Development of alternative strength reduction factors for interwythe shear failure of precast concrete insulated wall panels

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- This study investigates variability and uncertainty specific to precast concrete insulated wall panels to develop alternative strength reduction factors that account for a flexural failure mode due to interwythe shear connector failure.
- Using data from previous analytical and experimental studies, the investigators explored three sources of uncertainty for precast concrete insulated wall panels: the variability of wythe connector data against average values, the accuracy of simplified design backbone curves, and the effectiveness of beam-spring models to capture the behavior of panels.
- The results provide proposed interwythe-controlled strength reduction factors for discrete and continuous connector systems for a range of safety indexes, which can be used by technical committees tasked with developing design standards or recommendations.

The rising popularity of precast concrete insulated wall panels as exterior cladding or load-bearing structural components has fostered the development of numerous design solutions, with particular attention paid to wythe connector systems. The customizable use of wythe connectors in various layouts, in conjunction with several options for insulation and concrete wythe configurations, has motivated investigators to develop and implement structural analysis methodologies to capture the mechanics of these structures and the limiting failure modes for a given design case. Many experimental and computational studies have confirmed the effectiveness of such modeling approaches, particularly when the panel is loaded in flexure in response to lateral force demands, such as wind or suction, or combined flexure and axial force if gravity forces are imparted on a load-bearing panel.[1–4] Although these results have increased confidence in the use of computational modeling for these structures, knowledge gaps remain with respect to the reliability of these design solutions given the inherent variability in the constitutive properties of panels. Building codes and design guidelines[5,6] specify strength reduction factors for monolithic structural concrete members (that is, those without insulation and wythe connectors), such as solid wall panels, but additional research is needed to develop strength reduction factors that sufficiently reflect the variability in the components of precast concrete insulated wall panel systems. For example, a thorough reliability-based analysis would need to account for potentially significant variations in the peak shear resistance, ductility, and fabrication techniques for wythe connectors that serve to transfer shear forces between the opposing concrete wythes.[7] In addition to wythe connector material variability, insulation properties will differ depending on the type of foam selected and the extent of the wythe-insulation bond that forms during panel fabrication.

This paper highlights a framework used to compute alternative interwythe shear-controlled strength reduction factors for precast concrete insulated wall panels in which the strength reduction factors are divided into two main categories as determined by the type of wythe connector system, either discrete or continuous. This framework was designed to reflect major sources of variability in the panels with a particular emphasis on their influence on the peak flexural resistance of an insulated wall panel. Three categories of uncertainty were considered in this study: variability of double shear test data compared with a representative average curve, accuracy of simplified backbone curves, and comparisons between beam-spring modeling results and experimental test data on larger-scale precast concrete insulated wall panel specimens. The outcomes of calculating variability with respect to experimental test data and modeling accuracy were then funneled into calculations for strength reduction factors that can be used when interwythe shear-controlled failure is anticipated for either connector classification.

Design safety for structural concrete members

Before discussing the reliability-focused methodology used in our study, it is essential to situate the probabilistic approach within its historical and theoretical context. The seminal work by MacGregor[8] marked a major milestone in the introduction of limit state design for reinforced concrete structures. MacGregor's approach offers a rigorous formulation of load and resistance factors based on the probabilistic distribution of sources of uncertainty in materials, dimensions, modeling assumptions, and loading conditions. It also formalizes the relationship among the safety index β, the resistance-to-load ratio R/U, and the corresponding coefficients of variation used to generate load and resistance factors. It is important to note that the safety index β corresponds to a target probability of failure for the structural member, which is typically set by code provisions to ensure an acceptable level of safety.[9]

Design methods have evolved significantly over time, gradually shifting from the allowable stress design approach to limit state design. The latter has become the standard in many countries and continues to be widely adopted. Allowable stress design assumes that a structure is safe as long as the applied loads remain below a threshold value defined by global safety factors. However, this method does not account for the variability of loads or material properties, nor does it consider different plausible failure modes; therefore, the conservative (or unconservative) nature of design solutions based on allowable stress design will fluctuate. Limit state design addresses these limitations by distinguishing between different types of failures: structural collapse, serviceability loss, and material damage. It uses separate partial factors for loads and resistance, which provides more-accurate control over the level of reliability based on actual uncertainties. Limit state design also accounts for material behavior, making it especially relevant in the design of mixed-material structures, such as those that combine steel, concrete, and timber. Across these configurations, limit state design ensures a consistent level of safety by incorporating all relevant variabilities.

A key contribution of MacGregor's approach[8] is that it highlights the various sources of uncertainty that affect the behavior of reinforced concrete structures. Generally, uncertainties can be considered either aleatory or epistemic. Aleatory uncertainty is related to variability that cannot be reduced but must be accounted for probabilistically. For example, fluctuations in material properties are classified as aleatory uncertainty. In contrast, epistemic uncertainty arises from incomplete knowledge, such as uncertainties in model parameters, assumptions, or measurement errors. Epistemic uncertainty can potentially be reduced through better experimentation or better modeling.[10,11] Both aleatory and epistemic uncertainties can be present at all stages of design, manufacturing, and construction, and they influence both resistance and applied loads on the structure. It is therefore essential to identify types of uncertainties as precisely as possible.

Material property uncertainty is especially relevant for concrete structures, where compressive strength can vary significantly depending on the mixture proportions, curing conditions, and placement methods. Uncertainties related to inaccuracies in modeling or fabrication are also relevant for concrete systems. There may be discrepancies between theoretical dimensions and those achieved in the field. Even small deviations in placement or alignment can have a significant impact on structural performance. Analytical models can also introduce uncertainty and, despite the growing knowledge base for models of precast concrete insulated wall panels, simplifications such as the use of backbone curves or other idealized assumptions do not always fully represent the expected behavior of the component. Finally, there is uncertainty in the loading conditions as the exact loads that a structure will experience over its entire service life are ultimately unknown.

The resistance factors (that is, strength reduction factors) developed within the scope of this study represent an initial phase of research needed to develop a comprehensive limit state design methodology for precast concrete insulated wall panels. The authors plan to pursue this larger effort after first examining the effect of varying the constitutive properties of insulated wall panels as part of a resistance-based reliability assessment for these components.

