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NONLINEAR ANALYSIS OF A LARGE RC CANTILEVER GRANDSTAND OF SAN SIRO MEAZZA STADIUM IN MILAN USING THE APPLIED ELEMENT METHOD

Nicola Scattarreggia[1], Martina Cogliano[2], Gian Michele Calvi[2], and Matteo Moratti[3]

[1] University School for Advanced Studies IUSS Pavia
Piazza della Vittoria, 15, Pavia 27100, Italy
e-mail: nicola.scattarreggia@iusspavia.it

[2] University School for Advanced Studies IUSS Pavia
Piazza della Vittoria, 15, Pavia 27100, Italy
{martina.cogliano, gm.calvi}@iusspavia.it

[3] Studio Calvi Ltd
Via S. Severino Boezio, 10, Pavia 27100, Italy
matteo.moratti@studiocalvi.eu

Abstract

Stadiums may suffer from vibration serviceability problems when a large group of individuals in a crowd are involved in a coordinated motion due to their common light weight and adopted line of sight. Considering old stadium structures may have been designed without considering the dynamic effects associated to people jumping up and down or moving in synchro to a played song, it is of relevant interest to assess their safety usually conducted using Finite Element models. In this work, the Applied Element Method, a low-cost computational approach based on the mechanical interaction between rigid bodies and zero-thickness nonlinear interface springs, is used to assess the nonlinear structural performance of a sector of the reinforced concrete San Siro Meazza stadium in Milan, characterized by a large RC cantilever. Once the numerical model has been validated using available experimental data and results obtained through traditional FE models, nonlinear static analyses have been performed to evaluate the vertical capacity and the potential collapse mechanism. Thereafter, dynamic analyses have been conducted with a view to assess the structural response using a possible dynamic jumping load model. Displacement and acceleration time-histories have highlighted the nonlinear structural response of the analyzed portion when subjected to periodic dynamic loadings with frequencies resonant with those of the structure. Finally, a theoretical collapse mechanism has been numerically obtained.

Keywords: reinforced concrete, stadium, dynamic analysis, Applied Element Method.

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1 INTRODUCTION

Stadiums are multifunctional facilities being used for various sport events as well as music concerts, thus they represent a global entertainment business. Grandstands are typically characterized by slender cantilever structures in such a way as to provide a good line of sight, thus they require special attention during the design to limit vibrations as well as to provide sufficient structural safety. While new stadiums are designed to provide comfort and a safe environment to spectators, considering the dynamic excitation associated to different possible human crowds, most old stadiums could have not been designed to withstand the dynamic loads produced by thousands of people jumping up and down or moving in sync to the rhythm of the song played. To this end, old stadiums require monitoring used in combination with the structural assessment to control security/safety (see e.g. [1]).

Structural performance assessment of stadiums is typically conducted using liner elastic models based on Finite Element Method (FEM) (e.g. [2]). In this work, the Applied Element Method, a rigid body plus springs discrete modelling approach, is selected to preliminary assess the nonlinear response of an RC cantilevered portion of the Giuseppe Meazza stadium in Milan, also known as San Siro stadium. The current structure is an assembly of three main parts, as a result of three main projects carried out since 1926 to increase the capacity and adapt the security measures to current standards. Over the past years, it has been the drama of important soccer games such as the European Champion Clubs' Cup and the World Championship. However, in addition to that, since 1980 the San Siro stadium has welcomed music concerts of national and international singers, with the first tour of Bob Marley & The Wailers, as well as Bob Dylan (in 1984), Michael Jackson (1997), Rolling Stones (2003), and, more recently, e.g. the Muse (2019). A long story that may have an end in 2026 after the welcome ceremony of the XXV Winter Olympic Games, considering the hypothesis of demolition (see e.g. [3]). Recent applications have shown that the AEM, originally conceived by Meguro and Tagel-Din (2000) [4] to simulate controlled structural demolition, does appear to be able to capture adequately the progressive failure of large-scale bridge systems (e.g. [5]) as well as the response under small deformations [6]. Thus, using historical and publicly available material, an AEM-based model was built for explicitly assessing the nonlinear static capacity of the above-mentioned portion of the San Siro stadium, while analyzing the influence of external steel plates on the structural response under vertical loads. Finally, nonlinear dynamic analyses were carried out to assess the structure response subjected to a theoretical jumping load model, while providing a theoretical potential collapse mechanism.

