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AN INVESTIGATION OF THE LOAD DISTRIBUTION ON A TIMBER DECK-STEEL GIRDER BRIDGE

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(The opinions, findings, and conclusions expressed in this report are those of the authors and not necessarily those of the sponsoring agencies.)

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SUMMARY

The load distribution on a 48.5-ft. span timber deck-steel girder bridge built to the Virginia Department of Highways and Transportation standard SS-4 requirements was investigated under two conditions. The first condition was concerned with the load distribution when the timber deck fasteners were tight and the second condition involved investigating the load distribution when the fasteners were loosened in several stages.

The results of the study indicated that the live load stresses were of the same general order of magnitude and that the lateral load distribution was generally the same whether the deck plank fasteners were loose or tight. The load distribution factor currently used for the interior girders was found to be too high, whereas the procedure used for distribution of the load to the exterior girders was slightly low. It is recommended that consideration be given to using a load distribution factor of $\frac{S}{5}$ for the evaluation of both the interior and exterior girders of timber deck-steel girder bridges similar to the one investigated. Compared to current procedures, this modification would result in a 20% reduction in the live load distribution to the interior girders and a 28% increase in the distribution to the exterior girders.

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INTRODUCTION

In recent years a safety inspection program for evaluating and rating bridges has been required by the Department of Transportation. Concurrent with the continuous evaluation program the load limits on many secondary road bridges are determined and the structures are posted for load limits where required. In this program, the load limits being posted on many timber deck bridges are lower than the loads these structures have sometimes carried in the past. In many instances these load limits force heavy trucks carrying stone, concrete, etc., to take circuitous routes to get to their destinations or reduce their loads and make additional trips. Obviously, some road users cannot understand the reasons for the load limits when previously some structures had carried the loads that are now considered to be excessive. Thus, the more restrictive load limits have in some cases been questioned and felt to be too low and unwarranted.

In addition to the fact that the load limits are more restrictive on some structures than those that were previously allowed, some engineers feel that information is lacking concerning the load distribution on the standard steel beam-timber deck bridge that is in use on many of Virginia's secondary and, in some instances, higher classification roads. Although most of the timber deck structures in Virginia are located on secondary roads, there are an impressive number in service. Roughly half the bridges (approximately 900) in one of Virginia's eight highway districts, for example, are timber deck type structures. While many of these are of very short length, it is apparent that a large number of timber decked-steel girder bridges are in service in Virginia when all eight highway districts are considered. Thus, it is apparent that the ratings and evaluations of highway bridges in service involve a large number of timber deck structures with many of these having restrictive load limits posted.

Somg general research, as well as investigations of the load distribution on timber bridges, has been conducted.[1,2,3,4] Most of this research has dealt with timber deck-timber girder type structures — although one laboratory study conducted by Agg and Nichols[5] was concerned with wood floors on steel floor joists. There have been no known investigations, however, of the load distribution on in-service timber deck-steel girder bridges like those used in Virginia.

PURPOSE AND SCOPE

Many variables could have some influence on the load distribution characteristics of timber decked bridges. Since it is virtually impossible to investigate the effects of many variables when working with in-service structures, only the load distribution to the steel girders and the effects of floor fasteners were investigated in this initial study. Consequently, a structure that carries very little traffic was selected for static load tests to determine the following: (1) The actual vs. the specification load distribution factors for various lateral positions of a truck loading positioned on the span to develop maximum moment at midspan, and (2) the effect on the load distribution to the steel girders of loosening the floor systems' fasteners.

Since loosening of the floor fasteners can readily occur due to factors such as aging and localized crushing of the floor planks adjacent to the fasteners, this variable was included in the study.

An additional purpose of the investigation was to assess the desirability of testing a second structure to include the effects of some other variables such as skews, plank thickness, and lane width. It was expected that the results of the measurements taken on one wooden deck bridge would help determine the need for additional field study and testing.

The work was limited to the testing of one standard Virginia Highway and Transportation Department wooden floor-steel girder bridge under static live load (without impact). Determined during the study were the stresses in the steel girders resulting from the various loading sequences (described later) and the load distribution to the steel girders as developed from the strain data. The stresses and load distribution characteristics of the structure under several situations of loosened floor fasteners were included in the scope of the work.

DESCRIPTION OF THE BRIDGE INVESTIGATED

A two-lane, 48.5-ft. simple beam span (Figure 1) that carries Rte. 715 across the South Fork of the Rockfish River in Nelson County was selected for study. This structure has a 23-ft. roadway and was built approximately five years ago to replace a bridge washed out during the 1969 flood. The dimensions and material sizes on the structure conform very closely to the standard Virginia SS-4 plan for wooden floor steel beam bridges that are designed for H20 loading. The nominal 5 in. x 10 in. floor planks are attached to the steel girders with the standard fasteners shown in Figure 2. 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 3.

The superstructure of the bridge is composed of 14-21 WF 68 steel beams spaced 19-3/4 in. on centers for the interior bays and 24 inches on centers for the first two exterior bays on each side of the span (Figure 4). Only 6 of the 14 girders are anchored to the abutments as indicated in Figure 4. All others simply rest on the abutments.

The total length of the bridge deck was composed of 64 wooden planks with 2 of these — 1 on each end of the span — resting on the abutment backwall rather than on the steel girders. The actual dimensions of the floor planks were approximately 4-3/4 in. x 9-1/2 in., and the center portion of the width of the deck floor was covered with a 3/4-in. bituminous wearing surface as shown in Figure 4.

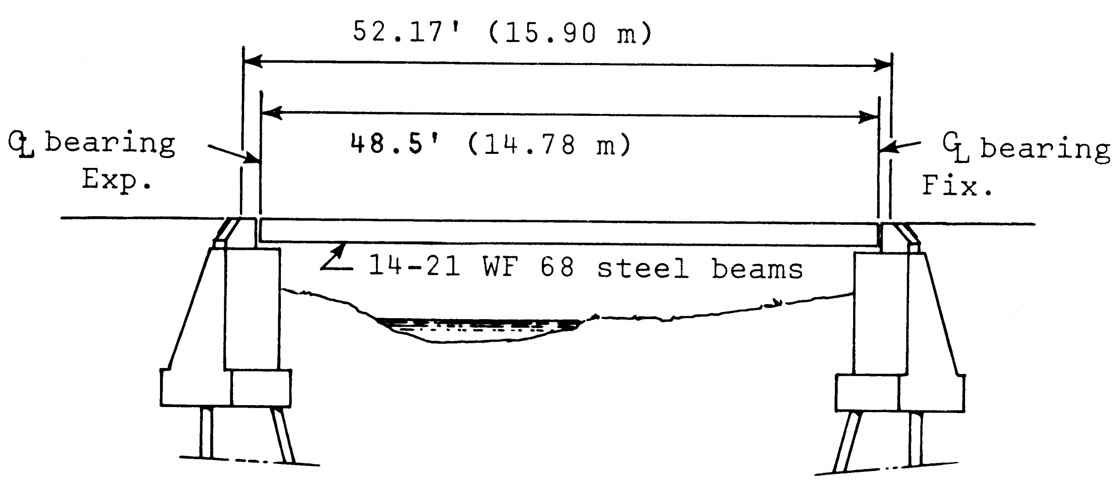


Figure 1. Elevation view of the steel girder — wooden floor bridge investigated.

