INFLUENCE OF TEMPERATURE ON HIGHWAY BRIDGE STRAIN MEASUREMENTS USING VIBRATING WIRE GAGES

Jeffrey A. Laman, Ph.D, PE: Associate Professor, Penn State University, State College, PA

J. Barrie Sellers, Ph.D.: President, Geokon Inc. Lebanon, NH

Jeffrey L. Schulz, P.E. Bridge Diagnostics, Inc. Boulder, CO

Introduction

When long-term field measurements are recorded, the effects of temperature changes on both the instrumentation and the structure must be quantified. Depending on the member material and boundary conditions, the apparent, or measured, strains will vary. In some cases the temperature-induced load effects on a structural member can be isolated through a relatively simple calculation, and in other cases, it must be determined experimentally. This paper focuses on how to address temperature effects when using vibrating wire (VW) strain gages for long-term structural monitoring on highway and railroad bridges.

Vibrating wire strain gages are made of steel and will, therefore, respond to temperature changes in the same way steel structural members respond. The first item the user should be aware of is the difference in thermal inertia between a VW strain gage and a structural member. What this means is that a VW gage, while attached to a girder, if left directly exposed to wind or sunlight variations, will contract or elongate much faster than the girder. Even if a strain gage is designed to be self-compensating for temperature changes, its *rate* of deformation change will be different than that experienced by the girders, meaning that apparent strains will be indicated by the gage. Put another way, since the two ends of the gage are anchored to the girder and for some reason the ambient temperature rises (sun coming out from behind a cloud, etc.), the wire in the gage will expand at a much faster rate than the girder, and the effect is to reduce the tension in the wire. The opposite is true if the temperature gets cooler. The only real way to minimize this thermal inertia effect is to cover and insulate the gage in order to include it as part of the monitored mass.

The effect of temperature on both the gage and the member is described below for both steel and concrete structures and for the two most common boundary conditions found on bridge structures.

Evaluating Temperature Effects on Free-Ended Members

The first boundary condition to be considered is a free-ended, or the no-restraint, condition that is typical for bridges that have an expansion joint at least at one end. Temperature effects on free-ended girders are usually not of much interest since there is no stress induced in the members with this boundary condition. Because stress is usually the desired parameter, most strain gages, including VW gages, have been designed to be self-compensating for temperature-induced strains to the extent possible. For this discussion, self-compensating refers to the ability of the gage to respond to “other” stresses in the member only, even if strains are occurring due to temperature. In other words, temperature-induced strains are automatically removed from the gage readings. In the following discussion, compressive strains are negative and tensile strains are positive.

1. Ends of a steel member are not restrained: any change in temperature will not result in either a compressive or tensile stress in the member. However, a strain will occur as follows:

$$\varepsilon_{Thermal\text{-}Steel} = (\boldsymbol{T_1}\text{-}\boldsymbol{T_0}) \times \boldsymbol{CF}_{Steel} \qquad (1)$$

where $\varepsilon_{Thermal\text{-}Steel}$ is the strain in a steel member, T_0 is the temperature when the initial reading was taken and T_1 is the temperature when the final reading was taken, and CF_{Steel} is the coefficient of thermal expansion for steel. If a self-compensating VW gage is attached to the steel member, this strain will not be indicated on the readout device because the wire inside the gage has expanded or contracted at the same rate as the steel member (assuming that the thermal inertia effects mentioned earlier have been minimized). The gage reading will be zero because the wire requires a change in its tension force in order for it to record any change in frequency.

Therefore, if other loads are applied to the member, the change in reading from the gage will be directly related to the stresses induced by loads only and is calculated by equation (2):

$$\boldsymbol{\sigma}_{Loads\text{-}Steel} = (\boldsymbol{R_1}\text{-}\boldsymbol{R_0})\boldsymbol{G}\boldsymbol{E}_s \qquad (2)$$

where $\boldsymbol{\sigma}_{Loads\text{-}Steel}$ is stress in the steel member induced by the loads only , R_0 and R_1 are the initial and final readings, G is gage batch calibration factor, and E_s is Young's modulus of steel. The batch calibration factor is supplied by the VW strain gage manufacturer. To recap, since the end of the girder was totally free and the gage self-compensating for temperature, the stresses due to the additional loads can be easily determined from the before and after gage readings.

