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ANALYSIS OF SOIL-STRUCTURE INTERACTION OF MAJOR RIVER-CROSSING BRIDGES

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SUMMARY

At the first part of this paper, topics concerning current state-of-the-art techniques for the seismic analysis of major river-crossing highway bridges are introduced in conjunction with soil-foundation-bridge interactions. These include earthquake accelerograms, multiple-support excitations and analyses of soil-pile-bridge interactions by Winkler-type spring model, boundary element model and finite element model. From the discussion, it is concluded that a full-scale three dimensional finite element model should be examined in order to develop a more effective simplified method which can be used in engineering practice. An existing truss-arch highway bridge in Southern Illinois is selected for the case study. The structure is located on deep alluvial soil deposits and the piers for the main span are on floating caissons and the piers of the approaches are on friction pile foundations. A three dimensional model of the bridge, along with its approaches, is developed. The preliminary results on the dynamic characteristics of the bridge are reported here.

INTRODUCTION

Unlike small bridges or overpasses, major river-crossing bridges have complex dynamic characteristics and usually a three-dimensional analysis is required to extract the characteristics. Several methods of dynamic analysis are currently available, if the foundation and the soil deposit are involved. Localized analysis for one span-pier system is often used to evaluate seismic performance of the bridge under the assumption that the most vulnerable parts of the bridge system are both ends of each pier and the bearings. However, this localization leads to the omission of global dynamic behavior which may play a critical role for bridges with large dimension. The simplest method could be just adding impedance matrices or springs and dashpots at the base of piers. This method has been very popular in cases where only the behavior of superstructure is of concern and a simple procedure is preferred.

This paper reports a part of an ongoing research project, the primary purpose of which is to develop an effective method for the seismic evaluation of major river-crossing bridges. A brief discussion on seismic input motions and multiple-support excitations are presented at first. The use of Winkler-type foundation models, boundary element models and finite element models is discussed. An existing truss-arch bridge in southern Illinois was chosen for a case study and preliminary results from a three dimensional FE model of the bridge are given and they will be used to detect the overall dynamic characteristics of the bridge for the further study (Kornkasem et al. 1999).

SEISMIC INPUT MOTIONS

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Due to infrequent seismic event in Mid-America, actual strong motion records in the region are not available. Artificially generated earthquake records at a bedrock location reflecting the characteristics of the faults and the topography in the region are necessary for the evaluation. Details about obtaining those records are not discussed in this paper. Complete sets of three orthogonal components of records are preferred, since in many cases it was observed that vertical components plays an important role in structural responses of large bridges. If a nonlinear three-dimensional finite element model is used for the site, the seismic wave at the bedrock will propagate through the soil domain and free field motions at each depth are easily obtained. Methods using one-dimensional wave propagation theory and an equivalent linear shear modulus, such as SHAKE, cannot represent vertical motion and nonlinear behavior of the site and the effectiveness should be carefully examined

EFFECS OF TRAVELING WAVES

In most cases of evaluating the dynamic response of a single structure to earthquake excitations, it has been assumed that the ground motion is a function of time only, not a function of space. This assumption will give satisfactory results for structures occupying relatively small areas. For bridges with long span lengths, the effect of spatial variation of ground motion becomes more important, and thus the assumption could be inappropriate.

Generally, to implement the feature of spatially varying ground motion to dynamic analysis of structures has been done by either deterministic or stochastic way. In deterministic ways of modeling it has been often assumed that a seismic wave propagates with constant velocity without changing its shape (e.g. Nazmy and Abdel-Ghaffar 1992; Leger et al. 1990). Traveling waves tend to reduce the maximum displacements, in that peak values of input seldom occur simultaneously for all supports because of the phase difference of the wave at each support. The reductions of the response do not always mean safer structures because they also introduce differential motions and result in the increase of internal forces. The amounts of the decrease of displacements and the increase of internal forces depend upon the geometry and stiffness of the structures and the seismic wave velocities. Even if these two effects can be seen clear for simple structures under harmonic excitations, the response is quite unpredictable and general tendency may not apply in complex situations such as long span bridges under earthquake loading, e.g. the maximum displacement can be significantly increased under certain circumstances like a specific wave velocity or angle of incidence (Nazmy and Abdel-Ghaffar 1992; Dusseau and Wen 1986). In real situations, however, ground motions at two distant points are not the same in general. Through the analysis of existing ground motion data, stochastic models of ground motions at spatially separated points can be constructed. In most cases, stochastic analyses have been done in frequency domain, often in conjunction with response spectrum methods (e.g. Kiureghian and Neuenhofer 1992). The response spectrum method has the advantage of its computational effectiveness over a time domain analysis and it has been reported that it could yield satisfactory results (Dumanoglu and Severn 1990).

