Effective framework for seismic analysis of cable-stayed-bridges Part 1: Modeling of the structure and of the seismic action.

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ABSTRACT: The spatial variability of the strong ground motion leads to significant aspects in the structural response of long-plan structures, such as long-span and medium-span bridges. In this study the structural responses of a prestressed concrete cable-stayed-bridge, with medium-length-span of 300 m, under 30 different seismic events, in both cases of synchronous and asynchronous motion at the base of the structure have been compared. Although the studies are principally conducted by a 3-D model of the structure, which one will call "frame model", developed using the finite element software Ansys, the structure has been studied using another model, which one will call "shell model", in order to evaluate how the differences in the modeling lead to different results. In the part 1 of the study, the authors put in evidence the differences among the models response in terms of modal characteristics of the principals modes. The comparison of the results obtained in both the models shows the similar behaviour of the two models, confirmed by the small differences in percentage of the values reached by monitored static and kinematical.

1 MODELING

1.1 *Modeling of the structure*

The bridge over the Cuiabà River is a prestressed composite cable-stayed bridge, built up from 2000 to 2001 in Mato Grosso, Brazil, with three spans of 71.75 m, 157 m, 71.75 m, for a total length of about 300m, by Rivoli S.p.A., after the design of Prof. Martinez y Cabrera.
The transversal section's total width is 22.4 m. The framework is made up of two longitudinal beams, with "π" section, with a width of 2.8 m and an height of 2.2 m, built up using segmental elements. Every beam has a width of 7.5 m and has a series of internal diaphragms, in order to make possible the anchoring of the cables. The piers have an height of 51.8 m over the foundation plinths, 39 m over the framework. Their section is approximately rectangular, with external maximum dimensions of 5 m x 3.5 m, constant over the height. The first seven cables are anchored in the lateral faces of the piers which, in that part, have an hollow section, while the two highest cables are anchored in the full section of the piers.

The total weight of the structure is $2.7*10^8$ N. All the monitored parameters, both static and kinematical, shown in Figure 2, have been considered in both sides of the bridge.
The studies are principally conducted by a 3-D model of the structure, developed using the finite element software *ANSYS*. The used model shown in Figure 2. is made up using frame elements, positioned in the centre of mass of the real elements, while the bridge's slab is modeled as a continuous, using shell finite elements. The cables are modeled with 3D spar elements having the feature of a bilinear stiffness matrix resulting in a uniaxial tension-only element; the stiffness is removed if the element goes into compression.

