New Generation of Precast Concrete Double Tees Reinforced by Carbon-Fiber-Reinforced Polymer Grid

I read with interest William Gamble's comments in the Discussion section of the January–February 2016 issue of *PCI Journal* regarding the paper "New Generation of Precast Concrete Double Tees Reinforced by Carbon-Fiber-Reinforced Polymer Grid"[1] by D. Lunn, G. Lucier, S. Rizkilla, N. Cleland, and H. Gleich, which appeared in the July–August 2015 issue of *PCI Journal*. In my opinion, Gamble's comments accurately reflect the intent of ACI 318-14[2] with respect to minimum flexural reinforcement, while the authors' responses fall short of addressing those concerns. As stated by Gamble, the primary purpose of minimum flexural reinforcement is to provide sufficient ductility so that the member does not fail at first cracking. Because all of the specimen flanges failed by rupture of the carbon-fiber-reinforced polymer (CFRP) grid immediately following cracking of the concrete, *their behavior would have been no different had they contained no reinforcement at all*. If CFRP grid is proposed to be implemented into the building code, the performance of a member reinforced with that material should match the intended behavior of the member with code-specified steel reinforcement.

With respect to the author's responses:

1. The authors claim that because the flange is not prestressed in the transverse direction, the $\phi M_n \geq 1.2M_{cr}$ provision of ACI 318-14 does not apply. Although this is technically true, the source of this requirement does not derive from the fact that the member is prestressed but from the type of steel reinforcement and how its properties are used when calculating the nominal flexural strength of the member. For nonprestressed members with Grade 60 (414 MPa) reinforcement, the yield strength of 60 ksi is used in flexural strength calculations, but the ASTM-specified minimum tensile strength is 1.5 times the yield strength (90 ksi [620 MPa]). This is the "unspoken" strain hardening that Gamble refers to. Although it is not obvious from the minimum flexural reinforcement equations in 9.6.1.2 of ACI 318-14, the provisions for nonprestressed beams are in fact derived from $\phi M_n \geq 1.0M_{cr}$ (Seguirant et al.[3]). Assuming the member fails by fracture of the reinforcement (full strain hardening), the margin between cracking and failure is $f_{su}/(f_y)\phi = 90/(60)(0.9) = 1.67$. For prestressed members, the stress in the reinforcement at nominal flexural strength f_{ps} can be as high as 270 ksi (1860 MPa), which is the ASTM-specified minimum tensile strength of the steel. No strain hardening is available, so assuming that the member fails by steel fracture, the margin between cracking and failure is $1.2/0.9 = 1.33$. It is clear that, like prestressed reinforcement, the CFRP grid material exhibits no strain hardening beyond the stress used to calculate the nominal flexural strength. Consequently, to ensure ductile behavior of flexural members reinforced with CFRP grid, it is entirely appropriate to apply provisions similar to those applicable to prestressed concrete. Remember that ACI 318-14 is written for steel reinforcement only and does not contemplate the use of nonprestressed reinforcement with no strain hardening capability. That is why the $\phi M_n \geq 1.2M_{cr}$ provision applies only to prestressed concrete in ACI 318-14.
2. The authors state that the minimum flexural reinforcement in the flange should be analogous to the requirements for one-way slabs in ACI 318-14, which is equivalent to the requirement for temperature and shrinkage reinforcement. I question this assumption. Like many provisions in ACI 318-14, the minimum flexural reinforcement requirements for slabs go back many years and were generally formulated for monolithic cast-in-place concrete; in this case, cast-in-place concrete one-way slabs. As noted in item 1, minimum flexural reinforcement requirements for nonprestressed beams and all pre-

stressed flexural members are based on some margin of safety between cracking and failure. Temperature and shrinkage reinforcement will generally not satisfy these margins, but as stated in the commentary to ACI 318-71,[4] "the minimum reinforcement required for slabs is a little less than that required for beams, since an overload would be distributed laterally and a sudden failure would be less likely." While this is probably true of monolithically cast concrete slabs, it is questionable whether it should also apply to statically determinate members, such as cantilever flanges. Based on the poor performance of these flanges in the testing program, my opinion is that it should not.

3. The authors say that because the CFRP grid reinforcement provides a tensile force of 5.38 kip/ft (78.5 kN/m) while the required Grade 60 (414 MPa) reinforcement provides only 4.54 kip/ft (66.3 kN/m), the grid with the 2.7 in. (69 mm) spacing meets the "analogous code requirement for needed reinforcement." This is an apples-to-oranges comparison. Again, the steel force is based on the yield strength while the CFRP force is based on the tensile strength. If the steel force were amplified by 1.5 to account for strain hardening, the resulting 6.81 kip/ft (99.4 kN/m) exceeds CFRP's 5.38 kip/ft by more than 25%.
4. The authors state that the recommended strength reduction factor of 0.75 was selected to be analogous to other brittle modes of failure, such as shear. Shear failures are naturally brittle, giving little or no warning before failure, but the code requires flexural behavior to provide sufficient warning (cracking and deflections) of impending failure. The point here is that flexural failures will not be brittle if sufficient reinforcement is specified to provide a reasonable margin between first cracking and failure. Plain concrete is defined in ACI 318-14 as concrete members with less than the minimum amount of reinforcement required by the code. If the flange fails at first cracking, it does not meet the minimum amount of flexural reinforcement required by the code, and by definition is plain concrete. As Professor Gamble points out, cantilever flanges do not fall within the permitted uses of plain concrete in ACI 318-14.

