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EXPERIMENTAL BEHAVIOR OF PRECAST HOLLOW CORE FLOOR DIAPHRAGMS SUBJECTED TO CYCLIC LOADING

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Abstract

This paper highlights the results of the initial experimental phase of a comprehensive research project on the seismic performance of precast untopped hollow-core floor diaphragms. The experimental specimen resembled a subdiaphragm region of a prototype shear wall-frame interactive system building where the in-plane seismic forces are to be transferred to a frame and a structural wall at both floor unit ends. A bidirectional test fixture was used for simultaneous control of in-plane lateral load and bending deformations. Global behavior, stiffness, strength and deformation demands on the connections and sliding of the longitudinal and transverse joints are examined under a sequence of increasing cyclic displacements

Keywords: *Precast Hollow-Core Slabs; Diaphragm; Experimental*

1. Introduction

Seismic behavior of precast concrete diaphragms has been studied due to the poor performance observed during the 1994 Northridge earthquake. Hawkins and Gosh [1] summarized a large portion of the provisions introduced in different codes [2, 3, 4] related to the seismic design of precast concrete diaphragms. Furthermore, the authors provided a list of unsolved seismic design issues for precast diaphragms in regions of high seismic risk, including the use of untopped and topped composite diaphragms, since the use of this type of diaphragms is not currently permitted in high seismic zones in the United States. However, appropriate seismic performance of untopped diaphragms could be achieved by ensuring with experimental evidence that the system will have strength and toughness equal to or exceeding that of a monolithic reinforced concrete diaphragm. In Italy, experimental studies reported out by Menegotto and Monti [5] demonstrated that the use of serrated-sinusoidal longitudinal joints on hollow-core slabs can significantly enhanced the performance of untopped hollow core slabs.

The widespread use of precast diaphragms in regions of moderate to high seismic risk and the increasingly importance of reducing post-earthquake damage has encouraged researchers to understand the behavior of diaphragms and their influence on the global response of the structures. A research program [6] has recently addressed the need for a comprehensive design methodology of precast concrete diaphragms with appropriate design forces levels and detailing to improve their seismic response. However, the aforementioned study was mainly focused on the performance of topped precast concrete diaphragms with high in-plane flexibility.

Considering the previous aspects, and given the use of tie bars as connectors embedded within selected filled voids of hollow-core units, the Research Center on Materials and Civil Infrastructure (CIMOC) at Universidad de Los Andes - Colombia, conducted an experimental study on precast untopped hollow-core diaphragms connections as an alternative floor system for residential and commercial buildings [7]. The study was aimed to experimentally investigate the ultimate capacity of cast-in place longitudinal hollow core joints and tie bars embedded into selected filled voids of hollow core slabs.

The aim of the study presented here is to experimentally evaluate the behavior of a diaphragm system composed by untopped hollow-core slabs using embedded tie bars as connectors to the surrounding elements of the lateral force resisting system (LFRS). The test setup was designed as a sub-assembly to reproduce the behavior of a subdiaphragm region within a shear wall-frame interactive system as lateral resisting system.

