Structural behaviour of network arch bridges with laterally inclined arches

Comportamiento estructural de puentes tipo network con arcos inclinados

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ABSTRACT

The paper describes a broad parametric study of the structural behaviour of network arch bridges with laterally inclined and parallel arches under vertical and lateral loading. Influence of the angle of arches, upper bracing type, f/L arch ratio, number and angle between the hangers on structural behaviour of the bridge members is investigated. Five separate studies are conducted using precise three-dimensional models with tubular arch profiles combined with orthotropic steel deck. Traffic and lateral wind load models include uneven loading across the width and length of the deck. Design recommendations based on the analysis of results concerning optimum values of each investigated parameter are presented.

RESUMEN

El artículo describe un amplio estudio paramétrico del comportamiento estructural de los puentes arco tipo network con arcos inclinados y paralelos bajo cargas verticales y horizontales. Se investiga la influencia del ángulo de los arcos, el tipo de arriostramiento, la ratio f/L del arco, y el número y el ángulo entre péndolas en el comportamiento estructural. Se realizan cinco estudios utilizando modelos tridimensionales con arcos de tubo estructural y tablero de losa ortótropa de acero. Los modelos de carga de tráfico y viento lateral incluyen cargas no uniformes. En base al análisis de los resultados obtenidos en el estudio paramétrico, se presentan recomendaciones para el proyecto de puentes tipo network.

KEYWORDS: network arch bridge, inclined arch, inclined hangers, tied arch
PALABRAS CLAVE: puente arco tipo network, arco inclinado, péndolas inclinadas, arco atirantado

1. Introduction

The network arch is a special type of tied arch bridge with inclined hangers where some of the hangers cross each other at least twice [1]. It was invented in 1955 by Per Tveit [1], who during the work on his master's thesis noted that multiple hanger crossing considerably improved the structural response in case of uneven or concentrated loads. In the recent years there has been an increased research activity on this subject [2-4]. Nevertheless, most of the studies have been conducted on systems with parallel arches. However, in the recent years a number of network arch bridges with laterally inclined arches have been constructed. Recent examples are Deba and Palma del Río bridges in Spain [5] and Dziwna bridge in Poland [6].

Following this increase it was reasonable to conduct an investigation in this field. One of the main objectives is to determine how different factors influence the inclined arch system in comparison to the one with parallel arches and to guide the designers in the process of choosing whether to use parallel or inclined arch geometry.

The analysis is divided into five individual studies where each one of the following parameters is investigated: the angle of network arches, type of the upper bracing between arches, f/L arch ratio, number of the hangers and the angle between the hangers and the arch. Each study is conducted on two groups of models (with inclined and parallel arch geometry) configured as close as possible to the real engineering models.

Vertical traffic and lateral wind load models defined per EN norms include uneven loading across the width and the length of the deck with the purpose to explore the advantage of inclined arch geometry in wider decks. The results of each study are described with emphasis on the behaviour under even and uneven loading.

Prior to the analysis described in this paper, the authors decided to undertake an initial two-dimensional analysis using only vertical loads. The main objectives were to investigate the behavior on larger number of models with a broader range of parameters in order to narrow down the investigation range and to determine the base models for the three-dimensional study.

2. Methodology

The present three-dimensional linear elastic analysis builds on the conclusions obtained in the initial analysis.

Certain factors are now narrowed down while those ones characteristic only to the three-dimensional models are now introduced (see Table 1).

Table 1. Overview of the variation factors used in the 2D and 3D analyses.

Factors varied in the analyses	2D analysis range	3D analysis range
Angle of the arches	-	0º-27.7º
Type of bracing	-	4 types
f/L ratio	1/5 – 1/9	1/5.5 – 1/7
n hangers	2x6 – 2x24	2x14 – 2x26
Angle of hangers	40º-60º	40º-55º
Type of hanger distribution	7 types	1 type (radial)
Arch rigidity	5 types	1 type
Tie rigidity	5 types	1 type

2.1 Structural models

The five separate studies are based on two base models with the following properties: inclination of the arches to the vertical plane $\alpha=0º$ in the parallel arch base model and $\alpha=27.7º$ in the base model with inclined arches. Height to span ratio f/L is taken as 1/6, the number of hangers as n=2x18, the angle between the hangers and the arches as $\alpha=45º$ and upper bracing as cross type truss. The base models use the radial hanger arrangement as proposed by Brunn and Schanack [2]. The hangers are modelled as steel rods with diameter D_h=60mm. Each one of these parameters is varied in each study while all the other parameters are kept constant. The total of 38 models were used (see Table 2).

Table 2. Overview of the number of models as a function of different factor variations.

