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Seismic retrofit assessment of a school building through operational modal analysis and f.e. modelling

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Abstract – The paper deals with the dynamic characterisation of a RC frame school building in central Italy before and after the seismic retrofitting, obtained by coupling the building with an innovative patented seismic dissipative protection system. Before the retrofit, ambient vibration tests were performed to evaluate frequencies and mode shapes for developing f.e. models describing the school dynamic behaviour in operational conditions. Several finite element models with increasing level of detail are presented, from the bare frame model, based on the assumptions and simplifications usually adopted for design purposes, to an upgraded model taking account of secondary and non-structural elements (e.g. internal and external walls, screeds, roofing, floor tiles and plasters) as well as the interaction between structure and retaining walls. The latter was used to develop the design model of the seismic retrofitting system, which aims to assure the immediate occupancy of the building in the case of severe earthquakes limiting damage to non-structural components. Tests were repeated after the retrofit to check consistency with numerical design predictions. Comparisons between experimental and numerical modal parameters are shown discussing the usefulness of ambient vibration tests.

Keywords: Building dynamic identification, ambient vibration test, operational modal analysis, RC frame building retrofitting, steel dissipative towers, finite element model upgrading

1. Introduction

Dynamic identification is an increasingly used technique in civil engineering, particularly for existing buildings. Generally, it is used: i) to calibrate structural models to be used for the design of

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27 repair, rehabilitation and retrofit works; *ii*) to assess and validate structural design models for final
28 testing; and *iii*) to monitor the structural health of buildings starting from the evaluation of changes in
29 their dynamic behaviour over time. Various testing techniques differing in terms of equipment, time
30 required, costs, and dynamic input can be adopted. However, Ambient Vibration Test (AVT) is one of
31 the most attractive method for the evaluation of the dynamic characteristics of buildings due to its
32 intrinsic advantages, such as the exploitation of ambient excitations as input instead of forced
33 vibrations, the use of portable and light instrumentation and the possibility to carry out tests without
34 disrupting buildings functionality. Due to the low amplitude range of vibrations ($\approx 10^{-5}$ g) produced by
35 the ambient excitation, only the dynamic behaviour of the building at very small strains can be
36 captured through AVTs.

37 Many ambient vibration tests have been executed in the last two decades for the dynamic
38 identification of civil structures such as buildings, bridges, towers, (e.g. [1-10]) and many studies have
39 been developed to assess factors affecting modal parameters (e.g. [11-12]). However, few examples
40 can be found in literature regarding dynamic tests performed on civil structures before and after
41 retrofitting works with the aim to validate a predictive f.e. structural model and to assess the dynamic
42 behaviour variation due to the interventions (e.g. [13-19]). Indeed, data from AVTs (i.e. from tests that
43 are able to capture the building dynamics only at small strains) are affected by contributions of non-
44 structural components and can be profitably used to calibrate the linear behaviour of f.e. models in
45 which nonlinearities can be later implemented to perform the seismic assessment of the retrofitted
46 structures. Numerical models should be at least modified to account for the material nonlinearity and
47 the contributions of non-structural components that during earthquake usually undergo damage that
48 reduce interaction phenomena with the structural members.

49 Finally, although finite element model updating based on experimentally obtained modal
50 parameters is a largely studied and well-known issue in civil engineering, only a few systematic
51 researches concerning the effects of non-structural elements on the overall response of buildings are
52 available in literature (e.g. [20-25]).

53 In this paper ambient vibration tests are exploited to assess a seismic retrofit of a strategic
54 building. In details, the identification of the modal features of an existing low-rise RC frame school
55 building before and after the seismic retrofit is presented discussing the f.e. model upgrading
56 descending from the tests results. The upgraded model includes contributions of the in-plane
57 deformability of floors, internal partitions, external infills and surrounding retaining walls. The
58 specific contribution of the latter features on the overall dynamic structural behaviour is also shown
59 and discussed. The refined model is adopted to design the retrofit system that is achieved with an
60 innovative patented dissipative protection system called “Dissipative Towers” [26-28] that foresees the
61 coupling of the existing building with new external rocking steel truss towers, pinned and equipped
62 with viscous dampers at the base. The refined model, which accounts for both structural and non-
63 structural components, was crucial for a proper design of the retrofitting system, which requires a
64 reliable prediction of the building displacements subjected to severe earthquakes in order to limit
65 damage to non-structural members. After the retrofit, the experimental dynamic response of the
66 building is compared with the numerical predictions in terms of modal parameters (natural frequencies
67 and mode shapes). Dynamic tests revealed important to (i) upgrade the design finite element model
68 and (ii) to check that changes in the modal parameters due to the retrofit agreed with those predicted
69 through the design model.

70 **2 School building description**

71 The school building is located in Camerino, in a high seismicity area in central Italy, as
72 demonstrated by the recent Central Italy earthquake that struck the municipality in 2016. Figure 1
73 illustrates the plan view and sections of the RC building, which is composed of three blocks (Block A,
74 Block B, and Block C) separated by expansion joints. Block A and Block C, constituted by a 4-storey
75 3×2 bay frame with an almost square plan (26.0×19.5 m), form the front of the building. Block B, at
76 the rear of the building, has 3-storeys and a rectangular plan (12.85×28.20 m), and is constituted by
77 frames with 2 bays of 4.0 m and 8.25 m in transverse direction and 7 bays of 3.6 m in longitudinal

direction (Figure 2(a)); this part of the building was erected on ancient masonry walls constituting the lower part of a pre-existing structure.

The columns have 40×40 cm square cross sections rotated by 45° with respect to the frame plane. The beams on the building perimeter are linearly tapered with cross sections of about 30×80 cm and 30×40 cm at the beam-to-column joint and at mid-span, respectively, whereas the inner beams have constant rectangular cross sections with varying dimensions. The mean compressive strength of concrete is $f_{cm} = 19.71$ N/mm² and is obtained from an experimental investigation on 22 core samples extracted from structural elements. The joint between the blocks (red dashed lines in Figure 1(a)) regards only beams and columns (columns straddling the joint have a triangular cross section) while non-structural components (e.g. screeds, floors and infill walls) are continuous through the joints. RC floors are made of prefabricated beams and 20 cm high clay blocks, on which a 4 cm thick slab is cast. Infills of the external frames consist of 30 cm thick brick cavity walls with intermediate insulation having height of about 1.20 m from the floor (below windows), as shown in Figure 2(a). Internal partitions are mostly made with 8 cm thick hollow clay blocks, with plaster on both sides; occasionally, very light infill plasterboard partitions are also present.

