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G. Chen et al., "Fatigue Assessment of Traffic Signal Mast Arms based on Field Test Data under Natural Wind Gusts," *Transportation Research Record*, vol. 1770, pp. 188-194, National Academy Press, Jan 2001.
The definitive version is available at https://doi.org/10.3141/1770-24

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Paper No. 01-2801

Fatigue Assessment of Traffic Signal Mast Arms Based on Field Test Data Under Natural Wind Gusts

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In recent years, several states including Missouri, Wyoming, California, and Texas experienced fracture failures of traffic signal mast arms. Almost all the failures are associated with the propagation of defects or cracks. It is therefore imperative to evaluate existing mast arms using a simple yet accurate procedure. A statistical methodology is proposed to predict the fatigue life of signal mast arm structures on the basis of field-measured strain data. The annual occurrence of various stress levels is determined using the historical wind speed data in the vicinity of a mast arm structure and the strain readings of the structure under specific wind gusts. For each stress level, the crack initiation and propagation lives are estimated with the strain-life approach and the Paris crack-growth-rate model. They are combined to account for variable stresses by means of Miner's rule and the root-mean-square model, respectively. The stress concentration factor around the arm-post connection is determined using a finite element model. The parameters in the life prediction models are determined with ASTM flat tension and compact tension tests. The proposed methodology was applied to a 12.8-m (42-ft) long octagonal mast arm and a 16.5-m (54-ft) long circular mast arm in Missouri. It is concluded that signal structures in perfect condition will not crack under natural wind gusts during their service life. However, the 16.5-m-long arm is likely to be vulnerable to tiny defects around the weld connection, but the 12.8-m-long arm is safe unless a visible crack exists.

In the past 6 years, a dozen traffic signal mast arms in Missouri fractured at the arm-post weld connection. Most of them lost their function after 1 to 2 years of service, whereas others stayed in service for about 20 years. Other states (*1; 2,* pp. 1107–1110) also experienced similar failures in signal mast arms during the last few decades. It is likely that these failures result from overstressing, poor welding quality, and low fatigue strength. To explore the direct causes for the failures and develop retrofit techniques for existing signal mast arm structures, the Missouri Department of Transportation initiated a research project on fatigue failure investigation of signal mast arms. As an important part of the project, the fatigue life of two in-service mast arms is predicted on the basis of field test data. A statistical methodology is introduced for estimating the cyclic loading on the mast arms due to natural wind gusts and assessing the fatigue condition of the existing signal support structures.

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FIELD TESTS OF SIGNAL MAST ARMS

Two signal mast arms, located at Providence and Green Meadows Boulevards and at Forum and Stadium Streets in Columbia, Missouri, were considered as typical structures in this study. Both structures were instrumented in the field to monitor truck- and wind-induced vibration. An anemometer was used to measure the speed of wind gusts. The signal support structure at Stadium and Forum Streets is shown schematically in Figure 1*a* and a typical arm-post connection in Missouri is detailed in Figure 1*b*. The cantilever mast arm is welded to a base plate that is bolted to the post. The mast arm and the post are made of circular steel pipes 31.75 cm by 11.81 cm by 16.5 m by 7 gauge (GA) (12.5 in. by 4.65 in. by 54 ft by 7 GA) and 36.83 cm by 28.96 cm by 8.23 m by 0.556 cm (14.5 in. by 11.4 in. by 27 ft by 0.2188 in.), respectively. The structure at Providence and Green Meadows Boulevards is similar except that the mast arm is octagonal. The dimensions of the arm and the post are 26.67 cm by 8.89 cm by 12.8 m by 7 GA (10.5 in. by 3.5 in. by 42 ft by 7 GA) and 36.20 cm by 29.21 cm by 8.38 m by 0.556 cm (14.25 in. by 11.5 in. by 27.5 ft by 0.2188 in.), respectively.

