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## INFLUENCE OF SUDDEN COLUMN LOSS ON THE DYNAMIC RESPONSE OF A MULTISTOREY STEEL FRAME

Multistorey steel buildings are proved to be very susceptible to situations when one of their columns loses its capacity as a result of an accidental action. The above mentioned case concerning a steel framed building is the subject of investigation presented in the paper. Structural system of analyzed building was designed in accordance with ultimate and serviceability limit states in the persistent and transient design situations. Then its integrity in accidental design situation was assessed. According to EN 1991-1-7 [1], the strategy based on limiting the extent of localized failure was assumed. Firstly, the static analysis of the structure in Autodesk Robot Structural Analysis Professional software was performed. Then, the static and dynamic GMNA analyzes (materially and geometrically nonlinear) of the structure in Autodesk Simulation Mechanical were carried out. Calculations were made in reference to plane frame, which is the repeatable load bearing system of considered building. FEM models were made with the use of beam and shell elements. The results of performed analyzes were compared and discussed. Concluding remarks were drawn and directions of future research were outlined.

**Keywords:** robustness, dynamic analysis, accidental action, column loss, steel framed building, FEM

### 1. Introduction

In the case of identified accidental actions classical methods of structure analysis may be used, however, taking into account highly dynamic nature of the problem. In accordance with EN 1990 [2], these actions are considered only in accidental design situation.

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If actions impossible to determine are considered, e.g. terrorist attacks or random loss of bearing elements capacity, standard EN 1991-1-7 [1] recommends using strategies based on limiting the extent of localized failure. In practice, the structure subjected to accidental actions has to be calculated in two stages:

- 1) Structure dynamic response on an action with the impulse nature (e.g. explosion, impact) to be determined. In this case accidental loads are carried by the whole structural system including appropriate dynamic properties.
- 2) Structural robustness to be modelled in the case when one of load bearing elements could lose its capacity in stage 1.

This paper focused on the second stage of calculations. The case of sudden column loss on the lowest storey of steel framed building was considered. The strategy based on limiting the extent of localized failure was assumed [1]. Firstly, the static analysis of the structure in Autodesk Robot Structural Analysis Professional software was performed to achieve suitable cross-sections of frames members. Then, the static and dynamic GMNA analyzes (materially and geometrically nonlinear) of the structure in Autodesk Simulation Mechanical were carried out. Calculations were made in reference to plane frame, which is the repeatable load bearing system of a building analyzed in [3] and [4], according to the static approach.

## 2. Analysis methods

Safety assessment of steel skeletal structures with reference to codification of design rules was widely described in [5], [6]. Possible, but very simplified method to analyze the structural robustness of a building is the static analysis. According to [7], in this approach calculations could be divided into three study cases:

- analysis of degraded structure due to the loss of the internal column on the lowest storey,
- design of the key element regarding the recommended value of accidental load  $A_d$ ,
- redundancy of ties to withstand the loss of any column on the lowest storey.

The static approach can be used in analysis of buildings in consequences class CC2 [5]. However it doesn't include the entire essential issue connected with the nature of analyzed phenomenon - the dynamic effect, which has to be taken into account during designing buildings in consequences class CC3. The loss of the capacity of element often happens in rapid way, which entails a sudden need to find the alternate paths of balance in structure. In this kind of situations the inertia of structure can have significant influence on forces distribution in load bearing components. Therefore the dynamic assessment of structural robustness on progressive collapse, widely applied and described, e.g. in [8] – [14] is more accurate approach for the mentioned problem.

### 3. Estimation of robustness on progressive collapse based on static analysis of frame

The single, repeatable steel frame with rigid joints (Fig. 1) representing the part of residential building (Fig. 2) was analyzed. According to Table A.1 in [1] this building is designated in the consequences class 2b (higher risk) and was widely analyzed in [4]: concerning design of the key element regarding the recommended value of accidental load  $A_d$  and analysis of degraded structure due to the loss of any column on the lowest storey. Naturally, including the connections between adjacent frames has substantial influence on obtained results – for example the transverse elements with length corresponding to spacing between each main frames and displacement boundary conditions could be added in model to take into account the catenary action associated with significant second order effects that plays an important role in resisting additional loads when structural column is destroyed under unexpected loads. Nevertheless, the simplification as an analysis of plane frame with connections between adjacent frames modeled as lateral restraints was assumed in this article.