Current strength reduction factors for structural concrete

According to section R21.1.1 of the American Concrete Institute's *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19)*,[5] conventional strength reduction factors for structural concrete members were developed to account for the possibility of understrength members due to variations in material strengths and dimensions, compensate for inaccuracies in design equations, reflect available ductility and required reliability, and reflect the importance of the structure. While provisions in ACI 318-19 section 21.2 reflect these four objectives for solid (not insulated) prestressed or nonprestressed concrete components, additional research is needed to develop strength reduction factors that directly reflect these purposes for precast concrete insulated wall panels.

Following are specific concerns related to the four objectives set forth in ACI 318 section R21.1.1:

- Compared with solid components, precast concrete insulated wall panels likely exhibit greater variability with regard to material strengths and fabrication techniques because numerous types and configurations of wythe connectors[7] and insulation foam are commercially available.
- Specific design methodologies and limit states for precast concrete insulated wall panels are not universally accepted, in part because of the increased complexity and immense variability of design configurations and performance metrics.[1,2,12,13]
- The overall performance of the wall panel may be heavily influenced by the strength and/or ductility of the wythe connectors,[1,12] in addition to the ductility of the wythe reinforcement.
- The structural importance of insulated wall panel components can vary significantly, ranging from non-load-bearing cladding to blast-resistant facades.[14]

Current acceptance criteria for fiber-reinforced grid connectors[15] specify prescriptive strength reduction factors for the design shear flow strength of the connectors, but these criteria do not consider the overall behavior of the wall panel (that is, the coupled behavior of the connectors and the prestressed and/or reinforced concrete wythes). For precast concrete insulated wall panel applications, we must consider not only the concrete component metrics but also those of the wythe connectors and the role that the insulation plays in the overall mechanical behavior of the walls.

Furthermore, existing strength reduction factors do not account for the interwythe shear failure that occurs when the wythe connectors fail as panels are subjected to flexural loading. Instead, existing strength reduction factors only cover three failure classifications that pertain strictly to structural concrete elements: tension controlled, compression controlled, and a transition region. The methodology described herein provides alternative strength reduction factors for precast concrete insulated wall panels, thereby addressing the aforementioned shortcomings and facilitating safe, reliable, and efficient design of these components.

Standard failure classifications

The *PCI Design Handbook: Precast and Prestressed Concrete*[6] defines tension-controlled failure as failure that occurs in a section where the reinforcing steel exhibits a maximum strain of 0.005 or more. According to ACI 318, the same classification is valid for net tensile strains at least 0.003 beyond the nominal yield strain of the bars.[5] Precast concrete insulated wall panels are often designed to meet tension-controlled criteria as a simplifying assumption strictly based on the performance of the concrete wythe sections; however, as Trasborg[16] has noted, this simplifying assumption does not account for the ductility that is achieved by composite action through the entire cross section. Naito et al.[17] found through experimental testing of precast concrete insulated wall panels that the applicability of tension-controlled strength reduction factors can also be limited by concrete crushing in the compression zone.

Compression-controlled sections are defined in the *PCI Design Handbook* as exhibiting a maximum strain of 0.002 or less (less than or equal to the nominal yield strain in ACI 318-19) in the extreme tension steel fiber. In general, precast concrete insulated wall panels are generally not designed with compression-controlled failure modes because of the nature and orientation of the loading schemes to which they are subjected in common applications.[18]

Last, the *PCI Design Handbook* defines sections limited by the transition region as those that exhibit strain in the extreme tension steel fiber of 0.002 to 0.005 (that is, between compression controlled and tension controlled), whereas the range specified in ACI 318-19 is between the nominal yield strain of the bars and a strain 0.003 greater than that value.

While the standard classifications are generally very effective toward quantifying the extent of member ductility as a result of the concrete and reinforcement performance, the descriptors do not capture a dominant failure mechanism that results from the progressive failure of wythe connectors, a phenomenon that can happen before the material-based limit states of the wythe sections are achieved.

Proposed interwythe shear limit state

The concept of composite action in precast concrete insulated wall panels is derived from the degree of shear force transferred and strain compatibility across the opposing concrete wythes, bounded by the theoretical noncomposite and fully composite behaviors. Theoretical noncomposite behavior implies that each concrete wythe has its own strain profile, which is completely independent of the other wythe's profile. Theoretical fully composite behavior is characterized by a single, continuous strain profile through the entire depth of the wall panel cross section. Noncomposite and fully composite responses are theoretical response extrema, and most practical designs for precast concrete insulated wall panels fall into the partially composite behavior classification. In this scenario, the strain profile cannot be continuous through the entire cross-section depth, but the strain profiles of the two individual wythes are not completely independent of each other and considerable interwythe shear slip can develop between them. Partially composite precast concrete insulated wall panels subjected to flexure can fail as a result of an interwythe shear failure (that is, the *unzipping* of wythe connectors[1] and/or the loss of the wythe-insulation bond) or a global flexural failure, which may be driven by the failure of steel reinforcement or concrete crushing.

In addition to the three aforementioned standard failure classifications for concrete components, our study proposes a fourth classification: interwythe shear-controlled failure. The strength reduction factors in this category account for the variability in mechanical properties of wythe connectors that can be associated with potential strength shortcomings in the context of premature failure modes (that is, the shear failure of the wythe connectors and the loss of the concrete wythe-insulation bond). In addition, the outcomes of this work can help streamline otherwise complex comprehensive analyses of precast concrete insulated wall panels in favor of an applicable strength reduction factor based on the anticipated performance of the panel given its wythe connector type and the design methodology used.

Quantifying sources of uncertainty

Before alternative strength reduction factors can be calculated, the underlying sources of uncertainty that they reflect must first be assessed and quantified. The parameter of interest for this portion of the methodology is the global uncertainty index Ω. For the purposes of this study, this index is calculated using the following three types of uncertainties:

- type a: variation in individual shear–displacement curves for a given type of wythe connector with respect to the mean curve from the sample set
- type b: errors caused by simplifying the raw shear compared with displacement history to a backbone curve for ease of implementation into structural analysis software
- type c: discrepancies between experimental test results on larger-scale insulated wall panel specimens and corresponding analytical estimates for these components generated using beam-spring modeling

The first source of uncertainty (type a) can be classified as aleatory because inconsistencies in the data are dependent on variations (both intentional and unintentional) of double shear test specimens. The second two sources (type b and type c) are epistemic because they depend heavily on modeling assumptions or approximations made during structural analysis.

