Design and Analysis of Precast, Prestressed Concrete, Three-Wythe Sandwich Wall Panels

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Precast concrete, three-wythe sandwich wall panels were developed with potentially improved thermal and structural performance compared with that of traditional two-wythe panels.

A three-wythe panel has three concrete wythes and two insulation layers. All three concrete wythes are connected by solid concrete regions, and the connections between successive concrete wythes are staggered in location so that no concrete path extends directly through the entire thickness of the panel.

This paper describes the design and analysis of the three-wythe panels. A series of two- and three-wythe panels were designed, and examination of the resulting panels' behavior provides insight into the anticipated capabilities of the three-wythe panels. Finite element analyses were performed to anticipate the behavior of the three-wythe panel at both the transfer of prestressing force and under service loads.

It was found that three-wythe panels can be designed using current design codes with special consideration of stresses that develop at the panel ends. A three-wythe panel can be treated as a composite panel and is suitable for longer spans compared with spans typical for a two-wythe panel. Transverse bending occurs in a three-wythe panel at the panel ends, and several approaches to reduce the transverse bending were introduced and evaluated, such as using partially debonded strands and shear connectors.

Precast concrete sandwich wall panels are often used as the exterior cladding of buildings and may also serve as bearing walls or shear walls. Typical sandwich panels are composed of two concrete wythes separated by an insulation layer. In this paper, such panels are referred to as two-wythe panels. Both concrete wythes in a sandwich wall panel are often the same thickness, and the surface of the exterior wythe may include architectural details, such as reveals, to provide the desired appearance of the panel.

Figure 1 shows three typical two-wythe panels. The panels are often described by a three-digit sequence of numbers in which each digit in the sequence denotes the thickness, in inches, of one of the layers in the panel. For example, a 3-2-3 panel is composed of two 3-in.-thick (75 mm) concrete wythes separated by a 2-in.-thick (50 mm) insulation layer.

As shown in Fig. 1, solid concrete regions (regions where the insulation layer is omitted) are provided for a variety of reasons, including the placement of inserts for lifting and handling, and connections to the foundation, roof, and adjacent panels. These locations of solid concrete have a significant adverse impact on the thermal performance of the panels. Kosny et al.[1] tested panels similar to configuration I presented in Fig. 1 and reported a 45% reduction in the thermal performance when solid concrete regions were added.

The research described in this paper is directed toward the development of precast concrete three-wythe sandwich wall panels (hereafter referred to as three-wythe panels). Three-wythe panels have the potential for improved thermal and structural performance compared with currently produced two-wythe panels. **Figure 2** shows typical three-wythe panels composed of three concrete wythes and two insulation layers. The locations of the solid concrete regions between successive concrete wythes are staggered, thereby eliminating all direct through-thickness thermal paths through solid concrete.

This paper describes the design and analysis of precast concrete three-wythe sandwich wall panels. A series of two- and three-wythe panels was designed, and examination of resulting panels' behavior provided insight into the anticipated capabilities of the three-wythe panels. Analyses were performed to anticipate the behavior of the three-wythe panel at both the transfer of prestressing forces and under service loads. Experiments investigated the behavior of three-wythe panels subjected to service load and at prestress transfer. Lee[2] and Lee and Pessiki[3] give the results of these experiments and complete details of the work presented in this paper.

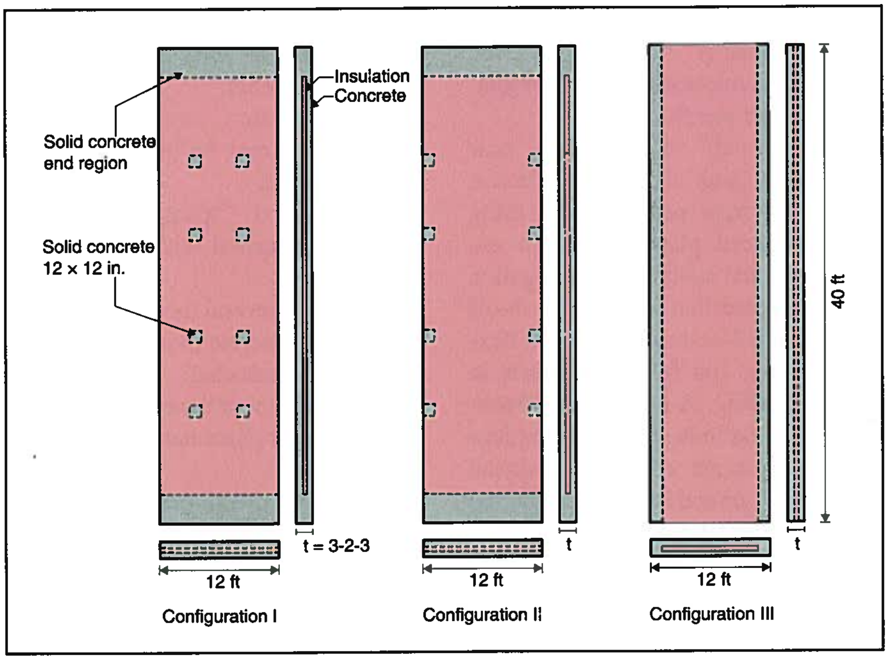

Fig. 1. Typical precast concrete two-wythe sandwich wall panel configurations. Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m.

BACKGROUND

Much of the current practice concerning the uses, design and detailing, manufacturing, and thermal performance of two-wythe panels is summarized in a two-part report prepared by the Precast/Prestressed Concrete Institute (PCI) Committee on Precast Sandwich Wall Panels.[4,5] In the past, several research projects have been conducted on precast concrete sandwich wall panels to improve their structural and thermal performance. Pfeifer and Hanson[6] tested several nonprestressed sandwich panels in flexure and found that different degrees of composite action could be achieved by varying types of wythe connectors and their spacing. Bush and Stine[7] tested precast concrete sandwich panels with continuous truss connectors. Results of the tests showed that a

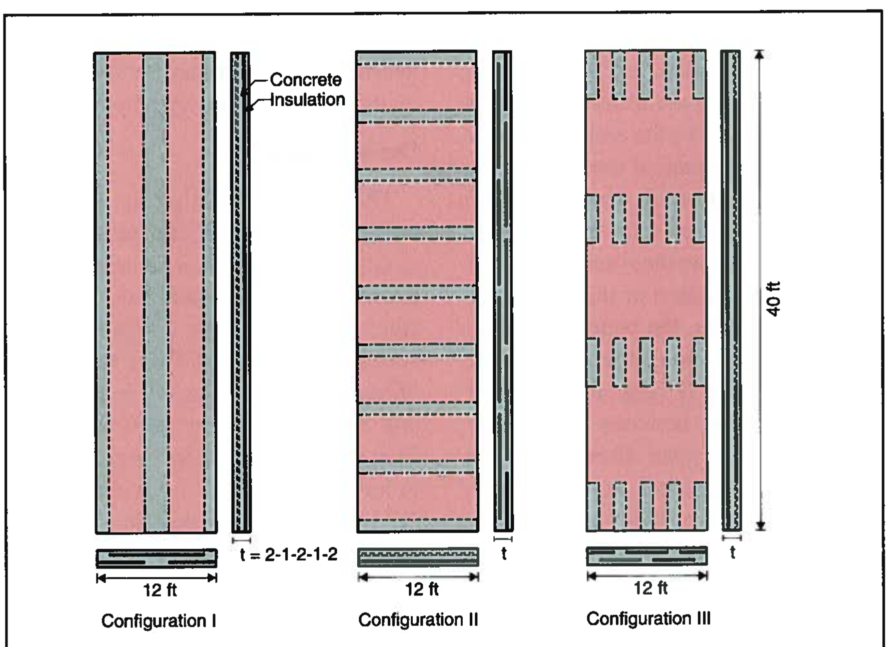

Fig. 2. Precast concrete three-wythe sandwich wall panel configurations. Note: 1 ft = 0.3048 m.

high degree of composite stiffness and flexural capacity could be achieved with truss connectors oriented longitudinally in the panels.

Einea et al.[8] developed a new structurally and thermally efficient precast concrete panel system using fiber-reinforced plastic bars. An experimental and analytical investigation was conducted that included push-off loading, small-scale specimens in flexural loading, and full-scale panels in flexural loading. A further experimental study by Salmon et al.[9] included four full-scale precast concrete sandwich panel tests. Lee and Pessiki[10,11] reported that the thermal performance of a three-wythe panel is superior to that of a two-wythe panel due to the increased length of the thermal path through the solid concrete in the three-wythe panel.

THREE-WYTHE PANELS

Figure 2 shows three general configurations of the three-wythe panel. Similar to the two-wythe panel, three-wythe panels are described with a five-digit number sequence. For example, a 2-1-2-1-2 panel is composed of three 2-in.-thick (50 mm) concrete wythes and two 1-in.-thick (25 mm) insulation layers (Fig. 2).

