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Dynamic response analysis of multi-storey building to a non-uniform excitation

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Abstract. In this paper the dynamic behaviour of multi-storey steel building is presented. The structure was subjected to a real, strong mining shock. During the analysis the time history analysis and the spectrum method were taken into account. The time history analysis concerns the uniform and non-uniform model of ground motion. A wave velocity of 300m/s was taken under consideration. The kinematic excitation was applied on the structure as support accelerations. The stress at selected points of structure were compared for all methods. The region nearby the connections was analysed particularly. The state of stress for connection zones was recognized.

1 Introduction

The mining shock was the typical dynamic load in the area of mining activity. The mining tremors may occurs very often and may be characterized by big energy. There are two main activity areas in Poland: Upper Silesian Coal Basin and Legnica-Głogow Copper Belt. In many cases the mining activity induce damage in buildings. The dynamic response analysis for buildings located in mining region are needed. The analysis allows to estimate the stress and displacements of structural elements. It is the method which helps to protect building against damage. Another issue is the wave passage effect. The non-uniform excitation indicated different object response than the uniform excitation. Some building shows vulnerability for wave propagation [1-4]. The time history analysis and response spectrum analysis are used to assessment the stress [5].

In this paper the multi-storey building was analysed. The building was subjected to uniform and non-uniform excitation. The dynamic response spectrum was calculated. The obtained results were compared with results from response spectrum method. The analysis allows to estimate the stress in elements during mining shock excitation. The analysis was conducted in Abaqus software [6].

2 Description of analysed construction

The five storey building was chosen to the dynamic analysis. The building was the steel structure on the plan of the rectangular. The building consist of five frames spaced in equal

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distance of 6 m. Total length of building was 24 m and the width 12 m. The total height of building was 17.8 m. The height of storey was not the same for each tier. The height of first storey was 4 m, the top storey 3.3 m and the rest of storeys 3.5 m. The main frames were created as a double span frames. The length of span equals 6 m. The elements of frame were made of typical section- double T sections. The external columns were made of HEB260; interior columns of HEB320 (it means the height of sections were 26 cm and 32 cm respectively). The lower beams (beams from 1st to 3rd floor) were made of IPE400 sections (40 cm height) and the rest of beams were made of IPE360 (height of 36 cm).

The beams and columns were joined together by rigid connections. That kind of connection allows to resist a bending moment.

The building was designed with a floor system consisting of a metal deck with a 15 cm lightweight concrete slab. It should be noted that the slabs were the same on each storey. Whole construction was supported by foundation slab.

The dimensions of analysed structure is presented in Fig. 1. The type of applied sections were also marked.

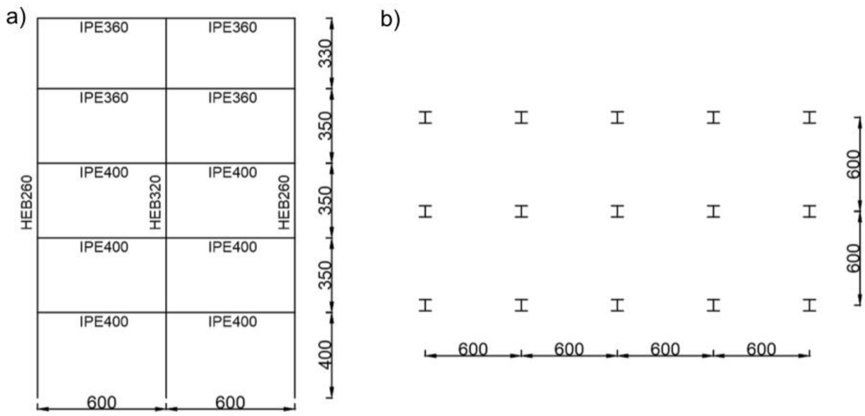

Fig. 1. Main dimension of analysed building: a) main frame, b) plan.

