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NONLINEAR SEISMIC ASSESSMENT OF COUPLED WALLS DESIGNED IN ACCORDANCE WITH EUROCODE 8

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Abstract

Experience from previous earthquakes have shown that wall structural systems experience less damage during earthquake compared to frame systems. Wall systems for functional and architectural reasons frequently have openings (windows, doors, elevators, esc.). Wall systems with regularly distributed openings represent efficient system for resisting earthquake loads. Coupling beams connecting the walls, if designed and detailed properly, increase seismic resistance of the building by distribution of inelastic deformations both vertically and in plan. Eurocode 8 introduced set of rules for design and detailing of coupled walls and coupling beams.

In order to access performance of coupled walls and beams designed in accordance with EC8, linear and nonlinear analysis of 11 story building was performed. Linear elastic modelling was done using software for linear analysis. The walls and coupling beams were designed and detailed in accordance with the provisions of Eurocode 8, part 1. Nonlinear model and assessment of inelastic response was conducted using Perform 3d CSI software for nonlinear analysis. For the modelling of coupled walls, wall section with fibers is used. The confined constitutive relationship is used for concrete edge elements, and unconfined relationship for concrete for the rest part. The reinforcement constitutive model was defined with bi-linear curve. Coupling beams are modelled using frame elements with shear hinge elements. Deformation capacities of elements was defined in accordance with EC8 provisions. Considering that EC8 doesn't provide provisions for deformation capacities of diagonally reinforced coupling beams, deformation capacities for these elements is defined in accordance with the provisions of ASCE 41-06 standard. Static nonlinear analysis is performed in accordance with EC8 provisions and deformation capacities of wall elements and coupling beams checked in accordance with the provisions EC8 part 1 and part 3, where applicable. Characteristic results are presented on the end of paper, with conclusions and recommendations.

Keywords: coupler walls, coupling beams, diagonal reinforcement, pushover analysis, shear hinge

1. Introduction

Coupled walls are shear walls intermittently connected with beams (coupling beams) along the height. Shear walls are designed to allow dissipative inelastic behaviour at the base of the walls. By connecting shear walls with beams, dissipative behaviour can also be enabled in the locations of coupling beams along the height of structure. The proportions of walls and stiffness of coupling beams determine the response of the coupled walls.

The difference in behaviour of shear and coupled walls can be explained by comparing the shear forces in base of the walls (Fig. 1). It can be noted that in addition to bending moments, additional axial forces are registered in coupled walls which are result of shear forces in the coupling beams. For the same external loads the forces at the base of coupled walls must be in equilibrium with the moment in equivalent shear wall as shown in equation below.

$$M = M1 + M2 + T \cdot L \tag{1}$$

The ratio between moment T·L of the coupled wall and moment M for the wall without openings, indicates the impact of the coupling beams on the coupling wall response. The contribution of strong coupling beams to the seismic response of coupled wall is significant because the above mentioned ratio is larger.

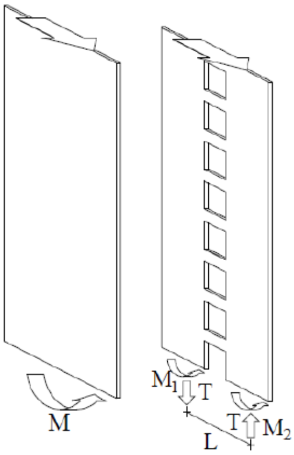

Figure 1. Comparison of internal forces at the base of different type of walls

2. Definition and provisions for coupled walls in accordance with EC8

In accordance with EC8 [1], coupled wall is structural element composed of two or more single walls, connected in a regular pattern by adequately ductile beams ("coupling beams"), able to reduce by at least 25% the sum of the base bending moments of the individual walls if working separately.