2 THE APPLIED ELEMENT METHOD FOR MODELLING RC STADIUMS

The Applied Element Method (AEM) is a discrete modelling approach in which a given solid body is modelled as a virtual assembly of small rigid units connected to each other by zero-thickness springs (normal and shear) uniformly distributed along element faces (see Figure 1). It can be easily adapted for modelling RC structures by explicitly coupling the mechanical contribution of concrete elements, whose properties are assigned to interface springs, with those of the embedded steel bars, usually represented by equivalent “reinforcement springs” that are placed in the same position of their real counterparts. The degrees of freedom (DOF) are assumed at the centroid of each rigid element, and the assembly of these rigid units through those springs make the behaviour of element collections deformable. Particularly, the material’s behaviour is represented by interface springs characterized by both normal k_n, and shear k_s stiffnesses (Figure 1). In the case of concrete, the stiffness of each spring depends on the volume assigned to it according to the total number at each interface, while steel reinforcement is represented by both normal and shear springs in the same location of the modelled counterpart. Material models can follow whatever uniaxial relationship but need to be implemented properly to account for non-standard phenomena, e.g. strain rate effects.

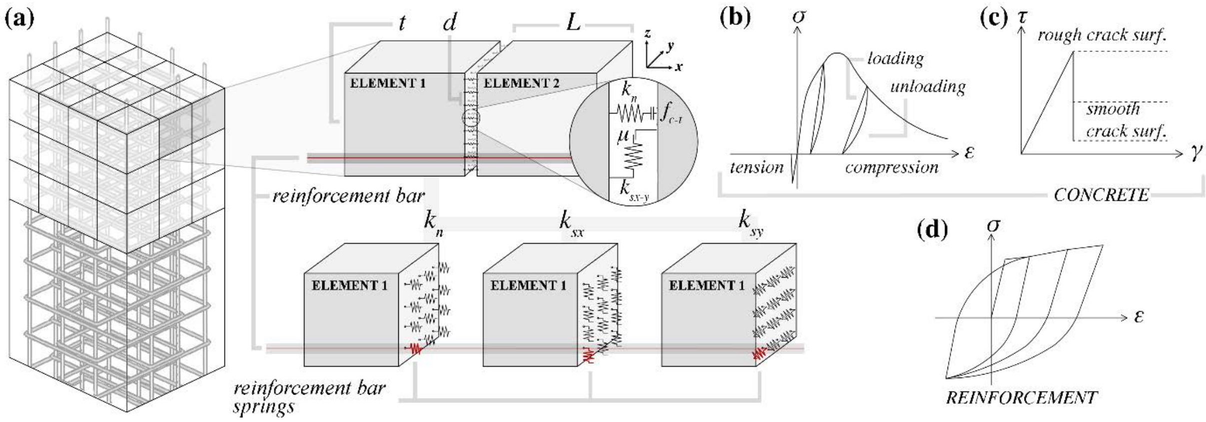


Figure 1: a) Schematic of modelling of reinforced concrete members using the AEM and b) material modelling: constitutive relationship of concrete and steel reinforcement.

However as described in [7], concrete response under uniaxial cyclic tension-compression loading can be modelled using the [8] model. To this end, Young's modulus of concrete (E_c), fracture (ψ_c) and compressive strain (ε_c) parameters are used to define an elastic bound, beyond which k_n is assumed as a minimum value (1% of initial k_n in this work) to avoid stiffness matrix singularities. Tension cracking is accounted for by neglecting the contribution of tensile strength right after reaching the peak stress f_{ct}. When considering shear-compression stress-states, instead, a simplified Mohr-Coulomb failure envelope is typically employed, and residual strength is assigned to account for some resistance after cracking due to the interlocking phenomena (typically assigned a roughness factor equal to 0.01). The hysteretic model proposed by [9] is typically assigned to normally-loaded reinforcement springs, whose failure takes place either if the considered faces reach their separation strain (the threshold that implies the cut-off of springs), or when the Euclidean norm of the surface traction vector reaches an ultimate strength f_u, as discussed in e.g. [10]. A separation among units is not permitted with static analysis, indeed such analysis diverges due to the stiffness matrix singularity, whilst it is possible in the dynamic stage where the addition of inertia forces. Once elements are separated collision can take place. Considering the nonlinear materials modelling is adopted the majority of structural sources of damping are explicitly considered, i.e. cracking of concrete, loading and unloading paths of both concrete and steel, cracks closure and opening, and additional sources of damping may not be needed.

3 BRIEF DESCRIPTION OF THE SAN SIRO MEAZZA STADIUM

The Giuseppe Meazza stadium, better known as San Siro stadium (its previous name from the district where it is located), shown in Figure 2a, is a multifunctional sports facility in Milan, Italy. It consisted of various substructures, mainly three portions (better known as "rings") and the roof, as result of the modifications since 1925. The so-called "first ring" (i.e. the lower part) is the original stadium composed of grandstands parallel to the soccer field and those added in the curves in 1940, able to welcome around 55,000 spectators. From a structural point of view, this portion is a very stiff RC-framed structure lying on the ground.

