EVALUATION OF PRESTRESS LOSSES FOR BRIDGE A7957 CONSTRUCTED WITH HIGH STRENGTH CONCRETE (FIELD STUDY)

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ABSTRACT

Two precast prestressed high-strength concrete (HSC) girders in the bridge A7957 were designed and erected on HWY 50, Missouri State, USA, for an in situ investigative study to monitor time-dependent losses. Vibration wire strain gages (VWSGs) were utilized as internal sensors within the beam cross sections to monitor the temperatures and strains. The measured prestress losses were compared to various expressions presented by AASHTO and the Precast/Prestressed Concrete Institute.

Keywords: Prestressed concrete, High-Strength Concrete, Prestress losses, Code Models.

INTRODUCTION

High strength concrete (HSC) allows greater design flexability for bridge designer in precast prestressed (PC/PS) concrete structures. It permits longer span structures that result from the use of more compact sections. HSC can be used to lower initial project cost by designing bridges with longer spans for a given girder cross section or reducing the number of girders by increasing the girder spacing[1]. ACI363-2R defines HSC as a type of concrete with specific 28-day strength of 8000 psi or greater[2]. It is clear that high strength is made possible by reducing porosity, inhomogeneity, and micro-cracks in the hydrated cement paste and the transition zone. HSC is considered to be more durable than conventional concrete. However, its production requires more attention to quality control than conventional concrete. Mix design of high strength concrete needs to use strong and durable aggregate, high cementitious materials content, and generally needs to have a low water-cementitious materials ratio (w/cm).

HSC mixture design can vary depending on locally available materials that make the fresh concrete workable and that ensure the strength development is as specified by the designer. With the variety of selected materials and requirements, many performance-related issues require more inspections. Differences in the amount of time-dependent losses are one example of an area currently under investigation. Understanding and predicting the prestress losses is essential to the design of concrete beams. If care is not taken to determine the prestress losses, the designer can potentially over-stress the structural members during serviceability states.

Prestress losses are the losses in tensile stress of prestress steel that affect the performance of prestressed concrete section. The tensile force in the tendon does not stay constant from the recorded value in the jacking gauge, but changes with time. The losses are classified into two categories: immediate and long term or time-dependent losses. The immediate losses take place during prestressing of the tendon and transfer the prestress to the concrete member. The elastic shortening (ES) and slip of the anchorage are the immediate losses. On the other side, the losses due to creep of the concrete (CR), shrinkage of the concrete (SH), and relaxation of the tendon (RE) are considered time-dependent losses[3]. Current empirical models used to determine the components of prestress losses in HSC are provided by the AASHTO LRFD Bridge Design Specifications (AASHTO 2102)[4] and the Precast/Prestress Concrete Institute (PCI) Design Handbook (PCI 2007)[5].

To date, there have been a limited number of full scale studies conducted to determine the long-term behavior of prestressed HSC beams. In a recent study by Myers et al. (2013) at Missouri University Science and Technology, two precast prestressed high-strength concrete (HSC) and high strength self-consolidating concrete beams were instrumented[1]. The HSC bridge spans a length of 48 ft (14.6 m) and has a width of 10 ft (3.0 m). The HS-SCC spans a length of 34 ft (14.6 m) and has a width of 10 ft (3.0 m). A total of 32 VWSGs with built-in thermistors were used in the beams and decks. Two data acquisition system boxes were used to monitor both bridges. The researchers incorporated two commonly used loss estimate models for calculating total prestress losses, including the AASHTO and Prestressed Concrete Institute (PCI). The researchers reported that the losses in HSC and HS-SCC bridges were approximately 6.21% and 4.86%, respectively, of the nominal jacking stress. It was concluded that the AASHTO LRFD method overestimated the prestress loss of HSC by 23 % and HS-SCC by 57% when measured modulus of elasticity of the material was used in the predicted model. The PCI Design Handbook was not as accurate and overestimated the total prestress loss by 24% for HSC and 85% for HS-SCC when measured modulus of elasticity of the material was used in the predicted model.

