ROBUSTNESS ANALYSIS OF MULTI-STOREY RC FRAME STRUCTURE FOR CORNER COLUMN LOSS

Radomir Folić [1]
Miloš Čokić [2]

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Summary: *Structural robustness represents the properties of a structural system to resist the progressive collapse i.e. to prevent the chain collapse of the structure, in the case of local failure of vital elements, by establishing the redistribution of effects and by enabling the formation of alternative load paths. Most often, in case of loss of one of the vertical element, the structure is subjected to large deformations, which makes the application of non-linear analysis necessary to study the behaviour of the structure until the formation of mechanisms or progressive collapse.*

Keywords: *Robustness, Progressive Collapse, RC Frame Structure, NSA, NDA*

1. INTRODUCTION

Most frequently, progressive collapse of buildings structures is initiated when one or more vertical bearing element is removed under extreme events (terrorist attacks, vehicle impacts, explosion, etc.). The chain reaction which occurs after a local failure is transferred to adjacent elements and leads to a progressive collapse of most part of the structure, or entire structure. The most extensive review of numerical and experimental research and codes devoted to progressive collapse, with comparative analyses is presented in [1]. Robustness of a building is the characteristic of structures is what constitutes resistance to progressive collapse. The collapse, the Roman Point Tower building (London in 1968) part gave rise to the first regulations and codes in UK, Canada, reviewed in [2]. In the European norms for structural engineering, the provisions about progressive collapse [3] were included for the first time in 2002 [5].
Analysis of the existing buildings is also important, since they often incur damage, especially after seismic events, so in paper [4] performances of damaged buildings were analyzed in several vertical column collapse scenarios.
Majority of papers considering robustness and progressive collapse is devoted to RC frame structures. In document [6], [7] and [10] possibility of Progressive collapse (PC) mitigation was discussed.
If robustness requirements are explicitly considered in design, it should be verified that the structure has a sufficient redundancy and possibilities to mobilise an alternative load

[1] Dr. Ing. Professor Emeritus, University of Novi Sad, Faculty of Technical Sciences, Trg Dositeja Obradovića 6, Novi Sad, Serbia, folic@uns.ac.rs, r.folic@gmail.com

[2] PhD student, University of Novi Sad, Faculty of Technical Sciences, Trg Dositeja Obradovića 6, Novi Sad, Serbia, cokicmilos@gmail.com

path [8], [9] and [13]. In [12] the Building Regulations in England and Wales for monolithic and precast concrete structures were considered. Four main procedures for structural analysis in case of column removal: static and/or dynamic linear, nonlinear static or dynamic, were proposed, the same as in [24].
PC of RC frame structural system of DCH RC building which is the most common type in Balkan region is investigated. Scenario in which the vertical corner columns were completely removed by incidental actions was analyzed, as most critical case [30].
In this paper the results of the behaviour of RC frame structure, after the removal of corner column are presented and analyzed in vertical and horizontal directions, using nonlinear methods

2. MATERIALS AND METHODS

Geometric and material properties of the structure

The subject of the analysis is office-residential building with 8 levels (ground floor+7 stories). The structural system of the building is a frame system [17]. The main structural elements of the analysed structure are RC slabs, beams and columns. The raster of the structure is shown in Fig. 1. The length of one span in the longitudinal (*X*) direction is 5.4 m (6x5.4 m total), and in the transverse direction (*Y*) 5.4 m (4x5.4 m total). The height of the each story is 3.0 m, so the total height of the building is 24.0 m. In order to simplify the modelling and calculation process, all vertical elements are fixed at the bottom level of the structure, i.e. soil-structure interaction is not included in the calculation and design.

