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Plastic behavior of integral bridge, consisting of supporting steel beams and concrete superstructure, under spatially varying seismic shock

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Abstract. The paper presents the dynamic response of an integral bridge to an earthquake registered in Central Europe. The acceleration history of the shock was scaled up to peak ground accelerations predicted for this seismic zone (0.4 g). The seismic action was implemented in the form of two models of three dimensional kinematic excitation: uniform and non-uniform (spatially varying). In the uniform model the assumption was made that the motion of all supports of the bridge was identical. In the case of the spatially varying excitation the wave passage effect was taken into consideration, assuming that the seismic wave propagated along the bridge forcing subsequent supports of the bridge to repeat the same motion with a time delay dependent on the wave velocity. The structural system of the integral bridge consisted of steel girders and crossbars whereas the superstructure was made of a concrete material. To represent the inelastic behavior of the integral bridge during the earthquake, plastic models of both the steel and the concrete material were implemented. For the steel material the classical metal plasticity model with the dynamic failure model of progressive damage, provided by the ABAQUS software, was applied. For the concrete material of the superstructure the concrete damaged plasticity constitutive model was taken into consideration. It turned out that when the non-uniform excitation model was imposed, the tensile damage (cracking) and the degradation of the support zones of the concrete deck were more significant than in case of uniform excitation. The non-uniform excitation model also caused considerably higher inelastic strains of the steel girders and crossbars than the uniform model. This resulted from quasi-static effects caused by ground deformations imposed on the bridge supports during the seismic shock.

Introduction

Spatial variability of ground motion during seismic shocks is often neglected in calculations of the dynamic response of simple engineering structures, like buildings. But in case of long bridges this simplification is not recommended [1, 2]. These typically multiple-support structures are exposed to so called non-uniform kinematic excitation during earthquakes. In the non-uniform model of kinematic excitation not only time but also spatial variability of the excitation is taken into consideration. Due to the length of the structure, which is comparable with the length of the seismic wave propagating in bedrock, different ground vibrations may act on particular supports of the bridge. The influence of the non-uniform seismic excitation on the dynamic response of large-dimensional multiple-support structures was considered by many researchers, e.g. [2, 3, 4]. The authors mention that the global dynamic response of a structure to seismic shock, that results from inertia forces, may be increased by quasi-static effects occurring due to changes of bedrock geometry.

Integral bridges consist of a primary structural system made of steel and a concrete superstructure. The primary structural system is usually designed as a grid composed of steel girders connected by crossbars that both take the shape of H-beams. The upper flanges of the beams are totally integrated with the bridge deck. The integrity of the steel grid and the concrete deck significantly increases the stiffness of the upper part of the whole structure. Due to that effect, some other members of the structure may be subjected to larger deformations and strains. That causes the work beyond the elastic range or even progressive damage during earthquakes. The non-uniformity of seismic excitation may also lead to additional quasi-static effects resulting in a further increase of global dynamic response.

The paper presents the influence of integration between a steel structural system and a concrete bridge deck on the seismic response of an integral bridge. The non-uniformity of seismic excitation, typical for long bridges, was taken into consideration.

3D model of the integral bridge with steel girders and a concrete deck

The dynamic response to spatially varying seismic shock was calculated for the three-span integral bridge. Such structures are typical in Central Europe as far as geometry and mechanical data of materials are concerned. The integral bridge consisted of supporting steel girders connected by crossbars, a reinforced concrete superstructure and piers. The length of the bridge was 95 m (the central span - 45.25 m, the extreme spans - 23.5 m). The height of the piers was about 8m.

The primary structural system of the bridge consisted of two steel girders shaped as H-beams located at a distance of 11 m. The height of the H-beams differed from 1.2 m (in the middle of the span) to 2.0 m (above the supporting piers), whereas the width of the beams was 0.8 m. The thickness of the H-beams' upper flange, bottom flange and web were: 35, 60 and 22 mm, respectively. The main beams were connected by steel crossbars 0.8 m high (1.8 m over the central piers) and 15 mm thick that made a structural system working as a grid. The reinforced concrete deck of the bridge was 16 m wide and 0.25 m high (1.6 % of reinforcement was applied). The upper flanges of the girders and crossbars were integrated with the concrete superstructure. The fixed boundary conditions reflected the high rigidity of the foundation rock. Fig. 1 shows the primary structural system of the bridge: the steel grid consisting of the H-beam girders and crossbars along with the concrete piers.

