DEVELOPMENT OF AN AUTOMATED BRIDGE MONITORING SYSTEM

Final Report

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PREFACE

This is the final project report for the study "Development of An Automated Bridge Monitoring System," Project 86-3, sponsored by the Joint Highway Research Advisory Council. John T. DeWolf was the principal investigator and Robert G. Lauzon and David F. Mazurek were graduate student research assistants.

This report has been abstracted from the two theses of the research assistants. Any investigation involving dynamic behavior and both laboratory and field tests involves great quantities of data. What is included is then only the overall findings, with limited supporting material. Both theses contain much additional information, and in addition, most test data is stored at the Department of Civil Engineering.

The original suggestion for the development of a monitoring system was made by John Judd, president of Vibra-Metrics in Hamden, Connecticut. He has continued to be involved in this work as an advisor and through a subsequent cooperative investigation. His assistance has been invaluable.

This investigation was preceded by other dynamic bridge studies, supported by the Joint Highway Research Advisory Council, and the reports from the earlier investigations contain much of the background work, including an extensive literature survey and a field study of the Founders Bridge in Connecticut. The references at the end of the report list the resulting publications.

In addition to the support of the Connecticut Department of Transportation, we are grateful to John Fikiet and Tom Marcellino, managers of the School of Engineering Electronics Shop and Machine

Shop, respectively. Joe Gartner, professor of mechanical engineering, also provided valuable assistance in setting up the laboratory data acquisition system, and Robert Sylvester from Bruel and Kjaer assisted in selection and use of the FFT analyzer for both the laboratory and field studies.

Chapter 1

INTRODUCTION

1.1 SIGNIFICANCE

Vibration monitoring through the use of accelerometers has been used in many areas. Nuclear power plants are continuously monitored, aircraft and space vehicles are monitored for problems, and some buildings have been monitored for wind and earthquake forces. Techniques for modelling and analyzing the dynamic performance of mechanical structures have been refined during the past decades. This has been combined with the developments in electronics instrumentation and computers so that dynamic studies can be used economically by engineers for a wide variety of engineering structures. The goal of vibration monitoring is to note changes to the structure that can precipitate failure.

Bridges are normally inspected at two year intervals. However, problems can occur between inspections. In 1987 the Federal Highway Administration began to consider increasing inspection intervals beyond two years for some bridges (5). Examples of recent failures include the Mianus River Bridge in Connecticut (15) and the Schoharie Creek Bridge in New York (6), where reports indicated that the failure was due to scouring and washout of one of the foundations. Failure occurred even though both bridges received regular inspections, either because the failure mechanisms were not noticed during inspection or because the mechanism developed between inspections. The collapse of a Rhode Island bridge was averted when a passerby observed severe midspan cracking of a primary girder (1,13). One engineer stated that the crack developed during a three day period prior to discovery.

Vibrational monitoring, which works well for other types of structures and machinery, is a potential tool for use in bridge monitoring. It has been the purpose of this study to establish guidelines for continuous bridge monitoring and to conduct tests on an actual Connecticut bridge using a preliminary prototype system.

This study is based on information developed in previous studies (2,3,4,7,8,9,10,11,12,14).

1.2 DESCRIPTION OF VIBRATIONAL BEHAVIOR

Vibrations are the oscillation of a point or body about a reference position. When the motion repeats itself exactly after a discrete amount of time the vibration is said to be periodic. The simplest form of a periodic vibration is called harmonic motion which when plotted as a function of time is represented by a sinusoidal curve, as shown in Figure 1.1. Useful definitions are the period T, which is defined as the time between occurrences of identical motion and the frequency f, which is defined as the inverse of the period. Amplitude is defined as the distance between the highest and lowest values of oscillation, while damping is the reduction in this amplitude as time goes on.

Most structures are multi-degree systems, i.e. they can involve vibrational behavior which is made up of a combination of sinusoidal curves with harmonically related frequencies. The motion of a piston in an internal combustion engine, Figure 1.2, demonstrates this.

The motion repeats itself during discrete amounts of time, yet it is not harmonic. The curve is reduced by frequency analysis into its component harmonic waves, shown in Figure 1.3. Frequency analysis, normally carried out with an FFT (Fast Fourier Transform) analyzer, can be used to approximate any periodic curve using any number of combinations.

When frequencies making up a periodic curve are plotted versus their respective amplitudes the result is a frequency spectrum and is shown in Figure 1.4. The peaks correspond to the resonant or natural frequencies. When resonance occurs, the displacement may become very large after a finite number of cycles. Resonant frequencies are unique for each structure and can be calculated knowing the stiffness and mass. Frequencies representing resonance have the largest amplitudes in the frequency spectrum. Each of these resonant or natural frequencies represents a mode shape, which can be thought of as the shape the structure takes when vibrating at that frequency. With each mode shape come stationary points or "nodes". As the mode shape becomes of higher order the number of nodes increases. More energy is needed then to excite these higher natural frequencies. The frequency spectrum is therefore dependent on the amount of energy input to the system.

1.3 APPLICATION TO BRIDGES

Bridge structures can be characterized by a series of complex springs and masses. The behavior is similar to a multi-degree of freedom spring mass system. Engineers are able to make good predictions of the static response, and with approximations they can even estimate the overall dynamic behavior.

The results of a small change in a bridge structure, as would occur if a failure mechanism were developing, would not usually be noticed by direct monitoring of the deformations, such as the deflection at the center of the span, or the monitoring of strains or forces in members or supports. However, the monitoring of the dynamic behavior, i.e. accelerations, should indicate when changes occur.

Accelerometers can be installed at various points on a bridge to monitor the performance. The data from these will be fed to a computer to develop a signature, or fingerprint, of the bridge. The frequency spectrum described in the previous section is one possible bridge signature.

The weakening of a structure is characterized by a reduction in stiffness, which in turn will lower the resonant frequencies and influence other modal properties. Elements which may be used in describing the vibrational signature are:

1. Resonant frequencies.
2. System damping corresponding to each resonance.
3. Mode shapes.

These dynamic characteristics are represented in the signature. Thus changes in the signature occur due to changes in the bridge structure, such as slippage in a connection or cracking of a girder.

Software can be developed for comparing the response of the bridge over time. When a change occurs, the computer will respond with a warning to indicate if an alert condition exists.

The proposed monitoring system would supplement the present inspection routine; it would not replace what is presently being done. The system would supplement the present procedure, indicating if changes are occurring, and which areas are in need of more careful inspection. It would provide a means of continuously monitoring the bridge between scheduled inspections.

1.4 INVESTIGATION APPROACH

The original desired objective was to develop a monitoring system and then to apply it to two or three prototype bridges. After careful study, the decision was made to utilize a model bridge to determine the monitoring approach needed. The model study allowed for introduction of failure mechanisms into the bridge. This resulted in the determination of which elements in the vibrational signature most readily indicate changes in the bridge's structure which can lead to collapse. The failure mechanisms included cracking of a main girder and slippage of a support.

After the completion of the model study, field monitoring was undertaken on a Connecticut bridge using actual vehicles.

1.5 ORGANIZATION OF REPORT

This report is summarized from R. Lauzon's M.S. thesis (10), and D. F. Mazurek's Ph.D. thesis (12), with additional field analysis results. The two theses contain much additional material, including reviews of the literature, which is not repeated in this report. A comprehensive literature review was done by S. LaShomb (8,9).

Chapter 2 describes the model tests. Chapter 3 reviews the test data, both to establish what should be included in the bridge signature and to determine what changes when failure mechanisms are introduced into the model bridge. Chapter 4 contains sample field data. Chapter 5 contains the general conclusions from the study.

