EXPERIMENTAL BEHAVIOR OF HOLLOW CORE CONNECTIONS IN UNTOPPED DIAPHRAGMS

Nelson Angel
PhD Student, Universidad de los Andes, Colombia
nm.angel55@uniandes.edu.co

Juan F. Correal
Associate Professor, Universidad de los Andes, Colombia
jcorreal@uniandes.edu.co

ABSTRACT: This paper highlights the results of the initial experimental phase of a comprehensive research project on the seismic performance of untopped hollow-core diaphragms. The experimental specimens resembled a real-scale diaphragm 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.

1. Introduction

Seismic behavior of precast concrete diaphragms has been studied due to the poor performance observed during the 1994 Northridge earthquake. Therefore, several design provisions were introduced in different codes such as the Uniform Building Code (ICBO, 1997), NEHRP Provisions (FEMA, 1998) and ACI 318 standard (ACI Committee 318, 1999). For example, ACI 318 requires for special seismic design category, a cast-in-place topping slab acting as composite or noncomposite diaphragm. However, untopped precast diaphragms of buildings designed for special seismic design category shall be permitted if tests show that the strength and toughness is equal to or exceeding that of a monolithic reinforced concrete diaphragm.

Hawkins and Gosh (Hawkins and Gosh, 2000) summarized a large portion of these provisions. 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 carried out by Menegotto and Monti (Menegotto and Monti, 1996) demonstrated that the use of serrated-sinusoidal longitudinal joints on hollow-core slabs can significantly enhanced the performance of untopped hollow core slabs.

Recently, a multi-university research team comprising the DSDM (Diaphragm Seismic Design Methodology) Consortium has carried out experimental and analytical research on precast diaphragms. This research developed a comprehensive design methodology (DSDM Consortium, 2014) including diaphragm design forces levels, diaphragm reinforcement and connector classification based on deformation capacity and overstrength factors to avoid fragile mechanisms of failure. However, the aforementioned study was mainly focused on the performance of topped diaphragms with high 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 untopped hollow-core diaphragms. This study was aimed to experimentally determine the global behavior of a diaphragm system composed by hollow-core slabs and the performance of the defined connectors between the slabs and elements of the lateral force resisting system. This study is expected to be the first step in assessing the use of untopped hollow-core diaphragms with tie bars as connectors in high seismic zones.

2. Experimental Program

The configuration of the tests specimens and the setup was selected based on the current seismic design and construction practice in Colombia, where most of the buildings are constructed using shear wall-frame interactive systems as lateral resisting system. Furthermore, all the frames in a building 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 120mm, limiting the span that could be achieved to 30 times the slab thickness. This limit has been established not for strength reasons but in order to prevent excessive floor vibration under service loads.

Thus, test specimens and setup resembled a real-scale diaphragm 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 hollow-core units ends. The test program examined the performance of the diaphragm region and the response of the tie bars used as connectors between hollow-core slabs and lateral force resisting elements. Four tests were conducted. The following sections detail the main features of the 4 tested diaphragms.

2.1. Diaphragm Specimens

Fig. 1 shows the test specimens and setup used in the experimental program. The diaphragm specimens consisted of a pair of 1200mm wide x 2980mm long x 100mm deep hollow-core units jointed together through a cast-in-place longitudinal shear keyway. Precast supporting beams were placed on both ends to support the slab units. 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). Supporting beams (B2 and B3) were 250mm x 300mm, while the beams on the transverse edges (B1) were 200mm x 300mm. Shear reinforcement emerging from the precast beams provided horizontal shear resistance and continuity between precast and cast-in-place concrete.

As the primary interest for the research, two types of connections between the slabs and the supporting beams were selected. Type D connectors were constructed from No. 3 deformed bars, commonly used as reinforcing bars. Type P connectors were fabricated from No. 4 plain bars. Connections between hollow-core slabs and supporting beams were achieved by breaking out the central cores at each slab end, placing the connectors in the cores, and filling with concrete as the cast-in-place section of the beams was 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 (Mejía-McMaster and Park, 1994), 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.

