Substructure Influence on Dynamic Stress Response of Superstructures in Composite Bridges

An Experimental Study

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The Virginia Council of Highway Investigation and Research has conducted a study of the dynamic stress response and vibration characteristics of two highway bridges with simply supported composite spans. A test vehicle, simulating an H20-S16-44 standard loading, made runs on the bridges. Both bridges had identical 66-ft 5-in. spans, but one had higher and less stiff piers than the other. Comparison of the data indicates that the stiffness of the substructure has an influence on the response of the superstructure to dynamic loading.

•THE NUMBER of simply supported composite highway bridge spans constructed has substantially increased in the past 10 years, and it appears that this type of bridge is continuing, if not increasing, in popularity. Its wide use can be attributed to ease of construction, economy of materials and aesthetic value. Utilization of the concrete slab to act structurally in conjunction with the steel beam, in addition to its normal function of spanning between the beams, has enabled the engineer to select a lighter steel section, resulting in an appreciable saving in costs. However, the lighter steel section, although satisfactory from a stress consideration, is more susceptible to the dynamic loads of highway traffic. One of the concerns of structural engineers in this type of construction is its frequently objectionable vibration characteristics. In many instances, certain combinations of amplitudes and frequencies of the oscillations of the bridge cause the public apprehension over the safety of the structure. Further, these oscillations have contributed to cracking in the bridge deck and may cause fatigue distress in some instances.

In an attempt to determine if any particular design features of a bridge were related to excessive dynamic response, a general survey of vibration characteristics of composite highway bridges in Virginia was conducted in the summer of 1960 (1). In this survey, amplitudes and frequencies of 67 composite spans excited by the crossing of a truck loaded to simulate an H15 standard loading were measured with an accelerometer pickup, a vibration meter, and a Brush recorder. In examining the resulting data, it was observed that in several instances bridges with very similar superstructures exhibited markedly different vibration responses. This led to a careful study of other bridge features that might influence the superstructure response, and the observation that the tops of relatively high piers oscillated with small, although measurable, amplitudes.

To examine in some detail the influence of the substructure on superstructure response to dynamic loads, a testing program was planned by the Virginia Council of

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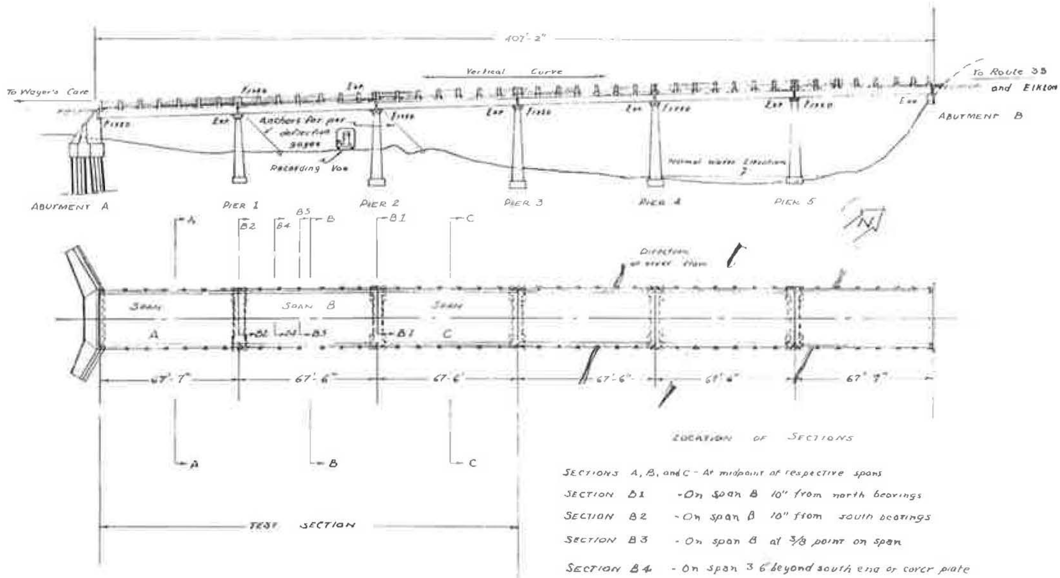

Figure 1. Profile and plan of Weyer's Cave Bridge, indicating test sections.

