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Damage scenario-driven strategies for the seismic monitoring of XX century spatial structures with application to Pier Luigi Nervi's Turin Exhibition Centre

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ABSTRACT

Damage and stiffness degradation in non-structural elements are known to strongly affect the global dynamic response of buildings subject to seismic excitation. This is particularly true for complex structural schemes, such as those characterizing XX century shell and spatial architectures. Moreover, degrading behaviours will strongly influence the design of seismic monitoring systems, which are deemed to be a non-invasive protection strategy for cultural heritage. This paper concerns the optimal sensor placement for vibration-based monitoring of one of the vaulted structures realized by Pier Luigi Nervi in the Turin Exhibition Centre. Based on finite element numerical models and optimization algorithms, the sensor placement in such peculiar structures should also take into account the possible effects of non-structural elements. The objective function used for optimizing sensor placement has been modified in order to account for the progressive damage in infill walls. The final scope of this work was, thus, to propose a damage scenario-driven sensor placement strategy accounting for concurrent damage scenarios, as those affecting spatial architectures.

1. Introduction

In recent years there has been an increase in the recognition of the cultural significance of modern architecture [1–3]. However, there are still challenges to secure its protection and conservation. In fact, many of the characteristics of modern architecture – such as the use of new and innovative construction methods and materials, the role of architecture in social reform, and the development of new building types and forms – challenge traditional conservation approaches and raise new methodological and philosophical issues [1,4]. An area of conservation that requires attention is seismic provision. In fact, modern architecture buildings were designed and built with no, or very limited, seismic provisions, due to the lack of reference technical standards at the time of their construction [5]. With a view to the restoration and renewal of these buildings, a careful assessment of the performance of their structures is a priority, especially when they are situated in a seismic risk area.

There is a growing body of literature that recognises the importance of the Modern Architectural Heritage [1,2,4], especially in the case of modern masters such as Pier Luigi Nervi. Recently the works of Pier Luigi Nervi have been the object of special attention: various papers and books have analysed his work [6–9], a special double issue of an international journal dedicated to Nervi [10], and the itinerant international exhibition "Pier Luigi Nervi. Architecture as challenge" has been set up since 2010 [11].

However, there remains a paucity of detailed studies on the structural behaviour and dynamic response of spatial of the 20th century. In the case of Pier Luigi Nervi there is very little information about effects of seismic loads on his structures, although they were realized mostly in seismic risk areas. In fact, the existing studies focus mainly on stylistic and architectural aspects, and much less on structural behaviour. The only studies addressing structural aspects concern the Palazzo del Lavoro in Turin (1961) [5,12], and the Palazzetto dello Sport in Rome (1960) [13,14].

In this paper an iconic building realized by Pier Luigi Nervi will be examined. Turin Exhibition Centre is a spatial structure masterpiece, admired for its daring and innovative conception. Nonetheless it has sadly been abandoned for a long time. This lack of maintenance is starting to induce serious preservation problems. The Municipality of Turin, that owns the building, is considering the assignment of a new function for the complex.

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The building consists of different blocks, including two famous halls: Hall B and Hall C. For the largest hall, known as Hall B, there is a relevant body of literature [6,15–17]. However, few studies have investigated the smaller Hall C [6]. Consequently, very little is known about its construction and how Nervi conceived its structure. Furthermore, no studies have to date analysed its structural behaviour or its response to ground motions.

This paper reports on the placement of sensors in a seismic monitoring system for Hall C in Turin Exhibition Centre. The authors examined the original design documents and the information from previous surveys. Subsequently, it was created a Finite Element (FE) model of Hall C to evaluate the vibration modes and to calculate the Modal Assurance Criterion (MAC) matrix [18,19]. The concept of MAC was used to define a general methodology for sensor placement in complex spatial structures.

Another issue addressed in this paper concerns the possible effects of non-structural elements (infill walls) on the seismic behaviour of spatial structures. In fact, both halls in Turin Exhibition Centre (Hall B and C) were realized without accounting for seismic action, but only for static configurations, in accordance with the technical standards of the time. The results of the numerical analysis demonstrate that infill walls undergoing seismic damage or degradation [20,21] may significantly affect the global dynamic response of these spatial structures, and consequently the optimal monitoring configurations. These observations led to the development of a novel numerical method to take into account different seismic damage configurations. This sensor placement strategy consists in an extension of the form of an objective function to be minimized in a penalty procedure and is presented in this paper.

The aims of this investigation were (i) to assess the vulnerability of historical spatial structures to seismic actions, with special attention to Nervi's structures; (ii) to propose efficient solutions for the optimal sensor placement (OSP) configurations in such complex architectures. This paper also seeks to contribute to increase existing knowledge about the great legacy of Pier Luigi Nervi. To this end, the present study is aligned with similar studies carried out by other authors on the work of great structural engineers such as Eduardo Torroja [22–24], Félix Candela [8,25,26], Robert Maillart [27], Eladio Dieste [28], Rafael Guastavino [29] thus contributing to the development of the discipline of structural criticism [23,30].

The paper begins with an overview of Turin Exhibition Centre and its construction innovations (Section 2). It then focuses on Hall C, describing the design and the structural conception conceived by Pier Luigi Nervi. Section 3 regards a seismic assessment of the structure, considering the potential seismic damage to non-structural elements. Section 4 describes the application of an optimal sensor placement strategy to two different cases: a first one corresponding to the undamaged structure, and a second scenario that considers a possible damage of the infill walls. Section 5 concerns the application of the novel sensor placement strategy. Finally, the main conclusions are presented in Section 6.

2. Turin Exhibition Centre

Pier Luigi Nervi (1891–1979) was one of the greatest and most inventive structural engineers of the 20th century [10]. With his masterpieces, scattered the world over, Nervi contributed to create a glorious period for structural architecture [11]. Nikolaus Pevsner, the distinguished historian of architecture, described him as "the most brilliant artist in reinforced concrete of our time" [31].

Considering the importance and the peculiarity of this legacy, a cross-fertilization is needed between engineers on one side and historians and architects on the other side, to lay the base for an advanced scientific approach to the documentation and conservation of the 20th century architectural heritage [6,32]. Currently the *Pier Luigi Nervi Project Association* contributes to keep track and knowledge about the life and work of Pier Luigi Nervi. At present the halls built by Pier Luigi Nervi are used occasionally for short exhibition events. This state of abandonment is inducing some preservation problems, in particular in the Hall B, caused by moisture and infiltrations [6]. In the past the building was interested by works of ordinary maintenance [32], and the only inspections were done in 2002 and interested the larger Hall B [33], while the Hall C was only visually inspected. The Turin Exhibition Centre (Palazzo di Torino Esposizioni) was built in 1948 for the 31st international Auto Show. The masterplan prepared by Roberto Biscaretti di Ruffia planned for a large complex on the remains of the Palazzo della Moda, damaged during World War II. The tender was committed to Nervi and Bartoli, whose project proposed a new construction system that combined prefabrication and the use of ferrocement. The building consists of the main Hall B and the smaller adjacent Hall C, both designed and constructed by Nervi (1947–1948, and 1950). In Fig. 1(a) it is reported the Turin Exhibition plan with the Hall C coloured in red[1].

