Evaluation of Corrosion-Resistant Basalt-Fiber-Reinforced Polymer Bars and Carbon-Fiber-Reinforced Polymer Grid Reinforcement to Replace Steel in Precast Concrete Underground Utility Vaults

“Evaluation of Corrosion-Resistant Basalt-Fiber-Reinforced Polymer Bars and Carbon-Fiber-Reinforced Polymer Grid Reinforcement to Replace Steel in Precast Concrete Underground Utility Vaults” by P. Archbold and G. Tharmarajah, in the September–October 2016 issue of *PCI Journal*,[1] is concerned with a potentially important topic, but it leaves far too much to one’s imagination in the description of the materials. The first problem is that the description of the basalt-fiber-reinforced polymer (BFRP) material gives the failure *stress* and modulus of elasticity. The description of the carbon-fiber-reinforced polymer (CFRP) material gives the failure *force* of one strand and the modulus of elasticity. Neither the areas nor the fracture strains are stated, while both seem to be needed.

I tried to work backward from the reinforcement ratios in Table 3 to find some of this information, but instead found another problem. The steel-reinforced slab has a stated reinforcement ratio ρ of 0.673%, but I cannot check this value. If a 12 mm (0.47 in.) steel bar has an area of 113 mm² (0.175 in.²), I get a steel reinforcement ratio that is much greater than the stated value.

$$\frac{A}{bd} = \frac{3(113)}{350(60)} = 0.0164$$

where

A = area of the reinforcement

b = width of concrete section

d = effective depth of the reinforcement in the cross section

The stated steel reinforcement ratio leads to an area of 141 mm² (0.219 in.²), leading to three bars with diameters d_b of 7.74 mm (0.30 in.). The measured load capacity leads to moments compatible with a slab containing 339 mm² (0.525 in.²) of tension steel acting at the stated yield stress, so it appears that the 0.673 is a typo, picked up from the next line. The reinforcement ratio of 0.673% for the BFRP bars leads to an area of 141 mm². The breaking force, area × failure stress, is compatible with the failure force listed in Table 2. However, apparently this is the fiber area as opposed to the gross area of the bars. A clear statement about this question is needed.

Table 3. Provided and balanced reinforcement ratio			
Test panels	**ρ, %**	**ρ_b, %**	**Failure mode**
Steel reinforced	0.673	3.186	Yielding of steel
BFRP reinforced	0.673	0.524	Concrete crushing
CFRP-grid reinforced	0.322*	3.620	Carbon-fiber-grid rupture
Unreinforced	0.000	0.000	Tensile/concrete rupture

Note: A = area of the reinforcement; b = width of concrete section; BFRP = basalt-fiber-reinforced polymer; CFRP = carbon-fiber-reinforced polymer; d = effective depth of the reinforcement in the cross section; ρ = reinforcement ratio = $\frac{A}{bd}$; ρ_b = balanced reinforcement ratio.

* Approximate.

The stated balanced reinforcement ratio, which is not defined, suggests that the failure strain of the BFRP is about 0.012, assuming that the balanced reinforcement ratio is that leading to crushing of the concrete at the same instant that the reinforcement fractures. This requires specific values of both the concrete and reinforcement strains. However, one cannot make such checks for the CFRP case because the failure stress and elongation are not given. The CFRP area is apparently about 67.6 mm^2 (0.105 in.^{2}) based on the reinforcement ratio. The stated balanced reinforcement ratio does not seem compatible with that for the BFRP material, but there is not enough information to allow a check. The reinforcement strands are at 75 mm (3.0 in.) spacing, suggesting five strands per layer for a total of 20 strands. If this is correct, the failure tensile force is 20 × 4.2 = 84 kN. This force is more or less compatible with the reported failure force. With this information, the failure stress is 84 kN/67.6 mm^2 = 1.242 kN/mm^2 = 1242 MPa (180.1 ksi). Dividing the stress by the modulus of elasticity gives failure strain (1242/235,400 = 0.00528), which seems quite low. A rather similar CFRP grid described in Seliem et al.[2] had a reported failure strain of about 0.01. The failure stress and strain and reinforcement area need confirmation.

If the CFRP area and modulus are correct, they can be used in a cracked-section elastic analysis to evaluate the conditions at the unfactored load. The stated service load, 17 kN (3.8 kip), leads to a service moment of 2.55 kN-m (1.88 kip-ft) plus the rather small dead load. In an elastic analysis, this leads to a reinforcement stress of about 655 MPa (95.0 ksi) and a strain of about 0.0028, more than twice that expected at service load in a member with steel reinforcement. Large cracks are to be expected.

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1. Archbold, Paul, and Gobithas Tharmarajah. 2016. "Evaluation of Corrosion-Resistant Basalt-Fiber-Reinforced Polymer Bars and Carbon-Fiber-Reinforced Polymer Grid Reinforcement to Replace Steel in Precast Concrete Underground Utility Vaults." *PCI Journal* 61 (5): 69–76.
2. Seliem, Hatem M., Lining Ding, William Potter, and Sami Rizkalla. 2016. "Use of Carbon-Fiber-Reinforced Polymer Grid for Precast Concrete Piles." *PCI Journal* 61 (5): 37–48.