Highway Investigation and Research, with the cooperation of the Structures and Applied Mechanics Division of the U. S. Bureau of Public Roads. Two bridges were selected as test structures. Each included in its superstructure a Virginia Department of Highways Standard 66-ft 5-in. span, virtually identical, with a 24-ft clear roadway width, a 7 ½-in. composite concrete slab and four 36-in. wide flange steel stringers. All comparative data listed in this paper are for the two like spans, one in each structure. These two spans are indicated as span B in Figures 1 and 2. The difference in the two structures was in the height of the similarly designed piers. The first structure, which was tested in the summer of 1961 is located on Rt. 276 near Weyer's Cave, Va., and is composed of six 66-ft 5-in. spans supported on one gravity abutment, one shelf-type abutment and five solid piers, on spread footings, of unsupported heights ranging from 18 to 22 ft measured from ground level to top of pier cap. The second structure, which was tested in the summer of 1962, is located on Rt. 729 across the Hazel River near Culpeper, Va., and is composed of three spans, a 66-ft 5-in. center span and two end spans of 61 ft 5 in. each. The substructure here consists of two shelf-type abutments and two solid piers of unsupported heights ranging from 14 to 15 ft, all of which are supported on timber piles. Pertinent details of both structures, including plan and elevation views, cross-sections of the 66-ft 5-in. spans, and details of the piers supporting the 66-ft 5-in. spans are shown in Figures 1, 2, 3 and 4.

The instrumentation and recording equipment, provided, installed and operated by personnel of the Structures and Applied Mechanics Division of the U. S. Bureau of Public Roads, were essentially the same as that used in the Illinois AASHO Road Test and in a number of similar bridge testing programs in other states. The equipment included an instrument trailer outfitted with oscillographs and amplifiers capable of permanently recording on sensitized paper through light-beam galvanometers the output of up to 48 strain or deflection gages. In effect, for the time of passing of the test vehicle on each of its runs, a complete recording was made for live-load strain at 34 bridge positions and live-load deflection at 12 positions. Pneumatic traffic tubes were installed at each end of the two bridges and several intermediate positions to operate air switches which recorded a signal on the sensitized paper each time a wheel crossed them. From these recordings the position of the test vehicle could be related to the resulting stresses and deflections. The oscillogram traces were easily converted to unit stresses or deflections by multiplying the ordinates measured from

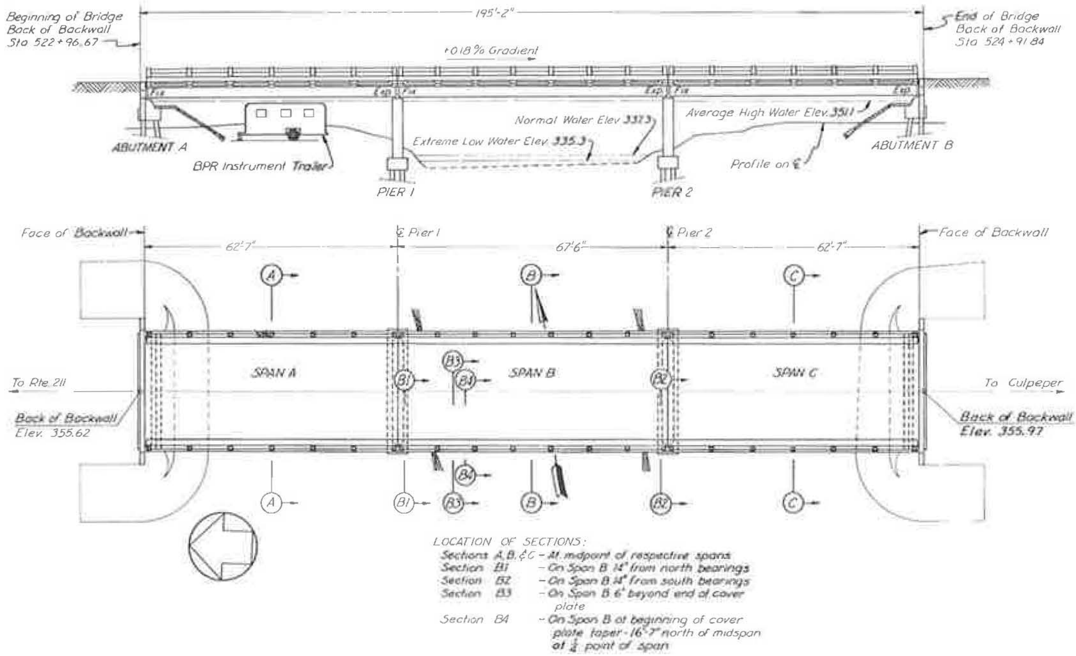

Figure 2. Profile and plan of Hazel River Bridge, indicating test sections.

the traces by constants computed from the equipment sensitivity and the individual gage characteristics. In addition to the oscillographs for recording the traces representing stresses and deflections, the instrument trailer also housed complete facilities to develop the sensitized record paper.

