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Design and Analysis of a Network Arch Bridge

BERNARDO MORAIS DA COSTA

Instituto Superior Técnico, University of Lisbon, Portugal
Contact: bernardo.costa@tecnico.ulisboa.pt

Abstract

The present dissertation aims the design and analysis of the hanger arrangement and the structural stability of a Network arch bridge – a tied-arch bridge with inclined hangers that cross each other at least twice. A comparative analysis with other types of hanger arrangements is also performed.

Possible solutions with respect to spans, materials and deck cross-section typology are presented and briefly discussed. Modeling using a tridimensional finite element model of the main bridge is described.

A detailed analysis of the hanger arrangement influence on the structural behavior is performed for the adopted solution. Four different arrangements of hangers – a vertical, a Nielsen and two different Network arrangements – are compared in terms of stress distributions, deflections, hangers' relaxation and fatigue behavior.

The linear stability analysis is finally performed for the different models, comparing their buckling modes and discussing the results with respect to different load patterns and load increments. The critical loads are evaluated using the European standards formulation, a simplified method and FEModel models.

Keywords: Network arch bridge, tied-arch bridge, bowstring bridge, roadway bridge design, hanger arrangement, arch stability analysis

Introduction

Arch bridges have outwardly directed horizontal forces on the arch ends. These important forces, proportional to the weight being carried out, the relation between bending and axial stiffness of the arch, the rise, and several other factors, can be visually understood from Figure 1, by the "will" of the loaded arch to "open".

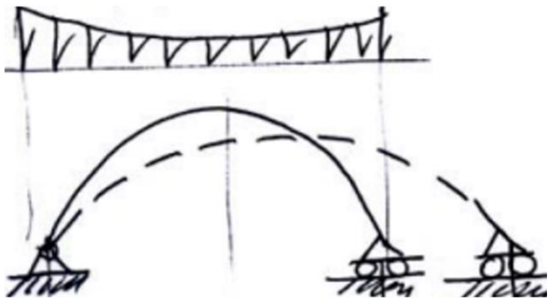

Figure 1 – Arch mechanism, expressed as a "will to open", when sustaining loads.

Tied-arch bridges, also known as *Bowstring* bridges, get their name from the way they withstand these forces. The deck is used as a tie (string) in tension to "hold" the top compressed arch (bow) (Figure 2).

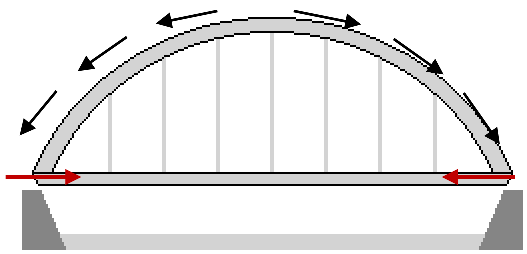

Figure 2 – Tied-arch bridge. Arch in compression, tie in tension

Network arch bridges are tied-arch bridges with inclined hangers that cross each other at least twice. To better understand it, this arrangement can be disassembled into three or more simpler sets of hanger arrangements, as for example the Nielsen arrangement of hangers, from Figure 3 to Figure 5, with hangers not necessarily with the same slope.

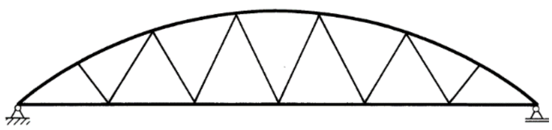

Figure 3 – Nielsen arrangement of hangers. 1 set of hangers.

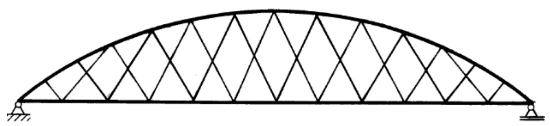

Figure 4 – Hangers cross each other once. 2 sets of hangers.

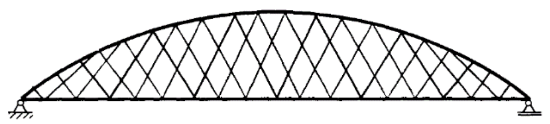

Figure 5 – Network arrangement of hangers – most hangers cross each other twice. 3 sets of hangers.

Using the Network arrangement of hangers in a tied-arch bridge, *Per Tveit* [1] refers that is possible to save between 40 % and 50 % of the cost of the superstructure, when comparing with other steel bridges.