EC 8 prescribes capacity design provisions for the design of coupling walls and beams. The coupling beams can be designed with the same provisions as conventional beams when cracking in both directions in unlikely to happen (verified by formula 5.48 in EC8 [1]), and prevailing flexural mode of failure is ensured with the acceptable application rule is:

$$l/h \geq 3 \tag{2}$$

If neither of the conditions are met, the resistance to seismic actions should be provided by reinforcement ensured along both diagonals of the beam. When this type of detailing is applied the inelastic behaviour of beam is expected along the entire length of the beam and whole beam is plastic region for dissipation of energy. Considering that beams is subjected on large shear forces, diagonally reinforced beams behave like a truss, no protection from shear failure is needed. Redistribution of seismic effects between coupling beams of different floors up to 20% is allowed, provided that the seismic axial force at the base of each individual wall (the resultant of the shear forces in the coupling beams) is not affected. Formula for calculating the required amount of reinforcement in diagonal columns according to EC8 [1] is given in following expression:

$$A_{sj} \geq \frac{V_{Ed}}{2 \cdot f_{yd} \cdot sin\alpha} \tag{3}$$

ν_{Ed}	is the design shear force in the coupling element ($\nu_{Ed} = 2 \cdot M_{Ed} / l$);
A_{sj}	is the total area of steel bars in each diagonal direction;
α	is the angle between the diagonal bars and the axis of the beam.

3. Modelling and building data for linear analysis

Analysis of coupled walls system was performed on the example building with 11 storeys. The layout of building is rectangular with dimensions 30x30 m. Ground floor height is 4.0 m, and for other storeys is 3.2m. The structure has reinforced concrete slab with thickness d = 16 cm. The beams are with dimensions b/d = 30/60 cm, columns have square cross section dimensions b/d = 60/60 cm and coupling beams are with dimensions b/d = 30/100 cm. All shear walls and have the same thickness of d=30cm. Characteristic layout of structure and characteristic frame are given on Fig. 2 and Fig. 3.

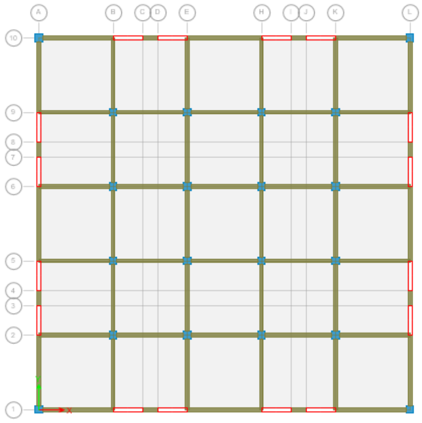

Figure 2. Characteristic layout of structure

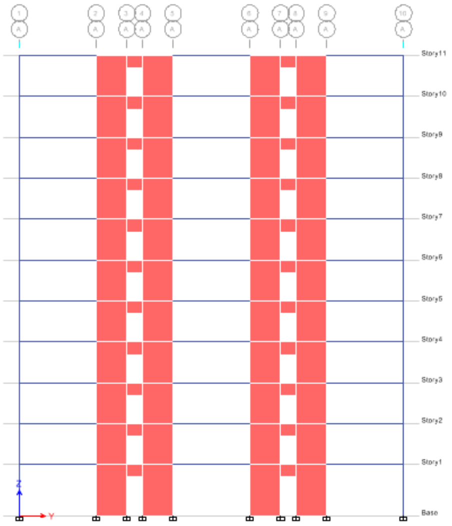

Figure 3. Characteristic cross section in axes A

Linear model was prepared in software for linear analysis. Geometric characteristics of elements and material properties are adopted in accordance with European regulations. Columns and beams are modelled as prismatic 3D beam elements and shear walls as shell elements. Stiffness properties are taken as one-half of the corresponding stiffness of the un-cracked elements including the coupling beams. It is noted that most of modern regulations give particular formulas for stiffness properties of coupling beams considering that the level of stiffness degradation of coupling beams is higher than on other elements [2]. For the adopted return period for the reference earthquake of 475 years, the peak ground acceleration $a_{gR} = 0.36g$. The ground type at the structure location is A. The structure is designed for high ductility class DCM. The structure is classified as coupled walls structure with behaviour factor $q = 3.6$.

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Lateral force analysis was performed in accordance with EC8 [1]. Total mass of structure was obtained from software with value 9597,92 ton and total seismic force $Fbx = Fby = 6779{,}2\ kN$.

