loads. Finally, exponential variation of the transverse load distribution factor with the flexural parameter (λ) is tentatively suggested for interior girders, and a constant distribution factor of 2.13 is recommended for exterior girders.

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Highway Bridge Vibration Studies

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The results of acceleration studies of highway girder bridges are presented. Deflection limitations and maximum span-depth ratios used in present bridge design codes do not necessarily ensure the comfort of bridge users. Vertical accelerations have been shown to be significant in producing adverse psychological effects on pedestrians and occupants of stopped vehicles. The effects on bridge accelerations of major bridge-vehicle parameters, including the properties of the bridge and the vehicle as well as the initial conditions of the roadway, were investigated analytically and compared to criteria for human response. Numerical solutions are obtained from a theory in which the bridge is idealized as a plate continuous over flexible beams for simple-span bridges and as a continuous beam with concentrated point masses for two- and three-span bridges. The vehicle is idealized as a sprung mass system. The results indicate that, for simple-span bridges, accelerations that might psychologically disturb a pedestrian are primarily influenced by bridge-span length, vehicle weight and speed, and especially roadway roughness. Less significant factors are

girder flexibility and transverse position of the vehicle. For the two- and three-span continuous bridges studied, roadway accelerations exceeded the recommended limit for comfort only when the effects of surface roughness were included.

This investigation is aimed at obtaining a better understanding of highway bridge vibrations by studying the effects of varying some of the parameters of the vehicle-bridge system. It is hoped that the results of the study will ultimately be useful in establishing design criteria that will directly regulate dynamic response characteristics.

Dynamic response is not specifically mentioned in current bridge design codes (1). Instead, deflection limitations and maximum span-depth ratios are specified in the hope of controlling vibrations; these code provisions do not, however, attack the problem directly. Other effects, such as acceleration and jerk, are known to be more significant than maximum deflection in producing adverse psychological reactions to bridge vibrations (complaints of disturbing bridge motions come primarily from pedestrians and from persons in halted vehicles).

The economical use of modern, high-strength steels may also be hindered somewhat by present code limitations if the design is controlled by deflection rather than strength requirements. Significant savings might be possible in the design of high-strength steel girder bridges if the somewhat arbitrary restrictions were replaced by dynamic response criteria.

The dynamic response of both simple-span and two- and three-span continuous-girder bridges has been investigated. The most successful recent analytical studies are those by Oran and Veletsos (2) for simple-span highway bridges and Veletsos and Huang (3, 4) for multispan continuous highway bridges. The computer programs developed by these researchers have been used, with some minor modifications, in this study.

Several reports (5, 6, 7, 8, 9) have discussed criteria for human response to vertical vibration. The report by Wright and Walker (5), which summarizes the effects of bridge flexibility on human response, suggests that human sensitivity to vibrations is most closely related to acceleration in the usual frequency range for multibeam highway bridges. Because most tests of vertical vibration perception have been conducted for harmonic vibrations, Lenzen (7) has suggested that the tolerance level be multiplied by a factor of 10 for short-duration peak accelerations. Wright and Walker (5) suggest a maximum acceleration magnitude of 2.54 m/s^2 (100 in/s^2) for comfort.

Because of the strong relation between acceleration and vibration perception, this study focuses on the variation of maximum bridge accelerations with significant bridge-vehicle parameters such as girder span and flexibility, vehicle weight and speed, axle spacing, and roadway roughness.

SIMPLE-SPAN BRIDGES

Method of Analysis

The method of analysis used in this paper for the dynamic response of simple-span, multigirder highway bridges was developed by Oran and Veletsos (2). Their computer program has been modified somewhat for this study to provide more acceleration information and to increase the capabilities for both input and output.

In this analysis the bridge is represented as a plate continuous over flexible beams. Both bending and torsional stiffnesses of the beams are considered, but composite beam-slab action is not. The major steps of the analysis are (a) determining the instantaneous values of the interacting forces between the vehicle and the bridge and the inertia forces of the bridge itself and (b) evaluating the deflections and moments produced by these forces. The second step, which is a problem of statics alone, is solved by a combination of energy principles and the Levy method of analysis for simply supported rectangular plates. Vertical deflections are represented in the form of a double Fourier sine series. The solution has been shown to converge fairly rapidly so that only a limited number of terms are required.

