Vibration Measurements on Simple-Span Bridges

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This paper presents results of field measurements of dynamic deflections of simple-span bridges due to the passage of a heavy vehicle. Data are presented for five bridges of typical design. No strain measurements are presented, and the discussion is limited to a consideration of midspan deflections. Typical dynamic deflection records are shown for the five bridges tested.

The maximum observed amplitudes of vibration for the test bridges vary from 18 to 40 percent of the maximum static deflection. These amplitudes are considerably greater than would be produced by a theoretical smoothly running load.

The frequencies of the observed vibration while the load is on the span is generally not the natural frequency of the structure but is apparantly related to the natural frequency of the vehicle on its suspension system. Experiments with a vehicle in which the springs had been blocked out of action indicate that considerably larger vibrations occur in this case than in the case of a completely sprung vehicle.

It is concluded that one of the most-important factors which influence the vibration of bridges is the dynamic characteristics of the vehicle itself. Another important factor is the condition of the roadway surface on the approaches, which may cause a vibration of the vehicle mass on its suspension system as it enters the span.

Also reported are experiments with a vertical accelerometer which may be used to measure vibrations without reference to a fixed point. The use of an accelerometer is shown to be a satisfactory method for measuring the free vibration of a bridge.

● THE modern tendency toward large span-depth ratios has focused attention on the problem of highway-bridge vibrations. Until recent years this problem has been generally ignored by engineers, with the result that there exists no satisfactory method by which a designer can predict the dynamic behavior of a bridge structure. This paper contains the results of a series of dynamic field tests conducted on several bridges in an attempt to obtain some of the much-needed information on this important problem.

The most-severe vibrations have been said to occur in multispan bridges. However, an understanding of the behavior of simple-span structures is desirable before a solution of the more-complex problem of continuous bridges is attempted. For this reason the project reported herein has been restricted to simple-span bridges.

The theoretical prediction of bridge vibration is complex, because there are many parameters involved; the reliability of such prediction is limited by the uncertainty regarding proper values of many of these parameters. These include not only the structural properties of the bridge but also the dynamic characteristics of the vehicle and the character of the roadway surface. The main purpose of this project was to define the problem more clearly by indicating which of the many parameters are of greatest significance.

The data presented are the dynamic midspan deflections of five simple-span bridges during the passage of a single heavy vehicle. Because of the general nature of the project's purpose, it was desirable to limit the measurements to midspan deflections, thus making it possible to test a larger number of bridges. When it becomes possible to predict dynamic deflections, the determination of dynamic stress increments will be greatly simplified.

A consideration of the results leads to certain conclusions regarding the amplitude, frequency, and damping characteristics of the induced vibration. These conclusions are significant, since they indicate the basic character and the major causes of the vibration.

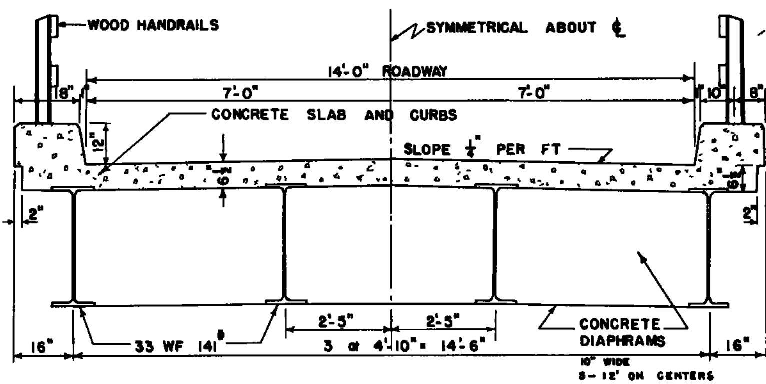
DESCRIPTION OF TESTS

The Structures

Cross-sections of the five test bridges are shown in Figures 1 through 5. These bridges are representative of the several types of simple-span structures currently in use in Massachusetts. These particular bridges were selected because their structural makeup and location presented a minimum of experimental difficulty. Simple structures were purposely chosen in order to minimize unnecessary complications which might make the analysis of test results more difficult.

Three of the structures are of the plate-girder type and two are steel-stringer bridges, one of which contains spiral reinforcement to provide composite action between the stringers and the concrete-deck slab. With the exception of the Ware Malboeuf Bridge, which has an open-steel deck, all of the bridges have concrete-deck slabs.

Also shown in Figures 1 through 5 are the span, weight per foot, and rigidity (EI) of the five test bridges. Two values of the rigidity are shown. The "design EI" includes those elements of the cross-section which are normally considered in the calculation of stresses. The "composite EI" includes all elements of either steel or concrete with the exception of railings. In these computations a modulus ratio of ten has been assumed.



SPAN = 69 FT
WEIGHT = 2860 LBS/FT
DESIGN EI = 0 90 × 10^{12} LB - IN^2
COMPOSITE EI = 2 10 × 10^{12} LB - IN^2

Figure 1. Conway Shirkshire Road Bridge.

Test Apparatus

The test vehicle used on all bridges is shown in Figure 6. It is a two-axle (six-wheel) Sterling-White 10-ton dump truck having a wheel base of 13 feet and a tread of 6 feet 6 inches. When loaded, the total weight of the vehicle was approximately 43, 000 lb. with 36, 000 lb. on the rear axle. The exact weights for the various test runs are given with the description of test results.

The information desired for each of the tests was a continuous record of the midspan deflection during the passage of the vehicle across the span and for a significant period thereafter. This information was obtained by a deflectometer specially designed for this purpose. The component parts of the instrument are shown diagrammatically in Figure 7 and the assembled instrument in use is shown in Figure 8. A short length of aluminum pipe of 1-inch diameter is held tightly against the underside of the structure by means of a compressed steel spring. Threaded to a cap at the bottom of this pipe is a ¼-inch-diameter rod which, in turn, depresses a small calibrated beam. As the bridge deflects, the rod is forced up or down, thus displacing the measuring beam. The pipe and rod are held in place by several ball-bearing guides. The entire system is mounted within a light steel tower held firmly against the ground or stream bed by the compressed spring. The tower is constructed in three units, each of which can be adjusted in length. Thus, any height up to a maximum of 24 feet can be accommodated by the instrument.

The measuring element is an aluminum beam ½ by ⅛ inch in cross-section on a 10-inch span. The midspan displacement of the measuring beam, which is equal to the deflection of the bridge, is obtained by two SR-4 (Type A-7) strain gages mounted on the beam. These gages were calibrated in a movable-head testing machine so that the gage readings could be converted directly to deflection.

The strain readings were recorded by a Sanborn strain-gage amplifier (Model 64-500A) and a Sanborn direct-writing recorder ("Twin-Viso" Model 60). This equipment produces a deflection-time record directly by means of a heated stylus writing on moving heat-sensitive paper. A choice of paper speeds is available, but in all tests a speed of 100 mm. per sec. was used.

