Effect of Alternative Truck Configurations and Weights on the Fatigue Life of Bridges

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The life of a bridge is principally influenced by repetitive loading resulting from vehicular traffic. The service data on the fatigue life of concrete, reinforcing bars, and prestressing steel show considerable scatter in their service life. This is due to both the stochastic nature of the imposed loading and the variability in their strengths as determined by the quality control in their manufacture. The fatigue life of partially prestressed concrete girder bridges, subjected to a spectrum of traffic imposed by the Poisson arrival of various categories is investigated. Each category examined has a different expected frequency of arrivals per unit time and a different distribution of gross weight. The allocation of the live load to the girders in skew and normal bridges is determined using the finite element method. The girders of the bridge are each assumed to be part of a series system consisting of four components: prestressing strands, reinforcing bars, cast-in-place concrete slab, and precast girder. The nine-axle B-train double trucks were found to be most damaging, whereas two-axle single trucks were least damaging. The incremental damage caused by each truck depends on the truck configuration, gross weight, axle-load distribution, and lateral load distribution. The median life of ordinary reinforcing bars is the lowest among the girder components.

Concrete structures, when subjected to fluctuation in strain during duty cycles, will eventually accumulate sufficient damage within the constituent materials of their components to limit their useful service life. These repetitive loadings can reduce bonding properties at the interface between steel and concrete and lead to cracks of substantial width, thereby allowing extensive deflections under service loads. The degree of damage is a function of many factors. Principally, these factors are the number and magnitude of the stresses during load cycles; the variability of loads; the configuration of the loads and their allocation to the structural components inducing the corresponding strains; and the degree of microscopic cracking with resultant change in constituent material properties.

A primary example of such structures of importance in civil engineering is the partially prestressed concrete (PPC) girder bridges that are subjected to repeated heavy loadings from truck traffic. The effect of loading on the bridge components is modified somewhat through different truck designs now necessary because of extreme truck weight allowed. If, as a result of these continual load fluctuations, significant cracking occurs in the concrete and material losses occur with the alteration of the material properties entailed, the structure may eventually fail in fatigue.

Nearly 45 percent of the nation's 600,000 bridges have been classified by FHWA as severely damaged and deteriorated because of heavy vehicles, natural environmental hazards, or lack of maintenance. Many studies using various experimental techniques, have been conducted to investigate the fatigue properties of prestressed concrete girders. In some instances, the reliability of the results of the tests was questioned by members of the prestressed concrete industry who claimed that inappropriate test techniques were used or that the experimenters were ignorant of material properties (*1*).

All available data on the fatigue life or cumulative damage of structural materials show considerable scatter and lack of deterministic predictability. This is due both to the variability of material properties among specimens and to uncertainties in predicting the deterioration of the structural material under cyclic loading. Moreover, simplified deterministic methods of analysis do not allow for uncertainties in calculating actual loads that will be applied to the structure. Most studies on the safe-life for concrete structures not only have ignored the considerable scatter induced by material variability and the randomness in anticipated loads but also have bypassed the necessity of censoring (i.e., aborted tests or run-outs in the specimens) the fatigue-test data.

Adopting Turner trucks may contribute significantly to fatigue damage of PPC bridges because cracks are more likely to develop in the bridge girders. Crack formation will accelerate fatigue damage and shorten the lives of these bridges. The present objective is to develop a model of fatigue reliability for PPC bridges using a theory of stochastic cumulative damage to evaluate the bridges in the nation's highway system.

