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SEISMIC PERFORMANCE ASSESSMENT OF RC FRAMED SYSTEM BUILDING DESIGNED TO EUROCODE HIGH CLASS DUCTILITY

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Keywords: *seismic performance, structural ductility, seismic damage index, reinforced concrete frame structures, pushover analysis, overstrength ratio*

ABSTRACT

A newly designed RC framed system building designed to Eurocode high-class ductility (DCH) is numerically investigated to assess its earthquake performance. The capacity curves obtained by non-linear static pushover analysis for different lateral load arrangements are compared to the initial response spectrum solution also to estimate the structural seismic overstrength ratio. The structural seismic performance assessment is based on an adopted floor damage index. The presented approach can be used as a pattern for evaluation of overstrength factor and seismic performance assessment. The results are indicative regarding low and midrise RC framed structures designed to Eurocode high ductility class (DCH).

1. Introduction

A newly designed mid-rise reinforced concrete frame structure to the requirements of Eurocode 8-1 [1 – 3] for high ductility class (DCH) has been numerically investigated by nonlinear static (N2 pushover [15]) analysis specified in [1], Annex B.

The main objectives of this study are to evaluate the over-strength factor of the structure, to identify the most likely mechanisms of sequential development of plastic deformations in the elements, and to evaluate the expected damage levels under specified earthquake actions.

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Part 1 of Eurocode 8 [1 – 3] defines the structural over-strength ratio as α_U/α_1, where α_1 is the value by which lateral seismic load is multiplied to cause any structural element reach its flexural resistance and α_1 is the value by which the lateral seismic action is multiplied to form a number of plastic hinges to develop overall structural instability. Past research [13, 14] suggest that the overstrength factor depends, among other factors, on structural redundancy and thus is an indicator of its level.

The mechanism of sequential development of member yielding is fundamental in understanding the process of spreading of structural damage with lateral load increasing.

Numerous research studies are focused on evaluation of structural damage level [12, 20 – 22] through damage indices approach. The approach can be generalized as developing functions that evaluate to 0,00 when no damages are present and to 1,00 to indicate structural failure or collapse with intermediate values describing structural damage development.

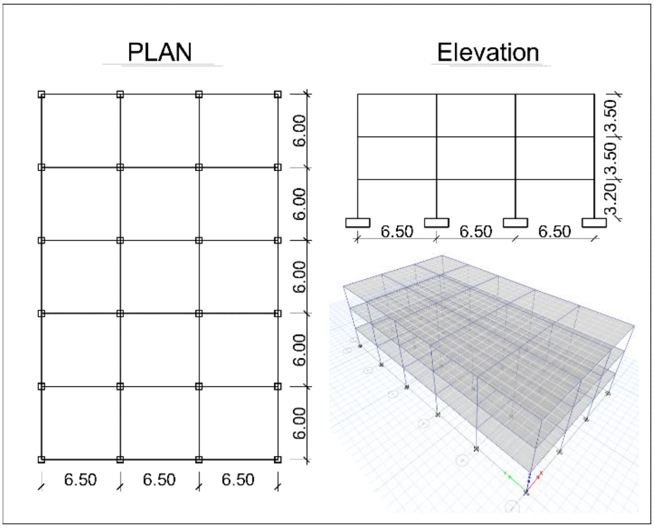


Figure 1. Structure drawing

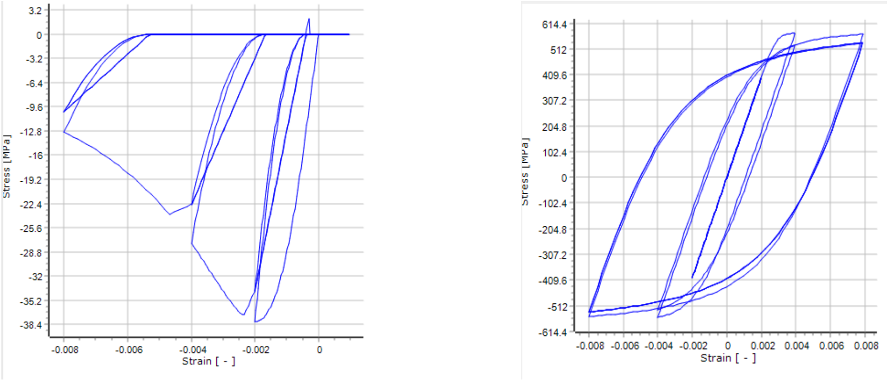
2. The studied model

The structure contains two floor slabs and a roof slab. It is composed of six transversal and four longitudinal frames. Frame spans are 6 m in the short (X) building direction and bays are 6,5 m (along length – Y). The cross-sections dimensions are 50/50 cm for columns with varying reinforcement in height and 60/40 cm for beams with variable reinforcement in length. The used materials are class C 30/37 for concrete and S 500C for reinforcement. The structure is regular in plan and in height with almost equal masses floor masses regularly distributed on the slabs. The structure is assigned III^{-rd} class of importance according to Eurocode 8-1 [1 – 3] with corresponding importance factor value $\gamma_I = 1,2$ and the adopted site location has reference ground acceleration $a_{gR} = 0,23\ g$ ($g = 9,81$ m/s^2), and soil class C. The response spectrum is Type 1. The model composition followed the guidance in [9 – 11].