The terms *face*, *center*, and *back* are used to indicate each concrete wythe in a three-wythe panel. The face wythe is cast against the concrete formwork if the panel is cast flat, and the back wythe is the finished wythe. In Fig. 2, the top wythe is the back wythe and the bottom wythe is the face wythe and is typically also the exterior face of the panel.

In configuration I, the 2-1-2-1-2 panel (Fig. 2), the solid concrete regions between wythes are staggered in the width direction of the panel. For this configuration, the panel cross section is uniform along the panel span. In configuration II (Fig. 2), the solid concrete regions between wythes are staggered in the span direction of the panel so that the panel cross section is uniform across the width of the panel. Finally, for configuration III (Fig. 2), the locations of solid concrete regions between wythes are staggered in both the width and span directions.

The three-wythe panel has several potential advantages:

- Increased thermal performance compared with that of a two-wythe panel;[10,11]
- Composite action between wythes may be provided by the concrete;
- Increased spanning capability compared with a two-wythe panel;
- Use of several locations of thickened concrete (where the wythes are connected) for potential placement of the embedded hardware for panel handling and connections;
- Increased corrosion protection (when placing all of the prestressing steel in the center wythe); and
- One-time prestressing operation (performed only when all of the prestressing steel is placed in the center wythe).

The three-wythe panel includes several potential disadvantages compared with a two-wythe panel, the most obvious of these may be increased production time and cost. Thus, the three-wythe panel may not be appropriate in many applications.

DESIGN STUDIES

The anticipated structural performance of three-wythe panels was examined by designing a series of two- and three-wythe panels using current codes and industry practices, making necessary assumptions for the unique features of the three-wythe panel.

Design Loads

This study focuses on the structural behavior of three-wythe panels, such as cladding panels, that are subjected to lateral load due to wind loads only. Design wind loads were computed using American Society of Civil Engineers' *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-02).[12] The panel treated here was assumed to measure 12 ft (3.7 m) wide by 40 ft (12 m) tall and was used in a building structure located at a northeast coastal area of the United States. For these regional conditions, the design wind pressure p was computed as 27 psf (130 kg/m^2) for pressure load and 30 psf (150 kg/m^2) for suction load. Other assumptions include a basic wind speed of 100 mph (160 km/h), an importance factor of 1.15, exposure C, an exposure coefficient of 1.04, a directionality factor of 1.0, and a topographic factor of 1.0.

In this paper, a uniform wind pressure of 32 psf (160 kg/m^2) is used as the service load for all panels for both pressure and suction. This value is often used in practice for routine panel design,[13] and this uniform value along the panel height was used to simplify the calculations for different panel heights. The application of a constant wind pressure is valid for panel heights up to 60 ft (18 m) in accordance with ASCE 7-02. Finally, a load factor of 1.3 was used to design both two- and three-wythe panels in this paper. Where the wind load was not reduced by a directionality factor, the load factor of 1.3 was allowed according to the American Concrete Institute's *Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05)*.[14]

Design Material Properties

All material properties were similar to those used in current practice for two-wythe panels. The material properties presented here also apply to the numerical analyses of the three-wythe panels described in a following section.

Concrete material properties for all wythes included a concrete compressive strength at transfer of prestressing force f'_{ci} of 3500 psi (24 MPa) and a design concrete compressive strength at 28 days f'_c of 6000 psi (41 MPa). The concrete moduli of elasticity E_{ci} and E_c are 3372 ksi (23,250 MPa) and 4415 ksi (30,440 MPa) at prestress transfer and 28 days, respectively. Poisson's ratio and the density of the concrete were taken as 0.2 and 150 pcf (2400 kg/m^3), respectively.

Prestressing steel used for all panels was $^7/_{16}$-in.-diameter (11 mm), seven-wire, low-relaxation strand. The yield stress f_{py} and the ultimate strength of the prestressing steel f_{pu} was 243 ksi (1680 MPa) and 270 ksi (1860 MPa), respectively. The modulus of elasticity of the strand E_{ps} was 28,500 ksi (197 GPa). The stress-strain relationship of the strand was obtained from the *PCI Design Handbook: Precast Prestressed Concrete*.[15]

Insulation material properties included a compressive strength of 25 psi (170 kPa), a modulus of elasticity E_i of 1.35 ksi (9.3 MPa), and a Poisson's ratio of 0.3. Wythe connectors were M-ties with a yield strength of 80.5 ksi (555 MPa) and modulus of elasticity of 30,000 ksi (207 GPa). Each M-tie had two 0.25-in.-diameter (6.4 mm) legs and was 4 in. (100 mm) high and 4 in. wide. As needed, M-ties having longer legs were used for thicker panels.

Design Considerations

Typical two-wythe panels are designed for flexure, shear, and deflection. The design of three-wythe panels is similar to the design of two-wythe panels except for special consideration of stresses at the panel end regions. Details of stresses at the panel end regions are described in the following.

Flexural Design—The flexural design of panels should satisfy the usual requirements, including resisting flexural stresses at service load, having adequate flexural strength for factored loads, and preventing abrupt failure modes. Flexural stresses are computed using a plane section assumption, and stresses are compared with limits provided in ACI 318-05. All panels are designed as class U members as given in ACI 318-05.

Flexural strength is calculated in the same manner as for other prestressed concrete flexural members. The design strength ϕM_n should be greater than or equal to the factored moment M_u. In prestressed sandwich panels, the flexural strength may be reached shortly after cracking. Such a failure occurs abruptly and without warning. This abrupt flexural failure is prevented by making ϕM_n exceed 1.2 times the cracking moment M_{cr}.[14]

Horizontal Shear Design—Typical sandwich wall panels are flexible in bending and, thus, panel design is generally controlled by flexure and not shear. For composite panels, however, sufficient strength must be provided to transfer horizontal shear between concrete wythes. For a three-wythe panel to act as a composite panel, horizontal shear must be transferred between adjacent concrete wythes.

Computation of horizontal shear strength V_{nh} requires potential failure modes to be determined. Once the potential failure modes are determined, shear strength can be computed using current design codes. The horizontal shear force V_u is computed using the actual change in compressive and tensile force in any segments as presented in Eq. (1).

$$V_u = \min(C,T) \tag{1}$$

where

C = the compressive force for each segment

T = tensile force for each segment

V_u is taken as minimum value between C and T.

Deflection—Deflection of a panel is checked at service load. According to an ACI Committee 533 report,[16] the deflection of any point on a panel is limited to $L/480$ but cannot be greater than 0.75 in. (19 mm) for cladding wall panels (where L is the length of the panel). There is no precise deflection limitation requirement for a cladding panel, however, in the current ACI 318-05 or *PCI Design Handbook*. Also, the limi-

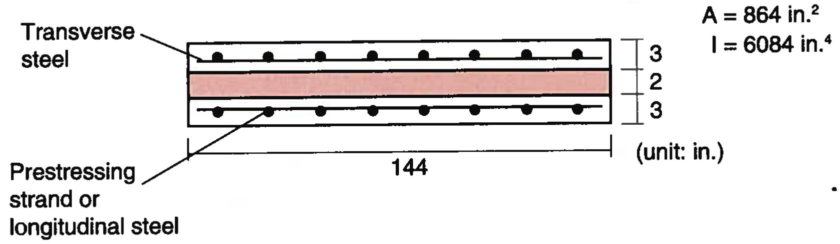

Fig. 3. Design of 3-2-3 precast concrete sandwich wall panel. Note: 1 in. = 25.4 mm.

Table 1. Design Summary of 3-2-3 Precast Concrete Sandwich Wall Panel

Strand No.	f_{ce}, psi	Moment Capacity, kip-in.			Longitudinal Steel* to Satisfy $\phi M_n > 1.2M_{cr}$	Transverse Steel*
		$1.2M_{cr}$	ϕM_n	M_f		
4	86	1209	361	832	46 No. 3	55 No. 3
8	171	1364	716	961	36 No. 3	62 No. 3
12	257	1520	1066	1091	24 No. 3	68 No. 3
16	342	1675	1410	1220	16 No. 3	77 No. 3
20	428	1830	1749	1349	6 No. 3	85 No. 3
24	513	1985	2083	1479	0 No. 3	95 No. 3
28	599	2140	2411	1608	0 No. 3	111 No. 3
32	684	2296	2733	1737	0 No. 3	127 No. 3

* Yield strength of reinforcement was assumed to be 60 ksi (414 MPa).

Note: No. 3 steel bar = 0.375 in. diameter = 10M (10 mm diameter). 1 in.^{2} = 645 mm^2; 1 in.^{4} = 0.4162 × 10^6 mm^4; 1 kip-in. = 0.113 kN-m; 1 psi = 0.006895 MPa.

tations in the ACI Committee 533 report are not typically applied in current practice.

