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Stresses in Hanger Plates of Suspended Bridge Girders

JAMES R. BELLENOIT, BEN T. YEN, and JOHN W. FISHER

ABSTRACT

Hanger plates in suspended span bridges are briefly examined for in-plane bending. The hanger plates are designed as tension members; however, measurements indicate that bending occurs as well. This bending is produced by the relative rotation at the ends of the plate and is caused by the frictional bond between the hanger plate, pin, and girder web assembly. A finite-element model of the hanger plates indicates that a nonuniform stress distribution exists across the width of the plate at the pinhole. The maximum stress concentration factors were calculated to be 4.6 and 1.8 for 6.9 MPa (1 ksi) axial and bending stress, respectively. A fatigue strength analysis was conducted to determine the life of the hanger plates.

A frequently adopted arrangement for short- and medium-span steel bridges is the three-span structure with a suspended portion in the middle (see Figure 1). The suspended steel girders are usually connected to the overhanging girders by hinges and hanger plates. This arrangement renders the suspended girders in a simply supported condition.

The hanger plates, which are attached to the girders by pins (as illustrated in Figure 2), are primary bridge components. The plates are designed to undertake the reactions of the rocker supports of the suspended span. Because the pins of hanger plates are assumed to rotate freely, the hangers are assumed to sustain tension forces.

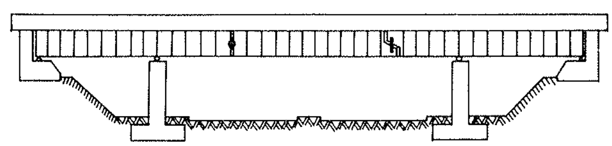

FIGURE 1 Three-span bridge with suspended girders.

In actual cases hanger plates may be subjected to in-plane bending because of friction at the pins, and to out-of-plane bending because of skewness of the bridge or other reasons. Broken hanger plates have been found in bridges (1). Some results of a brief study on in-plane bending of hanger plates are presented here.

MEASURED STRESSES

Because hanger plates are assumed to take tension forces, live-load stresses in hanger plates are expected to be tensile in nature. Measurements by electrical resistance strain gauges on hanger plates of two highway bridge girders indicated that strains varied from tension to compression, or vice-versa, as vehicles traversed the bridges (2).

Examples of recorded strain-time traces are shown in Figure 3. These results imply that the hanger plates were subjected to more than simple tension.

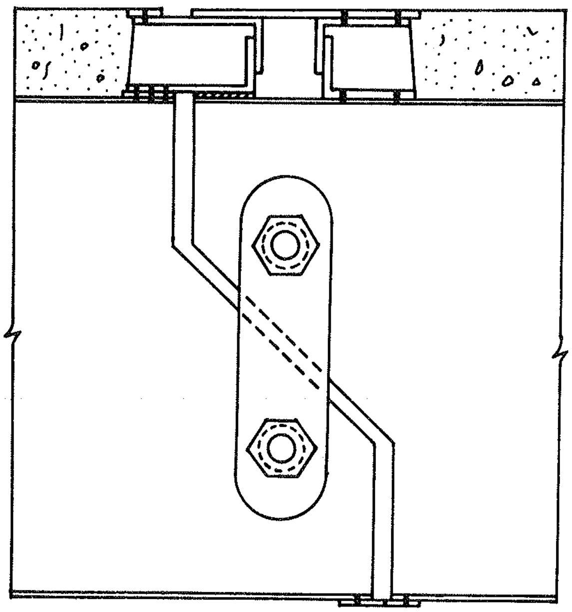

FIGURE 2 Hanger plate.