The testing procedure for both the Weyer's Cave Bridge and the Hazel River Bridge was essentially the same, although a greater number of runs were made on the Hazel River Bridge. The test results provided a very complete account of the stresses and deflections developed in both structures from the heavy H20-S16 loading passing over the bridges at a complete range of normal speeds and a full range of transverse positions.

The test procedure consisted of a 3-axle tractor-trailer, loaded to simulate an H20-S16-44 standard loading, passing across the test spans at speeds from creep (approximately 5 mph) to 45 mph in 5-mph increments and in three lateral positions for the Weyer's Cave Bridge and in five lateral positions for the Hazel River Bridge. Ninety-six crossings of the test vehicle were made in the Weyer's Cave tests and 189 crossings at Hazel River. The bridge responses measured and analyzed were midspan live-load deflections, live-load strains at 34 selected positions (32 on the Hazel River Bridge), and longitudinal displacement at pier tops.

From these measurements, the following characteristics of the test structures were determined and compared:

1. Transverse live-load distribution to the four stringers,
2. Position of the neutral axis in the stringers,
3. Fundamental frequency of vibration,
4. Logarithmic decrement of the bridge oscillation,
5. Amplitudes of vibration, and
6. Impact factors based on strains and deflections.

Only a portion of the results of this investigation is presented in this paper. However, detailed reports of these two tests, entitled A Dynamic Stress Study of the Weyer's Cave Bridge, 1963, and A Dynamic Stress Study of the Hazel River Bridge, 1964, are available from either the U. S. Bureau of Public Roads, Structures and

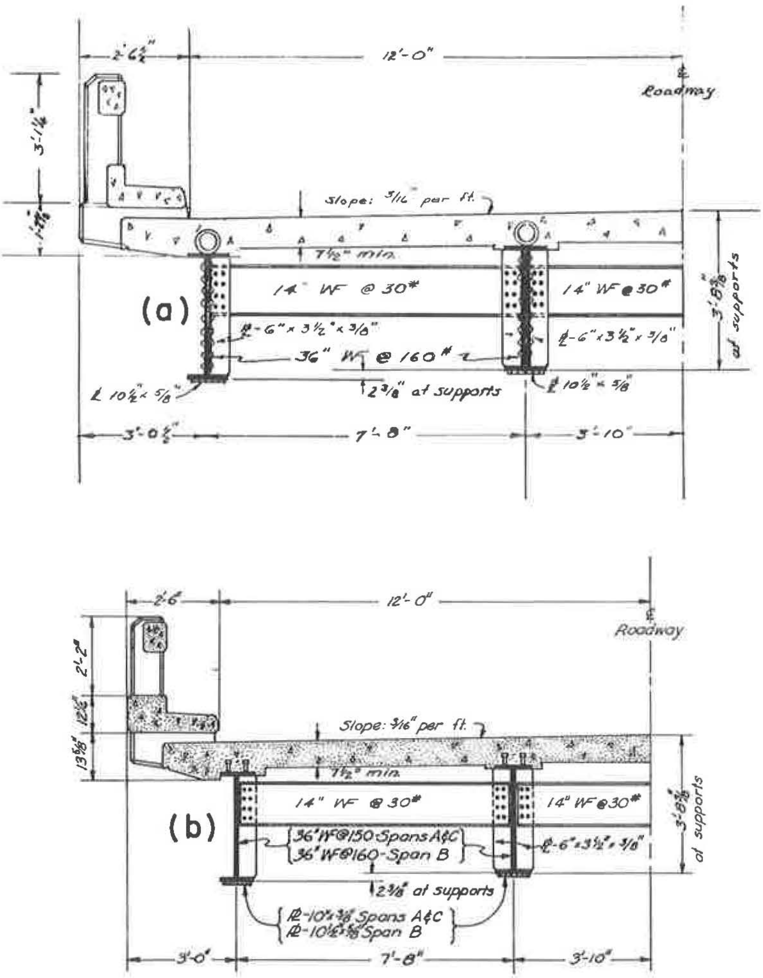

Figure 3. Half-transverse section of virtually identical 66-ft 5-in. spans B: (a) Weyer's Cave Bridge; and (b) Hazel River Bridge.