2.1. Design and construction innovations

Nervi pointed out how the limited amount of time allotted for construction and the unusual size of the building created a series of uncommon construction problems that were difficult to resolve using traditional building systems. Nervi employed new construction procedures that he studied for some years before this project. In fact, he had already successfully used these procedures with his engineering firm Nervi and Bartoli, although only with the realization of experimental buildings. Some of these are the small storehouse in the Magliana area in Rome (1946) [34], the wharf Conte Trossi in San Michele di Pagana (1947) and the ceiling of the pavilion at the Milan Fair (1947) [15].

For Pier Luigi Nervi, the Turin Exhibition represented the first concrete possibility to apply the principle of structural prefabrication, which united in a single large-scale vaulted structure his highly personal use of ferrocement with the extensive use of prefabricated elements. This combined use of two different technologies for the construction of large concrete shells would become one of the distinctive trait of Nervi's work. In fact, in the fifties concrete prefabricated elements were especially used in simple buildings, and the precast industry and standardization was not as advanced and broad as in the following decades [35].

Dating back from the thirties, thin shell roofs in concrete were seen as "the starting point for the specific solution of the vaulting problems" [36]. Vaulting forms were created taking inspiration by simple ideas grounded in the laws of nature [37] both empirically and mathematically. After World War II, structural research, particularly into thin concrete shells for vaulted structures, achieved extraordinary results. Researchers and designers of the time adopted a highly pragmatic approach to the industrialisation of construction systems, optimal use of materials, and structural analysis [38]. Nervi was one of the leading structural designers of the period together with Eduardo Torroja, Anton Tedesko, Nicolas Esquillan, and many others.

Turin Exhibition Centre is one among many spatial buildings that were built at the time. In the same decade other daring and innovative vaulted buildings were constructed all over the world. In the U.S.A., Eero Saarinen designed the Kresge Auditorium for the Massachusetts Institute of Technology (1953–1956), while in

[1] For interpretation of color in Fig. 1, the reader is referred to the web version of this article.

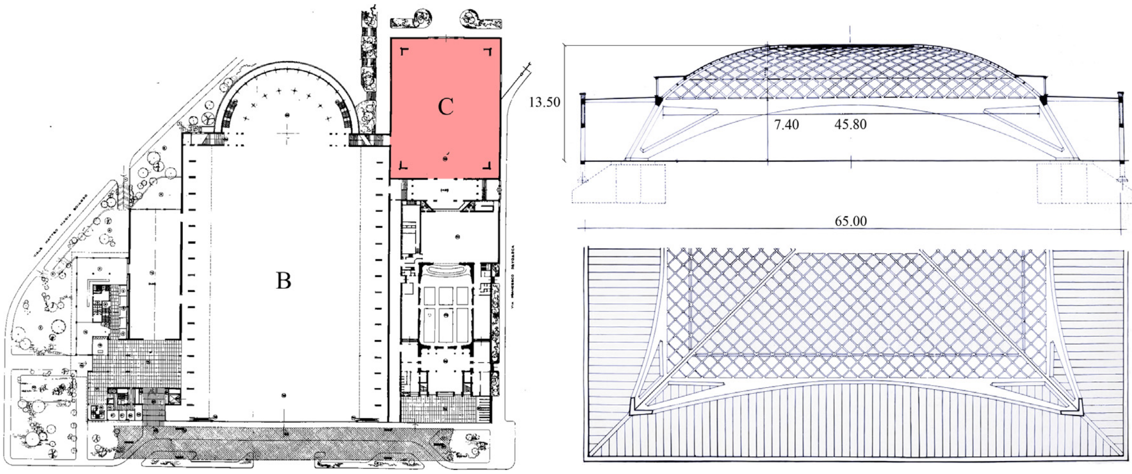

Fig. 1. (a) Turin Exhibition plan: the large rectangular space in the centre with the semi-circular apse at the end is Hall B, built in 1948 (B); the smaller rectangular shape in the upper right hand corner is Hall C, built a year later (C). (b) Plan and section of Hall C (Pier Luigi Nervi, drawing from Rassegna tecnica della Società degli ingegneri e architetti in Torino).

Sidney Jørn Utzon designed the famous Opera House (1956). In Europe, Nicolas Esquillan completed the CNIT in 1958, while Eduardo Torroja, Eugène Freyssinet, and Franz Dischinger had built their most iconic thin vaulted structures during the thirties, and were strongly influencing designers and engineers with their buildings and lessons.

In particular, it is possible to make a parallelism between Torroja and Nervi. Both of the two great builders also wrote books and articles in which they explained their reflection and aesthetics. Torroja began designing concrete structures in the late twenties, and in the mid thirties created three major works that characterize his vision: the Zarzuela Hippodrome Roof in Madrid (1935), the Fronton Recoletos of 1935, and the Market Hall at Algeciras (1934). Billington says that the major differences between the two designers are two: "first, whereas Nervi sees shells as ribbed Torroja sees them as ribless, hence at Algeciras the dome is smoothly curved inside and out; second, since domes tend to spread horizontally, Nervi designed exterior ribbed buttresses as the shell supports, whereas Torroja avoids buttresses by connecting vertical supporting columns with a polygonal ring of horizontal ties" [37].

2.2. Hall C

In 1949 the directors of the Turin Exhibition decided to enlarge the building by adding another hall, Salone C. They again entrusted the design to Nervi, who had to face two main problems. Firstly, the restricted time allowed to build the Hall with a size of 65×50 m. Construction was to start in the autumn after the closing of the exhibition and had to be ready by the following June. Secondly, the conceptual problems of harmonizing the new work with the character of the previous one without repeating its shapes [39]. After many trials in which Nervi also tried a cross vault, he adopted a vaulted roof completed by a perimetric slab of about 10 m in length. Fig. 1(b) shows the plan and a section of Hall C. This solution permitted the use of the structural prefabrication already employed for the half-dome of Hall B [40]; Fig. 2(a) shows a picture of the Hall C during its construction in 1950, with the assembly of the diamond elements in ferrocement on the vault centring already used in the half-dome of Hall B. Moreover, this solution was preferred because it expressed the advantage of a more organic and expressive static formula and a significant financial advantage. For the perimetral area Nervi adopted a new system of undulated ferrocement beams, which he had contemplated many times but had never actually used before. Immediately after the completion of the pavilion, Nervi registered patent no. 465636 in Rome on 19 May 1950, entitled "Building procedure for creating flat or curved load-resisting surfaces consisting of grids of reinforced concrete ribbing, possibly finished with connecting concrete slabs between the ribs" [17]. This patent, together with the patent no. 445781 registered on August 26th, 1948, was gradually developed until their final configuration with the construction of Turin Exhibition Centre's Halls. These patents would be adopted by Pier Luigi Nervi in the following decades until becoming a typical feature of the globally recognized Nervi's style [16,17].