3 Numerical model

To carry out the dynamic analysis of structure the numerical model was created. The model represents whole structure. The building was presented as a multi storey frame. In the 3D model main dimensions like height, span length, spacing of frame and the real shape of cross sections were taken into account. In case of complex shape of cross section and numerous elements connections, in the numerical model some simplifications were applied. The model is shown in Fig. 2.

The main frames, except one of them, were modelled using beam elements (see Fig. 2a). Decreasing numbers of finite elements in whole model was possible by using beam elements. That allow accelerate the calculation without deterioration the quality of results. As mentioned, one of the frame was modelled by different than beams elements. The frame in the middle of building was represented by shell elements. The different kind of elements allow to calculate the stress distribution and deformation of middle frame. The cross section of girders and columns were made of plates which form a double T section (see Fig.3). The shell thickness- correspond to a real dimensions of cross section. The middle frame is presented in Fig. 2b.

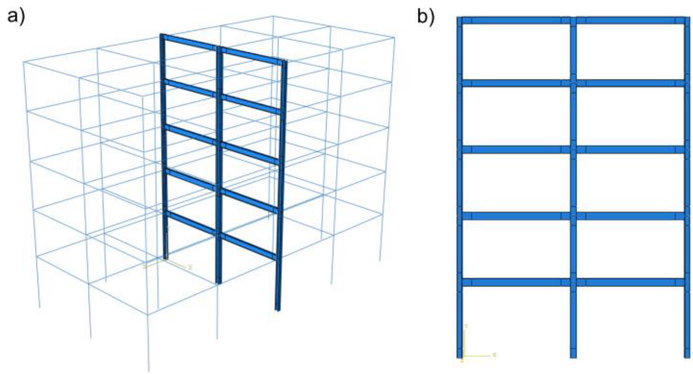

Fig. 2. Numerical model of building: a) isometric view b) main frame- front view.

In the numerical model the rigid, moment resistant joints were assumed. The connections were modelled as a welded connection. The exemplary connection between columns and beams is presented in Fig. 3. The elements were tied (there was compatibility of element displacements between column sides and beams ends). The rest of structural elements (modelled by beams) were coupled to the middle frame. The translations and also rotations were transmitted from beams to shells.

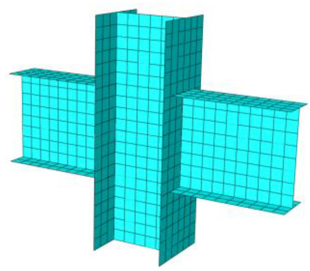

Fig. 3. Details of the frame structural node.

To represent the actual construction behaviour, the mass of non-structural elements were taken into account. The mass of the secondary elements were added to structure as inertiae masses. The masses were evenly distributed along each beams. The mass of concrete slab and 40% of live load were included into inertia.

The numerical model was supported at the end of columns. The fixed (each displacements equals 0) boundary conditions were used. The numerical model was created with Abaqus software [6]. The mesh consisted of 4092 beam elements (B31- 3D beam with 1st order interpolation) and 36504 shell elements (S4R - 4 nodes element with reduced integration). Used elements are available and describe in Abaqus element documentation. The size of elements were taken from convergence analysis.

4 Data of kinematic excitation

The dynamic response analysis of multi-storey building was carried out for a mining shock. The real, strong mining shock was taken into account. The shock was registered near Szombierki, in the Upper Silesian Coal Basin in Poland [7]. The region where the mining tremor was registered is the main region of mining activity in Poland. The shock was represented by the time history of ground accelerations in three directions: two horizontal- NS, WE and vertical. The accelerations record are presented in Fig. 4.

