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DISPLACEMENT BASED-DESIGN CRITERIA FOR BRIDGES

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SUMMARY

The displacement-based criterion is the main one in bridges designing. The paper includes the analysis of movable bearings traveling and some practical recommendations for estimating the value of this traveling. The full displacement consists of four parts: displacement caused by the elastic pier deformation, displacement caused by the soil and foundation deformation, displacement caused by the non-synchronous piers excitation, displacements caused by the inelastic pier deformation. As the result of the analysis the role of each part of full pier displacement is estimated. Among mentioned components of complete displacement the incoherent excitations of piers is most important. It can be about 50% of the whole displacement

INTRODUCTION

Bridges are important structures providing the efficiency of the life support system for earthquake prone regions. The experience of past earthquakes shows that bridges piers get damaged beginning with earthquake intensity I=8 on MSK scale. The requirements for bridges piers computations and designing are included in the building codes of many countries. But there are no recommendations and demands for calculations of movable bearings traveling. Meanwhile the longe traveling of movable bearings causes the fall of decks from piers. Such heavy bridges damages occurred during all strong earthquakes. Very often in these cases bridges spans and piers were in a satisfactory condition. Fig. 1 illustrates the view of damages under consideration. In the report of Prof. Barr [1], presented at the XII European Conference the fall of decks from piers was pointed as one of the most important damages of bridges. The paper includes the analysis of movable bearings traveling and some practical recommendations for estimating the value of this traveling.

THE MOVABLE BEARING TRAVEL ESTIMATION

The displacement-based criterion is the main one in bridges designing. The control of movable bearings travel is necessary irrespective of ways of designing. Using the two-level designing, in future the

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multi-level designing, and the designing of damages scenario we have to apply the displacement-based criterion not only for kinematical computations but also for piers behavior analysis.

Fig.1. A typical example of a deck fall

In the present report, the components of full pier displacement are analyzed. The full displacement consists of the following parts

- Displacement caused by the elastic pier deformation
- Displacement caused by the soil and foundation deformation
- Displacement caused by the non-synchronous piers excitation
- Displacements caused by the inelastic pier deformation

The first three displacement components are important both for strong, infrequent and for relatively weak, frequent earthquakes. The calculation of these components is important for estimating the movable bearings travel for bridges in operation. The full sum of components should be calculated to estimate the ultimate movable bearings travel and to design the displacements restrictors. At last the inelastic pier displacement determines the damages of pier and possibility of its collapse.

The simplest diagram for estimating displacements caused by the elastic pier deformation is presented in fig.2. This diagram includes 8 degrees of freedom:

Y_1,Y_2 are the horizontal displacements of piers as a rigid body;

u_1,u_2 are the elastic displacements of the tops of piers;

Z_1,Z_2 are the vertical displacements of piers as a rigid body;

φ_1, φ_2. are the rotational displacements of piers as a hard body.

The kinematical analysis of the presented diagram permits to receive the following formula for moveable bearing travel ξ:

$$\xi = Y_1 + u_1 - \varphi_1 z_A - Y_2 - u_2 + \varphi_2 H_2, \qquad (1)$$

The complete list of symbols for formula (1) is shown in Fig.2.

To estimate the displacement components in accordance with (1), the dynamic system equations, describing the “span-pier-soil” interaction are constructed. These equations are rather complex and can be presented in the following way:

$$\mathbf{M\ddot{q}}+\mathbf{B\dot{q}}+\mathbf{Rq}+\mathbf{\dot{q}}^{\mathbf{T}}\mathbf{Aq}=-\mathbf{M\ddot{q}}_0-\mathbf{\dot{q}}^{\mathbf{T}}\mathrm{L}\mathbf{\ddot{q}}_0 \qquad (2)$$

where

M, B and **R** are inertia, damping and stiffness matrices;

$\mathbf{q}$={Y_1,Y_2,φ_1,φ_2,Z_1,Z_2,u_1, u_2} is the vector of generalized displacements;

$\mathbf{q}_0$ is the vector of kinematic excitations;

A and **L** are matrices used for describing the geometrical nonlinearity of the system.