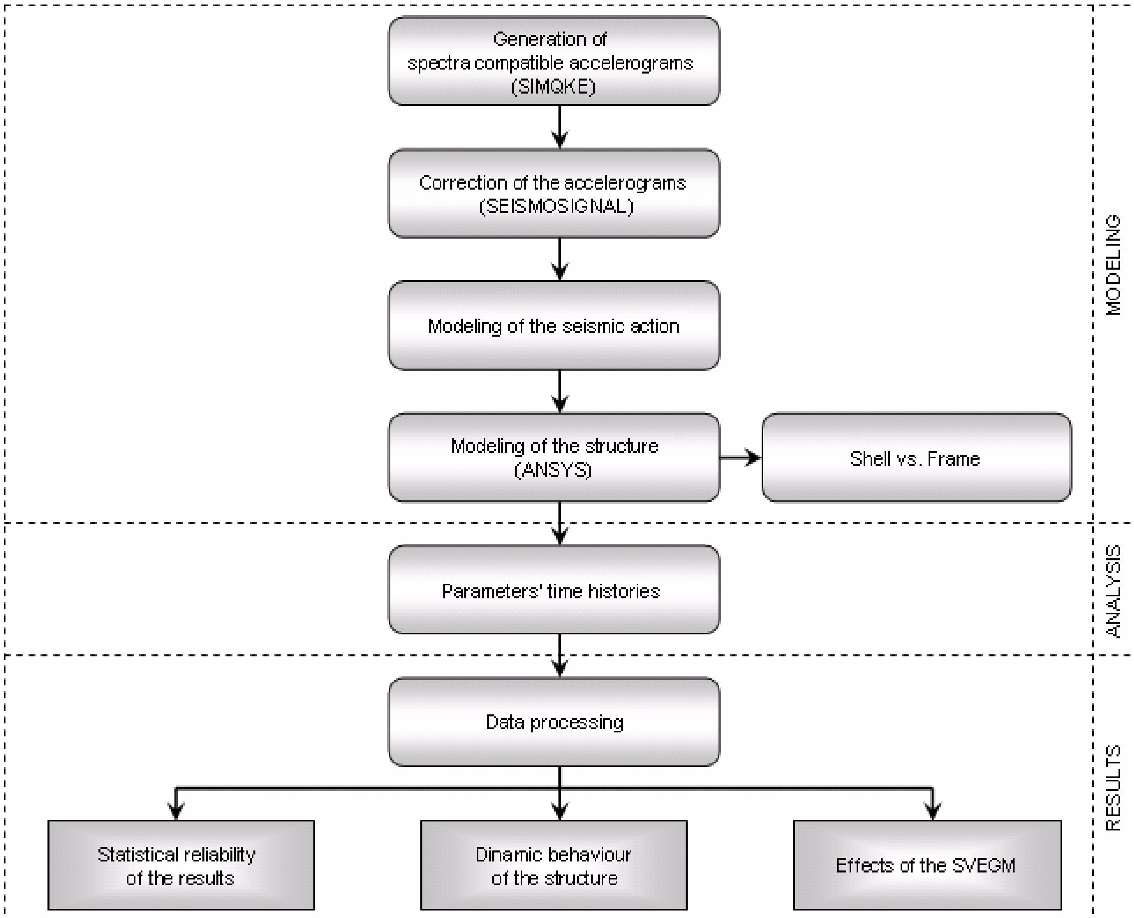
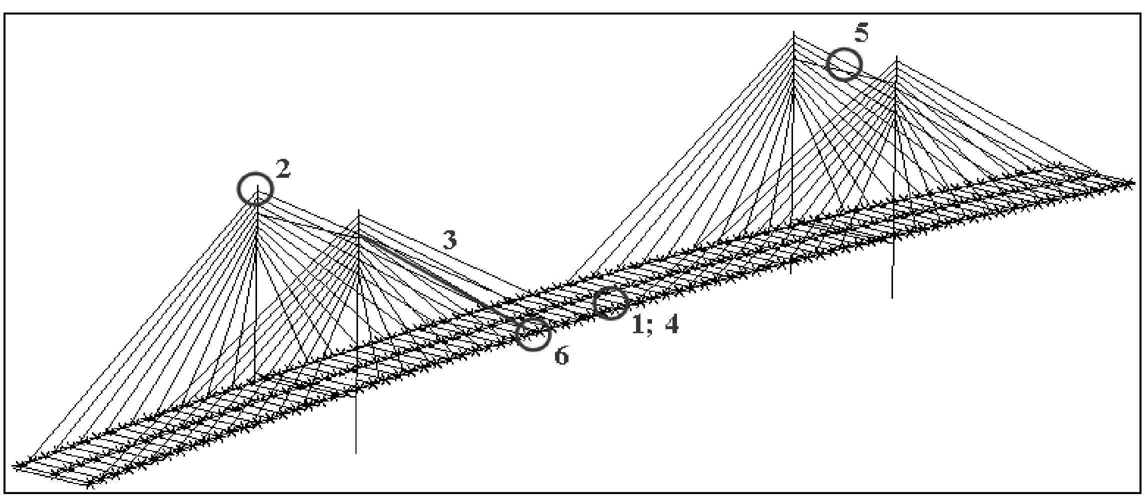


Figure 1. Studies done.



1. Vertical displacement in the middle of the span	4. Axial force in the middle of the long. beam
2. Transversal displacement of the pier	5. Axial force in the upper transverse
3. Axial stress of the 7^{th} cable	6. Moment at the connection of the 7th cable

Figure 2. 3-D model of the bridge and monitored parameters.

The piers have fixed ends at their bases, while the abutments are modeled with hinges restraints. The pretensions applied on the strands and on the cables are modeled by the imposition of an equivalent thermal variation, generating traction forces equal to the imposed ones.

It has been neglected the eccentricity between the barycentres of the longitudinal frames and the ones of the bridge's slab.

