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ANALYSIS OF A POTENTIAL COLLISION OF BUILDINGS DURING EARTHQUAKE BASED ON COMPUTER SIMULATION

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Preliminary notes
In the cases when expansion joints between adjacent buildings are not sufficient, certain parts of the structure collide during earthquake. Then it comes to local damage or even destruction of parts of the building because of the impact forces of great intensity. In order to avoid collision, particular attention has to be paid to design of aseismic joints. In the paper, a parametric analysis of the potential collision of two multi-storey buildings is carried out by the use of software package SAP2000v15, with the goal to examine what occurs if the expansion joints have been designed in compliance with Serbian standards, and if yet there are collisions in some cases. The flexible base model in interaction with soil was designed. The accelerograms of real earthquakes were applied in dynamic analysis. Quantities of seismic forces, bending moments and axial forces in the case of collision, as well as necessary width of expansion joints to avoid it, were computed. Conclusions about influence of considered parameters on collision occurrence were accomplished.

Keywords: *collision of buildings, earthquake, expansion joint, joint width*

Analiza mogućeg sudara zgrada za vrijeme zemljotresa na osnovu kompjutorske simulacije

Prethodno priopćenje
U slučajevima kada dilatacijske razdjelnice između susjednih zgrada nisu dovoljne, za vrijeme zemljotresa sudaraju se pojedini dijelovi objekata. Pri tome dolazi do lokalnih oštećenja ili čak rušenja dijelova zgrada jer su sile udara velikog intenziteta. Da bi se to izbjeglo, treba posebnu pažnju posvetiti projektiranju aseizmičkih razdjelnica. U radu je prikazana parametarska analiza potencijalnog sudara dvije višespratne zgrade, primjenom programskog paketa SAP2000v15, s ciljem da se ispita što se dešava ako su razdjelnice projektirane u skladu sa standardima koji važe u Srbiji, a ipak dođe do sudara u nekim razmatranim slučajevima. Korišten je model s fleksibilnom osnovom u interakciji s tlom. U dinamičkoj su analizi korišteni akcelerogrami realnih zemljotresa. Određene su vrijednosti seizmičkih sila, momenata savijanja i normalnih sila u slučajevima kada bi došlo do sudara zgrada, kao i potrebna širina dilatacijskih razdjelnica da bi se izbjegao sudar, a izvedeni su i zaključci o utjecaju pojedinih parametara na pojavu sudara.

Ključne riječi: *dilatacijska razdjelnica, sudar zgrada, širina razdjelnica, zemljotres*

1 Introduction

Expansion joints between the adjacent structures in cities are important for two reasons: they facilitate unimpaired expansion of buildings due to seasonal temperature variations and provide a free space where the buildings can oscillate during earthquakes. Based on the requirements for the design of structures in seismic areas, aseismic joints are designed between the adjacent buildings in order to avoid a collision of buildings during earthquakes. In some cases, despite the respect of those regulations, certain parts of the structure collide if separation joints are not sufficient, as it was demonstrated during numerous earthquakes. Then it comes to local damage or even destruction of some parts of the buildings because the impact forces are of great intensity. In order to avoid this, special attention should be paid to the design of aseismic joints, which is the subject of this paper. This issue is the most topical in densely built-up city districts.

The largest number of studies in this field deals with the collisions of two adjacent buildings. Anagnostopoulos (1988) was the first to simulate by computer the earthquake induced collision of adjacent building. Maison and Kasai (1990, 1992) simulated the collision of the buildings of different heights using the SUPER-ETABS software. Mouzakis and Papadrakakis (2004) modelled a collision between a rigid and a flexible building using DRAIN-TABS. The DRAIN-2DX software was used by Jeng and Tzeng (2000) [5] for researching the fire risk in Taipei City, as well as Karayannis and Fawata (2005) for research of the buildings whose floor structures are at different levels. Jankowski (2005) [4] improved the earthquake collision simulation introducing a non-linear highly elastic model. In the recent years, many authors have been researching the collision of spatial models, as well as a number of terraced buildings and the phenomenon of considerably higher damage at the end buildings in respect to the ones positioned in the middle (End building pounding).

The goal of this paper is to investigate, by the use of dynamic analysis, whether the separation joints which are designed according to Serbian regulations [25] are sufficient for avoiding a collision of adjacent buildings during a possible earthquake, and what is the increase of cross section forces in critical structural elements in comparison with the design according to the equivalent static load method. For this purpose, a parametric study was conducted using the software package SAP2000v15.

2 Examples of collision of buildings during earthquakes that occurred in the past

Figure 1 Olive View Hospital, San Fernando 1971, prior and after the collapse

If the adjacent buildings have different dynamic characteristics, their oscillations due to the effects of an earthquake will not be in the same phase. There are frequent cases of founding several lamellas of one housing block on a continuous foundation floor, where even though they have the same height and same dynamic characteristics, due to the foundation floor and soil interaction there occurs the out of phase oscillation of lamellas. If the expansion separation joints between the buildings are insufficiently large, that is, if they are smaller than the sum of amplitude oscillations of adjacent buildings, certain parts of buildings may collide. For instance, there is the collapse of the staircase tower of the "Olive View Hospital" in California due to collision with the main hospital building during the San Fernando earthquake 1971, Fig. 1.

Figure 2 Earthquake in Mexico City, September 19, 1985

There were no earthquakes like the Mexico City one, with as many examples of mutual collision of structures. The Mexico City earthquake of September 19, 1985, had the magnitude of M=8,1, and the most severe effects were felt in Mexico City which was 350 km from the epicentre. The cause of the damage to the buildings was the too small separation joints between the structures (Fig. 2). The damage was in some cases only local, and in some the collision caused collapse of the entire upper part of the structure above the damage points, especially in the cases when the structures of different heights collided. Collapse of the upper parts of buildings is one of the specific characteristics of the Mexico City earthquake. Not only collisions of adjacent buildings were the cause of this, but also the abrupt changes of stiffness and bearing capacity of the structure along its height.

From these examples it can be concluded that when it came to the collision of buildings, damage necessarily occurred, and often destruction of certain structural elements, which in turn led to the destruction of an entire floor (Fig. 2), or to the failure of entire structure (Fig. 1). If the buildings did not collide, it is assumed that damage would probably occur but they would not be demolished.

