Dynamic Studies of Two Continuous Plate-Girder Bridges

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This paper describes and presents partial results of tests conducted by the Bureau of Public Roads and the Oregon State Highway Department on two three-span, continuous plate-girder bridges in Oregon. Measurements include deflection of the girders and strain in the girders, stringers, and floor beams under test vehicles approximating the AASHO H20-S16-44 truck at speeds of 5 mph. to 45 mph. Test data are presented in curves showing variation of stress and deflection with speed. Comparisons are made with stresses and deflections calculated according to existing specifications. Measured stresses in general are found to be lower than calculated values. By comparison of the results from the two bridges, deck surface condition is considered to be an important factor in the vibration of one of the bridges.

● BRIDGE specifications have been based on calculated stresses from static loads, empirical impact allowances, and satisfactory past performance. The need for strain measurements in structural members under moving loads has been recognized. Since the development and application of electronic strain-recording equipment made these measurements possible, a number of reports of tests of bridges under moving loads have been published. The present report adds two three-span, continuous, plate-girder bridges to a list that includes a pony truss (1) and rolled beam spans (2, 3).

In the spring of 1953 the Physical Research Branch of the Bureau of Public Roads completed the assembly of a portable bridge-test unit. The unit, contained in a house trailer, consists of the equipment necessary to record varying strains and deflections and a darkroom for developing the oscillograms. Agreement was reached between the Bureau of Public Roads and the Oregon State Highway Department for the test unit to be employed on an Oregon highway bridge during the summer of 1953 as a cooperative project of the two agencies.

The North Dillard Bridge over the South Umpqua River on the Pacific Highway (US 99) in Douglas County was selected for the test. It is a three-span, continuous, plate-girder bridge with two variable-depth girders. The selection of this particular bridge was based primarily on three factors: (1) an appreciable amount of vibration has been observed in the structure; (2) most of the floor panels had transverse deck cracks, indicating the possibility that some damage was being caused by the vibration; and (3) no previous report had covered dynamic stresses and vibrations on a highway bridge of this type.

The spans are 121 feet, 160 feet and 121 feet, center to center of bearing. The girder depth, back to back of flange angles, varies from 5 feet 10½ inches at the outside ends of the side spans and at the center of the center span to 9 feet 2½ inches over the interior piers. The four piers, of dumbell pattern, are founded on rock. The heights from footing to bearing are about 35 feet for the exterior piers, 43 feet for the north interior pier, and 38 feet for the south interior pier. There is a 50-foot 3-inch reinforced-concrete approach span at each end. The deck is 30 feet wide with two 3-foot 6-inch sidewalks.

Figures 1 and 2 are views of the bridge. Figure 3 shows critical details of the structure. The bridge is on tangent. The profile of the bridge is a 600-foot convex vertical curve with the deck grade varying from level to 0.8 percent. The south end is 2.77 feet lower than the north end.

The bridge was opened to traffic on November 29, 1950. The average daily traffic is 7,000 with 13 percent heavy trucks and combinations. The south city limit of the town of Winston is 0.2 mile north of the bridge, and the edge of the unincorporated town of Dillard is 0.5 mile south. Along the highway between the two towns are three roadside fruit stands, several residences, and road connections to a sawmill and a gravel plant. These features were important considerations in the operation of test vehicles, since they ne-

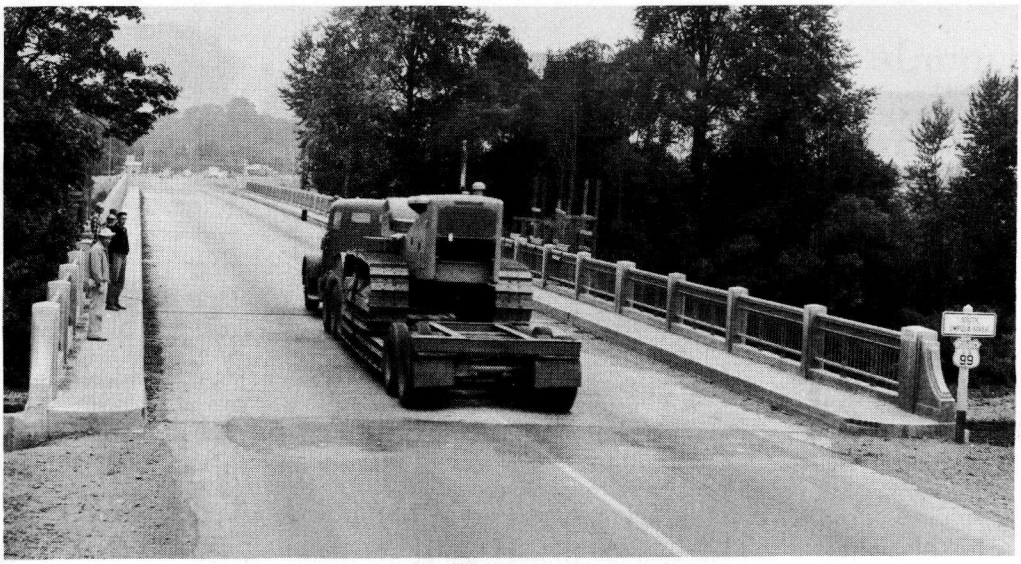


Figure 1. View of Vehicle A on North Dillard Bridge.

cessitated additional precautions to prevent traffic interference.

The program consisted of four series of tests with vehicles approximating the H20-S16-44 truck traveling at speeds of 5 mph. to 45 mph. In Series I, girder deflections and positive moment strains were measured. Negative moment girder strains were

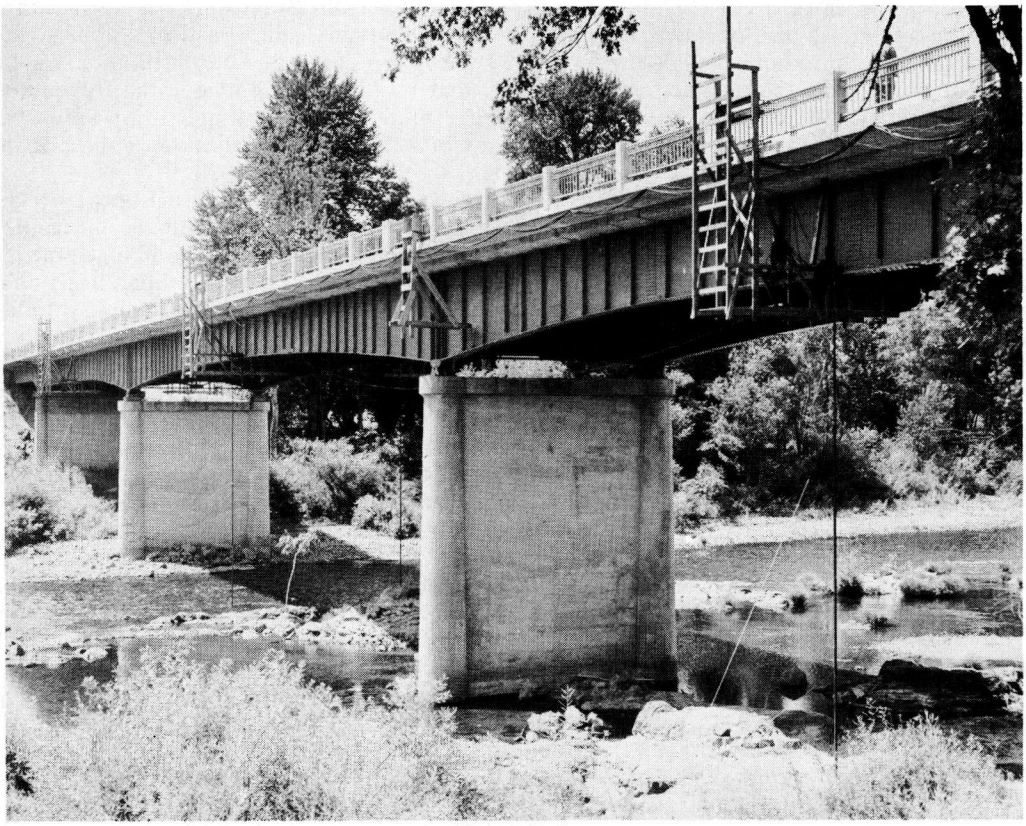


Figure 2. View of North Dillard Bridge showing girders and piers.

