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WAVE PASSAGE AND INCOHERENCE EFFECTS ON DYNAMIC RESPONSE OF A PRE-STRESSED VIADUCT UNDER A NATURAL EARTHQUAKE

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Abstract

The aim of this study was to assess the influence of spatial variation of ground motion on a 4-span pre-stressed concrete viaduct (160 m long) subjected to an earthquake. A seismic shock recorded in January 2012 in Poland was used as the kinematic excitation of the viaduct. In the dynamic analysis two models of non-uniform kinematic excitation were used. Firstly, a model of excitation in which only the wave passage effect was taken into account was applied. Secondly, a model of excitation considering the incoherence effect only was assumed. In both models of non-uniform kinematic excitation ground motion records at one point and a seismic wave velocity had to be known to generate earthquake records for all supports of the structure. In the analysis of the incoherence effect an effective engineering method for conditional stochastic simulation of ground motions was performed in the time domain. The results obtained for both models of non-uniform kinematic excitation were compared to those obtained for the model of uniform excitation that assumed identical motion of viaduct supports. The analysis revealed that the dynamic response of the viaduct to the shock is strongly determined by the assumed model of kinematic excitation. If the model of non-uniform kinematic excitation is applied the dynamic response may differ even up to 60% in relation to the response obtained for the model of uniform excitation. To assess the influence of seismic wave velocity on the dynamic response of viaduct to the earthquake various wave velocities were implemented. The analysis proved that the dynamic response of the viaduct strongly depended on the wave velocity. In case of the model of non-uniform excitation including the wave passage effect only, the dynamic response of the structure declined with the decrease of wave velocity. It is a consequence of the reduction of the average amplitudes of kinematic excitation. In case of the model of non-uniform kinematic excitation considering the incoherence effect only, the dynamic response of the viaduct substantially increases with decreasing wave velocity. For low wave velocities the dynamic response of viaduct is considerably greater for the model including the incoherence effect than for the model of uniform excitation. This occurs due to the quasi-static effects which result from changes of subsoil geometry during seismic shocks. Therefore, the application of conventional model of uniform excitation may lead to underestimation and non-conservative assessment of the dynamic response of a long structure, especially for low wave velocities.

Keywords: seismic response, spatial variation of ground motion, incoherence effect, wave passage effect

1. Introduction

In standard analysis of a dynamic response of structures to kinematic excitations, spatial variation of ground motion is commonly neglected although seismic waves present high variability in space. It means that calculations of the dynamic response of a structure to an earthquake are based on the assumption that motions of all points of the ground beneath a structure are identical. However, the influence of the spatial variation of excitation on the dynamic response of large structures may be significant. These structures are exposed to spatially different ground vibrations, since dimensions of their bases are comparable with the length of seismic waves. Generally, authors claim that the dynamic response to a non-uniform kinematic excitation is smaller than the dynamic response to a uniform excitation. The decrease of the dynamic response is caused by a reduction of average amplitudes of excitation. On the other hand, some authors mention that quasi-static effects that result from differences in a kinematic excitation in particular points of foundations may lead to an increased response.

The spatial variation of ground motion causing the non-uniformity of kinematic excitation of large dimensional structures has been comprehensively analysed in the last decades. At that time in Taiwan a measuring system SMART-1 was put into operation [1, 2]. The system, consisting of a dense network of accelerometers, recorded ground motions during seismic events and continuously monitored a zone of high seismic activity. The results of the vibration measurements at points located at a short relative distance allowed to determine the wave propagation velocity in the ground, the coefficients of coherence of vibrations at neighbouring points and the registration of the effects of increasing or reduction of amplitudes resulting from the local soil properties. In 1993 a large experimental site EUROSEISTEST located near Thessaloniki (northern Greece) was established. The project included integrated experimental and theoretical research studies in seismology, earthquake engineering, soil dynamics and structural engineering [3, 4].

In recent studies authors refer to three reasons of the spatial variation of seismic ground motion [5]: (a) finite velocity of wave propagation in the ground (wave passage effect), (b) loss of coherency resulting from reflection, interference and scattering of seismic waves in the ground (incoherence effect), (c) difference in ground conditions in particular points of subsoil beneath a structure (local soil effects).

The first reason papers of spatial variation of ground motions, i.e. the wave passage effect, occurs due to the fact that seismic waves reach the subsequent supports of the structure with a certain time delay dependent on the wave velocity. At the earliest the wave passage effect was considered by Werner [6]. On the basis of theoretical analyses and observations of destruction of large-dimensional structures during earthquakes, the author noticed that the wave passage effect led to the reduction of the dynamic response in relation to the response obtained for the uniform excitation. Morgan et al. [7] introduced the concept of *transit time* – the time of a shockwave transition under a structure - that is equal to the ratio of the length of the base of structure to the velocity of wave propagation in the ground. In this approach the kinematic excitation acting on the foundation at a certain moment is an averaged excitation during that time, so the wave passage effect causes the reduction of the effective excitation and, in consequence, the reduction of the global dynamic response of the object.

The second reason of the spatial variation of ground motion, i.e. the loss of coherency of seismic waves, occurs due to the scattering and interfering of waves arriving from a source. It is denoted as the incoherence effect. A number of empirical coherency models for spatially varying ground motions has been introduced on the basis on the data from SMART-1 [8, 9, 10]. Also semi-empirical models, for which functional forms are based on analytical assumptions but their parameter evaluation requires recorded data, are widely used [11, 12, 13]. as Finally, analytical models of coherency are considered. These studies have shown that the incoherence effect tends to increase with the increasing distance between supports, or with increasing frequency of ground motion.