Strain and potential energy of the system are written in terms of the deflections. Enforcing the condition that the total energy must be a minimum yields a set of equations for determining the Fourier coefficients, which can then be substituted into the appropriate moment and deflection expressions.

In the dynamic analysis, the mass of the slab is assumed to be uniformly distributed and the mass per unit length of each beam is assumed to be constant. The vehicle is represented by a single-axle, two-wheel loading that consists of a sprung mass and two equal unsprung masses. The springs are assumed to be linear elastic and to have equal stiffnesses. Damping has been neglected for both the vehicle and the bridge. The vehicle is assumed to move at constant velocity. The dynamic deflection configuration of the structure is represented by a Fourier series with time-dependent coefficients.

Again the total energy of the system can be written in terms of the displacement coordinates and their derivatives. Dead load deflections and unevenness of the roadway surface are included in the appropriate energy terms. The equations of motion are formulated by applying Lagrange's equation.

The procedures used to evaluate the dynamic response of the bridge-vehicle system can be summarized as follows:

1. The governing differential equations of motion are solved by means of a step-by-step method of numerical integration to determine the generalized coordinates and their first two derivatives. The time required for the vehicle to cross the span is divided into a number of small intervals, and the governing equations are satisfied only at the ends of these intervals by means of an iteration scheme.
2. The interacting forces between the vehicle and the bridge and the inertia forces of the bridge are evaluated.
3. The dynamic deflections and the bending moments induced in the bridge are determined from the dynamic forces acting on the bridge.

Acceleration Studies

The purpose of this investigation is to study the major parameters that affect the accelerations of simple-span highway bridges under moving loads. The study is based on the previously described analysis method and computer program. Bridge data were taken from Standard Plans of Highway Bridge Superstructures of the U.S. Bureau of Public Roads (10). The study is restricted to steel I-beam bridges with noncomposite reinforced concrete decks. The effect of side curbs is not taken into account.

Parameters that affect bridge accelerations can be classified as (a) bridge parameters, such as beam span and stiffness; (b) vehicle parameters, such as velocity and transverse position of the wheels; and (c) construction parameters, such as roadway roughness. The accuracy of the analysis depends, of course, on such solution parameters as the number of terms chosen for the deflection series expressions and the number of integra-

tion steps. For the simple-span bridge study, satisfactory convergence was obtained by dividing the time required for the vehicle to cross the span into 400 integration steps. The total computer time required to obtain the response of a bridge with a span of 18 m (60 ft) is approximately 150 s on a CDC 6500 computer.

Bridge Parameters

The effect of span length on maximum midspan accelerations is shown in Figure 1. The bridges are standard designs for an HS20-44 loading with a 13-m (44-ft) roadway width. Six steel beams support the 190-mm (7.5-in) reinforced concrete deck. Beam sizes range from W21×62 for the 6-m (20-ft) span to W36×300 for the 21-m (70-ft) span. Corresponding fundamental bending frequencies are 16 and 4 Hz.

The vehicle is represented by a single-axle, two-wheel, 32 667-kg (72 000-lb) loading that moves across the span at a constant velocity of 96 km/h (60 mph). The stiffness of each tire spring is 1051 kN/m (6000 lbf/in). Unsprung loads and damping are neglected. As the vehicle travels along near the edge of the bridge, the inside wheel is over beam 2. The exterior beams are shown to have somewhat higher midspan accelerations than the interior beams. Figure 1 also shows a definite increase in maximum accelerations for the standard bridge designs as the span is decreased.

Figure 2 shows the variation of the midspan acceleration of each beam as the vehicle crosses an 18-m (60-ft) span. Accelerations of beam 1 and beam 6 are out of phase all the time. Note that the maximum acceleration of beam 1 occurs when the vehicle enters the span but the maximum acceleration of beam 6 occurs when the vehicle is at midspan or leaving the bridge.

