Vulnerability of Damaged Structures: The Concept of the Scenario of Related Non-Linear Analyses

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Abstract:

The research presented in the paper employed the general approach in the analysis of robustness of reinforced concrete skeletal (frame) structures in the events of severe damage caused by strong earthquakes. Such damage requires adequate rehabilitation measures. Similar problems are related to incident situations, such as the structures which were bombed in Serbia in 1999, fire effects or sudden collapse of one or several structural elements. In comparison to the classical approach, where the data on the condition of the building are obtained using non-destructive methods, by taking samples, by empirical assessment or by modelling using the linear analyses, taking into account the ultimate limit state and service life calculation according to the regulations (Codes), the procedure presented in this research enables to make a decision on the structure condition by the simulation through the scenario of mutually related analyses. The developed procedure is based on the non-linear numerical analyses, observing the principles of performance based seismic design (PBSD). The example of a 10 storey building was used to present the developed procedure which determines adaptability of the system in an incident situation, which is, in addition, exposed to the earthquake action. The building is designed according to EN 1992 and EN 1998, observing the capacity design method calculation (CDM). The stiffness matrix for the vertical action effects is used as the initial stiffness matrix for the non-linear analysis, which simulates the collapse of individual columns at the base floor, forming a couple of possible scenarios. The stiffness matrix at the end of the non-linear analysis, which simulates the collapse of individual columns, is used as the initial stiffness matrix for the non-linear static pushover analysis for bidirectional seismic action (X and Y directions). The target displacement analysis was conducted according to the capacity spectrum method, considering the response spectrum generated according to ATC 40 regulations. The assessment of the building condition and vulnerability analysis was conducted based on the calculated global and inter-storey drifts. Regarding that according to PBSD methodology, performances of the system are considered applying non-linear numerical models, bringing it to the near-collapse, i.e. collapse state, the implementation of the procedure developed in this manner, enables making reliable decisions about the building condition. This is very useful for defining the rehabilitation, strengthening strategies, or for potential partial change of statical system of the building.

Keywords: vulnerability, risk analysis, reliability, robustness, structural failure, structural integrity, performance, redundancy, partial and progressive collapse

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1. Introduction

The prevention of collapse is a general safety problem for structural engineering. Various methods have recently been introduced in structural design in order to minimize risks resulting from different actions including strong earthquake. Progressive collapse may occur as a result of all kinds of incidental loads, i.e. schematic event → local damage → progressive collapse. This implies a new approach to structure safety, often called "the philosophy of structural design" [10]. The location, type and magnitude of incidental loads are unpredictable variables, therefore it is impossible to describe their influence on buildings. A progressive collapse is a chain reaction, which occurs after the damage, or loss of load bearing capacity of a small part of a structure under incidental loads [5]. Recently in structural design was increased attention to provide robustness as properties to withstand progressive collapse of the structures. Namely, when several structural member suddenly fail due accident, fire, attack or earthquake and the building collapses progressively (complete failure or major part of the building) [4] and [6].

Many of the contemporary building codes specify that the consequences of structural failure should not be disproportional to the effect causing the failure. The level of robustness of a structure should be analyzed in terms of the causes and consequences of failure; i.e. the consequences of structural damages should not be disproportional to the original cause. Most important seismic response parameters influenced on effects of repair methods of RC structures are stiffness, strength and ductility [1]. The design and details of member and parts is particularly important for achieved robustness of structures. The Eurocode EN 1990 and EN 1991 [10], [11], [12] provides the general principles for achieving structural robustness. Authors in investigation [16] on steel frame structures established that the number of stories and bays (spans) increased, the capacity of the structure to resist progressive collapse under lateral loading also increased, because additional elements participated in resisting progressive collapse.

The paper presents investigation regarding the vulnerability to progressive collapse of a model representing RC framed building when its seismic design was made according to the provision of the EN 1992 and EN 1998 using the capacity design method. The aim of those investigations is analysed performance mention RC frame structure and it's ability to resist progressive collapse with serious damage columns in ground floor in different locations under seismic action (different scenario). Impact of strong earthquake or bomb impact may cause a critical column as a result of overstress and or buckling to lose a part or whole its load bearing capacity. For realization this aim procedure based on the non-linear numerical analyses, observing the principles of performance based seismic design is developed. The example of a 10-storey building was used to demonstrate the developed procedure, which determines adaptability of the system in an incident situation.

2. Vulnerability and robustness of structures

The consequences of structural failures are divided into: direct and indirect consequences. Direct consequences related to the vulnerability of the structures. Indirect consequences are related to loss of system functionality, as result of local failure, and are related to the robustness of the structures. A robust system has to keep its integrity even in the case of accidental actions. Relationship between robustness, vulnerability and consequences in term of the risk assessment framework adopted to analyse the robustness of structures can be present as:

Exposure → Vulnerability → Robustness (COST Action TU0601) [12].

