State-of-the-Art of Precast/Prestressed Sandwich Wall Panels

Prepared by

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A5 Composite Cladding Panel
A6 Composite Loadbearing Panel

NOTATION

a = depth of equivalent rectangular stress block
a = width of panel being stripped
A = area of concrete at cross section considered
A_b = area of reinforcing bar or stud
A_{cr} = area of crack interface
A_{ps} = area of prestressed steel in tension zone
A_s = area of mild steel reinforcement
A_{vf} = area of shear-friction reinforcement
b = width of compression face of member
b = length of panel being stripped
c = distance from extreme fiber to neutral axis
C = resultant compressive force
C = coefficient of thermal expansion
C_u = factored compressive force
C_w = stud group adjustment factor
d_p = distance from compression fiber to centroid of prestressed reinforcement
D = dead load
e = eccentricity of design load or prestressing force measured from centroid of section
E_c = modulus of elasticity of concrete
EI = flexural stiffness of compression member
f_a = unit stress of structural steel
f_b = bending stress due to stripping; subscript denotes direction
f_c' = specified compressive strength of concrete
f_{ci}' = concrete compressive strength at time considered
f_{pc} = compressive stress in concrete at centroid of cross section due to prestress (after allowance for all prestress losses)
f_{ps} = stress in prestressed reinforcement
f_{pu} = specified tensile strength of prestressing steel
f_w = resultant stress on weld
F_a = allowable bending stress of structural steel
F_h = resultant horizontal shear force
F_w = design strength of weld
F_y = yield strength of structural steel
h = total depth of section
H = horizontal force
I = moment of inertia of section resisting external loads
I_g = moment of inertia of gross section
I_{xx}, I_{yy} = moment of inertia of weld group with respect to its own x and y axes, respectively
I_p = polar moment of inertia
K_u' = coefficient = $M_u(12{,}000)/bd_p^2$
l = total span length
l_d = development length
l_e = embedment length
l_w = length of weld
L = live load
M = unfactored service load moment
M_{cr} = cracking moment
M_n = nominal moment strength at section
M_u = factored moment due to applied loads
M_x = moment due to stripping with respect to x-axis
M_y = moment due to stripping with respect to y-axis
P = applied axial load
P_c = nominal tensile strength of concrete element
P_n = nominal axial load capacity
P_s = nominal tensile strength of steel element
P_u = factored applied axial load
q = load per unit area
Q = statical moment about neutral axis
r = radius of gyration at cross section of a compression member
R = fire endurance of composite assembly
R_1, R_2, R_3 = fire endurance of individual course
R_{DL} = reaction due to dead load
R_u = reaction due to total factored load
S = section modulus
S_w = section modulus of weld group
t = thickness of section
T = resultant tensile force
T = temperature
T_u = factored tensile force
U = required strength to resist factored loads
v = shear force per unit
V = total applied shear force
V_c = nominal shear strength of concrete element
V_{nh} = nominal horizontal shear at plane considered
V_s = nominal shear strength of steel element
V_u = factored applied shear force
w = uniform load
W = wind load
x = overall dimension of a stud group
x = distance to centroid of a weld group
y = distance from centroid of individual area to centroid of gross section
y = overall dimension of a stud group
y = distance from centroid to fiber considered
y = distance to centroid of a weld group
$\bar{y}$ = distance from one surface to centroid of section
Z = with subscript; plastic section modulus
α = with subscript; modification for development length
α = strain gradient across thickness of wall panel
β_d = ratio of factored axial dead load to factored axial total load
Δ = deflection
Δ_{temp} = thermal bow in wall panel
μ_e = effective shear-friction coefficient
ρ = A_s/bd = ratio of non-prestressed reinforcement
ρ_p = A_{ps}/bd_p = ratio of prestressed reinforcement
ϕ = strength reduction factor
ω_{pu} = $\rho_p f_{ps}/f_c'$

Fig. 1.5.a. Atlanta City Detention Center, Atlanta, Georgia – Gate Precast Company.

INTRODUCTION

Precast/prestressed sandwich wall panels are composed of two concrete wythes (layers) separated by a layer of insulation. One of the concrete wythes may be a standard shape, such as a flat slab, hollow-core section, double tee, or any architectural concrete section produced for a single project. In place, sandwich wall panels provide the dual function of transferring load and insulating the structure. They may be used solely for cladding, or they may act as beams, bearing walls, or shear walls.

Precast/prestressed concrete sandwich wall panels are used as exterior and interior walls for many types of structures. These panels may readily be attached to any type of

Fig. 1.5.b. Ganglehoff Center, St. Paul, Minnesota – Spancrete Midwest Company.

structural frame, e.g., structural steel, reinforced concrete, pre-engineered metal and precast/prestressed concrete. The panels are precast at a manufacturing plant, trucked to the project site and erected by cranes. Panels generally span vertically between foundations and floors or roofs to provide the permanent wall system, but may also span horizontally between columns.

In this report, precast/prestressed concrete sandwich wall panels will be referred to as "sandwich panels" or simply as "panels."

Sandwich panels are similar to other precast/prestressed concrete members with regard to design, detailing, manufacturing, handling, shipping and erection; however, because of the presence of an intervening layer of insulation, they do exhibit some unique characteristics and behavior. Where sandwich panel design and manufacturing parallels other precast/prestressed products (in particular, solid wall panels), this report refers to existing technologies. Where there are differences, this report presents this new material or expands upon previously published material.

Interest in sandwich panels has been increasing within the past few years because manufacturers are looking for new, viable product lines and architects/engineers are pleased with the energy performance and general aesthetics of the panels. In addition, contractors have found that the use of sandwich panels allows their project site to be quickly "dried in," allowing other trades to work in a clean, comfortable environment.

Fig. 1.5.c. Mystic Lake Casino, Minneapolis, Minnesota – Spancrete Midwest Company.

Fig. 1.5.d. Pepsi-Cola Bottling Co. of Oshkosh Inc., Oshkosh, Wisconsin – Spancrete Industries, Inc.

Fig. 1.5.e. Erection of sandwich wall panels – Spancrete Industries, Inc.

Fig. 1.5.f. Storage of sandwich wall panels – Spancrete Industries, Inc.

The purpose of this report is to present to the architect, engineer, contractor, precast/prestressed concrete producer and owner current North American practices concerning uses, design, detailing, manufacturing and thermal performance of sandwich panels. An additional purpose is to share experience gained over the past decades, and by doing so, permit specifiers, suppliers and users to benefit. Because design practices relating to sandwich panels vary, the Committee did not attempt to prepare a "Recommended Practice" at this time. However, the methods and descriptions in this report provide ample guidance for obtaining satisfactory results to those who follow them.

This report was prepared by the members of the PCI Committee on Precast Sandwich Wall Panels. The document was subsequently reviewed and balloted for publication by the Technical Activities Council. All reasonable care has been taken to ensure the accuracy of the material presented. The document, however, should only be used by those experienced in the applicable engineering areas dealt with here; recommendations contained herein should not be substituted for sound engineering judgment.

Fig. 1.5.g. Pettit National Ice Center, West Allis, Wisconsin – Spancrete Industries, Inc.

Fig. 1.5.h. Lacks Industries, Grand Rapids, Michigan – Fabcon Incorported.

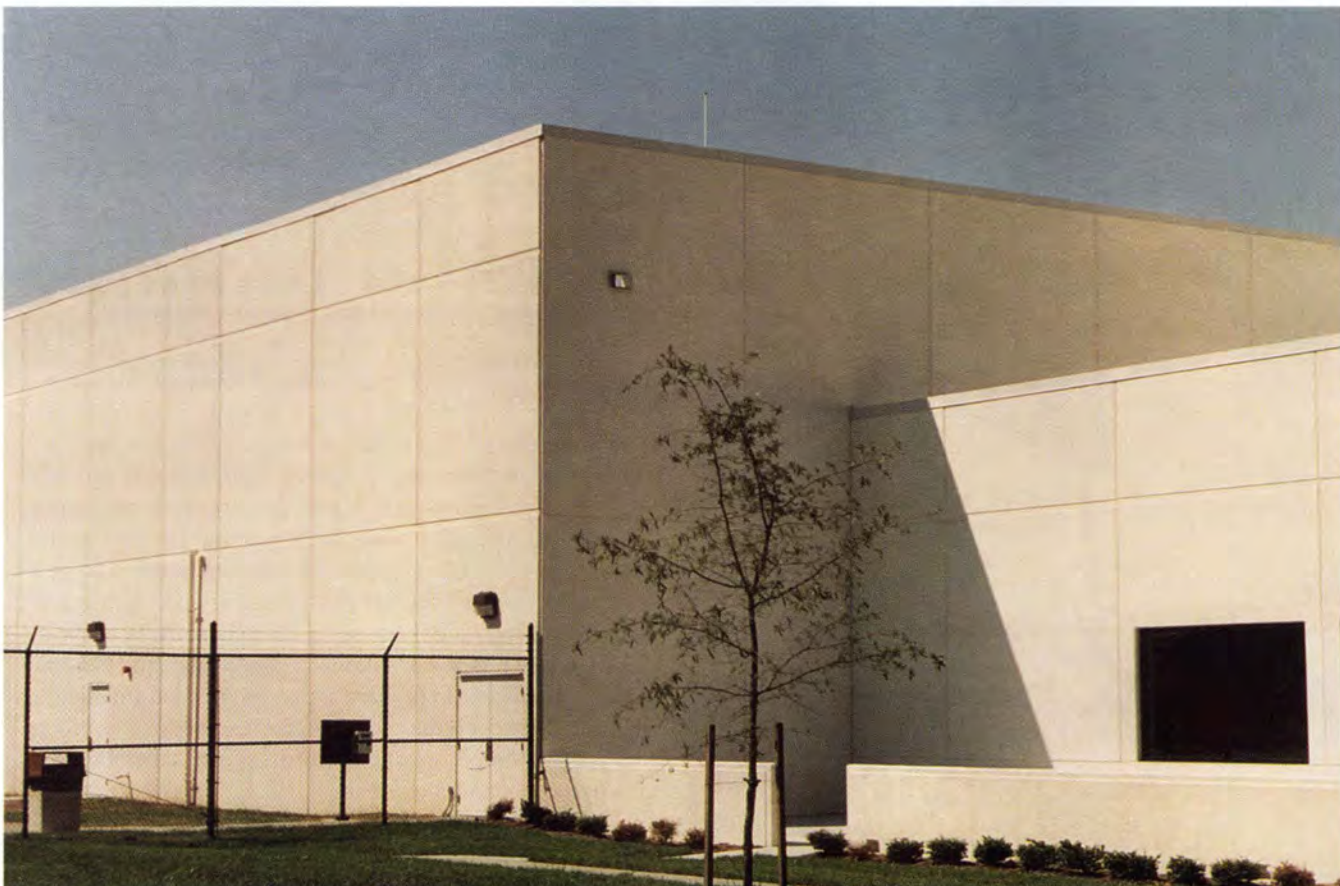


Fig. 1.5.i. Phillip Morris Plant, Newport News, Virgina – Tindall Concrete VA, Inc.

Fig. 1.5.j. Gymnasium & Aquatic Center, Colorado Springs, Colorado – Stresscon Corporation.

CHAPTER 1 — GENERAL

1.1 HISTORY

The Committee was unable to determine the first use of sandwich panels in the United States, but it is known that sandwich panels have been produced in North America for more than 40 years.

The initial sandwich panels were of non-composite type and consisted of a thick structural wythe (or hollow-core slabs, double tees, single tees), a layer of insulation and a non-structural wythe. Composite flat panels were manufactured later.

1.2 MATERIALS

Materials used in the manufacture of sandwich panels are the normal materials found in a precast/prestressed plant. These include structural concrete, reinforcing bars, mesh, steel embedments and prestressing strand. Materials unique to sandwich panels are insulation of various types and a variety of wythe connectors. Both of these materials are discussed in detail in the body of this report.

1.3 ADVANTAGES

Sandwich panels have all of the desirable characteristics of a normal precast concrete wall panel such as durability, economy, fire resistance, large vertical spaces between supports, and use as shear walls, bearing walls, and retaining walls. Sandwich panels can be relocated to accommodate building expansion. In addition, the insulation provides superior energy performance as compared to many other wall systems. There is virtually no limit to the R-value that can be provided. The hard surface on both the inside and outside of the panel provides resistance to forklift damage and vandalism and a finished product requiring no further treatment.

1.4 DESCRIPTION OF PANEL TYPES

1.4.1 Non-Composite

A non-composite panel is analyzed, designed, detailed and manufactured so the two concrete wythes act independently. Generally, there is a structural wythe and a non-structural wythe. The structural wythe is the thicker of the two.

1.4.2 Composite Panels

Composite panels are analyzed, designed, detailed and manufactured so that the two concrete wythes act together to resist applied loads. The entire panel acts as a single unit. This is accomplished by providing full shear transfer between the wythes.

1.4.3 Semi-Composite Panels

Semi-composite panels are analyzed and designed as composite or partially composite panels during stripping, shipping and erection, but as non-composite panels for in-place loads. Experience indicates that early bond between certain insulation types and the concrete wythes provides sufficient shear transfer for composite action during handling, but for design purposes this bond is considered unreliable for the long term.

Fig. 1.5.k. Thermoform Plastics, White Bear Lake, Minnesota – Fabcon Incorporated.

Fig. 1.5.l. Henderson County Courthouse, Hendersonville, North Carolina – Metromont Materials Corporation.

Fig. 1.5.m. Deublin Company, Waukegan, Illinois – J.W. Peters & Sons, Inc.

Fig. 1.5.n. Harrison County Jail, Indiana – Concrete Technology Inc.

Fig. 1.5.o. Alfa Laval Flow, St. Charles, Missouri – Fabcon Incorporated

Fig. 1.5.p. Bausch & Lomb, Lynchburg, Virginia – Tindall Concrete VA, Inc.

1.5 APPLICATIONS

Sandwich panels provide economic, attractive and energy efficient hard walls, and are found on virtually every type of structure including residential buildings, office buildings, low-temperature environments, controlled atmospheres, warehouses, industrial buildings and justice facilities. The most common use of these panels is for exterior walls, but they have been used as internal partition walls, particularly around temperature controlled rooms. Thick, high R-value sandwich panels have been used in sub-zero freezer applications as well.

Sandwich panels have also been used in architectural applications. The exterior wythe can receive the same architectural treatment used on any other architectural panel.

Figs. 1.5.a through 1.5.p show various applications of sandwich panels in structures across the United States.

CHAPTER 2 — DESIGN AND DETAILING CONSIDERATIONS

2.1 GENERAL INFORMATION

The design of precast/prestressed concrete sandwich wall panels is similar to that of typical precast/prestressed concrete members once a selection of the type of panel to be designed and manufactured is made, i.e., non-composite, composite or semi-composite. Once the panel type is selected, the section properties are calculated and the distribution of forces determined. The panels are checked for stresses resulting from transfer of prestress forces, handling, transportation, in-place service loads, and resistance to in-place factored loads. Deflections due to in-place service loads are also checked. The criteria used to evaluate the acceptability of the stresses and deflections are as stated in the current version of ACI 318-95 Building Code[1] and applicable local building codes.

The keys to successful concrete sandwich wall panel design are to ensure that the actual structural behavior of the panel coincides as nearly as possible to the predicted behavior and the original design assumptions, and to detail the panel and connections to accommodate anticipated movement. Because present knowledge of the behavior of sandwich panels is primarily based on observed phenomena and limited testing, some difference of opinion exists among designers concerning such matters as degree of composite action and the resulting panel performance, the effectiveness of shear transfer connectors and the effect of insulation type and surface roughness on the degree of composite action. Current and future research will undoubtedly provide better tools that can be used for more accurate predictions of behavior. This chapter addresses some of the successful design and detailing practices used for concrete sandwich wall panels.

2.2 WYTHE THICKNESS AND SIZE OF PRESTRESSING STRAND

Wythe thickness commonly used for precast/prestressed concrete sandwich wall panels has ranged from a minimum of 2 in. (51 mm) to as thick as required for the imposed loading. Most panels are made as thin as possible, but the wythe thickness is generally determined by the panel type and final use. For example, the required fire resistive rating may be the determinant for the thickness. A non-composite panel usually requires a thicker wythe(s) than a composite panel with the same load and span conditions. The use of form liners or reveal strips on the exterior wythe requires the exterior wythe to be thicker than a plane panel, so that code requirements for concrete cover and strength can be satisfied.

The maximum strand diameter that may be used is related to wythe thickness. Satisfactory results have been experienced using $^3/_8$ in. (9.5 mm) diameter strand in 2 in. (51 mm) thick wythes containing a $^3/_4$ in. (19 mm) maximum aggregate size. A wythe thickness of 3 in. (76 mm) is sufficient when using $^1/_2$ in. (13 mm) diameter strand. A thickness of 6 in. (152 mm) or greater generally employs two layers of strands.

When the wythe thickness to strand diameter ratio is less than six, special reinforcement is provided at the ends of the panel to limit splitting cracks over the strands during detensioning. With low thickness-to-strand diameter ratios, special attention during production is given to the as-placed strand location in order to maintain required cover and to minimize the risk of splitting cracks.

2.3 STRAND LOCATION AND FORCE

Strands are generally pulled to normal levels of initial tension and located at the centroid of each wythe so there is no tendency of the wythe to "camber." Composite panels that have wythes of unequal thickness or of unequal section properties have strands normally placed at the centroid of each wythe, but the initial tension is adjusted in one of the wythes so the resultant prestress force coincides with the centroid of the composite section. This is done to eliminate or reduce the initial bow or camber in the panel.

Some designers report success in intentionally introducing an inward bow to composite panels by the use of an eccentric prestress force. The intent of this is to offset the observation that "composite sandwich wall panels always bow outwards." The use of such a design should be carefully thought out because the introduction of an initial bow may create field alignment problems.

A uniform strand spacing across the panel width is generally used. Unavoidable transverse eccentricity may occur in L-shaped and cut-back panels, but the effect of this eccentricity is generally negligible.

2.4 WYTHE CONNECTORS

2.4.1 General Considerations

Wythe connectors are used to tie the two wythes together and keep the panel intact during handling and service conditions. These connectors penetrate the insulation and are bonded to each wythe. The typical wythe connector spacing ranges from 16 x 16 in. to 48 x 48 in. (406 x 406 mm to 1219 x 1219 mm). In composite panels, connectors may be used to create a shear transfer between the wythes. Wythe connectors come in many different sizes, shapes and materials. Some of the types and shapes are: C-tie, Z-tie, M-tie, cylindrical metal sleeve anchors, hairpin, circular expanded metal, welded wire trusses, plastic or fiber-composite pins, and areas of solid concrete.

Wythe connectors serve a variety of functions. If the panel is cast and stripped in a flat position, the connectors must be capable of resisting the tension created between the wythes during stripping. The ties are also used to transfer wind and seismic forces between the wythes. In composite panels, the connectors provide resistance to horizontal bending shear between the wythes. In non-composite panels, the type and arrangement of connectors are detailed to minimize horizontal shear transfer so that the wythes may act independently. Wythe connectors may be used in various combinations. For example, in a composite panel design, solid blocks of concrete may be used as a shear transfer mechanism and metal C-ties can be used to prevent the wythes from separating.

2.4.2 Shear Connectors

Shear connectors are used to transfer shear forces between the two wythes. Because wall panels are usually designed as one-way structural elements, shear forces are generated due to longitudinal bending in the panels. In some cases, the shear connectors may be used to transfer the weight of a non-structural wythe to the structural wythe.

Some shear connectors are stiff in one direction and flexible in the other. These are called one-way shear connectors. Examples of these are longitudinal steel wire trusses, solid ribs of concrete, M-ties, flat sleeve anchors and small diameter bent bars. These connectors are shown in Fig. 2.4.2.a. Care must be taken in the manufacturing process to maintain the intended orientation of the connectors.

