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Analytical approach to flexural response of partially composite insulated concrete sandwich walls used for cladding

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ABSTRACT

An analytical model was developed to predict the flexural response, and determine the impact of various shear connector parameters on the behavior, of architectural partially-composite precast concrete insulated panels. The sandwich system incorporates two concrete wythes with expanded polystyrene foam and angled shear connectors in between. The model accounts for material nonlinearity and cracking and incorporates idealized load–slip relationships to determine the amount of shear force transferred between wythes, which controls the degree of composite action. The model agrees well with experimental results of a large database from literature, having an average difference in peak loads of −1.0% with a standard deviation of 13.4%. A comprehensive parametric study investigated the effect of connector spacing, diameter, insertion angle, shear modulus of insulation, and connector material, namely steel and fiber-reinforced polymer, on the degree of composite action and hence on the load–deflection response. These parameters were also shown to critically affect failure mode, where a transition from a connector's material or pull-out failure to a flexural failure of the reinforced concrete wythe occurs. It was also shown that insulation bond alone contributes substantially, about 47% composite action by strength. Finally, for design purposes, the model can estimate the size and spacing of shear connectors to give a desired degree of composite action.

1. Introduction

Insulated concrete wall panels (also known as sandwich panels) are commonly used as exterior members in building construction. Insulated panels are typically composed of two relatively thin (50–150 mm) precast concrete wythes that surround a layer of rigid foam insulation. Shear connectors provide structural continuity between the wythes. Relative to other wall assemblies, insulated panels are advantageous as they combine structural, thermal, and architectural properties into a single unit [1].

Panels are classified based on their degree of composite action (amount of longitudinal shear force transferred between wythes). Fully composite (FC) walls have complete shear transfer (wythes act as one unit) while non-composite (NC) walls have zero shear transfer (wythes act independently). Panels with shear transfer between these extremes are partially composite. Relative to non-composite panels, fully composite panels are advantageous as they achieve design loads with less material, but as they are more subject to bowing, some designers prefer non-composite walls [2]. The flexural behavior of fully and non-composite walls can be predicted using approaches available in reinforced concrete design codes. Predicting partially composite wall behavior is more cumbersome as the shear transfer between the two wythes through the insulation and the connectors needs to be accounted for accurately [1,3]. The three structural designations are illustrated in Fig. 1.

Steel or Fiber Reinforced Polymers (FRP) shear connectors are commonly used. FRP is beneficial as its low thermal conductivity relative to steel allows for walls with higher thermal efficiency. Generally, low degrees of composite action are provided from pin-type connectors [4,5] while higher degrees are provided from continuous truss-type connectors [6,7] FRP grid [8], or by using solid concrete regions [5]. Various common connector types are shown in Fig. 2.

The insulation (typically Expanded (EPS) or Extruded (XPS) polystyrene) contributes a non-negligible amount to composite action with higher composite action coming from EPS [9]. However, the long-term performance of the insulation contribution is a matter of debate as the insulation-concrete bond may fail after repeated freeze–thaw cycles [1].

Allen [10] presented comprehensive approaches to designing sandwich elements with focus on members with linear-elastic skins (e.g. FRP) commonly used in aerospace applications. Bush