Current bridge specifications attempt to control vibrations by limiting the maximum live load deflection, which may unjustly penalize designs that use more flexible, high-strength steel beams. Figure 3 shows the variation of maximum midspan accelerations as beam stiffness is reduced. The basic design uses five W36×230 A36 steel beams. Equal strength can be provided by five W36×182 A572-50 beams. Although the maximum acceleration increases somewhat as stiffness is reduced, the relation is certainly less severe than an inverse proportion.

Vehicle Parameters

It is well known that vehicle speed has a strong influence on bridge vibrations. In Figure 4 maximum midspan accelerations for a five-beam bridge are plotted versus vehicle speed. Bridge accelerations are almost directly proportional to vehicle speed over the indicated speed range.

The effect of the transverse position of the vehicle on a five-beam, 18-m (60-ft) span bridge was studied by determining the maximum midspan accelerations for six cases. Results indicate that the accelerations of the edge beams are greatest when the vehicle travels along the edge of the roadway and tend to decrease when the vehicle travels near the centerline of the bridge. In contrast, center-beam accelerations increase as the vehicle moves toward the centerline and are slightly larger than edge-beam accelerations when the vehicle straddles the centerline. In most practical situations, however, edge-beam accelerations are larger than those of the interior beams.

Surface Roughness

Several test reports (11, 12, 13) have indicated that surface roughness is a very significant factor affecting the vibration of highway bridges. These reports have recommended that the bridge surface should be as smooth as possible. For all of the results previously reported here, the bridge surface was assumed to be smooth.

Surface roughness is assumed to be represented by

Figure 1. Maximum midspan accelerations for simple-span steel beam bridges.

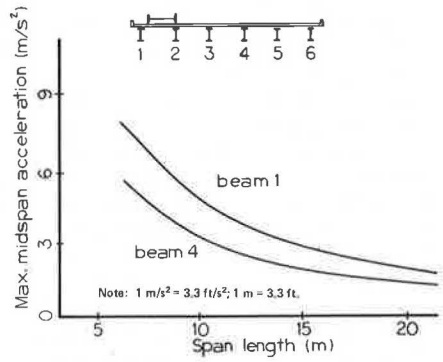


Figure 2. Acceleration history curves.

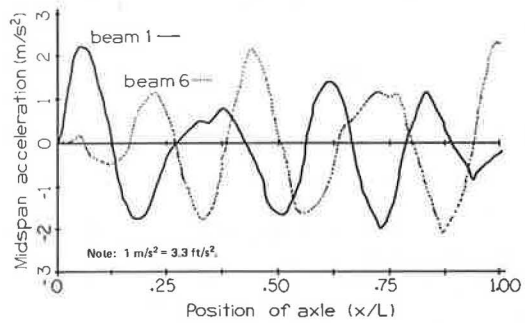


Figure 3. Acceleration versus beam stiffness for simple-span bridge.

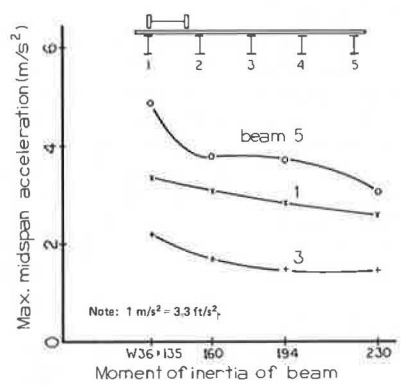
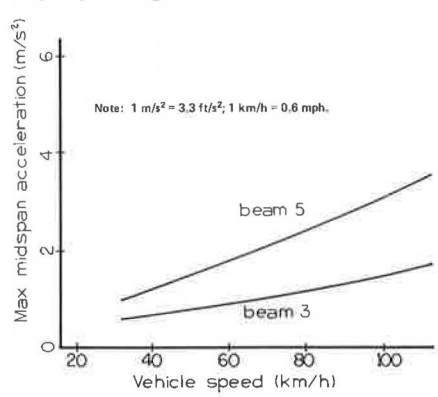


Figure 4. Acceleration versus vehicle speed for simple-span bridge.



some number of half sine waves passing through the supports. Both the number of half sine waves and the amplitude can be varied. It is assumed that the shape and the amplitude of bridge surface roughness are the same under both wheels of the vehicle.