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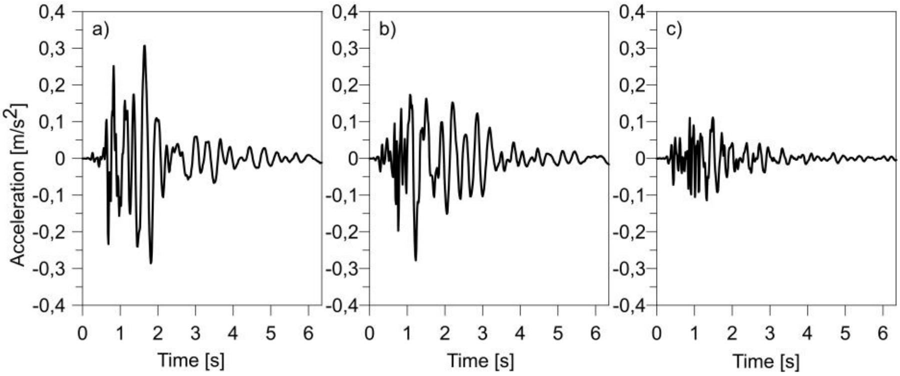

Fig. 4. Time histories of accelerations of the mining shock in: a) NS, b) WE, c) vertical direction.

Based on the figures 4a and 4b it can be seen that the maximum amplitudes in horizontal direction reach 0.3 m/s^2 , whereas the amplitude in vertical direction equal 0.12 m/s^2. In the analysis the mining shock was scaled up by 4. The maximum horizontal acceleration inputted into numerical model equals 1.2 m/s^2 and vertical 0.5 m/s^2. The Fourier analysis of the signal indicated that the dominant frequencies were located within the range of 2-5 Hz for all directions.

The time histories of acceleration were input to the structure as the kinematic excitation of supports. In the numerical model the varying ground motion model were taken into account [8, 9]. The uniform and non-uniform ground motion were applied. The non-uniform excitation model takes into consideration wave passage effect with defined wave speed- the building supports repeat the same motion with a delay depend on the wave velocity. In this study the wave speed of 300 m/s was assumed. It correspond to clayey sands ground. In the uniform excitation model, each supports move in the exactly the same way during analysis time.

5 Response Spectrum Analysis

The dynamic analysis presented in this paper include also the dynamic response spectrum method. This analysis based on spectral curve, specific for Upper Silesian Coal Basin region. The acceleration response spectrum is presented in Fig. 5. The spectrum curve was determined for damping ratio equals 5% and for ground type C (sand) [7]. The spectrum method allows to estimate the maximum stresses in structural elements. The complex simulation (like time history analysis) was not needed in response spectrum method.

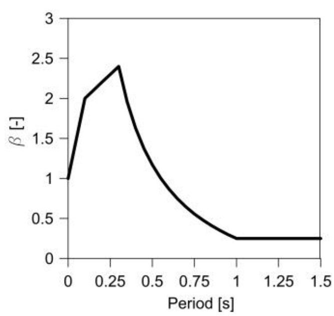

Fig. 5. The acceleration spectral curve.

6 Results

6.1 Uniform excitation

The dynamic response of analysed building was evaluated by the time history analysis. The Hilber-Hughes-Taylor direct integration method was used for the solution of equations of motion. The Rayleigh model of mass and stiffness proportional damping was applied. In the dynamic calculations the dead load and also a part of live load were taken into consideration. During analysis the stress in many points of structure were calculated. In this paper the results for representative points were presented only. The localization of chosen points is presented in Fig. 6. Point P1 and P2 were located in the middle of the span on 1st and 5th floor respectively. Point P3 was located in frame connection.

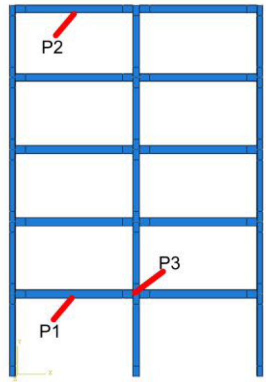

Fig. 6. Representative points chosen during analysis.

The first step of analysis was the calculation of stress for uniform excitation. The time history of stress obtained during calculation was presented in Fig. 7. Fig. 7a presents the stress distribution in point P1; Fig. 7b shows stress in point P2, and Fig. 7c represents results for point P3.