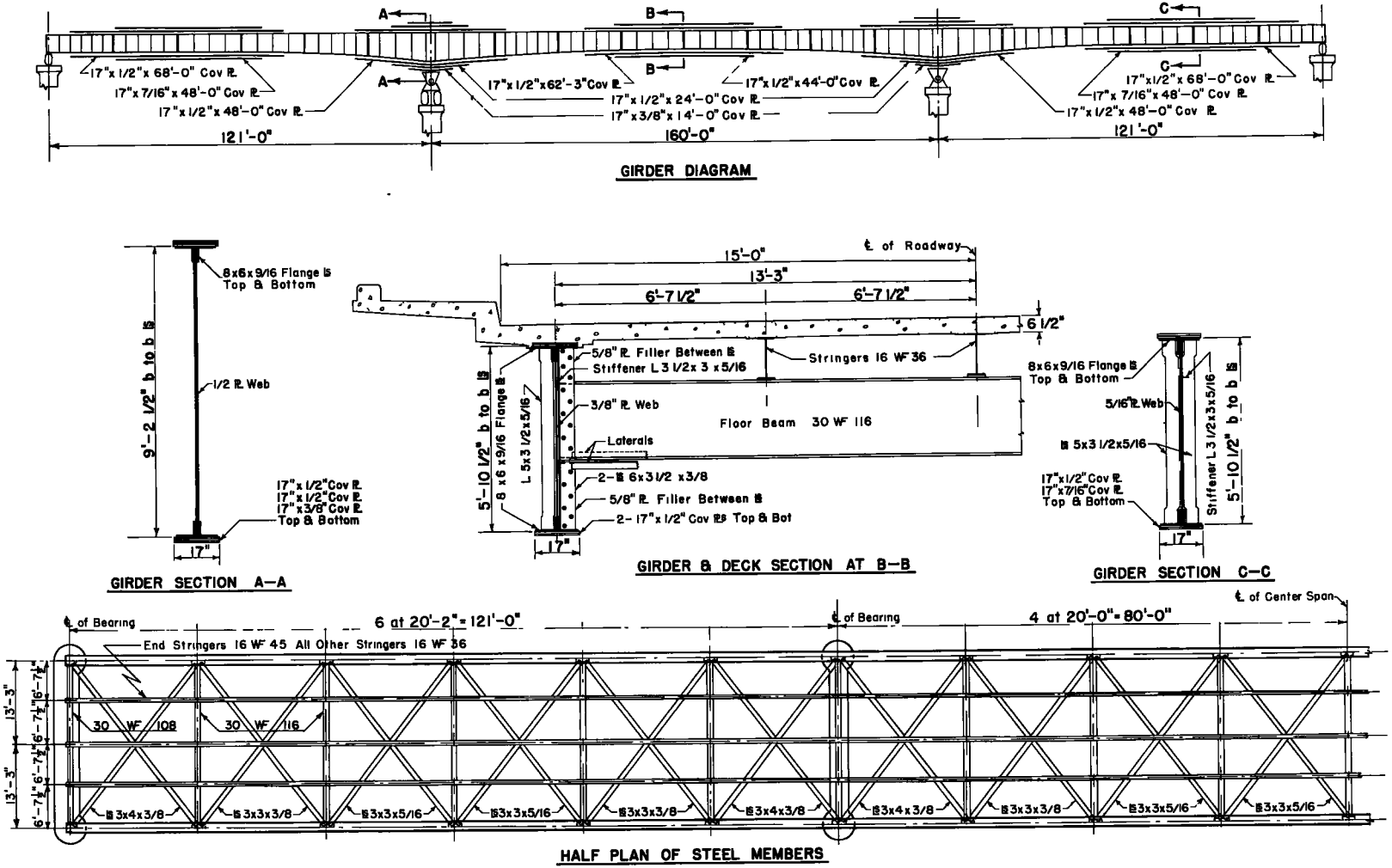


Figure 3. Details of bridges.

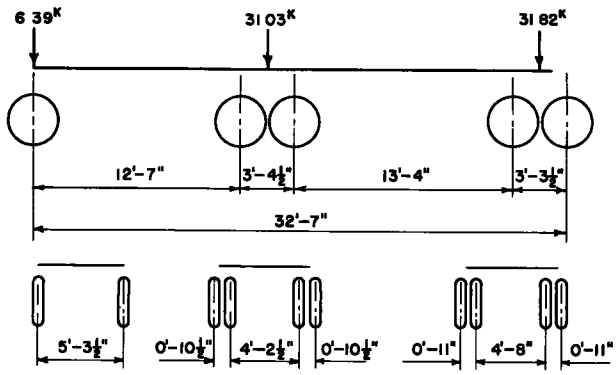
measured in Series II. Stringer strains were measured in Series III and floor beam strains in Series IV.

Magnetic strain gages were used in Series I and II. SR-4 resistance strain gages were used in Series III and IV. The deflectometers operated on the same principle as a magnetic strain gage. A coil housed in a pipe coupling was attached to the girder flange. An iron core in the coil was attached by a brass rod to a pipe mast from the ground or stream bed. Deflection of the girder produced relative motion between the coil and the core, thereby producing a change in the characteristics of the circuit.

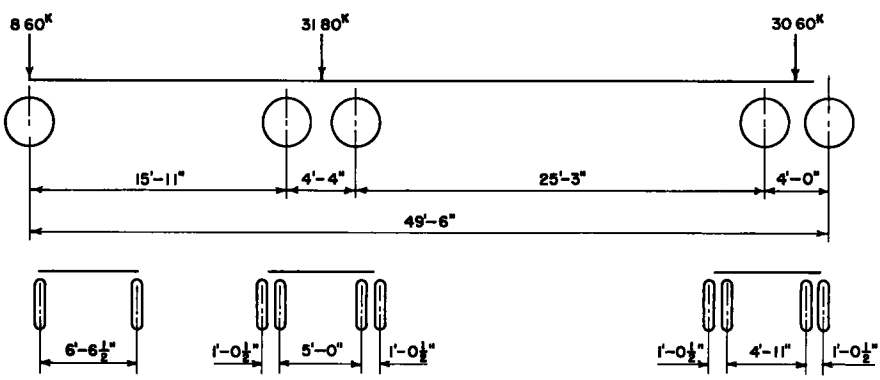
The strain recording equipment consisted of a power supply, bridge and attenuator panels, and two oscillographs. Light-beam galvanometers in the oscillographs exposed the strain traces on sensitized paper. Each oscillograph was connected to handle 15 active traces. A separate power supply and bridge and attenuator panels were provided for the deflectometers. The output was wired to the same two oscillographs used for strain recording.