The consequences of failure vary significantly and depend of factors: hazard, property of structures, and location of damaged element and robustness of structures. Robustness in EN 1991-1-7 defined the ability of a structure to withstand abnormal events without being damaged to an extent disproportionate to the original cause. Collapse resistance can be influenced in various ways. One possibility is through the structural robustness. In a robust structure, no damage disproportionate to the initial failure will occur. The quantification of robustness or related characteristics such as vulnerability presented and discussed in [13-17].

Principles and measures necessary for design and evaluation of structural robustness [17] are:

- identification and specification of accidental design situations and actions,
- verification of overall stability and stiffness of a structure,
- verification of vulnerability of structural elements and details,
- indirect design of alternative load paths by means of internal and perimeter, horizontal and vertical ties.

Guidelines and other FEMA document related to seismic rehabilitation of buildings are given in [7-

9]. Basic principles of risk identification and assessment comprise structural system, type of structural element (brittle or ductile), identification of hazards and hazard scenarios, probability for local damage and global failure, quantitative analysis of direct and indirect consequences of damage and failure. If one or more elements fail, the remaining structure is able to redistribute the loads and thus prevent a failure of the entire structure. The loss of a major structural element typically results in load redistributions and member deflections (structural redundancy). It shall be verified that the structure has sufficient redundancy and possibilities to mobilise so-called alternative load paths. Redundancy significantly increases system robustness, and is often the only viable means of ensuring sufficient robustness. Achieving redundancy may be difficult and or expensive, and is therefore often limited to critical parts of the system [6].

3. Scenario of related non-linear analyses

The concept of the scenario of related non-linear analyses, presented in this research is based on determining the structural condition while observing the Performance-Based Seismic Design (PBSD) principles. The classical approach in the assessment of a damaged structure starts from the non-destructive methods, sample taking, empirical assessment and design of structures using linear analyses. The ultimate limit state and service life calculations are conducted according to the regulations, with contemporary European codes EN 1992 [2] and EN 1998 [3] being primary. However, these codes do not define in detail the areas in which are considered buildings, which were during their service life damaged by incident action, such as the effects of the projectiles (structures in Serbia bombed in 1999), sudden collapse of one or several structural elements and similar incident situations. On the other hand, the aspects indicated by the European regulations for calculation of damaged structures, mostly apply to the implementation of linear statical analyses and design of reinforced concrete (RC) elements. The procedures defining the non-linear analyses of structures are minimized. In comparison to the European regulations, the American regulations FEMA 273 [7], FEMA 274 [8] and FEMA 356 [9] consider the assessments of conditions and rehabilitation of the structures, which were exposed to earthquake action in detail. However, they do not take in consideration other incident actions and relatedness of numerical models before and after sustaining damage. For the purpose of clearer defining and more accurate calculation of a such complex problem, the scenario of related non-linear analyses is developed, which must be performed in order to make adequate decisions on the condition of the structure. In figure 1 is presented a flow chart of scenarios of the related non-linear analyses.

In the first part, the calculation is conducted implementing the linear static analysis (LSA) and spectral modal analysis (SMA). Further, by designing in compliance with EN 1992 [2] and EN 1998 [3] codes, the necessary reinforcement in RC cross-sections is determined. In the second part a new 3D model of building is created for the non-linear analysis. At the first step of the second part, the static effects in the cross-sections of the structures are calculated only for vertical gravity actions (dead, super dead, live and similar loads). The system stiffness matrix at the end of this analysis is used as the initial stiffness matrix in the non-linear static pushover analysis (NSPA) for bidirectional seismic action (X and Y directions), which is conducted in the second step of the second part. As a non-linear response of the system the pushover curves, target displacement, forces and moments in cross sections, global drift (*DR*) and inter-storey drifts (*IDR*) are obtained. In the third part, the simulation of incident action on the structure is conducted, so the action causing decreases of bearing capacity, stability and serviceability of individual columns. The system stiffness matrix of the previous analysis is used as initial stiffness matrix of the non-linear analysis (NA) which simulates structural damage. In this case, development of non-linear strains in the system can be monitored, as well as the redistribution of static influences in the system. It is of particular importance that such analysis can serve for monitoring of plastic strain in the zone of severely damaged columns, i.e. monitoring of development of plastic hinges on the beams that are located in the local collapse zone. The number of possible scenarios n of structural damage and collapse of individual columns can be considerable. In the fourth part, the system stiffness matrix at the end of the structural damage analysis is used as initial stiffness matrix of NSPA analysis for bidirectional seismic action (X and Y direction), whereby the pushover curves are obtained as non-linear responses of the system, as well as target displacement, forces and moments in cross-sections, global and inter-storey drifts. Upon the completion of the structural calculation according to all the predefined scenarios, it is determined whether it is necessary to rehabilitate the structure, strengthen or partly alter the static system in the column collapse zone. If it is necessary to conduct some of the proposed procedures, the structure is redesigned, and possibly the procedure is reiterated for the purpose of verification and comparison of the obtained solutions. As it can be noticed, the calculation of the damaged structure conceived in this manner, simulates the realistic behaviour in the conditions of action and of incident situation and seismic actions, by firstly exposing the structure to the action of the gravitational load, and then by exposing the structure loaded in this way to the incident action. After that, the vulnerability of the structure is verified on the deformed and damaged structure, while the gravitational load is just transferred from the initial analysis to the following ones, and the incident and seismic action are defined in the corresponding analyses.