The average velocity and position of the test vehicle was determined by use of two simple mechanical switches placed in the roadway over the supports of the structure. Each switch consists of two steel strips held apart by insulated spacers. When the wheels of the vehicle pass over the switch, the two strips are forced together, closing a circuit which causes a separate writing arm on the Sanborn recorder to mark a "pip" on the test record. Knowing the paper speed and span of the bridge, the average vehicle velocity may be easily computed.

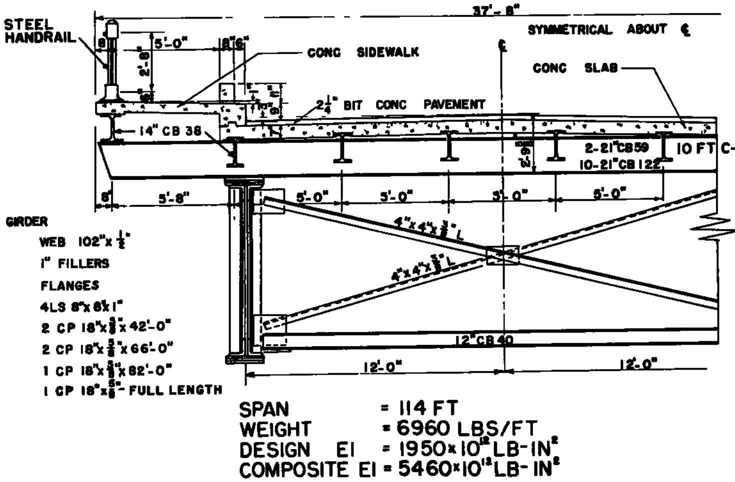


Figure 2. Gilbertville Ware River Bridge.

Test Procedure

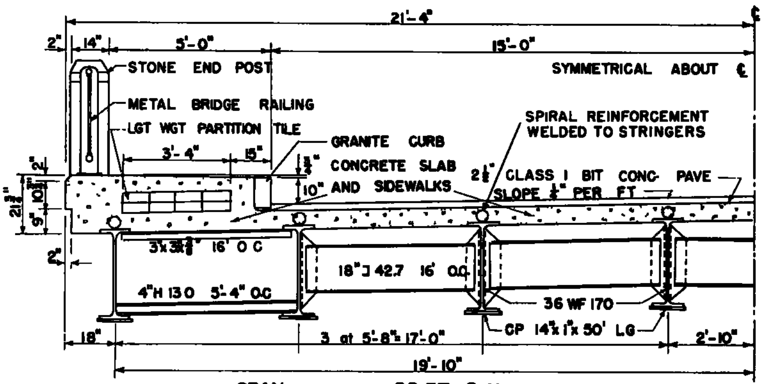
The testing of each bridge began with the measurement of static deflections. The test truck, straddling the roadway centerline, was moved into various positions along the span and the midspan deflection recorded. On the girder bridges the deflections were measured at the bottom flange of one girder, while on the stringer bridges the deflections were measured at midspan of the stringer nearest the centerline of the roadway. Moving load tests were then conducted at vehicle speeds beginning at 10 mph. and increasing by 3-mph. to 5-mph. increments up to the maximum attainable velocity.

The maximum velocity was limited by the vehicle and the condition of the approaches. In general, maximum speeds of approximately 40 mph. were attained. During the moving load tests, care was taken to ensure that the vehicle speed was as constant as possible and that the truck maintained the centerline position as it crossed the span.

Measurements were taken for passages in both directions.

TEST RESULTS

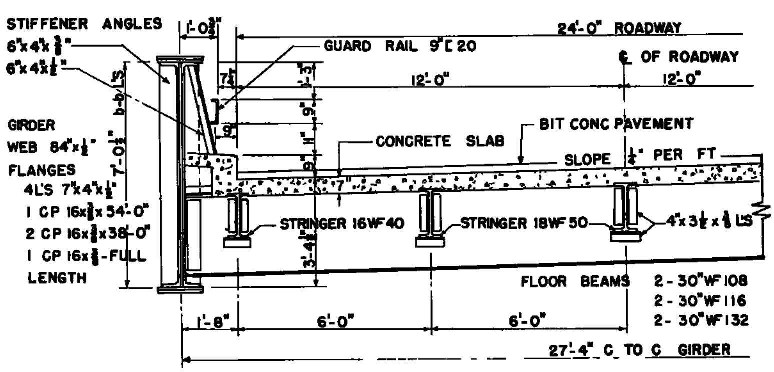
Typical deflection-time records for the five test bridges are shown in Figures 9 through 14. It is impossible to present in this paper more than a few of the many records which were obtained. However, additional records would provide little more information, since there is no apparent correlation between the characteristics of the vibration and the speed of the vehicle. In general, the records shown are among those which indicated the largest vibrations.



SPAN = 88 FT-8 IN
WEIGHT = 7830 LBS/FT
DESIGN EI = 744×10¹² LB-IN²
COMPOSITE EI = 912×10¹² LB-IN²

Figure 3. Townsend Main Street Bridge.

These records have been replotted from the original traces in order to provide more-convenient scales. The ordinate is taken as the ratio of dynamic to maximum static deflection at midspan. The abscissa is the position of the rear axle in terms of span length. A time scale is also provided. A static-deflection curve, based upon the measured static deflection, is shown in each figure for reference purposes. This is the so-called crawl deflection which would result from a very-slow passage of the vehicle. Also shown in each figure are the axle weights for that test run, the velocity of the vehicle, and the maximum static deflection. The residual vibration after the load has left the span is shown for a limited period of time.



SPAN = 86 FT
WEIGHT = 5120 LBS/FT
DESIGN EI = 627×10¹² LB-IN²
COMPOSITE EI = 720×10¹² LB-IN²

Figure 4. Townsend South Street Bridge.

DISCUSSION OF TEST RESULTS

As stated previously, there is no satisfactory theoretical solution available for the highway-bridge-vibration problem. However, in order to interpret the test results intelligently, it is helpful to enumerate some of the basic theoretical concepts involved.

The causes of the vibration may conveniently be separated into the following classifications: (1) the simple passage of a mass load across the span which produces vibration even though the roadway be perfectly smooth; (2) the vertical motion of the vehicle mass on its suspension system which is induced on the approaches and continues to some degree as the vehicle crosses the span; (3) sharp surface irregularities on the structure itself, which produce an "impact" effect involving sudden changes in the applied force. Each of these three causes are discussed below in general terms in order to form a basis for the discussion of the actual test results.

Cause 1

When an unsprung mass load crosses a bridge span with a perfectly smooth roadway surface, free vibrations are superimposed on the static or crawl deflection. The frequency of this vibration is essentially the same as the natural frequency of the structure. In this connection it should be noted that, due to the mass of the load, the natural frequency of the bridge continually changes as the vehicle crosses the span. However, in most practical cases the maximum variation in natural frequency does not exceed 20 percent.

