Framework for analyzing the flexural resistance function of a precast concrete insulated wall panel: Beam-spring modeling and pertinent limit states

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- The main objective of this paper is to provide a streamlined framework in which a beam-spring analysis can be used to generate the flexural resistance function of a single-span precast concrete insulated wall panel and, consequently, used to identify, characterize, and quantify pertinent limit states.
- In addition to its applicability for standard design requirements (cracking, deflections and service-level stress checks), beam-spring analysis can also be a powerful tool for inelastic or performance-driven applications, such as seismic, blast loading, and forensic investigations.
- The model can be built in most commercial structural analysis software packages, making beam-spring analysis an approachable solution for precast concrete insulated wall panels without the need for more complex finite element models.

Precast concrete insulated wall panels have become increasingly popular design solutions for exterior cladding systems because they jointly provide enhanced thermal performance and satisfactory resistance to applied lateral and/or gravity loads. Insulated wall panels have also become more widely used in high-risk facilities that require design for blast resistance while maximizing the aforementioned thermal performance.[1] A typical concrete insulated wall panel is constructed with a layer of rigid insulation sandwiched between two opposing prestressed or nonprestressed concrete wythes. To develop composite action across the panel cross section, wythe connectors are embedded in each of the two wythes and span the layer of insulation. In most cases, such connectors are manufactured using low-thermal-conductivity materials, such as fiber-reinforced polymers, to minimize thermal bridging through the connector and thus further enhancing the thermal performance of the wall panel. The level of composite action achieved in a given wall cross section heavily depends on the inherent behavior of the concrete wythes, the strength and stiffness properties of the wythe connectors, the thickness of the insulation layer, loading and boundary conditions, and the particular design limit state of interest, and therefore, composite action is a property of the panel and not the connector itself. For these reasons, analysis of precast concrete insulated wall panels can vary significantly in complexity and computational effort required, though the "Influence of Connector Ductility on Panel Limit States" section of this paper will analyze the impact that various types of shear connectors have on the composite action, strength, and ductility of an insulated wall panel.

Traditional analysis methods for precast concrete insulated wall panels have relied on simplifying assumptions, such as assuming a level of composite action when checking stresses, calculating deflections, or determining ultimate flexural strength. Furthermore, some approaches recommended designing for 100% composite action simply by providing a sufficient quantity of wythe connectors on each half-span to equal the nominal interwythe shear demand at the ultimate strength of the panel.[2] (See the "Composite Action Theory for Precast Concrete Insulated Wall Panels" section for the underlying theory for this assumption.) Although this approach may be appropriate for other types of composite structures that have stiffer connectors, such as steel-concrete composite floor systems, the relatively higher flexibility of polymer-based wythe connectors often results in a progressive connector failure mechanism instead of all connectors reaching their peak capacity and thus failing simultaneously.[3,4]

The main objective of this paper is to provide a streamlined framework in which a beam-spring analysis can be used to generate the flexural resistance function of a single span precast concrete insulated wall panel and consequently used to identify, characterize, and quantify pertinent limit states. Furthermore, the flexural resistance function can then be used in design or analysis applications requiring in-depth examinations of performance-driven limit states or damage levels, such as those necessitating blast, impact, or seismic resistance. The structure of the beam-spring analysis approach in this case consists of fiber-based beam-column and discrete spring elements used to represent the concrete wythes and shear resistance provided by wythe connectors and insulation, respectively. The model can be built in most commercial structural analysis software packages, making beam-spring analysis an approachable solution for precast concrete insulated wall panels without the need for more complex finite element models. After validating a beam-spring analysis model against a set of experimental test data on insulated wall panels, the framework summarized here facilitates deeper examinations of critical limit states, observations of expected failure modes, and the capability to optimize the design of the wall panel cross section. Many previous studies have largely used the beam-spring analysis approach to assess the performance of the wall panel strictly in the elastic range; however, this framework provides a unified methodology that is applicable across the entire response history of a monotonically loaded panel in flexure. Therefore, in addition to its applicability for standard design requirements (for example, cracking, deflections and service-level stress checks), beam-spring analysis can also be a powerful tool for inelastic or performance-driven applications, such as seismic, blast loading, and forensic investigations. These applications will be demonstrated throughout the course of this paper, first by conducting a deep examination of the inherent mechanics of precast concrete insulated wall panels, followed by the implementation of such mechanics to develop the beam-spring analysis model, then using a streamlined approach to interpret the analysis results, and finally concluding with several case studies, including those containing experimental test data for further validation of the methodology. The resulting framework, therefore, will provide an essential set of skills for design professionals, precast concrete fabricators, wythe connector manufacturers, researchers, and forensic structural engineers tasked with understanding the behavior of and subsequently optimizing the performance of precast concrete insulated wall panels.

Application of flexural resistance functions

A flexural resistance function is generally the relationship between the level of load resistance (in the form of force, pressure, moment, and so forth) exhibited by a structure and the corresponding deformation (in the form of deflection, support rotation, and so forth) at a known point on the structure. This function, when expressed in terms of moment resistance, also inherently contains all of the pertinent limit states throughout the loading history in flexure and, for precast concrete insulated walls, can include the onset of flexural cracking, wythe connector and insulation bond failure, and the ultimate flexural resistance. Not all of these milestones may be pertinent for applications where the design of these panels is likely to be controlled by lifting/handling or standard service-level design loads (such as dead, live, and wind loads); however, it may facilitate more straightforward calculation of stresses and deflections, even at relatively lower levels of applied load. In contrast, for panels subjected to more severe loading conditions—such as blast, impact, or seismic demands—the complete flexural resistance function is often needed to solve the dynamic equation of motion as part of a larger generalized single-degree-of-freedom analysis, as in PCI's *Blast-Resistant Design Manual* and the U.S. Army Corps of Engineers Protective Design Center's *Single-Degree-of-Freedom Blast Effects Design Spreadsheets (SBEDS)* (PDC-TR 06-01 revision 1).[1] More specifically, blast-resistance design methods (which will be an intended application of the beam-spring analysis framework for the purposes of this paper) often use dynamic increase factors to increase material design strengths as an approximate way to account for high-strain rate effects during the blast-induced flexural response of the panel. A satisfactory blast-resistant design is contingent on ensuring that the maximum blast-induced deformation of the panel remains at or below a given response limit that corresponds to a given level of damage to the component.[5] Therefore, the significance of the flexural resistance function in this design check is twofold:

- Calculation of the maximum deformation inherently relies on the flexural resistance function because it is needed to solve the equation of motion.

- The levels of damage can be either prescriptively or directly tied to the structural performance milestones along its history.

Traditionally, prescriptive damage levels have been the most widely used;[5] however, recent studies have demonstrated the applicability of performance-driven blast limit states derived from the flexural resistance function of a precast concrete wall panel.[6,7] Therefore, a major objective of this paper is to use a robust beam-spring analysis framework to generate flexural resistance functions while simultaneously monitoring the pertinent structural performance milestones and their implications for blast-resistant design of precast concrete insulated wall panels.

Precast concrete insulated wall panel behavior

Composite action theory for precast concrete insulated wall panels

In the context of precast concrete insulated wall panels, composite action is defined as the degree of shear force transfer between the opposing concrete wythes (acting in the two-dimensional planes at either interface of concrete and insulation) relative to the theoretical noncomposite and fully composite extrema. Theoretical noncomposite behavior implies that each concrete wythe has its own strain profile completely independent of the other, and thus zero shear force is shared between the two (**Fig. 1**). In this case, each wythe effectively behaves as its own beam and the total flexural resistance of the wall panel is calculated by summing the resistances of the two independent wythes. Practically speaking, noncomposite response is difficult to achieve because zero interwythe shear transfer requires that any wythe connectors have zero stiffness, which is not physically possible, and the concrete–insulation interfaces are frictionless and free of any adhesion. Previous research has demonstrated that a chemical reaction between hydrating cement paste in concrete and select types of rigid insulation facilitates the formation of a chemical adhesion mechanism that must be overcome whether wythe connectors are present.[8]

Contrarily, theoretical fully composite behavior is characterized by a single, continuous strain profile through the entire depth of the wall panel cross section (that is, through both concrete wythes and the insulation layer) as illustrated in Fig. 1. A continuous strain profile can only exist if the assumption that plane sections remain plane is valid. Furthermore, no interwythe shear slip is, therefore, possible in this scenario if the plane sections remain plane theory is to remain valid. Zero shear slip in conjunction with the nonzero shear force required for composite action suggests that wythe connectors must exhibit infinite stiffness. Because such connector behavior is not physically possible, neither is a fully composite panel response in realistic applications.

Because noncomposite and fully composite responses are purely theoretical, all practical precast concrete insulated wall panel designs fall into the partially composite behavior classification. Bounded by the theoretical noncomposite and fully composite moment curvature responses (Fig. 1), partially composite behavior occurs when the shear force transferred between the two concrete wythes is nonzero but less than the resultant magnitude of the force couple resulting from cross-sectional equilibrium in the fully composite scenario.

