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SEISMIC ANALYSIS OF A TRUSS-ARCH BRIDGE CROSSING THE MISSISSIPPI RIVER

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SUMMARY

The initial part of an ongoing research project to determine the seismic behaviour of an existing truss-arch bridge and its approach structures carrying I-57 crossing the Mississippi River is presented herein. In this part, an analytical investigation of the effects of different foundation models on dynamic characteristics of the overall bridge structure was performed. Comparisons of the results from nonlinear time-history analyses using fixed-base and equivalent-linear-soil-spring models, with and without pile-soil-pile interaction, have shown that the dynamic characteristics of the overall bridge structure cannot be realistically represented by the fixed-base model, thus indicating the need to include the foundation models in the seismic analysis of the entire bridge system.

INTRODUCTION

Following the completion of Phase I of the Illinois Department of Transportation's Seismic Bridge Condition Survey in early 1991 in which bridges were ranked with respect to their potential for damage by an earthquake, six bridges with various sizes and types of construction were selected for further study Phase II to determine preliminary seismic retrofit designs and cost estimates. Among these bridges which were ranked within the top 20 highest risk bridges was the bridge carrying Federal Aid Interstate Highway Route 57 over the Mississippi River at Cairo, Illinois.

Because of its long spans, the I-57 Mississippi River Crossing Bridge was chosen for detailed analysis and seismic evaluation in this project. Three-dimensional models are being used for linear and nonlinear seismic analyses of the existing bridge-foundation system. By applying the substructure method, Soil-Foundation-Superstructure Interaction (SFSI) problems can be divided into four consecutive steps: (1) modelling of the bridge superstructure, (2) determination of the foundation characteristics, (3) determination of the foundation input motions in the absence of the superstructure and (4) analysis of the seismic response of the bridge superstructure supported on the foundation springs and subjected to the foundation input motions. To begin developing insight into the seismic behaviour of the system, analyses are being done initially with the computer program SAP2000 (Computers and Structures Inc., Berkeley). Results of the nonlinear time-history analysis of several cases for which the bridge foundations are differently modelled are presented. These results also serve as a reference case for more elaborate analyses that will be performed subsequently.

LOCATION AND DESCRIPTION OF THE BRIDGE SYSTEM

The bridge, carrying F.A.I. Route 57 over the Mississippi River at Cairo, spans across the Mississippi River, with its north abutment in Illinois and its south abutment in Missouri. Two approach structures lead into the main channel crossing. The north approach consists of 9 spans of concrete deck supported on steel plate girders between piers 1 and 11 (497.8 m. or 1659 ft). The main channel crossing consists of a three-continuous-span, truss-arch structure over main and auxiliary navigation channels, between piers 11 and 14 (566.4 m. or 1857 ft). The south approach is similar to the north approach but has only 5 spans of concrete deck supported on steel

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plate girders, between piers 14 and 18 (326.1 m. or 1087 ft). Figure 1 shows an elevation view of the bridge including soil profile.

The substructures of both approaches are of similar construction, consisting of reinforced concrete columns connected by spandrel beams except at pier 9 where reinforced concrete diaphragm walls are integral with the columns throughout their length. For the main crossing, the pier columns are connected by reinforced concrete spandrel beams and diaphragm walls from the top of the footing up to about two-thirds the height of the columns. The piers of the approach structures and main channel crossing are supported on the friction pile foundations and caissons, respectively.

MODELLING OF THE BRIDGE STRUCTURE

An analytical model of the entire bridge (Figure 2) was made to represent the structure as shown on as-built construction drawings provided by the Illinois Department of Transportation (IDOT). To account for cracking of concrete, the flexural stiffness of pier columns and walls was determined using 50% of the gross EI, while 75% of the gross EI was used for the deck as recommended by ATC-32 (1996). The shear stiffness was based on the shape of the cross section according to established principles of mechanics of materials and was not reduced.

The bridge floor system consists of reinforced concrete deck acting compositely with 7 welded steel plate girders in the approach spans and with 9 steel plate girders in the main channel crossing. A 3-D grid model was used for the bridge deck system. The grid model is preferable because it represents the overall characteristics of the bridge deck system with good accuracy and requires less computational time and effort than models using shell elements.

