BRIDGE FATIGUE DUE TO DAILY TRAFFIC

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This paper presents a summary of loading history studies that have been conducted on girder-slab bridge structures. Characteristics of the bridges, bridge locations, and loadings are examined to present uniform code criteria. A technique that considers random load application is presented for possible design consideration. The method incorporates distribution of truck type, location of road, simple and continuous spans, and probable induced field stresses.

•DURING the past 5 years, various universities and state and federal highway agencies (1, 2, 3, 4, 5, 6, 7, 8, 9, 10) have conducted cooperative field studies to determine the loading history of bridge structures. These tests have provided information on the magnitude of induced bridge girder stresses and the vehicles that induce these stresses.

A thorough study of these data shows that the induced live-load stress ranges are low (1.0 to 3.0 ksi) in comparison with the design live-load stresses. This then suggests that the present fatigue design criteria may be too restrictive and some revision might be in order. The design guides also should account for the characteristics of the actual vehicles that traverse the bridges, as well as the random nature of the loads.

Therefore, it is the intention of this paper to present one possible method of fatigue design that does consider random load characteristics and actual induced stresses.

VEHICLE CHARACTERISTICS

During the various load history tests, classification counts were made of truck types crossing a bridge structure. If the structure was near a weighing or loadometer station, the gross weight, weight distribution, and axle spacings of the vehicles were also obtained. Data collected during the bridge tests in Alabama (1), Connecticut (2), Minnesota (3), Maryland (4, 5, 6, 7, 8), Michigan (9), and Virginia (10) are given in Tables 1 through 5. During many of these tests only classification counts were made.

The percentage of distribution of trucks for the various bridge tests is given in Table 1. The classifications are based on five truck types (Fig. 1). Although some reports list other truck types, most can be categorized under these five classes. Table 1 also gives the type of road system associated with each test. These data suggest that road systems can be divided into three classifications: metropolitan, urban, and rural. An average of the data for these classifications is given in Table 2. This distribution would then be used, instead of more reliable data, for fatigue analysis.

Table 3 gives the mean gross weights for five truck types and bridge tests. The ranges in gross weights are used to tabulate induced girder moments for each truck type.

Table 4 gives the percentage of the total load distributed to each axle for the various truck types. Only the tests conducted in Maryland and Connecticut provided such information. The average of these values will be used for describing typical vehicles.

Table 5 gives the average spacing between axles for the various truck types. Data were obtained from tests conducted in Virginia, Maryland, and Connecticut.

By using the resulting data from Tables 4 and 5, one can develop typical trucks

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Table 1. Percentage of distribution of trucks by test site.

Test Site	Truck Type 2D	3	2S-1	2S-2	3S-2	Total Trucks per 24 Hours	Classification[a]
Ala. 1	26.3	14.3	4.4	21.2	32.8	570	Rural, S
Ala. 2	23.2	5.3	3.3	15.4	52.8	1,090	Rural, I
Conn.	20.4	3.4	3.95	30.9	41.7	5,416	Urban, I
Minn.	31.0	27.1	2.5	5.8	33.6	2,551	Metropolitan, I
Md. 1	17.2	1.3	7.9	29.0	44.6	782	Rural, I
Md. 2	15.3	1.5	7.2	30.0	46.0	940	Rural, I
Md. 3	38.1	19.6	8.9	15.6	17.8	1,528	Metropolitan, I
Md. 4	19.8	5.9	6.7	21.3	46.3	1,468	Rural, S
Md. 5	26.8	6.2	5.6	20.8	40.6	542	Rural, S
Md. 6	27	11	5	18	39	925	Rural, S
Mich. 1	15	2.1	16.8	43.6	22.5	600	Rural, S
Mich. 5	11.9	2.2	17.5	34.6	33.8	690	Urban, S
Mich. 7	12.9	7.5	17.1	30.7	31.8	972	Urban, S
Va. 1	9.3	1.4	7.4	29.7	52.2	1,430	Urban, I
Va. 2	10.9	1.0	5.6	18.9	63.7	870	Urban, I

[a]S = state route, and I = Interstate.

Table 2. Percentage of distribution of trucks by road type.

Truck Type	Road Type Metropolitan	Urban	Rural
2D	35.0	13.0	21.0
3	23.0	3.0	6.0
2S-1	6.0	10.0	7.0
2S-2	11.0	30.0	25.0
3S-2	25.0	44.0	41.0

Figure 1. Truck types.

