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Inter-wythe Slip Design Criteria for Non-Composite Insulated Walls

Abdelrahman Awawdeh
University of Nebraska - Lincoln, aawawdeh2@huskers.unl.edu

Fray F. Pozo-Lora
University of Nebraska - Lincoln, fpozo-lora2@unl.edu

Marc Maguire
University of Nebraska - Lincoln, mmaguire8@unl.edu

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Abdelrahman Awawdeh, MS, Durham School of Architectural Engineering & Construction
University of Nebraska – Lincoln, Omaha, NE

Fray F. Pozo-Lora, Ph.D., Civil & Environmental Engineering School,
Pontificia Universidad Católica Madre y Maestra, Santo Domingo, Dominican Republic

Marc Maguire, Ph.D., Durham School of Architectural Engineering & Construction
University of Nebraska – Lincoln, Omaha, NE

ABSTRACT

Non-composite insulated wall connector design is governed by ICC-ES AC320. This standard works entirely in the loading domain, asking the engineer to prevent connector failure due to tension and shear loading. In this paper, the authors discuss additional criteria related to thermal loading and out-of-plane wind loading that create displacement demand in the non-composite connectors. Loads suitable for such analyses are not well defined. Loads are assumed and demonstrated herein and shown to cause significant displacement demand on connectors. Limited non-composite wythe connector testing is available, and some results are presented here. A comparison indicates that outright failure of non-composite connectors is unlikely for current designs, but fatigue due to thermal and wind loading may be of important consideration, in particular for tall panels.

Keywords: Design Criteria, Non-Composite, Insulated Wall Panels, Thermal Loading, Out-of-Plane Loading, Inter-Wythe Slip.

INTRODUCTION

Sandwich wall panels (SWP) are lately gaining popularity but have been in use for many decades. SWPs typically consist of two layers of concrete, usually termed wythes, separated by a layer of insulation and tied together with connecting elements, often called wythe connectors[1]. Engineers often classify SWP in three categories: fully composite, non-composite, and partially composite panels[2]. The only difference between the three categories is the shear transfer capabilities of the wythe connectors. Strong and stiff shear connectors can be used to foster more composite behavior, whereas weaker non-composite wythe connectors are intended not to transfer shear between the wythes.

The design of partially and fully composite insulated walls has been the subject of much contemporary research[3–8]. The interaction of the layers with composite SWP is complex, but the out-of-plane (OOP) mechanics are generally understood[5,9]. Further, it is thought that non-composite insulated walls are fairly well understood, though there are no peer reviewed published papers or design codes outlining their mechanics outside of ICC-ES AC320[10] – a semi-codified document that will be discussed in subsequent sections. Non-composite panels follow the same mechanics as partially composite insulated walls, but the connectors cannot transfer as much shear. Most – if not all – engineers simplify the non-composite design to a simple philosophy: the larger wythe carries the design loads, the thin exterior wythe carries little, and make sure the outside wythe does not come off. This philosophy hinges on one, often implicit, assumption: the connectors do not carry shear loading from OOP loads.

This paper intends to take a critical look at the way non-composite panels are designed and suggests that additional checks are needed, potentially indicating a height limit may be needed for given connectors and suggesting boundary conditions be controlled. Additionally, while this paper is not able to answer questions about this design, it will raise several questions that the engineering community should consider.

NON-COMPOSITE MECHANICS

The following sections form a discussion on the mechanics of non-composite SWP. Currently, AC 320 provides the only guidance outside of proprietary wythe connector suppliers. The AC320 design philosophy is simply to make sure the connectors can hold the dead load of the outer wythe and prevent delamination under the wind suction. The authors argue that the design philosophy should be that the connectors must handle all potential *movement* from environmental and mechanical loading in addition to these loads.

DEAD LOAD

According to AC320, non-composite SWPs are designed to carry the tension and shear loading from their tributary area of the outer wythe as it hangs off of the structural wythe during handling, in addition to service dead load and wind loading acting in tension as seen in Fig. 1 (more discussion on the wind in a later subsection). The equations for this are presented herein. AC320 also provides deformation limits for such (should not exceed 0.1 inches under gravity load):

$$\Delta g = \frac{Q_g * d_A^3}{12E_{AB} * I_A} \tag{1}$$

Where:

Δg = Displacement due to gravity load, inch or mm
Qg = Gravity load on the connector, typically the weight of the fascia layer of the tributary area for the connector

$$d_A = d_d + \frac{2h_v}{3}\left[1 - \frac{1}{1 + {h_v}/{d_d}}\right] \tag{2}$$

Where:

d_A = Connector bending length, a function of insulation thickness and embedment, inch or mm.
d_d = Insulation thickness, inch or mm.
h_v = Embedment length of the connector in the concrete, inch or mm.
E_{AB} = Flexural modulus of elasticity as determined in Section 4.1.3 of AC320 [10], psi or Pa.
I_A = Moment of Inertia of the connector, in^4 or mm^4.

Equations (1) and Equation (2), assume a specific type of connector (prismatic beam-like pin connector), but can be modified as needed for a connector of interest by looking up load on a load versus deflection curve, though such approaches are may not endorsed by ICC-ES. The loading is then checked to determine if it satisfies an interaction equation between tension and shear on the connector.

$$\left(\frac{P_s}{P_t}\right) + \left(\frac{V_s}{V_t}\right) \leq 1 \tag{3}$$

Where:

P_s = Applied service tension load.
P_t = Service tension load.
V_s = Applied service shear load.
V_t = Service shear load.

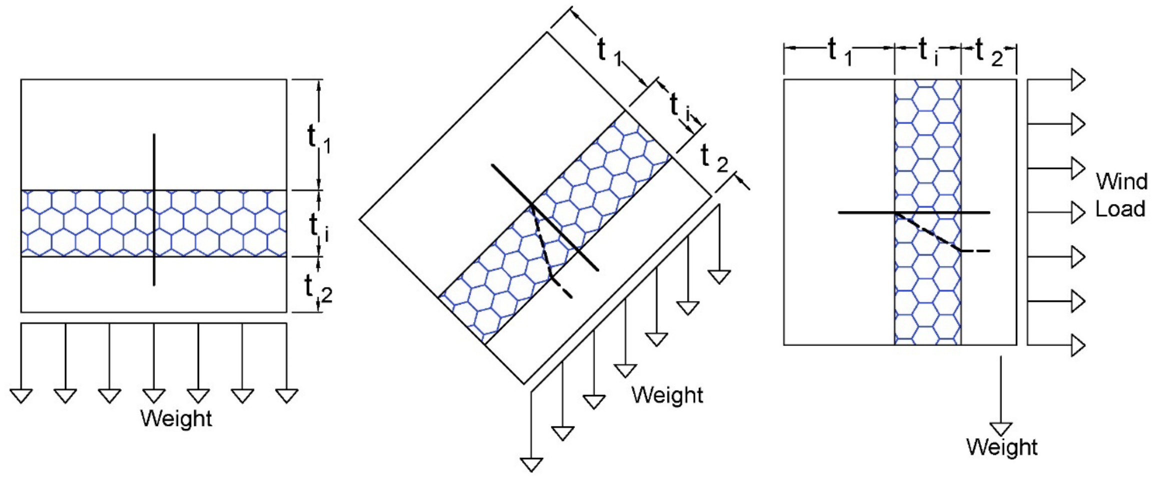


Fig. 1 Actions on wythe connectors discussed in AC320

THERMAL LOADING

Non-composite SWPs are usually assumed not to transfer shear load between the two wythes, which is convenient because resolving these actions can get complex. However, if one makes this assumption, the connectors must be able to handle the deformations associated with other actions, particularly thermal loading.

