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ASPECTS REGARDING EC3-2 PROCEDURE FOR STABILITY VERIFICATION OF THE FREE-STANDING CIRCULAR ARCHES

BY

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Abstract. The paper presents aspects related to the verification of the general stability of the bridge superstructures on circular arches with rigid vertical hangers, bridges of open-type or free-standing, without upper bracing.

A numerical worked analysis is carried out for a single-track railway bridge with a span of 30 m, which is based on the normative SR EN 1993-2:2007. Eurocode 3: Design of steel structures. Part 2: Steel Bridges (EC3-2).

The worked analysis carried out can be useful in deepening the knowledge of the procedure presented in the normative, as well as in the designing of the structures on arches in the field of bridges or other types of metal structures.

Keywords: circular steel arches, rigid hangers, calculation parameters, general stability, Euronorm SR EN 1993-2:2007

1. Introduction

Bridges on metal arches are widely used in the construction of road and railway bridges, offering economical solutions, simple from the point of view of

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construction and assembly and can also provide an aesthetic and architectural aspect often superior to other constructive solutions.

In the work (Moga P. et. all, 2020) four constructive solutions were analysed for the steel superstructure of a railway bridge with a span of 30 m: bridge on steel plate beams, bridge on trusses, bridge on circular arches with vertical hangers with upper bracing (closed bridge) and bridge on circular arches with vertical hangers without upper bracing (open bridge or free-standing arches).

For all four superstructures, the bridge's decking is identical (i.e. stringers, cross-girders and the bracing system), with the exception of the distance between the main elements, which is B=5.50 m for the bridge on steel plate girders and with the free-standing arches, and B=5.20 m, for the trusses bridge and for the bridge on the arches with upper bracing.

This difference resulted from the condition of compliance with the clearance gauge, but the cross-section of the deck elements remained the same.

This paper presents an analysis on the verification of the general stability of the steel deck on open arches, i.e., the stability of the arches in the vertical plane and in the out of plane, the calculation being carried out in accordance with the norm SR EN 1993-2:2007. Eurocode 3: Design of steel structures. Part 2: Steel Bridges (EC3-2).

2. Buckling lengths, critical forces

2.1. 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, Ncr is given by the equation:

$$N_{cr.y} = \frac{\pi^2 \cdot E I_y}{\left(\beta \times \bar{s}\right)^2} \tag{1}$$

where: 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. 1.

2.2. Influence of rigid hangers' configuration on critical buckling force for the in-plane of arch buckling

In some cases, the vertical configuration of the rigid hangers (used in most cases), Fig. 2.(a) may be modified as inclined hangers, obtained by evenly dividing the distances between the joints on the arch and those on the beam, resulting in a slight tilt of them, Fig. 2.(b), or radially arranged hangers, perpendicularly to the arch tangent at equidistant points on the arch, Fig. 2.(c), may be used.

By arranging the hangers in a modified configuration from the vertical one, a more uniform distribution of axial and bending efforts on the arches results and, at the same time, increases the critical buckling force in the arch plane.

Processing some data from the work (Romeijn A., 2006), which shows that the critically distributed load is proportional to a coefficient, given in the paper, it is proposed to introduce a coefficient, which we call the correction coefficient (or of configuration), with which the value of the critical buckling force obtained with the equation (1) and Fig. 1 is multiplied, for a configuration of type (b) or (c) hangers, and results are as follows:

$$N_{cr.y}^{b);c)} = \frac{\pi^2 \cdot E I_y}{\left(\beta \cdot \bar{s}\right)^2} \tag{2}$$

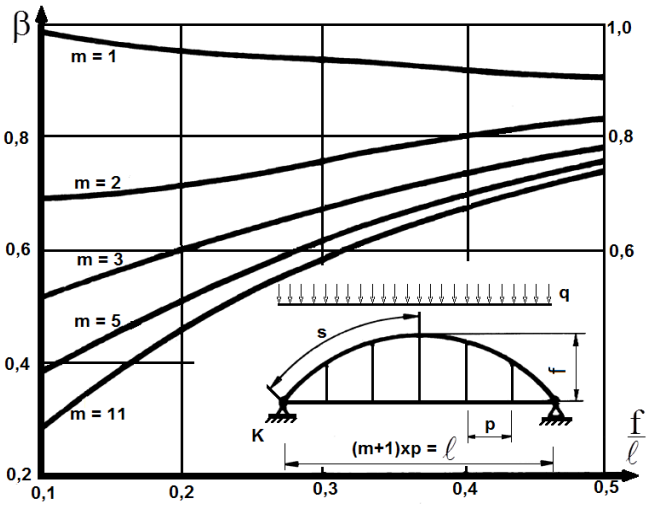


Fig. 1 − Buckling factor β.

