The Effects of Design and Manufacturing Parameters on Early Age Cracking of Prestressed Concrete Girders

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Abstract:

In recent years, the use of long prestressed girder spans has gained wide popularity. This demand necessitated to use of large size girders with high strength concrete up to 10 ksi which is common nowadays. *In addition, long term durability has been linked to the use of low water/cement ratios and the concrete industry has been successful in driving the W/C ratios to the range between 0.30 to 0.34 with the use of polycarboxylate. This new trend comes with minor drawbacks in the form of cracking at the top flanges of these girders. This paper presents a discussion of the cause of the experienced cracking and possible solutions to minimize undesired cracking.*

Keywords: Prestressed girders, Concrete, Shrinkage, Thermal effects, Cracking, Restraint shrinkage

INTRODUCTION

In recent years, the trend to use prestressed girders for long span bridges has gained wide popularity. However,20 years ago, a 120 ft. prestressed girder was considered long, currently the use of 160 ft and longer girder is becoming more common. This practice requires the use of large size girders with high strength concrete to achieve the designer's objective.

During the production of these girders, the occurrence of transverse cracks (perpendicular to the beam length) in the girder top flange has been observed. Some of the transverse cracks have even propagated into the girder web. The reason for the cracks has been unclear; however, it is most often blamed on the girders manufacturer and attributed to poor curing and handling.

Although accounting for restraint shrinkage, and thermal effects at the early age of prestressed girders is a sound design practice, it is not required by the AASHTO design specifications and most designers neglect these effects. Neglecting shrinkage, and thermal effects has been justified due to the fact that traditional AASHTO type girders have narrow top flanges and no steel reinforcement other than two lightly prestressed strands (pulled up to 5 kips). However, the new generations of Bulb Tee girders most often have 48 inch wide flanges and significant amount of mild reinforcement in addition to four top prestressing strands pulled between 5 to 10 kips.

Shrinkage of concrete takes place soon after casting, and lasts throughout its service lifetime. The overall shrinkage of concrete consists of several types: plastic shrinkage, autogenous shrinkage, drying shrinkage, thermal shrinkage and carbonation shrinkage (Aïtcin et al. 1997). For conventional concrete, shrinkage is taken as drying shrinkage which is the strain associate with the loss of moisture from concrete under drying conditions (Khayat and Long 2010). However, for high strength concrete (> 6 ksi) with a low w/c ratio, a supplementary problem occurs at early ages due to autogenous shrinkage. This is a result of the lack of moisture transfer with the environment that needs to be taken into account (Holt 2001). Achieving the desired high concrete strength (up to 10 ksi) is generally obtained through driving the w/c ratios to the range between 0.30 to 0.34. This extremely low w/c ratios are possible through the use of polycarboxylate which is known to lead to drawbacks in the form of high concrete shrinkage at early age. Such shrinkage generates significant volume changes that are known to cause tensile stress depending on the level of concrete restraint.

Another factor attributed to the early-age volume change in concrete is thermal change prior to prestress release, which depends on the type of concrete mix and curing conditions. Hydration of Portland cement results in the generation and dissipation of heat and typically an increase in the temperature of freshly placed concrete. To control cracking, it is generally recommended that the temperature differential between the concrete and the surrounding environment should be less than 35 ^{o}F to minimize the effects of thermal contraction (Gajda and Vangeem 2002).

Once at the storage area, the girders are left exposed to the natural environment and are subject to thermal changes. Since the large girder flanges are subject to thermal radiation from the sun, temperature differential between exposed surfaces and girder web can reach up to 40 to 70 °F. Such temperature gradient could cause differential volume changes and result in tension in the top flange of the girder.

Combining the effects of restraint concrete shrinkage and thermal radiation could result in significant tensile stresses in the beam top flange. While these stresses alone might not be sufficient to crack the concrete, superimposing the tensile stress due to the prestress can lead to high stress beyond the concrete modulus of rupture and result in the occurrence of concrete cracking in top flange.

In addition, the intended use of the prestressed girders could magnify the observed cracking problem. Generally, the girders are used for the construction of superstructure, but there are cases where prestressed girders are used as bent caps to support the super structure. While the applied loads are different, the design criteria are basically the same. However, significant amount of reinforcement is added in the top flanges to address negative moment demand. The following example presents a similar case and the deficiencies in this type of application.

CASE STUDY

Modified Florida 78’’ Bulb-T (FBT-78) Beam is designed as helper Bent supporting 11 AASHTO Type II girders in a Bridge in Florida. Figure 1 shows the bent elevation.

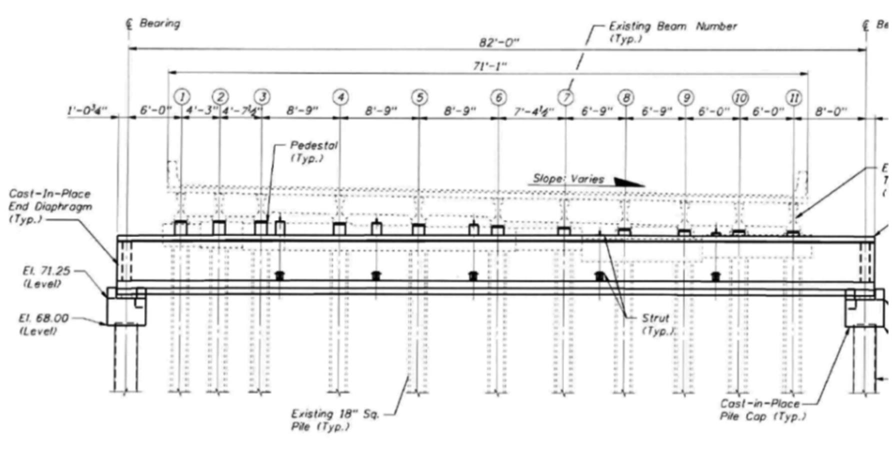

Figure 1 Bridge Elevation

Figure 2 shows the cross section dimension of the modified FBT-78 Beam, while the section properties of the Beam are given in Table 1.

