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SESSION G: CIVIL ENGINEERING

Pushover analysis with various material nonlinearity models

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ABSTRACT

In this paper, the nonlinear behavior of a reinforced concrete five-story building exposed to seismic action designed according to EN 1992-1-1 and EN 1998-1 is analyzed. In addition to the required load capacity, the local ductility conditions are also met in the critical sections of the structure. Nonlinear static (pushover) analysis of the structure subjected to different lateral force distributions (uniform and modal) was performed using four material nonlinearity models (infrmFB, infrmFBPH, infrmDB, and infrmDBPH) for two orthogonal directions. The analysis was carried out using the SeismoStruct software package for seismic analysis of structures. The seismic performance assessment of the structure was based on the pushover analysis results for each applied nonlinear material model.

KEYWORDS

Pushover analysis, Material nonlinearity, Seismic performance

1. INTRODUCTION

Significant global attention has been devoted to seismic protection, as experience has shown that destruction during strong earthquakes can have catastrophic proportions, with the real possibility of a large number of human victims. Given that the seismic risk in urban areas increases over time, a number of experimental and theoretical studies have been carried out in the world in order to study the behavior of structures during earthquakes [1]. Proper calculation of the seismic resistance of structures is based on the concept of designing structures to achieve acceptable behavior due to seismic effects. The main goal is for buildings to resist weak earthquakes without damage, moderate earthquakes with controlled damage to non-structural elements, and for strong earthquakes, it is necessary to prevent building collapse through controlled damage (plasticization)[2].

Due to its mechanical properties, reinforced concrete is a material that can ensure the favorable behavior of structures exposed to seismic forces [3]. If properly designed and executed, a reinforced concrete structure can develop the necessary ductility and protect against local or global collapse. Skeletal structures are the most common application of construction. They are characterized by relatively high flexibility due to the development of relatively small seismic forces. For normal loading, the structure remains in the linear-elastic range of material operation, and the relationship between load capacity and deformations is simple. In the event of an earthquake, and depending on its strength, some parts of the structure move into the non-linear area of material operation.

The main goal of modern regulations and norms for the calculation of structures in seismic areas is to ensure a sufficient capacity for dissipating seismic energy during the design seismic effect, but without a significant reduction in the local and global bearing capacity for horizontal and vertical loads, i.e., without reaching the level and distribution of damage that would cause the collapse of the building [4]. This goal is achieved by applying programmed behavior in the design of structures. In EN1998-1 [4], the concept of seismic-resistant design is based on the controlled reduction of loads through the reduction of forces, which is achieved by applying a behavior factor that practically reduces the values of the elastic response spectrum. The required bearing capacity is determined for the design level of seismic impact, which is several times lower than the values that would appear in the elastic response. This allows for a non-linear response for a true seismic effect, where the structure during an earthquake is exposed to seismic forces approximately equal to its nominal load.

The seismic analysis consists of the calculation of the dynamic characteristics of the structure, the determination of seismic forces based on the mechanical properties of the structure of the object and the specified ground displacement, and the calculation of the impact on the structure due to the action of induced seismic forces [5]. When the influences in the structure due to the action of seismic forces are determined, the dimensioning of all critical sections for the relevant combination of seismic action and other loads is carried out. After that, with the application of adequate structural solutions and the elaboration of details specific to aseismic design, the necessary bearing capacity and sufficient deformation capacity of the considered structure are ensured.

Nonlinear static analysis is a simplified method for evaluating seismic response that, along with its ease of application (compared to the nonlinear dynamic method), provides an accurate overall assessment of structural behavior [2]. Nonlinear static (pushover) analysis was employed in this study. This technique is widely recognized and utilized in numerous studies focused on evaluating the seismic performance of structures [6], [7], [8], [9], [10]. The analysis was conducted using the seismic analysis software SeismoStruct [11].