During the field tests, both truck- and wind-induced vibrations were recorded. However, the strain of the mast arm caused by truck passage is significantly lower. Therefore, the following fatigue life prediction is mainly focused on the effect of natural wind gusts. A total of 31 wind events (518 s of accumulated time) were recorded for the signal support structure at Providence and Green Meadows Boulevards and 26 events (451 s of accumulated time) were recorded at Stadium and Forum Streets. The wind speed, v, measured during the tests ranges from 2.67 to 13.33 m/s (6 to 30 mph). Each structure was instrumented with 12 strain gauges (*3,* pp. 1111–1114) as shown in Figure 2. Among the 12 strain records, emphasis is placed on analyzing the responses at Strain Gauges 1, 3, 5, and 7. They measured the longitudinal strains located 10 cm (4 in.) away from the arm-post connection. Presented in Figure 3 is a typical stress-time history at Strain Gauge 3, which is converted directly from the measured strain at a wind gust of 7.33 m/s (16.4 mph).

EVALUATION PROCEDURE

Fatigue failure of a properly designed and carefully constructed structure usually takes place after a long period of service. It results from the accumulated effect of a significant amount of minor damage. Therefore it is prudent to assume that, for the purpose of predicting the fatigue stress, natural wind gusts are repeatable every 10 years in a statistical sense.

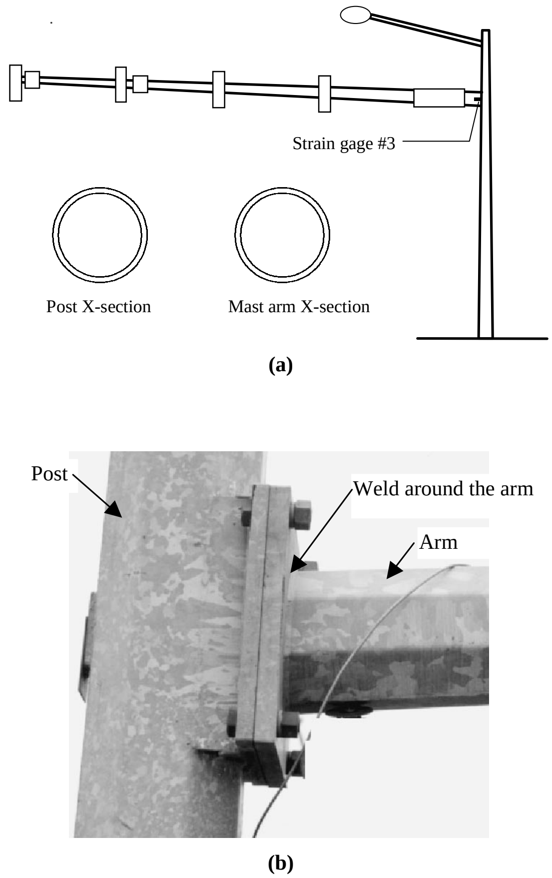


FIGURE 1 Traffic signal support structure at Stadium and Forum Streets: (*a*) elevation view; (*b*) typical arm-post connection.

Both instrumented structures were modeled using an ABAQUS finite element program. Their fundamental frequencies were found to be slightly less than 1 Hz, which is in good agreement with the test data presented in Figure 3. These frequencies are significantly larger than the dominating frequency of natural wind fluctuation (*4*), for example, less than 0.1 Hz. Therefore, wind fluctuations will not induce the resonant vibration of the mast arm structures. Although dependent on the direction and angle of wind gusts, the dynamic responses of the mast arms are most likely proportional to the wind pressure or the square of average wind speed. If the ratio between the wind-induced stress and the square of wind speed is of a similar statistical distribution to that for a range of the measured wind speeds, the same distribution can be reasonably extended to higher wind speed ranges that are rarely recorded in field tests. The following procedure is recommended to estimate the number of stress cycles at various levels due to natural wind gusts and to predict the fatigue life of an instrumented signal structure:

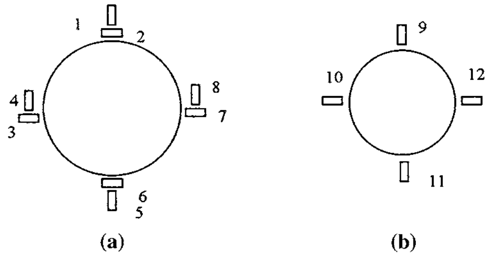


FIGURE 2 Strain gauge location on mast arm at Stadium and Forum Streets: (*a*) base; (*b*) midspan.