Calculations of sway frame according to ultimate limit state in persistent design situation including appropriate imperfections resulted in members cross-sections presented in Fig. 1 (*Initial Frame*). Then the structure was analyzed including various scenarios of its degradation, which are showed in Figs 3 a–e [4]. Calculations were performed in Autodesk Robot Structural Analysis Professional software [15] with use of beam elements.

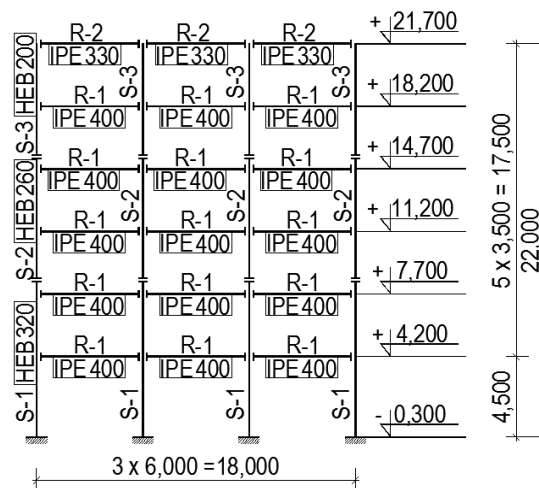


Fig. 1. Members of repeatable load bearing system resulting from ULS (*Initial Frame*)

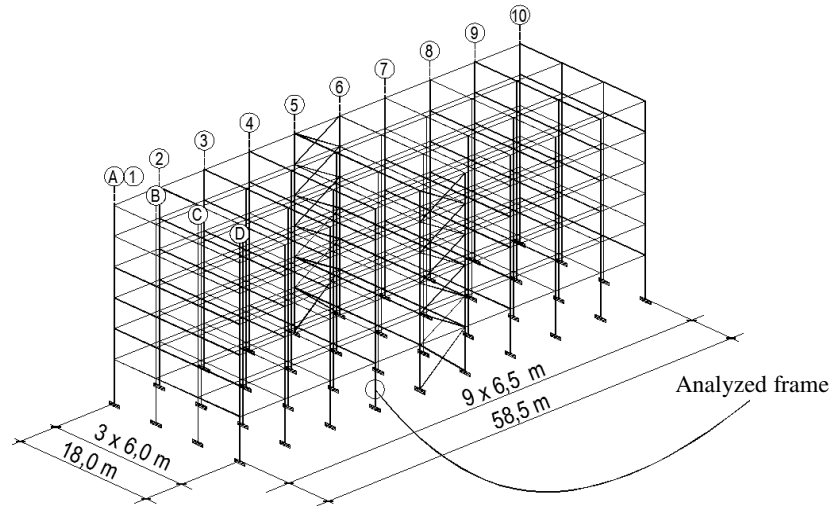


Fig. 2. Residential building taken into consideration in accidental design situation

