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Seismic Retrofitting of an Existing Structure

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***Abstract*:** *Many buildings in Albania continue to be designed with the old national standards which are not renewed for more than 30 years. So even the buildings designed and constructed in recent years if will be designed with European Codes under design earthquake will be heavily damaged or collapse.*

The article gives the analysis and design for adopting a reinforced concrete structure built several years ago with new European Codes that in the near future will become effective even in our country.

Initially the article describes the conditions of the existing structure and then gives the performed nonlinear analysis (adaptive pushover) and structural measures suitable for this type of structure.

The analyzed structure is very common in Albania and this study gives recommendations for the methodology of design and the most appropriate strategy of retrofitting these kinds of structures in order to achieve the required level of performance and increase their level of security based on European Codes.

Keywords: *Seismic retrofitting, EuroCodes, existing structure*

1. Introduction

Even some attempts in recent years in Albania as a result of various problems sometimes even political up to date design and constructions standards for reinforced concrete and masonry structures are not renewed. The lack of adaptation of new design standards has forced a number of designers to work directly based on Eurocodes but meanwhile more designers continue to work with the old standards. This has led to a cacophony of design approaches and that for a very seismic country like Albania will lead to very large problems if a design earthquake occurs. Some of the structures designed with Albanian old codes even when are well detailed in global viewpoint can pose problems as a result of insufficient reinforcement detailing and seismic demand change.

In the article through the example of a historic masonry building (part of palace of prime minister) designed by the Albanian codes and retrofitted based on Eurocodes recommendations aimed to give the problems that could have such type of structures and suggest to the entire community of engineers in Albania and politicians that is now essential to the designer of structures to apply European standards.

The methodology of existing structures control and retrofit passes through the following stages [1,2]:

- Dimensions and geometric data information, reinforcement bars and detailing, material of the existing structure
- Static analysis design as a new structure but with geometry and characteristics of existing material (simulation design).
- Check of structure deformations, etc. Comparison of provided dimensioning and reinforcement with required dimension and reinforcement. If provided dimensions and reinforcement are not sufficient then must be done the nonlinear analysis.
- Choose the strategy of intervention, analysis and control the retrofitted structure.

Below the article will give in detail all these stages.

2. Modelling of Structure

2.1. Existing Structure

The design of existing building is done in 1980. The building is divided in five separate structures. The first structures which will be analyzed in the article serve as secondary entrance, hall and offices premises for the complex of Prime Minister Palace. It has 3 above ground stories with almost regular form in plan.

Fig. 1: View of the structure

We have the drawings of all structural elements and reinforcement. We don't have data for other details and other possible changes during the construction.

From observations of the concrete elements is seen that the dimension of the structural elements are the same as in the design. We have done non-destructive and some destructive tests for taking the exact characteristics of the materials, and checking the height of the slab.

The building was designed based upon Albanian Design Codes. We have the final design drawings so taking into account also the real material characteristics we can consider that we have a very good level of recognition of the existing structure.

The dimensions are given in the figure below.

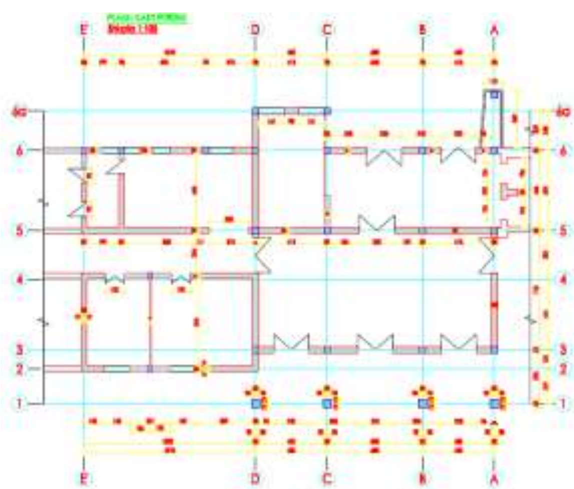

Fig. 2: Plan of the structure

2.2. Concrete Properties Investigations

Up to now there are used 4 main methods for evaluation of Concrete properties. Based on their characteristics their results are more or less reliable.