Each of the three sources will produce two statistical measures needed to calculate Ω. The first metric is the coefficient of variation, represented by Δ_1^i, where the superscript i will range from a to c as indicated by the corresponding uncertainty-type descriptions noted in the previous paragraph. The inclusion of this value is necessary because it captures the inherent variability of the sample set, calculated as the standard deviation σ divided by its mean μ (Eq. [1]). The second metric is the coefficient of variation divided by the square root of the population size n (Eq. [2]). This value is represented by Δ_2^i, where the superscript labeling convention is the same as for the coefficient of variation. This metric is used as a secondary indicator of variability that places a larger emphasis on the population size to facilitate and capitalize on more-accurate depictions of uncertainty when a larger number of data points can be provided. The global uncertainty index Ω is then calculated as the square root of the sum of the squares of all six Δ metrics across the three sources, (Eq. [3]).

$$\Delta_1^i = \frac{\sigma}{\mu} \tag{1}$$

$$\Delta_2^i = \frac{\Delta_1^i}{\sqrt{n}} \tag{2}$$

$$\Omega = \sqrt{\left(\Delta_1^a\right)^2 + \left(\Delta_2^a\right)^2 + \left(\Delta_1^b\right)^2 + \left(\Delta_2^b\right)^2 + \left(\Delta_1^c\right)^2 + \left(\Delta_2^c\right)^2} \tag{3}$$

where

Δ_1^a = coefficient of variation considering uncertainty type a

a = descriptor for uncertainty type a, variation in individual shear–displacement curves for a given type of wythe connector with respect to the mean curve from the sample set

Δ_2^a = coefficient of variation divided by the square root of the population size considering uncertainty type a

Δ_1^b = coefficient of variation considering uncertainty type b

b = descriptor for uncertainty type b, errors caused by simplifying the raw shear compared with displacement history to a backbone curve for software-based structural analysis

Δ_2^b = coefficient of variation divided by the square root of the population size considering uncertainty type b

Δ_1^c = coefficient of variation considering uncertainty type c

c = descriptor for uncertainty type c, discrepancies between experimental test results on larger-scale insulated wall panel specimens and corresponding analytical estimates for these components generated using beam-spring modeling

Δ_2^c = coefficient of variation divided by the square root of the population size considering uncertainty type c

As will be discussed in the following subsections, quantification of each of these metrics will require a comprehensive assessment of data compiled from numerous research efforts on double shear wythe connector tests, experimental testing of insulated wall panels, and beam-spring analyses.

Variability of wythe connector test results (uncertainty type a)

Compilation of double shear test data This effort builds on a comprehensive dataset procured from multiple experimental campaigns, each aiming to characterize the mechanical behavior of precast concrete insulated wall panel wythe connectors loaded in shear. Naito et al.[7] evaluated the performance of numerous commercially available wythe connectors subjected to double shear loading. Each connector configuration was repeated with 3 to 5 specimens, resulting in published data for a total of 14 test specimens: 9 for discrete connectors (types A, B, and C as labeled in the source paper) and 5 for continuous connectors (types D-1 and D-2). However, only 3 of the continuous connector specimens from that study were usable in our study because one had an error with displacement data acquisition and another was initially damaged. The discrete connectors in the study by Naito and colleagues were either glass-fiber-reinforced polymer (GFRP) grids or pins with extruded polystyrene (XPS) insulation, whereas the continuous connectors were all carbon-fiber-reinforced polymer (CFRP) grids, with specimen configurations D-1 and D-2 using XPS and expanded polystyrene (EPS) insulation, respectively.

Although this initial dataset provided a reliable starting point, the number of observations was deemed insufficient for statistical modeling and probabilistic calibration. Therefore, data from a group of 22 double shear tests conducted by Maguire and Al-Rubaye[13] for insulated tilt-up concrete panels were also used for the discrete connector category, resulting in a total population size of 31. Among the tested configurations, four connector types (labeled as types B, C, D, and E by Maguire and Al-Rubaye) were identified as compatible with the classification framework used in our study (that is, there were adequate details about each test and the full shear-displacement curve). All four connector types from that study were manufactured with reinforced polymers and XPS insulation.

The tests conducted by Naito et al.[7] contained bonded insulation, while most of the tests by Maguire and Al-Rubaye[13] were deliberately debonded using two sheets of insulation, creating a shear break in between. Although these test programs differed in that way, we decided to group them together in this study to capture the effect of insulation bond and residual friction in the context of the variability of wythe connector test results.

Connector type A from Naito et al.[7] and type C from Maguire and Al-Rubaye[13] were the same product, and Naito type B and Maguire type E were also the same. However, we treated these overlapping specimen types as unique specimens because their fabrication and testing programs were parts of different projects with varying equipment and personnel.

An additional set of 40 valid continuous wythe connector test results was procured from Bunn,[19] for a total population size of 43 for the continuous connector category. All of the specimens from Bunn's study were fabricated using a CFRP grid system; 15 of them used XPS insulation, and the other 25 used EPS insulation. The relatively large dataset from Bunn contained variations of insulation thickness (2 and 4 in. [50 and 100 mm]), specimen height and width, and grid spacing (and therefore the number of CFRP grids installed).

Data processing A crucial step before calculating the global uncertainty index in this study was normalization of the shear–displacement curves. Although the connectors were grouped into two broad categories (discrete and continuous), the samples exhibited significant design variations, such as width, insulation type, and thickness, which directly affected their shear resistance. This heterogeneity complicates comparative analyses unless prior data treatment is applied. When the purpose of the exercise is largely to assess the variability of repeated specimens, specimens with different types of connectors or even varying arrangements of the same connector type should not be penalized for deliberate design differences when compared with the average curve for the entire group. Ideally, each slight variation in a double shear test configuration would have its own average curve; however, such a restrictive grouping strategy would lead to a small population size, which could compromise the effectiveness of the statistical analysis methods used in our study. The normalization approach, therefore, represents a compromise solution that preserves the relatively larger population size of the heterogeneous test specimen group while focusing on the relative variability of the shear–displacement curves instead of raw magnitudes.

