16th World Conference on Earthquake Engineering, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 4978

Registration Code: S-T1462773387

SEISMIC PERFORMANCE OF A STEEL FRAMED HALL WITH BOLTED CONNECTIONS SUBJECTED TO SPATIALLY VARYING GROUND MOTION

J.M. Dulinska[(1)], D. Jasinska[(2)], P. Boron[(3)]

[(1)] *Professor, Institute of Structural Mechanics, Cracow University of Technology, Poland, jdulinsk@pk.edu.pl*
[(2)] *Associate professor, Institute of Structural Mechanics, Cracow University of Technology, Poland, djasinska@pk.edu.pl*
[(3)] *Teaching assistant, Institute of Structural Mechanics, Cracow University of Technology, Poland, pboron@pk.edu.pl*

Abstract

In the paper the effect of spatial variability of ground motion on the dynamic response of a steel framed hall with bolted connections to a seismic shock was assessed. Constitutive parameters of the steel material were determined experimentally to guarantee proper assessment of non-linear behavior of the structure. The dynamic responses of the hall to uniform and non-uniform kinematic excitation were compared. It occurred that although, for both cases of excitation, the hall lost dynamic stability and the bolted connections were partially disintegrated, some substantial differences were recognized. Despite the fact that the model of uniform excitation resulted in larger global deformations, due to greater values of maximal average accelerations, smaller values of yield measures, and connection degradations were reported in this case. This phenomenon is caused by quasi-static effects occurring in case of non-uniform excitation. Due to excitation non-uniformity, rafter-to-rafter and column-to-rafter connections were affected by additional torsion and bending, resulting from different motions of column footings, leading to larger deterioration of end plates, and failure of bolts. Hence, disregarding non-uniformity of excitation led to non-conservative results, and caused an underestimation of dynamic response of the hall.

Keywords: steel frame, bolted connection, nonlinear seismic response, plastic behavior, spatially varying ground motion

1. Introduction

Spatial variability of ground motion during seismic shocks is often neglected in the dynamic analysis of engineering structures and calculations of the dynamic response of a structure to a seismic shock are based on the assumption of identical movements of every point of the ground beneath the structure. Such a simplification is not recommended for large-dimensional structures (so called multiple-support structures) dimensions of which are comparable with the length of the wave propagating in the ground.

The issue of seismic influences on multiple-support structures and non-uniform kinematic excitation has been comprehensively studied since the 1960's. At that time a dense network of accelerometers was established in Taiwan to recognize spatial variability of ground motion during seismic events [1, 2]. It occurred that points located at short distances indicated significant differences in amplitudes and phases of acceleration.

Authors investigating spatial variability of kinematic excitation claim, that the dynamic response to non-uniform excitation is smaller than the response to uniform excitation. The decrease of the dynamic response is caused by the reduction of average amplitudes of kinematic excitation. However, the authors mention that quasi-static effects, resulting from changes of bedrock geometry may lead to an increase of the dynamic response.

Many researchers investigate bridges as most common multiple-support structures [3, 4]. They noticed that the dynamic behavior of these structures strongly depends not only on inertia forces but also on bedrock deformation during a shock. For long bridges a simplified model of uniform excitation is not recommended [5, 6].

Apart from typical multiple-support structures, like bridges and dams, large-dimensional industrial halls may also be affected by non-uniformity of kinematic excitation [7, 8]. The above-mentioned structures, especially those made of steel, indicate strong nonlinearities, both material and geometrical, while exposed to heavy earthquakes. Material nonlinearity results in local plastic behavior of a primary steel structural system, like yielding with associated plastic flow or plastic hinges. Geometrical nonlinearity, which occurs in large displacements, may lead to a global loss of dynamic stability, or even a total collapse of a structure [9].

The nonlinear performance of connections linking particular members of a primary structural system, like column-to-rafter or rafter-to-rafter roof connections, seems to be of crucial importance in the dynamic analysis [10, 11, 12, 13]. Connections, usually designed as frictional links, are exposed to degradation or even disintegration during strong seismic shocks. In connections, end-plates and panel zones are elements that dissipate the seismic energy released during an earthquake [14]. Partial loss of contact resulting in the decrease of contact surface between connection's members or even total separation of connection's elements may occur due to seismic action.

For these reasons nonlinear behavior of a primary structural system of a steel framed hall is the key issue of the dynamic analysis, and even a small deformation of ground resulting from non-uniform kinematic excitation may cause additional phenomena, e.g. torsion of a structure, that enlarge its seismic response. Hence, the simplifying assumption of uniform kinematic excitation may lead to a non-conservative assessment of dynamic responses of multiple-support steel framed halls to a seismic event.

The main objective of this study was to recognize the influence of kinematic excitation non-uniformity on the dynamic response of a steel hall to a strong seismic shock. The dynamic behaviour of the framed hall subjected to non-uniform kinematic excitation and to simplified uniform excitation were compared. The primary structural system of the hall consisted of steel frames with bolted rafter-to-rafter and column-to-rafter connections. To guaranteed proper assessment of non-linear behaviour of the steel material its constitutive parameters were determined experimentally. Differences in global dynamic response of the hall in case of uniform and non-uniform excitation were pointed out. Furthermore, nonlinear behavior of bolted connections in both cases of excitation was assessed and explained in details.

