 20.

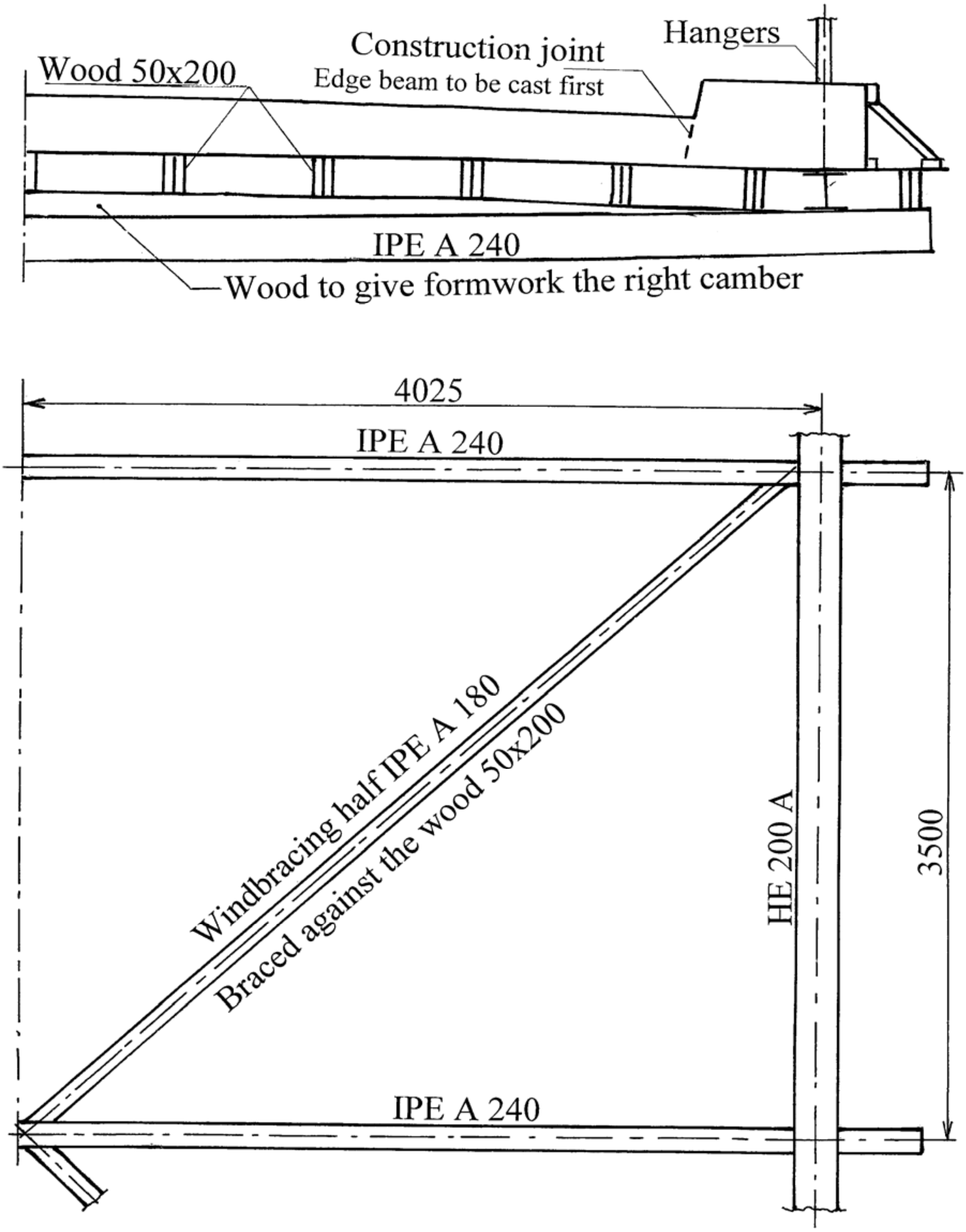


Fig. 56. Half tie and temporary lower chord for the Skodje bridge in fig. 33.

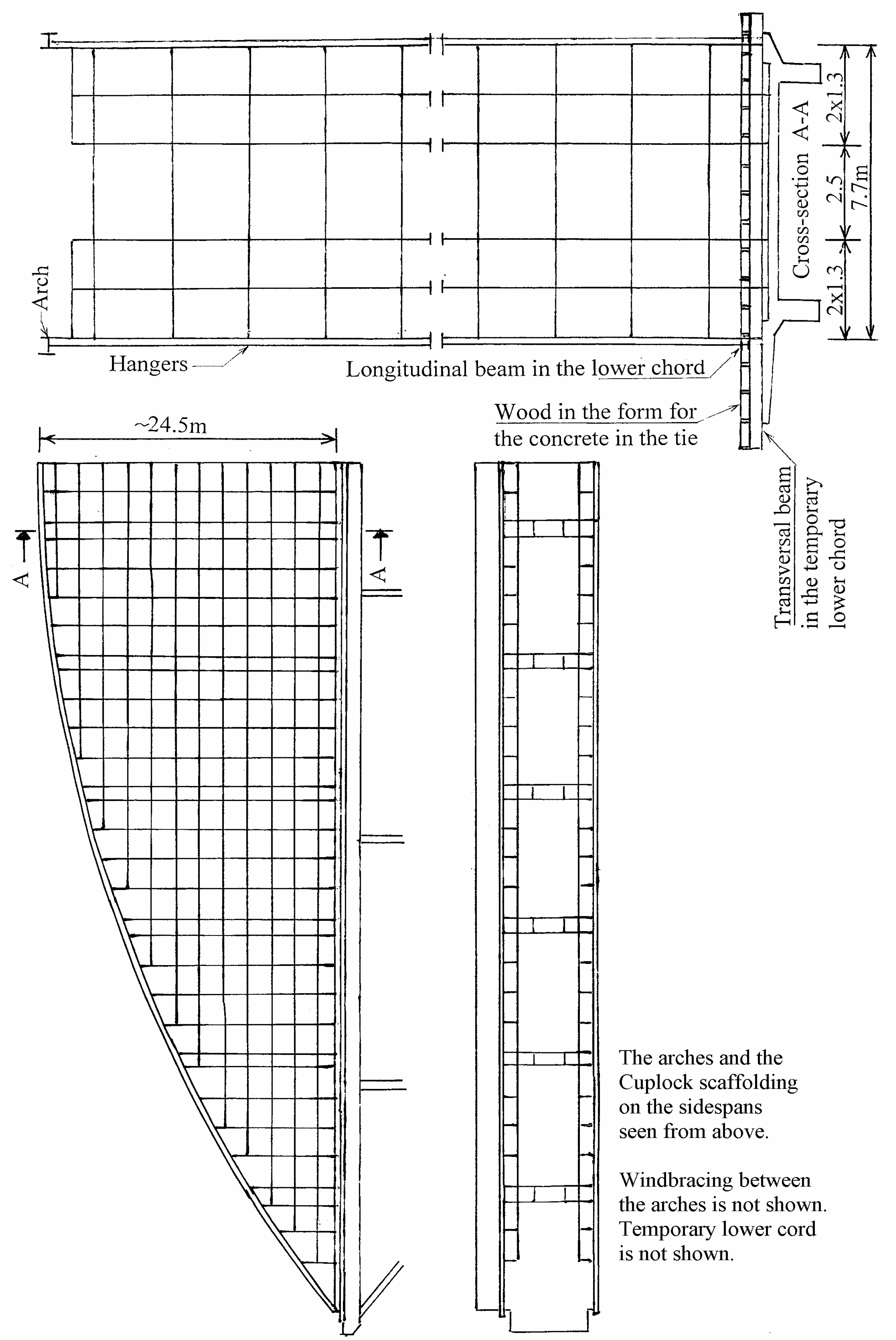


Fig. 57. Scaffolding for the erection of a steel skeleton on a side-spans of the Skodje Bridge. See fig. 33.

REMOVING THE TEMPORARY LOWER CHORD OF THE SKODJE BRIDGE

The temporary lower chord of the proposed Skodje Bridge is shown in fig. 56. Fig. 57 shows the scaffolding for the erection of the steel skeleton for the Skodje Bridge on the approach. Fig,. 22 on page 12 shows how the lower end of a hanger is fastened to the temporary lower chord. The purpose of the rest of this chapter is to present a simple method of removing the formwork of the lower chord. This method can be compared to other methods of removing the formwork.

It deserves a mention that during his lecture tours the author has often encountered the opinion that making the temporary lower chord a permanent part of the tie might save work. The advantages of the temporary lower chord are explained on page 30a.

Before the step by step explanation of the removal of the temporary lower cord with formwork, a short summary will be presented:

At the start 4 columns over 3 metres long are fastened to the ends of two extra long transversal beams in the temporary lower chord. See fig. 59. Then the connections at the ends of the longitudinal beams above the transversal beams are removed. The screws between these longitudinal beams and the lower ends of the hangers must also be removed.

Now the two extra long beams are forced down till the wheels at the top of the columns at their ends are resting at the edges of the concrete tie. See fig. 59. Over 5 metres of the longitudinal beams and the wooden form above them follow the long transversal beams down. This makes a wooden platform that is used when removing the temporary lower chord. By being rolled on the wheels seen in fig. 59, the platform can be moved along the edges of the concrete slab. The platform must have a solid wooden fence.

<u>Steps in removing the formwork and temporary lower chord after the lower chord has been cast.</u>

1. The wooden form at the edge of the concrete slab is removed around two extra long transversal beams in the temporary lower chord. These beams are in the middle of the span. Here the shear force due to wind during the casting of the concrete tie is so small that it can be taken in the plywood of the formwork.

2. The extra long transversal beams will carry over 5 metres of the temporary lower chord and its formwork. This formwork above the extra long transversal beams will be part of the platform for removing the rest of the lower chord.

3. The longer transversal beams are about 2.5 metres longer than ordinary transverse beams in the lower chord. Ordinary longitudinal wooden beams, 50x200 mm^2, see fig. 58, rest on the transverse beam. The formwork on top of the wooden beams is mostly plywood plates 14x500x1500 mm^3.

4. Fig. 59 shows part of a cross-section of the wagon for removing the temporary lower chord.

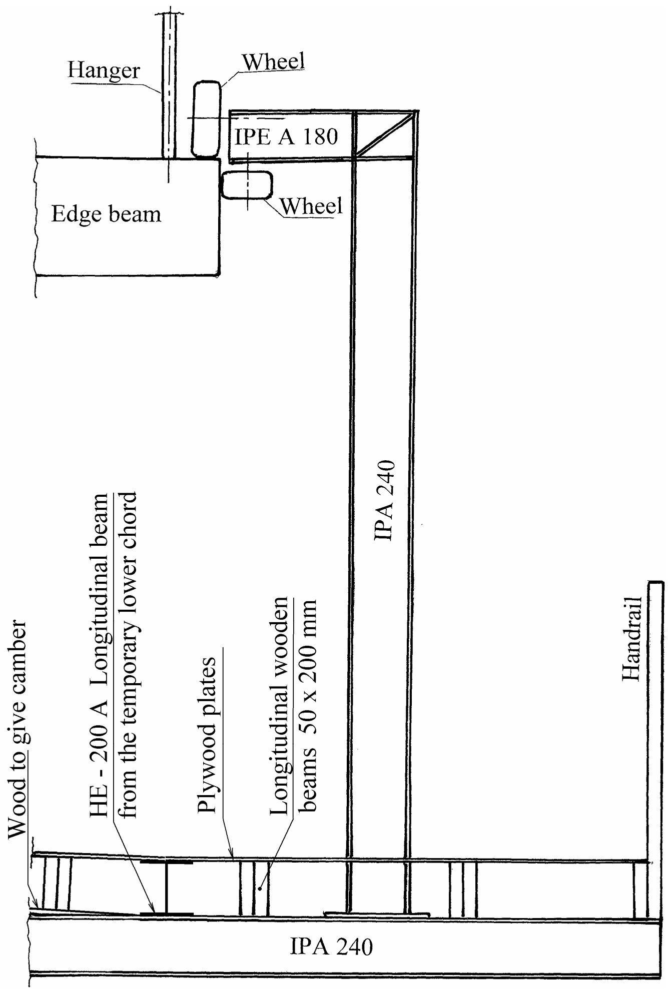


Fig. 59

5. Four columns over 3 metres long are fastened to the extra long transversal beams. At the top of the columns there are wheels that roll at the edge of the concrete slab. See fig. 59. These columns support the platform for removing the temporary lower chord.

6. Before the platform can be lowered, someone in a basket at the end of a movable arm must go down under the concrete slab and remove the screws at the ends of the longitudinal beams just above the two long transversal beams. Screws between the hangers and the longitudinal beams in the platform must also be removed. See fig. 22. The removal of screws can also be done from a wagon travelling on the lower flanges of the long transversal beams.

7. When the screws mentioned in step 6 have been removed, the two extra long transverse beams can be lowered. To make sure that the longitudinal beams with the wooden plate above follow the transversal beams down, the wooden plates must be well greased or there must be a plastic film on top of the wooden plates that are to be part of the platform for removing the lower chord.

8. In the platform for removing the temporary lower chord there must be many nails between the plywood plates and the longitudinal wooden beams. There must also be many nails in a piece of wood on top of the transverse steel beams that give the concrete slab the right camber. This piece of wood must be securely fastened because it has to take up some of the bending moment in the transverse beams.

9. Fig. 60 shows joints in the longitudinal wooden beams. These joints allow the plywood and the wooden beams, to which it is fastened, to move down with the rest of the platform.

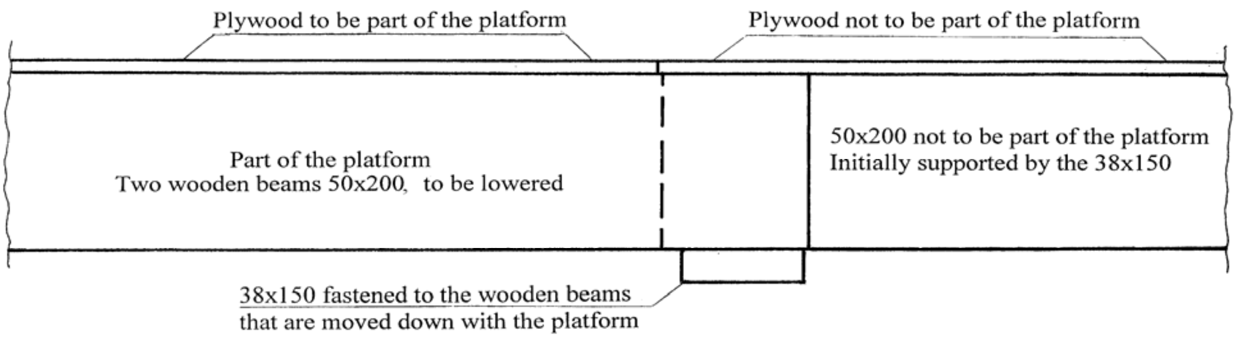


Fig. 60. Joint in wooden beam that lets the platform for removing the temporary chord be lowered

10. The platform for removing the lower chord is lowered so that the wheels at the top of the columns roll on the edge of the concrete slab. See fig. 59.

11. Next to the columns are ladders to facilitate access to the platform after it has been lowered.

12. Before the removal of the rest of the temporary lower cord, a solid railing must be put up around the platform that has been lowered. At both ends of the platform this railing reaches almost up to the temporary lower chord to give the workers protection from beams and formwork torn loose from the temporary lower chord. See fig. 61.

13. Jacks fastened to the lower ends of the hangers above the lane can move the platform. No personnel are allowed on the platform when it is moved.

14. Now to the actual removal of the temporary lower chord.

15. Members of the windbracing weigh 30 kg. Their screws in the transverse beams are removed. Then the members are removed by hand.

16. The temporary transverse beams with the wood that gives camber weigh around 300 kg. When the transverse beams are loosened from the longitudinal steel beams and the windbracing, they are free to be removed.

17. The transverse beams have holes in the ends for fastening a rope so they can swing out to the edge of the bridge. Later they are lifted on to the bridge deck and taken away by trucks.

18. The longitudinal beams in the lower chords could be made up of beams that weigh up to 400 kg. When their longitudinal joints are undone and the longitudinal beams are loosened from the lower end of the hangers, ropes are fastened to their ends.

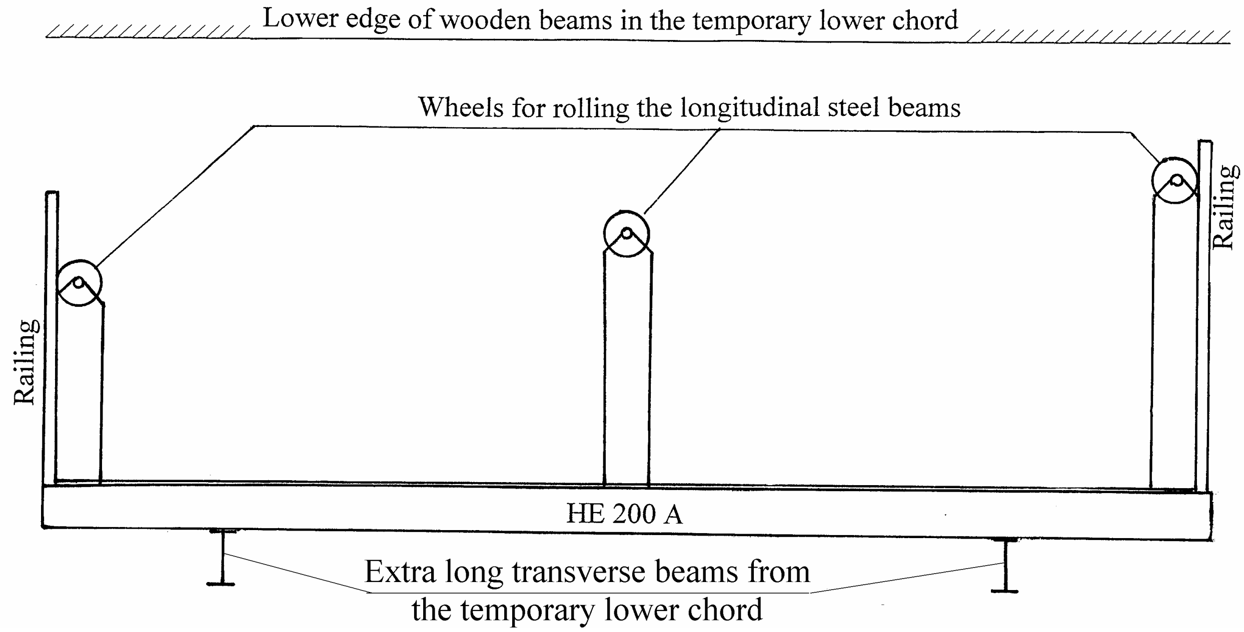


Fig. 61. Longitudinal cross-section of the platform for removal of the temporary lower chord

19. The longitudinal steel beams roll across the platform on wheels on columns on the platform. See fig. 61. After they glide off the platform, they hang suspended at the edge of the bridge till they are lifted and loaded on to trucks.

20. The wood above the transverse beams is loosened by hand and brought to the outer edge of the platform and lifted on to the roadway. The longitudinal wooden beams weigh around 25 kilograms.

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

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

ERECTION OF NETWORK ARCHES IN COSTAL AREAS AND WIDE RIVERS

In coastal areas network arches can be lifted in place by big cranes. See also p. 29k.

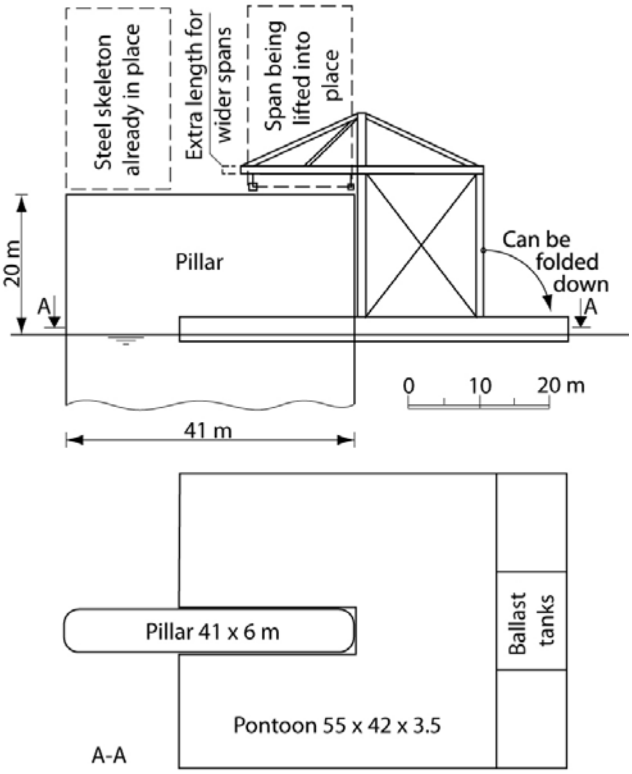


Fig. 61b. Crane for erection of optimal network arches in wide rivers

The crane shown in fig. 61b could be built to erect network arches in wide rivers. It could 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. 61b is shown erecting one half of the steel skeleton for a combined road and rail bridge with spans of 160 m. Rack 2003. The bridge has four arches. Each of the two steel skeletons shown in fig. 61b weighs ~1000 tons.

When both steel skeletons in fig. 61b 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. For tall pillars pontoons and cranes on top of the pillars would probably be a better idea.

NIELSEN’S BRIDGES

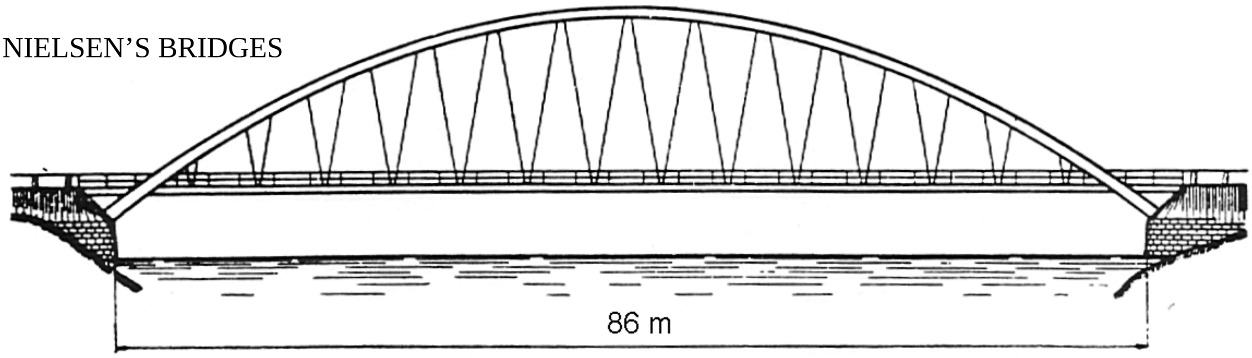


Fig. 61a. Bridge over Øster-Dalelven in Sweden. Design of this bridge started in 1921.

The Nielsen bridges are the forerunners of the network arch. See fig. 61a. They were developed by Octavius F. Nielsen who worked for a Swedish subsidiary of Christiani & Nielsen in Copenhagen. Around 60 of these bridges were built in Sweden between the two World Wars. Ostenfeld 1976 p. 219. Four of these bridges were shown in the doctoral thesis of O. F. Nielsen which was handed in in 1929. Nielsen 1930.