2.1 The retrofit system and the need of dynamic tests for the design

Intrinsic geometry of beams and columns, as well as of infills, makes the building vulnerable to seismic actions, despite its social and strategic value. The seismic retrofit of the building was thus achieved through a patented dissipative protection system called “Dissipative Towers” [26-28]. In details, two external rocking steel truss towers pinned and equipped with viscous dampers at the base have been positioned in plan according to Figure 1(a). Towers interact with the building at the floor levels, except at the first one ((Figure 1(b) and Figure 2(a)), through steel members which are connected to steel plates anchored to the external frames. Steel members are erected on RC thick base plates that are centrally pinned through a spherical support to the foundations. Viscous dampers are arranged vertically between the base and foundation plates (one device per vertex for tower A and two

for tower B), so that the rocking of the tower base, due to the building horizontal displacements, can activate all the devices. Dampers are included into an articulated quadrangle (Figure 2(b)) that amplifies the device displacements thanks to a leverage system. Furthermore, thick steel plates duly anchored to adjacent columns and beams are used to structurally connect adjacent blocks of the building.

From a conceptual point of view, the tower stiffness promotes a linear displacement profile and a constant inter-story drift, preventing soft-story collapses, while viscous dampers largely enhance the building dissipative capacity. Since the energy dissipation through dampers is very high, linear dynamic response spectrum analyses are not allowed to assess the seismic performance of the retrofitted system [29] and dynamic nonlinear analyses, involving the use of acceleration time histories, are required. However, nonlinearities are limited to dissipative devices since “Dissipative Towers” are dimensioned to assure a linear elastic behaviour of structural members. For the investigated case study, the retrofit system was designed not only to assure a linear behaviour of the building, but also to limit damage of non-structural elements in case of severe earthquakes (e.g. for actions normally corresponding to the life safety limit state) [29]. For this purpose, the design of the retrofitting system must be carried out suitably considering the overall initial stiffness of the building, which is largely affected by contributions of both structural and non-structural components. Indeed, the design numerical model should be able to accurately predict the building displacements and inter-storey drifts to which structural and non-structural damage are related.

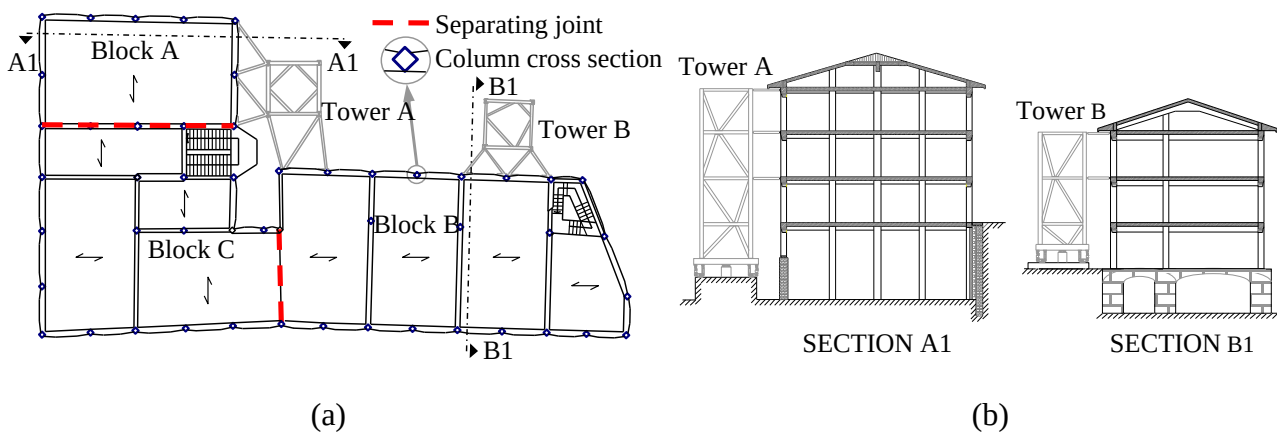


Figure 1. (a) Plan view and (b) sections of the building.



(a)



(b)

Figure 2. (a) Front and rear view of the school building; (b) viscous dampers at the base of the towers.

Thus, an estimation of the dynamic properties of the building in its operational condition (before the retrofit) is crucial for the development of a reliable numerical model that can be used for the retrofit design and, at the same time, a validation of the design model (i.e. including the retrofit system) is important, considering that the retrofit system must guarantee the building usability (with minor and fast repair interventions) after a severe earthquake.

3 Measurements and operational modal analysis

Ambient Vibration Tests (AVTs) are used to perform a dynamic characterization of the building before and after the retrofit. Modal information from tests executed before the retrofit were used to develop a numerical design model able to account for the building behaviour in its real service conditions while modal information from tests executed after the retrofit were used to validate the design model and its compliance with the real retrofitted structure.

3.1 Instrumentation and measurements

To measure the building vibrations due to ambient excitation, low noise piezoelectric

139 accelerometers with a sensitivity of 10 V/g, a frequency range ($\pm 10\%$) of $0.07 \div 300$ Hz, and a
140 broadband resolution of 1 μg root mean square were used. Sensors were connected to a 24-bit data
141 acquisition system with an input range of ± 5 V by means of low-noise coaxial cables. The maximum
142 measurable accelerations were ± 0.5 g. A laptop with dedicated software was adopted to store and
143 process the signals (Figure 3).

144 Three accelerometers per floor were used (Figure 4(a)) that are enough to draw the global mode
145 shape of the building capturing the coupled roto-translational motions of floors assumed to be rigid in
146 their plane. For each configuration, 1000-second long records, sampled at a rate of 2048 Hz (the lower
147 limit allowed by the adopted acquisition system), were acquired. This time length largely exceeds the
148 limit of about 1000-2000 times the fundamental period of the building, which is the acquisition length
149 recommended to obtain an accurate estimate of the modal parameters with ambient vibration
150 measurements [30]. In Figure 4(b) the time histories registered by accelerometers 2AX, 2AY and 2BX
151 during one of the tests carried out before and after the retrofitting, are reported. The Root Mean Square
152 (RMS) values of the measured accelerations are reported to show that the excitation levels during the
153 measurements carried out before and after the retrofitting were comparable. RMS of the measured
154 accelerations were calculated considering signal bands filtered in the frequency range that mainly
155 characterise the dynamic behaviour of the building, i.e. 3-5 Hz.