Chapter 2
MODEL TESTS

2.1 BRIDGE MODEL

A schematic of the model bridge and experimental setup is given in Figure 2.1. A cable connected to a variable speed motor was used to pull a test vehicle across the bridge. The model was configured as either a single or double span bridge with pinned supports. The information from accelerometers attached at different locations along the bridge model was fed to a signal analyzer and computer.

Structural Cross Section - The test span consisted of aluminum angles and plates bolted together to make up the cross section shown in Figure 2.2. This arrangement was used for many reasons. Aluminum was chosen because of its reduced Young's modulus in comparison with steel. Also less load was needed to produce a given deflection with an aluminum section than with a steel section of the same size. The goal here was not to produce a specific amount of deflection, but rather have a structure flexible enough to undergo vibration and produce a clear signature. Also, the test span was built from angles and plates so that individual pieces could be altered. Following testing, the cross section could then be repaired by replacing the altered piece. Additionally, 2 by 1/2 inch aluminum plates were bolted across the bottom width of the beam at 2.5-ft intervals to connect the two I-shape beams.

The vehicle traveled across the span riding along the top outer angles. A guiderail, made from a 3/4 inch square piece of wood was attached at the top center. Intermittent cuts in the wood piece were made to ensure that the stiffness of the bridge was not increased by the guiderail.

In some tests, mass was added to the bridge model by bolting different weights along the span at short intervals. Roadway roughness was modeled in other tests by adding strips of duct tape across the deck at evenly spaced intervals.

Test Vehicle - Three different two-axle test vehicles were used. The variable in the vehicles was the weight, equal to approximately 6.5 oz., 3.5 lbs and 35 lbs. Tests with the heavy weight vehicle demonstrated that the mass of this vehicle was so large that the bridge performance was reduced from a multi-degree system to single-degree system. The ratio of the mass of the model vehicle to the mass of the bridge also is much greater than would occur in normal highway bridges subject to the longest trucks. Therefore, the results presented in this report are based only on tests with the light and medium weight vehicles.

The vehicles were propelled, one at a time, across the span by two small wires tied to a cable that traveled in a trough supported above the test span as shown in Figure 2.1. The two wire arrangement kept the vehicle moving at the same speed as the cable. The cable was attached to a drum which was driven by an electric motor with a variable speed control. The opposite end of the cable was attached to a weight and pulley system to maintain tension in the cable at all times. The speed controller allowed for the determination of variation in vehicle speed on the vibrational signature.

2.2 INSTRUMENTATION

Accelerometers were fastened on the underside of the test span to measure the response of the span as the vehicle traversed it. Accelerometers consist of a piezoelectric material sandwiched between the base of the accelerometer and a small mass. During vibration the small mass is subject to acceleration which in turn stresses the piezoelectric material. This material is very sensitive to changes in stress, which result in small voltage changes. These small voltages were amplified and input into a frequency analyzer for processing into a frequency spectrum. The analyzer used in this study was a Bruel & Kjaer, Dual Channel Signal Analyzer Type 2032. The accelerometers were also manufactured by Bruel & Kjaer, model Type 2644. These accelerometers had a range of 0.5Hz to 100 KHz, more than covering the range of frequencies expected in the tests.

Two accelerometers were used to establish a normalized frequency spectrum. The accelerometers included a field and a reference accelerometer. Dependency on energy input was taken into account by normalizing the response of each one. The reference accelerometer was normalized by establishing its peak response as unity. The field accelerometer was normalized by dividing its response by the peak response of the reference accelerometer. The resulting amplitude on the frequency spectrum was independent of the energy input due to the dynamic loading.

The location of the accelerometers also affects the amplitude of the frequency spectrum. Measurements taken at a node point for a particular mode shape could not show accelerations and therefore that mode shape, and frequency associated with it, would go undetected. The placement of the reference accelerometer at a node point would be detrimental in the effort to normalize the amplitude of the frequency associated with it. For this reason the reference was placed at a location not coincident with a nodal point. The field accelerometer was placed at points along the length of the beam to ensure the measurement of all the frequencies.

2.3 TEST PROCEDURE AND DATA ANALYSIS

Tests were first performed using a conventional modal analysis technique, the primary purpose being to verify the ambient vibration approach used in the vehicle tests. The modal analysis procedure employed was the impact hammer technique. A number of vertical bending resonances were readily identified in the 0-200 Hz bandwidth.

Comparisons were made with a free vibrational analysis using the finite element analysis capabilities of GT STRUDL. Beam elements were used to model each of the I-shaped sections.

The Ambient Vibration Method was employed for most of the vehicle tests. A manual trigger was used to initiate data acquisition. For most measurements, the analyzer was triggered as the test vehicle entered the bridge. Fifteen averages were taken of each measurement to smooth the response spectra.

Raw data was obtained in the form of digital frequency response plots, including autospectra and cross-spectra. The elements of a vibrational signature are extractable from the spectra. Natural frequencies are approximately given by the frequencies corresponding to resonant peaks. Mode shapes are obtained by compiling phase and scaled amplitude information at each measurement station. Phase data is given by cross-spectra, while amplitudes are obtained from autospectra. Scaling and phasing are performed with respect to the reference station.

In summary, the Ambient Vibration Method provided nearly the same results as the conventional impact hammer approach for resonant frequency and mode shape determination, but overestimated damping.

2.4 INTRODUCTION OF FAILURE MECHANISMS

Once a base signature was determined for the model bridge, based on using the Ambient Vibration Method to determine natural frequencies and mode shapes, failure mechanisms were introduced, one at a time. These included introduction of a crack through one of the two I-shaped sections, and support slippage.

Crack Approximation - The location of a crack in a typical bridge girder will most likely form at a region of maximum stress. One example of this type of failure was the Rhode-Island viaduct mentioned earlier, where a simply supported steel highway bridge girder cracked at midspan. For the test model a crack was located at midspan for the single span case and at the 2/10 point for the two span case, as shown in Figure 2.3.

The lower flange at one I-shaped section was first cut and tests were conducted. The crack was then extended into the web. Figure 2.4 shows the three levels of cracking. The maximum reduction in the moment of inertia for the combined cross section for crack level III was 33 percent.

Support Slippage - The two span bridge model was used to study support slippage. Since the two-span bridge contained a sufficent number of support redundancies, the structure remained quite stable subsequent to local support releases. The middle support case is reported here. The release was developed by removing one of the two through-pin bearing blocks, leaving a partial, eccentric support. Figure 2.5 illustrates support slippage at the middle support.

Chapter 3

MODEL TEST RESULTS

3.1 MODEL BRIDGE SIGNATURE

The approach described in the previous chapter was used to determine resonant or natural frequencies and mode shapes. These are used as the basis for defining the bridge signature.

Both flexural modes, involving longitudinal bending deformations, and torsional modes, involving twisting deformations, were measured. The lowest flexural mode shapes are shown in Figure 3.1 for the single span and double span configurations. The twisting mode shapes correspond to the flexural mode shapes.

The lowest natural frequencies are listed in Table 3.1, based on the vehicle tests of the bridge model.

3.2 INFLUENCE OF TEST VARIABLES

Before vibrational signature monitoring to detect structural degradation can be initiated, the influences of normal operating conditions must be determined and accounted for. Therefore, characterization tests were performed to assess the nature of the bridge-vehicle system and how it influences vibrational response. The variables considered were vehicle velocity, roadway roughness, and vehicle mass.

<u>Vehicle Velocity</u> - The effects of vehicle velocity were evaluated by measuring the dynamic response of the model bridge for velocities of 1, 2, 4, and 8 ft/sec using both the light- and medium-weight vehicles.

