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LINEAR VS. NONLINEAR PROCEDURES FOR THE SEISMIC ASSESSMENT AND RETROFIT OF EXISTING RC BUILDINGS

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Abstract

In the context of seismic assessment and retrofit of existing reinforced concrete buildings, practitioners prefer to employ linear analysis and in particular the Linear Dynamic Procedure that uses the well-known Response Spectrum Analysis. This is attributed mainly to the fact that engineers are more familiar with this method, which is the one mostly employed in the design of new structures, but also because linear analysis is easier and faster to use. In the present work, it will be demonstrated that, despite these obvious advantages, the use of linear analysis in the assessment and strengthening of older RC structures is a very conservative practice that should be avoided, because it leads to a significant under-estimation of the member capacities. By means of examples and real case-studies that are analyzed with the full code-based seismic assessment methodologies, according to both ASCE 41 and Eurocode 8, it will be explained why the nonlinear methods, combined with a good knowledge of the structural configuration, can be very beneficial, lead to lighter interventions and prevent unnecessarily disruptive and costly works.

Keywords: Nonlinear Analysis, Linear Analysis, Pushover Analysis, Nonlinear Dynamic Analysis, Seismic Assessment, Seismic Retrofit

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1 INTRODUCTION

Practitioners generally prefer to employ linear analysis methods, which implicitly assume small deformations, limited and evenly distributed damage throughout the structural members, and an approximately elastic performance of all the structural components. This is attributed to the fact that engineers are more familiar with these methodologies that are similar to well-known procedures for the design of new building, but also because linear methods are easier and faster to use.

The approximations of elastic analysis do not constitute a significant problem in the design of new structures, since the engineers are able to choose the strength and stiffness characteristics of the structural components, in order to have a reasonable distribution of inelasticity, without large concentrations of deformations at particular, more vulnerable locations of the building. This, together with careful detailing of the members (e.g. closely spaced stirrups in RC members, or diagonal reinforcement, where needed) and the introduction of a uniform behavior factor (q-factor) that accounts for the inelastic response (implying that approximately inelasticity will be evenly distributed in the different locations of the structure) provides an efficient, and reasonably accurate framework for the design of new buildings with a high level of reliability.

On the contrary, this is rarely the case in the context of the seismic assessment and retrofit of existing structures. Older buildings have been designed and constructed before the introduction of the early earthquake resistance codes, without special considerations to withstand seismic actions in a manner similar to today’s practice. As a result, very frequently they exhibit irregular arrangement of their structural members, with uneven distribution of the strength, stiffness and mass, which adversely affects its behavior under earthquake loading (e.g. irregularities in plan or elevation, soft ground stories, short columns, coupling beams between large shear walls, indirect supports on beams etc.).

Because of this behavior, the use of elastic procedures for the analysis of existing buildings may lead to serious inaccuracies in the estimation of the force and the deformation demand on the structural components. What is more, in the majority of cases this approximation leads to the underestimation of the displacement demand of the members in the locations with concentrations of inelastic deformations that are the most vulnerable under seismic loading. In order to overcome all these problems, all the standards for structural assessment have proposed larger safety factors and procedures that are quite conservative.

The present paper attempts to carry out a comparative study on the use of the different linear and nonlinear procedures within the framework of the main methodologies for structural assessment and retrofit of existing buildings. Two case studies will be presented and analyzed, according to both the American [1, 2, 3, 4] and the European [5, 6] standards, and the results from the linear and the nonlinear procedures will be compared. It will be shown that the former provide very conservative estimates of the seismic capacity, and lead to interventions that are extremely expensive, and works that impose significant restrictions in the operation of the building. Similar conclusions have also been drawn by other recent studies [7, 8, 9].

2 LINEAR AND NONLINEAR METHODS OF ANALYSIS

Within the context of all modern codes for structural assessment and retrofit, four different analytical methods are proposed with small variations between the different standards:

- The *Linear Static Procedure* LSP, a static type of analysis with no variable load.
- The *Linear Dynamic Procedure* LDP, which is essentially the Response Spectrum Method (RSA). The linear dynamic procedure is the method of analysis that is typically em-

ployed for the design of new structures; thus, it is the method of analysis that engineers are more familiar with.

- The *Nonlinear Static Procedure* NSP, which is the well-know pushover analysis, either in conventional or adaptive mode.
- The *Nonlinear Dynamic Procedure* NDP, which is the nonlinear dynamic time-history analysis.

The names of the methods above are the ones that are used by ASCE-41 and the different American guidelines. The corresponding names in the Eurocodes are: (i) the lateral force analysis, (ii) the modal response spectrum analysis, (iii) the non-linear static (pushover) analysis and (iv) the non-linear time-history dynamic analysis. Within the context of this paper the American nomenclature will be adopted, since it is more consistent and intuitive.

2.1 Linear methods of analysis

In both linear analytical methods, the term *linear* implies a linearly elastic member behavior. The basic rules of elasticity apply, the stiffness distribution throughout the structure remains unchanged, and an unrealistic, linear force vs. displacement curve is obtained with the load increase.

A lateral, pseudo-seismic force distribution that is assumed to approximate the earthquake loading is applied to the elastic structural model, in order to calculate the internal forces and the system displacements. In the Linear Static Procedure the lateral load profile is an inverted triangle. The Linear Dynamic Procedure instead is somehow more sophisticated, since the profile of the lateral forces is not arbitrary anymore, but rather it is calculated as a combination of the modal contributions of the different modes of vibration of the structure. The derived action effects are then compared with the members' capacities for the selected performance level, always in terms of forces, and, if the capacities are larger than the demands, the structure is considered safe.

Because it is assumed that during a large seismic event the structure will sustain damage, a reduced ‘cracked’ stiffness is employed for the analysis (in a very coarse - almost crude- manner), in order to account for the reduction of stiffness due to material inelasticity. The cracked stiffness is given as a fraction of the uncracked stiffness using factors that are given in the different standards through tables.

Because of their approximate nature, the use of the linear procedures is permitted only in cases of very regular constructions that sustain limited damage and do not undergo large inelastic deformations. More specifically, the demand-to-capacity ratios should be relatively small for all structural components, and below unity for all the brittle failure types. Furthermore, there should be no strength or stiffness discontinuities or irregularities (in plan, in elevation or torsional) in the structural configuration.

Unfortunately, this is rarely the case with older construction. Existing RC buildings have not been designed to withstand seismic actions, and usually experience force concentrations, significant damage at specific, more vulnerable locations, large structural displacements beyond the domain of geometric linearity, as well as material deformations that exceed the elastic threshold.

2.2 Nonlinear static (pushover) analysis

According to ASCE 41-23, Section 7.4.3.1, the definition of the Nonlinear Static Procedure (NSP) or pushover analysis is the following: “A mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components of the building shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement is exceeded.“ The structural behavior is no longer linear, and the analysis accounts for the geometrical and material nonlinearities, as well as the redistribution of internal forces due to the sustained structural damage. Stresses are not proportional to strains, forces are not proportional to displacements and bending moments are not proportional to curvatures.

