Design Guide for Insulated Wall Panels using Beam-Spring Modeling Techniques, Part I 2025-03-20

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Part I –Theory and Modeling:

1.1 Introduction:

Precast concrete insulated sandwich wall panels have gained in popularity over the years. Typical applications include schools, gymnasiums, food processing plants, justice facilities and commercial/warehouse buildings. They provide a hard, durable surface both inside and out that is installed quickly in a single operation. Panels can be designed for blast-resistance, and are often load-bearing, carrying roof and floor loads, as well as providing the lateral shear resistance for the building.

Reasons for their increased use in place of more traditional materials include:

Speed of erection: A typical warehouse can be erected in a week. Cold weather is not an issue, since the panels are cast in a temperature-controlled environment and shipped to the site when needed. Weight is reduced and smaller cranes are often allowed when compared to tilt-up construction.

Design flexibility: The casting procedure allows for a large variety of finishes and patterns, including inset brick and stone.

Thermal efficiency: Edge-to-edge, top-to-bottom rigid insulation can be used within a relatively thin wall, providing a high effective R value. The thermal mass of the concrete provides an added benefit by slowing heat transmission through the wall, flattening out temperature swings.

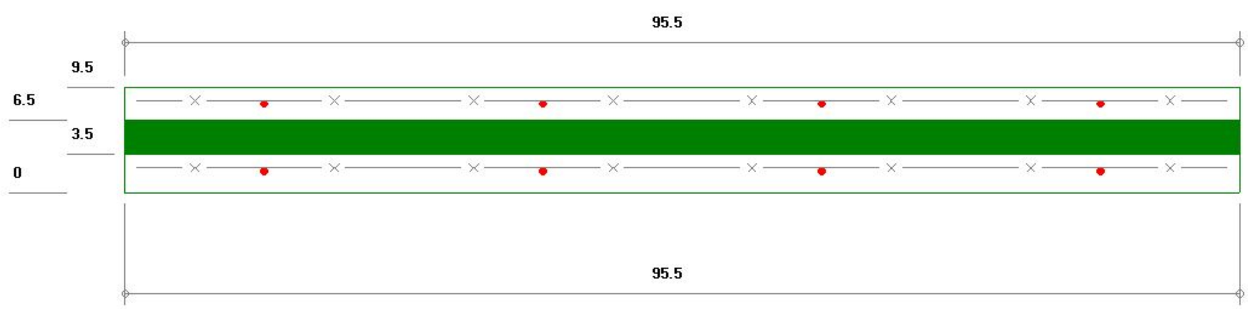
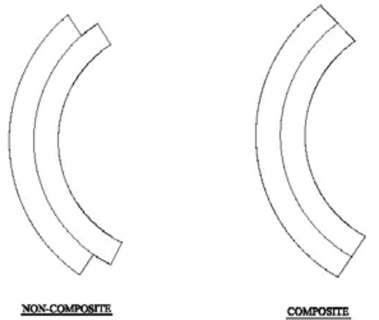


Figure 1: Cross-section through a typical insulated wall panel. In this example, the face wythe is ½ inch (1.25 cm) thicker to account for ½ inch (1.25 cm) deep reveals.

Figure 2: Loadbearing panels supporting a steel roof (Losch 2005)

There are different types of panel designs to consider:

Non-composite: With non-composite panels, the concrete wythes act independently (Figure 3a). This design is typically used when a high insulation value is required, such as for a cooler or freezer building. The wythes are isolated by high-performance rigid insulation and are connected together solely by thermally non-conductive pin connectors (Figure 3b). The pins are made of either a fiberglass and vinyl ester or polypropylene plastic or other non-conductive material. The interior structural wythe is usually much thicker than the exterior face wythe.



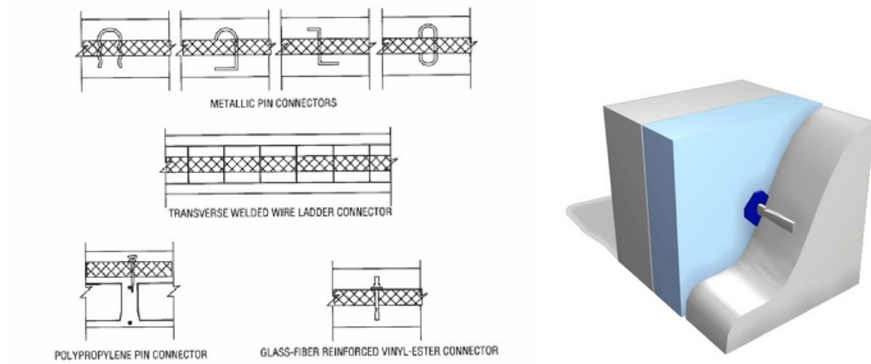


Figure 3a: Non-composite versus composite behavior (Losch 2005) *Figure 3b: Non-composite wythe connectors (Losch et al. 2011)*

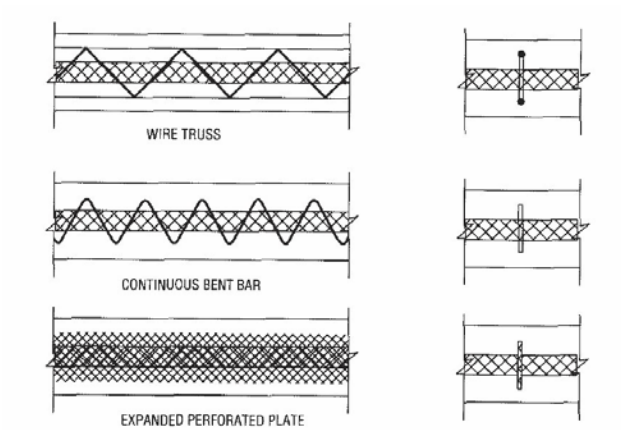


Figure 3c: Composite wythe connectors (Losch 2005)

Fully-composite: With fully-composite panels, the wythes act together as a unit for full horizontal shear transfer. A typical composite panel is eight times stiffer, can take three times the stress without cracking and has twice the ultimate strength of a non-composite panel of similar thickness. Composite panels are typically less expensive to make than non-composite panels, primarily because they can carry more loads and can be made taller and thinner. The inside and outside wythes are usually stressed with prestressing strand and made of equal thickness in order to minimize internal strains. Typically, steel truss type connectors and solid concrete sections are required to produce what has historically been considered to be a fully-composite panel. In actuality, It has been argued that no panel is truly fully-composite for all design limit states (Taylor Sorensen, Dorafshan, and Maguire 2018).

