

Insulated Wall Panels – Interface Shear Transfer

by

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A typical insulated wall panel consists of insulation sandwiched between two concrete wythes of equal or unequal thickness that are connected by various types of shear connectors. These connectors pass through the insulation that is sandwiched between the two concrete wythes. While such panels are highly developed, the traditional methods for the design of interface shear force ignore the influence of actual panel stresses under cracking loads.

For the given loads, these connectors undergo shear deformation, the extent of which determines the degree of composite action. Therefore, it is important to understand the shear transfer mechanism through these connectors as it relates to the composite action of these panels.

The purpose of this paper is to present a rational method for the determination of the interface shear force and the design of shear connectors. The scope of this paper is limited to

welded wire truss connectors.

A typical cross section and an elevation of an insulated wall panel are shown in Fig. 1.

The classical method to determine the interface shear force for composite action is based on the familiar expression:

$$F_h = \frac{V_u Q}{Ib}$$

where

F_h = interface shear force

V_u = applied ultimate shear

Q = static moment of compression area about centroid of whole section

I = moment of inertia

b = width of shear plane

In general, this method underestimates the magnitude of the interface shear force because the computed flexural moment of a typical wall panel due to external loads is usually less than the moment required to crack the panel. For ductility reasons, most building codes require that the flexural

strength of a panel be greater than the cracking strength.

Another method to determine interface shear force is outlined in the PCI Design Handbook (Fourth Edition). This method is based on the assumption that the interface shear force is taken as the lesser of the total compressive force capacity of the concrete in the compression zone or the total tensile force.

$F_h = C$ when $C < T$; $F_h = T$ when $T < C$, where $C = 0.85f'_c A_c$ and $T = A_{ps}f_{ps}$, in which:

C = concrete capacity of section in compression zone

T = total tensile force in steel

f'_c = compressive strength of concrete

A_{ps} = total area of prestressing steel

f_{ps} = stress in prestressing steel at nominal strength of member

This method of computation may overestimate the interface shear force, particularly for short panels. For production reasons, it is standard practice to have the same prestress level for as