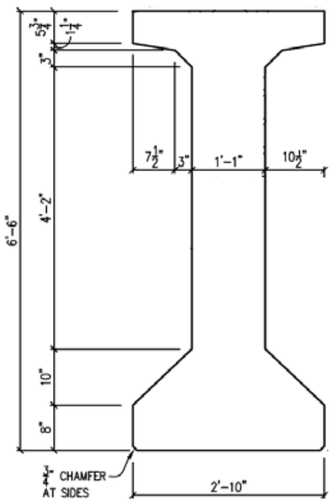

Table 1 Beam Section Properties

A	1433.63 in^2
I	977410.13 in^4
Y_b	36.49in
Y_t	41.51in
S_b	26785.70 in^3
S_t	23546.38 in^3
Perimeter	247.5 in

Figure 2 Beam Cross Section

The girders were cast and moved to the storage area without any signs of cracking. Inspection of the girders prior to shipping to the work site revealed hairline cracks at the top flanges of all cast girders. The transverse cracks extend along the girder length and appear to have started from the top flange surface and propagated into upper part of the web. A Typical crack map is shown in Figure 3.

Figure 3 illustrates extensive cracking along the length of the beam top flange. Majority of those cracks have propagated into the web by as much as 23in. The majority of the hairline cracks vary in width between 0.002 and 0.005 inch.

As most often the case, the girder manufacturer was blamed for the cracking and while that might be justified in some cases, the fact that every girder experienced the same type of cracking pointed out an inherent problem that required further investigation.

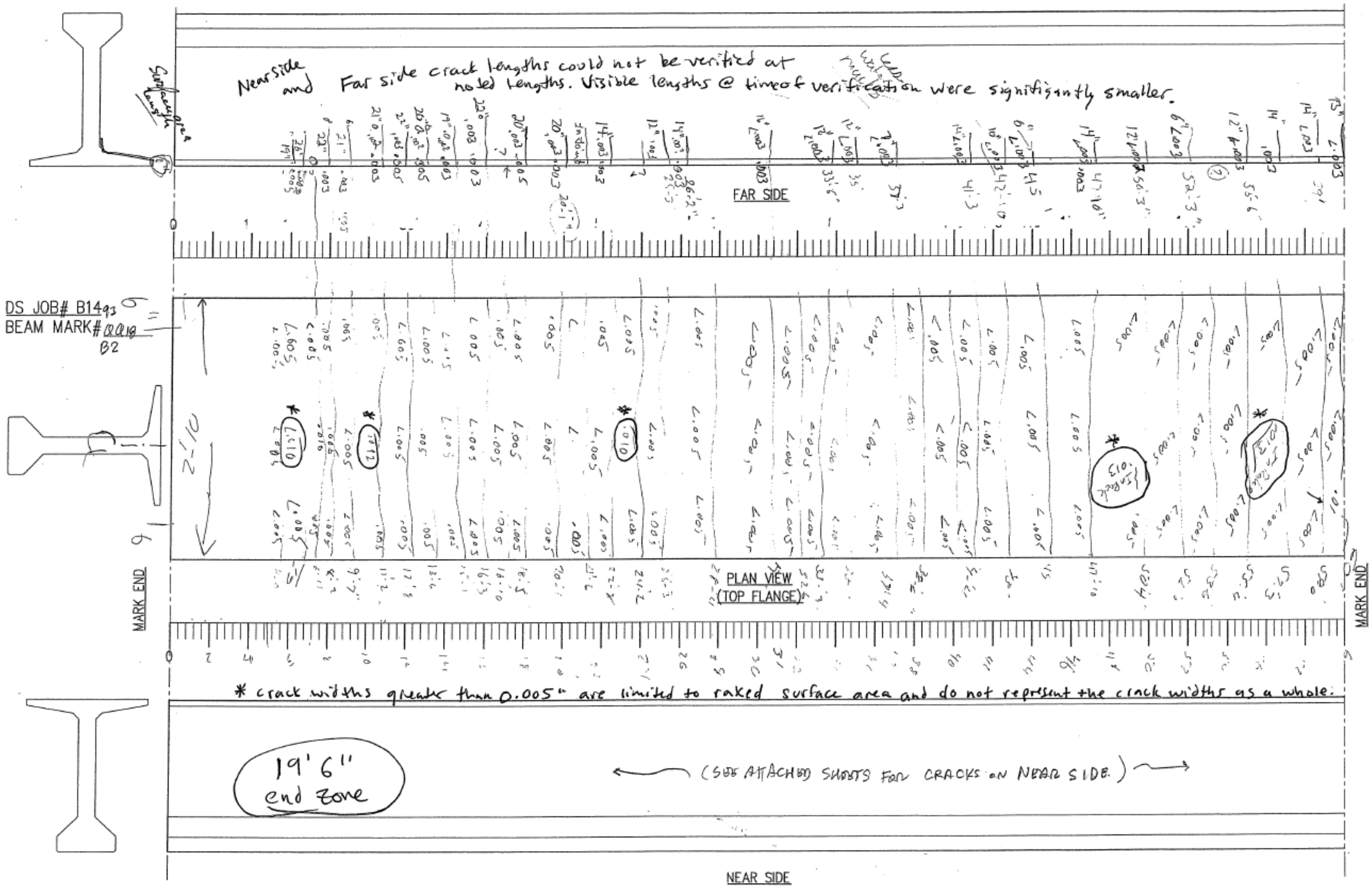

Figure 3 **Crack Map of Modified FBT-78 Beams After Casting**

The Modified FBT-78 Beam is designed using 48-270ksi low relaxation 0.6’’ diam. strands in the bottom flange and five (5) 0.6’’ diam. strands in the top flange as shown in Figure 4. The jacking force for each strand is 43.9 kips and 10 kips for the bottom and top strands, respectively.