The frequency spectra shows autospectrum values, which are related to the accelerations, versus frequency. An example is shown in Figure 3.2 for velocities of 2 and 8 ft/sec. This was based on using the light-weight vehicle. Examination of the figure indicates that as velocity increases, the overall spectral response is increased. In addition, the comparative amplitude of various resonant peaks, related to the acceleration, vary with velocity. However, the peaks present at one velocity are generally visible at the other velocity. Thus, while velocity variations do not create new peaks nor move existing ones, the intensity of resonances is dependent on cart velocity.

Corresponding natural frequencies are given in Table 3.2. Looking at the resonant frequencies, the light cart results were very consistent, displaying a maximum deviation of about 1%.

Roadway Roughness - Roadway roughness was varied by creating bumps using different thicknesses and spacings of 1/4 in. wide duct tape strips. Six different conditions were examined, including no strips (smooth), single and triple strips at 9 in. and 18 in. spacings, and strips of random size and spacing. Tests were performed using both the light- and medium-weight vehicles, using a velocity of 4 ft/sec (120 cm/sec).

Frequency spectra for the smooth roadway and the roadway with triple-thick strips spaced at 9 in. are shown in Figure 3.3 for the light cart. This figure indicates that as roadway roughness increases, the overall spectral response is increased. As was found for velocity variations, roadway roughness changes influence the comparative amplitude and intensity of resonant peaks, but do not create any new peaks or cause significant measurable changes in the position of the natural frequencies.

Vehicle Mass - Comparing the resonant frequency results just given with those obtained from the verification hammer test results (where there are no mass variables), the discrepancy associated with the light cart influence is very small, causing a maximum frequency reduction of less than 1% for the fundamental mode. However, the mass influence of the heavy-weight vehicle is greater, being up to 7% for the fundamental mode. The vehicle-to-span mass ratio for the light and heavy carts are approximately 0.01 and 0.1, respectively. For large highway bridges, the light vehicle to span mass ratio is more reasonable when considering individual vehicles.

Modal amplitudes displayed some variations between the light- and heavy-weight vehicle results. However, these deviations did not consistently follow a particular trend, and as a result could have been due to experimental error and not specifically attributable to mass variation. This is especially true for the fundamental mode, where differences in modal amplitude were usually under 2%, suggesting minimal mass influence on mode shapes.

Results of Altering Test Variables - The changes in vehicle speed, vehicle mass and roadway roughness, which relate to actual conditions on bridges, indicate that while the magnitude of the accelerations change, the natural frequencies and mode shapes do not change under a normal variation in these values. Thus, the bridge signature useful for monitoring for structural degradation can be based on natural frequencies and mode shapes.

3.3 DEGRADATION TESTS

The laboratory bridge model allowed for the evaluation of structural degradation and its influence on the vibrational signature. The model permitted the examination of various failure modes, thereby providing an indication of how the dynamic properties are consequently affected. The types of structural deterioration considered included support failure and crack propagation.

Crack Development - Results are presented in this report for a crack developing in one of the I-shaped girders in the double span model. The crack development was carried out in three stages as shown in Figure 2.3. The crack was located in one span at the point of maximum bending moment. As the crack developed, the overall bridge vertical bending moment of inertia was reduced to about 81%, 68%, and 67% of the original cross-section.

Resonant frequency results are given in Table 3.3. The first bending frequency displays steady and significant change resulting from the crack. A plot of the frequency change as a function of percent reduction in bridge vertical bending inertia is given in Figure 3.4. This plot suggests that the initial crack development causes only slight frequency changes, but as the crack increases, the rate of frequency change also increases. Bending modes B2 and B3 did display some frequency changes, but they were not nearly as large as occurred for the first mode.

The mode shapes of the first bending resonance (B1) are given in Figure 3.5. Of the three lowest bending resonances, this mode displayed the most change as the crack progressed. The deviation is greatest at the location of the defect (e.g., 78% change in modal amplitude for the largest crack). At locations remote to the defect (e.g., other span), virtually no impact to the mode shape is evident. The other two resonances were not influenced nearly as much, since the crack location was near stationary points for both modes.

Released Supports - Results are presented in this report for the two span model with a partial release of the middle support as shown in Figure 3.6. Other studies were made with partially released end supports.

The resonant frequency results are given in Table 3.4. Because of the released support, a new mode (B1A) appeared at about 19 Hz. This mode was global, and the character of its mode shape was similar to the first bending mode of a simply-supported beam, except that the eccentric restraint at the middle support introduced a torsional component to the motion.

The first and third vertical bending resonances displayed virtually no change. This is because these resonances are developable for the bridge configured as either a single- or double-span, and the support release occurred at a nodal point common to both arrangements. However, Mode B2 exhibited a sharp increase in resonant frequency as a result of the defect as shown in the Table.

The greatest impact to modal amplitudes also occurred along the central Stations, 12-22, for Mode B2, and this is illustrated in Figure 3.4. This figure covers the full double span length. The largest deviation resulted at the released support.

<u>Finite Element Study with Spacing Between I-Shaped Girders</u> - The model bridge was comprised of two individual girders, and the girders were rigidly fastened together with cross-ties over the length of the bridge. Thus, the girders could not behave independently and, in reality, more closely modeled a box girder rather than a multiple-girder bridge. As a result, it was suspected that the resonant frequency changes due to cracking might be larger if the girders were independent.

To investigate this possibility, a finite element analysis of the laboratory model was performed, the results and details of which are given in Reference (11). As part of the analysis, various mathematical models were created which emulated the existing girders, but separated them a given distance. The girders were connected by a flexible deck and with transverse stiffeners. The results indicate that significantly greater resonant frequency changes can occur when the girders are separated than when they are rigidly tied together. For example, the fundamental vertical bending frequency was lowered by 18% when one of the girders was cracked as in the laboratory.

Chapter 4

FIELD TESTS

Actual tests on a Connecticut Bridge (second level of the unused overpass in West Hartford over I-84) were performed to determine if vibrational signatures in the form of frequency response plots were obtainable for a full-scale bridge using the same equipment and methodology employed in the laboratory. The steel-girder/slab bridges consist of a series of simply-supported spans.

Two measurement stations were used for each set of tests. The stations were located on outside girders in the same span at opposite longitudinal quarter-points. The instrumentation setup was identical to that used in the laboratory. Excitation was provided using either a loaded 18.5 ton dump truck or a full-sized pickup truck. Measurements were initiated when a vehicle entered the instrumented span.

Figure 4.1 shows frequency response plots obtained at the same measurement station for two separate tests. One case reflects 10 averaged records obtained with the dump truck traveling at 30 mph, while the other represents three averages using the pickup truck traveling at 40 mph. The character of these plots shows that most major resonances occur at identical frequencies despite variations in vehicle mass and velocity. This follows the model test results. As expected, these variations do influence the intensity of the resonances.

These tests and others conducted on the same overpass demonstrate that signatures can be determined for actual bridges in the same manner as for the model bridges.

Work is now underway in a follow-up investigation supported jointly by the Connecticut Department of Higher Education and Vibra-Metrics of Hamden, Connecticut to conduct tests on a bridge during normal traffic loading.

Chapter 5

SUMMARY AND CONCLUSIONS

Laboratory tets have shown that major structural deterioration is readily detectable, based on comparison of vibrational signatures. Significant structural changes which can lead to failure are noticeable prior to collapse.

The Ambient Vibration Method provides nearly the same results as more conventional modal analysis techniques for resonant frequency and mode shape determination. Resonant frequencies and mode shapes are not influenced by vehicle velocity or roadway roughness. However, the intensity of a mode is dependent on velocity, roadway roughness, and vehicle mass. Resonant frequencies can be influenced by mass variables, while vehicle mass influences on mode shapes appear to be minimal. The range of vehicle masses during normal traffic loading is not so great, however, that significant changes in frequencies will prevent monitoring using vibrational signature analysis.

