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"BOWSTRING" ARCHES IN LANGER SYSTEM WITHOUT WIND BRACING

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Abstract

Arch bridges are slender structures and can be efficiently used in the range of medium to large spans. These structures have an improved aesthetic aspect and in the same time a low construction height, with obvious advantages regarding reduced costs in the infrastructuers and their foundations.

For this type of structures usually composite or orthotropic decks are used. Lately, innovative solutions have been used in designing arch bridges, especially discarding the top wind bracing system. The level of axial forces and bending moments in the arches and tie imply the design of sections with sufficient stiffness and strength in both directions in order to ensure the general stability of the arches, without the need for top wind bracing. Moreover, the cross section of the arches is not constant, but shifts in accordance with the variation of the bending moments, in order to ensure their lateral stability.

This paper studies a road bridge with parallel Bowstring arches, with a span of 108m and a carriageway 7.00m wide, sustained by a deck made up of crossbeams 2m apart and a concrete slab. The main beams are held by ties arranged in the Langer system, 10 to 14m apart from each other. The sag of the arches is 18m high.

The analyzed structure was proposed for construction in the city of Oradea and is used for crossing the "Crişul Repede" river, between Oneştilor street on the left bank and the Sovata, Fagului and Carpaţi streets on the right bank.

The performed analyses have the following two main objectives: to establish the critical load for which the failure of the arches occurs by instability and to underline the influence of different wind bracing systems on the bridge's collapse loads respectively.

Keywords: arch, wind bracing, stability

1. INTRODUCTION

Most arch bridges with spans over 250m built up until now have their hangers arranged in the Langer system because this leads to lower overall heights. In the interval of spans under 250m, arch bridges are built in the "bowstring" system (system that defines structures in which the compression forces in the arches are balanced by the tensile forces in the ties, also called stiffening girders). This type of structure – "bowstring" – is based on a Swedish bridge concept and is very efficient from the consumption of steel point of view. A fundamental principle states that the bending moments must be supported by the arches and ties proportional with their bending stiffness, aspects which was first pointed out by prof. M. Ritter at the IABSE Congress in 1940 (International Association for Bridge and Structural Engineering). The optimal allocation of steel between the two elements must be taken into account, and furthermore, in order to obtain maximum efficiency, the arch must be made in such a way as to muster sufficient resistance to lateral buckling without neglecting it's aesthetic shape. This type of structure was first implemented in Sweden, being very efficient over a wide interval of spans up to 120m.

2. PURPOSE OF THE PAPER. DESCRIPTION OF THE ANALYZED STRUCTURE

The purpose of this paper is to establish whether or not it is convenient, from an economic point of view, to adopt upper wind bracing for arch bridges in the "bowstring" system with Langer arranged hangers.

For aesthetic reasons, the current trend is to design steel bridges in the "bowstring" with low overall heights. This solution has certain implications as follows: the axial forces and bending moments in the arches and stiffening girders are bigger, resulting in an increase in the cross sections and the necessity of arranging the hangers in the Langer system and the impossibility of adding upper wind bracing due to the requirements of the gauge.

The high level of stress in the arches and ties imply designing sections with sufficient resistance and stiffness on both directions in order to ensure the overall stability of the arches witouh having the need for a wind bracing system. Adopting such a solution will result in arches with varying cross sections, with the dimensions variations being dictated by the general stability.

In this paper, a road bridge is studied with twin parallel arches in the bowstring "system", with a span of 108m and a 7.00m wide carriageway, supported by a deck made up of two stiffening girders, crossbeams every two meters and a concrete slab. The stiffening girders are sustained by hangers arranged in the Langer system at distances between 10m and 14m. The deflection of the raches is of only 18m, and the transverse distance between them is of 9.20m (Fig.2,3).

The bridge's infrastructure consists of two massive abutments with indirect foundations on large diameter bored piles 1.50m across and 20.00m long (Fig.3).

The analyzed structure was proposed for erection Oradea city, in order to ensure the crossing of the "Crişul Repede" river between „Oneştilor" street on the left shore, and the Sovata, Fagului and Carpaţi streets on the right one.

