EDITION 19.12.03

PRELIMINARY DESIGN OF NETWORK ARCH ROAD BRIDGES

With two examples spanning 93 and 120 metres

These instructions are written for future workshops on the design of network arches. Frequent references to the author's homepage are supposed to lead to a deeper understanding of optimal 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.

Most of the plans for optimal network arches that the author has heard about seem to have moderate spans. It might be a good idea to start with moderate spans to gain experience. Later greater savings per metre of bridge can be gained by using network arches in bridge projects where longer spans are asked for. See fig. 1.

The advice in this publication will be applied to two Norwegian bridges. One bridge is at Lonevaag (Lonevåg) on an island called Osterøy. It is situated 15 kilometres north-east of Bergen. The span will be 93 metres. The other is at Nybergsund. The span will be 120 metres. The instructions will be written in Times New Roman. The text relevant only to the Lonevaag Bridge is written in Batang. The text relevant only to the Nybergsund Bridge is written in Arial.

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 makes up only 10 % to 15 % of the total steel weight.

These instructions should help bridge designers to choose dimensions that can be put into the computer program when network arches are to be designed. They will also give some of the data on steel weights 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 the 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 Norway the room necessary for snow-clearing equipment often decides the minimum width of the footpaths. Then only one footpath is often wide enough for all the pedestrians. 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.

In the Lonevaag Bridge the lane for vehicles is 7.5 metres wide. The only footpath is 2 metres wide.

In the Nybergsund Bridge the lane for vehicles is 8.5 metres wide. The only footpath is 2.5 metres wide.

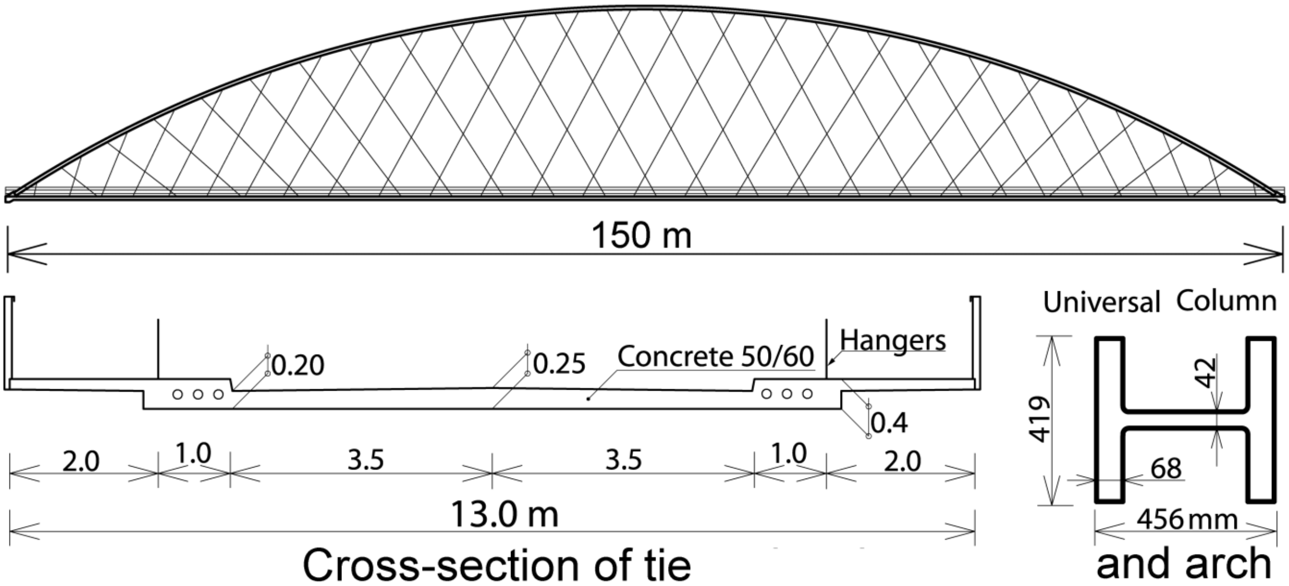

Fig. 1. Suggested network arch with a span of 150 m.

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 less than 150 m the cost of concrete, reinforcement and formwork per metre span is more or less independent of the span.

For a sequence of spans under 150 m we are probably near an optimum if the costs of the structural and prestressing steel in one span are nearly the same as the cost of 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.

For the Lonevaag Bridge the Public Roads Authority suggests a span of 93 metres.

For the Nybergsund Bridge the Public Roads Authority suggests a span of 120 metres.

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

For spans up to 170 m arches made of universal columns and/or American wide flange beams are strongly recommended by the author. These compact cross-sections can take high buckling stresses when used in network arches. Furthermore they lead to simple details. See TNA figs. 16 to 19. Because the bridges for Lonevaag and Nybergsund a footpath only on one side the two arches are different.

For the Lonevaag Bridge the universal columns in the arch are assumed to be 0.4 m and 0.38 m wide. For the Nybergsund Bridge the universal columns in the arch are assumed to be 0.42 m and 0.41 m wide.

4. Decide on the rise of the arch

For aesthetic reasons the author favours a rise of 15 % of the span. This rise of the 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]

A rise of the arch of 0.15 times the span is chosen both for the Lonevaag Bridge and the Nybergsund Bridge. The Lonevaag bridge has hilly surroundings. This could have made a 16 % rise of the arch acceptable.

5. Decide on the distance between the planes of the arches

Local rules and regulations will usually decide the distance between the inner edge of an arch and the outer edge of the nearest traffic. The rules of the Norwegian Public Roads Authority ask for 0.5 metres between the arch and the road. The widths of the arches in the wind portal are found in page 12.

The minimum distance between the planes of the arches in the Lonevaag bridge will be: 7.5+ 2x0.5+ ½(0.419+ 406)=8.91 metres. 9 m is chosen.

For the Nybergsund Bridge the minimum distance between the planes of the arches will be 8.5+0.5x2+½(0.456+419)=9.94 metres. 10 m is chosen.