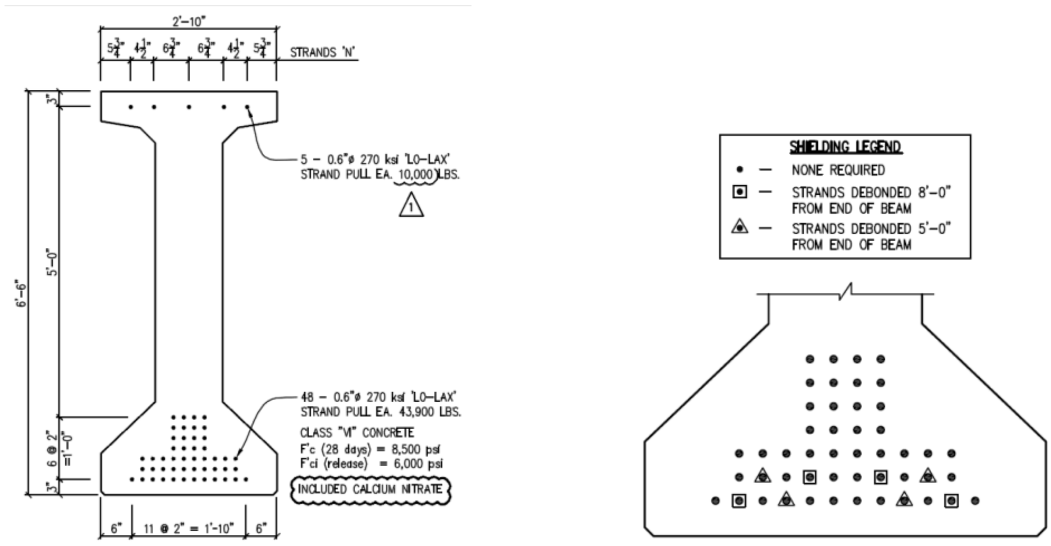

Figure 4 Strand Pattern

A total of 4 strands in the bottom flange are debonded up to a distance of 8 feet from the beam ends and 4 other strands are debonded for 5 feet to reduce the concrete stress at the beam ends as shown in Figure 4.

ASSESSMENT OF THE PROBLEM

Preliminary investigation of available information indicates that the girders are designed and detailed in a manner that is fully AASHTO Design code and FDOT compliant. The girders were wet cured for approximately three days, up to the transfer of the prestress according to FDOT current requirements. The initial opinion is that the cracking is due to either deficient concrete mix design or a designer error. Both probable causes are investigated in the following sections.

Effect of the Concrete Mix Design

The production method of these beams is consistent with past practices and the manufacturer confirmed that curing is also consistent and followed FDOT criteria. Since construction method could not be the cause of the cracking, verification of the suitability of the design mix is important to narrow down the cause of the cracking. An investigation of the concrete mix design is performed to determine whether the concrete mix contributed to the cracking problem.

Chemical admixtures are added to concrete in very small amounts mainly for the entrainment of air, reduction of water or cement content, plasticization of fresh concrete mixtures, or control of setting time.

The concrete mix design calls for four different types of admixtures, namely; air entraining admixture, Stabilizer admixture retarder, water-reducing admixture and corrosion-inhibiting admixture. The effect of each of these elements is discussed below.

Air entrainment is the process whereby many small air bubbles are incorporated into concrete and become part of the matrix that binds the aggregate together in the hardened concrete. Air entrainment has now been an accepted fact in concrete technology for more than 45 years. Generally, there is no standard dosage for the admixture and the exact quantity of air-entraining admixture needed for a given air content of concrete varies due to differences in concrete-making materials and ambient conditions.

Stabilizer admixture retards setting time of concrete by controlling the hydration of portland cement and other cementitious materials while facilitating placing and finishing operations. The recommended dosage according to the manufacturer data sheet is in the range of 260 ± 65 mL/100 kg of cementitious materials. The concrete mix design calls for 22.5 Oz which is the lower bound of the recommended range (198 ml/100 kg of cement)

Water-reducing admixture is a water reducing agent that is added to the concrete to improve workability (slump) at a lower w/c than that of control concrete. The basic role of water reducers is to deflocculate the cement particles agglomerated together and release the water tied up in these agglomerations, producing more fluid paste at lower water contents. The recommended dosage according to the manufacturer data sheet is in the range of 130-975 mL/100 kg of cementitious materials. For most applications, dosages in the range of 195-650 mL/100 kg will provide excellent performance. The concrete mix design calls for 45.0 Oz (397.2 ml /100 kg) which is within the recommended range.

The use of water reducing admixtures is associated with water bleed and high rate of shrinkage. Generally, increased attention needs to be placed on curing and protection due to the potential for shrinkage cracks and bleeding experienced when water reducers are used.

Corrosion-Inhibiting, CI, admixture is a calcium nitrite based corrosion-inhibiting admixture which is used in reinforced concrete to delay steel corrosion by repassivating defects on the steel surface. According to the manufacturer data sheet, one of the side effects of using this product is that the concrete setting times is generally accelerated. In most applications a retarding or hydration control admixture must be added to the concrete mixture to offset the acceleration effects of corrosion-inhibiting admixture.

From the manufacturer data sheet, the corrosion-inhibiting admixture is recommended for use within a dosage range of 3.8-23.0 L/yd^3 (5.0-30.0 L/m3) of concrete, depending upon the severity of the corrosion environment and the anticipated chloride loading of the structure. According to the concrete mix design for this project 576 Oz/CY of "CI" admixture is used which equates to 17 L/ L/yd^3. This high content of "CI" admixture will result in reduction in hydration rate and should be compensated for by using proper proportions of retarding or hydration control admixtures.

Further detailed examination of the mix plant batch records from the placement indicated that, on average, the actual dosage of Stabilizer admixture used in the concrete mixture was 55 fl oz/yd^3 which is adequate for the dosage of calcium nitrite admixture in the mixture and that it was effective in slowing down the accelerating effect of the calcium nitrite admixture to permit proper placement and finishing of the 78" FBT modified beams. Therefore, the lack of proper control over the accelerating effect of the calcium nitrite admixture can be ruled out as the cause of the cracking.

