

BULETINUL INSTITUTULUI POLITEHNIC DIN IAȘI

Publicat de

Universitatea Tehnică „Gheorghe Asachi” din Iași

Volumul 67 (71), Numărul 4, 2021

Secția

CONSTRUCȚII. ARHITECTURĂ

DOI: 10.2478/bipca-2021-0034



INFLUENCE OF DESIGN PARAMETERS ON GENERAL STABILITY OF TIED ARCHES WITH AN UPPER BRACING

BY

ALEXANDRA D. DANCIU*, ȘTEFAN I. GUȚIU, CĂTĂLIN MOGA and
MIRCEA SUCIU

Technical University of Cluj-Napoca, Faculty of Civil Engineering

Received: February 16, 2022

Accepted for publication: September 15, 2022

Abstract. The paper shows the influence of some design parameters on the general stability of a single-track railway bridge, having the structure made of circular steel arches with rigid vertical hangers.

The following structural parameters were considered in this analysis: closed bridge with an upper winding made of horizontal members without links (diagonals) between them; closed bridge, with upper winding made of horizontal elements and ties made in the form of cross diagonals; static scheme of the arch in the transverse direction – hinged, fixed or portal frame. The analysis carried out is based on the normative SR EN 1993-2:2007. Eurocode 3: Design of steel structures. Part 2: Steel Bridges (EC3-2). The results obtained can be useful in the design of the arches structures for the field of bridges or other types of metal structures.

Keywords: circular steel arches; design factors; general stability; Euro-norm SR EN 1993-2:2007; worked analysis.

*Corresponding author: *e-mail*: Alexandra.DANCIU@cfdp.utcluj.ro

© 2021 Alexandra D. Danciu et al.

This is an open access article licensed under the Creative Commons Attribution-NonCommercial-NoDerivatives 4.0 International License (CC BY-NC-ND 4.0).

1. Introduction

Bridges on steel arches, in various constructive forms, are relatively widely used in the construction of road and railway bridges, offering economical solutions, simple from a constructive and mounting point of view, and can also provide an aesthetic and architectural aspect superior to other constructive solutions.

For a railway bridge with a span of 30 m, four constructive solutions for the steel superstructure were analysed (Moga *et al.*, 2022): bridge on steel plate beams, bridge on trusses, bridge on arches with vertical hangers with upper winding (closed bridge) and bridge on arches with vertical hangers without upper bracing (open of free-standing bridge), Fig. 1.

In all four constructive variants, Fig. 1, the railway track on the bridge is an open-type, with the deck made up of stringers located at a distance of 1800 mm inter-axes and cross-girders at a distance of 5000 mm, both beams of the deck being made up as steel plate girders, and the in-site joints are made with high-strength bolts.

In Fig. 1 the geometry of the structure is presented.

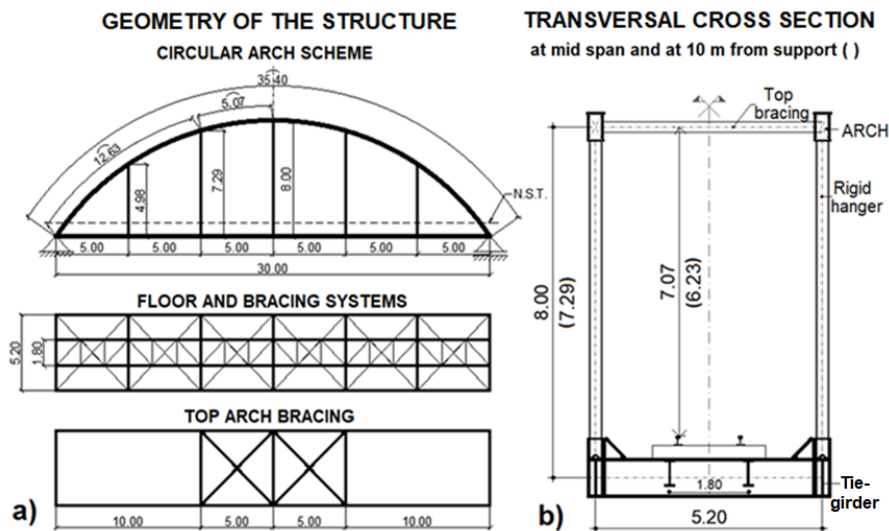


Fig. 1 – Tied arch: a) Geometry of the structure.
b) Transversal section considering the structure gauge.

This paper presents an analysis on the influence of some design parameters and structural conformation on the general stability of a railway bridge, having the superstructure carried out on circular metal arches with rigid vertical hangers, respectively:

- the influence of the stiffness of the upper bracing between the arches, in the case of closed bridges.

- the influence of the static arches scheme in the transverse direction: hinged, fixed arches or portal frame.

The analysis carried out is based on the normative SR EN 1993-2:2007. Eurocode 3: Design of steel structures. Part 2: Steel Bridges (EC3-2. Annex D), for the assessment of the buckling lengths and critical buckling forces, in the in-plane of the arch and in the out of plane of the arch and euro-norm (SR EN 1991-2.2005) for railway bridge actions.

Analyses and information on the influence of the geometric configuration of the hangers on the way the arches behave and take over loading are presented in the paper works (Danciu *et al.*, 2016; da Costa, 2013; Romeijn, 2006; Smit, 2013; Vlad *et al.*, 2015).

