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HUMAN INDUCED VIBRATIONS IN A PEDESTRIAN TIMBER BRIDGE

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Abstract. *The mountain landscape fits well with slender timber structures. Among them timber bridges are becoming very popular, and the designers are extending the span toward the hundred meters threshold. For a long pedestrian timber bridge in the Italian Alps, the authors had the chance to acquire measurements of the structural response which were collected for periods of few days and repeated in different seasonal weather conditions. A previously developed elaboration of the collected data leads to the validation of a numerical model. In this paper, the numerical model is used to investigate the dynamic response of the structural system which, according to the current structural code, can be ignored along the design process.*

1 INTRODUCTION

The designer of pedestrian bridges is allowed to work in the static context. It is therefore quite frequent, that after the construction of the bridge, the crossing pedestrians feel vibrations, a structural monitoring program starts and, eventually, damping devices are added “a posteriori”.

Despite the objective fact that the bridge damping properties are difficult to be foreseen “a priori”, a few further steps in the design process could avoid such a negative post-construction duty. There is no problem in claiming for the availability of a numerical tool able to analyze the dynamic structural response; it will be generally based on a finite element discretization. But, the model itself offers serious areas of uncertainties, mainly in the modeling of the joint behavior and, as said, in the definition of the damping contribution of materials and links.

The next two steps, however, are even more critic in view of a successful design accounting for the expected vibration:

1) an accurate description of the load-cases causing vibration;
2) an adequate definition of the associated limit state.

The two items listed above are discussed in the next section. Then, a further section is devoted to the illustration of a case study in the Italian Alps. The results of the numerical analyses follow.

2 GOVERNING RELATIONS

2.1 Dynamic excitations

A pedestrian bridge is entering vibrations as a consequence of environmental and man-made excitation characterized by a variability in time and space of the load intensity.

The environmental actions are mainly those associated with wind and earthquake but this paper is focused on the man-made excitation which can be categorized into three types:

1) single or couple of persons running across the bridge;
2) people covering the whole deck and progressing along the bridge;
3) the passage of a service car associated to the maintenance plan (of course this last action will only be possible when a direct access exists, i.e., the bridge can be accessed by ramps and not by stairs and/or elevator).

2.2 Limit state

An overview of the proposed strategies introduced to limit the vibration amplitudes in pedestrian bridges is reported in the books [1, 2]. Among these suggestions, the most appealing one is extracted from book [2] and can be summarized as it follows.

One considers a harmonic component of the motion:

$$x(t) = X \cos(2\pi f t)$$

where X denotes the amplitude of the vibration and f its frequency. It is well known that the amplitude of the associated acceleration is $X(2\pi f)^2$ from which one derives the quantity:

$$Z = [X(2\pi f)^2]^2 / f = 16\,\pi^4 X^2 f^3$$

which can be made dimensionless by dividing it by the reference value of $Z_0 = 10$ mm^2/sec^3.

A dimensionless vibration measure is then introduced as:

$$S = 10 \log (Z/Z_0) = 22 \log (X^2 f^3)$$

which is the logarithm of a dimensionless quantity; for the sake of specification, one introduces a fictitious unit called "vibrar".

By plotting a log-log diagram in the frequency-amplitude plane, the lines of constant $X^2 f^3$ are straight lines oriented as in Figure 1. It is a praxis to give evidence to the lines/domains corresponding to:

a) S= 17.5 vibrar: the domain from the origin to this line is regarded as a no-damage domain;
b) S= 40 vibrar: when in this layer, minor damage (e.g., plaster cracks) is detected;
c) S= 72.5 vibrar: when in this layer, damage to load-bearing parts is detected;
d) the outer domain is associated to the bridge destruction event.