LOAD MODELS

Truck weight is considered to be well defined by the inverse Gaussian distribution (*2*). In this study, the trucks are grouped into six categories on the basis of the number of their axles. Table 1 shows the categories of trucks and the average and standard deviations of their gross weight in each class. Because of the lack of statistical data on such trucks, the coefficient of variation of a Turner truck weight is assumed to be .30. Hwang and Nowak (*3*) found that the mean value of dynamic load factors and coefficient of variation for pre-

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TABLE 1 Truck Categories and Their Statistical Parameters

Class	A.W.[a] (KN)	S.D.[b] (KN)	Axle Spacing (m)								Axle Weight Distribution		
2-Axle Single	70.	26.6	5.1	-[c]	-	-	-	-	-	-	33.6	66.4	-
3-Axle Single	124.	30.0	5.1	1.3	-	-	-	-	-	-	30.1	69.5	-
4-Axle Semi-Trailer	144.	53.9	3.6	7.7	1.1	-	-	-	-	-	17.9	38.3	43.8
5-Axle Semi-Trailer	226.	78.7	3.6	1.3	8.7	1.2	-	-	-	-	13.9	48.1	38.0
5-Axle Split	240.	78.3	3.7	1.3	7.7	2.8	-	-	-	-	11.9	47.5	40.5
Nine-Axle B-Train Double	494.	148.	3.0	1.3	7.3	1.3	1.3	1.3	7.2	1.3	9.91	11.3	11.3[d]

[a] Average gross weight of a truck.
[b] Standard deviation of truck weight.
[c] Not applicable.
[d] All other 6-remaining axles have the same weight.

stressed concrete girder bridges are span-dependent. The dynamic load factor (I) is assumed to be log-normal; its coefficients of variation are equal to .53 and .62 for bridge spans of 80 and 100 ft, respectively. The mean value of I is constant and equal to 0.12.

ALLOCATION OF LOAD

Khaleel and Itani (*4*) investigated the live load distribution for slab-girder bridges. They proposed an analysis procedure expressed in the form of an algorithm. The maximum bending moment is given by

$$M = M_0 \frac{b}{D} KI \qquad (1)$$

where

b/D = lateral load distribution factor,
K = skew reduction factor,
I = impact allowance, and
M_0 = maximum static moment (half the truck load applied to a single isolated girder).

The maximum static moment for a k-axle truck (or semitrailer or tractor) is expressed by

$$M_0 = \frac{W}{8a} a^2 + aA_k(s, \delta) + B_k(s, \delta) \qquad (2)$$

where

a = span of bridge,
W = gross weight of truck,
s = vector of axle spacings,
δ = vector of ratios of axle weight to gross weight, and
$A_k(.) > B_k(.)$ = functions of s and δ.

The lateral distribution and skew reduction factor are assumed to be normally distributed. Khaleel (*5*) used the finite element method to derive expressions for b/D and K for both single-span and continuous-skew girder bridges. Expressions of D-values for exterior and interior girders are presented in Figures 1 and 2.

Live and dead loads are allocated to girders on the basis of the model described in Equations 1 and 2. Al-Zaid et al.

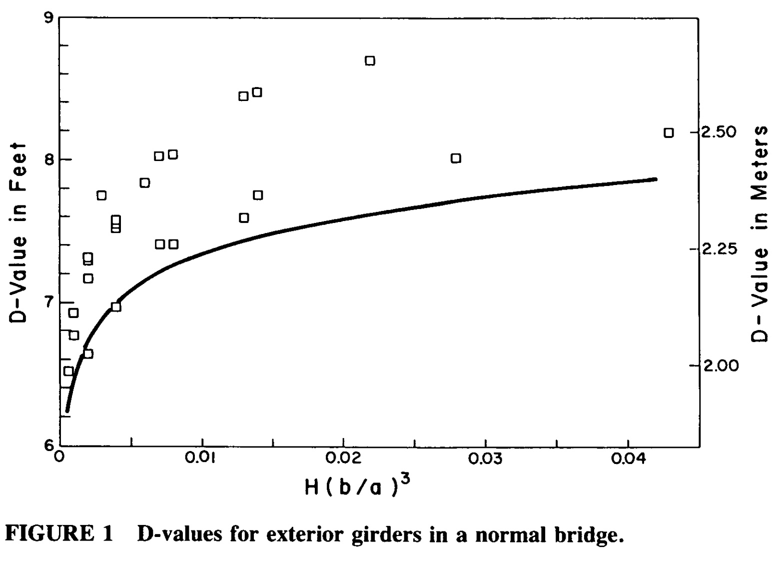

FIGURE 1 D-values for exterior girders in a normal bridge.

