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NETWORK ARCHES FOR RAILWAY BRIDGES

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Abstract. *Network arches have inclined hangers that cross each other at least twice. It seems to happen more often for railway bridges than for road bridges that structural elements above the bridge deck are acceptable, which justifies investigations of the applicability of network arches.*
The tie can be a longitudinally prestressed concrete slab. This gives less noise, and the additional self-weight favours the structural behaviour. Alternatives with steel and composite bridge decks are discussed. For double track railway bridges spanning up to 100 metres the arches can be rolled H-sections. For larger spans welded box sections are applicable.
For the design of network arches the hanger arrangement is important. Small bending moments in the arches and small hanger forces are obtained when the upper hanger nodes are placed equidistantly and all hangers cross the arch with the same angle. The cross angle's size depends on several parameters. Hints for a good choice are given. The maximum hanger forces vary little, thus all hangers have the same cross-section.
To ensure passenger comfort and the stability and continuity of the track, deformations of railway bridges are constricted. A network arch is a stiff structure with small deflections and therefore suitable to comply with such demands even for high speed railway traffic.
A network arch railway bridge with a concrete tie usually saves more than half the steel required for tied arches with vertical hangers and concrete ties.

1 INTRODUCTION

It has been shown in STEIMANN[iii] as well as in BRUNN & SCHANACK[ii] – both applying the European Standards – that network arches are suitable for railway bridges. In the following the general design and the most important details of a network arch railway bridge shall be described by means of the example of a 100 m spanning double track railway bridge, Figure 1.

Fig. 1. Visualisation of the double track 100 m spanning example railway bridge

2 BRIDGE DESIGN

Following the design advice given in this article leads to savings of about 60 % of structural steel compared with conventional tied arch bridges with vertical hangers.

2.1 The arches

The arches of railway bridges up to double track loading and with spans up to about 100 m can consist of rolled H-profiles connected by butt-welds executed in situ. In the considered example bridge with an arch rise of 17 % of the span American wide flange profiles W360x410x634 are sufficient for the mid-segments. Slightly larger profiles (W360x410x900) form the shafts of the wind portal frame. The length and therefore the bending moments of the shafts can be decreased by giving the end-segments a smaller radius. ARCELOR[i] provides rolled profiles of steel S 460 ML with a constant curvature. The use of such high-strength steel is favoured because of the predominant normal forces acting in the arch which also contribute to very slender arch profiles.

The shape of the railway clearance gauge permits the formation of the portal frame cross bar according to a truss with diagonal struts below to further decrease the length of the portal frame shafts.

The arches are supported by the closely spaced hangers giving high in-plane buckling resistance.

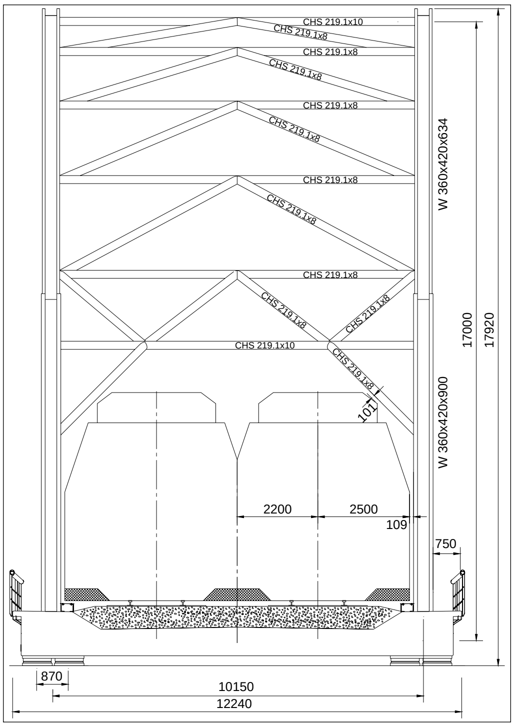

Fig. 2. Front view of the example 100 m span railway bridge, unit: [mm]

The need of sufficient out-of-plane buckling resistance makes the application of a wind truss essential. Since the supports given by the wind truss are spaced more widely than the hanger connections, the weak axis of the H-sections has to be horizontal. In cases where such sections are insufficient, welded box sections are a feasible solution, STEIMANN[iii].

The struts of the wind truss are recommended to be made of slender hollow sections in order to meet structural and aesthetic demands. The arch rise should be about 15 % of the span; larger arch rises decrease internal forces but respecting aesthetics it should not exceed 17 % of the span, TVEIT[v].