Some of the hangers were meant to relax due to live load on part of the span. The slope of the hangers made the bending moments in the arches almost negligible. The hangers were steel rods anchored in the concrete arches and in the middle of the longitudinal concrete beams in the tie. See fig. 61b.

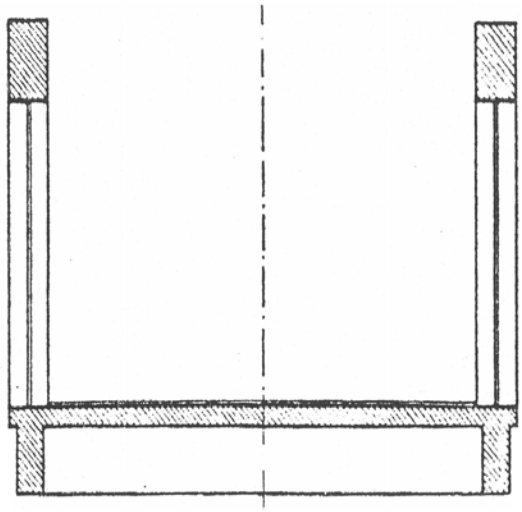


Fig. 61c. Cross-section of a Nielsen Bridge

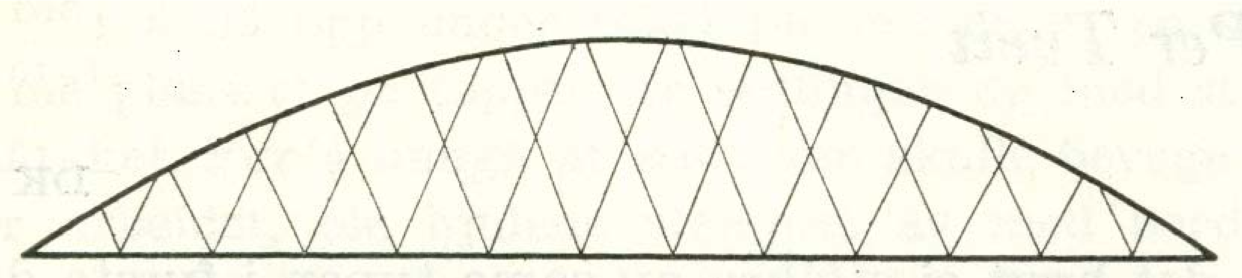


Fig. 61d. Bridge from Nielsen's patent 1925.

Over the years vehicle weight and speed have increased, but hangers breaking has not been a problem. See p. 22. How Nielsen's bridges were calculated is explained at length in his doctoral thesis. Nielsen 1930. The calculations were simpler when all hangers had the same slope and the bridge probably looked better. In the Nielsen bridges that were built no hangers crossed each other. It was probably not found to be necessary, but in the patent from 1926 some hangers crossed each other. See fig 61d.

In his master's thesis this author found that slimmer chords, stronger materials and bigger loads make it advantageous to let some hangers cross each other at least twice. In Japan bridges like the one in fig. 27 are called Nilsen-Lohse Bridges. The author would be inclined to call them network arches, but he does not think that they are optimal.

A famous German professor wrote that he could not understand why Nielsen had stopped halfway and not let the hangers meet at the arch. There was a point that he failed to see. See page 8.

Fig. 61e. The Castelmoron Bridge built in France in 1933

The longest Nielsen bridge was built in France in 1933. See fig. 61e. It had a span of 145 m.

TROUBLE WITH LOWER END OF HANGERS COMING OUT OF THE CONCRETE TIES.

Bridge authorities are reluctant to accept lower ends of hangers that come directly out of the concrete like in fig. 6, 6a 6c,, 43 and 100. These lower ends of hangers can not be changed. The main trouble is corrosion just above the concrete. The cement prevents corrosion in the concrete. The prestress in the ties of optimal network arches makes this is extra likely to be so. Often salt is used in the winter to remove snow. When the snow melts a layer of salt is left on the steel just above the concrete tie. This encourages corrosion.

Careful maintenance can counteract the problem. It has not been a problem with the Steinkjer Bridge. On the Bolstadstraumen Bridge some of the corrosion protection can be seen through the covering paint. The person responsible refused to renew the paint cover just above the concrete till all corrosion protection was to be overhauled in five years time.

What can be done? There are two groups of remedies. 1. Extra careful corrosion protection of the steel just above the concrete. 2. Avoiding fastening the hangers directly to the steel.
1. A. More frequent upkeep of the corrosion coating that is used on all the steel.
1. B. Extra durable corrosion coating just above the concrete. For instance: A plastic coating that also covers the concrete nearest to the steel rod.
2. A. Fastening the lower end of the hangers to rods through the tie. Suggested for the Brandanger Bridge on page 94. That drawing is not shown here.
2. B. Fastening the lower end of the hangers to steel beams under the tie. This would increase the area of steel that could not be replaced and would have to be given good corrosion protection.

Wondering how the lower end of the hangers in the Nielsen Bridges had been surviving I asked the Swedish bridge office in Borlänge. They answered that they had had trouble with nearly half the hangers that came directly out of the concrete and were keen to avoid such designs.

The author still prefers hangers that come directly out of the concrete.

The influence lines and other information on network arches calculated by the author serve two main purposes. One is to give readers a chance to check the author's statements on network arches. The other is to help engineers who want to design network arches choose suitable dimensions. http://pchome.grm.hia.no/~ptveit/

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. It is much more important to know the hangers' tendencies to become relaxed. Therefore this chapter and pp. 26 to 29j provide information that can be of help when searching for skeleton lines that lead to acceptable tendencies for the relaxation of hangers.

When transferring values from one span to another, general model laws apply. See for instance Maier-Leibnitz 1941. Here values of the span to be calculated are in parentheses. n is the ratio between the lengths of the calculated span and the span to be calculated. m is a factor that can be chosen freely. It indicates dimensions perpendicular to the plane of the arch. To get to the new span (a model) from the span that has been calculated the following equations can be used:

Length	$(L) = L/n$	Cross-section	$(A) = A \cdot E/n^2 \cdot m \cdot (E)$
Stiffness	$(I) = IE/n^4 \cdot m \cdot (E)$	Nodal load	$(P) = P/n^2\, m$
Bending moment	$(M) = M/n^3\, m$	Deflection	$(f) = f/n$

In the winter when the days are short, one only needs to remember that the ordinates of the influence lines for axial forces are independent of the span. Ordinates of the influence lines for bending moments are proportional to the length of the span.

The dimensions of the lower chords in the Steinkjer Bridge can be found in fig. 6a on page 6.

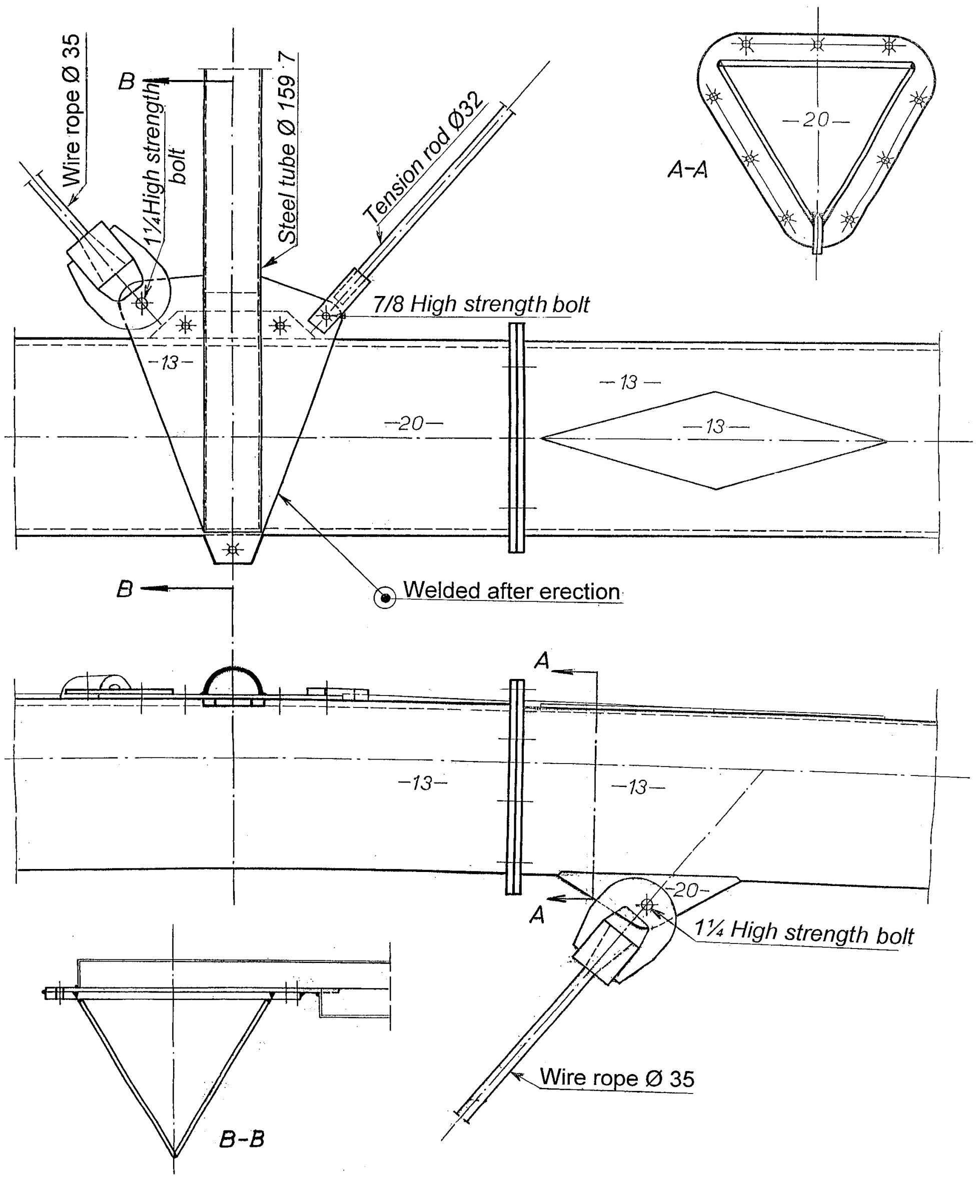


Fig. 62. Structural details around the second tube in the windbracing of the network arch in Steinkjer

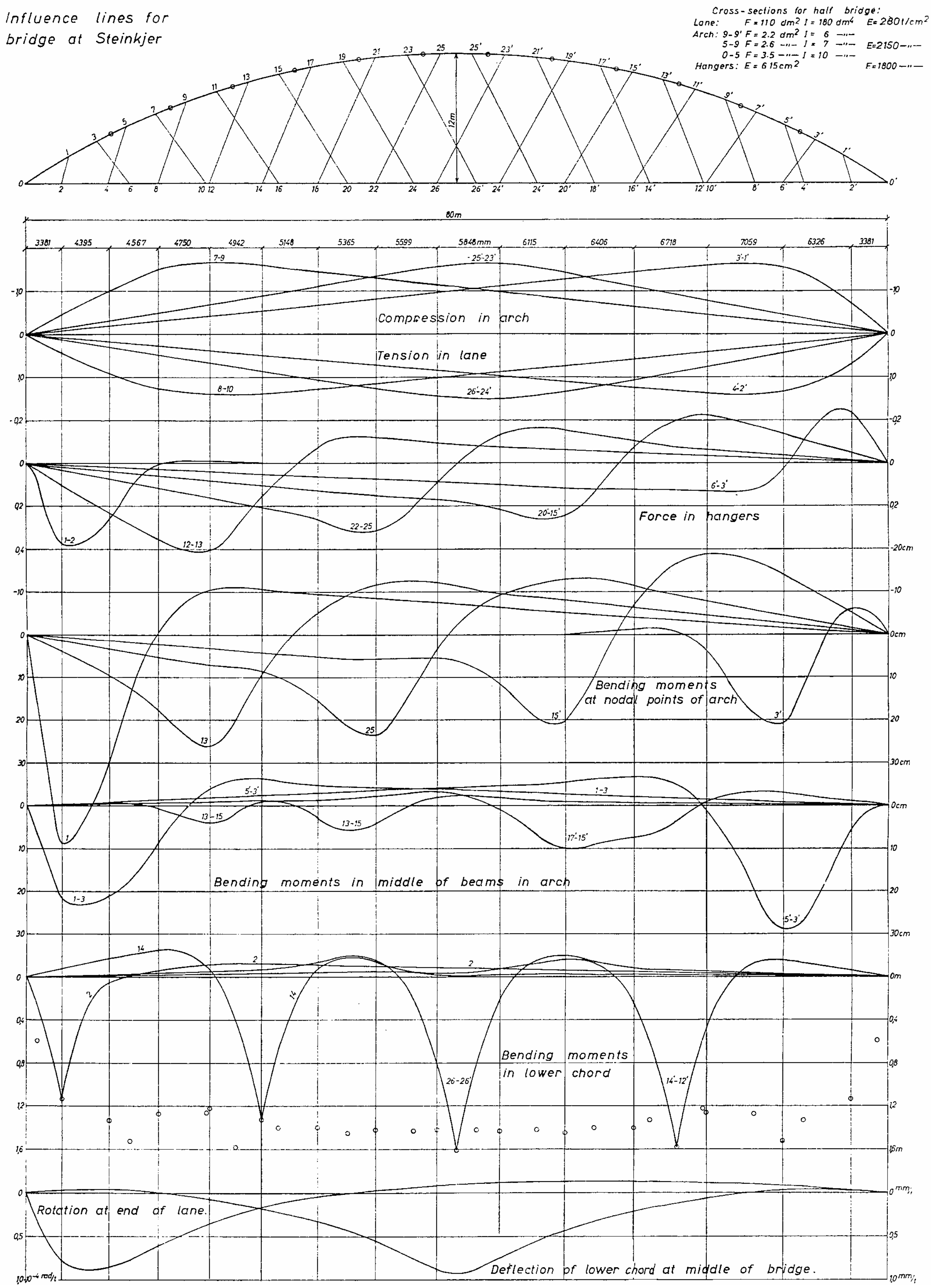


Fig. 63. Typical influence lines for the bridge at Steinkjer. The difference in slope between two adjoining hangers is 1.8°. The slope of the steepest hanger is 74.4°.

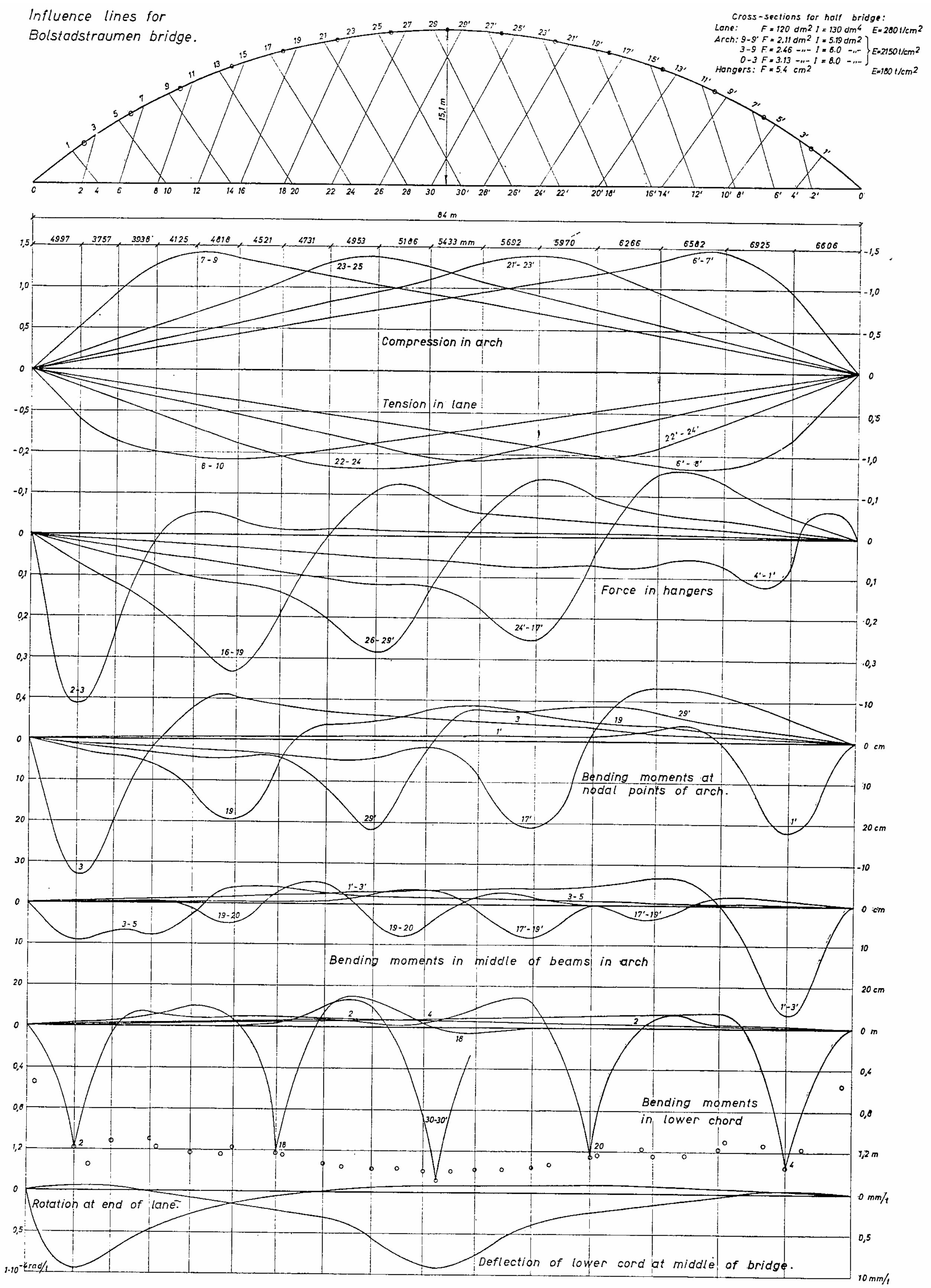


Fig. 64. Typical influence lines for the bridge over Bolstadstraumen. The difference in slope between two adjoining hangers is 1.7°. The slope of the steepest hanger is 73.8°.

VIENNA 1980

Two network arches both spanning 200 metres were calculated for the IABSE congress in Vienna in 1980. Tveit 1980a and 1980b. For the calculation of these network arches an optimisation program called FEMOPT was used. Mr. Georg Andersen designed it in partial fulfilment of his licentiate degree. Andersen 1979.

The design called 200A, see pages 60 to 68, has steeper hangers than the design called 200B. Pp. 69 to 72 give information on the dimensions and geometry of 200B. 200A and 200B have nearly the same cross-sections. From the influence lines in figs 65 and 77 it can be seen that the steeper hangers in 200A give smaller bending moments, but less resistance to relaxation of the hangers.

The skeleton lines for 200A have the same shape as the skeleton lines for the network arch for the Åkvik Sound. The difference is a factor 135/200, which is the ratio of the spans. The good thing about the skeleton lines presented in this publication is that they give very even maximum stress in the hangers. This is to some extent load dependent, but for spans between 150 and 200 metres little can probably be gained by examining other skeleton lines.

Most of the symbols in the diagrams will be immediately understandable to an experienced structural engineer. The stress index in fig. 71 on page 66 might be an exception. It is the stress in the member found by Navier's formula divided by the nominal yield stress. The stress index is calculated in five places in the member. The highest stress index has been given as output.

The bending moment diagram in fig. 71 has thick and thin lines. The thin lines are the result of the first calculation which is a standard linear-elastic calculation, carried out assuming that hangers take compression etc. The thick line is calculated by non-linear methods assuming sagging hangers etc. When the difference between two lines is small, only the thick line has been drawn.

Vienna 200A

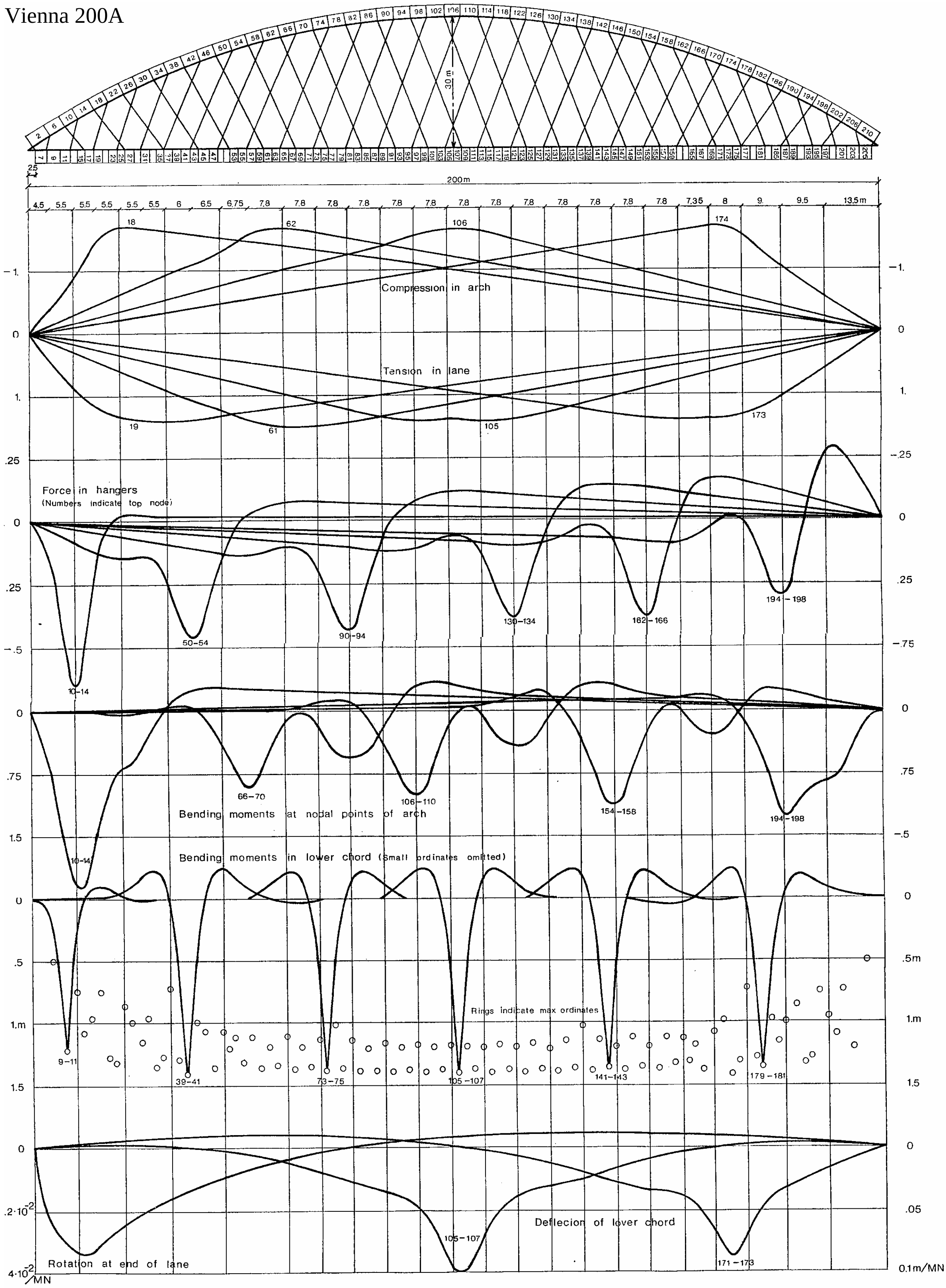


Fig. 65. Typical influence lines for a network arch spanning 200 m designed for the IABSE Congress in Vienna 1980. Tveit 1980a.