The numerical values of the coefficient are given in Table 1.

Table 1.

Values of parameter Kc

Number of hangers	Hangers configuration	f/L			
		0.1	**0.2**	**0.3**	**0.4**
3	**a)**	1.00	1.00	1.00	1.00
	b)	1.10	1.33	1.49	1.46
	c)	1.15	1.52	2.00	1.98
5	**a)**	1.00	1.00	1.00	1.00
	b)	1.19	1.52	1.52	1.44

Number of hangers	Hangers configuration	f/L			
		0.1	0.2	0.3	0.4
	c)	1.40	2.39	2.39	2.60
11	a)	1.00	1.00	1.00	1.00
	b)	1.45	1.74	1.57	1.45
	c)	2.66	5.26	4.25	3.62

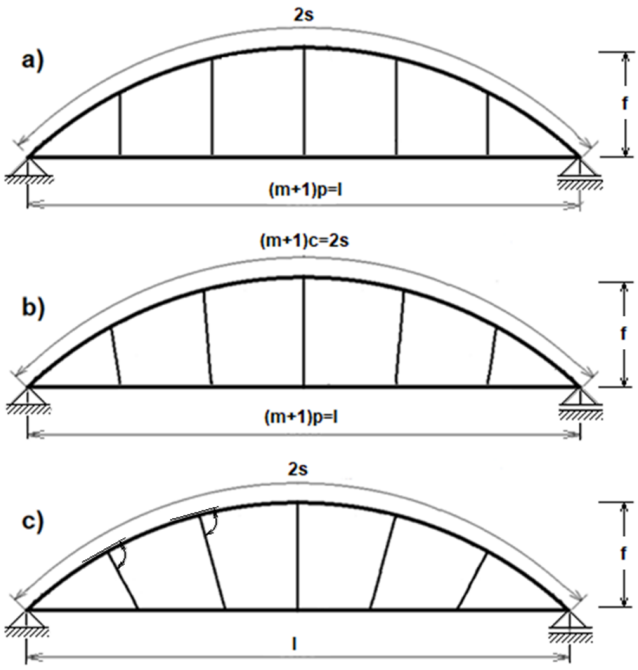


Fig. 2 − Hanger configurations.

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

For the arches with vertical hangers, the critical buckling force in the out of plane buckling of the arch is given by the equation:

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

where: l – is the span of the arch, EI_z – the out of plane flexural stiffness, β - is the coefficient of the buckling length.

In the out of plane of non-bracing arches according to EC3-2, the coefficient of buckling length is calculated with the equation:

$$\beta=\beta_1\cdot\beta_2 \quad (4)$$

The coefficients β_1 and β_2 shall be taken in accordance with Table 2 and Table 3, taken from the normative EC3-2.

Table 2.

Factor β_1 values

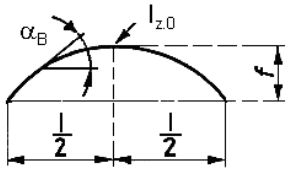
	Factor β_1						
1	f/l	0.05	0.10	0.20	0.30	0.40	
2	I_z - ct.	0.50	0.54	0.65	0.82	1.07	
3	$I_z=\frac{I_{z.0}}{\cos\alpha_B}$	0.50	0.52	0.59	0.71	0.86	

Table 3.

Factor β_2 values

	Loading mode	**β_2**	**Comments**
1	conservative (the deck is fixed to the top of the arch)	1	q – total load
2	by hangers	$1-0.35\frac{q_H}{q}$	q_H – load part transmitted by hangers
3	by posts	$1+0.45\frac{q_{st}}{q}$	q_{St} – load part transmitted by posts

For the evaluation of the coefficient β_2, EC3-2 does not specify how the rate q_H/q is assessed or evaluated. For the evaluation of this rate, it is hypothesized that the deflection of the arch for the antisymmetric loading, which produces the maximum deflection and bending moment at L/4 of the span, Fig. 3, is approximately equal to the deflection of the tie-beam (Romenijn

