PRECAST CONCRETE SANDWICH PANELS FOR FLOOR AND ROOF APPLICATIONS

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ABSTRACT

Structural floor systems represent a major portion of both the cost and weight of precast concrete building frames. Hollow cores (HC) are considered one of the most common precast floor systems due to their advantages in terms of economy, lightweight, structural capacity, and ease of production and erection. This paper presents the development of a new precast/prestressed floor system that is alternative to HC planks. The proposed floor system consists of sandwich panels that have comparative weight and structural capacity to HC planks while being efficient in thermal and sound insulation. These panels can be easily produced as they do not require specialized equipment for fabrication, which eliminates the need for high initial investment. The proposed floor panels consist of an internal wythe of insulation and two external wythes of concrete similar to precast concrete sandwich wall panels. The two concrete withes are designed to be fully composite through the use of shear connectors. Two types of shear connectors are presented 1) glass fiber-reinforced polymer GFRP ties for roof applications; and 2) steel ties for floor applications.

This paper presents the design and detailing of two 26 ft long and 4 ft wide specimens that were produced and tested at the Structural Laboratory of the University of Nebraska - Lincoln. One panel was produced using GFRP ties and without any intermediate concrete connectors (fully insulated), while the other panel was produced using steel ties and intermediate concrete connectors. The two specimens were tested in flexure under point load at the mid span. The load-deflection relationships have indicated that the two panels have full composite action and comparative structural performance to the HC planks.

Keywords: Sandwich Panel, Hollow Core, Shear Connector, Composite Action, Flexural Capacity.

INTRODUCTION

Structural floor systems represent a major portion of both the cost and weight of precast concrete building frames. Also, structural floor systems in multi-story buildings have an impact on the overall building height and design of other building systems. Many approaches have been used to improve the structural and construction efficiency of floor systems, some of these were sought to minimize the weight, depth, and cost of structural floor systems through the use of higher strength materials and improved construction techniques.

Hollow core (HC) precast prestressed concrete floor systems[1] are the common solution for several floor applications, especially where flat soffit, long span, and light weight floors are required. The number and size of strands in the bottom flange determine the ultimate load/span capacity of the planks. HC planks are produced using specialized equipment to ensure consistently, high quality, and efficiency of production. HC planks are grouted together to produce a diaphragm action and flat soffit. Enhanced structural performance can be achieved by using a composite topping, which can result in a span-to-depth ratio of up to 40. Despite these advantages, HC planks have poor thermal and sound insulation, and require high initial investment for production equipment.

Rip-slab floor[2] is a modified precast prestressed concrete double-tee with a 2 in. thick concrete slab and 8 in. deep ribs, for a total depth of 10 in. Testing the ultimate load capacity of the rib-slab with a dapped end connection has confirmed the feasibility of this floor system. The Rip-slab floor elements are economical, structurally efficient, and can be easily produced. However, they do not provide either flat soffit or thermal and sound insulation.

Filigree wide slab system[3] was originally developed in Great Britain and is presently used under the name of OMNIDEC. Filigree precast panels are thin reinforced concrete slabs (can be pretensioned[4]) with steel lattice truss that are used as formwork for the composite cast-in-place concrete topping The steel truss ensures composite behavior between precast and cast-in-place concrete and provides the panel with the required stiffness during erection. The typical thickness of the prefabricated slab is 2.25 in., but the total thickness of the panel varies. The panels are structurally efficient and easy to produce. They have a typical width of 8 ft and flat soffit that eliminates the need for false ceiling. The main disadvantage of this system is the low thermal and sound insulation.

This paper presents the development of a new precast/prestressed floor system that is alternative to HC planks. The proposed floor system is expected to have flat soffit, light weight and adequate structural capacity while being efficient in sound and thermal insulation and does not require specialized equipment for fabrication. Table 1 compares the proposed floor system with the existing systems in terms of the criteria listed before. The proposed system consists of an internal wythe of insulation and two external wythes of concrete similar to precast concrete sandwich wall panels. The two concrete wythes are designed to be fully composite through the use of shear connectors. In this study, two types of shear connector were used: 1) GFRP ties, when thermal insulation is needed as in roof applications; 2) steel ties, when thermal insulation is not a priority as in most floor applications. The design, fabrication, and testing of the proposed panels is presented in the following sections.

