Simplified Model for Partially-Composite Precast Concrete Insulated Wall Panels Subjected to Lateral Loading

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ABSTRACT

A simplified model to predict the partially composite response of precast concrete insulated wall panels to lateral loading is proposed. A conventional insulated wall panel design consists of two concrete wythes which sandwich a foam insulation layer. Shear ties are used to connect the two concrete wythes through the insulation layer – these ties can be designed to resist flexural demands via composite, partially composite or non-composite action of the section. Numerous types and configurations of shear ties (many of them proprietary) are available with a large range of constitutive properties, and thus the response of a wall panel can vary based on the type of tie used in its construction. Standard design practices do not consider the constitutive properties of the tie, compatibility of the wythes, or locations of the ties within the panel. A simplified model is proposed which incorporates the relative shear tie deformation as well as the tie's constitutive properties. The approach superimposes the sum of the force couple developed in each shear tie and the capacity of each concrete wythe via an iterative process by which the strength, deformation and level of composite action is calculated at each load increment. This model provides an accurate estimate of the load-deformation responses of these panels when subjected to lateral loads (perpendicular to the panel). The method is used to illustrate the sensitivity of flexural response in a precast insulated wall panel to tie properties and placement. The proposed model is demonstrated to be an effective tool for efficient flexural performance-based design and selective detailing of precast concrete insulated wall panels.

Keywords: Insulated wall panel, Partially-composite behavior, Precast concrete, Shear tie

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1. INTRODUCTION

Energy efficiency in buildings has become a major concern due to the threat of climate change and the increasing cost of utilities. Collins [1] demonstrated that energy saving benefits could be achieved by using a composite cladding system comprised of two concrete wythes (interior and exterior) with insulating foam sandwiched between them. The insulated wall panel concept further developed with research on the effect of insulation properties on the flexural stiffness of panels by Pfiefer and Hanson [2]. With the recent emphasis on sustainable construction, researchers and developers have refined the design of insulated panels to increase thermal resistance without compromising the structural integrity of the wall panel [3]. Shear ties are used to connect the two concrete wythes through the insulation layer – these ties can be designed to resist flexural demands via composite, partially composite, or non-composite action of the section. Thermally resistive shear ties comprised of carbon fiber-reinforced polymers (CFRP) or glass fiber-reinforced polymers (GFRP) have been shown to provide the flexural resistance required to resist the relevant design loads [4,5]. Though the thermal resistance of these panels is of great importance, it is also necessary to understand the structural behavior and failure limit states of these structures.

Accurate methods for calculating the composite response of insulated panels under service and ultimate loading are necessary for proper design of the panel system. Multiple approaches to assess the response of composite structures have been developed for a variety of structural applications. Research on composite action has been conducted on steel-concrete composite beams [6,7], concrete-timber structures [8–12], wood-based sandwich panels [13], and insulated concrete panels [14,15]. Most of these approaches assume linear elastic behavior in the structural components (including the shear ties) and do not account for a gap between interior surfaces of the structural materials where shear transfer occurs. Shear transfer relies on a combination of friction

between the insulation and concrete layers with the combined shear loading mechanism in the ties. Although experimental research has shown that a partial degree of composite action can be achieved through friction between the layers [16], the ties are needed to ensure shear capacity under service and ultimate loading. The gap created by the presence of the insulation layer between the wythes results in the development of shear, flexure and potentially axial force in the ties as the panel undergoes flexural deformation due to lateral loading (perpendicular to the panel's exterior surface). The moment developed in the ties results from a force couple created when opposing shear forces, separated by an eccentricity, are transferred to both concrete wythes at the locations of tie embedment. Traditional approaches are not easily extended to capture the localized flexural behavior of the ties.

Recently, approaches have been developed specifically for insulated wall panel systems. Benayoune et al. [17] showed that the ultimate strength and degree of composite action of a precast wall system is highly dependent upon the stiffness of the shear tie. Salmon and Einea [14] developed a method that, although effective for calculating thermal bowing deflections, is not applicable for determination of flexural strength. Bai et al. [18] developed a discrete model incorporating shear deformations and independent flexural behaviors of concrete wythes to create models of symmetric partially composite concrete sandwich panels. The model was limited to small elastic deformations and does not consider variation in deformation behavior of discretely placed tie types and their corresponding failure modes. Hodicky et al. [19] performed experimental testing on insulated concrete sandwich panels to examine the effect of varying geometric and material properties of different FRP ties on shear flow strength. By manipulating the spacing of distributed shear ties and thus the interaction surface area between the concrete and insulation, a change in the overall magnitude of shear stresses between the concrete wythes was observed. Chen