2. Model of non-uniform kinematic excitation

The equation of motion of a general multi-degree of freedom structure under kinematic non-uniform excitation can be formulated as follows [15]:

$$\begin{bmatrix} M_{ss} & M_{sg} \\ M_{gs} & M_{gg} \end{bmatrix} \cdot \begin{Bmatrix} \ddot{u}_s^t \\ \ddot{u}_g \end{Bmatrix} + \begin{bmatrix} C_{ss} & C_{sg} \\ C_{gs} & C_{gg} \end{bmatrix} \cdot \begin{Bmatrix} \dot{u}_s^t \\ \dot{u}_g \end{Bmatrix} + \begin{bmatrix} K_{ss} & K_{sg} \\ K_{gs} & K_{gg} \end{bmatrix} \cdot \begin{Bmatrix} u_s^t \\ u_g \end{Bmatrix} = \begin{Bmatrix} 0 \\ F_g \end{Bmatrix} \tag{1}$$

where:
s, g – degrees of freedom of structure and ground, respectively,
$[M]$, $[C]$, $[K]$ – mass, damping and stiffness matrix, respectively,
$\{\ddot{u}_s^t\}$, $\{\dot{u}_s^t\}$, $\{u_s^t\}$ – vectors of total accelerations, velocities, and displacements of structure respectively,
$\{\ddot{u}_g\}$, $\{\dot{u}_g\}$, $\{u_g\}$ – vectors of accelerations, velocities and displacements of ground motion, respectively,
$\{F_g\}$ – vector of reaction forces.

The vector of total displacements of a structure $\{u_s^t\}$ (as well as vectors of total velocities and accelerations) consists of two parts: dynamic component $\{u_s^d\}$ and quasi-static component $\{u_s^p\}$, i.e.: $\{u_s^t\} = \{u_s^d\} + \{u_s^p\}$.

The quasi-static component equals:

$$\{u_s^p\} = [R] \cdot \{u_g\} \tag{2}$$

where: $[R] = -[K_{ss}^{-1}] \cdot [K_{sg}]$.

After taking into consideration (2) and assuming small damping, (1) is equivalent to:

$$[M_{ss}] \cdot \{\ddot{u}_s^d\} + [C_s^s] \cdot \{\dot{u}_s^d\} + [K_s^s] \cdot \{u_s^d\} = \left(-[M_{ss}] \cdot [R] - [M_{sg}]\right) \cdot \{\ddot{u}_g\} \tag{3}$$

Since the dynamic response of a structure to kinematic excitation is obtained by numerical integration of (3), it depends on ground accelerations vector $\{\ddot{u}_g\}$. Individual components of this vector represent time histories of ground accelerations at particular supports of a structure. Since time histories of accelerations are registered at selected points, the application of formula (3) requires additionally an assumption of a model of kinematic excitation. On the basis of this model ground accelerations at particular supports could be specified.

There are three phenomena responsible for non-uniformity of kinematic excitation [16]: wave passage effect (difference in time when a wave reaches various points of the structure foundation), incoherence effect (loss of coherence resulting from reflections and refractions of a wave), local soil effects (differences in ground conditions in particular points of subsoil beneath a structure). Among the abovementioned reasons of kinematic excitation non-uniformity, the wave passage effect plays a central role.

In this study a simple model of non-uniform kinematic excitation, including the wave passage effect only, was adopted. It was assumed that subsequent points of ground on the way of the wave propagation repeat the same motion with a certain time delay dependent on wave velocity. Neither loss of coherence nor changes of amplitudes with regard to different local ground conditions were taken into consideration. Hence, the adopted model of non-uniform kinematic excitation required time histories of vibrations registered only at one reference point and wave velocity in the ground, which depends on the type of subsoil.

3. Basic description of the analyzed steel framed hall

The calculations of the dynamic response to a strong earthquake were performed for an existing industrial steel framed hall. The hall had a rectangular shape of the following dimensions: the width 21.0 m and the length

36.0 m. The primary structural system of the hall consisted of 7 single-story steel frames arranged regularly at spacings of 6 m in the longitudinal direction. Each frame had straight vertical columns and tapered rafter sections. No interior columns were mounted; the frames were created as a clear span. The height of the main frames measured from the base level (concrete underlayment) varied from 6.5 m at the eave struts to 7.5 m at the ridgeline of the roof. The frames were fixed at the base. Both columns and rafters were made of straight, rolled H section profiles: HEB 300. The geometry and main dimensions of the hall are presented in Fig. 1.

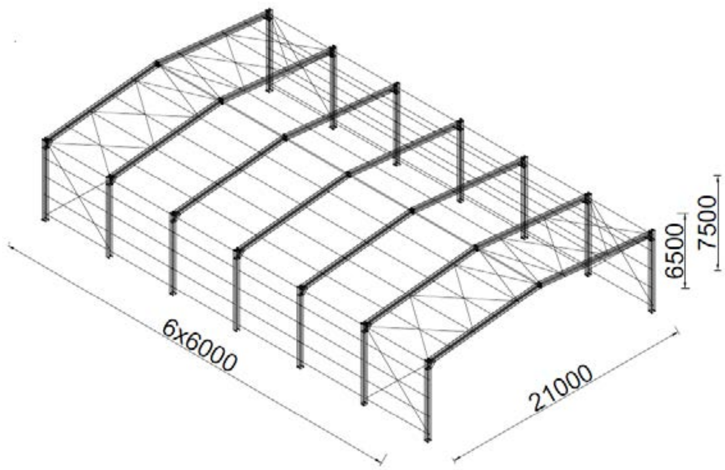

Fig. 1 – Main dimensions [mm] of the steel framed hall

Bolted end-plate rafter to rafter and column to rafter connections are shown in Fig. 2. Both were designed as frictional contact connections. Accordingly to Eurocode 3-8 [17] a friction coefficient of 0.2 was assumed. This value is usually expected in case of surfaces with no pretreatment. The roof ridge constructed as a bolted apex connection is shown in Fig. 2a. The connection consists of two end plates and three rows of 24 mm diameter bolts (two bolts in a row). The end plates were made of 25 mm thick steel sheets of dimensions 30 cm x 41.5 cm. The end plates were welded to the rafters and joined together by the bolts. The bolts were pre-tensioned with the force of 200 kN. The typical behavior of such a connection under the dead and live load from snow results in tension of the bottom side and compression of the upper side of the connection. For this reason two rows of bolts were placed in the bottom part, and one in the upper part of the end plate.