2. Experimental program

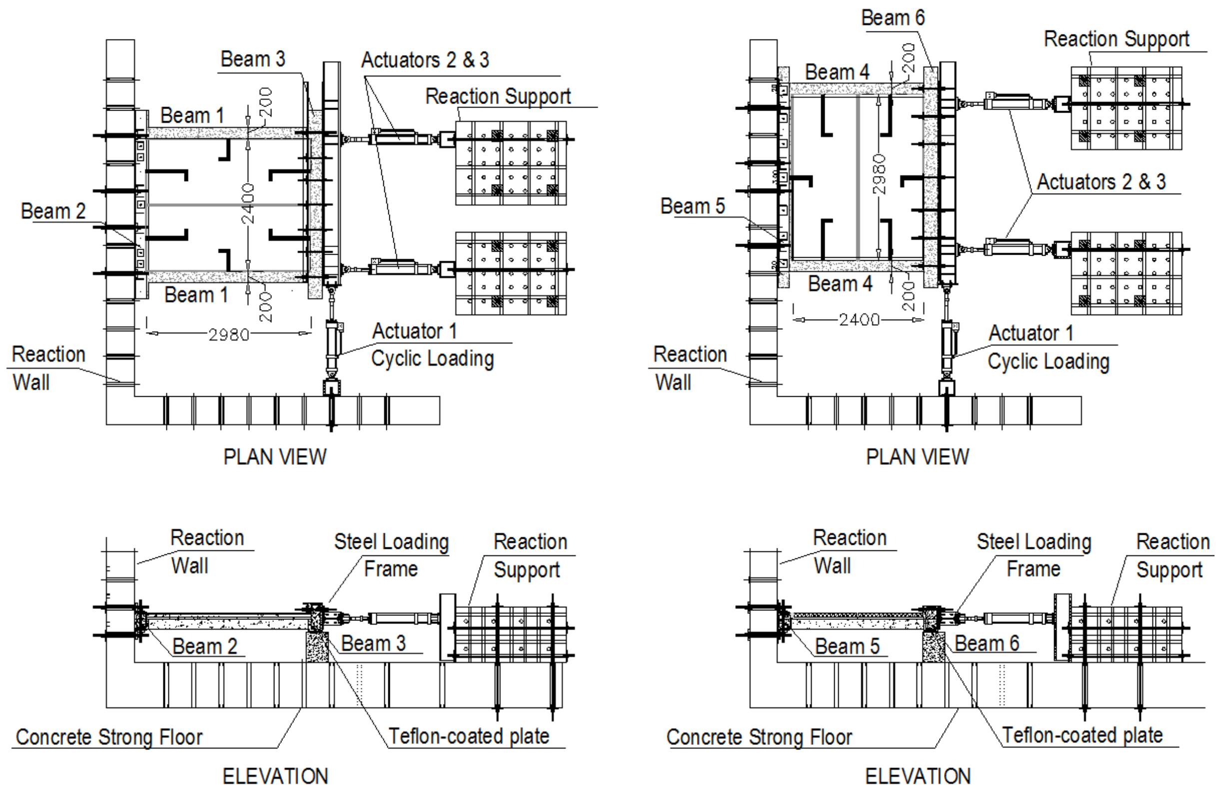
The configuration of the tests specimens and setup was selected based on the behavior of a shear wall-frame interactive system as lateral resisting system, where all the frames are intended to contribute to the lateral and vertical load resistance. Precast diaphragms systems using hollow-core slabs are mainly aimed for commercial and residential buildings, and it is usual practice to span one bay per unit. Considering the previous aspects, hollow-core slabs thickness varies between 80mm and 250mm, limiting the span that could be achieved to 30 times the slab thickness. This limit has been established not for strength reasons but to prevent excessive floor vibration under service loads.

Test specimens and setup resembled subdiaphragm region of a prototype shear wall-frame interactive system building where the in-plane seismic forces are to be transferred to a frame and a structural wall. The test program examined the in-plane cyclic performance of the diaphragm region composed of hollow-core slabs loaded in two perpendicular directions. The first (specimen A) was carried out with the cyclic load applied perpendicular to the longitudinal joints of the hollow-core slabs. The second (specimen B) was subjected to cyclic loading applied parallel to the longitudinal joints of the hollow-core slabs. Fig. 1 shows the two test specimens and setup used in the experimental program.

2.1 Diaphragm Specimens

Both specimens consisted of a pair of 1200mm wide x 2980mm long x 100mm deep precast hollow-core units jointed together through a cast-in-place concrete longitudinal shear keyway. Precast supporting beams were placed on both ends to support the slab units. Precast beams on the transverse edges of the diaphragm (running parallel to the longitudinal keyways of the hollow-core slabs) were included to represent the confinement provided by the other structural elements of the lateral force resisting system (i.e. spandrel beams, interior frames, shear walls). For specimen A supporting beams (B2 and B3) were 250mm x 300mm, while transverse beams (B1) were 200mm x 300mm. For specimen B the supporting beams (B4) were 250mm x 300mm, while transverse beams (B5 and B6) were 200mm x 300mm. Stirrups emerging from the precast beams provided horizontal shear resistance and continuity between precast elements and cast-in-place concrete. To reproduce the continuity within the diaphragm, beams B2 and B3 in specimen A, and beams B5 and B6 in specimen B were extended 300mm beyond the joints.

In order to provide structural integrity reinforcement, No. 3 deformed bars were used as longitudinal and transverse ties to connect hollow-core slabs to surrounding beams. To achieve a proper connection between hollow-core slabs and the supporting beams the central cores at both unit ends were broken-out, placing the connectors in the cores, and filling with concrete as the cast-in-place portions of the beams were poured. Connectors extended into the filled voids a distance equal to the transfer length of the pretensioned tendons in the hollow-core units (750mm) according to Mejia-McMaster and Park [8] and terminated by 90-degree end hooks and 180-degree end hooks within the hollow-core slabs and the supporting beams, respectively. Styrofoam pads were used to prevent cast-in-place concrete penetration inside the cores that were not used to anchor the connections. Transverse ties were placed at hollow-core units' mid-span following the procedure explained above, using the development length within the hollow core provided by AC 318 [2].



