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Increased Shear Strength of AASHTO Girders using Web Welded Wire Reinforcement

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***Abstract*:- Reactive powder concrete (RPC), known as ultra-high-performance concrete (UHPC), is currently used in producing bridge girders with superior mechanical properties. This paper investigates the use of orthogonal welded wire reinforcement (WWR) in shear reinforcement of precast/prestressed bridge I-girders. Two RPC I-girders were fabricated using grade 80 WWR meshes for shear reinforcement. The RPC girders were tested until strength failure is achieved, and girder shear capacity is quantified. The average shear capacity for the tested girders at failure was 2068 kN, compared to an estimated capacity of 2104 kN calculated using AASHTO LRFD equations. The outcomes of this research showed that orthogonal WWR can be successfully used in fabricating cost-effective shear reinforcement of RPC I-girders with superior performance. The availability and ease of construction of WWR could be efficiently used in producing economic ultra-high strength RPC I-girders for bridge construction.**

Keywords: Reactive powder concrete (RPC); random steel fibers; welded wire reinforcement (WWR); I-girders; shear capacity

1. INTRODUCTION

Reactive powder concrete (RPC) was introduced to the United States construction industry by the U.S. Army Corps of Engineers in the late 1980s. RPC became commercially available in early 2000s when the Federal Highway Administration (FHWA) investigated the possible use of RPC in improving the US highway infrastructure conditions. RPC applications included prestressed girders, precast concrete piles, retrofit of deteriorating bridge elements, bridge decks overlays, and security related construction projects.

RPC mixes include three main constituents: (1) preblended powder of fine portland cement, quartz flour, micro-silica, and sand, (2) high strength steel or organic fibers, and (3) high range water reducing agent (super plasticizers). Proprietary RPC mixes, available in the US construction market, is produced by Lafarge, and commercialized under the name Ductal. Proprietary RPC mixes are characterized by self-consolidating concrete flowing ability, early high strength, final compressive strength in excess of 150 MPa, increased durability and resistivity to alkali-silica reactivity (ASR) [1, 2, 3, 4].

Different research programs focus on developing economic non-proprietary RPC mixes, incorporate RPC mixes in the accelerated bridge construction (ABC) projects [5], and increase the environmental compliance of construction projects by fabricating smaller structural sections of superior strength; thus, minimizing the cement manufacturing and consumption [6, 7, 8]. The main objective of this research is to develop economic, self-compacting, RPC mixes using local mix constituents. Developed non-proprietary mixes will be used in fabricating precast/prestressed I-girders, reinforced with welded wire reinforcement to improve girders capacity. Shear strength of fabricated girders will be experimentally calculated, and compared to strength calculated using AASHTO LRFD equations [9]. The overall cost and performance of fabricated girders will be compared to results of comparable girders fabricated using proprietary RPC mixes.

2. LITERATURE REVIEW

RPC mixes are successfully used in the fabrication of different bridge components with superior strength and durability including in-situ connections between prefabricated bridge elements, rehabilitation of aging bridge elements poured using normal strength concrete mixes, and the full construction of new RPC structural bridge elements. In addition to higher strength, RPC displays higher performance against environmental loading resulting from freeze-thaw cycles and lower alkali-silica reactivity due to the high content of supplementary cementitious materials [10, 11]. In addition, the use of micro and nano-sized SCMs in RPC mixes in partial replacement of cement result in reduced carbon footprint and increased sustainability of construction projects. In the period from 2006 to 2019, departments of transportation (DoTs) across the United States constructed 200 bridges using RPC mixes in fabricating different precast or in-situ bridge elements. RPC bridges and their construction dates are shown in Figure 1