Figure 5 shows the results of determining the maximum midspan accelerations for 0 to 19 half sine waves of 13-mm (0.5-in) amplitude roadway roughness. Again, a 32 667-kg (72 000-lb) vehicle traveled at 96 km/h (60 mph) near the edge of the roadway on a five-girder, 18-m (60-ft) span bridge. It is evident that the accelerations are not influenced by up to 3 half sine waves of roughness; however, the effect increases markedly when the number of half waves exceeds 5 and reaches its peak when the surface roughness consists of 12 half sine waves. For the two peaks at 7 and 12 half sine waves, the times required for the vehicle to cross a single roughness wave correspond fairly closely to the first two natural frequencies of the bridge. The maximum accelerations for a rough roadway surface are as much as five times those for the same bridge with a smooth deck. Of course, the probability of a perfectly periodic deck roughness is rather small for a real bridge. But the significance of the effect is obvious. For the nearly resonant condition, maximum accelerations are approximately proportional to the amplitude of the surface roughness.

CONTINUOUS-BEAM BRIDGES

Method of Analysis

A general theory for the analysis of continuous bridges was developed by Huang and Veletsos (14). The computer program used in this study, which was developed by Huang (4), was previously used by Nieto-Ramirez and Veletsos (15) in an extensive study of the dynamic response of three-span bridges. Application of the program here is limited to two- and three-span symmetric beam bridges.

In this analysis the bridge is idealized as a single, continuous beam and the resulting system, which has an infinite number of degrees of freedom, is replaced by a discrete system with a finite number of degrees of freedom by replacing the distributed mass by a series of concentrated point masses and considering the beam stiffness to be distributed as in the original structure. Bridge damping is assumed to be of the absolute viscous type and is approximated by dashpots located at mass coordinate points. The analysis is based on ordinary beam theory and neglects the effects of shearing deformation and rotary inertia.

Because the bridge has been idealized in this analysis as a single beam, the rolling effect of the vehicle cannot be considered. Even when it is treated as a plane system, however, a vehicle is a complex mechanical system. In this analysis a tractor-trailer vehicle is represented by a three-axle load unit that consists of two interconnected masses (Figure 6). Each axle is represented by two springs in series and a frictional mechanism that simulates the effect of friction in the suspension system. The second spring is active only when the axle force exceeds the limiting friction value. Viscous damping is neglected.

Writing the equations of motion for the vehicle and the concentrated masses of the bridge yields a set of simultaneous, second-order differential equations, equal in number to the number of degrees of freedom of the bridge-vehicle system. These equations are solved by a numerical integration scheme in which the evaluation of the interacting forces between the bridge and the vehicle is a major intermediate step. As the integration of the differential equations is carried out, the values of all coordinates, accelerations, and interacting forces are determined. Values of corresponding deflections and moments at any section can then be determined by statics.

Acceleration Studies

Both two- and three-span symmetric, continuous-girder bridges have been studied extensively (16); because the results are similar, only two-span bridges will be discussed here. Bridge data are again those of the U.S. Bureau of Public Roads (10), and the study is restricted to steel continuous-beam bridges with reinforced concrete decks. Unless stated otherwise, the roadway surface is assumed to be smooth.

The accuracy of the analysis now depends on the number of lumped masses chosen as well as the number of integration steps. Good stability of the solution was obtained by dividing the time required for the vehicle to cross the bridge into 2000 integration steps. The bridge was modeled by lumping the masses at the quarter and midpoints of each span, as suggested by Huang (4).

Bridge Parameters

Figure 7 shows the maximum nodal accelerations for three standard-design bridge spans. The roadway width is 13 m (44 ft) and the 190-mm (7.5-in) slab is supported by six rolled steel beams. Bridge damping is taken as 2 percent of critical damping. The vehicle is an HS20 three-axle truck moving at a speed of 96 km/h (60 mph). As in the case of simple-span bridges, higher accelerations occur on the shorter spans. Maximum accelerations for two-span bridges were found to be about 1.5 times larger than those for three-span bridges of equal span length. Highest accelerations occur in the simple-span bridges.

