On the definition of seismic recovery interventions in r.c. buildings by non-linear static and incremental dynamic analyses

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Abstract**—** Incremental Dynamic Analysis (IDA) is a parametric analysis method that allows evaluating the structural performance under seismic loads more accurately than traditional static and dynamic analyses. With respect to a single non-linear analysis, the incremental dynamic analysis has the advantage to evaluate the structural performance under different levels of intensity, scaling proper ground motion records, until the structure collapses or until a fixed level of deformation is reached. In this study the potentialities of incremental dynamic analysis have been investigated in identifying the damaged elements in existing irregular r.c. buildings and a comparison with the results of static Pushover Analyses has been performed. In particular a strongly irregular building has been considered, representative of a particular manufacture and of an historical period of economic growth and speculation; it has not structural problems but suffers from abandonment and weathering effects. An interpretation of IDA procedure has been proposed, with the set of a mean IDA capacity curve, then bi-linearized in order to close the pushover procedure according to the extended N2 method. The aim is to underline in a specific case study how the choice of a methodology can affect the definition of recovery interventions, especially in the case of historical buildings, where the criterion of minimum intervention should be followed in order to preserve the original features.

Keywords**—** Incremental Dynamic Analysis; non-linear analysis; r.c. buildings; recovery interventions; seismic vulnerability.

I. Introduction

THE second postwar in Italy has been characterized by a great increase of the building sector, along with both technical innovation and speculation, which led to an inadequate constructional quality and often to a very high seismic vulnerability of these buildings, as seen in the last years. In fact a lot of problems can occur in very damaged structures, in case of bad or insufficient maintenance or in case of inaccurate design.

Moreover, the risk is that those buildings, which are representative of a particular manufacture, can be heavily modified or even partially or totally demolished if the historical issue is not contemplated in seismic recovery.

The increase of computer processing capacity and the growing interest of the scientific community for the seismic structural design, determined, in the last years, the development of more complex analysis methods able to give more reliable seismic evaluations, by taking into account the secondary effects, the dissipative effects, the non-linear behavior of materials and structures [1-6]; the results affect the choice of structural recovery interventions. As a consequence it is necessary to develop non-linear analyses, in order to evaluate the post-elastic behavior of the structure, correctly define the position of the plastic hinges and understand the kind of failure. Currently, non-linear seismic analyses (especially non-linear static pushover) are very common in practice and in codes because they represent a balance between result reliability and computational effort; however these methods are not able to completely consider the torsional effects due to the structural irregularity or to evaluate the parameters during the time; so a dynamic non-linear analysis is requested to obtain more reliable results, in particular when existing buildings are concerned. Such an analysis consists in determining the seismic response through a non-linear model and by using seven different accelerograms (according to the Italian code) based on the expected seismic risk [7]. It is possible to predict the seismic capacity of structures compared to the local seismic demand, verifying the integrity of structural elements and the necessity to make recovery interventions based on the expected damage for a given level of ground shaking. In order to obtain an high accuracy, a reliable and complete structural model is needed; moreover the accelerograms should be properly chosen. Although a significant computational effort is requested, the non-linear dynamic analysis allows to identify the parameters (displacements, forces) that characterize the structural response in the time and to identify the expected damage.

The incremental dynamic analysis represents an extension of

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the non-linear dynamic analysis, recently proposed to properly estimate the structural performance under seismic loads through one or more ground records, scaled in order to obtain one or more response curves. The concept of the Incremental Dynamic Analysis (IDA) has been introduced by Bertero [8] and has been successively developed in different ways by some researchers, such as Bazzurro and Cornell [9], Yun et al. [10], Mehanny and Deierlein [11], Dubina et al. [12], Psycharis et al. [13]. The classical procedure has been proposed in FEMA [14] as "incremental dynamic analysis" and then systemized in a standard way by Vamvatsikos and Cornell [15,16] and Vamvatsikos and Fragiadakis [17]. The IDA allows to understand the range of response under different levels of a ground motion record, even with the more severe ones, underlining how all the analyzed parameters can differ from one ground motion record to another; moreover it shows the structural behavior at each step of the ground motion increase, taking into account stiffness and strength degradation [15].

