EXPERIMENTAL EVALUATION AND FIELD-MONITORING OF SELF-COMPACTING-CONCRETE PRESTRESSED BOX BEAMS FOR DEMONSTRATION BRIDGE

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ABSTRACT

The advantages offered by self-consolidating-concrete (SCC) in terms of construction quality and speed are yet to be fully utilized in highway bridges due to questions on design and construction perceived to influence performance of the built members. A demonstration bridge project in Michigan is allowing the state's Department of Transportation to evaluate the short and long-term structural performance of SCC in prestressed bridges. The M-50/US-127 Bridge over the Grand River (Jackson, Michigan) features SCC prestressed box beams in 3 of its 6 beams. Three different mix design approaches to SCC are being evaluated against a reference normally consolidated concrete mix (NCC). Before implementation, performance of the SCC beams was evaluated through full-scale flexure and shear testing to ensure similar performance to the NCC beams. The SCC beams met the nominal design capacities and their performance was essentially the same as the NCC beams. With this validation, the demonstration bridge with its SCC beams was completed in October 2005. A strain and temperature continuous monitoring system was placed on the SCC beams and one NCC beam to evaluate long-term performance. Data retrieved since December 2005 indicates that the SCC beams are performing similarly to the NCC beam.

Keywords: Self-Consolidating-Concrete; SCC; Prestressed Concrete; Box Beam, Demonstration Bridge; Structural Testing; Flexural Behavior; Shear Behavior; Continuous Monitoring

INTRODUCTION

Self-consolidating-concrete (SCC) for bridge construction is yet to be widely accepted as an approved material by State Departments of Transportation due to questions regarding its quality control, structural performance and long-term behavior. Thus, while the advantages offered by SCC in terms of improved construction quality, speed, and safety due to its high and stable flow are gaining increased attention from transportation agencies, its use for highway design will require clarification of the before mentioned doubts. The demonstration project described in this paper attempts to provide some answers to the above concerns.

A common understanding of SCC is of a concrete that (a) has the fluidity that allows self-consolidation without external vibration, (b) remains homogeneous during and after placement, and (c) flows easily through reinforcement. Thus, the most important performance criterion that differentiates SCC from conventional concrete are those related to its fresh state. Achieving all of the mentioned performance requirements is not easy and it requires a compromise between the many factors influencing the mixture proportioning[1].

SCC achieves its fresh-property advantages through specially proportioned mix designs that significantly deviate from what we currently consider as ideal and developed through many years of experience and research. These deviations have raised concerns about their consequences on known qualities of concrete, including material properties, structural performance issues, and durability parameters. To this end, considerable work has been done since the introduction of SCC in the development of mix designs, characterization of its rheology and mechanical properties, and evaluation its in-situ properties. However, relatively fewer research efforts in the US and abroad are evaluating the structural performance of members built using SCC and developing design recommendations.

A research and demonstration project was thus initiated with the Michigan Department of Transportation (MDOT) through the Federal Highway Administration (FHWA) Innovative Bridge Research and Construction (IBRC) program. The goal is to implement and characterize SCC in a selected number of precast/prestressed beams of a new bridge and compare their behavior to those from conventional concrete through structural testing and long-term monitoring.

The experimental M-50/US-127 Bridge (Fig. 1a) over the Grand River in Jackson, Michigan, incorporates three beams using self-consolidating-concrete (SCC). The bridge (Fig. 1b) consists of six 27 in. x 36 in. prestressed box beams of 52 ft in length spaced at 8.5 ft, for a total bridge width of 46.5 ft. Of the six beams three were cast using three different SCC mix designs and three were cast using one normally consolidated concrete (NCC) mix. The three SCC beams along with one of the NCC beams were instrumented for long-term in-service evaluation. Before the SCC beams were placed in the demonstration bridge, their adequate flexure and shear performance was confirmed through full-scale testing. This paper provides an overview of the manufacturing details of the SCC and NCC beams, the structural testing program, and preliminary data from the field monitoring phase.
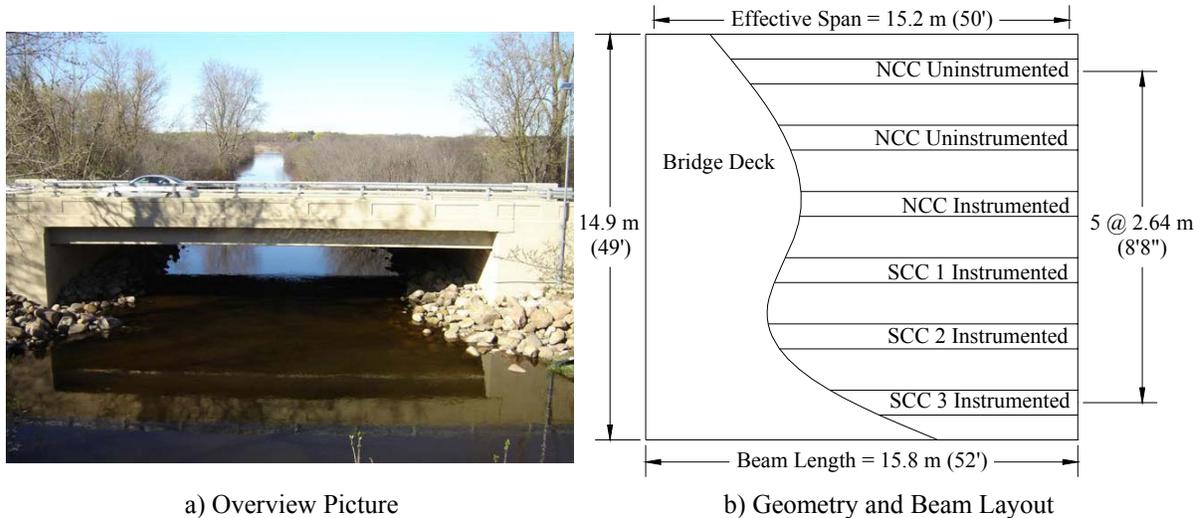
| a) Overview Picture | b) Geometry and Beam Layout |