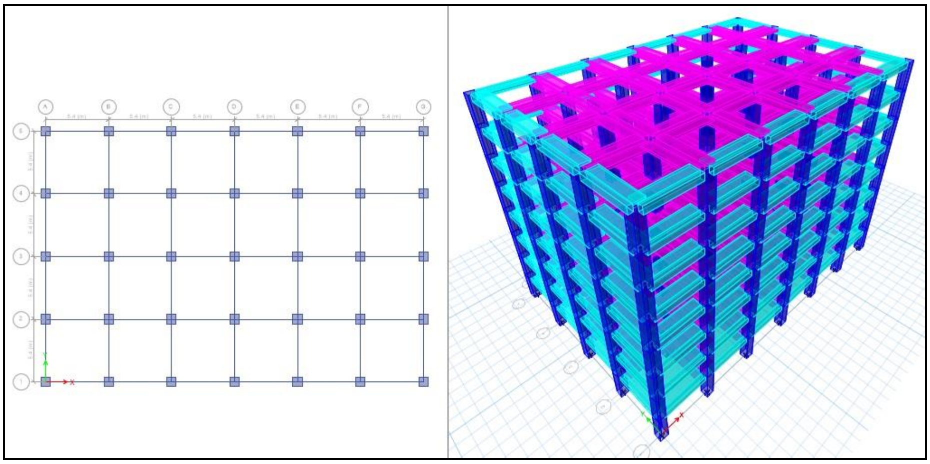

Figure 1. Building analysis model

Material properties of concrete C30/37 [16] and reinforcing steel class C (f_{yk} = 500 MPa, k = 1.25) [16] have been adopted for model analysis. The value of The structure is designed for the medium ductility class (DCM) behaviour [17]. The structural design is done according to the European building design standards [3], [15], [16] and [17], and the calculations are performed using [23]. The structural behaviour is analysed by performing the non-linear static analysis (NSA) and non-linear dynamic analysis (NDA) methods. Geometric characteristics of the cross-section properties of the beams and columns are shown in Table 1.

Table 1. Geometric characteristics of structural elements

Element	**Dimensions** [cm]	**Rebar**
Plate:	16	Ø12/20
Beams: *b/d* [cm]	30/50	3+2 Ø22
Beam effective flange width: $b_{eff,i}$	90/16	Ø12/20
Columns: d_x/d_y [cm]	60/60	12 Ø22

The calculations of the structure are done according to the methodology and recommendations given in [3], [15], [16] and [17]. There are two phases of modelling and calculation process for the building structure analysis. The first phase includes the creation and analysis of the model M0 that is used for linear-elastic analysis of the structure and design of elements. The second phase includes the creation of building model M1 which is used for the robustness and progressive collapse analysis of the structure and comparative analysis of the results.

Loads and actions

The applied loads are as follows: permanent (dead) loads (DL or G_i) – self-weight of structural elements and an additional permanent load; live load (LL or Q_i) and seismic load (S_i). Load combinations and design values of actions for calculations are used according to [3] for horizontal analysis and according to [8]. Load combinations used for the vertical analysis are:

$$\omega \cdot (1.2 \cdot DL + 0.5 \cdot LL) \tag{1}$$

In the case of non-linear static pushdown analysis (NSPDA) and non-linear dynamic pushdown analysis (NDPDA), ω equals 1 for an undamaged mode, while after removal of elements for a corresponding scenario, this value incrementally increases proportionate to the combination of loads (1), until failure in the observed point, according to the function presented in Fig. 2. In case of t_0-t_1 time period analysis (Fig.2) when fast non-linear (modal) analysis (FNA) and NDA are applied, this value is 1 on the undamaged model, whereby after the removal of element for adopted scenario, remains equal to 1 (Fig. 2).

In the case of the vertical analysis, FNA was used exclusively for determining the values of displacement of the corresponding points after the removal of vertical elements (after t_0, Fig. 2), because it does not provide sufficiently reliable results as the dynamic nonlinear direct integration method (nonlinear dynamic analysis – NDA), or as the solving of the dynamic motion equations.

NDA was used for determining the values of displacement of the corresponding points after the removal of vertical elements (after t_0, Fig. 2) and also for pushdown analysis, where ω was incrementally increased proportionate to the combination of loads (1), until failure in the observed point is reached.

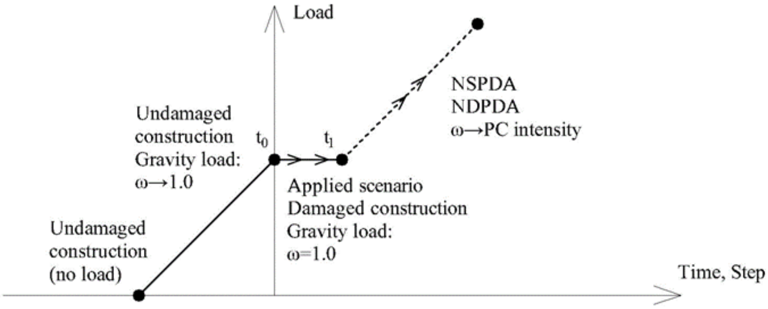

Figure 2. Schematic of load applied in NSPDA and NDPDA, eq. (1)

