

Welded-wire reinforcement versus random steel fibers in precast, prestressed concrete bridge girders

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Reactive-powder concrete (RPC) is a new class of concrete developed in France in the mid-1990s. RPC is a high-strength ductile material made of a special combination of fine sand, cement, quartz flour, silica fume, steel fibers, water, and high-range water-reducing admixture (HRWRA). The technology of RPC is widely known as ultra-high-performance concrete (UHPC).

The Association Francaise de Génie Civil (AFGC), in its *Interim Recommendations for Ultra High Performance Fibre-Reinforced Concrete*,¹ and the Japan Society of Civil Engineers (JSCE), in its draft *Recommendations for Design and Construction of Ultra High Strength Fiber Reinforced Concrete Structures*,² define UHPC as a cementitious composite that has a compressive strength in excess of 21.7 ksi (150 MPa) and contains steel fibers for ductile behavior. The enhanced strength and durability of UHPC are mainly due to optimized particle grading, steel fibers, and extremely low water-to-powder ratio, which produces a tightly packed mixture.

Random steel fibers represent about 2% by volume or 6% by weight of UHPC. They are also considered the largest component of the mixture because they are 0.5 in. (13 mm) in length and 0.008 in. (0.2 mm) in diameter. Steel fibers used in UHPC mixtures have a modulus of elasticity of 29,790 ksi (205,400 MPa) and a tensile strength exceeding 290 ksi (2000 MPa).³

Several researchers have studied the effect of random

Editor's quick points

- Fiber-reinforced ultra-high-performance concrete (UHPC) bridge I-girders were compared with high-performance concrete (HPC) I-girders reinforced with Grade 80 WWR. The comparison focused on shear capacity and material cost.
- The shear capacity of the HPC girders reinforced with WWR was comparable to that of the fiber-reinforced UHPC girders, but the overall behavior of the HPC was more accurately predicted by the AASHTO LRFD equations.
- The material costs for the HPC girders reinforced with WWR were 64% less for the concrete and 60% less for the steel. Because standard procedures for mixing, placing, and curing can be used for HPC, it is anticipated that production costs would also be significantly lower than for UHPC.

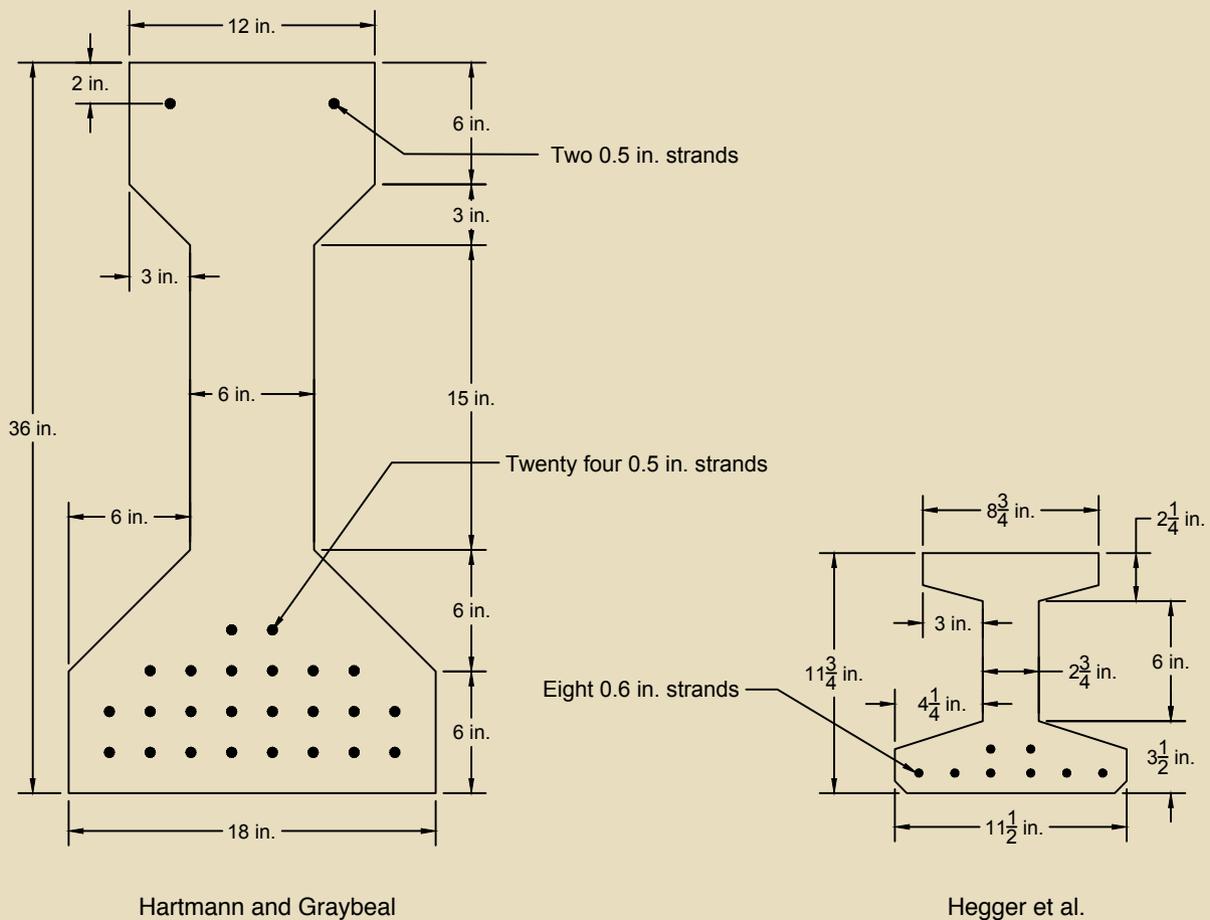


Figure 1. Ultra-high-performance concrete girders tested by Hartmann and Graybeal and Hegger et al. Source: Data from Hartmann and Graybeal 2002 and Hegger, Tuchlinski, and Kommer 2004. Note: 1 in. = 25.4 mm; Grade 270 = 1860 MPa.

steel fibers on the mechanical properties and durability of UHPCs.⁴⁻⁶ They reported that steel fibers have a significant effect on controlling crack width, improving bond strength of prestressing strands, and increasing the compressive strength because of their confinement effects. They also reported that fiber orientation has a substantial impact on flexural strength, which decreases when the fibers are aligned perpendicular to the flexural tensile stresses. Therefore, special attention should be given during placement of fiber-reinforced UHPC to ensure proper consolidation and alignment of fibers.

The application of UHPC in the design and construction of bridge I-girders has been initiated by the Federal Highway Administration (FHWA) and a few state departments of transportation since the early 2000s. Hartmann and Graybeal tested the flexure and shear capacity of American Association of State Highway and Transportation Officials (AASHTO) Type II girders made of UHPC with 2% (by volume) random steel fibers and twenty-four 0.5-in.-diameter (13 mm), Grade 270 (1860 MPa), low-relaxation, 7-wire prestressing strands.⁷ **Figure 1** shows the cross section of the tested girders where no mild steel reinforcement was used.

Three shear tests were conducted using a single point load and a shear span ranging from 2 to 2.5 times the girder depth. The average shear capacity of the three tests performed on the AASHTO Type II UHPC girders was 441 kip (1960 kN) with a standard deviation of 59 kip (270 kN). The FHWA report FHWA-HRT-06-115³ has more information on these tests.

Hegger et al. completed several tests investigating the shear capacity of I-shaped, prestressed concrete girders made of the same UHPC used by the FHWA with 2.5% (by volume) random steel fibers.⁸ These girders were reinforced with eight 0.6-in.-diameter (15 mm), Grade 270 (1860 MPa), low-relaxation, 7-wire prestressing strands (Fig. 1). The average ultimate shear capacity of these girders was 61.4 kip (273 kN), and the average tensile stress across the shear failure plane was 2.28 ksi (15.7 MPa). Given the small depth and thin web of these girders, the average shear strength was found to be similar to that of the AASHTO Type II UHPC girders tested by the FHWA, which was 2.31 ksi (15.9 MPa).