156 For the measurements carried out before the retrofitting works, only 3 accelerometers were
157 available, two of which, considered as references, were located at point A on the second floor (2AX
158 and 2AY), while the third sensor (roving sensor) was moved around to all the other positions. For the
159 analysis carried out after the retrofitting works, 12 accelerometers were available and only one
160 configuration was necessary. However, three different acquisitions were carried out keeping the same
161 sensor configuration, to better estimate the variability of the parameters identified thanks to data
162 redundancy. Further details can be found by the reader in [31].

163 During tests, the air temperature and relative humidity were monitored outside the building; a
164 temperature range of 19-22 °C and a relative humidity of about 60 % was observed during

165 measurements carried out before the retrofitting works in August 2012 while a temperature range of
166 17-20 °C and a relative humidity of about 74 % was registered during tests performed after the retrofit
167 in May 2013. Moreover, the wind velocity was quite low during both tests.

168
169 Figure 3. Measurement equipment during tests.

170
171 Figure 4. (a) Layout of accelerometers; (b) filtered acceleration time histories with RMS values, before and after
172 retrofitting.

173 Environmental parameters, especially wind velocity and environmental temperature, are known
174 to affect the modal parameters of structures [32-35]; however, considering similarities of excitation
175 levels and environmental conditions relevant to the two tests, it can be concluded that changes in the
176 modal parameters of the building can be almost completely attributed to effects of the retrofiting
177 works.

178 *3.2 Signal processing and operational modal analysis*

179 Standard signal processing techniques were applied to all the recorded data before carrying out
180 modal analyses. First, a correction of the spurious trends of signals was performed by subtracting the
181 contribution resulting from the signals fitting with a third-degree polynomial. Then, the records were
182 filtered with a low-pass filter with a cut-off frequency of 20 Hz to avoid aliasing phenomena. Finally,
183 the signals were down-sampled at 51.2 Hz to reduce the number of data and make subsequent analyses
184 faster. The modal parameters of the building (natural frequencies, damping ratios mode shapes) were
185 identified using two output-only techniques implemented in Matlab environment [36]: the Enhanced
186 Frequency Domain Decomposition (EFDD) method [37,38] and the Covariance-driven Stochastic
187 Subspace Identification (SSI-COV) method [39-42]. Considering that very close results have been
188 obtained with the two methods, only results from the SSI-COV method are herein presented. In this
189 work, a model order of 50 is used and, in the stabilisation diagrams (Figure 5), a mode is assumed
190 consistent with reference to frequency, damping ratio and mode shape when, by increasing the model
191 order, it shows a natural frequency variation $< 1\%$ (green cross), a damping ratio variation $< 2\%$ (red
192 circle), and a Modal Assurance Criterion (defined in the following section) $MAC > 0.98$ (cyan star),
193 respectively.

194 *Comparison between mode shapes*

195 To compare mode shapes and obtained from measurements before and after the building
196 retrofiting, the Modal Assurance Criterion (MAC) [43], was used.

197

Figure 5. Stabilisation diagrams (SSI-COV) for measurements before and after retrofitting works.

This criterion is defined as

$$(1)$$

and provides a numeric value that assesses the correspondence between two mode shapes. MAC value is 1 for perfectly matching mode shapes (parallel vectors) and 0 for completely different mode shapes (orthogonal vectors). In the following, MAC values are presented in matrix form exploiting a greyscale ranging from white (MAC = 0) to black (MAC = 1).

4 Modal features from ambient vibration tests

4.1 Tests before retrofitting

The left half side of Table 1 presents the resonance frequencies f and damping ratios ξ of the first seven modes identified before retrofitting. The values of damping ratios ξ relevant to the first seven modes vary between 0.9 and 2.6 %, showing a quite high variability, which is rather usual for buildings and civil engineering structures when identified through ambient vibration testing. As for mode shapes, a 3D isometric view is reported in Figure 6(a) (the last floor of block A is not included because it was not monitored). Furthermore, the basement results to be practically fixed since its modal displacements are negligible compared to those of the upper floors. The first three modes are those typical for low-rise RC frame buildings and can be assimilated to a first transverse mode with

216 torsional component, a first torsional mode, and a first longitudinal mode with torsional component. A
 217 significant torsional component is always present due to the L-shaped plan of the building. The
 218 subsequent two mode shapes cannot be clearly assimilated to standard mode shapes while the sixth and
 219 the seventh are similar to the second torsional and transverse modes, respectively.

220 4.2 Tests after retrofitting

221 The right half side of Table 1 reports values of the natural frequencies f and the damping ratios ξ
 222 of the first seven modes identified while the identified mode shapes are shown in Figure 6(b). The
 223 latter appear like those obtained from tests executed before retrofitting and small differences cannot be
 224 clearly recognised through the graphical representation. The first three modes are again those typical of
 225 low-rise RC frame buildings: a first transverse mode, a first torsional mode, and a first longitudinal
 226 mode, all with a significant torsional component. The fifth and the sixth modes are assimilable to
 227 second modes, longitudinal and transverse, respectively.

228 4.3 Comparison between the results of the tests performed before and after retrofitting

229 Despite above mentioned similarities, significant changes in the building dynamic behaviour are
 230 found from the comparison of results obtained from tests performed before and after the seismic
 231 retrofit, denoted with subscripts b and a , respectively. It is worth noting that the modal parameters of
 232 real buildings identified by means of operational modal analysis at different times are generally
 233 affected by a certain variability.

234 Table 1. Modal parameters before and after retrofitting.

mode	Before		After		Mode shape
	$f_{ex,b}$ [Hz]	$\xi_{ex,b}$ [%]	$f_{ex,a}$ [Hz]	$\xi_{ex,a}$ [%]	
1 st	3.61	1.46	3.60	1.49	first transverse
2 nd	3.70	1.69	3.82	1.96	first torsional
3 rd	4.00	1.14	4.14	1.49	first longitudinal
4 th	4.41	0.92	5.00	1.17	/
5 th	7.25	1.16	7.69	1.15	/
6 th	8.69	2.58	9.54	2.18	/
7 th	9.89	2.41	10.50	3.30	/

Figure 6. Resonance frequencies and mode shapes (a) before and (b) after retrofitting.