The dynamic response of long bridges, which are typical multiple-support structures exposed to the non-uniform kinematic excitation during earthquakes, has been extensively studied by many authors. Zerva [14] presented a detailed analysis of the dynamic response of two- and three-span bridges exposed to the spatially varying excitation considering both the loss of coherency and the wave passage effects. Norman et al. [15] compared experimental and numerical results for a multiple-support excitation of a long bridge. Sextos et al. [16] studied the influence of spatial variability of earthquake ground motions on curved bridges. Bi et al. [17]

presented the combined effects of spatial variation of ground motion and local site amplification on a bridge response. The effects of the soil-structure interaction and local soil conditions on the seismic response of multi-span bridges were also investigated by Zembaty & Rutenberg [18] and Carbonari et al. [19]. The phenomenon of spatial variation of ground motions represented as the combined effect of three causes, i.e. the loss of coherence of the motion with distance, the wave-passage effect and the local site condition, were analysed by Lupoi et al. [20], Sextos et al. [21] and Bi et al. [22]. Saxena et. al. [23] estimated the relative significance of these three effects on the seismic response of long multi-span reinforced concrete highway bridges. Dulinska & Jasinska [24] considered the dynamic behaviour of an integral bridge under the non-uniform kinematic excitation using the concrete damage plasticity material model.

The aim of this study was to assess the influence of spatial variation of ground motion on a 4-span pre-stressed concrete viaduct subjected to an earthquake. In the dynamic analysis two models of kinematic excitation were used. Firstly, a model of non-uniform kinematic excitation in which only the wave passage effect was taken into account was used. Secondly, a model of excitation with the incoherence effect was applied. The results obtained for the non-uniform models of kinematic excitation were compared to those obtained for the uniform excitation that assumed identical motion of the supports of the viaduct. To assess the influence of a seismic wave velocity on the dynamic response of the viaduct to the earthquake various wave velocities were implemented.

2. The Main Data of the Viaduct

The evaluation of the dynamic response to an earthquake was conducted for an existing 4-span pre-stressed concrete viaduct with elastomeric bearings located in Poland. The structure is 157.5 m long. The length of the longest span is 56 m. The height of the piers varies from 7 to 21 m. The abutments are situated 32 and 34 m away from the extreme piers. The height of the cross section of the concrete deck changes along the length of the viaduct form 1.9 m at the abutments to 3.3 m at the central pier. The general view of the viaduct and geometry of the cross section of the girder above the central pier are presented in Fig. 1. The bridge is equipped with elastomeric bearings as linking elements between the superstructure and the piers. The elasticity modules are taken as 4.4 and 3.8 GPa for the superstructure and the piers, respectively. The fixed boundary conditions reflected the high rigidity of the foundation subsoil. The numerical model of the viaduct was created with the ABAQUS software (Fig. 2). The deck and girders were discretized with the 10-node quadratic tetrahedron elements of 50 cm. There were three bands selected above the supports where the mesh was more dense with the elements of about 20 cm. For the piers and abutments the 8-node linear brick elements of 80 cm were applied.

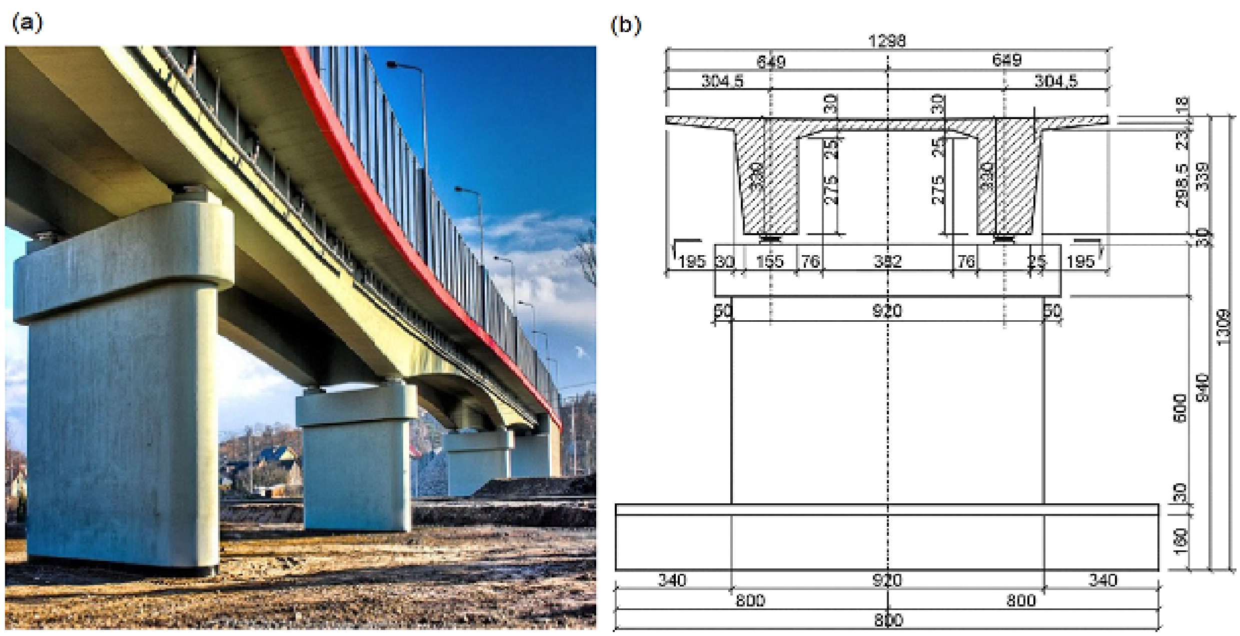

Fig. 1 – The general view (a) and geometry of the cross section the central pier of the viaduct (b)

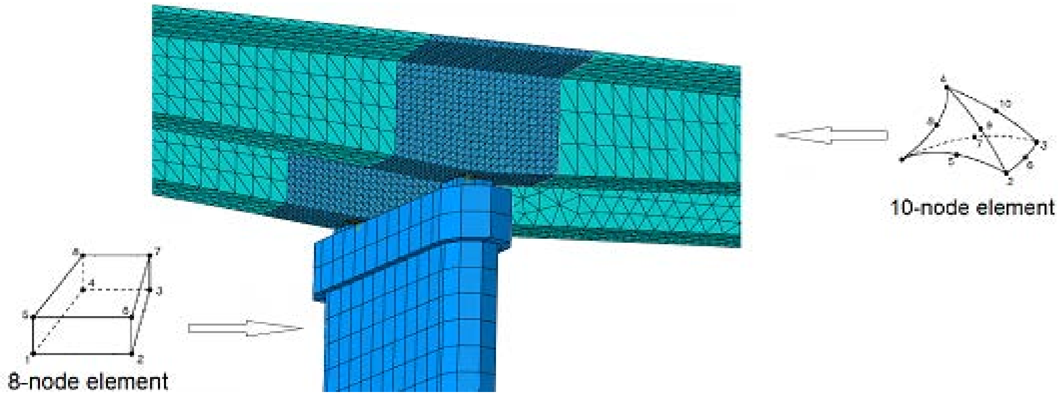

Fig. 2 – The fragment of the numerical model with the mesh of the primary structural system

