Urban Maglev Fiber Reinforced Concrete Girder Design, Development and Testing

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ABSTRACT: This paper discusses the work performed by General Atomics and Mackin Engineering in the development of Steel Fiber Reinforced Concrete (SFRC). This paper discusses the status of subscale testing of long beams manufactured with SFRC. Two beams were manufactured without conventional reinforced steel, one with prestress steel strands and the other two without prestress strands. These beams were tested to failure to evaluate the effect of pre-stressing the SFRC girders, and to evaluate the long span beam properties and distribution of fibers. This effort is a continuation of the Guideway Steel Fiber Reinforced Concrete Hybrid Girder Design as discussed at the Maglev 2006 conference in Dresden Germany. This paper will summarize the results of these tests and the resulting conclusions.

1 INTRODUCTION

General Atomics (GA), in conjunction with the United States Air Force, originally developed Steel Fiber Reinforced Concrete (SFRC) over a decade ago; in early 2004, GA and San Diego State University further optimized the mix design with excellent results and presented them in papers at both the Maglev 2004 and 2006 proceedings. Since then, substantial development has occurred and testing has been performed.

Concrete is a quasi-brittle material with a low capacity for deformation under tensile stress. In conventional reinforced concrete, steel reinforcing bars are added to prevent cracking caused by tensile stresses induced by the imposed dead and live loads, creep deformation and drying shrinkage. In SFRC, the solid rebar is replaced with uniformly distributed and randomly oriented steel fibers along with the required amount of prestressing steel to provide the necessary pre-compression.

In order to fully evaluate the effect prestressing has on SFRC girders, and assess the impact span length and fiber distribution have on SFRC material properties, three quarter-scale beams were fabricated and tested. Each of the quarter-scale beams was 12” wide x 24” deep x 29’- 0” long. The first beam was fabricated in December 2006. It was cast with steel fibers at a 3.0% dosage rate, but no prestressing strands or other reinforcing steel. The second and third beams were fabricated in November 2007. The second beam was cast with steel fibers at the same 3.0% dosage rate in the upper third (compression zone) of its cross section, a higher 3.4% dosage rate in the lower third (tension zone) and the 3.2% average dosage rate in the middle third (neutral axis zone). The second beam contained no prestressing strands or other reinforcing steel. The third beam was cast with steel fibers at the same 3.2% average dosage rate as the second beam, and a longitudinal duct 6” from the bottom of the beam that was required for the post-tensioning strands. The beam was post-tensioned after it was wet cured for 28 days.

Each of the three quarter-scale beams were instrumented with strain gauges. The non post-tensioned beams were incrementally loaded until the beams cracked and eventually failed. The post-tensioned beam was incrementally loaded until the beam cracked, but did not fail.

This paper will summarize the results of these tests and the resulting conclusions.

2 STATIC LOAD TESTING

2.1 Purpose of static load testing.

The purpose of the static load testing of the quarter-scale beams was to evaluate SFRC beams in the following areas:

The need for prestressing steel in long span SFRC beams to carry the tensile stresses induced by the imposed dead and live loads.

The need for reinforcing steel in long span SFRC beams to reduce the effects of creep, shrinkage and thermal expansion, and to carry the shear stresses induced by the imposed dead and live loads.

2.2 Description of quarter-scale test beams.

In October 2006, GA designed a quarter-scale High-Strength Steel Fiber Reinforced Concrete (HS-SFRC) test beam.

The beam was 12” wide x 24” deep x 29’- 0” long. The design capacity was estimated based on a 25’- 0” clear span with a 12” x 24” weldment used as a bearing at each end. (See Figure 1) The material properties that were used to determine the capacity of the beam are listed in Table 1.

Figure 1. Quarter-scale post-tensioned static load test beam.

In October 2007, General Atomics designed two more quarter-scale HS-SFRC test beams; one beam included a post-tensioning system with seven strands each comprised of seven wires designed by DYWIDAG-Systems International USA, Inc. of Bolingbrook, Illinois. (See Figure 2)

Both beams were 12” wide x 24” deep x 29’- 0” long. The design capacities were estimated based on a 26’- 0” simple span with a 24” long W8x24 steel beam used as a bearing at each end. (See Figures 2 and 3) The material properties that were used to determine the capacities of the beams are listed in Table 2.

