Damage limitation requirements according Eurocode 8 for flexible reinforced concrete low-rise buildings

Altin Seranaj [1], Forcim Softa[2], Agim Seranaj[3], Fanjola Koroveshi[4]

[1] *Department of Building Construction and Transportation Infrastructure, Polytechnic University of Tirana,*
[2]*Department of Mechanics of Structures, Polytechnic University of Tirana, Albania*
[3]*Department of Mechanics of Structures, Polytechnic University of Tirana, Albania*
[4]*Department of Civil Engineering, Epoka University, Albania*

ABSTRACT

This study presents an analytical assessment for the reinforced concrete frame constructed with flat beams. The typology is known as frame with “embedded beam in the slab”, with a total height 25cm. Many similar structures have been constructed all around Balkans regions and Middle East. Sometime these structures have only in the perimeter the normal beam (thickness more than the slab). As it is well known the main advantages of using this type of reinforced concrete frames are that, they provide both, low economical cost for initial construction and a good architectural flexibility. Anyway in some literature, these types of structures are not recommended as being the only system for seismic forces resistance of a building.

The response of the frame with flat beams is compared with two other structural typologies, classical frame and uncoupled wall systems. Structures with 3 and 5 floors are selected for this analyse. The effect of infill masonry hallow clay bricks is also considered.

The criterions of comparisons are based mostly on Eurocode 8 and especially on damage limitation states. This criterion is chosen in this study due to the fact that this parameter is connected not simply to deformation but to the financial loss to. This investigation is made by using software ETABS. The investigation as was expected show that the frame with flat beams has much deformation than the classical frame and the uncoupled wall systems. The interstory drifts are much difficult to accommodate as per Eurocode 8 requirements. When infill walls are symmetrical in plan and well distributed in height the effect is positive in all structures. When are not continues on the ground story only the uncoupled wall systems represent consistent interstory drifts. Two other structural typologies are similar on their ability to present the negative effect of soft story.

Keywords: *Flat beams, Hallow brick masonry infill, soft story, damage limitation, construction cost.*

INTRODUCTION

In the last decades, huge numbers of the reinforced concrete structures are constructed using beams inside reinforced concrete slabs made will hallow clay/concrete/polymers block. However, in most cases, due to lack of deep strong beams, which can form with columns strong frame actions, the resulted transverse stiffness may be low. This may lead to potential damage even when subjected to earthquakes with moderate intensity.

These types of structures are representing low initial cost for construction and very good architectural flexibility on buildings, but on the other side this structures are flexible and the deformation that follow the earthquake response even for moderate earthquake cane cause a lot of damages on non-structural elements and so, an important economical loses.

Substantial research efforts have been devoted to investigating the performance of engineering structures during earthquakes such as reinforced concrete buildings, minarets, masonry and wooden buildings. It was reported that hundreds of thousands of buildings suffered different types of damage during these earthquakes.

This study presents an analytical assessment for the reinforced concrete frame constructed with flat beams. The depth of the beams on the examples studied in this article is 25cm, as normal practices in Albania. Typical buildings are shown on Figure 1.

Figure 1 Reinforced concrete frame structure with embedded beams into the slab

Their reaction under earthquake forces is compared with the reaction of classical reinforced concrete frame and wall systems. We have analyzed also this structures even taking into consideration the effects of infills with hallow bricks. According the Eurocodes requirements, the designer have to take into consideration the infills during the analysis on the cases they have a negative influence on the reaction of the structures. In the literature the infills are called, as non-structural elements. This may lead to big problems, not only to the other professionals, but even to the civil engineers. They calling so (non-structural) would lead to do “dangerous” modifications during the constructions of building, due to the changing of the considerations that the designer have taken during the design.

On many recent earthquakes on Turkey, Italy, Taiwan, Haiti, Indi, Pakistan, Algeria, Greece etc., have shown that the mistakes made during the design and construction due to the negative influence of infill walls have led to considerable damages.