2.1. The materials model

The material used in the structural model are inbuilt into the SeismoStruct [23] program. For concrete, the Mander et al. [18] model is used. This model represents well the performance of confined and unconfined concrete. The model assumes constant confinement factor throughout the entire strain range for scaling the stress-strain relationship. Values for mean peak concrete compressive strength f_c' = 38 MPa at strain ε_c = 0,002 and initial elastic modulus E = 33 GPa define the implemented concrete model. The resulting stress-strain curve is shown in Fig. 1.

The Menegotto-Pinto [19] model was used for modelling the reinforcement steel bars and stirrups. This model is well known and has been implemented in SeismoStruct [23] and other structural software for nonlinear seismic analysis. The specified mean strength is 575 MPa, for lower-bound strength 500 MPa is used and the program default values for the rest parameters. The resulting stress-strain curve are shown in Fig. 2.



Concrete C30/37 **Reinforcement S 500C**

Figure 2. Stress-strain curves for the modelled materials [23]

2.2. The structural elements model

The distributed hysteretic performance of the structural elements – columns and beams, was implemented in the numerical model by meso-scale fiber-sections model of distributed plasticity. Both columns and beams were described by 6 fiber cross sections along its lengths using 204 control points in each of them. The cross sections define the longitudinal reinforcement at the specified location along element length. This allows precise specification of reinforcement along element length.

2.3. The seismic loads model

The seismic action on structures for the adopted approach is modeled as lateral forces with specified initial distribution and incrementation rules to predefined magnitude level under constant gravity loads. Eurocode 8-1 [1 – 3] requires the use of at least two lateral seismic load vertical representations specified as uniform that is proportional to the mass regardless of elevation and modal that is consistent with lateral force distribution in elastic analysis. SeismoStruct [23] software provides the options to apply both constant (conventional) and adaptive (modal) load distributions [23] the later based on lateral load vector updating on every incrementation step to reflect current eigen vectors. The software also provides options for implementing force based and displacement based incrementation controls. Based on the above, two sets (in X – short and in Y – long building direction) of loads composed of six lateral load distributions respectively were initially used.

The internal names used for the different loading arrangements have the form of X-F_rect, where the first letter (X or Y) specifies the direction of loading, the second letter (F or D) the type of load incrementation – force or displacement, and the last part specifies the type of distribution: rect – conventional uniform, proportional to the floor masses, tri – conventional proportional to the floor masses and based on first mode eigen vector, and adapt – based on load vector updating on every incrementation step. Graphical comparison of resulting capacity curves for each lateral seismic load distributions are shown on Table 1.

The comparison of the capacity curves for different loading schemes confirms that the displacement based conventional incrementation of uniform vertical distribution of lateral loads is not representative for the considered whole building seismic performance case but is rather applicable to investigating soft story cases. The capacity curves for all other cases are similar in shape and indicate stable results with comparatively small dispersion. The comparison of capacity curves, based on force controlled incrementation, indicates that the adaptive case is averaging the estimates of the uniform distribution, which corresponds to that in the lateral force method, and the triangular (or fixed modal shape) distributions. The comparison of the two adaptive incrementation cases reveal slight differences possibly due to the different ways the initial force distributions were applied to the spatial structural model in SeismoStruct [23]. The values indicate similar initial structural stiffness in both horizontal directions.

