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Autor(en): **Bieniek, M.P. / Testa, R.B. / Kratky, R.J.**

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Fatigue Problems in Suspension Bridges: A Case Study

Problèmes dus à la fatigue dans les ponts suspendus: un exemple

Dauerfestigkeitsprobleme von Hängebrücken: Ein Beispiel

M.P. BIENIEK
Professor
Columbia University
New York, NY, USA

R.B. TESTA
Professor
Columbia University
New York, NY, USA

R.J. KRATKY
Consulting Eng.
Weidlinger Associates
New York, NY, USA

H.B. ROTHMAN
Consulting Eng.
Weidlinger Associates
New York, NY, USA

SUMMARY
This paper contains an investigation of the fatigue damage in various components of a suspension bridge. An experimental and analytical program leading to the determination of the range and number of cycles of the live-load stresses in the deck system is presented together with estimates of the crack growth rates at certain critical locations. The effect of a stiffening scheme on the torsion-induced fatigue stresses is outlined.

RESUME
Cet article contient une étude des dommages dus à la fatigue dans les différents éléments d'un pont suspendu. Un programme expérimental et analytique permet de déterminer l'amplitude et le nombre de cycles de contraintes dues à des charges mobiles dans le tablier, de même qu'une estimation de la vitesse de propagation des fissures dans certains endroits critiques. L'effet d'une méthode de raidissement sur les différences de contraintes dues à la torsion est indiqué.

ZUSAMMENFASSUNG
Dieser Artikel behandelt die Dauerschwingfestigkeit verschiedener Teile einer Hängebrücke. Ein experimentelles und analytisches Programm für die Bestimmung der Grenzspannungen und die Zahl der Lastspiele für die Fahrbahnkonstruktion unter Verkehrsbeanspruchungen wird beschrieben und die Rissfortpflanzungsrate an gewissen kritischen Stellen abgeschätzt. Der Einfluss eines Versteifungssystems auf die Wechselspannungen infolge Torsion wird angedeutet.

1. INTRODUCTION

Suspension bridges provide a number of unique and difficult problems of fatigue of their components. The main factors which cause fatigue damage in suspension bridges are the following:

-Due to the effectiveness of the cables in supporting the dead loads, the live loads control the stress levels in the stiffening girders or trusses, the deck systems and the bracings, with frequently occurring total stress reversals.

-The relatively high flexibility of suspension bridges leads to much higher secondary stresses than in any other type of bridges.

-The response of a suspension bridge to wind loading is predominantly oscillatory, i.e. of fatigue causing type.

-Simple increase of cross-sectional dimensions of affected elements leads very seldom to improved stress conditions; in fact, the level of secondary stresses is often increased as a result of such a design change.

This paper contains a study of the first two factors, performed for the case of the Manhattan Bridge in New York City. The Manhattan Bridge was completed in 1909, over the East River, as a link between the boroughs of Manhattan and Brooklyn. A general view and cross-section of this bridge are shown in Fig. 1. The bridge carries unusually heavy traffic loads from three roadways (on two levels) and four subway tracks. The flexibility of the system and the asymmetric loads from the pairs of off-center subway tracks produce large torsional deformations, with one side of the bridge deflected up to 8 feet more than the other. Abnormal deterioration of the bridge components, especially of the floor system, and its relation to the twist was observed within a few years of bridge opening and has continued ever since. In a series of reconstruction jobs, the upper lateral bracings were removed, the upper roadways demolished and rebuilt, suspender ropes replaced, tower links altered, fractured chords and lower laterals restored, and innumerable repairs were made to cracked floor members.

The study described in this paper utilizes a number of tools which became only recently available to the bridge engineer. The investigation of the problems of the Manhattan Bridge consists of the following:

-Detailed computer analysis of the entire structure modelled as a three-dimensional system.

-Strain-gage measurements of stresses at several critical locations, performed under static test loads and, with continuous recording, under typical traffic conditions.

-Fatigue and fatigue crack propagation analysis.