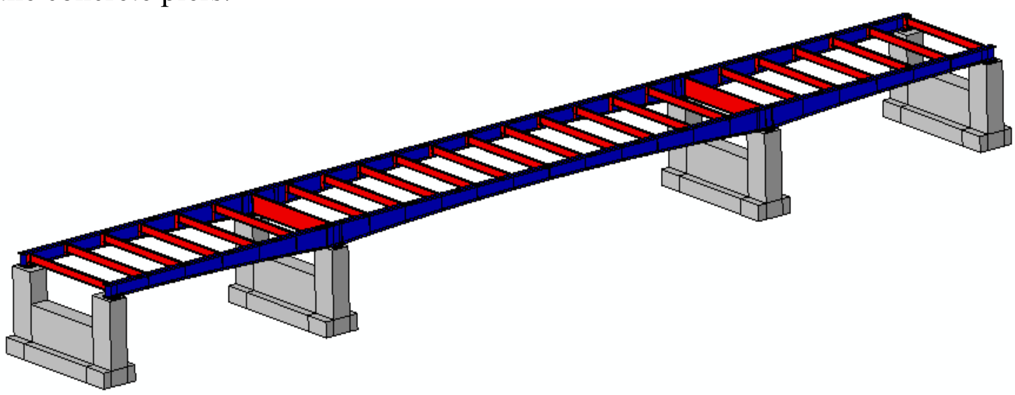

Fig. 1: Primary structural system of the bridge: the steel grid consisting of the H-beam girders and crossbars along with the concrete piers

The bridge was fitted with steel-laminated elastomeric bearings as linking elements between the deck of the bridge and the piers. The cross sectional area of the bearings was determined by the allowable pressure on the bearing support. The length and the width were assumed 1.2 and 0.9 m, respectively. The height of the bearings (100 mm) was assumed on the basis of the limitation of its horizontal stiffness and was controlled by movement requirements in the seismic area.

A 3D finite element model of the structure was developed using the ABAQUS software [5].The concrete superstructure of the bridge was discretized by circa 48000 continuum 8-node shell elements. The reinforcing rebars of a diameter φ 20 mm, located every 8 cm and 16.5 m long, were modeled in the ABAQUS software as a "fuzzy layer" [5] and implemented into the concrete deck above the piers. The primary structural system, i.e. the steel grid, was modeled with 53000 8-node continuum shell elements and 4500 3D quadratic brick 20-node finite elements above the piers. In summary, the total number of degrees of freedom was about 0.8 million. The piers were discretized with a coarse mesh of about 2000 3D linear brick 8-node finite elements, since their stiffness was incomparably larger in comparison with the rest of the structure. Practically they worked like rigid bodies. The FE mesh, densified in supports' zones, where concentration of stresses was predicted, is presented in Fig. 2.

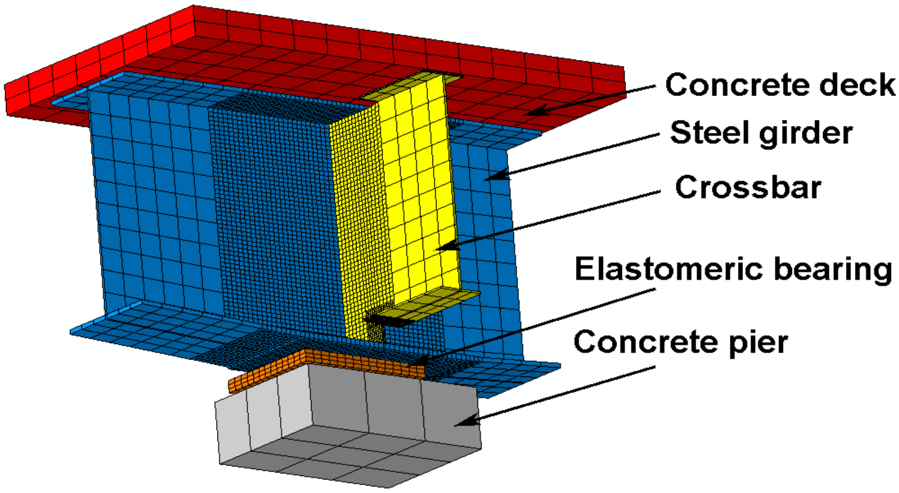

Fig. 2: Fragment of the integral bridge above the pier with FE mesh; densification of the mesh was applied for the H-beam girder and crossbar in the supports' zone

For the dynamic analysis the Rayleigh model of mass and stiffness proportional damping was applied. The damping coefficients α and β were determined for damping ratios ξ_1, ξ_2 (2.5%) referring to natural frequencies of the bridge f_1 and f_2, respectively.

