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Performance building evaluation based on inspection, testing and numerical analysis-case study

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ABSTRACT: The trend of building additional stories upon existing structures in the area of Skopje city, particularly on existing masonry structures that, as a rule, were built prior to the enforcement of the seismic regulations in the country, is increasing. At the initiative of the Karposh municipality in Skopje, the Institute of Earthquake Engineering and Engineering Seismology, IZIIS, was realized a project, whose purpose was definition of a methodology for assessment of the existing building stock in the municipality from the aspect of seismic stability. Within the frame of this project, a case study has been realized to evaluate seismic stability of a selected residential structure after its enlarging and building of additional stories. Performance evaluation was done based on visual inspection, field testing for system identification as well as on numerical analyses. Definition of seismic site potential was carried out, also. The main purpose of the performed analyses was to give insight into the performance of this integral structure during future earthquakes.

1 INTRODUCTION

The main design philosophy in most of the contemporary seismic codes is based on protection of human lives under strong earthquakes and partially controlled damage during the occurrence of the so called frequent earthquakes. Macedonian seismic design code follows this practice.

In conditions of an increasing trend of building of additional stories and enlargements of existing structures, there inevitably arises the question about the seismic stability and safety of such new “hybrid” structures. This is even more emphasized by the fact that, in most of the cases, the existing structures represent masonry structures built prior to the passing of the seismic regulations in the country. After performance of works on reconstruction, enlargement, building of additional storeys, adaptations, etc. on existing structures, their structural system and dynamic characteristics are changed. If such works are designed and realized non-professionally, the seismic stability of the structure could be disturbed.

However, practice in the Republic of Macedonia has shown that such an important problem is not paid sufficient attention and that the existing seismic resistance of such buildings could be even reduced in this case. The valid regulations in Republic of Macedonia allow building of additional stories upon structures without seismic strengthening if the mass of the structure is not increased for more than 10% regardless the seismic resistance/non-resistance of the existing building, considering that the behavior of the structure under an earthquake will not essentially be changed. Following such code prescription without any criticism could lead toward increasing of the seismic risk of such kind of buildings.

This current engineering practice has been the motivation for the cooperation between the IZIIS and Karposh municipality in Skopje within which projects on definition of a methodology for geo-referenced inventory toward seismic stability and safety of the existing building stock in the municipality are being realized Necevska-Cvetanovska et al. (2012). This is a pioneering activity at national level whose final goal is to increase the level of seismic protection of citizens and structures in this municipality. The final objective of these projects is definition of a two-level methodology for geo-referenced inventory toward seismic stability and safety of the existing building stock in the Karposh municipality, as well as upgrading of GIS of the municipality with new attributes.

Presented briefly in this paper are the results from the case study realized within the frames of this project, i.e., from the analysis of the achieved stability of the enlarged structure with constructed additional story at Blvd. Partizanski odredi 48a for gravity and external seismic effects, whereat three individual states of the structure were analyzed: only existing masonry structure, only newly designed RC structure for enlargement and building of other stories and the integral structure. Generally, the procedure consists of elastic-static analysis, then analysis of elements up to ultimate state of bearing and deformability capacity as well as analysis of the dynamic response of the system to real seismic effects with intensity and frequency content that are expected at the considered location. For the needs of the mentioned analyses, a complete visual inspection of the structure, field tests for determination of the seismic potential of the soil as well as non-destructive experimental tests on the structure for the purpose of identification of the bearing system and definition of its main dynamic characteristics, were done previously.