A., 2006), or advanced numerical methods of calculation can be used, (da Costa, 2013, Smit T.J.M., 2013):

$$\text{Arch deflection: } \delta_{arc} \approx \frac{5}{384} \frac{q_H (L/2)^4}{E I_{arc}} \tag{5}$$

$$\text{Stiffening girder deflection: } \delta_G \approx \frac{5}{384} \frac{(q-q_H)(L/2)^4}{E I_G} \tag{6}$$

Equalizing the two deflections it is obtained [3]:

$$q_H = \frac{q}{1+\frac{E \cdot I_G}{E \cdot I_{arc}}} \tag{7}$$

In the paper (da Costa, 2013) it is considered that the entire load is taken over by the arch, respectively the ratio $q_H/q=1$, and therefore the coefficient $\beta_2=0.65$. This hypothesis is not always conservative because it results in a shorter buckling length, i.e., a higher critical buckling force.

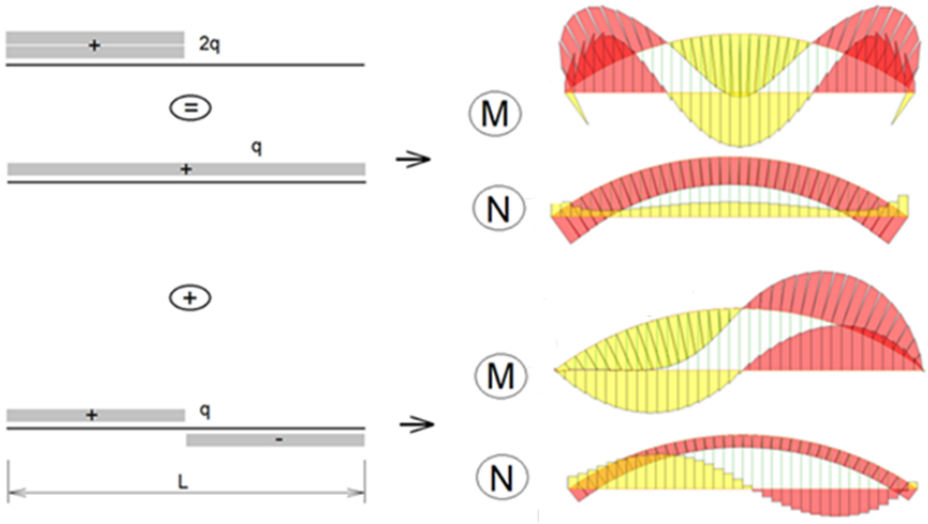


Fig. 3 − Schemes to evaluate the rate q_H/q.

3. Worked case study

3.1. Analysed structure

The following is an application for the evaluation of the parameters for checking the general stability of a steel deck on free-standing arches, the calculation being carried out in accordance with the normative SR EN 1993-2:2007. Eurocode 3: Design of steel structures. Part 2: Steel Bridges (EC3-2).

In the case of the open bridge (without upper bracing), the rise of the arch results from the optimization conditions, so for a ratio f/L = 1/5 = 0.2, the adopted rise of the arch is f=6000 mm, and the distance chosen between the axes of the arches, to ensure the gauge width (STAS 4392-84), is B=5500mm.

The geometry of the open bridge superstructure is presented in Fig. 4.