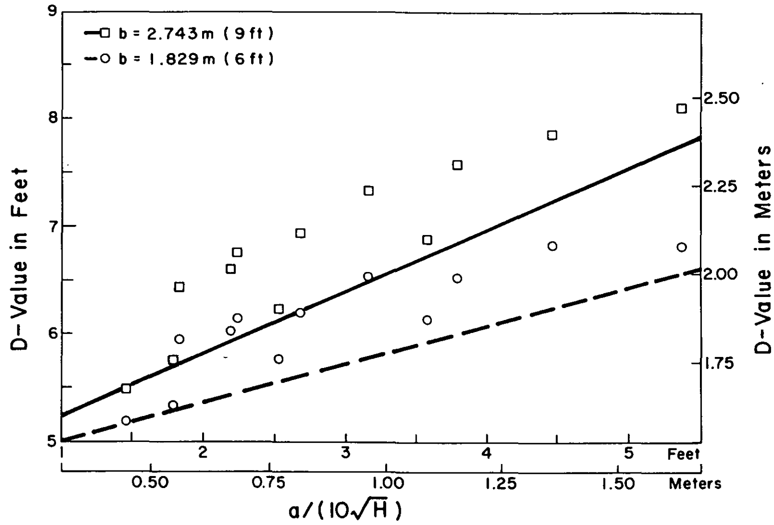

FIGURE 2 D-values for interior girders in a normal bridge.

(*6*) developed a general time-step analysis procedure that considers the effects of time- and cycle-dependent creep, concrete shrinkage and relaxation of prestressing steel. This model calculates stress in cracked or uncracked composite sections; it is used here to compute maximum and minimum stresses in bridge components, with maximum stress resulting from total load and minimum stress from dead load only. These stresses are then used to compute median life (β) and shape parameter (α) for cumulative-damage distribution for bridge life under stochastic loading.

Cumulative Damage Distribution

Khaleel (*2*) derived composite fatigue life that has the following formula:

$$\bar{\beta} = \frac{1}{\sum_{i=1}^{k} \frac{\lambda_i}{\hat{\beta}_i}} \tag{3}$$

where

- $\bar{\beta}$ = units of average daily truck traffic (ADTT),
- k = number of traffic categories,
- $\hat{\beta}_i$ = bridge fatigue life resulting from arrival of all trucks in category i, and
- λ_i = expected number of truck per day for ith category.

For each passing truck, β_{ij} is calculated on the basis of engineering data presented by Khaleel (*2*) and the shape and scale parameters from the B-S distribution fitted to all stress ranges. Then $\hat{\beta}_i$ is calculated as

$$\hat{\beta}_i = \frac{1}{\sum_{j=1}^{N_i(t)} \frac{1}{\beta_{ij}}} \tag{4}$$

Equation 3 is the Miner's rule in central tendency (i.e., median). It has been empirically shown to hold better than any other fatigue-life rule. The harmonic mean, Miner's rule, is rigorously derived as the actual value of the median life under the assumptions specified.

Moreover, Khaleel (*2*) calculated the corresponding shape parameter for the distribution of the fatigue life for the structure under the mixture of loads imposed, which is given by

$$(\bar{\alpha})^2 = \bar{\beta} \sum_{i=1}^{k} \frac{\lambda_i}{\hat{\beta}_i} \left[\hat{\alpha}_i + \frac{1}{\hat{\beta}_i} \right] \tag{5}$$

Thus, a distribution has been found to which the median life (i.e., composite fatigue life) can be calculated under a mixture of traffic using Miner's rule and the coefficient of variation determined so that it is possible to calculate "the fraction of life used," that is, the probability of failure after any fraction of the characteristic life has been spent. But, as is known empirically, this probability depends upon factors other than the median life. The formula in Equation 3 recognizes this influence through the corresponding cumulative-damage distribution and the associated $\bar{\alpha}$.