In pushover analysis, a structural model that consists of nonlinear members is loaded with a predefined lateral load profile with a gradually increasing loading factor λ until the displacement of a selected ‘Control Node’, typically located at the center of mass of the top storey of the building, reaches the so-called ‘Target Displacement’. The Target Displacement represents an approximation of the maximum displacement demand under the selected level of earthquake ground motion. The lateral force profile is expected to approximate the earthquake loading, and several different types of force distributions can be employed, triangular, uniform, modal or even adaptive distributions that change from step to step.

During the analysis the sequence of the plastic-hinge formation, the members’ failures, and the change in the loading paths and the redistribution of forces are identified. Since the stiffness is no longer constant, but rather it is updated at every step, the structure gradually softens as plastic hinges develop at the locations of structural damage. As a result, the force vs. deformation curve, which is the so called capacity curve, is not linear any more, but has a characteristic parabolic shape as the structural deformations increase disproportionally with the level of lateral loading. In other words for the same level of load increase, the increase of the deformations get larger as we push further on in the inelastic range.

Pushover analysis provides crucial information on response parameters that cannot be obtained with conventional elastic methods (either static or dynamic). The response characteristics that can be obtained with pushover analysis include (i) the realistic force demands on potentially brittle elements, such as axial demands on columns, moment demands on beam-to-column connections or shear force demands on short, shear-dominated elements, (ii) estimates of the deformation demands of elements that have to deform inelastically, in order to dissipate energy, (iii) consequences of the strength deterioration of particular elements on the overall structural stability, (iv) identification of the critical regions, where the inelastic deformations are expected to be high, (v) identification of strength irregularities in plan or elevation that cause changes in the dynamic characteristics in the inelastic range, (vi) estimates of the interstorey drifts, accounting for strength and stiffness discontinuities, (vii) sequence of the member’s yielding and failure and the progress of the overall capacity curve of the structure, and (viii) verification of the adequacy of the load path, considering all the elements of the system, both structural and non-structural.

Compared to the elastic procedures, pushover analysis treats inelasticity in a more explicit manner and being more ‘displacement-based’ it is more suitable for performance-based engineering. Of course, these benefits come with the additional cost of having to model accurately the inelastic load-deformation characteristics of both structural and non-structural members, as well as increased computational effort. However, since the recent computer developments have now rendered it possible to undertake nonlinear analyses in much faster times than in the past, pushover seems to be much better suited for the evaluation of the structural behavior of the existing RC buildings.

2.3 Nonlinear dynamic analysis

According to ASCE 41-23, Section 7.4.4.1, the definition of the Nonlinear Dynamic Procedure NDP, or nonlinear dynamic time-history analysis is the following: “A mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components of the building shall be subjected to earthquake shaking represented by ground motion acceleration histories to obtain forces and displacements". The objective of the method is to assess the capacity of the structure, considering the deformability, the strength and the hysteretic behavior of all structural members that are subjected to the specified earthquake ground motion.

The Nonlinear Dynamic Procedure constitutes a sophisticated approach for examining the inelastic demands produced on a structure by a specific seismic loading. The basis, the modelling approaches, and the acceptance criteria of the NDP are similar to those for the NSP. One additional complication with respect to the NSP is that now the monotonic force-displacement curves are not sufficient for structural modelling, and the full hysteretic, loading & unloading rules need to be introduced for all the structural members (or at least those that we expect to behave inelastically). These rules should realistically reflect the hysteretic energy dissipation in the elements over the range of displacement amplitudes expected in the seismic design situation. Furthermore, the mass distribution and the equivalent viscous damping of the structure should be defined, in order to correctly model the inertia forces that are introduced in the structure from the dynamic vibrations.

Regarding the modelling of the seismic action, instead of the lateral force distributions that are used in the linear procedures and pushover analysis, acceleration time-histories are now applied at the foundation level of the building. These accelerograms can be real recorded seismic actions, or artificial and synthetic accelerograms that match a given target (usually code-defined) spectrum.

2.4 Applicability of the different analytical methods

Linear Static Procedure: The LSP is the most basic method of the four with many approximations and very limited accuracy, even for relatively simple structural configurations. It is only allowed for small symmetric buildings, and it is employed in a conservative manner with large safety factors. In general, it should be avoided for everyday application, but for the most simple and regular buildings.

Linear Dynamic Procedure: Because the loading is calculated through the combinations of several modes (including higher ones), the LDP is suitable for tall and asymmetric buildings, where higher mode effects are of importance. However, as an elastic method, it inherently has limited accuracy in the case of large inelastic deformations, which are very common in existing buildings under large earthquake loading. Hence, the results can be very inaccurate when applied to buildings with highly irregular structural systems, unless the building is capable of responding almost elastically at the selected Seismic Hazard Level. Similarly to the LSP, it is employed conservatively with higher safety margins, in comparison with the nonlinear methods.

Nonlinear Static Procedure: Due to the explicit modelling of inelasticity, the NSP is more suitable when large inelastic deformations are expected. In such cases, the structural response can be modelled with satisfactory accuracy, allowing for a less conservative approach. The NSP is generally a more reliable approach for characterizing the performance of a structure than the linear procedures. However, it cannot accurately account for changes in dynamic response as the structure degrades, and it is not suitable, when higher-mode effects are of importance, e.g. with taller buildings (more than 10-15 floors). In general, the NSP is a valid approach for the seismic assessment of existing buildings; however, it should be used with caution, when the structural response is determined by more than one modes.

Nonlinear Dynamic Procedure: The nonlinear dynamic time-history analysis involves fewer assumptions than the nonlinear static procedure, and it is the most sophisticated method for structural analysis. It is more accurate than the NSP, and it is subject to fewer limitations regarding the load and the structural configuration. The NDP is able to model both the inelastic material behavior and higher mode effects for a given earthquake record. It directly provides the maximum global displacement demand produced by the earthquake on the structure, eliminating the need for approximations, and it is generally suitable for any structural configuration and any earthquake loading. However, the main disadvantage of the method is a significant one: it is difficult to use, and specialized knowledge is often required, e.g. for the selection of suitable accelerograms, or the interpretation of results.

3 CODE-BASED CHECKS AND ACCEPTANCE CRITERIA

3.1 General

As in the standard design methodologies, the requirement of the capacity checks is that the component strength is larger than the demand on the component, i.e.:

$Q_C \geq Q_U$ for ASCE 41, or $R_d \geq E_d$ for Eurocode 8

Q_U or E_d is the design value of the action effect for the seismic design situation for the selected hazard level. Q_C or R_d is the corresponding resistance of the element, considering specific material parameters (e.g. lower-bound, nominal or mean value of the strength), based on the type of analysis (linear or nonlinear), the type of the action (ductile or brittle) and the selected performance level.

A basic distinction is done in both standards between the *deformation-controlled*, ductile actions (e.g. bending in a member without significant axial loads) and the *force-controlled*, brittle actions (e.g. shear), and different approaches are followed for the capacity checks in the two cases, as will be described below.