1.2 *Modeling of the seismic action*

The seismic input has been assigned by applying spostograms at the contact-points structure-soil, these are obtained by the double integration of 10 artificial accelerograms (Fig. 3), generated with the *Simqke* software [Gasparini, Vanmarcke, Liu, 1976], witch generates statistically independent artificial acceleration time-histories; the artificial motion is:

$$z(t) = I(t)\sum_{n} A_n \sin(\omega_n t + \varphi_n) \tag{1}$$

where *I(t)* is an opportunely chosen deterministic envelope function.
The ten generated accelerograms are different by a random phase angle, and are opportunely filtered by the use of the Butterworth filter, applied with the *Seismosignal* software. The response spectra of the accelerograms are spectra-compatible with the ones proposed by the Eurocode 8 for an "A-type" terrain.
Table 1 shows the principal characteristics of the accelerograms generated by the use of *Simqke* software.
It has been considered a propagation velocity of the seismic waves *Vs* in the soil of 1000 m/s: consequently, the time delay corresponding to this velocity, through the two abutments is about 0.3 s (Tab. 2).
The applied spostogram has been then translated in function of the time delay; the asynchronism is taken into account only in the longitudinal direction.

Table 1. Characteristics of the accelerograms generated by the use of *Simqke* software.

	ACCELEROGRAM	
PARAMETERS:	Horizontal	Vertical
Duration	30 s	30 s
Initial transitory	4 s	4 s
Final code	20 s	24 s
PGA	0.35 g	0.32 g
Number of Simqke cycles	10	10

Table 2. Time delay [s] of the arrival of the shear waves at the points of contact between structure and soil.

v [m/s]	Abutment 1	Pier 1	Pier 2	Abutment 2
1000	0.00000	0.07125	0.22825	0.29950

Therefore, the analysis conducted with the finite element program ANSYS, is a nonlinear time stepping analysis, done with the Newmark's method, with a Newton-Rhapson approach.
Although it can be used a modified Newton-Rhapson iteration, in which every step is divided into different sub steps; it has been used the original Newton-Rhapson method, because it converges more rapidly than the modified Newton-Rhapson iterative process, due to the expense of additional computation in the modified approach.
The values of the parameters, obtained in both the cases of synchronous and asynchronous motion have been compared, by considering the average of the 10 maximum registered values

reached in the time history of each parameter. It has been considered the time histories for each couple of horizontal and vertical generated spostograms, considered in its 3 combinations, maximizing the ground motion in the three orthogonal directions:

seismx + 0.3 seismy + 0.3 seismz

0.3 seismx + seismy + 0.3 seismz

0.3 seismx + 0.3 seismy + seismz

The monitored parameters responses have been evaluated in both the sides of the bridge.

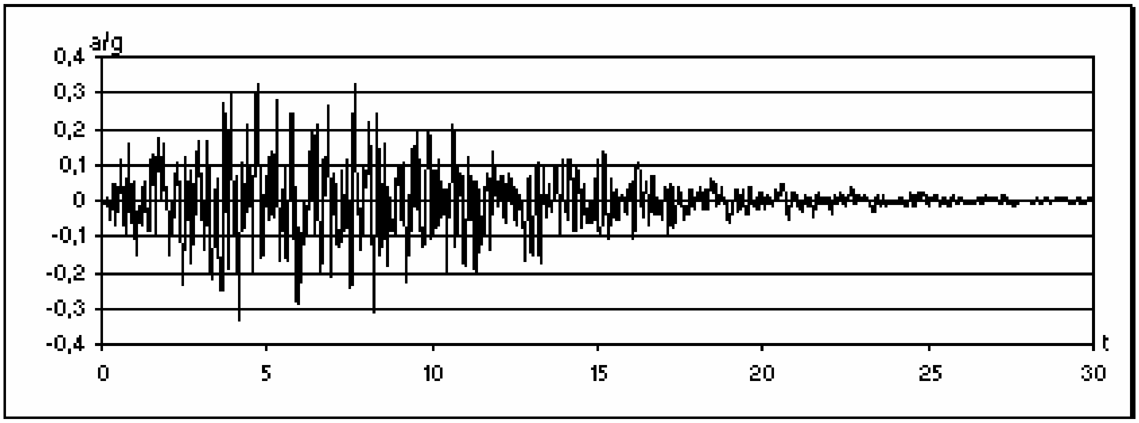


Figure 3. Examples of horizontal accelerograms used.