Additional Design Considerations—Additional panel design considerations are stripping, handling, transportation, and erection loads. Precast concrete panels are sometimes governed by these service loading cases.

Comparison of Two- and Three-Wythe Panel Designs

Two- and three-wythe panels were designed according to the design considerations previously described. A series of two- and three-wythe panels were designed for the same set of requirements, and the resulting designs were compared to gain insight into the anticipated performance of the three-wythe panel compared with that of the two-wythe panel.

For two-wythe panels, the 3-2-3 panel (configuration I) was considered (Fig. 1). The panel was 12 ft (3.7 m) wide, and both face and back wythes were assumed to be prestressed with the same prestressing force. For three-wythe panels, configuration I panels were investigated (Fig. 2). These panels were also 12 ft wide and had one 2-ft-wide (0.6 mm) concrete rib between the face and center wythes and two 1-ft-wide (0.3 m) concrete ribs between the center and back wythes. For both two- and three-wythe panels, all prestressing strands were assumed to be tensioned to 70% of their maximum strength, and time-dependant prestressing losses were assumed to be 15%.

Two-Wythe Panels—Table 1 and **Fig. 3** summarize the design of a series of 3-2-3 panels. The table summarizes moment capacities of the panel with 4 to 32 strands. As an example, when the panel has 12 strands, the effective prestressing force f_{ce} is 257 psi (1.77 MPa). Based on the panel cross section and strand area, moment capacities of 1.2 times the cracking moment $1.2M_{cr}$, the design flexural strength ϕM_n, and the controlling moment (based on the allowable stresses) M_f were computed and the results are shown in Table 1. M_f was computed when the stress in the extreme tension fiber reached $6\sqrt{f_c'}$ while $7.5\sqrt{f_c'}$ is allowed for class U members in ACI 318-05. For the 3-2-3 panel with 12 strands, twenty-four $^3/_8$-in.-diameter (10M) reinforcing steel bars (12 in each wythe) would need to be added to satisfy the requirement that $\phi M_n > 1.2M_{cr}$. It is noted here that not every panel presented in Table 1 was a practical design. The table indicates the capabilities of the panels as the number of strand was varied for a given cross section.

Three-Wythe Panels—Table 2 and **Fig. 4** summarize the design of a series of 2-1-2-1-2 panels when all prestressing strands were placed in the center wythe. Six different panel thicknes es (2-1-2-1-2, 2-1-3-1-2, 3-1-3-1-3, 2-2-2-2-2, 2-2-3-2-2, and 3-2-3-2-3) were investigated. Because of space limitations, only the 2-1-2-1-2 panel is shown in Table 2. Results for the other panels are given in references 2 and 3. Transverse reinforcement requirements shown in the table are discussed in the later section.

Spanning Capabilities—Based on the moment capacities shown in Tables 1 and 2, the maximum panel span limits were computed for a uniform wind load of 32 psf (160 kg/m²) over the entire panel span. **Figure 5** shows the relationship of the maximum span limits from the $1.2M_{cr}$, ϕM_n, and M_f requirements with respect to the number of strands for the 3-2-3 and 2-1-2-1-2 panels.

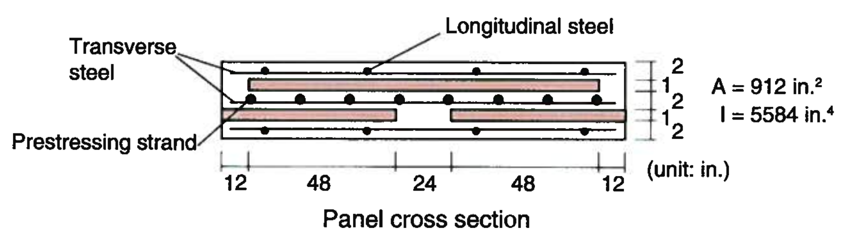

Fig. 4. Design of 2-1-2-1-2 precast concrete sandwich wall panel. Note: 1 in. = 25.4 mm.

Table 2. Design Summary of 2-1-2-1-2 Precast Concrete Sandwich Wall Panel

Strand No.	f_{ce}, psi	Moment Capacity, kip-in.			Longitudinal Steel* to Satisfy $\phi M_n > 1.2M_{cr}$	Transverse Steel*
		$1.2M_{cr}$	ϕM_n	M_f		
4	81	1109	436	762	33 No. 3	35 No. 3
8	162	1245	851	875	21 No. 3	44 No. 3
12	243	1380	1244	988	9 No. 3	53 No. 3
16	324	1516	1613	1101	0 No. 3	64 No. 3
20	405	1652	1960	1214	0 No. 3	80 No. 3
24	486	1788	2282	1328	0 No. 3	95 No. 3
28	567	1923	2577	1441	0 No. 3	111 No. 3
32	648	2059	2846	1554	0 No. 3	127 No. 3

* Yield strength of reinforcement was assumed to be 60 ksi (414 MPa).

Note: No. 3 steel bar = 0.375 in. diameter = 10M (10 mm diameter). 1 in.² = 645.16 mm²; 1 in.⁴ = 0.4162 × 10⁶ mm⁴; 1 kip-in. = 0.113 kN-m; 1 psi = 0.006895 MPa.

As shown in Fig. 5, an increase in the number of strands increased the spanning capabilities of both the two- and three-wythe panels. An increase in the number of strands increased both the required prestressing force and the area of reinforcement in the panel. Therefore, as $1.2M_{cr}$, ϕM_n, and M_f increased, the maximum span limit also increased.

All panel-spanning capability plots exhibit similar features for both the two- and three-wythe panels (Fig. 5). When panels have a small number of strands, ϕM_n is less than $1.2M_{cr}$. When panels have a relatively large number of strands, ϕM_n is greater than $1.2M_{cr}$. This is because the flexural strength increases more rapidly than the cracking strength as strands are added to a given panel section.

As discussed previously, abrupt failure of the panel is prevented by ensuring that ϕM_n is greater than $1.2M_{cr}$. If ϕM_n is increased to $1.2M_{cr}$ by adding longitudinal reinforcement shown in Tables 1 and 2, the stress limits control the spanning capability of the panels (Fig. 5). This was true for both the two- and the three-wythe panels. The extreme fiber tension stress limit of $6\sqrt{f'_c}$ at full service load controls the panel-spanning capability for composite two- and three-wythe panels—unless other member design classifications are assumed (in class T members, the extreme fiber tension is limited up to $12\sqrt{f'_c}$ at full service load).

Figure 6 provides a summary of the spanning capabilities of all panels. Because the maximum spanning capabilities for all panels were controlled by stress limits, only the maximum panel spans from the stress limits are plotted in Fig. 6. As shown in the plot, a longer panel span was obtained with increased panel thickness. This is because the moment of inertia increases with panel thickness.

Two different locations of prestressing were investigated for the three-wythe panels:

- When all prestressing strands are located in the center wythe (C); and
- When all prestressing strands are located in the face and back wythes (FB).

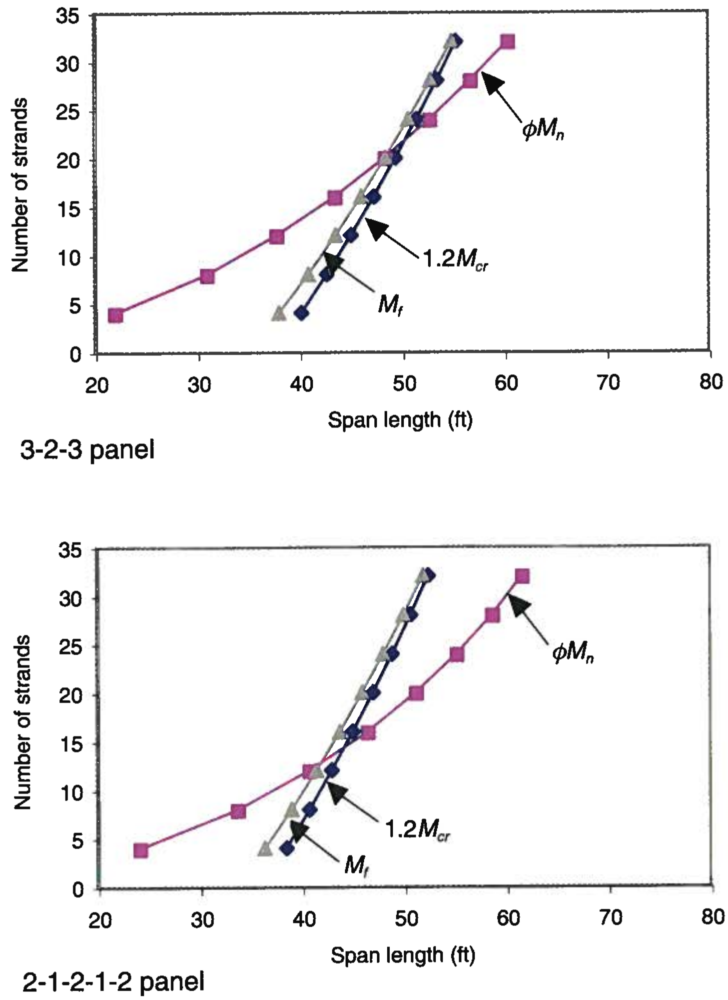

Fig. 5. Spanning capability of precast concrete two- and three-wythe sandwich wall panels. Note: 1 ft = 0.3048 m.

