BASE SHEAR REDISTRIBUTION BETWEEN THE R/C DUAL SYSTEM STRUCTURAL COMPONENTS

V. Sigmund[1] and I.Guljas[2] and M.Hadzima-Nyarko[3]

[1] *Professor,* [2] *Assoc. professor* [3] *Assistant*
University of Osijek, Faculty of Civil Engineering,Drinska 16a,31000 Osijek, Croatia
Email:sigmund@gfos.hr

ABSTRACT :

The experience provided by the recent earthquakes showed that buildings with structural walls have had satisfactory performance for collapse prevention. Conversely, some of the dual system buildings (wall frame) were rated as severely damaged therefore implying an urgent need for strength assignments redefinition between the structural components. The elastic response of dual buildings is mainly governed by the wall response and the frame contribution could be neglected (bellow 15% of the base shear what makes them a secondary member). In the inelastic range the situation could drastically change and frame contribution increases (even up to the 50%) but these elements are then under-designed and not capable to carry the increased load.

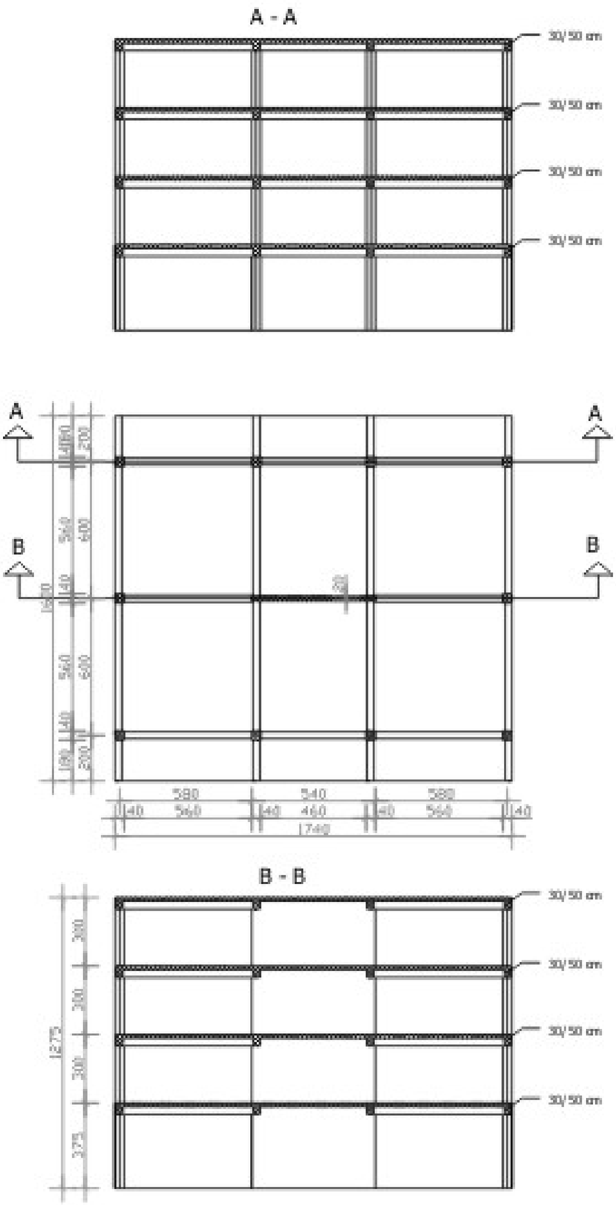
A set of dual building structures were selected, designed and analyzed. The model buildings were 4, 7 and 10 stories high with two frames and one wall-frame in longitudinal and four frames in the transverse direction. Column dimension varied thus causing stiffness and strength redistribution among the wall and frame, in elastic and inelastic range. Seismic analysis including elastic, nonlinear static (push-over) and nonlinear dynamic analysis have been done. Observed were base shear distribution, displacement and ductility demands of the wall and frames through system degradation. The results have shown that wall contribution diminishes as deformations are increasing and its share could go up to almost 50% what required high ductility of the frame elements. That they do not have if they were originally designed as “secondary” seismic elements according to the material Eurocodes.