Factors	Models with parallel arches	Models with inclined arches
Angle of the arches	6 models	
Type of bracing	4 models	4 models
f/L ratio	4 models	4 models
n hangers	4 models	4 models
Angle of hangers	4 models	4 models

All the models have equal dimensions of the deck with length L=120m and width

B=23m. The length was chosen based on two case studies: Deba and Palma del Río river bridges. The arch and the tie are modelled as circular hollow sections (CHS) with constant diameter (D) and thickness (t). Comparison of the model geometry with the case study bridges is presented in Table 3, and the base models are shown in Figures 1 and 2.

Table 3. Arch and tie cross section comparison

Case studies with CHS	D [mm]	t [mm]	D/t [mm]	L [m]	L/ D
Deba	800	45	17.8	110	138
Palma del Río	900	50	18.0	130	144
Present model	1000	30	33.3	120	120

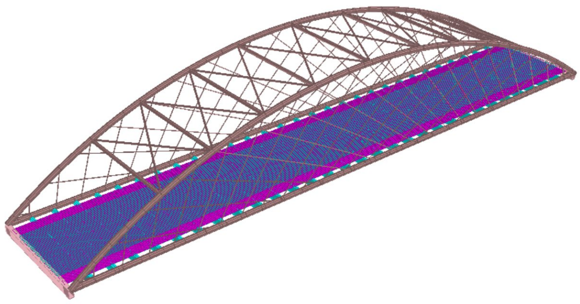


Figure 1. Three-dimensional base model with parallel arches.

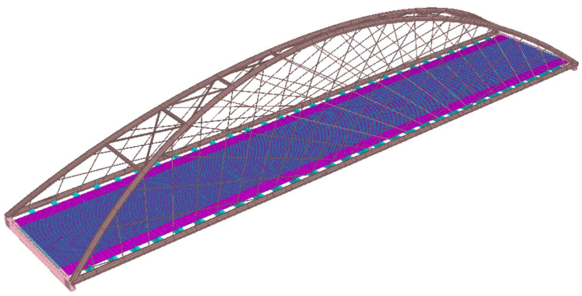


Figure 2. Three-dimensional base model with inclined arches.

A number of the network arch bridges built in the previous 50 years were designed with prestressed concrete decks. Recently the appearance of the composite decks (Deba river, Palma del Río river, Dziwna river bridge) has brought some variety in the deck construction of the network arch bridges. One of the main advantages of the bridges with inclined arches can be seen in the cases of uneven loading across the width of the bridge deck. Therefore, it was decided to use a wider deck in order to investigate the influence of the uneven loading. Hence, the deck in this analysis is wider in comparison to the above-mentioned bridges.

The models in this study are designed with orthotropic steel deck (t_{deck}=12mm). The distance between the arches at the piers is 23m. Bridge deck consists of four traffic lanes and two footways. The total width of the traffic deck is 14.4m (4 tracks x 3.25m + 2 x 0.7m free space on each side of the deck). Each footway is 2.4m wide. Dimensions of the orthotropic deck and longitudinal ribs in the traffic deck and footways are calculated following the recommendations for orthotropic plate thickness and details EN1993-2 Annex C [7]. The front view of the bridge, the bridge deck and the orthotropic deck details are shown in Figures 3, 4 and 5.

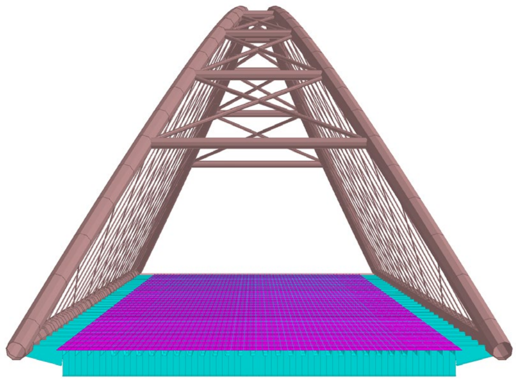


Figure 3. Front view of the base model.

Intermediate cross beams are spaced every 4m and are modelled as I section (h=1500, b=550, t_f=30, t_w=15mm) with the deck plate acting as the superior flange. End cross beams are modelled as rectangular sections (h=1500, b=800, t=15mm). All the bracing types are designed as CHS with 9 cross beams and corresponding diagonal members leaving necessary clearance gauge.

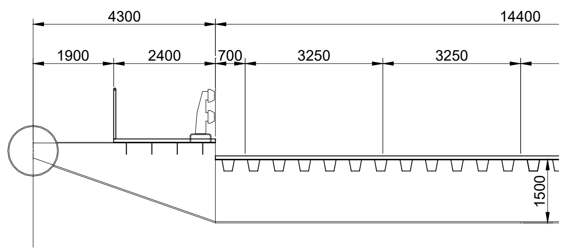
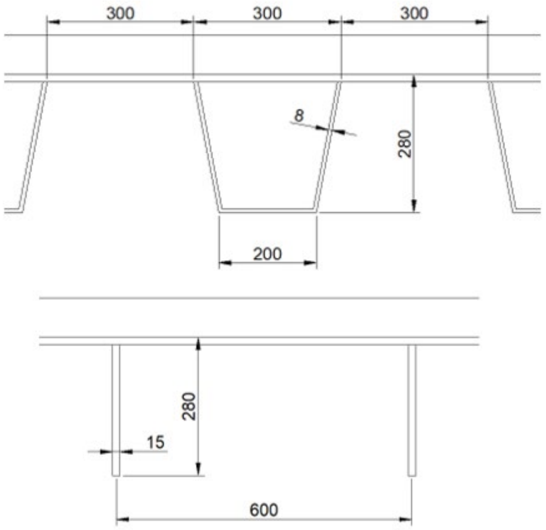