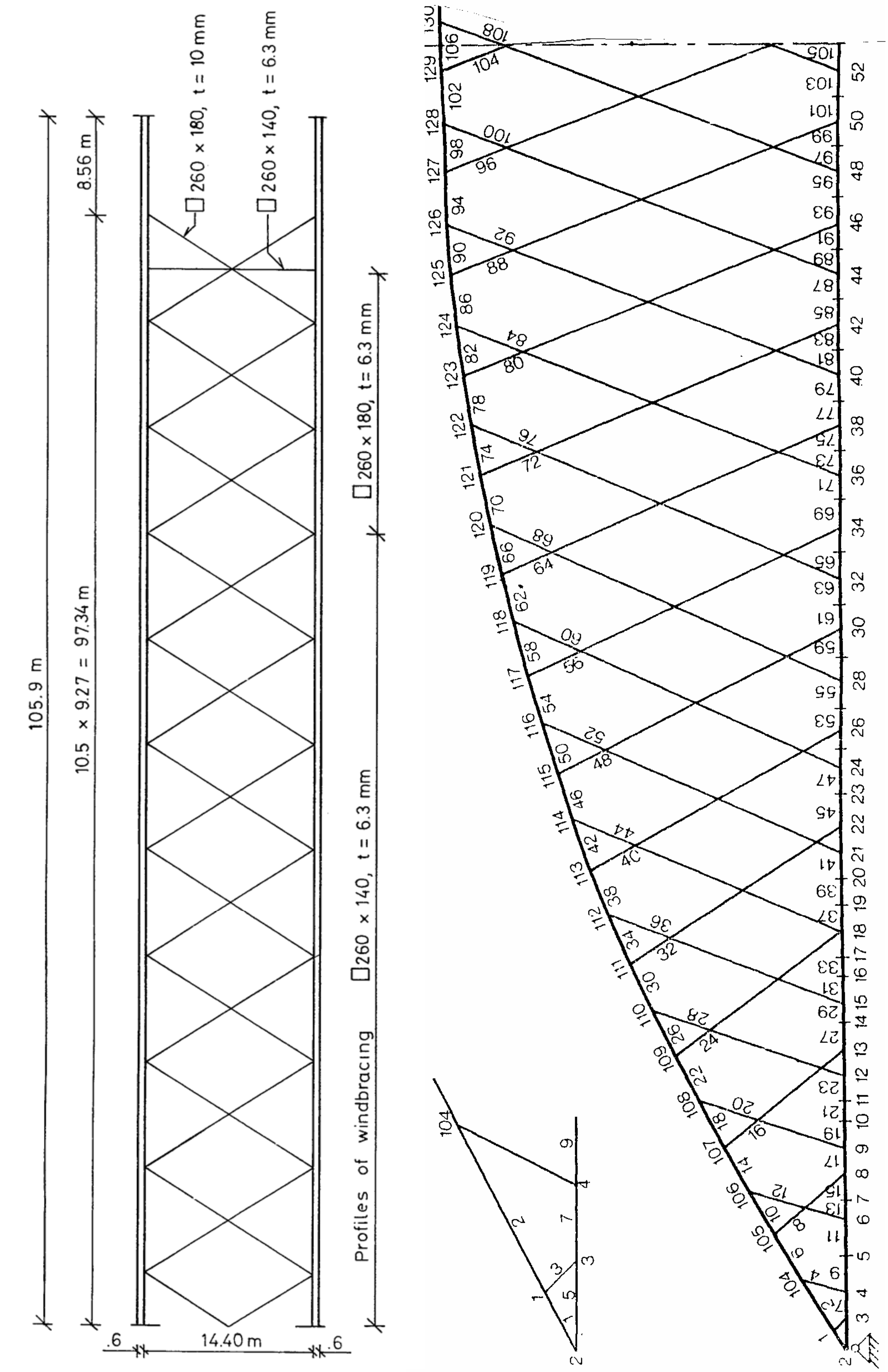


Fig. 66. Windbracing and co-ordinates of 200A

Coordinates of the arch

Node	x	y
1	-98.70	.8490
104	-94.56	3.453
105	-91.20	5.447
106	-87.81	7.368
107	-84.38	9.217
108	-80.90	10.99
109	-77.39	12.69
110	-73.85	14.31
111	-70.27	15.86
112	-66.66	17.33
113	-63.01	18.72
114	-59.34	20.04
115	-55.64	21.27
116	-51.92	22.42
117	-48.17	23.50
118	-44.40	24.49
119	-40.60	25.40
120	-36.79	26.24
121	-32.97	26.98
122	-29.12	27.65
123	-25.27	28.23
124	-21.40	28.74
125	-17.52	29.15
126	-13.64	29.49
127	-9.745	29.74
128	-5.849	29.91
129	-1.950	29.99

Coordinates of the lane

Node	x	y
2	-100.0	0.
3	-97.50	0.
4	-95.50	0.
5	-92.50	0.
6	-90.00	0.
7	-88.50	0.
8	-86.50	0.
9	-84.50	0.
10	-82.50	0.
11	-81.00	0.
12	-79.00	0.
13	-77.00	0.
14	-75.00	0.
15	-73.50	0.
16	-71.50	0.
17	-70.00	0.
18	-68.00	0.
19	-66.00	0.
20	-64.00	0.
21	-62.00	0.
22	-60.00	0.
23	-57.50	0.
24	-55.50	0.
25	-54.00	0.
26	-52.65	0.
27	-50.70	0.
28	-48.75	0.
29	-46.80	0.
30	-44.85	0.
31	-42.90	0.
32	-40.95	0.
33	-39.00	0.
34	-37.05	0.
35	-35.10	0.
36	-33.15	0.
37	-31.20	0.
38	-29.25	0.
39	-27.30	0.
40	-25.35	0.
41	-23.40	0.
42	-21.45	0.
43	-19.50	0.
44	-17.55	0.
45	-15.60	0.
46	-13.65	0.
47	-11.70	0.
48	-9.750	0.
49	-7.800	0.
50	-5.850	0.
51	-3.900	0.
52	-1.950	0.

Fig. 67. Co-ordinates for 200A

Cross - sections used in computations

	A	γ	I	W	E ***	Thickness of top and bottom plate of arch in mm.	
	m^2	MN/m^3	m^4	m^3	MN/m^2		
All hangers	.0015	.08	.0	.0	168000		
Arch, node 2 to 104	.1118	.09	.0366	519	210000	55	Member 1 and 2
" " 104 to 107	.0998	.09	.0307	.0441	"	45	" 6 to 14
" " 107 to 114	.0962	.09	.0289	0418	"	42	" 18 to 42
" " 114 to 118	.0938	.09	.0278	.0403	"	40	" 46 to 58
Member 3 and arch above node 118	.0 914	.09 **	.0267	.0388	"	38	" 3 and above 62.
Edge beam	2.274	.03035 *	.018	.06	38000		

The high specific gravity caters for weight of asphalt, guardrails, railings and lower end of hangers

The high specific gravity caters for weight of windbracing and upper end of hangers.

Fig. 68. Cross-sections used in the calculation of 200A

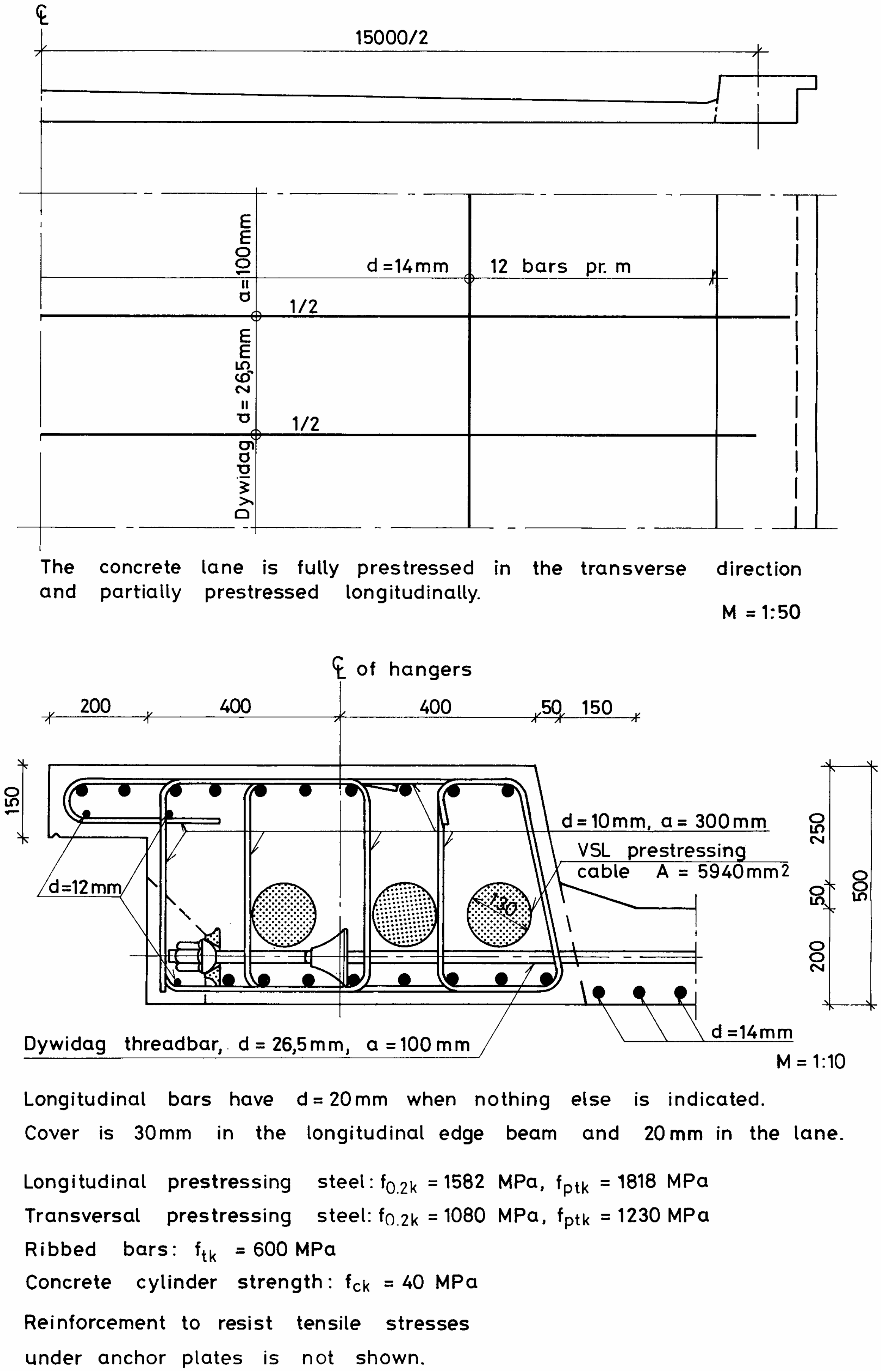


Fig. 69. Some reinforcement of edge beam and lane for 200A. Tveit 1980a. The thickness of concrete cover outside the reinforcement was smaller 25 years ago.

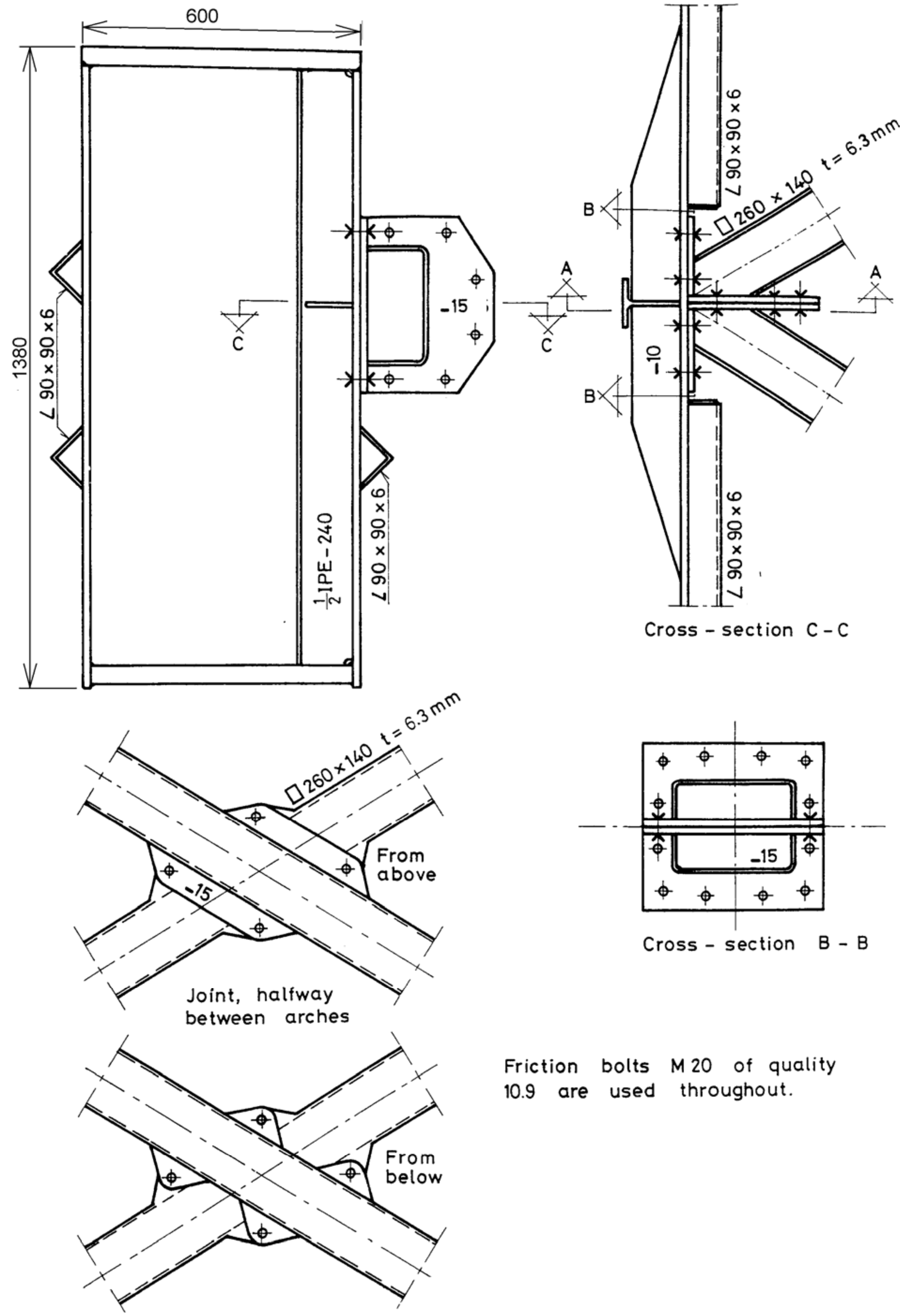


Fig. 70. Details of arch and windbracing in 200A. Tveit 1980a.

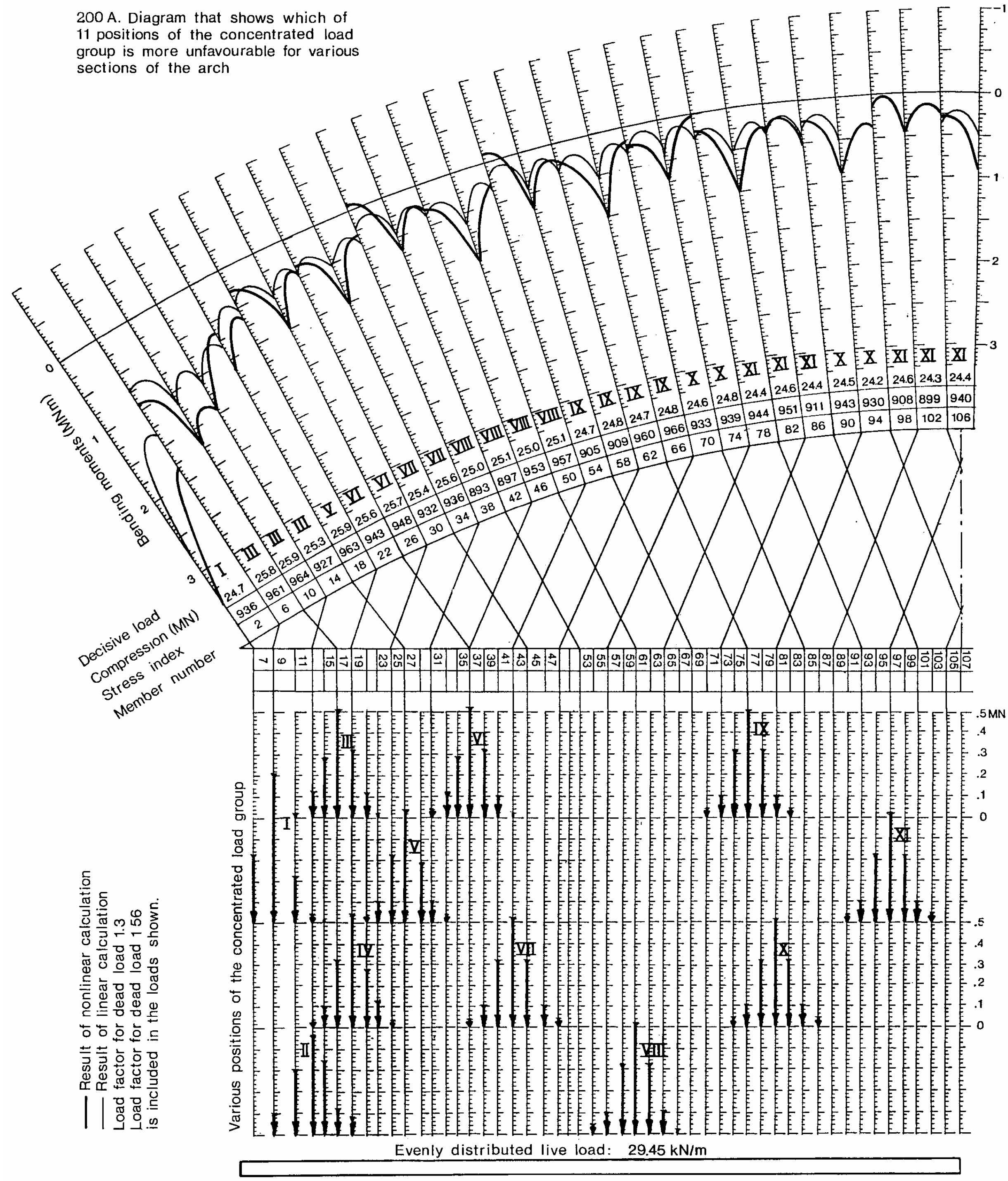


Fig 71. Forces and stress indexes for loads that decide the dimension of the arch of network arch 200A

The diagram shows how non-linear calculations influence the forces in the arch. The definition of the stress index is found on page 59. The load groups are found in fig. 23 page 13. The co-ordinates are found in fig. 67. The dimensions of the network arch are found in figs 68 to 70.

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. 72 shows how the network arch called 200A reacts to a very big load on one side of the span. The left 54% of the span carries a 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 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 gives 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.

Considerable bending moments due to relaxation of hangers are needed to make the two load cases in fig. 73 lead to equal maximum stresses.