Figure 1. Site of the analyzed structure

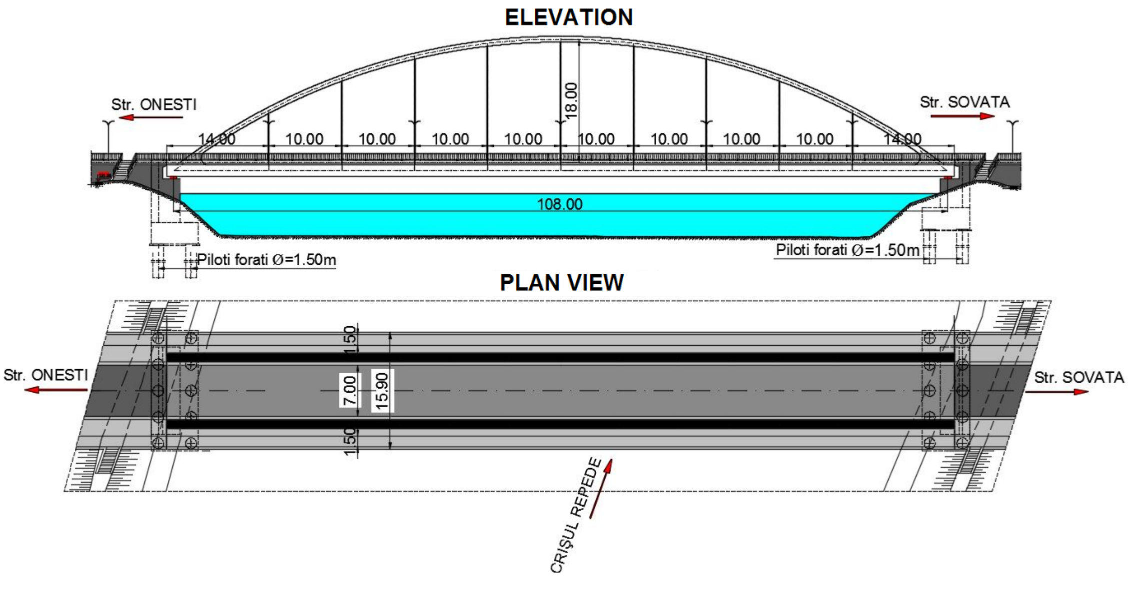


Figure 2. Elevation and plan view of the bridge

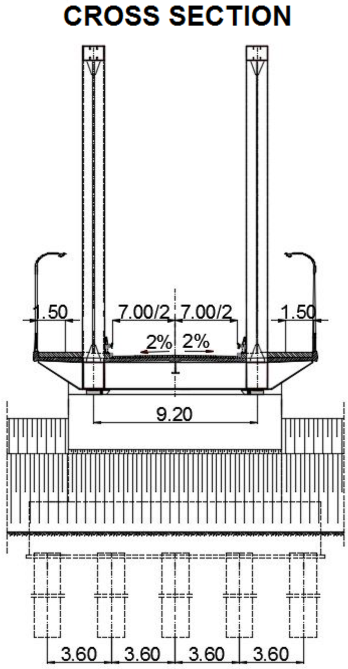
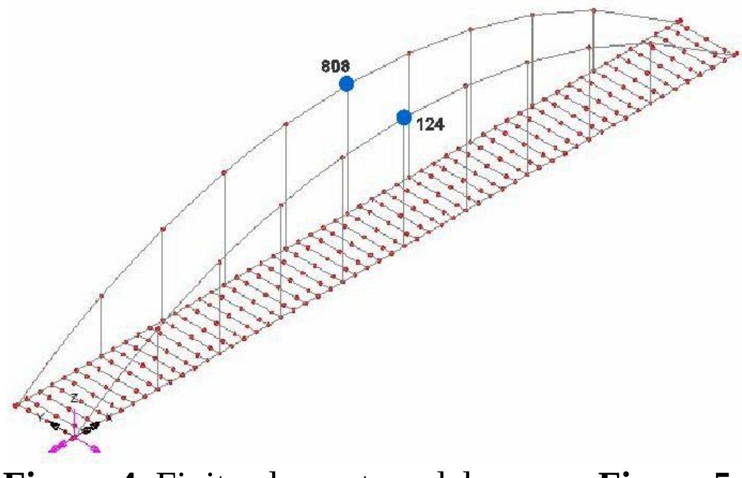


Figure 3. Cross section of the bridge

3. DESCRIPTION OF THE F.E.M. MODEL

Pentru a analiza riscul apariţiei fenomenelor de instabilitate generală a arcelor a fost In order to study the risk for occurrence of general instability phenomena of the arches, a three dimensional finite element model with beams was used (Fig.4).