The magnetic strain gages and the deflectometers were calibrated with dial indicators before installation and after removal. The strain-recording equipment contained calibrating facilities for SR-4 resistance strain gages.

Road tubes were installed across the bridge deck at each pier and at the center of the center span. The road tubes operated air switches which recorded the progress of the test vehicle on one trace on the oscillogram of each oscillograph. This record and the 0.1-second lines that were printed automatically on every oscillogram permitted accurate determination of the location of the test vehicle at all times. It also made possible an accurate calculation of vehicle speed.



TRUCK "A" 69,240 POUNDS



TRUCK "B" 71,000 POUNDS

Figure 4. Dimensions and weights of test vehicles.

The strain gages and the strain recording equipment, including the oscillographs, were components of the Bureau of Public Roads test unit. The deflectometers, deflectometer power supply, and bridge and attenuator panels were loaned to the project by the Institute of Traffic and Transportation Engineering of the University of California.

Two test vehicles were used. Vehicle A was a military-type, 2½-ton truck chassis converted to a logging truck with a logging trailer. The vehicle was loaded with steel sheet piling. Vehicle B was a diesel truck-tractor with an equipment semitrailer. The load was a track-type tractor. Figure 4 shows critical dimensions and axle loads of the two vehicles. Vehicle B is shown on the bridge in Figure 1.

For Series I, six deflectometers and 24 magnetic strain gages were installed. A deflectometer was installed on the outside lower flange of each girder at the center of each span. Strain gages were installed on the inside and outside top and bottom flanges of each girder at the center of each span. They were installed on the lower surface of the top flanges and on the upper surface of the lower flanges. The gage axis was located ⅞ inch from the outside edge of the outstanding flange angle.

Series I was divided into four subseries. Series Ia was a set of runs with the test vehicle centering the roadway center line. Runs were started with Vehicle A. Four or six runs, two or three in each direction, were made at each multiple of 5 mph. from 5 mph. to 30 mph. by the speedometer. The highest speed attainable by Vehicle A under the test conditions was 30 mph. Vehicle B was then procured and runs with it were at 5 mph. and at each multiple of 5 mph. from 20 to 45 mph. by the speedometer. There is some variation between speedometer speed and actual speed calculated from the air switches and the oscillogram time lines. The highest actual speed was about 43 mph.

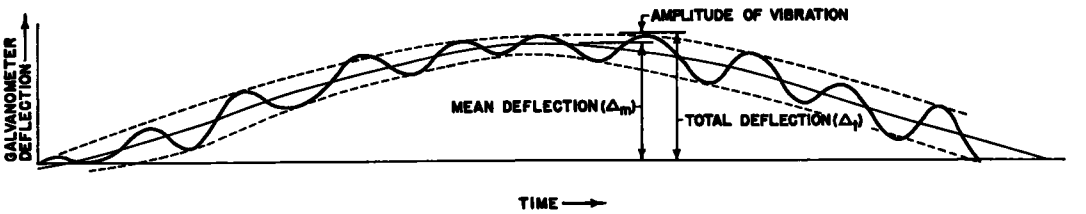


Figure 5. Typical deflection trace.

For an understanding of the terms used on the curves and in the discussion of the test results, attention is invited to Figure 5. This is a typical deflection trace from an oscillogram. The ratio of ordinate to abscissa from the trace has been doubled for clarity. The trace takes the form of a vibration curve with a frequency of about 2 cps. superimposed on a deflection curve representing the passage of the vehicle across the center span. To take the results from a trace, envelope curves to the vibration curve are drawn. The maximum ordinate of the upper envelope curve and the maximum ordinate of a line midway between the envelope curves are converted to deflection by the calibration curve and are termed the total deflection (Δt) and the mean deflection (Δm). The difference between the total deflection and the mean deflection is the amplitude of vibration. Strain traces are analyzed in an identical manner. Strains are converted to stress by an assumed modulus of elasticity of 30 million psi. The terms used are total stress (σt), mean stress (σm) and amplitude of stress oscillation.

In this preliminary report only the results from runs with Vehicle B will be presented. Vehicle A would add little to the picture. Only the stresses in the lower flanges will be presented. These are the critical stresses in each section. The total and mean stresses in the lower flanges of each girder in the center span for all Series Ia southbound runs and northbound runs were averaged separately to see whether or not the direction of the vehicle has an effect on the stress and deflection. Since no directional effect was evident, the results of the runs in the two directions were averaged to give a single stress curve and a single deflection curve for the center span of each girder for Series Ia. Figure 6 shows the stress curves for the two girders. The highest mean stress is about 90 percent of the calculated stress without impact, based on the current AASHO speci-

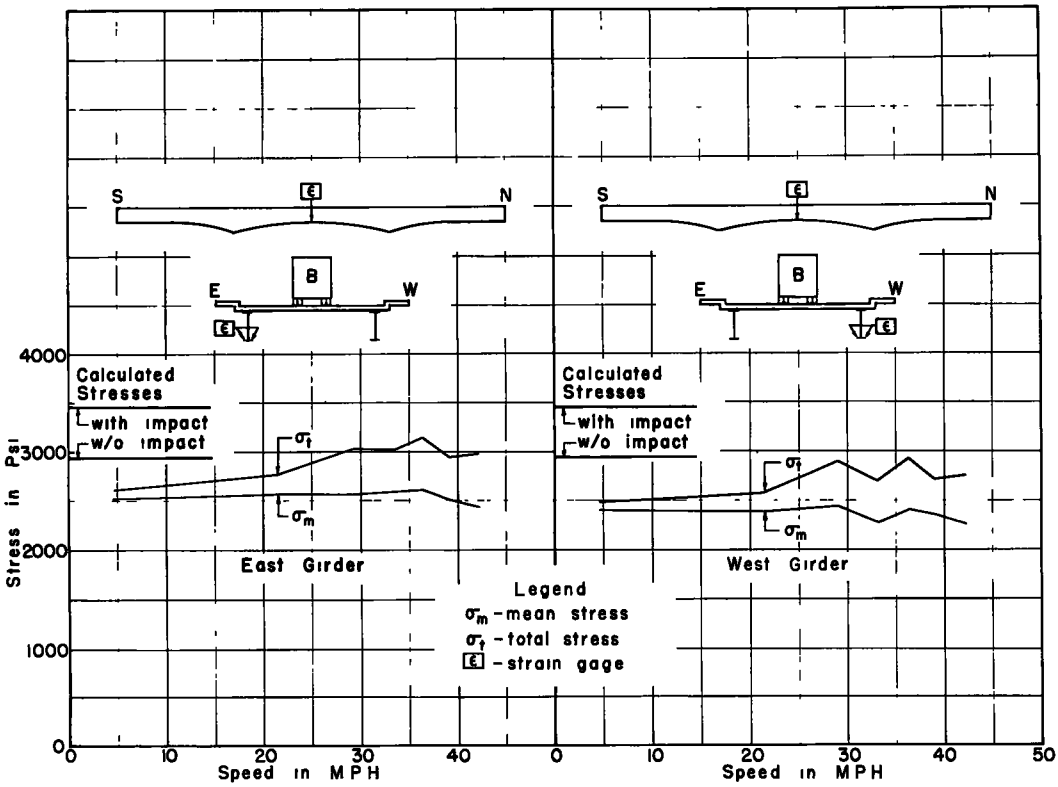


Figure 6. North Dillard Bridge stress curves, Series Ia, Vehicle B. center span.

fications.[1]

The highest total stress is about 7 percent greater than the calculated stress without impact but about 9 percent less than the calculated stress with impact. The increase in total stress resulting from an increase in the amplitude of vibration between 20 and 30 mph. is typical of the span. Figure 7 shows the corresponding deflection curves. The deflection follows the same general pattern as the stress except that a sharp increase in amplitude in the east girder occurs about 29 to 35 mph. The greatest observed total deflection is 0. 55 inch and the greatest amplitude of vibration is 0. 13 inch, both occurring at 42. 6 mph. in the east girder.