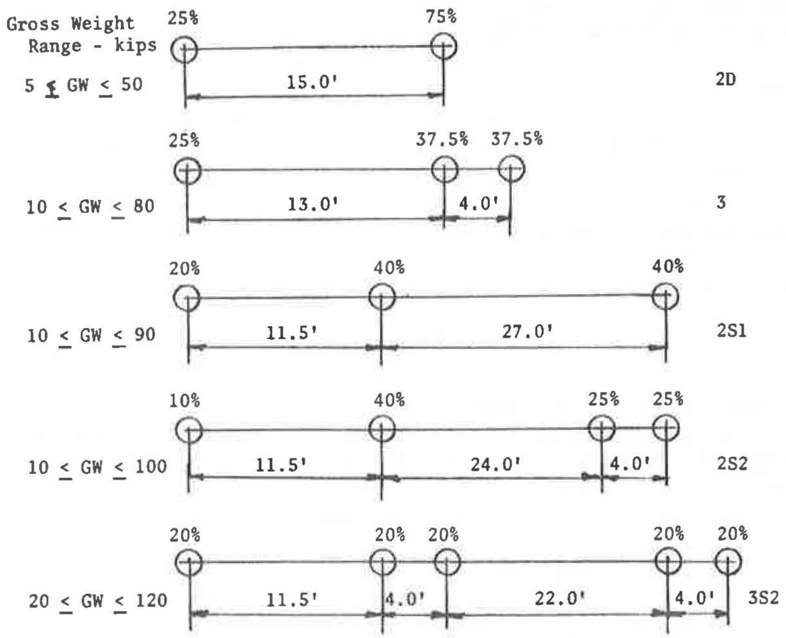

Table 3. Vehicle gross weight and standard deviation.

Item	Truck Type 2D	3	2S-1	2S-2	3S-2
Test site					
Va.					
Mean G	13.1	22.4	29.7	38.5	54.9
S	5.51	9.91	15.93	9.86	13.96
Mich.					
Mean G	15.0	—	36.6	37.0	48.7
S	5.0	—	9.7	13.0	13.7
Md. 1					
Mean G	14.7	32.4	31.4	43.2	56.6
S	6.32	12.89	9.67	15.19	19.79
Md. 2					
Mean G	13.0	48.0	29.8	38.0	53.0
S	—	—	—	—	—
Conn.					
Mean G	15.7	38.4	54.7	45.8	48.2
S	—	—	—	—	—
Average					
Mean G	14.3	35.3	36.1	40.5	52.3
S	5.6	11.4	11.8	12.7	15.8
Range	3 ≤ G ≤ 25.5	12 ≤ G ≤ 58.1	12 ≤ G ≤ 59.7	15 ≤ G ≤ 65.9	21 ≤ G ≤ 83.9

Note: G = gross weight, kips; and S = standard deviation.

(Fig. 1). This is necessary to determine probable induced girder moments caused by the five truck types.

SIMPLE SPAN STUDY

Relation of Induced to Design Maximum Moments

To relate the probable induced stresses from various truck types to the design stresses, relationships between induced maximum moments (M) and AASHTO live-load design maximum moments (M_A) have been calculated and tabulated. These values are given in Table 6 as a ratio of M/M_A for various truck types and the corresponding range in gross weights for the typical truck types. The length of spans ranges from 40.0 to 140.0 ft.

Reduction Factor

As noted previously, the observed stresses during the field tests were less than the design live-load stress. This is partly because of the truck types that induce these stresses as opposed to the standard AASHTO HS-20-44 truck and partly because of differences in load distribution, material properties, and the unaccounted-for contributions of automobiles and parapets. With this observation in mind, a study was conducted (11) to relate vehicle characteristics to corresponding induced stresses and bridge stiffness. The results of this study provided the following general equation:

$$(f_r)_{test} = \frac{A + B(G)}{S/12L} \tag{1}$$

where

$(f_r)_{test}$ = observed induced stress range at (a) centerline of the girder and (b) off the end of the cover plate,
G = vehicle gross weight,
S = girder section modulus,
L = span length in feet, and
A, B = constants obtained from linear regression analysis.

Centerline of Girder—A relationship between the induced stresses and design stresses for the centerline of the girder can be given by the following equation:

$$\frac{f_{test}}{f_{design}} = \frac{A + B(G)}{S/12L} \times \frac{S}{M\left(\frac{\overline{S}}{5.5}\right)(1 + I)} \tag{2}$$

where

M = calculated absolute maximum moment caused by a set of wheel loads of an AASHTO truck,
$\overline{S}/5.5$ = distribution factor,
$\overline{S}$ = girder spacing, and
I = impact factor = $\left(\frac{50}{L + 125}\right)$.

The AASHTO moment can be computed by M = (108L - 1,680) kip-in., where L = feet. Defining $\frac{f_{test}}{f_{design}}$ = F and substituting in the M equation gives

$$F = \frac{[A + B(G)]\ 12L}{(108L - 1{,}680)\left(\frac{\overline{S}}{5.5}\right)(1 + I)} \tag{3}$$

Assuming that several trucks can occur on the bridge at the same time during the field

Table 4. Percentage of weight distribution by axle.