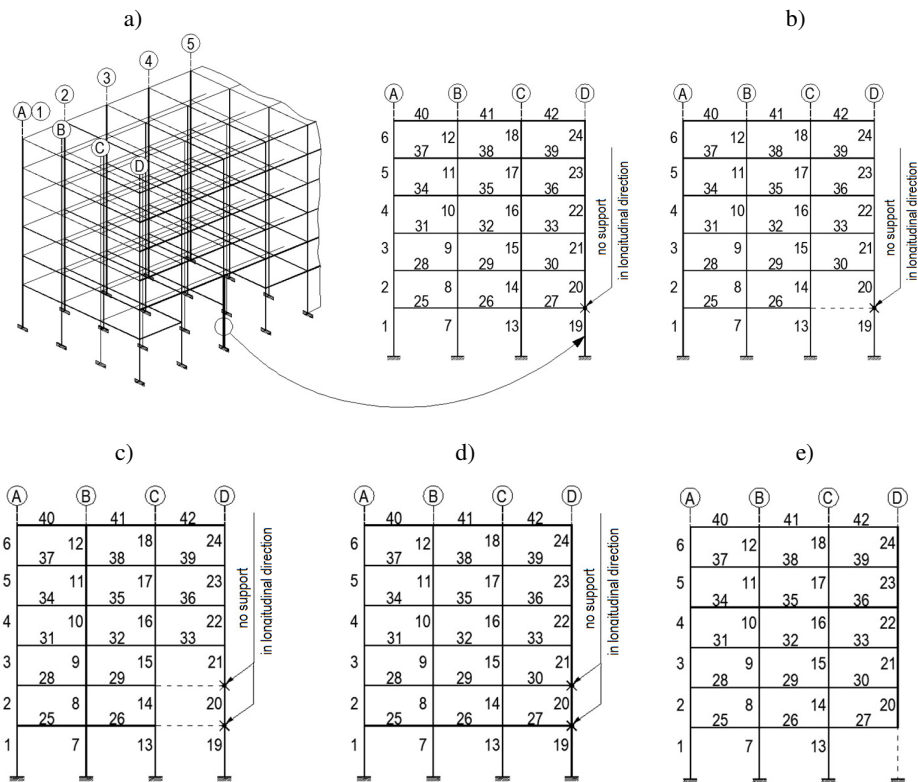


Fig. 3. Various scenarios of analyzed frame degradation due to external blast [4]

Static analyzes of degraded frame in accidental design situation resulted in significant increase of members cross-sections (*Upgraded frame*). Selected sections are shown in Table 1.

Table 1. Sections fulfilled Ultimate Limit States resulting from frame static analyzes

Frame	Main structure elements				
	column S1	column S2	column S3	girder R-1	girder R-2
Initial	HEB 320	HEB 260	HEB 200	IPE 400	IPE 330
Upgraded	HEB 500	HEB 280	HEB 200	IPE 750x173	IPE 330

Finally case e) (removing the side column on the lowest storey) proved to be most disadvantageous and was assumed to further analysis including dynamics effects.

## 4. Estimation of robustness on progressive collapse based on frame dynamic analysis

### 4.1. Initial Frame analysis

Structural system of analyzed building designed in accordance with ultimate and serviceability limit states in the persistent and transient design situations was taken under consideration. Geometrically and materially nonlinear analysis (GMNA) was conducted in Simulation Mechanical software, featuring integrated Autodesk Nastran FEA solver [16].

FE model of analyzed frame was made with the use of rectangular shell elements. Analysis of solution convergence including influence of discretization was carried out. The mesh size had been gradually decreased until it reached 40 mm, for which satisfactory solution convergence at acceptable calculation time was obtained. Finally the entire model consist of about 94 thousands of finite elements. However, in this case sufficient accuracy of dynamic analysis can be obtained using beam elements [8], [17], [18], shell elements were used in this study to more precise capture the potential plastic zones in most critical points of structure (Fig. 4).

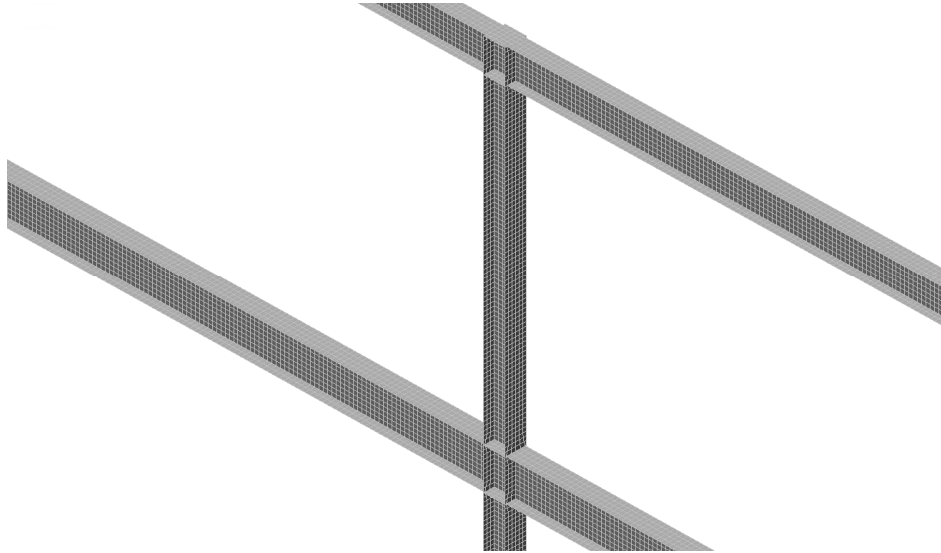


Fig. 4. View of the part of the FE model of *Initial Frame*

Dead load was modelled as lumped mass applied to top flanges of girders while live load as external load applied to top flanges of girders on side nave of frame (Fig. 5). Lateral restraints of girders' top flanges due to presence of floor slabs were assumed. To simplify numerical calculations, a side sway of the frame due to wind action was not taken into consideration. Furthermore, neither global nor local imperfections were included in analysis.