In the case of the side spans, a difference in stress and deflection characteristics due to the direction of the vehicle might be expected for two reasons: The influence line for moment at the midpoint of the end span of a continuous series is not symmetrical, nor is the axle and load arrangement of the vehicle longitudinally symmetrical. Calculations indicate that the vehicle should produce higher stresses when proceeding across a side span from the approach than when proceeding across a side span from the center span. On the other hand, when the vehicle proceeds from the center span onto a side span, the side span is in a state of vibration before the vehicle enters the side span.

Figure 8 shows the stress curves for the side spans. First span and third span effects are shown by separate curves. While the highest total and mean stresses are similar, the reaction to speed is different. In the first span effect, where the structure is still until the vehicle enters the span, the highest stresses occur at about 40 mph.,

[1] Since shear developing angles were used at 21-inch to 24-inch spacing in the region of positive moment, the calculated girder stresses and deflections presented in this report are those obtained by considering composite action of the girder with that portion of the deck allowed under AASHO specifications. A value n=10 was used in computing both stresses and deflections.

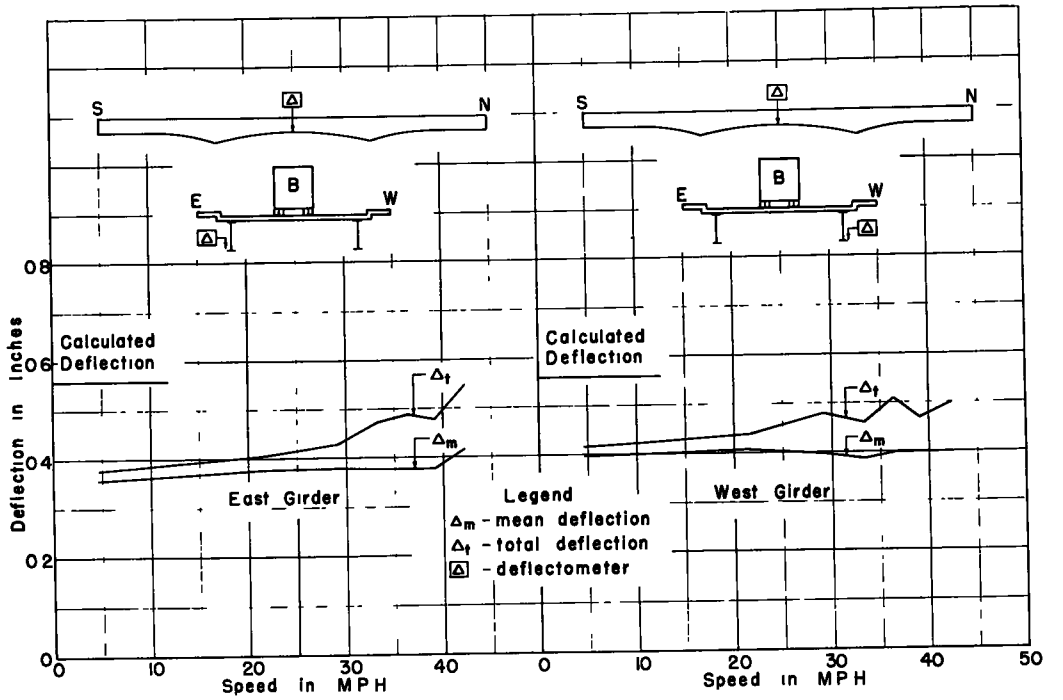


Figure 7. North Dillard Bridge deflection curves, Series Ia, Vehicle B, center span.

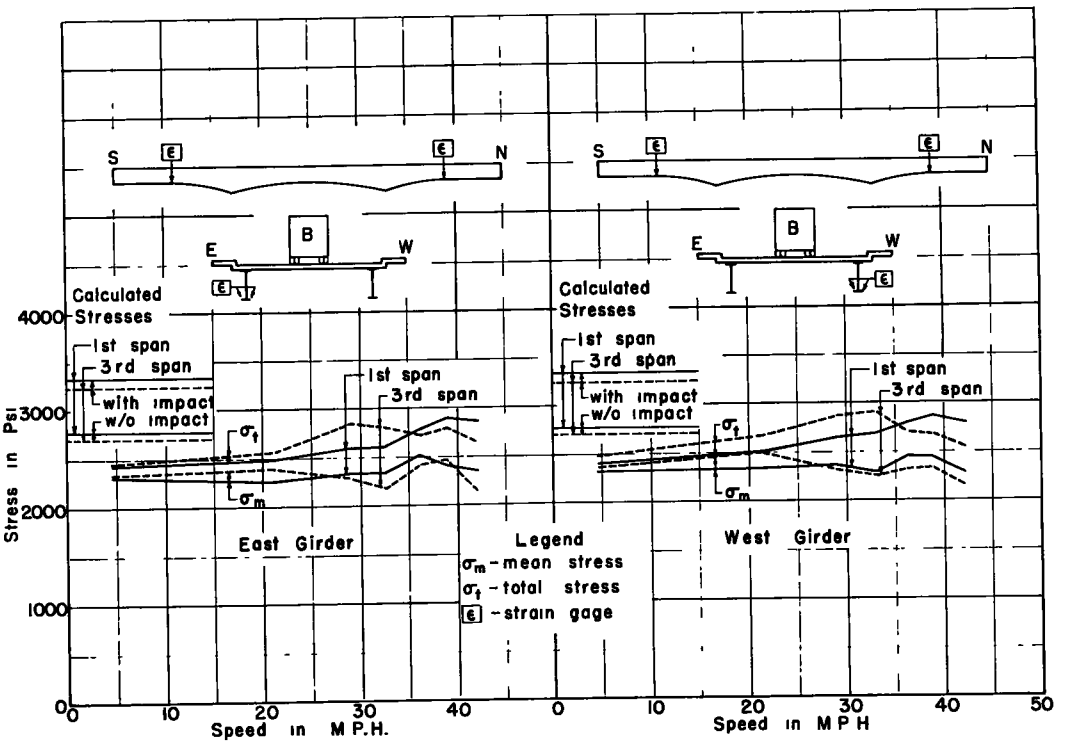


Figure 8. North Dillard Bridge stress curves, Series Ia, Vehicle B. side spans.

while in the third span effect, where the structure is vibrating from the passage of the vehicle across the other two spans, the highest stresses occur at about 30 mph.