3 Formulation of the building collision issue

Conditional equations that define the behaviour of spatial structures under static and dynamic loads are derived under the assumption that the floor is infinitely in-plane rigid. That is why displacement of each floor in the horizontal plane is defined by three parameters: the displacement components u_j and v_j in direction of the x and the y axis, respectively, as well as rotation θ_j around a vertical axis z (Fig. 3) [15].

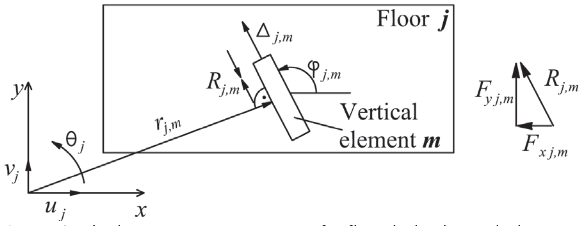

Figure 3 Displacement components of a floor in horizontal plane and relation between vertical and floor element

Displacement vector $\boldsymbol{u}$ of the whole structure is defined in terms of $3N$ components:

$$\boldsymbol{u}^{\mathrm{T}} = \left[u_1\, v_1\, \theta_1 \cdots u_j\, v_j\, \theta_j \cdots u_N\, v_N\, \theta_N\right], \tag{1}$$

where N is the number of floors in considered structure.

If the position of the m-th vertical element is determined by angle $\varphi_{j,m}$ and normal distance $r_{j,m}$ from the origin, then the displacement of the vertical element $\Delta_{j,m}$ in its plane, and the floor level j is given as:

$$\Delta_{j,m} = u_j \cos\varphi_{j,m} + v_j \sin\varphi_{j,m} + \theta_j r_{j,m}. \tag{2}$$

When all the floors are considered, the relation between $\boldsymbol{\Delta}_m$ and $\boldsymbol{u}$ is:

$$\boldsymbol{\Delta}_m = \boldsymbol{a}_m \boldsymbol{u}, \tag{3}$$

or in the developed form:

$$\begin{bmatrix} \Delta_{1,m} \\ \vdots \\ \Delta_{j,m} \\ \vdots \\ \Delta_{N,m} \end{bmatrix} = \underbrace{\begin{bmatrix} \cos\varphi_{1,m} & \sin\varphi_{1,m} & r_{1,m} & \cdots & 0 & 0 & 0 & & 0 & 0 & 0 \\ \vdots & \vdots & \vdots & & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ 0 & 0 & 0 & \cdots & \cos\varphi_{j,m} & \sin\varphi_{j,m} & r_{j,m} & & 0 & 0 & 0 \\ \vdots & \vdots & \vdots & & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ 0 & 0 & 0 & \cdots & 0 & 0 & 0 & & \cos\varphi_{N,m} & \sin\varphi_{N,m} & r_{N,m} \end{bmatrix}}_{a_m} \cdot \left[u_1\ \ v_1\ \ \theta_1\ \cdots\ u_j\ \ v_j\ \ \theta_j\ \cdots\ u_N\ \ v_N\ \ \theta_N\right]^{\mathrm{T}}, \tag{4}$$

where matrix $\boldsymbol{a}_m$ is called a transformation matrix.

Relation between vertical and floor elements can be, as shown in Fig. 3, displayed via horizontal support whose direction in the plane of the floor coincides with the direction of the vertical element. The reaction of this support is marked with $\boldsymbol{R}_{j,m}$, and its components in the directions of axes and the moment about the origin are defined by a vector $\boldsymbol{F}_{j,m}$:

$$\boldsymbol{F}_{j,m} = \begin{bmatrix} F_{xj,m} \\ F_{yj,m} \\ M_{oj,m} \end{bmatrix} = \begin{bmatrix} \cos\varphi_{j,m} \\ \sin\varphi_{j,m} \\ r_{j,m} \end{bmatrix} \cdot \boldsymbol{R}_{j,m}. \tag{5}$$

When all the floors are considered, this relation reads:

$$\boldsymbol{F}_m = \boldsymbol{a}_m^{\mathrm{T}} \boldsymbol{R}_m, \tag{6}$$

where the matrix $\boldsymbol{a}_m^{\mathrm{T}}$ is transposed matrix $\boldsymbol{a}_m$.

Stiffness matrix $\boldsymbol{Z}_m$ of vertical element m is given by the matrix equation:

$$\boldsymbol{R}_m = \boldsymbol{Z}_m \boldsymbol{\Delta}_m, \tag{7}$$

where $\boldsymbol{\Delta}_m$ and $\boldsymbol{R}_m$ are vectors whose coordinates represent the horizontal displacements of vertical elements and the force vector in the floor level. If we use Eqs. (7) and (3), Eq. (6) looks like:

$$\boldsymbol{F}_m = \boldsymbol{a}_m^{\mathrm{T}} \boldsymbol{Z}_m \boldsymbol{a}_m \boldsymbol{u}. \tag{8}$$

Now the reactions of the connections of all the vertical elements can be written as:

$$\boldsymbol{F} = \sum_{m=1}^{M} \boldsymbol{F}_m, \tag{9}$$

where M is the total number of vertical elements. The equilibrium condition of the forces $\boldsymbol{P}$ and corresponding reactive forces is:

$$\boldsymbol{F} = \sum_{m=1}^{M} \boldsymbol{F}_m = \boldsymbol{P}, \tag{10}$$

i.e.

$$\sum_{m=1}^{M} \boldsymbol{a}_m^{\mathrm{T}} \boldsymbol{Z}_m \boldsymbol{a}_m \boldsymbol{u} = \boldsymbol{P}. \tag{11}$$

Eq. (11) can be written as

$$\boldsymbol{K}\boldsymbol{u} = \boldsymbol{P}, \tag{12}$$

where:

$$\boldsymbol{K} = \sum_{m=1}^{M} \boldsymbol{a}_m^{\mathrm{T}} \boldsymbol{Z}_m \boldsymbol{a}_m, \tag{13}$$

is the stiffness matrix of the whole structure, which in this case establishes a relation between the external load and unknown displacements $\boldsymbol{u}$ of the floor structures.