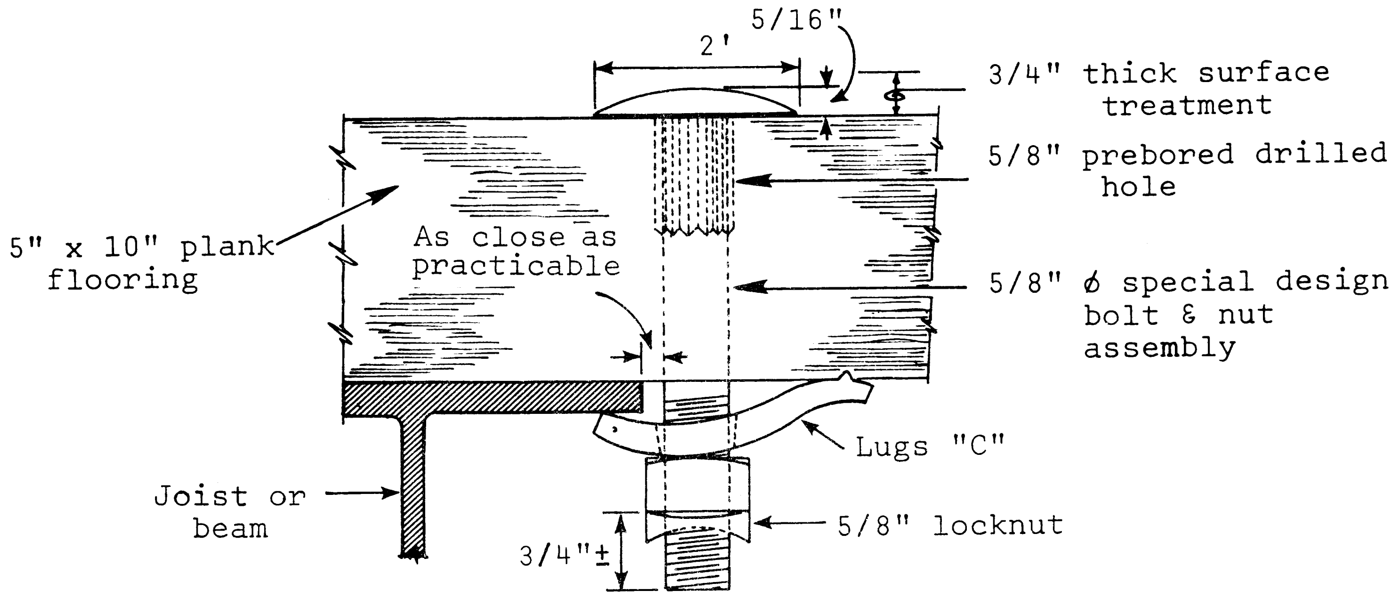


Figure 2. Detail of floor plank fasteners.

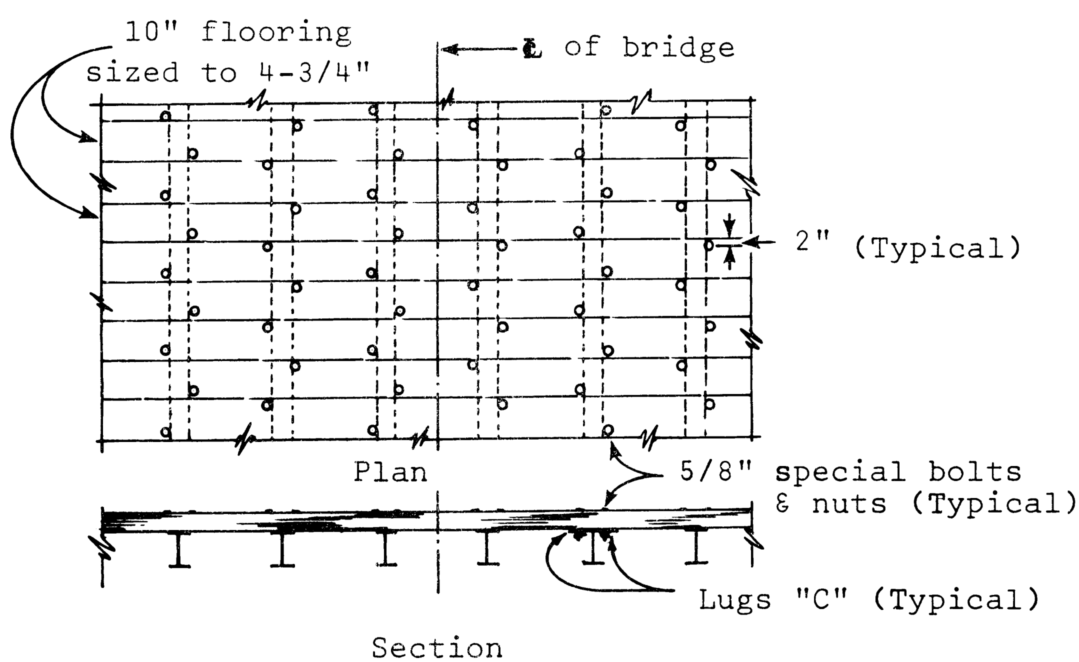


Figure 3. Partial plan and sectional view showing the floor fastener arrangement.

INSTRUMENTATION AND LOADING

Instrumentation

The Rte. 715 bridge was instrumented with 16 electrical resistance SR-4 wire type strain gages attached to the lower and upper flanges of the steel girders at the selected locations indicated in Figure 4. Since preliminary calculations had indicated that the most highly stressed members under loading would be the exterior and first interior girders, the first 3 girders on one side of the structure were all gaged. Because of the close beam spacing it was not considered necessary to gage all of the remaining girders. Therefore, strain gages were applied to the lower flanges of 8 of the girders and are designated as L_1 through L_8 in Figure 4. Upper flange gages, designated as U_1 through U_6, were placed on 3 selected girders. A gage, W_1, was placed on the web of one member and another gage was placed on the underside of a floor plank at a distance halfway between the adjacent supporting girders.

All of the strain gages were placed on the girders at midspan. Although this location is not the maximum moment location for certain truck axle configurations, the axle dimensions on the truck that was used for loading purposes was not known at the time the gages were being applied. The strains obtained during the tests, therefore, would be slightly lower at midspan than those that might have been obtained had the gages been placed at the maximum moment point. This difference, however, is slightly less than 1% and is not significantly related to the objectives of the investigation.

Truck Loading

A privately owned dump truck, which is normally contracted for Virginia's Department of Highways and Transportation hauling, was used to apply axle loads to the bridge. The truck was loaded with crushed stone and the front and rear axle weights, as well as the total weight, were recorded at a nearby rock quarry. The truck axle dimensions and average axle loads are detailed in Figure 5.

The truck loading was nearly the same as the type 3 unit loading designated in the Manual for Maintenance Inspection of Bridges.(6) The type 3 unit loading has a total weight of 23 tons, whereas the truck used in this study weighed 22.7 tons. The distance between the front and the first rear axle of the truck used, however, was 1.67 ft. shorter than the 15 ft. designated for the type 3 unit. Thus, for the 48.5 ft. 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 (2567 in.-kips actually applied vs. 2524 in-kips for type 3). As opposed to the other legal loading types described in the manual,(6) it should be noted that the type 3 unit will develop the maximum moment in a 48.5 ft. span.