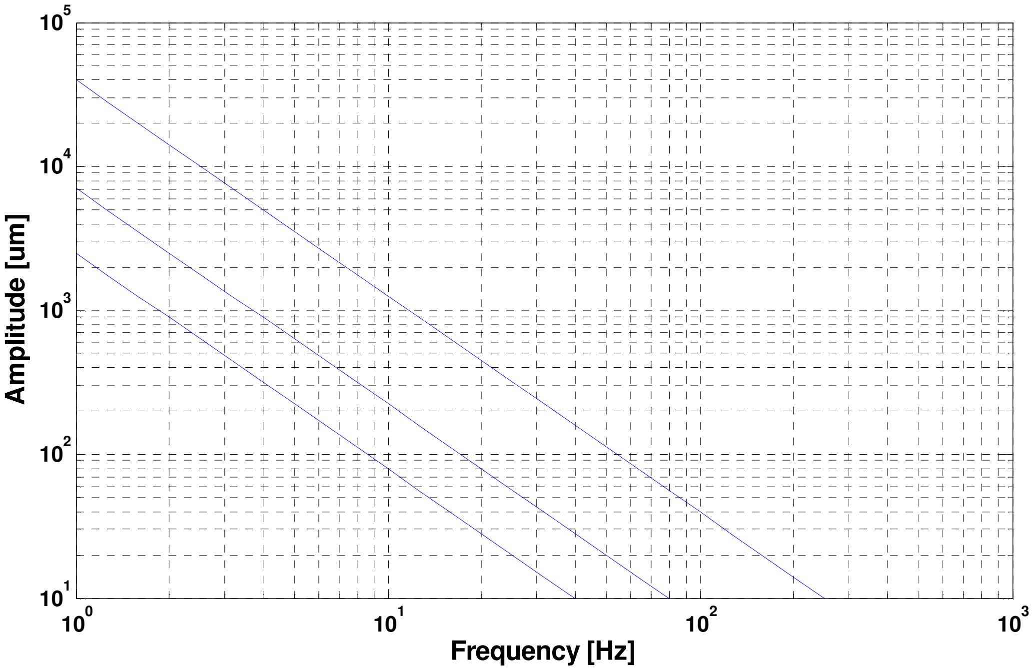

Figure 1 – The frequency vibration plot, with emphasis on the 4 damage zones: I – the no-damage zone contains the left-bottom corner; II – the straight line at 17.5 vibrar limits the possible plaster cracks area; III - the straight line at 40 vibrar limits the zone of possible damage to load bearing structural parts; IV - the straight line at 72.5 vibrar limits the zone of damage to load bearing structural parts

3 THE CASE STUDY

Earthquake engineers are familiar with the word "Tolmezzo", the name of a large village in Italy, where earthquake records were collected during the seismic events of spring and late summer in 1976. Nearby there is the largest natural lake in the area called the "Lake of the three municipalities" (or, simpler, the "Cavazzo Lake") with an emissary channel which was a barrier for the jogging activity across the surrounding park. Thus the two sides were linked by a pedestrian bridge assembling steel and timber components (Figure 2 and 3) to be harmonized with the surrounding environment. Instead, reinforced concrete was limited to the foundation components.

Figure 2 – A general view of the bridge which represents the case study of this manuscript.