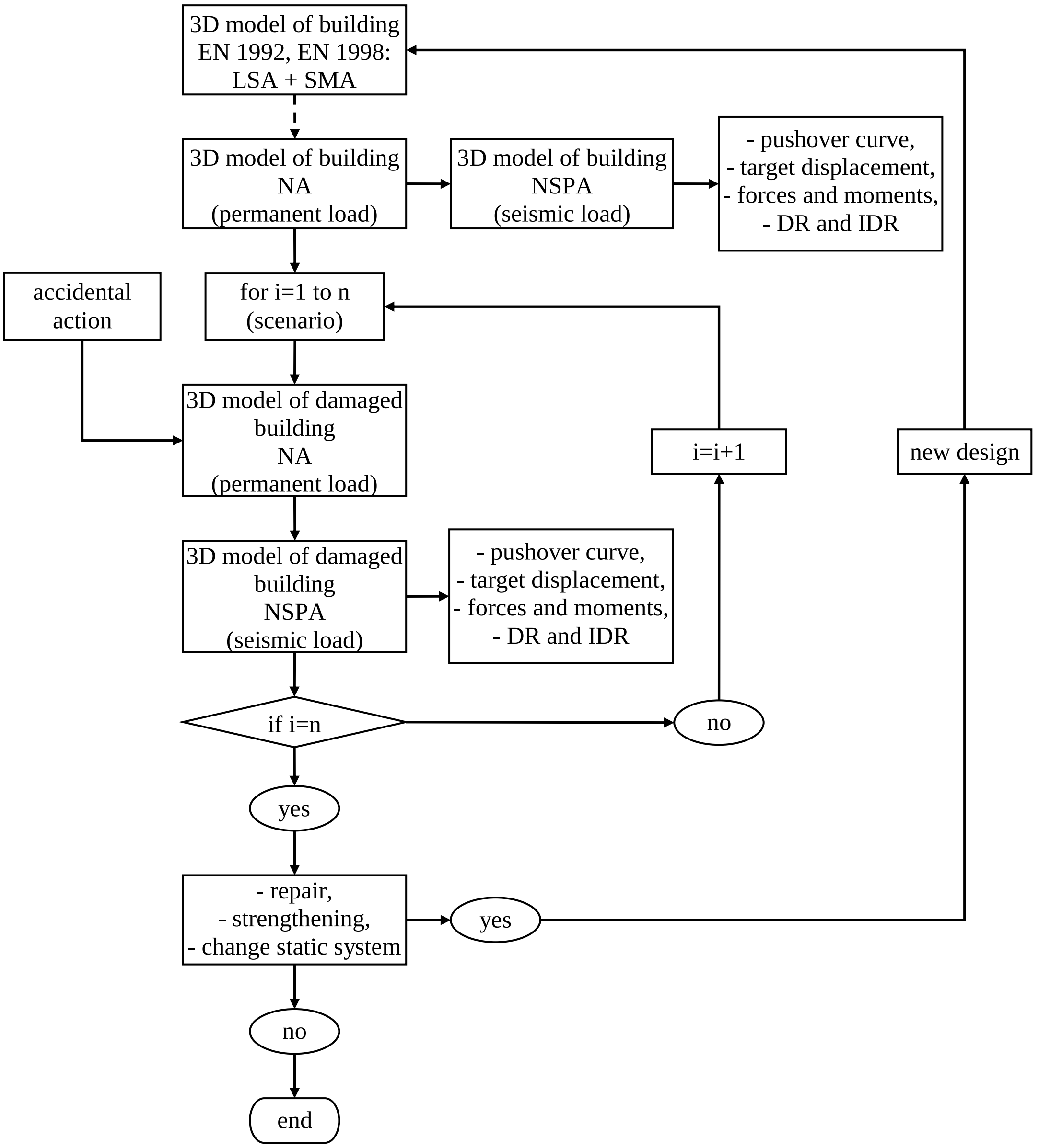


Figure 1. Flow chart of scenarios of the related non-linear analyses

4. Numerical analyses

Verification of the developed scenario of related non-linear analyses is performed on the example of a 10 storey buildings with a frame static system. In figure 2 are displayed the 3D model and layout of a 10 storey building. Previously the building was designed in compliance with EN 1992 [2] and EN 1998 [3] codes, taking into consideration the calculation concept according to the Capacity Design Method. The floor plan dimensions of the buildings are 36x24m, while the dimensions of a bay are 6x6m. The building is designed for concrete class C25/30. From the ground floor to the fifth floor, the dimensions of the external columns are 50x60cm, and of the internal ones 60x70cm, while from the fifth floor to the tenth floor, dimensions of the external columns are 40x50cm, and of the interior ones 50x60cm. From the ground floor to the fifth floor, the dimensions of the beams are 35x60cm, and from the fifth floor to the tenth floor they are 30x60cm. The thickness of floor slabs of all storeys is 20cm.

