Simplified Design Check of Thermal Stresses in Composite Highway Bridges

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A simple formula intended for use as a design check of thermal stresses in simply supported composite highway bridges is described. It is based on a series of field tests of various bridges ranging in span from 47 ft 3 in. to 71 ft 5 in. The formula relates the thermal stress at the bottom of the girder to the temperature difference between the top and bottom of the slab and the depth of the bridge.

• ALTHOUGH a small body of literature exists on thermal stresses in composite beam construction (defined as a flexural member consisting of a steel beam with a concrete slab firmly attached to its top) the quantitative analysis contained in such literature is generally too complex to be of direct use to bridge designers (see Refs. 1 and 2 for all related references). The few existing specifications contained in the construction codes of various organizations are little better. Germany's code (DIN 1078, 1958) for temperature effects in composite construction is as follows:

A. Indeterminate structure: a straight line variation of $\pm 15°$ C shall be assumed between the top of concrete slab and bottom of steel girder.
B. Statically determinate structures: the thermal effect shall be allowed for by an additional shrinkage of 10×10^{-5}. These stresses shall be combined with live load stresses as follows: (a) Full live load + half temperature difference. (b) Full temperature variation and live load reduced by 1% per meter of span to 40 meters span, then constant 40% reduction.
C. Shearing forces due to temperature difference may be distributed as a triangular shearing force diagram at the end of the girder with a length equal to the effective slab width. (In association with this shearing force the code also specifies the use of heavy end anchorages tieing the slab and beam together at their interface.)

Austria, Sweden, and Japan are the only other countries with a thermal stress provision in their codes, and these codes are all essentially based on the German code. The United States has no direct provision, although Section 1.2.15 of the 1961 AASHO Standard Specifications for Highway Bridges states: "Provision shall be made for stresses or movements resulting from variations in temperature."

Obviously, none of the existing codes gives the designer any direct information on how to convert temperatures to stresses, on which all elastic design is based. For this reason, a series of field tests were conducted by the Virginia Council of Highway Investigation and Research in cooperation with the U. S. Bureau of Public Roads in 1964 to determine, if possible, a simple method to predict thermal stresses.

FIELD STUDIES

Six simple span composite bridges (all at least 2 yr old) were selected for testing. They ranged from 47 ft 3 in. to 71 ft 5 in. in span, as indicated in Table 1, and all had

Paper sponsored by Committee on Bridge Design.

TABLE 1

COMPOSITE BRIDGES INVESTIGATED

Va. Bridge	Avg. Slab Thickness (in.)	Girder Spacing (ft)	Girder Size	Bridge Span C. to C. (ft)
On Rt. 671 over Rt. 11-A at Lexington	8	$8\frac{5}{12}$	24 WF 100 with 10- × $\frac{3}{4}$-in. lower cover plate	$47\frac{1}{4}$
On Rt. 635 over Pedlar River at Pedlar Mills	$7\frac{1}{2}$	$7\frac{2}{3}$	33 WF 141	$51\frac{5}{12}$
On Rt. 252 over Hays Creek at Brownsburg	$7\frac{3}{4}$	$7\frac{2}{3}$	36 WF 160	56
On Rt. 256 over South River at Grottoes	$7\frac{1}{2}$	$8\frac{1}{6}$	36 WF 150 with $10\frac{1}{2}$- × $\frac{5}{8}$-in. lower cover plate	$61\frac{1}{4}$
On Rt. 250 over South River at Waynesboro	7	$6\frac{1}{2}$	36 WF 150 with 11- × $\frac{3}{4}$-in. lower cover plate	69
On Rt. 257 over Dry River near Dayton	$7\frac{3}{4}$	$7\frac{2}{3}$	36 WF 170 with $10\frac{1}{2}$- × $\frac{9}{16}$-in. and 9- × $\frac{3}{8}$-in. lower cover plates	$71\frac{5}{12}$

conventional Lubrite plate bearings. On each bridge an interior girder was instrumented for strain reading by a 10-in. Whittemore gage at the upper and lower flanges. (Facia girders were not instrumented in this series of tests as they are subject to unusual temperature conditions caused by solar radiation on the lower flanges and are generally over-designed.) Numerous strain readings were periodically taken under dry, zero live-load conditions. Because of its age, the concrete was assumed to be in a post-shrinkage state. The Whittemore gage was calibrated by an invar bar before each set of readings. Simultaneously with the strain readings, temperatures at the top of the slab and at the upper and lower flanges of the girder were also taken with a quick response surface thermometer. An infrared thermometer, which proved greatly superior to the surface thermometer, was used in the latter portion of the studies.

From these data and the following equation, thermal stresses at the measured positions in the girder could be obtained:

$$f = eE - cET \tag{1}$$

where

f = thermal stress in girder;
e = measured strain;
E = modulus of elasticity (30×10^6 psi);
c = coefficient of expansion ($6.5 \times 10^{-6}/^\circ$F); and
T = temperature change.

Except for regions close to the end of the girder, thermal stresses are essentially constant along the length of the beam.

For comparative purposes, temperature changes, T, were related to conditions early in the morning, when the entire bridge was almost at a uniform temperature state.

