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SYNOPSIS

A study related to the experimental data obtained from the full-scale tests of Meloland Road Overcrossing is presented. The major objective of this study is to demonstrate that the soil-structure interaction effects play a significant role in the dynamic response of the structure. Initially, the results from the static deformation of the bridge under the quick-release loads are presented. These results were obtained by applying a special integration technique on the experimental acceleration records. Then a preliminary analytical model of the bridge is fitted to the experimental data in order to evaluate the values of the soil-structure springs along with other parameters of the bridge. Finally, a Finite Element model for the foundation of the pier was developed in order to obtain the load- (or strain-) dependent values of the pier foundation stiffnesses. These values are compared to those obtained from the bridge model.

INTRODUCTION

The Meloland Road Overcrossing, shown in Fig. 1, is a two-span reinforced concrete box girder bridge with monolithic abutments. Both the abutments and the single-column central pier are supported with pile foundations. The bridge is located within a quarter of a mile of the Imperial Fault near El Centro, California and, because of its importance and nearness to a very active fault, is heavily instrumented with strong motion accelerographs. During the 1979 Imperial Valley earthquake, this structure was strongly shaken and a large suite of strong motion records were collected for the structure. Since that time, a number of researchers (Douglas et al., 1984; Levine and Scott, 1989; Werner et al., 1985 and 1987) have investigated the response of this structure to that earthquake and its soil structure interaction characteristics. An ambient vibration survey was carried out by Gates and Smith (Gates and Smith, 1983) in 1980, and full-scale quick-release static and dynamic field tests were conducted in May of 1988 (Douglas et al., 1990).

The shaking amplitudes achieved in these tests were representative of earthquake motion. Two levels of quick-release initial ram loads were used, 21.2 kips and 141 kips. In addition to the complete instrumentation of the bridge deck during these experiments, measurements were made to estimate the extent to which the soils in the approach fills and central pier foundation participated in the response of the structure. The field test measurements were used for the direct calculation of the dynamic properties of the structure, such as natural frequencies and mode shapes. Recently, at the University of Nevada, Reno, an algorithm was produced for integrating accelerograms obtained from quick-release experiments to velocity and displacement records (Douglas et al., 1990). In addition to producing the displacement time histories, the algorithm provides reliable estimates of the offset or release displacement of the structure; and, therefore, it was used for the determination of the static deformation of the bridge under the initial quick-release loads from the acceleration records obtained during the experiment (Douglas, Maragakis & Nath, 1990).

The main objectives of this paper are the following: (1) to provide results from the full-scale dynamic tests of the bridge that demonstrate the effects of soil-structure interaction, (2) to present recent studies to fit a preliminary analytical model to the experimental mode shapes and natural frequencies obtained from the high-amplitude quick-release field tests in order to evaluate the soil-structure interaction springs along with other parameters of the structure, and (3) to present the results of an analytical Finite Element model which was used to determine the spring values of the pier foundation.

REPRESENTATIVE MELOLAND STATIC DISPLACEMENT RESULTS

As was mentioned in the introduction, an integration algorithm for quick-release accelerograms was recently developed at the University of Nevada, Reno, which allows for the determination of the displacement time histories as well as for the reliable estimate of the initial release displacement of the structure. Details about this algorithm are provided in Douglas et al., 1990. This integration technique was applied on all of the acceleration records obtained from the full-scale tests of the Meloland Road Overcrossing, thus allowing the determination of the static displacements of the bridge under the initial quick-release loads. Some representative results of this analysis are provided below.

Figures 2 and 3 show the release configuration of the hammerhead, the central pier, and the pier foundation at low and high amplitudes respectively. The foundation displacements are evidence of the presence of significant soil-structure interaction. By comparing the two figures, it can be seen that the pier is in single curvature bending at low amplitude while it is in double curvature bending at high amplitude. This is a strong evidence of the nonlinear behavior of the structure.