The shear capacity of the fiber-reinforced UHPC girders outperforms that of conventionally reinforced concrete

girders. This conclusion was based on the comparison with the AASHTO Type II girders made of conventionally reinforced high-strength concrete and tested by Tawfiq.⁹ This comparison indicates that fiber-reinforced UHPC girders have 63% more shear capacity than similar high-strength concrete girders with standard shear reinforcement. This significant increase was attributed to the random steel fibers that provide tensile load-carrying capacity across the cracked plane.

Despite the strength and durability advantages of fiber-reinforced UHPC, the high material cost and lack of design provisions represent the main impediments to its quick and widespread use in bridge construction. In addition, the lengthy mixing procedures, special placement requirements to ensure proper consolidation and orientation of fibers, and longer curing and setting times significantly increase the production cost. Also, failure of fiber-reinforced UHPC girders generally occurs when the random steel fibers are debonded from the concrete, which is a sudden failure that is highly dependent on the orientation of the fibers relative to the failure plane.

Alternatively, welded-wire reinforcement (WWR) has been used increasingly in the precast, prestressed concrete industry because of its ease of installation, high strength, and economy due to significant labor savings. WWR is manufactured from cold-worked steel wires welded in an orthogonal configuration. According to ASTM A497,¹⁰ WWR must have a minimum tensile strength of 80 ksi (550 MPa), yield strength of 70 ksi (480 MPa), and weld shear strength of 35 ksi (240 MPa).

Several researchers have studied the efficiency of using WWR as shear reinforcement.¹¹⁻¹⁴ They reported that the close spacing of longitudinal and horizontal wires is effective in controlling the distribution of diagonal cracks, and

the presence of cross wires provides sufficient anchorage for shear reinforcement. In addition, the high strength and consistent spacing of WWR significantly reduce reinforcement congestion in high-shear zones.

The objective of this research is to investigate, through full-scale testing and cost analysis, the shear capacity and economy of using high-performance concrete (HPC) reinforced with WWR compared with fiber-reinforced UHPC in precast, prestressed concrete bridge I-girders. This comparison focuses on shear capacity and material cost and does not include flexural capacity, production cost, or durability.

HPC mixture development and properties

Table 1 shows the mixture proportions and cost of three HPC mixtures developed using local materials for this investigation. Material cost was estimated based on typical prices in Nebraska in 2008, which were \$90/ton (\$99.3/tonne) for portland cement, \$600/ton (\$662/tonne) for silica fume, \$15/ton (\$16.5/tonne) for Class C fly ash, \$10/ton (\$11/tonne) for fine sand, \$15/ton (\$16.5/tonne) for limestone, and \$15/gal (\$4/L) for HRWRA.

Type III cement was used to improve early strength, an important property for the early release of prestressing force. Silica fume and class C fly ash were used to fill the voids in the mixtures, improve the durability and flowing ability, and reduce the carbon footprint of the product. All of the mixtures have from 20% to 30% supplementary cementitious materials. The $\frac{3}{8}$ in. (9.5 mm) limestone was selected because of its availability, economy, and strength while maintaining the mixture flowing ability and resistance to segregation. **Figure 2** shows the grading of the fine and coarse aggregates used in these mixtures based on sieve

Table 1. Mixture proportions and material cost of HPC mixtures

Material	HPC 1	HPC 2	HPC 3
Fine sand, lb/yd ³	1580	2255	1580
$\frac{3}{8}$ in. limestone, lb/yd ³	672	0	672
Cement Type III, lb/yd ³	1050	1120	1050
Class C fly ash, lb/yd ³	30	240	300
Silica fume, lb/yd ³	150	240	150
HRWRA, lb/yd ³	62	71	54
Total water, lb/yd ³	297	301	295
<i>w/cm</i> ratio	0.20	0.19	0.20
Total cost, \$/yd ³	177	215	168

Note: HPC = high-performance concrete; HRWRA = high-range water-reducing admixture; *w/cm* = water-cementitious materials ratio. 1 in. = 25.4 mm; 1 lb/yd³ = 0.593 kg/m³; \$1/yd³ = \$1.308/m³.

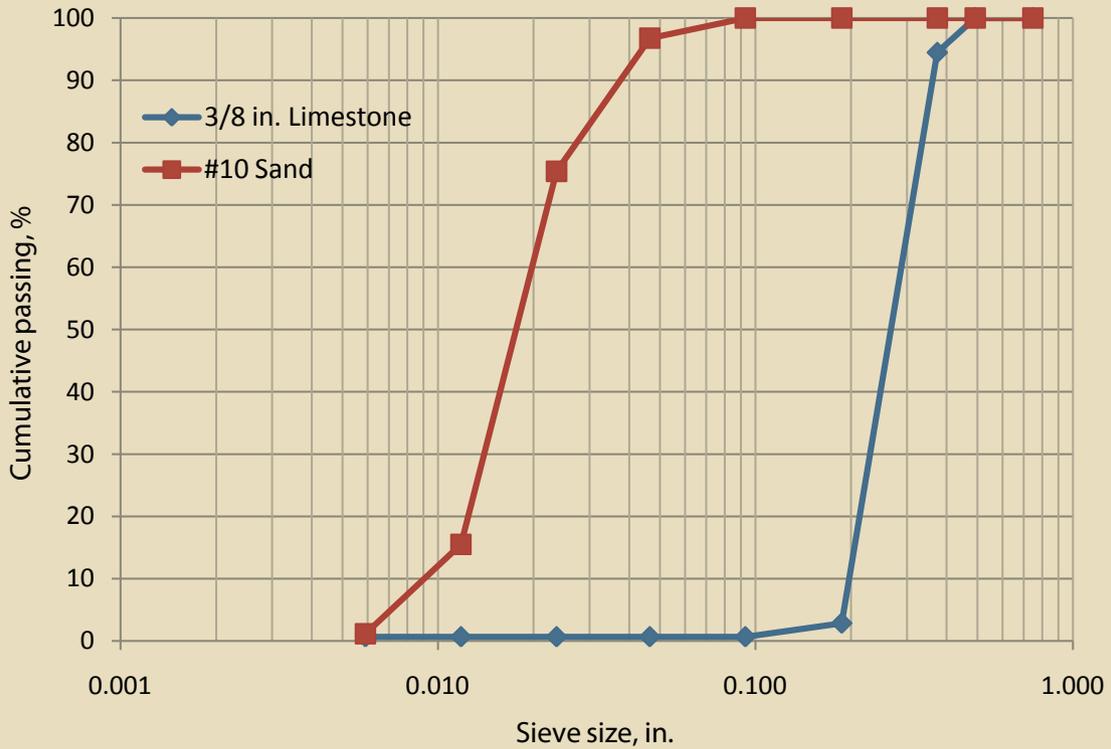


Figure 2. Grading of the aggregates used in mixture development. Note: 1 in. = 25.4 mm

analysis. Water–cementitious materials ratios (w/cm) about 0.2 were adopted to improve the strength and durability.

Figure 3 shows the average compressive strength of the developed mixtures versus time. HPC 2 had the highest 1-day

compressive strength at 13.4 ksi (92 MPa) and HPC 1 had the highest 28-day compressive strength at 17.2 ksi (119 MPa) using standard moist curing for all mixtures.