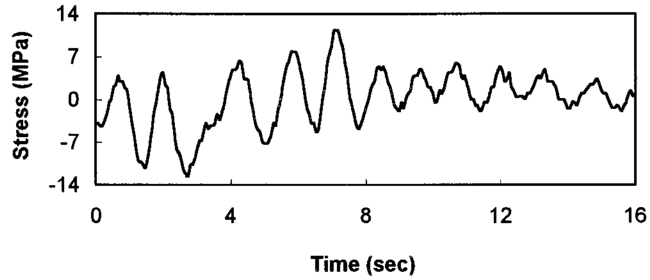


FIGURE 3 Stress time history from Gauge 3.

1. Analyze the historical wind gust records (10 years) in the vicinity of the instrumented structures to determine the statistical distribution of the wind speed,
2. Determine the number of cycles at various stress levels (normalized by the square of wind speed) from the field test data on the instrumented mast arms,
3. Extrapolate the stress distribution in Step 2 into the corresponding stress for the rare wind gusts of higher speed,
4. Compute the number of cycles corresponding to different stress ranges by multiplying the wind speed distribution by the load spectrum from Steps 2 and 3,
5. Determine the number of cycles that the mast arm can endure before a fatigue failure occurs under different cyclic loads of constant amplitude, and
6. Divide the results in Step 4 by those in Step 5 to calculate the minor damages and combine them to predict the fatigue life of the signal structure under a variable stress loading.

WIND-INDUCED STRESS ON SIGNAL MAST ARMS

Wind Speed Distribution at Columbia, Missouri

The wind speed information at Columbia, Missouri, is provided by the National Climatic Data Center (NCDC). The data were collected during the period 1969 through 1978. They include the monthly and annual statistics on the occurrence of wind events at various hourly mean wind speeds in 16 horizontal directions. Since only the distribution of wind speed is needed, the wind gusts with the same wind speed were grouped into one category regardless of their direction. Figure 4 shows the annual statistics of wind speed. The wind speed generally follows a logarithmic normal distribution.

Horizontal Vibration of Mast Arms

Strain Gauges 3 and 7 were used to measure the longitudinal strains at two sides of the mast arm near the base. The strains are associated with the out-of-plane (horizontal) vibration of the signal support structure. Since their responses are of the same magnitude, only the records at Strain Gauge 3 are used for the following analyses. The

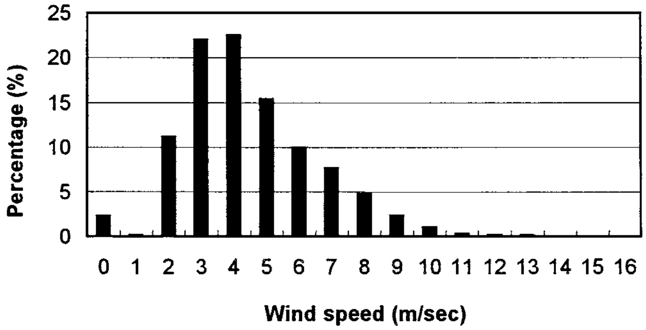


FIGURE 4 Annual wind speed statistics from Columbia, Missouri.

strains measured in the field are converted to stresses by multiplying by the modulus of elasticity of steel materials. The resulting stress-time history is shown in Figure 3.

Load Spectrum due to Natural Wind Gusts

As explained before, the fundamental frequency of the arm structures is significantly higher than the excitation frequency of wind gusts. Therefore, the responses of the mast arm mainly correspond to the vibration of the fundamental mode of the structure as observed from the field tests. To determine the fatigue load on the mast arm, the stress range from peak to valley of each cycle of vibration, as shown in Figure 3, is computed for every event. The minimum stress range considered in the calculation is 1.378 MPa (0.2 ksi) for the Providence and Green Meadows mast arm and 3.445 MPa (0.5 ksi) for the Stadium and Forum mast arm. The total number of cycles can then be counted for each stress level. Figure 5 (*solid bars*) presents the number of stress cycles divided by the total test time as a function of stress level at the arm-post connection, which is referred to as the load spectrum.