Fig. 1 Picture and Plan View with Beam Layout of M-50/US-127 Bridge

MANUFACTURING AND MATERIAL PROPERTIES

Several methods to proportion SCC have been developed, and, while there is thus no single accepted procedure, it is commonly agreed that all methods are bounded by two main approaches[1]:

- Mixes with moderate w/c ratios and use of high-range-water-reducers (HRWR) and viscosity-modifying-admixtures (VMA) to provide fluidity and increase stability.
- Mixes without any VMA, but with lower w/c ratios to reduce free water content and provide stability and use of a relatively high content of HRWR to provide high-fluidity.

Three SCC mix designs that bound the accepted methods noted above were designed using their water/cement ratio as the guiding parameter. A baseline normally consolidated concrete (NCC) mix was also considered. The concrete mix designs were developed by MSU and the Premarc Corporation (Grand Rapids, MI) in consultation with Degussa Admixtures Inc. following the project Special Provisions[2] and PCI's guidelines for SCC in precast/prestressed elements[3]. All mixes were designed for a design compressive strength at 28-days of 5500 psi. Trial batches were made and their performance was evaluated through spread-flow, J-ring and L-box tests and manufacturing procedures were evaluated through mock-up beams. The four mix designs for the project were as given in Table 1. Beam production was completed by Premarc during the month of June 2005.

In addition to conventional material testing during production, concrete properties were evaluated at the day of each test for all beams. Compressive strength was evaluated for both flexure and shear tests and split tensile strength was determined for the shear tests only. A summary of the average day-of-test concrete properties is given in Table 2. It can be seen that the compressive strength of all mixes was well above the targeted design value. Finally, the prestressing strand used was seven wire low-relaxation, 0.6-in. nominal-diameter 270 ksi strand from American Spring Wire Co. (Bedford Heights, OH).

Table 1 Concrete Mix Designs

		SCC1	SCC2	SCC3	NCC*
Constituents (lbs)					
Cement	Type-III	700	700	700	564
Sand (oven dry)	2 NS	1,591	1,513	1,320	1,354
Coarse Aggregate (oven dry)	17A	1,350	1,350	1,450	1,883
Water		256	285	320	151
Air	6 % (target)	6.00%	6.00%	6.00%	6.00%
Water/Cement Ratio		0.37	0.41	0.46	0.38
Sand/Total Aggregate Ratio		54.1%	52.9%	47.7%	41.8%
Admixtures (oz/cwt)					
Air Entraining	MBAE90	1	1	1	1.9
HRWR	Glenium® 3400	15	12	10.7	11.3
VMA	Rheomac® VMA	1	2	6	0

* Standard Mix Design from Premarc Corp. – Mix No. 564-3400

Table 2 Concrete Compressive and Tensile Strength at the Day of Tests

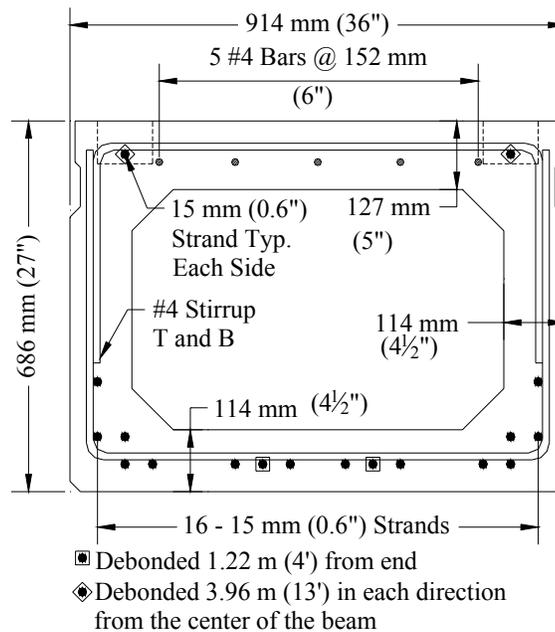| Beam	Flexure Tests			Shear Tests				
	Age (days)	f'_{c_avg} (psi)	Std.Dev. (psi)	Age (days)	f'_{c_avg} (psi)	Std.Dev. (psi)	f'_{t_avg} (psi)	Std.Dev. (psi)
NCC	41	8,196	437	55	8,560	134	646	105
SCC1	43	8,290	300	58	7,519	158	609	57
SCC2	42	6,754	290	66	7,711	430	576	38
SCC3	40	6,685	236	54	6,953	118	514	73