In most cases, it has been assumed that seismic waves travel along ground surface without consideration of soil-structure interaction. However, the interaction of traveling waves through soil and the foundation of bridges may critically affect the behavior of the superstructure. As an example, vertically incident waves cause the phase difference of acceleration on the foundation, and it results in rocking motion at the base of bridge piers. For a more accurate differential motion at each support to be obtained, a three-dimensional FE analysis should be used where input motions travel along the bedrock and propagate through soil deposit, whether the FE model of the site is coupled with the superstructure model or not.

SOIL-STRUCTURE INTERACTIONS

Most major river-crossing bridges in Mid-America are founded either on floating piles or caissons in deep alluvial soil deposit. Even if there have been a extensive researches on the detailed modeling and behavior of pile foundations under dynamic loading for a long time, the foundations of bridges are often simplified with equivalent springs and dashpots. In geotechnical area, however, only pile or pile group stiffness or behavior within piles has been a major concern, and superstructures have been ignored or oversimplified. Therefore it is required that a model dealing the superstructure and the foundation with the same level of importance be developed. Currently, roughly three classes of methods are available to model dynamic behavior of pile foundations; finite element models, boundary element models and Winkler-type foundation models. Discussions on these methods are closely related to the topics of nonlinearity and coupling of two systems. Analytical methods, which were widely used in early stage of the researches on the dynamic behavior of pile foundations, are not suitable for irregular site conditions and complex structures.

Despite the extensive time needed for computations, FE analysis is considered to be the most accurate method for the combined system of the superstructure, the foundation and the soil deposit if proper boundary conditions are provided. Recently, a simplified quasi-3D nonlinear FE model (Finn et. al. 1996) and a full-scale 3D FE model using a plastic constitutive law of the soil medium (Cai et. al. 1995) were developed, and it was shown that nonlinearity and three dimensionality play an important role. For the most rigorous result to be obtained, wave passage effect of bedrock motions and possible formation of gaps at soil-foundation interface have to be taken into account for bedrock input motion as well as nonlinear soil behaviors.

The boundary element methods have advantages over the finite element method in that computational time can be significantly reduced and radiation of waves can be precisely reproduced. Even if several hybrid models have been reported (Guin et al. 1998; Pavlatos and Beskos 1994) where FEM were used for structural components and BEM for soil domain, BEM is not suitable for representing nonlinear behavior in soil medium. Therefore the effect of nonlinearity in soil deposit must be closely examined before a BEM is applied.

As another alternative to the FEM approach, Winkler-type foundation models easily allow the nonlinear behavior of soil medium (e.g. Nogami et al. 1992; Elassaly et al. 1995) and the time-domain analysis through frequency independent mass, springs and dashpots (Nogami et al. 1992). The accuracy of this approach critically depends upon proper representation of the pile-to-pile interaction and the effect of far-field motion. Wang et al. (1998), through the comparison of various spring-dashpot systems, pointed out that radiation damping and p-y curves have significant effect on the response of nonlinear systems. Zheng and Takeda (1995) showed that frequency independent mass-spring models are valid only in a certain range of natural frequency of the system.

A three-dimensional FE model including whole soil domain and the entire bridge spans will be quite large and it might not be appropriate procedure for engineering practice. In other methods, however, there are several limitations whose validity are not fully evaluated and verified. As a result, in order to develop a more computationally effective method, it is necessary to conduct a series of analyses using a coupled full-scale FE model with all necessary nonlinear features to obtain a reference case for the use in evaluating the other simplified methods.

