

Girder-End Cracking in Prestressed I-Girders

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1. Introduction

There has been much interest in the stresses created during the early-age of a prestressed concrete girder. A recently completed research project to instrument and monitor Missouri's first HPC Bridge (Gopalaratnam and Eatherton, 2001) provided some useful insights regarding the early-age response of HPC prestressed girders. Figure 1 shows cracking which was observed in the girders after the transfer of prestressing. These cracks are attributed to a combination of residual stresses due to curing/hydration, and stresses created by the transfer of the prestressing force (Earney, 2000, Earney, 2002, and Gopalaratnam, et. al, 2001).

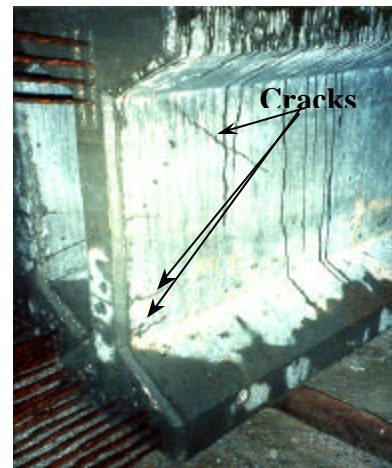


Figure 1. Early-age cracking in HPC girder.

1.1 Background information

This project was undertaken on behalf of the Missouri Department of Transportation (MoDOT) after cracking was observed in more than 100 of the 900 PC I-girder bridges in the state. The data from a previous project that instrumented four HPC girders in Missouri's first HPC bridge provided the information needed to analyze the early-age behavior of these girders (Eatherton, 1999). The instrumentation in these girders was monitored from the beginning of the girder's casting until one year after the opening of the bridge. The instrumentation consisted of thermistors, strain gaged bars, and vibrating wire strain gages at several locations in each of two cross sections. Additionally, two stirrups were instrumented with strain gages at the end of each girder.

1.2 Objective

The objective of this study was to develop an analytical method for predicting the vertical stresses that are created in the end of PC I-girders during early age. It is believed that the two factors contributing principally to this are the restrained thermal expansion and contraction during hydration, and the transfer of the prestressing force. A finite element model was developed using ANSYS (v5.4, 1999) to analyze the residual stress development during hydration. A model created by Gergely and Sozen (1967) was modified to analyze the vertical tensile stresses created during the transfer of the prestressing force.

1.3 Literature review

Stresses developed due to stressing operations were first investigated in the early sixties by Mashall and Mattock (1962). These studies were largely empirical, but did provide some insight for future studies. The authors speculated that the cracking might be initiated by restrained shrinkage and thermal contraction provided by the form during curing, however no analysis of this hypothesis was reported.

Research has been conducted on the thermal strains created in girders due to in-service temperature variations (Saetta, et al, 1995), and on the development of heat during curing in HPC girders (Khan, et al, 1998, Steeg, et al, 1996). However, to the best of the author's knowledge, no literature exists on methods for calculating residual stresses due to restraint provided by the formwork.

An analytical model to predict the vertical tensile stresses in the girder end region of PC Beams was proposed by Gergely and Sozen (1967). The model, shown in Figure 2, assumes that when a section is taken at the bottom of the girder-end, the portion of the internal stress distribution acting on the section is not sufficient to resist the large prestressing force and a reaction moment is created internally. This moment thus creates tensile stresses at the girder end. Gamble (1997) and Kannel, et al (1997) noted cracking similar to that found in Missouri in girders used in Illinois and Minnesota bridges, respectively. Professor Gamble observed that the model proposed by Gergely and Sozen could be used to accurately predict the location of these cracks. No mention was made of prediction of the stress levels in the girder-end, however.



Figure 2. Gergely-Sozen model for predicting vertical stresses due to prestressing force.

2. Vertical End Stresses due to Prestress Transfer

The Gergely and Sozen (1962) model is used with some modifications based upon experimental data from a companion investigation (Eatherton, 1999, Gopalaratnam and Eatherton, 2001) to evaluate vertical end stresses created during the transfer of the prestressing force. The model, shown in Figure 3, assumes an unbalanced internal moment is created due to the prestressing force. The location of the maximum unbalanced moment is found by solving for the unbalanced moment as a function of the amount of web included in the section, “y”.

$$M = P(y + h_f - y_{ps}) - C(y + h_f - y_f) - \left(s_f - \frac{y}{3m} \right) y^2 \frac{t_w}{2} \quad (1)$$

Computing the first derivative of Eq. 1 and setting it equal to zero allows establishing the depth at which the moment reaches a maximum value.