The main truss members were modelled by frame elements with the connections assumed to be rigid. Due to the stiffening effect caused by the bolted gusset plate connections and overlap of cross sections at a connection, an analysis based upon the centerline-to-centerline geometry of the members is likely to be too flexible. This stiffening effect was taken into account by using a rigid-end factor, which is defined as the length fraction of each end offset assumed to be rigid for bending and shear deformation. The mass contributed by the frame elements is lumped at each joint for each of the three translational degrees of freedom.

Fixed and expansion steel bearings are used throughout the bridge. The bearings were modelled in such a way that the fixed bearings could rotate and the expansion bearings could both translate and rotate within the allowable limits in the longitudinal direction. They were pinned against transverse movements. At the main truss structure, the allowable rotation which is defined as the rotation that can take place freely, is +/- 0.192 radian and +/- 0.25 radian for the fixed bearings and expansion (rocker) bearings, respectively. Similarly, the allowable expansion, defined as the longitudinal translation that can take place freely, after which the bearings become stiff is +/- 0.23 m. (8.9 in.) for the expansion bearings. These values, used in the analyses, were determined according to the as-built drawings with an assumption of 50 °F ambient conditions. The gapping and stiffening behaviour of the bearings as described above was modelled using nonlinear gap and hook elements. An example of the idealised force-displacement relation for the expansion bearing is illustrated in Figure 3.

Expansion joints are located at piers 11 and 14 at the transition between the main truss and approach spans. The allowable expansion at ambient temperature of 50 °F is +/- 0.24 m. (9.3 in.). In the 3-D model, the adjoining members at each side of the expansion joints were modelled as separate members connected by the nonlinear elements.

MODELLING OF THE BRIDGE FOUNDATIONS

Different foundation models are used in the time-history analysis: the fixed-base model, and the equivalent-linear-soil-spring models with and without Pile-Soil-Pile Interaction (PSPI). The first model is the model in which the support conditions are assumed to be fixed for all degrees of freedom. The model is used to evaluate effects of the foundation modelling on behaviour of the bridge superstructure and also serve as a comparison case for more detailed Soil-Structure Interaction analyses. In the second model, the stiffness matrix of the pile foundations is obtained without consideration of the PSPI effects. In the third model, the PSPI effects are taken into account.

The stiffness of the pile foundations is computed according to a proposal by Lam and Martin (1986), included in the Seismic Design Guidelines for Highway Bridge and recommended by the American Association of State

Highway and Transportation Officials (AASHTO). The equivalent-linear stiffness of each pile is calculated based on the estimated soil modulus according to an assumed level of shaking, which shall be checked against values obtained from the response-history analysis for verification. The single pile stiffness matrices are then statically condensed to the foundation-structure-interface node to develop 6x6-stiffness matrices for the pile groups using basic matrix operations. The stiffness of each pile group is input at the base of the pier columns in the second model.

The pile group effects are accounted for by applying the interaction-factor method originally introduced by Poulos (1968). The static-interaction-factor method has been shown by a number of researchers to yield reasonable predictions of pile group stiffness during earthquake shaking. This method uses the Mindlin solution to evaluate the response of a point within the interior of a semi-infinite linearly elastic isotropic homogeneous mass (half space mass) as a result of the application of a harmonic or impulse load at another point in the half space mass. The superposition is then used to incorporate the stiffness of single piles modified by interaction factors into the pile group stiffness. This was done for the third model using the method described in Poulos and Davis (1980).

The computed displacements at the base of the pier were compared with the displacement level assumed to estimate the soil properties to verify the initially estimated soil modulus. The determination of the pile group stiffness and the seismic analysis were repeated with the appropriately adjusted soil properties until the convergence with acceptable tolerance was reached.

SITE RESPONSE ANALYSIS AND INPUT GROUND MOTIONS

Since the magnitude 8 earthquakes that had occurred in the Midwest region predated the development of modern seismological instruments, no recorded accelerograms from such strong earthquakes seems to have been available. As a result, synthetic accelerograms generated for NEHRP B sites (Hwang, 1998) as a function of moment magnitude and epicentral distance are chosen to be used in the investigation. Since the bridge is located approximately 40 km. north-east from the New Madrid seismic zone, motions corresponding to a moment magnitude of 7.5 and epicentral distance of 40 km. were used as outcrop motions in the site response analyses. The shear wave velocity of the top rock layer was set to 1 km/s (3,300 ft/s).

