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STUDY OF THE COMPOSITE ACTION OF HOLLOWCORE PANELS USED IN GIRDER-SLAB SYSTEM

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Floor construction with precast hollowcore panels produced by Lafarge Precast Edmonton results in a commonly used girder-slab system. Continuity between the elements is ensured by bent rebars and shear studs. Once all these elements are installed, a structural concrete is poured between the reinforced concrete panels and over the entire floor. The extent of composite action between the rigid diaphragm and the steel beams is not known. Therefore, its potential benefit is not taken into account in the current design procedures for the steel structure. The main components of this research project are the following: an experimental program consisting of a series of 6 large-scale shear tests were carried out. The outcome of this research shows that there is a potential for a composite action between a hollowcore plank and a standard hot rolled W shape. It was found that there is enough confinement to develop the steel stud strength when the beam is connected to the precast prestressed concrete panels using a 1/2" shear stud embedded between the planks and under two to three inches of concrete topping.

Keywords: Precast prestressed concrete panel, Floor system, Shear transfer device (STD), Nelson stud, Embedment, Push-out tests.

1 INTRODUCTION

This paper describes the tests performed to study the composite action of precast prestressed hollowcore panels used in girder slab system. The tests were carried out in the structural laboratory at the Université de Sherbrooke in collaboration with Lafarge Precast Edmonton, located in Alberta, Canada. These tests were conducted in order to characterize the shear stud behavior between the hollowcore precast floor panels, and the steel frame to which it connects. These results will be instrumental for a subsequent research project aimed at determining the degree of composite action when the panels are connected onto the steel beams.

Lafarge Precast Edmonton provided 12 - 8" thick standard hollowcore precast floor panels. Consultants SteelSSALG supplied six – 4’ high W12x40 stubs with 1/2" standard shear studs, and six rigid skids used to support the samples during the loading. The technical staff at Université de Sherbrooke assembled the panel sections, installed the concrete forms around the samples, and poured a 5000 psi concrete topping as shown in Figure 1. Three pairs of identical full-scale samples were tested considering different shear stud layout for each set.

2 EXPERIMENTAL PROGRAM

The objective of this program is to evaluate the shear strength of a standard 1/2" stud at 3 different spacing. The six samples are named in a format that represents the number of stud used and by the sample number. The six samples are described as follows: NB1S1 and NB1S2 have one stud per beam flange, NB2S1 and NB2S2 have two studs equally spaced per beam flange, NB3S1 and NB3S2 have three studs equally spaced per beam flange. For each sample, S1 represents the first specimen and S2 the second.

2.1 Description of Tested Specimens

Each specimen consists of two sets of two hollowcore planks of 3' x 4' x 8" mounted on each flange of a W12x40 stub. Each flange supports an identical stud layout (one, two or three 1/2" stud AWS type A - F_u=61 ksi) embedded within a 5000 psi - 2" to 3" thick concrete topping poured between and over the 3' x 4' hollowcore panels. The concrete topping was poured in two sequences; the first pour for samples NB1S1, NB1S2, NB3S1 took place on November 14th 2017, while the second pour occurred on November 29th 2017 for samples NB2S1, NB2S2 and NB3S2. The 4" long 1/2" stud has a length to diameter ratio equal to 8 which is greater than 4 as required by clause 17.7.2.1 of the Design of Steel Structures Canadian standard (CAN/CSA S16-14 2014). Figure 1 illustrates the test setup.

Figure 1. Test setup.

2.2 Test Setup

The methodology followed for the push-out tests is similar to the one reported in Jayas and Hosain (1988). The tests were performed using a dynamic press MTS with a capacity of 2563 kips. Loading was applied in displacement control at a rate of 7.8×10^{-5} in/s. For all tests, the load and the vertical displacement were respectively measured by the load cell and the displacement sensor inside the MTS machine. Displacement transducers were added to the steel beam into the vicinity of the panel-topping interface to measure the vertical displacement of the W-beam. The load-slip curves, presented in the section 3, shows the slip recorded by a displacement transducer located at the mid-height of the web of the W12x40 beam. More details on the experimental program can be found in Barriere *et al.* (2018).

2.3 Material Characterization

2.3.1 *Compression tests on concrete topping*

In order to characterize the concrete topping, compression tests were carried out on 12 small 6”x12” cylinders from the two different concrete pours. The tests were performed according to ASTM C39/C39M-18 standard, on a MTS 1124 kips capacity press and loading was applied in displacement control at a rate of $4x10^{-4}$ in/s. The results are presented in Table 1.

Table 1. Compressive strength of concrete topping at 28 days.

Specimen	Strength (psi)
S1 – batch 1	5323
S2 – batch 1	5207
S3 – batch 1	5062
Average batch 1	5192
S1 – batch 2	4960
S2 – batch 2	4844
S3 – batch 2	4888
Average batch 2	4902
Global Average	5047

2.3.2 *Tests on shear stud*

In order to characterize the shear studs, tensile tests were performed on three specimens where the following dimensions were measured: length, section diameter, and reduced notch diameter and notch width. The tests were conducted on a MTS 647 Hydraulic Wedge grip-112 kips capacity press. Loading was applied in displacement control at a rate of $9.8x10^{-4}$ in/s up to the yield strength, and at a rate of $3.3x10^{-3}$ in/s up to the ultimate strength. The load and the displacement were respectively measured by the load cell and the displacement sensor of the MTS machine.