2. NONLINEAR STATIC ANALYSIS

Nonlinear static analyses are often used in seismic calculations and evaluations of structures as indicators of structural yielding and possible failure mechanisms. The main advantage of such procedures compared to linear ones is that employing nonlinear analyses, the nonlinear characteristics of the material are directly taken into account, resulting in more realistic values of internal forces and deformations of the structure [2].

The capacity of the frame structure is determined by a non-linear static ("pushover") analysis. The pushover method represents an acceptable alternative to the complex nonlinear dynamic calculation method, which uses the equations of motion [12]. With this analysis, in addition to the gravity load, the structure is also exposed to the lateral forces from the earthquake, and in this way, a capacity curve known as a "pushover curve" is obtained [5]. The capacity curve is usually determined to show the first form of structural response based on the assumption that the structural response is governed primarily by the fundamental mode of vibration. The goal of this analysis is to define the dependence of the displacement of the control node (usually a point on the last floor or roof) and the shear force at the base that causes the first yielding in the structure. Plastic hinges form when the sectional capacity of individual members is reached. The incremental lateral load continues to be applied until the "target displacement" is reached and a non-linear static capacity curve is formed. To obtain the capacity curve, which represents the horizontal displacement of the top of the structure, pushover analysis is applied up to the control displacement value corresponding to 150% of the target displacement value. Given that the structure is a system with several degrees of freedom, the pushover curve of such a system is converted into the pushover curve of an equivalent system with one degree of freedom so that the capacity of the structure can be compared with the given seismic demand [5], [13].

The force-displacement relationship can be established for any force and displacement, but is usually established for the total shear force and displacement of the top of the building. According to EN 1998-1 [4], the calculation is carried out for uniform and modal distribution of lateral forces. In the case of uniform load distribution, seismic forces in individual levels are proportional to floor masses, while in the case of modal distribution, seismic forces are distributed according to the height of the building according to the same law adopted in the elastic analysis.

With the design of the structure and adequate construction, it is necessary to prevent excessive damage to the supporting structure due to the impact of the design earthquake. Also, an appropriate degree of protection must be provided in the event of damage and limitations in the functioning of the facility. Pushover analysis evaluates the seismic performance of the object and combines the behavior of load-bearing and non-load-bearing elements, thus forming a description of the overall degree of structural damage for different levels of seismic action, which is a significant advantage of the pushover method [14].

The analysis to assess the seismic performance of the object in this work was carried out in the structural analysis program SeismoStruct [11]. SeismoStruct is a finite element-based program that has the ability to use various nonlinear modeling techniques. While calculating the behavior of spatial frames during analysis with static and/or dynamic effects, the program can analyze structural elements with the help of material and geometrically nonlinear models and calculate the response of the structure and its elements.

3. NUMERICAL ANALYSIS

A five-story reinforced concrete building (Figure 1) was analyzed in accordance with the provisions given in EN 1992-1-1 [15] and EN 1998-1 [4]. The building meets the requirements of regularity in terms of base and height. The frame construction of the building consists of six frames perpendicular to the longer side and four frames perpendicular to the shorter side of the building, with a frame spacing of 5 m in both directions. The basic dimensions of the building are 15 x 25 m, and the floor height is constant at 2.8 m. The columns are square-shaped with dimensions of 45 x 45 cm, and the beams are T and L cross-sections with dimensions: flange width 185 cm, flange thickness 16 cm, rib width 45 cm, and rib height 19 cm. The cross-sections as well as the adopted layout of the element reinforcement are shown in Figure 2.

Figure 1: Model of structure

During the analysis, the effects of seismic action and gravity (constant and variable) loads were considered. Seismic impacts were determined by multimodal analysis with a design spectrum for the horizontal direction. EN 1998-1 [4] prescribes the elastic response spectrum for horizontal seismic action with the following parameters: type 1 elastic response spectrum, soil category C (S=1.15, Tb=0.2s, Tc=0.6s, and Td=2s), maximum ground acceleration ag=0.2g, correlation factor due to damping η=1, and building significance factor γ=1. For the analyzed construction, a high ductility class DCH was adopted, for which a total behavior factor of q=5.85 was determined.