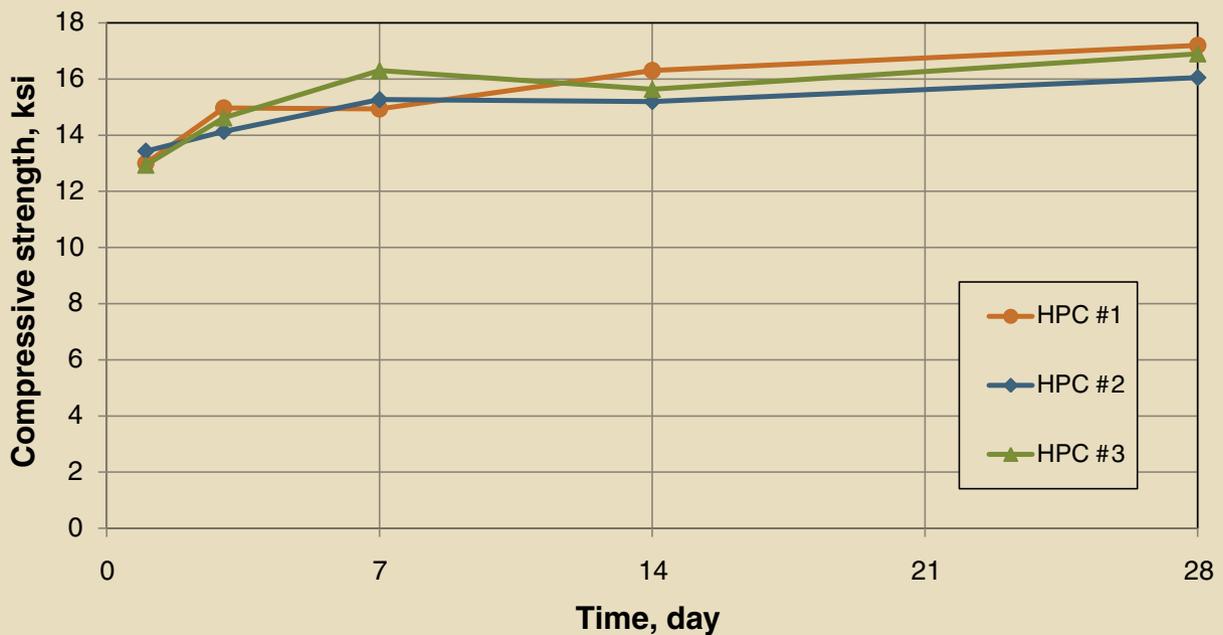


Figure 3. Compressive strength development of the high-performance concrete (HPC) mixtures. Note: 1 in. = 25.4 mm.

Table 2. Tests performed on the developed mixtures

Property	ASTM test	HPC 1	HPC 2	HPC 3
Flowability, in.	C1611	25	29	26
Compressive strength, ksi	C39	17.2	16.05	16.9
Splitting strength, psi	C496	914	1107	785
Modulus of elasticity, ksi	C469	4968	6173	5908
Modulus of rupture, psi	C78	1031	1585	1326
Length change, %	C157	0.04	0.09	0.03

Note: HPC = high-performance concrete. 1 in. = 0.305 mm; 1 psi = 6.895 kPa; 1 ksi = 6.895 MPa; \$1/yd³ = \$1.308/m³.

Table 2 shows the results of the tests performed on the HPC mixtures to evaluate the modulus of elasticity, modulus of rupture, compressive strength, splitting tensile strength, flowing ability, and length change. All mixtures resulted in self-consolidating concrete (SCC), as they achieved an average spread from 25 in. to 29 in. (640 mm to 740 mm) and a visual stability index of zero according to the ASTM C1611¹⁵ slump flow test. This is mainly due to the high dosage of HRWRA. However, it is recommended that the final dosage of HRWRA be determined according to the batched quantity and mixer shear. The results in Table 2 indicate that mixture HPC 2 achieved the highest values of flowability, early strength, modulus of rupture, and splitting strength. Therefore, HPC 2 was chosen for producing the test girders.

All mixtures were developed using a high-energy paddle mixer that had a 5.5 horsepower motor and a batch capacity up to 18 ft³ (0.5 m³). Trial mixtures were made in 4.5 ft³ (0.083 m³) batches to ensure the consistency of the mixture and reliability of test results. The mixing procedure used in these mixtures is slightly different from that of conventional concrete. First, all granular materials were dry mixed for 2 to 3 min. Then, water and HRWRA were added together to the dry mixture, and mixing continued for 10 to 12 min until adequate flowing ability was observed. A slump flow test was performed, and the mixture was considered acceptable if average spread was at least 22 in. (590 mm). Additional HRWRA was added and mixing resumed for another 2 to 3 min if flowability was insufficient.

HPC girder design, detailing, and production

To evaluate the shear capacity of the welded-wire-reinforced HPC girders compared with that of the fiber-reinforced UHPC girders, two full-scale AASHTO Type II girders were designed, fabricated, and tested. The two girders were 18.5 ft (5.6 m) long. **Figure 4** shows the cross section and reinforcement details.

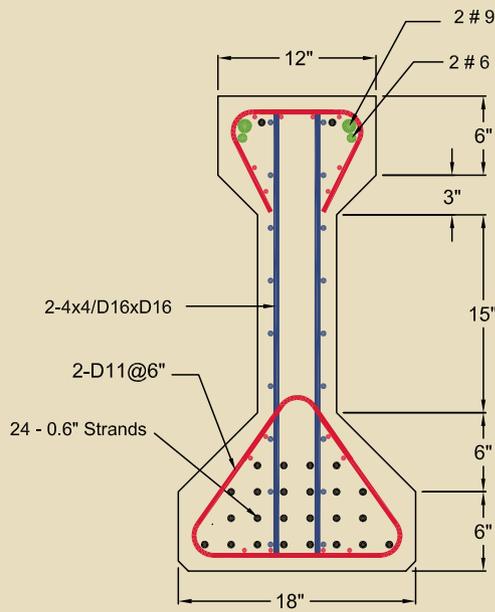
The two girders were designed according to the 2007 AASHTO LRFD Bridge Design Specifications¹⁶ to carry a single

point load of 663 kip (2950 kN) 6 ft (1.8 m) from the centerline of the support (shear span-to-depth ratio equals 2). This load was calculated to generate a shear force equal to the average shear capacity of the UHPC girders tested by FHWA. The two girders were pretensioned using twenty-four 0.6-in.-diameter (15 mm), Grade 270 (1860 MPa), low-relaxation prestressing concrete strands tensioned to 202.5 ksi (1396 MPa). The girders were overdesigned in flexure to ensure that the dominant mode of failure was shear failure.

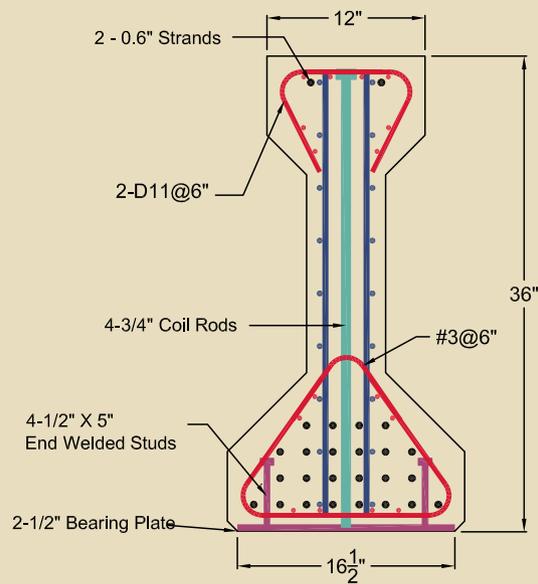
Two partially prestressed (101 ksi [698 MPa]) 0.6-in.-diameter (15 mm) strands were placed in the top flange to control cracking at release, in addition to two no. 6 (19M) and two no. 9 (29M) Grade 60 (420 MPa) reinforcing bars used as compression reinforcement. Shear reinforcement consisted of two Grade 80 (550 MPa), 4 in. × 4 in. (100 mm × 100 mm), D16 (MD100) WWR along the girder length. Details of shear design calculations are shown in the last section.

The end zone was reinforced using four ¾-in.-diameter (19 mm) headed coil rods at 2 in. (50 mm) spacing along the girder axis and welded to the bearing plate. Two ½-in.-thick (13 mm) steel plates were placed at the girder ends and anchored using four ½-in.-diameter headed studs on each plate. The bottom and top flanges were reinforced using D11 (MD70) WWR at 6 in. (150 mm) spacing along the girder length for confinement. This reinforcement was made triangular in shape because of the difficulty of having two consecutive bends with only 4 in. (100 mm) spacing. **Figure 5** shows the auxiliary reinforcement of the girder before concrete placement.