2 PERFORMANCE BUILDING EVALUATION OF THE SELECTED CASE STUDY BASED ON INTEGRATED APPROACH

2.1 Case study – structural characteristics

The existing residential structure at Blvd. Partizanski Odredi 48a was built in 1958 as a structure consisting of B+GF+3, proportioned 30.70 m/9.50m at plan. From structural aspect, the structure represents a massive structure with bearing walls constructed of solid bricks in cement lime mortar in both orthogonal directions and fine ribbed floor structure. According to what the occupants say, during the earthquake of 1963, the structure suffered minimal damage that was repaired in 1964-65. The only interventions for strengthening of the structure involve reinforced concrete elements with a height of 1.5 m constructed in the form of buttresses in the basement of the structure up to the terrain level. The newly designed enlargement and additional storeys of the existing structure consist of a new RC structure that is designed such that it “bridges” the existing masonry structure. For the purpose of enabling communication between the existing masonry structure and the enlargement at individual levels, individual parts of the existing masonry, parapets below windows and new openings were demolished. In such a way, “the new integral” structure is enlarged and two additional storeys are built whereat its total height from the fixation at the foundation to the top is H = 23.12 m, (Fig. 1).

In accordance with the defined methodology for seismic performance evaluation Necevska-Cvetanovska et al. (2012, 2013, 2014), for the selected case study the following activities were carried out:

- Definition of site seismic parameters and seismic safety criteria
- Experimental non-destructive test for definition of the real dynamic characteristics of the structure using ambient vibration method

- Field testing for system identification using nondestructive testing methods
- Numerical analyses consist of elastic (static), nonlinear dynamic analysis and analysis of loading and deformation capacities of structural elements. These analyses were realized for three different constructional phases: existing masonry structure, new RC structure which bridging the existing one and the integral structure.

Selected results from these four groups of activities are presented further in the paper.

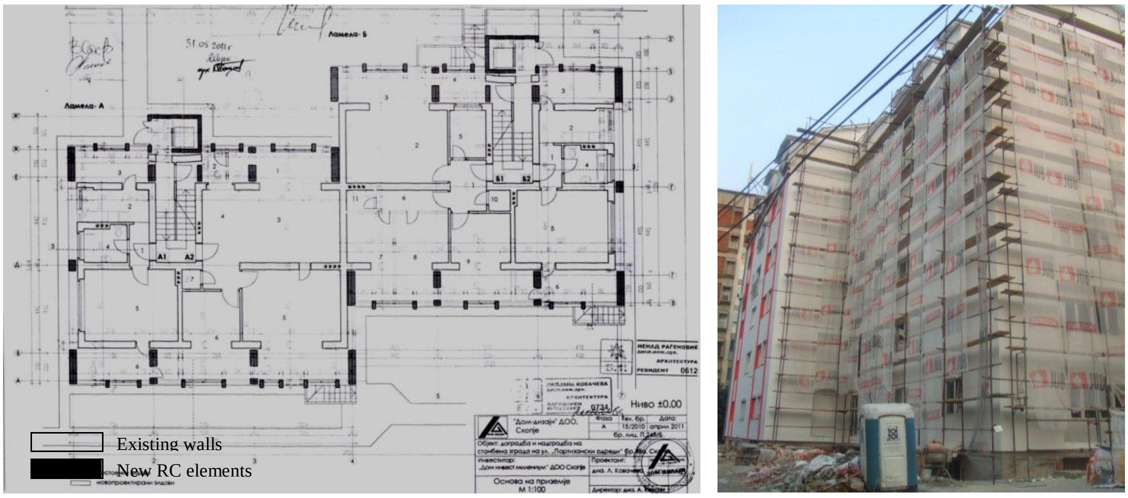

Figure 1. Ground floor plan of the existing structure with new RC part and photo during the construction

2.2 *Definition of seismic site parameters and seismic safety criteria*

For the considered site, detailed regional and local seismotectonic, geophysical and geomechanical investigations were carried out and seismic parameters with their intensity and frequency content were defined, Sesov et al. (2012). Based on seismic risk analysis, the maximum acceleration has been given as a seismic parameter for two seismic risk levels: design and maximum expected earthquake for the given location.