2. <u>Ends of a concrete member are not restrained:</u> As was the case with the steel girder example above, any change in temperature will not result in either a compressive or tensile stress in the member however, a strain will occur since the concrete girder can freely expand and contract due to temperature changes. This strain can be calculated by:

$$\varepsilon_{Thermal\text{-}Concrete} = (T_1 - T_0) \times CF_{Concrete} \qquad (3)$$

where $\varepsilon_{Thermal\text{-}Concrete}$ is the thermally-induced strain in the concrete and $CF_{Concrete}$ is the coefficient of thermal expansion for concrete. It should be noted that $CF_{Concrete}$ is usually just an estimate as it can change over time and even vary throughout the same structure due to inconsistent mixing of the concrete before pouring. Regardless, theory holds that if a VW gage is attached to the concrete member, a change in strain WILL be measured because the steel wire inside the gage has expanded or contracted at a slightly different rate than that of the concrete beam due to the difference in coefficients of thermal expansion. The measured strain will not be zero because not all of the deformation was compensated for by the gage. Therefore, the apparent (measured) strain will be as follows:

$$\varepsilon_{Measured\text{-}Concrete} = -(T_1 - T_0) \times K \qquad (4)$$

In the above equation, K is the difference between the two coefficients of expansion as follows:

Table 1. Coefficients of Thermal Expansion

Steel:	12.2 ppm/°C	6.7 ppm/°F
Concrete:	≈10 ppm/°C	≈5.5 ppm/°F
Difference (K):	2.2 ppm/°C	1.2 ppm/°F

Note that this is approximately 20% of the actual change in strain. This quantity is negative because the steel wire elongated more than the corresponding concrete. This causes a reduction in the wire tension and a corresponding lower vibrational frequency.

If additional loading has taken place, the stresses due to these loads can be calculated by correcting the measured strains as follows:

$$\sigma_{Loads\text{-}Concrete} = ((R_1 - R_0)G + ((T_1 - T_0) \times K))E_c \qquad (5)$$

where $\sigma_{Loads\text{-}Concrete}$ is the stress induced by the loads only and R_0, R_1, T_0 and T_1 are the initial and final strain and temperature readings, G is the batch calibration factor, and E_c is Young’s modulus of concrete. Notice that the sign for the correction is positive. This would normally be negative, but because the correction value of Equation (4) was negative, a positive value results.

Summary of Temperature Effects on Free-Ended Members

When strain gages, VW or otherwise, are designed to be self-compensating for temperature, they theoretically will not register strain on free-ended members due to temperature changes. The exception is the concrete example above where the gage is not completely compensating because the materials are different. Therefore, for free-ended members, temperature-induced deformations must be calculated rather than measured. Self-compensating strain gages can be thought of in terms of stresses as well. As the above situation illustrates, temperature-induced strains occurred, but stresses did not, and indeed, the gages (at least on steel) read zero change. However, since there is a difference in the coefficients of thermal expansion between the concrete member and the steel gage, the gages will register about 20% of the actual thermally-induced strain.

Evaluating Temperature Effects on Members with End Restraints

A good example of a structure with significant end restraints would be an integral abutment bridge. For this end condition, the temperature-induced strains will be directly related to the internal forces of the beams. Note that it is very difficult to obtain a fully-restrained (meaning no displacement allowed at all) bridge system due to the huge temperature-induced forces involved. One complication in this situation is the temperature gradient between the top and bottom of the typical bridge girders. This can lead to uneven expansion and contraction throughout the cross-section and induce a combination of axial and bending stresses. To further compound this problem, the resultant of the girder restraint at the abutments may not be applied concentrically with the member's neutral axis, again leading to the possibility of flexural stresses being induced. To help quantify this type of response, gages can be installed at both the top and bottom of the girders to determine the location of the neutral axis.

The discussion below will consider that the structural members and strain gages are heating and cooling uniformly and therefore avoiding the above-mentioned thermal inertia phenomenon.