2.2 The hangers and hanger connections

A 100 m network arch should be equipped with about 48 hangers per arch plane, which has economical and structural reasons. The extra costs caused by additional hangers and their connections have to be balanced against the material costs that can be saved due to smaller internal forces in arches and tie.

Each set of hangers – with the same sense of direction – has an offset to the arch plane of half the hanger's diameter, which allows the hangers to pass each other without deflections. This eccentricity causes torsional moments in the arch profiles, which are partially taken by the wind bracing. The direction of the eccentricity changes from each hanger connection to the next, so that the torsional moments counterbalance each other as long as all hangers are in tension. Due to the inclined hangers in network arches some hangers might take compression which makes them relax. In Section 3 a hanger arrangement will be introduced which ideally avoids hanger relaxation.

The hangers should consist of smooth bars made of high strength steel such as S 460 ML with a circular cross-section. A diameter of 60 mm is sufficient for a 100 m double track railway bridge using 48 hangers per arch plane. Maximum hanger forces in Ultimate Limit State do not exceed 1062 kN.

At their intersections the hangers are protected by a sheathing of slit open plastic tubes and tied together with elastic rubber bands. This couples the deflections out of the arch plane, increases damping and prevents the hangers from banging against each other.

The hanger connections along the arch constitute a detail with high requirements, especially in terms of fatigue. If the connection plate is aligned to the slender arch transversally, as recommended for arches made of H-sections or rectangular box sections, the space for the connection is very limited. Therefore, detail solutions common for tied arches with vertical hangers and larger arch profiles providing more space might not be applicable for network arches. For more details see TEICH[iv].

2.3 The bridge deck

The stiff longitudinal structural behaviour of network arches with its closely spaced lower hanger nodes leads to the fact that the decisive bending moments in the tie are to be found in transverse direction. Thus, the distance between the arches and therefore the transverse span of the bridge deck should be minimised. For double track railway bridges this distance can be as small as 10.15 metres if the footpaths lie on cantilevers outside of the arches, Figure 2.

2.3.1 Concrete deck

The tie of a network arch can be made very slender when making it of concrete. This is important, when bridges cross rivers, channels, motorways or other sites where a small deck depth is demanded. The deck consists of longitudinal edge beams below the arches containing longitudinal prestressing tendons, and the slab spanning between the edge beams. In BRUNN & SCHANACK[iii] it has been shown that bridge slabs with and without transverse prestressing are feasible solutions for railway bridges. The decisive factors are costs as well as the stiffness and the slenderness of the bridge deck.

A very slim design can only be achieved by transverse prestressing. For the example bridge with a 43-centimetre-thick deck of C50/60 concrete spanning 10.15 metres between the hangers, 370 thread bars DYWIDAG type 36D placed at a distance of 27 cm along the tie are sufficient, Figure 3. As in this case, compression reinforcement might be necessary.

The prestressing in the longitudinal direction (6 tendons DYWIDAG type 6827 in each edge beam) mainly counteracts the horizontal thrust of the arches and is therefore a function of the span, arch rise and all vertical loads acting on the tie. Increasing the depth and therefore the self-weight of the bridge deck consequently increases the required longitudinal prestressing and the cross sections of all other primary structural members of the bridge. To counteract the resulting higher costs, the transverse prestressing tendons can be omitted, as they are not necessarily required in a thicker slab. In the example of the 100 m double track railway bridge it was found that a non-prestressed deck can have a depth of only 47 cm without compression reinforcement. The economical advantage to the prestressed version is

