LIVE LOAD TEST RESULTS OF MISSOURI'S FIRST HIGH PERFORMANCE CONCRETE SUPERSTRUCTURE BRIDGE

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ABSTRACT

High performance concrete (HPC) is becoming more widely utilized in highway bridge structures. Bridge A6130 is the first fully HPC superstructure bridge in Missouri. To study the overall behavior of the bridge under live load, a static live load test was developed and carried out in June, 2002. 64 embedded vibrating wire strain gages and 14 embedded electrical resistance strain gages were used to acquire the changing strain rate in the bridge caused by the varying live load conditions. Girder deflections and rotations were also recorded using external sensors and a data acquisition system. The ASSHTO specifications (1994) live load distribution factor recommended for design was compared to the measured value and found to be overly conservative. The AASHTO LRFD specification (2002) live load distribution factor found appeared appropriate for HPC bridges. Two finite element models were developed and analyzed using ANSYS to investigate and compare the continuity level of the MoDOT interior bent detail to measured values.

Keywords: Load Test, High Performance Concrete, Instrumentation of Bridges, Load Distribution Factor, PC Girder Continuity.

INTRODUCTION

Through optimization of mix proportions using chemical admixtures and pozzolanic materials, high performance concrete (HPC) with design compressive strengths between 69 MPa (10,000 psi) and 90 MPa (15,000 psi) have been successfully produced with conventional materials and concrete production methods. The latest developments in the pretensioned concrete industry, including the use of 15.24-mm (0.6-in.) diameter strands, have also enhanced the benefits of HPC enabling designers to take advantage of higher strength concretes. High performance bridges with HPC and large diameter prestressed strands are becoming more and more attractive to designers. To implement more widespread use of HPC in Missouri, the Missouri Department of Transportation (MoDOT) has co-sponsored a research study in Missouri to investigate both the early-age and later-age performance of a widely used PC bridge system in Missouri that includes the use of HPC and larger prestressing strands. Monitoring of these structures during the construction period and the service life can provide a beneficial understanding of the entire behavior of these structures and therefore offer useful reference for designers, contractors and researchers.

To investigate the overall behavior of the bridge under live load, a static live load test was carried out in June, 2002. During the live load test, 64 embedded vibrating wire strain gages (VWSG) and 14 embedded electrical resistance strain gages (ERSG) were used to acquire the changing strain rate in the bridge caused by the varying live load conditions. Girder deflections were also recorded using LVDT and precise surveying equipment. In addition, 4 inclinometers were used to obtain the angle deformation due to the varying live load. Two data acquisition systems (DAS) were used to capture the strain gauges, LVDT and inclinometer readings.

The test results including strain and deflection were presented and found to be reasonable. Based upon the strain and deflection data captured, the load distribution among the girders was studied. The ASSHTO specifications and AASHTO LRFD live load distribution factor recommended for design was compared to the measured value. To investigate the continuity of the MoDOT interior girder bent detail, two finite element models were developed considering different boundary conditions and analyzed using ANSYS. These models were compared to measured values to access the continuity level of the MoDOT detail.

BRIDGE INTRODUCTION AND LOAD TEST PROGRAM

Bridge A6130 was designed as a five-span bridge in Pemiscot County located near Hayti, Missouri. The span lengths of the bridge are 15.5m (50.9-ft), 17m (55.8-ft), 17m (55.8-ft), 17m (55.8-ft) and 15.5m (50.9-ft), respectively. The width of the road is 11.5m (37.7-ft) with 410mm (16.1-in.) safety barrier curbs. Loading criteria for the design was MS18 (modified), which is equivalent to AASHTO HS20-44 modified. This is the first bridge in Missouri that fully implement HPC into the superstructure of the bridge which includes the girders and bridge deck.

Precast prestressed beams were designed to incorporate high-strength/high performance concrete (HS/HPC). The required design compressive strength was 70MPa (10,152 psi) with a required release strength of 52MPa (7542 psi). The use of 15.24mm (0.6-in) diameter pretensioned strands is employed to make full use of the high-strength concrete. All twenty main span girders used in the bridge are MoDOT type 2 girders. The use of HS/HPC enabled the designers to reduce the number of girders from 6 using conventional strength concrete to 4 using HS/HPC. HPC is also used in the cast-in-place deck with a thickness of 230mm (9.055-in) including the use of mineral admixtures to obtain a highly impermeability concrete. The abutment and bent lines are projected on a skew at an angle of 48^{o}.