Figure 4 shows the transverse static displacements (positive to the West) of the south approach fill and the bridge deck caused by the low amplitude 21.2-kip ram load. The approach fill measurements were taken at roadway level at stations 1 and 2. The bridge deck measurements at stations 3 through 13 were taken on top of the concrete barrier rail at the edges of the deck 17 inches above pavement level. All stations are 20.67 feet apart.

Figure 5 shows the vertical displacements of both the West and East edges of the bridge deck and the approach fill. Positive numbers indicate up. As can be seen from this figure, there is a net upward displacement of the bridge deck because of the difference in the amplitudes of the West and East side vertical displacements. This makes sense, of course, because there was a 15-kip upward force component applied by the ram. The movements of the approach fill in both Figs. 4 and 5 are a result of soil-structure interaction.

Finally, Fig. 6 shows the extent of soil involvement at the central pier foundation. The lower curve shows the transverse release displacement configuration out to 47.5 feet away from the edge of the upper pile cap. It can be seen that significant involvement occurs out to a distance of 10 or 15 feet. This is about one pile cap diameter, which is 15 feet in width. Also shown in the curve is a plot of the first arrival times for the accelerograms in question. The slope of the curve is about 1250 feet per second for the apparent wave velocity. As these were first arrivals, it is presumed that they measure the p wave velocity at these close distances. A shear wave velocity of about one-third of this p wave velocity would be consistent with the soft soils at the site.

DESCRIPTION OF THE BRIDGE FINITE ELEMENT MODEL

The bridge was modeled analytically by using the SAP-90 computer program (Wilson and Habibullah, 1989), which is a general purpose linear finite element program that can perform static and dynamic analyses. Modal and static analyses were performed in this study. The bridge's beam and column elements were modeled as three-dimensional translational and rotational soil-structure interaction springs, which are shown in Fig. 7. Nodes 1 through 21 (Fig. 7) of the deck correspond to the actual experimental stations where measurements were taken during the field tests. This allows for the direct comparison of the measured and calculated static and modal deformations of the bridge deck. For a more accurate representation of the torsional vibrations of the bridge deck, the torsional mass moment of inertia terms for the deck were included in the mass matrix of the structure using the lumped-mass method.

RESULTS FROM THE MODEL

The model which is presented herein represents a preliminary model of the Meloland Road Overcrossing obtained by fitting the SAP-90 model discussed above to four experimental dynamic mode shapes and natural frequencies (Douglas et al., 1990). The four experimental modes used to identify the parameters of the bridge model represent the 3.22 hz first transverse mode, the first and second vertical modes at 3.44 and 4.65 hz respectively, and the first 6.61 hz torsional mode. The first and second experimental vertical modes are represented by the data points in Figs. 8 and 9. The first experimental transverse deck modal displacements are shown in Fig. 10, and the associated deck rotations are shown in Fig. 11. The final mode shape used in the model was the experimental first torsional mode. The deck rotations of this mode are shown in Fig. 12.

The approach used to fit the analytical SAP-90 model to the above suite of data was to define a merit or criterion function consisting of the sum of the weighted squared errors for all experimental natural frequencies and modal displacements and rotations discussed above. The weighting factors on all of the mode shape data were defined so that each modal data point contributed equally to the criterion function. The natural frequency data was weighted to 10 relative to each individual modal displacement point except for the torsional mode which was assigned a weighing factor of 5. This merit or criterion function was then minimized to identify the significant structural parameters. For this problem, the structural variables which were included are: (1) all of the foundation soil-structure interaction springs shown in Fig. 7 with the exception of the longitudinal springs at the abutments and the pier rotational spring about the y axis, (2) the effective moments of inertia of the deck and pier, and (3) the effective torsional constant for the deck. These structural parameters are shown in Tables 1 and 2. The number of variables was further limited by requiring that the model be symmetric. This symmetry constraint required that the corresponding spring constants at the two abutments be the same and that the pier foundation springs in the two coordinate directions be the same.

A pattern search minimization or optimization procedure to identify the structural parameters from the criterion function was implemented on an IBM PS2 Model 70 386 microcomputer. This pattern search method was developed based on the work of Hooke and Jeeves, 1961.