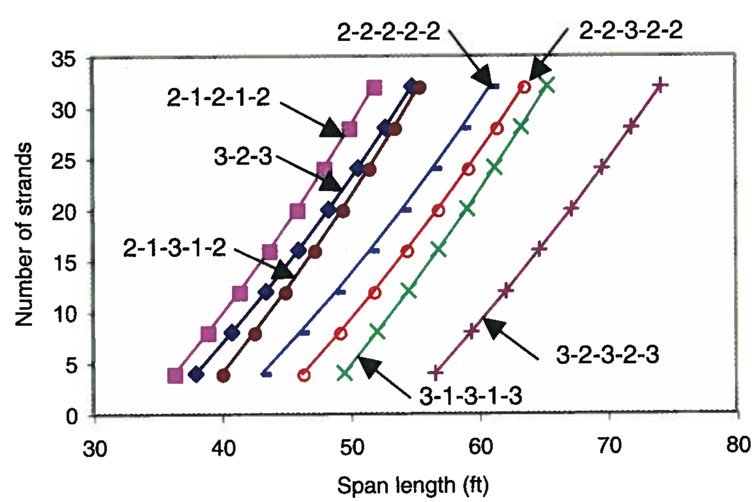

Fig. 6. Summary of spanning capabilities for precast concrete two- and three-wythe sandwich wall panels. Note: 1 ft = 0.3048 m.

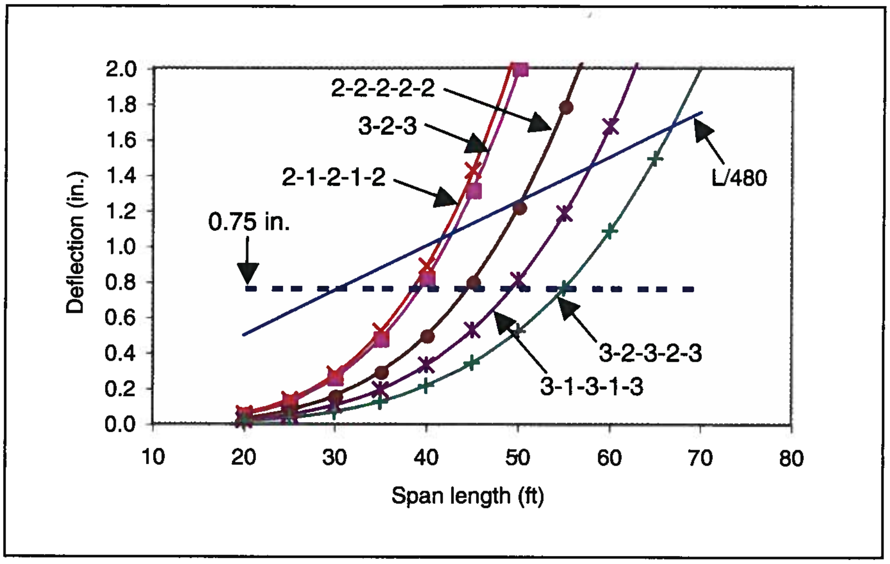

Fig. 7. Midspan deflection for precast concrete two- and three-wythe sandwich wall panels. Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m.

The spanning capabilities for three-wythe panels were the same for both the C and FB cases. This is because, as explained previously, all span limits were governed by stress limits.

Deflection of Panels—Figure 7 compares the midspan deflection of two- and three-wythe panels under full service load. All deflections were computed assuming a simply supported panel with 32 psf (160 kg/m²) uniform service load and an uncracked cross section. Depending on the prestressing in the panels, cracks or material nonlinearity of the panel may occur, but these potential effects were ignored. Superimposed on the deflection plots are two deflection limitations—*L*/480 and 0.75 in. (19 mm)—obtained from the ACI Committee 533 report.

As shown in Fig. 7, spanning capabilities of the panels changed depending on which deflection limitation was applied. When applying the *L*/480 deflection limitation, the 3-2-3 panel was limited to a 43 ft (13 m) span while the 3-2-3-2-3 panel could be used for a 67 ft (20 m) span. Alternatively, when the 0.75 in. (19 mm) deflection limit was applied, the panel span was limited to 38 ft (12 m) for the 3-2-3 panel and 55 ft (17 m) for the 3-2-3-2-3 panel.

Limiting the maximum panel deflection to 0.75 in. (19 mm) greatly limits the range over which the three-wythe panel may be used. For a span of 55 ft (17 m), the 0.75 in. midspan deflection was equal to *L*/880, which is much less than *L*/480 (maximum permissible deflection for roof and floor construction given by ACI 318-05). Thus, the 0.75 in. deflection limit may be conservative for three-wythe panels.

Reinforcement Calculation—Longitudinal reinforcement is required to prevent the abrupt flexural failure of the panels (satisfying the requirement of $\phi M_n > 1.2M_{cr}$), and results are included in Tables 1 and 2. Longitudinal reinforcement was assumed to be equally placed in all concrete wythes, and total area of reinforcement per panel is indicated in the tables. In current practice, the effective prestressing force of a panel is in the range of 150 psi (1.0 MPa) to 600 psi (4.1 MPa). For these stress levels, the required longitudinal steel area was small (Tables 1 and 2).

The required transverse reinforcement to resist horizontal shear force was also computed, and results using the shear friction method[14] are shown in Tables 1 and 2. For results presented here, a failure mode was assumed. For a two-wythe panel, the interface between two concrete wythes is assumed to crack. The assumed failure mode for the three-wythe panel is shown in **Fig. 8**. These assumed failure modes were determined by considering the maximum shear stress locations in the respective panels.

The horizontal shear force to be resisted was computed using Eq. (1). In these calculations, the additional longitudinal reinforcement indicated in Tables 1 and 2 was included as part of the tensile force. As shown in Tables 1 and 2, when the number of strands increases in the panel, the required transverse reinforcement increases. The amount of required transverse reinforcement was similar for both two-wythe and three-wythe panels.

The amount of required transverse reinforcement shown in Tables 1 and 2 is more than typical amounts used in current practice for two-wythe panels. The solid concrete regions were assigned a shear strength of 80 psi (0.6 MPa) to design the panel for horizontal shear. In addition, when the horizontal shear force was computed from the design wind load and panel span—instead of from Eq. (1)—the horizontal shear force demand was less, so less reinforcement was needed.

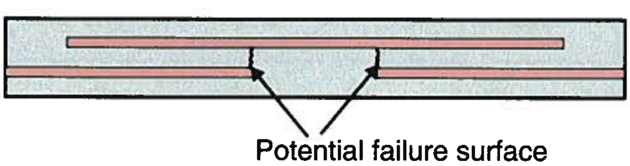

Fig. 8. Horizontal shear failure mode for the configuration I panel.

FEM ANALYSES

Finite element method (FEM) analyses were performed to gain further insight into the expected behavior of the three-wythe panels at prestressing force transfer and at full service loads. Transverse bending occured at the end of the panel at transfer of prestress, creating stresses that might lead to cracking in the longitudinal direction of the panel. Features of the transverse bending are described, and several solutions to reduce bending are introduced and investigated using the FEM analyses.

The configuration I panels were investigated in this study (Fig. 2). Material properties used in the FEM analyses were the same as described previously in the design studies.

Description of FEM Model

Model Geometries and Element Types—Figure 9 shows an example of the quarter-symmetry FEM model of a 3-2-3-2-3 panel. For the configuration I panel, two symmetry conditions existed along the midwidth and across the midspan of the panel. As a result, a quarter-symmetry model was used. This model geometry is typical and is used for most of the FEM analyses presented in this paper.

Various finite-element mesh arrangements and convergence studies were investigated, and the finite-element mesh arrangement shown in Fig. 9 was selected. As discussed, transverse bending at transfer of prestressing causes rapid stress variations at the end region of a panel, and the fine finite-element mesh was used in that region. Three different finite-element mesh regions were used along the panel span to reduce the total number of elements and analysis execution time. Solid elements were used to model the concrete and insulation in the FEM. Shell elements were used to model a steel plate.

