RESULTS OF AMBIENT VIBRATION TESTING OF BRIDGES

James H. Gates (I)
Monte J. Smith (II)
Presenting Author: James H. Gates

SUMMARY

This paper presents a summary of the experimental results from the ambient vibration testing of 57 typical bridges. Mode shapes and natural frequencies were determined for each bridge from the ambient data. These results were compared to the computed results from STRUDL models of the bridges. A system identification was performed on selected bridges using a complete quadratic procedure on the STRUDL model.

A comparison between ambient vibration results and higher force level results for two bridges, Meloland Road Overcrossing in California and Rose Creek Interchange in Nevada, is discussed.

INTRODUCTION

Ambient vibration testing is an economical means of testing structures for their dynamic response characteristics. This study involved the testing of 57 bridges by ambient vibration techniques. The bridges selected for this study represent a cross section of the types of bridges located in California. The results of this study can be applied in the modeling of bridges for dynamic and static analysis.

FIELD STUDY PROCEDURE

The field study used four two second period Kinemetrics Model SS-1 Ranger Seismometers; a Nimbus Instruments ES-6C Engineering Seismograph; a calibrator fabricated by the Caltrans Transportation Laboratory; and a Bruel and Kjaer Model 7003 FM tape recorder. The natural frequencies of each bridge were measured in three directions in addition to the transverse mode shapes. Bridges instrumented for strong motion also had their vertical mode shapes measured. The seismometers were used to gather all the velocity data by selectively placing them on the bridge structure. At each test set up, a base point was selected so that the mode shapes could later be extracted. The seismograph was used as an amplifier to the tape recorder to obtain a nominal one volt signal. The seismograph was modified to accept six channels of coaxial input and output. The calibrator was used to calibrate all the seismometers together so that the readings from one could be compared to the readings from another. The analog tape data was used as a storage medium for later reduction.

(I) Senior Bridge Engineer, California Department of Transportation, Sacramento, California, USA

(II) Assistant Transportation Engineer, California Department of Transportation, Sacramento, California, USA

At each bridge the seismometers were first calibrated to each other. Then the seismometers were oriented transverse to the structure and measurements made at intervals along the length of the bridge leaving one seismometer at a base point. A single seismometer was then set up to measure the vertical and horizontal frequencies of the structure.

REDUCTION OF DATA

The equipment used to reduce the data included a Norland Corporation Model 3001 programmable waveform analyzer with a 3701R disk drive, an Axiom EX-801 micro printer, a Hewlett Packard 1311A cathode-ray display, and a Hewlett Packard 7041A X-Y plotter. The waveform analyzer would read a portion of data from the tape recorder. This data was then analyzed using programs written for the analyzer.

First, the calibration functions were determined for the seismometers. A Fourier analysis was then performed on the data as a first step in determining the natural frequencies and mode shapes. The transfer functions were then determined for the waveforms with respect to the base point. These transfer functions were then combined into the power spectral density function and the natural frequencies of the structure determined. When the transfer functions were combined, taking into account their location on the bridge, the mode shapes could be determined. The transfer functions were determined in polar coordinates and the data was then averaged with the previous data in rectangular coordinates. This averaging tended to remove the noise and resulted in cleaner waveforms.

The ambient results were then compared to a computer analysis of the bridge. The analysis models were constructed using STRUDL, a multipurpose structural analysis computer program maintained by McAUTO of St. Louis, Missouri. The models were composed of three dimensional beam elements with lumped masses. The abutment soil stiffness and the foundation soil stiffness were modeled using linear foundation springs. The properties for the beam elements were determined using current Caltrans practice. Shear deformations were ignored. The modulus of elasticity of the concrete was assumed to follow the standard American Concrete Institute formula for concrete. The moments of inertia were adjusted for cracking in the section. Hinges in the structure were modeled as one foot long members with one end released for longitudinal force, transverse moment and longitudinal moment.

SYSTEM IDENTIFICATION

Bridges that have been instrumented for strong motion by the California Division of Mines and Geology were selected for a system identification analysis. The system identification process involved the fitting of the model natural frequencies and mode shapes to the measured natural frequencies and mode shapes. The parameters that were identified were the moments of inertia of the superstructure plus the foundation and abutment foundation stiffnesses. The parameters were selected such that the weighted least squares error could be minimized. The mode shapes and natural frequencies were assumed to be the complete quadratic combination of the parameters. The coefficients for the complete quadratic were determined by performing multiple STRUDL analysis runs as follows: An analysis of the STRUDL model was made and then the mode shapes and natural frequencies were extracted. A parameter was then changed in order to measure the change in the mode shapes and natural frequencies for each change of a combination of parameters. When the coefficient matrix could be assembled, then a fitting program was utilized to determine the best set of parameters. This process was run iteratively until the parameters converged.