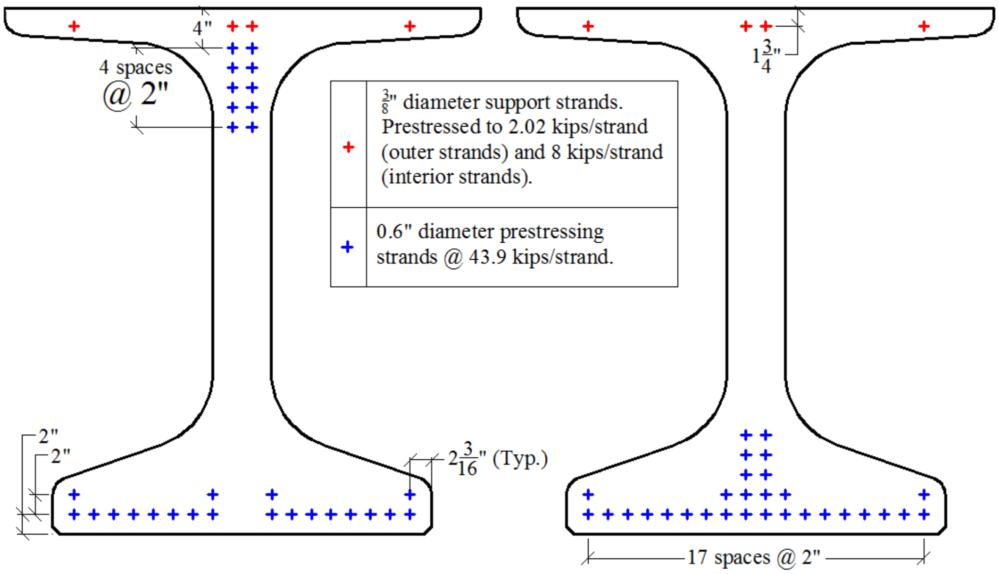
In a study conducted by Roller et al. (2011)[6], four 131 ft (40 m) long full scale bridge girders were instrumented in Louisiana to evaluate the prestress losses in HSC bulb-tee girders for the Rigolets Pass Bridge. The measured total prestress losses derived from concrete strains corrected for temperature and load effects were found to be less than corresponding values calculated using AASHTO LRFD Bridge Design Specifications.

BRIDGE DESCRIPTION

The A7957 Bridge on Highway 50 is located in Osage County, Missouri. The bridge has three spans with PC/PS concrete girders. The bridge was designed to be simply supported for dead load and continuous for live load via a CIP deck, as seen in Figure 1. Each span was designed with concrete mixtures of different compressive strength. The two exterior spans are 100 ft (30.5 m) long and one interior is 120 ft (36.6 m) long. The superstructure is supported by two intermediate bents and two abutments. The bridge has a superelevation of 2.0%.

Fig. 1 View of the A7957 Bridge

Each span implemented four PC/PS Nebraska University 53 (NU53) girders. The NU 53 girder was developed by the University of Nebraska’s Center for infrastructure Research in cooperation with the Nebraska Department of Roads. The girder’s cross section provides several advantages during construction, giving designers more flexability to increase strand capacity and reduce stress concentration in the edges by curved fillets (see Figure. 2). Span one with HSC was utilized for this study. The beams were prestressed by 30, Grade 270 steel tendons: 20 straight and 10 harped at double harping points. The 0.6 (15 mm) diameter tendons were 7-wire, low-relaxation strands. Four additional 3/8 in. (9mm) diameter prestressing strands were added within the top flange of each girder for crack control. The jacking force per strand was approximately 44 kips, slightly overstressed to 45 kips to compensate for anchorage losses.



a) End view b) Mid-span view

Fig. 2 Cross section view of NU 53 girder

The target 28-day compressive strength of HSC was 8000 psi (55.2 MPa) and the specified release strength was 6500 psi (44.8 MPa). The mixture proportion of HSC mix design is presented in Table 1. Steam curing regime was utilized to accelerate the hydration process of all PC/PS girders. The maximum temperature of steam regime did not exceed 120 °F (49 °C). The precast girders and deck panels were fabricated in August 2013 at County Materials Corporation, located in Bonne Terre, Missouri, USA. Erection began in September 2013. The deck slab was cast from the east side to the west of the girder, after the erection of girders at the site in October 2013. The bridge entered into service (i.e., opened to traffic) during the middle of 2014 after the roadway was completed.

