NEW PROCEDURE FOR FATIGUE DESIGN OF HIGHWAY BRIDGE GIRDERS

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The wide variety of heavy truck traffic and bridge girder weld conditions combined with reported low measured stress levels under random traffic suggests there must currently be inconsistent safety margins against bridge fatigue. This paper discusses a probabilistic load model that forecasts histograms of highway bridge loading and that can be used to predict fatigue life and to properly size girder sections. A reliability or risk approach to choosing safety factors on material and load is also described. The simplified design procedure based on the truck loading model permits cross sections to be designed or checked against fatigue by a simple formula that also includes as parameters truck volume, span length, weld category, and location.

•CONSIDERABLE attention has been devoted recently to the possibility of fatigue failure in steel girder highway bridges. Field measurement studies under actual random traffic conditions have been made in several states (1). These tests have shown that in most cases actual stress histories experienced by bridge girders are considerably below the allowable AASHTO code standards. This is partly because fatigue is treated in the AASHTO code as a byproduct of the yield or overload analysis, which requires high distribution factors, all lanes loaded, conservative truck weights and dimensions, and so on.

In introducing higher strength steels, continuous spans, and welding details, fatigue often controls the required girder size. In view of the low measured stresses, modifications in the current specification appear appropriate. The current code also does not distinguish between the wide range encountered in both gross truck weight distributions and annual truck volumes. Although the AASHTO code suggests you may do otherwise, based on traffic and loadometer surveys (2), it does not offer any alternatives. This paper presents a detailed procedure for fatigue life design that incorporates these factors and illustrates it with several examples.

A further indication of the need for design changes is that many specifications are evolving toward a probabilistic basis for choosing safety factors. One common example is separate or split safety factors on load and strength. This paper also illustrates how this can be done for the fatigue design problem. A material safety factor is introduced to account for fatigue life variabilities, and a second safety factor on load is used to account for uncertainties in future load growth and possible errors in analysis.

FATIGUE LIFE ANALYSIS PROCEDURE

A new fatigue code format that is based on a more realistic evaluation of fatigue loading and material properties and yet can be simplified enough for practical design is discussed. The goal of the fatigue life analysis procedure was to incorporate field measurements, laboratory data, and state records of truck weights and volumes so they could be used for evaluating and predicting girder fatigue life (3).

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In the fatigue prediction calculations (3), it was found that two truck types, including single and tractor-trailer vehicles, would sufficiently represent all bridge truck loadings. The truck physical parameters were defined from a survey of actual data rather than from extremely unfavorable cases as in the yield design provisions of the AASHTO code. For a particular roadway type and location, a local gross weight distribution and percent by volume of each truck type based on state records were used.

By using a static analysis of the bridge girders, the live-load bending moment range at any point along the girder was found for both truck types. From these static analyses, a computer program calculated the bending moment range histogram for the critical bridge location. The flow chart of fatigue calculations is shown in Figure 1. The bending moment also includes a dynamic impact factor that increases the maximum moment and decreases the minimum value and thereby considerably raises the moment range. This is handled in the program by calculating an envelope of the moment pulse as the truck moves across the bridge. The analytical model of truck loadings also included a truck headway distribution for the important effects of truck loading superpositions caused by closely spaced trucks or trucks passing each other.

These calculated moment histograms have compared favorably to histograms of field measurements. It must be emphasized that the computer program is only used to develop a set of tabulated parameters in a specification, but it is not needed for everyday design or checking cross sections. (This is illustrated below in a simplified design procedure.) The calculations also showed that it was sufficient to consider the fatigue life at some critical location such as the span center on some representative bridge length, and for any other span location and length the results could be directly extrapolated by computing static moment ranges based on a tractor-trailer vehicle. The basic idea is that fatigue damage depends on the stress range experienced by an element and that the tractor-trailer loading is sufficiently representative for comparing locations.