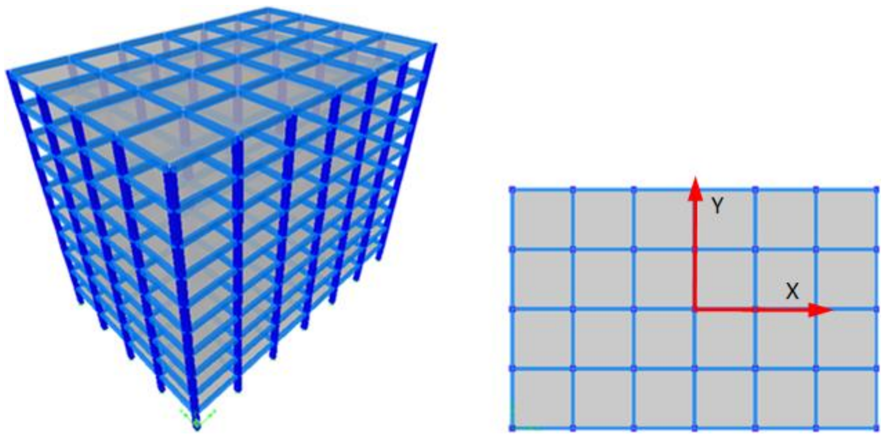


Figure 2. 3D model and layout of the 10-storey building

The building load is calculated as a permanent one (dead weight of the structural building elements and the additional permanent load g=3kN/m²), useful (occasional p=3kN/m²) and seismic load. The building was designed for the return period of reference seismic action of T=475g, design ground acceleration a_g=0.3g, ground type C, ductility class DCH and behaviour factor q=5.85. The building is insensitive to torsion since the stiffness centres coincide with the centres of mass of all the floors and are located along one vertical. However, in calculation of seismic effects, and additional value of eccentricity of 5% is assumed for both orthogonal directions, so for the seismic combinations, it can be considered that the building is moderately sensitive in terms of torsion. Seismic combinations are calculated for the bidirectional action of the earthquake. The number of characteristic modes, which was taken in consideration is 30, while in calculation of the static influences the P-Δ effects are taken in consideration as well. In all the nodes, the criterion that the ratio of the sum of the moments on the ends of the columns and the sum of the moments on the ends of the beams is higher than 1.3 is satisfied. Since structural system of the building is formed by columns and beams as linear elements, and slabs as surface elements, such buildings belong to the group of non-stiff and movable systems. For these systems, it is very important to limit the relative inter-storey drift according to $d_r v \leq 0.01h$, where h is the height of the floor, v is reduction factor which takes into consideration the lower return period of the seismic event and refers to the limit state of serviceability, d_r design inter-storey relative horizontal drift, calculated as the difference between mean horizontal drifts d_s at the top and the bottom of the observed floor [3]. In figure 3a are presented the global drifts DR of the 3D model of the frame building calculated using LSA and SMA analyses whose maximum values are lower than 0.2%, while in the figure 3b are presented inter-storey drifts IDR of the 3D model of frame buildings and ultimate drift values according to the previous expression, in the functions of the inter-storey drift IDR_{EC8} for X and Y directions. Maximum values of inter-storey drifts are lower than the limit value of the inter-storey drift IDR_{EC8} for v=0.5.

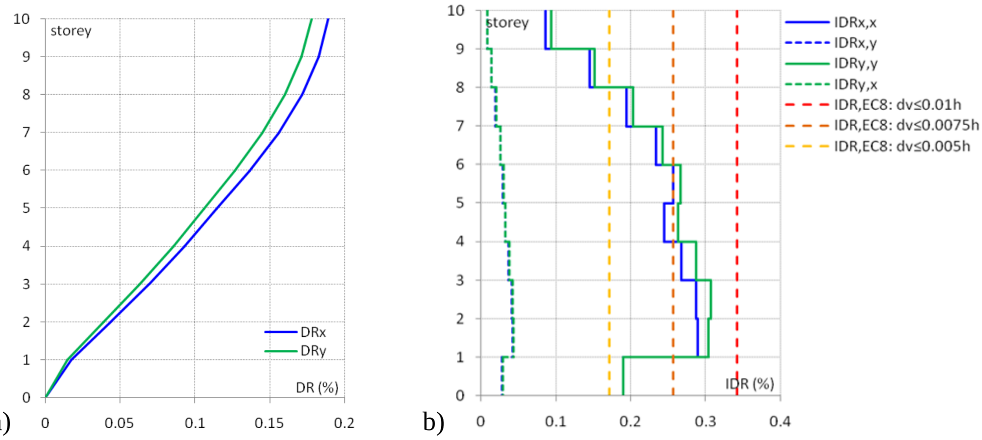


Figure 3. a) global drifts DR, b) inter-storey drifts IDR of the 3D frame building model

The sudden removal of a load-bearing element, ground floor column in this analysis, generates in the damaged structure dynamic effects and structural response is non-linear. Following the linear elastic analysis, a Damage-Capacity Ratio can be computed for each structural element [14]. In this paper we used performance criteria. The scenario of column collapse was conducted by elimination of individual columns of the ground floor of a 10-storey building. In figure 4 are presented nine scenarios of collapse states of ground floor columns (or sudden removal corner, edge and internal columns), with the following possible options:

- the scenarios displaying collapse states of outer columns only correspond to the possible damage of these columns due to the terrorist action comprising detonation of explosive in a vehicle parked in the immediate vicinity of the building,
- the scenarios displaying collapse states of inner columns only correspond to the potential damage of these columns due to the terrorist action comprising detonation of explosive planted inside the building,
- the scenarios displaying collapse states of both interior and exterior columns corresponds to the possible damage o these columns due to intensive earthquakes.