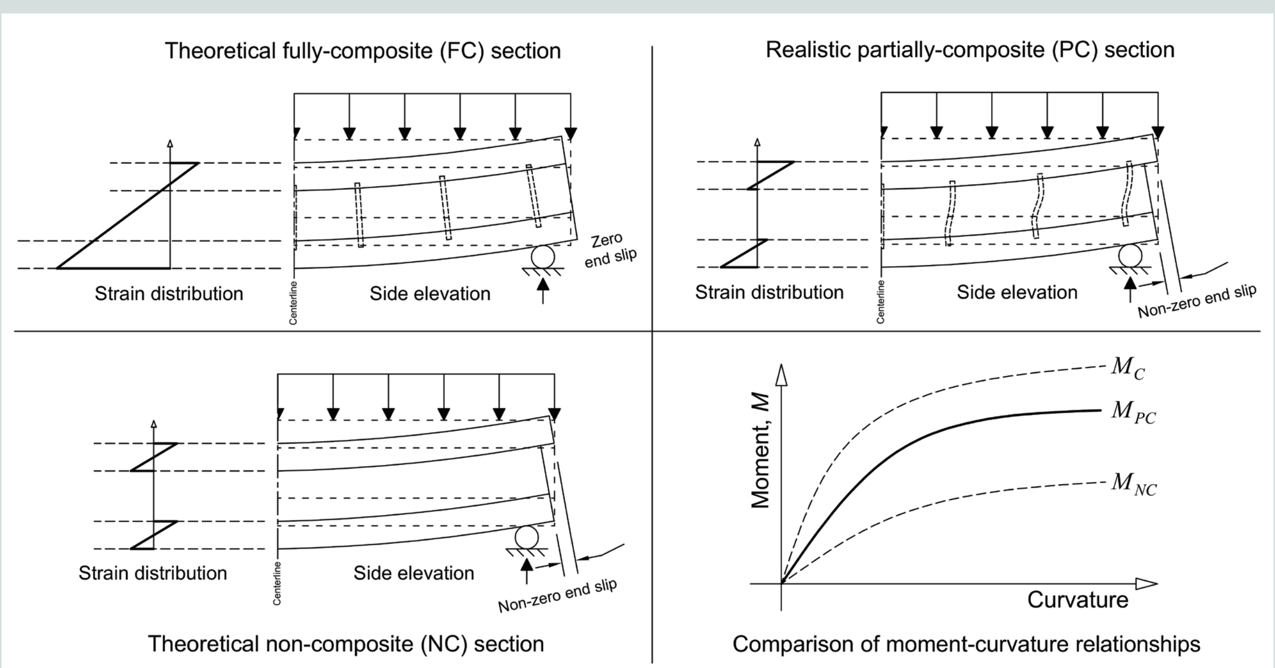

Figure 1. Precast concrete insulated wall panel composite behavior classifications and relative moment-curvature responses. Note: M_{FC} = fully composite moment response; M_{NC} = noncomposite moment response; M_{PC} = partially composite moment response.

Although the strain profile cannot be continuous through the entire cross-section depth in this case, the strain profiles of the two wythes are not completely independent of each other and considerable interwythe shear slip can develop as shown in Fig. 1.

Limit states in flexure

The following sections summarize and provide pertinent information for the possible limit states of a precast concrete insulated wall panel loaded in flexure. In real wall panel applications, flexure is typically induced by lateral wind pressure or suction that can cause outward or inward lateral deflection, depending on the location of the panel in the structure relative to the direction of the wind. Lateral deflection may also be caused by second-order load-deflection effects when an applied gravity force is exerted on a load-bearing wall panel. Please refer to "State of the Art of Precast/Prestressed Concrete Sandwich Wall Panels" for more information on these and other applicable loading conditions.[2]

Onset of flexural cracking Like most reinforced concrete members, the onset of flexural cracking in insulated wall panels occurs when the strain in the tension-most fiber of the cross section reaches the strain corresponding to the peak tensile strength of the concrete. Panels without prestressing are generally expected to crack at significantly lower levels of applied lateral loading, and thus this limit state may be especially critical for such members. Many precast concrete insulated wall panels will be designed below the cracking threshold at service, often aided by prestressing to achieve Class U (uncracked). Traditional mechanics of flexural cracking use the modulus of rupture as the aforementioned tension strength limit because this parameter is determined using a plain concrete specimen that is loaded in flexure until the cracking moment (also the ultimate moment capacity in the case of the unreinforced plain concrete section) is reached in accordance with ASTM C78[9] or C293.[10] For theoretical noncomposite panels with wythes of equal thickness, the onset of flexural cracking is expected to occur at the tension-most face of both wythes, and due to the noncomposite assumption, both wythes should crack simultaneously (Fig. 1). Therefore, when using a beam-spring analysis model (with fiber cross sections representing the wythes) to detect the onset of flexural cracking, it is worthwhile to monitor the extreme tension fibers for both wythes. For theoretical fully composite panels, only one extreme tension fiber is present at the tension-most outer face of the entire panel cross section (Fig. 1). Last, for the practical partially composite case, the extreme tension fiber is again expected to manifest at the tension-most face of the section (that is, in the same location as the theoretical fully composite case); however, some cracking is also possible at the tension-most fiber of the other wythe, with tensile strains growing larger at that face as composite action through the overall cross section trends closer to 0% (that is, approaches the theoretical noncomposite case) as demonstrated in Fig. 1. Severe flexural cracking may disrupt the embedment of wythe connectors installed in the cracked wythe; however, significant loss of interwythe shear transfer is not expected because flexural cracks typically develop transverse to the span direction and the strong axis of the wythe connectors should align with the span direction. Therefore, even if a transverse crack were to intercept a wythe connector, the connector would likely be able to bridge the crack without complete loss of embedment and subsequently shear transfer. Further experimental testing is needed to fully support the mechanics behind this phenomenon, but such a disruption in shear transfer is currently not accounted for in the beam-spring analysis framework discussed here.

Crushing of extreme compression fiber As with standard reinforced concrete cross sections, a compression-driven failure mode is possible if the allowable compressive stress is exceeded at the extreme compression fiber of the cross section. Although not commonly a controlling limit state for precast concrete insulated wall panels, it can occur at ultimate flexural strength if nonprestressed wythes are heavily overreinforced or if excessive prestressing is applied to a section, with the exception of a high level of composite action present between the two concrete wythes for either of these situations. Most beam-spring analysis models will incorporate expected material property definitions such as the expected shear-strain relationship for concrete in compression. The assumption of the Whitney stress block in compression is not designed for use in fiber cross sections and will not facilitate the most fruitful examinations of strain distributions within the cross section, which is an underlying indicator to determine the composite behavior. Such constitutive material models will specify the means to determine the strain corresponding to peak compressive stress. For example, the Hognestad model calculates this value as a function of cylinder compressive strength and cylinder modulus of elasticity[11] and Popovics recommends a function of the fourth root of the peak compressive strength.[12] Regardless of the constitutive model used in the beam elements, the extreme fiber compression strain (and the corresponding extreme fiber stress) can be extracted from the appropriate fiber within the beam element discretization to ensure that the maximum allowable values are not exceeded. Exceeding this limit state may also cause a numerical instability in the model because the solver then has to interpolate on the postpeak region of the stress-strain curve as the fibers that have not yet exceeded this milestone remain below the peak in the still monotonically increasing portion of the curve.

Interwythe shear failure of insulation-concrete bond The previous two sections focused on the limit states of the concrete wythes, but an interwythe shear failure mechanism is also plausible in response to loading in flexure. Due to the relationship between the flexural curvature in the member and the corresponding interwythe shear deformations,[3] the magnitude of interwythe shear slip is expected to increase as the applied lateral loading on the panel increases. Experimental double shear tests have demonstrated that in specimens where fresh concrete is placed against rigid insulation, a chemical bond is expected to form at the concrete–insulation interface.

Realistically, a perfect bond may be difficult to achieve in a precasting plant if placement of the insulation board on top of the fresh concrete placed as the bottom wythe (that is, with respect to the orientation in the formwork during fabrication) is delayed or walked on by the fabrication crew.[8] The opposite side of the insulation is much more likely to have a more robust bond at its interface with the top wythe because fresh concrete will be placed directly onto that surface. Theoretically, if either one of the two insulation–concrete interfaces (that is, on either side of the insulation board) does not develop the bond, a shear break exists, the bond mechanism will be bypassed, and shear resistance will be provided by some combination of friction and the mechanical strength of the wythe connectors in shear. Friction at that interface can provide significant shear resistance regardless whether a chemical bond forms in the first place.

The concrete–insulation friction and bond mechanisms are much stiffer than typical polymer-based wythe connectors alone,[8,13] especially due to the large surface area over which they act compared to a discrete or one-dimensional, linearly distributed layout of flexible wythe connectors. Therefore, a significant loss of interwythe shear stiffness and, consequently, composite action is possible at relatively low levels of applied lateral load at this limit state. When extruded polystyrene (XPS) insulation is used and installed to be bonded to the concrete, a drastic and sudden shedding of shear resistance can be observed as the bond interface fails[8] and the remaining concrete–insulation friction and mechanical strength of the wythe connectors further engages. This phenomenon is less relevant for bonded expanded polystyrene (EPS) insulation because the bond failure typically is not sudden but rather involves a more gradual internal shearing of the EPS foam beads[8] instead of abruptly popping away from the adjacent concrete.

Interwythe shear failure of wythe connectors An extensive database exists that contains the results of experimental research on various types and configurations of wythe connectors for precast concrete insulated wall panels.[13] Despite the relatively large dataset, there has been limited research demonstrating the direct correlations between the limit states of the wythe connectors and the failure modes of an insulated wall panel in which they are installed. This gap may be the result of the insulated wall panel design objectives, many of which are centered around early-age properties for lifting and handling and initial prestressing and ensuring that the panel does not experience flexural cracking once erected. Although these limit states are important for precast concrete construction, they occur at relatively smaller levels of applied loading and thus correspondingly smaller out-of-plane deflections, which limit the interwythe shear deformations of the wythe connectors. Nonetheless, the ultimate strength of an insulated wall panel is also an important milestone that must also meet building code and design guideline strength requirements. Because this milestone effectively corresponds to the peak resistance of the component, large out-of-plane deformations will cause significant shear demands and corresponding deformations to occur in the wythe connectors, especially in those in the regions of the highest shear demand.[3,4] Therefore, understanding the pertinent failure modes of different types of wythe connectors will allow engineers to properly account for such mechanisms in a beam-spring analysis and ultimately ensure that an unwarranted interwythe shear failure of the connectors does not keep the precast concrete insulated wall panels from satisfying the required design provisions.

Most types of wythe connectors are manufactured using fiber-reinforced polymers (FRPs) to provide a balance between structural integrity and low thermal conductivity, the latter to avoid discrete points of thermal bridging across a desired continuous insulation layer.[14] One downside of common types of FRP materials is their inherent lack of ductility caused by the brittle nature of fiberglass strands encased in resin. At the ultimate strength limit state, the ductility of the wythe connectors can have a substantial influence on the peak resistance of the insulated wall panel and can trigger an unzipping interwythe shear transfer mechanism.[3] At low levels of panel composite action at the strength limit state (recall that composite action varies along the load-deformation history of an insulated wall panel as demonstrated by Gombeda et al.[3]), shear deformations are expected to develop at relatively low levels of applied lateral loads and these deformations will be greatest in magnitude at the locations of the highest shear force (that is, at the end of a simply supported panel subjected to a uniformly distributed out-of-plane load). Therefore, the first wythe connectors to fail (that is, their peak shear resistance is reached for the purposes of the beam-spring model) will be concentrated near the highest shear demands. Once these first connectors fail, the remaining interwythe shear demand must be redistributed to the next-closest row of connectors until they fail and so on. This mechanism will propagate until either the loss of wythe connector resistance causes the panel (now with substantially lower composite action) to fail in concrete crushing, reinforcement rupture, or overall instability or the maximum expected applied lateral load is reached before the unzipping mechanism propagates to near the center of the panel span length (where the shear demands are theoretically zero if panel symmetry is upheld regardless of the level of applied load).