In order to study the influence of the continuity of the diaphragm, beams B2 and B3 were extended 300mm at both ends beyond the joints in two of the tested specimens. Table 1 lists the test parameters for each specimen.

Hollow-core slabs were fabricated with a specified concrete strength of 60 MPa. Cast-in-place concrete were poured with average measured compression strength of 28 MPa. Connections and diaphragm reinforcement were design to meet the provisions of PCI Design Handbook (Industry Handbook Committee, 2004) for a prescribed force equal to the actuators capacity (350 kN).

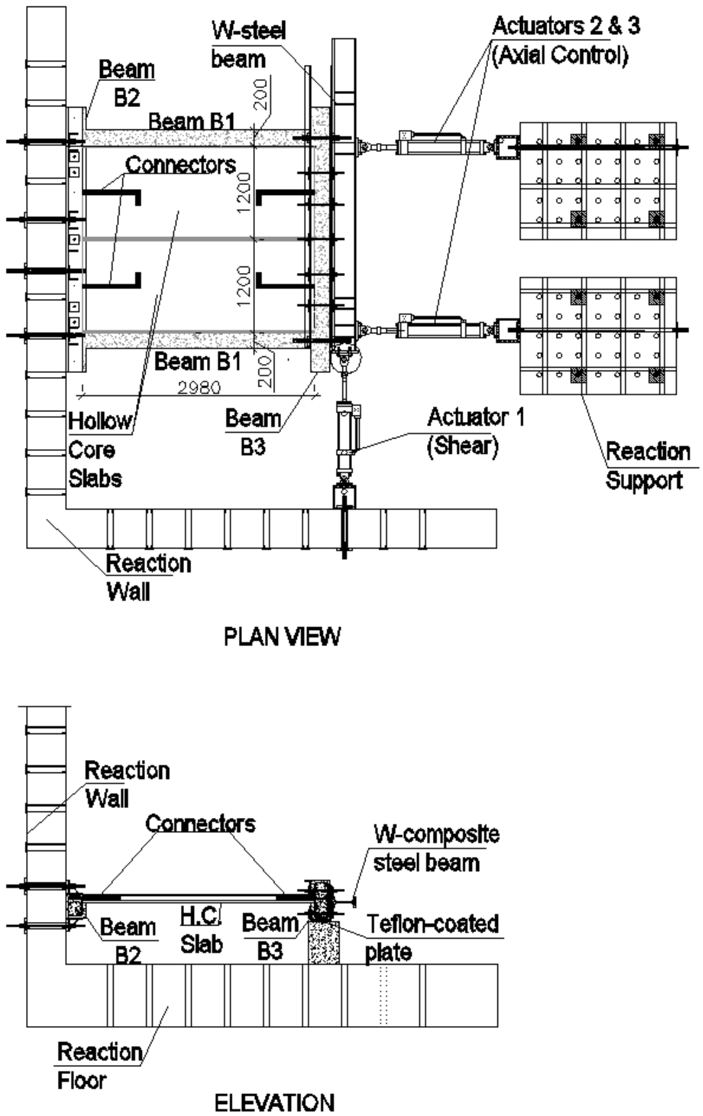

Fig. 1 – Tests specimens and setup.

Table 1 – Configuration of the specimens.

Specimen / Test	Connectors	Yield strength (MPa)	Beam extension (mm)
D1-J0	Deformed bars	420	0.0
P1-J0	Plain bars	240	0.0
D2-J3	Deformed bars	420	300.0
P2-J3	Plain bars	240	300.0

2.2. Test Setup and Instrumentation

A bidirectional test fixture was developed to allow for simultaneous control of in-plane lateral load and bending deformations at the specimen through the use of three actuators. Demand was applied through displacement control of actuator 1 (shear). Actuators 2 and 3 held axial deformation constant to keep pure shear deformation on the specimen. The test specimen was connected and fastened to the lab reaction wall on one end (beam B2) through four 32mm post-tensioning bars (600kN of post-tensioning force each), providing a fixed end, while the other supporting beam (B3) rested vertically on a couple of Teflon pads, providing freedom of horizontal movement with minimal friction force. The actuators were connected through a W-composite steel beam to the free end of the specimen (beam B3). Steel beam was anchored to beam B3 through twelve post-tensioning bars (two 32mm and ten 15mm diameter).

