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SEISMIC RESPONSE OF RC BUILDINGS WITH DIFFERENTLY REINFORCED SHEAR WALLS

Miloš Čokić*, Radomir Folić*, Boris Folić**

* *University of Novi Sad, Faculty of Technical Sciences, Trg Dositeja Obradovića 6, Novi Sad, Serbia, cokicmilos@gmail.com; folic@uns.ac.rs, r.folic@gmail.com*

** *University of Belgrade, Innovation Centre, Faculty of Mechanical Engineering, Kraljice Marije 16, Belgrade, Serbia, boris.folic@gmail.com*

ABSTRACT
A post-elastic behaviour of buildings with reinforced concrete (RC) walls is analysed, using the method of nonlinear static analysis. Two models, both with different RC wall reinforcement detailing, are used for the comparative analysis of the building behaviour. Walls are modelled as column-rigid beam elements with fiber nonlinear hinges, but with different distribution and quantities of the reinforcing steel in both model. The model with RC walls that have boundary elements is chosen as a referent model for comparative analysis. Based on the results of nonlinear analysis method, target displacement and inter-story drift values, some conclusions and recommendations are given.

KEY WORDS: Buildings, RC walls, seismic nonlinear analysis, reinforcing method

SEIZMIČKI ODGOVOR AB ZGRADA SA RAZLIČITO ARMIRANIM ZIDOVIMA

REZIME
Analiziran je sistem post-elastičnog ponašanja zgrada sa armirano-betonskim (AB) zidovima, primenom metode nelinearne statičke analize. Za uporednu analizu ponašanja zgrade koriste se dva modela, oba sa različitim detaljima armiranja AB zidova. Zidovi su modelirani kao sistemi stubova i krutih greda sa vlaknastim modelima plastičnih zglobova, ali sa različitim rasporedom i količinama armature u oba modela. Kao referentni model za komparativnu analizu, odabran je model sa AB zidovima koji imaju ivične elemente. Na osnovu rezultata dobijenih primenom metode nelinearne analize, ciljnih i međuspratnih pomeranja, date su odgovarajuće preporuke.

KLJUČNE REČI: Zgrade, AB zidovi, seizmička nelinearna analiza, metode armiranja

INTRODUCTION

Modelling of a structural system depend on the complexity of the system and the software package that is used. RC walls are generally modelled in the following two ways (Fig. 1):

- System of two frame elements – A column with geometric characteristics of the wall; a beam or link element with very high stiffness properties (Figure 1a). In this approach, the modelling process of nonlinear hinges is very complex. The accuracy of the results and calculation time are dependant to a certain extent on the modelling accuracy of nonlinear hinges;

- Shell elements. Walls are modelled as multi-layered shell elements (Figures 1b and 2). The accuracy of the results and calculation time are dependant to a certain extent on the density properties of 2D finite elements network.

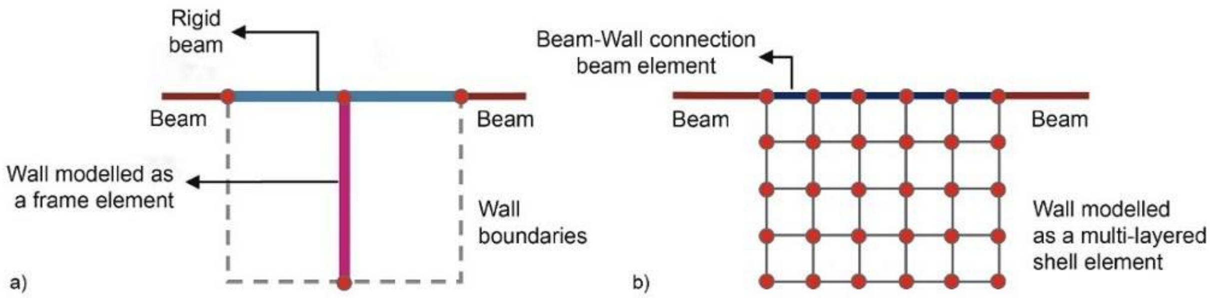

Figure 1. Wall models: a) frame elements system; b) shell element, after (Fahjan, et al. 2010)
Slika 1. Modeli zidova: a) sistem linijskih elemenata (stub – kruta greda); b) ravanski element (Fahjan i dr., 2010)

The modelling of walls as a multi-layered shell element or frame element system was analysed in papers (Kubin et al. 2008; Fahjan et al. 2010; Sukumar et al. 2016). The 3D model of the structure in (Ajmal et al. 2012) in (Fahjan et al. 2012) did a comparative analysis on plane (2D) model of a building frame. Based on the results of their research, it can be concluded that modelling of walls with frame elements will give similar results compared to structures with shell wall-element models.

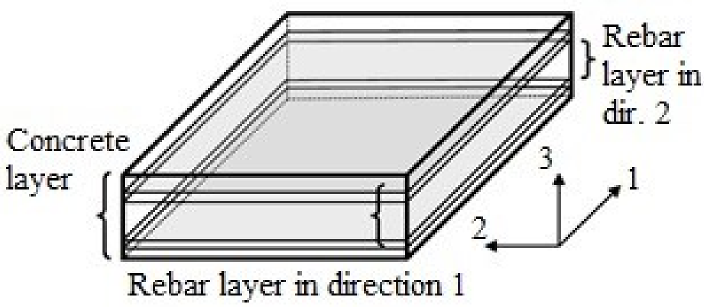

Figure 2. Multi-layered shell element, after (Fahjan, et al. 2010)
Slika 2. Višeslojni ravanski element, prema (Fahjan i dr., 2010)

In paper (Oh et al., 2002) the effect of boundary element details of structural walls on their deformation capacities was studied. Structural walls considered in this study had different sectional shapes of boundary element details, with barbell-shaped RC wall as well. Only rectangular RC walls were studied. As expected, wall specimen without boundary elements has shown the lowest performance, compared to other specimens. In paper (Darani & Moghadam, 2012) is investigated the effect of wall aspect ratio, axial force, and boundary element characteristics on the behaviour of low-rise shear walls and failure mode of walls.