After the Second World War, during the 1950s, the stadium increased its capacity up to 90,000 spectators with the construction of the middle part better known as "second ring". The stadium was the theatre of the 1965 and 1970 European Champion Clubs' Cup. Subsequently, during the 1970s the stadium was designed to welcome the UEFA league, and it was during this period that there was a restoration of the stadium 40 years old. The second ring consists of RC frames to which are linked cantilevers supporting the stands. Particularly, 14 independent grandstands compose the second ring; gaps are also provided with the first ring.

Finally, subsequently to the designation as the drama of the World Championship of 1990, following a new rationale for the security of spectators rather than increasing its capacity, the stadium was subjected to a massive restoration. Thus, it was further expanded with the construction of the third ring (i.e. the upper portion). It is an independent structure, with eleven towers (30-m high) that support ten caisson beams 50-m long which are linked two main cantilevers supporting the stands. As result, the third ring is composed of 10 grandstands. The corner towers are 50-m height and support the trussed structure of the roof.

(a) **(b)**

Figure 2: a) Perspective view and b) transversal section of the San Siro Meazza stadium in Milan. (In the red box is the so-called "second ring" under investigation).

As above described, similarly to other large structures, the San Siro stadium is constituted of sub-structures featuring different dynamic properties that require supervision through appropriate Structural Health monitoring systems (SHM) (see e.g. [11]), that assist in its management while posing special attention to the structural safety. It was during recent significant soccer events that caused vibration levels higher than those usually recorded (specifically recorded in the so-called "green sector"). Particularly, recorded acceleration time-histories reported a peak of 2.5 m/s^2 in the vertical direction at the knee of the main cantilever, while a peak response of 1.2 m/s^2 was recorded in the horizontal direction. As result, the study of the structural performance of a such portion of the structure has been considered of interest.

The second ring is essentially constituted of frames, which feature the following main components:

- Two columns with a cross-section of 1.20x0.70m^2.
- A beam of variable depth, from 0.25m to 2.90m, and 0.70m wide, characterized by the main cantilever.
- Transversal beams along the height of the frame with a cross-section of 1.90x0.50m^2 to connect the columns.

Finally, it is noticed that during the restoration of the 1970s, the lower main cantilevers were strengthened with the introduction of steel plates, laterally bolted to the concrete.

4 NUMERICAL MODELLING

The numerical model developed consists of a single internal frame in a sector (composed by the assembly of parallel frames), thus taking advantage of the results carried out by Cogliano et al. 2023. Indeed, the vertical behaviour of an internal frame can be properly reproduced with a 2D model in which loads are derived considering an area of influence equal to 5 m (distance between two consecutive frames in the direction parallel to the field).

The discretization adopted was devised with a view to balance computational accuracy and analysis time. Indeed, the selection of the appropriate mesh sizes for each structural element/region was guided, as proposed by Scattarreggia et al. (2022) [12], by the expected damage distribution. A coarser mesh was therefore allotted to regions that would not suffer extensive damage or which with unlikelihood undergo large deformations, while more refined element subdivisions were applied to regions which are deemed to potentially trigger the collapse or exhibit significant damage/deformation. As shown in Figure 3the upper part of the frame was modelled using an equivalent bilinear material considering is not of relevance to the response of the portion under investigation, indeed modelled fully nonlinear.