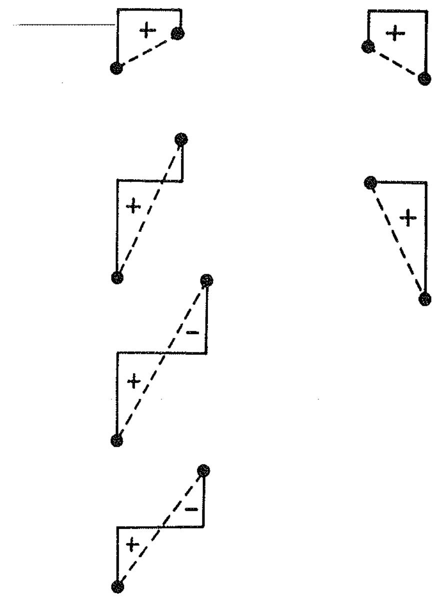

FIGURE 4 Stress distribution across hanger plate width.

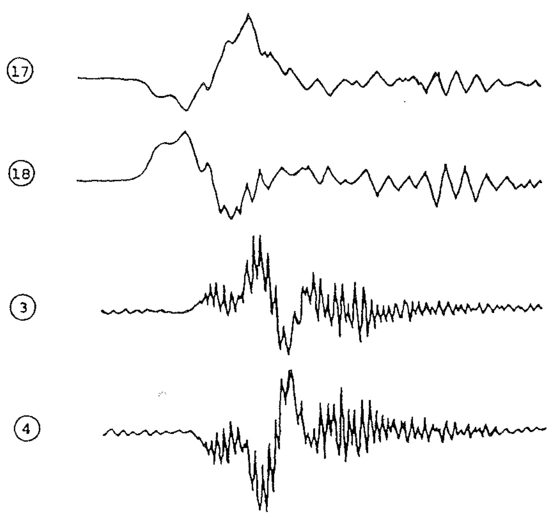

FIGURE 3 Recorded strain traces.

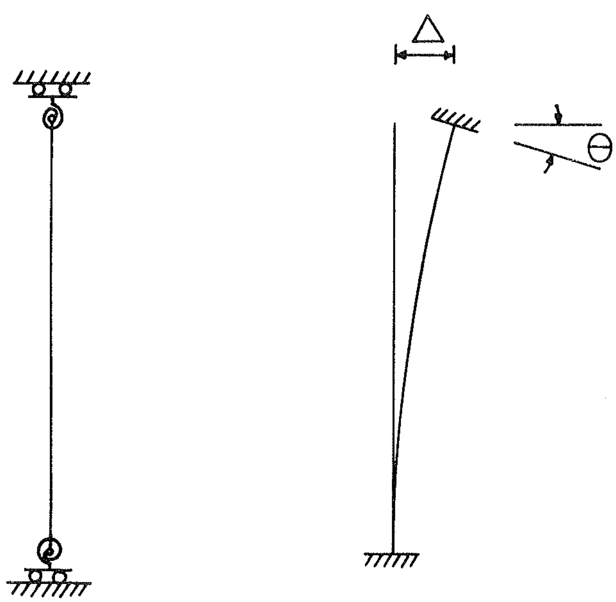

FIGURE 5 Model for hanger plate analysis.

From the strain traces, stress distributions in hanger plates were established. Examples are plotted in Figure 4. These are estimated stresses across the hanger plates at a specific instance. The distributions clearly indicate that the hanger plates were subjected to bending in addition to axial forces. In other words, the hanger plates were not acting as simple tension members, as assumed.

CAUSE OF BENDING STRESSES

A hanger plate subjected to tension and in-plane bending may be modeled as a pin-connected tension member with restraints against end rotations (see Figure 5). The restraints could be induced by friction at the pins or bonding between the hanger plates and adjacent girder webs. When relative rotation or transverse displacement between the pinholes occurs, the hanger plate undergoes bending.

The relative rotation and transverse displacement of a hanger plate in a bridge are difficult to calculate. Their magnitudes depend on the geometry and configuration of the hanger plates and pins, as well as on the arrangement of the bridge girders and the roadway deck in the vicinity. If it is assumed that the hanger plates rotate with the girders such that the relative rotation between the ends of the hanger plate is equivalent to that between the ends of the cantilever and the suspended girder, and that the transverse displacement is compatible to this rotation, then the bending stresses in the hanger plate would be directly proportional to the relative rotation.