3. Data of the Seismic Shock

In this study a real seismic shock of a magnitude 3.8 in Richter scale with the epicenter in Jarocin (Poland), registered in January 2012, was used as the kinematic excitation of the viaduct. Time histories of velocities in three directions were recorded by a seismic station located near the epicenter. However, for the dynamic analysis time histories of accelerations, obtained by differentiating the registered velocity data, were used. The time histories of accelerations in three directions and their frequency spectra are shown in Figs 3 and 4, respectively. The dominant frequencies are located within the range from 2 to 4 Hz. For this study purposes the amplitudes of accelerations were scaled up in order to obtain the maximum value of acceleration equaled 0.4 m/s^2. This is a value of horizontal peak ground acceleration (PGA) of earthquakes predicted in seismic hazard studies and a seismic risk map for the area where the shock occurred [25].

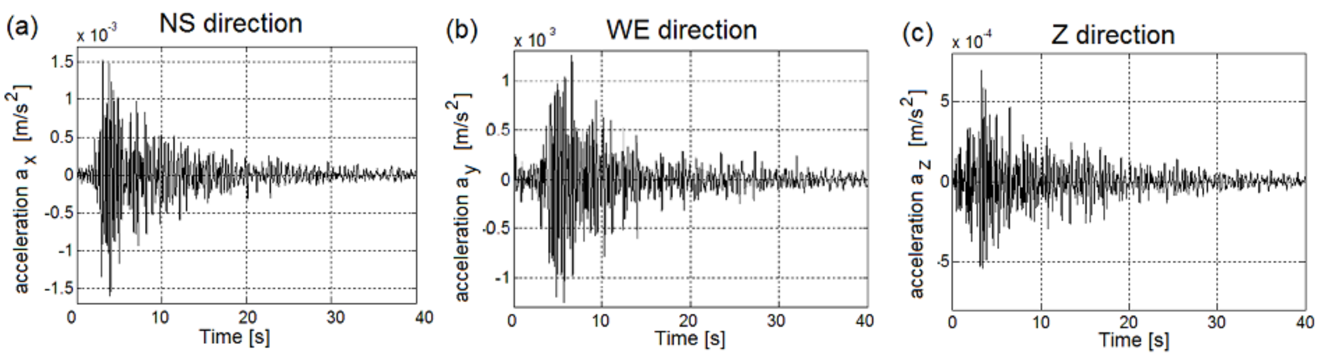

Fig. 3 – Time histories of ground accelerations in: (a) north-south direction (NS), (b) west-east direction (WE), (c) vertical direction (Z)

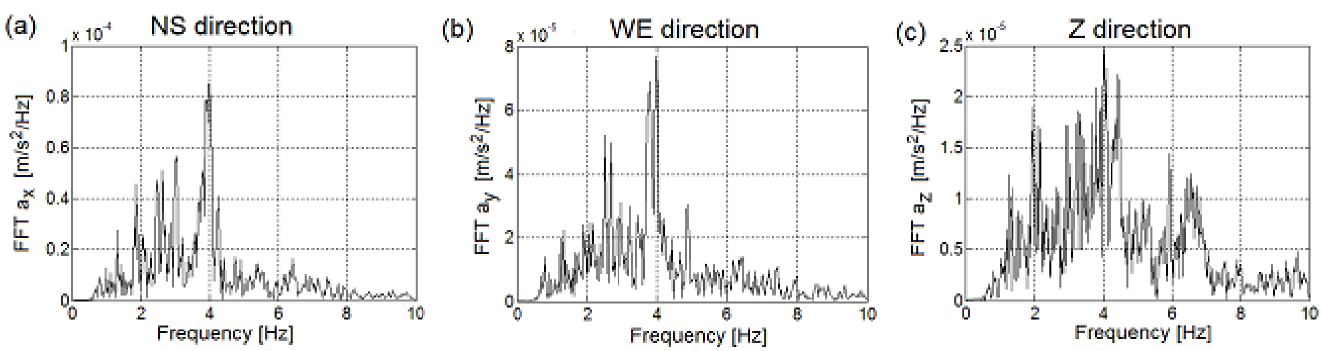

Fig. 4 – Frequency spectra of ground accelerations in: (a) north-south direction (NS), (b) west-east direction (WE), (c) vertical direction (Z)

4. Simulation of Earthquake Ground Motions for Particular Supports of the Viaduct

In the study two models of non-uniform kinematic excitation were assumed: the model of excitation in which only the wave passage effect was taken into account and the model of excitation in which only the incoherence effect was considered. Additionally, for comparative analysis purposes, the model of uniform kinematic excitation was implemented, in which identical motions of all supports of a structure were assumed.

It was assumed for both cases of excitation models that the known earthquake time histories in three directions (see Fig. 3) are specified for the first support of the viaduct. Hence, for both models of ground motion, the time histories of accelerations for the other four supports of the viaduct had to be simulated. It was also assumed that the north-south (NS) direction is parallel to the longitudinal axis of the viaduct, whereas the west-east (WE) direction is perpendicular to it. The separation distances of the particular supports from the first support were assumed accordingly to the viaduct geometry as 34, 90, 124 and 156 m, for the consecutive supports, respectively.

The finite velocity of shock waves is a basic and dominant reason of the non-uniform kinematic excitation which has to be considered in case of large-dimensional structures. To assess the influence of the wave velocity on the dynamic response of the viaduct to the earthquake the following wave velocities were implemented for both models of ground motion: 250, 500, 1000 and 2000 m/s. The lowest value of 250 m/s can be applied for sands, whereas the highest value of 2000 m/s can be used for stiff rocky subsoils. The calculations were also carried out for the uniform kinematic excitation, that refers to the *infinite* wave velocity.