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. For fig. 2 a concrete cube crushing strength of 50 MAa is assumed.

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

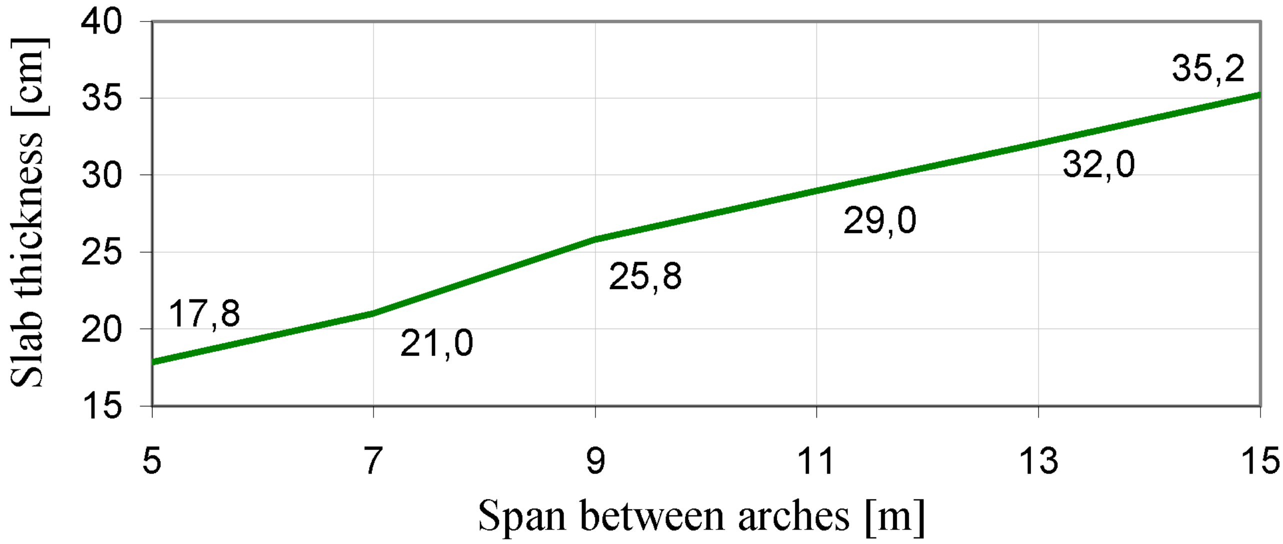

Fig. 2. Necessary thickness of slab between the arches. [Teich and Wendelin 2001 p. 109]

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 loads and ample for Norwegian loads. We will use a parabolic variation of the thickness of the slab. This is to make sure that the maximum transverse reinforcement is decided by the maximum bending moment in the middle of the slab. The deflection of the slab can be counteracted by a suitable camber.

Since the forces in the slab will be controlled by the computer calculation it is not necessary to make the preliminary calculation of the slab very carefully. If the thickness of the slab is reduced later in the design process, one must make sure that the reduced weight of the tie does not lead to too big bending moments due to the relaxation of hangers. See TNA pp. 69-70.

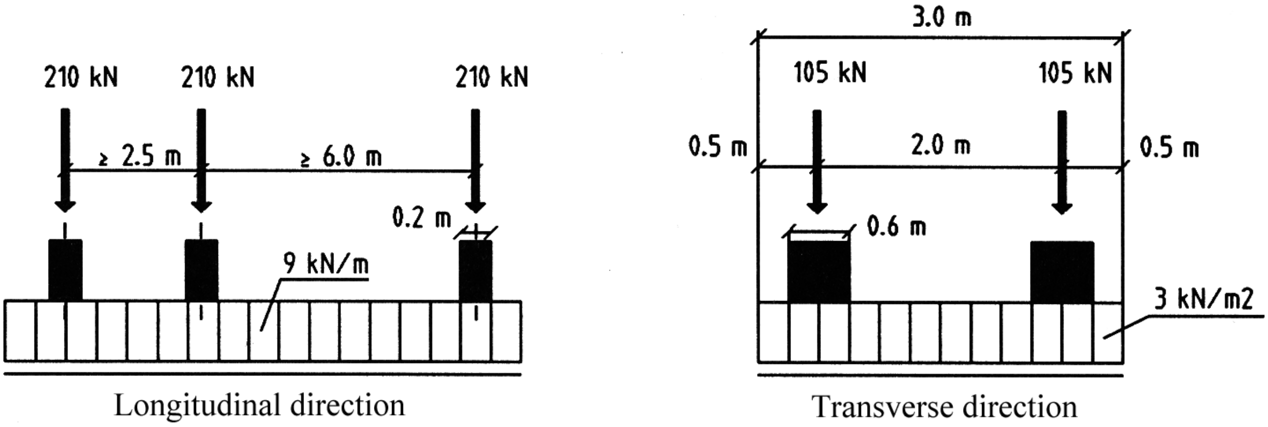

Fig. 3. Norwegian traffic loads between the arches.

The Norwegian traffic loads are considerably smaller than the loads used in the European Union. They are shown in fig 3. On the pavement 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 arches, the load outside the hangers is 2 kN/m^2.

8. Decide on the shape of the lower chord

The edge beam should normally be 0.4 to 0.5 m deep. It must have room for the longitudinal prestressing cables and must take the transversal forces from the hangers. See fig. 1. The thickness of the footpaths could be varying from 0.15 m at the outer edge to 0.18 to 0.24 m at the root. This should be on the safe side since the thickness of the footpath of the bridge at Steinkjer, TNA fig. 6 is 0.12 m. It is in good shape after 38 years.

For the Lonevåg bridge the depth of the tie is chosen to be 0.4 metres under the bigger arch and 0.45 metres under the smaller arch. The bigger arch is nearest to the footpath. The depth of the universal column is expected to be 0.393 m in the bigger arch and 0.381 m in the smaller arch. See page 12. The suggested shape of the lower chord in the Lonevaag Bridge is shown in fig 4.