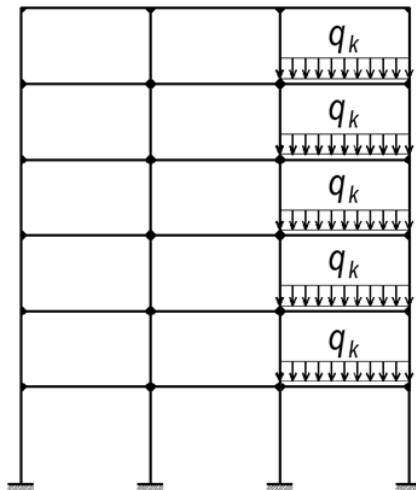


Fig. 5. Live load case included in frame dynamic analysis

Modal analyzes of complete frame and without a side column (degraded) were performed to determine natural frequencies and mode shapes. The analyzes were performed twice – including and not including mass of ceilings lying on girders. Obtained frequencies are shown in Table 2. Values of frequencies were used to determine suitable time step for dynamic analysis.

Table 2. Natural frequencies of the *Initial Frame*

Natural frequencies of analyzed structure [Hz]						
Mass of ceilings	included			not included		
Mode	1	2	3	1	2	3
Complete frame	0.51	1.45	2.53	3.09	8.43	14.31
Degraded frame	0.43	1.38	2.04	2.60	7.69	9.75

Based on results obtained from modal analyzes, the time step was set as 0,05 s [19]. Trial dynamic analysis revealed that further reducing of time steps had negligible influence on results. Implicit integration method available in Autodesk Simulation Mechanical was used in dynamic analysis.

The characteristic of the elastic-plastic material model with isotropic hardening [20] used in analysis is shown in Fig. 6 [21]. The following material parameters were assumed:

$$\begin{aligned} \sigma_y &= 235 \text{ MPa,} \\ \sigma_u &= 360 \text{ MPa,} \\ \varepsilon_{st} &= 0.02, \\ \varepsilon_b &= 0.04, \\ \varepsilon_u &= 0.30. \end{aligned}$$

Damping of the structure was defined with use of Rayleigh’s method by setting the mass-proportional damping coefficient  $\eta = 2.0$  and stiffness-proportional damping coefficient  $\delta = 4.0$ . This values were assumed based on probationary analysis, to achieve noticeable damping [22]. Correlation between this coefficients was obtained based on formula (1) for two first natural frequencies  $\omega_1, \omega_2$ .

$$\begin{bmatrix} \xi_i \\ \xi_j \end{bmatrix} = \frac{1}{2} \begin{bmatrix} \frac{1}{\omega_i} & \omega_i \\ \frac{1}{\omega_j} & \omega_j \end{bmatrix} \cdot \begin{bmatrix} \eta \\ \delta \end{bmatrix} \tag{1}$$

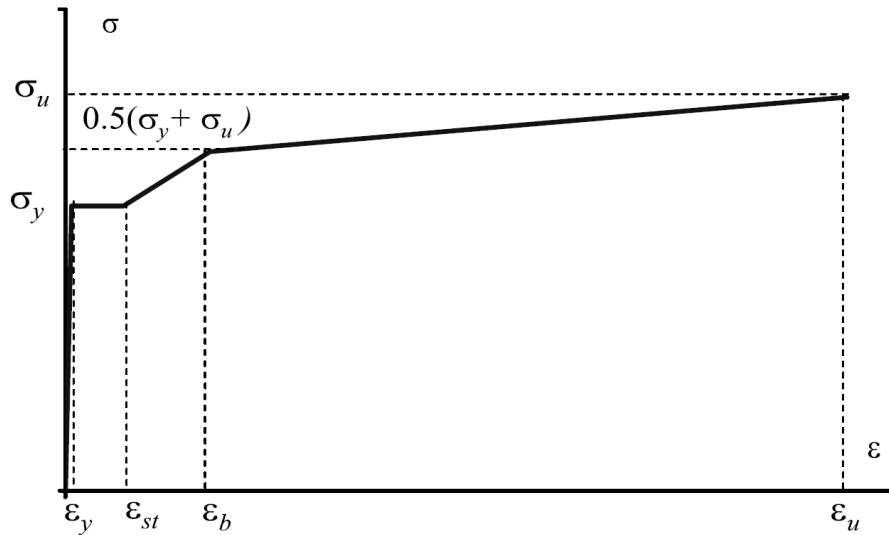


Fig. 6. Simplified stress-strain curve (in uniaxial tensile test) used in analysis, based on [21]