For research purposes, two different mixing procedures were followed in the production of the two girders (2.25 yd³ [1.72 m³] each) to determine the most practical and efficient procedure. The mixing procedure used in the production of girder A was recommended by the precast producer and was different from the one used in making the lab trial mixtures as presented in the previous section. In the new procedure, all cementitious materials were mixed with all of the water and HRWRA for 2 to 3 min.



Mid-Section



End-Section

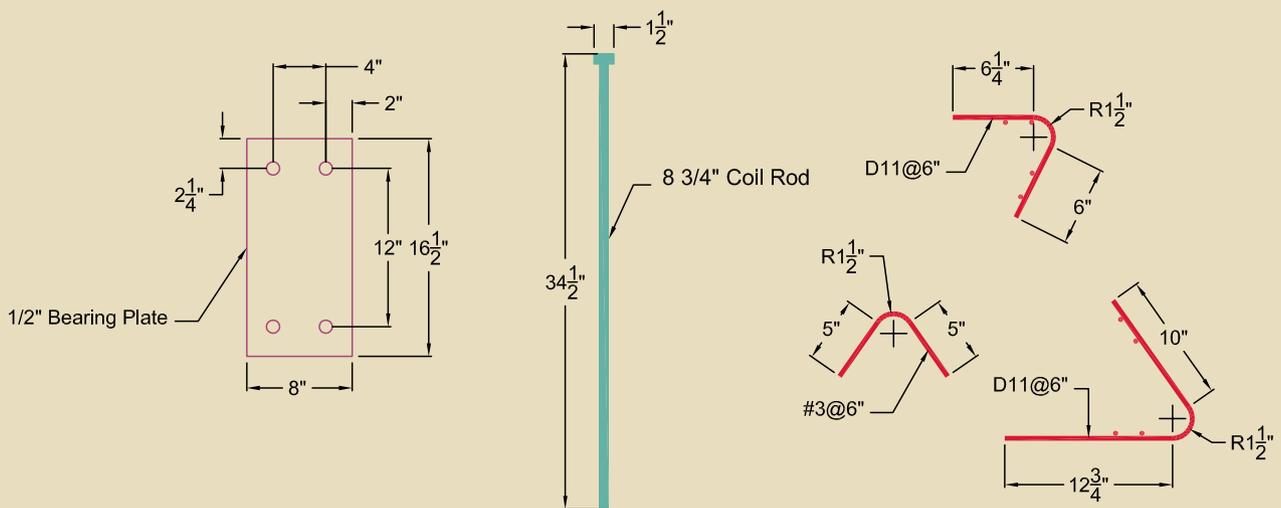


Figure 4. Cross sections and reinforcement details of tested girders. Note: WWR = welded-wire reinforcement. 1 in. = 25.4 mm; D11 = MD70; D16 = MD100; no. 3 = 10M; no. 6 = 19M; no. 9 = 29M.

Then, fine sand was added to the mixture and mixing continued for 10 to 15 min. An additional dosage of HRWRA was used if the slump flow was inadequate.

This procedure did not provide adequate flowability or consistency; it resulted in lumpy and stiff cement balls and excessive heat of hydration. A large dosage of HRWRA had to be added to achieve adequate spread. On the other hand, the mixing procedure used in the production of girder B was successful, as it was exactly the same procedure followed in making the lab trial mixtures. It resulted in a flowable concrete with an average spread of 30 in. (760 mm) without using an extra dosage of HRWRA.

Table 3 lists the actual quantities used in the production



Figure 5. Auxiliary reinforcement of the tested girders.

of girders A and B along with the associated material cost per cubic yard. Cylinders taken from the two batches were tested for compressive strength at different ages (Fig. 6). For the AASHTO Type II girder A, the unsuccessful mixing procedure generated a significant amount of heat that eventually resulted in shrinkage cracks, which adversely affected strength gain. Although a compressive strength of 18 ksi (124 MPa) was achieved after 3 days, the strength declined to less than 15 ksi (104 MPa) after 56 days. For the AASHTO Type II girder B, the successful mixing procedure combined with heat curing resulted in a 1-day compressive strength of 13 ksi (90 MPa) and a strength of 21 ksi (140 MPa) at the time of testing (130 days).

Figure 7 shows the end zone of girder B at prestress release. None of the girder ends experienced any visible cracking due to splitting. Each girder had two end diaphragms that were dimensioned and reinforced (Fig. 8) to anchor prestressing strands, which is the current practice of bridge construction in some states. No deck was placed on either of the two girders, similar to the girders tested by the FHWA in 2001.

Shear testing of WWR HPC girders

Figure 9 shows the test setup of the two 18.5-ft-long (5.64 m) AASHTO Type II girders made of HPC and Grade 80 (550 MPa) WWR to experimentally evaluate their shear capacity. Load was applied vertically to the top flanges using two 400 kip (1800 kN) hydraulic jacks acting on a steel beam supported on a loading plate grouted to the top flange

Table 3. Design and cost of HPC used in girders A and B

Material	Quantity, lb/yd ³	
	Girder A	Girder B
Fine sand	2075	2075
Cement Type III	1120	1120
Class C fly ash	240	240
Silica fume	240	240
HRWRA	93	67
Total water	293	279
Cost, \$/yd ³	246	215

Note: HPC = high-performance concrete; HRWRA = high-range water-reducing admixture. 1 lb/yd³ = 0.593 kg/m³; \$1/yd³ = \$1.308/m³.

of the girder. Each girder was instrumented with a slide wire potentiometer 6.25 ft (1.91 m) from the girder end. The potentiometer location was aligned vertically with the point of load application to measure the vertical deflection. Surface-bonded electrical resistance strain gauges were attached to the girder at various locations to determine the strain profile at different sections. Load was applied gradually, and the girder was visually inspected at increments of 100 kip (445 kN) until the ultimate capacity was reached.

Figure 10 shows the load-deflection response of the two tested girders as well as one of the fiber-reinforced UHPC girders tested by FHWA (girder 24S). For girder A, the peak load was 647 kip (2878 kN), which corresponds to

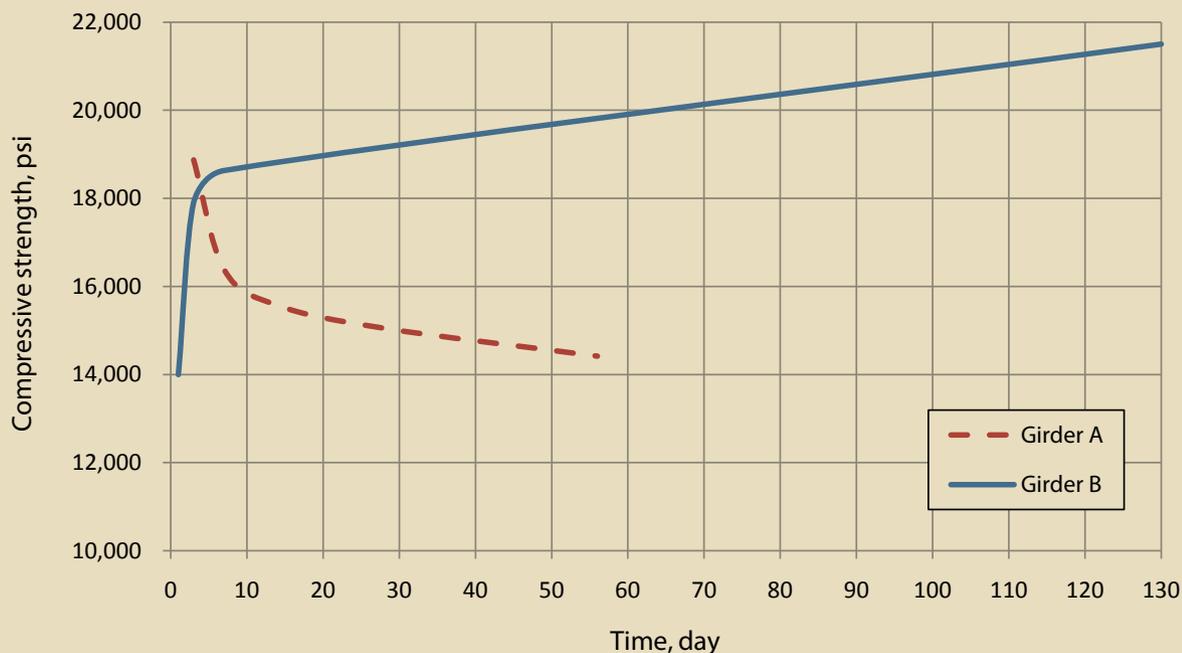


Figure 6. Compressive strength development of high-performance concrete used in Girders A and B. Note: 1 psi = 6.895 kPa.