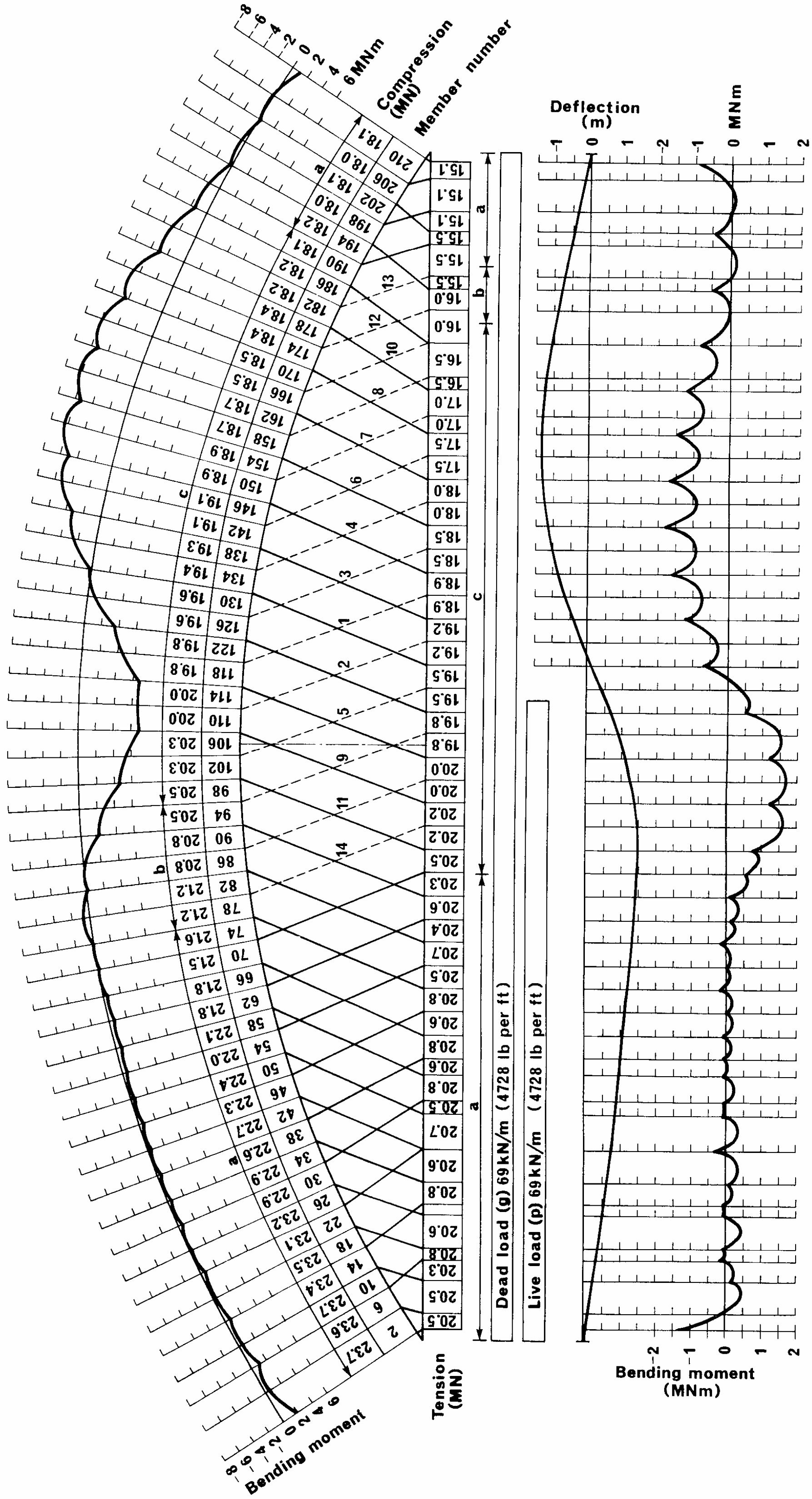


Fig. 72. Forces and deflections due to extremely skew load on 200A

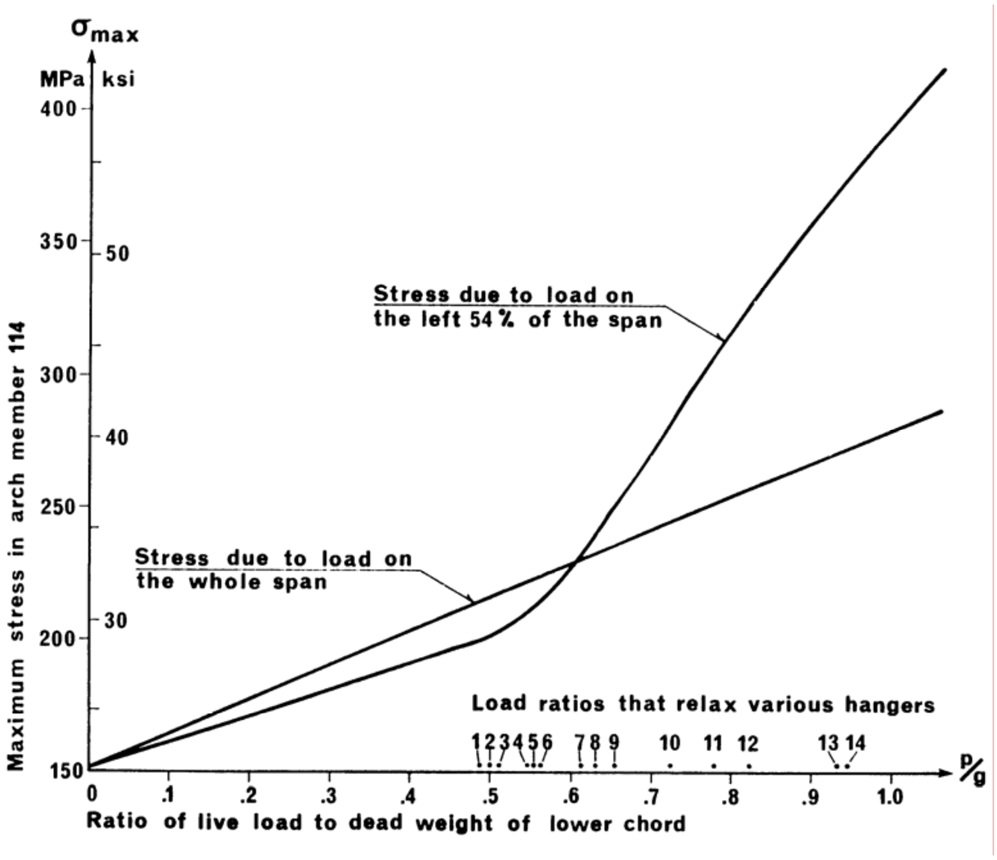


Fig. 73. Development of maximum stress in the arch member numbered 114 in fig.72

Fig. 73 applies to bridge 200A which has the arch shown in fig. 70. 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 bending moments that occur after some hangers have relaxed.

Fig. 73 shows how maximum stresses in member 114 increase with increasing evenly distributed load. The loaded length of 54% of the span and member 114 have been chosen because it is the combination 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.

For fig. 73 the span in fig. 72 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 would, however, 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. Fig. 73b shows the development of maximum stress in member 114. For moderate live loads, live loads on the whole span are decisive. 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.

For slim network arches, it is best to avoid the relaxation of hangers in the serviceability limit state. It simplifies calculations if relaxation of hangers is avoided also in the collapse limit state. Furthermore the network arch where few hangers relax can better take an increase in live loads.

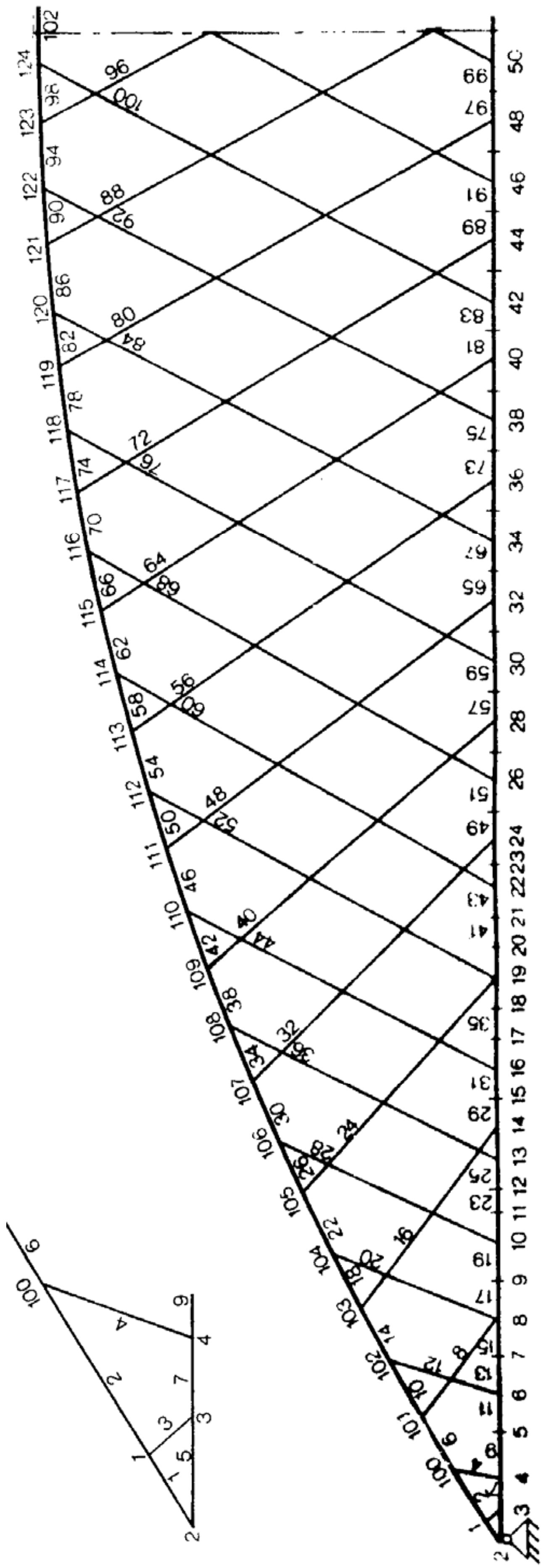


Fig. 74. Skeleton lines for 200B, which is a network arch spanning 200 metres. Tveit 1980a.

Coordinates of the lane

Node	x	y
2	-100.0	0.
3	-98.00	0.
4	-96.00	0.
5	-93.00	0.
6	-90.50	0.
7	-88.00	0.
8	-85.50	0.
9	-83.00	0.
10	-80.50	0.
11	-78.50	0.
12	-77.00	0.
13	-75.00	0.
14	-73.00	0.
15	-71.00	0.
16	-69.00	0.
17	-67.00	0.
18	-65.00	0.
19	-63.00	0.
20	-61.00	0.
21	-59.00	0.
22	-57.00	0.
23	-55.50	0.
24	-54.00	0.
25	-52.00	0.
26	-50.00	0.
27	-48.00	0.
28	-46.00	0.
29	-44.00	0.
30	-42.00	0.
31	-40.00	0.
32	-38.00	0.
33	-36.00	0.
34	-34.00	0.
35	-32.00	0.
36	-30.00	0.
37	-28.00	0.
38	-26.00	0.
39	-24.00	0.
40	-22.00	0.
41	-20.00	0.
42	-18.00	0.
43	-16.00	0.
44	-14.00	0.
45	-12.00	0.
46	-10.00	0.
47	-8.000	0.
48	-6.000	0.
49	-4.000	0.
50	-2.000	0.

Coordinates of the arch

Node	x	y
1	-98.70	.8490
100	-95.40	2.938
101	-91.88	5.051
102	-88.32	7.084
103	-84.72	9.038
104	-81.07	10.91
105	-77.38	12.70
106	-73.65	14.40
107	-69.89	16.02
108	-66.09	17.55
109	-62.25	19.00
110	-58.39	20.36
111	-54.49	21.64
112	-50.57	22.82
113	-46.62	23.92
114	-42.64	24.93
115	-38.65	25.84
116	-34.63	26.67
117	-30.60	27.41
118	-26.55	28.05
119	-22.49	28.60
120	-18.42	29.06
121	-14.33	29.43
122	-10.24	29.71
123	-6.148	29.90
124	-2.049	29.99

Fig. 75. Co-ordinates for 200B, which is a network arch spanning 200 metres. Tveit 1980a.

Cross - sections used in computations

	A	γ	I	W	E***	Thickness of top and bottom plate of arch in mm.	
	m^2	MN/m^3	m^4	m^3	MN/m^2		
All hangers	.0015	.08	.0	.0	168000		
Arch, node 2 to 100	.1118	.09	.0366	.519	210000	55	Member 1 and 2
" " 100 to 102	.0998	.09	.0307	.0441	"	45	" 6 and 10
" " 102 to 110	.0962	.09	.0289	.0418	"	42	" 14 to 42
" " 110 to 114	.0938	.09	.0278	.0403	"	40	" 46 to 58
Member 3 and arch above node 114	0.914	.09**	.0267	.0388	"	38	" 3 and above 62.
Edge beam	2.274	03035*	.018	.06	38000		

* The high specific gravity caters for weight of asphalt, guardrails, railings and lower end of hangers.

** The high specific gravity caters for weight of windbracing and upper end of hangers.

*** In diagrams where nonlinear analysis has been used, E is divided by 1.32.

Fig. 76. Cross-sections used in the calculation of 200B.

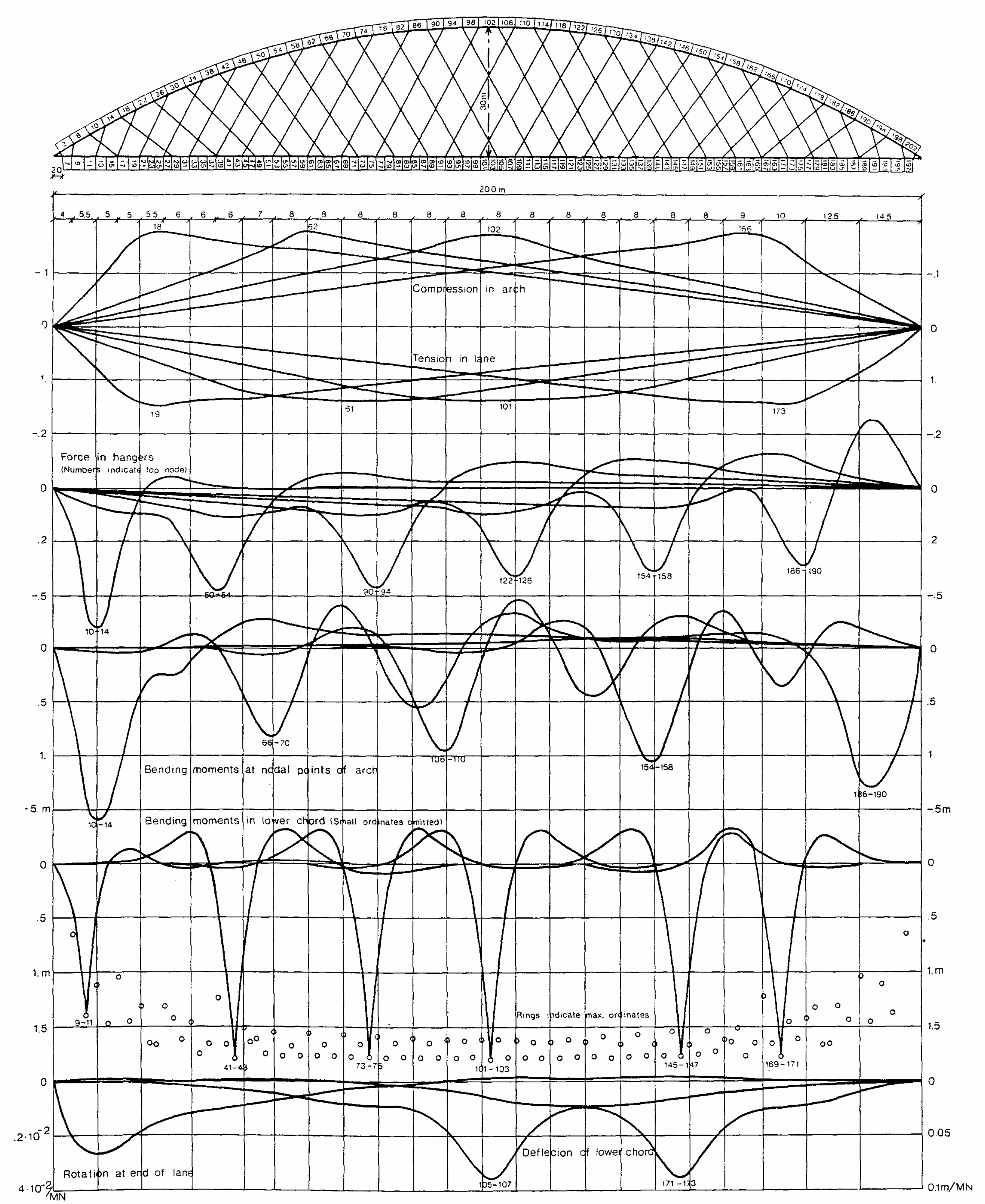


Fig. 77. Influence lines for a bridge spanning 200 m calculated for the IABSE congress in Vienna 1980

ÅKVIK SOUND, NORWEGIAN EDITION

The Åkvik Sound bridge network arch was designed because Norsk Stålforbund (Norwegian Steel Society) and the bridge office of The Norwegian Public Roads Administration were looking for competitive steel bridges with spans between 120 and 300 m.

The author presented the network arch at a seminar in April 1997. The two organisations came up with money to hire a student to assist the author in the design of a network arch for the Åkvik Sound in northern Norway. Alexandra Jay, a French student of mechanical engineering was, hired for the job.

Alexandra Jay proved very good at calculating the network arch by means of the ANSYS program. She also drew the diagrams for the network arch for the Åkvik Sound presented in this section. Originally the diagrams in figs 78 to 96 were not meant for publication. The result of our combined efforts is presented on pages 9 to 12 in this publication. In order to save work, the skeleton lines from 200B multiplied by the ratio of the spans (135/200) were used for the Åkvik Sound network arch. Tveit 1980a. 200B can be found on pages 69 to 72.

The arch of the Åkvik Sound Bridge has been calculated using members that are straight between the nodal points. Constant curvature along the arch gives a 7 mm maximum deviation from a straight line between two adjacent nodal points. In the ultimate limit state the maximum axial force in the arch is around 10 MN. This gives an additional ~20 MPa bending stress in the nodes of the arch. The bending moments due to continuous curvature of the arch are indicated in part of fig. 91.

Since the shapes of the skeleton lines of 200B and the Åkvik Sound Bridge are the same, there must be considerable similarities in the influence lines of the two spans. The ordinates are practically the same for the axial forces in the chords. See figs 77, 81 and 82. There is more on this on page 56.

There is also considerable likeness between the influence lines for the bending in the lower chord in figs 77 and 85. Had the spans been models of one another, the influence ordinates would have been proportional to the spans. See about model laws on page 56.

The maximum influence ordinate for bending in the lower chord of 200B is 1.8 m. This can be seen from fig. 77. The corresponding influence ordinate for the Åkvik Sound should have been 1.8*135/200=1.22. As can be seen from page 82 it is 1.33. This is partly because the stiffness (EI) of the arch in 200A is 8.87 times the stiffness of the tie. In the network arch of the Åkvik Sound the stiffness of the arch is 3.09 times the stiffness of the lane. Thus the arch in the 200A takes more bending than the arch in the Åkvik Sound. The area (stiffness) of the hangers will have a smaller influence.

The author assumes that the relative stiffness of the chords and the area of the hangers influence the size of the ordinates for bending moments in the arches. The maximum influence ordinates in the arches for the bending in the nodal points between member 90 and 94 and between member 130 and 134 are proportional to the span of the two arches.

From the three preceding paragraphs we can conclude the obvious. Model laws apply to the influence lines of normal forces in the chords. Bending in the chords is considerably influenced by the relative stiffness of the chords and possibly, but to a lesser extent, by the stiffness of the hangers.

Since there is little bending in the chords of a network arch, there is little need to know the exact magnitude of the bending moments in the chords before calculations by a computer program.

Deflection of the Åkvik Sound Bridge can be found using the influence lines in fig. 88. The biggest deflection due to live loads can be found in the middle of the span. It is about 81 mm. The span of the bridge is 1667 times this deflection.

Fig. 90, page 87, gives forces when the smaller edge beam is cast. A HE 200B in the temporary lower chord has been assumed for these calculations. The stiffness of the part of the edge beam that has already been cast has been disregarded. Just below the middle of page 27 there is some good reasoning on the forces in the temporary lower chord.

The right hand side of the diagram shows the forces when 18.2 metres at both ends of the span carry the concrete of the lighter edge beam before it is hardened. The left side of the diagram shows the forces when 32.5 metres at both ends of the span carry the concrete of the lighter edge beam before it is hardened.

In order to prevent the relaxing of hangers, the casting starts from the ends of the span and proceeds towards the middle. The loads are as follows:

Reinforcement	3.6 kN/m
Wooden formwork	0.2 kN/m
Prestressing cables	0.7 kN/m
Sum:	4.5 kN/m
Weight of edge beam	9.7 kN/m
Sum:	14.2 kN/m

In addition to these weights comes the weight of the steel in the lower chord. It is 1.7 kN/m. A load factor of 1.2 for dead load has been used for all loads during the casting of the small edge beam. From fig. 90 it can be seen that the edge beam is not overly distorted during casting. Thus there is only small built-in bending moments in the small edge beam. The tension in the hangers is big enough to keep them reasonably straight while the edge beam is cast.

The steel in the longitudinal beam of the temporary lower chord has f_y=460 N/mm^2. The longitudinal beam in the temporary lower chord in figs 21 and 22 is HE 220 AA. It has a bending moment capacity 205 kNm. This is about ten times greater than the biggest bending moment in the temporary lower chord in fig. 90.

In the arch the bending moment capacity is more than 40 times the bending moment. The tensile capacity of the temporary lower chord is over 2 MN. The prestressing cables take some of the tensile force during and after the casting of the tie. Thus the deflection shown in fig. 90 will not occur. Tension in the prestressing cables while the edge beams are cast will reduce the friction due to wobble.

When the smaller edge beam is cast, there is very uneven load on the two arches. The hangers under the stronger arch will restrain the relaxation of the hangers under the weaker arch. This effect has been disregarded.

In the ultimate limit state the maximum force in a hanger is 539 kN. In figs 90 to 96 the bending moments are given in the rectangle to the right of the points where they occur.

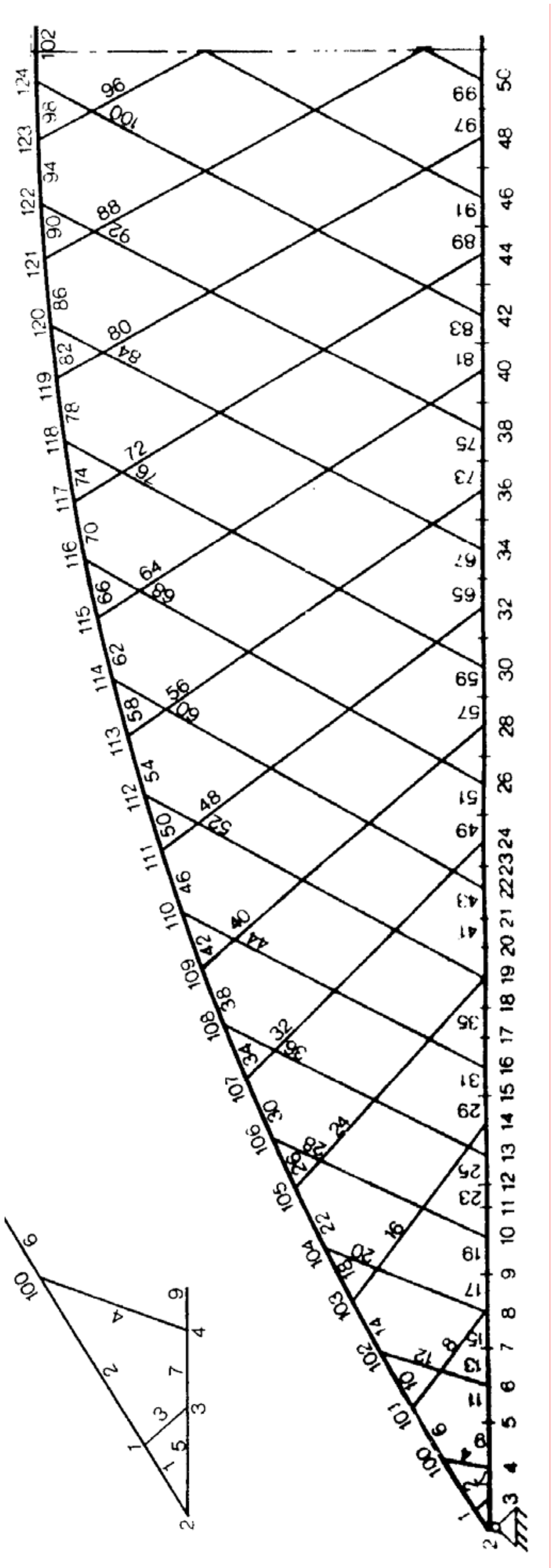


Fig. 78. Skeleton lines for the Åkvik Sound network arch

Coordinates of the arch		
Node	X	Y
1	-66.6225	0.573075
100	-64.395	1.983
101	-62.019	3.409
102	-59.616	4.782
103	-57.186	6.100
104	-54.722	7.364
105	-52.232	8.573
106	-49.714	9.720
107	-47.176	10.814
108	-44.611	11.846
109	-42.019	12.825
110	-39.413	13.743
111	-36.781	14.607
112	-34.135	15.404
113	-31.469	16.146
114	-28.782	16.828
115	-26.089	17.442
116	-23.375	18.002
117	-20.655	18.502
118	-17.921	18.934
119	-15.181	19.305
120	-12.434	19.616
121	-9.673	19.865
122	-6.912	20.054
123	-4.150	20.183
124	-1.383	20.243
125	0	20.250

Coordinates of the lane		
Node	X	Y
2	-67.500	0.00
3	-66.150	0.00
4	-64.800	0.00
5	-62.775	0.00
6	-61.088	0.00
7	-59.400	0.00
8	-57.713	0.00
9	-56.025	0.00
10	-54.388	0.00
11	-52.988	0.00
12	-51.975	0.00
13	-50.625	0.00
14	-49.275	0.00
15	-47.925	0.00
16	-46.575	0.00
17	-45.225	0.00
18	-43.875	0.00
19	-42.525	0.00
20	-41.175	0.00
21	-39.825	0.00
22	-38.475	0.00
23	-37.463	0.00
24	-36.450	0.00
25	-35.100	0.00
26	-33.750	0.00
27	-32.400	0.00
28	-31.050	0.00
29	-29.700	0.00
30	-28.350	0.00
31	-27.000	0.00
32	-25.650	0.00
33	-24.300	0.00
34	-22.950	0.00
35	-21.600	0.00
36	-20.250	0.00
37	-18.900	0.00
38	-17.550	0.00
39	-16.200	0.00
40	-14.850	0.00
41	-13.500	0.00
42	-12.150	0.00
43	-10.800	0.00
44	-9.450	0.00
45	-8.100	0.00
46	-6.750	0.00
47	-5.400	0.00
48	-4.050	0.00
49	-2.700	0.00
50	-1.350	0.00

Fig. 79. Co-ordinates for the Åkvik Sound network arch

CROSS-SECTIONS USED IN COMPUTATIONS FOR OUTPUT FROM 03/03/98 TO THE MIDDLE OF MAY 1998

	A (mm^2)	γ (Ns^2/mm^4) (10^{-9})	I (mm^4) (10^6)	W (mm^3)	E (Mpa)	
All hangers	1256	7,85			210000	Diameter 40 mm
Arch,node 2 to 105	50060	7,85	553,7	2721000	210000	UC 356*406*393
Arch, node 105 to 125	43300	8,3*	468,5	1939000	210000	UC 356*406*339,9
Edge beam	1500000	3,04**	10630		30000	Height 450 mm

The cross-sections used in the computations apply to the strongest arch, but the influence lines can be used for the weaker arch as well.