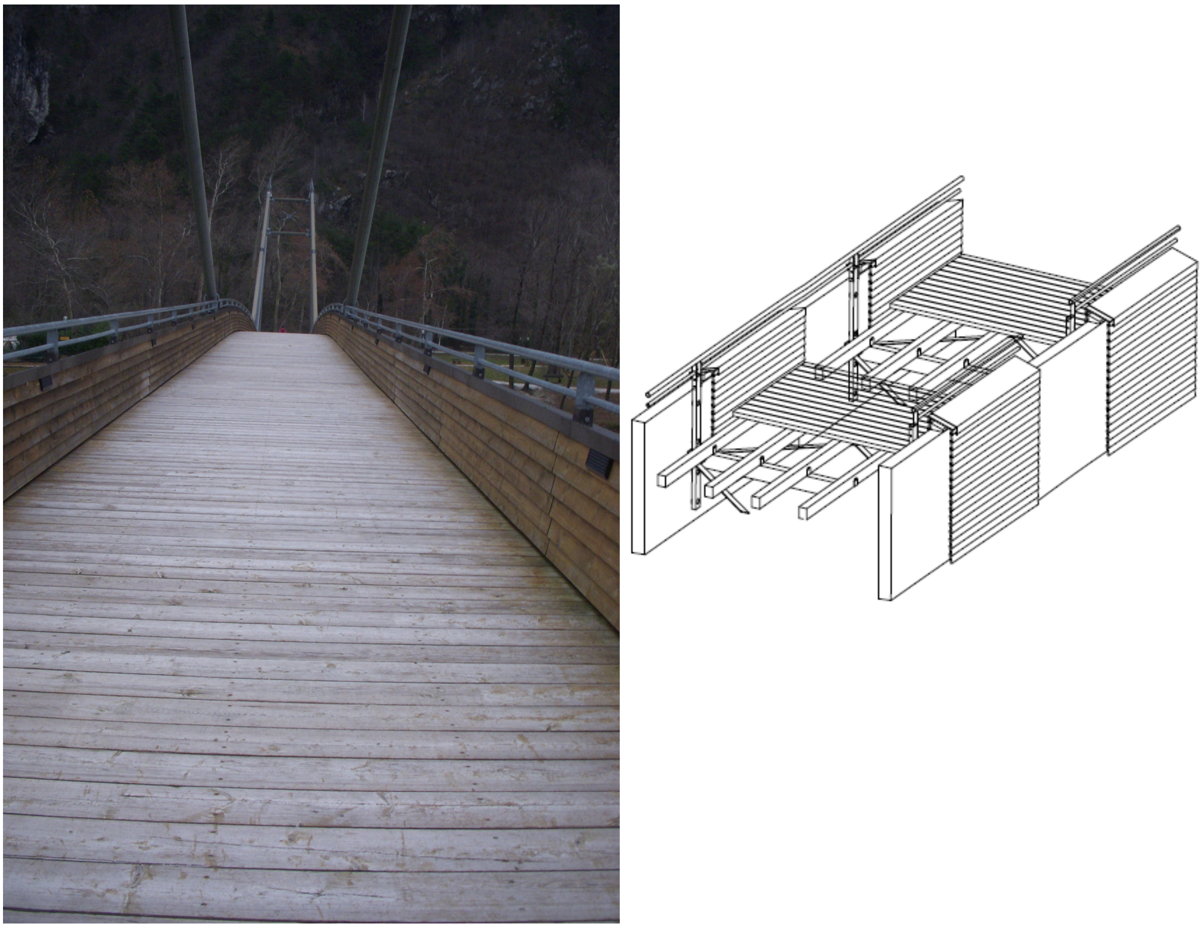

Figure 3 – Details of the walking path across the bridge.

Despite the bridge seems to be of a cable-stayed type, there are no cables. The static scheme consists of a vertical antenna on both ends of the bridge supporting the deck at one third of the span (from each side) by tubular steel elements. The span is 83 m and the double-beam deck width is 4 m, of which 3.22 m represents the free crossing width.

Glued laminated timber of high strength GL28c is adopted for all structural elements, except for the lateral cladding, made by larch, and the walking deck, which is made by glued laminated timber of type GL24c [3-4].

Two lateral curved beams of section 20 by 194.1 cm (Figure 3), with neoprene supports at the ends, are linked by a sequence of H-shaped, transversal tubular steel elements of type S355JR. The transversal elements of length 3.64m are braced and support five timber beams on which the walking platforms are mounted.

The beams are anchored at the thirds to the steel antennas on the two sides. The height of the antennas is 15 m, and they are made by elements of tubular steel section of external diameter 457. mm and thickness 14.2 mm. The link antenna-beam is made by elements of tubular steel section of external diameter 273 mm and thickness 8 mm.

The pedestrian bridge is regarded as category 3, which means that two live loads have to be considered:

- $q_{1.d}$ = isolated load of 10kN over an area of 70 by 70cm;
- $q_{1.e}$ = distributed load of 4.0 kN/m^2 resulting from a dense crowd.

In addition, the effects of snow, wind and earthquake are taken into account.

In order to have a numerical model of the system, a finite element model was created within the software environment Mentat-Marc [5], by paying particular attention to the wood material modelling [6]. The behavior of the link between the longest oblique elements and the deck is not a priori known, and for this reason a preliminary model of the steel portal was implemented to clarify this aspect. The numerical model of the whole bridge is shown in Figure 4.