Figure 2. Quarter-scale post-tensioned static load test beam.

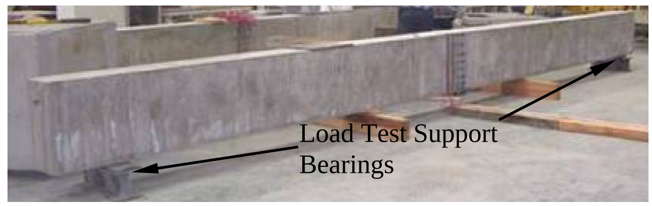

Figure 3. Quarter-scale <u>non</u> post-tensioned static load test beam.

Table 1. Design material properties: 1st test beam.

Material Properties	Value
28–Day Compressive Strength	10,500 psi
28–Day Elastic Modulus	7,200,000 psi
28–Day Flexure (Cracking) Strength	1,600 psi
28–Day Ultimate (Failure) Strength	2,500 psi
Steel Fiber Yield Strength	150,000 psi
Steel Fiber Elastic Modulus	29,000,000 psi
Steel Fiber Dosage Rate	3.0 %
Steel Fiber Projected Area Coefficient	40.5 %

Table 2. Design material properties: 2nd & 3rd test beams.

Material Properties	Value
28–Day Compressive Strength	10,500 psi
28–Day Elastic Modulus	7,200,000 psi
28–Day Flexure (Cracking) Strength	1,600 psi
28–Day Ultimate (Failure) Strength	2,500 psi
Steel Fiber Yield Strength	150,000 psi
Steel Fiber Elastic Modulus	29,000,000 psi
Average Steel Fiber Dosage Rate	3.2 %
Steel Fiber Dosage Rate – Upper 1/3rd Compression Zone	3.0 %
Steel Fiber Dosage Rate – Middle	3.2 %

1/3rd Neutral Axis Zone	
Steel Fiber Dosage Rate – Lower 1/3rd Tension Zone	3.4 %
Steel Fiber Projected Area Coefficient	40.5 %

2.3 Test specimen material properties.

Three, 4” diameter x 8” high test cylinders were cast to determine the 28-day minimum compressive strength for each of the test beams (total of nine test cylinder specimens). In addition, three, 4” wide x 4” deep x 14” long sub-scale test beams were cast to determine the 28-day flexure (cracking) and ultimate (failure) strengths for each of the test beams (total of nine test beam specimens). The test results for the cylinder and sub-scale beam specimens for both the non post-tensioned and post-tensioned beams are summarized in Tables 3 and 4.

Table 3. Test specimen material properties: 1st test beam.

Material Properties	Non Post-Tensioned Beam Properties	Post-Tensioned Beam Properties	Design Properties
28–Day Compressive Strength	10,800 psi	-----	10,500 psi
28–Day Elastic Modulus	7,100,000 psi	-----	7,200,000 psi
28–Day Flexure (Cracking) Strength	2,300 psi	-----	1,600 psi
28–Day Ultimate (Failure) Strength	2,600 psi	-----	2,500 psi

Table 4. Test specimen material properties: 2nd and 3rd test beams.

Material Properties	Non Post-Tensioned Beam Properties	Post-Tensioned Beam Properties	Design Properties
28–Day Compressive Strength	12,300 psi	11,400 psi	10,500 psi
28–Day Elastic Modulus	7,600,000 psi	7,300,000 psi	7,200,000 psi
28–Day Flexure (Cracking) Strength	2,400 psi	1,800 psi	1,600 psi
28–Day Ultimate (Failure) Strength	2,700 psi	2,500 psi	2,500 psi

These test results verified the properties of the test specimens for the three static load test beams equaled or exceeded the properties used to design the quarter-scale test beams.

2.4 Quarter-scale test beam static load testing.

In February 2007, San Diego Precast Concrete performed a static load test to confirm the design capacity of the first simply supported HS-SFRC quarter-scale test beam; the beam was not post-tensioned. Eight steel weights were provided in three different sizes to produce a potential maximum applied load of 57,500 lb. (See Table 5)

Table 5. Steel weights: 1st test beam.