Support failures can cause large changes to both resonant frequencies and mode shapes. These failures may also lead to the creation of new modes. Crack propagation can cause substantial shifts in certain resonant frequencies. The rate of frequency change appears to increase with increasing crack size. Mode shapes are also heavily influenced by crack propagation. The greatest changes occur in the vicinity of the defect. Thus, once it has been determined that a structural defect exists, mode shapes could be used to isolate its location.

An automated system must be capable of determining the vibrational signature, based on resonant frequencies and mode shapes. Once the elements of the signature are determined, the automated system must make a decision as to whether or not a dangerous condition exists.

Continuing research is now underway at the University of Connecticut and at Vibra-Metrics in Hamden, Connecticut, to build a full scale field monitoring system for placement on a Connecticut bridge. This system will be used to refine the signature analysis techniques and to use normal traffic loading for continuous monitoring.

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TABLE 3.1 Natural Frequencies for Model Bridge

Mode	Natural Frequencies	
	Single Span	Double Span
Bending 1	8.3 Hz	32.3 Hz
Bending 2	31.5 Hz	46.6 Hz
Torsion 1	38.2 Hz	73.3 Hz
Bending 3	69.0 Hz	118.1 Hz
Torsion 2	80.2 hz	

TABLE 3.2 Natural Frequencies for Varied Vehicle Velocity

Cart Velocity, in feet per second	MODE		
	B1	B2	B3
1	32.3	46.0	119.0
2	32.2	45.9	119.2
4	32.3	46.2	119.0
8	32.6	45.8	119.4

TABLE 3.3 Resonant Frequencies for Cracked Bridge

Mode No.	Test*	Natural Frequency, In Hertz	Percent Change
B1	A	32.32	-
	B	32.02	0.9
	C	30.96	4.2
	D	29.01	10.2
B2	A	46.58	-
	B	45.98	1.3
	C	45.38	2.6
	D	45.07	3.2
B3	A	118.1	-
	B	116.1	1.7
	C	115.7	2.0
	D	115.5	2.2

* A: Baseline bridge
B: First crack increment
C: Second crack increment
D: Third crack increment

TABLE 3.4 Resonant Frequencies for Released Middle Support

Mode No.	Natural Frequency in Hz: Baseline with Full Supports	Natural Frequency in Hz: with Released Support	Percent Change
B1A	-	18.91	-
B1	32.32	32.35	0.1
B2	46.58	60.17	29.2
B3	118.1	117.3	0.7