- Level 1: For design earthquakes, structures should behave in linear range with possible limited nonlinear deformation in selected structural elements i.e. ductility demand for masonry should be <1,5 and for RC < 2,5
- Level 2: For maximum expected earthquakes, both, structural and nonstructural elements are deeply in nonlinear range of behavior but stability of the structure is not jeopardize i.e. ductility demand for masonry should be <2,5 and for RC < 4,0

For the considered location the level of design earthquake should be 0.26g and for the maximum expected earthquake it should be 0.34g. To define dynamic structural response, three different types of earthquakes were used (El Centro, N-S, 1940, Petrovec N-S 1979, Robic N-S, 1968).

2.3 *Definition of the real structural dynamic characteristics using ambient vibration methods*

Ambient vibration methods were used to define experimentally structural dynamic characteristic which further in the analysis were applied for: (1) calibration of structural models and (2) through comparison with analytically obtained values to assess any deviation between designed and constructed building. In the considered case the main dynamic characteristics of the structure after the performed enlargement and built additional stories by application of the ambient vibration technique were: $T_1^{y\text{-}y}$=0.273s, $T_2^{x\text{-}x}$=0.259, T_3^{tor}=0.228s. It is exactly the comparison between the experimentally and analytically obtained periods made by the designer engineer ($T_1^{x\text{-}x}$=0.91s, $T_2^{y\text{-}y}$=0.65s) that speaks for itself that the structure, although constructed in compliance with the project, deviates from the designed behavior and is much more rigid in both orthogonal directions. This means that the reinforced concrete structure of the enlargement and the additionally built stories does not behave independently, but on the contrary, it behaves along with the existing structure in the elastic phase. This is due to the rigid joints from the floor along with the floor base that bridge the expansion joint between the existing and the new floor structure of the enlargement. Hence, the imposed conclusion is that, in reality, a completely independent structure cannot be constructed, which should be taken into account in the design phase as well, Krstevska et al. (2012).

2.4 Field testing for system identification using non-destructive testing methods

The identification of the existing structural system using non-destructive testing and visual inspection has shown that the existing masonry structure belongs to the system of "plain masonry" with reinforced concrete floor structure. In accordance with the valid Rulebook on Construction of Structures in Seismic Regions, (PIOVS 81) in regions with Intensity of IX degrees, as is the Skopje one, only ground floor structures are allowed for this type of plain masonry system, Sendova et al. (2012).

2.5 Numerical analyses of the structural system

2.5.1 Methodology for structural analysis

For the needs of analytical verification of the seismic safety and stability of the structure, three individual states of the structure were treated as follows:

- Existing state of the original masonry structure (PS- existing state) and after demolition of some of individual elements (NPS – newly designed conditions);
- Newly constructed reinforced concrete structure by which the existing structure is bridged (AB)
- The state of an integral structure – the entire reinforced concrete structure with the included existing masonry structure (AB+NPS)

For each of the individual states, the following types of analysis were performed:

(1) Elastic, i.e., static and equivalent seismic analysis by use of the finite element method and SAP 2000 computer program (Structural Analysis Program, UC Berekely, California). For the needs of such analysis, a mathematical model was prepared for each of the individual states. Equivalent seismic force analysis was carried out according to the national regulations.

(2) Analysis of elements up to ultimate state of strength, bearing capacity and deformability resulting in Q-δ relationships for each element and cumulatively for each story, displacement capacity δ_u and ductility capacity defined as $\mu=\delta_u/\delta_y$. Displacement and ductility capacities are defined using the methodology and a corresponding package of computer programmes for optimal design of new and performance evaluation of

existing structures, (UNDP/UNIDO Project 1984, Necevska-Cvetanovska et al. 2000). Such an analysis was carried out separately for the existing masonry structure, for the state prior and after demolition of the individual parts for the purpose of communication with the enlarged part and separately for the newly constructed reinforced concrete structure bridging the old structure.