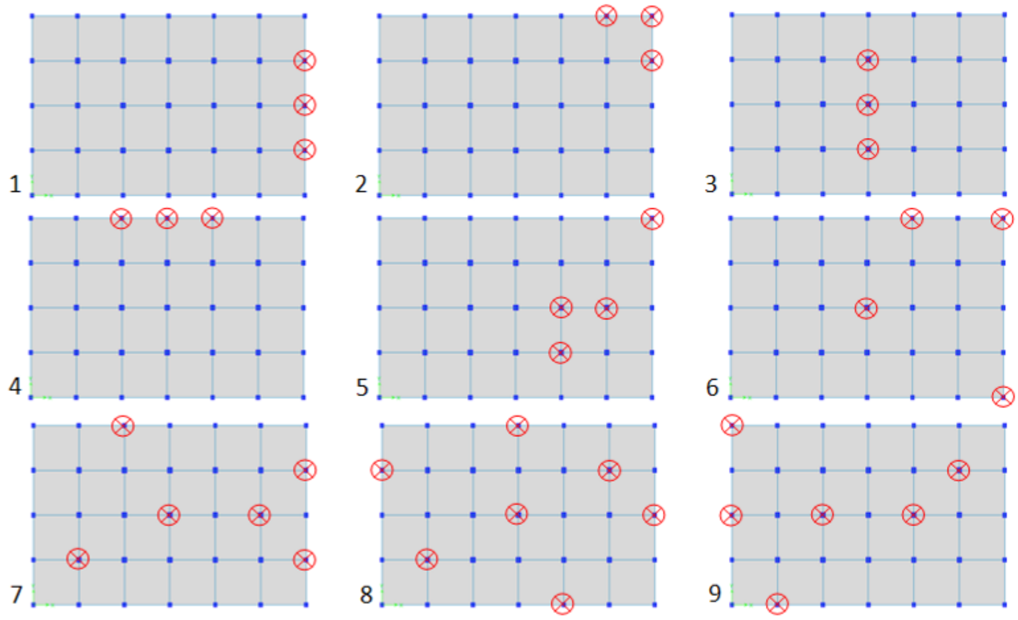


Figure 4. Scenarios of building damage (collapse of ground level columns)

In figure 5 are presented NSPA pushover curves for the conducted NSPA analyses and for undamaged buildings and for nine predefined scenarios. The non-linear behaviour of the buildings for the X direction was considered separately from that considered for the Y direction. In case of the X direction, the displacement of the highest node of the building in the centre of mass for the degree of freedom (DOF) in the X direction was monitored, while in case of the Y direction the displacement of the highest node of the building in the centre of mass for the degree of freedom in the Y direction was monitored. In the course of performing the NSPA analyses, bidirectional seismic action was assumed for each direction using the rule $1EQ_X+0.3EQ_Y$, i.e. $1EQ_Y+0.3EQ_X$. By considering the bearing capacity in the non-linear domain, for all NSPA pushover curves, it can be concluded that it is the highest for the undamaged building, which was to be expected. However, from the aspect of realized non-linear displacements, maximum displacements were obtained in the case of the first scenario for the X direction and the fourth scenario for the Y direction. The results obtained in this way describe the vulnerability level of the building in case when only outer columns are damaged. Considering initiation of stiffness in the linear domain, it can be stated that not all the NSPA pushover curves start from the zero. This is the result of application of related non-linear analysis, which assumes that firstly there were collapse states in the columns and then the NSPA analysis was conducted. For such scenarios, the level of initial drift is most frequently shifted towards the positive value. In certain cases, the levels of maximum realized drifts are lower than the maximum realized drifts, which were obtained for undamaged buildings. In such situations, the collapse of the buildings sets in sooner, so the lower the level of maximum realized drift, the sooner the collapse state of the building sets in.

In the second part of the research, target displacement analyses were conducted according to the CSM (Capacity Spectrum Method), assuming that the response spectrum is generated according to the ATC 40 [1] codes. In table 1 are presented the calculated target displacement parameters according to the CSM for PGA=0.3g for the undamaged building and possible scenarios, separately for the X direction and separately for the Y direction.