Due to the similarity of these typologies of structures we are here below on the figure presenting some photo from the structures affected by the earthquake “Izmir (Kocaeli), Turkey earthquake, Aug. 17, 1999, with intensity 7.4 Richter [1]. Typical buildings are shown on Figure 2.

The same types of structures are constructed these last 20 years in Albania. The future strong earthquake will come and similar pictures as above (even worse) we may see after that earthquake (if we will). An urgent need is to emphasize the problem that the flexible structures present and after increasing the requirement on the design and constructions have to be decided. Central and local authority have to collaborate in efficient way in order to do vulnerability analysis and other studies in order to construct a sustainable strategy for earthquake disaster preparedness and management.

Figure 2 Reinforced concrete frame structure with embedet beams into the slab, after earthquake Gölcük, Turkey.

2 DESCRIPTION OF THE BUILDINGS

The investigated buildings are 3 and 5 floors multi-story reinforced concrete structure with beams depth 25 cm embedded on slabs. For the normal classical frame and wall systems the beams have the normal high 60cm. The elevation of the building and the floor plan are shown in Figure 3 and Figure 4. The ground floor is 4.2m height and for other story height is 3.15m.The total height of the building above the basement is 10.5 m for the structure with 3 floors and 16.8m for the structure with 5 floors. The dimensions in plane are the same for both buildings 13.8x10.4m.

Footings with tie beams represent the foundation. Concrete C25/30 is used. The corresponding modulus of elasticity amounts to Ecm= 31GPa (EN1992/Table 3.1). Steel S500 Class C is used. The structure will be designed for ductility class DCM.

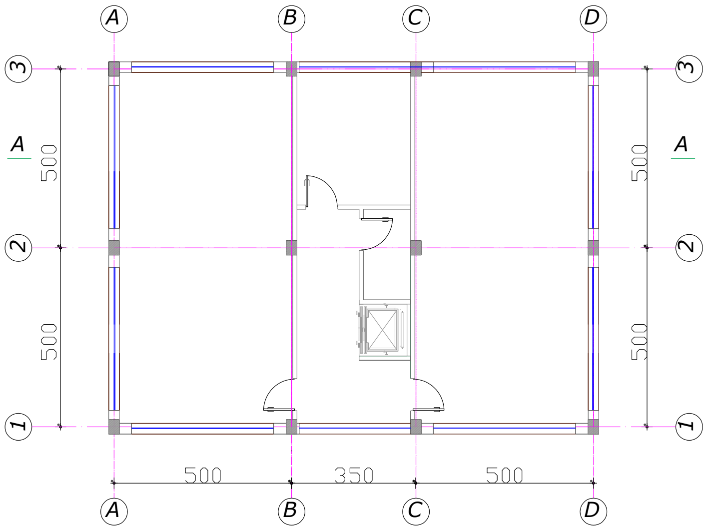

Figure 3 Plane of the model (3 floors and 5 floors)

