

WALKING-INDUCED VIBRATION OF A FOOTBRIDGE

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Keywords: Structural Dynamics, Footbridges, Dynamic loads, Pedestrian-Structure Interaction, Vibration, Design-Criteria, FE-model.

Abstract. *In the last years, the need for structures able to link the functional and aesthetic role has led engineers and architects to design footbridges characterized by long span, light materials and increasing slenderness. The low ratio between permanent and variable loads makes recent footbridges more sensitive to the dynamic loads, such as the forces transmitted by pedestrian. Excessive vibrations may arise when the bridge natural frequencies are very close to those characterizing the human activities: walking, jogging and running. This issue became relevant after the Millenium Bridge inauguration, when an excessive lateral sway motion was triggered by the synchronization between bridge and pedestrian movements.*

The dynamic behavior of a lively footbridge over-passing the Serio river near Seriate (about 50 km far from Milan), Italy, is investigated in this paper. The suspension bridge, 63.75 m long, is composed of a timber deck on a steel grid. The footbridge model, based on the as-built design data, is implemented in the ANSYS framework. The numerical frequencies computed through the FE model match those identified from the results of the experimental campaign of ambient vibration measurements in a fully satisfactory way. Since the bridge displays several frequencies in the range excited by human activities, its response to crossing pedestrians is investigated. The dynamic interaction pedestrians-footbridge is analyzed for different classes of traffic with two approaches. First, the FE model developed in ANSYS is excited by the distributed harmonic load model for a pedestrian stream, applied on the bridge coherently with the corresponding mode shape, as suggested by the Hivoss Guideline. Second, an ad-hoc developed numerical code is adopted to compute the bridge dynamic response to moving forces. This code reads as input data the structural matrices computed in ANSYS and integrates the equations of motion of a system in which the pedestrian is modeled as a constant vertical travelling force along the footbridge deck. The vibration serviceability under the vertical component of pedestrian load is assessed by comparison to comfort criteria. The results of the Hivoss guideline show accelerations exceeding the value of comfort. The transient analysis predicts lower values within the limits. The two sets of values can be interpreted as an upper and lower bound of the actual response.

1 INTRODUCTION

The Seriate footbridge at study (Figure 1), about 64.0 m long, follows the recent aesthetic trend of light and slender footbridges. The structural solution includes a slender deck supported by a main spatial system of suspension cables and equipped by two lateral stabilizing cables with opposed curvatures, one on each side. At the bridge ends, two steel frames support the suspension cables and the backstays. Before the opening, the footbridge was subjected to dynamic tests, performed by the Laboratory of Vibration and Dynamic Monitoring of Structures (VibLab), Politecnico di Milano [1]. The tests included both operational modal testing and human-induced vibration due to the crossing of different groups of pedestrians, moving at several step frequencies. The dynamic behaviour of the structure turned out to be characterized by frequencies within the range of values perceived by human beings.

In a previous work [2] a numerical Finite Element (FE) model was developed within the ANSYS framework, to reproduce the experimental dynamic properties. In this work the FE model, capable to match the experimental frequencies, is adopted to simulate the pedestrians-structure dynamic interaction. In a first study on the vibration serviceability performance due to human-walking activity, a harmonic analysis is carried out with ANSYS, according to the Hivoss guidelines. Two different traffic classes are considered, coherently with the role of the bridge, connecting two cycle routes located in the "Serio Park". Subsequently, the pedestrians-footbridge dynamic interaction is investigated in more detail with an ad-hoc developed numerical Fortran code, named INTER: the structural matrices computed by ANSYS are read in input and the bridge equations of motion are integrated. Forcing terms are given by the pedestrians' weight travelling at a constant speed v on the deck, along straight trajectories parallel to the bridge longitudinal axis. The moving forces, simulating groups of pedestrians, can act with different spatial configurations, entering and exiting the bridge at different time instants.

The accelerations due to walking, predicted by the two numerical codes, have been compared to the comfort criteria prescribed by Hivoss guideline in order to estimate the vibration serviceability assessment under the vertical component of pedestrian load.

2 THE SUSPENSION FOOTBRIDGE

2.1 Bridge description

The "Seriate footbridge" (shown in Figure 1a), 63.75 m long, connects two cycle routes in the "Serio Park" (close to Milan, Italy). The timber deck on a steel grid (Figure 1b) has a width ranging between 2.5 m at the entrance and 5 m at midspan; the longitudinal steel girders are slightly curved with a rise of 1.3 m. The transverse beams of the steel grid (Figure 1b), spaced 1 m apart, fall into two categories: the main girders, connected to the hangers, with a tapered section and the secondary ones, with an IPE 120 cross-section. The stringers are a pair of IPE 330 beams at the edges, and a central beam with a circular section ($\phi=298.5$ mm), deemed to stabilize the deck on the horizontal plane. A series of X-braces, shown in Figure 1b and connecting the main transverse girders, provides stiffness in the horizontal plane. The timber deck (shown in Figure 1b) has only a minor structural role, providing the walking surface and support for the pedestrian. The ends of the transverse main girders are crossed by stabilizing cables, whose sliding in the longitudinal direction is allowed by the interposition of a polymeric layer between the two contact surfaces.