To perform the normalization, each point of an experimental data curve was divided by its own peak force value. That calculation transformed each curve into a unitless profile with a maximum value of 1, allowing for shape comparisons independent of actual force magnitudes. **Figure 1** compares the average curves generated before and after the shear force was normalized. The effect of the normalization strategy is clearly shown; the spread of data is focused on the relative variability among specimens as opposed to physical differences in the specimen design configurations. To compute the average curve of a population set, each shear force–displacement curve is interpolated on a set of predefined shear displacement values to facilitate averaging of the force values at each displacement. This method allows the construction of a representative normalized average curve, which in turn enables evaluation of the dispersion of the individual tests. For the discrete connectors, an average curve was calculated for each connector type from Naito et al.[7] (types A, B, and C) and Maguire and Al-Rubaye[13] (types B, C, D, and E). For the continuous connectors, two average curves were generat-

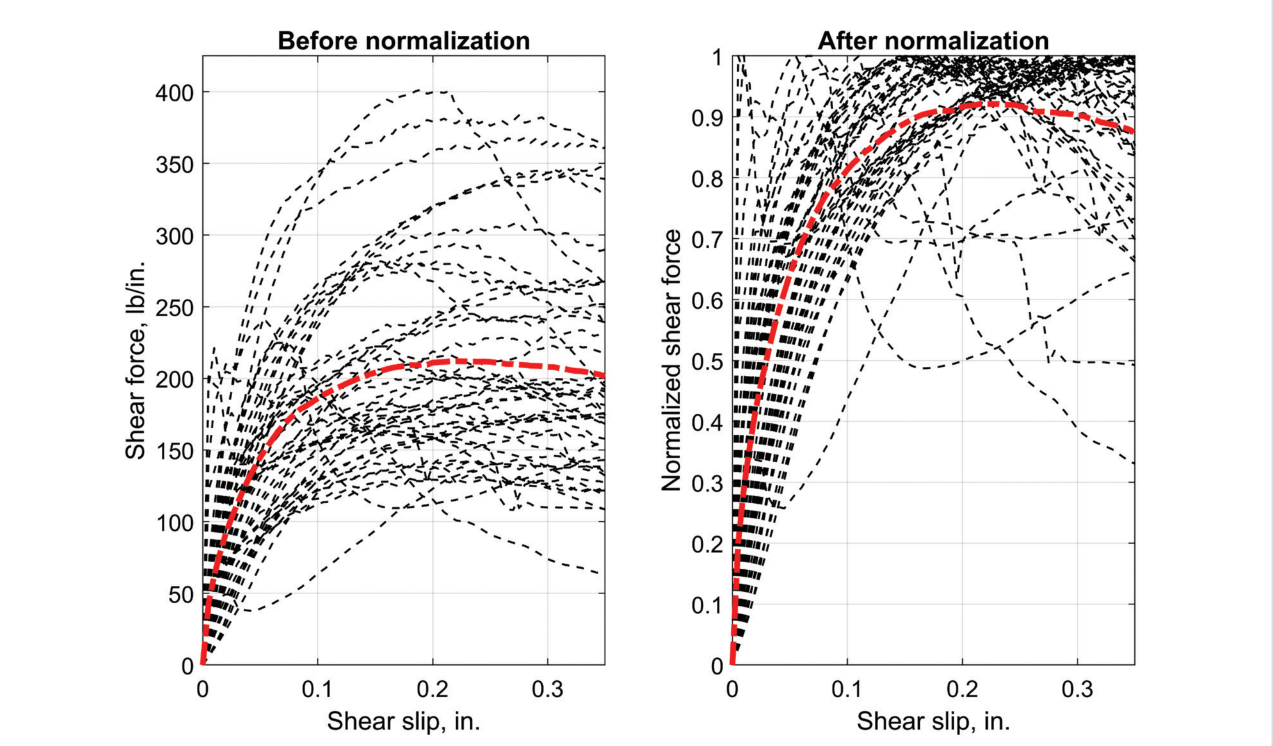

Figure 1. Comparison of average shear-displacement curves (red line) before and after shear force normalization. Note: 1 in. = 25.4 mm; 1 lb/in. = 175.1 N/m.

ed, one each from Naito et al.[7] (type D) and Bunn,[19] because the connector product type was the same across all of those tests. For the purposes of this study, this normalization method was applied only for the calculation of Δ_1^a and Δ_2^a, which measures the variability among experimental curves within a given group.

The intent of this methodology was to not limit the shear-displacement response to discrete values such as the peak load or the force at a specific displacement. These points alone are insufficient to fully capture the mechanical behavior of the connectors. To provide a more comprehensive representation, an approach based on strain energy, calculated as the area under a shear force–displacement curve, was adopted. This method offers two advantages. First, it accounts for the entire mechanical response, including the elastic phase, post-peak behavior, and energy dissipation related to ductility, and second, it allows for a global trend to emerge, avoiding the overinterpretation of local variations. For each curve, the strain energy is calculated by using a trapezoidal numerical integration scheme to sum the area under the curve between each pair of consecutive points. The calculated total strain energy for each double shear specimen is then divided by the corresponding strain energy of the average curve for its population set. The mean and standard deviation of the resulting unitless ratios are then computed before the statistical parameters needed to calculate the contributions of this source of uncertainty to the global uncertainty index are determined (**Table 1**).

Accuracy of idealized backbone curves (type b uncertainty)

To quantify the second source of uncertainty selected in this study, we created two types of simplified backbone curves to approximate the performance of a specific connector, insulation type, and installation configuration from available

Table 1. Statistical parameters for variability of wythe connector data

Connector category	n	μ	σ	Δ_1^a	Δ_2^a
Discrete	31	0.98719	0.06396	0.06479	0.01164
Continuous	43	0.94597	0.08091	0.08554	0.01304

Note: a = descriptor for uncertainty type a, variation in individual shear–displacement curves for a given type of wythe connector with respect to the mean curve from the sample set; n = population size; Δ_1^a = coefficient of variation considering uncertainty type a; Δ_2^a = coefficient of variation divided by the square root of the population size considering uncertainty type a; μ = mean of population; σ = standard deviation of population.

experimental test results. The first option for backbone curve generation used in this study is based on a general format for extracting critical points from raw test data proposed by Naito et al.[7] This method involves determining the slope of the initial linear-elastic segment of the curve K, finding the maximum shear force V_{max} from the dataset, and dividing V_{max} by K to yield an approximate yield shear displacement Δ_y. The last critical point on the curve is the displacement Δ_u, which corresponds to a force that has degraded to one-half of V_{max} in the post-peak region of the raw data curve. In our study, an additional point was added for the last recorded displacement value to better capture any significant ductility that a particular specimen exhibited. This additional point does not need to be consistent across all curves because each individual curve has its own unique backbone curve.