KEYWORDS: r/c dual system, Eurocodes, earthquake, primary and secondary seismic elements

1. MODEL BUILDINGS

Modern seismic codes (EN 1998-1) make a distinction between the primary and secondary seismic elements, as far as their role and contribution to earthquake resistance of the building is concerned. “Primary seismic members” should be modeled in the structural analysis, designed and detailed for earthquake resistance. Secondary seismic elements only need to satisfy the rules of the material codes plus the requirement that they maintain support of gravity loads when subjected to most adverse displacements and deformations induced in them in the seismic design situation.

In the dual wall-frame buildings, wall easily, due to its higher stiffness in the elastic analysis, attracts higher portion of the horizontal loads. That makes it a “primary seismic element” and frames are designed for practically no horizontal loads. When the frames are exposed to expected non-linear displacements of the system then induced forces are much greater than the ones obtained during the linear elastic analysis. If the frames were designed for these forces than they are no longer “secondary seismic member”. So, the distinction between primary and secondary elements is lost as well as the idea to simplify the design by not considering frames in the structural model for seismic analysis of the building.



Stories: 4				
		Wall/frame		
clmn (cm)	beam (cm)	b_{eff} (cm)	Wall length (cm)	Wall -thick (cm)
40x40	30/50	111,2	540	20
50x50	30/50	110,5	550	20
60x60	30/50	109,8	560	20
Stories: 7				
		Wall/frame		
clmn (cm)	beam (cm)	b_{eff} (cm)	Wall length (cm)	Wall -thick (cm)
50x50	30/50	110,5	550	20
60x60	30/50	109,8	560	20
70x70	30/50	109,1	570	20
Stories: 10				
		Wall/frame		
clmn (cm)	beam (cm)	b_{eff} (cm)	Wall length (cm)	Wall -thick (cm)
60x60	30/50	109,8	560	20
70x70	30/50	109,1	570	20
80x80	30/50	108,4	580	20

Table 1 Geometrical data of the models

Figure 1 4-story model building

In order to point this problem out, a set of model buildings has been defined, designed and verified. The model buildings represent the well-known “Tsukuba building” which in the lateral direction has dual structural system consisting of boundary frames integrally linked to the central structural wall (5.00*0.20m) by moment-resisting beams. In transverse direction the building is a pure frame building having three moment-resisting frames (Fig.

1). The model buildings have 4, 7 and 10 stories with standard loading for business/dwelling buildings (g=9 kN/m2 and q=3.5kN/m2) located in the IX seismic zone (with design PGA=0,3g) grounded on the ground type „B“. Frame-beam dimensions were kept constant while frame-column dimensions varied as follows: a) 40x40; 50x50 and 60x60cm; b) 50x50; 60x60 and 70x70cm; c) 60x60; 70x70 and 80x80cm for 4, 7 and 10 stories respectively. Wall/floor area is kept constant at 0.36% (wall is 5,00x0,20m). All buildings were designed by linear-elastic modal analysis methods according to the Eurocodes EN 2 and EN 8 and for high ductility (DC H) of their elements.

2. NON-LINEAR NUMERICAL ANALYSIS

Numerical model for calculating the nonlinear response of reinforced/concrete building structures was developed and is explained in detail in Lopez (1988). Structure is defined in terms of its geometry and moment-curvature relationship for its individual elements. The frame elements consist of the linearly elastic member with nonlinearities concentrated in two nonlinear springs at the member ends (Giberson, 1969). Nonlinear response of wall elements includes flexure and shear components. Each wall member consists of several subelements so that each subelement can be subjected to a different stage of inelastic action in order to allow inelastic propagation through the story height. Force-deformation relationship of the members for monotonically increasing loads have been defined using the **empirical approach**. This combination has been verified as the one giving very good results in Sigmund (2003). Shear force and shear displacement relationship were calculated by taking only reinforcement for shear carrying capacity of the section.