(a) Test A configuration (b) Test B configuration

Fig. 1 – Tests specimens and setup

Hollow-core slabs were fabricated with a specified concrete strength of 60 MPa. Cast-in-place concrete was poured with average measured compression strength of 28 MPa. Connections and diaphragm reinforcement were designed to meet the provisions of PCI Design Handbook [4] for a prescribed force equal to the actuators capacity (350 kN).

2.2 Test Setup and Instrumentation

A bidirectional test fixture was developed to allow simultaneous control of in-plane lateral load and bending deformations of the specimen through the use of three actuators. Cyclic demand was applied through displacement control of actuator 1 (shear). Actuators 2 and 3 provided a pair corresponding to the moment induced by actuator 1, while reproducing the continuity of the diaphragm beyond the setup boundaries.

The test specimen was connected and fastened to the lab reaction wall on one end (beam B2 and B5 for test A and B respectively) through four 32mm post-tensioning bars (600kN of post-tensioning force each), providing a fixed end, while beams B3 and B6, for test A and B respectively, rested vertically on a couple of Teflon pads, providing freedom of horizontal movement with minimal friction force. The actuators were connected through a steel loading frame to the free end of each specimen (beam B3 and B6 for test A and B respectively)). The steel loading frame was anchored to the concrete beams through twelve 32mm post-tensioning bars.

The specimens were instrumented internally and externally to measure the response. A total of 31 and 29 instruments were installed on specimens A and B respectively to measure displacements, force and strain data as indicated in Fig.2. Linear variable differential transformers (LVDTs) were placed on transverse and longitudinal joints to measure relative sliding between hollow-core slabs and between hollow-core slabs and adjacent beams. LVDTs were installed to measure the opening at the transverse joint between the hollow-core slabs and the supporting beams. Furthermore, two strain gauges were installed in each tie connector to measure the axial strain. Actuators loads were measured using external load cells installed at the end of each hydraulic actuator. Actuator 1 was controlled through an external LVDT.

2.3 Loading Protocol

Tests were conducted under displacement control at quasi-static rates (1.27 mm/sec). The cyclic protocol (Fig. 2) consisted of three cycles at increasing levels of shear displacement or shear distortion in accordance with the protocol developed for the PRESSS (Precast Seismic Structural Systems) program [9]. Shear distortion (SD) is calculated as Δ/ L, where Δ is the applied shear displacement and L is the center-to-center distance between beams B2 and B3 (3130 mm) and beams B5 and B6 (2675 mm) for test A and B respectively.

