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Wind Dynamic Response of a Suspension Pedestrian Bridge

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Summary

The paper presents wind dynamic response of a suspension pedestrian bridge. Dynamic response of the structure was obtained by performing two analyses: modal superposition (linear analysis) and direct integration of equation of motion (geometric nonlinear analysis). The aerodynamic instability of the suspension structure was studied too. All the analysis was carried out by LUSAS (FEM) program.

KEYWORDS: Geometric nonlinear analysis, Dynamic response

1. INTRODUCTION

After the disaster of Tacoma Narrows Bridge in 1940 attention focused on the dynamic problems of suspension bridges. The great span length of suspension bridges makes their static and dynamic behavior under the action of lateral forces an important problem. The most significant forces are due to wind and to the earthquakes.

In order to obtain a correct response of the structure subjected to dynamic loads it is absolute necessary to carry out nonlinear dynamic analysis.

Time history response of the suspension bridge will be obtained in two load cases: mean velocity wind action and for gust of wind.

2. PEDESTRIAN SUSPENSION BRIDGE

Pedestrian Suspension Bridge was designed as a link between Burjuc and Tisa, over Mures River.

The superstructure consist in a steel continuous lattice girder over three spans (40,00 m + 120,00 m + 40,00m) (Figure 1.).

The deck is a triangular system with 1,55 m high and 2,20 m width and stiffening frame at superior and inferior sole.

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The suspension system is formed by the main parabolic suspension cables and vertical hangers. Each main suspension cable is it made from three steel carrying cables with 60 mm in diameter. The vertical hangers are made from 6 round steel bars with 21 mm in diameter, placed at 5,00 m one to another along the central span.

The steel towers has A shape in longitudinal and transversal plane. The high of the towers is 25,40 m. The towers are supported on two caisson indirect column foundations.

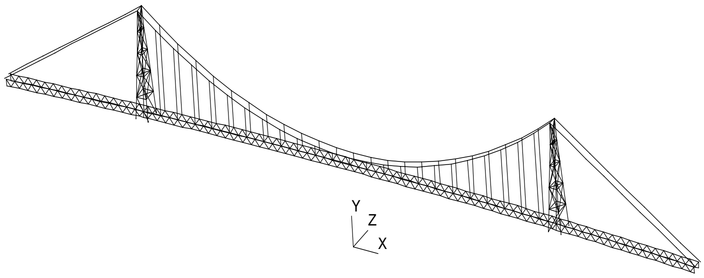

Figure 1. Finite element model

3. FINITE ELEMENT MODEL

The structure has a pronounced spatially behavior. This is the reason that lead to a discrete spatially model. The finite elements (3D beam) were chosen to allow running nonlinear geometric analysis. Discrete model of the structure has 3088 nodal points and 1330 beam finite elements, [6].

4. STRUCTURE ANALYSIS

4.1. Static analysis

The geometric nonlinearity of the suspension system of the structure require static nonlinear analysis (modified Newton – Raphson), [3].

The wind pressure acting in global direction Z is 2 kN/m^2 according to the standards.

A comparative study between linear and nonlinear geometric static analysis is illustrated in Figure 2. Lateral flexibility of the suspension system is revealed by

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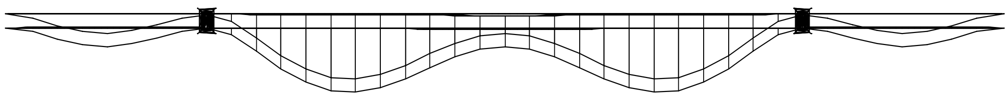
the evaluation of the displacements of the main suspension cables in both linear and geometric nonlinear static analysis.

Maximum displacement for the middle of central span
0,248 m – linear static analysis
0,246 m – geometric nonlinear static analysis



Maximum displacement for the cables
5,300 m – linear static analysis
1,206 m – geometric nonlinear static analysis

Figure 2. Maximum displacement for wind pressure

4.2. Modal analysis

The necessity of running a modal analysis result from the prescription of different European codes for vibration response of the suspension structure, [1], [4].