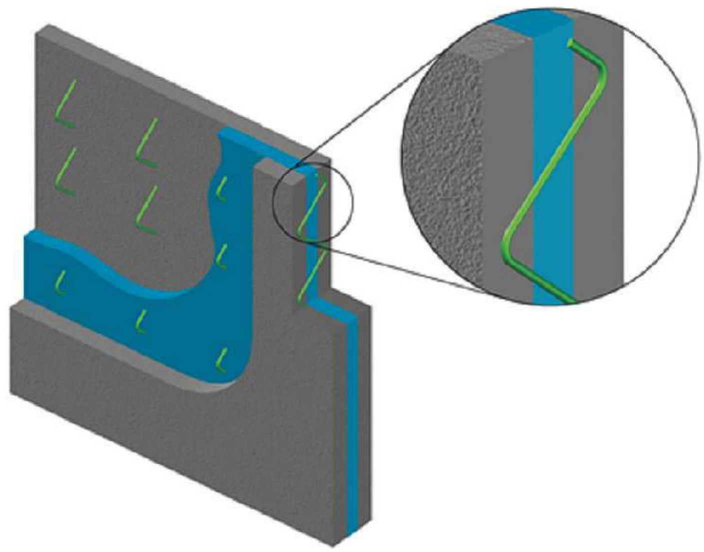


Figure 4: Typical partially-composite precast concrete insulated panel (Adapted from Al-Rubaye et al. 2018)

Partially-composite: Partially composite panels (the focus of this paper) provide less than full shear transfer between wythes (Fig. 4). They behave in a manner in-between composite and non-composite. The degree of composite action is often determined by load tests performed by an independent testing lab or in-house by the connector manufacturer. Proprietary partially-composite wall systems have become available which combine the high insulating value of non-composite panels with the strength and slenderness of composite panels. This is accomplished using non-conductive truss, grid or individual connectors between the wythes for shear transfer. Partial composite action provides sufficient strength for most applications in much the same way partially composite floor systems are usually more economical.

Partially composite walls are typically thinner and lighter than a non-composite sandwich wall panel, while still providing excellent thermal and structural efficiency with proper detailing. Historically, walls that were designed as fully composite are likely close to fully composite structurally, but require penetration of the insulation with solid concrete sections to exhibit such behavior, making them less attractive relative to increasingly stringent insulation requirements.

Analyzing the strength, ductility and flexibility of a partially-composite panel is more difficult than for a fully-composite or non-composite design. This is because the degree of composite action varies with the span, thickness and wythe connector layout, among other factors. Bunching the connectors near the ends will increase capacity, for example, though this is not advised by most connector manufacturers for analysis reasons. Paradoxically, shorter spans require more connectors per unit area to achieve the same degree of composite action. A panel may exhibit 100% composite action for ultimate strength, but only 25% composite behavior for deflection, further complicating the analysis.

1.2 Background:

Traditionally, composite wall panels have been fabricated using continuous steel wire trusses for the wythe connectors (Fig. 3c). Often, solid zones of concrete are also used at the top and bottom to bolster the shear capacity at the panel ends. The system is inexpensive, but it introduces thermal bridging through the trusses and solid zones, reducing the effective R value of the panel (Taylor Sorensen et al. 2019). This could be overlooked for some milder climates, as most heat transfer occurs through the roof and windows. In addition, there is thermal lag due to the panel mass, reducing daily temperature swings. In cold climates, however, cold spots and condensation have been known to form at the solid zones (Fig. 5).

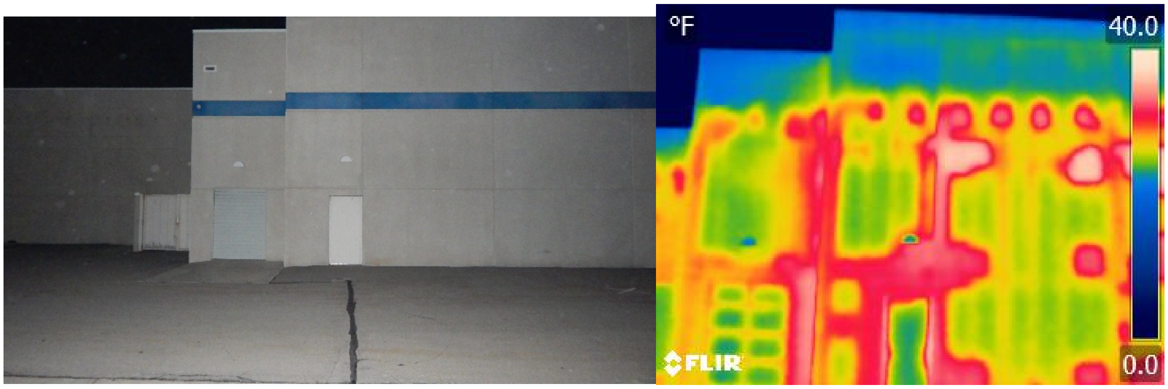


Figure 5. Color and Thermal images of precast concrete insulated wall panels containing steel truss connectors and solid zones . Note: °C = (°F – 32)/1.8. (courtesy of Taylor Sorensen)

To eliminate this thermal bridging, researchers and manufacturers have developed numerous alternative non-conductive wythe connector types (Fig. 6). Most are constructed from fiber-reinforced polymer (FRP), using either glass or carbon fibers. Since these connectors are more expensive than the standard steel trusses, there is an economic incentive to use less of them, or only as many as is necessary. For the vast majority of applications, full 100% composite action is not required to satisfy design loadings. This is because the panel thickness is usually dictated by the insulation value required, not by strength requirements.

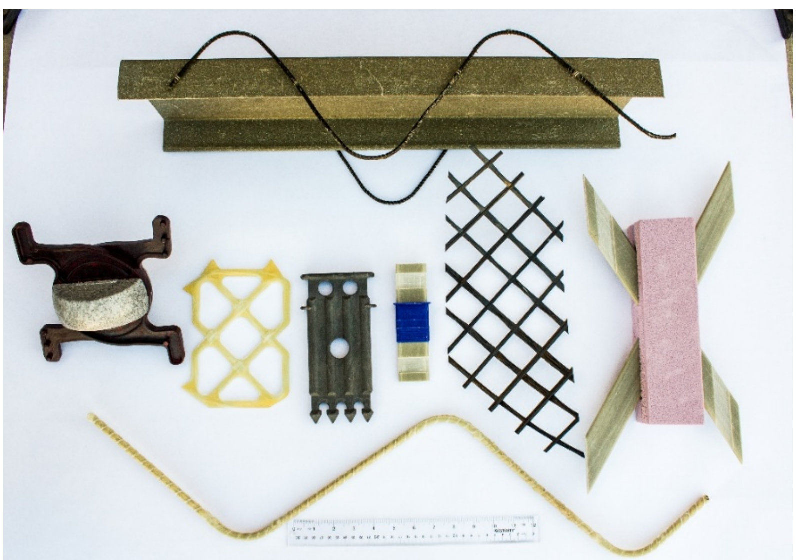


Figure 6: Wythe connectors used for partially-composite insulated wall panel fabrication (PCI Insulated Wall Panels Committee)

Even so, a standard design method for partially composite panels is needed that will be acceptable to the Engineer of Record and building code officials. Presently, this is achieved through empirical testing of the various panel and connector types. The goal of the testing is to provide percent composite values for strength, stress and deflection that the precast engineer can rely on to design the panels.