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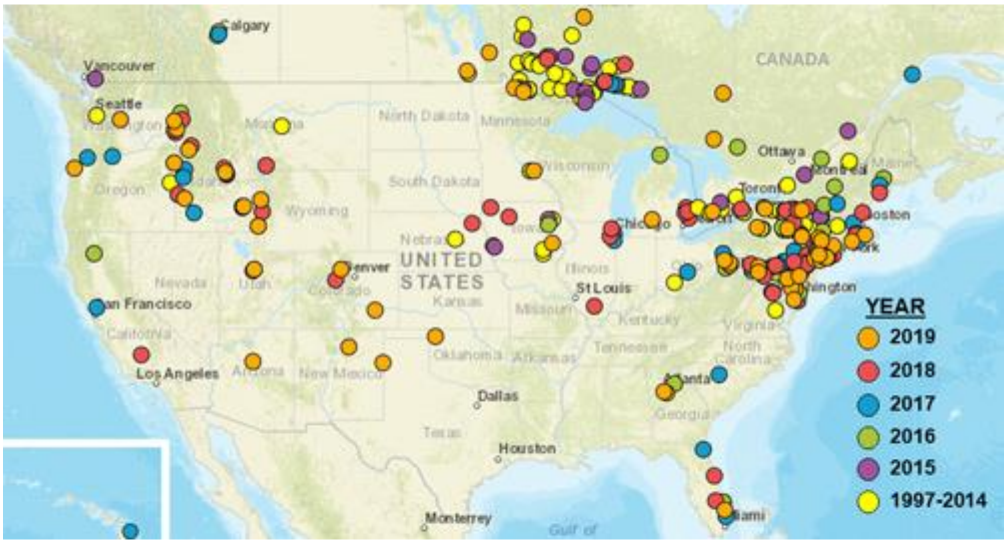

Figure 1. RPC bridge map in the United States (Courtesy of the FHWA)

Larger prestress strands and steel confining rebars are being used in fabricating RPC I-girders to increase girders flexure capacity [12], and welded wire reinforcement (WWR) is used in girders shear reinforcement. WWR is fabricated using reinforcing steel mesh made of parallel series of high strength steel bars welded in square or rectangular mesh. WWR could be fabricated using plain wires, deformed wires, or a combination of both types according to the WWR-concrete bond strength. Small diameter, closely spaced WWR meshed provide a uniform stress distribution and better crack control for improved serviceability. The longitudinal reinforcement bars of WWR provides the mesh with overall stability of its vertical wires during construction, which improves the quality control procedure outcomes.

The use of WWR has substantially increased in precast/prestressed concrete industry during the recent decade. WWR increased market share is attributed to ease of construction, construction cost savings, and avoiding schedule overruns. According to specifications, deformed welded wire reinforcement for concrete should have a minimum tensile strength of 550 MPa, minimum yield strength of 480 MPa, and weld shear strength of 240 MPa [13]. Steel content of precast structure could be reduced by 27% when WWR meshes are incorporated in member fabrication, in addition, WWR contributes are easier in placement, which contributes to the construction site safety. WWR handling and placement is shown in Figure 2 [14]. The WWR is manufactured from cold-worked steel wires, welded in orthogonal meshes. The cold working process results in higher yield strength. However, it significantly decreases the ductility of WWR meshes [15]

Figure 2. Structural WWR handling and placement in construction sites [14]

WWR ease of construction and precise prefabricated dimensions enable bridge designers to cast I-girders with congested web reinforcement for increased shear capacity, as shown in Figure 3. Self-consolidating concrete flowing and passing abilities are required to avoid internal voids and poor surface finishing. RPC mixes with spread diameter in excess of 60 cm. is required to prefabricate heavy WWR reinforced bridge girders [16]

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Figure 3. WWR meshes for web shear reinforcement of precast I-girders

To-date, the specifications of WWR used in different structural applications including flexure, axial, shear, and torsion reinforcement are presented in the Building Code Requirements for Structural Concrete, ACI 318-19, and Commentary on Building Code Requirements for Structural Concrete, ACI 318R-19 [17], as shown in Table 1.

Table 1. Applications of ASTM A1064 and A1022 deformed reinforcement [17, 18]

Usage	Application	Maximum yield strength permitted for design	Acceptance	
			Deformed WWR	Deformed wire
Flexure; Axial Force Shrinkage and temperature	Special seismic systems	420 MPa	No	No
	Other	560 MPa	yes	Yes
Lateral support of longitudinal bars or concrete confinement	Special seismic systems	700 MPa	Yes	Yes
	Spirals	700 MPa	No	Yes
	Other	560 MPa	Yes	Yes
Shear	Special seismic systems	420 MPa	Yes	Yes
	Spirals	420 MPa	No	Yes
	Shear friction	420 MPa	Yes	Yes
	Stirrups, ties	420 MPa	Yes	Yes
	Hoops	560 MPa	Yes	No
Torsion	Longitudinal & Transverse	420 MPa	Yes	Yes

The impact of WWR mesh spacing on the efficiency of using WWR in shear reinforcement of I-girders was investigated. Full scale beams with different WWR mesh sizing were tested in shear. The experimental investigation showed that shear cracks were better controlled when WWR mesh had smaller spacing for longitudinal and vertical wires [19]. The deformation in WWR results in increased shear capacity of fabricated girders due to the diagonal cracks improved distribution pattern [20]. WWR girders display a significant increase in capacity under static loads as compared to its performance under dynamic loading. The ACI code provisions over-estimate WWR performance under cyclic loading, thus, it is recommended to use minimum web-reinforcement for girders subjected to dynamic loads [21]. Overall, the WWR improved mechanical characteristics, higher WWR-concrete bond, and improved plant quality control in producing WWR results in higher girders capacity, as compared to conventional construction using reinforcing steel bars [22]. The following section of this research paper describes the research objectives, research methodologies followed, and the outcomes of this research.