The 3D discrete model was subjected to modal analysis is order to obtain the first 15 eigenvalues and mode shapes.

In Figure 3. are presented the most significant mode shape of the suspension bridge: the first symmetric mode of lateral vibration of the deck (Mode No. 1), the first anti-symmetric mode of vertical vibration (Mode No. 2), the first symmetric mode of lateral vibrations of the towers (Mode no. 9) and the first mode of torsional vibrations (Mode no. 11).

4.3. Wind effect according to different European prescription

Once eigenvalues and mode shape obtained from modal analysis, it can be use to evaluate wind velocity corresponding to specific dynamic effects.

Not all the specific effects were reflected in European prescription. This is the reason that wind critical values are evaluated only according to some prescription.

Evaluation of wind response involve studies about limited amplitude oscillations, divergent amplitude oscillations and non-oscillatory divergent movements, [1], [4], [8], [9].

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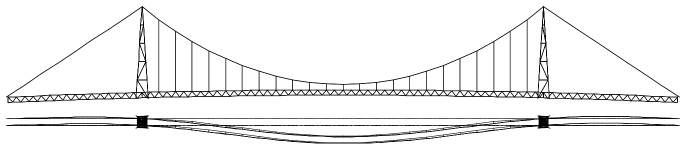
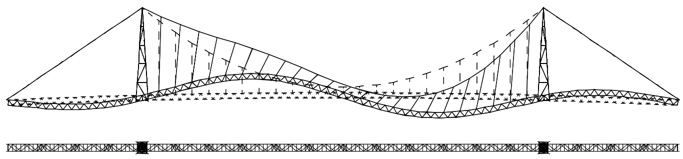
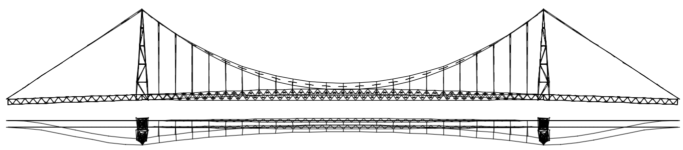


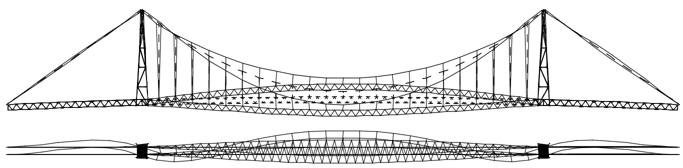
Modal shape 1   T = 1,190 s



Modal shape 2   T = 0,638 s



Modal shape 9   T = 0,228 s



Modal shape 11   T = 0,218 s

Figure 3. The most significant modal shapes

Conclusion: Pedestrian suspension bridge present aerodynamic stability for wind turbulence, for von Karman gust and flutter effect (the most dangerous dynamic effect of the wind acting on suspension bridges), [6].

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4.4. Resonance amplitude

The first 15 natural frequencies of the structure were in the range of 0,00 – 6,00 Hz. Wind response will be studied in the same range of frequency.

The structure response is presented for two nodal points which offer the most significant information.

For the tower - the maximum lateral displacement of a nodal point lead to the top of the tower.

For the deck - the maximum lateral displacement of a nodal point corresponding to the middle of the central span.

The structure response in frequency domain due to wind excitation is presented in Figure 4. and Figure 5.

The resonance amplitude of 4,39 Hz at the top of the tower is related to the first natural frequency of symmetric lateral vibration mode (Mode no. 9). In the same way, the resonance amplitude of 0,84 Hz at the middle of the central span is related to the first natural frequency of symmetric lateral vibration mode (Mode no. 1). The amplitude in X and Y global directions can be neglected instead of Z global direction.