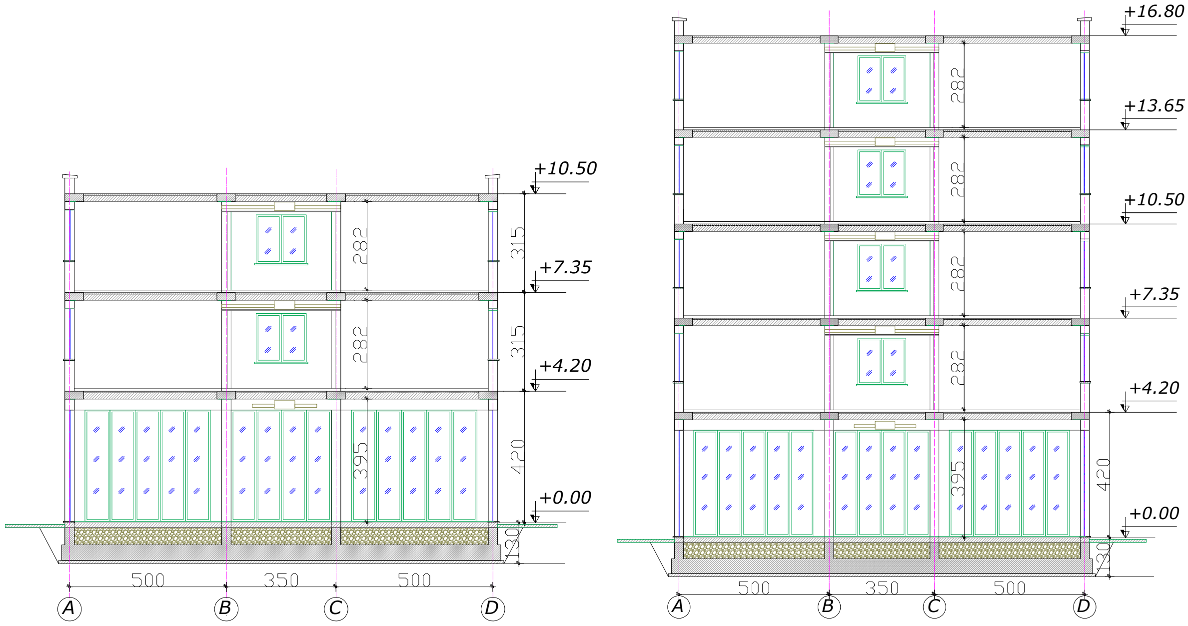

Figure 4 Section of the model (3 floors and 5 floors)

3 VERTICAL LOADS AND SEISMIC ACTION

The permanent loads "G" are represented by the self-weight of the structure and additional permanent load uniformly distributed equal to 3 kN/m2.. In the case of investigated building (which represents an office building – category B (EN 1991/Table 6.1)), the variable-live load in terms of uniformly distributed load amounts to 2kN/m2 (EN 1991/Table 6.2). The variable-live loads are, in a seismic design situation, reduced with a factor of $\Psi_{2i} = 0.3$ (EN 1990/Table A.1.1).

Based on the unit weight of the masonry and plaster a load 9 kN/ml is considered only over the perimeter beams. The self-weight of the vertical and horizontal elements (columns and beams) are automatically generated in program ETABS [2].

For the design of the building the design response spectrum is used (i.e. elastic response spectrum reduced by the behavior factor q).

The reference peak ground acceleration amounts to a_{gR} = 0.25g. The values of the periods (T_B, T_C, T_D) and of the soil factor (S), which describe the shape of the elastic response spectrum, amount to T_B = 0.15s, T_C = 0.5 s, T_D = 2.0 s and S = 1.2 (EN 1998-1/Table 3.2). The building is classified as importance class II (EN 1998-1/Table 4.3) and the corresponding importance factor amounts to γ_I = 1.0 (EN 1998-1/4.2.5(5)P). Therefore the peak ground acceleration is equal to the reference peak ground acceleration $a_g = \gamma_I * a_{gR}$ = 0.25g. Using the equation in EN 1998-1/3.2.2.2 the elastic response spectrum was defined for 5% damping.

The floor masses and mass moments of inertia are determined according to EN 1998-1/3.4.2. Complete masses resulting from the permanent load (self-weight of the structure + 3 kN/m 2) are considered, whereas the masses from the variable-live load are reduced using the factor $\Psi_{Ei} = \varphi \cdot \Psi_{2i}$. Factor Ψ_{2i} amounts to 0.3 in the case of an office building (EN 1990/Table A.1.1).

4 STRUCTURAL MODEL

A three-dimensional structural model is used. The basic characteristics of the model are as follows: All elements are modeled as line elements. Rigid offset for the interconnecting beams and columns elements are taken into account. All elements are fully fixed in foundation Frames and walls are connected together by means of rigid diaphragms (in horizontal plane) at each floor level. The elastic flexural and shear stiffness properties are taken to be equal to the uncracked elements. Infills are considered in the model on the specified cases.