3. RESEARCH OBJECTIVE AND METHODOLOGY

The main objective of this research is to fabricate bridge I-girders using non-proprietary RPC mix and welded wire reinforcement with superior strength and durability. Girders strength are calculated using AASHTO LRFD equations, and compared to girder

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strength measured through lab shear strength testing. Girder strength and economy is compared with results attained by testing girders fabricated using proprietary RPC mixes. Research methodology includes:

1. Economic, self-compacting, non-proprietary RPC mixes are designed
2. Two AASHTO Type II I-girders are fabricated using the developed RPC mix. Fabricated girders are reinforced using orthogonal WWR for increased shear capacity
3. Girder shear strength is calculated using AASHTP LRFD design equations (research analytical phase)
4. Girders are instrumented with strain gages and LVDTs to measure girder stresses and deflection under applied loads
5. Girders are tested using point load acting at a 1.8-meter shear span
6. Loading is increased until shear failure is attained, and Load deflection curve is developed (research experimental phase)
7. Research and analytical and experimental phases are compared for data validation
8. Validated results are compared with comparable girders fabricated using proprietary RPC mixes

4. RESEARCH EXPERIMENTAL PHASE

The research experimental investigation includes two phases: Phase I – to develop economic non-proprietary self-consolidating RPC mixes to be used in fabricating precast/prestressed bridge I-girders with superior mechanical characteristics, and Phase II – fabricate 2 AASHTO Type II girders using the developed RPC mixes and WWR for web shear reinforcement. The shear capacity of fabricated girders will be tested and compared to the capacity calculated using AASHTO LRFD equations. The mechanical performance and cost of fabricated girders will be compared to similar girders fabricated using proprietary RPC mixes and tested at the Federal Highway Administration laboratory.

4.1. Development of Non-Proprietary RPC Mixes

The RPC mixes developed in this research are designed for fabricating economic high strength precast/prestressed I-girders. As an industry requirement, RPC mixes should attain the following:

Fresh concrete mix characteristics include a mixing time lesser than 20 minutes. This is specified by precast facility managers to successfully pour large span girders without cold joints formation between consecutive pours. Developed mixes are required to be self-consolidating due to the heavy reinforcement of girders. A minimum spread diameter of 60 cm. was targeted.

Hardened concrete mix characteristics include a minimum 24-hour compressive strength of 70 MPa for early release of strands, and a minimum final strength of 105 MPa at 28-day age. A targeted maximum material cost of $500 per cubic yard was specified for increased economic feasibility

The RPC mixes are designed to satisfy the afore-mentioned characteristics. First, the random steel fibers incorporated in proprietary mixes are eliminated to reduce the material cost and mixing time. Second, type III portland cement is used to achieve high early strength required for prestressing strands release. Third, class C fly ash was used as an economic SCM to replace quartz flour to reduce mix production cost. In addition to fly ash, micro-silica is used to increase the concrete strength, minimize voids ratio, and improve RPC durability. Finally, high range water reducers (HRWRs) are used to maintain flowing ability at a water-to-powder ratio less than 0.2. Different non-proprietary RPC mixes were produced given the afore-mentioned design considerations. Based on flowing ability and compressive strength test results, RPC mixes, shown in Table 2, were initially selected for potential use in the fabrication of RPC AASHTO Type II girders.

Table 2: Non-proprietary economic UHPC mix designs (kg./m^3)

Mix	Cement	Micro-silica	Fly ash	Fine Sand	C. Aggregate	Water	HRWR
Mix A	630	90	180	1353	0	135	37
Mix B	625	80	80	1455	0	156	21
Mix C	630	90	180	948	403	144	37
Mix D	670	145	145	1353	0	144	43
Mix E	630	90	180	948	403	140	43

The afore-mentioned mixes were produced in 0.015 m^3 using high energy paddle mixers. Two-step mixing procedures were used in producing the UHPC mixes, as follows:

1. Granular constituents were pre-blended. This dry mixing process includes cement, class C fly ash, silica fume, and fine sand. Pre-blending procedure ranges from 2-3 minutes;
2. The pre-defined amount of water and HRWR was added to the blended constituents. Mixing continued till sufficient flowing ability was achieved. This procedure ranged from 12 to 15 minutes.