Fig. 2 Cross-Section of Test Beams

In addition to the 6 *field* bridge beams (Fig. 1b) 8 *test* beams and 3 *substitute* beams were produced. For each concrete type one test beam was fabricated and instrumented for flexural and shear evaluation, respectively. Three additional substitute field NCC beams were produced should the performance of the SCC beams was found to be unacceptable. In all, 17 beams were produced. The test beams were replicas of the beams to be used in the M-50 /US-127 Bridge, specifically: 52 ft long 27 in. x 36 in. prestressed box beams. The only difference between the field and test beams was the omission of slab ties on the test beams. Each box beam was reinforced with 18 0.6-in. diameter prestressing strands in four rows. Top flange longitudinal reinforcement consisted of 5 #4 bars. Shear reinforcement consisted of #4 open stirrups spaced at every 6 in. for 6 ft from each beam end and at every 12 in. for the remainder of the beam. A typical cross section of the test beams is shown in Fig. 2.

EXPERIMENTAL PLAN AND PROCEDURES

Before the SCC beams could be accepted for implementation in the demonstration bridge, their performance was to be evaluated through full-scale testing[4] to ensure that their flexural and shear performance was equal or better than that of a conventional beam. Details on the flexure and shear testing are provided in the following sections.

FLEXURE TESTS

The flexural capacity of the box beams was assessed through four-point bending tests. The 52-ft beam was simply supported to create an effective span of 50 ft. Loading was applied by means of two servo-controlled actuators spaced 8 ft apart and centered about the beam. The flexural tests were provided with diverse instrumentation to monitor overall performance and test setup control[4]. The most relevant monitored responses were global beam deformations, strand movement/slip, bottom strand strains at two critical sections, and concrete strains at the top compressive fiber. An overview drawing of the test setup is given in Fig. 3.
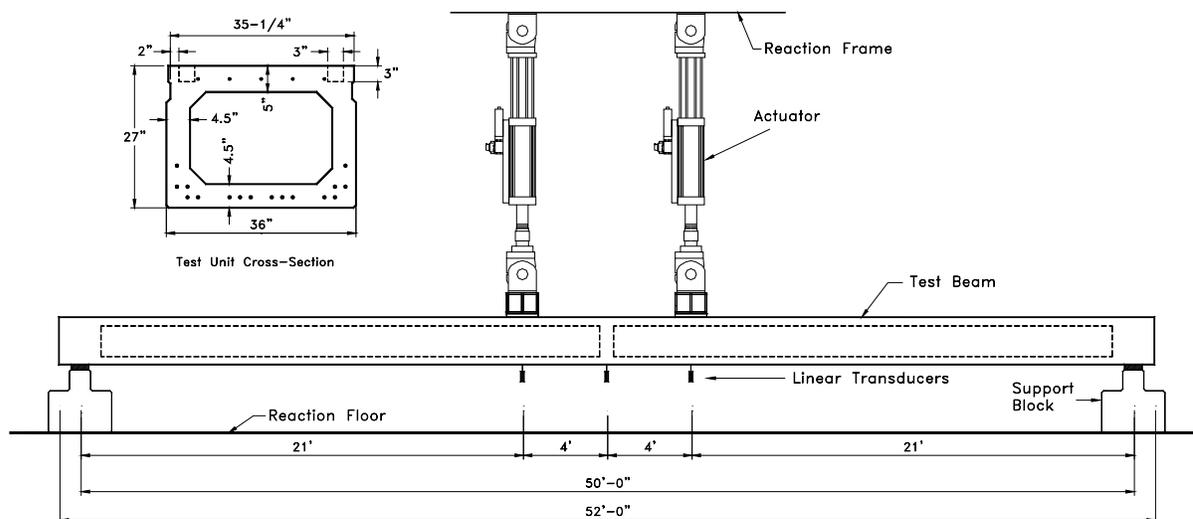

Fig. 3 Flexural Test Setup

The flexural tests were conducted by monotonically loading the beam up to failure. The beams were initially loaded in force control in increments of 10 kips at a rate of 0.01 kip/sec. and loaded under displacement control once the section showed significant nonlinear force-displacement response in typical intervals of 0.25 inches at a rate of 0.0025in./sec.

SHEAR TESTS

The shear capacity of the box beams was also assessed through four-point bending tests, with loading points 8 ft apart; but with the difference that the shear spans were considerably reduced to increase shear demands, see Fig. 4. However, reducing the shear spans led to overhanging cantilevers beyond the simple supports of the setup. To minimize the resulting negative moments, and their effect on the beam behavior, it was not desired to reduce the shear spans too much.

The setup was thus determined by the need to keep the beam overhangs at a minimum and minimizing flexure-shear interaction to ensure a shear-dominated failure. The shear test for the NCC beam had a shear span of 11 ft. The flexural and shear capacities of the beam for this configuration were too close. Thus, the test setup for the subsequent SCC beams was changed so that the shear span was reduced to 9 ft, as shown in Fig. 4.