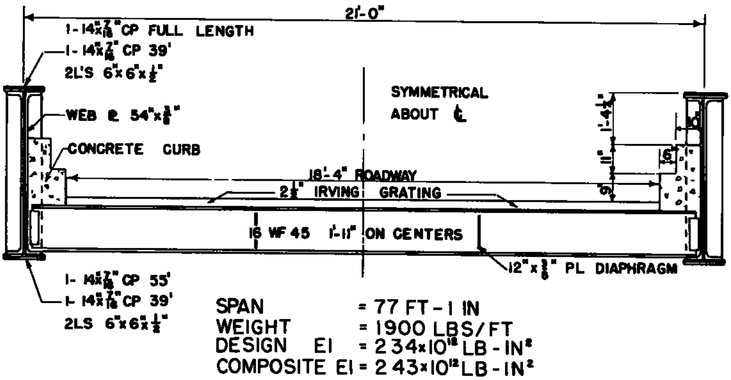


Figure 5. Ware Malboeuf Bridge.

For the structures and vehicle speeds included in this test program, the amplitude of vibration caused by an unsprung load on a smooth surface would be about 5 or 10 percent of the maximum static deflection. As will be discussed below, the test results show much-larger vibrations.

In the usual practical case the major part of the load is supported on springs. In general, the effect of the springs is to reduce the amplitude of vibration. However, this is a relatively minor effect when the vehicle is traveling on a perfectly smooth surface.

Cause 2

The second cause of vibration may be the most important. As the vehicle enters the span there is usually some degree of vertical oscillation of the vehicle mass on its suspension system, which results in a periodic variation in the applied force. This is roughly equivalent to a traveling alternating force of some magnitude superimposed on the gravity force of the mass load. Theoretical solutions for this case which were developed in the study of railroad-bridge vibrations indicate that the predominant frequency of the vibration is the same as the frequency of the alternating force rather than the natural frequency of bridge.

The amplitude of these vibrations depends not only upon the magnitude of the alternating force but also upon the relation between the frequency of the force and the natural frequency of the structure. If the two frequencies are the same, a resonant condition exists and extremely large vibrations may result. Complete resonance cannot be attained, however, because the natural frequency of the bridge changes as the vehicle crosses the span.

The amplitudes are also limited by the damping characteristics of both the structure and the vehicle. However, theoretical solutions indicate that, in practical cases, the vibration resulting from the condition described above can be much larger than that caused by a smoothly running load.

Some of the ideas expressed above are illustrated by the model-analysis results shown in Figure 15. These results are entirely qualitative and should not be considered as an indication of the magnitude of vibration in actual structures. The model bridge was a 1½-by-¼-inch steel bar to which weights were attached to provide a natural frequency typical of the bridges tested. The model vehicle was a steel mass supported on four wheels which could be pulled across the span at constant speed. The vehicle was constructed so that springs could be inserted between the wheels and the mass. The quantity measured is the bending strain at midspan which is nearly proportional to the midspan deflection.

The top trace in Figure 15 shows the variation in strain as the load crosses the span when no springs are present and the mass is supported directly by the wheels. The second trace indicates that, when the springs are inserted, the vibration is appreciably reduced. The bottom trace shows the much-larger strain variation resulting from the passage of a spring-born mass which had been artificially excited to vertical oscillation just before it entered the span. The natural frequency of the spring-borne mass was approximately equal to that of the unloaded structure, and even larger vibrations were prevented only by the high degree of damping in the spring system. The bottom trace in Figure 15 is similar in several respects to the deflection measurement of the actual structures.

Cause 3

The third possible cause of vibration listed above is the effect of sharp irregularities or bumps in the roadway surface on the bridge itself. In the case of an unsprung vehicle, this results in a large and suddenly excited vibration at the natural bridge frequency. The presence of springs on the vehicle complicates the problem but generally reduces the magnitude of the vibration. It is believed that, on most modern highway bridges, the surfaces are sufficiently smooth to prevent this effect from being the major cause of vibration.

Figure 6. Test Vehicle.

Static Deflections. The maximum static deflections which were measured on the five test bridges are given in Figures 9 through 14. For comparison with theoretical values, an effective rigidity, EI, based upon these deflections, has been computed for each bridge and is shown in Table 1. In general, the composite rigidity described previously is more indicative of actual behavior. However, except for the Townsend South Street Bridge, there is considerable difference between experimental and computed values.

In the case of the Gilbertville Bridge, which is a deck-girder structure, the results indicate that only a part of the floor system contributes to the rigidity. The results for the Townsend Main Street Bridge, a fairly wide stringer bridge, indicate that the entire width of cross-section is not fully effective. The experimental effective rigidities of the Conway and Ware bridges are more difficult to explain, because they are considerably larger than any value which could be computed from the cross-section. Possible explanations include a higher concrete modulus than assumed in the computations and some degree of restraint at the supports.

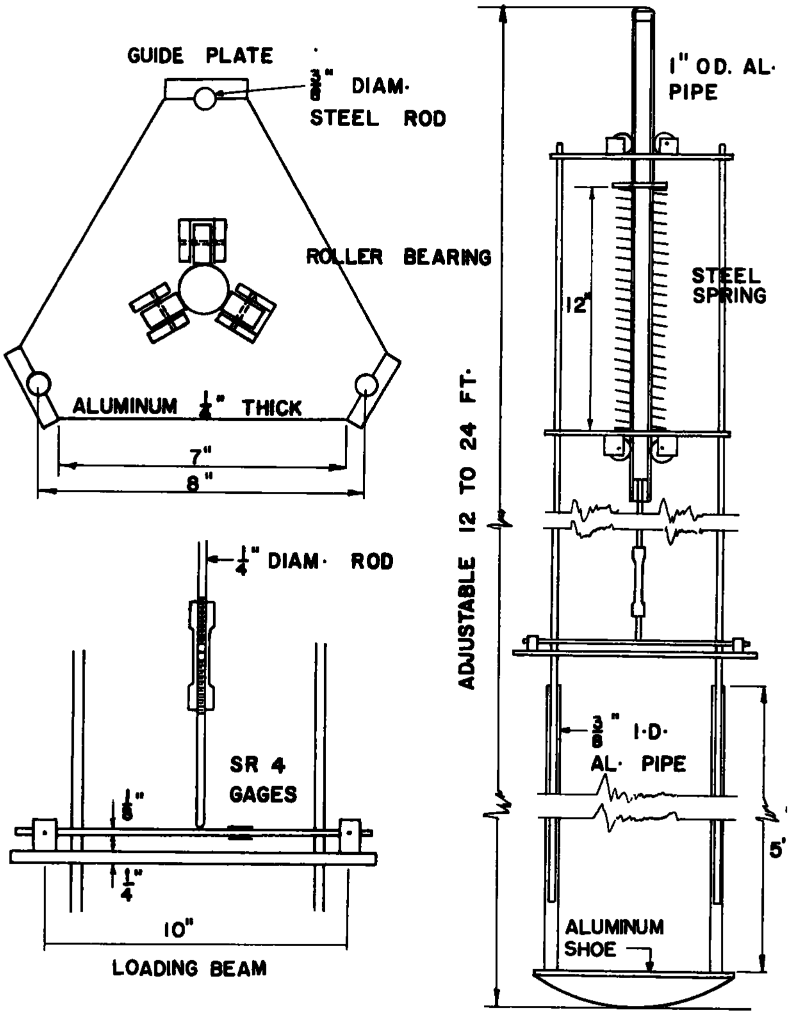


Figure 7. Deflectometer.