et al. [20] conducted bending tests on eight large scale concrete sandwich panels and accounted for a variation in the distribution of FRP shear ties. The results showed that different types of shear ties produce varying levels of composite action, further motivating the need to include the properties and spatial distribution of ties into analysis methods of sandwich panels. Tomlinson and Fam [21] developed an approach to model the flexural response of partially-composite insulated wall panels with varying arrangements of shear ties and wall panel geometries. This methodology consists of an iterative approach that assumes a linear slip profile and integrates strain discontinuities caused by the ties to obtain an updated slip profile, however, shear tie constitutive properties are restricted to the elastic range. This may be suitable for elastic-brittle materials; however, ductile materials, which include metals and numerous plastics will realistically yield due to inter-laminate shear forces at large panel deformations.

This paper proposes a simplified model to assess the performance of both prestressed and non-prestressed concrete insulated non-loadbearing panels under out-of-plane loading. As stated in the title, the elements examined in this manuscript are insulated wall panels intended for use as part of the building envelope. Use of these panels for use as roof or slab elements was not considered in this paper. There are some cases such as refrigeration buildings that could also utilize the design concept for the roof diaphragm, however the main application is for wall panels. These elements are designed to resist out-of-plane forces such as those generated from wind loads or pressure demands from blast events.

As an alternative to the aforementioned methods by others, the proposed model provides a simplified method for calculating panel performance from service loads through ultimate strength for a wide range of cross-section geometries and shear tie configurations. The intended application of the model is for use as a design and analysis tool and facilitates ease of implementation by the

user without the prerequisite of commercial finite element software. It allows the user to easily input all relevant geometric and material parameters and determine the behavior of the panels without the need for building complicated finite element models. The proposed model incorporates the elastic and inelastic constitutive properties of the wythe-to-wythe shear ties, thus capturing the effect of tie ductility on the overall panel. Proper use of the model enables a simplified yet effective way to design and selectively detail tie arrangements for a desired level of performance and/or an intended failure mode or limit state in response to lateral loads. The proposed model is particularly applicable for the design of precast insulated wall panels to resist impulsive lateral loads such as blast and impact.

2. BACKGROUND

Insulated precast concrete wall panels consist of a layer of foam insulation sandwiched between two layers, or wythes, of concrete. The insulating layer allows the panel to provide high thermal resistance without sacrificing the advantages offered by precast concrete construction, including design aesthetics, economic fabrication with higher quality control, or rapid erection. Shear ties which connect the two wythes through the insulation layer are commonly used to resist the shear and tension forces generated during fabrication, shipping, erection, and service life. The combinations of shear and tension on the ties vary over the loading history of the panel. In most cases compression between the wythes is assumed to be transmitted through the insulation.

While a physical connection between the concrete wythes is necessary to create composite action, this connection can allow heat to transfer directly between the wythes due to “thermal bridging,” thereby decreasing the thermal efficiency of the panel [22]. However, the effects of thermal bridging can be diminished or effectively eliminated by utilizing shear ties made with material having low thermal conductivity with small cross-sectional area. Numerous shear tie

systems (many of them proprietary) are currently available and offer varying levels of strength, stiffness, thermal resistance and cost. Shear ties may be placed at discrete locations or distributed along the length of the panel. They can be used to allow the interior and exterior wythe to work in tandem to resist external lateral loads, thereby inducing flexural response in the panel. For insulated panels constructed with shear ties, the flexural behavior can be classified as non-composite, composite or partially composite. These scenarios are examined for a typical section and span as illustrated in Figure 1A.

Composite behavior is defined in this paper as the flexural resistance achieved by assuming a linear strain profile over the panel section (see Figure 1B), similar to that in a solid concrete cross-section. Non-composite behavior exists when the interior and exterior wythes act in flexure independently to resist applied loads, and the flexural contribution of the insulation layer is assumed to be negligible. This is equivalent to the flexural response of two stacked slabs with a frictionless interface between them. Due to compatibility, the stacked slabs must have the same curvature, rotation and flexural displacement along the length. This results in measurable relative slip between the two wythes (Figure 1C). Between these two extrema is the case of partially composite behavior, which is characterized by a level of shear force transfer between the interior and exterior wythes through the connection between them (Figure 1D). These connectors allow relative slip between the panels (though less than that of non-composite panels) as a function of their shear resistance. The degree of composite action dictates the moment-curvature resistance of the section and hence the load-deflection behavior of the panel (Figure 1E). As the degree of shear transfer increases, the response will trend toward that of a composite section – conversely, decreasing levels of shear transfer will trend toward the non-composite response.

