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DICAM

DAMAGE DETECTION IN A PRECAST STRUCTURE SUBJECTED TO AN EARTHQUAKE: A NUMERICAL APPROACH

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Abstract.

Seismic risk mitigation is a relevant issue in European regions, such as Italy, characterized by the presence of a large stock of vulnerable existing industrial buildings.

Through a case study, the article proposes an integrated novel approach for the diagnosis of structures after a seismic event. The suggested monitoring system is based on recording the accelerations of a real structure during an earthquake and on their introduction as input into a numerical model, suitably tuned, in order to outline a possible post-earthquake scenario.

The leading idea of this approach is to provide an estimation of the health and residual life of monitored structures, and to detect and quantify the damage, some of the crucial issues of Structural Health Monitoring (SHM). The technique is applied to a real structure, an industrial building liable of some seismic vulnerabilities. It did not undergo an earthquake, so the real accelerations could not be recorded. For this reason, they are acquired from a second numerical model subjected to real and simulated earthquakes.

1. Introduction

One of the most important issues in civil and mechanical engineering is the detection of structural damages, defined as changes of material properties, boundary conditions and system connectivity, which adversely affect the system performance. The damage identification process generally requires to establish the existence, localization, type and intensity of the damage.

Recent seismic events in Italy have clearly shown the high seismic vulnerability of existing precast (industrial) buildings that often reveal an inadequate safety level against seismic actions [1–3]. These events produce structural damages and, in the worst cases, lead to loss of life. Therefore, an integrated structural monitoring system for existing buildings is very useful for determining the effects of an earthquake, in particular for precast concrete structures.

In this paper a novel approach to perform the diagnosis of a structure is presented. The method is based on recording the floor accelerations of a real structure during a seismic action, and on their introduction as nodal accelerations in a refined numerical model of the structure, strongly nonlinear, endowed with an elastic plastic (softening) damage constitutive law. The model is then able to detect the existence, the position and the amount of damage induced in the structure by the earthquake, providing a possible post-earthquake scenario.

Another important purpose of this work is the quantification of the damage through the calculation of damage indices. The method is based on recording floor accelerations of a real structure during a seismic action, and on their introduction as nodal accelerations in a refined Numerical Model (NM) of the structure, strongly nonlinear, endowed with an elastic plastic (softening) constitutive law. The model is then able to detect the existence, the position and the amount of damage induced in the structure by the earthquake, providing a possible post-earthquake scenario.

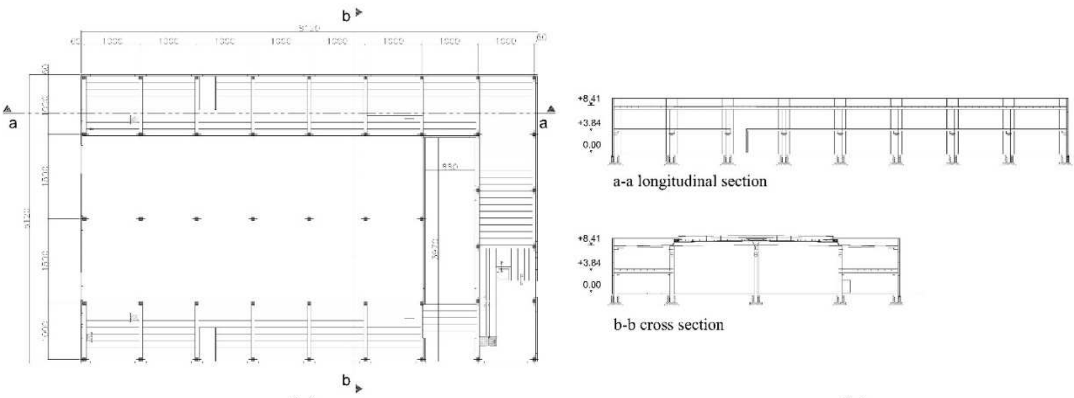
It will be proposed a *global damage index*. This index is representative of the health of the entire structure and it is used for the emergency conditions, when the plastic regime is activated (e.g. building evacuation).