This is due not only to signal acquisition and processing but also to random changes in a number of factors such as amount and distribution of masses inside the building as well as environmental conditions (e.g. wind, temperature, humidity). However, these uncertainties generally induce much smaller variations in modal parameter values than those identified for the case under discussion.

Interesting considerations can arise when observing the changes in the values of resonance frequencies and damping ratios, as well as when comparing mode shapes [44]. To facilitate the reader, the values of the first seven resonance frequencies identified before and after the building retrofit are listed in Table 2. Except for the first mode, a general increase, ranging between 0.12 and 0.85 Hz, with an average value of about 6 % can be observed after the retrofit. This can be interpreted as a consequence of a general increase in the stiffness of the building coupled with steel towers. In particular, it is worth observing that the first resonance frequency is practically unchanged and that the second and the third frequencies undergo a small increment.

Table 2. Experimental resonance frequencies before ($f_{ex,b}$) and after ($f_{ex,a}$) the building retrofit.

mode	$f_{ex,b}$ [Hz]	$f_{ex,a}$ [Hz]	$(f_{ex,b}-f_{ex,a}) / f_{ex,b}$ [%]
1 st	3.61	3.60	-0.2
2 nd	3.70	3.82	3.0
3 rd	4.00	4.14	3.6
4 th	4.41	5.00	13.4
5 th	7.25	7.69	6.1
6 th	8.69	9.54	9.8
7 th	9.89	10.45	5.7

251 Conversely, the higher frequencies present greater increments, particularly the fifth, the sixth and
252 the seventh increase by about 6.1 %, 9.8 % and 5.7 %, respectively. This behaviour is consistent with
253 the adopted retrofitting system that foresees stiff steel towers free to rotate at their base and only
254 connected to the building in correspondence of the floors. As a consequence, towers do not add much
255 stiffness with respect to modes that involve an almost-linear deflection of the building, i.e. the first
256 three modes. Conversely, they strongly increase the stiffness with respect to higher modes that imply a
257 non-linear deflection of the building vertical profile, i.e. with non-uniform values of inter-storey drift.
258 Furthermore, the increase in the resonance frequency values is also partially due to the stitching of
259 joints separating the building blocks (before the retrofit the interactions among blocks are only due to
260 non-structural components). This effect is more pronounced for modes involving relative movements
261 between blocks (e.g. the fourth), as will be shown later through a refined f.e. model of the structure.

262 Regarding damping ratios, typical values ranging between 1.5-3.0% are obtained, as usual for
263 AVTs on RC low-rise buildings. It should be remarked that the dissipative contribution of the towers
264 cannot be captured with AVTs, because dissipative devices are not activated by low amplitude
265 vibrations. Finally, mode shapes identified before and after the building retrofit remain quite similar
266 and the differences cannot be clearly identified graphically; thus, differences are captured analytically
267 by means of MAC in Figure 7a. Very little differences can be observed for the first four modes while a
268 variation in the higher modes (especially the fifth and the sixth) appears evident; this confirms
269 considerations already made about changes of the natural frequencies, which are more evident for
270 higher modes. Indeed, the rigid steel towers connected at the base through a spherical hinge contribute
271 to linearize the profiles of horizontal displacements of the building. Thus, the tower-building
272 interactions are more evident for higher modes, which are characterised by nonlinear profiles of
273 horizontal displacements whereas effects on lower modes are less pronounced, since displacement
274 profiles are closer to linearity. For lower modes, the contribution of the towers to the regularisation of
275 mode shapes can be better appreciated by means of the MAC between the experimental mode shapes
276 and a perfectly linear deflection.

277

278 Figure 7. (a) MAC between mode shapes before and after retrofitting; (b) profile of modal displacements at AX
279 and AY before and after retrofitting, and MAC values with an ideal linear deflection of the building.

280

281 Figure 7b shows components of modal displacements of the first three modes in the two principal
282 directions of the building in correspondence of alignments AX and AY for both the pre-retrofitted
283 (blue lines) and retrofitted (red lines) conditions. In order to compare the profiles of modal
284 displacements with respect to an ideal linear deflection of the building, MAC values between the
285 experimental modal displacements and a linear trend are computed and reported in the figure with the
286 relevant colour. A general increase in the MAC value, corresponding to a regularisation of the modes
287 (i.e. mode shapes closer to a linear shape), is obtained, especially for the second and the third modes.

288 5 3D finite element models

289 5.1. Building before retrofitting

290 A predictive refined finite element model for the seismic retrofit design of the existing building
291 was developed in SAP2000 code [45]. The numerical model is based on available structural drawings
292 of the building and in-situ measurements as well as destructive and non-destructive tests on the
293 structural materials, i.e. extraction of concrete core samples to estimate the strength and elastic

294 modulus of concrete, sonic tests to control the homogeneity of concrete elements and surveys using an
295 electromagnetic cover meter to investigate position, depth and size of steel reinforcement.

296 Beams and columns are modelled with 2-nodes frame elements while slabs and walls are
297 schematised with 4-nodes shell elements having six degrees of freedom (dof) per node. Prismatic
298 frame elements are used for columns and internal beam sections, whereas non-prismatic frame
299 elements are chosen for tapered external beams. About the latter, the major moment of inertia (bending
300 in vertical plane) of the cross section is assumed to vary along the beam axis with a parabolic law,
301 whereas the minor moment of inertia (bending in horizontal plane) varies linearly. The shell elements
302 are discretised into almost rectangular elements with an area of about 0.1 m^2 . This value was obtained
303 according to preliminary convergence analysis, by gradually reducing the shell size up to a non-
304 significant variation in the values of the natural frequencies. To consider the stiffening effect due to the
305 intersection of the members at the beam-to-column joints, rigid-end offsets equal to the 70% of the
306 nominal overlapping length are assumed for frame elements connecting to the nodes. The columns of
307 Block A and Block C are fully restrained at the building basement. Being the building founded on
308 cemented sandstone, this assumption is assumed to be quite representative of the structural behaviour
309 of the building at operational condition. The columns of Block B are, instead, rigidly connected to the
310 shells of the masonry walls constituting the foundations.