As noted in the Introduction, there are numerous factors that affect the composite strength of an individual panel – too many to be covered solely by empirical testing. It would be desirable to have a generalized design methodology that can be easily implemented. Utah State University conducted testing towards this end, funded by the Precast/Prestressed Concrete Institute (PCI), and others have been working on this problem for decades (Einea et al. 1991, 1994; Pessiki and Mlynarczyk 2003; Frankl et al. 2008; Gombeda et al. 2017; Al-Rubaye, Sorensen, and Maguire 2017b).

Until recently there was limited knowledge, outside of oft held trade-secret information, available for the analysis and design of partially composite sandwich wall panels in a general manner. Analysis and design procedures varied based on the proprietary system selected, though are thought to be somewhat similar. For instance, many relied on an effective section approach or “percent composite” approach that follows similar procedures to the design of a solid panel with penalized section properties and is limited to elastic design assumptions in many cases. PCI produced a “state-of-the-art” document, which in-lieu of codified procedures has been adopted by many as a code-like document (Losch et al. 2011). In 2024, PCI published the PCI 150-23 Standard: “Specification for the Design of Precast Concrete Insulated Wall Panels”. This document has been adopted by reference into ACI 319 and will have the force of law when 319 is adopted by the building codes. Several methodologies for the design of partially-composite panels are included in this Standard.

1.3 What is the Beam-Spring Method?

Before a standard design method can be implemented, a standard model for the wythe connector properties needs to be developed. While different connector systems may use specialized techniques that may be applicable to only their system, researchers have been working on ways to make the analysis process more generalized and accessible. One of the earliest known solutions to the concrete sandwich wall panel problem was completed by Holmberg and Plem (1965), who modified work by Granholm (1949) and Newmark (1951), however these approaches are largely not suitable for contemporary design due to complexity and only being applicable to steel wire truss type connectors.

Many connector manufacturers have implemented some version of a truss (axial or Vierendeel) for elastic analysis, which has been shown suitable (Al-Rubaye, Sorensen, and Maguire 2017a) Recently, Gombeda et al. (2017) and Tomlinson and Fam (2015) have developed methods capable of predicting the full moment curvature response. However, this level of complexity is not needed for design of composite wall panels, which are almost exclusively designed in the elastic range and engineers are reluctant to perform complicated analyses they do not understand and have difficulty checking. Olsen et al. (2017) provided a simplified approach, limiting iteration, and solving directly for elastic deflections and cracking moments as well as the ultimate moment (not ultimate deflection, which is more complicated). Engineers largely design partially composite walls in the elastic range and typically need to check three major limit states (cracking, deflections and ultimate strength). Because engineers can follow the simplified approach by Olsen et al. (2017) it is one of the most widely adopted methods in the industry, with nearly half of the known connector systems implementing it or in the process. The advantages of each of these contemporary solutions is that a phenomenological approach for connector behavior has been developed.

Nearly all contemporary approaches to this problem recognize that connectors come in all shapes, sizes, and materials, reflecting the industry. Rather than try to come up with a physics-based model for specific connectors (i.e., truss, grid, pin etc.), a generalized shear load versus displacement relationship is used. This behavior largely behaves like a shear spring element, which is why Al-Rubaye (2018) evaluated the use of a generalized matrix model approach termed the beam-spring model (see Figure 7).

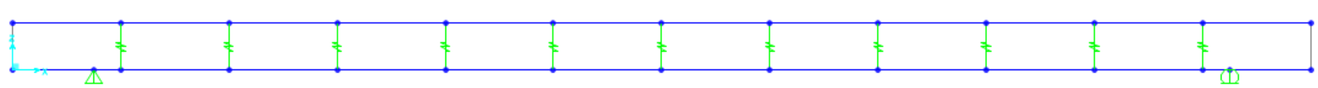


Figure 7: Example Beam Spring Model using gross section properties for wythes, spring elements for connectors, pin-and-roller boundary conditions

How researchers and practitioners arrive at this load versus displacement behavior is not uniform. Naito et al (2011) performed shear testing on various connector types using a double shear approach that uses two connectors per specimen. Olsen et al. (2017) used a large specimen that limited pinching effects (see Figure 8).

There are two International Code Council Acceptance Criteria that cover connector systems: AC320 and AC422 (ICC Evaluation Service 2010, 2015), however their use is not widespread. AC320 implies a single shear test that is similar to those produced by Naito et al. (2011). AC422 is written specifically for a double shear test tailored for only one specific connector system. The rest of the literature has used some variation of these programs. The true answer is probably none of these and there is concerted effort at PCI to develop a standard testing procedure to enable wider implementation of the above contemporary methodologies.

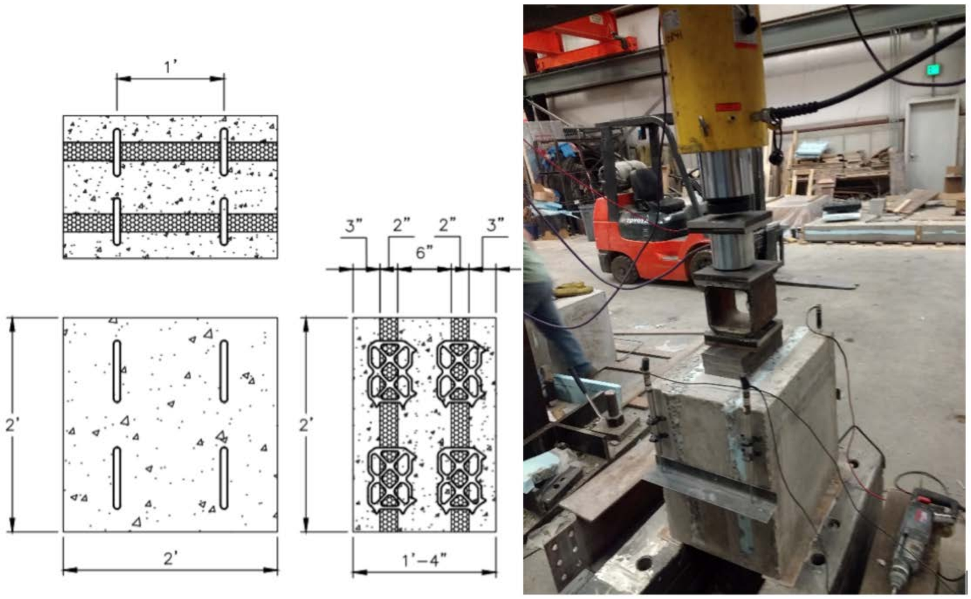


Figure 8: Example Double Shear Testing Setup (Adapted from Olsen et al., 2017)

To give the reader an idea of how these research results are to be implemented, this paper will summarize the results of Olsen et al. (2017). This will illustrate a matrix analysis technique, the beam-spring model, which is easily understood and accessible to engineers and architects.