Concrete strain and beam deflection were the basic components to be monitored during the load test. A total of 64 internal vibrating wire strain gauges (VWSG), 14 internal bonded electrical resistance strain gauges (ERSG) and 16 internal thermocouples were embedded in the PC girders and CIP deck. A data acquisition system (DAS-1) with sufficient channels was designed and assembled for the project. In total, 6 girders were instrumented and 4 locations in the deck. VWSG and ERSG were embedded in girders B13, B14, B23 and B24 at mid-span section and near support section as illustrated in Figure 1. These strain gauges were used for the live load test study. VWSG were used to determine a profile of strain along the depth of the section as well as the temperature profiles during various stages of fabrication, construction and service. ERSG were used as redundant gauge for strain measurement similar to previous studies by Gross and Burns[1]. More specifics about the instrumentation plan for this bridge may be found in a paper by Yang, Shen, and Myers[2].

LVDT's and surveying equipment was used for deflection measurement. Seven LVDT's were used for deflection measurement. As shown in Figure 1, four LVDT's were used for the beam midspan deflection measurement and three were used at the midpoint of the deck between girders. Inclinometers were place on the deck to obtain the slope deformation as shown in Figure 2. A second data acquisition system (DAS-2) was shipped from the Univ. of Missouri-Rolla to the bridge site to acquiring data from the LVDT's and inclinometers. Surveying equipment was used to measure the deflection of the second span girders as highlighted in Figure 2.

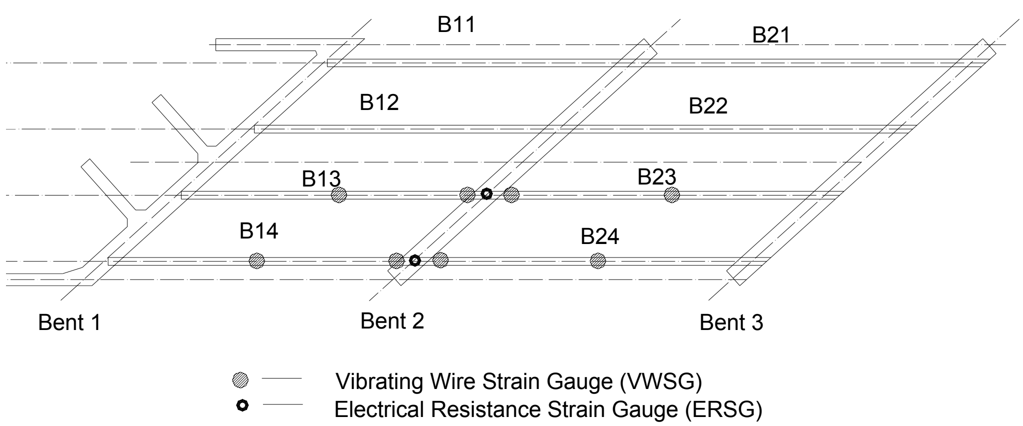


Fig. 1 Strain Gauge Connected with DAS-1

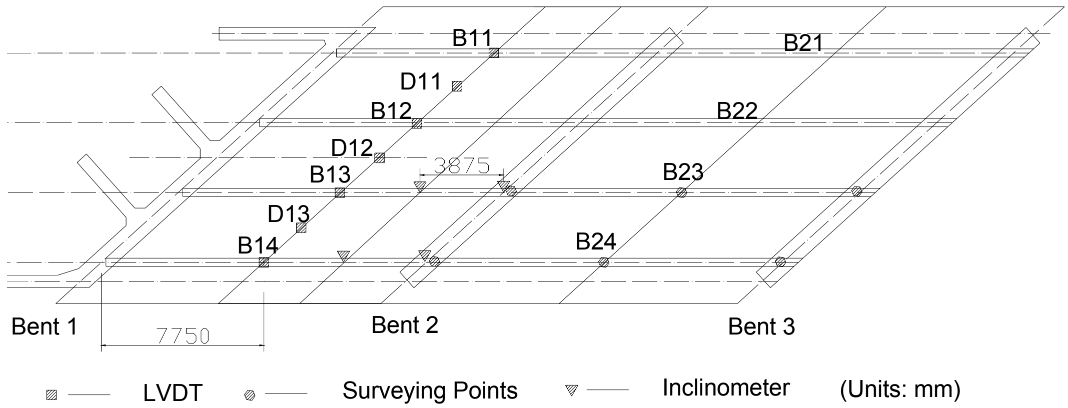
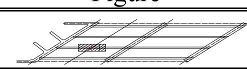
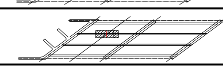
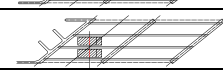
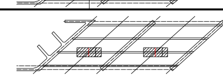
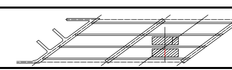


