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

Second Edition

Prepared by

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PREFACE

This report is an update of the original state-of-the-art report published in the March–April and May–June 1997 issues of the *PCI Journal*.[1] The original report was later published as a reprint (JR-403). After the publication of the original report, the use of precast/prestressed concrete sandwich wall panels became more widespread in the United States. Also, the publication led to a better understanding of the proper use, performance, and technical aspects of these specialty panels.

Based on the interest in these panels and the passage of time from the initial report, the committee decided that an update to the state-of-the-art report was a necessary intermediate step prior to the preparation of a recommended practice committee report. This report updates both the text and the associated design examples to current codes and design practices. In addition, newer, more-applicable photographs of the manufacturing process and completed, in-place panels have replaced the previous photographs.

CONTENTS

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

b = clear span of panel

b_p = height of parapet

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 panel section at midheight, due to out of plumbness, thermal bow, and load effects, relative to ultimate design load (for P-D analysis)

e_P = eccentricity of axial roof or floor load on panel, or prestressing force measured from centroid of section

E_c = modulus of elasticity of concrete

EI = flexural stiffness of compression member

f = net stress on concrete cross section

f_a = unit stress of structural steel

f_b = bending stress due to stripping; subscript denotes direction

f_{bv} = bending stress during the erection of the panel

f'_c = specified compressive strength of concrete

f'_{ci} = concrete compressive strength at time considered

f_{pc} = concrete 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_r = modulus of rupture of concrete

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_1 = overall depth of the composite panel section

h_2 = insulation thickness within the composite panel section

H_u = factored load reaction in horizontal direction

I = moment of inertia of section resisting external loads

I_g = moment of inertia of gross section

I_p = polar moment of inertia

I_{xx}, I_{yy} = moment of inertia of weld group with respect to its own x and y axes, respectively

K'_u = coefficient = $M_u(12{,}000)/bd_p^2$

l = clear span length

l_d = development length

l_e = embedment length

l_w = length of weld

L = live load

L_R = roof 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 = total prestress force after losses

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

Q = statical moment about neutral axis

Q_E = effect of horizontal seismic (earthquake-induced) forces

r = radius of gyration at cross section of a compression member

R = fire endurance of composite assembly

R = roof live load

$R_1R_2R_3$ = fire endurance of individual course

R_{DL} = reaction due to dead load

R_u = reaction due to total factored load

S = section modulus

S_{DS} = design, 5% damped, spectral response acceleration parameter at short periods

S_w = section modulus of weld group

t = thickness of section

t_{ns} = thickness of fascia wythe (typically outer wythe)

t_s = thickness of structural wythe (typically inner wythe)

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 weld group

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_i = initial panel bow

Δ_{temp} = thermal bow in wall panel

Δ_w = bow in wall panel due to wind

e_t = net tensile strain in extreme tension steel

μ_e = effective shear-friction coefficient

ρ = a redundancy factor based on the extent of structural redundancy present in a building

ρ = A_s/bd = ratio of nonprestressed reinforcement

ρ_p = A_{ps}/bd_p = ratio of prestressed reinforcement

ϕ = strength-reduction factor

ϕ_k = stiffness-reduction factor

ω_{pu} = $\rho_p f_{ps}/f'_c$

INTRODUCTION

Precast/prestressed concrete sandwich wall panels are composed of two concrete wythes (layers) separated by a layer of rigid foam plastic insulation. One of the concrete wythes may be a standard shape, such as a flat slab, hollow-core section, double tee, or any custom architectural concrete section. In place, sandwich wall panels can 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 structural frame, including structural steel, reinforced concrete, pre-engineered metal, and precast/prestressed concrete. The panels are fabricated at a precast concrete manufacturing plant, shipped 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 exhibit some unique characteristics and behavior. Where sandwich panel design and manufacturing parallel other precast/prestressed concrete 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 increased within the past decade since the first state-of-the-art report was published. Manufacturers continue to seek new, viable product lines, and architects and engineers are pleased with the energy performance and 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.