Figure 4. Bridge deck modelled as orthotropic plate (in mm)



2.2 Determination of the loads

The permanent actions include the self-weight of the bridge structure and the dead load of the road surfacing and fixed equipment. The variable actions consist of vertical traffic loads determined in accordance with EN 1991-2 [8] and horizontal wind loads determined per EN 1991-1-4 [9]. The simultaneity of the traffic loading systems is considered by using two Load groups 1A and 4 per EN 1991-2 [8]. Distribution of the traffic loads on the deck is based on Load groups 1A and 4 acting: a) over the whole bridge span and whole width, b) over half of the span and whole width and c) over the whole span and half width (Table 4).

Table 4. Overview of the traffic loads in the analysis

Traffic loads	Whole span – whole width	Half span - whole width	Whole span – half width
Load group 1A Heavy vehicles positioned centrally between two arches	$P_{A,1}$	-	-
Load group 1A Heavy vehicles positioned closer to one arch	$P_{B,1}$	$P_{B,2}$	$P_{B,3}$
Load group 4 (crowd loading)	$P_{C,1}$	$P_{C,2}$	$P_{C,3}$

The difference between the $P_{A,i}$ and $P_{B,i}$ loads is the position of the heavy vehicle regarding the distance to the arch. The carriageway including the central reservation is divided into 4 notional lanes of 3m and a remaining area of 2.4m.

The wind loads are determined considering that the deck is located at height z=20m and the top of arch at maximum height z=40m in all bridge models in order to obtain the same lateral loading for the purpose of analysis. The highest basic wind velocity for the Spanish territory 29m/s is taken in the analysis. Considering the return period of 100 years and the coefficient c_{prob}=1.04, $v_b(100)$=30.16m/s is obtained. Wind forces acting perpendicular to the bridge span are presented in Table 5.

Table 5. Wind forces acting on the arch and the deck

Bridge element	Fw [kN/m]
Deck	6.37
Arch	4.60

2.3 Load combinations

Load combinations of the parametric analysis are determined in accordance with the EN1990 [10]. In total, there are 31 load combinations. Prior to the parametric analysis, the deck together with longitudinal and transversal steel beams was dimensioned using load cases (LC11-LC14) (see Table 6). The rest of the load cases were used first for the dimensioning of the arches, ties, hangers and upper bracing and later for the parametric analysis. The load cases are divided into six load combination groups (see Table 6): group 10 used for deck dimensioning, group 20 with wind load as the only variable action, group 30 with traffic load as the only variable action, groups 40 (not shown as not relevant) and 50 with traffic loading as a leading variable action combined with wind in positive and negative direction and group 60 with wind loading as the only variable action used for serviceability limit state.

Table 6. Overview of the traffic loads in the analysis

Load cases	Self-weight G_{sw}	Dead load $G_{as}+G_{hr}$	Traffic load $P_{x,i}$	Wind load W±
LC11	1.00 G_{inf}	1.00	1.35 $P_{A,1}$	-
LC12	1.00 G_{inf}	1.00	1.35 $P_{C,1}$	-
LC13	1.35 G_{sup}	1.35	1.35 $P_{A,1}$	-
LC14	1.35 G_{sup}	1.35	1.35 $P_{C,1}$	-
LC21	1.00	1.00	-	1.5 W+
LC22	1.35	1.35	-	1.5 W+
LC23	1.00	1.00	-	1.5 W-
LC24	1.35	1.35	-	1.5 W-
LC31	1.35	1.35	1.35 $P_{A,1}$	-
LC32	1.35	1.35	1.35 $P_{B,1}$	-
LC33	1.35	1.35	1.35 $P_{B,2}$	-
LC34	1.35	1.35	1.35 $P_{B,3}$	-
LC35	1.35	1.35	1.35 $P_{C,1}$	-
LC36	1.35	1.35	1.35 $P_{C,2}$	-
LC37	1.35	1.35	1.35 $P_{C,3}$	-
LC51	1.35	1.35	1.35 $P_{A,1}$	0.9 W-
LC52	1.35	1.35	1.35 $P_{B,1}$	0.9 W-
LC53	1.35	1.35	1.35 $P_{B,2}$	0.9 W-
LC54	1.35	1.35	1.35 $P_{B,3}$	0.9 W-
LC55	1.35	1.35	1.35 $P_{C,1}$	0.9 W-
LC56	1.35	1.35	1.35 $P_{C,2}$	0.9 W-
LC57	1.35	1.35	1.35 $P_{C,3}$	0.9 W-
LC61	1.0	1.0	-	0.2 W+
LC62	1.0	1.0	-	0.2 W-