Figure 7. End zone of test girder B after release.

an ultimate shear capacity of 432 kip (1922 kN), and the maximum deflection was 0.93 in. (23.6 mm). For girder B, the peak load was 746 kip (3318 kN), which corresponds to an ultimate shear capacity of 498 kip (2215 kN), and the maximum deflection was 1.03 in. (26.2 mm).

The load-deflection diagrams show that the two girders exhibited linear behavior up to the flexure cracking load of about 450 kip (2002 kN), then nonlinear behavior up to failure. The lower shear capacity and modulus of elasticity of girder A compared with girder B is mainly due to the lower concrete strength that resulted from using an improper mixing procedure. Comparing the load-deflection response of the WWR-reinforced HPC girder B with that of fiber-reinforced UHPC girder 24S indicates that despite that the two girders had almost the same capacity, 24S showed higher softening behavior at earlier load, which resulted in significantly higher deflections. It should be noted that girder 24S had a span of 24 ft and was loaded 7.5 ft from the support.

Table 4 lists the predicted flexure cracking load, shear cracking load, ultimate load, and maximum deflection. These values were calculated using a design compressive strength of 15 ksi (100 MPa) and according to the 2007 AASHTO LRFD specifications. Flexural cracking load was calculated based on service III limit state, while ultimate shear capacity was calculated using the new simplified procedure for shear design shown in the last section. Shear cracking load was calculated using the principal tensile stress, and maximum deflection was calculated using the bilinear cracked section analysis.

A comparison of the observed values with the predicted ones indicates that predictions are slightly conservative and relatively accurate. This means that the well-established theories of flexure and shear design of reinforced concrete girders provide consistent and reliable predictions of the

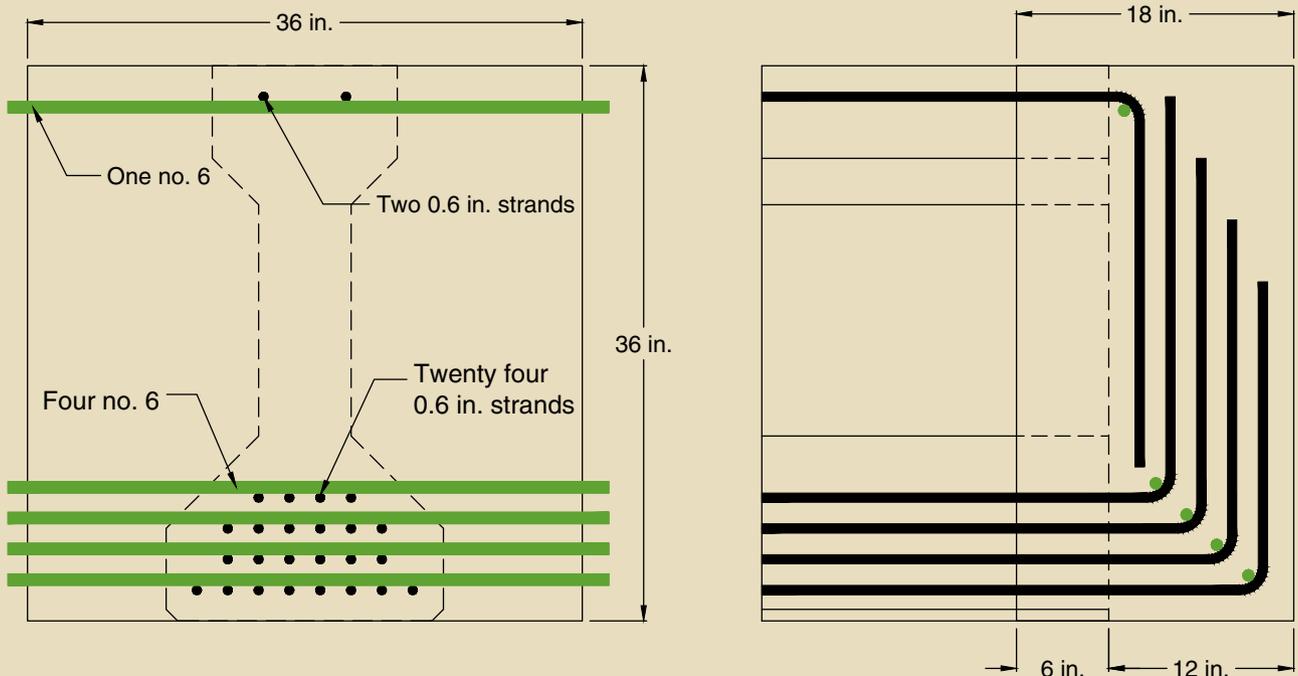


Figure 8. Dimensions and reinforcement details of end diaphragms. Note: 1 in. = 25.4 mm; No. 6 = 19M.

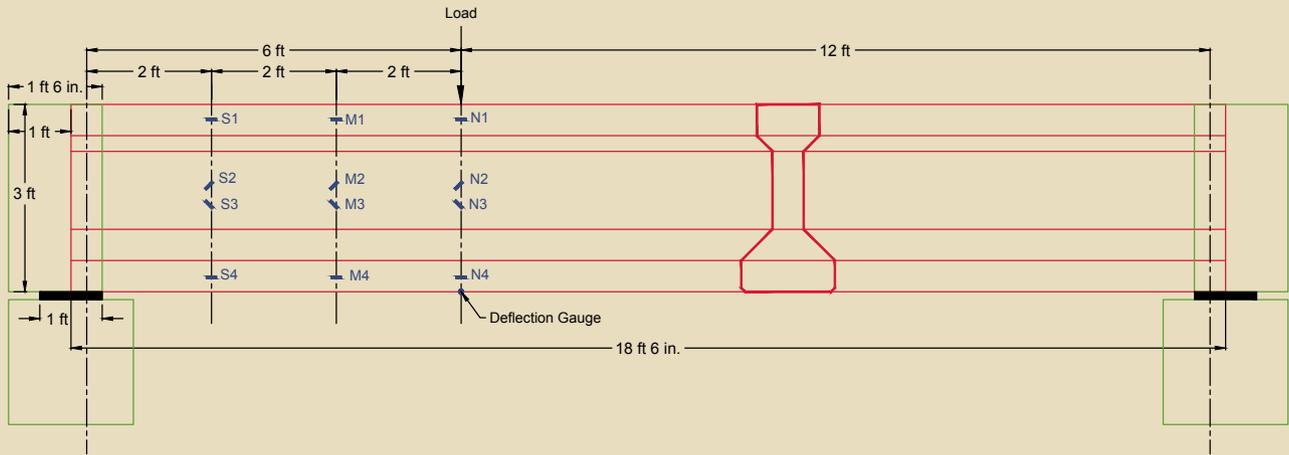


Figure 9. Test setup and strain gauge locations. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.

flexure and shear capacity of HPC girders reinforced with WWR.

Alternatively, predicting the shear capacity of fiber-reinforced UHPC girders is not covered by most of the standard structural design codes. In the AFGC recommendations, the shear capacity of fiber-reinforced UHPC is the sum of the concrete contribution and fiber contribution, which was predicted to be 247 kip (1100 kN) for the FHWA-tested AASHTO Type II girders.³ This value is substantially less than the actual capacity of the tested girders, which indicates

the lack of theory and experiments required to ensure consistency and reliability of capacity predictions.

