Seismic design of an irregular structure with Adaptive Pushover Analysis

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ABSTRACT

Recently we are using the European Codes for the design of structures for earthquake resistance but in some cases even European Code doesn't gives realistic results. For irregular structures the European Code gives very simplified recommendations that if will be used for the design gives results that could produce a very heavy and expensive construction.

The present paper in the first part will provide information about the EN-Eurocode 8 for Design of Structures for Earthquake Resistance and its limitations. After that will be presented adaptive pushover procedure for nonlinear analysis of irregular structures.

The APO theory will be used for the analyses of an existing structure. Firstly in the paper is given the information about the conditions of the existing structure. After that is performed the nonlinear analysis (adaptive pushover) and are given the structural measures suitable for retrofitting this type of structure.

In the end, the paper makes recommendations on 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, Adaptive Pushover Analysis*

INTRODUCTION

In recent years in Albania had been constructed numerous buildings both private and public but despite several attempts the design and constructions standards for reinforced concrete 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. In the article through the example of a building designed by the Albanian codes and retrofitted based on Eurocodes recommendations aimed to give the problems that could bring the use of existing codes and suggest to the entire community of engineers in Albania that is now essential to the designer of structures to apply not only European standards but to use the new methodologies as well.

The methodology of existing structures control and retrofit passes through the following stages:

- Dimensions and geometric data informations, 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 reinforcement with required reinforcement. If provided reinforcement is 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.

EXISTING STRUCTURE

The design of existing building is done in 2009. The building is divided in two separate structures. The first structure which will be analysed in the article will serve as offices premises. It has 2 above ground stories with irregular form in plan. The dimensions are about 8m with 24.3m at max point.

The first structure is divided in 2 main axes in longitudinal direction (one main span) and 5 axes (4 spans) in transversal direction. Columns dimensions are 40 x 40cm and 40 cm diameter. The side beams and transversal beams are 40 x 60cm, while the beams for the port between axis “3”and “4” are 30x40cm and 25x60 cm. The slab height is 17 cm. 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 “K T P 1985”. 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.

Figure 1. View of the structure

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 6 Schmid hammer tests as described in UNI EN12504-2 standard.

For the core tests we have used the correction given by Masi (2005) [7]

$F_{c,i}=(C_{h/D} \times C_D \times C_s \times C_d)f_{core,i}$

Where $C_{h/D}$ correction for h/D different from 2.
C_D correction for D different from 100mm
C_s correction for steel presence influation
Cd correction for core disturbance

From this expression can have the following characteristics
Concrete properties from tests

Self weight	γ=2455 kg/m^3
Cylinder concrete compressive strength	fck= 270 daN/cm^2
Cubic concrete compressive strength	Rck= 330 daN/cm^2
Design compressive, tensile strength	f_{cd} = 180 daN/cm^2
	f_{ctm} = 28.5 daN/cm^2
	$f_{ctk\ 0.05}$ = 19.4 daN/cm^2

Structure evaluation based on Eurocodes

As recommended by the Eurocodes[] and the reference documents, 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[5,7]:

- 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.

Seismic action

Seismic identification zone

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 seismic hazard should be given only with one parameter a_{gR} on ground type "A" that correspond to rock or rock like geological formations, including 5m weak formations (soil) at surface. The values of a_{gR}(maximum acceleration PGA) are taken from the Probabilistic hazard map of Albania recommended recently by a group of authors [6]. The return period of the reference event is T_R=475years that corresponds to a life time of 50years.

The horizontal PGA in ground type A for the city of Tirana is taken PGA=0,25g

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 behaviour factor "q". For the ultimate limit state for local soil conditions (ground type C) this factor is taken 3.2.