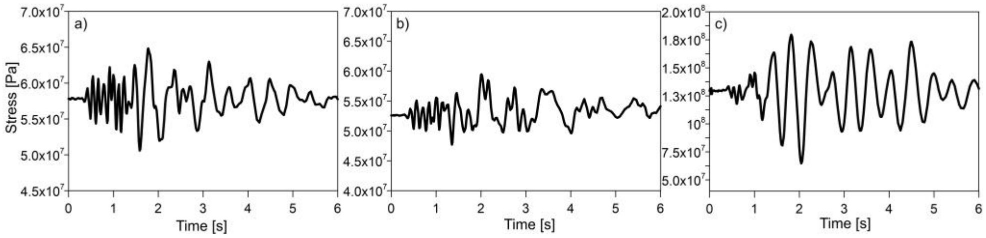

Fig. 7. Time history of stress in point: a) P1, b) P2, c) P3.

In Fig.7 the maximal principal stress is presented. The analysis of stress in point P1 shows that the values of stress oscillated around the constant stress. This stress comes from the dead load of structure. In case of point P1, the dead load stress equals 58 MPa. Based on the presented time history of stress, two regions of stress intensification can be identified. The first range occurred around 2nd second of excitation time and the another was set between 3-3.5 s of shock. Both intensification ranges occurred in the time of local increasing of shock acceleration.

The maximum value of stress in point P1 reach 65 MPa, and appears in 1.9 s of shock. It is the time of maximum peak ground acceleration. The peak stress obtained in point P1 during mine shock excitation is about 15% greater than stress achieved due to the dead load. The minimum value of stress equals 51 MPa.

The stress distribution presented in Fig. 7b (for point P2) shows similar variability in time as stress in point P1. The course of stress indicated two intensification regions too. It is worth to mentioned that this zones appeared at the same moment for points P1 and P2. The maximum stress in point P2 achieved almost 60 MPa, whereas minimum 47 MPa. It was clearly to see that the stress oscillated about value 52 MPa which comes from the self-weight of structure. The stress which occurred in the second intensification region, also reach the significant value. The peak stress (in 3.5 s time of shock) reach 57 MPa and it set 110% of initial stress (stress for dead load).

In addition the stress in frames connection was analysed. The results were presented in Fig.7c. The stress indicate the different variability than in other points. The stress oscillating with lower frequency and show steadily value of stress during whole excitation time. The stress in point P3 achieved maximum value in 2nd second of shock equals 180 MPa. The peak value of stress was almost 40% greater than stress for self-weight.

To complete the frame analysis, the stress distribution in connection was analysed. The map of stress in lower connection was shown in Fig. 8. The figure presents the Misses stress distribution in time of the maximal stress appearance.

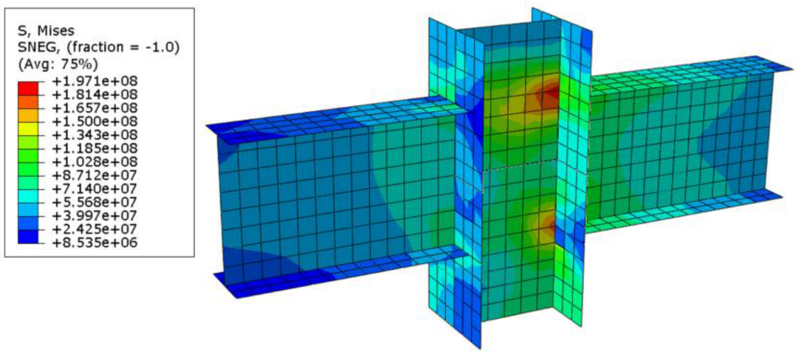

Fig. 8. Mises stress distribution in 2nd second time of shock.

