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EFFECT OF TRANSIENT THERMAL GRADIENT ON STRESSES IN COMPOSITE BRIDGES

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Composite steel-concrete bridges experience higher thermally induced stresses than their concrete and steel cousins. These thermal stresses which can result from support restraints, debris accumulating in expansion joints, or from non-uniform thermal gradients, can lead to significant damage in the concrete deck. Conventional heat transfer theory in solids, in three-dimensional finite element formulation, is used to perform a sequentially coupled thermal-stress analysis in a selected single-span case study bridge. Actual environmental boundary conditions for a selected geographical region are used to develop the thermal profile. The vertical thermal gradient is shown to be largely non-linear as opposed to existing models, such as AASHTO. The thermally induced tensile stresses in the concrete are shown to be significant compared to service load stresses and constitute 60% of the tensile strength of the concrete deck.

Keywords: Temperature profile, Finite element analysis, Cold regions, Construction cracks, Steel, Concrete.

1 INTRODUCTION AND JUSTIFICATION

Bridges undergo continuous heat exchange with their surroundings thus developing a transient thermal profile within their cross section. The thermal gradient is largely affected by the thermal diffusivity of the constituent materials. It is this difference between the low and high thermal diffusivity of concrete and steel, respectively, that results in a significant thermal gradient within the transverse cross-section of composite bridges. This thermal gradient, coupled with different coefficients of thermal expansion for concrete and steel and the presence of shear connectors that prevent slip between the steel girders and concrete deck, make the thermal stresses in composite bridges particularly high. Design codes, such as the AASHTO LRFD Bridge Design Specifications prescribe hypothetical vertical temperature profiles in various types of bridges located in different geographical regions (AASHTO 1989).

The provided bridge expansion mechanisms can only accommodate overall deformations due to annual variations in temperatures and uniform strain components. The vertical thermal gradient resulting from diurnal variations produces three components of thermal stresses: (1) a uniform component (the average strain) which produces axial movement without stress if the movement is unrestrained; (2) a linear component (curvature-inducing strain), which produces vertical deflections and curvature without stress if the vertical movement is completely unrestrained; and (3) a nonlinear component (a stress-inducing strain) that leads to non-uniform self-equilibrating stresses. These self-equilibrating stresses result from the continuity of the cross section and the assumption that plane sections remain plane. These stresses will lead to considerable damage in the structure in the form of deck cracks that lead to the corrosion of the steel reinforcement (Ramey *et al.* 1997). Limited studies have been dedicated to investigating the thermal gradients and the resulting thermal stresses in composite bridges (Fu *et al*. 1990).

Previous studies—on which the proposed AASHTO gradient is based—have failed to take into account the presence and effect of a longitudinal thermal gradient that will produce self-equilibrating stresses in the transverse direction, regardless of whether the vertical thermal gradient is uniform or not, or whether the section is restrained or not. This is mainly due to the continuity of the section, the Poisson's effect, and displacement compatibility at the interface of consecutive sections. While some previous studies considered a two-dimensional model (Berwanger and Symko 1975, Emanuel and Hulsey 1978), most existing research is based on a one-dimensional model (Emerson 1973, Priestley and Thurston 1979), environmental boundary conditions synthesized using predictive sinusoidal models rather than actual measured data—and in some cases—steady state conditions, and simplistic finite element models. The various theoretical and experimental studies that had been conducted on composite bridges were synthesized by Kennedy and Soliman (1987), who proposed a simple one-dimensional vertical temperature distribution within the section that is uniform through the depth of the steel beam and linear through the concrete deck. This distribution led to simple formulation of thermal stresses in simple and continuous spans.

2 BRIDGE MODEL

The Colquitz River Bridge near Victoria in British Columbia, Canada was used as a case study because of its perfectly symmetrical cross section that makes it typical of composite bridges. The bridge length is 83 meters divided into five spans of varying lengths, and its width is 11.9 m. The bridge cross-section is shown in Figure 1 and the material properties are summarized in Table 1. The thermal coefficient of expansion of concrete varies between 7.2 E-6/^{o}C and 11.7 E-6/^{o}C. A constant value was assumed near the higher end—closer to the thermal coefficient of expansion of steel. Lower values would produce more dramatic thermal stresses. A single span of 18.3 meters was modeled in this study having pinned and fixed supports—boundary conditions that would produce the highest thermal stresses of all possible support conditions as was shown by Dilger *et al.* (1983). The City of Fargo, ND (Latitude: 46˚, 52’, 38”; Longitude: 96˚, 47’, 22”; Elevation: 275 m), with its extremely cold winters and hot summers, was selected for the purpose of highlighting the thermal stresses produced by diurnal cycles through large thermal gradients.