RESULTS

The results come from three sources: the system identification, the compared runs, and the comparison of ambient data to the results of large force level shaking.

Ambient Vibration Testing

Ambient vibration testing is relatively cheap and easily performed. In bridge structures, it gives an indication of the dynamic properties of the bridge. Bridges are highly nonlinear because of the expansion joints in the bridge and the foundation stiffnesses. Since bridge superstructures are very stiff, the response of the structure is highly dependent on the stiffness of the foundations. As the force level increases the boundary springs become softer so that the fundamental period of the structure increases. Also the effective inertia of a concrete member will decrease with increasing load levels, increasing the fundamental period of the structure.

One of the problems with ambient vibration testing is that the structure properties depend on the load level. Ambient loads may range anywhere from heavy truck traffic and high winds to automobile traffic and light winds. In some of the bridges, more than one first and second natural period were observed. This is explained by the varying load levels and the varying properties of the structure at these levels.

Ambient vibrations also contain many natural frequencies due to the exciting force. When field data is obtained it is very difficult to remove these unwanted frequencies. One method of removing them is to obtain a large amount of data and averaging the results. This tends to remove the non-structural frequencies from the measured data. Another method used is to determine the mode shapes at the apparent structure natural frequencies. This helps to separate the directions of the frequencies and also to remove the non-structural frequencies. It should be noted that it is assumed that each of the principal directions of a structure have their own natural frequencies. This may not always be the case. Many times the first mode of vertical motion is also a mode of longitudinal motion because of the way the stiffness of the structure is arranged and the mass is distributed along the structure.

Fundamental Natural Frequencies Observed

Figures 1 and 2 show a plot of the measured fundamental natural frequencies of bridges according to the type of structure and its characteristic dimensions. It is interesting to note the good correlation of the measured fundamental natural frequency to the ratio of the structure length times height to its width times depth. Pier walls generally do not follow the pattern since they add a lot of stiffness to the system compared to other substructure types. The lines were fit to the data in a least squares sense.

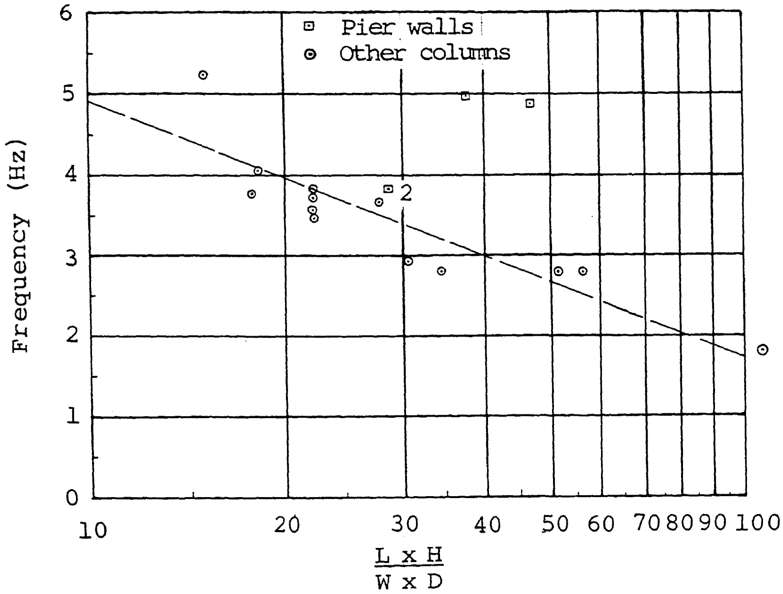

Figure 1 Plot of Frequency vs. Length x Height: Width x Depth Ratio for Concrete Reinforced Box Girders

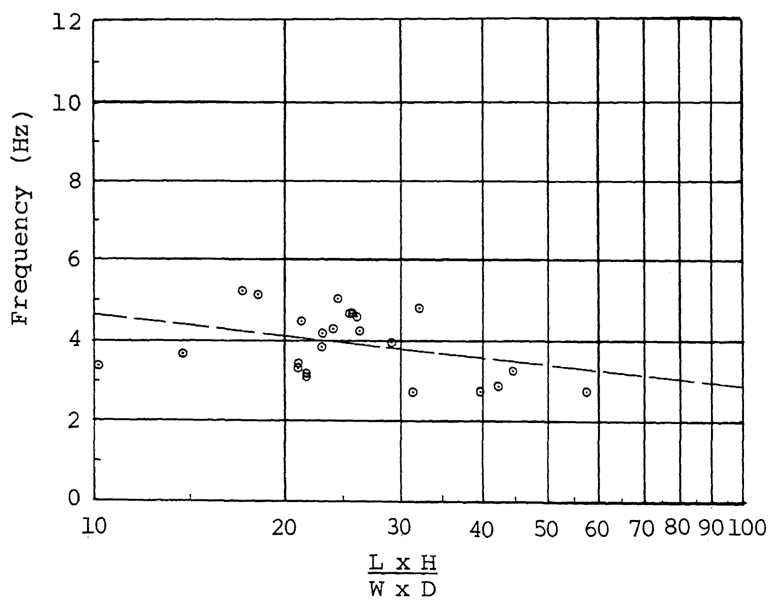

Figure 2 Plot of Frequency vs. Length x Height: Width x Depth Ratio for CIP Prestressed Box Girders