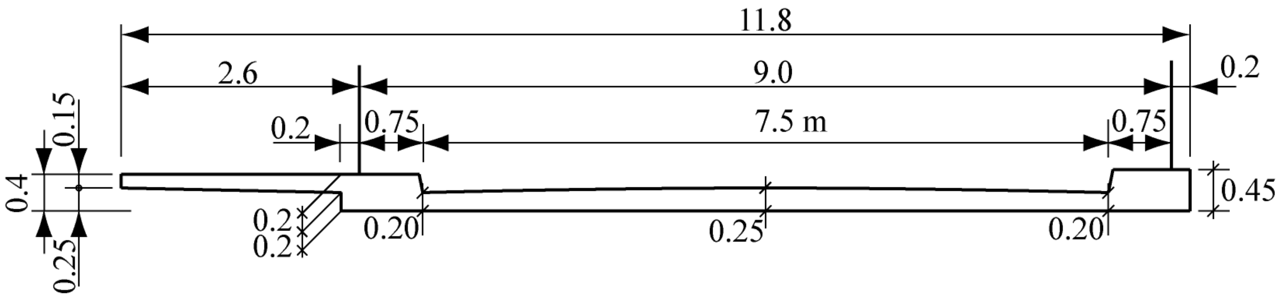

Fig. 4. Suggested shape of the lower chord of the Lonevaag Bridge.

The span divided by the combined depth of the tie and arch is 93/(0.45+ 0.381)=112 for the smaller arch. The same value for the Bolstadstraumen Bridge is 91. Bolstadstraumen Bridge has been the world's most slender arch bridge for the last 38 years.

For the Nybergsund bridge depths of the edge beams in the tie are as in the Lonevaag bridge. The depth of the universal column in the bigger arch is expected to be 0.4 m. Then the same slenderness of the bridge is 116/(0.4 +0.406)=144. The suggested shape of the lower chord of the Nybergsund bridge is shown in fig. 5, see next page.

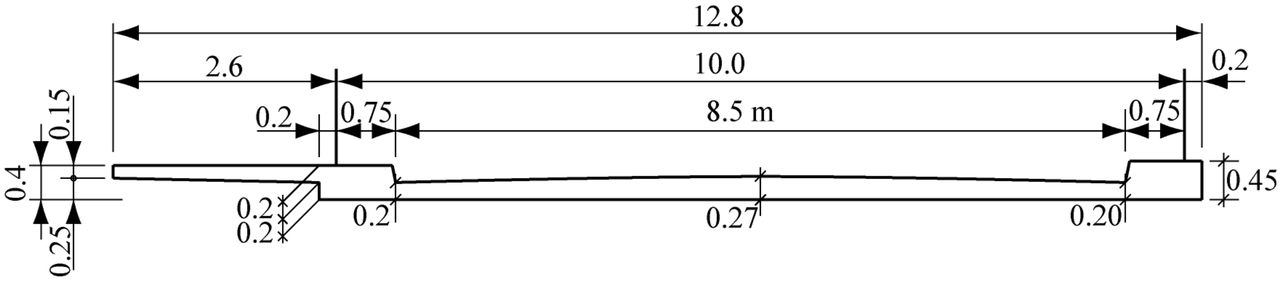

Fig. 5. Suggested shape of the lower chord of the Nybergsund Bridge.

9. Calculate the permanent load

The layer of asphalt is assumed to be 0.1 m thick. The weight of the asphalt is assumed to be 23 kN/m^3. The asphalt layer is supposed to be 0.05 m thick when a light tie could lead to relaxation of many hangers. The relaxation of many hangers can lead to big bending moments in the chords.

For the Lonevaag Bridge the weight of concrete, asphalt and steel is 52.6 kN/m of bridge under the bigger arch and 40.1 kN/m under the smaller arch. 1 kN/m per arch of guardrails and railings are included. Half the asphalt weighs 4.3 kN/m.

For the Nybergsund Bridge the weight of concrete, asphalt and steel is 52.7 kN/m under the bigger arch and 40.0 kN/m under the smaller arch. 1 kN/m per arch of guardrails and railings are included. Half the asphalt weighs 5 kN/m

Steel with a yield stress of 430 MPa (S 460 ML) has been assumed for the dimensions used in both bridges. The steel weight per m^2 can be taken from fig. 6. It is fig. 9 in TNA. The dot in the diagram indicates the steel weight of a bridge designed for the Åkvik Sound in Norway. See TNA pp. 9 to 11. Cold bending of the profiles in the arch will give a small reduction of the notch impact values. 60 % of the steel weight is expected to be structural steel.

The steel weight on the bigger arch is about 10% bigger than the steel weight on the smaller arch. As the steel weight is only about 6 % to 7 % of the weight of the concrete and the asphalt, very precise calculations of the steel weight are not necessary