The structural parameters for the model which were identified are shown in Tables 1 and 2. In Table 1, the numerical values shown are the identified parameters associated with this preliminary model. The longitudinal spring at the abutments was assigned the fixed value

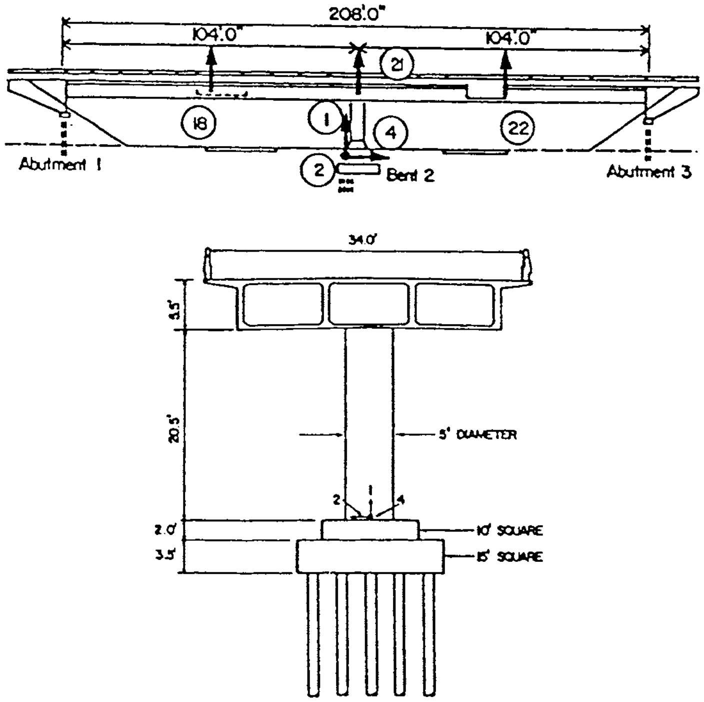

Figure 1: Elevation and Section of the Meloland Road Overpass (Rojahn et al., 1982)

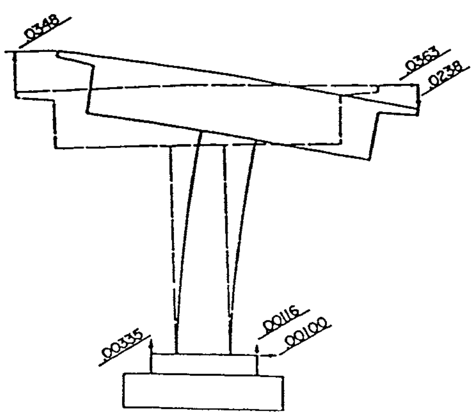

Figure 2: Release Configuration of Pier Section Including Foundation Displacements Caused by the Low Amplitude 21.2-kip Ram Load

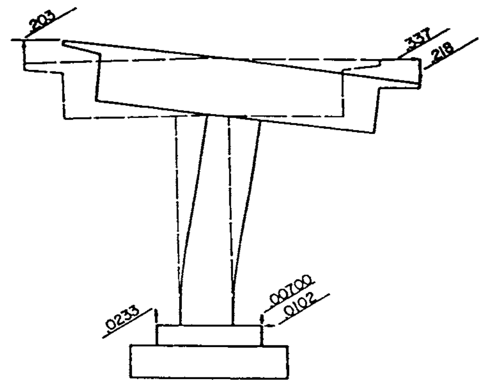

Figure 3: Release Configuration of Pier Section Including Foundation Displacements Caused by the High Amplitude 141-kip Ram

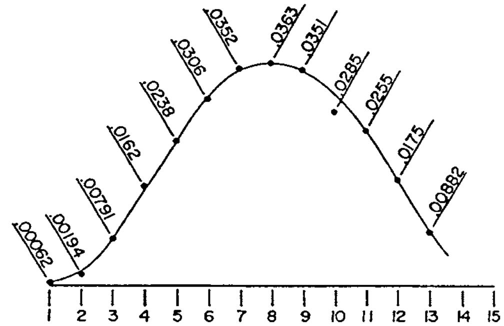

Figure 4: Transverse Bridge/Roadway Release Displacement Configuration Caused by the Low Amplitude 21.1-kip Ram Load