A total of 15 instruments were installed on each specimen to record displacement, force and strain data. Three linear variable differential transformers (LVDTs) were placed on transverse and longitudinal joints to measure relative displacements between hollow-core slabs and between hollow-core slabs and supporting beams (B2 and B3). For tests D2-J3 and P2-J3, two LVDTs were installed at the fixed end to measure the opening at the transverse joint between the hollow-core slabs and the supporting beam B2. Strain Gauges were installed at each connector to calculate the tie forces using the measured axial strain. Two more strain gauges were located at the top exterior rebars of beams B1, near the fixed end of the specimens. Actuators loads were measured using external load cells installed at the end of each hydraulic actuator. Actuators were controlled through additional LVDTs, one per actuator.

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 (Priestley, 1992).Shear distortion is calculated as Δ / L, where Δ is the applied shear displacement and L is the center-to-center distance between the mid-section of beams B2 and B3, equal to 3130 mm.

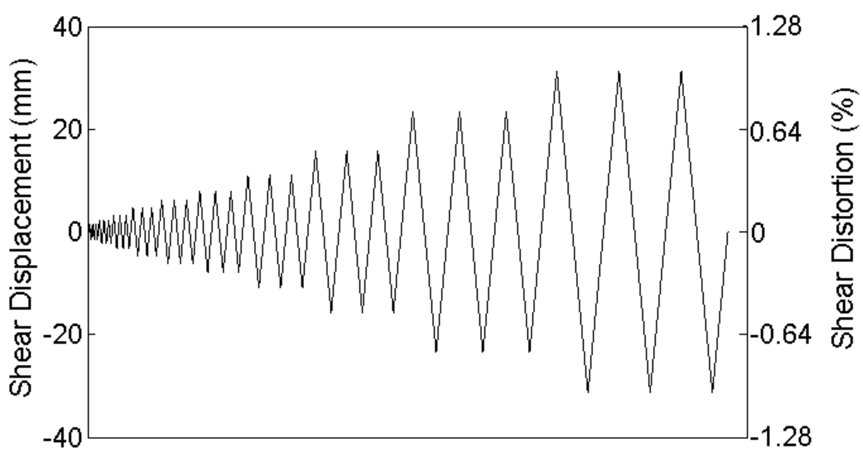

Fig. 2 – Cyclic protocol of shear displacements.

3. Test Results and Observations

Fig. 3 summarizes the load – displacement response of each test. In general, the extension of the beams (specimens D2-J3 and P2-J3) has a stiffening effect due to the elastic behavior of the extended portion throughout the tests, while specimens D1-J0 and P1-J0 exhibited a response influenced by the flexibility and damage observed on the external faces of these extended beams. D1-J0 and P1-J0 tests produced wider hysteresis loops, which is more energy dissipation, with lower demands on axial control actuators. Tests on specimens D2-J3 and P2-J3 were prematurely stopped at shear distortions of 0.35% and 0.25%

respectively because one of the axial actuators that controlled bending on the specimen reached its maximum load capacity.

In general, specimens exhibited cracking along transverse and longitudinal joints, followed by the progressive concrete spalling of the upper layers on these joints. In subsequent cycles the cracking opening propagated. As it was described previously, beam joints on specimens D1-J0 and P1-J0 were damage at the final cycles and exhibited diagonal cracking on the exterior faces. The connectors, however, did not fracture and were capable of sustaining more deformation. No substantial damages were observed in the hollow-core slabs during the post-tests inspections.