The computed bending moment histogram is then converted to a stress histogram by dividing moment by an equivalent elastic girder section modulus Z_{eq}. The fatigue life is computed by Miner's damage rule in which each stress range level causes damage in inverse proportion to the fatigue life for that stress level. Because only one constant, Z_{eq}, is needed to relate loading to stress, it becomes convenient to determine fatigue life versus Z_{eq} for a given truck volume. Thus, the loading information is contained in the bending moment histogram, whereas Z_{eq} contains all the information on the bridge girder section. This calculation uses the result of Fisher, Frank, Hirt, and McNamee (4) that only live-load stress range and not dead load affects fatigue life. Figure 2 shows a plot of truck volume versus Z_{eq} for different design lives based on a moment histogram calculated from a representative sampling of Ohio truck weight records (5).

The fatigue checking and design method, however, does not require in each case a computer for calculating bending moments and fatigue damage. A literature survey of fatigue information showed that the fatigue S-N curve or stress range versus the number of cycles to failure is a straight line on log-log paper with essentially the same slope regardless of steel or weld type. This assumption permits extrapolating both weld category and a safety or risk factor without further recalculation of bending moments. The weld category is treated by a single term D_f that, like an equivalent stress concentration factor, moves the fatigue curve parallel to itself (Fig. 1). The same holds for the material safety factor N_s, which can be treated as a risk value inasmuch as safety factors greater than 1 correspond to definable risk levels such as $1/_{100}$ or $1/_{1,000}$ of a fatigue failure during the girder lifetime (3). The various factors are summarized in the following equations.

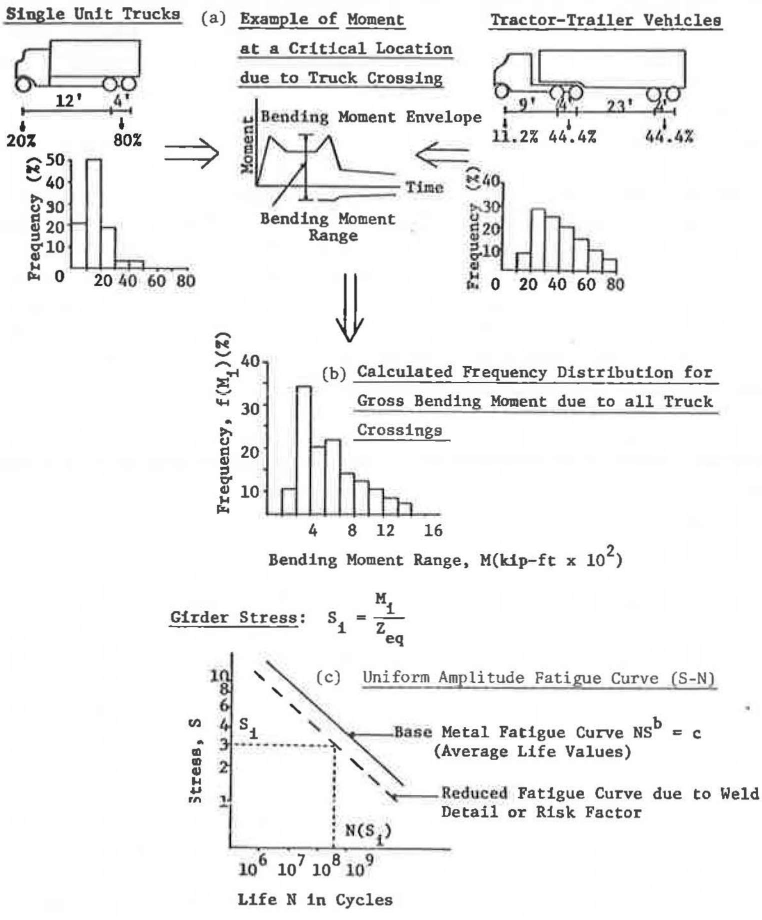
The uniform amplitude fatigue curve is

$$NS^b = c \tag{1}$$

where

N = number of cycles to failure;
S = uniform amplitude stress;

Figure 1. Flow diagram of fatigue evaluation.



(d) Life Prediction: Damage, $D = V\Sigma D_i = \frac{V}{c}\Sigma S_i^b f(M_i) = \frac{V}{Z_{eq}^b c}\Sigma M_i^b f(M_i)$

$\text{Life} = \frac{1}{D}$, years

b = fatigue slope, equal to about 3 in all cases; and
c = constant.