Other shear connectors are stiff in at least two perpendicular directions and will consequently transfer both longitudinal and transverse horizontal shears. Examples of these are solid blocks of concrete (often located at lifting points),

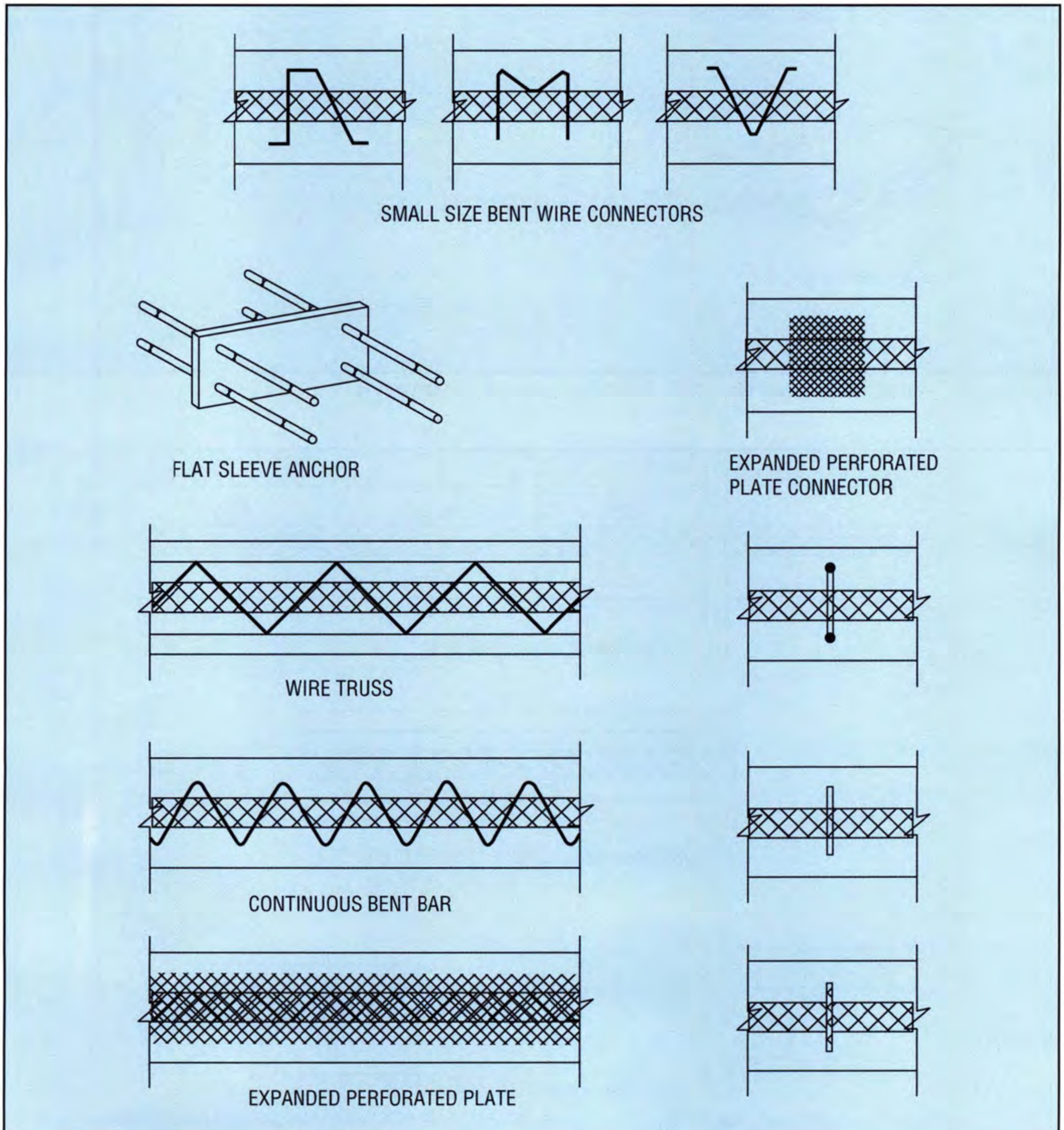


Fig. 2.4.2.a. One-way shear connectors, stiff in only one direction.

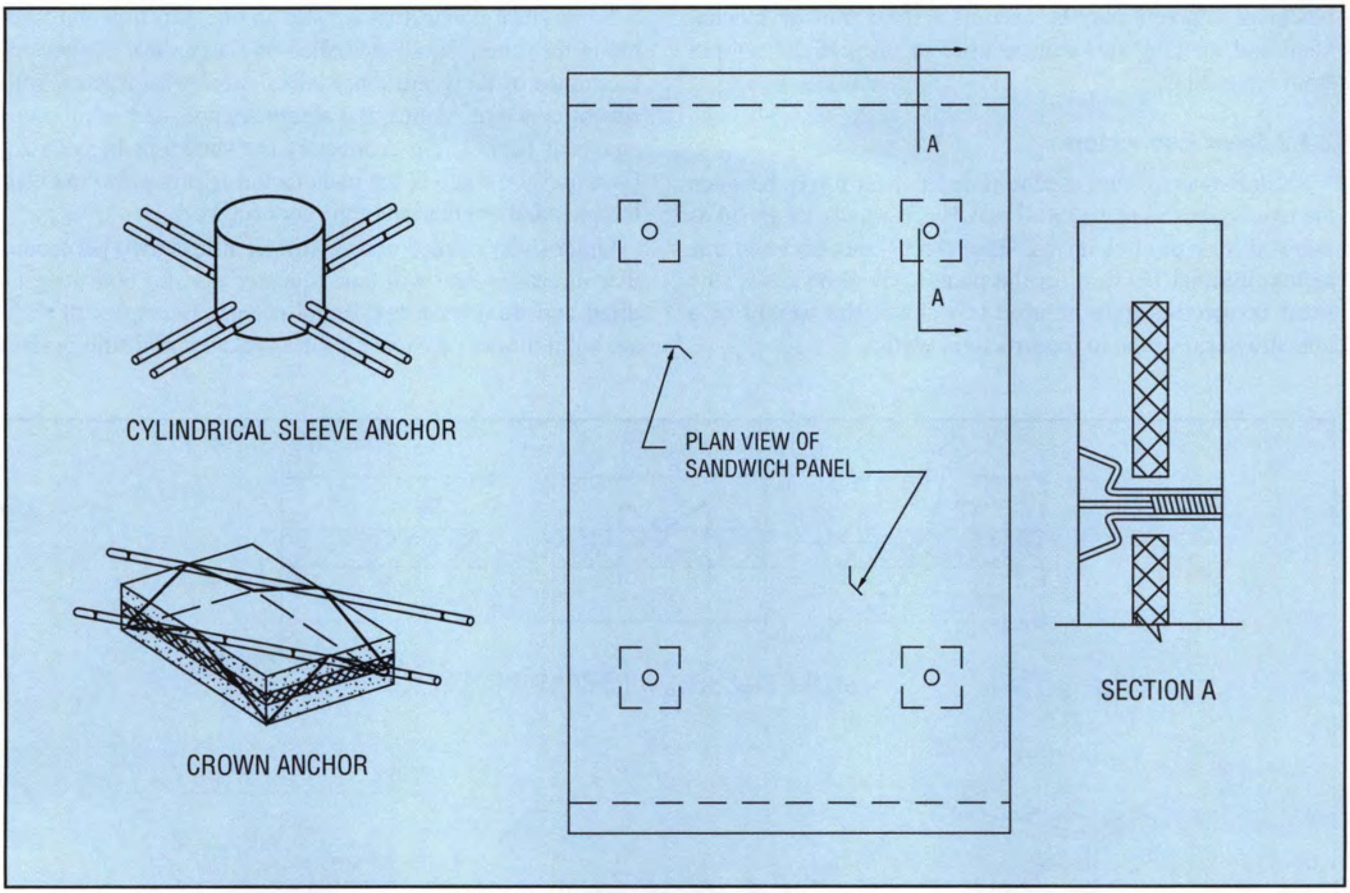


Fig. 2.4.2.b. Two-way shear connectors, stiff in at least two perpendicular directions.

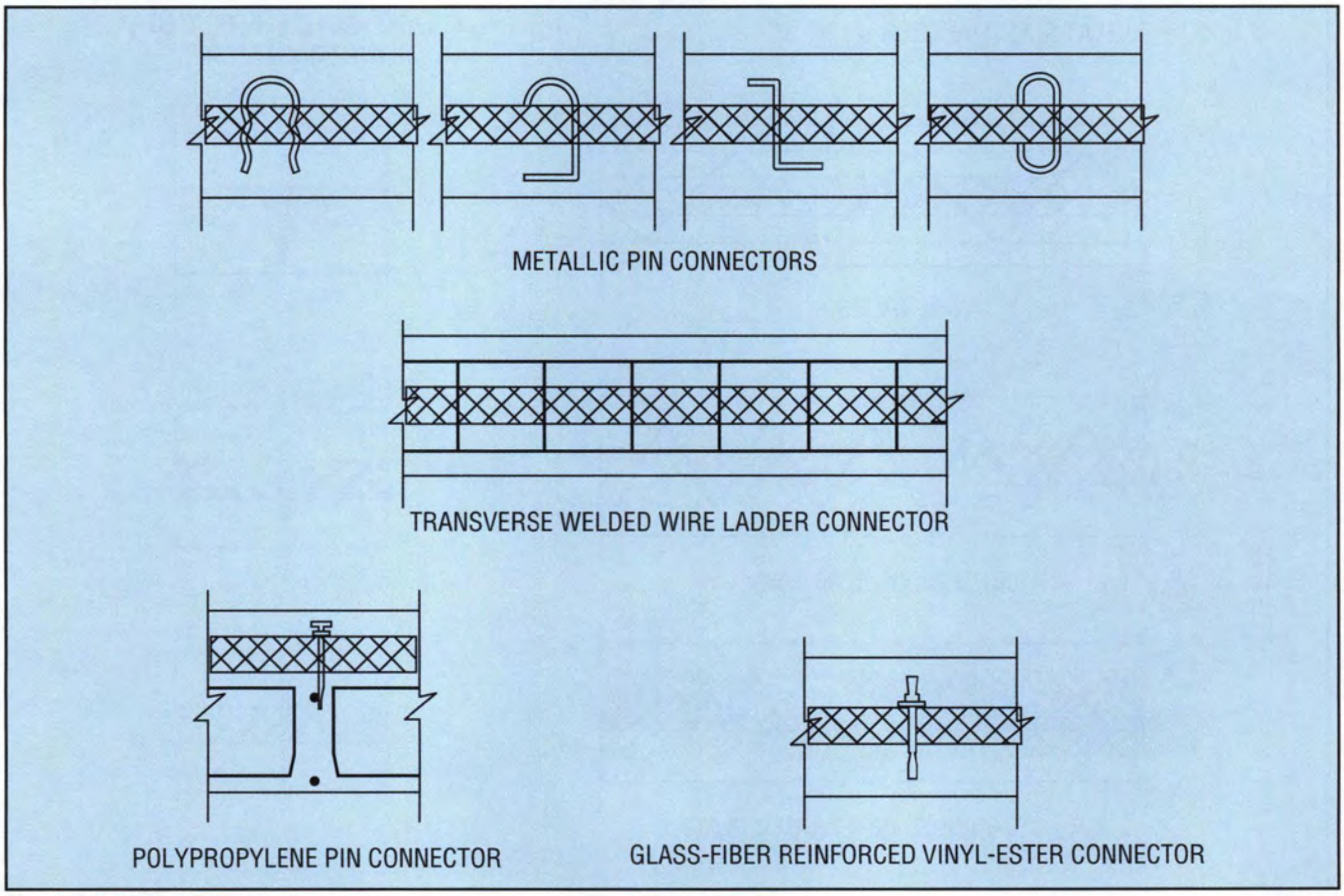


Fig. 2.4.3. Non-composite connectors.

connection plates, cylindrical sleeve anchors and crown anchors. These are shown in Fig. 2.4.2.b.

Capacities of shear connectors may be obtained from the connector manufacturer or, in some cases, calculated using allowable bond stresses for plain smooth bars along with allowable steel stresses for bending, shear and axial forces. When solid areas of concrete are utilized, a commonly used ultimate shear resistance value (ϕv_{nh}) is 80 psi (552 kPa).

The insulation between the wythes may in some cases provide a shear resistance between the wythes. Rough faced dense insulation provides more shear transfer than slick faced insulation. Shear resistance that may be available from bonded insulation is considered temporary. In semi-composite panels, the assumption is made that the insulation provides sufficient shear transfer to create composite action during stripping, handling and erection, but the shear transfer is not relied on to provide composite action for resisting service loads.

2.4.3 Non-Shear Connectors

Non-shear connectors are tension elements that are not capable of transferring longitudinal shear forces between the wythes. They are used in non-composite panels to transfer normal forces between wythes and in composite panels as auxiliary connectors to the shear connectors when the spacing of the shear connectors is large. These connectors are capable of resisting tension and compression but no significant amount of shear; consequently, their contribution to composite action is usually neglected. However, certain "non-shear" connectors may provide some shear resistance. Use of panels with these connectors may be justified by providing data to the proper building officials, following the approval procedures of ACI 318-95, Section 1.4.[1]

Examples of non-shear connectors are plastic pins, metal C-ties, hairpins, and continuous welded ladders. These connectors are shown in Fig. 2.4.3.

2.5 PANEL WIDTH, THICKNESS AND SPAN

Sandwich wall panels are manufactured in virtually all the same shapes and sizes as solid panels. In general, the larger the panel, the greater the economy because there are fewer pieces to form, strip, load, transport, erect, and connect. The maximum size is limited only by the handling capability of the plant, erection equipment, transportation restrictions, and the ability of the panel to resist the applied stresses.

Sandwich panels have been made as wide as 14 ft (4.3 m) and as tall as 75 ft (23 m). Overall thickness has varied from 5 in. to greater than 12 in. (127 to 305 mm). Wythe thickness is discussed in Section 2.2. Insulation thicknesses have commonly varied from 1 to 4 in. (25.4 to 102 mm). The most common ranges in sizes are:

Wythe thickness: 2 to 8 in. (51 to 203 mm)
Insulation thickness: 1 to 4 in. (25.4 to 102 mm)
Overall thickness: 6 to 12 in. (152 to 305 mm)
Width: 8 to 12 ft (2.4 to 3.6 m)
Span: 10 to 50 ft (3 to 15.2 m)

2.6 BOWING CONSIDERATIONS

Bowing in sandwich panels is a deflection caused by differential wythe shrinkage, thermal gradients through the panel thickness, differential modulus of elasticity between the wythes and creep from storage of the panels in a deflected position. These actions cause one wythe to lengthen or shorten relative to the other. When wythes are interconnected, such differential wythe movement may result in curvature of the panel, i.e., bowing. Because most sandwich panels exhibit some degree of composite interaction due to shear transfer by either bonded insulation and/or by the stiffness of wythe connectors, bowing in all types of sandwich panels is common.

Bowing is a complicated and sometimes controversial topic. The present state-of-the-art is not sufficiently advanced to precisely predict the amount of bow in any given panel.

Reasons for this inability to precisely predict bow are:

- Shrinkage, creep and modulus of elasticity of the concrete cannot be precisely predicted.
- Actual thermal gradients and their shape are not precisely known.
- The degree of restraint provided by external connections is not precisely known.
- The degree of composite action in semi-composite panels is generally not precisely known.
- An exact analytical model for each of the above, or the interaction among them, has not been established.

With all these unknowns, it is still possible for the designer to adequately account for the bowing characteristics of composite and semi-composite panels. In this regard, it is similar to the imprecision in accurately predicting the camber of a double tee. The most important consideration is to realize that bowing will occur and to establish a reasonable value for the magnitude of bowing, often based on experience. Connections between the panel and the structural and non-structural systems should then be made so that no distress is experienced in any of the elements due to forces that develop in the connections.

Some designers have found that by using a smaller thermal gradient through the panel than the project site atmospheric temperature difference, a reasonably accurate prediction of panel behavior can be made. The design examples in the Appendix reflect this approach. Detailing as related to bowing is discussed in Section 2.11.

Some useful observations made by those experienced with composite and semi-composite sandwich panels are:

- Panels bow outwards most of the time.
- Panels heated by the afternoon sun will bow more than those that are not, i.e., panels on the south and west elevations will bow more than those on the east and north elevations.
- Panels bow daily due to transient thermal gradients.
- Sandwich panels experience a greater thermal gradient than solid panels of equal thickness. This is due to the superior thermal properties of sandwich panels.
- Panels stored in a bowed position will remain in the bowed position after erection. This may be due to "locked-in" creep.

- Differential shrinkage can occur between the wythes due to relative humidity differences between interior and exterior exposures.
- Panels containing wythes with different moduli of elasticity, such as panels with wythes containing different concrete strengths but with equal levels of prestress, will bow ("curl") due to differential shortening and creep of the wythes after prestress transfer.

In addition to the bowing characteristics of an individual panel, the possibility of differential bowing between adjacent panels must also be considered. Differential bowing may occur because:

- Adjacent panels may have different flexural stiffness, such as a flat panel adjacent to a stemmed panel.
- Adjacent panels may be connected to the structure at different levels.
- Adjacent panels may be supported at different footing elevations creating differing span lengths and locations of maximum bowing.
- Adjacent panels may have different shrinkage, creep and/or modulus of elasticity properties, especially if they were cast at different times.
- Adjacent panels may have different initial bows because of non-uniform storage conditions.
- Panels that meet at corners will not have identical bowing characteristics due to the different exposures. L-shaped panels at corners will also bow differently than adjacent flat panels.

In order to maintain integrity of caulking, connections are detailed so that adjacent panels move together perpendicular to their plane. The connections are also detailed so that volume change forces do not build up parallel to the plane of the panels. As noted previously, experience has shown that sandwich panels located on the south and west building elevations bow more than those on the north and east elevations. This is due to the dominant sun exposure on the south and west elevations. It is emphasized that architects/engineers should anticipate this and should check with the local precasters for expected bowing characteristics.

It is also important that the architect and the structural engineer realize that any calculation of anticipated sandwich panel bowing is approximate. The exact amount of actual bowing cannot be determined. It is essential that all parties understand there will be bowing, that experience with similarly configured panels is the best method of predicting the magnitude of bow, and that the panel connections be detailed accordingly.

2.7 FLEXURAL DESIGN

2.7.1 Level of Prestress

Many designers use a minimum level of effective prestress for each wythe ranging between a minimum of 225 psi up to 600 psi (1.55 to 4.14 MPa); these values have been successfully used to ensure that the panel will perform properly during stripping, handling, erection, and under service loads.

Table 2.7.2. Load distribution for 2/3/6 non-composite panel.

Wythe (in.)	Moment of inertia (in.^{4} per ft width)	Load (percent)
2	8	4
6	216	96
Total	224	100

Note: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 in.^{4} per ft = 1,365,587 mm^4/m.

2.7.2 Non-Composite Panels

The flexural design of non-composite sandwich panels is identical to that of solid panels that have the same sectional properties as the structural wythe(s) of the non-composite panels. One additional consideration is the distribution of loads between the wythes for a panel that has two "structural" wythes. The distribution of loads is based on the relative flexural stiffness of each wythe. Once this distribution is made, each wythe is then individually designed as a solid panel. Deflections are calculated using the sum of the wythe flexural stiffness. For example, the load distribution for a 2/3/6 non-composite panel would be as shown in Table 2.7.2.

As a practical matter, the 6 in. (152 mm) wythe in this case would probably be designed for 100 percent of the load. The deflection calculation would be based on a moment of inertia of 224 in.^{4}/ft width (3.06×10^8 mm^4/m).

The theoretical stress distribution for a 2/2/3 sandwich panel is shown in Fig. 2.7.2. The 2 in. (51 mm) wythe resists 23 percent of the load and the 3 in. (76 mm) wythe resists 77 percent of the load.

2.7.3 Composite Panels

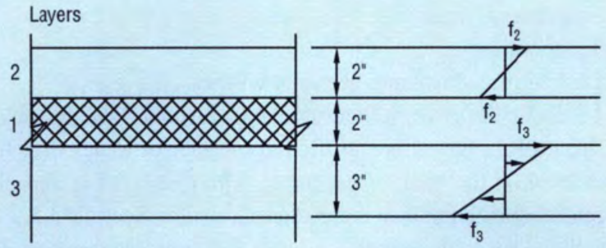
The flexural design of composite panels is similar to that of solid panels that have the same cross-sectional (or flange) thickness. The differences are:

1. A mechanism for horizontal shear transfer between wythes must be provided and analyzed.

2. The calculation of the sandwich panel section properties must account for the thickness of the individual wythes, the location of the composite centroid and the lack of concrete at the location of the insulation. A generic section property calculation for a composite flat sandwich panel (see Fig. 2.7.3.a) and some comparisons between solid and sandwich flat panel section properties are listed in Table 2.7.3.

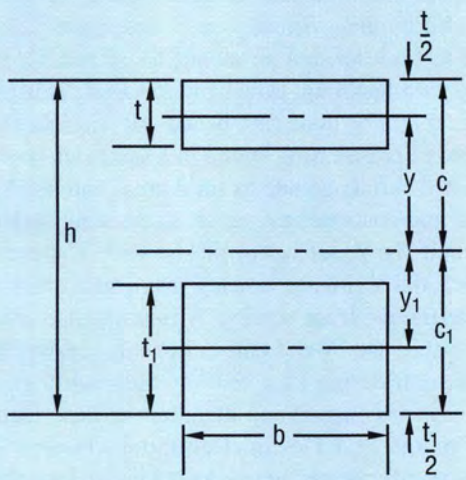
The calculation for horizontal shear transfer is made using the strength concept as given in the PCI Design Handbook, Fourth Edition (Section 4.3.5)[2] for composite flexural members. In this approach, the maximum compressive capacity of the concrete is compared to the maximum tensile capacity of the reinforcement. The lesser of the two forces must be resisted by the shear transfer mechanisms located in the horizontal shear span. The tensile capacity of the reinforcement will be the lesser force in all practical cases. For simply supported members loaded uniformly, the horizontal shear span may be taken as one-half the clear span length.

