

## MAXIMUM AMPLITUDE DYNAMIC TESTS OF A HIGHWAY BRIDGE

Bruce M. Douglas (I)  
James A. Richardson (II)  
Presenting Author: J. Richardson

### SUMMARY

The results of a series of maximum amplitude lateral dynamic tests on a reinforced concrete highway bridge are presented. The largest lateral loads applied were about one and one-half times the design earthquake loads and resulted in maximum accelerations of the deck of about ten percent of gravity. The results indicate that the overall rotation of the pile foundations are the major contributor to the soil/structure interaction phenomenon rather than the lateral stiffness of the pile groups. These in situ tests also strongly suggest that the bridge's neoprene elastomeric bearings stiffened substantially with age in the one to four percent strain range.

### INTRODUCTION

In May 1982, a series of large amplitude dynamic tests were conducted on the Rose Creek Interchange which is located about ten miles southwest of Winnemucca, Nevada. This bridge is a four hundred foot long reinforced concrete box girder bridge, and has five spans supported on single column piers. At each abutment, the deck is isolated from the abutment by five neoprene elastomeric bearing pads having a dimension of 3 x 10 x 14 inches and a durometer hardness of 52 at the time they were shipped to the job site in 1971. Additional details regarding the bridge may be found in references 1, 2, 3, 4 and 5. The bridge is pile founded (Fig. 3) in order to bypass a ten foot layer of relatively soft clay material.

### STATIC AND DYNAMIC TESTS

This series of dynamic tests were conducted by using the pullback and quick release method of excitation (3,4). One hydraulic ram was used at each of the bridge's four piers to deform the structure statically in the lateral direction. The initial release deflections of the deck were measured with dial gages at both abutments and the centerline of the bridge (Table 3). Free vibration was induced by suddenly releasing the fluid from all the hydraulic jacks. A previous series of quick release experiments were performed on this bridge (1,2) where the maximum lateral loads were 50,000 lbs. or about one quarter of the design earthquake load. In this series of experiments the largest total lateral load applied was about one and one-half times the design earthquake load. These were applied monotonically in a series of four increments indicated in all the tables by 9T, 25T, 40T and 56T respectively. These indicate the individual ram loads in tons. The rams were inclined at 45° so that the total lateral release loads for the four lateral test cases were respectively 51, 141, 226, and 317 kilo pounds.

---

(I) Professor of Civil Engineering, University of Nevada, Reno, Nevada, USA

(II) Graduate Research Assistant, University of Nevada, Reno, Nevada, USA

In contrast to the previous experimental work on this bridge (1), where modal displacement data was obtained for the bridge deck only, additional dynamic response measurements were made in this new series of tests to describe the detailed experimental dynamic response of the footings. All pier foundations were excavated in order to make direct measurements of the lateral foundation response. In addition, two vertical acceleration response measurements were made at each side of the eight foot wide column (Fig. 3) in order to be able to directly measure the foundation rotation effects.

Figures 1 and 2 illustrate the experimental first and fourth modes respectively for the 56T load case. The solid dots represent the measured modal displacements of the deck, the solid squares represent the lateral modal displacement of the pile caps and the solid triangles represent the sum of the two measured vertical modal displacements obtained eight feet apart at each side of the bridge piers (Fig. 3). The modal rotation of the pile cap may be obtained by dividing this sum by eight feet. The open samples in Figs. 1 and 2 indicate the corresponding quantities obtained from the optimized analytical model which will be discussed in the next section. As is evident in Figs. 1 and 2, the fit between the experimental and optimized analytical results is very good.

In the first series of experiments (1) the elastomeric bearing pads were "frozen" to the abutments by sand and gravel that had accumulated over the years. For this new series of experiments the elastomeric bearing pads were washed clean to insure that they were the only mechanism transmitting loads from the deck to the abutment. The dial gage measured deflections of the bearing pads and the deck at the center of the bridge, just prior to quick releasing the rams, are given in Table 3 for each load case.

Estimates of the lateral displacements of the foundation at the instant the rams were released are given in Table 4, and estimates of the upward/downward displacement of the outer piles (Fig. 3) for each pier foundation is given in Table 5. These quantities were estimated by observing that the response of the bridge, when all four rams were equally loaded, was nearly unimodal in the fundamental mode (3,4); and then scaling the measured centerline bridge deck displacement in Table 3 in proportion to the measured fundamental modal displacements for that load case.

