# A simplified method to assess the seismic vulnerability of school buildings

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# A SIMPLIFIED METHOD TO ASSESS THE SEISMIC VULNERABILITY OF SCHOOL BUILDINGS

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## *Abstract*

The Italian school building asset consists of over 40,000 units, 60% of which was built before the introduction of the national seismic design code. When dealing with existing buildings, a lot of structural information, such as dimensions of resisting elements, material properties, joint connection details, loading conditions, are not accessible. This might lead to uncertainties in structural analyses. In the last few decades, several structural health monitoring (SHM) techniques have been used to compensate such lack of data and perform more reliable analyses. However, SHM might result in costly and time-consuming surveys that can also be affected by uncertainties due to environmental effects, measurement noise, and signal processing errors. This paper proposes a simplified methodology to evaluate the seismic vulnerability of existing buildings by presenting a straightforward SHM procedure to collect the fundamental information required to build a finite element model (FEM). The methodology was applied to an existing school facility built in the 70s located in Melzo (Italy). Firstly, a series of non-destructive tests based on ambient vibrations and harmonic horizontal excitations induced by a vibrodyne was performed. Output-only and Input-Output techniques were utilized to obtain the dynamic properties of the structure, i.e. natural frequencies, vibration mode shapes and damping coefficients. Subsequently, the experimental results were used to create and calibrate the FE model. Nonlinear analyses were carried out to estimate the maximum capacity of the structure in terms of acceleration and base shear at different performance levels. Finally, the vulnerability index was computed following the guidelines provided by the current Italian seismic codes. The proposed methodology is advantageous for both damage detection and seismic retrofitting strategies.

*Keywords: school building; structural health monitoring; vulnerability index; retrofitting strategy; nonlinear analysis*

## 1. Introduction

Strong earthquakes affecting urban areas may have disastrous consequences where buildings have been designed without appropriate seismic codes. This is even more critical when it comes to national heritage and strategic buildings such as schools. Current Italian standard [1] requires the evaluation of the seismic vulnerability index for existing structures through the assessment of the maximum bearing capacity. The seismic vulnerability of an existing structure depends on several key components and it can be described as its susceptibility to damage by ground shaking of a given intensity. The aim of the seismic vulnerability assessment is to obtain the probability of a given damage level due to an earthquake.

Recently, a new Italian standard [1] introduced a guideline to compute the seismic vulnerability index for existing buildings: it uses the maximum bearable seismic action and the Ultimate Limit State (ULS) earthquake intensity that should be used to design a new building with the same characteristics of the existing one as a reference measure. Nevertheless, current Italian standard [1] does not provide any specific procedure which to be followed for analysis and computation of maximum bearing capacity. It can be interpreted as an implicit recognition of the singularity of each structure that comes from different designs and uncertainties [2]. Therefore, a methodology that combines specific processes, e.g. field testing and numerical modeling, to compute a reasonable vulnerability index with the purpose of being replicated on a large number of buildings is still lacking. In this work, an integrated approach for the seismic vulnerability assessment of existing buildings is proposed and applied to an existing RC school in North Italy. The method includes a preliminary analysis of the building and a survey using nondestructive methods. Subsequently, dynamic modal characteristics are identified to calibrate numerical FE models and perform seismic nonlinear analysis. Finally, different options for the vulnerability index computation are discussed.

## 2. Methodology for seismic vulnerability

This research deals about the identification of a reliable methodology that could be employed to perform the seismic vulnerability assessment of existing buildings. With this aim, a preliminary step consists in the field investigations that include both static and dynamic tests to calibrate further FE procedures for the numerical analysis. The field investigations start with a survey of the original design documentation. This step can be critical because design tables and reports are often not available, even for governmental and public buildings. In this case, an inspection would be necessary: the survey of experts able to detect possible deterioration conditions, structural and non-structural peculiarities, and materials characteristics represents the next step. Finally, the field investigations are completed by the dynamic measurements and the identification of the modal characteristics through consolidated output-only numerical procedures. In parallel, the use of forced vibrations techniques for dynamic identification is also considered using the eccentric mass shaker (vibrodyne) with potential interaction effects between different building's modules (hammering). Lastly, the numerical model of the existing building is prepared and validated against the identified modal characteristics. The structural nonlinear behavior is also modeled to include the post-elastic response in the numerical analyses. The seismic vulnerability assessment is completed by computing the vulnerability index through different approaches with reference to the standard requirements.

A case study of an existing school building in the northern Italy is also adopted to evaluate in detail the proposed procedure on a real test-bed. It is characterized by some specific aspects that can be met frequently in existing buildings: the age of construction ('60s), the design with outdated standards, the use of reinforced concrete for the frames, the presence of structural joints that can origin hammering phenomena.

## 3. Case study

The case study on which the previously described approach has been applied is the "Mascagni School" in Melzo (Milan). It is a complex of three reinforced concrete buildings built in 1976. The school consists of three separate structures that include classrooms, a gym and a canteen (Fig. 1). This research is applied to the



building containing the classroom. The structure is a two-story building with a rectangular plan of about 98.5 m x 20 m and a height of 6.8 m. It is divided into three separated blocks by two expansion joints (Fig. 2). During the inspection, it was observed that the expansion joints opening consists of about 2.75 cm filled with polystyrene material.