* The high specific gravity caters for weight of windbracing and upper end of hangers
** The high specific gravity caters for weight of asphalt, guardrails, railings and lower end of hangers

Fig. 80. Cross-sections used in the final calculations for the Åkvik Sound network arch

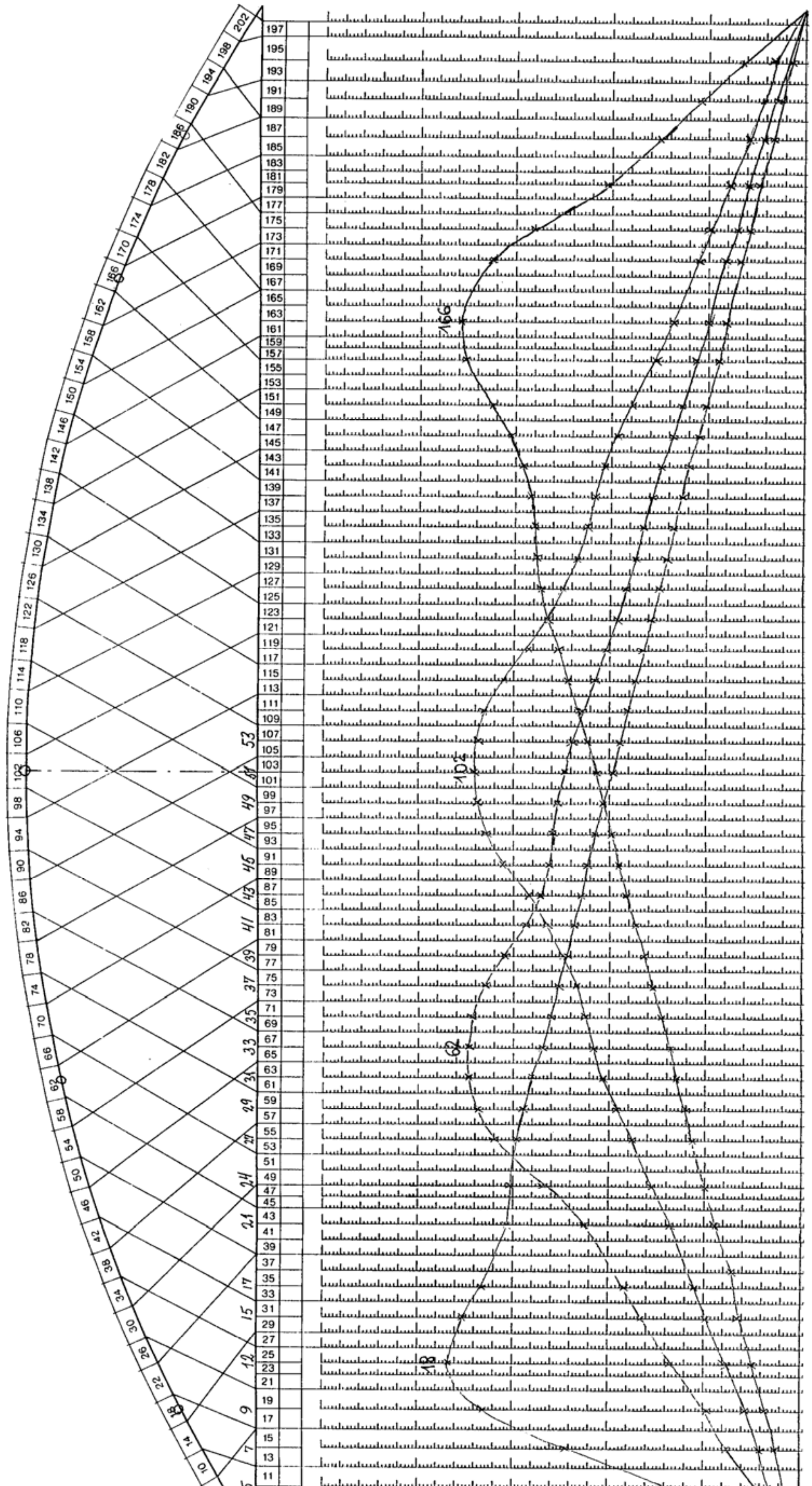


Fig. 81. Influence lines for compression in the arch of the Åkvik Sound network arch

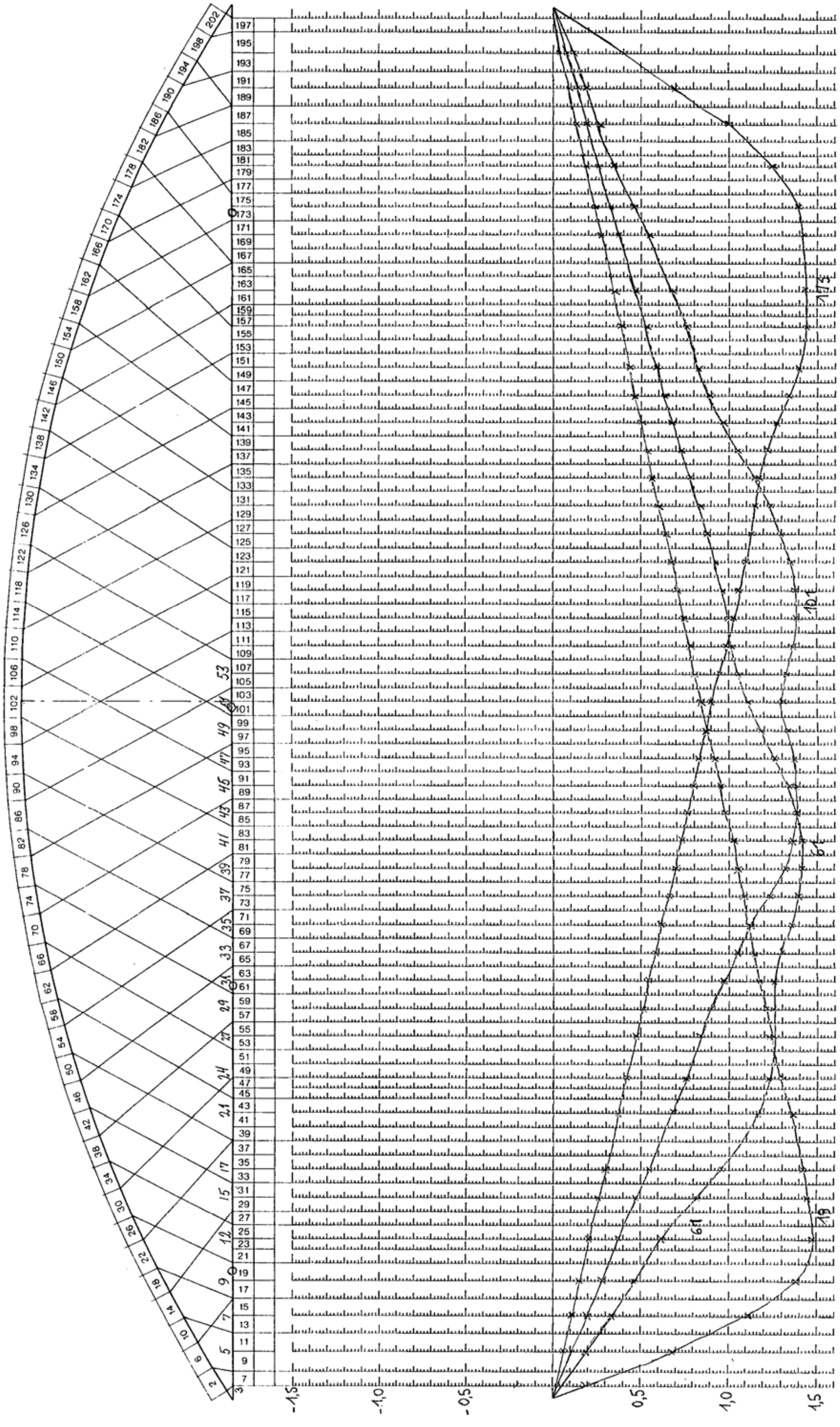


Fig. 82. Influence lines for tension in the tie of the Åkvik Sound network arch

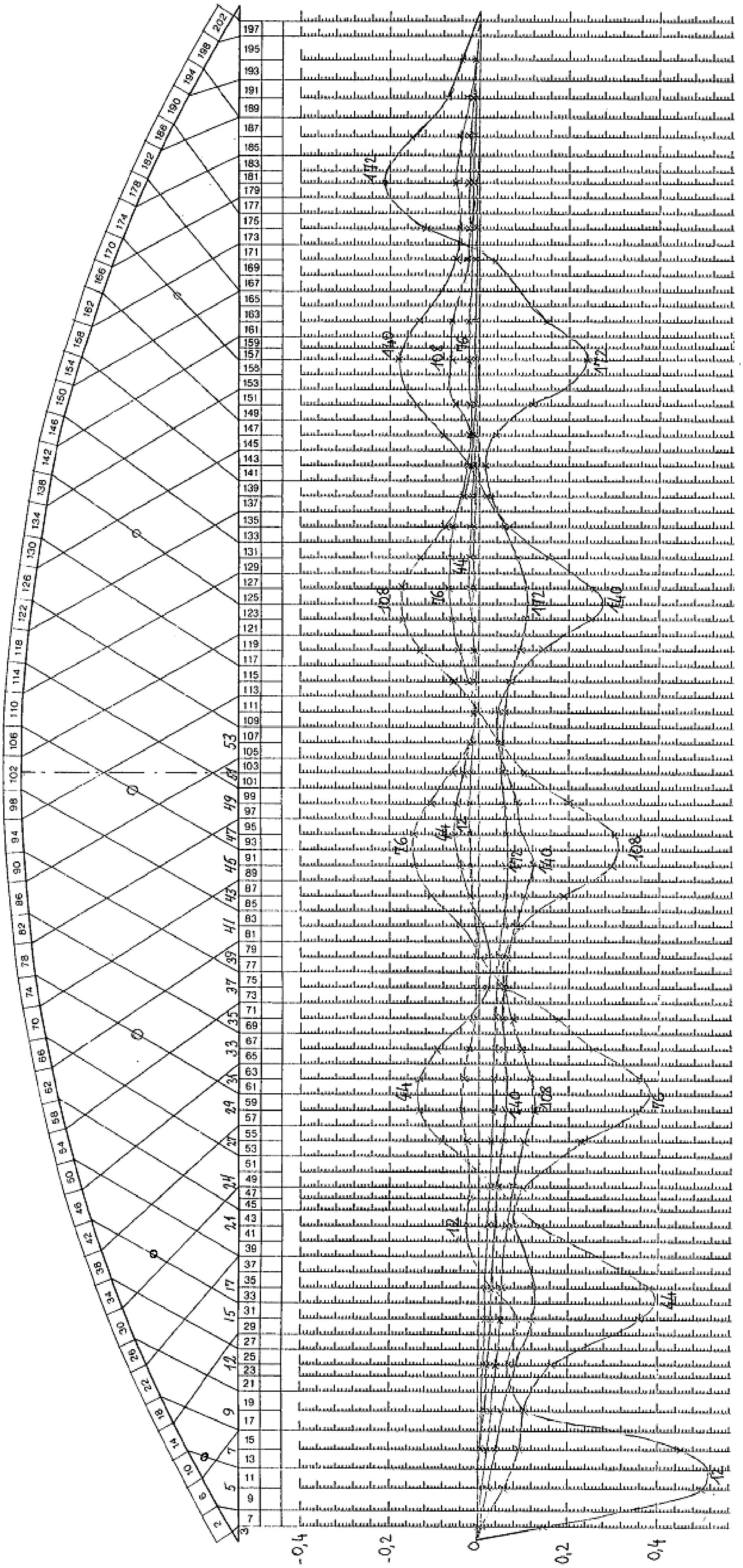


Fig. 83. Influence lines for the force in the hangers in the Åkvik Sound network arch

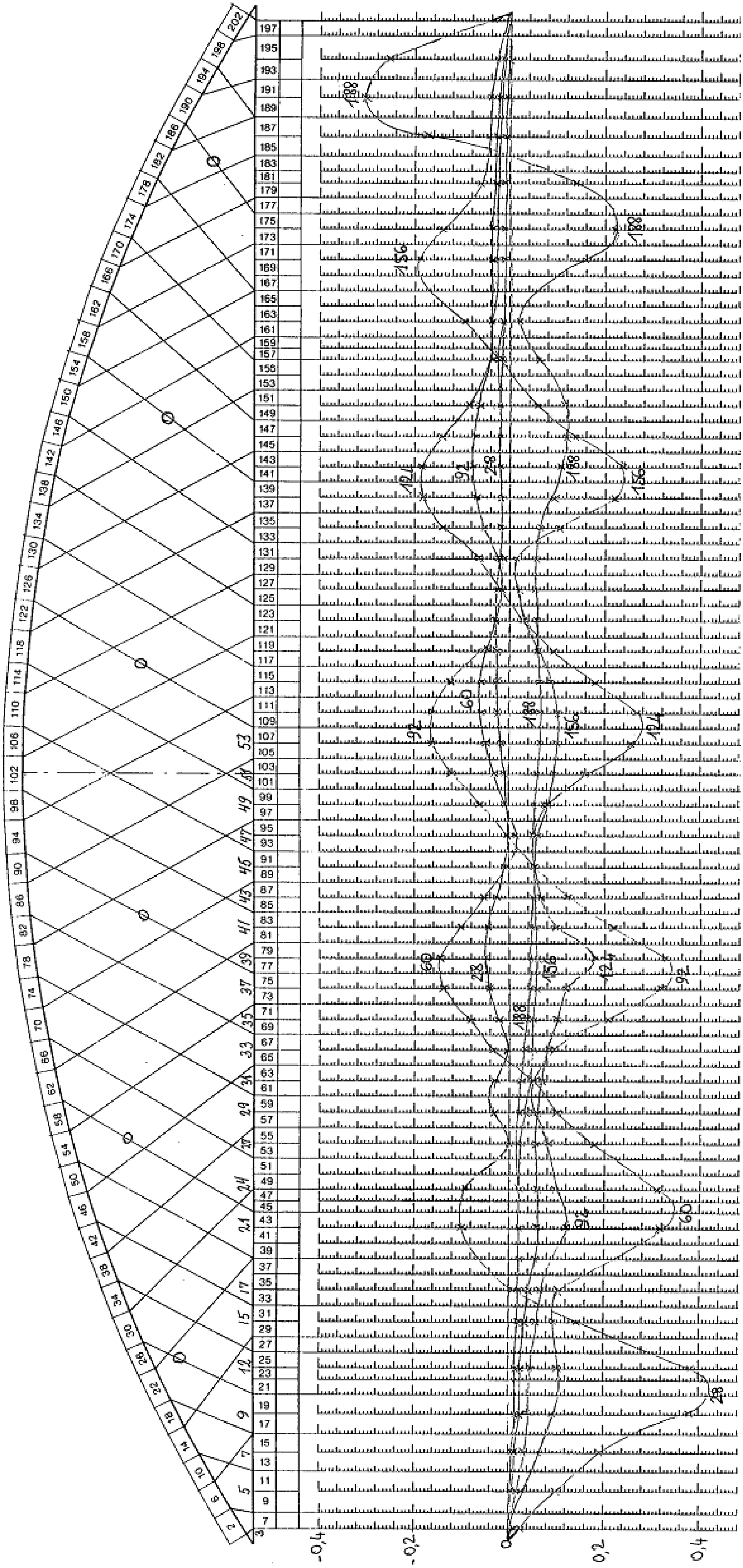


Fig. 84. More influence lines for the force in the hangers of the Åkvik Sound network arch

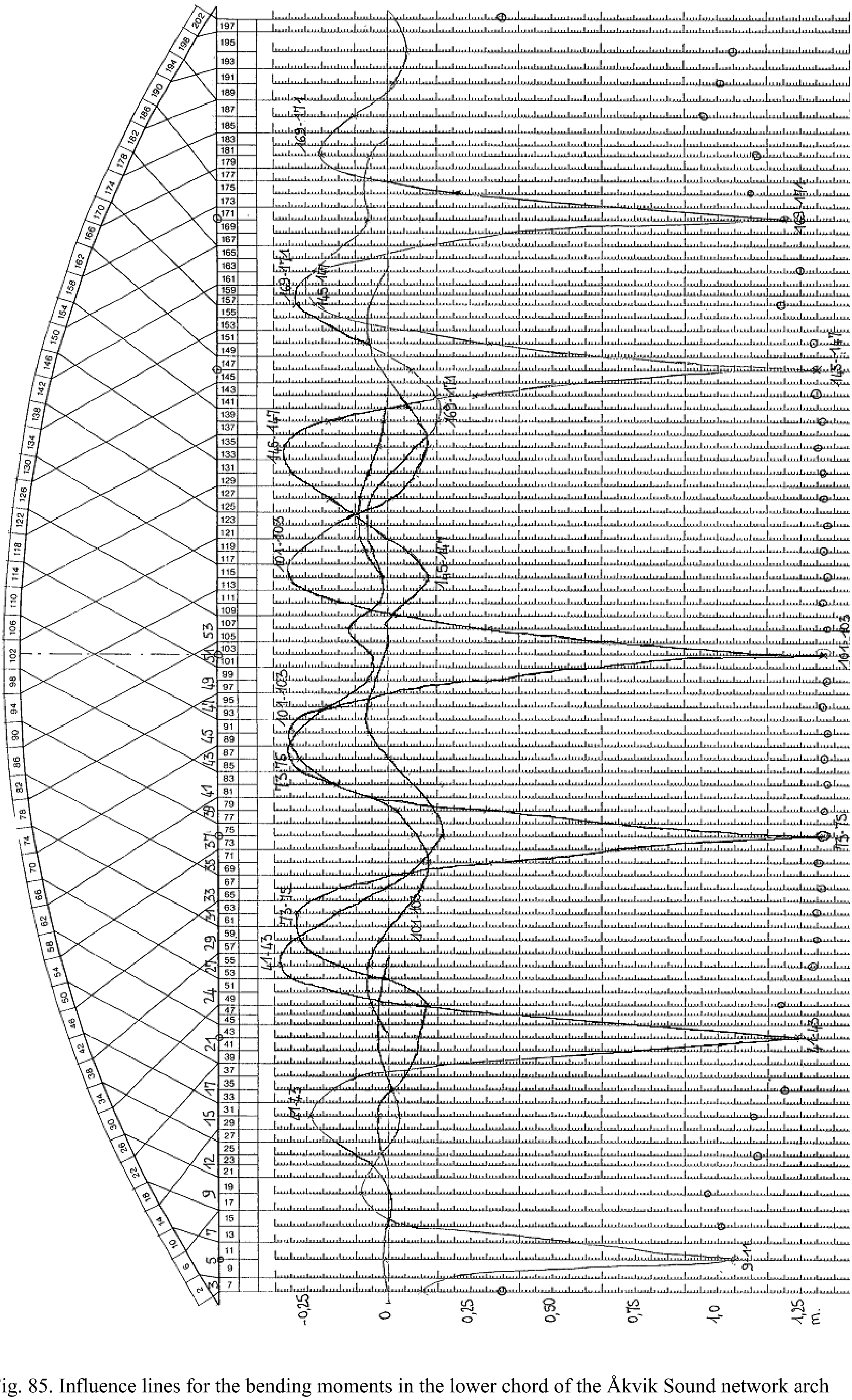


Fig. 85. Influence lines for the bending moments in the lower chord of the Åkvik Sound network arch

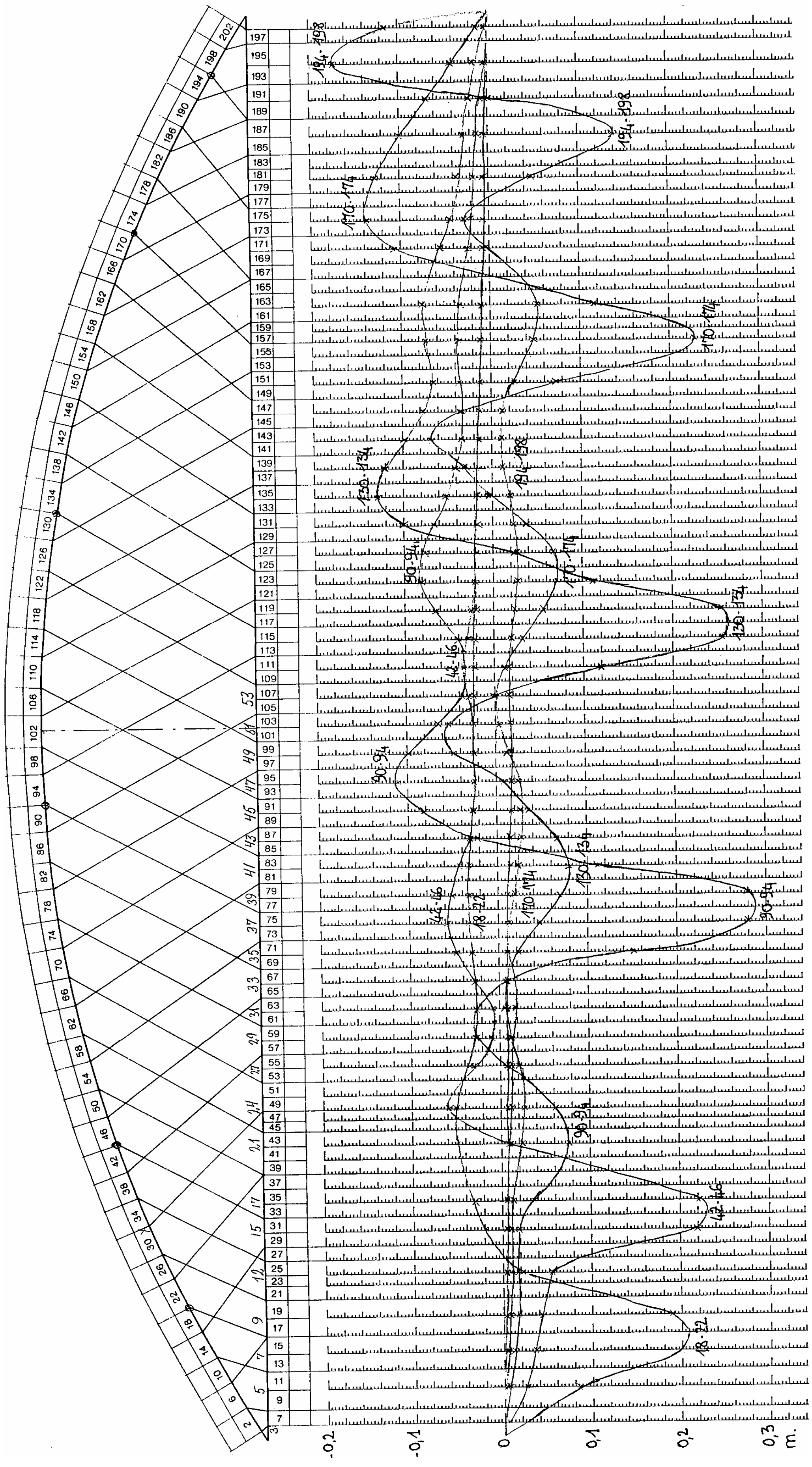


Fig. 86. Influence lines for the bending moments in the arch of the Åkvik Sound network arch