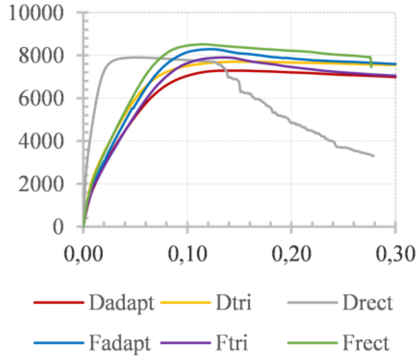
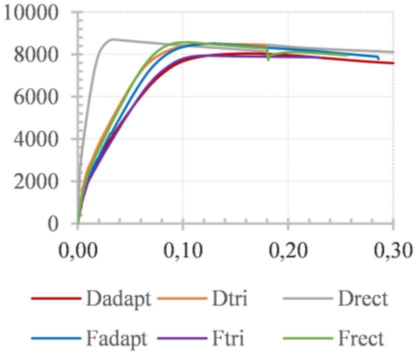
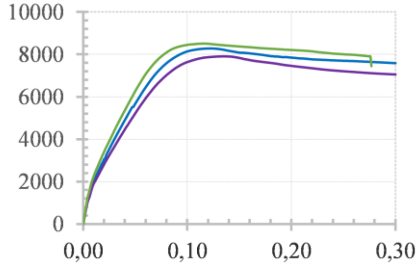
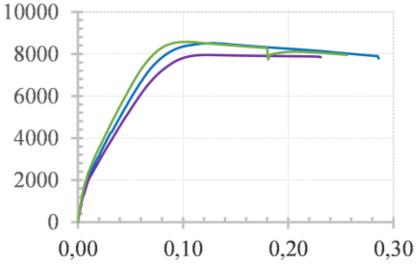
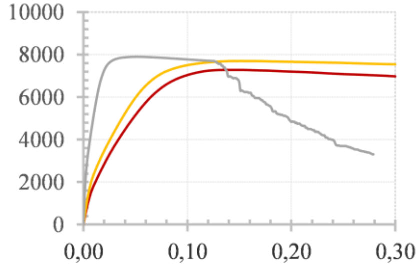
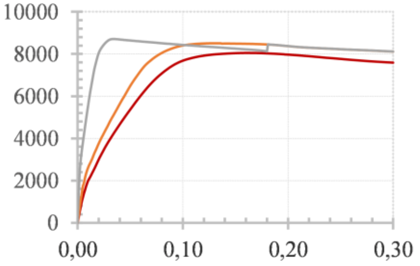
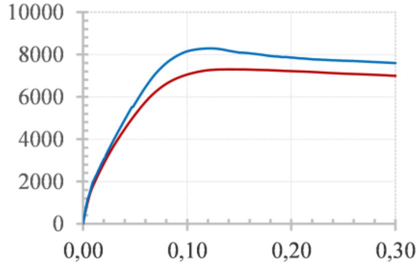
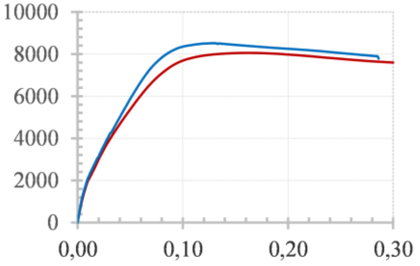
3. Results from the analysis

3.1. Structural performance levels

The separate parts of Eurocode 8 distinguish between new buildings design and assessment of existing and newly designed structures. Part 1 of Eurocode [1 – 3], which specifies new building design, defines two structural performance levels: the Near Collapse (NC_1) level and the Damage Limitation level (DL_1). The required structural performance is controlled by satisfying requirements for Ultimate Limit States (ULS) and Serviceability Limit States (SLS). Part 3 of Eurocode 8 [4 – 6] is focused on structural performance assessment and retrofitting, and recognizes three performance levels. The Near Collapse (NC_3) level is reached when the residual loadbearing capacity of structural members for gravity loads is still present, but secondary structural members are damaged to a large extent. The Significant Damage (SD_3) is reached when the structure possesses capacity to work under gravity but the capacity to resist lateral loads is reduced. The Damage Limitation (DL_3) level is present when structural members work almost entirely in the elastic range of their capacity curve.

The studied structure is designed to Part 1 of Eurocode 8 [1 – 3] for Near Collapse (NC_1) performance level. By taking into account that approximately $NC_1 \approx SD_3$, the structure also satisfies the performance level definition for Significant Damage (SD_3) of Part 3 of Eurocode 8 [4 – 6]. The target displacements for performance levels DL_3, SD_3, and NC_3 were evaluated under different loading arrangements. The results are shown in Fig. 3. The displacements of the center of masses at roof level obtained from the capacity curves, corresponding to the initiation of the plastic branch (d_y) of the bilinear idealization and reaching ultimate base shear at d_u, are also displayed.

Table 1. Capacity curves for different initial loading arrangements

	In X (short) direction	**In Y (long) direction)**
All distributions and controls		
Force-based loadings		
Displacement loads		
Adaptive incrementing		

Note: The vertical axis represents *Vb* – base shear value in kilo Newton, the horizontal axis *d* – displacement in meters of the center of masses at structure roof level.

The values of displacements defining the performance levels (DL3, SD3 and NC3) between different loading arrangements show small variability in both major structural directions (X and Y). The values for d_y and d_m defining the capacity curves are sensitive to initial lateral loads and incrementing procedures.