2.1 Dimensioning of coupled walls

The dimension of considered coupled wall is given in the Fig. 4.

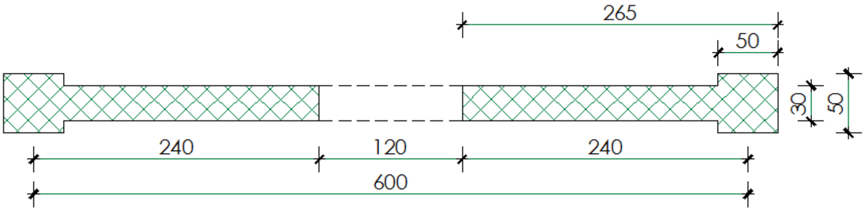

Figure 4. Characteristic cross section of coupled wall

In accordance with EC8 [1], the control was performed if coupling beams reduce by at least 25% the sum of the base bending moments of the individual walls if working separately.

Total base moments obtained with lateral force method for coupled walls and uncoupled walls are given below:

Coupled walls: $\mathrm{Mp} = 2 \cdot 2651 = 5302\,\mathrm{kNm}$

Un-coupled walls: $\mathrm{Mp} = 2 \cdot 4827 = 9654\,\mathrm{kNm}$

The difference is 45%, which satisfies criterion for the walls to be treated as coupled.

During the action of strong earthquakes in (selected) critical regions, significant non-linear deformations should be allowed to occur (in the case of shear walls, this is zone in the base of the wall. It is possible, even desirable, for the cross-sectional forces-moments and shear force to be redistribute between the walls in order to obtain better utilization and more uniform stressing of the elements. It is also possible to perform redistribution of forces in coupled walls including axial forces that are result of seismic loads. In wall no.1 there is an axial tensile force that reduces the bending capacity of the wall and increases edge longitudinal reinforcement, while in wall 2 there is an axial compressive force with the opposite effect. For this reason, it makes sense to redistribute part of the moment M_1 and entrust it to wall no.2, which will result in equalizing the load capacity and amount of reinforcement in both walls.

It is recommended that the percentage of redistribution does not exceed 30% according to EC8 [1], so that crack openings and damage during the impact of earthquakes of minor and medium intensity remain within acceptable limits. EC8 [1] (5.4.2.4 (2)) provides a procedure for redistribution the effects is walls, with condition that the total required bearing capacity is not reduced. The shear forces are redistributed together with the bending moments, in such a way that it does not have a significant impact on the relationship between bending moments and transverse forces in individual walls. In walls subjected to large fluctuations of axial force, as e.g. in coupled walls, moments and shears should be redistributed from the wall(s) which are under low compression or under net tension, to those which are under high axial compression. Due to this effect, often confinement of whole section of wall has to be performed because of high axial forces in critical regions of the coupled walls.

The adopted reinforcement for the critical region of coupled wall is given on Fig. 5. It is noted that the reinforcement is not symmetric on both sides and this is result of axial forces in the walls.