Combination of loads used for the horizontal analysis of structural behaviour according to [3] is:

$$1.0 \cdot G_i + 0.3 \cdot Q_i + 1.0 \cdot S_i \tag{2}$$

Vertical load

There are two different types of vertical loads on the construction: the weight of the structural elements and the additional permanent load (G_i) and the variable-live load (Q_i). The adopted value of the permanent constant load is g_{pl} = 3.0 kN/m^2 on all floors. The load intensity of the variable-live load amounts to q = 3.0 kN kN/m^2 [15] on all floors, except on the roof slab at which the load intensity is equal to q_r = 1.0 kN/m^2 [15]. The self-weight load of façade elements, which is imposed on all façade beams except the roof façade beams is equal to g_f = 10.0 kN/m. The value of the reduction factor of the live loads is $\psi_{2,i}$ = 0.3 [3].

Horizontal (seismic) actions

To calculate the peak ground acceleration (PGA) action on the structure, an elastic response spectrum, type 1 [17] is used, for ground type B [17], with the reference PGA which amounts to $a_{gR} = 0.30{\cdot}g$. Since the building has an office-residential function, it corresponds to the class of importance II, for which the value of the importance factor is γ_I = 1.0 [17], so calculated value of the PGA is equal to $a_g = \gamma_I{\cdot}a_{gR} = 0.3{\cdot}g$ [17]. The adopted damping value is 5%, after [17]. For more about damping please see [19]. Eccentricity ratios of 5% for both directions are included.

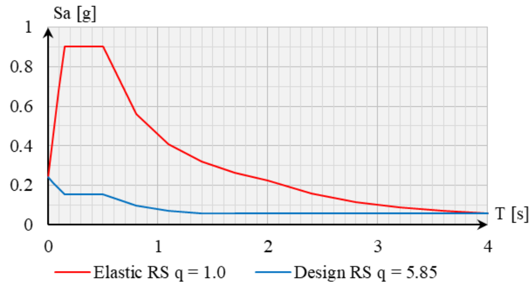

Figure 3. Elastic and design response spectrum

The adopted value of the behaviour factor q is equal to 5.85 [17]: Seismic base shear force for each horizontal direction is calculated with the correction factor value λ = 0.85 [17]. Elastic and design response spectrums are shown in Fig. 3.
In the Fig. 4 the earthquake record of the Imperial Valley earthquake (El Centro, 1940), from the PEER NGA strong motion database record, which was used for FNA and NDA is presented. The presented ground acceleration record is scaled for the adopted design value of $PGA = 0.3g$.

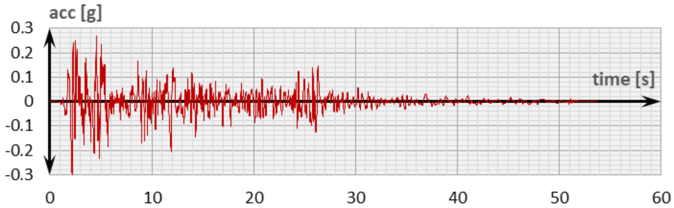

Figure 4. Imperial Valley (El Centro, 1940) ground acceleration records

Adopted properties and simplifications of structural models:
A spatial (3D) model is used for the structure's analysis, which is conducted in [23]. The following parameters, assumptions and simplifications are adopted:
- Second-order (*P-Δ*) effects are included in the calculation;
- Concrete frames rigid factor is 0.5.

In the case of vertical NSA, FNA and NDA, the following parameters, assumptions and simplifications are adopted:
- RC plates are included in the calculation models with corresponding effective widths within beams, i.e., the plates are not treated as surface elements. Robustness of the structure after removing of bearing elements (b. e.) according to the corresponding scenarios is calculated taking into consideration non-linear behaviour of the walls, columns and beams with corresponding effective widths.

In case of horizontal analysis, the following parameters, assumptions and simplifications are adopted: [22]
- Cracked structural elements properties are included in the calculation;
- Elastic flexural stiffness properties of the columns are reduced to 70% and to 35% for the beams;
- Torsion stiffness is calculated as 10% of elastic torsion;
- The shear stiffness of the columns and beams is reduced to 40% of its elastic stiffness.

Linear-elastic analysis model
Linear-elastic structural model was used for the design of structural elements, according to [16], [17].