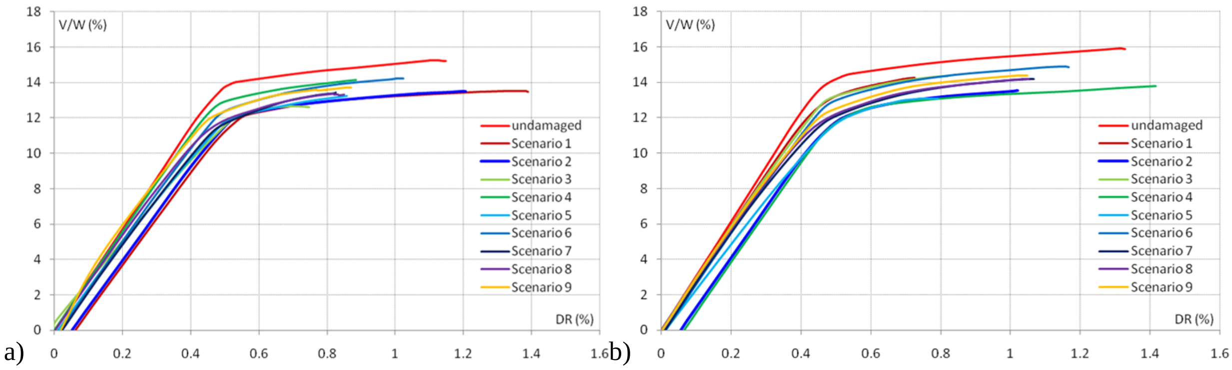


Figure 5. NSPA pushover curves for predefined scenarios: a) monitoring of the drift for degree of freedom in the X direction, b) monitoring of the drift for degree of freedom in the Y direction

Apart from the parameters, such as: the level of target displacement D_t, total shear force of the ground floor for the target displacement level V_t, effective period of vibrations for the target displacement level $T_{eff,t}$ and coefficient of effective damping for the target displacement level $\xi_{eff,t}$, also presented are departures in percents per scenario, in comparison with the model of the undamaged building. The effective vibration period for the target displacement level $\xi_{eff,t}$ corresponds to the secant period of vibrations in terms of capacity, i.e. in the ADRS (acceleration-displacement response spectra) format. This vibration period is considerably higher than the elastic period of building vibration, since development of elastic-plastic strain in certain cross-sections is taken into account. The most problematic scenarios for the X direction are the third, fifth and seventh, since for the seismic demand the target displacement could not be realized (N/A), i.e. capacity of non-linear deformations of the structure is too small. The most problematic scenarios for the Y direction are the first, third and fifth scenarios, where target displacement could not be realized, either. For all the scenarios of the column collapse, the values of effective period of vibrations for the target displacement level $T_{eff,t}$ and of the coefficient of effective damping for the target displacement level $\xi_{eff,t}$ increase, in comparison to $T_{eff,t}$ of the undamaged building. On the other hand, the total shear force of the ground floor for the target displacement level V_t decreases in all the scenarios, while the level of the target displacement D_t increases.

Table 1. Calculated parameters of the target displacement level according to the CSM method for *PGA*=0.3g

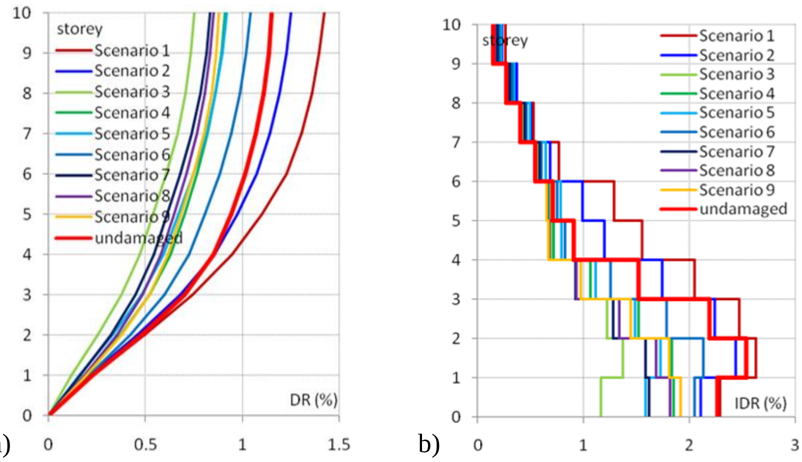
monitoring X DOF								
Scenario	D_t (cm)		V_t (kN)		$T_{eff,t}$ (s)		$\xi_{eff,t}$ (%)	
Undamaged	25.5		14407.9		2.74		19.9	
1	29.0	+13.7%	12834.9	-10.9%	2.95	+7.7%	20.9	+5.0%
2	28.9	+13.3%	12868.7	-10.6%	2.98	+8.8%	21.3	+7.0%
3	N/A	-	N/A	-	N/A	-	N/A	-
4	26.7	+4.7%	13770.6	-4.4%	2.90	+5.8%	21.1	+6.0%
5	N/A	-	N/A	-	N/A	-	N/A	-
6	27.6	+8.2%	13674.2	-5.1%	2.91	+6.2%	20.9	+5.0%
7	N/A	-	N/A	-	N/A	-	N/A	-
8	27.6	+8.2%	13088.4	-9.2%	3.03	+10.6%	22.1	+11.1%
9	26.2	+2.8%	13375.5	-4.8%	2.94	+7.3%	21.6	+8.5%
monitoring Y DOF								
Scenario	D_t (cm)		V_t (kN)		$T_{eff,t}$ (s)		$\xi_{eff,t}$ (%)	
Undamaged	25.4		14847.8		2.69		19.8	
1	N/A	-	N/A	-	N/A	-	N/A	-
2	28.9	+13.8%	13082.1	-11.9%	2.92	+8.6%	20.4	+3.0%
3	N/A	-	N/A	-	N/A	-	N/A	-
4	28.9	+13.8%	12980.5	-12.6%	2.89	+7.4%	20.8	+5.1%
5	N/A	-	N/A	-	N/A	-	N/A	-
6	27.3	+7.5%	14159.3	-4.6%	2.85	+5.9%	20.7	+4.5%
7	28.5	+12.2%	13645.4	-8.1%	2.98	+10.8%	21.4	+8.1%
8	28.1	+10.6%	13611.8	-8.3%	2.98	+10.8%	21.0	+6.1%
9	27.1	+6.7%	13792.1	-7.1%	2.88	+7.1%	20.5	+3.5%