Authors' response

Basalt-fiber-reinforced polymer (BFRP) reinforcement used in the panel tests was in rod form, and the carbon-fiber-reinforced polymer (CFRP) was in grid form.[1] Hence, the rupture stress and modulus of elasticity of BFRP bars and strength of a single strand were provided in the paper. The panels used 12 mm (0.47 in.) bars, so the strength given for BFRP was based on 12 mm BFRP bars. The area of each strand varied slightly for CFRP grid. Therefore, the strength of a single strand was given for CFRP. Since BFRP bars and carbon-fiber-grid reinforcement were used to replace steel in precast concrete panels, the main aim of providing rupture stress and modulus of elasticity was so that similar material properties of steel could be compared. Although fracture strain is an important parameter, the modulus of elasticity and rupture stress influence the service-load-level behavior (such as deflection) and ultimate load behavior (such as failure mode) of precast concrete panels. Therefore, the comparison was limited to rupture strength and modulus of elasticity.

As stated in the Test Specimens section of the paper, the panels tested were representative samples from actual underground vaults available on the market. In the commercial panels, the reinforcement percentage was calculated using 1000 mm (39 in.) wide by 100 mm (4 in.) thick specimens, which had six bars spaced at 160 mm (6.3 in.). A similar arrangement was adopted in reported panels as well. Due to limitations in the test arrangements, the cover concrete was reduced to 15 mm (0.59 in.) on either side, assuming no structural deficiency due to the provision of additional side cover on either side of the panels. Therefore, the reinforcement percentage was calculated as a ratio between the steel reinforcement area of six bars (6 × 112.2 = 673) divided by total area of the 1000 mm long panel (1000 × 100 mm^2). A similar calculation was made for BFRP bars as well. The ratio provided by the precast concrete manufacturer was also a similar value. Hence, the 0.673% was derived from the assumption of 673 mm^2 (1.04 in.^{2}) reinforcement over a length of 1000 mm.

Balanced reinforcement ratio is the amount of reinforcement in a concrete cross section that leads to simultaneous concrete crushing and fiber-reinforced polymer (FRP) rupture failure. The calculation of balanced reinforcement ratio was made according to Eq. (8.3) of the ACI 440.1R guideline, which is applicable for FRP reinforcement.

$$\rho_{fb} = 0.85\beta_1 \frac{f_c'}{f_{fu}} \frac{E_f \varepsilon_{cu}}{E_f \varepsilon_{cu} + f_{fu}}$$

where

f_c' = compressive strength of concrete = 57.1 MPa

f_{fu} = design tensile strength of FRP = (mean tensile strength – 3 standard deviations) (environmental factor [1.0 or 0.9 for carbon])

E_f = modulus of elasticity of FRP = 54,000 MPa

ε_{cu} = ultimate strain of concrete = 0.003

The determined balanced reinforcement ratio using equations from *Eurocode 2*[3] also showed similar results.

If a 1000 mm (39 in.) panel is considered, it requires 14 strands to reinforce one layer. Hence, failure stress = 14 × 4.2 × 4 × 1000/322 = 730.43 N/mm^2 (105.94 ksi). This translates to a strain value of 0.003 = 730.43/235,000. This is a similar strain value of concrete assumed in this analysis with an expectation of simultaneous concrete crushing and FRP rupture. As mentioned in the paper, although the CFRP-reinforced panels were designed using a balanced section, the failure load was slightly higher than the predicted failure load.

The authors of the paper also agree with the comment mentioned in the discussion regarding the crack width because CFRP-reinforced panels showed larger crack widths of more than 1 mm (.0394 in.), where 0.5 mm (0.02 in.) crack widths were accepted for FRP-reinforced sections, considering the corrosion-resistant nature of these materials. Hence, larger crack widths were expected and the experimental investigation showed that based on service load requirements, CFRP-reinforced panels cannot be used with the described amount of reinforcement. Also, further studies using higher amounts of reinforcement can help to understand the behavior of such panels.

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1. Archbold, Paul, and Gobithas Tharmarajah. 2016. "Evaluation of Corrosion-Resistant Basalt-Fiber-Reinforced Polymer Bars and Carbon-Fiber-Reinforced Polymer Grid Reinforcement to Replace Steel in Precast Concrete Underground Utility Vaults." *PCI Journal* 61 (5): 69–76.
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Use of Carbon-Fiber-Reinforced Polymer Grid for Precast Concrete Piles

There is a glaring error in a metric conversion of the dimension of the strand size on page 38 of "Use of Carbon-Fiber-Reinforced Polymer Grid for Precast Concrete Piles" in the September–October 2016 issue of *PCI Journal*.[1] The width of 7.544 mm, which has too many significant figures, is not 4.016 in. but rather should be 0.297 in. if 7.544 mm is correct.