Truck Type	Axle A Md.	Axle A Conn.	Axle B Md.	Axle B Conn.	Axle C Md.	Axle C Conn.	Average A	Average B	Average C
2D	25	41	75	59	—	—	33	67	—
3	25	33	75	67	—	—	29	71	—
2S-1	20	27	40	40	40	33	24	40	36
2S-2	10	19	40	36	50	45	14	38	48
3S-2	20	18	40	42	40	40	19	41	40

Table 5. Vehicle axle spacing.

Truck Type	Span A to B Va.	Span A to B Md.	Span A to B Conn.	Span B to C Va.	Span B to C Md.	Span B to C Conn.	Average A to B	Average B to C
2D	14	16	15.7	–	–	–	15.2	–
3	14	18	19.1	–	–	–	(13 + 4) or 17.0	–
2S-1	11.0	12	11.8	29.0	28.0	23.9	11.6	27
2S-2	11.0	12	12.1	27.0	28.0	28.7	11.7	(24 + 3.9) or 27.9
3S-2	12.0	12	11.1	30.0	30.0	33.3	11.7	(4.1 + 22 + 4) or 30.1

Table 6. M/M_A for a simple span bridge.

Truck Type	Gross Weight (kips)	Length (ft) 40	60	80	100	120	140
2D	20	0.365	0.327	0.312	0.304	0.299	0.296
	40	0.730	0.654	0.624	0.608	0.598	0.591
	60	1.096	0.981	0.936	0.912	0.897	0.887
	80	1.461	1.308	1.248	1.216	1.196	1.183
3	20	0.340	0.313	0.303	0.297	0.293	0.291
	40	0.680	0.627	0.606	0.594	0.587	0.582
	60	1.020	0.940	0.908	0.891	0.880	0.873
	80	1.360	1.254	1.211	1.188	1.174	1.164
2S-1	15	0.164	0.163	0.176	0.183	0.188	0.191
	30	0.327	0.326	0.352	0.367	0.376	0.382
	45	0.491	0.488	0.528	0.550	0.564	0.573
	60	0.654	0.651	0.704	0.734	0.752	0.765
	75	0.818	0.814	0.881	0.917	0.940	0.956
	90	0.982	0.977	1.057	1.100	1.128	1.147
2S-2	20	0.201	0.212	0.230	0.240	0.247	0.251
	40	0.401	0.424	0.460	0.480	0.493	0.503
	60	0.602	0.636	0.690	0.720	0.740	0.754
	80	0.803	0.847	0.919	0.960	0.987	1.006
	100	1.003	1.059	1.149	1.200	1.234	1.257
3S-2	20	0.199	0.208	0.229	0.241	0.248	0.252
	40	0.398	0.416	0.459	0.481	0.495	0.505
	60	0.597	0.624	0.688	0.722	0.743	0.757
	80	0.796	0.831	0.917	0.962	0.991	1.010
	100	0.995	1.040	1.146	1.203	1.238	1.262
	120	1.193	1.248	1.375	1.443	1.486	1.515

Table 7. Simple span reduction factor for F-truck types 2D and 3 at center span.

Length (ft)	Gross Weight (kips) 10	20	40	60	80
40	0.0714	0.1171	0.2087	0.3002	0.3917
60	0.0604	0.0991	0.1766	0.2541	0.3315
80	0.0567	0.0931	0.1658	0.2386	0.3113
100	0.0551	0.0938	0.1610	0.2316	0.3022
120	0.0542	0.0890	0.1586	0.2281	0.2977
140	0.0538	0.0883	0.1573	0.2262	0.2952

Table 8. Simple span reduction factor for F-truck types 2S-1, 2S-2, and 3S-2 at center span.

Length (ft)	Gross Weight (kips) 20	40	60	80	100	120
40	0.0989	0.1512	0.2035	0.2558	0.3081	0.3604
60	0.0837	0.1279	0.1722	0.2164	0.2607	0.3050
80	0.0786	0.1201	0.1617	0.2033	0.2448	0.2864
100	0.0763	0.1166	0.1570	0.1973	0.2377	0.2780
120	0.0713	0.1149	0.1546	0.1943	0.2341	0.2738
140	0.0745	0.1139	0.1533	0.1928	0.2322	0.2716

tests, the actual stresses are increased by ($\bar{S}/5.5$), thus increasing the reduction factor F, which gives

$$F = \frac{[A + B(G)]\,L}{(9L - 140)\left(1 + \frac{50}{L + 125}\right)} \qquad (4)$$

The coefficients A and B in Eq. 2 are obtained from an empirical equation, which depends on the five truck types. A close examination of these five equations (12) indicates that two equations can readily represent the response of the bridge to the five truck types. The final equation for types 2D and 3 is

$$F = \frac{[0.1835 + 0.0328\,(G)]\,L}{(9L - 140)\left(1 + \frac{50}{L + 125}\right)} \qquad (5)$$

For truck types 2S-1, 2S-2, and 3S-2, the equation is

$$F = \frac{[0.3338 + 0.01874(G)]\,L}{(9L - 140)\left(1 + \frac{50}{L + 125}\right)} \qquad (6)$$

The reduction factor F (Eqs. 3 and 4) is given in Tables 7 and 8 for various span lengths and gross weights. The factors give the ratios of the observed stresses to the design stresses for simple span, composite girder-slab bridges.