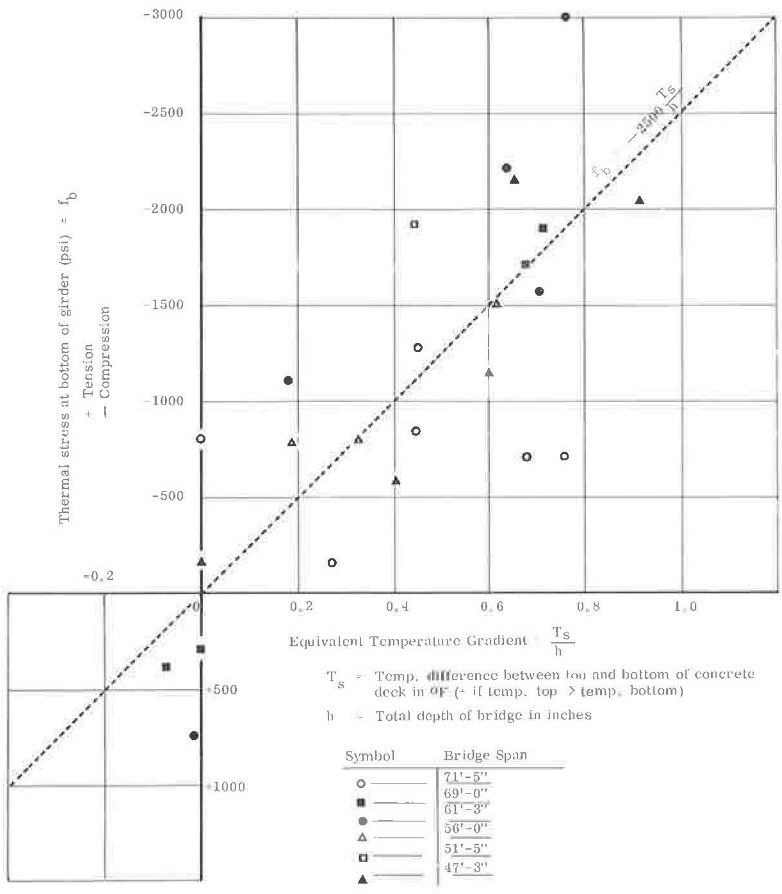

Figure 1. Plot of measured stress vs temperature gradient.

The initial simplifying approach taken was to reduce the more complex theoretical equations cited at the beginning of this paper to a few basic parameters by the process of eliminating small order terms. This process lead to two dimensionally correct equations in terms of the interface shear, F, and the interface moment, Q, as in Eqs. 2 and 3:

$$F = k E_c A_c T \tag{2}$$

and

$$Q = k_1 h F \tag{3}$$

where

E_c = modulus of elasticity of concrete;
A_c = area of concrete slab;
T = temperature changes from top of slab to bottom of girder;
h = total depth of bridge; and
k_1 = experimental nondimensional constants.

It was hoped that if a consistent set of k and k_1 values could be found, all thermal stresses in the bridge could thereby be computed from the known F and Q. However, the scatter of these constants proved too great for meaningful analysis. Therefore, as a next best course, thermal stresses in the lower flange (those generally controlling design) were plotted against the equivalent temperature gradient T_s/h, where T_s is the temperature difference between the top and bottom of the concrete slab and h is the total depth of the bridge. The results are shown in Figure 1. Although there is more scatter about the median line than is generally desirable, a trend does seem evident.

Three possible causes of the scatter are (a) neglect of secondary parameters such as material property variations and span length; (b) experimental error in measuring both the strain and the temperature (which could account for as much as ± 500 psi); and (c) the nonlinear nature of the friction at the movable bearing. Depending on the tendency of the girder to move thermally one way or the other, the friction restraint could cause the stresses to be raised or lowered as much as several hundred psi. The limiting sliding value of the coefficient of friction of 0.1 of Lubrite plates cannot always be assumed because full sliding may not actually be occurring at the time of measurement. To substantiate the friction hypothesis, a random selection of experimental data was compared with the author's more complex theories (previously cited), and it was found that much better agreement between theory and experiment could be obtained if bearing restraint was introduced into the force system.

Although the use of more sensitive and expensive instrumentation would probably reduce the scatter, it is believed that because of the action of statistical balancing, the position of the final median line would not be significantly affected.

CONCLUSIONS

Until a better design method is found, it is suggested that thermal stresses in simply supported composite highway bridges can be approximately checked by data obtained from this study.

Since thermal stresses are generally of secondary magnitude, the following formula is believed reasonable:

$$f_b = 2,500 \frac{T_s}{h} \tag{4}$$

where

f_b = thermal stress in bottom flange of girder (+ if tension, - if compression), psi;
T_s = temperature difference between top and bottom of slab (+ if temperature at top is greater than temperature at bottom, and - if temperature at top is less than temperature at bottom), ° F; and
h = total depth of bridge (slab + girder), in.

The value of T_s must be obtained on a regional basis, since the climate of the United States is too varied to permit specification of a fixed value. For guidance, however, it may be said that from tests conducted on bridges in the region of Charlottesville, Va., the maximum $+T_s$ is about 40 F, occurring in the summer, and the maximum $-T_s$ is about 10 F, occurring in the winter. For interior girders, the entire girder is at approximately a constant temperature equal to that at the bottom of the slab, so T_s may also be interpreted as the temperature difference between the top and bottom of the bridge.

ACKNOWLEDGMENTS

Grateful acknowledgment is made to the Virginia Council of Highway Investigation and Research and to the U. S. Bureau of Public Roads for their joint sponsorship of this study. Special mention is also made of the assistance of James Martin.

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