EDITION 10.03.03

PRELIMINARY DESIGN OF NETWORK ARCH ROAD BRIDGES

Examples with spans of 135 and 160 metres

These instructions are written for future workshops on the design of network arches. Those that take part in the workshops are meant to design their network arches prior to coming to the workshop. The author hopes that these instructions will be of use to engineers that do not take part in the workshops.

Network arches for road bridges seem to be very competitive for spans of 135 to 160 m. For longer two lane spans the arches can seldom be universal columns or American wide flange profiles. Then closed box sections seem to be the answer. Network arches might be competitive for spans up to 300 m and above.

The advice in this publication will be applied to two bridges with spans of 135 and 160 m. The instructions will be written in Times New Roman. The text relevant only to the 135 m span is written in Batang. The text relevant only to the 160 m is written in Arial.

The designs in these instructions are made in accordance with the loads and codes of the European Union. The data for the 135 m span are supported by the graduation thesis that Stephan Teich and Stefan Wendelin made in Grimstad in 2001. [Teich & Wendelin 2001] A revised version of the thesis can be found in the author's homepage. See next paragraph.

In these instructions the author will often refer to "The Network Arch" (TNA). That is over 100 pages on network arches that can be found on the Internet at: http://pchome.grm.hia.no/~pert This home page will be updated at irregular intervals.

Because the axial forces are dominant in the chords, a simple preliminary calculation can give reliable information on the amount of steel needed for the chords. It is more difficult to decide on the steel needed in the hangers, but that is only 10 to 15 % of the total steel weight.

These instructions are for bridge engineers who want to design optimal network arches. They offer a choice of dimensions that can be put into a computer program when network arches are designed. They will also give some of the data needed to compare network arches to other alternatives.

It saves time to make the preliminary design of a network arch in the following sequence:

1. <u>Decide on the width of the parts of the bridge that carry traffic</u>

That means deciding on the width of the roadway and footpaths. This decision will depend on the expected traffic. If the traffic over the bridge is expected to grow quickly over the years, it is sometimes best to build network arches for the traffic expected in the relatively near future and plan for another network arch to be built when the traffic warrants it and funds are available. [Tveit 2000].

In network arches it is recommended to put the footpath outside the arches. This reduces the span of the slab between the arches. See fig. 1 on the next page. When the distance between the arches is up to 15 to 18 m, a simple slab can be used between the arches. For slabs spanning more than 12 m transverse prestressing should be considered. Both spans in these instructions use the tie that is shown in fig.1. The author thinks that the distance between the traffic and the arches is bigger than necessary considering the solidity of the lower part of the arch.

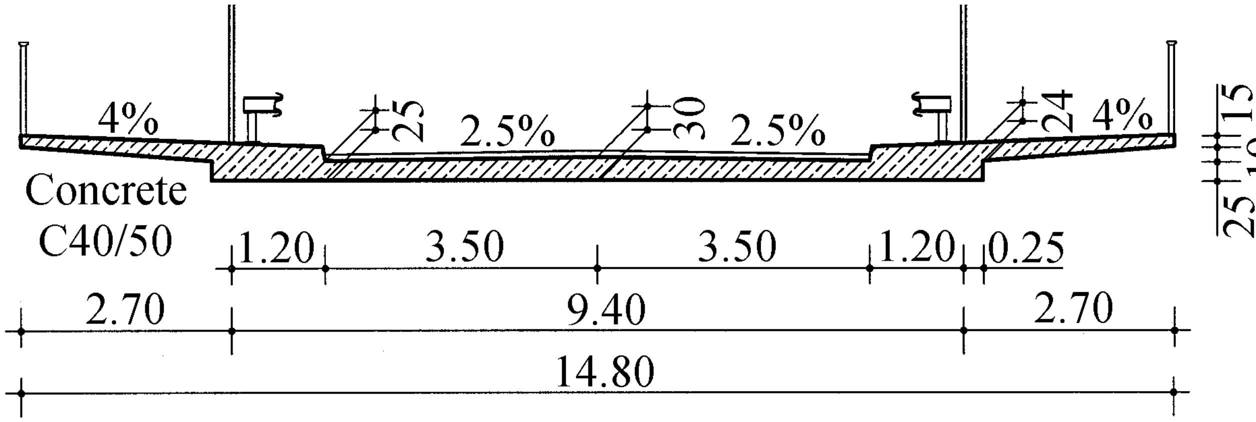

Fig. 1. Cross-section of the tie in both spans.

2. Decide on the span

Often the site, more or less, decides the length of the span. Since the network arches are so light and use so little material, they should normally have longer spans than other bridges that could be used for the same site. For spans under 160 m the cost of concrete, reinforcement and formwork per metre span is nearly independent of the span.

For a sequence of spans under 160 m we are probably near an optimum if the costs of the structural and prestressing steel in one span are nearly the same as a pillar. This rule disregards the fact that the method of erection has great influence on the optimal length of a span. Methods of erection can be found in TNA pages 6, 12, 15, 20, 21 and 50. See also the index on page 101.

One span is 135m.

The other span is 160m.

3. Make an educated guess on the width of the arch

For spans up to 170 m arches made of universal columns and/or American wide flange beams are highly recommended by the author. These compact cross-sections can take high buckling stresses when used in network arches. Furthermore they give simple details. See TNA fig. 16 to 19.

For the 135 m span the universal columns in the arch are assumed to be 0.456 m wide.

For the 160 m span the universal columns in the arch are assumed to be 0.475 m wide.

At the end of the arches the profiles are a couple of centimetres wider.

4. Decide on the rise of the arch

For aesthetic reasons the author favours a rise of 15 % of the span. This rise of arch will be assumed. A bigger rise would give smaller steel weights. Most Japanese network arches have a rise between 15% and 17% of the span. [Nakai 1995]. The author would consider a rise above 15 % of the span if the bridge has hilly surroundings.

A rise of the arch of 0.15 of the span is chosen for both spans.

5 Decide on the distance between the planes of the arches

According to fig. 1 the distance between the planes of the arches is 9.4 m.