The dimensioning of the structural elements was carried out based on the authoritative combinations in the software package TOWER 8. For the dimensioning of the supporting elements, concrete C30/37 and reinforcements B500B were used. The columns are reinforced with 12Ø28 and are confined with stirrups UØ12/10cm over the length of the critical zone and over the remaining length UØ12/15cm. The beams at the support are reinforced in the upper zone with 5Ø20 and in the lower zone with 3Ø20, and in the field in the lower zone with 3Ø20 and in the upper zone with 2Ø20. Beam flanges are reinforced in the upper zone with Ø12/10cm and in the lower zone with Ø12/20cm in both orthogonal directions.

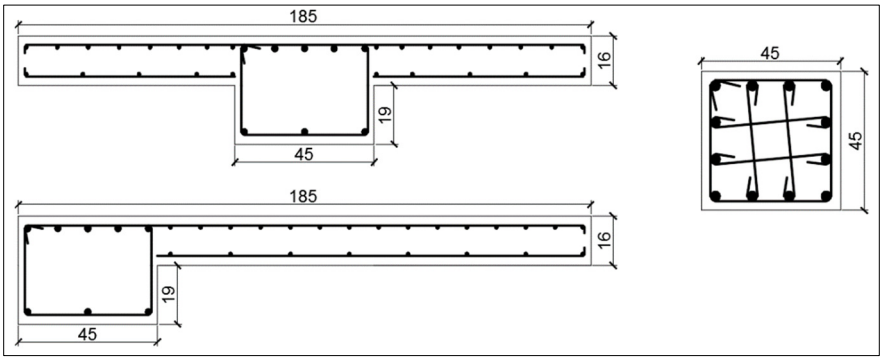

Figure 2: Cross sections of elements with adopted reinforcement

The steel and concrete models used in the SeismoStruct program are shown in Figure 3. The concrete model is based on the constitutive law for confined concrete proposed by Mander et al. [16], while the reinforcement steel is modeled using the Menegotto-Pinto [17] stress-strain relationship.

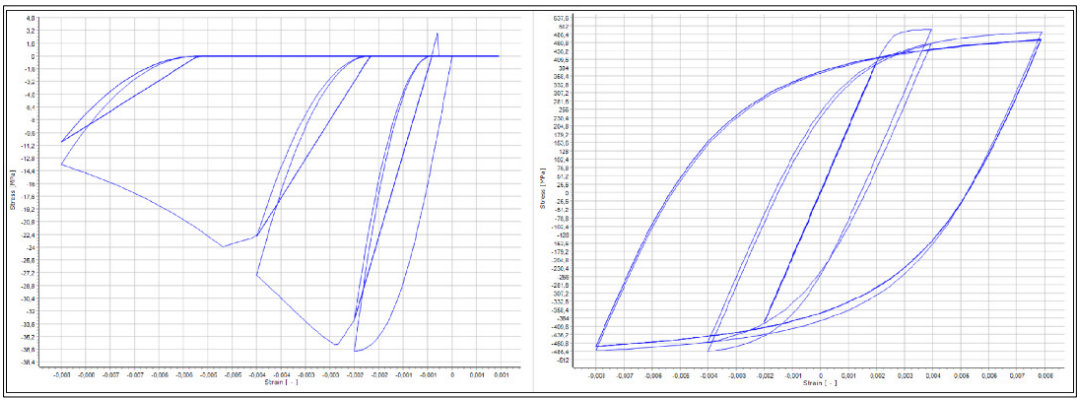

Figure 3: a) concrete and b) steel models from SeismoStruct

In addition to the required load capacity, the conditions of local ductility are also met in the critical sections (Table 1) [4], [15].