Off End of Cover Plate—As described for the ratio of f_{test} to f_{design} at the centerline of the structure, a similar ratio can be developed at the end of the cover plate:

$$\frac{f_{test}}{f_{design}} = \frac{A + B(G)}{S/(12L)} \times \frac{S}{M\left(\frac{\bar{S}}{5.5}\right)(1 + I)}$$

where

M = calculated moment at end of cover plate caused by a set of wheel loads of an AASHTO truck,
$\bar{S}/5.5$ = distribution factor,
$\bar{S}$ = girder spacing, and
I = impact factor = $\left(\frac{50}{L + 125}\right)$.

The AASHTO moment is determined by M_{16} kips = $16\left[(1 - N)\left(\frac{L}{2}\right)(1 + N) - 7\right]12$ kip-in. This equation was developed by positioning a set of wheel loads on an influence line diagram for moment at the cover plate of the beam shown in Figure 2. The equation for moment only contains the effect of two wheels (P = 16 kips) spaced at 14.0 ft. The 4-kip axle was assumed to be off the structure. As shown in Figure 3, the ratio of cover plate length to span length (N) is compared to span length. The limiting value of N so that the 4-kip axle remains on the girder is shown by the bound line. A plot of the data for the test bridges is also given. As can be seen, most of the bridges fall beyond the limiting N value; thus, the 4-kip load can be neglected. If the 4-kip load is to be included, the additional moment is given by the following equation: M_4 kips = 12(1 + N) [L(1 - N) - 28] kip-in., which can then be added to the previously defined M_{16} kip equation.

Defining $F = \frac{f_{test}}{f_{design}}$ and substituting in the M_{16} kip equation into M of the general equation give

$$F = \frac{[A + B(G)]\,12L}{16\left[(1 - N)\,\frac{L}{2}(1 + N) - 7\right]12\left(\frac{\bar{S}}{5.5}\right)(1 + I)} \qquad (7)$$

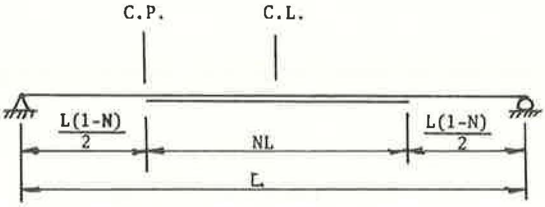

Figure 2. Cover-plated, simple span structure.

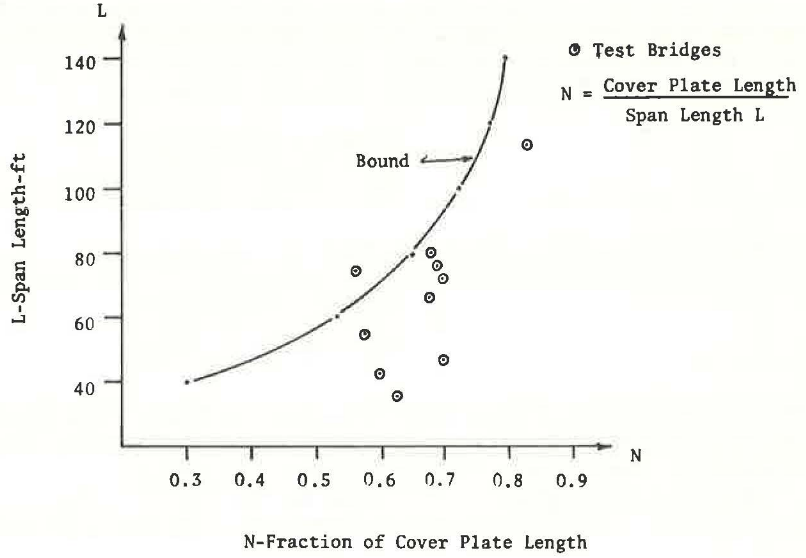

Figure 3. Span length versus fraction of cover plate length N.

Table 9. Simple span reduction factor for F-truck types 2D and 3 at cover plate end.