To see whether the load spectrum shown in Figure 5 (*solid bars*) is representative for all wind gusts regardless of wind speed, the test data were grouped into three subsets according to the corresponding wind speeds. For the signal support structure at Stadium and Forum Streets, the wind speed of each subset ranges up to 5.8 m/s (13 mph), 5.8 to 7.11 m/s (13 to 16 mph), and greater than 7.1 m/s (16 mph). For the mast arm at Providence and Green Meadows Boulevards, it ranges up to 7.1 m/s (16 mph), 7.1 to 8.9 m/s (16 to 20 mph), and greater than 8.9 m/s (20 mph), respectively. The load spectrum for each subset can be determined in the same fashion. All three spectra are compared with the overall spectrum (*solid bars*) in Figure 5*a* for the Stadium and Forum mast arm and in Figure 5*b* for the Providence and Green Meadows signal support structure. As can be seen, they are very similar in terms of stress distribution though small differences in detail exist at several points. Therefore, the response distribution based on the complete set of test data can be used for other wind speed ranges.

Annual Number of Cycles at Various Stress Levels

The wind speed distribution presented in Figure 4 and the load spectra in Figure 5 are used to predict the number of cycles of vibration a mast arm may be subjected to at different stress levels. For each wind speed from Figure 4, the occurrence frequency at various stress levels can be computed by multiplying the square of the wind speed

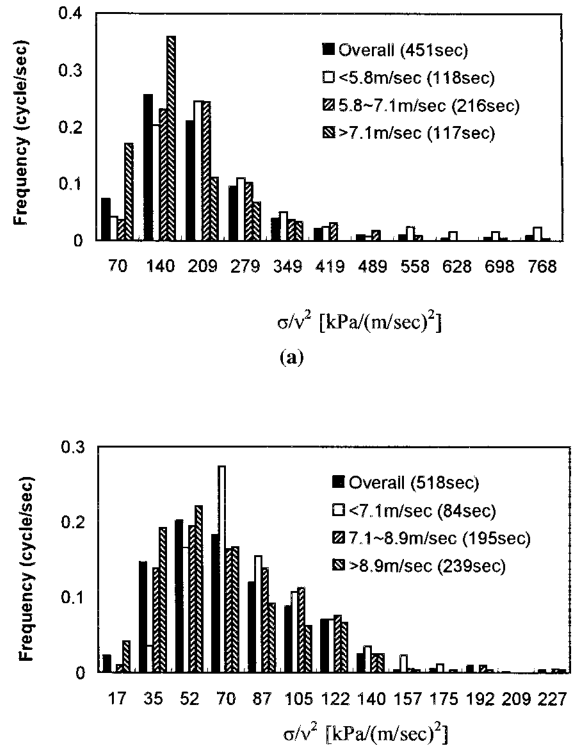


FIGURE 5 Load spectra due to natural wind gusts: (*a*) Stadium and Forum mast arm; (*b*) Providence and Green Meadows mast arm.

by the ratio of stress over the square of wind speed in Figure 5. After every wind speed in Figure 4 is taken once, the occurrence frequencies corresponding to the same stress level are added into the total number of cycles per second. The annual number of cycles of vibration at various stress levels is then obtained. Tables 1 and 2 give the predicted annual occurrence of different levels of stress at the Sta-

TABLE 1 Crack Initiation Life of Mast Arm at Stadium and Forum Streets

Stress Range (MPa)	Annual Loading Cycles	Fatigue Life
4.5	1.404×10^{7}	*
13.7	5.412×10^{6}	*
27.1	2.559×10^{6}	*
45.2	8.401×10^{5}	*
63.4	2.445×10^{5}	*
81.5	1.086×10^{5}	*
99.7	5.767×10^{4}	*
117.8	4.335×10^{4}	*
136.0	1.295×10^{4}	*
154.1	8.216×10^{3}	*
172.3	4.773×10^{3}	*
199.4	5.378×10^{3}	*
235.6	2.133×10^{3}	*
271.9	7.807×10^{2}	5.15×10^{9}
308.0	6.934×10^{2}	1.08×10^{9}
344.3	5.825×10^{1}	2.64×10^{8}
380.6	8.159×10^{1}	7.32×10^{7}
416.9	4.663×10^{1}	2.24×10^{7}