The large vault of the Hall rests on four arches with a plane inclined according to the resultant of the thrust of the vault and the weight of the perimetral roof system. Fig. 2(b) shows an interior of the hall. The horizontal rigidity of the roof system, appropriately reinforced by two-way reinforcing in the upper slab, was conceived to redistribute and balance the thrust of the vault. In order to bring evidence of this redistribution effect, Nervi assumed for the undulated slabs a rectangular frame model embracing the vault. Horizontal actions were not explicitly taken into account in the design. In fact, in his paper Nervi shows how he conceived the structure of the hall and the calculation methods performed for its assessment [39]. Nervi explains that the building was conceived as a static system and "the vault was calculated both as a thin membrane vault, and as a vault formed by elemental arches each resistant on their own. The sizing of the sections was the result of a reasoning of approximation: the arches, the horizontal reaction of the floors, and then the strengthening of these elements were verified for each hypothesis on the basis of reasoning, considerations and attempts to investigate and understand the actual static behaviour of system" [39,41]. There is no mention of any calculations related to wind actions.

3. FE modelling and analysis

Architectural heritage structures require an accurate process of documentation to gather any useful information, as well as a multidisciplinary approach to clarify their most controversial aspects. Original drawings, the description of relevant historical events and intervention evidence were collected and compared for this study. All the significant data as well as the information gathered from a survey provided the geometric and dimensional characteristics of the structure, as well as the classification of the elements

Fig. 2. Hall C of Turin Exhibition Centre during its construction in 1950: (a) assembly of the diamond elements in ferrocement on the vault centring © Archivio MAXXI Roma, Fondo Nervi. (b) An interior of Hall C today (photo by the Author).

and the materials employed. The simplified geometric model was subsequently used for the construction of the FE model [42,43]. The main structural elements of the building are evidenced in Fig. 3.

3.1. Numerical modelling

The finite element model (FEM) was constructed in ANSYS software (see Fig. 4b) on the basis of the original documentation gathered. At this stage of the analysis linear models were preferred, also due to the lack of experimental data on material properties. The ribs of the vault and the ring beam are modelled as mono-dimensional elements (BEAM), whereas the ferrocement filling slabs and the arches are modelled with bi-dimensional elements (SHELL). The perimetral ferrocement slab is modelled neglecting its undulated elements and realizing an equivalent slab. The structure was reasonably assumed to be clamped at the base. Furthermore, in this linear FEM, the continuum underlying the structure was disregarded, based on the assumption that in ambient vibration conditions soil-structure interaction has no effects on vibration modes. A 2% modal damping was assumed for all frequencies. Material properties were deduced partly from the values founded in literature in Nervi's projects and partly from typical values of the period [34,41]. The infill walls, built in hollow bricks,

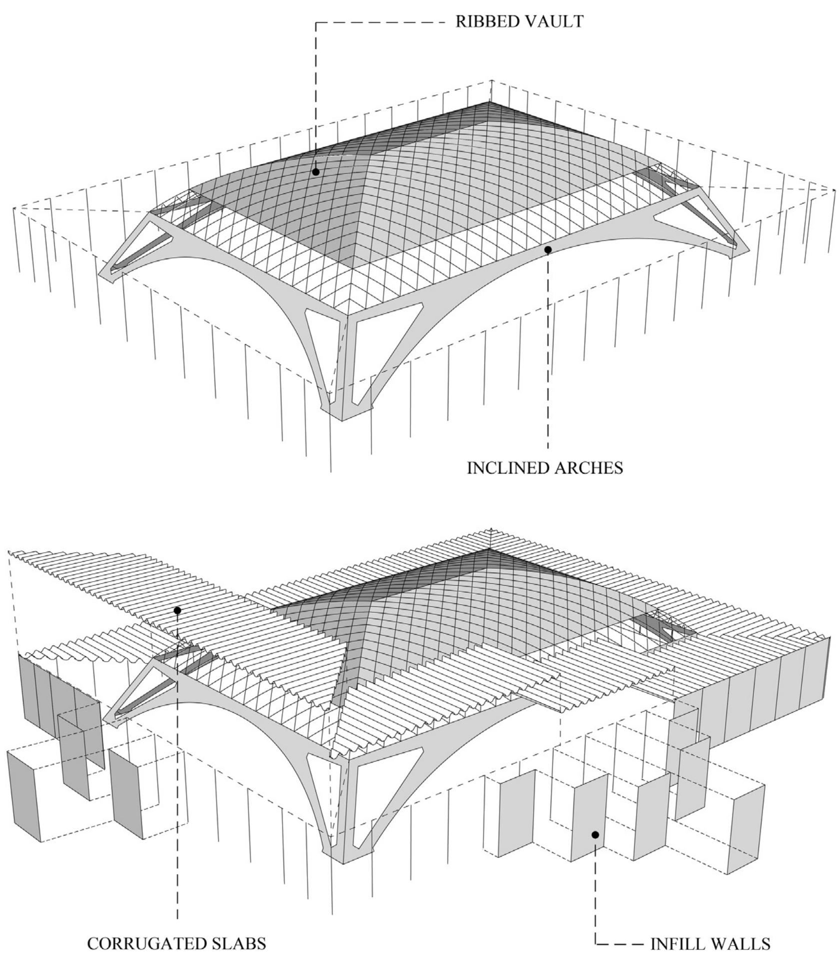

Fig. 3. The structure consists of a vaulted roof supported by four inclined arches. It is completed by a perimetral slab of about 10 m in length, sustained by a series of slim pillars and infill walls.