In X (short) direction

	Dadpt	Drec	Dtri	Fadap	Frec	Ftri
DL	0,078	0,029	0,070	0,074	0,069	0,078
SD	0,100	0,039	0,090	0,094	0,089	0,100
NC	0,173	0,073	0,157	0,164	0,154	0,173
d_y	0,071	0,061	0,072	0,065	0,065	0,077
d_m	0,144	0,146	0,122	0,115	0,115	0,135

In Y (long) direction

	Dadpt	Drec	Dtri	Fadap	Frec	Ftri
DL	0,078	0,029	0,070	0,074	0,069	0,078
SD	0,100	0,039	0,090	0,094	0,089	0,100
NC	0,173	0,073	0,157	0,164	0,154	0,173
d_y	0,071	0,061	0,072	0,065	0,065	0,077
d_m	0,144	0,146	0,122	0,115	0,115	0,135

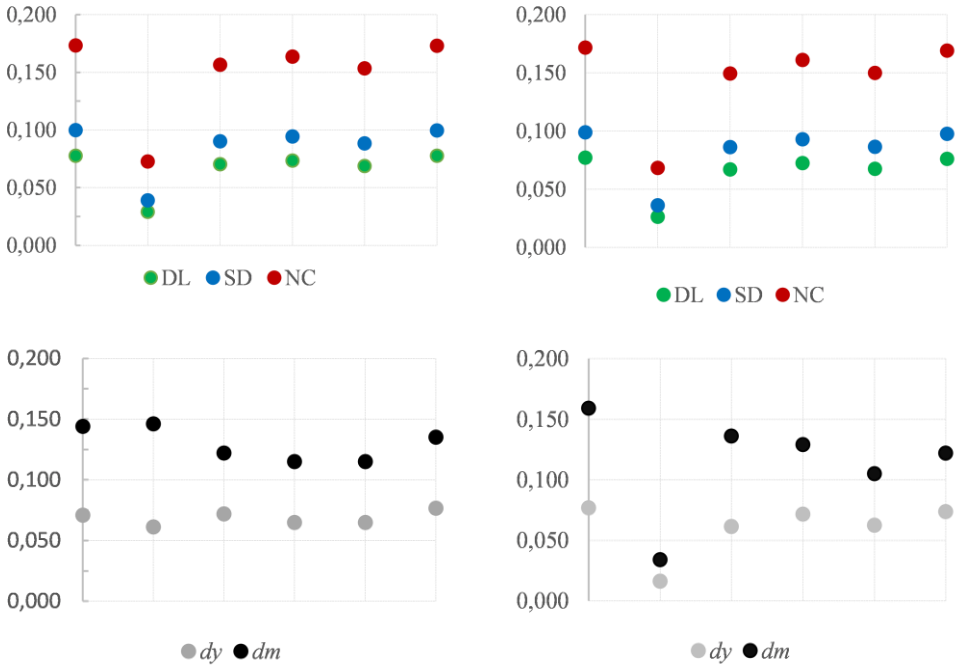


Figure 3. Structure roof displacements corresponding to code defined performance levels and capacity curve parameters

3.2. Structural over-strength ratio

Eurocode 8-1 [1 – 3] defines overstrength ratio α_U/α_1 as the ratio between lateral load when sufficient number of plastic hinges are formed to the load corresponding to first hinge creation. The code adopted value for this ratio without explicit numerical analysis for regular both in plan and in height reinforced concrete frame system structures is 1,3 and the maximum permitted value after analysis is 1,5. The overstrength ratio takes part in the evaluation of structure performance factor q by the relation:

$$q = q_0 k_w \; ; \qquad q_0 = 4,5\frac{\alpha_U}{\alpha_1} , \tag{1}$$

where k_w = 1,0 for the considered DCH framed structural framed system. Additional rules govern the cases of irregular in plan or in height structures. The structure performance factor is the link between elastic and non-linear structural design analysis. Although the code specified values are conservative, the applied limits to the overstrength ratio ($1,3 \leq \alpha_U / \alpha_1 \leq 1,5$) for framed systems, result in possibility to increase the initial value of q by up to 15 %.

We can assume that until reaching lateral load incrementation level of α_1 the structure behaves elastically. By increasing the lateral loads, the structure responds by developing plastic deformations and involves more of its members in the inelastic response until reaching its ultimate strength at α_U level of load incrementation. Further load increasing leads to deteriorated overall structural performance and collapse at the end. The definitions and value ranges of performance and overstrength factors are well described [7] and studied [13, 14]. By following the approach of [13 – 15] the overstrength ratio can be defined as overstrength performance factor Ω_1:

$$\Omega_1 = \frac{\alpha_U}{\alpha_1} = \frac{V_y}{V_1} = \frac{V_y^*}{V_1^*} , \tag{2}$$

where the (*) symbol indicates that the referenced values are transferred to the equivalent SDOF system, V_e and V_y are the base shears in a system with unlimited elastic performance and in a system with limited strength respectively.