Whereas the first type of backbone curve in our study used 5 points, the second type of backbone curve used 11 points (including the origin). This second type of curve provides a more accurate understanding of the actual evolution of the curves and thus improves the fidelity of the backbone estimation. A logarithmic scale was used to skew the number of new displacement points for the backbone curve to earlier in the shear-displacement response where more significant milestones generally occur. The curve was defined using a set of shear displacement values (determined by the logarithmic scale), and the corresponding shear force values were interpolated from the raw data. **Figure 2** illustrates the improved accuracy of the enhanced backbone curve compared with the methodology proposed by Naito et al.[7]

None of the backbone curves were normalized because each specimen has its own distinct raw data and backbone curve. The strain energy approach detailed in the previous section was also followed for this backbone curve assessment to capture the full behavior of each specimen throughout its entire loading history. **Table 2** presents the pertinent statistical parameters after the strain energy for each backbone curve was divided by the corresponding value from the raw curve for each specimen.

Accuracy of the beam-spring model (type c uncertainty)

As discussed earlier in this paper, one of the underlying reasons for implementing strength reduction factors is to compensate for inaccuracies in design equations or analysis approaches. Strength reduction factors for precast concrete insulated wall panels need to address uncertainty related to the accuracy of flexural resistance models of a partially composite insulated wall panel relative to the corresponding experimental test results. In our study, we used the beam-spring analysis approach to model the panels because it is popular in precast concrete design practices and has a proven record for estimating the flexural response of various types of precast concrete insulated wall panels.[1,3,4,12,20] An important attribute of this model type is its general ability to monitor the status (that is, progression along the specific backbone curve) of each wythe connector row at each loading increment imposed on the panel to facilitate direct correlations between wythe connector property milestones and the global limit states of the panel.

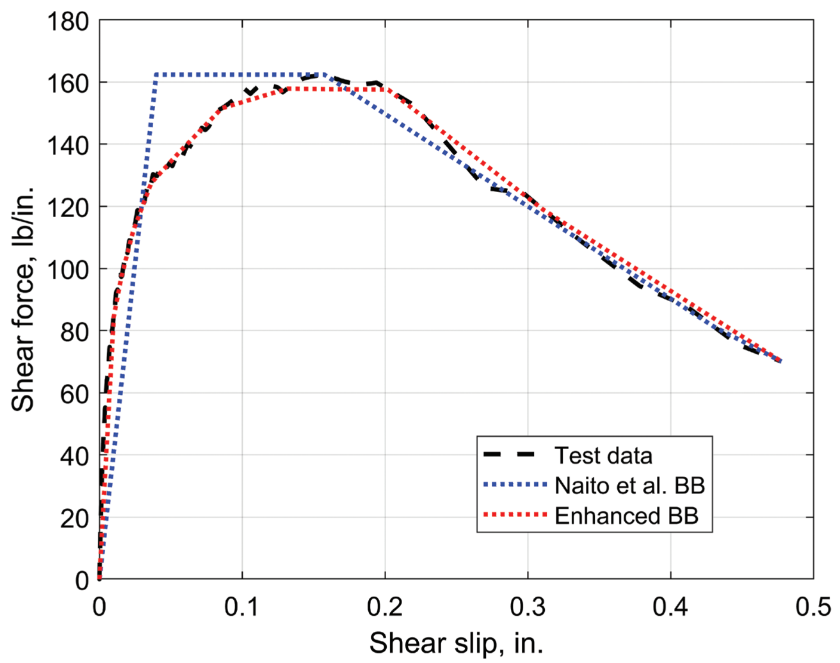

Figure 2. Accuracy of the two backbone curve approaches compared with experimental double shear test data. Note: enhanced BB = enhanced 11-point backbone curve generated using the method presented in this study; Naito et al. BB = backbone curve generated using the method from Naito et al. (2012). 1 in. = 25.4 mm; 1 lb/in. = 175.1 N/m.

Table 2. Statistical parameters for accuracy of backbone curves

Connector category and backbone curve format	n	μ	σ	Δ_1^b	Δ_2^b
Discrete, Naito et al. BB	31	1.01210	0.10329	0.10206	0.01833
Discrete, 11-point BB	31	1.02019	0.04805	0.04710	0.00846
Continuous, Naito et al. BB	43	0.99704	0.07163	0.07184	0.01096
Continuous, 11-point BB	43	1.01324	0.02462	0.02430	0.00371

Note: 11-point BB = enhanced 11-point backbone curve generated using the method presented in this study; b = descriptor for uncertainty type b, errors caused by simplifying the raw shear compared with displacement history to a backbone curve for software-based structural analysis; n = population size; Naito et al. BB = backbone curve generated using the method from Naito et al. (2012); Δ_1^b = coefficient of variation considering uncertainty type b; Δ_2^b = coefficient of variation divided by the square root of the population size considering uncertainty type b; μ = mean of population; σ = standard deviation of population.

For this study, we used an open-source analysis platform to create the partially composite beam-spring models, following the framework specified by Lallas and Gombeda,[20] to generate the flexural resistance function and extract pertinent limit states. The framework to calculate new strength reduction factors presented in this paper uses the peak moment resistance limit state. Although other pertinent panel limit states exist, we chose the peak moment limit state to help capture the full extent of the interwythe shear response, including peak shear resistance and ductility.

The test matrix for this effort included 30 larger-scale precast concrete insulated wall panel specimens: 15 with discrete wythe connectors and 15 with continuous wythe connectors. For the discrete connector dataset, three of the panels were originally tested by Maguire and Al-Rubaye[13]; these specimens contained connector type A as defined by Maguire and Al-Rubaye. The remaining 12 specimens in the discrete connector dataset were from Naito et al.[17] Of those 12 panels, 9 contained connector type B and 3 contained type C, as defined by Naito et al.[7] The dataset for panels with continuous connectors included 3 that were tested by Maguire and Al-Rubaye,[13] 9 tested by Naito et al.,[17] and 3 tested by Trasborg.[16] All of these specimens contained the CFRP grid connectors labeled as D-1 or D-2 by Naito et al.[7]

The peak flexural resistance extracted from the results of the partially composite beam-spring model was divided by the corresponding peak value from the experimental resistance function for each specimen. Statistical analysis of these ratios followed the same procedure described previously in the sections on variability of wythe connector test results and accuracy of idealized backbone curves. This analysis determined the coefficient of variation and other parameters (**Table 3**) needed to compute the general uncertainty indexes. The results shown in Table 3 generally imply that compared with the inherent wythe connector properties and the accuracy of the backbone curves, more variability is associated with the accuracy of flexural resistance models.