In table 2 are presented the calculated parameters of the maximum realized displacement level (pre-collapse state) for the undamaged building and possible scenarios, separately for the X direction and separately for the Y direction. Apart from the parameters such as: maximum realized displacement level D_{max}, total shear force of the ground floor for the maximum realized displacement level V_{max}, effective period of vibrations for the maximum realized displacement level $T_{eff,max}$ and coefficient of effective damping for the maximum realized displacement level $\xi_{eff,max}$, also presented are departures in percents per scenario, in comparison with the model of the undamaged building. In this case the values $T_{eff,max}$ and $\xi_{eff,max}$ are not uniform (unequivocal), but are both positive and negative since the maximum displacements are reached in the scenarios, which are both higher and lower than the maximum displacements of the undamaged building. The total shear force of the ground level for the maximum realized displacement level V_{max} is lower in all the scenarios in comparison to the V_{max} of the undamaged building. The situation is similar for the maximum realized level of displacement D_{max}, except for the first scenario of the X directions and fourth scenario of the Y direction.

Table 2. Calculated parameters of the maximum realized target displacement level (pre-collapse state)

monitoring X DOF								
Scenario	D_{max} (cm)		V_{max} (kN)		$T_{eff,max}$ (s)		$\xi_{eff,max}$ (%)	
undamaged	37.9		14970.3		3.29		24.6	
1	45.9	+21.1%	13275.6	-11.3%	3.67	+11.6%	25.4	+3.3%
2	29.2	-22.9%	13928.3	-6.9%	3.02	-8.2%	22.4	-8.9%
3	24.7	-34.8%	12417.1	-17.1%	2.84	-13.7%	19.2	-21.9%
4	29.9	-21.1%	13273.8	-11.3%	3.48	+5.8%	24.6	0%
5	28.3	-25.3%	13011.5	-13.1%	3.05	-7.3%	20.6	-16.3%
6	33.8	-10.8%	13990.0	-6.5%	3.20	-2.7%	23.6	-4.1%
7	27.2	-28.2%	13206.4	-11.8%	2.95	-10.3%	21.1	-14.2%
8	28.1	-25.9%	13122.8	-12.3%	3.06	-7.0%	22.3	-9.3%
9	28.7	-24.3%	13491.5	-9.9%	3.07	-6.7%	22.9	-6.9%
monitoring Y DOF								
scenario	D_{max} (cm)		V_{max} (kN)		$T_{eff,max}$ (s)		$\xi_{eff,max}$ (%)	
undamaged	43.8		15634.8		3.41		24.9	
1	23.9	-45.4%	14040.0	-10.2%	2.71	-20.5%	18.3	-26.5%
2	33.7	-23.1%	13303.7	-14.9%	3.14	-7.9%	22.9	-8.0%
3	27.0	-38.3%	14115.8	-9.7%	2.89	-15.2%	20.7	-16.9%
4	46.7	+6.6%	13564.0	-13.2%	3.61	+5.9%	25.1	+0.8%
5	26.1	-40.4%	12875.6	-17.6%	2.88	-15.5%	18.5	-25.7%
6	38.5	-12.1%	14631.9	-6.4%	3.33	-2.4%	24.5	-1.6%
7	35.2	-19.6%	13976.1	-10.6%	3.29	-3.5%	23.9	-4.0%
8	34.7	-20.8%	13949.0	-10.8%	3.28	-3.8%	23.8	-4.4%
9	34.6	-21.0%	14164.9	-9.4%	3.22	-5.6%	23.7	-4.8%