Unit Weight	Quantity	Total Weight
12,500 #	3	37,500 #
5,000 #	3	15,000 #
2,500 #	2	5,000 #
Totals	11	57,500 #

The beam had an overall length of 29’- 0” and a clear span of 25’- 0”. The beam was supported at each end with a 12” x 24” weldment used as a bearing.

The load for the beam was applied in 12,500 lb increments by slowly stacking the steel weights at mid span until the beam cracked and eventually failed.

The test beam supported a single 12,500 lb weight without cracks; however, it cracked and failed (collapsed) when the next 12,500 lb weight was added. The actual cracking load was not estimated because the load increment was too large to obtain an accurate result.

In January 2008, US Concrete Precast Group (formerly San Diego Precast Concrete) performed two additional static load tests to confirm the design capacities of the simply supported HS-SFRC quarter-scale test beams; the third beam was post-tensioned, but the second beam was not. Eleven steel weights were provided in four different sizes to produce a potential maximum applied load of 93,750 lb. (See Table 6)

Table 6. Steel weights: 2nd and 3rd test beams.

Unit Weight	Quantity	Total Weight
18,750 #	3	56,250 #
12,500 #	1	12,500 #
5,000 #	3	15,000 #
2,500 #	4	10,000 #
Totals	11	93,750 #

The following photograph (See Figure 4) shows the assortment of steel weights (totaling 81,250 lb) that produced the maximum load placed on the post-tensioned test beam.

Figure 4. Post-tensioned beam supporting 81,250 lb load.

Each beam had an overall length of 29’- 0”; the post-tensioned beam had simple span of 25’- 0” whereas the non post-tensioned beam had a simple span of 26’- 0”. The beams were supported at each end with a 24” long W8x24 steel beam used as a bearing.

The load for the post-tensioned beam was typically applied in 2,500 lb increments by slowly stacking the steel weights at mid span until the beam cracked (but did not fail).

The post-tensioned test beam supported a combination of weights totaling 50,000 lb without cracks; however, it microscopically cracked when the next two 2,500 lb weights were added. In addition, the test beam supported 81,250 lb with no visible cracks; but, when this load was sustained overnight, visible cracks appeared. Post testing analysis confirmed the beam had microscopically cracked between 50,000 lb and 55,000 lb at $\pm$ 52,400 lb.

The load for the non post-tensioned beam was applied in 2,500 lb increments by slowly stacking the steel weights at mid span until the beam cracked and eventually failed.

The non post-tensioned test beam supported a combination of weights totaling 7,500 lb without cracks; however, it cracked and failed (collapsed) when the next 2,500 lb weight was added. Post testing analysis confirmed the beam had cracked between 7,500 lb and 10,000 lb at $\pm$ 9,200 lb.

The ranges for the cracking loads are found in Table 7 and the failure loads are summarized in Table 8.

Table 7. Results of first crack load testing.

Test Number	Span Length	Cracking Load	
		Minimum	Maximum
1	25’- 0”	12,500 #	25,00 #
2	26’- 0”	7,500 #	10,00 #
3	25’- 0”	50,000 #	55,00 #

Table 8. Results of failure load testing.

Test Number	Span Length	Failure Load	
		Minimum	Maximum
1	25’- 0”	12,500 #	25,000 #
2	26’- 0”	7,500 #	10,000 #
3	25’- 0”	No Failure	No Failure

Strains were measured at six locations on each of the three test beams with strain gages wired to an IOtech StrainBook/616 electronic strain gage measurement system. (See Figure 5)

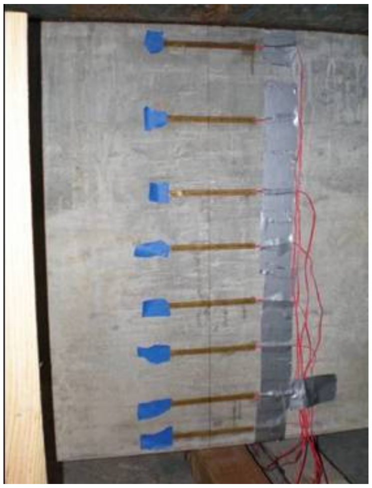

Figure 5. Test beam strain gages.