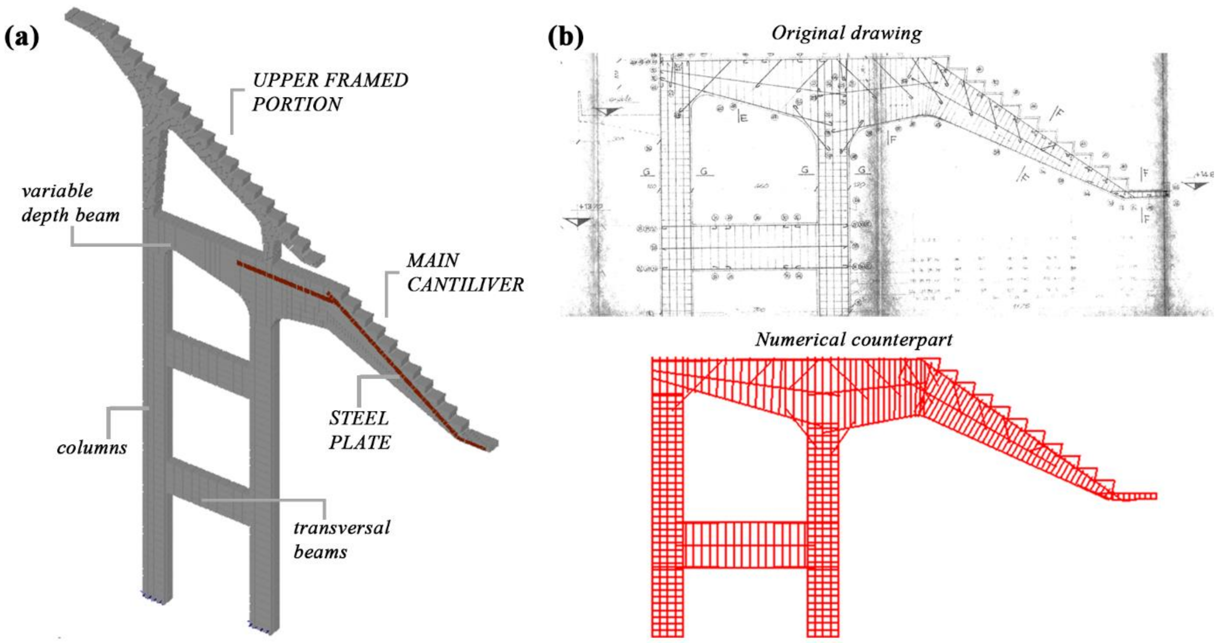


Figure 3: Numerical modelling using the AEM of an isolated frame: (a) solid model and (b) comparison between original drawings and numerical idealization of reinforcement details for the portion of main interest.

A coarse discretization was thus employed for the upper part (indicated in dark grey in Figure 3) (Δ=0.3-0.6m, where the parameter Δ indicates the min-max dimensions, in meters, of the mesh units constituting the considered discretized structural element) and bottom part of the frame's columns (Δ=0.2-0.4m, which is reduced up to 0.15-0.3 at the pier-arches connections). The optimal level of discretization of the variable depth cantilever was iteratively determined using different numbers and orientation of triangular and hexahedral elements; the employed mesh layout (hexahedral elements have a variable dimension, from Δ=0.15-0.2m at the cantilever tip to Δ=0.15-0.4m at the top knee) resulted in only minor dissimilarities in terms of the global response, governing failure mechanisms and stress-strain path with respect to more detailed models. Finally, the elements constituting the transversal beams were characterized by a medium size mesh (Δ=0.15-0.35m). As a result of the adopted discretization, approximately 7,000 elements were employed for the isolated frame (i.e. around 40,000 degrees of freedom), shown in Figure 3. As previously discussed, the second ring was subject to a massive restoration during the 1970s, with the addition of steel plates. From a numerical point of view, the retrofit model was built by explicitly modelling the steel plates (herein assumed of 250mm constant height and 8 mm thick, with a mesh discretization Δ=0.15-0.008m), while assuming perfect adhesion at the interface with concrete.

5 CROSS-VALIDATION WITH TRADITIONAL FEM MODELLING AND EXPERIMENTAL RESULTS

As a first verification of the developed AEM model, some comparisons against results obtained with the 2D FE model developed in Sap 2000 [13] by Cogliano et al. (2023) [14] were carried out, considering both static analysis as well as eigenvalues. Starting with the first, and as indicated in Table 1, the results of the AEM model were compared with those obtained using the FEM-based one. Two structural response quantities were examined: the total vertical base reaction, and the vertical deformation at the lower knee of the main cantilever. Very minor differences were observed for forces, which is quite reassuring, whilst higher differences are observed in terms of deformation because of the fully nonlinear behaviour of the AEM model over the construction stages (indeed some cracks are observed before the application of the external steel plates).

Parameter	AEM	FEM	AEM/FEM
Total vertical base reaction [kN]	2756	2836	0.97
Vertical deformation at cantilever tip [mm]	23	15	1.53

Table 1: Static results analysis under permanent loads (retrofit included).

For what concerns the eigenvalue analysis, and considering the structure configuration as for the static analysis case, Figure 4 shows a comparison between FEM and AEM models for the predominant vibration modes along x, and z translational axes (i.e. $T_{1,x}$ is in the horizontal direction, while $T_{1,z}$ is in the vertical direction). It was noticed that the horizontal mode, with a frequency of about 2.5 Hz, involves a movement of the whole frame, while the vertical mode is mainly characterized by the motion of the main cantilever, with a frequency of about 4.5 Hz.