(3) Analysis of the dynamic response of the system to actual seismic effects with defined intensity and frequency content expected at the considered location. Applying modeling by concentrated masses assuming concentration of distributed structural characteristics of individual stories, a nonlinear dynamic analysis was performed by use of a storey hysteresis model obtained from the previously performed analysis of elements up to ultimate state.

2.5.2 Comparative analysis for different structural states – selected results

In this part, comparative presentation of the most important results obtained from the performed analyses of the three different states: (1) the newly designed state of the masonry structure itself (NPS), (2) the newly constructed reinforced concrete structure itself (AB) and (3) the integrated structure as an entirety consisting of the newly constructed structure and the old masonry structure, (AB+NPS) were given. The existing state of the structure prior to the demolition of some individual parts (PS) was not considered due to the obtained similarity of the results with the NPS state.

Main Dynamic Parameters

Table 1 shows comparatively the experimentally and the analytically obtained fundamental periods for the corresponding orthogonal directions. What can be concluded is that the elastic state in which the structure was at the moment when the dynamic characteristics were measured experimentally, the structure, i.e., the enlargement and the additional stories did not behave separately and independently of the existing masonry structure, but on the contrary, the structure behaved as an entirety, which was confirmed by the very close values of the experimentally obtained fundamental periods to those obtained from the analysis of the integrated structure (AB + NPS).

Table 1. Comparative presentation of the obtained fundamental periods (in sec)

Direction	PS	NPS	AB	AB+NPS	Ambient vibration test
Longitudinal, x-x	0.243	0.259	0.643	0.265	0.259
Transverse, y-y	0.274	0.278	0.541	0.269	0.273
Torsion	0.226	0.244	0.424	0.251	0.228

Elastic Analysis

Presented in the figures 2 to 4 are the ductility demands in the longitudinal and transverse direction, respectively, obtained from the dynamic response of the structure in the three individual states under the maximum expected earthquake, along with the ductilities allowed in accordance with the defined design criteria. With the exception of the Robic earthquake, after demolition of individual parts of the structure, under the design and maximum earthquake, the masonry structure exhibits ductility that is greater than those allowed by the seismic safety criteria (Fig. 2). As to the reinforced concrete structure that bridges the masonry structure, it exhibits a relatively good behavior in longitudinal direction (Fig. 3), but in transverse direction, because of the enormous rigidity acquired due to the dimensions and the position of the walls, it requires ductility that is much greater than those allowed in accordance with the seismic safety criteria, particularly in the case of the Petrovac earthquake.