The purpose of this report is to present to the architect, engineer, contractor, precast/prestressed concrete producer, and owner the 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 for the benefit of specifiers, suppliers, and users. 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

Figure 1.5.a. University Commons Student Housing, Georgia State University, Atlanta, Ga.

guidance for obtaining satisfactory results.

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 PCI Technical Activities Committee. All reasonable care has been taken to ensure the accuracy of the presented material. The technical portions of this 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.

The publication of the material contained herein is not intended as a representation or warranty on the part of PCI or any other person named herein that this information is suitable for any general or particular use or promises freedom from infringement of any patent or patents. Anyone making use of this information assumes all liability from such use.

Figure 1.5.b. Proximity Hotel, Greensboro, N.C.

Figure 1.5.c. Academy of World Languages, Cincinnati, Ohio.

Figure 1.5.d. Desmarais Building, University of Ottawa, Ottawa, Ontario, Canada.

Figure 1.5.e. Morris County Correctional Facility, Morris Township, N.J.

Figure 1.5.f. J. L. Mann High School, Greenville, S.C.

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 50 years.

Early sandwich panels were of the noncomposite type and consisted of a thick, structural wythe (or hollow-core slabs, double tees, or single tees), a layer of rigid insulation, and a nonstructural wythe. Composite sandwich panels were manufactured later.

1.2 MATERIALS

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

Figure 1.5.g. PEI-Genesis, South Bend, Ind.

Figure 1.5.h. Lucas Oil Stadium, Indianapolis, Ind.

Figure 1.5.i. Silas Center, Winston-Salem, N.C.

Figure 1.5.j. Ralph Engelstad Arena, Grand Forks, N.Dak.

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. They can be relocated to accommodate building expansion. In addition, the insulation provides superior energy performance and moisture protection compared with many other wall systems. The hard surface on the inside and outside of the sandwich panel provides resistance to forklift damage, theft, and vandalism and a finished product requiring no further treatment if desired.

Figure 1.5.k. Alameda County Juvenile Justice Center, San Leandro, Calif.

Figure 1.5.l. Jack Britt High School, Fayetteville, N.C.

Figure 1.5.n. Nordstrom Oaks, Irvine, Calif.

Figure 1.5.m. Louisiana Offshore Oil Port Building, Covington, La.

Figure 1.5.o. John H. Stroger Jr. Hospital of Cook County, Chicago, Ill.

Figure 1.5.p. Willow Creek Elementary School, Fleetwood, Pa.

Figure 1.5.q. Budweiser Warehouse, Roebuck, S.C.

Figure 1.5.r. University of Kentucky Patient Care Facility, Lexington, Ky.

Figure 1.5.s. Centralia High School, Centralia, Ill.

Figure 1.5.t. 151 First Side, Pittsburgh, Pa.

Figure 1.5.u. Juniper-Poplar Hall, University of South Florida, Tampa, Fla.

Figure 1.5.v. New Jersey Aquarium, Camden, N.J.

1.4 DESCRIPTION OF SANDWICH PANEL TYPES

1.4.1 Noncomposite

A noncomposite sandwich panel is analyzed, designed, detailed, and manufactured so that the two concrete wythes act independently. Generally, there is a structural wythe and a nonstructural wythe, with the structural wythe being the thicker of the two.

1.4.2 Composite

Composite sandwich 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 in bending. This is accomplished by providing full shear transfer between the wythes.

1.4.3. Partially Composite

Partially composite sandwich panels have shear ties connecting the wythes, but the connectors do not provide full composite action. The bending stiffness and strength of these panel types fall between the stiffnesses and strengths of fully composite and noncomposite sandwich panels. Section 2.7.5 covers the design of this panel type.

1.5 APPLICATIONS

Sandwich panels provide economical, attractive, and energy-efficient hard walls and are found on virtually every type of structure, including residential buildings, schools, office buildings, low-temperature environments, controlled atmospheres, warehouses, industrial buildings, justice facilities, and hospitals. The most common use of sandwich panels is for exterior walls, but they have been used as internal partition walls, particularly around temperature-controlled rooms. For example, highly insulated (high *R*-value) sandwich panels have been used in subzero freezer applications.