311 The masses of the structural elements (i.e. RC beams and columns) are automatically computed
312 by the software according to assigned frame element cross sections and material properties. The
313 masses of the external walls are uniformly distributed along the perimeter beams, whereas the masses
314 of the floors, composed of structural slabs and non-structural elements (screeds, roofing, floor tiles and
315 plasters), as well as those of the internal partition walls and furniture (considered as equivalent
316 distributed loads) are considered lumped at beams that are orthogonal to the slabs orientation. Live
317 loads are not considered initially, in order to simulate the real condition of the building during the test
318 and to allow the model validation through comparisons of numerical and experimental results; in

319 Table 3 the values of self-weight of both structural and non-structural elements are listed for
320 completeness.

321 Materials are assumed to behave elastically, with properties reported in Table 4; the static
322 Young's modulus of concrete is derived from the mean value of the concrete strength ($f_{cm} = 19.71$
323 N/mm²) as suggested by the Italian Standards [29]. The reduced modulus of elasticity, $E_{c,red}$, is
324 assumed to be 65% of the static modulus, while the dynamic modulus of elasticity is obtained by
325 increasing the static modulus by about 20% [46], to capture the dynamic behaviour at very low
326 amplitude vibrations. The values of the static elastic modulus and mass of the retaining walls, as well
327 as those of internal and external walls, are chosen according to the Italian Standards [47], depending
328 on the masonry typology. Due to the lack of information available in the literature, the dynamic
329 modulus of elasticity is obtained by increasing the static modulus by 20%, analogously to the concrete.

330 The following six different models with increasing degree of accuracy are developed.

331 - **Mod. 0a** and **Mod. 0b** (Figure 8(a)) are the bare frame models usually developed for design
332 purposes. The internal partitions and external walls are not modelled, a rigid diaphragm is considered
333 for each floor (i.e. infinite in-plane stiffness and null out-of-plane stiffness of the slabs), stairs and
334 foundation walls at the base of Block B are modelled with shells. For the verifications in terms of
335 Ultimate Limit States (ULS), the reduced value of the static modulus of the elasticity of concrete,
336 $E_{c,red}$, is considered to account for concrete cracking (Mod 0a), whereas, for the verifications in terms
337 of Service Limit States, the full static value is usually adopted to consider uncracked concrete
338 conditions. In this case, to detect the behaviour at very low strain levels during operational conditions
339 (ambient vibrations), the dynamic elastic modulus $E_{c,dyn}$ is adopted (Mod 0b).

340 - **Mod. 1** takes account of the contribution of the external retaining wall by modelling the
341 basements of Blocks A and C with shell elements, having dynamic modulus of elasticity $E_{rw,dyn}$, and by
342 considering the stiffness contribution given by the soil surrounding the wall. The latter is included by
343 means of supports restraining horizontal displacements and located at the first level in the front part of
344 Blocks A and C.

345

Table 3. Loads of structural and non-structural members.

	Typical floor	Last floor	Roof	Stairs of block C	Stairs of block B
Structural [kN/m ²]	3.20	3.20	2.80	3.75	3.00
Non-structural [kN/m ²]	1.80	0.60	0.60	1.20	1.20
Non-structural [kN/m]	4.60				

346

347

Table 4. Elastic modulus [N/mm²] and weight [kN/m³] of concrete members and walls.

Concrete		Retaining wall		External wall		Internal wall	
$E_{c,st}$	25497	$E_{rw,st}$	3360	$E_{w,st}$	4583	$E_{w,st}$	3208
$E_{c,dyn}$	30720	$E_{rw,dyn}$	4032	$E_{w,dyn}$	5500	$E_{w,dyn}$	3850
$E_{c,red}$	16573	$E_{rw,red}$	/	$E_{w,red}$	/	$E_{w,red}$	/
w	25	w	22	w	20	w	12

348

349

- **Mod. 2** considers the in-plane deformability and bending stiffness of the floors by removing

350

diaphragms and modelling slabs with anisotropic shells, which have inertia and stiffness characteristics

351

equivalent to those of the reinforced concrete slab with a T-cross section coupled with screed and tiles,

352

in the longitudinal direction, and to those of the slab flange coupled with screed and tiles, in the

353

transverse direction. Furthermore, the self-weight plus other loads imposed on the floor are considered

354

as uniform loads applied to the shells.

355

- **Mod. 3** takes into account the contributions of the internal partitions and external walls, which

356

are modelled with shell elements having elastic modulus $E_{w,dyn}$ and thickness (masonry and plaster) of

357

0.12 m and 0.2 m, for internal partitions and external walls, respectively.

358

- **Mod. 4** (Figure 8(b)) accounts for the contribution of the soil surrounding the building; the

359

supports located at the floor level are replaced with springs that are orthogonal to shells representative

360

of the retaining walls. The spring stiffness is 8144 kN/m, corresponding to a subgrade reaction value of

361

80000 kN/m³, according to Bowles [48], who suggests the range 80000-96000 kN/m³ for a medium-

362

dense sand. This final model is very accurate and rather complex, involving 2403 frame elements,

363

69291 shells, and 70743 nodes.

364 Numerical natural frequencies and mode shapes are obtained through an eigenvalue analysis and
 365 are compared with the experimental ones. Values of the natural frequencies of the first four modes of
 366 each model are graphically shown in Figure 9 proving a direct comparison with the corresponding
 367 experimental modes. For each model, the type of mode shape is also indicated, when it is clearly
 368 similar to a standard mode for buildings, i.e. transverse, longitudinal, and torsional modes, with
 369 reference to the directions x and y and to the rotation ϕ (around z), respectively.