		Gross Weight (kips)				
N	Length (ft)	10	20	40	60	80
0.50	40	0.0537	0.0954	0.1788	0.2623	0.3457
0.60	40	0.0618	0.1097	0.2057	0.3016	0.3975
0.70	40	0.0763	0.1355	0.2539	0.3724	0.4908
0.55	60	0.0534	0.0950	0.1780	0.2611	0.3441
0.60	60	0.0580	0.1030	0.1930	0.2830	0.3730
0.65	60	0.0639	0.1135	0.2128	0.3120	0.4112
0.65	80	0.0628	0.1116	0.2091	0.3067	0.4042
0.70	80	0.0707	0.1256	0.2355	0.3453	0.4551
0.75	80	0.0822	0.1460	0.2736	0.4012	0.5288
0.75	100	0.0818	0.1453	0.2724	0.3994	0.5265
0.80	100	0.0992	0.1762	0.3302	0.4843	0.6383
0.85	100	0.1284	0.2281	0.4274	0.6268	0.8262
0.80	120	0.0993	0.1764	0.3306	0.4847	0.6389
0.85	120	0.1286	0.2284	0.4280	0.6277	0.8273
0.90	120	0.1874	0.3330	0.6240	0.9151	1.2060

Table 10. Simple span reduction factor for F-truck types 2S-1, 2S-2, and 3S-2 at cover plate end.

		Gross Weight (kips)					
N	Length (ft)	20	40	60	80	100	120
0.50	40	0.0713	0.1223	0.1734	0.2244	0.2755	0.3265
0.60	40	0.0819	0.1407	0.1994	0.2581	0.3168	0.3755
0.70	40	0.1012	0.1736	0.2461	0.3186	0.3911	0.4636
0.55	60	0.0709	0.1218	0.1726	0.2234	0.2742	0.3250
0.60	60	0.0769	0.1320	0.1870	0.2421	0.2972	0.3523
0.65	60	0.0848	0.1455	0.2062	0.2670	0.3277	0.3884
0.65	80	0.0833	0.1430	0.2027	0.2624	0.3221	0.3818
0.70	80	0.0938	0.1610	0.2282	0.2955	0.3627	0.4299
0.75	80	0.1090	0.1871	0.2652	0.3433	0.4214	0.4995
0.75	100	0.1085	0.1863	0.2640	0.3418	0.4195	0.4973
0.80	100	0.1316	0.2258	0.3201	0.4144	0.5086	0.6029
0.85	100	0.1703	0.2923	0.4143	0.5363	0.6583	0.7803
0.80	120	0.1317	0.2261	0.3204	0.4148	0.5091	0.6035
0.85	120	0.1705	0.2927	0.4149	0.5371	0.5493	0.7814
0.90	120	0.2486	0.4268	0.6049	0.7830	0.9612	1.1390

Assume again that several trucks occur simultaneously on the bridge during the field tests. This increases the stresses by an assumed amount of ($\overline{S}$/5.5). The reduction factor is, therefore,

$$F = \frac{[A + B(G)]\ L}{16\left[(1 - N)\left(\frac{L}{2}\right)(1 + N) - 7\right]\left(1 + \frac{50}{L + 125}\right)} \quad (8)$$

A and B in Eq. 5 were obtained from a study of loading history field data (12). Data indicate that coefficients that represent the behavior of the five truck types can be reduced into two categories:

$$F = \frac{[0.0720 + 0.025(G)]\ L}{16\left[(1 - N)\left(\frac{L}{2}\right)(1 + N) - 7\right]\left(1 + \frac{50}{L + 125}\right)} \quad (9)$$

for truck types 2D and 3, and

$$F = \frac{[0.1211 + 0.0153(G)]\ L}{16\left[(1 - N)\left(\frac{L}{2}\right)(1 + N) - 7\right]\left(1 + \frac{50}{L + 125}\right)} \quad (10)$$

for truck types 2S-1, 2S-2, and 3S-2. The reduction factor F (Eqs. 6 and 7) is given in Tables 9 and 10 for various span lengths, fraction of cover plate length, and gross weights.

CONTINUOUS SPAN STUDY

M/M_A Three-Span Structure

By using the typical trucks shown in Figure 1, the maximum moments induced by these vehicles on a symmetrical two-span structure were determined. The moments in question were located at the interior support (point 1) and the midspan (point 2) of a two-span structure of length 2L. The values were then related to the AASHTO design moments as R_1 (M/M_A at interior support) and R_2 (M/M_A at midspan). The gross weights for these various truck types were assumed to be equal to the maximum values and are (a) for 2D, 50.0 kips; (b) for 3, 80.0 kips; (c) for 2S-1, 80.0 kips; (d) for 2S-2, 100.0 kips; and (e) for 3S-2, 100.0 kips. The resulting ratios for the five truck types are given in Table 11.

M/M_A Three-Span Structure

By using a similar procedure to that for a two-span structure, critical moments were evaluated in various three-span structures. The locations of the critical moments were selected at midspan of the end span (point 1, R_1); interior support (point 2, R_2); and midspan of the center span (point 3, R_3). M/M_A of the induced moments for these three points, according to the typical trucks and AASHTO loadings, are given in Tables 12 through 16 for the various truck types.