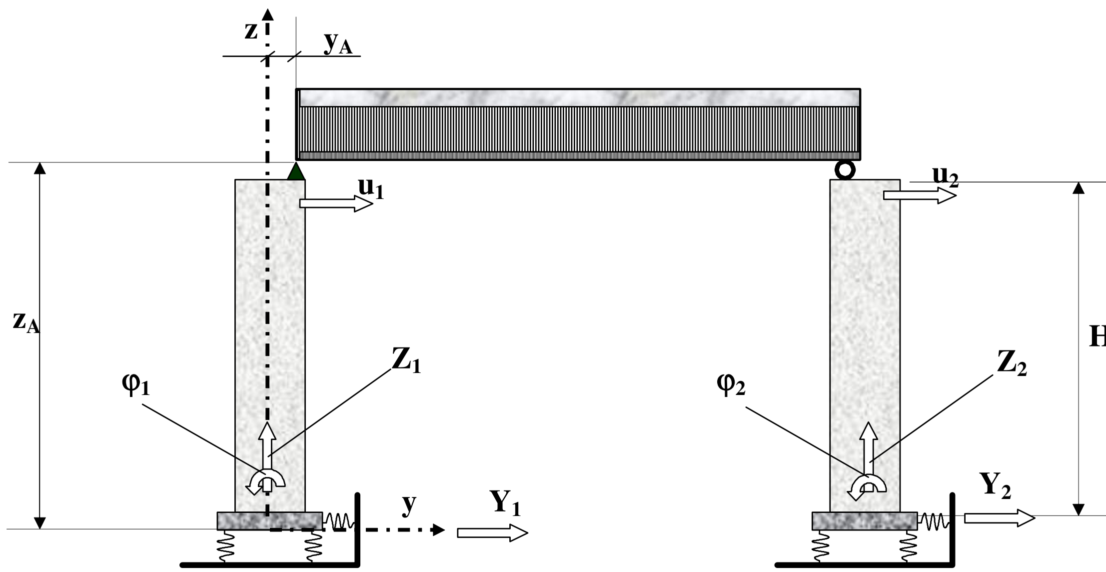

Fig.2. The simplest diagram for estimating the “soil-pier-span” interaction

Some types of simplified equations received from (2) on the base of asymptotic method are presented in the report. For each type of the simplified equations, the conditions of their application are formulated. These conditions depend on the ratio of piers height and thickness as well as on the ratio of moments of inertia of piers and spans.

The peculiarities of the received equations are the correlation between the vertical and horizontal oscillations and the availability of nonlinear members including products of generalized velocities particularly the products with $\dot{\varphi}_2$.

To estimate the third component of full displacement, random bridge vibrations were caused by incoherent independent excitation of each pier were considered. The complete excitation U_0 was presented as the sum of excitations of each pier u_k:

$$U_0=\sum_{k=1}^{N}u_i\ , \qquad (3)$$

where N is the number of piers.

For each excitation u_k seismic displacements and loads matrices were obtained. Unlike the case of coherence input, we get not only solitary one matrix of seismic loads but N matrices, i.e. one matrix for each pier excitation. In this case the bearing travel to be estimated is a double sum of its components

$$\Delta=\sum_{j=1}^{n}\sum_{k=1}^{N}\delta_j^{(k)}\ , \qquad (4)$$

where

$\delta_{j,k}$ is a component of the bearing travel for the mode "j", caused by the excitation of pier "k"; n is the number of structure modes taken into account.

To estimate the design value of Δ, we have to use the following formula

$$\Delta_d = \sqrt{\sum_{i=1}^{n}\sum_{j=1}^{n}\sum_{k=1}^{N}\sum_{r=1}^{N}\chi_{ijkr}\varepsilon_{ij}\delta_i^{(k)}\delta_j^{(r)}} \quad (5)$$

where

χ_{ijkr} is the correlation coefficient between the excitations of piers "k' and 'r' for modes 'i' and 'j';

ε_{ij} is the correlation coefficient of modes 'i' and 'j'.

Methods of estimating correlation coefficient ε_{ij} are well known. In these investigations the formulas of professor Ter-Kiurigian [2] and professor A.A.Petrov [3] were used.

The estimation of the correlation coefficient χ_{ijkr} involves some difficulties caused by the lack uniform approach to setting the excitations correlation. Therefore five variants of excitations correlation were considered.

1. Coherent piers excitations. In this case all values of χ_{ijkr} are equal to 1. For the bridge with regular construction the bearing traveling Δ=0.
2. Noncorrelated excitations. In this case the value of χ_{ijkr} is equal to 1 if i=j and χ_{ijkr}=0 if i≠j. At that $\Delta_d = \sqrt{\sum_{i=1}^{n}\sum_{j=1}^{n}\sum_{k=1}^{N}\varepsilon_{ij}\delta_i^{(k)}\delta_j^{(k)}}$. If the modes are noncorrelated, $\Delta_d = \sqrt{\sum_{j=1}^{n}\sum_{k=1}^{N}\delta_j^{(k)2}}$. For the bridge with regular construction the bearing traveling $\Delta=\delta\sqrt{2}$, where δ is the displacement of the top of the pier.
3. Excitations set by using the hypothesis of a "frozen" wave [3].In this case the value of χ_{ijkr} can be estimated by formula

$$\chi_{lsjr} = \cos\left(\frac{k_i + k_r}{2V_2}\cdot L_{sl}\right) \quad (6)$$

where p_i is the frequency of the mode 'i' of the structure, L_{kr} is the distance between pier 'k' and pier 'r', V is the velocity of shear waves in the soil.