Figure 1 - Flexural behavior of insulated concrete wall panels

3. COMPUTATIONAL MODELING OF PANEL RESPONSE EXTREMA

Before discussing the development and performance of the proposed approach, the authors will first illustrate the underlying mechanics of precast insulated wall panels constructed with shear ties via a computational modeling approach, implemented using OpenSees [23]. An illustration of the OpenSees model is shown in Figure 2 where C , L , and P represent the shear tie spacing, span length, and panel fabrication length, respectively. Since the model was developed in 2D, multiple rows of shear ties at the same location along the panel length are lumped together. Concrete wythes are modeled as *Displacement-Based Beam-Column Elements* [23] with ten discretized fiber cross-sections (integration points) every 30.5 cm (12 in.) along their length. Shear ties are represented with *Two Node Link Elements* [23] (Figure 2c) and assembled together to form the wall system (see Figure 2b). The flexural resistance of the insulation layer is not physically represented in this

model but is instead incorporated into the resistance of the discrete tie as noted in Naito et al. [24]. Concrete in the wythe beam-columns is modeled using a uniaxial material with non-linear compression behavior and linear tension softening (material type *Concrete02* [23]). Linear concrete tension softening is used to facilitate convergence when the beam-column element experienced cracking. Reinforcing steel is modeled with idealized uniaxial *ElasticMultiLinear Material* [23] stress-strain behavior.

The link elements consist of orthogonal springs in the direction parallel to its local principal axis (*S1*) to support compression as well as transverse (*S2*) to implement the shear force-deformation relationship of the ties (Figure 2d). The axial resistance of the discrete ties (*S1*) is modeled using an *Elastic Uniaxial Material* [23] with constitutive properties based upon the compressive stiffness of the insulation. Through experimental testing of extruded polystyrene (XPS) foam, a compressive stiffness value of 0.244 kN/cm per cm² of link element tributary area was deemed sufficient for use in the model. The shear-deformation behavior of the ties in the *S2* springs is modelled using an *ElasticMultiLinear Material* [23] as illustrated in Figure 2d where V_m and V_u represent the tie force at peak response and ultimate (i.e., fracture), respectively. D_y , D_m and D_u denote the tie deformations at the onset and end of the peak force plateau and at ultimate response, respectively. This material model can be constructed using data from experimental testing or tie manufacturer specifications. Link elements were discretely placed at the location of every shear tie along the length of the panel and connected the corresponding nodes of the two wythes with full translational and rotational connectivity. Additional link elements with zero resistance in *S2* were used at intermediate nodal locations along the length of the wythe beam-columns where no ties were present, as well as at the ends of the panel, to maintain overall stability of the slender wythes. The analysis is conducted using a hybrid incrementation scheme,

incorporating both displacement and force-controlled methods. Force-controlled incrementation is used until the peak panel response is reached (i.e., tangent slope of the panel resistance function reaches zero) and the remainder of the analysis is run displacement-controlled. This hybrid approach is used to improve efficiency of the model while ensuring that all regions of panel response are properly captured.

Figure 2 - Schematic of model built using computational methods

Using this computational component-based model, the effect of shear tie stiffness on the degree of composite action of a panel can be illustrated. As the stiffness of the ties approaches zero, the response of the panel approaches the non-composite response. On the contrary, when the tie stiffness approaches infinity, the response approaches composite behavior. An illustration of this concept is shown in Figure 3 where the non-composite and composite responses of a given panel are used as lower and upper bounds of strength (vice-versa for end slip), respectively. The prototype will panel considered for this study is a uniformly loaded simply-supported panel with

overall length and span length equal to 3.66 m (12 ft.) and 3.05 m (10 ft.), respectively; symmetrical wythe cross section dimensions of 81.3 cm (32 in.) wide and 7.62 cm (3 in.) and thick; and XPS insulation thickness equal to 5.08 cm (2 in.). Two #16 Gr. 420 (#5 Gr. 60) bars were placed at the middle of the thickness of each wythe, and concrete compressive strength was taken as 41.4 MPa (6000 psi). Shear spring elements were placed at 40.6 cm (16 in.) on center along the length of the panel, starting at 20.3 cm (8 in.) in from the edge. For simplification, shear tie constitutive properties for this initial investigation were modeled using *Elastic Uniaxial Material* [23] rather than the prescribed multilinear approach in order to directly illustrate the effect of tie stiffness on the panel response. The effect of tie ductility is illustrated later in the paper as part of a realistic parametric study which will require the implementation of the multilinear shear tie material model. Three levels of hypothetical tie stiffness were used to examine the effects of increasing composite action: low (17.5 kN/cm), medium (43.8 kN/cm) and high (175 kN/cm). The influence of panel cross-section geometry and material properties are examined in a realistic prototype wall panel later in this paper.