Instrumentation for the shear tests differed from the flexural tests by the provision of instrumentation to assess shear behavior in a critical shear section; defined 29 inches into the shear span from the points of load application (see Fig. 4). A shear critical region was thus defined over a distance of +/- h/2 from the critical shear section. A shear deformation panel (24 in. square) was placed centered about the shear critical section to measure average shear deformations in this section. Strain gages in the strand, stirrups and top concrete surface were also placed in the shear critical area. The test protocol for the shear beams was the same as for the flexure tests.
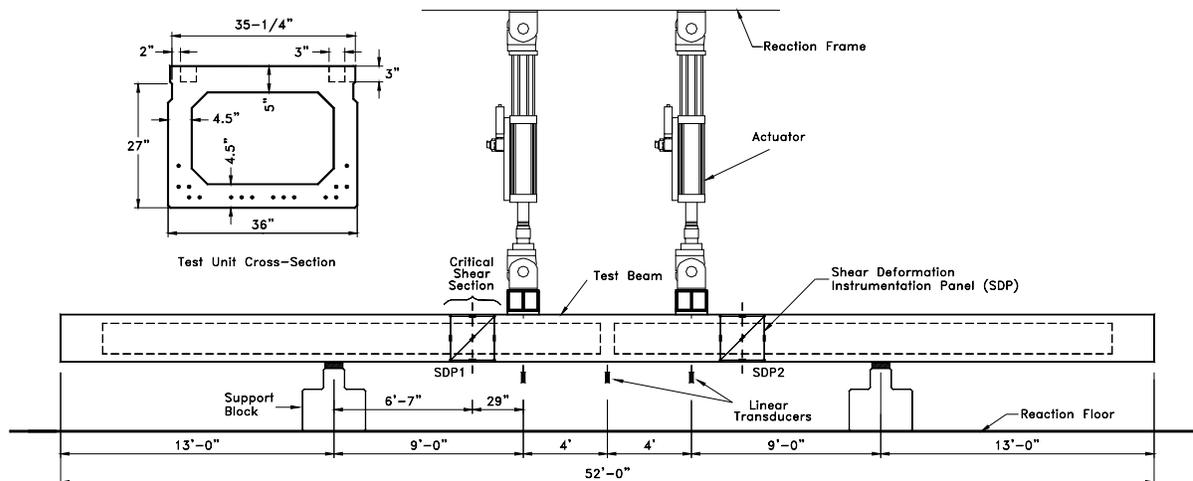
Fig. 4 Shear Test Setup

EXPERIMENTAL OBSERVATIONS AND RESULTS

FLEXURE RESPONSE

All test beams behaved as expected for the flexural response of a prestressed concrete member. The appearance of flexural cracks, their paths, and their widths for all beams was consistent with expected behavior. It should be noted that the beams had two open pockets on the top flange used to flame-cut the top flange strand during production (see Fig. 2). These cavities, centered about the beam, were approximately 3 in. x 3 in. x 4 in., with the long dimension along the beam length. The sectional capacity of the beam was thus lower in this section. With the exception of the SCC2 beam, whose pockets were grouted by the producer, all beams were tested with the top flange pockets ungrouted.

Since the beams were tested without the deck compression flange, the section was essentially over-reinforced. Failure was thus dictated by crushing of the compression flange. The failures were explosive since the concrete compressive strengths were relatively high (Table 2) and also due to the high prestressing level. However, the overall response was ductile and strain gages on the strands indicated that they had yielded before failure. No slip was measured by the transducers mounted on the prestressing strands at the beam end. A view of the typical failure mode for the flexure tests is shown in Fig. 5.

A plot of the applied force per actuator versus the beam maximum (center) displacement for all flexure beams is shown in Fig. 6a. It can be seen that the overall response of all beams was essentially equal, with the NCC beam reaching a slightly higher load at failure and the SCC1 beam showing the largest deformation at failure. Fig. 6b shows the moment versus curvature response of the mid-span section for all beams. Section curvature was calculated from strain gage readings on the top surface of the concrete compression flange and the strain gages in the first row of prestressing strands. It can be noted that in both figures the traces do not start at zero. The reason is that dead load contribution to the moment was added in order to compare the achieved capacities with the design limits.
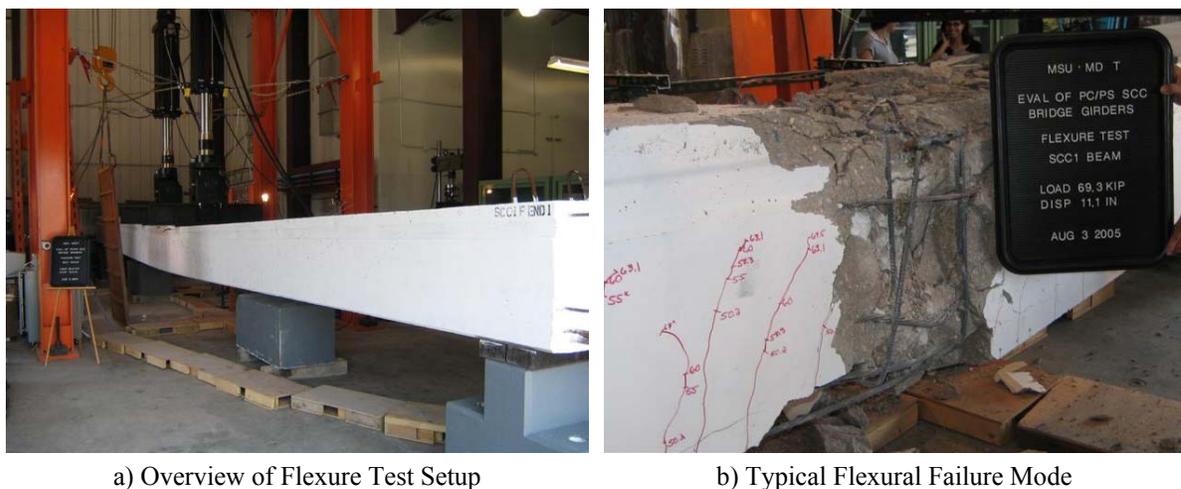
a) Overview of Flexure Test Setup   b) Typical Flexural Failure Mode