2 MODAL ANALYSIS' RESULTS

It has been performed a nonlinear time-step analysis, using the time histories of the applied spostograms. Significant errors may arise in a time history analysis, principally for two reasons [Chopra, 2000]:

1. The use of the tangent stiffness instead of the secant one;

2. The use of a constant time step, that may delay detection of the transitions in the force-deformation relationship.

Taking into account those two aspects, it's fundamental to choose an adequate type of analysis and an adequate time step Δt.
It is also necessary an adequate time step to describe the highly irregularities of the earthquake ground motion, also considering that the finest the time step is, the major is the required time for the analysis. After having compared the results obtained, under a same seismic event, in the cases of different time steps, one has chosen to use a time step of 0.05 s.
The time history of the differences of the values reached by the axial stress of the 7th cable, under a same seismic event, under two different time steps (0.01 and 0.05 s), is showed in Figure 4.
The comparison of the monitored parameters and the time required to perform the analysis shows that a time step of 0.05 s allows good results, similar to the ones obtained with a time step of 0.01, with differences of the maximum values less than 1%, together with a lower time of analysis.
The periods of the structures have been determined with a modal analysis, after having performed a static nonlinear analysis, in order to evaluate the modal parameters of the structure once reached the deformed configuration of the structure under the applied static loads.

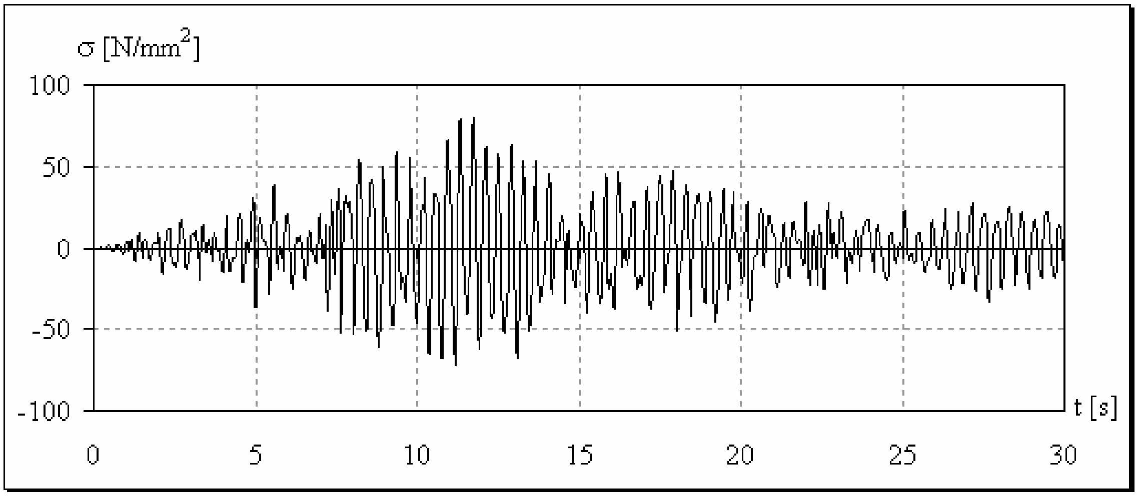


Figure 4. Time histories of the differences of the values of the 7th cable's axial stress, reached in the following two analysis:
Analysis 1: Time step = 0.01 s
Analysis 2: Time step = 0.05 s.

It has been compared the modal results obtained in the used model (shell model) with the ones obtained in a different model, which one will call "frame model"; the substantial difference between these two models is how the bridge's slab is modelled: a gridwork in the frame model and a two-dimensional continuous in the shell model.

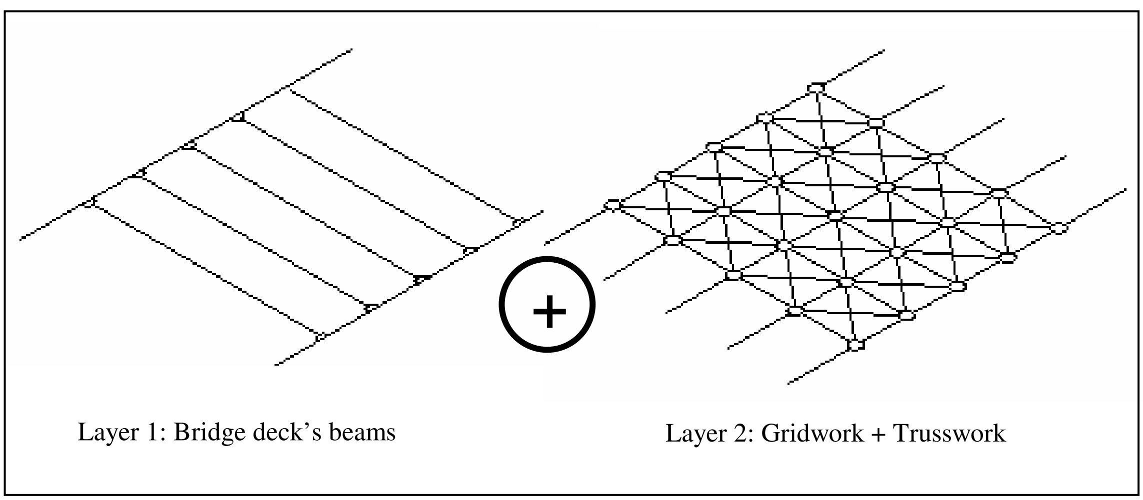


Figure 5. Layers of the 3D model of the bridge.