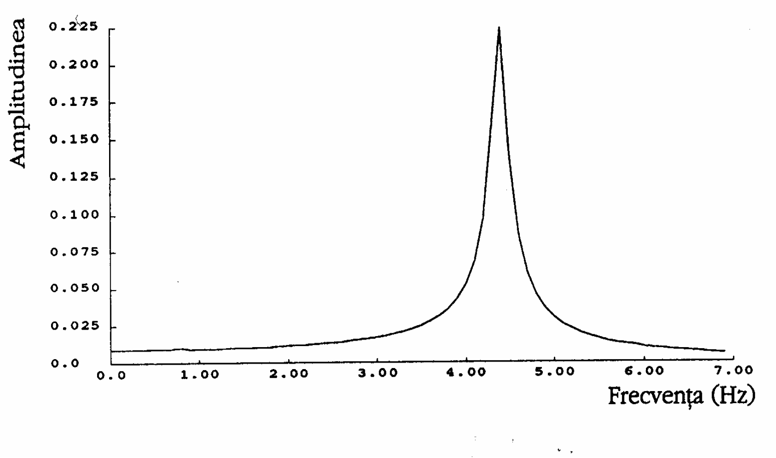

Figure 4. Resonance amplitude for the top of the tower due to wind excitation

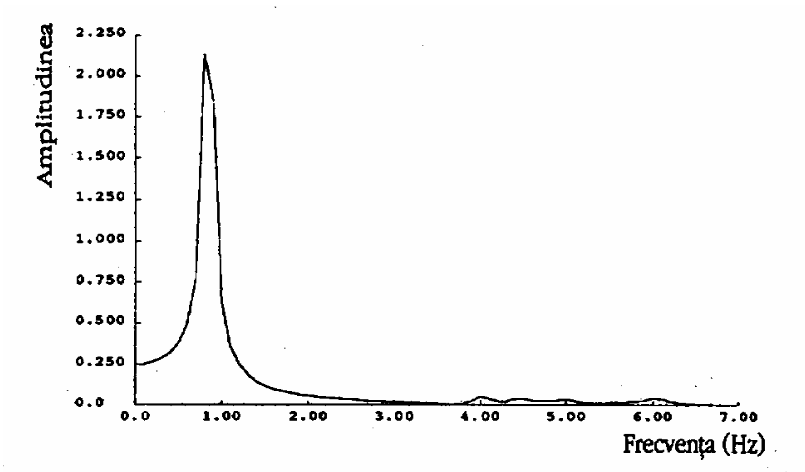

Figure 5. Resonance amplitude for the middle of the central span due to wind excitation

4.5. DYNAMIC WIND ANALYSIS

4.5.1. Wind mean pressure

The real wind forces were considered acting in one step. Time history response of the structure is presented in Figure 6 and Figure 7 for the same nodal point (top of the tower and middle of the central span). Lateral displacements of nodal points oscillating around the same values obtained in static analysis for wind pressure.

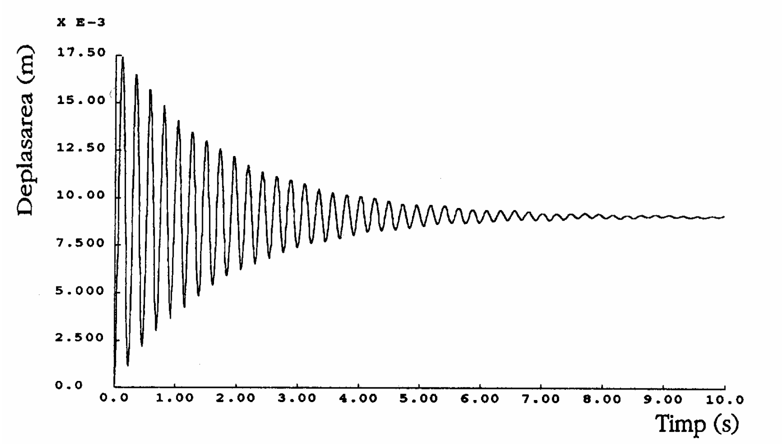

Figure 6. Time history response in displacements for the top of the tower.