The suspension system supporting the deck is composed of:

- steel main pylons, slightly inclined with respect to the vertical plane, and arranged in pairs creating an A-shaped portal. The portals support the suspension system and the

backstays cables as shown in Figure 1a;

- 2 main suspension cables, $\phi = 60$ mm, supporting the longitudinal girders through 42 vertical hangers, of diameter 16 mm;
- backstays cables, $\phi = 60$ mm, connecting the pylons top to the ground;
- 2 stabilizing cables of opposed curvatures $\phi = 40$ mm.

The suspension system is not symmetric about the vertical plane crossing the longitudinal bridge axis: the two hangers connected to the same main transverse girder have different length and the main cables do not follow the same parabolic function. All the cables were pre-tensioned during construction.



Figure 1. Seriate Footbridge: (a) overall view; (b) footbridge deck, detail of longitudinal and transverse beam.

2.2 FE model

The bridge model, shown in Figure 2, is based on the as-built design data and has been implemented within the ANSYS framework. The girders of the steel grid, the pylons and the hangers are modeled with "BEAM 188". This beam element, with 6 degrees of freedom (DOFs) at each node, includes shear-deformation effects according to Timoshenko beam theory. Cables and braces are modeled with "LINK 180", a spar element transmitting axial force only, with 3 DOFs at each node [3]. The timber planks and the handrails are modeled as lumped masses applied on steel grid without any contribution to the overall stiffness; their weight is included in the dead load. The boundary conditions of the footbridge model, as the constraint conditions between adjacent elements, are inferred from the technical drawings.

The cables geometric stiffness and the variation of configuration associated to dead loads have been accounted for in a preliminary non-linear static analysis. Once the model correctly reproduces the design value of tension in cables and the deformed geometry, the modal analysis has been performed. Further details on the derivation of the FE model are found in [2].

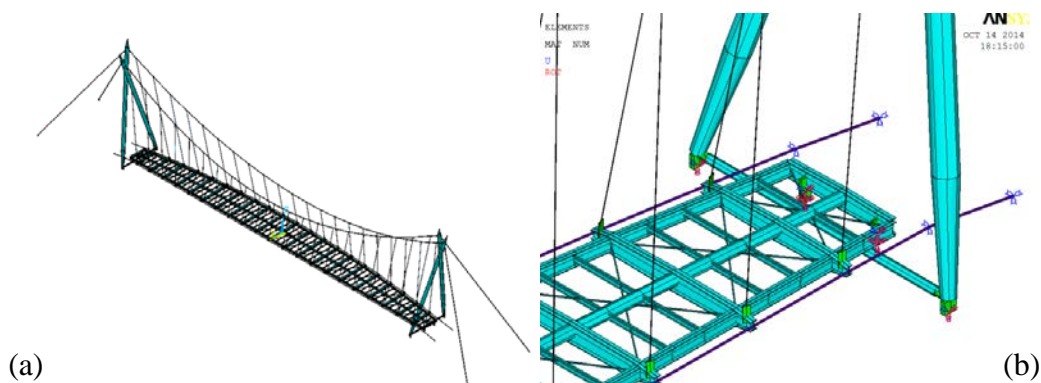


Figure 2. Finite element model of "Seriate Footbridge": (a) overall view; (b) detail of steel grid.

The results of modal analysis, in terms of frequencies and mode shapes, are compared with those obtained through the experimental campaign [1] in Table 1, also listing the percentage errors on the frequencies and the MAC (Modal Assurance Criterion, [4]) values computed on the mode shapes. The good correlation between the numerical and experimental dynamic properties makes the FE model a solid base for further numerical calculations.