1.3.1 Data:

Example connector properties are presented in Figure 9. Clearly, the behaviors of different systems provide different properties. For engineering purposes in the elastic range, the engineer requires elastic stiffness, elastic limit and ultimate load values. There is currently no standard governing how to select these values, where especially for elastic stiffness, there is some subjectivity. For the purposes of modeling, the exact curve can be used, but for engineering a safe structure most engineers pick a secant stiffness. AC422 arbitrarily selects a secant stiffness at 50% of the ultimate strength of the connector, which is likely an adequate design assumption.

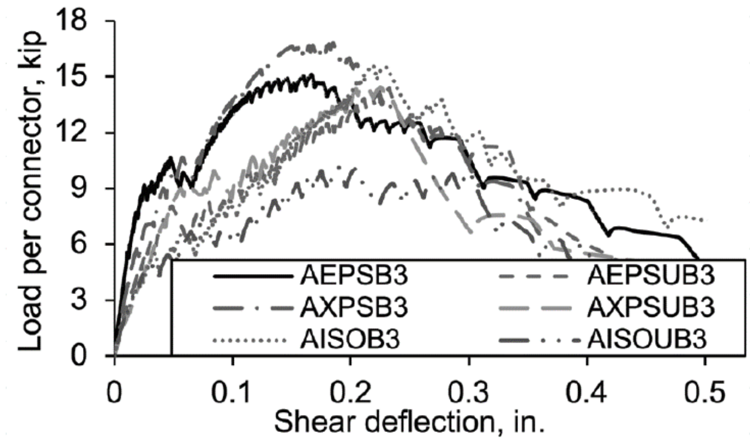


Figure 9: Example of Connector System Load versus Deflection Relationship (Adapted from Olsen et al. 2017)

Elastic stiffness properties can be assigned to the stiffness of shear spring elements in a beam-spring model (see Figure 7, additional details on constructing a beam-spring model are found in Al-Rubaye et al. (2018). Alternatively, these properties can be input into several other contemporary analysis techniques like those mentioned in the previous section.

To validate these assumptions large scale testing was performed. Figure 10 illustrates the ability of the beam-spring model to predict the elastic behavior (i.e., up through panel cracking) of the large-scale panels. Overall the models were able to predict deflection and cracking moment to within 10%, assuming cracking occurs at 7.5sqrt(f'c). Many engineers will design the panel elastically for ultimate loads to ensure safety and adequacy of the analysis techniques.

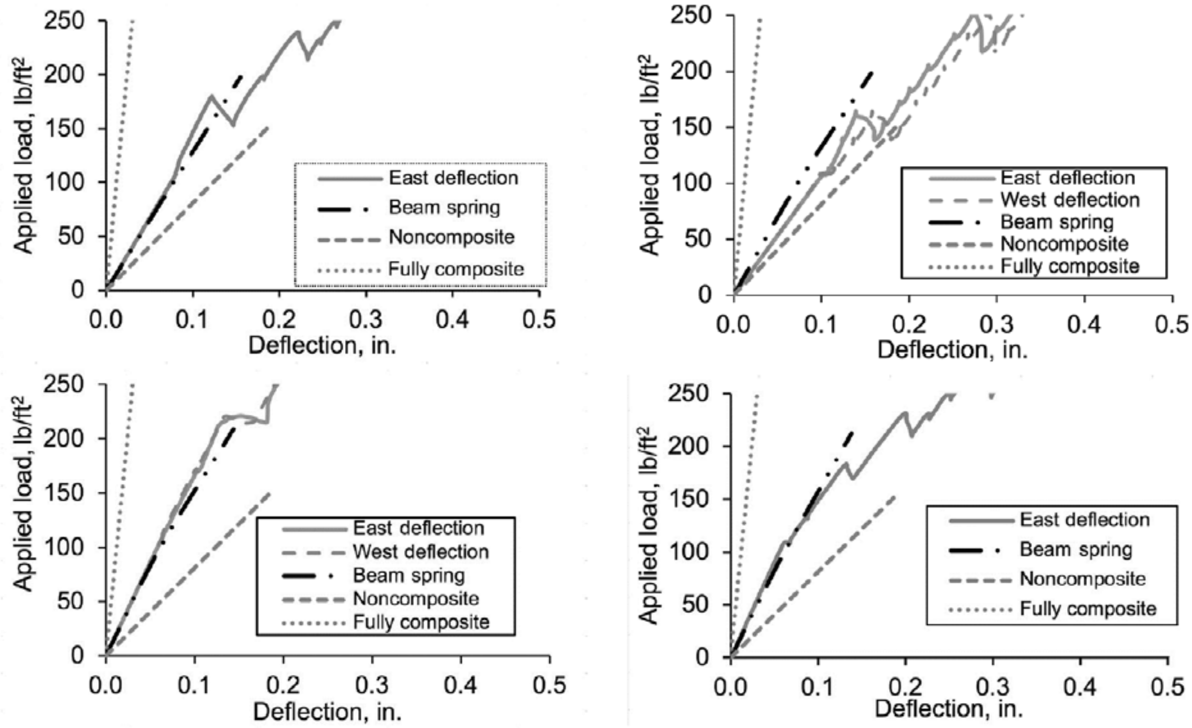


Figure 10: Fit of beam spring model to large scale partially composite sandwich wall panel tests (adapted from Olsen et al 2017)

1.3.2 Explanation:

The Beam-Spring model is intended to be a general-purpose model that can be used for any type of connector, whether truss, bar or plate. This general-purpose analysis model can produce accurate results and various methods are used routinely in the industry. These structures are being designed safely throughout the United States and Canada. It is anticipated that, when coupled with the coming standardized connector shear values, the beam-spring model should allow for the analysis of a wide-variety of panel system types. Code adoption of this or other analysis methodology and connector test method will allow innovations in wythe connector technology to continue, with confidence that the resultant designs will be code-compliant.

1.3.3 Design Curves:

It is expected that wythe connector strain curves will be provided by the system manufacturers, based on the PCI standard testing method (see Fig. 11). These values can be input to a beam-spring matrix, simplified analysis (Salam, 2021), or specialized software, to meet the requirements of the PCI 150 Design Standard. Maximum ultimate connector capacity (Fu) will be limited by a strength reduction factor (ϕ factor) ranging from 0.5 to 0.75.