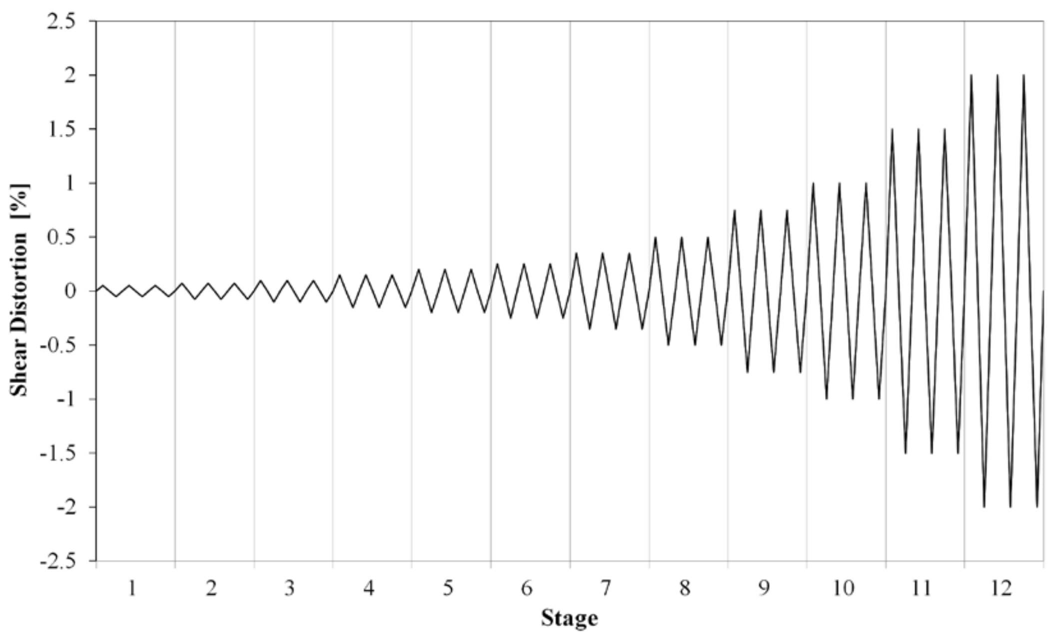


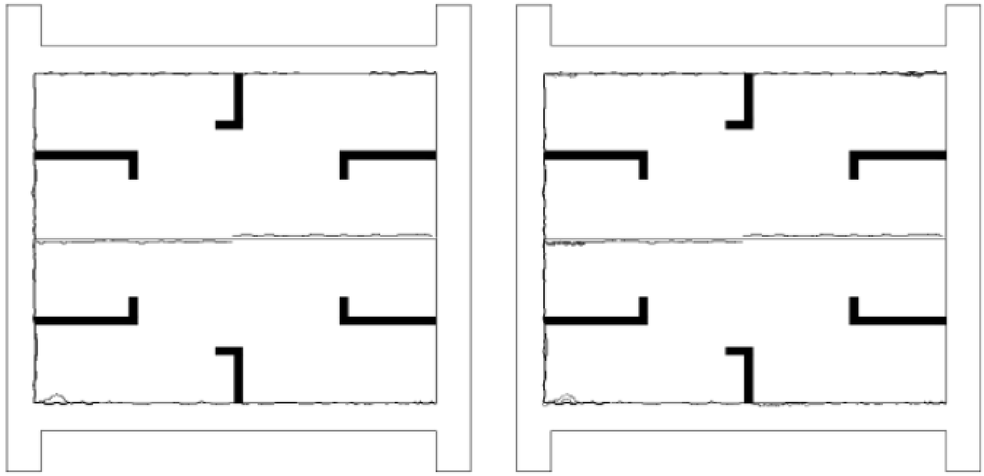
Fig. 2 – Cyclic protocol of shear distortion

3. Test results and observations

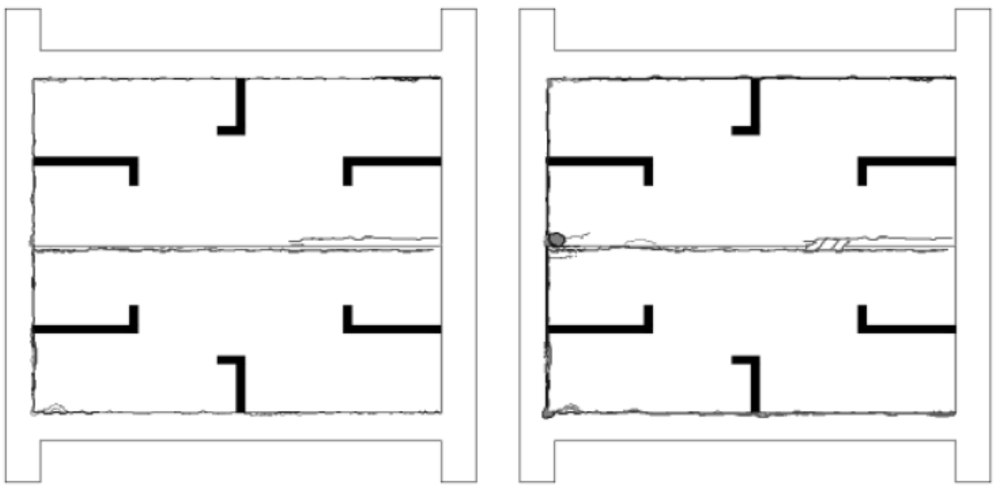
The response of the specimens was assessed through observed crack patterns and failure modes, as well as the lateral force versus displacement response of each specimen.

3.1 Specimen A

Prior to testing, Specimen A exhibited shrinkage cracking at the interface between the cast-in-place concrete and the hollow-core slabs along the transverse and the external longitudinal joints. Cracks extending along the longitudinal joint between the hollow-core slabs appeared during the first stage (SD=0.05%). Up to Stage 4 (SD=0.15%) the cracks propagated as the residual cracks width increased. As the test continued, it was observed detachment of the cast-in-place concrete along the longitudinal joints. In the cycles near the end of the test, crushing in the corners of the hollow-core slabs and in the contact area with the cast-in-place concrete of the beams was observed. The test was ended after the application of 10 stages up to a shear distortion of 1.0% (31.3 mm). Crack patterns at different stages are shown in Fig. 3.