A general procedure using finite element models can be constructed by putting a series of horizontal and vertical dashpots at the vertical boundaries of the soil deposit to absorb scattering waves radiated from the structure. The equation of motion for a general case is

$$\begin{bmatrix} M_{11} & M_{12} & M_{13} \\ M_{21} & M_{22} & M_{23} \\ M_{31} & M_{32} & M_{33} \end{bmatrix} \begin{Bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \\ \ddot{u}_3 \end{Bmatrix} + \begin{bmatrix} C_{11} & C_{12} & C_{13} \\ C_{21} & C_{22} & C_{23} \\ C_{31} & C_{32} & C_{33} \end{bmatrix} \begin{Bmatrix} \dot{u}_1 \\ \dot{u}_2 \\ \dot{u}_3 \end{Bmatrix} + \begin{bmatrix} K_{11} & K_{21} & K_{13} \\ K_{21} & K_{22} & K_{23} \\ K_{31} & K_{32} & K_{33} \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \\ u_3 \end{Bmatrix} = \begin{Bmatrix} P_1 \\ P_2 \\ P_3 \end{Bmatrix} - \begin{Bmatrix} 0 \\ c_{s,p}\dot{u}_2 \\ 0 \end{Bmatrix} \tag{1}$$

where subscript 1, 2 and 3 stand for the degree of freedom of internal nodes, vertical boundaries and horizontal boundaries at the bottom, respectively, and $c_{s,p}$ is wave velocity for S-wave or P-wave.

$$\begin{aligned} &\begin{bmatrix} M_{11} & M_{12} \\ M_{21} & M_{22} \end{bmatrix} \begin{Bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \end{Bmatrix} + \begin{bmatrix} C_{11} & C_{12} \\ C_{21} & C_{22} \end{bmatrix} \begin{Bmatrix} \dot{u}_1 \\ \dot{u}_2 \end{Bmatrix} + \begin{bmatrix} K_{11} & K_{12} \\ K_{21} & K_{22} \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix} \\ &\quad = \begin{Bmatrix} P_1 \\ P_2 \end{Bmatrix} - \begin{Bmatrix} 0 \\ c_{s,p}\dot{u}_2 \end{Bmatrix} - \begin{bmatrix} M_{13} \\ M_{23} \end{bmatrix} \{\ddot{u}_3\} - \begin{bmatrix} C_{13} \\ C_{23} \end{bmatrix} \{\dot{u}_3\} - \begin{bmatrix} K_{13} \\ K_{23} \end{bmatrix} \{u_3\} \end{aligned} \tag{2}$$

$$\begin{aligned} &\begin{bmatrix} M_{11} & M_{12} \\ M_{21} & M_{22} \end{bmatrix} \begin{Bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \end{Bmatrix} + \left(\begin{bmatrix} C_{11} & C_{12} \\ C_{21} & C_{22} \end{bmatrix} + \begin{bmatrix} 0 & 0 \\ 0 & diag[c_{s,p}] \end{bmatrix} \right) \begin{Bmatrix} \dot{u}_1 \\ \dot{u}_2 \end{Bmatrix} + \begin{bmatrix} K_{11} & K_{12} \\ K_{21} & K_{22} \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix} \\ &\quad = \begin{Bmatrix} P_1 \\ P_2 \end{Bmatrix} - \begin{bmatrix} M_{13} \\ M_{23} \end{bmatrix} \{\ddot{u}_3\} - \begin{bmatrix} C_{13} \\ C_{23} \end{bmatrix} \{\dot{u}_3\} - \begin{bmatrix} K_{13} \\ K_{23} \end{bmatrix} \{u_3\} \end{aligned} \tag{3}$$