Traffic Influence Model

Consider a structure that sustains random loads in service but in which each load is allocated to all its multiple components. The strain (or stress) for each component, for a given geometry of the structure and a fixed type of load, is a complex but deterministic function found by using finite element analysis. Unfortunately the magnitudes, intensity, and allocation of total load to each component are random. Moreover, it is assumed the load order is random, the sequences of a given type form an identically independent distributed sequence, and the frequency of loads of each category is a stationary stochastic process.

The total cumulative damage sustained by the structure up to time $t > 0$ is the damage from all categories of load fluctuations (*2*) and is given by

$$Y_t = \sum_{i=1}^{k} \sum_{j=1}^{N_i(t)} Y_{i,j} \tag{6}$$

where

$Y_{i,1}, Y_{i,2}, \ldots$ = incremental damages to structure from successive loads of type $i = 1, 2, \ldots, k$;

k = number of truck categories; and

$N_i(t)$ = frequency of loading under each category assumed to be a Poisson process.

The probability of survival of the structure (*2*) is given by

$$Pr[T \geq t] \cong 1 - \Phi\left[\frac{1}{\alpha}\,\xi\left(\frac{t}{\beta}\right)\right] \tag{7}$$

where $\xi(x) = x^{1/2} - 1/x^{1/2}$ and Φ is cumulative normal distribution function.

VERIFICATION OF MODEL

The method for calculating the number of cycles to fatigue failure is verified by comparison of experimental and predicted data. Holmen (*7*) studied the effect of various load histories on the fatigue behavior of plain concrete. Experimental results from two- and multistage constant amplitude loadings (*7*) are compared against theoretical results. In two-stage loading, the test specimen was exposed to a constant stress amplitude of a given level until a fixed number of cycles was completed. Then, in the second stage, the level was changed and maintained until failure. The calculated number of cycles to failure are compared with the experimental ones as shown in Tables 2, 3, and 4. For the multistage constant-amplitude loading, the predicted and experimental numbers of cycles to failure are 1,338,907 and 1,502,931, respectively. The maximum stress in the multistage case ranged from 0.12 to 0.88 of the concrete compressive strength, f'_c. The results for Alternatives A and B of the two-stage loading show good agreement between the experimental and theoretical results. The maximum stress levels in the two-stage loading are very high; this fact explains the differences in the predicted and experimental results. The experimental and theoretical results show that the presence of small amplitudes in a loading histogram seems to reduce the sequence effects. Bridges are generally subjected to small amplitudes of stresses; therefore, it is reasonable to ignore the load-order effect.

RESULTS

A simply supported bridge with a span of 80 ft is considered. The reinforced concrete slab is supported by 10 AASHTO Type IV precast and pretensioned girders spaced at 8 ft as shown in Figure 3. The girders were designed according to the AASHTO specifications (*8*). The traffic is grouped into six categories on the basis of number of truck axles. Moses and Ghosn (*9*) conducted a study using weigh-in-motion technology to obtain reliable information on bridge traffic. A FORTRAN program was written to implement the traffic model. This program is available from the lead author of this paper.

Effect of Truck Configuration

For purposes of investigating the effect of truck configuration on bridge fatigue life, the ADTT is assumed to consist entirely of one category of trucks of varying gross weights. Figure 4 shows the composite fatigue life of two of the girder components (i.e., prestressing steel and reinforcing steel) versus the truck category (two-axle single, three-axle single, four-axle semitrailer, five-axle semitrailer, five-axle split, and nine-

TABLE 2 Multistage Constant Amplitude Loading: Experimental and Theoretical Results

$\frac{\sigma_{max}}{f'_c}$	Applied Cycles	β_{ij}	$\frac{\lambda_i}{\beta_{ij}}$
.88	380	656	.579
.82	1,069	3458	.309
.75	3,470	2,4071	.144
.68	9,882	167,556	.059
.61	24,917	1,166,326	.021
.54	54,588	8,118,579	.006
.47	105,561	5.65×10^{7}	1.86×10^{-3}
.40	177,354	3.93×10^{8}	4.51×10^{-4}
.33	256,555	2.74×10^{9}	9.37×10^{-5}
.26	314,583	1.91×10^{10}	1.65×10^{-6}
.19	315,864	1.33×10^{11}	2.38×10^{-6}
.12	238,708	9.24×10^{11}	2.58×10^{-7}
N_F [a]	1,502,931	–[b]	1,338,907

[a] N_F is the total number of cycles to fatigue failure.
[b] Not applicable.