(a) Stage 1 (SD=0.05%) (b) Stage 5 (SD=0.20%)



(c) Stage 7 (SD=0.35%) (d) Stage 10 (SD=1.00%)

Fig. 3 – Crack pattern of Specimen A at different stages

Fig. 4 shows the lateral force – displacement (shear distortion) response of the specimen loaded perpendicular to the longitudinal joints of the hollow-core slabs. The hysteretic behavior is plotted up to a shear distortion of 1.0%. During the early stages of the test, the hysteretic curve exhibited a nonlinear behavior, mainly due to the gradual opening at the transverse joints. The hysteretic curve shows stable behavior of the diaphragms specimen, with an increasing strength trend up to a shear distortion SD of 0.25%. This initial trend can be characterized by the shear resistance along the longitudinal joints and the diagonal elastic struts developed within each hollow-core slab (Fig 5 (a)). An unsymmetrical response of the specimen was observed during these stages, with the forces in the positive direction being greater than those recorded in the negative direction at the same displacement level. This behavior was probably due to the initial cracking concentrated at one corner joint near the fixed end of the setup. As the test continued, the response became more symmetrical.

For shear distortions above 0.25%, the test specimen exhibited a nearly constant strength due to the increasing crushing of the diagonal struts developed as a consequence of the near rigid body behavior of the hollow-core slabs. A pinching effect was observed as a result of the residual opening in the transverse joints during reloading. An increase in both stiffness and strength was observed once the gaps closed and the strut mechanism was activated in the opposite direction.

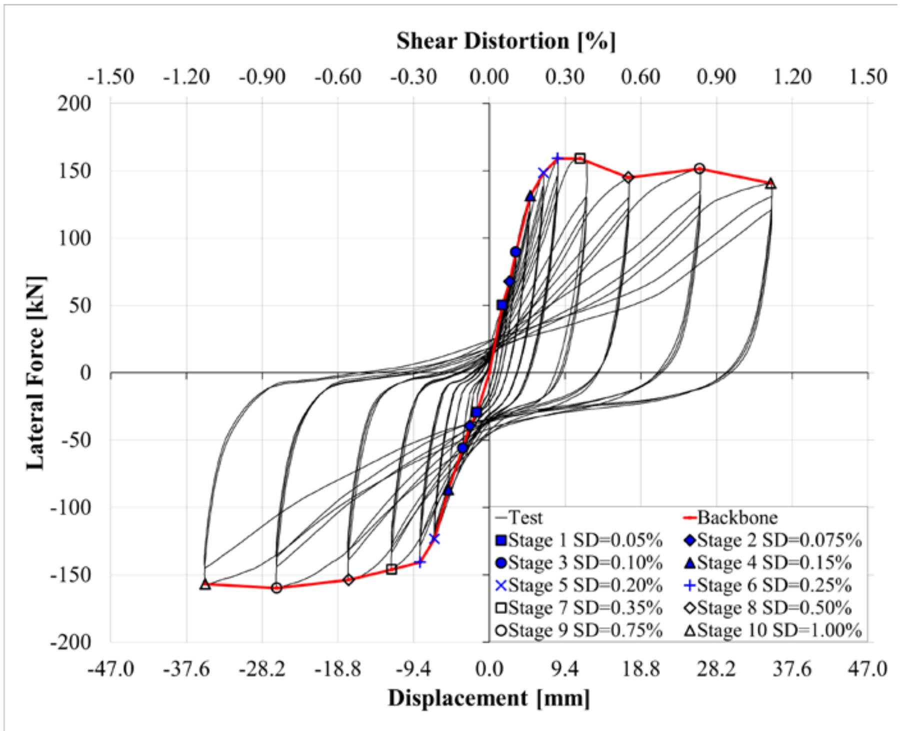
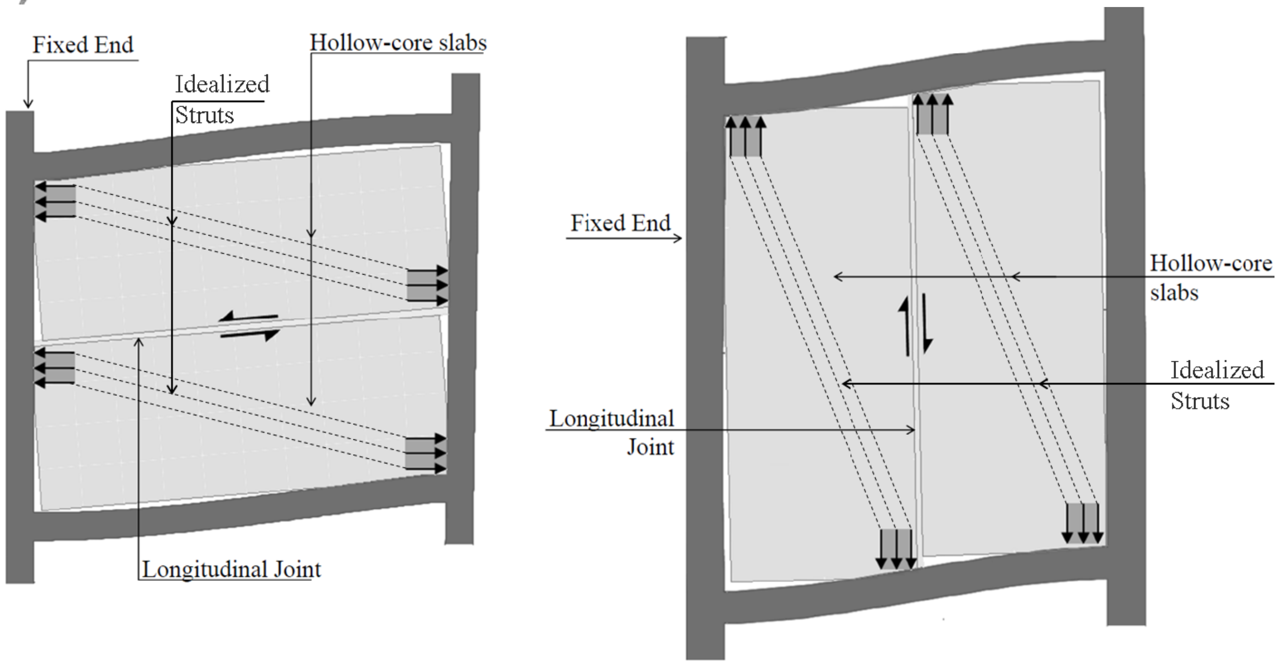