A sample calculation is shown in Fig. 2.7.3.b for a 3/2/3 composite panel. Each wythe is prestressed with eight $^3/_8$ in.



$$I_2 = (1/12)(12)(2)^3 = 8 \text{ in.}^4$$
$$I_3 = (1/12)(12)(3)^3 = 27 \text{ in.}^4$$
$$S_2 = 8/1 = 8 \text{ in.}^3$$
$$S_3 = 27/1.5 = 18 \text{ in.}^3$$
$$M_2 = [I_2/(I_2 + I_3)]M = (8/35)M = 0.23M$$
$$\therefore\ f_2 = (0.23M)/8 = 0.029M$$
$$M_3 = [I_3/(I_2 + I_3)]M = (27/35)M = 0.77M$$
$$\therefore\ f_3 = (0.77M)/18 = 0.042M$$

Fig. 2.7.2. Stress distribution in a non-composite sandwich panel.



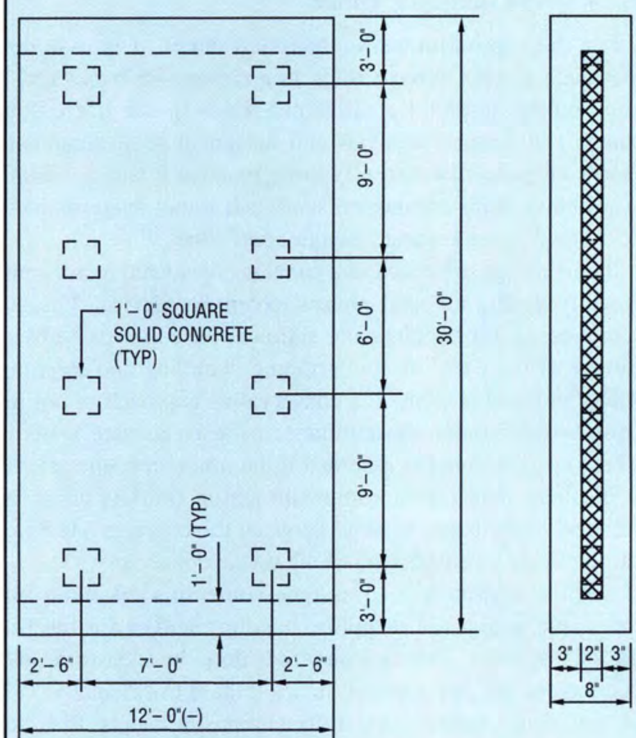
$$A = b(t + t_1)$$
$$c = \frac{\left[\frac{1}{2} bt^2 + bt_1(h - \frac{1}{2} t_1)\right]}{A}$$
$$I = \frac{bt^3}{12} + bty^2 + \frac{bt_1^3}{12} + bt_1 y_1^2$$
$$S = \frac{I}{c} \qquad S_1 = \frac{I}{c_1}$$
$$r = \left(\frac{I}{A}\right)^{1/2}$$

Fig. 2.7.3.a. Section properties of composite sandwich panel.

Table 2.7.3. Example comparisons of solid vs. sandwich panel section properties.

Panel configuration	Total thickness (in.)	I_{solid} (in.^{4} per ft width)	S_{solid} (in.^{3} per ft width)	I_{comp} (in.^{4} per ft width)	S_{comp} (in.^{3} per ft width)
2/2/2	6	216	72	208	69
3/3/3	9	729	162	702	156
4/2/2	8	512	128	472	109/ 129

Note: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 in.^{3} per ft = 53,763 mm^3/m; 1 in.^{4} per ft = 1,365,587 mm^4/m.



Given: $A_{ps} = 8(0.085 \text{ in.}^2) = 0.68 \text{ in.}^2$
$f_{ps} = 262$ ksi
$f'_c = 5$ ksi

Solution: $T = (0.68 \text{ in.}^2)(262 \text{ ksi})$
$= 178$ kips
$C = (0.085)(5 \text{ ksi})(3 \text{ in.})(144 \text{ in.})$
$= 1836$ kips
$F_h = 178$ kips

Area of concrete for shear transfer located within the horizontal shear span:
(4)(12 in.)(12 in.) + (12 in.)(14 in.) = 2304 in.^{2}

$V_{nh} = (0.080 \text{ ksi})(2304 \text{ in.}^2)$
$= 184 \text{ kips} > 178 \text{ kips}$
$\therefore$ Composite action is achieved.

Fig. 2.7.3.b. Sample calculation of horizontal shear in a 3/2/3 composite sandwich panel.

(9.5 mm) diameter 270 ksi (1862 MPa) strands and the concrete strength is 5000 psi (34.5 MPa). The panel has a 1 ft (0.305 m) solid horizontal band of concrete at each end and eight 1 ft (0.305 m) square solid concrete blocks located at the lifting points. Although the calculations shown in Fig. 2.7.3.b employ only the available concrete area in shear, if necessary the shear transfer capacity of this section can be enhanced by introducing shear-friction reinforcement at the solid concrete sections located at the lifting points.

Unless full composite action is verified by tests, some manufacturers and designers choose to use a stiffness less than that based on full composite behavior.

2.7.4 Semi-Composite Panels

For the semi-composite sandwich panel, it is assumed that such panels behave both as a composite panel and a non-composite panel at different times in the life of the panel. The flexural analysis and design of semi-composite sandwich panels is currently done in several ways. Certain proprietary semi-composite sandwich panel systems have developed along separate design approaches.

In one design approach, the panel is considered to act compositely during all load phases except in-service. The assumption of initial composite action is based on past observations of the panel during stripping, handling and erection. After the panel is erected, a conservative approach of not relying on horizontal shear transfer to resist service loads is taken, i.e., the panel is assumed to be non-composite after it is in place. The initial composite action (horizontal shear transfer) is attributed to bond between the concrete and insulation plus any contribution of the wythe connectors.

Another approach is to assume something less than full composite action for stripping, handling and/or for the full life of the panel. This is sometimes done by measuring deflections of the panel and comparing them to calculated values for non-composite and fully composite panels, thus deriving an assumed percentage of full composite behavior.

2.7.5 Other Considerations

An important design check to be made is the ACI 318-95[1] requirement (Section 18.8.3) that the flexural strength of the panel exceeds 1.2 times the cracking strength. In prestressed sandwich panels, the level of prestress, and thus the cracking strength, may be sufficiently high so that supplemental mild steel reinforcement needs to be added to increase the flexural strength.

2.8 LOADBEARING DESIGN

2.8.1 Loading

Loadbearing sandwich wall panels are designed for various loads, such as:

- Self weight
- Roof (and floor) $D + L$
- Wind
- Seismic
- Load from adjacent panels
- Soil (lateral pressure)
- Temperature
- Differential shrinkage between wythes

Lateral soil pressure loading can occur when the soil level in front of the panel is significantly higher or lower than the soil level on the back of the panel. Thermal bowing and differential shrinkage bowing are explained in Section 3.3.2 of the PCI Design Handbook, Fourth Edition[2] and Section 2.6 of this report; each effect may create forces between adjacent panels.

The loadings listed above are external, i.e., they act on the entire structure. In the case of a sandwich wall panel, they relate to the panel in its erected position. Panels also are designed for loads imposed during stripping, yard handling, travel and erection. The panel self weight in a horizontal, flat position, with equivalent static load multipliers to account for stripping and dynamic forces, will often govern the panel design. Forces imposed during manufacturing and erection are discussed in Section 5.2 of the PCI Design Handbook, Fourth Edition.[2]

2.8.2 Design

Sandwich wall panels placed vertically are designed similar to beam-columns. Prestressing in the long direction is highly recommended and is particularly effective as a method of providing virtually crack-free panels. Panels are designed to limit tension so as not to exceed $5\sqrt{f_c'}$ during stripping, yard handling, travel and erection. This portion of the design might be done first because it will usually govern the amount of prestressing strand in a sandwich wall panel.

Restrained shrinkage due to solid areas and bond between insulation and concrete may result in cracking under service conditions if the panel is not prestressed. Thermal bowing and differential shrinkage bowing may cause cracking if the panel is restrained from bowing by intermediate connections to the structure and by end connections that resist rotation.

The strength design of a bearing wall sandwich panel is the same as for compression members as described in Section 4.7 of the PCI Design Handbook, Fourth Edition.[2] Slenderness effects are the moments caused by the eccentricity of axial loads due to deflections resulting from wind and seismic or gravity forces, and out of plumb resulting from erection tolerances and bowing. These secondary effects can be accounted for in the panel design by using the moment magnification method or the second order (P-Δ) analysis as described in Section 3.5 of the PCI Design Handbook, Fourth Edition.[2] Interaction curves for various wythe thicknesses and strand configurations can be determined using the methods described in Section 4.7 of the PCI Design Handbook, Fourth Edition,[2] or from available computer programs.

Stresses for service loading are also checked. Tension under service loading should not exceed $7.5\sqrt{f_c'}$ in order to keep the panel from cracking. As stated above, the stresses during handling are usually more critical than service load conditions.

Panels spanning horizontally are designed similar to beams.

2.8.3 Non-Composite Panels

For flat panels, the interior wythe is usually assumed to be the structural wythe and is designed to support the panel loads. For double tee wall panels, the wythe containing the stems is generally chosen to be the structural wythe. If the non-structural wythe does not bear on the structure below, its weight must be transferred to the structural wythe. Connectors are used to support the non-structural wythe as described in Section 2.4.2. Loading and design are the same as described in Sections 2.8.1 and 2.8.2 above. Examples of non-composite designs are detailed in Appendix A. Flexural design of non-composite panels is described in Section 2.7.2.

2.8.4 Composite Panels

Handling and service loads are resisted by the two wythes acting in unison. In order to provide for composite behavior, measures must be taken to ensure transfer of the calculated shear between the wythes in the direction of panel span. Rigid ties, welded wired trusses or regions of solid concrete that join both wythes are used to achieve composite panels. Loading and design are the same as described in Sections 2.8.1 and 2.8.2. Examples of composite panel designs are detailed in Appendix A. Flexural design of composite panels is described in Section 2.7.3.

2.8.5 Semi-Composite Panels

For effects of slenderness, the sum of the individual moments of inertia may be used. Loading and design are as described in Sections 2.8.1 and 2.8.2 . Flexural design of semi-composite panels is described in Section 2.7.4.

2.9 SHEAR WALL CONSIDERATIONS

2.9.1 General

Sandwich wall panels may be arranged to resist lateral loads imposed on a structure by acting as vertical cantilever beams from the foundation. For most configurations, the portion of the total lateral force that each wall resists is based on the bending and shear resistance of the wall. The design of shear wall buildings is performed in accordance with Sections 3.7 and 3.10 of the PCI Design Handbook, Fourth Edition.[2]

Connecting long lengths of wall panels together can result in an undesirable buildup of forces due to volume change; thus, it is preferable to resist the lateral loads by individual panels or by only small groups of panels.

If there is uplift at the base of the shear wall, a connection similar to that shown in Section 2.10.2.4 can be designed to transfer the force to the foundation. Connections to the roof or floor similar to Sections 2.10.3 or 2.10.5 must transfer the diaphragm forces to the shear wall panel and satisfy structural integrity requirements.

Example designs of shear wall panels can be found in Appendix A and in Section 3.7 of the PCI Design Handbook, Fourth Edition.[2]

2.9.2 Non-Composite Panels

Lateral load resistance of a non-composite panel is based on the stiffness of the structural wythe only. Shear wall connections at the foundation and at the roof or floor diaphragm are made to the structural wythes.

2.9.3 Composite Panels

The bending and shear resistance of a composite panel are based on the composite section of the sandwich panel. Connections can be made to either wythe and are usually located at solid areas where the insulation is interrupted.

2.9.4 Semi-Composite Panels

Lateral load resistance of a semi-composite panel is based on the structural wythe, the same as for a non-composite panel. The comments for non-composite panels above are the same for semi-composite shear wall panels.

2.10 EXTERNAL CONNECTIONS

2.10.1 General

This section includes examples of connection details of sandwich wall panels to foundations, to other framing members of a structure, and to each other. The details included are neither all inclusive or necessarily the best possible arrangements. Most of the details come from sandwich wall panel system brochures, from precasters' standard connections and from actual construction projects. The details are included to present ideas and to illustrate connections commonly used. Consultation with local precasters is recommended when planning connection design. Precasters can advise which types of connections would be most appropriate and economical for a given sandwich wall panel application.

Selection of a connection detail for a specific project requires consideration of strength requirements, energy performance and load transfer paths. Production, erection, serviceability and durability should also be considered. This state-of-the-art report presents connection ideas and generalities; thus, detailed design information is not shown on the sketches. Sizes of plates, weld size and lengths and joint dimensions have been purposely omitted. The connections should be designed using the principles and examples of the PCI Design Handbook, Fourth Edition[2] and PCI MNL-123, Design and Typical Details of Connections for Precast and Prestressed Concrete, Second Edition.[3] The requirements for structural intègrity, as outlined in the PCI Design Handbook, Fourth Edition[2] Section 3.10, and ACI 318-95,[1] Chapters 7 and 16, must also be satisfied.

The following detail sketches are arranged in groups according to the location of the connection on the panel and what the panel is connected to. Within each category, the description of the details, including features and considerations, is given followed by sketches of the details. The details include the following categories:

- Panel base to foundation connections
- Panel top to roof connections
- Panel to lintel beam connections

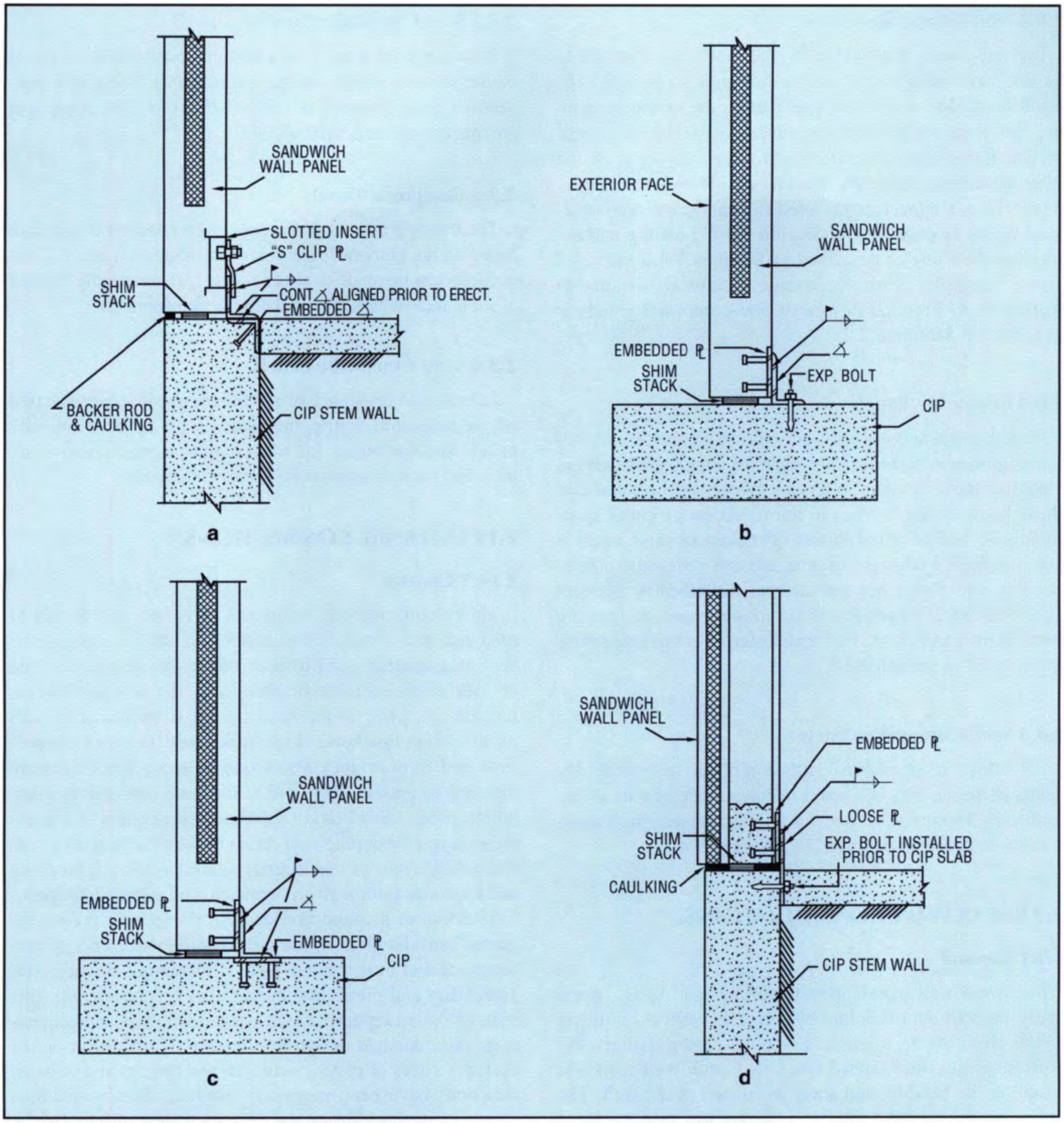


Fig. 2.10.2.1. Non-shear wall to foundation connections.

- Panel to intermediate floor connections
- Corner panel connections
- Panel to panel connections

2.10.2 Panel Base to Foundation Connections

Five types of panel base foundation connections are illustrated and discussed:

- Non-shear-wall panels to concrete foundations
- Panels connected to foundations and to floor slabs
- Slotted connections
- Shear wall panels
- Panel bracing

2.10.2.1 Non-Shear-Wall Panels to Concrete Foundations (Figs. 2.10.2.1.a, b, c, d, and e)

The panels are cast with either embedded weld plates or slotted inserts. The concrete foundation walls or footings have embedded weld plates or drill-in expansion anchors. The sandwich wall panel can either extend underground to the footing or sit at the floor level on top of a cast-in-place concrete wall. The panels are set on shims and then the joint may be partially grouted for non-loadbearing panels or completely grouted for loadbearing panels. Panel support during erection is achieved by temporary bracing or attachment to a braced structure.

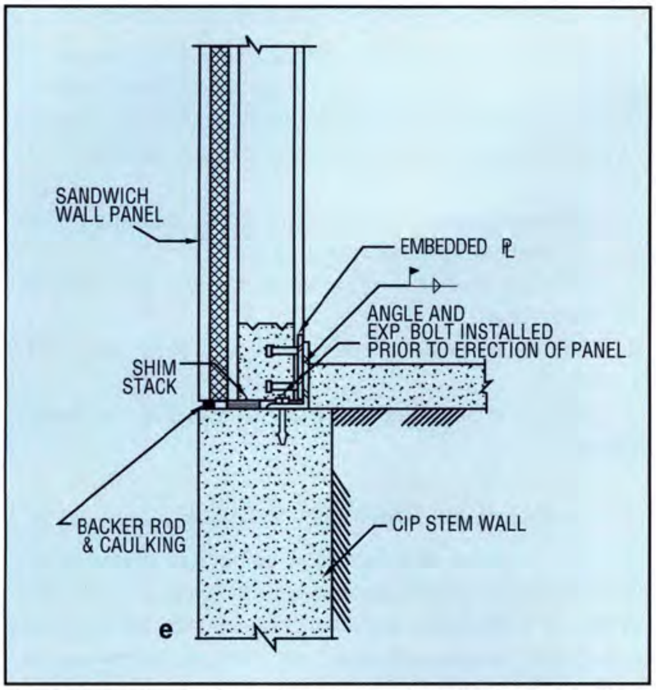


Fig. 2.10.2.1 (cont.). Non-shear wall to foundation connections.

Features:

1. Drill-in expansion anchors eliminate tolerance problems.

2. Extending panels to footing eliminates cost of forming and pouring cast-in-place concrete walls.

Considerations:

1. Care needs to be taken concerning coordination and tolerances between embedded plates in cast-in-place walls or footings and embedded plates in precast panels.

2. Need temporary bracing if panel is erected before rest of structure, or two cranes must be used simultaneously.

3. Connection may not be concealed or needs to be protected from corrosion.

4. Need to grout under both wythes unless panel is designed to support outer wythe from inner wythe.

5. A solid bottom [eliminating insulation for 1 ft (0.305 m) or so] may reduce the insulating properties.

6. S-clip and expansion anchor connections do not provide much uplift resistance.

2.10.2.2 Panels Connected to Foundations and Floor Slabs (Figs. 2.10.2.2.a and b)

The comments in Section 2.10.2.1 apply to this section also. In addition, the panels are connected to the floor slab with reinforcing bars, coil rods or threaded rods. The connections acting together can provide moment resistance.

