Discussion

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Hollow-core slabs with cast-in-place concrete toppings: A study of interfacial shear strength

The following comments relate to "Hollow-Core Slabs with Cast-in-Place Concrete Toppings: A Study of Interfacial Shear Strength" by Ryan M. Mones and Sergio F. Breña from the Summer 2013 issue of *PCI Journal*.

I think that it also needs to be noted that some unstated factors contribute to the conclusions in this paper as they transition into real field practices. These include surface preparation, using the proper concrete mixture, appropriate placement and consolidation techniques, hot- and cold-weather concreting practices, and proper curing methods.

The surface of the field-installed hollow-core unit should be saturated surface dry or drier (we conducted our own tests at Morse Brothers [now Knife River] that validated this fact) at the time of topping installation. I have seen sawdust, hydraulic oil, dirt, construction debris, and so forth not cleaned off just prior to casting the topping. The concrete mixture should conform to ACI specifications and should attain the specified strength.

We have all seen excessive water added to delivered concrete, rendering it substandard. Observation and concrete testing are always a plus to validate construction practices.

The field applied concrete should be properly screeded to maintain the minimum thickness and consolidated to ensure a good precast/ready-mixed concrete interface contact. ACI's hot- and cold-weather concrete practices should be observed during all weather conditions (hot, cold, wet, snowy, and windy). Concrete curing should conform to these hot- and cold-weather practices.

Most contractors and builders understand the principles of providing good, code-conforming structures, and they do just that. It is the odd 1% or 2% that need to be reminded, and maybe watched, to ensure good performance of these hollow-core composite topping systems. Hollow-core systems with composite topping provide great solutions to conform to fire codes and to support loads with superior span-to-depth ratios and can increase construction speed.

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Authors' response

We would like to thank Pat Hynes for his interest in our paper and the comments on construction practices related to concrete toppings. The authors are in agreement with the comments and would like to further emphasize the need for quality control when casting a concrete topping on-site. The references that we discuss in our response can provide additional information on the factors that lead to a high-quality composite bond.

Raths and Hoigard[1] provide many best-practice tips for bonding topping slabs with precast concrete and present observations from cases where debonding has occurred. Research by Djazmati and Pincheira[2] has shown that compacting overlay concrete using a vibrator can have a dramatic effect on the shear strength of the resulting bond. Shin and Lange[3] studied the mechanisms leading to early-age debonding of overlays due to shrinkage and ambient temperature changes.

Quality construction is paramount when placing a topping on precast concrete that does not have steel reinforcement crossing the composite interface. Horizontal shear is transferred through bond between the precast concrete unit and cast-in-place concrete topping. Although our research was performed in a laboratory setting, efforts to clean the surface of the precast concrete specimens were limited to what could be reasonably expected at a construction site. We did not purposely contaminate the surface to simulate the detrimental effects that poor surface conditions might have on topping bond, but we agree that adequate construction practices must be followed prior to casting a topping. Our research shows that typical composite hollow-core slabs will not fail in horizontal shear if the proper design and construction practices are followed.

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Precast concrete, steel-braced, hybrid pipe rack structure

The following comments relate to "Precast Concrete, Steel-Braced, Hybrid Pipe Rack Structure" by Sebastián F. Vaquero, Damián R. Correa, and Sergio F. Wolkomirski from the Fall 2013 issue of *PCI Journal.*

This paper contains a number of erroneous statements. Because the structure discussed in the paper was constructed abroad, it would have been of little consequence to U.S. readers if the reference to the seismic provisions of the American Institute of Steel Construction (AISC) 341-05 had not been made. It gives the false impression that a structure like that is compliant with U.S. codes. The structure presented does not meet a number of the requirements of AISC 341. Following is a short list:

- The braced frames are not special because they lack any special detailing whatsoever.
- The braced frames are not even concentric because the cantilevered portions of the columns, where the primary portion of the seismic load is, cannot be considered small eccentricities.
- The frames cannot be qualified as composite per AISC 341 because the frame beams, which lack an encased structural steel section, are not composite.
- The chevron configuration is a poor choice, especially at the base tier, given the steep brace angle.
- The chevron brace in the upper tier imposes a significant axial and shear demand on the beams, for which they are obviously not designed. The beams are not laterally braced, nor do their end connections provide continuity.
- The sum of the shear strength of the stitches does not appear to exceed the tensile strength of the channel comprising the brace.

- It is mentioned in several instances that seismic forces amplified with an overstrength factor are used. Unfortunately that is not good enough because it is required that the brace connections be capable of developing the expected compressive strength of the brace. (It takes a small lateral drift to cause strain in the braces corresponding to their axial strength.)

Specific code requirements aside, a couple of important seismic considerations are missing: the columns being subjected to a seismic demand in two directions. It is especially troubling that the columns are considered to develop a plastic hinge at the base despite the statement that a strong column–weak beam concept is used. Combined with the plastic hinges of the beams, the resulting mechanism does not appear to be kinematically admissible.

The simple act of inverting the base tier chevron could eliminate so many design issues.