2. Description of the structure

The industrial building is sited in a small Italian town 80km distant from Ancona, the main city of Marche region. This area was subjected to the Umbria-Marche earthquakes that struck the central Italy in 1997 and 2016. The structure has a simple and geometrically regular structural scheme, which is typical for r.c. precast industrial structures (Fig. 1). The plan is a rectangle of 80 m in the longitudinal direction and 50 m in the transversal one (Fig. 2).

Figure 1. External view of the building.

All elements are precast, except for some in-situ cast concrete substructures. The first floor heigh is 3.84m while the second floor one is equal to 4.57m; it is assumed z = 0 m at the finished ground floor, z = 3.84 m at the first level and z = 8.41 m at the roof.



(a) (b)

Figure 2. (a) Plan view and (b) vertical sections of the building.

3. FEM model calibration with linear and nonlinear analysis

The examined industrial building is a precast structure: typically, the connections among the elements should be modeled as cylindrical or spherical hinges depending on the constraint degree offered by the connections. All columns are modeled as one-dimensional finite elements fixed at the ground level (z=0 m). In Tab. 1 the results of the eigenvalues analysis are shown; the two FEM models provide very similar periods, participant masses and modal shapes (Fig. 3), confirming that they match satisfactorily and can be used for the present purposes.

Table 1. Modal parameters of the structure (in bold the participation masses of the main modes)

MODE (n°)	**Frequency (%)**		**Error (%)**	**Partic. mass X (%)**		**Particip. mass Y (%)**	
	MIDAS	SEISMO		MIDAS	SEISMO	MIDAS	SEISMO
1	**0.938**	**0.947**	0.95%	0.000	0.000	**9.378**	**9.473**
2	**0.968**	**0.974**	0.63%	0.004	0.013	**33.568**	**32.758**
3	**1.002**	**1.006**	0.46%	**55.070**	**53.342**	0.526	0.598
6	**1.185**	**1.209**	2.03%	0.000	0.000	15.419	0.783
7	**1.279**	**1.248**	2.42%	0.100	0.055	**5.726**	**7.351**
8	**1.354**	**1.275**	6.15%	0.001	0.000	1.194	2.388

The fiber model approach [4,5] is used in both software in order to have a realistic description of the post-elastic behavior of the reinforced concrete structure. To perform dynamic analysis in both software, each time-history is obtained using the Newmark's integration algorithm [6]. The accuracy of the Newmark's method depends on the period of the excitation; in the present case each step of integration ($\Delta t = 0.01$ sec) satisfy the relation $\Delta t \leq 0.318\, T_n$, where T_n (0.162 sec) is the period of the vibration mode that allows to reach the 85% of the participating mass [7].

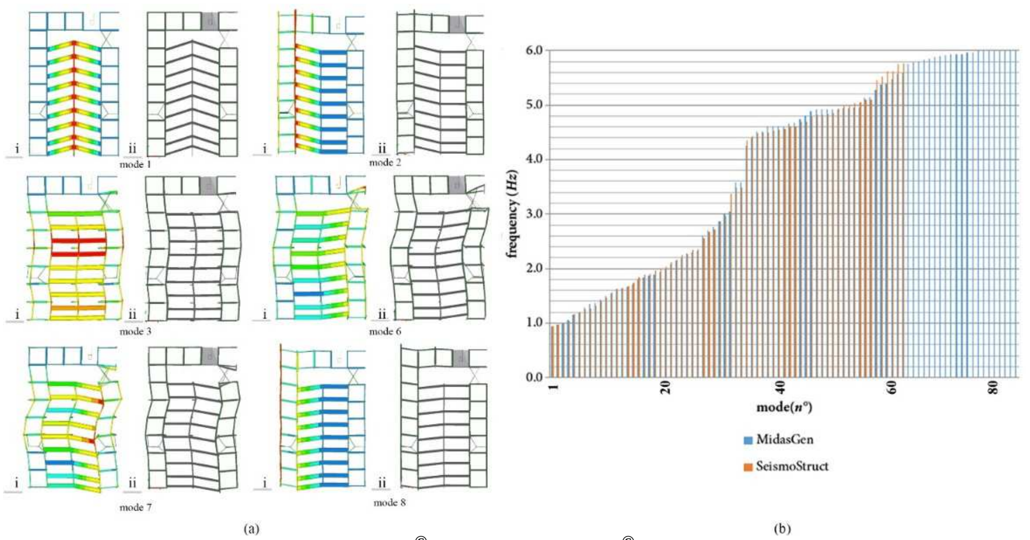


Figure 3. (a) Modal shapes with Seismostruct© (i) and MidasGen© (ii) (b) Frequency comparison between the two models.