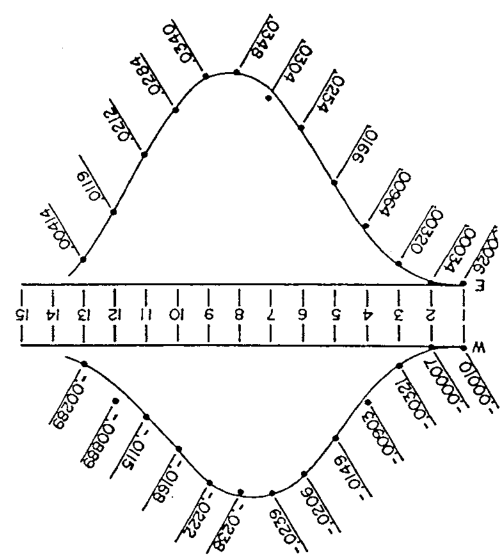

Figure 5: Vertical Release Displacement Configuration at Both the East and West Sides of the Bridge/Roadway Caused by the Low Amplitude 21.2-kip Ram Load

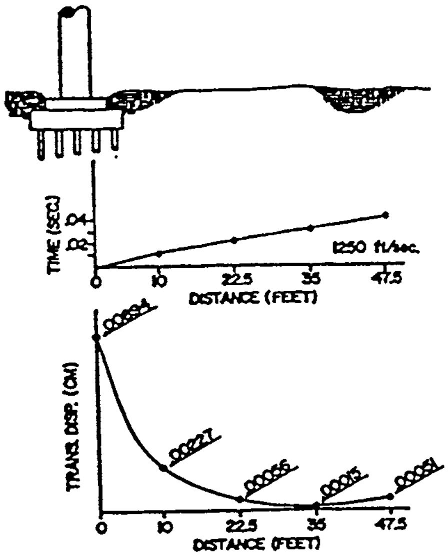

Figure 6: Vertical Release Displacement Configuration of the Soils Adjacent to the Pier Foundation Caused by the high Amplitude Ram Load and the Associated Travel Time Curve Developed from the First Arrival Times

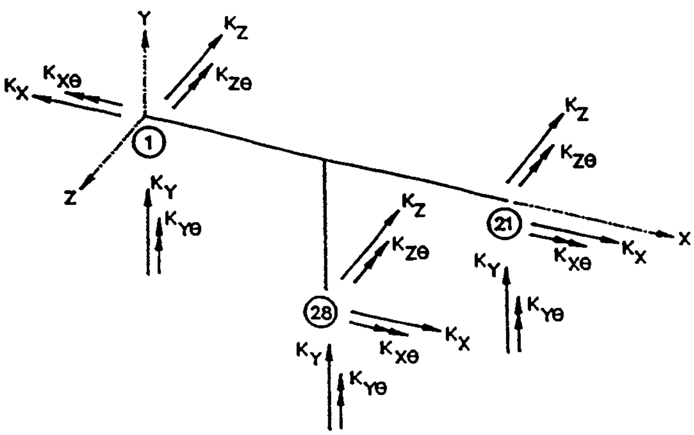

Figure 7: Finite Element Model of the Bridge

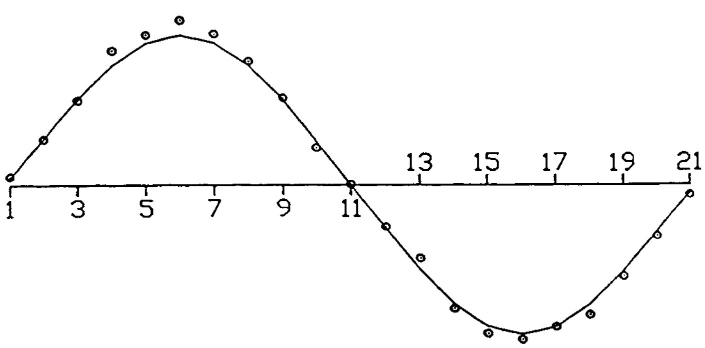

Figure 8: First Vertical Mode (3.44 Hz)