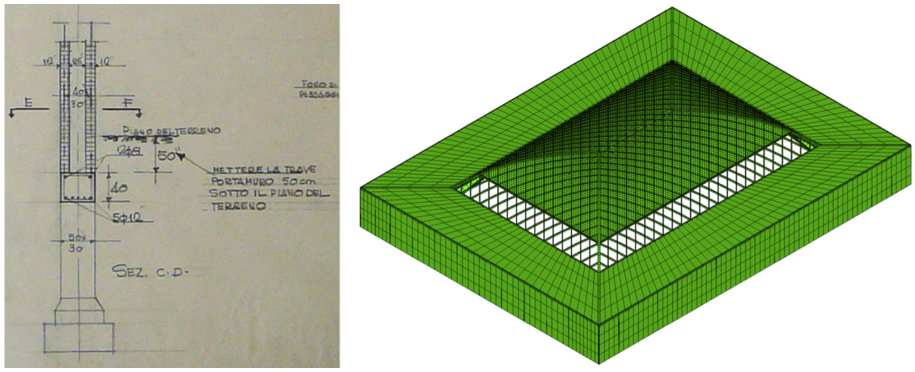

Fig. 4. The original scheme showing a section of an infill wall, reporting its dimensions, together with the pillars' foundation (a) [41]. A 3D view of the global finite element model of the hall (FEM1) showing its FE mesh (b) [43].

Table 1
Materials and properties.

Material	E GPa	Poisson's Ratio	ρ kg/m^3
Ferrocement	26	0.2	2500
Masonry (infill walls)	2	0.2	1100
Reinforced concrete	30	0.2	2500

are placed between the pillars and the corrugated slabs (Fig. 3). As a result, the elastic moduli are assumed as shown in Table 1, which is consistent with the information reported in [34,41].

To simulate different damage scenarios two FEMs were built: FEM1 considers the structure in its integrity, with infill walls; FEM2, instead, represents the structure without infill walls, in order to simulate a full damage state for non-structural elements during the ground motion. FEM1 has 19,153 elements and 25,912 nodes. FEM2, instead, counts 18,197 elements and 24,464 nodes. After a static validation of the models, the FE eigenvalue problems were solved in order to evaluate the main modal characteristics in both damaged and undamaged state.

3.2. FEM vibration modes

3.2.1. Model accounting for non-structural elements (FEM1)

The modal shapes have been calculated considering dead loads. Some of the modes extracted from the model are listed in Fig. 5. Table 2 illustrates the modal properties of the structure. The first two modes, for example, are governed by the ridge ribs and have a small participation, similarly to other modes that are not reported. Thus, for the purposes of this study, only the modes reported in Table 2 will be considered.

It was decided that the variation of the modal frequencies would be studied as a function of the damage assumed in non-structural elements. The damage was simulated as a reduction of the elastic modulus of the infill walls. Fig. 6 reports the variation of the main modal frequencies as a function of the apparent Young modulus of the infill walls. Values of the modulus smaller than 2.0 [GPa] correspond to a damage in the infill walls.

3.2.2. Model without infill walls (FEM2)

It was then decided to analyse the behaviour of the structure without the infill walls, in order to simulate the possibility that non-structural elements undergo damage phenomena during the

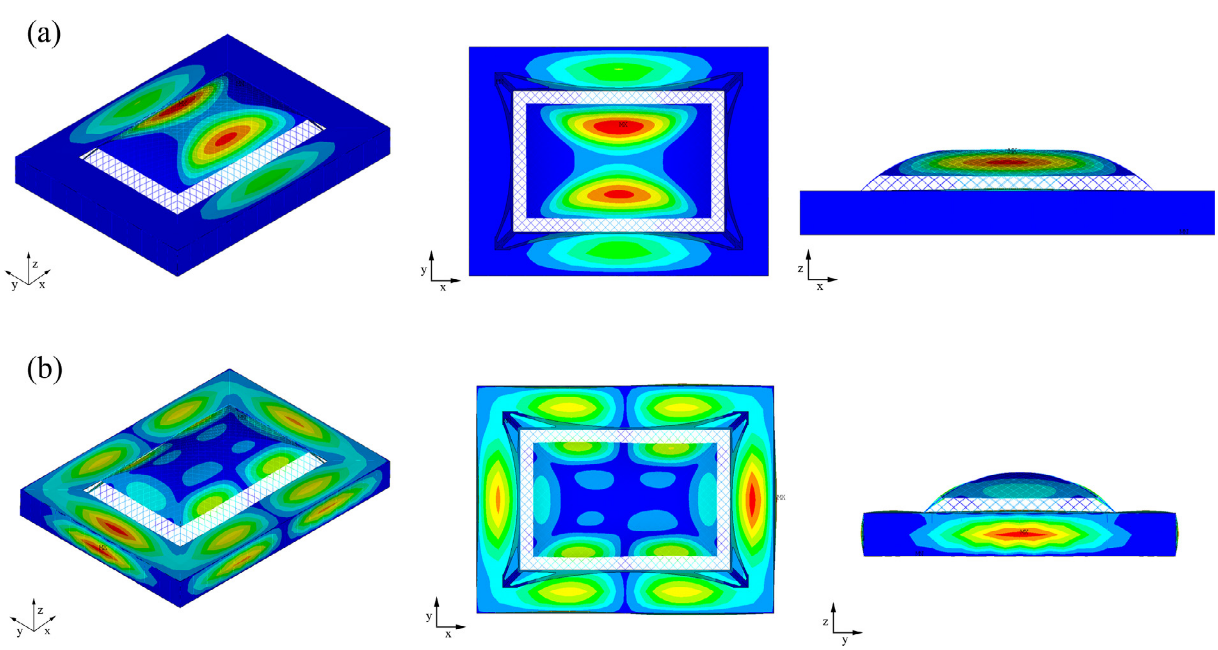

Fig. 5. Third (a) and sixth (b) modal shapes.

Table 2
Modal properties of the structure and classification of the main vibration modes, considering the presence of the infill walls.

Mode	Frequency	Cumulative mass fraction			Mode classification
	Hz	x	y	z	
3	4.168	0.000	0.135	0.000	1st bending Y: vault, arches, slabs, walls
4	4.330	0.000	0.000	0.670	1st vertical: longer slabs, vault, arches
6	4.788	0.247	0.000	0.001	1st bending X: vault, arches, slabs, walls
7	4798	0.013	0.000	0.258	2nd vertical: smaller slabs, arches
8	4801	0.358	0.000	0.005	2nd bending X: slabs and walls
9	4828	0.000	0.401	0.000	2nd bending Y: vault, arches, slabs, walls
11	5.025	0.000	0.376	0.000	3rd bending Y: vault, arches, slabs, walls
14	5.225	0.379	0.000	0.000	3rd bending X: vault, arches, slabs, walls