Tables 12 through 16 also give the various span lengths and the proportions of end spans to the center span. The classification of gross weights of the vehicles was assumed to be the same maximum values as those given previously.

DESIGN CRITERIA

In a general design of a bridge girder, fatigue analysis is performed after the section has been determined according to static dead- and live-load stress conditions. This fatigue analysis is based on a predetermined number of induced load applications (i.e., 100,000, 500,000, or 2,000,000 cycles) at a maximum induced stress obtained from the AASHTO truck loading. It is probably unrealistic to penalize the structure with absorption of these high stresses when it is known that the actual induced stresses

Table 11. M/M$_A$ for two-span bridge.

Truck Type	Spans (ft) 80 R_1	80 R_2	100 R_1	100 R_2	120 R_1	120 R_2	140 R_1	140 R_2
2D	0.474	0.812	0.412	0.785	0.364	0.768	0.325	0.757
3	0.757	1.248	0.658	1.218	0.581	1.198	0.520	1.185
2S-1	0.652	0.919	0.600	0.956	0.545	0.981	0.496	1.000
2S-2	0.835	1.135	0.761	1.193	0.689	1.229	0.625	1.254
3S-2	0.811	1.121	0.747	1.177	0.680	1.214	0.619	1.240

Table 12. M/M$_A$ for three-span bridge.

Truck Type	Midspan Length (ft)	N	R_1	R_2	R_3
2D	80	0.6	0.872	0.709	0.825
	80	0.8	0.814	0.554	0.819
	80	1.0	0.785	0.537	0.814
	100	0.6	0.824	0.623	0.795
	100	0.8	0.785	0.485	0.791
	100	1.0	0.764	0.468	0.787
	120	0.6	0.797	0.554	0.776
	120	0.8	0.767	0.430	0.772
	120	1.0	0.751	0.413	0.770
	140	0.6	0.779	0.499	0.764
	140	0.8	0.755	0.386	0.760
	140	1.0	0.742	0.370	0.758

Table 13. M/M$_A$ for three-span bridge, truck type 3.

Truck Type	Midspan Length (ft)	N	R_1	R_2	R_3
3	80	0.6	1.313	1.132	1.257
	80	0.8	1.249	0.883	1.251
	80	1.0	1.216	0.857	1.247
	100	0.6	1.261	0.995	1.226
	100	0.8	1.216	0.774	1.221
	100	1.0	1.192	0.747	1.217
	120	0.6	1.230	0.886	1.205
	120	0.8	1.196	0.687	1.092
	120	1.0	1.177	0.661	1.198
	140	0.6	1.210	0.797	1.191
	140	0.8	1.182	0.617	1.188
	140	1.0	1.167	0.592	1.185

Table 14. M/M$_A$ for three-span bridge, truck type 2S-1.

Truck Type	Midspan Length (ft)	N	R_1	R_2	R_3
2S-1	80	0.6	0.227	0.935	0.908
	80	0.8	0.863	0.734	0.911
	80	1.0	0.920	0.741	0.913
	100	0.6	0.843	0.884	0.942
	100	0.8	0.921	0.690	0.946
	100	1.0	0.962	0.682	0.949
	120	0.6	0.897	0.817	0.567
	120	0.8	0.957	0.635	0.883
	120	1.0	0.989	0.620	0.974
	140	0.6	0.933	0.752	0.986
	140	0.8	0.981	0.583	0.990
	140	1.0	1.007	0.565	0.993

Table 15. M/M$_A$ for three-span bridge, truck type 2S-2.

Truck Type	Midspan Length (ft)	N	R_1	R_2	R_3
2S-2	80	0.6	0.938	1.221	1.086
	80	0.8	1.027	0.956	1.090
	80	1.0	1.112	0.944	1.093
	100	0.6	0.995	1.133	1.136
	100	0.8	1.112	0.884	1.142
	100	1.0	1.173	0.863	1.147
	120	0.6	1.075	1.038	1.174
	120	0.8	1.164	0.807	1.073
	120	1.0	1.211	0.782	1.185
	140	0.6	1.128	0.951	1.202
	140	0.8	1.199	0.737	1.208
	140	1.0	1.238	0.711	1.213

Table 16. M/M$_A$ for three-span bridge, truck type 3S-2.

Truck Type	Midspan Length (ft)	N	R_1	R_2	R_3
3S-2	80	0.6	0.957	1.168	1.083
	80	0.8	1.035	0.916	1.091
	80	1.0	1.115	0.921	1.197
	100	0.6	1.010	1.104	1.142
	100	0.8	1.118	0.862	1.150
	100	1.0	1.175	0.850	1.156
	120	0.6	1.085	1.020	1.182
	120	0.8	1.169	0.793	1.081
	120	1.0	1.214	0.774	1.195
	140	0.6	1.137	0.939	1.211
	140	0.8	1.203	0.728	1.218
	140	1.0	1.241	0.705	1.223

Table 17. Truck distribution on three-span bridge.