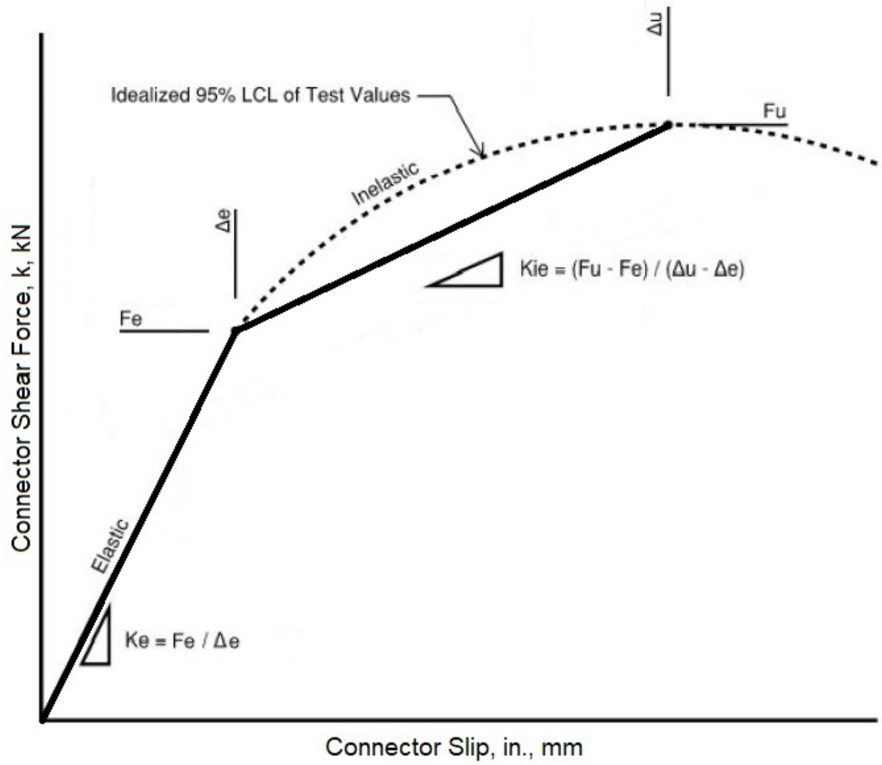


Figure 11: Schematic Wythe Connector Slip (strain) vs. force graph (LCL = Lower Control Line, Ke = elastic stiffness, Kie = inelastic stiffness)

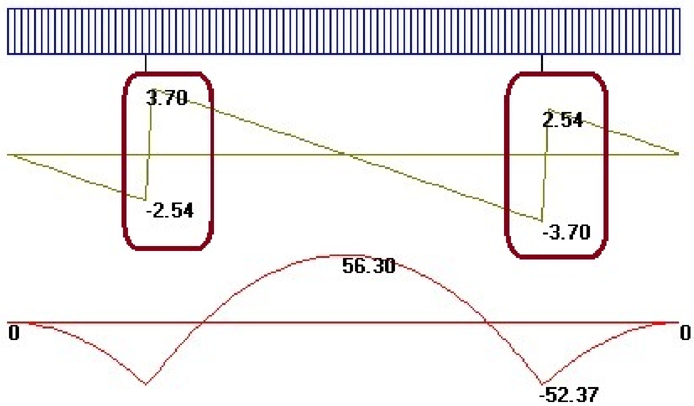
1.4 Advantages of the Beam-Spring Method:

Previous partially composite analysis methods relied either on a percent composite analogy (described below), manufacturer tables, or “black box” proprietary software. There were no recognized standards or agreement among the manufacturers regarding connector stiffness testing methods or phi-factors. Nevertheless, this panel type has performed well and continues to gain market share over other wall types, mainly due to its superior thermal performance, lower weight, and thinner profile. Here are some of the advantages of the beam-spring method for the engineer:

Not proprietary: Generic plane frame structural analysis software is all that is required for calculation. With agreed-upon standard connector stiffness values, an analysis performed by one engineer will match that performed by any other engineer. It’s not a “black box” that must be blindly trusted, but a repeatable, rational calculation method.

Unequal wythes: Most testing of wythe connector systems utilizes equal-thickness concrete wythes. It can be the case, however, where one wythe needs to be substantially thicker. This may be due to a prescriptive requirement, such as for jail or prison walls, or to accommodate pockets, embedded connections or lifters. As beam-spring uses a rational frame analysis, the forces and stresses in each wythe can be determined individually, similar to linked walls with an axial force component. This allows the thinner wythe to contribute to the panel strength, unlike with a non-composite design.

Shear reversals or non-uniform loads: Panel handling, typically with lifting at fifth points, generates shear reversals at the lifter locations. The percent composite and other legacy methods assume a simple span condition without any shear reversals. With a simple span, the connector slip increases linearly towards the member ends, with increasing shear force taken by the connectors. When a shear reversal occurs, the connectors in the area can no longer engage significantly, as there is very little connector slip. At this point, according to beam-spring analysis, composite action essentially disappears and individual wythe moments and stresses increase significantly. The legacy analysis methods do not take this loss of composite action into account.

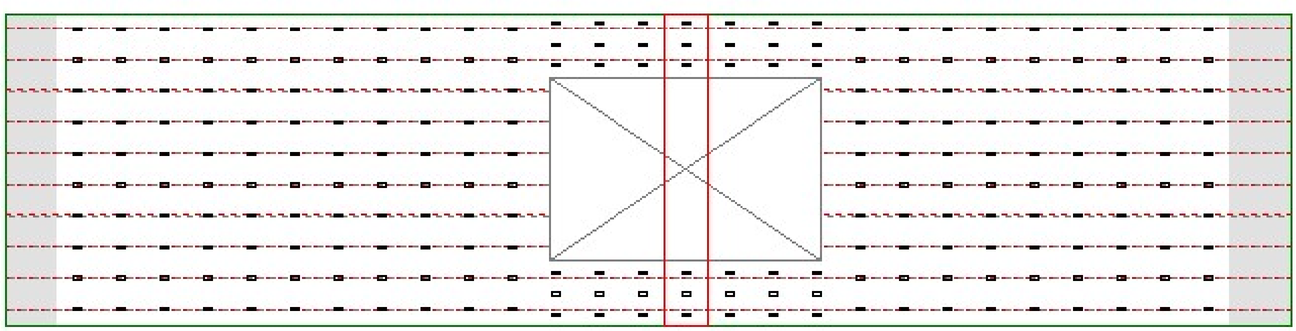


Shear reversals with 5th point lifting

The question then has to be asked – why do we not see more issues such as cracking when using the legacy methods? There may be other mechanisms at play, but, to-date, there has not been much testing of panels with shear reversal.

Panels with mid-height connections are also subject to shear reversal, as well as panels with large concentrated loads along the span. Beam-spring can account for all these conditions, while the legacy methods cannot.

Non-uniform connector spacing: Many panels have large openings. As such, it is not always possible to distribute the full number of wythe connectors alongside the opening. Beam-spring analysis can model fewer connectors at these locations. If the opening is near mid-height, wythe shear stresses may be low, so the missing connectors will likely have only a negligible effect on panel strength.



Wythe connector layout at a large opening

1.4.1 Limits of Applicability:

The beam-spring method depends on standard wythe connector shear stiffness curves. These are usually generated by double-shear tests of lab specimens according to PCI standards. If these are not available, then less than ideal means must be employed to get these values. It may be possible to back-calculate connector shear stiffness from panel test results. This can be a trial-and-error process involving tuning the beam-spring model element values to roughly match panel test results for stresses and deflection. In the Author's opinion, this is still better than to solely rely on percent composite or other legacy methods.