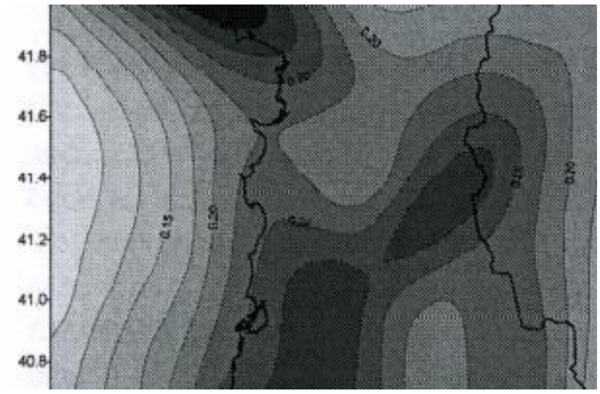


Figure 2. Peak ground acceleration Map of Albania

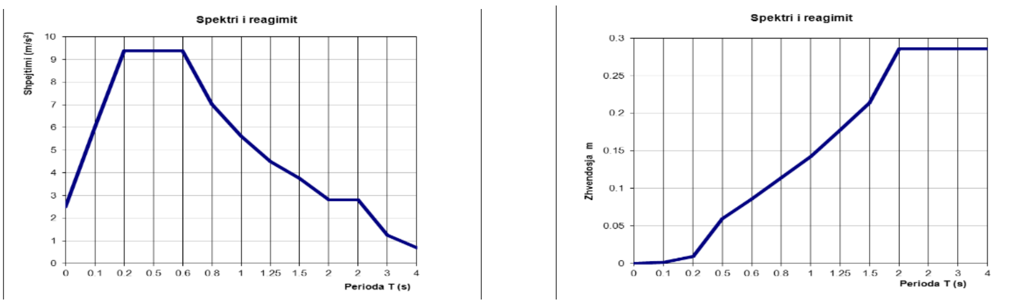


Figure 3. Graphical view of the elastic acceleration and displacement spectrum for soil type C

Dynamic 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 analyse and the determination of internal forces is done by spectral method with concentrated masses in the centre 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[4] is given below in fig. 3.

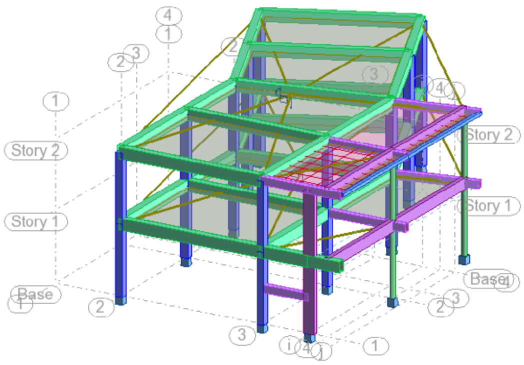


Figure 4. Graphical view of the linear model

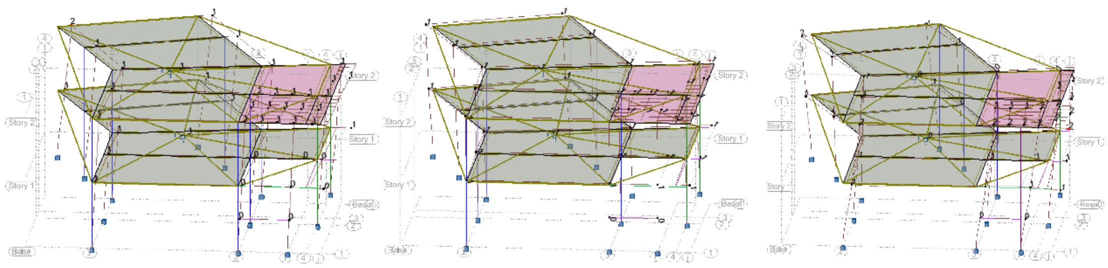


Figure 5. Graphical view of the three first modes

Table 1. Modal results of the structure

Case/ Mode	Frequency (Hz)	Period (sec)	Rel.mas. UX (%)	Rel.mas. UY (%)	Cur.mas UX (%)	Cur.mas UY (%)	Total mass (kg)
4/1	1.67	0.60	11.09	72.01	11.09	72.01	704610.46
4/2	1.68	0.59	80.78	88.76	69.69	16.75	704610.46
4/3	2.04	0.49	90.96	92.47	10.18	3.71	704610.46
4/4	4.71	0.21	91.08	98.70	0.13	6.23	704610.46

As seen from the modal results the structure is not regular and torsion influence its seismic behaviour. After the determination in advance the fragile or ductile behaviour for each element, with forces obtained from seismic combination is checked the strength of all the elements. From these results we can see that although nearly in limit the columns strength are assured (average safety coefficient is approximately around 1:03) 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.

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 torsionally 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.