Non-linear analysis model
In addition to parameters, assumptions and simplifications that are used for all models, for the post-elastic vertical and horizontal analysis models, the following are used as well:
- Material properties for non-linear behaviour of concrete, and reinforcing steel [16];
- Columns and beams are designed as confined RC elements [16] and [18];

- Effective flange widths are calculated according to [16] and [17];

Non-linear hinge properties

Hinge properties are defined according to [20] and [21] in [23]. In the beams, it is anticipated that plastic hinges will emerge because of the effects of bending moments about the horizontal local axis of an element. In the case plastic hinges form in the columns, they will be affected by the axial force and bending moments about both local axes of the element. Column and beam non-linear hinges are modelled at the 5% element length distance from the nodes.

Adopted scenarios for robustness and progressive collapse analysis

For the analysis of robustness and progressive collapse of structure, scenario with the removed corner column in the ground floor was adopted. M0 corresponds to the condition in which the structure is undamaged, while the model M1 corresponds to the condition of the damaged structure. Fig. 6 show the scenario used in the analysis.

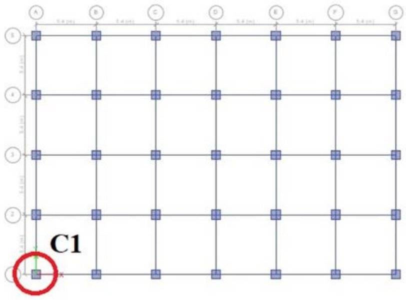

Figure 6. Damage scenario M1 – corner column is removed in the ground floor (base view)

Models for vertical NSA, FNA and NDA

In the analysis models, surface load acting on the plates is reduced to linear uniform load acting on the beams, taking into consideration the active surface of the plates, in accordance with the boundary support conditions. Plates are included in the calculation models with corresponding effective widths of the beams, i.e., plates are not treated as surface elements. By this, certain bearing capacity of the plates which does not coincide with the effective beam widths is ignored. The consequence of this simplification is that the results may indicate lower system robustness than the actual one, but the calculation favours safety. More details about four methods for pushdown analysis (linear static, linear dynamic, nonlinear static and nonlinear dynamic) can be found in [24].

3. RESULTS AND DISCUSSION

Vertical analysis

In the case of the analysis of the column removal scenario for t_0-t_1 time period (Fig.2), vertical displacements of characteristic points are presented for the case with the moment when the column is removed, with the time increment $\Delta t = 0.01s$. (Fig. 7) The results

presented in Fig 7. Show that there is a difference between the results obtained using FNA and NDA. NDA results show bigger displacement values than FNA (approximately 3 cm).
As already mentioned, FNA is not implemented for pushdown analysis because it does not offer sufficiently reliable results, which could be obtained using NDA.

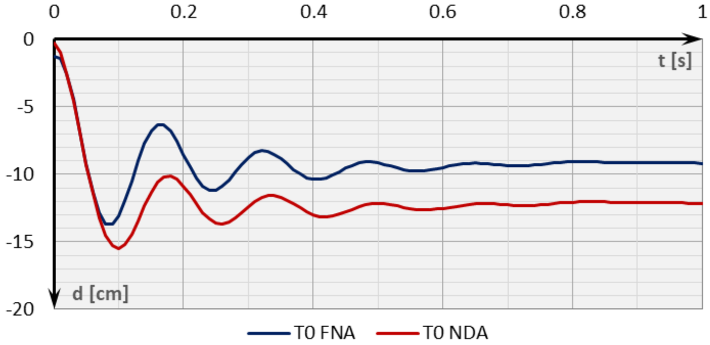

Figure 7. Vertical deformations after removal of vertical elements $(\omega = 1)$

NSA and NDA is performed according to the described procedures. Two approaches were taken:

- On a damaged structure loaded according to (1) for $\omega = 1$, the set load is being incrementally increased for the value $\omega > 1$. This is pushdown analysis method with the use of uniform (UNI) load distribution. (Fig. 11)
- On a damaged structure loaded according to (1) for $\omega = 1$ only reactive load of the removed elements is applied, but acting in the opposite direction. The set load is being incrementally increased for the value $\omega > 1$. This is pushdown analysis method with the inverse proportional point load (IPPL) distribution. (Fig. 12)

Both of the mentioned pushdown methods are described in detail in [25], [26] and more papers in this research area.