Loading—Two types of loading were used: prestressing force and service loads. Prestressing force was imposed as nodal forces over the transfer length at the end of the panel. The transfer length was computed according to ACI 318-05 (section R12.9). For all FEM analyses discussed in this paper, a prestressing force of 21.7 kip (96.5 kN) per strand was used and the force was uniformly distributed over the transfer length of 24 in. (610 mm). To examine stresses in a panel under service loads, a 32 psf (160 kg/m^2) uniform wind load was applied and both wind pressure and suction were considered.

Boundary Conditions—For all of the analyses in this study, simple supports were assumed where connection hardware attaches the panel to the foundation. In addition, where symmetry conditions exist, symmetry boundary conditions were used.

FEM Models—Two different FEM models were considered. In the first

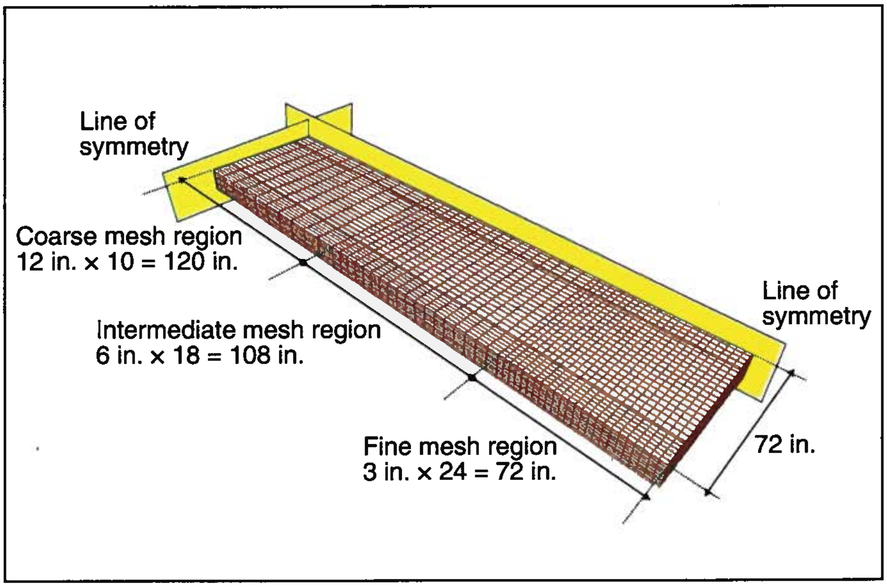

Fig. 9. Finite element method (FEM) model geometry and finite-element mesh. Note: 1 in. = 25.4 mm.

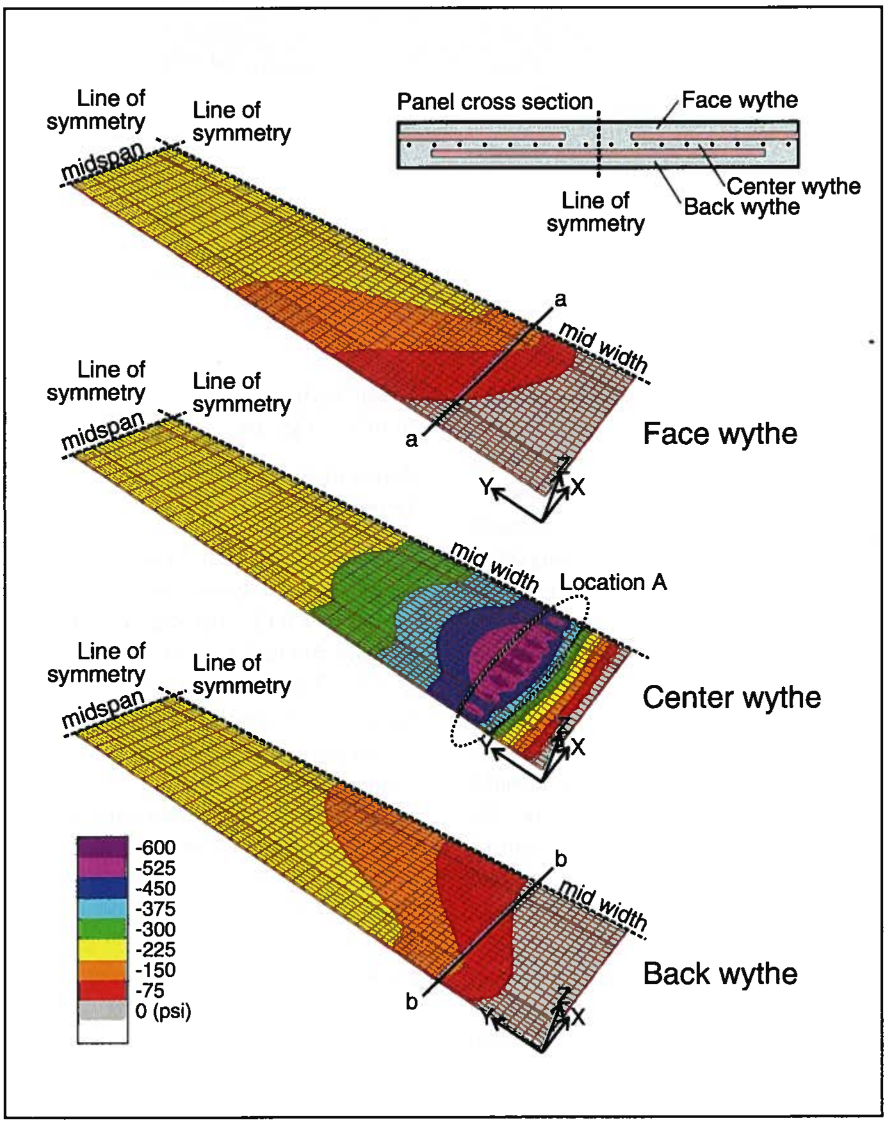

Fig. 10. Y-direction normal stress contours under the action of initial prestressing force P_i. Note: 1 psi = 0.006895 MPa.

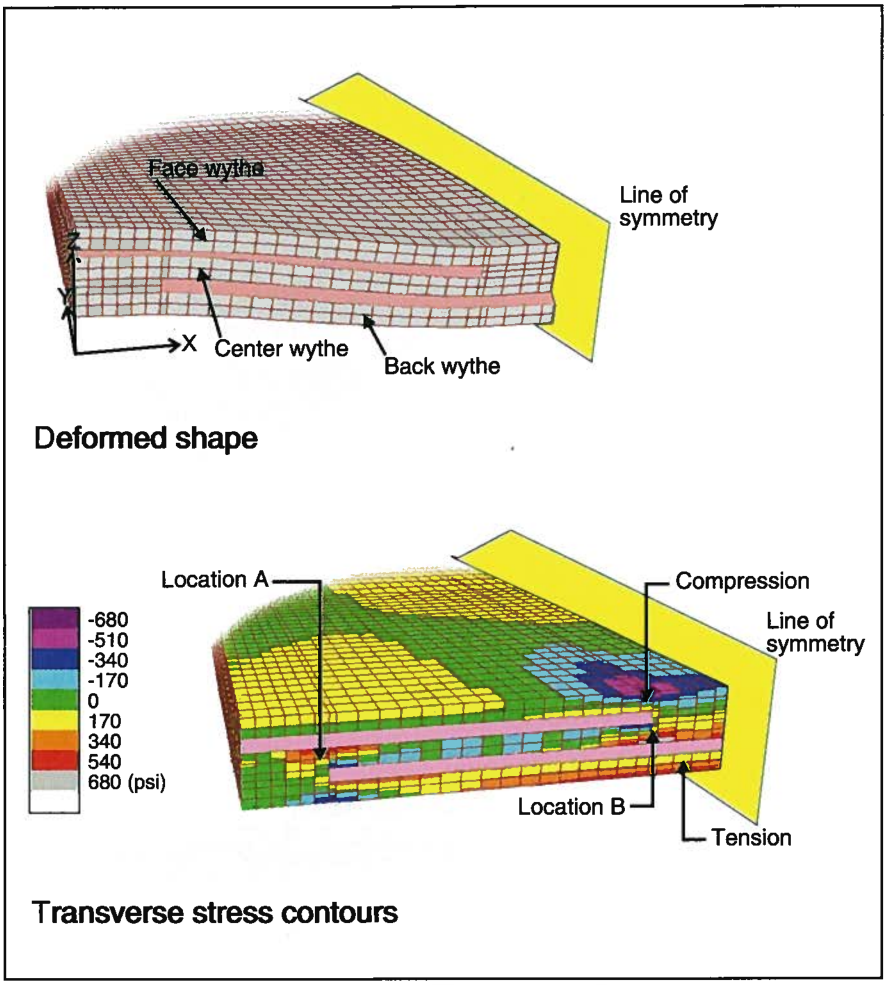

Fig. 11. Transverse bending. Note: 1 psi = 0.006895 MPa.

model, both the concrete and the insulation were modeled with solid elements and it was assumed that the concrete and insulation were perfectly bonded together so that all required stresses (tension, compression, and shear) could be transferred between the two materials. The first model is called the perfect bond model in this paper.