Fig. 1 – Structure overall view

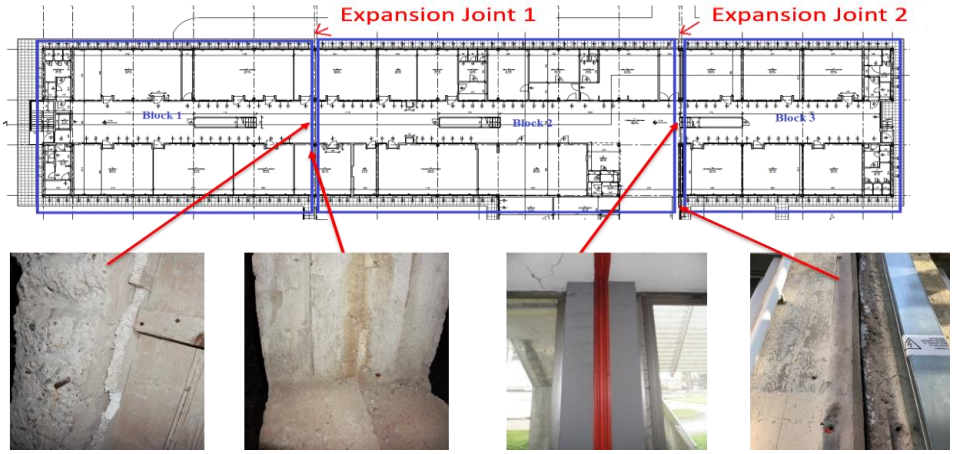



Fig. 2 - Expansion joints filled with polystyrene material

During the field investigation, the building did not show significant cracks or structural damages, while it was possible to observe by visual inspection a limited degradation of concrete surfaces, with spalling and corrosion of the outer reinforcement bars. The original design documents were not available and only few structural data were found on an existing BIM model prepared recently by the municipality. The lack of information useful for the finite element model has been filled by some non-destructive tests.

## 3.1 Non-destructive tests

A series of non-destructive tests were conducted to obtain the structural parameters such as the concrete module of elasticity and strength, the concrete cover and the steel reinforcements detail in the main structural elements. Tests with thermal camera were also performed to detect structural elements that were not reported in the available technical drawings and BIM model. The device consists in an infrared camera able to detect the different degrees of irradiation emitted by the different surface materials. Indeed, concrete elements (blue areas) show a lower temperature with respect to the masonry elements, lighting systems and aluminum ventilation shafts (orange and yellow areas) (Fig. 3a).

To identify the material characteristics, a sclerometer test was carried out according to the guidelines provided by the UNI-EN [3] (Fig. 3b). Results indicated 31.5 MPa as the average concrete strength for the beams, columns and shear walls. Therefore, the concrete class C25/30 was assumed for the subsequent numerical analyses. Furthermore, a pachometer was used to collect information about spacing, cover and size of steel reinforcement bars inside the concrete elements (Fig. 3c). The identified typical column cross section is shown in Fig. 3d.




(a) (b) (c) (d)

Fig. 3 - Thermal camera test (a), sclerometer test (b), pachometer test (c), and column cross section (d)

## 3.2 Dynamic tests

A wireless sensor network was used to collect accelerations at different building positions. The network consisted of five sensing units equipped with MEMS (Micro Electro Mechanical Systems) triaxial accelerometers (numbered from 50 to 54), with a low noise (7 μg/√Hz) and a dynamic range of 90 dB. The units also implemented GPS receivers allowing to create a local network of synchronized instruments using absolute time, in which one sensor assumes to be the 'Master' and the others are 'Slaves'. The Master unit was implemented to communicate with the other ones, collecting data from the Slaves units and coordinate the connection with a remote server. The network can be connected to a PC to manage the data recording, downloading and processing in real time through the remote connection. Fig. 4a shows the MEMS accelerometers positioning on the building.

The dynamic characterization of a structure was achieved through ambient vibration and forced vibration tests. The last ones have been performed using a vibrodyne to assess the efficiency of the adopted accelerometers. The vibrodyne was fixed to the RC shear wall connected to elevator containment (Fig. 4b) at the ground floor level. The device can perform within 0-33 Hz and apply a variable force in the range 0-40 kN. Different harmonic excitations were applied along the building longitudinal direction (North-South) by varying rotation frequencies from 0 to 30 Hz (corresponding to a varying force of 0 to 40 kN).

(a) (b)

Fig. 4 – Installation examples of MEMS accelerometers (a) and vibrodyne positioning on the shear wall (b)

Different sensor configurations were considered for dynamic tests. Each one was set individually in each block to verify the efficiency of the expansion joints. Furthermore, the configurations were designed in such a way to record torsional modes characteristics. Four configurations were finally considered for ambient vibration tests (S1A, S2A, S3A and S4A), while for forced vibration tests the sensing units were arranged in V1 and V2 configurations (Fig. 5). Each test had duration of about 15 minutes with a sampling frequency fixed at 200 Hz.