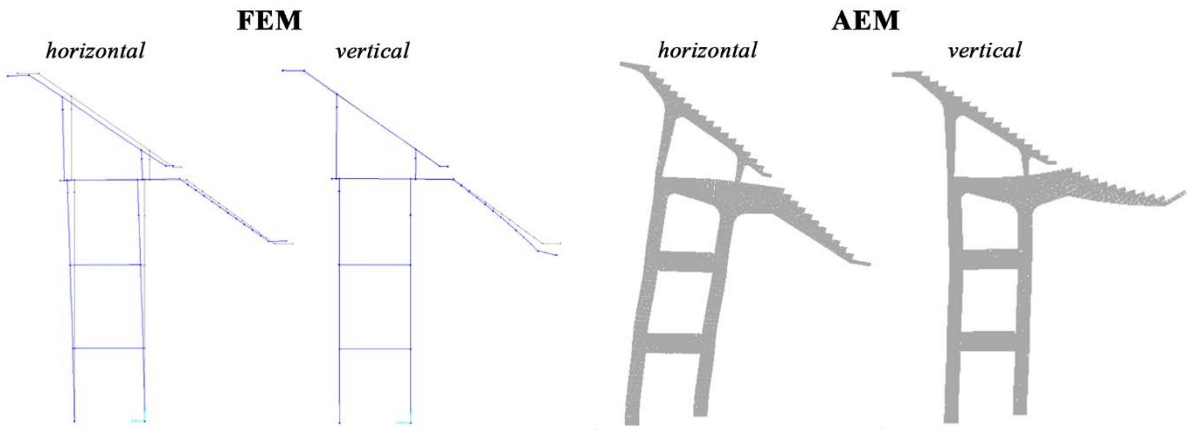


Figure 4: Modal shapes for the fundamental vibration modes using the FEM and AEM models.

Therefore, similarly to the static analysis comparison, the dynamic results obtained using the AEM were consistent with those from the FE model, being also aligned with those obtained from experimental results as summarized in Table 2. Indeed, dynamic identification tests handled by Politecnico di Milano in 2006, as shown by the spectra in the frequency domain in

Figure 5, highlighted that the dynamic behavior of the second ring is governed by few harmonic components. Particularly, a first mode with a frequency of about 2.2 – 2.3 Hz characterized the horizontal movement of the second ring frames, whilst a second mode with a frequency of about 4.4 – 4.6 Hz distinguished the vertical movement of the main cantilever.

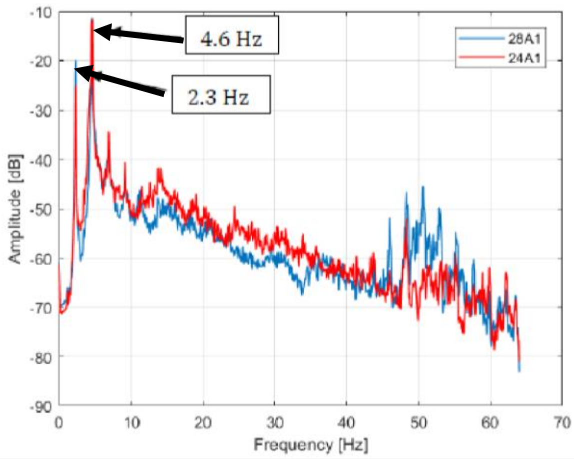


Figure 5: Frequency spectra for both the vertical and horizontal accelerations.

Mode Case	Frequency [Hz]		
	Experimental	FEM	AEM
Horizontal movement	2.3	2.14	2.50
Vertical movement	4.6	4.39	4.50

Table 2: Comparison of natural frequencies between experimental data, FEM and AEM results.

6 NONLINEAR STATIC ANALYSES

The capacity and failure mechanism of the developed AEM-based model has been analyzed through an incremental linearly distributed vertical load until the collapse. Such analysis has been carried out considering both the model with and without the steel plates (used as retrofitting measure). The pushdown curves shown in Figure 6 are expressed in terms of vertical base reactions from loads (both the permanent and incremental load) acting on the main cantilever (left *y*-axis), with respect to the vertical displacement monitored at the cantilever knee (*x*-axis). In addition, in the right y-axis, the incremental load is expressed in terms of uniformly distributed live loads acting on the main cantilever.

The incremental load has been applied in correspondence of steps and, as can be inferred from Figure 6, a peak vertical base reaction of about 760 kN and 1070 kN (or in terms of a uniformly distributed load of 3.2 kN/m^2 and 10.4 kN/m^2) has been achieved for the model with and without steel plates, respectively. Therefore, the introduction of steel plates (according to what is described in Section 4) has led to an increase of peak strength/force of about +40% with respect to the model without retrofitting. On the other hand, both models appear to fail for a vertical displacement (monitored at the lower cantilever knee) between 160-180 mm. Figure 7 reports the associated failure mechanisms as a result of the pushdown analyses. Particularly, the comparison between Figure 7a and Figure 7b shows the impact of the retrofitting intervention on the shift of the section where the failure initiates. Indeed, in the model without retrofitting (Figure 7a) the failure starts along the inclined portion of the main cantilever, between the fifth and sixth step (from the top knee), while in the model with retrofitting (Figure 7b) such failure is impeded and thus, according to the capacity principles, the failure is shifted to a section located at about the center of the main transversal girder with variable depth.