UHPC cylinders produced for compressive strength testing were cured in two different settings including moisture and thermal curing. The compressive strength test results showed that the use of thermal curing and type III Portland cement is required to achieve the targeted early high strength. Moisture cured specimens displayed a 20% reduction of concrete early compressive strength (24-hour strength). The two curing techniques resulted in equal compressive strength for specimens at age of 14-days, as shown in Figure 4

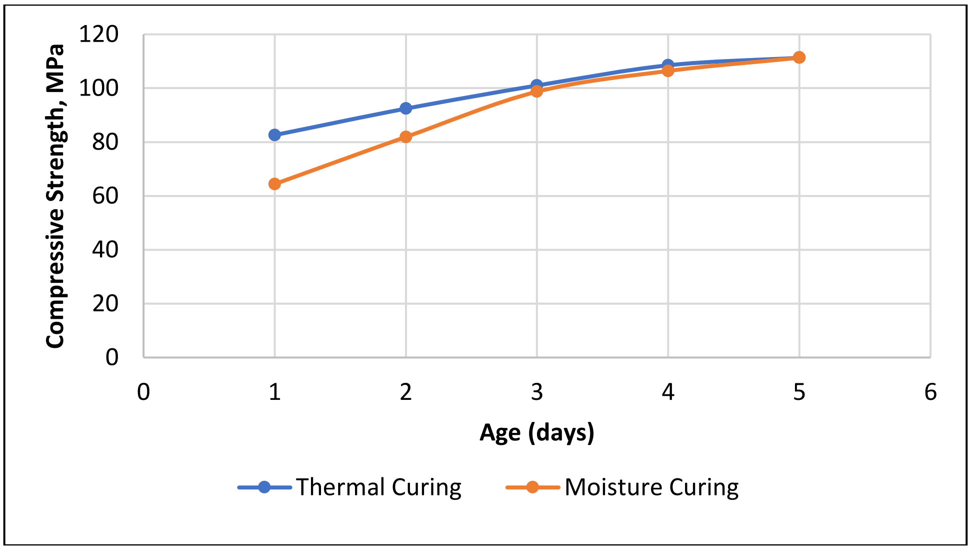

Figure 4: Compressive strength of thermal and moisture cured concrete specimens

The compressive strength testing results for developed non-proprietary UHPC mixes are shown in Figure 5. Given the different mixes compressive strength test results, mix C is selected for fabricating the WWR girders.