The aim of this study is to compare the results of an incremental dynamic analysis with those of a pushover procedure, evaluating the sensitiveness of both analyses in terms of definition of recovery interventions, focusing on reinforced concrete constructions built more than forty years ago, characterized by great irregularity both in plan and in elevation. More precisely the comparison has been carried out in terms of requested displacement, typology of collapse mechanism and number of crashed elements. This comparison is also suggested by the common incremental loading nature characterizing IDA and static pushover analysis. In this way it is possible to evaluate how the choice of the methodology affects both the individuation of the elements requiring recovery intervention and the choice of the intervention typology; this has consequences on technology, performances and economic cost. Moreover the entity of interventions should be limited when historical buildings are concerned, in order to preserve their integrity and their original configuration. The implementation of more refined and onerous analyses is so justified by the possibility to properly predict the structural problems and the consequent recovery interventions.

II. The Incremental Dynamic Analysis: state of the art

The extended N2 method, based on pushover analysis and implemented in Eurocode 8 [18], allows to determine the seismic demand based on the period of the equivalent SDOF system [19]. When irregular 3D structures are concerned, dynamic spectral analyses combined with 3D pushover analyses are more suitable [20]. Seismic capacity can be determined through different empirical formulas; in the present work the formulas provided by Eurocode 8-3 for the calculation of the ultimate chord rotation and the shear strength of RC elements, have been used.

Incremental Dynamic Analysis consists in processing nonlinear dynamic analyses of a structure, using different and proper ground motion records, each scaled to several intensity levels, in order to fully describe the structural behavior until instability occurs [15]. Through a proper interpolation of the results, IDA curves can be obtained, each showing the relation between a parameter representing the damage measure (DM, such as peak roof drift) versus a parameter of intensity (IM, such as peak ground acceleration). The seismic assessment is performed by comparing seismic demand and capacity for different limit states, defined for each IDA, given the IM level.

Following the standard procedure explained in [15] the first step is to establish the scale factor (SF) λ to be applied to the unscaled time-history. The IM of a scaled accelerogram is monotonically increased with the scale factor λ. The Peak Ground Acceleration (PGA), Peak Ground Velocity and x = 5% damped Spectral Acceleration at the structure first-mode period are some examples of scalable intensity measure.

The DM is a scalar parameter representing the structural response under a seismic loading: maximum base shear, node rotations, peak story ductilities, peak roof drift, floor peak interstorey drift angles, etc. The chosen ground records are scaled from a low IM to higher IM levels until structural collapse occurs. For each increment of the IM, a nonlinear dynamic time history analysis is performed, and the related DM can be obtained. An IDA curve represents the plot of the chosen DM versus the IM, resulting from one or more dynamic nonlinear analyses: the DM is represented at each level of the IM of the scaled ground motion. So the IDA curve is a set of discrete points which can be interpolated, i.e. with linear approximation.

Since a single IDA curve is not sufficient to cover a wide range of structural responses, several analyses with different ground motion records are needed; in this way, several IDA curves parameterized with the same IM and DM can be obtained. While a single IDA is a deterministic curve, a set of IDA curves is related to the randomness of ground motion and thus a probabilistic approach is requested. The IDA curves can be separately fit, so obtaining the statistics of the parameters, or a parametric model of the mean DM corresponding to the fixed IM can be fit involving all the curves simultaneously. Due to the high computational effort, some simplified and approximated methods have been set up. Vamvatsikos and Cornell [16] define the force–deformation curve in initial loading of a single degree of freedom (SDF) system in order to match the curve of the real multi-degree of freedom structure and find the peak deformation of the SDF. Several force distributions are requested. Moreover, the elastic stiffness of the SDF system should be estimated from the IDA curve. Alternatively, Chopra and Goel [21,22] suggest estimating seismic demands through modal pushover analysis (MPA).

III. The case study: the "Ex convitto Duni" in Matera

A. Description of the building

The analyzed building is the "ex convitto Duni", at present the Provincial school office, in Matera (in the South of Italy), designed in 1971 by Piergiorgio Corazza and Emanuele Plasmati. The project originally contemplated two twin buildings (A and B), each constituted by two parts (A, A’, B, B’), independent up to second floor and finally connected by a transversal part named “C”. The execution of the building began in 1979, in a different place than the one contemplated in the project, determining extemporaneous adjustments of the area. Moreover only the buildings A’’, B’’ and C were accomplished, while the buildings A’ and B’ were postponed (Figs. 1-2(a)-(b)).

