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Seismic Strengthening of Existing RC Structure Through External 3D Exoskeleton

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1 Abstract

The seismic hazard in the southern regions of Europe is known as one of the most critical issues when considering the improvement of the existing buildings in terms of energy and structural behavior. The use of integrated plug-and-play systems can be a solution to the most common obstacles occurring during the different phases in the building practices: from the design to the realization. Within the framework of the European project Pro-GET-onE, a case of structural strengthening obtained by applying a steel exoskeleton connected to the reinforced concrete (RC) structures of an existing building has been presented. The modelling, the linear and non-linear analyses were conducted with finite element software. They have been implemented for the pilot case of Athens, and the results have been achieved in relation to different parameters such as PGA, shear exploitation and displacement in the plastic phase. This approach determines an increase in the global stiffness of the structural system with a consequent reduction in displacements. Depending on the actual plasticization of the RC frames, the resulting excursion in the plastic phase of the exoskeleton is proven to provide a dissipation of extra energy and a stiffness increase in the existing building.

Keywords: earthquake; steel exoskeleton; seismic retrofit; reinforced concrete structures; shear walls.

2 Introduction

The Mediterranean countries of the European Union are indicated as the areas with the highest probability of natural earthquakes in Europe [1]. In these areas, recent seismic events have shown how the issue of seismic vulnerability is relevant for existing buildings of reinforced concrete (RC) since many of these were designed without any reference to anti-seismic criteria or with overpassed standards.

Seismic strengthening solutions can be distinguished according to the number of resistant elements involved and to the strategy adopted. There are local interventions that may involve the strengthening of structural elements or global interventions that can change the behaviour of the building by inserting additional structural systems. Using one or more techniques, two main strategies are outlined, one concerns the increase in capacity while the other reduces demand on the existing structures.

The choice of the strategy depends on numerous factors, including invasiveness, cost, global behavior and critical aspects of the structure.

This study is included in the European project Pro-GET-onE and therefore depends on some characteristic constraints that determine its strengths [2]. The strategy of the project proposes a type of integrated seismic improvement solution that excludes the displacement of the inhabitants and at the same time entails an energetic requalification through a system of volumetric architectonic additions with a consequently new performing envelope. These objectives can be achieved thanks to the positioning of the new strengthening structures outside the existing building, through the use of a steel exoskeleton as shear wall.

The use of infill shear walls has a wide application area for the strengthening of vulnerable structures [3]. Shear walls can be made in reinforced concrete or with rigid steel frames. The first case is the most widespread and researchers tested both external wall schemes implemented in perpendicular [4] or parallel [5] to the side of the building. Alternatively, the use of steel shear walls has already been studied and tested in literature [6–12]; even following the same design logic of intervention as Feroldi et al [13], further proof of the importance of this topic in the European territorial context. In these studies, it is possible to find numerous recurrent points in the application of this strategy such as the tangible increase in stiffness, strength and capacity of the structure usually linked to a reduction in the overall displacements. In some cases, this method leads to a reduction in ductility and a variable evaluation of the effects.

The evaluation of the seismic improvements achieved for the student house of Athens (prototype of the EU project) with the addition of the external steel frames (GET-system) has been performed by using SAP2000 [14] complies with the Eurocodes [15–20]. Below, the initial state (IS-ATH) and the project (PR-ATH) evaluations will be presented together with a specific paragraph for the Near Collapse limit state analysis.

3 Seismic vulnerability assessment

3.1 Athens case study

3.1.1 Description of the student house

The building belongs to the National and Kapodistrian University of Athens campus, in Zografou. The case study represents a part of the entire building of the student house; it is divided with an expansion joint from the rest and for this study it is considered independent (see figure 1).

Figure 1. Picture of the Athens Student House

The structure consists of five floors, the basement of 3.9m height while the four upper floors of 3.0m height each, including the flat roof, with a total plan size of 22.35x12m. The horizontal structures are made of 18cm thick concrete slabs that can be considered as diaphragm constraints. The vertical elements are piers of 125x25cm, and the beams have different heights (from 55cm to 70cm) depending from the spans and a web of 25 cm width. Below it is possible to see a horizontal cross section in figure 2 and the finite element model in figure 3.