On the cases were the infill walls with hallow bricks is taken into consideration the equivalent struts concept is used [3]. Adiferent width are sugested on the literature for the equivalent struts. We have considered 0.25d (d- length of the struts) and modul of elastiity Ecm = 1.6GPa and poison coeficient i ν= 0.15. Here below on figure 5 is presented the 3d model constructed and analised with ETABS program.

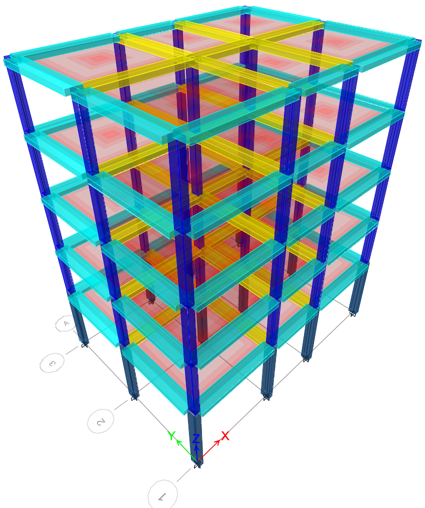

Figure 5 Mathematical models 3d. R/C frame with embeded beams in to the slabs.

5 DETERMINATION OF BEHAVIOR FACTOR

The mathematical model is needed in case of wall systems, for the determination of the structural type of the building. In our case the constructed model have a shear resistance over 65% of the total [4]. So the wall system is defined.
The behavior factor q for each horizontal direction is calculated by equation (EN 1998-1/5.1)

$$q = q_0 \text{ x } k_w \qquad (1)$$

where q_0 is the basic value of the behavior factor and k_w is the factor associated with the prevailing failure mode in structural system with walls. In case of frame systeme acording we have: $q = 1\text{x}q_0$ were $q_0=3*\alpha_u/\alpha_1=3*1.3=3.9$ so: q=1x3.9=3.9.
In case of uncoupled wall system in each of the two horizontal directions and will be designed as a DCM (Ductility Class Medium) structure.

6 MODAL RESPONS SPECTRUM, PERIODS AND NUMBER OF MODS FOR EFFECTIVE MASSES

The first nine modes have been sufficient to satisfy the requirements in EN 1998-1/4.3.3.3(3) (the sum of the effective modal masses amounts to at least 90% of the total mass). For each studiet case we have used the short definitions as below:
FSC- for Frame system clasical (normal beams). FSCW- for Frame system clasical (normal beams) but considering infill walls over ground floor. FSCTW- for Frame system clasical (normal beams) but considering infill walls Total stories. FSE- for Frame system clasical (embedet beams on the slab). FSEW- for Frame system clasical (embedet beams on the slab) but considering infill walls over ground floor. FSETW- for Frame system clasical (embedet beams on the slab) but considering infill walls Total stories. UWS- for uncapled wall system clasical (embedet beams on the slab). UWSW- for uncapled wall system clasical (embedet beams on the slab) but considering infill walls over ground floor. UWSTW- for uncapled wall system clasical (embedet beams on the slab) but considering infill walls Total stories. On the Figure 6 are presentet the modes of vibrations.