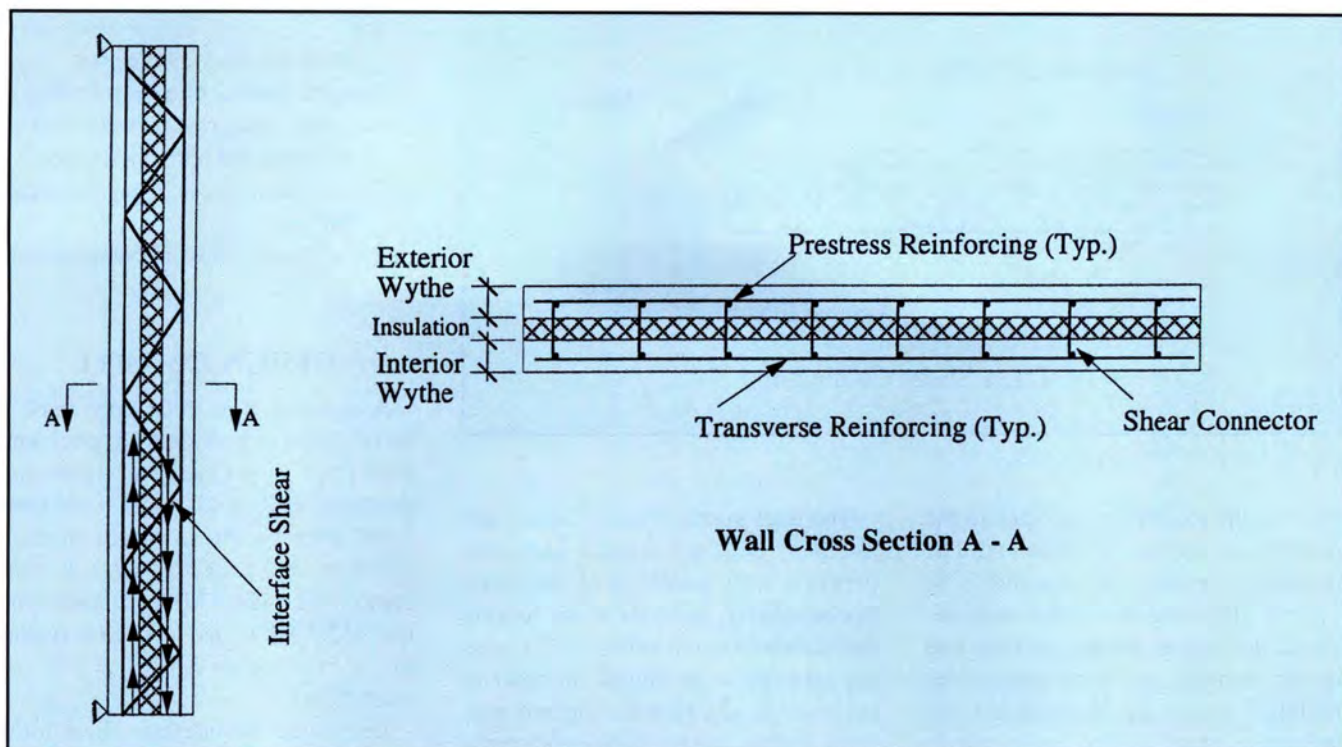


Fig. 1. Typical cross section and elevation of insulated wall panel.

many different design spans as possible.

The following alternate method is presented to determine the interface shear force in a typical insulated wall panel. In the author's opinion, the procedure is more realistic, as explained below:

1. The interface shear force is a function of the difference between the average compressive stress f_m at the critical section and the average compressive stress f_e at a section located at

the end of the prestressing transfer zone. Because most panels have a low steel ratio, these stresses should correspond to the cracking load as illustrated in Fig. 2.

2. The interface shear force is also a function of the location of the critical section and the prestress transfer zone from the end of the panel (*all* ratio) for various design spans. Based on this reasoning, the interface shear force can be defined as follows:

$$F_h = bt(f_m - f_e)$$

where

F_h = interface shear force

b = width of compression wythe

t = thickness of compression wythe

f_m = average stress level of compression wythe at critical section

f_e = average stress level in compression wythe at end of prestress transfer zone

The cracking load is defined as the flexure load that will produce a tensile

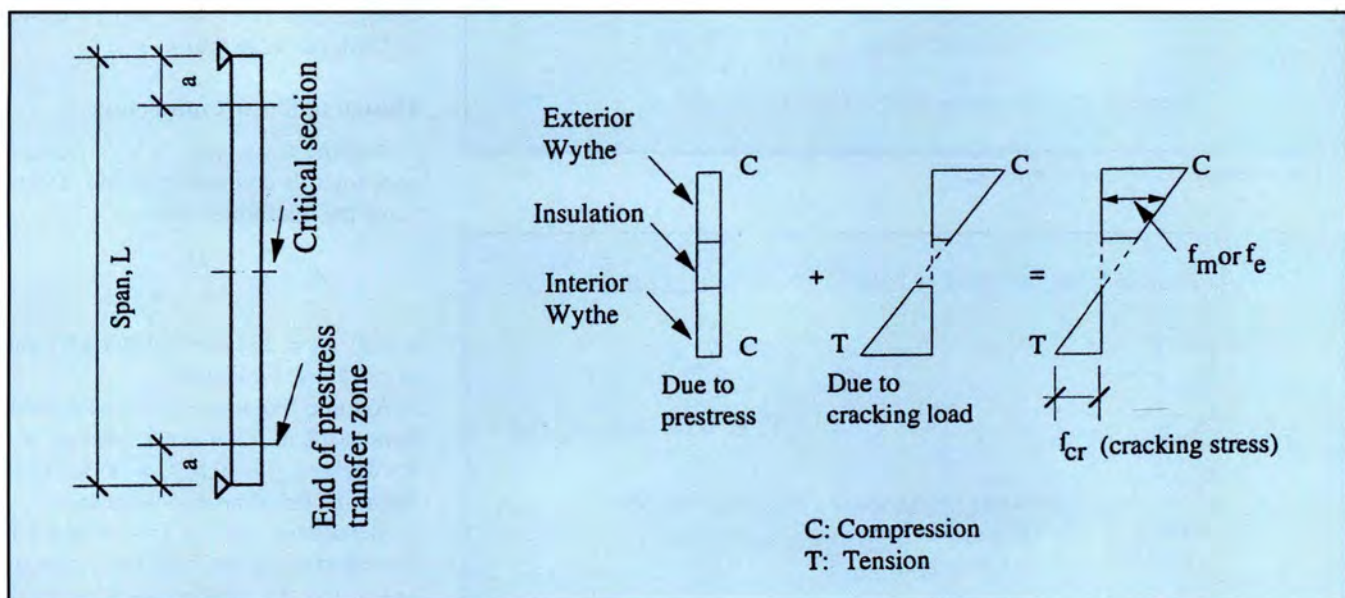


Fig. 2. Average compressive stress under cracking load.