The rate of propagation of the aforementioned unzipping mechanism will heavily depend on the ductility of the wythe connectors,[3] in addition to the layout/configuration of the connectors.[4] Therefore, connectors with greater ductility can allow the panel to accrue larger out-of-plane deformations before achieving its ultimate strength milestone. Though most FRP materials are not designed to exhibit significant ductility, certain FRP connectors have demonstrated the ability to develop relatively large shear deformations[13] due to a gradual splitting or delamination mechanism as fiberglass strands attempt to debond or separate from the surrounding polymer resin. Although the connector has reached its peak shear resistance when this mechanism occurs, its significant postpeak resistance causes a delay of the unzipping mechanism, which results in higher panel ductility. The splitting/

delamination mechanism is also encouraged by the development of both shear and flexural stresses in non-truss-style wythe connectors. Delamination is largely due to internal, microscopic shear failures occurring in the connector body that result from a nonuniform distribution of strain through its thickness, a distribution that must be caused by bending and not purely axial or direct shear forces. Truss-style connectors theoretically only develop axial forces (due to the inherent structural mechanics of their namesake), which hinders the development of this relatively ductile mechanism. Some proprietary or prototype wythe connectors may comprise an unreinforced polymer or composite material. For these types of connectors, their shear deformation capacity should, theoretically, correlate back to the stress-strain relationships of the polymer or composite material from which they are manufactured. For example, many types of polycarbonates, nylons, or impact-resistant polymers are designed for relatively larger ultimate tensile strains, which can facilitate the development of a stable plastic hinge in the member or extended ductility in some cases. A drawback of unreinforced polymers is relatively lower ultimate tensile strength and stiffness (due to the lack of fiberglass strands), which can require a larger quantity of wythe connectors on a panel half-span to satisfy interwythe shear demands. One of the major advantages of the beam-spring analysis approach compared with more simplified methods is the ability to accept the entire load-deformation relationship of the wythe connectors that inherently contains all pertinent information regarding strength, stiffness, and ductility.

Construction of the beam-spring model

Figure 2 presents the beam-spring analysis approach described herein. Construction of the model begins by establishing the pertinent constitutive properties (step A), including

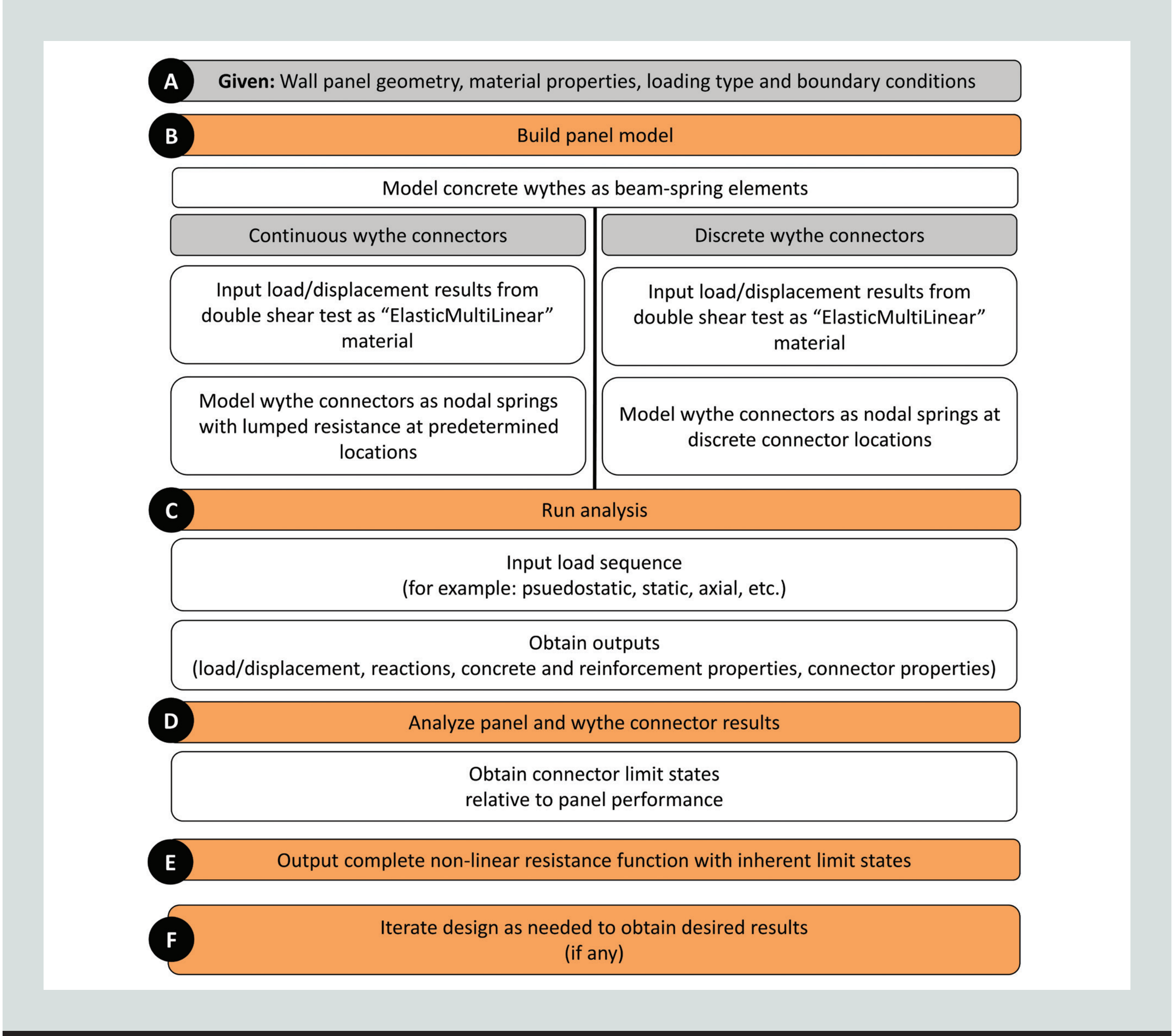

Figure 2. Framework for precast concrete insulated wall panel analysis using beam-spring analysis.

those of the concrete wythes and the interwythe shear resistance. The model can then be built using standard beam and spring elements (step B) for the concrete wythes and shear resistance, respectively, to facilitate ease of implementation. After running the analysis (step C), the overall performance of the panel and the individual rows of wythe connectors can be examined (step D) to determine their interrelationships and influence on pertinent limit flexural states. Last, the full, nonlinear resistance function can be determined (step E) either for comparison with experimental test data (to validate the modeling approach for a given application) or performance design trials, with iterations as needed (step F) to achieve specific design objectives.

Main attributes of the computational model

The general structure of a beam-spring model for this application consists of beam–column elements with fiber cross sections that represent the prestressed or nonprestressed concrete wythes and discrete spring elements populated with the constitutive relationship of the wythe connectors and insulation in shear (that is, along the span length of the panel) (**Fig. 3**). This relationship is generally quantified as the total interwythe shear resistance as a function of shear deformation, and this curve is typically determined from double shear tests of a double-wide mock-up insulated wall panel section with an appropriate wythe connector layout and insulation in accordance with an acceptable test method, such as AC422.[15] The curve resulting from the double shear tests will, therefore, reflect the conditions in which the sample was designed or prepared (for example, purposely unbonded insulation and connector spacing) and quantify the total shear resistance at each deformation increment. The stress-strain relationships for concrete and steel reinforcement can be adopted from previous research or literature. Recommended concrete stress-strain models are from Hognestad[11] or Popovics,[12] and those for steel reinforcement can generally be derived from knowing the material type and grade or from obtaining actual stress-strain results from the steel mill or an independent testing laboratory. A design-oriented set of stress-strain curves for prestressed reinforcement can be found in the *PCI Design*