Frequencies of Vibration

The natural frequency of the structure is best indicated by the residual vibration which continues after the load has left the span. In all of the tests conducted, the first mode of vibration was predominant, and the following discussion is limited to a consideration of this mode.

The natural frequencies obtained from the residual vibrations are given in Table 1. Some difference was observed between various test runs, but the percentage variation is insignificant. The Ware and Townsend South Street bridges display an interference effect which is apparent in a periodic variation in the amplitude of the residual vibration

TABLE 1

COMPUTED AND EXPERIMENTAL RIGIDITY

(Values of EI are 10^{-12} lb.-in^2)

Bridge	Computed		Experimental			
	Design EI	Composite EI	Static Deflection ins	Effective EI	Frequency cps	Effective EI
Conway	0.90	2.10	0.179	2.87	6.00	4.33
Gilbertville	19.50	54.60	0.085	27.50	4.28	38.80
Townsend-Main	7.44	9.12	0.162	6.77	3.77	12.50
Townsend-South	6.27	7.20	0.146	6.78	4.17	8.89
Ware	2.34	2.43	0.221	3.26	5.50	3.84

(see Figures 13 and 14). The reason for this phenomenon is uncertain, but it may be the result of vibrations in the floor system superimposed on that of the main girders.

Effective rigidities based upon natural frequency have been computed and are shown in Table 1. In all cases the values are greater than those based upon static deflections. Although this cannot be completely explained, it is probable that the causes are a greater participation of secondary elements and a greater degree of frictional restraint at the ends of the span.

While the load is on the span, the frequency of vibration is generally not the natural frequency of the bridge. This seems to suggest that it is a forced rather than a free vibration. It is believed that this forced vibration is caused by the vertical oscillation of the vehicle mass on its suspension system.

TABLE 2

MAXIMUM OBSERVED AMPLITUDES OF VIBRATION

Bridge	Natural Frequency cps	Truck Velocity mph	Max. Amplitude / Max. Static Deflection
Townsend-Main	3.8	29.3	0.30
Gilbertville	4.0	30.5	0.33
Townsend-South	4.2	25.5	0.40
Ware	5.2	17.9	0.18
Conway	6.0	35.0	0.27

In order to verify this conclusion, the natural frequency of the vehicle was measured by means of the accelerometer described elsewhere in this paper. When the stationary vehicle was excited by artificial means, the natural frequency was found to be about 3.1 cps. Unfortunately, this value cannot be accepted with certainty because a modern vehicle is a complex, nonlinear dynamic system. The predominant frequency of the moving vehicle varies, depending upon the mode of vibration excited by the roughness of the roadway. Measurements were also taken of the vertical accelerations of the truck body as the vehicle moved along a roadway.

These records are difficult to interpret because of the many modes of vibration appearing in the acceleration trace. However, the predominant frequency apparently varies between 2 and 3 cps. Many of the test records of bridge deflection display frequencies of vibration falling within this range, even though the natural bridge frequency is well outside the range.

In Figure 9 the predominant frequency during the entire time of crossing is approximately the measured truck frequency and cannot possibly be related to the natural frequency of the bridge. In Figures 10, 11, 12, and 13 the vibration begins at the truck frequency but degenerates into the natural bridge frequency before the vehicle has left the span. This may be explained by the fact that the truck oscillations which had been induced on the approaches were damped out before the crossing had been completed. The test records shown in Figures 10 and 11 are for the same structure but the vehicle was traveling in opposite directions.

The frequency of the vibration at the beginning of the passage is different in the two cases, which is probably due to the fact that the predominant truck frequency was also different. The test record for the Ware Bridge, shown in Figure 14, is more difficult to interpret. Apparently the vertical truck motion is less severe than that encountered on the other bridges.

Most of the other test records obtained but not presented herein display the same frequency characteristics as those described above. It is therefore concluded that the frequency of vibration, at least during the first part of the crossing, is related to the natural frequency of the vehicle.

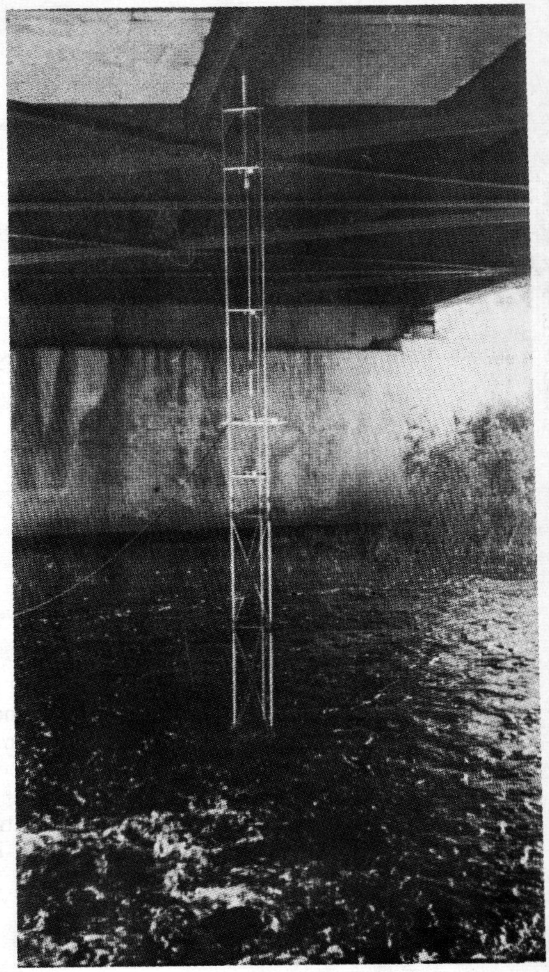


Figure 8. View of Deflectometer in use.

Amplitudes of Vibration

The first conclusion that is drawn from an inspection of the test records is that the amplitude of vibration is much larger than that which would be caused by a smoothly running load. In fact, the vibration caused by such a load is so small in comparison to the observed vibration as to be almost negligible. A second conclusion is that there is little evidence of the effect of sharp surface irregularities on the structure itself. In practically all cases the vibration is initiated when the truck enters the span and continues throughout the passage. Surface irregularities are probably a contributing factor but not the major cause of vibration. From a study of frequencies as well as amplitudes, it appears that the major cause of vibration, at least in these particular structures, is a periodic vertical oscillation of the vehicle as it enters the span.

A study of all test records fails to reveal any correlation between the amplitude of vibration and the speed of the vehicle. However, the range of test speeds is limited, and it is reasonable to expect that higher speeds would generally cause larger vibrations. This conclusion is based upon the assumption that higher speeds would induce more-violent truck oscillations.