By reaching the base shear V_1, the structure exits the elastic performance and its members are reaching their ultimate strength capacity. These capacities were established by response spectrum analysis where the design base shear value of V_d was used.

The difference between V_1 and V_d lies in their different nature. The value of q predefined in Eurocode 8-1 [1 – 3] has also the role to transform the analysis to the linear elastic response region, while the value from non-linear analysis reflects the structural performance as described by the model. The design base shear V_d is obtained in the structure, designed to resist seismic action defined by $S_d(T)$ design response spectrum by modal linear elastic analysis. The $S_d(T)$ spectrum is based on q performance factor defined by Eqn. 1. The base shear value V_1 is obtained through nonlinear analysis by identifying the first member yield on the structure capacity curve. Both values can be transformed to the equivalent SDOF system.

The value of the structural performance factor, referred here as q_1, can be expressed as in [13, 14, 17] by the equation:

$$q_1 = \frac{V_e^*}{V_1^*} = \frac{V_e^*}{V_y^*}\frac{V_y^*}{V_1^*} = q_\mu \Omega_1 . \tag{3}$$

This value differs from the value of the performance factor (q) in Eqn. 1. The difference is by a factor of $k = q / q_1$. Additionally strength representing q_S performance component referred to as observed overstrength in the literature [13, 14, 17] can be defined as $q_S = \Omega_d = V^*_y / V^*_d$ so that $q = q_\mu q_S = q_\mu \Omega_d$. After taking into account that $q = V^*_e / V^*_d$, where V^*_d is the base shear from response spectrum analysis, we obtain:

$$k = \frac{q}{q_1} = \frac{V_e^*}{V_d^*}\frac{V_1^*}{V_e^*} = \frac{V_1^*}{V_d^*} = \frac{V_e^*}{V_y^*}\frac{V_y^*}{V_1^*}\frac{V_1^*}{V_e^*} = \frac{q_\mu \, q_S}{q_\mu \, \Omega_1} \;\; ;$$

$$\frac{\alpha_U}{\alpha_1}=\Omega_1=\frac{V_y^*}{V_1^*}=\frac{V_y^*}{V_1^*}=\frac{V_y^*}{V_d^*}\frac{V_d^*}{V_1^*}=\frac{q_S}{k}\;;\;\; q_S=\Omega_1 k\;; \tag{4}$$

$$q=q_\mu\, q_S=q_\mu\Omega_1 k \quad\longleftrightarrow\quad q=k_w q_0=4{,}5\frac{\alpha_U}{\alpha_1}\,.$$

Values of k coordinate the relations between the linear response spectrum and the non-linear solutions. In the last line of Eqns. 4 the current performance factor q on the left references value obtained by solution while on the right is the code defined value. Eqns. 4 complete the set of internal dependencies between performance ratios and factors.

The described values and relations can be tracked in Fig. 4, where the results obtained by pushover analysis from displacement based adaptive loading for the X (short) building direction are shown.

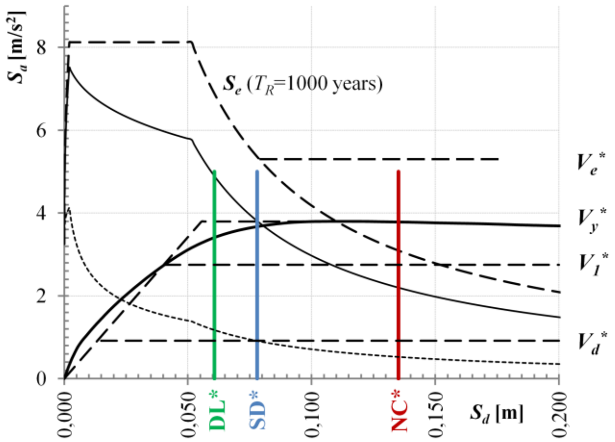


Figure 4. Background to behavior factors

The results for the overstrength ratio and performance factors from pushover analysis for different load arrangements and from linear response spectrum analysis are shown in Table 2.

The obtained results are in good agreement with those in other studies [13]. The values for k are higher than the average for RC framed structures [13], where average reference values of 1,33 for irregular and 1,46 for regular frames are reported. All obtained overstrength factor q_S values are higher than the established limit $q_S \geq 2{,}0$ for low- and medium-rise RC buildings.

The results indicate that the values for the overstrength ratios α_U / α_1 in both X (short) and Y (long) directions can be increased to 1,4 for the next structural design iteration.

All the results show that the designed structure possesses reserves in both strength and ductility under the considered seismic action.