Table 1: Satisfying the condition of local ductility (EN 1998-1)

Beams	Columns
Reinforcement coefficient of the tensioned zone ρ=0,7486 % ≤ ρ_{max}=0,752 % Compression zone reinforcement coefficient ρ'=0,4489 % ≥$\rho_{tens}/2$ Length of the critical zone l_{cr}= 1 m ≥ $l_{cr,des}$ = 0,652 m	Total reinforcement coefficient ρ_{min}= 0,01 ≤ ρ = 0,0334 ≤ ρ_{max} = 0,04 Length of the critical zone l_{cl}/h_c = 2,45/0,45 = 5,44 > 3 →l_{cr} = 0,675m Fulfillment of requirements for adequate section confinement $\alpha \cdot \omega_{wd} \geq 30 \cdot \mu_\phi \cdot \nu_d \cdot \varepsilon_{sy,d} \cdot \frac{b_c}{b_0} 0{,}035$ (0,333>0,2406)

After defining all the geometric and mechanical characteristics of the structure, a non-linear static "pushover" analysis was carried out (Figures 4 and 5) [11]. The procedure was carried out for two orthogonal directions (X and Y) and two distributions of lateral forces - uniform and modal distribution. Four different models were created in the seismic analysis program to show the different effects of material nonlinearity: inelastic frame element (*infrmFB* and *infrmDB*) and inelastic plastic-hinge frame element (*infrmFBPH* and *infrmDBPH*).

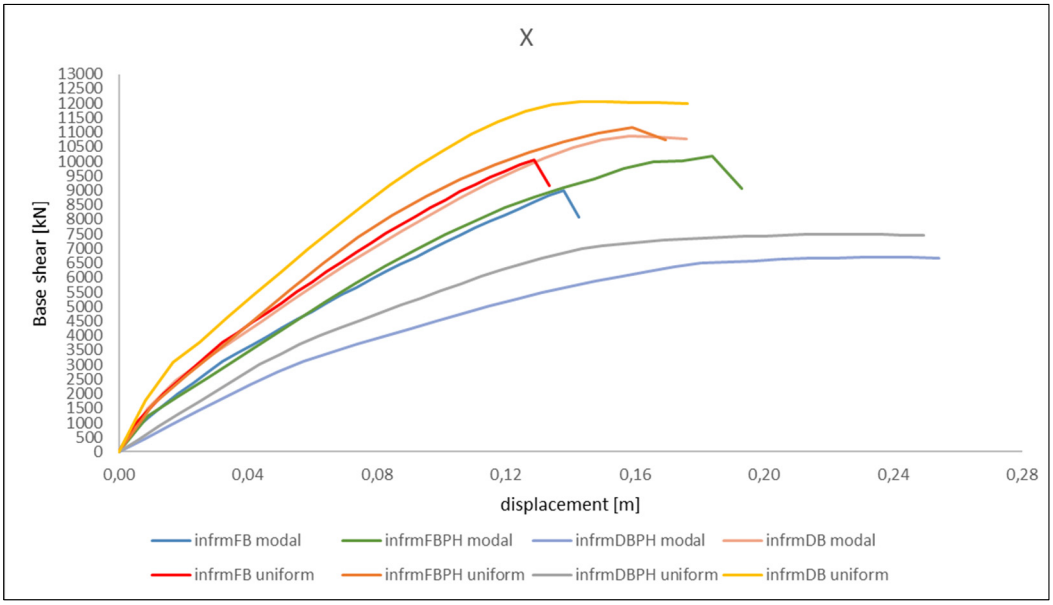

Figure 4: Pushover curve for X direction (SeismoStruct)

