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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Abridgment

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

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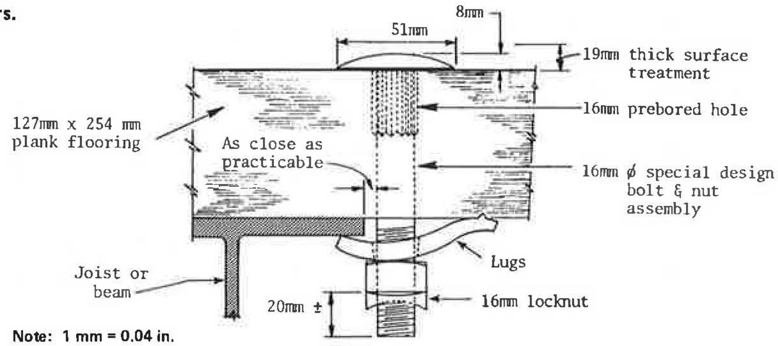
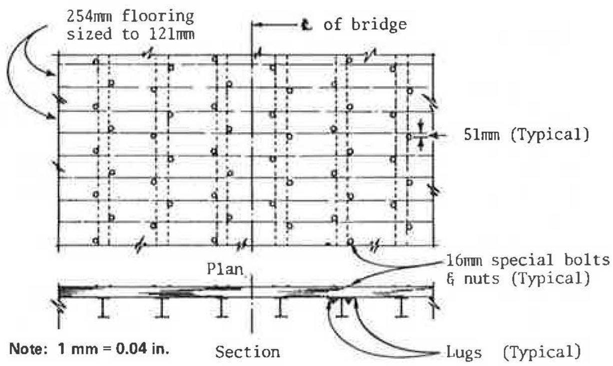
Some general studies, as well as investigations of load distribution, have been conducted on timber bridges (1, 2, 3, 4). Most of this research has concerned structures with timber decks and timber girders. A laboratory study conducted by Agg and Nichols (5) was concerned with wood floors on steel floor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

Figure 1. Floor plank fasteners.



Note: 1 mm = 0.04 in.

Figure 2. Floor fastener arrangement.



Note: 1 mm = 0.04 in.

ported here deals with the load distribution on an in-service bridge with a timber deck and steel girders.

DESCRIPTION OF TEST STRUCTURE

The test bridge was a 7-m (23-ft) wide by 14.8-m (48.5-ft) long simple-span structure that conforms to the Virginia standards for wooden floor-steel beam bridges designed for H20 loading. Figure 1 shows the standard fasteners with which the nominal 127 × 254-mm (5 × 10-in) floor planks are attached to the steel girders. The fastener bolts are inserted through predrilled holes in the planks and locked to the upper flange of each girder in the staggered arrangement shown in Figure 2. The deck floor is covered with a 19-mm (0.75-in) bituminous wearing surface.

The superstructure of the bridge is composed of 14 21WF68 steel beams spaced 0.5 m (19.75 in) apart on centers for the interior bays and 0.6 m (24 in) apart on centers for the first two exterior bays on each side of the span. Only 6 of the 14 girders are anchored to the abutment seats; all others simply rest on the abutment beam seats.

INSTRUMENTATION AND LOADING

Strain gauges were applied to 8 of the 14 girders at midspan. The structure was loaded with a truck that simulated the type 3 loading designated by the American Association of State Highway and Transportation Officials (AASHTO) (6). The type 3 loading has a total weight of 20.9 Mg (23 tons); the truck used in this study weighed 20.6 Mg (22.7 tons). The distance between the front and the first rear axles of the truck, however, was 0.51 m (1.67 ft) shorter than the 4.57 m (15 ft) designated for the type 3 load. Thus, for the span investigated, the loading used produced midspan moments in the girders that were very close to those that would be developed by the type 3 legal load unit: 290 032 N•m (213 347 lbf•ft) actually applied versus 285 174 N•m (209 774 lbf•ft) for type 3. It should be noted that the type 3 loading, unlike other types of legal loadings, will develop the maximum moment in a 14.8-m (48.5-ft) bridge span.

The study determined the stresses in the steel girders that resulted from various loading sequences. Load distribution characteristics of the structure were determined under varying conditions with loosened floor fasteners. More details on the instrumentation, the type of loading, and the test procedures are available in another report (7).