* Number of Cycles > 10^{10}

TABLE 2 Crack Initiation Life of Mast Arm at Providence and Green Meadows Boulevards

Stress Range (MPa)	Annual Loading Cycles	Fatigue Life
2.0	1.742×10^7	*
6.0	5.515×10^6	*
10.9	3.401×10^6	*
17.0	9.522×10^5	*
24.8	4.648×10^5	*
34.8	1.269×10^5	*
44.6	2.814×10^4	*
54.4	1.289×10^4	*
69.4	6.387×10^3	*
89.3	1.207×10^3	*
109.2	2.943×10^2	*
129.0	3.044×10^1	*
148.9	2.029×10^1	*

Number of Cycles > 10^{10}

dium and Forum and the Providence and Green Meadows mast arms, respectively.

Vertical Vibration of Mast Arms

The bending stresses at the top and bottom of mast arms are associated with the vertical vibration of the signal support structures. Such structures are typically more flexible out of plane than in plane. It is likely that the horizontal vibration is stronger than the vertical vibration since its natural frequency is relatively closer to the predominant frequency in wind fluctuation. The field test data at Strain Gauges 1 and 5 confirm that the stress associated with the vertical vibration is less than one-third of that with the horizontal vibration. Therefore, only the bending stress at the side of the arm-post connection needs to be considered for the assessment of fatigue life, at least for the mast arms under consideration.

STRESS CONCENTRATION AT ARM-POST CONNECTION

A computer model (global model) of the entire mast arm structure was used to determine the global distribution of internal forces and moments under a typical natural wind gust. Another computer model (local model) was set up to investigate the local distribution of stress around the arm-post weld connection. A 1.14-m (45-in.) segment of the mast arm was cut, and refined finite element meshes were generated for the segment with a detailed modeling of the weld profile. It was observed from field inspections that the weld leg of the instrumented structures is typically 0.635 cm ($\frac{1}{4}$ in.) long on the base plate and 1.11 cm ($\frac{7}{16}$ in.) long on the mast arm wall. There are over 36,000 three-dimensional solid elements with the minimum element sizing 0.1016 cm by 0.1016 cm by 1.016 cm (0.04 in. by 0.04 in. by 0.4 in.). The external loads on the refined model are the force and moment at the cut section of the entire signal structure from the global model. It is noted that for the octagonal arm, a 1.588-cm (0.625-in.) radius was used to simulate the transition of the mast arm wall around the corner of the octagonal section for realistic modeling. This treatment also eliminates the singularity of the stress amplitude in numerical computation.

The finite element analysis indicates that the longitudinal stress in the direction of the mast arm centerline is always dominant under wind loads from any direction. The maximum stress occurs near the toe of welding at the corner of the octagonal section and at the uppermost point of the circular section. To see this effect clearly, the stress distribution along the centerline of the arm is given in Figure 6. It can be seen that the longitudinal stress is at the maximum at the toe of the weld and rapidly drops to its asymptotic value in the area away from the weld connection. The stress concentration factor, presented in Figure 6, is defined as the ratio of stresses at any point along the mast arm to those at the point far away from the weld toe. The maximum concentration factor is determined from Figure 6 to be 2.88 for the octagonal arm and 2.63 for the circular arm. The Missouri Department of Transportation has recently introduced a so-called fatigue-resistant weld profile with varying slopes. To understand the effect of weld profile on stress concentration, three profiles are selected as shown in Figure 7*a*. Their effects on stress distribution are presented in Figure 7*b*. It can be observed that the actual weld yields the smallest concentration of stress even though its weld leg on the base plate is the shortest, mainly because the actual weld has the longest weld leg on the mast arm wall, whereas the fatigue-resistant weld is the shortest. Therefore, it is concluded that the stress concentration depends on the weld length along the mast arm wall regardless of the slope of a weld profile near the toe, and the fatigue-resistant design could lessen the stress concentration if used to lengthen the weld leg on the arm wall.

PREDICTION OF FATIGUE LIFE OF SIGNAL MAST ARM STRUCTURES

A well-designed engineering structure may potentially fail under low-amplitude cyclic loading in two stages: initiation and crack propagation. In general, the first stage lasts significantly longer than the second stage. The fatigue life of the instrumented mast arms is predicted as follows.