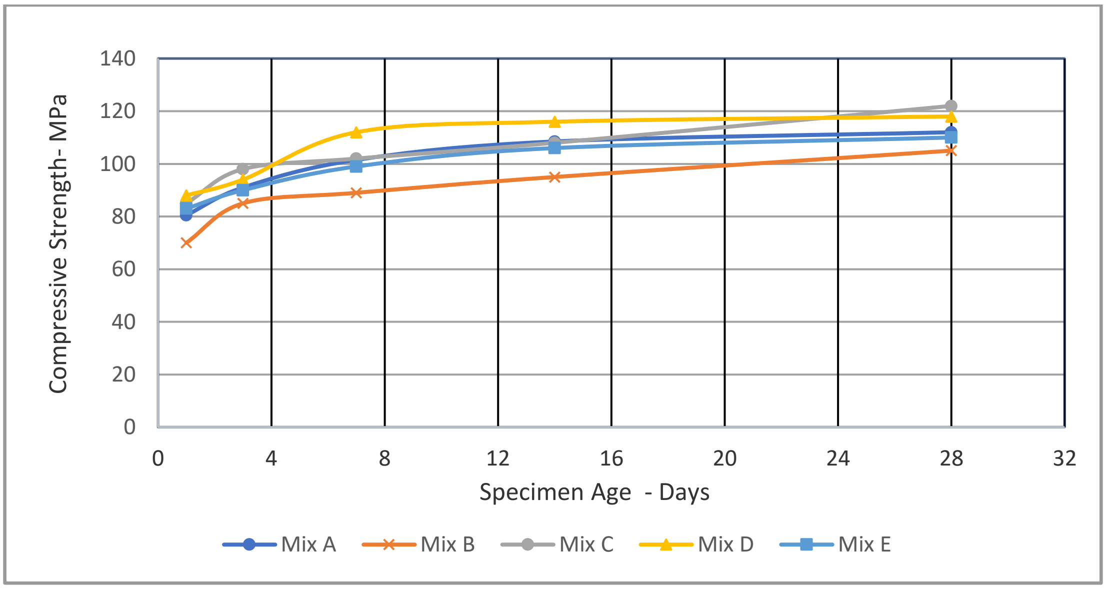

Figure 5: Compressive strength test results of non-proprietary UHPC mixes

4.2. Fabrication of I-Girders using Non-Proprietary RPC Mixes and WWR

Two AASHTO Type II girders are fabricated using the non-proprietary RPC mixes developed. Fabricated girders have a span of 5.65 m. The girders have a shear capacity sufficient to withstand a point load of 3340 kN acting at a distance of 1.8 m. from girders support. The girders flexure reinforcement included 24 – 15.2 mm low-lax strands of 1860 MPa ultimate capacity. The compression reinforcement included regular steel bars and two partially prestressed strands. Two 4 x 4 – D16 x D16 WWR meshes of 560 MPa capacity (grade 80) are used for shear reinforcement. The cross sections of fabricated girders are shown in Figure 4

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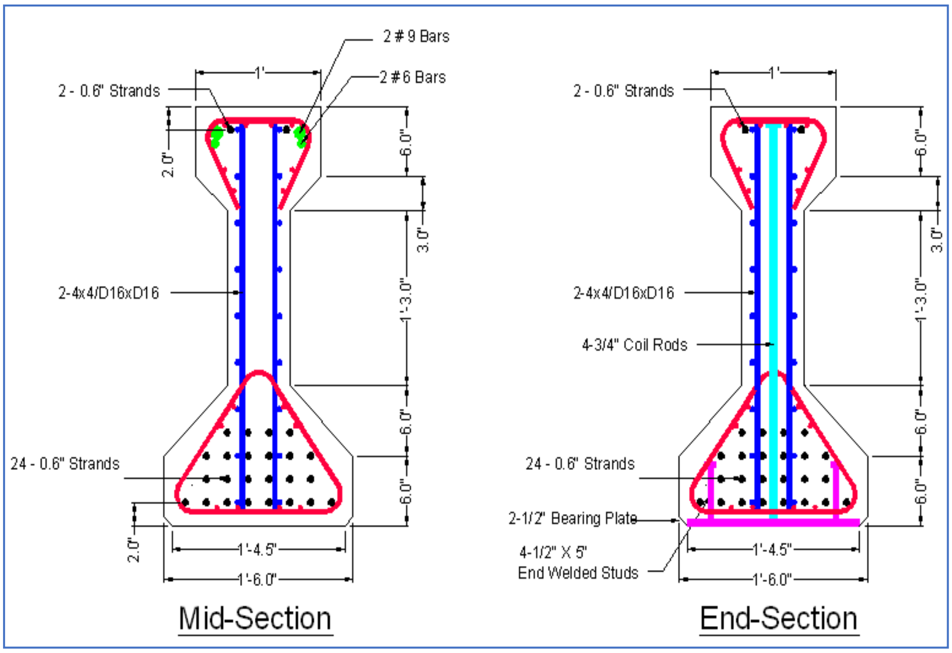

Figure4: AASHTO Type II girders section details

4.3. Shear Testing of Girders

The two AASHTO Type II girders, with span of 5.65 m, were tested in shear using a point load through a similar test setup. Loads were vertically applied to the top flanges through two hydraulic jacks acting symmetrically on a small steel beam. The load point-bearing assembly was a steel plate grouted to the top flange. The girder was supported on roller bearings at 7.5 cm from the girder ends. The girder test setup is shown in **Figure 5**.

