Shallow precast concrete floor without beam ledges or column corbels

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- Current precast concrete floor systems require the use of inverted-tee beams with ledges to support hollow-core slabs and column corbels to support beams, which results in a low span-to-depth ratio and reduced height clearance.
- A shallow precast concrete floor system utilizing hollow-core slabs that reduces floor depths by eliminating permanent beam ledges or column corbels is proposed for use in multistory office buildings.
- Beam–hollow-core slab connections and beam–column connections performed well in full-scale tests, and shear capacities were accurately predicted using the shear friction design method from ACI 318-14.

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Precast concrete floor systems usually consist of hollow-core slabs supported by precast/prestressed concrete inverted-tee beams that are supported on column corbels or wall ledges. These floor systems are covered by either a thin nonstructural cementitious topping or a 2 to 3 in. (51 to 76 mm) structural composite concrete topping. They are suitable for multistory office buildings because of their speed of construction, economy, and high structural performance. Despite these advantages, precast concrete floor systems have two main drawbacks: a low span-to-depth ratio and the presence of floor projections, such as column corbels and beam ledges. For instance, it is not uncommon to have an inverted tee with span-to-depth ratio of 12 for a hollow-core floor with a bay size of 30 ft (9.1 m). In addition, this floor would have a 12 in. (305 mm) deep ledge below the hollow-core slab and a 16 in. (406 mm) deep column corbel below the beam soffit.[1] Alternatively, post-tensioned, cast-in-place concrete floors can achieve a span-to-depth ratio up to 45 with a flat plate floor system,[2] which results in reduced floor height and savings in architectural, mechanical, and electrical building systems.

The main objective of this paper is to present the development of a 10 in. (254 mm) deep precast, prestressed concrete beam for multistory office buildings. The beam only projects 2 in. (51 mm) below the 8 in. (203 mm) hollow-core slabs it supports. With a 2 in. composite topping, the total structural depth is 12 in. (305 mm), corresponding to a span-to-depth ratio of 30 for a 30 ft (9.1 m) bay size. The proposed beam is only 4 ft (1.2 m) wide and simple to form, thus allowing production in existing prestressing beds and eliminating the concerns about difficulties in making 8 ft (2.4 m) wide beams that were presented in a previous study.[3]

The paper is organized as follows: first, existing precast concrete floor systems are reviewed; second, design and detailing of the developed shallow floor beam and its connections with hollow-core slabs and columns are discussed; third, the construction sequence of the floor system is presented; fourth, the experimental verification is demonstrated; and finally, conclusions and recommendations are summarized.

Existing precast concrete systems

Low et al.[3,4] developed a shallow floor system for multistory office buildings. The system consisted of hollow-core slabs; 8 ft (2.4 m) wide, 16 in. (406 mm) deep prestressed beams; and single-story precast concrete columns fabricated with full concrete cavities at the floor level. Column reinforcement was passed through the cavity in the beam and mechanically spliced at the project site to achieve its continuity. The large beam width and single-story columns discouraged its adoption by U.S. precast concrete producers.

Composite Dycore Office Structures[5] developed a floor system consisting of a shallow soffit beam, floor slabs, and continuous precast concrete columns with blockouts at the beam level. In this system, precast concrete beams and floor slabs act primarily as stay-in-place forms for the major cast-in-place concrete operations required to complete the floor system, which was costly and time-consuming.

A system developed by Mid-State Filigree Systems Inc.[6] consists of reinforced precast concrete floor panels that serve as permanent formwork. The panels are composite with cast-in-place concrete and contain the reinforcement required in the bottom portion of the slab. They also contain a steel lattice truss that projects from the top of the precast concrete unit. One of the main advantages for this system is a flat soffit floor, which does not require a false ceiling. However, this system requires extensive fieldwork techniques to produce.[7]

Simanjuntak[8] developed a shallow ribbed slab configuration without corbels. This was accomplished by threading high-tensile steel wire rope through pipes embedded in the floor system and holes in the columns. The main drawback of that system was the need for a false ceiling to cover the slab ribs.

Thompson and Pessiki[9] developed a floor system of inverted-tee beams and double tees with openings in their stems to pass utility ducts. This floor system may be appropriate for parking structures, but it does not provide either the shallow floors or flat soffits desirable for multistory residential and office buildings.

Hanlon et al.[10] developed a total–precast concrete floor system for the construction of a nine-story flat-slab building. This system consisted of precast concrete stair/elevator cores, prestressed concrete beam-slab units, prestressed concrete rib-slab floor elements, variable-width beam slab, and integrated precast concrete columns with column capital. The need for special forms to fabricate these components, special handling, and high-capacity cranes for erection were the main limitations of this system.

Bellmunt and Pons[11] developed a new flooring system that consisted of a structural grid of concrete beams with expanded polystyrene (EPS) board in between. The grid had beams in two directions every 32 in. (813 mm). The floor was finished with a light paving system on top and a light ceiling system underneath. This system had many advantages, such as light weight, flat soffits, and good thermal insulation. However, some of its disadvantages included floor thickness and the unique fabrication process of EPS forms due to the special connections required.

The Peikko Group developed the Deltabeam in 1989, which is made of welded steel plates forming a hollow trapezoidal shape with holes in the side plates (webs). The bottom plate extends beyond the web to act as a ledge to support the hollow-core slabs. Once the hollow-core slabs are placed, cast-in-place concrete is placed to fill the inside of the beam, where it creates a composite topping if needed. The beam height varies based on the required span and is typically less than that of conventional precast, prestressed concrete inverted-tee beams. Deltabeam might require shoring during erection and, in some cases, adding shims to the ledge plate to raise the slabs to match the elevation of the top plate.

Design and detailing of the proposed shallow floor system

In order to present the design and detailing of the proposed shallow precast, prestressed concrete floor system, the floor

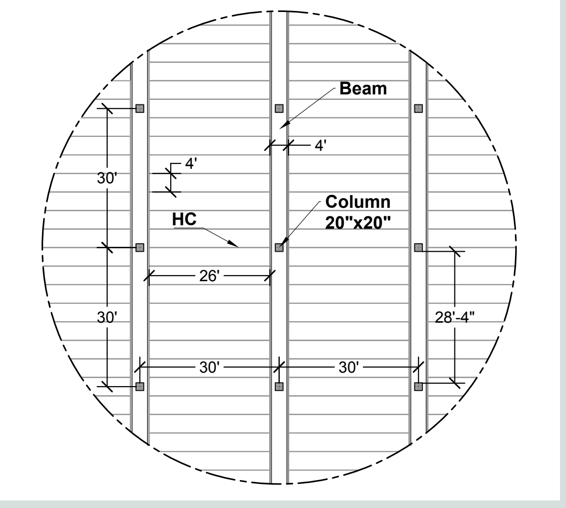

Figure 1. Floor plan of an example office building. Note: HC = hollow-core. 1″ = 1 in. = 25.4 mm; 1′ = 1 ft = 0.3048 m.