Effects of Design Prestress, Thermal and Shrinkage at Transfer

Table 2 summarizes all assumptions used for analyzing of the stress condition of the beam top flange after prestress transfer.

Table 2 Beam and Strand Properties

Bottom Strands		
	Section w/o debonded strands	Section with debonded strands
Number of Bottom strands	48	40
Jacking force per strand	43.9kips	43.9kips
Stress loss at transfer (assumed)	8%	8%
Initial jacking force, P_i	1938.62 kips	1615.52 kips
Y_b	36.49 in	36.49 in
$e_b= Y_b-Y_{bs}$	29.24 in	28.59 in
Top Strands		
Number of Top strands	5	5
Jacking force per strand	10.0 kips	10.0 kips
Stress loss at transfer (assumed)	8%	8%
Initial jacking force, P_i	46.0 kips	46.0 kips
Y_t	41.51 in	41.51 in
$e_t= Y_t- 3$	38.51in	38.51in

Beam Details	
Beam Length, L	84.13 ft

Concrete strength at transfer	6.0 ksi
Concrete strength at 28 days	8.5 ksi
Concrete unit weight	0.15 kcf
Beam unit weight, w (=0.15*A/144)	1.49 kip/ft

Top Flange Stress Due to Prestress

Recent research showed that in deep members the average tensile strength of concrete in direct tension can be significantly lower than $7.5\sqrt{f_c'}$ and is typically in the order of $4\sqrt{f_c'}$ (Collins and Mitchell 1997; fib 1999). Recently, Tuchscherer and Bayrak (2009) investigated seven 54'' deep by 20ft long prestressed concrete bridge beams to study the concrete cracking at release, and the results indicated that the extreme fiber tensile stress should be limited to $4\sqrt{f_c'}$ in order to prevent cracking of the top flanges.

While the above research indicates that the $7.5\sqrt{f_c'}$ tensile strength limit is too high to prevent cracking, the limited number of tests and size of specimens used prevent adaption of these results. Current design codes (ACI, AASHTO) specify the upper limit for the tensile strength and research on super high strength concrete indicates even higher tensile limit. In this study, the code accepted modulus of rupture is used in the analysis.

Based on the information presented in Table 2, the concrete stress in the top flange can be calculated at transfer stage with Eq. (1).

$$f_t = \frac{P_i}{A} - \frac{P_i e}{S_t} + \frac{M_g}{S_t} \qquad (1)$$

The top concrete stress along the beam length is shown in Table 3 and compared with the allowable stress per as specified in FDOT guidelines.

Table 3 Top Stress Due to Prestress at Transfer Along The Modified FBT-78 Beam

Section	0.0L	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L
Distance from beam end, ft	0	8.413	16.825	23.238	33.65	42.063	50.475	58.888	67.300	75.713	84.125
Top Stress, ksi	-0.727	-0.706	**-0.518**	-0.384	-0.303	-0.276	-0.303	-0.384	**-0.518**	-0.706	-0.727
Allowable Tension, ksi	-0.93 ($12\sqrt{f_{ci}'}$)		-0.465 ($6\sqrt{f_{ci}'}$)							-0.93 ($12\sqrt{f_{ci}'}$)	

It should be noted that the FDOT design guidelines allow a maximum tensile stress of $12\sqrt{f'_{ci}}$ at the outer 15% of the span length. It can be seen from Table 3 that the top stress at 0.2L section exceeds the calculated allowable tensile stress based on the design release strength. However, while the calculated top concrete tensile stresses in these areas are high it is not likely sufficient to crack the concrete at transfer. It should be pointed out that the specified 28 day concrete strength is 8.5 ksi and research has shown that the modulus for this type of strength could be as high as $12\sqrt{f'_{ci}}$.

Top Flange Stress Due to Restrained Shrinkage and Temperature Change

Early-age volume change occurs in concrete due to changes in temperature and moisture content. The volume change associated with a change in moisture content is referred to as drying shrinkage. Significant volume changes are known to cause cracking depending on the level of restraint in the concrete member and the interaction of internal forces (i.e., prestress and Selfweight). As mentioned earlier it is generally recommended that the temperature differential between the concrete and the surrounding environment should be less than 35 °F to minimize the effects of thermal contraction. The use of a protective covering such as tarps and constant wetting of the exposed concrete surfaces help to retain heat. This method of curing, if applied correctly is effective in maintaining the maximum temperature differential below 35 ^{o}F during early curing and before transfer of the prestressing force. However, this curing environment is generally discontinued after prestress release. The prestressed girders, once prestress transfer is completed, are generally transported from the casting beds to a general storage area. Generally, the prestress release takes place between one to three days of casting and while the compressive concrete strength is achieved, the concrete continue to cure for several additional days until maturation.

Restraint concrete shrinkage and thermal gradient could result in significant tensile stress depending on the type of concrete mix, curing conditions and storage environment. It should be noted that the rate of drying of the concrete surface will be influenced by the prevailing ambient conditions. Consequently, the prevailing ambient conditions will also influence the potential for cracking.

It can be seen from Figure 5 that there is a significant restraint at the girder to flange due to the amount of longitudinal and transverse reinforcement specified by the designer. In addition to the five (5) top strands there are 6 #7 longitudinal bars at the same level as the strands. There are 240 # 5 bars at variable spacing which translates to an effective steel bar at an average of 8 inches. Add to that 22 #4 bars and we get a very heavily reinforced top flange that is fully restraint by reinforcement.