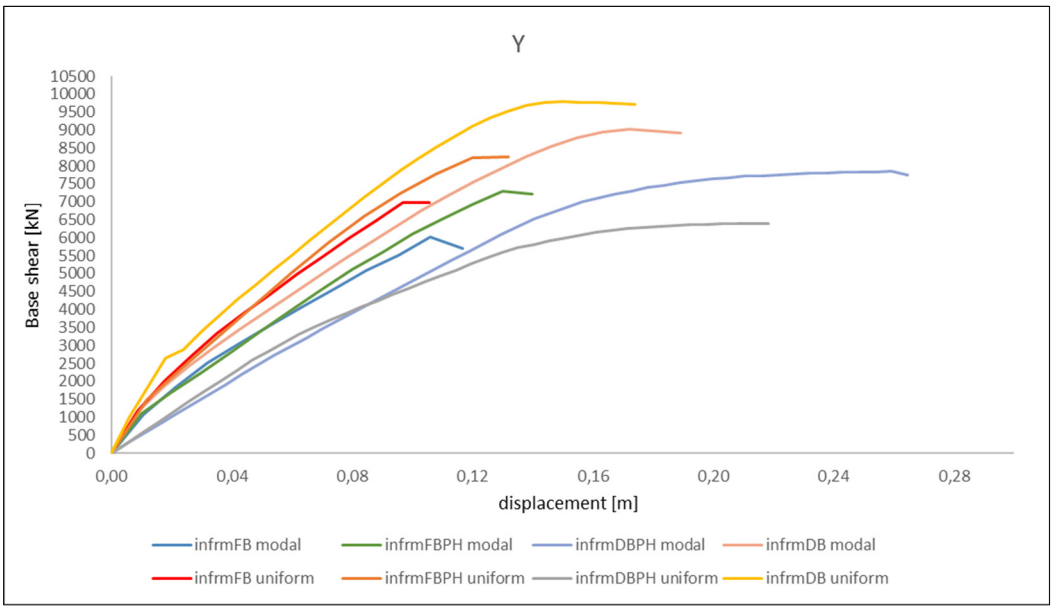

Figure 5: Pushover curve for Y direction (SeismoStruct)

InfrmFB- To simulate the nonlinear response of reinforced concrete frame structures, it is common to use simplified models such as two-node beam-column elements. Among these, force-based elements (Flexibility-FB) are especially popular because they predict longitudinal plasticity propagation, represent axial-flexural interactions, and satisfy force balance and strain compatibility [18]. The stress and deformation states in beam-column element sections are obtained by integrating the nonlinear uniaxial response of the material fibers. The section is divided into these fibers to capture inelasticity along the element's length and depth. Among SeismoStruct's four inelastic frame element types, the *infrmFB* element is the most accurate. It can represent inelastic behavior along the entire length of a structural member, even with only one element per member. The main advantage of this material nonlinearity model is its ability to use a single element to simulate a nonlinear response [11]. This reduces the number of degrees of freedom compared to using several displacement-based elements.

InfrmFBPH- The plastic hinge integration method overcomes problems with biased responses caused by deformations in force-based beam-column finite elements [19]. The method integrates deformations over certain plastic hinge lengths at the ends of the element. A positive feature of this method is that it gives exact solutions for linear problems. Incorporating the plastic length of the hinge into the integration method provides an objective response of elements and sections, which is important for the softening behavior of reinforced concrete structures. Plastic rotations are defined consistently and related to deformations in plastic hinges [19]. The advantages of such a formulation are not only a shortened analysis time (fiber integration is performed only for the two-membered end part) but also a complete control of the length of the plastic hinge (expansion of inelasticity), which allows overcoming the localization problem.

InfrmDB- This formulation is based on element stiffness. The displacement-based approach follows standard finite element method procedures where cross-sectional strains are interpolated from an approximate displacement field [20]. To approximate the response of a nonlinear element, a constant axial strain and a linear curvature distribution are applied along the entire length of the element (correct only for prismatic linear elements). Consequently, *infrmDB* elements should be used with small length members, leading to the need for mesh refinement, to achieve good accuracy in the case of higher-order strain distributions. It is usually necessary to define 4 to 6 elements per structural element [20].

InfrmDBPH- The plastic hinge integration method integrates deformations over specified lengths of plastic hinges at the member ends [19], [20]. All non-linear deformations of the element are collected in these rotational springs, while the rest of the element remains elastic. The *infrmDBPH* element consists of 3 sub-elements, two joints at the edges of the member, which model plastic rotational deformations around the second and third local axes, and an elastic element in the middle, which models the part of the element that remains elastic.