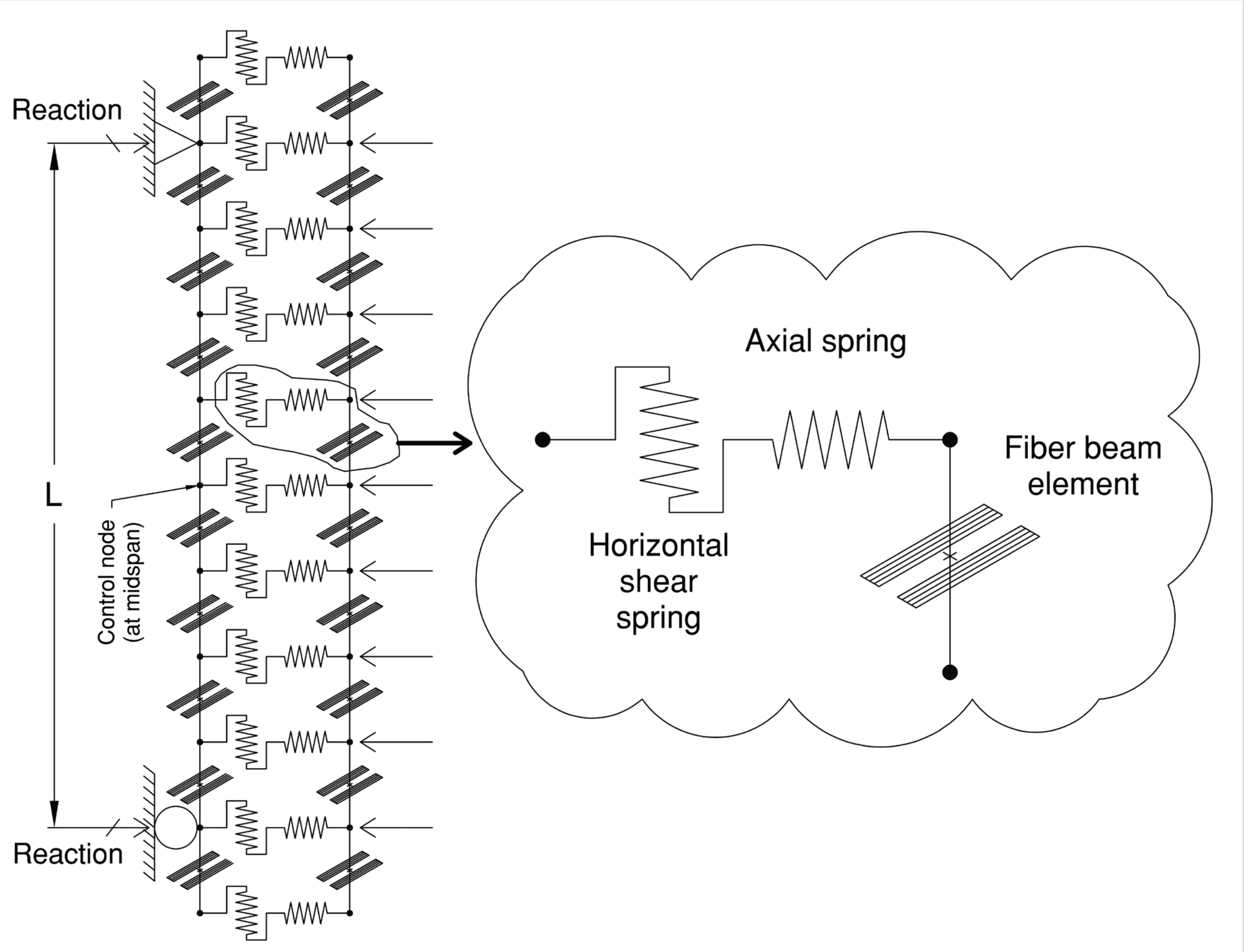

Figure 3. Typical structure of the computational beam-spring analysis for precast concrete insulated wall panel used herein. Note: L = length.

Handbook: Precast and Prestressed Concrete.[16]

Depending on the program in which the beam-spring model is built (see the "Main Attributes of the Computational Model" section), slight modifications to the aforementioned constitutive relationships/models for concrete, steel reinforcement, and interwythe shear resistance may be beneficial in facilitating numerical convergence and the computational stability of the model. A prime candidate for such an adjustment is the tension portion of the concrete stress-strain relationship. An open crack cannot transfer stress across it; however, a subtle amount of tensile strength degradation is possible before the crack fully forms, even in concrete members without the addition of fiber reinforcement. After the extreme tension fiber of the panel's concrete cross section reaches the strain corresponding to peak tensile strength, the model will look to interpolate the postpeak tension response on the next iterations with increasing strain in that fiber. If a stress-strain relationship assumes the degradation back to zero stress happens instantaneously, the numerical solver may struggle to find equilibrium convergence on subsequent iteration steps because the relationship is no longer a mathematical function (that is, it now has multiple strength values possible at a given strain or, in applicable engineering terms, an infinite stiffness) in that case. Implementation of some degree of tension softening (degradation from peak tensile strength down to zero stress with a finite negative slope) can facilitate more straightforward convergence in the panel's postcracking response. Typically, the model will prompt for a peak tensile concrete strain and an ultimate tensile concrete strain, the beginning and end of the tension softening portion of the relationship, respectively. The former can be calculated as the quotient of the flexural strength (that is, modulus of rupture as discussed previously) and the elastic modulus. For cases where postpeak tension softening is integrated to facilitate numerical convergence, a limit of two times the peak tensile concrete strain is recommended for the ultimate tensile concrete strain; any larger ratios may cause excessive artificial tensile strength to be accounted for when analyzing the resistance of the section. To facilitate further postcracking convergence, the available concrete tensile strength past the ultimate concrete tensile strain shall be set to zero for all greater-in-magnitude values of tensile strain. A similar strategy to promote numerical convergence of the model can also be used for the interwythe shear resistance relationship. Because the aforementioned unzipping wythe connector mechanism manifests due to connectors experiencing a drop in postpeak shear resistance, fine adjustments to any abrupt or brittle portions of the interwythe shear relationship procured from testing may be needed to prevent the model from numerically failing before the full mechanism develops.

Implementation of the beam-spring model in structural analysis software

A beam-spring analysis can be coded into most computer programming languages or platforms; however, for practical design applications, an open-source or commercially available structural analysis software package is recommended. In addition, several specialized precast concrete design software packages have modules specifically built for precast concrete insulated wall panels. Some design software packages will provide the user with a visual interface in which all pertinent input parameters can be defined and may set default fundamental analysis parameters, such as individual element types, number of integration points, and solver tolerance. With the use of any design software, the engineer or user should verify that the program is generating the desired output for a given case, which can be accomplished using simple, small tests where the results can be straightforwardly calculated by hand.

Open-source platforms typically require some application-level programming (that is, inputting predefined command lines into an input file with a predefined structure rather than coding from scratch using a base language) that is usually straightforward after a brief learning curve. The advantages of using this type of modeling structure are a generally low or even zero cost to use the software, lower computational costs (such as time and computer memory), and ease of data extraction. The last advantage is that each requested dataset is saved in a separate output file that the user provides a name and directory for in advance, so the user doesn't have to search through a more complex user interface or large output reports as is common in some commercial software packages. An open-source analysis platform was used for a series of case studies later in this paper (see the "Comparison with Experimental Test Data and Failure Modes" section) and, therefore, it will be helpful to provide more specific details pertaining to the development of a model. (This discussion is not needed for most commercial design software given their own specific guidance.) Previous research has shown that displacement-based beam–column elements have been effective for representing prestressed or nonprestressed concrete wythes,[3,4] partly due to the inherent complexity of the model and the ability to capture the spread of plasticity along the element length. The latter is particularly important at the ultimate strength limit state as the panel is expected to undergo significant lateral deflection, and the state of a plastically deformed reinforced concrete cross section must be monitored relative to the interwythe shear mechanism to properly identify the controlling failure mode at that limit state. The former reason is especially relevant because these elements must also transfer the reactions from the two node link elements that represent the wythe connectors. The number of elements should be determined based on the spacing between wythe connector rows (that is, each row of wythe connectors is represented by a node along the length of the panel), and the optimal number of integration points can be determined from a sensitivity study performed after the model is initially built. Two node link elements in the structural analysis software use multiple springs to represent the three permissible degrees of freedom for wythe connectors: translation parallel to the panel thickness (Spring1, Fig. 3), translation along the panel length (Spring2, Fig. 3), and rotation of the connector about an axis transverse to the panel length and in the plane of the panel's flat surface. Spring1 should be populated with a stiffness value that accounts for the behavior of the rigid insulation layer in compression and, in a numerical sense, ensures the stability of the model as lateral loads are applied to the panel. Spring2 is populated with the interwythe shear versus deformation relationship derived from experimental double-shear testing of a mock-up wythe connector and insulation assembly as discussed previously. Last, the rotational spring should also be used if the shear resistance of the connectors is measured in a direct shear test, that is, not using the double-shear method that inherently also captures the flexibility of the connectors (due to a moment arm that exists within the insulation layer thickness) in addition to the shear force capacity. More specific details about the structural analysis modeling structure can be found in Gombeda et al.[3]

Calculating panel resistance functions

This section presents a series of case studies conducted using an open-source structural analysis platform following the procedure outlined in Fig. 2, to demonstrate how the flexural resistance function and the inherent behavior of precast concrete insulated wall panels can be appropriately captured using the beam-spring analysis framework. Comparisons with experimental test data on insulated wall panel specimens and the results of beam-spring modeling will be used to validate the proposed framework. This effort will be followed by a parametric study to assess the influence of varying wythe connector ductility on the sequential development of pertinent wall panel limit states in flexure.

Comparison with experimental test data and failure modes

This case study intends to provide a holistic review of a wide variety of panels and their corresponding flexural resistance functions in an effort to provide a ubiquitous match from the beam-spring modeling results with experimental test data. The panels chosen for analysis in this paper were emulative of a variety of precast concrete insulated wall panel applications including both load-bearing and non-load-bearing cladding panels and blast-resistant facades. The specimens can be classified using three main facets:

- P represents prestressed, and N represents nonprestressed.
- D represents discrete, and C represents continuous connectors.
- E represents EPS, and X represents XPS insulation.

Pertinent fabrication details for these panels can be found in **Table 1** provides pertinent fabrication details for these panels. The panels with an asterisk were not analyzed using the beam-spring approach and were instead included for discussion of precast concrete insulated wall panel failure modes only. The other panels were used extensively in the analysis portion of this paper. The analyzed panels were selected due to the availability of mechanical properties obtained from direct testing of the constituents (that is, double shear tests for connectors and compressive strength reported pursuant to ASTM C39[17]). In addition, the selected panels showcase a variety of failure modes, aiming to highlight the relationships between precast concrete insulated wall panel characteristics and common failure modes. Of particular importance are panels where flexural failure was ultimately driven by the wythe connectors, effectively creating an interwythe shear controlled mechanism.

The modeled precast concrete insulated wall panels in this section were obtained from test data from five sources.[4,18–21] Tested panels were uniformly loaded using either an air[18] or water bladder[18,20] or with a load tree.[20,21] Nafadi et al.[19] opted to apply two symmetric lateral point loads emulative of wind pressure from suction. In addition, several sources employed an additional axial load[18,19] when testing load-bearing panels. All of the analyzed sources tested precast concrete insulated wall panels to failure using either displacement-controlled[4,20,21] or load-controlled[18,19] test setups. Gombeda et al.[4] and Naito et al.[21] employed a displacement rate of 0.01 in./sec (0.25 mm/sec), while Trasborg[20] did not specify a displacement rate. Maguire and Al-Rubaye[18] employed a load rate of 2 lb/ft^2 per minute (9.72 kg/m^2 per minute), and Nafadi et al.[19] increased their lateral loads in 1 kip (4.45 kN) increments until failure was achieved. All panels were simply supported on emulative roller pins and were loaded flat with the exception of those tested by Nafadi et al.,[19] which were loaded vertically. **Figure 4** shows analyzed precast concrete insulated wall panel wythe connector layouts (Table 1 references the corresponding panel configurations), where *j* is the row of wythe connectors from the outermost edge of the panel span increasing toward the panel midspan. Figure 4 shows variable connector layouts with increased wythe connectors toward the edge and fewer connectors towards the midspan. Figure 4 also shows continuous wythe connectors where j was determined by dividing the half-span into regular intervals for lumping the shear resistance in the analysis.