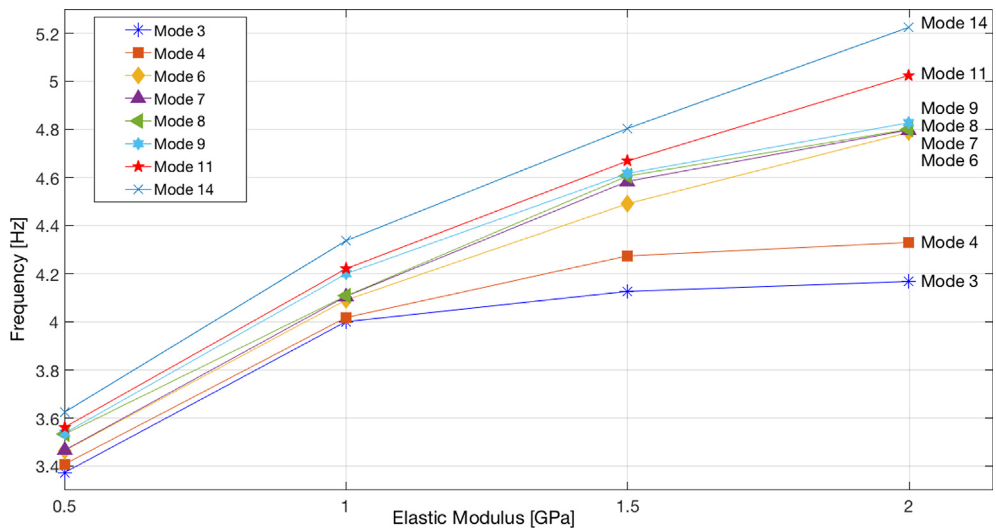

Fig. 6. Mode and frequency variations depending on the damage of the infill walls.

ground motion. Also in this case, the modal shapes have been calculated considering dead loads. Some of the main modes extracted from the model are listed in Fig. 7. Table 3 shows the modal properties of the structure, the principal modes and their classification.

3.3. Comparison between the two models and emerging issues

The main issue to be raised concerns the out-of-the-plane movements of the arches, as highlighted by low frequency vibration modes with high participation. This type of behaviour is induced by the inertial mass of the arches. It can also be observed that, when not effectively contrasted, these out-of-plane movements constitute a serious vulnerability factor for the entire Hall, as they may induce overturning, pounding and interaction with structural and non-structural elements (e.g. the corrugated slabs). In addition, a low ductility mechanism is associated to overturning, which results in important amplification of the seismic response. Another critical aspect lies in the bending (translational) mode of

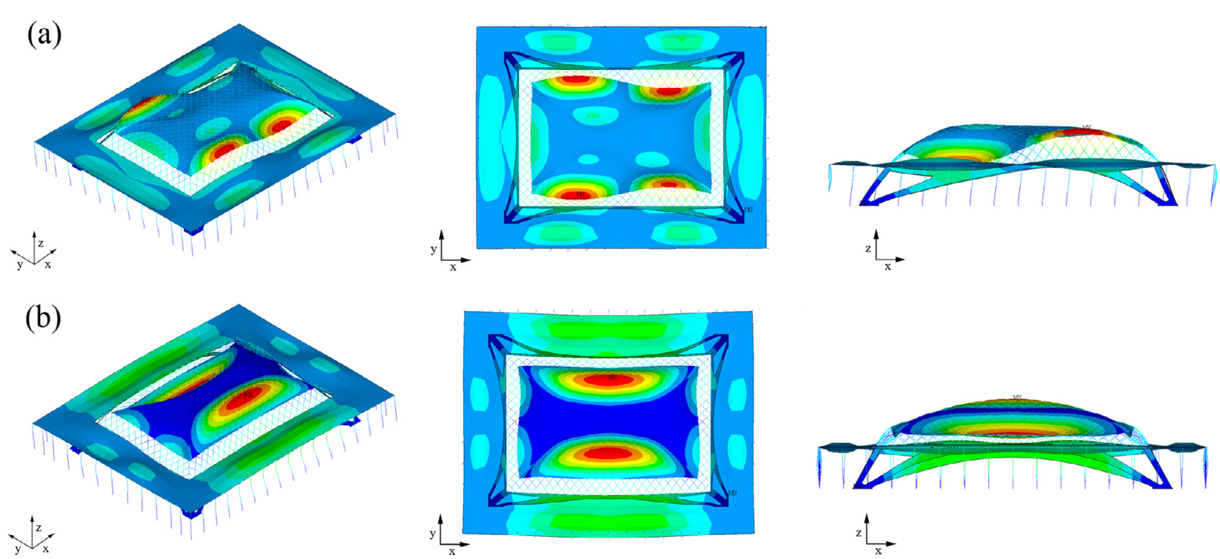

Fig. 7. 23th (a) and 24th (b) modal shapes.

Table 3
Modal properties of the structure and classification of the main vibration modes, for the structure without the infill walls.

Mode	Frequency	Cumulative mass fraction			Mode classification
	Hz	x	y	z	
1	3.026	0.000	0.028	0.000	
2	3.041	0.000	0.000	0.425	longer slabs
3	3.136	0.012	0.000	0.000	
4	3.160	0.000	0.000	0.211	shorter slabs
11	3.488	0.013	0.000	0.000	
12	3.561	0.000	0.006	0.000	
14	3.711	0.000	0.020	0.000	
20	4.443	0.010	0.000	0.000	
22	4.579	0.001	0.000	0.004	
23	4.711	0.892	0.002	0.000	1st bending X: vibration of smaller arches out of their plane
24	4.726	0.000	0.617	0.000	2nd bending Y: In-phase vibration of longer arches
25	4.746	0.005	0.310	0.000	3rd bending Y: mode of the vault
31	5546	0.000	0.000	0.012	2nd vertical: arches
36	6148	0.013	0.000	0.000	
37	6253	0.000	0.000	0.079	
43	7064	0.046	0.000	0.000	2nd bending X: vault
45	7221	0.000	0.000	0.259	4th vertical: vault

the rigid cap of the roof with respect to deformable contour ribs. Due to Nervi's structural conception, in Hall C this mode happens to fall in an amplified region of the typical seismic response spectrum. Also due to the low ductility associated with these mechanisms, in case of moderate seismic events, the ribs can be affected by local interaction with the roof.

The analyses in Section 3 evidence an inherent vulnerability of this kind of structure to seismic actions. This is expected to happen especially in the presence of heavy structural elements, which are conceived to withstand important gravity loads. Nervi's building has a complex structural scheme. In addition, the model must take into account the effect of non-structural elements, as well as their possible mechanical degradation during a strong motion. In particular, infill walls exhibit a typically elastic-brittle behaviour. Such premises constitute critical aspects that will affect the design of a vibration-based monitoring system.

4. Sensor placement

Vibration based structural health monitoring (SHM) represents a non-invasive technique which exploits the natural vibration of structures. It is useful to describe the structural conditions of historical buildings and to optimize the plan of maintenance as well as the restoring interventions [44,45]. A SHM system is the result of the integration of several sensors, devices and auxiliary tools.