Series Ib was a repetition of Series Ia, with the exception that all runs were made with the east tires of the vehicle a foot from the east curb. Figure 9 shows the stresses in the center span. The calculated stresses are based on simple beam distribution to girders. The curves for measured stress show that, due primarily to the torsional rigidity of the structure, a greater proportion of the load than assumed was transferred to the unloaded girder. The laterals are in the plane of the bottom flange of the floor beam and they, with the floor slab and main girders, form a box section over 4 feet deep having a significant torsional stiffness, the deep floor beams and their connections being stiff enough to prevent distortion of the cross section. Again, the abrupt increase in amplitude between 20 to 30 mph. in the loaded girder will be noted. The unloaded girder shows a greater amplitude of stress oscillation than the loaded girder.

To determine the damping effect of additional stationary mass or dead load on the vibration of the structure, Vehicle A was parked at the center of the center span with tires touching the west curb. Series Ib was then repeated as Series Ic with Vehicle B making all the runs. Figure 10 shows the stress curves under these conditions. New strain and deflection zeros were established after Vehicle A was parked on the span. The stresses in Figure 10 are those caused by the moving vehicle only. They do not include the stress caused by the parked vehicle. The highest total stresses occur at a different speed than in Series Ib, but the magnitude of these stresses is virtually unchanged by the parked vehicle.

The fourth subseries under Series I was It, readings taken under normal heavy truck traffic on the bridge. The data from these tests have been scaled from the oscillograms and tabulated, but they have not yet been analyzed in detail. Several records were taken with more than one vehicle on the structure, but no stresses or deflections greater than

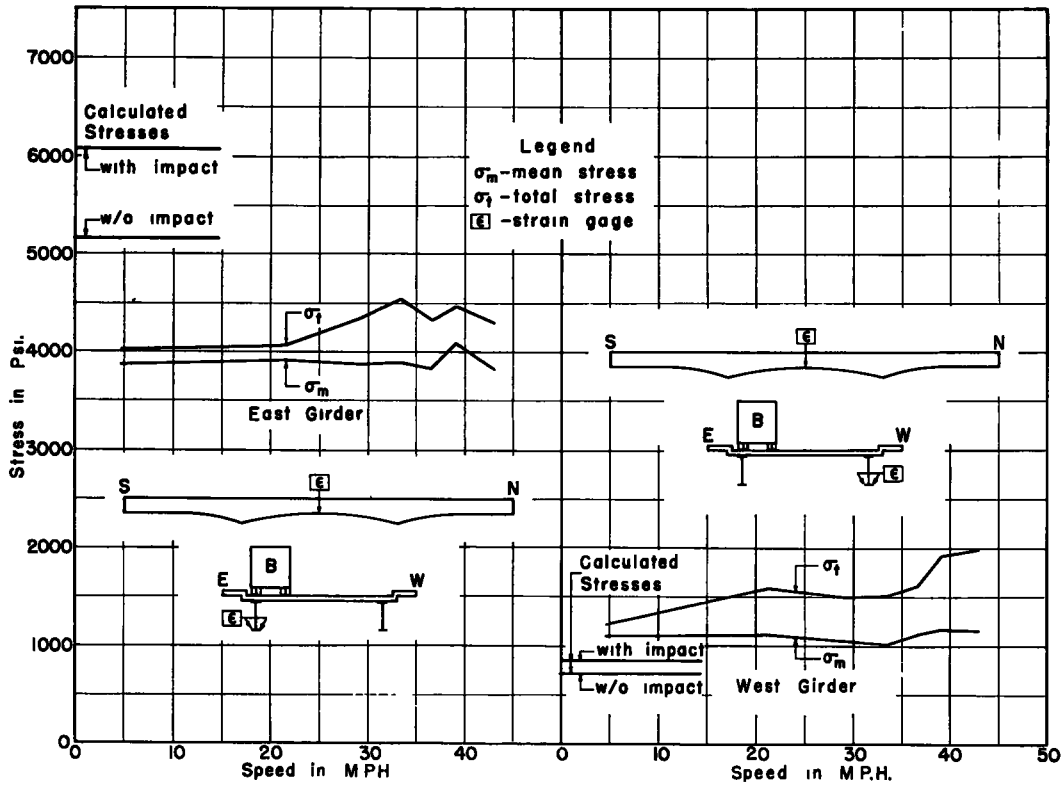


Figure 9. North Dillard Bridge stress curves, Series Ib, Vehicle B, center span.

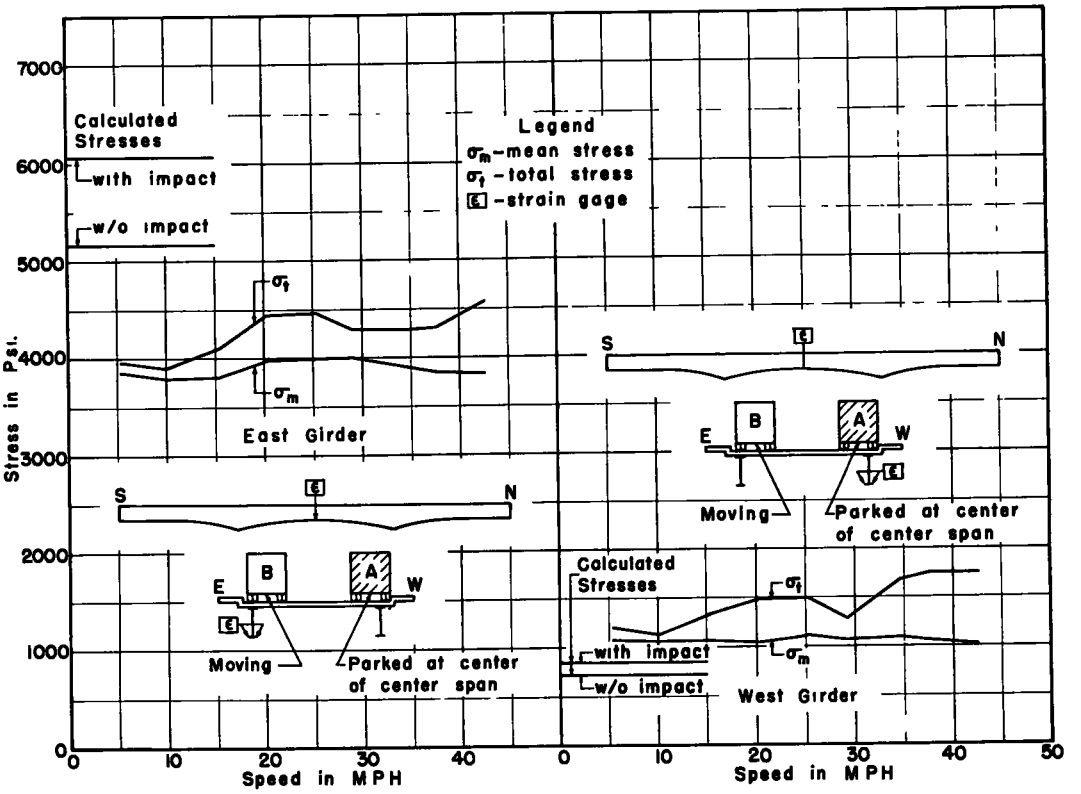


Figure 10. North Dillard Bridge stress curves, Series Ic, center span. Stresses from moving vehicle only.