| Mode | Mode Type | Frequency | | | MAC |
|------|----------------------|----------------------|-------------------|----------------------|-------|
| | | Experimental (Hz) | Numerical (Hz) | ε (%) | |
| 1 | Vertical-flexural | 1.025 | 1.079 | 5.22 | 0.995 |
| 2 | Vertical-flexural | 1.475 | 1.565 | 6.07 | 0.994 |
| 3 | Vertical-torsional | 1.924 | 1.997 | 3.77 | 0.908 |
| 4 | Vertical-transversal | 1.953 | 2.109 | 7.99 | 0.842 |
| 5 | Vertical-flexural | 2.168 | 2.311 | 6.58 | 0.984 |
| 6 | Vertical-torsional | 2.754 | 2.635 | -4.33 | 0.975 |
| 7 | Vertical-flexural | 2.861 | 2.827 | -1.17 | 0.996 |
| 8 | Vertical-torsional | 3.691 | 3.645 | -1.24 | 0.957 |
| 9 | Vertical-flexural | 4.121 | 4.076 | -1.10 | 0.988 |
| 10 | Vertical-torsional | 4.385 | 4.409 | 0.55 | 0.982 |
| 11 | Vertical-flexural | 5.645 | 5.512 | -2.36 | 0.958 |
| 12 | Vertical-torsional | 6.006 | 5.875 | -2.18 | 0.982 |
| 13 | Vertical-flexural | 7.217 | 7.255 | 0.52 | 0.976 |
| 14 | Vertical-torsional | 7.490 | 7.394 | -1.28 | 0.983 |

Table 1. Comparison between experimental and numerical results

3 VIBRATION SERVICEABILITY DUE TO WALKING: HIVOSS GUIDELINE

The first assessment of the serviceability conditions of the footbridge has been carried out following the Hivoss guideline (HG). For the sake of completeness, the main features of the HG of interest in this work will be summarized in the following. The guideline indicates the critical ranges for the natural frequencies f of the footbridges subjected to pedestrian walking as it follows [5]:

- for vertical and longitudinal vibrations: $1.25 \text{ Hz} \leq f \leq 2.3 \text{ Hz}$
- for lateral vibration: $0.5 \text{ Hz} \leq f \leq 1.2 \text{ Hz}$

Moreover, footbridges with natural frequencies in the range $2.5 \text{ Hz} \leq f \leq 4.6 \text{ Hz}$ might be excited to resonance by the second harmonic of pedestrian loads [4]. In this case, the critical frequency range for vertical and longitudinal vibration expands to $1.25 \text{ Hz} \leq f \leq 4.6 \text{ Hz}$ [4]. Lateral vibrations are not affected by the second harmonic of pedestrian loads. The "Seriata footbridge" shows 10 natural frequencies in the critical range. In particular, the first frequency is in the range characterizing the lateral vibration, while the remaining ones might produce vertical and longitudinal vibrations. It must be specified that, starting from the fifth mode, the 2nd harmonic of pedestrians load has to be considered.

The HG [5] proposes a load model to simulate the walking activity induced by a group of pedestrians. A uniformly distributed harmonic load $p(t)$ [N/m²] represents the equivalent pedestrian stream:

$$p(t) = P * \cos(2\pi f_s t) * n' * \psi \quad (1)$$

In eqn. (1), $P * \cos(2\pi f_s t)$ is the harmonic load due to a single pedestrian. P is the component of the force due to a single pedestrian ($P = 280 \text{ N}$ for vertical load, $P = 140 \text{ N}$ for longitudinal load and $P = 35 \text{ N}$ for lateral load) with a walking step frequency f_s . The step frequency f_s is assumed equal to the footbridge natural frequency under consideration; n' is the equivalent number of pedestrians on the loaded surface S . The reduction coefficient ψ takes into account the probability that the footfall frequency approaches the critical range of natural frequencies under consideration. HG adopts two load models to calculate the response of the footbridge depending on pedestrian density and traffic class (Table 2) [5]:

- load model for TC1 to TC3 (density $d < 1.0 \text{ P/m}^2$)
- load model for TC4 to TC5 (density $d \geq 1.0 \text{ P/m}^2$)

| Traffic class | Density (pedestrian/m ²) | Description |
|---------------|---|---------------------|
| TC1 | 15P/(B×L) | Very weak |
| TC2 | 0.2 P/m ² | Weak |
| TC3 | 0.5 P/m ² | Dense |
| TC4 | 1.0 P/m ² | Very dense |
| TC5 | 1.5 P/m ² | Exceptionally dense |

Table 2. Pedestrian traffic classes and density (from [5])

In the first class TC1 only 15 pedestrians are moving on the whole area $B \times L$ of the footbridge. The equivalent number of pedestrians n' depends on the structural damping ratio ξ , on the loaded surface S and on the number of pedestrians n on the surface ($n = S * d$):

$$\text{TC1 to TC3} \quad n' = \frac{10.8\sqrt{\xi * n}}{S} \quad \left[\frac{1}{\text{m}^2} \right] \quad (2)$$

$$\text{TC4 to TC5} \quad n' = \frac{1.85\sqrt{n}}{S} \quad \left[\frac{1}{\text{m}^2} \right] \quad (3)$$

The value of the reduction coefficient ψ is depicted in Figure 3, for the two situations in which the excitation frequency is in the first or second harmonic of the pedestrian load.