The results for tensile capacity are presented in Table 2. These values, obtained with the stress-strain curve, are higher than the values supplied by Nelson Stud Company shown as 49 ksi and 61 ksi for the yield and ultimate strength respectively.

Table 2. Tensile strength of Nelson studs.

Specimen	Fy (ksi)	Fu (ksi)
S1	58.0	73.5
S2	51.3	74.3
S3	51.8	74.7
Average	57.3	74.2

3 RESULTS

In this section, Table 3 presents for each specimen the vertical displacement measured at the maximum load P_{max}, the observed failure mode of the stud, and the ultimate shear load. The load-slip curve for each of the 6 samples is then presented in Figure 2.

As per CAN/CSA S16-14, Clause 17.7.2.2, the factored resistance of a shear connector in a solid slab (q_{rs}) is given by Eq. (1);

$$q_{rs}=0.50\varphi_{sc}A_{sc}\sqrt{f'_c E_c}\leq\varphi_{sc}F_uA_{sc} \quad (1)$$

Table 3. Test results.

Specimen	Shear load per stud at failure - P_{max} (kips)	Stud failure mode	Slip at P_{max} (in)
NB1S1	21.56	Sheared	0.0045
NB1S2	14.55	Bent	0.0150
NB2S1	25.77	Sheared	0.0740
NB2S2	29.74	Sheared	0.0820
NB3S1	19.15	Sheared/Bent	0.0590
NB3S2	18.84	Sheared/Bent	0.0230

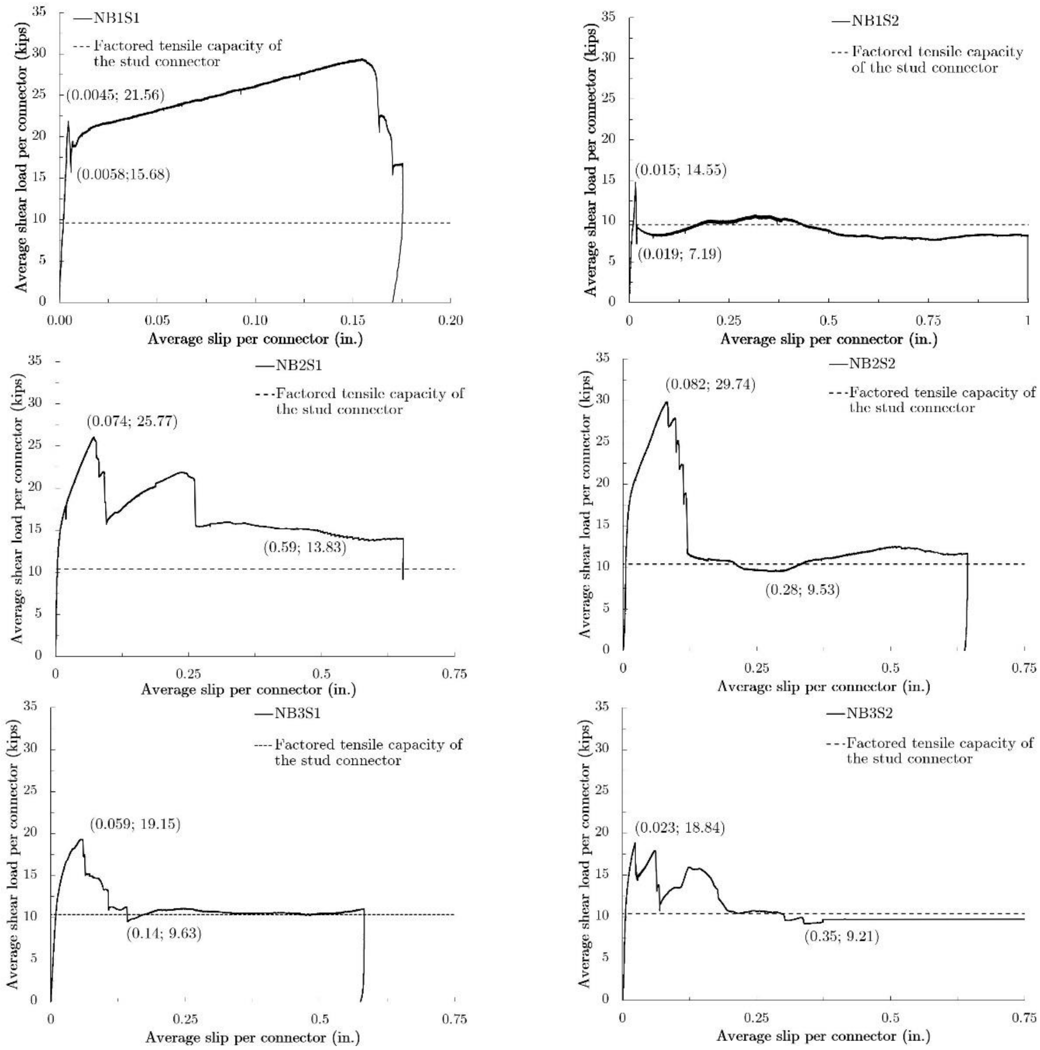

Figure 2. Load-slip curves for the 6 specimens.

