# Dynamic Behavior of Cable Supported Bridges Affected by Corrosion Mechanisms under Moving Loads

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**THE DYNAMIC BEHAVIOR OF CABLE SUPPORTED BRIDGE SUBJECTED TO MOVING LOADS AND AFFECTED BY CORROSION MECHANISMS IN THE CABLE SUSPENSION SYSTEM IS INVESTIGATED**

## COMPUTATIONAL METHODS:

- ***Cable Formulation**:* The theoretical formulation is consistent to large deformation theory based on Green-Lagrange's strain measure and the second Piola-Kirchhoff stress, whereas the material behavior was assumed to be linearly elastic. The weak form can be derived by using the principle of D'Alembert as follows:

$$\sum_c \int \sigma_n \delta\varepsilon_n dV_0 + \sum_c \int \mu_c \ddot{\underset{\sim}{u}} \delta \underset{\sim}{u} dV_0 = \sum_c \int \underset{\sim}{g}_c \delta \underset{\sim}{u} dV_0 + \sum_c \underset{\sim}{F} \delta \underset{\sim}{u} dV_0$$

- **Girder Formulation**: The interaction between moving loads and bridge motion was considered introducing non-standard contributions arising from Coriolis and centripetal inertial forces, which are, mainly, produced by the coupling behavior between moving system and bridge deformations.

$$\ddot{\bar{u}}_z = \frac{\partial^2 \bar{u}_z}{\partial t^2} + \frac{\partial^2 \bar{u}_z}{\partial x \partial t} 2c + \frac{\partial^2 \bar{u}_z}{\partial x^2} c^2, \text{ with } c = \frac{\partial x}{\partial t}$$

The mass and loading functions during the external load advance, can be written by the following expressions:

$$\rho = \lambda H\left(x_1 + L_p - ct\right) H\left(ct - x_1\right), \quad f = pH\left(ct - X\right) H\left(X + L_p - ct\right)$$

$$\rho_0 = \lambda_0 H\left(x_1 + L_p - ct\right) H\left(ct - x_1\right), \quad m = p \cdot e\, H\left(ct - x_1\right) H\left(x_1 + L_p - ct\right)$$

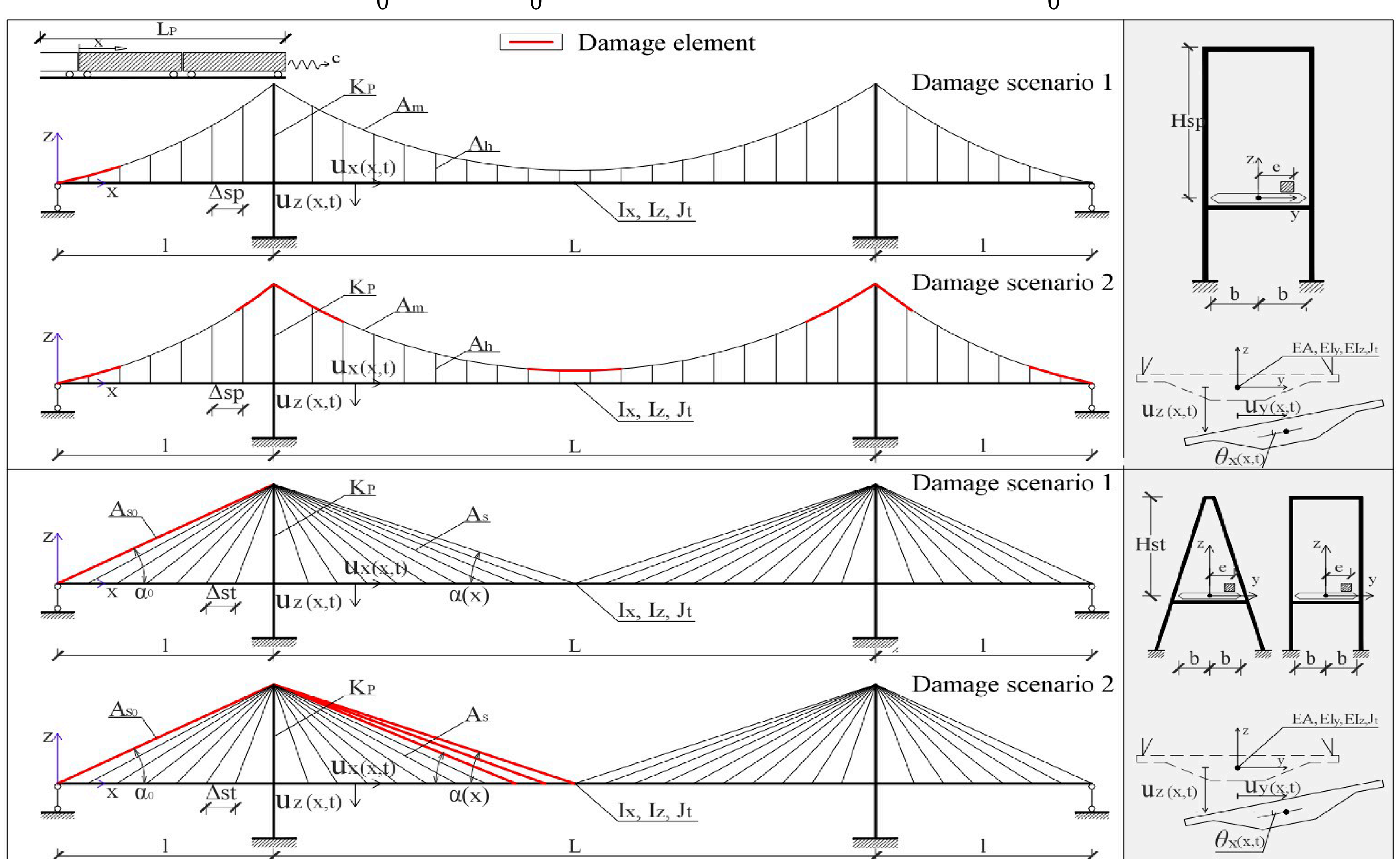
The dynamic equilibrium equations were derived in explicit form, consistently with a variational approach, in which both internal and external works were evaluated by means of the following relationship:

$$\sum_c \int \left( N\delta\varepsilon_n + M_x \delta\chi_x + M_y \delta\chi_y + M_t \delta\theta_x \right) dL + \sum_c \int \mu \ddot{\underset{\sim}{u}} \delta \underset{\sim}{u} dL + \sum_c \int \mu_0 \ddot{\underset{\sim}{u}} \delta \underset{\sim}{u} dL =$$

$$\sum_c \int \underset{\sim}{g}_G \delta \underset{\sim}{u} dL + \sum_c \underset{\sim}{F} \delta \underset{\sim}{u} dL + \sum_c \int \rho \left( \ddot{\bar{u}}_z + 2c\dot{\bar{u}}_z + c^2 \bar{u}''_z \right) \delta \bar{u}_z dL + \sum_c \int \dot{\rho} \dot{\bar{u}}_z \delta \bar{u}_z dL$$

- ***Corrosion Mechanism Formulation**:* The cable corrosion mechanism was formulated, consistently with a Continuous Damage Mechanics approach in which cable deterioration results in the reduction in cable cross-sectional area. The effective modulus of elasticity $E_{eff}$ and the corresponding stress for corroded cable can be defined as :

$$D = \frac{A_{eff}}{A_0} = \frac{A_0 - A^*}{A_0}; \quad \sigma_{eff} = \frac{\sigma}{1 - D}; \quad E_{eff} = \frac{A_{eff}}{A_0} E$$




*L*= 1050 m; *l*= 350 m; $H_{sp}$= 168 m; $H_{st}$= 210 m; *b*= 10 m; $\Delta_{st}$= 10 m; $\Delta_{sp}$= 20 m; $L_p$= 1050 m; *e*= 5 m; *A*= 9.54 m$^2$; $\sigma$g=3.58E8 Pa; $A_m$= 0.488 m$^2$; $A_h$= 0.0106 m$^2$; $A_{s0}$= 0.248 m$^2$; $A_s$= g $\Delta_{st}$ /($\sigma$g sin$\alpha$); $I_y$= 3.057 m$^4$; $I_z$=113.11 m$^4$; *Jt*= 1.0332 m$^4$; *g*= 30000 kg/m; *p*= 30000 kg/m; *E*= 2.05E11 Pa; $K_p$ = 7357500 N/m;

**Figure 1.** Structural model of the bridge, damage scenarios and bridge properties utilized in the results

## RESULTS AND CONCLUSIONS:

Results are presented for cable supported bridges based on both suspension and cable-stayed configurations, adopting similar properties for the main constituents of the bridge structures. The analyses are reported for bridge structures with different damage mechanisms in the cable system (fig.1).The results show that H-shaped tower bridge typologies are more affected by the damage mechanisms than the A-shaped tower bridge.