4.1 Determination of ground motion records considering the wave passage effect only

In the model of non-uniform kinematic excitation, in which the wave passage only is taken into account, it was assumed that the seismic waves propagated parallel to the longitudinal axis of the viaduct. The subsequent supports of the viaduct repeated the same motion as the first support with a certain time delay dependent on wave velocity. Hence, the ground motion record for the particular support of the viaduct was determined by shifting the known record by the time in which the wave covered the distance between the first and the analyzed support.

4.2 Determination of ground motion records considering the incoherence effect only

In the model of non-uniform kinematic excitation, in which only the incoherence is considered, the conditional stochastic modelling method of ground motion records, given by Jankowski & Wilde [26] and Jankowski [27], was applied. The authors propose an effective engineering method for conditional stochastic simulation of ground motions to study the response of long structures.

In order to generate earthquake records for all supports of a structure, the ground motion record at one support and a seismic wave velocity have to be known. The simulation of ground motion records for the other supports of a structure is performed in the time domain. However, the time domain formulation involves computations of convolution integrals and may be relatively troublesome [27] if the spatial correlation function depends on all frequencies the spectrum of the earthquake consists of. The simulation of ground motion records in the time domain can be much easier performed, when the frequency dependence of the spatial correlation function is simplified. The method simplifies the frequency dependence of spatial correlation to one function so that only the correlation of the predominant frequency of the earthquake is taken into account. Since a band of frequencies which dominates the response of engineering structures like bridges is narrow, in this study only one spatial correlation function was assumed to represent the correlation for the band of predominant frequencies. The spatial correlation function of the field takes the form (Eq. 1):

$$K(\xi) = \sigma^2 e^{\left(\frac{-f_d|\xi_j-\xi_i|}{2\pi v\alpha}\right)} \tag{1}$$

where: $\xi_{i\,(j)}$ – separation distance at the point i (j) of the field, f_d – predominant frequency of the shock, v – wave velocity, α – space scale parameter ($\alpha > 0$), which depends on local geological conditions, σ – standard deviation of the recorded shock.

Jankowski & Wilde [26] and Jankowski [27] describe the numerical procedure, based on the acceptance-rejection theorem, which can be used for the simulation of ground motion records. The procedure required the assumption of the predominant frequency of the shock and the space scale parameter α, which depends on local geological conditions and specifies the degree of correlation of points of the field. The parameter α should be obtained experimentally on the basis of the data recorded in the region of seismic activity with dense arrays. The higher the value of α is, the higher the correlation between points of random field is expected. In the study this algorithm was applied to determine unknown time histories of ground motions for all supports on the basis of the specified earthquake record at one location. The predominant frequency of the shock equalled 4 Hz was assumed (see Fig. 3). The space scale parameter was assumed as 1, that corresponds to the highest incoherency of the field. Three components (longitudinal NS, transverse WE and vertical Z) of the generated time histories of accelerations of all supports, obtained for wave velocity v = 250 m/s, are presented in Fig. 5.

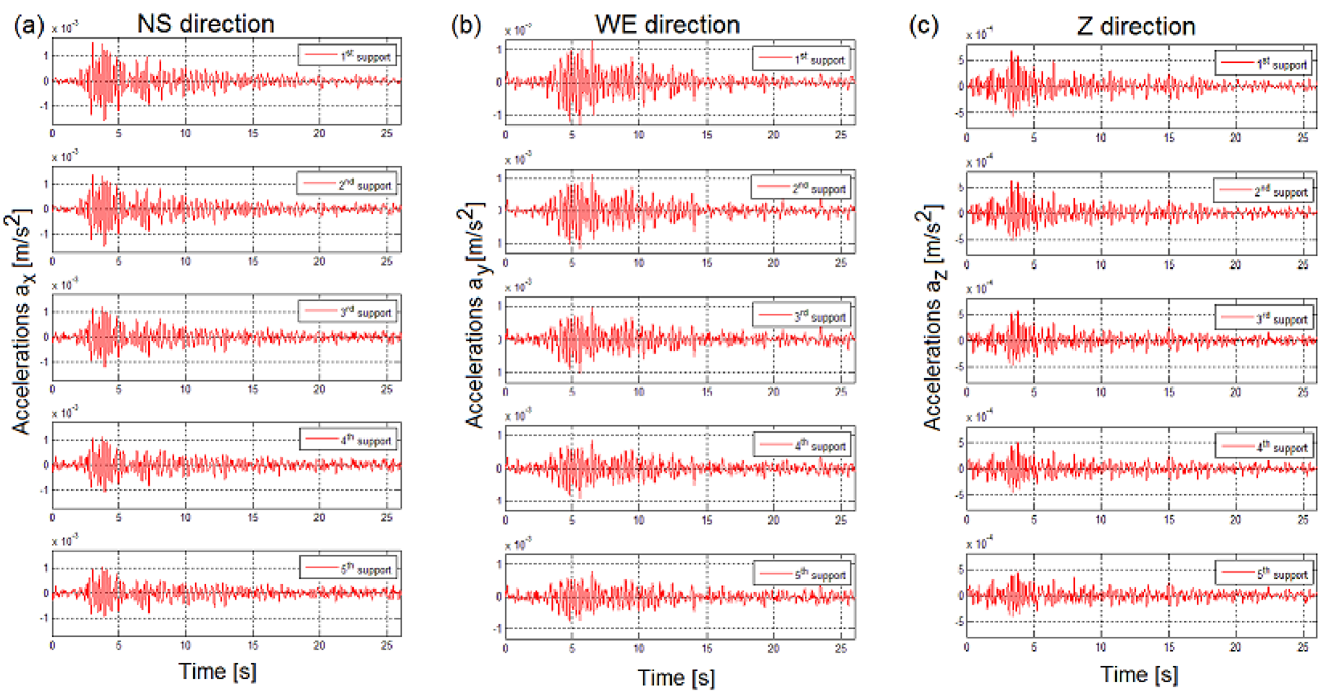

Fig. 5 – Time histories of ground accelerations simulated for all supports for wave velocity 250 m/s: (a) longitudinal component NS, (b) transverse component WE, (c) vertical component Z

The autocorrelation functions of generated time histories of accelerations obtained for the wave velocity 250 m/s are shown in Figs 6a, 7a and 8a for longitudinal NS, transverse WE and vertical Z direction, respectively. The autocorrelation function at time $t = 0$ reduces to the average power of the signal, i.e. to the Root Mean Square value (RMS). Hence, it is clearly visible that all generated time histories of ground motions indicate a reduction of average power for the consecutive supports. The reduction of 40 % and 45 % between the first and the last support can be noticed for the horizontal component NS and WE, respectively. Smaller reduction of the RMS value, reaching 30 %, can be observed for the vertical component Z. The comparison of frequency spectra of ground motions of the first (known) and the last support (generated) are presented in Figs 6b, 7b and 8b for three directions, respectively. The values of FFT functions on the last support are reduced in the same extent as the autocorrelation functions. Nevertheless, the predominant frequency remains unchanged.