3. Results and discussion

The influence of the variation of each parameter on the behaviour of hanger axial forces, arch and tie axial forces, vertical and lateral bending moments in the arch and tie and finally the combined normal stresses in the arch and tie is analysed for both models with inclined and with parallel arches. The study of the influence of the bracing type is complemented with the analysis of the lateral displacements of the arches. In the following paragraphs, only the most relevant results obtained in the analysis are presented.

3.1 Influence of the angle of arches

This analysis is done on six models where the distance between the arches at their highest point is changed from 23m to 2m in regular steps of 4.2m. This results in the angles between the arch and the vertical plane shown in Figure 6.

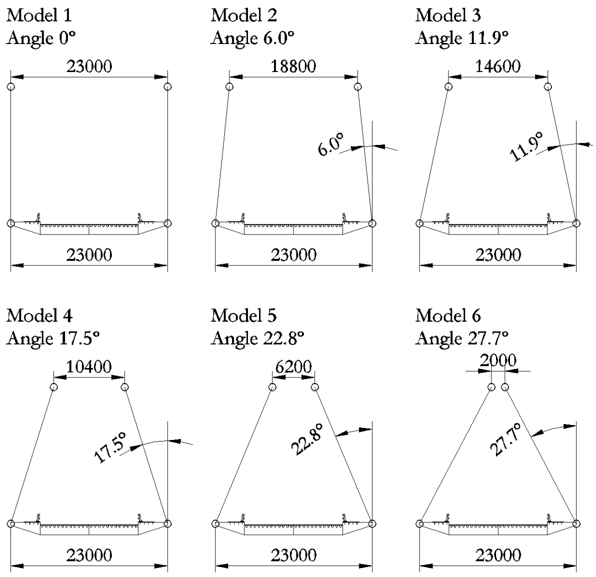


Figure 6. Models for the analysis of the influence of the angle of the arches

The f/L ratio is kept constant in this analysis, and it refers to the vertical height of the bridge and not to the height of the arch in its own plane. Therefore, the increase in the angle of the arch to the vertical plane results in the increase of the height of the arch in its own plane as well as the overall structural weight.

Table 7 Weight comparison of the six models with varying angle of the arches

Model nº	Angle of the arches [º]	Structural weight [tons]	Weight comparison [%]
Model 1	0.0	1123	100.0
Model 2	6.0	1114	99.3
Model 3	11.9	1107	98.6
Model 4	17.5	1101	98.0
Model 5	22.8	1096	97.6
Model 6	27.7	1093	97.3

On the other side, a smaller distance between the arches leads to the decreased weight of the upper bracing. Finally, the decrease in the weight due to the arch bracing is higher than the increase due to the arch height. A comparison in the model structural weight is shown in Table 7.

All the load cases with combined traffic and wind loads including even and uneven loading show an increase in the hanger axial forces with an increase of the angle of the arches to the vertical plane. This increase shows only minor variation up to 12º. It is then that the values start increasing up to 21% in case of the most critical load cases (see Figure 7). LC53 is the only exception and shows little variation.

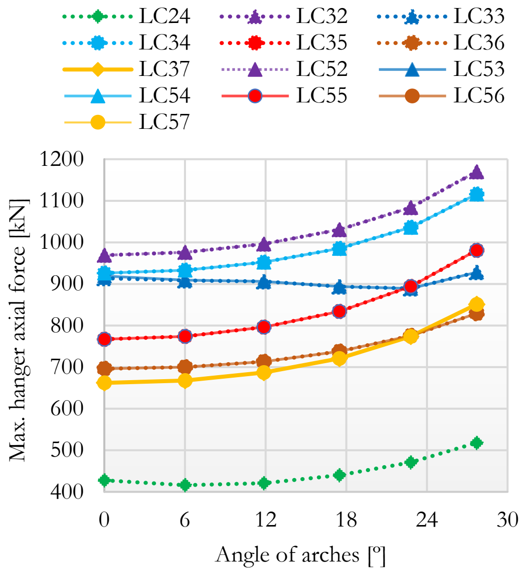


Figure 7. Influence of the angle of arches on maximum hanger forces

The increase of the axial forces in the arch (up to 3%) and the decrease of the tension forces in the ties (up to 5%) describes the influence of the change in the angle of the arches on the axial forces in the chords (arches and ties). The vertical bending moments increase with the increase of the angle of the arches due to the lower rigidity of the hangers. However, the increase in the angle of the arches contributes to the higher lateral rigidity of the chords which results in the lower lateral bending moments.