Because the interior wythe has heating, ventilation, and air conditioning (HVAC) and is insulated with a foam layer, the exterior wythe temperature fluctuates, creating a temperature gradient[11]. The gradient between these two wythes defines the deformation of the connectors. On the other hand, the weak and flexible connectors usually employed for non-composite SWP connectors are assumed to take little or no loading. Because of this assumption, these thermal deformations are almost completely unrestrained by the connectors.

If one continues with this assumption, that means the connectors must accommodate any movement caused by these temperature fluctuations. Ultimately, this deformation must be checked by the allowable deformation of the connector. Checking such local deformations of a given connector for a given load is straightforward. Thermal loading and its actions are demonstrated in Fig. 2 for a boundary condition such that the exterior wythe is floating and Fig. 3 such that the exterior wythe is grouted. Both boundary conditions exist in practice. In this case, the most critical connectors are those at the top and/or bottom.

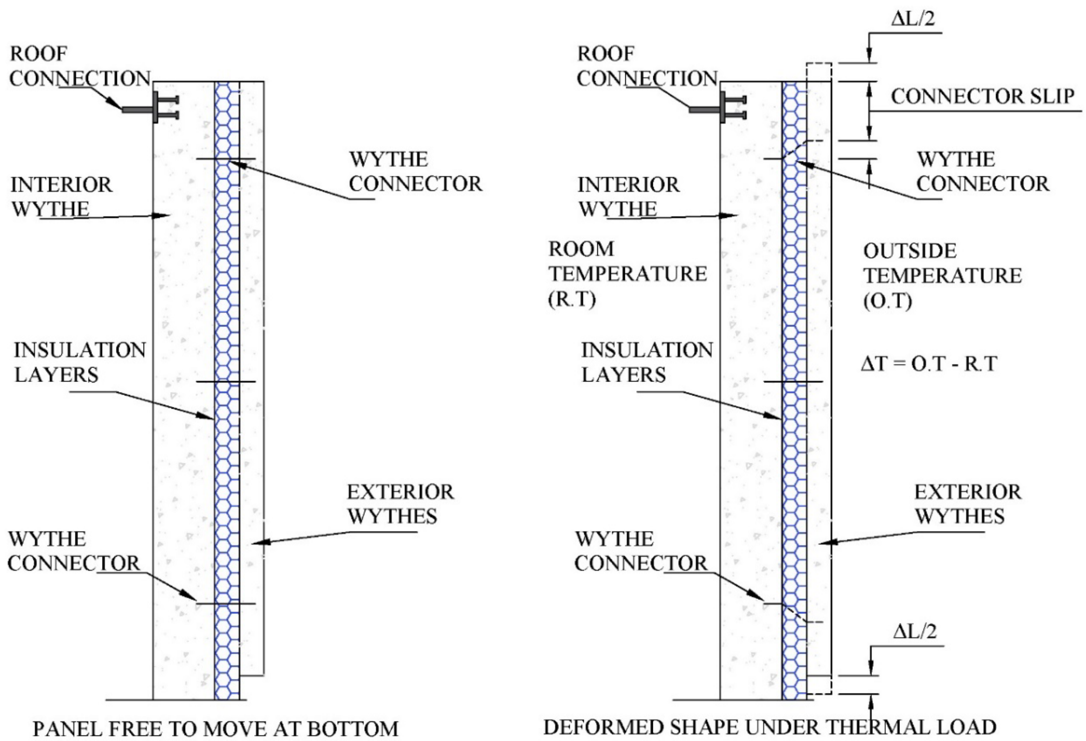


Fig. 2 Mechanics of SWP Under Thermal Load with Floating Outer Wythe

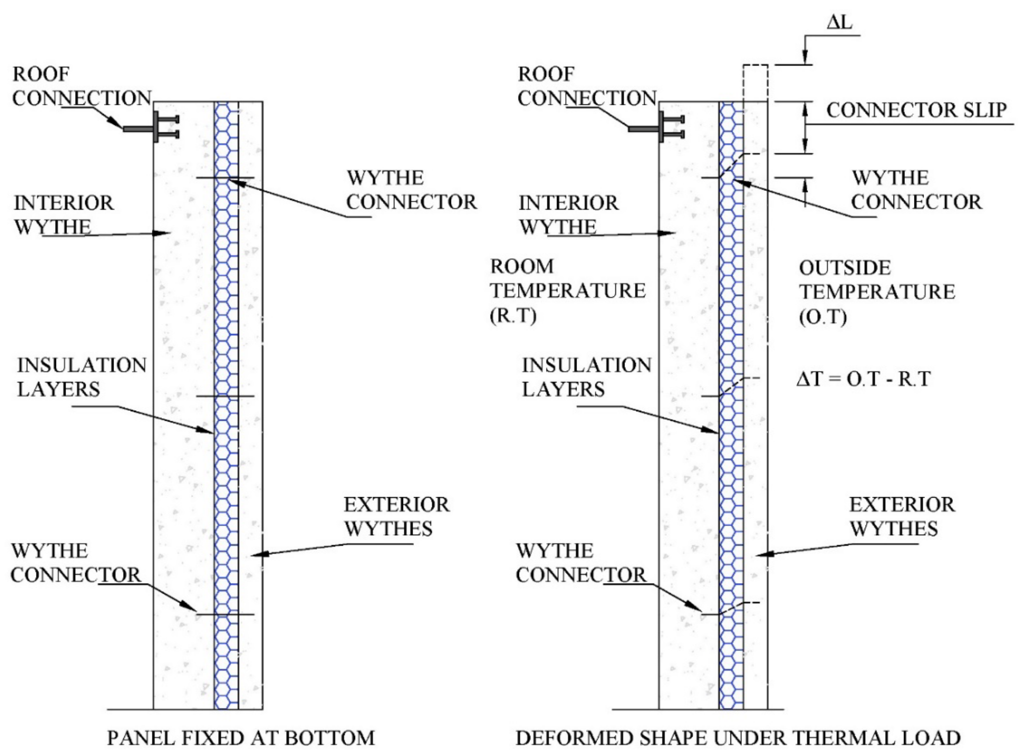


Fig. 3 Mechanics of SWP Under Thermal Load with Fixed Outer Wythe

To calculate the maximum thermal deformations, familiarity with the mechanics of Equation 1 is required. For panels with floating exterior wythes (i.e., ungrouted), the slip in the top and bottom connector locations is half of that calculated by Equation 4 such that the wythe expands about its centerline in both directions. Alternatively, if the bottom is fixed (i.e., grouted), the deformation is forced vertically. Comparing the two cases, it is clear that these thermal deformations are twice as severe in the latter case.

$$\Delta L = \alpha * L * \Delta T \tag{4}$$

Where:

α = Coefficient of thermal expansion
L = Length of panel
ΔT = Temperature change

OUT-OF-PLANE LOADING

To simplify the discussion on OOP loading, wind load will be discussed, because seismic loading likely comes with additional concerns, but similar action and kinematics, that will be considered outside the scope of this paper. Further, seismic loading is infrequent when compared to thermal and wind load frequency.