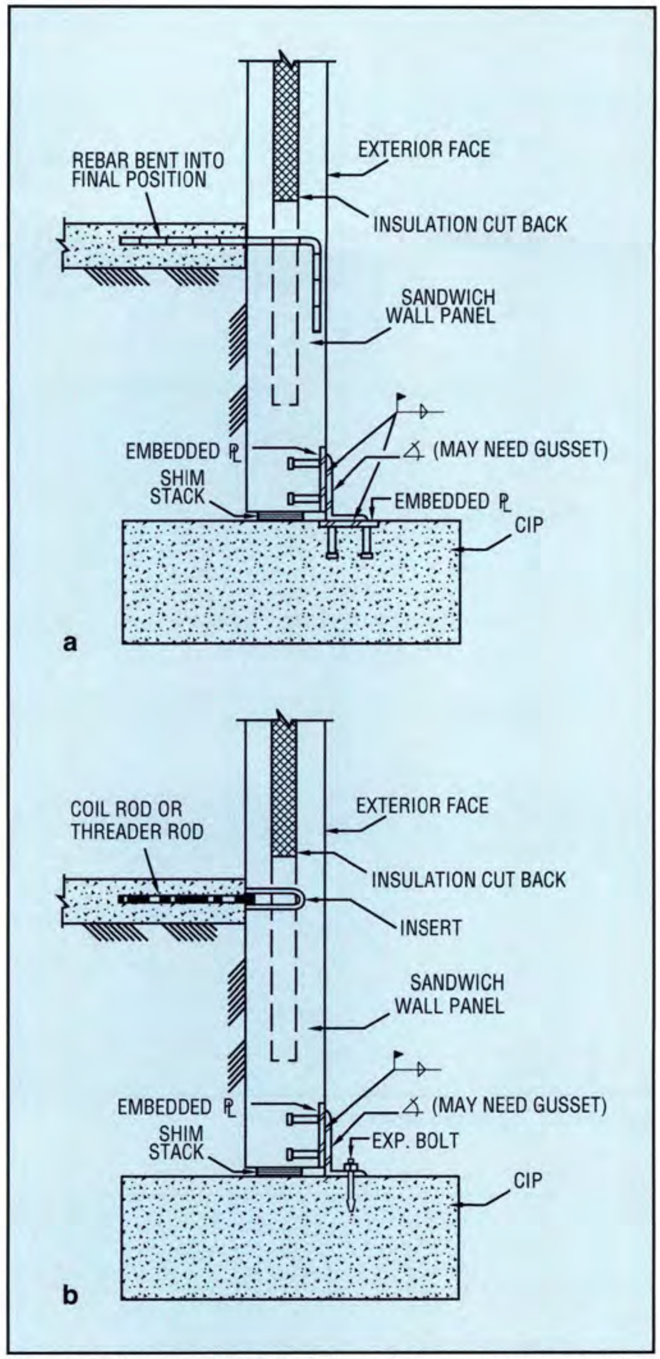
Features:

1. Moment resistance at panel base is achievable from forces developed at the foundation and at the floor level. Some panels may be cantilevered from the base and not connected to structure at top (eliminates wind bracing in the structure).

2. Connection to floor reduces unsupported panel height.

3. Extending panels to footing eliminates cost of forming and pouring cast-in-place concrete walls.

Considerations:



Figs. 2.10.2.2. Sample connections for panel bases used for retaining wall.

1. The force in the floor slab connection may be quite large. Forces due to differential temperature and shrinkage between wythes should be accounted for in addition to wind suction and axial load eccentricity.

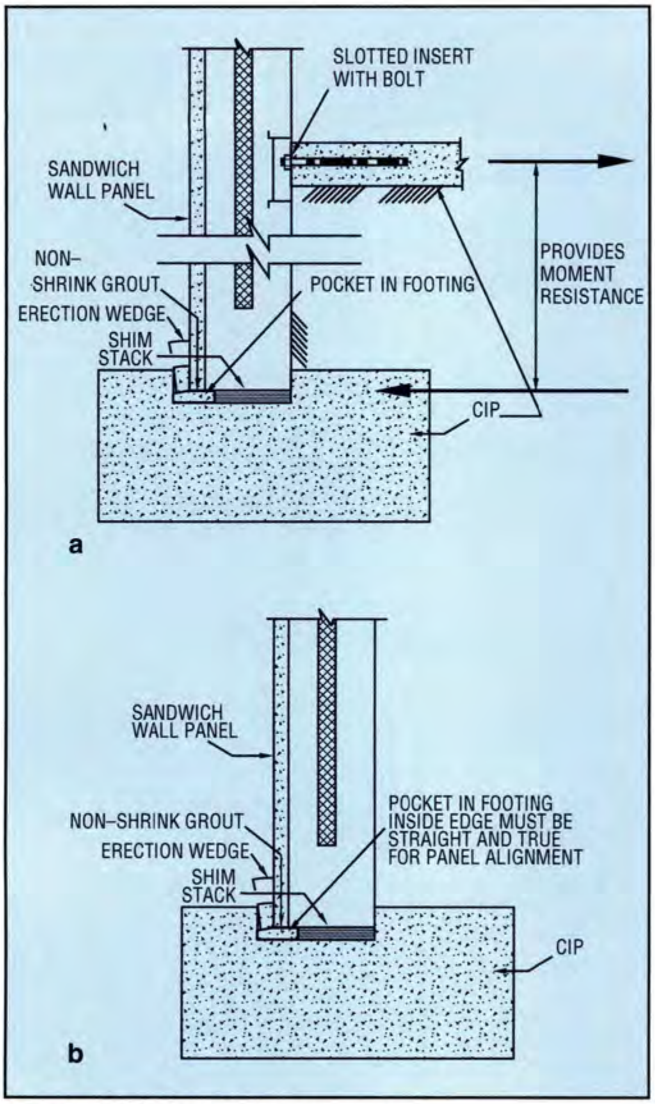
2. Panel moments at the floor slab are resisted by strands that may not be fully developed.

3. Temporary bracing is needed until the floor slab is poured.

4. Care needs to be taken concerning coordination and tolerances between embedded plates in cast-in-place footings and embedded plates in precast panels.

5. Future removal of the floor slab, thus eliminating the connection.

6. Solid bottoms may reduce the thermal efficiency.



Figs. 2.10.2.3. Slotted base connections.

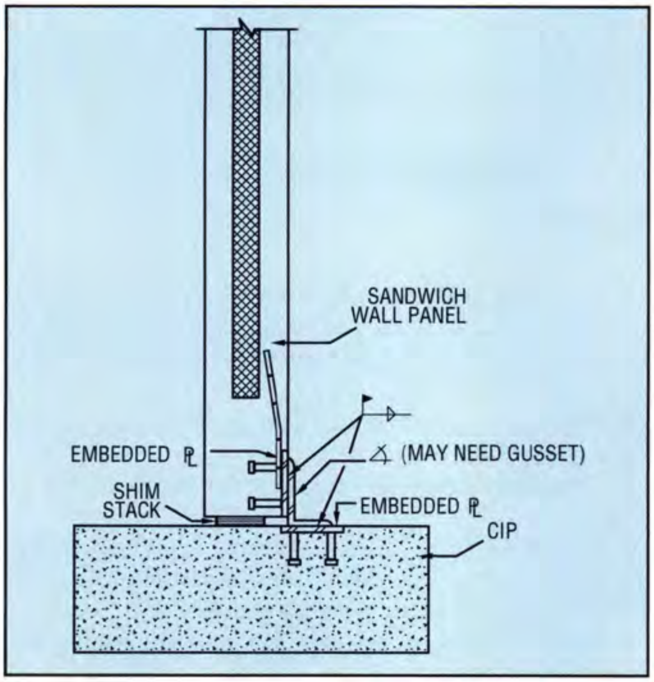


Fig. 2.10.2.4. Shear wall panel to foundation connection.

2.10.2.3 Slotted Connection (Figs. 2.10.2.3.a and b)

The panels are placed in continuous slots cast in the cast-in-place footing. Shims and temporary wedges are used until the slot area is grouted. Panel support during erection is achieved by temporary bracing or attachment to a braced structure.

Features:

1. No embedments. No coordination or tolerance problems with embedments not lining up.

2. Speeds up erection — no welding or bolting at foundation.

Considerations:

1. There is no positive connection to footing, especially for uplift.

2. Stability is dependent on wedging panel in slot before grouting.

2.10.2.4 Shear Wall Panels (Fig. 2.10.2.4)

A welded connection between embedded plates in the cast-in-place foundation and the precast panel is used. The panels and foundations and their connections are designed for the uplift forces and structural integrity requirements. Panel support during erection is achieved by temporary bracing or attachment to a braced structure.

Features:

1. A simple, economical method to resist wind and seismic forces.

2. Several panels can be connected together to increase the lateral load resistance and structural stiffness.

Considerations:

1. Care needs to be taken concerning coordination and tolerances between embedded plates in cast-in-place foundations and embedded plates in precast panels.

2. Temporary bracing is needed if the panel is erected before the rest of the structure, or two cranes must be used simultaneously.

2.10.2.5 Panel Bracing (Figs. 2.10.2.5.a and b)

Loadbearing wall panels are usually erected before the adjacent structure because the roof and floor members bear directly or on haunches or ledges on the panels. A common way to temporarily brace the panels until the rest of the structure is constructed is to use pipe braces and concrete deadmen. The deadmen and bracing are moved as the work progresses.

A different form of temporary bracing, available in one system, uses a beam with temporary columns; the roof framing is erected on this system before the panels are erected. As the panels are put up, the beam is welded to the panels and the columns are then removed. Temporary cable guys are usually required to brace the structure until the roof deck diaphragm is in place and all of the panels are erected. There are numerous alternative arrangements that are commonly used, including complete precast framing with, for example, roof double tees. This requires temporary bracing for panels only on the leading edge of the construction.

Features:

1. Bracing systems permit the sandwich walls to support the roof framing and eliminate permanent beams and columns adjacent to the panels.

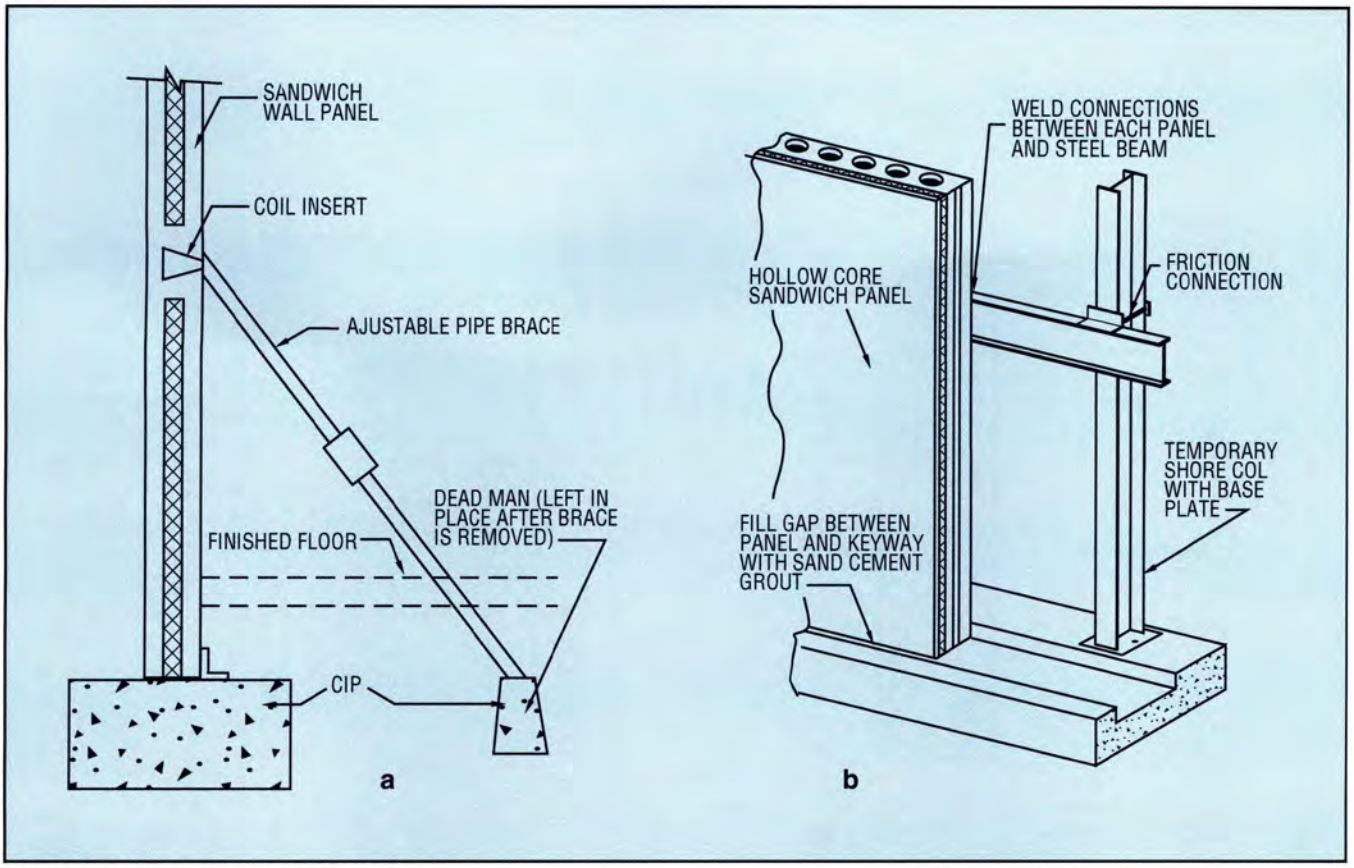


Fig. 2.10.2.5. Temporary panel bracing.

2. Deadmen, pipe bracing and patching inserts are eliminated by the temporary column and beam system.

Considerations:

1. Deadmen are often temporary. Buried deadmen, which may be permanent, need to be low enough so they do not act as supports for the floor slab or parking lot.

2. Inserts for pipe braces may need to be patched after the bracing is removed.

3. Cable guys must be strong enough to resist lateral forces of the building with panels and not just the lightweight and open steel structure.

4. Pipe braced panels need to be erected before perimeter joists are installed.

2.10.3 Panel Top to Roof Connections

Five types of panel top to roof connections are illustrated and discussed:

- Non-loadbearing wall panels to beams or joists
- Loadbearing wall panels to roof joists
- Loadbearing wall panels to roof decks
- Loadbearing wall panels to hollow-core slabs
- Loadbearing wall panels to beams or joist girders

2.10.3.1 Non-loadbearing Wall Panels to Beams or Joists (Figs. 2.10.3.1.a, b, c, and d)

The panels are cast with either embedded weld plates or slotted inserts. Welded connections are designed for wind and/or seismic loads, and to satisfy structural integrity requirements. If the panels are intended to act as shear walls, these connections are designed for the roof diaphragm forces.

Features:

1. Embedded weld plates or slotted inserts together with steel roof members give latitude in erection tolerance.

2. Connections are somewhat flexible and accommodate movements.

Considerations:

1. Roof framing has to be erected before wall panels. If not, the panels need temporary bracing.

2. Solid concrete at the embedded plates reduces the insulating properties.

2.10.3.2 Loadbearing Wall Panels to Roof Joists (Figs. 2.10.3.2.a, b, and c)

Roof joists can either bear directly on the top of the wall panel, in pockets or on steel member ledges or haunches welded to embedded plates in the panels. Panels are designed for the eccentric loading of the roof joists.

Features:

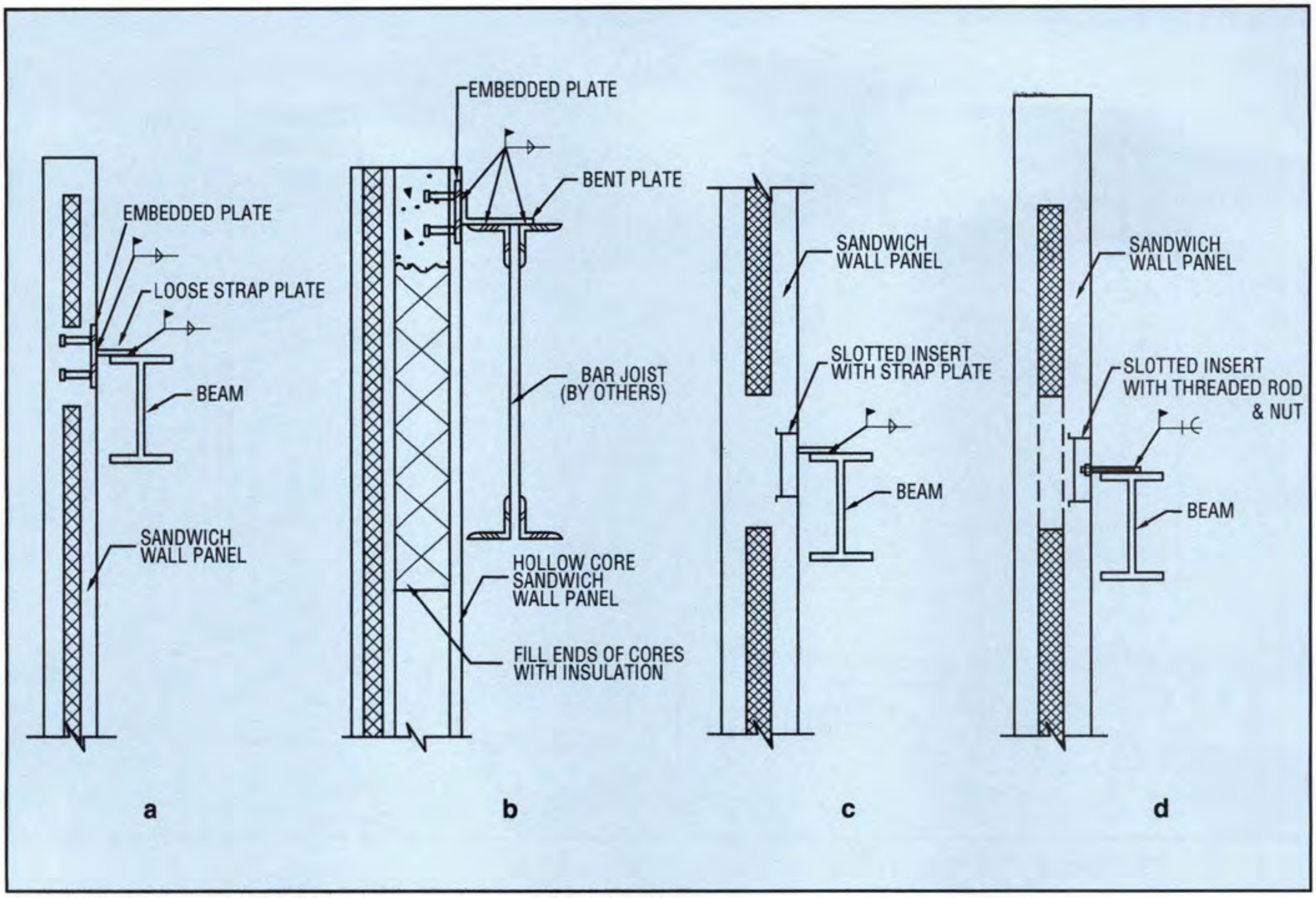
1. Loadbearing panels eliminate exterior beam and column framing.

2. A continuous angle or beam ledge permits variable joist spacing.

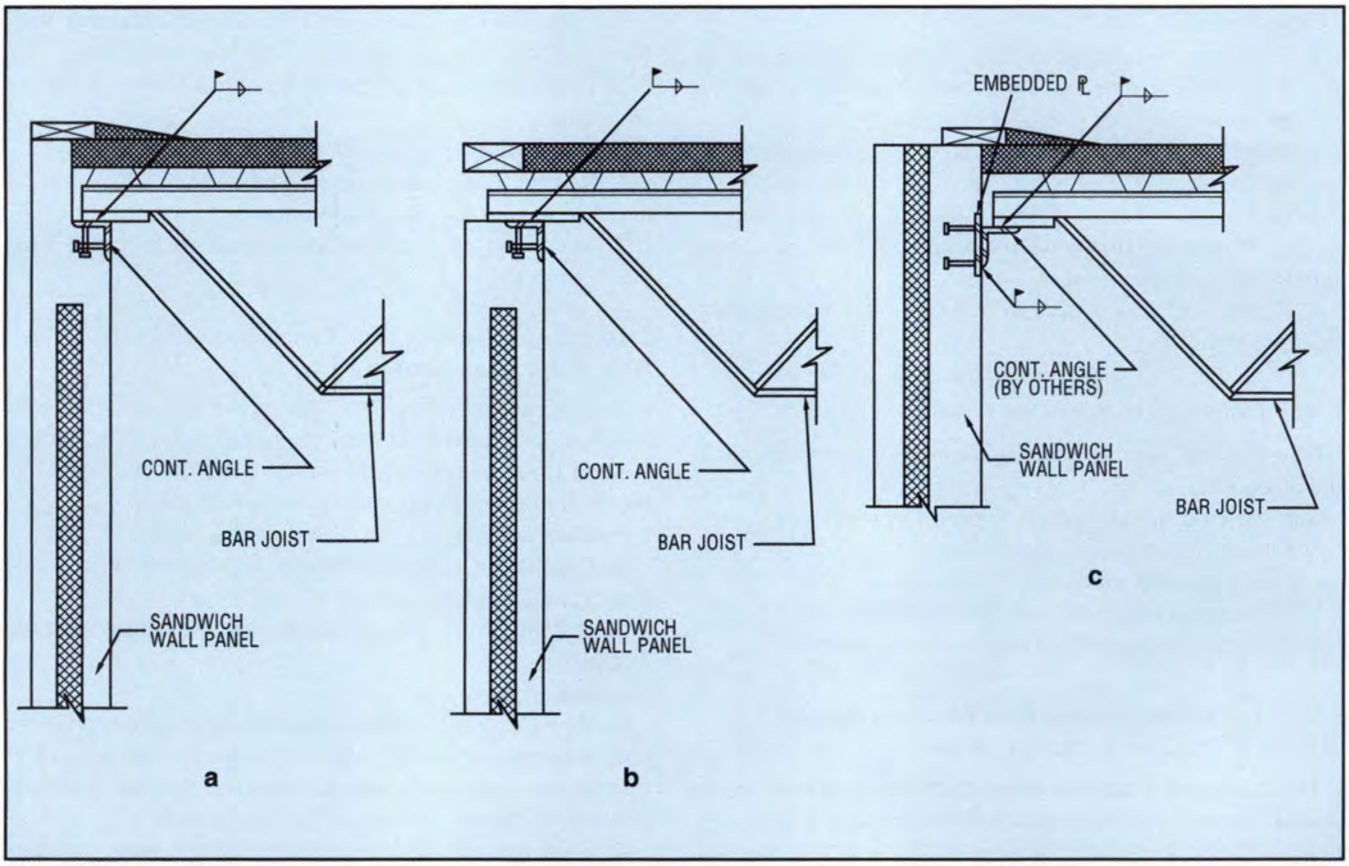
Considerations:

1. Walls have to be erected and temporarily braced before roof framing is erected. Temporary columns can be used to support the edge beam, thus allowing the roof to be erected first and eliminating bracing of the wall panels.