Total stiffness degradations were 71%, 55%, 28% and 25.5% for specimens D1-J0, P1-J0, D2-J3 and P2-J3 respectively. It should be notice that the reported stiffness degradation was lower for the specimens with the extended beams due to the lower level of shear displacement achieved in these tests. It seems that the behavior of the specimens was influenced by the beams around the hollow-core slabs and the relative short length of the slabs units, contributing to the overall stiffness and diaphragm integrity once continuity between hollow-core slabs and adjacent beams is lost due to the failure of the longitudinal and transverse joints. Strength degradation was not observed.

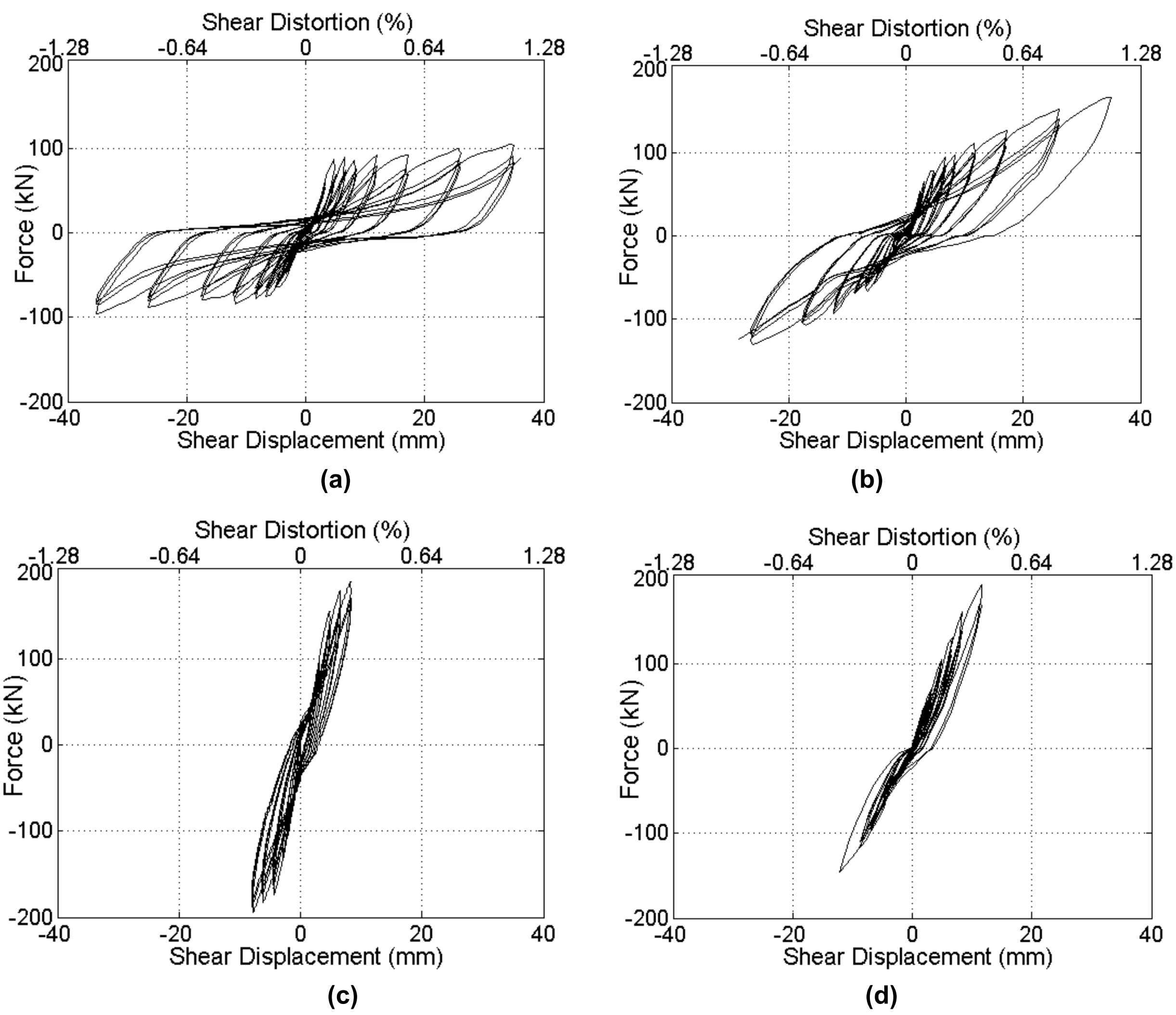

Fig. 3 – Specimen responses: (a) D1-J0; (b) P1-J0; (c) D2-J3; (d) P2-J3

Fig. 4 shows the strain history recorded in the most demanded connector in each test. Tension tests on No. 3 deformed bars and No. 4 plain bars indicated an average yield strain of 0.0021 mm/mm and 0.0022 mm/mm respectively. As observed in strain records, the connections composed by deformed bars did not yield, while the most demanded plain connection on test P1-J0, reached a strain of 0.0027 mm/mm or 129% of the yield strain. This strain was obtained for a lateral displacement of 23.5mm, equal to 0.75% of shear distortion.

It should be notice that the use of plain bars enables bond failure to propagate along the connection, thus making large ultimate strain possible when compared with deformed bars. As can be seen in Fig. 4, strain demands on plain connections are higher than those observed for deformed connections at the same level of shear displacement on the specimens.