Similar simulations were performed for the wave velocities of 500, 1000 and 2000 m/s. The obtained results bear resemblance to the ones described above. The higher wave velocity was assumed, the smaller drop in average power at the consecutive supports was obtained. In case of wave velocity of 500 m/s the difference of average power (as well as maximum acceleration) for the first and the fifth support was about 20 %, whereas in case of wave velocity 2000 m/s - less than 10 %. It is also worth noticing that the generated time histories of ground motions become less scattered with the increasing velocity of the wave.

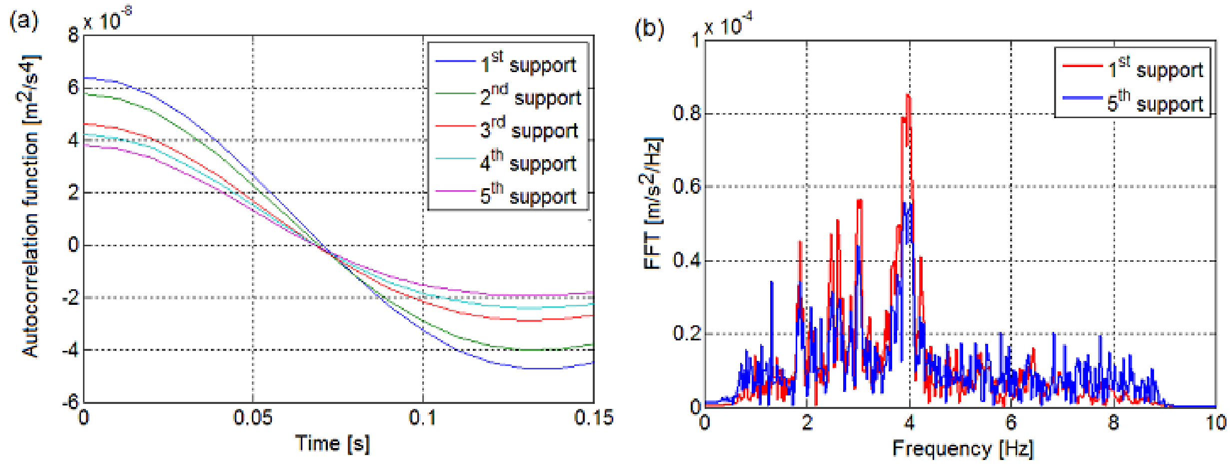

Fig. 6 – (a) Autocorrelation functions of generated time histories of accelerations and (b) comparison of frequency spectra of the ground motions for the first and for the last support (longitudinal component NS)

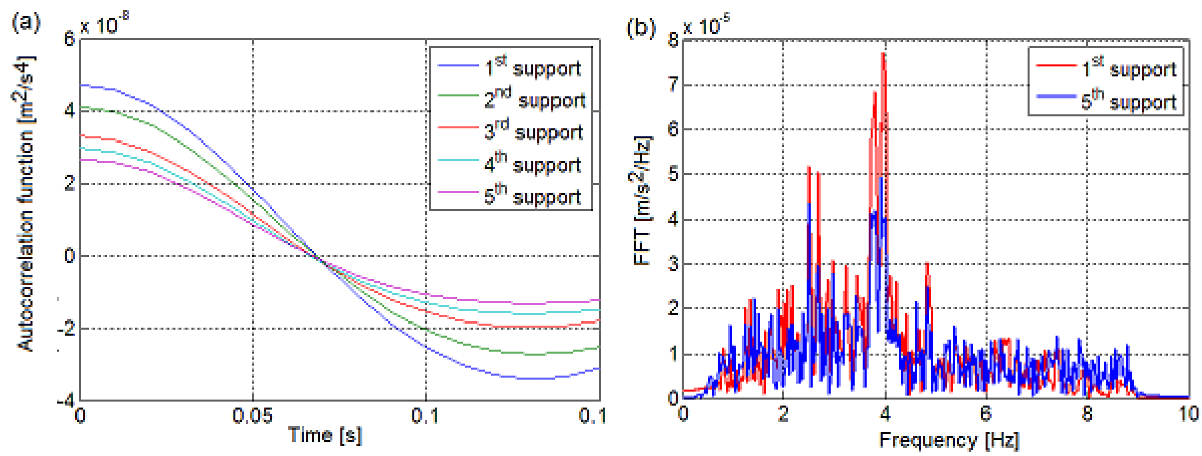

Fig. 7 – (a) Autocorrelation functions of generated time histories of accelerations and (b) comparison of frequency spectra of the ground motions for the first and for the last support (transverse component WE)