6. Decide on the quality of concrete

A high concrete strength usually gives a more durable structure. In Norway a cube crushing strength of 55 MPa is recommended. In Germany slightly lower strengths tend to be favoured.

7. Choose the dimensions of the concrete slab between the arches

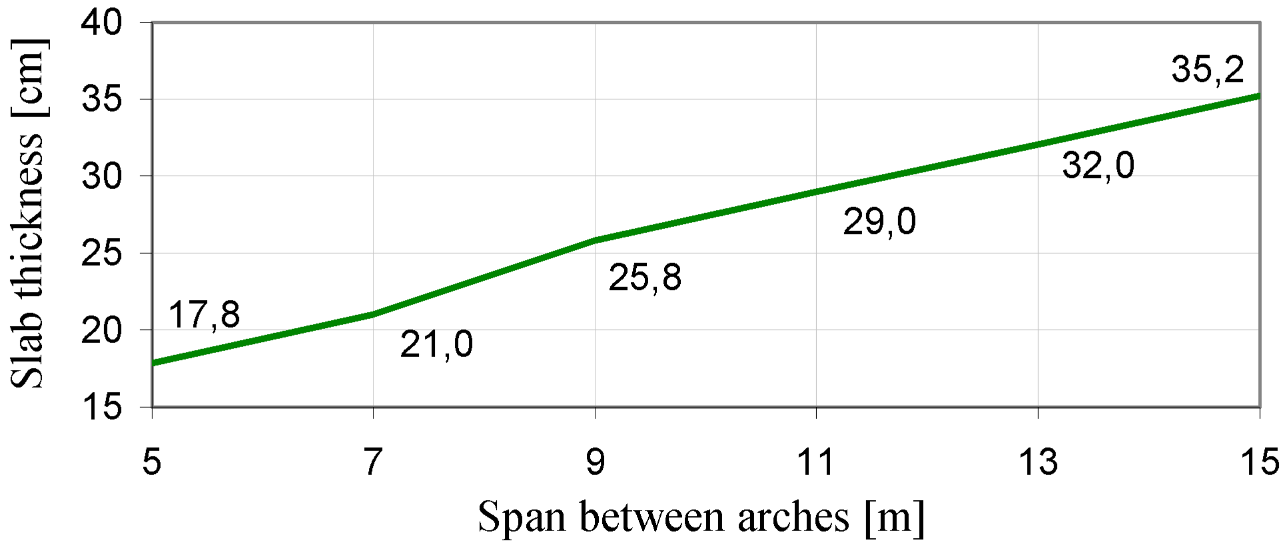

Fig. 2. Necessary thickness of slab between the arches. [Teich and Wendelin 2001]

The maximum thickness of the slab between the arches can be taken from fig. 2. The thickness is on the safe side for the loads in the Euronorm. Thus it is sufficient for most national codes. The thickness of the slab in fig. 1 is 2 cm bigger. That can be a good thing because the highest stress might occur slightly to the side of the middle of the slab.

Since the forces in the slab will be controlled by the computer calculation, it is not worthwhile to make the preliminary calculation of the slab very carefully. The deflection of the slab can be counteracted by a suitable camber.

8. Decide on the shape of the lower chord

This decision has been made when adopting the cross-section of the tie shown in fig. 1. Maybe the plate should be made 1 or 2 cm thicker between the middle of the slab and the gutter.

9. Calculate the permanent load in the serviceable limit state

The permanent load is calculated for one arch. The layer of asphalt is 8 cm thick. The weight of the asphalt is assumed to be 24 kN/m^3. The weight of the concrete is assumed to be 25 kN/m^3.

The concrete weighs	49.76 kN/m
The asphalt weighs	6.72 kN/m
Railings and guardrails are assumed to weigh	1 kN/m
Permanent load on the tie in the serviceability limit state	57.48 kN/m

Assumed steel weight of the steel structure above the tie in the serviceability limit state:

For a span of 135 m: 8 kN/m.

For a span of 160 m: 9.5 kN/m.

10. Find the live load in the serviceability limit state

The maximum evenly distributed live load on one arch is shown in fig. 3.

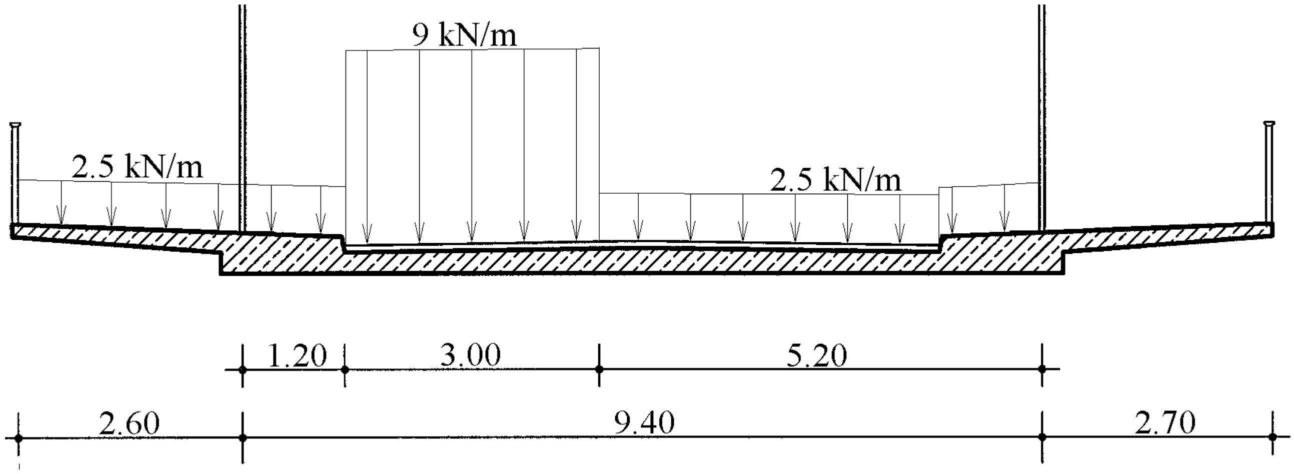

Fig. 3. Evenly distributed live loads on one arch.