Applied Mechanics Division, or the Virginia Council of Highway Investigation and Research. Included in the second report are comparisons of the various measured responses of the superstructures and pier tops of the two bridges with the supporting test data.

Four of the most important conclusions are presented and discussed with the supporting experimental data in the following sections of this paper.

1. The lower flange midspan stresses and deflections of the Hazel River Bridge were appreciably smaller for each speed than the corresponding values for the Weyer's Cave Bridge. These comparisons can be observed in Table 1 which indicates that for the interior beams, 2 and 3, the Weyer's Cave Bridge stresses range from 8.2 percent (at 40 mph) to 23.4 percent (at 10 mph) above the corresponding Hazel River Bridge stresses. Also, the Weyer's Cave Bridge lower flange midspan deflections were larger than the corresponding values for the Hazel River Bridge. The percentage differences ranged from a low of 5.7 percent (20 mph) to 31.0 percent (10 mph).

2. The amplitudes of oscillation of the stringers increased with increased speeds for both bridges. The amplitudes for the Hazel River Bridge stringers were appreciably

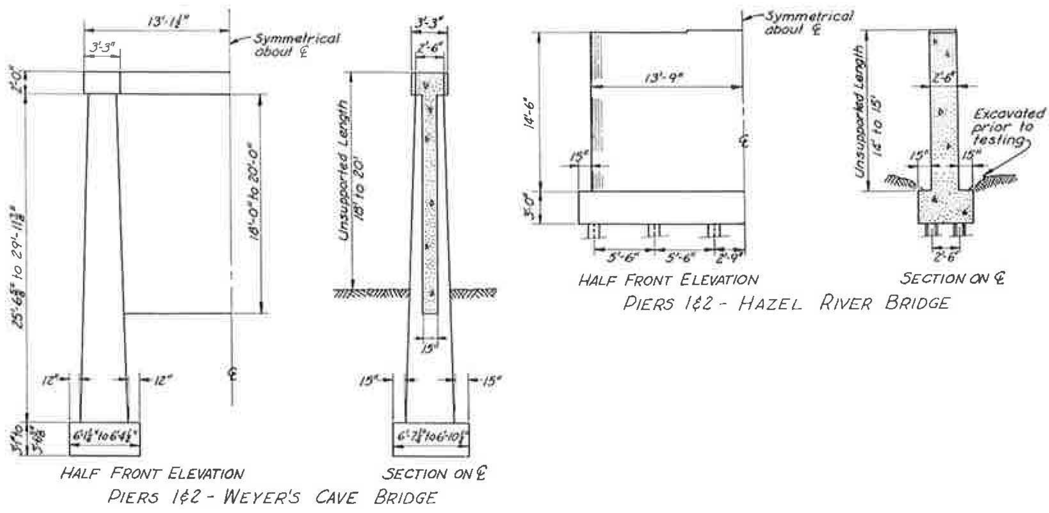

Figure 4. Details of pier supporting two 66-ft 5-in. spans B of Weyer's Cave Bridge and Hazel River Bridges.

TABLE 1

COMPARISON OF MIDSPAN STRESSES, DEFLECTIONS, DOUBLE AMPLITUDES AND PIER TOP MOVEMENTS

Speed			Peak Longitudinal Displacements of Pier Tops from Equilibrium Position						Midspan Peak Stresses[a] (psi)		Midspan Peak Deflections[a] (in.)		Midspan Double Amplitude[a] (in.)	
Nominal (mph)	Average (mph)		Pier 1 (in.)		Pier 2 (in.)		Pier 1 + Pier 2 (in.)							
	Weyer's Cave	Hazel River	Weyer's Cave	Hazel River	Weyer's Cave	Hazel River	Weyer's Cave	Hazel River	Weyer's Cave	Hazel River	Weyer's Cave	Hazel River	Weyer's Cave	Hazel River
Creep	-	3.6	0.017	0.013	0.014	0.011	0.031	0.024	2710	2250	0.185	0.145	0.015	0.010
10	9.6	9.2	0.017	0.014	0.018	0.011	0.035	0.025	2885	2340	0.190	0.145	0.035	0.010
15	15.7	15.0	0.017	0.014	0.020	0.012	0.037	0.026	2630	2320	0.175	0.160	0.037	0.030
20	21.0	19.8	0.020	0.016	0.021	0.012	0.041	0.028	2930	2530	0.185	0.175	0.060	0.010
25	26.4	24.6	0.018	0.016	0.015	0.012	0.033	0.028	2580	2330	0.180	0.150	0.047	0.020
30	31.2	29.2	0.020	0.013	0.021	0.012	0.041	0.025	2900	2600	0.185	0.155	0.055	0.015
40	40.3	37.6	0.025	0.016	0.018	0.012	0.043	0.028	2910	2690	0.200	0.175	0.092	0.040
Flank	41.7	45.7	0.020	0.015	0.027	0.014	0.047	0.029	2925	2460	0.190	0.175	0.083	0.055