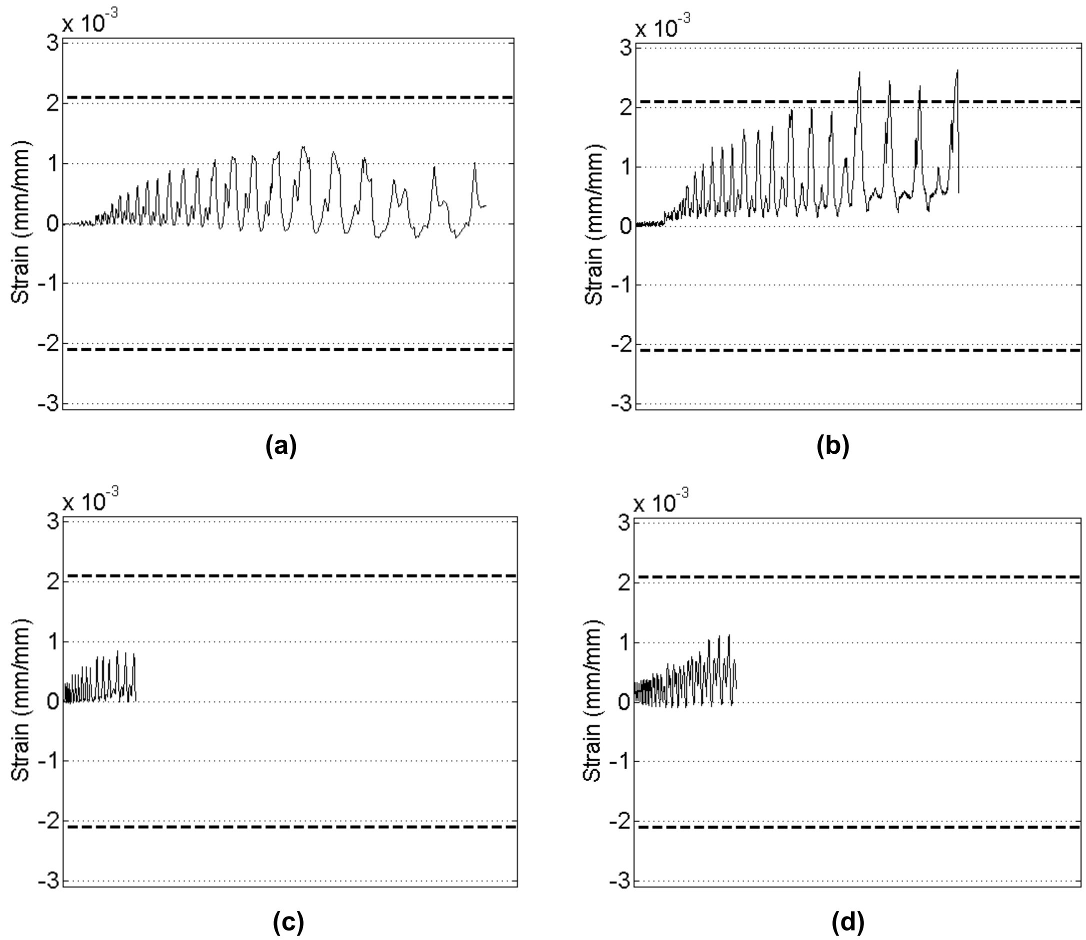

Fig. 4 – Strain at most demanded connection: (a) D1-J0; (b) P1-J0; (c) D2-J3; (d) P2-J3

Fig. 5 shows the transverse joint sliding history recorded at the fixed end of the specimens, between beam B2 and hollow-core slabs. Transverse joints accumulate residual sliding even from the early cycles of the tests. The accumulation of sliding is more evident in the specimens with no beam extensions (D1-J0 and P1-J0), reaching measured values of sliding up to 5.0 mm. Test results also indicated that the transverse joint sliding in not sensitive to the type of connection used.

When the longitudinal joints shear capacity were exceeded, significant shear slip along the keyways was observed, reaching values up to 14.0 mm in the case of test D1-J0, as shown in Fig. 6. With the exception of the test D2-J3, the longitudinal joints did not exhibit a residual sliding due to cyclic loading. In general, for a prescribed level of shear displacement on the specimens, sliding at longitudinal joints is not sensitive to the stiffening effect due to the incorporation of beams joints length.