Although the headquarter was provisory, the settlement is nowadays unchanged and maintenance has been neglected.

The building was realized in reinforced concrete; some floors consist of reinforced concrete precast beams with an infill made of hollow clay blocks, while other floors include steel joists and reinforced concrete slab. Structural elements do not show apparent qualitative defects, so they were properly realized. However the weathering is apparent and determined the expulsion of the concrete cover and an incipient oxidation of the exposed steel bars. Moreover the lack of maintenance and the absence of external finishing, together with several acts of vandalism, caused the break of some tile elements of the external infill and of a floor of the basement. At the third and fourth floors of the unfinished building B’’, a leakage from the roofing determined blazed humidity, expulsion of the concrete cover and oxidation of steel bars. So the structure is not affected by static problems: degradation depends on the dismissal and the incompleteness of the finishing, which precipitated the weathering.

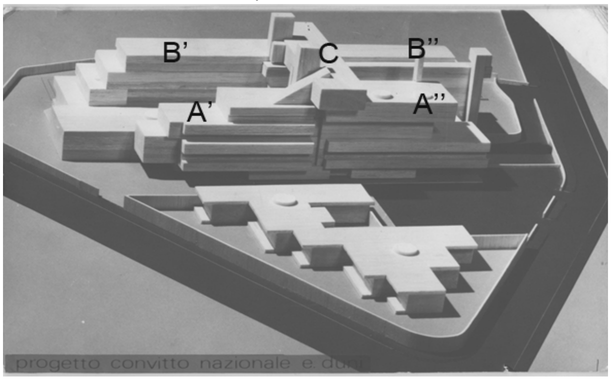
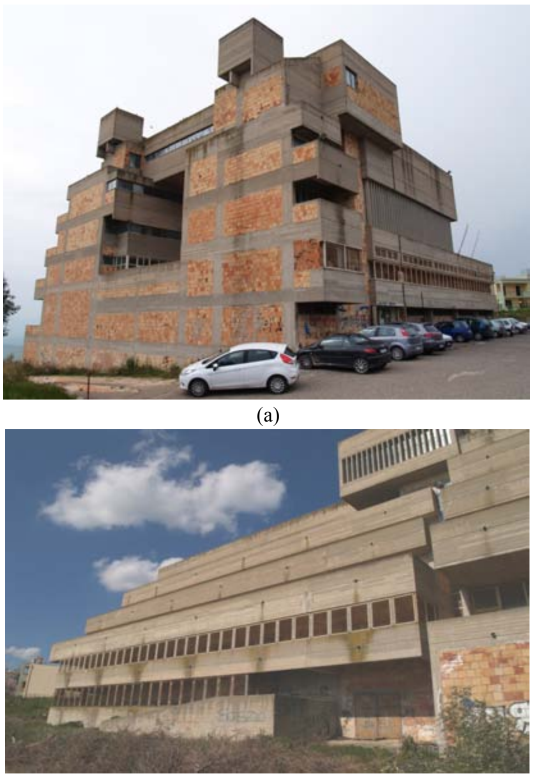


Fig. 1 Model of the originally designed building


(a)

(b)

Fig. 2 Views of the building: (a) East view; (b) South view

B. *Phases of knowledge and mechanical modeling*

Cognitive analyses have been carried out, with growing levels of knowledge, in order to settle a reliable structural model allowing to perform refined nonlinear dynamic analyses. An historical investigation has been carried out in order to recollect the project and the transformations based on the different uses. A geometric survey has been made, with the identification of cracks, deformations and damages due to weathering. A set of destructive and non-destructive analyses (coreboring, sclerometer tests) has been carried out to identify concrete strength and its state of preservation; executive structural design has been taken into account to create a detailed model.

After setting a complete acknowledgement of the building, a tridimensional structural model has been implemented by the software Seismostruct (Seismosoft); beams and columns have been represented through linear element, perfectly constrained at the basis; rigid diaphragms have been adopted at each floor level (Fig. 3).