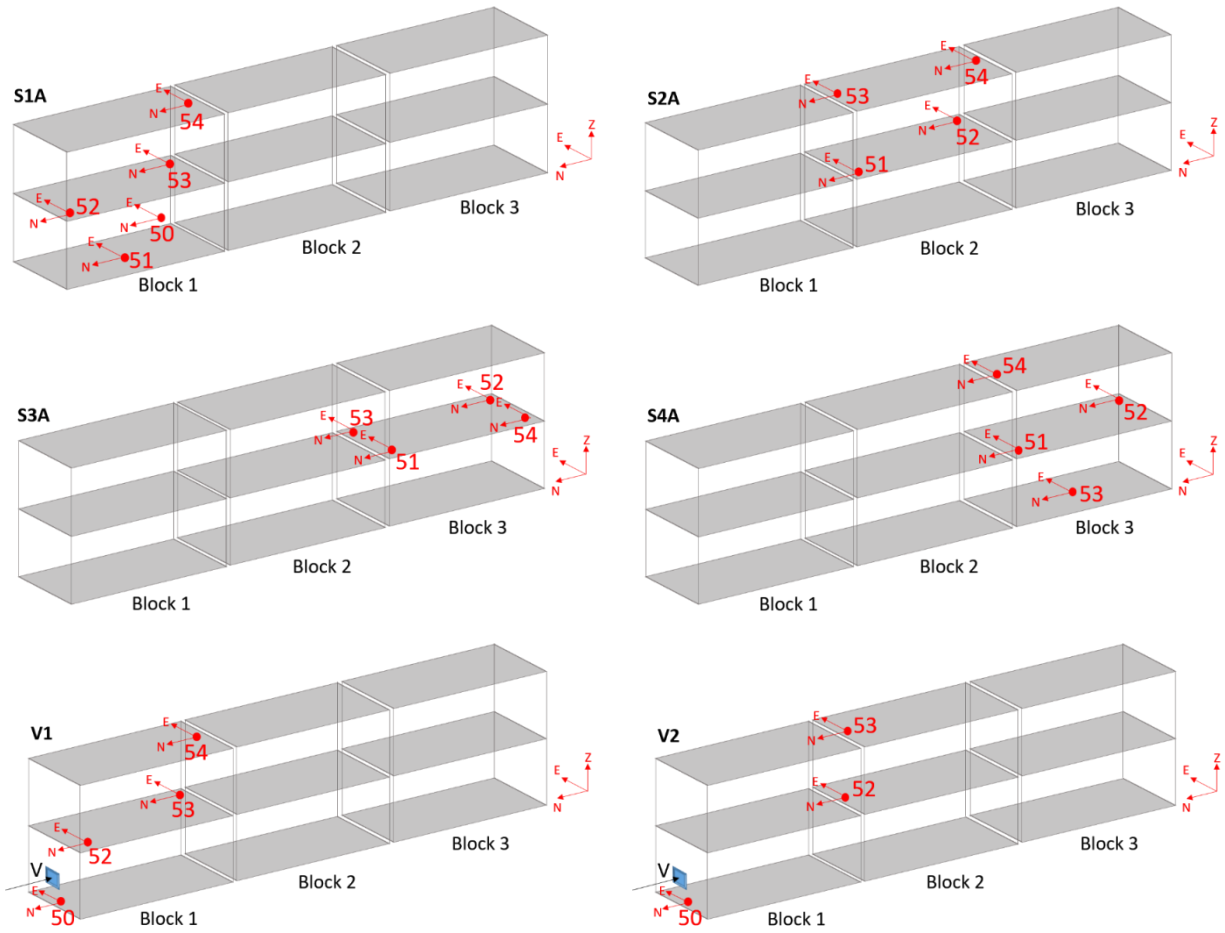



Fig. 5 - Accelerometer configurations for both ambient and forced vibration tests

### 3.2.1 Identification of modal parameters

Data processing first consists in extrapolation of the raw recorded signals corresponding to each configuration. Then, a low-pass filter is used setting the cut-off frequency at the value of 20 Hz. The resulting signals were processed through FDD (Frequency Domain Decomposition) and RDT (Random Decrement Technique) using MATLAB [4] for ambient vibration tests. FDD identification algorithm is based on the Power Spectral Density (PSD) matrix decomposition using the Singular Value Decomposition (SVD); RDT is a time domain procedure in which the linear structural response is transformed into a random decrement function [5]. The singular values diagram of the PSD matrix as function of frequencies was computed, as well the CPSD matrix through the Cross Power Spectral Density (CPSD) function in Matlab. The graphic representation of the singular spectrum allowed identifying the peaks corresponding to the main natural frequencies. An example of the processed signals for the S1A configuration is represented in Fig. 6, where the peaks are clearly visible at the same frequency values in both FDD and RDT output-only approaches.