The maximum observed amplitudes of vibration for all of the test bridges are shown in Table 2. These values are the numerical averages of the positive and negative displacements from the neutral position in one complete cycle of vibration. In general, these amplitudes were not constant across the span, and the average amplitude during passage of the load is somewhat less than the maximum. It should be emphasized that the maximum amplitudes shown in Table 2 are not necessarily related to the maximum total deflection. Only by coincidence does the maximum downward amplitude of vibration occur at the same point as the maximum static deflection. However, this paper is concerned primarily with the vibration itself, rather than with total deflection.

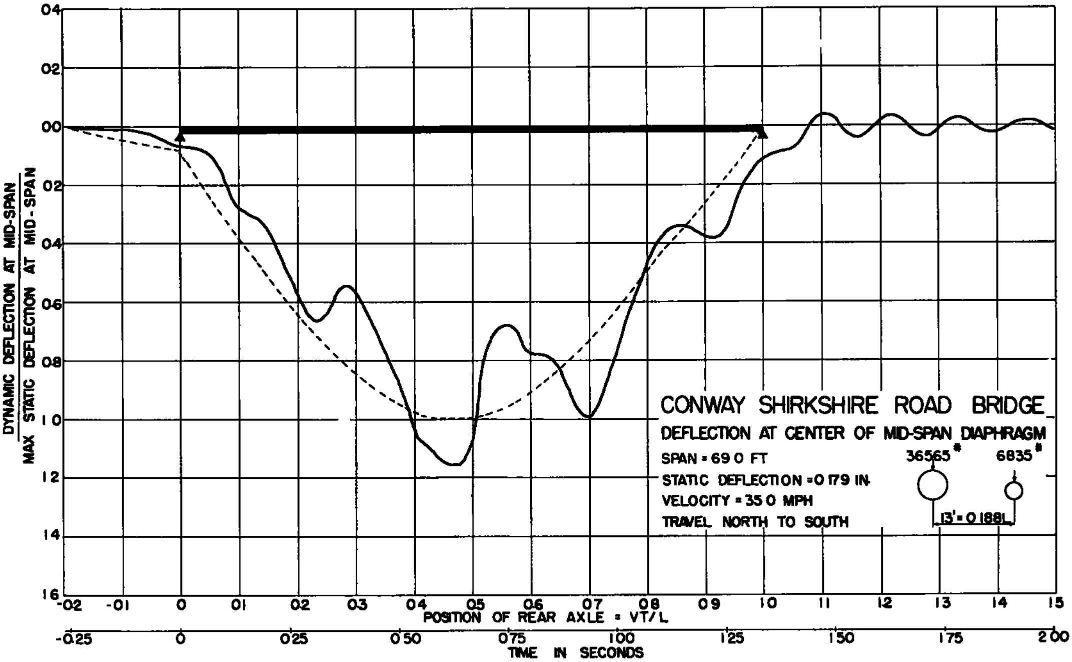


Figure 9.

The maximum amplitudes in Table 2 do not represent unusual cases. Several other records for each of the bridges displayed amplitudes nearly as large as those shown.

If, as concluded above, the major cause of vibration is an alternating magnitude of the applied force, the two-most-significant parameters are the amplitude of the truck oscillation and the natural frequency of the bridge in relation to the natural truck frequency. Although a very limited number of bridges are included, the maximum ampli-

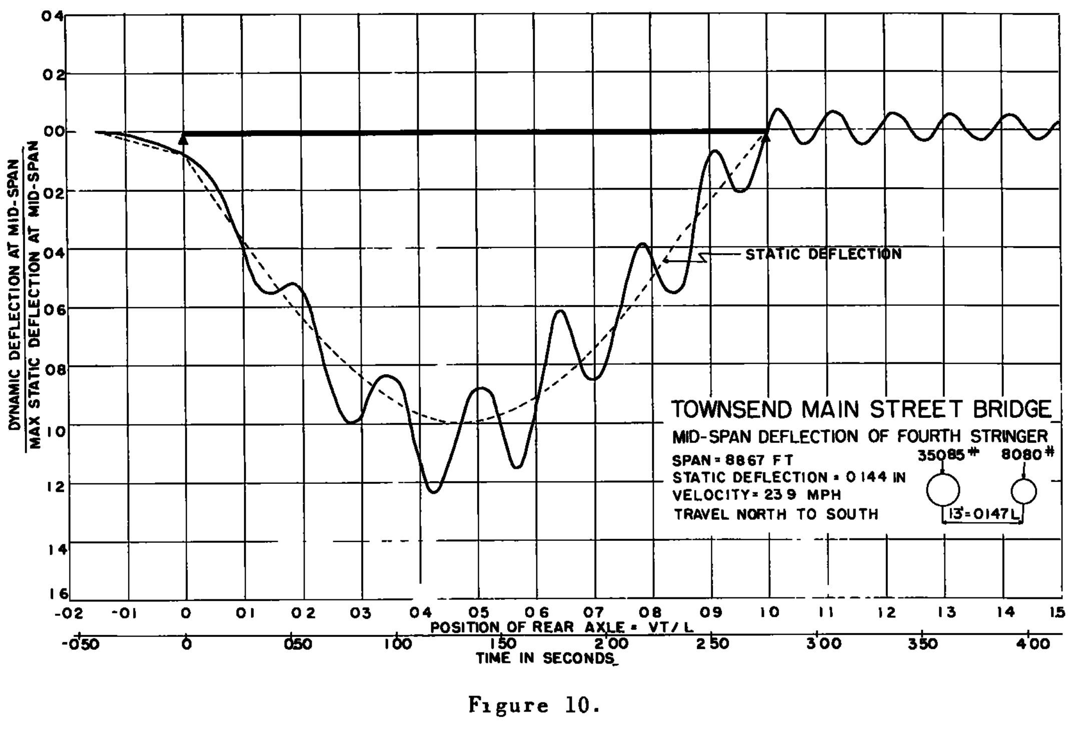


Figure 10.