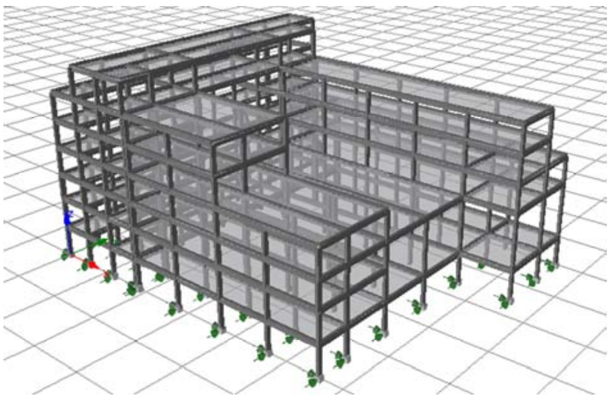
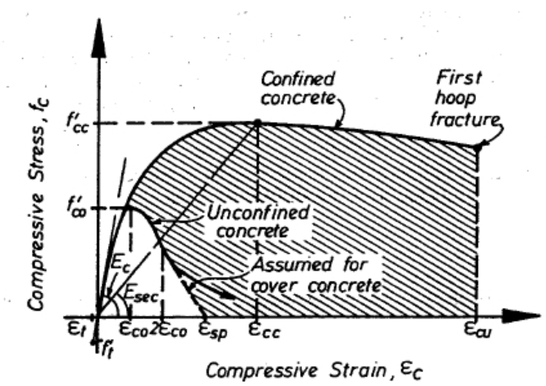
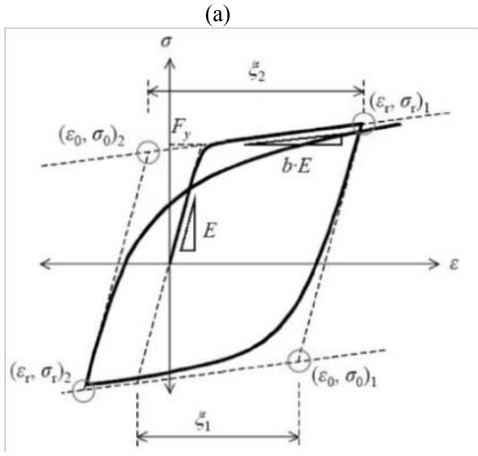


Fig. 3 Model of the building in Seismostruct



(a)



(b)

Fig. 4 Constitutive laws for confined concrete and steel bars: (a) Mander et al. law for confined concrete; (b) Menegotto – Pinto law for steel bars.

A fibre approach for RC frame analysis has been used, with the Mander, Priestly and Park constitutive law [23] for the confined concrete and the Menegotto – Pinto law [24, 25] for steel bars (Fig. 4(a)-(b)).

IV. Nonlinear analysis

Nonlinear analyses have been carried out on the selected building. For the IDA analysis the maximum base shear has been related to the peak roof drift, in order to make a direct comparison with the pushover analysis. The comparison has been carried out only in the *x*-direction (parallel with A and B buildings) because the participating mass in *y*-direction is 52.3%, so less than the minimum 75% recommended to run pushover analysis with the main force distribution proportional to the mass multiplied by the first mode shape.

Due to the high computational effort, ten artificial ground motion records, properly generated with the software REXEL (www.reluis.it) have been used and calibrated to fit the Eurocode 8 elastic response spectrum for ground type A with a PGA=0.11g (Figs. 5-6). The accelerograms have been scaled by a SF λ from 0.2 to 2, in a step-by-step procedure, in order to carry out IDA analyses. Since the Static Pushover (SPO) curve refers to base shear versus peak roof drift, they will be considered as IM and DM respectively.

For each analysis the minimum SF λ that causes the limit state of collapse has been determined and an IDA curve for each ground motion has been obtained by interpolating the points in correspondence of which IM e DM have been measured.

The mean IDA curve has been obtained from the ten single IDA and the maximum displacement has been assumed where at least six single IDAs are reached. (Fig. 7).