4. Antiphase piers excitations. In this case the value of χ_{ijkr} is equal to 1 if i=j and χ_{ijkr}=-1 if i≠j. For the bridge with a regular construction the bearing travel Δ=2δ.
5. The pier excitations include two parts. The first part is noncorrelated oscillations and the second part is a "frozen" wave.

In each cases of excitations

$$0 < \Delta < 2\delta \quad (7)$$

The inequality (7) points to the importance of both piers excitations correlation and the correlation coefficient estimation. Some additional seismological data is necessary to estimate this coefficient. In particular, the displacement response spectrum for the original ground and the correlation function of the points oscillations of original ground are necessary.

The authors proposed to estimate piers displacement in conditions of incomplete seismological information on the basis of the fifth variant of excitations correlation.

To estimate the inelastic pier displacement, the model of concrete taking into account the formation of cracks was considered. In this model the pier stiffness depends on the maximum elastic displacement during the history of pier loading.

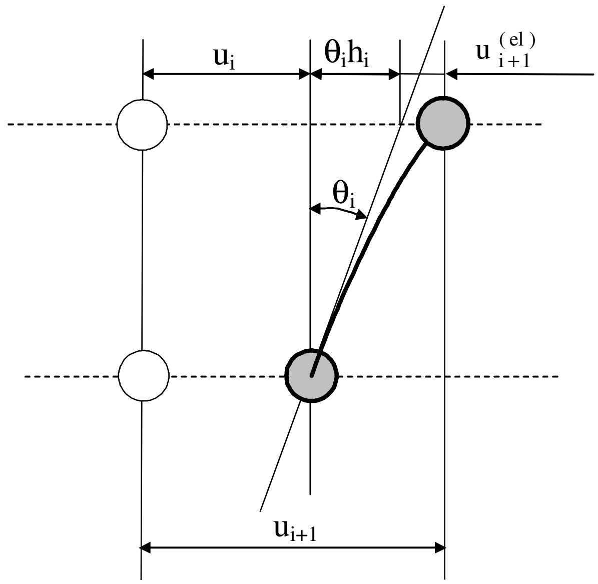

Fig.3. Fragment of the pier design diagram.

$$R(u) = \begin{cases} \dfrac{C(u_{max}^{(el)})u}{1+u^2\kappa(u_{max}^{(el)})}, & \text{при} \quad u_{max}^{(el)} > u_{lim} \\ C_0 u, & \text{при} \quad u_{max}^{(el)} \leqslant u_{lim} \end{cases} \qquad (8)$$

In this formula $C(u_{max}^{(el)})$ is the stiffness of a pier section, which depends on the maximum of elastic part of the mutual displacement of the ends of section $u_{max}^{(el)}$ (Fig.3); C_0 is the initial value of the section pier stiffness; u_{lim} is the limit elastic displacement the excess of which causes the pier damages for the section under consideration; $\kappa(u_{max}^{(el)})$ is a damage parameter. The value of κ increases linearly from 0 to 1, as the value $u_{max}^{(el)}$ increases. The typical view of dependence R(u) is presented in fig. 4. The characteristic points of this diagram, shown in fig.4, were calculated using the geometrical parameters of the cross-sections of the pier section and their reinforcement.

The time-history analysis of seismic oscillations equations made it possible to analyze the values of non-elastic piers displacements. An example of such displacements calculation is presented in Fig.5a. The graphical chart of pier damaging corresponding to these displacements is presented in Fig. 5b. The inelastic piers displacements turned out to be rather small even for large damaging of the pier.

This result seems contradictory to the existing ideas about inelastic deformations of piers, because firstly all field investigations point to large inelastic residual seismic displacements of piers, which can be clearly seen. Secondly falls of decks are put down to nonlinear inelastic piers displacements. Therefore, inelastic displacements u_{nl} are considered to be bigger than elastic ones u_e in building codes. For example, in Eurocode-8 u_{nl}=1.6 u_e [1].