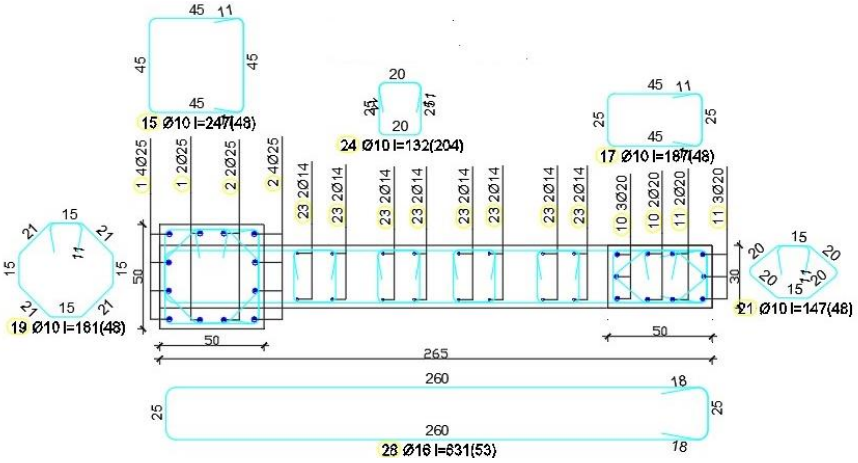

Figure 5. Reinforcement of coupled wall in the critical region

2.2 Dimensioning of coupling beams

In EC8, there are rules for reinforcing coupling beams as primary structural elements. Coupling beams act as high beams with shear forces as dominant load. According to EC8 [1], coupling beams are usually reinforced with diagonal reinforcement in the form of elements that resemble pillars. Classic reinforcement of connecting beams is done only if the shear forces are small. Experiments have shown that diagonally reinforced coupling beams have significantly better inelastic behaviour than classically reinforced.

Assessment of criterions given in formula 5.48 in EC8 [1] is done and it is noted that coupling beams are supposed to be reinforced with diagonal reinforcement.

Redistribution of seismic effects between coupling beams for different floors up to 20% is performed and the results are shown in the table 1. The difference is exceeded only on the last four floors. This can be ignored because the coupling beams on the last four floors can be classically reinforced. Also, the calculation of seismic forces performed by lateral force method, which doesn't take effect of higher mode effects on seismic response. Considering this redistribution and diagonal reinforcement is justified also on higher floors.

Table 1 – Redistribution of effects in coupling beams

Storey	VED (kN)	Required reinforcement	Adopted bars	Reinforcement cm2	Actual force	Difference (%)
11	144	3,12	4ø12	4,52	208,31	-44%
10	54	1,17	4ø12	4,52	208,31	-285%
9	131	2,84	4ø12	4,52	208,31	-59%
8	291	6,31	4ø12	4,52	208,31	29%
7	439	9,52	4ø18	10,18	469,17	-7%
6	574	12,45	4ø18	10,18	469,17	18%
5	701	15,21	6ø18	15,27	703,75	0%
4	820	17,79	6ø18	15,27	703,75	14%
3	933	20,24	6ø22	22,81	1051,24	-13%
2	1036	22,48	6ø22	22,81	1051,24	-1%
1	1078	23,39	6ø22	22,81	1051,24	3%
	Σ=6201				Σ=6332,8	2,1%

The adopted reinforcement in the coupling beams, for the first three floors, is given on Fig. 6.

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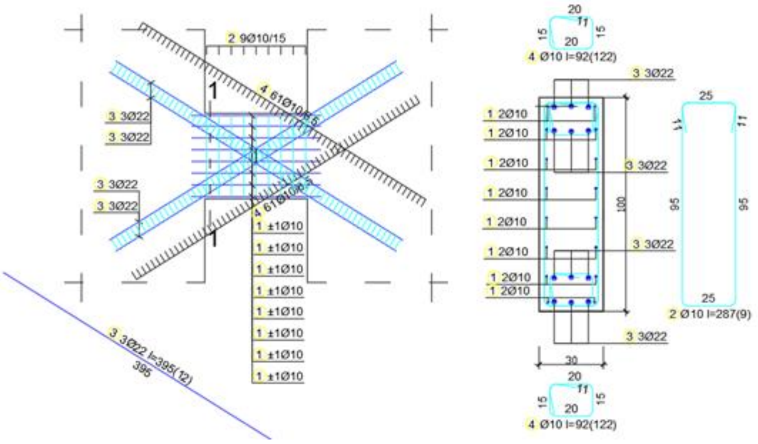

Figure 6. Reinforcement of coupling beam on lower storeys

4. Nonlinear analysis

Planar model of coupled wall is created in PERFORM 3d software package [3] (Fig. 7). The floor diaphragms were modelled as rigid with the masses lumped at the corresponding centre of gravity. The mass is modelled in accordance with the results of linear analysis for combination of dead load and 30% of live load.

4.1 Modelling for nonlinear analysis

The modelling of coupled walls is performed by modelling wall section with fibers. The confined constitutive relationship is used for concrete edge elements, and unconfined relationship for concrete for the rest part. The reinforcement constitutive model was defined with bi-linear curve.

The coupling beams are modelled using frame elements with shear hinge.