All of the discussion above centers on the ductility requirements for minimum flexural reinforcement. The code also permits alternative "over-strength" provisions where the design strength must be greater than or equal to the required strength times an amplification factor: $\phi M_n \geq 1.33 M_u$ for nonprestressed members and $\phi M_n \geq 2.0 M_u$ for prestressed members. Again, the larger multiplier for prestressed members reflects the lack of strain hardening available beyond the stress used to calculate M_n. According to the commentary of ACI 318-71, these provisions are intended to provide sufficient extra reinforcement for safety of large members where the ductility requirements would be excessive. In my opinion, a double-tee flange does not qualify as a large member and should not require an excessive amount of flexural reinforcement to result in ductile behavior.

Based on the discussion above, my opinion is that double-tee flanges reinforced with CFRP grid should be designed in accordance with provisions similar to those used for prestressed concrete, namely $\phi M_n \geq 1.2 M_{cr}$ or $\phi M_n \geq 2.0 M_u$. As long as a brittle mode of failure in flexure is prevented by the selection of an appropriate quantity of CFRP grid, I do not see why the typical ϕ factor of 0.9 for flexure should not be used. However, according to the results of the tests,[1] the term M_{cr} should be reevaluated for double-tee flanges. Figure 8 shows that the measured modulus of rupture of the double-tee flanges ranged from approximately $7\sqrt{f_c'}$ to $10\sqrt{f_c'}$, with a median value of approximately $9\sqrt{f_c'}$. The modulus of rupture of members with "normal" proportions and exposure is typically assumed to be $7.5\sqrt{f_c'}$. The higher modulus of rupture for double-tee flanges is not unexpected because they are shallower and less subject to shrinkage stresses than "normal" concrete members, both of which tend to increase the modulus of rupture of the concrete.[3] In order to specify a quantity of CFRP grid reinforcement sufficient to provide a reasonable margin between cracking and failure, I suggest calculating M_{cr} with an assumed modulus of rupture of $9\sqrt{f_c'}$.

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1. Lunn, D., G. Lucier, S. Rizkilla, N. Cleland, and H. Gleich. 2015. "New Generation of Precast Concrete Double Tees Reinforced by Carbon-Fiber-Reinforced Polymer Grid." *PCI Journal* 60 (4): 37–48.
2. ACI (American Concrete Institute) Committee 318. 2014. *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)*. Farmington Hills, MI: ACI.
3. Seguirant, S., R. Brice, and B. Khaleghi. 2010. "Making Sense of Minimum Flexural Reinforcement Requirements for Reinforced Concrete Members." *PCI Journal* 55 (3): 64–85.
4. ACI Committee 318. 1971. *Commentary on Building Code Requirements for Reinforced Concrete.* ACI 318-71. Detroit, MI: ACI.

Authors' response

The authors would like to thank the reader for his valuable contribution to this discussion of "New Generation of Precast Concrete Double Tees Reinforced by Carbon-Fiber-Reinforced Polymer Grid."[1] The reader raises an important consideration with respect to the reserve strength of steel reinforcement provided by strain hardening. As noted by the reader, some Grade 60 (414 MPa) mild steel reinforcement can exhibit a 50% difference between the ultimate tensile strength and the yield strength, and there is no doubt that, when present, this overstrength provides an additional margin of safety. The authors would like to add that other types of common steel reinforcement include ASTM A706[2] Grade 60 steel, which specifies a minimum ultimate-to-yield ratio of 1.25, and ASTM A1064[3] welded-wire reinforcement (WWR), which has a specified ultimate-to-yield ratio of just over 1.1. With WWR, there is also no requirement that the material stress-strain curve exhibit a distinct yield point. Currently, ACI 318-14[4] does not differentiate between these types of reinforcement with regard to minimum reinforcement and overstrength requirements. Thus, it is not guaranteed that steel-reinforced flanges will exhibit significant excess capacity above yield.

The fiber-reinforced polymer (FRP) grid reinforcement under discussion is typically used in lieu of WWR. FRP materials are linear elastic, so they have no yield plateau and exhibit no strain hardening behavior. However, for FRP, significant differences exist between the statistically guaranteed tensile strength typically specified for design and the strength actually exhibited by the materials at rupture. The ratio between actual FRP rupture and guaranteed design strength is often of a magnitude similar to the difference between yield and rupture for traditional steel reinforcement. Therefore, it is the authors' opinion that it is unnecessary to modify the minimum reinforcement or overstrength requirements. It should be noted that the lowest ratio of measured failure load to service load for a full-thickness test specimen with a 2.7 in. (69 mm) grid spacing was 3.7. The authors consider this factor of safety to be well within the range of acceptable design practice.