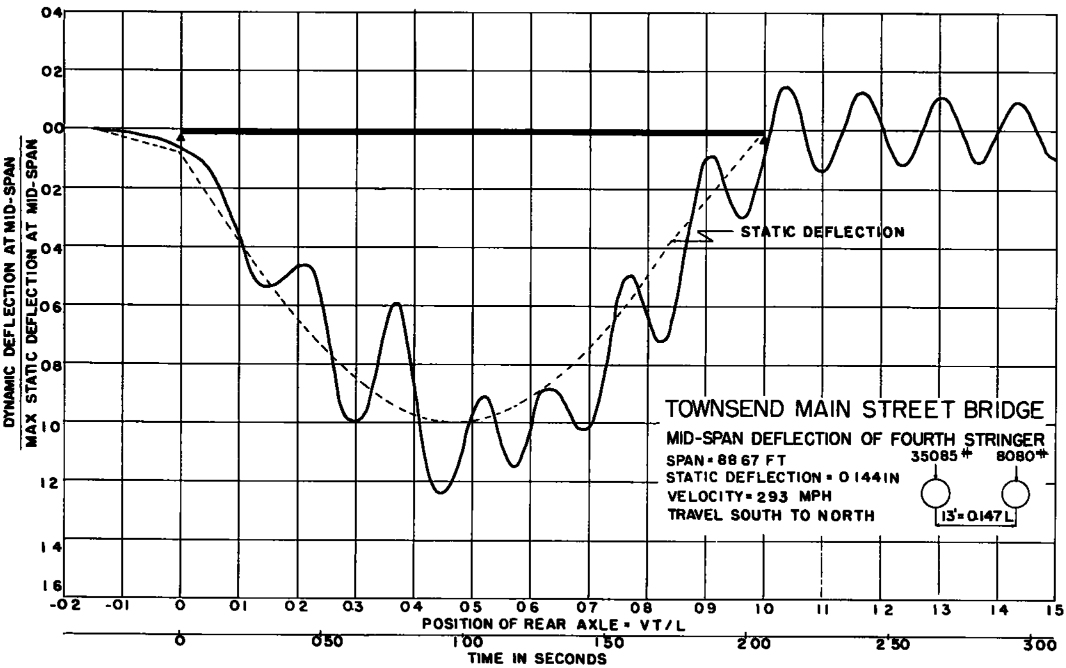


Figure 11.

tudes shown in Table 2 generally verify this statement.

The three bridges having low natural frequencies approaching that of the truck produced the largest amplitudes in proportion to the static deflections. The fact that the Townsend South Street Bridge displayed the largest amplitude can be explained by the fact that it has the roughest approach roadway of the three low-frequency bridges. The Conway Bridge, which experienced vibration greater than is consistent with its natural

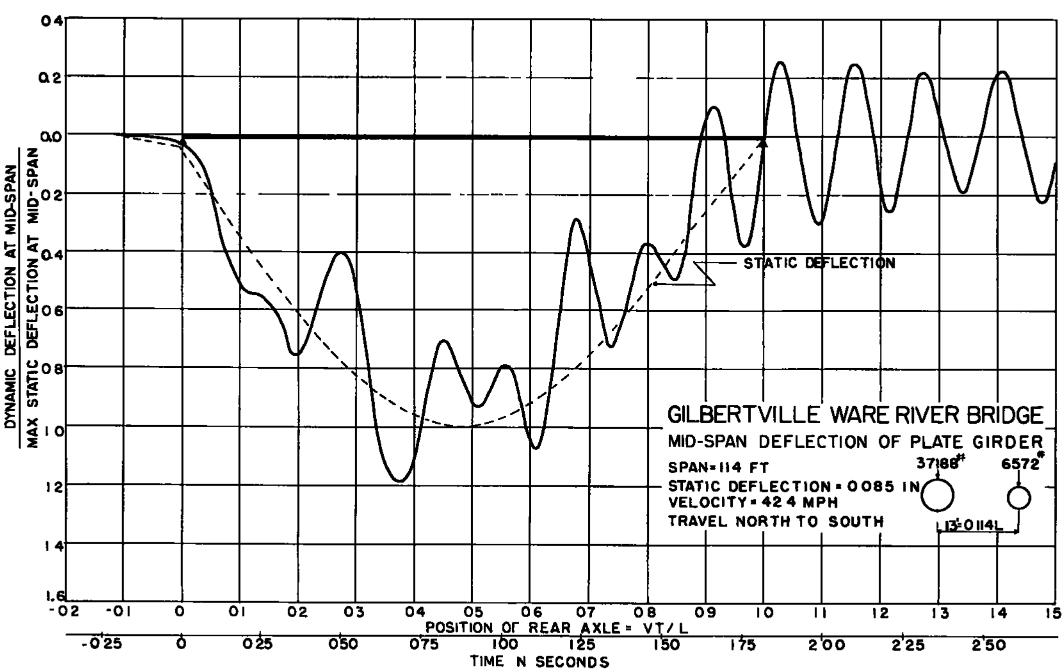


Figure 12.

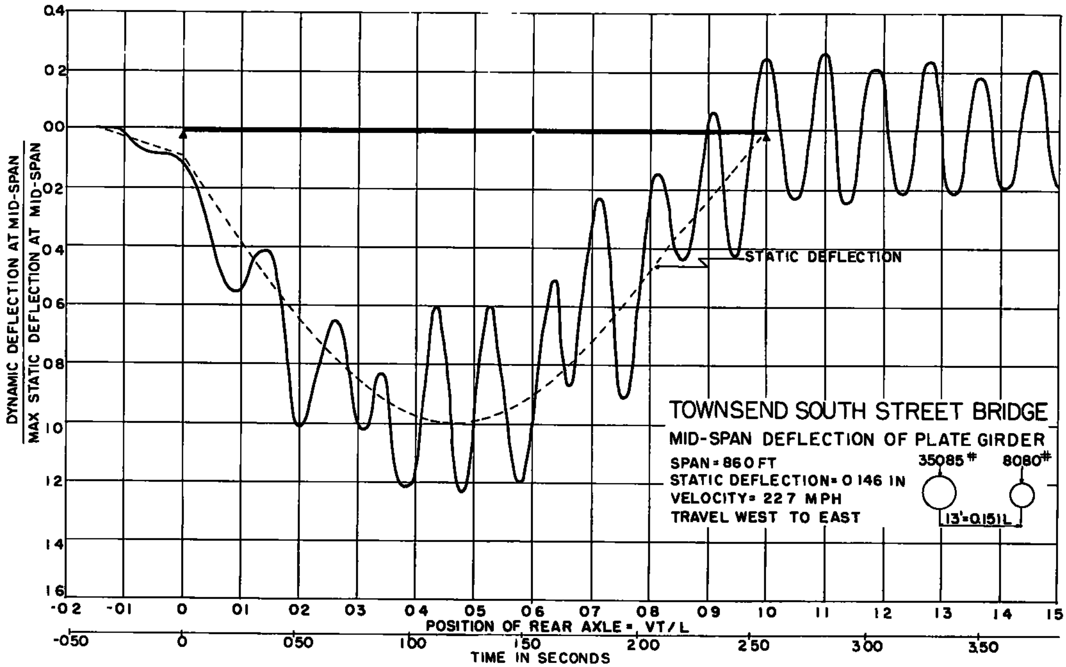


Figure 13.

frequency, has a gravel approach roadway which is the roughest of all five bridges.

It should be noted that the five test bridges are not unusual with regard to the roughness of the approaches. With the exception of the gravel approach roadway to the Conway Bridge, all approaches have bituminous surfaces in reasonably good condition.