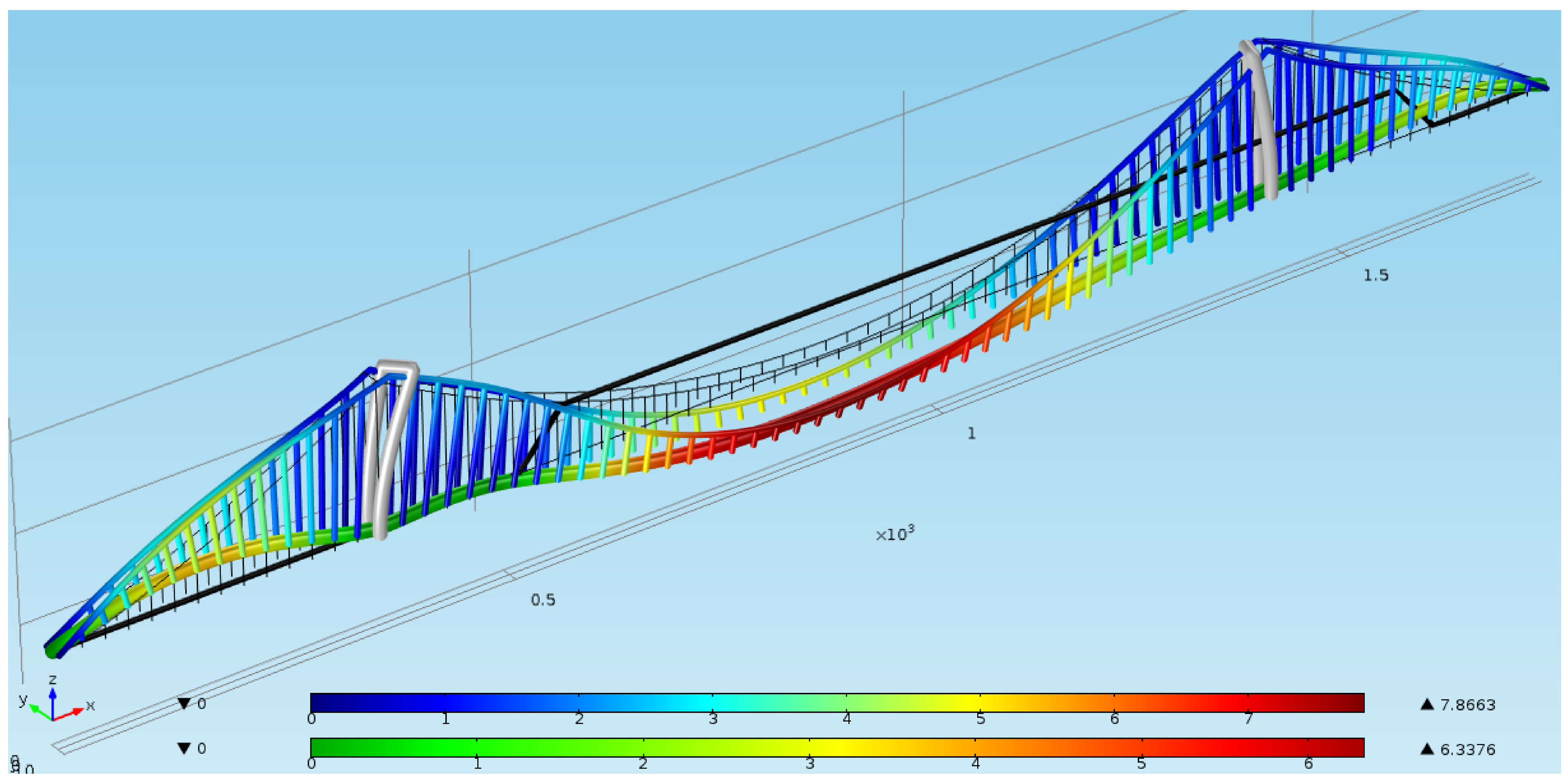


**Figure 2**. Suspended bridge: Comsol Multiphysics 4.2a model

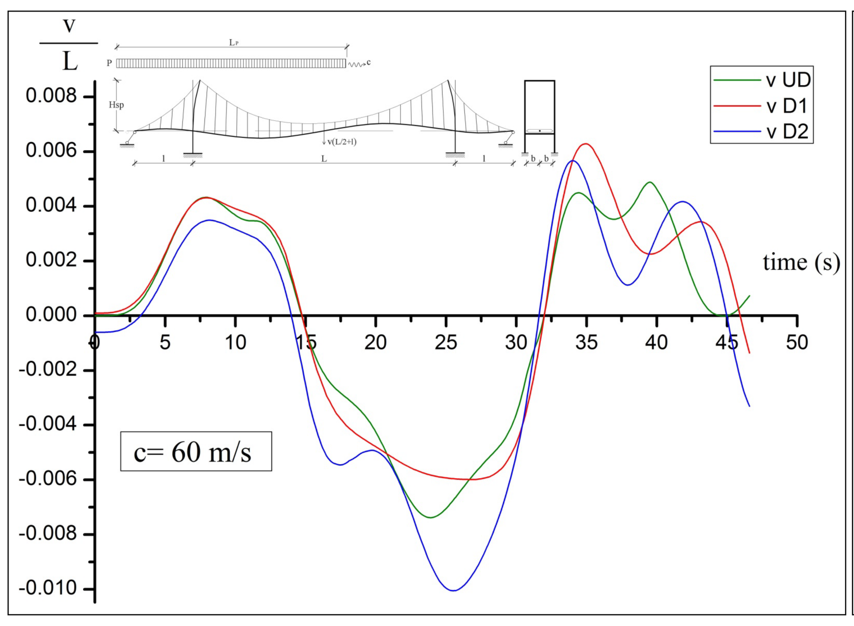



**Figure 3**. Suspended bridge: Time History of the midspan displacement

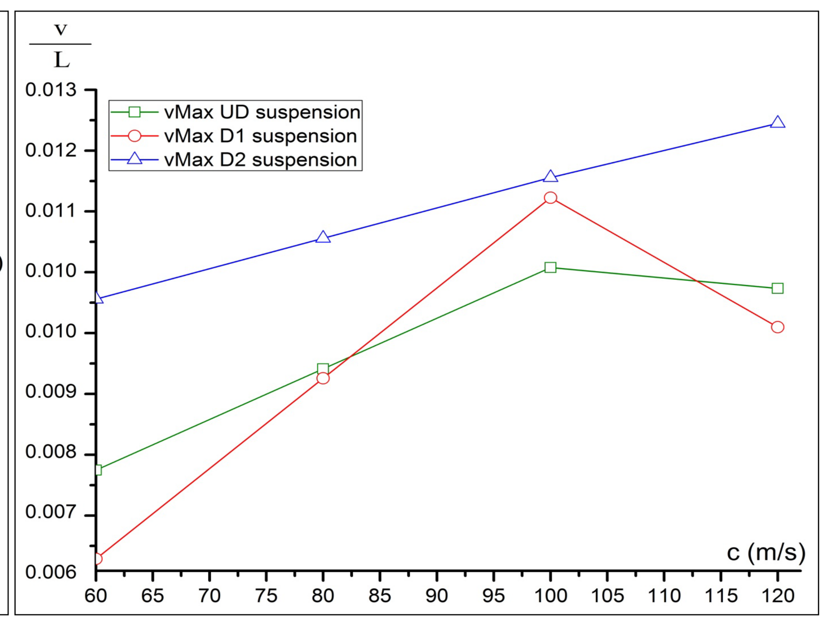



**Figure 3**. Suspended bridge: Maximum vertical midspan displacement - influence of the speed

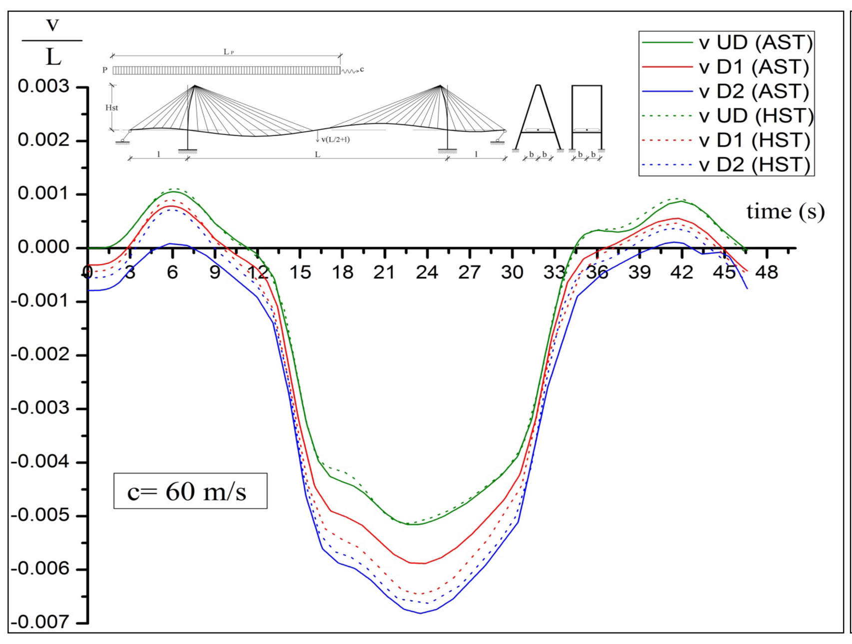



**Figure 4**. Cable-stayed bridges: Time History of the midspan displacement – c = 60 m/s

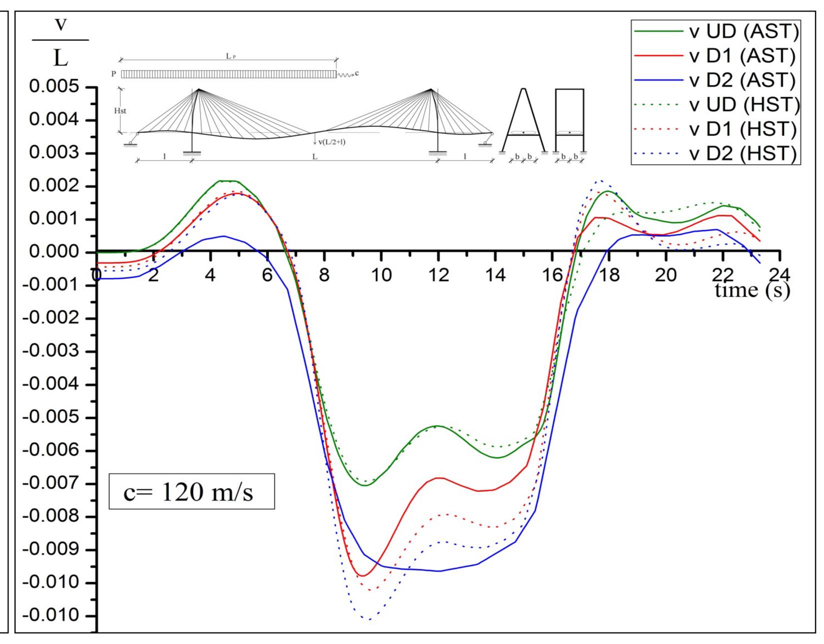