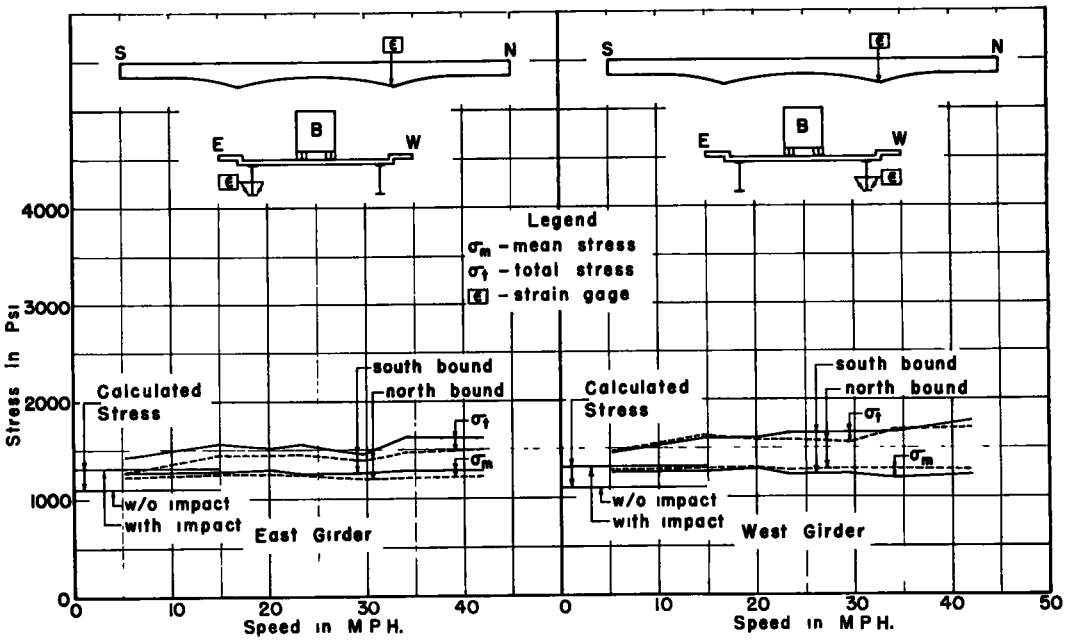


Figure 11. North Dillard Bridge stress curves, Series IIa, Vehicle B, center span.

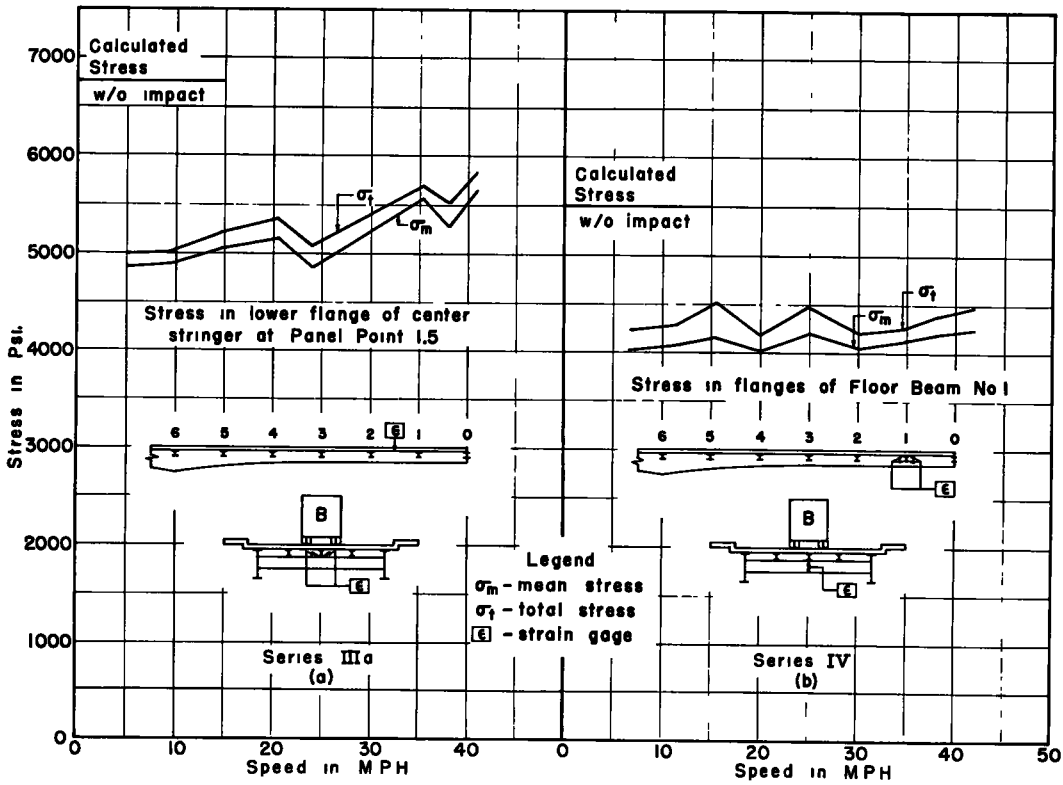


Figure 12. North Dillard Bridge floor-system stress curves, Vehicle B, north span.

those measured in Series Ib were found.

For Series II, the twelve strain gages were removed from the inside girder flanges and were installed on the girder flanges near the north interior pier. Eight gages, inside and outside of top and bottom flanges on both girders, were installed in the center span. Four gages, outside of top and bottom flanges of both girders, were installed in the north span. All twelve gages were 2 feet 6 inches from the pier centerline. This distance was necessary to avoid the special girder details at the pier. Figure 11 shows the strain curves for the gages on the center span side of the pier. The negative moment stresses show a higher degree of uniformity than the positive moment stresses and less variation with speed. The measured negative moment stresses are consistently greater than the calculated stresses with impact.

For Series III, resistance strain gages were installed on both sides of the top and bottom flanges of the three stringers at two cross-sections of the bridge. One cross-section was at the center of the second floor panel from the north end of the north span and the other cross-section was over the floor beam next south. Runs were made with the vehicle centered on the deck, hence, centered over the center stringer and with the west wheels of the vehicle directly over the center stringer. At the center of the panel, maximum stresses were produced in the lower flange of the center stringer with the vehicle centered over the stringer. Figure 12a shows these stresses. The unusual feature of these curves is the variation of both mean and total stress with speed and the comparative uniformity of the amplitude of stress oscillation. The highest total stress observed is only about 87 percent of the calculated. Composite action of the concrete deck is not considered in the calculated stress, since no shear devices were installed. Composite action is evident in the magnitude of the lower flange stresses and in the relationship of upper and lower flange stresses.

Figure 12b shows the stress curves for Series IV, the flange stresses at the center