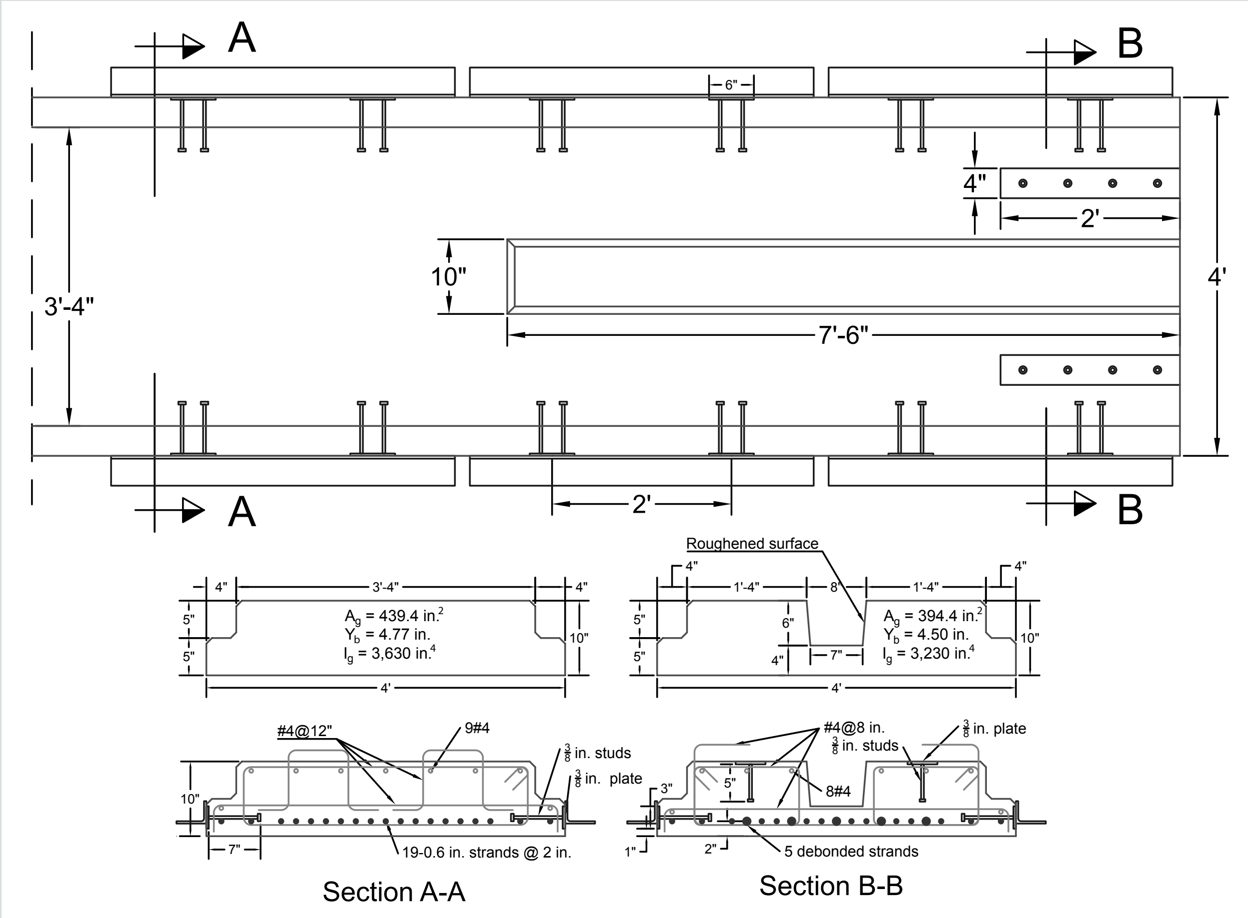

Figure 2. Plan view (top), section view (middle), and reinforcement details (bottom) of the floor beam. Note: A_g = gross area of precast concrete section; HC = hollow-core; I_g = gross moment of inertia; Y_b = distance from bottom fiber to center of gravity of the section. 1″ = 1 in. = 25.4 mm; 1′ = 1 ft = 0.3048 m; #4 = no. 4 = 13M.

plan of an example office building is employed as shown in **Fig. 1**. In this example, 20 in. (508 mm) square columns are spaced at 30 ft (9.1 m) in both directions. The proposed 10 in. (254 mm) thick floor beams are used in only one direction and supported by the columns, while the 8 in. (203 mm) thick hollow-core floor slabs are used in the other direction and supported by the beams. This layout shows that the beam width is 4 ft (1.2 m) and its length is 28 ft 4 in. (8.6 m), while the hollow-core slab width is 4 ft and length is 26 ft (7.9 m). The design of the example building for only gravity loads is presented assuming a cast-in-place concrete topping with an average thickness of 2.5 in. (63.5 mm) and a live load of 100 lb/ft^2 (4.8 kN/m^2). The beam was designed to be a simple span for its weight and the weight of the slabs and a continuous span for the topping and live load. **Figures 2** to **4** show the dimensions and reinforcement details of the precast concrete floor beam, hollow-core slab, and column components.

To achieve the structural capacity of the proposed 10 in. (254 mm) deep precast, prestressed concrete beams for the given loads and spans, the following unique features are used:

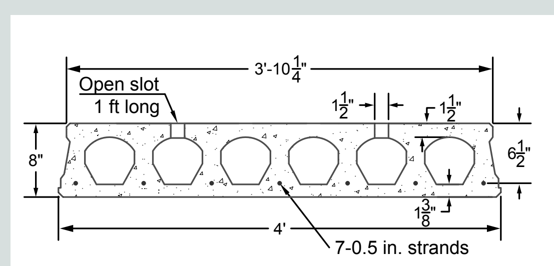

Figure 3. Cross section of a typical 8 in. thick hollow-core slab. Note: HC = hollow-core. 1″ = 1 in. = 25.4 mm; 1′ = 1 ft = 0.3048 m.

- A relatively wide 48 in. (1219 mm) beam accommodates a large number of prestressing strands in one row and maximizes prestressing eccentricity. In addition, 0.6 in. (15 mm) diameter prestressing strands are used to help concentrate the prestressing force in one row and simplify production. Figure 2 shows that only nineteen 0.6 in. diameter Grade 270 (1860 MPa) low-relaxation straight