Standard beam-spring analysis is an elastic analysis (inelastic behavior will be covered in Part II). Connector shear-slip curves have two components, the elastic portion and the inelastic one (see diagram). The transition from elastic to inelastic is not always clear. For simplicity, the elastic limit can be set as ½ of the ultimate limit. An inelastic analysis is fraught with complications. It requires multiple runs to attain convergence, which can be subject to oscillations. Also, there is no "retreat" to elastic behavior once in the inelastic range. Pattern loads can also confuse the results.

As such, a conservative procedure is to use connector elastic values for ultimate loads. This provides a factor of safety to ensure that the connectors will not go into the inelastic range under service loads. If the elastic slip limit is exceeded, then more connectors can be added at that location.

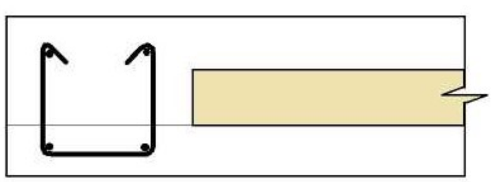
1.5 Implementation:

The beam-spring method can be used for both discrete and linear wythe connector types, as long as the stiffness per unit length is equivalent.

1.5.1 Panel Fabrication:

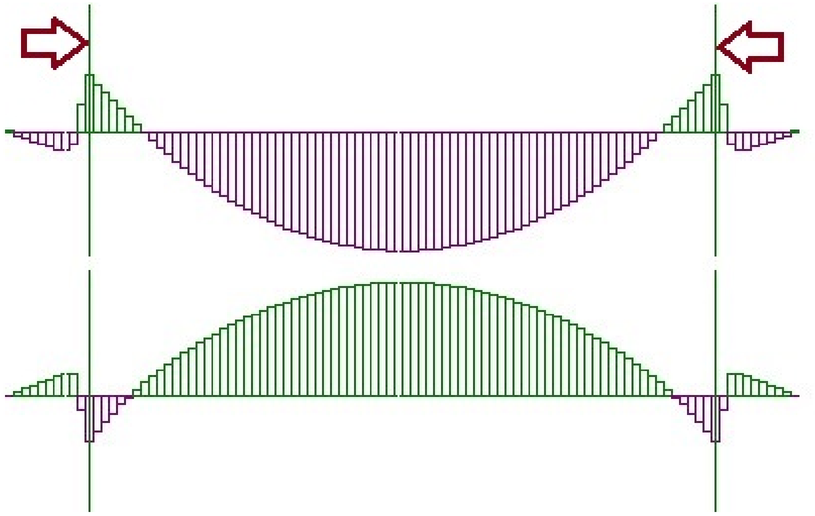
An important consideration regarding the fabrication of partially composite panels is to avoid un-intended (and un-modeled) solid zones. These can short-circuit thermal performance and also cause shrinkage cracking. This cracking is most likely to occur in the thinner wythe of panels with unequal wythe thicknesses. The thin wythe shrinks faster than the thick wythe. It can be considered as similar to a canvas stretched on a frame.

Solid zones are often necessary or desirable, especially at panel ends. This is acceptable as long as they are accounted for in the beam-spring model. Solid zones are modeled as extremely stiff nodes (connectors) between the wythes. When used at panel ends, they can significantly improve composite action for stresses and deflection. Composite ultimate strength is not considered unless the solid zones are large enough to take the full capacity of the opposing wythe reinforcing capacity in horizontal shear. According to ACI 318-19 Table 16.4.4.2, the shear capacity of the plain concrete interface with "1/4 in. amplitude intentional roughness" is 80 psi. This is further modified by a 0.75 Phi-factor for shear, netting 60 psi. If the solid zone area is not sufficient, a reinforcing cage can be used to provide metal ties in shear-friction across the shear interface:



A reinforcing cage at both member ends can give full composite action for simple-span ultimate strength.

When end solid zones are modeled with beam-spring, each wythe behaves like a beam with fixed ends. The analysis will show a "rebound" increase in flexural stresses where the panel insulation meets the solid zone:



Beam-spring model shows a flexural stress rebound in each wythe at the solid zone interface with the insulation (tension stress shown in green).

Although the end solid zones provide a significant flexural stress reduction, it is not nearly as much, and not the same as, a member that is fully composite across the full span (using continuous concrete ribs or similar means).

1.6 Comparison to the Percent Composite Method:

1.6.1 History:

The percent composite method became popular in the 1990s as a way to justify the design of composite insulated wall panels in a manner that engineers were familiar with and could be comfortable with. The primary composite systems at the time used continuous steel trusses for the wythe connectors. These systems usually tested out at 85% composite or better for strength, so the percent composite strength reduction was basically another phi-factor used to ensure an adequate design.

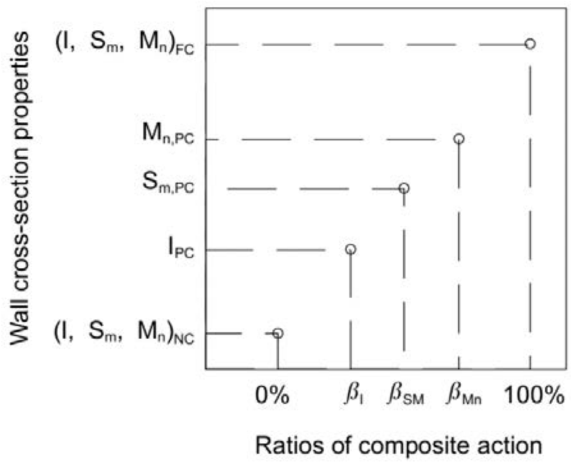
Over time it became apparent that the percent composite values for flexural stress and deflection differed from that used for strength. These values were usually lower than the strength value. The differences became more pronounced when using glass-fiber composite connectors instead of steel trusses. As such, the percent composite method was refined to provide separate percent values for section modulus (stress) and moment of inertia (deflection). As an example, a panel could be 80% composite for strength, 60% for flexural stress and 40% for deflection.

Percent composite properties are based on the percent difference between full composite and non-composite capacity, section modulus and moment of inertia of the section. For example, 80% composite moment of inertia ("IPC") would be:

IPC = I(0%) + (I(100%) - I(0%)) * 0.80

The deflection (moment of inertia) value is important for the P-Delta slenderness analysis of load-bearing panels, so this is not simply a service issue, but a consideration for strength as well.

A common question that is still asked of system manufacturers is: “What percent composite value should I use for your system?” The answer is complicated because it depends on many factors, such as the member span, for instance.



Interpolation to determine section properties of a partially composite panel.

1.6.2 Advantages:

The percent composite analogy is easy to understand and employ with the same design methods used for solid panels. Since no insulated panel without continuous solid zones can be truly 100% composite, it provides a numerical way to justify highly composite panels as "almost" fully composite. Also, until recently, there was not a widely accepted alternative to percent composite design.