[a]Average of beams 2 and 3.

less than the corresponding amplitudes of the Weyer's Cave Bridge, as can be observed in Table 1 for the two interior beams, 2 and 3, for centerline runs. Figures 5, 6 and 7 show the double amplitudes plotted against speed for each of the four stringers, for the three instrumented spans of each structure and for test vehicle runs on the centerline as well as the two curb positions. As previously mentioned, span B of each structure is the one for which comparisons can be made as they are virtually identical Virginia Department of Highways 66-ft 5-in. span standard designs. Whereas spans A and C of the Weyer's Cave Bridge are identical to the spans B, spans A and C of the Hazel River Bridge are 61 ft 5 in. in length.

It can be observed from Figures 5, 6 and 7 that the double amplitudes of the midspan positions of the four beams are sensitive to the path of the test vehicle. The double amplitudes of beam 1 are the greatest when the test vehicle is in the east curb lane and the double amplitudes of beam 4 are the greatest when the test vehicle is in the west curb lane.

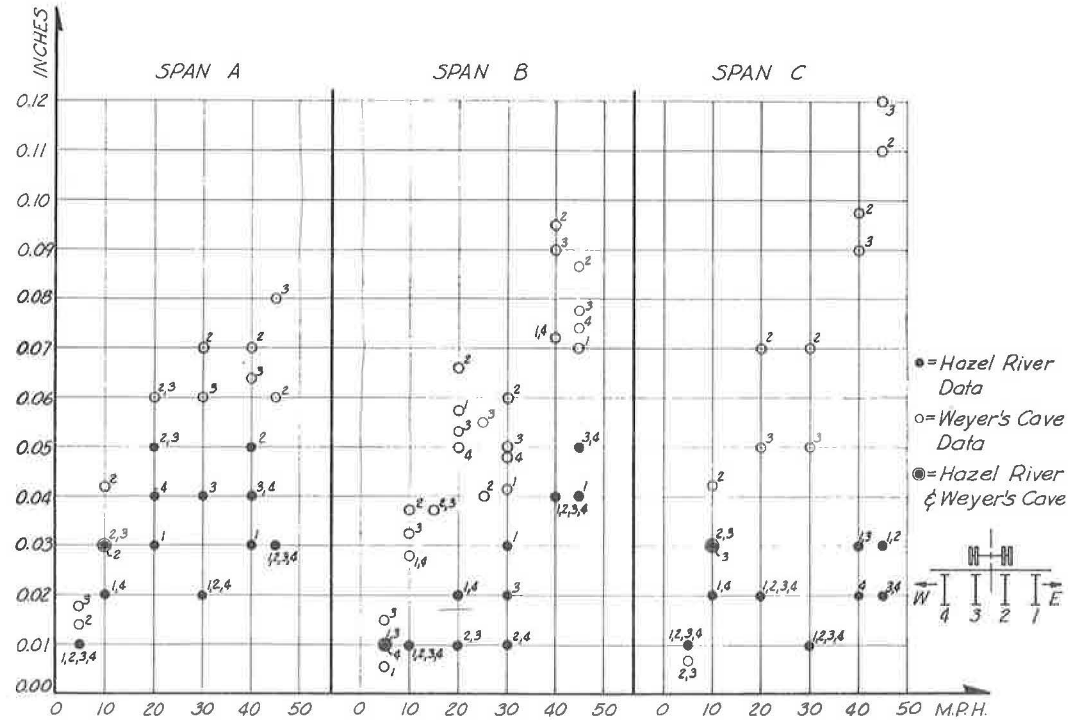

Figure 5. Average midspan double amplitudes vs nominal speeds (centerline).