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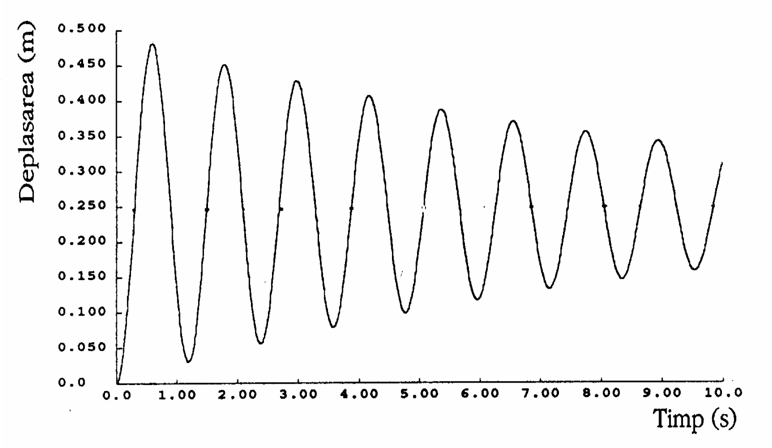

Figure 7. Time history response in displacements for the middle of the central span.

4.5.2. Gusty wind

The real wind velocity is permanently changing in time and space. A result of different study and measurements on wind velocity is illustrated in Figure 8.

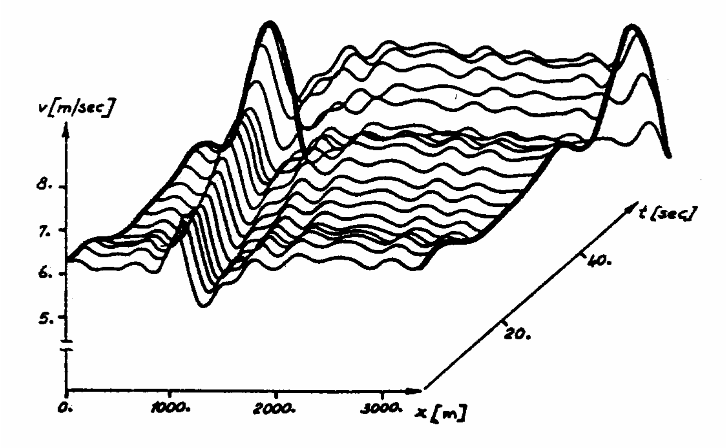

Figure 8. Wind velocity space and time distribution

Dynamic response on wind excitation can be obtained by experimental or by analytical approach.

Mathematical method can be a stochastic approach in frequency domain – unacceptable for suspension bridges (cannot evidence the flutter effect) or a deterministic approach in time domain – which require time history of air flow in

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time and space for direct integration of equation of motion at different moment of time.

Suspension bridge was subjected on a deterministic approach. Air flow was modeled in time steps of 0,05 s for 12,0 s range (Figure 9). The 12,0 s range is long enough to cover the dynamic response of the structure which has first natural period 1,19 s.

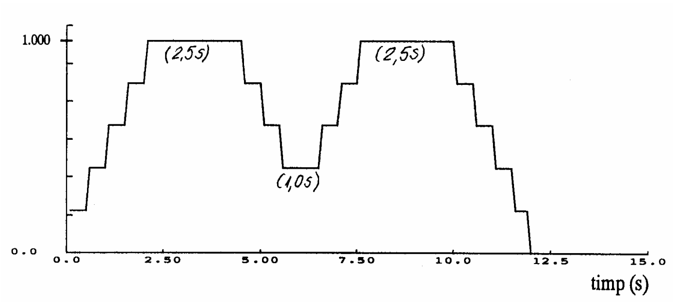

Figure 9. Wind velocity time distribution

Space air flow was neglected – considering constant pressure of the wind along the bridge.