WIND LOADING

Wind loading is anecdoctally not thought to cause any shear load in the connectors; however, this is a false assumption. It is true that the action of wind suction will cause load axial load on a given connector based on its tributary area. However, under OOP wind, the structural wythe will deform in bending, and the exterior wythe (because the connectors keep it attached) must also remain at the same OOP deflection. If this is the case, then the exterior wythe will also have the same curvature and rotation as the structural wythe (this is also the basis for all composite beam theory since Newmark[12]). Provided the connectors can keep the layers in contact, thus meeting this assumption, the rotation of the two wythes will cause shear in the connectors at their connected faces and then actually rotate away from each other (i.e., slip). The mechanics of this deformation is the same as those used for partially composite panels but has been ignored. This local deformation is defined in Fig. 4.

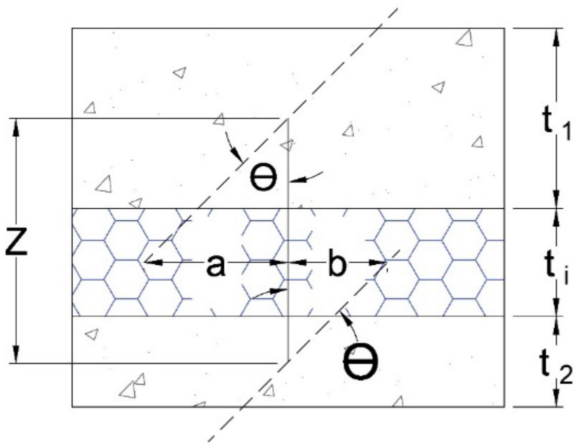


Fig. 4 Relationship Between Rotation and Connector Slip

Using the relationship in Fig. 4, the slip generated by wythe rotation can be calculated as shown in Equation 5 and Equation 6.

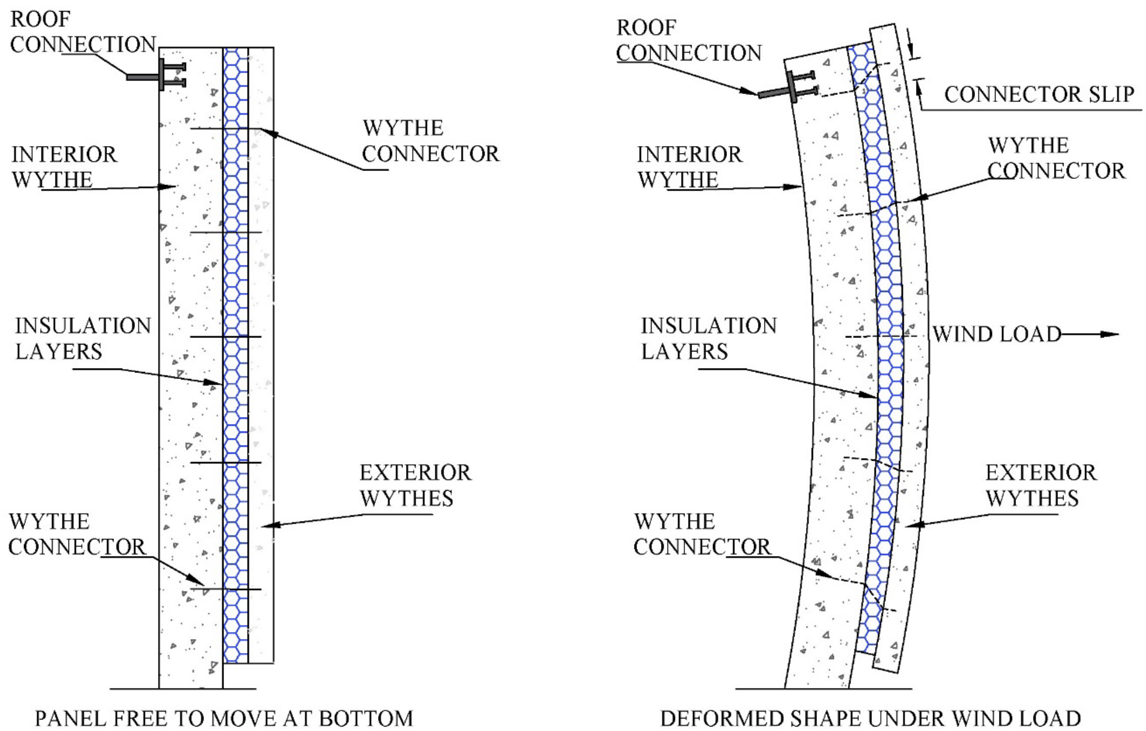
$$Slip = a + b = \theta * Z \tag{5}$$

$$Z = \frac{t_1}{2} + t_i + \frac{t_2}{2} \tag{6}$$

Where:

θ = Rotation at a connector location
t_1 = Thickness of interior wythe
t_2 = Thickness of exterior wythe
t_i = Thickness of insulation layer
Z = centroidal distance between concrete wythes

The global mechanics of non-composite panels under wind suction load are illustrated in Fig. 5 and 6 for floating and fixed base conditions, respectively. Clearly, because the slip follows the rotation of the structural wythe, the critically loaded connectors are the top and bottom connectors in the floating wythe situation and the top connector in the fixed base situation.



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173 Fig. 5 Mechanics of SWP Under Wind Load with Floating Outer Wythe