Lu & Henry (2015) tested six walls to investigate the seismic behaviour of RC walls with distributed minimum vertical reinforcement in accordance with provisions in NZS 3101:2006. They developed detailed numerical models of lightly RC walls to understand the behaviour of the test walls, and to investigate the performance of walls with minimum vertical reinforcement. Results from these analyses showed that wall size, reinforcement type and concrete strength had a significant effect on the cracking behaviour and lateral drift capacity of RC walls.

In his paper (Milev, 2016), Milev discussed the problems and solutions in the design of RC wall structures, and among them, the local ductility requirements and checks after (EN1998-1, 2004). "*Local ductility of ductile walls can be ensured by providing the confined boundary elements in the critical zone of the wall. However the procedure for calculation of the length of confined boundary elements is complicated and is partly clear in Eurocode 8 even for the case of walls with rectangular cross section. In author's opinion the procedure is iterative even for the simple cases.*" (Figure 3)

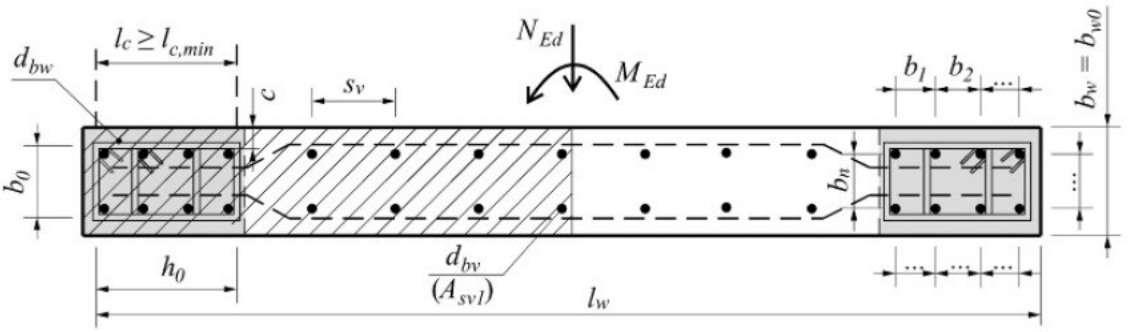

Figure 3. Rectangular cross-section RC wall with boundary elements (M1), after (Milev, 2016)
Slika 3. AB zid pravougaonog poprečnog preseka sa utegnutim ivičnim elementima (M1), prema (Milev, 2016)

In this paper, two mathematical models (M1 and M2) were used, in order to analyse and compare the behaviour of DCM wall structural system (EN1998-1, 2004) that contain RC walls with and without boundary elements.

The main difference between the wall M1 and M2 is in the different position and quantity of the rebar in wall-elements. RC walls in M1 are designed according to (EN1998-1, 2004), (EN1992-1, 2004) as the walls with boundary elements (Figure 3). M2 have RC walls without boundary elements and it is compared to the referent model M1. RC walls in M2 are designed as fully unconfined RC walls. With the exclusion of boundary elements, all other propositions given in (EN1998-1, 2004), (EN1992-1, 2004) were adopted in their design. Geometrical characteristics (length, width, height) of the walls are the same in both models.

The results of this analysis are used to compare the behaviour of a wall structural building system with - and without boundary elements in RC walls and the effect of equal reinforcement distribution in RC walls without boundary elements on the post-elastic behaviour of the structure. Nonlinear static analysis (NSA) was used in the analysis of the structural system behaviour. NSA method was used to perform pushover analysis. To fully observe the behaviour of the structures in post-elastic zone, global (GDR) and inter-story drifts (IDR) were used for the comparative analysis.

MATERIALS AND METHODS

The subject of the analysis is an office-residential building with 11 levels (basement, ground floor + 9 stories). The structural system of the building is a wall system (EN1998-1, 2004). The main structural elements of the analysed structure are RC slabs, walls, beams and columns. The raster of the structure is shown in Figure 4. The length of one span in the longitudinal (*X*) direction is 4.8 m (8x4.8 m total), and in the transverse direction (*Y*) 5.4 m (5x5.4 m total). The height of basement and the ground floor is 3.6 m, while the height of the other 9 stories is 3.2 m, so the total height of the building is 36.0 m. In order to simplify the modelling and calculation process, all vertical elements are fixed at the bottom level of the structure, i.e. soil-structure interaction is not included in the calculation and design.