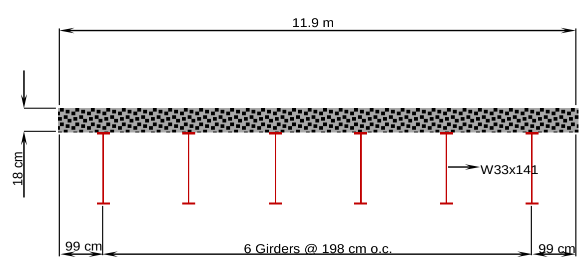


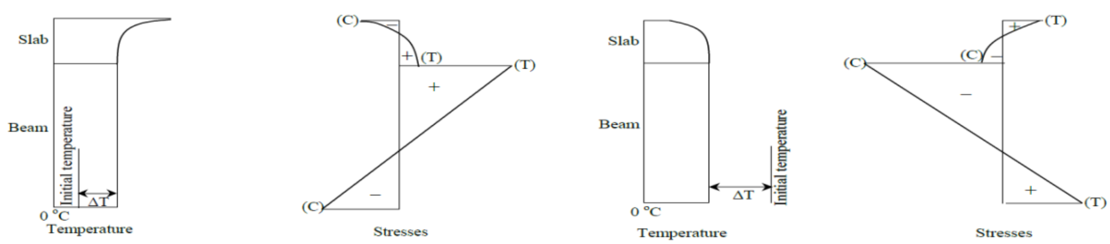
Figure 1. Cross section of Colquits river bridge.

When performing thermal stress computations for bridges, two cases of temperature profiles have to be considered in order to estimate the full range of critical stresses for design. The first is when a positive thermal gradient results from having the top of the concrete deck at a temperature higher than the bottom of the deck, which is at the same temperature as the steel beam. The second case is when the top of the concrete deck is at a temperature lower than the bottom of the deck and steel beam, and a negative thermal gradient ensues. The first case can occur on a sunny day in the summer when the top of the deck is heated by the sun radiation faster than the shaded steel beams that remain at a uniform temperature—equal to that of the ambient air temperature. The first case can also take place on a sunny winter day, but with much lower temperature difference, ΔT, between the top of the deck and the steel beams due to the lower intensity of solar radiation. The second case can occur on a winter or a summer day when the top of the deck is suddenly drenched with cold rain or snow.

Table 1. Bridge model material properties.

Bridge Component	Unit Weight (Kg/m^3)	Modulus of Elasticity (MPa)	Poisson's Ratio	Coefficient of Thermal Expansion (cm/cm/^{o}C)	Grade	Compressive Strength (MPa)
Concrete Deck	2,400	26,435	0.2	10.8 E-6	NA	28
Steel Girders	7,850	200,000	0.3	11.7 E-6	ASTM A992	NA

The most critical case is when the thermal and service load stresses are of an additive nature. In a fixed-pinned single span bridge as considered, a positive thermal gradient similar to the one depicted in Figure 2(a) will produce the associated shown stresses that would be additive to dead and live load stresses. A negative thermal gradient, as the one depicted in Figure 2(b), will produce thermal stresses that are of reverse sign to those of service loads, and are consequently not detrimental. Since larger differential temperatures would obviously induce higher stresses (although not proportionally), the case of a hot sunny day in summer would therefore constitute the most critical case of stresses. The time domain chosen for this transient analysis consists therefore of a 24-hour time span occurring over June 4. The selection was based on the highest radiation intensity day of the year for Fargo, ND, since it has been shown that the solar radiation has the highest influence on thermal gradients in bridges (Tong 2002). A transient time step of one hour, divided into four (4) equal increments, was used for the transient thermal simulation.



(a) (b)

Figure 2. Thermal gradient and corresponding thermal stresses: (a) positive gradient; (b) negative gradient.