The geometric characteristics (area and inertia) of the gridwork have been evaluated by the application of the kinematical criterion: applying forces equipollent to the ones applied on the continuous, the corresponding deformations should be equal [Toniolo, Malerba, 1981].
Therefore, the framework has been built by the superimposition of the two layers separately obtained in the cases of forces parallel to the deck (trusswork) and perpendicular to the deck (gridwork) (Fig. 5). Finally, the complete deck framework has been implemented by the finite element software ANSYS by the orderly superimposition of the bridge's real beams (Fig. 6).

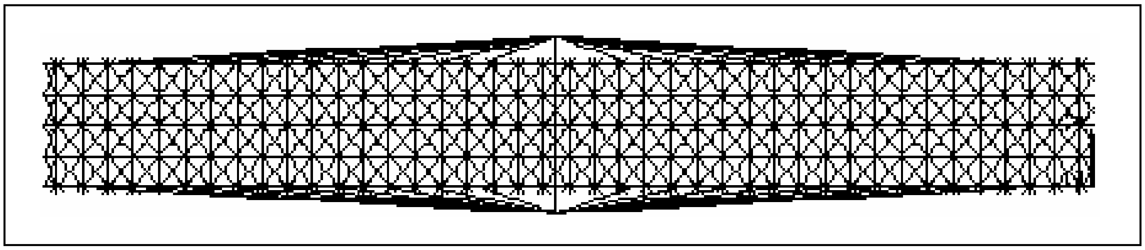


Figure 6. Assembled 3-D model of the bridge.

The comparison of the results obtained in both the models shows the similar behaviour of the two models, confirmed by the small differences in percentage of the values reached by monitored static and kinematical parameters (Tabs 3, 5.)

The modes exciting the 90% of the mass of the structure are 207. The characteristics of the first ten period of the structure are shown in Table 4.

Table 3. Comparison between the frame model and the shell one: vertical displac. in the middle of the span.

Vertical displacement in the middle of the span $U_{z,1}$ [cm]			
Load conditions:	Shell	Frame	%
1. Live on the three spans	-21.79	-21.77	0.12
2. Live on the central span	-26.92	-26.89	0.12
3. Live on the lateral span	3.83	3.83	0.01