2. Main objectives

The first aim of this thesis consists on designing a Network arch bridge that can cross over Llobregat River, in Barcelona (Spain), 170 meters wide. This bridge should have a total length of around 300 m, considering the approach spans on both sides, for crossing also a set of railway and roadway lanes. For aesthetical reasons and environmental integration of the bridge, these approach spans are also studied. Moreover, this dissertation intends to identify the advantages or disadvantages of adopting a Network arrangement of hangers and in which situations should it be considered.

A second aim of this work is to investigate the structural influence of the different hangers' arrangements on the bridge behavior. Four different hangers arrangements are studied using tridimensional SAP2000 FEModels, namely: i) a Vertical hangers arrangement, ii) a Nielsen hangers arrangement, iii) a Network hangers arrangement with constant slope, and iv) a Network hangers arrangement with variable slope. The influence of the following aspects are investigated: i) resulting stresses distributions on the arch, ties and hangers, ii) total stiffness of the structure and expected deflections, iii) number and importance of relaxing (compressed) hangers, and iv) global stability of the structure.

Finally, it is also a main objective of this work to investigate the stability of the arch, describing and comparing the multiple possible approaches. A linear stability analysis is performed, for the different models and arrangements studied, considering five different load patterns, and discussing the different ways of incrementing the bridge loads up to the bucking load. The different procedures to obtain this buckling load are also investigated using the proposed formulation from the European standards, a simplified method proposed by *Outtier et al.*[2] and comparing the results with the ones obtained using FEModel linear and nonlinear analysis.

3 Adopted Solution

The adopted solution (Figure 6) has a total length of 350 m, with two approach viaducts with balanced spans of 25.5 – 34 – 25.5 m, and a tied-arch span of 180 m, separated by expansion joints. A composite steel-concrete deck, 26.6 m wide, is the most economical solution. The main tied-arch span comprises the following structural elements: i) 2 steel (S420N) ties with a box-section (1.411 x 1.344 x 0.030 m); ii) 35 steel (S355N) ribs with a variable I cross-section [middle cross-section: h = 1.8 m, b =0.8 m, t_w = 0.012 m, t_f^{top} = 0.020 m, t_f^{bottom} = 0.040 m. End-cross-sections: h=1.0m, b= 0.4 m, t_w = 0.012m, t_f^{top} = 0.020m, t_f^{bottom} = 0.040 m]; iii) a concrete (C40/50) slab, 250 mm thick, with ϕ25//0.10 longitudinal reinforcement bars (A500); iv) 2 steel (S420N) arch box-sections (1.400 x 1.200 x 0.040 m) leaning 79° inwards; v) 2 x 70 steel (S460N) hangers, with a 80 mm diameter, a Network arrangement with hangers equidistant (5 m distant) ate the deck level, and coincident with the ribs / ties intersection, and a constant 65° slope; and vi) other secondary steel elements such as 7 bracing box-section beams linking the arches and 2 box-section end-cross-girders (Fig. 6).

The adopted constructive procedure includes the launching of the steel part of the arch structure through the approach bridge decks, until its final position, with the aid of a floating pontoon, as can be seen in Figure 7.