Table 1: Superstructure Properties for Prestressed Concrete Structures

Superstructure Moment of Inertia	Range of Fitted Values (% of gross)
Vertical Motion	120 - 140
Transverse Motion	100 - 120
Torsional	200 - 315

Table 2: Superstructure Properties for Reinforced Concrete Structures

Superstructure Moment of Inertia	Range of Fitted Values (% of gross)
Vertical Motion	36 - 97
Transverse Motion	49 - 120
Torsional	40 - 200

From the fundamental mode shapes observed, the contribution to the mode shapes from the stiffness of the abutments was observed. The displacements at the abutments of end diaphragm abutments with monolithic wingwalls were much smaller than the seat type abutments. When modeling the abutments it is very important to include the stiffness of the abutments.

The fundamental longitudinal frequencies generally agreed with the computer model. In many suspended spans, an equivalent spring was added to model the hinge stiffness. Bridges that were skewed did not generally exhibit an increase in stiffness due to the skew of the columns. These bridges generally acted as if the columns were not skewed. This effect was especially noted at the San Juan Bautista Bridge (Bridge Number 43-31). This bridge also demonstrated another modeling problem. The bridge is composed of simple spans on two column bents. Thus the bridge had a joint at each bent and there were many natural frequencies. The mode shapes extracted from the experimental data could not be matched by the computer model. A lot of the difficulty was caused by the lack of an accurate hinge model. In the computer model the hinges were frictionless member releases. The actual hinge has friction and a limited movement. The use of a nonlinear analysis would eliminate this problem.

Measured Section Properties

The measured section properties of the structures showed a marked difference depending on the superstructure type. In the models compared to the measured results, the assumed section properties generally depended on the type of structure. Prestressed structures were assumed to have the full gross section properties at a cross section. Reinforced concrete structures were assumed to have 40 percent of the vertical motion moment of inertia, 70 percent of the gross transverse moment of inertia and 40 percent of the gross torsional moment of inertia. With these modeling assumptions the structure models were generally looser than measured. This is probably due to the fact that in the computer model the effects of slab flares and web thickening were not taken into

Table 3: Foundation Springs from System Identification

Abutment Foundation Springs	Range of Fitted Values (Pinned - Fixed)
Longitudinal	14,860 - 158,900 k/ft
Transverse	4,387 - 180,700 k/ft
Vertical Rotation	8,471 - 10,290,000 ft(k)/deg
Transverse Rotation	0 - 525,700 ft(k)/deg
Column Foundation Springs	**Range of Fitted Values (Pinned - Fixed)**
Longitudinal	4,749 - 292,500,000 k/ft
Transverse	1,145 - 248,800,000 k/ft
Longitudinal Rotation	129,400 - 433,000,000 ft(k)/deg
Vertical Rotation	3,895 - 714,300,000 ft(k)/deg
Transverse Rotation	1,906 - 278,400,000 ft(k)/deg

account. Also at the low ambient force level, the cracked section would be larger.

When utilizing the system identification procedure, the properties could be better determined. Table 1 shows the range of superstructure moments of inertia for prestressed concrete structures and Table 2 shows the range of superstructure moments of inertia for reinforced concrete structures using system identification. The results show that cracking has a large effect on the properties of the superstructure. In prestressed concrete structures the cracking is eliminated by the prestress force. The measured values are well defined. On the other hand, the length of span greatly influences the amount of cracking in reinforced concrete structures. At the Rose Creek Interchange, a five span reinforced concrete structure, the moments of inertia in the vertical direction varied from 36 to 62 percent of gross. These results reflect the amount of cracking in a section, which is influenced by the reinforcing in the span and the span length. In the transverse direction, the inertias varied from 49 to 120 percent of the gross. This reflects the amount of cracking along the spans.

<u>Foundation Springs</u>

In the bridges where system identification was not performed, the column foundations were assumed fixed and springs were modeled at the abutments. The abutment foundation springs were generally calculated according to the assumption that a Rankine pressure distribution would form behind the abutment. The soil modulus assumed was 48 kips/cubic foot (7.63 MN/cubic meter) and an equivalent pile spring of 480 kips/foot (7.01 MN/meter). Using these springs the response was generally a little stiffer than that predicted by the computer model. This would suggest that the assumed springs are probably too loose.