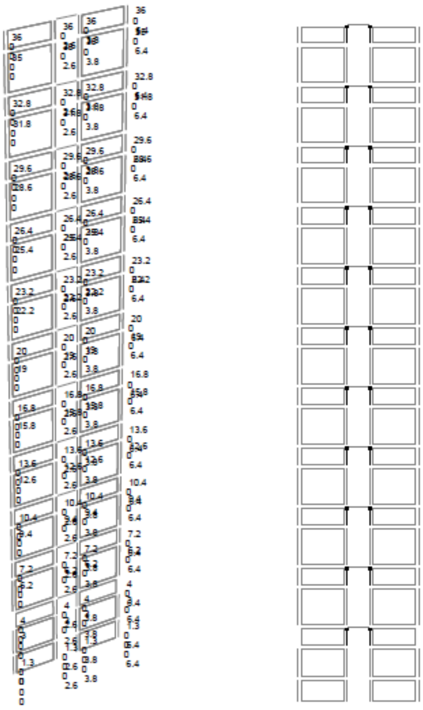

Figure 7. Reinforcement of coupling beam on lower storeys

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Main location for dissipation of seismic energy is in the base of the wall. Nonlinear behaviour in the base of the wall is gained by yielding of vertical reinforcement on the edges of the walls. Modelling of wall elements is done with Shear Wall elements consisting of number of fiber elements (reinforcement, confined and unconfined concrete). For analysed coupled wall, 10 characteristic cross sections were defined in accordance with adopted reinforcement from linear analysis. Fiber section of wall is consisted from two components. First component includes concrete (confined and unconfined) and reinforcement of edge elements. Second component includes shear reinforcement. Length of plastic hinge is defined in accordance with the provisions of EC8, part 3 [4]. The rotation is controlled with Rotation Gage elements.

4.2 Modelling of coupling beams

In PERFORM [3], coupling beams for shear walls can be modelled using Frame elements or Wall elements in accordance with the guidelines from CSI [5]. Considering dimensions of coupling beams b/d=30/100 and the length of 120cm, the coupling beams can be considered moderately deep. In accordance with model with frame elements was adopted.

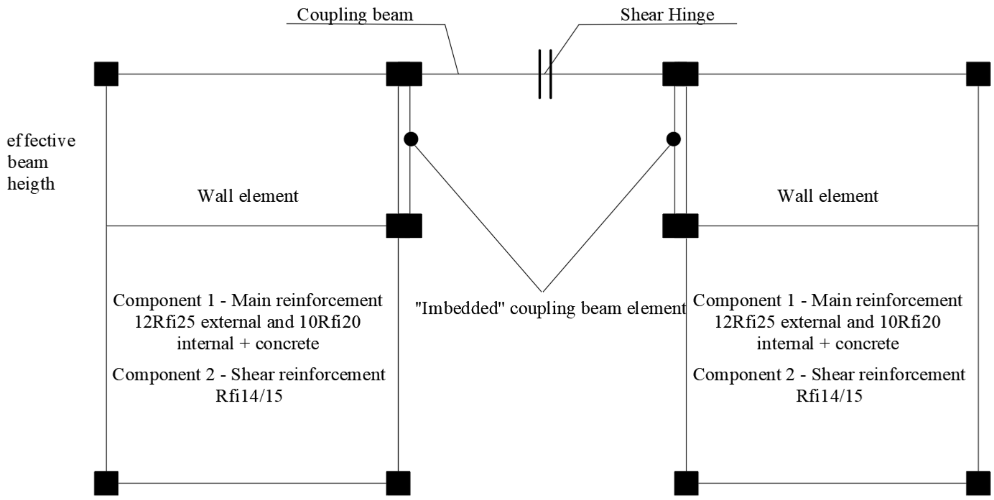

Figure 8. Schematic representation of nonlinear model of coupling wall on first floor

Moderately deep coupling beams were modelled using Frame elements, as shown in Fig. 8. The coupling beam was modelled with shear hinges because the shear behaviour governs and the coupling beams are diagonally reinforced. The coupling beam compound component consists of two elastic beam segments and a rigid-plastic shear hinge. The coupling beam element must be connected to the piers by "imbedded" beam elements. If this is not done, the coupling beam will be effectively pin-connected to the wall. These beams should be stiff enough in bending to provide a stiff connection to the piers and have negligible axial stiffness to avoid stiffening the piers. Bending in the coupling beam is transferred to the piers by these vertical beams (as a tension-compression couple).