The rise of the arch adopted is $f = 8000\text{mm}$ that complies with the gauge condition. The clearance gauge is given in STAS 4392-84, and it was chosen for traffic speeds $100\text{km/h} < v < 160\text{ km/h}$, and the distance between the axis of the pillars of the contact line $L < 40\text{ m}$. The f/L value is: $1/3.75 = 0.27$, greater than that resulting from the optimal condition. The distance between the axis of the arches B results from the width of the clearance gauge, an inter-axes arches $B=5200\text{ mm}$ value is adopted.

2. Buckling length, critical forces

In the case of elements subjected mainly to compression, such as arches, the use of high-grade steels is not economical given that, by increasing the strength, the coefficient of slenderness increases, and the class of the cross-section tends towards a higher value. If the class cross-section reaches Class 4, the calculations will be operated with the area A_{eff} .

Following pre-design and subsequently verification at ultimate limit state (ULS) and serviceability limit state (SLS) (strength, stability, deformation and own vibration frequency), the arch is made of S235 steel, with a box cross-section as shown in Fig. 2.

Buckling lengths and critical forces for the in plane buckling of arches

According to EN 1993-2: 2005 (SR EN 1993-2: 2007), the critical buckling force for the in plane buckling of arches, N_{cr} and the buckling force are given by the equation:

$$N_{\text{cr},y} = \frac{\pi^2 \cdot E I_y}{(\beta \bar{s})^2} \quad (1)$$

And the critical buckling length by Eq. (2):

$$L_{cr,y} = \beta \cdot \bar{s} \quad (2)$$

where: $\bar{s}$ - is the half length of the arch; EI_y - is the flexural stiffness in the plane of the arch; β - is the coefficient of the buckling length in plane of the arch, Fig. 3.

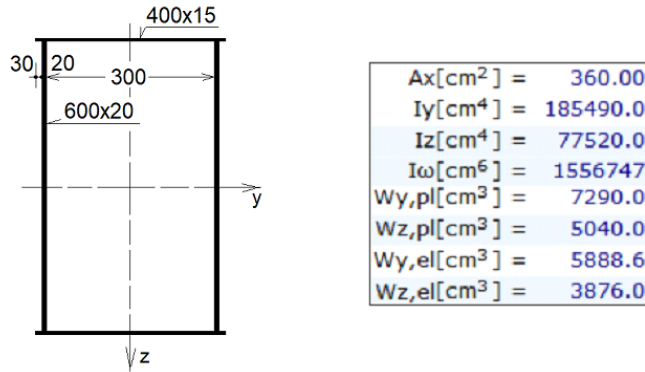


Fig. 2 – Arch cross-section and the design characteristics.

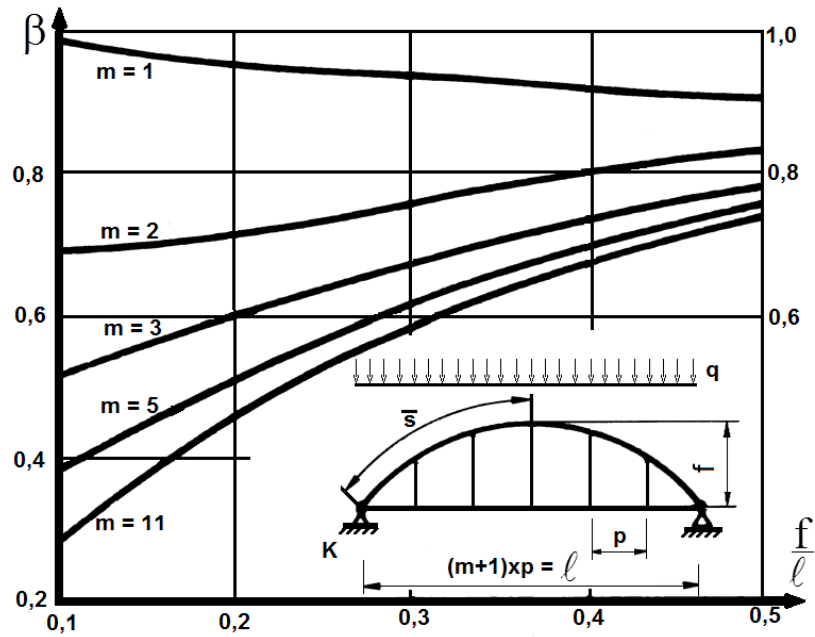


Fig. 3 – Buckling factor β .

In this case we have $5(m+1)=30$, $m=5$, $f/L=1/3.75=0.27$, and from Fig. 3 it is obtained $\beta=0.6$.

The buckling length in-plane of arch: $L_{cr,y} = \beta \times \bar{s} = 0.6 \times 1770 = 10.62\text{m}$.

The critical buckling force: $N_{cr,y} = \frac{\pi^2 \cdot E I_y}{(\beta \times \bar{s})^2} = 3.4 \times 10^6 \text{ daN}$.