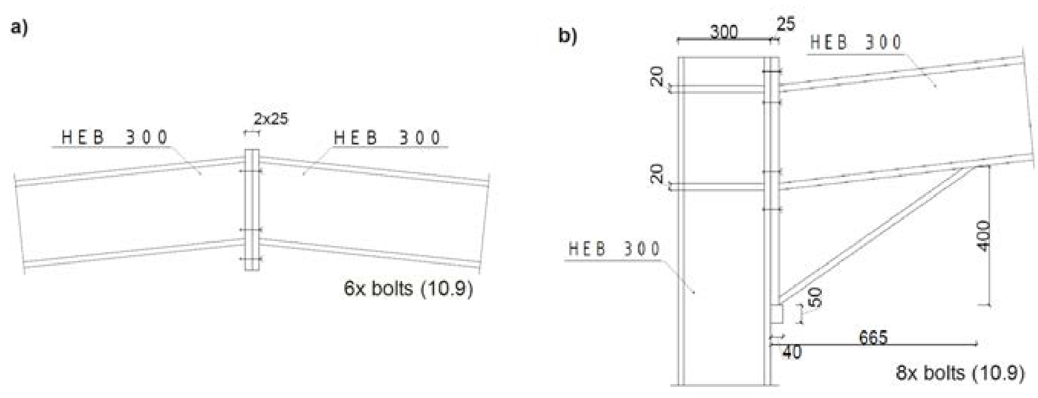

Fig. 2 – Details and dimensions [mm] of: (a) bolted apex connection between rafters in the roof ridge, (b) end-plate joint connection between the column and the rafter

Connections between columns and rafters were created as end-plate joints. The end plates were welded to the rafters and fastened to the columns by eight 24 mm diameter bolts arranged in four rows. The bolts were pre-tensioned with the force of 200 kN. Additionally, the connection was strengthened by a haunch. The details and dimensions of the apex and column-rafter connections are presented in Fig. 2b.

The secondary structural system of the main hall consisted of roof purlins, girts, eave struts and bracings. The roof area was equipped with purlins: horizontal beams spanning between frames. The purlins were the principal members of the roof secondary support system supporting roof panels, transferring loading to the frames and helping stabilize the roof. The girts constituted the principal members of the wall secondary support system. They, like the purlins, transferred the loads imposed on the covering system of the wall panels onto the frames. Both the purlins and the girts were designed as simply supported beams, connected to the main steel frames by bolts. They did not constitute continuous beams. This fact was of a crucial importance for the dynamic analysis. Since purlins and girts, constructed as simply supported beams did not stiffen the whole structure as much as continuous beams, the entire structure was relatively soft in the direction perpendicular to the planes of the main frames, despite wall and roof bracings, used to increase the out of plane stability. The torsion of the rafters was almost free in this case. The displacements of frames in the out-of-plane direction were also less limited.

4. Experimentally determined constitutive parameters for steel material

The elements of the analyzed frame were made of structural steel of a commercial symbol S235JR. The elasto-plastic model of the steel material was assumed for the analysis. The real nonlinear behavior of the structural steel during the dynamic analysis was guaranteed by experimentally determined material parameters. The stress-strain curve for the structural steel was obtained on the basis of a tensile test of a rectangular steel specimen, which was performed using the Zwick-Roell universal testing machine. For comparison, a numerical simulation of the above mentioned test was also carried out. The numerical process was conducted with the ABAQUS software. The specimen was discretized by the SHELL S4R finite elements. In the numerical calculations the parameters of the non-linear elasto-plastic steel material were taken from the experiment.

The theoretical strain-stress curve was determined as a result of the experiment. Hence, it could be stated that the parameters of the elasto-plastic model of steel were verified experimentally.
Figures 3a and 3b show the experimental tensile test and its numerical simulation, respectively, for a specimen made of the structural steel material. The comparison of the curves obtained from the numerical simulation and from the experimental tensile test demonstrated in Fig. 3c shows high conformity. The experimentally obtained yield curve data of the structural steel are summarized in Table 1.

The elasticity modulus of 195 GPa was also obtained from the experimental test. The Poisson's ratio of 0.3 and the mass density 7850 kg/m^3 were assumed.

The material of the bolts was also described as an elasto-plastic. The yield stress of 900 MPa and the limit stress of 1003 MPa were assumed for the steel material of bolts on the basis of literature [18]. The elasticity modulus of 210 GPa was used.

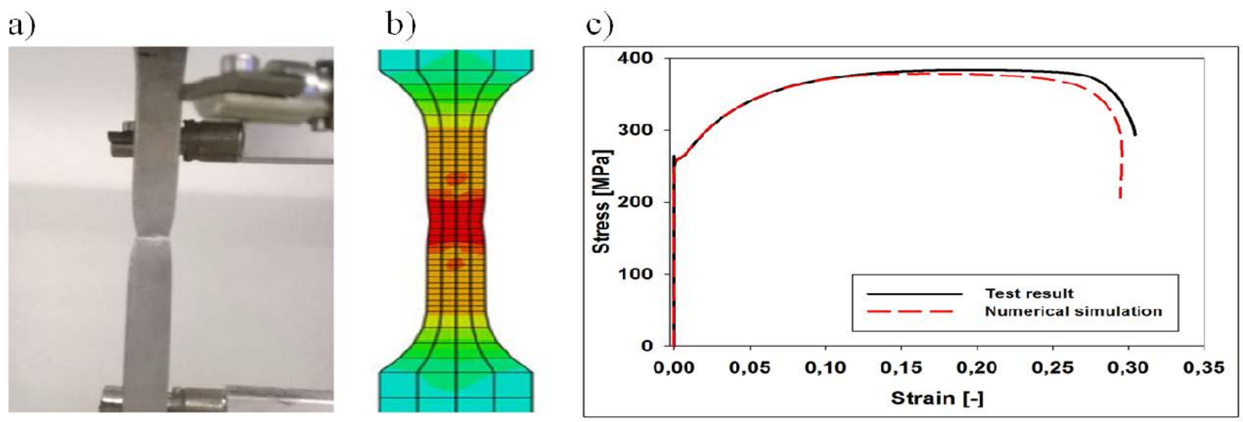

Fig. 3 – Tensile test of the steel material: (a) experiment, (b) numerical simulation, (c) comparison of stress strain curves obtained from experimental tensile test and computer simulation