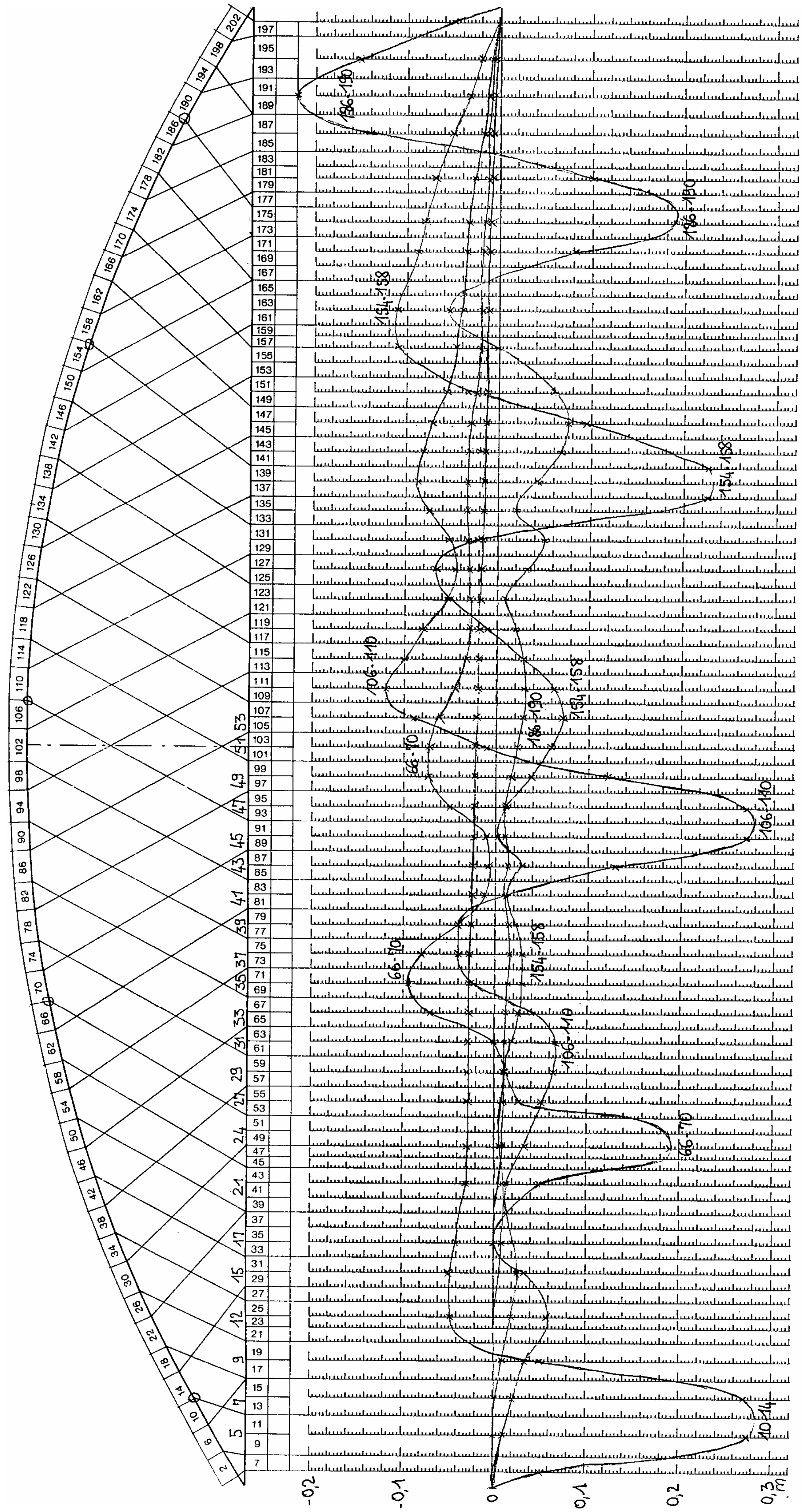


Fig. 87. More influence lines for the bending moments in the arch of the Åkvik Sound network arch

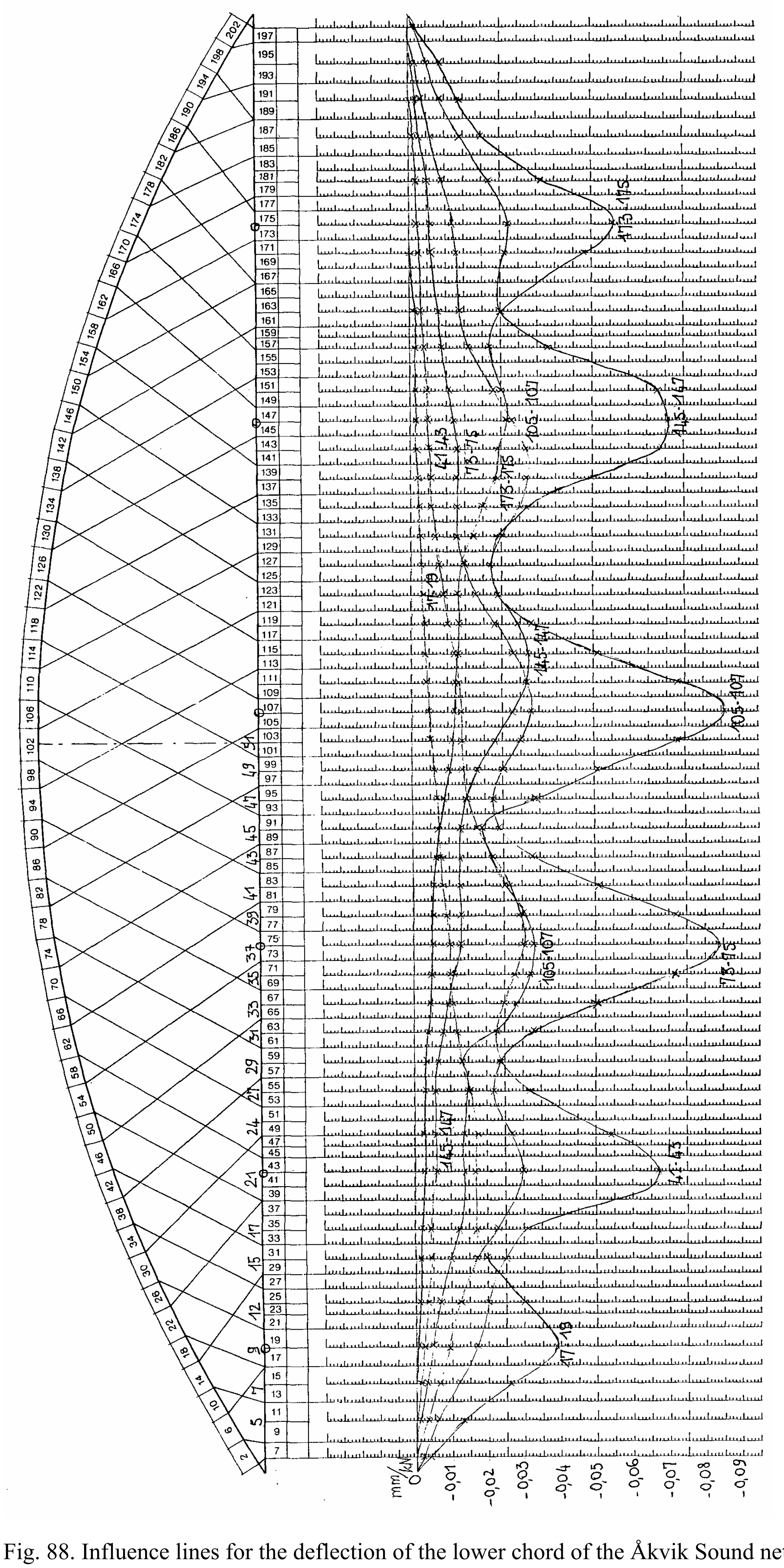


Fig. 88. Influence lines for the deflection of the lower chord of the Åkvik Sound network arch

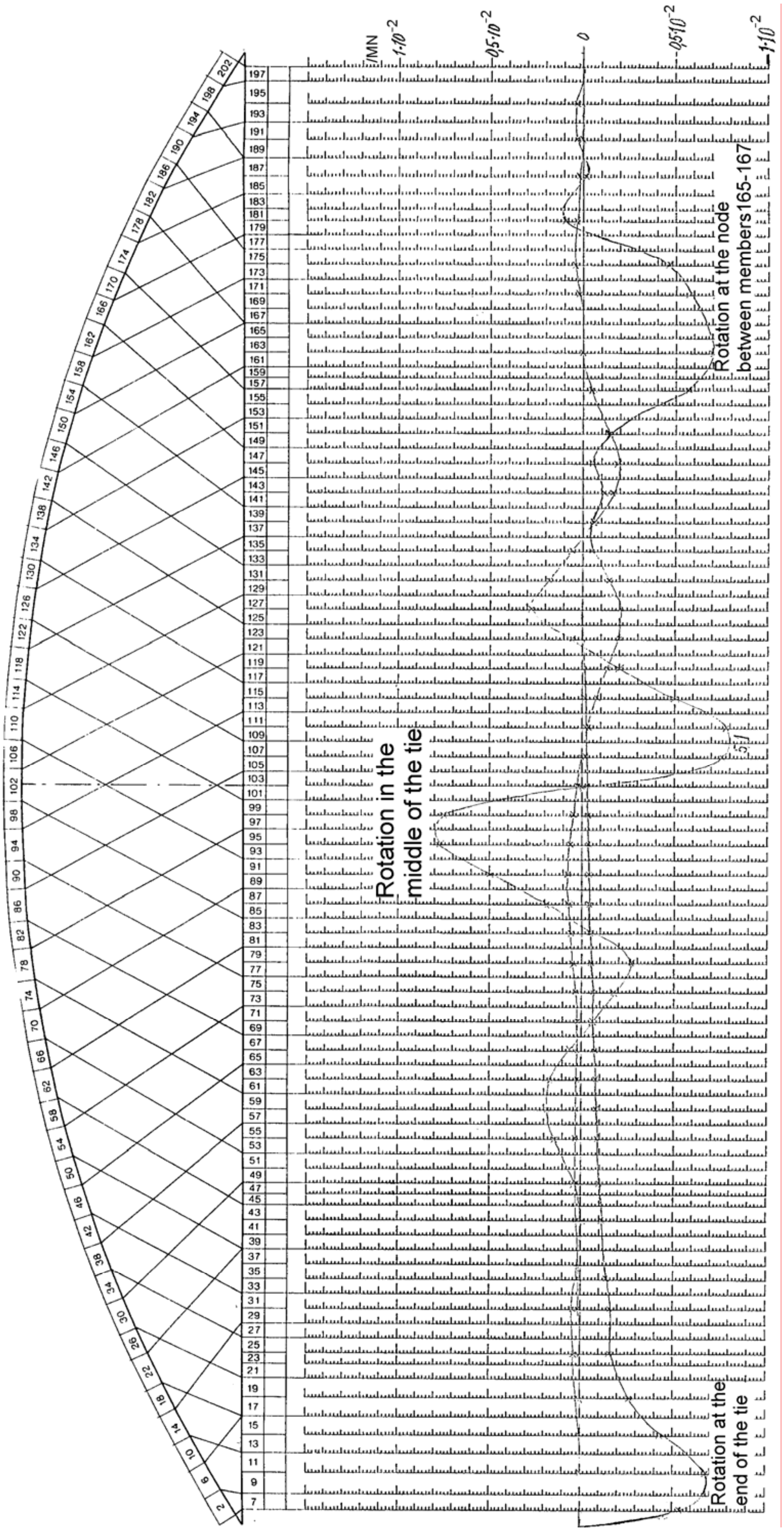


Fig. 89. Influence lines for the rotation in the lower chord of the Åkvik Sound network arch

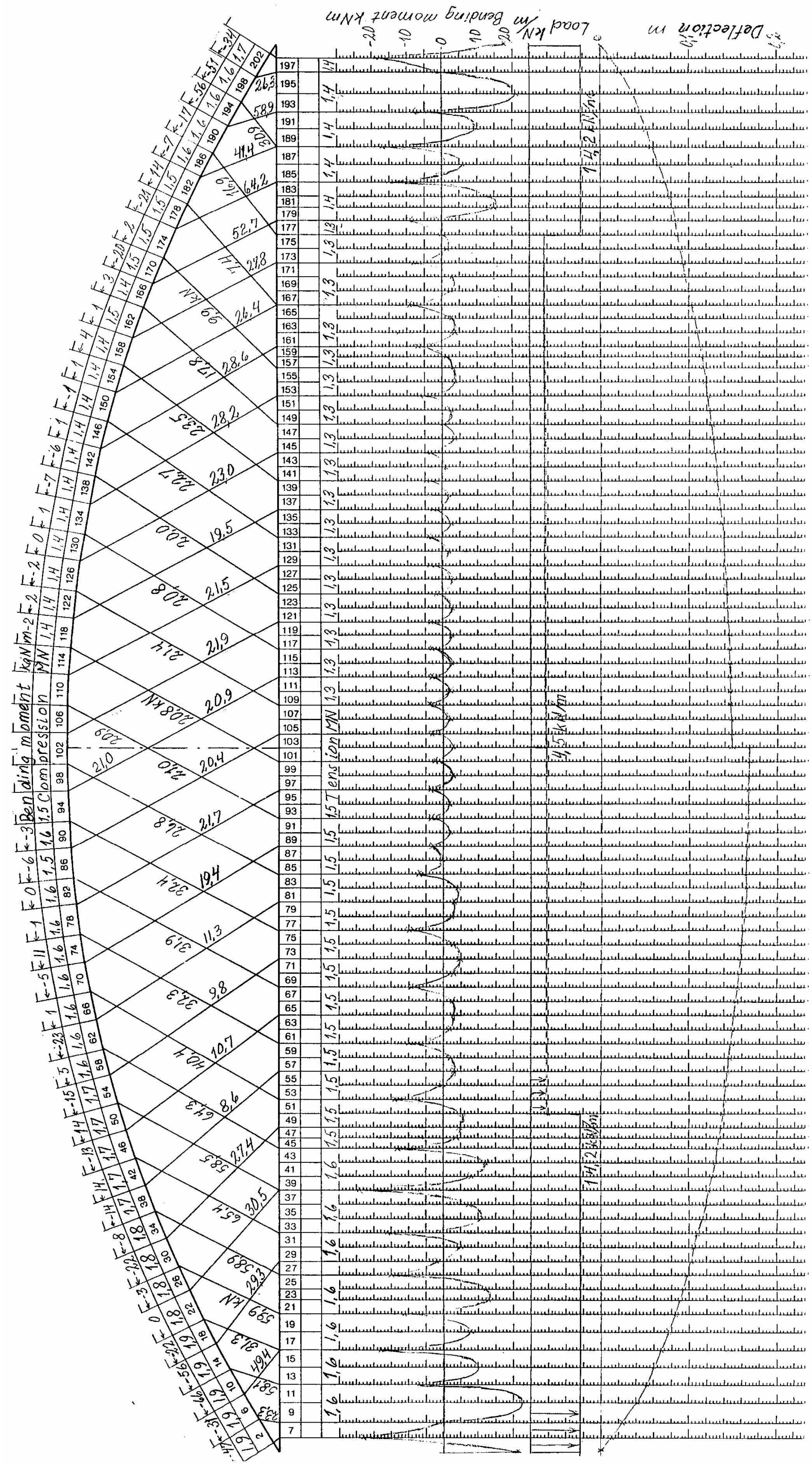


Fig. 90. Forces in the Åkvik Sound network arch when the smaller edge beam is cast. Loads and stiffnesses are explained on p. 74.

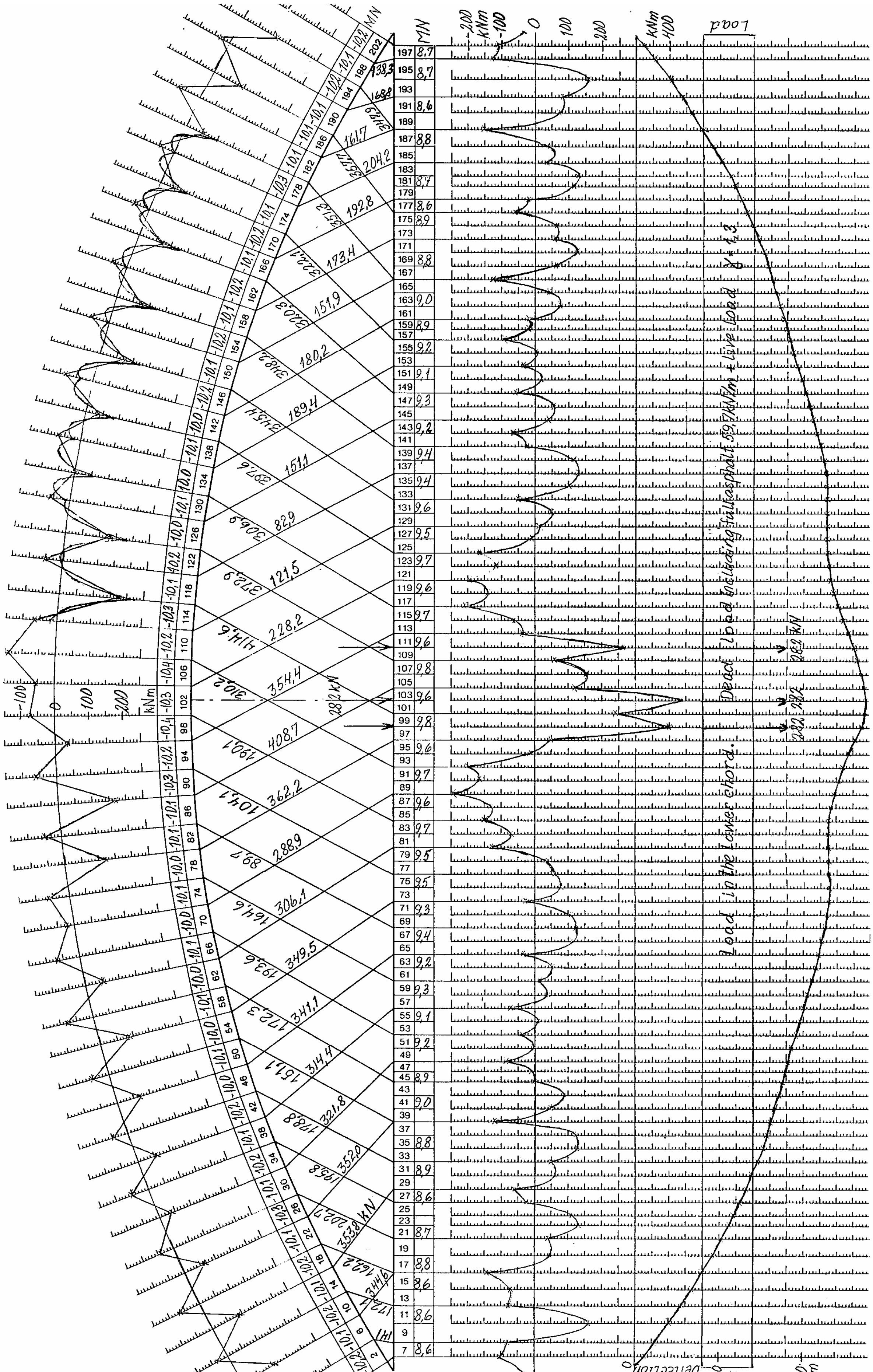


Fig. 91. Maximum load on the Åkvik Sound network arch in the ultimate limit state. The wheel loads are in the middle of the span