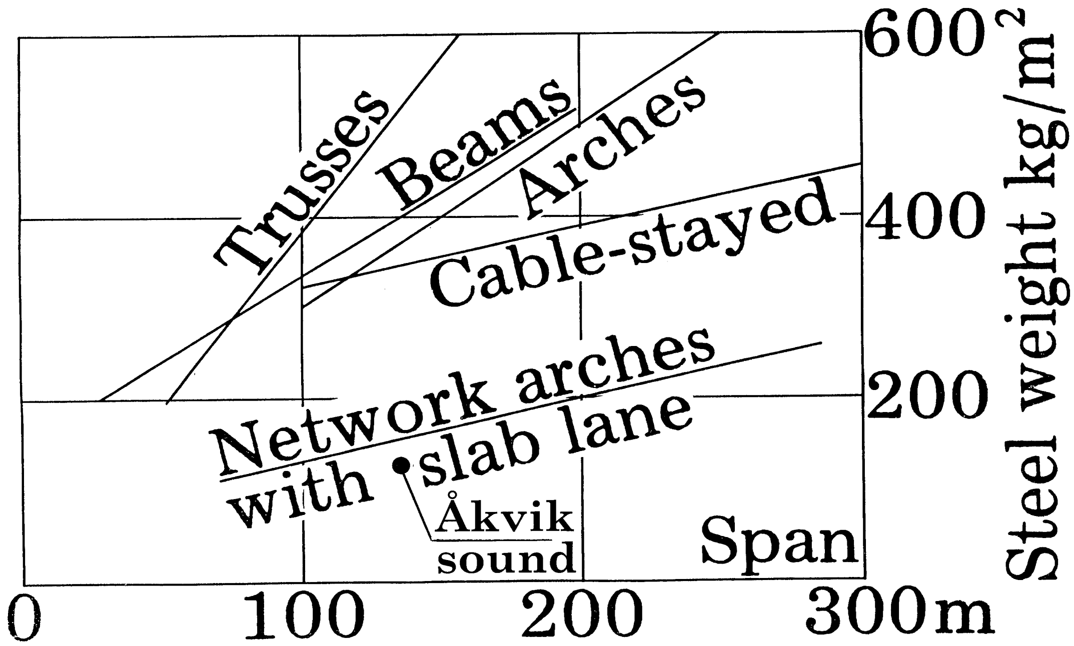

Fig. 6. Amount of steel in various types of highway bridges.

For the Lonevaag Bridge fig. 6 gives a steel weight of around 7 kN/m of bridge under the bigger arch and around 5 kN/m of bridge under the smaller arch. For the Nybergsund Bridge fig. 6 gives a steel weight of around 10 kN/m of bridge under the bigger arch and around 7 kN/m of bridge under the smaller arch.

10. Find the live load on the bridge in the serviceability limit state

Local loads and codes have to be used. How the even traffic loads are placed can be seen from figs. 7, 8, 9 and 10.

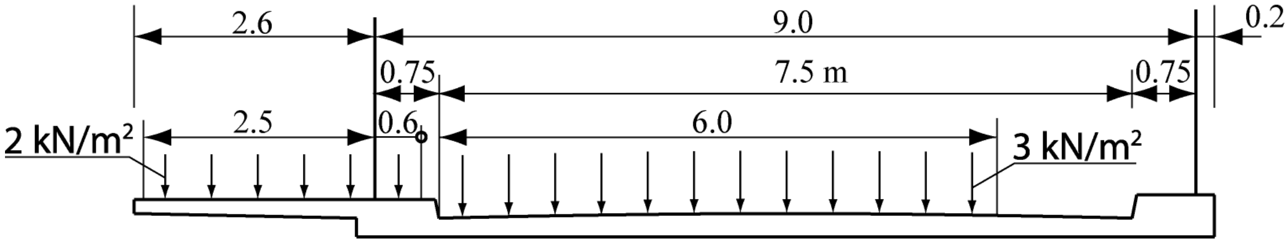

Fig. 7. Live loads on the bigger arch of the Lonevaag Bridge.

On the bigger arch the evenly distributed live load is 17.4 kN/m. The concentrated load is 368 kN.

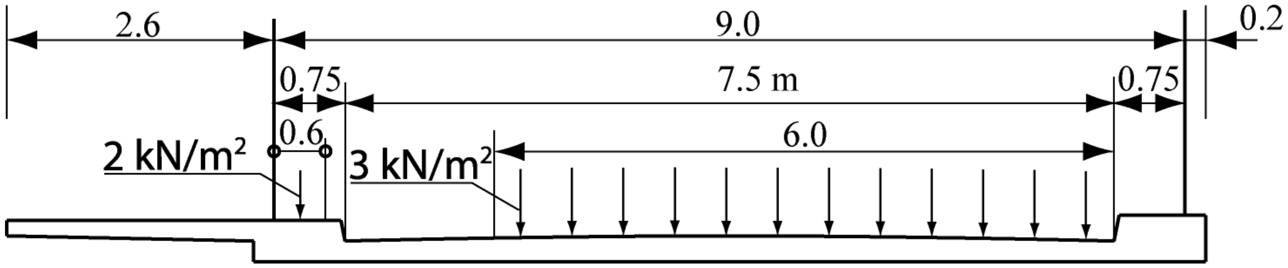

Fig. 8. Live loads on the smaller arch of the Lonevaag Bridge.

On the smaller arch the evenly distributed live load is 10.5 kN/m. The concentrated load is 368kN.

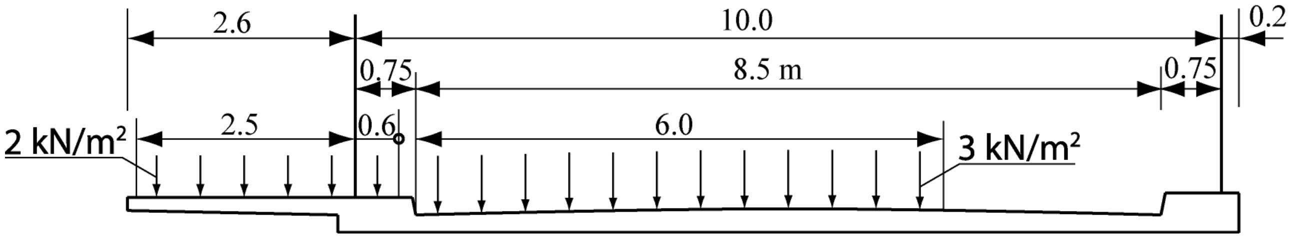

Fig. 9. Live loads on the bigger arch of the Nybergsund Bridge.

On the bigger arch the evenly distributed live load is 18.1 kN/m. The concentrated load is 394 kN.