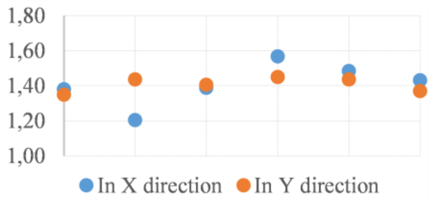
Eurocode 8-1 [1 – 3] uses the approach to magnify the reference peak ground acceleration $a_{g,R}$ by structural importance factor γ_I to apply different performance requirements to structures. This reflects in changing the mean reference return period T_R and the probability of exceedance P_R in T_L – the structure design lifespan. For the case the implementation of structure importance factor $\gamma_I = 1{,}2$ changes the mean reference return period T_R to 975

(≈ 1000) years from its reference value $T_{NCR} = 475$ (≈ 500) years and the probability of exceedance P_R to 5 % from its reference value $P_{NCR} = 10$ % in structure design lifespan of $T_L = 50$ years.

Table 2. Structure behavior factors

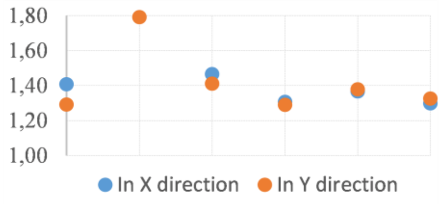
$\alpha_U / \alpha_1 = \Omega_1 = V^*_y / V^*_1$

	Dadpt	Drec	Dtri	Fadpt	Frec	Ftri
X	1,38	1,20	1,39	1,57	1,48	1,43
Y	1,35	1,44	1,41	1,45	1,44	1,37



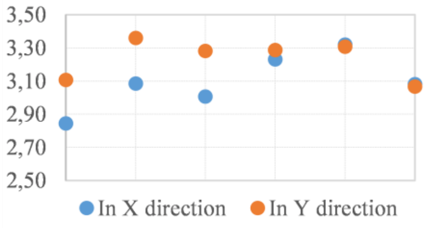
$q_\mu = V^*_e / V^*_y$

	Dadpt	Drec	Dtri	Fadpt	Frec	Ftri
X	1,41	1,97	1,47	1,31	1,37	1,30
Y	1,29	1,79	1,41	1,29	1,38	1,33



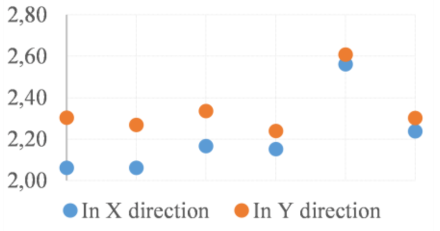
$q_S = \Omega_1 k = V^*_y / V^*_d$

	Dadpt	Drec	Dtri	Fadpt	Frec	Ftri
X	2,84	3,08	3,01	3,23	3,32	3,08
Y	3,11	3,36	3,28	3,29	3,31	3,07



$k = V^*_1 / V^*_d$

	Dadpt	Drec	Dtri	Fadpt	Frec	Ftri
X	2,06	2,06	2,17	2,15	2,56	2,24
Y	2,30	2,27	2,34	2,24	2,61	2,30



Note: The results from displacement controlled regular initial load distribution are displayed just for reference purposes and are not indicative for the considered structure.

3.3. Evaluation of structural expected damage levels

The structural expected damage under the action of design earthquake loading has been evaluated by adopting local displacement based damage index based on the Powell and Allabahadi [22] formulation of DI_μ to interstory drifts and Bracci et. al. [12] global damage index *gDI* based on weighted vertical loads:

$$DI_\mu = \frac{\delta_m - \delta_y}{\delta_u - \delta_y} = \frac{\mu - 1}{\mu_U - 1};$$

$$gDI = \frac{\sum w_i DI_i^{m+1}}{\sum w_i DI_i^m}; \quad \sum w_i = 1{,}0; \quad gDI \le 1{,}0; \tag{5}$$

$$w_i = \frac{(\text{Total tributary area})_i}{\sum (\text{Total tributary area})_i},$$

where δ_m is maximum interstory drift, δ_y – the drift at yield, δ_u – the drift value defining story failure, w_i – an importance factor based on story gravity loads, normalized to whole structure weight at foundation level, and m – a weighting factor taken here as 0,0.