The diagram containing the arch stress analysis (Figure 8) clearly demonstrates that the overall stresses decrease for the load combination groups 20 and 50 and increase for the load combination group 30 with an increase in the angle of the arches. It is concluded that the optimum angle lays between 18º - 24º. Results also show that the inclined arch geometry results in lower arch and tie stresses under uneven loading (LC52, LC53 and LC54).

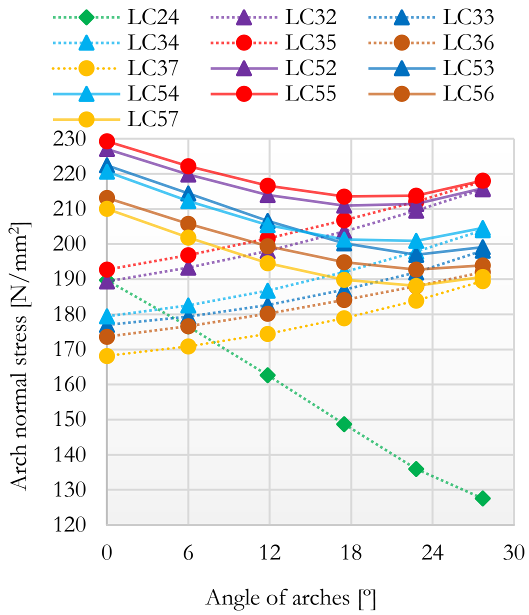


Figure 8. Influence of the angle of arches on normal stresses in the arch

The tie stress diagram shows a steady decrease for all the load cases (Figure 9). The values of the stresses are lowest for the angle of 28º for all the load cases.

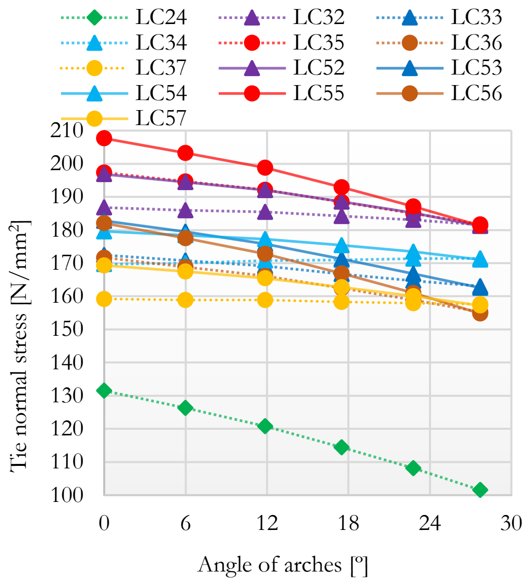


Figure 9. Influence of the angle of arches on normal stresses in the ties

3.2 Influence of the type of upper bracing

Four types of upper bracing (Vierendeel, K-truss, Double Warren and Cross type) were used in this analysis (Figure 10). Cross members are modelled as CHS sections D=1000mm, t=25mm in case of Vierendeel system and D=600 t=12.5mm for the rest of the truss systems. Diagonal members are modelled as CHS sections with D=250mm and t=10mm.

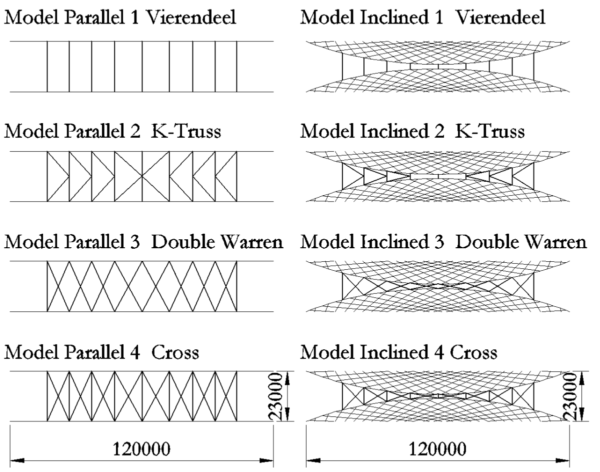


Figure 10. Models for the analysis of the influence of the type of bracing

No significant effect is produced in the axial forces in the hangers (difference below 3%) for different bracing systems even under the predominant lateral loading. The bracing component only introduces small additional axial forces in the chords. The analysis shows that the parallel bridge model has a higher dependence (4% change in axial force) on the bracing type than the inclined arch model (1% change in axial force). This is due to the fact that the inclined arch system is itself more resistant to the lateral forces than the parallel system. The optimum system in terms of the arch and tie axial forces is the Double Warren truss.

Change in the bracing system of the parallel arch model from the Vierendeel type to the truss type produces a decrease in the stresses in the range of 1.7% to 2.6% for the arch and 1.7% to 4.0% for the tie. In case of the models with inclined arches, changes are not significant.