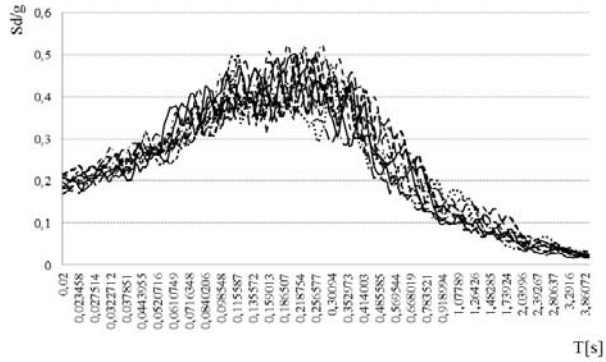


Fig. 5 Acceleration spectra for the selected accelerograms (5% damping) (REXEL).

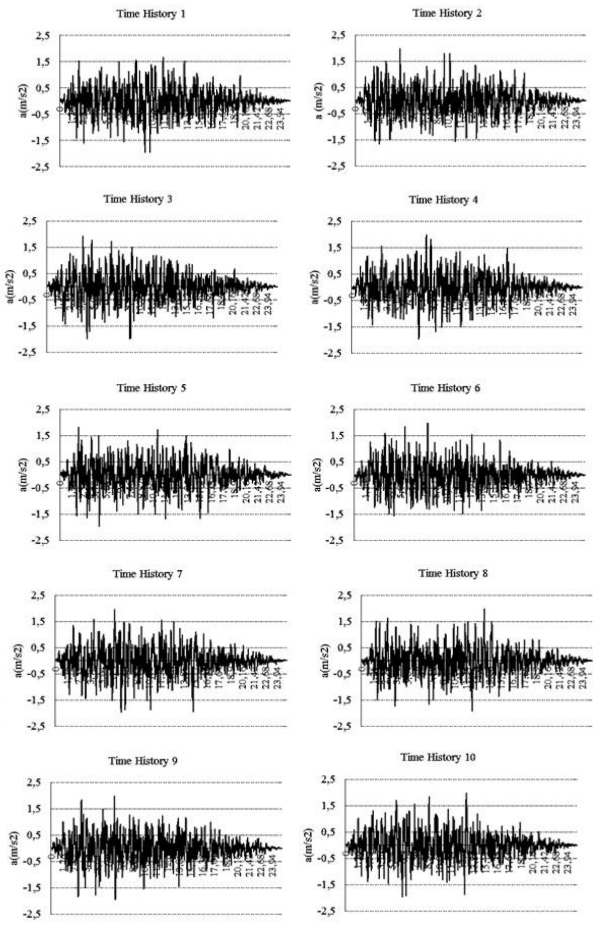


Fig. 6 Time histories.

The mean IDA has been compared with the pushover curves, obtained with two different lateral load patterns: one obtained according to the first modal shape and one obtained with an adaptive pushover [26], a more accurate and reliable method for irregular structures.

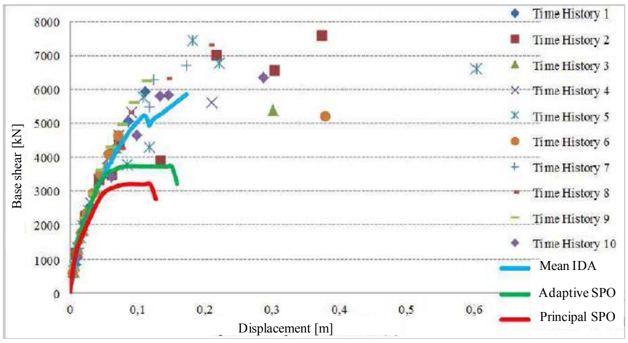


Fig. 7 Set of IDA curves and mean IDA.