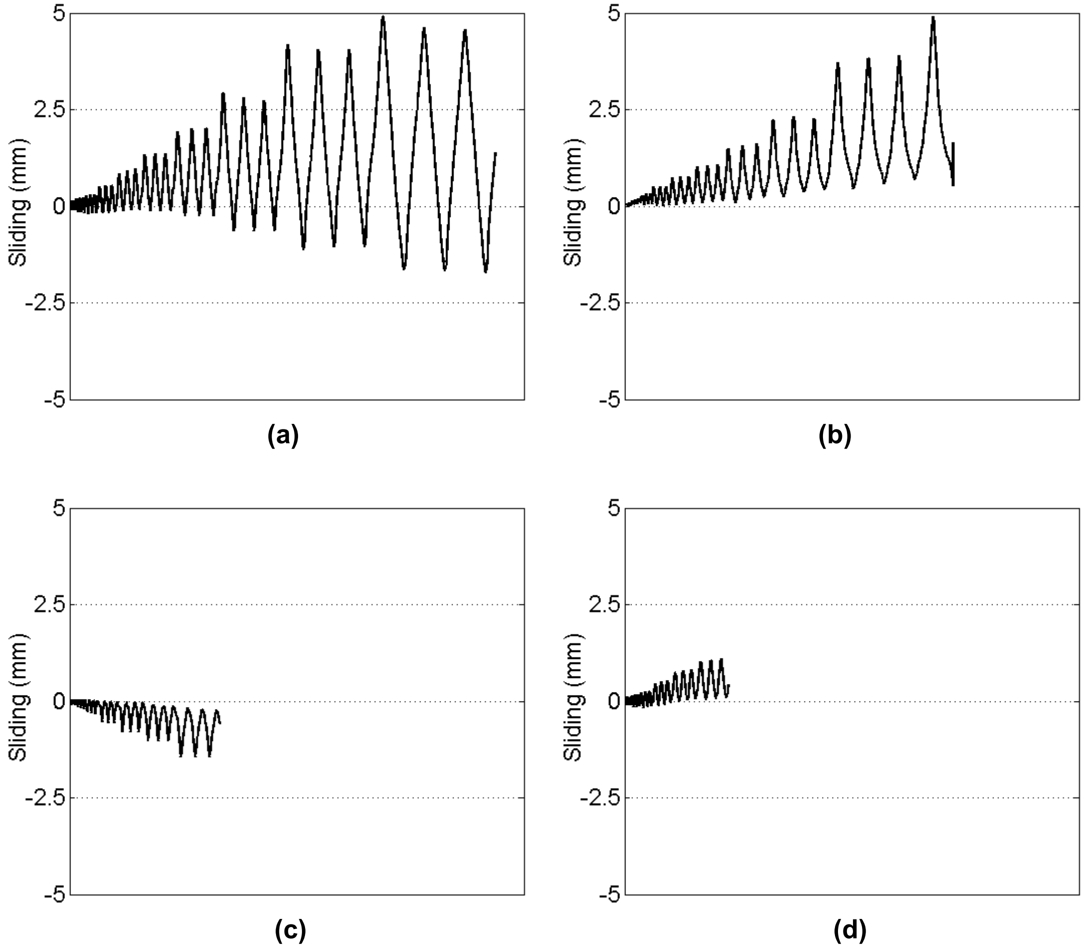

Fig. 5 – Sliding at transverse joints: (a) D1-J0; (b) P1-J0; (c) D2-J3; (d) P2-J3