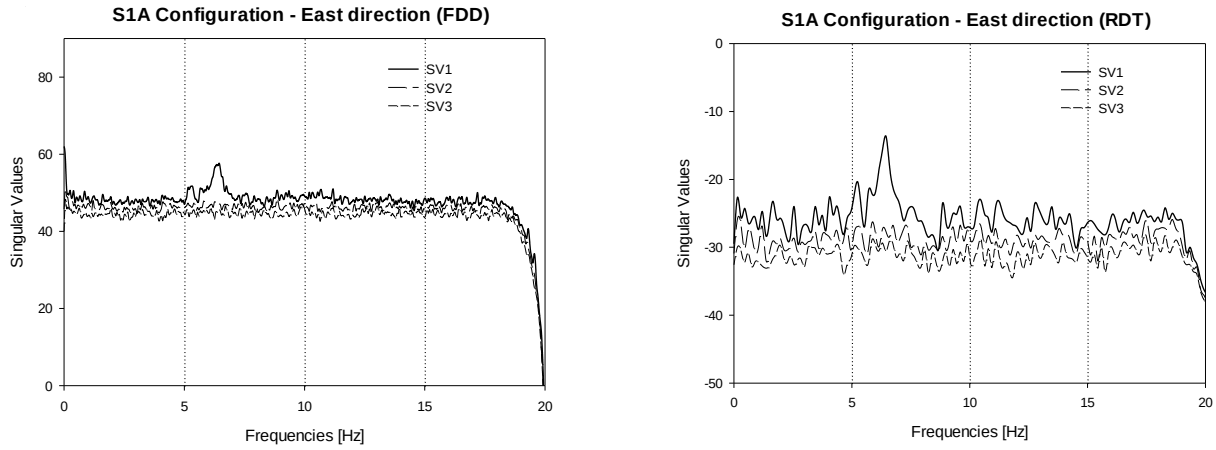



Fig. 6 – Signal processing using FDD (a) and RDT (b) for configuration S1A in E-W direction

As expected, results show that the structure is more rigid in the transversal direction (E-W) since the principal axes of the columns are oriented along this direction. Furthermore, the identified modes are flexural.

Table 1 - Identified natural frequencies for S1A configuration for Block 1

| Configuration | Direction | FDD [Hz] | RDT [Hz] |
|---|---|---|---|
| S1A | E-W | 6.45 | 6.44 |
| | | 5.30 | 5.24 |
| | N-S | 9.75 | 9.74 |
| | | 13.40 | 13.34 |

MEMS recorded data from the vibrodyne tests in N-S direction have been analyzed through the FRF method for V1 and V2 configurations (for Block 1 and Block 2 respectively). A wide range of frequencies from 5 Hz to 30 Hz was applied during the tests. In order to measure the input load, an accelerometer (#50) was placed close to the vibrodyne. Normalizing the output with respect to the input in terms of Fourier Transform, the main natural frequencies of the structure were identified. Each FRF was computed and the resonance frequencies were identified where the real part approach zero values. On the other hand, peaks directions of the imaginary part determine the associated mode shapes. Table 2 presents the results for V1 and V2 configurations with respect to those ones computed by output-only techniques for both Block 1 and Block 2.

Table 2 - Comparison of natural frequencies between different techniques, Blocks 1 and 2