At the beginning of the analysis the frame was not degraded, so column was taking over all forces intended to it. Loads were applied in quasi-static way – dead load through first 10 000 seconds, live load through next 5 000 seconds and finally through another 5 000 seconds no additional load was applied, which was intended to stabilize the forces in structure (Fig. 7). In this part of analysis the time step was set equal 1 000 seconds.

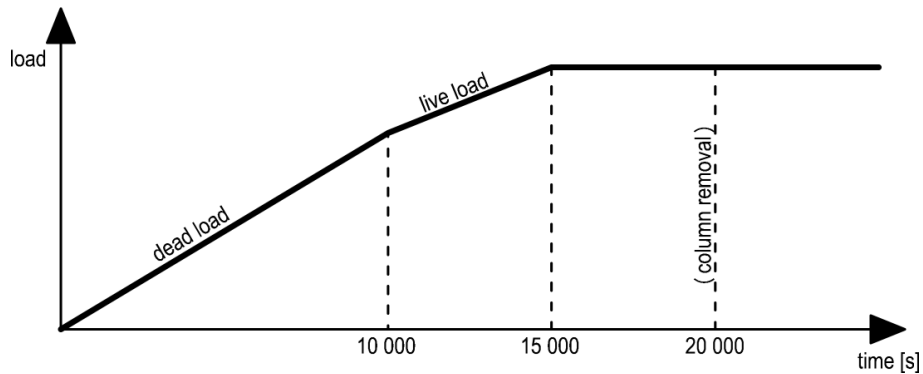


Fig. 7. Load multiplication factor curve in dynamic analysis

After described time the whole supporting constraints in side bottom column were removed rapidly during one time step and behavior of such degraded structure was analyzed. In this part of analysis time step was set as 0.05 s. Supporting zones of side girders started working above plastic limit after

about 0.15 s (Fig. 8) as well as vertical displacement of side columns was growing very rapidly with average speed about 2 m/s. Analysis was stopped when the equivalent strain in mentioned zones exceeded 30% which corresponds to average relative elongation of steel samples during fracture (Fig. 9). At that moment of calculations, maximum vertical displacements amounted 1 105 mm and didn't stabilized, which means that the side nave of analyzed frame is going to collapse (Fig. 10–11).