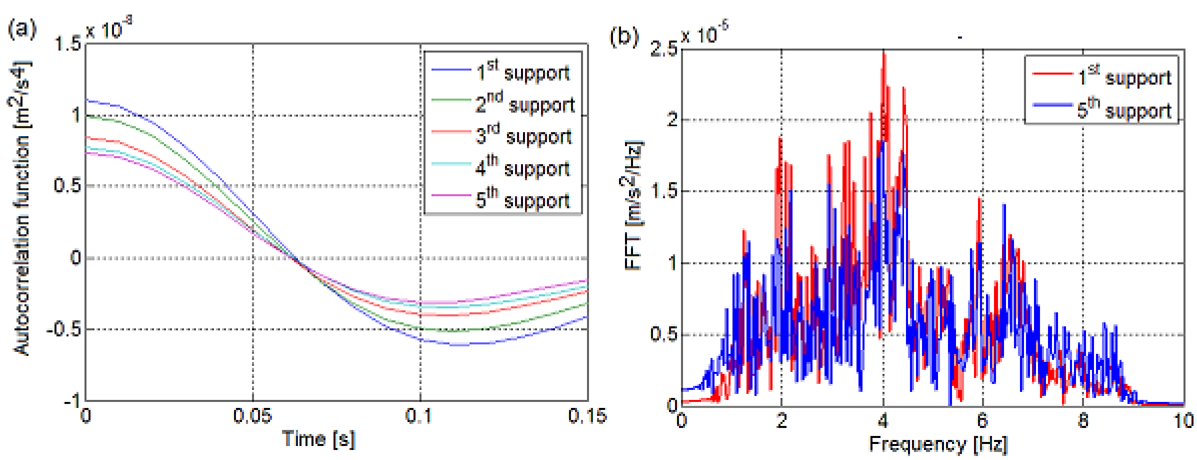

Fig. 8 – (a) Autocorrelation functions of generated time histories of accelerations and (b) comparison of frequency spectra of the ground motions for the first and for the last support (vertical component Z)

5. Numerical Results

The ground motion records generated for all supports were applied to conduct the dynamic analysis of the viaduct subjected to the earthquake. The Hilber-Hughes-Taylor time integration algorithm for a direct step-by-step solution, provided by the ABAQUS software, was used for full time history analysis. The Rayleigh model of mass and stiffness proportional damping was applied. The damping coefficients α = 0.965 (mass proportional damping) and β = 0.0024 (stiffness proportional damping) were determined assuming damping ratio of 5% for the first (2.38 Hz) and second (4.32 Hz) natural frequencies corresponding to the first and second vertical bending mode shapes.

5.1 Representative points chosen for the dynamic analysis of the viaduct

The influence of the wave passage and the incoherence effects on the dynamic response of the viaduct was assessed at eight cross sections – four near the supports and four in the middle of each span. At each section five points were selected for the analysis – two at middle bottom of each girder, two at middle top of each girder and one in the middle of the deck. The selected sections and points (with symbols used to describe their location) are presented in Fig. 9a and 9b, respectively.

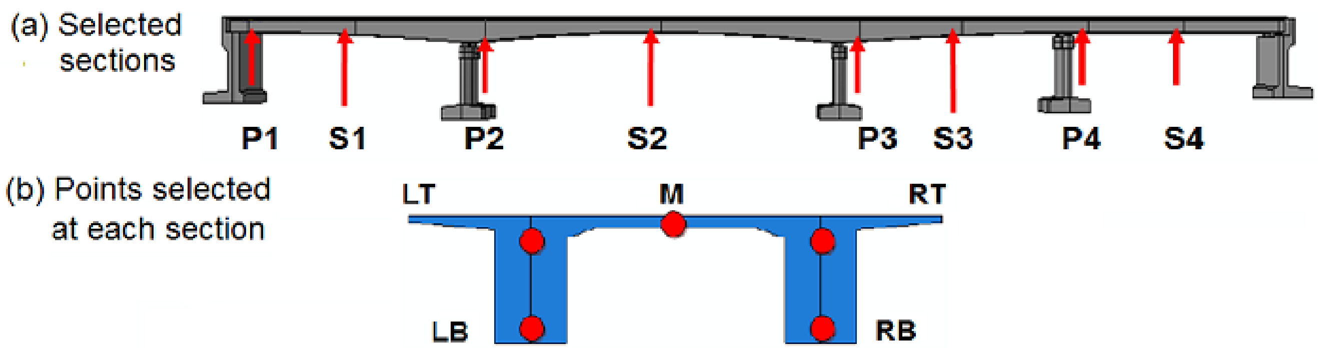

Fig. 9 – Cross sections (a) and points at each cross section (b) with their symbols selected for the dynamic analysis

5.2 The influence of the wave passage effect on the dynamic response of viaduct

The influence of wave passage effect on the dynamic response of viaduct is analyzed in Fig. 10, for the representative point P2RB (next to the second support in the bottom of the right girder). The comparison of maximum principal tensile stresses obtained for the uniform and non-uniform excitation with the wave velocity of 500 m/s indicates a significant decrease of stresses in case of the non-uniform excitation.

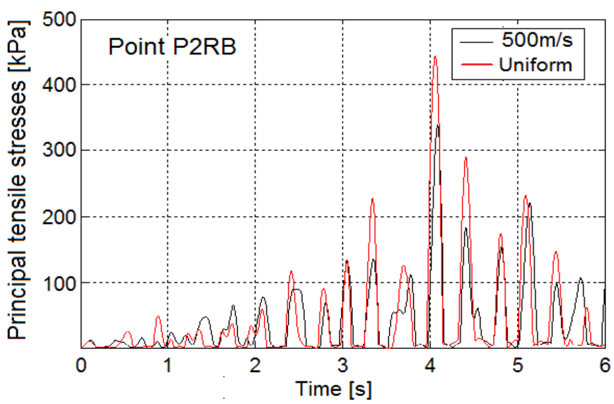

Fig. 10 - Time histories of principal tensile stresses at point P2RB obtained for the uniform excitation and non-uniform excitation considering the wave passage only (wave velocity 500 m/s)

The dependence of maximum principal tensile stresses on the wave velocity is shown in Fig 11. It can be observed that at the representative points situated next to the first support (points P1) and next the second support (points P2) the maximum principal tensile stresses decrease with the decreasing wave velocity. Similar tendencies for bridges of various structural solutions were reported by other authors [5] (especially for bridges equipped with elastomeric bearings). The decrease in the dynamic response occurs due to a reduction of effective kinematic excitation acting on a structure, caused by the wave passage effect [7].