The effect of increasing tie shear stiffness on the pressure-deflection and end slip response is illustrated in Figure 3a and 3b, respectively. The successively darker shading between curves shows the panel response trending toward the composite behavior as tie stiffness increases. Similarly, end slip trends toward zero (i.e., composite response) as tie stiffness is increased. The plots show that a 10-times increase in tie stiffness (low to high) increases the yield strength by over 50% and decreases the panel deformation at yield by over 50%. The results therefore indicate that constitutive properties of the ties have a direct influence on the overall strength and stiffness of precast insulated wall panels.

Figure 3 - Computational results for a) applied pressure and b) panel end slip

4. PROPOSED MODEL

Though effective for demonstrating partially composite action, the computational model in Figure 2 is more suited to research applications rather than design due to its component-based assembly and computational effort. A new model for designing and analyzing partially-composite precast insulated wall panels subject to lateral loading was therefore pursued. The proposed model accounts for the contribution of each shear tie based on its constitutive properties and placement within the panel. The model can be used for any combination of panel cross-section geometry and arrangement of shear ties, including uneven and asymmetrical distributions, and is also compatible with non-uniform loading conditions. It is the responsibility of the user to procure shear tie constitutive relationships prior to performing the analysis. The model is currently limited to

224 statically determinate panels and further research by the authors will seek to determine efficient
225 ways to allow for the model to be compatible with indeterminate structures (e.g., multi-span wall
226 panels). A flowchart outlining the proposed model framework is shown in Figure 4. A description
227 of the procedure is followed by a validation against experimental data and comparison with the
228 previously discussed computational model.

Non-Composite & Composite Response

for $i=1 \dots \dots$ Number of displacement increments

for $k=1 \dots \dots$ Number of % composite iterations

Partially Composite Response

Figure 4 - Framework for proposed model

Before starting the procedure, two moment-curvature analyses are performed for the panel cross-section as an initializing step: one assuming a composite response and the other non-composite response as shown in Figure 5. The moment-curvature relationship for the composite section, $M_C(\phi)$, is calculated assuming a continuous linear strain profile through the entire cross section, including both concrete wythes and the sandwiched insulation layer. For these analyses, the Hognestad stress-strain model [25] was used for concrete in compression; a multitude of other concrete models can potentially be used as an alternative. To capture cracking, an elastic-brittle concrete tensile stress-strain relationship was used, with failure at the modulus of rupture. A multilinear material model was implemented for non-prestressed and prestressed steel based on a fit with material test data. For the non-composite section, moment-curvature relationships are calculated for both the interior, $M_{bot}(\phi)$, and exterior wythe, $M_{top}(\phi)$ and summed to obtain the non-composite moment-curvature relationship, $M_{NC}(\phi)$, as shown in Equation 1. These non-composite and composite responses are used as lower and upper bounds of panel strength, respectively, for assessing partially-composite behavior.

$$M_{NC}(\phi) = M_{top}(\phi) + M_{bot}(\phi) \quad \text{Equation 1}$$

The deflections of the composite and non-composite panel configurations are determined as a function of the applied load. The determination of the moment–deflection resistance of each panel at non-composite and composite behavior is detailed in Figure 5 and summarized as steps A through E in Figure 4. First, an incremental load, w_i is applied until the maximum moment is reached at step N (step A). The resulting moment distribution is then calculated using static equilibrium (step B). The curvature over the length of the panel is determined in step C for both the composite and non-composite panels by combining the moment distribution from step B with the moment-curvatures obtained in the aforementioned initializing step. For each incremental load,

253 the panel deflection can be computed in step D from the determined curvature distribution through
254 virtual work using Equation 2 or via other methods where $\phi_i(x)$ is the curvature over the length, L ,
255 at each load increment, w_i , and $m_\delta(x)$ is the virtual moment distribution over the length due to the
256 virtual unit force at midspan. For this model, both $\phi_i(x)$ and $m_\delta(x)$ are discretized with integration
257 points every 2.54 cm (1 in.) along the length of the panel. The user can adjust the density of
258 integration points as needed to efficiently calculate the deflection of a specific panel. Iteratively
259 incrementing the loads via steps A through D allows for the calculation of the full load-deflection
260 response of the panel with both composite and non-composite sections (step E).