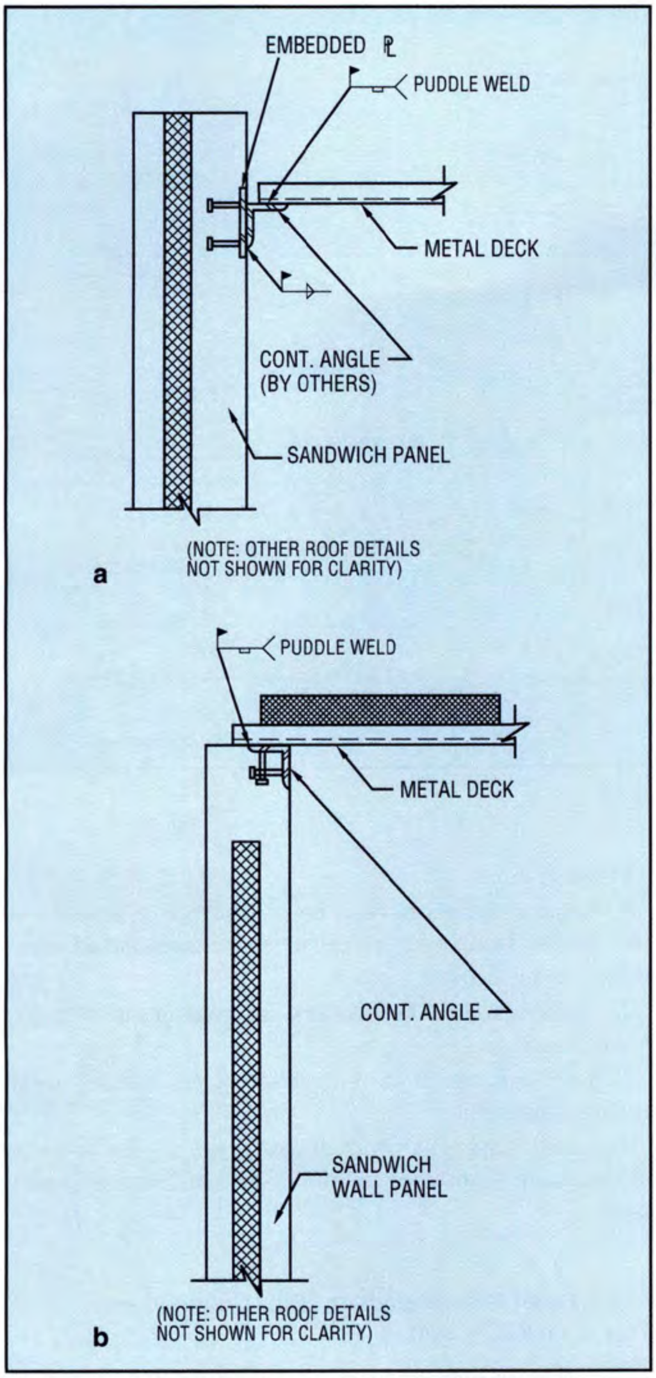
2. Solid concrete at the embedded plates reduces the insulating properties.



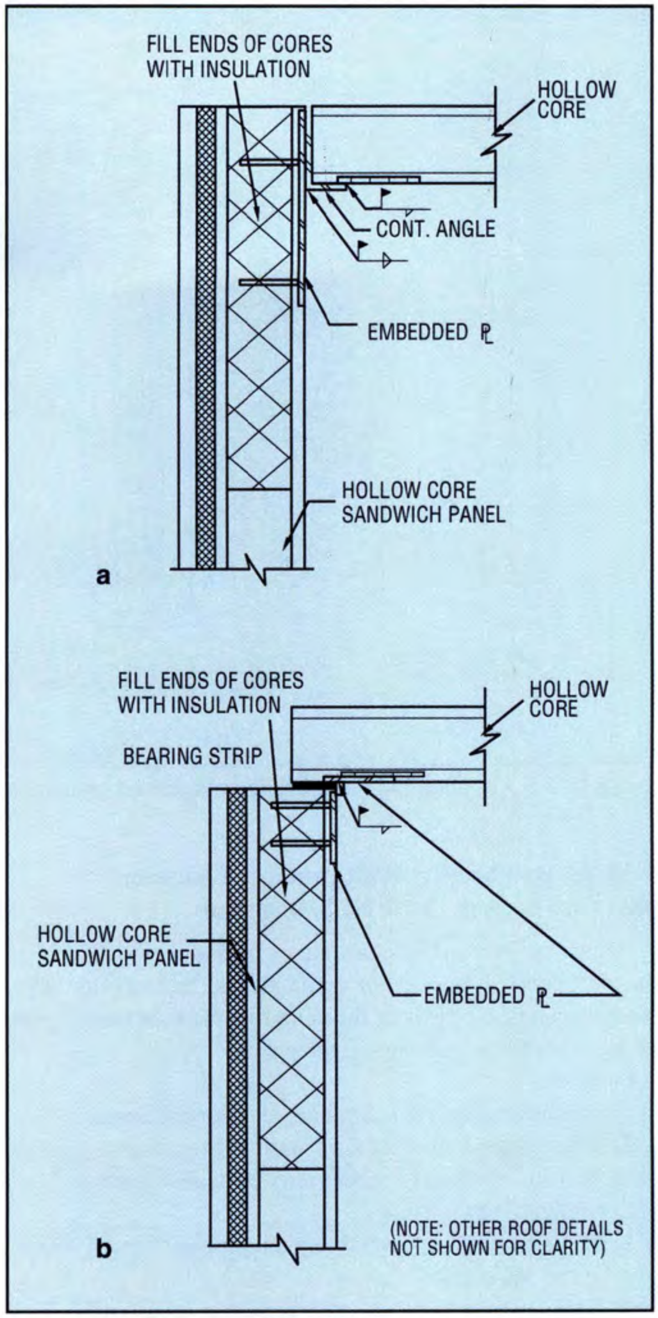
Figs. 2.10.3.1. Non-loadbearing panels to beam or joist connections.



Figs. 2.10.3.2. Loadbearing panels to roof joist connections.



Figs. 2.10.3.3. Loadbearing panels to metal deck roof connections.



Figs. 2.10.3.4. Loadbearing panels to hollow-core slabs.

2.10.3.3 Loadbearing Wall Panels to Metal Deck Roofs (Figs. 2.10.3.3.a and b)

A metal deck or wood deck can either bear directly on top of the wall panel or on the steel member or wood member ledges.

Features:

1. Loadbearing panels eliminate edge beams or joists.

2. Ledge members can be installed to meet roof slopes.

Considerations:

1. Walls have to be erected and temporarily braced before the roof deck next to the wall can be installed.

2. Solid concrete at the embedded plates reduces the insulating properties.

2.10.3.4 Loadbearing Wall Panels to Hollow-Core Slabs (Figs. 2.10.3.4.a and b)

Hollow-core slabs can either bear directly on top of the wall panel or on steel member ledges. Panels are designed for the eccentric loading of the hollow-core slabs.

Features:

1. Loadbearing panels eliminate edge beams or joists.

2. Ledge members can be installed to meet the slope of hollow-core slabs.

Considerations:

1. Walls have to be erected and temporarily braced before hollow-core slabs are erected.

2. Solid concrete at the embedded plates reduces the insulating properties.

3. Special embedded plates in hollow-core slabs are required.

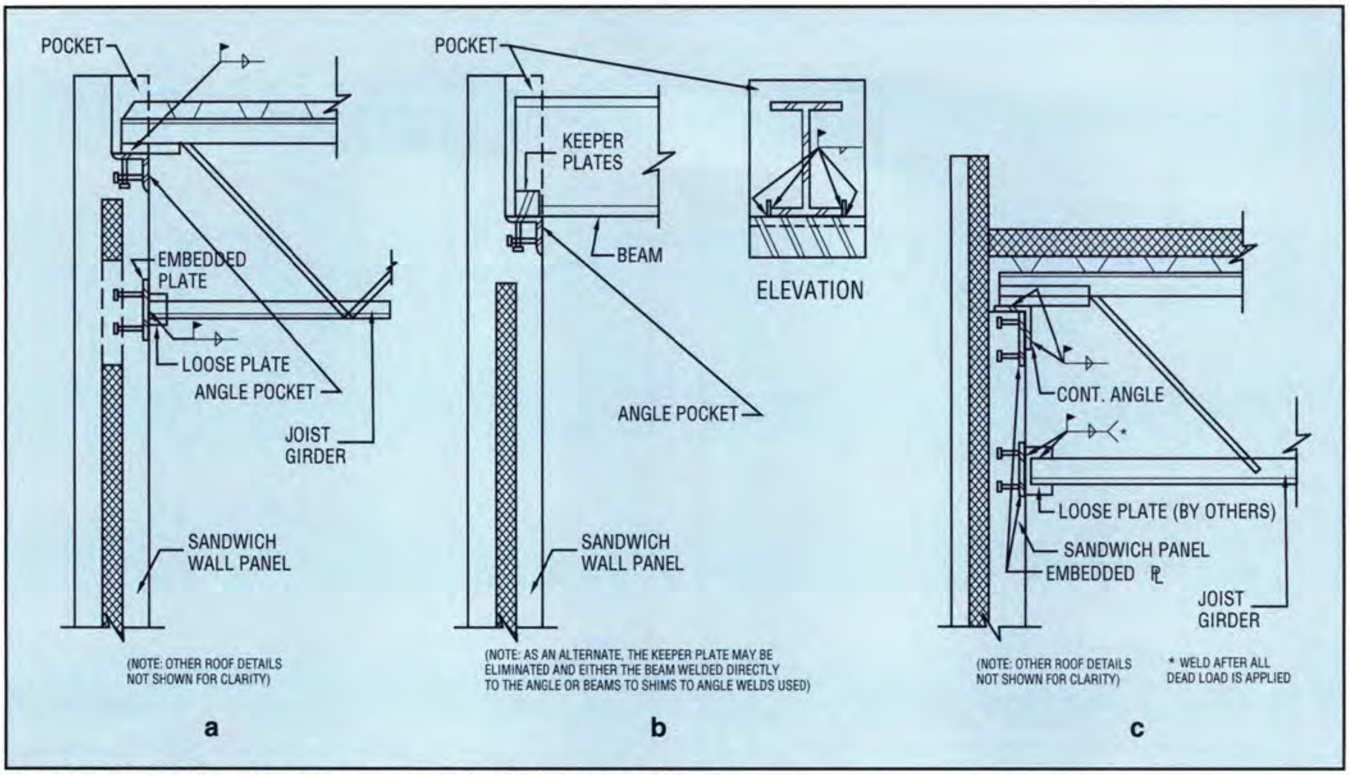


Fig. 2.10.3.5. Loadbearing panels to beams or joist girders connections.

2.10.3.5 Loadbearing Wall Panels to Beams or Joist Girders (Figs. 2.10.3.5.a, b, and c)

Beams or joist girders can either bear directly on top of the wall panel in pockets or on haunches. Beams with large reactions should bear near the center of the structural wythe in order to reduce bending on the panel.

Features:

1. Loadbearing panels eliminate exterior columns.

2. A pocketed connection eliminates the need for haunches and eliminates eccentricity of beam reaction.

Considerations:

1. Walls have to be erected and temporarily braced before roof beams are erected.

2. Solid concrete at the bearing area or at embedded plates reduces the insulating properties.

2.10.4 Panel to Lintel Beam Connections (Figs. 2.10.4.a, b, c, and d)

When door openings are the entire panel width or wider, lintel beams are usually employed to support the panels above the opening. The lintel system and the connections can be designed to minimize the crane setting time. Alternatively, haunches on the full height panels with pockets in the door panels may be used (see Fig. 2.10.4.b). Door panels can be connected to adjacent panels with weld plates but the crane setting time will be longer. The design of lintel beams and their connections must account for the torsion of eccentric loads as well as for flexure and shear.

Features:

1. Quick, easy erection is attained; tolerances are adequate.

2. Wide openings can be achieved with economical framing and connections.

Considerations:

1. Adjacent panels have to be erected before lintels and door panels. Temporary bracing of door panels and adjacent panels can be difficult.

2. Torsion on lintel beams can cause rotations if not properly designed.

3. Solid concrete at the embedded plates reduces panel insulating properties.

4. Careful thought needs to be given to the bowing and restraint to bowing of the door panel and adjacent panels.

2.10.5 Panel to Intermediate Floor Connections (Figs. 2.10.5.a, b, and c)

Loadbearing panels can support intermediate floors or mezzanines with continuous ledge members (see Fig. 2.10.5.a). Nominal or lateral connections (see Figs. 2.10.5.b and c) as well as bearing connections to intermediate floors need to be designed for the forces that result from restraining panel bowing.

Features:

1. Loadbearing panels eliminate exterior beam and column framing for mezzanines and intermediate floors.

2. Intermediate connections reduce bowing and unsupported height.

Considerations:

1. Large forces may develop due to restraint of panel bowing.

2. Any eccentricity of bearing connections that will cause torsion on support beams needs to be considered.

3. Solid concrete reduces insulating properties.

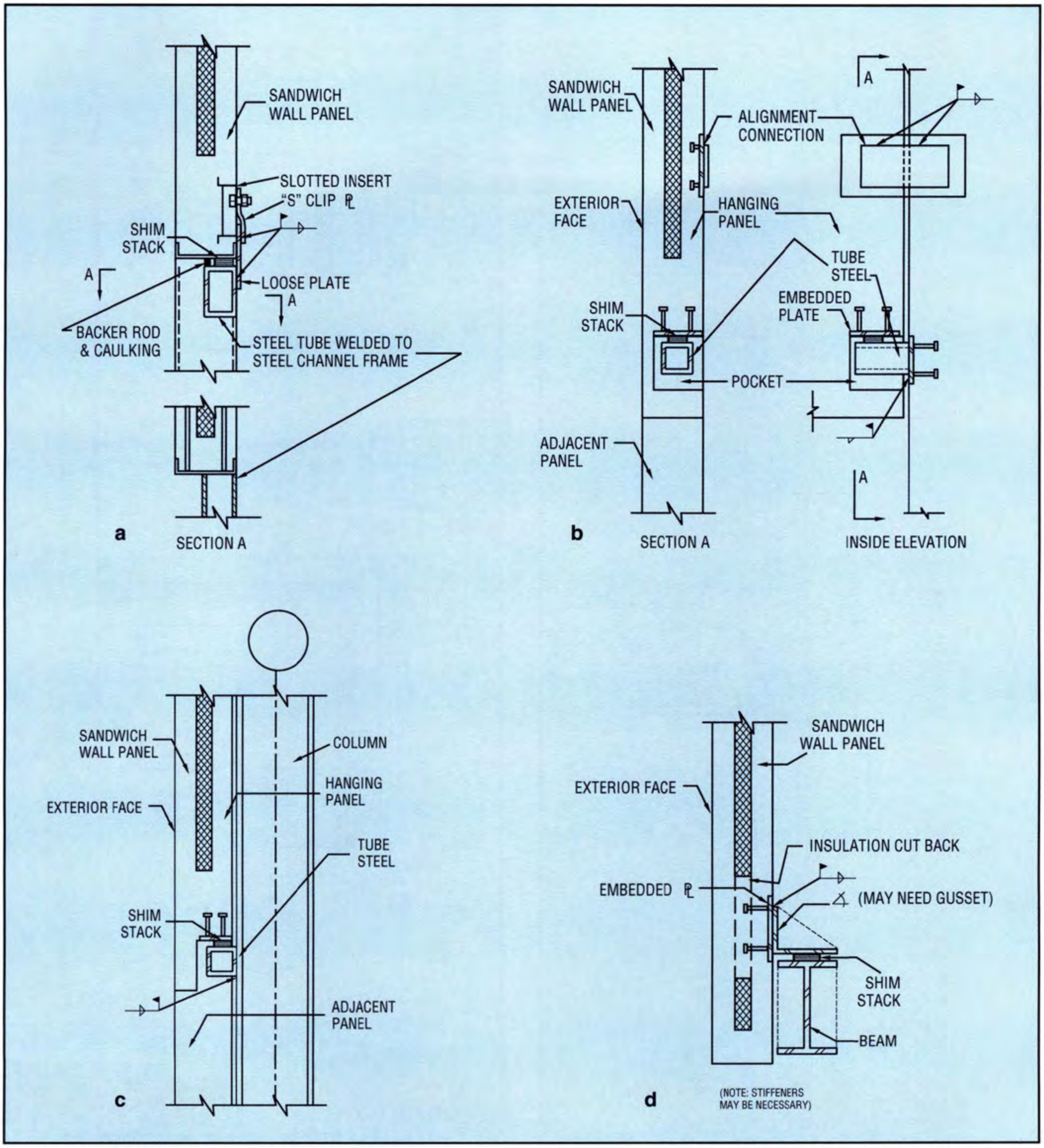


Fig. 2.10.4. Panel to lintel beam connections.

2.10.6 Corner Panel Connections (Figs. 2.10.6.a, b, c, d, e, f, and g)

At corners, the bowing of panels perpendicular to each other may cause unacceptable separation and possible damage to the joint sealant. It may be desirable to restrain bowing at the corners with one or more connections between panels or to a corner column (see Section 3.3.2, PCI Design Handbook, Fourth Edition[2]). Separate corner panels can also be used to help the bowing separation problem (see Figs. 2.10.6.b, c, d, e, f and g). Mitered corners should have a quirk detail, usually 1 x 1 in. (25.4 x 25.4 mm) (see Figs. 2.10.6.b, c, d and g).

Features:

1. Connecting panels together or using special corner panels eliminates separation and alignment problems.

2. Corner columns can be eliminated if sandwich panels are loadbearing.

Considerations:

1. Corner connection forces due to bowing can be high.

2. Connections are rigid.

3. Solid areas reduce the insulating properties.

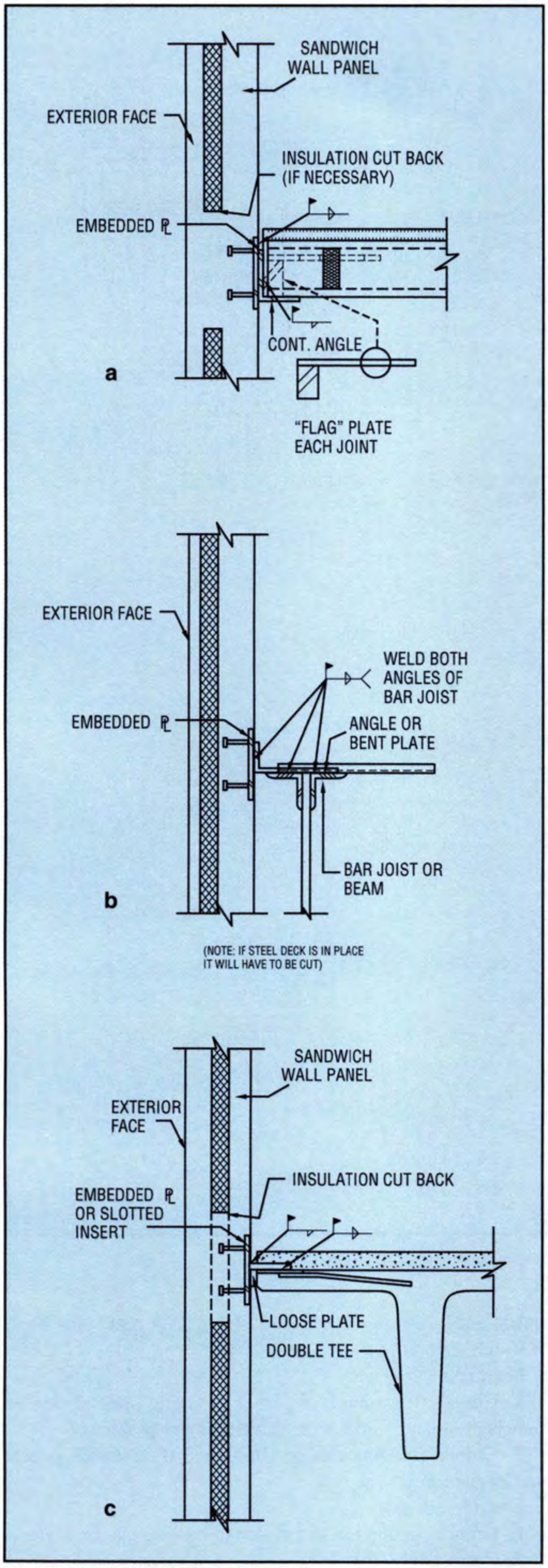


Fig. 2.10.5. Panel to intermediate floor connections.