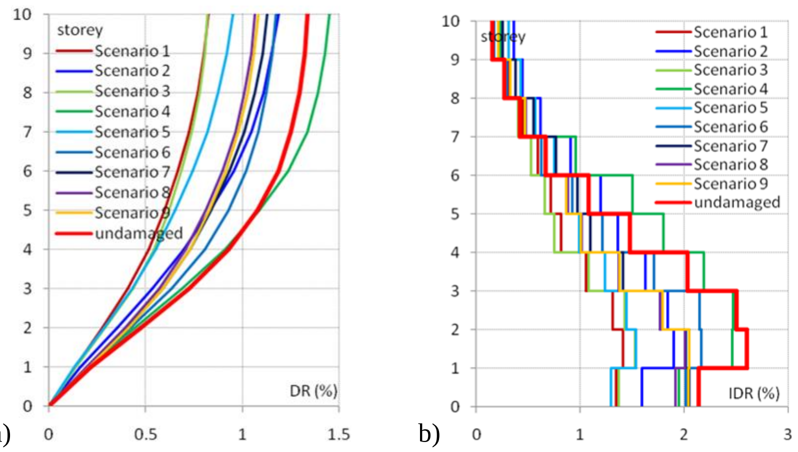
In the third part are considered global *DR* and inter-storey drifts *IDR* for the maximum realized level of displacement (pre-collapse state). In figure 6 are displayed the drifts for the X direction, while in figure 7 are presented the drifts for the Y direction. Considering global drifts *DR* in figure 6a it can be stated that they are higher in case of the first and second scenarios, in relation to the global drifts of an undamaged buildings. All the other global drifts are lower. This clearly indicates that the collapse states develop much earlier, than it is the case in undamaged buildings, except in case of the first and second scenarios where the ductility value is increased, but, in turn, the bearing capacity of the entire building in the non-linear domain is reduced. The inter-storey drifts *IDR*, presented in the figure 6b, also indicated the higher values in the first and the second scenario from the ground floor to the sixth floor. However, all the inter-storey drifts above the sixth floor are almost higher than the inter-storey drifts of the undamaged building. This clearly indicates the fact that in all the scenarios, higher values of inter-storey drifts develop on higher floors. From the ground floor to the sixth floor, the inter-storey drifts are smaller for all the scenarios, except for the first and second scenario, even though in case of the second scenario they also decrease at the ground and first floor levels. The lower values of inter-storey drifts indicate a lower level of vulnerability of infill walls of the building. However, these values of inter-storey drifts should be considered in correlation with the realized global drifts of the building. As already determined, all the global drifts are lower in comparison with the global drifts of the undamaged building, except in case of the first and second scenario, so the higher values of these inter-storey drifts cannot be realized since the early collapse state sets in.



a) b)

Figure 6. Maximum realized displacement level (pre-collapse state) of the 3D model of the frame building for the X direction: a) global drifts *DR*, b) inter-storey drifts *IDR*

By considering the global drifts *DR* in figure 7a it can be concluded that they are higher in case of the fourth scenario, in comparison to the global drifts of the undamaged building. All the other global drifts are lower. In a similar fashion, it can be concluded that it clearly indicates that the collapse states develop much sooner that it is the case for the undamaged buildings, except in the case of the fourth scenario where the value of the ductility is increased, but the bearing capacity of the entire building is greatly reduced in the non-linear domain (the lowest in comparison with all the other scenarios). The inter-storey drifts *IDR*, presented in figure 7b, indicates somewhat higher values in the fourth scenario from the third to the seventh floor. However, all the inter-storey drifts above the seventh floor are almost higher than the inter-storey drifts of the undamaged building. Similarly to the previous statements, the lower values of inter-storey drifts indicate the lower vulnerability of infill walls of the building, but these values of inter-storey drifts should be considered in correlation with the realized global drifts of the building. As already determined, all the global drifts are lower in comparison with the global drifts of the undamaged building, except in the fourth scenario, so that the higher values of these inter-storey drifts cannot be realized, since the early collapse state sets in.



a) b)

Figure 7. Maximum realized displacement level (pre-collapse state) of the 3D model of the frame building for the Y direction: a) global drifts *DR*, b) inter-storey drifts *IDR*

5. Final remarks and conclusions

In order to prevent progressive collapse it is necessary to ensure maximum ductility, redundancy and structural continuity and therefore the capability of a structure to develop alternative load transfer paths in case a vital element fails [4]. A risk in structural engineering is commonly analysed and evaluated by means of quantitative criteria for identified possible hazard scenarios, possibilities of the undesired events and cost of damages. Robustness is an emergent concept related with structural damage tolerance.

In this paper the parameters such as: maximum realized displacement level D_{max}, total shear force of the ground floor for the maximum realized displacement level V_{max}, effective period of vibrations for maximum realized displacement level $T_{eff,max}$ and coefficient of effective damping for maximum realized displacement level $\xi_{eff,max}$, in percents per scenario in relation to undamaged structure are analysed. Global drifts *DR* are higher in case of the first and second scenarios for X direction and in case of the fourth scenario for Y direction, in relation to the global drifts of undamaged building. All the other global drifts are lower. This indicates that the collapse states develop much earlier, than it is the case in undamaged buildings, except in case of the first and second scenarios where the ductility value is increased, but, in turn, the bearing capacity of the entire building in the non-linear domain is reduced. The lower values of inter-storey drifts indicate the lower vulnerability of infill walls of the building, but these values of inter-storey drifts should be considered in correlation with the realized global drifts of the building. As determined the higher values of these inter-storey drifts cannot be realized, since the early collapse state sets in.

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