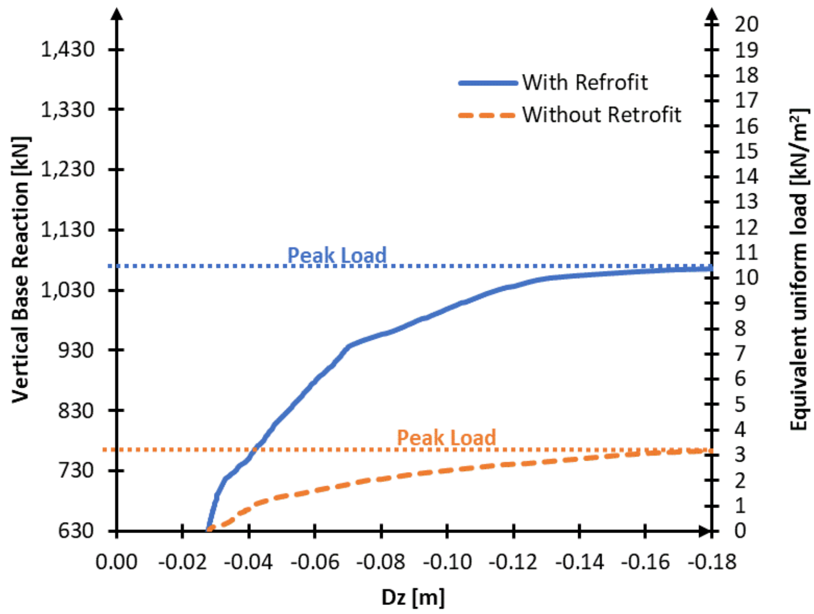


Figure 6: Comparison of the nonlinear static capacity curves between the model with and without retrofit.

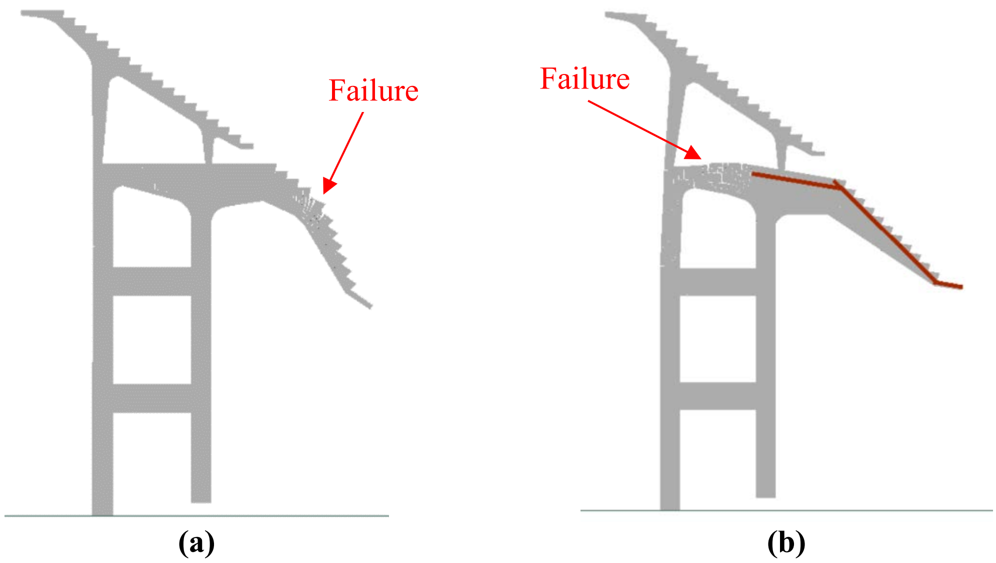


Figure 7: Theorical failure mechanism for the model a) without and b) with retrofitting.

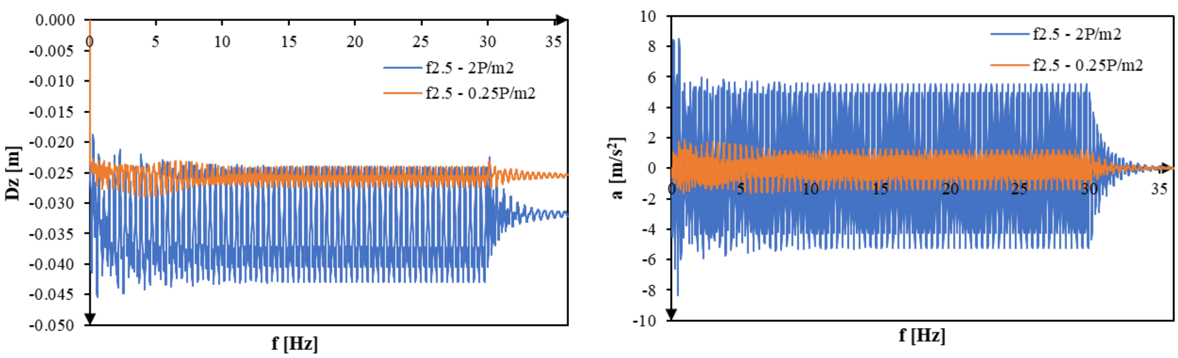
7 PRELIMINARY NONLINEAR DYNAMIC ANALYSES

The dynamic response of the structure has been preliminary analyzed through nonlinear time-history analyses considering a possible dynamic crowd action model applied on the main cantilevered portion.