Table 1 HSC mixture proportions

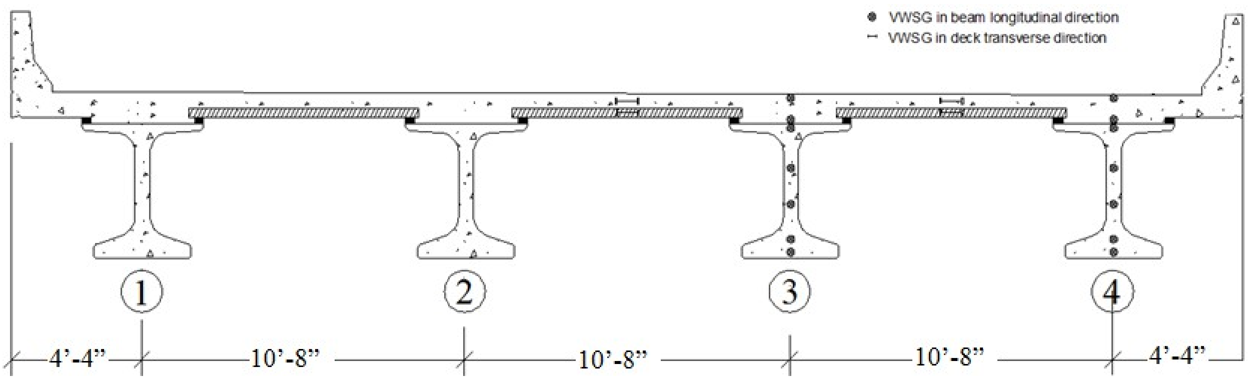
Type	Material	(HSC)
Coarse Aggregate, (lb/yd^3)	(1/2”)Lead Belt, Park Hills Stone Masonry Grade E Dolomite	1780
Fine Aggregate, (lb/yd^3)	Weber, Cristal City Sand/Class A Ledges 4-1	1085
Cement, (lb/yd^3)	Portland Cement – Type I	800
w/c	---	0.32
Chemical Admixtures, oz/yd^3	Air Entraining Agent	8.0
	Water Reducer and Retardant	9.2
	High Range Water Reducer	17.2

Design Air Content (%)	---	5.5

Notes: 1 lb/yd^3=0.593 kg/m^3, I oz. /yd^3=37 g/m^3

MONITORING SYSTEM

The HSC girders produced for span 1 of the A7957 Bridge were instrumented to obtain data for the measured strain and temperature. Two instrumented girders (namely: S1-G3 and S1-G4) were monitored from fabrication through service life. The VWSGs locations within instrumented PC/PS girders are illustrated in Figure 3.