The cross-section being of Class 1 shall be operated with the gross area of the section and will be obtained:

$$\bar{\lambda}_y = \sqrt{\frac{A \cdot f_y}{N_{cr,y}}} = \sqrt{\frac{360 \cdot 2350}{3.4 \cdot 10^6}} = 0.50 \Rightarrow \chi_y = 0.88 \text{ (curve b of buckling)}.$$

Buckling lengths and critical forces for the out of plane buckling of arches

In the out of plan of the arch, according to EC3-2, Annex D, the critical buckling force, in the case of arch systems fitted with upper bracing or end frames (portal frames), the critical force shall be determined with the equation:

$$N_{cr,z} = \frac{\pi^2 \cdot E I_z}{(\beta \cdot h)^2} \quad (3)$$

The critical buckling length for the out of plane of arch is:

$$L_{cr,z} = \beta \cdot h \quad (4)$$

where: β - is the coefficient of the buckling length in plane of the arch, h is taken according to Fig. 4 (EC3-2).

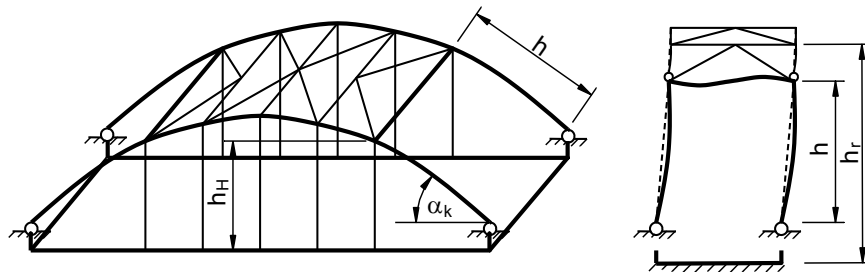


Fig. 4 – Geometrical buckling characteristics of the final portals for arches.

With the geometrical data of the arch shown in Fig. 5, the geometric design parameters are evaluated:

$$h = \frac{L_{T2}}{\sin \alpha_k} = \frac{729}{\sin \alpha_k} [\text{cm}]; h_r = \frac{(L_{T2} + L_{T3})/2}{\sin \alpha_k} = \frac{(729 + 800)/2}{\sin \alpha_k} = \frac{765}{\sin \alpha_k} [\text{cm}]; \frac{h}{h_r} = \frac{729}{765} = 0.95$$

The upper bracing is made of the following members:

- horizontal members made of circular hollow sections (CHS):

$D \cdot t = 245 \times 10$, with $I = I_0 = 4187 \text{ cm}^4$, $A = 59.6 \text{ cm}^2$, $i = 8.38 \text{ cm}$, $\lambda = 520/8.38 = 62$.

- cross diagonals made of circular hollow section (CHS):
 $D \cdot t = 140 \times 8$, with $A = 33.2 \text{ cm}^2$, $I = 725 \text{ cm}^4$, $i = 4.67 \text{ cm}$, $\lambda = 721/4.67 = 154$.
 where i is the radius of gyration.

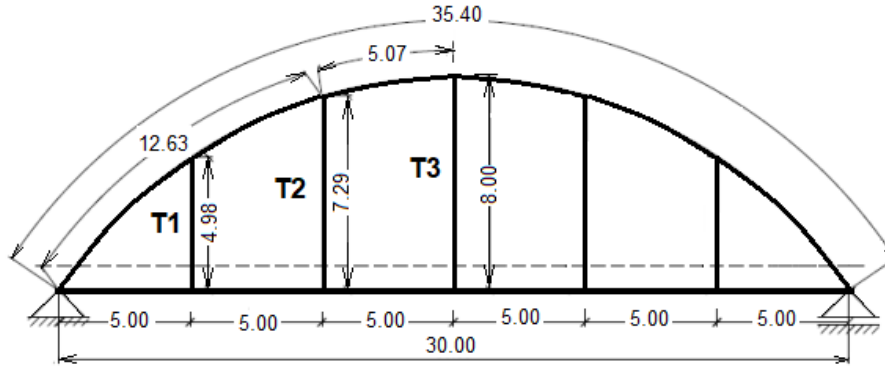


Fig. 5 – Geometrical arch design characteristics.

Considering the stiffness of the bracing given by the horizontal CHS (top strut) it is obtained: $\eta = \frac{I \times b}{I_0 \times h} = \frac{77520 \times 520}{4187 \times 1238} = 7.78$.

Taking for I_0 evaluation two CHS working together due to diagonals, it results $I_0 \approx 2 \cdot A_{CV} \cdot d^2 = 2 \cdot 59.6 \cdot 252^2 = 7.57 \cdot 10^6 \text{ cm}^4$ and it is obtained $\eta_0 \approx 0$.

Case 1: Hinged arch for the out of plane of arch

In this case, the diagrams from Fig. 6 are used to evaluate the value of β factor.

For $h/h_r = 0.95$ and $\eta = 7.78$ it is obtained $\beta = 1.4$, respectively $\beta \cdot h = 1.4 \cdot 1238 = 1733 \text{ cm}$.

For $h/h_r = 0.95$ and $\eta \approx 0$ it is obtained $\beta = 1.2$, respectively $\beta \cdot h = 1.2 \cdot 1238 = 1486 \text{ cm}$.

Case 2: Fixed arch for the out of plane of arch

For $h/h_r = 0.95$ and $\eta = 7.78$, from Fig. 7 it is obtained $\beta = 0.8$, respectively $\beta \cdot h = 0.8 \cdot 1238 = 990 \text{ cm}$.

For $\eta \approx 0$ it is obtained $\beta = 0.6$, respectively $\beta \cdot h = 0.6 \cdot 1238 = 743 \text{ cm}$.