The second model was a non-linear model in which the insulation could resist only compression and the wythe connectors could resist only tension. Both models gave similar results. The non-linear model, however, required significantly greater computational effort. Only results from the perfect bond model are presented in this paper. Results of the non-linear model are given in the Lee dissertation.[2] In addition, when modeling the panel in FEM analyses, the panel configuration was turned upside down. Thus, the face wythe was located at top of the FEM model, and the back wythe was located at the bottom of the FEM model, as shown in **Fig. 10**.

Panel Behavior at Prestress Transfer

The configuration I panels were the focus of the analyses (Fig. 2). The panels were 12 ft (3.7 m) wide and 50 ft (15 m) long and had a panel thickness of 3-2-3-2-3. The panels had sixteen $^7/_{16}$-in.-diameter (11 mm), 270 ksi (1860 MPa) prestressing strands, all placed in the center wythe.

Prestressing Stress Distribution of Three-Wythe Panels—Figure 10 shows Y-direction normal stress contours under the initial prestressing force P_i at the center of each concrete wythe. Only a quarter-symmetry portion of the panel is shown, and the stress contours were plotted by using average stress values of all connected concrete elements. For the center wythe, the prestressing force distribution was relatively uniform across the panel width (Fig. 10) and stress concentration occurred where the strands were located (location A). The maximum Y-direction normal compressive stress was about 650 psi (4.5 MPa) at location A and was about 250 psi (1.7 MPa) at midspan.

The face and back wythes did not have uniform prestress distribution across the panel width. For any transverse section across the panel width (line a–a), the stress in the face wythe was greater at the midwidth of the panel and lesser at the edge of the panel. The opposite prestress distribution occurred in the back wythe (line b–b). These non-uniform stress distributions can be explained as a shear lag effect in the three-wythe panel. The prestressing force was applied on the center wythe only, and it was transferred to the face and back wythes through the concrete ribs.

A relatively uniform prestress distribution was obtained in all three wythes at the midspan of the panel. For the panel shown in Fig. 10, stresses at midspan of the panel were in a range of 248 psi to 252 psi (1.71 MPa to 1.74 MPa) and were within ±1% of the initial prestressing force of the panel f_{ci} (250 psi [1.72 MPa]).

Transverse Bending at Prestress Transfer

Features of Transverse Bending—Figure 11 shows the deformed shape at the end of the three-wythe panel and the transverse stress contours. As shown in the deformed-shape diagram in Fig. 11, the panel end distorted due to prestressing force in the center wythe. The midwidth part at the panel, where symmetry exists, had a downward displacement, and the free edge of the panel had an upward displacement. This deformed shape corresponds to the tension and compression stresses in the transverse direction (Fig. 11). The back wythe's bottom surface at the midwidth of the panel was in tension, and the face wythe's top surface was in compression. The maximum transverse tension stress occurred at locations A and B, as shown in Fig. 11. These stresses were locally concentrated.

Figure 12 illustrates the cause of the transverse bending that occurred at prestress transfer. The panel cross

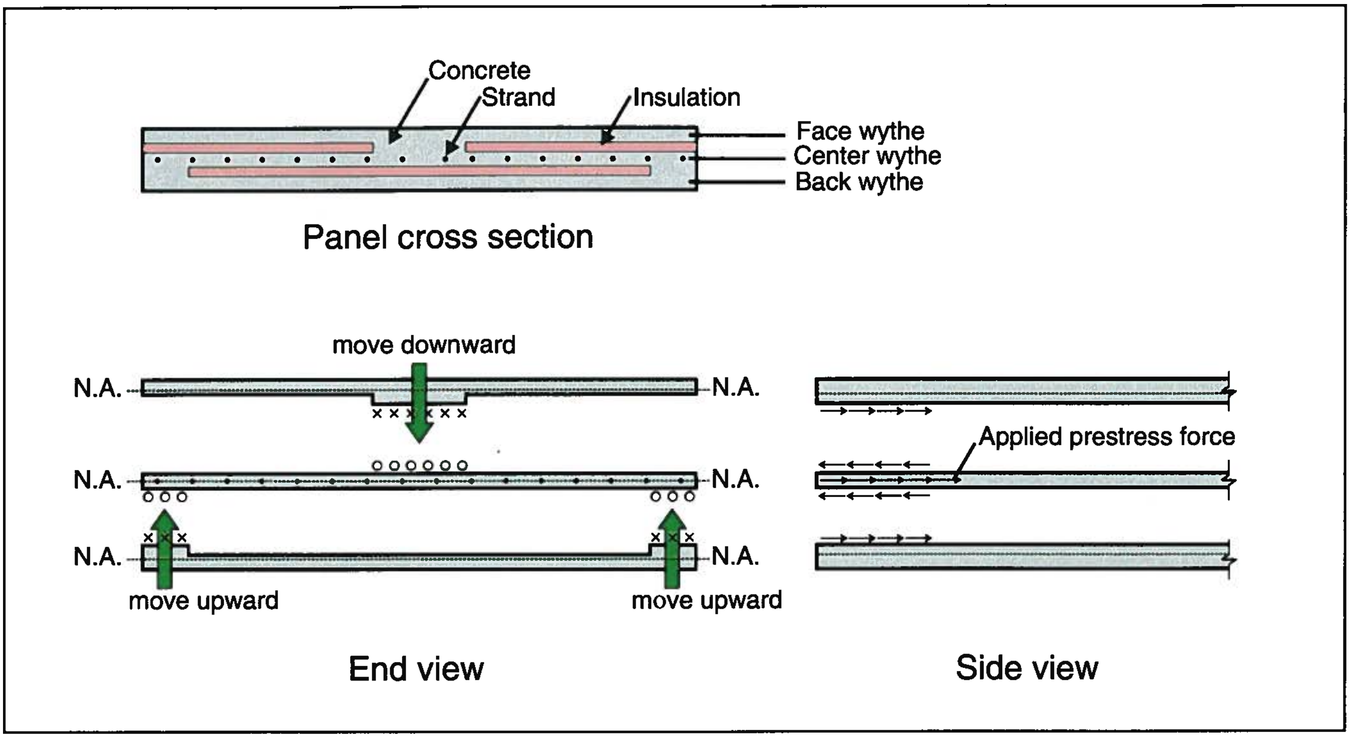

Fig. 12. Illustration of the cause of the transverse bending at prestress transfer. Note: N.A. = neutral axis.

section is shown in Fig. 12, and forces that acted on each concrete wythe are shown in the end view and side view. When the prestressing force was applied to the center wythe only, prestress was transferred between wythes through the concrete ribs. As a result, the prestressing forces were applied eccentrically to the face and back wythes. Edges of the panel displaced upward due to the force eccentricity in the back wythe, and the midwidth of the panel displaced downward due to the force eccentricity in the face wythe. This simple explanation agrees with the result from the FEM analysis (Fig. 11).

Effect of Prestressing Force Location on Transverse Bending—Three different prestressing force locations were considered:

- Prestressing force applied in the center wythe only (C);
- Prestressing force applied in the face and back wythes (FB); and
- Prestressing force applied in the face, center, and back wythes (FCB).

Two different panel cross sections were considered in this study and are shown in **Fig. 13**. The first panel was the same as configuration I (1-2 rib panel) shown in Fig. 2. The second panel had three concrete ribs between the face wythe and the center wythe and two concrete ribs between the center wythe and the back wythe (2-3 rib panel). It is noted that the 2-3 rib panel may actually be a more practical panel configuration for panel handling and the installation of embedded lifting hardware.

Table 3 shows analysis results for various prestressing force locations in three-wythe panels. Results include the maximum transverse tension stresses in each panel and their locations. As presented in Table 3, the FCB cases had the lowest maximum transverse tension stresses, and the C cases had the highest maximum transverse tension stresses. For a given prestress location, the 2-3 rib panel had lower stresses than the 1-2 rib panel (Table 3).

For both the 1-2 rib and 2-3 rib panels, the maximum transverse tension stresses occurred in the center wythe (locations 1 and 4) for the C cases and in the outer wythes (locations 2 and 5) for the FB cases. For both of these cases, these maximum tension stresses are attributed to transverse bending. For the FCB cases, the maximum transverse tension stresses occurred in locations 3 and 6. Examination of the stress contour plots suggests that these maximum stresses were associated with radial stresses between the lines of prestressing force application, rather than from transverse bending of the panel.