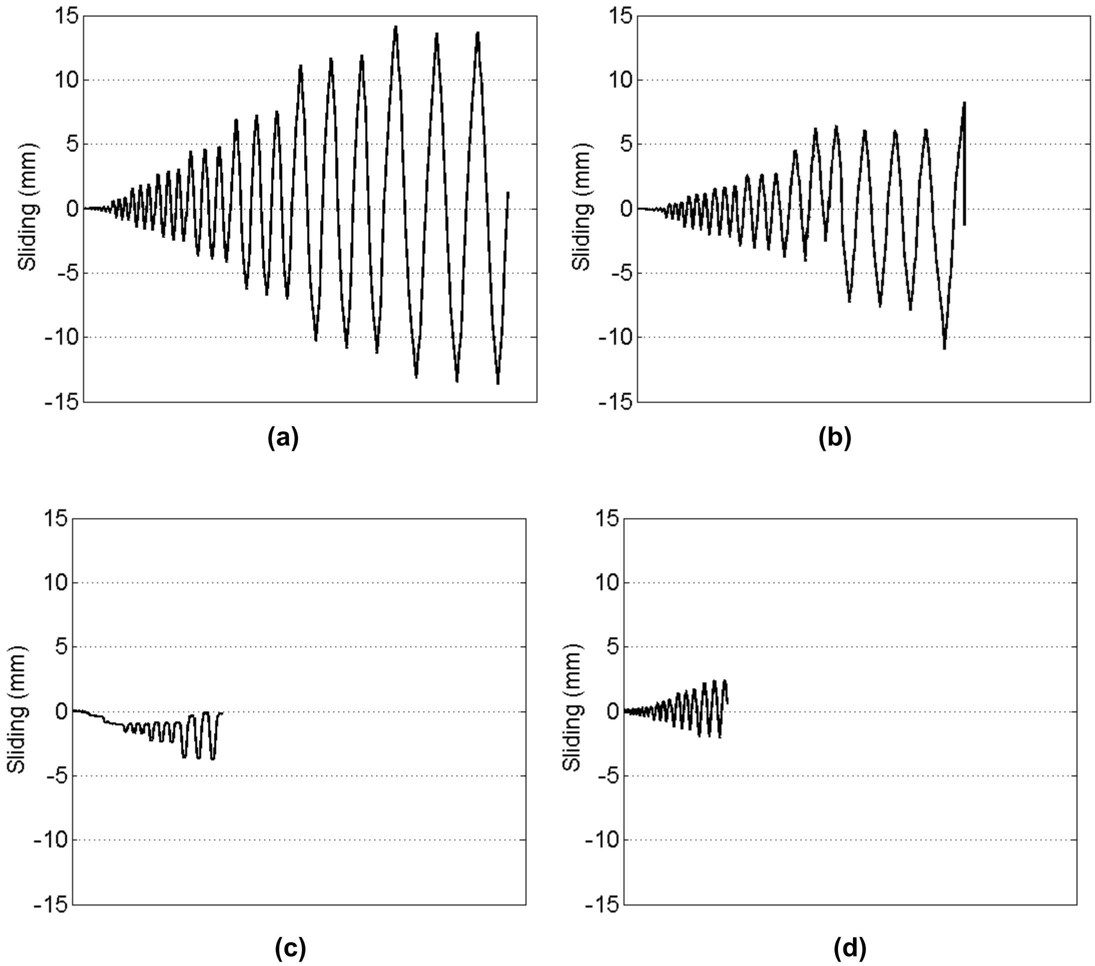

Fig. 6 – Sliding at longitudinal joints: (a) D1-J0; (b) P1-J0; (c) D2-J3; (d) P2-J3

4. Summary and Conclusions

The experimental research program examined the cyclic performance of four specimens representative of a real-scale hollow-core diaphragm region where the in-plane seismic forces are to be transferred to a frame and a structural wall at both hollow-core units ends. As the primary interest for the research, deformed bars and plain bars were used as connections placed in filled voids of hollow-units and supporting beams. The influence of the continuity of the diaphragm, provided by the extension of beams in the corners was also studied. From the test observations and discussions presented, the following conclusions are drawn:

The behavior of the tested specimens was influenced by the stiffness of the beam joints. Extension of beams, representing the continuity of the diaphragm, provided a stiffer response of the precast diaphragm. The stiffness of the specimens without beam joints extension significantly degraded after the formation of diagonal cracking on the exterior faces of the corner joints.

The connectors used in the four specimens exhibited low deformation demands. One of the connections on test P1-J0 reached a deformation slightly over the yield strain.

The plain bar connections exhibited higher strain demands than those obtained for deformed bar connections for a prescribed level of shear displacement, due to the lower bond between concrete and plain bars making larger strains possible.

Sliding on transverse and longitudinal joints initiated since the early stages of the tests. The measured joint sliding was not sensitive neither to the type of connection used nor the stiffness provided by the extension of the beams.