TABLE 3 Predicted and Experimental Number of Cycles to Failure Based on Two-Stage Constant Amplitude Loading: Alternative A

Specimen Number	Number of Cycles			
	Experimental		Predicted	
	.75 f_c'[a]	.90 f_c'[b]	.75 f_c'[a]	.90 f_c'[b]
1	4980	136	8824	239
2	4110	520	2680	335
3	6710	344	5735	287
4	6760	195	8515	243
5	2260	187	3804	317
-	4136[c]	276[c]	4926[c]	236[c]

[a] Maximum applied stress (the first stage) was maintained until a fixed number of cycles were completed.
[b] Maximum applied stress (the second stage) was maintained until failure.
[c] Average number of cycles of the five specimens at a certain maximum stress level.

axle B-train are Categories 1 to 6, respectively). The nine-axle B-train double trucks were the most damaging, whereas the two-axle single trucks were least damaging. The four-axle semitrailer trucks are less damaging than three-axle single trucks, even though their average weight is higher than that of three-axle single trucks. The incremental damage caused by each passing truck depends on its gross weight, configuration, axle load distribution, and lateral load distribution on the bridge.

Effect of Concrete Strength

The compressive strength of concrete was varied to study its influence on the fatigue life of the structural components subjected to the spectrum of loads. The effect of increasing f_c' from 30 to 40 MPa yields a slight increase in the fatigue life of prestressing and nonprestressing steel. The fatigue lives of cast-in-place slab and precast girder were increased 233 and 355 percent, respectively. The results are shown in Figure 5.

Effect of Change from Nominal Values

Figure 6 shows that the effect of uniform reduction of the nominal area of reinforcing steel on the fatigue life of prestressing steel is negligible. Reducing the area of prestressing steel by 20 percent, however, results in 62 and 79 percent reductions in the fatigue life of prestressing and reinforcing steel, respectively.

Effect of Partial Prestressing Ratio

Several girders were designed with the same moment capacity but with different partial prestressing ratios (PPRs) to inves-

TABLE 4 Predicted and Experimental Number of Cycles to Failure Based on Two-Stage Constant Amplitude Loading: Alternative B

Specimen Number	Number of Cycles			
	Experimental		Predicted	
	.90 f_c'[a]	.75 f_c'[b]	.90 f_c'[a]	.75 f_c'[b]
1	108	22140	90	18348
2	74	10940	62	9145
3	40	710	294	5288
4	45	4340	151	14448
5	80	4110	209	10681
-	69[c]	8448[c]	161[c]	11582[c]

[a] Maximum applied stress (the first stage) was maintained until a fixed number of cycles were completed.
[b] Maximum applied stress (the second stage) was maintained until failure.
[c] Average number of cycles of the five specimens at a certain maximum stress level.