For an initial concrete strength f'_{ci} of 3500 psi (24 MPa), the modulus of rupture of the concrete was 444 psi (3.1 MPa), as computed by $7.5\sqrt{f'_{ci}}$. The panels that exhibited maximum transverse stresses above 444 psi were expected to crack at prestress transfer. These cracks may be undesirable in practical applications, particularly if cracks are present in the face or back wythes. Therefore, there is a need to reduce the stresses to below the cracking stress of the concrete.

Approaches to Reduce the Transverse Bending Stresses—Two approaches to reduce transverse bending stresses at prestress transfer were studied. The first approach uses partially debonded strands (half of the total number of strands) to apply the prestressing force more gradually at the end of the panel. The second approach uses shear connectors between concrete wythes to cause all wythes to shorten more uniformly at prestress transfer.

Four different panel end conditions were investigated to determine the ways that conditions influence transverse bending stresses at prestress transfer. **Table 4** describes the differ-

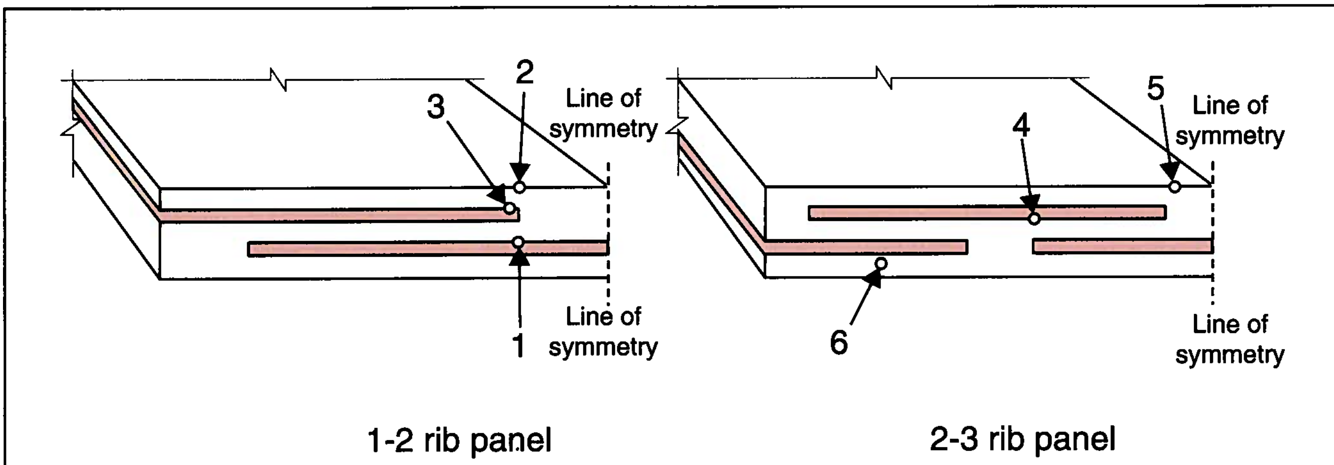

Fig. 13. Two different panel cross sections considered in the study of the effect of prestressing force location on transverse bending.

Table 3. Maximum Transverse Tension Stresses for Various Prestress Locations

	Prestressing Location	Maximum Transverse Stress, psi	Location
1-2 Rib Panel	C	754	1
	FB	482	2
	FCB	255*	3
2-3 Rib Panel	C	687	4
	FB	379	5
	FCB	155*	6

* Maximum stresses are radial stresses around line load from prestress.

Note: C = center wythe; FB = face and back wythes; FCB = face, center, and back wythes. 1 psi = 0.006895 MPa; 1 kip-in. = 0.113 kN-m; 1 psi = 0.006895 MPa.

ent end conditions (cases A, B, C, and D). Only the 1-2 rib panel was investigated in this study, and only the case in which the prestressing force was applied in the center wythe was considered. This combination created the largest transverse stresses (Table 3). Comparing the maximum transverse tension stress in the center wythe, 37%, 26%, and 12% stress reductions were obtained for cases B, C, and D, respectively, compared with case A.

In case B, stresses vary with respect to the debonded length. The maximum transverse tension stresses were reduced with increased debonded length, and the stress reduction occurred for both the 1-2 and 2-3 rib panels. Based on the analysis results, half of the total number of strand requires a debonded

Table 4. End Conditions to Reduce Transverse Bending Stresses

Panel End Condition	Modification	Description of Panel End Condition
Case A	None	Basic case (no attempt was made to reduce the transverse bending stresses)
Case B	Debond	Partially debonded strands (half of the strands were debonded for 6 ft [2 m] at the end of the panel)
Case C	Shear connector	Steel plates placed at the end of the panel were used as shear connectors
Case D	Shear connector	Solid concrete regions through the entire panel thickness placed at the end of the panel were used as shear connectors

length of at least 6.0 ft (1.8 m) to keep the panel from cracking.

An accurate prediction of the transverse cracking in three-wythe panels is complicated by uncertainties such as panel imperfections caused during panel production, bond action between concrete and insulation, and thermal and shrinkage effects in a panel. Lee and Lee and Pessiki describe a test program that further investigates panel behavior at prestress transfer.

Panel Behavior at Service Loads

The general flexural behavior of three-wythe panels was investigated at service loads. All analyses treat a 3-2-3-2-3 panel considering both 1-2 rib and 2-3 rib panels. Prestressing force and self-weight of the panel were ignored to more clearly show the stresses caused by wind loading. The perfect bond FEM model with the simply supported, quarter-symmetry boundary conditions was used for all FEM analyses. The model's finite-element mesh is shown in Fig. 9.

Elastic Load-Deflection Behavior of Three-Wythe Panels—Figure 14 plots the uncracked elastic load-deflection behavior of the 1-2 and 2-3 rib panels. For comparison, the load-deflection responses of composite and non-composite panels are also plotted.

As indicated in Fig. 14, the load-deflection behavior of both the 1-2 and 2-3 rib panels was similar to that of the composite panel. In addition, comparing the 1-2 rib and 2-3 rib panels, the 2-3 rib panel was slightly stiffer than the 1-2 rib panel, which was expected because of greater shear lag effects in the 1-2 rib panel.

Panel Flexural Stress Distribution at Service Loads—Figure 15 depicts the flexural stress distribution at a service wind load of 32 psf (160 kg/m²). The quarter-symmetry panel configuration is shown in Fig. 15. Figure 15 also plots the flexural stress distribution at selected Y locations across the panel width (lines a-a, b-b, c-c, and d-d) on the back wythe at Z = 0. The flexural stress distribution is also shown through the panel thickness at X = 36 in. (910 mm), which was at a quarter width of the panel (locations e, f, g, and h).

A non-uniform flexural stress distribution existed across the panel width

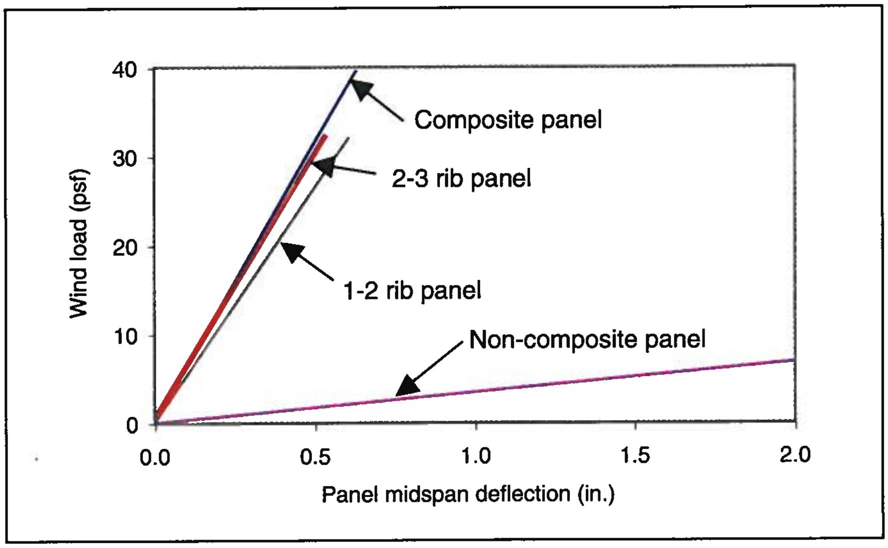

Fig. 14. Uncracked elastic load–versus–deflection behavior of 1-2 rib and 2-3 rib panels. Note: 1 in. = 25.4 mm; 1 psf = 4.882 kg/m².

(Fig. 15). This non-uniform stress distribution was relatively greater at the end of the panel (line d-d) and became more uniform at the midspan of the panel (line a-a). Relatively larger stresses occurred at the panel edge compared with the stresses in the mid-width of the panel. This was due to the shear lag effect in the panel. The flexural stress distribution through the panel thickness was also non-uniform. Non-uniform stress distribution was relatively greater at the end of the panel (location h) and decreased toward the midspan of the panel (location e).