3.2 Capacity checks for linear methods

When using the linear methods of analysis, the checks are always performed in terms of forces for both the deformation and the force-controlled actions. For the ductile mechanisms of failure, inelasticity is taken into account in the two codes in a similar, but not exactly the same way. In ASCE 41, inelasticity is taken into account by the so-called m-factors, whereas in Eurocode 8 it is considered with the selected behavior factor q. The philosophy of both factors is the same, i.e. to account for the capability of ductile members to deform beyond their yield point. However, there are two main differences between the q and the m-factors. The most important difference is that, whereas the m-factors are member specific (i.e. different m-factors may be assigned to the different structural components), the q-factor is based on the entire capability of the building to absorb energy. Secondly, the m-factors operate on the capacity side of the inequality, effectively increasing the strengths, whereas the q-factor is employed to decrease the demand on the components (both factors assume values equal or larger to unity).

For the deformation-controlled actions the component demand is calculated from the set of linear analyses. For the force-controlled actions the component demand is calculated based on capacity design considerations (i.e. estimate of the maximum action that can be developed in a component, based on a limit-state analysis), taking into account the expected strength of the components that deliver forces to the component under consideration, in order to make sure that a failure in the force-controlled action is avoided.

For the deformation-controlled actions the capacity of the components shall be based on the expected strengths. On the contrary, for the force-controlled actions the capacities shall be based on lower-bound strengths.

3.3 Capacity checks for nonlinear methods

For the deformation-controlled actions the quantities checked are deformations (rather than forces), as these are calculated from the nonlinear analysis. For the force-controlled actions, the component demands are again forces, but now they are calculated directly from the nonlinear analysis. Note that the demands are not to be determined from capacity design considerations as in the linear case, since inelasticity is explicitly accounted for by the nonlinear analysis method, and the capacity design concept is no longer needed.

For the deformation-controlled actions the component capacities are taken as permissible inelastic deformation limits that are determined considering all coexisting forces and deformations at the target displacement. For the force-controlled actions, the component capacities are taken as lower-bound strengths that are determined considering all coexisting forces and deformations at the target displacement. Contrary to the deformation-controlled actions, the checks for the force-controlled actions are performed again in terms of forces.

4 APPLICATION EXAMPLES

In order to investigate the effect that the method of analysis has on the outcome of the structural assessment and retrofit methodologies, two application examples will be examined. The first example pertains to the assessment of an industrial building with short columns, whereas the second example is a real case study of the strengthening of a small residential building with a soft ground floor that has been severely damaged from two consecutive earthquakes. ASCE 41 [1] and ACI 369.1 [2] will be employed in the first case, while the second investigation will be carried out with the provisions of Eurocode 8 Part-3 [5].

All the analyses and all the checks have been carried out with SeismoBuild [10], a finite element package dedicated to the seismic assessment and strengthening of reinforced concrete buildings, which is capable of performing linear and nonlinear, static and dynamic analysis.

5 ASSESSMENT OF AN INDUSTRIAL BUILDING WITH SHORT COLUMNS

In the current section the structural assessment and strengthening of an industrial building with short columns will be presented [11]. The building is a typical design of the late 1980s and it consists of two rectangular floors of approximately 880m^2 each (Figure 1).

The concrete grade is C16/20 (f_{ck}=16MPa, $f_{c,mean}$=24MPa), the steel grade is S400 (f_{sk}=400MPa, $f_{s,mean}$=444MPa) for longitudinal bars and S220 (f_{sk}=220MPa, $f_{s,mean}$=244MPa) for the stirrups. There is adequate longitudinal reinforcement (for instance a typical rectangular column has 8∅20 of rebars), however the shear reinforcement is just ∅8/15 for the beams and ∅8/30 for the columns.

The infill walls have been constructed in the perimeter of the building up to a certain height, in order to create openings and provide light in the interior, forming a series of vulnerable, short columns in the entire perimeter. They are relatively strong with good quality ceramic bricks and mortar of relatively high strength. The combination of strong infills and short columns is the most important characteristic of the building and constitutes a serious structural problem related to its seismic behavior, as will be shown in the next sections.

Figure 1: 3D rendering of the industrial building (front view and side view)

The building will be analyzed according to ASCE 41 for a single Performance Objective ‘g’ (see Table C2-8 in ASCE 41-23), which combines the (3-C) Performance Level for Life Safety, and the BSE-1E Seismic Hazard Level with a 20% probability of exceedance in 50 years.

Code-based checks in shear and bending for all the members were carried out. It is noted that bending is a deformation-controlled (ductile) failure mechanism, for which reason the checks were performed in terms of forces (bending moments) for the linear analyses, but in terms of the plastic hinge rotation in the nonlinear methods. All the checks are expressed in terms of the demand-to-capacity ratio (DCR) for every member. The DCR is the proportion, by which the demand is larger than the capacity; if DCR<1 the member is safe, if instead DCR>1 the member fails.

5.1 Capacity checks with the Nonlinear Static Procedure, NSP

With the static nonlinear (pushover) analysis, the checks have been carried out for both shear and plastic hinge rotation for all the members. The DCRs are calculated as the envelope of all the results that are extracted from the executed pushover analyses. No failures have been observed for the checks in plastic hinge rotation, and the maximum DCR ratios was 0.536 (Figure 2).

On the contrary several failures were observed in the shear checks. As expected, almost all the failures were located at the short columns in the perimeter, confirming the fact that lightly reinforced short columns are indeed an element of increased vulnerability in existing buildings. The maximum demand-to-capacity ratio is 1.431, but all the short columns fail under the prescribed seismic loading; DCRs range from 1.236 to 1.431 in the ground floor, and from 1.040 to 1.274 in the second floor (Figure 3). In total 44 members have failed.