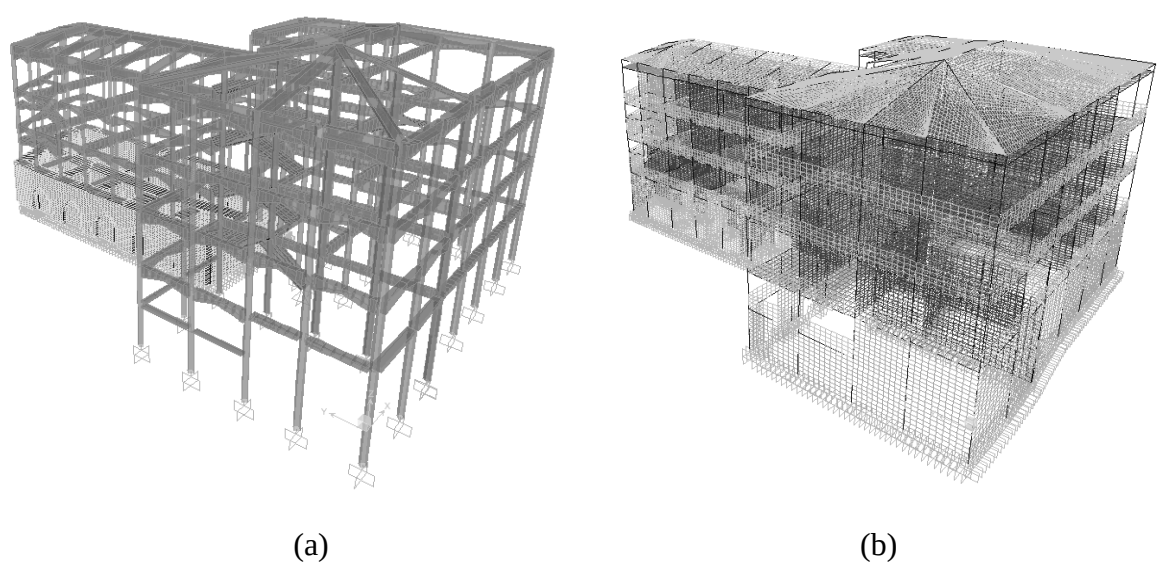


Figure 8. 3D f. e. model of the building pre-retrofitting: (a) extruded view of the global model Mod.0a; (b) global model Mod. 4 (links not in view).

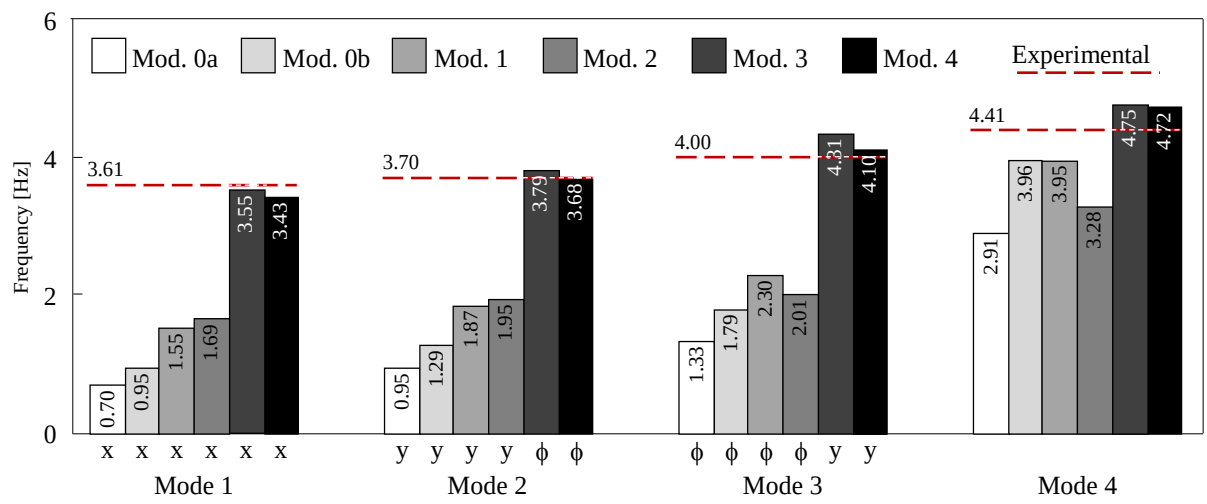


Figure 9. Natural frequencies obtained with f.e. models.

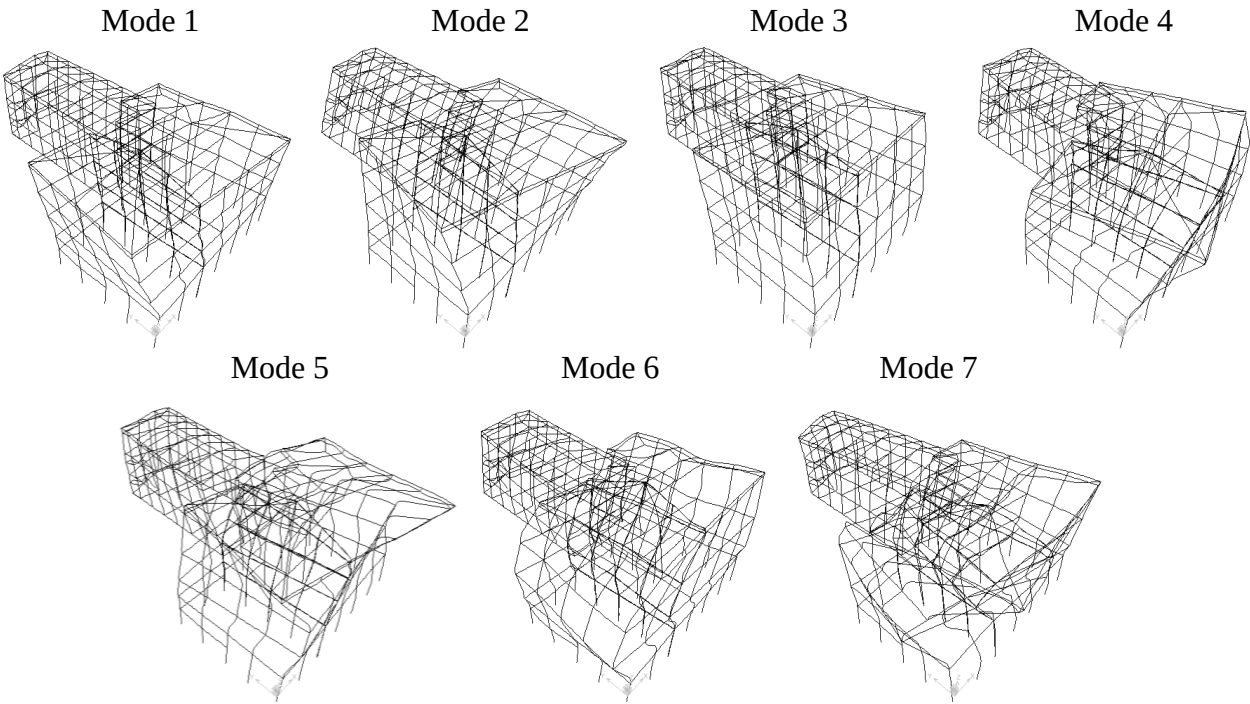
377 The bare frame model (Mod. 0a), usually adopted for structural design, shows values of natural
378 frequencies much lower than the experimental ones (e.g. the first frequency is about one fifth). A
379 significant increase in these theoretical values of about 30 % is obtained with Mod. 0b, thanks to a
380 much higher value of the modulus of elasticity assumed for the concrete, $E_{c,dyn}$ instead of $E_{c,red}$
381 (increment of about 85%). It is worth noting that also the sequence of the modes disagrees with the
382 experimental one: the first theoretical mode is mainly a transverse mode, the second is a longitudinal
383 mode, and the third a torsional mode, whereas the first three experimental modes were mainly
384 transverse, torsional and longitudinal modes, respectively. A further significant increase in the values
385 of the first three natural frequencies (of about 63 %, 45 % and 28 %, respectively) is obtained with
386 Mod. 1, thanks to the stiffening contribution given by the retaining walls and supports positioned at the
387 ground level in the front part of the building.