There is a more serious problem, not identified or discussed. The failure force of specimen CN is given as 2755 kN (619.4 kip). This force, divided by the gross area, gives a failure stress of 21.7 MPa (3150 psi), while the reported concrete strength was approximately 38 MPa (5500 psi). This makes the comparisons of the benefits of the various spirals pointless. The Fig. 9 strains clearly show that there was something wrong with specimen CN compared with the other specimens.

The gross area multiplied by the stated concrete strength is a force of 4816 kN (1083 kip). No specimen reached this force. The normal concrete column strength equation uses 0.85 f_c' rather than the full concrete compressive strength f_c', leading to 4094 kN (920.4 kip), but it is not clear whether this reduction should apply to a member that is this short, especially with the confining steel collars at the ends. The reinforced specimens all had longitudinal reinforcement, but the quantities are so small that the contributions to strength can be neglected. In the two steel-reinforced cases, the steel can contribute about 120 kN (27 kip), and the CFRP cases would appear to contribute considerably less. The behavior of the CFRP material in compression is not discussed.

Most equations for the required spiral steel quantity involve the term A_g/A_{core} (where A_g is gross area and A_{core} is core area). See ACI 543R-121 section 5.5[2] or chapter 20 of the *PCI Bridge Design Manual*,[3] for example. For the two specimens with steel spirals, this ratio is 3.05. This is an almost insurmountable value, and supplying a spiral that can lead to a failure load larger than that of a member without a spiral will be very difficult, if not completely unbuildable.

The specimens reinforced with CFRP spirals present other questions. ACI 318-14[4] has a requirement for a minimum amount of circular spiral reinforcement—Eq. (25.7.3.3)—stated as a steel ratio with slightly altered notation.

$$\rho_s = \left(\frac{A_g}{A_{core}} - 1\right)\frac{f_c'}{f_{sp}}0.45 \qquad \text{(ACI 25.7.3.3)}$$

where

ρ_s = spiral steel ratio = minimum volume of confinement reinforcement/volume of confined concrete

f_{sp} = usable strength of the spiral, usually taken as the yield stress

The spiral steel ratio ρ_s for a known spiral is computed as

$$\rho_s = \frac{4A_{sp}}{D_{core}s}$$

where

A_{sp} = area of steel spiral

D_{core} = diameter of confined concrete core measured to outside of spiral

s = pitch

The strain in steel spirals at column failure was extensively studied and reported in the Richart et al. papers.[5–7] The strains varied considerably, generally in the range of 0.004 to 0.009. None apparently reached 0.01. Many different types of steel were used, with a wide range of strengths.

A recommendation made by Richart et al. was stress in spiral steel f_{sp} when strain is 0.005.

The intent of ACI Eq. (25.7.3.3) is that the strength of the core of the column after the concrete outside of the spiral has spalled off should be slightly larger than the strength of the original intact column, ignoring the spiral, and that there should be some ductility that does not exist in a tied column.

Applying ACI Eq. (25.7.3.3) to a member with a CFRP spiral introduces questions about the value of f_{sp} because the equation was derived with steel spirals with various degrees of ductility. A straightforward application of ACI Eq. (25.7.3.3) with a core diameter of 318 mm (12.5 mm) and a strength of 640 MPa (93 ksi) leads to a required minimum value ρ_s of 0.0159. With the spiral area of 7.933 mm^2 (0.01230 in.^{2}) (the single-layer case) and a spacing of 44.55 mm (1.754 in.), the supplied ρ_s equals 0.00224. Thus, there can be no expectation that the CFRP spiral adds to the strength of the member, nor can using two layers be expected to add strength.

The two bending tests on pile sections are of interest, but they would have been more convincing if the pretensioned reinforcement had been the same. Also, the description of the pile with the CFRP spiral is not consistent between the text and Fig. 12. The figure shows 12.7 mm (0.5 in.) special strands, while the text says that they were 13.2 mm (0.520 in.) in diameter, so the area is not defined but might be about 0.164 in.^{2} (106 mm^2) if 13.2 mm is correct. The compression strain shown in Fig. 16 for the pile with the CFRP grid ends at a bit less than 0.003 strain, which is about what would have been expected in any flexural test, and shows no benefit of the spiral on strain. What does the curve for the control specimen look like? Were strand specimens tested to determine the actual stress-strain properties and failure stress? The predicted moment capacities listed in Table 5 are substantially smaller than the value I computed for the control specimen using a standard Grade 270 (1860 MPa) stress-strain curve for the steel. My value is 4% smaller than the reported test moment, not 21%. In addition, the asterisks in Table 5 should have been located one row higher.

I would attribute the slightly higher moment capacity of the pile with the CFRP grid to the 10% higher compressive strength rather than to the CFRP grid. Its contribution of the force in the compression zone must be small. The moment capacities reported in kip-ft units must actually be kip-in.

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