Maximum evenly distributed live load on one arch:

p=9·3·7.7/9.4+2.5·3.8·10.1/9.4+2.5·5.2·2.6/9.4=28.6 kN/m

The maximum concentrated live loads on one arch are shown in fig. 4. The axle loads are really placed 1.2 m apart, but here they are drawn together. Bear in mind that the axle loads distribute quite a lot before they reach the arch.

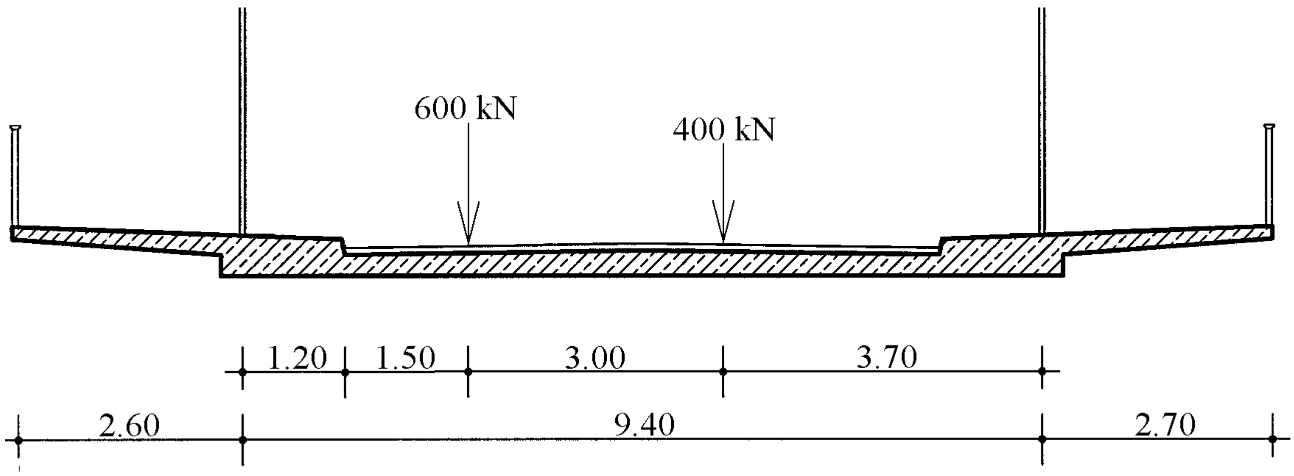

Fig. 4. Concentrated live loads on one arch.

P=600·6.7/9.4+400·3.7/9.4=585 kN

11. Find the maximum loads on your network arch in the serviceability and the collapse limit state

The permanent load in the serviceable limit state is 57.5+ 8=65.5kN/m for the 135 m span and 57.5+ 8.5=67kN/m for the 160 m span.

The permanent load on the tie in the serviceable limit state is given under point 9.

If half the asphalt is worn away, the permanent load on the tie becomes 57.48-3.36=50.1 kN/m for both spans. The maximum evenly distributed live load on one arch in the serviceability state is calculated in subsection 10.

The evenly distributed live load in the collapse limit state is 28.6·1.5=42.9 kN/m for both spans.

12. Make a tentative decision on the system lines on the span
The hangers should be placed equidistantly along the arch. Page 26 in TNA gives some advice on the arrangement of the hangers. A distance between 2.6 and 4 m seems a reasonable choice. The distance depends partly on the span of the network arch. Smaller spans should have shorter distances between the nodes to reduce bending due to the curvature between the nodes in the arch.

The network arches in Steinkjer and Bolstadstraumen, (TNA pages 6 and 7 and pages 56 to 58), were built on timber structures resting on piles in the riverbed. All hangers should have about the same maximum tension. The slope of the hangers influences the tension.

For the bridge at Steinkjer the average maximum hanger tension was 93% of the maximum hanger tension. For the Bolstadstraumen Bridge the average maximum tension in the hangers was 91.5% of the maximum hanger tension. Such results are dependent on codes, loads and weights.

If a temporary lower chord is used for the erection, the choice of distance between the nodes could also depend on the timber beams available for supporting the formwork in the temporary lower chord. The choice could depend on the deflection that can be tolerated when casting the slab between the transversal beams in the temporary lower chord.

The skeleton lines shown in fig. 5 come from [Tveit 1980]. The skeleton lines on the left, which are called Vienna 200A, gave the best results for the 200 m span designed for the loads and the concrete strengths used then. With increasing loads and higher concrete strength the skeleton lines on the right will become more relevant. In TNA they are called ViennaB.

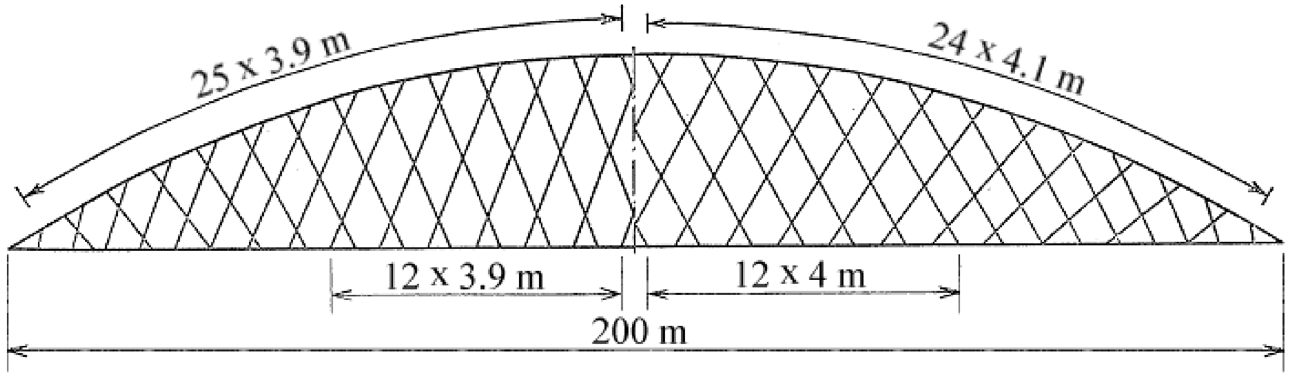

Fig. 5. System lines for two different ratios of live load to permanent load. [Tveit 1980]

In these instructions the skeleton lines in ViennaB reduced by a suitable factor will be used. The geometry of Vienna 200B reduced by a factor of 0.675 has been used by [Teich and Wendelin 2001]. The same skeleton lines will be used for our 135 m span.