The finite element model of the bridge section was constructed using homogenous isotropic solid elements. Both concrete deck and steel girders were modeled using 3D 8-node linear hexahedral heat transfer mesh elements with temperature as a single degree of freedom at each node, and second-order (quadratic) interpolation in three dimensions. The model (shown in Figure 3) comprised the full span length, width, and depth of the bridge with all six supporting steel girders. The refined mesh resulted in nine integration points along the depth of the concrete deck, thus allowing accurate representation of non-linear behavior in bending and retrieval of thermal profile, stresses, and strains at each integration point. Shear connectors, in the form of "tie" connectors, were continuously provided between the bottom of the concrete deck and the top flange of the steel girders to ensure full composite action zero slip. Translation and rotation displacement restraints were provided at the end of the model as appropriate. The stress-strain behavior of the concrete is modeled to include compressive crushing, tensile softening, and cracking. The stress-strain behavior for the concrete deck in tension is shown in Figure 4.

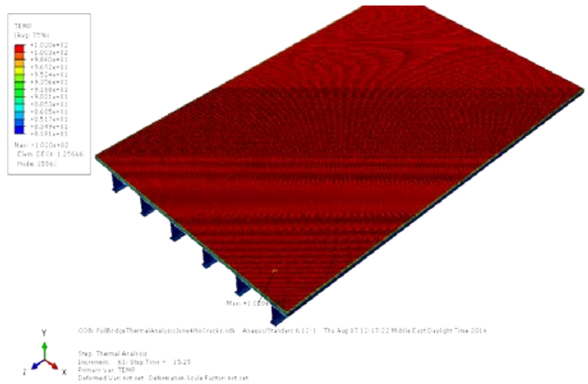


Figure 3. Finite element model of the bridge.

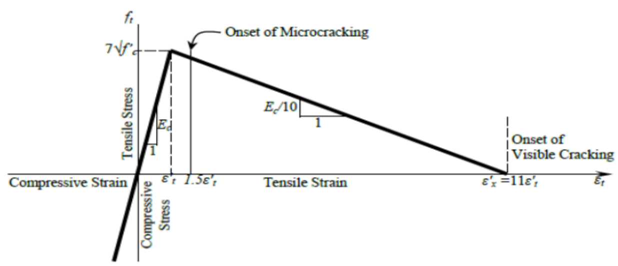


Figure 4. Concrete stress-strain model in tension.

3 RESULTS

Sequentially-coupled non-linear transient thermal-stress analysis is conducted in which the stress/deformation field in the bridge depends on the temperature field. This is performed by first conducting an uncoupled heat transfer analysis, and then a stress/deformation analysis of the model. The simulation time domain spanned over a period of 24-hours that represents a full day (June 4). Results of the 3D finite element analysis show that the vertical temperature distribution in the web of the steel girder is almost uniform, which agrees with previous studies (Kennedy and Soliman 1987). Figure 5 provides the vertical temperature distribution at midspan of the interior girder at the time of maximum thermal gradient, along with the thermal profile as calculated using the AASHTO and the Kennedy and Soliman models. Due to solar radiations reflected from the ground, the temperature at the bottom surface of the concrete deck tends to be slightly cooler right over the two steel girders than the regions between the two girders and under the overhang.

The absolute maximum temperature differential between the concrete deck and the steel girder is critical for calculating the bridge thermal stresses resulting from the various components of thermal strains (uniform, linear, and non-linear). On June 4, the temperature differential reached a maximum value of 10.7 ^{o}C (19.3 ^{o}F) at the location of the interior girder when the temperature in the concrete deck reached a maximum value of 38.6 ˚C (102 ^{o}F) at time 15:15. This value is less than half that suggested by Kennedy and Soliman, which is 22.3 ˚C.

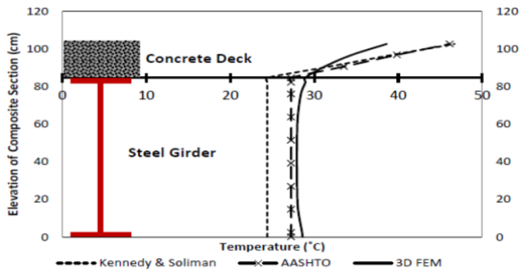


Figure 5. Vertical temperature distribution at interior girder (15:15; June 4).