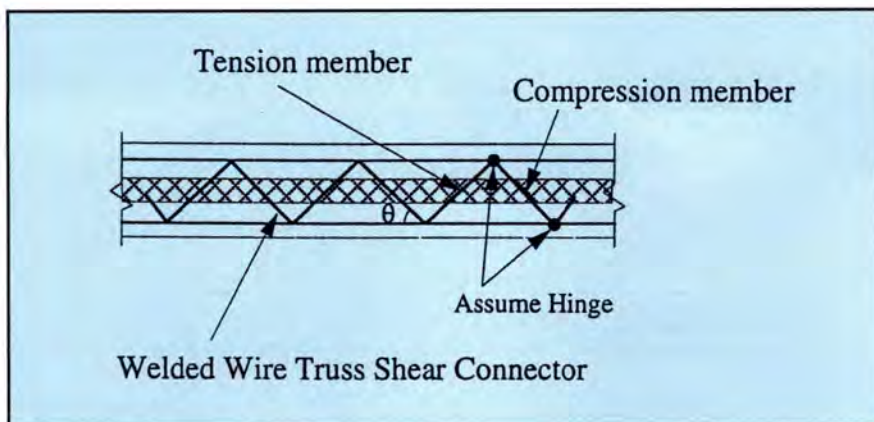


Fig. 3. Typical welded wire truss.

stress in the tension wythe equal to the modulus of rupture of concrete. The modulus of rupture is assumed to be $7.5\sqrt{f'_c}$. Because this force acts between the critical section and the end of the member, the total force to be resisted along the shear plane for the entire panel can be expressed as $F_h = 2bt(f_m - f_e)$.

WELDED WIRE TRUSS SHEAR CONNECTORS

A typical welded wire type shear connector is shown in Fig. 3.

The total interface shear force is resisted by both the tension and compression web members of the truss. For simplicity, the ends of the tension and compression members of the truss are assumed to be hinged. In order to prevent any slip between the two concrete wythes, the buckling characteristics of the compression web members must be considered. The thickness of the insulation will thus have an appreciable effect on the shear capacity of the truss. The interface shear force can be written as:

$$F_h = A_s N \cos \theta (f_{st} + f_{sc})/2$$

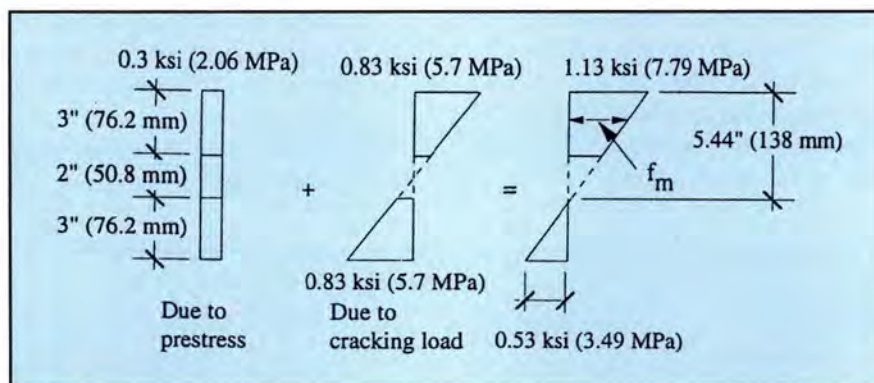


Fig. 4. Average compressive stress f_m .