In order to have realistic accelerations to reintroduce as input into the damage identification process, a 25 seconds spectrum-compatible time history has been generated through the software Simqke_GR and used as ground acceleration at the base of the columns.

This ground acceleration is used also to check the agreement of the two software in the post-elastic regime. A comparison between displacement (output) obtained by the two models subjected to the same ground acceleration (input), is reported in Fig. 4 and shows a very good agreement.

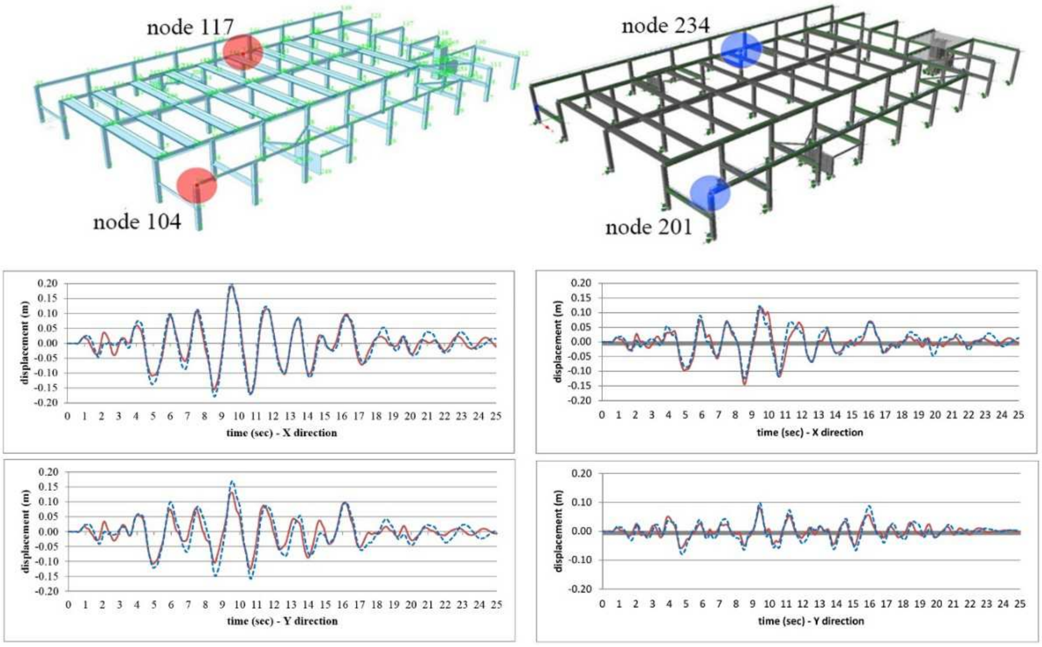


Figure 4. Comparison between the displacement [m] obtained by the two software

4. SHM implementation and damage indices

Accelerations are derived from the SeismoStruct© software and then reintroduced as input into the MidasGen© software to simulate the process of the proposed monitoring system.
At this stage, the nonlinear dynamic analysis in MidasGen© can be performed to detect the amount of structural damage and to calculate the damage indices, as subsequently discussed.