When designing a SHM system, it is necessary to first perform an accurate analysis of the structural behaviour, in order to monitor the most expressive and sensitive parameters. In order to design an optimal monitoring system for the Hall C, that is not currently monitored, the FE models were firstly used to identify the main structural portions and elements that affect the dynamic behaviour of the structure. Based on the FE vibration modes, the measuring points (henceforth termed setups) are defined. The placement of sensors is a crucial stage of the design of a monitoring system. The optimal location of sensors can be trivial in structures with simple geometry where the problem can be solved simply relying to the experience of the operator, or with a trial-and-error approach. However, architectural heritages are often complex structures calling for more sophisticated strategies, such as optimal sensor placement (OSP) [45].

OSP techniques play a significant role in enhancing the quality of modal data for SHM. This is particularly true for large civil structures, where a limited number of sensors are available to monitor a huge number of movements [46]. A popular OSP approach is the one proposed by Kammer [47] and consists in a minimisation of the norm of the Fisher information matrix. Another way to solve the problem is the modal kinetic energy-based method as proposed by Salama [48] and Chung [49]. A deep review of OSP methods with particular emphasis on vibration measurements can be found in [50] and [51]. Some robust approaches to the solution to the OSP problem refer to the concept of Auto-MAC matrix [18,52]. As for optimization techniques, many engineering applications rely on genetic algorithms (GAs) as their use is known to reduce the probability to incur in local minima.

In the case of complex civil structures such as Hall C, a simple and robust criterion for the solution of the problem of optimal placement of accelerometers is to be chosen. It refers to the mentioned MAC [18], a matrix containing element indices of the correspondence of two modal vectors. In fact, the elements of the matrix can take values between 0, which indicates vectors devoid of correlation, and 1, which indicates vectors equal. The generic term of MAC can be defined as:

$$MAC_{ij} = \frac{(v_i^T v_j)^2}{(v_i^T v_i)(v_j^T v_j)} \tag{1}$$

where v_i and v_j are respectively the i-th and j-th column of matrix v, which is the reduced modal matrix of the degrees of freedom corresponding to possible measurement positions [18]. The goal of the OSP is to minimize the elements outside the diagonal of the MAC matrix. The genetic algorithm searches for the best set of positions that allow to minimizes each term MACij ($i \neq j$). This allows to distinguish the modes to be monitored on the basis of a given numbers of accelerometers. The objective function used to solve the problem of the OSP is based on the above stated definitions, as it derives from the simple subtraction of terms related to the MAC calculated using the modal matrix (MAC_R) with those of the ideal MAC (MAC_I):

$$error = \frac{\sum_i \sum_j (MAC\_R - MAC\_I)}{n} \tag{2}$$

where Eq. (2) represents the objective function to be minimized and n is the number of elements of the matrix.

The OSP algorithm was applied to the model of Hall C of Turin Exposition Centre. The FE models enabled the location of the main structural elements that affect the dynamic behaviour of the structure and the calculation of the solution of the eigenvalue problem, whose eigenvectors are depicted in Tables 2 and 3. The measuring points are defined based on the FE vibration modes. The setup chosen has 68 possible positions for accelerometers corresponding to 174 channels. Positions were distributed on the ribbed vault, as well as along the principal ribs, on the two arches, and on the perimetral slabs. The possible positions are reported in Fig. 8. Accordingly, the potential channels were distributed as follow: 68 channels in the X direction, 68 channels in the Y direction, and 38 channels in the Z direction. In addition to their correct positioning, different scenarios were explored by varying the number of sensors from 2 to 40. The choice about the top bound of 40 sensors was based on previous experiences [45,53] and the typical cost of SHM systems. It was also investigated if and how the optimal sensor placement could change, considering the presence or the absence of the non-structural elements such as the infill walls. For this reason, the OSP algorithms were applied first on the structure with infill walls, then on the structure without. The

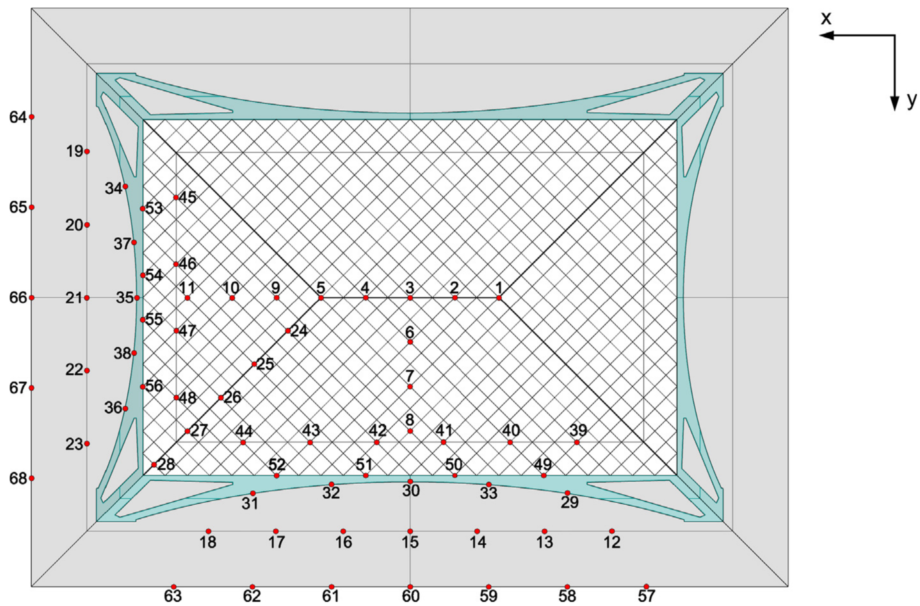

Fig. 8. Possible positions for the accelerometers.