2. STRAIN MEASUREMENTS

Since a preliminary analysis, utilizing a simplified model of the bridge, failed to detect abnormally high stresses even in the areas of the deck system with most severe damage histories, a stress measurement program was initiated. Strain gages were placed on the floor beams of the upper and lower roadways (Fig. 1). The strain gage locations were chosen on elements which were free from cracks, close however to suspected stress concentrations. Fig. 2 shows typical arrangements of the strain gages on the upper and the lower floor beams. All the gages were placed symmetrically on both sides of the floor beams in order to be able to monitor the weak-axis bending.

The strain measurements under static loading were performed with the objective of gaining some basic insight into the working of the bridge and providing a

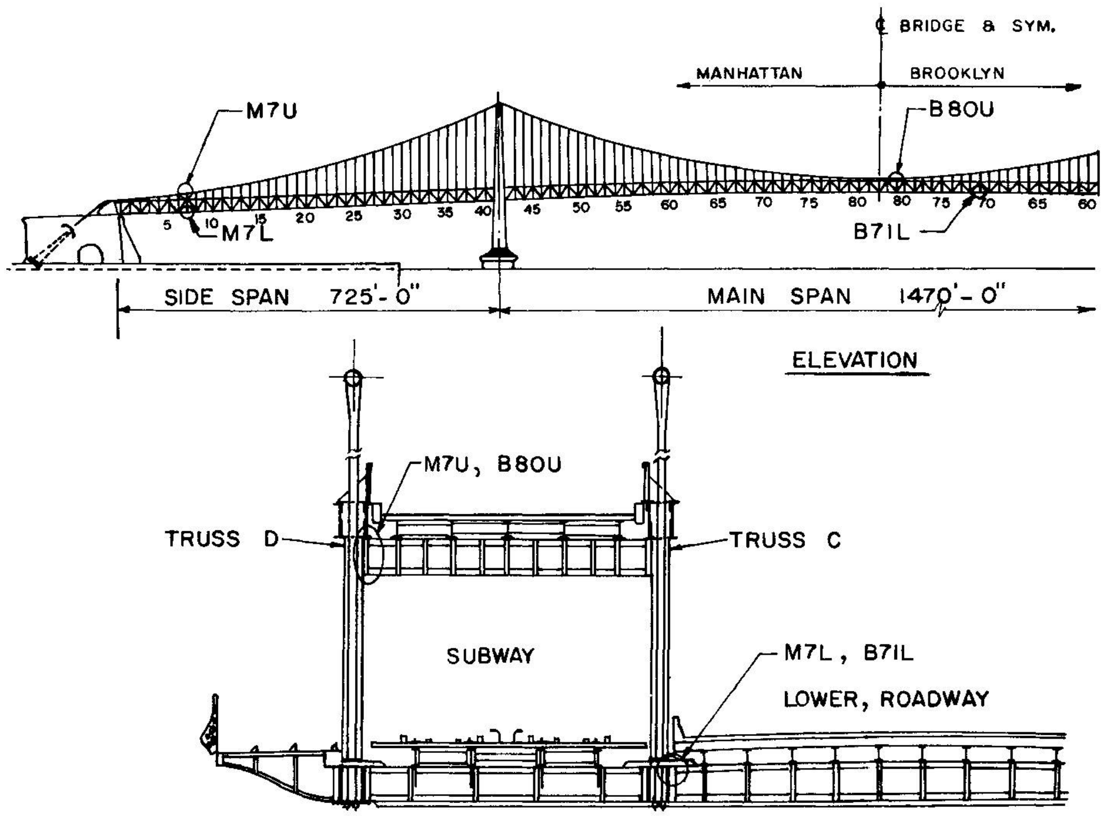

Fig. 1 Manhattan Bridge: Elevation and Typical Cross Section; Strain Gage Locations are Indicated