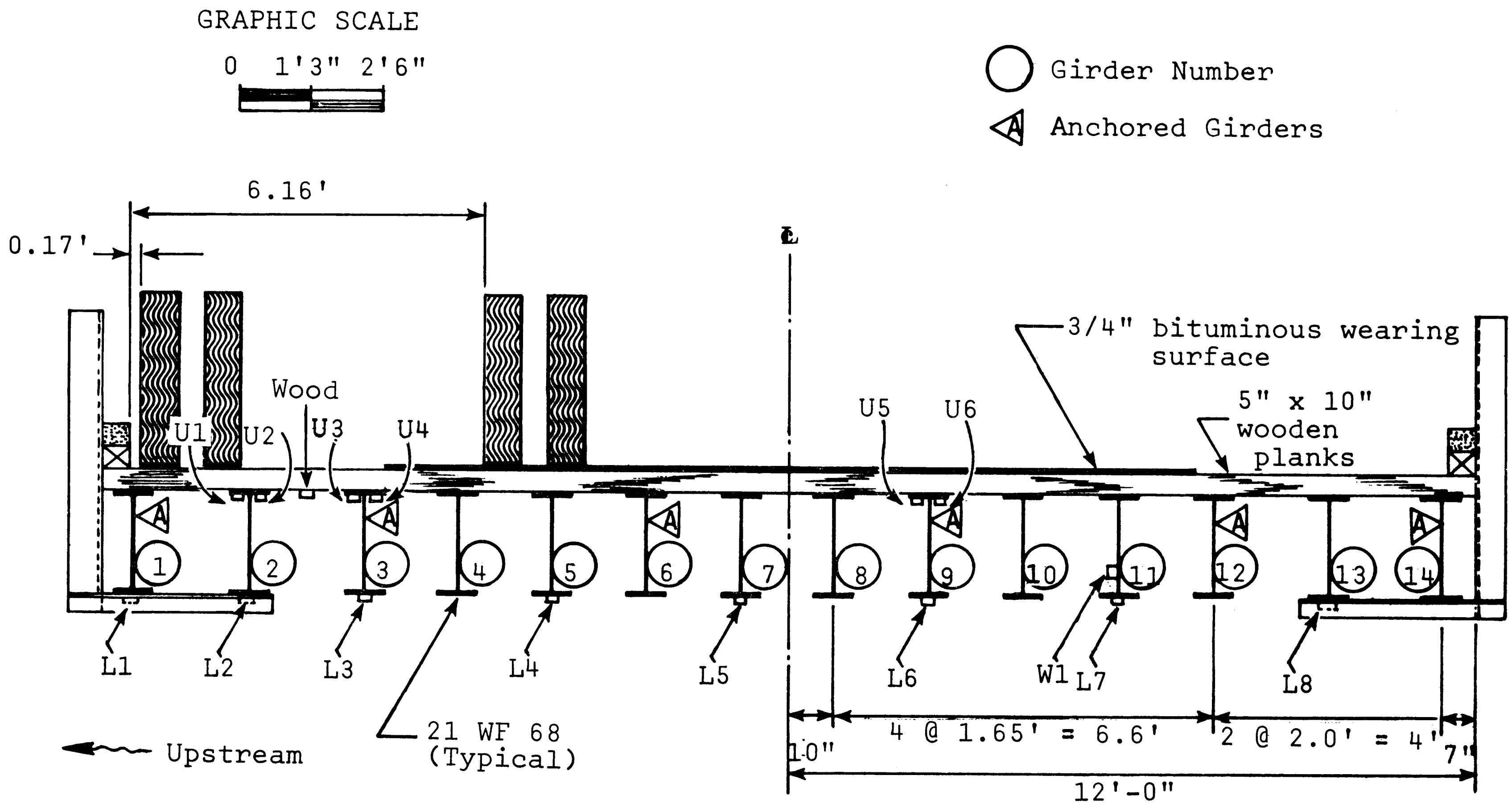
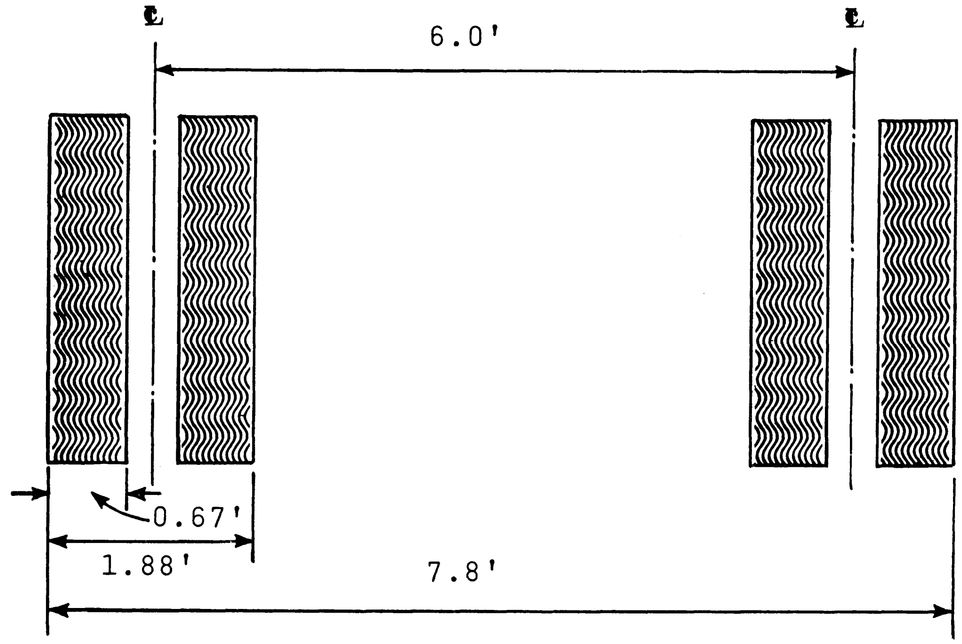
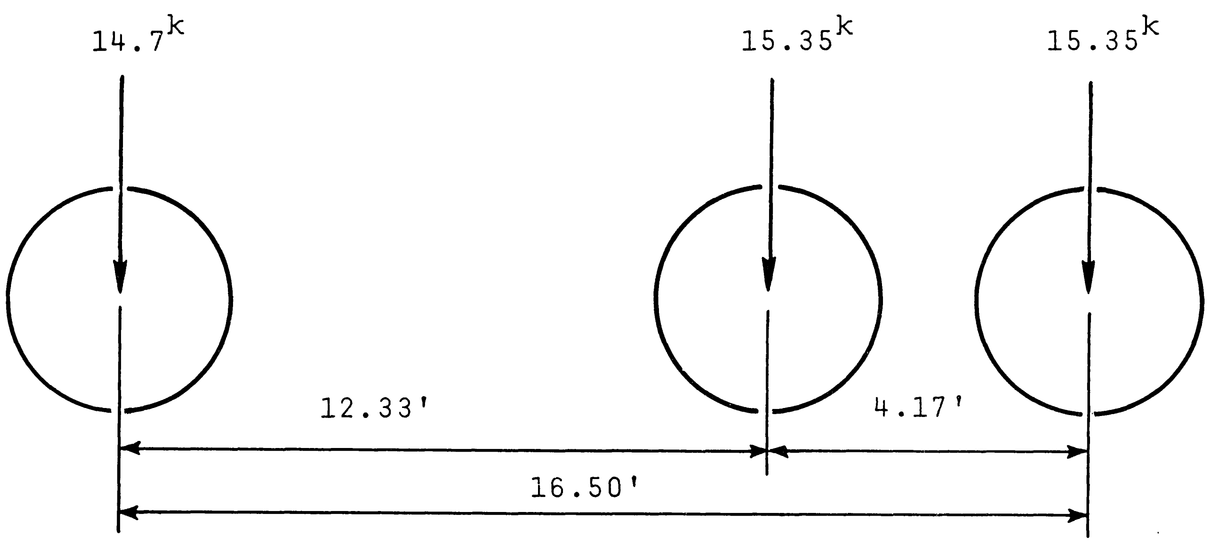


Figure 4. Transverse section of the bridge superstructure showing dimensions, locations of strain gages, and a typical lateral position of the rear axle of the truck used for loading the span.



(a) Rear axle dimensions



(b) Axle loads in kips

Figure 5. Axle dimensions and average axle loads on the truck used for loading the bridge.