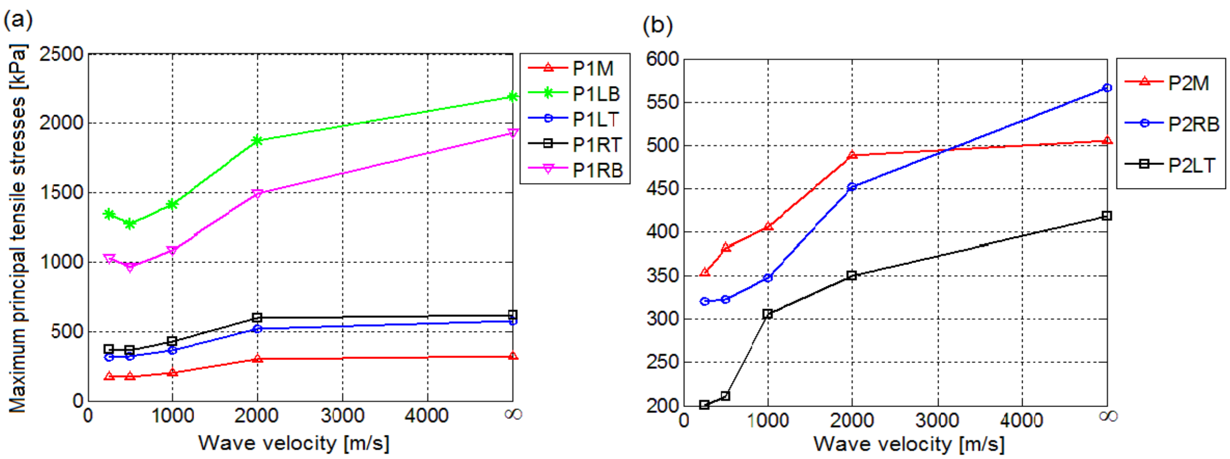

Fig. 11 – Dependence of maximum principal tensile stresses on wave velocity obtained for non-uniform excitation considering the wave passage effect for points next to: (a) the first support, (b) the second support

5.3 The influence of the incoherence effect on the dynamic response of viaduct

The influence of incoherence effect on the dynamic response of viaduct is presented in Fig. 12 for the representative point S2RB. It can be noticed that in case of non-uniform excitation considering the incoherence effect only the reverse situation occurred than in case of the wave passage effect. The comparison of maximum principal tensile stresses indicates that the dynamic response obtained for the non-uniform excitation (wave velocity of 500 m/s) is about 20 % greater than the response obtained for the uniform excitation.

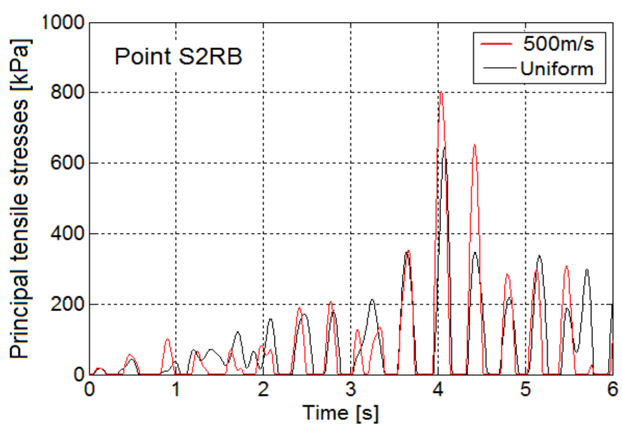

Fig. 12 - Time histories of principal tensile stresses at point S2RB for the uniform excitation and the non-uniform excitation considering the incoherence effect only (wave velocity 500 m/s)

The dependence of maximum principal tensile stresses on the wave velocity is presented in Fig. 13. Although the average power of kinematic excitation is reduced for the consecutive supports (see Figs 6-8) the dynamic response of the viaduct, in terms of maximum principal tensile stresses, tends to increase for the wave velocities lower than 1000 m/s. The loss of cohrence and the strong scattering of the generated time histories of accelerations, that occurs for low wave velocities, impose significant quasi-static effects resulting in the increase of the dynamic response of structure.

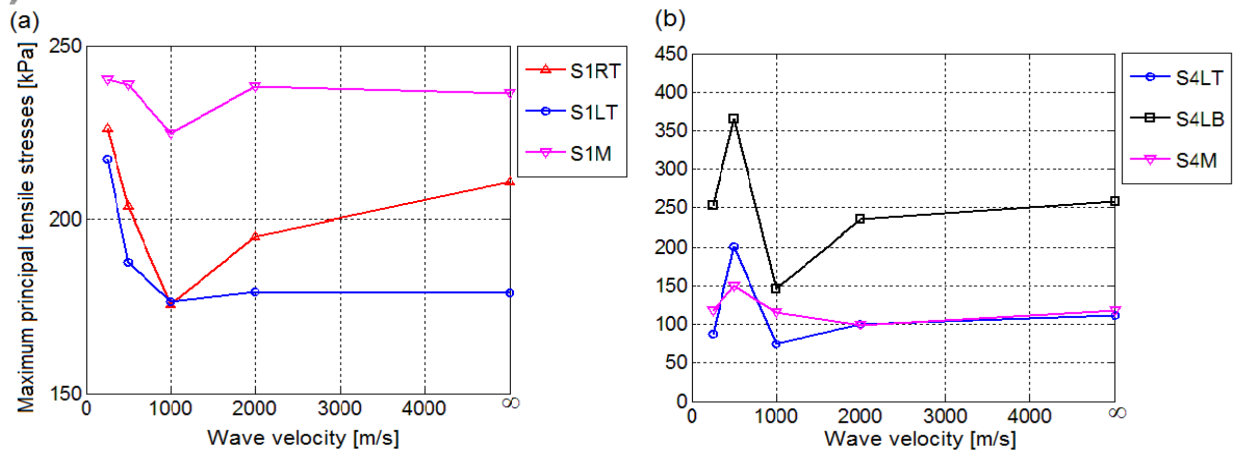

Fig. 13 – Dependence of maximum principal tensile stresses on wave velocity obtained for non-uniform excitation considering the incoherence effect only for points next to: (a) the first support, (b) the second support

5.4 Comparative analysis of the obtained results

The dependence of dynamic response on the wave velocity obtained for two models of non-uniform excitation, one considering the wave passage and the other considering the incoherence effect only, are compared in Fig. 14.