#### SYSTEM IDENTIFICATION METHODOLOGY AND RESULTS

The basic usefulness of full scale lateral dynamic tests on bridges is to identify the structural parameters which are significant in determining the seismic response of the bridge. One of the important observations made previously (1,2) is that these tests are "in effect" full scale tests of the foundation (boundary) elements which support the bridge. The analytical model used for the system identification work is described in Ref. 1. The system identification method used herein, which is an extension and generalization of the method used in Ref. 1, is described by Reid (6). Sunder and Sanni (7) have also presented a related generalization of the method described in Ref. 1.

The basic system identification results obtained for this series of dynamic tests are presented in Table 1. The experimental modal information used for all system identification results given in this table were the modal dis-

placements and natural frequencies of modes 1 and 4. The frequency of mode 2 was not sufficiently distinct from mode 1 to allow its simple extraction, and both modes 2 and 3 were not excited at a sufficiently high amplitude to define them reliably. The first eight rows of data in Table 1 give the experimental lateral (subscript T) and rotational (subscript R) springs determined for the four load cases labeled 9T through 56T (see Fig. 3). The rotation spring was located at the top of the pile cap because that is the level at which the foundation rotation was measured. The numerical subscript in these first eight rows indicate the pier number.  $K_N$  and  $K_S$  indicate the total lateral stiffness of the five elastomeric bearing pads at the north and south end of the bridge respectively. The associated static displacements of the five pads at the instant of quick release is given in Table 3.  $I_{eff}$  is the fraction of the gross transverse moment of inertia of the deck which is effective in the analytical model.

In addition, lateral ambient vibration data for modes 1 and 4 collected by Gates and Smith (8) were used to identify the related ambient level properties. The order of magnitude of displacements associated with this data is at least an order of magnitude smaller than that associated with the quick release data. In order to extract the lateral and rotational stiffnesses from this data, since no foundation level measurements were available, it was necessary to "slave" the rotational and lateral foundation stiffnesses together. The "slaving" ratios were obtained by averaging all the other rotational vs lateral stiffness ratios obtained from all the quick release load cases. Reid (6) has shown that it is not possible to independently extract the lateral and rotational stiffnesses of the foundation elements in the absence of modal foundation response measurements.

When comparing the foundation stiffnesses presented here with the results in Ref. 1 it should be remembered that the foundation stiffnesses in Ref. 1 represent an "effective" lateral stiffness obtained by incorrectly assuming that the foundations did not rotate. If the "slaving" ratios noted above are used to find this "effective" stiffness from the 9T load data in Table 1, reasonable agreement between the results in Ref. 1 and the 9T load case in Table 1 are obtained. Note that the experimental loads in these two cases were nearly equal.

In a correlation study (2), a large empirical cyclic correction factor was invoked following Prakash (9) and Davisson (13) which gave an apparent agreement between the geotechnically determined foundation stiffnesses and the experimentally determined stiffnesses. Based on the results of this experimental series involving 230 separate quick release excitations and in excess of a thousand significant cycles of vibration, no evidence of a cyclic degradation of the pile group foundation stiffnesses was observed (3,4). Removal of the cyclic correction factor from the geotechnically determined foundation stiffnesses given in Ref. 2 give lateral foundation stiffnesses in acceptable agreement with those shown in Table 1.

Finally the distribution of deflection of the piers at deck level caused by a unit load acting at the top of the column is shown in Table 2. The significant thing to notice is that foundation flexibility contributes very significantly to the overall column flexibility and that foundation rotation contributes the major share of the total foundation caused flexibility. It should be further observed that these foundation flexibilities have a

substantial impact on the distribution of seismic forces.

#### STIFFNESS OF ELASTOMERIC PADS

By using the stiffness of the neoprene elastomeric bearings obtained in Table 1 it was possible to compute the shear modulus of the bearings which were eleven years old at the time these bridge tests were conducted. The associated maximum shear strains of pads at the instant of quick release was calculated from the data in Table 3. The results are plotted in Fig. 4 as the open triangles and circles. The fundamental frequency of the bridge varied slightly over the range of test amplitudes, but was on average about 2.5 hz. The above shear modulus data on the eleven year old bearings was compared to data obtained by Imbsen and Schamber (10) for new neoprene bearings having the same durometer hardness and bearing manufacturer at a frequency of 3 hz. These results are shown by the squares and hexagons in Fig. 4. From Fig. 4 it is clear that the aged bearings have a substantially higher shear modulus than the new bearings in the strain range of one to four percent. Tests conducted on fifteen year old small flexible neoprene couplings suggest a similar increase in stiffness with age (11). In outdoor tests on twenty year old neoprene from pipe gaskets a similar increase in stiffness was observed (12).