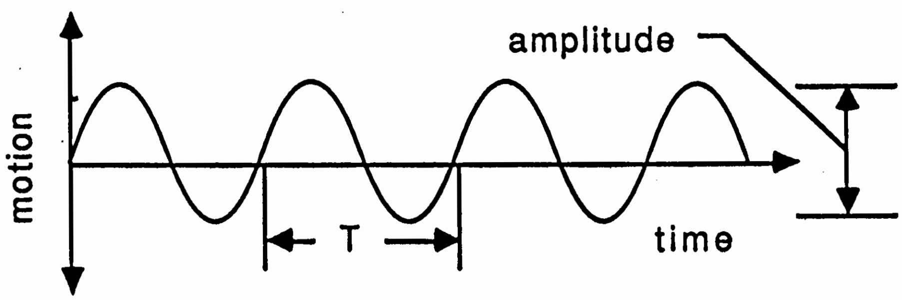


Figure 1.1 Harmonic Motion

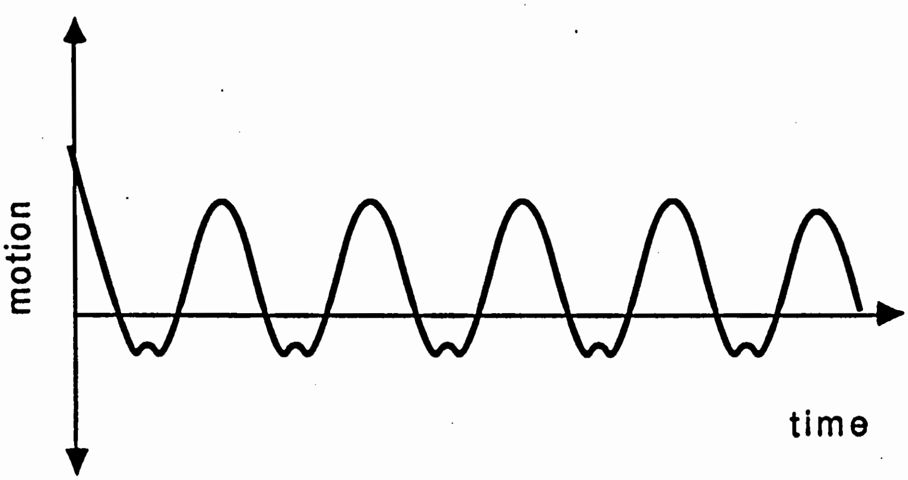


Figure 1.2 Piston Motion

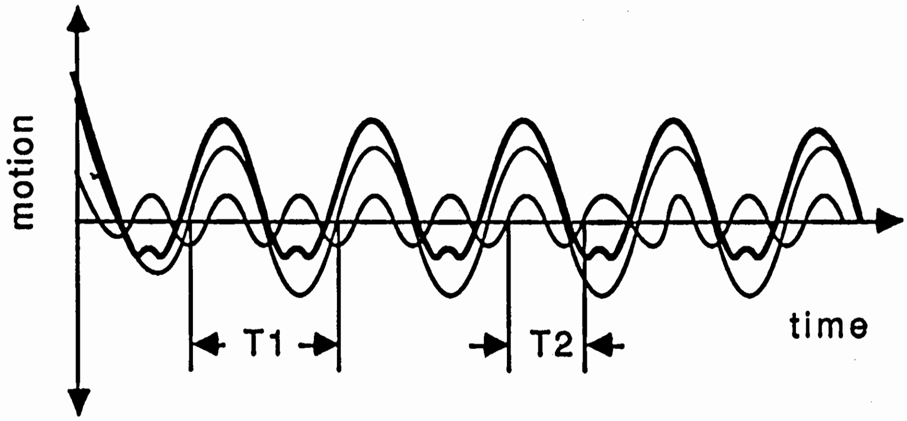


Figure 1.3 Wave Components

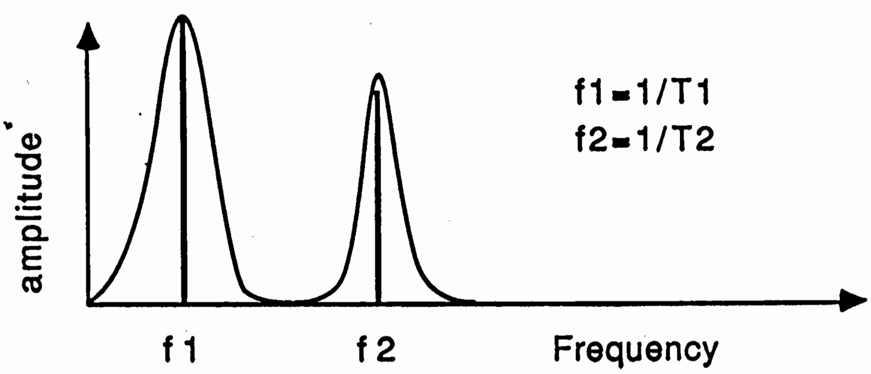


Figure 1.4 Frequency Spectrum

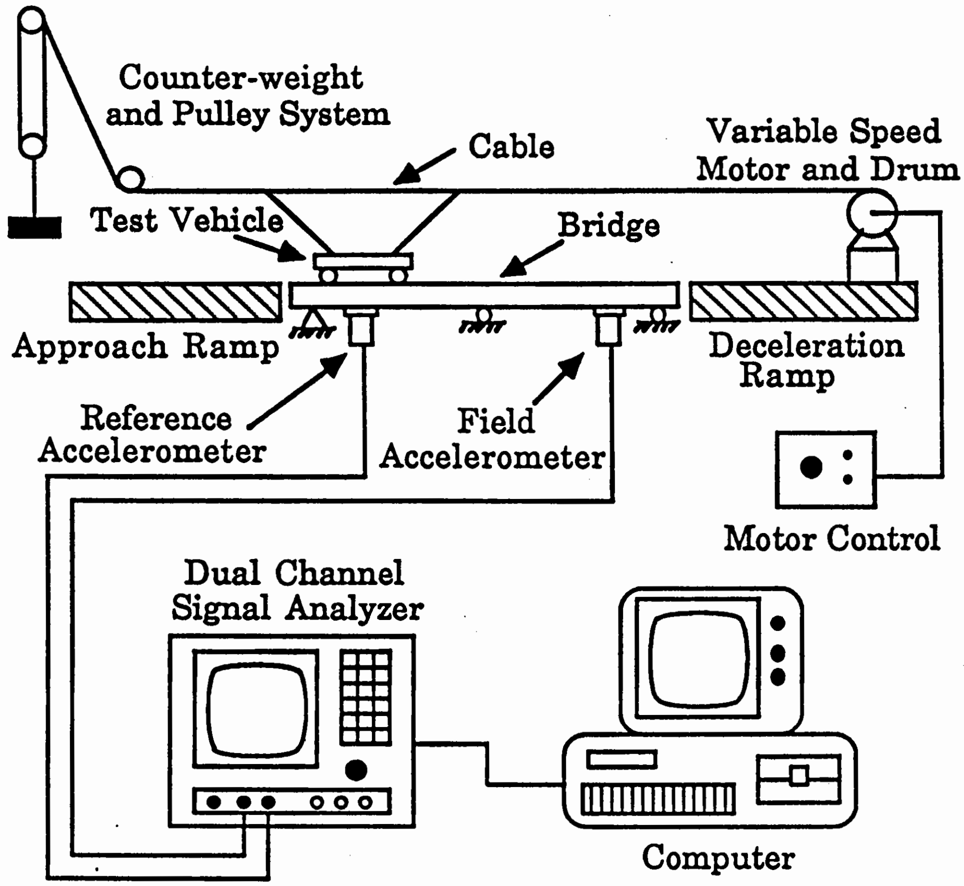


Figure 2.1 Model Bridge Test Setup