388 With Mod. 2 a slight increase in the values of the first two natural frequencies is obtained, due to
389 the bending stiffness of the shells introduced to model the floor. Vice versa, there is a decrease of
390 about 13-17 % in the values of the third and the fourth natural frequencies that, being related to modes
391 involving in-plane deformations of floors, are much more affected by the removal of the diaphragms
392 that constrain the floor nodes avoiding in-plane deformations. It is worth noting that for all the models
393 discussed so far, the longitudinal mode anticipates the torsional one in disagreement with the
394 experimental results.

395 With Mod. 3 important modifications to the modal properties of the building are observed: the
396 sequence of the modes now agrees with the experimental one, the values of the first three natural
397 frequencies become almost twice those of Mod. 2 and quite close, even if slightly higher, to the
398 experimental ones. These evidences confirm that the contribution of internal partitions and perimeter
399 walls is of primary importance in the dynamic behaviour of the building for low-level vibrations.

400 Finally, Mod. 4, in which the supports located at the ground level are removed and the
401 contribution of the soil surrounding the retaining wall is modelled with elastic springs, shows values of
402 the natural frequencies that are slightly lower than those of Mod. 3 and close to the experimental ones.

403 Figure 10 shows the first seven mode shapes obtained with Mod. 4, whereas Table 5 illustrates
 404 the comparison between numerical (Mod. 4) and experimental modal parameters, i.e. the percentage
 405 difference between the values of natural frequencies and MAC values between mode shapes. A good
 406 agreement between numerical and experimental values of the natural frequencies is obtained, with a
 407 difference of about -5 % and -0.54 % for the values relevant to the first and the second modes and
 408 ranging between 2.50 % and 10.41 % for the higher modes.



409
 410 Figure 10. Mode shapes of Mod. 4 (shells and links not in view).

411
 412 Table 5. Comparison between experimental and numerical (Mod. 4) modal parameters of the building before
 413 retrofitting.

	Experimental $f_{ex,b}$ [Hz]	Numerical $f_{n,b}$ [Hz]	$(f_n - f_{ex})_b / f_{ex,b}$ [%]	MAC
1 st mode	3.61	3.43	-4.99	0.93
2 nd mode	3.70	3.68	-0.54	0.99
3 rd mode	4.00	4.10	2.50	0.97
4 th mode	4.41	4.72	7.03	0.98
5 th mode	7.25	7.55	4.14	0.75
6 th mode	8.69	9.07	4.37	0.72
7 th mode	9.89	10.92	10.41	0.94

414 The MAC values show a very good correspondence with values ranging between 0.93 and 0.99
415 for the first four mode shapes and the seventh one, while higher modes present slightly lower values. It
416 is worth mentioning that in practical applications the dynamic response of low-rise RC buildings is
417 usually accurately estimated considering the contribution of the first modes.

418 5.2. Building after retrofitting

419 Starting from Mod. 4, a numerical finite element model that includes external towers and plates
420 used to connect adjacent structural members close to the separating joints (Figure 11(a)) is developed
421 and used for the retrofit design Hereafter, this model is referred to as Mod. 5 and is used for the
422 estimation of the dynamic parameters of the retrofitted structure, which will be compared with results
423 of experimental tests executed after the retrofit.

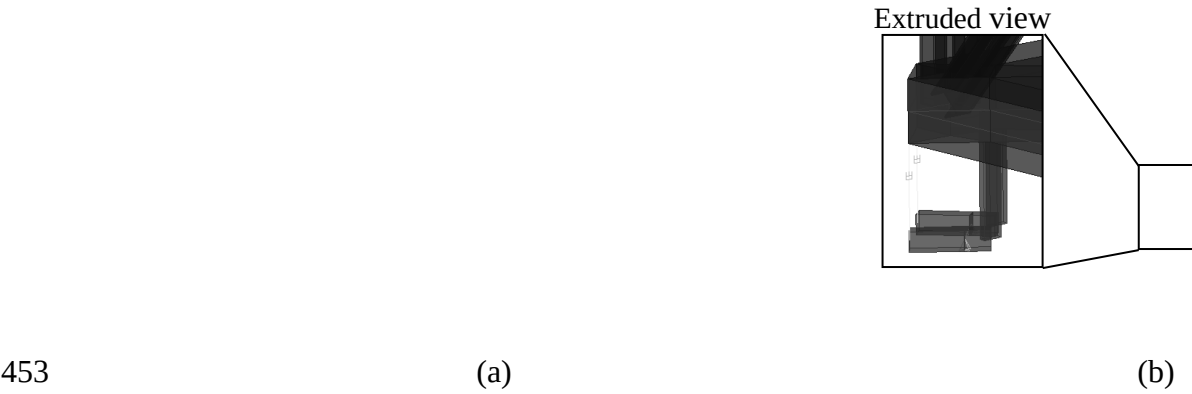
424 The steel towers are modelled with frame elements, while 0.5 m thick shell elements are used for
425 the base plate. A hinge at the centroid of the intrados of the base plate restrains possible tower
426 translations, while rotations are free. Articulated quadrangles that include dissipative devices, each
427 modelled with two elastic frames (one horizontal and one vertical) and one link element, simulating
428 the damper, are located at the vertexes of the base plate. The horizontal elastic frame is pinned at 1/3 of
429 its length, while the vertical frame that includes the link is pinned both at the base plate and the
430 horizontal frame (Figure 11(b)). Considering that the viscous dampers are not activated by very low
431 amplitude vibrations as those registered during ambient vibration tests, the dissipative contribution of
432 the devices is not accounted for. On the other hand, their elastic stiffening contribution is modelled by
433 means of link elements. Assuming that the fluid inside the device does not flow between the two
434 chambers of the device, the elastic stiffness K can be estimated, considering the fluid compressibility,
435 as

$$436 \quad (2)$$

437 where A (70.04 cm²) and h (5.5 cm) are the cross section and the height of the fluid chamber,
438 respectively, and B (105500 N/cm²) is the bulk modulus of the viscous fluid. The towers are connected