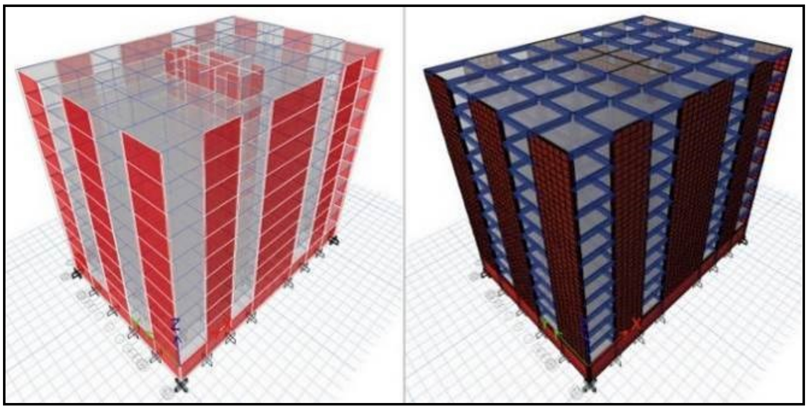

Figure 4. Analysis model
Slika 4. Model za analizu

Material properties of concrete C35/45 (EN1992-1, 2004) and reinforcing steel class C (f_{yk} = 500 MPa, k = 1.15) (EN1992-1, 2004) have been adopted for model analysis. The structure is designed for the medium ductility class (DCM) behaviour (EN1998-1, 2004). The structural design is done according to the European building design standards (EN1998-1, 2004), (EN1992-1, 2004) and (EN1991-1, 2002), and the calculations are performed using (ETABS, 2016). The structural behaviour is analysed by performing NSA and NDA methods. The N2 method (EN1998-1, 2004) is used for the calculation of target displacement values. The position of the walls in the structural system is shown in Figure 5. Geometric characteristics of the cross-section properties of the walls are shown in Table 1.

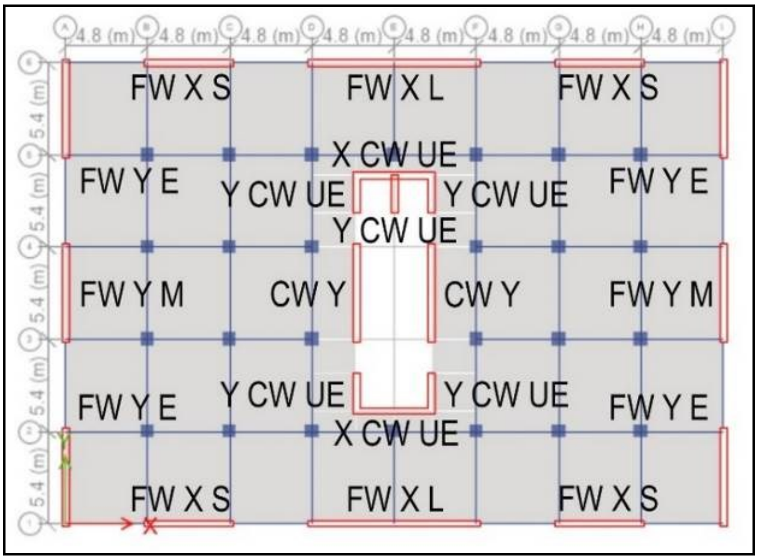

Figure 5. Walls position – plan view
Slika 5. Pozicije zidova – presek u osnovi

Table 1. Geometric characteristics of structural elements
Tabela 1. Geometrijske karakteristike konstruktivnih elemenata

Level	Basement – 10th story	Wall length on all floors	L_W (full/B.E/I.E) [m]
Plate: d_{pl} [cm]	16	FW X L	10.0 / 1.8 / 6.4
Beams: b_w/h [cm]	40/60	FW X S	5.2 / 1.0 / 3.2
Columns: d_x/d_y [cm]	80/80	FW Y E, FW Y M, CW Y	5.8 / 1.1 / 3.6
Walls: b_w [cm]	40	CW X UE	4.4 / 0.9 / 2.6
		CW Y UE	2.2 / 0.7 / 0.8

The calculations of the structure are done according to the methodology and recommendations given in (EN1998-1, 2004), (EN1992-1, 2004) and (EN1991-1, 2002).

The first structural model M1 is the referent model, where RC walls are modelled with boundary elements (Figure 3). The other model is used for comparative analysis with M1. In M2 walls are fully unconfined RC elements.

The applied loads are as follows: permanent loads (G_i) – self-weight of structural elements and an additional permanent load; live load (Q_i) and seismic load (S_i). The adopted value of the permanent constant load is g_{pl} = 3.0 kN/m^2 on all floors. The load intensity of the variable-live load amounts to q = 3.0 kN kN/m^2 (EN1991-1, 2002) on all floors, except on the roof slab at which the load intensity is equal to q_r = 1.0 kN/m^2 (EN1991-1, 2002). The self-weight load of façade elements, which is imposed on all façade beams except the roof façade beams is equal to g_f = 10.0 kN/m on beams and 3.0 kN/m on RC walls. The value of the reduction factor of the live loads is $\psi_{2,i}$ = 0.3 (EN1992-1, 2004).

To calculate the peak ground acceleration (*PGA*) action on the structure, an elastic response spectrum (RS), type 1 is used, for ground type C and the value of the $PGA = 0.2 \cdot g$ (EN1998-1, 2004). The adopted damping value is 5%. Eccentricity ratios of 5% for both directions are included. The maximum and the adopted value of the behaviour factor is q = 3.0 (EN1998-1, 2004).

ADOPTED PROPERTIES AND SIMPLIFICATIONS

A spatial (3D) model is used for the structure's analysis, which is conducted in (ETABS, 2016). The following parameters, assumptions and simplifications are adopted:

- RC plates are horizontally rigid diaphragms
- Second-order (P-Δ) effects are included in the calculation
- Cracked structural elements properties are included in the calculation

In addition to parameters, assumptions and simplifications that are used for all models, for the post-elastic analysis models, the following are used as well:

- structural elements are modelled with material properties for nonlinear behaviour of concrete (EN1992-1, 2004), (Mander et al., 1988) and reinforcing steel (EN1992-1, 2004) (Figure 6)
- The behaviour of RC is described by a Takeda hysteretic model and the Kinematic model of hysteresis was used for the reinforcement. Both models are an integral part of the software package (ETABS, 2016),
- elastic flexural stiffness properties reduction for the walls, beams and columns from linear-elastic model are excluded from the calculation, because their behaviour will be determined by P-M-M hinges and constitutive relationships shown in Figure 6,
- elastic shear stiffness properties reduction for the walls (for in-plane actions) from linear-elastic model is excluded from the calculation, because its behaviour will be determined by shear hinges and constitutive relationships shown in Figure 7,
- effective flange widths are considered in nonlinear analysis and were calculated according to (EN1998-1, 2004). The width of the effective beam flange is equal to 50 cm on the side of the beam.