In the case of usual buildings (less than 15 ÷ 20 stories) collision due to earthquake action is more dangerous reality considering the structure as in-plane system than as space one. Therefore, in most such cases, it is justified to apply the 2D analysis rather than the 3D analysis. However, for very tall buildings a real danger is greater if they are considered as spatial systems, because the displacement of such buildings in the wind direction, as well as perpendicular to the wind direction, is greater than due to the seismic forces. As the torsional stiffness of these structures is usually small and their vertical components suffer heavy loads, there is a potential collision of such buildings. So the potential collision of tall buildings is more likely to be caused by wind than by earthquake, and because of that the analysis of the wind action should be performed in the areas where this impact is significant.

Probably every earthquake that occurs in a larger city induces the appearance of collision of the parts of adjacent structures, but the question is, what is the dominant reason for the damage, or as worse, destruction. Usually there are several different factors that can be combined one with the other, and in the case of total destruction it is very difficult to accurately determine the cause: primary or complementary.

Two buildings are taken into consideration, and the assumption of the two-dimensional behaviour of both buildings is considered justified. This means that the distribution of mass is symmetrical and that vertical bearing elements of both buildings are distributed symmetrically in orthogonal directions, so the centres of mass and centres of rigidity do not significantly vary in individual floors. Also adopted are the usual assumptions in analysis of the horizontal forces in buildings: 1) floors are rigid in their planes; 2) the mass of the building is concentrated on certain floors; 3) vertical bearing elements are the in plane girders. Therefore, isolated columns, frame girders or bearing walls are treated as internal connections which limit the motion of floors in horizontal directions (floor rigidity). Floor rigidity is defined as a horizontal force at the level of the floor which is required so that the observed floor would move relative to the lower floor for a unit amount. If the floor height between two floors $i-1$ and i equals h_i, if the sum of the moments of inertia of all vertical elements between those two floors equals I_{d} then the corresponding story stiffness is:

$$K_i \geq \frac{12EI_{\mathrm{d}}}{h_i^3}, \tag{14}$$

where E is the modulus of elasticity of vertical elements.

If the time variation of horizontal ground displacements at the level of the foundation during an earthquake is $u_{\mathrm{g}}(t)$, the acceleration of the ground is $\ddot{u}_{\mathrm{g}}(t)$. The floor acceleration is equal to the sum of acceleration created by the earthquake and relative acceleration.

$$a_i = \ddot{u}_{\mathrm{g}} + \ddot{u}_i. \tag{15}$$

Equations of motion of the floors, taking into account the damping, are presented in the matrix form:

Equations of motion of the floors

$$\boldsymbol{M}\ddot{\boldsymbol{u}} + \boldsymbol{C}\dot{\boldsymbol{u}} + \boldsymbol{K}\boldsymbol{u} = -\boldsymbol{M}\boldsymbol{e}\ddot{\boldsymbol{u}}_{\mathrm{g}}(t), \tag{16}$$

where $\boldsymbol{M}$ is diagonal mass matrix and $\boldsymbol{K}$ three-diagonal rigidity matrix,

$$\boldsymbol{M} = \begin{bmatrix} m_1 & 0 & 0 & 0 \\ 0 & m_2 & 0 & 0 \\ 0 & 0 & \ddots & 0 \\ 0 & 0 & 0 & m_n \end{bmatrix}, \tag{17}$$

$$\boldsymbol{K}=\begin{bmatrix}(k_1+k_2) & -k_2 & 0 & 0 & 0 & 0\\ -k_2 & (k_2+k_3) & -k_3 & 0 & 0 & 0\\ \vdots & \vdots & \ddots & \vdots & \vdots & \vdots\\ 0 & 0 & 0 & -k_{n-1} & (k_{n-1}+k_n) & -k_n\\ 0 & 0 & 0 & 0 & -k_n & k_n\end{bmatrix}, \quad (18)$$

while $\boldsymbol{u}$ is the vector of generalized coordinates u_i and $\boldsymbol{e}$ is the vector of order n whose elements are all equal to one.

$$\boldsymbol{u}=\begin{bmatrix}u_1\\ u_2\\ \vdots\\ u_n\end{bmatrix} \text{ and } \boldsymbol{e}=\begin{bmatrix}1\\ 1\\ \vdots\\ 1\end{bmatrix}. \quad (19)$$

The matrix $\boldsymbol{C}$ is adopted in principle, in the form of proportional matrix $\boldsymbol{C} = \alpha\boldsymbol{M} + \beta\boldsymbol{K}$ where coefficients α and β are obtained from the expression:

$$\alpha = 2\xi_i \frac{\omega_i\omega_j}{\omega_i-\omega_j} \text{ and } \beta = \frac{2\xi_i}{\omega_i-\omega_j}. \quad (20)$$

Here ω_i and ω_j are natural frequencies of two natural forms, while ξ_i and ξ_j are adopted relative modal damping. Normally, the same damping is introduced, i.e. $\xi_i = \xi_j$, which is analogous to the system of one degree of freedom.

In Fig. 4 are presented two adjacent buildings at a mutual distance d. It is considered that the buildings are exposed to the same seismic excitation which is manifested in the form of the known time function of soil acceleration $\ddot{u}_g(t)$.

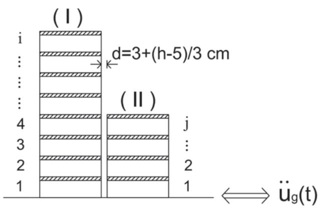

Figure 4 Width of the separation joint according to the Code [25]

Differential equations of building motion are given by the Eq. (16) as:

- for building (I)...n_1 equations in the form

$$\boldsymbol{M}_1\ddot{\boldsymbol{u}}_1 + \boldsymbol{C}_1\dot{\boldsymbol{u}}_1 + \boldsymbol{K}_1\boldsymbol{u}_1 = -\boldsymbol{M}_1\boldsymbol{e}_1\ddot{u}_g(t) = \boldsymbol{f}_1, \quad (21)$$

- for building (II)...n_2 equations in the form

$$\boldsymbol{M}_2\ddot{\boldsymbol{u}}_2 + \boldsymbol{C}_2\dot{\boldsymbol{u}}_2 + \boldsymbol{K}_2\boldsymbol{u}_2 = -\boldsymbol{M}_2\boldsymbol{e}_2\ddot{u}_g(t) = \boldsymbol{f}_2. \quad (22)$$