The fatigue damage D_i by Miner's criterion for a single application of stress range S_i is

$$D_i = \frac{1}{N(S_i)} \tag{2}$$

where

$$N(S_i) = \frac{c}{S_i^b}$$

The cumulative damage D may be found from the computed histogram of bending moments:

$$D = \sum D_i = V \sum \frac{f(M_i)}{N(S_i)} = \frac{V}{c} \sum S_i^b\ f(M_i) \tag{3}$$

where

V = truck volume, and
$f(M_i)$ = frequency or percentage of live-load bending moment ranges equal to M_i from the calculated moment histogram.

The stress is assumed to be moment M_i divided by an equivalent elastic section modulus Z_{eq} or

$$S_i = M_i/Z_{eq} \tag{4}$$

Substituting Eqs. 2 and 4 into Eq. 3 gives

$$D = \frac{V}{Z_{eq}^b c} \sum M_i^b\ f(M_i) \tag{5}$$

Failure occurs by Miner's law when D, the cumulative damage, equals 1. Note that the loading terms are incorporated in the moment frequency inside the sum in Eq. 5, whereas the fatigue category, girder properties, and truck volume are outside the sum and can be incorporated in the constant Z_{eq}. [In Moses and Garson (3) Z_{eq} appears as DR (design ratio), i.e., $1/Z_{eq}$.] A general expression for Z_{eq} is

$$Z_{eq} = \frac{ZD_f}{GN_sN_L} \tag{6}$$

The variables in Eq. 6 are as follows:

1. G is the distribution factor expressed as the percentage of the total live-load truck bending moment on the bridge as the truck goes to an individual girder. Based on some reported field measurements of random traffic, G equals about S′/25 (3).
2. S′ is the girder spacing in feet.
3. N_s is the safety or risk factor. A review of fatigue life variabilities in tests showed that a value of 1.75 would seem equal to risks less than one fatigue failure that occurs in the expected life of 10,000 bridges (3). Some results of laboratory fatigue tests tend to show that fatigue life has a log normal distribution. Thus, equal probability fatigue curves (P-S-N lines) would plot on a log-log S-N diagram as a series of straight lines. This enables the risk or safety factor to be reflected as a single constant value S_f at all stress levels.
4. Z is the girder section modulus, in in.^{3}.
5. D_f is the weld category factor that is analogous to a stress concentration (ratio of fatigue curve stress intercept to fatigue intercept for cover plate determination).

Table 1 gives some suggested values for various weld categories based primarily on lab tests (4). Because the value of c in Eq. 1 is also needed to compute fatigue damage, the weld category values are given as ratios of the value for a cover plate termination.

6. N_L is the load factor to account for future truck growth in volume and gross weight distribution as well as any errors in stress analysis and impact factor. A value of 1.5 was used. [Some studies of load growth show mean weights increasing 2.5 percent per year (3).]

Because the value of Z_{eq} is fixed by Eq. 5 for a given loading and fatigue life, a simplified design procedure uses values of Z_{eq} for different roadway categories and locations. For example, combining gross weight data from 12 rural Ohio locations (5) to determine weight distribution for calculating the bending moment histogram gave a value of Z_{eq} equal to 1,700 in.^{3} for a 100-year life, 36 trucks per hour, and the center of a 60-ft simple span girder. On some Maryland weight records (6) Z_{eq} was 1,333 in.^{3} for the same life and bridge. These values of Z_{eq} should be taken only as illustrative because they were based on very selective locations. Further study of weight records will be needed. However, only truck weight distribution must be considered as span; volume and life factor out.

FATIGUE DESIGN PROCEDURE

The following steps are required by the suggested fatigue design procedure.

Select the section modulus value Z_{eq} now designated as $(Z_{eq})_{tab}$ for the roadway category into which the design location is expected to fall. The tabulated values besides the truck weight characteristics are based on specified truck volume, bridge length, impact factor, and location of critical weld. Adjustments to account for these quantities follow.

Adjust the value of Z_{eq} by any change in the desired service life and expected traffic volume (Eq. 5) so that it has the same fatigue damage. The section modulus Z_{eq} and volume V must satisfy the formula

$$\frac{V}{Z_{eq}^{b}} = \text{constant} \tag{7}$$

If V is the truck volume used to calculate the tabulated value of Z_{eq}, then the modified Z_{eq} for another truck volume, V, would be

$$Z_{eq} = (Z_{eq})_{tab} \left(\frac{V}{V_{tab}}\right)^{1/b} \tag{8}$$

where V is the actual truck volume expected at the site, and V_{tab} is fixed (e.g., 36 trucks per hour for a 100-year life).