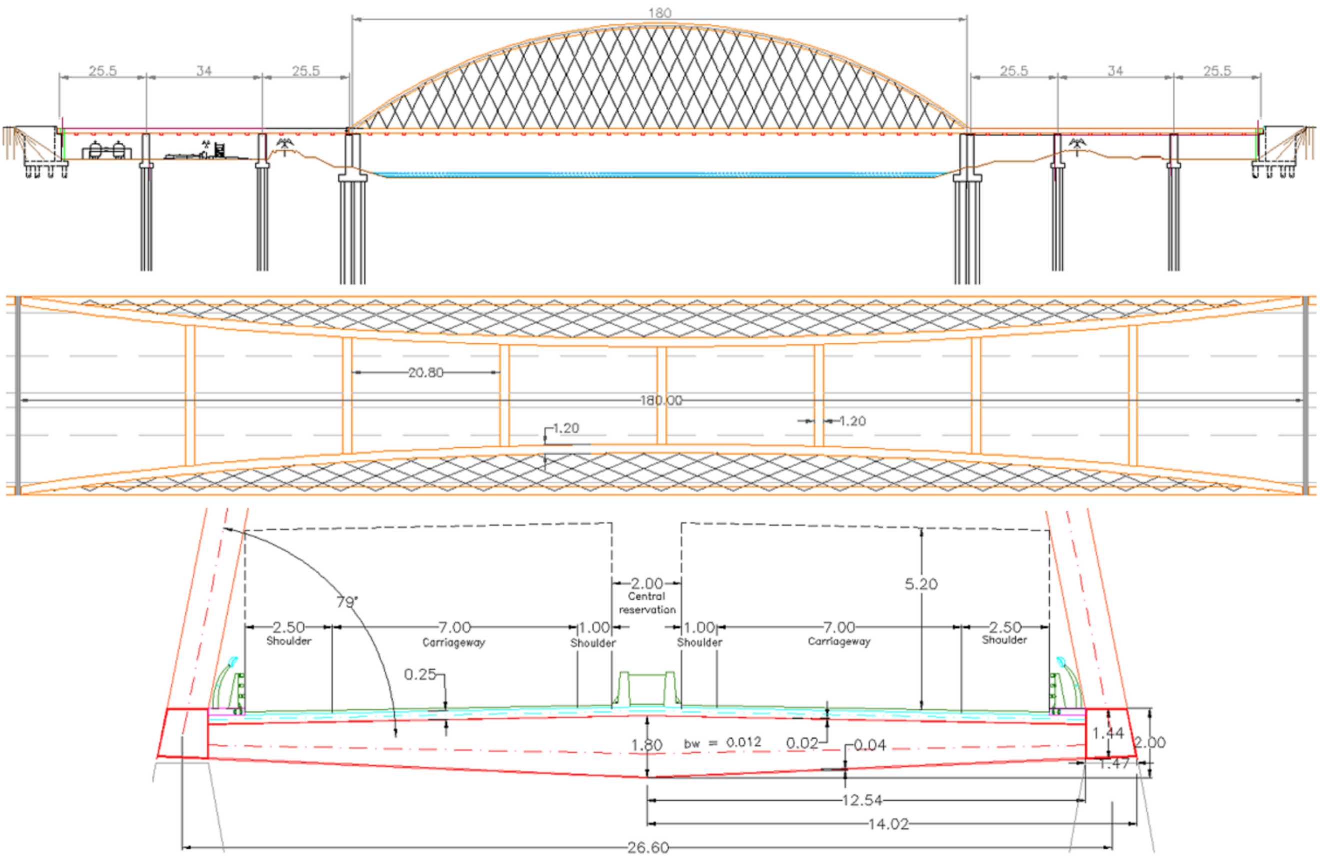

Figure 6 – Elevation view, plan view and deck's cross-section view of the designed bridge.

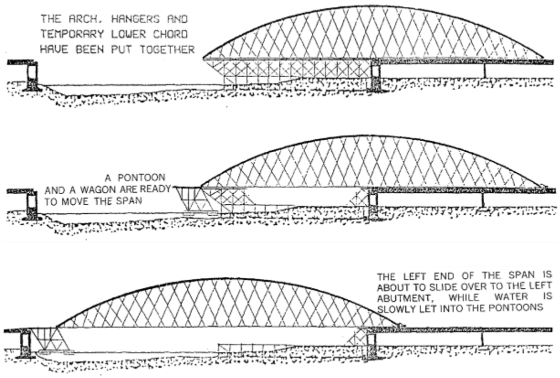

Figure 7 - Pushing the arch through the access viaduct. Sequence by Per Tveit [1].

Then precast slabs are then put in place supported by the ribs' top flanges and finally concreting operations are done in a symmetrical form with respect to the arch. Headed studs on the ribs' top flange and ties' inner web assure the deck's composite behavior.

A database collected by *Gonçalves, P.* [3] gives useful information on several built tied-arch bridges. The comparison of these results with the adopted solution (a star in the graphics) is presented in Figure 8.

4. Structural analysis

4.1 Overview

The network arch can be compared to a beam with a compression (arch) and a tension (tie) zone. A higher arch decreases axial forces in the chords and it's mainly aesthetic reasons that limit this height. The network hangers' arrangement act like a web, taking some of variation of the shear, while most of it, is taken by the vertical component of the arch axial force.

4.2 Deck

The tied-arch span imposes two important events to its deck: a) global bending moment of the deck; and b) tension of the slab due to the referred arches' "will to open".