Influence of Vehicle Springs

In an attempt to investigate the effect of the vehicle springs, additional tests were

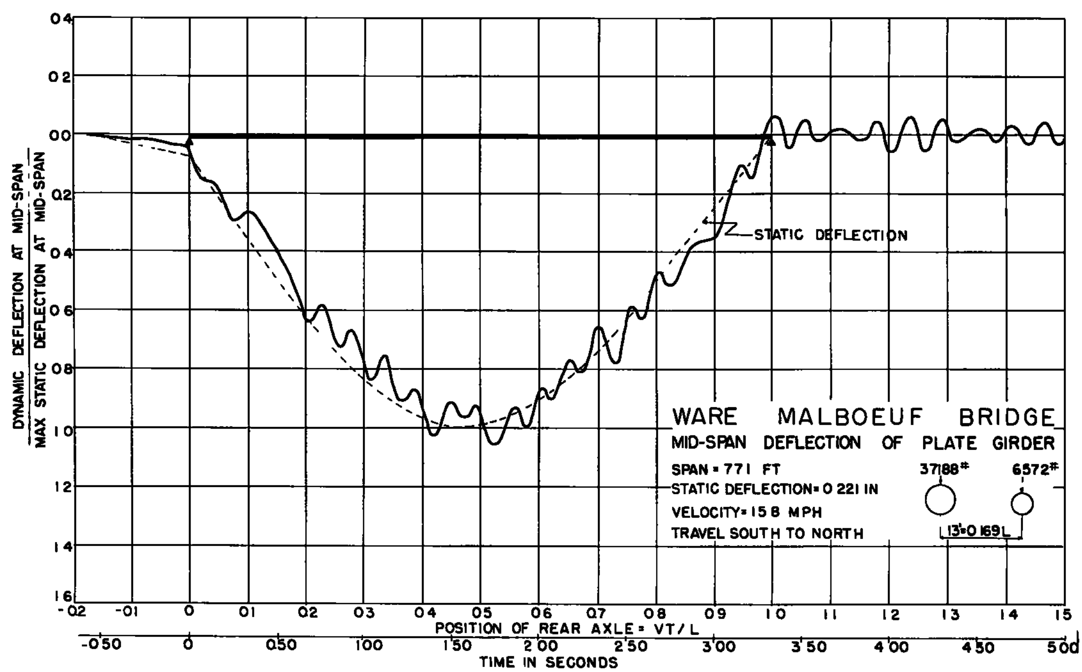


Figure 14.

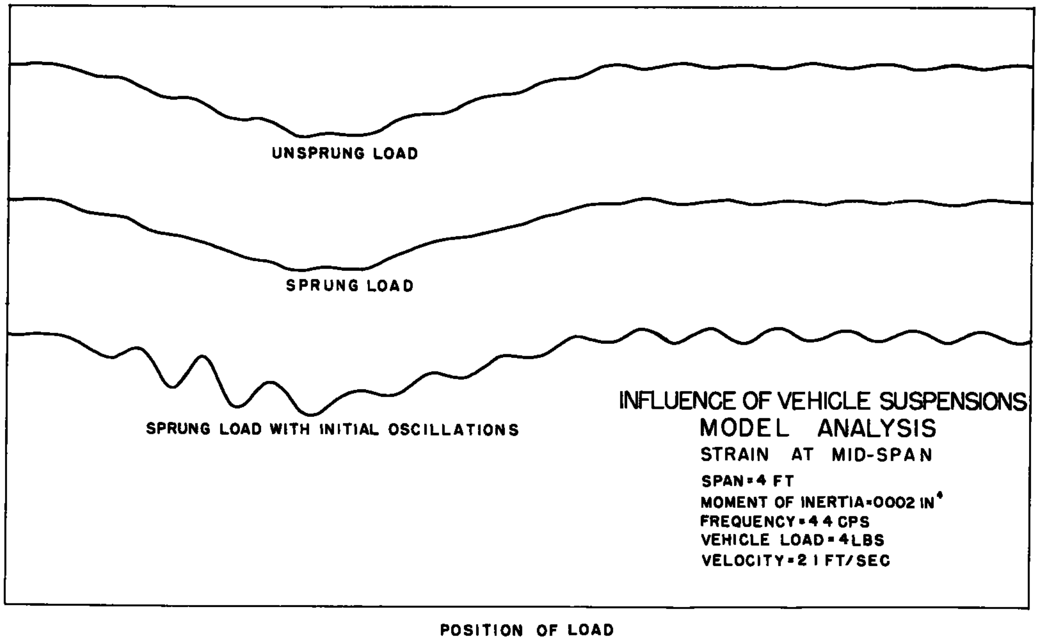


Figure 15.

made on the Townsend Main Street Bridge. In this test series the springs of the truck were blocked, and the vertical freedom of the vehicle was limited to the effect of the pneumatic tires. A different truck, similar to the one previously described, was used in these tests.

The results are shown in Figure 16, where a comparison is presented between a test run during which the springs were blocked and one during which the springs were acting normally. It is immediately obvious that the vibration is much-more severe in the former

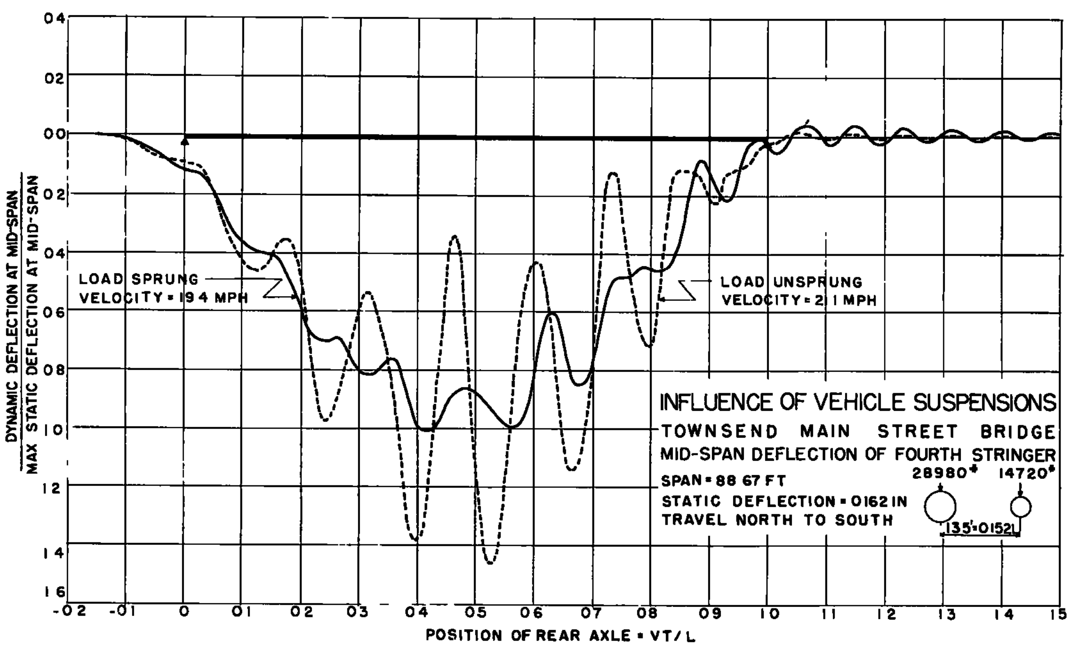


Figure 16.

case. In fact, the amplitude of vibration reached a rather alarming 56 percent of the static deflection. The large vibrations resulted primarily from the vertical oscillation of the vehicle on its tires, which was noticeably violent during all runs for which the springs had been blocked.