Due to the limited load capacity of the axial actuators, control of these actuators has been modified for the current tests, allowing horizontal displacements perpendicular to the shear displacement of actuator 1 but controlling rotation on the free end of the specimen. This consideration has reduced the load demand on the axial actuators, achieving all the stages of the lateral displacement protocol.

5. Acknowledgements

This research is supported by the Colombian Administrative Department of Science, Technology and Innovation COLCIENCIAS and the precast concrete producers Manufacturas de Cemento S.A through grant 1204-562-37641. The authors are also grateful for the support from Dywidag Systems DSI Colombia.

6. References

ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI318-99) and Commentary (ACI318R-99)", 1999

Diaphragm Seismic Design Methodology DSDM Consortium, "Seismic Design Methodology Document for Precast Concrete Diaphrgagms. Research Report", 2014.

FEMA. "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures FEMA 303", 1998.

HAWKINS, Neil, GHOSH, S.K., "Proposed Revisions to 1997 NEHRP Recommended Provisions for Seismic Regulations for Precast Concrete Structures: Part 3 – Diaphragms", *PCI Journal*, Vol 45, No. 6, November 2000, pp. 50-59.

Industry Handbook Comittee. "PCI Design Handbook: Precast and Prestressed Concrete 6th ed.", 2004

International Conference of Building Officials ICBO. "Uniform Building Code 1997", 1997.

MEJIA-MCMASTER, Juan, PARK, Robert, "Tests on Special Reinforcement for the End Support of Hollow-Core Slabs", *PCI Journal*, Vol 39, No. 5, September 1994, pp. 90-105.

MENEGOTTO, Marco, MONTI, Giorgio, "Diaphragm action of precast floors: Behavior and Modeling", *Proceedings of the 11th World Conference on Earthquake Engineering*, No. 768, June 1996.

PRIESTLEY, Nigel, "The U.S.–PRESSS Program Progress Report. Third Meeting of the U.S.–Japan Joint Technical Coordinating Committee on Precast Seismic Structural Systems (JTCC-PRESSS)", 1992.