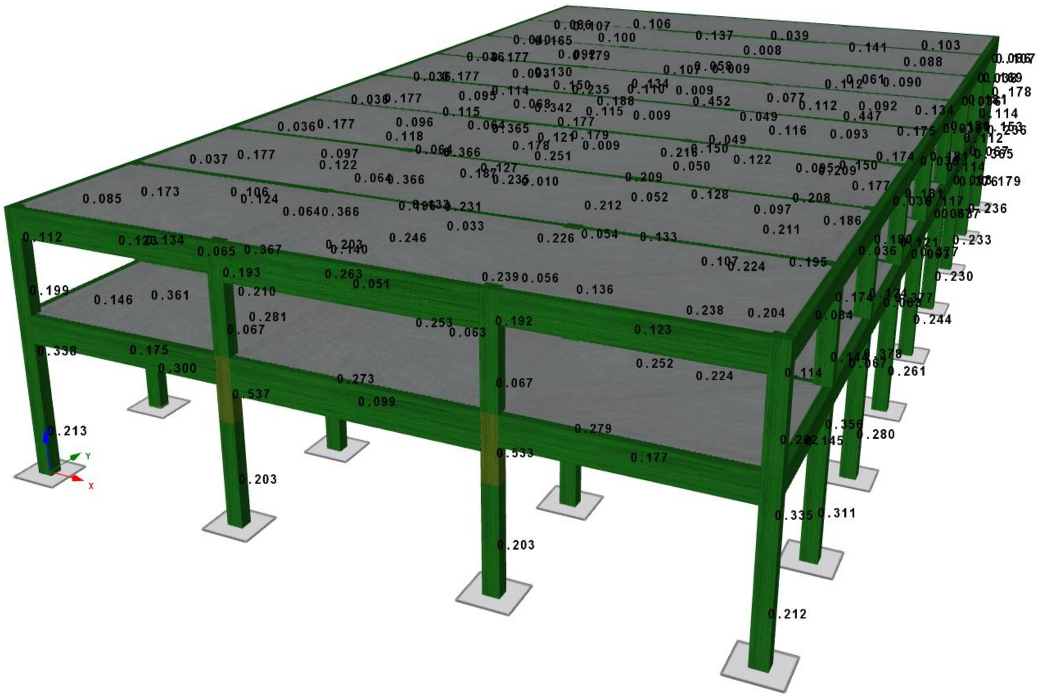

Figure 2: Plastic hinge rotation checks with the Nonlinear Static Procedure, NSP

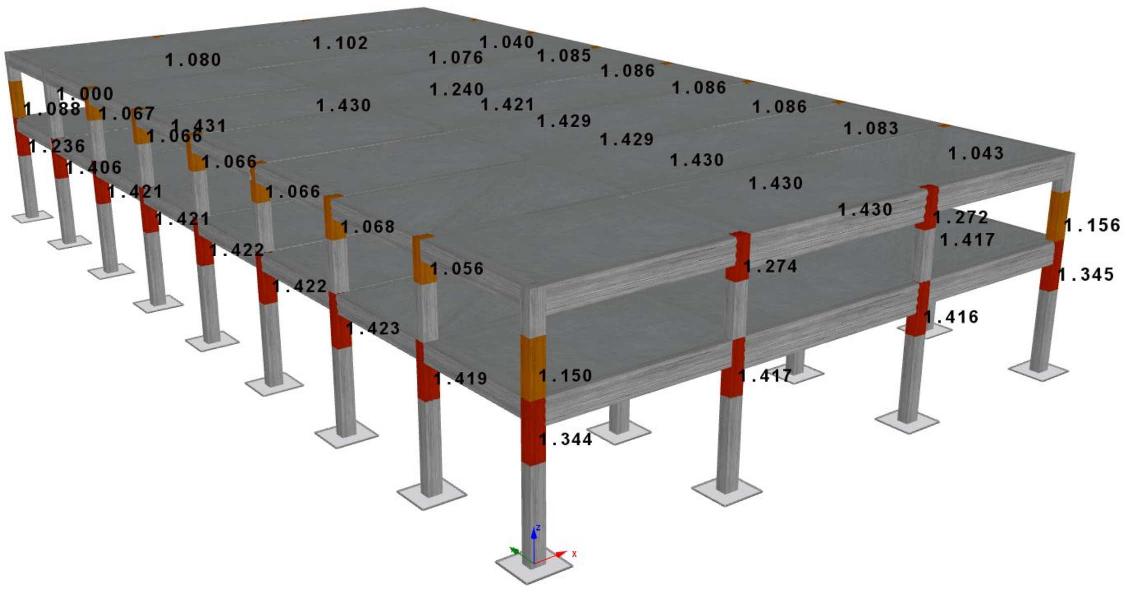

Figure 3: Shear checks with the Nonlinear Static Procedure, NSP

5.2 Capacity checks with the Linear Dynamic Procedure, LDP

Between the two linear methods of analysis, the Linear Dynamic Procedure LDP that makes use of the well-known Response Spectrum Analysis (RSA) is more accurate in the calculation of the seismic demand on the structural members, and it was chosen in the current investigation. The 8 linear dynamic analyses required by ASCE-41 ran faster than the 8 push-over analyses (3 seconds, instead of 27 seconds in an average Intel i7 processor – both sets of analyses were executed using the advanced parallel computational capabilities of SeismoBuild). However, the shear demand with LDP on the short columns is considerably higher, as shown in Figure 4 (up to 664 kN with LDP, with respect to a maximum of 285 kN with

NSP). It is noted that shear is a brittle type of failure and the demand is calculated with capacity design considerations. taking into account the expected strength in bending of the components that deliver forces to the component. This is done, in order to make sure that a failure in the force-controlled action is avoided, since the results from the elastic analyses are expected to be quite inaccurate.

The higher demand is obviously reflected on the results of the capacity checks in shear. The maximum demand-to-capacity ratio in shear is now 5.17, whereas almost all structural members fail (Figure 4). It is noted that the increased DCR ratios are attributed to the increased calculated demand, but also to higher safety factors that are generally employed in the less accurate, linear methods of analysis. In total 112 members have failed, all 72 columns and 40 of the beams.

Bending is a ductile action, and whereas in the nonlinear methods the checks are carried out in terms of deformations, in the linear methods they are carried out in terms of bending moments, which is usually more conservative. This is highlighted in Figure 5 that shows the bending checks with the linear dynamic procedure. Whereas with pushover analysis, no failures are observed (the maximum DCR is equal to 0.536), with the linear dynamic procedure 59 failures occur and the maximum DCR is equal to 2.678.