Table 1: Comparing the proposed against existing floor systems

Criteria	Hollow core	Rip-slab	Filigree wide slab	Proposed Sandwish Floor Panel
Does not Need Special Equipment to Produce	✗	✓	✓	✓
Does not Need Cast-in-place Topping	✓	✓	✗	✓
Have adequate Thermal Insulation	✗	✗	✗	✓
Have adequate Sound Insulation	✓	✗	✓	✓
Have Flat Soffit	✓	✗	✓	✓

DESIGN APPROACH

The proposed panel is designed to be fully composite. The flexural capacity of the composite panel is that of a solid panel that has the same cross section as the two concrete wythes. Shear connectors are used to transfer horizontal shear forces between the concrete wythes. This force can be calculated using the strength method given in the PCI Design Handbook, 7th Edition 2010 Section 5.3.5 “Horizontal Shear Transfer in Composite Components”[5]. In this method, the horizontal shear force is taken as the lesser of the maximum compressive force in concrete and maximum tensile force in the reinforcement/prestressing. This force is then used to determine the required number of shear connectors over the horizontal shear span, which is one-half the clear span for simply supported panels. Most manufacturers of shear connectors use the same method to determine the amount of shear connectors for composite panels and distribute these connectors uniformly along the horizontal shear span. In this study, a triangular distribution of the horizontal shear force along the shear span is used to determine the most efficient distribution of shear connector. In addition, the flexural capacity was determined using the strain-compatibility for two loading stages: 1) the panel without topping was designed to carry the topping weight (25 psf) plus the weight of construction loads (50 psf); and 2) the panel with topping was designed to carry the live load (100 psf) plus any superimposed dead loads (weight of flooring or ceiling).

EXPERIMENTAL INVESTIGATION

Two panels were fabricated and tested at the Structural Laboratory of the University of Nebraska-Lincoln. Each panel was 26 ft long, 4 ft wide, and 8 in. thick. Both Panels were longitudinally reinforced with seven 0.6 in. diameter grade 270 low-relaxation prestressing strands tensioned to 31 kips, which is the maximum jacking force for 0.5 in. diameter strands. The researchers used 0.6 in. diameter due to the unavailability of 0.5 in. diameter strands at the time of panel fabrication. The 8 in. thick sandwich panels consisted of two concrete wythes. The top concrete wythe is 1 in. thick and the bottom concrete wythe is 3 in. thick and they are separated by a 4 in. thick layer of extruded polystyrene (XPS) as shown in Fig. 1 and 2. Glass Fiber-Reinforced Polymer (GFRP) ties were used in panel 1 as shear connectors in addition to 12 in. wide solid concrete block at each end as shown in Fig. 1. Steel ties and concrete connectors were used in panel 2 as shear connectors. The concrete connectors were 9 in. wide solid block at each end, 3 in. wide rip in each side, and two 3 in. wide rips 8.75 ft apart from each end in addition to the gap between the steel ties and the insulation.