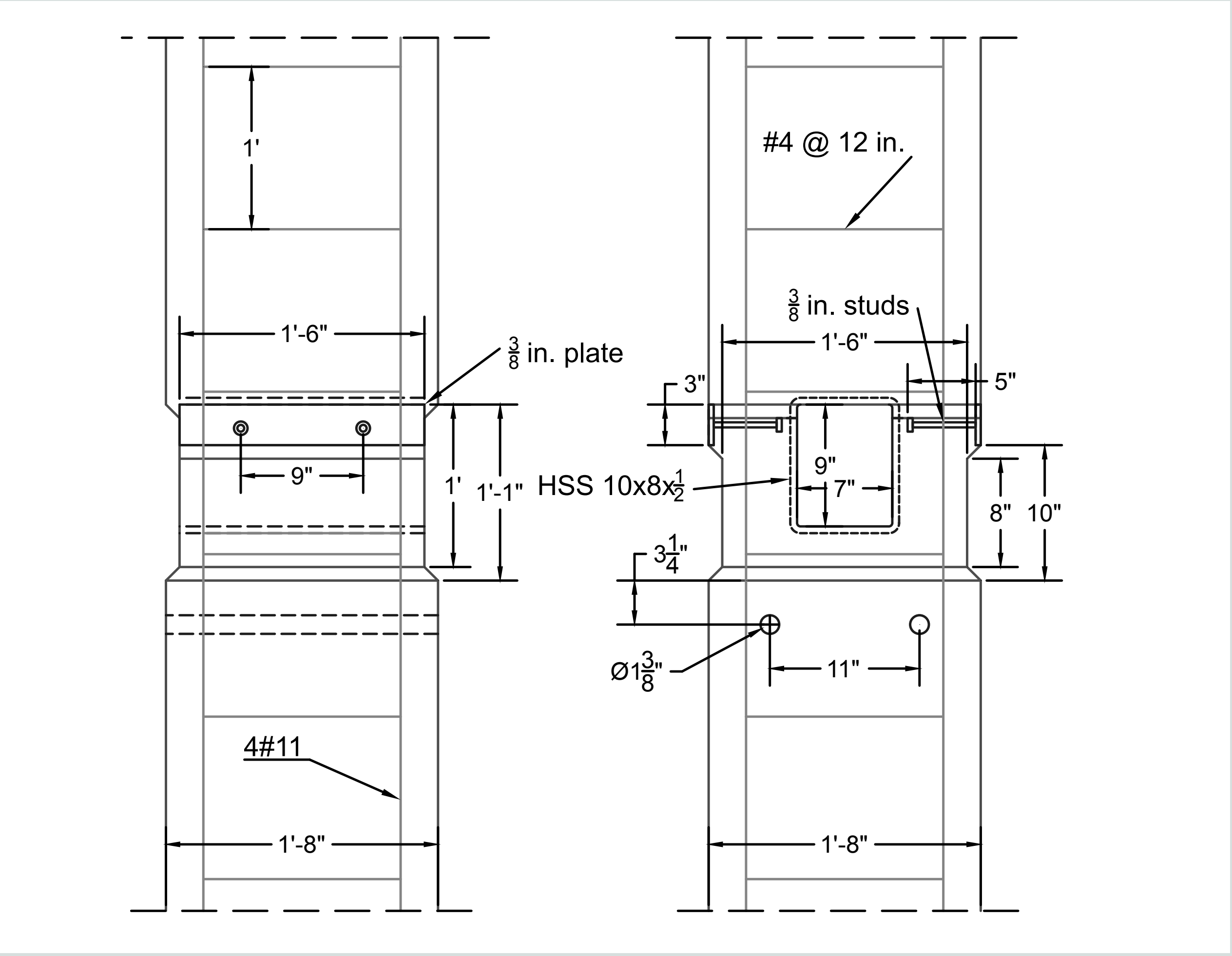

Figure 4. Dimensions and reinforcement details of the column. Note: HSS = hollow structural steel. 1″ = 1 in. = 25.4 mm; 1′ = 1 ft = 0.3048 m; #4 = no. 4 = 13M; #11 = no. 11 = 35M.

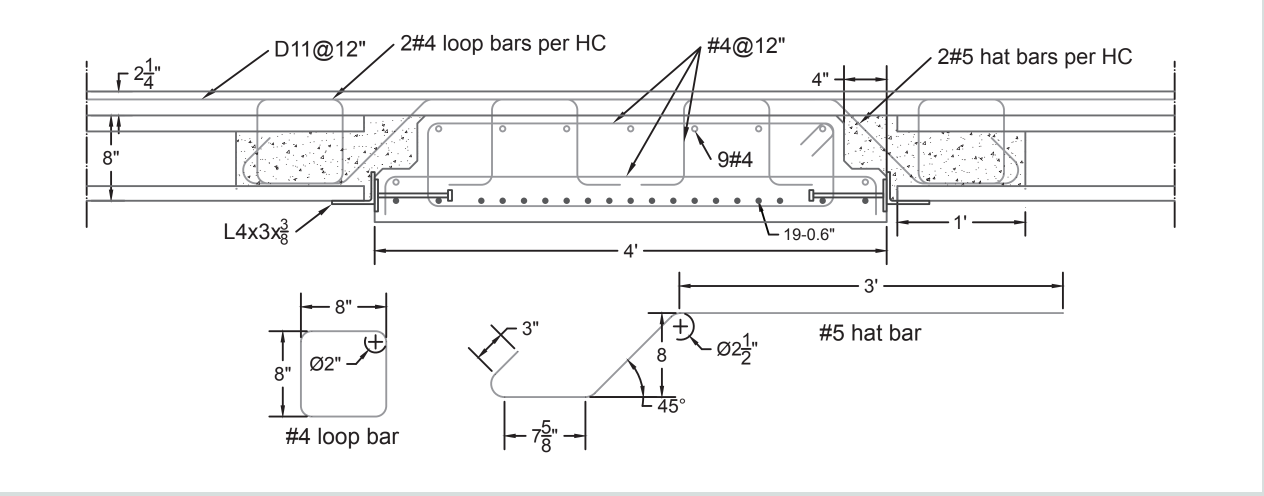

Figure 5. Beam–hollow-core connection with cast-in-place concrete topping. Note: HC = hollow-core. 1 ″ = 1 in. = 25.4 mm; 1′ = 1 ft = 0.3048 m; #4 = no. 4 = 13M; #5 = no. 5 = 16M.

strands with five debonded strands are needed in this example for the American Concrete Institute's *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)*[14] Class T. The design assumed concrete strengths of 6 and 8 ksi (41 and 55 MPa) at release and final, respectively. A summary of the beam design can be found in Morcous and Tadros.[13]

- Embedded side plates along the beam (Fig. 2) allow steel angles to be welded to the plates, forming steel ledges to support hollow-core slabs (**Fig. 5**). In addition, the hollow-core slabs (Fig. 3) have open slots in the top flange over the voids to accommodate the reinforcement required to connect the slabs to the beam and the topping (that is, loop bars and hat bars) as shown in Fig. 5. When the voids are filled during placement of the topping concrete, the resulting reinforced concrete joint is, in effect, a hidden ledge that will transfer vertical shear from hollow-core slabs to the floor beam even if the steel angles are removed. Therefore, no fire protection of the steel angles would be required because they are considered temporary. This detail was already patented in 2014 by Morcous and Tadros.[14] It should be noted that the same connection details could be used with 10 in. (254 mm) thick hollow-core slabs instead of the 8 in. (203 mm) slabs shown in this paper. In this case, where the depth of the hollow-core slabs is the same as the depth of the beams, the soffits of the two floor elements line up and the floor may be considered to have a flat soffit. This option would allow the spans to increase from 30 ft (9.1 m) to nearly 40 ft (12.2 m), making the system even more efficient.

- A continuous span for topping weight and live load improves the flexural capacity of the beam and reduces deflections. This continuity is achieved by creating openings in the columns at the beam level (Fig. 4) and troughs (that is, open channels) at the beam ends as shown in Fig. 2. The six no. 8 (25M) negative moment reinforcement required to resist the topping weight is placed in the troughs and through the column void (**Fig. 6**). These troughs are then filled with 4 ksi (28 MPa) cast-in-place concrete before placing the concrete topping, which makes the beams continuous for the topping weight. Additional continuity reinforcement, nine no. 8 (25M) reinforcing bars, is placed in the cast-in-place concrete topping to resist live-load effects. The continuity reinforcement and the cast-in-place concrete create a hidden corbel that provides interface shear resistance between the beam and column.[13] The temporary steel angle corbels may be removed after construction while maintaining adequate support capacity of the beams on the columns. Figure 6 shows a cross section at the column location with reinforcement and cast-in-place concrete topping in place.