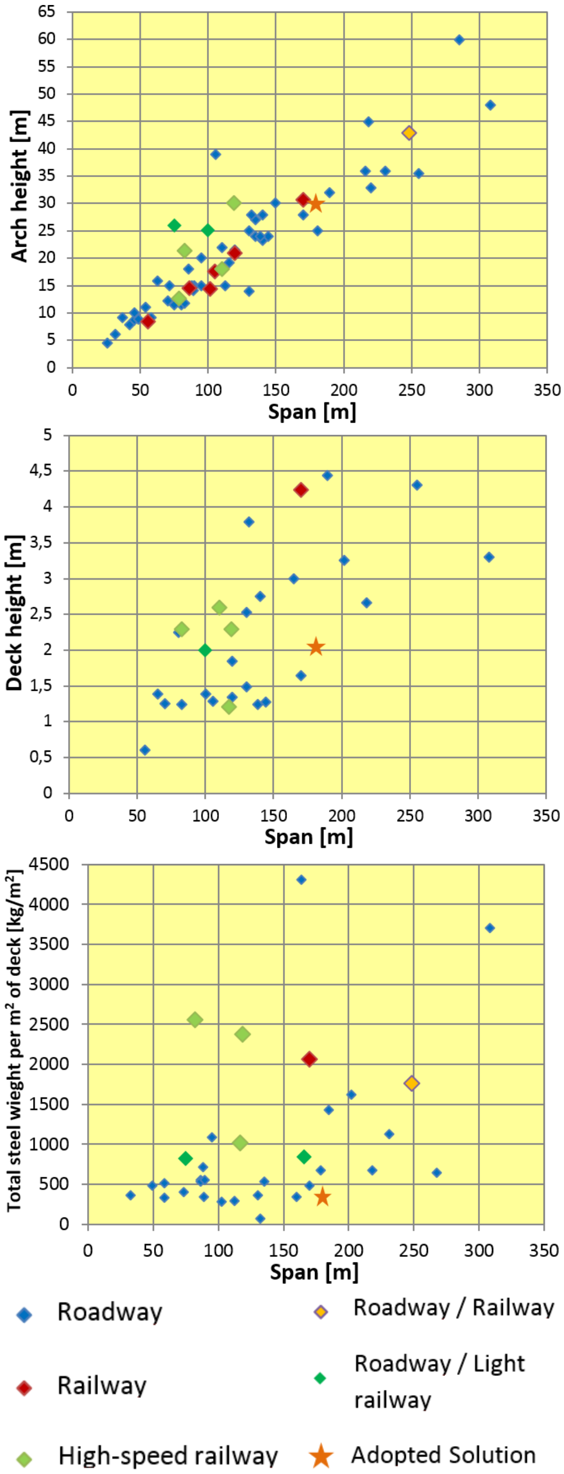

Figure 8 – Adopted solution comparison with built tied-arch bridges. Database collected by *Gonçalves, P.* [3]

Taking into consideration the constructive procedure, tension of the slab comes only from the SDL (Superimposed Dead Loads) and variable loads, as before that, the precast slab's do not have stiff solid concrete connecting them and only the ties are there to resist the horizontal forces from the arch. The same occurs with the transversal compressions of the composite behavior slab-rib.

Ribs, which are a class 4 section, must resist a constructive process without the aid of scaffolding. Therefore, as usual the composite section is only activated for SDL and variable loads, and for the DL (Dead Loads), only the steel section resists.

For the same reason, the ties, for the DL, will support alone the arch "will" to open, and, at both ends of the span (at the corners), it also supports basically alone all following loads since the connection to the slab is barely mobilized. Moreover, the interaction with the arch causes important bending moments in these corners.

The resulting stresses on the deck, at the conditioning cross-sections, for the Ultimate Limit States, are summarized in Table 1.

Table 1 – Deck's main conditioning stress results

Structural Element	f_{Ed}	τ_{Ed}	f_{yd}
Slab's rebars	403	-	435
Ribs	213	161	355
Ties	392	59	420
Units		MPa	

4.3 Arches

The arch springs (corners) are also over stressed from the interaction between both chords (arch & tie). Though, the highest bending moments are in-plane, they occur in the same arch sections and are due to wind action. The elastic stress verification, at the arch spring, are presented in Table 2.