The story drift ratio δ_r in Eurocode 8-1 [1 – 3] limit for DL_1 is $v\delta_r = 0{,}01h$, where δ_r is evaluated for design seismic action that corresponds to $\mathrm{NC}_1 \approx \mathrm{SD}_3$ performance levels and $v = 0{,}5$ for the considered structure. This formulation can be expressed as $\delta_{u,\mathrm{DL1}} = 0{,}01h$, $\delta_{u,\mathrm{NC1}} = 0{,}01h / v = 0{,}02h$ and $\delta_{u,\mathrm{NC3}} = (4/3)\,\delta_{u,\mathrm{SD3}} = 0{,}027h \approx 0{,}03h$ after taking into consideration the specified in A.3.2.3 clause of Eurocode 8-3 [4 – 6] ratio between member chord rotations $\theta_{um} / \theta_{\mathrm{SD}} = 4/3$ and the geometrical relations between drifts and chord rotations. In the current study the ultimate displacement $\delta_u = 0{,}03h$, where h is the relevant height, was taken to cause damage index $DI\mu$ to reach its ultimate value of 1,0 thus to indicate floor collapse. This value is within the 4 % of h threshold drift value specified in [11] where it is established that seismically detailed columns maintain stable lateral strength. The evaluation of the story yield drift ratios δ_y is based on the assumption of linear distribution of displacements in height when reaching the yield point in the structure capacity curve i.e. plastic deformations (damages) are starting to develop in all floors simultaneously. This assumption leads to conservative results both for local and global damage indices. Both the adopted values of displacements, corresponding to damage initiation δ_y and structural failure δ_u states, suggest that the computed damage index values indicate code tolerable damages, rather than level of observed or expected loss of structural bearing capacity for gravity loads or structural collapse.

Compliance between structural observed damages classed into damage states and story damage index DI_μ values is adopted as in the widely used formulation originating in [20, 21] and is given in Table 3. Although the values are calibrated against local damages in elements, they are also applicable to element assemblies. When applied to groups of elements on floor level the described damages are expected to occur only in some or even in one element, when DI_μ value is close to the lower boundary and in many lateral load-resisting elements, when the value is close to the upper boundary.

Table 3. Damage states

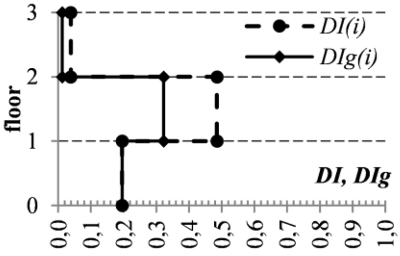
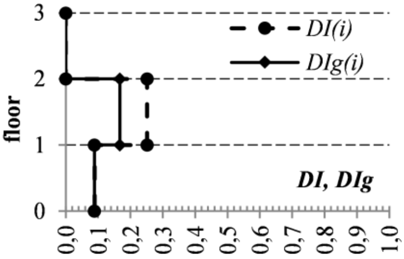
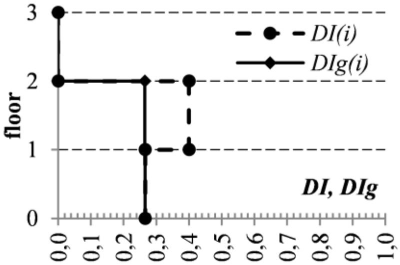
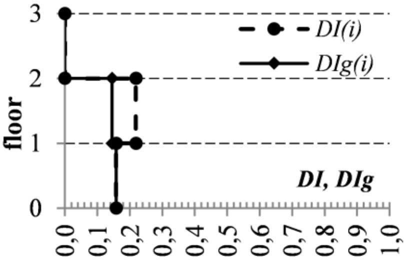
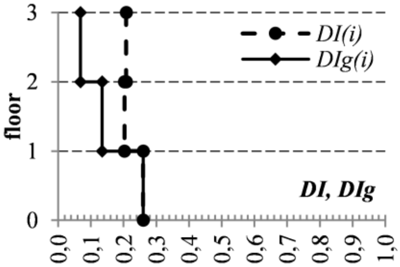
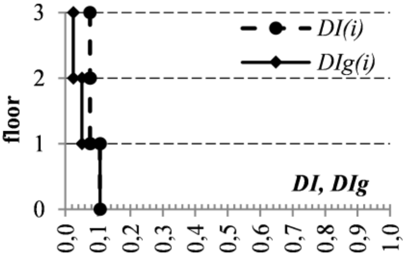
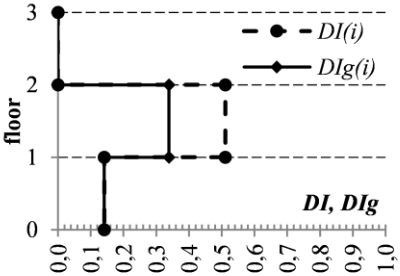
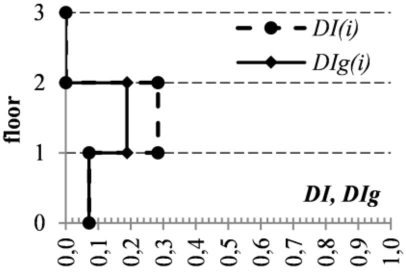
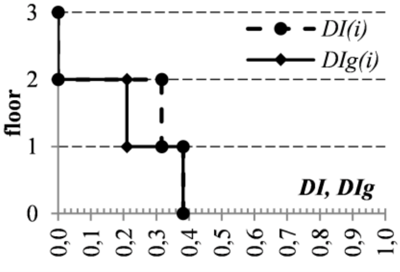
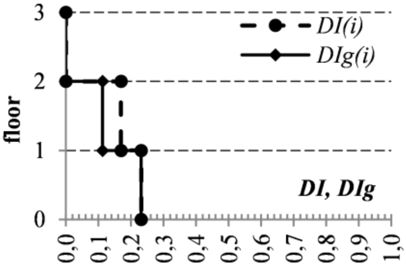
Damage state	**Values of DI_μ**		**Description of damages**
	from	**to**	
Slight	0,00	0,20 (0,25)	Initial cracks in structural elements.
Moderate	0,20 (0,25)	0,40 (0,50)	Significant cracks in structural elements. Concrete spalling in some elements.
Heavy	0,40 (0,50)	1,00	Significant large cracks in structural elements. Reinforcement buckling in some elements is observed.
Collapse	> 1,00	–	Partial or full collapse of the structure.