Figure 5 - Response estimation for composite and non-composite panels

$$\Delta_i = \int_0^L \phi_i(x) m_\delta(x) dx \quad \text{Equation 2}$$

To compute the load-deflection response of a partially-composite panel, $M_{PC}(\Delta)$, the iterative procedure outlined in Figure 4 and illustrated in Figure 6 is used following the completion of step E in Figure 5. This part of the procedure increments panel displacement, calculates the resulting deformation and force developed in each shear tie for the increment, and uses the shear force couple along with the non-composite moment to determine the partially-composite moment

at that displacement increment. An initial level of panel composite action is assumed (step F) and checked for convergence upon completion of the remaining steps in the analysis (through step M). For the first iteration, the non-composite panel response is used. If a specified tolerance is met, the framework advances to the next displacement increment. If not, the level of composition action calculated from step M is used as the initial condition for the next iteration at the same displacement increment, and the calculations are repeated from step F. The program structure for this analysis is presented in Figure 4 and illustrated in Figure 6.

For each displacement increment, i , the panel rotation at each tie location, j , is determined from the respective curvature distribution (step G) using virtual work (Equation 3) or an alternative method (step H). This is repeated for each iteration of composite action, k . At each tie location, the shear deformation in the tie is computed from the rotation at the base of the tie (step H) and compatibility as illustrated in step I (Figure 6) using Equation 4 and Equation 5. The resulting shear force at each tie is determined from the tie's deformation and its mechanical properties (step J). It is important to note that the shear tie load-deformation backbone curves shown in step J of Figure 6 were developed as simplified approximations of test data to facilitate ease of convergence and stability for the model, as previously done by Naito et al [24]. This requires the user to procure test data for the given shear ties in a panel and develop the backbone curves to approximate the shear tie constitutive behavior. For discrete ties, mechanical properties can be obtained either from manufacturer data or experimental testing. For panels constructed with distributed ties, the force contribution per length can be lumped at discrete locations and the analysis can use this same procedure. Steps H through J are iterated until the contribution of all ties within the loaded half span have been accounted for.

For ties that lie outside the span, a linear slip profile is assumed outward from the last two ties within the span, and all remaining tie deformations are extrapolated accordingly. The total shear force contribution of all ties on the half panel is then summed using Equation 6 (step K) and multiplied by eccentricity, e , taken as the distance between wythe centroids as illustrated in Figure 6K, to determine the supplemental force couple caused by the shear ties. For panels with asymmetrical shear tie distributions and/or non-uniform loading, the tie shear force contributions are first summed from the location of the peak panel bending moment outward to either support and the side with the lowest total shear force is used as a conservative estimate to calculate the total supplemental moment resistance. The supplemental moment resistance is added to the non-composite moment (step L) at each displacement increment with the caveat that the total partial composite resistance cannot exceed the composite resistance, as shown in Equation 7. Finally, the percent composite action (PCA) of the panel is determined (step M) with Equation 8. The calculated PCA value is used to interpolate between the non-composite and composite moment and curvature responses (see Equation 9 and 10) before the onset of the second iteration. The partially-composite response curve obtained from this interpolation is then used to compute the updated relative tie deformations. The procedure, Steps F to M, are iterated until the PCA at each displacement step converges. This iterative procedure represents an efficient way to calculate the partially-composite response of insulated wall panels.

Partially Composite Response

(F) Guess % Composite at each load step. Assume non-composite initially. Use % composite from previous step for following steps.

(G) Determine curvature along span, $\phi_{i,k}(x)$, for load step i and guess of level of composite action, step k . Interpolate partially composite moment-curvature from non-composite and composite section analysis.

(H) Utilize virtual work or other method to determine panel rotation $\theta_{i,j,k}$ at each tie location along span for given distributed load.

(I) At each tie location, j , determine the relative tie deflection, $\delta_{i,j,k}$ taking the eccentricity, e , as the distance between the centroid of each wythe.

(J) Determine shear force in each tie, $f_{i,j,k}$ using deflections from step (I) and constitutive tie properties.

(K) Determine total additional shear contribution from ties, $F_{i,k}$ from summation of shear forces in each tie of panel for current increment of loading, i .

(L) Superimpose moment contribution from total tie force couple ($F_{i,k} \times e$) onto non-composite moment.

(M) Calculate % composite for each load step, i , and calculate the difference from previous step, $k-1$ (or initial if $k=1$). Compare difference with tolerance. If not within tolerance, repeat iteration k .