The results obtained can be explained by two reasons.

1. The inelastic behavior of a pier has two stages. The first stage includes displacements from the beginning of nonlinear deformations (point 1 in Fig.4) to the limit displacements u_{lim}, corresponding to the maximum of the pier response (point 2 in Fig.4). The second stage includes displacements bigger than the limit ones.

During the first stage the piers stiffness is positive and it prevents a great increase of displacements. At point ‘2’ a plastic hinge appears in the body of the pier and the pier turns into a

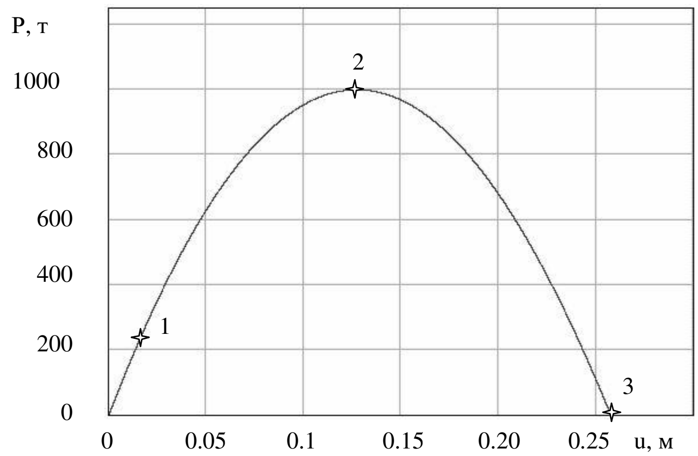

Fig.4. An example of R(u) diagram
1 – transfer from elastic to nonelastic stage of the work;
2 –the actual breaking point;
3 – conditional breaking point

mechanism the displacement of which can be very significant, up to a complete fall of the pier. It is the second stage that is usually observed during field investigations after strong earthquakes. In designing the second stage of piers behavior should not be allowed, therefore, the first stage was investigated in this research.

2. The long movable bearings travel is caused not only by the nonlinear piers displacement but also by other reasons, among them the incoherent excitations of piers is most important. Other reasons being excluded, the remaining part of the displacement appears to be not very long.

The presented displacement estimations are random values. The statistical analysis of these estimations is presented in the report. In this analysis the earthquake input was regarded as random excitation with the acceleration amplitude distributed in accordance with Veybull law. The output parameters are estimated using both analytical and numerical methods. On the basis of this analysis, the vulnerability function for bridge pier can be constructed. As the result of the analysis the role of each part of full pier displacement is estimated. As an example of such analysis displacements of a pier presented in Fig.6 were estimated. The results of this estimation are presented in Fig.7.

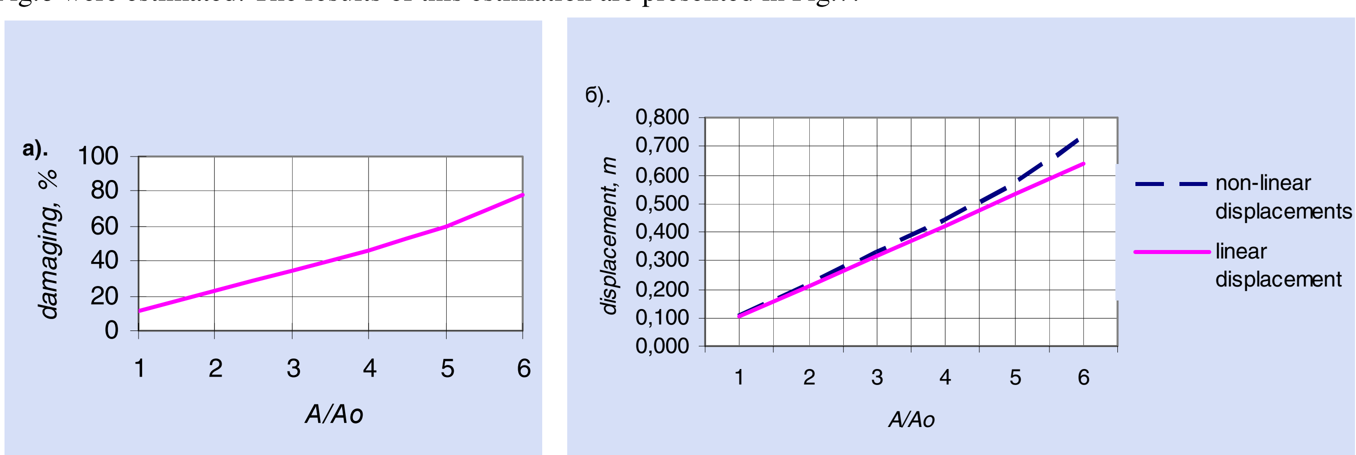

Fig.5. Dependencies of the pier damaging (a) and the heard of pier displacement (b) on relative acceleration A/A_0