The effect of varying girder stiffness was investigated for a bridge with two 18-m (60-ft) spans. Although the results shown in Figure 8 are somewhat irregular, maximum accelerations for that bridge would not be significantly increased by replacing the A36 beams with smaller, high-strength steel beams. The important point here is, of course, that deflection control is not directly related to vibration control.

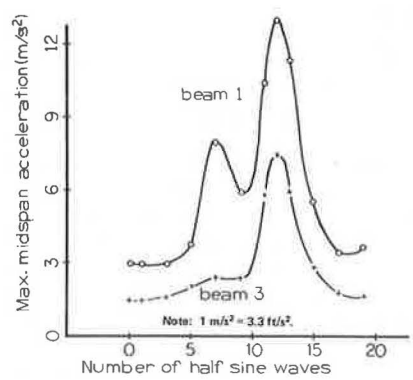


Figure 5. Acceleration versus roadway roughness for simple-span bridge.

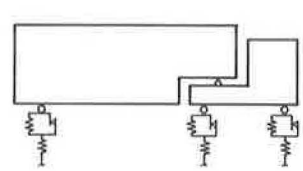


Figure 6. Three-axle vehicle model.

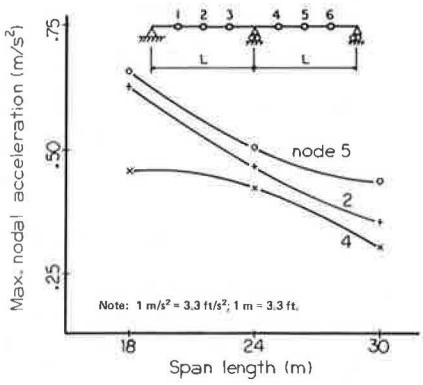


Figure 7. Maximum accelerations for two-span steel-beam bridges.

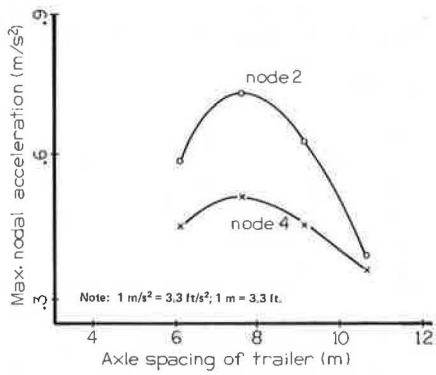


Figure 9. Acceleration versus axle spacing for two-span bridge.

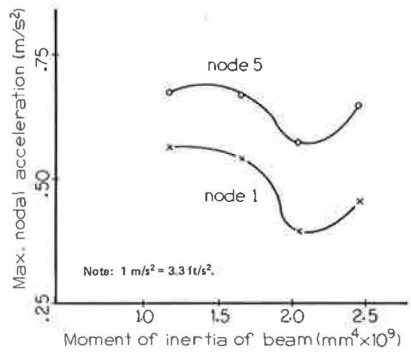


Figure 8. Acceleration versus beam stiffness for two-span bridge.

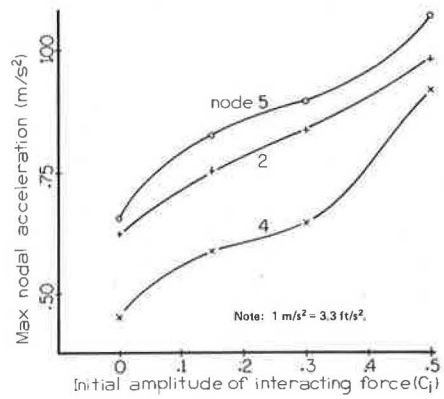


Figure 10. Effect of initially oscillating vehicle on bridge acceleration.

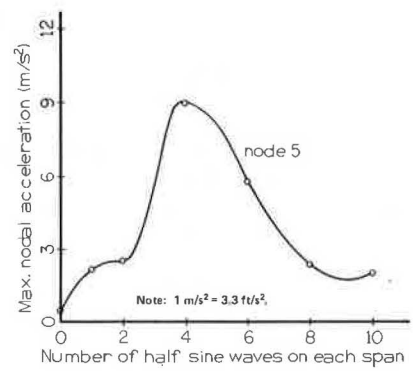


Figure 11. Acceleration versus roadway roughness for two-span bridge.