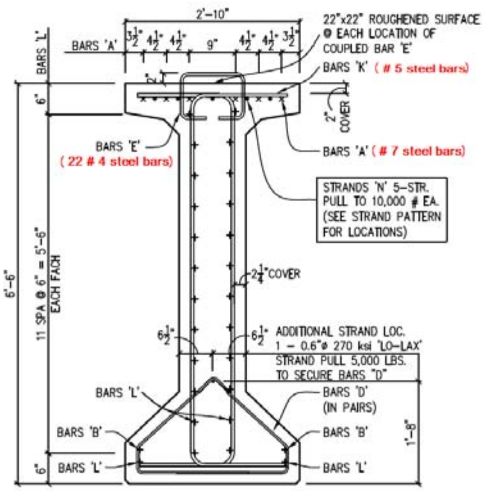

Figure 5 Reinforcements Details

Shrinkage is the time-dependent strain measured in an unloaded and unrestrained specimen at constant temperature (Gilbert and Mickleborough 2002). Concrete starts shrinking when drying commences and continues to increase with time at a decreasing rate. A larger drying surface to volume ratio, the more rapid the rate of drying and the higher magnitude of shrinkage strain (Gilbert and Mickleborough 2002). Figure 6 illustrates the shrinkage effects on a beam element. It can be seen that the beam top surface is subject to compression due to the shrinkage effects.

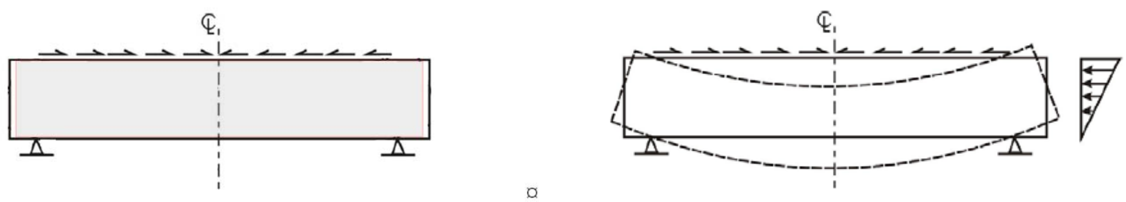

Figure 6 Shrinkage Effects

Note that in the top part of Figure 6 the concrete beam is unrestrained. The two point supports provide external restraints to the bottom flange and the heavy reinforcement in the top flange provide an internal restraint. The bottom flange is completely under compression from the applied prestressing which protects it against cracking. The stress condition in the top fibers is highly influenced by the degree of restraint. For example, if the beam end is fixed without allowing the shortening of the top fibers, the shrinkage of concrete would induce tension stresses instead of compression. Similarly, if the concrete is heavily reinforced as the case in these girders, concrete shortening due to shrinkage will be restrained by the bonded steel reinforcement since the steel does not shrink, which results in an equal, opposite tension stress in the concrete. While this tensile stress itself may not be sufficient to crack the concrete, however, it could be sufficient when combined with other tensile stresses from other external and internal loading.

Hence, the effects of restrained shrinkage and temperature change shall be taken into account, and superimposed to the stress calculations based on the stress due to prestress release. several assumptions are made to account for unknown parameters, as follows, for the purpose of analysis.

- All strands are released at Day 4 (Concrete placement occurs at Day 1).
- Simply consider the top flange only to determine the restraint degree.
- The age-adjusted effective modulus of elasticity of concrete $E''_c = E_c$.
- 30 microstrain (30×10^{-6}) for concrete shrinkage at Day 4
- Assume 70^o F temperature change between the time of concrete placement and Day 4, when prestress release occurs, based on Figures 7 and 8.

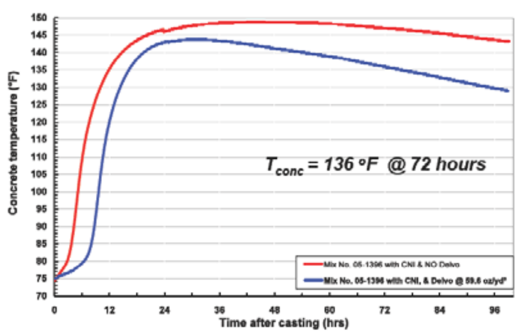

Fig.7 Temperature Rise in Concrete

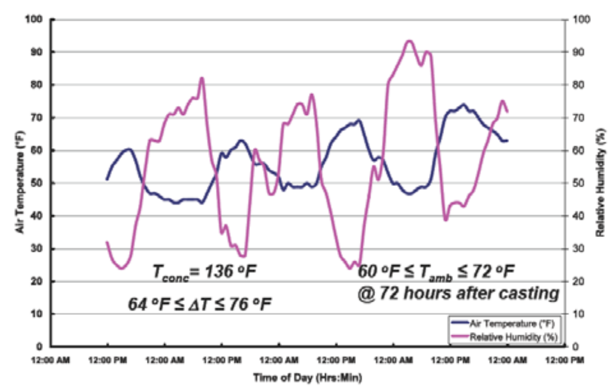

Fig.8 Environmental Condition from 2/18 to 2/21/11

The degree of restraint, R, from the top reinforcements could be calculated by Eq (2) as follows.

$$R = \frac{A_S E_S}{A_S E_S + A_C E_C} \qquad (1)$$

Where, E_c=4696ksi (f_c'=6ksi); E_s=29,000ksi; A_s=4.685 in^2 (6-#7 & 5-0.6 in. dia. Strands); A_{cg}=276.6 in^2 (top flange area); A_c=A_{cg}-A_s=271.9 in^2. Then R can be calculated as equal to 0.0962.

Hence the tensile stress due to the restraint shrinkage and the temperature change in top flange can be calculated as below.

$f_t = RE''_C(\alpha\Delta T + \varepsilon_{sh})$=0.0962*4696*(6*10^{-6}*70+30*10^{-6})=0.203ksi

Table 4 shows the top stress in girder top flange due to the effects of strands prestress and effects of restraint shrinkage and the temperature.