Case 3: Closed end frame (end portal)

In this case, the diagrams from Fig. 8 are used to evaluate the value of β factor.

Fig. 8 shows that the stiffness of the bracing and of the end cross-girder are considered equally, which constitutes a particular case.

For $\eta=7.78$ it is obtained $\beta \approx 0.9$ and results $\beta \times h = 0.9 \times 1238 = 1114$ cm.

For $\eta=0$ it is obtained $\beta \approx 0.55$ and results $\beta \times h = 0.55 \times 1238 = 681$ cm.

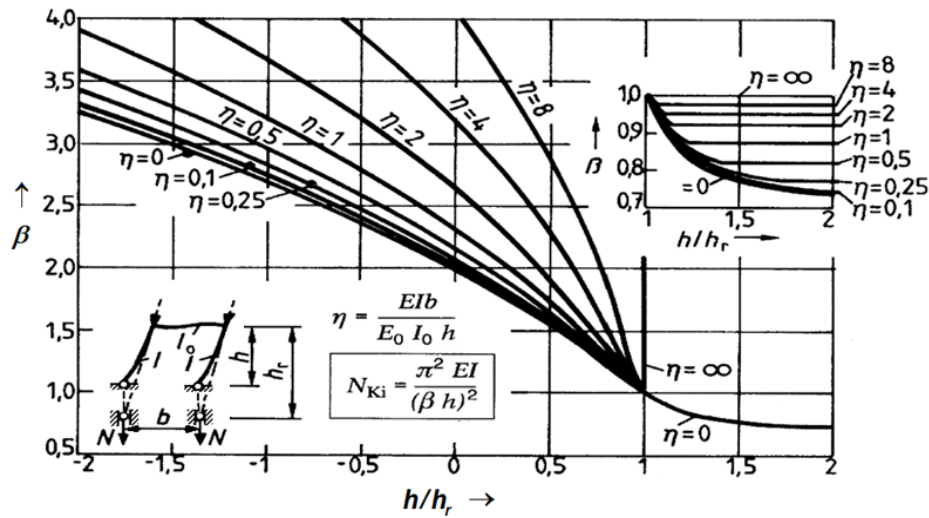


Fig. 6 – Buckling length factor β - hinged arches.

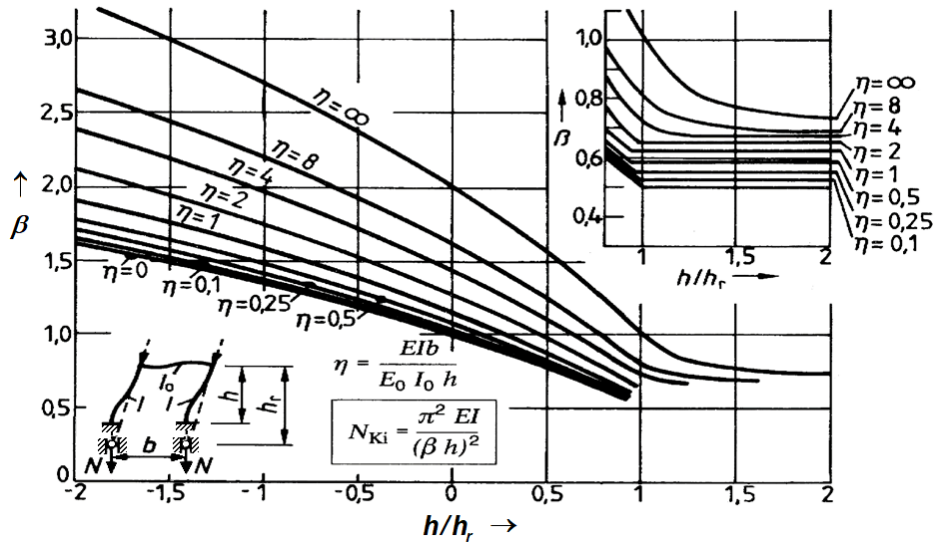


Fig. 7 – Buckling length factor β - fixed arches.

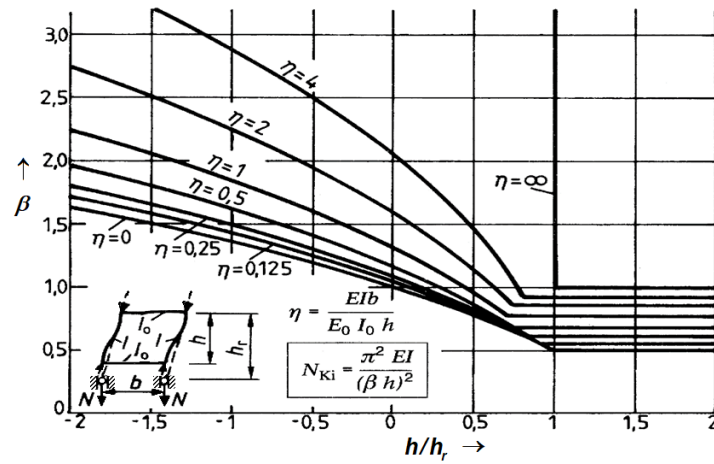
Fig. 8 – Diagrams of buckling length factor β - final portals.

Table 1 summarizes the values of buckling lengths in the in plane of the arch and in the out of plane for the cases analyzed.