Implications when assuming composite action response extrema Because some traditional analysis approaches for precast concrete insulated wall panels have relied on simplified assumptions for composite action behavior, we assessed the implications of assuming either the theoretical noncomposite or fully composite bounds for the calculation of strength reduction factors. To accomplish this objective, we modified the partially composite beam-spring models used to generate the parameters in Table 3 by changing the wythe connector property definition. The backbone curves in the two node link elements were replaced with hypothetical linear-elastic stiffnesses of 1.0×10^{-10} kip/in. (1.8×10^{-8} kN/m) and 1.0×10^{10} kip/in. (1.8×10^{12} kN/m) for the theoretical noncomposite and fully composite response extrema, respectively. Following the approach used in the previous partially composite exercise, the peak value of each beam-spring model's noncomposite and fully composite flexural resistance function was divided by its corresponding peak experimental moment resistance in each case. **Table 4** shows drastic differences in the Δ_i^c values for

Table 3. Statistical parameters for assessing the accuracy of the beam-spring models

Connector category	n	μ	σ	Δ_1^c	Δ_2^c
Discrete	15	0.89506	0.08954	0.10004	0.02583
Continuous	15	0.73568	0.13456	0.18291	0.04723

Note: c = descriptor for uncertainty type c, discrepancies between experimental test results on larger-scale insulated wall panel specimens and corresponding analytical estimates for these components generated using beam-spring modeling; n = population size; Δ_1^c = coefficient of variation considering uncertainty type c; Δ_2^c = coefficient of variation divided by the square root of the population size considering uncertainty type c; μ = mean of population; σ = standard deviation of population.

Table 4. Statistical parameters for beam-spring model response extrema

Connector category	Theoretical composite action response	n	μ	σ	Δ_1^c	Δ_2^c	Δ_1^c % error relative to PC case, %
Discrete	NC	15	0.48811	0.17748	0.36361	0.09388	263.5
	FC		1.50859	0.35766	0.23709	0.06122	137.0
Continuous	NC	15	0.37140	0.16200	0.43618	0.11262	138.5
	FC		1.11097	0.35121	0.31613	0.08162	72.8

Note: c = descriptor for uncertainty type c, discrepancies between experimental test results on larger-scale insulated wall panel specimens and corresponding analytical estimates for these components generated using beam-spring modeling; FC = fully composite; n = population size; NC = noncomposite; PC = partially composite; Δ_1^c = coefficient of variation considering uncertainty type c; Δ_2^c = coefficient of variation divided by the square root of the population size considering uncertainty type c; μ = mean of population; σ = standard deviation of population.

the noncomposite and fully composite cases relative to the partially composite case, with errors ranging from 72.8% to 263.5% across the discrete and continuous wythe connector categories. Also, as expected, the mean of the peak flexural resistance ratios is significantly lower than 1 for the lower-bound noncomposite cases and greater than 1 for the upper-bound fully composite cases. These observations highlight the importance of modeling insulated wall panels with expected material properties and response mechanisms to avoid an unnecessary penalty when computing reliability metrics for precast concrete insulated wall panels.

Calculation of global uncertainty indexes

Based on the statistical parameters calculated for wythe connector variability, backbone curve accuracy, and beam-spring modeling accuracy, we used Eq. (3) to compute the global uncertainty index values for the discrete and continuous wythe connector systems and panels. **Table 5** shows that in this study, discrete connector systems demonstrated less uncertainty relative to the continuous connector systems.

Table 5. Global uncertainty indexes Ω

Analysis case	Ω
Discrete, Naito et al. BB	0.16050
Discrete, 11-point BB	0.13152
Discrete, NC	0.38126
Discrete, FC	0.25356
Continuous, Naito et al. BB	0.22012
Continuous, 11-point BB	0.20923
Continuous, NC	0.45872
Continuous, FC	0.33777

Note: 11-point BB = enhanced 11-point backbone curve generated using the method presented in this study; FC = fully composite; Naito et al. BB = backbone curve generated using the method from Naito et al. (2012); NC = noncomposite.

In addition, compared with the simplified backbone curve format proposed by Naito et al.,[7] the 11-point backbone curve reduced the global uncertainty indexes for the discrete and continuous connectors by 18.1% and 4.95%, respectively. As discussed previously, a higher global uncertainty index will generally facilitate a lower, more-penalizing strength reduction factor to account for greater design uncertainty. Employing the simplified noncomposite and fully composite assumptions had a negative impact on the global uncertainty index for both the discrete and continuous connector systems. It should be noted that when global uncertainty indexes were computed for the noncomposite and fully composite cases, the backbone curve accuracy statistical parameters were omitted from the calculation of Ω. This decision was made because the approximation of shear resistance is no longer needed at these two theoretical response extrema. However, the variability of the wythe connectors was included because those connectors were physically installed, even if their expected shear force–displacement record was not accounted for when the flexural resistance of the panel was estimated. The noncomposite assumption led to 189.9% and 119.2% increases in the global uncertainty indexes for the discrete and continuous connectors, respectively. Similar observations were made when assessing the fully composite assumption; the global uncertainty indexes increased by 92.8% and 61.4% for the discrete and continuous connectors, respectively.

Computation of strength reduction factors

The procedure described in this paper to calculate strength reduction factors is based on the approach outlined by MacGregor.[8] Before discussing the procedure and its results, it is important to note a limitation of the analyses performed in this study. The strength reduction factors were calculated solely from the resistance side of the traditional design equations used for precast concrete structures. The authors recognize that a complete and thorough reliability assessment and the subsequent development of strength reduction factors would also require an in-depth examination of the variability of the loading conditions and cases imposed on the component. However, the scope of this particular study was limited to assessing the variability of wythe connector properties and the accuracy of the beam-spring modeling approach commonly used to design these panels. Therefore, the intention is for committees that develop building code provisions or design guidelines to use the findings of this study as part of a larger holistic examination of the reliability of precast concrete insulated wall panel systems.