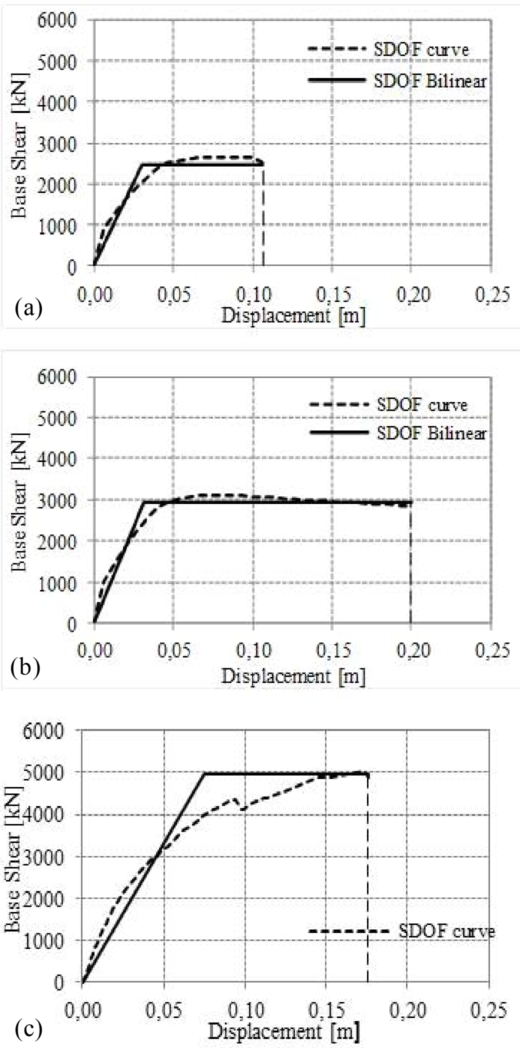
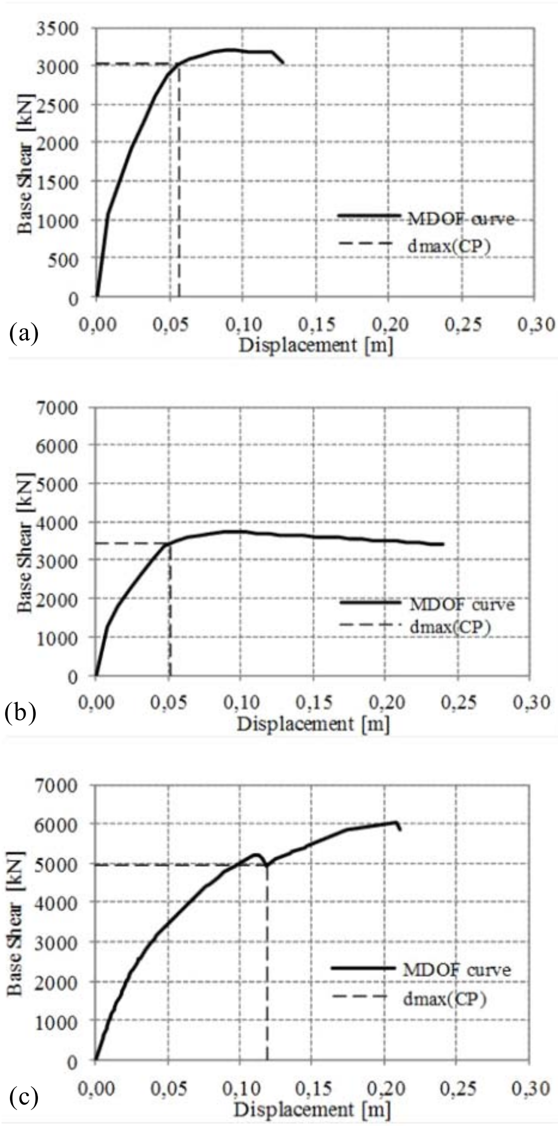


Fig. 8 SDOF systems: (a) Principal SPO; (b) Adaptive SPO; (c) IDA.

The results show that the static pushover gives more conservative results both in terms of displacements and base shear. For an immediate comparison each curve has been bi-linearized according to the extended N2 method [19-20] with reference to the SDOF system (Fig. 8). For each curve the passage to the MDOF has been obtained through the modal participation factor and the design displacement spectrum for the city of Matera, so deriving the corresponding demand of displacement (Fig. 9).



Fg. 9 MDOF systems with requested displacement: (a) Principal SPO; (b) Adaptive SPO; (c) IDA.

A. Results

For the principal pushover a requested displacement equal to 0.057 m has been obtained, for the adaptive pushover 0.052 m; the mean IDA led to a displacement equal to 0.119 m.