Of the panels tested by Maguire and Al-Rubaye,[18] NCE3 behaved the most similarly to a fully composite panel due to the inadvertent solid zones produced during panel fabrication, while the panels with discrete connectors (NDX5, NDX6, NDX7, and NDX8) exhibited lower levels of composite action. Panels NDX5 and NDX8 failed in interwythe shear, while NDX6 and NDX7 achieved peak flexural capacity when the steel reinforcement yielded. At the peak flexural capacity for panels NDX6 and NDX7, Maguire and Al-Rubaye[18] observed that the outermost wythe connectors exhibited slip comparable to the yield plateau observed on the load-slip profile for the wythe connectors, which aligns with the results of the beam-spring modeling analysis conducted in this paper. Although precast concrete insulated wall panels NDX5 and NDX8 both failed in horizontal shear, NDX8 achieved a higher flexural capacity than NDX5 due to the optimized configuration of wythe connectors.

Table 1. Panel model parameters

Panel identification (original panel identification)	Connector type	Insulation type (thickness, in.)	Concrete wythe type (thickness, in.)	f'_c, psi	Longitudinal reinforcement per wythe	Span, ft
PCE1* (PCS2†)	CFRP grid	EPS (2)	prestressed (3)	8396	two ⅜ in. strand	10
NCE2* (PCS3†)	CFRP grid	EPS (2)	nonprestressed (3)	8410	two no. 5 bars	10
PDX1‡ (PCS5†)	FRP pin	XPS (3)	prestressed (3)	8787	two ⅜ in. strand	10
PCX1‡ (PCS6†)	CFRP grid	XPS (3)	prestressed (3)	8686	two ⅜ in. strand	10
NDX1‡ (PCS7†)	FRP pin	XPS (3)	nonprestressed (3)	8860	two no. 5 bars	10
PDX2‡ (PtXX§)	FRP pins	XPS (3)	prestressed (3)	6003	two ⅜ in. strand	10
NDX2‡ (RtXX§)	FRP pins	XPS (2)	nonprestressed (3)	6003	two no. 5 bars	10
PCE2‡ (PtEG§)	CFRP grid	EPS (2)	prestressed (3)	8396	two ⅜ in. strand	10
NCE1‡ (RtEG§)	CFRP grid	EPS (2)	nonprestressed (3)	8410	two no. 5 bars	10
PDE1* (PtEL§)	SS C clip	EPS (2)	prestressed (3)	8222	two ⅜ in. strand	10
NCE3* (A‖)	CFRP grid	EPS (4)	nonprestressed (2.5)	3785	three no. 4 bars	40
NDX5* (B#)	GFRP pin	XPS (2)	nonprestressed (4)	6163	six no. 4 bars	40
NDX6* (C#)	FRP truss	XPS (2)	nonprestressed (4)	6163	six no. 4 bars	40
NDX7* (D#)	GFRP X pin	XPS (2)	nonprestressed (4)	6322	six no. 4 bars	40
NDX8* (E#)	FRP pin	XPS (2)	nonprestressed (4)	5496	six no. 4 bars	40
NDX3‡ (DT23XUU**)	DT23††	XPS (2)	nonprestressed (3)	6105	two no. 5 bars	10
NDX4‡ (DT24XBH**)	DT22 and DT24††	XPS (2)	nonprestressed (3)	5206	two no. 5 bars	10
NDE1‡ (DT24EBE**)	DT22 and DT24††	EPS (2)	nonprestressed (3)	5206	two no. 5 bars	10
PCE3* (EPS1‡‡)	CFRP grid	EPS (4)	prestressed (2)	8497	two ⅜ in. strand	19
PCX2* (XPS1‡‡)	CFRP grid	XPS (4)	prestressed (2)	8497	two ⅜ in. strand	19

Note: Specimen panel identifications use P for prestressed and N for nonprestressed, D for discrete and C for continuous connectors, and E for EPS and X for XPS insulation. DT = ductile tie; EPS = expanded polystyrene; f'_c = concrete compressive strength; FRP = fiber-reinforced polymer; GFRP = glass-fiber-reinforced polymer; SS = stainless steel; XPS = extruded polystyrene. No. 4 = 13M; no. 5 = 16M; 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 psi = 6.895 kPa.

* This panel was not analyzed using the beam-spring approach and was instead included for discussion of precast concrete insulated wall panel failure modes only.

† Data from Naito, Hoemann, Shull, and Saucier (2011).

‡ This panel was used extensively in the analysis portion of this paper.

§ Data from Trasborg (2014).

|| Data from Maguire and Al-Rubaye (2022).

Data from PCI Industry Handbook Committee (2021).

** Data from Gombeda, Naito, and Quiel (2021).

†† Data from Gombeda, Naito, and Quiel (2020).

‡‡ Data from Nafadi, Lucier, Yaman, Gleich, and Rizkalla (2021).

More important, the wythe connectors in NDX8 nearer to midspan did not exhibit large slips, while the end wythe connectors exhibited slip values equal to the maximum slip observed along the load-slip profile for the wythe connectors. Gombeda et al.[4] also examined varying wythe connector configurations (prioritizing shear resistance at the end of the precast concrete insulated wall panel with most wythe connectors placed in this region in Fig. 4) and found that compared with uniform configurations, varied wythe connectors exhibited lower end slip values at peak precast con-

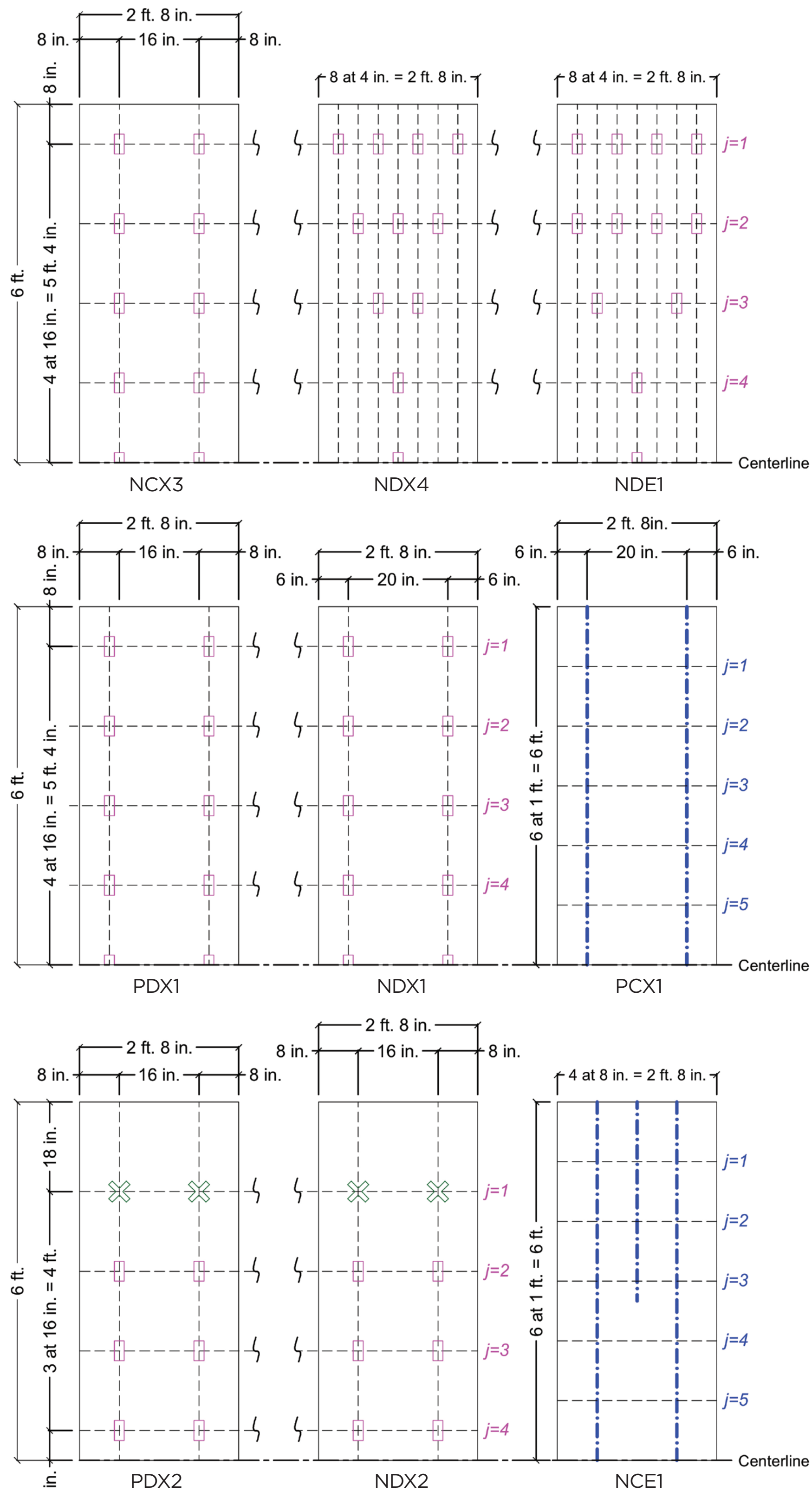

Figure 4. Modeled precast concrete insulated wall panel layouts. Note: Specimen panel idenfitifcations use P for prestressed and N for nonprestressed, D for discrete and C for continuous connectors, and E for EPS and X for XPS insulation. EPS = expanded polystyrene; *j* = row of wythe connectors from the outermost edge of the panel span increasing toward the panel midspan location; XPS = extruded polystyrene. 1 in. = 25.4 mm; 1 ft = 0.305 m.

crete insulated wall panel flexural capacity. This behavior is the direct result of higher concentrations of shear resistance at the end regions of the panel where shear demands will be highest. Furthermore, the increased shear resistance at the ends of a precast concrete insulated wall panel allow for improved flexural capacity of the panel (**Table 2**).