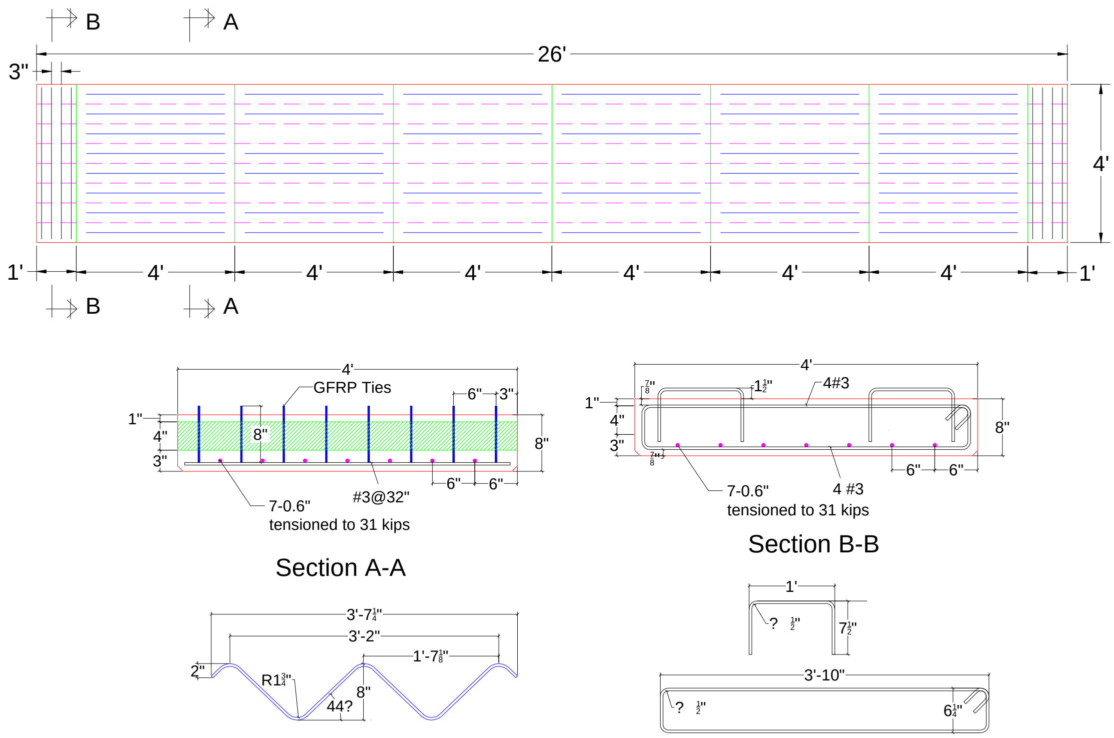

Figure 1: Floor panel 1 with GFRP ties

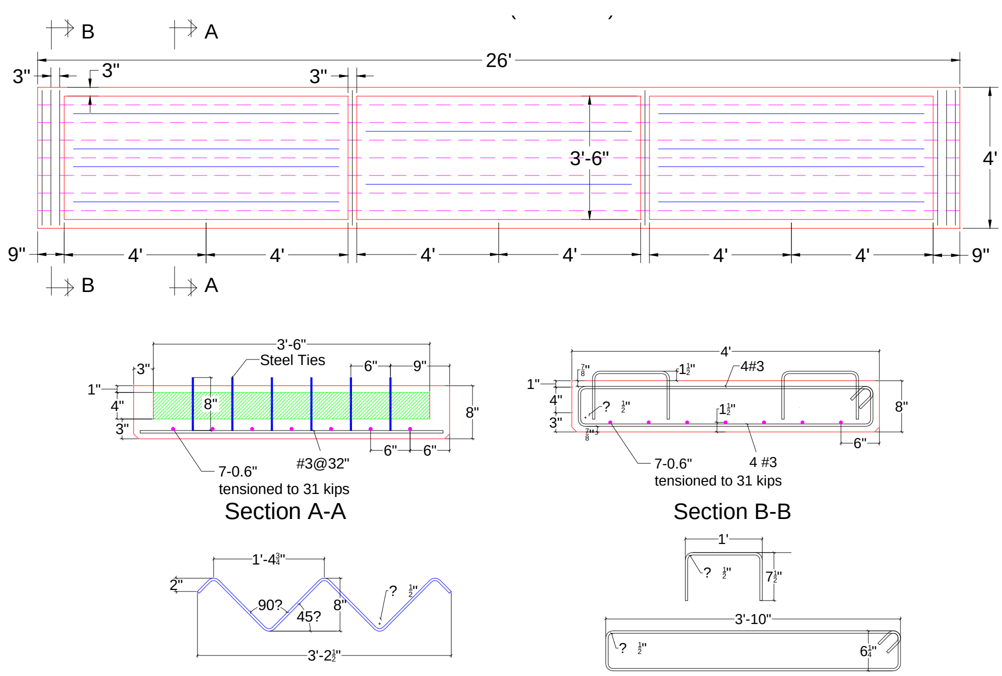

Figure 2: Floor panel 2 with steel ties

The fabrication process of the two panels includes: production of GFRP and steel ties, preparation of XPS foam panels, and casting of concrete wythes. The preparation of the XPS foam panels includes: melting slots for inserting ties, inserting GFRP ties into the foam slots, and filling the remaining gaps with canned expandable foam as shown in Fig. 3. Excess foam is removed with a long, flat fine tooth blade.

The fabrication of the floor panels proceeded as follows: first, chamfers were stapled to the bed at the appropriate spacing; second, the seven 0.6 in diameter strands were threaded through the bulkheads and prestressing abutments; third, strands were chucked at both ends and tensioned to 31 kip, forth, self-consolidating concrete with 25 in. spread was delivered by ready mix for casting the bottom and top wythes as shown in Fig. 4; and fifth, after the concrete reached the required release strength (6 ksi), forms were stripped and strands were released gradually.