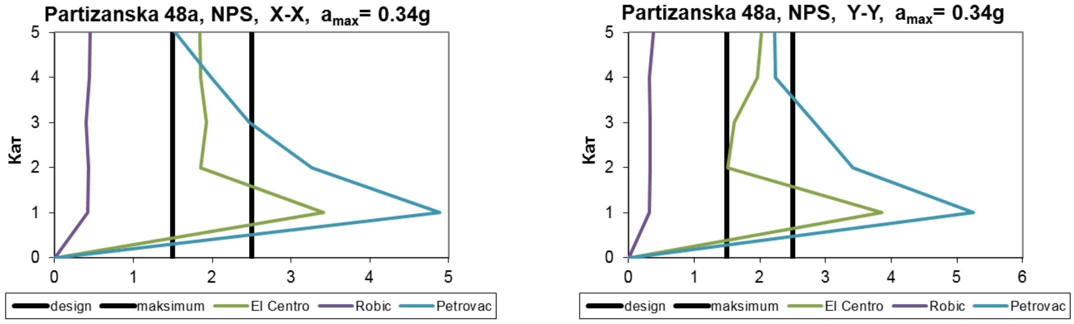

Figure 2. Ductility demand, NPS existing state

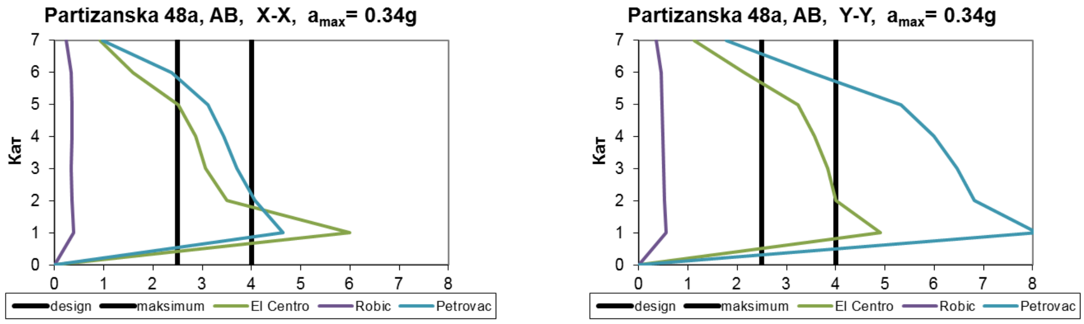

Figure 3. Ductility demand, only enlargement and additionally built stories, AB

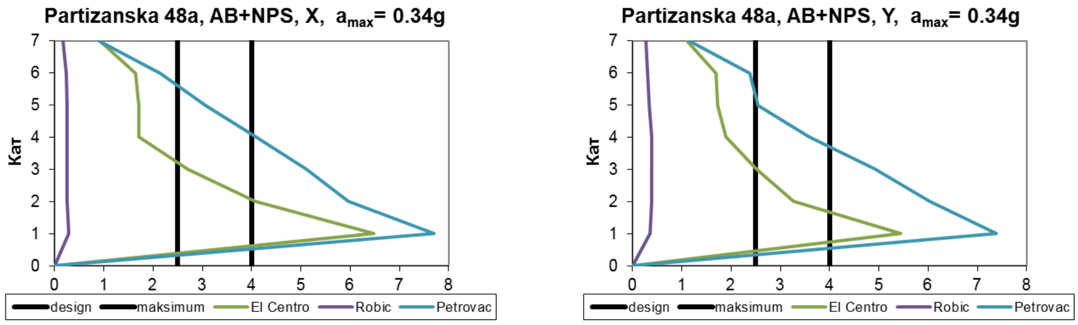

Figure 4. Ductility demand, integrated structure, AB+NPS

Analysis of Bearing and Deformability Capacity

The results from performed analysis imposed that there is a big difference in stiffness and strength of the existing masonry structure and the newly constructed structure. The existing masonry structure at the ground level has capacity of 2924 kN and 2070 kN in longitudinal and transverse direction, respectively, which is less than the necessary capacity defined by the national regulations (PIOVS 81) which amounts to 20% of the total weight of the structure, i.e. 4016 kN. Quite the contrary, at the ground floor, the newly constructed reinforced concrete structure has a capacity of 8544 kN and 28128 kN in longitudinal and transverse direction, respectively, which is considerable and, at the same time, unnecessarily more than the necessary capacity defined with the national regulations (PIOVS 81) amounting to 10% of the total weight of the structure i.e. 2628 kN.

Considering that the structures will behave separately after the trigger of the expansion joints, this difference in bearing and deformability capacity, under a certain earthquake level, may lead to damage to the existing masonry structure (exhaustion of bearing capacity or deformability) while, at the same time, the newly constructed part behaves completely in the elastic range. This means that, with this design solution, the design engineer does not provide the same level of designed seismic protection for both structures.

Figs. 5 and 6 graphically show the comparison between the ductility demand versus capacity for actual earthquake effects with intensity of a_{max}=0.34g, for each of the orthogonal directions and for both structures taken separately. Hence, it is clear that the ductility of the masonry structure is considerably lower than the required and vice versa, that the ductility capacity of the newly constructed structure is considerably higher than the required ductility. This again speaks for itself about the big differences in the behavior of both structures, which is unfavorable because under the same earthquake intensity, the structures will behave in a different way.