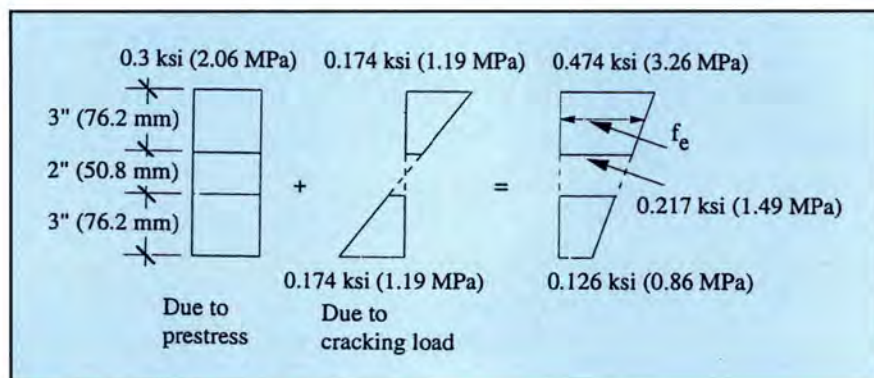


Fig. 5. Average compressive stress f_e .

where

A_s = area of truss web member

N = total number of truss members

θ = angle in degrees between web member and horizontal plane

f_{st} = allowable stress in tension member

f_{sc} = allowable stress in compression member

DESIGN EXAMPLE

Assume an 8 in. (203 mm) thick x 96 in. (2438 mm) wide wall panel with a 30 ft (9.14 m) span, an insulation thickness of 2 in. (51 mm), a prestress level after prestress losses in each wythe of 300 psi (207 MPa), f'_c (concrete) = 5 ksi (34.6 MPa), f_y (steel) = 60 ksi (413.7 MPa), and a concrete modulus of rupture of $7.5\sqrt{f'_c} = 530$ psi (3.65 MPa).

Determine the interface shear force and the total corresponding area of shear steel using welded wire truss connectors and "C" type connectors.

The solution is as follows:

1. The interface shear force is:

$F_h = 2bt(f_m - f_e)$. The average compressive stress at the critical section and at the end of the transfer zone is shown in Figs. 4 and 5.

2. At the critical section:

$f_m = (1.13 + 0.507)/2 = 0.818$ ksi (5.64 MPa). At the end of the prestress transfer zone, assume a transfer length for $3/8$ in. (9.5 mm) diameter strands of 20 in. (505 mm), $f_e = (0.474 + 0.217)/2 = 0.346$ ksi (2.38 MPa), $F_h = 2bt(f_m - f_e) = (2)(96)(3)(0.818 - 0.346) = 271$ kips (1205 kN).

Design of Shear Connectors

Assume truss type "VVV" connectors for full composite action. Determine the reinforcing area:

$$A_s = \frac{2F_h}{N \cos \theta (f_{st} + f_{sc})}$$

where $F_h = 271$ kips (1205 kN) and $N = 300$ truss members.

Assume five trusses with web members at 12 in. (304 mm) spacing, $\theta = 45$ degrees, $f_{st} = 0.6$, $f_y = 36$ ksi (248 MPa), f_{sc} for compression member:

Slenderness ratio = kl/r , where k is the coefficient for buckling strength (= 1), l is the unsupported length [= 2.82 in. (71.7 mm)], and r is the ra-