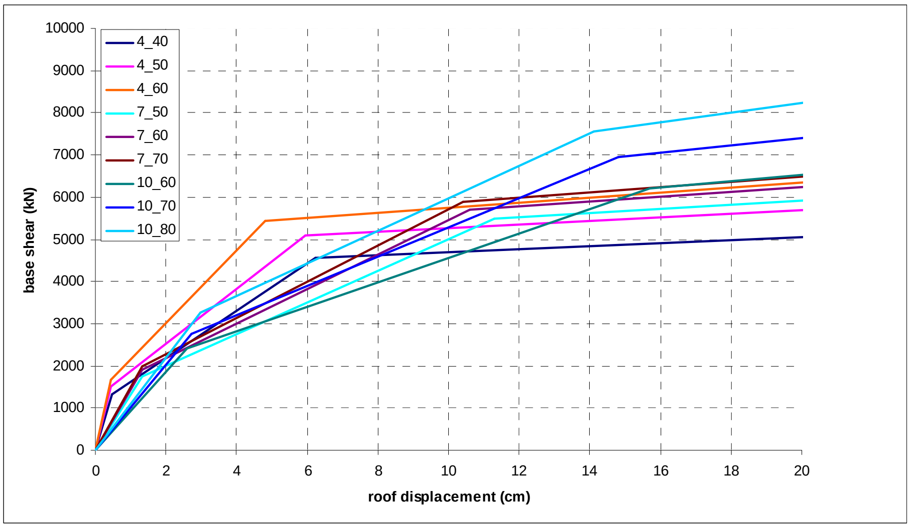


Figure 2 Base-shear Roof-displacement of the buildings

2.1. Non-linear static (push-over) analysis

For evaluation of the attained structural capacity static non-linear pushover analysis has been performed. Incremental horizontal loading was applied in a form that represents the first structural mode (triangular lateral load). Calculated relationship between the base shear and roof level displacement represents the overall nonlinear characteristics of the structure and its structural capacity.

The initial elastic stiffness of the structures did not differ significantly by increasing the frame-column dimensions which indicates that wall stiffness plays a major role in the elastic phase. Yield and post-yield stiffness increased from 10 to 15% . In the Figure 2 and Table 2 presented are the results of the push-over analysis for the 4, 7 and 10 stories building. Ultimate base shear is determined by the roof displacement equal to 2% of the building height. Base-shear at the crack opening is set as the 25% of the ultimate base shear. Generally, increase of the column cross-section (frame stiffness and strength) increased the overall structural lateral stability.

Wall-response dominated the overall structural behavior in all model buildings. Higher buildings (with 7 and 10 stories) and stronger columns have shown unfavorable local behavior in the wall and frame-columns.

Development of the plastic hinges and required local ductility is shown on the Figures 3, 4 and 5 for 4, 7 and 10 story buildings respectively. Plastic hinges were opening at the wall bottom (when the frame contribution is neglectable), but also on the first and second floor (for frames having stiffer columns) which could be very unfavorable. That required higher column ductility, which is not provided by default in a „secondary seismic member".

Table 2 Base-shear and Roof-level displacement characteristic values

Stories	**4**			**7**			**10**		
Base shear (kN); Roof displacement (cm)	**Column section**			**Column section**			**Column section**		
	40x40	**50x50**	**60x60**	**50x50**	**60x60**	**70x70**	**60x60**	**70x70**	**80x80**
BS_C (kN)	1321	1501	1681	1752	1902	2002	2402	2752	3251
BS_Y(kN)	4570	5090	5450	5480	5700	5900	6200	6950	7555
BS_U (kN)	5254	5945	6695	7058	7559	7959	9607	11008	13009
d_c (cm)	0,46	0,45	0,45	1,34	1,32	1,32	2,58	2,72	2,96
d_y (cm)	6,22	5,95	4,80	11,3	10,6	10,4	15,7	14,8	14,1
d_u (cm)	25,59	25,86	25,67	43,17	43,12	43,55	61,34	61,63	61,50