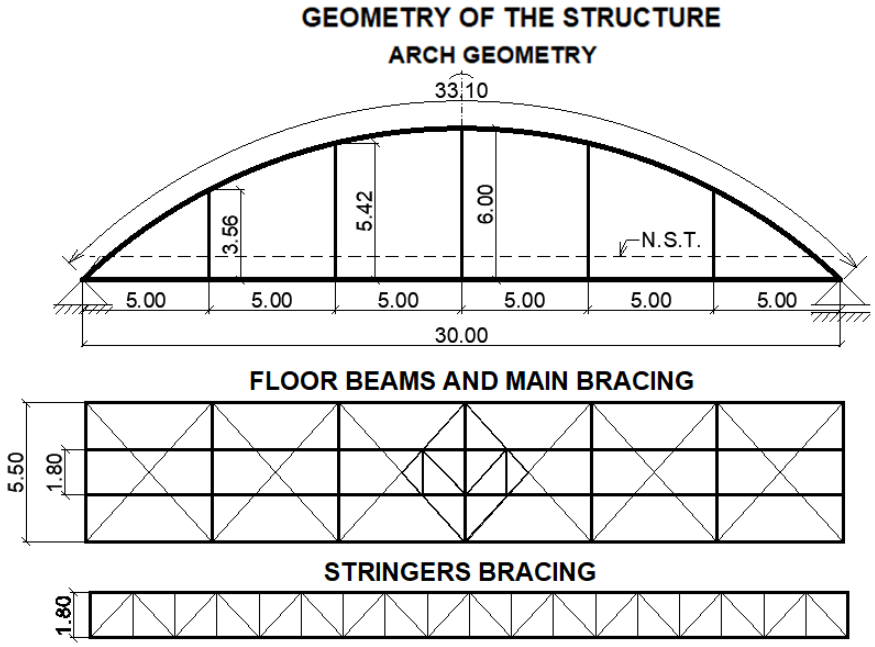


Fig. 4 − The geometry of the structure.

The deck cross-section and the cross-sections of the deck elements (stringers, cross-girders, tie-girders) are shown in Fig. 5.

In the case of elements loaded mainly at compression, such as arches, the use of high-grade steels is not always very economical, given that by

increasing the strength the coefficient of slenderness increases and the class of the cross-section tends towards a higher value, and if it reaches to section Class 4, the calculations will operate with the A_{eff} area.

Following a pre-design step, the arch is made of S235J2 steel grade, with a box section, with the dimensions and the resistance characteristics shown in Fig. 6.

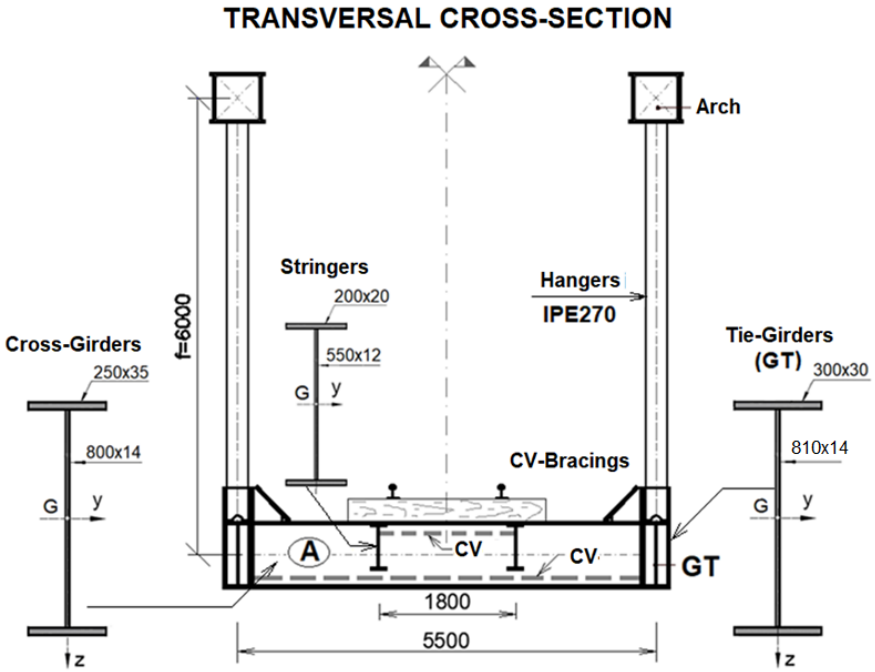


Fig. 5 − Transversal cross-section of the structure.

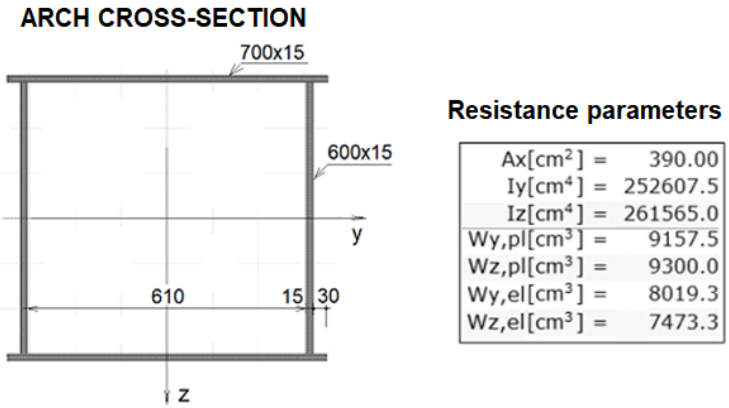


Fig. 6 − Cross-section and resistance parameters of the arch.

3.2. Design parameters

Buckling length and critical force for the in-plane arch buckling

In this case we have (m+1)5=30; m=5; f/L=1/5=0.20, and from Fig. 1 it is obtained β=0.5.