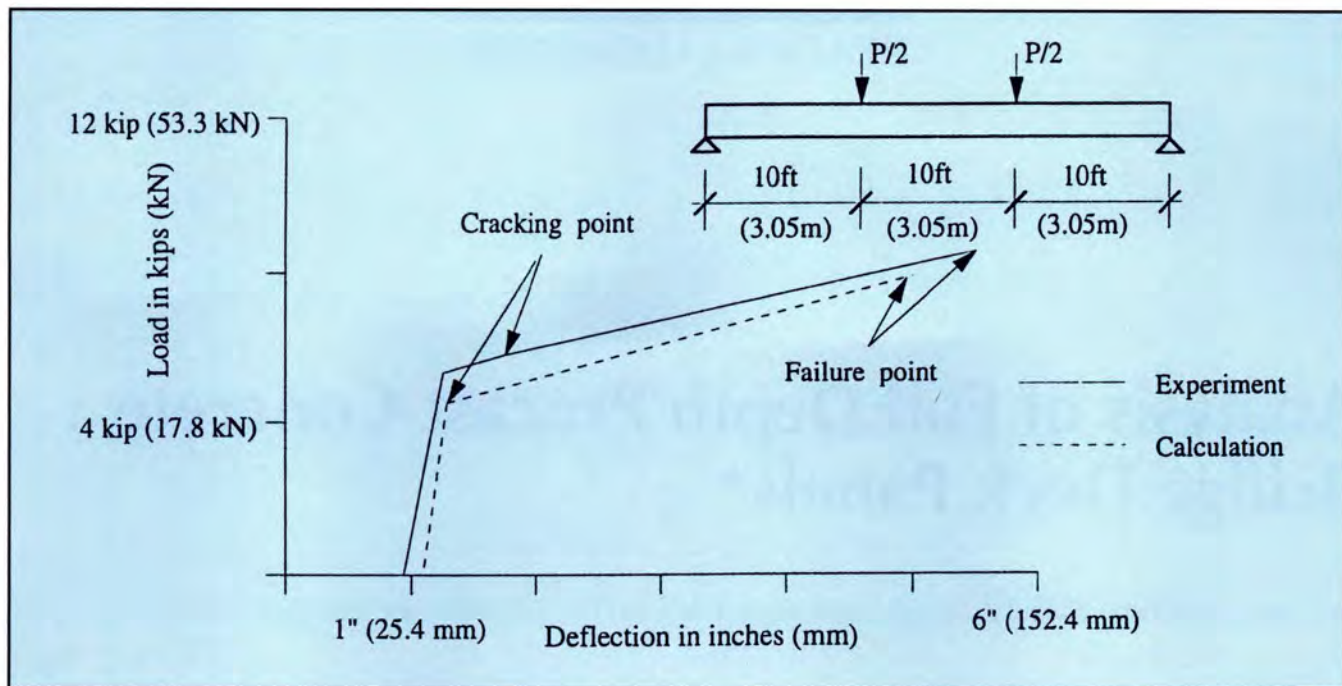


Fig. 6. Predicted and test load-deflection curves of panel.

dius of gyration ($= 0.25d$), where d is the diameter.

$Kl/r = 45.1$ for wire diameter of say 0.25 in. (6.36 mm), f_{sc} for slenderness ratio of $45.1 = 29.3$ ksi (201.81 MPa).

Therefore:

$A_s = (2)(271)/(300)(0.707)(36 + 29.3) = 0.04 \text{ in.}^2$ (25.8 mm²). A_s provided is 0.049 in.² (31.61 mm²) for a 0.25 in. (6.36 mm) diameter compression member. Hence, $0.049 \text{ in.}^2 > 0.04 \text{ in.}^2$ (ok).

Therefore, use five trusses with $1/4$ in. (6.36 mm) diameter web members spaced at 12 in. (304 mm).

EVALUATION OF TEST PANEL

To verify the analysis, a test panel was examined. The panel was 8 in. (203 mm) thick x 96 in. (2438 mm) wide with a bottom wythe of 3 in. (76.2 mm), 2.5 in. (63.5 mm) of insu-

lation and a 2.5 in. (63.5 mm) thick top wythe. The panel was 31.75 ft (9.69 m) long with a design span of 30 ft (9.14 m).

The panel had $14 - 3/8$ in. (9.5 mm) diameter low relaxation strands and was designed as 100 percent composite using welded wire trusses per the design example. Based on a prestress level of 398 psi (2.74 MPa) after prestress losses, the predicted and test load-deflection curve results are plotted in Fig. 6.

CONCLUSIONS

1. Based on very limited testing, it appears that the horizontal reinforcement based on the average stress at the critical section and at the end of the prestress zone is close to being realistic for achieving 100 percent composite action using welded wire truss shear connectors. The test results

show that the horizontal stress reinforcement based on allowable steel stress can provide adequate ductility between the cracking limit and ultimate stress range.

2. The requirements for the cracking moment and ultimate capacity are fully met at the time of failure. The failure is ductile because it occurred through yielding of the truss tension web member.

3. More research is recommended to study the composite action achieved by various connectors and to understand fully the behavior as it relates to the shear deformation of various connectors.

4. There is also a need for more tests to achieve maximum thermal efficiency compatible with the fully composite action despite the fact that these are two contradictory requirements.