In these cases must be used nonlinear dynamics analysis or adaptive nonlinear static analysis [3]. Hereafter are given some figures that for the case of our structure illustrate the encountered problems.

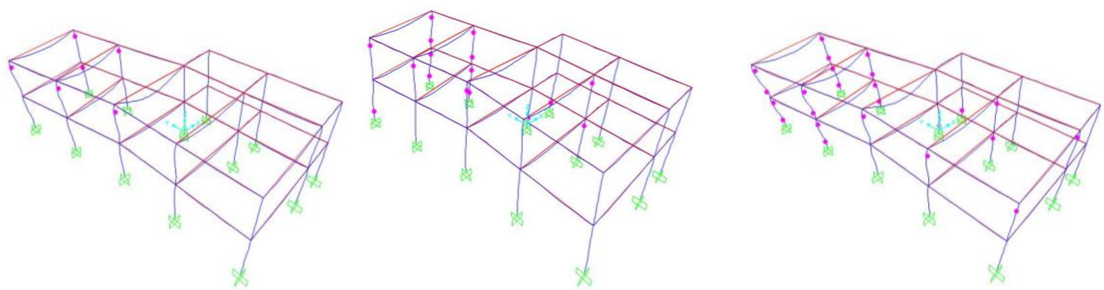


Figure 5. Graphical view of the structure for three incremental following steps

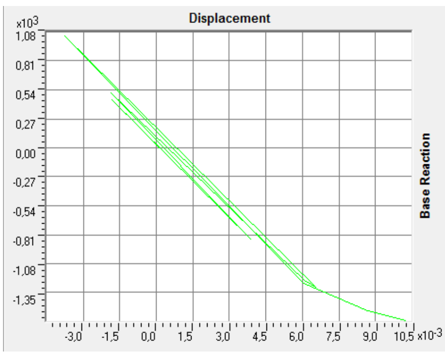


Figure 6. Graphical view of the base force versus displacement for the structure(capacity curve)

From the shown results can be clearly seen that the capacity curve, deformation base shear force relationship is not continuous in one direction and cannot be taken a performance point of the structure.

This means also that we can't determine the required maximum displacement and the corresponding base shear force with any of the known methods. (ATC40, FEMA 356)[2,1]

Consequently we cannot check the deformation capabilities (θ) or strength (M_{Rd},V_{Rd}) of the elements of the structure.

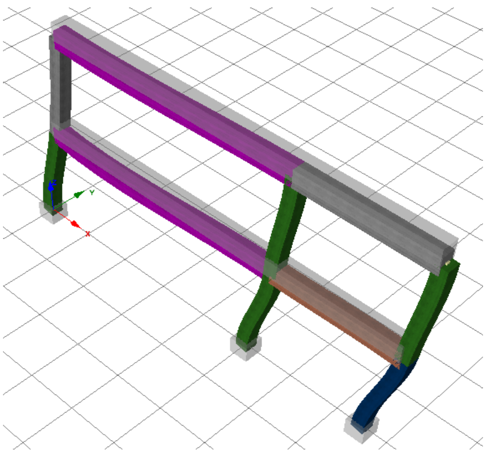
Adaptive static nonlinear analysis (Adaptive pushover)

In this analysis, the distribution of horizontal incremental loading isn't held constant but varies according to modal forms and participation factors obtained from the analysis of its eigenvalues forms after each load step [3,8].

The analysis can consider the degradation (Softening) of structure elements strength, the change of eigenvalues forms after each load step and change the internal forces due to spectral amplification.

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

The used methodology is quite similar to the classical nonlinear static analysis (PO) so in the figures below are given only some results of deformation capacity the strength check of the most loaded frame (frame B).



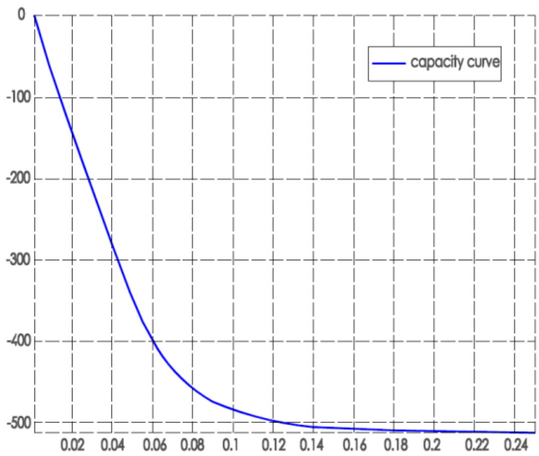


Figure 7. Graphical view of performance criteria check for frame B of the structure;
Figure 8. Graphical view of capacity curve only for frame B of the structure