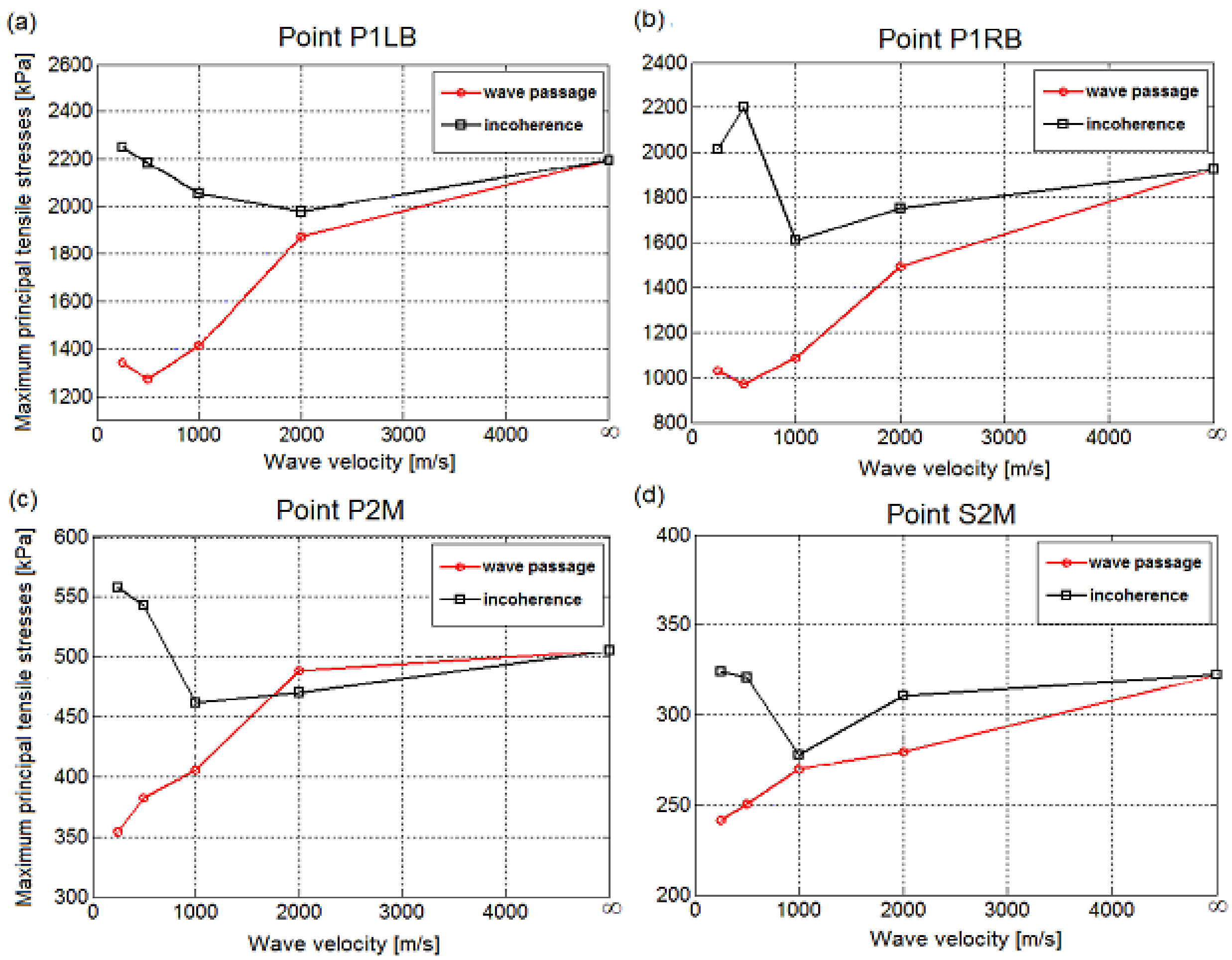

Fig. 14 - Dependence of maximum principal tensile stresses on wave velocity obtained for the non-uniform excitation considering the wave passage effect only (red line) and the incoherence effect only (black line) for points: (a) P1LB, (b) P1RB, (c) P2M, (d) S2M

The dependencies of the dynamic response of viaduct on the wave velocity, obtained for both models of non-uniform excitation, present different trends. Especially, they differ significantly for the wave velocities located in a range from 250 to 2000 m/s that represents real values of velocities in grounds of different stiffness.

In case of the model of excitation for which the wave passage effect is taken into account the maximum principal tensile stresses at all representative points indicate a similar relation: the fall of stresses with the decrease in wave velocity. At some points the reduction reaches 50 %.

In case of the model of excitation for which the incoherence effect is considered the charts present slight reduction of stresses with the decreasing wave velocity up to 1000 m/s. However, the stresses obtained for lower velocities (500 and 250 m/s) are greater than those obtained for the uniform excitation (i.e. for *infinite* wave velocity). The maximum differences in the dynamic responses of viaduct for various wave velocities reach 60 %.

6. Conclusions

The influence of spatial variation of ground motion on the 4-span viaduct subjected to the earthquake was assessed. On the basis of the analysis the following conclusions may be formulated:

- The analysis revealed that the dynamic response of the viaduct to the shock is strongly determined by the assumed model of kinematic excitation. If the model of non-uniform kinematic excitation is applied the dynamic response may differ even up to 60% in relation to the response obtained for the model of uniform excitation.

- In case of the model of non-uniform excitation including only the wave passage effect the dynamic response of the structure persistently declines with the decrease of the wave velocity. It is a consequence of the reduction of the average amplitudes of kinematic excitation.

- In case of the model of non-uniform kinematic excitation considering only the incoherence effect, the dynamic response of the viaduct substantially increases with the decreasing wave velocity. This occurs due to the quasi-static effects which result from changes of subsoil geometry during seismic shocks. For low wave velocities the dynamic response is considerably greater than the response obtained for the model of uniform excitation. Therefore, the application of conventional model of uniform excitation may lead to underestimation and non-conservative assessment of the dynamic response of a long structure, especially for low wave velocities.

- The analysis proved that the dynamic response of the viaduct strongly depends on the wave velocity, especially for velocities up to 2000 m/s. For that reason, the correct estimation of a seismic wave velocity in the ground is a necessary condition of an accurate dynamic analysis.

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