For the problem where [M] is a diagonal matrix and there are no external forces,

$$\begin{bmatrix} M_{11} & 0 \\ 0 & M_{22} \end{bmatrix} \begin{Bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \end{Bmatrix} + \left(\begin{bmatrix} C_{11} & C_{12} \\ C_{21} & C_{22} \end{bmatrix} + \begin{bmatrix} 0 & 0 \\ 0 & diag\left[c_{s,p}\right] \end{bmatrix} \right) \begin{Bmatrix} \dot{u}_1 \\ \dot{u}_2 \end{Bmatrix} + \begin{bmatrix} K_{11} & K_{12} \\ K_{21} & K_{22} \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix} = -\begin{bmatrix} C_{13} \\ C_{23} \end{bmatrix} \left\{ \dot{u}_3 \right\} - \begin{bmatrix} K_{13} \\ K_{23} \end{bmatrix} \left\{ u_3 \right\} \quad (4)$$

CASE STUDY

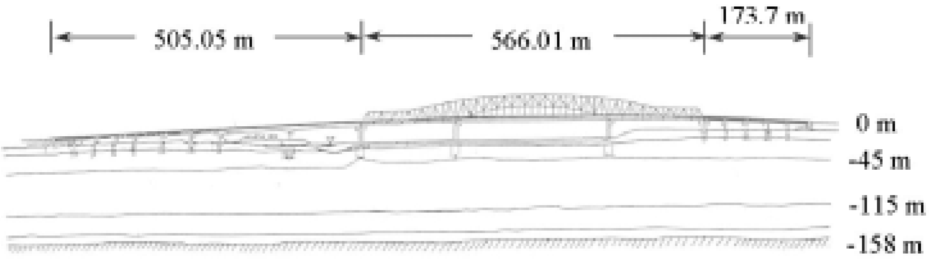

Figure 1. Description of the Cairo Bridge

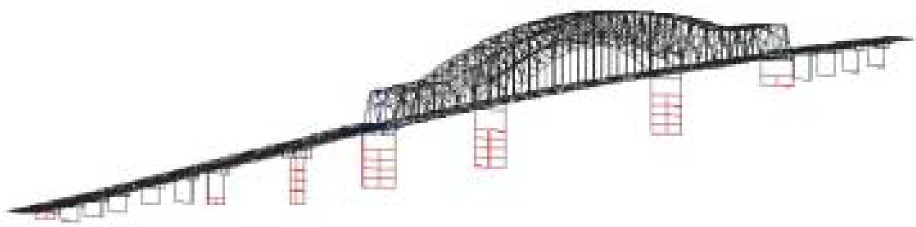

Figure 2. FE model of the bridge.

The Cairo Bridge is located over the Mississippi River in Southern Illinois. The bridge consists of approach spans and a main span (Figure 1). The soil deposit under the bridge is about 160m thick and consists of 5 layers of sand and clay. In order to examine the dynamic characteristics of the bridge, a three-dimensional finite element model was constructed using SAP2000(Figure 2). The modal characteristics of the bridge showed that there are two longitudinal modes around the period of 2.15sec and 47 modes above the period of 1.0 sec. It is also noteworthy that it has a significant number of modes associated with the torsion of approach spans and they may play an important role in the seismic behaviour of the bridge.

Artificially generated acceleration time histories developed for the use in the New Madrid seismic zone (Hwang, 1998) were applied at the bedrock to obtain ground motions. Three types of models were used for time-domain analyses. In model A, the base of each pier is fixed and the model is completely linear, while the base is still fixed but nonlinear behavior in bearings and expansion joints were taken into account in model B. In model C, in addition to nonlinear behavior, six spring elements were attached at the base of each pier to account for the foundation impedance. Absolute values of maximum displacements along the centerline of the deck were given for each model. The result shows that there exist significant differences between longitudinal and transverse displacements from model A and C or B and C(Figure 4) and implies that soil-structure interactions play a more important role in those displacements than in vertical one. Transverse displacements are observed to be so high that those can cause severe damage to bearings. Significant torsion is present due to the difference between shear centers and mass centers of deck and truss sections.