We have done 2 core tests as described in UNI EN12504-1 standard and several Schmid hammer tests as described in UNI EN12504-2 standard together with ultrasonic tests (Sonreb).

For the core tests we have used the correction given by Masi (2005) [4]

$$Fc,i=(Ch/D \times CD \times Cs \times Cd)fcore,i \quad (1)$$

Where Ch/D correction for h/D different from 2, CD correction for D different from 100mm, Cs correction for steel presence influation, Cd correction for core disturbance

From this expression we have the following characteristics?

Concrete properties from tests

Self-weight	g=2455 kg/m3
Cylinder concrete compressive strength	fck= 120 daN/cm2
Cubic concrete compressive strength	Rck= 170 daN/cm2
Design compressive, tensile strength	fcd = 80 daN/cm2

3. Structure Evaluation Based On Eurocodes

3.1. General

As recommended by the Eurocodes and the reference documents [1,2,3,5,6] structural evaluation of existing buildings in general requires an «additional» limit state. The new buildings are design to fulfil the hierarchy of resistances and appropriate ductility, and evaluated structures are design according to these requirements.

These requirements are based on the determination of three states of damage of the structure

- limit state with limited damage (immediate occupancy) IO
- limit state with significant damage (from damage control- life safety) LS
- limit state of structural stability (total or partial collapse) CP

The evaluation of the existing structure proceeds according to the following steps:

- Identification of existing data
- Determination of levels of recognition and selection of computer models
- Determination of seismic loads in every limit stage
- Modelling and Analysis
- Verification of elements

The first two items we have described in the beginning of the article, the others are given below.

3.2. Seismic Action

Albania is a very seismic zone. In the existing Albanian code the seismic input is taken from an Intensity map multiplied by soil conditions and some other factors. According to EC8[3] seismic hazard should be given only with one parameter agR on ground type "A" that correspond to rock or rock like geological formations, including 5m weak formations (soil) at surface. The values of agR (maximum acceleration PGA) are taken from the Probabilistic hazard map of Albania recommended (not officially) recently by "Geoscience Institut". The return period of the reference event is TR=475years that corresponds to a life time of 50years.

The horizontal PGA in ground type A for the city of Tirana is taken PGA=0,30g

Based on the values of PGA in rock and for the specific type of terrain is calculated the design spectrum for three limit states based on EC8 formulations and soil condition classifications. The design spectrum is taken by reducing the corresponding elastic spectrum with the appropriate structure behavior factor "q". For the ultimate limit state for local soil conditions (ground type C) this factor is taken 1.5[3].

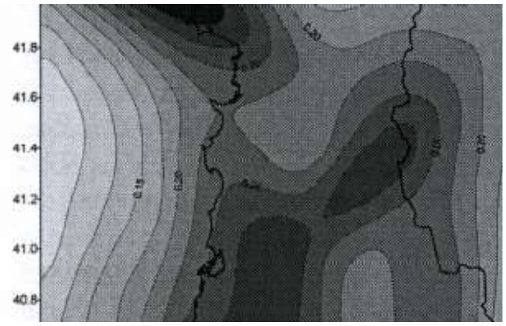

Fig. 3: Peak ground acceleration Map of Albania

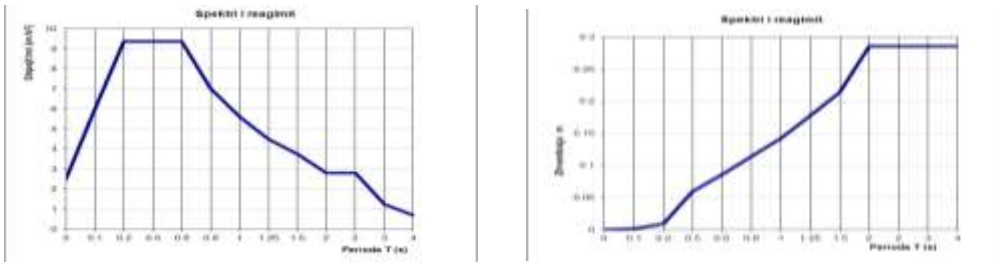

Fig. 4: Graphical view of the elastic acceleration and displacement spectrum for soil type C