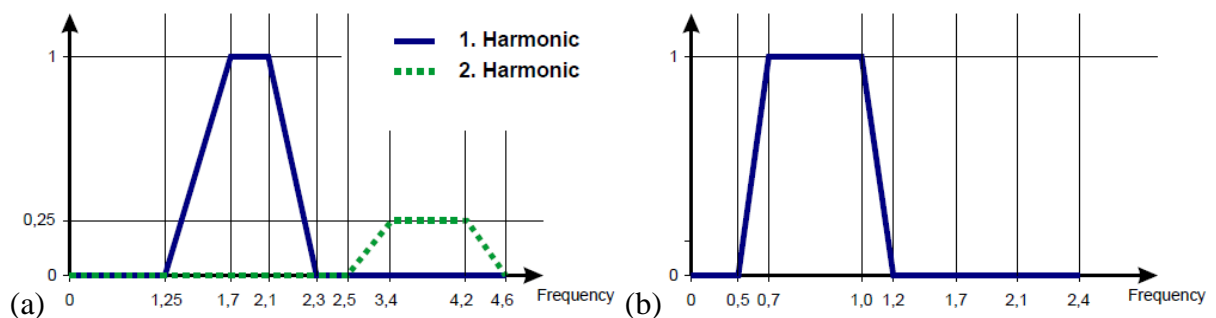


Figure 3. Reduction factor: (a) vertical and longitudinal load, (b) lateral load (from [5])

The direction of the harmonic load is the same of the half-waves characterizing the mode shapes associated to the natural frequency considered in the dynamic analysis (Figure 4).

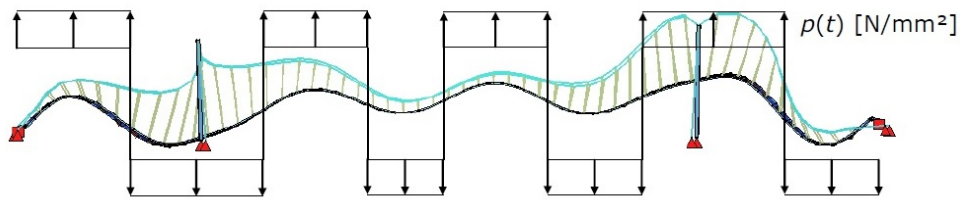


Figure 4. Application of a harmonic load according to mode shape (from [5])

The fulfillment of vibration serviceability conditions requires that the computed values of acceleration stay within the acceleration limits corresponding to four classes with a decreasing degree of comfort (Table 3). Two different traffic classes are here considered: TC1 (very weak traffic), associated to a comfort class CL1, and TC2 (weak traffic), associated to a comfort class CL2. Since the footbridge is located in a country park, the probability to find a "dense traffic" is very small and higher traffic classes are not taken into account.

| Comfort classes | | | |
|-----------------|-------------------------|---------------------------|---------------------------|
| Comfort class | Degree of comfort | Vertical a_{lim} | Lateral a_{lim} |
| CL1 | Maximum | $< 0.5 \text{ m/s}^2$ | $< 0.1 \text{ m/s}^2$ |
| CL2 | Medium | $0.5 - 1.0 \text{ m/s}^2$ | $0.1 - 0.3 \text{ m/s}^2$ |
| CL3 | Minimum | $1.0 - 2.5 \text{ m/s}^2$ | $0.3 - 0.8 \text{ m/s}^2$ |
| CL4 | Unacceptable discomfort | $> 2.5 \text{ m/s}^2$ | $> 0.8 \text{ m/s}^2$ |

Table 3. Comfort classes with common acceleration ranges (from [5])

As shown in Table 1, there are 10 natural frequencies in the critical bandwidth. For each frequency, and for each traffic class considered, a dynamic analysis is performed. Each analysis is characterized by a harmonic load acting up and down coherently with the corresponding mode shape directions and by an excitation frequency equal to one of the natural frequency of the bridge. The maximum accelerations have been computed with a harmonic analysis in ANSYS. The numerical results are evaluated, in three cross sections of the deck (at half-span and at a 1/4 of span, both sides) for three points: two are placed at the base of handrails and the third one is on the longitudinal axis of the footbridge deck (Figure 5).