It results: $L_{cr.y}=\beta\cdot\bar{s}=0.5\cdot3310/2=827.5$ cm $=8.28$ m.

$$N_{cr.y}=\frac{\pi^2\cdot E\cdot I_y}{\left(\beta\times\bar{s}\right)^2}=\frac{\pi^2\cdot2.1\cdot10^6\cdot252607}{\left(828\right)^2}=7.63\times10^6 \text{ daN}.$$

Cross-section Class: $\frac{c}{t}=\frac{610}{15}=40.7<42\cdot\varepsilon=42 \Rightarrow$ Class 3.

Non-dimensional slenderness $\bar{\lambda}_y$

The cross-section being Class 3 will be operated with the gross area of the section.

$$\bar{\lambda}_y=\sqrt{\frac{A\cdot f_y}{N_{cr.y}}}=\sqrt{\frac{390\cdot2350}{7.63\cdot10^6}}=0.346 \Rightarrow \chi_y=0.94 \text{ (curve b of buckling)}$$

For comparison, the design parameters for stability and configuration type b) and c) in Fig. 2 are evaluated, and the results are presented in Table 4.

Table 4.

Buckling design parameters

ARCH CONFIGURATIONS	$N_{cr.y}$ x10^4 [kN]	$\bar{\lambda}_y$
a)	7.63	0.346
b)	11.60	0.281
c)	18.24	0.224

Buckling length and critical force for the out of plane buckling of the arch

Using the flexural inertia moments of the arch and of the tie-beam it is obtained:

$$q_H=\frac{q}{1+\frac{E\cdot I_G}{E\cdot I_{arc}}}=\frac{q}{1+\frac{185490}{252607}}=\frac{q}{1.734}=0.577\cdot q \Rightarrow \beta_2=1\text{-}0.35\cdot0.577=0.80$$

For $\beta=\beta_1\cdot\beta_2=0.65\cdot0.80=0.52$, l=30m, it results $L_{cr.z}=\beta\cdot l=15.6$ m .

It is obtained:

$$N_{cr.z}=\frac{\pi^2\cdot E\cdot I_z}{(\beta\cdot l)^2}=\frac{\pi^2\cdot 2.1\cdot10^6\cdot 261565}{(1560)^2}=2.23\cdot10^6 \text{ daN}$$

$$\bar{\lambda}_z=\sqrt{\frac{A\cdot f_y}{N_{cr.z}}}=\sqrt{\frac{390\cdot 2350}{2.23\cdot10^6}}=0.64 \Rightarrow \chi_z=0.81 \text{ (curve b of buckling)}.$$

In the case of open bridges (without upper bracing), one of the service limits states (SLS) verification condition is to limit the horizontal arch deflection to the value of $\delta_H=f/1500$, a condition from which a much larger cross-section in the horizontal direction of the arch (rigidity about z-z axis) can result, or a stronger end cross-girder compared to the intermediate ones.

4. Checking of an arch subjected to axial forces and bending moment

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

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}}+C_{mi.0}\frac{M_{y.Ed}+\Delta M_{y.Ed}}{\frac{M_{y.Rk}}{\gamma_{M1}}}\le 0.9 \qquad (8)$$

where:

- N_{Ed} – is the design value of the compression force;
- $M_{y.Ed}$ – is the design value of the maximum moment about the y-y axis of the member obtained from first order analysis without considering imperfections;
- $C_{mi.0}$ – is the equivalent moment factor, according to Annex A of SR EN 1993-1-1;
- χ_y – is the reduction factor due to flexural buckling.

In the case of members with a uniform section, of cross-section classes 1, 2 or 3, subjected to axial compression and monoaxial or biaxial bending, the verification equation (8) may be put in the following simplified forms:

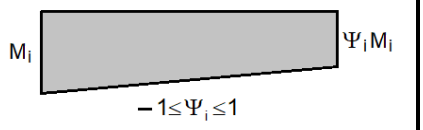
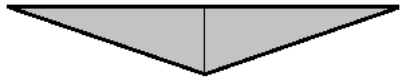
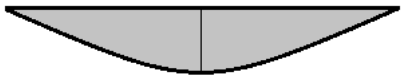
$$\frac{N_{Ed}}{\frac{\chi_{min} N_{Rk}}{\gamma_{M1}}}+C_{my.0}\frac{M_{y.Ed}}{\frac{M_{y.Rk}}{\gamma_{M1}}}\le 0.9 \text{ - monoaxial bending} \qquad (9)$$

$$\frac{N_{Ed}}{\frac{\chi_{min}N_{Rk}}{\gamma_{M1}}}+C_{my.0}\frac{M_{y.Ed}}{\frac{M_{y.Rk}}{\gamma_{M1}}}+C_{mz.0}\frac{M_{z.Ed}}{\frac{M_{z.Rk}}{\gamma_{M1}}}\leq 0.9 \text{ - biaxial bending} \quad (9)$$