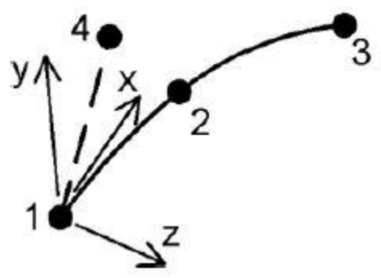


Figure 4. Finite element model **Figure 5.** Thin beam finite element [1]

The bridge's deck, which supports the carriageway, made up of the stiffening girders, the crossbeams and the concrete slab was modeled using a planar beam grid using thin curved beam elements with four nodes (Kirchhoff) (Fig.5), which don't have the deformation under shear force formulation included. These finite elements each have three degrees of freedom on the end nodes, a displacement and a relative rotation for the third node, situated in the middle of the bar and the fourth node is used for defining the element's xy local plane (Fig.5).

The geometrical characteristics of the cross beams sections were predetermined using the shear lag concept, applied to composite steel-concrete superstructures, because the concrete slab is linked with the cross beams by means of flexible connectors.

The supporting hangers were modeled with isoparametric unidimensional finite elements as curved bars with three nodes (two at the ends and one in the middle), each having three degrees of freedom formulated as translations.

The two abutments of the bridge and their foundations were not included in the model because they have little influence on the structure's response under the considered loads.

In all the analyses done, the behaviour of the material was considered as linear-elastic.

4. PERFORMED ANALYSES. RESULTS

In order to evaluate the possibility of general instability phenomena of the bridge's arches occurring (lateral buckling or torsion), the static analyses were done in two steps. In the first step, a stability eigenvalue analysis (buckling) was done, considering the loads from the structure's body weight in the final stage of construction (service) and a 10kN/m uniform distributed load. This was used to determine the shapes corresponding to the arches loss of stability and the corresponding load factors, which would lead by multiplication of the intial considered loads, to an approximate value of the critical loads. From the structure's geometrical point of view, there were no imperfections considered during errection, which could have led to eccentricities between the elements axes or the application of the loads. The first four loss of stability shapes and their corresponding load factors are presented in figures 6 through 9.

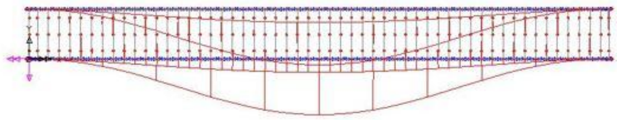


Figure 6. Mode 1, λ_1=42.34

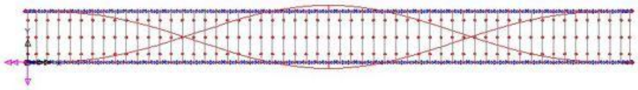


Figure 7. Mode 2, λ_2=57.02

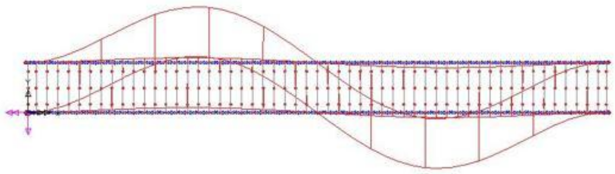


Figure 8. Mode 3, λ_3=59.48

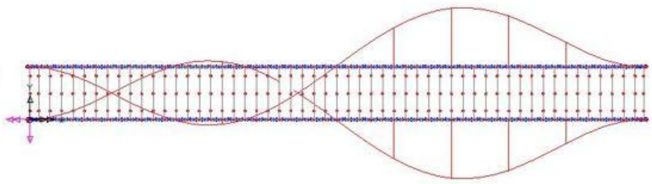


Figure 9. Mode 4, λ_4=61.02

Starting from the allure of the deformed shapes of the structure under general buckling of the arches, geometric nonlinear analyses were done in the second step of the study, aiming to establish the structure's response under the effect of the applied exterior loads. An initial deformed shape of the structure was considered, according to each of the shapes presented in figures 6 through 9, based on the following formula:

$$e_i = e_0 \cdot sin\frac{n\pi x_i}{L} \tag{1}$$

The following physical sizes intervene in the upper formula:

e_i is the lateral displacement value of the point defined by the abscissa x_i through which the initial deformation of the arches was modeled;

e_0 is the maximum lateral displacement value of the lateral deformation at the mid span;

n is the number of half-waves considered for the initial deformation of the arches;

L is the span of the bridge.

According to the specifications in [7], the maximum lateral displacement of the arches was considered in the interval between L/2000-L/500.

In order to evaluate the risk of general instability phenomena of the arches occurring in service, load model LM1 was applied besides the structure's body load in the final stage, according with [8].

The criterion used to stop the analysis was considered for the state of deformation of the structure, according to which, after reaching a certain value for the transverse displacement of the arches, the structures becomes nonfunctional due to the lack of space for the gauge. The two points situated at the keys of the arches were monitored for lateral transverse displacement (Fig. 4). The maximum value allowed for the lateral displacement was considered 0.12m (Fig. 10).

After the nonlinear analyses were done, load-displacement (P-Δ) response curves of the structure were traced, and by using the maximum displacement criterion, the values of the load factors for the maximum deformation state were determined.

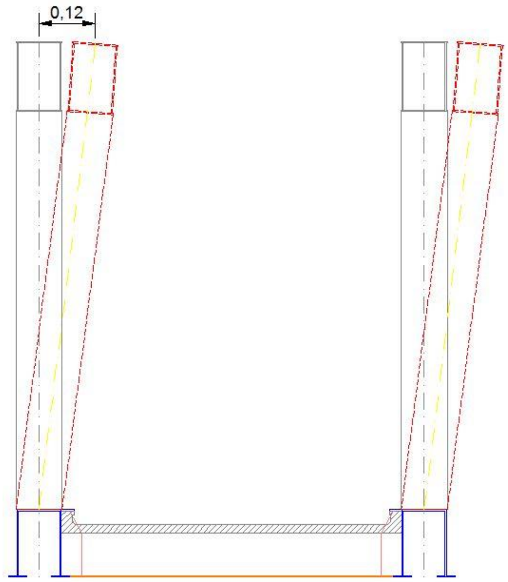


Figure 10. Deformed shape of the deck

In figures 11 and 12 the P-Δ curves are represented for the points at the keys of the two arches, with overlaped maximum displacement dictated by the deformation criterion, as well as values of the load factors corresponding to loads from body weight, load model 1 and the combination of the two at the ultimate limit state. The initial deformation was considered for a single half-wave, having a maximum value of $e_0=L/500$.