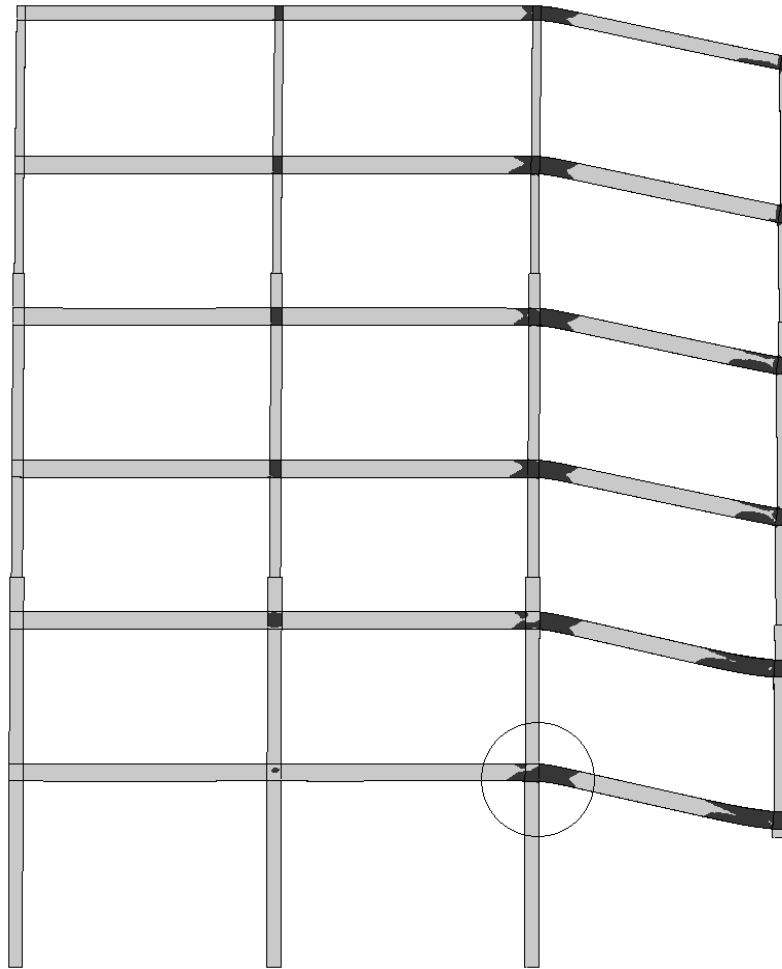


Fig. 8. Plastic zones (dark areas) in degraded *Initial Frame*

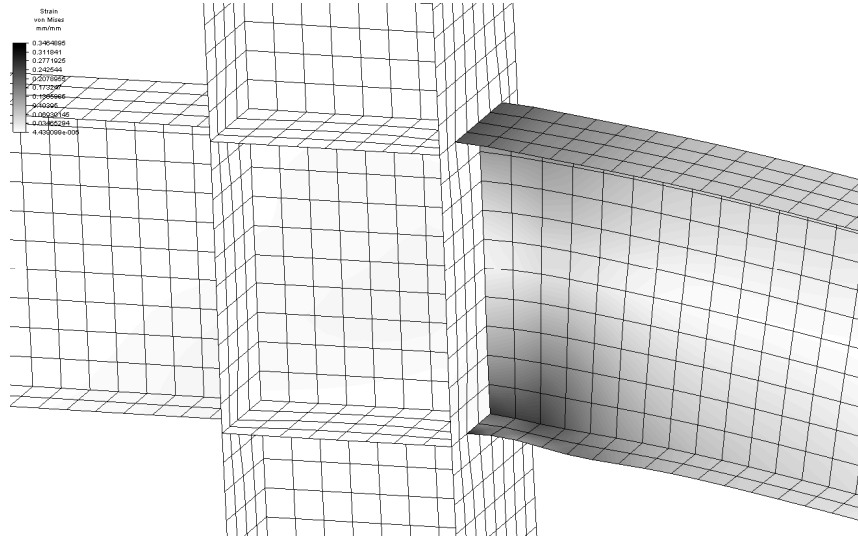


Fig. 9. Equivalent strain map around joint marked by a circle in Fig. 8

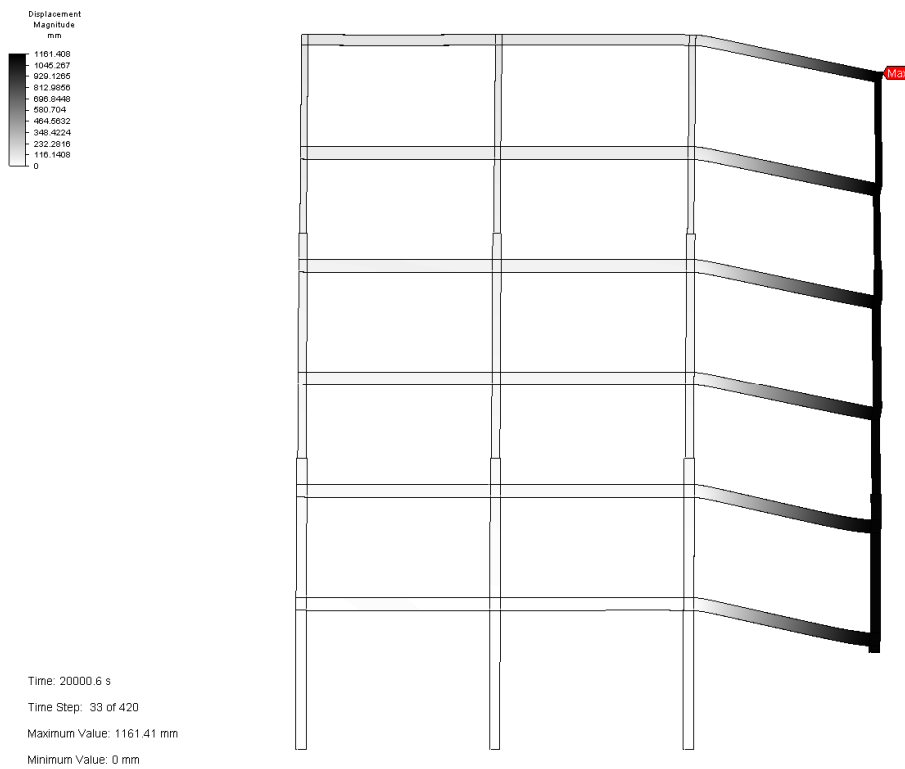


Fig. 10. Vertical displacement map in degraded *Initial Frame*

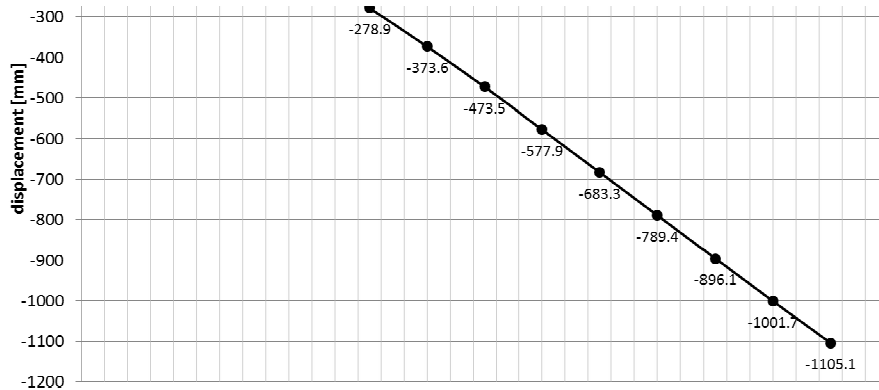


Fig. 11. Vertical displacement versus time of the top where the column was removed derived from dynamic analysis of Initial Frame

#### 4.2. Upgraded Frame analysis

In this stage, structural system of analyzed bridge designed in accordance with ultimate limit state in accidental design situation was taken under consideration.

Analysis assumption remained the same as Initial Frame. Natural frequencies calculated for upgraded Frame are presented in Table 3.

Table 3. Natural frequencies of upgraded Frame

Natural frequencies of analyzed structure [Hz]						
Mass of ceilings	included			not included		
Mode	1	2	3	1	2	3
Complete frame	0.88	2.18	3.45	4.24	10.59	13.55
Degraded frame	0.76	2.12	3.33	3.65	10.15	13.55

Based on results obtained from modal analysis, the step for dynamic analysis was set as 0.05 s, the same as Initial Frame.

Analysis revealed that maximum vertical displacement occurred 0.5 second after column's removal. Inertia forces increased in comparison to static analysis, but induced only temporary plasticization, not only in connection between column S-1 (HEB 500) and column S-2 (HEB 500) on axis C (Fig. 12). Finally stresses stabilized on level about 205 MPa (Fig. 13).

Fig. 12. H-M-H stress map in upgraded Frame 4 (0.5 s after column removal)

The maximum equivalent stress at the connection of bottom girder with side column was occurred a bit earlier – about 0.2 s after column's removal and reached value about 225 MPa and finally stabilized about 160 MPa, which is significantly less value in comparison to 205.5 MPa received from static analysis (Fig. 14). It proves that dynamic effects led to a different way of forces distribution. Finally structure did not lose its stability, so it was resistant to assumed accidental action (side column removal).

Selected results of analysis are shown in Fig. 64–1

Fig. 13. H-M-H stress versus time of the point in connection between column S1 and column S2  
in *Upgraded Frame*

Fig. 14. H-M-H stress versus time of the point in girder-to-column joint in *Upgraded Frame*