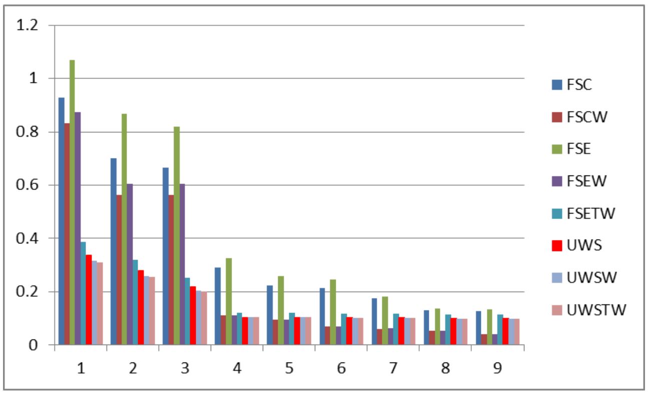

Figure 6 Modal periods for the models with 3 stories

From the above Figure we can conclude that the structure with the reinforced concrete frame that has the beams embeded nto the slab is very flexible. Even the structur with the normal frame is flexble to. This conclusion can be easly drawng if we refrer to the easy conservative expresion for modal periods determination as: **T= 0.1xN** , were N is the floor number. In case the infills are considered over the ground floor, normaly the periods will be rediused but still they

are fare from the normal values. Only wen the infill walls are considered on the total building elevation the values of the periods came closer to the normals. The uncapled wall system also present normal value of the periodes for all the cases studiet.
Here on the Figure 7 below are presentet the periods for the models with 5 floors.
The same comment are valid as above for the models with 3 floors.

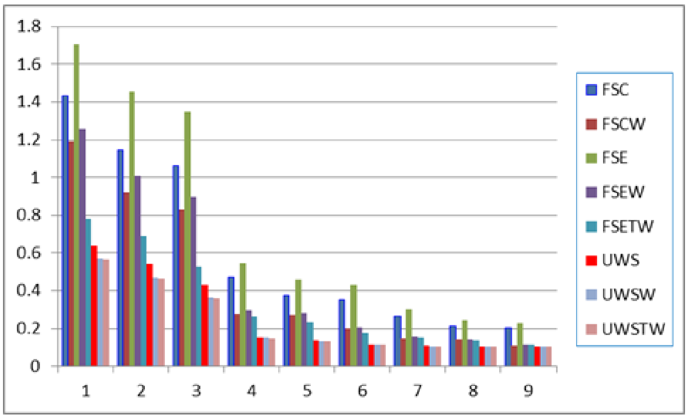

Figure 7 Modal periods for the models with 5 stories

7 DAMAGE LIMITATIONS REQUIREMENS

The damage limitation requirement should be verified in terms of the interstorey drift (d_r) (EN 1998-1/4.4.3.2) using equation

$$d_r \cdot \nu \leq \alpha \cdot h \qquad (2)$$

Story drift d_r is evaluated as the difference of the average lateral displacements d_S in CM at the top and bottom of the story. The analyzed building is classified as importance class II and the corresponding reduction factor ν amounts to 0.5 (EN 1998-1/4.4.3.2(2)). α is factor which takes into account the type of the non-structural elements and their arrangements into the structure. It amounts to 0.005, 0.0075 and 0.01 (EN 1998-1, equations 4.31, 4.32 and 4.33). For a clear presentation are presented with line the three values of α. It can be seen that the most severe drift limit (α = 0.005, for building having non-structural elements of brittle materials attached to the structure) is exceeded. The deformation of structural members is connected also to all the elements nonstructural, equipemets etc as they are mey be connected together. For this reasons this limitations are important to the importance not only on the financional los but also to the los of services of inportants equipement ect.

First we have presented the comparision betwin the three type of structures FSC, FSE and UWS. Due to the similarity of results and the limited page number of the articles we are presenting here the results only for the X- direction.

Easly from the Figure 8 we can se that the damage limitation requirement acording Eurocode is much dificultt to be acomodated form the case of the structure FSE. If for the first floor the infills have to be on the ductile nature for FSC and FSE structures for the first floor anly the FSE structure have this requirement. The other type UWS show a good performance due to the stiffnes presented by r/c walls.