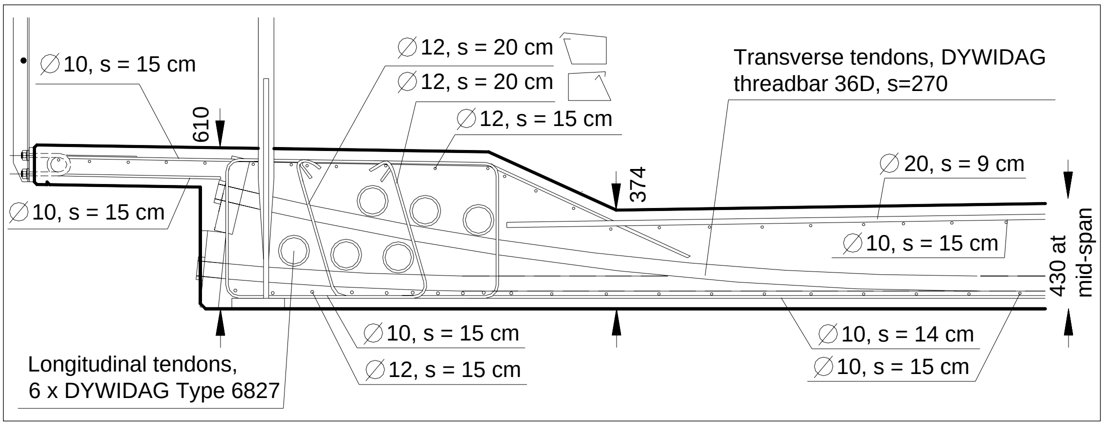

Fig. 3. Cross section of concrete slab with transverse prestressing

with 3 % negligible and might be – where decisive and necessary – outweighed by the great advantage of the higher stiffness when using prestress. Pay heed that the codes specify high demands on maximum deformations of railway bridges.

At both ends of the bridge deck the edge beams and the slab are widened forming the end cross girders. Their task is to form stiff beams between the bearings and to complete the wind portal frames. Hence, the end cross girders distribute eccentric vertical forces, bending moments about the longitudinal axis of the bridge, reduce deflections, and support the edges of the plate-like slab. An additional function of the widened edge beams is to provide space for anchoring the longitudinal tendons.

Ideally, two pot bearings per arch root point should be used, so that a large part of the arch bending moments in transverse direction is directly transferred into the abutments.

Drainage is an important point when designing a network arch bridge. A slim bridge deck might make it difficult to accommodate longitudinal drainage pipes with the necessary incline. This must already be incorporated when designing the bridge deck. In most cases enough space must be provided for spouts going through the edge beams. If the longitudinal incline is not large enough for the application of pipes, open canals which can be cleaned regularly could be a solution. Another alternative for increasing the incline is to apply a camber to the bridge deck in the longitudinal direction. Such a camber might be employed anyway, because a horizontal lower surface seems to sag, to the human eye.

A feasible erection method is, besides using scaffolding, alternatively the application of a light temporary lower steel chord as described in TVEIT[v]. The bridge, erected on a nearby construction site, is moved into its final position before the concrete deck is cast. Once the bridge is completed, the temporary lower chord, which also carries the formwork, is removed.

2.3.2 Steel deck

The design of the deck in structural steel also represents a feasible solution for network arches. As shown in STEIMANN[iii] advantages of this deck version are a very short erection time and the reduction of the total bridge weight to about the half that of the network arch with concrete deck.

For double track railway bridges the deck plate should be stiffened by longitudinal ribs distributing loads to cross girders arranged every 2.5 metres. These cross girders span between the stiffening girders that lie in the arch planes. Together, the stiffening girders, the deck plate and the longitudinal ribs form the tension chord of the arch bridge, Figure 4.

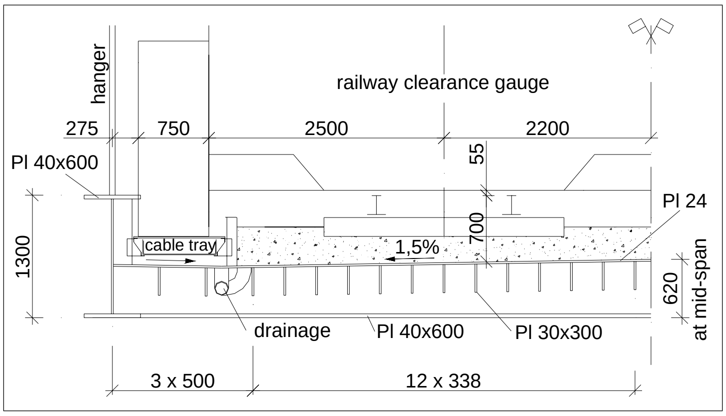

Fig. 4. Cross section of a double track orthotropic steel deck, unit: [mm]

The deck depth of such orthotropic plate is defined by the height of the cross girders. Limiting the plate thicknesses to a maximum of 40 mm – in order to use the maximum yield and tensile strength – leads to a cross girder height at mid span of 620 mm for a transverse span of 11 metres.

For single track railway bridges it is recommended to omit the longitudinal ribs and support the deck plate exclusively by cross girders arranged every 0.7 metres. This results on the one hand in a reduction of welds subjected to fatigue strains and on the other hand in a more even load distribution to the stiffening girder with therefore smaller bending moments, Figure 5.