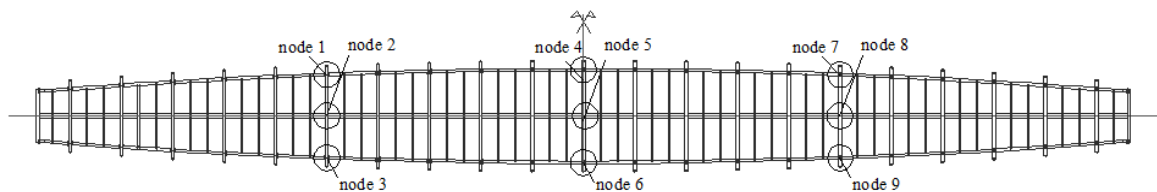


Figure 5. Cross sections and corresponding points considered in the analysis

The "Seriata footbridge" exhibits natural mode shapes with a flexural or torsional-flexural behavior, except for the 4th mode ($f = 1.95 \text{ Hz}$), characterized by a displacement field in the transverse horizontal plane. For each flexural mode, the significant output response parameter is the amplitude of the vertical acceleration due to the related harmonic load; lateral accelerations are considered, due to the harmonic load tuned to the 4th mode. Figure 6 and Figure 7 depict the results for the traffic classes TC1 and TC2, respectively. The maximum values of footbridge acceleration, at the selected nodes and for each mode, are depicted in Figures 6a, 7a and 6b, 7b for vertical vibration and lateral vibration respectively. Horizontal lines denote the limit values proposed by Hivoss guideline.

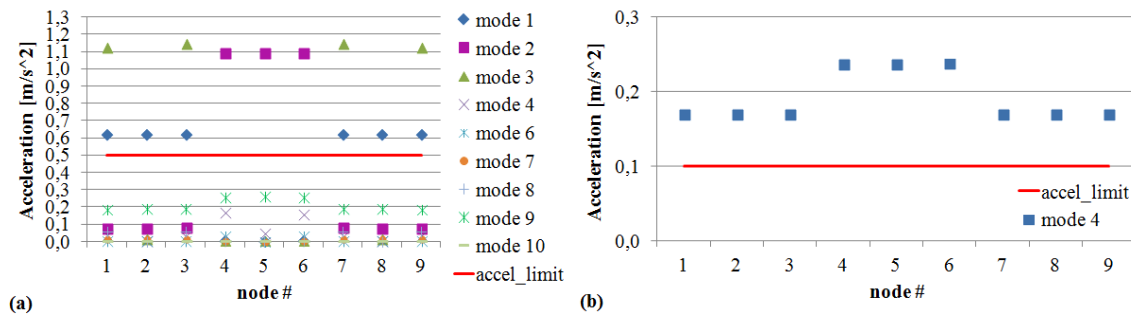


Figure 6. Vibration serviceability assessment, traffic class TC1: (a) vertical vibration, (b) lateral vibration.

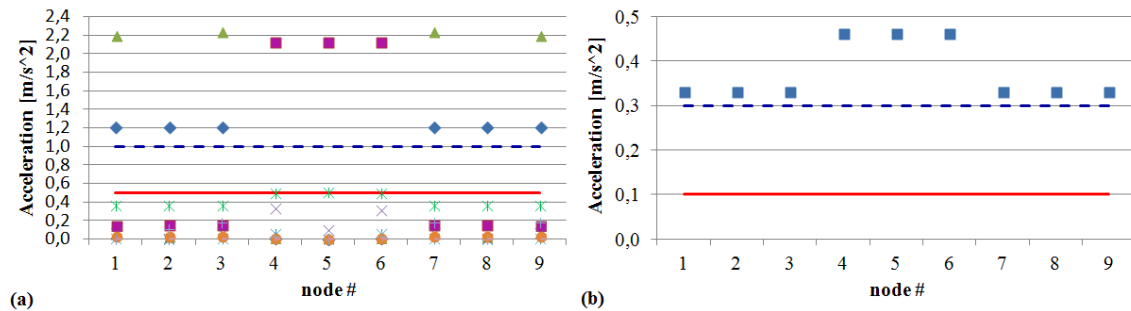


Figure 7. Vibration serviceability assessment, traffic class TC2, (a) vertical vibration, (b) lateral vibration.

In Figure 7, having the same legend as in Figure 6, the dashed and the solid line represent the upper and the lower bound of the limit acceleration value, respectively, for the comfort class CL2. The results show that the accelerations due to the harmonic loads tuned with the first 4 mode shapes exceed the comfort limits in both traffic classes considered. Conversely, starting from the 6th mode shape, the maximum acceleration are lower than those prescribed by HG. The 5th mode doesn't appear in the Figures 6-7 since its reduction factor ψ is equal to zero, making the load amplitude null. When the excitation frequency exceeds the value of the 4th mode, the second harmonic of pedestrians loads comes into play. Until now, there is no hint in the literature that significant vibrations have occurred due to the second harmonic of pedestrians [5] and the numerical results confirm this finding.