The bridge is located over deep alluvial soil deposits with a thick layer of soft to stiff clay soils near the ground surface. For this soil profile, the bedrock motions are expected to be modified by the soft soil deposits resulting in lower frequency motions at ground surface which are believed to be critical for long period structures primarily long-span bridges. To account for such matter, site response analyses were performed for several soil profiles to determine reasonable bounds on the expected soil profile at different locations using the computer program SHAKE91 (Idriss and Sun, 1992).

Due to wave scattering or kinematic interaction effects, the support motions are generally different from the free-field motions. Nonetheless, a number of studies manifest that the foundation-input motions can approximately considered equal to the free-field motions based upon the concept that the effects of the presence of the pile foundation on the support motions or seismic wave scattering are expected to be insignificant if the dominant seismic wave lengths are much larger than the horizontal dimension of the foundations (Fenves et al., 1992).

At the Cairo area, the approximate shear wave velocity is 183 m/s (600 ft/s) and a typical dimension of the pile foundation is 18.3 m (60 ft) resulting in the prediction that wave scattering effects is important for periods of vibration less than 0.1 sec. Since this vibration period is small enough that wave scattering effects can be neglected for this structure, the free-field motions were used as the input motions to the bridge system.

As a result of the site response analyses, three components of ground motions having approximate peak acceleration of 0.7g for horizontal directions and 0.5g for vertical direction were obtained and used as input motions at base of the structure in the time-history analyses.

RESULT SUMMARY AND CONCLUSION

Dynamic characteristics in the form of modal periods of the bridge structure for different foundation models are listed in Table 1. Examples of modes of vibration are also depicted in Figure 4. The seismic-induced forces at the truss bearings and impact force, which is referred to the force experienced at the truss bearings due to their gapping and stiffening characteristics as previously described, as well as reactions at selected piers are presented in Table 2. Results from the nonlinear time-history analyses of all three models can be summarised as follows.

1. Modelling the foundation causes the period of the structure to increase. The modal periods of the bridge were elongated by 5-30% when the foundation flexibility was considered.

2. The more flexible support conditions increase the periods and maximum displacements of the structure, resulting in the smaller forces experienced at most structural parts of the bridge but larger impact forces at the truss bearings. The analysis results show a trend of increasing impact force at the expansion bearing as the support conditions become more flexible.

3. By comparing the results from model 2 & 3, it can be concluded that whether or not the PSPI or group effect is taken into account does not significantly affect the overall dynamic characteristics of the bridge system. The difference in modal periods between model 2 and model 3 is between 5-10%. Differences in forces vary with a tendency to decrease for model 3.

These results suggest that the fixed-base model cannot represent the dynamic characteristics of the overall bridge structure, thus indicating the need to include the modelling of foundation characteristics in seismic analyses of an entire bridge system. The results also indicate modest differences in dynamic characteristics and bridge responses between model 2 and model 3 implying that the PSPI or group effect could for all practical purposes be neglected.