Table 4 Top Stress Along The Modified FBT-78 Beam

Section	0.0L	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L
Distance from beam end, ft	0	8.413	16.825	23.238	33.65	42.063	50.475	58.888	67.300	75.713	84.125
Pre-stress, ksi	-0.727	-0.706	**-0.518**	-0.384	-0.303	-0.276	-0.303	-0.384	**-0.518**	-0.706	-0.727
T & Sh, ksi	-0.203	-0.203	-0.203	-0.203	-0.203	-0.203	-0.203	-0.203	-0.203	-0.203	-0.203
Total, ksi	-0.930	-0.909	-0.721	-0.587	-0.506	-0.479	-0.506	-0.587	-0.721	-0.909	-0.930
Allowable Tension, ksi	-0.93 ($12\sqrt{f'_{ci}}$)		-0.465 ($6\sqrt{f'_{ci}}$)							-0.93 ($12\sqrt{f'_{ci}}$)	

Effects of the Beam Support Location During Storage

Once the girders are transported to the storage areas they are placed on wood blocks 3 feet away from the girder ends as illustrated in Figure 9 until transporting to the job site. The change in the support conditions alters the stresses in the beams due to the change in the effect of the self-weight of the girder.

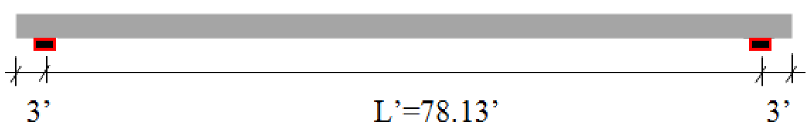

Figure 9 Storage of Prestressed Beam at Two Points (Not to scale)

Figure 10 schematically shows the bending moment diagram for the beam under its own self-weight.

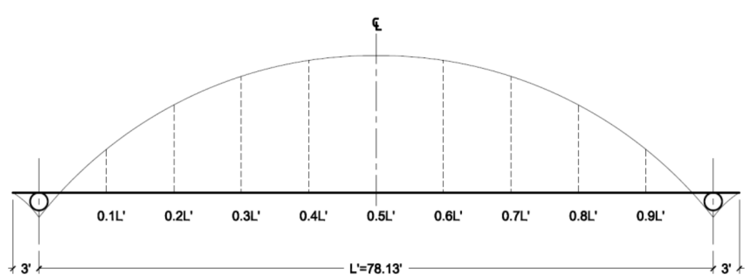

Figure 10 Schetch of the Beam Moment Diagram under Own Weight (Not to scale)

Therefore, the concrete stress in the top flange can be calculated at storage stage, as shown in Table 5.

Table 5 Comparison of Beam Top Stress

Section	0.0L	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L
Distance from beam end, ft	0	8.413	16.825	23.238	33.65	42.06	50.475	58.888	67.300	75.713	84.125
Transfer, ksi	-0.727	-0.706	**-0.518**	-0.384	-0.303	-0.276	-0.303	-0.384	**-0.518**	-0.706	-0.727
Storage, ksi	-0.727	-0.803	-0.614	-0.506	-0.399	-0.372	-0.399	-0.506	-0.614	-0.803	-0.727
Difference, ksi	0	-0.097	-0.096	-0.122	-0.096	-0.096	-0.096	-0.122	-0.096	-0.097	0

It can be seen that the top tension increase, but not significant, after the beams are transported to the storage area, due to the change of self-weight condition. Such increase in tensile stress shall be superimposed to the total tensile stress at prestress transfer as shown in Table 4. Table 6 shows the tensile stress on top girder flange right after the change of supports at storage site.

Table 6 Beam Top Stress at Storage Site (Day 4)

Section	0.0L	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L
Distance from beam end, ft	0	8.413	16.825	23.238	33.65	42.06	50.475	58.888	67.300	75.713	84.125
Top stress, ksi	-0.930	-1.006	-0.817	-0.709	-0.602	-0.575	-0.602	-0.709	-0.817	-1.006	-0.930

Effects of Shrinkage under Storage Conditions

The shrinkage of concrete starts when drying commences and continues to increase with time. As stated in AASHTO LFRD Specification, shrinkage is affected by several factors as follows:

- Aggregate characteristics and proportions
- Average humidity at the bridge site
- W/C ratio
- Type of cure
- Volume to surface area ratio of member
- Duration of drying period

The influence of the above listed factors are taken into account in the calculation of the shrinkage strain. For concretes devoid of shrinkage-prone aggregates, the strain due to shrinkage, ε_{sh}, at time, t, may be taken as:

$\varepsilon_{sh} = k_s\, k_{hs}\, k_f\, k_{td}\, 0.48 \times 10^{-3}$ (LRFD A5.4.2.3.3-1)

in which:

k_s = factor for the effect of the volume-to-surface ratio of the component (Note that the surface area should include only the area that is exposed to atmospheric drying.)

k_{hs} = humidity factor for shrinkage

k_f = factor for the effect of concrete strength

k_{td} = time development factor

Where:

$$k_{hs} = 2.00 - 0.014H \quad \text{(LRFD A5.4.2.3.3-2)}$$

$$k_s = \frac{\dfrac{t}{26e^{0.36(\frac{V}{S})} + t}}{\dfrac{t}{45+t}} \frac{1064 - 94(\frac{V}{S})}{923} \quad \text{(LRFD AC5.4.2.3.2-2)}$$

$$k_f = \frac{5}{1+f'_{ci}} \quad \text{(LRFD A5.4.2.3.2-4)}$$

$$k_{td} = \frac{t}{61-4f'_{ci}+t} \quad \text{(LRFD A5.4.2.3.2-5)}$$

Where: t = maturity of concrete (day), defined as age of concrete between time of loading for creep calculations, or end of curing for shrinkage calculations, and time being considered for analysis of creep or shrinkage effects.

In addition, LRFD Specification states: if the concrete is exposed to drying before 5 days of curing have elapsed, the shrinkage as determined in LRFD A5.4.2.3.3-1 should be increased by 20 percent.

Considering the reality of the casting of the modified FBT-78 Beam, the prestress release occurred on the 3rd day after casting, after which the girders are moved to the storage yard in the same day. During the first three days, the girders are wet cured while sitting in the formwork and the top surface is covered by a tarp. During this initial period the beam is cured under wet conditions and drying of the concrete is limited.