Fig. 5 Flexural Test Overview and Typical Failure Mode

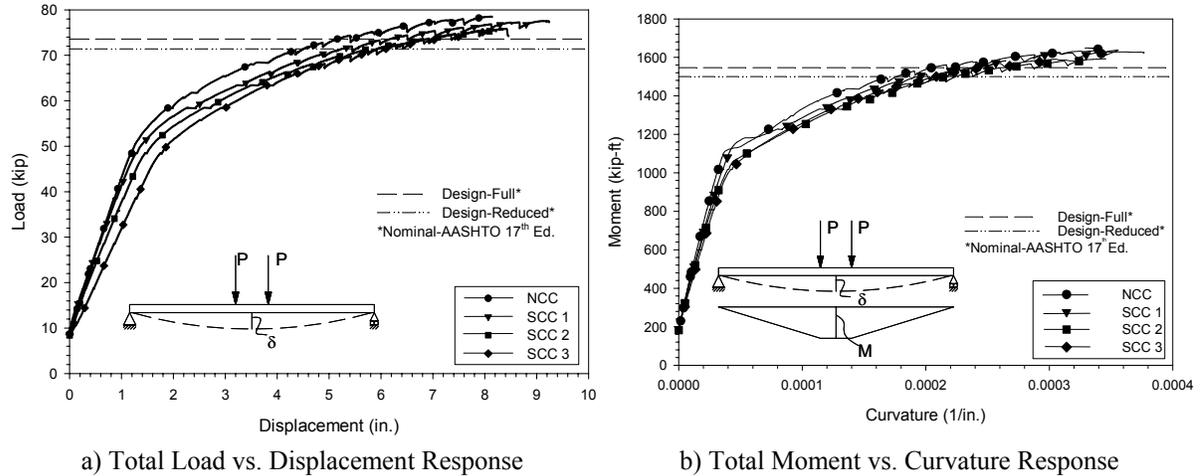a) Total Load vs. Displacement Response   b) Total Moment vs. Curvature Response

Fig. 6 Representative Flexural Test Results

The plots in Fig. 6 also show two response limits corresponding to the design nominal capacity, which is the design capacity for the bridge with the assumed design concrete compressive strength of 5,500 psi. The sectional capacity was determined using the simplified approach presented in the 17th Edition of the AASHTO Standard Specifications[5], since these were the guidelines used by MDOT for the design of the demonstration bridge. The limits are shown for both the full and reduced section. The full cross section refers to the section capacity neglecting the existence of the before-mentioned top flange pocket. The reduced section assumes that the compression flange reduces to an effective width of 26 in., upon subtracting the width of both pockets (3 in. each) and the strip of concrete on the outside (2 in. each), see Fig. 3. It can be seen that the sectional capacities of all beams exceeded the required design capacities.

The results in Fig. 6 also show that the NCC beam reached the highest section capacity and that the SCC3 beam had the lowest (Table 3). Also, the SCC1 beam had the highest deformation at failure and the NCC beam had the lowest. Since the beams had different concrete strengths at the day of test (Table 2), their normalized response with respect to concrete compressive strength was evaluated[4]. This showed, however, that concrete compressive strength was not a significant parameter. Thus, in spite of having different concrete compressive strengths, the overall load and moment capacities of the beams as shown in Fig. 6 was essentially equal.

Table 3. Maximum Achieved Capacities of Flexural Beams

	Max. Total P-Load (kip)	Max Applied Center Disp. (in.)	Max. Total Moment (kip-ft)	Design Moment* Full/Reduced (kip-ft)	Actual to Design Ratio (w/ Reduced)
NCC	78.5	8.1	1,648.8	1,546/1,499	1.10
SCC1	77.5	9.2	1,628.4	1,546/1,499	1.09
SCC2	75.9	8.5	1,592.8	1,546/1,499	1.06
SCC3	75.7	8.2	1,590.0	1,546/1,499	1.06

* AASHTO Standard Specifications – 17th Edition[5]

SHEAR RESPONSE

Interpretation of results from the shear tests required consideration of the partially bonded/debonded strands in the top flange, which modified the shear and flexural capacity of the beam along its length; and of the cantilever overhang continuity over the supports, which allowed the beam redistribute loads beyond the supports. Degradation of the beam was concentrated through shear deformations along the shear critical zone as seen in Fig. 7a. The inclination of shear cracks was very shallow due to the high level of prestress. This same prestressing, which extended beyond the supports, allowed the beam to redistribute member demands towards the end of the beam even after the critical shear section reached its capacity. Thus, while the shear capacity of the critical section was reached (further discussion later), complete shear failure of the beams was not reached, except for one test unit. The SCC3 beam exhibited a flexure-shear failure upon crushing of the compression zone in flexure (see Fig. 7b). This failure was not a typical shear failure but followed from the combined flexure and shear loading and the weakening of the beam due to shear cracking.

a) Distress in Shear Span and Critical Shear Region   b) Flexure-Shear Failure of SCC3 Shear Test Unit

Fig. 7 Typical Shear Distress in and Flexure-Shear Failure of SCC3 Beam

The total (with self-weight) shear force versus center displacement for all of the shear test beams is shown in Fig. 8a. While this plot shows that the response of the NCC beam was considerably different than the SCC beams, the difference is due to the respective shear spans in the test setups. The total moment-curvature response at the critical shear section for all beams is shown in Fig. 8b. In this plot it can first be seen that the response of all beams was very similar. Secondly, it can also be observed that the moment-curvature response at the critical section abruptly "softens" followed by an almost linear response attributed to the redistribution of loading on to the cantilever overhang.