Figure 11 plots the readings of strain gauges M1 and M4 at the top and bottom flanges of girders A and B for a section that was 4 ft (1.2 m) from the support centerline. Compressive strains at the top flange were found to be as expected because they increased linearly with the load up to failure. Tensile strains at the bottom flange were also found to be as expected because they increased gradually with the load up to the yielding of the prestressing strands

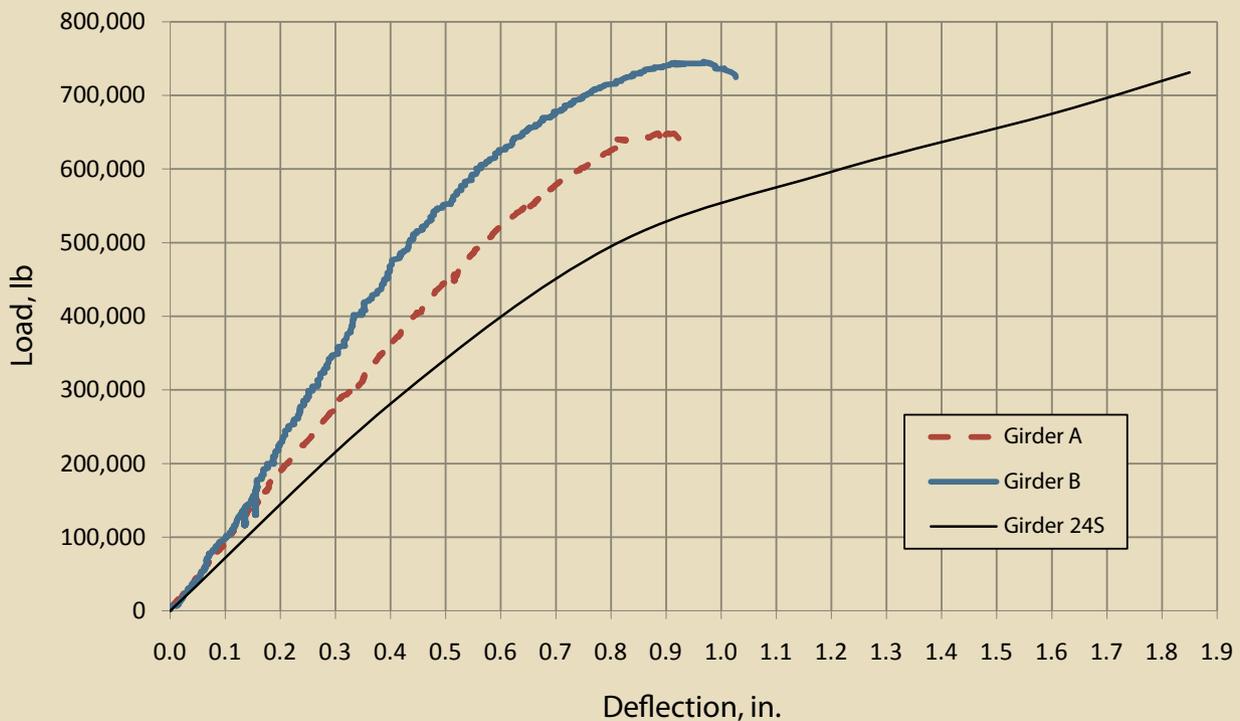


Figure 10. Load-deflection relationships for girders A and B versus girder 24S. Note: 1 in. = 25.4 mm; 1 lb = 4.448 N.

Table 4. Comparing predicted versus actual capacities of girders A and B

Parameter	Prediction	Girder A	Girder B
Flexure cracking load, kip	420	450	480
Shear cracking load, kip	120	140	150
Ultimate shear capacity, kip	429	432	498
Ultimate deflection, in.	0.80	0.93	1.03

Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

at a load of about 550 kip (2446 kN). Tensile strains continued to increase significantly with a slight increase in the load until failure occurred. The readings of strain gauge M4 in girder B were incomplete because of cracking of the concrete at the strain gauge location.

Figure 12 plots the readings of strain gauges M2 and M3, located parallel to the diagonal compression and diagonal tension directions, respectively, in the web of girders A and B for the same section. These readings indicate that compression strains increased linearly with the load up to failure, while the tension strains increased linearly until diagonal cracks occurred, which is evident in the sudden increase of the M3 gauge readings in girder A. The readings of the M3 gauge in girder B became unreliable after cracking; apparently the strain gauge was damaged or detached from the concrete surface.

Figure 13 shows girders A and B at failure. Despite the difference in the shear capacity of the two girders, their failure mechanisms were similar. Typical diagonal shear cracks were initiated in the web at midheight and propagated down toward the support and up toward the top flange. Due to the triangular shape of the top flange reinforcement,

the concrete cover of the top flange spalled and additional diagonal cracks developed at flatter angles as the load increased. As the load approached the ultimate capacity, the web cracks widened, causing concrete spalling within the high-stress area of the web. At failure, substantial cracking and rotation of the end diaphragm close to the loading point occurred due to the slippage of prestressing strands anchored in the diaphragm.

Based on the observed cracking pattern, it was determined that the primary failure mode was diagonal tension initiated by the slippage of underdeveloped prestressing strands (theoretical development length was about 12 ft [3.6 m]). The higher strength and quality concrete of girder B resulted in greater bond strength with the strands than that of girder A. This was also confirmed by the higher values of the ultimate load and ductility of girder B compared with that of girder A (**Fig. 10**).

Discussion

Table 5 shows a comparison between two sets of girders: fiber-reinforced UHPC girders obtained from the literature^{3,8} and welded-wire-reinforced HPC girders tested in

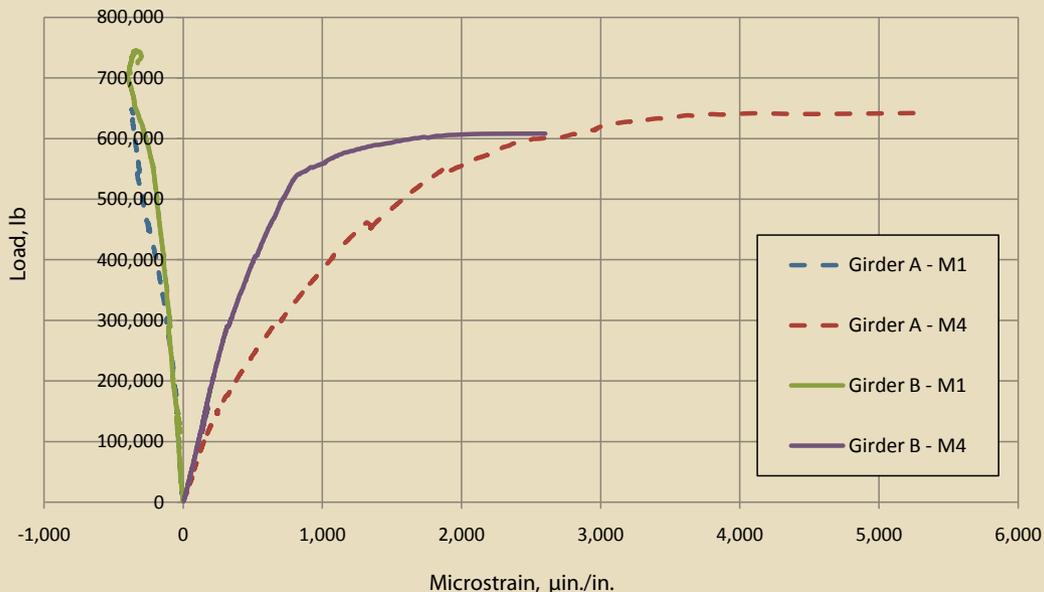


Figure 11. Readings of M1 and M4 strain gauges for girders A and B. Note: 1 in. = 25.4 mm; 1 lb = 4.448 N.