Table 1
Buckling length of arch

In-plane buckling		
Tied-arch with vertically shaped hangers		$L_{cr,y}=10.62 \text{ m}$
Out-of-plane buckling		
Type of arch support	Stiffening parameter η	$L_{cr,z} \text{ [m]}$
Two-hinge arch	$\eta=7.78$	17.33
	$\eta=0$	14.86
Fixed arches at supports	$\eta=7.78$	9.90
	$\eta=0$	7.43
Arches with end portals	$\eta=7.78$	11.14
	$\eta=0$	6.80

In the actual design situations, the arch supporting in the transverse direction is an elastic fixed in the end cross-girder, and in the pre-design phase, the buckling length for the out of plane of the arch, can be considered equal to the length of the arch between the support and the first horizontal bracing (top strut), $\beta \cdot h \approx 12.63 \text{ m}$. Using advanced computing programs, a more accurate solution can be achieved.

For $L_{cr,z} = \beta \cdot h \approx 12.63 \text{ m}$ it is obtained:

$$N_{cr,z} = \frac{\pi^2 \times E \times I_z}{(\beta \times h)^2} = \frac{\pi^2 \times 2.1 \times 10^6 \times 77520}{(1263)^2} = 1.0 \times 10^6 \text{ daN}$$

$$\bar{\lambda}_z = \sqrt{\frac{A \times f_y}{N_{cr,z}}} = \sqrt{\frac{360 \times 2350}{1.0 \times 10^6}} = 0.92 \Rightarrow \chi_z = 0.65 \text{ (curve b of buckling).}$$

3. Aspects regarding the structure design

The structure of the steel deck was checked using a spatial calculation program, SAP2000, combined with an analytical assessment for the direct effect of the wind action and of the nosing force, taken over by the main horizontal bracing.

In assessing the calculation actions, account was taken of: the safety coefficient $\gamma_g=1.35$ - for permanent actions, $\gamma_{Q1}=1.45$ - for the LM71 convoy actions, $\gamma_w=1.5$ - for wind action, $\Psi_{0,w}=0.75$ - the coefficient of combination for wind action and $\Phi=\Phi_3=1.14$ - the dynamic coefficient. For loading group 11, given in (SR EN 1991-2:2005), the traction force and the braking force are considered with their integral characteristic values, while the nosing force is reduced by 50%.

The permanent design load, applied simplified to each stringer, was evaluated: $g_{Ed,g}=0.5 \cdot \gamma_g \cdot g_{perm}=0.5 \cdot 1.35 \cdot 30 \approx 20 \text{ kN/m}^2$.

The uniformly distributed loads and the vertical concentrated forces by the action of the LM71 convoy, for each stringer:

$$Q_{Ed}^{LM71}=0.5 \times \gamma_{Q1} \times \Phi_3 \times (LM71)=0.5 \times 1.45 \times 1.14 \times (LM71)=0.827 \times (LM71) .$$

The LM71 loading model given by SR EN 1991-2 (SR EN 1991-2:2005) and the loading convoy applied in this case, for each stringer, are shown in Fig. 9.

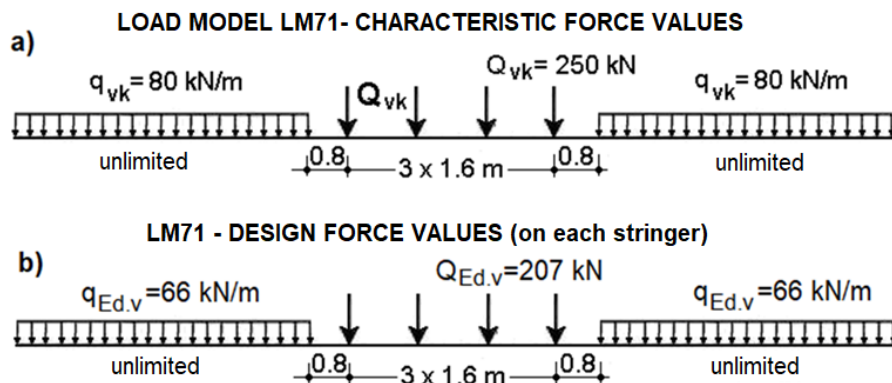


Fig. 9 – Load model LM71 and the design loading on each stringer.

The vertical actions of the convoy and indirect actions of the wind and the nosing force will be taken over by the tied-arch, Fig. 10.

DIRECT AND INDIRECT DESIGN ACTIONS

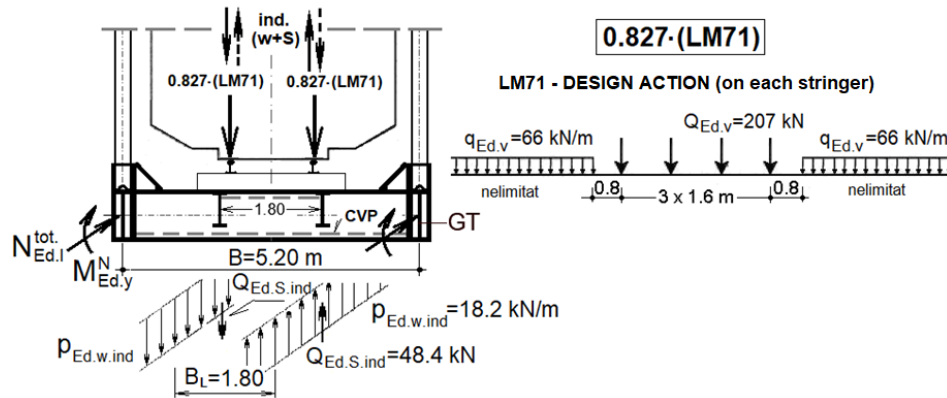


Fig. 10 – Vertical traffic actions and indirect wind and nosing actions.