Since the loading tests were conducted over a three-day period, the weight of the truck was rechecked several times during this period to monitor possible load shifting and moisture accumulation or evaporation in the crushed stone. Although the axle weights varied several hundred pounds between weighings, the small variations in relation to the total axle weights had no significant effect on the strains recorded for the various truck positions on the bridge deck. Thus, the axle loads shown in Figure 5 are the average for the three-day test period.

Loading Procedure

The truck was initially positioned on the span in several longitudinal and lateral positions to check for possible malfunctioning of the strain gages and to experimentally determine the longitudinal position of the wheel loads required to develop maximum strains at midspan. As would be expected, the maximum strains occurred at midspan when the first rear axle (with reference to the front axle) was positioned at that point. Positioning the first rear axle several inches on either side of midspan, however, had little or no effect on the strain readings.

To determine the load distribution to the steel girders under various loading conditions, strains were recorded at all gage points for a number of lateral positions of the truck. A typical loading position showing the rear axle with reference to the wheel guard on one side of the bridge is illustrated in Figure 4. Although the sidewalls of the truck tires were against the wheel guard for the particular loading position illustrated, the tire treads bearing on the deck surface were 2 in. from the guard. The remaining lateral positions of the truck with reference to the face of a wheel guard are tabulated in Table 1. Minor variations in the lateral position of the truck of usually less than an inch occurred when the vehicle was repositioned for each loading series. A typical view of the truck being positioned on the timber decked bridge is shown in Figure 6.

Strains were recorded at all gage points for each of the lateral loading positions when all of the deck plank fasteners were tight. When all of the fasteners were loose, and for several conditions where only those at certain girders were loosened, strains were recorded at all gages for some selected loading position. From these strain data, stresses, moments, and load distribution factors were calculated.

Table 1

Lateral Positions of the Truck Loading with Reference to the Wheel Guard on One Side of the Superstructure

Position No.	Distance to edge of rear axle tire tread nearest the wheel guard, ft.
1	0.17
2	0.63
3	0.75
4	1.83
5	2.25
6	2.83
7	3.17
8	4.00
9	4.50
10	5.08
11	6.42
12	8.17
13	9.75
14	14.08

Figure 6. The truck being positioned on the 48.5 ft. timber deck span to obtain maximum moment at midspan of the steel girders.

RESULTS

General

The moments, stresses, and distribution factors reported are those developed only by the static live loading applied to the timber deck-steel girder bridge. No dead load moments or impact factors are included. Therefore, if one is interested in the total stresses that might be developed in each of the instrumented girders for a particular truck loading position, they could be found by adding in the dead load moments and applying the appropriate impact factor (0.29 in this case) to the actual live load moments or stresses. To satisfy the main purposes of the study, i.e., investigating the load distribution for various lateral loading positions and for various loosened plank fastener situations, the live load moments and stresses are all that need be considered.

By applying one line of wheels from the actual axle loads (Figure 5) to the 48.5 ft. span a maximum live load moment at mid-span of 2,567 in.-kips can be obtained. If all of this moment were carried by a single girder with no lateral load distribution occurring, a stress of 18.35 ksi 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 6.0 ksi or less. The highest stress developed for all of the lateral axle positions on the span was 7.22 ksi and this occurred at an exterior beam when the fasteners on girders 1-5 were loosened and the tread of the nearest tire was 2 in. from the curb as shown earlier in Figure 4. 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 2 ft. apart.

Some of the typical representative results of the investigation are presented in Figures 7 through 10. In each of these figures the midspan stresses on the lower flanges of the steel girders are shown for both the tightened and loosened floor fastener conditions. For those girders in the span that were not instrumented, the stress distribution shown in Figures 7-10 is assumed to be linear between the adjacent girders.

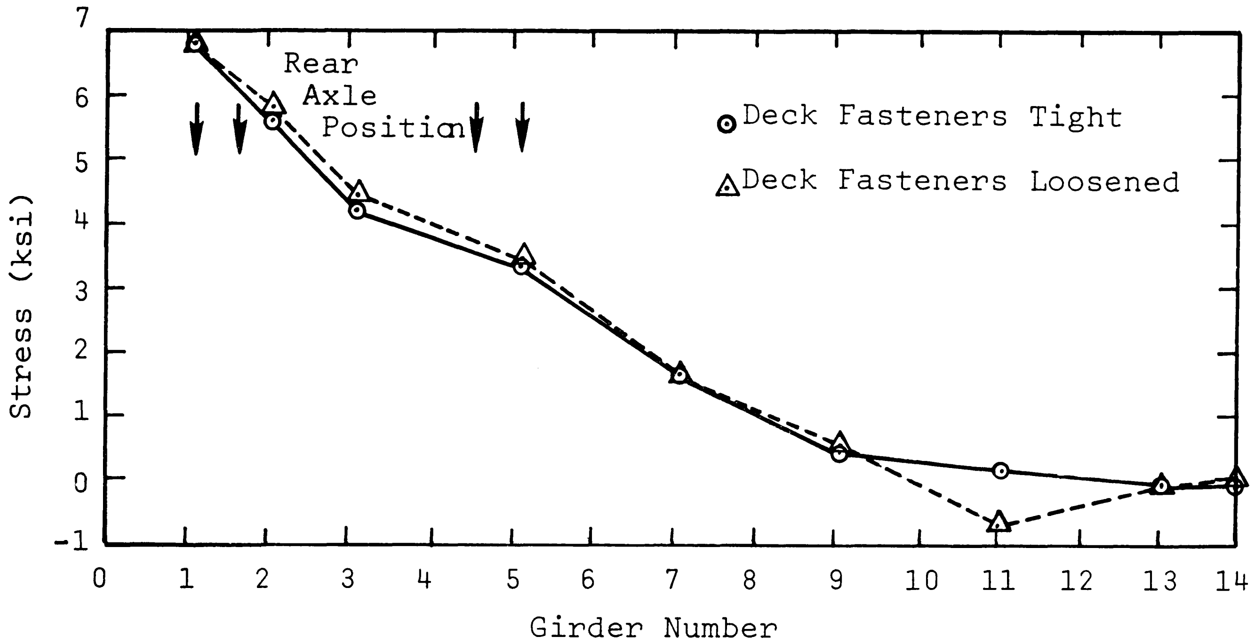


Figure 7. Lateral midspan stress distribution produced by the truck loading at position 1.

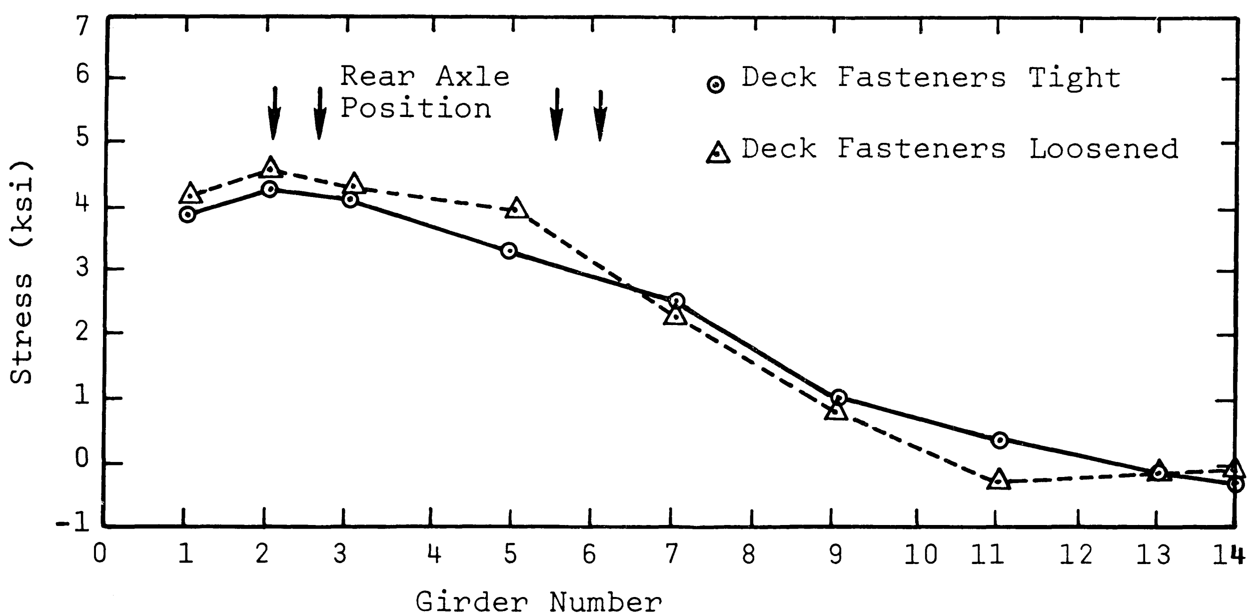


Figure 8. Lateral midspan stress distribution produced by the truck loading of position 4.