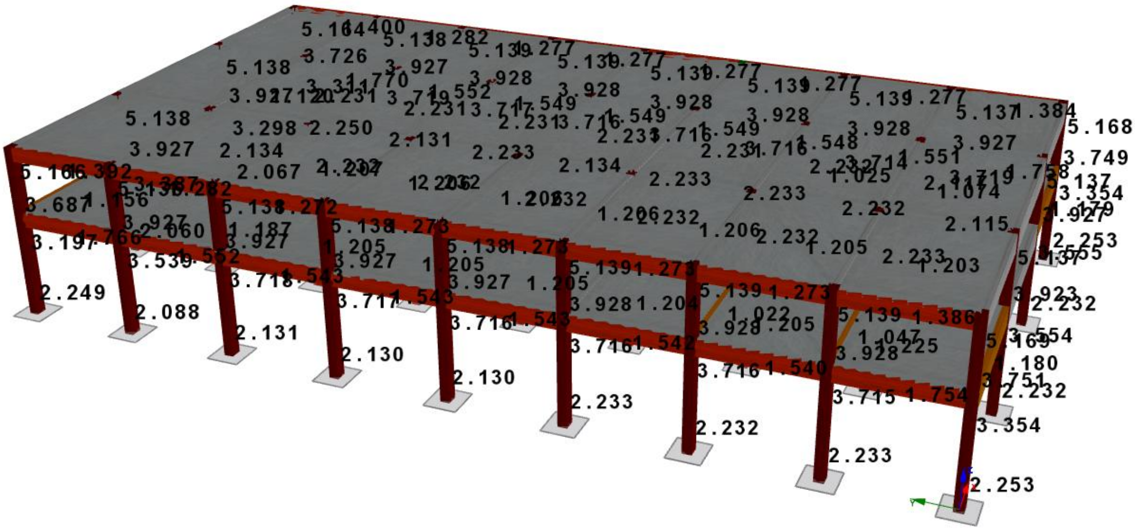

Figure 4: Shear checks with the Linear Dynamic Procedure, LDP

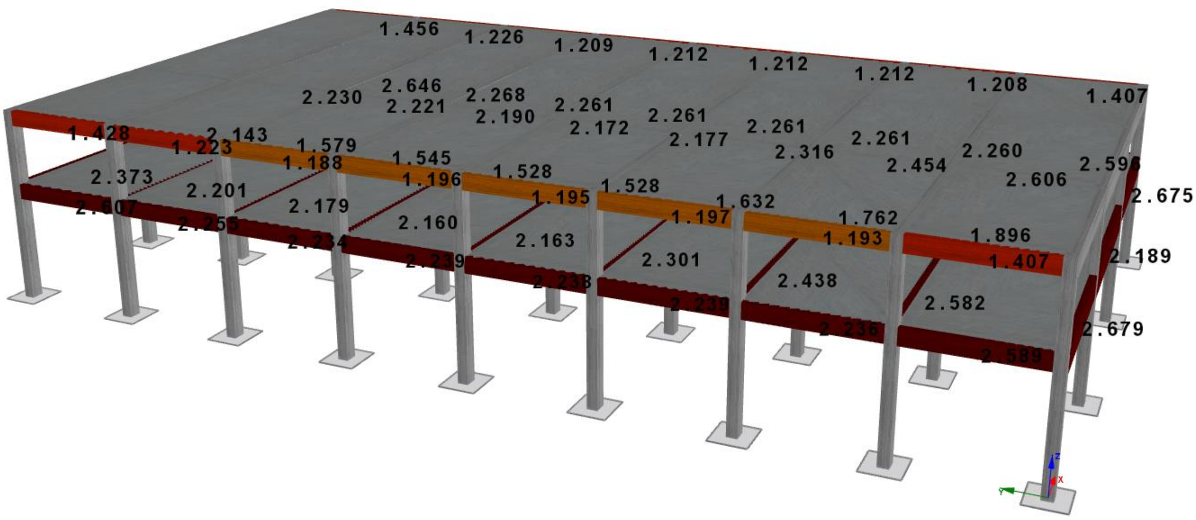

Figure 5: Bending checks with the Linear Dynamic Procedure, LDP

5.3 Capacity checks with the Nonlinear Dynamic Procedure, NDP

As explained in the previous section, the nonlinear dynamic time-history analysis involves fewer assumptions than the nonlinear static procedure, thus it is more accurate and it is subject to fewer limitations. However, this increased accuracy and flexibility does not come without a cost. The Nonlinear Dynamic Procedure is very expensive in terms of computational resources (NDP: 7 minutes and 18 seconds for 11 dynamic analyses. NSP: 27 seconds for 8 pushover analyses with the same Intel i7 processor). One additional complication with the NDP is the selection or generation and the scaling or matching of the accelerograms, with which the dynamic analyses are run. Note however that in the case of SeismoBuild this process does not pose a significant challenge, since SeismoBuild utilizes several SeismoArtif [12] algorithms for the automatic creation of artificial accelerograms that match the acceleration spectrum for the different seismic hazard levels. For every dynamic analysis (according to Chapter 16 of ASCE 7 [4] at least 11 analyses are required), a pair of accelerograms is automatically created for the X and Y directions.

The additional computational time is compensated with more accurate, and less conservative results. Indeed, if one gets the envelope for all the nonlinear dynamic analyses and carries out the capacity checks, it can be observed that the maximum demand-to-capacity ratio is now 1.309, which is 9% less than the 1.431 value of the maximum DCR in the NSP. Moreover, only 20 members have failed, as opposed to the 44 members that failed with pushover analysis (Figure 6). The additional computational cost pays off with more accurate and less conservative results, and consequently lighter structural interventions for the seismic strengthening.

Regarding the bending checks, again no failures have been observed in plastic hinge rotation, as in the case of the nonlinear static procedure. However, the DCR ratios are now decreased; 0.326 in the NDP as opposed to 0.536 in the NSP.

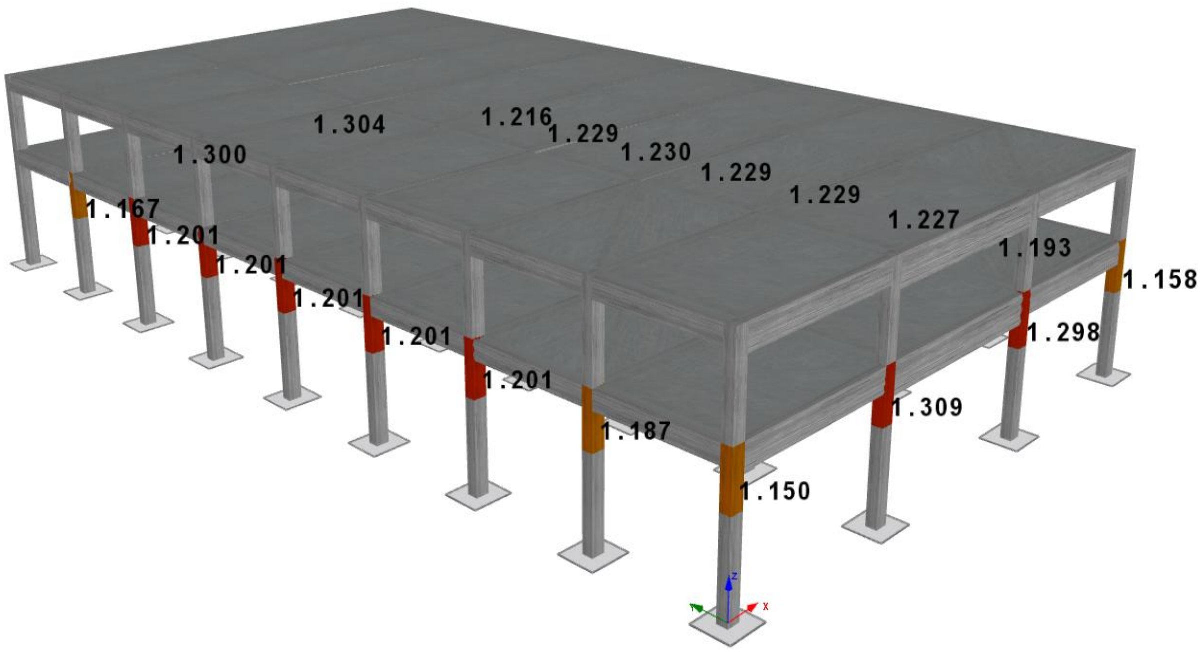

Figure 6: Shear checks with the Nonlinear Dynamic Procedure, NDP

6 ASSESSMENT AND STRENGTHENING OF REAL 2-STORY BUILDING

In the current section the structural assessment and strengthening of a 2-story residential building is presented [13]. The building is relatively small with an approximate area of $200m^2$ per floor (Figure 7). The ground floor was constructed in 1967 and the second floor in 1980. It had a soft ground floor with extremely weak columns, some of which were confined by infilled walls forming also short columns. All the ground floor columns (18 in total) were damaged during the two 2014 Kefalonia earthquakes in Greece [14], of magnitudes 6.1 and 6.0. Four of the columns were very severely damaged with the complete deterioration of the concrete, the fracture of hoops and the local buckling of the longitudinal reinforcement (Figure 8). It is noteworthy that the damage has been concentrated at the columns of the ground level only. The beams at the ground level, the strip footings at the foundation and the entire second floor (including the non-structural components) were intact.