Both systems of equations can be presented as a system of equations of the order $n=n_1+n_2$:

$$\boldsymbol{M}\ddot{\boldsymbol{u}} + \boldsymbol{C}\dot{\boldsymbol{u}} + \boldsymbol{K}\boldsymbol{u} = \boldsymbol{f}(t), \quad (23)$$

where:

$$\boldsymbol{M} = \begin{bmatrix}\boldsymbol{M}_1 & 0\\ 0 & \boldsymbol{M}_2\end{bmatrix}, \boldsymbol{C} = \begin{bmatrix}\boldsymbol{C}_1 & 0\\ 0 & \boldsymbol{C}_2\end{bmatrix}, \boldsymbol{K} = \begin{bmatrix}\boldsymbol{K}_1 & 0\\ 0 & \boldsymbol{K}_2\end{bmatrix}, \quad (24)$$

$$\boldsymbol{u} = \begin{bmatrix}\boldsymbol{u}_1\\ \boldsymbol{u}_2\end{bmatrix}, \boldsymbol{f} = \begin{bmatrix}\boldsymbol{f}_1\\ \boldsymbol{f}_2\end{bmatrix}. \quad (25)$$

In principle, the buildings oscillate mutually independently. However, if the relative dynamical parameters (mass, rigidity, frequency) are considerably different for the adjacent buildings, and if at that the width of the separation joint d between them is relatively small, collision of certain floors may occur. The conditions of creation of contact at the same height can be presented by the relation:

$$u_j^{(\mathrm{I})} - u_j^{(\mathrm{II})} = d, \quad (26)$$

where $u_j^{(\mathrm{I})}$ and $u_j^{(\mathrm{II})}$ are displacement of the floors number j of the buildings (I) and (II). Regarding that the response of the system for the given accelerogram as a forced load is sought for, the solving of the equations is performed by direct numerical integration.

4 Modelling and analysis of a potential collision of two buildings

Displacements and deformation of buildings during earthquakes depend on the behaviour of the entire system consisting of: the frame structure, foundation structure and geological environment where the building is founded. The classical mathematical model for analysis of interaction of multi-storey frames at seismic action takes into account that there is the total fixation in the support nodes. Many authors have contributed to the improvement of such soil-structure interaction model, have studied an impact of interaction on the dynamic response of a system, and have proposed various analytical and numerical solutions [16].

In this paper the parametric analysis has been conducted implementing the software package SAP2000v15 and the numerical integration applying Newmark's method for the given accelerograms with 5 % of damping as a standard initial value. A Newmark constant average acceleration integration scheme (β=0,25) is implemented in the dynamic time-history analyses. The non-linear effects are introduced applying geometrical non-linearity via P- Delta effect, while the material non-linearity is introduced by introduction of plastic hinges for the moment and normal forces at the beginning and the end of members according to the FEMA356 [22].

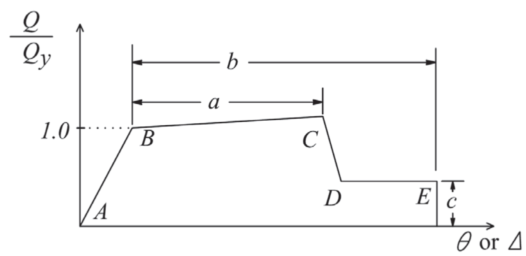

Figure 5 Load – deformation relationship in plastic hinges

In the design was used the model with flexible base and soil interaction (flexible base model). The foundation structure and multi-storey frames have been modelled applying linear structural elements, and the soil applying surface structural elements. The soil is modelled as an elastic half-space by 2D shell elements to a depth of 10 m below the level of funding. Adopted characteristics of the soil are: E=10^5 kN/m^2, ρ=1,8 t/m^3 and ν=0,3. Modelling of soil by springs, i.e. the use of Winkler's soil model has significant limitations and therefore it is not chosen in this analysis. In the case of Winkler's soil model the springs are mutually independent, and consequently the soil deformation below one building does not affect the deformation of the soil below the adjacent building, what is not real behaviour.

Herein, interaction of soil and foundation structure is modelled by gap elements [16]. Due to different stresses in the soil at the contact of the smaller and the taller building foundations, it is realistic for the soil to separate from the foundation slab in some instant of time. This is overcome by nonlinear gap elements which enables modelling of uplift phenomena in vertical direction. This is especially important in the case of collision of narrow but tall buildings. The opening of gap element is adopted to be zero, while spring stiffness is equivalent to the soil stiffness in the direction of the z-axis [20]. Also gap elements are used for modelling of the mutual connection of two adjacent buildings, as only mutual pressure action is permitted in the analysis [16]. The spring stiffness of these gap elements is adopted as k=2000 kN/m.

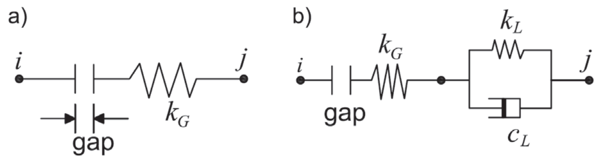

Figure 6 Gap element (a), gap element in combination with Kelvin-Voigt element (b)

The purpose of the gap element (Fig. 6a) was to transfer only the pressure force, only when the aperture is closed. The force-deformation dependency is given

$$f_G = \begin{cases} -k_G\left[(u_i - u_j) - open\right] & \text{for } (u_i - u_j) < open \\ 0 & \text{for } (u_i - u_j) > open \end{cases}, \quad (27)$$

where k_G is the stiffness of the element, u_i and u_j are node displacement and *open* is the separation joint width. Dissipation of energy during collision can be taken into consideration by introducing damping.

The model can be improved combining the gap element with viscous damper and elastic spring (Kelvin-Voigt element), Fig. 6b. Force-deformation dependency is expressed:

$$f = k_L d_L + c_L \dot{d}_L, \quad (28)$$

where k_L is the linear rigidity of the spring, c_L is damping coefficient and d_L is deformation in the direction of the element. The stiffness and damping of contact element is proposed by Jankowski [4]. The stiffness k_G must be a hundred times higher than stiffness k_L in order to render the gap element absolutely rigid after closing [4]. Kelvin-Voigt's element is not used in this study.