Crack Initiation Under Variable Stress Cycles

The strain-life approach (*5*) is used to predict the life of mast arms in the first stage. It requires the use of the following stress-strain and strain-life relationships:

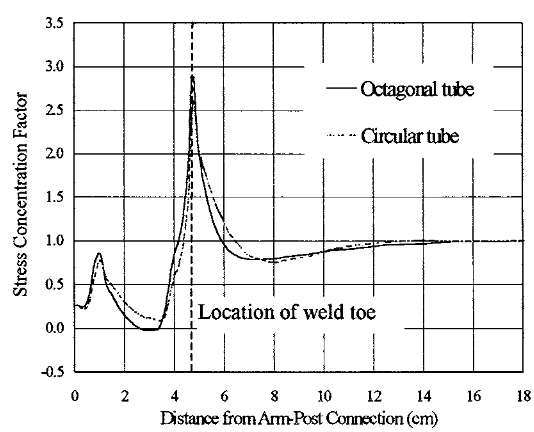


FIGURE 6 Stress distribution along centerline of mast arms.

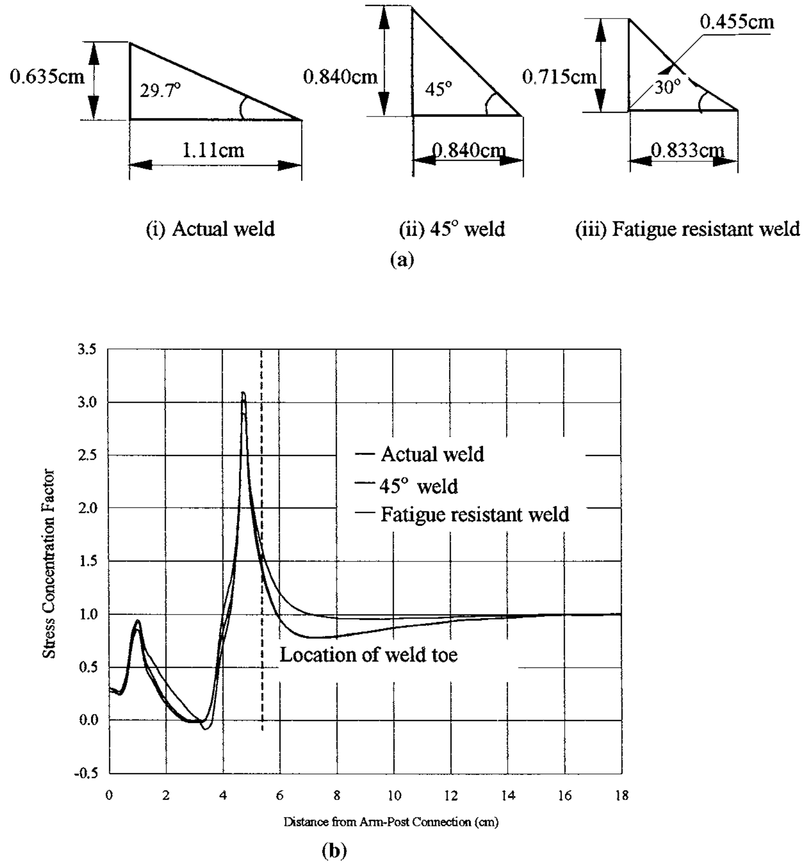


FIGURE 7 (*a*) Various weld profiles; (*b*) stress concentration factors.

$$\frac{\Delta\epsilon}{2} = \frac{\Delta\sigma}{2E} + \left(\frac{\Delta\sigma}{2K'}\right)^{1/n'} \tag{1}$$

$$\frac{\Delta\epsilon}{2} = \frac{\sigma_f'}{E}(2N_f)^b + \epsilon_f'(2N_f)^c \tag{2}$$

where

$\Delta\epsilon$ = cycle strain range,
$\Delta\sigma$ = cyclic stress range,
E = modulus of elasticity,
K' = cyclic strength coefficient,
n' = cyclic strain hardening exponent,
N_f = minimum number of cycles for initiation of crack,
σ_f' = fatigue strength,
ϵ_f' = ductility coefficient, and
b, c = constants in strain life model.