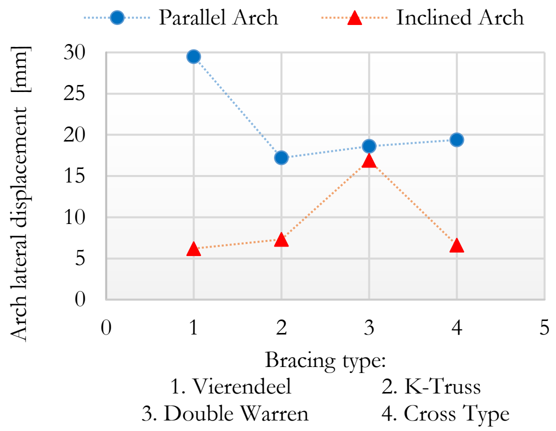


Figure 11. Top of arch lateral displacements (LC62)

All three types of the truss bracing produce acceptable lateral displacements under predominant lateral wind loads (ϱ<H/1000) for the parallel arches achieving the best results (ϱ<H/1160) with K-truss. Lateral displacements in the inclined arch model are several times smaller coming close to the values below H/2500 (ϱ<H/3200 for Cross type) (Figure 11).

3.3 Influence of the ratio f/L

Each one of the two model groups consist of four models with f/L ratios ranging from 1/5.5 to 1/7.0 in the increments of 0.5 (Figure 12).

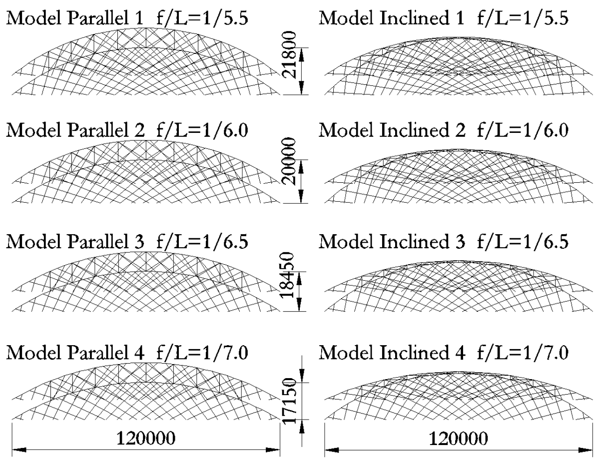


Figure 12. Models for the analysis of the influence of f/L ratio

The maximum hanger forces are obtained for the load cases with predominant traffic loading (LC52). In both model groups there is a decrease in the maximum hanger forces up to 10% with the change of f/L from 1/5.5 to 1/7.

For all load cases, the same f/L variation shows an increase in the arch and tie axial forces. The maximum arch axial forces are increased by 17.7% (parallel) and 20.2% (inclined) while the tie forces are increased by 21% in both groups.

For symmetric load cases (LC52), decreasing the height decreases the forces in the hangers which leads to lower concentrated forces in the arch and the tie and hence the lower vertical bending moments. For non-symmetric loads the higher bridge allows for the hangers to spread from one point in the tie to two points in the arch located on either side of the bridge. This is not possible for the low bridge heights where the two hangers are attached to the same side of the arch. Hence decreasing the bridge height increases the bending moments in non-symmetric load cases (LC53). (Figure 13).

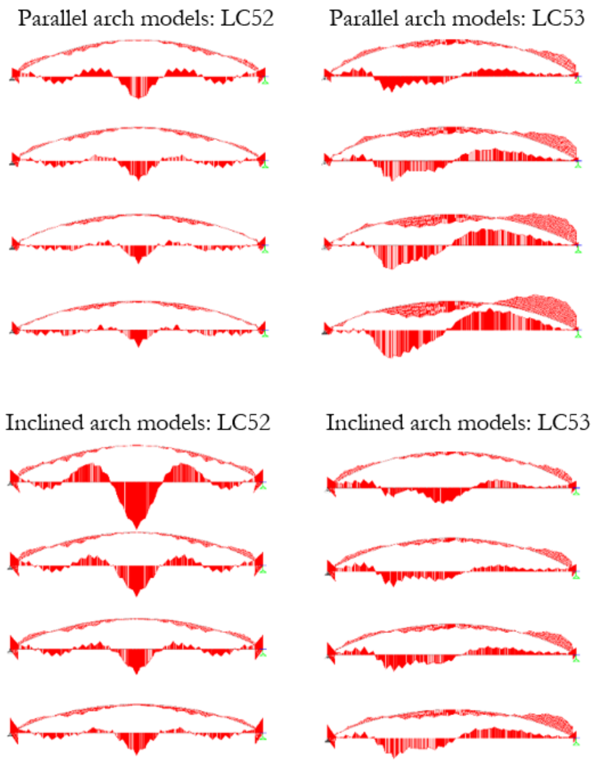


Figure 13. Influence of f/L ratio on vertical moments (f/L ratio from top to bottom of each model group: f/L=1/5.5, 1/6.0, 1/6.5 and 1/7.0): LC52 and LC53

The highest normal stresses are obtained for the LC52 in both systems. For the ratio f/L=1/6.5 both the inclined and the parallel arches have similar stresses. In this analysis, the parallel arches are more suitable for ratios 1/(f/L) above 6.5 and inclined arches for the ratios below this value. The tie stresses increase proportionally for both systems and are generally lower for inclined arches (Figure 14).