Notes: 1 in. = 25.4 mm

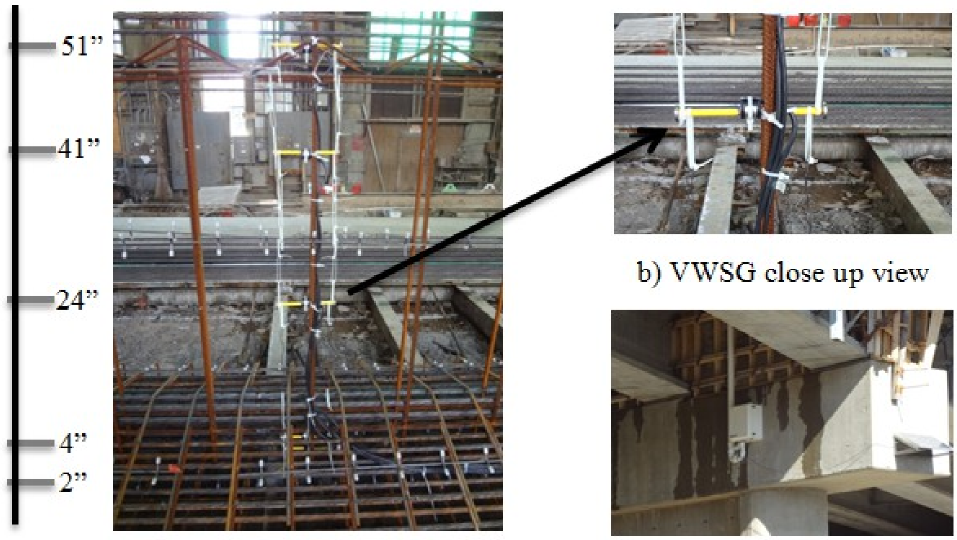
Fig. 3 Bridge A7957 cross section

VIBRATING WIRE STRAIN GAUGES (VWSGS):

A total of 26 vibrating wire strain gauges with built-in thermistors (type EM-5) were utilized to measure the strain and temperature for the PC/PS girders. The VWSGs were installed in the mid-span and east side of the girder. The standard pattern in the mid-span consisted of five gauges over the height of the girder and two more in the slab above the girder. Images of the VWSGs within the girder's height are shown in Figure 4. During construction, VWSG readings were made prior to strand release, after strand release, during transportation, erection, and before and after casting the deck slab concrete. Monitoring of the bridge is ongoing.

DATA ACQUISITION SYSTEM

The data from the VWSGs were recorded by a data acquisition system (DAS). The DAS used was Campbell Scientific CR800 box which works wirelessly. Following the erection of the girders, the CR800 DAS was anchored to the interior side of the intermediate bent pier caps for long-term monitoring. A cellular antenna, which was also anchored to the interior side of the bent 2 pier cap, was used to send the data from the CR800s in real time back to the researchers at Missouri S&T during fabrication of the precast PC girders and the different stages of the bridge construction. Measurements were taken at five minute intervals.



a) VWSGs in the midspan

b) VWSG close up view

c) DAS attached to the pier bent cap

Fig. 4 VWSGs Installation

MATERIAL PROPERTIES

Material property tests were performed on specimens collected from the same batch of HSC girders to have adequate predictions for the prestress losses. All the tests followed standard ASTM guidelines. A summary of the tests, test methods, and results are presented in Table 2.

Table 2 Summary of HSC tests and results

Tests	Test Method	Specimens	Concrete Age	Member (Span-Girder) S1-G3	Member (Span-Girder) S1-G4
Compressive Strength, (psi)	ASTM C39-12	4-in. diameter x 8-in. long cylinder	Release	6896	7635
			28 days	10774	9733
			365 days	10236	10642
Modulus of Elasticity, (ksi)	ASTM C469-10		Release	4435	4717
			28 days	5223	5143
			365 days	5648	5604
Modulus of Rupture, (psi)	ASTM C78-10	6 x 6 x 21/24 in. beams	28 days	587	653
Coefficient of thermal Expansion, (με/°F)	ASTM C490-11	4-in. diameter x 48-in. long cylinder	180 days	4.97	

Note: 1 ksi= 6.896 MPa; 1 in. = 25.4 mm; με/°C= 1.8 με/°F

PRESTRESS LOSSES

ELASTIC SHORTENING LOSSES

The VWSGs embedded in the concrete girder were utilized to measure elastic shortening (ES) as an indirect measurement. These measurements were obtained by subtracting the strain reading immediately after release from the baseline strain measurement recorded just before release. Measurements were taken at the level of the strand's c.g.s. The measuring strain was corrected due to the thermal effect and multiplied by the modulus elasticity of the prestressing strands (E_{ps}=28500 ksi) to determine prestress losses, as demonstrated in Eq. (1).

(1)

(2)

Equation (2) was used to calculate the change in stress from ES. In Eq. (2), f_{cgs} is the stress of the concrete at the centroid of the prestressing strands, E_{ps} is the modulus of elasticity (MOE) of the prestressing strands, and $\boldsymbol{E_{ci}}$ is the MOE of the concrete at release. Equation (3) was used to estimate the stress of the concrete at release.

(3)

Where, P is the estimated force immediately after release, A is the cross-sectional area, Ig is the gross moment of inertia (uncracked section), e is the eccentricity of the strand, and M is any moment applied to the beam.