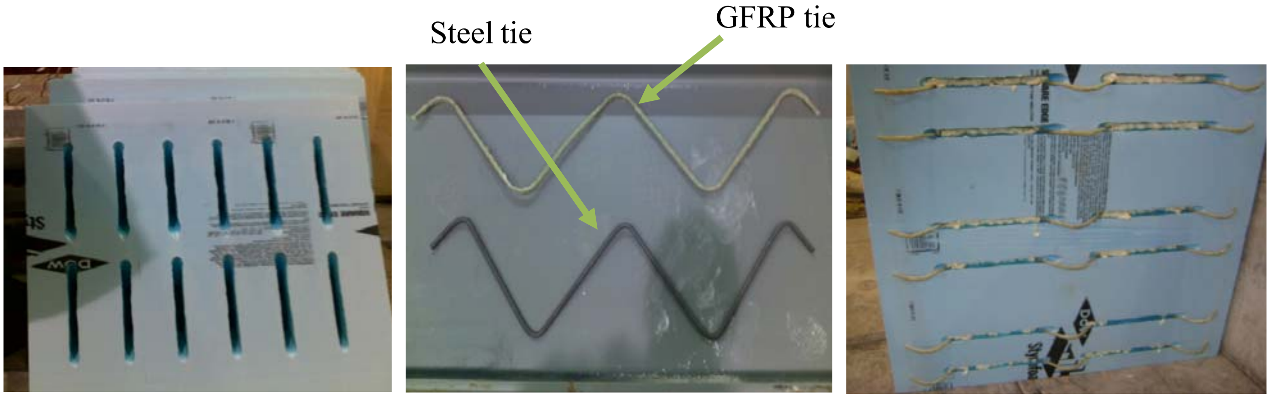

Figure 3: Inserting ties into slotted foam and filling the gap with expandable foam

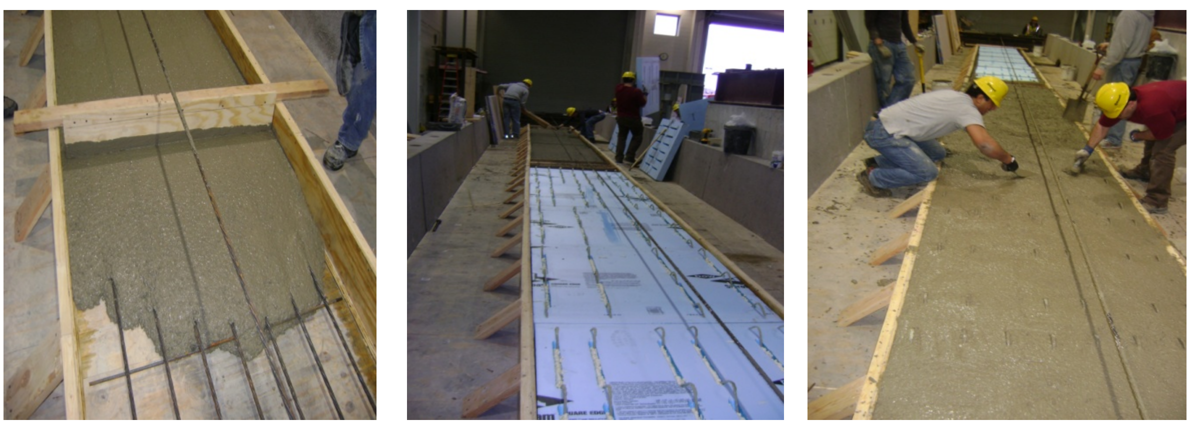

Figure 4: Casting the bottom wythe, installing foam panels, and casting the top wythe

FIRST TEST (WITHOUT TOPPING*)*

The first test was conducted to determine the behavior of the panels without topping. At the time of the test, the concrete strength was 9.6 ksi. One point load was applied at mid-span of the panel using hydraulic jack and load cell. Roller supports were placed 25.67 ft center to center. Specimen deflection was recorded using one potentiometer located at mid-span under the point load as shown in Fig. 5. The net camber (after subtracting the self-weight deflection) of the two panels was approximately 0.25 in.

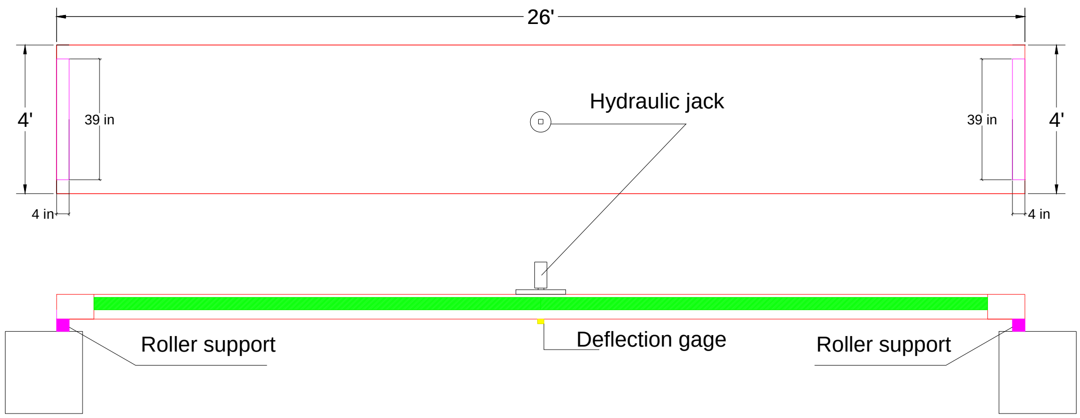

Figure 5: Test Setup

Fig. 6 plots the load-deflection relationships of the two panels when loaded up to the cracking load. The plot indicates that the relationship of panel 1 is linear, and the relationship of panel 2 is non-linear. Panel 1 showed a higher level of ductility than panel 2, which can be explained by the fact that GFRP ties have significantly lower modulus of elasticity than steel ties, which allows higher relative movements between the top and bottom concrete wythes. In addition, panel 2 has more concrete connectors that restrain this relative movement.