From the curve of the relations in the function of displacement (incremental loading step) and the curve of the criterion of performance achieved for d_{max} is seen that the columns capacity to absorb the plastic deformation is greater than that of the beams so we have chosen to increase the flexural capacity of the beams and only confining the columns to achieve the performance that we have agree with the investor, that under the design earthquake the structure must achieve an acceptance criteria between Immediate Occupancy(IO) and Collapse Prevention(CP).

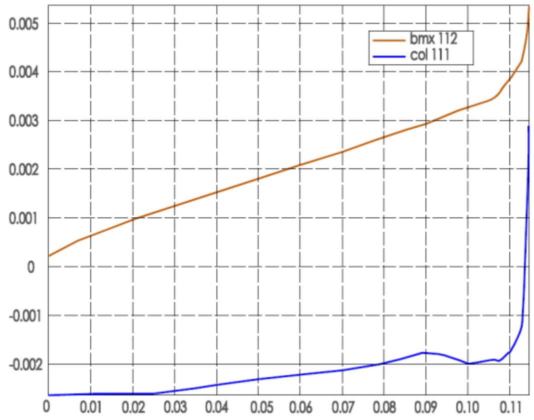


Figure 9. Graphical view of θ-incremental deformation for the side column and the first story beam.

Results

From the obtained results, as illustrated in the above figures the flexural strength of the beams is 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 recognises 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 the third type, increasing the 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 impossible 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.

Below are given the results for the most loaded frame (frame B).

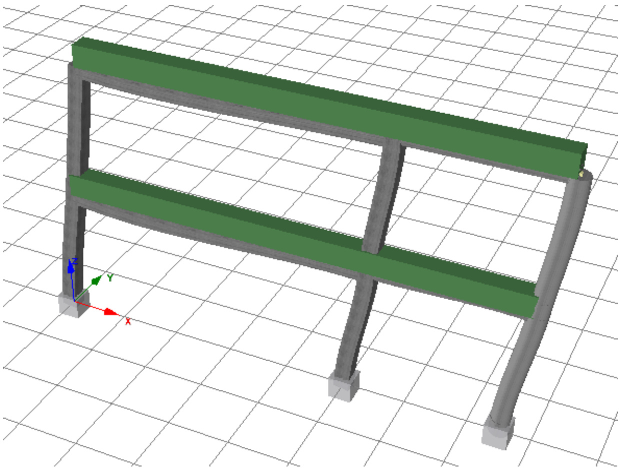


Figure 10. Graphical view of performance criteria check for frame B of the structure after the increasing of capacity of structural elements. (no performance criteria is reached)

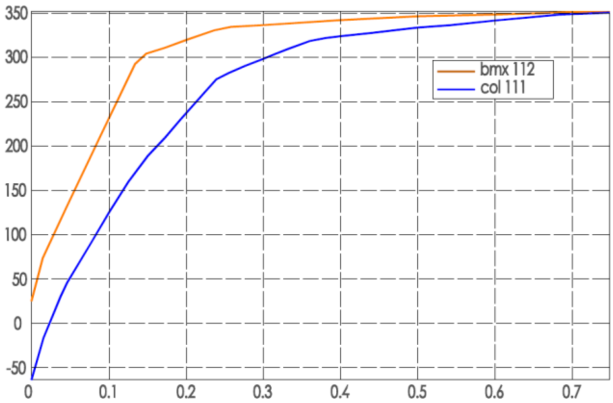


Figure 11. Graphical view of M-incremental deformation step percentuality for the side column and the first story beam after the increasing of capacity of structural elements

CONCLUSION

The use of Adaptive pushover analysis is highly efficient and the results are consistent and close to the results from nonlinear dynamical analysis. But the use of this method remains limited to specialized software and modelling requires plenty of time and care. If these analysis will not be implemented in ordinary commercial software the use of nonlinear dynamic analysis, although time-consuming will continue to remain the most widespread method for the calculation of torsionally eccentric structures.

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