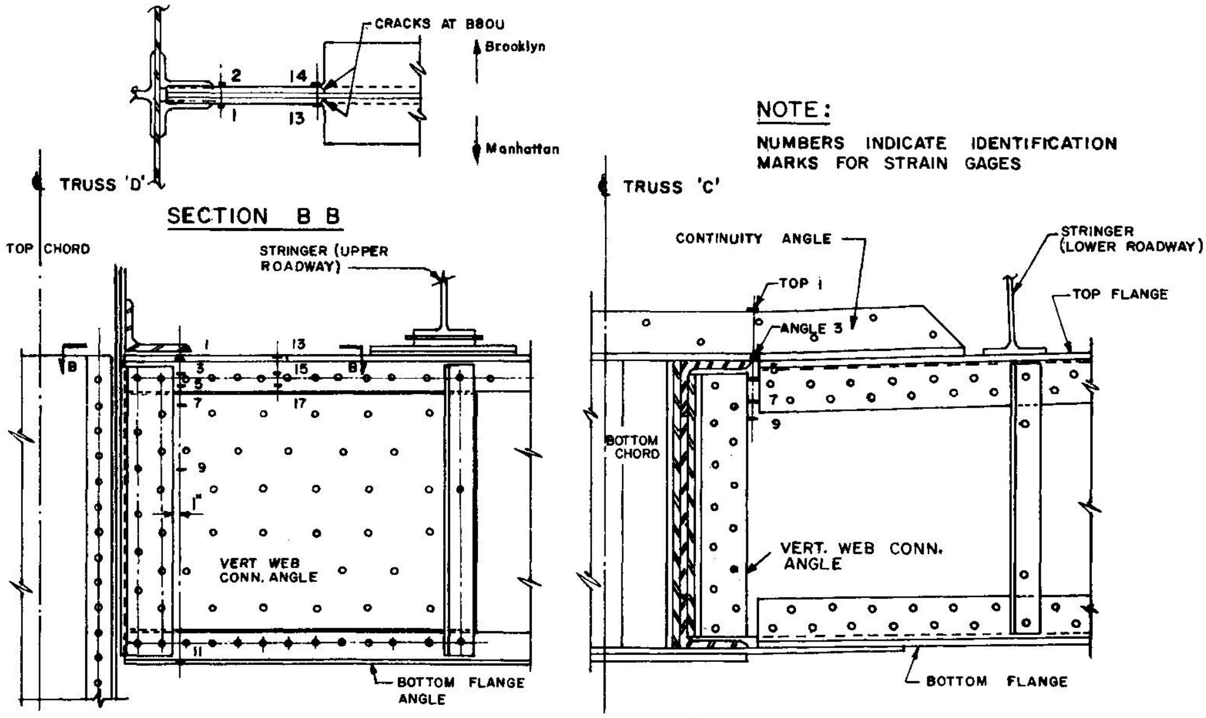

Fig. 2 Strain Gages on Upper (Left) and Lower (Right) Floor Beams

means of verification of future computer modelling of the bridge. The test load consisted of two subway trains (10 empty cars in each train) moving abreast on the tracks between trusses A and B. The trains were stopped at thirteen points along the length of the bridge and the gage readings were taken. Table 1 contains a few representative stress data.

Table 1 Static-load Stresses (psi) in the Upper Floor Beams

Location; Gage No.	Load Position L/2 Manhattan Side Span	L/4 Main Span	L/2 Main Span
M7U; 1	+12,840	-11,370	-4,980
M7U; 2	-11,490	+11,640	+5,190
M7U; 13	- 8,610	+10,230	+2,520
M7U; 14	+11,820	-17,550	-3,480

Since gages 1 and 2, and similarly 13 and 14, are located on opposite sides of the floor beam, the comparison of the stresses at gage 1 and gage 2 (and at 13 and 14) reveals very strong weak-axis bending (Fig. 3). The comparison of gage 1 and gage 13 (and 2 and 14) indicates the bending curvature reversal between the cord of the truss and the first stringer (Fig. 3). This last effect was evidently caused by excessively large friction between the stringer and the floor beam.

In order to obtain data on stresses in the floor beams under normal traffic conditions, continuous recordings of the inputs from several gages were made during normal traffic on the bridge. The recording periods were from 7:30 to 9:30 a.m. (heavy traffic to Manhattan), from 4:30 to 6:30 p.m. (heavy traffic to Brooklyn), and from 11:00 p.m. to 1:00 a.m. (light traffic). Sample traces of strain records are shown in Fig. 4. It was evident that the stresses in the floor beams caused by direct loading by various vehicles were a relatively small fraction of the total stress ranges. The prevailing contributions to the stresses in the floor beams came from the overall deformations of the bridge, with largest peaks generated by coincidental meeting of two or more subway trains at certain locations along the length of the bridge. By counting the stress cycles on the continuous records, stress range histograms were extrapolated to cover a thirty-year period of the bridge life. One of these histograms is shown in Fig. 5, for the gage No. 14 at the location M7U (an upper roadway floor beam).