This postulation can be examined by comparing the influence lines of the relative rotation of the girder ends and the recorded live-load strains in the hanger plates. Figure 6 shows such a comparison. The recorded strains at a point of a hanger plate

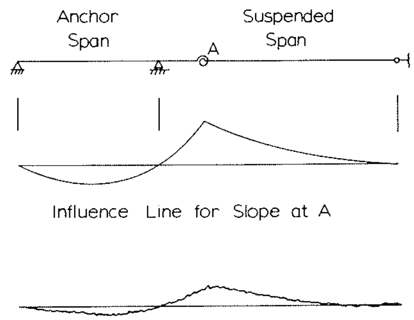

FIGURE 6 Influence line for slope versus recorded strain trace.

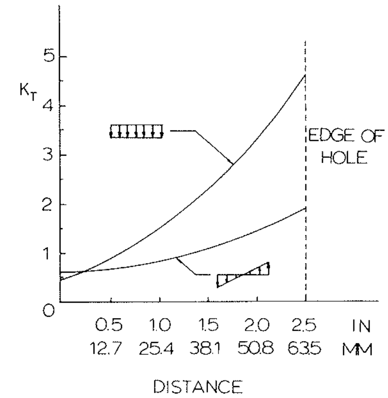

FIGURE 7 Stress distribution at pinhole.

were produced by a single truck traversing a bridge at a crawl speed. The similarity of the curves is obvious.

To explore this further, the relative rotations at the 12 hanger plates of 6 girders of a bridge are estimated. The axial forces in the hanger plates, evaluated from measured strains in the plates, are applied to the respective girders. The resulting rotations are given in Table 1, together with the

TABLE 1 Bending Moment in Flange Plates

Girder	Hanger Plate	θ	$M_{comp.}$	$M_{meas.}$
G1	W	0.000451	22	0
	E			8
G2	W	0.000623	30	2
	E			0
G3	W	0.000935	45	8
	E			6
G4	W	0.000887	40	27
	E			24
G5	W	0.000216	10	33
	E			6
G6	W	0.000113	5	13
	E			0

respective computed bending moments, assuming that the ends of the hanger plates are bonded to the girder webs. The corresponding bending moments from measured strains are also listed. By comparing the computed and measured bending moments it is evident that the bending of the hanger plates is less than the computed value. Nevertheless, the hanger plates do appear to have rotated with the girders and to have sustained bending moment.

ESTIMATION OF FATIGUE STRENGTH

Under tensile axial forces and bending moments, live-load stresses at the pinholes of hanger plates could be quite high. The result is possible fatigue crack growth at the pinholes. To estimate the fatigue endurance, it is necessary to know the stress magnitudes and their variation with time (3).

The stress distribution at a pinhole of a hanger plate is evaluated by a finite-element analysis. The results are shown in Figure 7. The stress concentration factor (K_T) is 4.6 at the edge of the pinhole for a uniform axial stress, and is 1.8 for a maximum bending stress of 6.9 MPa (1 ksi). The nonuniform nature of stresses in the hanger plate produces nonsymmetrical stresses at the pinhole. For the case shown, the axial and maximum bending stresses are equal, where the highest stress at the edge of the hole is 3.2 times the highest stress in the plate.

The variation of stresses at a pinhole can be estimated from the stress history of the hanger plate. For the plate discussed in Figure 7, the stress histogram from measurement is shown in Figure 8. Most of the stress ranges (live-load stresses)

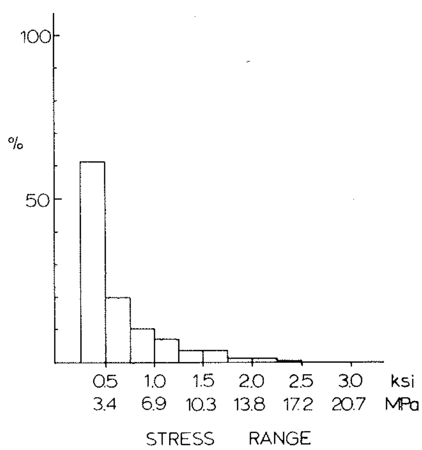

FIGURE 8 Histogram of stress range.

were low, with a maximum value of 20.7 MPa (3.0 ksi). By assuming that the stress distribution shown in Figure 7 is applicable all the time, the maximum stress range at the pinhole would be 66.2 MPa (9.6 ksi).