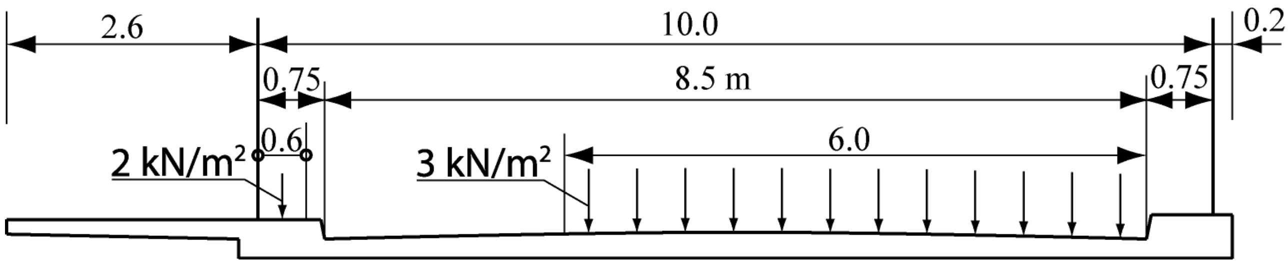

Fig. 10. Live loads on the smaller arch of the Nybergsund Bridge.

On the smaller arch the evenly distributed live load is 11.3 kN/m. The concentrated load is 394 kN.

11. Make a tentative decision on the system lines of the span

The hangers should be placed equidistantly along the arch, but the member nearest to the end of the arch should be 30 % to 50 % longer than the other members in the arch. Page 26 in TNA gives some advice on arrangement of the hangers. A distance between 2.6 m and 4 m seems a reasonable choice.

The distance between the nodes in the arch depends partly on the span of the network arch. Smaller spans should have shorter distances between the nodes in order to reduce bending due to the curvature between the nodes in the arch. For the two bridges designed for Vienna the nodes are placed equidistantly in the middle half of the tie. In the Steinkjer Bridge and the Bolstadstraumen Bridge there is a constant difference in the slope of two hangers next to each other. The author can not say what is best.

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 chosen of distance between the nodes could depend on the deflection that can be tolerated when casting the slab between the transversal beams in the temporary lower chord. This deflection, and the strength, decides the dimensions of the timber beams supporting the plywood in the temporary lower chord. For temporary lower chords, see TNA pages 12 and 52 to 55 and index page 101.

11.1 Hanger forces in the Lonevaag bridge

For the Lonevaag Bridge we will use the system lines for the Steinkjer Bridge, TNA p. 59, multiplied by a factor 93/80=1.16. In fig. 11 on the next page we check the hangers' resistance to relaxation in the bigger arch because here we find the bigger ratio of live load to permanent load. Thus the resistance to relaxation is smallest for this arch.

The influence lines and the load cases are shown in fig. 11. Hanger 20'-15' is examined. It might not be this hanger that is subject to the biggest tension. Nor may it be it the hanger that will relax for the smallest load, but it gives a good indication of the biggest hanger force and the hangers' tendency to relax

By means of the influence lines in fig. 11 one can find the maximum tension in some hangers. Influence lines cannot normally 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 line gives hanger forces on the safe side for load cases that make some hangers relax. There is more on this point in TNA page 33 and fig. 45.

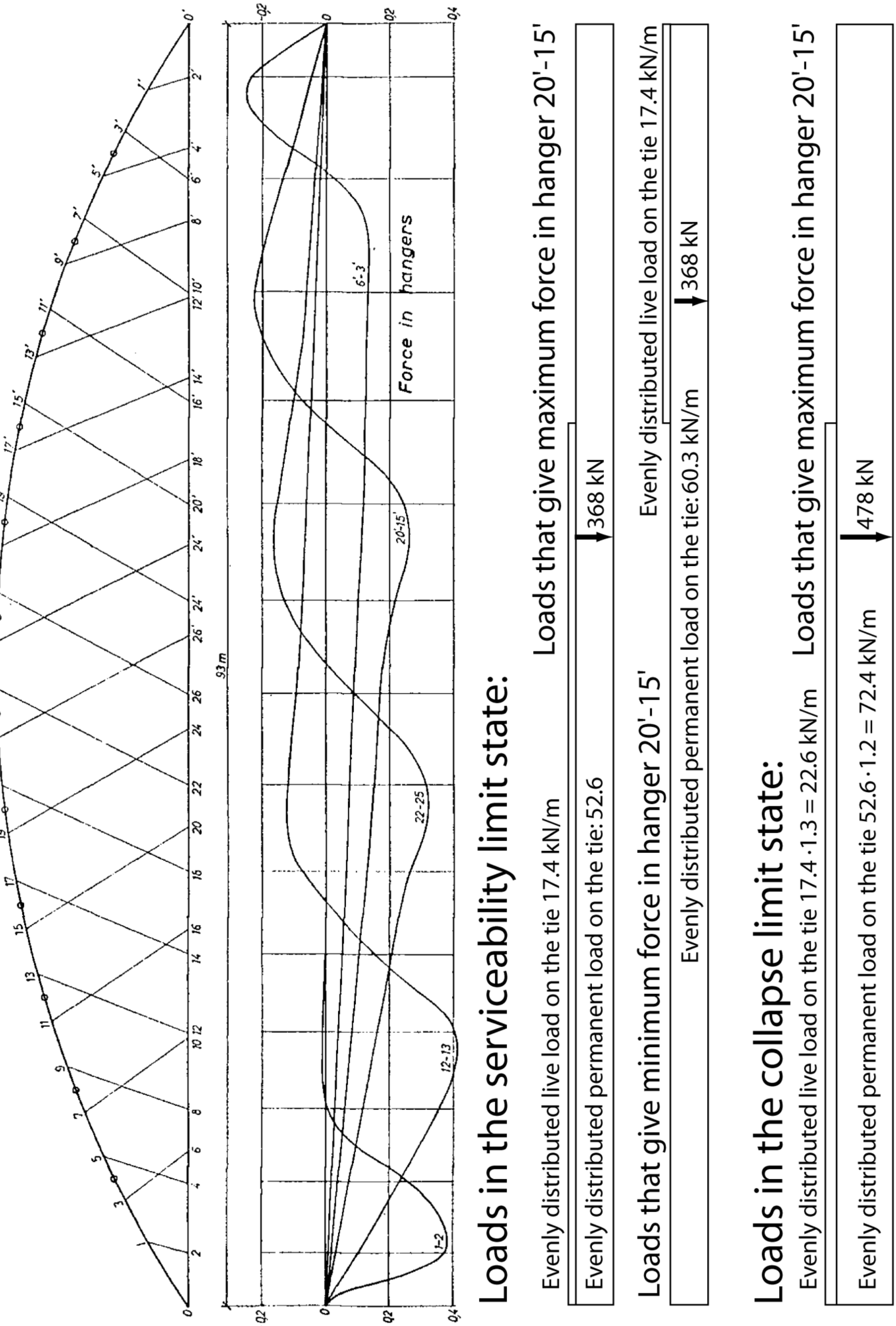

Fig. 11. Loads on the influence line for the axial force in member 20'-15' in the Lonevaag Bridge.