439 at each floor level, except for the first, by means of elastic elements hinged at the ends. Moreover, to
440 model the intervention aimed at the structural connection of the three building blocks, frame elements
441 are introduced to simulate the stiffness characteristics of the steel plates positioned on adjacent
442 columns and beams, straddling the joints between two blocks. With regard to the materials, the values
443 of the elastic moduli are assumed accordingly to the design parameters: $E_a = 200000$ MPa for the steel
444 elements of the towers and $E_{c,d} = 38770$ MPa for the concrete of the base plates.

445 Table 6 presents a comparison between the modal parameters obtained from the numerical model
446 and the experimental tests executed after the building retrofit in terms of percentage difference for
447 frequencies and MAC values for the identified mode shapes (Figure 12). A good agreement between
448 the numerical and experimental values of the natural frequencies is obtained, with a difference of less
449 than 5% for values relevant to the first three modes as well as the fifth and the sixth. More scattered
450 values are obtained for the fourth and the seventh modes. The MAC values show a very good
451 correspondence for the first four mode shapes, with values ranging between 0.97 and 0.99, and a good
452 agreement for the higher mode shapes, with values ranging between 0.80 and 0.86.



453 (a) (b)

454 Figure 11. 3D f. e. model of the building post-retrofitting: (a) global model Mod. 5 (links not in view); (b) detail
455 of the dissipative system.

456 Table 6. Comparison between the experimental and numerical (Mod. 5) modal parameters of the building after
 457 retrofitting.

mode	Experimental $f_{ex,a}$ [Hz]	Numerical $f_{n,a}$ [Hz]	$(f_{n,a} / f_{ex,a}) / f_{ex,a}$ [%]	MAC
1 st	3.60	3.48	-3.33	0.99
2 nd	3.82	3.85	0.79	0.99
3 rd	4.14	4.23	2.17	0.98
4 th	5.00	5.57	11.40	0.97
5 th	7.69	8.06	4.81	0.86
6 th	9.54	9.41	-1.36	0.80
7 th	10.45	11.87	13.59	0.86

458

459

460 Figure 12. Mode shapes of Mod. 5 (shells and links not in view).

461 5.3 Comparison between results of FE models before and after retrofitting

462 When comparing the numerical results obtained with Mod. 4 and Mod. 5 (building before and
 463 after retrofitting, respectively) in terms of natural frequencies (Table 7) and mode shapes by means of
 464 the MAC criterion, considerations analogous to those already made when discussing the experimental
 465 results hold. Due to the tower flexural stiffness and the structural connections of the building blocks,
 466 an increase in the natural frequencies of the building is observed.

Table 7. Comparison between the numerical modal parameters of the building before (Mod. 4) and after retrofitting (Mod. 5).

mode	$f_{n,b}$ [Hz]	$f_{n,a}$ [Hz]	$(f_{n,a} - f_{n,b}) / f_{n,b}$ [%]	MAC
1 st	3.43	3.48	1.46	0.97
2 nd	3.68	3.85	4.62	0.99
3 rd	4.10	4.23	3.17	0.99
4 th	4.72	5.57	18.01	1.00
5 th	7.55	8.06	6.75	0.99
6 th	9.07	9.41	3.75	0.99
7 th	10.92	11.87	8.70	0.96

This increase is low for the first three modes (“linear” modes), ranging between 1.46 and 4.62 %, while it becomes more important for higher modes, with an increment ranging between 3.75 and 8.70 %. Finally, a significant increase of 18.01 % is observed for the fourth mode, as it involves in-plane relative rotations between the building blocks, which are more restrained after the retrofitting. With reference to mode shapes, values of MAC greater than 0.95 demonstrate that very small differences are observed for the first seven modes, consistently with the features of the protection system.

6 Conclusions

In this paper, the experimental and numerical modal properties of a school building before and after seismic retrofitting with external steel towers equipped with dissipative devices have been presented. Since the retrofitting system must guarantee the building usability after severe earthquakes, the design must be carried out with a model addressing the actual dynamic properties of the building, which strongly depends on both structural and non-structural members. In this framework, experimental modal parameters obtained from ambient vibration measurements are used to upgrade a conventional structural f.e. model of the building to be used for the retrofit design, including contributions of the in-plane deformability of floors, internal partitions, the external infills and the

486 surrounding retaining walls. The role of the above usually neglected aspects on the overall dynamic
487 structural behaviour is shown by progressively refining the building modelling.

488 Natural frequencies and mode shapes of the refined f.e. model are in good agreement with the
489 experimental ones, with both reference to fundamental and higher modes, demonstrating the reliability
490 of the model. The model was used for the design of the seismic protection system and provides the
491 expected modal parameters of the retrofitted building that are used to assess the actual stiffening
492 contributions of the retrofit.

493 By comparing the modal parameters of the building before and after retrofitting, an overall
494 increase in the resonance frequencies has been observed. This increase is low for the first three modes
495 (predominantly the first transverse, torsional and longitudinal modes), while it is more pronounced for
496 the higher modes, as expected by numerical predictions. Furthermore, by comparing the mode shapes
497 obtained before and after the building retrofit by means of the modal assurance criterion, it resulted
498 that the first mode shapes remain almost unchanged while those related to the higher modes undergo
499 greater variations, consistently with the peculiarities of the adopted protection system and the
500 numerical expectations.

501 Overall, the dynamic identification of the building through ambient vibration tests revealed
502 crucial for the development of a refined reliable f.e. model for the seismic protection system design,
503 and the subsequent assessment of the actual stiffening contribution of the retrofit (dynamic proof test).
504 This is of paramount importance when the seismic retrofit is designed to limit damage to non-
505 structural components under seismic actions, because both structural and non-structural elements
506 contribute to the overall dynamic response of the construction.

507 **Data availability statement**

508 Some or all data, models, or code that support the findings of this study are available from the
509 corresponding author upon reasonable request.

510

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