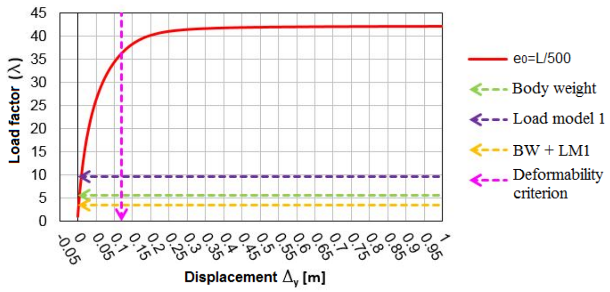


Figure 11. Curve P-Δ, node 124

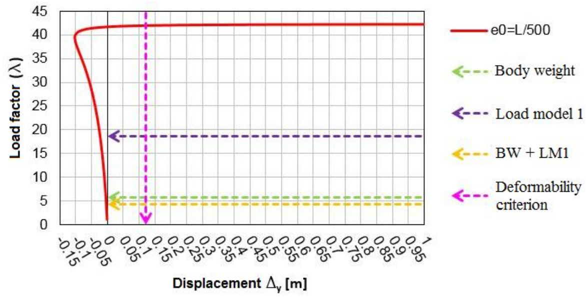


Figure 12. Curve P-Δ, node 808

In order to ensure the structure against the occurring of the general instability phenomena, the possibility of adding an upper wind bracing system between the two arches was analyzed, with respect to the conditions imposed by the gauge. Three distinct systems for the upper wind bracing were studied, which are frequently used in the design of this type of structures: a system made up of transverse beams – system 1 (Fig. 13), the one made up of transverse beams and diagonals arranged in the shape of the letter “X” – system 2 (Fig. 14) and the one ade up of transverse beams and diagonals arranged in the shape of the letter “K” – system 3 (Fig. 15).

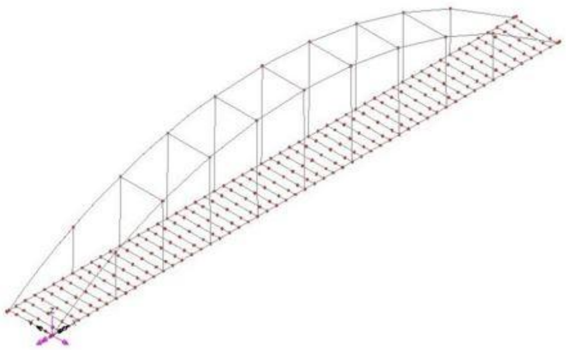


Figure 13. Wind bracing system 1

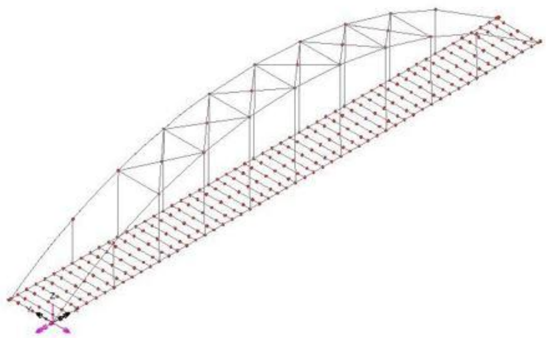


Figure 14. Wind bracing system 2

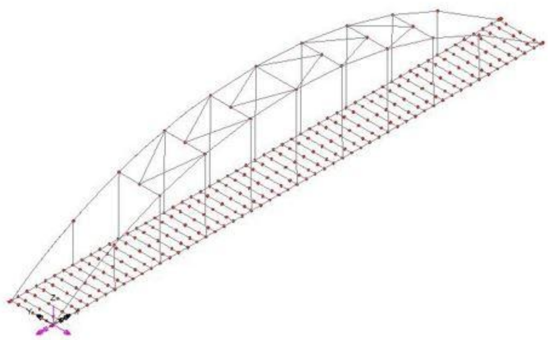


Figure 15. Wind bracing system 3

Using these models, more geometric nonlinear analyses were performed with the purpose of determining the load factors corresponding to the chosen deformation criterion. The P-Δ curves for the three wind bracing systems are presented in figures 16-18.