4.1 *Damage indices and localization of the damage*

One of the objectives of this work is to compute a specific damage index that can be implemented within a system of real-time SHM and that can be used to quantify the damage state at each time step and the possible incipient collapse [8]. In this paper the ductility damage index DI_μ is used. This index was proposed by Powell and Allahabadi [9], and is given by:

$$DI_\mu = \frac{\delta-\delta_y}{\delta_u-\delta_y} = \frac{\mu-1}{\mu_u-1}, \tag{1}$$

where δ is the maximum inelastic displacement during an earthquake, δ_y is the yelding displacement and δ_u the ultimate displacement, i.e. the capacity, both computed with a preliminary nonlinear static (pushover) analysis. This index is representative of the conditions of the entire structure and belong to the category of *Global Damage Index*, according to the classification reported in [10]. The $\boldsymbol{DI_\mu}$ does not consider damage accumulation [11], but depends on the displacements beyond the elastic threshold, one of the most important parameters to keep under control in precast structures.

For the calculation of the damage index a pushover analysis must be preliminary performed. This is usually achieved with a master node coincident with the center of mass of

the last floor that is representative of the structural behavior [12]. In this case, however, the classic approach would be incorrect considering that the capacity curve is strictly dependent on the choice of the node, because of the in-plane deformability of the horizontal floors. For these reasons a "*Global Control Pushover Analysis*" (GCPA) is proposed, monitoring which node reaches first the target displacement, set at 0.30 m, under monotonic loading. In this way, it is possible to detect which portion of the structure reaches first the collapse, and then to perform a pushover analysis monitoring the selected nodes in each direction.
The pushover analyses have put in evidence which part of the structure reaches first the target displacement in both directions (Fig. 5 a-b). For brevity only the comparison between the pushover curves obtained with lumped and diffused plasticity approach is reported (Fig. 5-c).

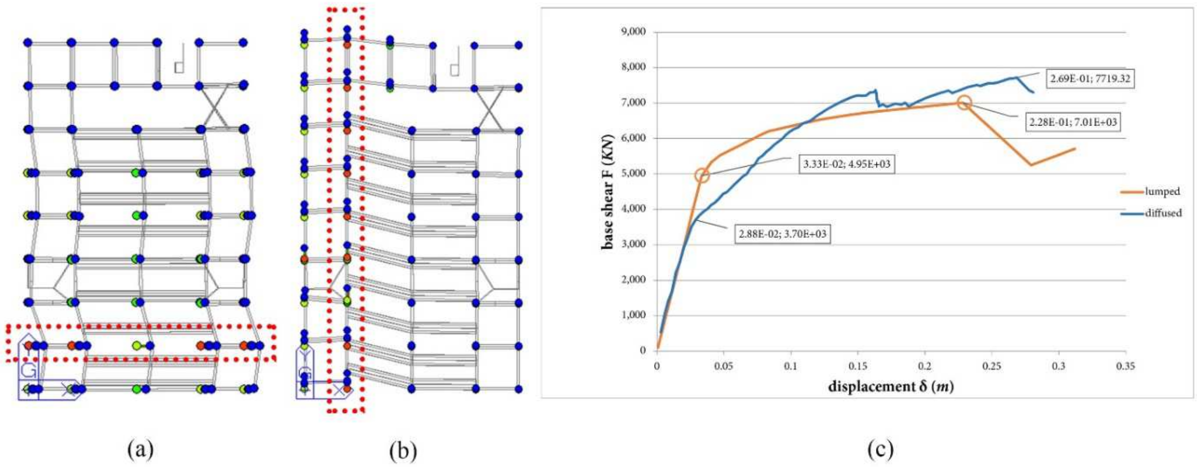


Figure 5. Pushover global control analysis. (a) x direction: δ_y = 0.033m, δ_u = 0.228m, T_b = 7010 kN (b) y direction: δ_y = 0.035m, δ_u = 0.232m, T_b = 6480 kN (c) comparison between pushover analysis conducted with lumped and diffused approach (x-direction)

Starting from the computed capacity curves, it is possible to obtain the yield and the ultimate displacement, needed in eq. (5).

4.2 *Quantification of damage*

In this section the damage index DI_μ is computed. The time histories of the displacements in two different nodes belonging to the structural portions identified by the GCPA are reported in Fig. 6. These represent the structural response in terms of displacement of the most vulnerable part of the structure. The two maximum displacements, highlighted in Fig. 6, are δ_{max} = 0.155m (*x* direction) and δ_{max} = 0.176m (*y* direction). In turn, they give the maximum damage indices, $DI_{\mu\ max}$ = 0.626 and $DI_{\mu\ max}$ = 0.714, respectively, in *x* and *y* directions.