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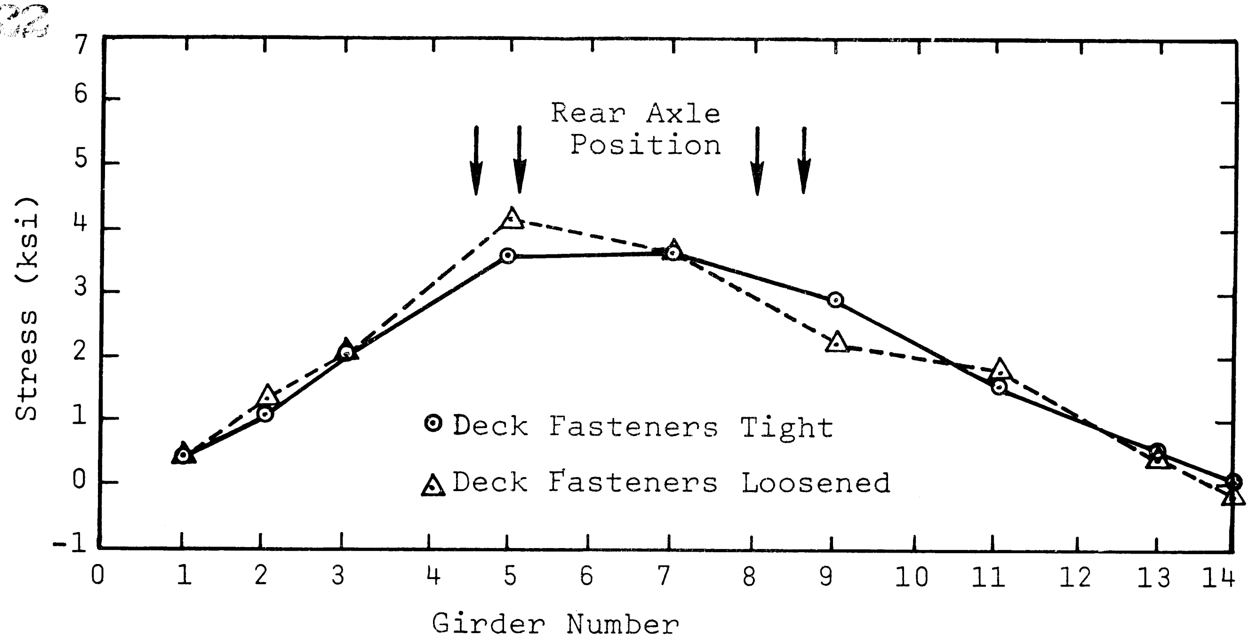


Figure 9. Lateral midspan stress distribution produced by truck loading at position 11.

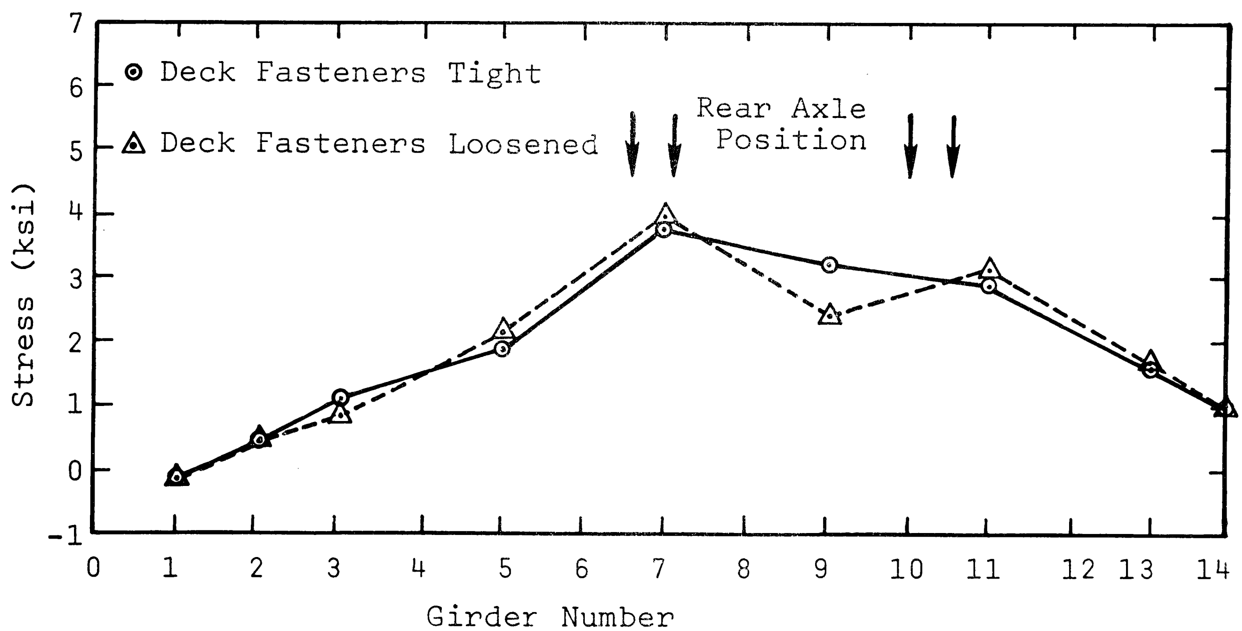


Figure 10. Lateral midspan stress distribution produced by the truck loading at position 13.

These data show several things. First, the live load stresses resulting from the truck loadings are laterally distributed in a reasonably consistent manner even with the fasteners loosened. Secondly, the live load stresses in the steel girders are reasonably low. While design stresses must include dead load and impact, the maximum stress recorded on the interior girders was only 31% of the 20 ksi allowable for A36 steel. In most instances the maximum stresses recorded for the interior girders were on the order of 25-30% of the total allowable. Normally, load stresses on the order of 40-50% of the total allowable are not unreasonable. Lastly, loosening of the deck plank fasteners does not have a very significant effect on either the magnitude or the lateral distribution of the stresses. Girder number 5, as designated in Figure 4, had the highest increase in stress when the fasteners were loosened. Data given in Figure 11 show this increase in stress for the five loading positions for which data were recorded. The maximum increase in stress was 15% when the truck was at loading position 4. For the other girders the stress increase was less than that observed on girder number 5 and in many instances, as can be seen in Figures 7-10, the stresses were less than those recorded when all the fasteners were tight.

Since only one truck was used to load the bridge, the stress data from loading one side of the symmetrical structure were superimposed on those obtained from loading the opposite side to determine the maximum stresses, assuming both lanes were loaded. From the face of the wheel guard on each side of the bridge loading positions equivalent to numbers 2, 5, and 7 in Table 1 were superimposed. As shown in Figures 12-14, a maximum stress of 6.3 ksi was obtained on the exterior girders and 5.9 ksi on the interior ones.