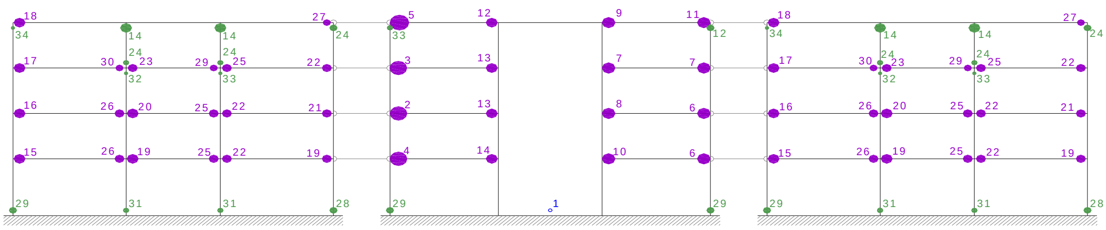


Figure 3 Opening of the plastic joints and required ductility, 4 stories-50x50cm columns

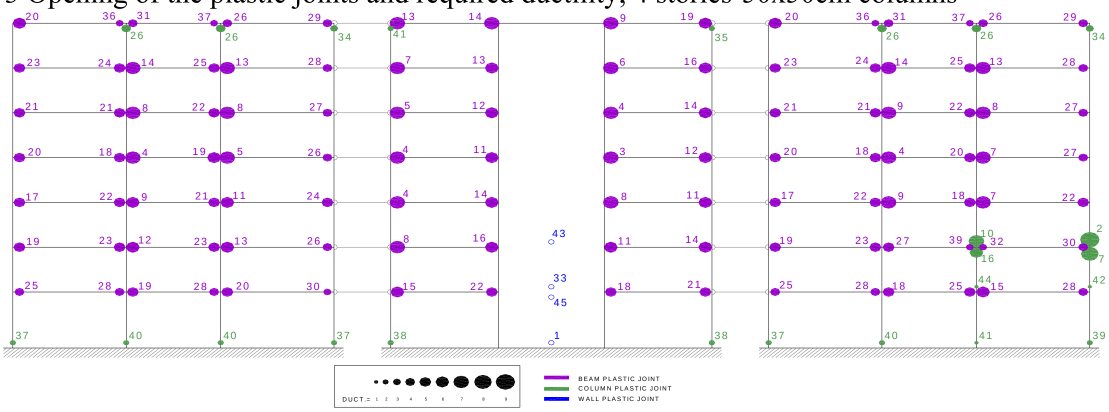


Figure 4 Opening of the plastic joints and required ductility, 7 stories-60x60cm columns

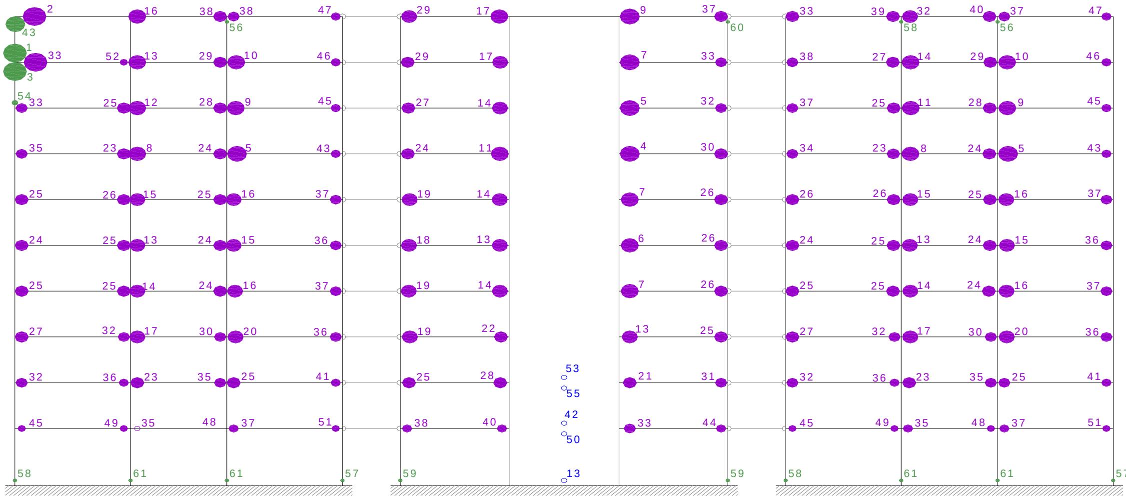
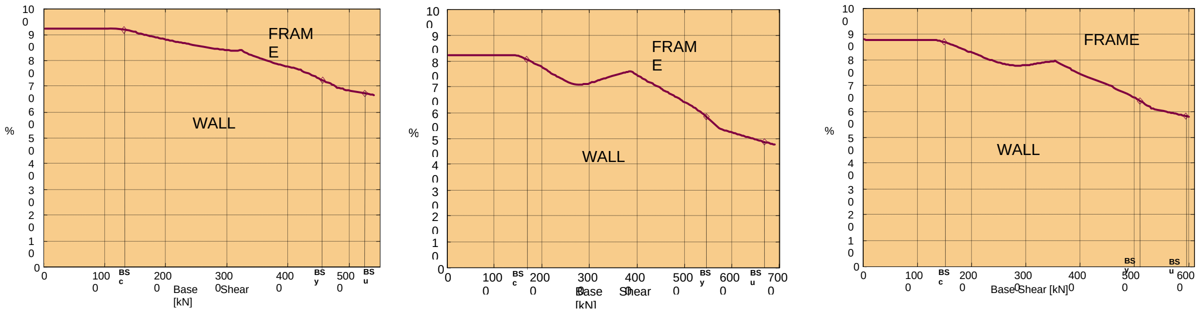