Table 1 – Constitutive parameters of the elasto-plastic steel material

Yield stress [MPa]	**Plastic strain [-]**	**Yield stress [MPa]**	**Plastic strain [-]**	**Yield stress [MPa]**	**Plastic strain [-]**	**Yield stress [MPa]**	**Plastic strain [-]**
245	0	259	0.00168	300	0.0215	370	0.096
250	0.0002	260	0.0025	320	0.033	375	0.112
251	0.00026	265	0.00631	330	0.0405	378	0.126
252	0.00032	270	0.0087	340	0.049	380	0.1375
255	0.00055	280	0.0127	350	0.06	383	0.173
257	0.001	290	0.0168	360	0.075	383	0.22

5. Seismic input data

A strong seismic shock of Northridge (1994) [19] was investigated in this study. The time histories of accelerations registered in three directions are shown in Fig. 5. The magnitude of the shock equaled 6.7. The maximal values of accelerations in horizontal directions NS and WE equaled 17.45 and 9.72 m/s^2, respectively. The maximal value of 10.28 m/s^2 was recorded in the vertical direction. The presented time histories of vibrations were applied as the kinematic excitation (accelerations) acting on the structure in three directions for both models (uniform and non-uniform).

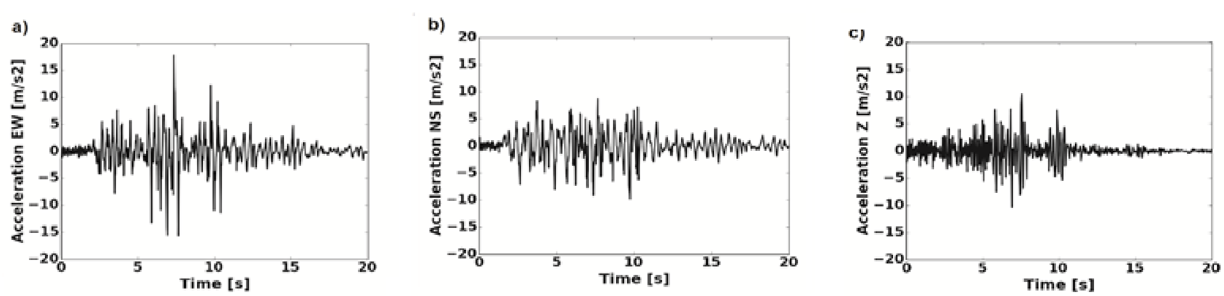

Fig. 5 – Time history of accelerations in: (a) horizontal NS direction; (b) horizontal WE direction, (c) vertical direction

6. Numerical model of the steel hall with bolted connections

6.1 Comments on the numerical model of the entire hall

In this study a three dimensional FE model of the steel framed hall was created using the ABAQUS code. Only one, central main frame was discretized with about 5000 8-node continuous shell finite elements SC8R, provided by the ABAQUS element library. The remaining frames as well as elements of the secondary structural system, i.e. purlins, girts and bracings, were modelled with beam elements. The dead load of the roof and the walls as well as the live load were replaced by concentrated forces. The seismic motion of the ground was applied directly to the column footings. These simplifications of the numerical model were implemented to minimize the size of the problem, and to reduce numerical effort.

6.2 Details of the numerical model of bolted connections

Nowadays, 3D numerical models of steel structures are usually created for dynamic analyses. However, particular members of these structures are often modeled with beam elements. In such models details of connections are not modeled at all. In more precise analysis shell elements are used, e.g. to discretize steel webs and flanges in rolled section profiles of rafters and columns as well as details of connections. The most precise models of connection members, like end plates, panel zones and bolts are built with solid elements. It is essential to obtain good agreement in the process of linking full scale experiment and FE modelling [20, 21].

In this study the entire connections (end plates, bolts and fragments of rafters and columns close to the end plates) were modelled with about 30000 3D brick linear C3D8R finite elements included in the ABAQUS library. In order to obtain accurate results the mesh of the zones adjacent to analyzed connections were densified. The numerical models of the apex and the column to rafter connections with a FE mesh are presented in Fig. 6.

The unilateral frictional contact between end-plates and bolts (both shanks and washers) as well as between two end plates, was modeled by surface-to-surface contact elements. To allow for misplaced bolts, holes in end plates were 2 mm oversized.

The pretension of the bolts was realized by generating initial thermal strains by assuming thermal expansion coefficient of the bolt material $1.08 \cdot 10^{-5}\ {}^{1}\!/_{{}^{0}C}$, and cooling the bolts by $530^{0}C$. The bolts' pretension caused initial compression of the end-plates resulting in a concentration of plastic zones (about 0.8 % equivalent plastic strain) around the bolt holes even before the dynamic shock had been imposed on the structure.