In literature are available various loading models to account for the dynamic excitation of people, considering various activities as well as characteristics of the crowd (synchronous or asynchronous movement of spectators) and the mutual interaction between structure and spectators, as being studied both experimentally and numerically (see e.g. [2]). Cogliano et al. (2023) [14] have preliminary evaluated the influence of the adopted loading model as well as the effect of the sync/async motion on the linear dynamic response of the second ring. However, herein we start analyzing the nonlinear response using the periodic loading model proposed by Ji and Ellis (1994) [15]. Particularly it has been considered the most severe loading scenario, i.e., assuming the synchronous motion of people (considering an average weight of 75 kg per person) jumping with frequencies resonant with those of the structure. Therefore, the loading function has been calculated for a frequency of 2.5 Hz and 4.5 Hz. And although the typical frequency of jumping crowd is between 1.5 Hz and 3.5 Hz (as for instance described in [2]), a loading model with a frequency of 4.5 Hz is of interest to analyze the nonlinear structure response of the main cantilever and to account for the potential changes in its dynamic properties (a task that could not be accomplished with elastic models). However, it has to be noted that also loading models with a frequency that is a sub-multiple of a critical frequency may need special attention.

The dynamic analyses were conducted assuming a different number of spectators present on the grandstand, i.e. 0.25 persons/m^2 and 2 persons/m^2 (all jumping simultaneously). The displacement and acceleration time-histories are monitored at the lower knee of the main cantilever.

In the case of the load model with a frequency of about 2.5 Hz, the displacement time-history shown in Figure 9a indicates that the model with 2 persons/m^2 produces a peak value about 1.6 times higher than the case with 0.25 persons/m^2. This trend is also observed in terms of acceleration (Figure 9b). Indeed, the model with 2 persons/m^2 produces peak displacement and acceleration of about 45 mm and 8 m/s^2, respectively, while when considered 0.25 persons/m^2 such values decrease to 28 mm and 2 m/s^2. Furthermore, in the case of 2 persons/m^2 the residual displacement (after 30 seconds of jumping and 5 seconds of free vibration) increases from about 23 mm to 32 mm, thus highlighting material nonlinearities have taken place. Indeed, the eigenvalue analysis conducted at the end of the analyses indicated a reduction of the structure frequency for the second mode (i.e. the one associated with the vertical movement of the main cantilever) of about 15% in the case of 2 persons/m^2. The frequency remained essentially constant in the case of 0.25 persons/m^2, thus indicating the elastic response of the structure.



(a) **(b)**

Figure 8: a) Displacement and b) acceleration time-history for a jumping load model with a frequency of 2.5 Hz and 0.25 and 2 persons/m ^{2}.

On the contrary, with a loading frequency very similar to that of the vertical mode of vibration (i.e. about 4.5 Hz), a collapse was observed for the loading model with 2 persons/m^2 after about 20 sec of excitation. Contrarily to the previous case, the eigenvalue analysis conducted at the end of the analyses indicated a reduction of the structure frequency for the second mode (i.e. the vertical one associated with the vertical movement of the main cantilever) of about 5% in the case of 0.25 persons/m^2. This result indicates a load with a frequency of 4.5 Hz more critical with respect to the previous one (i.e. with 2.5 Hz), producing some nonlinearities.

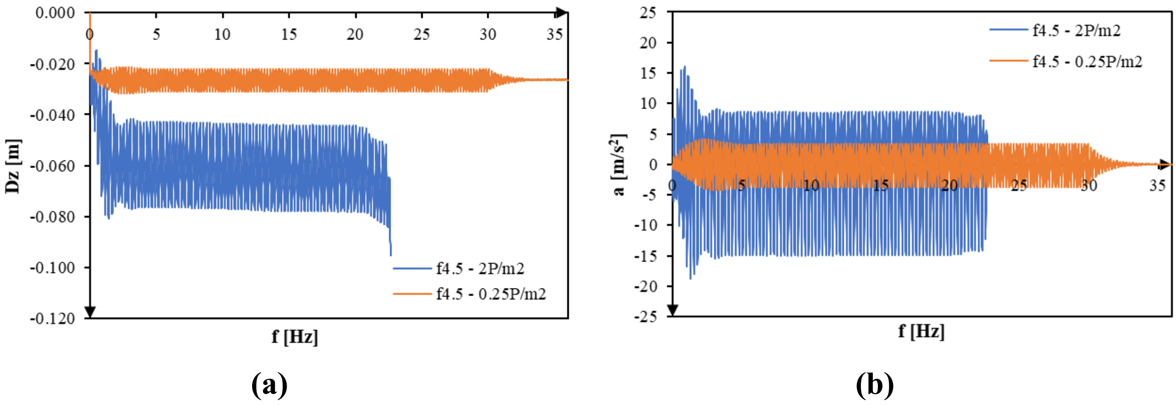


Figure 9: a) Acceleration and b) displacement time-history for a jumping load model with a frequency of 4.5 Hz and 0.25 and 2 persons/m^2.