Figure 6 - Response estimation for partially composite panels

$$\theta_{i,j,k} = \int_0^L \phi_{i,k}(x) m_{\theta j}(x) dx \quad \text{Equation 3}$$

where $\phi_{i,k}$ is the curvature distribution of the non-composite section at load increment w_i , $m_{\theta j}(x)$ is

the virtual moment distribution over the length due to the applied moment at each tie location, j .

$$e = \frac{t_1}{2} + t_2 + \frac{t_3}{2} \quad \text{Equation 4}$$

$$\delta_{i,j,k} = e \tan \theta_{i,j,k} \quad \text{Equation 5}$$

where e is the eccentricity between the geometric centroids of each wythe and $\theta_{i,j,k}$ is the rotation at each tie location calculated using Equation 3.

$$F_{i,k} = \sum_{j=1}^N f_{i,j,k} \quad \text{Equation 6}$$

where N is the number of ties on the panel span and $f_{i,j,k}$ is the force in each shear tie, j , due to the deformation of each tie $\delta_{i,j,k}$ from the incrementally applied load w_i .

$$M_{PC\ i,k} = \min(M_{C\ i}, M_{NC\ i} + F_{i,k} e) \quad \text{Equation 7}$$

where $M_{C,i}$ is the moment capacity of the composite panel at deflection Δ_i , and $M_{NC,i}$ is the moment capacity of the non-composite panel at deflection Δ_i .

$$PCA_{i,k} = 100 \frac{M_{PC\ i,k} - M_{NC\ i}}{M_{C\ i} - M_{NC\ i}} \quad \text{Equation 8}$$

$$\phi_{PC\ i,k+1} = PCA_{i,k} (\phi_C - \phi_{NC}) + \phi_{NC} \quad \text{Equation 9}$$

$$M_{PC\ i,k+1} = PCA_{i,k} (M_C - M_{NC}) + M_{NC} \quad \text{Equation 10}$$

where ϕ_C , ϕ_{NC} , M_C and M_{NC} are the composite and non-composite curvature and moment response vectors, respectively.

5. COMPARISON WITH EXPERIMENTAL TEST DATA

The results of several recent experimental tests on precast insulated wall panels are used to validate the proposed model. Six wall panel configurations are considered as shown in Figure 7. Test data are obtained from Naito et al. [26], Trasborg [27] and Tomlinson and Fam [28]. Panels IWP 1 to 2 and IWP 4 to 6 measured 3.66 m (12 ft.) in total length and were subject to a uniformly distributed load within a simply-supported span of 3.05 m (10 ft.) as illustrated in Figure 7. Panel IWP 3 measured 2.7 m (8.86 ft.) in total length and was loaded in four-point bending as detailed in Figure 7. The shear ties used in each panel are shown in Table 1 and Figure 8. Shear tie constitutive properties were adopted from experimental tests [24,29] and tie manufacturer

329 specifications. The tie shear force-deformation relationships are shown in Figure 9. Note that since
330 tie type D is a distributed shear tie, a tributary width of 203.2 mm (8 in.) was used to transform it
331 to into an equivalent discrete tie for use in the model. It should be noted that for the following
332 comparisons, iterations of percent composite action were stopped once the solution converged to
333 within a 1% difference of the composite action guess at each load increment.

Figure 7 - Tested insulated wall panel configurations from [26–28]

336

Table 1 – Shear tie matrix

Identification	Source	Type	Material	Size
A	Thermomass®	Non-Composite (MC)	GFRP pin	MC 20/50
B	Thermomass®	Composite (CC)	GFRP pin	CC 150-50-50-50
C	Thermomass®	X Series	GFRP pin	X60-305
D	Altus Group®	C-Grid®	CFRP grid	C50 – 1.8 x 1.6
E	Tomlinson and Fam [29]	N/A	BFRP pin	B45T6

Figure 8 - Illustration of shear ties with dimensions

Figure 9 – Force-deformation relationships of shear ties used to construct panels

An illustration of the model is first used to show displacement-dependent shear tie and panel behavior. Figure 10 illustrates the series of calculations that constitute the partially composite member analysis. These include (a) the base rotation in each tie and the associated shear deformation, (b) the force in each row of shear ties and the percentage of composite action, and (c) panel pressure capacity as a function of the midspan displacement for panel IWP 4 (see Figure 7d). As expected, tie rotation and deformation increase with increasing panel deflections. Also, Figure 10a shows that the shear ties closest to midspan (row $j=5$ – see Figure 7d) exhibit lower deformation compared to those closest to the supports (row $j=1$), again as expected. The effect of shear tie deformation on the force in each row is illustrated in Figure 10b. A low ductility, high strength tie (type C) near the edge (row $j=1$) begins to fail at a much smaller panel deflection than the remaining higher ductility ties. Strength degradation of ties in row $j=1$ has a significant influence on the level of composite action (Figure 10b) as the peak row force and peak composite action occur at the same panel deflection. Figure 10c further shows that the panel's lateral pressure capacity is deformation-dependent. These observations are contrary to many design assumptions which only consider strength limit states of insulated wall panels [30].