1.6.3 Limitations:

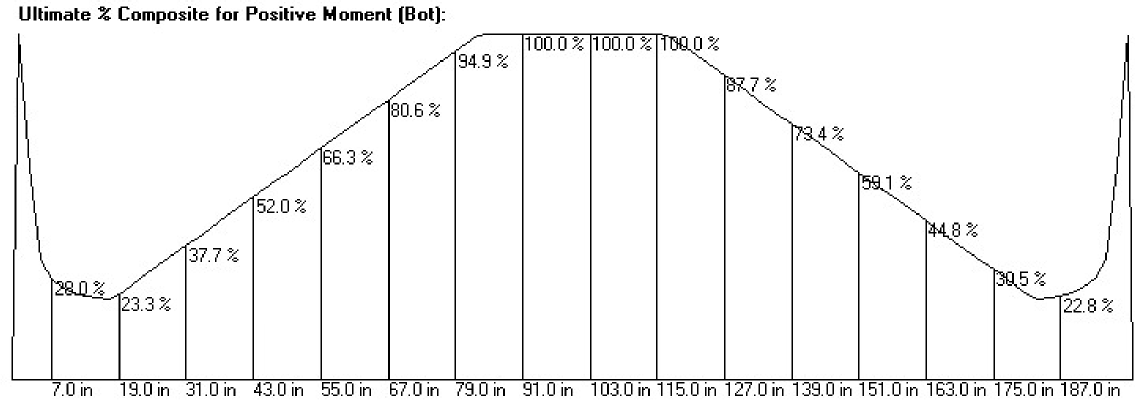
Percent composite is in many ways an over-simplification of insulated wall behavior. It is not based on a rational analysis but is instead based on observation and limited testing. Panels are tested in simple-span while deflection and stresses are measured. This is compared to theoretical 0% and 100% panel designs to provide the % composite ratio. The panels are then tested to failure to provide an ultimate strength ratio. The drawback with this approach is that these values are limited to a specific span, thickness, load type, connector layout, etc. Without many more tests, it can be guesswork to extrapolate percent composite values to any other combination.

Because it is more of an analogy than a method, percent composite is not applicable or reliable for some conditions. Situations where the percent composite analogy breaks down include unequal wythes, concentrated loads, intermediate connections and fifth point handling.

1.6.4 Strength Design:

Percent composite for ultimate strength is determined by the ability to transfer horizontal shear from the reinforcing in one wythe through the wythe connectors to the other wythe. The PCI report, "State of the Art of Precast/Prestressed Concrete Sandwich Wall Panels" (2011) contains a sample calculation using solid concrete zones (Fig. 2.7.4b). A calculation using wythe connectors instead of solid zones would be similar. (Wythe connector stiffness should not be combined with solid zones because the solid zones are much more rigid and would prevent the connectors from significantly engaging.)

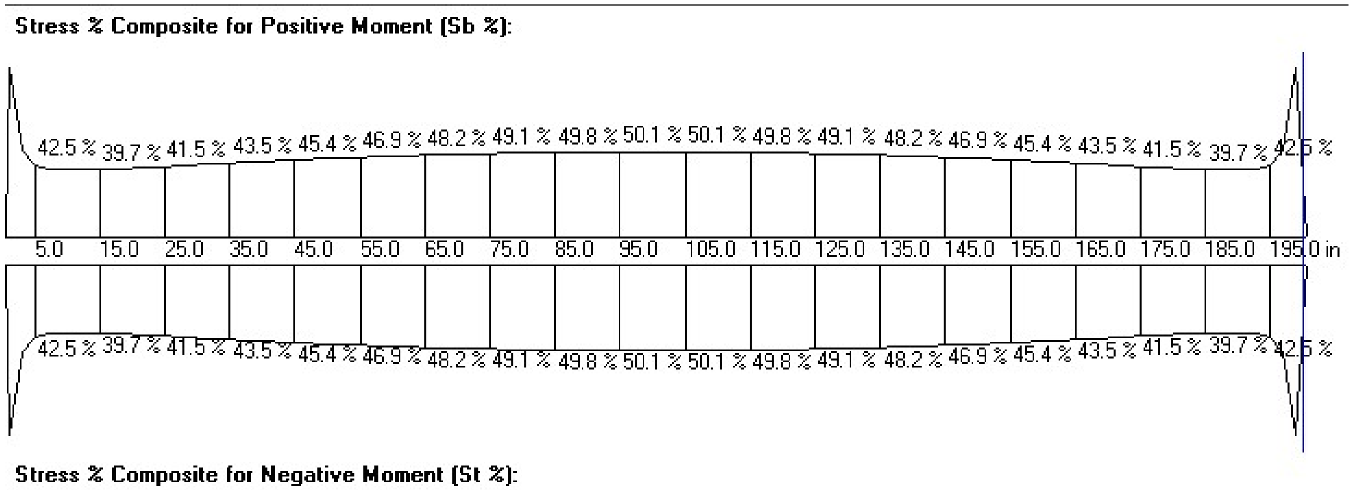
This method is suitable for simple spans only. An intermediate support could develop a plastic hinge. If one wythe has more flexural reinforcing than the other, than the composite percentage would differ for positive vs. negative moment. Composite percentage is greatest at mid-span and decreases toward the member ends (see below). This is because there are fewer connectors to engage near the ends.



Percent composite for ultimate strength graph (along span)

1.6.5 Stress Check:

Percent composite for stress is usually found as follows. A typical panel is instrumented and load-tested in the service range and the extreme fiber stresses are noted. From this, an equivalent partially composite section modulus is back-calculated. That modulus is compared to a theoretical 100% composite section and a 0% composite section to arrive at a percent composite value. Less commonly, the percent composite value is attained by comparing the actual extreme fiber stress to the theoretical 100% stress and the 0% stress. This approach gives different results from using the section modulus, it is more conservative but is difficult to apply to a load-bearing analysis.



A beam-spring frame analysis shows that stress percent composite is relatively constant along a simple span. This is not the case for multiple spans.

As noted previously, stress percent composite is greatly reduced when there are intermediate supports, as is the case with panel handling. In addition, unequal wythes can cause the percent composite analogy to break down. It's possible that the extreme fiber stress will occur on the insulation face of the thicker wythe instead of the exterior face of the thin wythe. A beam-spring analysis is required to find these stresses.

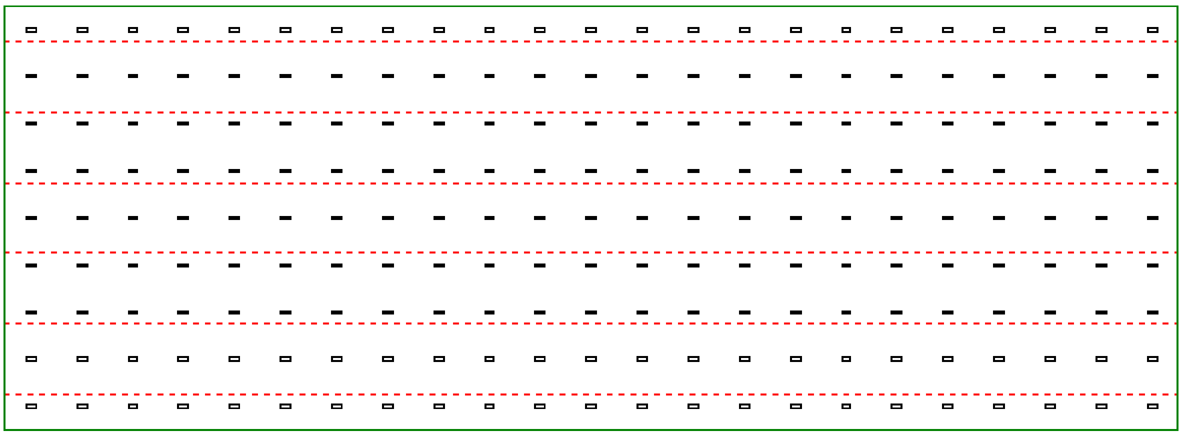
1.6.6 Deflection Check:

As with the stress check, mid-span deflection is noted on the test panel and compared to idealized 100% and 0% deflections to calculate a percent composite moment of inertia. The deflection percent composite is also relatively constant along a simple span but is greatly reduced if there are any intermediate supports.