Select, from a structural analysis, coefficients for the location of the critical cross section and span length, and adjust for any differences in impact factor. These coefficients are

$$Z_{eq} = \frac{(Z_{eq})_{tab}}{C_{XL} C_I} \tag{9}$$

where the variables C_{XL} and C_I are as follows:

1. C_{XL} is the ratio of bending moment range, found by using a tractor-trailer vehicle for the design section, to the bending moment range used in calculating $(Z_{eq})_{tab}$ values. In the examples, this was based on the center of a 60-ft span. Table 2 gives C_{XL} values for various simple span lengths and locations.

2. C_I is based on the ratio of the expected impact factor to the value used to calculate the $(Z_{eq})_{tab}$ values. The impact factor used to calculate the damage was arbitrarily taken as 20 percent. Thus, if any other impact value I is used then

Figure 2. Z_{eq} versus truck volume and design life.

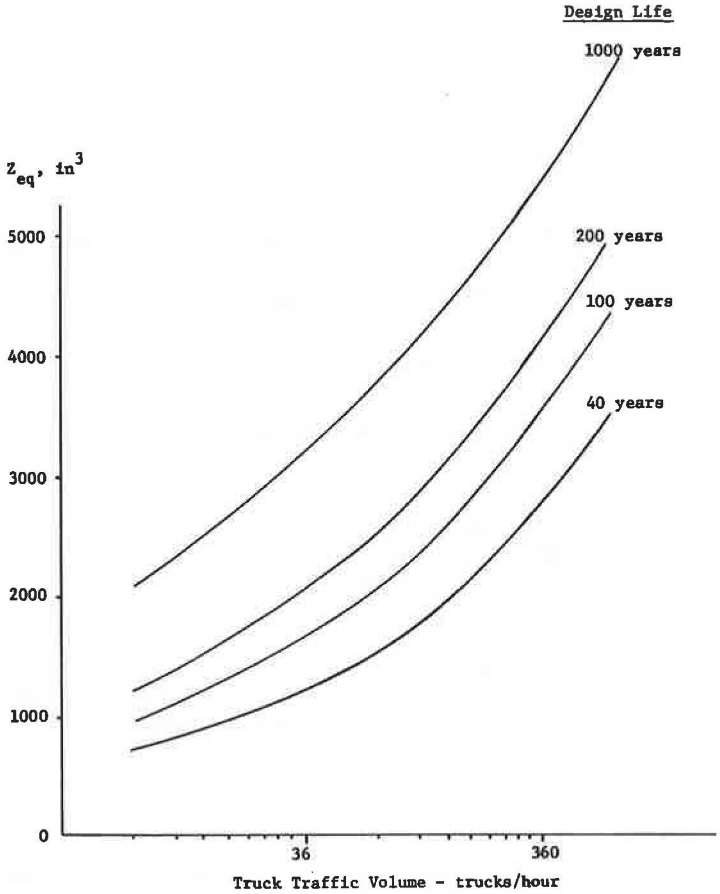


Table 1. Suggested design detail factors (D_f).

Design Factor	Detail
3.56	Rolled sections
2.58	Welded beams and girders
2.22	Welded flange splices
1.00	Cover plate terminations

Table 2. C_{XL} values for various simple span lengths and locations.

Length (ft)	Weld Location 2/10	3/10	5/10	7/10	8/10
20	6.16	4.88	4.38	4.88	6.27
30	3.59	2.83	2.50	3.08	3.98
40	2.56	1.98	1.72	2.20	2.78
50	1.87	1.52	1.31	1.48	1.87
60	1.37	1.10	1.00	1.05	1.37
70	1.07	0.85	0.758	0.820	1.07
80	0.873	0.69	0.606	0.672	0.873
90	0.740	0.581	0.508	0.566	0.740
100	0.642	0.503	0.436	0.491	0.642
110	0.565	0.442	0.382	0.433	0.565
120	0.508	0.394	0.340	0.388	0.508

$$C_I = \frac{1.2}{I} \tag{10}$$

where I is the impact factor. For example, if the AASHTO value is used

$$I = 1 + \frac{50}{L + 125}$$

where L is span length.