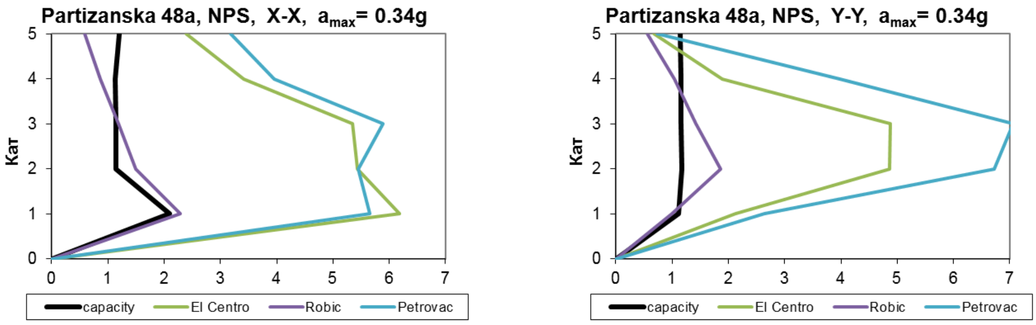

Figure 8. Ductility demand versus capacity, NPS

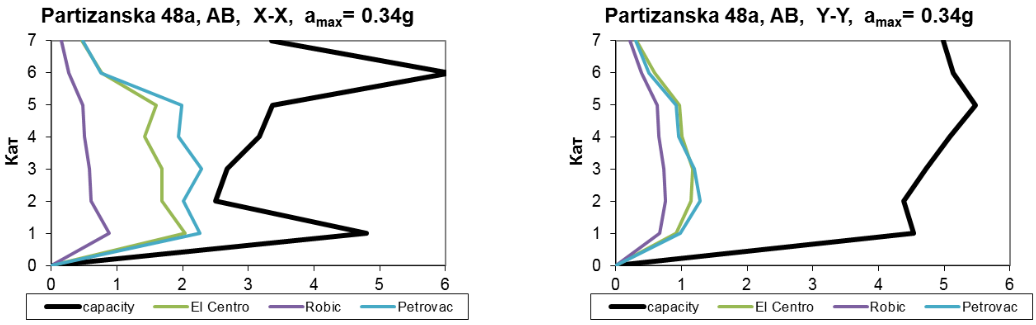

Figure 9. Ductility demand versus capacity, AB

3 CONCLUSIONS

- Performance evaluation of a selected residential structure after enlarging and building of additional stories was done based on visual inspection, field testing for system identification as well as on numerical analyses. The main purpose of the performed analyses was to give insight into the performance of this integral structure during future earthquakes.

- The comparative analysis of the bearing capacity and deformability points to a drastic difference between the stiffness and the strength of the existing masonry structure and the newly constructed reinforced concrete structure. With such design solution, the designer does not provide the same level of designed seismic protection of both structures.
- The ductility capacity of the masonry structure is considerably lower and vice versa, the ductility capacity of the newly constructed structure is considerably greater than ductility demand. This again points to a big difference in the behavior of both structures because, under the same earthquake intensity, the structures will behave in a different manner.
- The experimental investigations of the structure by use of the ambient vibration technique show that, in the elastic range, the entire structure behaves integrally, which is due to the continuous construction of the floor bases and the floor over the expansion joints. The above conclusion is also confirmed analytically. In the post-elastic range of behavior this connection will break and the integrity of the structure (as designed now) will be disturbed. So, the existing masonry structure and newly constructed RC structure will behave as a separate structure during future earthquakes.
- During the design of enlargement and additional stories, the designer has a professional obligation to provide equal level of designed seismic protection of the existing and the newly constructed part. The integrated structure should exhibit harmonized behavior during future earthquakes and should enable equal level of protection of the occupants.

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