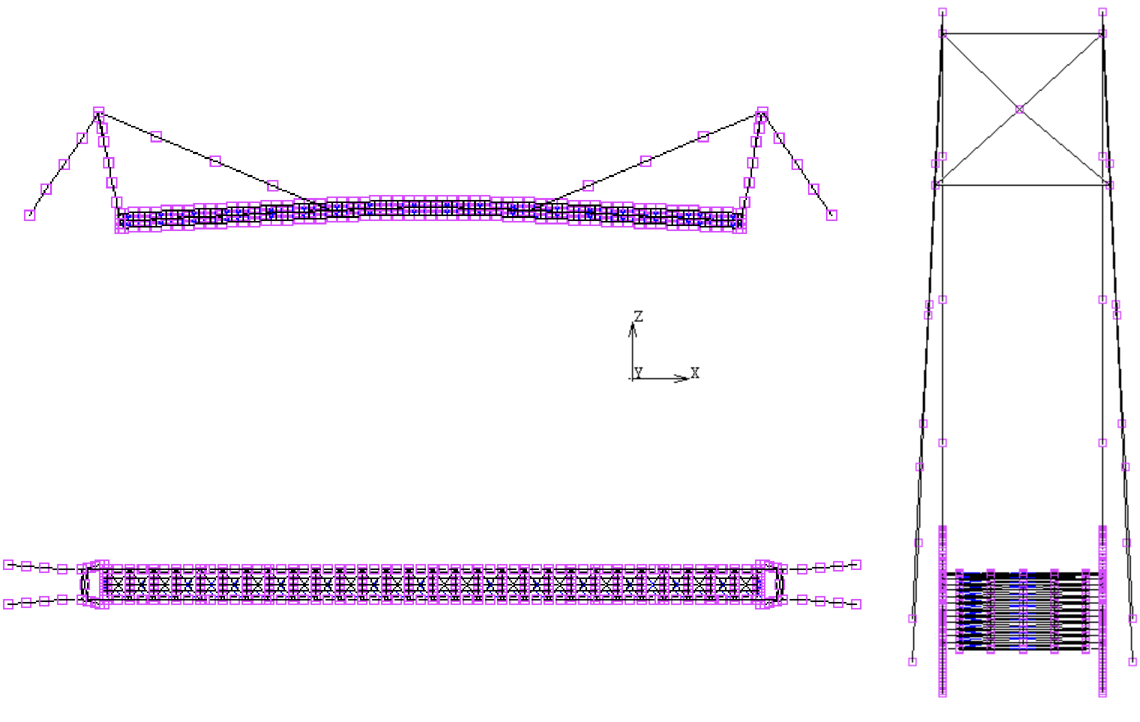

Figure 4 – Beam elements and shell elements used to discretize the pedestrian bridge.

4 SELECTED RESULTS FROM THE DYNAMIC ANALYSES

The results of the analyses reported in this section pursue the goal of showing the feasibility of the simulation. A deeper study on the damping properties would be required in order to produce more realistic time histories of the response.

First, the analysis is conducted by considering the crossing of a car of mass 2 t along the bridge at a speed of 10m/s. The load case is a sequence of pulses at 29 different positions along the bridge. Moving masses are also added to refine the dynamic analysis. Just for sake of illustration, the single pulse in position 12 would produce the response summarized in Figure 5.

The dynamic analysis is conducted in 1000 steps, with a time step of 0.18 sec. The response in terms of vertical displacement time history is plotted in Figure 6 for the node 101, the one of the maximum deflection under pulse 12.

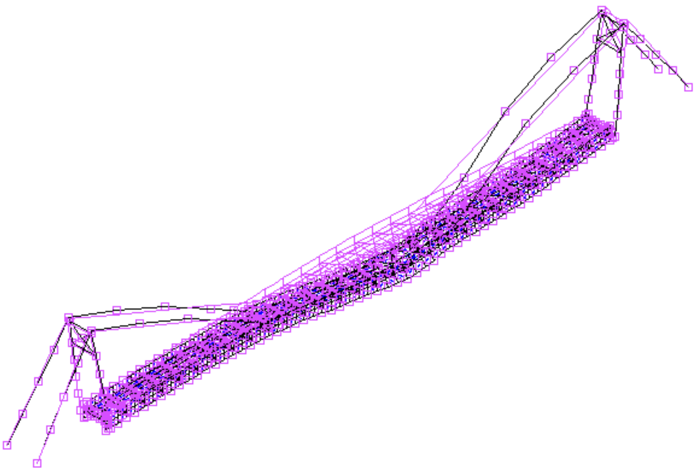

Figure 5 – The bridge response under the action of pulse 12, in a global sequence of 29 by which the crossing of a car is simulated.