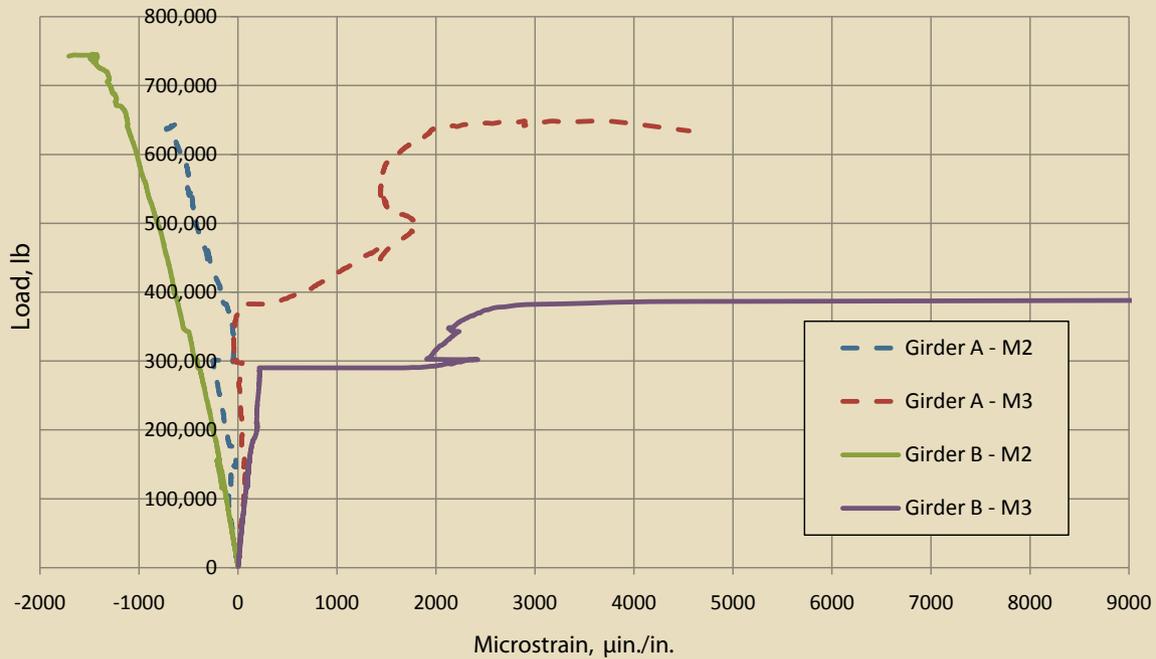


Figure 12. Readings of M2 and M3 strain gauges for girders A and B. Note: 1 in. = 25.4 mm; 1 lb = 4.448 N.

this study.

The results of the shear tests performed on each set of girders show that the average shear capacity of the non-proprietary welded-wire-reinforced HPC is comparable to that of commercial fiber-reinforced UHPC. The variation of test results also indicated that using WWR results in a more consistent and predictable shear capacity than when using random steel fibers. Despite having comparable shear capacity for the two sets of girders, the shear failure mechanism was different.

Figure 13 shows that the WWR was able to uniformly distribute the cracks and maintain the girders' integrity even after the concrete cover had spalled and the girders were unable to carry any additional load. In spite of the breakage of the welds at failure in some locations, cross wires provided adequate anchorage of vertical wires in the top and bottom flanges even after failure. The fiber-reinforced UHPC girder that had the highest shear capacity had two large parallel shear cracks that released in a brittle fashion at failure. Also, although none of the bottom strands slipped, the two top flange strands broke at several locations during the failure, resulting in girder disintegration.³

Table 5 also shows the material cost of the girders (excluding the cost of prestressing strands) calculated for the purpose of comparison. The cost of Grade 80 (550 MPa) WWR used for girder shear reinforcement and that used for top-flange and bottom-flange reinforcement was estimated to be \$160/yd³ (\$215/m³), assuming a unit price of \$0.50/lb (\$1.10/kg). According to the quantities listed in Table 4, the cost of the used HPC is \$215/yd³ (\$289/m³). This results

in a total cost of \$375/yd³ (\$504/m³), which is 62.5% less than the cost of commercial fiber-reinforced UHPC (about \$600/yd³ [\$806/m³] for concrete and \$400/yd³ [\$538/m³] for steel fibers).

These savings are in addition to the significant savings in production costs because the proposed HPC with WWR follows industry-standard procedures for batching, mixing, placement, and curing, which makes it attractive to U.S. precast, prestressed concrete producers. In contrast, the production of fiber-reinforced UHPC represents a significant disadvantage to U.S. precast, prestressed concrete plants because they are not set up for a 45 min mixing cycle and 48 hr intense heat curing before removal of the element from the prestressing bed. This alone could double the cost of the finished product. Again, the authors emphasize that this investigation and comparison deal only with the shear capacity of UHPC and the corresponding material cost. The benefits of other mechanical and durability characteristics of UHPC were not considered in this comparison.

Table 5 also shows the ACI 318-08 and 2007 AASHTO LRFD specification limits on nominal shear stress. These limits are stipulated in ACI 318-08 sections 11.3.2 and 11.4.7.9 and AASHTO section 5.8.3.3. Comparing these limits with the actual shear capacity of the tested girders indicates that ACI 318-08 limits underestimate the shear resistance of reinforced concrete sections, which is not the case for the AASHTO LRFD limits.



Girder A



Girder B

Figure 13. Shear failure of the tested AASHTO Type II girders made of high-performance concrete (HPC) and welded-wire reinforcement (WWR).

Conclusion

The objective of the research was to investigate the structural performance and economics of using Grade 80 (550 MPa) welded-wire reinforced precast, prestressed HPC girders compared with using fiber-reinforced UHPC girders. The shear capacity of two full-scale AASHTO Type II girders reinforced with WWR was experimentally evaluated and compared with that of similar girders made of commercial fiber-reinforced UHPC and tested by FHWA in 2001. This paper also presented the properties of the developed non-proprietary HPC used in the production of the two test girders. This investigation has led to the following conclusions:

- The use of Grade 80 (550 MPa) WWR in shear reinforcement of HPC I-girders results in shear capacity comparable to that of fiber-reinforced UHPC girders. Moreover, experimental results have shown that precast, prestressed HPC I-girders reinforced with WWR

have more consistent and predictable results than those of fiber-reinforced UHPC I-girders.

- An economical, nonproprietary HPC that satisfies the following requirements can be successfully developed and applied to precast, prestressed concrete bridge I-girders:
 - flowing ability with at least 22 in. (560 mm) spread
 - standard batching, mixing, placing, and curing procedures at existing U.S. plants to ensure efficient production
 - a minimum 24 hr compressive strength of 12 ksi (83 MPa) for early release of prestressing
 - a design compressive strength of at least 15 ksi (103 MPa) to ensure the applicability of the current AASHTO LRFD specifications
- The material cost analysis of the HPC girders reinforced with WWR showed a 64% savings compared with fiber-reinforced UHPC in terms of concrete cost and a 60% saving in terms of steel cost. This translates to an average material cost savings of 62% in addition to the significant production cost savings of following standard concrete mixing, placement, and curing operations and the ease of handling WWR that reduces reinforcement congestion.

Shear design calculations

The shear capacity was calculated according to the 2007 AASHTO LRFD specifications simplified design equation. The critical section was taken to be 6 in. (150 mm) from the centerline of the applied load because the width of the loading plate was 12 in. (305 mm). This section was designed to have the average shear capacity that the FHWA girders had (441 kip [1960 kN]).

$$P = \text{load} = 663 \text{ kip}$$

$$V_u = \text{factored shear force acting on the section of investigation} = 442 \text{ kip}$$

$$M_u = \text{factored moment acting on the section of investigation} = 29,172 \text{ kip-in.}$$

$$d_v = \max\left(0.9d_e, 0.72h, d_e - \frac{a}{2}\right) \text{ AASHTO article 5.8.2.9}$$

where

$$d_v = \text{effective shear depth}$$

Table 5. Comparison of the shear capacity and material cost

Girder	Material	Tested by	Material cost, \$/yd ³	28-day compressive strength, ksi	Overall span, ft	Shear span, ft	Applied load, kip	Shear capacity, kip	Average shear stress, ksi	ACI Limit, ksi	AASHTO limit, ksi
28S	UHPC + fibers	FHWA	1000	27.6	28.0	6.50	500	384	2.01	1.62	5.18
24S	UHPC + fibers	FHWA	1000	27.6	24.0	7.50	731	503	2.63	1.62	5.18
14S	UHPC + fibers	FHWA	1000	27.6	14.0	6.00	766	438	2.29	1.62	5.18
Average				27.6	22.0	6.70	666	441	2.31	1.62	5.18
18A	HPC + WWR	UNL	375	15.0	18.0	6.00	647	432	2.28	1.19	2.81
18B	HPC + WWR	UNL	375	19.0	18.0	6.00	746	498	2.63	1.34	3.56
Average				17.0	18.0	6.00	697	465	2.46	1.27	3.19
Small-scale I-girders	UHPC + fibers	Hegger et al.	1000	29.3	10.5	5.25	123	62	2.28	1.67	5.49