The design of these beam–hollow-core and beam–column connections is conducted using the shear-friction design method of ACI 318-14 section 22.9.[12] Grade 60 (414 MPa) reinforcing bars and 4 ksi (28 MPa) cast-in-place concrete are used to create a shear-transfer mechanism between the precast concrete beam and column components and between precast concrete hollow-core and beam components. A coefficient of friction equal to 1.0 is used between cast-in-place concrete placed against hardened precast concrete, assuming that the contact surface is intentionally roughened. The beam–hollow-core connection is assumed to be a pinned connection, while the beam–column connection is assumed to be a moment-resisting connection, as the continuity reinforcement extends beyond the negative moment region. Flexural capacities of both the midspan and end-span sections are calculated using the strain compatibility approach. This approach assumes the following about the beam:

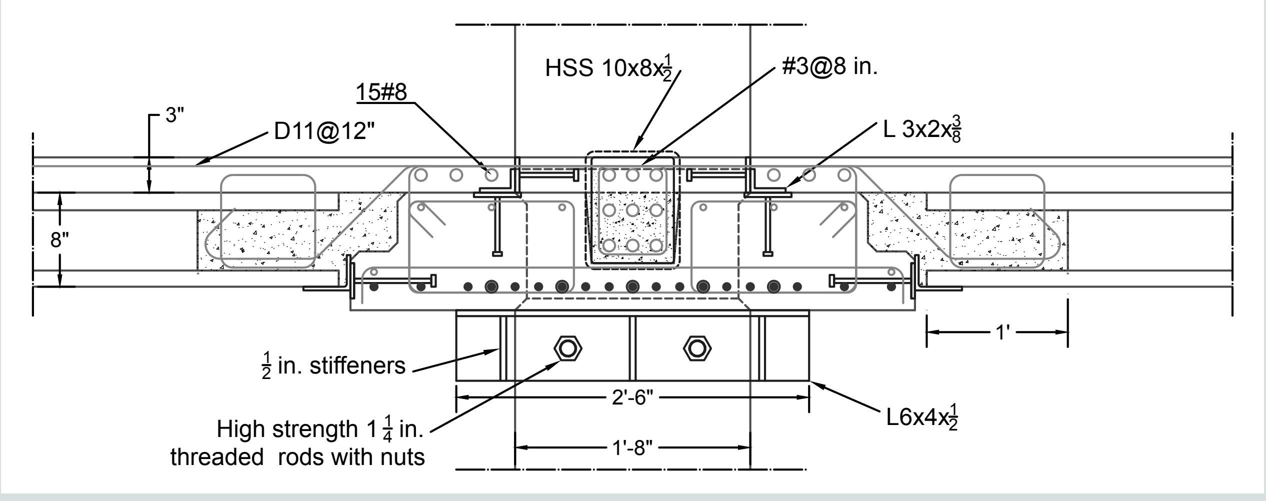

Figure 6. Beam–column connection with cast-in-place concrete topping. Note: HC = hollow-core. 1″ = 1 in. = 25.4 mm; 1′ = 1 ft = 0.3048 m; #4 = no. 4 = 13M; #5 = no. 5 = 16M.

- simply supported noncomposite for prestressing force and beam and hollow-core self-weight
- continuous noncomposite for topping weight
- continuous composite for live load and superimposed dead load[15]

Construction sequence

The construction sequence of the floor system using the proposed 10 in. (254 mm) deep beam is as follows:[16,17]

1. Multistory continuous precast concrete columns are erected, and temporary steel corbels are installed at each floor level. The temporary corbels can be steel angles with stiffeners that are anchored to the column using high-strength threaded rods through holes in the precast concrete columns.

2. Precast rectangular beams are placed on temporary corbels. Steel angles are welded to the steel plates on top of the beams and plates on the column sides to stabilize the beams during hollow-core erection. Welding may be done at the precasting plant.

3. Steel beam ledges are installed for supporting the hollow-core slabs. These ledges can be steel angles welded to the plates embedded on the sides of the beams.

4. Hollow-core slabs are placed on the steel ledges for the entire floor.

5. Specially shaped steel bars (called hat bars) are placed in the hollow-core keyways. In addition, beam continuity reinforcing bars are placed in the beam recess and through the column opening.

6. Grout or flowable concrete is used to fill hollow-core keyways, the beam recess, shear keys between hollow-core slabs and beam sides, and gaps between beam ends and column sides.

7. An additional layer of beam continuity reinforcement is placed on top of the beam through the column opening and on each side of the column. In addition, topping reinforcement is installed.

8. Cast-in-place concrete topping is placed to provide a level floor surface.

9. Temporary steel corbels and ledges can be removed after the topping concrete reaches the required strength. They can also stay in place if their appearance does not negatively affect the aesthetics of the floor.

Experimental investigation

The experimental investigation presented in this paper was carried out to evaluate the shear capacity of the ledgeless beam–hollow-core connections and corbelless beam–column connection. It should be noted that the specimens tested in this section are slightly different from those presented in Fig. 1 to 6, but the concepts used in designing and detailing the tested connections are the same.

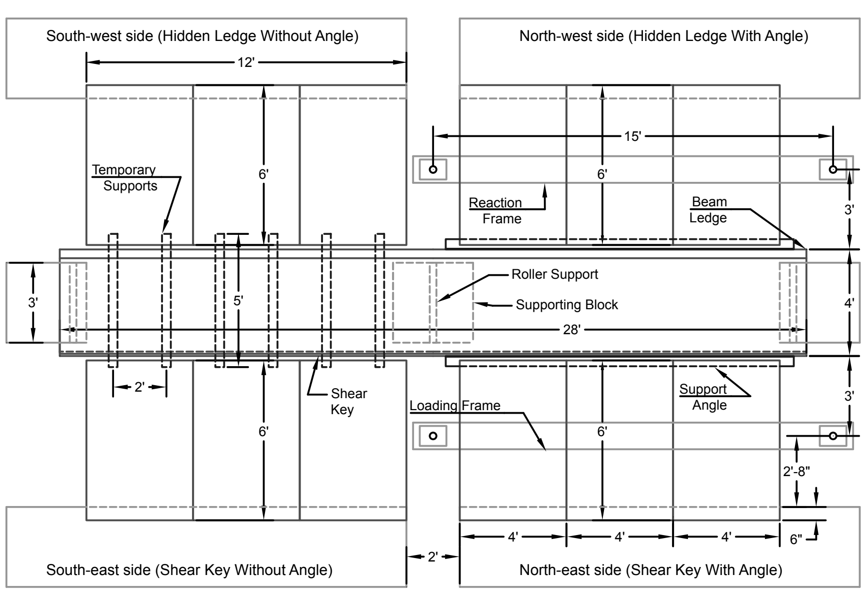

Figure 7. Plan view of the precast concrete components of test specimen. Note: 1″ = 1 in. = 25.4 mm; 1′ = 1 ft = 0.3048 m.