Note: The values in parenthesis mark possible variations in boundaries between the damage states.

The results for story DI_μ and global gDI damage indices are shown in Table 3. The obtained story damage index DI_μ values from all load procedures both in short (along axis X) and long (along axis Y) directions indicate that the structure will exhibit only slight to initial-moderate damages within some elements.

Two separate trends can be distinguished, one of them pointing to damages predominantly affecting the second floor, and the other suggesting comparable first floor involvement.

Table 4. Evaluated story and global damage indices at design earthquake level

	In X (short) direction	In Y (long) direction
D_adapt		
F_adapt		
D_tri		
F_tri		
F_rect		

3.4. Structural damage development sequence

Establishing the sequence of expected damage development by element sections is of primary concern for structural seismic performance evaluation. Results both in X and Y building directions suggest two different damage development sequences. The first sequence indicates damage initiation starting at internal columns on the $2^{\text{-nd}}$ floor ceiling locations and its development to column sections near the floor. The alternative sequence suggests damage initiation, starting at foundation level of frame internal columns, and its development to beams and in height. In all cases, the rotational column capacities θ_u are reached after roof displacements reach the NC_3 global structural performance level $d_{u,\mathrm{NC3}}$.

The established damage development sequences are in close agreement with the results for story damage indices DI_μ above for the different loadings.

Inclusion of foundations and taking into consideration of the soil-structure interaction effects in the structural model on next design iteration step is expected to shift the results towards the first mentioned damage development path.

4. Conclusions

The current evaluation of the seismic performance of newly designed RC framed system building confirms its adequate design to satisfy the desired performance level under earthquake actions. The obtained behavior factors values by static nonlinear analysis fall within the thresholds of other researches. The overstrength ratio results, the established damage development path(s) and the evaluation of damages caused by design earthquake action by the damage index approach indicate measures towards possible improvements in next design iteration steps. The approach can be used as a pattern for seismic performance numerical assessment and evaluation of overstrength factors.

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ОЦЕНКА НА СЕИЗМИЧНОТО ПОВЕДЕНИЕ НА СТОМАНОБЕТОННА СГРАДА С РАМКОВА КОНСТРУКТИВНА СИСТЕМА, ПРОЕКТИРАНА ЗА ВИСОК КЛАС ДУКТИЛНОСТ ПО ЕВРОКОД

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Ключови думи: *земетръсно поведение, дуктилност на конструкции, сеизмичен индекс на повреди, стоманобетонна рамкова конструкция, push-over анализ, нелинеен статичен анализ, завишена носимоспособност*

РЕЗЮМЕ

Проведено е числено изследване за установяване на земетръсното поведение на стоманобетонна сграда с рамкова конструкция, проектирана за висок клас на дуктилност (DCH) по Еврокод. Капацитивните криви, получени чрез нелинеен статичен анализ при различни начални разпределения на натоварванията, са съпоставени и с резултатите от линейния анализ със спектри на реагиране за установяване на коефициента на сеизмична завишена носимоспособност на конструкцията. Оценката на сеизмичното поведение на конструкцията е извършена чрез прието представяне на етажен индекс на повреди. Използваният подход може да служи за образец при установяване на стойности на коефициента на завишена носимоспособност и оценка на поведението на конструкции при сеизмични въздействия. Получените резултати са индикативни по отношение на ниско- и средноетажни стоманобетонни рамкови конструкции, проектирани за висок клас на дуктилност (DCH) по Еврокод.

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