Table 2 – Arch corner elastic verification (MPa)

Stress	N_{Ed}	$M_{Ed\ 3\text{-}3}$	$M_{Ed\ 2\text{-}2}$	Total	Limit
$\sigma_{x,Ed}$	-221	52	133	405	420
τ_{Ed}	-	-	-	20	242

And
$$N_{b,Rd} = \frac{\chi \times A \times f_y}{1{,}0} = 67588\ kN \geq N_{Ed} \qquad (1)$$

With: $X_y \sim X_z =$ 0.790 and N_{Ed} = 44926 kN

While the instability occurs with an out-of-plane movement for the all observed buckling modes, in-plane movements are simultaneously visible in every single buckling mode, since the arches are inclined at 79^0 with the horizontal plan and cannot buckle outwards without some vertical movement. Since instability in both axis of the arch cannot be easily divided, in a conservative and simple way, λ_y is assumed to be equal to λ_z, such as χ_y and χ_z. The resultant ULS verification is presented according to *EN1993-1-1*[4]:

$$\frac{\mathrm{N}_{Ed}}{\frac{X_y \times \mathrm{N}_{Rk}}{\gamma_{M1}}} + k_{yy} \times \frac{M_{y,Ed}}{X_{LT} \times \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \times \frac{M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \qquad (2)$$

$$\frac{44926}{67588} + 1.059 \times \frac{4455}{36000} + 0.529 \times \frac{8985}{28462} = 0.963 \leq 1.0$$

4.4 Hangers

Hanger's verifications are the following:

1) Maximum axial force.
2) Characteristic axial force (limited to 50% of the ultimate resistance).
3) Fatigue.
4) Relaxation.

1), 2) and 3) are easily checked, by correctly adapting the hangers' characteristics, and 2) resulted the most conditioning. The 4), however, presents great interest to be discussed, since it is entirely related to the hangers' arrangement and live loads patterns and significance.

Relaxation can be seen as the consequence of the hangers' inability to sustain compression forces, and, at both ends of the span, hangers do compress due to a truss-beam like behavior. Notice the similarities in Figure 9 and Figure 10.

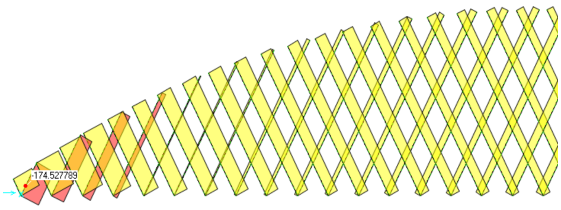

Figure 9 – Hangers' axial forces for DL (half span shown). Leaning inwards' hangers are compressed.

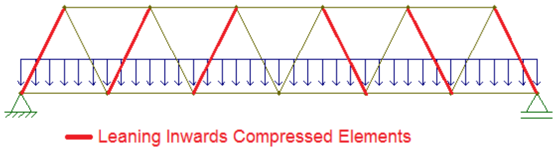

Figure 10 – Truss-beam scheme. Leaning inwards diagonals compressed.

Then, for half-span loading, other central hangers tend to relax (dashed hangers in Figure 11).

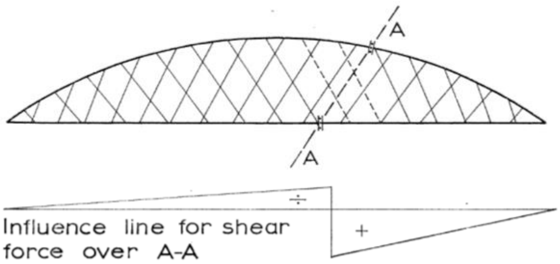

Figure 11 - Influence line for shear force over A-A, related to the influence line of the dashed hangers.

Hanger's relaxation may or may not have significant consequences on the structure since it changes considerably its structural behavior. Therefore it might be of interest to prevent it by adopting an appropriate hangers' arrangement. *Per Tveit* [1] and *Brunn & Schanack* [5] give multiple useful advices on this subject, leading in the present case to the 65º slope adopted, which prevents relaxation for a half-span live load patterns.

For the full-span load case, it is decided to study the required prestress forces in the first 8 leaning inwards hangers of each arch corner to avoid relaxation. For that, an influence matrix of the effects of pre-stressing each hanger was built. This allows obtaining the right combination of prestress that prevents relaxation. Two prestress combinations, applied in different periods were required to improve the structure behavior during the construction stages.