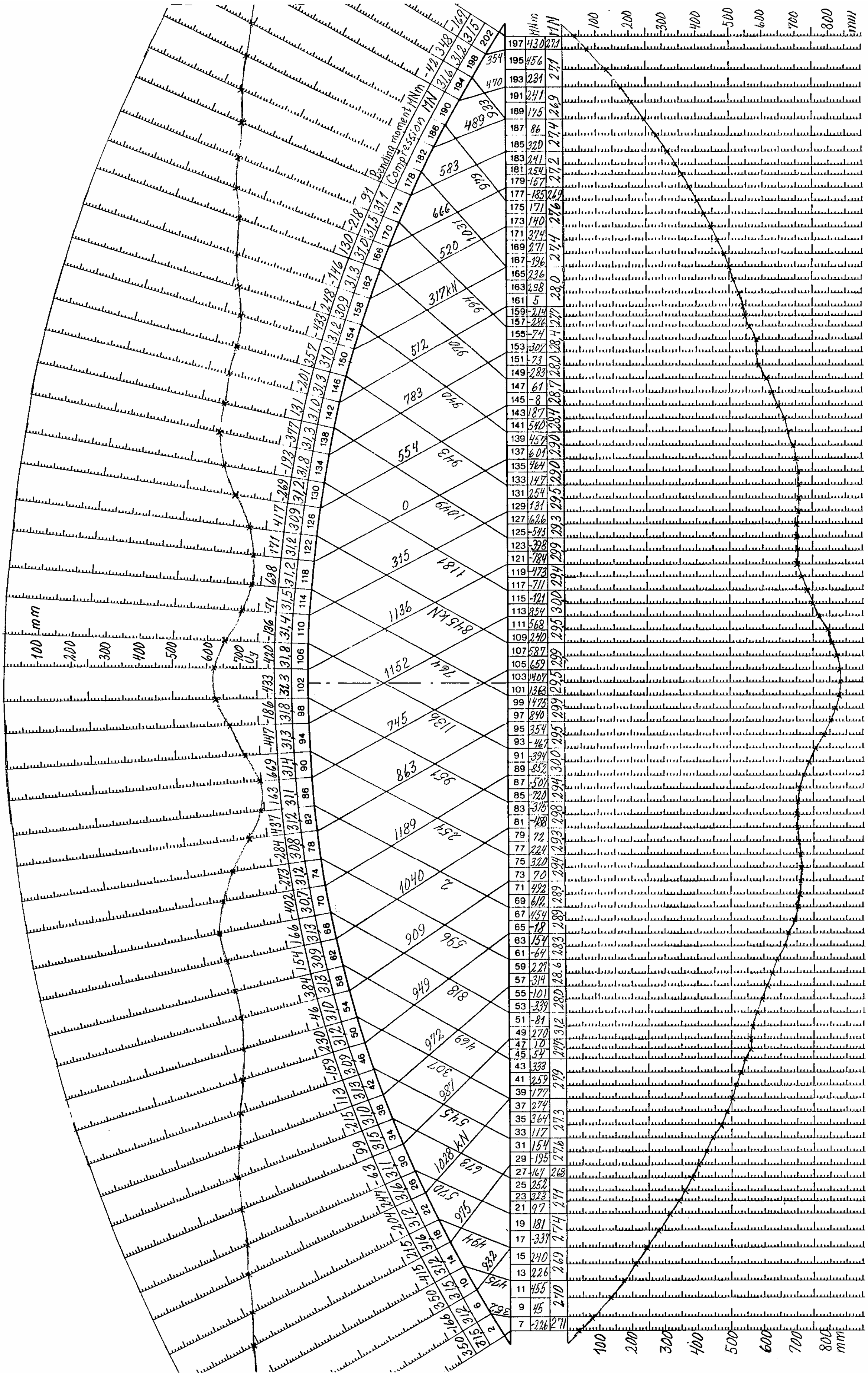


Fig. 92. Forces and deflections just before elastic buckling due to the evenly distributed load on the Åkvik Sound network arch

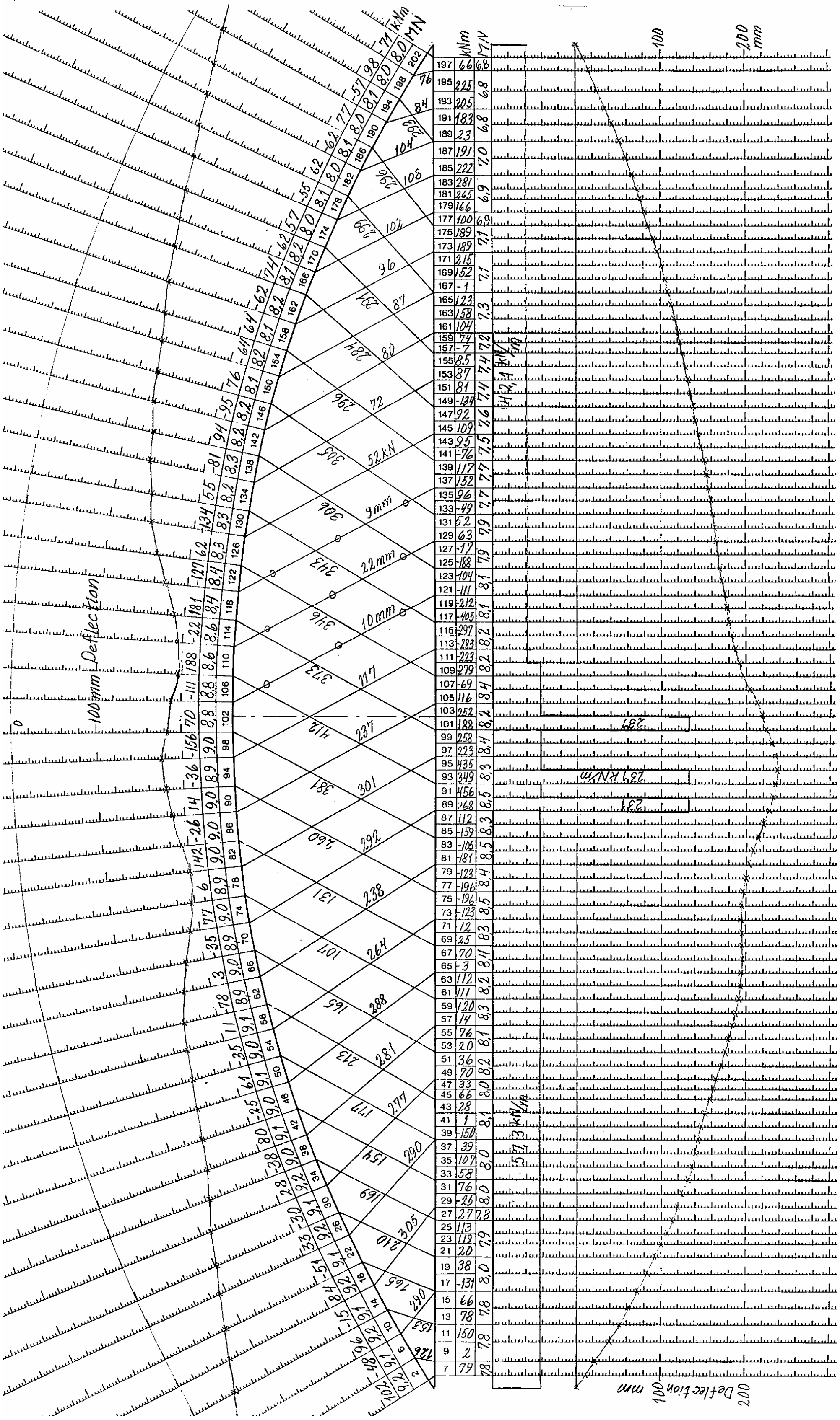


Fig. 93. Forces due to live loads on the left 54% of the Åkvik Sound network arch in the ultimate limit state. Half the weight of asphalt on the whole span is assumed. The shortening of three relaxed hangers is given in mm.

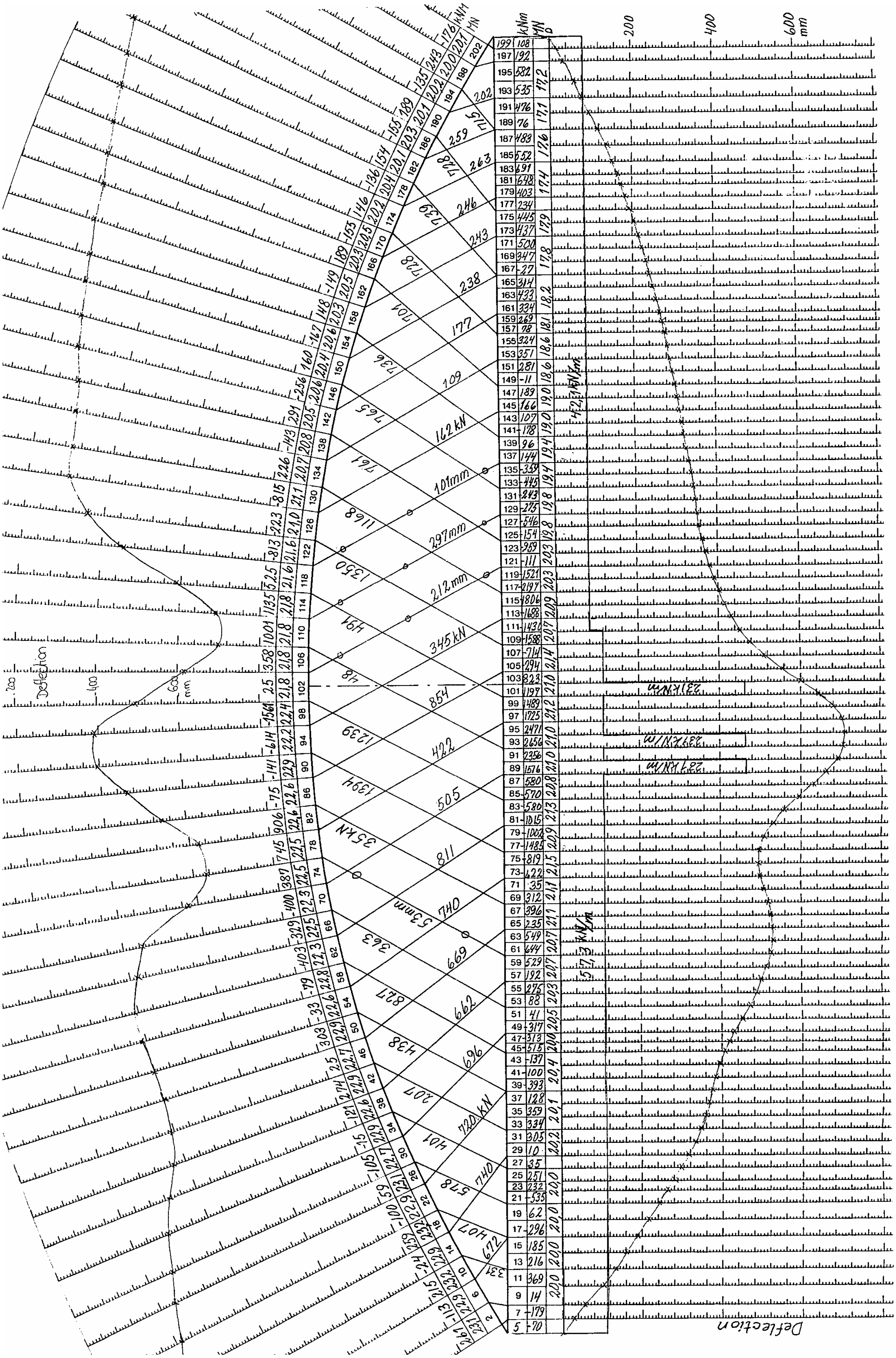


Fig. 94. Forces and deflections just before elastic buckling due to load on the left 54% of the Åkvik Sound network arch. The shortening of three relaxed hangers is given in mm.

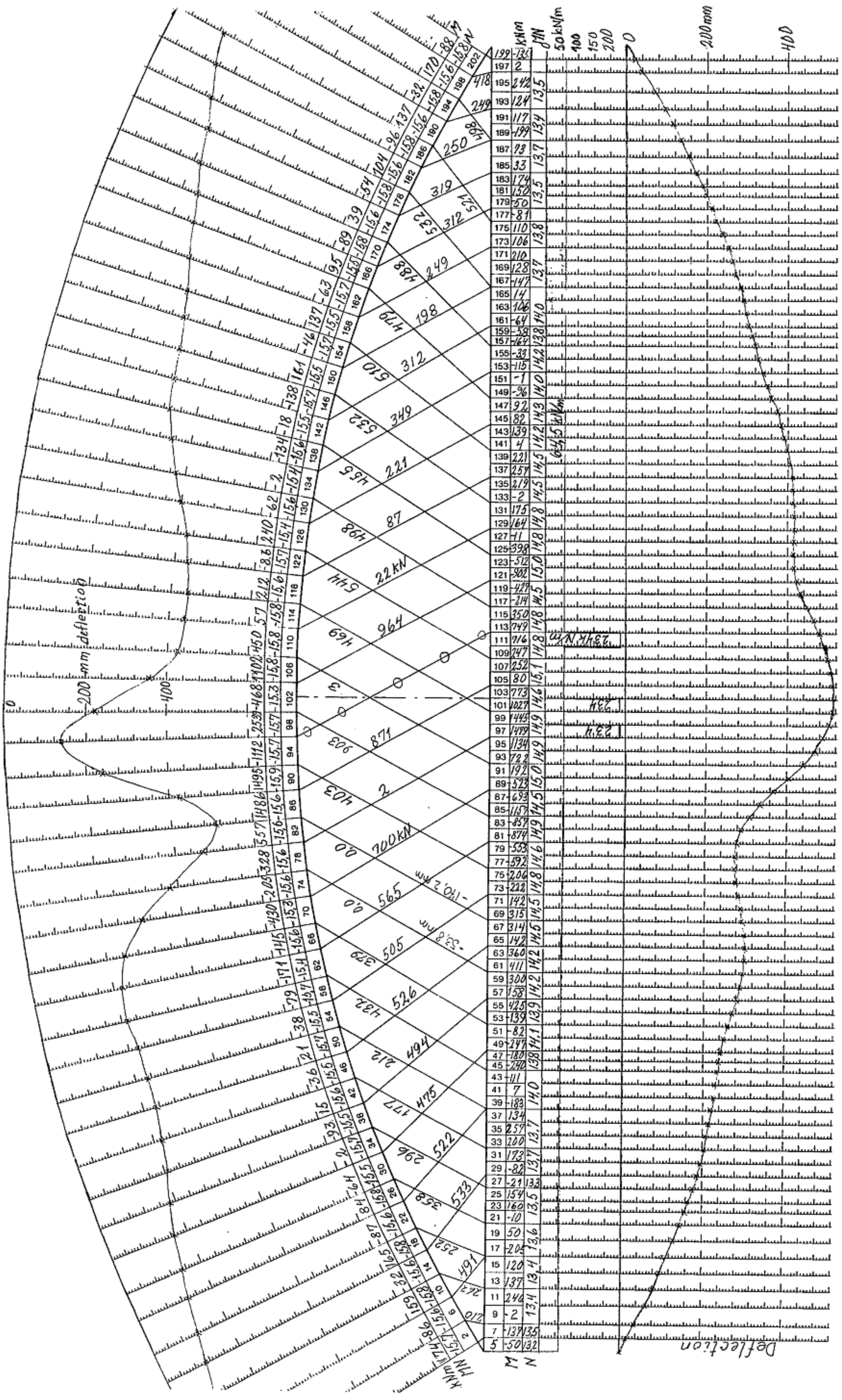


Fig. 96. Forces and deflections just before buckling when one hanger in the middle of the Åkvik Sound network arch is broken

NETWORK ARCH BRIDGE OVER THE RIVER LUZNICE (Czech Republic) Built 2004

Designer: Ladislav Šašek, PhD, Mott MacDonald, Prague

Basic information: **Investor: South-Bohemian Region** **Contractor: SWIETELSKY**

The bridge is located in the lovely valley of the river Lužnice. Very nice historical scenery of the town Bechyně just underneath the castle called for a slender and fragile structure, which will blend into its environment, particularly where the dimensional relationships and scale are concerned.

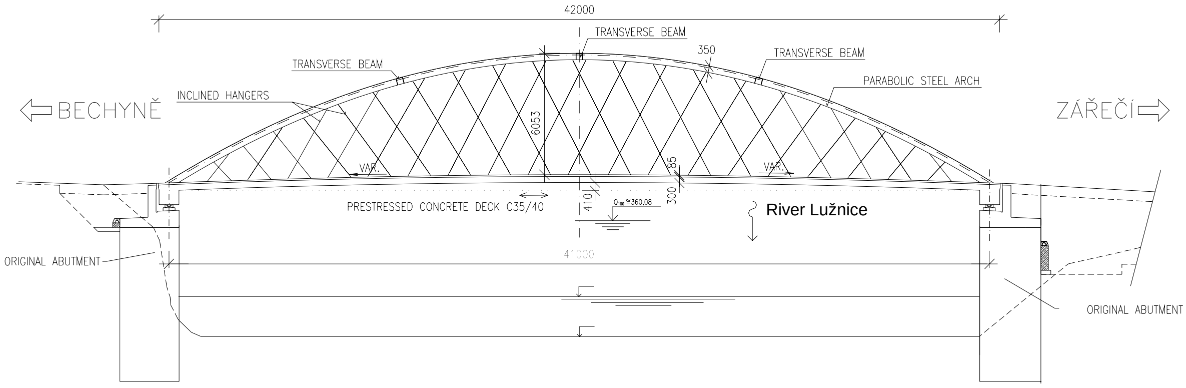


Fig. 96a. Longitudinal section

The structural form is a tied arch where the arch is made of steel and the tie is prestressed concrete deck with many inclined hangers that have multiple intersections. It has two steel arches with the rise of 6.05 m. The sections of arches are welded, in the shape of an inverted letter U. Hangers are made from stainless steel rods. The concrete slab thickness varies from 250 mm – 300 mm below road line, 500 mm below hangers to 180 mm below sidewalks.

The bridge span is 41 m. The construction can be very slender as the bending in the steel arches and concrete tie is small. 7 days after hardening of concrete the hangers were tightened and the deck was prestressed by 4 tendons. After stressing, the structure was able to carry the load and the scaffolding was removed. In order to control stresses, great care was needed in the adjustment of the hangers.

Night lighting is designed for daily use and is supplied by two lamp posts on both sides of the bridge. Lighting for special occasions is created with 36 lamps placed in the inverted U sections of arches. The conception of this type of lighting focuses on emphasising the network of stainless hangers as a characteristic feature of the bridge.

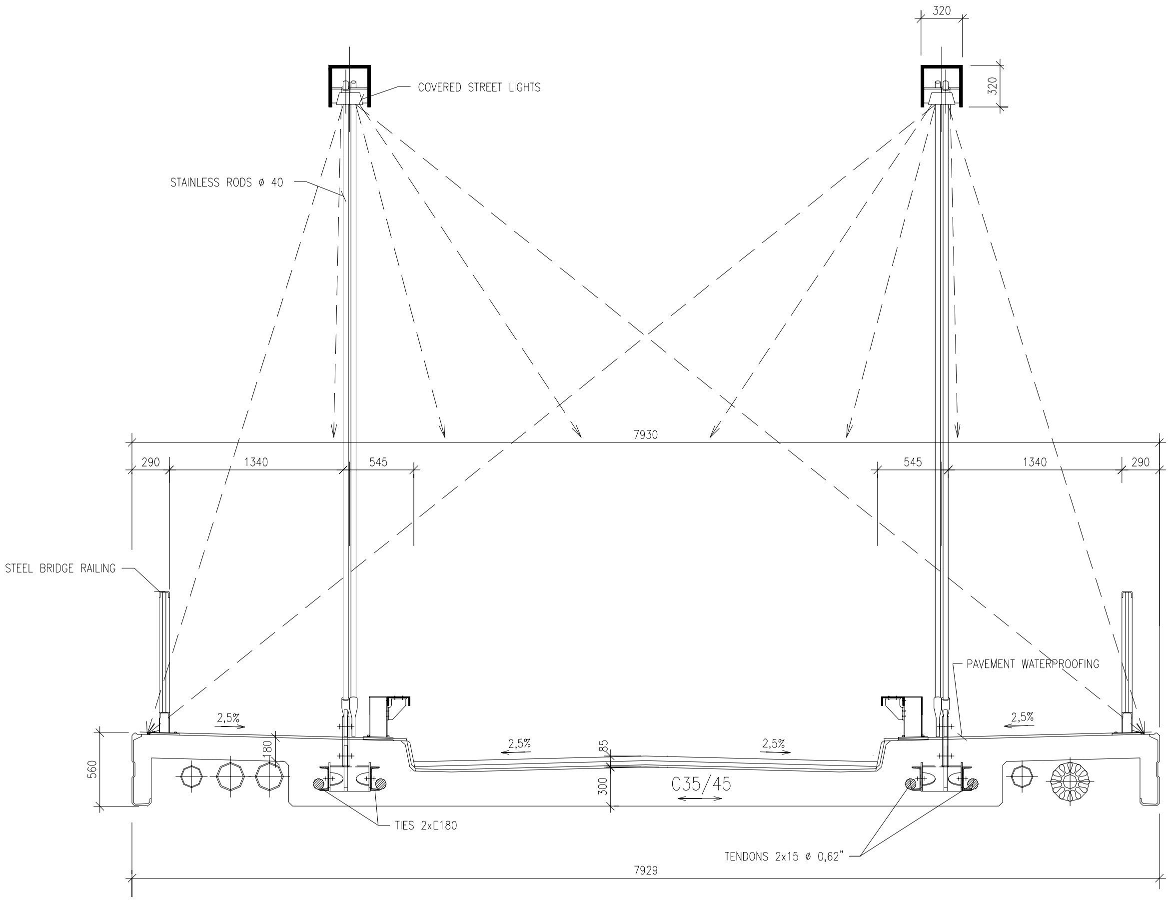


Fig. 96b. Cross-section

Fig. 96c. Bechyne network arch by day

Fig. 96d. Bechyne network arch by night

COMMENTS BY PER TVEIT ON THE NETWORK ARCH AT BECHYNE

I want to congratulate Dr. Ladislav Šašek on the design of the network arch at Bechyne. I never knew about it until it was finished. Dr. Šašek looked at my homepage and went ahead with the design. I am most impressed.

The method of erection suggested on page 92c is very like the method that I suggested in June 2004, Tveit 2004, but that is a coincidence. The way Dr. Šašek hides the cables and tubes under the footpath is new and very recommendable. See fig. 96b. (previous page). I would have liked the arch to be part of a circle.