NONLINEAR HINGE PROPERTIES

The properties of confined concrete in structural elements are calculated according to (Mander et al., 1988). Ultimate strain value in confined concrete core $\varepsilon_{cu,c}$ is calculated according to expression given in (Paulay & Pristley, 1992). Plastic hinges are modelled as fiber cross sections. Nonlinear behaviour of structural elements (walls, columns and beams) is modelled with nonlinear plastic hinges. P-M-M nonlinear fiber hinge models are used to analyse the effects of axial forces and bi-directional moments on nonlinear behaviour of the system.

Contrary to the approach where only P-M-M hinges were included to model the nonlinear behaviour of ductile RC walls, shear hinges were also included to model RC walls behaviour. The method that was applied is described in (Gerin & Adebar, 2004). Shear hinge model stress – strain relationship in wall *FW Y M I* is shown in Figure 7. The same methodology was applied for other RC walls.

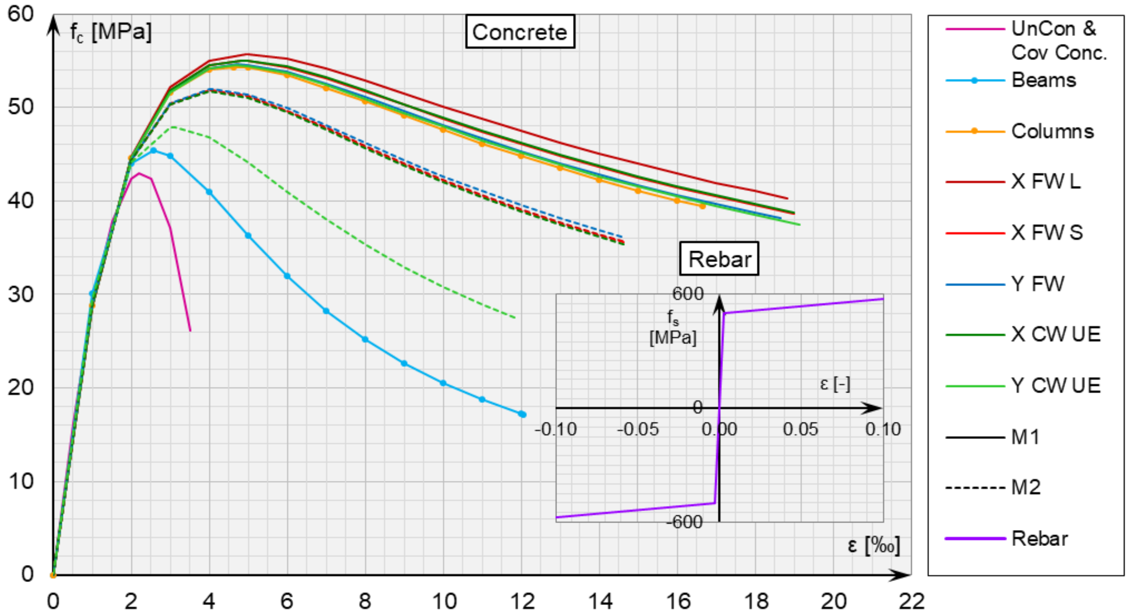

Figure 6. Materials stress-strain relationship
Slika 6. Konstitutivne veze materijala

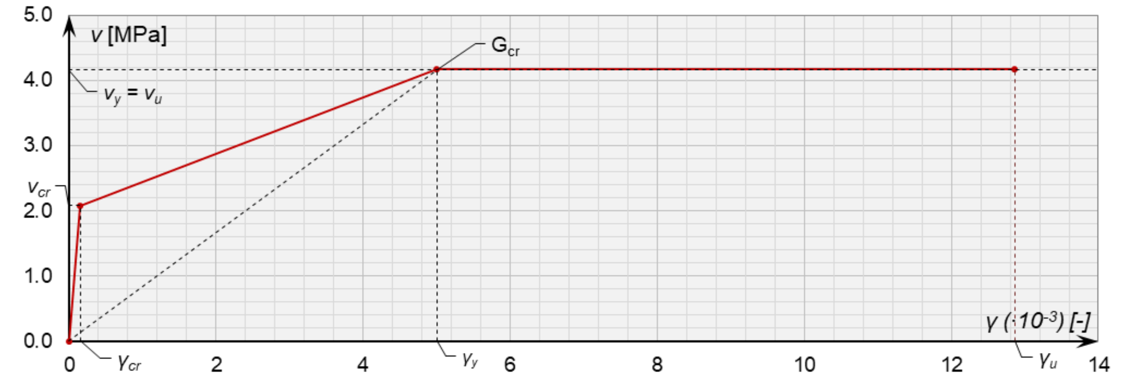

Figure 7. Shear hinges stress-strain relationship in wall FW Y M I
Slika 7. Konstitutivna veza za smičući plastični zglob u zidu FW Y M I

To obtain shear hinge model, it is necessary to calculate the values shown in Figure 7. Cracking shear stress v_{cr} is equal to the lesser value of (Gerin & Adebar, 2004), (ACI 318-11, 2011).