Note: UHPC = ultra-high-performance concrete; WWR = welded-wire reinforcement. 1 ft = 0.305 m; 1 kip = 4.448 kN; 1 ksi = 6.895 MPa; \$1/yd³ = \$1.308/m³.

d_e = distance from the top of the girder to the resultant of the prestressing force

h = total height of the composite section

a = depth of the Whitney stress block

d_v = 28.35 in. (720.1 mm)

$$\epsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po} \right)}{E_s A_s + E_p A_{ps}}$$

AASHTO Eq. (5.8.3.4.2-4)

For

$$0 \leq \epsilon_s \leq 0.006$$

where

ϵ_s = strain in nonprestressed longitudinal tension reinforcement

M_u should not be taken less than $|V_u - V_p|d_v$

N_u = factored axial force, taken as positive if tensile

V_p = component in the direction of the applied shear of the effective prestressing force, positive if resisting the applied shear

A_{ps} = area of prestressing reinforcement on the flexural tension side of the member

f_{po} = locked-in difference in strain between concrete and prestressing steel times the modulus of elasticity of the prestressing strands

E_s = modulus of elasticity of nonprestressed reinforcement

A_s = area of nonprestressed reinforcement on the flexural tension side of the member at the section under consideration

E_p = modulus of elasticity of prestressed reinforcement

$$\epsilon_s = \frac{\left[\frac{29,172}{28.35} + (0.5)(0) + |442 - 0| - (5.21)(202.5) \right]}{(29,000)(0) + (28,500)(5.2)}$$

$$= 0.0028$$

$$0 \leq 0.003 \leq 0.006 \text{ OK}$$

$$\beta = \frac{4.8}{1 + 750\epsilon_s} \quad \text{AASHTO Eq. (5.8.3.4.2-1)}$$

where

β = factor indicating the ability of diagonally cracked concrete to transmit tension and shear; the previous equation is used when the minimum amount of shear reinforcement is provided

$$\beta = \frac{4.8}{1 + 750(0.0028)} = 1.55$$

$$\theta = 29 + 3500\epsilon_s \quad \text{AASHTO Eq. (5.8.3.4.2-3)}$$

where

θ = angle of inclination of diagonal compressive stresses

$$\theta = 29 + 3500(0.0028) = 38.8 \text{ deg}$$

$$V_c = 0.0316\beta\sqrt{f'_c}b_vd_v \quad \text{AASHTO Eq. (5.8.3.3-3)}$$

where

V_c = shear resistance contribution of the concrete

$\sqrt{f'_c}$ = compressive strength of the concrete

b_v = effective web width taken as the minimum web width within the depth d_v

$$V_c = 0.0316(1.55)\sqrt{15}(6)(28.35) = 32.3 \text{ kip (144 kN)}$$

$$V_{sv} = \frac{A_{sv}f_yd_v \cot \theta}{s_v} \quad \text{AASHTO Eq. (5.8.3.3-4)}$$

where

V_{sv} = shear resistance contribution of the vertical steel

A_{sv} = vertical mild shear reinforcement provided over the distance s_v

f_y = yield stress of the mild shear reinforcement

s_v = spacing of the vertical reinforcement

$$V_{sv} = \frac{(0.96)(80)(28.35)\cot(38.8)}{12} = 226 \text{ kip (1000 kN)}$$

$$V_{sh} = \frac{A_{vh}f_yd_v}{s_h} \quad \text{AASHTO Eq. (5.8.3.3-4)}$$

where

V_{sh} = the shear resistance contribution of the horizontal steel

A_{vh} = horizontal mild shear reinforcement provided over the distance s_h

s_h = the spacing of the horizontal reinforcement

$$V_{sh} = \frac{(0.96)(80)(28.35)}{12} = 182 \text{ kip (810 kN)}$$

$$V_n = (V_c + V_{sv} + V_{sh} + V_p) \quad \text{AASHTO Eq. (5.8.3.3-1)}$$

where

V_n = nominal shear resistance

$$V_n = (32.3 + 226 + 182 + 0) = 440 \text{ kip (1960 kN)}$$

$$442 \text{ kip} \approx 440 \text{ kip} \quad \text{OK}$$

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Notation

- a = depth of the Whitney stress block
- A_{ps} = area of prestressing reinforcement on the flexural tension side of the member
- A_s = area of nonprestressed reinforcement on the flexural tension side of the member at the section under consideration
- A_v = vertical mild shear reinforcement provided over the distance s_v
- A_{vh} = horizontal mild shear reinforcement provided over the distance s_h
- b_v = effective web width taken as the minimum web width within the depth d_v
- d_e = distance from the top of the girder to the resultant of the prestressing force
- d_v = effective shear depth
- E_p = modulus of elasticity of prestressed reinforcement
- E_s = modulus of elasticity of nonprestressed reinforcement
- f'_c = compressive strength of the concrete
- f_{po} = locked-in difference in strain between concrete and prestressing steel times the modulus of elasticity of the prestressing strands
- f_y = yield stress of the mild shear reinforcement
- h = total height of the composite section
- M_u = factored moment acting on the section of investigation, should not be taken less than $|V_u - V_p|d_v$
- N_u = factored axial force, taken as positive if tensile
- P = load
- s_h = spacing of the horizontal reinforcement

- s_v = spacing of the vertical reinforcement
- V_c = shear resistance contribution of the concrete
- V_n = nominal shear resistance
- V_p = component in the direction of the applied shear of the effective prestressing force, positive if resisting the applied shear
- V_{sh} = shear resistance contribution of the steel oriented horizontally
- V_{sv} = shear resistance contribution of the steel oriented vertically
- V_u = factored shear force acting on the section of investigation
- w/cm = water–cementitious materials ratio
- β = factor indicating the ability of diagonally cracked concrete to transmit tension and shear
- ϵ_s = strain in nonprestressed longitudinal tension reinforcement
- θ = angle of inclination of diagonal compressive stresses

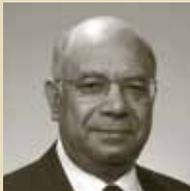
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Synopsis

Several experimental investigations have indicated that the shear capacity of fiber-reinforced ultra-high-performance concrete (UHPC) girders outperforms that of conventionally reinforced high-strength concrete girders. However, the exceptionally high material and production cost of fiber-reinforced UHPC girders limits its use in bridge applications. This paper presents the experimental work and cost analysis performed to evaluate the shear capacity and economics of using welded-wire reinforcement (WWR) in

precast, prestressed high-performance concrete (HPC) bridge I-girders. The development of an economical, practical, and nonproprietary HPC is presented. Two full-scale American Association of State Highway and Transportation Officials (AASHTO) Type II girders were designed according to the *AASHTO LRFD Bridge Design Specifications* and fabricated using the developed mixture and Grade 80 (550 MPa) WWR. The shear testing of the two girders indicated that welded-wire-reinforced HPC girders have shear capacity comparable to that of fiber-reinforced UHPC girders while being more economical. In addition, the production of welded-wire-reinforced HPC girders complies with the current industry practices and eliminates the mixing, consolidation, and curing challenges associated with the production of fiber-reinforced UHPC girders.

Keywords

Bridge, girder, high-performance concrete, HPC, shear capacity, steel fibers, UHPC, ultra-high-performance concrete, welded-wire reinforcement, WWR.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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