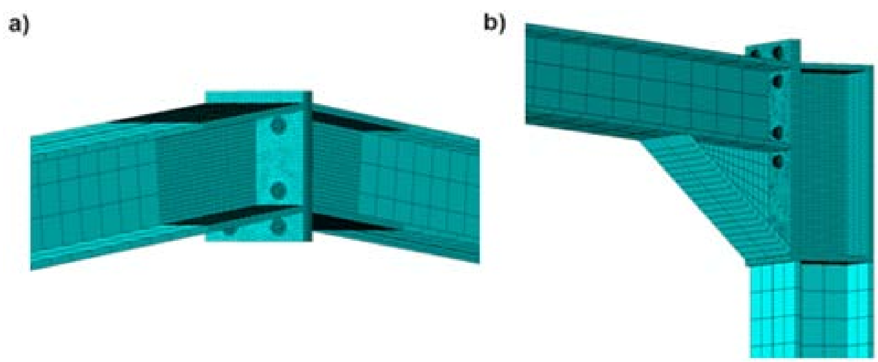

Fig. 6 – Details of bolted connection models with a FE mesh: (a) the apex connection, (b) the column-to-rafter connection

7. Comparison of dynamic responses of the hall to the seismic shock obtained for uniform and non-uniform kinematic excitation

The dynamic responses of the steel framed hall to the seismic shock were evaluated by the time history analysis using the Hilber-Hughes-Taylor direct integration method for the solution of equations of motion. A minimal time step increment of 10^{-5} s was necessary for this highly nonlinear analysis to obtain convergence.

The Rayleigh model of damping, proportional to the stiffness and the mass of the structure, was applied with coefficients determined for damping ratios 2.5 % referring to the first and the second circular frequencies.

In the dynamic analysis of the hall both, uniform and non-uniform, models of kinematic excitation were taken into consideration. In the model of non-uniform excitation the wave velocity of 100 m/s was assumed that corresponds to coarse sands.

7.1 Global dynamic loss of stability of the frame

The final configurations of the hall fragment after the analyzed seismic shock (see Fig. 5) for both excitation models are presented in Fig. 7. The nonlinear dynamic behavior of the hall under the shock was reported similar for both, uniform and non-uniform kinematic excitation. From the very beginning of the shock the main frame performed oscillations around the initial configuration up to about 8 s. At that moment the amplitudes of ground motion had grown substantially and the frame lost its global dynamic stability. Three separate yielding zones were diagnosed on the entire frame. The first one was localized in the lowest parts of both columns with the maximal value of equivalent plastic strains of 2.7 %. The second and third zones covered both, the apex and the column-to-rafter connections. They both underwent partial deterioration.

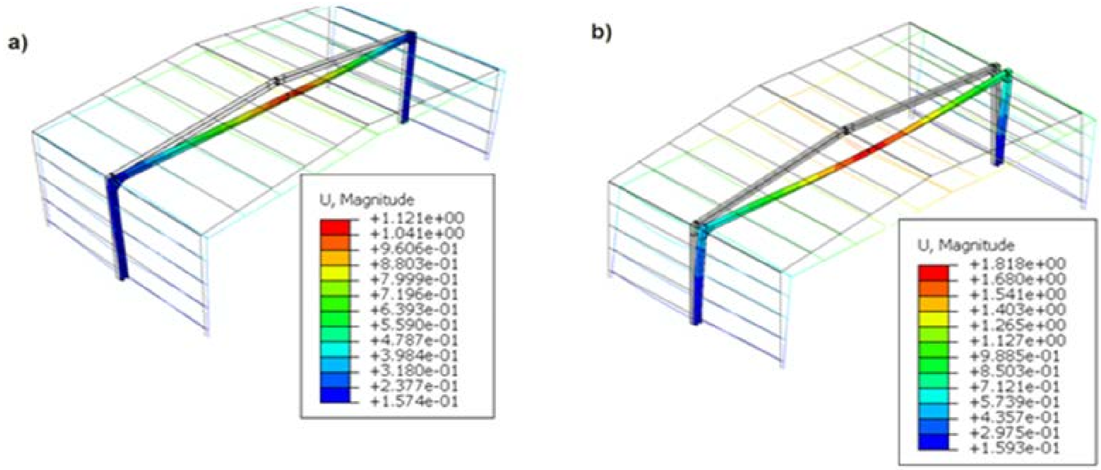

Fig. 7 – The final configuration of the main frame resulting from the loss of dynamic stability: (a) non-uniform excitation - the apex connection displaced by 1.1 m and rotated by 40^{o}, (b) uniform excitation - the apex connection displaced by 1.8 m and rotated by 45^{o}

Despite the large displacements and rotations caused by the phase of strong ground motion, the hall did not collapse. It stabilized in a new state of equilibrium configuration and from now on (8 s) to the end of the shock oscillated around this new configuration.

In case of uniform excitation (see Fig. 7b) the apex connection of the frame is displaced by approximately 1.8 m. The rotation of the roof ridge, reaching about 45^{o} is also clearly visible. In case of non-uniform ground motion (see Fig. 7a) the displacement and rotation of the apex connection achieved the values of approximately 1.1 m and 40^{o}, respectively. Hence, they both were smaller than in case of non-uniform excitation. That effect can be explained by the fact that the average acceleration amplitudes applied to the foundation were smaller when non-uniformity of excitation is taken into consideration. It resulted in the reduction of inertia forces acting on the overground part of the structure.

7.2 Dynamic behavior of the apex connection

Nonlinear dynamic behavior of the bolted apex connection under the seismic shock was extensively examined and presented in details in Figs 8-12. The dynamic responses are compared for both, uniform and non-uniform, cases of kinematic excitation.

Firstly, the final configurations of the end plates of the apex connection along with the distribution of equivalent plastic strains (PEEQ) obtained for both excitation models are analyzed in Fig. 8. The end plates and the bolts are shown separately to reveal the plastic strain concentrations in the end plates in the vicinity of the bolts' holes and in the bolts' shanks.

It could be noticed that the maximal value of this plastic measure in the end plates is larger in case of non-uniform than in case of uniform kinematic excitation: it reached 0.041 (see Fig. 8a) and 0.031 (see Fig. 8b), respectively. Hence the application of the non-uniform excitation model led to a 25 % PEEQ increase.

The loss of contact between the end plates is also evident for both discussed excitation models (Figs 8 and 10). It is clearly visible that a large displacement and rotation of the bolted apex connection is accompanied by a partial separation of the end plates. However, the split of the end plates was greater for non-uniform excitation.