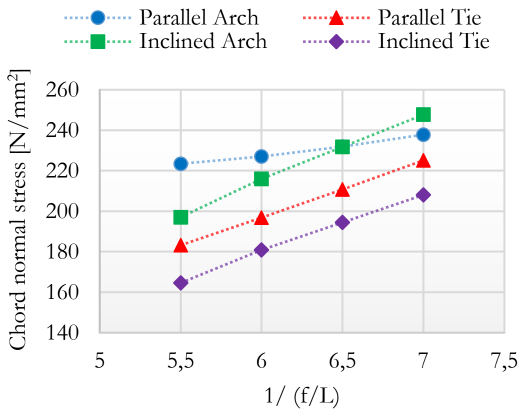


Figure 14. Influence of f/L ratio on normal stress in the arch and tie (LC52)

3.4 Influence of the number of hangers

Both model groups in this analysis are modelled with the following number of hangers: 2x14, 2x18, 2x22, and 2x26 (Figure 15).

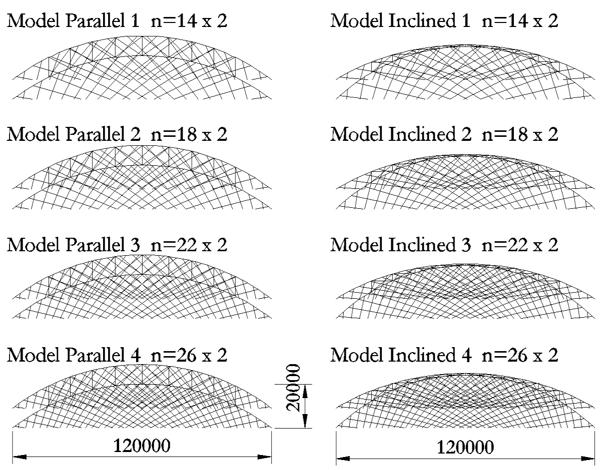


Figure 15. Models for the analysis of the influence of the number of hangers

The analysis shows that the maximum hanger forces are significantly reduced with the increase in number of hangers. This reduction is slightly more significant in the case of the inclined arches. The disadvantage of hanger increase is that certain hangers can lose the tension if the number of the hangers is higher than the 2x26 (Figure 16).

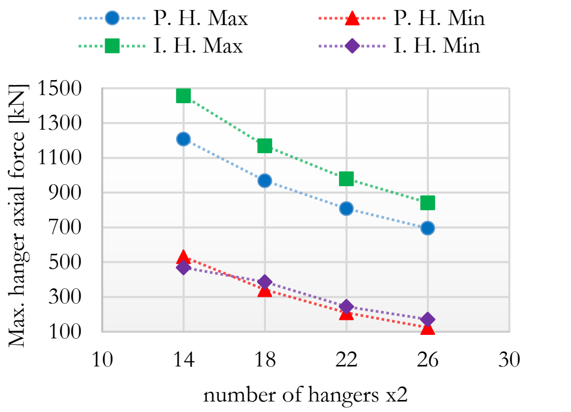


Figure 16. Influence of the number of hangers on maximum (H. Max) and minimum (H. Min) hanger forces (P-Parallel model; I-Inclined model)

The higher number of the hangers has limited effect on the axial forces in the chords (below 3%). However, it contributes to the higher rigidity of the whole hanger system causing the reduction in vertical bending moments in arches and ties in both model groups. This reduction is bigger in parallel arch system (tie average reduction 28.9% and arch average reduction 15.5%) than in inclined arch system (tie average reduction 14.4% and arch average increase 2.8%). The lateral bending moments are not significantly influenced by the number of hangers. There is a decrease in the combined stresses with the increase of the number of hangers. The rate of this change is almost constant, and any decrease is only seen for the change from 2x14 to 2x18 hangers.

3.5 Influence of the angle of hangers

The angle of the hangers in this analysis is taken as the angle between the arch and the hanger. It is varied from 40º-55º in steps of 5º for the parallel and inclined arch models (Figure 17).

The maximum forces in the hangers are reduced as they become more vertical from 40º to 55º. This reduction is more pronounced for inclined arch model (53%) than the parallel arch model (36%). Maximum hanger forces are lower in the parallel arch model for the angles 40º-50º, while at the angle 55º these forces become similar for both models and do not show further significant reduction (Figure 18).