Figure 5 Opening of the plastic joints and required ductility, 7 stories-60x60cm columns

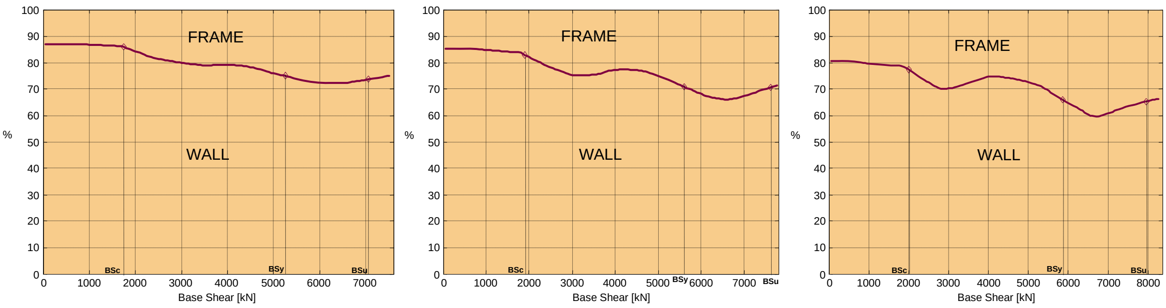
2.1.1 Distribution of the base shear among the wall and columns

Distribution of the base shear among the wall and frame-columns is visible on the Figures 6, 7 and 8. It is obvious that the wall contribution diminishes as the plastic deformation increases. In the elastic range wall takes from 83 to 93% of the toal base shear. At the ultimate state it takes only 48 to 63% of the base shear which means that frame elements do not represent a „secondary seismic member“ as inelastic deformation increases.



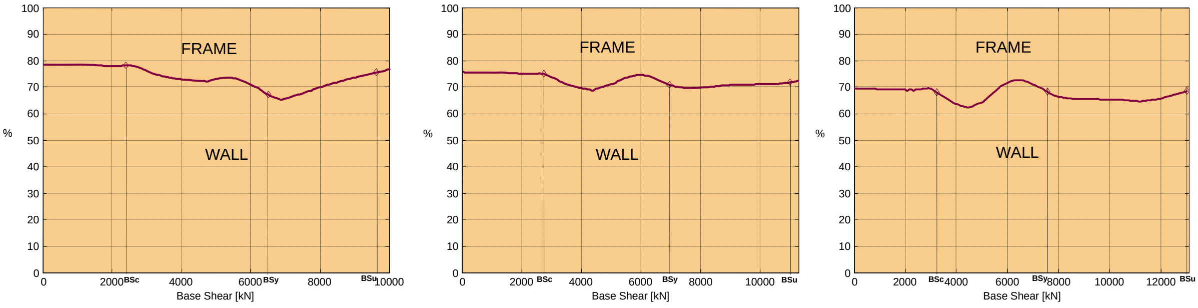
a) **b)** **c)**

Figure 6 Distribution of the base shear among the wall and columns for 4 story building and column dimensions of a) 40x40 cm; b) 50x50 cm; c) 60x60 cm



a) **b)** **c)**

Figure 7 Distribution of the base shear among the wall and columns for 7 story building and column dimensions of a) 50x50 cm; b) 60x60 cm; c) 70x70 cm



a) b) c)