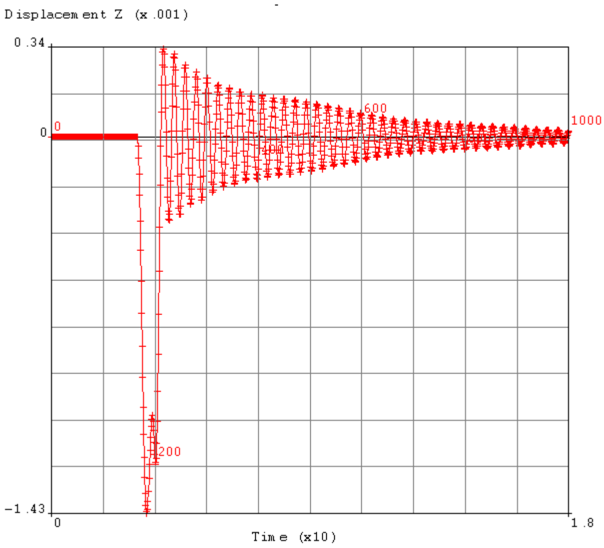

Figure 6 – Time history of the vertical displacement (in meters) of the bridge node (labeled 101) of maximum deflection in Figure 5, under the action of pulse 12 (with time expressed in seconds).

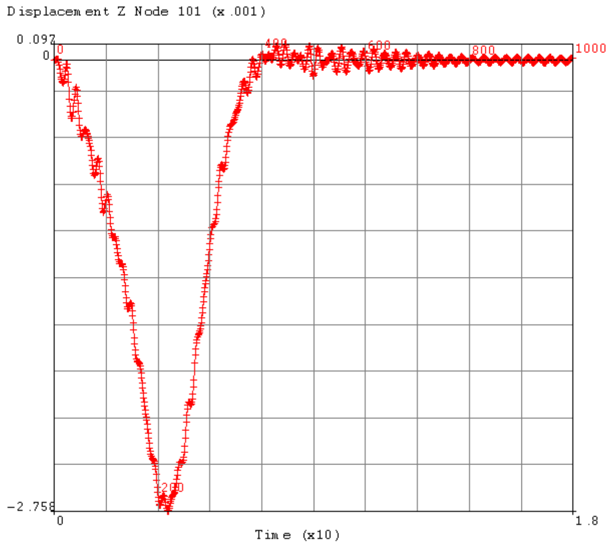

Figure 7 – Time history of the vertical displacement (in meters) of the bridge node 101 during the car crossing (with time expressed in seconds).

The car crossing analysis, i.e., the response to the entire sequence of 29 pulses, is summarized, again for node 101, in Figure 7. It is seen that, after a peak of 2.75 mm, a free oscillation of 0.1 mm amplitude starts and then decays with a damping of 1%, which was introduced as default in the model. A comparison with experimental data will then allow a more realistic simulation. The mode associated with this free oscillation is provided in Figure 8; it corresponds to a natural frequency of 2.7 Hz. The amplitude of the free oscillation is marked in the diagram of Figure 1 by a cross as shown in Figure 9.