4 VIBRATION SERVICEABILITY DUE TO WALKING: TRANSIENT ANALYSIS

The HG provides an engineering approach to the prediction of dynamic effects due to walking pedestrians. The bridge modal properties must be determined, and the dynamic load-induced model is applied in a harmonic analysis. However, the need arises of a more sophisticated approach to compute the bridge response, able to account in a more realistic way the human walking activity. Different models for the time variation of the vertical force induced by walking derived from experimental tests can be found in the literature. The bridge response to a moving variable force can only be computed with a numerical transient analysis, able to account for the force variation both in time and in space. To this aim, in this work a numerical code named INTER, written in Fortran language and originally developed to study the bridge-vehicle dynamic interaction [7], has been modified to analyze the interaction between pedestrians and bridge. Modifications have also affected the routines related to the bridge geometry. The updated INTER 2.0 version of INTER is able to consider bridges with different geometry, such as suspension bridges with an arch deck.

INTER 2.0 performs the transient analysis of a footbridge subjected to moving forces (pedestrians) in the time domain, by integrating the equations of motion with the Newmark's

method of constant acceleration. The structural matrices (stiffness and mass) of the footbridge, extracted from ANSYS at the end of the non-linear analysis, are read as input data. Thus, the stiffness matrix takes into account the stress stiffening, i.e. the stiffening of the structure due to its stress state (pre-tension applied in each cables). The relevant information about the bridge geometry and the characteristics of pedestrian stream are also input data.

As a first approximation, each pedestrian is modeled as a moving force travelling along the bridge deck with a constant velocity v and along a rectilinear trajectory parallel to the bridge axis. The pedestrian's position on the bridge deck is identified by the transverse and longitudinal coordinates x_p, y_p . Under the above assumptions, the former remains constant during motion, the latter varies linearly in time. The moving force is vertical and no horizontal force is transmitted from pedestrian to bridge. In the first implementation of the algorithm, the force amplitude is constant, and only its position on the bridge changes at each time step. Given the time step Δt of the numerical integration, at the generic instant $k\Delta t$, the pedestrian's position y_k along the longitudinal axis is computed from the pedestrian's position y_{k-1} at the previous $k-1$ step as:

$$y_k = y_{k-1} + v * \Delta t \quad (4)$$

The equations of motion of the footbridge are:

$$\mathbf{M}\ddot{\mathbf{q}} + \mathbf{C}\dot{\mathbf{q}} + \mathbf{K}\mathbf{q} = \mathbf{N}(x_p, y_p(t))^T * F_p \quad (5)$$

where \mathbf{q} is the vector listing the lagrangian coordinates of the footbridge FE model, \mathbf{M} and \mathbf{K} are the ANSYS mass and stiffness matrices respectively, \mathbf{C} is the Rayleigh damping matrix, computed by INTER 2.0, and $\mathbf{N}(x_p, y_p(t))^T * F_p$ is the pedestrian forcing term. Since F_p is a constant term equal to the pedestrian's weight, the time variation is all included in the term, $\mathbf{N}(x_p, y_p(t))^T$, a vector of interpolating shape functions that transfers each load from its point of application to the proper nodes of the FE mesh.

The need for a shape functions vector arises from the following rationale. At a generic time instant, the pedestrian does not stay on a mesh node. In fact, he can follow a *quasi-random* rectilinear trajectory, only constrained to move parallel to the bridge axis and with a x_p coordinate inside the deck grid. At each time step, the shape functions transform the moving force into equivalent nodal loads on properly selected nodes. As shown in Figure 8, if the pedestrian trajectory coincides with a line of nodes, the loaded nodes will be those placed immediately before and after the force position. In the opposite case, the loaded nodes will be the four nodes of the mesh surrounding the pedestrian's position. The shape functions values, depending on the pedestrian's position $(x_p, y_p(t))$, change at each time step and their implementation takes care of unloading the nodes left behind and loading the approaching nodes. At each time step the terms of $\mathbf{N}(x_p, y_p(t))^T$ are all zero, except for the rows corresponding to the DOFs related to the nodes directly or indirectly loaded by the pedestrian.

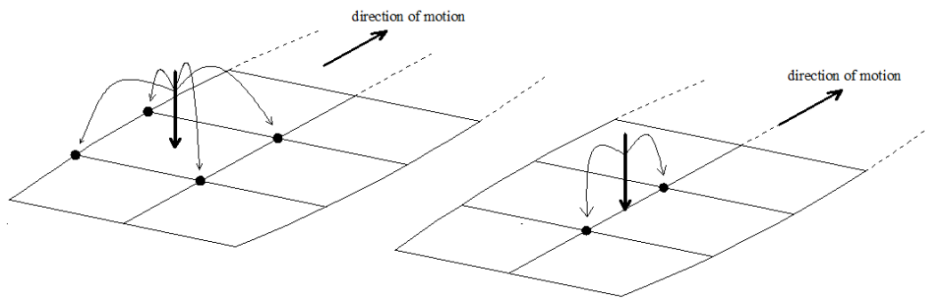


Figure 8. Loaded nodes: (a) pedestrian outside a line of nodes, (b) pedestrian on a line of nodes.