RC walls have P-M-M plastic hinges, which are located at the 15% of and 85% element total height, on all floors except the basement, ground and 1st floor, with the plastic hinges length of 0.3*L*, where *L* is a clear length of the element. Dominant nonlinear behaviour of RC walls is usually expected in the first two floors (ground and 1st floor) (Milev, 2016). For that reason, on these stories, the distributed plasticity approach was used, which means that each wall on the basement, ground and 1st floor have 5 P-M-M hinges that are distributed along their full length. These hinges have the same length of 0.2*L*, but they are located at the 0.1*L*; 0.3*L*; 0.5*L*; 0.7*L* and 0.9*L* of the element total height. Shear hinges are modelled as a shear stress and strain function *(v-γ)*. They are located on 0.5*L* on each wall on which P-M-M plastic hinges are modelled and they integrate the entire section across its height in the calculation.

Among several expressions (Zhao, 2011), the commonly used equation given by (Paulay & Pristley, 1992) was used for the calculation of the plastic hinge length, because of its application simplicity.

MODELLING OF THE STRUCTURAL ELEMENTS

RC walls, columns and beams have nonlinear properties. The walls are modelled as nonlinear column – rigid beam elements (Figure 1). Each wall section is defined with many fiber sections, whose behaviour is described using constitutive stress – strain relationship functions (Figures 6 and 7).

Rebar quantities in walls, columns and beams, used in analysed models are presented in Tables 2 and 3. Rebar quantity is calculated based on the wall model design:

- M1 – design with confined boundary elements (Figure 3). Wall models in M1 are designed according to (EN1998-1, 2004), (EN1992-1, 2004), for a DCM structural behaviour (EN1998-1, 2004). Wall models include confined boundary elements at the ends and unconfined element between them (Figure 3). This model is used as a referent (comparison) model.
- M2 – model with reinforcement quantity calculated according to (EN1998-1, 2004), (EN1992-1, 2004), for evenly distributed bars. This model is fully unconfined along its length.

Indexes *I*, *II* and *III* in Table 2 represent the story position of RC walls. Index *I* refers to basement, 1st floor and 2nd floor. Index II refers to RC walls placed in 3rd, 4th and 5th floor. Index II refers to RC walls positioned from 6th to 10th floor.

The modelling of core walls is done by designing the each wall segment of the core according to design actions in nonlinear analysis. They are modelled as a group of multiple column-rigid beam elements.

The reinforcement in columns and beams is the same in M1 and M2. The focus of the paper is to analyse the effect of walls reinforcement detailing on behaviour of RC building. In this way, the reinforcement in beams and columns did not affect the differences in structural system's behaviour between the models and has no direct impact on difference of the results in the comparative analysis. The reinforcement amount in columns and beams by their position (Figure 8) are shown in Table 3. All columns have the same amount of reinforcement.. Fibers division was used for plastic hinges modelling.

Table 2. Reinforcement quantity in RC walls
Tabela 2. Usvojena količina armature u AB zidovima

Wall	**Vertical reinforcement**		**Confinement**	**Shear reinforcement**
Model	**M1 (B.E. / I.E.)**	**M2**	**M1**	**M1, M2**
Properties	**(n_{bL}) d_{bL} / s_{bL}** [mm/cm]	**Whole RC wall** [mm/cm]	**$(n_{sw,d}$ / $n_{sw,b})$ d_{sw} / s_{sw} [mm/cm]**	**d_{sw} / s_{sw}** [mm/cm]
FW X L I	(32)Ø20/12.00 / (84)Ø10/15.10	(138)Ø16/14.75	(3 / 15) Ø10/10	Ø20/20
FW X L II	(32)Ø18/12.01 / (84)Ø10/15.10			Ø18/20
FW X L III	(32)Ø16/12.03 / (84)Ø10/15.10	(138)Ø14/14.76		Ø16/20
FW X S I	(18)Ø16/12.63 / (40)Ø10/13.00	(74)Ø14/14.53	(3 / 8) Ø10/10	Ø16/20
FW X S II	(18)Ø12/12.69 / (40)Ø10/13.00			Ø14/20
FW X S III				Ø12/20
FW Y E I	(18)Ø22/13.97 / (48)Ø10/14.74	(94)Ø16/12.63	(3 / 8) Ø10/10	Ø18/20
FW Y E II	(18)Ø18/14.03 / (48)Ø10/14.74	(94)Ø14/12.64		Ø14/20
FW Y E III	(18)Ø14/14.09 / (48)Ø10/14.74			
FW Y M I	(18)Ø18/14.03 / (48)Ø10/14.74	(94)Ø14/12.64	(3 / 8) Ø10/10	Ø18/20
FW Y M II				Ø14/20
FW Y M III	(18)Ø14/14.09 / (48)Ø10/14.74			
CW Y I	(18)Ø14/14.09 / (48)Ø10/14.74	(94)Ø14/12.64	(3 / 8) Ø10/10	Ø14/20
CW Y II				
CW Y III				Ø12/20
CW X UE I	(18)Ø12/11.26 / (36)Ø10/14.06	(62)Ø14/14.78	(3 / 8) Ø10/10	Ø16/20
CW X UE II				
CW X UE III				Ø14/20
CW Y UE I	(14)Ø12/11.76 / (12)Ø10/13.40	(32)Ø14/14.90	(3 / 6) Ø10/10	Ø12/20
CW Y UE II				
CW Y UE III				Ø10/20