The following parameters have been varied in the analysis: the bearing structural system, combinations of heights of adjacent buildings, width of separation joint and type of accelerogram. In the cases where the structures had the equal number of floors, the adjacent lamellas are modelled as having different structural systems or common foundation floor, because, in the opposite case the oscillation could be of the same phase, so there would be no need to analyse the case of separate foundations.

The buildings are modelled in the following structural systems: skeletal, mixed-type and panel. The number of floors is chosen: 3-storey, 6-storey, 9-storey and 12-storey. Different combinations of structural systems and height of neighbouring buildings are considered, and those shown in Fig.7(a ÷f) are separated as typical cases.

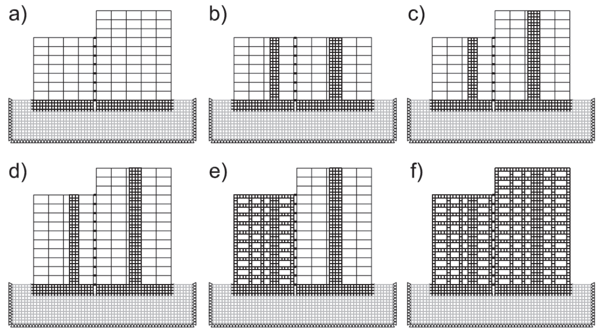

Figure 7 Typical cases of considered models

In all the models, one building is the four-bay frame and the other is five-bay frame, with the spans of 5 m, floor height 3 m and accompanying load of 50 kN/m on the beams of the frames. The modulus of elasticity was adopted as E=30 GN/m^2 for all the structural elements. For all the buildings are selected rectangular cross-sections of the basic elements, but of different dimensions along the building height, as shown in Tab. 1.

Table 1 Dimensions of structural elements in considered models.

Part of a building along its height	Column dimensions / cm	Beam dimensions / cm	Shear wall dimensions / cm
ground floor and 1st to 3rd story	60×80	50×60	300×30
from 4th to 6th story	60×60	40×60	300×20
from 7th to 9th story	40×60	40×50	200×15
from 10th to 12th story	40×40	30×50	200×15

According to [25] the aseismic joints are designed for irregular floor planes of high-rise structures and for those with irregular heights. The separation joint width is 3 cm. For each 3 m of structural height (of the structures taller than 5 m) the separation joint width is increased for 1 cm.

For the high-rise buildings taller than 15 m as well as for the lower flexible structures, such as skeletal structures without stiffeners, the separation joint width is specified by the design, so as not to be smaller than the doubled value of deformation of adjacent structural segments, and must not be smaller than the previously mentioned values. The separation joint width must be larger than the maximum amplitudes of separated structures $d = \Delta\alpha + \Delta\beta$, where displacements $\Delta\alpha$ and $\Delta\beta$ should include elastic and plastic displacements of the structure, and interaction with the soil.

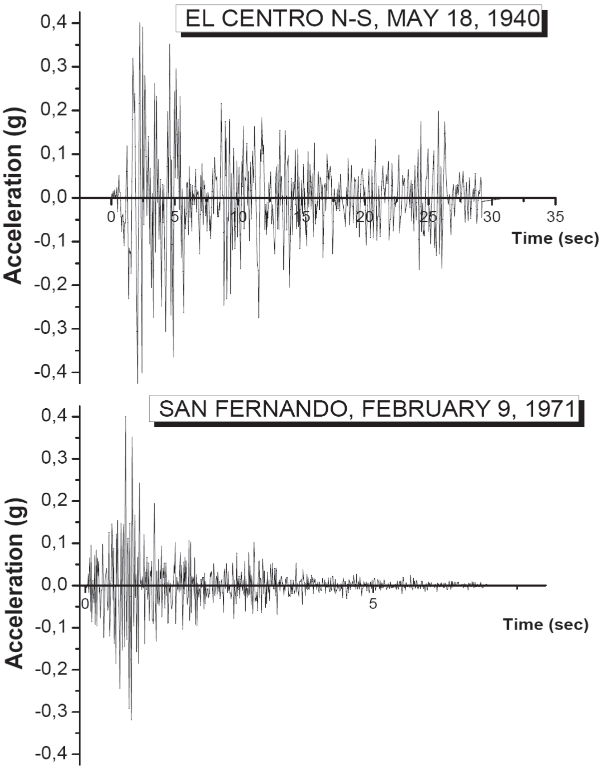

Figure 8 Earthquake accelerograms used in the simulation of collision

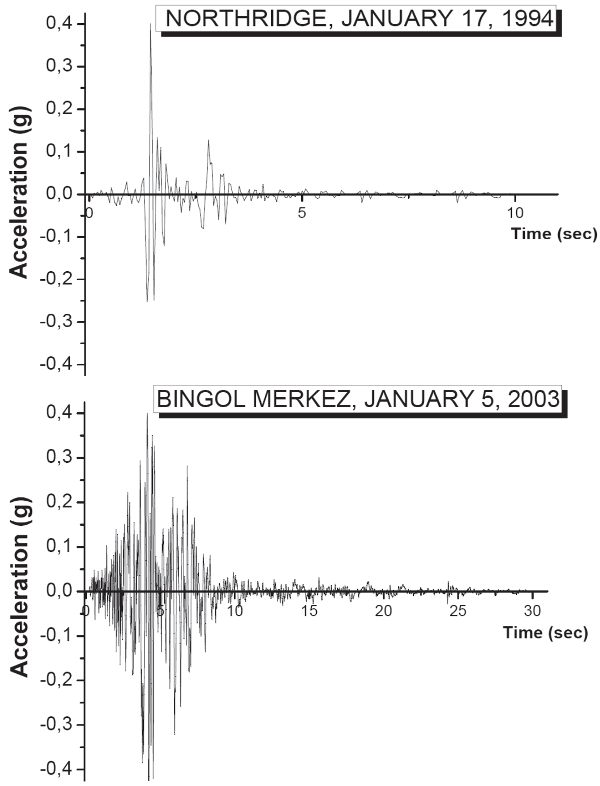

Figure 9 Earthquake accelerograms used in the simulation of collision

If the displacement of the structures is calculated on the basis of the action of equivalent static load, then the displacement obtained in this way should be multiplied by the ductility factor, i.e. $d \geq C_\mathrm{d} \cdot \mathit{\Delta}_\mathrm{el}$, but our national regulations [25] do not provide values for the ductility factor C_d. This factor is mentioned in some foreign regulations and depends on the characteristic of the structure and applied material. Recommendations of the Council of Applied Technology USA are as follows: for reinforced concrete skeletal structure without stiffening C_d=2 ÷ 6; for reinforced concrete skeletal structure with reinforced concrete walls C_d=5; or structures with bearing walls of reinforced concrete C_d=4 [24].