To illustrate convergence of the proposed iterative approach, the error percentage between assumed and calculated percent composite values, PCA (see Equation 8), of panel IWP 4 (see Figure 7) was plotted as a function of iteration step, k , for the first 25.4 mm (1 in.) of midspan panel displacement in increments of 2.54 mm (0.1 in.) as shown in Figure 11. The results show that for this particular example, the percent composite level for each displacement increment converges to within the specified tolerance of $\pm 1\%$ composite action in ten steps or less, depending on the displacement increment. It is important to note that the number of iteration steps

363 required for convergence will vary based on the complexity of the model and the tolerance
 364 specified by the user.

Figure 10 - Displacement-dependent shear tie and panel behavior for IWP 4

Figure 11 – Error percentage between assumed and calculated percent composite values as a

function of iteration step, k , for the first 25.4 mm (1 in.) of panel midspan displacement for IWP

4

Figure 12 presents a comparison between the proposed model and experimental testing data for the six panels. Multiple tests were conducted on most panels, resulting in a distribution of measured response. For those panels a shaded region that bounds the experimental response is plotted. The exception is Panel IWP 3 (Figure 12c), for which only one set of data was available. Panel response calculated using the previously discussed computational modeling approach is also included in subplots a-b and d-f for additional comparison. Panel IWP 3 is not examined using computational modeling due to the use of concrete rails which encroach on the insulation layer and complicate the cross-sectional geometry. Both applied pressure and panel end slip are shown as a function of midspan deformation. The results indicate that the proposed model and computational approach show good conservative agreement with the experimental results. The

381 pressure capacity of both models trends toward the lower bound of the experimental results, and
382 end slip response from the models is larger in every case considered here. The results also compare
383 especially well for panels constructed with discrete shear ties (Figure 12a-e). The accuracy of the
384 proposed model with both experimental results and computational modeling validates the proposed
385 iteration-based model for use in predicting the response of insulated wall panels.

Figure 12 - Comparison of proposed model and the computational model with experimental data for panels IWP 1 to 6 (subplots a to f, respectively)

6. PARAMETRIC STUDIES

The proposed model can be used to predict the response of wall panels constructed with a broad range of concrete wythe cross-section geometries, insulation layer thicknesses and shear tie arrangements. To illustrate the practicality of using this method as a design and analysis tool and to determine the effect of tie ductility on overall panel response, two parametric studies involving four hypothetical insulated wall panel configurations (Figure 13a through d) are conducted.

6.1. Parametric Study A – Applicability of the model as a design tool

The first study examines the applicability of the model as a design tool, incorporating the effects of varying shear tie arrangements and constitutive properties on global panel response. All panel configurations have the same wythe cross section geometry, overall length and simply-supported span as IWP 4 (see Figure 7). The constitutive properties of the ties are depicted in Figure 14. Four hypothetical tie types are considered: C, F, G, and H. Type C is a larger tie with higher strength and ductility compared to the other 3, and it is used only in IWP 7 (as shown with the larger tie markings in Figure 13a). Ties G and H have the same initial and unloading stiffness as F but have two and three times the ductility of F, respectively. The arrangement of ties with their corresponding constitutive properties are designed such that the panel IWP 7 (with tie type C), IWP 8 (with tie type G), and IWP 9 (with tie type H) have comparative moment capacity. The panels are examined using the proposed model. The pressure versus deformation response for all panel cases is shown in Figure 15 along with the corresponding percentage of composite action at the peak capacity. The results indicate that similar flexural strengths can be achieved with several tie configurations, such as a few closely spaced strong connectors (IWP 7) or a larger number of low strength connectors (IWP 8 and 9). Due to the high stiffness and strength of the type C ties, the IWP7 panel reaches its peak strength at a much lower deformation than IWP 8 or 9, which both

use a denser distribution of connectors with lower strength and stiffness. The ties in Panels IWP 8 and 9 have the same yield strength, but the ties in IWP 9 have increased ductility and are placed closer together near the end of the panel. Figure 15 shows IWP 8 and 9 with nearly the same peak pressure capacity, but IWP 9 reaches a higher global panel deformation due to the increased ductility. This also illustrates the effect of tie configuration, where panels with varying layouts and different ties can achieve similar peak panel strength yet with significantly different panel deformations. As illustrated in this parametric study, the tie properties and arrangement can be used to customize the global flexibility, strength and ductility of an insulated panel in accordance with a variety of design objectives.