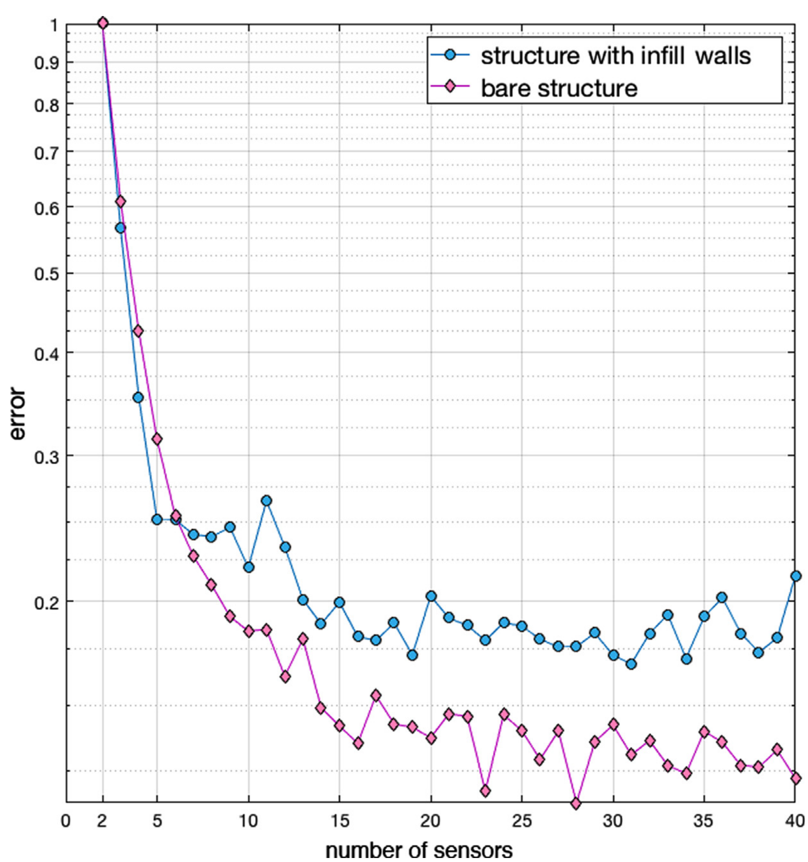

Fig. 9. Normalized error function of the structure accounting for different scenarios: undamaged and bare structure.

algorithm required some numerical constraints, such as a limited number of sensors and a limited number of potential positions. With these premises different setups were considered.

4.1. An OSP based monitoring system for Hall C

As seen in the previous sections, non-structural elements may appreciably affect the vibration modes, and consequently the OSP. For this reason, it was decided to develop a damage scenario-driven sensor placement strategy, and the optimization algorithm was run on the structure accounting for damage scenarios. A first setup was designed based on the undamaged configuration, i.e. the setup accounting for the infill walls. The second setup, instead, was associated to the bare structure, in the assumption that all non-structural elements would undergo a complete degradation at the end of the strong motion. The algorithms were developed using the MATLAB software.

The charts in Fig. 9 depict the final error in the objective function corresponding to the two optimized configurations. In this case the abscissa represents the number of sensors used by different sensor placements. While error for the undamaged structure stabilizes at around 15 sensors, the bare configuration appears to call for additional sensors. This result can be ascribed to the influence of local movements activated at the infill walls (non-structural elements), the dynamics of which are captured with higher spatial resolution.

Subsequently, the optimization was performed by weighting different contributions on the basis of the importance of the different modes. The introduction of the weighting coefficients led to an improvement of the decoupling performance.

The optimal solutions found by the algorithm in the investigated range of modes are illustrated in Fig. 10, for the two cases (with and without infill walls). From the optimal sensor configurations (Fig. 10a and b) the distribution of the sensors in the two layouts is clearly different. In setup 2–2 (b), where the infill walls are not taken into account, the sensors are mainly located in the arches and, contrary to setup 1–2 (a), the vertical movements appear to be more important. In general, setup 2–2 is characterized by monitoring of vertical vibrations. By contrast, in setup 1–2, where the non-structural elements are taken into account, many sensors are distributed on the vault and on the perimetral slabs. Most of the sensors are positioned along the minor defence line.

Based on these findings, an integrated design strategy of the monitoring system could be appropriate for this type of structures, for which non-structural elements can play an important role during an earthquake. The integrated design should consist in a union of the two different configurations. Fig. 10(c) reports a direct overlapping of the previous configurations (setup 1–2 (a) and setup 2–2 (b)).

More generally, the objective function for OSP in such complex structures should take into account many structural configurations corresponding to different damage scenarios potentially activated during a seismic event.

5. An optimal sensor placement strategy accounting for damage scenarios

As it has emerged from Section 4.1, the influence of non-structural elements and their interaction with the structure cannot be neglected when designing a dynamic monitoring system. In particular, it has been observed that when non-structural elements, such as infill walls, undergo seismic damage or degradation, they may significantly affect the global dynamic response.

Such an aspect is even more important for historical spatial structures, realized in a period when seismic or other dynamic actions were not taken into account in the design process. These structures were frequently conceived on the basis of gravity loads

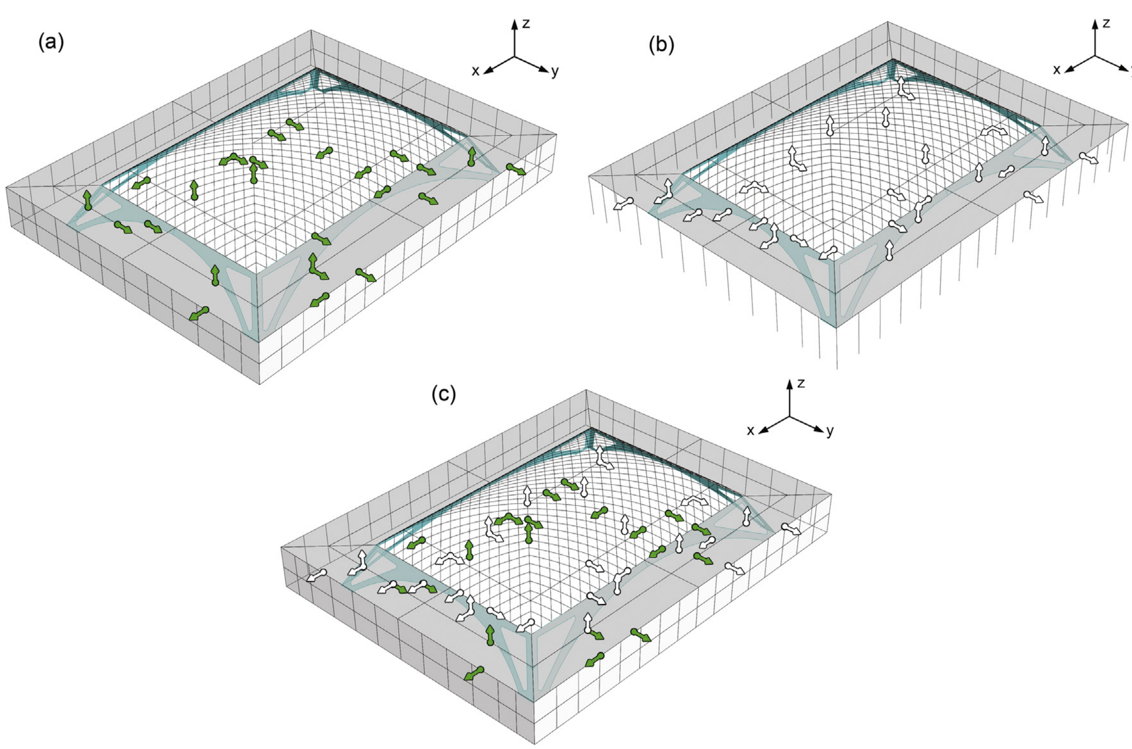

Fig. 10. Sensor configuration layout Setup 1–2 (a), Setup 2–2 (b), and their overlapping (c). The sensors of Setup 1–2 are coloured in green, while the sensors of Setup 2–2 are coloured in white.