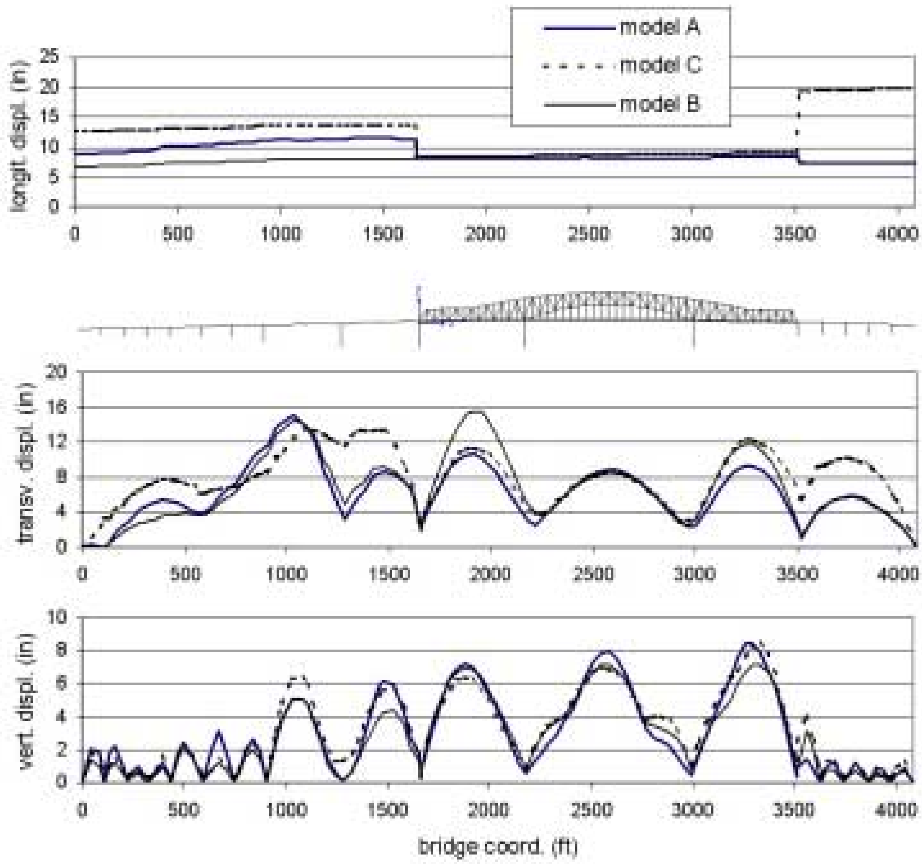

Figure 4. Max. displacements along the deck

FUTURE REAEARCH

The goal of this research is to develop an effective tool for the seismic evaluation of major river-crossing bridges. Different classes of methods, including detailed full-scale FE model, will be applied to various types of bridges and site conditions. Effects of soil nonlinearities, seismic wave propagation, soil-pile-bridge interactions and so on will be evaluated and then the relative importance of each effect will be determined. With all the information, a fully evaluated set of procedure for the purpose will be suggested.