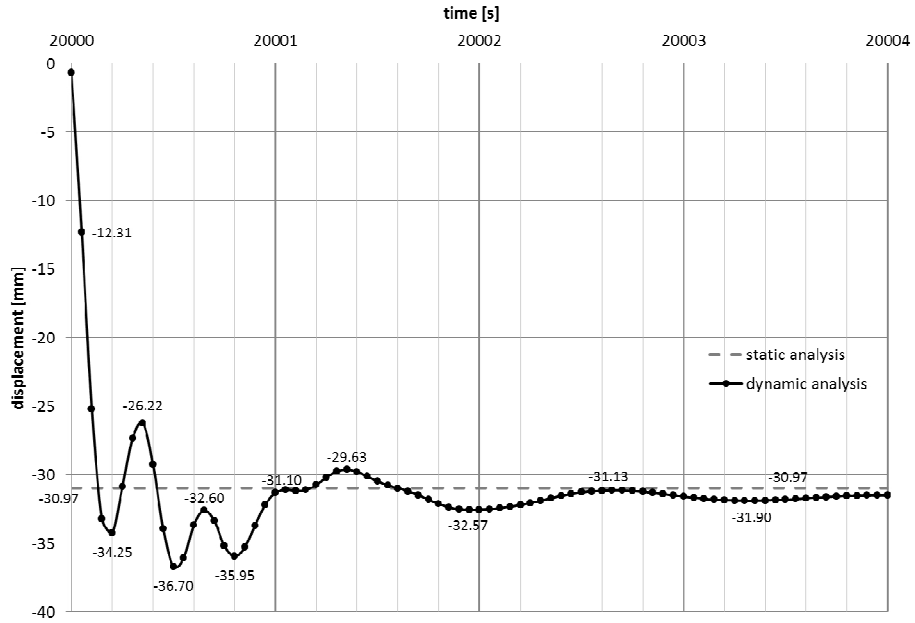


Fig. 15. Vertical displacement versus time of the point where the column was removed in *Upgraded Frame*

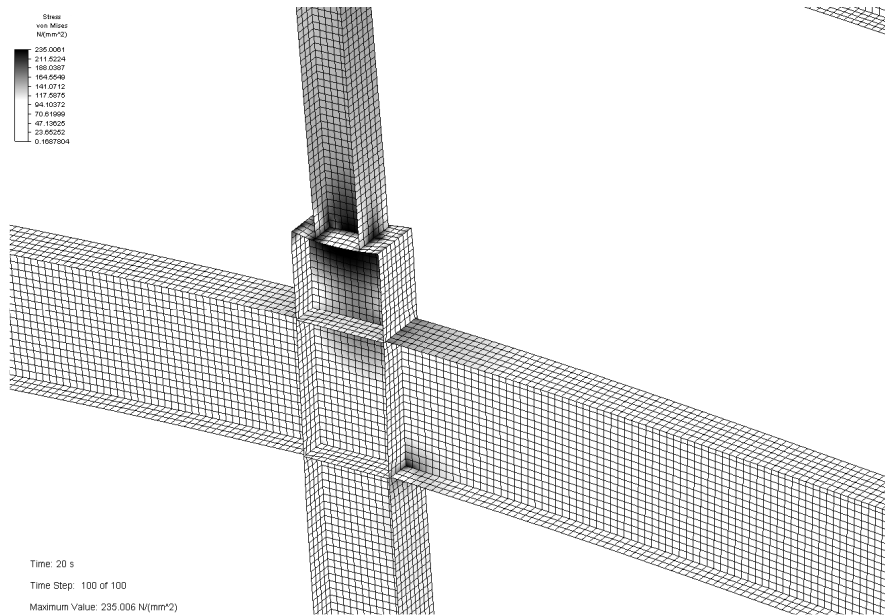


Fig. 16. H-M-H stress maps around column-to-column joint derived from static analysis of *Upgraded Frame*

## 5. Summary

Performed calculations presented in the paper revealed that sections of degraded structure's elements determined in static analysis (*Upgraded Frame*) using beam elements are sufficient to take over the designed load even with including dynamics effects of phenomenon. Simultaneously, frame with initial elements (*Initial Frame*) partially collapses in the case of side column loss. In comparison to static analysis a few percent increase of stresses and displacements was observed in dynamic analysis. In the case of plane frame with members of class 1 – 3 cross-section sufficient accuracy of calculations can be obtained using beam elements [8], [17], [18]. Despite of that, shell elements were used to more precise capture the potential plastic zones. Due to the necessity of the use of enormous sections, both for bottom columns as well as for girders, the possibility of the use of additional bracings to ensure integrity of structure subjected to accidental actions should be considered. Future research should take into consideration a spacious behavior of a structure instead of plane frame analysis. Furthermore, future investigations should also account for an influence of high temperature and distortion speed on material properties to improve accuracy of analysis results, as well as stiffness of steel joints (use of semi-rigid joints) and initial side sways due to imperfections and wind action.

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