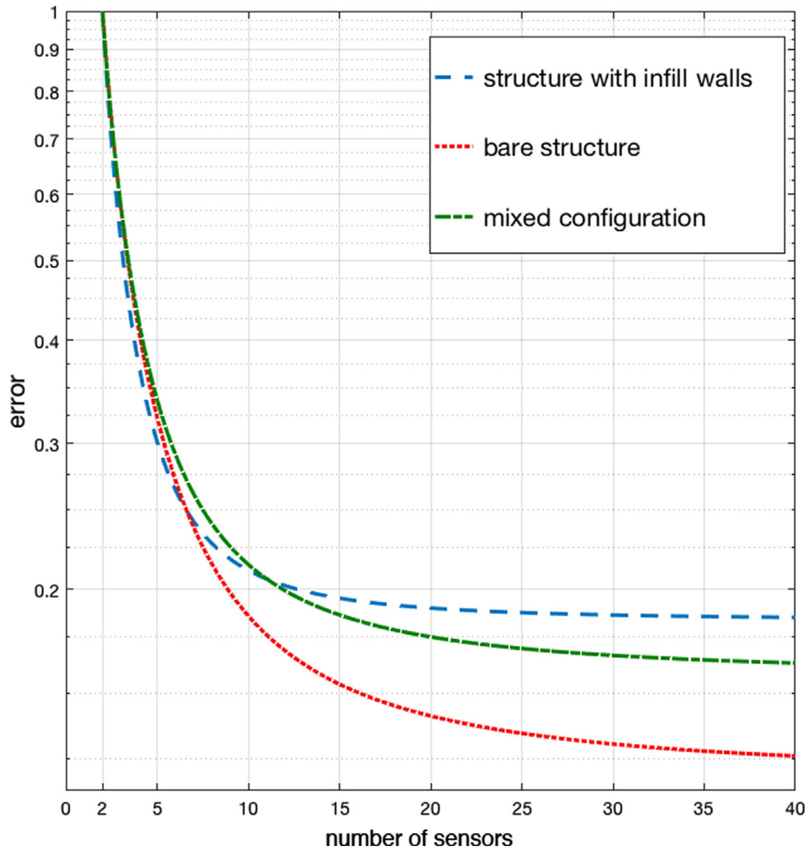

Fig. 11. Results obtained from the new OSP configuration.

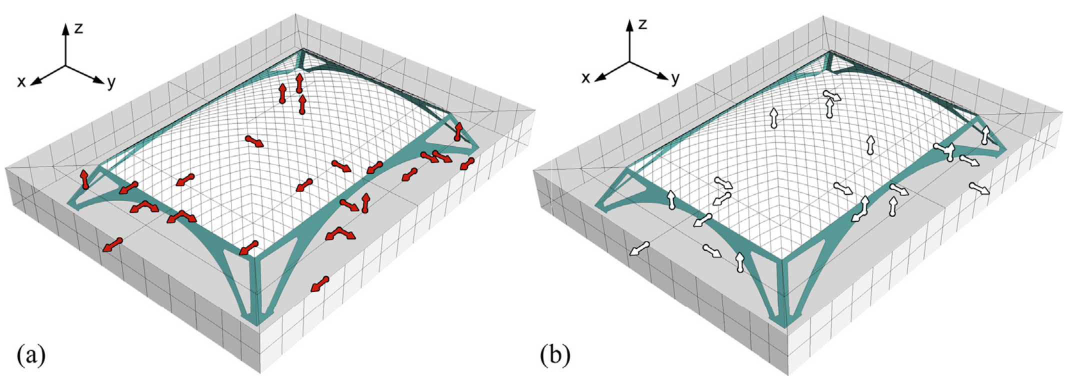

Fig. 12. Sensor configuration layout Setup 1 (a), Setup 2 (b).

and this purpose is clearly expressed in their design. A development of these observations could consist in a modification of the form of the objective function that takes into account different seismic damage scenarios. Eq. (2) is accordingly updated:

$$error = error_w + error_b \tag{3}$$

where $error_w$ represent the structure with the infill walls, and $error_b$ is the bare structure without the infill walls. The results obtained from the new objective function are presented in Fig. 11, where the trends of normalized error functions are reported and compared for the three investigated scenarios.

Fig. 12 reports the optimal sensor configurations obtained for the mixed configuration. Also in this case, an additional optimization was performed by weighting different contributions on the basis of the importance of modes (Setup 2). The integrated design strategy of the monitoring system permits a more rational use of the instruments. Indeed, most sensors appear to be localized in the elements that govern the seismic response of this complex structure: the arches and the vault.

6. Conclusions

This paper concerns OSP methods to be used in the design of seismic monitoring systems for concrete spatial structures, with special emphasis on the vaulted structures realized by Pier Luigi Nervi. Sensor placement strategies for such peculiar structures should include in their formulations the possible damage and stiffness degradation in infill walls and other non-structural elements.

Accordingly, the objective function used for optimizing sensor positions has been modified based on a damage scenario-driven strategy.

A numerical study was carried out on Hall C of Turin Exhibition, designed and constructed by Nervi in 1950. The preliminary analysis for this building confirmed that, in most cases, large concrete buildings conceived in the post-war period are inherently vulnerable with respect to seismic actions. This is especially true in the presence of heavy structural elements conceived to withstand important gravity loads, like in Hall C. As a result, after the recent reclassification of the Italian territory, the seismic actions cannot be neglected. Turin Exhibition complex is located in a low seismic area but, in view of its reuse, it cannot be considered safe with respect to earthquakes. In fact, the vibration modes of Hall C highlight a number of vulnerability factors to seismic actions. Critical mechanisms appear to affect the longer arches in the out-of-plane direction, since the structural designers in that period did not usually take these aspects into account. Other relevant seismic mechanisms might be associated to the horizontal sliding of the upper part of the roof. It is worth noticing that some of the evidenced mechanisms cannot be prevented by defence lines designed for wind actions, as is frequent in standard buildings. On these premises, seismic monitoring systems is highly recommended for the seismic preservation and conservation of these historical spatial architectures.

To conclude, numerical investigations have confirmed that non-structural elements strongly affect the sensor placement of vibration-based monitoring systems. In this respect, this study demonstrated that the damage-scenario driven OPS strategy is an efficient design tool.

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