Stresses developed on the lower side of the upper flanges of the girders were of the same order of magnitude as those recorded on the lower flanges. While in some instances the stresses on the upper flange of a particular girder would be slightly higher than those on the lower flange, the variances are probably due to the dissimilar locations of the gages and to localized stresses developed by the wheel loads and fastener attachments on the top flanges. The highest stress recorded on an upper flange was 6.24 ksi.

One strain gage was located on the web of girder number 11 at a distance of 6.4 in. from the bottom side of the lower flange. This is at a point approximately 30% of the depth from the bottom flange of the girder. The maximum stress developed at this point was 1.28 ksi when the truck was at position 14 on the span.

A final gage, which was installed on a wooden plank halfway between girders 2 and 3, indicated a maximum live load stress of 1.04 ksi when the wheel loading was directly above the gage. Assuming an allowable bending stress of 1.85 ksi for a southern pine dense structural 72 grade lumber with a modulus of elasticity of 1.6×10^6 psi, the stresses developed in the floor plank appear to be well below that allowable.

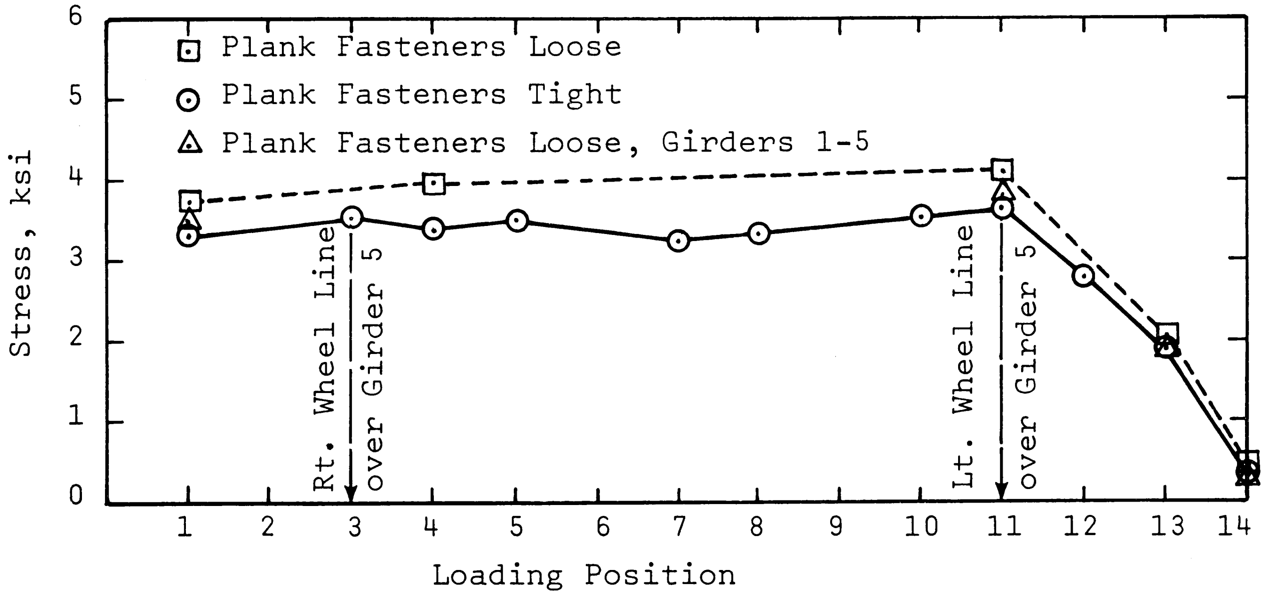
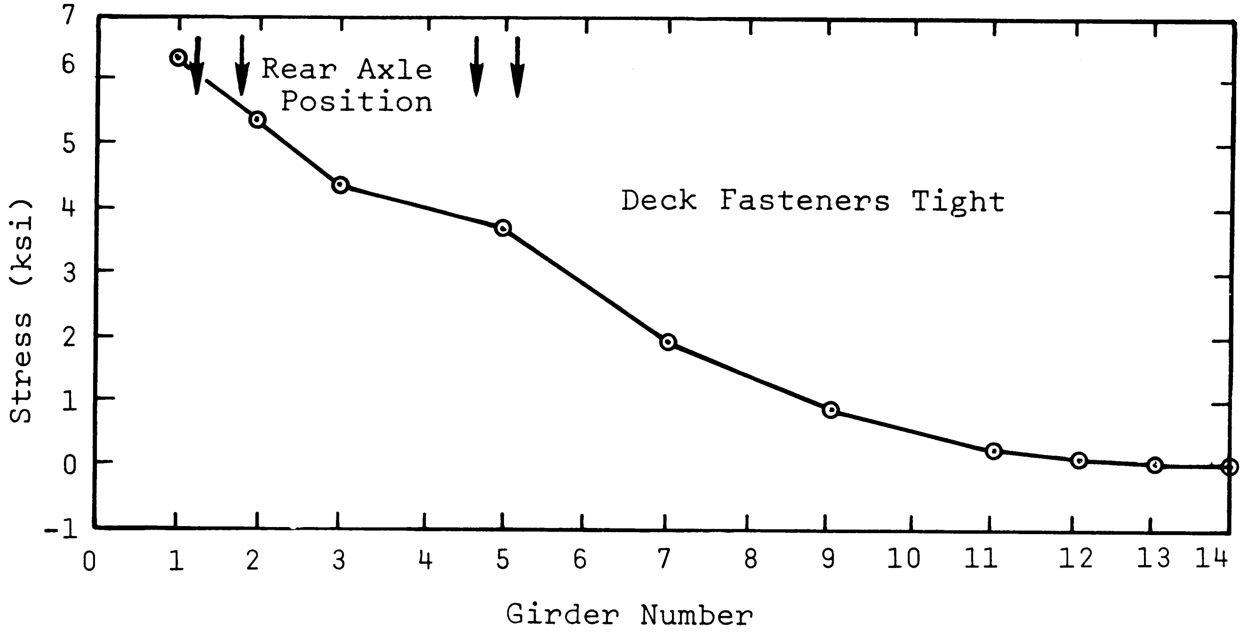
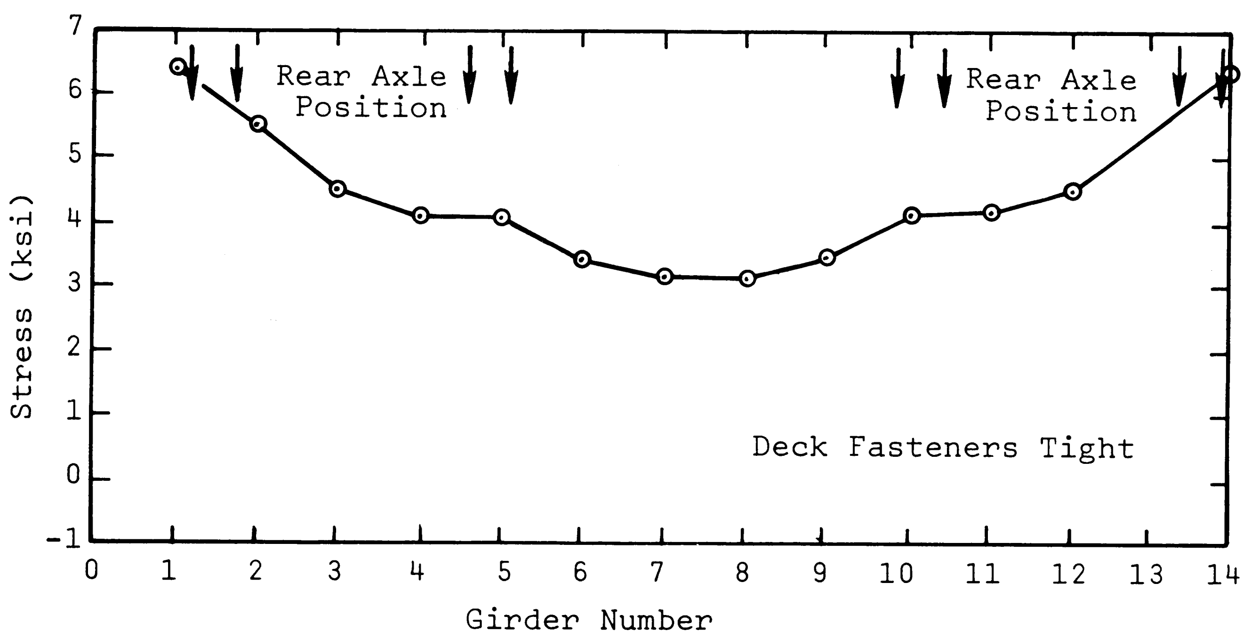