**Figure 5.**. Cable-stayed bridges: Time History of the midspan displacement – c = 120 m/s

The comparison show how cable-stayed bridges are much more affected by the presence of the damage, since larger values of the bridge displacements with respect to the undamaged configuration are observe.

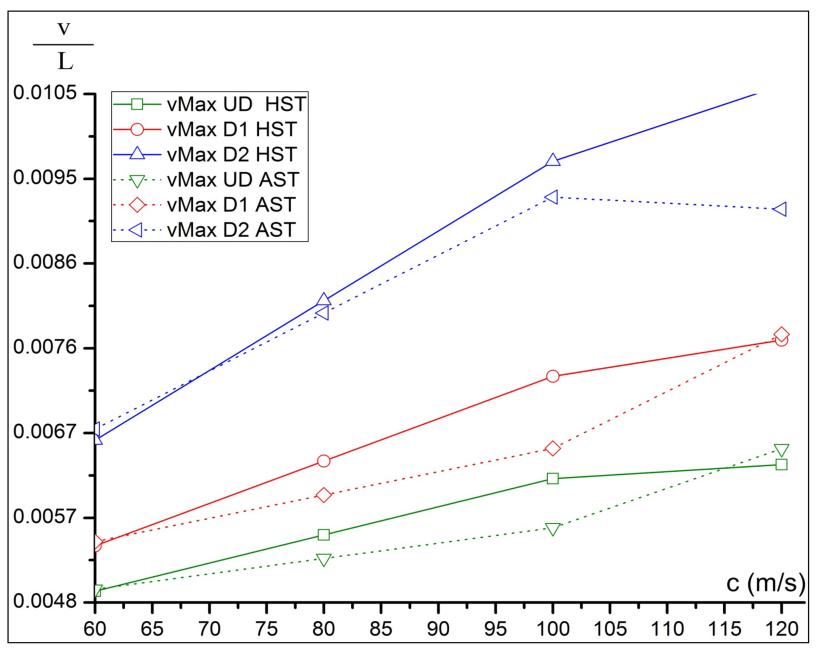



**Figure 7**. Cable-stayed bridge: Maximum vertical midspan displacement - influence of the speed

## REFERENCES:

1. Bruno D., Greco F., Lonetti P., Dynamic impact analysis of long span cable-stayed bridges under moving loads. *Engineering Structures*, **30**, 1160-1177 (2008).
2. Huang J., Wang R., Tang T., Formulation for cable state of existing cable-stayed bridge, *Proceedings of the 26th Southern Africa Transport Conference* (SATC 2007), 745-751 (2007).
3. BrunoD. , Greco F., Lonetti P., Pascuzzo A., Dynamic analysis of long span cable-stayed bridge under the action of moving loads, *International conference IBSBI 2011 - Innovations on bridge and soil-bridge interaction*, 513-520 (2011).