Based on the obtained calculation results for two different lateral load distributions, a very similar form of force-displacement dependence is noticeable, as well as the fact that the differences between the base force values at the same horizontal displacement value do not exceed approximately 10%. This indicates the regularity of the construction and the achieved uniformity of stiffness and load capacity for the height of the building. A manual verification of the results was performed. The calculation of the target displacement, as well as the comparison with the results from the software, is shown in Table 2 for the modal lateral load distribution and Table 3 for the uniform lateral load distribution. Figure 6 shows the relative floor displacement of the object in the X and Y directions.

Table 2: Calculation of target displacement for modal lateral load distribution

	Γ	F_y*	d_m*	E_m*	d_y*	T*	$S_{e(T*)}$	d_{et}*	d_t*	d_t	SeismoStruct
DB X	1.06	10856.61	0.15	395.66	0.23	0.65	5.18	0.06	0.06	0.06	0.06
DB Y	1.06	9011.74	0.17	399.84	0.26	0.75	4.51	0.06	0.06	0.07	0.07
DBPH X	1.06	6693.19	0.24	623.58	0.29	0.93	3.65	0.08	0.08	0.08	0.08
DBPH Y	1.06	7843.11	0.24	723.00	0.29	0.86	3.92	0.07	0.07	0.08	0.08
FB X	1.06	9942.93	0.23	888.81	0.28	0.75	4.51	0.06	0.06	0.07	0.07
FB Y	1.06	7652.32	0.23	703.92	0.28	0.86	3.94	0.07	0.07	0.08	0.08
FBPH X	1.06	10190.58	0.13	448.66	0.17	0.58	5.83	0.05	0.05	0.05	0.06
FBPH Y	1.06	8126.6	0.13	432.64	0.15	0.61	5.52	0.05	0.05	0.06	0.07

Table 3: Calculation of target displacement for uniform lateral load distribution

	Γ	F_y*	d_m*	E_m*	d_y*	T*	$S_{e(T*)}$	d_{et}*	d_t*	d_t	SeismoStruct
DB X	1.04	12042.04	0.16	558.24	0.23	0.61	5.54	0.05	0.05	0.05	0.05
DB Y	1.04	9796.84	0.15	392.32	0.22	0.66	5.11	0.06	0.06	0.06	0.06
DBPH X	1.04	7500.9	0.22	680.41	0.27	0.84	4.05	0.07	0.07	0.07	0.07
DBPH Y	1.04	6397.81	0.21	473.55	0.28	0.92	3.67	0.08	0.08	0.08	0.08
FB X	1.04	10802.06	0.21	882.08	0.25	0.67	5.02	0.06	0.06	0.06	0.06
FB Y	1.04	8527.14	0.19	594.68	0.25	0.75	4.49	0.06	0.06	0.07	0.07
FBPH X	1.04	11351	0.24	1037.85	0.30	0.71	4.74	0.06	0.06	0.06	0.06
FBPH Y	1.04	9228.28	0.12	562.3	0.13	0.52	6.50	0.04	0.04	0.05	0.06