The same skeleton lines were also used in the Norwegian network arch for the Åkviksound. Data for that network arch is found in TNA pp.73 to 93. If the ratio of live load to permanent load is higher than 0.70 to 0.75, the hangers should be less steep than in 200B and another hanger arrangement should be chosen.

When calculating the effect of concentrated loads, they are converted into evenly distributed loads by multiplying by 2 and dividing by the length of the influence line. This is because the maximum ordinate of an influence line is usually nearly twice as big as the average ordinate. This is the case for the influence lines for the axial forces in fig. 63 and 64 in TNA.

The model laws given in TNA page 56 are useful when utilizing a set of influence lines. We can use the influence lines for the Norwegian Åkviksound for our spans. The influence lines for the 135 m span can be used as they are.

For the 160 m span the influence lines for the axial forces can be used as they are. Only the length of the influence line has to be changed. For the influence lines for the bending moments we also have to multiply the ordinates by a factor 160/135=1.185.

Supporting the use of the influence lines of the Norwegian Åkviksound bridge, is the fact that we are likely to have about the same relative stiffness between the arch and lane in our bridge. The axial force gives the major part of the stresses in the arch. Thus it is not necessary to have a very precise prediction of the bending moments in the arch. The bending in the tie influences only the longitudinal reinforcement but not the dimensions of the cross-section.

The skeleton lines for Vienna 200A and 200B (TNA pages 59 to 72) are reasonably well suited for spans that are to be erected using a temporary lower chord. The nodes are equidistantly placed in the arch and in the middle half of the tie. To make things easy for ourselves we choose the skeleton lines from Vienna 200B for the spans of 135 and 160 m. See TNA pp. 69 to 72. This distribution of hangers will give nearly the same maximum force in all hangers. In the rest of these instructions a geometry like the one in Vienna 200B will be assumed.

What geometry to use is very much influenced by the hanger's resistance to becoming relaxed. Why this is important is explained in TNA pp. 69 and 70 and maybe better in [Tveit 1987 pp. 2191-2193]. Here the only comment is that the network arch functions much like a truss as long as none or only a few of the hangers relax. When hangers relax, the bending moments increase. A moderate increase is not dangerous because the axial force is much reduced in the load cases that lead to relaxation.

Fig. 6 shows the load intensity and the hangers', resistance to becoming relaxed for the best network arch in [Tveit 1980]. The system lines to the left in fig. 5 have been used. The drawing is explained in TNA p. 26 onwards. The thin curved lines indicate the load intensity when the span is loaded from the left.

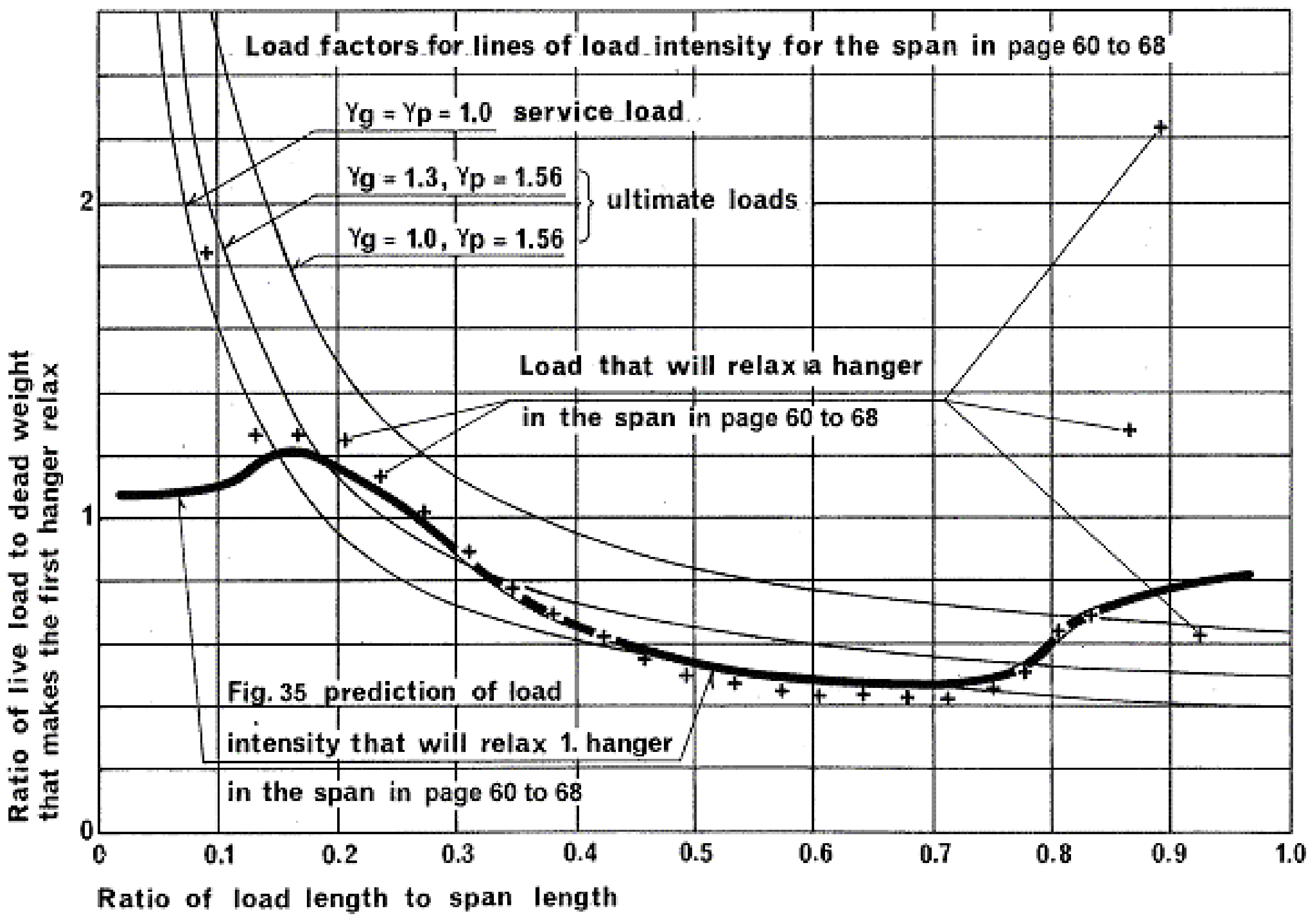

Fig. 6 Relaxation of the first hanger according to calculation (+)
and prediction according to fig. 38 in TNA p 29. (Thick line)