Increasing the amount of FRP flange reinforcement will not necessarily provide a ductile failure mechanism. FRP rupture is always brittle, but ACI 318-14 does not require flexural failures to be ductile. Instead, it penalizes brittle failure mechanisms with lower strength reduction factors, such as for an overreinforced, compression-controlled section in flexure. This brittle failure mechanism is permitted by ACI 318-14, but it is penalized with a low strength reduction factor. As stated in the paper[1] and in the previous discussion, the authors recommend a strength reduction factor of 0.75 for the case of carbon-fiber-reinforced polymer (CFRP) grid rupture rather than the 0.9 used for tension-controlled flexure or the overly conservative 0.55 provided by ACI 440.1R-06[5] for sections controlled by FRP bar rupture. The overly conservative value recommended by ACI 440.1R-06 is intended to prevent global failure in the event of rupture of one FRP bar in a small group of bars. The carbon-fiber-reinforced polymer (CFRP) grid reinforcement is believed to have a more uniform distribution of reinforcement at a tighter spacing than the equivalent FRP bar reinforcement such that a premature failure of a single FRP strand is unlikely to result in a global failure.

When considering the possible failure modes of steel-reinforced flanges compared with FRP-reinforced flanges, it is worth discussing corrosion failure. The assumption that the failure mode for a steel-reinforced flange will remain ductile over time may be optimistic if that steel-reinforced flange is subjected to a corrosive environment (as parking structure flanges can be). After several years of corrosion, the probability of a steel-reinforced flange (particularly a WWR-reinforced flange) remaining ductile and maintaining its original safety factor is less certain. CFRP-reinforced flanges generally maintain their strength in highly corrosive environments, while steel-reinforced flanges generally do not.

In considering designs for brittle versus ductile failures, the reader provides the example of an undesirable brittle shear failure being avoided by ensuring that a ductile flexural failure will govern member behavior. The authors fully agree with the reader that brittle failure modes should be avoided whenever possible in design. A designer can avoid brittle failures with CFRP-reinforced flanges by designing for a ductile global flexural failure to control. It is unlikely that a flanged concrete section would exhibit an ultimate capacity of more than 3.7 times the service load if that section were efficiently designed for global flexure.

With regard to concrete tension capacity, it is the authors' opinion that the widely adopted concrete tensile strength of $7.5\sqrt{f_c'}$ psi ($0.62\sqrt{f_c'}$ MPa) is sufficient for the design of this type of member. The concrete tensile stresses at failure in the experimental program varied from $7\sqrt{f_c'}$ psi ($0.58\sqrt{f_c'}$ MPa) to $10\sqrt{f_c'}$ psi ($0.83\sqrt{f_c'}$ MPa), which is within the range of the generally accepted value.

The authors would again like to thank the reader for his insightful and thoughtful comments and appreciate the chance for discussion.

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Characterization of the Shear Behavior of Z-Shaped Steel Plate Connectors Used in Insulated Concrete Panels

In the March–April 2016 *PCI Journal* article "Characterization of the Shear Behavior of Z-Shaped Steel Plate Connectors Used in Insulated Concrete Panels,"[1] there appears to be an initial study on this Z-shaped metal wythe tie. I do respectfully have to caution the authors on their conclusions based on the small number of test specimens. This limited sampling should be further expanded to better validate the correlations expressed in this article. In addition, the use of any metal wythe ties establishes thermal short circuits, which can significantly reduce the effective *R*-value of the panel. Metal ties also interrupt the continuous insulation of the individual precast concrete panels, potentially resulting in interior cold and condensation spots. Further testing should be done to determine the reduction of the effective *R*-value when using these suggested metal wythe ties.

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Reference

1. Goudarzi, Nabi; Yasser Korany, Samer Adeeb, and Roger Cheng. 2016. "Characterization of the Shear Behavior of Z-Shaped Steel Plate Connectors Used in Insulated Concrete Panels." *PCI Journal* 61 (2): 23–37.

Authors' response

Regarding the discussion on the article titled "Characterization of the Shear Behavior of Z-Shaped Steel Plate Connectors Used in Insulated Concrete Panels,"[1] the authors appreciate the comments made by Pat Hynes. Our paper had reported a preliminary experimental study on the shear behavior of steel Z-shaped connectors used in insulated concrete panels. The paper showed that Z-shaped connectors can reach the plastic shear strength of their material and possess large shear stiffness. Thus, Z-shaped connectors can improve the out-of-plane strength and stiffness of insulated panels up to the strength and stiffness of a fully composite panel. This optimizes the structural design of insulated panels. We also would like to note that our study has been complemented by further numerical and analytical investigations, the results of which are under preparation for publication in the near future.

Nonetheless, other factors should also be considered, including cost and availability of interlayer connectors and the nonstructural performance of the insulated panels. Any interlayer connector system creates some level of thermal bridging that cannot be avoided. There is a recognized trade-off between enhancing structural efficiency and minimizing thermal bridging. More research is needed to use these panels in situations where the *R*-value governs the design. In projects where tall panels need to be installed and the *R*-value is not a governing design concern, the Z-shaped connectors can be used to maximize the out-of-plane strength and stiffness of insulated concrete panels.

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