Secondly, the separation of the end plates resulted in further indicators of degradation of the apex connection: the bolts affected by additional tension resulting from the split underwent significant yielding in case of excitation non-uniformity. The comparison of equivalent plastic strains in the shank of the bolt A, located in the central area of the apex connection, is shown in Fig. 9.

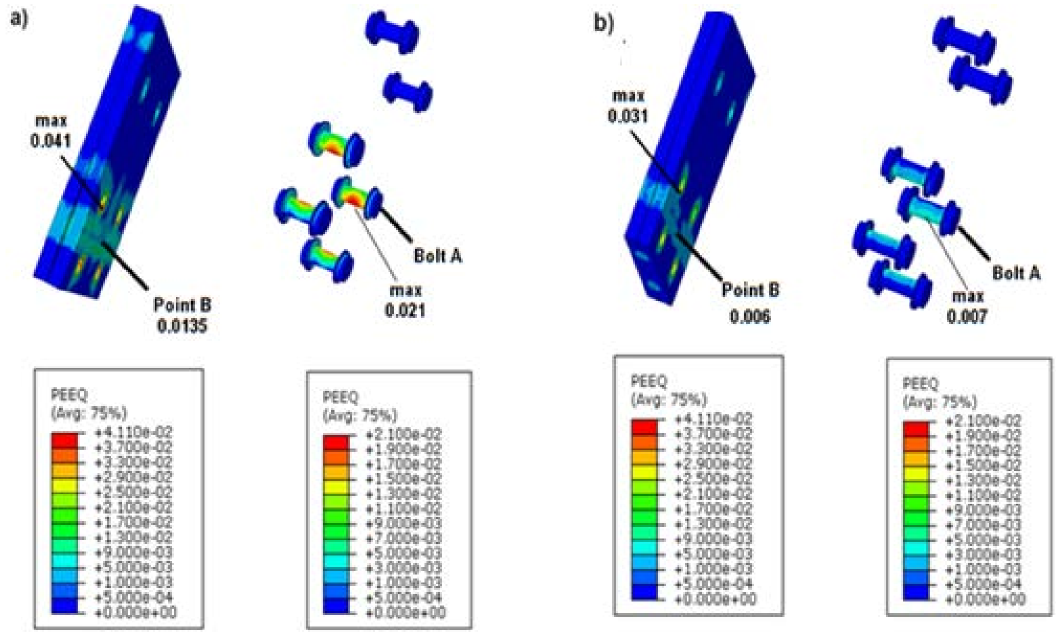

Fig. 8 – Nonlinear dynamic behavior of the bolted apex connection (equivalent plastic strains in the end plates and bolts): (a) non-uniform excitation, (b) uniform excitation

The distribution of PEEQ in the bolts' shank for non-uniform excitation with the maximal value of approximately 2.1 % is shown in Fig. 9a. It can be noticed that the entire cross-section of the bolt shank went plastic. That points towards total failure of the pre-tensioned middle bolts. For uniform excitation the middle bolts show hardly any sign of yielding. Only the part of the cross-section was affected with the maximal value of 0.7 % as presented in Fig. 9b. In both cases the distribution of plastic measure indicates the effect of bending of the bolts. It occurred due to the partial split and the deterioration of the end plates.

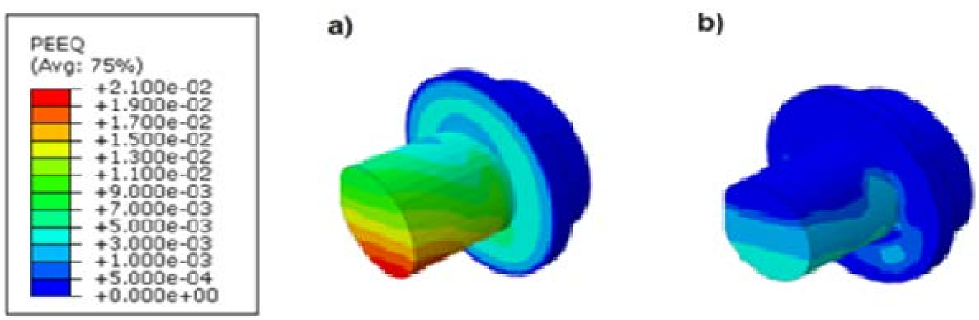

Fig. 9 – Comparison of the yielding in the shank of the pre-tensioned steel bolt A bent due to the end plates deterioration: (a) non-uniform excitation, (b) uniform excitation

The distribution of contact stresses for both analyzed cases are shown in Fig. 10. In case of non-uniform excitation contact loss concerned the entire central area of the apex connection (Fig. 10a), whereas in case of uniform excitation (Fig. 10b) contact stresses around the central bolts were still reported.

The differences in the apex connection behavior, pointed out in Figs 9 and 10, demonstrate that the application of a proper excitation model might be of crucial importance. Even though the displacements of the connection are greater in case of uniform excitation (see Fig. 7), the opposite situation is recognized for the yield measures. If the model of non-uniform kinematic excitation is taken into consideration, the apex connection is affected by additional torsion resulting from different motion of the frame footings in direction perpendicular to the frame plane. This torsional motion produces more noticeable deterioration of the end plates and total failure of the central bolts. As the simplified uniform model of excitation does not reveal such effects, its application prevents the proper assessment of the connections seismic response.