Truck Type	Frequency (percent)	Trucks/Day	Trucks/Year
2D	22	220	80,500
3	10	100	36,500
2S-1	8	80	29,200
2S-2	15	150	54,800
3S-2	45	450	164,000

are much lower. The stresses that are induced also depend on the traffic characteristics; therefore, a random loading criterion appears more suitable.

Miner's Technique

Incorporation of various vehicle loading conditions, a random process, easily can be established by application of Miner's theory (13). The theory is expressed by

$$\sum\left(\frac{n_i}{N_{f_i}}\right) = 1.0 \tag{11}$$

where

n_i = number of induced cycles at a constant stress level f_i, and
N_{f_i} = number of induced cycles to institute failure at stress level f_i.

The estimated life of a structural element is then determined by solving Eq. 11:

$$N_{life} = \frac{1.0}{\sum\left(\frac{n_i}{N_{f_i}}\right)} \tag{12}$$

where $\sum\left(\frac{n_i}{N_{f_i}}\right)$ represents the damage estimate that is induced during 1 year. Therefore, a bridge could be designed in terms of years rather than cycles.

The following is the procedure for checking the fatigue life of a structural weldment:

1. Determine the probable number of trucks per day at a bridge location.
2. Determine the probable percentage of distribution of truck types at a bridge location (use Table 2 if traffic data are not available).
3. Determine the number of vehicle applications per year, i.e., percentage × daily population × 365.
4. Determine M/M_A in regard to the type of structure, vehicle type, and vehicle gross weight (Tables 5 and 10 through 15).
5. Modify AASHTO design live-load stress according to percentage of M/M_A—$f_i = f_{design} \times M/M_A$—for each truck type.
6. Determine the failure cycles at the induced stress f_i for the given truck type, where the failure cycles N_{f_i} are computed from the following equations (11):

$$\log N_{f_i} = 8.87 - 2.65 \log f_i \tag{13}$$

for cover-plated beams, and

$$\log N_{f_i} = 10.637 - 2.94 \log f_i \tag{14}$$

for plain and butt-welded beams.

7. Compute the estimated life of structure in relation to each of the five truck types by using Eq. 12.
8. Determine whether this estimated life is satisfactory.

Root-Mean-Square Technique

As an alternate to Miner's procedure, the influence of the five truck types can be combined into one common denominator by evaluating the root-mean-square (rms) of their stresses (14). This stress is then used to evaluate the fatigue life.

The general equation used for determining rms stress is

$$f_{rms} = \{[(f_{2D})^2 + (f_3)^2 + (f_{2S-1})^2 + (f_{2S-2})^2 + (f_{3S-2})^2]/5.0\}^{1/2} \tag{15}$$

where f_{2D}, f_3, f_{2S-1}, f_{2S-2}, and f_{3S-2} = stresses induced by 2D, 3, 2S-1, 2S-2, and 3S-2 truck types respectively.

The following procedure is used for checking the fatigue life of a structural weldment with the rms technique:

1. Determine the probable number of trucks per day at bridge location.
2. Determine the number of vehicle applications per year, i.e., daily population (DP) × 365.
3. Determine M/M_A in regard to the type of structure, vehicle type, and vehicle gross weight (Tables 5 and 10 through 15).
4. Modify AASHTO design live-load stress according to percentage of M/M_A—$f_i = (f)_{design} \times M/M_A$—for each truck type.
5. Use computed stresses f_i, for each of the five truck types, to compute f_{rms} as given by Eq. 15.
6. Determine failure cycles N_f at the f_{rms} stress level by using Eqs. 13 and 14.
7. Compute estimated life of structure, i.e., $N_{years} = \dfrac{N_{f_{cycles}}}{365 \times DP}$.
8. Determine whether this estimated life is satisfactory.

The reduction factors given in Tables 7 through 10 were not listed as part of the design procedure. These factors can be used with modifying factors to obtain a more realistic estimate of the actual induced stress. Therefore, the final stress f_i is computed as

$$f_i = f_{design} \times M/M_A \times F \tag{16}$$

If a conservative estimate is required, F = 1.0 would be used.

APPLICATION

Examination of truck classification data (5, 6) and a load history study of a three-span continuous bridge yielded the truck distributions given in Table 17. (The average daily traffic was 1,000 trucks. The values for the number of trucks per day times 365 equals the trucks per year.)

The bridge to be examined under these loadings (Table 17) has three spans: 72, 90, and 72 ft long (Fig. 4). The bridge is composite in the positive moment region and has a 7-in. concrete slab. The girders are spaced at 7-ft, 7-in. intervals. The section properties of a typical interior girder at sections A, B, and C and the design live-load moments and stresses are given in Table 18. With this information, the induced stresses caused by the five truck types can now be determined.