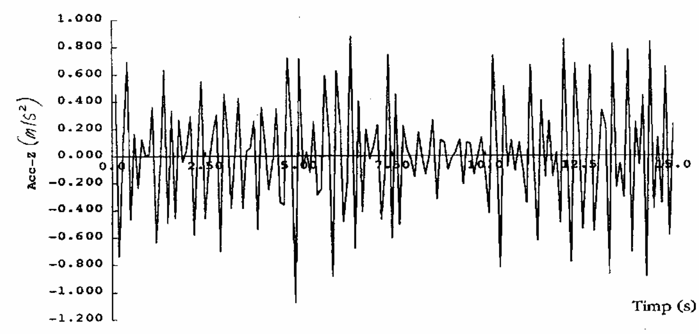

Figure 10. Time history in acceleration of the top of the tower in global direction Z shows a maximum acceleration of 0,12g.

Dynamic analysis include 12,0 s of wind pressure and 3,0 s without any excitation.

For gusty wind it was defined a specific load function which multiplied static wind load for all the members of the structure (Figure 9.).
Time history response of the structure is illustrated in Figures 10 - 13 for different quantities.

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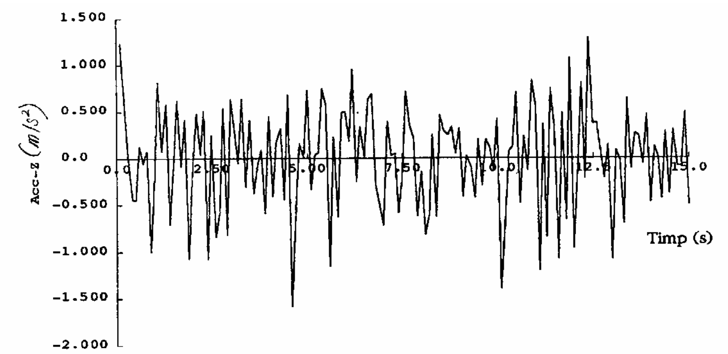

Figure 11. Time history in acceleration of the middle of the central span in global direction Z shows a maximum acceleration of 0,15g.

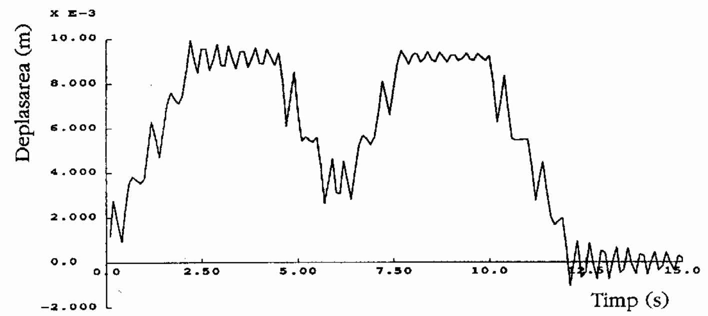

Figure 12. Time history displacement shows an oscillation around displacement value evaluated in static analysis for top of the tower.

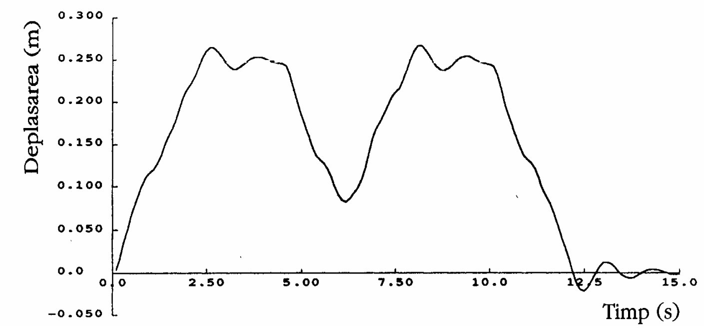

Figure 13. Time history displacement shows an oscillation around displacement value evaluated in static analysis for middle of the central span.

5. CONCLUSIONS

The geometric nonlinearity of the suspension system requires static nonlinear analysis.

Dynamic response of suspension bridges is highly influenced by the damping ratio. Choosing a wrong value for damping ratio can corrupt the response.

Geometric nonlinearity has to be taken into account for dynamic analysis too. Direct integration of equation of motion is the solution for flexible structure.

The pedestrian suspension bridge is well conformed from static and dynamic point of view.

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