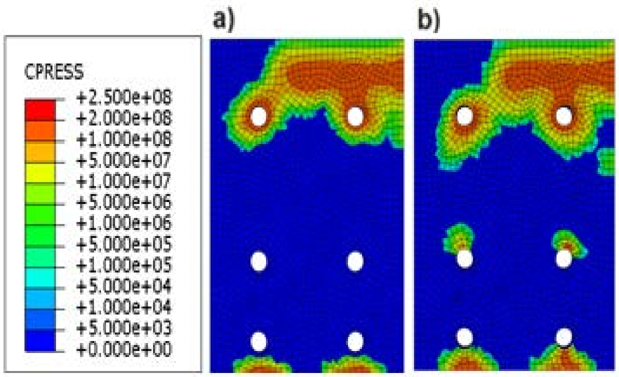

Fig. 10 – Comparison of final contact stresses (CPRESS) on the internal surface of the end plate of apex connection: (a) non-uniform excitation, (b) uniform excitation

The development of equivalent plastic strains and maximal principal mechanical strains at the shank of bolt A is displayed in Fig. 11a and 11b, respectively. Both measures are significantly greater in case of non-uniform excitation. The presented time histories also demonstrate that the oscillations of strains, resulting from ground vibration, were negligibly small in comparison with the rapid increase in strains that occurred due to the loss of dynamic stability at about 8 s.

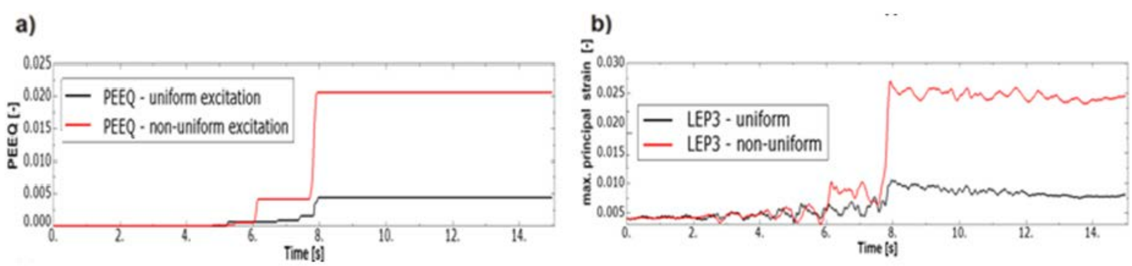

Fig. 11 – Comparison of time histories at the shank of bolt A obtained for non-uniform and uniform excitation: (a) equivalent plastic strains, (b) maximal principal mechanical strains

Finally, two charts in Fig. 12 illustrate the comparison of stress-strain curves for the pre-tensioned shank of bolt A for non-uniform (red line) and uniform (black line) excitation models. In both cases, after the global loss of stability of the frame, the bolt underwent plastic yielding and the substantial jump of principal strain was observed, reaching a level of approximately 2.7 % for non-uniform and 1.1 % for uniform excitation. Several stages of elastic loading-unloading cycles were observed, with the final elastic stress-strains oscillations around the new state of equilibrium configuration (see Fig. 7), that begun after the phase of the strong ground motion had been finished.

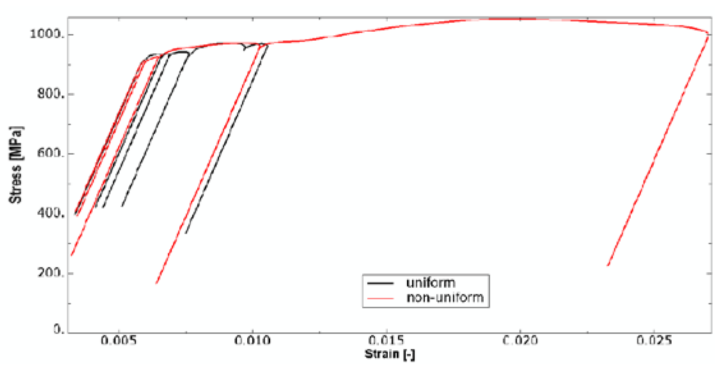

Fig. 12 – Comparison of the stress-strain curves for the pre-tensioned steel bolt obtained for non-uniform and uniform excitation

7.3 Dynamic behavior of the column-to-rafter connection

The final configurations of the column-to-rafter connection along with the distribution of equivalent plastic strains obtained for both excitation models are presented in Fig. 13.

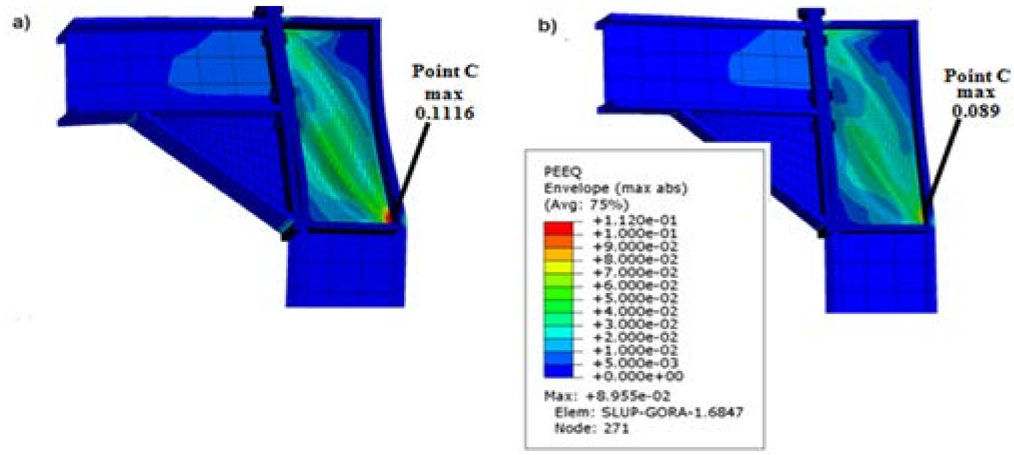

Fig. 13 – Comparison of equivalent plastic strains and deterioration of the end plates of the bolted column to rafter connection: (a) non-uniform excitation, (b) uniform kinematic excitation

It could be noticed that the maximal value of this plastic measure in the end plates is larger in case of non-uniform than in case of uniform kinematic excitation, reaching 11.16 % (see Fig. 13a) and 8.9 % (see Fig. 13b), respectively. It means that the application of the non-uniform excitation model led to approximately 25 % PEEQ increase, as was the case for apex bolts.