Tables 12 through 16 are used. It will be assumed that the center span is 100 ft (90 ft actually) and the end span ratio equals 72/90 or 0.80.

The induced stresses caused by the five truck types are computed by multiplying the design stresses by the M/M_A factors. These stresses must also reflect the passage of a single vehicle rather than all lanes loaded as is assumed in the original design. This can be achieved by using a new distribution factor of S/11.0 (15). Thus a ratio of S/11.0 to the AASHTO distribution factor S/5.5 gives a factor of 0.50. Therefore, the resulting stresses caused by the various truck types (Table 19) are computed as

$$\text{Stress}_{\text{truck type}} = (\text{design stress})\left(\frac{M}{M_A}\right)\left(\frac{S/11.0}{S/5.5}\right)$$

The rms stress for R_1 = 5.95, for R_2 = 3.37, and for R_3 = 6.15.

The fatigue life of a plain or butt-welded girder subjected to these stresses is obtained by Eq. 14. The stresses at midspan of the center girder will govern. The cycles to failure ($N_f \times 10^6$) are given in Table 20 (rms = 209.2). These resulting N_f values and the frequencies of applied stresses per truck type will now be used to determine the estimated failure life with Miner's and the rms techniques.

Miner's Technique

The damage index is n_i/N_{f_i} and is given in Table 21. The estimated life is the reverse of n_i/N_{f_i} or

Figure 4. Three-span continuous bridge.

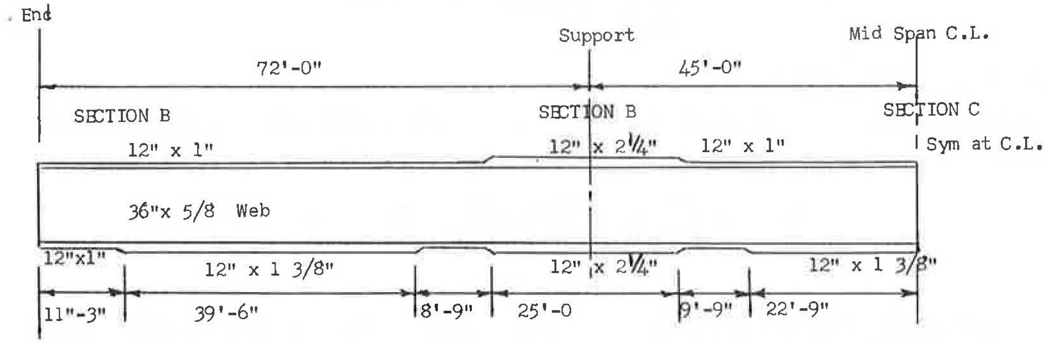

Table 18. Interior girder descriptions for a three-span continuous bridge.

Section	Type	Section Modulus (in.^{3})	Design Moment (kip-ft)	Stress (ksi)
A, support	Noncontinuous	849.0	-635.0	-9.0
B, endspan	Continuous	739.0	690.0	11.37
C, midspan	Continuous	739.0	716.0	11.59

Table 19. Stresses caused by various truck types.

	Stress (ksi)		
Truck Type	R_1 (end span)	R_2 (support)	R_3 (midspan)
2D	4.47	2.67	4.57
3	6.95	3.48	7.05
2S-1	5.25	3.10	5.50
2S-2	6.30	3.97	6.60
3S-2	6.35	3.89	6.65

Table 20. Cycles to failure.

Truck Type	Cycles to Failure at R_3 (midspan)
2D	501.0
3	140.1
2S-1	290.6
2S-2	170.1
3S-2	166.3

Table 21. Damage index.

Truck Type	$n_i \times 10^3$	n_i/N_f
2D	80.5	0.000161
3	36.5	0.000261
2S-1	29.2	0.000100
2S-2	54.8	0.000320
3S-3	164.0	0.000990

$$N_{life} = \frac{1}{\sum \frac{n_i}{N_{f_i}}} = \frac{1}{0.00183} = 546 \text{ years}$$

Root-Mean-Square Technique

The estimated bridge life is computed from the rms failure life of $N_f = 209.2 \times 10^6$ cycles:

$$N_{life} = \frac{N_f}{365 \times DP} = \frac{209.2 \times 10^6}{365 \times 1{,}000} = 574 \text{ years}$$

CONCLUSION

A methodology has been presented by which random truck loading on a bridge can be considered relative to the fatigue response of welded plate elements.

Examination of field data, as reported by various states, has resulted in a series of typical trucks that were used as loads in evaluating induced girder moments. The induced field stresses were compared to the calculated stresses, and this resulted in the determination of reduction factors. These factors may be used to modify design stresses.

Further studies should be conducted in developing accurate single-vehicle load distribution factors S/11.0. The tables also should be refined to reflect other moment locations along the girders. The suggested fatigue design procedure is derived from field tests on multiple beam and slab bridges and, therefore, should only be used on similar structures.

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