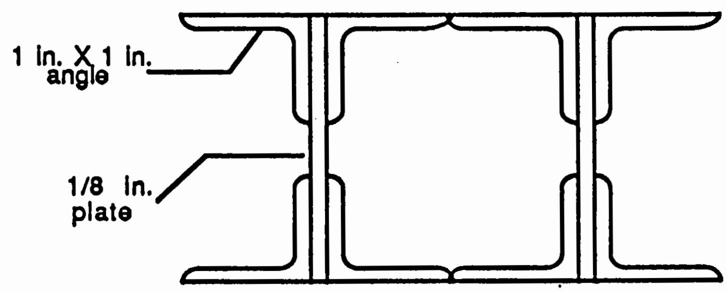


Figure 2.2 Cross Section of Test Span

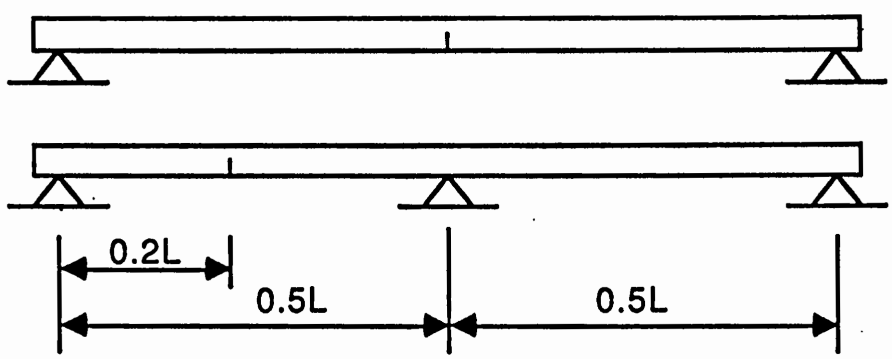


Figure 2.3 Crack Location

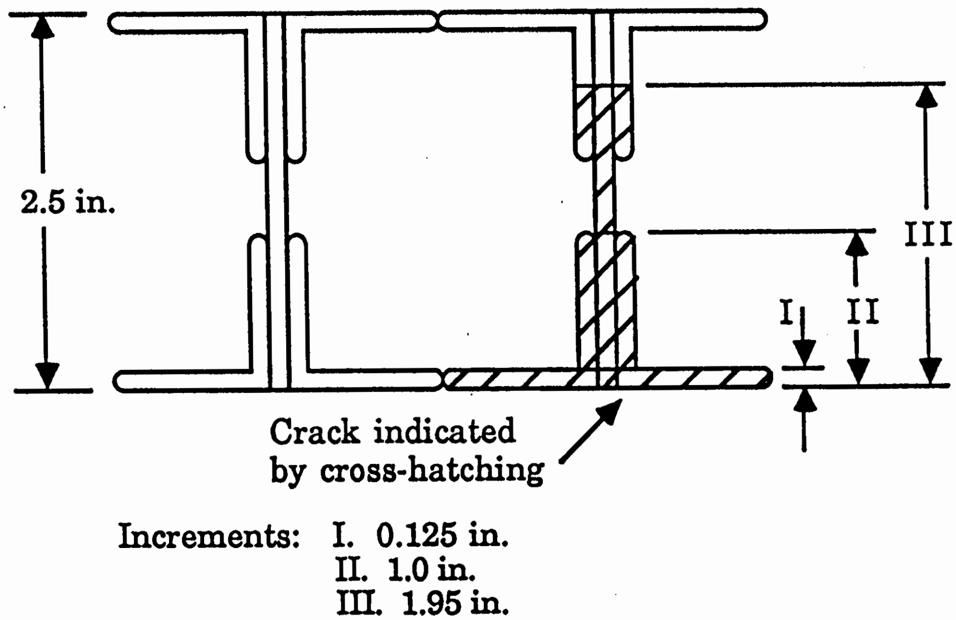


Figure 2.4 Crack Development

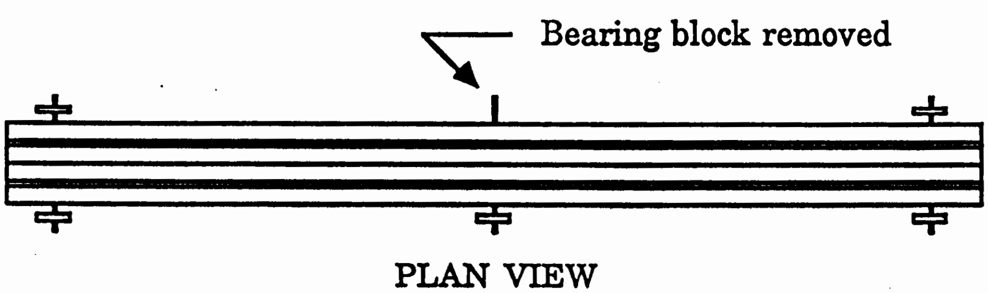
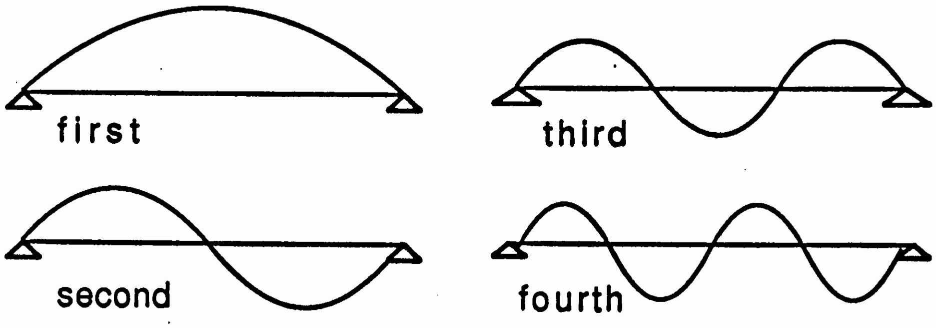
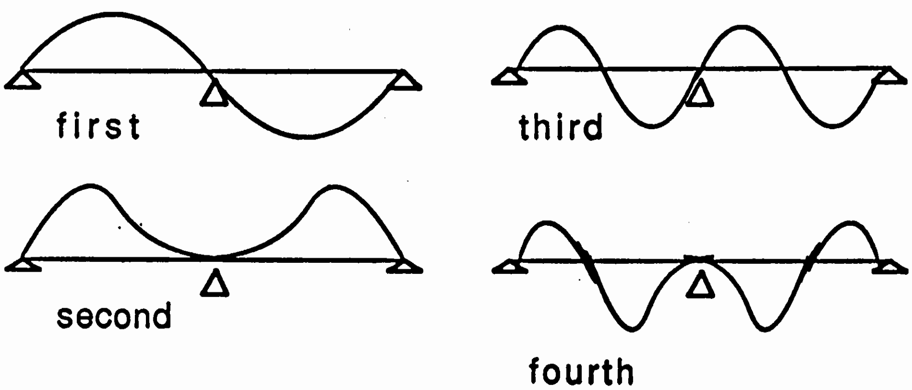