The structure of the deck was analyzed for two loading cases (or hypotheses):

- Case 1: Loading over all the span (full loading), Fig. 11;
- Case 2: Loading over half the span of the bridge, Fig. 12.

CASE 1 OF LOADING

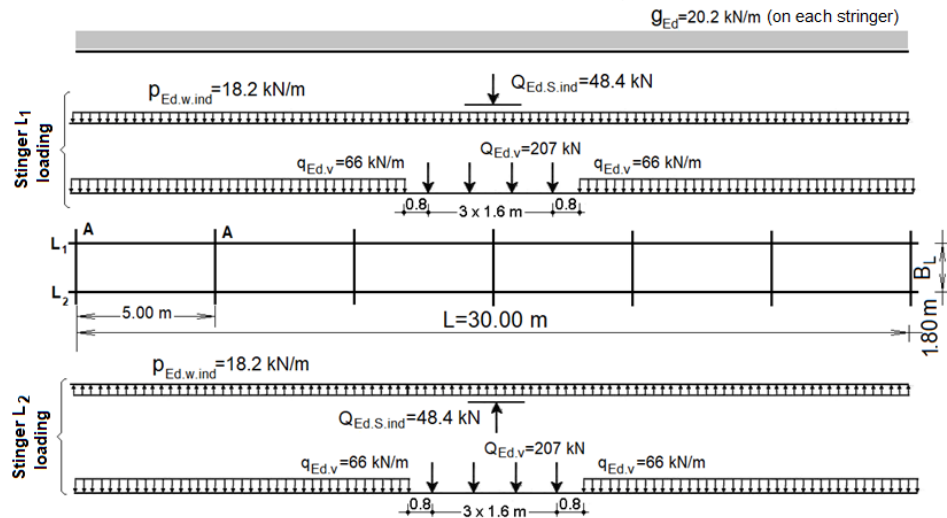


Fig. 11 – Loading Case 1.

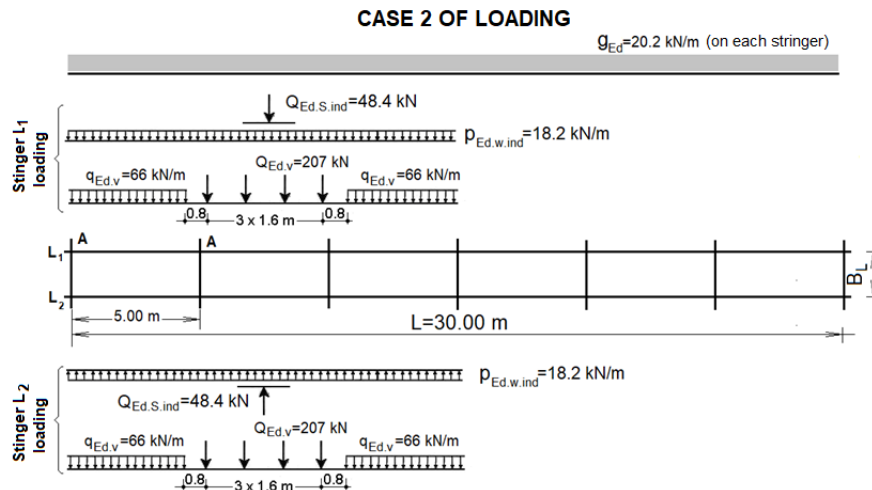
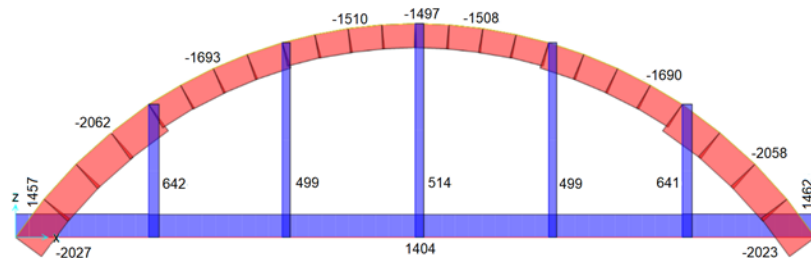


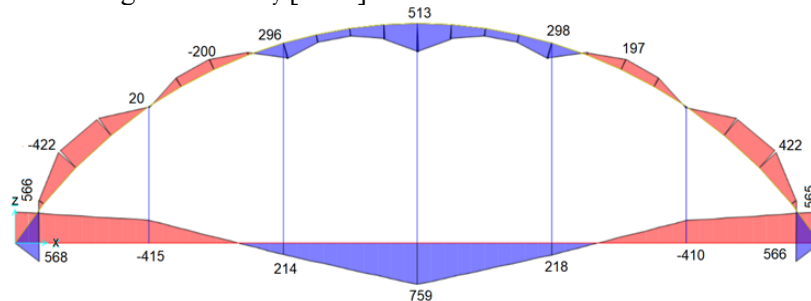
Fig. 12 – Loading Case 2.