Fig. 2 Deformation Measurement Points for Live Load Test

Two MoDOT dump trucks from Missouri District 10 were used for the load test. A SHD 5935 truck loaded up to 201.6 kN (47380 lb) and a SHD 6032 truck loaded up to 218.8 kN (49220 lb) were used in the live load test. The two trucks are of identical configuration but are slightly different in overall and axle weights. Six load cases were implemented for the load test. The truck location in each case is described in Table 1. These load cases were selected to study both load distribution across the width of the bridge and bent rotation at the interior bent.

Table 1 Load Case Description

Load Case	Description	Figure
A	One truck centered over mid-span of B13	
B	One truck centered over mid-span of B12	
C	Two trucks centered over mid-span of B13; one on each side	
D	One truck centered over mid-span of B13 and one truck centered over mid-span of B23	
E	Two trucks centered over mid-span of B23; one on each side	
F	One truck centered over mid-span of B23	

LIVE LOAD TEST RESULTS

The curvature was determined from the slope of a linear trend line fit based on the recorded strain data. Using the composite section properties and modulus of elasticity of the concrete from match cured specimens, the moment at the section was determined. Thermal effects were investigated, but determined to be minimal and not impact the curvature results. From the strain profile, it can be seen that the embedded strain gauges used for the bridge instrumentation worked very well even when subjected to relatively small load. The strain profiles for much of the bridge cross-sections exhibited a very close linear relationship. The fitted straight line also demonstrated the beam theory assumption that plane sections remain plane for the composite section.

The live load distribution factors for design were attained from AASHTO[3] 3.23.2 and 3.23.1 and AASHTO LRFD[4] 4.6.2.2. For moment calculation with two lanes loaded, the recommended AASHTO distribution factor for exterior girders B11 and B14 is 1.614 and for interior girders B12 and B13 is 1.969 respectively. Both are controlled by fatigue and they are used for design. Applying AASHTO[3] Table 3.23.3 to consider skew effects of the support, the reduction factor of 0.88 is found. This reduces the distribution factors to 1.420 for girders B11 / B14 and 1.735 for girders B12 / B13, respectively. Similarly using AASHTO LRFD[4], much lower distribution factors were obtained as shown in Table 2 considering two lanes loaded and one lane loaded. There is no specification on fatigue effect on load distribution factor in LRFD.

For each live load case in the load test, midspan deflection of the four girders monitored in Span 1 was measured with LVDT's. To examine the distribution of load across the bridge, individual girder deflections were totaled, and then each individual girder response was divided by this total. The result is a fraction of the total bridge response that each individual composite girder carried. To avoid confusion with load distribution factor as defined in the AASHTO specification, a calculated term entitled *load distribution coefficient* is used to represent the fraction of the load that causes the maximum response in any individual girder for that particular truck crossing. The responses used for calculation can be strain or deflection[5]. Based on deflection data, the load distribution coefficient for span one girders B11, B12, B13 and B14 can be obtained as listed in Table 2. Load case E and load case F are not listed because trucks were place in the second span only for these two cases.

Table 2: Load Distribution Factor and Tested Load Distribution Coefficient for the First Span

	AASHTO Load Distribution Factor			Load Distribution Coefficient			
Beam	AASHTO Standard Two Lanes Loaded	AASHTO LRFD Two Lanes Loaded	AASHTO LRFD One Lane Loaded	Load Case A 1 truck	Load Case B 1 truck	Load Case C 2 trucks	Load Case D 1 truck
B11	1.420	0.774	0.643	0.044	0.243	0.139	0.014
B12	1.735	0.731	0.608	0.194	0.572	0.462	0.152
B13	1.735	0.731	0.608	0.618	0.202	0.934	0.707
B14	1.420	0.774	0.643	0.144	-0.018	0.465	0.126