The concentrated load is included in the load intensity by multiplying it by two and dividing it by the loaded length. This is reasonable when the influence lines for the hangers are triangles. This is a rough approximation considering the shape of the influence lines for ViennaB and for the Åkviksound bridge. See TNA pp. 74, 83 and 84, but it is really fig. 36 TNA that matters.

The crosses indicate the load intensity that has been calculated to relax the first hanger for various loaded lengths. The thick line indicates the prediction of the relaxation of hangers according to fig. 38 in TNA.

In fig. 7 the system lines on the left of fig. 5 are used for the span of 135 m. Fig. 7 shows some of the same lines as fig. 6. The thick line in fig. 7 shows the prediction of load that will relax the first hanger when the span is loaded from the left. The thin lines show the load intensity when the span is loaded from the left.

Looking at fig. 6 and 7 we can see that the hanger's tendency to becoming relaxed is biggest when the load is on a little more than half the span. We can also see that the ratio between the hangers' resistance to becoming relaxed and the load is bigger in fig. 7 than in fig. 6. This indicates that relaxation of hangers is a smaller problem for the span in fig. 7. We cannot be sure of this because the arch of span in fig 6. is much stiffer. [Teich and Wendelin 2001] found that relaxation of hangers was not a problem for the span in fig. 7.

For the 160 m span it is assumed that the relaxation of hangers will not create grave problems because here the live load is less intense since the concentrated load is a smaller part of the total load.

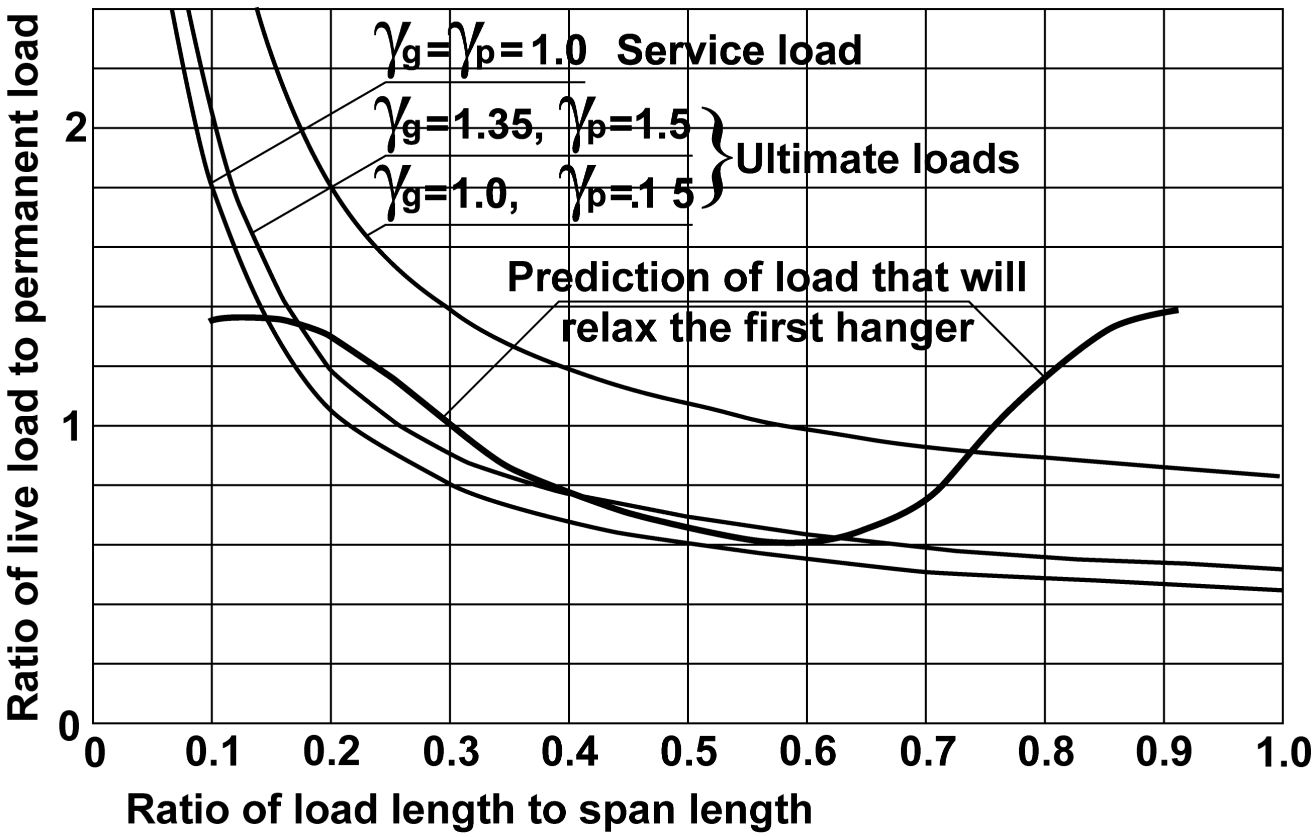

Fig. 7. Prediction of relaxation of the first hanger in the 135 m span according to fig. 38 in TNA p. 29 compared to load intensity on the span.

Now come considerations for the various parts of the bridge.