3.3. Linear Analysis

Structural modelling aspects and the determination of seismic action given above is done in the same manner as for a new building according to EuroCodes 8 recommendations. The analyses and the determination of internal forces is done by spectral method with concentrated masses in the center of masses of each story. The combination of seismic loads and other actions is made according to EC1.

Model of the structure is the same as for a new building and the contribution of non-structural elements is neglected. The 3D model of the structure is given below in fig. 3.

Fig. 5: Graphical view of the linear model

The modal results are given in the table and figures below.

TABLE I: Modal Results of Structure

Case	Mode	Freq (Hz)	Period (sec)	Rel.mas. UX (%)	Rel.mas UY (%)	Cur.mas UX (%)	Cur.mas UY (%)	Total mass UX (kg)	Total mass UY (kg)
4	1	2,18	0,46	0,01	51,49	0,01	51,49	3703836,0	3703836,0
4	2	2,45	0,41	62,01	56,08	62,00	4,59	3703836,0	3703836,0
4	3	2,79	0,36	74,34	81,31	12,33	25,23	3703836,0	3703836,0

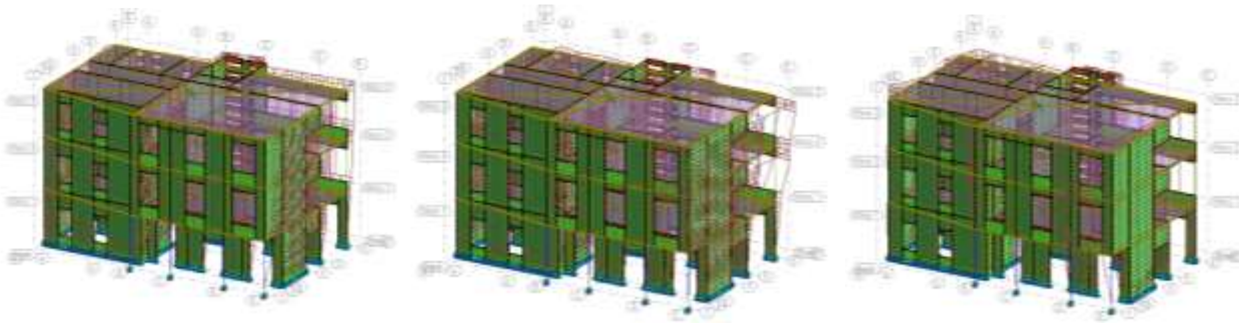

Fig. 6: Graphical view of modal forms

As seen from the modal results the structure is not regular and torsion influence its seismic behavior. After the determination in advance the fragile or ductile behavior for each element, with forces obtained from seismic combination are checked the strength of all the elements. From these results we have seen that although the primary idea as a rigid structure the deformations are nearly in limit and the first story can be considered as weak story. The columns strength cannot be assured (average safety coefficient is approximately around 0:93) while the beams meet the criteria of resistance in shear but did not meets the criteria of flexural resistance. To get a more accurate picture of the way the structure behaves it is necessary to do nonlinear analysis.