Calculation of forces in hanger 20'-15'

Positive inf. area: 7.40 m.   Negative inf. area: 3.67 m.   Sum of inf. areas: 3.73 m.

Serviceability limit state:

Maximum hanger force: $3.73 \cdot 52.6 + 7.4 \cdot 17.4 + 368 \cdot 0.23 = 410$ kN/m

Minimum hanger force: $3.73 \cdot 52.6 - 3.67 \cdot 17.4 - 368 \cdot 0.23 = 47.7$ kN/m
There is no danger of relaxation of hangers regardless of wear of asphalt.

Collapse limit state:

Maximum hanger force: $3.73 \cdot 72.4 + 7.4 \cdot 22.6 + 552 \cdot 0.23 = 564$ kN

Minimum hanger force when half the asphalt is worn away, $\gamma_g = 1.0$ and $\gamma_p = 1.5$:
$3.73(52.6-4.3) - 3.67 \cdot 22.6 - 478 \cdot 0.23 = -13$ kN

The hanger will relax in the collapse limit state. This leads to a smaller maximum hanger force. The deviation from the optimal in the present design is insignificant. See TNA pp. 69-70. From the original calculation of the hangers in the Steinkjer bridge the author has found that the hanger 20'-15' is not the hanger that has the least resistance to relaxation, but it can be concluded that all the hangers have so much resistance to relaxation that the relaxation of hangers will not lead to big bending stresses in the chords.

To take 564 kN which is the maximum hanger force under the bigger arch a cross-section of $564 \cdot 10^3 1.1/430 = 1443$ mm^2 is needed. A hanger diameter of 45 mm is chosen. Area=1590 mm^2.

For the smaller arch a cross-section of similar calculation of the hanger can be carried out. In order to save time and paper a hanger diameter of 41mm is chosen. A=1320 mm^2.

If the hanger length has to be adjusted, steel wires are best. See TNA fig. 56. Otherwise round steel rods are recommended.

11.2 System lines for the Nybergsund Bridge.

The longest span in the lower chord of the Lonevaag Bridge in fig. 11 is 5.52 m. This is so much that the geometry of the Steinkjer Bridge cannot be used for the Nybergsund Bridge. It would lead to too long members in the temporary lower chord and it would make the design of the temporary lower chord difficult.

The author suggests the system lines in fig.12 for the Nybergsund Bridge. These skeleton lines have a reasonable resistance to relaxation. See fig 13. The slopes of the hangers are influences by the slope of the hangers in the Steinkjer Bridge. The difference in slope between two adjoining hangers is 1.1°. The randomly chosen system lines in fig. 12 might have to be changed when we have the first results of the computation of the hanger forces.

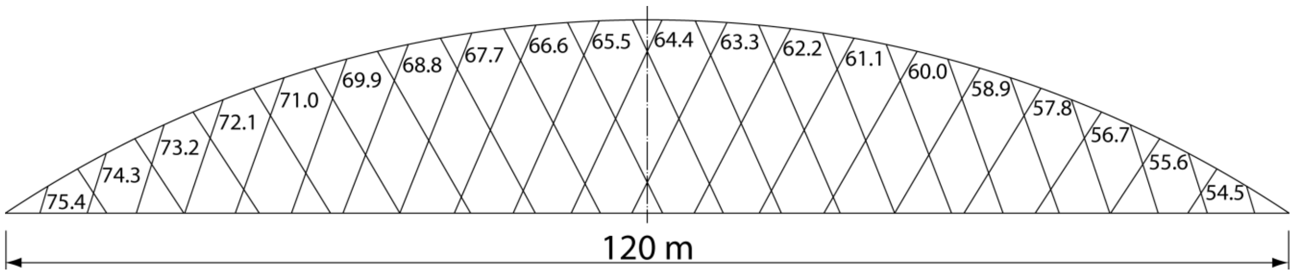

Fig.12. System lines for the Nybergsund Bridge. Span 120 m

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.

Nybergsund is as far away from the sea as you can get in Norway. The winters are long and cold. Thus steel skeleton consisting of the steel in the arch and the hangers plus a temporary lower chord can be used for the erection. The steel skeleton is erected on the ice and lifted on to the pillars. See TNA p. 30a.

The steel skeleton will weigh at least 190 tons. The weight will depend on how much of the wood that is put in place before the steel skeleton is lifted on to the pillars. The concrete tie is cast in the spring. TNA p. 31a. and TNA fig. 90 and page 74. The steel skeleton of the Lonevaag bridge will weigh about 110 tons. In be erected on the approach to the bridge and be lifted in place by the craneship "Uglen". It can lift 2 x 150 metric tons to 60 m above sea level.

Since we do not have any influence lines that can be used to find the axial forces in the hangers in the Nybergsund Bridge, it might be best to make guess of the necessary hanger dimension. This would probably make later alterations of the hanger dimension more likely. Guess a hanger diameter of 50 mm (A=1960 mm^2) under the bigger arch and a hanger diameter of 45 mm (A= 1590 mm^2) under the smaller arch.