Constitutive parameters for concrete and steel materials used in dynamic calculations

To represent the inelastic behavior of the bridge under a strong seismic shock, plasticity models were assumed as constitutive models of both concrete and steel materials. The plastic behavior of concrete was represented by the Concrete Damage Plasticity Model [6, 7]. The model, implemented in the ABAQUS software, consists of the combination of non-associated multi-hardening plasticity and scalar damaged elasticity to describe the irreversible damage that occurs during the fracturing process. The yield surface is controlled by two variables representing equivalent plastic strains: $\tilde{\varepsilon}_t^{pl}$ and $\tilde{\varepsilon}_c^{pl}$, associated with failure mechanisms under tension and compression loading, respectively. The degradation of the elastic stiffness is characterized by two different damage variables: d_t - for tension and d_c - for compression. These variables are functions of equivalent plastic strains. They can take values from zero – which represents undamaged material, to one – which denotes total loss of strength. The essential constitutive parameters of the Concrete Damage Plasticity Model (CDPM), used in the dynamic calculations of the bridge, were adopted according to laboratory tests [8]. The CDPM is especially recommended for calculations of concrete structures subjected to dynamic loadings, like earthquakes [4, 5, 7].

The undamaged elastic-plastic response of the steel material behavior was represented by the classical metal plasticity model. It uses Mises yield surfaces with associated plastic flow, which allow for isotropic hardening behavior. The model was used in conjunction with the model of progressive damage and failure implemented in the ABAQUS software [5]. Criteria for damage initiation resulting from both main mechanisms of fracture - ductile and shear - were taken into consideration. The ductile as well as the shear criteria are phenomenological models for predicting the onset of damage due to nucleation, growth, coalescence of voids (in case of ductile) and the onset of damage due to shear band localization (in case of shear). Two state variables, ω_D and ω_S, are responsible for the initiation of ductile and shear damage, respectively. The ductile criterion model assumes that the equivalent plastic strain at the onset of damage, $\tilde{\varepsilon}_D^{pl}$, is a function of stress triaxiality and strain rate. The shear criterion model assumes that the equivalent plastic strain at the onset of damage, $\tilde{\varepsilon}_S^{pl}$, is a function of the shear stress ratio and strain rate. The state variables, ω_D and ω_S, can take values from zero – which represents lack of plastic strains, to one – which denotes the initiation of ductile and shear damage, respectively. Once the initiation criteria are met the damage evolution of steel starts. The model assumes that damage is characterized by the progressive degradation of the material stiffness, leading to failure. Both the ductile and shear initiation criteria were used to analyze the behavior of the steel bridge grid.

The essential constitutive parameters of the steel model were assumed according to [5]. The ductile criterion was specified in terms of the plastic strain at the onset of damage as a tabular function of the stress triaxiality; the shear criterion was specified in terms of the plastic strain at the onset of damage as a tabular function of the shear stress ratio.

Seismic input data

In this study a real seismic shock of magnitude 3.2 in Richter scale, that occurred in Poland (December, 2004), was used as the kinematic excitation of the integral bridge. The registered amplitudes of acceleration were scaled up in order to obtain the maximum value of acceleration equal to 4 m/s^2 (0.4 g). This value is an appropriate peak value of ground acceleration (PGA) for the strong seismic shocks that occur in Central and Southern Europe [1]. Fig. 3 shows time history of ground accelerations in three directions.

Two models of kinematic excitations, uniform and non-uniform, were applied in the seismic analysis. In the model of non-uniform kinematic excitation the wave passage along the bridge was taken into consideration; it was assumed that the ground points in the direction of wave propagation repeat the same movement with a certain time delay dependent on the wave velocity. The wave velocity of 300 m/s was applied.

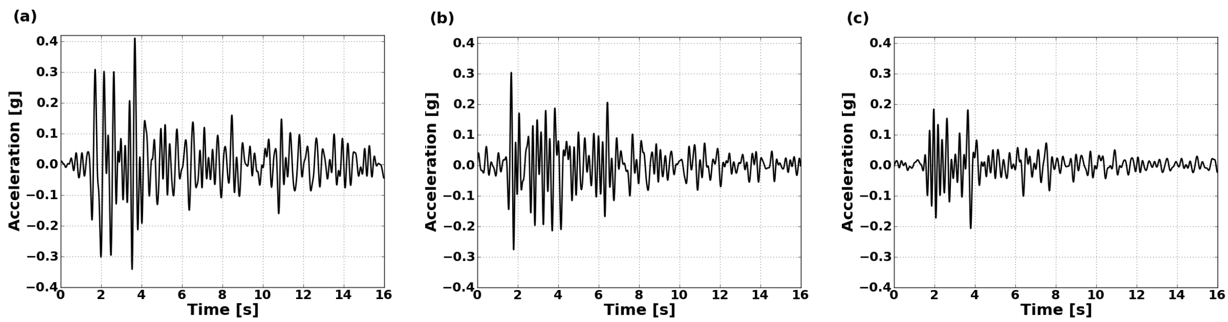

Fig. 3: Time history of ground accelerations: (a) horizontal direction X, parallel to wave propagation, (b) horizontal direction Y, perpendicular to wave propagation, (c) vertical direction