The force-displacement and moment-curvature plots feature lines indicating the nominal capacities calculated for the design beam, i.e., with a design compressive strength of 5,500 psi. The nominal capacities were calculated according to the 17th Edition of the AASHTO Standard Specifications[5] as well as the simplified sectional analysis procedures recommended in the 2nd Edition of the AASHTO LRFD Design Specifications[6]. The

apparent reduced shear capacity of the tested beams compared to the design value from the Standard Specifications is due to flexure-shear interaction effects, which will result in a compromise between the capacities calculated for pure flexure and pure shear failure[7,8]. This effect is not explicitly taken into account in the Standard Specifications; however, the LRFD Specifications addresses this issue for the determination of shear capacities, which is based on the principles of the modified compression field theory[7].

The presence of shear in a section also decreases the flexural capacity of a section due to the additional horizontal tensile forces created by the diagonal compressive struts in the section web. This reduced moment capacity can be determined by considering the additional tensile strains generated in the section due to shear[7,8]. These calculations were performed with the program Response 2000[9] for the sectional shear capacity equations from the AASHTO LRFD specifications[6]. A shear-moment interaction diagram computed by Response 2000 for the design beam (f'_c = 5,500 psi) is shown in Fig. 8c. It can be seen that the reduced flexural capacity corresponds very closely with the onset of softening in the moment-curvature traces for the SCC1 beams in Fig. 8b, which defines the capacity curve level shown for the AASHTO-LRFD specifications in this plot.

a) Total Shear Force vs. Displacement Response

b) Total Moment vs. Curvature Response

c) Design Section Moment-Shear Interaction Diagram

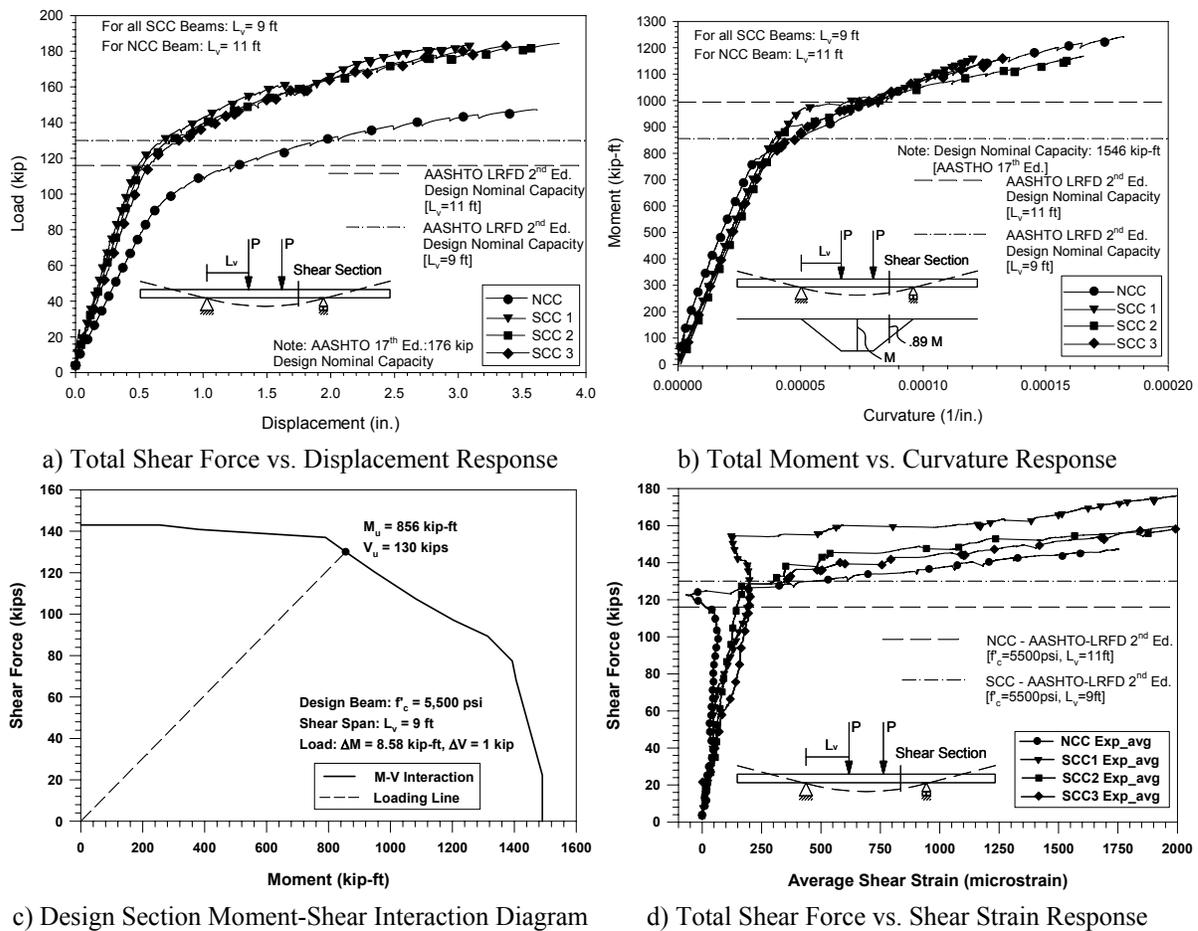d) Total Shear Force vs. Shear Strain Response