Figure 13 – Panel configurations IWP 7-10 (a-d) used in the parametric study (all dimensions in mm)

Figure 14 – Shear force-deformation relationships of shear ties used in the two parametric studies

Figure 15 – Estimated pressure-deformation response of IWP 7 through IWP 9

6.2. Parametric Study B – Effect of tie ductility on global panel response

In the second parametric study, a progressive shear tie failure mechanism is used to illustrate the effects of tie ductility on the global composite percentage and overall response of a panel. Under uniform load with simple boundary conditions, the largest inter-layer slip in an idealized panel will occur at each end of the panel and approach zero at midspan. Consequently, ties placed at the end of the panel will contribute more shear resistance and reach their fracture deformation before ties located closer to midspan. Similarly, at later stages of the response when displacements are larger, ties located closer to midspan will engage and provide increasing shear contributions. To illustrate this concept, panel configuration IWP 10 (Figure 13) is analyzed with three different shear ties F, G, and H (Figure 14). Figure 16a, c, and e present estimated pressure – deflection responses of the IWP 10 with ties F, G and H, respectively. Three critical milestones, first yield of the reinforcement, peak capacity and ultimate response (defined here as a global panel deformation of 15.5 cm (6.1 in.)), are chosen to illustrate the progression of shear tie failures. Figure 16b, d, and f illustrate the shear force in each tie along the length of the panel half-span (due to symmetry) at the three milestones for tie type F, G, and H, respectively. When the steel reinforcement in the tension wythe yields, all tie types (F-H) provide the same resistance with the four connectors nearest the end of the panel in the plastic range and the others in the elastic range. At this level of response, the IWP 10 tie arrangement and mechanical properties produces a panel with 60% of the composite strength (i.e. milestone 1 in Figure 16a, c, and e). At the peak strength of each panel (i.e. milestone 2), the majority of ties are at or near their plastic capacity. As the ductility of the connectors increases, the ties can maintain the internal shear couple at larger deformations and thereby allow the panels to utilize their overall flexural strength longer. For example, ties G and H both enable 63% composite action at the peak capacity, versus 59% for tie

451 F. The actual strength and deformation level at peak for tie H, however, is higher than that for G
452 because both the composite and non-composite panel strengths increase at larger deformations.
453 Tie type H is able to remain stable under larger deformations than G because of the greater ductility
454 in the tie. Tie type F achieves lower composite action at peak strength because the plastic
455 deformation capacity of the tie is not large enough to allow all ties to become plastic before the
456 end tie loses strength and the panel “unzips.” As the deformation increases past the peak strength,
457 progressive shear tie failures occur as illustrated in the decrease in tie strength and drop in flexural
458 capacity. For tie F, five rows of ties fail when the panel reaches the deformation of 15.5 cm (6.1
459 in.). If the ductility of the tie is doubled (tie G), three rows of ties fail - if the ductility is tripled (tie
460 H), tie integrity is maintained. Therefore, the flexural strength and percentage of composite action
461 increases at the ultimate deformation level with increases in tie ductility.

Figure 16 – Effect of progressive tie failure on performance of panel IWP 10

7. CONCLUSIONS

A simplified model was developed to assess the flexural behavior of partially-composite precast concrete insulated wall panels. The approach accounts for concrete wythe flexural behavior

as well as the constitutive properties and geometric arrangement of the ties used to transfer shear forces between the wythes. To determine the percentage of composite action between the wythes, the proposed model uses an iterative method to calculate relative tie deformations and resulting shear forces. These forces create force couples which are superimposed onto the non-composite panel response at each iteration step as an additional moment. An OpenSees computational model was also used to illustrate the accuracy of the proposed model for estimating both interface slip and global flexural response. Using these two modeling approaches, the individual performance of each shear tie can also be closely monitored and compared. The proposed model accurately estimates the pressure-deformation and end slip response of six experimental tests representative of insulated wall panel construction. Also, the proposed model and the OpenSees computation model show good agreement, indicating that the approximations used in the iteration-based model are acceptable.

Two parametric studies were performed to illustrate the effectiveness of the proposed model as a simplified design and analysis tool and to show the effects on tie ductility on overall panel response. Specifically, the approach was used to illustrate how tie properties and placement can directly impact the global pressure-displacement response. The results of the study show that constitutive properties and spatial arrangement of the tie connectors can be configured to vary the panel's flexural ductility and stiffness while achieving the same strength in response to lateral pressure. Tie ductility is also shown to directly impact the deformation capacity of the panel. Low ductility ties result in early strength degradation of the tie, considerably reducing the level of composite action as well as the panel overall deformation capacity and strength.

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