ACKNOWLEDGEMENTS

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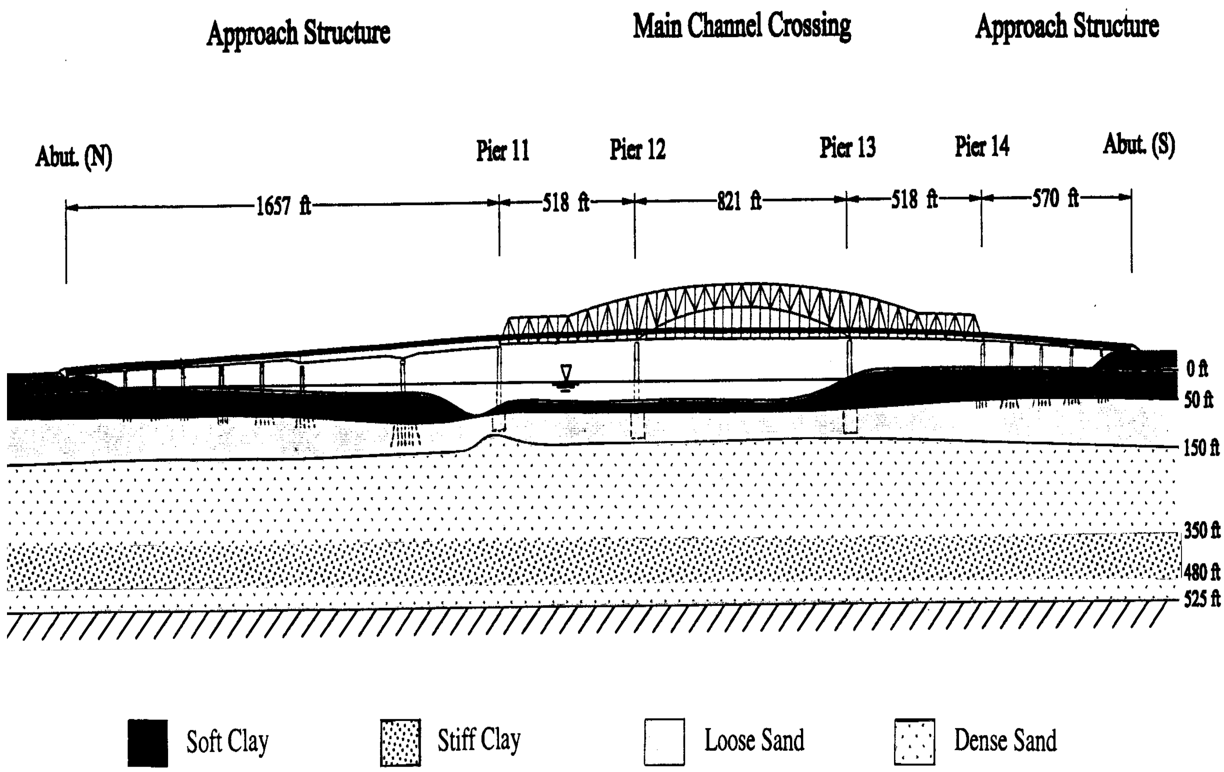