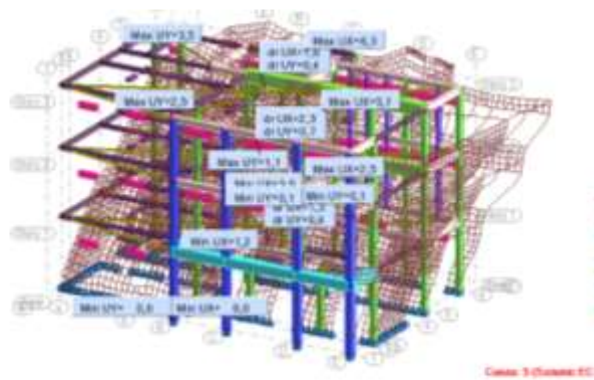

Fig. 7: Deformation of structure for earthquake motion in "x" direction

3.4. Static Nonlinear Analysis

Nonlinear static analysis is the simplest method for nonlinear analysis of structures. This analysis can usually be done with concentrated plasticity models that are the classic uses but recently distributed plasticity models are used as well.

Without treating the aspects of the method we shall give only some problems which are also reflected in our analysis of the structure.

In difference from the linear analysis in this method cannot be made a combination of results in both directions but each direction must be considered separately. For each direction are taken into consideration two types of distribution of forces, one according to normalized first mode deformations and the second according to proportional mass of each floor.

The method cannot take into account the effects of progressive degradation of strength, the redistribution due to of the plastification and the change of modal characteristics. Also in torsional eccentric structures the first mode has important effective mass participation in both directions and may not be disconnected from other modal forms. This mean simply that we cannot evidenced a first mode that effects only one direction to get real performance of the structure for each direction separately [4, 6].

In these cases must be used nonlinear dynamics analysis or adaptive nonlinear static analysis.

3.5. Nonlinear Dynamics Analysis

This analysis can consider the degradation (Softening) of structure elements strength, the change of deformations forms after each acceleration value and change of the internal forces due to deformation accumulation.

This type of analysis gives satisfactory results for torsional eccentric structures and structures for which the higher modes influence the seismic response.

The nonlinear dynamic analysis needs also a time history of acceleration of all the possible earthquakes that can occur in the site. After the evaluation of the PGA, the seismic input is taken as a real accelerograms from ESD database using the software Rexel [7] and scaling the accelerograms with the software Seismosoft [8]. The chosen accelerogram according to EC8 spectra is compatible with PGA taken from probabilistic seismic hazard analysis for TR = 475 years [3]. For taking into account the convulsion from surface to the depth in which is generated the input in the model a scaled factor is applied to the accelerograms. Three components of the one of the selected accelerograms are given in Figure below.