Diagrams of axial forces and bending moments for the two loading cases (hypotheses) from the permanent actions, vertical actions of the LM71 convoy and from the indirect wind action and nosing, for the most loaded tied-arch system, are shown in Fig. 13 – Loading Case 1 and Fig. 14 – Loading Case 2.

Axial force N [kN]



Bending moment M_y [kNm]

Fig. 13 – Diagrams N and M_y – Loading Case 1.

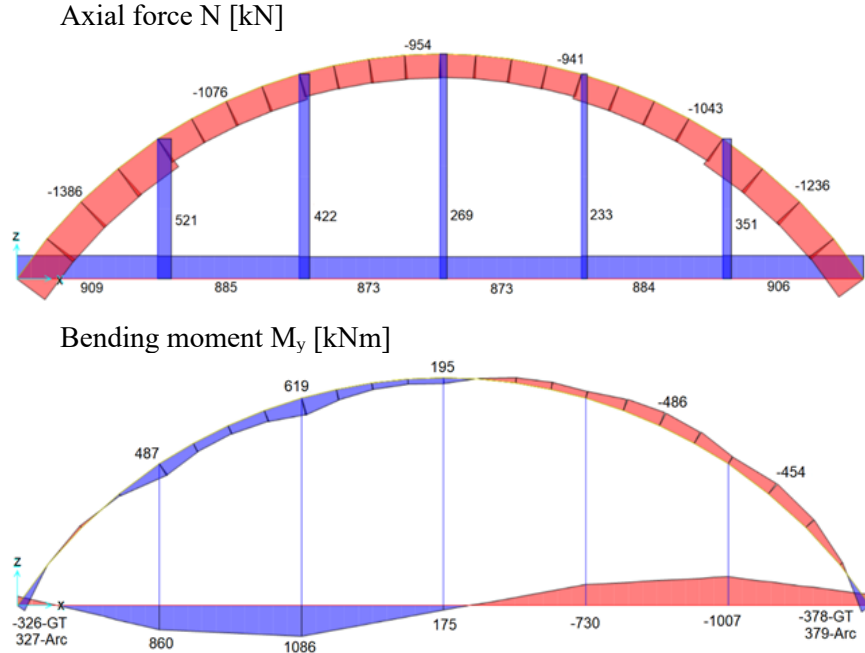


Fig. 14 – Diagrams N and My – Loading Case 2.

4. Compression and bending check of the arch cross-section

According to SR EN 1993-2, § 6.3.3, a simplified equation may be used for the bending and compression check of a steel member:

$$\frac{N_{Ed}}{\chi_y N_{Rk}} + C_{mi,0} \frac{M_{y,Ed} + \Delta M_{y,Ed}}{M_{y,Rk}} \leq 0.9 \quad (5)$$

$\gamma_{M1} \qquad \qquad \gamma_{M1}$

In the case of members with a uniform section, falling within cross-section classes 1, 2 or 3, subjected to axial compression and monoaxial or biaxial bending, the verification Eq. (5) may be put in the following simplified forms given in Eq. (6) or Eq. (7):

$$\frac{N_{Ed}}{\chi_{min} N_{Rk}} + C_{my,0} \frac{M_{y,Ed}}{M_{y,Rk}} \leq 0.9 \quad \text{- monoaxial bending} \quad (6)$$

$\gamma_{M1} \qquad \qquad \gamma_{M1}$

$$\frac{N_{Ed}}{\chi_{min} N_{Rk}} + C_{my,0} \frac{M_{y,Ed}}{M_{y,Rk}} + C_{mz,0} \frac{M_{z,Ed}}{M_{z,Rk}} \leq 0.9 \quad \text{- biaxial bending} \quad (7)$$

$\gamma_{M1} \qquad \qquad \gamma_{M1} \qquad \qquad \gamma_{M1}$

where:

N_{Ed} – is the design value of the compression force;

N_{Rk} – characteristic value of compression capacity;

$M_{y,Ed}$ – is the design value of the maximum bending moment about the y-y axis, obtained by an analysis of first order, without taking into account imperfections;

$M_{z,Ed}$ – is the design value of the maximum bending moment about the z-z axis, obtained by an analysis of first order, without taking into account imperfections;

$M_{y,Rk}$ – characteristic value of bending moment capacity;

$M_{z,Rk}$ – characteristic value of flexural moment capacity;

$C_{mi,0}$ – is the equivalent bending moment factor as set out in Annex A (SR EN 1993-1-1). It is denoted as $C_{my,0}$ or $C_{mz,0}$, depending of the axis the bending occurs.

γ_{M1} – partial safety coefficient for buckling resistance;

$\chi_{min} \cdot \chi_y$ - the reduction coefficient.