Fig. 4 – Hysteresis curve for specimen A.

Strain measurements taken in the tie connectors indicated that plastic strains developed at all the ties placed in the transverse joints after the application of Stage 8 (SD=0.50%). Furthermore, the opening of the transverse joints measured at the tie connectors location and the observed behavior indicate that these connectors did not contribute significantly to the diaphragm resistance by shear friction mechanism but by kinking in these bars crossing the joints.

3.2 Specimen B

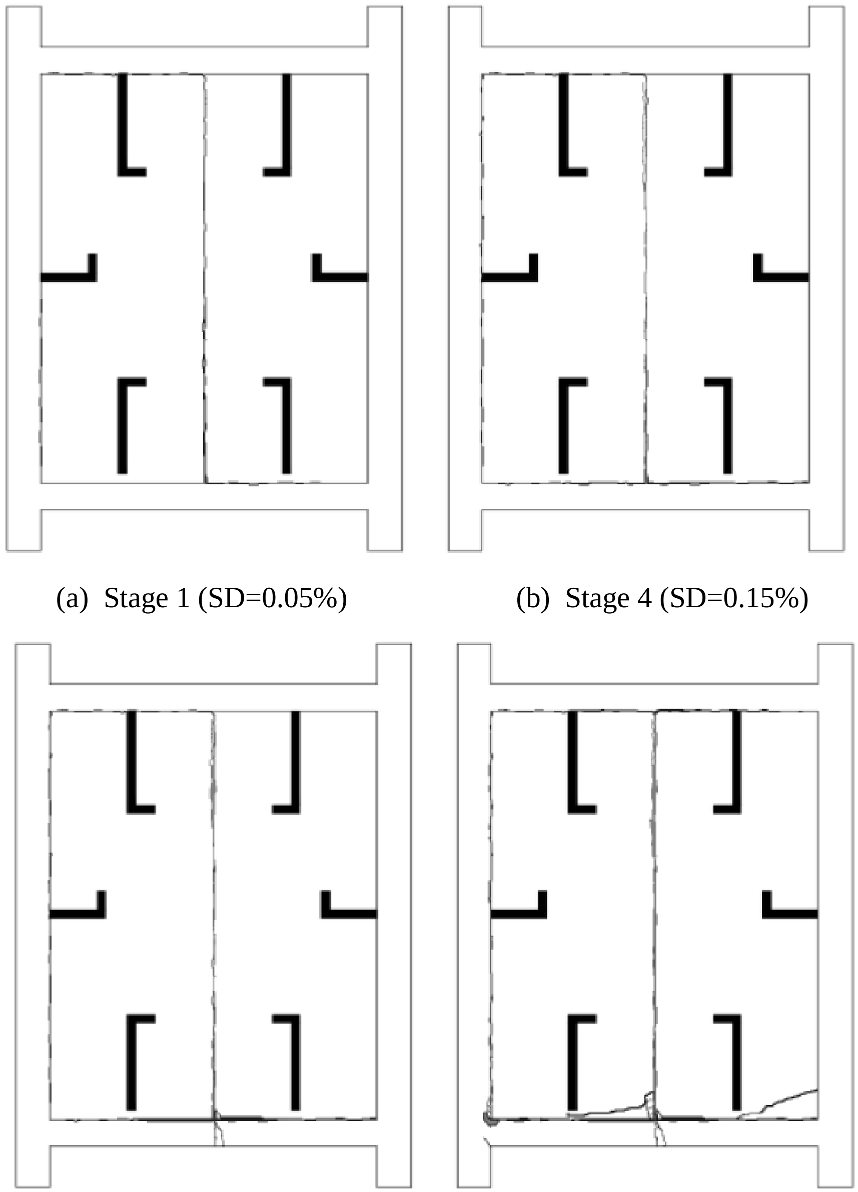
Figure 6 shows the crack pattern at different stages during testing of Specimen B. In Specimen an initial shrinkage cracking also developed and propagated at the interface between the cast-in-place concrete and the hollow-core slabs along the transverse and the longitudinal joints. Sliding along the longitudinal joint between the hollow-core slabs and rigid body rotation of the slabs were found to be the predominant mode of deformation. Incipient spalling of the cast-in-place concrete along the longitudinal joint between the hollow-core slabs was observed up to Stage 5 (SD=0.2%). Vertical cracks developed at the mid-span of the supporting beams. These cracks developed due to the activation of the inclined struts within each hollow-core slab, demanding flexural stresses in the plane of the diaphragm on the supporting beams, additional to the bending when acting as part of the lateral force resisting system.