To determine the parameters in the foregoing equations, several flat-sheet specimens of rectangular cross section were made and tested on the MTS810 machine. The test results are presented in Figure 8. The parameters in Equations 1 and 2 are identified by curve-fitting the strain-stress and the strain-life models with the experimental data; they are given in Table 3. The corresponding parameters of A-595A steel determined using the monotonic stress-strain relation and other associated material properties are also given in Table 3. There is general agreement between the two sets of data.

The fatigue life at each stress level in Tables 1 and 2 is determined from Equations 1 and 2 and included in the last column of those same tables. The fatigue lives corresponding to various stress ranges are combined with Miner's rule to determine the fatigue life accounting for variable amplitude stress cycling. On the basis of this rule, both the Stadium and Forum and the Providence and Green Meadows arms are found to be fatigue adequate to survive natural wind gusts.

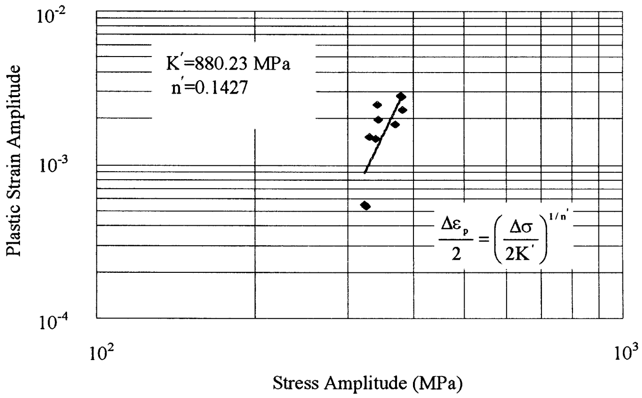
Crack Propagation Under Variable Stress Cycles

Even if there is a small surface crack (defect or discontinuity) near the toe of the weld profile of a mast arm, the arm can still support the external loads until the defect propagates around the cross section of the arm. The crack growth rate is related to the range of the stress intensity factor, ΔK (in megapascals), by the following Paris equation:

$$\frac{da}{dN} = C(\Delta K)^m \tag{3}$$

where

a = crack length (m),
N = number of stress cycles applied to structure, and
C, m = material constants of crack growth model.



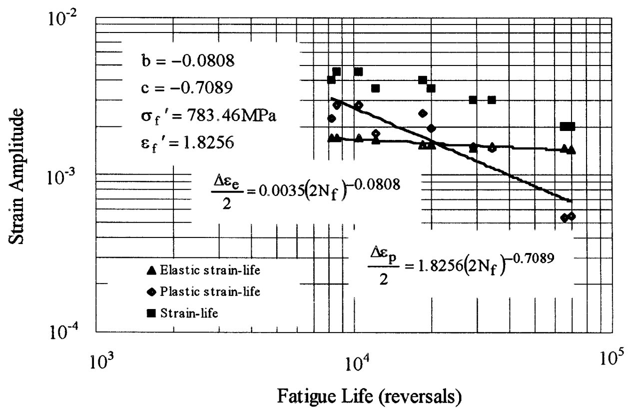


FIGURE 8 Parameters in crack initiation prediction model: (*a*) plastic strain and stress relation; (*b*) strain and fatigue life relation.

To determine these constants, several compact tension specimens were made and tested in the laboratory. The notch on the specimens was oriented at 90 degrees with the test arm axis. The material constants can be estimated as $C = 4.48 \times 10^{-12}$ and $m = 3.11$ from the test data shown in Figure 9.

For a thumbnail surface crack, the stress intensity factor is a function of the stress range at the arm-post connection and the crack size. For any initial crack size, a stress intensity factor can be calculated for each stress level. To account for the variable amplitude loading, the intensity factors corresponding to various stress levels are combined with the root-mean-square rule. The resulting factor is substituted in Equation 3 to determine the crack propagation life in this study. For instance, a 0.127-cm (0.05-in.) crack will lead to the fracture of the Stadium and Forum mast arm after 40 years of service and the Providence and Green Meadows mast arm will not fracture even after 100 years. Even a 0.265-cm (0.104-in.) crack will not cause the collapse of the Stadium and Forum arm in 9 years and the Providence and Green Meadows arm for over 100 years. Thus, it is concluded that the first arm is likely vulnerable to tiny defects around the arm-post weld connection, but the second arm is not unless a visible crack exists.