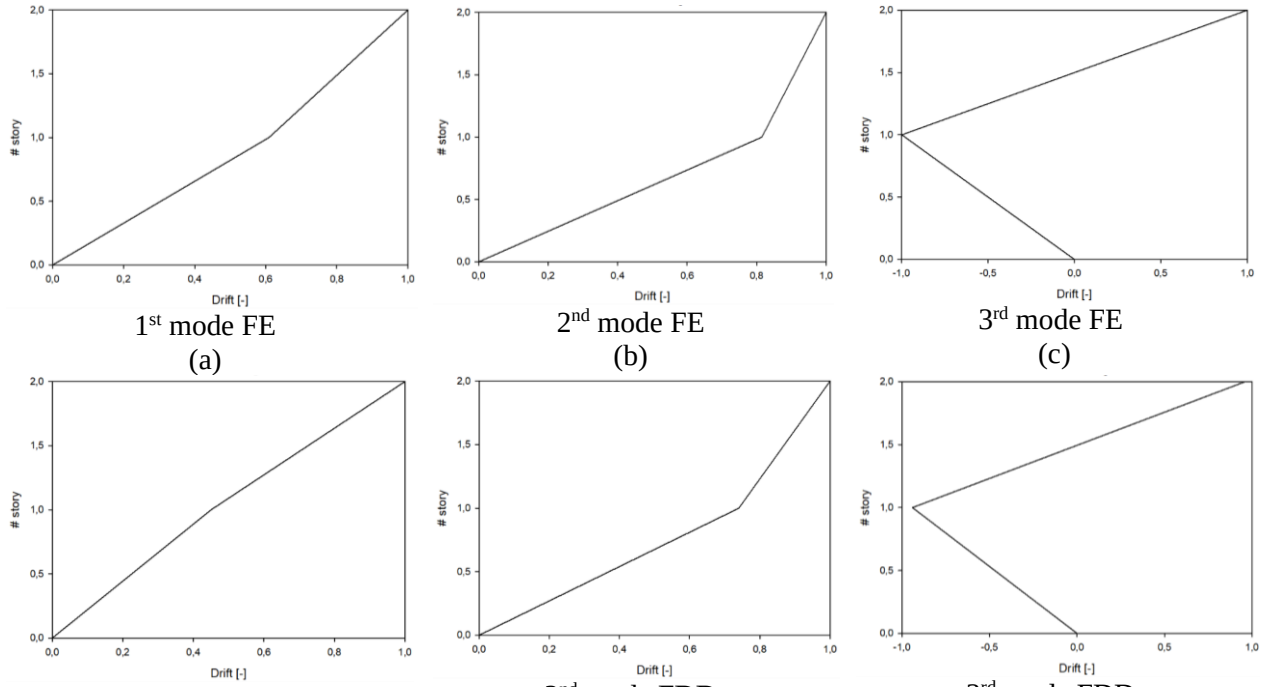
| Block | Configuration | Direction | FRF [Hz] | FDD [Hz] | RTD [Hz] |
|---|---|---|---|---|---|
| 1 | V1 | N-S | 5.0 | 5.3 | 5.2 |
| | | | 10.0 | 9.7 | 9.7 |
| | | | 14.6 | 13.4 | 13.3 |
| 2 | V2 | N-S | 5.0 | 5.3 | 5.4 |
| | | | 10.0 | 7.3 | 7.3 |
| | | | 15.0 | 17.6 | 17.5 |



### 3.3 Finite Element Model

From the available documentation and the information collected during on-site inspections, a numerical model has been created in SAP2000 and then refined to reproduce the building response. Fig. 7 shows a perfect match between the behavior of the model and that of the real structure: this is underlined by the results of the mode shapes of Block 1 computed using the FE model and those obtained with FDD experimental methodology. Finally, Table 3 reports the FE computed natural frequencies and those ones obtained by the output-only techniques.




1st mode FE (a) 2nd mode FE (b) 3rd mode FE (c)

1st mode FDD (d) 2nd mode FDD (e) 3rd mode FDD (f)

Fig. 7 - Quantitative comparison between FEM (a,c,e) and experimental (b,d,f) mode shapes for Block 1

Table 3 - Comparison between FDD, RDT and FE natural frequencies for Block 1

| Configuration | Modes | FDD [Hz] | RDT [Hz] | FE [Hz] | Participation mass ratio |
|---|---|---|---|---|---|
| | 1st mode | 5.33 | 5.30 | 5.40 | 0.91 |
| **S1A** | 2nd mode | 6.38 | 6.50 | 6.40 | 0.52 |
| | 3rd mode | 13.40 | 13.34 | 13.20 | 0.97 |

To assess the seismic vulnerability of the school building, it was necessary to include nonlinearities within the FE numerical models developed in the previous step. With this aim, nonlinear constitutive laws for construction materials and plastic hinges [6] were introduced through the adopted FE code.

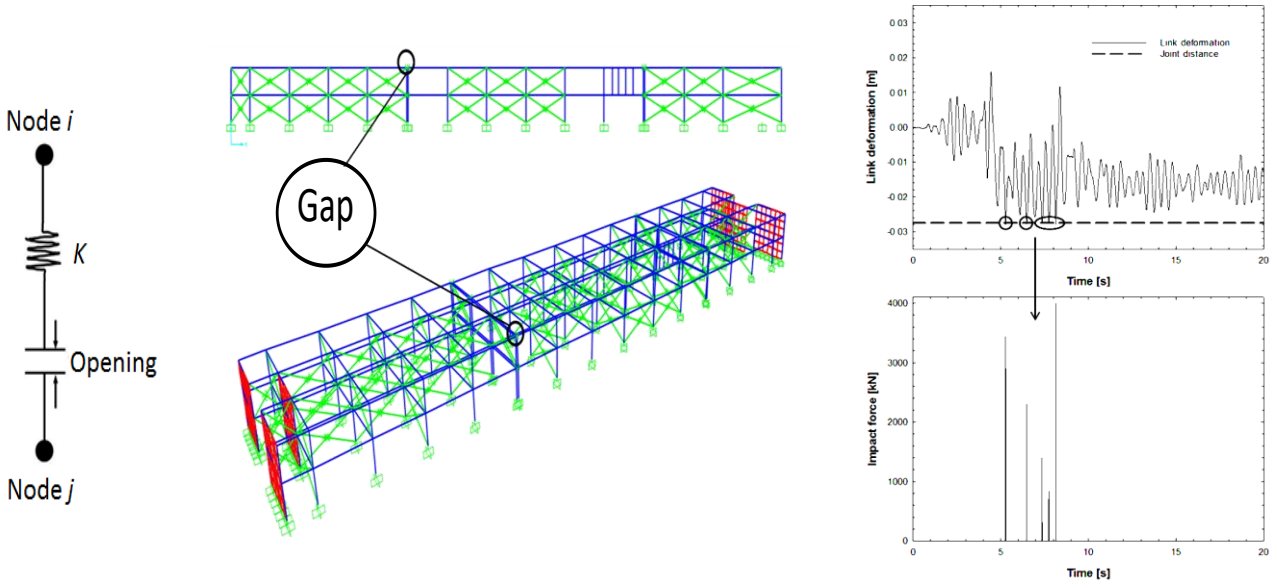
To consider possible collapses mechanisms because of hammering effects between adjacent blocks, gap elements were defined to model the expansion joints. The gap element is able to connect two adjacent nodes to model the contact conditions. Thus, it reacts with compression interaction forces when adjacent blocks approach each other, while it does not provide tensile forces. The impact compression force is set to be generated exclusively when the opening parameter (2.75 cm) is exceeded. Fig. 8a shows the gap element