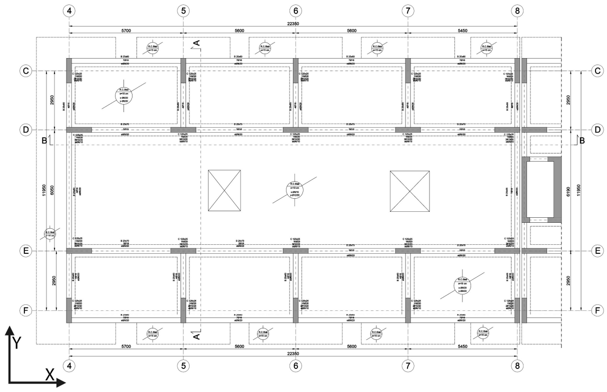

Figure 2. Horizontal cross section of the type plan

The reinforcement bars have been defined based on a report resulting from a structural survey aimed at obtaining an intermediate level of knowledge that allowed the use of a confidence factor of 1.2. The C20/25 ($f_{ck,cyl}$=20Mpa) concrete class and the S400 (f_{yk}≥400MPa) deformed ribbed bars were used.

3.1.2 Seismic parameters

Three elastic reference spectra are defined for the analyses corresponding to the three main limit states used in the verifications following the requirement of the EC8-1 [19]. The acceleration anchor values are reported related to the seismic zones defined in the Greek national standards EAK2000 [21]. The soil could be classified as B while the building is classified with an importance class III, so each acceleration value is multiplied by 1.2.

In this work the limit states of Damage Limitation (DL), Significant Damage (SD) and Near Collapse (NC) are used with the following parameters:

- LS DL – P_{VR}=20%; T_R=225yrs; **a_g=0.1512g**;
- LS SD – P_{VR}=10%; T_R=475yrs; **a_g=0.192g**;
- LS NC – P_{VR}=2%; T_R=2475yrs; **a_g=0.337g**.

3.2 Performed seismic analyses

The structural performance of the IS-ATH was evaluated using the Modal Linear Analysis (MLA), the Static Pushover Analysis (SPA), the Linear Dynamic Analysis (LDA) and the Non-Linear Dynamic Analysis (NLDA).

The MLA was used to determine the dynamic behavior of the structure, identifying the periods (T_i) and the percentages of activated mass of the main vibrating modes.

The SPA was used to calculate the behavior factor q to be used in LDA, and the displacement to the damage limit (u_{LS-DL}), the latter used as a target shift for the NLDA [13]. The SPA was performed along the two main directions of the structure and in both ways, with two different load distributions. More precisely: the "uniform" distribution, with lateral forces proportional to the masses and a "modal" pattern, proportional to lateral forces determined in elastic response spectrum analysis using an adaptive procedure where there was a torsional component.

Non-linear behaviour of all the resistant elements involved in the analyses was considered with reference to the EC8-3 [20]. For beams and columns ductile mechanisms, chord rotational hinge with concentrated plasticity was used. While, the brittle behaviour of the elements has been evaluated using the ultimate shear resistances of all the sections with reference to EC2 [17] for element with confinement bars. These force-controlled hinges were placed at the ends of the beams and in the middle of each column.

The *q-factor* is obtained, with reference to EC8-1 [19], deriving the overstrength ratio α_u/α_1 directly from the SPO capacity curves. Two *q-factors* will be distinguished for both the main directions, meaning two different design response spectra applied.

Once the behavior factors have been defined, the structural elements have been verified in terms of resistance through the LDA by evaluating the capacity/demand ratios (C/D), where the demand values are obtained using the LS-SD. In addition, the inter-story drifts are also verified, as defined in EC8-1 [19] using the damage limitation elastic response spectra. For both checks, the anchorage values of acceleration a_g has been reduced until stresses and deformations became lower than the capacity of the existing elements identifying the vulnerability of the structure.