The concrete grade was found approximately equal to C16/20 (f_{ck}=16MPa, $f_{c,mean}$=24MPa), and the steel grade was S220 (f_{sk}=240MPa, $f_{s,mean}$=280MPa) for both the longitudinal and the transverse reinforcement. The members, especially the columns, were lightly reinforced with ∅14 or ∅16 longitudinal rebars and∅6/30 to ∅6/50 hoops in the columns and ∅8/25 hoops in the beams. It is noteworthy that the shear reinforcement is considerably larger in the beams, rather than the columns, an indication that the building had been designed for gravity loads only.

Infilled walls that affect the structural stiffness were present at the second floor, but also in some locations at the ground level. These infills at the ground floor had been constructed in the perimeter and up to a certain height in order to retain the soil from the inclined landscape, and formed short columns in several locations. The combination of strong infills at the second floor and the short columns caused by the geometric constraints at the ground floor resulted in a very vulnerable structure, which was seriously affected by the 2014 earthquakes.

The building was analyzed according to the provisions of Eurocode 8, Part-3 for the Limit State of Significant Damage SD. The peak ground acceleration of the region is 0.36g and the analyses were carried out with a 10% probability of exceedance in 50 years, i.e. a return period of 475 years.

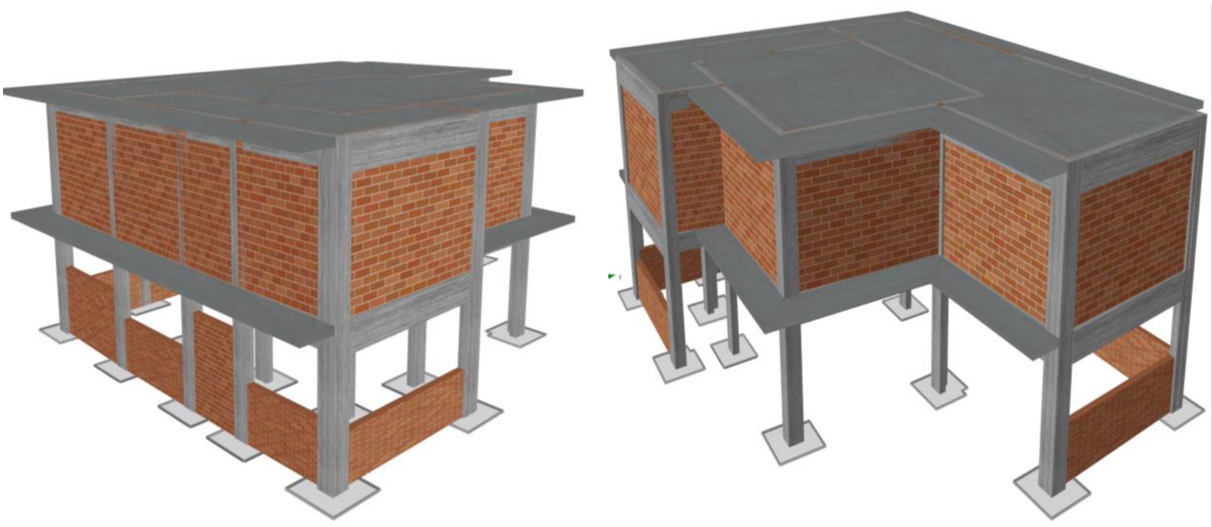

Figure 7: 3D rendering of the 2-story building (front view and back view)

Figure 8: Damage sustained by the 2-story residential building

6.1 Capacity checks for the existing building

The checks have been carried out in shear and chord rotation for all the members. The values of the demand-to-capacity (DCR) ratios for the checks in shear and bending with the Nonlinear Static Procedure are displayed in Figure 9 and Figure 10, respectively. The maximum DCR ratios are 3.500 in shear and 2.803 for chord rotation. This is in accordance with the on-site, post-earthquake observations and indicates that the building would require strengthening even it was not damaged from the seismic events.

It is noteworthy that the demand exceeds the capacity in all 18 columns of the ground level in shear, and in 11 out of the 18 ground floor columns in chord rotation, confirming again the increased vulnerability of soft ground stories and short columns in older, lightly reinforced buildings. What is also very interesting is the fact that the nonlinear static analysis of SeismoBuild managed to identify correctly the locations where increased damage occurred, as depicted in Figure 11.

As expected, with the Nonlinear Dynamic Procedure the demand-to-capacity ratios are generally smaller (maximum DCR equal to 2.393 in shear and 0.659 in bending). What is more noteworthy however is that the DCR values fitted the locations and the extent of damage even better than the NSP (Figure 12), another indication that the NDP is the most accurate method for structural analysis.