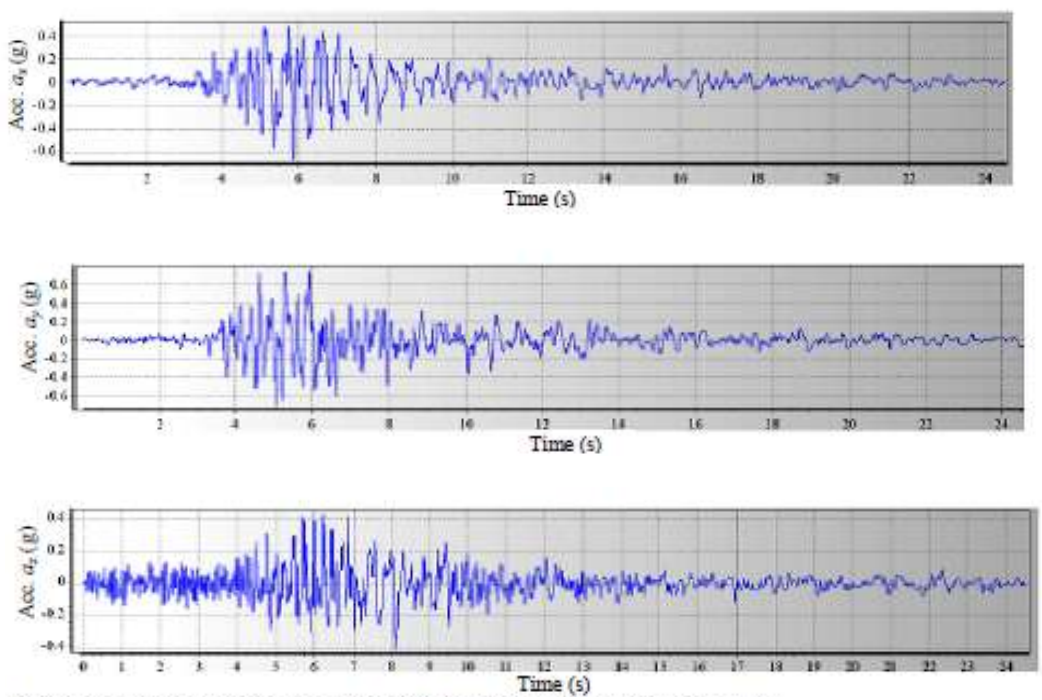

Fig. 8; Time history of acceleration in x, y and z direction for the Montenegro earthquake

In the figure below are given only some results of stresses capacity check of the most loaded plane.

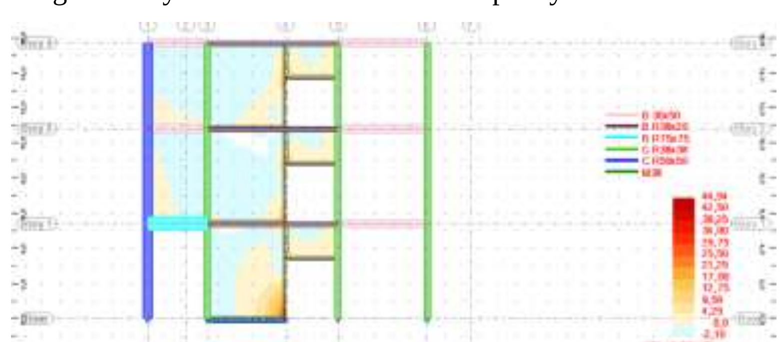

Fig. 9: stresses in vertical direction for time 5.8s. Acceleration in x direction

3.6. Results

From the obtained results, as illustrated in the above figures the flexural strength of the beams and out of plane work of some masonry panels are more problematic. To rehabilitate the structures we can use four different approaches.

1. Increasing the global capacity (strengthening). This can be done by the addition of cross braces or new structural walls.
2. Reduction of the seismic demand by means of supplementary damping and/or use of base isolation systems.
3. Increasing the local capacity of structural elements. This approach recognizes the existing capacity of the structures, and adopts a more cost-effective approach to selectively upgrade local capacity (deformation/ductility, strength or stiffness) of individual structural components.
4. Selective weakening retrofit. This is an intuitive approach to change the inelastic mechanism of the structure.

From these four types of retrofit strategy approaches we have chosen to apply a combination of first and third type, increasing the global and local capacity of structural elements because as it’s seen from the results the structure has limit stiffness for accepted performance allowed drifts (cannot apply type 4), the addition of walls or braces is of course restricted due to architectural requirements, and the use of seismic base isolation systems is quite expensive.

In our case, for this purpose we have used for the reinforcement of the beams longitudinal carbon fiber strips both in middle and supports and for columns confinement carbon fiber web.

Fiber design and placement of needed fibers is done according to Italian recommendation CNR-DT 200/2004 and then we checked the structure with reinforced sections with distributed plasticity model.

From the obtained results can be seen that after strengthening of elements the structures performance is improved and all elements meet the performance criteria in flexure, shear strength, deformative capacity and the surface layer of column concrete that in the existing structure crush and spall out is now assured.

The shear mechanisms and out of plan kinematic work of masonry panels need further studies and shall be one of our objectives in the future.

4. Acknowledgements

We want to thanks the Eng. Nicola Brini for his work in the non-destructive test for determination of material characteristics.

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