Finally, the NLDA was used to verify that the displacements of the structure subjected to accelerograms that match the elastic response spectra in damage limitation are lower than the u_{LS-DL} target shift. Furthermore, the NLDA was used to investigate the behavior of the structure in the LS-NC.

3.3 Results of the Initial State

The MLA has provided the following parameters that characterize the dynamic behavior of the IS-ATH:

- 1st mode; T=0.91s; transversal(Y); 80% of activated mass (m_a);
- 2nd mode; T=0.86s; torsional(Z); m_a=80%;
- 3th mode; T=0.85s; longitudinal(X); m_a=82%.

Figure 4 shows the worst capacity curves obtained from the SPA with a total base shear value of:

$$V_{X,IS} = 4374.612\ kN \tag{1}$$

$$V_{Y,IS} = 2861.986\ kN \tag{2}$$

respectively in the X and Y direction. These values are related to the formation of the first plastic hinge in the LS-SD. From these curves it was possible to determine the following structural factors for the IS-ATH:

$$q_X = 2.701 \qquad q_Y = 2.812 \tag{3}$$

These values have been used to reduce the elastic spectrum in the LDA in order to verify the ductile mechanisms. While the fragile mechanisms have been verified with a factor q = 1.5 as indicated in the Eurocodes [19].

The stresses obtained from the combination with the LS-SD show that the verifications are satisfied to a 42% of the design seismic action. While the LS-DL deformation check is satisfied by applying a 15% reduction to the anchoring acceleration.

The results obtained by the NLDA confirm this data. In fact, two load combinations, among the seven analyzed, present displacements greater than the target identified with the non-linear static analysis (u_{LS-DL}). As can be seen in figure 5.

4 Project solution, 3D exoskeleton

4.1 GET system approach

The additional structure provided by the project consists of steel frames (two columns and a beam) for each floor, with bracings in the transversal direction, connected to the existing reinforced concrete frame at the column-beam joints. These rigid frames increase the stiffness of the existing RC frames toward the in plane horizontal actions. These frames are connected in longitudinal direction with additional beams hinged to create the space suitable for housing the volumetric additions. The connection between the two structures is a cylindrical hinge connected to the exoskeleton by means of a flange and connected to the concrete joint with an UPN profiles fixed along the perimetral beams. This joint allows the rotation in the plane perpendicular to the existing façade. A picture of the finite element model is showed in figure 3. S275 structural steel was used with the following sections:

- HEA 280 for columns;
- HEA 160 for transversal beams;
- HEB 140 for longitudinal beams;
- ϕ76.1x3.2 for vertical concentric braces;
- ϕ114.3x7 for vertical concentric braces in the last two spans near the seismic joint in order to avoid torsional behavior in the two principal vibrating modes.

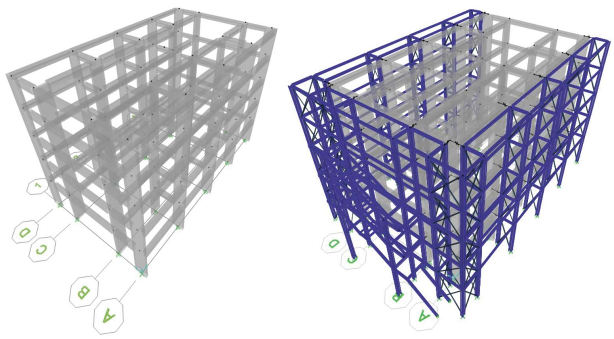

Figure 3. Finite element models of the IS-ATH and of the PR-ATH

4.2 Results of the project solution

The post-intervention results were obtained using the same analyses and procedures used for the initial state.