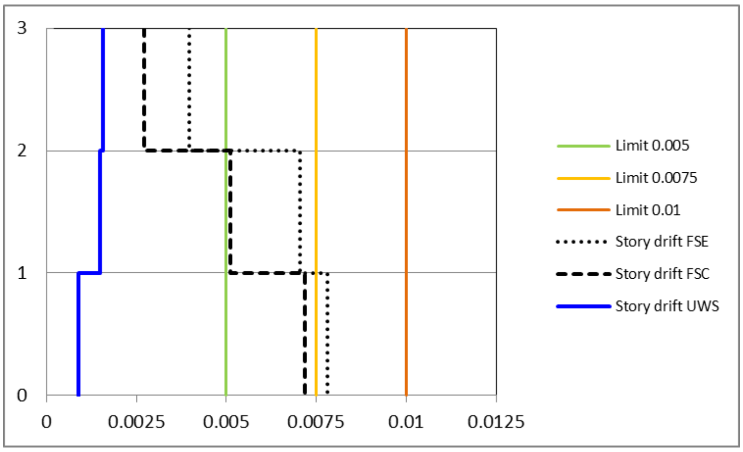

Figure 8 Comparisons of the models FSE, FSC and UWS

Below are presented on Figure 9 the cases were is taken into consideration the infill walls with hallow bricks (without in ground floor). The structures FSCW are FSEW are presenting that the requirement of damages on the ground floor dos not correspond to the normal constraction practice in Albania (building having non-structural elements of brittle materials attached to the structure) [5] . In fact this case bring attention to the phenomeno of soft story created by the absence of the infills in ground floors. The other case of structure UWSW is presenting much consistent value of demages limitations. The financial los due to the deformations of the structures UWSW under a frequent earthquake will be not inportant. Even the structures are of low higt we se that the requarement of damages limi state are dificult to accomodate. For building that frame structures with embedet beams onto the slab the second criterion of design have much sensitivity than the normal frames.

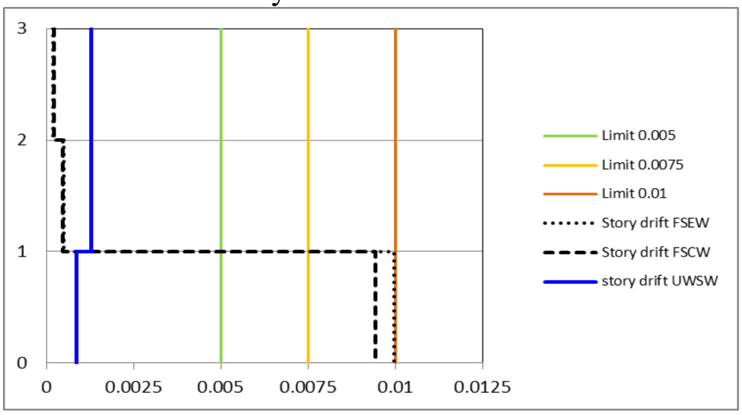

Figure 9 Comparisons of the models FSEW, FSCW and UWSW

For the models were are included in all the floors the infills the interstory drifts are similar. On the literature in fact is mentioned that due to the nature of the masonry infills (heterogenuy of element) this conclusion have to bee carfelly stated. Experiments has shown that the factors that influence on the total frame and infills reactions are to meny and this is still a subject that Enginiers are working on.

Below on the Figure 10 and Figure 11 are presented the results for the models that have 5 floors. The same coments as for the model with 3 floors are valid.