Superimposed Transverse Bending—As noted, transverse bending of the three-wythe panels was a concern at prestress transfer. This transverse bending was also studied in combination with the service loads.

The 1-2 rib and 2-3 rib panels were investigated with the four different end conditions (cases A, B, C, and D) listed in Table 4. Both pressure and suction wind loads were considered. All panels were 10 ft (3 m) wide and 50 ft (15 m) long, and all were 3-1.5-3-1.5-3 panels. The 10 ft panel width was selected to match the width of the prototype panels treated in an experimental program.[2,3] Separate FEM analyses were performed for prestress transfer and at 32 psf (160 kg/m²) service load. These two sets of stress results were superimposed.

For 1-2 rib panels under the combined action of prestressing force and service loads, all panel end conditions had maximum stresses that were greater than the cracking stress ($7.5\sqrt{f'_{ci}}$), except for case C. Thus, crack formation is likely. Case A had the greatest maximum transverse stress, followed by cases B, D, and C. In contrast, under the action of a suction wind load, all transverse stresses were less than the cracking stress.

For the 2-3 rib panels under the combined action of prestressing force and service loads, all panel end conditions had maximum stresses that were less than the cracking stress, except for case A. Case A had the greatest maximum transverse stress followed by cases B, C, and D. Under the action of a suction wind load (similar to the 1-2 rib panels), all transverse stresses were less than the cracking stress.

CONCLUSIONS

Following are the major conclusions based on the design studies in this paper:

- Three-wythe panels can be designed using current design codes with special considerations given to transverse stresses at the panel ends.
- Composite behavior of the three-wythe panel is provided by solid concrete regions. As a result, the three-wythe panel is suitable for longer panel spans compared with two-wythe panels.
- Longitudinal and transverse rein-

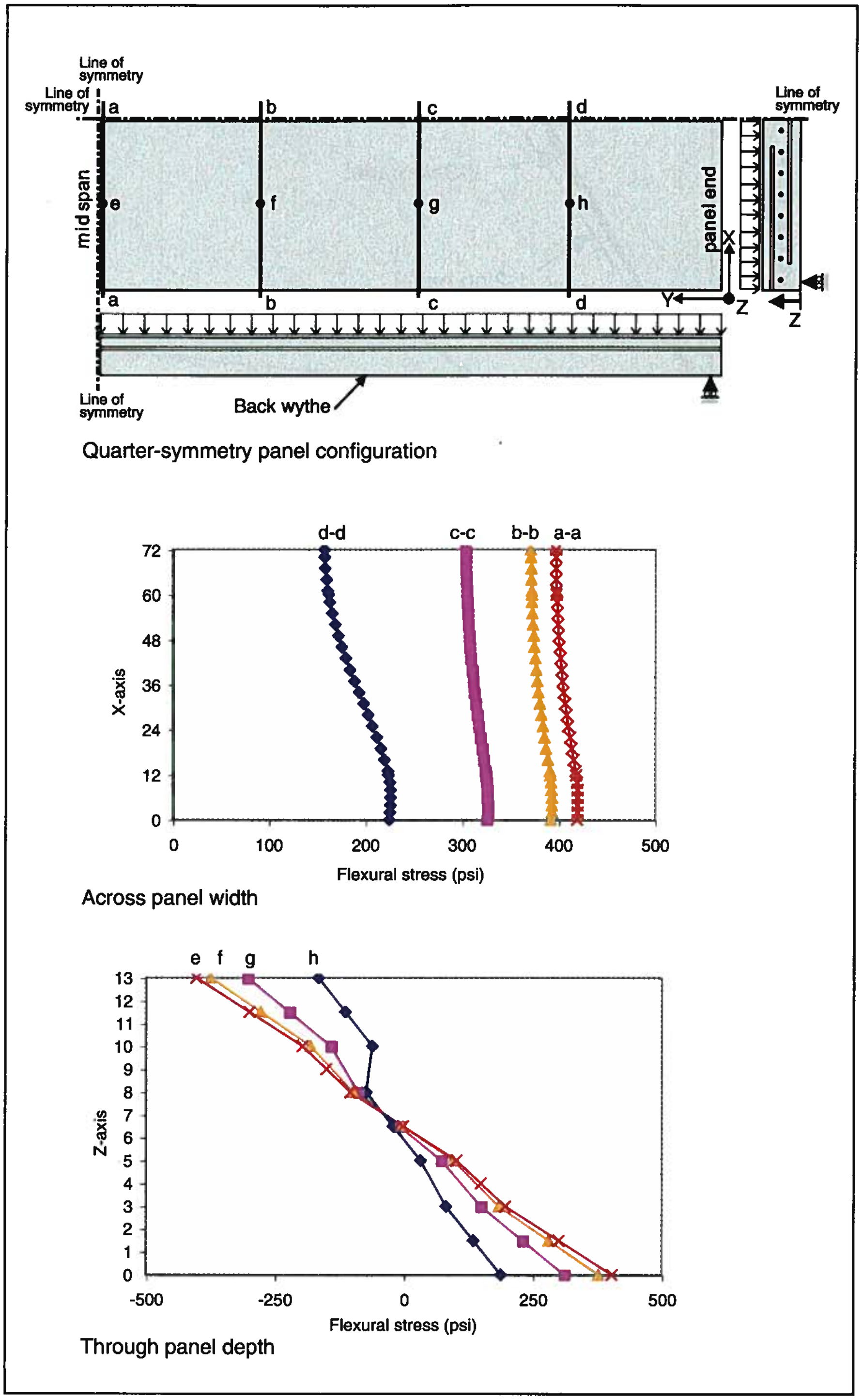

Fig. 15. Flexural stress distribution through panel width and thickness for 1-2 rib panel. Note: 1 psi = 0.006895 MPa.

forcement should be provided to prevent abrupt flexural failure and horizontal shear failure of the panel, respectively. The amount of the longitudinal reinforcement is small for practical panels.

- Fixing the maximum panel deflection at 0.75 in. (19 mm) greatly limits the range over which the three-wythe panel may be used compared with a deflection limit of *L*/480.

Major conclusions were based on the FEM analyses in this paper:

- Transverse bending occurs locally at the ends of a three-wythe panel at prestress transfer unless the prestressing force is applied uniformly in all three concrete wythes. Several approaches may be used to reduce the transverse bending that occurs, such as using partially debonded strands and shear connectors (steel plates and solid concrete blocks).
- When the prestressing force is applied only at the center wythe, the longitudinal stress distribution is non-uniform in the back and face wythes, both across the width and through the depth of the panel. This non-uniform stress distribution is due to a shear lag effect and is greatest at the ends of a panel.
- The flexural stress distribution in a three-wythe panel at service loads is non-uniform, both across the width and through the depth of the panel. This non-uniform stress distribution is significant at the ends of the panel, but a relatively uniform stress distribution occurs at the midspan of the panel.
- Transverse bending of three-wythe panels, caused by differential prestressing force between wythes, was investigated under the action of prestressing force and service load. The transverse bending stresses vary depending on the layout prestressing force, and the direction of the service load (pressure or suction).

ACKNOWLEDGMENTS

A PCI Daniel P. Jenny Fellowship funded this research. Additional funding was provided by PCI, the Pennsylvania Infrastructure Technology Alliance, and the Center for Advanced Technology for Large Structural Systems (ATLSS) at Lehigh University. Technical support was provided by High Concrete Structures Inc., Composite Technologies Corp., Dayton Superior Corp., H. Wilden and Associates, Metromont Prestress Co., Morse Bros. Inc., Nitterhouse Concrete Products, Owens Corning, Stresscon Corp., and Tindall Corp. We gratefully acknowledge support from these sponsors. The findings and conclusions presented in this paper are those of the authors and do not necessarily reflect the views of the sponsors.

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APPENDIX

Notation

C = compressive force in any segment as in Eq. (1)

E_c = concrete modulus of elasticity at 28 days

E_{ci} = concrete modulus of elasticity at transfer of prestress

E_i = insulation modulus of elasticity

E_{ps} = prestressing steel modulus of elasticity

f'_c = concrete compressive strength at 28 days

f'_{ci} = concrete compressive strength at transfer of prestress

f_{ci} = initial prestressing force of a panel

f_{pe} = effective prestressing force of a panel

f_{pu} = ultimate strength of prestressing steel

f_{py} = yield strength of prestressing steel

L = panel span length

M_{cr} = cracking moment

M_f = controlling moment based on the allowable stresses

M_n = nominal flexural strength

M_u = factored moment

p = design wind pressure

P_i = initial prestressing force

T = tensile force in any segment as in Eq. (1)

V_{nh} = horizontal shear strength

V_u = horizontal shear force

ϕM_n = design flexural strength