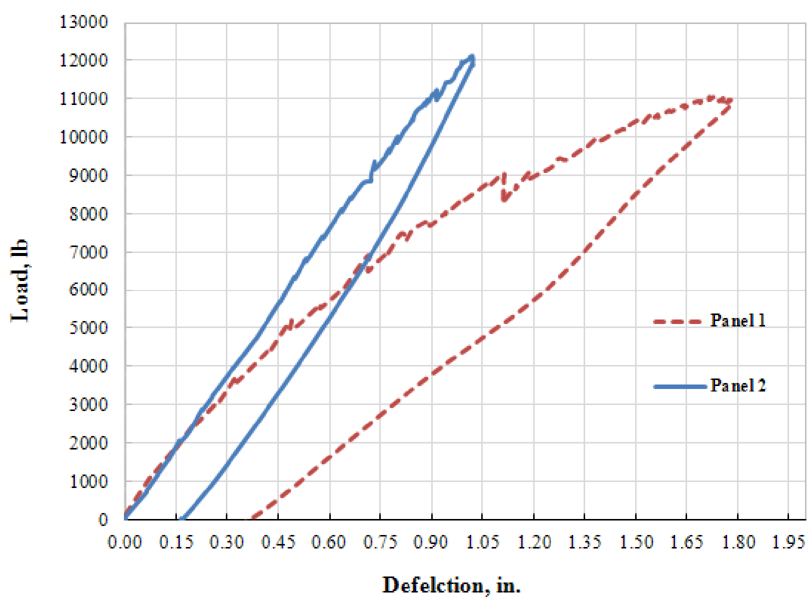

Figure 6: Load-deflection relationships for the two panels without topping

THE SECOND TEST (WITH TOPPING)

The second test setup is similar to the first one except that it was conducted after casting 2 in. concrete topping over the top of the two panels. Concrete strain gauges were attached to the top surface to measure the strain in extreme compression fibers. Fig. 7 shows the load deflection relationships of the two panels. In this figure, the left vertical axis shows the applied load in pounds, while the right axis shows the corresponding uniform load (i.e that results in similar deflection) in pound per square foot. The load-deflection relationships show a linear behavior up to the cracking load, which was approximately15 kip for the two panels. A non-linear relationship continued until the ultimate load was reached, which was approximately 33, kip for panel 1 and 34 kip for panel 2. It should be noted that the point load equivalent to a live load of 100 psf is 5.2 kip and the corresponding deflection is 0.18 in.

For strength and prestress loss calculations, the following values of the concrete compressive strength for the two panels were used: 8.4 ksi at release, and 10.8 ksi at final, and 3.4 ksi for the topping. These values represent the average compressive strength of the tested cylinders at release and at the testing time. Prestress loss calculations were performed according to the 7th Edition of the PCI Design Handbook (2010), which resulted in a total prestress loss of approximately 18%. The nominal flexural capacity of the panel section (ΦM_n) was calculated using strain compatibility and assuming a fully composite section and a resistance factor (Φ) of 1.0. This resulted in a theoretical capacity of 226 kip.ft, depth of compression block of 2.224 in, and ultimate stress in prestressing strands of 270 ksi. It should be noted that the two panels were made of the same concrete and had the same prestressing force.