On the bridges where system identification was performed, both the abutment and the column foundation springs were determined. These results are summarized in Table 3. The values in Table 3 reflect both foundations that are fixed and

Table 4: Meloland Road Overcrossing - Natural Frequencies

	Observed Natural Freq. (hz)	
	Ambient	Earthquake
Mode 1 Vert.	3.52	2.78
Mode 2 Vert.	4.88	4.39
Mode 1 Trans.	3.42	2.49
Mode 2 Trans.	7.32	5.03
Mode 1 Long.	4.98	2.69

Table 5: Rose Creek Interchange - Natural Frequencies

	Observed Natural Freq. (hz)	
	Ambient	Large Forces
Mode 1 Vert.	2.78	2.5
Mode 2 Vert.	3.66	3.5
Mode 3 Vert.	4.20	4.0
Mode 1 Trans.	2.83	2.7

pinned. Fixed foundations were generally at the upper end of the range and the pinned foundations at the lower end of the range. The pinned results do not equal zero, indicating that keys and other concrete features are not fully pinned in any direction. The range of values represents all the foundation types. It is necessary to utilize foundation springs in a computer model to obtain the correct response. Some of the foundation values were checked against present methods for determining pile and foundation stiffness. Usually the order of magnitude of these were correct.

Structural Response and Force Level

Two of the bridges presented the opportunity to compare the results from an ambient loading with a higher level loading.

Meloland Road Overcrossing (Bridge Number 58-215) is a two span reinforced concrete bridge located south of El Centro, California and is instrumented for strong motion recording by the California Division of Mines and Geology. In 1979 the Imperial Valley Earthquake caused fault rupture 0.5 mile (0.8 km) from the bridge and shook the bridge to a maximum acceleration level of 0.5 g. Table 4 summarizes the measured natural periods of the structure for both the ambient and the earthquake motions. Note how the structure softened up during the earthquake motion and the properties of the structure were less than those measured at the ambient level.

Rose Creek Interchange is located outside of Winnemucca, Nevada. It was tested at the ambient level in this study. Bruce Douglas of the University of Nevada, Reno has conducted many high amplitude tests on the structure (Ref. 1).

Table 5 lists the comparisons of the results as obtained at the higher force levels and the results as obtained in the ambient vibration study. This structure also exhibits a looser behavior at the higher force levels.

CONCLUSIONS

The results of the testing of 57 bridges by ambient vibration testing are presented in this paper. These results show that the current dynamic analysis models used for bridges are generally satisfactory. The results also show the importance of carefully modeling the boundary conditions at the supports. Portions of the structure where full releases were assumed were shown to have some fixity. The determination of accurate and representative section properties was found to be important (cracked sections, section thickenings).

From the results of the system identification performed on the reinforced concrete bridges, Meloland Road Overcrossing and Rose Creek Interchange, the longitudinal superstructure moment of inertia was determined to be between 40% and 60% of the gross value. The transverse superstructure moment of inertia was determined to be between 60% and 80% of the gross value. The torsional moment of inertia was determined to be approximately equal to the gross value.

From the results of the system identification performed on the prestressed concrete bridges, Painter Street Overcrossing and Route 280-680/101 Separation, the longitudinal superstructure moment of inertia was determined to be between 120% and 140% of the gross value. The transverse superstructure moment of inertia was determined to be between 100% and 120% of the gross value. The torsional moment of inertia was determined to be about 200% of the gross value.

Figures 1 and 2 provide a graphical illustration of the expected fundamental natural frequencies for typical bridges. These figures can be used as a guide to designers to estimate first mode frequencies and to check the validity of computer models.

A significant difference in structure response between the ambient and earthquake force levels was found at Meloland Road Overcrossing. This difference was determined to be caused primarily by the nonlinear response of the soils at the abutment.

REFERENCES

1. Douglas, Bruce M., Charles D. Drown, and Monty L. Gordon. "Experimental Dynamics of Highway Bridges." Proceedings of the Second Specialty Conference on Dynamic Response of Structures: Experimentation, Observation, Prediction and Control. New York: American Society of Civil Engineers, 1980. 698-712.

2. Douglas, Bruce, Mehdi Saiidi, James Richardson, and James Hart. "Results From High Amplitude Dynamic Tests and Implications for Seismic Design." Proceedings of the 15th Joint Meeting of U. S. - Japan Panel on Wind and Seismic Effects. Tsukuba, Japan, 1983.

3. Gates, James H. and Monte J. Smith. Verification of Dynamic Modeling Methods by Prototype Excitation. Sacramento: California Department of Transportation, (FHWA/CA/SD-82/07), 1982.