As illustrated in Table 2, the maximum load distribution coefficient is 0.934 for B13. Load distribution factors based on AASHTO LRFD[4] were comparable to the tested distribution coefficients. Minor variation observed is expected since the tests trucks were not located according to the designed lanes, but rather located to acquire the worst case factor for the specific girder studied. An additional factor also includes the short span and large skew of the bridge. These fundamentally account for why some coefficients were slightly higher than the calculated factors by AASHTO LRFD[4]. Load distribution factors found based on AASHTO[3] specification are substantially higher than the live load test result values obtained and illustrated in Table 2. Therefore, AASHTO[6] design codes are generally too conservative for the live load distribution factor calculation and subsequently the load rating in terms of strength limit requirements. Based on the load test conducted herein, AASHTO LRFD[4] provides more appropriate load distribution factors for design.

The girders were designed as simply supported member prior to casting the cast-in-place deck and bent continuity detail. After the girders and bent were cast integrally, they were designed as a continuous beam structure. From the data above, it may be noted that for each load case negative moment develops at the near-end support section. To investigate the boundary condition, two finite element models were developed and analyzed using ANSYS, only considering girder B13, B23 and half of B33 using the composite section, applying load as live load multiplied by the load distribution factor measured. These models are used to compare the continuity level of the MoDOT interior bent detail to measured values. Since the test results are all available at each instrumented section for load case A, load case A was studied using finite element method (FEM). One model is a continuous beam model and the second is a model where the girder is fixed at the bent (beams fixed at bent model).

The response of the actual structure when subjected to the live load is much closer to the fixed end model compared to that of the continuous beam model. Use of the continuous beam approach for design would naturally yield less accurate results by underestimating the interior bent negative moment and over-estimate the midspan positive moment. The fixity level of the interior bent is nearly fixed based upon the rotation and moment levels measured and predicted by the beam fixed at bent model. It therefore is advisable to consider the girder continuity as fully fixed. To simplify the design calculations, the fixed end model is conservative and acceptable for calculating both positive and negative moment in the loaded span for the continuity detail used. For the conjunctive girder in the adjacent span (Span 2), a continuous beam model is conservative for design if Span 1 is the only span loaded; a continuous beam model will provide conservative design moments for the adjacent girders in Span 2.

CONCLUSIONS

As described herein, a static live load test was undertaken for a recently completed HPC bridge in Missouri. Further details related to the live load test and the FEM results can be found in a paper submitted to the Transportation Research Board annual meeting[7].The following conclusions are drawn based on the test results observed and FEM's developed:

1. The load test results were found to be reasonable, both in terms of deflection and strain data. The LVDT and inclinometers were very reliable and responded to minor variation in applied load. Due to the small nature of the applied live load, high girder stiffness and short relative span lengths, deflection readings were small. Subsequently, surveying equipment used for deflection measurements was not viable.

2. The embedded strain gauges used for the bridge instrumentation worked very well even when the effect of the live load was relatively minimal. The strain profiles for much of

the bridge cross-sections exhibited a very close linear relationship. The squared coefficients of determination are near 1.00 for fitting the load test data as shown in these figures. The fitted straight line also demonstrated the beam theory assumption that plane sections remain plane for the composite section.

3. The tested actual bridge live load distribution coefficients were found to be comparable to the live load distribution factors calculated using the AASHTO LRFD[4] specifications, but significantly lower than those factors calculated using the AASHTO[3,6] standard specification. The distribution factors recommended by AASHTO appeared to be overly conservation based on the live load test herein. Therefore, the AASHTO[6] specification design codes are generally too conservative for the live load distribution factor calculation and subsequently the load rating in terms of strength limit requirements using these design guides. The AASHTO LRFD[4] specification appears appropriate for HPC bridges is recommended instead.

4. The test and analysis results demonstrated that the MoDOT continuity detail at the interior bent is nearly fixed based upon the rotation and moment levels measured and predicted by the beam fixed at bent model. It therefore is advisable to consider the girder continuity as fully fixed for these types of bridge systems. To simplify the design calculations, the fixed end model is conservative and acceptable for calculating negative moment in the loaded span for the continuity detail used. This assumes there would not be softening in the negative moment region of the bent detail due to cracking from temperature variations or overloads over time.

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