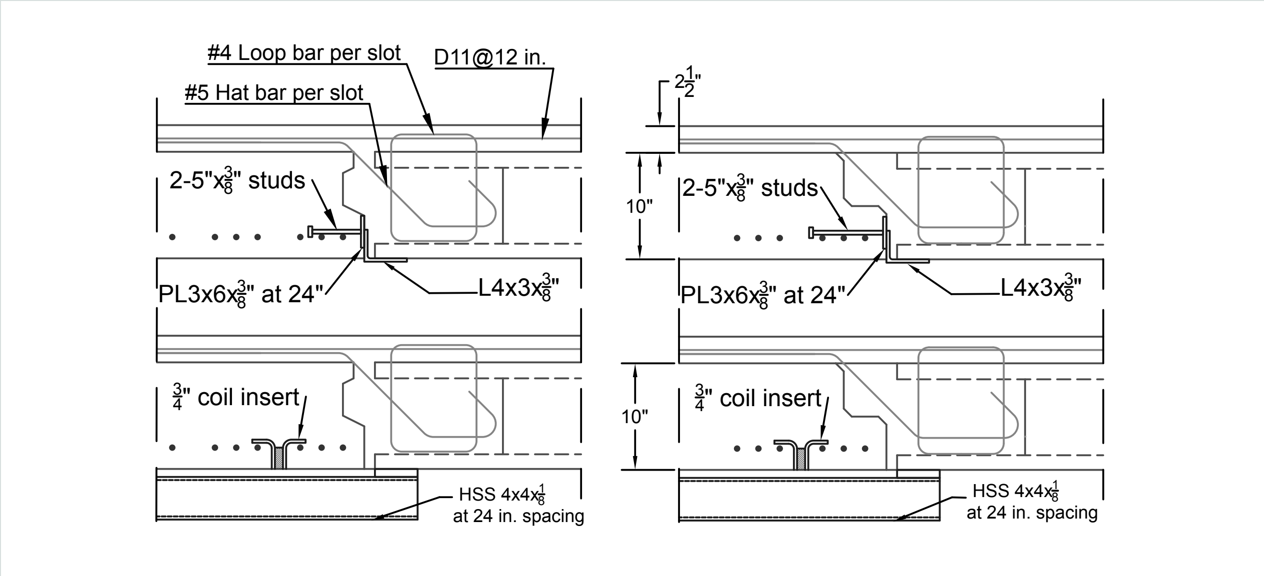

Figure 8. Details of the four tested beam–hollow-core connections. Note: HSS = hollow structural steel; PL = plate. 1″ = 1 in. = 25.4 mm; #4 = no. 4 = 13M; #5 = no. 5 = 16M.

Beam–hollow-core slab connection

The full-scale test specimen shown in **Fig. 7** consists of a 28 ft (8.5 m) long by 10 in. (254 mm) thick by 48 in. (1219 mm) wide precast concrete rectangular beam and twelve 6 ft (1.8 m) long by 10 in. thick by 48 in. wide hollow-core segments. In the test setup, the beam was supported by three roller supports (two end supports and one middle support) to minimize beam deflection while testing the capacity of beam–hollow-core connections. The beam was fabricated with two options for the ledgeless hollow-core connection: shear key and hidden ledge. For each option, two configurations were used to support hollow-core plans during construction: temporary hollow structural steel tubes (HSS) that were 4 × 4 × ¼ in. (100 × 100 × 6 mm) were attached to the beam soffit using ¾ in. (19 mm) threaded rods and coil inserts embedded in the precast concrete beam, which can be removed after the topping was hardened, and 4 × 3 × 3/8 in. (100 × 76 × 9 mm) steel L-shaped angles welded to preinstalled beam side plates that remained in the specimen during testing. These configurations

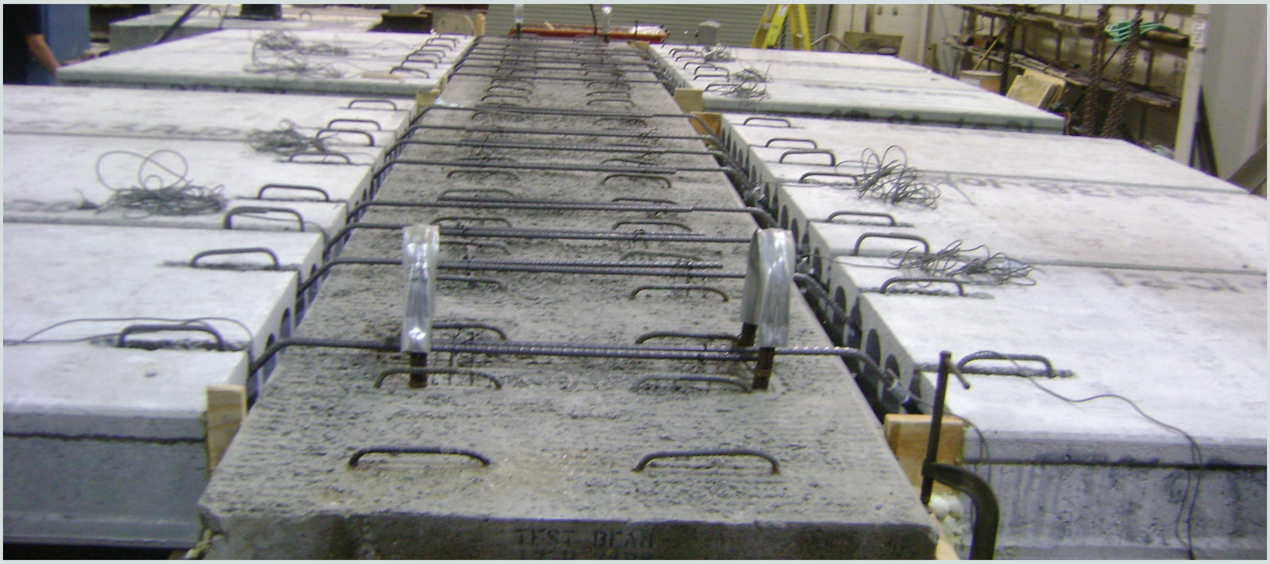

Figure 9. Installation of beam–hollow-core connection reinforcement.

Table 1. Specified and measured concrete compressive strength at time of testing

Component	Specified strength, psi	Measured strength, psi
Precast concrete	8000	9390
Grout	4000	8037
Topping	3500	5678
Note: 1 psi = 6.895 kPa.		

were used to evaluate the contribution of the temporary ledge to the capacity of the connection. **Figure 8** shows the four different combinations of beam–hollow-core connections tested in this investigation: hidden ledge with angle (northwest side), shear key with angle (northeast side), hidden ledge without angle (southwest side), and shear key without angle (southeast side). The same reinforcement was used in the four connections. Hollow-core slabs used in this specimen have two 1 ft (0.3 m) long by 1.5 in. (38 mm) wide slots in the top surface to allow placing connection reinforcement.