The fatigue strength of hanger plate pinholes has not been defined. Conservatively, category C of AASHTO allowable stresses may be adopted (4). For multigirder bridges, the threshold value for category C is 69.0 MPa (10 ksi). Because the maximum live-load stress could be higher than this threshold, fatigue crack propagation must be considered.

Results of studies have indicated that fatigue crack growth in ferrite-pearlite steels follow the rule (5) that

$$da/dN = 3.6 \times 10^{-10} \quad (\Delta K)^3 \tag{1}$$

where

a = crack size,
N = number of cycles,

ΔK = range of stress intensity factor = $f(a) S_r \sqrt{\pi a}$,
S_r = constant amplitude stress range, and
$f(a)$ = correction factor for crack shape, stress gradient, and so forth (6).

Equation 1 may be rearranged and integrated to give an estimated life (N):

$$N = \int dN = \int_{a_i}^{a_f} \left(da / \{ 3.6 \times 10^{-10} [f(a) S_r \sqrt{\pi a}]^3 \} \right) \quad (2)$$

For the hanger plate shown in Figure 7, an initial corner flaw of a_i = 2.54 mm (0.1 in.) is assumed with a detectable final crack size of a_f = 25.4 mm (1.0 in.). The constant amplitude stress range is estimated from the stress histogram by using Miner's hypothesis and is equal to 5.8 MPa x 3.2 = 18.5 MPa (2.7 ksi). By incorporating the stress gradient of Figure 7 into an expression for the connection factor f(a), the resulting estimate life is 506 x 10^6 cycles. If 2,000 cycles per day are induced by trucks, it would take many years for the crack to grow. Thus, if the hanger plate is made of steel with adequate toughness against brittle fracture, there should be ample time for inspection if a crack would ever develop.

CONCLUSIONS

In conclusion, the following points are restated.

1. Hanger plates of suspended bridge girders are subjected to bending as well as axial forces.
2. In-plane bending results from friction at the pin and the relative rotations of the girders at the hanger plate.
3. Live-load stresses at the edge of pinholes are higher than those in the hanger plates.
4. Fatigue cracks could grow from pinhole edges.

For the case studied, there is ample time for inspection. Further studies are needed on the adequacy of current design assumptions and, particularly, on the behavior of plates in relation to girder geometry and bridge dimensions.

ACKNOWLEDGMENT

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Design of the Cable-Stayed Girder Weirton-Steubenville Bridge

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ABSTRACT

When completed, the Weirton-Steubenville Bridge will be the sixth cable-stayed girder bridge constructed in the United States. The design to be constructed at a $20 million cost was chosen in 1983 from three bridge designs presented for construction bids. Crossing the Ohio River between Weirton, West Virginia, and Steubenville, Ohio, the new bridge will be 1,965 ft from abutment to abutment and have a main span of 820 ft. A concrete, inverted Y-shaped tower, which rises 365 ft above the supporting pier, features above its apex a 140-ft-high pylon that supports a dual-plane cable system. Materials specified for the composite bridge were placed where their properties would provide the greatest advantages without sacrificing integrity and function. Fascia girders are I-girders with webs skewed at 10 degrees from the vertical, thus reducing cable-connection eccentricity, material quantities, and steel fabrication costs. The composite superstructure consists of longitudinal stringers, transverse floor beams, and a concrete deck--all treated as an orthotropic system. Further,