For different variations in the bending moment, the equivalent bending moment factor shall be evaluated with the equations given in Table 5.

Table 5.

Equivalent bending moment factor

	$C_{mi.0}=0.79+0.21\Psi_i+0.36(\Psi_i-0.33)\frac{N_{Ed}}{N_{cr.i}}$
	$C_{mi.0}=1-0.18\frac{N_{Ed}}{N_{cr.i}}$
	$C_{mi.0}=1+0.03\frac{N_{Ed}}{N_{cr.i}}$

Checking of the arch subjected to compression and biaxial bending was carried out for two loading hypotheses:

- loading over all the span (full loading);
- loading over half the span of the bridge.

The resistances of the cross-section of the arch are:

$N_{Rk}/\gamma_{M1}=A\cdot f_y/\gamma_{M1}=390\cdot 2350/1.1=833\cdot 10^3$ daN ;

$M_{y.Rk}/\gamma_{M1}=W_{y.el}\cdot f_y/\gamma_{M1}=8019\cdot 2350/1.1=1713\cdot 10^4$ daNcm=1713 kNm

$M_{z.Rk}/\gamma_{M1}=W_{z.el}\cdot f_y/\gamma_{M1}=7473\cdot 2350/1.1=1597\cdot 10^4$ daNcm=1597 kNm .

Following the automated spatial calculation of the structure (Moga et. all, 2020), the maximum loading was obtained from the M_{max} + $N_{related}$ combination, and the result obtained is:

$$\frac{N_{Ed}}{\frac{\chi_{min}N_{Rk}}{\gamma_{M1}}}+C_{my.0}\frac{M_{y.Ed}}{\frac{M_{y.Rk}}{\gamma_{M1}}}+C_{mz.0}\frac{M_{z.Ed}}{\frac{M_{z.Rk}}{\gamma_{M1}}}=0.75<0.9$$

5. Conclusions

The stability check of the metal arches is a particularly important problem, which is why there are many materials in the technical literature, but some of them are difficult to apply in the current design work.

The 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.

In the arch plane these design data are presented for the arches with vertical hangers and for those without hangers, with rigid supports - for four static systems: 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. By arranging the hangers in a modified configuration from the vertical one, a more uniform distribution of the axial and bending efforts on the arch results and, at the same time, increases the critical buckling force in the arch plane.

In the out of plane of the arc, the evaluation of the critical forces for non-winding arches, loaded by hangers or vertical posts, and for upper-winding arches between them (closed bridges) is presented.

In the case of open bridges, without upper-winding between arches, in the calculation of stability in the out of plane of the arch, the coefficient is included, and for its evaluation, in EC3-2 does not specify how to assess or evaluate the reports q_H/q and q_{st}/q.

The analysis of the general stability of a deck on open-type bridge (without top bracing), i.e. the evaluation of the parameters for the stability of arches in the in-plane of arch and in the out of plane of arch, carried out in accordance with the norm SR EN 1993-2:2007, may be useful in the design of arch structures for the field of bridges or other types of metal structures.

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ASPECTE PRIVIND STABILITATEA GENERALĂ A ARCELOR CIRCULARE LA PODURILE DESCHISE ÎN CONFORMITATE CU EC3-2

(Rezumat)

În lucrare se prezintă aspecte legate de verificarea stabilității generale a podurilor pe arce circulare cu montanți rigizi, de tip deschis - fără contravântuire superioară.

Este efectuată o analiză numerică pentru un pod de cale ferată normală cu deschiderea de 30 m, care are la bază normativul SR EN 1993-2:2007. Eurocod 3: Proiectarea structurilor de oțel. Partea 2: Poduri de oțel (EC3-2).

Analiza efectuată poate fi utilă în însușirea procedurii prezentate în normativ, precum și în activitatea de proiectare a unor structuri pe arce din domeniul podurilor sau ale altor tipuri de structuri metalice.