model with the relevant parameters between connection nodes i and j. Parameter k is the gap element stiffness that was assumed 102-104 times the connected elements one.

The gap elements were positioned at the contact nodes of each floor to model potential hammering effects. In that case, all elements may be affected because of internal forces propagation and redistribution, and local or global collapses may also occur. Fig. 8b shows the whole FE model with gap elements at the expansion joints. It was employed to perform non-linear dynamic analyses for seismic vulnerability assessment. Fig. 8c shows an example of dynamic analysis with gap element activation due to hammering at several time instants. The generation of the contact force at the pounding between the adjacent blocks can be noted.




(a) (b) (c)

Fig. 8 – Gap element model (a) and building FE model with gap elements (b) and hammering effect performing nonlinear time history analysis (c)

## 4. Seismic vulnerability assessment

The seismic vulnerability of a building depends on the lack of some main features that may affect fundamental structural components. Current Italian standard [1] introduces a new parameter $\zeta_E$ as a vulnerability index for a straightforward comparison between the maximum bearable seismic action of the existing structure and that one required to design a new one on the same soil and with the same vibrational characteristics [7]. The maximum PGA (Peak Ground Acceleration) as a comparison parameter is prescribed.

This section explores the regulatory recommendation ($PGA_{max}$) in detail and compares it with an alternative formulation to compute the vulnerability index (maximum spectral acceleration at the reference period). Both formulations are presented below and applied to the school building under consideration, composed by three blocks and expansion joints, performing dynamic non-linear analyses.

### 4.1 Vulnerability index – Method 1

The seismic vulnerability index for existing building introduced by current Italian regulations is based on the following relationship [1]:

$$\zeta_E = \frac{PGA_{Collapse}}{PGA_{Design}} \tag{1}$$



where $PGA_{Collapse}$ is the maximum bearable seismic action of the existing structure in terms of PGA, while $PGA_{Design}$ is the design peak ground acceleration at the Collapse Limit State (CLS) for the new building with the same characteristics of the existing one. The nonlinear characteristics have to be introduced in order to reproduce the potential collapse mechanisms. For the school building they are related to the potential generation of plastic hinges at the main structural components of the RC frame and hammering phenomena.

To define the maximum bearable seismic action $PGA_{Design}$ at Eq. (1), an iterative procedure at is used. To this aim, seven SLC spectrum compatible records were selected in both horizontal directions using the GMSM (Ground Motion Selection Modification) procedure based on the seismic energy principle [8]. The selected records are compatible with the spectral acceleration at the reference period of structure (0.19 s) and with the seismogenetic parameters of the site. Furthermore, OpenSignal software [9] was used to select the seven records.

Dynamic nonlinear analyses were performed by applying accelerations in both directions simultaneously. Accordingly with FEMA [10], the maximum inter-story drift associated to the collapse prevention limit state for buildings with shear walls in both directions is set as 2% (for Block 1 and Block 3), while for framed buildings it is defined as 4% (for Block 2). The dynamic response of the building in terms of maximum average inter-story drift was calculated for the seven time histories and compared with the maximum allowable inter-story displacement representative of the collapse prevention limit state (CP). If the computed drift was lower than the target displacement, the records were scaled based on PGA until the average of the maximum inter-story drift for all seven records reached the target. At this iteration, $PGA_{Collapse}$ value was identified as the average of the PGAs of the seven records. Fig. 9 illustrates the spectra of the seven selected records and the mean spectrum compatible with the site spectrum, for both horizontal directions. The target drift was reached firstly at Block 1 (2% of drift), while the second one was more flexible with respect to the other ones (Fig. 10). Furthermore, Block 3 reached 1.7% showing more strength with respect to the first one. Finally, $PGA_{Collapse}$ (average value of PGA of seven final scaled records) resulted 0.337 g, while the vulnerability index equals to 1.57 (Table 4).