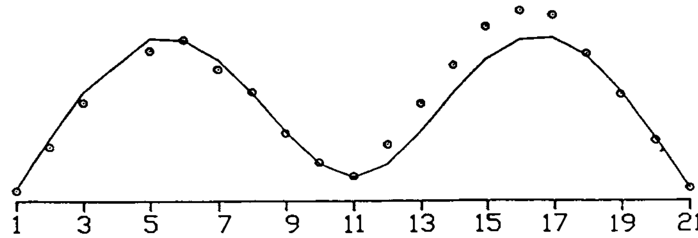

Figure 9: Second Vertical Mode (4.65 Hz)

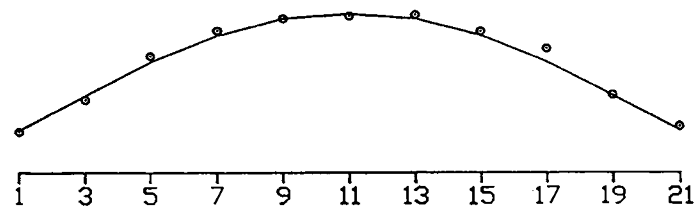

Figure 10: Deck Translations Associated with 3.22 Hz Transverse Mode

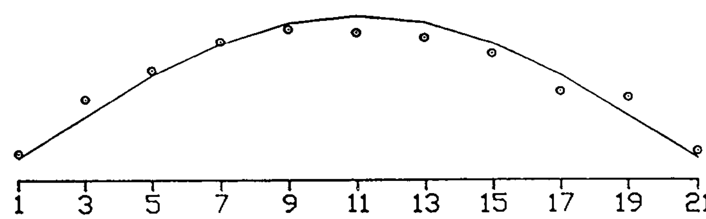

Figure 11: Deck Rotations Associated with 3.22 Hz Transverse Mode

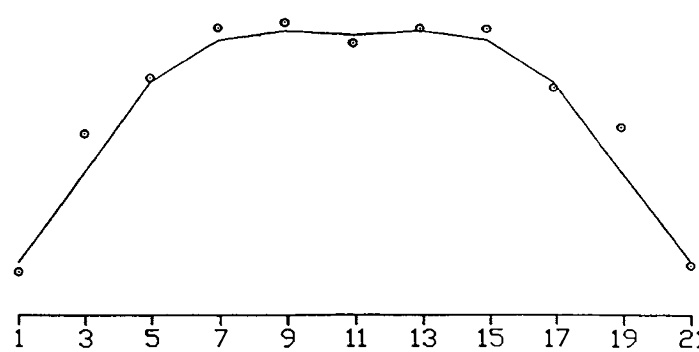

Figure 12: Deck Rotations Associated with 6.61 Hz Torsional Mode

shown, and the rotational spring about the y axis at the pier was also given the fixed value shown. This spring was calculated by computing the rotational stiffness which would be consistent with the lateral stiffness of the individual piles of this foundation. Table 2 shows the gross geometric section properties for the deck and the pier column, and the fraction of the values used in the optimized model. The torsional constant, J, for the deck was obtained by solving the thin wall multicell St. Venant torsion problem. It should be recalled that this theory assumes that no warping restraint of any cross section in the deck is present. As will be demonstrated in the next section, this does not represent the actual situation in this structure.

It should be noted that the identified soil-structure interaction springs in the model are, in general, very much larger than those reported by Levine and Scott, 1989. Selected springs from Table 1 were also compared with those of Crouse (3) and found to be in general agreement with his.