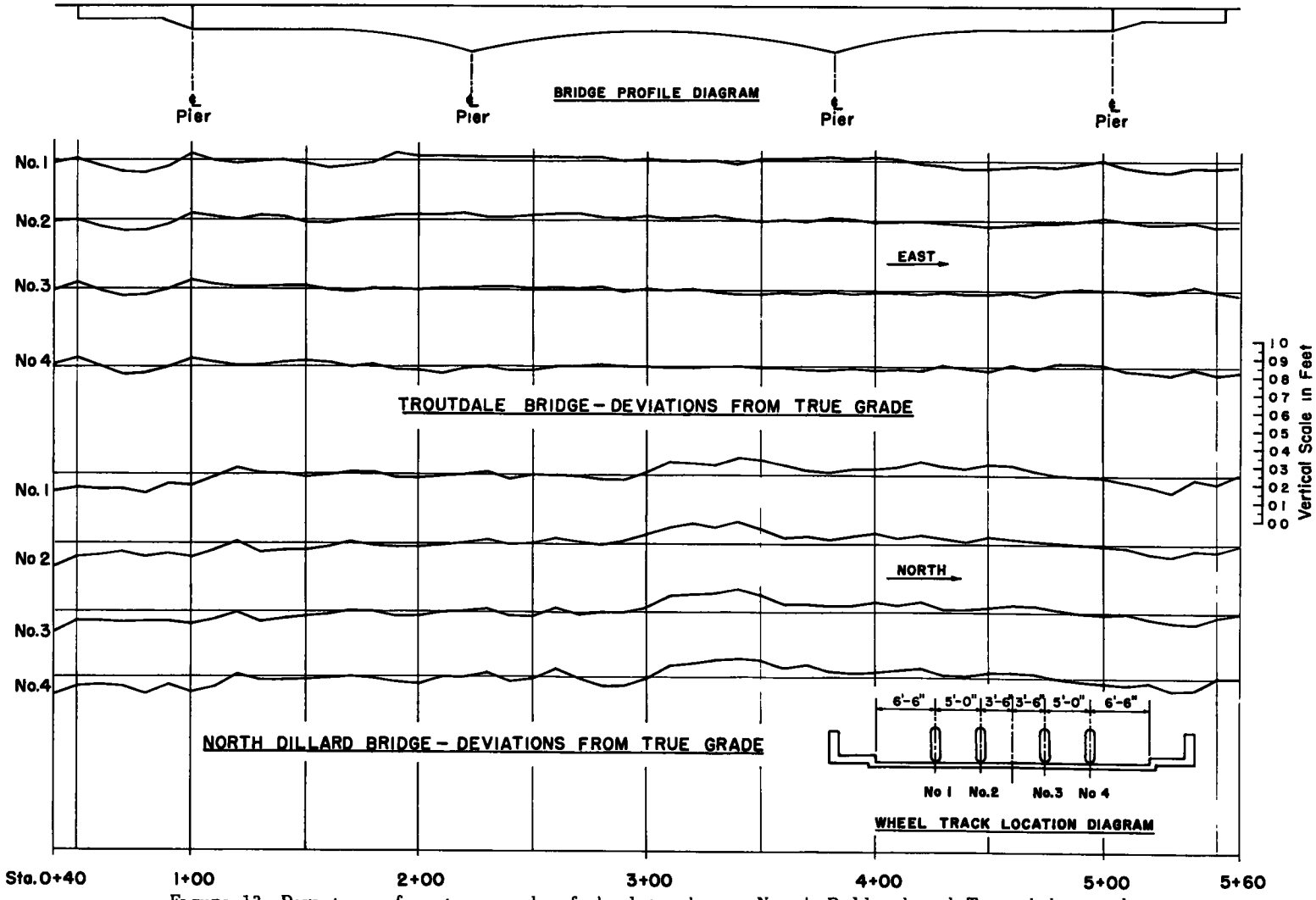


Figure 13. Deviations from true grade of wheel tracks on North Dillard and Troutdale bridges.

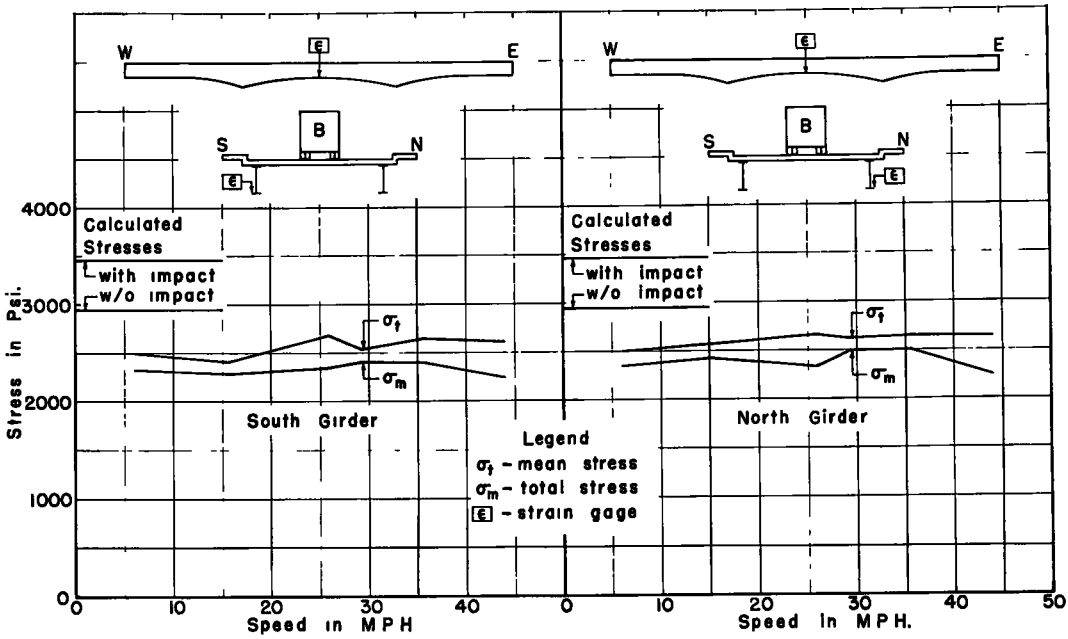


Figure 14. Troutdale Bridge stress curves, Series Ia, Vehicle B, center span.

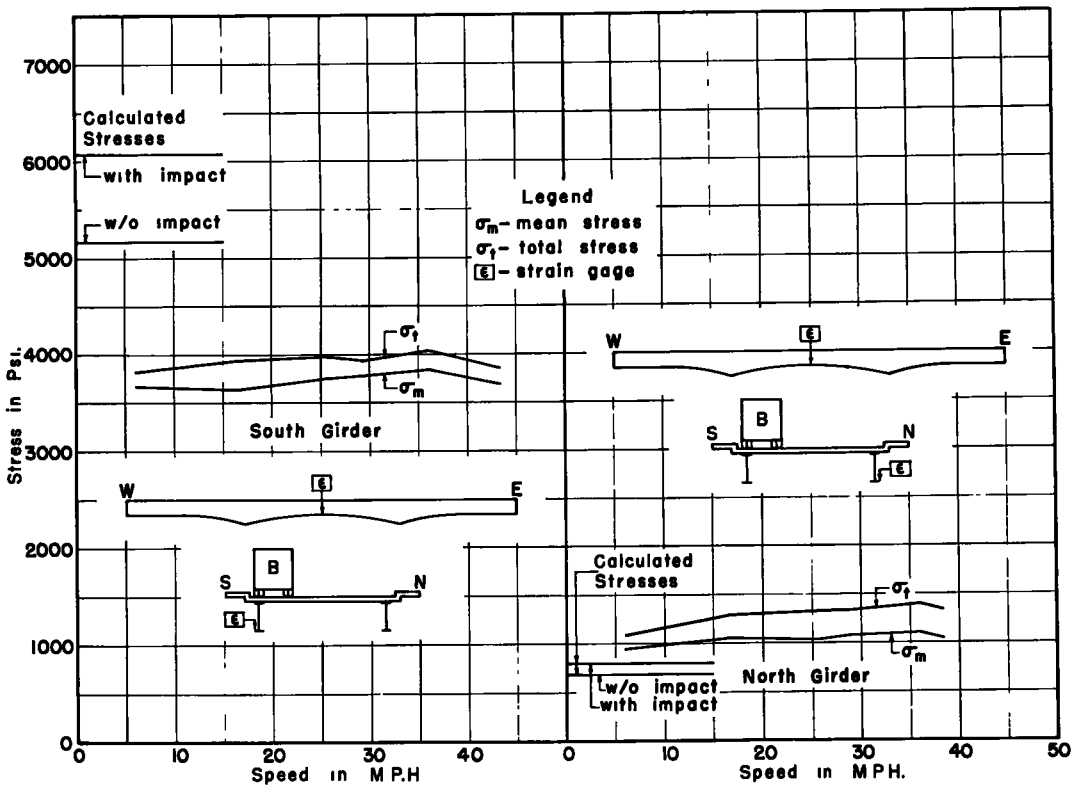


Figure 15. Troutdale Bridge stress curves, Series Ib, center span.