#### CONCLUSIONS

1. Full scale dynamic tests of bridges can in effect be full scale dynamic tests of complete foundations. In this paper the experimental lateral and rotational stiffnesses of four pile group foundations and their representative displacements have been presented. These experimental values should be useful for geotechnical correlation studies.

2. Evidence that the shear modulus of neoprene bridge bearings may increase substantially with age has been inferred from in situ dynamic testing of a highway bridge.

3. No evidence of a cyclic degradation of foundation stiffnesses was observed over the very large number of significant cycles of vibration involved in this series of high amplitude dynamic tests.

4. From a detailed analysis of the data obtained from these tests it is clear that the soil/structure interaction phenomenon of the pile foundations contribute very significantly to the distribution of seismic loads. Without a proper assessment of these parameters in the design phase, the distribution of the seismic loads could be in error by at least a factor of two (2).

#### ACKNOWLEDGMENTS

We wish to thank Dr. Ian Buckle for his helpful comments relating to bridge bearings and Dr. Mehdi Saïidi, our colleague, for his helpful comments and advice. A significant portion of this study was funded by NSF Grant CEE-8108124; however, the results and conclusions are those of the authors alone.

#### REFERENCES

1. Douglas, B.M., and W.H. Reid, "Dynamic Tests and System Identification of Bridges", Journal of the Structural Division, ASCE, Vol. 108, No. ST. 10, Oct., 1982, pp. 2295-2312.
2. Douglas, B.M., and G.M. Norris, "Bridge Dynamic Test Implications for Seismic Design", Journal of Technical Topics in Civil Engineering, ASCE, Vol. 109, No. 1, Apr. 1983, pp. 1-22.
3. Douglas, B.M., M. Saïidi, and J. Richardson, "Dynamic Response of Highway Bridges", Proceedings U.S.-P.R.C. - Bilateral Workshop on Earthquake Engineering, Harbin, China, Aug. 30, 1982, pp. B-7-1 to B-7-15.
4. Douglas, B.M., M. Saïidi, J. Richardson, and J. Hart, "High Amplitude Dynamic Tests and Implications for Seismic Design", Proceedings 15th Joint Meeting of U.S./Japan Panel on Wind and Seismic Effects, UJNR, May 17-28, 1983, 22 pp.
5. Saïidi, M., and J. Hart, "Nonlinear Seismic Response of Short Reinforced Concrete Highway Bridges", Proceedings of Eighth World Conference on Earthquake Engineering, July 21-28, 1984, San Francisco, CA., 8 pp.
6. Reid, W.H., "A Method of System Identification for Full Scale Structures Described by Complex Computer Models", M.S. Thesis, University of Nevada Civil Engineering Department, in preparation.
7. Sunder, S.S., and R. Sanni, "Foundation Stiffness Identification for Off-shore Platforms", Preprint of paper submitted to the Journal of Applied Ocean Research, July, 1983, 30 pp.
8. Gates, J.H., and M. Smith, "Verification of Dynamic Modeling Methods by Prototype Excitation", Cal Trans Report, No., 1982, 192 pp.
9. Prakash, S., Soil Dynamics, McGraw-Hill Book Company, Inc., New York, N.Y., 1981.
10. Imbsen, R.A., and R. Schamber, "Earthquake Resistant Bridge Bearings", FHWA/RD-82/165 Final Report, May 1983, 279 pp.
11. Case History Reprint, "15 Year Shelf Aging Test of Neoprene", Elastomers Notebook, E.I. Dupont DeNemours & Co., Inc.
12. Maylasian Research and Development Board, "Natural Rubber in Bridge Bearings", Regional Office, 204 South Spring Street, Cary, Illinois.
13. Davisson, M.T., "Lateral Load Capacity of Piles", Highway Research Record, No. 333: Pile Foundations, Highway Research Board 1970.