(a) Specimen A (b) Specimen B

Fig. 5 – Scheme of the strut resistance mechanism developed within the specimens

After Stage 5 (SD=0.2%), progressive spalling along the longitudinal joints took place as the vertical cracking width at the supporting beams increased as a consequence of the more pronounced rigid body rotations of the slabs. At Stage 9 (SD=0.75%) splitting of the hollow-core slabs at the corners occurred probably due to lack of compatibility. The test was ended during the application of Stage 10 (SD=1%) due to a problem with the test control. It is worth noting that, despite the early end of the test, a relative deformation of 1% is extremely large and not relevant in the design of diaphragms. Moreover, the damage experienced in the previous stages indicated that diaphragm designs with hollow-core slabs should be performed not only in terms of strength but also considering limits for the in-plane deformation that account for damage.



(a) Stage 1 (SD=0.05%) (b) Stage 4 (SD=0.15%)

(c) Stage 7 (SD=0.35%) (d) Stage 10 (SD=1.00%)

Fig. 6 – Crack pattern of Specimen B at different stages

Fig. 7 shows the response of the specimen loaded parallel to the longitudinal joints of the hollow-core slabs. The response on this direction was characterized by an initial high stiffness when compared to Specimen A. In the subsequent stages a reduction of the stiffness was observed due to the increasing spalling of the cast-in place concrete of the longitudinal joint.

As with Specimen A, an early hysteretic behavior was observed due to the gradual opening at the transverse joints. The increasing strength trend exhibited up to shear distortions SD of 0.20% can be associated to contribution of the shear resistance along the longitudinal joints and the diagonal elastic struts developed within each hollow-core slab (Fig 5 (b)). Subsequent stages showed an increase in the lateral strength with a reduced stiffness that can be related to the accumulated damage along the longitudinal joints, leading to a strut mechanism within the slabs.

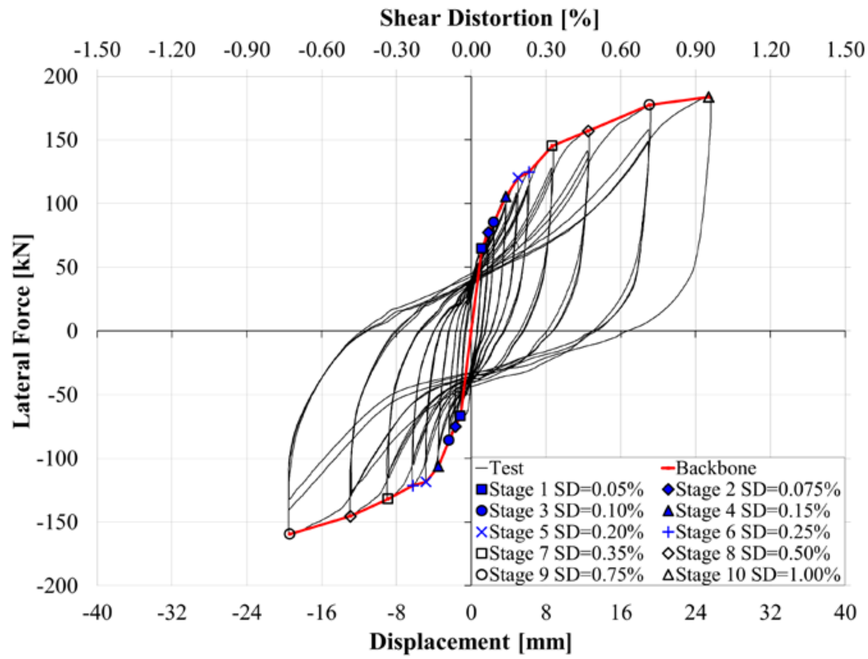


Fig. 7 – Hysteresis curve for specimen B.