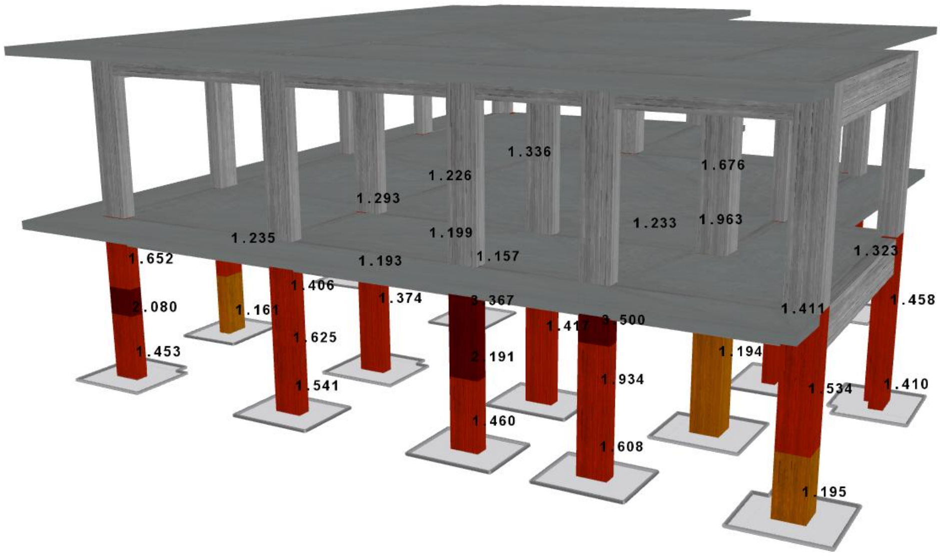

Figure 9: Shear checks with the Nonlinear Static Procedure, NSP

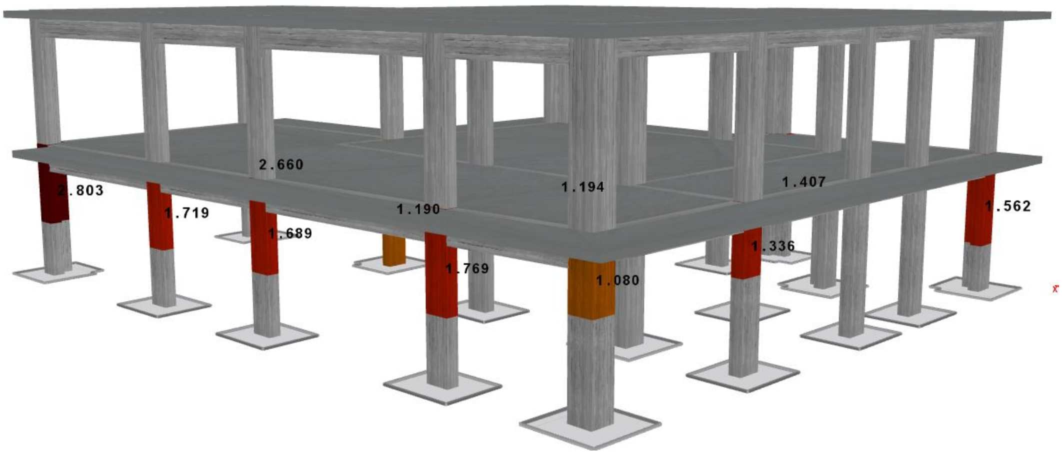

Figure 10: Chord rotation checks with the Nonlinear static procedure, NSP

On the contrary, with the Linear Dynamic Procedure the demand-to-capacity ratios are very high with values up to 6.720 in shear and 11.031 in bending. In total 43 out of the 80 structural elements have failed (Figure 13). What is more important is that now the failures are not confined to the ground floor columns; in several beams of the first floor and columns of the second floor the demand exceeds the capacity. This differs from what has been observed from the post-earthquake survey, and it is yet another indication that the linear methods of analysis are very conservative and they are not suitable for older, weak buildings that are expected to exhibit a strongly inelastic behavior during a large seismic event.

6.2 Seismic retrofit with reinforced concrete jackets

Because all the columns at the ground level were severely damaged, the construction of strong reinforced concrete jackets was actually the only technically acceptable solution for the strengthening of the building. The retrofit scheme consisted of the strengthening of all the columns at the soft story with a 10 cm wide reinforced concrete jackets, while the other structural members (members on the 2nd floor, beams, foundation system) were left unstrengthened.

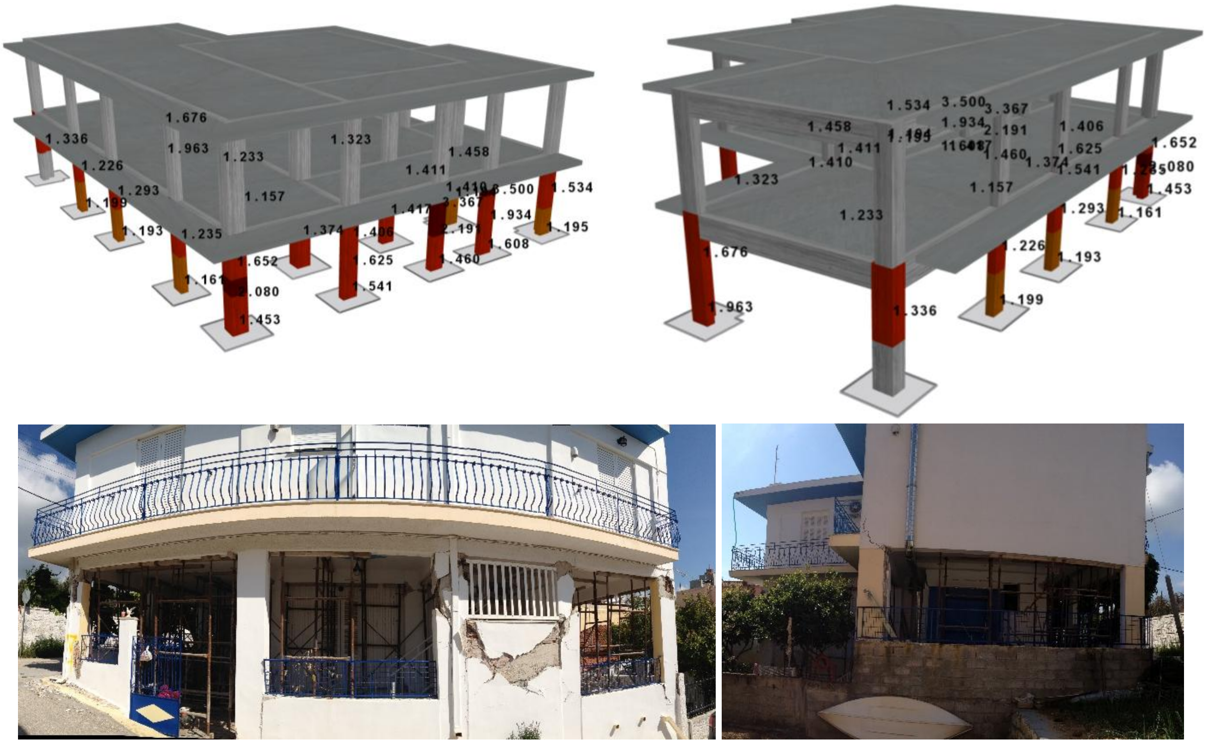

Figure 11: Comparison between the shear checks with the Nonlinear Static Procedure NSP in SeismoBuild and the actual damage sustained by the building in the 2014 Kefalonia earthquakes