The mathematical expression used to compute strength reduction factors ϕ herein follows the format of Eq. (4), where γ_r is defined by MacGregor[8] as the ratio of the mean strength of the member to its design strength.

$$\phi = \gamma_r e^{-\beta\alpha\Omega} \tag{4}$$

where

α = separation function

For the purposes of this paper, γ_r is set to 1, because inaccuracies in the design methodology have already been factored into the calculation of the Ω values (see the previous section on the accuracy of the beam-spring model). The parameter α (also known as the separation function) is set to 0.75, based on the procedure and examples published by MacGregor. This parameter has been shown to be approximately equal to 0.75 through a triangular approximation, which uses 0.75 times the linear summation of the uncertainties in load and resistance rather than the square root of the sum of squares of these uncertainties that appear in the probability-based design equation. This approximation separates the two uncertainties in a linear format, allowing independent equations to be written for the load and resistance reduction factors (see Ang and Tang[9]). In Eq. (4), β (also referred to as the safety index) is directly derived from an accepted probability of failure for a given structural component. More specifically, β is the multiple of standard deviations away from the mean to a point at which the area under the standard normal distribution curve to the left of that point equals the accepted probability of failure.

In this paper, we do not recommend a specific probability of failure for an insulated wall panel at its peak limit state. (That decision would be the responsibility of a design standard technical committee or other governing body.) Instead, the alternative strength reduction factors proposed herein are presented as a function of varying β values (and their corresponding probabilities of failure). The results could easily be expanded to accommodate additional values of β, if needed or desired, because the underlying Ω values would not change using this approach, unless additional data on wythe connector systems were integrated into the previous steps to calculate the Ω values.

Tables 6 and **7** present the proposed interwythe-controlled strength reduction factors for the discrete and continuous connector systems, respectively, examined in this study. The probabilities of failure considered range from 1×10^{-11} to 1×10^{-6} purely as part of an exercise to calculate strength reduction factors across a range of hypothetical β values. The results show that, as is expected with this type of reliability-driven analysis, the strength reduction factors decrease or become more restrictive for design as the accepted probability of failure decreases (or the safety index increases as shown in **Fig. 3**). This trend is qualitatively justified by acknowledging that a strength-driven failure of a structural member is theoretically less likely as the margin between the design strength and the expected strength widens. More specific results within the context of wythe connectors demonstrate that a higher-fidelity backbone curve approach (that is, an approach containing a greater number of points) and modeling the expected partially composite behavior of the insulated wall panel using the beam-spring methodology facilitated the largest and most liberal strength reduction factors in all cases. Using the simplified backbone curve approach proposed by Naito et al.[7] led to slightly lower ϕ factors, generally decreasing by only a few hundredths, especially at larger probabilities of failure. Reductions in the ϕ factors are more significant when the noncomposite and fully composite assumptions are used to generate the flexural resistance function of the wall panel. This effect is more pronounced for the theoretical noncomposite assumption, leading to minimum ϕ reductions of 0.188 and 0.174 for discrete and continuous connector systems, respectively, when compared with the 11-point backbone curve approach to model the partially composite response.

Table 6. Proposed inter-wythe-controlled strength reduction factors for discrete connector systems

Probability of failure p_f	Safety index β	Proposed strength reduction factor ϕ			
		11-point BB	Naito et al. BB	FC	NC
1×10^{-1}	1.28	0.881	0.857	0.784	0.693
1×10^{-2}	2.33	0.795	0.755	0.642	0.514
1×10^{-3}	3.09	0.737	0.689	0.556	0.413
1×10^{-4}	3.72	0.693	0.639	0.493	0.345
1×10^{-5}	4.26	0.657	0.599	0.445	0.296
1×10^{-6}	4.75	0.626	0.565	0.405	0.257

Note: 11-point BB = enhanced 11-point backbone curve generated using the method presented in this study; FC = fully composite; Naito et al. BB = backbone curve generated using the method from Naito et al. (2012); NC = noncomposite.

Table 7. Proposed inter-wythe-controlled strength reduction factors for continuous connector systems

Probability of failure p_f	Safety index β	Proposed strength reduction factor ϕ			
		11-point BB	Naito et al. BB	FC	NC
1×10^{-1}	1.28	0.818	0.810	0.723	0.644
1×10^{-2}	2.33	0.694	0.681	0.554	0.449
1×10^{-3}	3.09	0.616	0.600	0.457	0.345
1×10^{-4}	3.72	0.558	0.541	0.390	0.278
1×10^{-5}	4.26	0.512	0.495	0.340	0.231
1×10^{-6}	4.75	0.475	0.456	0.300	0.195

Note: 11-point BB = enhanced 11-point backbone curve generated using the method presented in this study; FC = fully composite; Naito et al. BB = backbone curve generated using the method from Naito et al. (2012); NC = noncomposite.

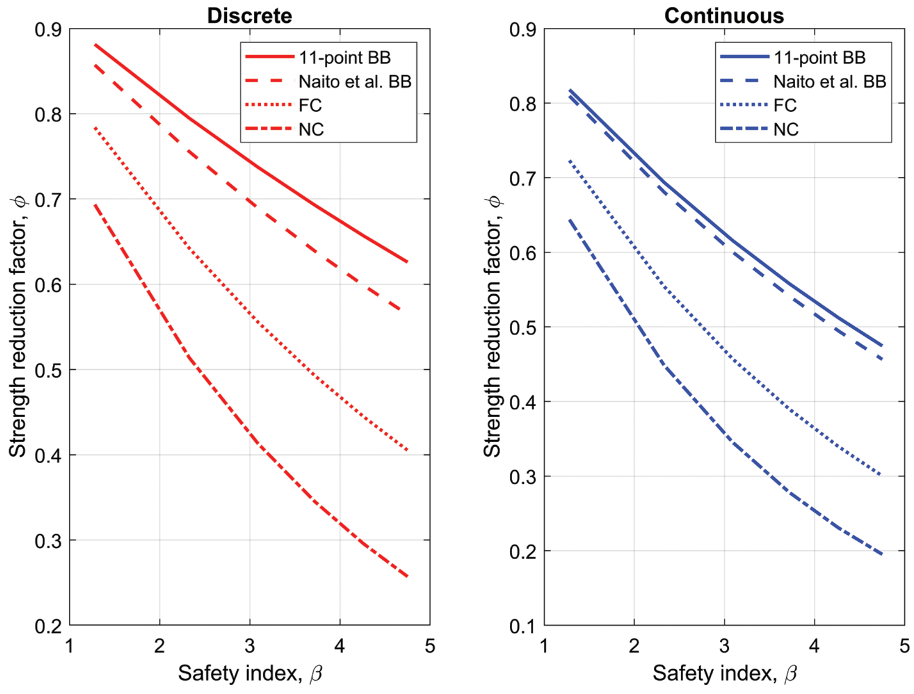

Figure 3. Plots of strength reduction factors *f* as a function of safety index *β*. Note: 11-point BB = enhanced 11-point backbone curve generated using the method presented in this study; FC = fully composite; Naito et al. BB = backbone curve generated using the method from Naito et al. (2012); NC = noncomposite.