The MLA has provided the following parameters:

- 1st mode; T=0.74s; longitudinal(X); m_a=82%;
- 2nd mode; T=0.63s; transversal(Y); m_a=82%;
- 3th mode; T=0.59s; torsional(Z); m_a=82%.

From the selected capacity curves (fig. 4) base shears at LS-SD are:

$$V_{X,EB} = 4263.03\ kN \qquad V_{X,GET} = 1665.121\ kN \tag{4}$$

$$V_{Y,EB} = 3723.46\ kN \qquad V_{Y,GET} = 3738.051\ kN \tag{5}$$

where $V_{i,EB}$, is the base shear of the RC frames, while $V_{i,GET}$ is the horizontal load taken by the GET-system. The following values are obtained for the q-factors:

$$q_X = 2.818 \qquad q_Y = 3.4361 \tag{6}$$

Therefore, proceeding with the LDA, the PR-ATH is verified in terms of resistance, with a seismic action equal to about 86% of the design value for the LS-SD.

As regards the verification of the displacements, with the NLDA, it is 100% satisfied with the design seismic action. It is possible to note this result from the following graph in figure 5.

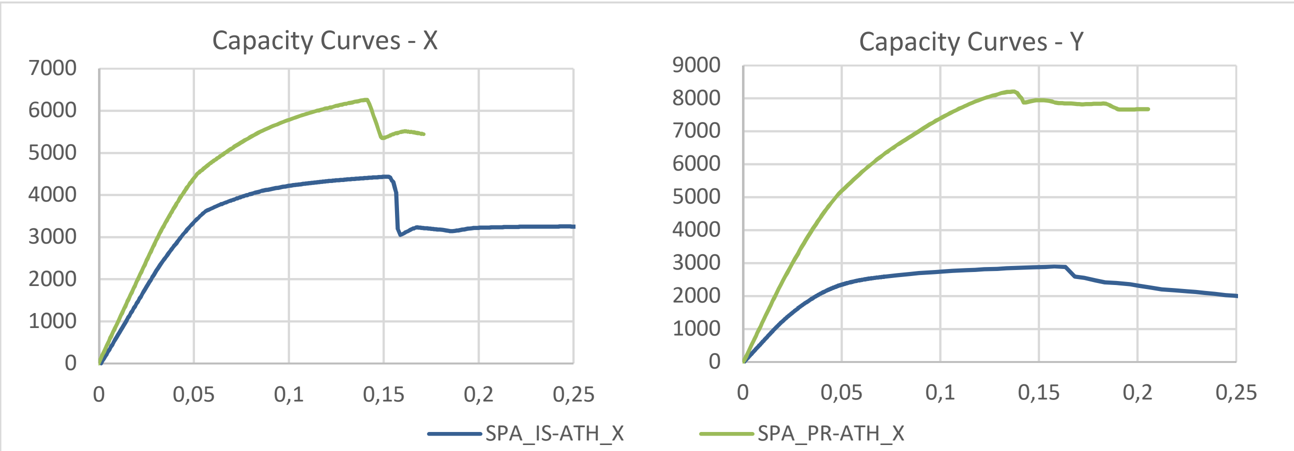

Figure 4. The capacity curves obtained from the static non-linear analyses before and after the strengthening