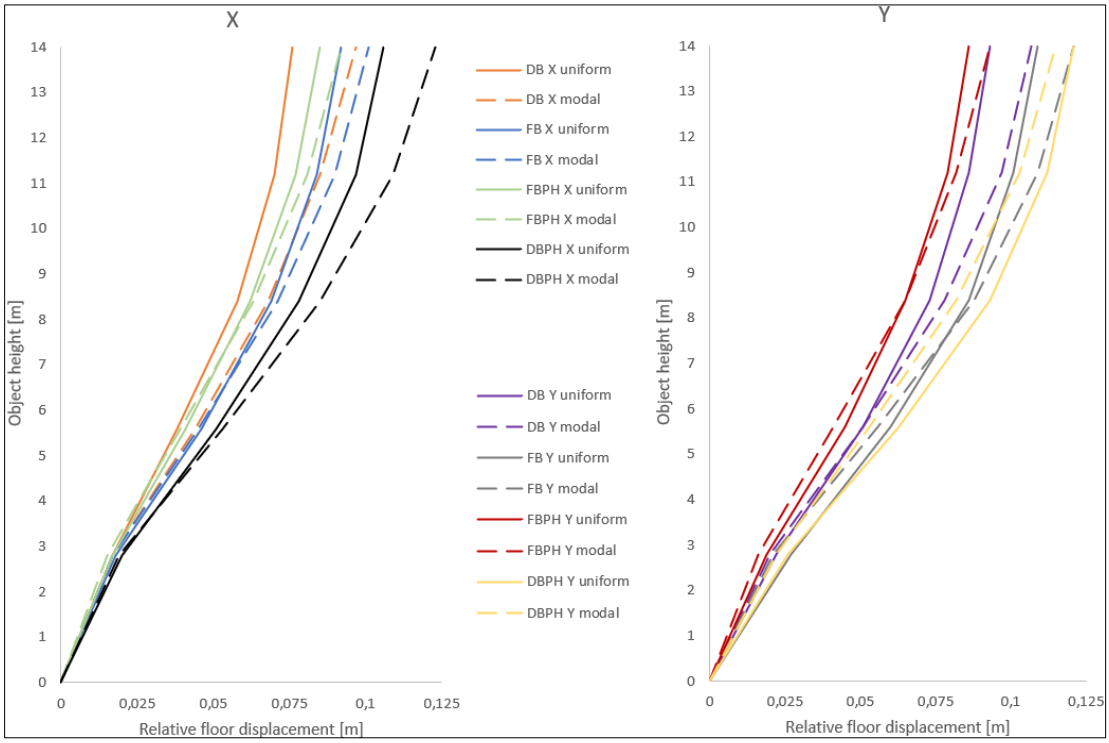

Figure 6: Relative floor displacement

The schedule for the appearance of plastic joints when reaching 150% of the target displacement is shown in Figure 7. The considered criteria for the appearance of plastic joints are concrete strain, yielding, fracture, chord rotation, and shear capacity. As can be seen in the picture, plastic hinges occur when shear capacity and yielding conditions are exceeded.