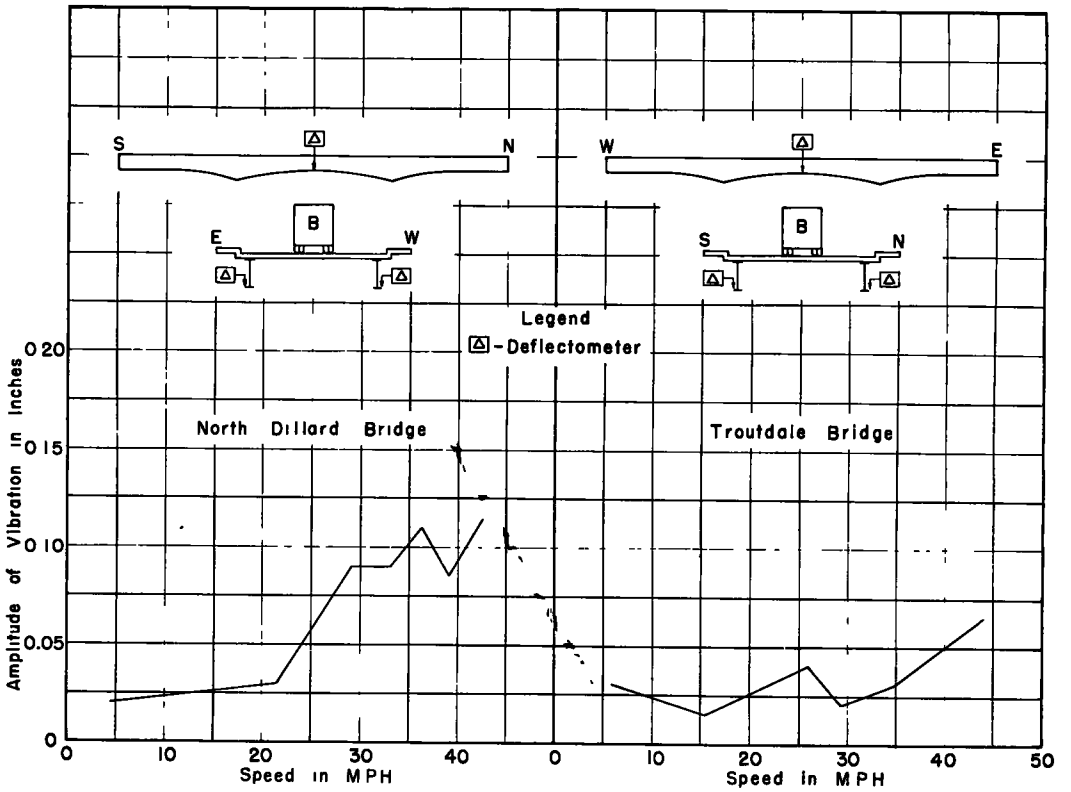


Figure 16. Amplitude of vibration curves, Series Ia, center span. Comparison between North Dillard and Troutdale bridges.

of the first floor beam from the north end of the north span. The floor beam showed a different response to speed variation than any other member tested. Here again the measured stresses are much lower than the calculated stresses. Preliminary analyses of the records of gages near the ends of the floor beams indicate that the low stresses at the center of the floor beam will be explained by partial fixity at the connection of the floor beam to the girder.

During the testing of the North Dillard Bridge, it was concluded that the deck roughness might be a contributing factor to the vibration. To verify this conclusion, it was decided that measurements of strain and deflection should be made on a similar bridge having a smooth deck. The Troutdale Bridge over the Sandy River on the Columbia River Highway (US 30) was fabricated and erected from the same superstructure plans as the North Dillard Bridge.

Profiles in each of the four wheel tracks were obtained on both bridges by taking elevations with a level at 10-foot intervals and making an accurately traced, graphical profile between each pair of level points. These profiles included 60 feet of roadway and approach structure at each end of the steel structure. Figure 13 shows the deviations of the 10-foot points from true grade, the true grade for each wheel track being represented by a horizontal line. To arrive at a numerical comparison of the roughness of the two structures, total deviation from true grade at the 10-foot points were computed. The total deviations in 520 feet for the four wheel tracks at North Dillard were 2.02 feet, 2.09 feet, 2.05 feet, and 2.00 feet (average, 2.04 feet). The deviations at Troutdale were 1.37 feet, 1.08 feet, 0.84 feet, and 0.84 feet (average, 1.03 feet). On the basis of this comparison, the Troutdale Bridge was selected for making comparable measurements of strain and deflection.

Differences other than deck roughness that might be significant in the vibratory characteristics of the structures include the following: The Troutdale Bridge is on a level

grade. The deck has few transverse cracks. The piers are of slightly different detail, are shorter (about 28 feet from footing to bearing), and are founded on piling.

Series Ia and Ib of the North Dillard tests were duplicated on the Troutdale Bridge, except that resistance strain gages were used and strain gages were placed only on the top and bottom outside flanges of the girders at the centers of the west and center spans. Figures 14 and 15 are the Ia and Ib stress curves for the center span of the Troutdale Bridge. The mean stress in the Troutdale Bridge averages about 5 percent less than those in the North Dillard Bridge. The amplitude of stress oscillation on the Troutdale Bridge is much less. Figure 16 shows the average variation in amplitude of vibration with speed for both girders in the center spans of the two structures as taken from Series Ia deflection records.

The analysis of the results to date has included only the consideration of stress and deflection. The members have been studied separately. Their interaction has not been considered. In the completion of the analyses of the data this will be done. It is hoped that an analysis of the structure for vibration can be made and correlated with the test results. As a matter of interest, the natural frequency of the first symmetric vertical mode with strong motion in the main span is about 1.7 cps. and its logarithmic decrement is about 0.065. The natural frequency of the first asymmetric mode having a node at the center of the main span and dominant motion in the side spans is about 2.7; its logarithmic decrement is about 0.076.

The measured stresses were lower than the stresses calculated on the basis of current AASHO specifications, except in the case of the unloaded girder under eccentric loading and negative moment stresses in the lower girder flanges. This latter item warrants further study. The measured stresses were considerably lower than the calculated stress in the stringers and floor beams and in the loaded girder under eccentric loading. The condition of the deck surface has little effect on the mean stresses in the girders, but it has a decided effect on the amplitude of vibration and on the total stress.

The original test program was prepared by Neil Van Eenam, of the Bureau of Public Roads. He was present during the early phases of testing and assisted in the general organization of the project. The Bureau of Public Roads' test unit was assembled by E. G. Wiles, who accompanied the unit to the test side and supervised its operation during the tests. G. S. Vincent represented the Bureau of Public Roads on the project, assisted in the field work, and consulted with the authors at frequent intervals during the analyses of data and offered helpful suggestions in the preparation of the report.

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