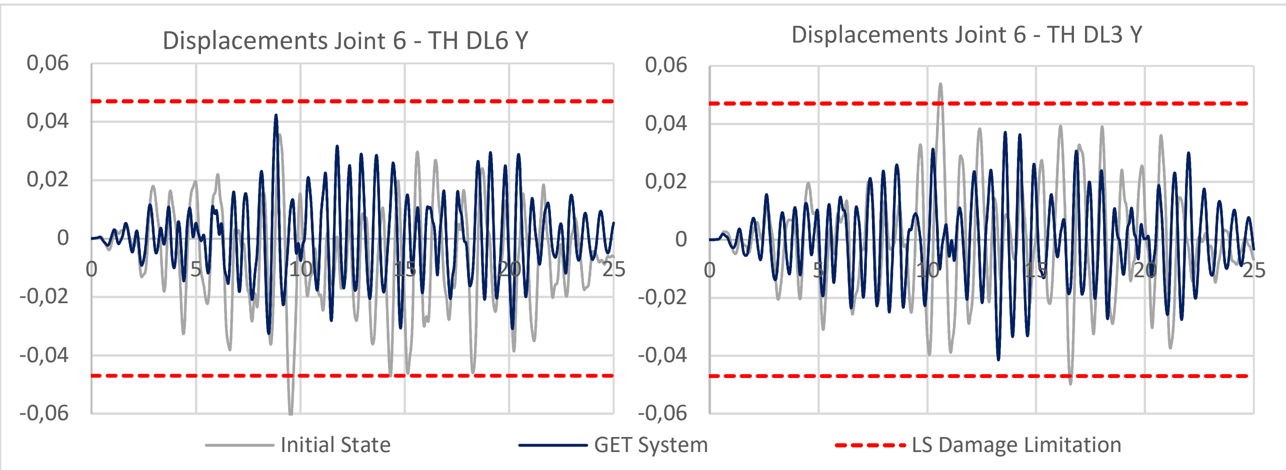

Figure 5. Comparison of the displacements of the joint 6 plotted during two seismic events before and after the intervention (generated with artificial accelerograms, combinations TH DL3 and TH DL6)

5 Consideration on LS-NC

The steel strengthening frames increase the overall resistance of the existing building with consequent reduction in displacements. The non-increase in dissipative capacity is confirmed by other similar studies [4, 10]. For this reason, the SPA has been used to study the displacement capacity for the LS-NC. The target displacement (TD) is not verified for most lateral load combinations due to the decrease of plastic hinges that have displacements beyond the elastic limit, resulting in a worsening in the LS-NC.

In order to better understand the behavior of the structure during a seismic event of great intensity the NLDA is also used. Seven artificial accelerograms combinations compatible with the LS-NC elastic response spectrum are used (Time History analyses, THn LS).

The results regarding the IS-ATH show:

- Four plastic hinges over the NC displacement limit in TH NC4;
- Brittle failure of a column in TH-NC5;
- Eight plastic hinges over the NC displacement limit in TH NC6;
- Two plastic hinges over the NC displacement limit in TH NC7.

The same artificial earthquakes applied to the post -strengthening situation do not show any plastic hinges formation except for the TH NC4 where, however, there are still three hinges over the NC displacement limit. From these results seams that an improvement is guarantee also in the LS-NC allowing the existing structure to withstand these earthquakes.

6 Conclusions

The results show that the external exoskeleton can be used to reduce the torsional effects in the primary vibrating modes in order to improve resistance to the lateral actions along the main directions of the existing structure.

The checks carried out with the LDA show that the GET-System produces an improvement in terms of resistance towards seismic actions of 44%. In terms of displacements, the verifications are satisfied confirming the possibility to minimize the damage for the frequent earthquakes.

The application of the steel exoskeleton involves an overall increase in stiffness and consequently an increase in the base shears of the structure. In longitudinal direction, a small stress reduction in the existing building has been noted and an increase in the total base shear of 35.51% (eqs. 1, 4). The small value is due to the fact that the GET-system design provides the frames only on one side and these frames are also without bracing due to architectural needs. In the Y direction the existing building manages to take a greater percentage of shear in the Y direction, and overall the total base shear is increased by 160.71% (eqs. 2, 5).

The reduction in displacements do not allow the same energy dissipation trough the plasticization of the non-linear hinges. This situation can lead to a decrease in the overall ductility of the structure in terms of displacement and can bring negative results in TD verifications in the LS-NC. The different results obtained from SPA and NLDA were analyzed, confirming the necessity of further insights. Finally, to enrich the potential of the steel exoskeleton, the insertion of dampers inside the steel frames or in the connection between the two structures will be a possible solution to obtain a reduction in seismic demand as well as the increase in strength and stiffness.

7 Acknowledgements

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