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

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

Figure 1.5.w. Aero Tech Warehouse, Colorado Springs, Colo.

Figure 1.5.x. El Paso County Correctional Facility, Colorado Springs, Colo.

Figure 1.5.y. Cabela's Retail Store, Lehi, Utah.

Figure 1.5.z. Healthridge Fitness Center, Olathe, Kans.

CHAPTER 2 – DESIGN AND DETAILING CONSIDERATIONS

2.1 GENERAL INFORMATION

The design of precast/prestressed concrete sandwich wall panels is similar to that of other precast/prestressed concrete members. Once the type of panel is selected (noncomposite, composite, or partially composite), section properties and the distribution of forces are determined. The panels are analyzed for stresses resulting from the transfer of the prestressing force, stripping, storage, transportation, and erection and are also analyzed for allowable stresses, deflections, and ultimate strength for in-place conditions. The criteria used to evaluate the stresses, deflections, and ultimate strength are in accordance with the current version of ACI 318, *Building Code Requirements for Structural Concrete*, and applicable local, state, and national codes.

The keys to successful sandwich panel design are to ensure that the actual structural behavior of the panel coincides as closely as possible with 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 the 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. Therefore, designers considering a departure from proven design approaches should consider a verification test program if the project application involves a large number of panels.

2.2 WYTHE THICKNESS AND SIZE OF PRESTRESSING STRAND

Most sandwich panels are made to be as thin as possible, but the wythe thickness is generally determined by the panel type and final use. For example, the wythe thickness may be controlled by the specified fire resistance for the project or the minimum concrete cover required for the reinforcement. A noncomposite panel usually requires a thicker wythe or wythes than a composite panel with the same load and span conditions. For a noncomposite panel, many designers assume an overall panel thickness of about 1.5 times the equivalent composite panel thickness as a general estimate. The use of formliners or reveal strips on the exterior wythe requires the exterior wythe to be thicker than a plain panel so that code requirements for concrete cover and strength can be satisfied.

In this report, a 2/3/6 panel is one that comprises a 2-in.-thick (51 mm) concrete wythe, a 3-in.-thick (76 mm) insulation layer, and a 6-in.-thick (150 mm) concrete wythe.

The maximum strand diameter that may be used is related to wythe thickness. Results have been satisfactory when ³⁄₈-in.-diameter (9.5 mm) strand in 2-in.-thick (51 mm) wythes containing ¾ in. (19 mm) maximum aggregate is used. Half-inch-diameter (13 mm) strand is commonly used in wythes 3 in. (76 mm) or thicker.

Additional transverse reinforcement is commonly provided at the ends of the panel to limit splitting cracks over the strands during detensioning. Such reinforcement may be in the form of reinforcing bars or untensioned strands placed perpendicular to the prestressed strand. With low thickness–to–strand diameter ratios, special attention during production is given to the position tolerance of the as-placed strand location to maintain the required cover and to minimize the risk of splitting cracks. Wider panels are more susceptible to splitting cracks than narrow panels. It may be necessary to increase the number of strands and reduce the release force to avoid cracking of this type.

2.3 STRAND LOCATION AND FORCE

Strands are generally located at the centroid of each wythe to minimize the tendency of the wythe to camber. Composite and partially composite panels that have wythes of unequal thickness or of unequal section properties have strands that are normally placed at the centroid of each wythe, but the prestressing force is adjusted in one of the wythes so the resultant prestressing force coincides with the centroid of the composite section. This is done to eliminate or reduce the bow or camber in the panel.

For noncomposite panels, prestressing of the facing layer is generally needed only to limit cracking during handling. Because the wythe connectors allow differential shortening of the wythes without transferring shear forces across the insulation, bow is not commonly observed in noncomposite panels.