5. Hangers arrangements and arch instability investigations

5.1 Hangers arrangement study

Different arrangements are studied, namely: "Vertical", "Nielsen", "Network" and "Optimized Network", and are respectively illustrated in Figure 12.

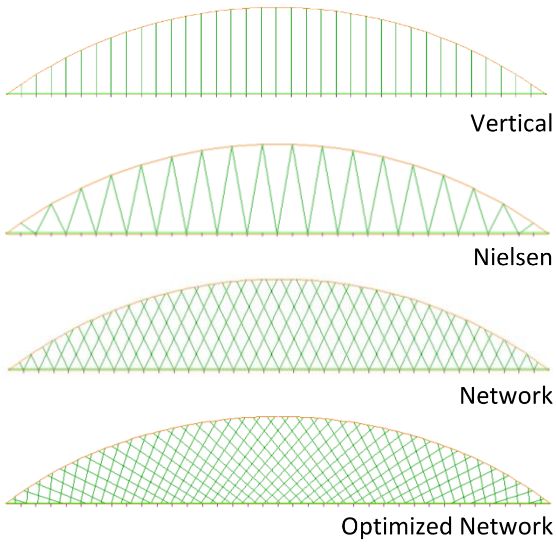

Figure 12 – Hangers arrangements studied.

The same materials, deck and arch cross-sections were adopted in the different models. The hangers' cross-section area was defined as inversely proportional to the number of hangers (Table 3). Two load distributions, matching the LM4 preconized in *EN1991-2*[6], were defined: i) "LD-All" – Load in all span length, and ii) "LD-Half" – Load in the left-half of the span (see the first two schemes of Figure 13). The main forces results and displacements are listed in Table 4, and lead to the following comments:

Table 3 – Hangers characteristics on the different models

	Vertical	*Nielsen*	*Network*	*Opt. Network*
Nº	2 x 35	2 x 34	2 x 70	2 x 80
A (m^2)	0.0101	0.0104	0.0050	0.0044
f_{yd} (Mpa)	460	460	460	460
N_{Rd} (kN)	4646	4784	2312	2021

Nº - 2 (arches) x number of hangers (per arch)

- Vertical arrangement is extremely vulnerable to half-span loading. It balances unsymmetrical loads by bending both the arch and the tie, since hangers do not connect different sections of the deck to the arch. This is connected with what is observed with deflections.
- The distance between hanger's nodes on the arch does not influence its results.
- A higher number of hangers fairly decrease the resultant bending moments on the ties.
- The adopted solution (Network) has the best results on the arch and on deflections.
- The Optimized Network arch has the best results on the tie.
- With almost the same number of hangers as the Vertical, the Nielsen arrangement behaves seemingly well, in respect to the main axial forces and bending moments.

The next step is to evaluate the hangers' behavior. An optimal arrangement solution accomplishes two goals:

1. Low maximum axial force. Since models differ on the number of hangers, the way to better access this is to measure N_{max} / N_{Rd}. N_{Rd} is proportional to the area, which was defined inversely proportional to the number of hangers.
2. Even axial forces (low $N_{variance}$). This prevents overdesigned hangers and/or different solutions for different hangers.

$N_{variance}$ is simply defined, in Eq.3.

$$N_{Variance} = \frac{\mathrm{N_{max}} - \mathrm{N_{average}}}{\mathrm{N_{average}}} \tag{3}$$

Therefore, the axial force results on the hangers are presented next, in Table 5, which raises the following considerations:

- To compensate, and clearly related to the disturbing results obtained previously, the Vertical arrangement shows the best results here.
- The Optimized Network arrangement finally reveals its benefits, having virtually no compressed hangers and a considerable even axial force between hangers.
- Nielsen arrangement is by far the most penalized, in contrast to the previous fairly good results. The N_{min} = -889 kN indicates an alarming compression value. In fact, this compression force alone will exceed the tension forces from the permanent loads. Moreover, an even bigger N_{max} leads to the very demanding results observed.

Table 4 – Main forces and displacements on the different hanger arrangements models.