The FE model of the bridge deck contains only beam elements. Nevertheless, the shape functions are those of a 4-node shell element with 12 DOFs, based on Kirchhoff theory, already implemented in the code. This choice is acceptable if we consider the real situation of the timber deck and has proven to be able to minimize numerical problems tied to high frequency oscillation in the response. Since loads on the bridge are only vertical, the "virtual mesh" simulates a bending plate element and the selected shape functions produce, in each node, nodal forces on the vertical DOF and on the two out-of-plane rotational DOFs.

4.1 Numerical results

Either a single pedestrian or a group of pedestrians can move on the deck. Different spatial configurations of pedestrians have been implemented in INTER 2.0 to represent both a regular stream, with positions placed into a "uniform grid", and a random group of pedestrians in a "chess grid" with variable distance between them. This work considers a group of 15 pedestrians, as in the traffic class TC1, placed with a *random-like* distribution (Figure 10). The center of gravity of the group is eccentric with respect to the bridge axis. The distance between the first and last pedestrian is 17 m; the position of the resultant force in longitudinal direction is 9.6 m behind the first pedestrian row. A velocity of 1.3 m/s is considered [8]. Each pedestrian crosses the bridge in 49 s and the last pedestrian exits the bridge 13,7 s after the first.

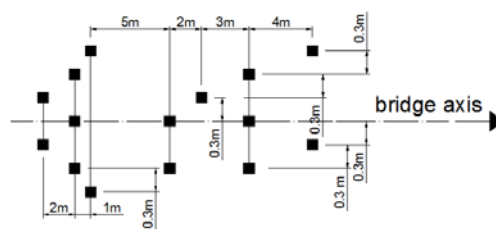


Figure 9. Distribution of pedestrians associated to traffic class TC1; the arrow defines the direction of motion.

The integration time step Δt is 0.001s, quite smaller than 1/10 of the period of the 14th mode. At time $t=0$ the first pedestrian enters the bridge; the analysis goes on for 25 s after the last pedestrians exit the bridge. The Rayleigh damping matrix is computed for a 1% damping, a value experimentally identified, on the first and fourth mode. The time history of displacements and accelerations at the nodes considered in the harmonic analysis (Figure 5) are the relevant response parameters. Figure 10 depicts the accelerations of the nodes 2-5-8 along the bridge axis. Figure 11 shows the same quantity for the lateral nodes 3-6-9. A different pattern is detected, with largest values in the lateral nodes, due to the torsional effect induced by the load scheme. The acceleration at midspan (nodes 5-6) when the resultant force reaches the cross section ($t \approx 30$ s), is coherent with the mode shape of the 6th torsional mode.

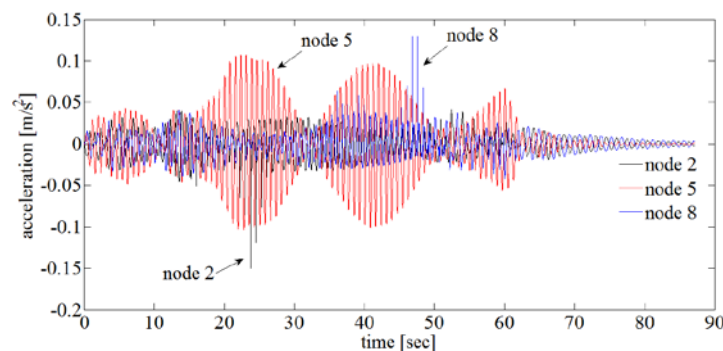


Figure 10. "Random" pedestrian stream: time history of acceleration, nodes 2-5-8.

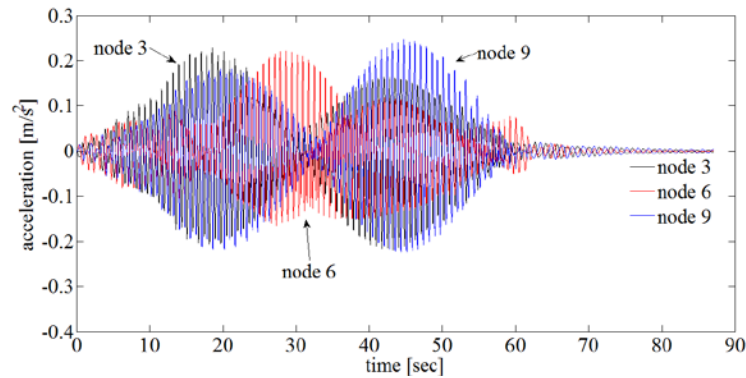


Figure 11. “Random” pedestrian stream: time history of acceleration for nodes 3-6-9.