Fig. 8 Representative Shear Test Results

A plot of the shear force vs. shear strain response in the defined shear critical section for all beams is given in Fig. 8c. In this figure, the measured strains in the deformation panels for each beam were averaged. Threshold limits indicate the nominal shear design capacity according to the AASHTO LRFD specifications. Two threshold lines are shown, one corresponding to the shear span setup of the NCC beam (11 ft) and the other to the shear span of the SCC beams (9 ft). From Fig. 8c it can be seen that there is a clear point at which the shear strains in the critical region increase at a very high rate and how this level corresponds very closely to the predicted shear capacity of the section. It can thus be concluded that the sections reached their shear capacity. From the responses it can also be observed that the post-"elastic" behavior is essentially flat, thus indicating that the section has essentially no more shear resistance.

It can be concluded that the shear response of all beams was very similar and in all cases the critical sections reached a shear capacity greater than that of the design beam. A summary of the maximum achieved capacities, defined as the point beyond which shear strains increase at a very rapid rate are given in Table 4. The SCC1 beam reached the maximum shear capacity and highest ratio (1.22) compared to the nominal strength. The lowest ratio of actual capacity to nominal strength was for the SCC3 beam. While the NCC beam apparently had the lowest shear capacity, this was due to flexural effects from the longer shear span.

Table 4 Maximum Achieved Capacities of Shear Beams at Critical Shear Section

	Maximum Total Shear (kip)	Maximum Total Moment (kip-ft)	Nom. Design Shear [a] (kip)	Nom. Design Moment [b] (kip-ft)	Actual to Design Ratio (Shear)
NCC	128.4	1102.3	116	994	1.11
SCC1	159.0	1046.6	130	856	1.22
SCC2	145.5	957.8	130	856	1.12
SCC3	140.3	923.3	130	856	1.08

[a] According to AASHTO LRFD[6] Simplified Section Analysis Method
[b] Based on moment-shear interaction envelope calculated using Response 2000[9] for AASHTO-LRFD criteria.

CONTINUOUS FIELD MONITORING

Upon implementation of the SCC beams in the demonstration bridge, the project includes installation, management, and data interpretation of an in-service performance monitoring plan. An instrumentation program based on the recommendations by the FHWA for demonstration projects using high-performance-concrete[10] was implemented. Four beams, one for each type of concrete (3 SCC and 1 NCC), were instrumented as shown in Fig. 1b. Type T thermocouples are being used to measure temperature through the cross section while strains are being measured with vibrating wire gages. The instruments were arranged at three sections as shown in Fig. 9. These consisted of a main section at mid-span (Section A), a redundant section with key instruments 1 ft from the centerline (Section B), and a quarter-point section (C) also with key instruments to evaluate response along the beam length.

a) Instrumented Sections
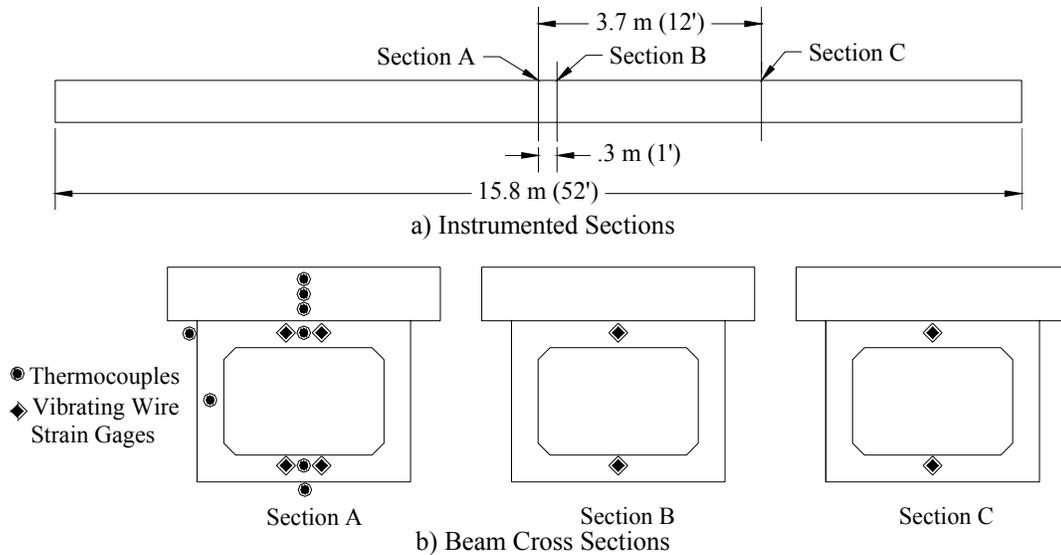

b) Beam Cross Sections

Fig. 9 Instrument Layout for Continuous Monitoring Plan

Readings from the vibrating wire strain gages were taken at five time events during the beam production and the completion of the bridge: (1) after concrete placement (baseline measurement), (2) after prestress release, (3) after cut of top strand, (4) after placement in the bridge, and (5) after deck casting. The measured strains and variations at the beam bottom flange of Section A for all instrumented beams are given in Table 5.