Based on these displacements, for each analysis the elements which collapse have been identified and quantified. For each element the compatibility of shear resistances with the limitations for fragile mechanisms and the compatibility of displacements with the limitation for ductile mechanisms have been verified. The main failure occurred in the columns of the second floor for shear fragility.

Although the requested displacement for the pushover analysis are smaller than those of the mean IDA, the number of the crashed elements is greater in the pushover analyses with 7 damaged columns for the principal pushover and 8 for the adaptive one; as to the IDA, only two among ten time-history analyses show respectively 5 and 4 damaged elements (Figs. 10-11). So the pushover analyses are very conservative and through the IDA the recovery intervention can be suitably reduced.

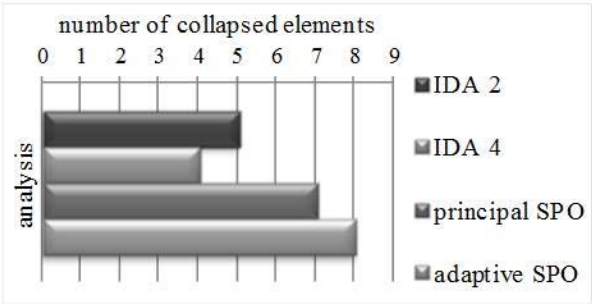
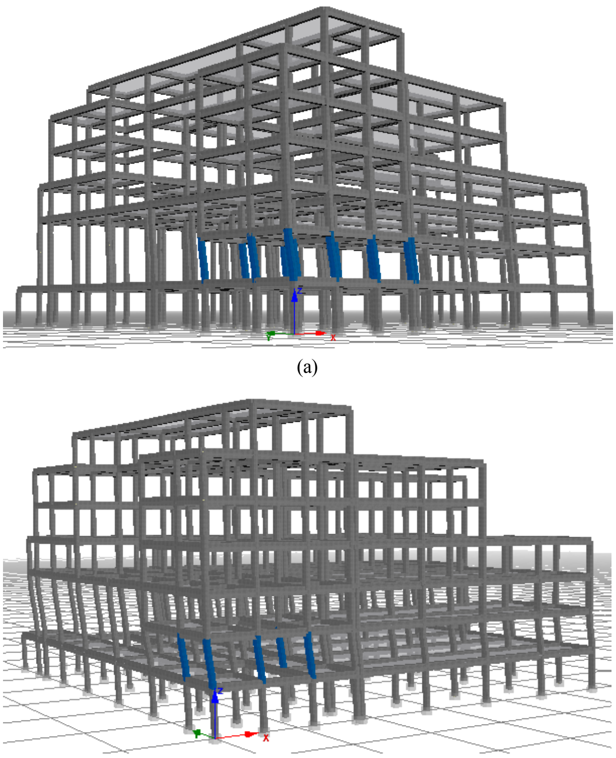


Fig. 10 Set of IDA curves and mean IDA.



(a)

(b)

Fig. 11 Results of the analysis: (a) collapsed elements in adaptive pushover; (b) collapsed elements in IDA 2.

V. Conclusion

This study shows the sensitivity of the analysis method in identifying the crashed elements that need seismic recovery interventions.

Static pushover analysis shows a good correlation with incremental dynamic analysis, but is obviously more conservative, especially for the limited capability of the fixed load distribution to predict higher mode effects in the post-elastic range when highly irregular buildings are concerned. Incremental dynamic analysis covers instead a wider range of structural response thanks to the different ground records having their own peculiarities. The simplified IDA that defines a single-degree of freedom system to approximate the static pushover curve (whose elastic stiffness is calculated from IDA curve) for a multi-degree-of freedom structure allows to reduce the computational effort.

The complexity and the extreme irregularity of the analyzed building shows how a dynamic incremental analysis can guarantee, with respect to the traditional nonlinear analysis, safety and a greater preservation of the building; this leads to the fulfillment of the minimum intervention criterion, particularly important in the case of historical buildings.

The reliability of an analysis is also related to the level of knowledge of the building. So destructive and/or non-destructive tests are recommended, in order to achieve a more realistic estimation of seismic vulnerability. As a consequence, a less conservative analysis such as incremental dynamic analysis requires a wider knowledge of the structure.

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