Figure 12 shows the displacement responses of the nodes along the bridge axis (2-5-8). The nodes 2 and 8, at the $\frac{1}{4}$ of the span, show quasi-specular results; each node attains the maximum displacement in the time interval when is reached by the center of the parallel forces. In Figure 13 the displacement responses of the nodes at midspan (4-5-6) have a similar pattern but different magnitude due to the eccentricity of the load pattern. Node 6 experiences the maximum value of displacement.

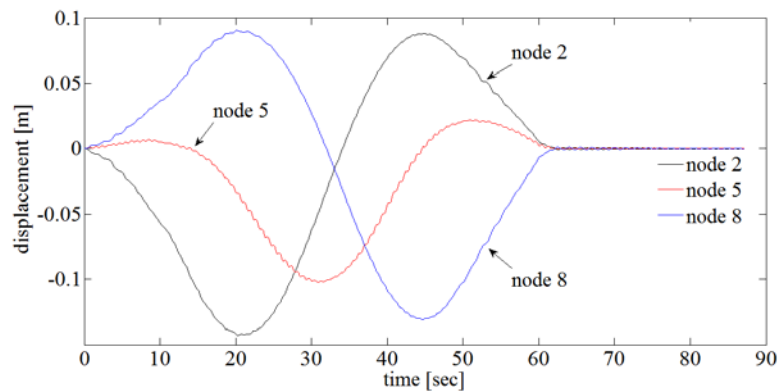


Figure 12. “Random” pedestrian stream: time history of displacement for nodes 2-5-8.

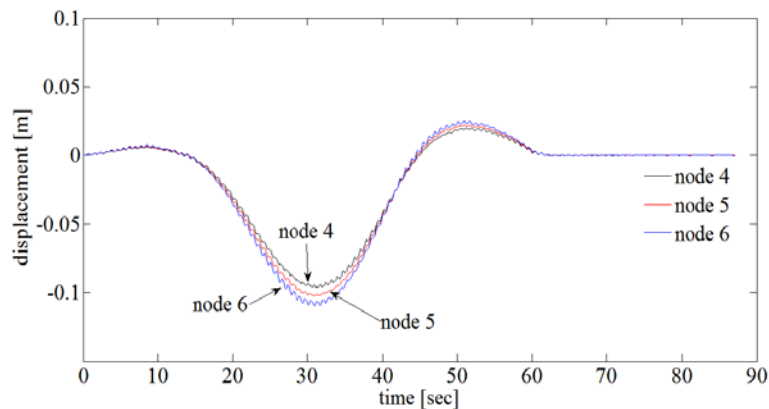


Figure 13. “Random” pedestrian stream: time history of displacement for nodes 4-5-6.

5 CONCLUSIONS

The results of serviceability assessment of a lively footbridge have been obtained with a harmonic analysis in ANSYS, as suggested by the HG, and with a transient step-by-step anal-

ysis performed with an *ad-hoc* numerical code named INTER. Two traffic classes are considered, very weak and weak (TC1, TC2), respectively associated to maximum CL1 and medium CL2 comfort classes. Ten natural frequencies of the footbridge are in the critical range characterizing the walking activity. Even though the pedestrian-induced loads are different in the two approaches, a comparison between the predicted acceleration values appears significant.

In the HG implementation, the 2nd harmonic of pedestrian load has to be considered starting from the 5th mode. The accelerations due to the harmonic loads tuned to the first 4 modes exceed the comfort limits in both traffic classes considered. Moreover, no significant vibrations have been detected due to the second harmonic of pedestrians load. The transient analysis is performed with the code INTER 2.0 under a stream of 15 pedestrian aimed to represent the TC1 condition. The peaks of the acceleration value are lower, but of the same order of magnitude of those induced by the harmonic loads for the higher modes. The results provided by HG and INTER 2.0 can represent an upper and lower bound of the expected acceleration. On one side, a very low value of damping ratio is related to a high value of the amplification factor in the harmonic analysis of HG. On the other side, the dynamic effect induced by the moving pedestrian, as computed by INTER 2.0, is underestimated by the approximations of constant force moving in a continuous way on the deck.

The numerical results hint that the vibration serviceability conditions of the footbridge may not be fulfilled. Further studies will address the issues tied to the transient analysis, namely the efficiency of the numerical integration, the modeling – in space and time - of the pedestrian-induced force and the effect of the space distribution of pedestrians. Final aim of the research work is the study of the dynamic interaction based on the adoption of a mechanical system for each pedestrian.

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