In the following analysis, it is assumed that the stress due to the shrinkage will commence after three days from casting and once the girders are moved to the casting yard. The entire beam surfaces are exposed for drying under storage.

The surface of the entire beam (drying surface) can be calculated by two components as below:

Total Surface=Perimeter of beam cross section multiplied by Beam length + Areas of both ends

The exposed relative humidity used in the calculation of shrinkage strain is assumed to be 75% in accordance with AASHTO LRFD Specification (Figure 5.4.2.3.3-1).

As discussed above the drying shrinkage strain for the initial 3 days should be ignored in the calculation of shrinkage stresses during storage. Table 7 shows the calculated shrinkage stresses at different ages. The "1.2 ε_{sh} " column of the table accounts for 20% increase in the stress according to AASHTO LRFD Eq. A5.4.2.3.3-1 since the concrete was exposed to drying before 5 days of curing.

Table 7 Concrete Shrinkage Strain with Time

Concrete Age, Days	Shrinkage Strain ε_{sh}, $\times 10^{-6}$ in/in	1.2 ε_{sh}, $\times 10^{-6}$ in/in	Tensile Stress due to restraint concrete Shrinkage, $\sigma_{Sh}=RE_c*(1.2\ \varepsilon_{sh})$, ksi
4	0.00	0.00	0.000
6	38.22	45.86	0.021
8	59.67	71.60	0.032
13	103.0	123.60	0.056
18	135.7	162.84	0.074
24	165.7	198.84	0.090
38	212.3	254.76	0.115
53	242.8	291.36	0.132
78	273.0	327.60	0.148
103	290.8	348.96	0.158
153	311.0	373.20	0.169
186	318.9	382.68	0.173
253	329.0	394.80	0.178
368	338.2	405.84	0.183
503	343.8	412.56	0.186
1003	355.6	426.72	0.193
2003	359.2	431.04	0.195

Figure 11 shows the plot for the concrete shrinkage strain vs. concrete age. It can be seen that approximately over 90% of shrinkage takes place in the first year.

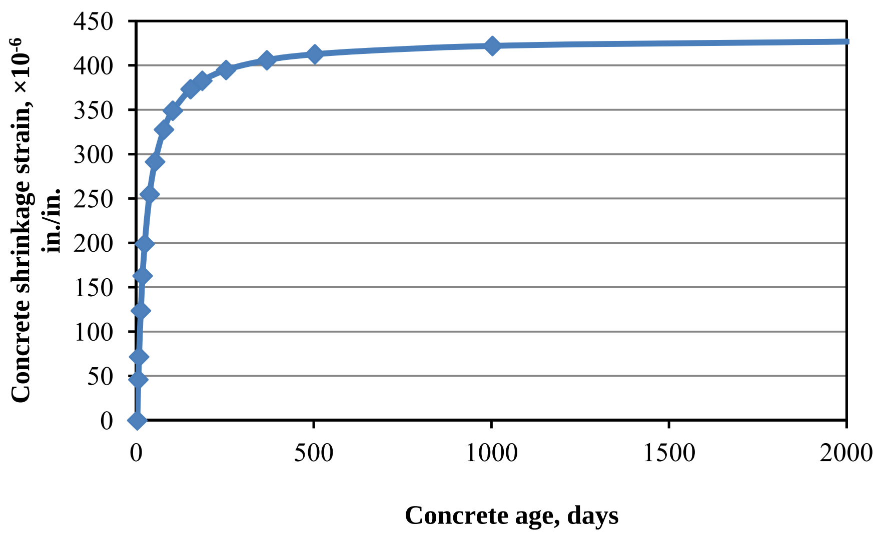

Figure 11 Girder Concrete Shrinkage vs. Concrete Age

Table 8 shows the total tensile stress at different ages due to the prestress, girder self-weight and restraint shrinkage. As can be seen, the net tensile stress in concrete could exceed the cracking modulus of rupture depending on the exposure time and the concrete design strength.

Table 8 Total Tensile Stress at Different Ages, ksi

Section		0.0L	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L
Distance from beam end, ft		0	8.413	16.825	23.238	33.65	42.063	50.475	58.888	67.300	75.713	84.125
After release (day 4)		-0.930	-1.006	-0.817	-0.709	-0.602	-0.575	-0.602	-0.709	-0.817	-1.006	-0.930
Day 8	σ_{Sh}	-0.032	-0.032	-0.032	-0.032	-0.032	-0.032	-0.032	-0.032	-0.032	-0.032	-0.032
	Net	-0.962	-1.038	-0.849	-0.741	-0.634	-0.607	-0.634	-0.741	-0.849	-1.038	-0.962
Day 13	σ_{Sh}	-0.056	-0.056	-0.056	-0.056	-0.056	-0.056	-0.056	-0.056	-0.056	-0.056	-0.056
	Net	-0.986	-1.062	-0.873	-0.765	-0.658	-0.631	-0.658	-0.765	-0.873	-1.062	-0.986
Day 18	σ_{Sh}	-0.074	-0.074	-0.074	-0.074	-0.074	-0.074	-0.074	-0.074	-0.074	-0.074	-0.074
	Net	-1.004	-1.080	-0.891	-0.783	-0.676	-0.649	-0.676	-0.783	-0.891	-1.080	-1.004
Day 24	σ_{Sh}	-0.09	-0.09	-0.09	-0.09	-0.09	-0.09	-0.09	-0.09	-0.09	-0.09	-0.09
	Net	-1.020	-1.096	-0.907	-0.799	-0.692	-0.665	-0.692	-0.799	-0.907	-1.096	-1.020

The storage period for the girders varies from project to project and assuming a 21 day storage period (Day 24). It can be seen that the maximum tensile stress is more than twice the value of the allowable stress. Assuming that the concrete strength 21 days after release (24 days concrete age) is approximately 95% of the 28 day strength (=8.075 ksi) and assuming a concrete module of rupture equal to $7.5\sqrt{f_c'}$ (ACI 318-08), the cracking stress can be calculated at 0.674 ksi.