In order to find the stress in the girder end due to this moment, a length of girder must be used (labeled “x” in Figure 3). Gergely and Sozen had suggested that “x” be assumed equal to the total height of the girder. It was suggested that this distance was greater than the transfer length of the post-tensioned strands used in the initial study. This condition is required so that the assumption of a linear distribution of internal longitudinal stress is accurate. Since data from stirrup strains during prestress transfer operations for the HPC project (Eatherton, 1999) were readily available, it was possible to evaluate the Gergely-Sozen recommendation for girder length to be considered for the prestressed HPC girders. Two stirrups were instrumented in the ends of each of the four girders. The strains in the stirrups at $h/2$ - and h -away were opposite in sign suggesting that the point of “zero-strain” occurs somewhere between these two locations. This point of “zero-strain” is located $x/2$ from the end for the Gergely–Sozen model. Due to the limited number of data points, the influence of girder geometry, prestressing force used, prestressing profile, and transfer length on the location of “x” cannot be ascertained for a general case. However, this location can be established experimentally from the data available.

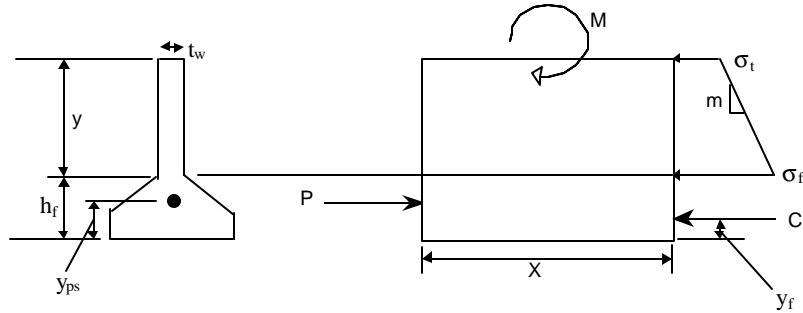


Figure 3. Modified Gergely-Sozen model.

Using linear interpolation, the location of “zero-strain”, $x/2$, was found to be in the range of 80 cm - 100 cm. If the Gergely–Sozen assumption is used, this distance would be $h/2 = 69$ cm for both girders. This length is shorter than that computed from experimental measurements of stirrup strain and produces stresses that are significantly higher. Table 1 lists the computed values for maximum unbalanced moment for each of these girders, the corresponding location, and the maximum vertical tensile stress produced using the Gergely – Sozen assumption for x , as well as the experimentally computed length, x .

The maximum tensile stresses in the girder ends due to prestress transfer are approximately 40% to 50% of the tensile strength of concrete (~4-5 MPa). While these stresses by themselves may not be sufficient to cause cracking, when they are considered in conjunction with the residual tensile stresses due to hydration/curing gradients horizontal girder-end cracking is very likely.

Table 1. Maximum tensile stress and location due to prestress transfer

Girder	Location of Max. Moment from girder bottom (cm)	Maximum Un-balanced Moment (MN-m)	Maximum Tensile Stress (experimental data, MPa)	Maximum Tensile Stress ($x = h$, MPa)
Long	58	413	2.4	5.2
Short	52	306	2.2	3.8

3. Residual Stress Profiles due to Differential Thermal Loads

Two types of elements are used in the numerical model: Four-node, two-dimensional plane, structural solid elements, and two-node, two-dimensional, point-to-point contact elements. Plane elements were used to model both the concrete and the steel mold, and contact elements were used to model the interface between the concrete and the steel mold. For the plane elements, a plane strain option is chosen, which allows for stress in the perpendicular, z , direction, but not strain. This constitutive model is well-suited for applications that model an “infinitely” deep section like a girder where perpendicular strains are negligible. These elements have two degrees of freedom at each node: x - and y -direction displacements. Small displacement theory is used in conjunction with linear elastic material behavior. The contact elements are designed such that they model two surfaces that allow for compressive normal forces as well as frictional sliding forces, but do not transfer tensile forces. This allows the concrete to separate from the steel mold preventing any normal tensile forces from developing at the steel-concrete interface. The frictional force is calculated as the compressive normal force times the coefficient of friction, μ .

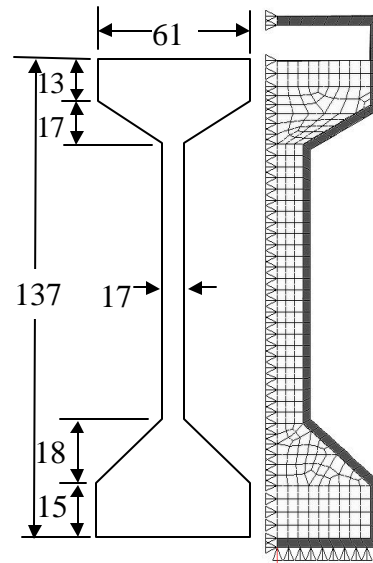


Figure 4. Model dimensions (cm) and finite element mesh details.