Dynamic response of the bridge to seismic shock

The results of the dynamic response of the integral bridge to uniform and non-uniform seismic excitations are shown in Figs 4-6. It could be observed that the behavior of the concrete material of the bridge deck as well as the steel material of the primary structural system exceeded the elastic range and underwent plastic effects in some zones of the structure.

The evolution of the tensile damage (cracking) of the concrete deck on the assumption of uniform and non-uniform excitations is presented in Fig. 4. For both models of excitation the progressive failure of the concrete material (a zone of non-zero tensile damage variable d_t) occurred above internal piers, where the upper part of the deck underwent significant tension. In case of uniform excitation (see Fig. 4a) a zone affected by concrete damage covered the distance of 0.6 m aside from the center of the pier. Once the amplitude of the ground vibrations had grown substantially (see Fig. 3a) the values of the tensile damage variable d_t rapidly increased at 4.17 s. The loss of the deck stiffness reached 57 % ($d_t = 0.57$). In case of non-uniform excitation much more considerable tensile damage and stiffness degradation took place (see Fig. 4b). A zone of damage covered the distance of about 5 m aside from the center of the pier and the stiffness degradation increased to 95 %. The maximal percentage of degradation appeared at 2.77 s, before the shock amplitudes reached PGA = 0.4 g. The increase of the concrete degradation occurred due to quasi-static effects.

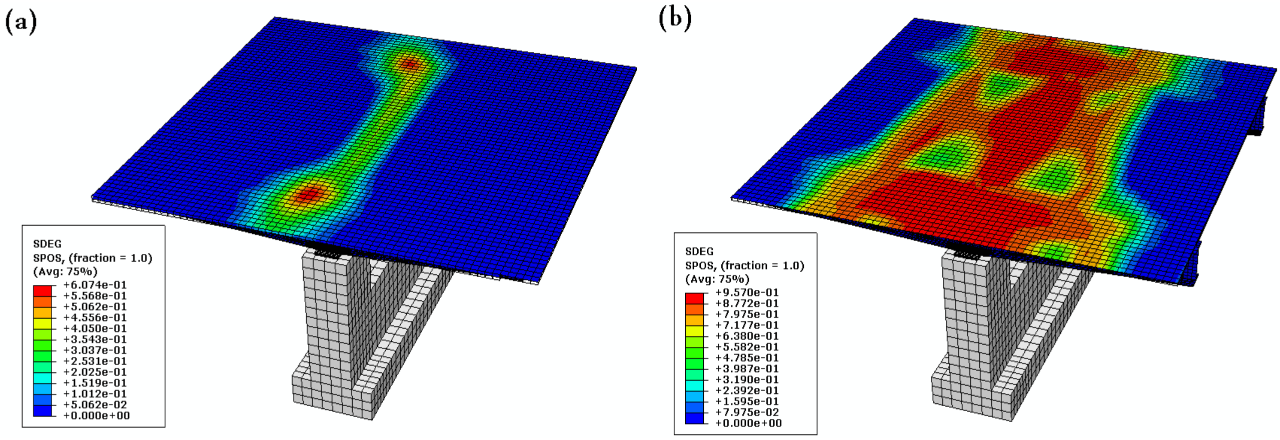

Fig. 4: Degradation of stiffness of the concrete deck of the bridge: (a) uniform excitation, (b) non-uniform excitation

The steel material of the primary structural system did not indicate any failure, but both initiation criteria, ductile and shear, were met in the steel members of the structure. The evolution of equivalent plastic strains in the steel members of the bridge on the assumption of uniform and non-uniform excitations is presented in Fig. 5a and 5b, respectively. It denotes that in both cases of excitations the behavior of steel was reported beyond the elastic range. However, in case of the uniform excitation, only a very small area was affected by plastic strains (see Fig. 5a).