The cross-section of the arch being Class 1 shall be operated with the gross area of the cross-section and with the plastic section modulus, the capacity of the cross-section under axial compression and bending moment being the following:

$$N_{Rk} / \gamma_{M1} = A \times f_y / \gamma_{M1} = 360 \times 2350 / 1.1 = 769 \times 10^3 \text{ daN}$$

$$M_{y,Rk} / \gamma_{M1} = W_{y,pl} \times f_y / \gamma_{M1} = 7290 \times 2350 / 1.1 = 1557 \times 10^4 \text{ daN} \times \text{cm} = 1557 \text{ kN} \times \text{m}$$

$$M_{z,Rk} / \gamma_{M1} = W_{z,pl} \times f_y / \gamma_{M1} = 5040 \times 2350 / 1.1 = 1077 \times 10^4 \text{ daN} \times \text{cm} = 1077 \text{ kN} \times \text{m}$$

The most unfavorable check resulted from load hypothesis 1, considering the maximum bending moment combined with the axial force, and the maximum axial force combined with the bending moment, leads to:

$$\frac{N_{Ed}}{\chi_{min} N_{Rk} \gamma_{M1}} + C_{my,0} \frac{M_{y,Ed}}{M_{y,Rk} \gamma_{M1}} + C_{mz,0} \frac{M_{z,Ed}}{M_{z,Rk} \gamma_{M1}} = 0.72 < 0.9.$$

For other ULS and SLS checks (deformations, dynamic behavior), as well as for the details design, the works (Moga *et al.*, 2018) and (Moga and Guțiu, 2020) were used.

5. Conclusions

The stability check of steel arches is an important technical problem, and in the technical literature there are many papers, but some of them are more difficult to apply in the current design work.

Euro-norm SR EN 1993-2:2007. Eurocode 3: Design of steel structures. Part 2: Steel Bridges (EC3-2), presents equations and diagrams with which critical buckling forces can be assessed for the in-plane arch and for the out of plane arch.

For the in-plane of arch, the design data are presented for the arches with vertical hangers and for those without hangers, with rigid supports. The static systems taken into considerations are double-hinged arch, double fixed arch, three-hinged arch and double fixed arch with a hinge at the top (crown). For other hanger's configuration systems (tilts, multiple system, etc.), no design information is presented in EC3-2.

For the out of plane of the arch, the evaluation of critical forces for upper-winding arches is presented for three types of transversally supported arches: hinged arches, fixed arches and arches with end portal frames (for the particular case where the stiffness of the top strut is equal to that of the end cross-girder).

The paper presents an analysis on the influence of some design parameters and structural conformation on the general stability of a 30 meters long closed railway bridge, having the superstructure made on circular steel arches with rigid vertical hangers.

The analysis carried out can be useful in the design of arch structures for the field of bridges or other types of metal structures.

REFERENCES

- da Costa, Bernardo Moraira, *Design and Analysis of a Network Arch Bridge*, Ms Degree, UT Lisboa, 2013.
- Danciu A.D., Guțiu Șt., Moga C., *Bi-dimensional Analysis of A 90 M Arch with Different Hanger Arrangements*, Sofia: Surveying Geology & Mining Ecology Management (SGEM), 2016.
- Moga P., Guțiu Șt., Danciu A.D., Moga C., *Poduri metalice. Manual practice de proiectare: structura de pod*, UTPress, 2022 (in Romanian).
- Moga P., Guțiu Șt., Moga C., *Construcții și poduri metalice. Bazele proiectării elementelor din oțel*, UTPress, 2018 (in Romanian).
- Moga P., Guțiu Șt., *Poduri metalice. Conformarea suprastructurilor*, UTPress, 2020 (in Romanian).
- Romeijn A., *Steel bridges*, Course CT 5125, TUDelft, 2006.
- Smit T.J.M., *Design and construction of a railway bridge with a network hanger arrangement*, TUDelft, 2013.
- Vlad M., Kollo G., Marusceac V., *A modern approach to tied-arch bridge analysis and design*, Acta Technica Corviniensis – Bulletin of Engineering, Vol. 8, No. 4, 2015
- * *SR EN 1991-2/2005. Eurocode 1: Acțiuni asupra structurilor. Partea 2: Acțiuni din trafic la poduri* (in Romanian).
- ** *SR EN 1993-2/2007. Eurocode 3: Proiectarea structurilor de oțel. Partea 1-1: Reguli generale și reguli pentru clădiri* (in Romanian).

* *SR EN 1993-2/2007. Eurocode 3: Proiectarea structurilor de oțel. Partea 2: Poduri de oțel (in Romanian).*

* *STAS 4392-1-1/2006: Căi ferate normale. Gabarite (in Romanian).*

INFLUENȚA PARAMETRILOR DE PROIECTARE ASUPRA STABILITĂȚII GENERALE A PODURILOR PE ARCE METALICE CU CONTRAVÂNTUIRE LA PARTEA SUPERIOARĂ

(Rezumat)

În lucrare se prezintă influența unor parametri de proiectare asupra stabilității generale a unui pod de cale ferată simplă, având structura de rezistență realizată pe arce metalice circulare cu tiranți verticali rigizi.

La efectuarea analizei se au în vedere următorii parametri de alcătuire structurală: închis, cu contravântuire superioară realizată din bare orizontale fără legături (diagonale) între bare; pod închis, cu contravântuire superioară realizată din bare orizontale și legături realizate sub formă de diagonale încrucișate; schema statică de rezemare a arcului în sens transversal – articulată, încastrată sau de tip cadru portal.

Analiza efectuată are la bază normativul SR EN 1993-2:2007. Eurocod 3: Proiectarea structurilor de oțel. Partea 2: Poduri de oțel (EC3-2). Rezultatele obținute pot fi utile în activitatea de proiectare a unor structuri pe arce pentru domeniul podurilor sau ale altor tipuri de structuri metalice.