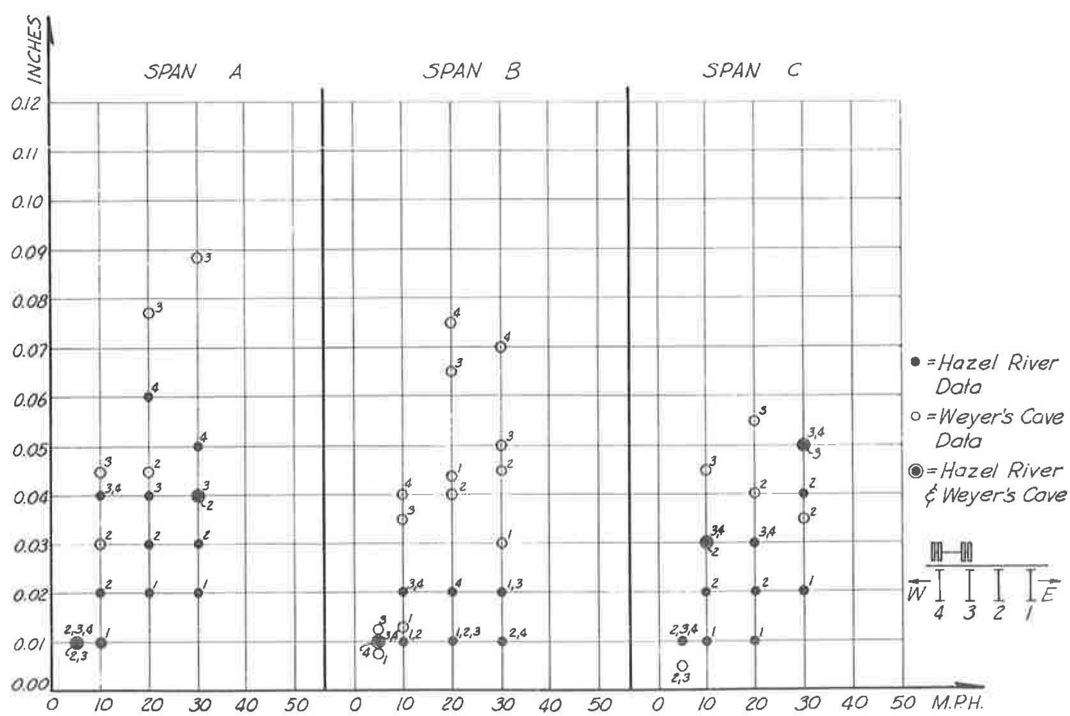

Figure 6. Average midspan double amplitudes vs nominal speeds (west curb).

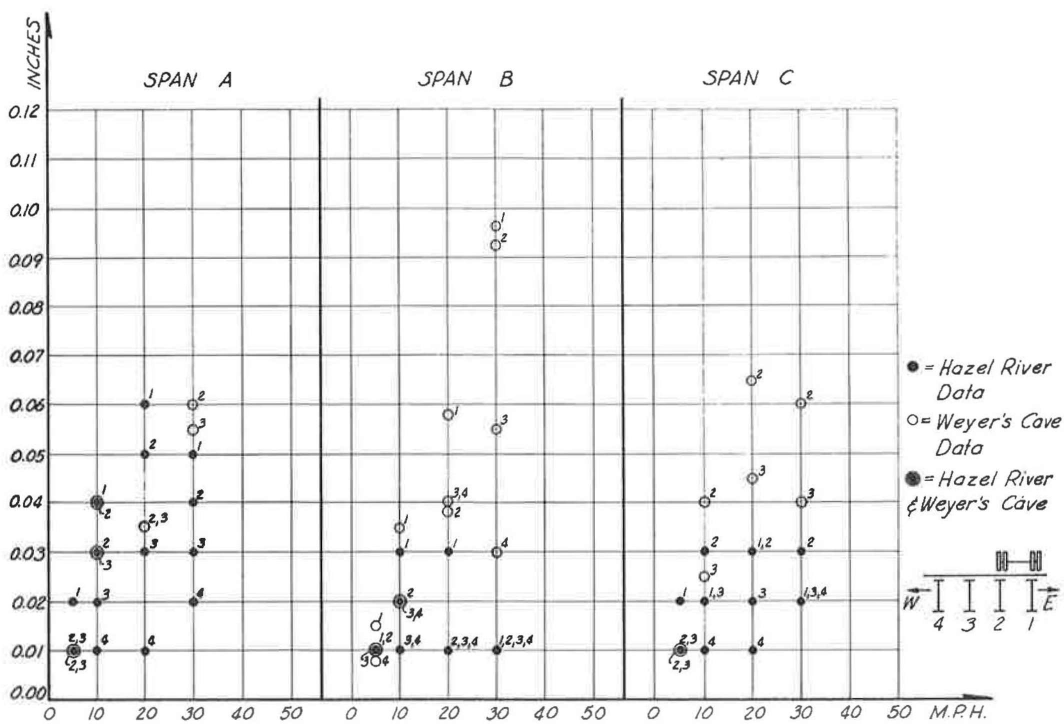

Figure 7. Average midspan double amplitudes vs nominal speeds (east curb).