RESULTS

If all of the live load moment developed by the line of the wheels of the test vehicle were carried by a single girder with no lateral load distribution, a stress of 126.5 MPa (18 350 lbf/in^2) would be developed in that member. Considerable lateral load distribution takes place, of course, and it was found that the highest stresses developed on the interior girders of the bridge were on the order of 41.4 MPa (6000 lbf/in^2) or less. The highest stress developed for all of the lateral axle positions used on the span was 49.8 MPa (7220 lbf/in^2); this occurred at an exterior beam when the fasteners on the four adjacent girders were loosened and the tread of the nearest tire was 51 mm (2 in) from the curb. In this case the resultant of the load (placed on the bridge by the two wheels of one side of a rear axle) would fall midway between the exterior and the first interior girders, which are spaced 0.6 m (2 ft) apart.

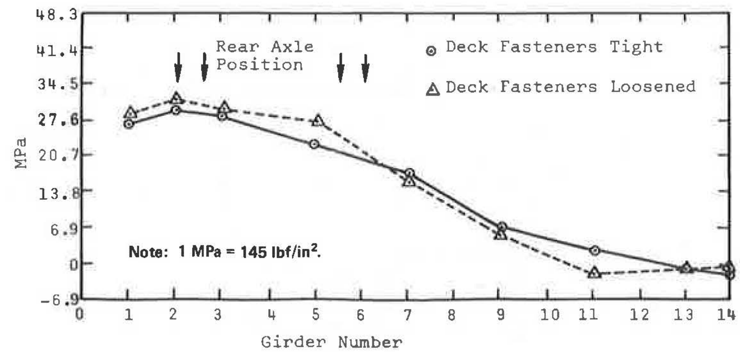
Figure 3 shows the midspan stresses on the lower flanges of the steel girders for conditions in which floor fasteners were tightened and loosened. These data show the following results.

1. The live load stresses resulting from the truck loadings are laterally distributed in a reasonably consistent manner, even when the fasteners are loosened.
2. The live load stresses in the steel girders are reasonably low. Although design stresses must include dead load and impact, the maximum stress recorded on the interior girders was only 31 percent of the 138 MPa (20 000 lbf/in^2) allowable for A36 steel. In most instances the maximum stresses recorded for the interior girders were on the order of 25 to 30 percent of the allowable total.
3. Loosening of the deck plank fasteners does not have a very significant effect on either the magnitude or the lateral distribution of the stresses.

LOAD DISTRIBUTION FACTORS

The current practice of the Virginia Department of Highways and Transportation is to distribute the live load

Figure 3. Lateral midspan stress distribution produced by truck loading position shown for conditions of loosened and tightened fasteners.



moments to the interior girders of all timber-deck bridges by using a factor of S/4, where S is the spacing in feet between adjacent girders specified by AASHO (8) (because the AASHO values for load distribution are empirically derived in customary units, no SI equivalents are given). The live load is distributed to the exterior girders by using the reaction of the wheel load obtained by assuming the flooring to act as a simple beam between the exterior and the first interior girders.

Load Distribution to Interior Girders

The load distribution factor of S/4 was found to be conservative in all cases for the interior girders. The highest load distribution to interior girders was developed by loading both lanes of the two-lane bridge. The highest load distribution factor obtained from the data would yield an equivalent formula of S/5.12. Considering the remaining lower stresses obtained in the investigation, the denominator of the formula would be larger. These data suggest that, for 127-mm (5-in) thick timber-deck bridges such as the SS-4 Virginia standard structure investigated, a distribution factor for the interior girders of S/5 would be adequate for legal limits.

Load Distribution to Exterior Girders

The load distribution factor determined for the exterior girders by proportioning the load as the reaction of a simple beam between the exterior and the first interior girders was found to be inadequate in some instances. Specifically, the Virginia procedure used to evaluate the exterior girders calls for positioning the resultant of the wheel line 0.55 m (18 in) from the curb, which corresponds to the study loading position in which the nearest tire tread is 191 mm (7.5 in) from the curb. By using the simple-beam assumption, a distribution factor of 191/609 (7.5/24), or 0.315, would be obtained. Moving the loading position from 191 to 51 mm (7.5 to 2 in) from the curb changes the calculated simple-beam distribution factor from 0.315 to 0.50 and the actual factor from 0.343 to 0.39. Thus, for the 140-mm (5.5-in) lateral change of wheel-load position, the actual change in load distribution was much less than that calculated by the procedure for simple-beam reaction. Because the 0.343 and 0.39 factors can be transformed respectively to S/5.38 and S/5.08, a general distribution factor of S/5 could be used. An analysis of other loading positions in the area between the first and second girders suggests that the use of a distribution factor of S/5 would be more realistic than the current procedure used for exterior girders.

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