The measured elastic shortening losses were determined and compared with the empirical equation adopted by AASHTO LRFD (2012) and the PCI Design Handbook (2007) with the actual MOE and approximate MOE specified by American Concrete Institute (ACI363-97) for HSC, presented in Eq. (4). Table 3 displays the results.

(psi) (4)

The comparison between measured to empirically determine is summarized in Figure 5. The measured elastic shortening values were typically higher than those predicted by the AASHTO LRFD, and PCI methods. The AASHTO LRFD method underestimated the ES losses by 25% and 30% when measured and predicted MOE were used, respectively. However, the PCI method tended to underestimate the ES losses of HSC by 35% and 32% when measured and predicted MOE were used, respectively. As a result, AASHTO LRFD method was considered more accurate than the PCI method.

Table 3 HSC elastic shortening losses

HSC								
Result Method	S1-G3				S1-G4			
	Strain (με)	Stress (psi)	% Jacking	P/M Ratio†	Strain (με)	Stress (psi)	% Jacking	P/M Ratio
Measured	632	18024	9.1	1.00	710	20235	10.2	1.00
AASHTO*	521	14855	7.5	0.82	490	13968	7.0	0.69
AASHTO**	483	13769.5	6.9	0.76	459	13086	6.6	0.65
PCI*	452	12869.4	6.5	0.71	425	12100.6	6.1	0.60
PCI**	463	13206.8	6.6	0.73	446	12697	6.4	0.63

* Methods using measured MOE, ** Methods using approximate MOE (Eq. 4), and † Predicted to measured ratio

Fig. 5 Comparison of HSC elastic shortening losses with empirical models using measured MOE

COMPARISON FOR TOTAL PRESTRESS LOSSES

The total prestress losses in PS/PC girders consist of elastic shortening loss (ES), shrinkage of concrete (SH), creep of concrete (CR), and relaxation of strand (RE) which are considered for serviceability cases[7,8]. These losses do not affect the ultimate strength of a girder, but they may lead to poor predication of service camber and deflection[9]. Empirical models have been provided by AASHTO LRFD and PCI to determine the components of prestress losses separately.

The strain readings at c.g.s from VWSGs were utilized to measure the total prestress losses in the concrete girder at 330 days. These values were determined through strain compatibility using the portion of prestress losses due to elastic shortening, creep, and shrinkage, but neglecting the portion of the prestress losses due to strand relaxation because relaxation occurs in the strand at a constant strain. The measured prestress losses at 330 days were compared to predicted losses presented by AASHTO LRFD and PCI using the measured elastic modulus of HSC.

It can be inferred from Figure 6 (in average) that AASHTO LRFD and PCI methods underestimated the total prestress loss by 1.7% and 2.4%, respectively. From the aspect of components comparison, the total amount of creep and shrinkage losses presented by AASHTO LRFD and PCI methods were overestimated by approximately 28% and 39%, respectively. Exterior girder (S1-G4) experienced higher measured losses than interior girders (S1-G3) and that might be because the difference in the MOE (~44 ksi).

Fig. 6 Comparison of HSC measured prestress losses

CONCLUSION

This full-scale study was conducted to determine the long-term behavior of prestressed HSC beams. Based on this research, the following conclusions can be drawn:

A DAS system and VWSGs have been successfully installed and are functioning adequately to collect strains and temperatures in the HSC girders of bridge A7957 during fabrication, erection, and service life.

HSC girders exhibited elastic shortening losses 19 ksi on average. The elastic shortening losses represented the significant part of measured prestress losses.

The average measured prestress loss of HSC girders at 330 days was 34 ksi. This value was determined through strain compatibility using the portion of prestress loss due to elastic shortening, creep, and shrinkage, but neglecting the portion of the prestress losses due to strand relaxation because relaxation occurs in the strand at a constant strain.

Both AASHTO LRFD and PCI empirical models underestimated the elastic shortening losses of HSC with either actual or predicted MOE.

In general, AASHTO LRFD tended to be more accurate than the PCI method in predicting HSC prestress losses.

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