Figure 2.5 Support Slippage



(a) Single Span



(b) Double Span

Figure 3.1 Mode Shapes for Model Bridge

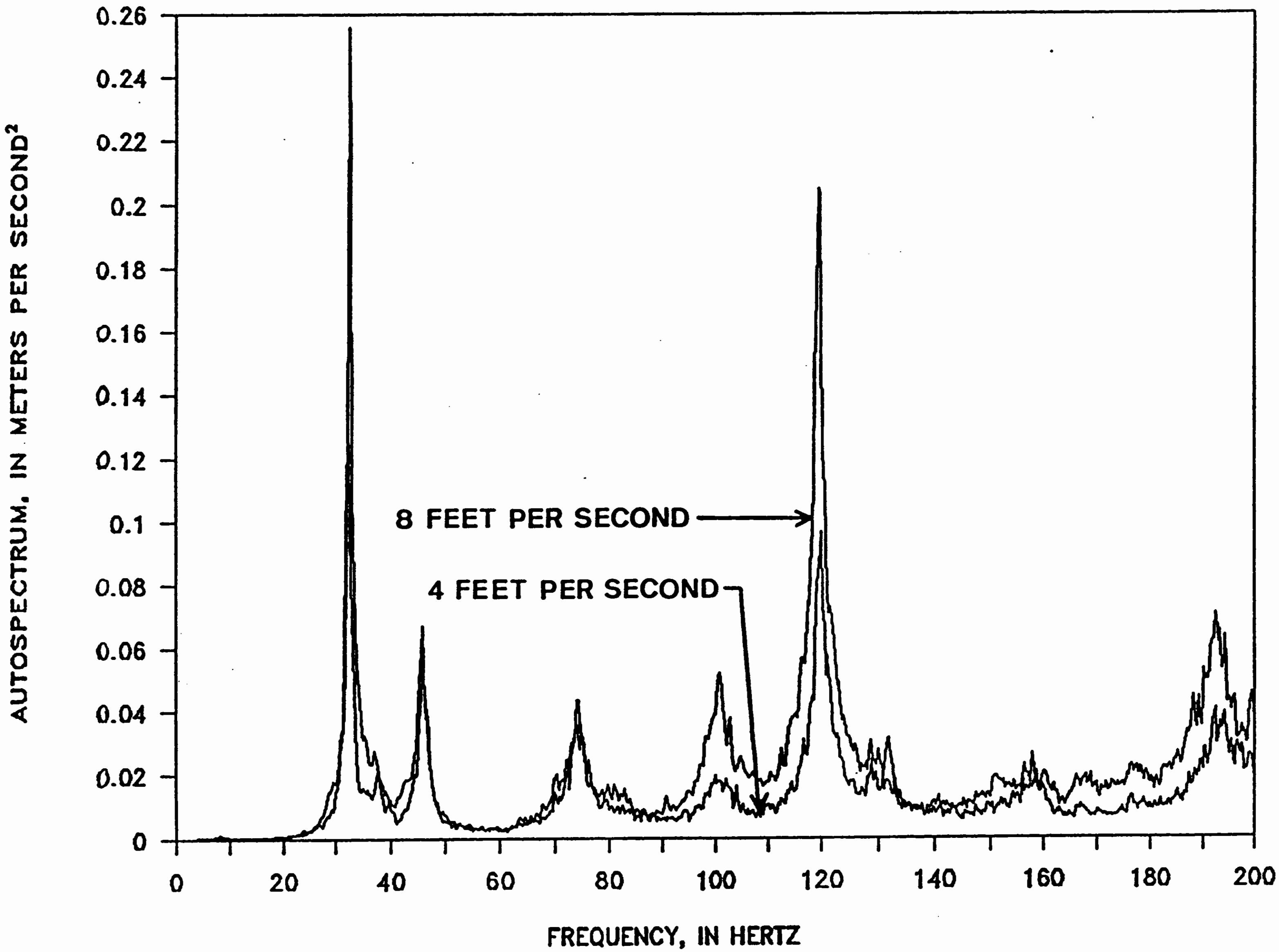


Figure 3.2 Frequency Spectra for Double Span Model Test - Variable Speed

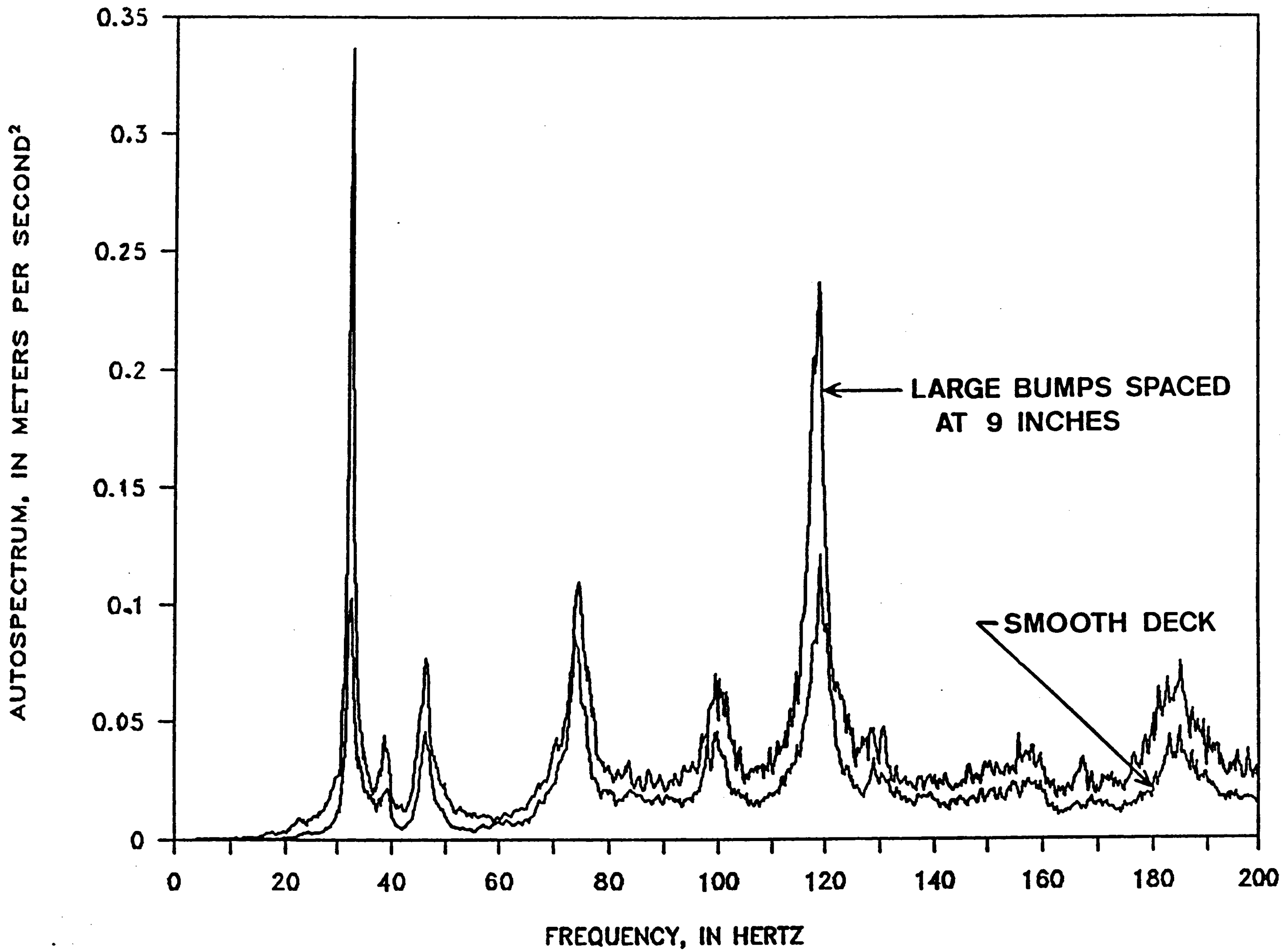


Figure 3.3 Frequency Spectra for Double Span Model Test - Variable Roadway Roughness

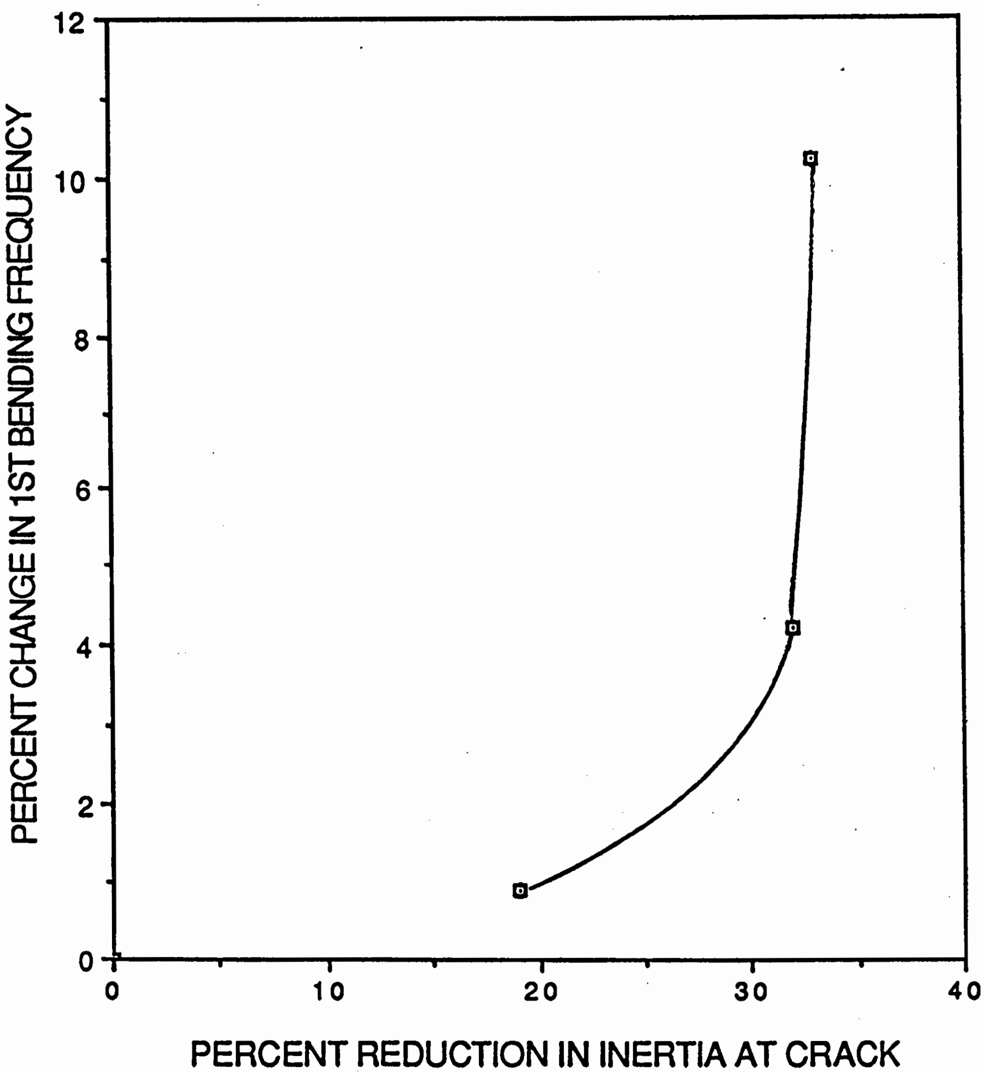


Figure 3.4 Change in Natural Frequency Versus Reduction in Moment of Inertia for First Bending Mode for Double Span Model with Crack