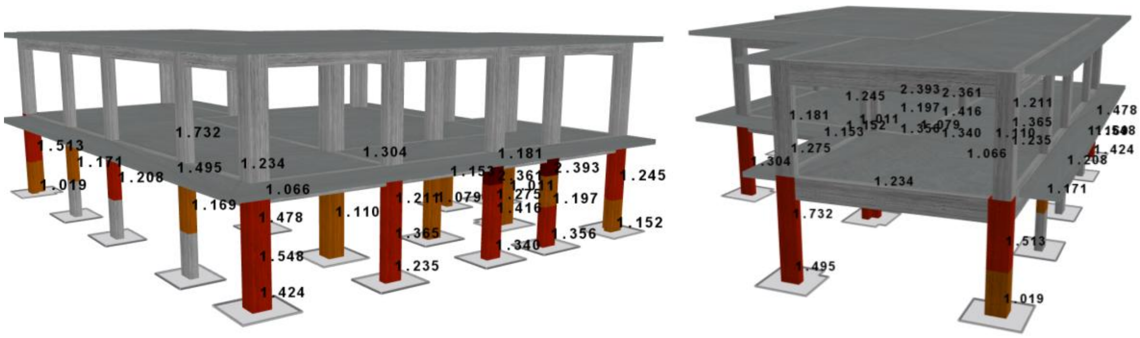

Figure 12: Shear checks with the Nonlinear Dynamic Procedure, NDP

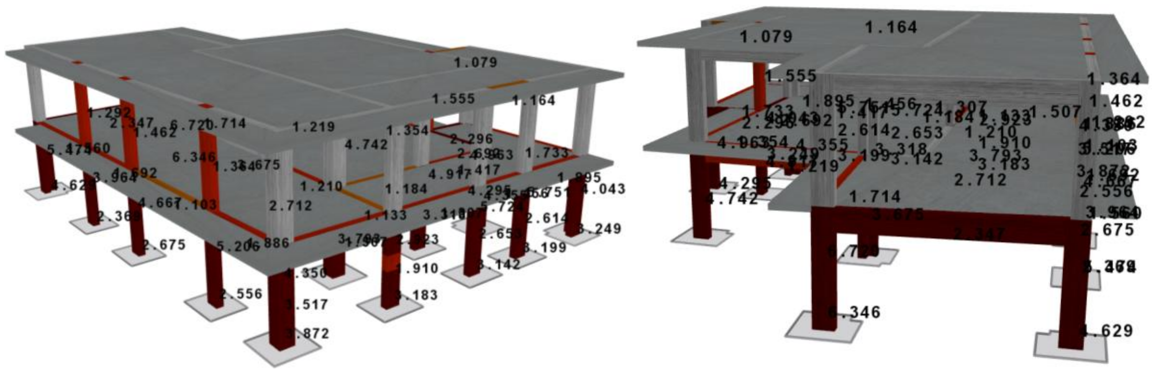

Figure 13: Shear checks with the Linear Dynamic Procedure, LDP

Looking at the Linear Static Procedure results for the strengthened building and the relevant code-based checks (in particular in shear, which was the critical check in the initial building), it can be seen that now none of the member fails. The maximum DCR ratios are now observed in the beams that were not strengthened with values up to 0.982. For the columns that have been jacketed the maximum DCR is equal to 0.568, considerably smaller than the 3.500 value of the original structure (Figure 14). The maximum DCR in columns is found on the second floor, which was not strengthened (DCR=0.978). Similar are the observations for the checks in chord rotation with a maximum DCR value equal to 0.413.

With the Nonlinear Dynamic Procedure NDP the results are better in terms of the maximum demand-to-capacity ratios; the maximum DCR in shear is equal to 0.881, and the maximum DCR in chord rotation is 0.201.

With the Linear Dynamic Procedure LDP instead the results are very conservative. Although the jacketed columns of the ground floor withstand the demand, in total 27 members fail in shear and 33 members fail in bending. The maximum DCR values are 3.345 in shear and 10.331 in bending moments.

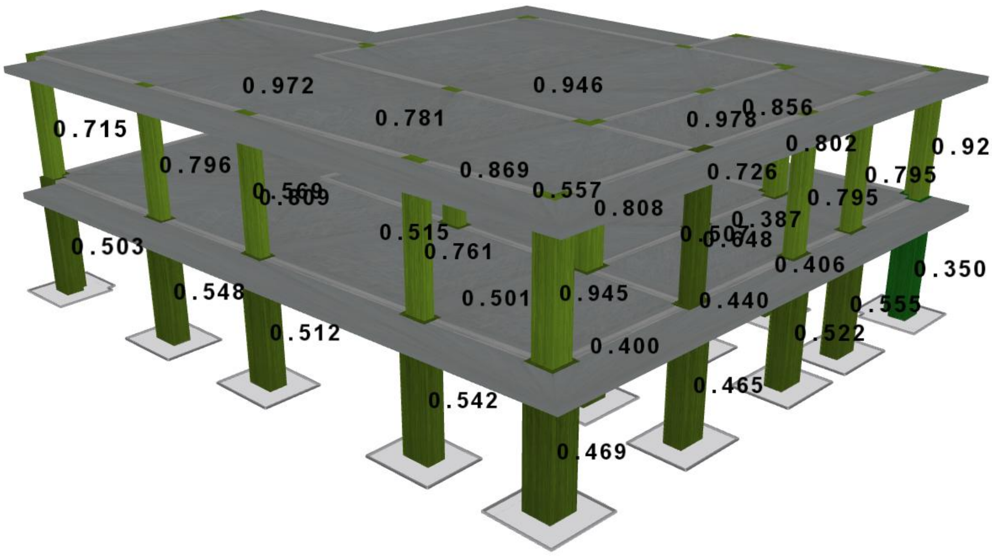

Figure 14: Shear checks in the strengthened building with the Nonlinear Static Procedure, NSP

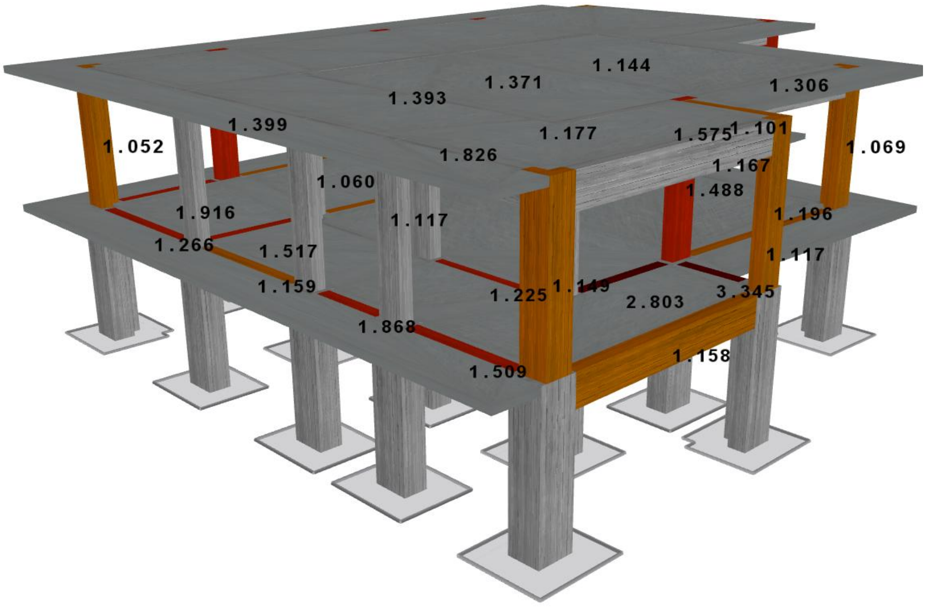

Figure 15: Shear checks in the strengthened building with the Linear Dynamic Procedure, LDP

Figure 16: General view of the building during the construction of the concrete jackets

Obviously, if the design of the strengthening scheme was based on the results from the linear dynamic analysis, the interventions would have been much more costly and more invasive, since retrofit of the components in the 2nd (undamaged) floor would have been required. By applying the Nonlinear Static Procedure and pushover analysis the intervention works were confined in the ground story columns only (Figure 16), without causing additional costs and unnecessary disruption to the other parts of the building that have not sustained damage.

7 CONCLUSIONS

In the present work, the full code-based procedures for seismic assessment and retrofit have been carried out for the case of two different buildings, one with short columns and one with a soft ground story. The analyses were carried out with both the linear and nonlinear assessment methodologies and the results were compared against each other.

The outcome of this investigation clearly shows that the nonlinear methods are more suitable for analyzing existing buildings. This is because the liner methods, due to the inherent inaccuracies, are overly conservative in their estimates of the seismic capacity and very frequently they lead to excessive interventions that are costly and cause significant and unnecessary disturbance in the operation of the building. This can be counterproductive from the public safety point of view, since very often owners are left with two options, none of which is appealing: the demolition and the reconstruction of the building or to do nothing (from experience, the ‘do nothing’ approach is what building owners usually choose when the required works tend to become extensive).

The use of nonlinear methods instead, combined with the good knowledge of the structural configuration can be very advantageous, and lead to lighter and less invasive interventions. In particular, the NSP is gradually becoming the 'standard' methodology for assessment and retrofit, because it is faster than the NDP, but also because of the simplicity in its application. Note that the potential of the Nonlinear Static Procedure in the assessment and retrofit of existing structures is indirectly acknowledged by all the modern assessment standards (in particular ASCE-41 and EC8 Part-3), since the description of the method is covered in considerably more detail, compared to the other three methods.

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