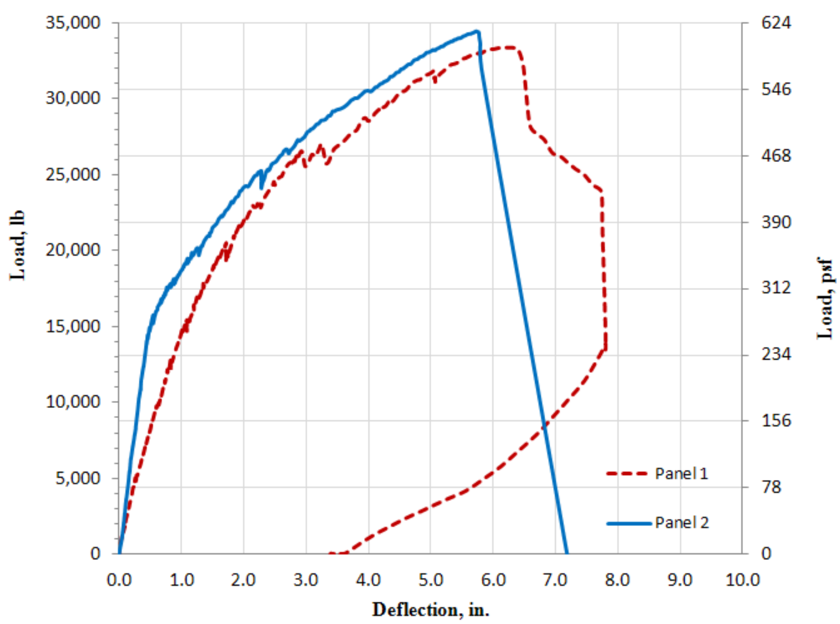

Figure 7: Load-deflection relationship for the two panels with topping

Table 2 compares the theoretical flexural capacity of each specimen with its measured flexural capacity obtained from testing. The ratios of measured-to-theoretical capacity indicate that panels 1 and 2 have flexural capacity higher than the theoretical capacity of a fully composite section. This means that the section is fully composite. The ratios of measured -to-theoretical capacity in Table 2 also indicate that GFRP ties in panel 1 and steel ties in panel 2 have achieved the full composite action.

Table 2: Comparing the theoretical against measured flexural capacity of test specimens

Panel	L_e (in.)	$M_{theoretical}$ (kip. In.)	$W_{O.W}$ (kip/in.)	$M_{O.W}$ (Kip.in.)	$P_{measured}$ (kip)	$M_{measured}$ (kip. in.)	$M_{total\text{-}measured}$ (kip. in.)	$M_{total.measured}$ / $M_{theoretical}$
Panel 1	308	2712	0.026	308.3	33.4	2571.8	2880.1	1.06
Panel 2	308	2712	0.028	332.0	34.5	2656.5	2988.5	1.10

Fig. 8 shows load strain relationships of the two panels at top fiber. The strain at mid-span top fibers in panel 1 indicates that the concrete strain didn't reach 0.003, while it reached 0.003 in panel 2. This behavior explains the failure mode of each panel, which is shown in Figures 9 and 10. Fig. 9 shows that Panel 1 had tension-controlled flexural failure. Also several cracks appeared in the top surface at each ends, where the concrete end blocks restrained the panel rotation (i.e. partial fixity). Fig.10 shows that panel 2 has compression-controlled flexural failure as the topping concrete reached its ultimate strain.