Due to the direct traffic load impact the welded steel deck is subjected to high fatigue strains, which is, however, unproblematic when applying suitable standard detail solutions. Since fatigue loading and deflection limitations of the deck plate are decisive, longitudinal ribs and cross girders should be made of S 235. The predominant normal forces in the stiffening girders favour the usage of S 460 ML as applied for hangers and arch.

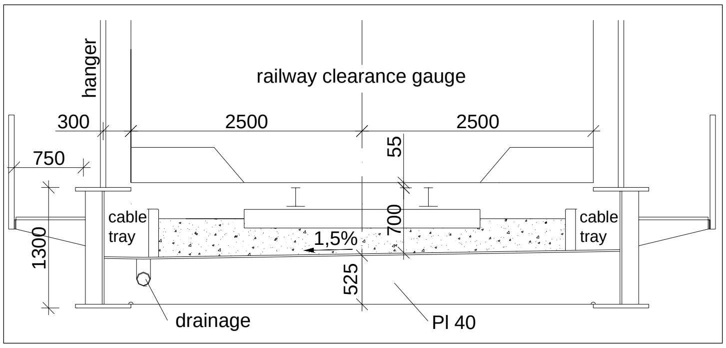

Fig. 5. Cross section of a single track steel deck, unit: [mm]

To utilise the structural behaviour of the network arch, the hangers should be connected to the tie independently from the cross girder spacings. Instead, the criteria presented in Section 3 should be obeyed. Even though this implies bending moments in the stiffening girder because of discrete traffic load distribution, the bending is small and it is still possible to limit the height of the stiffening girders by the rail's top edge.

2.3.3 Composite deck

In cases where minimisation of the deck depth is not required a composite deck is applicable, which can be fabricated economically.

The deck structure shown in Figure 6 consists of 2.5-metre-spaced S355-steel cross girders that act in composite action with the concrete slab of C35/45 by means of headed shear studs. The longitudinal reinforcement ∅20 mm / 10 cm and the stiffening girders in the arch planes take the tensile forces acting in the lower chord of the arch bridge.

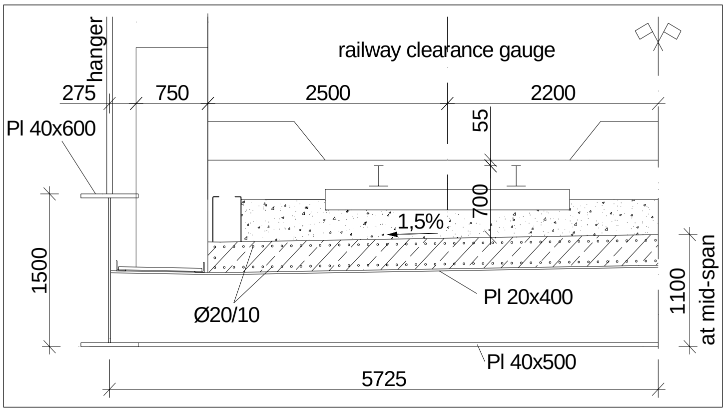

Fig. 6. Cross section of a double track composite deck, unit: [mm]

In order to distribute the tensile forces directly from the arch root points to the entire concrete slab, stiff end cross girders with a box section und load distribution plates between the upper flanges of the cross girders may be applied.

2.3.4 Best suitable solution

The deck designs, which have been introduced, fulfil all demands of Ultimate Limit State as well as Serviceability Limit State according to the Eurocode. However, the decision on one of these solutions has a sizeable influence on the erection method, the steel weight and therefore the erection costs of the bridge superstructure.

Whereas a 100-metre spanning double track network arch bridge with a concrete deck has a total weight of 1640 tonnes and requires 376 tonnes of structural steel, an analogue network arch with steel deck only weighs about 940 tonnes.

In principle the application of a concrete deck seems to be reasonable, since it leads to immense savings of structural steel, a smaller deck depth and noise reduction. Furthermore the additional self-weight favours the structural behaviour of the network arch and less exposed steel surface, which requires corrosion protection, saves maintenance costs.

Nevertheless the application of an orthotropic steel plate may be a more economic solution, because the considerably reduced total weight benefits the erection of the superstructure. For example a bridge with a steel deck can be mounted completely off-site and then easily be lifted or moved to the final position. In addition this shortens the operating breaks, which is often decisive for the railway company.