1.7 The Beam-Spring Model:

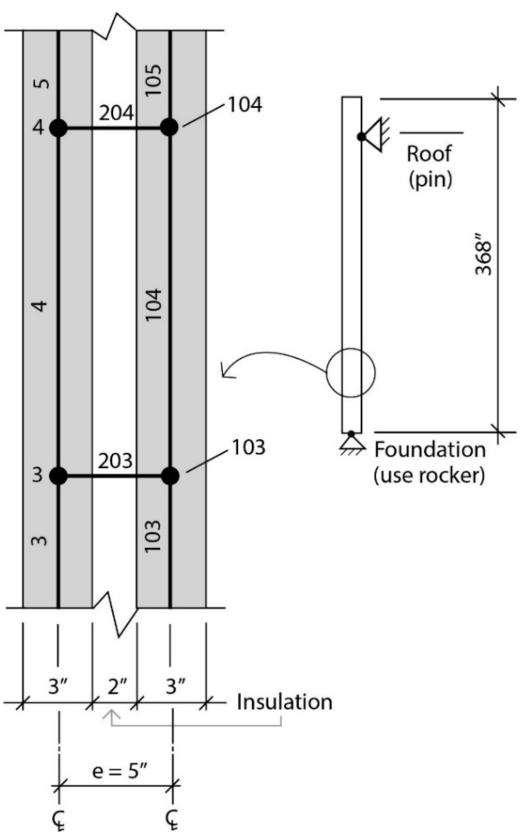
As noted previously, generic plane frame structural analysis software can be used to create a beam-spring model. Below is one approach. There are also other variations which can give similar results.

For this wall example, 23 wythe connector rows are assumed to be spread over the 368" wall height at 16" longitudinal and lateral spacing:



Wall in plan, 368" tall by 144" wide

1.7.1 Member and joint locations: For simplicity, a 16" wide strip will be analyzed, instead of the full 144" width. Members and nodes (joints) are input and numbered as so:

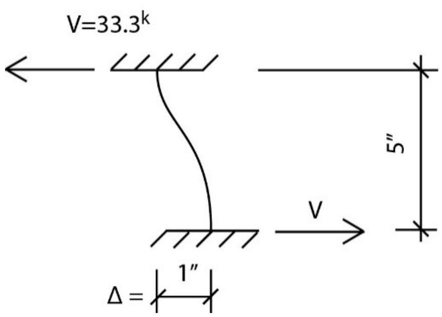


There are 3” thick wythes with 2” thick insulation. Center to center of the wythes is therefore 5”, where the joints will be placed, 16” on center, with concrete wythe and polymer connector members connected to them. The only joint releases will be at the foundation and roof supports. All the other joints can transfer moment as well as axial and shear forces. A rocker joint is used at the foundation, assuming both wythes are supported for gravity.

1.7.2 Member properties:

An area (A), moment of inertia (I) and modulus of elasticity (E_c) is required for each member. Each concrete wythe in our example has a 16” by 3” cross-section, therefore area (A) is 48 in^2 and I = 36 in^4. The concrete modulus of elasticity is modified by β_{dns}, depending on the load type, per ACI 318-19 6.6.4.4.4. The β_{dns} factor for wind load is zero, since it is a transitory load. β_{dns} for dead load is 1.0, since it is a sustained load. For example, assuming 6000 psi concrete and primarily wind load, a combined β_{dns} can be estimated as 0.1. Therefore E_c = 4415/(1 + β_{dns}) = 4415/1.1 = 4014 ksi. E_c for PΔ deflection is further modified by a stiffness reduction factor, ϕ_k. Per ACI 318-19 R6.7.1.1, ϕ_k may be taken as 0.875. Net E_c for PΔ analysis becomes 4014*0.875 = 3512 ksi.

For wythe connectors, the area can be any reasonable value, since connector tension is not relevant to this analysis, only shear. In this case, we’ll use A = 1 in^2. For polymer-based connectors, a modulus of elasticity of 4350 ksi is reasonable. Again, the value used for E itself is not important, as EI is the critical value. The moment of inertia will be tuned to provide a flexural stiffness equal to the tested double shear stiffness, Ke. Ke equals the force at the connector elastic limit, Fe, divided by the shear slip at that same limit, Δe. 2.0k / 0.06” = 33.3 k/in. The moment of inertia for a 1” shear slip under a 33.3k force can be found from the following formula:



$$I = \frac{V\ell^3}{12E\Delta}$$

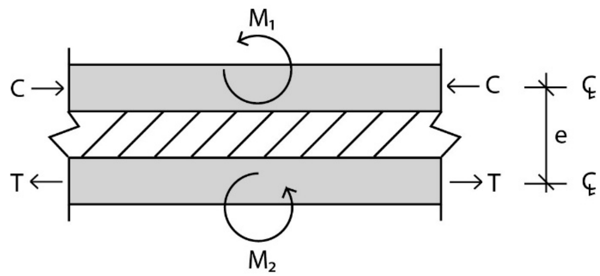
$$I = \frac{33.3^k(5'')^3}{12(4350\text{ksi})1''} = 0.08\text{in}^4$$

Moment of inertia for each wythe connector needs to be 0.08 in^4 to match the connector double-shear stiffness, Ke.

1.7.3 Loadings:

Standard loads and combinations such as Dead, Live, Roof, Seismic and Wind can then be applied to members and/or joints as appropriate. Special load types will be discussed in Part II. This includes P-Δ moment, differential temperature strain, unequal prestress, eccentric axial loads and local effects.

The analysis will predict the axial tension or compression in each concrete wythe, as well as the moment taken by each wythe and the connector slip at each row along the member length. The member capacity will be a combination of the tension-compression moment couple between the wythes and moments taken by each wythe individually. The higher the moment couple is in relation to the wythe moments, the higher the degree of composite action.



*Total moment = T * e + M_1 + M_2, assuming T = C*

Maximum slip should be checked to ensure that the connectors are still in the elastic range. Otherwise, an inelastic analysis would be required. If wythe moments exceed the wythe cracking stress, then the analysis should add cracked section properties. These topics are covered in Part II.

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