Conclusion

This paper details the development of alternative inter-wythe-controlled strength reduction factors for precast concrete insulated wall panels loaded in flexure. The peak flexural resistance of the panel was chosen as the main limit state because this response milestone generally manifests when most panel wythe connectors are past their proportional limit, which introduces a much greater level of uncertainty for the panel compared with the uncertainty during its elastic response. The procedure used to develop these factors in this study was fundamentally based on principles of limit state design, and the proposed strength reduction factors were quantified based on a comprehensive dataset of wythe connector test results and performance metrics gathered from larger-scale insulated wall panel experimental programs. Reliability-focused contributions from three major categories of panel behavior were assessed: variability of wythe connector test results against a representative average curve, accuracy of simplified shear-displacement backbone curves used as input to computational models for panels, and effectiveness of such models to capture physically observed responses of experimental wall panel specimens. The combined uncertainty of these categories was then calculated, and, when complemented with a set of plausible design safety indexes, the results facilitated recommendations for new strength reduction factors for partially composite precast concrete insulated wall panels controlled by interwythe shear failure.

Based on the information presented in this paper, the following conclusions can be drawn:

- When assessing the variability of wythe connector double shear test data against the appropriate average curve, it was most effective to normalize the shear-displacement response to the peak shear resistance value. This strategy avoids introducing unnecessary uncertainties between different configurations of test specimens within the same connector category, especially given the relatively limited population size of applicable test data.

- The first two sources of uncertainty examined in this study (variability of wythe connectors against their average curve and accuracy of simplified backbone curves) were measured using the strain energy under the shear resistance–displacement curve. This approach helped quantify the full interwythe shear response and capture any notable ductility.

- Wythe connectors were grouped into two distinct categories—discrete and continuous—due to possible disparities with installation strategies, modeling techniques, and failure mechanisms. The final discrete and continuous connector datasets had population sizes of 31 and 43 tests, respectively, and the coefficients of variation were 0.065 and 0.086, respectively, when comparing the ratios of normalized strain energy for the test results to those of the respective average curves.

- The enhanced 11-point backbone curve approach generally facilitated more-accurate representations of double shear test data than the approach proposed by Naito et al.[7] Compared with Naito and colleagues' approach, the enhanced backbone option led to 53.9% and 66.2% reductions in the coefficient of variation for the discrete and continuous connectors, respectively.

- After procuring a dataset of 15 experimental tests each for panels fabricated with discrete and continuous connectors, coefficients of variation between the peak flexural resistance of the beam-spring model results normalized to the corresponding experimental test value were 0.100 and 0.183, respectively. When the panels were modeled, traditional assumptions of noncomposite or fully composite behavior were associated with coefficients of variation that were 73% to 263% greater than the expected partially composite case. Therefore, these assumptions are not recommended for use in this framework.

- The alternative strength reduction factors were, as expected, highly dependent on both the selected safety index and the representative global uncertainty index. As one example, the computed factors were 0.881 and 0.818 for discrete and continuous wythe connector systems, respectively, when we assumed a lenient probability of failure of 0.1 and its corresponding safety index of 1.28. The authors recommend that technical committees tasked with developing design standards or recommendations consider these results after an appropriate safety index is agreed upon.

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Notation

a = descriptor for uncertainty type a, variation in individual shear–displacement curves for a given type of wythe connector with respect to the mean curve from the sample set

b = descriptor for uncertainty type b, errors caused by simplifying the raw shear compared with displacement history to a backbone curve for software-based structural analysis

c = descriptor for uncertainty type c, discrepancies between experimental test results on larger-scale insulated wall panel specimens and corresponding analytical estimates for these components generated using beam-spring modeling

i = generic descriptor for the uncertainty type

K = slope of the initial linear-elastic segment of the curve

n = population size

p_f = probability of failure

R/U = resistance-to-load ratio

V_{max} = maximum shear force

α = separation function

β = safety index

γ_r = ratio of the mean strength of the member to its design strength

Δ_1^a = coefficient of variation considering uncertainty type a

Δ_2^a = coefficient of variation divided by the square root of the population size considering uncertainty type a

Δ_1^b = coefficient of variation considering uncertainty type b

Δ_2^b = coefficient of variation divided by the square root of the population size considering uncertainty type b

Δ_1^c = coefficient of variation considering uncertainty type c

Δ_2^c = coefficient of variation divided by the square root of the population size considering uncertainty type c

Δ_i^c = both coefficient of variation considering uncertainty type c and coefficient of variation divided by the square root of the population size considering uncertainty type c

Δ_1^i = coefficient of variation uncertainty source i

Δ_2^i = coefficient of variation divided by the square root of the population size n for uncertainty source i

Δ_u = displacement corresponding to a force that has degraded to one-half of V_{max} in the post-peak region

Δ_y = yield shear displacement

μ = mean of population

σ = standard deviation of population

ϕ = strength reduction factor

Ω = global uncertainty index

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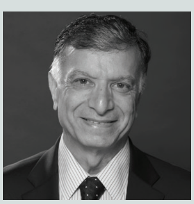

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Abstract

This paper details the development of alternative strength reduction factors that capture the behavior and variability of precast concrete insulated wall panels failing in interwythe shear at the peak flexural resistance limit state. Previous research has shown that this milestone generally manifests when most wythe connectors in a panel are past their proportional limit, which introduces a greater level of uncertainty for the panel compared with the uncertainty during its elastic response. The methodology addressed three types of uncertainties: the reliability of assessments of the variability of wythe connector data against average values, the accuracy of simplified design backbone curves, and the effectiveness of beam-spring models to capture the behavior of panels. The results facilitated recommendations for new strength reduction factors as a function of plausible safety indexes. These factors were as high as 0.881 and 0.818 for discrete and continuous wythe connectors, respectively, when assuming a very lenient probability of failure of 0.1 and a corresponding safety index of 1.28.

Keywords

Beam-spring modeling, flexural performance interwythe shear, limit state design, partial composite action, precast concrete insulated wall panel, reliability analysis in structural engineering, shear-displacement behavior, strength reduction factor, uncertainty, wythe connector.

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