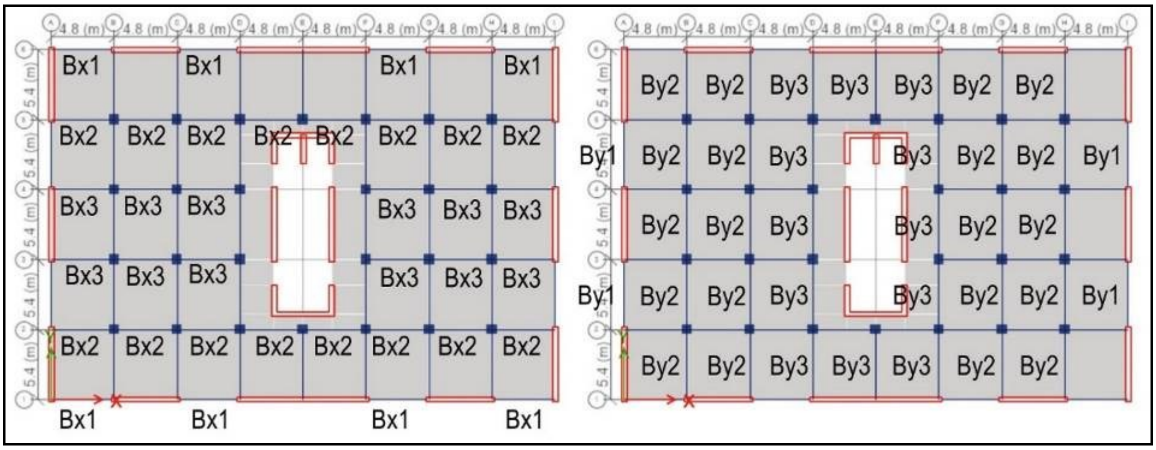

Figure 8. Beam positions – plan view
Slika 8. Pozicije greda – pogled u osnovi

Table 3. Reinforcement quantity in columns and beams
Tabela 3 . Usvojena količina armature u stubovima i gredama

Level		**Basement – 10th story**				
Reinforcement		**Flexural**				**Confinement/Shear**
Columns:		32 - Ø18				Ø10/10
Beams:		**Flexural**				**Confinement/Shear**
B_{X1}	*top*	6 - Ø25	**B_{Y1}**	*top*	6 - Ø25	Ø10/10
	bottom	6 - Ø25		*bottom*	5 - Ø25	
B_{X2}	*top*	5 - Ø22	**B_{Y2}**	*top*	3 - Ø25	Ø10/10
	bottom	5 - Ø22		*bottom*	3 - Ø25	
B_{X3}	*top*	3 - Ø25	**B_{Y3}**	*top*	6 - Ø22	Ø10/10
	bottom	3 - Ø25		*bottom*	6 - Ø22	

RESULTS AND DISCUSSION

MODAL ANALYSIS

The load dependant Ritz (LDR) vector is used for the modal analysis in the linear-elastic design and NSA models. The value of T_1 corresponds to the first translational periods in the X or Y direction or the first rotational period R. The value of T_2 corresponds to the translational periods in the X or Y direction or the rotational period R, which refers to the period value in which the structural system reaches at least 90% of the sum of effective modal masses in one of the 2 translational directions or one rotational direction. The structure is torsionally stiff. Values of the periods used are shown in Table 4.

Table 4. Periods of vibration of the structures
Tabela 4. Periodi vibracija konstrukcija

MODEL	T_{M1} [s]	m_{M1} [%]	T_{M2} [s]	m_{M2} [%]
Y_1	0.868	66.64	0.860	66.64
Y_2	0.079	91.12	0.078	91.19
X_1	0.674	67.26	0.666	67.38
X_2	0.069	91.11	0.068	91.17
R_1	0.583	65.53	0.577	65.64
R_2	0.053	90.38	0.052	90.46

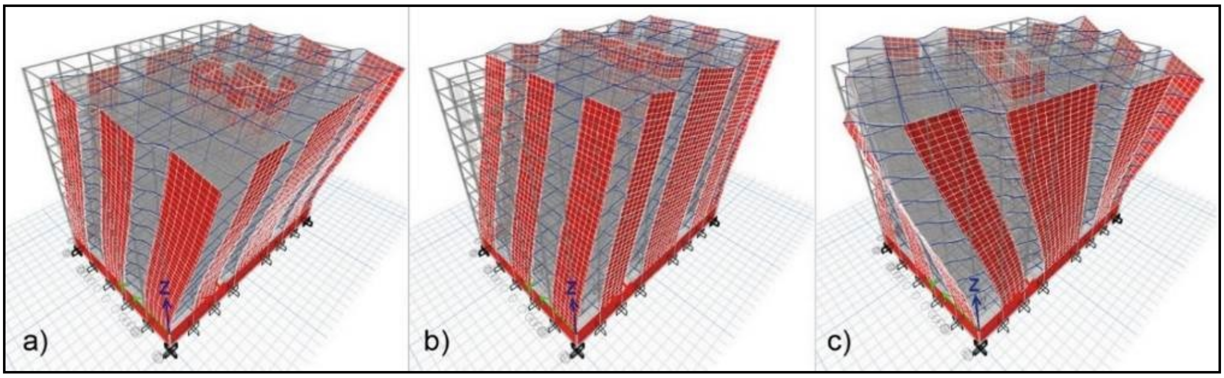

Figure 9. First three fundamental periods of vibration of the structure:
a) T_1 – translation (*Y* dir.); b) T_2 - translation (*X* dir.); c) T_3 – rotational
Slika 9. Prva tri osnovna perioda vibracija konstrukcije:
a) T_1 – translacioni (*Y* pravac); b) T_2 - translation (*X* pravac); c) T_3 – rotacioni

NONLINEAR STATIC PUSHOVER ANALYSIS

A NSA is performed for both main (*X* and *Y*) directions. Two different load distribution patterns are used for the analysis: mass proportional (PROP) and modal (MOD). PROP load distribution is the mass-proportional load distribution and modal (MOD) represents the 1st mode load distribution for appropriate direction. The results of NSA for both directions are shown in Figure 10.