Figure 9 shows the specimen before placing the 2.5 in. (64 mm) thick cast-in-place concrete topping. The reinforcement of beam–hollow-core connections consisted of the hat bars and loop bars (Fig. 5). The hat bars were placed over the beam in the hollow-core slots and keyways to resist the vertical shear between the beam and hollow-core slabs. The loop bars were placed in the hollow-core slots to resist the horizontal shear between the hollow-core slabs and the topping. Twenty-four strain gauges were attached to the reinforcement (six strain gauges in each connection): three gauges to the hat bars and three gauges to the loop bars. After grouting the hollow-core keyways, slots, and shear keys, topping reinforcement was installed. Eight strain gauges were attached to the topping reinforcement (two in each connection). Finally, the concrete topping was placed and temporary ledges were removed after the concrete reached the specified strength. **Table 1** summarizes the specified and attained concrete strengths at the time of testing for the precast concrete, grout, and topping.

To evaluate the shear capacity of the proposed beam–hollow-core connections, hollow-core slabs were loaded at their midspan on one side using a loading frame while the other side of the beam was clamped using a reaction frame to maintain specimen stability. Testing was performed using two jacks applying two concentrated loads (one load per jack) to a steel spreader beam to create uniform load on the hollow-core slabs at 3 ft (0.9 m) away from the beam–hollow-core connection. Loading continued to failure while the deflection under the load was measured using a potentiometer attached to the soffit of the middle hollow-core slab.

The beam–hollow-core connection was tested in two stages. In the first stage, the hollow-core slabs were loaded up to 100 kip (445 kN), which created a shearing force at the connection of 16.5 kip (73.4 kN) per hollow-core slab. This value is the ultimate shear force due to factored dead and live loads. In the second stage, the hollow-core slabs were loaded to failure. The factored load applied to shear the beam–hollow-core connection using shear friction theory was predicted to be 209 kip (927 kN) (104.5 kip [463.5 kN] each side or 34.9 kip [154.5 kN] per hollow-core slab). In addition, the factored loads applied to fail the composite hollow-core slabs in flexure and shear were predicted to be 315 kip (140 kN) (157.5 kip [70 kN] each side or 52.5 kip [23 kN] per hollow-core slab) and 240 kip (1068 kN) (120 kip [534 kN] each side or 40 kip [178 kN] per hollow-core slab), respectively. The following subsections present the configurations of the tested connections.

Hidden ledge with angle (northwest side) Two 130 kip (578 kN) jacks were used to test the connection. In the first stage of loading, the specimen performed well under ultimate design load with no signs of failure or cracking. In the second stage, the hollow-core slabs were loaded to 257 kip (1143 kN). The test was stopped after reaching the ultimate load capacity of the used jacks. The applied load created a shear force of 42.8 kip (190.4 kN) at the beam–hollow-core connection. This value is almost 2.6 times the demand and 23% more than the design capacity of the connection. At that load, the connection did not crack, but shear cracks were observed in the other end of the hollow-core.

Shear key with angle (northeast side) Two 400 kip (1779 kN) jacks were used in this test. The specimen performed well under ultimate design load with no signs of failure or cracking. In the second stage, the hollow-core slabs were loaded to 240 kip (1068 kN) without cracking the connection. The test was stopped due to shear failure of the hollow-core slabs. The applied load created a 40 kip (178 kN) shear force on each hollow-core. This value is almost 2.4 times the demand and 15% more than the design capacity of the connection.

Hidden ledge without angle (southwest side) Two 400 kip (1779 kN) jacks were used in this test. The specimen performed well under ultimate design load with no signs of failure or cracking. In the second stage, the hollow-core slabs were loaded up to 203 kip (903 kN) without cracking the connection. The test was stopped because of shear failure of the hollow-core slabs. The applied load created 33.8 kip (150.3 kN) shear force on each hollow-core. This value is almost 2.05 times the demand and very close to the design capacity of the connection.

Shear key without angle (southeast side) Two 130 kip (578 kN) jacks were used in this test. The specimen performed well under ultimate design load, with no signs of failure or cracking. In the second stage, the hollow-core slabs were loaded up to 227 kip (1010 kN) without cracking the connection. The test was stopped due to shear failure of the hollow-core slabs. The applied load created 37.8 kip (168.1 kN) shear force on each hollow-core. This value is almost 2.3 times the demand and 8% more than the design capacity of the connection.

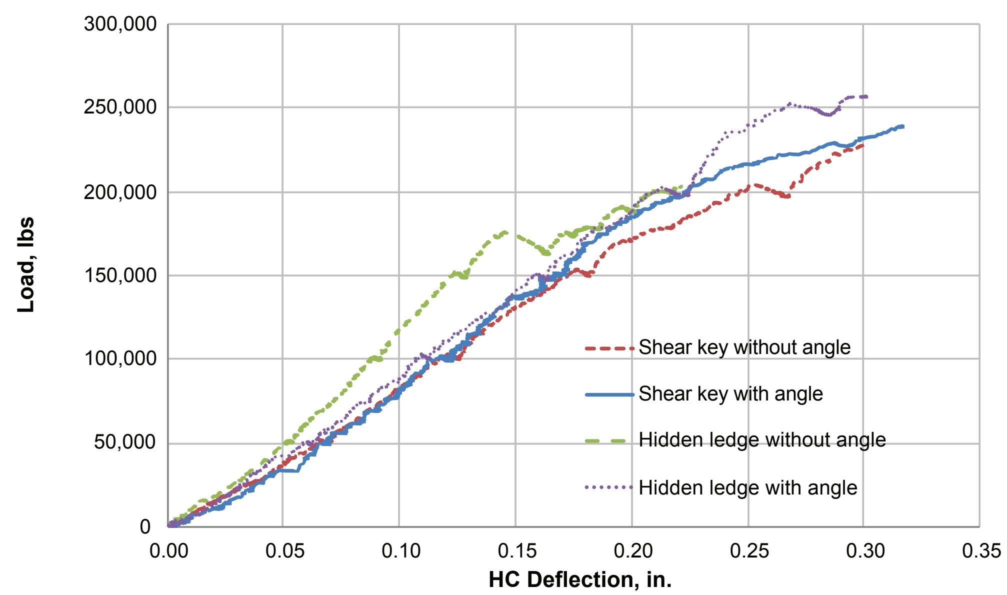

Figure 10. Load-deflection relationships for the four beam–hollow-core connection tests. Note: HC = hollow-core. 1 in. = 25.4 mm; 1 lb = 4.448 N.

Figure 10 presents the load-deflection relationships of the four tested connections (Tests A through D). These relationships indicate that all connections behaved similarly regardless of the presence of temporary ledges, which indicates the adequacy of the connection even after the temporary ledges are removed.