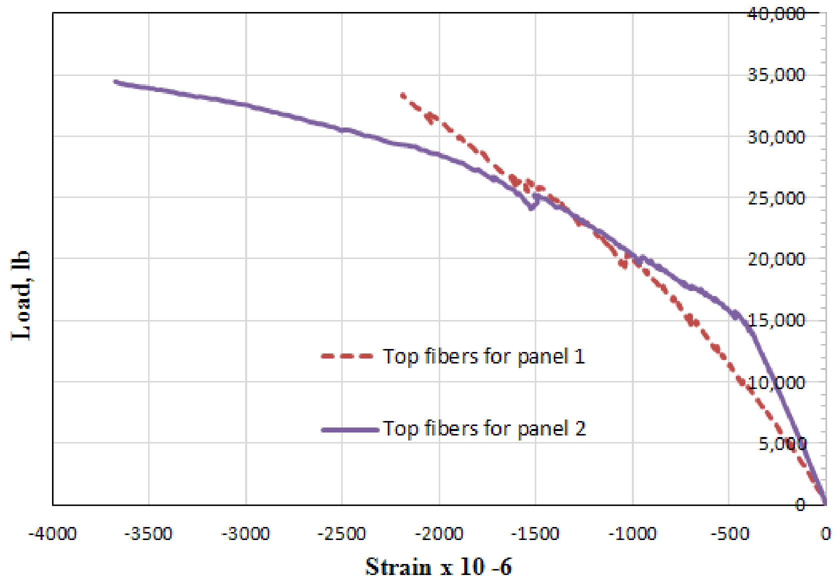

Figure 8: Load-strain relationship of top fibers at mid-span

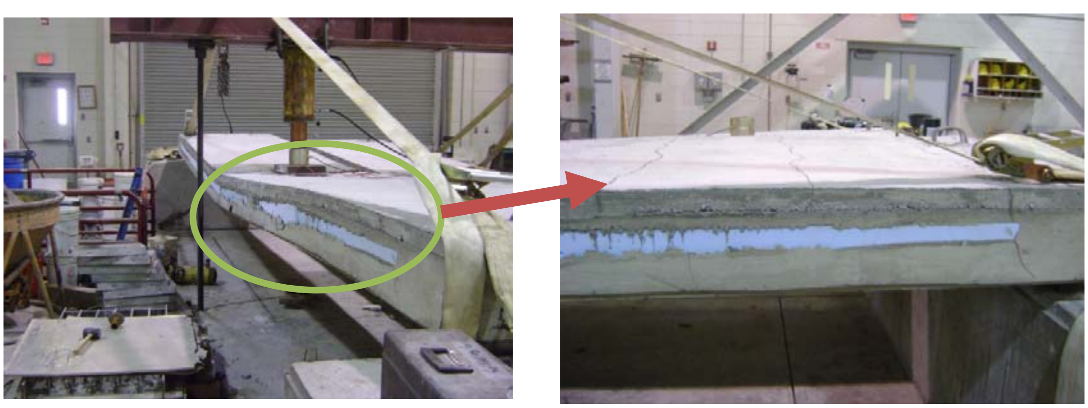

Figure 9: Failure mode of panel 1

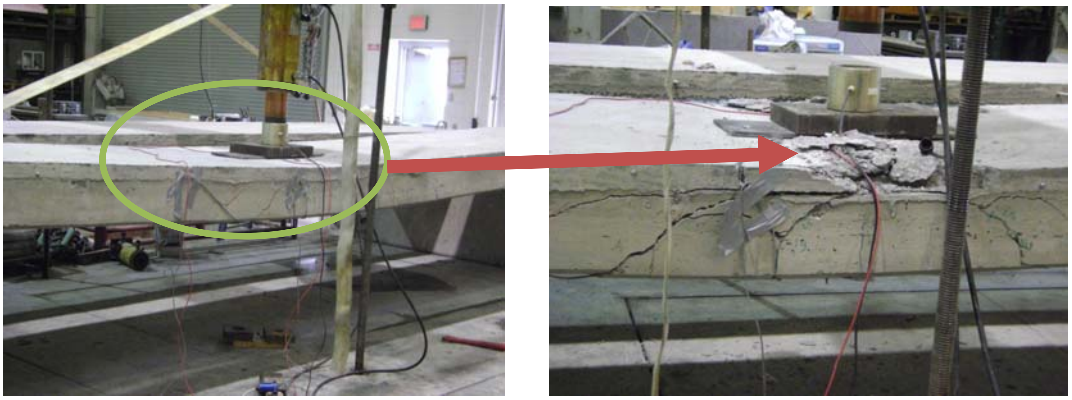

Figure 10: Failure mode of panel 2

ANALYTICAL MODELS

In order to predict the behavior of precast concrete sandwich floor panels with different number and distribution of ties, two modeling methods were investigated. The first method is the planar truss method in which the top-chord members represent the top wythe, bottom-chord members represent the bottom wythe, and diagonal members represent tie legs. Fig. 11 shows the two planar truss models developed for the two panels. In each model, truss elements are assumed to be located at the centerlines of actual elements and have the equivalent section properties. For example, the geometric properties of a diagonal member in the end of the panel 1 are equal to eight times the geometric properties of one tie leg. Connections between the diagonal members and top and bottom chord members are assumed to be pinned with rigid end zone equal to the portion of tie leg embedded in concrete. The two models are assumed to be simply supported and subjected to 6.5 kip point load that represents the equivalent service live load 100 psf in terms of deflection. Analysis results of the two truss models are listed in Table 2.