Seismic separation joints must be sealed, so as to prevent atmospheric water penetration or for esthetic reasons, providing that the sealing must be either on the surface or performed with foams which would not resist the structure oscillations. Therefore, the material used to seal or conceal the separation joint must be either very flexible, so as not to resist the structure, or brittle, so as to fall off as soon as an earthquake has begun [8].

The width of separation joints in the models is calculated according to the Regulations [20], and in dynamic analysis is varied from 3 cm to 15 cm.

Analysis was performed for all models due to seismic load described by the following real earthquake accelerograms: El Centro Earthquake May 18, 1940; San Fernando, California, Earthquake February 9, 1971; Northridge, Pacoima January 17, 1994; Bingol Markez Yinderlik ve Iskan Mudurlugu January 15, 2003. All accelerograms were scaled to the maximum value of acceleration a_max=0,4g.

5 Discussion of parametric analysis results

The diagrams in Figs. 10 ÷ 15 present the effect of collision of typical adjacent buildings whose models are depicted in Fig. 7(a ÷ f). The graphs represent the dependence between the width of separation joint and the maximum force of collision as well as the number of collisions which took place during accelerogram duration.

No clear trend of dependence between the width of the separation joint, force of the collision and number of the collisions can be determined. Each case is individual and should be treated separately. There is only one rule - with the increase of the separation joint width, the number of collisions decreases; but the force of collisions does not necessarily decrease. If it is adopted in the model that there is no separation joint, or if it is very small, then the collisions of lower floors occur, which matches the results obtained by other authors. The width of the separation joint according to the regulations [25] is completely satisfactory only for the certain types of earthquake San Fernando and Northridge.

Frequency characteristic of the accelerogram is a dominant factor having effects on the onset of collisions, as can be seen on the example of the recording El centro and Bingol Markez where the width is not meeting the requirements of the regulations. This is not the case only in insufficiently rigid structures in the zones of high seismic intensity, for which the condition of limitation of absolute displacement of the top of the structure and relative displacement of adjacent floors is very problematic, but also in the case of considerably more rigid structures with mixed-type structural systems.

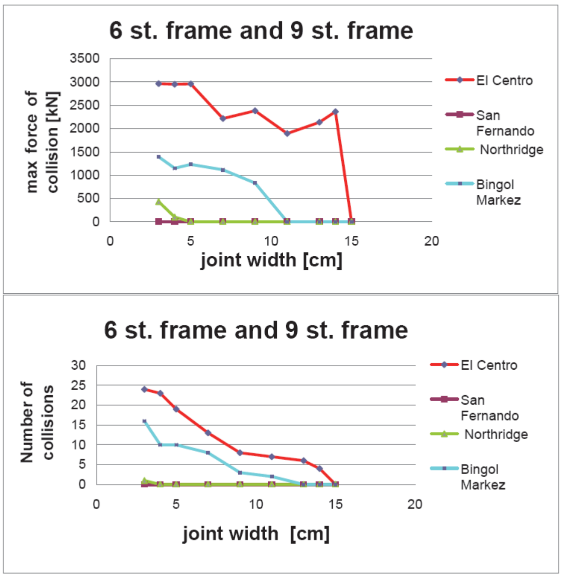

Figure 10 Effect of the collision of 6-story and 9-story skeletal buildings

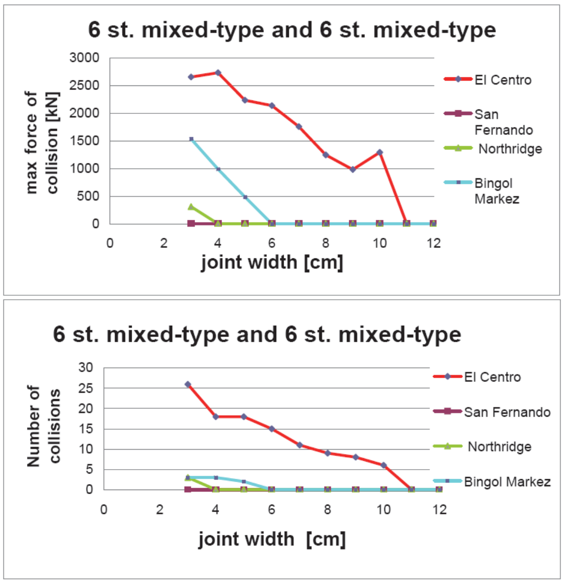

Figure 11 Effect of the collision of two 6-story buildings in mixed-type structural system on the same foundation floor

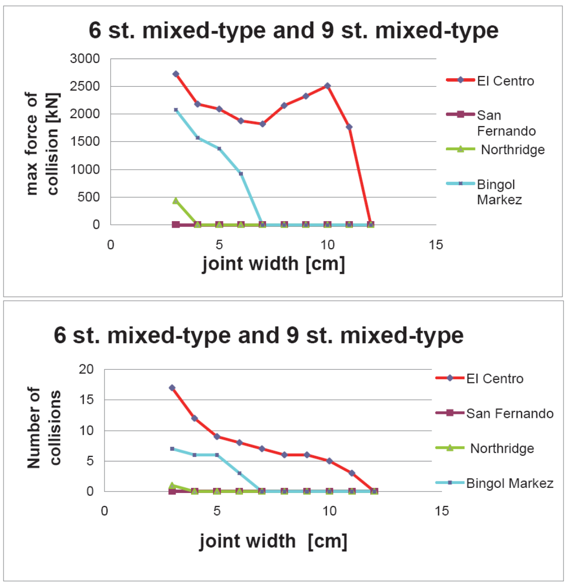

Figure 12 Effect of the collision of 6-story and 9-story buildings in mixed-type structural system