Figure 8 Distribution of the base shear among the wall and columns for 10 story building and column dimensions of a) 60x60 cm; b) 70x70 cm; c) 80x80 cm

For 4-story model frame contribution in the elastic range was neglectable (from 8 to 19%). By approaching the inelastic deformation of 1.5% of the MDR its contribution increased to 33 to 52%.
For 7-story model frame contribution in the elastic range was from 14 to 22%. By approaching the inelastic deformation of 1.5% of the MDR its contribution increased to 26 to 34%.
For 10-story model frame contribution in the elastic range was from 21 to 31%). By approaching the inelastic deformation of 1.5% of the MDR its contribution remained about the same.
Observation of the capacity curves indicated that discrepancy between the elastic and inelastic frame contribution is bigger when its contribution, according to the elastic analysis, could be neglected.

2.2. Non-linear dynamic analysis

Nonlinear dynamic time history analysis of the models was calculated using LARZWD (1992) and a set of three recorded ground motions that covered the frequency range of interest: Bar N-S and Petrovac N-S recorded during the 1979 Monte-Negro earthquake and El Centro N-S recorded at 1940. Maximum accelerations were scaled so that ground motion spectral intensities were similar for the same earthquake zone (IX zone MCS). The set of three different ground motions was used.

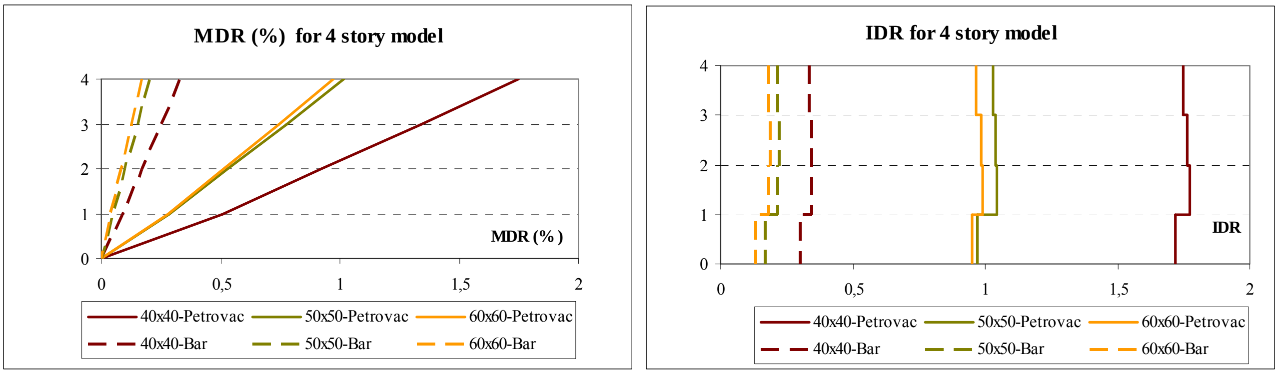


Figure 9 Calculated MDR and IDR of the 4-story model for Petrovac & Bar records

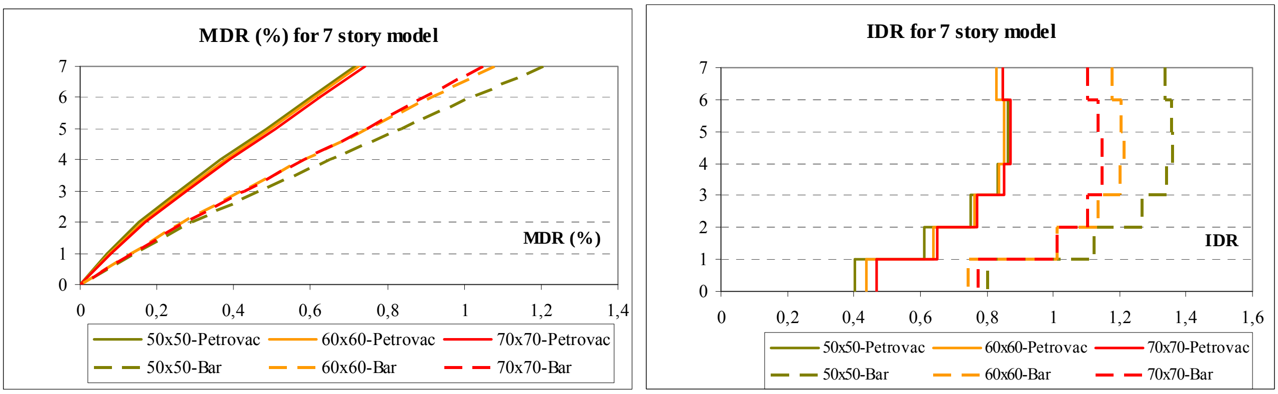
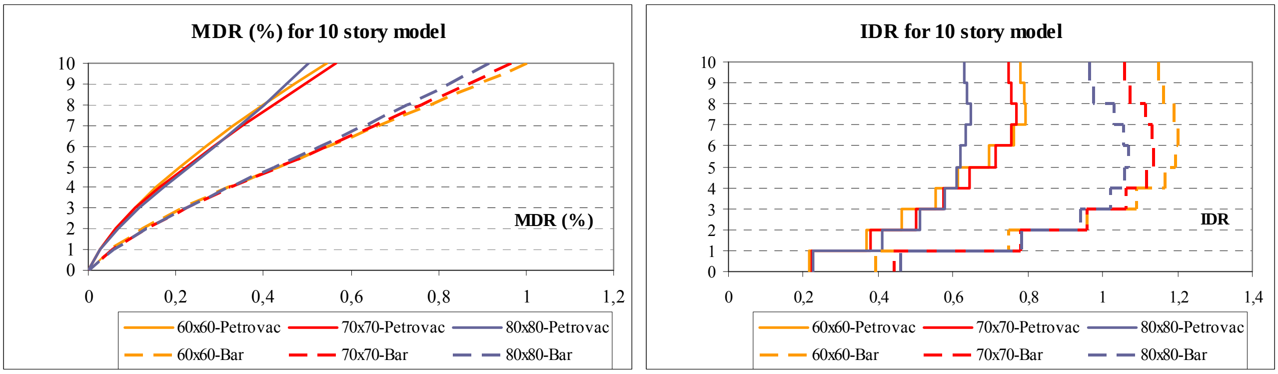
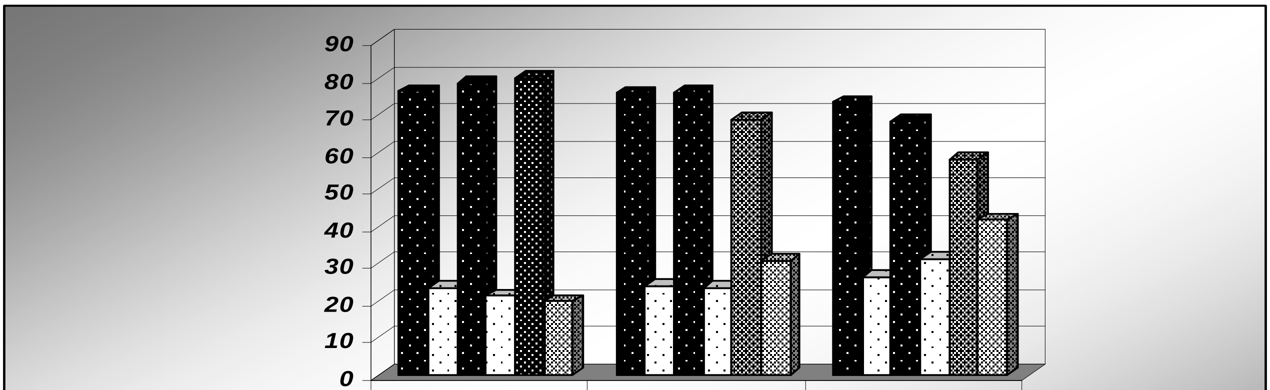