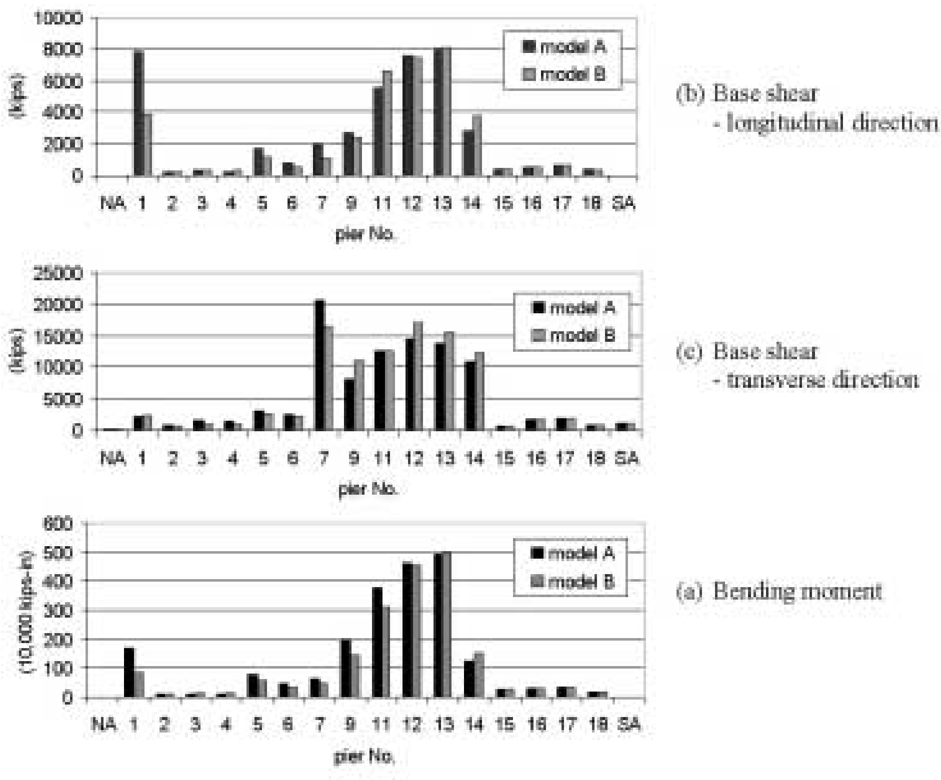

Figure 4. Reaction Forces at the base of each pier

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REFERENCES

Cai, Y. X., Gould, P. L. and Desai, C. S. (1995), "Investigation of 3-D nonlinear seismic performance of pile-supported structures", *Performance of Deep Foundations under Seismic Loading*, ASCE Geotech. Special Pub. No. 51, 17-31.

Dumanoglu, A. A. and Severn, R. T. (1990), "Stochastic response of suspension bridges to earthquake forces", *Earthquake Eng. Struct. Dyn.*, **19**, 133-152.

Dusseau, R. A. and Wen, R. K. (1986), "Responses of a long span arch bridge to differential abutment motions", *Proc. 3rd U.S. Nat. Conf. Earthquake Eng.*, EERI, 2049-2060.

Elassaly, M., Ghali, A. and Elbadry, M. M. (1995), "Influence of soil conditions on the seismic behaviour of two cable-stayed bridges", *Canadian J. Civ. Eng.*, **22**, 1021-1040.

Finn, W. D. L., Thavaraj, T. and Wu, G. (1996), "Seismic Analysis of pile foundations: State-of-art", *Proc. 11th WCEE*, paper no. 2073.

Finn, W. D. L. and Wu, G. (1996), "Nonlinear seismic analysis of pile foundations", *Proc. 11th WCEE*, paper no. 1414.

Guin, J. and Banerjee, P. K. (1998), "Coupled soil-pile-structure interaction analysis under seismic excitation", *J. of Struct. Eng.*, ASCE, **124**, no 4, 434-444.

Hwang, H. M. (1998), *Artificial acceleration time histories*, Center for Earthquake Research and Information, University of Memphis.

Kiureghian, A. D. and Neuenhofer, A. (1992), "Response spectrum method for multiple-support seismic excitation", *Earthquake Eng. Struct. Dyn.*, **21**, 713-740.

Kornkasem, W. et al. (1999), "Seismic analysis of a truss-arch bridge across the Mississippi River", Presented for publication on the 5th U.S. Nat. Conf. Lifeline Earthquake Eng.

Leger, P., Ide, I. M. and Paultre, P. (1990), "Multiple support seismic analysis of large structures," *Compt. Struct.*, **36**, 1153-1158.

Nazmy, A. S. and Abdel-Ghaffar, A. M. (1992), "Effect of ground motion spatial variability on the response of cable-stayed bridges," *Earthquake Eng. Struct. Dyn.*, **21,** 1-20.

Nogami, T., Otani, J., Konagai, K. and Chen, H.-L. (1992), "Nonlinear soil-pile interaction model for dynamic lateral motion", *J. of Geotech. Eng.*, ASCE, **118**, no 1, 89-106.