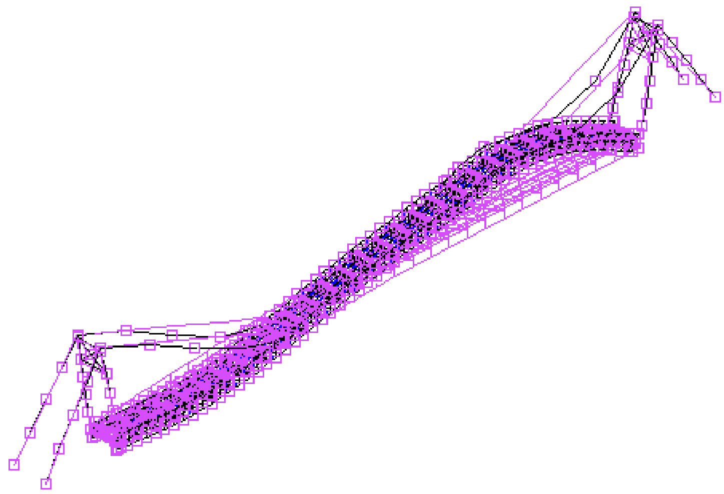

Figure 8 – The damped free oscillation after the car crossing: frequency 2.7 Hz. and damping 1%.

It is worth noticing that apart from the fact that the masses are one tenth of the one of the car, the car crossing load is also a rough representation of two persons running along the bridge (since the walking kinematics is ignored). Thus, such a passage would generate a free oscillation of amplitude 0.01 mm, as from Figure 7 after dividing by 10.

The case of people clustering on the bridge and moving along the same direction required a fully different analysis whose results are summarized in Figures 10 and 11. A second point is then added by this analysis in Figure 9. The walking is simulated by a moving half of the weight from the left to the right node and vice versa with frequency 1Hz. This also gives rise to an external horizontal component of 0.1 time the weight in the overloaded node.

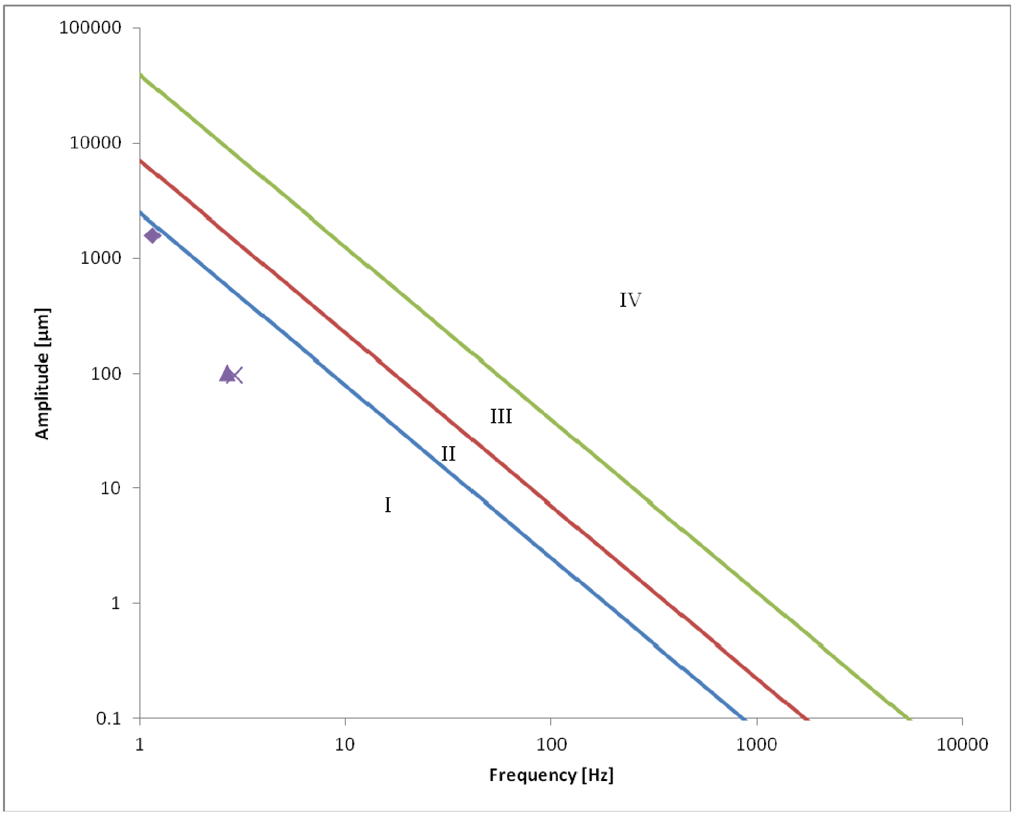

Figure 9 – Free oscillations induced by car and persons crossing: representations in the diagram introduced as Figure 1. Triangle and cross correspond to the car crossing and a vibrodine test simulation: the vibration is along the vertical axis. The third mark corresponds to the dense clustering of people and reports the vibration along the transversal axis.