Hidden ledge without angle by loading the hollow-core as cantilever (southwest side) In the previous tests, load was applied at the midspan of the hollow-core, and the failure mode was shearing hollow-core slabs without cracking the connections. Therefore, to investigate the shear capacity of the connection, hollow-core slabs were loaded

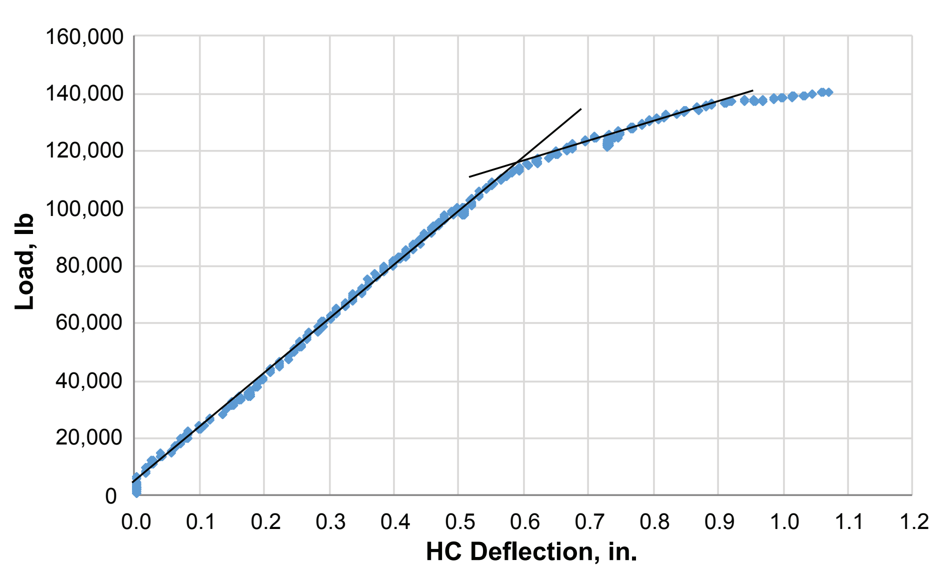

Figure 11. Load-deflection curve for the beam-hollow-core connection loaded as cantilever. Note: HC = hollow-core. 1 in. = 25.4 mm; 1 lb = 4.448 N.

Table 2. Summary results of hollow-core–beam connection tests

<table>
<tr><th>Test ID</th><th>Test title</th><th>Applied load, kip</th><th>Hollow-core connection measured capacity, kip</th><th>Hollow-core connection design capacity, kip</th><th>Hollow-core shear demand, kip</th><th>Hollow-core shear capacity, kip</th><th>Observation</th></tr>
<tr><td>A</td><td>Hidden ledge with angle</td><td>257</td><td>42.8</td><td rowspan="5">34.9</td><td rowspan="5">16.5</td><td rowspan="5">40.0</td><td>Test stopped because the capacity of the loading jacks was reached</td></tr>
<tr><td>B</td><td>Shear key with angle</td><td>240</td><td>40.0</td><td>Hollow-core shear failure</td></tr>
<tr><td>C</td><td>Hidden ledge without angle</td><td>203</td><td>33.8</td><td>Hollow-core shear failure</td></tr>
<tr><td>D</td><td>Shear key without angle</td><td>227</td><td>37.8</td><td>Hollow-core shear failure</td></tr>
<tr><td>E</td><td>Hidden ledge without angle (hollow-core loaded as cantilever)</td><td>147</td><td>49.0</td><td>Hollow-core shear failure in the clamped side and several cracks in the connection</td></tr>
</table>

Note: The average was 40.7, the standard deviation was 5.69, and the coefficient of variation was 14.0%. 1 kip = 4.448 kN.

as cantilever in this test by removing the concrete block supporting the free end of the hollow-core slabs. The hollow-core slabs were loaded at the midspan as before while clamping the other end to maintain specimen stability. Testing was performed without temporary steel angles by applying a uniform load on the cantilevered hollow-core at 4 ft (1.2 m) from the center of the beam while measuring the deflection at midspan of the hollow-core. The reaction frame was clamping hollow-core slabs at 5 ft (1.5 m) from the center of the beam.

Figure 11 shows the load-deflection relationship. This plot indicates that the three composite hollow-core slabs on the southwest side were able to carry 140 kip (623 kN), which corresponds to a total shear force of 147.7 kip (657 kN), includ-

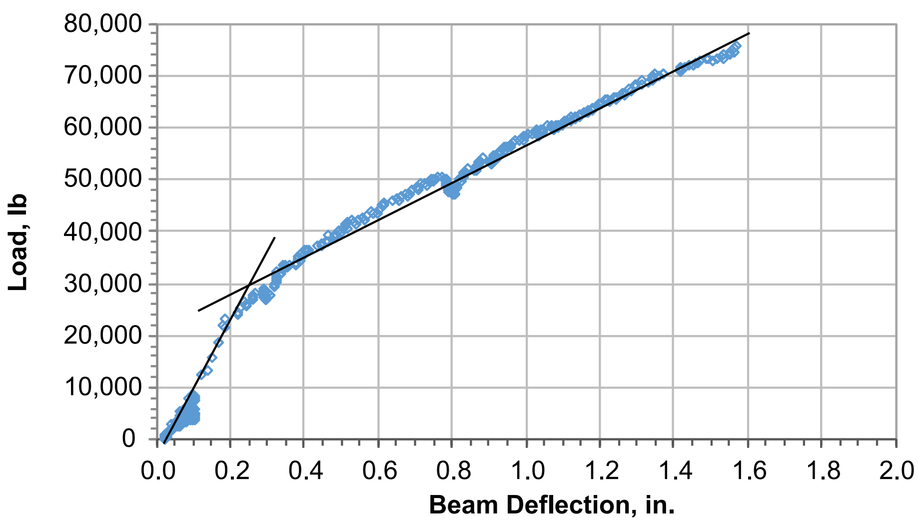

Figure 12. Load-deflection relationship of beam negative moment testing. Note: 1 in. = 25.4 mm; 1 lb = 4.448 N.

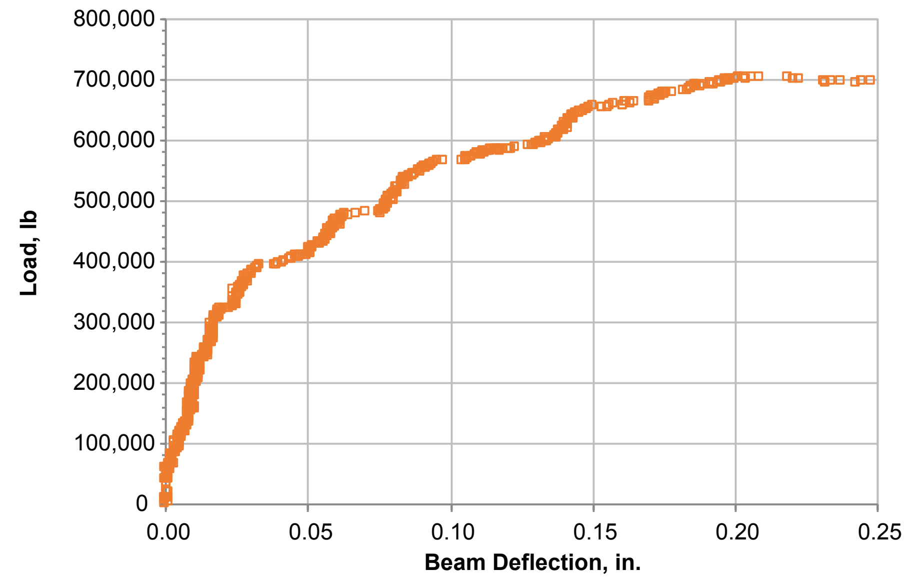

Figure 13. Load-deflection relationship for beam–column connection shear testing. Note: 1 in. = 25.4 mm; 1 lb = 4.448 N.

ing the self-weight of the hollow-core and topping (49.2 kip [218.8 kN]) per hollow-core slab. This is almost three times the demand and 40% more than the design capacity of the beam–hollow-core connection. The test was stopped due to shear failure of the hollow-core at the clamped side and severe cracking of the connection. **Table 2** summarizes the results of testing beam–hollow-core connections and the predicted capacity.