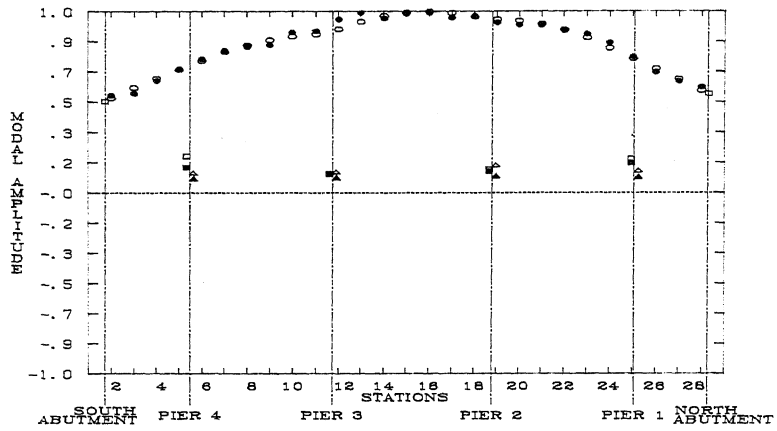


Fig. 1. Mode 1 for 56T Load Case,  $f = 2.32$  Hz.

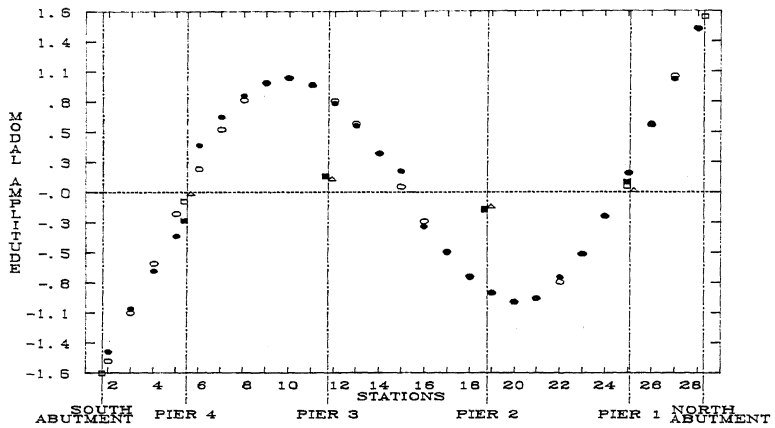


Fig. 2. Mode 4 for 56T Load Case,  $f = 4.88$  Hz.

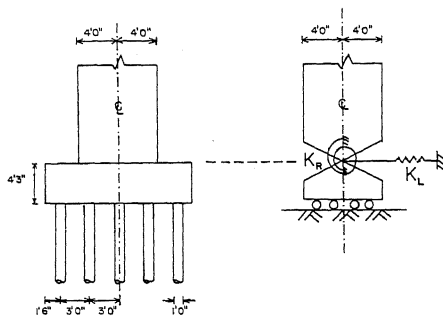


Fig. 3. Pile Foundation and its Associated Analytical Model.