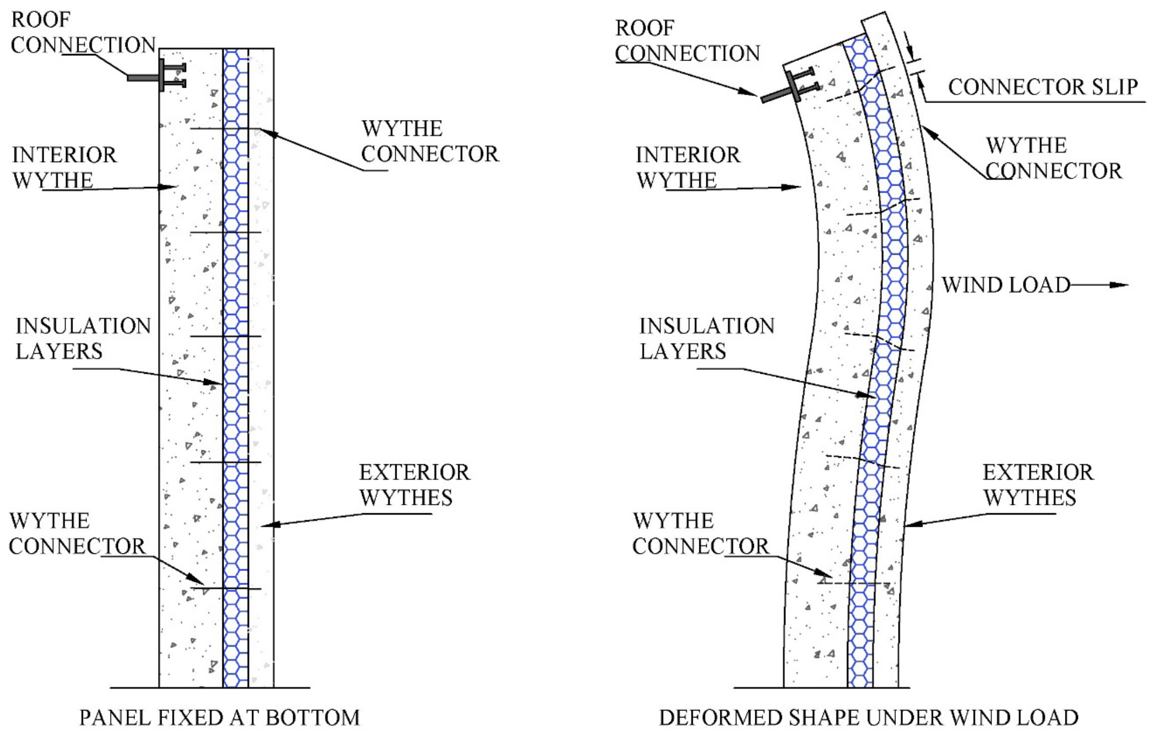

Fig. 6 Mechanics of SWP Under Wind Load with Fixed Outer Wythe

To calculate the slip at a connector location due to bending of the panel under uniform wind load, the rotation (θ) is calculated using Equation 7 for the floating wythe condition and using Equation 8 for the fixed-pin connection. Fig. 4 illustrates the relationship between the rotation and slip in the connector, and Equations 5 and 6 (fundamental beam mechanics) are used to calculate the connector slip using the calculated rotation.

$$\theta(x) = \frac{-w}{24EI}(L^3 - 6L * x^2 + 4x^3) \tag{7}$$

$$\theta(x) = \frac{-w * x}{48EI}(6L^2 - 15L * x + 8x^2) \tag{8}$$

Where:

θ = Rotation
w = Wind load
L = Length of panel
x = location along the panel
E = Modulus of elasticity
I = Moment of inertia of the *structural wythe*

One will notice that these are fundamental relationships familiar to all structural engineers, not complex sandwich theory[13,14]. The use of sandwich theory would produce similar answers but is not straightforward. Just like the discussion on thermal loading, because the connectors are assumed to be very flexible and weak, they cannot restrain this OOP bending. Therefore, the important consideration is that the connectors can handle the deformations imposed by this action.

LOADS

Even though the mechanics outlined above are easy to identify and implement, determining the loading that is appropriate and for which scenarios are not. The effect of diurnal and annual temperature swings on the mechanical performance of insulated wall panels is not known. The sun comes up daily, causing at least 18,250 temperature cycles of variable magnitude. The difference between the maximum and minimum temperatures in a year is very large. The magnitude of the thermal gradient is likely related to solar radiation, the thermal mass of the structure, ambient temperatures, and exterior convection.

The effects of wind are better understood. The design loading in ASCE 7-16 is a 700-year load, but there are equations to reduce this to a different return period, and the hazard tool provides several wind velocities for different return periods. Loads of consequence are likely infrequent during wind events but depending on the state of the panel (cracked, or uncracked), the slip generated could be significant.

TEMPERATURE LOADING

To illustrate temperature loads, three cities were selected to represent hot, cold, and moderate areas. These cities were Phoenix, Arizona; Grand Forks, North Dakota; and Omaha, Nebraska, to represent hot, cold, and moderate, respectively. Temperature data were obtained from the National Oceanic and Atmospheric Administration [15] for the past 50 years for each location. The maximum and minimum daily temperatures were plotted, from which the average daily temperature swing and yearly temperature swing were calculated, as shown in Fig. 7. These temperatures are air temperatures only and are used as surrogates for concrete temperature because such loads are unknown. These air temperatures are used only to provide an estimate and context for the forthcoming analyses. The temperature swing is reported, rather than the gradient (internal minus external temperature changes) because if the internal wythe is thermally stationary, the exterior wythe will expand and contract independently, and these swings will result in repeated deformations of the connectors.

The yearly probability for 20 deg. F and 40 deg. F daily changes are reported in Table 1(a) for context. The temperature swing histograms for all three cities can be seen in Fig. 8. Seasonal temperature swings are another infrequent, but certain, cyclic temperature load. For the purposes of this discussion, this is considered the difference between the maximum and minimum temperature values for one year. Such a temperature change will require the non-composite connectors to accommodate these wythe deformations. Table 2 shows the average seasonal temperature swing for the three cities. Fig. 7 illustrates an example of a daily and seasonal temperature swing for the city of Omaha for 2021 only.

Table 1 Average daily temperature swings

City	Average Temp. Swing (ΔT) F	Standard Deviation (σ) F	Probability of ΔT > 20 F	Probability of ΔT > 40 F
Phoenix	24.02	5.91	75.2%	0.34%
Grand Forks	20.06	9.13	50.3%	1.45%
Omaha	21.41	8.68	56.5%	1.62%

Table 2 Seasonal temperature swings

City	Average Yearly Temp. Swing (ΔT) F	Standard Deviation (σ) F
Phoenix	83.1	4.27
Grand Forks	120	6.26
Omaha	114	7.41