The first four entries in Table 3 give the natural frequencies associated with the identified model and the relevant experimental values. The solid lines in Figs. 8 to 12 represênt the computed mode shape data for the first four modes used in the optimization process. As can be seen from the tables and the figures, the agreement between the model and the experimental data is quite good.

TABLE 1. Foundation Soil-Structure Interaction Springs

		Spring Value	
SPRING DIRECTION		$K_{1,21}$	K_{28}
	X	1.00 E8	5.88 E7
	Y	6.78 E7	1.23 E8
	Z	2.09 E7	5.88 E7
	$X\theta$	1.16 E10	9.72 E9
	$Y\theta$	8.92 E9	2.51 E9
	$Z\theta$	1.32 E9	9.72 E9

TABLE 2. Geometric and Material Properties

PROPERTIES	GROSS SECTION	% USED
I_{zz}	230 #/ft^4	70 %
I_{yy}	4140 #/ft^4	46 %
J	614 #/ft^4	67 %
I_{col}	30.7 #/ft^4	42 %
E = 5.29 E8 #/ft^2		
v = 0.15		

TABLE 3. Natural Frequencies

MODE SHAPE	MEASURED	CALCULATED
Transverse	3.220	3.222
1st Vertical	3.441	3.469
2nd Vertical	4.653	4.620
Torsional	6.610	6.563
High Order Transverse	22.10	27.65

FOUNDATION FINITE ELEMENT MODEL DESCRIPTION AND RESULTS

In order to develop independent values for the foundation springs at the central pier, the three-dimensional finite element analysis program IMAGES-3D was used. The 25 wooden pier foundation piles, spaced on a square grid at 3 feet on centers, were modeled with three-dimensional beam elements; while the soil medium was modeled with three-dimensional brick elements. The half plan view of the foundation model is shown in Fig. 13, and a section through the centerline is shown in Fig. 14. The soils below the foundation were modeled with six layers (Fig. 14). The depth of each layer was taken from the foundation soil profile information provided by Norris, 1988. The sixth layer below the piles was included to model the pile plunging effects. Soil elements were also included to model the soil Passive Resistance at the pile cap level to the right and left of the foundation. The pile cap was modeled with three-dimensional brick elements. The horizontal limits of the soil elements extended out 20 feet on each side of the pile elements.

This model was used for the estimation of the load- (or strain-) dependent lateral, vertical, and rotational stiffnesses of the foundation. In each case, a curve of the foundation stiffness against the load was developed. The curve for the lateral stiffness is shown in Fig. 15. To develop this curve, Poisson's ratio was taken to be equal to 0.5 for the saturated layers. For the unsaturated layers, it was estimated based on values provided by Bowles, 1982. Then, for a certain load value, the modulus of elasticity of each layer was initially assumed to be equal to the static value provided by Bowles, 1982. The foundation model was then used to calculate the strain of each layer. Based on the strain values and a strain-modulus of elasticity curve, a new value of the modulus of elasticity for each layer was estimated. The iterations continued until sufficient convergence for the modulus of elasticity of each layer was achieved. In this case, the deflection of the pile cap was calculated and it was plotted versus the corresponding load value. This procedure was repeated for several load values to produce the curve shown in Fig. 15. Table 4 shows a