Arrangement:		Vertical		Nielsen		Network		Opt. Network		Units
	LD:	All	Half	All	Half	All	Half	All	Half	
Arch	$M_{33,max}$	1631	**-12203**	937	916	848	688	996	817	kNm
	N_{max}	-8492	-4513	-7767	-5633	-7664	-5576	-7811	-5635	kN
Tie	$M_{33,max}$	-1563	**-10955**	934	1114	732	737	607	656	kNm
	N_{max}	5339	**9721**	4931	3653	4967	3677	4738	3517	kN
	δ_{max}	132	**860**	62	78	57	38	65	38	mm

Table 5 – Hanger axial forces on the different hanger arrangements

	Arrangement:	Vertical		Nielsen		Network		Opt. Network		Units
	LD:	All	Half	All	Half	All	Half	All	Half	
Hangers	N_{min}	215	58	46	**-889**	-118	-168	14	-65	kN
	N_{max}	344	222	584	1114	283	304	248	211	kN
	$N_{average}$	277	138	289	144	144	72	154	77	kN
	N_{Rd}	4646		4784		2312		2021		kN
	$\frac{N_{max}}{N_{Rd}}$	7%	5%	12%	23%	12%	13%	12%	10%	-
	$N_{Variance}$	24%	61%	102%	672%	96%	322%	61%	174%	-

From these results, compressed hangers were removed from the Nielsen model, LD-Half was applied and a bending moment of 10606 kNm was obtained, opposing to the previously obtained 688 kNm with compressed hangers. Final conclusions are noted:

- The Nielsen arrangement has severe relaxation issues. For unsymmetrical loads it sees many of its hangers relaxed. This changes the apparently good results obtained in Table 4, since hangers cannot mobilize compression. For this reason, when there is a live load in the nearness of relaxing hangers, the effects of incrementing that live load are very much like the ones in the Vertical Model. So, accordingly to a higher or lower importance of the live loads, the Nielsen arrangement behaves respectively, more closely to the Vertical Model or more closely to the Network models.

- The Vertical model, for unsymmetrical loads, gets bending moments on the arch 17 times greater than the ones of the Network arch.

- The Network arrangement has the lowest forces and bending moments on the arches. Its disadvantages to the Optimized Network, regarding the hangers, can be partially compensated by applying appropriate prestress.

Finally, during the analysis, a few more remarks were noted:

- Within the same arrangement, the greater axial stiffness of the hangers, the more uneven forces result.
- The higher the number of hangers, the lower bending moments on the ties.
- More steep hangers give smaller hanger forces but bigger stress variations and relaxations problems.

5.2 Arch Instability analysis

The same arrangements are analysed considering 5 different LD (load distibutions) of the same LM4 preconized in *EN1991-2*[6] (Figure 13). The results of the instability analysis were obtained from incrementing Live Loads only, on the FEModel. This is a conservative approach which revealed to be sufficiently accurate. The results are listed in Table 6.

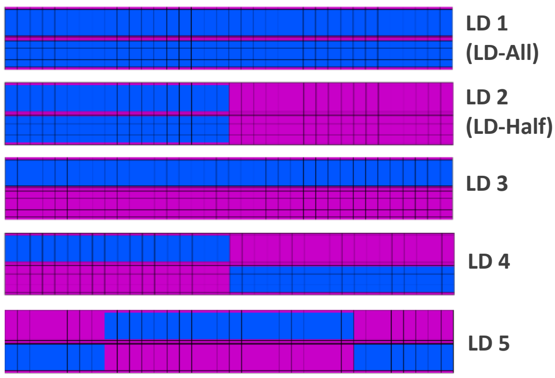

Figure 13 – Load Distributions applied in this study. Blue color corresponds to a 5kN/m2 vertical uniform distributed load.

Table 6 – Instability Analysis Results on the FEModels

Arrangement	LD:	1	2	3	4	5
Vertical	λ	12.2	22.0	24.2	24.3	24.4
	N_{Ed} [kN]	8492	4513	6200	5578	5111
	$N_{FE,el}$ [kN]	103761	**99316**	150152	135562	124630
Nielson	λ	16.9	28.2	33.6	31.6	32.8
	N_{Ed} [kN]	7769	5162	5682	5062	4637
	$N_{FE,el}$ [kN]	**131215**	145498	190989	159954	152288
Network	λ	17.0	26.1	33.8	33.9	33.9
	N_{Ed} [kN]	7677	5576	5604	4644	4636
	$N_{FE,el}$ [kN]	**130231**	145478	189221	157435	157164
Opt. Network	λ	16.3	26.2	32.5	32.6	32.6
	N_{Ed} [kN]	7811	5635	5696	4675	4982
	$N_{FE,el}$ [kN]	**127343**	147775	184872	152338	162277

λ = buckling factor   **N_{Ed} =** maximum compression force the LD applied $N_{FE,el} = \lambda \times N_{Ed}$

According to Per Tveit [1], in a normal network arch the decisive load cases are maximum load on the whole span (LD1), which was confirmed.