When comparing panels PDX1 and NDX1, the wythe connectors in PDX1 (a prestressed panel) experienced significantly less shear force at the onset of cracking and the connectors passed their peak shear resistance at the panel's peak moment resistance, whereas the connectors in NDX1 (nonprestressed) were significantly closer to their peak shear force at the onset of cracking but just reached their peak shear force at the peak panel capacity. This behavior indicates that the connectors are sufficient for the capacity of the nonprestressed panel and are overloaded in the precast concrete panel. Furthermore, the modeling results outlined in **Fig. 5** through **7** provide a good fit

Table 2. Comparisons of experimental and beam-spring modeling peak moment response

Panel identification (original panel identification)	Peak test moment, kip-in.	Deflection at peak moment, in.	Peak model moment, kip-in.	Deflection at peak moment, in.	Model/test peak moment
PCE1* (PCS2†)	293.66	1.92	212.32	0.56	0.72
NCE2* (PCS3†)	304.53	3.96	214.97	3.76	0.71
PDX1‡ (PCS5†)	278.32	5.15	235.73	7.82	0.85
PCX1‡ (PCS6†)	247.69	3.06	218.87	1.01	0.88
NDX1‡ (PCS7†)	266.74	6.36	236.68	3.82	0.89
PDX2‡ (PtXX§)	316.97	3.95	261.05	2.42	0.82
NDX2‡ (RtXX§)	315.74	5.23	297.17	3.44	0.94
PCE2* (PtEG§)	355.70	1.36	234.98	1.51	0.66
NCE1‡ (RtEG§)	263.16	1.43	222.61	1.74	0.85
PDE1* (PtEL§)	266.60	1.14	222.26	5.08	0.83
NCE3* (A||)	681.30	13.41	409.31	10.53	0.60
NDX5* (B#)	540.17	4.15	509.59	5.08	0.94
NDX6* (C#)	932.98	11.94	897.40	20.00	0.96
NDX7* (D#)	931.07	13.68	896.83	20.00	0.96
NDX8* (E#)	703.50	6.16	922.36	8.37	1.31
NDX3‡ (DT23XUU**)	139.17	5.99	166.19	4.00	1.19
NDX4‡ (DT24XBH**)	235.57	5.84	208.76	4.57	0.89
NDE1‡ (DT24EBE**)	222.95	6.38	212.72	4.04	0.95
PCE3* (EPS1††)	431.70	1.17	375.25	1.41	0.87
PCX2* (XPS1††)	797.64	1.87	803.17	2.26	1.01

Note: Specimen panel identifications use P for prestressed and N for nonprestressed, D for discrete and C for continuous connectors, and E for EPS and X for XPS insulation. EPS = expanded polystyrene; XPS = extruded polystyrene. 1 in. = 25.4 mm, 1 kip-in = 0.113 kN-m.

* This panel was not analyzed using the beam-spring approach and was instead included for discussion of precast concrete insulated wall panel failure modes only.

† Data from Naito, Hoemann, Shull, and Saucier (2011).

‡ This panel was used extensively in the analysis portion of this paper.

§ Data from Trasborg (2014).

|| Data from Maguire and Al-Rubaye (2022).

Data from PCI Industry Handbook Committee (2021).

** Data from Gombeda, Naito, and Quiel (2021).

†† Data from Nafadi, Lucier, Yaman, Gleich, and Rizkalla (2021).

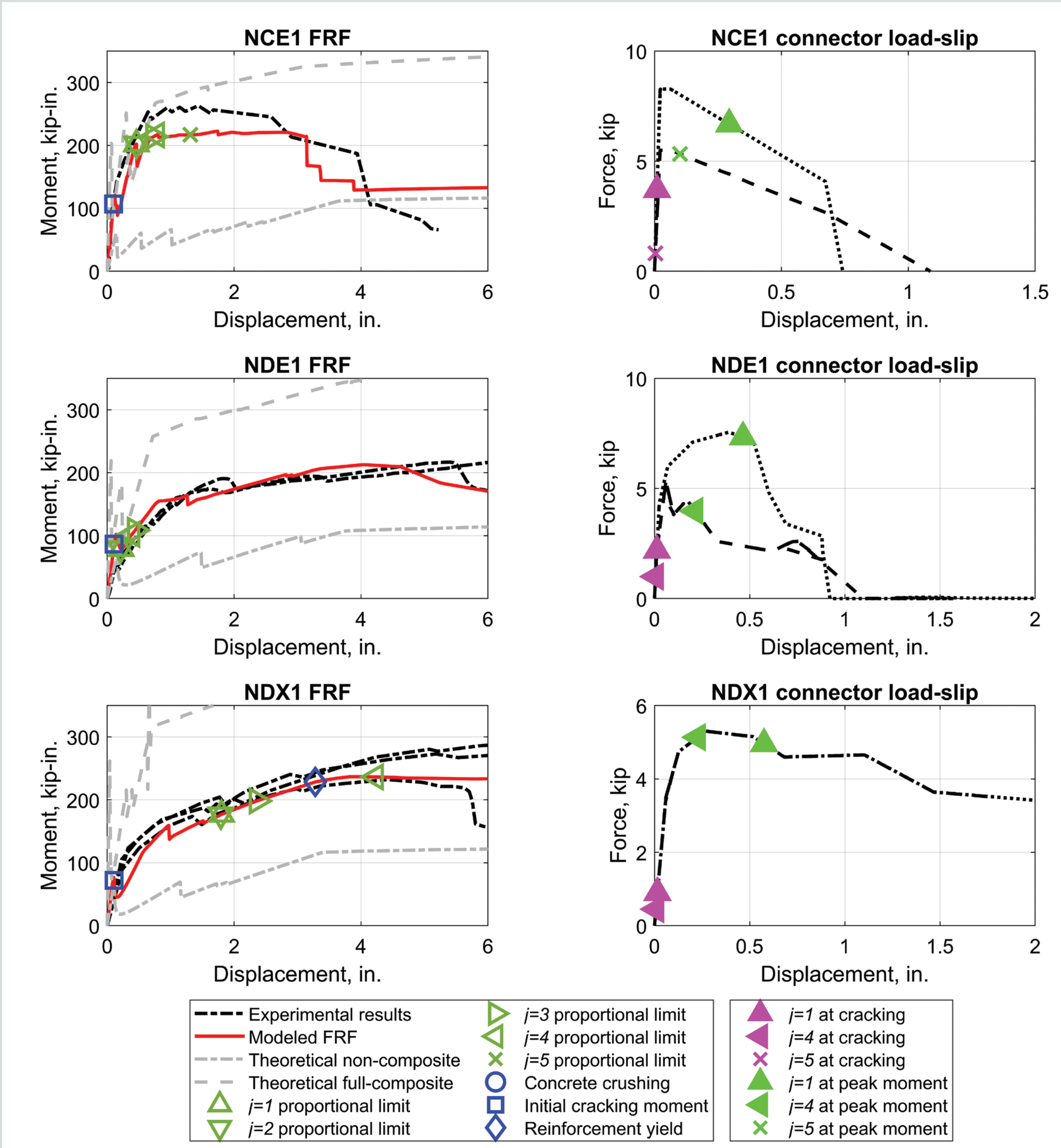

Figure 5. Resistance functions and connector load-slip profiles for panels NCE1, NDE1, and NDX1. Note: Specimen panel identifications use N for nonprestressed, D for discrete and C for continuous connectors, and E for EPS and X for XPS insulation. EPS = expanded polystyrene; FRF = flexural resistance function; *j* = row of wythe connectors from the outermost edge of the panel span increasing towards the panel midspan location; XPS = extruded polystyrene. 1 in. = 25.4 mm; 1 kip = 4.448 kN; 1 kip-in. = 0.113 kN-m.

to the experimental data, indicating that the beam-spring analysis accurately predicts precast concrete insulated wall panel behavior given the mechanical performance of the constituent materials that compose the precast concrete insulated wall panel component. In general, the precast concrete insulated wall panel model results provided a conservative estimate of strength, with the majority of the ratios of model to experimental peak moment resistance being less than 1 (Table 2).

At peak moment capacity in panels NCE1 and PCX1, the continuous wythe connectors are only just past their proportional limit (Fig. 5 and 7, respectively). The discrete wythe connectors analyzed for this study have a wider range of load-slip behavior compared with the selected continuous wythe connectors, but all of the analyzed precast concrete insulated wall panels with discrete connectors exhibited the onset of cracking with their connectors near the beginning