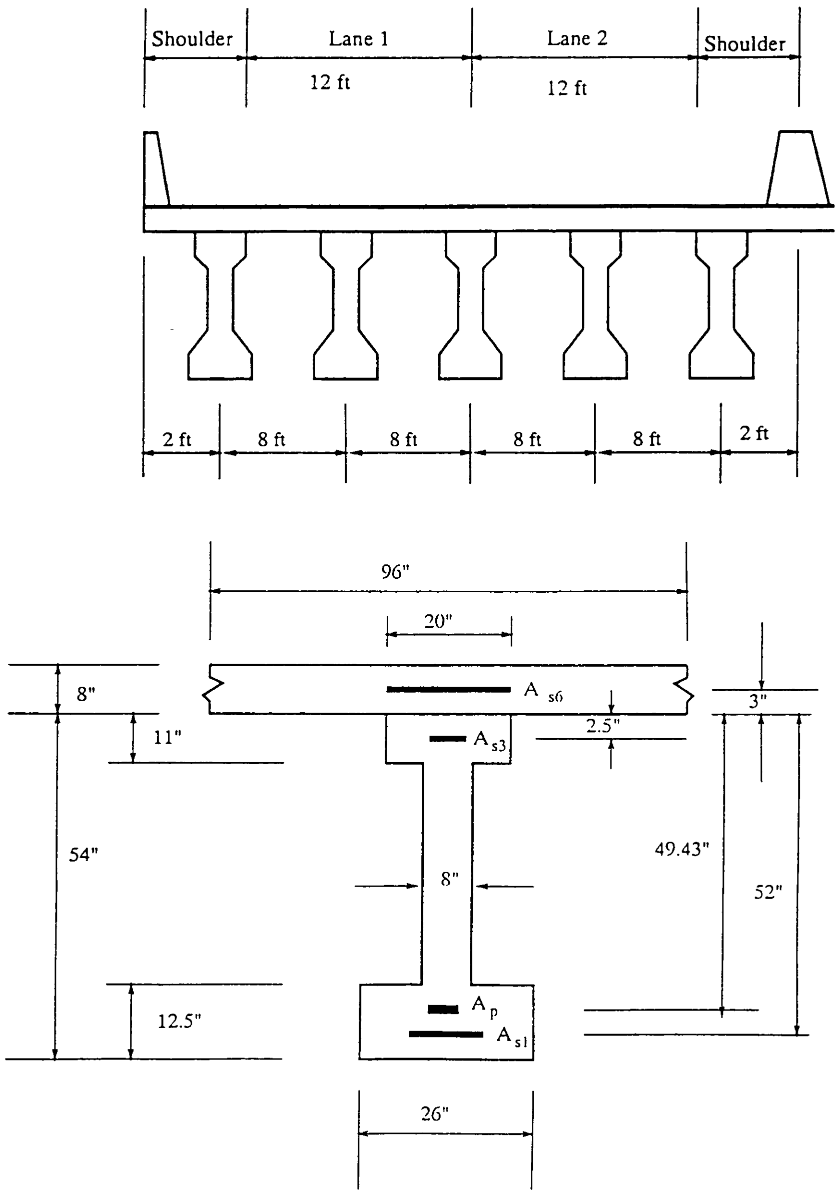

FIGURE 3 Cross section of a typical slab-and-girder bridge.

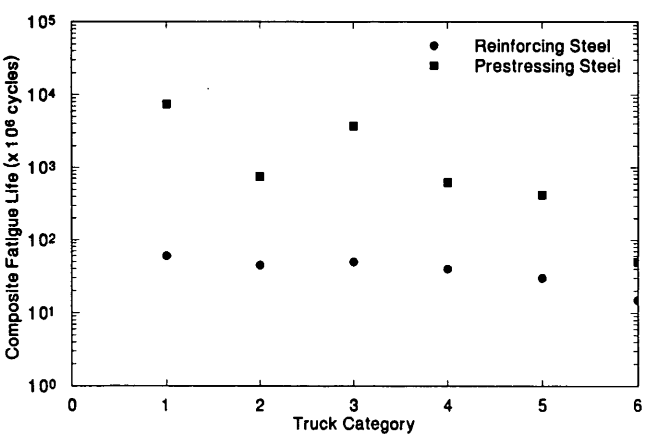

FIGURE 4 Effect of truck configuration on fatigue life.