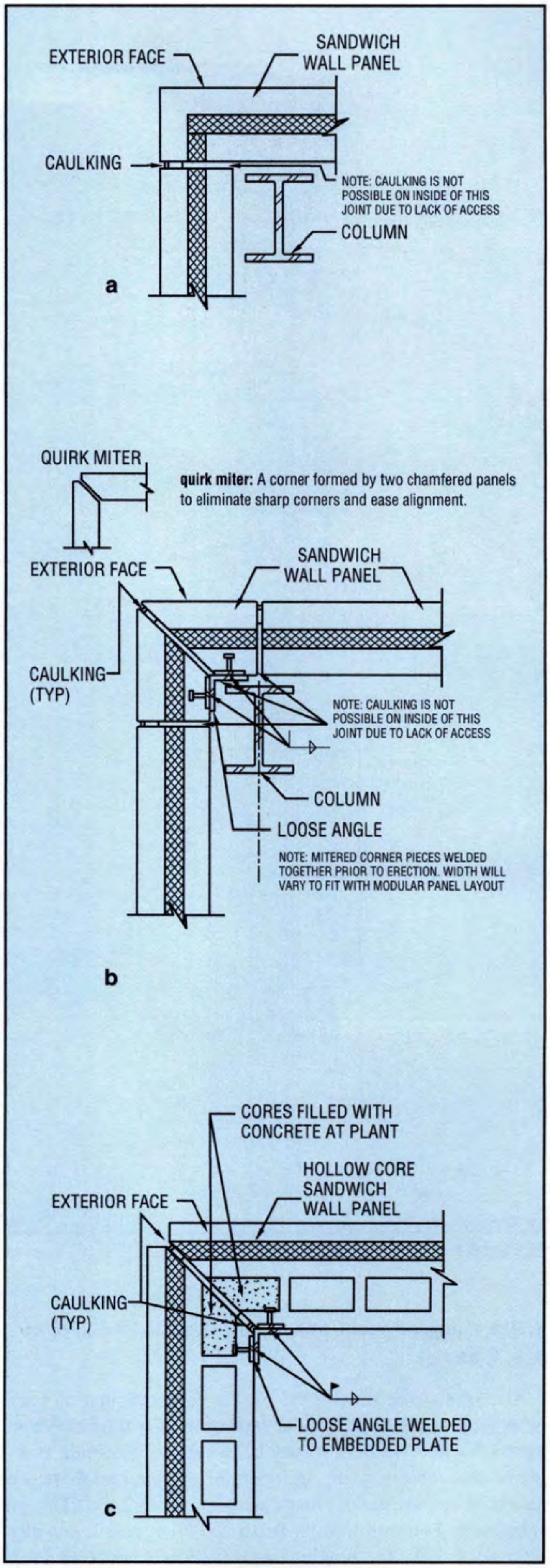


Fig. 2.10.6. Corner panel connections.

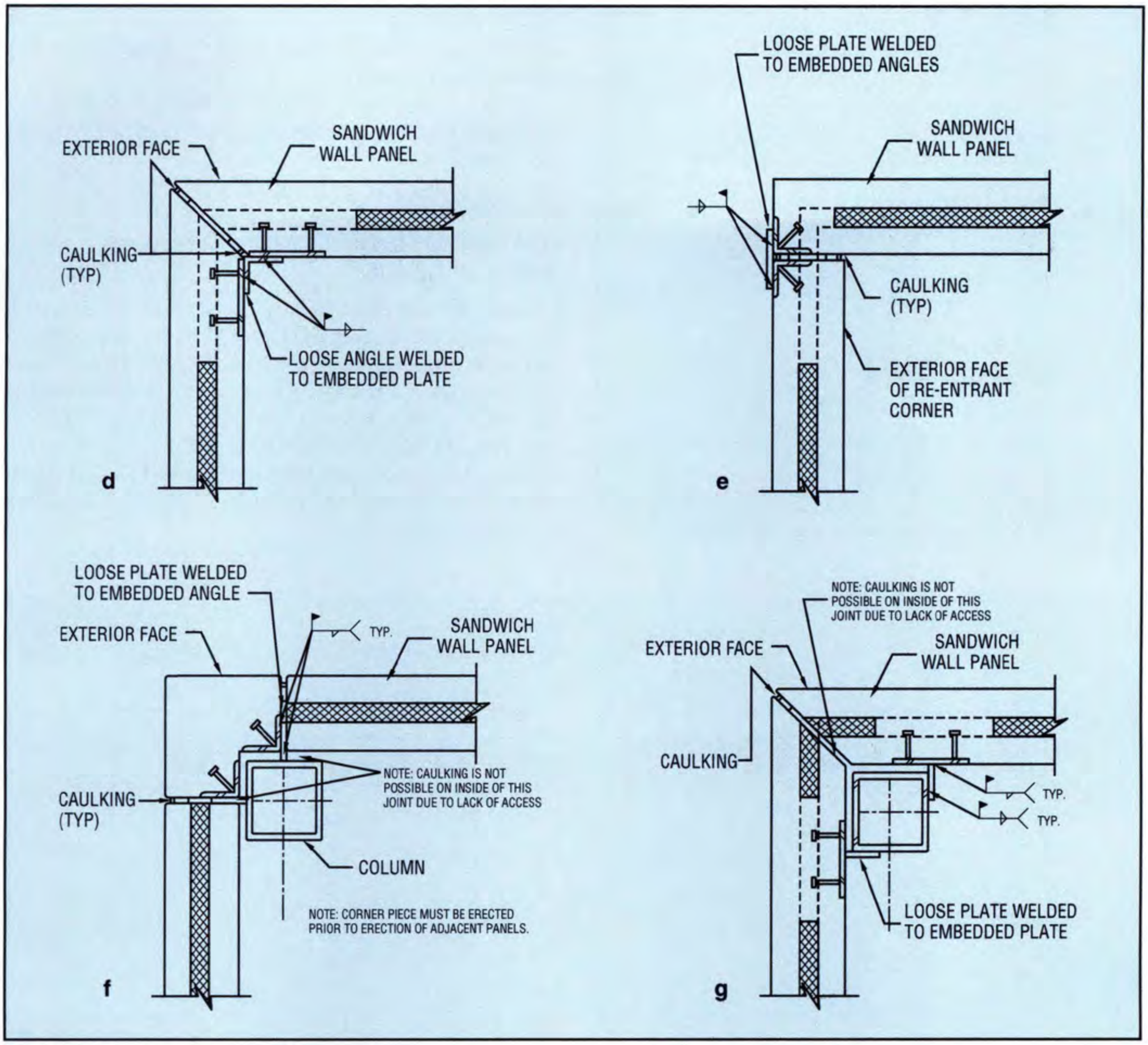


Fig. 2.10.6 (cont.). Corner panel connections.

2.10.7 Panel-to-Panel Connections (Figs. 2.10.7.a, b, c, d, e, f, and g)

Four types of panel-to-panel connections are illustrated:

- Panel-to-panel shear wall connections
- Panel-to-panel alignment connections
- Panel-to-panel joint caulking
- Panel-to-panel, horizontal joint connections

2.10.7.1 Panel-to-Panel Shear Wall Connections (Fig. 2.10.7.a)

In general, sandwich panels are not tied to each other with rigid connections in order to prevent the buildup of volume change forces. In some cases, however, panels need to be connected together to increase their shear wall resistance.

Features:

1. Panels are a simple, economical method to resist wind and seismic forces compared to cross-bracing or moment resistant frames in the structure.

2. Several panels can be connected together to increase the lateral load resistance.

Considerations:

1. Rigid, unyielding connections.

2. Solid areas reduce the insulating properties.

2.10.7.2 Panel-to-Panel Alignment Connections (Figs. 2.10.7.b and c)

Depending on their height, sandwich wall panels may need more than one or two connections between panels to help alignment during erection and to control differential bowing in the finished structure. If possible, the alignment connections should allow panels to move horizontally, parallel to the wall face.

Features:

1. Allows erector to align panels.

2. Differential bowing between adjacent panels is reduced.

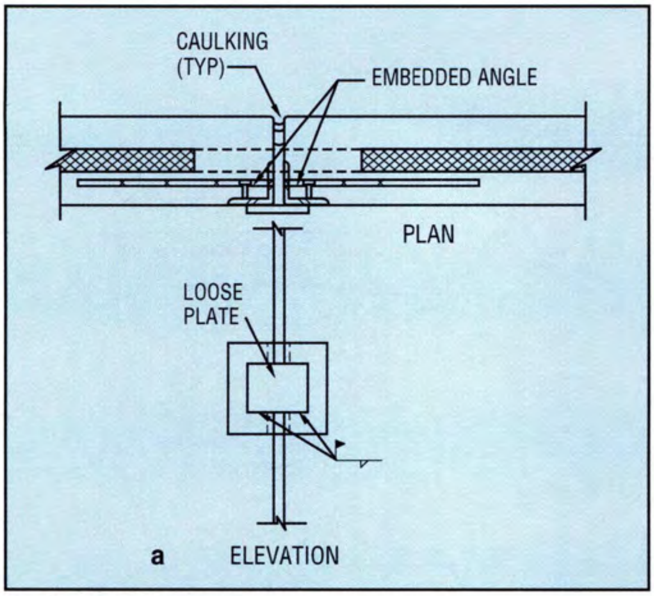


Fig. 2.10.7.a. Panel-to-panel shear wall connections.

Considerations:

1. If connections are rigid, volume change forces may cause connection distress.

2. If two panels have a differential bow, a large force may be required to pull them together resulting in large connection forces.

2.10.7.3 Panel-to-Panel Caulking Connections (Figs. 2.10.7.d and e)

Sandwich wall panel joints are normally caulked on both the outside and the inside surface. Most building codes do not require these joints to be protected against fire. In special cases when a fire rating is required, insulating materials can be used as referred to in Section 8.6 of PCI MNL 124, Design for Fire Resistance of Precast Prestressed Concrete[4] and in Section 9.3.6.6 of the PCI Design Handbook, Fourth Edition.[2] An example is presented in Section 2.13 of this report.

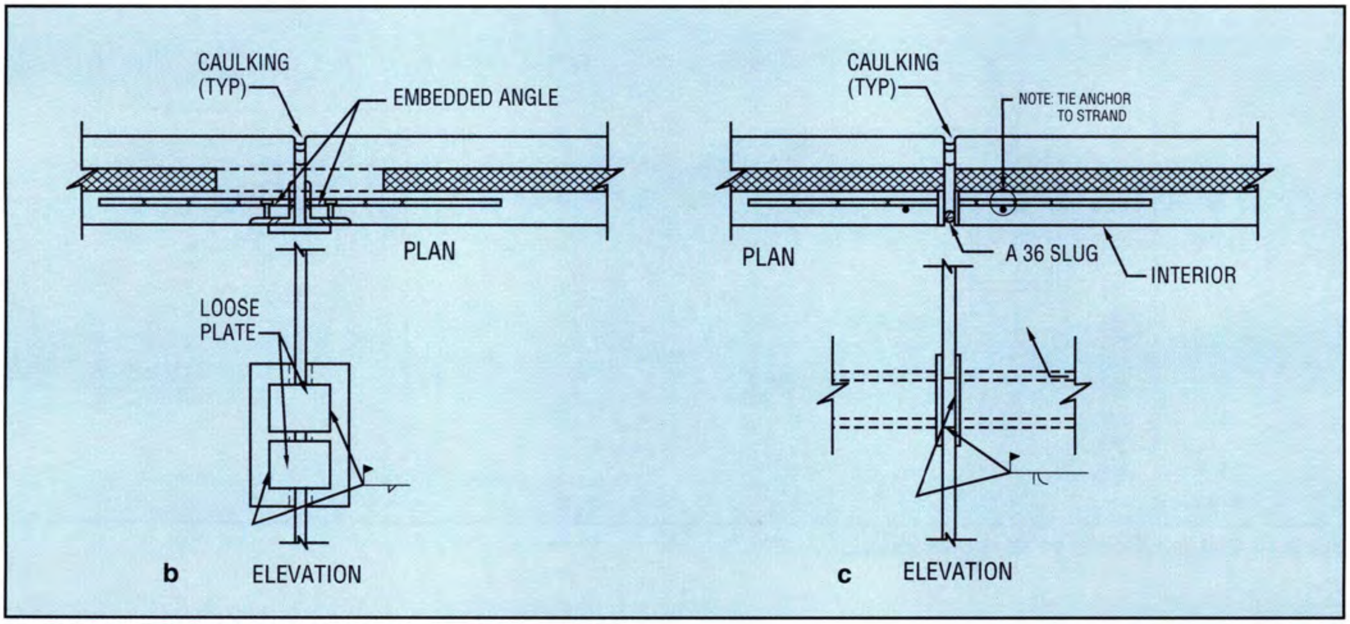


Fig. 2.10.7.b and c. Panel-to-panel alignment connections.

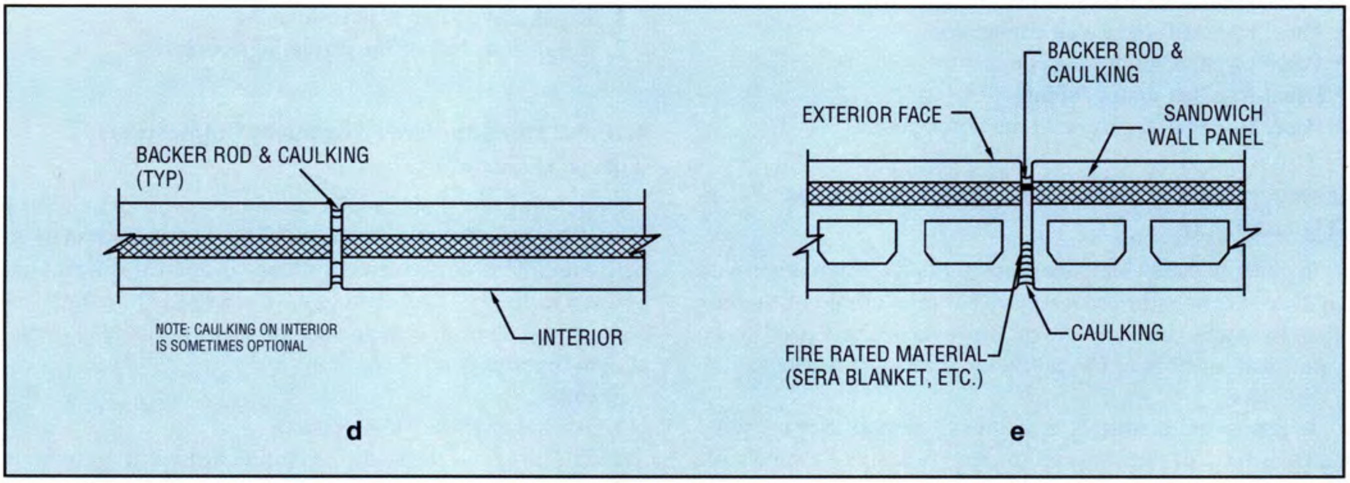


Fig. 2.10.7.d and e. Panel joint caulking.

2.10.7.4 Panel-to-Panel, Horizontal Joint Connections (Figs. 2.10.7.f and g)

Sandwich panels placed vertically above each other can either be supported individually from the structure or be stacked using shims and grout. Connection plates between panels can be used for alignment or to transfer lateral forces.

2.11 DETAILING CONSIDERATIONS

2.11.1 Joints

The normal detailed joint width between adjacent panels is $^1/_2$ in. (13 mm). This allows for some tolerances in the as-manufactured width of the panels. A $^1/_2$ in. (13 mm) joint is also a good dimension for the installation of the caulking. Although detailed as $^1/_2$ in. (13 mm), actual in-place joint widths may vary from $^1/_4$ to $^3/_4$ in. (6.3 to 19 mm). These joint widths are easily caulked. Joint width tolerances are given in the PCI Design Handbook, Fourth Edition (see Fig. 8.3.4).[2]

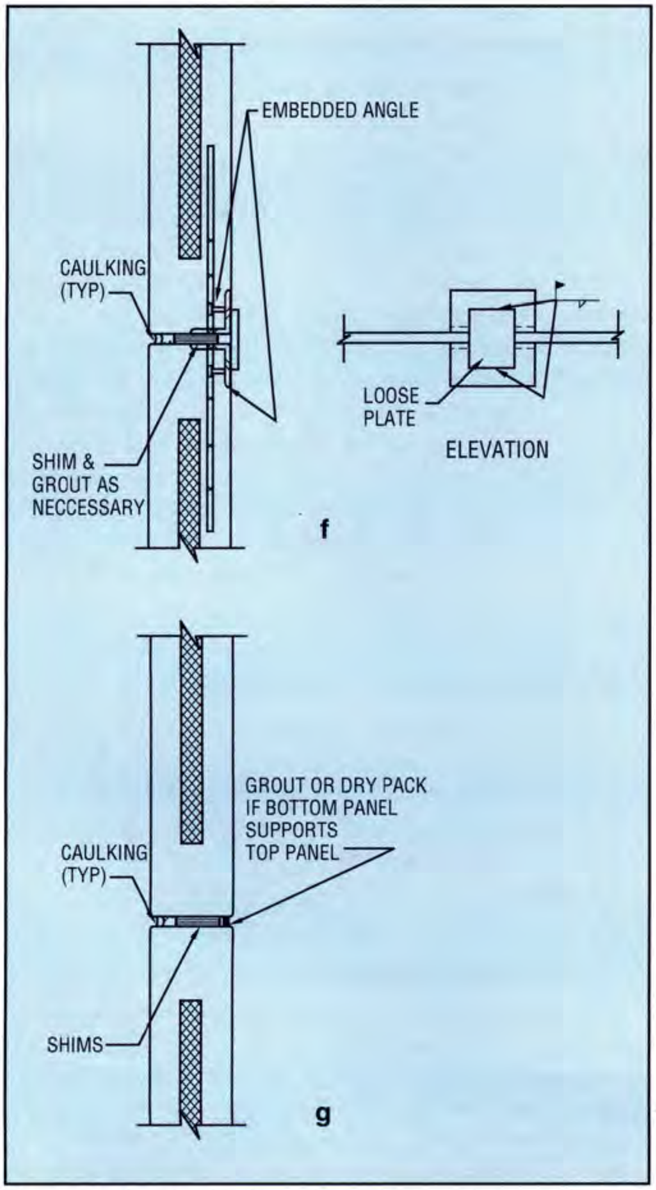


Fig. 2.10.7.f and g. Horizontal panel-to-panel connections.

Corner joints are normally detailed as either butt joints or quirk miters. Butt joints are more easily fabricated because there is less formwork involved. When butt joints are used, it is important that the detailer either terminate the insulation several inches from the panel edge adjacent to the joint or detail the insulation to turn the corner to meet with the insulation in the panel on the adjacent elevation. Quirk miters must be formed, finished and erected with care so that the final appearance is acceptable. Section 2.10.6 discusses other corner joint considerations.

2.11.2 Clearances

As recommended in the PCI Design Handbook, Fourth Edition (see Table 3.2.3),[2] the normal clearance detailed between sandwich panels and the structure is 1 in. (25.4 mm). Any detailed clearance less than this value may cause problems. The problems are generally due to errors in the plumb or location of the structure, not because of the as-built panel dimensions. Usually, panels are erected plumb and straight, but in some cases they must be erected to follow the structure; as a result, the panels may not be plumb or straight. The 1 in. (25.4 mm) clearance usually allows for necessary tolerances. Clearances greater than 1 in. (25.4 mm) may allow for more tolerance in the structure but will cause greater load eccentricities in the sandwich panel connections. In some special cases, such as sandwich panels on a concrete frame, larger clearances [2 in. (51 mm)] may be necessary.

2.11.3 Interactions With Other Trades

An important consideration in relation to other trades is that all sandwich panels will move or bow during the life of the structure. Anything connected to them must be able to accommodate such movement.

An example is a block wall constructed perpendicular to a composite sandwich panel. The panel may bow outwards on a daily basis, whereas the block wall will probably not be as sensitive to daily temperature changes. If the joint between them is caulked, the caulking will eventually fall out. In lieu of a caulked joint, a sealed slip joint can be used. It is important that the architect be aware of this phenomenon so that a suitable detail can be provided.

Similar occurrences occur at mezzanines that are not connected to the panels, dropped ceilings that may have their edge support attached to the panel (the edge may move sufficiently to permit ceiling tiles to fall out), and joints between slabs on grade and panels that extend down past the slab to the footing. All of these joints, and others similar to them, will open and close as the panels bow.