This experiment emphasizes two important conclusions: (1) the dynamic characteristics of the vehicle are of primary importance and (2) the general effect of the springs is to reduce the amplitude of vibration.

Damping Characteristics

In any theoretical solution of the vibration problem, the damping characteristics of the structure are of importance. The damping coefficients observed in the five test bridges are given in Table 3. The coefficient S is contained in the equation

$$A_n = A_o \, e^{-2\pi nS}$$

where A_n is the amplitude of the nth cycle and A_o is the initial amplitude. The coefficients given were obtained from the residual vibration by taking A_o at the first distinct cycle of free vibration after the load left the span and A_n at a convenient number of cycles thereafter. The damping is not exactly exponential, but the assumption is sufficiently accurate for practical purposes.

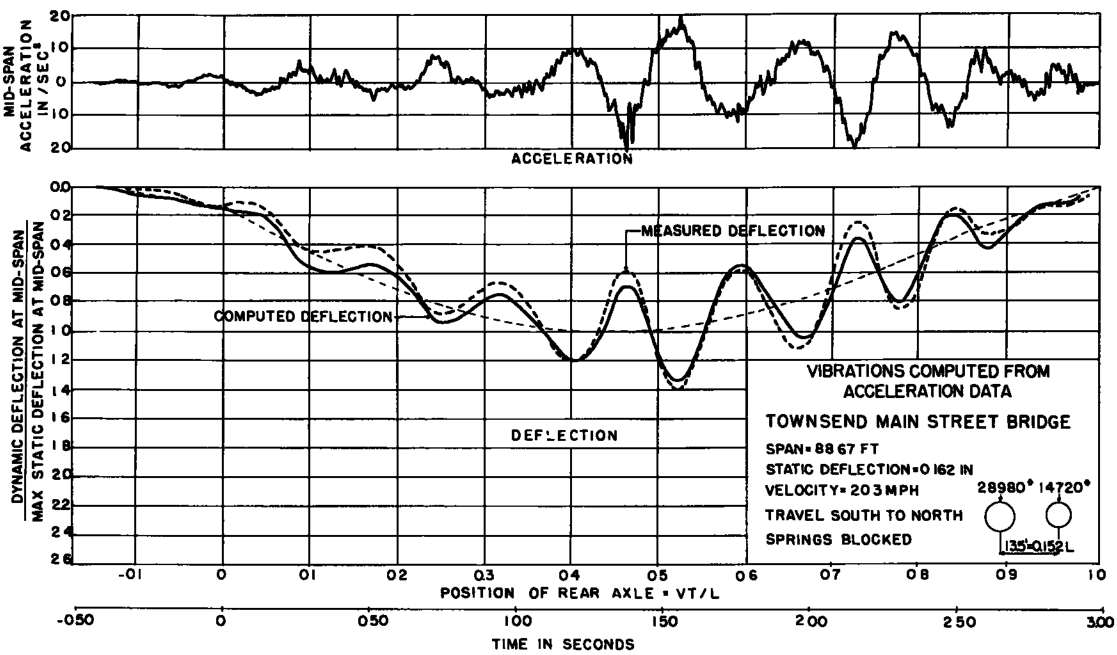


Figure 17.

It may be observed in Table 3 that the two bridge types display radically different coefficients and that there is remarkable agreement between the bridges of each type. However, the number of structures is probably not sufficient to warrant definite conclusions.

In general, the damping is less than might be expected. For example, in the case of girder bridges, the tenth cycle has an amplitude of approximately 62 percent of the first cycle. The same quantity for stringer bridges is 25 percent.

VERTICAL-ACCELERATION MEASUREMENTS

As a part of this project, experiments were conducted to determine the feasibility of measuring bridge vibrations by means of an accelerometer. In this procedure, deflec-

TABLE 3

DAMPING COEFFICIENTS

$A_n = A_o e^{-2\pi nS}$

Bridge	Type	Damping Coefficient, S
Townsend-Main	Stringer	.020
Gilbertville	Girder	.007
Townsend-South	Girder	.007
Ware	Girder	.008
Conway	Stringer	.024

tions are obtained by double integration of the acceleration records.

No commercial accelerometer was obtainable which had the desired ranges of frequency and amplitude. For this reason, an instrument was especially constructed for this project. It consists essentially of a mass supported partially by strain-gage wires. When vibration occurs, the strain in the wires, which is proportional to the acceleration, is recorded by the Sanborn system.

A typical acceleration record is shown in Figure 17. Theoretically, a complete deflection record can be produced by double integration of the acceleration record. However, the accelerations associated with the crawl deflection are very small, and an extremely accurate record would be required to produce a total deflection curve. This is not a serious defect, since the oscillation about the crawl deflection is of primary interest. A simplified procedure was adopted in which each half cycle of oscillation is considered independently and assumed to be a half cycle of simple harmonic motion. The maximum displacement in a half cycle is then given by $\ddot{A}\pi^2/T^2$, where $\ddot{A}$ is the maximum acceleration and T is the length in seconds of the half cycle.

A typical result of the approximate procedure described above is shown in Figure 17, where it is compared with a directly measured deflection record. The agreement is generally good, although the accelerometer cannot attain the accuracy of direct-deflection measurements. The residual vibration, which is not shown in Figure 17, can be accurately determined by the accelerometer, because the high-frequency accelerations are much-less prominent. Thus, the natural frequency and damping coefficient of a structure are easily determined by acceleration measurements.

The principal advantage of the accelerometer is that its use does not require access to the underside of the bridge. It need only be placed on the structure and connected to the recording device. Thus, the dynamic behavior of a bridge can be determined rapidly. If the oscillatory motion alone, rather than a complete deflection record, is desired, the use of an accelerometer shows considerable promise.

CONCLUSIONS

The experimental program was limited in scope, and the conclusions listed below should not be applied to structures or conditions dissimilar to those encountered in the test program.

1. The most-important single factor which influences the amplitude of vibration is the vertical oscillation of the vehicle as it approaches the span. Although the magnitude of oscillation depends upon the roughness of the roadway, this factor is of primary importance, even on surfaces which are normally considered to be smooth.
2. The natural frequency of the vehicle and the type of its suspension system are also of primary importance.
3. As a result of the foregoing, the amplitude of vibration is several times that which is predicted by computations based upon smoothly running loads.

ACKNOWLEDGMENTS

This study was undertaken as a part of the program of the Joint Highway Research Project which was established at the Massachusetts Institute of Technology by a grant from the Massachusetts Department of Public Works for research in the field of highway engineering. The work was done for the Bridge Division of the department. The Maintenance Division cooperated by furnishing vehicles and manpower for the tests.

The authors wish to express their appreciation to the following members of the department staff who have been most helpful throughout this study: J. G. Rundlett, bridge engineer; John McGovern, bridge-maintenance engineer; Earl Herrick, assistant bridge-maintenance engineer; and J. A. Cannon, superintendent of equipment and repairs. At M. I. T., Y. C. Loh, M. E. Alper, and Donald Gunn provided valuable assistance.