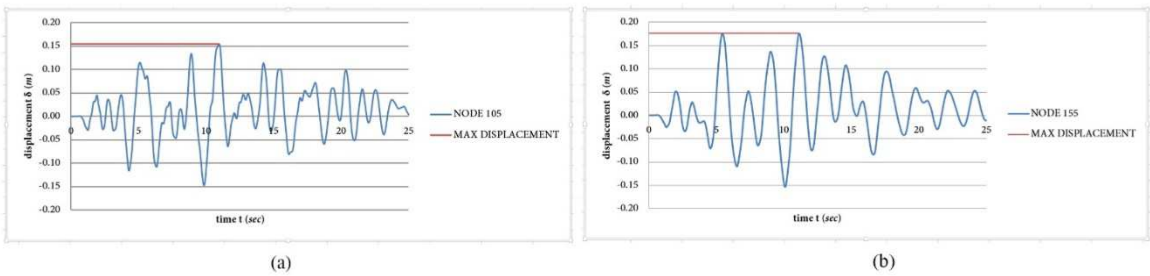


Figure 6. Displacements obtained from nonlinear time history analysis.
(a) x direction: δ_{max} = 0.155m; (b) y direction: δ_{max} = 0.176m.

A noticeable result of the proposed approach concerns the possibility of tracking the development of damage level at each time step. In fact, the combination between the updating of the model allowed by SHM techniques and the damage model built with the static nonlinear analysis, permits to know how damage evolves in time, as shown in Figure 7. From Fig. 7 it is possible to point out another consideration. There is a difference between the two principal directions *x* and *y*. In the *x*-direction (Fig. 7a) the damage index rises slower than in the *y*-direction, having some intermediate steps before reaching its peak. In the *y*-direction (Fig. 7b), instead, the damage index goes from zero to its maximum value in few steps that present a significant amplitude. This latter is clearly most dangerous from a practical point of view.

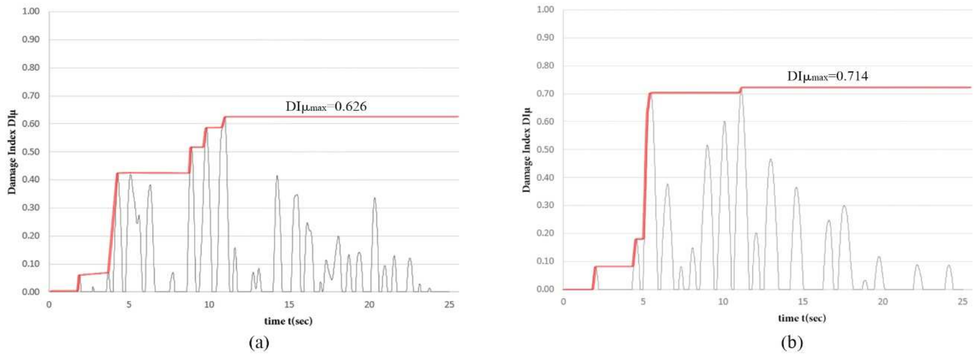


Figure 7. Evolution of damage indices during the seismic event.(a) x-direction: DIμ max = 0.626. (b) y-direction: DIμ max = 0.714.

4.3 *Damage scenarios using real earthquakes*

The purpose of this section is the quantification of the damage using real recorded earthquake. The Umbria-Marche earthquake (1997) characterized by a PGA of 1.69 m/sec^2 is considered. The two maximum displacements obtained from the Umbria-Marche earthquake are δ_{max} = 0.101 m in the *x*-direction (Fig. 9a) and δ_{max} = 0.093 m in the *y*-direction (Fig. 8b). These correspond to the maximum damage indices, $DI_{\mu\ max}$ = 0.346 and $DI_{\mu\ max}$ = 0.296, respectively, in *x* and *y* directions.