Measurement of the local strains in the tie connectors indicated that plastic strains developed at all the ties placed in the transverse joints after the application of Stage 7. As with Specimen A, the measured plastic strains are related to kinking the tie connectors crossing the joints. Fig. 8 shows the backbone curves for both tests.

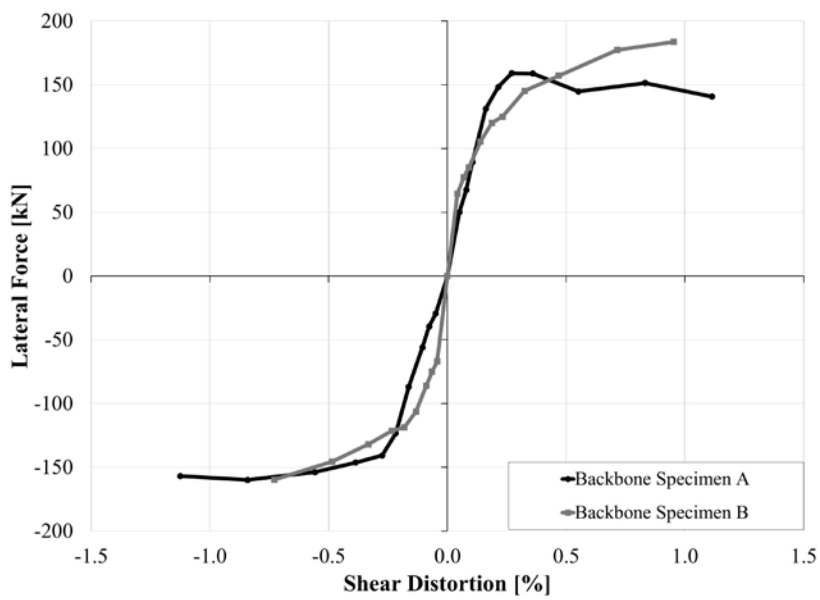


Fig. 8 – Backbone curves.

4. Summary and conclusions

This paper presents the results of an experimental study to assess the response of two untopped hollow-core subdiaphragms regions. Large-scale sub-assemblies were built and tested under reversed cyclic loading parallel and perpendicular to the longitudinal joints of the precast slabs. Based on the results of this investigation, the following conclusions can be drawn:

The hysteretic response of the test specimens exhibited a stable behavior that can be categorized in two phases: an initial stage with an increasing strength trend that accounts for both shear resistance along the longitudinal joints and diagonal struts developed within each hollow-core slab; and a second phase with a reduced stiffness, that can be simplified to nearly constant strength, due to the increasing crushing of the diagonal struts controlled by the rigid body behavior of the hollow-core slabs.

The overall behavior of the specimens was described by cracking along the longitudinal and transverse joints. In addition, it was observed during the tests that the transverse joints opened significantly as a result of the rigid body behavior of the hollow-core slabs. In the final stages of the tests, concentrated stresses at the hollow-core slabs corners and at the cast-in place concrete of the beams in contact with these corners caused significant damage. For the specimen B, vertical cracks developed at the mid-span of the supporting beams, where the inclined struts demanded the beams in the horizontal plane. Consequently, for design purposes of untopped hollow-core diaphragms, an adequate estimation of the strut mechanism forces should be considered in order to perform an appropriate design of the beams in the plane of the diaphragm.

The measurements taken during the tests showed plastic strains on the tie connectors that can be related to kinking action. The magnitude of the demanded strains on these connectors should be also considered in the design process to accomplish the codes provisions associated to structural integrity reinforcement.

Based on the results and observations from the tests presented here, it appears that diaphragm designs with hollow-core slabs should be performed not only in terms of strength but also considering limits for the in-plane deformation that account for acceptable damage on the diaphragm elements.

Additional test data are still needed to compare the results and observations presented herein with diaphragms systems that are entirely cast-in-place or comprise cast-in-place topping, composite or non-composite, with hollow-core slabs.

5. Acknowledgements

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