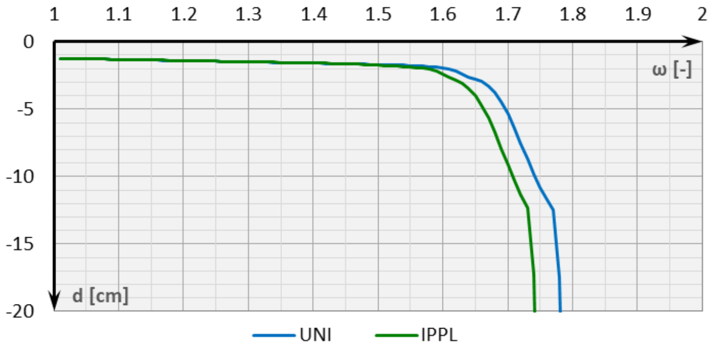

Figure 8. NDA pushdown curves $(\omega - t)$

In this way, obtained pushdown curves, provide the view of the relation of gravitational force intensity – vertical deformation (NSA) and vertical deformation – time (NDA) in characteristic points. However, NDA pushdown curves can be converted into gravitational force intensity – vertical deformation $(\omega - d)$ curves because load – time function for NDPDA is familiar. (Fig. 8)

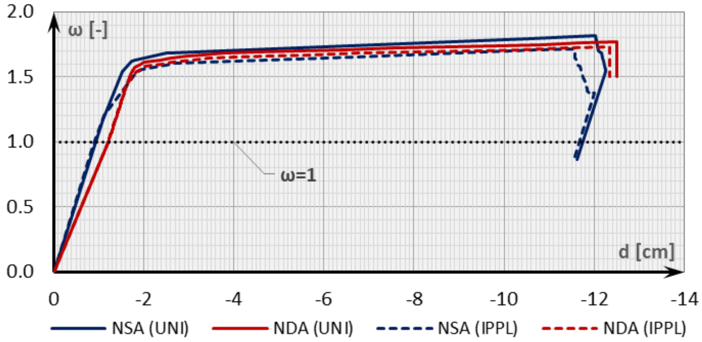

Figure 9.. NSA and NDA pushdown curves $(\omega - d)$

Point at the top of the removed corner column (C1) was chosen for the analysis, because the highest deformation and first onset of collapse is expected there. The obtained values are presented in Fig. 9. ω represents the incremental load factor according to (1).
Fig. 9 shows pushdown curves for mentioned cases (UNI and IPPL), where the existing gravity load on the damaged structure was incrementally increased until collapse (UNI) and the case where the reactive forces of removed elements were incrementally applied on the damaged system, with the opposite direction of action (IPPL). Ultimate displacement point for NDA is determined at the points where NDPDA curves (Fig 8.) have significant tangent value increase between two referent points.
Pushdown curves obtained by NDA show higher displacement capacity in the referent point (C1), than the curves obtained by NSA. Difference between the displacements at ultimate force capacity points are 4.0% for UNI and 6.9% for IPPL load distribution (Fig. 9). Also, for both methods (NSA and NDA) the displacement values at the ultimate force capacity points are higher for UNI than IPPL load distribution, 4.19% for NSA and 1.36% for NDA method.
For both analysis methods (NSA and NDA) and both load distributions (UNI and IPPL), maximum vertical displacement values before the collapse are in the range of 11.5-12.5 cm and maximum value of ω is in the range of 1.72-1.82.

Horizontal analysis
NSA
A NSPA is performed for both main (X and Y) directions. Mass proportional load distribution pattern (PROP) was used for the analysis. The results of NSPA for both directions are shown in Fig. 10.

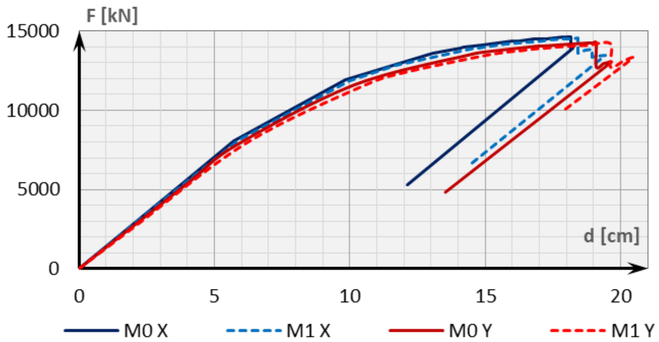

Fig. 10. Pushover curves for different scenario in X direction

Fig. 10 shows that, as expected, M0 will have a bit higher shear force capacity (0.58% in X and 0.64% in Y direction), and lower displacement capacity than M1 (1.38% in X and 2.72% in Y direction). Also, as expected, structural system has higher shear and lower displacement capacity in longitudinal (X) direction, compared to transverse (Y) direction.