13. Lower chord

The axial force in the lower chord can be found using the influence line in TNA fig. 82. We do not normally need to do this in order to have dimensions to put into the computer. We do need to calculate the force in the lower chord if we want to find the amount of prestressing cables needed. It would however be tempting just to look at previous designs to find the likely amounts of prestressing cables needed.

14. Hangers

Use the influence lines in TNA fig. 83 to find the axial force in one or more hangers. The author suggests that hanger 108 is examined first. The other hangers have about the same hanger force so it is not necessary to examine more than one hanger to find the dimensions to put into the computer program.

If three hangers are to be examined, the author suggests that the hangers 44 and 172 could also be examined. If the hanger length has to be adjusted, it is probably best to use steel wires in the hangers. See TNA fig. 56. Otherwise round steel rods are recommended. See TNA fig. 17.

14a. Find the maximum load in the collapse limit state

Find the smallest cross-section of a hanger that can take this load. Influence lines can normally not be used for load cases that make some hangers relax. Do not worry about this. In load cases that make hangers relax there is an increase in the bending moments, but the maximum hanger force is reduced. Thus the influence lines give hanger forces on the safe side for load cases that make some hangers relax. This effect is shown in TNA fig. 45 and explained in TNA page 32.

The influence lines for the 135 m and 160 m span are shown in fig. 8. See next page. The ordinates of the two sets of influence lines are the same. The influence line for the 160 m span are longer than the influence line for the 135 m span. The ordinates are the same. Thus the areas under the influence lines for the 160 m span can be found by multiplying the areas for the 135 span by a factor 160/135.

Calculation of forces in hanger 108

Span 135: Positive inf. area: 6.19 m. Negative inf. area: 3.02 m. Sum of inf. areas 3.17m.

Serviceability limit state:

Maximum hanger force: $3.17 \cdot 57.5 + 6.19 \cdot 28.6 + 585 \cdot 0.3 = 534.8$ kN

Minimum hanger force: $3.17 \cdot 57.5 - 3.02 \cdot 28.5 - 585 \cdot 1.65 = -0.6$ kN

Collapse limit state:

Maximum hanger force: $3.17 \cdot 77.6 + 6.19 \cdot 42.9 + 878 \cdot 0.3 = 774.9$ kN

Span 160: Pos. influence area: 7.34 m. Neg. inf. area: 3.58 m. Sum of inf. areas 3.76 m.

Serviceability limit state:

Maximum hanger force due to live load: $3.76 \cdot 57.5 + 7.34 \cdot 28.6 + 585 \cdot 0.3 = 601.6$ kN

Minimum hanger force due to permanent load: $3.76 \cdot 57.5 - 3.58 \cdot 28.6 - 585 \cdot 0.165 = 17.3$ kN

Collapse limit state:

Maximum hanger force due to live load: $3.76 \cdot 77.6 + 7.34 \cdot 42.9 + 878 \cdot 0.3 = 870.1$ kN

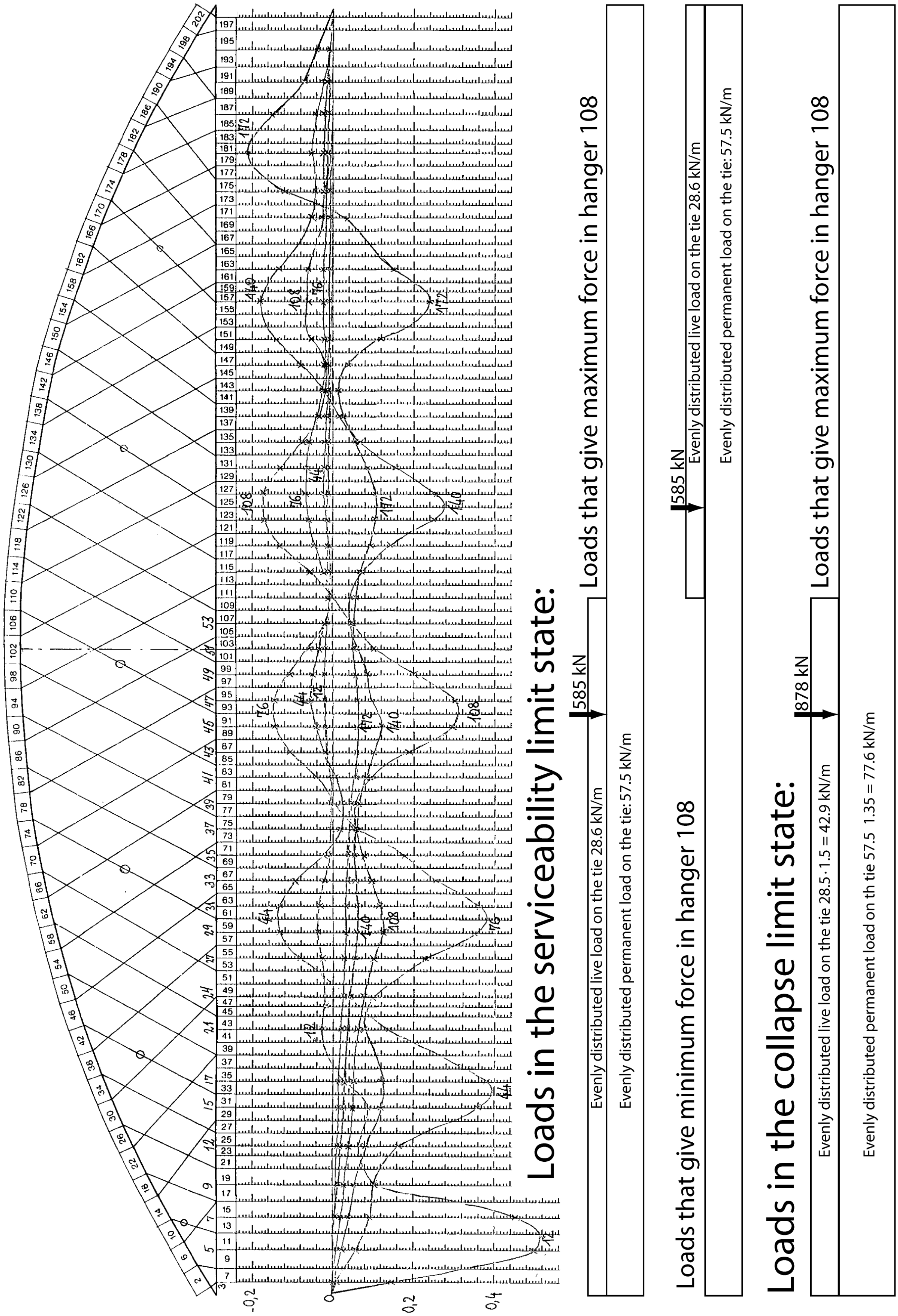

Fig. 8. Loads on the influence line for member 108 in 135 m span.