3. STRESS ANALYSIS

The first-order deflection theory was used for a detailed stress analysis. It was felt that a linearized theory would be sufficiently accurate while allowing use of superposition and taking advantage of the system's symmetry. It was thus possible to model the bridge structure very realistically, as a three-dimensional system. Every panel point was taken into account and the major structural components including the floor beams, were modelled as individual elements. The stringer-floor beam interaction was analyzed on a smaller (local) model for various conditions of the reactions between the stringers and the floor beams. Accuracy of the analysis was confirmed by comparing its stress predictions with the data obtained by the strain-gage measurements.

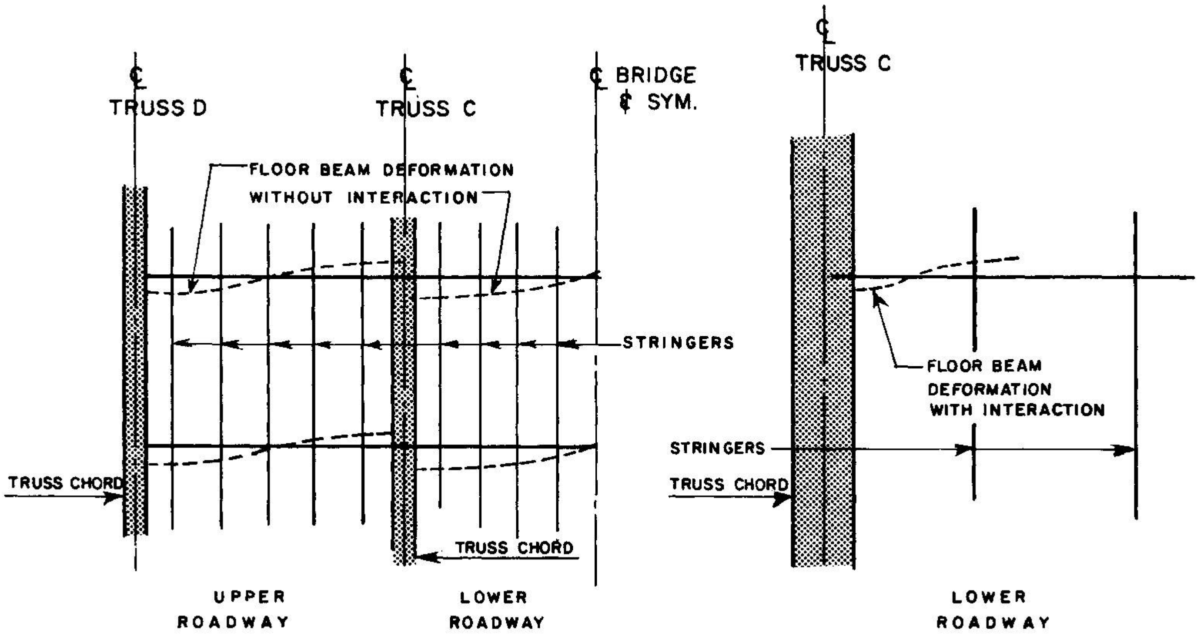

Fig. 3 Weak-axis Bending of the Floor Beams; Figure at Right Shows Aggravated Bending Due to Stringers Interaction

By combining the loads from the four subway tracks and from the roadways, the maximum stresses for strength design and the stress ranges for fatigue design were established. The analysis was performed for the present state of the bridge and for several proposed stiffening schemes. The stress ranges in the upper floor beams determined analytically for the existing configuration and for the proposed stiffening scheme are shown in Fig. 7.

The most important lessons learned from this analysis are the following:
-Three-dimensional modelling of the bridge structure is absolutely necessary if correct determination of stresses and deformations is to be achieved.
-Stresses which may be treated as "secondary" in other types of bridges become often of primary importance in suspension bridges.