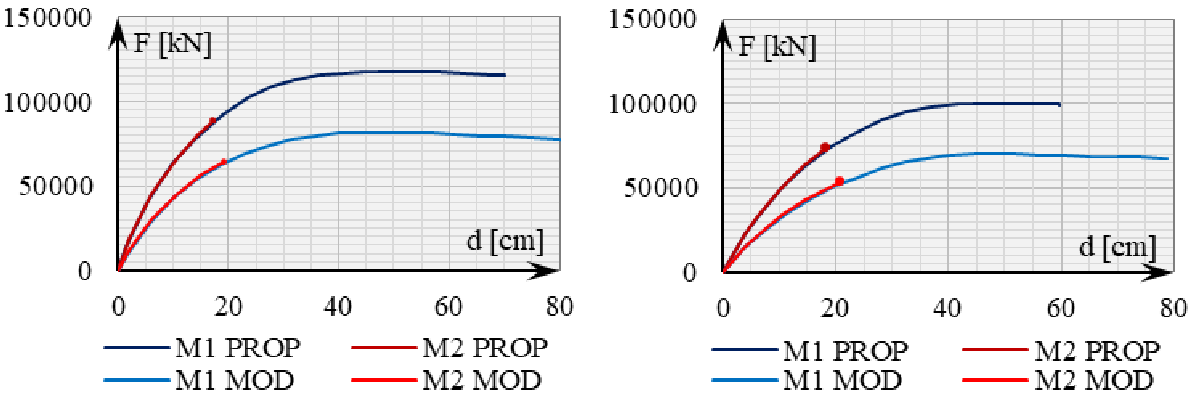

Figure 10. Pushover curves in *X* (left) and *Y* direction (right)
Slika 10. Pushover krive u *X* (levo) i *Y* pravcu (desno)

Based on NSA results (Figure 10), it is evident that there is a negligible difference in the behaviour of two building models until a certain point. The main difference between M1 and M2 can be spotted "deep" in the non-linear zone. While M1 pushover curve has a "smooth" shape until the last calculated value, M2 reaches its ultimate capacity point much earlier, compared to the first two models, which is expected, because there is no confinement in the RC walls. That leads to overall, lower ductility of the walls and the structure and inability of the walls to receive the stresses which may be absorbed by the confined RC walls in M1 and M2. However, for *PGA* equal to $a_g = 0.2 \cdot g$, the target displacement values show that all three structures remain in low-to-mid ductility zone, and the wall design still does not impact different behaviour properties, because full ductility potential has not been reached yet. Target displacement and corresponding force values are displayed in Table 5.

Table 5. Target displacement and corresponding shear force values
Tabela 5. Vrednosti ciljnih pomeranja i odgovarajućih smičućih sila

Model	$d_{t,x,prop}$ [cm]	$d_{t,y,prop}$ [cm]	$F_{t,x,prop}$ [kN]	$F_{t,y,prop}$ [kN]	$d_{t,x,mod}$ [cm]	$d_{t,y,mod}$ [cm]	$F_{t,x,mod}$ [kN]	$F_{t,y,mod}$ [kN]
M1	10.89	9.68	66746.72	47413.58	12.69	12.03	49851.65	36883.53
M2	10.91	9.70	67355.53	47921.89	12.72	12.03	50197.99	37097.54

The relationship between these values may be described as $d_{T,M1} < d_{T,M2}$ and $F_{T,M1} < F_{T,M2}$ in both *X* and *Y* direction for PROP load distribution. The relationship between these values may be described as $d_{T,M1} < d_{T,M2}$ and $F_{T,M1} < F_{T,M2}$ in X direction and $d_{T,M1} = d_{T,M2}$ and $F_{T,M1} < F_{T,M2}$ in Y direction for MOD load distribution. However, the highest percentage difference between these values is 1% and they can be neglected.

Values of global drifts for each floor (GDR_i) are calculated according to:

$$GDR_i = \frac{d_i}{\overline{H}_i} \tag{1}$$

where:
- d_i is displacement of the *i*-th storey for referent *PGA* and
- $\overline{H}_i$ is the height of the *i*-th storey from the ground floor.

Values of inter-story drifts for each floor (IDR_i) are calculated according to:

$$IDR_i = \frac{d_i - d_{i-1}}{H_i - H_{i-1}} = \frac{\Delta d_i}{\Delta H_i} \tag{2}$$

where:
- d_i is displacement of the *i*-th storey for referent *PGA*,
- H_i is the height of the *i*-th storey from the ground floor,
- Δd_i is relative inter-story drift of the *i*-th storey for referent *PGA* and
- ΔH_i is the height of the *i*-th storey from the ground floor.

Values of global drifts (GDR) and inter-story drifts (IDR) obtained in nonlinear static pushover analysis for PGA of 0.2*g* are shown in Figure 11. Maximum global drifts (GDR_{max}) and maximum inter-story drifts (IDR_{max}) values and their percentage differences in M2, compared to M1 are shown in Table 6.