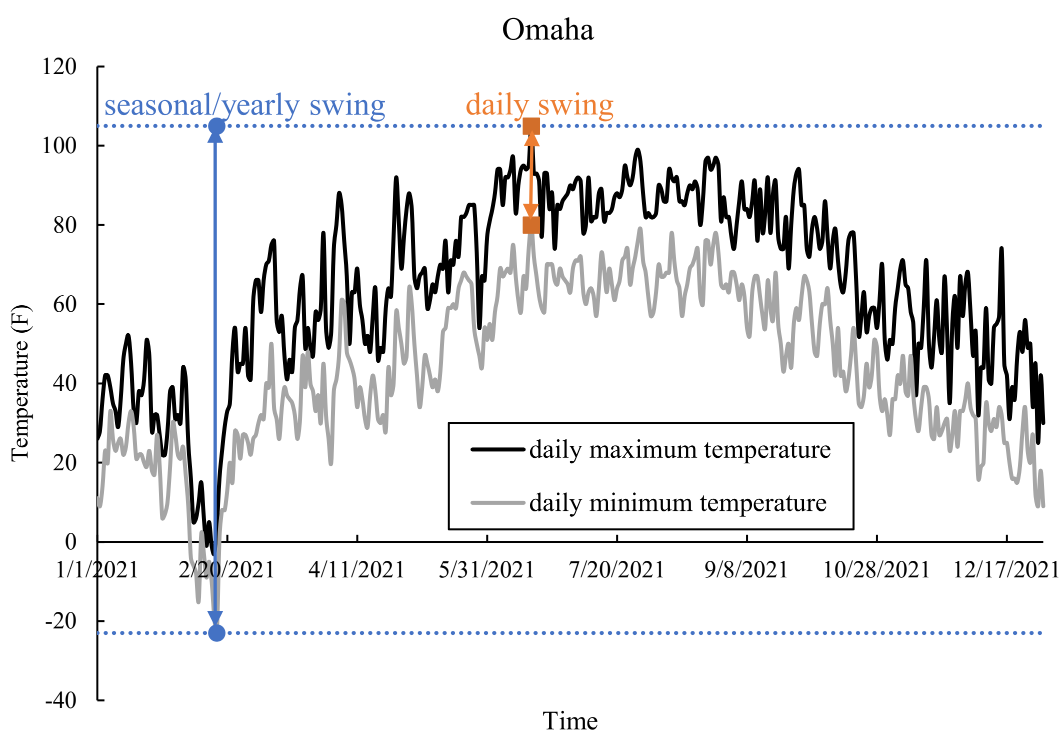

Fig. 7 Example of seasonal and daily swing for Omaha city

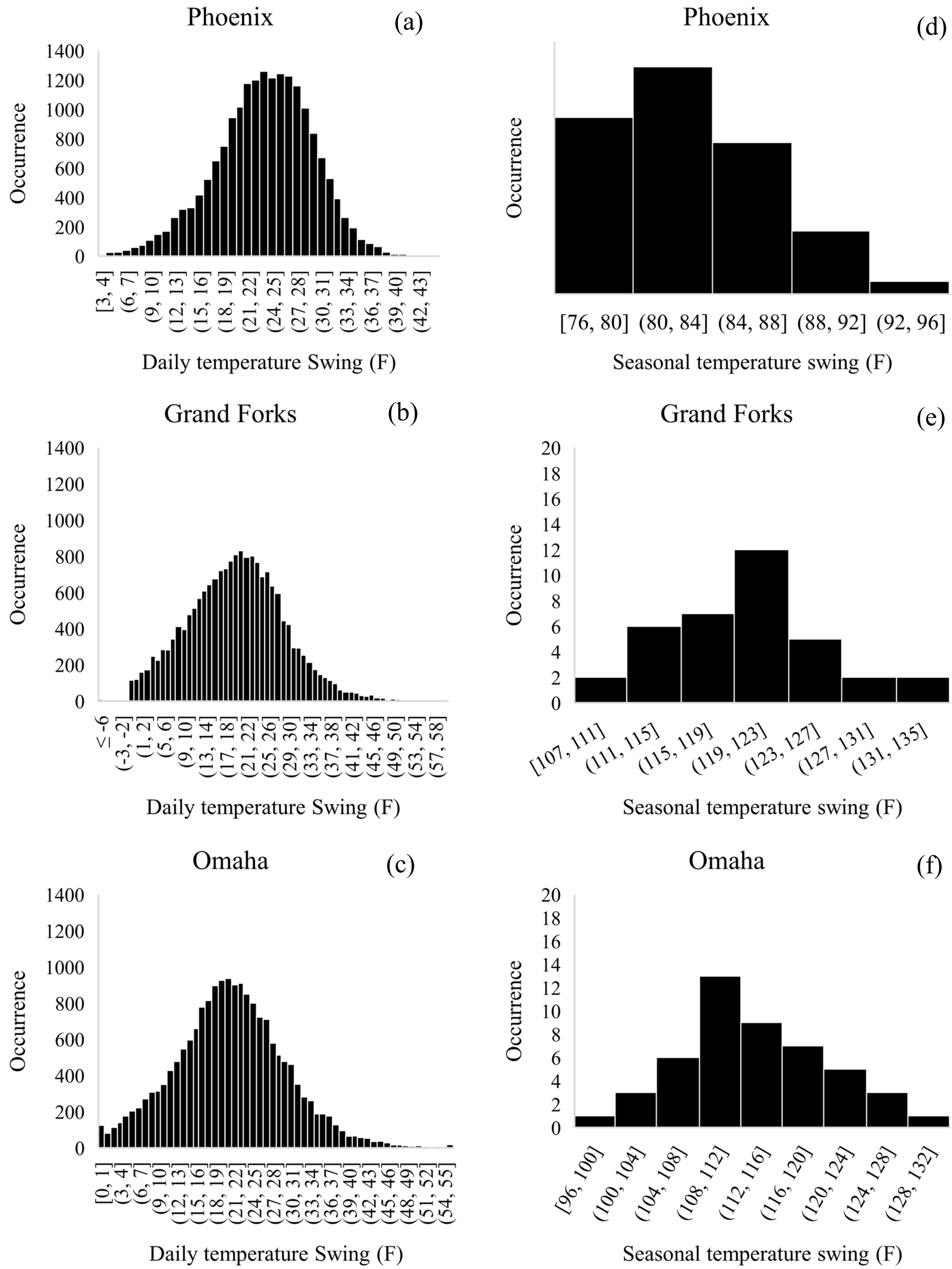

Fig. 8 Daily (a), (b), and (c) and seasonal (d), (e), and (f) temperature swing histograms for Phoenix, Grand Forks, and Omaha

These temperatures are ambient temperatures and only illustrative. Actual temperature gradients are unknown as there are no experimental programs that have collected such data for insulated walls. The AASHTO LRFD[16] bridge design specification provides a design model for the thermal gradient. The model divides the United States map into four zones of solar radiation, see Fig. 9 (a), and used two lines from the top of the girder and one straight line from the bottom, as seen in figure Fig. 9 (b), which has shown to be accurate for box girder bridges[17], but it is unlikely to be accurate for SWP. A similar approach could be undertaken for SWP, which would have implications for both non-composite and composite SWP designs.

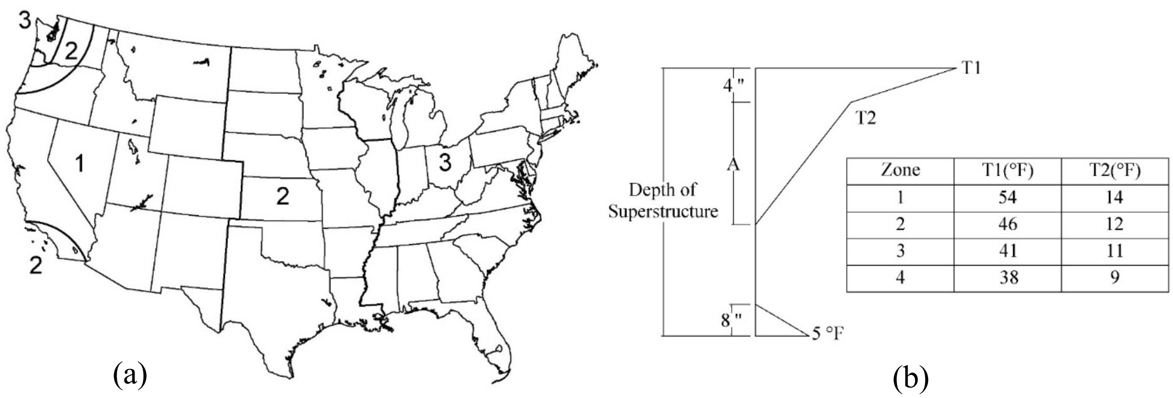

Fig. 9 (a) Solar radiation zones (b) updated vertical design gradient [16].