Vehicle Parameters

The values for the geometric, mass, and suspension parameters of the vehicle model were chosen to make the model closely represent a typical, heavily loaded tractor-trailer. The spacing of the tractor axles, although taken to be 3.7 m (12 ft), might be considered variable. As shown in Figure 9, the maximum acceleration for two 18-m (60-ft) spans occurs when the trailer axle spacing is about 8 m (25 ft) or 0.42 times the span length. The critical axle spacing ratio for three-span bridges is 0.37 to 0.43.

A comparison of maximum accelerations was also made by using one-, two-, and three-axle vehicle models. Maximum accelerations were about the same for two- and three-axle vehicle models, but they were about two-thirds of the magnitudes produced by the single-axle model. As in the case of simple-span bridges, a marked increase in acceleration with vehicle speed was found.

Maximum accelerations were also computed for different values of frequency ratio (the ratio of the natural frequency of the vehicle on its tires to the natural frequency of the bridge). Generally, the magnitudes of accelerations at the nodes were about the same for all values of frequency ratio although the midspan accelerations were slightly higher when the vehicle and the bridge had the same natural frequency.

A real vehicle is likely to be oscillating somewhat as it enters a bridge because of irregularities in the approach pavement and a possible discontinuity at the abutment. The most significant parameter for representing initial oscillations is the initial-axle-force variation (C_i). The initial axle force is equal to $(1 + C_i)$ times the static force. Figure 10 shows maximum nodal accelerations for four values of C_i. The same C_i value is assumed for each axle. According to Nieto-Ramirez and Veletsos (15), $C_i = 0.15$ might correspond to a 3-mm (0.125-in) pavement irregularity and $C_i = 0.50$ might represent a large discontinuity at the abutment. It can be seen that the initial oscillation causes a 30 to 50 percent increase in maximum acceleration for this bridge-vehicle system, which is assumed to have a smooth deck surface. An investigation that considered a phase angle difference between initial oscillations of the trailer axles found that maximum accelerations varied less than 20 percent.

Surface Roughness

All of these two-span studies assumed a smooth bridge-deck surface. The effect of surface roughness has been investigated for a standard design bridge with two 18-m (60-ft) spans that was loaded by an HS20 truck. The deck surface was represented by an integral number of half sine waves. The influence of roadway roughness is shown in Figure 11. With four half waves of roughness in each span, the frequency of oscillation of the interacting forces is close to the fundamental natural frequency of the bridge and the nearly resonant response occurs. Thus, for both simple and continuous bridges,

surface roughness seems to be the most significant factor affecting roadway accelerations.

CONCLUSIONS

Analytical studies have shown that the significant parameters that influence bridge accelerations are vehicle speed and weight, bridge-span length, and surface roughness. Maximum acceleration levels were found to be rather high for typical simple-span bridges; however, accelerations for two- and three-span continuous bridges exceeded the suggested comfort limit only when severe effects of surface roughness were included.

Current specifications attempt to control bridge vibrations by limiting girder flexibility. In this study, only a small increase in maximum acceleration resulted when girder flexibility was increased by replacing A36 beams with smaller, high-strength steel beams. Thus, using more efficient, high-strength steel designs may be possible without adversely affecting user comfort.

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The aid of W. H. Walker of the Civil Engineering Department of the University of Illinois in making available the computer programs used in this study is gratefully acknowledged. This analytical study is a part of the Joint Highway Research Project, Bridge Vibration Studies, which involves the cooperative efforts of the School of Civil Engineering of Purdue University and the Indiana State Highway Commission. The research is being carried out under the sponsorship of the Federal Highway Administration, U.S. Department of Transportation.

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Abridgment

Load Distribution on a Timber-Deck and Steel-Girder Bridge

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Some general studies, as well as investigations of load distribution, have been conducted on timber bridges (1, 2, 3, 4). Most of this research has concerned structures with timber decks and timber girders. A laboratory study conducted by Agg and Nichols (5) was concerned with wood floors on steel floor joists. The study re-