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Nomenclature

b	panel cross-section width
d_i	depth of reinforcement at ith layer (1 = topmost)
f_c	stress in concrete at strain ε_c
f_c'	concrete compression strength
f_{cr}	concrete rupture stress
f_{frp}	stress in FRP reinforcement or shear connector at strain ε_{frp}
f_s	stress in steel reinforcement at strain ε_s
f_u	ultimate strength of steel reinforcement
$f_{u,in}$	insulation tensile strength
$f_{u,sc}$	ultimate strength of shear connector
f_y	yield stress of steel reinforcement or connectors
h_f	depth of the façade wythe
h_{in}	depth of the insulation layer
h_s	depth of the structural wythe
l_e	shear connector embedment length
n	number of transverse rows of shear connectors
s	shear connector longitudinal spacing
A_{in}	area of insulation subject to shear
A_s	longitudinal reinforcement cross-sectional area
A_{sc}	shear connector cross-sectional area
C	distance from edge of concrete to center of connector used for pullout calculations
E_c	concrete modulus of elasticity
E_{frp}	FRP reinforcement or shear connector modulus of elasticity
E_m	post-yield modulus of steel reinforcement and connectors accounting for strain hardening
E_s	steel reinforcement or shear connector modulus of elasticity
E_{sc}	shear connector modulus of elasticity
F	applied flexural load
F_{Ci}	concrete force in ith layer (1 = topmost)
F_{ri}	reinforcement force in ith layer (1 = topmost)
$F_{u,FC}$	fully composite panel ultimate load
$F_{u,NC}$	non-composite panel ultimate load
$F_{u,PC}$	partially composite panel ultimate load
G_{in}	insulation shear stiffness
I_{sc}	shear connector moment of inertia
L	panel span
L_{sc}	length of shear connector passing through insulation
M	applied moment on panel
V_{dw}	contribution to longitudinal shear resistance from dowel action at slip, δ_s
V_{in}	contribution to longitudinal shear resistance from the insulation at slip, δ_s
V_L	longitudinal shear force transferred between wythes at slip, δ_s
$V_{L,FC}$	V_L in a fully composite panel, causes $\varepsilon_{sc} = 0$
V_{sc}	contribution to longitudinal shear resistance from shear connectors at slip, δ_s
V_{tr}	contribution to longitudinal shear resistance from truss action at slip, δ_s
$V_{tr,max}$	maximum contribution to longitudinal shear resistance before connector failure
X	shear connector transfer length
α	factor for bond calculations
α_1	bond characteristics factor for tension stiffening
α_2	nature of loading factor for tension stiffening
γ_c	concrete density
δ	panel deflection
δ_{cl}	midspan panel deflection
δ_s	differential wythe slip between wythes
ε_{ac}	strain in truss connectors
ε_c	concrete strain
ε_c'	concrete strain at peak compressive stress, f_c'
ε_{cr}	concrete rupture strain
ε_{ci}	strain in concrete in ith layer (1 = topmost)
ε_{cu}	peak concrete strain
ε_{ri}	strain in reinforcement in ith layer (1 = topmost)
ε_{sc}	differential strain between concrete wythes
$\varepsilon_{sc,NC}$	differential strain between concrete wythes for a non-composite section
ε_{su}	ultimate strain in steel reinforcement
$\varepsilon_{u,frp}$	ultimate strain in FRP reinforcement
ε_y	yield strain of steel reinforcement or shear connectors
θ	shear connector insertion angle (0° = normal to panel face)
κ_u	degree of composite action by strength methods
ρ_v	shear connector reinforcement ratio (total area of connectors/bL)
φ	shear connector diameter
ψ	curvature of section under bending
ψ_f	curvature in facade wythe
ψ_s	curvature in structural wythe

and Wu modified Allen's work to incorporate the added shear stiffness from truss connectors [7] while Salmon and Einea used a similar approach to evaluate thermal bowing in panels [6]. Both techniques work well for truss connectors in linear-elastic conditions only. Bai and Davidson developed a discrete model to define pre-cracking behavior of sandwich panels [3]. Their model isolates flexure and shear deformations with reasonable accuracy until cracking. Pantelides et al. used a truss analogy to model insulated panels with GFRP shell connectors [11]. This model created a bilinear response based on yielding of the longitudinal reinforcement. Naito et al. used idealized load–slip relationships from double shear push through tests to predict shear deformation and partial composite behavior [12].

This paper presents a numerical model which predicts the complete behavior of a partially composite insulated concrete panel in flexure. The model accounts for concrete cracking and nonlinearity, both steel and FRP shear connectors of various diameters, spacing and insertion angle, various failure modes (Fig. 3) including connectors yielding, rupture or pullout, and the load–slip constitutive relationships of the insulation and connectors. The model is verified against results from five experimental programs. A comprehensive parametric study is then conducted focusing on the impact of varying shear connector properties on the flexural load–deflection relationships, failure modes and the degree of composite action.

2. Model development

The flexural response of partially composite insulated concrete wall panels was modeled using software developed with MATLAB. The analytical procedure and development of the model are described in this section. The model involves a number of key components, namely: material constitutive relationships, shear connection mechanisms, moment–curvature responses of the wythes, and developing load–deflection and load–slip relationships for the panel system in flexure, where slip is the relative longitudinal displacement between the two wythes and reflects the