Based on the previous results, the following considerations can be drawn. The Umbria-Marche earthquake is characterized by a medium intensity but it presents a quite long cyclic behavior with similar values of peak accelerations over time. This entails mild increment in time of the damage index, especially for the *x*-direction (Fig. 8a). In the *y*-direction (Fig. 8b) it can be seen a similar qualitative behavior, with a final value of the damage index slightly lower.

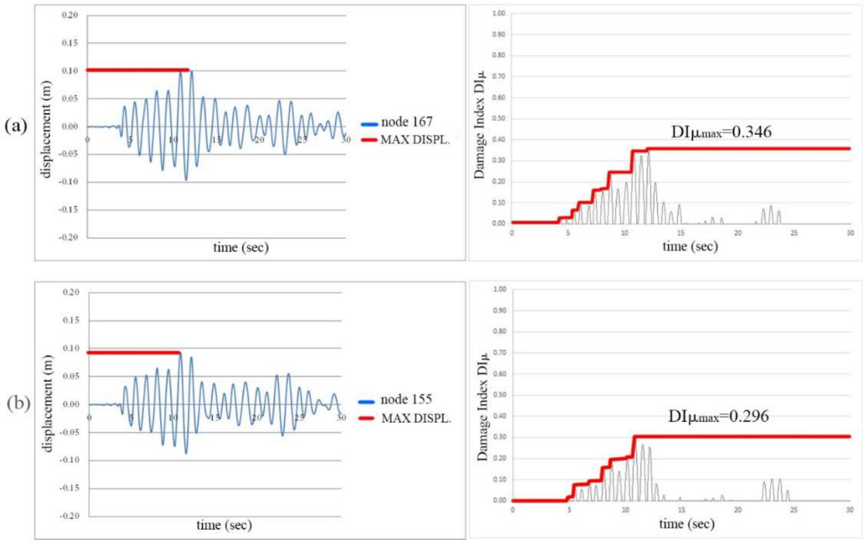


Figure 8. Maximum displacement and damage indices obtained with Umbria-Marche earthquake:
a) *x* direction δ_{max} = 0.101; DIμ max = 0.346, b) *y* direction δ_{max} = 0.093; DIμ max = 0.296

5. Conclusions

In this paper a new approach to implement a SHM system of an ordinary building has been proposed in order to outline a possible post-earthquake scenario and to calculate damage indices. The building chosen as case study is an industrial one which can be susceptible of damage during a seismic event due to some horizontal and vertical irregularities.

The excellent convergence reached between two commercial software has allowed a SHM simulated process. In fact, it was possible to extract nodal accelerations from the Seismostruct© model as if they were measured from accelerometers installed on the real structure. Consequently, these acquired data became the input for the MidasGen© model, which has been analyzed with the aim of outlining a possible post-event damaged scenario.

It is worth to remember that this work is based on a numerical simulation because (fortunately) the building in exam never underwent an earthquake, making impossible to measure real data.

For the diagnosis of the structure, a damage model is implemented using a ductility damage index; the latter is based on the displacements beyond the elastic threshold, one of the most important parameters to keep under control in prefabricated structures. This index requires an estimation of the yield and ultimate displacements of the structure, that can be determined by a preliminary static nonlinear analysis.

Considering the story deformability, an alternative way to perform nonlinear static analyses is proposed, not by defining a-priori control node, but by performing a *"Global Control Pushover Analysis"*. In this way, the nodes that firstly reach the ultimate displacement can be identified. Then, traditional pushover analyses can be carried out using those nodes as control ones in order to deduce the capacity curve of the structure in each direction.

The most relevant information resulting from the proposed procedure are the location and the magnitude of the damage. In this way, it is possible to obtain an important instrument for the choice of the evacuation routes in emergency conditions. Moreover, this approach is a powerful NDD tool for the evaluation of post-seismic structural conditions, which is one of the fundamental aspects not only related to the safety of the occupants, but also concerning the economic point of view.

Further developments of this work should be consider an ambient vibration survey in order to detect the main modal parameters of the structures and to calibrate a more reliable model for the nonlinear analysies. Other advances in SHM are focused on the use of wireless sensors networks (WSN), which are less expensive and more flexible than the wired ones[13].

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