Figure 11. Stress of midspan of girder #5 for various lateral positions of the truck loading showing the effect of several loosened fastener conditions.



(a)



(b)

Figure 12. (a) Lateral midspan stress distribution produced by truck loading positioned at 0.63 ft. (18 in. to resultant of nearest wheel line) from the curb for single lane loading.
(b) Midspan stress distribution with both lanes loaded by superposition. Each truck is positioned 0.63 ft. from the curb in the left and right lanes.

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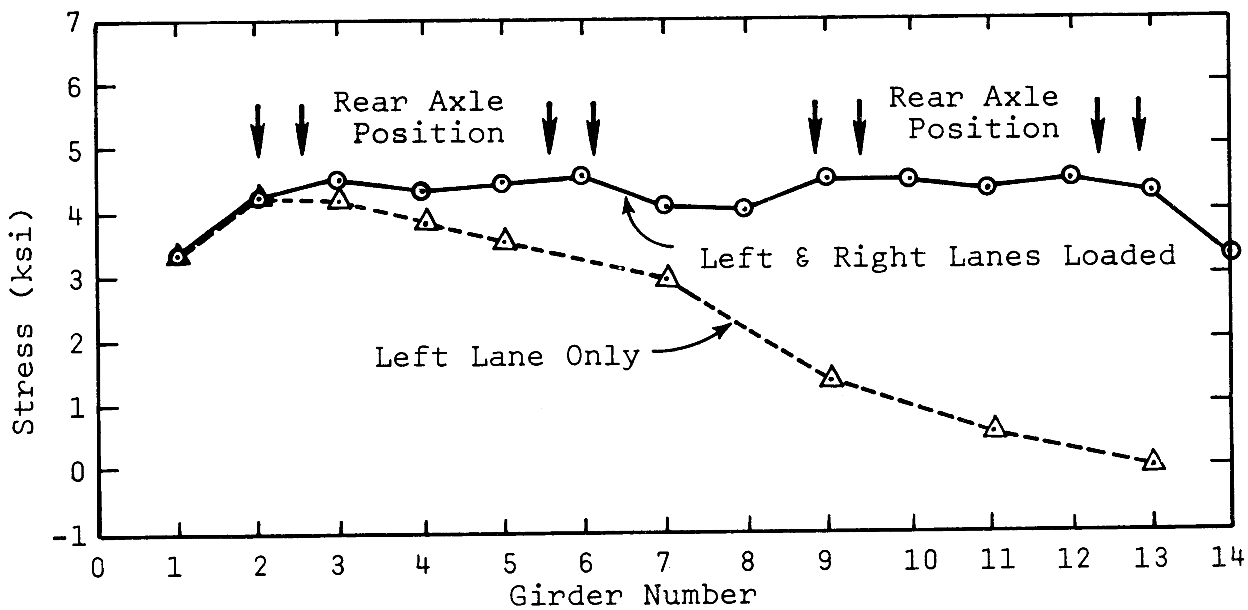

Figure 13. Midspan stress distribution with both lanes loaded by superposition. Each truck is positioned 2.25 ft. from the curb in the left and right lanes.

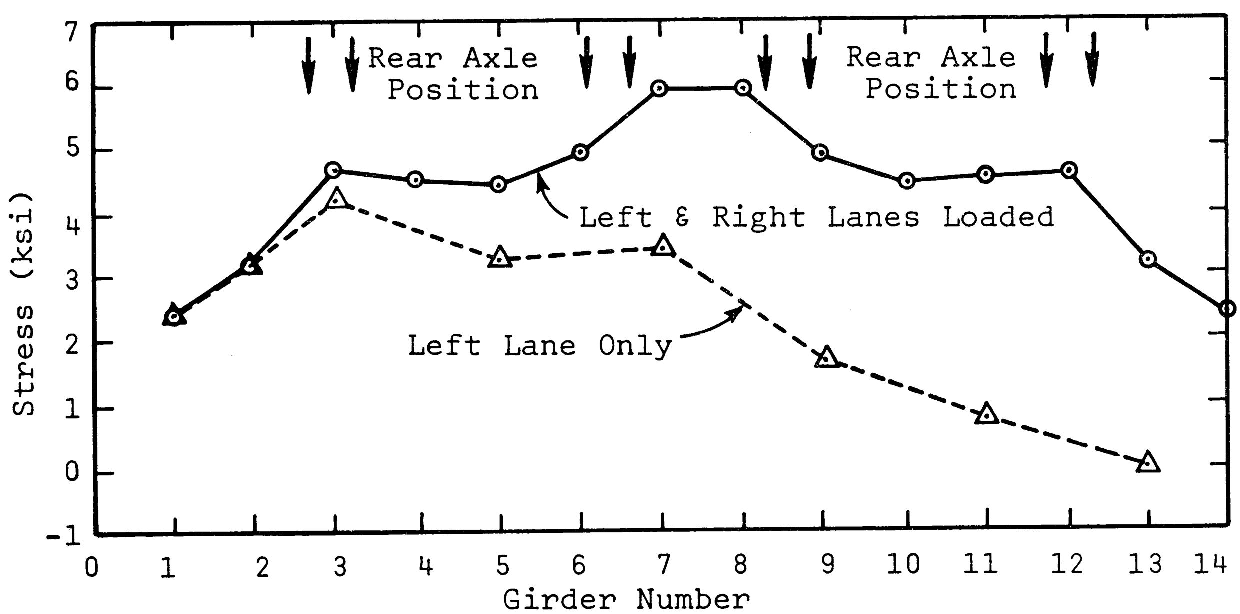

Figure 14. Midspan stress distribution with both lanes loaded by superposition. Each truck is positioned 3.17 ft. from the curb in the left and right lanes.

Load Distribution Factors

The current practice of the Virginia Department of Highways and Transportation is to distribute the live load moments to the interior girders of all timber plank decked bridges by using a factor of $\frac{S}{4}$, where S is the spacing between adjacent girders. 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 first interior girders.

Load Distribution to Interior Girders

For the interior girders the load distribution factor of $\frac{S}{4}$ was found to be conservative in all cases. For the 19.75 in. interior girder spacing this formula would yield a distribution factor of 0.41, and for the 24 in. spacing a factor of 0.50. By proportioning the actual load moments in the girders to the 2,567 in.-kip maximum moment applied, it was found that the highest load distribution to interior girders was developed by the two-lane loading shown in Figure 14. This distribution would be 0.322, which is equivalent to $\frac{S}{5.12}$. Although a higher stress of 6.24 ksi was developed on the top flange of girder number 2, the girder spacing of 2 ft. at this location would yield a distribution factor of $\frac{S}{5.88}$. Considering the remaining lower stresses obtained in the investigation, the denominator of the formula would be larger.