3. The pier top longitudinal displacements were of comparable magnitude to the vertical oscillations of the deflected stringers at the midspan positions and were sensitive to the lateral position of the test vehicle on its runs. The magnitudes of the pier top movements of the Hazel River Bridge were considerably less than those of pier top movements of the Weyer's Cave Bridge.

The pier top displacement data are summarized for centerline runs in Table 1 and are given in more detail, including east and west curb runs, in Table 2.

For the centerline runs, the two gages on each pier moved in unison, but for the curb runs, there were noticeable differentials in the movements of the two gage positions. Larger pier movements occurred on the side of the test vehicle location, indicating slight twisting of the pier about a vertical center axis. The peak movements ranged from 0.007 to 0.018 in., with a great majority of the values falling in the 0.011- to 0.016-in. range.

It is interesting to compare the size of the pier movements with the double amplitudes (Table 1) of the midspan position of the span B stringers for each bridge. The pier movements for the Hazel River Bridge are consistently smaller than the corresponding values for the Weyer's Cave Bridge; however, they are larger in proportion to the double amplitudes of the midspan position of the 66-ft 5-in. span B. This, of course, follows from the fact that the midspan double amplitudes of span B of the Hazel River Bridge are substantially smaller than the corresponding values for the Weyer's Cave structure.

In the analysis of the strain gage and deflection gage data taken from the oscillogram tapes, the direction of the displacements and signs of the strains, as well as the magnitudes of the displacements and stresses, can be readily determined. Also on the oscillogram tapes were signals recorded when the test vehicle crossed air hoses laid across the bridge decks. From this information, it was observed that the pier tops were displaced toward the span on which the vehicle was located. It was also observed that the frequency of vibration of the piers with the vehicle off the structure was

TABLE 2

PEAK LONGITUDINAL DISPLACEMENTS OF PIER TOPS FROM EQUILIBRIUM POSITION

Nominal Speed (mph)	Pier[a]	Hazel River Under Beam 2 (in.)	Hazel River Under Beam 3 (in.)	Hazel River Avg. (in.)	Weyer's Cave Under Beam 2 (in.)	Weyer's Cave Under Beam 3 (in.)	Weyer's Cave Avg. (in.)
(a) East Curb Runs							
Creep	1	0.015	0.010	0.012	0.018	0.012	0.015
	2	0.010	0.007	0.008	0.016	0.010	0.013
10	1	0.014	0.009	0.012	0.019	0.020	0.020
	2	0.012	0.007	0.010	0.016	0.007	0.012
20	1	0.017	0.011	0.014	0.021	0.017	0.019
	2	0.011	0.010	0.010	0.017	0.010	0.014
30	1	0.013	0.012	0.012	0.018	0.015	0.017
	2	0.013	0.009	0.011	0.020	0.018	0.019
(b) Centerline Runs							
Creep	1	0.013	0.013	0.013	0.017	0.017	0.017
	2	0.010	0.012	0.011	0.015	0.013	0.014
10	1	0.015	0.014	0.014	0.017	0.016	0.017
	2	0.011	0.011	0.011	0.017	0.018	0.018
20	1	0.016	0.017	0.016	0.019	0.020	0.020
	2	0.011	0.014	0.012	0.020	0.022	0.021
30	1	0.012	0.014	0.013	0.020	0.020	0.020
	2	0.012	0.013	0.012	0.022	0.020	0.021
40	1	0.017	0.016	0.016	0.026	0.023	0.025
	2	0.011	0.013	0.012	0.016	0.019	0.018
Flank	1	0.016	0.014	0.015	0.019	0.020	0.020
	2	0.014	0.015	0.014	0.033	0.020	0.027
(c) West Curb Runs							
Creep	1	0.010	0.014	0.012	0.014	0.018	0.016
	2	0.010	0.013	0.012	0.012	0.015	0.014
10	1	0.010	0.016	0.013	0.017	0.016	0.017
	2	0.008	0.013	0.010	0.010	0.015	0.013
20	1	0.010	0.018	0.014	0.018	0.021	0.020
	2	0.007	0.016	0.012	0.014	0.019	0.017
30	1	0.009	0.015	0.012	0.017	0.018	0.018
	2	0.010	0.015	0.012	0.014	0.019	0.017

[a]Piers 1 and 2 support virtually identical spans B of the two bridges.

in close agreement with the frequency of the superstructure. This would indicate that the movements of the piers resulted from a forced vibration, contributed by the superstructure.