In the collapse limit state the maximum hanger force has been found to be 774.9 kN and 870.1 kN. Other hangers might have greater force. Use 825 kN and 930 kN when finding the necessary hanger cross-sections for the 135 m and the 160 m span.

For the 135 m span the necessary cross-section is:
$A = N_{t,Rd} \cdot \gamma_{MO}/f_y = 825 \cdot 10^3 \cdot 1.1/430 = 2110\ mm^2$
Choose D=55 mm $A=2380\ mm^2$

For the 160 m span the necessary cross-section is:
$A = 930 \cdot 10^3 1.1/430 = 2379\ mm^2$
Choose D=58 mm $A=2830\ mm^2$

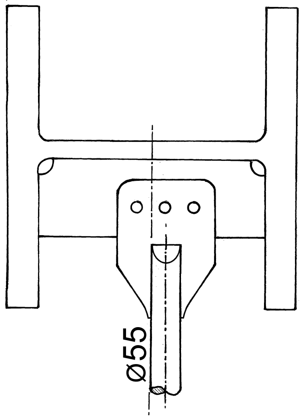

Fig. 9. Upper end of hanger.

14b. Find the stress variation in the fatigue limit state
Find a cross-section of hanger that will take this load. Use the bigger of the cross-sections found in 15a and 15b in the computer calculation of your network arch. It is here assumed that fatigue is not decisive partly because of the carefully executed and well rounded connection that is indicated in fig. 9 and 10. [Teich and Wendelin 2001] found that fatigue was decisive and led to 5 mm bigger diameter of the hangers. A wrong hanger dimension has little influence on the other members of the network arch. Thus it is not important if the preliminary hanger dimensions are too small.

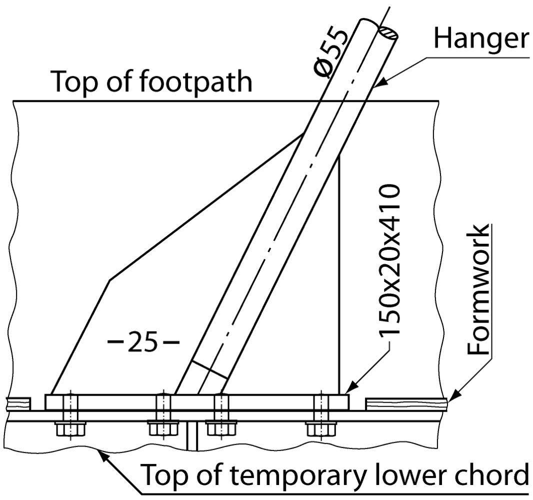

Fig. 10. Lower end of hanger.

15. Arch
The relevant influence line from fig. 81 in TNA can be used to find the axial force in the top of arch. See fig. 11.

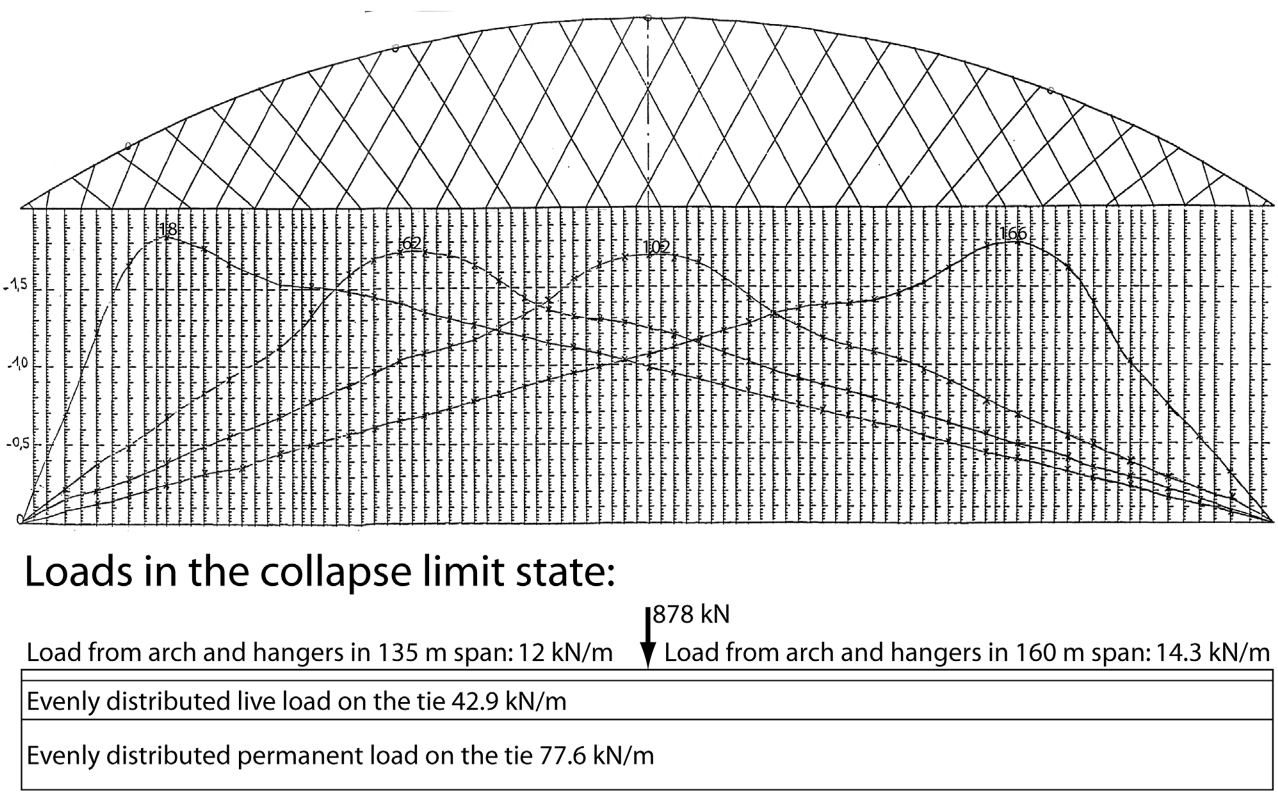

Fig. 11. Influence lines and loads for the top of network arches spanning 135 m and 160 m.