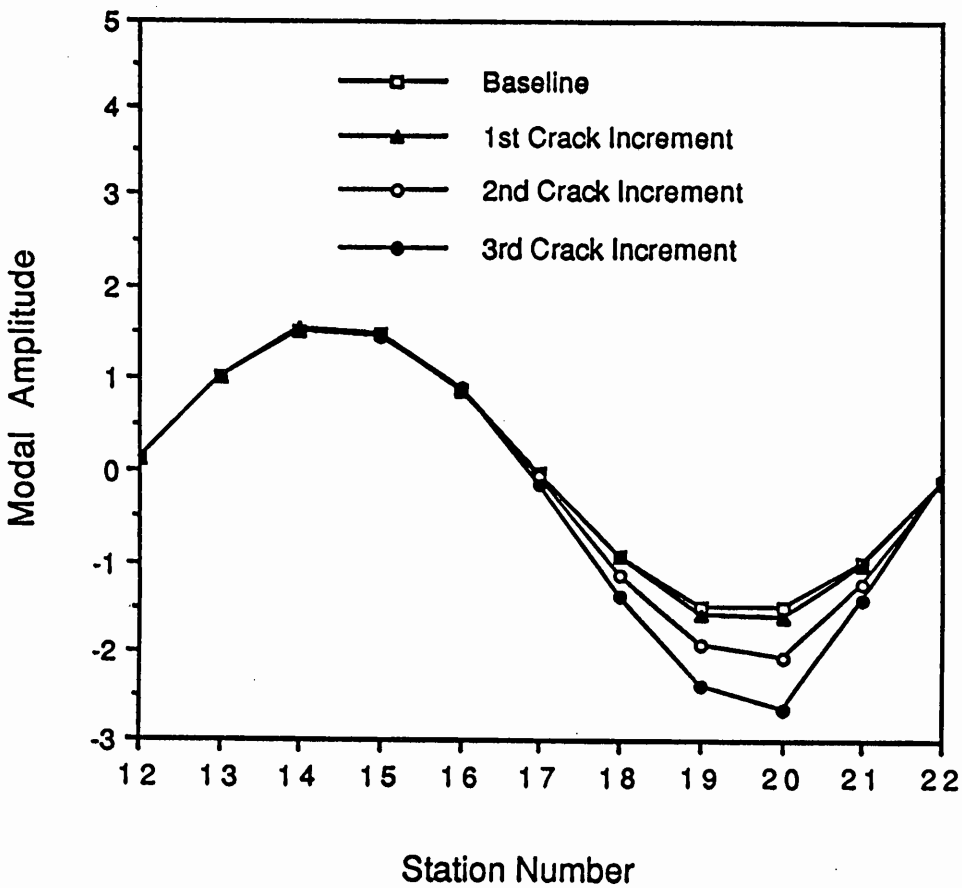


Figure 3.5 Change in Mode Shape for Crack Development in Double Span Model

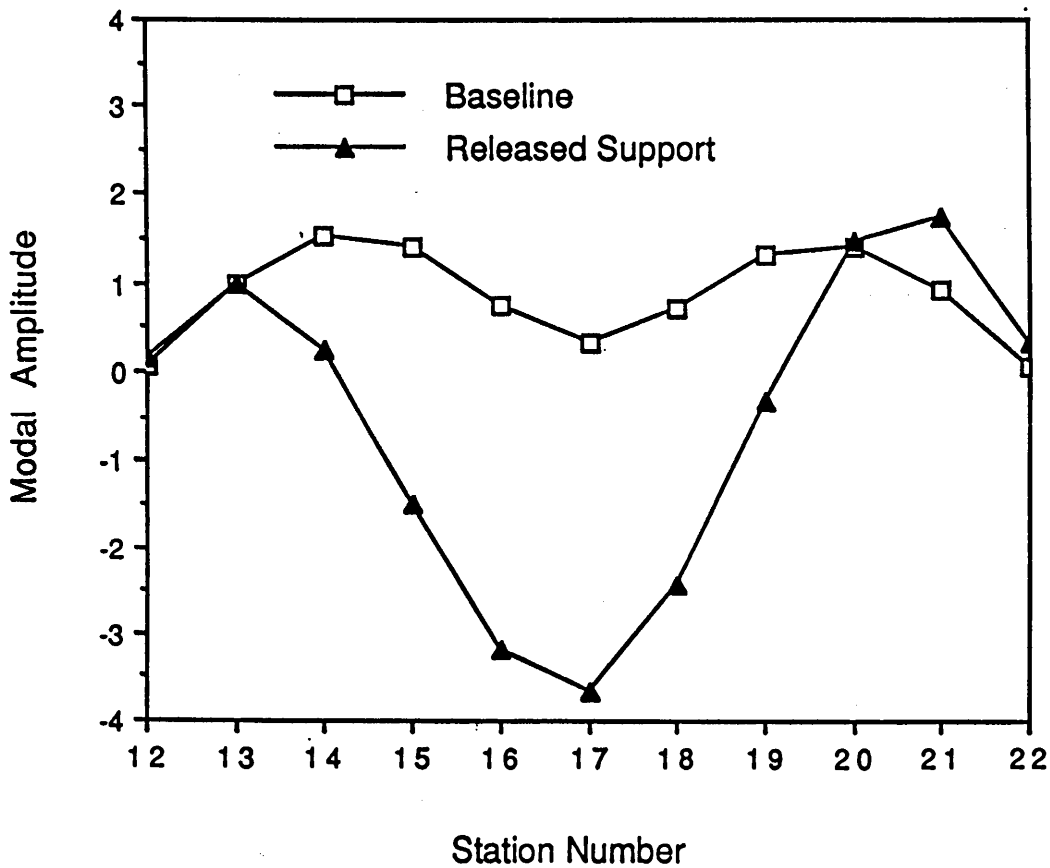


Figure 3.6 Change in Mode Shape for Partial Release of Center Support in Model Test

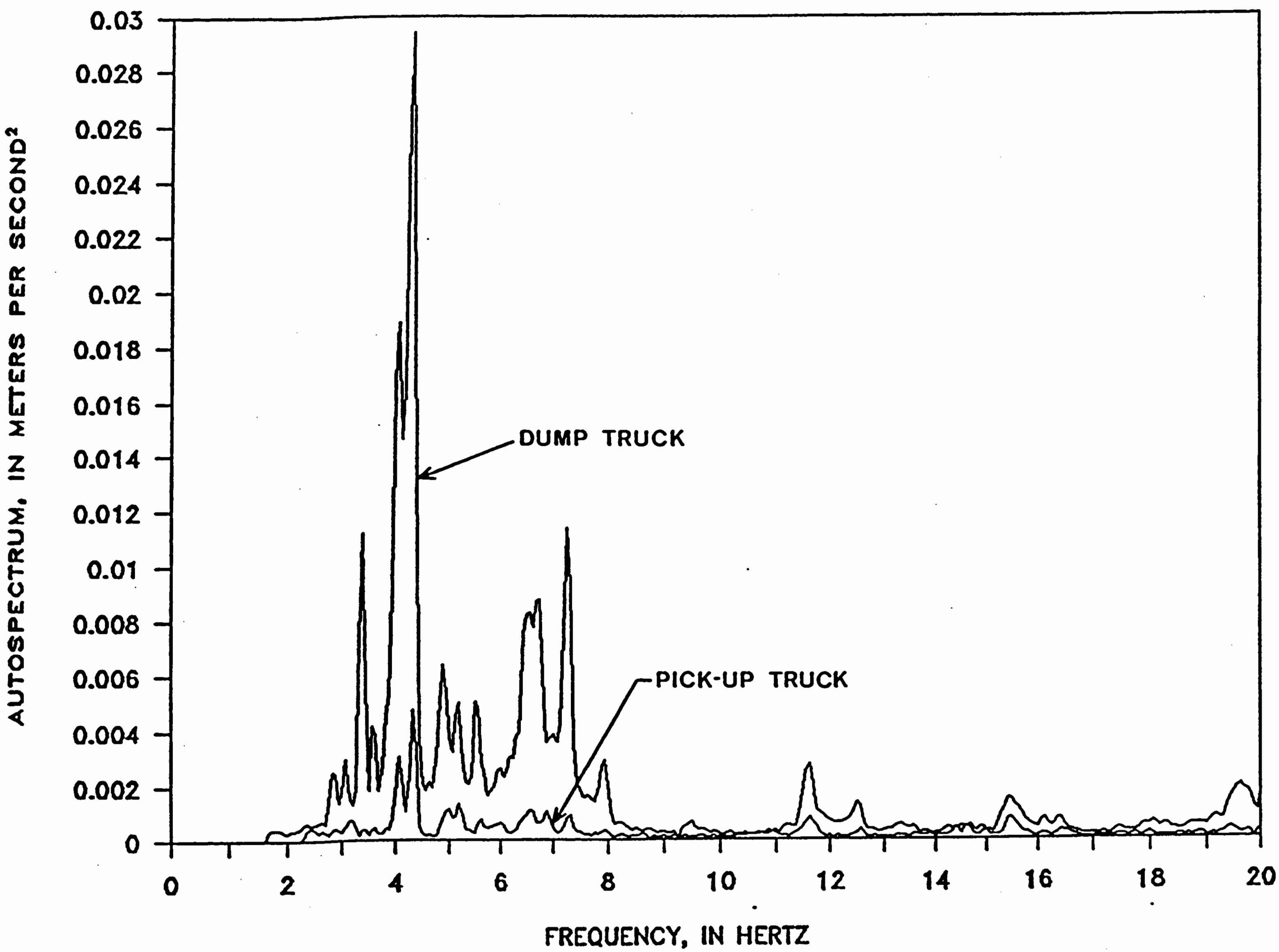


Figure 4.1 Field Test Frequency Spectra - Different Vehicles