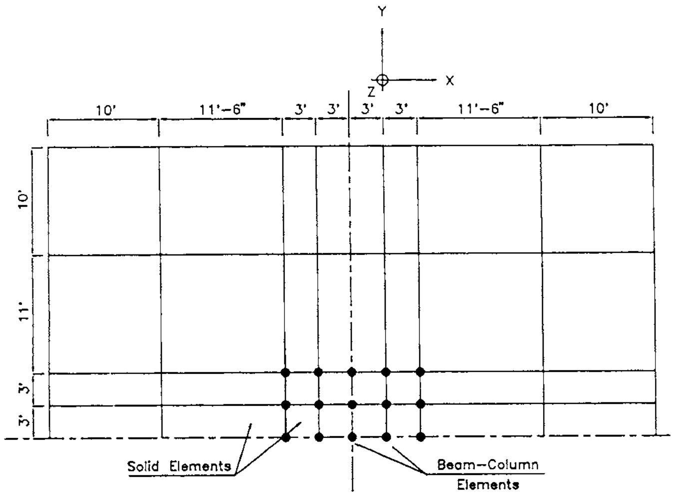

Figure 13: Half-Plan View of Pier Foundation Model

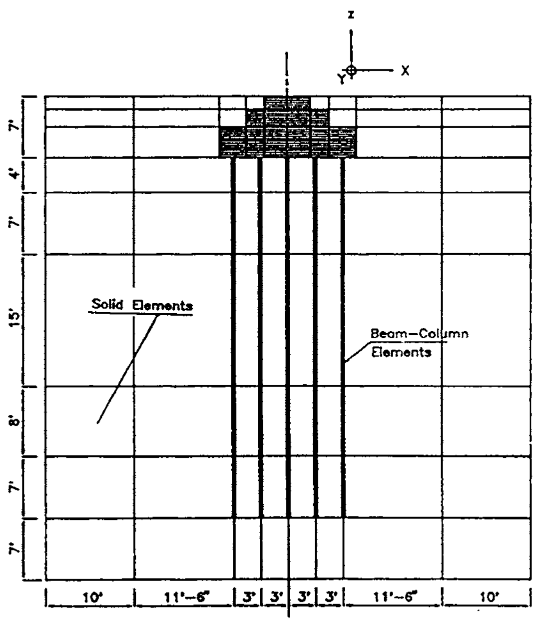

Figure 14: Section of Pier Foundation Model

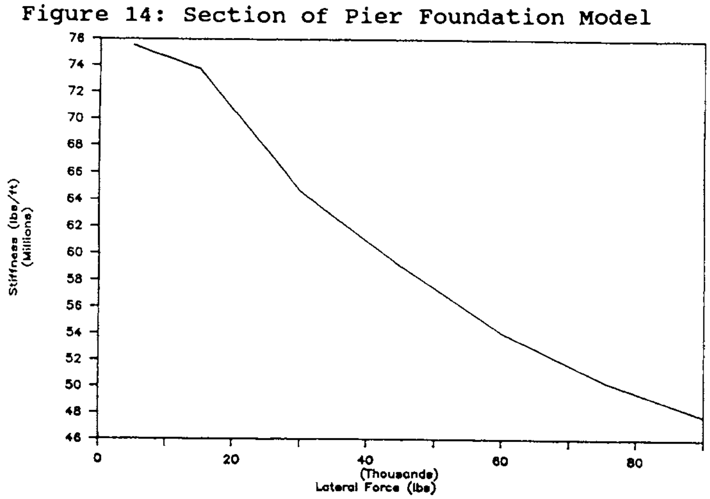

Figure 15: Lateral Foundation Stiffness Versus Lateral Load

TABLE 4. Comparison of Foundation Springs from Bridge Model and the Foundation Model

SPRINGS	BRIDGE MODEL	FOUNDATION MODEL
Lateral (#/ft)	5.9 E7	6.6 E7
Vertical (#/ft)	1.2 E8	1.06 E8
Rotational (#*ft)	9.7 E9	4.23 E9

comparison of the Foundation Spring values from the Bridge Model and the Foundation Model at the same loading level.

CONCLUSIONS

In the first part of this paper, some results related to the static deformations of Meloland Road Overcrossing under the initial quick-release ram loads have been presented indicating the presence of strong soil-structure interaction. These results were obtained by applying a special integration technique on the accelerograms obtained from the full-scale tests of the Meloland bridge performed in May 1988. Then a preliminary model of the bridge was developed by fitting it to the mode shapes and frequencies obtained from the quick-release field test data. This model produced values for the soil-structure interaction springs at the abutments and the central pier foundation. Finally, a Finite Element model of the pier foundation was developed and used to find the load amplitude-dependent values of the foundation springs. At the level of the experimental loads, these values compared reasonably well to those predicted by the bridge model.

ACKNOWLEDGEMENTS

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