4.3 Deformation capacities and limit states

Length of plastic hinge and rotation capacity is calculated in accordance with EC8 part 3 provisions.

The calculated length of plastic hinge is $l_p = 130cm$, and the rotation capacity is 0.09 rad.

In EC8, the deformation capacity of coupling beams is not defined, so the deformation capacities were taken from the American regulations, ASCE 41 [5], for diagonally reinforced coupling beams. For Collapse Prevention, the level of object damage plastic shear rotation (shear dilation) is 0.03 radians. For the beam span of 120cm displacement across the shear hinge is 3.6cm.

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Following deformation limit states are defined:

- Bending rotation in shear walls was checked with rotation gages for the location within plastic hinge and outside – Check was performed in accordance with no-collapse requirement defined in EC8-part 1;
- Coupling beam shear deformation. This covers the shear hinges in all coupling beam elements in accordance with no-collapse requirement defined in EC8-part 1 ;
- Tension strain, hinge region, 1%. This covers tension strain in all strain gages in the hinge regions. The D/C ratio for any gage is the tension strain in percent. This is mainly for interest, but also checks that the steel strains are not excessive.
- Compression strain, hinge, crushing. This covers compression strain in walls hinge regions. This checks that there is no significant concrete crushing in the hinge regions.
- Tension strain-This covers tension strain in all strain gages above the bottom story. This checks that there is no significant hinging outside the hinge regions;
- Compression strain, upper stories, crushing. This covers compression strain in all strain gages above the bottom story. This checks that there is no significant concrete crushing outside the hinge regions.

Following strength limit states are defined:

- Shear force capacity for walls in accordance with the no-collapse requirement defined in EC8-part 1 (2004) corresponding to NC. The shear capacity of walls is checked whether the elements remain in the elastic range regarding shear during the seismic action;

4.3 Nonlinear static (PUSHOVER) analysis

Pushover analysis is a nonlinear static analysis carried out under conditions of constant gravity loads and monotonically increasing horizontal loads in accordance with N2 method [6] from EC8.

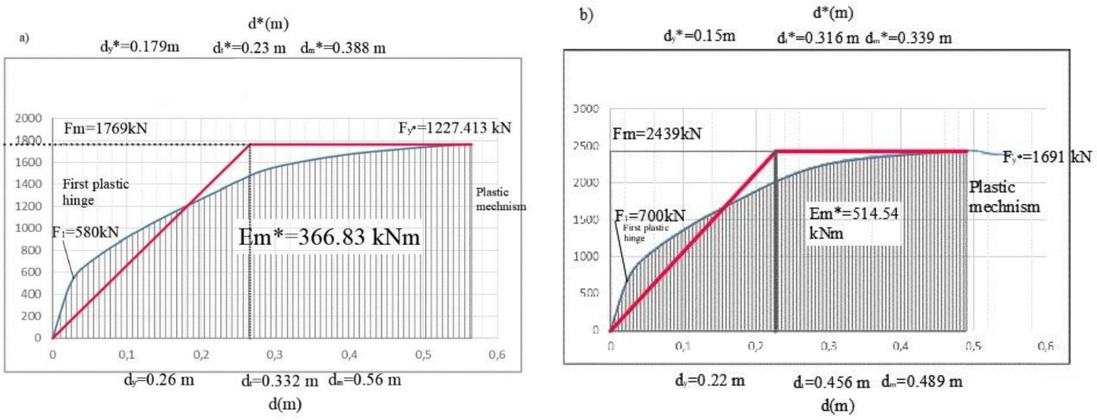

Figure 9. Total base shear force F with a) modal pattern; b) uniform pattern of forces.

According to EC8 at least two vertical distributions of the lateral loads should be applied. After obtaining the results, the most unfavourable results obtained by these two analyses are used in the design. For this case study, vertical distribution was done with "modal" and uniform pattern. The relation between base shear force and the control displacement (the "capacity curve") was determined for values of the control displacement ranging between zero and the value corresponding to 150 % of the target displacement and presented on the Fig. 9a and Fig. 9b.