Based on the Fig. 8 it can be noticed that the most charged elements occurred in column. Maximum stress appeared near the place where the beams joint to the column. It was caused by bending moment occurred in connection. The maximum value of stress equals about 200 MPa and was located nearby upper flange plate of girder. The stress in beams were significant lower than in column. The peak value of stress was 120 MPa and it was almost 1.7 times lower than maximum stress in connection.

6.2 Comparison of results obtained for uniform and non-uniform excitation

In the next step, the analysis for non-uniform excitation was conducted. The stress analysis was executed for the same points as previous calculation. The results obtained during both steps of analysis were compared. The comparison were presented in Fig. 9.

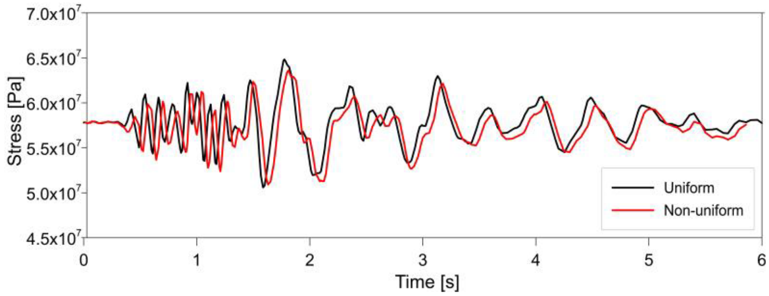

Fig. 9. Comparison the time history of stress in point P1.

Based on the results presented in Fig. 9 it can be seen that the stress obtained for uniform and non-uniform excitation were similar. The time history of stress indicated identical variability in time for both variants of excitation. The stress value for each points were also similar: the differences for stress in points P1 reach 1 MPa, so they did not exceed 10%. The maximum difference occurred form 1-2 s of excitation time. It is worth to mention that the distribution of stress were lagged in time about constant value during the whole shock. The translation was the effect of mining shock wave speed. Hence, the difference in stress values obtained in this point, for uniform and non-uniform excitation were unnoticeable. Only the time delay appeared. Similar situation occurred in points P2 and P3.

6.3 Estimation of results by response spectrum analysis

The last part of analysis was the response spectrum analysis. The calculation for chosen points of frame was conducted. The obtained results were compared to the value of stress from previous calculations. The stress comparison were presented in Table 1.

Table 1. Comparison of peak stress for THA and RSM.

Point	THA stress [MPa]	RSM stress [MPa]	Difference [%]
P1	65	72	10
P2	60	65	15
P3	180	209	17

The results summarized in Table 1 indicate that stress comes from response spectrum method (RSM) were greater than from time history analysis (THA). In each case the spectrum method brought the significant higher stress than the THA. The greatest difference equals 17% and it was reached for point P3 (the point in connection). For the other points the differences equals 10% for point P1 and 15% for point P2. The presented results indicated that the spectrum analysis provide to the secure stress level.

7 Conclusions

The dynamic response analysis of a multi-storey building to a mining shock allowed the formulation some conclusions.

The dynamic analysis for uniform excitation demonstrates, that the strong mining shock caused a significant increase of stresses in structural elements. The greatest increasing of stress was observed near connection of structure. The achieved stress were about 40%

greater than stress comes from the dead load. The maximum stress in connection reach 190 MPa. The stress which appeared in beams, indicated slight increase.

The observation of stress distribution achieved during analysis with uniform and non-uniform excitation, shows that the value of stress was similar for both cases. The analysis with non-uniform excitation indicate a unnoticeable difference in comparison with uniform excitation. The peak value of stress in structure was identical in each case of ground motion model. It shows that the analysed building- compact in plane with short sides- was invulnerable to wave passage effect.

The comparison of results from time history analysis and response spectrum method revealed that the spectrum method allow to estimate the solution. The maximum stress obtained during spectrum analysis were greater than stress from full time analysis. In that case the response spectrum method was the upper assessment of stress. The difference between results from THA and spectrum method were about 10-17%.

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