There are also a number of puzzling conclusions:

- "A braced reinforced concrete frame designed using the same force reduction factor as that of a conventional moment frame with moderate ductility would behave adequately during an earthquake." What does that mean? What is this prediction based on? What are *braced concrete frame*, *moderate ductility*, and *adequate behavior*? Phrases with little meaning, if any.
- "General reinforcement detailing requirements are adequate, and there is no need to use special detailing." What makes that frame special if no component of that frame (beams, columns, braces, gusset plates) has special detailing? Again, what does *adequate* mean? Just to remind the authors, special detailing such as additional reinforcement is not required only for strength, it is required for better confinement and ductility.
- "The brace members and their connection can be designed using a procedure similar to that for braces in steel structures." Is this a conclusion, the final point of this paper?

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Authors' response

We would like to thank Alex Mihaylov for his interest in this case study.

Like many other international readers, we appreciate the positive influence of U.S. journals in our profession. In fact, the Argentine codes are mainly based on the U.S. codes (with a slight influence of the New Zealand Codes), of course adapted to our local needs. Furthermore, when some design criteria are not properly addressed in our codes, it is a common local practice to use the U.S. codes as a tie breaker. Due to this factor, we decided to include the seismic provisions of AISC (341-05) as a reference.

We want to start our response clarifying that we did not intend to discuss in detail the requirements of special concentrically braced frames (SCBFs) because the existing literature covering this particular subject is abundant. Although many remarkable professionals have studied this system extensively, guidelines for its use in newly constructed reinforced concrete frames still need to be developed.

According to AISC 341 for composite special concentrically braced frames (C-CBFs), minor eccentricities are permitted if they are accounted for in the design, as we did. In our opinion, the term *minor eccentricities* is referring to the distance between the beam-to-column joint and the beam-to-brace joint. In both directions the cantilevered portion of the column was designed to absorb elastically the resulting flexure, the SCBF was designed to absorb the resulting longitudinal shear, and the special reinforced concrete moment frame was designed to absorb the resulting transverse shear. Both systems were designed to provide confinement and ductility.

Nevertheless, for C-CBF the code clearly states that beams shall be either structural steel or composite structural steel and that the structural steel braces shall meet the requirements for SCBF. It is worth noting that we only comply with the latter requirement.

As Mihaylov already knows, to focus the ductile behavior of the SCBF into the braces, the provisions limit the following:

- member slenderness
- width-to-thickness ratios (compactness)
- compressive strength
- spacing of stitches (interconnection locations of double-channel braces) in the case of built-up members

In addition, the bracing connections should be designed using the required tensile and compressive strength recommended by the code. All of these requirements, and others, were addressed in the design of the steel braces complying with AISC 341.

Despite Mihaylov's unfavorable opinion, the chevron configuration is allowed by the code. As stated in our paper, the steel V brace was found to be the best solution for this particular case because of the following:

- The connection between the steel brace and the frame is located away from the column base plastic hinge region.
- All tolerances were able to be met between the different systems by leaving sockets on the pier.

The axial and shear demands on the longitudinal reinforced concrete beams were taken into consideration in the design. The strong force demand on the beams is one of the main reasons we decided to add a steel section to the beam-to-corbel connection.

According to AISC 341-05, "As a minimum, one set of lateral braces is required at the point of intersection of the V-type bracing, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points." The reinforced concrete structure proved to have sufficient out-of-plane strength and stiffness to ensure stability. For this reason no lateral bracing is necessary.

As stated in our paper, we used an overstrength factor of Ω_o only where reliable inelastic response or energy dissipation could not be provided (beam corbel, column corbel, and steel embed plates connecting the brace frame to the reinforced concrete structure) to focus the inelastic behavior on the steel braces and on the special reinforced concrete moment frame plastic hinges. Subsequently, we carefully detailed them to ensure that the estimated ductility demands could be reliably accommodated.

A beam sidesway mechanism (as shown in Fig. 5 of our paper) occurs as a result of strong column–weak beam design. This kinematic mechanism is perfectly valid, and the ductility demand at the plastic hinges in the beams and at the column bases is moderate for this mechanism and can easily be provided in design. We recommend Park and Paulay's books as references on the subject.

Regarding our conclusions, we should make the following comments:

- The authors agree with Mihaylov that additional tests and guidelines still need to be developed to fully comprehend this type of hybrid structure.
- When we say that "General reinforcement detailing requirements are adequate, and there is no need to use special detailing," we mean that no additional requirements are necessary besides the ones we already discuss in our paper.

Our conclusions were based on the series of tests shown in the references. As many readers already know, to check for an adequate seismic behavior, the following parameters are fundamental:

- the ability of a structure to dissipate the ground motion energy (related with ductility)

- degradation of lateral stiffness
- hysteretic behavior
- lateral drift

Current seismic codes assume that the lateral loading system for newly constructed reinforced concrete structures is either moment-resisting frames, coupled walls, or shear walls. Guidelines for the use of steel braces in newly constructed reinforced concrete frames still need to be developed. The intention of our paper is to make a humble contribution on the subject.

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