Beam-column connection

Two tests were conducted to evaluate the performance of the corbelless beam–column connection shown in Fig. 6: testing the beam continuity system to evaluate the negative moment capacity at the connection and testing the shear capacity of the connection after removing the temporary corbel. For the first test, the load was applied at the unsupported end of the beam (in other words, cantilevered) while the other end was clamped to prevent tipping over. One 400 kip (1779 kN) jack was used to apply a concentrated load to the beam at 9 ft (2.7 m) from the center of the column while the deflection of the cantilevered end was measured. **Figure 12** shows the load-deflection relationship for this test. This plot indicates that the beam had a cracking load of approximately 30 kip (133 kN) and an ultimate load of 76 kip (338 kN), which corresponds to a negative moment at the critical section of 672 kip-ft (911 kN-m), including self-weight. The predicted capacity of the composite beam using strain compatibility analysis was found to be 667 kip-ft (904 kN-m).

For the second test, the beam was supported from both ends and loaded symmetrically at 3 ft (0.9 m) from the center of the column on each side. Two 400 kip (1779 kN) loading jacks and two 12 in. (305 mm) square loading plates were used to apply the load on the top surface of the concrete topping to failure. **Figure 13** shows the load-deflection curve of that test. This curve indicates that the maximum load was 704 kip (3131 kN), which resulted in a shear force (627 kip [2789 kN]) that was significantly higher than the demand of the example building when loaded with 100 lb/ft^2 (4.8 kN/m^2) live load (308 kip [1370 kN]). The nominal shear capacity calculated using shear friction theory was found to be 614 kip (2731 kN). The measured capacity was close to the theoretical nominal capacity. It should be noted that this high shear capacity was achieved despite the fact that the specimen was already cracked in the first test.

Research implementation

The proposed shallow floor system was implemented in the construction of the Farmers Mutual Building, located at 1220 Lincoln Mall in Lincoln, Neb. This five-story office building was jointly designed by e.construct.USA LLC and Concrete Industries Inc. All interior beams were 25 ft (7.6 m) long shallow inverted-tee beams (30IT15) that were supported on temporary steel corbels and made continuous through column openings similar to the proposed floor system. The floor was designed for 100 lb/ft^2 (4.8 kN/m^2) live load, and the beams were made of 8000 psi (55,160 kPa) concrete pretensioned with fourteen 0.5 in. (12.7 mm) diameter Grade 270 (1860 MPa) low-relaxation strands. A 2 in. (51 mm) thick composite topping made of 4000 psi (27,580 kPa) cast-in-place concrete was added after all precast concrete components were erected. The beam pocket was reinforced using nine no. 8 (25M) Grade 60 (414 MPa) bars in three rows and filled with 5000 psi (34,475 kPa) flowable concrete. The temporary corbel sleeves were filled with 8000 psi (55,160 kPa)

Figure 14. Farmers Mutual Insurance Office Building in Lincoln, Neb., (top) and beam–column connection (bottom). Courtesy of Concrete Industries Inc.

grout to match the strength of the precast concrete column after the steel angles were removed. The exterior beams were conventional rectangular and L-shaped beams that were simply supported on column corbels and are hidden in the exterior walls. **Figure 14** shows the building during construction. The precaster and contractor expressed satisfaction with the simplicity, efficiency, and economy of the proposed framing system.

Conclusion

Current precast concrete floor systems require the use of inverted-tee beams with ledges to support hollow-core slabs and column corbels to support beams, which results in a deep floor system that reduces the clear floor height in addition to the already low span-to-depth ratio. The proposed floor system with 10 in. (254 mm) deep floor beams and either 8 in. (203 mm) or 10 in. thick hollow-core slabs corresponds to a total floor depth of 12 to 13 in. (305 to 330 mm). The relatively deep concrete beam ledges and column corbels are eliminated. Economic and structural efficiency, ease and speed of construction, quality, and aesthetics are the main advantages of the proposed system. Shallow floor depth results in additional savings in mechanical, electrical, and architectural systems. This paper presents details of the beam design and the connections with hollow-core slabs and columns. Full-scale testing of beam–hollow-core connections and beam–column connections was conducted to evaluate the behavior and shear capacity of these connections. Based on analytical and experimental results, the following conclusions can be made:

- A 10 in. (254 mm) deep precast, prestressed concrete beam can be designed to resist the gravity loads for an office building with 30 ft (9.1 m) bay size and 100 lb/ft^2 (4.8 kN/m^2) live load using prevailing production and erection techniques and commonly used 8 in. (203 mm) thick hollow-core slabs.

- The proposed beam–hollow-core connections performed well because the shear capacity exceeded the predicted values and significantly exceeded the demand. None of these connections failed, and the tested hollow-core slabs instead failed in shear prior to the failure of the connections. The capacity of the proposed beam–hollow-core connections can be predicted using the shear friction design method from ACI 318-14.[12]

- The proposed beam–column connection performed very well because it had adequate flexural capacity to achieve beam continuity and shear capacity to transfer shear without temporary steel corbels. These capacities can be accurately predicted using the strain compatibility and shear friction design methods from ACI 318-14,[12] respectively.

- Because the shear capacity of the test connections without temporary ledges or corbels was adequate, steel angles can removed or left in place without affecting the fire rating of the building.

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Notation

A_g = gross area of precast concrete section

I_g = gross moment of inertia

Y_b = distance from bottom fiber to center of gravity of the section

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Abstract

The use of shallow floor systems in office buildings is desirable because it reduces the overall building height and saves on the cost of architectural, mechanical, and electrical building systems. Precast, prestressed concrete floors consisting of hollow-core slabs on inverted-tee beams are known for their superior quality and speed of construction. However, when the depth of the hollow-core slab, inverted-tee beam ledge, and column corbel are added, the total floor depth becomes significantly larger than that of cast-in-place, post-tensioned slabs.

This paper presents a system by which the hollow-core slabs are framed next to the beam rather than on top of a ledge, and the beams are framed into the column without the aid of a permanent concrete corbel. For the development of the system, a 30 by 30 ft (9.1 by 9.1 m) bay size is considered typical for office floors. Hollow-core slabs that are 8 in. (203 mm) deep are supported on 10 in. (254 mm) deep beams using a new beam–hollow-core connection that is designed using shear-friction theory. Methods of temporary support until the composite topping is cured are presented. Full-scale testing confirmed satisfactory performance.

A beam–column connection is also developed using column recesses at the beam location and reinforcing bars through a void in the column to allow the beam to be continuous and its reaction to be resisted by the column without the conventional corbel. A temporary steel angle support is used until the connection grout is hardened. Full-scale testing of the beam–column connection showed excellent behavior. The main advantages of this shallow floor system are its high span-to-depth ratio (up to 30) and its efficient and economical production and erection techniques. Some of the features of the developed system were implemented in a four-story office building in Lincoln, Neb. Experience with this application is also discussed.

Keywords

Beam ledge, column corbel, hollow-core, shallow floor system.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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