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5. Results

In the Fig. 10a and Fig. 10b presents plastic deformations at the target displacement and displacement when plastic mechanism is formed for the modal load distribution. From the figure it can be seen that preferred mechanism of plastic deformations was formed. At lower intensities of horizontal forces, plastic deformations occurred in the coupling beams, and with the increase of forces, plastic mechanisms are additionally formed at the base of the walls.

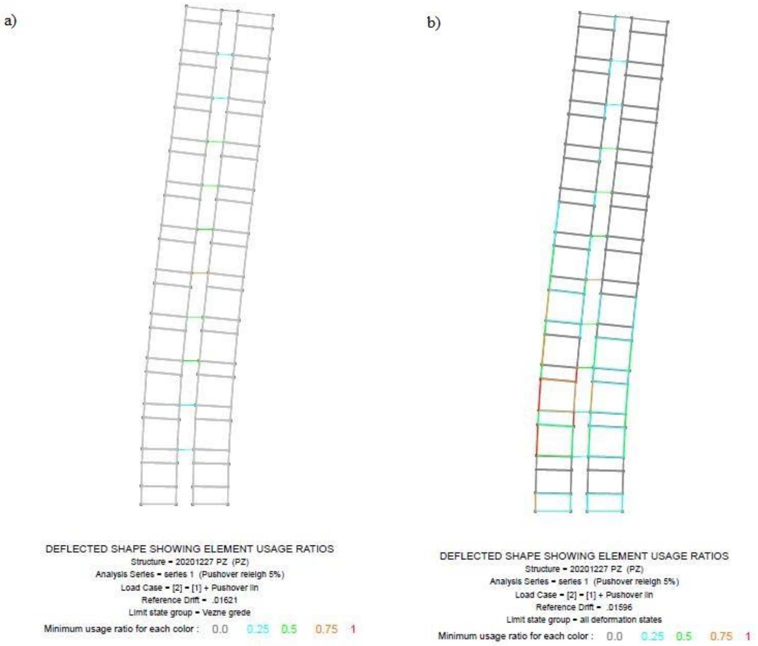

Figure 10. Plastic deformation at the a) target displacement; b) displacement when plastic mechanism is formed for the modal load distribution.

In the Fig. 11a and 11b presents plastic deformations at the target displacement and displacement when plastic mechanism is formed for the uniform load distribution. From the figure it can be seen that plastic deformations occurred simultaneously in the coupling beams and at the base of the walls.

From the Fig. 10 and Fig. 11 it can be seen that with the uniform distribution of forces along the height of the structure, plastic deformations appear earlier at base of the walls. The subject distribution of forces along the building height serves to present the possibility of the appearance of a "weak storey" in the structure. From the analysis of this structure it can be concluded that the deformation capacity of the coupling beam will be exhausted before deformation capacity plastic hinge at the base of wall is exhausted.

With pushover analysis, it is also possible to check the value of the ratio α_u/α_1 given in EC8, which directly affects the value of the behaviour factor q. Analogous to the definition given in EC8, Article 5.2.2.2(4), the values α_u and α_1 multiplier of horizontal seismic design action at formation of global plastic mechanism and at formation of first plastic hinge in the system, respectively. Equation below provides value of ratio.

$$\mathrm{a_u/a_1} = \mathrm{F_m/F_1} = 2432/2000 = 1.216 \tag{4}$$

It can be concluded that the recommended value corresponds to the recommended value in EC8 for DCM which is 1.2. The value of the ratio for the structure with non-coupled walls is 1.0, so this increase is the ratio is product of additional locations for energy dissipation within the shear plastic hinges in coupling beams.