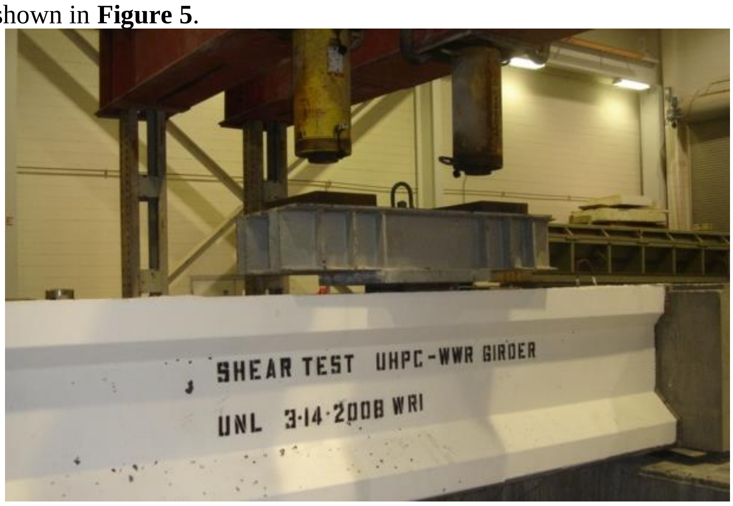

Figure 5: AASHTO Type II shear test setup

The girder was instrumented with linear variable differential transformer (LVDT) at the bottom extreme fibers at a distance of 1.90 m from the girder end. The LVDT location was aligned vertically with the point of load application. The LVDT is used to measure the maximum vertical deflection under shear loading. Surface-bonded electrical resistance strain gages are attached to the girder at various locations to capture the girder strain profile. Strain gages instrumentation was located along three vertical instrumentation lines along the shear span, as shown in Figure 6.

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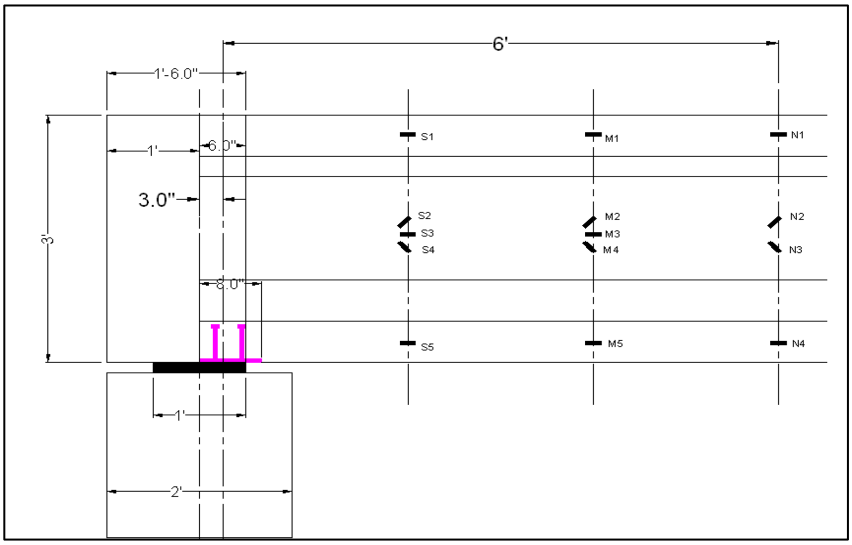

Figure 6. Strain gage instrumentation profile

The two girders were loaded until their ultimate capacity was reached and failure was achieved. Failure was predefined to be concrete crushing or development of wider shear cracks initiated at the bottom flange; and accompanied by strand slippage. The first shear test was completed on girder A. The girder shear span was 1.80 m, resulting in a shear span-to-depth ratio of 2.0. The peak load deflection was 24 mm. The load deflection response of the girder shows that the elastic (linear) behaviour of the girder was altered at a load of 2130 kN. Despite of the girder softening behaviour, additional load-carrying capacity was displayed. The reserve shear capacity was due, in part, to the WWR, used as shear reinforcement. The WWR improvement to the cracking pattern resulted in a better post-cracking performance of the web concrete. The girder shear capacity, at a peak load of 3310 kN, was 2207 kN. Similarly, girder B was tested in shear. Girder B displayed a significant reserve after softening behaviour was achieved. Girder B linear load-deflection relation was altered at a load of 1243 kN. The girder failed in shear at an ultimate load of 2880 kN. The girder shear capacity at the peak load was 1920 kN. Strain gage readings were not used for the validation of softening behaviour or ultimate capacity of the tested girders due to the spalling of concrete at lower loading levels, which resulting in strain gages damage. The load-deflection response of the two girders is shown in Figure 7.