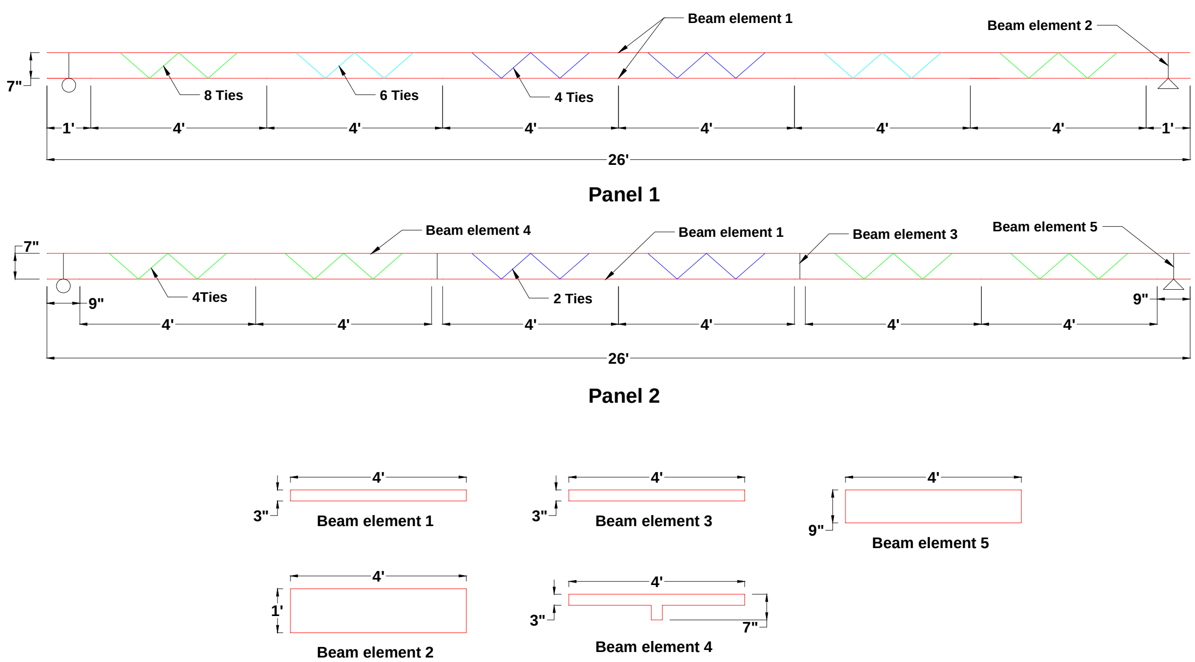

Fig. 11: Truss models of the two test panels

The second modeling method is developing three dimensional FE models in which the top and bottom wythes are modeled as shell elements, and tie legs are modeled as frame elements. Fig. 12 shows the model developed for the panel 1. In each model, shell and frame elements are assumed to be located at the centerlines of actual elements and have their exact section properties. Connections between the frame and shell elements are assumed to be pinned with rigid end zone equal to the portion of tie leg embedded in concrete. The two models are assumed to be simply supported and subjected to 6.5 kip point load that represents the equivalent service live load 100 psf in terms of deflections. Analysis results of the truss and FE models are listed in Table 2.

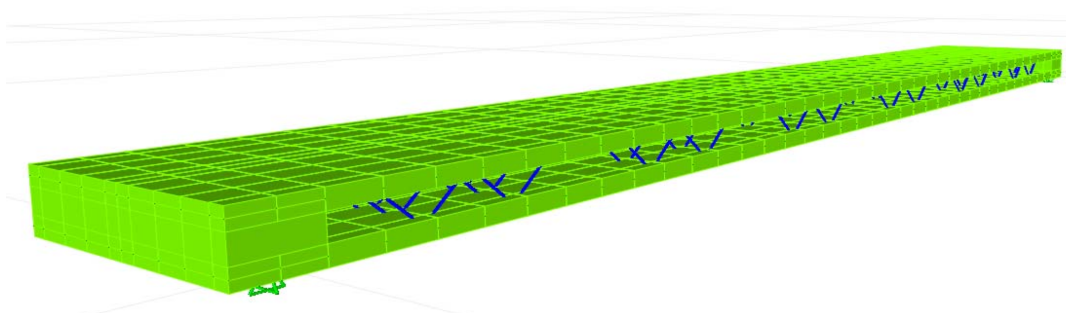

Fig. 12: 3D FE model of the panel 1

Table 3: Comparing the measured deflection versus that calculated using truss and FE models

Panel	L_e (in.)	E(ksi)	I_g (in.^{4})	P (Kip)	D_{truss} (in.)	D_{FE} (in.)	D_{actual} (in.)	D_{actual} / D_{truss}	D_{actual} / D_{FE}
Panel 1	308	5813	3744	6.5	0.38	0.39	0.38	1.0	0.97
Panel 2	308	5813	3776	6.5	0.22	0.21	0.2	0.91	0.95

Table 3 presents the theoretical deflections of the two specimens calculated using truss and FE models under 6.5 kip point load applied at mid-span. Comparing these values against the actual deflections measured during testing indicates that both planar truss models and 3D FE models provide very reasonable estimates of panel defections under service load

CONCLUSIONS

Based on the results of the experimental and analytical investigations, the following conclusions are made:

1. The fabrication of proposed panels using the procedure presented in the paper is simple, efficient, economical, and does not required specialized equipment
2. The proposed panels have full composite action under ultimate load. Their ultimate flexural capacity exceeded the theoretical capacity calculated using strain compatibility.

3. The number and distribution of ties required to achieve full composite action should be calculated using the PCI Design Handbook method for horizontal shear in composite members, but using triangular distribution of the horizontal shear along the shear span.
4. Calculating deflections of the proposed floor panels using the truss models and FE models results in consistent and realistic deflection predictions. Truss models are recommended due to their simplicity and computational efficiency.

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