Figure 10 Calculated MDR and IDR of the 7-story model for Petrovac & Bar records





	60x60	70x70	80x80
wall10	77	76	74
frame10	23	24	26
wall7	79	76	69
frame7	21	24	31
wall4	80	69	58
frame4	20	31	42

Figure 12 Calculated distribution of the base-shear forces between the wal and frame-elements

During the inelastic dynamic analysis distribution of the base shear among the wall and frames depended on the frequency characteristics of the excitation that exciteed all modes. Frames took over from 20 to 42% for 4 story building, from 21 to 31% for 7 story building and from 23 to 26% for 10 story building in the average for all three excitations. Frame-element contribution has been higher in the structures that had undergone higher inelastic deformation (4-story building).

3. SUMMARY

In order to point out the problem in distinguishing between the „primary“ and „secondary“ seismic members in dula wall-frame buildings 9 model structures having 4, 7 and 10 stories have been designed according to the Eurocodes EN 1992 and 1998 by linear-elastic analysis and designed according to the EN 1998 capacity design rules. Wall/floor area was kept constant at 0,36%. According to the linear elastic analysis, the wall could be designed as „primary seismic element“ according to the EC8 rules, as it takes over from 85 to 94% of the total base shear for 4 and 7 story buildings. Therefore, the frame-columns could be designed as „secondary“ seismic elements for vertical loadings only and according to the EC2.
Non-linear static (push-over) and dynamic (time hostory) analysis of the model structures were done. As the structures were enetering into nonlinear range, frames were taking over bigger part of the base shear and the wall took over from 48 to 83% of the total base shear. Contribution of the higher modes, that was obvious in the non-linear dynamic response, contributed to the lower influence of the wall. Due to the frame-wall interaction, frame sections have had high ductility demand although they had been designed for gravity loadings only.
Therefore, it seems that in dual (frame-wall) buildingsa, there is no clear distincion among the „primary“ and „secondary“ seismic elements and that neglecting of the frame-elements contribution could endanger the overall structural stability. Although wall dominated the dynamic response, contribution of the higher modes started a pronounced interaction among the wall and frame-elements causing higher than expected non-linear behavior of the both.

REFERENCES

Sigmund, V., Bošnjak-Klečina, M., Guljaš, I., Stanić, A.. (2000), A comparisson between the buildings designed according to the HRN and EC8, *Građevinar*, **52:7**, 379-389.
Sigmund, V., Guljaš, I., Stanić, A.(2002)., Non-linear response of structures designed according to the HRN and EC8, *Građevinar*, **54:1**, 1-14;
EC8: (2004), Design of structures for earthquake resistance, *CEN*, Brussel, 2004;
Lopez, R., Sozen, M. A. (1988, 1992), LARZWD and LARZWS Computer Programs for Nonlinear Analysis of Planar Reinforced Concrete Structures Incorporating Frame and walls, University of Illinois, Urbana
Sigmund,V, Brana, P, Herman, K. (2000), Two methods for estimating nonlinear characteristics of r/c frame members, *The Third Japan-Turkey Workshop on Earthquake Engineering*, **Vol. 1**, 87-98., Istanbul, Turkey
Sigmund V., Herman, K., Guljaš, I.(2000), An evaluation of the displacement controlled design procedures, 12 World Conference on Earthquake Engineering, Auckland, New Zealand
Aoyama, H. (1981), A method for the evaluation of the seismic capacity of existing r/c buildings in Japan, NZNSEE, **14:3**
Y.Inada (1993), Force displacement models for solid shear walls of reactor buildings, Earthquake resistance of reinforced concrete structures, University of Tokyo, Japan
Stanić, A., Sigmund, V., Guljaš, I.(2003), Behavior of the walls under in-plane horizontal loadings, fib-2003 Symposium, Athens, Greece
Stanić, A.; Sigmund, V.; Guljaš, I.(2004), Seismic capacity of structural members using neural networks, XIII WCEE, Vancouver, Canada