4. FATIGUE CRACKING PREDICTIONS

Within the strength of materials methodology, the cumulative fatigue damage in a structure subjected to variable-amplitude cyclic loading can be estimated with the aid of one of the existing rules. Among them, Miner's rule appears to be reasonable and enjoying widespread acceptance. Its use requires the knowledge of the S-N diagram of the material, obtained from constant-amplitude fatigue tests, and the histograms (or "load spectra") of stress ranges imposed on the element to be analyzed. According to Miner's rule, the damage, D, is

$$D = \sum_i n_i/N_i \qquad (1)$$

where n_i = number of actual cycles of stress range S_i;

N_i = number of cycles to failure at stress range S_i.

Failure occurs when D = 1.

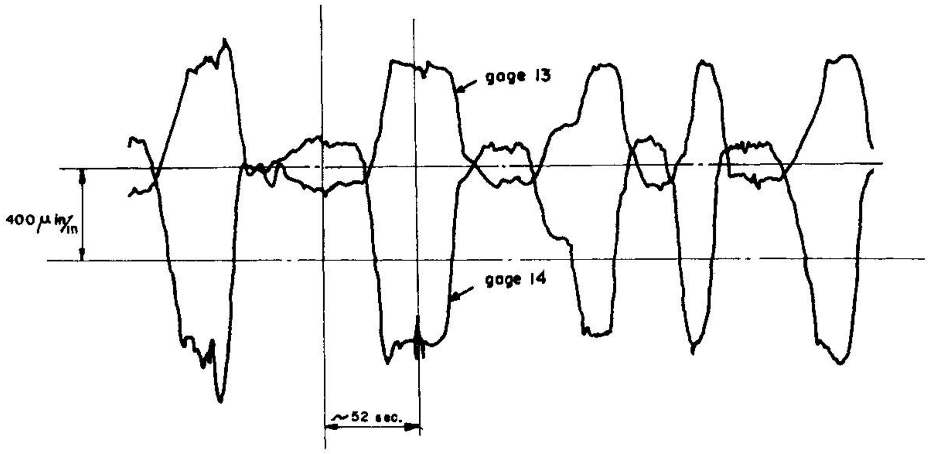

Fig. 4 Samples of Stress Records; Location M7U, Gages 13 and 14

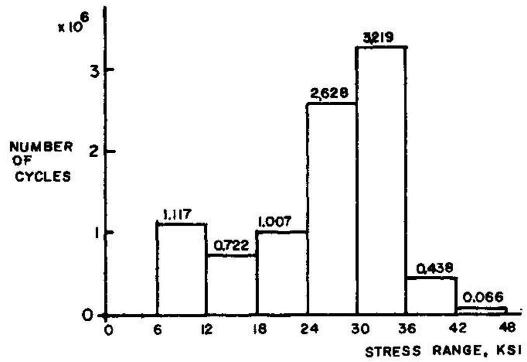

Fig. 5 Histogram of Stress Ranges; Location M7U, Gage 14

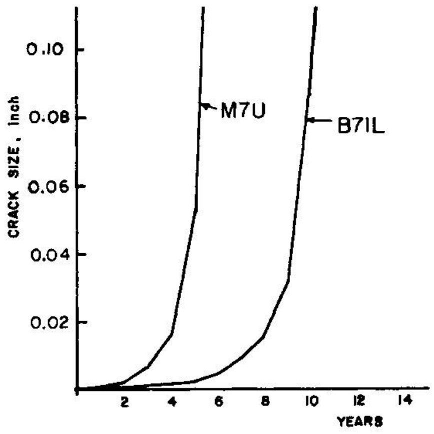

Fig. 6 Crack Growth at Locations M7U and B71L (Initial Crack Length Assumed at 0.001 in.)