Where q_{rs} represents the factored shear strength of a shear connector (N), F_u the tensile strength of the stud (MPa), and A_{sc} the cross-sectional area of the connector (mm^2), φ_{sc} = 0.80, f'_c= 35 MPa and Ec = 26622 MPa. The tensile strength, referenced by the supplier, of a 1/2" diameter steel stud is equal to 420 MPa, which corresponds to a factored shear strength, qrs, of 42.6 kN = 9.6 kips. This value appears on each load-slip curve. For each curve, the factored tensile strength of a standard 1/2" shear stud is shown along with the recording of the load as a function of the vertical displacement. For each curve except one (NB1S1), a peak at the onset of the cracked concrete was captured, followed by a plateau associated to the progressive failure of the shear stud. These results are discussed in detail in section 4, Analysis.

4 ANALYSIS

The results obtained from these six samples are now discussed in this section. From the load/slip curves (Figure 1), it was observed that all samples reached a maximum load higher than the theoretical strength of the studs. This can be seen by the peak in the average shear load per connector followed by a plateau close to the capacity of the stud. It is fair to assume that the peaks correspond to the added contribution of the shear capacity of the concrete before cracking. Therefore, the shear stress inside the concrete was calculated by considering P_{max} minus the strength of the shear studs divided by the sheared concrete plane. An evaluation of the concrete shear surface was made by adding the average concrete topping thickness to the 8” of concrete between the panels. It was observed that all failures occurred in one shear plane between the planks. The end of each plateau corresponds to the complete split failure of the specimen. Finally, the shear studs of all samples were either sheared off or bent over the W12 beam, which confirms this configuration offers an excellent confinement around the studs.

The first sample NB1S1 within the one-stud series reached a failure load of 21.56 kips which is highly in excess of the theoretical strength of 9.6 kips for a ½” stud. This over strength is probably due to an unexpected restraint within the W12 column in contact with the bottom-skid, and shall not be considered as a reliable result. The second sample NB1S2 shows a failure load of 14.55 kips which is slightly higher than the stud capacity. If the theoretical stud shear strength is subtracted from the measured failure load, this leaves a stress of 5.8 psi within the concrete. The only reliable sample within this one-stud series shows a displacement at failure higher than the 2 and 3-studs series, which is due to a lesser amount of stud, and a possible rotation of the hollowcore panel with only one stud at its midspan. For sample NB1S2, the plateau slightly remains under the shear stud strength of 9.8 kips. This observation is most likely due to some voiding in the concrete observed around the failed stud, as detailed in (Barriere *et al.* 2018).

For the two studs series, both specimens NB2S1 and NB2S2 respectively failed at 25.77 and 29.74 kips per stud. Failure was observed at the interface between the stud and W12 flange. The first peak corresponds to a concrete shear stress contribution of 52.2 psi for sample NB2S1, and 72.5 psi for sample NB2S2. All studs within this group were sheared-off from the beam flange. For these two samples, the plateau mostly shifted above the shear stud strength. Sample Both samples definitely exhibit a better contribution of the concrete shear strength between the panels.

The three studs series NB3S1 and NB3S2 respectively shows a maximum load per stud of 19.15 and 18.84 kips. The observed mode of failure was a combination of bent and sheared studs. For these two samples, an almost identical concrete stress of 40.6 psi for NB3S1 and 39.2 psi for NB3S2 are calculated. Both samples of this series shown a loading plateau almost equal to the shear stud strength. These results lead to the conclusions presented next.

5 CONCLUSIONS

Tests on floor construction with precast Hollow panels were performed in the structural laboratory of the University of Sherbrooke in order to describe the shear stud capacity characterizing the composite action between the precast panels and the steel beam supporting it. The tests were carried out at the request of the Lafarge Precast Edmonton at December 2017 and

January 2018. Six full scale specimens were load tested until failure to determine if the confinement around the shear stud was sufficient to develop its tensile strength. Based on the results of five reliable samples presented in this paper, the following conclusions are issued:

1. When the stud is confined between two standard hollowcore planks and under 2” to 3” of concrete topping, the results show that there is potential for a composite action between the planks and the steel beam supporting it;
2. For two- and three- studs set-up, it appears that it would be acceptable to calculate the shear strength of a single stud q_{rs} using Clause 17.7.2.2 of S16 with marginal modification;
3. Since the overall strength of a three stud configuration is comparable to a two stud layout, the results show there would be no need to use more than 2 studs per 4’ wide panel.

A series of full scale tests in bending should be performed in order to assess the applicability of Clause 17.9.3 of S16 needed to evaluate the value of Q_r which is the value of the sum of the factored resistances of all shear connectors between points of maximum and zero moment. This will be needed to calculate the factored bending capacity of a composite beam.

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