Choose a stringer spacing S' and weld detail. Table 1 gives values of D_f for various weld details.

Calculate the required girder section modulus Z by using Eqs. 6, 8, and 9.

$$Z = (Z_{eq})_{tab} \frac{GN_sN_L}{D_f \cdot C_{XL} \cdot C_I}\left(\frac{V}{V_{tab}}\right)^{1/6} \tag{11}$$

A general equation for checking the elastic section modulus in terms of truck volume and bridge and weld characteristics is

$$Z = (Z_{eq})_{tab} \frac{\left(\frac{S'}{25}\right)(1.75)(1.5)}{D_f \cdot C_{XL} \cdot C_I}\left(\frac{V}{36}\right)^{1/3} \tag{12}$$

EXAMPLES OF FATIGUE DESIGN

Several examples illustrate the simplified fatigue design procedure described. The examples are not inclusive, and an extensive study needs to be taken of (a) truck weight variations at different locations, (b) a larger range of continuous span bridges, and (c) additional weld categories. However, all elements of the design procedure are indicated.

Example 1

Example 1 is a 60-ft simple span with a Maryland truck weight histogram (6). It has a 100-year service design with 36 trucks per hour expected volume. The design calls for a rolled beam on 7-ft center spacings. Z_{eq} for this truck weight histogram was 1,333 in.^{3}. The impact factor by the AASHTO code is I = 1 + [50/(60 ft + 125)] = 1.27. Therefore, C_I = 1.2/1.27 = 0.95. For a rolled beam, Table 1 gives D_f = 3.56. Table 2 gives C_{XL} equal to 1.0 for this case. Substituting in Eq. 12 gives, for the required girder section modulus,

$$Z = 1{,}333 \frac{\left(\frac{7\text{ ft}}{25}\right)(1.75)(1.5)}{(3.56)(1.0)(0.95)} = 293 \text{ in.}^3 \tag{13}$$

Example 2

Example 2 has the same bridge data and truck volume as in example 1 except the truck weight histogram is from Ohio (5). Z_{eq} for this case was 1,700 in.^{3}; thus,

$$Z = 1{,}700 \frac{\left(\frac{7\text{ ft}}{25}\right)(1.75)(1.5)}{(3.56)(1.0)(0.95)} = 293 \times \frac{1{,}700}{1{,}333} = 375 \text{ in.}^3 \tag{14}$$

Example 3

Example 3 has the same data as example 1, but plate girders with twice the spacing (14 ft) rather than rolled beams were used. Table 1 gives D_f = 2.58 for a welded plate girder. The section modulus now required is

$$Z = 1{,}333\,\frac{\left(\frac{14}{25}\right)(1.75)(1.5)}{(2.58)(1.0)(0.95)} = 2 \times \frac{3.56}{2.58} \times 293 = 815 \text{ in.}^3 \tag{15}$$

Example 4

Example 4 is a simple span bridge with the same data as example 1, but a rolled beam with cover plate at 2/10 span is used with a spacing of 7 ft, 11 in. In this case both the section at midspan and the section at the weld cutoff must be checked. Table 1 gives a value of D_f = 1.0 for a cover plate termination and 2.58 for a welded beam at the midspan. Table 2 gives the value of 1.37 for C_{XL} at the weld cutoff. Thus,

$$Z_{\text{midspan}} = 1{,}333\,\frac{\left(\frac{7^{11}/_{12}}{25}\right)(1.75)(1.5)}{(2.58)(1.0)(0.95)} = 455 \text{ in.}^3$$

$$Z_{2/10\text{cutoff}} = 1{,}333\,\frac{\left(\frac{7^{11}/_{12}}{25}\right)(1.75)(1.5)}{(1.0)(1.37)(0.95)} = 850 \text{ in.}^3 \tag{16}$$