CONCLUSIONS AND RECOMMENDED FUTURE RESEARCH

From the foregoing analysis, the following conclusions can be drawn:

1. Both the wind speed and the ratio between stress and the square of wind speed follow a logarithmic normal distribution. Since the ratio is insensitive to wind speed, its distribution can be used for weak and strong wind gusts.
2. The average stress in the signal arm structure at Forum and Stadium Streets is significantly larger than that at Providence and Green Meadows Boulevards because of its greater span length.
3. The intensity of the vertical vibration caused by natural wind gusts is less than one-third that of the horizontal vibration.
4. The stress concentrates at the weld toe of the arm-post connection. The maximum stress occurs at the uppermost point of the circular section with a concentration factor of approximately 2.63 and at the corner of the octagonal section with a concentration factor of approximately 2.88. The stress concentration depends on the length of weld leg along the mast arm wall. The fatigue-resistant design proposed by the Missouri Department of Transportation may therefore be helpful.
5. Neither of the instrumented signal structures will crack during their service life. However, the mast arm at Stadium and Forum Streets is likely to be vulnerable to the development of a crack, but the mast arm at Providence and Green Meadows Boulevards is not unless a visible crack has developed.

Although the fatigue-resistant weld profile has been shown by the analytical study to be effective in reducing the stress concentration, its actual performance needs to be verified experimentally. In addition, past studies (*6*) led to the recommendation of new fatigue design specifications, which have recently been approved by AASHTO (*7*). On the basis of the AASHTO specifications, the maximum stress range at the arm-post weld toe was determined to be from 42 MPa for the Stadium and Forum arm to 45 MPa for the Providence and Green Meadows arm for Fatigue Category II. This range is substantially greater than the average stress range of 13 MPa and 5 MPa, respectively, observed from field data. Future study should be directed at the evaluation of more in-service structures based on field data to confirm the code recommendations.

TABLE 3 Parameters in Stress-Strain and Strain-Life Models

Material	E (MPa)	K′ (MPa)	n′	σ_f' (MPa)	ε_f'	b	c
Mast Arm Steel	222.5×10^3	880.23	0.1427	783.46	1.8256	-0.0808	-0.7089
A-595A Steel	199.8×10^3	793.04	0.1463	792.35	0.9943	-0.0878	-0.5~-0.7*

* Estimated based on experience.

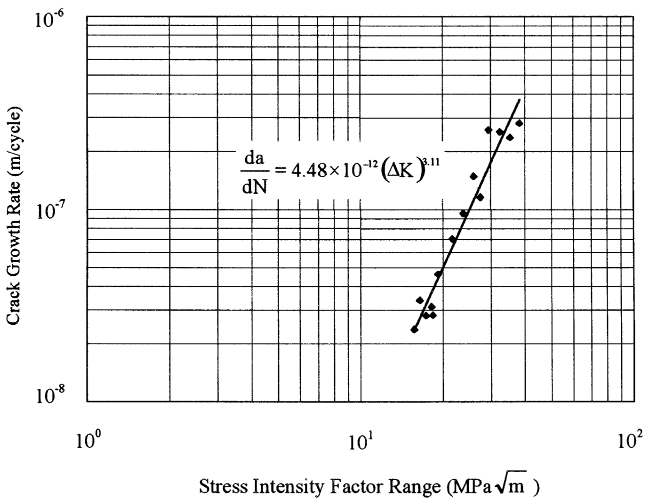


FIGURE 9 Crack growth rate versus stress intensity factor range.

ACKNOWLEDGMENTS

This study was supported by the Missouri Department of Transportation with Vince Imhoff and Tom Allent as Project Coordinators and by the University Transportation Center at the University of Missouri–Rolla. The support is greatly appreciated.

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Publication of this paper sponsored by Committee on General Structures.