The gradients shown in the AASHTO LRFD code (Fig. 9 b) are higher than those implied by Fig. 8, likely because ambient temperature swings are not the same as thermal gradients and the concrete retains thermal energy due to its thermal mass as ambient temperatures change. Further, the temperature gradients in SWP are likely higher than those in a bridge because of the wall insulation and the HVAC regulating temperatures of the interior wythe.

WIND LOADING

In contrast to temperature loads, wind is clearly stated and quantified in the ASCE 7-16[18]. For the purposes of this analyses, the ASCE Hazard Tool was used to determine sub-design recurrence interval loading and example loads are presented in Table 3 for the three locations above. To calculate the pressure from wind speed, ASCE 7 provides the following equation 9 assuming the K coefficients are set to 1.0 for simplicity.

$$q_z = 0.00256 * V^2 \text{ (lb/ft}^2\text{)} \tag{9}$$

Where:

V = wind speed (mph)

The average 10-year and 50-year loads were 14.66 (lb/ft^2) and 21.24 (lb/ft^2), respectively and not appreciably different between locations studied in this manuscript.

Table 3 Wind speeds and loads for the three cities

Location	10-year wind speed (mph)	50-year wind speed (mph)	10-year wind load (lb/ft^2)	50-year wind load (lb/ft^2)
Omaha	77	90	15.18	20.74
Phoenix	75	96	14.40	23.59
Grand Forks	75	87	14.40	19.38
Average	76	91	14.66	21.24

WYTHE CONNECTORS

In the preceding section, the actions of different loads were described. AC320 describes means of testing non-composite wythe connectors (based on ASTM E488-09) but only requires the reporting of their static strength. Except for the dead load tension and shear and wind load tension, the slips generated from thermal and wind do not necessarily test the strength of the connector. Rather, the OOP bending and thermal mechanics require that a non-composite connector be flexible and accommodate deformations that are otherwise unrestrained. If such deformations exceed the deformation capability of a given connector, it will likely fail, and the exterior wythe delaminate.

Because of the lack of data on such connectors, a total of six double-shear specimens were tested to quantify the shear load vs displacement behavior for the non-composite connectors following the procedure outline in reference [19]. This testing was only performed to provide a point of reference for this paper, so the connector itself is only described as a common, proprietary, non-composite connector.

The specimens were double shear type specimens[20] with dimensions of 4 ft by 2 ft and consisted of three layers of concrete separated by two layers of foam. Both outer concrete wythes were 2 in. while the inner concrete wythe was 5 in. The insulated foam layers were composed of two 1-in. thick XPS foam sheets to ensure no insulation contribution to the shear strength and provide a debonded plane. Two different arrangements of connectors were used for each group of three double-shear specimens to determine if there were any differences in specimen configuration. The first group had four connectors connecting the exterior wythe to the central wythe wythes, a total of eight for the entire specimen, while the second group had six connectors connecting the exterior wythe to the central wythe, with a total of twelve connectors. The six-connector version is shown in Fig. 10; the four-connector version was identical, except the center row of connectors was removed.

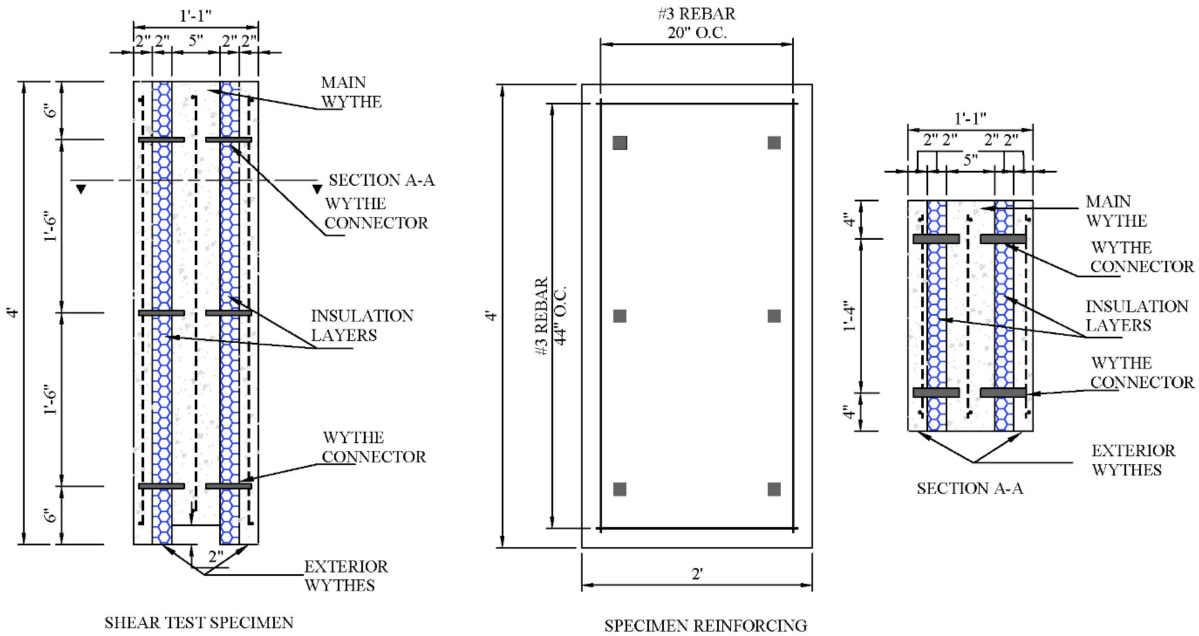

Fig. 10 Typical double shear specimen

A hydraulic ram was used to apply the load concentric to the inner concrete wythe. The load was measured using a 100-kip load cell. The two outer wythes were supported by the load frame while the inner wythe was free to move as shown in Fig. 11. The relative movement of the inner wythe was measured as the average of four total string potentiometers placed on the mid-height of the specimen. After sensor installation, the specimen was loaded until complete separation of the wythes.

Fig. 11 Double shear specimen test setup

Fig. 12 presents the load versus deformation curves for the two types of specimens. Specimens were labeled NC-[number of connectors per exterior wythe]-[iteration number]. The NC-4 series specimens and the NC-6 series specimens generally had the same near-linear behavior following what looked like an early vertical line attributed to breakaway friction of the insulation layers. Table 3 presents the normalized maximum strength and the slip at maximum strength for each of the six total tests. The strength of the connector was normalized by the average of all six tests to obscure which connector was tested, further, the load is superfluous to the discussion. These results indicate the two configurations were functionally identical. The average slip at maximum for these connectors was 0.544 in.