Table 5 Key Event Strain Measurements in Beam Bottom Flange at Section A

Event	Date	NCC ε_{AB} ($\mu\varepsilon$)	SCC 1 ε_{AB} ($\mu\varepsilon$)	SCC 2 ε_{AB} ($\mu\varepsilon$)	SCC 3 ε_{AB} ($\mu\varepsilon$)
After prestress release	6-22-05 – 6-30-05	-221.75	-273.93	-268.15	-209.61
After top strand cutting	8-2-05	-430.11	-458.84	-442.77	-525.63
In place at bridge site	9-21-05	-457.51	-510.72	-486.19	-598.97
After deck casting	10-2-05	-352.25	-398.79	-376.71	-481.08

For continuous in-service monitoring, the instruments were connected to a datalogger with built-in signal conditioning and setup for on-site data retrieval through a PC. The data logger is powered by a 12-volt rechargeable battery and is recharged by a 10 W solar panel. Data is collected automatically every 2 hours by a data acquisition program using the datalogger's software. With 8 thermocouples and 8 strain gages per beam, reading of all instruments takes approximately 1.5 minutes. Data is stored in the memory module of the datalogger and downloaded to a portable PC once a month.

The monitoring system has been active since mid December 2005. Fig. 10 shows the temperature and strain history at the beam's Section A bottom flange taken at 12:00 pm. Fig. 10b shows a general increase in compressive strains after an initial stable period, which corresponds to the gradual temperature increase over the summer months (Fig. 10a). While the strain histories for the different beams have different starting points, their change with

time has been similar. Investigation on the reasons behind the initial differences and further analyses on the collected data is in progress.
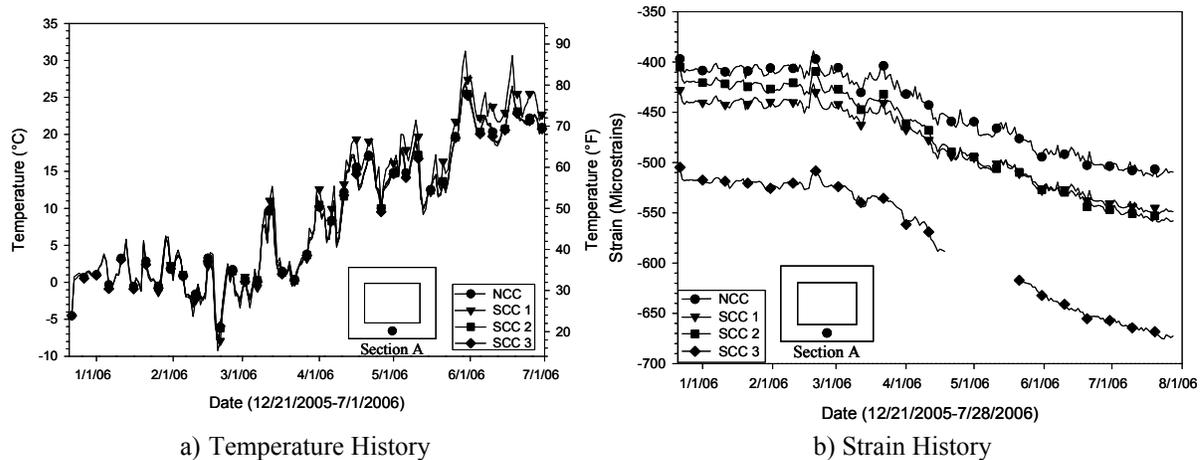

a) Temperature History   b) Strain History

Fig. 10. Representative Plots from Continuous Monitoring at Beam Bottom Flange Section A

CONCLUSIONS

The M-50/US-127 demonstration bridge over the Grand River in Jackson, Michigan, is permitting the short- and long-term evaluation of SCC for prestressed concrete bridges. Three different SCC mixes, developed to bound current approaches to SCC mix design, are being evaluated in the project. Structural testing of the full-scale prestressed bridge beams evaluated the flexure and shear response of the different SCC beams against conventional NCC beams, and continuous monitoring is in place to evaluate long-term performance.

Flexural tests showed that the overall behavior of the SCC beams was very similar to that of the conventional beam (NCC) and that the absolute capacities of the SCC beams were only marginally lower. The flexural capacities of all three SCC beams thus exceeded the required design capacity by 6 to 9 percent. The shear behavior and capacities of the SCC beams were also found to be adequate and very similar to that of the NCC beam. Cracking paths and widths were consistent in all beams and the failure levels closely matched analytical predictions. The shear capacity of SCC beams was 8 to 22 percent higher than the calculated nominal design shear value.

Even though of long-term behavior of the SCC prestressed box beams are still to be fully evaluated, their short-term flexural and shear response determined through the testing program indicates that the SCC beams safely satisfy their prescribed design requirements. Thus, while resolving all of the concerns on SCC is still work in progress, results from this project are expected contribute its safe and efficient use in highway bridges.

AKNOWLEDGEMENTS

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