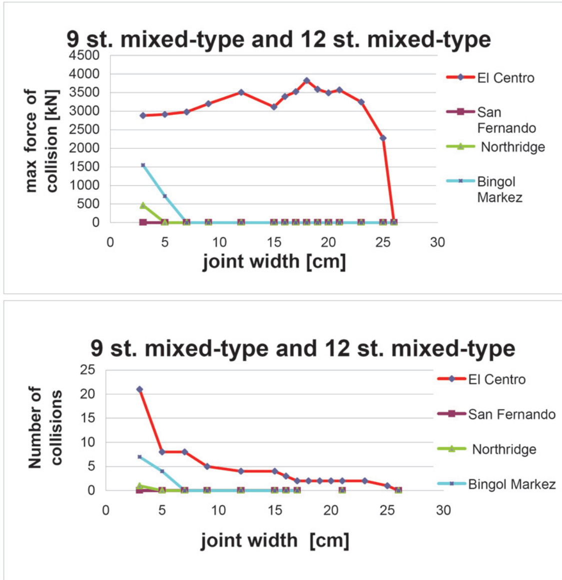

Figure 13 Effect of the collision of 9-story and 12-story buildings in mixed-type structural system

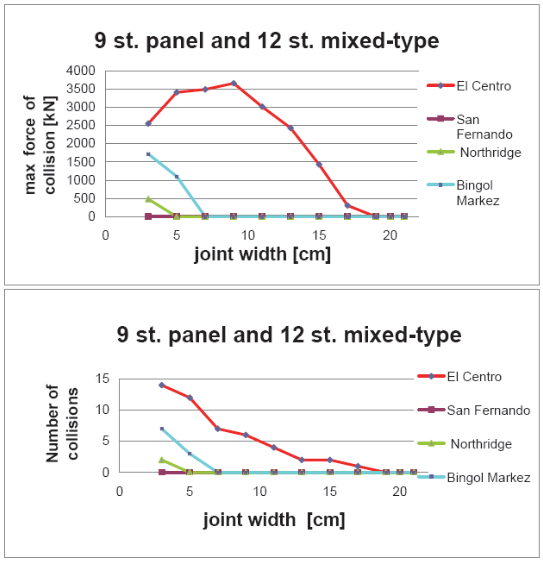

Figure 14 Effect of the collision of 9-story panel building and 12-story building in mixed structural system

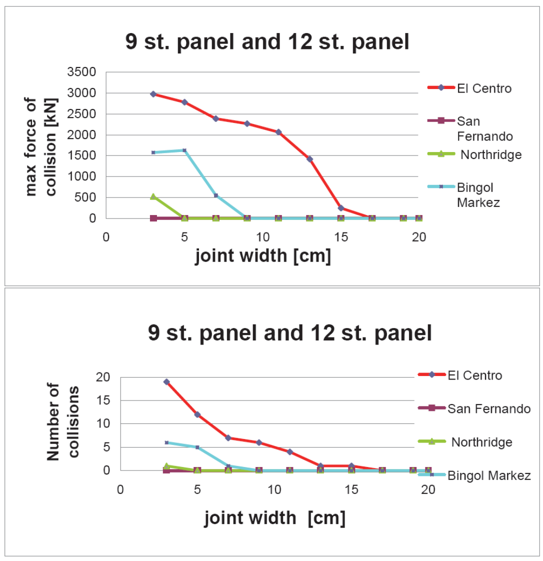

Figure 15 Effect of the collision of 9-story and 12-story buildings in panel structural system

From the presented examples, it can be concluded that for the unfavourable response of the structure, not only the width of the separation joint and amount of maximum acceleration of the soil are important, but also the predominant period of soil oscillations, as well as frequency characteristics of an earthquake. From the examples 1 and 3 in Tab. 2 it can be seen: that for the same number of floors and the same accelerogram in the case of collision of rigid structures (in the mixed-type system) there is a double increase of impact of the collision force in the columns, than in the case of purely skeletal structures.

In Tab. 2 are given comparative results of bending moments and normal forces in the columns of the critical floor of the taller building. Internal forces are calculated according to: 1) the Code which is in force in Serbia [25] adopting the second category of soil, IX seismic zone, the seismic coefficient of 0,1 and the second category of a building; 2) EC8 adopting soil category B, the third category of facility significance (γ = 1,0), type of spectra 1, the behaviour factor 2, the ratio a_g/g = 0,4 and viscous damping ζ = 5 %; 3) dynamic analysis of the buildings collision for the adopted joint width according to EC8. Nonlinear dynamic analysis has been carried out by direct integration for the time step of 0,02 s, with all accelerograms scaled to 0,4g, adopting damping of 5 %, and including the geometric nonlinearity by P-Delta option. Minimum size separation joint required by [20] does not satisfy the first, third and fourth case, which results in a significant increase of the internal forces in the columns of critical floor of the taller building. It is evident that in the cases where the collisions occurred, the values of bending moments obtained by the dynamic analysis exceed multiple times the values obtained by the equivalent static load method according to regulations [20] and by the multimodal analysis according to EC8.

Table 2 Systematization of a part of results of parametric analysis

Case (Fig.7)	Serbian Regulation		EC8		Dynamic analysis		Influences in the columns of the critical floor according to the regulations [20] , EC8 and dynamic analysis									
							31/81	EC8	Dyna.	31/81.	EC8	31/81	EC8	Dyna.	31/81.	EC8
	$d_{requ.}$ / cm	$\sum S_{calc.}$ / kN	$d_{requ.}$ / cm	$\sum S_{calc.}$ / kN	$d_{requ.}$ / cm	$\sum S_{calc.}$ / kN	M / kN·m	M / kN·m	M / kN·m	Δ / %	Δ / %	N / kN	N / kN	N / kN	Δ / %	Δ / %
a) 6-s.s.+ 9-s.s.	9	910	9	2779	15	2473	93	262	694	646	165	-109	-268	-735	574	174
b) 6-m.s.+ 6-m.s.	9	710	9	2543	11	2750	45	114	251	458	120	-76	-42	-139	83	231
c) 6-m.s.+ 9-m.s.	9	1338	9	3894	10	2779	48	132	377	685	186	-82	-241	-270	229	12
d) 9-m.s.+ 12-m.s.	12	1683	12	4613	26	3822	58	123	418	621	240	-458	-218	-351	-23	61
e) 9-p.s.+ 12-m.s.	12	1683	12	4613	19	3653	58	123	352	507	186	-458	-218	-450	-2	106
f) 9-p.s.+ 12-p.s.	12	1877	12	6019	17	1772	52	167	206	296	23	-150	-350	-728	385	108

Skeletal system (s.s.), mixed-type system (m.s.), panel system (p.s.)