Because the sandwich panel manufacturer is the one most experienced with the behavior of panels, it is good practice that the precaster be consulted together with the general contractor and professional engineer early in the project. The manufacturer can also assist the general contractor in coordinating these details with other trade shop drawings.

2.11.4 Corners

At corners of buildings, composite and semi-composite panels will bow in orthogonal directions ("fishmouth") if not restrained. If the fishmouthing is not eliminated, the corner caulking material may fail. This phenomenon is described in the PCI Design Handbook, Fourth Edition (Section 3.3.2).[2] Connection details are discussed in Section 2.10.6 of this report. Some caulking failure at the joint next to the corner has also been observed. The use of appropriate connections on the last and second last panels on an elevation has been successful in eliminating this problem. These panel-to-panel connections should be detailed to minimize significant volume change restraint forces, as discussed in Section 2.10.7.2.

2.11.5 Openings

Openings in panels may be detailed as being completely contained within the panel (punched) or as blockouts in the panel sides, top or bottom. In some cases, wall openings are formed by hanging a partial panel from two adjacent panels. These latter panels are commonly called "hanging panels" and are discussed in Section 2.11.6.

Panel openings should have re-entrant corners reinforced with diagonal bars to limit the width of corner cracks. This reinforcement should be in both wythes. ACI 318-95[1] calls for two diagonal #5 bars, but in the smaller wythe thickness it may only be possible to use #3 bars because of clearance considerations. Punched openings located near one edge of a panel are very susceptible to cracking and it is sometimes advisable to eliminate the insulation in this area and reinforce the side with additional reinforcement.

Openings formed by blockouts in the panel edges cause handling problems. Twisting of the panels during stripping or erection may cause corner cracks. Some producers utilize strongbacks to reduce the twisting effect.

In some cases, panel openings are so extensive that an insufficient section modulus is left to keep the applied stresses below the limits specified in ACI 318-95,[1] Chapter 18, or even below the cracking limit. In these cases, the designer can only satisfy the ultimate criteria. This is normally accomplished by the use of additional longitudinal mild steel reinforcement. The addition of supplemental prestressed strand is not practical because long line production methods are normally used.

2.11.6 Hanging Panels

Hanging panels are those that are hung from lintels (concrete or steel) or from adjacent panels. The details for these panels must consider the support of the panel and the bowing characteristics of the adjacent panels. The connections for these panels are typically classified as either bearing or alignment connections.

Because of the indeterminacy of the load path when more than two bearing connections are used, designers normally limit the number of bearing connections to two per panel. Bearing connections are typically detailed to allow for larger than normal tolerances because of the difficulty associated with designing and installing a field retrofit if the bearing connection or its support is mislocated. For example, if the connecting hardware is 4 in. wide and 6 in. high (102 and 152 mm), the embedded panel plate might be detailed 8 in. wide and 10 in. high (203 and 254 mm). Erection drawings should clearly show the erector which of the connections are bearing and instruct him as to the extent of welding required before the panel self weight can be released from the crane.

Hanging panels that are rigidly attached or supported from adjacent panels will move with the adjacent panels. If the hung panel is also firmly attached to the structure, structural distress in the connections may occur. Another case to consider is when a similar situation as above exists and the hung panel is supported by a structural lintel. The hung panel will tend to move with the adjacent panels and pull the lintel with it. Sliding bearing connections have been successfully used to allow for this movement.

2.12 REINFORCEMENT REQUIREMENTS

2.12.1 Minimum Transverse Reinforcement

ACI 318-95, Section 16.4.1,[1] states that "for one-way precast, prestressed wall slabs not wider than 12 ft (3.66 m), requirements for shrinkage and temperature reinforcements may be waived." Because precast/prestressed panels are made in widths suitable for shipping and because much of the transverse shrinkage can occur prior to erection, internal volume change stresses do not build up in the panels at the same magnitudes as longer cast-in-place walls.

Satisfactory results have been achieved for many sandwich wall panel projects with panels that have no mesh or other transverse reinforcement except possibly #3 or #4 bars at the top and bottom and at the lifting points. For precast, non-prestressed sandwich panels, the recommendations of the PCI Design Handbook, Fourth Edition (Section 5.2.4),[2] are generally followed. One major recommendation is that a minimum reinforcement ratio of 0.001 be provided in each direction of the panel.

2.12.2 Additional Reinforcement for Handling

The analysis and design procedures for wall panels contained in Chapter 5 of the PCI Design Handbook, Fourth Edition,[2] are normally used for the determination of flexural stresses during panel handling. This analysis generally results in additional transverse reinforcement detailed at the lifting points.

2.12.3 Prestressed Release Reinforcement

Most producers provide additional transverse reinforcement at the ends of sandwich panels. This reinforcement is provided to limit any longitudinal cracks that may result from bursting stresses when the prestressing force is transferred to the member. This reinforcement is generally most effective when it is placed between the strand and the exterior face of each wythe.

2.13 FIRE RESISTANCE

The fire endurance of sandwich wall panels can be estimated by using the procedure as outlined in the PCI Design Handbook, Fourth Edition,[2] and in PCI MNL 124.[4] Some panel systems have been tested to provide these data. Once the fire endurance of each individual element is known, the estimated fire endurance of the assembly can be calculated using the formula:

$$R = (R_1^{0.59} + R_2^{0.59} + R_3^{0.59})^{1.7}$$

To show the application of the above equation, two examples are given.

Example 1 — Calculate the estimated fire endurance of a panel consisting of a 3 in. (76 mm) inside wythe, 2 in. (51 mm) of expanded polystyrene insulation, and a 2 in. (51 mm) exterior wythe. The concrete contains siliceous aggregate.

R_1 = fire endurance of 3 in. (76 mm) wythe = 45 minutes

R_2 = fire endurance of 2 in. (51 mm) polystyrene = 5 minutes (as determined from fire tests; reference PCI Design Handbook)

R_3 = fire endurance of 2 in. (51 mm) wythe = 26 minutes

Substituting the above values in the given equation:

$$\begin{aligned} R &= (R_1^{0.59} + R_2^{0.59} + R_3^{0.59})^{1.7} \\ &= [(45)^{0.59} + (5)^{0.59} + (26)^{0.59}]^{1.7} \\ &= 148 \text{ minutes} = 2 \text{ hours } 28 \text{ minutes} \end{aligned}$$

In general, the joints between sandwich wall panels do not require special treatment. PCI MNL 124[4] contains discussion related to the treatment of joints. Local building codes and MNL 124 should be referenced for specific project requirements. Should special joint treatments be required, interpolation of the graphic values of MNL 124 is accepted practice.

Example 2 — Determine the fire endurance of a 1/2 in. (13 mm) wide butt joint between 6 in. (152 mm) thick panels utilizing a 1 in. (25.4 mm) thickness of ceramic fiber blanket.

From Fig. 8.5 of MNL 124,[4] the fire endurance of a 1 in. (25.4 mm) joint is approximately 1.7 hours and the fire endurance of a 3/8 in. (9.5 mm) joint is approximately 3 hours.

By interpolation, the estimated fire endurance of a 1/2 in. (13 mm) wide joint is:

$$\begin{aligned} R &= 3 - [(3 - 1.7)(0.5 - 0.375)/(1 - 0.375)] \\ &= 2.74 \text{ hours} \end{aligned}$$

CHAPTER 3 — INSULATION AND THERMAL PERFORMANCE

3.1 GENERAL INFORMATION

Concrete sandwich wall panels provide the best structural insulating system by positioning the most vulnerable part of the assembly, the insulation, between two durable layers of concrete. With this insulating technique, construction time is reduced while the owner receives a durable, low maintenance, fire resistant wall assembly that delivers the highest R-value per unit cost.

Parallel to the development of sandwich walls by the concrete industry is an urgency in the construction industry to develop procedures to calculate the actual performance characteristics of building envelopes. This move to building energy performance is manifested in the most recent ASHRAE standard published in cooperation with the Department of Energy (DOE) and the Environmental Protection Agency (EPA). This document, better known as ASHRAE Standard 90.1,[5] based on the previous Standards 90-A and 90-75, sets strict compliance guidelines for building design and calculation of performance for the entire constructed facility. This standard focuses on all new construction, covering most building types. As a standard, 90.1 has been adopted into the local building codes of all 50 states as well as the three national building codes, BOCA, SSBCI, and ICBO. The final form of this standard is a part of the New Model Energy Code, also known as the Energy Policy Act of 1992.

ASHRAE Standard 90.1[5] is an adopted energy language that contributes to the initial design, the analysis of a design, the construction, and the operation of new facilities. Building codes based on these latest revisions to energy performance now include tighter minimum U-values for each assembly contributing to a building's envelope. This effect has mandated the improvement of calculation procedures, energy testing, and material production. Energy calculations have been adopted to benefit the owner in selecting envelope assemblies as well as benefiting the producer in marketing a quality product.

These calculation procedures are covered in Section 3.4. In addition to requiring tighter performance in every assembly contributing to the envelope, the energy codes have also reached to the manufactured materials used to produce the given assemblies. Insulation materials have been significantly affected by these code requirements and are discussed in Section 3.2.

3.2 INSULATION TYPES

Although there are many insulation types on the market today, insulated concrete sandwich walls utilize a cellular (rigid) insulation because it provides those material properties that are most compatible with concrete. These material properties include moisture absorption, dimensional stability, coefficient of expansion and compressive and flexural strengths. The selection of the insulation type to enhance energy performance is as important as the reinforcement needed to enhance structural performance. Depending on site location, climate variables, and operating conditions, insulation selection can affect the longevity of the panel's intended effectiveness.

Cellular insulation used in the manufacture of sandwich panels comes in two primary forms, thermoplastic and thermosetting. The thermoplastic insulations are better known as molded expanded polystyrene (beadboard) and extruded

Table 3.2.a. Physical properties of various insulating materials for sandwich wall panels.

Physical property	Polystyrene						Polyisocyanurate		Phenolic	Cellular glass
	Expanded			Extruded			Unfaced	Faced		
Density (lb per cu ft)	0.7-0.9	1.1-1.4	1.8	1.3-1.6	1.8-2.2	3.0	2.0-6.0	2.0-6.0	2.0-3.0	6.7-9.2
Water absorption (percent volume)	< 4.0	< 3.0	< 2.0	< 0.3			< 3.0	1.0-2.0	< 3.0	< 0.5
Compressive strength (psi)	5-10	13-15	25	15-25	40-60	100	16-50	16	10-16	65
Tensile strength (psi)	18-25			25	50	105	45-140	500	60	50
Linear coefficient of expansion (in./in./°F) × 10^{-6}	25-40			25-40			30-60		10-20	1.6-4.6
Shear strength (psi)	20-35			—	35	70	20-100		12	50
Flexural strength (psi)	10-25	30-40	50	40-50	60-75	100	50-210	40-50	25	60
Thermal conductivity (Btu-in./hr/ft²/°F)	0.30	0.26	0.23	0.20			0.18	0.10-0.15	0.16-0.23	0.35
Maximum use temperature	165°F			165°F			250°F		300°F	900°F

Note: 1 lb per cu ft = 16.02 kg/m³; 1 psi = 0.006895 MPa; 1 in./in./°F = 1.800 mm/mm/°C; 1 Btu-in./hr/ft²/°F = 0.1442 Wm/m²/°C; $t_c = (t_f - 32)/1.8$.

expanded polystyrene (extruded board). Thermosetting insulations consist of polyurethane, polyisocyanurate and phenolic. The physical properties of these thermosetting insulations are listed in Table 3.2.a.

The previously mentioned insulation types are addressed in nationally recognized ASTM standards for material production. These standards present quality control minimums to the manufacturer for each product matrix. The references for the cellular insulations mentioned above in ASTM are listed in Table 3.2.b.

A concrete sandwich panel is a unique environment for an insulating material. During manufacture of the panel, the insulation is exposed to high temperatures from hydration and applied heat from accelerated curing. These high temperatures, as well as loading from foot traffic during production, high moisture levels from the curing of a plasticized mix, and compressive forces and flexural stresses, all require that the insulation being used exhibit superior performance. Once the panel is cured and erected into place, the insulation is then exposed to a continuous moisture and vapor drive that continues to affect the insulating material.

In some locations, freeze-thaw cycles during the building's lifetime produce forces on the insulation that work to break or tear apart individual cells. For example, a molded polystyrene insulation has a high moisture absorption rating. A building with a high moisture drive will cause this insulation to absorb potentially large amounts of moisture. When exposed to a freeze-thaw cycle, the weak bond between the beads or cells of the insulation breaks down and the insulation begins to disintegrate. This process can be mitigated by choosing an extruded polystyrene.

In some facilities, sandwich panels are exposed to extremely high interior operating temperatures. The physical property of an insulation to withstand these temperatures can cause the panel to fail to perform as intended throughout the lifetime of the building. For instance, polystyrene insulation has a relatively low melting temperature. These insulation types begin to shrink when temperatures reach 160°F (71°C). Selection of a protected polyurethane or polyisocyanurate insulation with melting temperatures above 350°F (177°C) can prevent possible structural weakness or thermal instability. The specifier should choose the insulation to be compatible with and resistant to the conditions to which it will be exposed.

Table 3.2.b. ASTM standard references for various types of insulation.

Type of insulation	ASTM designation	ASTM type
Expanded polystyrene	ASTM C-578	Types I, II, VIII, IX, XI
Extruded polystyrene	ASTM C-578	Types IV, V, VI, VII, X
Polyurethane	ASTM C-591	Types 1, 2, 3
Polyisocyanurate	ASTM C-591	Types 1, 2, 3
Phenolic	ASTM C-1126	Types I, II, III

3.3 ENERGY PERFORMANCE

3.3.1 Thermal Transmission

Thermal transmission is usually the most important physical property for the insulation in a concrete sandwich wall. The ability of the panel to resist energy flow is affected by the ability of the insulation system to resist the transfer of energy. Chapter 8 of ASHRAE Standard 90.1[5] addresses various wall assemblies and the calculation of the thermal transmittal of those assemblies.

In order to construct an insulated panel, structural elements (i.e., wythe ties) must often pass through the insulation layer or the insulation layer is fit into gaps between those structural elements. This construction practice interrupts the otherwise continuous insulation layer and, thus, provides the potential for conductance of energy. These interruptions are also known as "thermal bridges." Thermal bridges can be steel, concrete, composites and plastics. A thermal bridge conducts energy at a much higher rate than the insulation, thus creating "short circuits" where they occur. The short circuit associated with the thermal bridge reduces the effectiveness of the insulation.

3.3.2 Performance Calculation

Thermal bridges in sandwich panels can create high potentials for condensation and thermal inefficiency based on the designed performance of the panels. ASHRAE Standard 90.1[5] addresses the calculation of the thermal bridge by mandating the use of two calculation procedures: Isothermal (Series-Parallel) Analysis and Zonal Method Analysis. These procedures used in their proper context represent the most effective prediction of the actual performance of the panels. Both of these calculation methods are addressed in Section 3.4.

It is beyond the scope of this report to prescribe or recommend a particular thermal performance calculation method. Opinions are varied concerning the proper method or combination of methods to be used with a particular panel configuration. Use of connections through the insulation and solid zones of concrete in and/or around the insulation has been common since the introduction of sandwich panels.

During the past two decades, insulation systems have been developed to minimize or eliminate the solid zones of concrete, the steel connections, or both. The selection of the insulation system for use in the sandwich panel should be based on the conditions in the building environment, the required structural performance, and the effect that the building codes have on the constructed assembly.

3.4 CALCULATION PROCEDURES

Calculation of the energy efficiency of a sandwich panel includes analyzing the panel for the effects of thermal bridging and accounting for the improved performance based on the use of concrete as a thermal storage material. Previously mentioned were the isothermal and zonal method analyses. These two calculations are provided in Chapter 20 of the 1993 ASHRAE Handbook of Fundamentals,[6] and are included in the latest energy code adoption ASHRAE 90.1.[5] Another useful publication is the Thermal Mass Handbook, Concrete and Masonry Design Provisions using ASHRAE/IES 90.1-1989.[7]

3.4.1 Zonal Method Analysis

The Zonal Method Analysis is a calculation based on a regularly spaced element interrupting an insulation layer. Steel studs in a cavity wall, steel bulb tees in a roof deck, and steel ties in a sandwich panel are examples of the elements considered by this analysis. The effect that these elements have on the insulation layer can be accurately calculated by this analysis, which accounts for the difference in U-value for the given percentages of area. The introduction of concrete zones or bridges into a given assembly may not be effectively accounted for by the Zonal Method due to the irregularity of the zone-to-panel area.

3.4.2 Isothermal Analysis

The Isothermal Analysis was derived to account for irregular construction elements in an envelope assembly. This analysis takes into consideration the U-value concentration of the materials through a parallel plane based on their overall area percentage. This analysis is backed by thermal testing to show the effect of the actual construction technique and the design on the in-place wall assembly.

3.4.3 Thermal Mass Calculation

Energy analysis by the two methods above is available in computer software or long-hand format and can be performed on a per job basis or on a standard assembly basis. The results of these calculations can show the effect that a panel assembly has on the entire building efficiency. Based on the latest Standard 90.1,[5] sandwich panels contribute greater efficiencies to the building operation than previously assumed. This contribution is significantly affected by the ability of the panels to initially resist energy flow, and can be evaluated.

A comparison of wall assemblies over the past decade by ASHRAE and the DOE has produced engineering results that show additional benefits of using concrete in wall construction. Chapter 8 in ASHRAE Standard 90.1[5] gives an equation for comparing the operating performance of walls with high mass to that of walls with low mass. Based on the specific location of a building and the operating conditions, overall energy performance can be enhanced due to the heat storage capacity of concrete. Concrete mass in a wall assembly provides a thermal lag during daily and seasonal climate fluctuations. This thermal lag can beneficially delay the overall transfer of energy from the warm side of the wall to the cold side.

Properly constructed and calculated concrete sandwich panels can take full advantage of these principles and provide significant advantages over other wall systems. This method has been made available by the DOE and ASHRAE as computerized procedures for calculation.

CHAPTER 4 — MANUFACTURE OF SANDWICH PANELS

4.1 GENERAL

The manufacture of sandwich panels generally follows the same procedures and practices as commonly used in the production of standard precast/prestressed products. These procedures are all described in detail in PCI MNL-116-85 Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products, Third Edition.[8] Special manufacturing considerations are described in the remaining portion of this chapter. See Figs. 4.1.a to 4.1.m.

4.2 STRESSING AND STRAND POSITIONING

The precasting plant must have the proper equipment for tensioning prestressing strand according to MNL-116-85[8] and the necessary equipment and form setup to ensure strand location in the final as-cast product. Strand location is extremely important in sandwich panel production to prevent warping, bowing and cracking. In long production lines, the strands are sometimes chaired to maintain accurate locations.

4.3 METHODS OF CASTING

4.3.1 Wet Cast (Normal Slump)

Sandwich insulated wall panels cast by the "Wet Cast System" are manufactured in long line steel forms with bulkheads separating each panel. The bottom wythe strand, reinforcing steel, embedments and other required materials are placed and the first layer of concrete is introduced and vibrated as shown in Fig. 4.3.1.a. Vibration techniques vary. Some commonly used methods are standard "spud" vibrators, vibrating drop screeds, grid vibrators and external form vibrators.

Insulation is then placed with wythe ties that connect the bottom layer of concrete and project into the top layer of concrete, as shown in Fig. 4.3.1.b.

The top wythe strand, reinforcing steel, and embedments are then placed and the final layer of concrete is cast and finished as shown in Fig. 4.3.1.c. Because vibration tends to make the insulation float, some manufacturers use reinforcing bar slugs placed between the top strand and the insulation to hold down the insulation.

Also, some manufacturers stress both the top and bottom strands in the initial production step. The remaining production procedures are then the same as described above.

4.3.2 Dry Cast (Zero Slump)

Dry cast refers to panels made with zero slump concrete placed by machines that extrude dry concrete as part of their manufacturing operation. Zero slump products started as hollow-core slabs, and the standard production procedures have been modified to permit the manufacture of sandwich wall panels.

4.3.3 Machine Cast

Manufacturers have developed various machine casting systems to improve the quality and reduce the cost of manufacturing sandwich wall panels. This system utilizes the "Wet Cast System" and/or the "Dry Cast System." Some of the machine-cast panel systems are described in this section.

Corewall (Wet Cast) System

The Corewall Panel System (see Fig. 4.3.3.a) utilizes machinery to produce panels with normal slump concrete or by the wet cast system. Corewall Inc. has developed machinery to place the concrete wythes and create various ribbed finishes in the top surface. After the machinery places the bottom wythe concrete, insulation is then placed with special wythe ties. The Corewall machinery then places the top wythe concrete and finishes the top surface with a proprietary roller system. The rollers can be changed to provide various patterns in the concrete surface. The panels may be formed to length or sawcut to length after curing.