The loss of contact between the end plates is also evident for both discussed cases. It is clearly visible that large displacement and rotation of the bolted column-to-rafter connection is accompanied by the partial separation of the end plates. The split of the end plates was greater for non-uniform than for uniform excitation as was the case for the apex connection. No bolts' yielding was detected in the column-to rafter connection.

8 Conclusions

In the paper the nonlinear dynamic response of a steel framed hall to a strong seismic shock was analyzed. Two kinematic excitation models, uniform and non-uniform, were applied in the calculations. The nonlinear behavior of the bolted apex and column-to-rafter connections were studied in details. The following conclusion, similar for both uniform and non-uniform cases of excitation, can be formulated on the basis of the analysis:

1. Global nonlinear behavior of the structural frame occurred – it lost dynamic stability in consequence of the phase of strong ground motion.
2. Local non-linear behavior and disintegration of the bolted connections took place - the contact surface between the end plates of the connections considerably decreased.

However, substantial differences in the dynamic response of the hall were recognized under different models of excitation. Despite the fact that the application of uniform excitation model resulted in much larger global displacements and deformations due to greater values of maximal average accelerations, smaller values of yield measures and connection degradations were reported in this case. This phenomenon is caused by so called quasi-static effects occurring in case of non-uniform excitation, in which connections were affected by additional torsion and bending resulting from different motion of the frame footings. These quasi-static effects led to more noticeable deterioration of end plates and even to total failure of central bolts.

The differences in the dynamic response of the hall to uniform and non-uniform seismic excitation described above indicate, that the simplified, uniform excitation model can lead to non-conservative results in calculations of the dynamic response of large dimensional steel halls to seismic shocks. Hence, ignoring the wave passage effect may cause underestimation of dynamic response of the steel hall.

9. References

[1] Abrahamson NA et al. (1987): The SMART 1 accelerograph array (1980-1987): A review. *Earthquake Spectra*, **3** (2), 263-287.

[2] Shin TC, Tsai YB, Liu CC, Wu YM (2003): Strong-motion instrumentation programs in Taiwan. *International Handbook of Earthquake and Engineering Seismology*. Academic Press, San Diego, CA, Part B: 1057-1062.

[3] Bo Z, Yuanqing W, Zhihua C, Yongijiu S, Yang J, Yihong W (2015): Research on the random seismic response analysis for multi- and large-span structures to multi-support excitations. *Earthquake Engineering and Egineering Vibration*, **14** (3), 527-538.

[4] Zanardo G, Hao H, Modena C (2002): Seismic response of multi-span simply supported bridges to a spatially varying earthquake ground motion. *Earthquake Engineering and Structural Dynamics*, **31** (11), 1325-1345.

[5] EN 1998-1:2004 Eurocode 8 - Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings, ENV 1998-1, CEN, Brussels.

[6] Dulinska JM, Jasinska D (2015): Plastic behavior of integral bridge, consisting of supporting steel beams and concrete superstructure, under spatially varying seismic shock. *Key Engineering Materials*, **626,** 438-443.

[7] Jasinska D, Dulinska JM, Boron P (2015): Influence of spatial variability of seismic shock on steel hall with experimentally determined elasto-plastic parameters of steel material. *Applied Mechanics and Materials*, 744-746, 884-889.

[8] Jasinska D, Dulinska JM, Boron P (2015): Nonlinear Dynamic Response of a Steel Frame with Bolted Apex Connection to a Strong Seismic Shock. *Proc. of the 2015 International Conference on Civil Engineering (CIVILENG 2015), Zakhyntos Island, Greece, 2015*, 154-161.

[9] Rahimi E, Estekanchi HE (2015): Collapse assessment of steel moment frames using endurance time method. *Earthquake Engineering and Engineering Vibration*, **14** (2), 347-360.

[10] Ghassemieh M, Jalalpour M, Gholampour AA (2014): Numerical evaluation of the extended endplate moment connection subject to cyclic loading. *Current Advances in Civil Engineering*, **2**, 35-43.

[11] Bahaari MR, Sherbourne AN (1996): 3D simulation of bolted connections to unstiffened columns - II. Extended endplate connections. *Journal of Constructional Steel Research*, **40** (3), 189-223.

[12] Mahmoud H, Elnashai A, Spencer B, Kwon O, Bennier D (2013): Hybrid Simulation for Earthquake Response of Semi-Rigid Partial-Strength Steel Frames. *Journal of Structural Engineering*, 139, 1134–1148.

[13] Calado L, De Matteis D, Landolfo R (2000): Experimental response of top and seat angle semi-rigid steel frame connections. *Materials and Structures*, **33** (8), 499-510.

[14] Hedayat A, Celikag M (2008): Enhancement of Panel zone (PZ) contribution to the ductility of post-Northridge welded connections. *WSEAS Transactions on Applied and Theoretical Mechanics*, **3** (5): 165-174.

[15] Clough RW, Penzien J (1993): *Dynamics of structures*, McGraw-Hill, Inc., NY.

[16] Zerva A (2009): *Spatial variation of seismic ground motions. Modelling and engineering applications*. CRC Press.

[17] PN-EN 1993-1-8 Design of steel structures: Design of joints.

[18] Johnson L, Palmiere E, Thackray R, Burgess I, Davison B (2013): Behaviour of axially loaded structural bolting assemblies in fire. *Application of Structural Fire Engineering, Proc. of International Conference in Prague, 2013.*

[19] Pacific Earthquake Engineering Research Center (PEER) (2015): https://ngawest2.berkley.edu, released on May, 2015.

[20] Ivan A, Ivan M, Both I (2010): Comparison of FEA and Experimental Results for a Steel Frame Connection. *WSEAS Transactions on Applied and Theoretical Mechanics*, **5** (3), 187-196.

[21] Tong G, Lianglong S, Guodong Z (2011): Numerical simulation of the seismic behavior of self-centering steel beam-column connections with bottom flange friction devices. *Earthquake Engineering and Engineering Vibration*, **10** (2), 229-238.