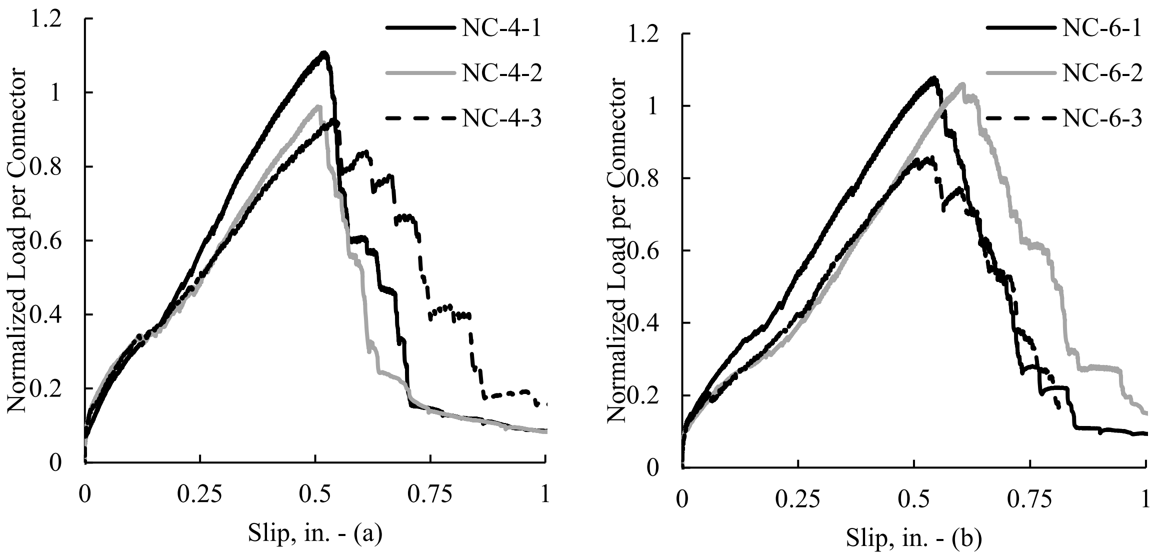

Fig. 12 Load vs slip results for (a) four connectors per wythe and (b) six connectors per wythe

Table 4 Double shear results

Specimen	Maximum Normalized Load per Connector	Slip at Max Strength (in.)
NC-4-1	1.11	0.521
NC-4-2	0.96	0.506
NC-4-3	0.93	0.546
NC-6-1	1.08	0.544
NC-6-2	1.06	0.606
NC-6-3	0.859	0.539
Average	**1.00**	**0.544**

In the context of the subsection preceding this one, the connectors need to accommodate movement of the wythes relative to each other as well as the in-place loads like the localized wind loading from their tributary area and the shear dead load. It seems that in the case of this connector, the displacements should not exceed 0.544 in. or the connector will fail.

Unfortunately, there is little information on cyclic testing of such connectors. It is unknown how repeated wind and thermal cycles will affect connector performance long term. AC320 has no guidance on this but does provide a cyclic testing regime geared toward seismic capacity as shown in Table 4, but not deformation capacity or hysteretic behavior. In reality the total number of cycles due to wind and thermal is not well understood, but based on the histograms of Fig. 8, the loads and number of cycles will not be reflected by the seismic cycles in Table 4.

Table 5 AC320 Shear cyclic load regime

Load Level	Number of cycles
$\pm V_s$	10
$\pm V_i$	30
$\pm V_m$	100

Where: Vi = A load midway between Vs and Vm, Vm = One-fourth the average ultimate shear load, Vref, in concrete of the tested strength, Vs = The maximum shear test load.

DISCUSSION

The above discussions about loading and slip calculations have been separated into this section, but the intent is to combine this information to determine if, with the best available knowledge, there should be changes to the way precast engineers think about non-composite SWP connector design.

Calculating the slip due to thermal loads requires an estimate of wythe temperatures, panel length, and coefficient of thermal expansion of the concrete (CTE). The CTE is estimated between 4.1 and 7.3 microstrain/degF[21] and for the analyses here is taken as 5 microstrain/degF. In lieu of better data, thermal swings of 20°F and 40°F for the floating and the fixed exterior wythe conditions are plotted for different panel heights up to 60 ft in. The best practice of keeping the exterior wythe floating keeps these low thermal swings minimal, but such slips are significant.

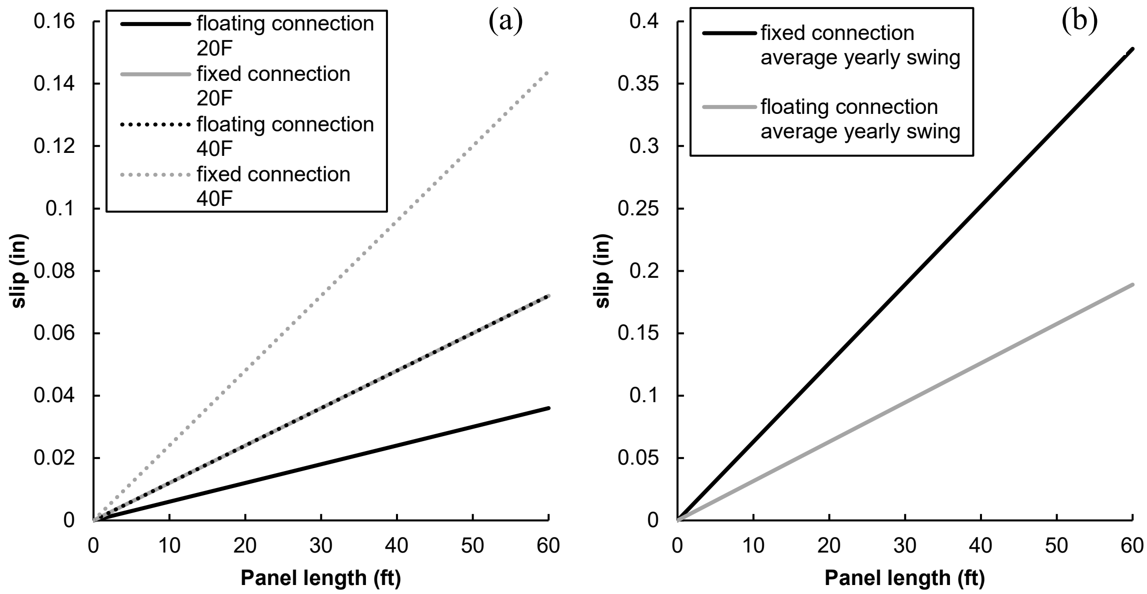

Fig. 13 (a) Slip vs panel length for floating and fixed connection for 20°F and 40°F (a) Slip vs panel length for floating and fixed connection for average yearly swing of three cities

Considering the displacement capabilities outlined in Table 3, such loads may approach displacements that will cause significant damage to the connector reducing its capacity over time. It seems that the temperature changes on the order of daily swings may only pose issues for larger panel heights and connectors that may not have the ductility of those tested here. However, the yearly temperature swings illustrated in Fig. 13b are very large and could limit the use of non-composite connectors at taller heights. At a minimum, such displacements would likely cause some damage, lowering the connector tensile and shear capacity. As contemporary panels continue to increase in height, this issue may well become exacerbated as the temperature slip is linearly related to the panel height.