AASHTO recommends that the vertical temperature distribution be uniform in the steel girders and linear in the superimposed concrete deck. The temperature differential in the concrete deck is based on a map of solar radiation zones in the United States, in which the city of Fargo, ND (located in zone 2) has a maximum positive vertical temperature differential of 18.9 ˚C between the top and bottom surfaces of the concrete deck. Figure 5 shows that the AASHTO maximum temperature differential is 8.2 ˚C higher than the one obtained here. This difference reaches 11.6 ^{o}C for the Kennedy and Soliman model. This conservative approximation by AASHTO leads to considerable error in assessing the thermal stresses. Another issue is the nonlinear thermal profile which produces a nonlinear thermal strain component whose effects on the design is unknown.

The vertical thermal profile proposed by AASHTO for the case study bridge is highly conservative. The reason is that AASHTO is based on a 1D conduction analysis and ignores the reflected solar radiation which decreases the positive thermal gradient by heating the bottom surface of the deck. Also, the AASHTO model does not consider the depth of the concrete deck or the overhang-to-depth ratio, which produces a shading effect on the exterior beam. AASHTO also assumes a linear vertical temperature distribution in the concrete deck at all times.

The highest positive value for the maximum principal stress is used as the maximum tensile stress in the concrete deck, and the highest negative value of the minimum principal stress is used as the maximum compressive stress in the concrete deck. As for ductile materials such as steel, the Von Mises effective stress is used for the determining the stress in the steel girders; these stresses are only positive since they predict the yielding failure of the material.

Figure 6 shows the stress/deformation results of the steel girders. The maximum Von Mises effective stress is shown to be at around 98 MPa (14.5 ksi), which is well within the yield strength of any structural grade steel. This maximum stress occurs at the top of the exterior girder near the supports of the bridge. Figure 6 shows the thermal stresses to produce lateral torsional buckling in the steel girders that increase from the centerline of the bridge towards the edges.

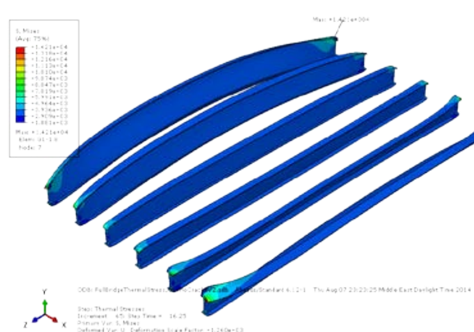


Figure 6. Maximum Von Mises stresses and deformation of steel girders.

The thermal stresses are compared to those obtained by previous studies in Table 2. The tensile stresses in the concrete deck are reasonably comparable with those obtained by Emanuel and Taylor (1985), and much higher than those obtained by Chen (2008) and Fu *et al.* (1990). This high discrepancy in the value of tensile stresses can be attributed to the fact that previous studies used the principle stresses to compute the thermal stresses in the steel girders, and the fact that these models have not considered the 3D effect of the thermal gradient on the bridge stresses.

Table 2. Thermal stresses comparison with previous models.

Maximum Stresses (MPa)	**3D FEM**	**Chen (2008)**	**Fu *et al.* (1990)**	**Emanuel and Taylor (1985)**
Compressive in Concrete Deck	−3.8	--	-1.87	−1.98
Tensile in Concrete Deck	2.16	0.61	1.14	2.83
Compressive in Steel Girder	--	--	-18.62	−16.25
Tensile in Steel Girder	97.98	--	6.76	0

4 CONCLUSIONS

The AASHTO and Kennedy and Soliman models are overly conservatively for bridges in cold regions, leading to overestimation in predicting the thermal stresses. A nonlinear thermal profile develops in the concrete deck as compared to the linear profile proposed by AASHTO and other models. This nonlinearity creates a nonlinear strain component that significantly affects the thermal stresses in the bridge. The stresses in the deck constitute 60% of the available tensile strength of concrete and are thus substantially large to warrant consideration in the design of steel reinforcement. Existing 1D and 2D models fail to account for the lateral torsional buckling stresses that the thermal gradient produces in the steel girders. They also fail to model the effect of the transverse thermal gradient on the transverse curvature and thermal stresses of the bridge.

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