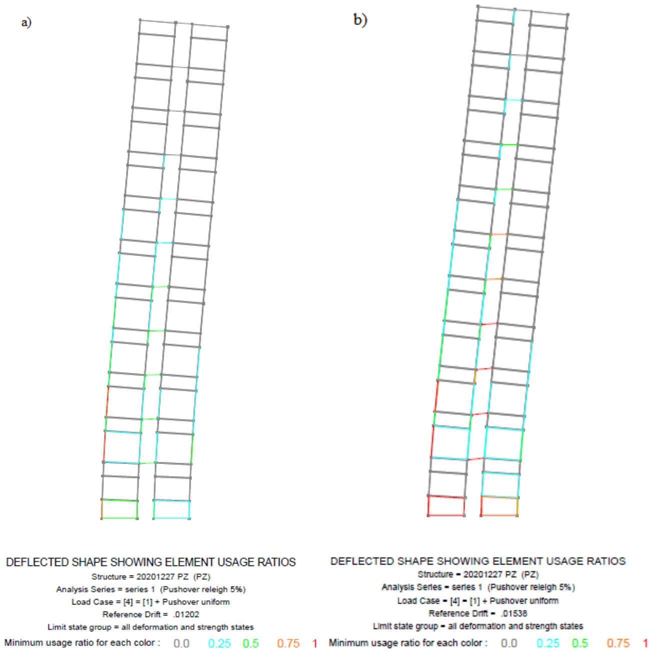

Figure 11. Plastic deformation at the a) target displacement; b) displacement when plastic mechanism is formed for the uniform load distribution.

In accordance with the “no collapse” requirement, load bearing capacity of brittle elements and deformation capacity dissipative zones (e.g. rotation of plastic joints) should be checked. Brittle failure of elements is shear failure of walls. Deformation capacity of plastic hinge rotation at the base of the walls and capacity of shear hinge in coupling beams was checked. Results are given in table 2.

Table 2 – Maximum D/C values for Redistribution of effects in coupling beams

	D/C ration for shear in wall	**D/C ration for plastic hinge at the base of the wall**	**D/C ration for shear hinge in coupling beam**
Pushover - uniform pattern	0.65	0.64	0.6
Pushover - modal pattern	0.57	0.62	0.58

6. Conclusions

Following conclusions and recommendations are obtained from results:

- Eurocode 8 recognizes coupled walls as a special type of structural elements that, with proper calculation and reinforcement details, have excellent ductility capacity. Connected walls enable the distribution of inelastic deformations at the base and in height;
- Dimensioning of coupled walls is done according capacity design method EC8, part 1. Redistribution of seismic effects between coupling beams of different floors up to 20% is allowed in order to obtain a more uniform reinforcement on the floors. It is possible, even desirable, for the cross-sectional forces-moments and shear force to be redistribute between the walls in order to obtain better utilization and more uniform stressing of the elements;
  According to EC8, connecting beams are usually reinforced with diagonal reinforcement in the form of elements that resemble columns. Classic reinforcement of connecting beams is applied only if the shear forces are with lower intensity;

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- It is noted that most of modern regulations give particular formulas for stiffness properties of coupling beams, which is not the case with EC8. Considering that the level of stiffness degradation of coupling beams is higher than on other elements this should be defined in next version of EC8.
- There are limited guidelines in EC8 for nonlinear analysis of coupled walls. There are no information related to selection of model for coupling beams with diagonal reinforcement and connections of coupling beams with coupled walls so the guidelines from ASCE 41 and CSI knowledge base were used. Also, EC8 part 3 guidelines for determination of plastic hinge length are the same as for uncoupled wall which should be investigated. Considering the extensive use of coupled walls the guidelines for nonlinear modelling of coupling beams should be provided in next versions of EC8.
- Results of nonlinear static (pushover) analysis show generally very good behaviour of coupled wall designed in accordance with “no collapse” requirement given in EC8. The brittle elements remained in elastic region of deformation and elements for dissipation have shown that D/C ration is below 1. The critical region at the base of the wall is two storey in accordance with the EC8 and it is noted that for subject structure plastic deformations have been noted also on the third storey so further investigation is necessary in order to check if critical regions should be extended for coupled walls in higher buildings.
- Value of the ratio α_u/α_1 was checked and it can be concluded that the recommended value corresponds to the recommended value in EC8 for DCM which is 1.2. The value of the ratio for the structure with non-coupled walls is 1.0, so this increase is the ratio is product of additional locations for energy dissipation within the shear plastic hinges in coupling beams.

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