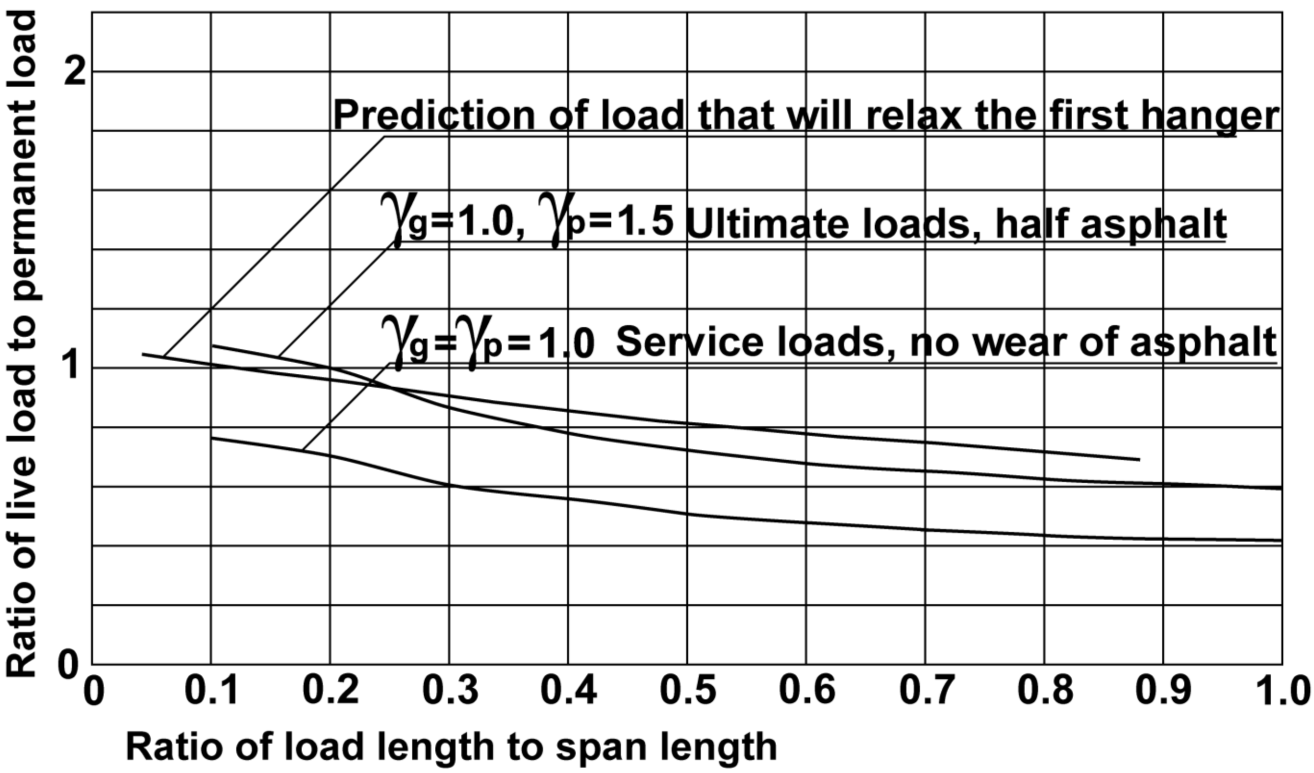

Fig. 13. Prediction of relaxation of the first hanger in Nybergsund Bridge (fig 12) according to fig. 38 in TNA p.29 compared to the load intensity on the span.

12. Arch

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 except at the wind portal. This arrangement gives the smallest bending moments 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 be sloping away from the end of the span as in all system lines in this paper.

As can be seen from the influence lines in fig. 14, 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.

Fig. 14 is taken from TNA fig. 63. It can be used to find the axial force in the top of the arch. The steeper hangers of the Nybergsund Bridge lead to bigger axial forces in the arch. This small increase is of no consequence. Formulas for calculating the axial forces in the chords can be found in [Tveit 1967 p. 251].

12.1 Find the maximum axial force in the arch in the collapse limit state

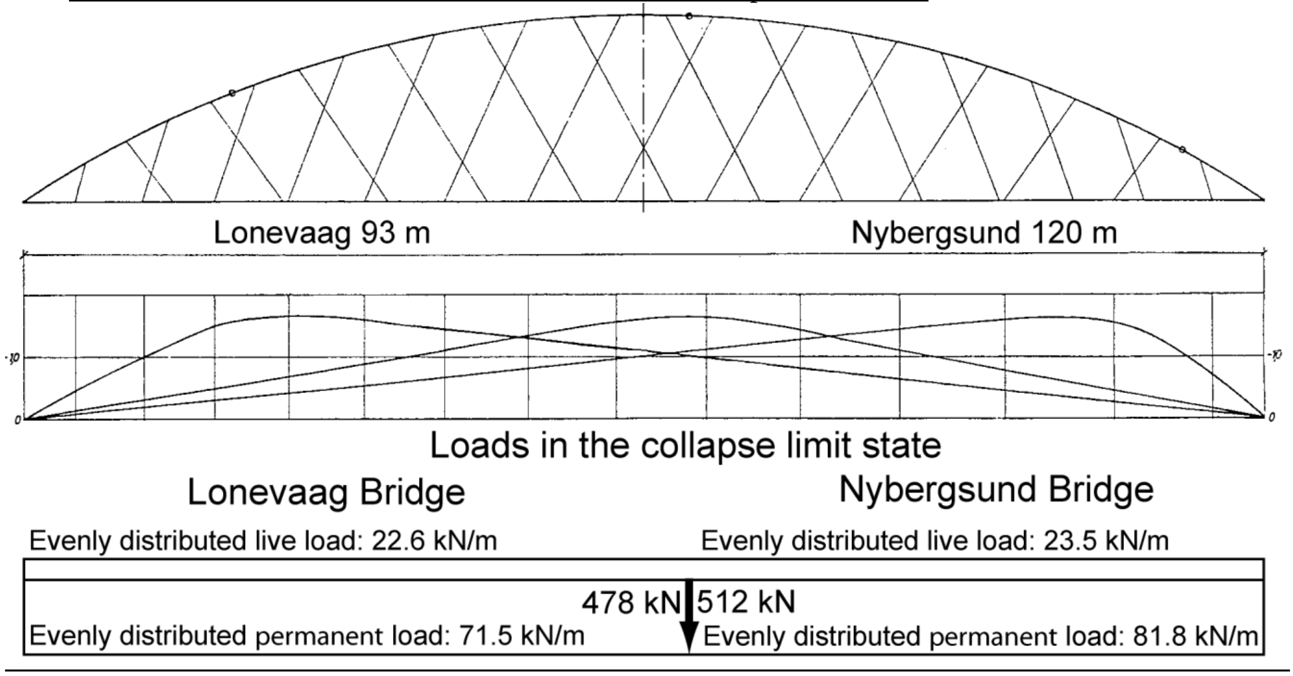

Fig 14. Influence lines and loads on the arch of the Lonevaag and the Nybergsund Bridge.