Some designers report success in intentionally introducing an inward bow to composite panels by the use of an eccentric prestressing force. The intent of this is to offset the observation that composite sandwich panels, over time, tend to bow outwards, probably due to differential shrinkage strain. Outward bowing can increase the stresses in load-bearing panels. The use of this design should be carefully considered because intentionally introducing bowing might create alignment problems during erection in the field.

2.4 WYTHE CONNECTORS

2.4.1 General Considerations

Wythe connectors can provide a variety of functions. For panels that are stripped from the form in the horizontal position, these connectors must resist the tensile forces caused by the weight of the lower wythe plus any form suction acting on that lower wythe. These connectors must also resist the tensile forces resulting from out-of-plane wind suction and seismic forces. In fully and partially composite panels, wythe connectors must resist horizontal, in-plane shear caused by flexural bending. Wythe connectors may allow the wythe to act independently but may be required to support the weight of the exterior wythe plus any suction forces when the panel is only bearing on the internal structural wythe.

These connectors may be used in conjunction with solid zones at corbels, embedments, pick points, and the like to resist out-of-plane forces while the solid zones resist the in-plane shear forces. There are many proprietary wythe connectors, such as C-ties, Z-ties, M-ties, cylindrical metal sleeve anchors, hairpins, circular expanded metal, welded-wire trusses, and plastic or fiber-composite pins. There are also proprietary wall-panel systems that use carbon-fiber mesh as wythe ties. The connector spacing ranges from 16 in. × 16 in. (400 mm × 400 mm) to 48 in. × 48 in. (1220 mm × 1220 mm).

Wythe connectors must have directional-tested confirmed shear and tension values along with test embedment depths. Connector manufacturers generally publish tensile and shear capacities for specified embedment depths. Manufacturers may also publish stiffness values for their connectors so that the level of composite action can be verified by the design professional of record.

2.4.2 Shear Connectors

Shear connectors are used to transfer in-plane shear forces between the two wythes. Sandwich panels are usually designed as one-way structural elements; shear forces are generated due to longitudinal bending in the panels. In addition, the shear connectors may be used to transfer the weight of a nonstructural wythe to the structural wythe.