NDA

Fig. 11 show the difference between the values of horizontal displacements of the top of the structure Δu_x and Δu_y in both horizontal directions for the selected scenario, in comparison to the scenario M0, which corresponds to an undamaged structural system. The values in Fig. 11 were obtained using the expression:

$$\Delta u_x = u_{x,1} - u_{x,0} \quad ; \quad \Delta u_y = u_{y,1} - u_{y,0} \tag{3}$$

where $u_{x,1}$ and $u_{y,1}$ are displacements of the top of the structure for a corresponding scenario, and $u_{x,0}$ and $u_{y,0}$ are displacements of the top of an undamaged structure (M0), for the used ground acceleration records (Fig. 4).

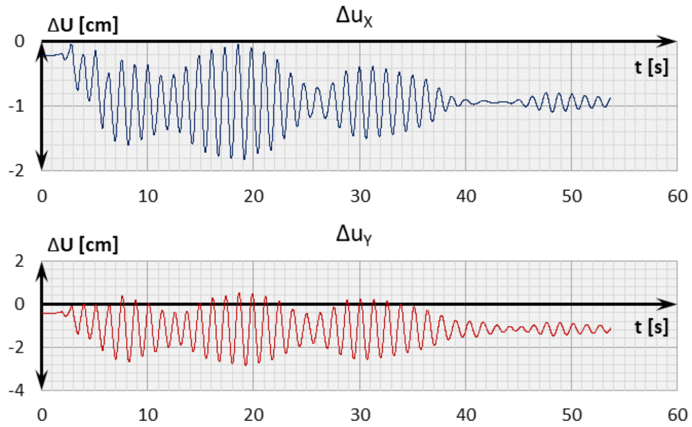

Fig. 11. *Time-displacement functions in X (left) and Y (right) direction for undamaged (M0) structure*

The highest displacement values for M0 are 12.24 cm in X and 12.73 cm in Y direction. The highest displacement values for M1 are 13.02 cm in X and 13.83 cm in Y direction Biggest difference between the horizontal displacements for the selected ground acceleration record (Fig. 4) is $\Delta u_{x,max} = 1.82$ cm for X and $\Delta u_{y,max} = 2.84$ cm for Y direction.

4. CONCLUSIONS

In this paper, NSA, NDA and FNA methods are applied in the analysis of the behaviour of DCH RC frame structure, after the removal of corner column. The behaviour of the building is analysed for vertical and horizontal direction.
It can be concluded that, from the aspect of both vertical and horizontal behaviour of the system, building will not be exposed to the risk of global or local progressive collapse in case of predicted scenario. The reason for this conclusion is the fact that the structure will keep enough residual capacity to carry 1.72-1.82 times bigger load than the analysis load combination (1), before it reaches the state of local progressive collapse. Also, results of the horizontal analysis show that the structural behaviour of the building will not have significant differences comparing to undamaged structure (max. 2.72%).
Results of NSA and NDA show that the structure is more resilient in longitudinal direction, then transverse, which is expected. Based on the results obtained, it can be concluded that for the adopted scenario of corner column removal, structural system resilience is more endangered in vertical, than it is in horizontal direction.

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АНАЛИЗА РОБУСНОСТИ ВИШЕСПРАТНЕ АБ ЗГРАДЕ У СЛУЧАЈУ ГУБИТКА УГАОНОГ СТУБА

Резиме: *Робусност представља својство конструктивног система да се одупре прогресивном колапсу, тј. да спречи ланчани лом конструкције у случају локалног губитка виталних елемената, успостављањем прерасподеле утицаја и омогућавањем формирања алтернативних путева оптерећења. Најчешће, у случају губитка једног од вертикалних елемената, конструкције је подвргнута великим деформацијама, због чега је примена нелинеарне анализе неопходна за проучавање понашања конструкције до формирања механизма или прогресивног лома.*

Кључне речи: *Робусност, прогресивни лом, АБ оквирна конструкција, НСА, НДА*