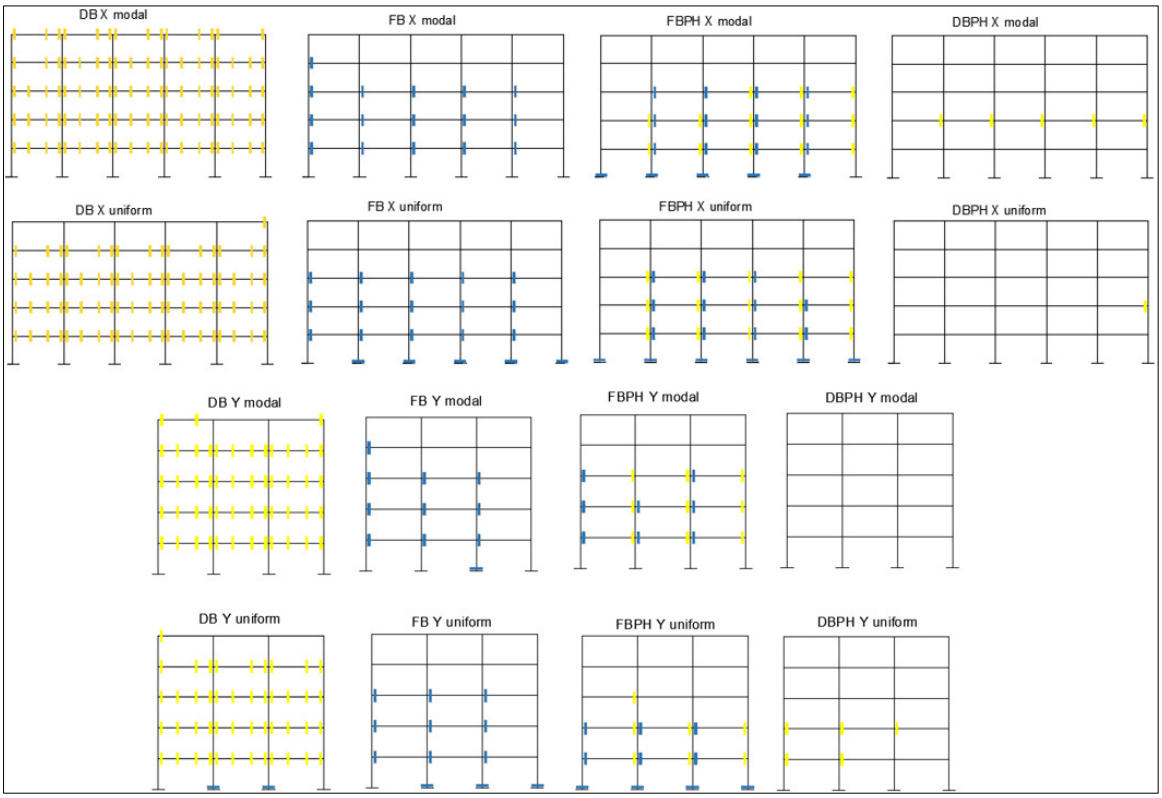

Figure 7: Schedule of appearance of plastic hinges for X and Y direction (yellow-shear capacity; blue- yielding)

Total chord rotation capacity θ_u is determined according to the expression (A.1) from EN 1998-3 [21], based on the formulations proposed by Panagiotakos and Fardis [22] and Biskinis and Fardis [23]. As shown in Table 4, the limit value of chord rotation at 150% of the target displacement was not exceeded anywhere.

Table 4: Calculation of chord rotation capacity

	Modal		Uniform	
	demand	limit	demand	limit
DB X	0.00106	0.04088	0.00117	0.04145
DB Y	0.00128	0.04360	0.00145	0.04333
DBPH X	0.00691	0.07303	0.00698	0.07259
DBPH Y	0.00788	0.08114	0.00916	0.08272
FB X	0.00609	0.06834	0.00651	0.06296
FB Y	0.00748	0.07440	0.00881	0.07097
FBPH X	0.00364	0.06932	0.00435	0.06319
FBPH Y	0.00383	0.07214	0.00464	0.07026

4. CONCLUSION

Pushover analysis of a five-story reinforced concrete building subjected to different lateral load patterns (uniform and modal arrangement) was performed using four different material nonlinearity models for two orthogonal directions. By observing the results of the seismic performance assessment of this object, the following conclusions are reached:

- The uniform loading scheme provides greater initial stiffness and shearing capacity of the base compared to the modal load distribution.
- *infrmFB* model- When reaching 150% of the target displacement, plastic hinges appear due to exceeding yielding in the beams on the 1st, 2nd, and 3rd floors; the maximum chord rotation is 0.0075 rad. On the pushover curve graphs, this model has the smallest displacement values.
- *infrmFBPH* model- When reaching 150% of the target displacement, plastic hinges appear due to exceeding the shear capacity and yielding in the beams on the 1st, 2nd, and 3rd floors and yielding in the columns at the supports; the maximum rotation of the chord is 0.00383 rad. This model has minimum relative floor displacement values for the Y direction.
- *infrmDB* model- When reaching 150% of the target displacement, plastic hinges appear due to exceeding the shear capacity in the beams on all floors. The chord rotation is the smallest in this model and is 0.00128 rad. This model has minimum relative floor displacement values for the X direction.
- *infrmDBPH* model- When reaching 150% of the target displacement, plastic hinges appear due to exceeding the shear capacity in the beams on the 1st and 2nd floors. The rotation of the chord is the largest in this model and is 0.00788 rad. This model has maximum relative floor displacement values. On the pushover curve graphs, this model has the highest displacement values.

Based on the analysis of the results, the models with the mean values of the considered criteria (shear capacity, yielding, relative floor displacement, chord rotation capacity) are *infrmFB* and *infrmFBPH* models. The *infrmDB* model gives the smallest values of chord rotation and relative story displacement, while the *infrmDBPH* model gives the highest values.

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