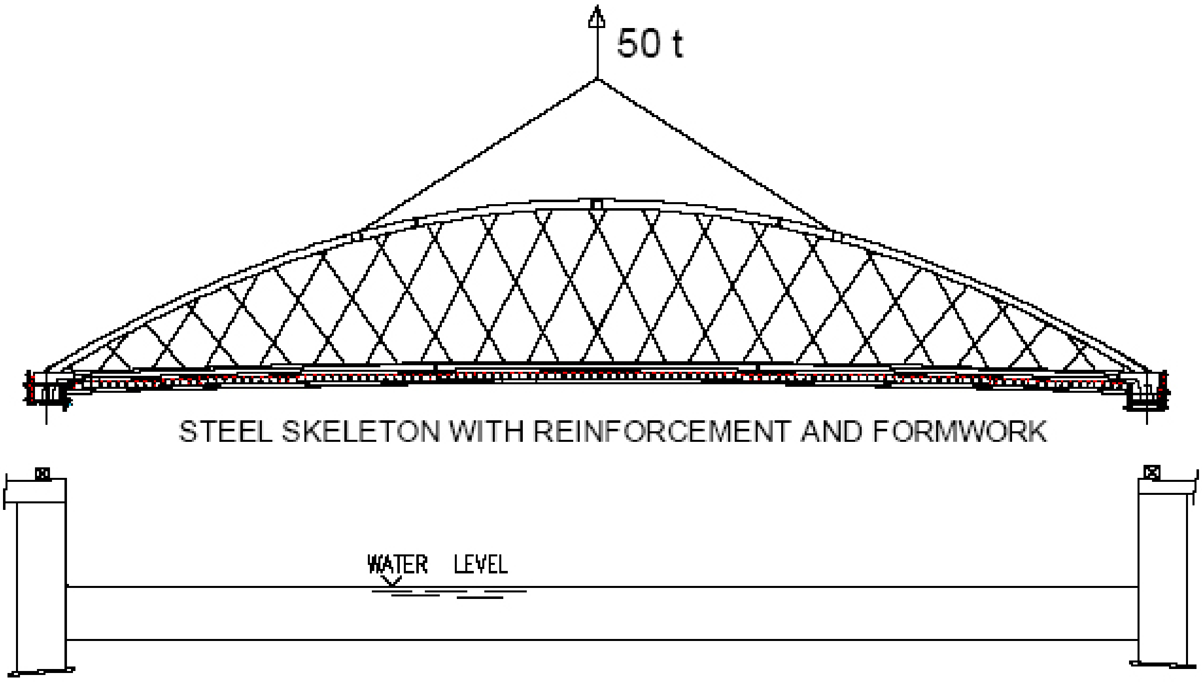


Fig 96e. Proposed method of erection for the Bechyne network arch

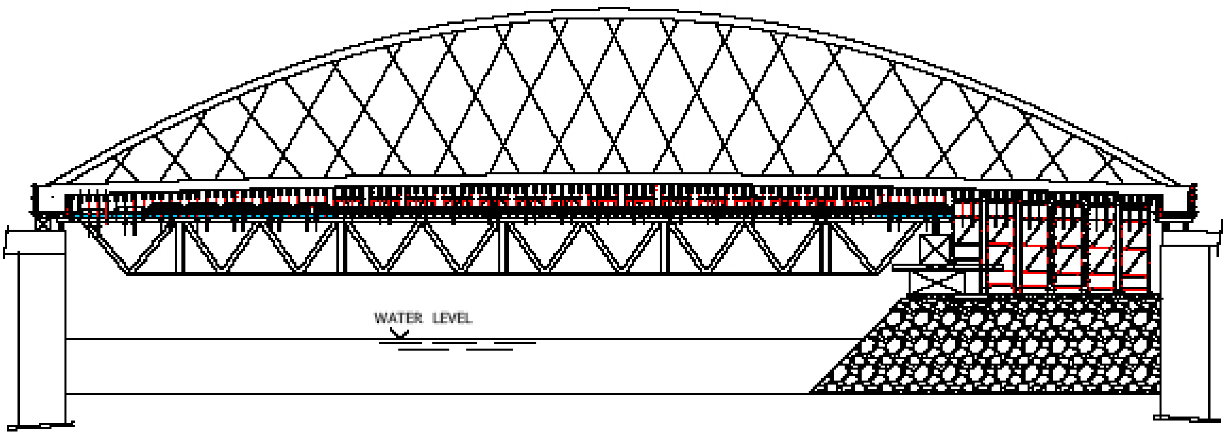


Fig. 96f. Contractor’s method of erection for the network arch at Bechyne

The amount of materials used per m^2 is:

Concrete	0.316 m^3
Reinforcement	66 kg
Prestressing steel	9.3kg
Construction steel	101 kg

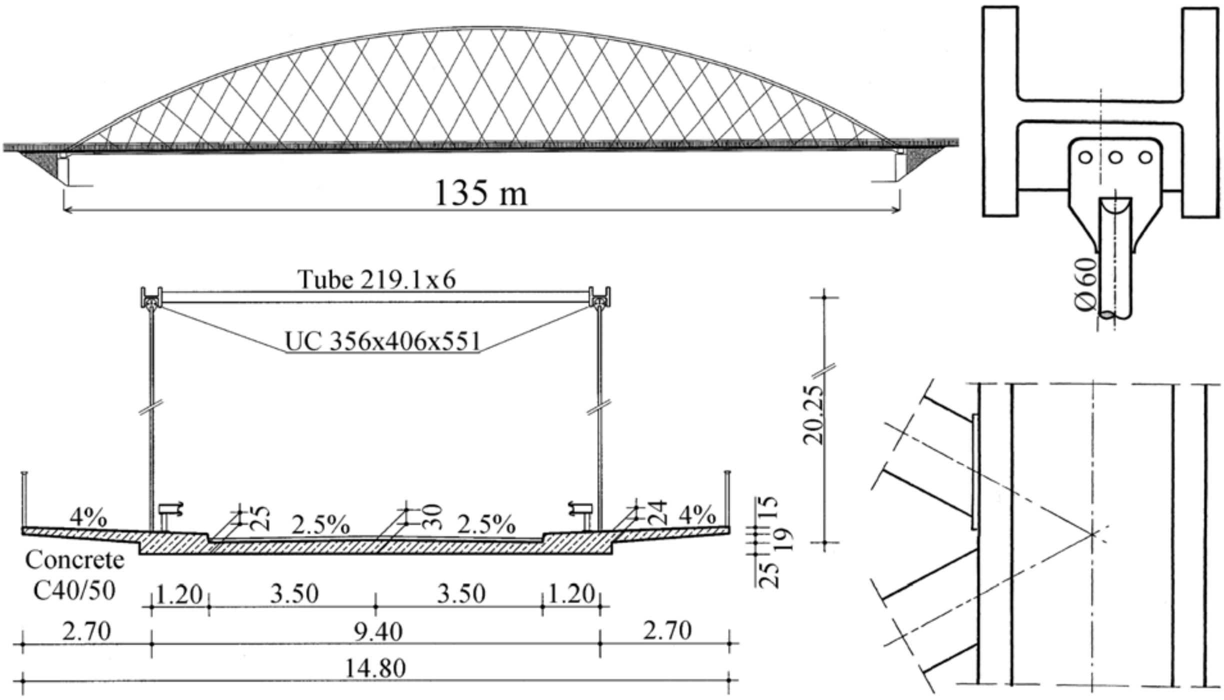


Fig. 97. The Åkviksound network arch designed according to EU codes. Teich and Wendelin 2001.

OPTIMAL NETWORK ARCHES COMPARED TO OTHER STEEL BRIDGES

Two very able students of Professor W. Graße in TU-Dresden wrote their graduation thesis in Norway in the summer of 2001. They calculated the optimal network arch in fig. 97. The resulting steel weights per m^2 of useful bridge area are shown on the left in fig. 98. The loads and codes of the EU were used. Where there was doubt, solutions that gave the bigger steel weight were adopted. A revised version of the students' graduation thesis can be found at http://fag.grm.hia.no/fagstoff/ptveit/ Fig. 98 compares the steel weight of the network arch in fig. 97 to steel weights in recent German arch bridges with vertical hangers. The year that the bridges were built is indicated. Bridges marked N have no wind bracing. In bridges marked S the arches slope towards each other.

It is surprising that the network arch tends to use less reinforcement in the tie than the bridges that have steel beams under the concrete slab. Part of the reason for this is the high amount of minimum reinforcement that is needed in the slab that lies on top of the elongating longitudinal steel beams in the tie. In the optimal network arch the moderate longitudinal prestress in the serviceable limit state reduces the need for minimum reinforcement. Furthermore it seems to the author that when the concrete slab can carry the concentrated load, not much extra reinforcement is needed to take the load to the edge beams.

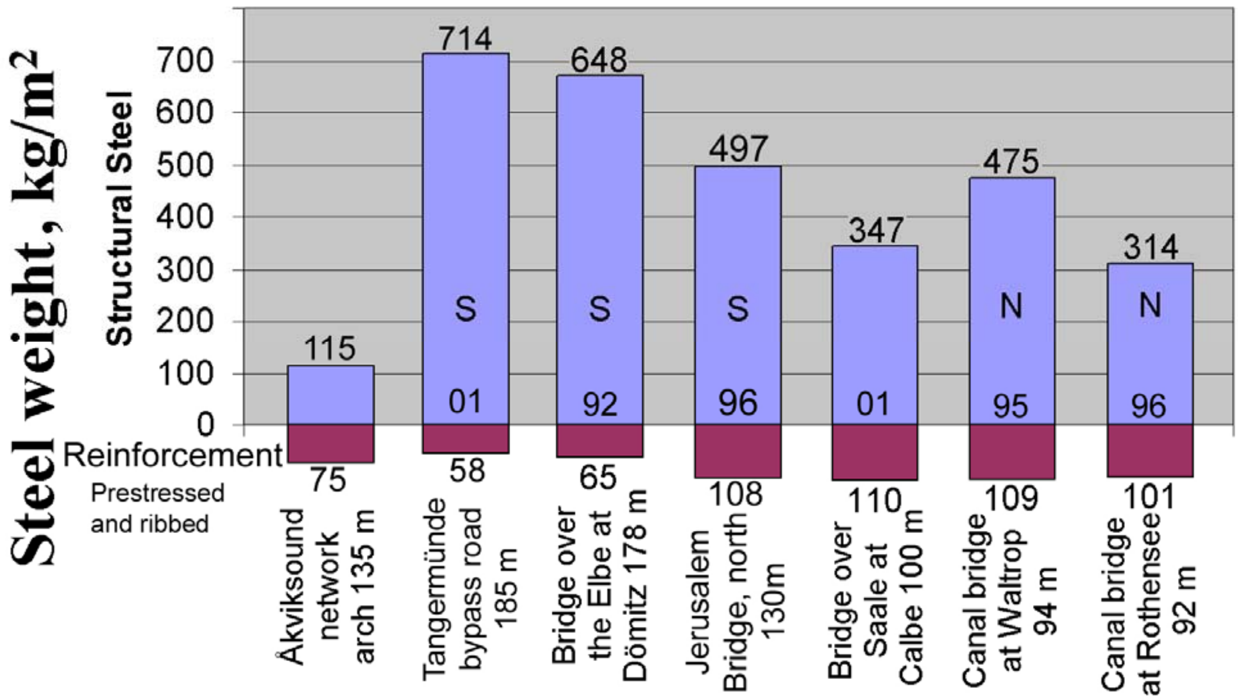


Fig. 98. Steel weight per square metre for various arch bridges

The Jerusalem Bridge in Magdeburg uses more than 4 times as much structural steel per m^2 as the Åkviksound Bridge. The spans are nearly the same. The Åkviksound Bridge comes out better because the area under the arches is included in the surface that is used for calculating the steel weight in kg/m^2. Different methods of erection contribute to the great variation in steel weight in fig. 98. A temporary tie for the Åkviksound network arch would need a steel weight of 12 kg/m^2.

In the Åkviksound network arch pre-bent H-profiles made of S 460 are assumed. In 2003 the steel mill said that these profiles would cost between 22% and 30% more than straight profiles in grade S 355.

Steel weight is not the only thing that matters. The table in fig. 99 indicates about other things that might be important. Among other things it gives reasons why the steel in network arches might cost less per ton than the steel in arch bridges with vertical hangers.

The references with the steel weights in fig. 98 can be found in [Tveit 2002]. [Herzog 1975] says that arches use about the same amount of steel as other steel bridges. This indicates that network arches use much less steel than other road and rail bridges.

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
Fabrication	15 to 30 times longer welds. 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 • Floating into place • Erection on side-spans • Erection on ice	Erection is more expensive with 2 to 4 times more steel.

Fig. 99 compares the optimal network arch to arch bridges with vertical hangers

SAVING OF COST BY USING NETWORK ARCHES INSTEAD OF OTHER STEEL BRIDGES

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 on materials. An incomplete list of publications can be found on pages 98 to 98b.

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.

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 is compared to an arch bridge with vertical hangers spanning 100 m built over the river Saale near Calbe in Germany, Fiedler and Ziemann 1997. See fig. 98.

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 a network arch designed by Teich and Wendelin 2001. See fig. 97. A revised version of their work can be found on the author's home page, at http://pchome.grm.hia.no/~ptveit/

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 the two spans are presented.

Permanent load per span:	Calbe		Network arch
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 m^2 :((9.0-2.5)·3·100+ 1390·2.5)0.981/10= 532 t
Network arch: 2205 m^2 :((9.0-2.5)·3·100+ 2205·2.5)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:	(0.9·2+2)100	0.12·207[m]	1·100	2·100	Σ 701 m²
Network arch:	(0.424·2+0.6)150	0.06·1528[m]	1·150	2·150	Σ 759 m²

The vertical load on the support is about 7 % smaller for the Calbe Bridge.
The area exposed to wind is approximately 8% smaller for the Calbe Bridge.
The useful area of the bridge is approximately 6 % 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	63 %
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. These additional facts 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. The network arch has simpler details and much shorter welds. See fig. 99 page 93a, 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.

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. Many good civil engineers will not believe that these savings are possible. Nevertheless the author hopes that most of them will consider trying network arches at suitable sites. 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.

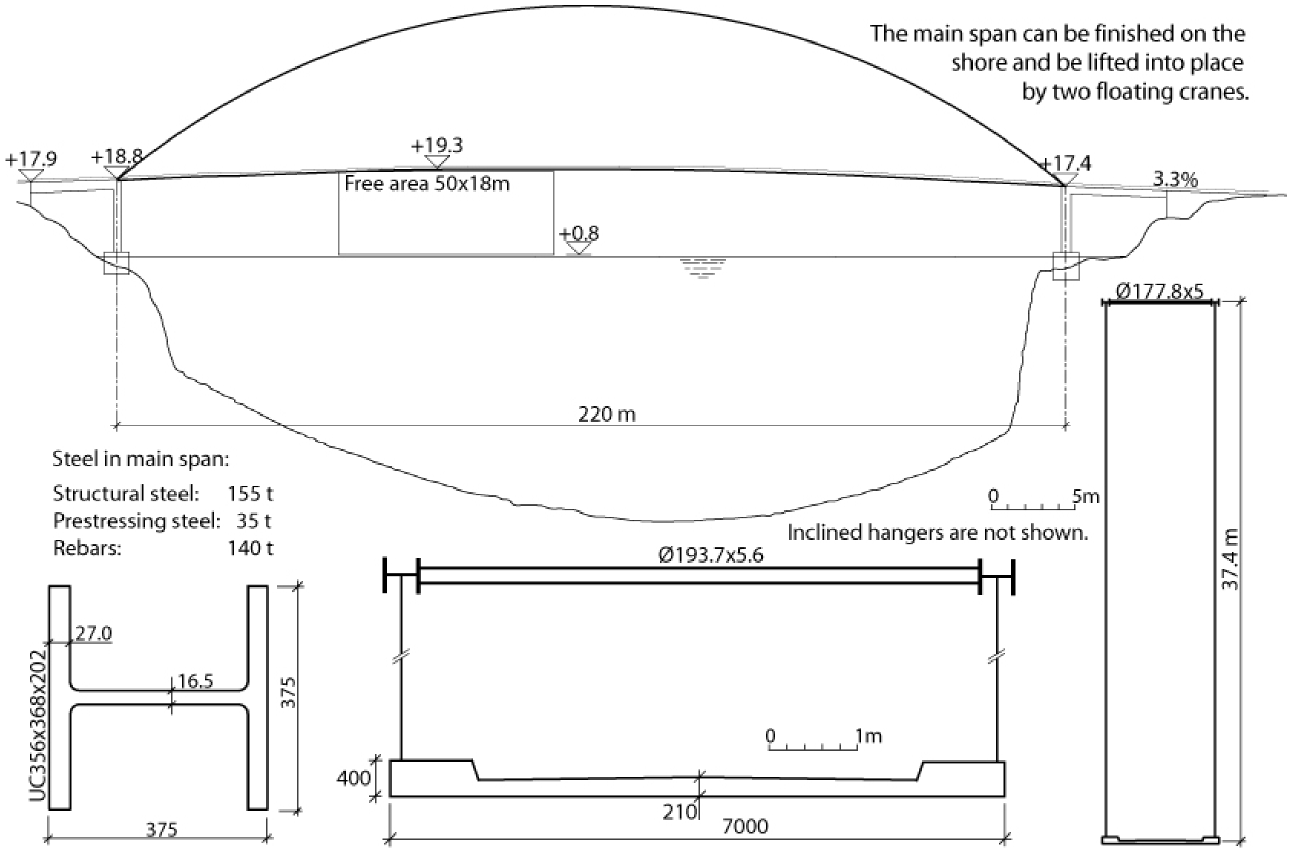


Fig. 99. The Brandanger Bridge in western Norway. The arches might be tubular in the end.

THE BRANDANGER BRIDGE AND HOW TO BUILD IT

The Bridge Technology Section of the Norwegian Public Roads Administration is improving on the design of the bridge in fig. 99. The design has been suggested by the author. It has a span of 220 m and a rise of 0.17 times the span. It 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.

The Brandanger bridge is likely to remain the world's most slender arch bridge for many years to come. In this slender span the bending in the middle of the tie due to wind is a problem. This might be overcome by a combination of higher concrete strength, tubes in the arch, deeper edge beams, a more aerodynamic lower chord or, as a last resort, including the side-spans in the tie.

You might wonder whether this very slim bridge will have unpleasant vibrations during strong winds. It will not, because it has roughly the same length and width as the Fyksesund Bridge. That is a Norwegian suspension bridge from around 1940. It was strengthened about 20 years later. The Brandanger Bridge is much stiffer and has an aerodynamically better tie; thus it will not have harmful vibrations. See also p. 24.

According to Brekke 2005 the Bridge Technology Section of the Norwegian Public Road Administration has found that 5 million euro can be saved by using the network arch instead of a relevant beam bridge. It will be up to the contractor to decide how the network arch should be produced and erected. Here the author would like to present some ideas that should not be taken too seriously, because of his limited knowledge of the production of steel structures. Still some of his ideas might be used in the production of other network arches.

<u>Production of the bracing between the arches.</u> First the end plates are welded to the horizontal tubes that are fastened between the arches just above the traffic. Then it can be seen how much the welding deforms the end plates that are to be screwed onto the arches. Next two posts with the correct distance between them could be erected for the welding of the plates on the ends of the other tubes between the arches. The end plates on the tubes are screwed to plates on the posts before the welding. The plates on the posts have a shape that contributes to making the end plates plane after they have been welded to the tubes. This will keep the cracks next to the arches small.

Production of the arches. The arches are made from steel S 460 M with a yield stress F_y=430 MPa for the relevant thickness. Higher steel strength could be considered. The arches could be subdivided into lengths of about 10 m. They are bent to the right curvature at the steel mill. First the plates at the upper ends of the hangers are fastened to the arches. If the shape of the arches has changed then they must be bent to the right shape. Heat that could dramatically reduce the strength of the arches must not be applied.

Fig. 19 shows the end plates between the arches. They must be bound in pairs before they are fastened between the parts of the arches. The end plates for the arches are first welded to one end of the arches. Then the other plate is welded to the other end of the arch in the joint. One must make sure that the joint has the right shape. This must be done by careful grinding of the ends of the arches.

After the welding is finished the two arches must be laid out on the ground to check the shape. In Steinkjer the author was surprised when the transverse plates in fig. 62 were not flat. Small adjustments were made by putting small, thin steel plates between the transverse plates.

Erection. First the concrete tie is cast with a suitable longitudinal and transverse camber. The longitudinal camber should be smaller than in fig. 99 because it is more economical to have more rockfill in the eastern approach and less under the tie when it is cast. The ends of the arches and rods for fastening the lower ends of the hangers stick out of the concrete. The lower ends of the hangers are shown in fig. 100.

Two parallel parts of arches with at least three tubes between them are joined together before they are lifted up to almost 40 m onto the scaffolding. See fig 99a. The chosen length depends on the lifting capacity of the crane. The weight is about a half tonne per m. The weight depends on the rise of the arch, the steel quality etc. Each of these parts must be supported at 4 points or more. When the temperature is right the last bit of the arch can be put in place.

If the joints in the arches are welded, then the distance between the joints can be increased to 15 to 20m depending on availability, transport problems, crane capacity on site etc. The joints can be like in fig. 19.

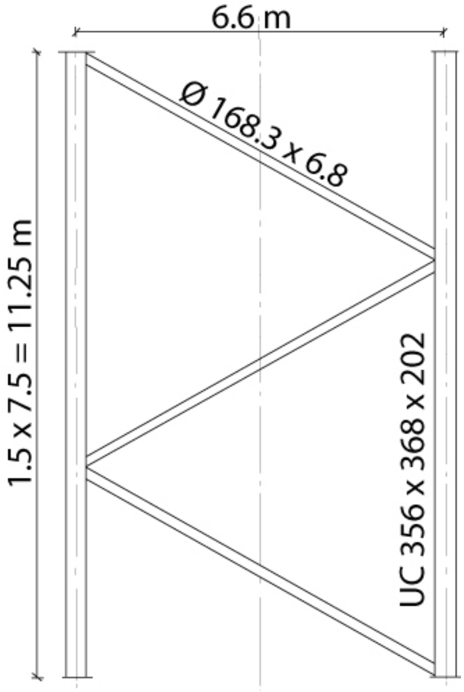


Fig 99a. Part of arch and windbracing ready for lifting.

When the arches are in the correct position and the hangers are put in, the lower end of the hangers can be welded to the rods sticking out of the concrete. See fig.100. The welds to these rods are made on site and have a lower stress than the other welds in the hangers.

Lifting in place of the main span. The main span is built on a very flat rockfill at the shore 4 km from the bridge. It might weigh 1200 to 1400 tons. It will be lifted in place by two or more big floating cranes. At least one floating crane must be used at each end of the span. Some floating cranes can lift over 2000 tons. The cranes can hook themselves to the plates at the ends of the arches. See figs 6b, 18, 39a.

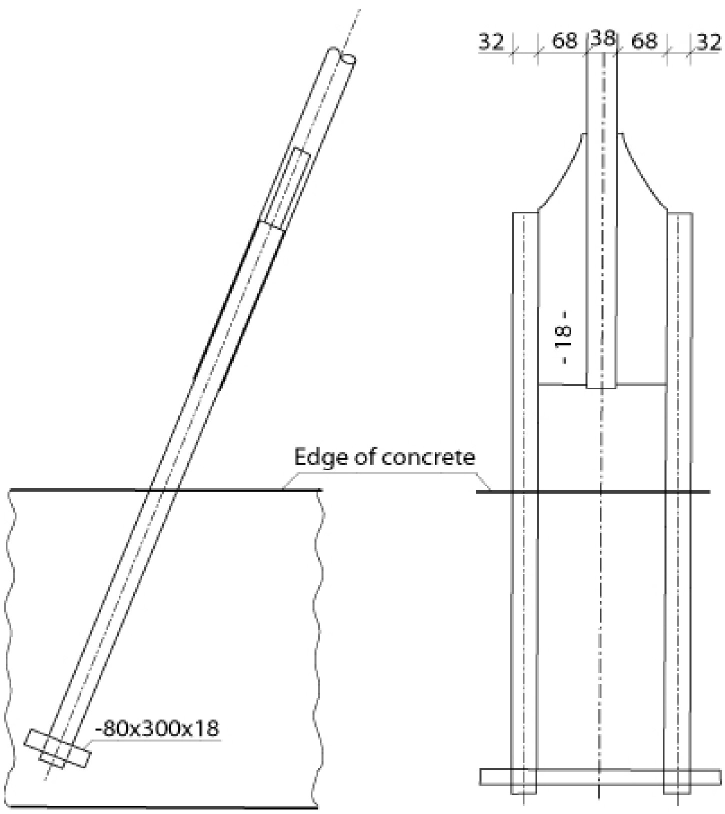


Fig. 100. Lower end of a hanger for the Brandanger Bridge

HOW TO MAKE A PRELIMINARY DESIGN OF A NETWORK ARCH FOR A ROAD BRIDGE

This chapter has been moved to http://fag.grm.hia.no/fagstoff/ptveit/ There the title is: "On the preliminary design of network arch bridges." The chapter gives simple advice on how to find dimensions for a suggested network arch. The purpose of such calculations is to arrive at the preliminary dimensions that can be put into a general frame program. The publication will also be a good help to anyone who wants to find the amount of materials needed in a network arch. The spans are 93, 120, 135 and 160 m.

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This list also contains references relevant to network arches not mentioned in this publication.

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