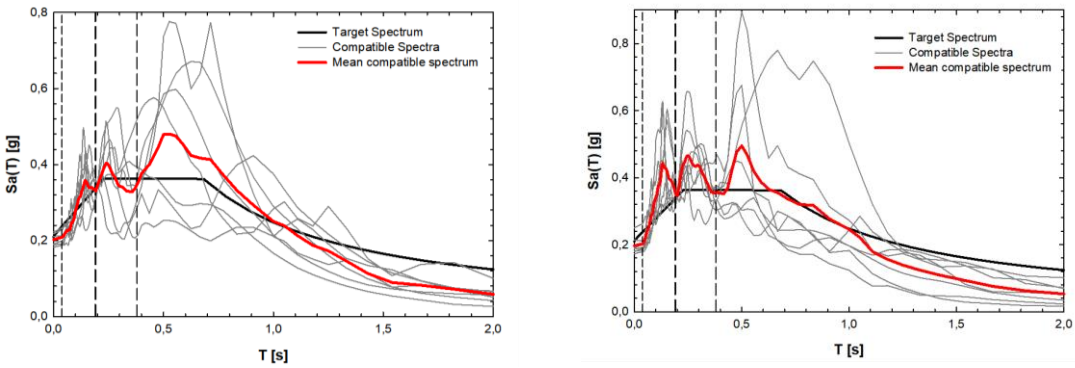


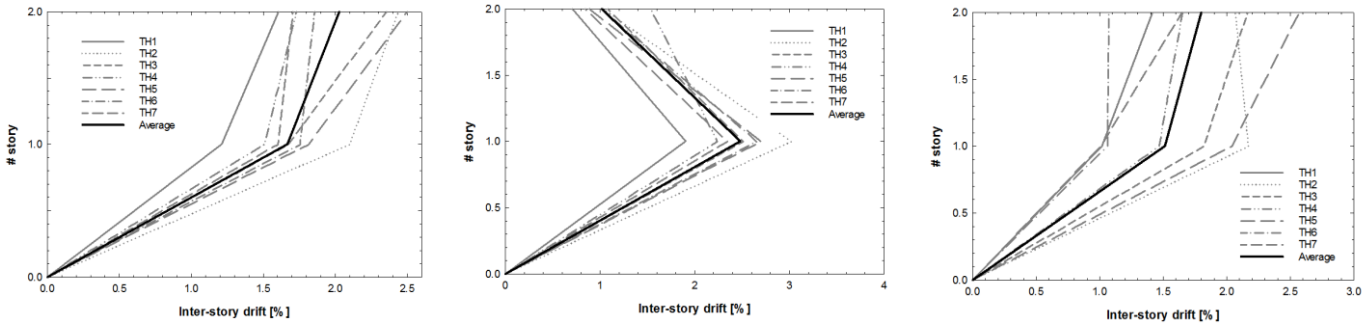

Fig. 9 - Input spectra at collapse: (a) x and (b) y direction

Table 4. Vulnerability index evaluated using Method 1

| $PGA_{Design}$ [g] | $PGA_{Collapse}$ [g] | $\zeta_E$ |
|---|---|---|
| 0.214 | 0.337 | 1.57 |






(a) (b) (c)

Fig. 10 – Inter-story drift related to collapse prevention limit for Block 1 (a), Block 2 (b) and Block 3 (c) using Method 1

## 4.2 Vulnerability index – Method 2

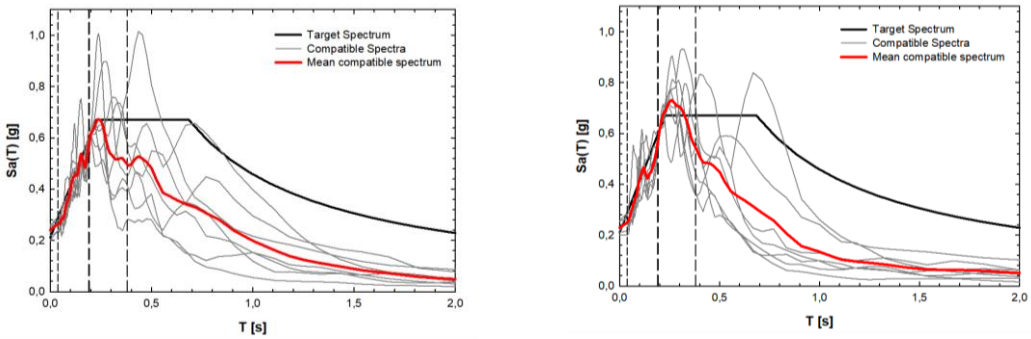
An alternative formulation to compute the vulnerability index ($\zeta'_E$) is proposed as follows:

$$\zeta'_E = \frac{S_a}{S_d} \tag{2}$$

where $S_a$ is the maximum bearable spectral acceleration of the existing building at the reference period of the structure, while $S_d$ is the CLS spectral acceleration that would be used in the design of a new building on the same soil, with the same characteristics, at the reference period. In this method with respect to the first one, the spectral acceleration at the reference period is considered as critical parameter instead of PGA.