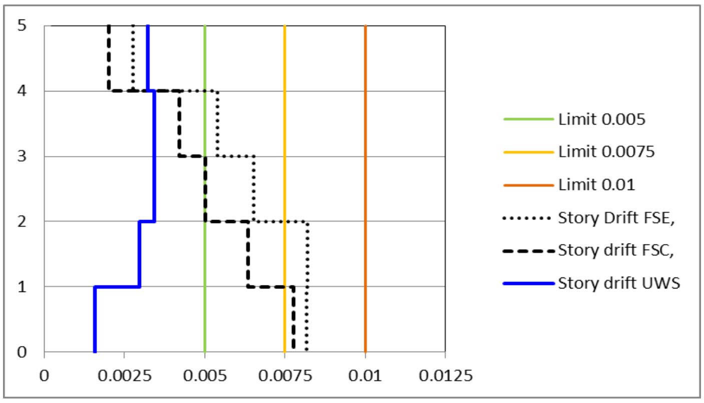

Figure 10 Comparisons of the models FSE, FSC and UWS

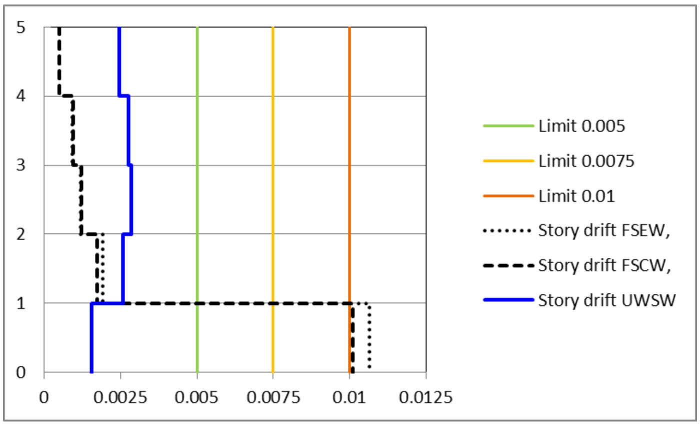

Figure 11 Comparisons of the models FSEW, FSCW and UWSW

8 SECOND ORDER EFFECTS CRITERION P-Δ

The criterion for taking into account the second order effect is based on the interstorey drift sensitivity coefficient θ, which is defined with equation (EN 1998-1/4.4.2.2(2))

$$\theta = P_{tot} * d_r / V_{tot} * h \qquad (3)$$

where dr is the interstorey drift, h is the story height, Vtot is the total seismic story shear obtained by modal response spectrum analysis and Ptot is the total gravity load at and above the story considered in the seismic design situation (G + 0.3Q,). On the following figures we are presenting the results only for the direction x.

In the investigated buildings, the second order effects need to be taken into account, mostly in the ground floors and especially when the building dos not have infill walls on the ground floor. The 5 floor building analysis with the Frame with embedded beams in to the slab is more sensitive to the P-Δ. For wall system, in all the cases the requirements of the second level design are easy fulfilled. In the Figure12 and figure 13 below are presented only the results in x direction of the model with 5 floors.