Figure 1: Bridge elevation

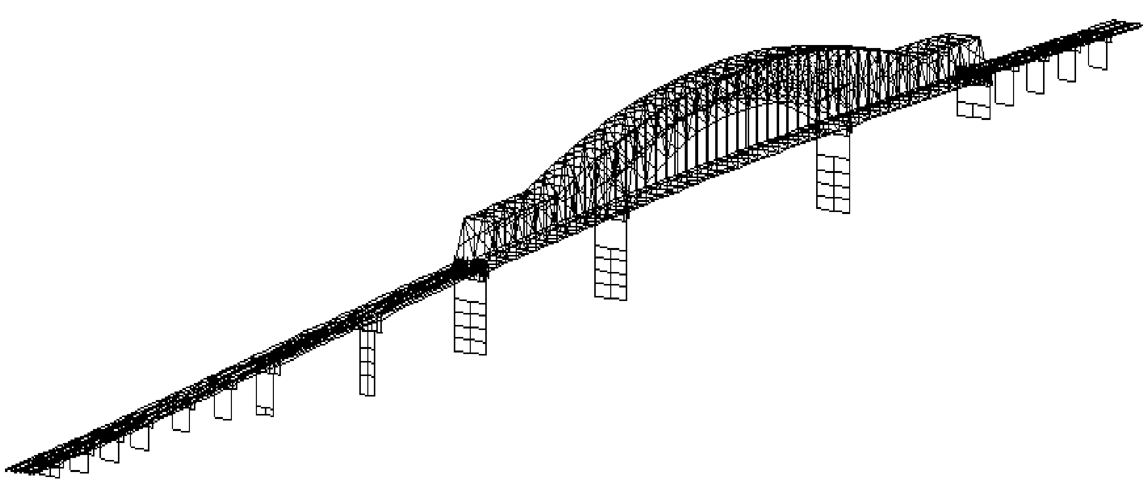


Figure 2: Structural model of the bridge

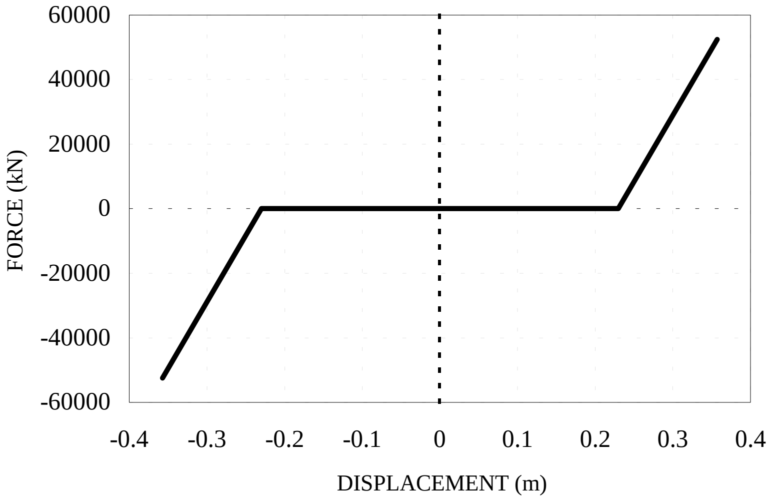
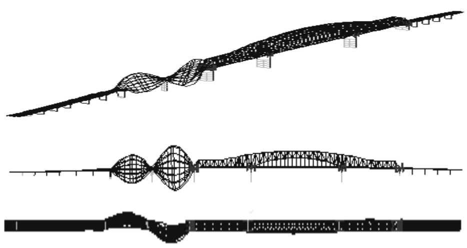
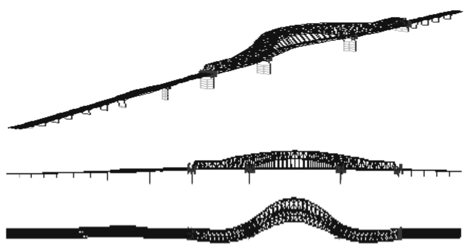


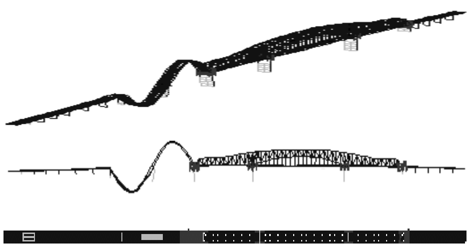
Figure 3: Idealised force-displacement relation for truss bearings



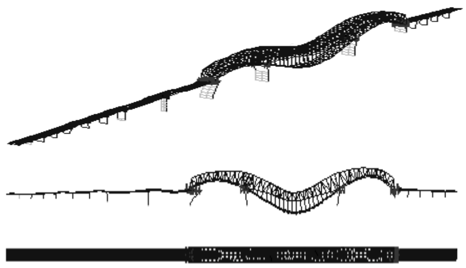
a) Torsional mode



b) Transverse mode



c) Vertical mode



d) Vertical mode

Figure 4: Examples of modes of vibration

Table 1: Dynamic Characteristics (modal periods) of three models

Mode	Periods (second)		
	Model 1 (Fixed-base)	Model 2 (without PSPI)	Model 3 (with PSPI)
1	2.272	2.718	2.873
2	2.185	2.271	2.559
3	1.653	2.160	2.270
4	1.520	1.526	1.977
5	1.498	1.492	1.528
6	1.342	1.239	1.486
7	1.203	1.201	1.238
8	1.188	1.186	1.198
9	0.993	1.021	1.185
10	0.968	0.976	1.030
11	0.910	0.957	0.978
12	0.870	0.940	0.968

Table 2: Seismic response of selected members

Member & Location	Model 1	Model 2	Model 3
At expansion bearing			
Axial force (kN)	8,238	6,285	6,828
Transverse shear (kN)	16,364	14,167	14,162
Transverse moment (kN-m)	21,243	14,758	18,625
Impact force (kN)	50,129	61,249	63,562
At fixed bearing			
Axial force (kN)	27,137	24,953	24,064
Longitudinal shear (kN)	15,670	16,257	14,781
Transverse shear (kN)	18,472	15,265	14,420
Transverse moment (kN-m)	33,182	32,300	26,365
At Pier 1			
Axial force (kN)	7,957	6,347	6,156
Longitudinal base shear (kN)	17,143	11,458	12,775
Transverse base shear (kN)	10,471	9,936	7,837
At Pier 9			
Axial force (kN)	27,671	35,121	31,857
Longitudinal base shear (kN)	10,577	9,768	10,012
Transverse base shear (kN)	48,723	42,705	10,052
At Pier 14			
Axial force (kN)	58,847	19,883	19,531
Longitudinal base shear (kN)	15,630	16,053	14,620
Transverse base shear (kN)	52,179	31,096	36,438