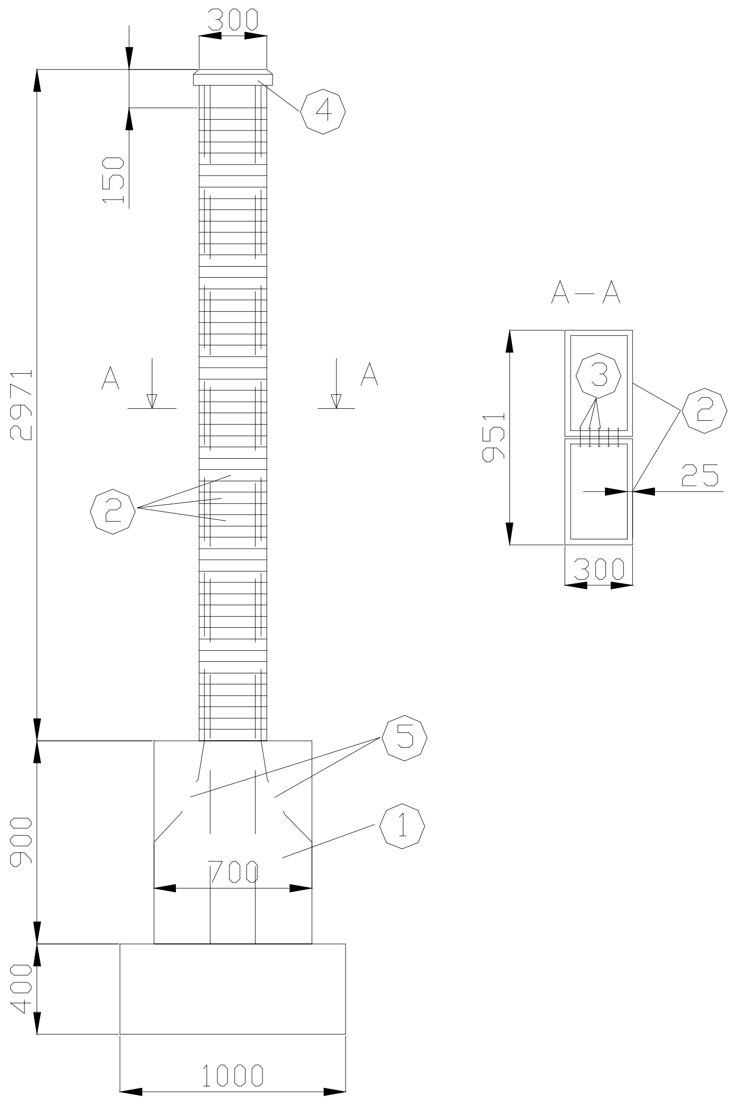

Fig.6. The pier diagram

1 - the monolithic concrete socle part; 2 – hollow box-shaped stressed concrete blocks; 3 - tension bars; 4 – the head of the pier; 5 - stressed reinforcement

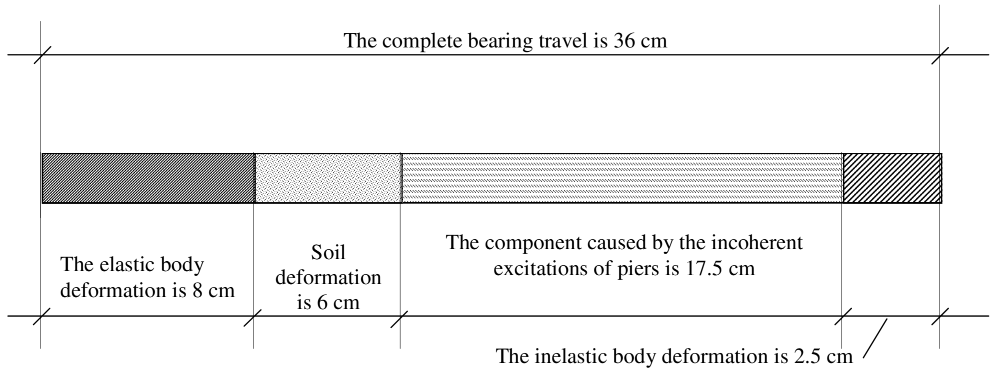

Fig.7. The diagram of bearing travel components

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