4. Logarithmic decrements as determined from the recorded traces of selected representative strain and deflection gages indicated that the oscillations of the Hazel

TABLE 3

AVERAGE LOGARITHMIC DECREMENTS

Position	Lower Flange Strains		Deflection Gages	
	Weyer's Cave	Hazel River	Weyer's Cave	Hazel River
Midspan A	0.137	0.117	0.074	0.134
Midspan B	0.064	0.143	0.063	0.131
Midspan C	0.085	-[a]	0.067	0.170
Pier 1	-	-	0.113	-[a]
Pier 2	-	-	0.108	-[a]

[a]Strains and deflections at these positions not adaptable to determinations of logarithmic decrements.

River Bridge damped out more quickly than did the oscillations of the Weyer's Cave Bridge. Logarithmic decrements of the oscillations recorded on the oscillograms were determined for as many traces as could be used for this purpose. However, only the oscillograms showing a regular decay pattern representative of viscous damping were used to compute the decrements and most of the strain and displacement recordings for the two short (61-ft 5-in.) end spans of the Hazel River Bridge were eliminated from consideration. Further, logarithmic decrements could not be determined from any of the traces of the Hazel River Bridge pier top movements because of the rapid dying out of the oscillations.

It is noted for comparison in Table 3 that the logarithmic decrements for the 66-ft 5-in. span B of the Weyer's Cave Bridge averaged 0.064 for the strain traces of the four stringers and 0.063 for the deflection traces. For span B of the Hazel River Bridge, the logarithmic decrements were 0.143 for the strain traces and 0.131 for the deflection traces. It is evident that the vibrations of the Hazel River Bridge center span died out consistently quicker than did the vibrations of the Weyer's Cave Bridge.

Also for comparison, it is noted that logarithmic decrements of 0.113 and 0.108 were determined for the two instrumented pier tops of the Weyer's Cave Bridge, whereas the pier top oscillations of the Hazel River Bridge were of such short time duration and of such an irregular nature that logarithmic decrements could not be determined. These relative results are consistent with what one would predict, inasmuch as the Weyer's Cave Bridge piers are 18 to 22 ft high and the Hazel River Bridge piers are 14 to 15 ft high, in each instance measured from the ground level to the top of pier cap.

SUMMARY AND CONCLUSIONS

The results of this study indicate that when a vehicle crosses a simply supported span, the tops of the supporting piers are displaced toward the center of the span. The amount of this displacement varies substantially with the stiffness of the piers.

Span B of the Weyer's Cave Bridge, virtually identical to span B of the Hazel River Bridge but supported on higher more flexible piers, showed the following noticeably different responses from the dynamic loading:

1. Midspan peak stresses and deflections were generally higher for the Weyer's Cave Bridge than for the corresponding measurements for the Hazel River Bridge;
2. Midspan amplitudes of vibration were greater for the Weyer's Cave Bridge than the corresponding values for the Hazel River Bridge; and
3. Vibrations were damped out less rapidly in the Weyer's Cave Bridge than in the Hazel River Bridge.

It may be concluded from this investigation that the stiffness of the substructure elements can, in some cases, affect the characteristics of the superstructure under dynamic loading.

Although it is obvious that excessive vibration in bridge structures of this type can be controlled by stiffening the bridge stringers themselves, it appears from the results of these tests that the superstructure vibration can be meliorated, to some extent, by selecting a more rigid substructure. Frequently, in the selection of types of piers to be used for a bridge structure, a choice is made between slender more flexible piers with a saving in material and more costly formwork, or heavier more bulky piers which utilize more material but require less expensive formwork. It is suggested that the second alternative would be the better choice for longer spans where objectionable vibrations are most likely to develop. It can also be pointed out, for example, that an increase from 20- to 24-in. diameter pier columns would result in more than doubling the moment of inertia of the columns and probably in a more tolerable vibration condition in the bridge deck.

REFERENCE

1. Progress Report No. 1, Vibration Survey of Composite Bridges. Virginia Council of Highway Investigation and Research.