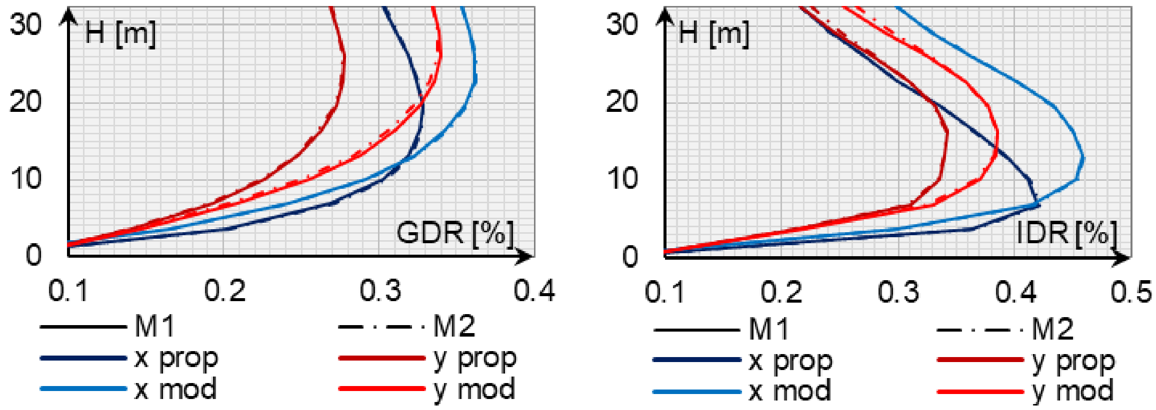

Figure 11. GDR (left) and IDR (right) in *X* and *Y* direction obtained by pushover NSA
Slika 11. GDR (levo) i IDR (desno) u X i Y pravcima, proračunati primenom NSA

Table 6. NSA GDR_{max} and IDR_{max} values and their percentual difference compared to M1
Tabela 6. Vrednosti GDR_{max} and IDR_{max} proračunate primenom i procentualne razlike u odnosu na M1

Load distribution	**PROP**		**MOD**	
Direction	**X dir.**	**Y dir.**	**X dir.**	**Y dir.**
$GDR_{max,M1}$ [%]	0.328	0.277	0.361	0.339
$GDR_{max,M2}$ [%]	0.329 (0.23%)	0.276 (-0.36%)	0.362 (0.25%)	0.337 (-0.59%)
$IDR_{max,M1}$ [%]	0.419	0.343	0.458	0.428
$IDR_{max,M2}$ [%]	0.422 (0.74%)	0.342 (-0.38%)	0.459 (0.37%)	0.428 (-0.09%)

Based on the analysis and comparison of *GDR* and *IDR*, displayed in Figures 11, maximum *GDR* and *IDR* values for M2 in comparison to M1 are negligibly small and they are shown in Table 6.

REINFORCEMENT AMOUNT COMPARISON

Another parameter that is used to compare the difference between the methods of RC walls rebar detailing is the vertical flexural rebar. M2 lacks the confining reinforcement in RC walls, so the difference is incomparable. Horizontal shear rebar and confining rebar quantity were excluded from the comparison analysis. The values of M2 are compared to M1. Although it is already shown (through pushover analysis) that M1 and M2 have the same structural response for the PGA = 0.2*g*, the amount of rebar in RC walls in the two models is much different. Longitudinal (vertical) bars have almost 30% higher volume in M2, than in M1.

FINAL REMARKS AND CONCLUSIONS

This paper analyses the impact of different quantity and arrangement of rebar in RC walls, which does not correspond with a typical form of RC walls with peripheral elements. (EN1998-1, 2004), (EN1992-1, 2004). Two models are considered, whereby M1 corresponds with case when seismic RC walls are designed in accordance with (EN1998-1, 2004), (EN1992-1, 2004) and it represents the reference model for a comparative analysis. Characteristics of RC walls in M2 are described in detail in the paper. Using the comparative analysis, the differences described by numerical values in the text, tables and figures from the aspect of displacements, GDR and IDR of the structural system exposed to the action of a seismic actions are established. The analysis of the obtained results leads to the following conclusions:

- Using a certain quantity of rebar with atypical arrangement in RC walls in relation to (EN1998-1, 2004), (EN1992-1, 2004), the results are achieved, which can to a certain extent deviate from the results obtained by the analysis of the reference model M1.

- The M2 (non-confined and evenly distributed along the wall length), will have almost the same structural response as M1 for the design *PGA*. The roof displacement values, *GDR* and *IDR*, and its percentage differences compared to M1 values for the designed *PGA* are very small, i.e. negligible. However, the big difference between M2 and M1, occurs in the plastic zone, after the target displacement is reached. It is established through pushover analysis. The structure will lose its bearing capacity much earlier, than in M1, with a lot less ductile response, which is by no means a good property. This type of response is the consequence of non-confined RC walls with much lower concrete stress limit than in confined parts of M1 RC walls that provide the ductility of the element and the structure as whole unit. M2 walls will have much higher amount of vertical rebar, but none of the confining rebar, which will lead to the smaller total amount of the 2 types of reinforcement in RC walls in M2, compared to M1.

By analysing the results obtained from M2, it can be concluded that with the choice to use a different approach of rebar detailing in the RC walls (evenly distributed, confined and unconfined rebar), the similar response of M1 will be achieved in many of the analysed parameters (displacements, GDR and IDR for the design PGA). This can be seen in the results of the NSA. On the other hand, M2 will have the same response as M1, but it will reach its ultimate capacity point much earlier, because of the lack of ductile properties of its RC walls.

Based on the results of the conducted analysis, it can be concluded that the existing approach according to (EN1998-1, 2004), (EN1992-1, 2004) and the approach applied in M2 will provide the similar results in terms of the seismic structural response for adopted *PGA* but not higher load intensities, and much higher rebar quantity than in M1. The approach according to (EN1998-1, 2004), (EN1992-1, 2004) is strongly **recommended** in RC wall design. Providing the confined boundary elements in critical zone of the RC walls, in wall-equivalent dual structural system building, contributes to a better behaviour of the entire structural system and its seismic response.

It is established that the percentage deviations of the results in M2 are unimportant in relation to M1, but are not negligible, either. The values in M2 indicate that this approach provides very unfavourable results, so irrespective of its cost-reducing potential, it **should not** be employed.

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