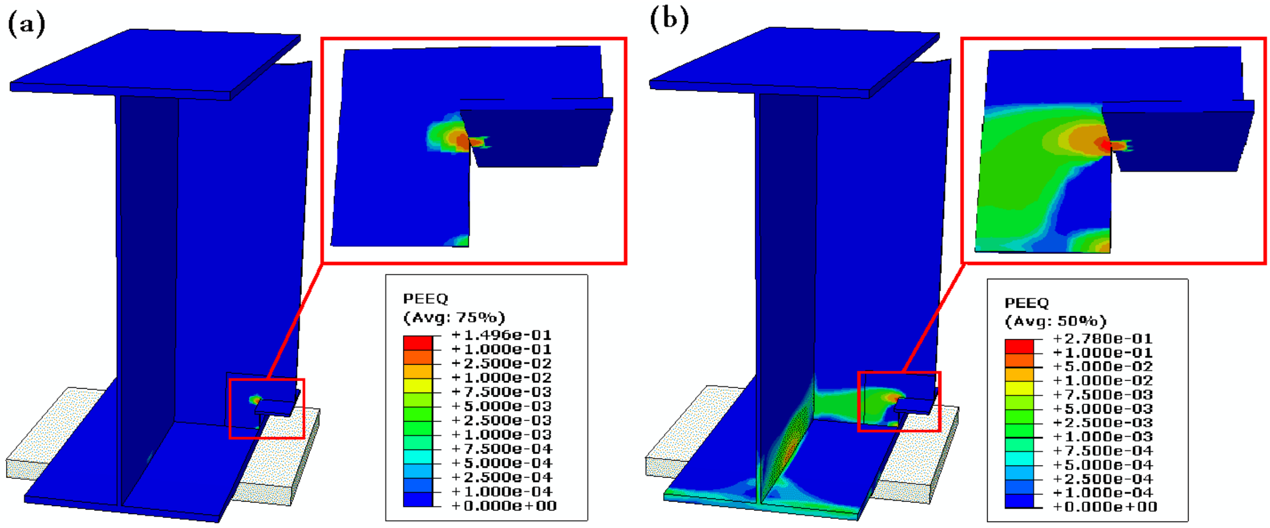

Fig. 5: Location of equivalent plastic strains in the steel structural system of the bridge: (a) uniform excitation, (b) non-uniform excitation

It is clearly visible that in case of the non-uniform excitation the area of equivalent plastic strains was incomparably larger. The equivalent plastic strains were accumulated mainly in the bottom parts of the crossbars connecting the main girders over the piers. This was due to the horizontal component of the seismic shock, perpendicular to the longitudinal axis of the bridge, which caused different motions of both steel girders in case of non-uniform excitation. The upper flanges of the girders were strongly integrated by the bridge deck, whereas the webs were connected only by the crossbars. Hence, the difference in both girders' displacements appeared mainly in their bottom part. In case of the non-uniform kinematic excitation the bottom flange of the girders also partially went plastic (see Fig. 5b). The differences in movements of subsequent piers caused a quasi-static effect: additional bending and shear of the bottom flange of the girders immediately behind the bearing.

The state variable ω_D, responsible for initiation of the ductile damage, reached 0.10 and 0.20 on the assumption of uniform and non-uniform excitation, respectively. The values of the state variable ω_S, indicating the initiation of shear damage, were larger than the values of state variable ω_D; they equaled 0.15 and 0.29, respectively. All values of state variables were less than 1, hence neither ductile nor shear damage of the primary structural system initiated in any case of excitation.

Conclusions

In the paper the results of calculations of the dynamic responses of the integral bridge, consisting of the steel primary structural system and the concrete deck, to uniform and non-uniform seismic excitation were presented. In summary, the following conclusions as well as some general remarks for engineering practice can be formulated:

- Tensile damage (cracking) and stiffness degradation of the concrete deck of the bridge occurred for both models of excitation; however in case of non-uniform excitation the loss of stiffness as well as the zones affected by tensile damage were considerably larger.
- In case of the uniform excitation the steel material of the primary structural system of the bridge remained almost elastic (excluding a small area of the crossbars), whereas in case of the non-uniform excitation the bottom parts of the girders and crossbars underwent plastic strains. However, neither ductile nor shear damage of the steel members initiated in any case.
- The integration of the primary structural system and the concrete deck significantly increased the stiffness of the upper part of the steel beams in comparison with the bottom parts. This enlarged the quasi-static effects which resulted from changes of subsoil geometry during the seismic shock.

The analysis of the integral bridge subjected to the seismic shock proved that the dynamic response could be underestimated on the assumption of uniform excitation, and both the concrete deck and the steel structural system of the bridge, may not properly assess the plastic behavior of the structure.

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