3. Ends of a steel member fixed: any change in temperature of the steel member will result in a build-up of stress. If both ends of the girder are 100% fully restrained, stresses are induced because of the temperature change, but no strain in the girder occurs. This is the only case in which the "built-up" thermal strains, and therefore stresses, can be calculated as follows:

$$\sigma_{Thermal\text{-}Steel} = \varepsilon_{Thermal\text{-}Steel}\, Es \quad = \quad (T1\text{-}T0) \times CF_{Steel} \times Es \tag{6}$$

Equation 6 can be considered an "upper bound" stress since in most cases, the bridge abutments are not fully rigid and will displace some amount and therefore, reduce the thermally-induced stresses.

Regardless of the level of fixity of the girder, the magnitude of the temperature-induced stress in the girder should be indicated accurately by the vibrating wire strain gage. This is because the temperature-induced deformation of the vibrating wire itself will cause a change in wire tension and a resulting change in it's vibrational frequency, therefore:

$$\sigma_{Thermal\text{-}SteelBeam} = (R_1 - R_0)GE_s \tag{7}$$

where σ is stress and R_0 and R_1 are the initial and final readings recorded on the strain indicator, G is gate batch calibration factor, and E_s is Young's Modulus of steel. This stress can now be used to calculate the temperature-induced forces being applied to the supports.

If other loads are applied to the structure in addition to the temperature-induced loading, the stresses due to these simultaneous loads cannot be easily separated from the temperature loads. Theoretically, separation is only possible if the end restraints are fully rigid because that is the only case in which temperature-induced stresses can be accurately calculated and then subtracted from the measured strains. However, the stresses due to other loads can be separated through an experimental process. This requires the reading of both the strain and temperature at frequent intervals over a period of time during which the temperature changes, but the external loading is constant. In order to do this properly, some type of automatic recording system should be used rather than manually-recorded measurements. From these readings, it is possible to calculate the changes of temperature and the corresponding changes of strain over the same time intervals. When these strain changes are plotted against the corresponding temperature changes, the resulting graph should show a straight-line relationship. The slope of the plotted line yields the overall thermal coefficient of expansion for the structure. This experimentally-determined structure thermal coefficient of expansion can be applied to the total strain and temperature data to remove the temperature-induced strains leaving only those strains produced by changing loads. Note that the experimentally-derived thermal factor may change with time and with construction activity due to the addition of steel and concrete materials.

4. Ends of a concrete member are fixed: any change in temperature of a concrete member will result in a change of stress in the member. However, similar to the free-ended concrete member discussed above, the magnitude of the temperature-induced stresses will not be compensated for accurately with the VW strain gage. Again, this is because the coefficients of thermal expansion are not the same for steel gage and the concrete member. Because the change in vibrational frequency of the wire in the gage is related to the coefficient of expansion of steel and not concrete, a correction must be applied to the observed readings. This correction is described below.

$$\sigma_{Thermal\text{-}ConcreteBeam} = ((R_1-R_0)G + ((T_1-T_0) \times K))E_c \tag{8}$$

where E_c is Young's Modulus for concrete.

As with the restrained steel member situation previously mentioned, the temperature and loading strains cannot be easily separated. Therefore, the experimental method described above is recommended for determining the loading strains and stresses.

Note that Equations (6) and (8) appear exactly the same. However, Equation (6) indicates stresses induced only by “other loads” because the ends are free, and Equation (8) assumes some level of end fixity and determines the stress due to temperature changes only. In other words, the first condition has no temperature-induced stresses, just stress due to other loads while the latter condition has both types of stresses present.

Summary

A brief outline of how to address temperature effects when long-term measurements are recorded on bridge structures with vibrating-wire strain gages has been presented. It is not intended to be fully complete or exhaustive (except for the authors), but it should give the user some good background knowledge on the various ways temperature effects can be addressed. In some cases, the temperature effects are of the most interest and in other cases, they would like to be removed so that the effects from other loads can be quantified.

As seen, the end conditions are very important for determining the origins of the stresses. On free-ended members, there should be no temperature-induced stresses to speak of. On restrained-ended members, the temperature effects can cause the dominating stresses. However, in order to separate the temperature-induced stresses from the other-loading stresses, an experimental approach must be used.