Table 9 shows the comparison between the calculated stresses along the girder with the cracking stress after 21 days release.

Table 9 Cracking Comparison at 21 Days after Prestress Release, ksi

Section		0.0L	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	1.0L
Distance from beam end, ft		0	8.413	16.825	23.238	33.65	42.063	50.475	58.888	67.300	75.713	84.125
21 days after release	σ_{cr} ksi	-0.674										
	σ_t ksi	-1.020	-1.096	-0.907	-0.799	-0.692	-0.665	-0.692	-0.799	-0.907	-1.096	-1.020
	% σ_{Cr}	151.3	162.6	134.6	118.5	102.7	98.7	102.7	118.5	134.6	162.6	151.3

It can be seen from Table 9 that the cracking stress is exceeded at all sections along the beam length except for the mid-span section, which clearly explain the reason for the observed cracking. It should be pointed out that the above assumptions are not conservative and in many cases the girders could be stored or left at the work site for extended period of time before placing the bridge deck and the observed cracking is likely to occur in these cases.

Moreover, additional creep induced tensile stress is likely and the effect of the temperature gradient could not be underestimated. In many cases the girders are stored side by side and the large girder flanges are subjected to the thermal radiation from the sun and the temperature deferential between the exposed surface and the girder web could reach 40 to 70 ^{o}F. Such temperature gradient could cause differential volume changes and result in tension in the top flange of the girder. In the case presented the top flange of the hanger beam was shortened and the girders were stored 2 ft. on center and the effect of the temperature gradient is insignificant.

Due to the large parameters involved and the complex interaction between these parameters, accurate prediction of top tensile stress is complicated. The work presented in this paper is intended for shedding light on other parameters that influence the cracking of prestressed girders and should be considered in the design process.

SUMMARY AND CONCLUSIONS

The geometry, high concrete strength and reinforcement details of the new class of prestressed girders should be taken into account during the design stage to minimize the potential for transverse cracking.

The stresses due to early age concrete shrinkage coupled with thermal effects and additional stresses due to storage conditions could result in a significant tensile stress that when superimposed to the design stress is sufficient to induce transverse cracking of the top flanges.

A case study is presented to illustrate the effect of these parameters. The causes for the top flange cracking of modified FBT-78 beam are investigated by looking into concrete mix, and the top tensile stress under the combined effects of prestressed strand, beam self-weight, thermal effect and restraint shrinkage. Based on the results, it can be concluded that the main cause for the top flange cracking of the beam studied herein is due to the heavily reinforced top bars/strands that restrain the concrete from shortening due to shrinkage, and the thermal effect before prestress transfer.

RECOMMENDATIONS

The following recommendations are made in order to avoid or minimize such cracking problem at early concrete age:

1. **Age at Prestress Release**
   - In general the prestress release should be carried out within 24 to 36 hours of pouring the beams to minimize restraining effect and associated tension stresses due to early age shrinkage.

   - Release top strands at 24 hours regardless of whether the initial concrete compressive stress is achieved or not. While in the bed the top flange is exposed and the top strands are tensioned to a lower value. Early release of these strands has the following benefits:

     - Applying early compression to the top flange to counter act the tension from early concrete shrinkage.
     - Releasing the restraining effect of the top strands will minimize the shrinkage effects at early age.

2. **Curing Period and Methods**
   - Consider using steam cure followed by a period of wet cure. Steam curing will help in early achieving of the initial specified compressive concrete strength and will allow releasing the prestress within 24 hours of pouring. Note that the early application of prestressing effects will help minimize shrinkage cracking.

3. Prestress Level for Top Strands

Consider increasing the prestress level for the top strands to 50% of the prestress level applied to the bottom strands. Increasing the prestress level coupled with early release of the top strands will help minimize shrinkage cracks in the top flanges.

4. Set guide lines and limits on the use of top reinforcement

State DOT's design guidelines should address this issue and alert the designer that counter measures should be used to control early shrinkage due to these restraining effects. Using significant amount of top reinforcement in the top flanges of the beams add significant restraining effects and will most often result in shrinkage cracking.

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REFERENCES

1. AASHTO LRFD Bridge Design Specifications (2010), American Association of State Highway and Transportation Officials, Washington, DC.
2. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary, American Concrete Institute, Farmington Hills, MI, 2008, 465pp.
3. Aìtcin, P.C., et al., "Integrated View of Shrinkage Deformation," Concrete International, V. 19, No. 9, Sept. 1997, pp. 35-41.
4. Collins, M.P., and Mitchell, D., *Prestressed Concrete Structures*, Response Publicatios, Toronto, ON, Canada, 1997, 614pp.
5. *fib*, "Structural Concrete, Textbook on Behaviour, Design and Performance, Updated Knowledge of CEB/FIP Model 1990," V.1, Féderation Internationale du Béton, Lausanne, Switzerland, July 1999, 269pp.
6. Gajda J. and Vangeem M., "Controlling temperatures in mass concrete," Concrete International, Jan. 2002, pp. 59-62.
7. Gilbert, R.I. and Mickleborough, N.C., *Design of Prestressed Concrete*, Spon Press, Taylor & Francis Group, New York, NY, 2002.
8. Holt, E. E., "Early Age Autogenous Shrinkage of Concrete," Espoo 2001, Technical Research Centre of Finland, VTT Publications 446.
9. Khayat, K. H. and Long, W.J. , "Shrinkage of Precast, Prestressed Self-Consolidating Concrete", ACI Materials Journal, V. 107, No. 3, May-June 2010, pp.231-238.
10. Tuchscherer, R.G. and Bayrak, O., "Tensile Stress Limit for Prestressed Concrete at Release," ACI Structural Journal, V.106, No.3, May-June 2009, pp.279-287.