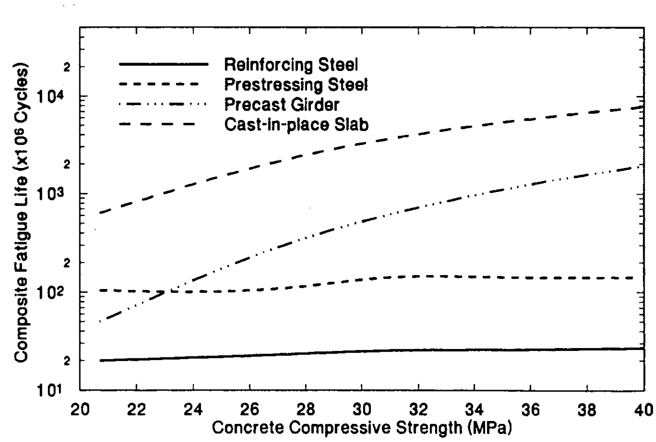

FIGURE 5 Effect of concrete strength on fatigue life.

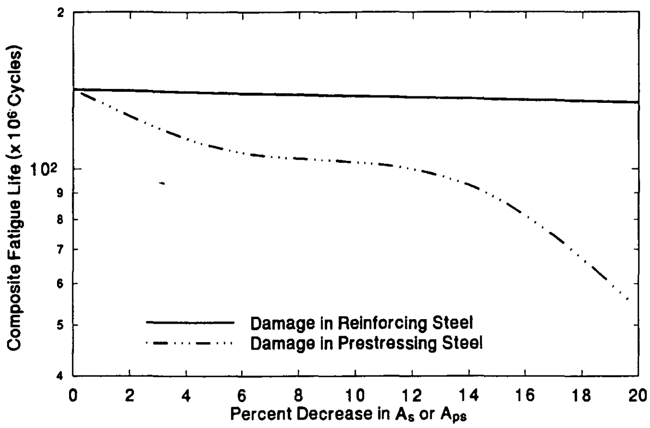

FIGURE 6 Effect of damage in reinforcement on fatigue life of prestressing steel.

tigate the effect of PPR on fatigue life. Figure 7 shows that a girder with a high PPR has a higher fatigue life than one with a low PPR.

SUMMARY AND CONCLUSIONS

An efficient method is presented for calculating the number of loadings a bridge member can withstand before a detectable fatigue crack develops. This method recognizes that the probability of survival depends on factors other than the median

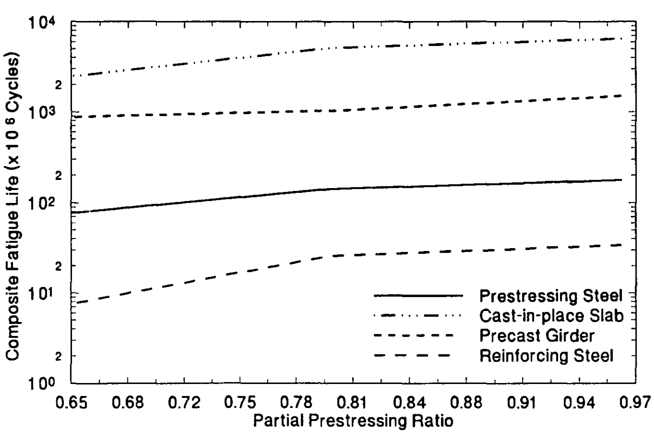

FIGURE 7 Effect of PPR on fatigue life.

life through the corresponding cumulative damage distribution and the associated $\overline{\alpha}$. The following major conclusions are drawn from the study:

- The nine-axle B-train double (i.e., one of the Turner) trucks were found to be most damaging, whereas two-axle single trucks were least damaging. The incremental damage caused by each truck depends on truck configuration, gross weight, axle-load distribution, and lateral load distribution factor.
- The median life of ordinary reinforcing bars is the lowest among the girder components. Cast-in-place slab, on the other hand, has the highest median life.
- Reducing the nominal cross-sectional area of prestressing steel reduces the fatigue life of all components significantly, whereas reducing the area of reinforcing steel has virtually no effect on the fatigue life of the structure.
- Girders designed with low PPR have a much shorter median fatigue life than those designed with high PPR.

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