A certain combination of the advantages of the concrete and the steel deck version can be achieved by the application of a composite bridge deck. Analogously to the steel deck the light steel structure of the composite deck network arch can be lifted and moved as a whole into the final position before the concrete slab is cast. And similar to the concrete deck the solid slab prestresses the inclined hangers effectively by the high self-weight and reduces noise from passing trains. In contrast, the large required structural height of the composite girders is disadvantageous.

As a conclusion it can be said that every single project with its specific requirements and local conditions will decide which solution is the most economic.

3 OPTIMISATION OF THE HANGER ARRANGEMENT

The arrangement of the hangers has sizeable influence on the structural behaviour of network arches. It decides the forces and, as is especially important in railway bridges, force variations within the network arch depending on many parameters like, for example, span, arch rise, number of hangers, loading or arch curvature. Small changes in the geometry may lead to a significant increase or decrease of the maximum internal forces. General statements about an optimal hanger arrangement, which gives reasonable results in a great variety of different bridges, should be made.

3.1 What is an optimal hanger arrangement?

Optimal structures are characterised by attributes such as: “safe/durable”, “economic/inexpensive”, “fast/easy to built”, “functional”, “aesthetic”, “ecological” et cetera. The complexity of these demands would cause extensive work satisfying them; consequently the number of considered attributes should be reduced.

Assuming that all structures pass an assessment according to legal standards before being built, the consideration of “safe/durable” can be omitted. As a good approximation and keeping in mind that the characteristics of network arches must not be changed, the remaining attributes can be summarised by satisfying one goal, the minimisation of maximum internal forces and force variations. Achieving this aim saves material, which can mean a less expensive structure, ecological advantages, easier erection due to less weight and leads to more slender arches and hangers, which might be a criterion for aesthetics.

Due to this simplification in the following the more appropriate word “improve” is used instead of “optimise”.

3.2 Investigations carried out to find an improved hanger arrangement

In BRUNN & SCHANACK[i] an optimisation process is carried out to find an improved hanger arrangement considering small maximum internal forces and small stress ranges. In a preliminary investigation two algebraic descriptions are introduced to create hanger arrangements similar to the ones considered as near optimal by former studies. Thereupon, it is possible to vary the geometry of the network arch bridge shown in Figure 1 within these descriptions and analyse the influence lines of the structure using a 3D-FEM-model by means of SOFiSTiK® structural analysis software. The results of 850 different hanger arrangements are compared searching for minimum internal forces.

A similarity is apparent when looking at the hanger arrangements giving best results. The connection lines of the hanger crossings seem to be congruent with the radii of the arch circle. By searching for explanations and studying theories of the optimisation of structures, a new, radially orientated type of hanger arrangement has been found. It reduces internal forces compared to hanger arrangements, thought to be near optimal by former studies, by about 20 % and provides easy applicability to network arches with varying span, number of hangers and arch rise. In the following a possible derivation of the radial hanger arrangement will be presented.

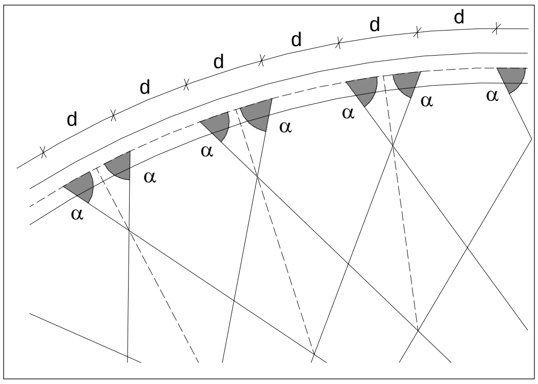

Fig. 7. Definition of the radial hanger arrangement

Since bending moments in arches depend on the line of thrust and bending moments ought to be reduced in arch bridges, it is necessary to align the line of thrust to the centre line of the arches. Due to easier fabrication the arches of network arches are part of a circle. It is a well-known fact that the line of thrust is circular if equal forces act radially towards the centre point. Thus, to decrease bending moments in the arch, loads have to be distributed by the hangers in such a way that their resulting forces are equal radial loads on the arch. Since hanger forces vary for different loads and load cases, a simplification must be made. Assuming equal maximum hanger forces, radial resulting forces are obtained if all hangers cross the arch with the same angle and the upper hanger nodes are placed equidistantly, Figure 7.