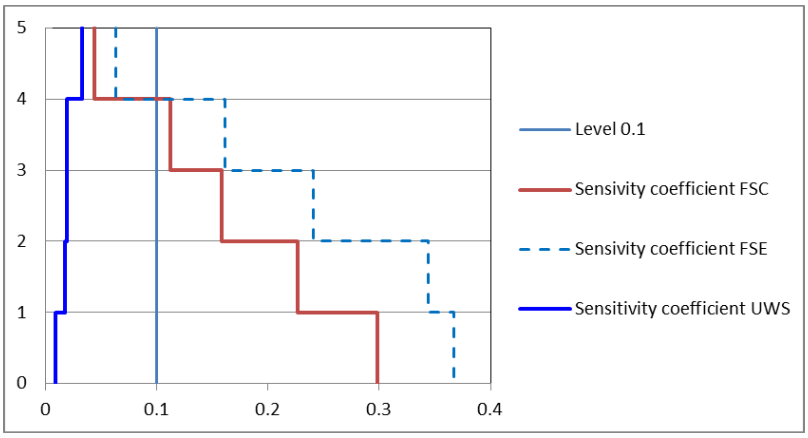

Figure 12 Comparisons of the models FSE, FSC and UWS

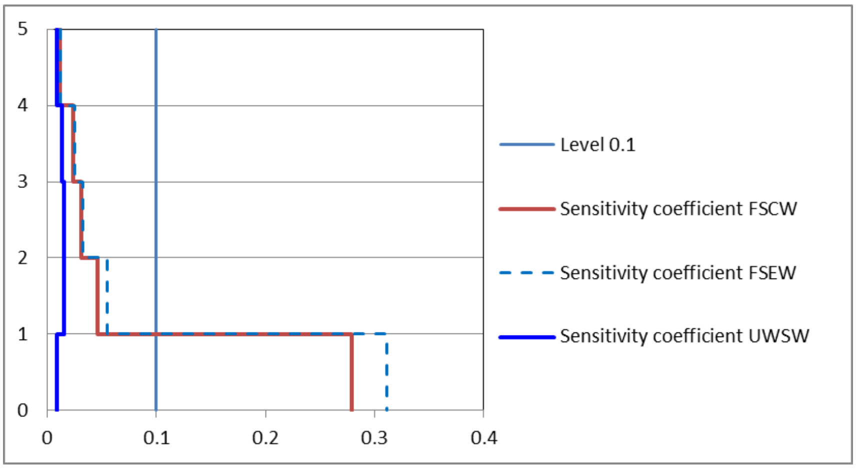

Figure 13 Comparisons of the models FSEW, FSCW and UWSW

For the 5 flor building the sensitivity coeficient hes considerable values. The request to increase the shear load designe on the vertical elements is biger for the frame with embedded beams into the slabs.

9 CONCLUSIONS

Based on the linear structural analysis of reinforced concrete frames with two different heights of models, 3 and 5 floors, we conclude:

Frame structures made of reinforced concrete with embedded beam have a higher flexibility than those with the classical frame with the normal beams. The UWS present consistent values for the modal periods on all studied cases.

Interstory drifts of the frame with embedded beams are the highest of the models studied. These facts bring attention to the fulfilment of the damages limitation requirement according eurocodes. The same situation applies with respect to control of the secondary P-Δ effect. The frame with embedded beams need a higher coefficient to apply for increasing the forces received from the linear analysis.

By consideration of the infills walls, it shows that they have the ability to substantially modify the structure response.

Based on this analysis and on other studies made about this topic, as well as technology implementation and experience in our country (Albania) we suggest that the terminology used for infill walls with hallow blocks “non-structural elements” is not appropriate [6]. It create

confusion in construction practice with the idea that modifications of infills can be easily made. Such modifications may be catastrophic in case of frame with beams embedded into the slabs due to the sensitivity that this typologies present.
Beside the life safety is the damage limitation requirement. For the same life safety fulfilment of different structure typologies different damage limitation capabilities (of course all they should accommodate the minimums required by codes) may be accommodates. For investors is important to know how their investment is exposed to the negative effects of the frequent earthquakes.

REFERENCES

[1] "Reinforced Concrete Frames and Wall Buildings" by H.Sezen et al. Structural Engineering Reconnaissance of the Kocaeli (Izmit) Turkey Earthquake of August 17 1999. Berkeley: Pacific Earthquake Engineering Research Center, (PEER Report 2000-09), December 2000

[2] ″ ETABS (2013)", Nonlinear version 13, Extended 3-D analysis of building systems Computers and Structures, Inc.Berkeley, U.S.A

[3] ″ Conceptual Seismic Design Guidance for New Framed Infill Buildings" Shabnam J. Semnani, Stanford University; Janise E. Rodgers, GeoHazards International; Henry V. Burton, Stanford University

[4] ″ Eurocode-8(2004): Design of structures for earthquake resistance" The European Committee for standardization, Final Draft, December 2004

[5] ″ Inxhinieria sizmike " /Niko Pojani Botime Toena 2003. ISBN 99927-1-747-5

[6] Seranaj A., Kokona E., Balilaj M.,″ REAGIMI I RAMAVE FLEKSIBILE PREJ BETONI TË ARMUAR NËN VEPRIMIN E FORCAVE SIZMIKE " / Buletini i shkencave teknike, Universiteti Politeknik i Tiranës. Albania