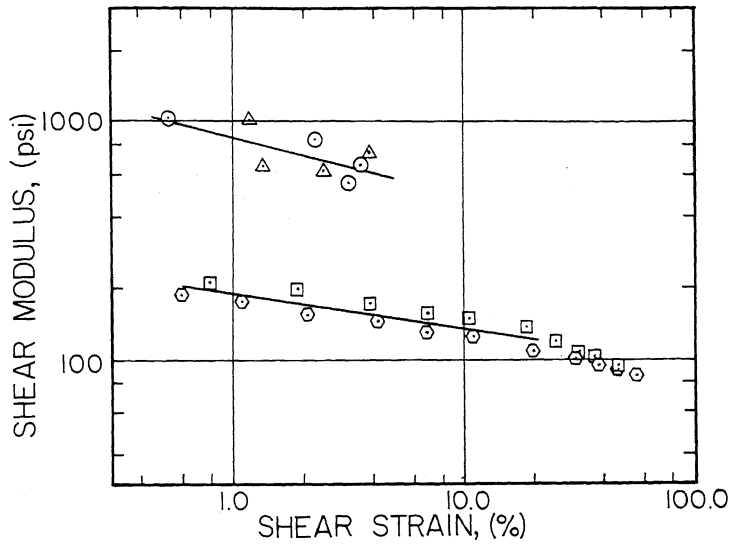


Fig. 4. Shear Modulus vs Shear Strain for Neoprene Bridge Bearings

TABLE 1. Bridge Properties From System Identification

Property	Load Case				
	Ambient	9 <sup>T</sup>	25 <sup>T</sup>	40 <sup>T</sup>	56 <sup>T</sup>
K <sub>T4</sub>	1.80	1.52	2.05	1.66	1.17
K <sub>R4</sub>	64,300	46,600	71,000	62,700	49,200
K <sub>T3</sub>	16.52	5.99	5.98	3.72	4.93
K <sub>R3</sub>	392,000	230,000	116,000	77,700	115,200
K <sub>T2</sub>	9.50	8.63	4.54	3.46	3.62
K <sub>R2</sub>	192,000	150,000	101,000	71,000	78,200
K <sub>T1</sub>	2.55	1.82	1.79	1.75	1.61
K <sub>R1</sub>	81,600	49,900	60,900	57,900	55,300
K <sub>S</sub>	.246	.238	.153	.145	.170
K <sub>N</sub>	.235	.244	.196	.130	.157
I <sub>eff</sub>	.602	.582	.620	.686	.603
E <sup>2</sup>	.081	.685	.142	.448	.202

K<sub>T</sub>, K<sub>N</sub>, K<sub>S</sub>, in units 10<sup>6</sup> lb/in  
 K<sub>R</sub> in units 10<sup>6</sup> lb-in

TABLE 2. Distribution of Transverse Pier Deflection Due to Unit Load Applied at Top

Load Case	Pier	Bending Deformation as Percent	Shear Deformation as Percent	Deformation Caused By Lateral Foundation Flexibility Percent	Deformation Caused By Rotation Foundation Flexibility Percent	Structural Deformation Percent Column 3 + 4	Foundation Caused Deformation Percent Column 5 + 6
9T	1	22	1.7	19.7	56.6	23.7	76.3
9T	2	47.1	3.6	8.9	40.4	50.7	49.3
9T	3	52.4	4.0	14.3	29.3	56.4	43.6
9T	4	20.4	1.5	21.9	56.2	21.9	78.1
25T	1	24.4	1.9	22.3	51.4	26.3	73.7
25T	2	36.9	2.8	13.3	47.0	39.7	60.3
25T	3	40.7	3.1	11.1	45.1	43.8	56.2
25T	4	27.1	2.1	21.6	49.2	29.2	70.8
40T	1	23.6	1.8	22.1	52.5	25.4	74.6
40T	2	29.7	2.3	14.1	53.9	32.0	68.0
40T	3	31.5	2.4	13.9	52.2	33.9	66.1
40T	4	24.4	1.8	23.9	49.9	26.2	73.8
56T	1	22.6	1.7	23.0	52.7	24.3	75.7
56T	2	31.5	2.4	14.2	51.9	33.9	66.1
56T	3	39.6	3.0	13.1	44.3	42.6	57.4
56T	4	19.6	1.5	27.5	51.4	21.1	78.9
A	1	30.4	2.3	19.5	47.8	32.7	67.3
M	2	52.2	3.8	9.0	35.0	56.0	44.0
B	3	66.5	5.0	6.6	21.9	71.5	28.5
I	4	25.1	1.9	22.8	50.2	27.0	73.0
E							
N							
T							

TABLE 3. Measured Release Displacements, (in)

Load Case	Bearing Pad South Abutment	Bridge Deck	Bearing Pad North Abutment
9 <sup>T</sup>		.023	.016
25 <sup>T</sup>	.041	.072	.067
40 <sup>T</sup>	.075	.114	.095
56 <sup>T</sup>	.117	.165	.106

TABLE 4. Estimated Lateral Release Displacements of Foundation Pile Caps, (in)

Load Case	Pier 1	Pier 2	Pier 3	Pier 4
9 <sup>T</sup>	.003	.002	.002	.003
25 <sup>T</sup>	.011	.009	.007	.007
40 <sup>T</sup>	.017	.014	.012	.012
56 <sup>T</sup>	.027	.020	.018	.024

TABLE 5. Estimated Vertical Release Displacements of Outer Piles in Foundation, (in)

Load Case	Pier 1	Pier 2	Pier 3	Pier 4
9 <sup>T</sup>	.003	.003	.003	.003
25 <sup>T</sup>	.009	.010	.009	.007
40 <sup>T</sup>	.013	.015	.010	.007
56 <sup>T</sup>	.020	.022	.020	.019