Remembering that all buckling modes are significantly out-of-plane, the results were not much affected by the hangers' arrangement. They were, in fact, affected and are mostly related to the previous stress distribution results of Table 4.

For the out-of-plane buckling, *EN1993-2*[7] presents two procedures to obtain the β (Buckling length factor): i) "Out-of-plane buckling of arches with wind bracing and end portals", and ii) "Out of plane buckling factors for free standing arches". The results from proceeding according to i) and ii), and the result of the FEModel, for the Network arrangement, are resumed in Table 7.

Table 7 – Instability procedures comparison

Procedure	N_{cr}
i)	207310 kN
ii)	19021 kN
FEModel	130231 kN

It can be concluded that both *EN1993-2*[7] procedures provide a lower and upper boundary of the FEModel result, but within an unsatisfying large interval. Actually, procedure i) and ii) predicts accurate results for the FEModels illustrated respectively in Figure 14 and 15.

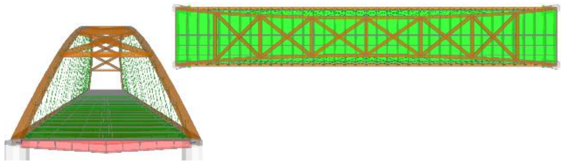

Figure 14 – FEModel with stiff wind bracing elements - $N_{FE,el}$ = 226265 kN.

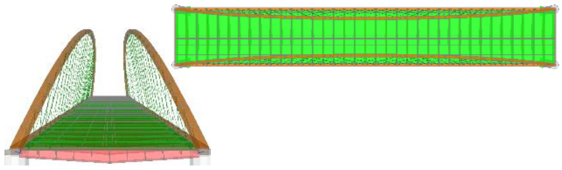

Figure 15 – FEModel with free standing arches - $N_{FE,el}$ = 25044 kN.

Finally, a study from *Outtier et al* [2], based on a database of more than 50 steel tied-arch bridges spanning from 45 to 200 m, where detailed linear and nonlinear elastic-plastic analysis were performed and compared to the *EN1993-2*[7] procedures, led into a simplified method of assessing instability by proposing an alternative formula to obtain β. The results of the N_{cr} obtained for the current Network bridge, from this alternative β, were unexpectedly high (N_{cr} = 404551 kN). In fact, β factors, obtained for spans greater than 150 m were suddenly low. After confirming with one of the researchers that this was not expected to happen, it was concluded it is worthy trying to improve the formula and to extend its validity domain in the future since it offers an easy and straightforward procedure.

6. Conclusions

The base case design here developed allowed substantial material savings when compared to many other tied-arch bridges.

The pre-design of the hanger's arrangement, facilitated by *Per Tveit*[1] and *Brunn & Schanack*[5], proved to be remarkably accurate on the benefits it predicted.

Both Network arrangements analyzed evidenced clear structural advantages over the Vertical arrangement. The Vertical arrangement only presented benefits for the hangers' forces, as a consequence of over requesting the bending stiffness of the chords.

On the Nielsen results, if significant unsymmetrical live loads exist, the severe relaxation of hangers leads this solution to behave similarly to the Vertical hangers' arrangement model.

With the inclination of the arches and with the presence of the bracing beams it is extremely unlikely to occur a pure in-plane buckling. The inclination of the arches also reduces the wind portal frames and the bracing beams length, resulting in a more stable solution.

Hangers' arrangements don't affect directly the stability of the arch significantly, but indirectly through the stress distributions. An integrated methodology for the simplified analysis of in-plane and out-of-plane buckling of the arch still needs to be developed.

In the author's opinion, for out-of-plane conditioned tied-arch bridges, *EN1993-2*[7] may be carefully used in the two situations analyzed and successfully compared: i) when no bracing beams exist, or ii) when the bracing beams form a really stiff structure.

Finally, this study hopes to have demonstrated that Network arch bridges can be competitive and structurally efficient when compared to other tied-arch bridges.

7. References

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