Lonevaag Bridge: Area under the influence line at the top of the arch is 80.4 m.

Nybergsund Bridge: Area under the influence line at the top of the arch is 103.8 m.

Calculation of the bigger arches

Lonevaag Bridge,maximum axial force at the top of the arch:
$80.4(71.5+22.6)+478\cdot1.6=8331\text{kN}$

Nybergsund Bridge, maximum axial force at the top of the arch:
$103.8(81.8+23.5)+512\cdot1.6=11749$ kN

For the 93 m span: Area at the top of the arch: $8331 \cdot 10^3 \cdot 1.4/340 = 34.3 \cdot 10^3$ mm^2
Choose British Universal Column UC 356x406x287   A=$36.6 \cdot 10^3$ mm^2.

For the 120 m span: Area at the top of the arch: $10503 \cdot 10^3 \cdot 1.4/340 = 48.4\ 10^3$ mm^2
Choose British Universal Column UC 357x406x393   A=$50.1 \cdot 10^3$ mm^2.

The factor 1.4 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.

In the tie the hangers could be placed equidistantly in the middle half of the span as in fig. 4 or more as in the Steinkjer and the Bolstadstraumen Bridge. See TNA fig. 63 and 64. Since the tie is shorter than the arch, the average distance between the nodes in the tie becomes slightly smaller than the distance between the nodes along the arch. See for instance TNA fig. 8. Different loads and codes give different optimal hanger arrangements.

12.1.1 Calculation of the smallest arches

Collapse limit state:

Lonevaag Bridge, maximum axial force at the top of the arch:
$80.4(54.1+13.7)+478 \cdot 1.6=6216$ kN

Nybergsund Bridge, maximum axial force at the top of the arch:
$103.8(63.4+14.7)+512 \cdot 1.6=8926$ kN

For the 135m span: Area at the top of the arch: $6216 \cdot 10^3 \cdot 1.4/340=25.6 \cdot 10^3$ mm^2.
Choose British Universal Column UC 356x406x235 A=$29.9 \cdot 10^3$ mm^2.

For the 160 m span: Area at the top of the arch: $8926 \cdot 10^3 \cdot 1.4/340=36.8\ 10^3$ mm^2.
Try British Universal Column UC 357x406x340
A=$43.3 \cdot 10^3$ mm^2.

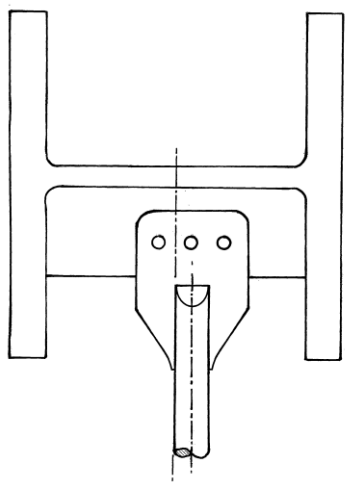

Fig. 15. Upper end of hanger.

13. Fatigue

In the road bridges at Lonevaag and Nybergsund the expected volume of traffic is small. This makes it less likely that fatigue will influence design. The hangers are the part of the bridge that is more likely to be influenced by fatigue. If the type of upper and lower end of hangers that is shown in fig. 15 and 16 is used, fatigue is not expected to be a problem. Thus we do not have to calculate the axial forces in the arch in the serviceability limit state.

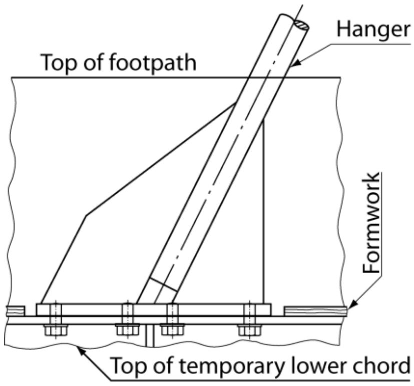

Fig. 16. Lower end of hanger.

14. Lower chord

The tensile force in the middle of the tie is between 2 % and 5 % smaller than the compressive force in the arch. The formulas for this are given in [Tveit 66]. Choose prestressing cables that can take this force. The cables go between the ends of the arch. Due to the friction it is the force in the middle of the span that decides how big the prestressing cables must be. Friction, creep and relaxation can be calculated.

There will be no or very little longitudinal tension in the concrete caused by the day-to-day traffic. Thus the transverse cracks in the tie will be small or non-existent in the serviceability limit state. A minimum reinforcement in the longitudinal direction will probably be able to take most of the longitudinal bending moment in the tie.

15. 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 m 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 in the wind portal has a smaller curvature, but the savings in material might not pay for the extra design work due to two instead of one curvatures of the arch. Two curvatures in the arches would also make it easier to attach a side-span to the network arch. In the Steinkjer Bridge the last two members 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 in the arch and the end of the bridge could be a little longer than the other members. This also would give more even hanger forces. If H-profiles 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 H profile. The cavity under the steel plate should be filled with concrete.

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Knowledgeable readers will understand that these hints are just a rough guide to 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 arch 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.

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Grimstad, March 10th 2003

Per Tveit