Shear connectors that are designed to be stiff in one direction

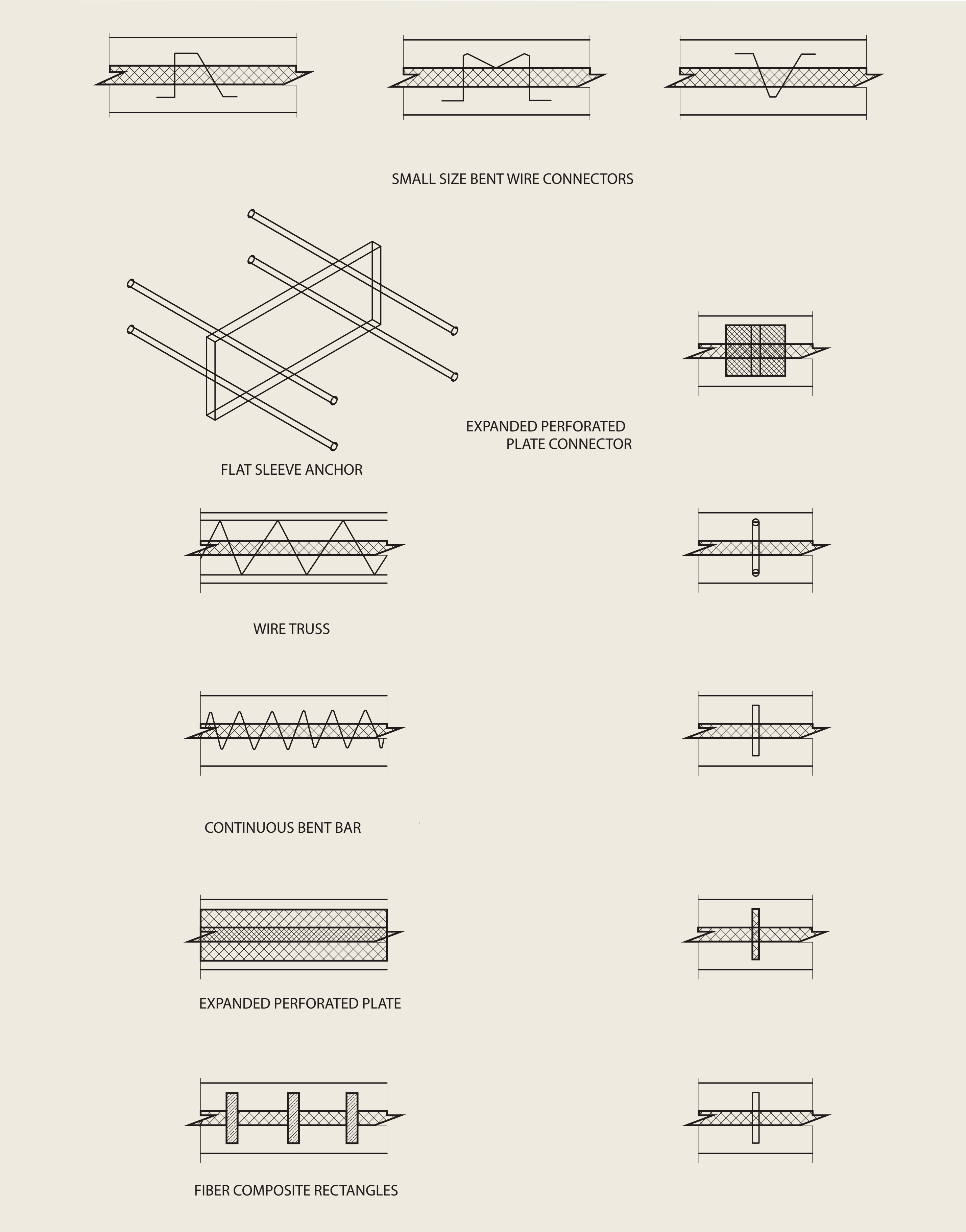


Figure 2.4.2.a. One-way shear connectors, stiff in only one direction.