The spectral acceleration at the denominator of Eq. (2) ($S_d$) is fixed by the design spectrum at site considering the first natural period of the existing building. For the school under study, it resulted 0.332 g at the reference period of 0.19 s (Fig. 9). Spectral acceleration $S_a$, representing the collapse state, is defined through dynamic non-linear analyses by iteratively scaling the site design spectrum. Thus, at each step seven records compatible with the scaled spectrum are selected. Coherently with Method 1, time histories were applied simultaneously in both horizontal directions and inter-story drift limits were used to identify the target spectral acceleration: i.e. 2% for blocks with shear walls (Block 1 and Block 3) and 4% for framed building (Block 2).

For each set of time histories, the dynamic response (in terms of maximum average inter-story drift) was computed and compared with the inter-story drift representative of the collapse prevention (CP) limit state. Differently for the previous Method 1, if the calculated inter-story drift was lower than the one corresponding to collapse, a new set of records is selected based on the scaled design spectrum. This procedure is repeated until the average of the maximum inter-story drift values for seven time histories reaches the drift value corresponding to the CP limit.

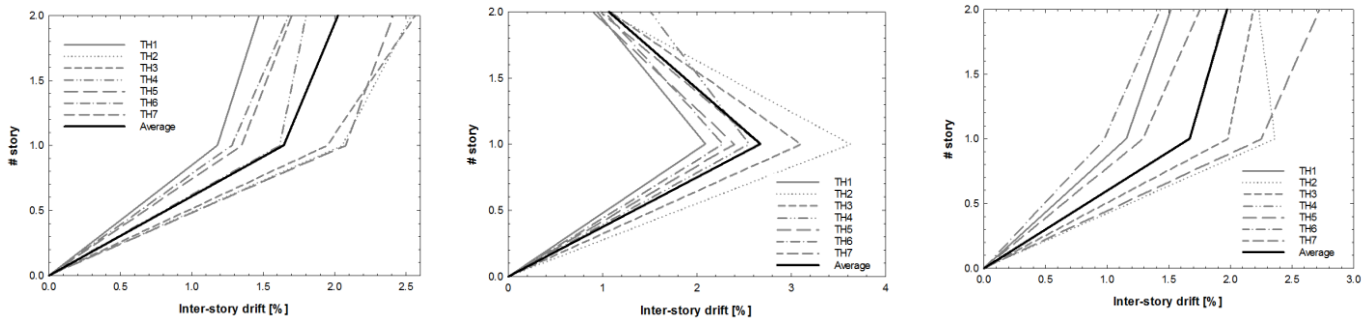



(a) (b)

Fig. 11 – Target spectra at collapse: x direction (a) and y direction (b)






(a) (b) (c)

Fig. 12 - Inter-story drift related to collapse prevention limit for Block 1 (a), Block 2 (b) and Block 3 (c) using Method 2

With respect to Method 1, this method is more accurate but less immediate because at each iteration step it is necessary to amplify the reference spectrum and then to select a new compatible set of records. For the school building, spectral acceleration $S_a$ was computed as the maximum bearable spectral acceleration corresponding to the earliest achievement of the CP limit drift in one of the three blocks. Fig. 11 shows the target spectra at collapse and the mean spectrum of seven compatible records, for both horizontal directions. The block that collapsed first is Block 1 with an average inter-story drift of 2%, while the second and third blocks reached 2.7% and 1.8%, respectively (Fig. 12). Finally, $S_a$ resulted 0.579 g, and consequently the vulnerability index was calculated as 1.74 (Table 5).

Table 5 - Vulnerability index estimated using Method 2

| $S_d$ [g] | $S_a$ [g] | $\zeta'_E$ |
|---|---|---|
| 0.332 | 0.579 | 1.74 |

## 5. Conclusions

A comprehensive methodology to assess the seismic vulnerability of existing buildings is presented in this paper and applied to a school building located in Northern Italy. The first step of the procedure consists in on-site investigations to evaluate the actual structural conditions and characterize building materials and structural dynamic characteristics. To this purpose, a wireless sensor network was used, and different structural identification methods were implemented to compare their efficiency for dynamic structural characterization. Frequency response functions were computed, and the modal characteristics were identified using input-output and output-only methods. The comparison highlights equivalent results in terms of modal characterization. The output-only method resulted more advantageous because it does not require the use of equipment to excite the structure. The second step of the procedure consists in the creation of a numerical model that is able to replicate the actual dynamic response of the building in both linear and non-linear domains. Finally, the seismic vulnerability index introduced by the Italian standard (Method 1) was calculated and compared with a new formulation herein presented (Method 2). Method 1 uses Peak Ground Acceleration (PGA) to determine the maximum capacity of the structure, whereas Method 2 is based on the spectral acceleration at the period of the structure. Method 1 is more conservative with respect to Method 2 since the maximum bearing capacity at collapse is evaluated using time histories scaled based on the acceleration at the ground instead of the spectral acceleration at the period of the structure. Overall, Method 2 was found to be more accurate despite it required more computational effort.



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## 7. Copyrights



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