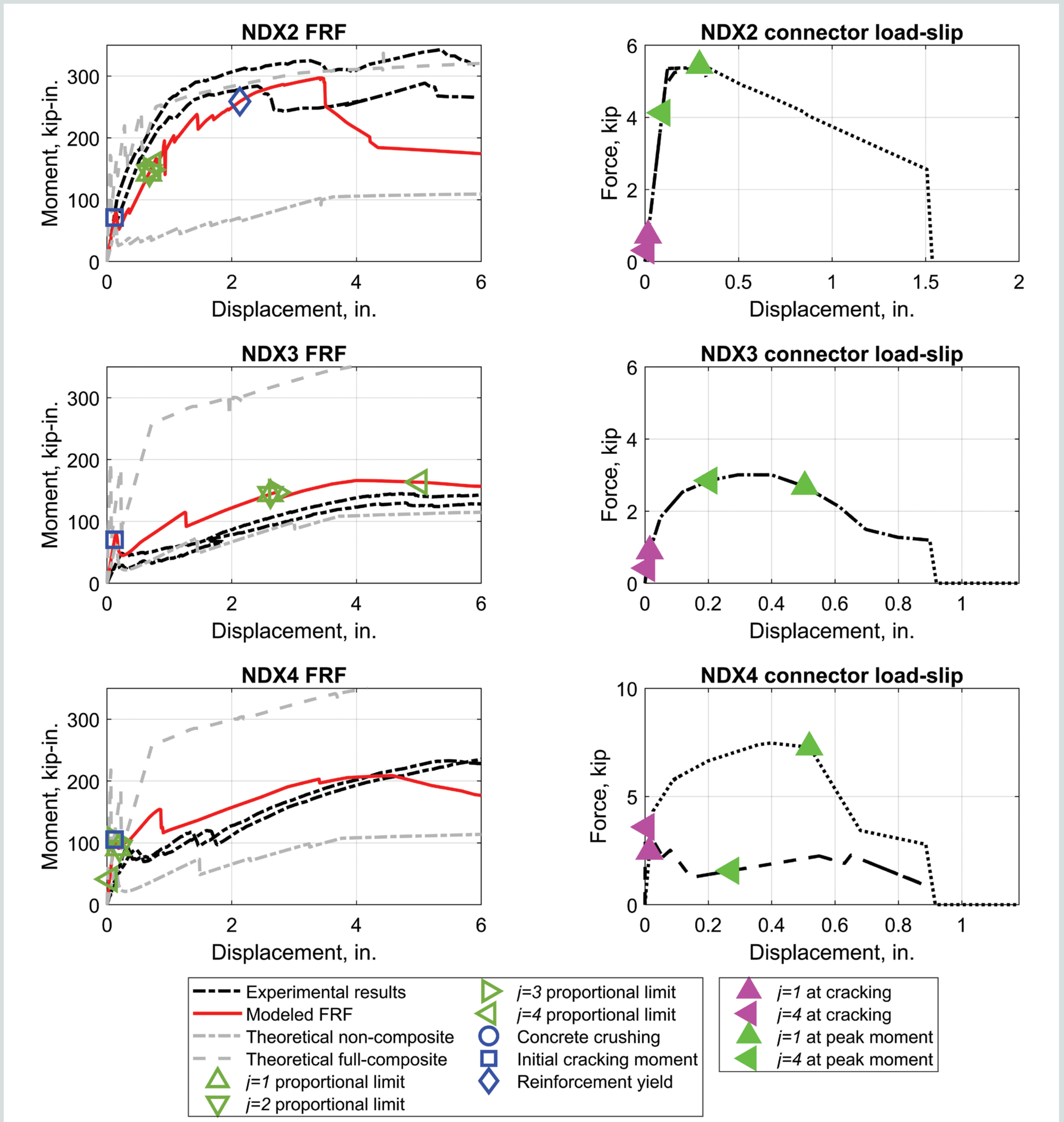

Figure 6. Resistance functions and connector load-slip profiles for panels NDX2, NDX3, and NDX4. Note: Specimen panel identifications use N for nonprestressed, D for discrete connectors, and X for XPS insulation. FRF = flexural resistance function; j = row of wythe connectors from the outermost edge of the panel span increasing toward the panel midspan location; XPS = extruded polystyrene. 1 in. = 25.4 mm; 1 kip = 4.448 kN; 1 kip-in. = 0.113 kN-m.

of the linear-elastic region of load-slip profile (with the exception of NDX1, where the connectors were all nearing peak at the onset of cracking). Panels NDX3, NDX4, and NDE1 show that components made with bonded insulation outperform those with unbonded insulation, but overall, the impact that insulation bond has on panel response is minor compared with the role that connector type plays. In particular, Nafadi et al.[19] found during testing that the EPS panel (PCE3 in this study) failed at a lateral load of 100 lb/ft^2 (4.72 kPa) while the sandblasted XPS panel (PCX2 in this study) that they tested failed at 175 lb/ft^2 (8.38 kPa), which is directly attributed to the higher bond strength between the concrete and insulation. The validation provided here for the beam-spring analysis relative to experimental data substantiates the use of beam-spring modeling for continued analysis of precast concrete insulated wall panels with different input

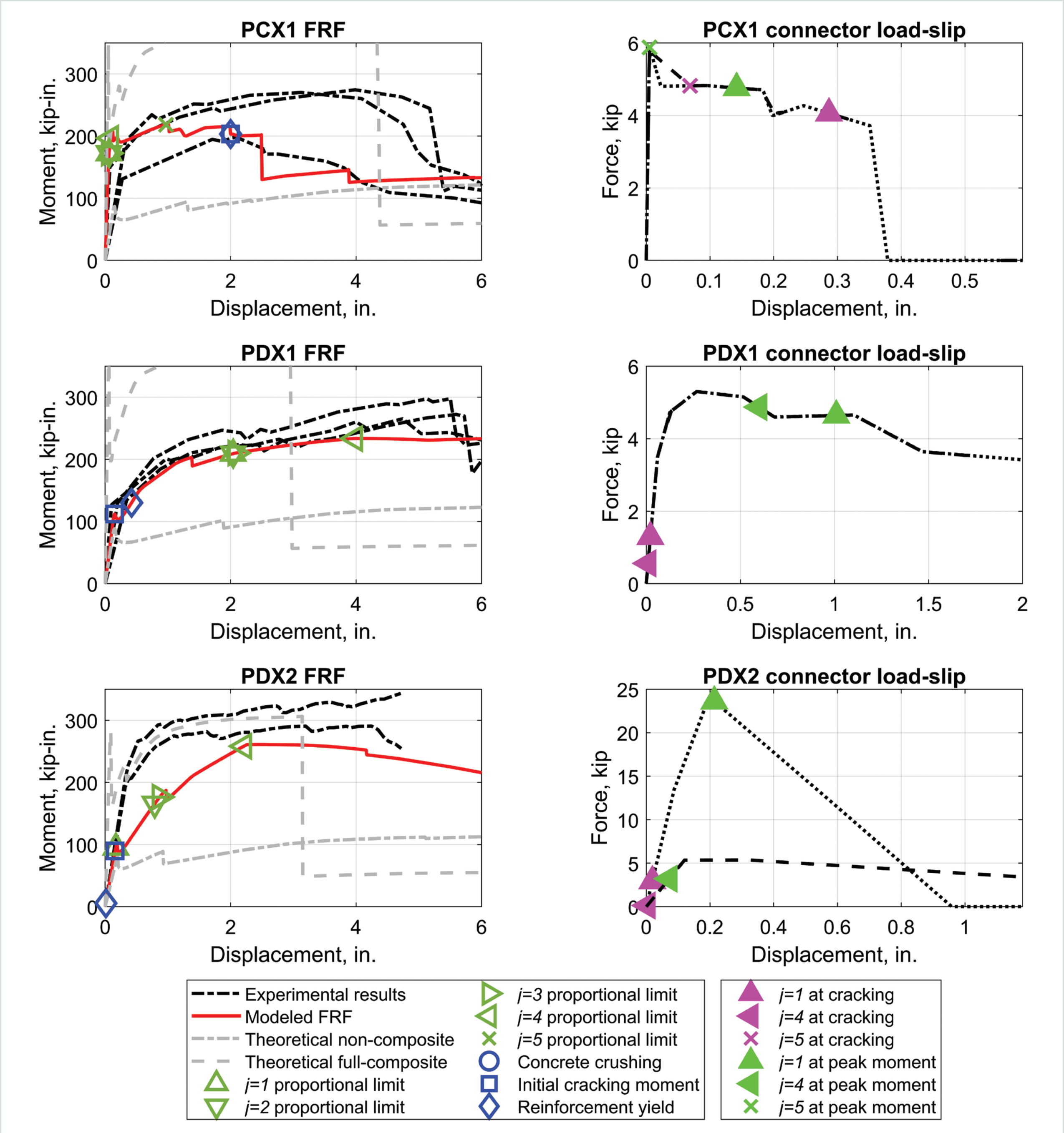

Figure 7. Resistance functions and connector load-slip profiles for panels PCX1, PDX1, and PDX2. Note: Specimen panel identifications use P for prestressed, D for discrete and C for continuous connectors, and X for XPS insulation. FRF = flexural resistance function; *j* = row of wythe connectors from the outermost edge of the panel span increasing toward the panel midspan location; XPS = extruded polystyrene. 1 in. = 25.4 mm; 1 kip = 4.448 kN; 1 kip-in. = 0.113 kN-m.

parameters to obtain desired flexural resistance functions and assess the failure modes and behavior of precast concrete insulated wall panels pursuant to different wythe, concrete, and reinforcement input properties.

Wythe connectors in rows *j* equal 1 (outermost) and the closest to midspan (*j* equal to 4 or 5, depending on the analyzed precast concrete insulated wall panel, denoted in Fig. 4) are plotted on the load-slip profile in Fig. 5 through 7, highlighted at the iteration of initial cracking moment in magenta and peak moment capacity in green to illustrate the progression of the connectors with the most and least shear demand at these limit states as the panel is loaded in flexure. The wythe connectors in row *j* equal 1 dictate the limit state as these progress along the load-slip profile and experience the most shear demand under flexural loading due to the inherent mechanics that govern precast concrete insulated wall panel limit states and behavior.

Influence of connector ductility on panel limit states

Following successful validation of the beam-spring modeling framework, the methodology was then used to examine the effect of connector ductility on the overall ductility and failure mechanisms of a series of theoretical precast concrete insulated wall panel configurations. The experimental panel configuration for this facet of research can be found in Fig. 4 and used discrete connectors for ease of analysis. The variable connector backbone curves can be found in **Fig. 8**, and the connector load-displacement was iterated using strain energy, which is taken as the area under the load-displacement curve. The increments of strain energy were 1, 2, 3, and 4 kip-in. (0.1, 0.2, 0.3, and 0.5 kN-m) and were increased by extending the displacement available to the theoretical wythe connector at peak load, which was taken as 6 kip/connector (27 kN/connector). Postpeak behavior for all of the connectors analyzed was taken to be identical in each case to ensure that the performance of the shear connectors and corresponding precast concrete insulated wall panels was dictated solely by the elastic region of the shear connectors. Furthermore, this section examined the extent to which the elastic region of the wythe connectors is exhausted when a precast concrete insulated wall panel is loaded and how this relates to precast concrete insulated wall panel peak flexural capacity to better understand the role that wythe connectors play in the ultimate strength limit state for precast concrete insulated wall panels.

With the extended ductility of the wythe connectors exhibiting high strain energies, these wythe connectors exhibited higher degrees of shear transfer between the concrete wythes and further contributed to improved precast concrete insulated wall panel ductility and overall moment capacity (**Fig. 9**). Shear transfer between layers is determined predominantly by the load-slip profile of the wythe connectors, with extended ductility yielding an improved degree of shear transfer. Of particular interest during this section of research was the performance of the wythe connectors during the load-response history of the precast concrete insulated wall panels under flexural loading. With the extended displacement at peak shear resistance for each case of varying strain energy, the wythe connectors were able to exhibit increased ductility at the point of peak panel moment capacity (Fig. 9). At the onset of panel cracking, all of the wythe connectors developed for this study were still on their initial linear rise of the elastic region of the load-slip profiles. This behavior indicates that, as expected, the onset of panel cracking is driven by the mechanical properties of the concrete. Figure 9 shows the performance of the connectors at the panel's peak flexural resistance and highlights that the theoretical wythe connectors in the outermost region of the panel are at the proportional limit of their respective load-slip curve, indicating that the outermost wythe connectors drove failure in the analyzed wall panels as they exhausted the proportional limit of load-slip capacity at peak panel flexural capacity.