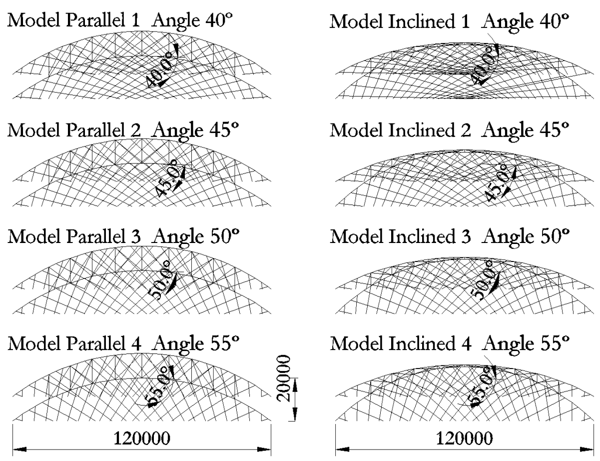


Figure 17. Models for the analysis of the influence of the angle of hangers

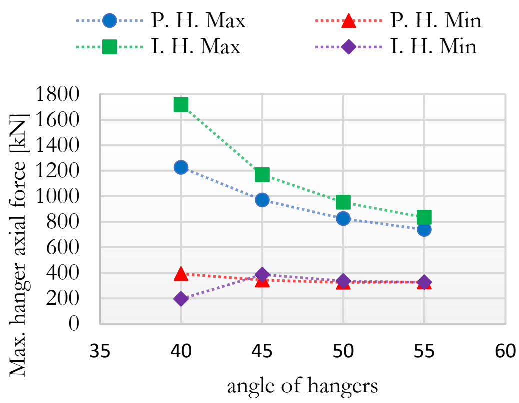


Figure 18. Influence of the angle of hangers on maximum (H. Max) and minimum (H. Min) hanger axial forces (LC52) (P-Parallel model; I-Inclined model)

The increase in the angle of hangers contributes to the reduction of arch axial force up to 4% in both model groups. The influence on the lateral bending moments is also not significant. The angle of the hangers has significant influence on the bending moments and plays an important role in the case of uneven loading. For both model groups, the change in the angle from 45º to 55º will cause the hangers to be more vertical which will decrease the moments for the loads distributed all over the span and will increase the moments for the loads distributed over the half-span. The optimum angles obtained are 45º for the parallel model and 50º for the inclined model (Figure 19).

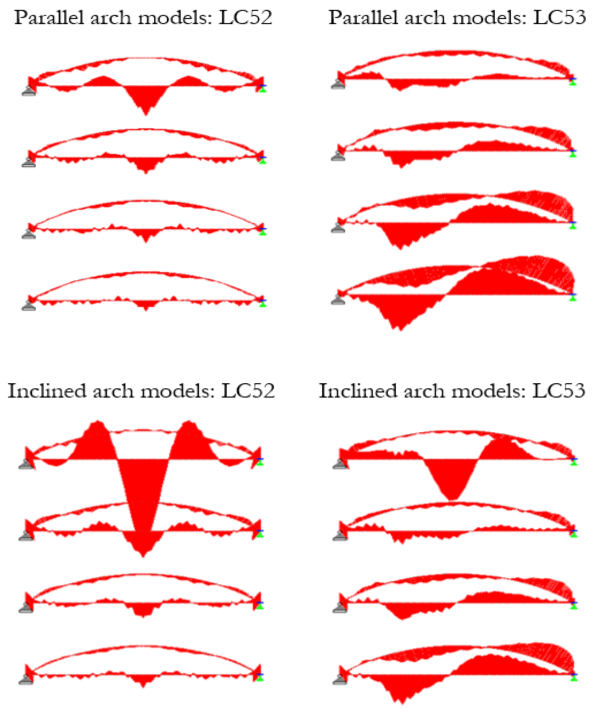


Figure 19: Influence of hanger-arch angle on chord vertical moments (hanger-arch angles from top to bottom of each model group: 40°, 45°, 50° and 55°)

4. Conclusions

A parametric analysis of structural behaviour of network arch bridges with inclined and parallel arches under vertical and lateral loading was described in this article.

The optimum angle between the arch and the vertical plane regarding the arch and tie stresses lays in the range of 18°-24°. Under combined traffic and wind loading with non-uniform distribution, lower stresses in the arch and tie are obtained for the inclined arch geometry. Lateral displacements in the inclined arch models are several times smaller than those of the parallel arch, with values below H/2500.

Inclined and parallel arches have similar stresses for the ratio f/L=1/6.5. Parallel arches are more suitable above and inclined arches below this ratio. Change in the f/L ratio from 1/5.5 to 1/7 increases the tie stresses in both systems. The optimum angle of hangers is 45° for parallel and 50° for inclined arch models while the reasonable number of hangers for both systems is 2x18 for the given geometry.

References

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