In Figure 10 are reported the initial instants of the theoretical collapse mechanism. The collapse starts with a flexural-shear failure in correspondence of the variable-depth transversal beam, consistently with the push-down results. The failure propagates with the separation, and in turn the cantilever rotates on top of the right column that is not able to balance the flexural action. As result, at 2 sec from the failure initiation, the main cantilever portion collapses rotating around a transversal axis on top of the right column, while the mechanism spread with the failure of the upper part of the structure. Following this instant, the collapse simulation is stopped considering it will be not realistic then, considering the first ring below is not modelled. Indeed, aside of a free mechanism up to a potential impact with the second ring itself, because of the presence of the first ring the collapsing portion would have impacted on it.

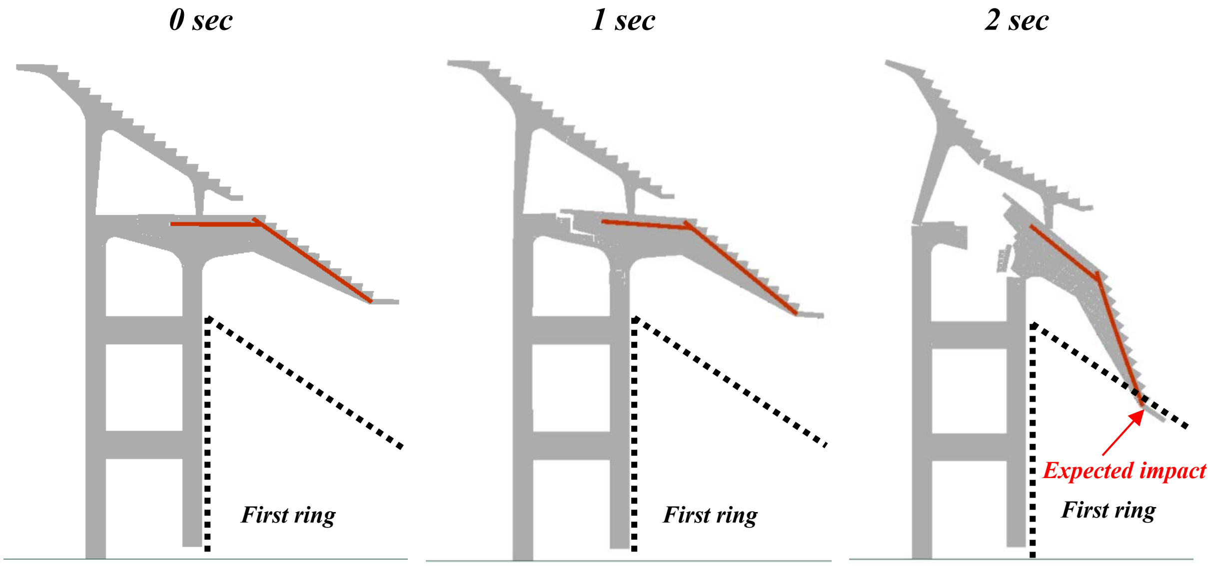


Figure 10 A theoretical collapse mechanism of the modelled system subjected to a theoretical dynamic excitation with a frequency of 4.5 Hz. (The time starts at the beginning of failure).

8 CONCLUSIONS

In this work, a preliminary investigation of the structural performance of an RC cantilevered grandstand, namely of the San Siro Meazza stadium in Milan, was carried out through the development and analysis of AEM-based models. Dynamic results obtained through eigen values analysis have shown that the AEM is able to reproduce the dynamic properties of large RC cantilever structures consistently with those from traditional FE models and experimental data. Nonlinear pushdown analyses, conducted by applying an incremental vertical load on the main cantilevered portion, have shown the influence on the ultimate vertical load and failure mechanism because of the retrofitting of the main cantilevers. Indeed, the introduction of the external steel plates would have led to a remarkable increase of the ultimate vertical capacity and a modification of the potential failure mechanism. Nonlinear time-history analyses have been conducted to preliminary assess the dynamic response of the structure (with the retrofitting) when subjected to a possible loading model representing human crowd. And although only two significant frequencies have been considered for the loading excitation, results have indicated that the structure may experience dynamic properties modification depending on the loading intensity and characteristics, while in some cases failure could occur. Finally, the nonlinear response of a such large RC cantilevered grandstand has to be further explored considering additional frequencies of the dynamic excitation, namely in the range of human activities, different loading models, crowd synchronism, as well as the mutual interaction between people and structure.

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