Spandeck (Wet Cast) System

The Spandeck Wall Panel System (see Fig. 4.3.3.b) utilizes machinery that produces hollow-core slabs [6, 8, 10 and 12 in. (152, 203, 254 and 305 mm)] as the bottom

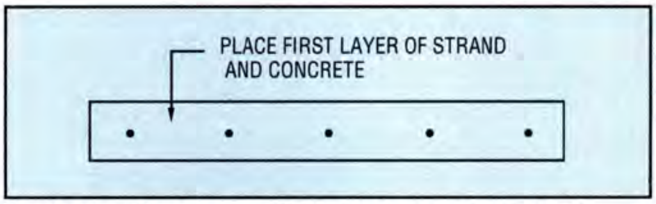


Fig. 4.3.1.a. First step — Place first layer of strand and concrete.

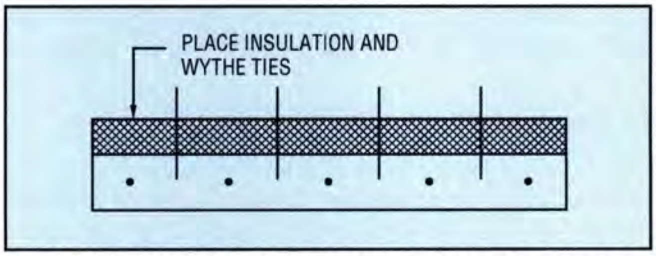


Fig. 4.3.1.b. Second step — Place insulation and wythe ties.

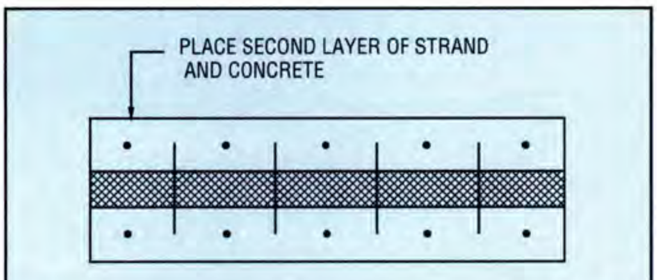


Fig. 4.3.1.c. Third step — Place second layer of strand and concrete.

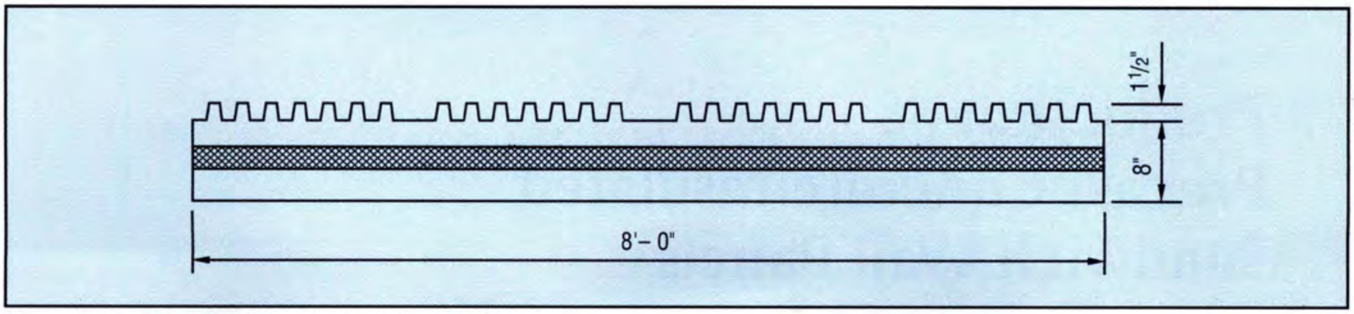


Fig. 4.3.3.a. Corewall Panel System (wet cast).

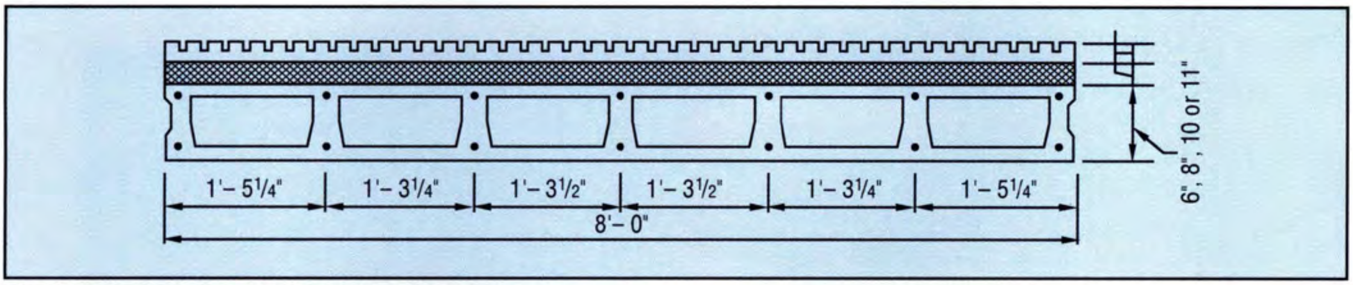


Fig. 4.3.3.b. Spandeck Wall Panel System (wet cast).

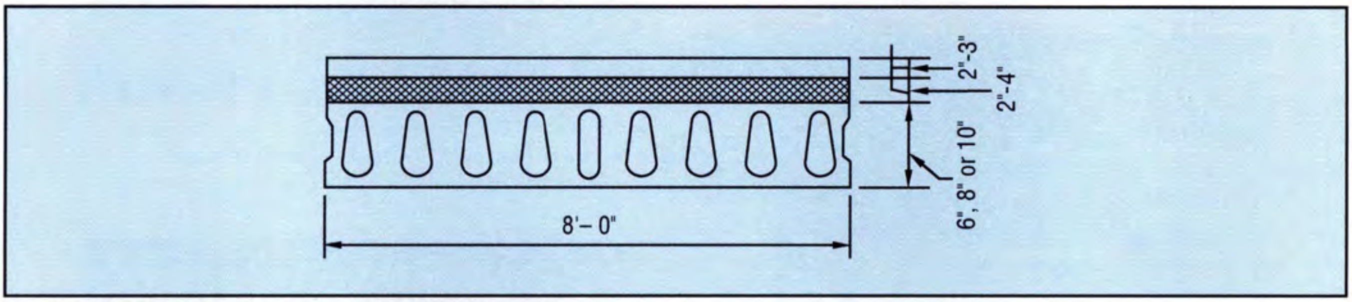


Fig. 4.3.3.c. Spancrete Wall Panel System (dry cast).

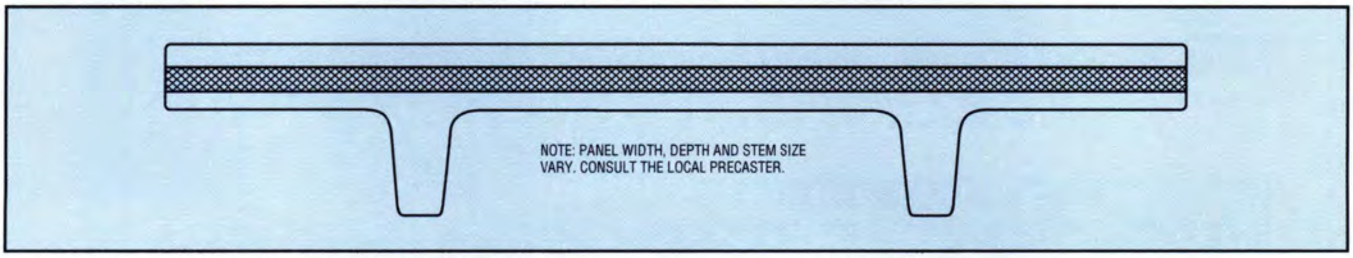


Fig. 4.3.4. Insulated double tee wall panel system.

wythe with normal slump concrete or by the wet cast system. Insulation is then placed with wythe ties and the Spandeck machinery places the top wythe concrete. The finish is produced by a moving mandrel. The mandrels are interchangeable, thus providing various patterns in the concrete surface. After curing, the panels are sawcut to length.

Spancrete (Dry Cast) System

The Spancrete Wall Panel System (see Fig. 4.3.3.c) also utilizes machinery that produces hollow-core slabs [6, 8 and 10 in. (152, 203 and 254 mm)] with zero slump concrete or by the dry cast system. Insulation is then placed with ties and the top wythe concrete is placed by machine and finished. After curing, the panels are sawcut to length.

4.3.4 Other Products (Wet Cast)

Insulated sandwich panels are not limited to flat panels and adaptations of hollow-core products. Sandwich insulation can be added to double tees, single tees, architectural shapes and other products, as shown in Fig. 4.3.4.

4.4 CURING

Curing of sandwich insulated wall panels is critical to product quality. It is sometimes necessary to have a

Production of Precast Concrete Insulated Sandwich Wall Panels

Contributed by: Budd Hilgeman, Concrete Technology Inc., Springboro, Ohio.

Fig. 4.1.a. Basic form with reinforcement in place; exterior side of panel is form surface; typically, architectural panels are 3 in. (76 mm) of face mix, 2 in. (51 mm) of insulation, and 3 in. (76 mm) of back-up concrete.

Fig. 4.1.b. Concrete being placed as a part of face mix, raking mix to cover all surfaces of form; concrete once placed, will be vibrated with both external and internal units; placement depth is 3 in. (76 mm) for face mix.

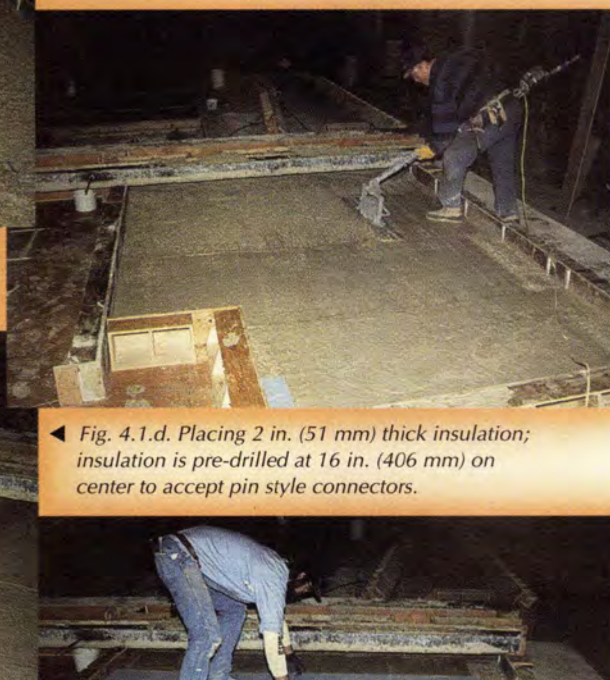


Fig. 4.1.c. Vibrating face mix using bull float vibrator.

Fig. 4.1.d. Placing 2 in. (51 mm) thick insulation; insulation is pre-drilled at 16 in. (406 mm) on center to accept pin style connectors.

Fig. 4.1.e. Placing 2 in. (51mm) thick insulation; insulation can go outer edge to outer edge, and/or can incorporate certain areas of panels that are non-insulated, depending on design criteria required.

◄ *Fig. 4.1.f. Insulation partially in place.*

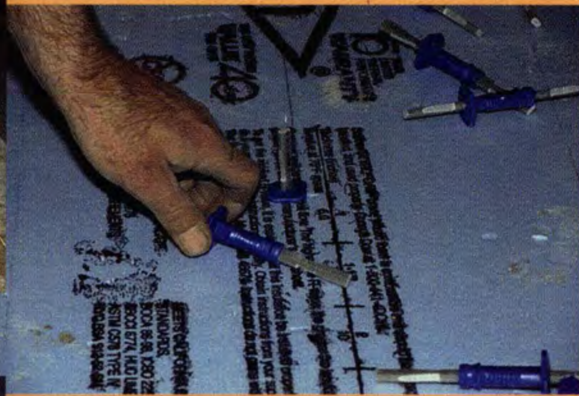


Fig. 4.1.g. Pin connector in place through insulation and full pin connectors. ►

◄ *Fig. 4.1.h. Placing pin connector through insulation.*

Fig. 4.1.i. Pin connector partially through insulation. ►

◄ *Fig. 4.1.j. Pin connectors in place.*

Fig. 4.1.k. Placement of concrete back-up over insulation. ►

◄ *Fig. 4.1.l. Blockout of foam around edge handling insert.*

Fig. 4.1.m. Vibration of backup mix using pencil vibrator. ►

curing system capable of providing external heat to accelerate the concrete heat of hydration so as to permit daily casting. The curing system usually incorporates controls to hold the preset temperature of the freshly placed concrete according to the procedures contained in PCI MNL-116.[8]

The maximum curing temperature needs to be carefully determined and monitored because some insulation materials become unstable at 160°F (71°C). Many manufacturers use a maximum curing temperature of 140°F (60°C). Under high heat, extruded polystyrene may expand its thickness by 50 percent, causing a "blowout," and expanded polystyrene may shrink causing gaps between the insulation and concrete.

Special attention (such as tenting of curing tarps) is given to the curing of the top and bottom wythes because heat applied from the bottom of the panel is prevented from reaching the top wythe by the presence of the insulation.

4.5 HANDLING

Handling, yarding, storage and shipping requires specialized equipment and personnel experienced with this product. Products should be stored so that their weight is supported on the structural wythe.

4.6 FINISHES

Finishes on sandwich panels can be varied depending on the design requirements, method of casting and budget for the project.

The bottom wythe finish provided by steel or wood forms is a smooth finish suitable for either interior or exterior use. Coatings such as paint or stain can be used to provide an excellent completed surface. Various textured finishes can also be provided on the bottom wythe by using form liners placed on the form.

The top wythe finish can be produced with a variety of finishes, both by hand and by machinery. These finishes include rake, rolled and imprinted. Designers usually check with local manufacturers to determine the cost of different finishes prior to detailing. A finish available from one manufacturer may not be available from another. Manufacturers have gone to great expense to develop certain finishes that they find are acceptable to a wide range of products. Variations from the norm may greatly increase the cost of the panels.

4.7 DETENSIONING

Prestressing strands are detensioned using the same procedures as with other prestressed concrete products. The strands are generally cut using cutting torches or saws. Special care is given when cutting strands located in thin concrete wythes so that strand impact forces are minimized.

CHAPTER 5 — PRODUCT TOLERANCES, CRACKING AND REPAIRS

5.1 TOLERANCES

5.1.1 Manufacturing Tolerances

The manufacturing tolerances as presented in PCI MNL 116[8] for "Insulated Wall Panels — Single-Story Structures" are typically used for both single-story sandwich panels and multistory sandwich panels. Because sandwich panels are usually cast in long line forms similar to other prestressed concrete products, the manufacturing tolerances of PCI MNL 117-96 Manual for Quality Control for Plants and Production of Architectural Precast Concrete Products[9] are not applicable to sandwich panels.

5.1.2 Erection Tolerances

The erection tolerances as stated for structural wall panels in the PCI Design Handbook, Fourth Edition,[2] are normally used.

5.1.3 Relationships Between Different Tolerances

Confusion sometimes arises concerning whether or not an in-place panel is within specified tolerances. To help clarify this situation, the first two paragraphs of Section 8.1.5 of the PCI Design Handbook, Fourth Edition,[2] are reprinted in their entirety.

> "A precast member is erected so that its primary control surface is in conformance with the established erection and interfacing tolerances. The secondary control surfaces are generally not directly positioned during erection, but are controlled by the product tolerances. Thus, if the primary control surfaces are within erection tolerances, the member is erected within tolerance. The result is that the tolerance limit for the secondary surface may be the sum of the product and erection tolerances.
>
> Since tolerances for some features of a precast member may be additive, it must be clear to the erector which are the primary control surfaces. If both primary and secondary surfaces must be controlled, provisions for adjustment should be included. The accumulated tolerance limits may have to be accommodated in the interface clearance. Surface and feature control requirements should be clearly outlined in the plans and specifications."

5.2 CRACKING

The addition of prestressing helps control cracking, but sandwich panels crack in ways similar to other concrete wall panels. Some of the same cracks are those that occur at reentrant corners, transverse cracks due to handling and longitudinal cracks due to prestress splitting forces or handling. In addition, some cracks have been observed at locations where the insulation is discontinuous, such as at the ends of panels detailed with a solid end block. None of these cracks are normally cause for rejection, but the designer, manufacturer, shipper and erector can and should take measures to reduce the occurrence of these cracks.

5.3 REPAIRS

Repairs to cracks in sandwich panels are generally not structural repairs. In other words, the repairs normally do not return the cracked panel to its original uncracked stiffness. The use of epoxy injection is not applicable in areas where insulation is located because once the epoxy penetrates the injected wythe, there then exists an unlimited path of travel for the epoxy at the concrete to insulation interface. "Painting" of the cracks with injection epoxy is sometimes used to seal the cracks.

Cracks that are located on the interior of the building and are transverse to the prestressing strands are usually small in width. If the interior of the structure is not in an aggressive environment, these interior cracks are usually not repaired. If the surface is to be painted, the painter can treat these cracks in the same manner used to treat any concrete crack prior to painting, such as spackling.

CHAPTER 6 — HANDLING, SHIPPING AND STORAGE OF SANDWICH PANELS

6.1 PANEL LENGTH AND WIDTH

Sandwich panel length and width vary due to project requirements, form size, handling equipment capabilities and transportation limitations. In some parts of the United States, shipping laws allow unescorted, permitted transportation of 12 ft (3.66 m) wide panels, but in other areas this maximum width is only 10 ft (3.05 m). Panels as wide as 14 ft (4.27 m) and as long as 60 ft (18.3 m) have been manufactured and transported.

Panels are handled in conformance with the design requirements. Panels are either back stripped using standard lifting devices or special vacuum lifts, or they are edge picked if cast on a special tilt table. Narrow panels may be handled using side clamp lifting devices. Choice of lifting points and stress analysis is done in accordance with the PCI Design Handbook, Fourth Edition, Chapter 5.[2]

6.2 SHIPPING

Sandwich panels are shipped either on edge or in the flat position. The shipping position is dependent on equipment availability, form face finish requirements, transportation equipment, and the flexural design of the panel.

When panels are shipped on their edge, consideration of localized bearing stresses must be given in order to prevent chipping and spalling. Non-composite panels should be loaded so that only the structural wythe sits on dunnage. Panels receiving a special finish are often edge shipped to prevent damage or staining to the finish.

When panels are shipped in the flat position, more panels can usually be shipped per load. Some items requiring attention are:

- Length of panels vs. length of trailer. If the trailer is flexible (such as a stretch trailer), over-stressing of the panel may result.
- Dunnage is usually positioned at the lift point locations.
- Job site access needs to be such that torsional twist of the trailer and panel is minimized or eliminated.

6.3 PANEL STORAGE

Plant storage is determined by the method of stripping and/or shipping. Ideally, the panel is stored in the same position in which it is shipped. This reduces handling, thus reducing cost and opportunities for damage.

Job site storage of sandwich panels is undesirable because the panels need to be stored on dunnage resting on clear, firm, flat areas. Areas such as these are not found on most job sites. Job site storage also creates additional handling unless the trailers are "dropped." On most industrial buildings, the general contractor requests that some panels be left out for equipment access during construction. These panels can be leaned against the structure adjacent to their final location, and should be securely tied off to the structure.

CHAPTER 7 — ERECTION OF SANDWICH PANELS

7.1 PANEL HANDLING AND JOB SITE STORAGE

Panel handling at the job site is basically the same as previously described in Chapter 6. The only difference may be the equipment used and the type of rigging employed. The panel manufacturer considers the type of equipment/rigging to be used and specifies proper handling techniques so that the panel is not damaged during the unloading process. Some manufacturers show handling diagrams on the precast/prestressed concrete erection drawings.

7.2 PANEL ERECTION

Panel erection is performed with proper sized cranes and rigging. Erection diagrams showing hook-up locations and tripping procedures are often a part of the manufacturer's drawings. Panel handling procedures are described in the PCI Design Handbook, Fourth Edition, Chapter 5,[2] and PCI MNL-127-85, Recommended Practice for Erection of Precast Concrete.[10]

7.3 PANEL BRACING

The use of temporary bracing is sometimes required during the erection of loadbearing panels. Most often, wall braces are used to acomplish this, an alternative being a temporary beam and column system. Wall braces are commonly either inclined pipe columns or tube columns, placed between the panel and either the floor slab or a deadman. Deadmen can be either temporary or permanent. Structure foundations, such as spread footings at columns, may also be used to temporarily brace the panels. Braces can be rented from several industry suppliers who will also provide load tables appropriate for the particular brace.

CHAPTER 8 — INSPECTION OF SANDWICH PANELS

8.1 PLANT INSPECTION

Inspection of sandwich wall panels at the manufacturing facility follows normal pre-pour and post-pour quality control procedures as outlined in PCI MNL 116.[8] Particular attention needs to be given to the location of the prestressing strands, the location of lifting devices and wythe thickness. Periodic inspection of the panels in storage is a good practice. Any cracking, chipping or spalling is more easily repaired prior to shipping. A final general inspection is usually conducted by the loading crew during the loading process.

8.2 JOB SITE INSPECTION

Remedial work is normally performed by the precaster prior to final inspection by the general contractor and owner's representative. Acceptance of the panels is based on fulfilling the written project requirements. These requirements normally reference PCI MNL 116.[8]

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