For 5 in. thick timber decked bridges like the SS-4 standard investigated, these data suggest that a distribution factor for the interior girders of $\frac{S}{5}$ would be adequate for legal load limits. Noting that the bridge design specifications[7] generally allow for a decrease in the distribution factor (increased denominator in the distribution factor) for strip and multiple layer flooring, this finding could be related to the 5 in. floor thickness on the study bridge.

Load Distribution to Exterior Girders

For the exterior girders the load distribution factor determined by proportioning the load as the reaction of a simple beam between the exterior and first interior girder was found to be inadequate in some instances. Specifically, the procedure used to evaluate the exterior girders calls for positioning the resultant of the wheel line 18 in. from the curb, as shown in Figure 15, which corresponds to the study loading position number 2 with the nearest tire tread 7-1/2 in. from the curb. Using the simple beam assumption, a distribution factor of $\frac{7.5}{24}$ or 0.315, would be obtained.

The maximum stress in the exterior girder was found to be 6.29 ksi (Figure 12), which is equivalent to a moment of 880.39 in.-kips. Proportioning this moment to the maximum live load moment of 2,567 in.-kips results in a distribution factor of 0.343. Therefore the load distribution factor obtained by current procedures is too low by approximately 9%. When the resultant of the wheel line is positioned at 19-1/2 in. from the curb, the simple beam distribution factor is 15% lower than that obtained from the load tests. As the load is positioned further from the curb and closer to the first interior girder the distribution factor calculated by this procedure would approach zero.

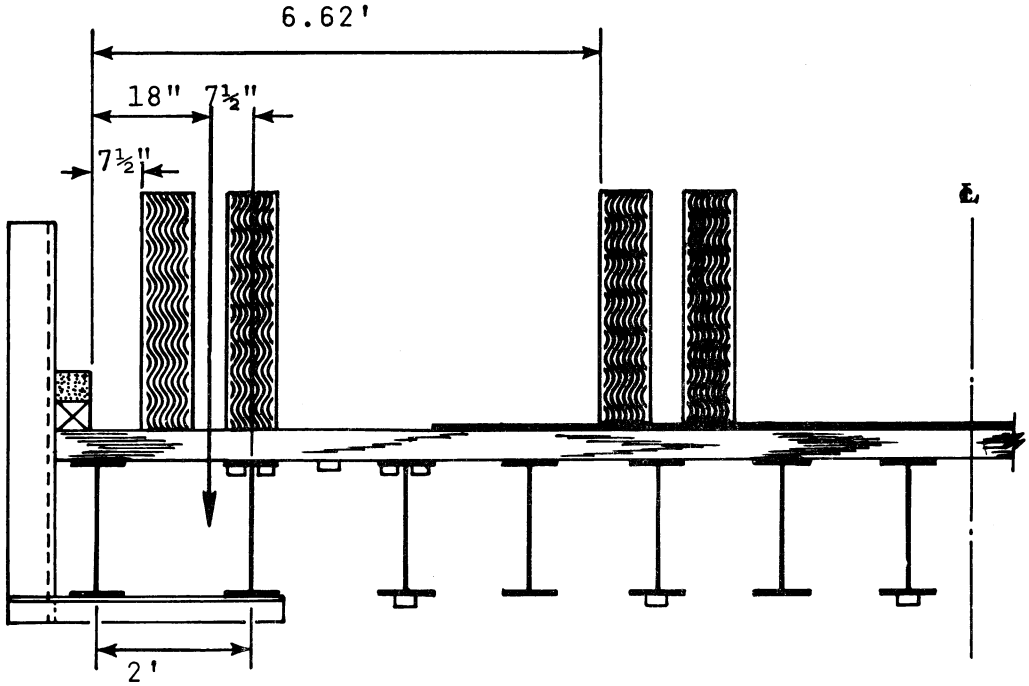

Figure 15. Wheel line positioned such that the resultant would be 18 in. from the face of the curb. (This is equivalent to loading position number 2, Table 1.)

If it is assumed that the load is positioned at 2 in. from the curb (study position number 1), the resultant of the wheel line would be midway between the exterior and the first interior girder. Using the simple beam procedure, a distribution factor of 0.50 would be obtained. The maximum stress for this loading position was 7.22 ksi — giving a moment of 1,010.22 in.-kips, which can be proportioned to the 2,567 in.-kips to obtain an actual distribution factor of 0.39. Therefore the simple beam distribution is conservative for the 2 in. truck loading position. Obviously, the choice of the loading position to be used for the distribution factor calculation is critical as it will determine whether the results are conservative or liberal.

In the foregoing examples, moving the loading position from 7½ in. to 2 in. from the curb changes the calculated simple beam distribution factor from 0.315 to 0.50, and the actual from 0.343 to 0.39. Thus, for the 5½ in. lateral change of wheel load position the actual change in load distribution was much less than that calculated by the simple beam reaction procedure. Noting that the 0.343 and 0.39 factors can be transformed respectively to $\frac{S}{5.83}$ and $\frac{S}{5.08}$, it is apparent that a general distribution factor of $\frac{S}{5}$ could be used. The use of a standard $\frac{S}{5}$ distribution factor for the exterior girders would be more representative of the load distribution variations observed in this investigation.

SUMMARY OF CONCLUSIONS

From the results of the load tests conducted on the Virginia Standard SS-4 timber deck-steel girder bridge described earlier in this report, the following conclusions are indicated.

1. The live load stresses resulting from the truck loading are laterally distributed in much the same manner whether the deck plank fasteners are tight or loose.

2. For similar loading positions the stresses in the girders are nearly the same, in most instances, with the floor fasteners loose or tight. While some girders are actually stressed slightly less for some loading positions when the fasteners are loosened, the maximum increase in stress of 15% was found on girder number 5. The maximum stress on this girder, with the fasteners loose, however, was considerably less than the maximum recorded on other girders with the fasteners tight.

3. For the interior girders the Virginia practice of using a load distribution factor of $\frac{S}{4}$ was found to be conservative in all cases. An analysis of the data obtained from the truck loading investigation suggests that a distribution factor of $\frac{S}{5}$ would be less conservative but adequate for the legal load limits currently allowed. A possible explanation for this result is suggested in the report.

4. For the exterior girders, the Virginia practice of positioning the resultant of a wheel line 18 in. from the curb and distributing the load by proportioning it

as the reaction of a simple beam between the adjacent girders was found to be slightly inadequate. The distribution factor obtained from this procedure was found to be approximately 9% too low. An analysis of other loading positions in the area between the first and second girders suggests that the use of a distribution factor of $\frac{S}{5}$ would be more realistic than the current procedure used on the exterior girders.

It should be noted once again that the foregoing conclusions apply only to bridges that conform to the dimensions of the standard SS-4 wooden plank deck-steel girder bridge with a 5 in. floor thickness. With proper judgment the results could have implications for similar conditions on other wooden deck-steel girder structures.

RECOMMENDATIONS

It is recommended that the Bridge Division and the Structures Research Advisory Committee consider the advisability of using a load distribution factor of $\frac{S}{5}$ for the evaluation of both the interior and exterior girders of timber decked-steel girder bridges which have a nominal 5 in. floor plank thickness and comply with the standard girder spacings designated for these type structures. As compared to current procedures this approach would be less conservative, giving a 20% reduction in live load distribution to the interior girders, and more conservative, with a 28% increase in live load distribution to the exterior girders. If it is deemed desirable to continue using the current load distribution procedures, no further study of these type structures would be required. If consideration is given to changing current practice, however, additional study and/or verification of the results obtained in this investigation may be desirable.

ACKNOWLEDGMENTS

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