The maximum recorded strain in the post-tensioned beam increased as the first twenty-two loads were incrementally applied. The maximum strain peaked at 52,500 lb and declined slightly at 55,00 lb; it then remained nearly constant as the next eight loads were incrementally applied up to 81,250 lb. Even though no visible cracks were observed after each applied load, the constant maximum strain indicates the post-tensioned beam microscopically cracked when the 52,500 lb load was applied.

Visible cracks were later observed in the post-tensioned beam after the 81,250 lb load was sustained overnight. This suggests once the microscopic cracks were initiated by the 52,500 lb load, they gradually propagated upward, as each subsequent load was applied, past the center of gravity of the post-tensioning strands with some cracks reaching the neutral axis of the beam. The steel fibers bridged the initial cracks in the beam, but when the cracks reached the strands, the strands as well as the fibers bridged the cracks. The cracks remained "closed" because the maximum load was not large enough to develop the strain in the strands that produces the minimum elongation needed to allow the cracks to open. After twelve hours of sustained load, the time dependent creep strain plus the static strain from the applied load produced the required elongation and the visible cracks appeared.

The quarter-scale post-tensioned test beam did not experience ductile failure because the tensile strain in the post-tensioning strands was still below the yield strain and the compressive strain in the concrete was less than the crushing strain. The beam did not experience brittle failure because the pre-compression provided by the post-tensioning strands prevented it by forcing the strands to yield in tension before the concrete crushes in compression.

The quarter-scale non post-tensioned test beams collapsed as a result of brittle failure. This occurred when the fibers pulled out after the concrete fractured but before the fibers yielded. A ductile failure would have occurred if the fibers had yielded before they pulled out. (See Figure 6)

Figure 6. Test beam brittle fracture.

3 CONCLUSIONS

The primary objective of the static load tests of the quarter-scale beams was to determine how significantly the length of a test beam affects its cracking strength.

For simply supported short beams, when a crack first appears at the bottom of the beam, the fibers form a bridge that holds the beam together by transferring the strain across the crack. But as the crack propagates upward toward the neutral axis, the fibers are subject to pullout from the concrete. The pullout process is very energy intensive; as a result, SFRC exhibits a stable load-deflection behavior in the region beyond first crack which places this material in the category of pseudo-plastic or tough materials.

However, for simply supported long beams, the maximum beam strain and stress significantly increases. In order for the concrete to transfer the larger strain to the steel fibers and across the crack, their pullout strength must be increased by lengthening the fibers. In other words the critical fiber length increases as the span length increases.

Unfortunately, from a manufacturing standpoint, continuing to increase the fiber length becomes counterproductive. This is due to the tendency of long fibers to bunch during mixing, resulting in a non-uniform distribution of fibers and a decreased pullout strength.

The rapid decrease in the cracking strength of long beams to a value less than 2% above the modulus of rupture is evidence the steel fibers provide little additional tensile strength above the modulus of rupture. Therefore, as with traditional reinforced concrete beams, steel prestressing strands must be added to SFRC beams to prevent brittle cracking of the concrete due to the tensile stresses induced by the imposed dead and live loads.

On the other hand, there is very good evidence presented in earlier research performed by many others that indicates steel fibers can eliminate the need for steel reinforcing bars to resist creep, shrinkage and thermal expansion. There is also encouraging evidence presented in current research performed by the Iowa DOT, Iowa State University and Lafarge North America for the FHWA to suggest steel fibers can eliminate the need for steel reinforcing bars to carry the shear stresses induced by the imposed dead and live loads.

Even though steel fiber reinforced concrete girders will require prestressing strands, their fabrication cost can be significantly reduced and their simplicity can be greatly improved when compared to the construction of conventional reinforced concrete girders with a complex reinforcement cage requiring many hours of manual labor to fabricate.

4 REFERENCES

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