The mesh was created to preserve as nearly as possible an aspect ratio of 1:1 for the plane elements. In all cases, mesh refinement studies were conducted to establish that mesh size had little influence on the stress solutions. Fig. 4 shows the girder dimensions and the element mesh for the model. The steel mold is one inch thick, which, with modifications to the web steel properties, accounts for the additional stiffness provided by the stiffeners on the sides of the mold. The top brace on the model is to simulate the bracing used to prevent the form from splaying open. This bracing is located 10 cm above the top of the concrete.

The boundary conditions were chosen to model the girder sitting on a rigid platform, and to reflect the symmetry of the cross-section. This was done by restraining vertical movement of nodes along the bottom edge of the form and by restraining horizontal movement of nodes along the centerline of the girder cross-section (Figure 4). The model is subjected to thermal loading and loading due to self-weight. Unit weights of $2,400 \text{ kg/m}^3$ for the concrete and $7,850 \text{ kg/m}^3$ for the steel were used. The temperature values were based on experimental data recorded during curing of the girders for the HPC bridge (Eatherton, 1999). The thermal loading was determined by taking the absolute

maximum difference between eight thermocouples at each of two cross-sections (end and midspan). The thermocouples were distributed along the depth of the girder. The maximum temperature differential was found to be 20°C.

In order to keep the finite element model as simple as possible, assumptions were made about the behavior of the curing concrete under temperature loading. It was decided that a single load step would be used. The temperature history of the girders was analyzed, and it was found that the maximum temperatures were reached shortly after steam curing begins (12 hours after casting). In contrast, the temperatures do not begin to approach ambient temperature until after 72 hours or more. When concrete cures, its stiffness increases with time. During the time when the steam curing is started, the concrete has a relatively low stiffness. Using the Branson (1977) method, the 12-hour modulus of elasticity is about 60% of the 28-day modulus of elasticity. The 12-hour modulus is 22.8 GPa, and the 28-day modulus is 39.3 GPa for HPC. In contrast, when the girders cool, the stiffness is about 75% to 80% of the 28-day value.

Based upon the temperature history of the girder, it was decided to model only the portion of the girder history from when maximum temperature is reached until the temperature returns to ambient level. It is assumed that no appreciable stresses develop in the concrete due to increasing temperature because a) the steel mold, which has a higher thermal coefficient of expansion, does not restrain volumetric change in the concrete and b) at early ages, the modulus of concrete is very low. The same is not true during dropping temperature, as the stiffness of the concrete is much greater. This procedure is conservative since the stiffness at the end of the curing period is used for the analysis. This was assumed to be the 72-hour stiffness, found using Branson (1977).

A plot of the principal stress contours is shown in Figure 5. The orientation of these stresses is not shown on the figure, but they were found to follow the contour of the steel mold. It can be seen that a uniform tensile stress is created in the web of the girder. The maximum principal tensile stress value was found to be around 3.0 MPa. This stress is located at the reentrant corner of the flange and web. The average tensile stress in the web was around 1.3 MPa. The pattern of stress concentrations is in conjunction with observed cracking patterns shown in Figure 1. When the maximum stress from curing/hydration is combined with the tensile stress created due to the transfer of the prestressing force, it is very likely that cracking will occur in the end-zone of PC I-girders.

The maximum stresses are located only in small, concentrated areas. This suggests that a change in cross-section that incorporated less sudden direction changes could reduce the maximum stress values. The average stresses are significantly less than typical modulus of rupture (MOR) values of HPC (~5.2 MPa). If the stress concentrations were eliminated, the girders would be much less likely to crack due to early-age behavior.

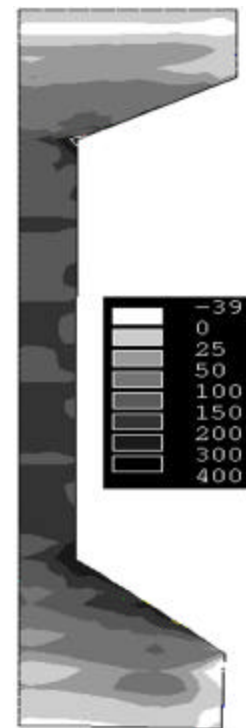


Figure 5. Principal residual tensile stress profile due to curing and hydration.

Values in psi, 1psi = 7kPa

4. Conclusions and Discussion

The transfer of the prestressing force was found to create vertical tensile stress of around 2.3 MPa. This value is less than 50% of typical MOR values, and would not likely cause cracking. However, when this value is considered in conjunction with the residual stresses created due to heat of hydration and curing, cracking can be expected to occur.

Reducing the temperature gradients during curing and hydration would help to reduce the residual tensile stresses, as would altering the geometry of the girder to reduce the sharpness of the transition from flange to web.

Stresses due to prestress transfer could possibly be reduced by increasing the thickness of the web or providing an end block. Both methods increase the area of concrete in tension, however they will also tend to increase the unbalanced moment by raising the resultant compressive force of the concrete.

5. Acknowledgements

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