Considering the mechanics under wind load described above, the slip for SWP varying in length from 10 ft to 60 ft was calculated for different structural wythe and insulation thicknesses. The calculation was done for an uncracked panel as well as a cracked panel (using $I_{cr} = 0.25*I_g$ per ACI 318[22] Section 6.6.3.1.1 for simplicity, though this could overestimate I_{cr} in a slender panel). Loads applied were 14.7 psf (10-year wind) and 21.2 psf (50-year), which a panel would be expected to resist at least five times and one time during its anticipated lifespan. The nomenclature x-y-z was used to represent the thickness of the interior wythe, insulation, and exterior wythe for x, y, and z, respectively. In Fig. 14, the slip was calculated for both floating and fixed end conditions with an insulation thickness of 2 inches and an interior wythe thickness of 5, 6, 7 and 8 inches for both cracked and uncracked panels. Because the slip is related to the thickness of the panel, as well as the rotation (a function of the height cubed) very large slips are shown.

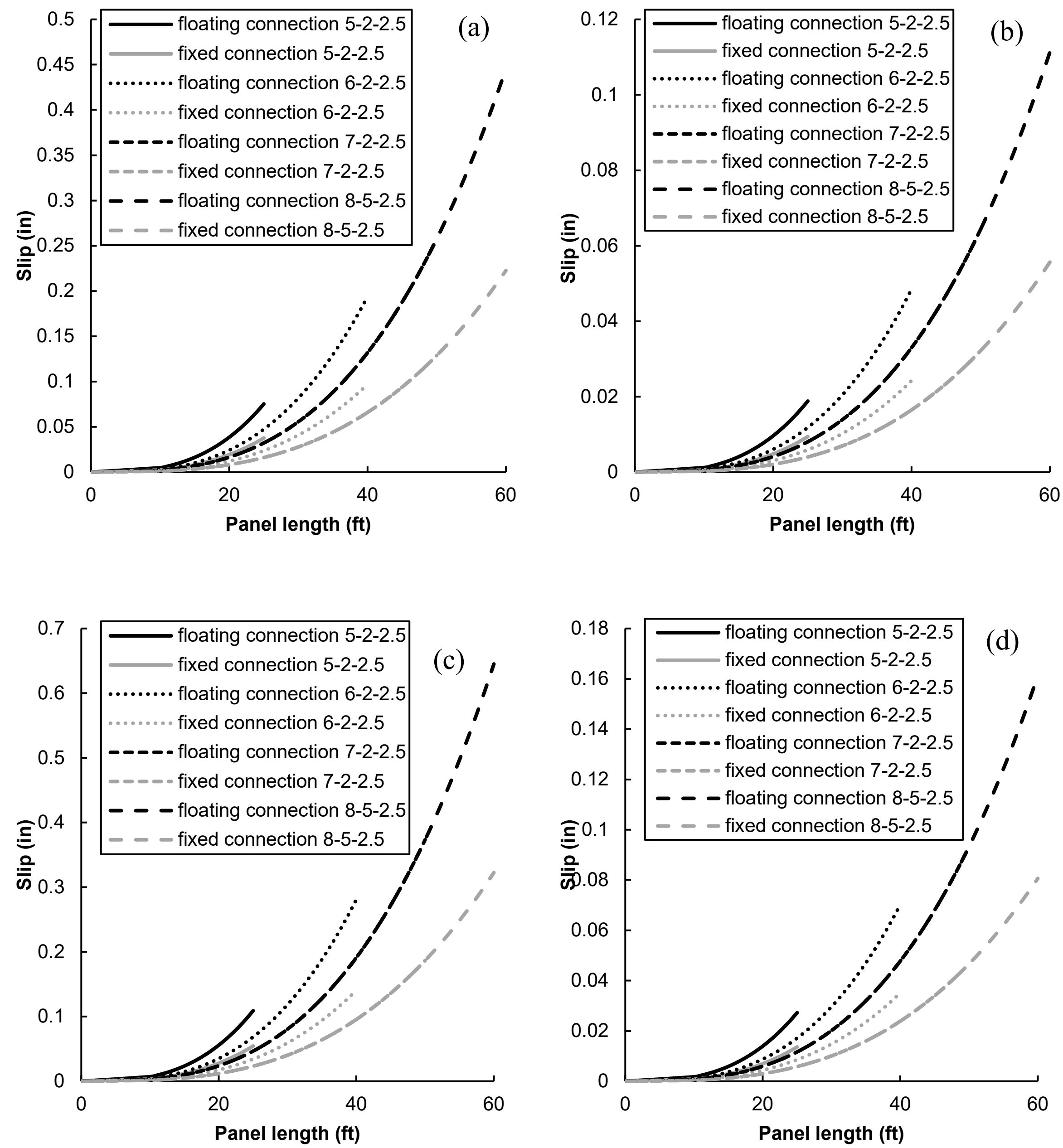

Fig. 14 Slip vs panel length for floating and fixed connection with 2-inch insulation under wind load for (a) cracked panel and 14.66 (lb/ft^2) load (b) uncracked panel and 14.66 (lb/ft^2) load (c) cracked panel and 21.24 (lb/ft^2) load (d) uncracked panel and 21.24 (lb/ft^2) load

While the wind loading at the above rate is not a very frequent event, this illustrates the importance understanding the slip capabilities of the connectors to be designed. If the connector slip capacity is 0.5 in. (per the tests herein) a 10-year wind event may well result in ruptured connectors and delamination in cracked panels. For uncracked panels, this slip at these loads is much smaller and may only contribute to possible strength reduction of the connector discussed in the thermal section. Such fatigue information is not widely available. For the 50-year wind events, induced slips are also very large, and assuming a cracked panel, has the potential to fail the connectors for tall panels. Further, if non-composite panel connectors are expected to remain intact under failure conditions like ultimate loads, where the fully cracked section properties are expected, it is unlikely that a connector with such displacement capacity can remain intact and are likely to fail prior to the nominal capacity of the member.

The purpose of the above discussion is to talk about the fundamental kinematics of the panels and get engineers to think about potential limits on connectors, particularly in taller panels. There is limited information available on displacement capacity and fatigue performance of in-service non-composite connectors, but the very limited testing program here indicates that issues are unlikely for the panel heights considered. Regardless, the mechanics discussed herein should become part of a formalized design document so as to determine if issues may arise in future designs as panel heights continue to go up.

CONCLUSIONS

This paper has discussed the design of connectors for non-composite panels. The ICC-ES AC320 document outlines the testing and design of such connectors, but works only in the load domain, rather than the displacement domain. Displacement demand from thermal and wind loading was discussed and the mechanics for calculating connector displacement demand was presented. Loads were introduced and discussed, in particular their shortcomings and future needs. Using the mechanics and assumed loads, connector displacement demand was calculated and presented. The results indicated large connector slip displacements may be large enough to cause fatigue and performance drop. However, there is such limited information on connector data, it is difficult to quantify these effects. This paper has generated more questions than answers, but the following conclusions can be made regarding the discussion above:

- The mechanics of thermal and OOP load-induced connector slip are relatively easy to implement and familiar to most engineers.
- There is limited guidance on thermal loading for SWPs, and this paper calls for additional study of thermal gradient loading in SWPs. This has implications in both non-composite and composite SWP.
- Connector slip from thermal and OOP loading is highly dependent on the length of the panel, and connectors in taller panels may experience issues with slip demand.
- Slip demand from thermal and OOP wind loading may cause fatigue in non-composite connectors in SWP, reducing their capacity, particularly in taller panels. There is no available data on this currently.

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