Another goal is to achieve small maximum hanger forces. Looking at the upper hanger nodes, the hanger forces counteract the forces caused by the deviation of the arch compression force and the resulting shearing force, see Figure 8. Bending moments are omitted because they do not contribute to the hanger forces. The forces N_1 and N_2 differ by experience not more than 3 %, while the shearing forces T_1 and T_2 differ exceedingly. As it can be seen both resulting forces R_N and R_T, that are to be taken by the hangers, are orthogonal to the arch centre line, which means radial. Simple vector analysis proves that the smallest hanger forces are achieved if the hangers cross the direction of the resulting arch forces symmetrically, which approves the radial hanger arrangement.

Another, easier description of this geometry is: “All hangers cross the arch with the same angle”. This theory works well with equal hanger forces.

As maximum hanger forces vary little, this geometry serves for further investigations, in which the only variable is the crossing angle between the hangers and the arch. Investigating different angles, optima can be found depending on all other parameters such as loading, stiffness of the arch and the tie, rise of the arch, number of hangers etc. For each network arch bridge project a different cross angle will give best results, considering small internal forces and force variations.

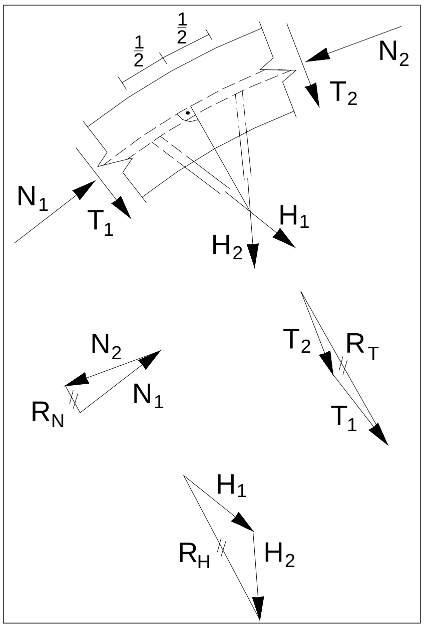

Fig. 8. Forces at the upper hanger nodes

3.3 Practical hints on near optimal hanger arrangements

The crossing angle α, see Figure 7, is to be found in a range between 45 and 60 degrees. The current research status does not provide a formula to determine optimal crossing angles for different bridges. However, in the following, hints for a good choice will be given.

- Considering fatigue a crossing angle slightly bigger than 45° will give best results, whereas small maximum internal forces occur for angles of about 55°.
- For a higher number of hangers an increased angle will be necessary.
- A smaller ratio of live load to dead load (e.g. road traffic loads instead of railway traffic loads) requires a bigger crossing angle.
- With increasing spans the mentioned load ratio will automatically be smaller; therefore increased crossing angles will satisfy minimisation demands.

The clamping of the arch at the ends of the bridge causes a disturbance range in the ideal structural behaviour of the circular arch. A slightly changed arrangement of the hangers will be necessary at the ends of the arch. In BRUNN & SCHANACK[ii], which is published on the internet, detailed instructions how to adapt the hanger arrangement in this area can be found in chapter 6.5.7.

4 SUMMARY

Compared to traditional tied arches with vertical hangers the special hanger arrangement of network arches reduces the bending moments in lower and upper chord to a tenth. Tensile and compressive forces are predominant in the whole structure whereas rigidity increases. This is of particular importance when considering the deformation restrictions of railway bridges especially when loaded by high speed railway traffic. Therefore network arches constitute a very attractive solution for railway bridges with large spans.

Railway bridges spanning up to about 100 m can have arches of H-sections. Higher requirements can be satisfied with slender box sections.

For the bridge deck, a concrete tie appears to be the best solution considering the structural behaviour of network arches, but economical advantages caused by easier erection may lead to a steel or a composite bridge deck as better alternatives. Design requirements and local conditions of each particular bridge project will decide the most economic deck design.

Network arches are very sensitive to changes in the hanger arrangement. Relaxing hangers and uneven distribution of maximum hanger forces are to be avoided. Applying the radial hanger arrangement, which means all hangers cross the arch with the same angle, provides low internal forces and ideally impedes hanger relaxation. Due to the disturbance range at the ends of the arch, caused by the clamping, an adaptation is necessary.

Wherever tied arches are considered to be applied for new railway bridges, a network arch hanger arrangement should be investigated. Material savings, the improved dynamic behaviour, a smaller deck depth and lower maintenance costs will lead to considerable economical advantages compared to conventional tied arch bridges.

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