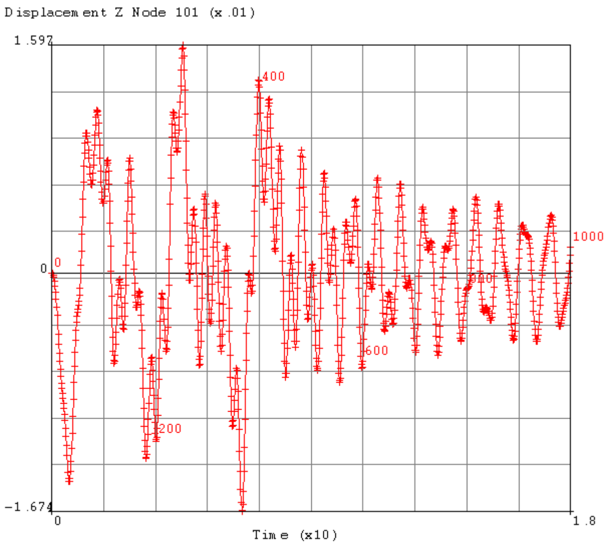

Figure 10 – The vertical displacement (in meters) of the bridge node labeled 101 during the dense clustering of people crossing (time expressed in seconds) for one half of the time history and then at rest on the bridge.

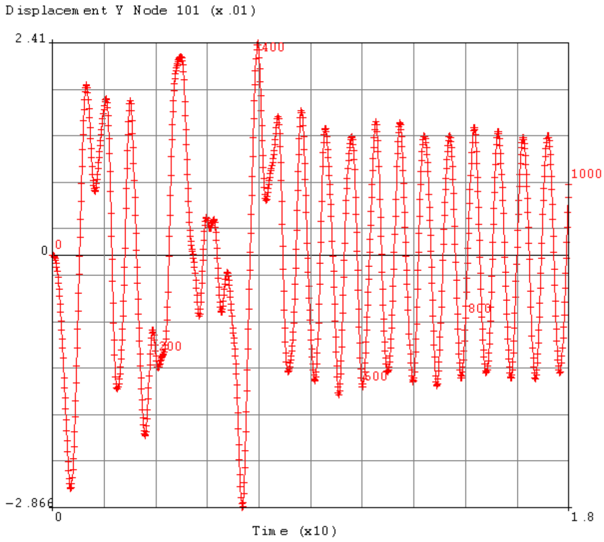

Figure 11 – The horizontal displacement (in meters) of the bridge node (say 101) during the dense clustering of people crossing (time expressed in seconds) for one half of the time history and then at rest on the bridge. This response, scaled by 10, is used to add the corresponding mark in Figure 9.

The result of the analysis, carried out without fixing the damping which is quite low, is exploiting many of the resource of the structural system, but one is moving 1000 persons along the bridge in a very aggressive way, both in terms of vertical load variation and of horizontal action.

The points representative of this situation of distributed crowd is first scaled to one tenth and then plotted (by a diamond) in the diagram of Figure 1, as shown in Figure 9. A third point obtained as free vibration after the numerical simulation of a vibrodine test (in the vertical direction) is also plotted by a triangle.

5 CONCLUSIONS

On a numerical model calibrated by the available experimental data, two design steps, additional to the static procedure required by structural codes in Europe, are discussed. The first one sees the development of a dynamic analysis; the second one the translation of the dynamic analysis results into a diagram of expected performance.

In this paper, one investigates the feasibility of a procedure whose goal is to avoid that feeling vibration when crossing the realized bridge statically designed results in the designer being obliged to a costly experimental campaign. A preliminary estimation of the expected vibration levels would immediately classify the performance as allowable or non-allowable.

It is worth noticing however that in this process the calibration of the model parameters and mainly of the damping properties [7-9] will always suggest a dynamic testing of the bridge at the proofing stage.

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