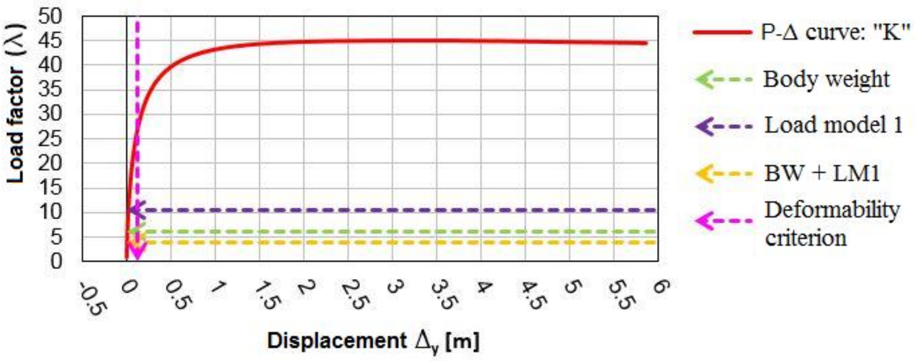


Figure 16. *P*-Δ curve, wind bracing system 1, node 124

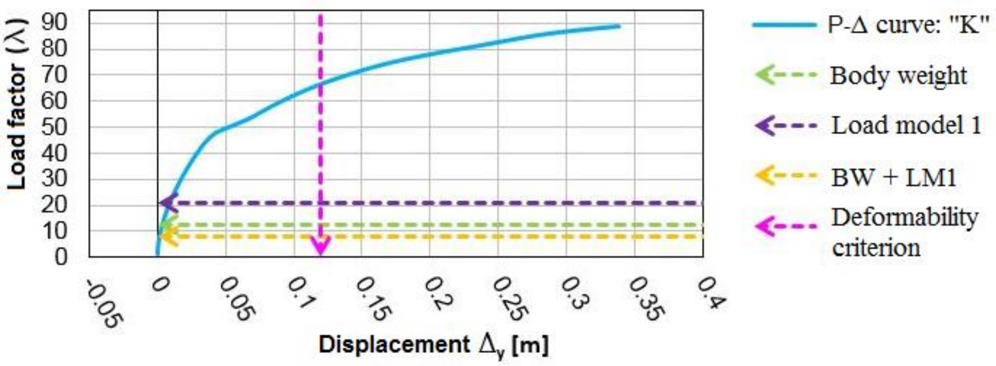


Figure 17. *P*-Δ curve, wind bracing system 2, node 124

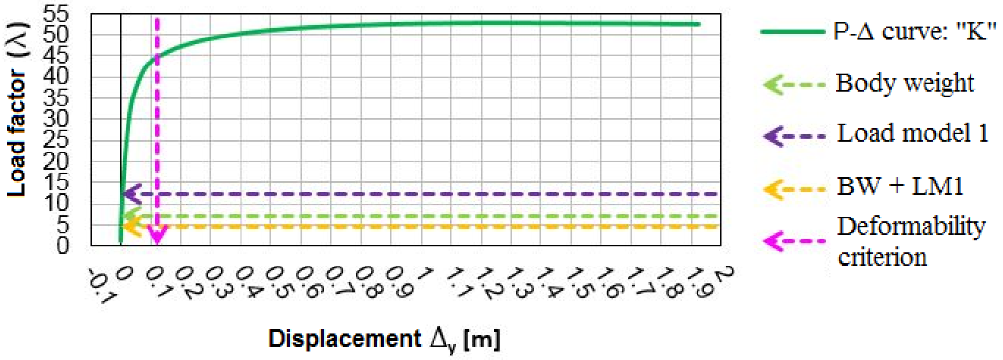


Figure 18. *P*-Δ curve, wind bracing system 3, node 124

For all the three considered cases, the maximum accepted lateral displacement of the arches keys was e_0=*L*/500.

5. CONCLUSIONS

In this paper, the phenomena of loss of general stability was analyzed for an arched, bottom carriageway, road bridge over the “Crişul Repede” river, in Oradea city. Two types of analyses were done using finite element models. In the first case, a linear static buckling eigenvalue analysis was done in order to determine the approximate values of the critical load. After this, geometric nonlinear analyses were performed considering deformed initial shapes of the structure following the allures of the previously determined loss of stability modes, in order to determine the load-displacement (P-Δ) curves, thus resulting the values of the critical load factors that lead to the ultimate limit state of deformation corresponding to the structure’s body weight combined with the load model 1.

By analyzing the obtained results, it can be said that the values of the critical load factors are quite high, even with the lack of upper wind bracing and there is no risk of loss of stability in the arches. Even in the worst case scenario, corresponding to the ultimate limit state under the combination between the structure’s own weight and the load model 1, the obtained value for the critical load factor was 3.70.

In figures 11 and 12 it can be observed that, up to a certain value of the load factor, the displacement tendency of the arches is symmetric and towards the inside of the bridge, but after a certain point, the arches take an antisymmetric shape, corresponding to the first loss of stability mode, presented in figure 6.

Regarding the efficiency of the wind bracing systems, it can be seen in figures 16-18 that the highest value of the critical load factor for the combination of permanent loads and load model 1, is obtained for the systems with the diagonals arranged in the shape of the letter “X” and it’s value is approximately 10.

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