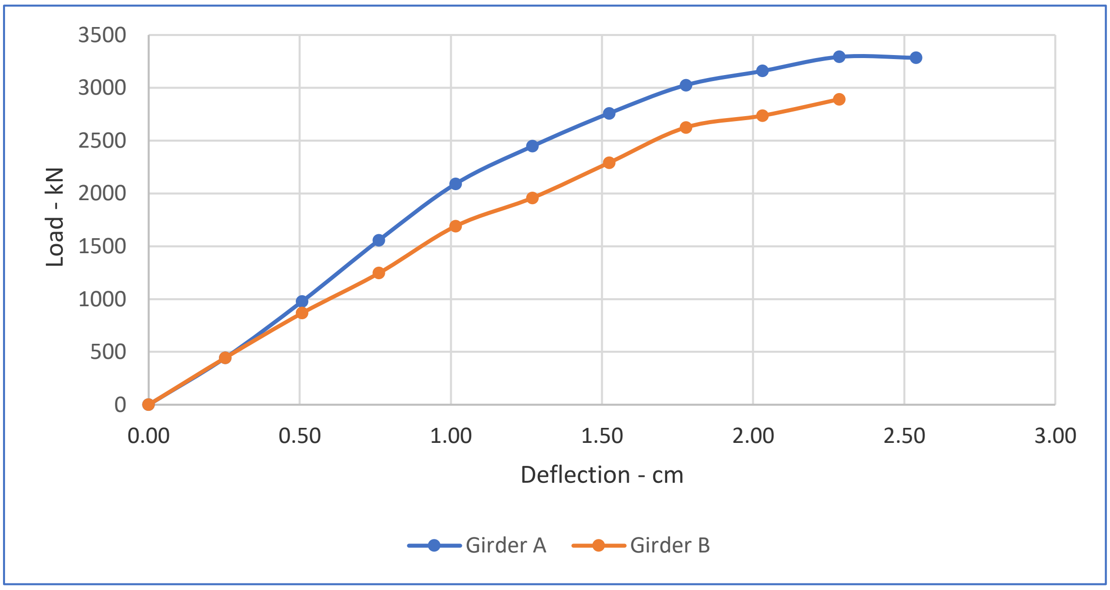

Figure 7. Load-deflection curve (girders A & B)

5. RESEARCH ANALYTICAL PHASE

According to AASHTO LRFD design equations, shear capacity of the precast/prestressed I-girder fabricated using WWR mesh is calculated as:

$$V_n = V_c + V_s + V_p \tag{1}$$

The girder critical shear section is directly below the acting point load, at a distance of 1.83 m from the end support centerline. At this section, a shear force equal to two thirds of the acting load, in addition to maximum flexural moments are applied. Due to the absence of harped strands, the prestressing strands are not contributing to the girder shear capacity. The concrete and orthogonal WWR contributions are calculated as follows:

5.1. Concrete Contribution to Shear Capacity, V_c

According to AASHTO LRFD (5.8.3.3-3), the concrete contribution to the girder shear capacity is calculated as:

$$V_c = 0.083\beta\sqrt{f_c'}b_v d_v \qquad (2)$$

In order to calculate the value of β, the quantities $\frac{V}{f_c'}$, ε_x are calculated as follows:

$$\frac{V_u}{f_c'} = \frac{V_u - \phi V_p}{\phi b_v d_v f_c'} \qquad (3)$$

$$\varepsilon_x = \frac{\frac{M_u}{d_v} + 0.5N_u + (V_u - V_p) - A_{ps}f_{ps}}{2(E_s A_s + E_p A_{ps})} \leq 0.001 \qquad (4)$$

For concrete compressive strength of 105 MPa, width of web (b_v) = 150 mm, section shear depth (d_v) = 720 mm, S = 305 mm, A_{ps} = 3359 mm^2, M_u = 4073 x 106 N.mm, f_c' = 105 MPa, and WWR f_y = 560 MPa

$$\frac{V}{f_c'} = \frac{2063000}{1.0\ x\ 150\ x\ 720\ x\ 105} = 0.182$$

$$\varepsilon_x = \frac{\frac{4073E6}{720} - 3359X0.7X1861}{2\ x\ 196500x3359} = 0.001$$

From AASHTO LRFD Table 5.8.3.4.2-1, values of β and θ are 1.79 and 36.1 respectively. The concrete contribution to shear capacity is calculated using equation (2) as follows:

$$V_c = 0.083\beta\sqrt{f_c'}b_v d_v = 0.083\ x\ 1.79\ x\ \sqrt{105}x150x720 = 164418\ Newton = 164\text{ kN}$$

5.2. Orthogonal WWR Contribution to Shear Capacity, V_s

According to AASHTO LRFD (5.8.3.3-4), the orthogonal WWR contribution to shear capacity is calculated as:

$$V_s = \frac{A_v f_y d_v(\cot\theta + \cot\alpha)\sin\alpha}{s} \qquad (5)$$

For 2 orthogonal WWR meshes of 4 x 4 – D16 x D16, A_v = 1239 mm^2, the ultimate shear capacity of WWR is calculated as follows:

For vertical WWR:

$$V_s = \frac{A_v f_y d_v(\cot\theta)}{s} = \frac{619\ x\ 560\ x\ 720\ x\ \cot 36.1}{305} = 1122493\text{ Newtons} = 1122\text{ kN}$$

For horizontal WWR:

$$V_s = \frac{A_V f_y d_v \cos\alpha}{s} = \frac{619\ x\ 560\ x\ 720}{305} = 818297\text{ Newtons} = 818\text{ kN}$$

Based on Equation (1), the ultimate shear capacity of tested AASHTO girders is:

$$V_n = V_c + V_s + V_p = 164 + 1122 + 818 = 2104\ kN$$

6. DISCUSSIONS AND RESEARCH FINDINGS

This research project investigated the possibility of fabricated economic I-girders with superior characteristics using non-proprietary RCP mixes and orthogonal WWR meshes for shear reinforcement. Non-proprietary RPC mixes were developed using type III portland cement, micro-silica, and fly ash as a preblended cementitious powder. A water-powder ratio less than 0.2 was used in mix development. A self-consolidating flowing ability was attained using high-range water reducers. Concrete mixes developed had a final cost less than $500 per cubic meter, compared to a proprietary RPC mix cost greater than $2000 per cubic meter.

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Orthogonal WWR meshes were used for girder shear reinforcement instead of incorporating random steel fibers. The elimination of random steel fibers resulted in expedited mixing of concrete, and a significant material cost reduction of the produced mixes. The shear capacity and material cost of produced girders are compared with similar girders fabricated using proprietary mixes and tested at FHWA Turner Fairbank Highway Research Center [23]. Results comparison are showed in Table 3

Table 3. Non-proprietary RPC girders vs. proprietary girders test results & material cost estimate

Girder	Material	Span (m.)	Shear Capacity (kN)
FHWA - 28S	Proprietary RPC & Steel Fibers	28	1708
FHWA - 24S	Proprietary RPC & Steel Fibers	24	2237
FHWA - 14S	Proprietary RPC & Steel Fibers	14	1948
Cost per m^3	**$2300**	**Average**	**1964**
WWR Girder A	RPC & WWR	18	2215
WWR Girder B	RPC & WWR	18	1921
Cost per m^3	**$780**	**Average**	**2068**

The research findings show that non-proprietary RPC mixes with superior mechanical characteristics can be produced using local materials at a fraction of the cost of comparable proprietary mixes. The absence of random steel fibers can be compensated by orthogonal WWR for improved girder shear capacity. Attained girder characteristics and significant cost saving will result in increased RPC and WWR meshes application in the bridge/heavy construction industry. Future research is required to investigate the reliability of non-proprietary RPC mixes [24], and the possibility of incorporating self-cleaning agents in project construction to reduce environmental pollution [25]. Finally, building information modelling and 3-D printing technologies should be incorporated in project construction [26,27,28]

Notation

E_c = static modulus of elasticity of concrete
w_c = specific weight of concrete
k_1 = correction factor for source of aggregate to be taken as 1 unless determined by physical test, and as approved by the authority of jurisdiction
f_c' = concrete compressive strength
f_t = concrete split tensile strength
f_r = concrete modulus of rupture
d_v = effective shear depth
V_n = nominal shear resistance of section considered
V_c = nominal shear resistance provided by tensile stresses in concrete
V_s = shear resistance provided by shear reinforcement
V_p = component in the direction of the applied shear of the effective prestressing force
β = factor relating effect of longitudinal strain on the shear capacity of concrete
b_v = width of web
ε_x = longitudinal strain in the web reinforcement on the flexural tension side of the member
A_{ps} = area of prestressing steel on the flexural tension side of the member
N_u = normal force acting on the section
F_{po} = parameter taken as modulus of elasticity of strands multiplied by locked-in differencein strain between the prestressing strands and concrete, for regular prestress levels = $0.7f_{pu}$
N_u = factored axial force, taken as positive for tension and negative for compression, kips
M_u = factored moment, not to be taken less than V_uD_v
V_u = factored shear force
Φ = Strength reduction factor, for shear = 0.9
f_y = yield strength of shear reinforcement, ksi

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2.2. Conflicts of Interest

The authors declare no conflict of interest.

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