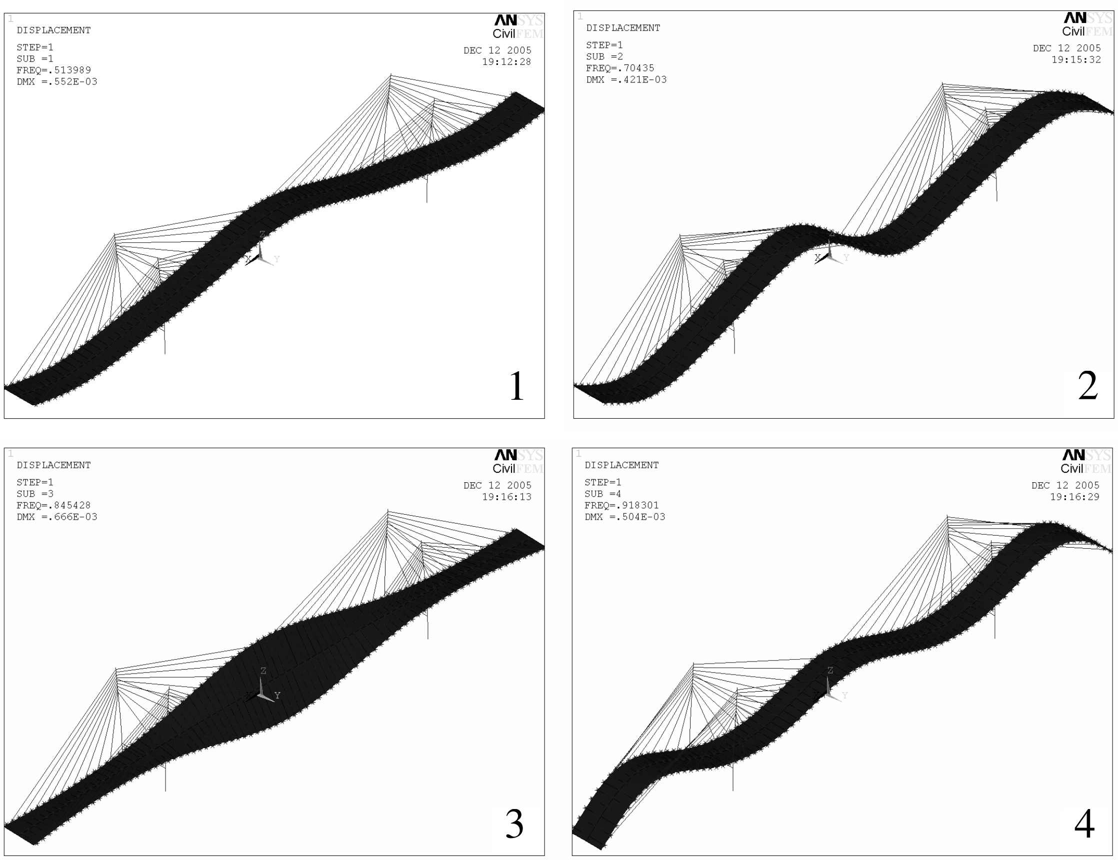


Figure 7. First four modal deformations.

In Figure 7 one can see the modal deformations of the first four modes. In Table 6 the periods and the percentage of excited mass of the modes exciting more than 15% of the mass of the structure are shown.

Table 4. Comparison between the frame model and the shell one: first 10 periods.

Mode	Shell	Frame	%
1^{th} mode	1.9456	1.9444	0.06
2^{th} mode	1.4197	1.4196	0.01
3^{th} mode	1.1828	1.1900	0.60
4^{th} mode	1.0890	1.0891	0.01
5^{th} mode	1.0025	1.0029	0.04
6^{th} mode	0.7943	0.7948	0.06
7^{th} mode	0.7781	0.7841	0.78
8^{th} mode	0.7533	0.7537	0.05
9^{th} mode	0.7182	0.7194	0.17
10^{th} mode	0.6571	0.6618	0.71

Table 5. Comparison between the frame model and the shell one.

Parameter	Shell	Frame	%
Pier's Transv. displ. Ux [m]	0.016376	0.01579	-3.58
Axial stress 7^{th} cable [N/m^2]	9.46E+08	9.43E+08	-0.27
Axial force upper transverse [N]	-4.34E+06	-4.17E+06	-3.97

Table 6. Modal characteristics of the modes exciting more than 15% of the mass of the structure

Mode	Period	Mass (percentage)					
		x	y	z	rotx	roty	rotz
4	1.089	0.0	0.0	27.8	4.4	0.0	0.0
5	1.003	0.0	0.0	0.0	0.0	40.5	0.0
8	0.753	0.0	42.9	0.0	53.1	0.0	0.0
9	0.718	0.0	0.0	0.0	0.0	0.0	21.2
18	0.379	31.7	0.0	0.0	0.0	2.2	0.4
32	0.241	0.0	0.0	0.0	0.0	0.0	21.5
35	0.199	0.0	29.4	0.0	3.1	0.0	0.0
40	0.185	38.4	0.0	0.0	0.0	2.5	0.4

3 CONCLUSIONS

In this study it is considered the dynamic response of a medium-span cable-stayed-bridge. The bridge is considered by different modelling. The seismic action is assigned by the application of different spostograms, obtained by the double integration of generated spectra compatible accelerograms.

In the part 1 one has described the numerical models of the structure and the modelling of the seismic action. In particular one has performed a SHELL model and a FRAME model to reproduce the behaviour of the bridge deck. The results in terms of:

- vertical displacement in the middle of the span
- transversal displacement at the pier
- axial stress in the 7^{th} cable

- axial force upper the transverse
- periods and percentage of excited mass

are compared and they show a good similarity among the models.

ACKNOWLEDGMENTS

Thanks to Professor D.M. Frangopol for his useful discussions on this study. The financial supports of University of Rome "La Sapienza" and COFIN2004 are acknowledged. Nevertheless, the opinions and the results presented here are responsibility of the Authors and cannot be assumed to reflect the ones of University of Rome "La Sapienza".

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