Example 5

Example 5 has the same data as example 1 but has a change in truck rate from 36 trucks per hour to 100 trucks per hour (100-year life). With Eq. 8, Z_{eq} is modified by

$$Z_{eq} = (Z_{eq})_{tab}\left(\frac{100}{36}\right)^{1/6} = 1{,}333 \times (2.78)^{1/3} = 1{,}800 \text{ in.}^3$$

and

$$Z = 293\left(\frac{1{,}800}{1{,}333}\right) = 413 \text{ in.}^3 \tag{17}$$

Example 6

Example 6 has the same data as example 1 but has an 80-ft bridge instead of a 60-ft bridge. The impact factor should be modified because the computed Z_{eq} value is based on 1.2 rather than on AASHTO values. By using Eq. 10

$$C_I = \frac{1.2}{1 + \frac{50}{80 + 125}} = 0.96$$

For 80 ft and a midspan location, C_{XL} = 0.606 (Table 2). Z_{eq} is now modified by these values with Eqs. 9 and 10:

$$Z_{eq} = \frac{1{,}333}{(0.606)(0.96)} = 2{,}300 \text{ in.}^3$$

and the required girder section modulus

$$Z = \frac{2{,}300}{1{,}333} \times 293 = 502 \text{ in.}^3 \tag{18}$$

Example 7

A three-span bridge located in Portage County, Ohio, was used and will be checked for adequate section modulus by the procedures presented. The cross section consisted of 36 W 150 beams (Z = 504 in.^{3}) at 7-ft, 11-in. spacing with cover plates at the supports. The symmetric three spans are 48, 60, and 48 ft with cover plates extending

6 ft on either side of the intermediate supports. The truck rate is 70 trucks per hour.

Fatigue checks will be done at A, center of first span; B, center of middle span; and C, cover plate termination location in first span.

A static bending moment analysis is done by using influence functions, and a tractor-trailer loading gave the following C_{XL} values based on the ratio of peak bending moment range to the center moment on a 60-ft simple span. At A, $C_{XL} = 1.25$; at B, $C_{XL} = 1.38$; and at C, $C_{XL} = 2.15$. A dynamic analysis gave an impact factor ratio value of C_I of about 0.94 applicable at all three locations. The detail factor at A and B is 3.56 for a rolled beam and 1.0 at C for a cover plate termination. Assuming the Ohio weight histogram (5), which would seem applicable at site, Z_{eq} was given above as 1,700 in.^{3} based on 36 trucks per hour. Thus, from Eq. 8:

$$Z_{eq} = (Z_{eq})_{tab}\left(\frac{70}{36}\right)^{1/3} = (1{,}700)(1.94)^{1/3} = 2{,}120 \text{ in.}^3$$

Substituting now for the required section modulus expression (Eq. 12) gives

$$Z_A = 2{,}120\,\frac{\left(\frac{7^{11}/_{12}}{25}\right)(1.75)(1.5)}{(3.56)(1.25)(0.94)} = 423 \text{ in.}^3 \quad (<504)$$

$$Z_B = 2{,}120\,\frac{\left(\frac{7^{11}/_{12}}{25}\right)(1.75)(1.5)}{(3.56)(1.38)(0.94)} = 385 \text{ in.}^3 \quad (<504)$$

$$Z_C = 2{,}120\,\frac{\left(\frac{7^{11}/_{12}}{25}\right)(1.75)(1.5)}{(1.0)(2.15)(0.94)} = 870 \text{ in.}^3 \quad (>504) \tag{19}$$

Because the actual section has a modulus of 504 in.^{3}, locations A and B are satisfactory although C is relatively unsatisfactory. (Fatigue checks should also be done at the pier.) It should be emphasized that the above checks on section size reflect the assumed values of risk N_m (1.75), the load factor N_L (1.5), and a truck weight histogram averaged from several locations.

CONCLUSIONS

The study confirmed changes generally needed in fatigue specification, i.e., a fatigue loading separate from yield loading, consideration of live-load stress range rather than maximum peak stress, and updating of the load analysis to reflect actual traffic and truck load conditions on a given roadway.

Further effort is needed to clarify several points. These include continuous span bridges (only a three-span has been done thus far), the number of roadway types with different gross weight distributions and type percentages, the girder distribution and detail factors, and the material and load safety factors. These must be found from further field and laboratory measurements and calibration with existing designs.

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