As shown in Fig. 9, the initial elastic panel performance is nearly identical for all of the shear connectors of increasing strain energy, but the panel's ability to withstand loading was

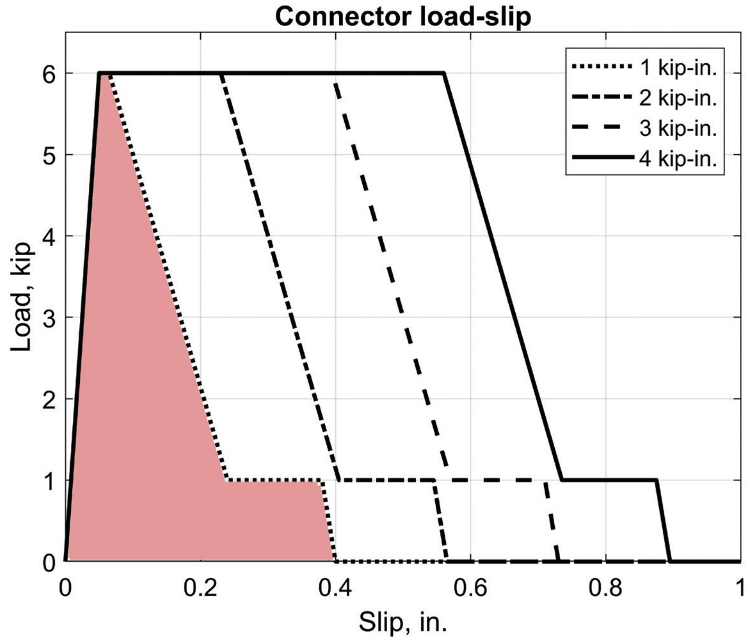

Figure 8. Theoretical variable ductility wythe connector load-slip behavior. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN; 1 kip-in. = 0.113 kN-m.

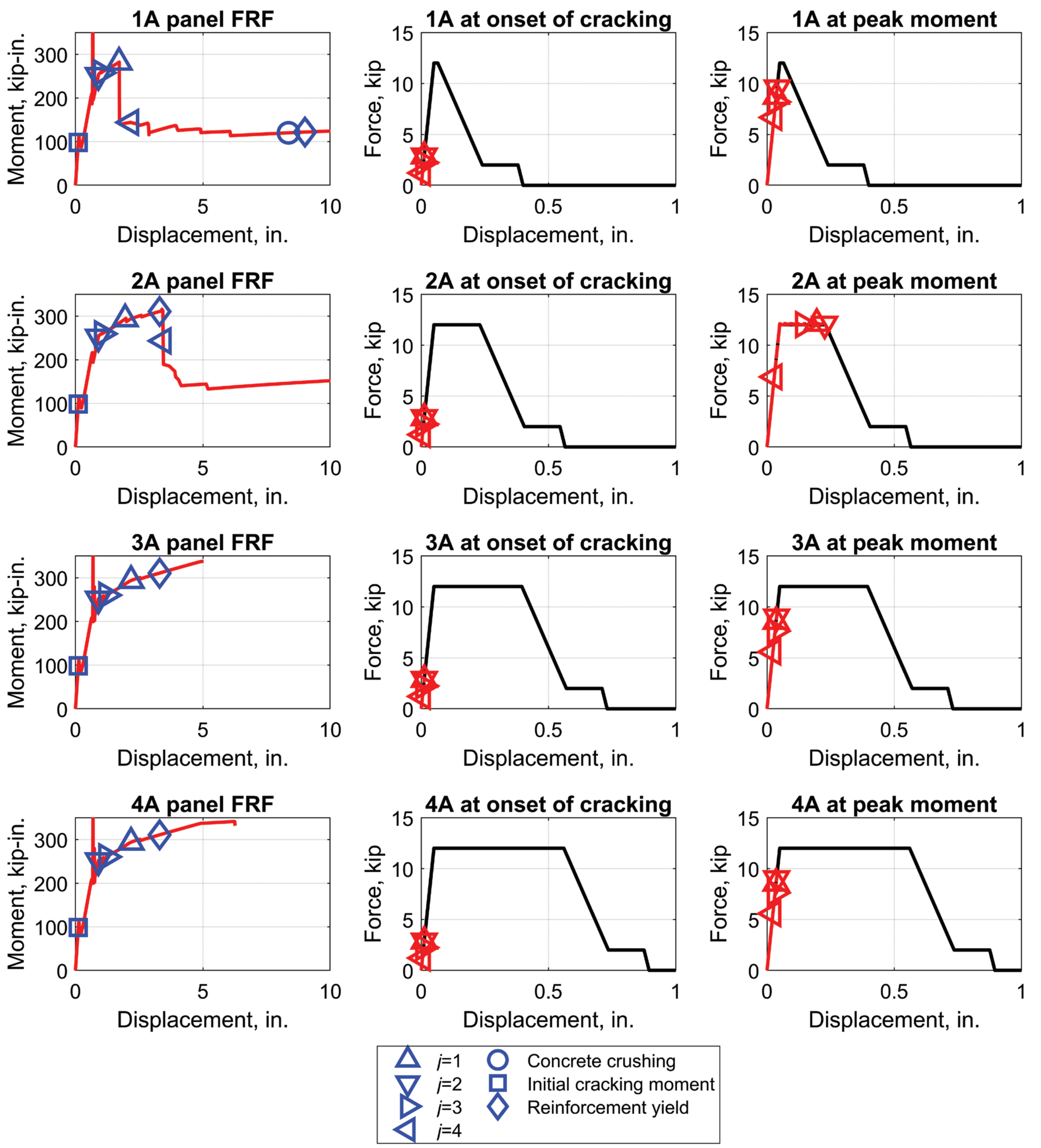

Figure 9. Theoretical variable ductility wythe connector resistance functions at onset of cracking and at peak moment capacity. Note: FRF = flexural resistance function; j = row of wythe connectors from the outermost edge of the panel span increasing toward the panel midspan location. 1 in. = 25.4 mm; 1 kip = 4.448 kN; 1 kip-in. = 0.113 kN-m.

then determined by the shear connector's ability to maintain load capacity at increasing displacements. Increasing the available ductility of the wythe connectors also increases the degree of shear transfer under sustained precast concrete insulated wall panel loading and further serves to improve the available degree of composite action, which is shown in the improved ductility of the moment-displacement curves (Fig. 9). In addition, as indicated by the outermost row of connectors (j equal to 1 in Fig. 9), the sustained ductility of the wythe connectors at increased deflection at the maximum load allows the precast concrete insulated wall panel to continue sustaining load after the wythe connector initially reaches its peak load. For example, the outermost wythe connectors all reach their initial peak load (6 kip [26.7 kN]) at or around a shear slip of 2 in. (50 mm); in the case of wythe connectors with strain energy equal to 1 kip-in. (0.1 kN-m), this achievement is directly related to precast concrete insulated wall panel failure (as shown by the immediate drop in moment capacity shown in Fig. 9), while in the increasing strain energy wythe connector cases, the panel is able to sustain load as the connector continues to deform at the maintained load.

Conclusion

This paper serves to demonstrate the usefulness of applying a beam-spring modeling framework to develop the flexural resistance function for a precast concrete insulated wall panel. This methodology can be applied to non-load-bearing or load-bearing applications that extend beyond the elastic region or special applications, such as blast or seismic loading. After highlighting the plausible flexural limit states and validating the modeling approach against a series of experimental test data, a case study demonstrated the extent to which wythe connector ductility will affect the performance of the panel, further demonstrating the usefulness of the computational design approach. More specific findings resulting from this study are as follows:

- The beam-spring modeling approach examined here provides a good fit with the experimental test results for precast concrete insulated wall panels, with the model peak moment capacity achieving 89% of the experimental peak moment capacity on average, indicating that it is a good fit for analysis of partially composite behavior. Furthermore, the results of this analysis show that the beam-spring model provided, on average, a conservative estimation of experimental strength. With the exception of NDX8 and NDX3, all model peak moment capacities were less than 100% of the experimental peak moment capacity.

- Observed failure modes for the provided precast concrete insulated wall panels include interwythe shear and panel flexural failure driven by either the reinforcing steel or wythe connectors, which can all be tied to the precast concrete insulated wall panels' mechanical properties and their subsequent interaction. In cases where the wythe connector-driven limit state is controlled, the connectors exceeded the proportional elastic limit of their load-slip profile, as was the case in panels NCE1, NDE1, NDX1, NDX2, NDX3, NDX4, PCX1, PDX1, and PDX2 (Fig. 5 through 7).

- Increasing the available strain energy of wythe connectors, in particular the ability to withstand force at increasing deformations, directly improves precast concrete insulated wall panel ductility and peak moment capacity. Extension of the proportional limit of the load-slip profile to increase strain energy directly related to precast concrete insulated wall panels improved ductility and peak moment capacity, with the limiting wythe connector deflection at sustained load determining when and how the precast concrete insulated wall panel will fail.

- These results support the use of the beam-spring approach for the analysis of insulated wall panels beyond the limits of elastic behavior, as is sometimes the case in alternative approaches. The inherent versatility of the model allows for applications where pertinent limit states exceed the elastic capacity of the section, as is the case in loading for blast, impact, and seismic considerations. The open-source and iterative nature of the beam-spring model make it a useful and easily implementable tool for precast concrete engineers looking to examine the composite action response of insulated wall panels.

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Notation

f'_c = concrete compressive strength

j = row of wythe connectors from the outermost edge of the panel span increasing toward the panel midspan location

L = length

M = moment response

M_{FC} = fully composite moment response

M_{NC} = noncomposite moment response

M_{PC} = partially composite moment response

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Abstract

This paper demonstrates the versatility of beam-spring modeling when calculating flexural resistance functions for precast concrete insulated wall panels. Modeling frameworks that accurately estimate the flexural strength of panels while identifying critical limit states are powerful tools for precast concrete engineers. Behavior quantification benefits panels that are designed to withstand conventional wind loading and be especially beneficial for more severe loading conditions, such as blast or seismic, where evaluating the nonlinear response of structures is pertinent. This paper provides a review of the mechanics and plausible limit states of these structures, develops the modeling framework reflecting the behavior of the components, and validates this methodology using comparisons with experimental test data and analytical case studies. Emphasis is placed on the influence of wythe connector shear resistance when determining efficient panel design configurations. This study shows that beam-spring modeling captures full nonlinear resistance functions and ductility of panels examined herein, providing insight on limiting failure modes for these structures.

Keywords

Beam-spring model, composite action, connector, insulated wall panel, wythe connector.

Review policy

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