Span 135 m: Area under the influence line is 114.1 m.

Span 160 m: Area under the influence line is 132.2 m.

Collapse limit state:

Maximum axial force at the top of the arch: $114.1(12+42.9+77.6)+878 \cdot 1.7= 16.6 \cdot 10^3$ kN

Maximum axial force at the top of the arch: $135.2(14.2+42.9+77.6)+878 \cdot 1.7= 19.7 \cdot 10^3$ kN

Then dividing the axial at the top of the arch force by the yield stress 430 MPa and multiplying the result by a factor 1.5 will give a preliminary area of the arch that can be used in a computer program.

For the 135 m span: Area at the top of the arch: $16.6\ 10^6 \cdot 1.4/340=68.4 \cdot 10^3$ mm^2
Choose British Universal Column UC 357x406x551 $A=70.19 \cdot 10^3$ mm^2

For the 160 m span: Area at the top of the arch: $19.7 \cdot 10^6 \cdot 1.4/340=81.1 \cdot 10^3$ mm^2
Try British Universal Column UC 357x406x634 $A=80.75 \cdot 10^3$ mm^2

The factor was found for arches made of steel EN 10113-3: S 460 ML with a yield stress of 430 MPa for a nominal thickness over 40 mm. If steel with a lower yield stress is used, the factor is likely to go down slightly.

As can be seen from the influence lines in fig. 11, the normal force in the arch increases towards the wind portal. It might be necessary to increase the cross-section of the arch one step before we get to the wind portal, but that can be taken care of after the computer calculations have started.

The arch should be part of a circle. For spans up to at least 160 m the arch should be a universal column or an American wide flange beam. Hangers along the arch should be placed equidistantly. This arrangement gives the smallest bending due to local curvature of the arch when the span is fully loaded. Two hangers at each nodal point would give bigger bending moments in the arch due to local curvature and less efficient support of the arch in buckling.

The hangers nearest to the ends of the arch usually have smaller maximum forces than the other hangers. Increasing the distance between the end of the span and the nearest upper end of a hanger can to some extent counteract this phenomenon. The first hanger in the tie should normally be sloping away from the end of the span as in fig. 4.

In the tie the hangers should be placed equidistantly in the middle half of the span. The distance between the nodes should be the same as, or slightly smaller than, the distance between the nodes along the arch. See fig. 4. Different loads and codes give different optimal hanger arrangements.

16. The wind portal

The wind portal needs a bigger cross-section than the rest of the arch. Assuming that a universal column or an American wide flange beam is used, a 30 % increase of the cross-section would be a reasonably educated guess for road bridges. Maybe the lowest 2 to 3 m of the wind portal should have a bigger cross-section. This can be put right after the computer calculation has started.

We get smaller bending moments in the wind portal if the H-profile here has a smaller curvature. That would also make it easier to attach a side-span to the network arch. In the Steinkjer Bridge the last member in the tie nearest to the north abutment were made shorter in order to reduce the bending moment where the side-span was attached. See TNA fig. 6 and compare the length of the main span to the same distance in fig. 63.

The members between the last node and the end of the bridge could be a little longer than the other members. This would give more even hanger forces. If H-profiles are used in the arches, the lower half of last member in the wind portal should have a steel plate on top of the arch. The cavity under the steel plate should be filled with concrete.

17. Calculate the effect of relaxing hangers

This effect should be calculated in the collapse limit state. For the tie the load factor for the permanent load should be assumed to be 1.0 because the weight of the lower chord restrains the relaxation of hangers.

In Norway steel studs are used in car tyres in the winter. This gives considerable wear on the asphalt road surface. The author has assumed that the hanger's maximum tendency to relax will occur when half the asphalt is left on the road surface. In most other countries less wear on the asphalt can be assumed.

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Knowledgeable readers will understand that these hints are just a rough guide for the dimensions that should go into the computer program, but the author hopes that this advice will be of use. Please note that these instructions could also be used to find an approximate steel weight of a network arches in order to compare it to other bridge alternatives.

These instructions are to be put into TNA. Would those who take part in workshops on network arches or otherwise use these instructions, please come up with suggestions for improving the text. It is not fair that the author should do his best to give good advice and his readers should abstain from pointing out mistakes and possible improvements to him.

References:

NAKAI H. et al. [1995] "Proposition of Methods for Checking the Ultimate Strength of Arch Ribs in Steel Nielsen-Lohse Bridges", *Stahlbau* Vol. 64, No. 5, 1995, pp.129-137.

Teich S. and Wendelin S. [2001], "*Vergleichsrechnung einer Netzwerkbogenbrücke unter Einsatz des Europäischen Normenkonzeptes*", (Comparative Calculation of a Network Arch using Euronorms. In German), Graduation Thesis of the TU-Dresden, 270 pages. A revised version of this graduation thesis has been made available on the author's home page http://pchome.grm.hia.no/~pert.

Tveit, P. [1966], "The design of network arches", *The Structural Engineer*, Vol. 44, July 1966, pp. 249-259.

Tveit, P. [1980] "Network arches." 11th IABSE Congress, held in Vienna, Austria, Final Report, IABSE, ETH- Höngerberg, CH-8039, Zürich, Switserland, IBSN 3 85748 0254, pp. 817-818

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

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