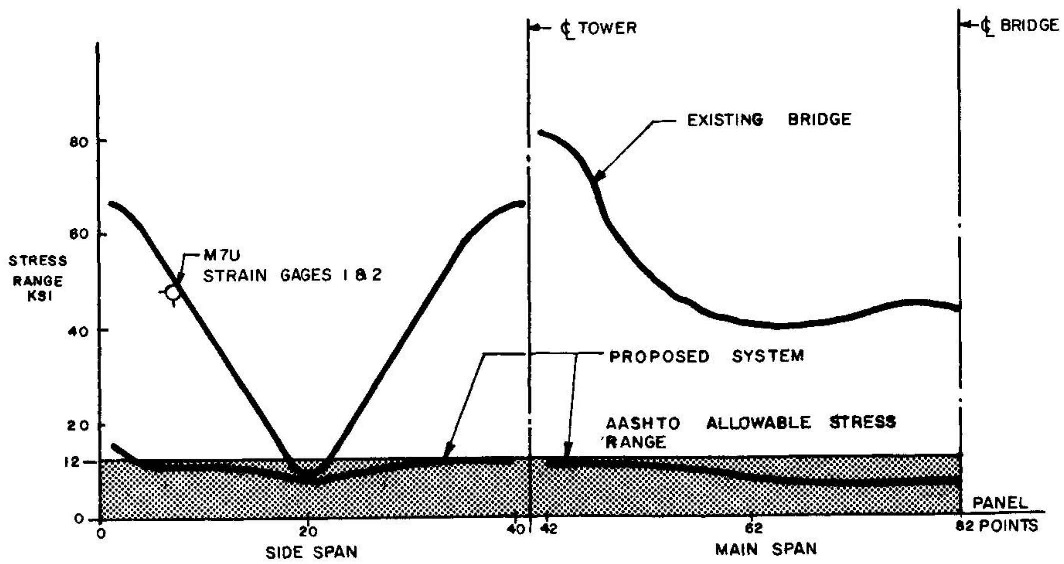

Fig. 7 Stress Ranges at Upper Roadway Floor Beams

Miner's rule has been applied to several locations on the floor beams, for which the experimental stress-range histograms were determined and the S-N curve of rolled A-36 steel was used. The results for 30-year service period are, for ex- example:

Location M7U : D = 0.77
Location B71L : D = 0.14

In reality, cracks in the floor beams in the vicinity of these and other locations were observed after a service period of less than 30 years since the last replacement and rehabilitation work. Thus, the existence of cracks must be attributed to one or a combination of the following factors:
-Initial flaws in the material;
-Initial cracks, notches, and other surface flaws causing local stress concentrations;
-The effects of corrosion;
-The possibility that at some locations the peak stresses were occasionally higher than the levels determined in the present investigation.

The above observations seem to point to the fracture mechanics methods as perhaps the more rational approach to the fatigue cracking problems in bridges. In order to develop an estimate of the rate of growth of hypothetical and, in some instances, existing cracks in the floor beams, the following formula was used ([1], [2])

$$\frac{da}{dN} = C\,(\Delta K)^n \tag{2}$$

where a = crack length, N = number of cycles, ΔK = range of stress intensity factor, C and n = material constants.

For A-36 structural steel, the values of the two material constants were adopted from Ref. [1] (p. 261) as

$$C = 3.6 \times 10^{-10}, \qquad n = 3$$

The stress intensity factor was determined under the assumption of uniform nominal stress in the analyzed element and small length of the crack as compared to the dimensions of the element. Thus, the expression for ΔK is

$$\Delta K = 1.12\ \Delta\sigma\ \sqrt{\pi a} \tag{3}$$

where $\Delta\sigma$ is the nominal stress range. The values of $\Delta\sigma$ with the corresponding number of cycles were taken from the strain-gage records described in Section 2. Integration of Eq. (2) was performed with the aid of the Euler method, with the time interval of 0.5 year (i.e. with the update of the value of "a" in the expression for ΔK on the right-hand side of Eq. 2 at half-year intervals). The initial crack length was assumed to be equal to 0.001 in. Typical results of this analysis are shown in Fig. 6. The general trend indicated by these theoretical predictions appears to be in agreement with the actual observation of the crack growth between two consecutive inspections. Given the erratic nature of some of the effects present in the system (among them, notably, the stick-slip contact between the stringers and the floor beams which causes the situation depicted in Fig. 3), the fracture mechanics approach is, in fact, remarkably successful in explaining the progress of damage of the components in question.

5. STIFFENING SCHEME

A number of stiffening schemes were considered. The solution which possesses the advantage of being effective, economical, and not detracting from the appearance of the bridge consists of strengthening the floor beams and restoration of the lateral bracing system at the level of the two upper roadways. The torsional rigidity of the bridge is thus considerably increased; the two pairs of trusses and the lateral bracings from two very effective torque tubes. The stress ranges in the floor beams of the modified system are shown in Fig. 7, where - for reference - the allowable stress range according to the AASHTO Specifications, [3], is also given.

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