Increase in the bending moment in the column of critical floor, amounts to no less than three to seven times (685 %). As for the normal forces, their increasing due to collision ranges up to six times (574 %) according to the Serbian Code, which confirms its backwardness in comparison with EC8 and other current regulations.

The collisions last only several hundredths of a second. In this interval, the structures change their velocities, i.e. negative accelerations occur, which causes immense collision forces. The structures cannot endure forces of this magnitude, and their influence is most prominent in the columns of the taller building at the level of the floor immediately above the top of the lower building. The columns sustain a large number of cyclic post-elastic deformations, and the accumulation of damage in the course of the longer duration of the earthquake exhausts the bearing capacity of the structure. Therefore, the local crushing of concrete is inexorable, and in the severe cases, collapse of upper floors.

6 Conclusion

On the basis of the conducted research, it can be concluded that the results obtained by the dynamic analysis are significant, because they fully justify a significant place held by the expansion joint in designing.

Because of the great influence of the frequency characteristics of the earthquake in a dynamic analysis, accelerograms for local earthquakes should be used, because it is concluded from the parametric analysis that despite the impact of the maximum acceleration, a crucial influence on response of the structure has a predominant period of record.

The spatial models, which have been dominant in the recent years because of the availability of commercial software, due to the interaction of reinforced concrete walls (from both directions) and floor structures, as a rule, yield unrealistically low displacements. This may mislead the designers if they follow the logic that the elastic displacement of adjacent buildings multiplied by the ductility factor can be assumed as referential when designing separation joints.

The development of the regulations aims at formulating the design concept on the basis of control of some of the important parameters describing behaviour and damage to the structures – "performance based design". Thus horizontal displacement and the width of separation joints in any case should be an inevitable factor in design of adjacent structures.

In the event of future changes of national regulations and harmonization with European regulations it is necessary to more precisely include the influence such as plastic deformations of the structure and soil interaction. Also, it is necessary to clearly define the ductility coefficient depending on the material and type of the structure pursuant to the EC8 and ATC recommendations.

Acknowledgments

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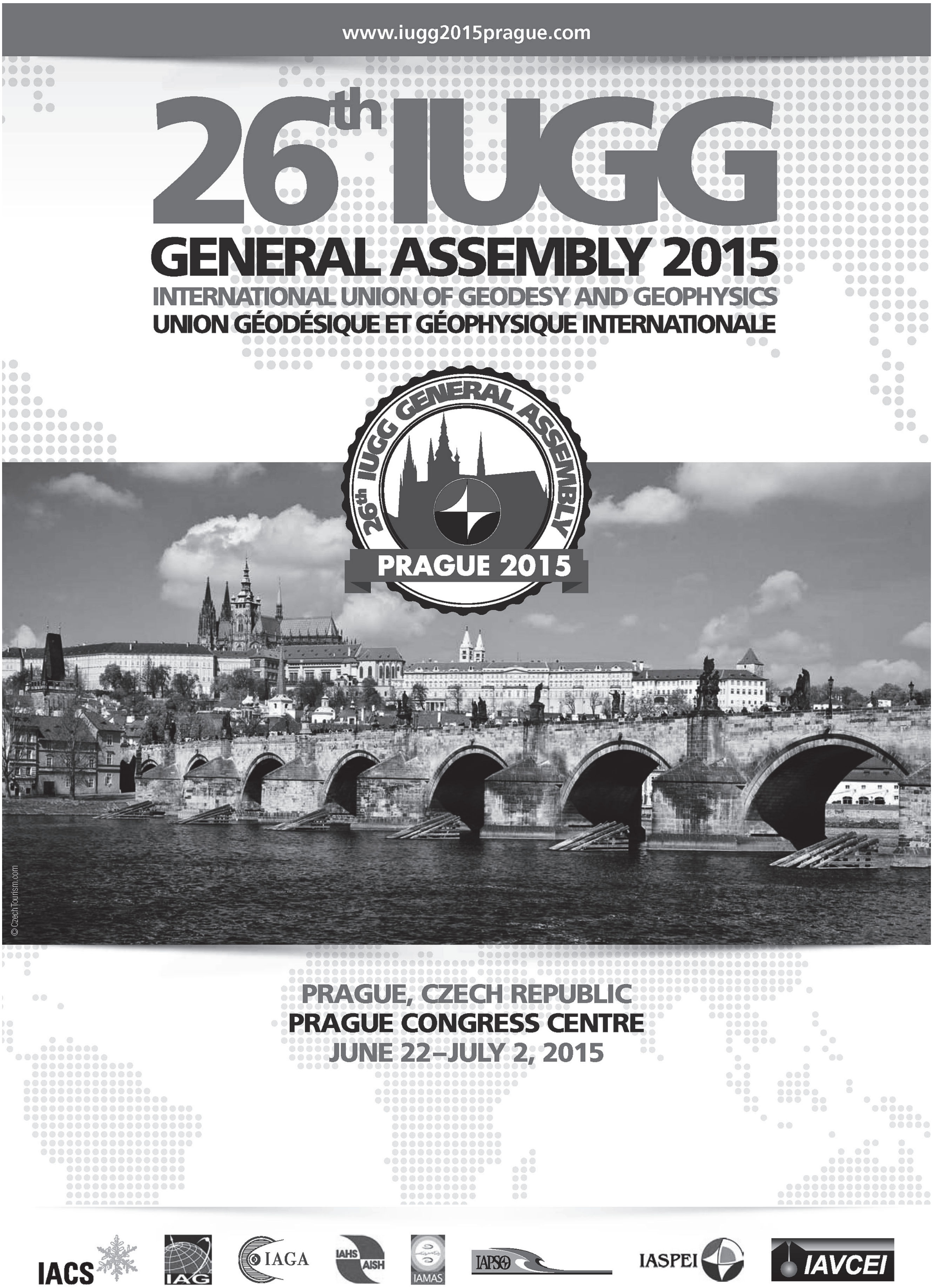
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