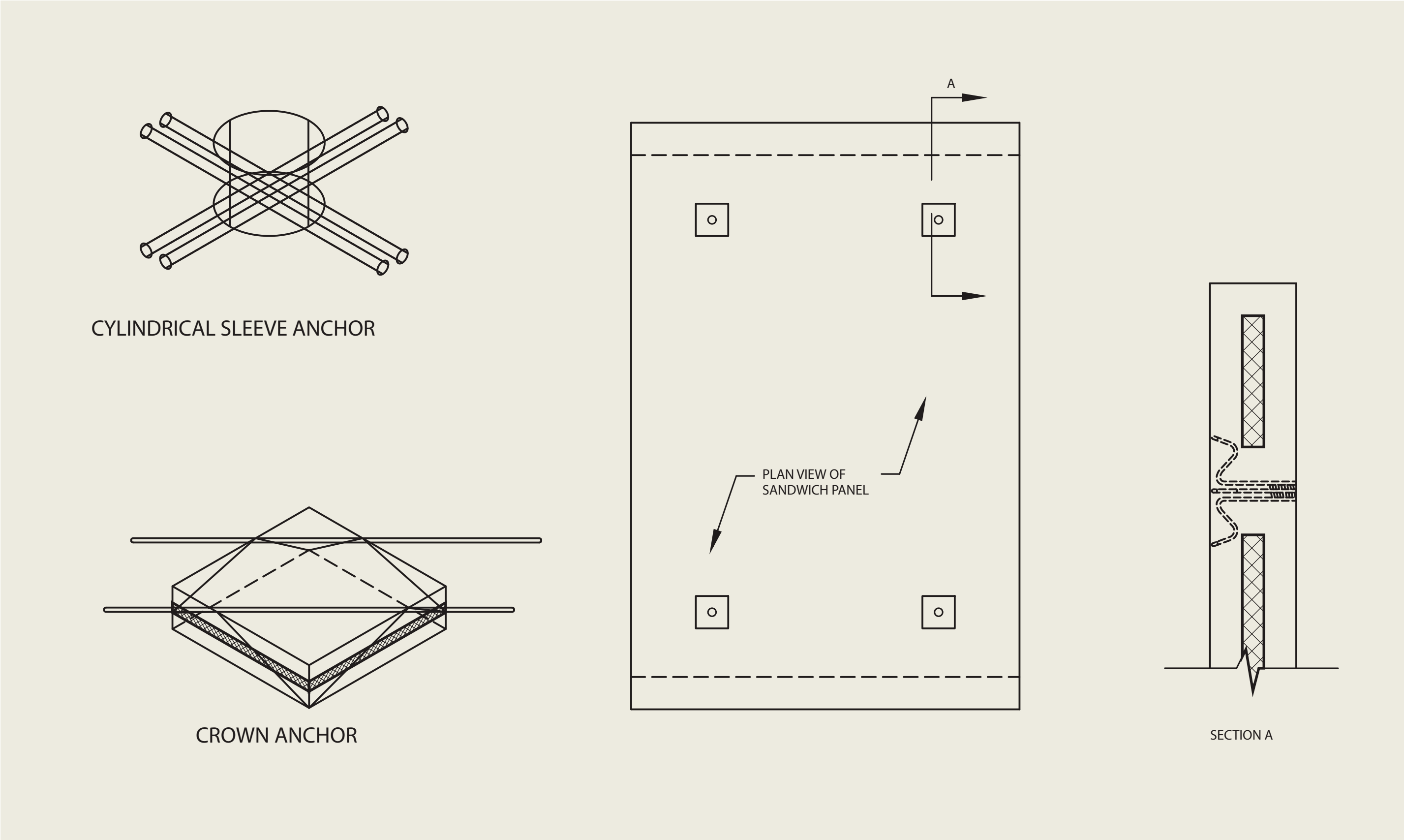


Figure 2.4.2.b. Two-way shear connectors, stiff in at least two perpendicular directions.

and flexible in another are called one-way shear connectors. Examples of these are longitudinal steel-wire trusses, solid ribs of concrete, flat sleeve anchors, fiber composite rectangles, and small-diameter bent bars (Fig. 2.4.2.a). Care must be taken in the manufacturing process to maintain the intended orientation of the one-way 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 zones of concrete (often located at each end of the panel and at lifting points), connection plates, cylindrical sleeve anchors, and crown anchors. Connection plates and crown anchors are normally installed in solid zones of concrete and can therefore be considered rigid shear connections (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 zones of concrete are utilized, a commonly used ultimate shear stress value is 80 psi (550 kPa) across the area of solid regions (see ACI 318-05, section 17.5.3).[2]

In some cases, the insulation layer itself may transfer shear 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, however, considered to be temporary. With noncomposite panels, the assumption is sometimes made that the insulation provides sufficient shear transfer to create composite action during stripping from the form, handling, and erection, but the shear transfer is not relied on to provide composite action for resisting service loads. It should be noted that certain tension connectors might also 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 as outlined in chapter 1 of ACI 318-05.

2.4.3 Noncomposite Connectors

Noncomposite connectors are generally considered capable of transferring only tension forces between the wythes. Some noncomposite connectors are, however, capable of transferring the dead load of the fascia to the structural wythe. They are used in noncomposite 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. Because these connectors are unable to transfer significant shear, their contribution to composite action is usually neglected. Examples of tension connectors are plastic pins, fiber composite connectors, metal C-ties, M-ties, hairpins, and continuous welded ladders (Fig. 2.4.3).

2.5 PANEL WIDTH, THICKNESS, AND SPAN

Sandwich panels are manufactured in virtually all of the same shapes and sizes as solid panels. In general, the larger the panels are, the greater the economy because there are fewer pieces to form, strip, load, transport, erect, and connect. The maximum size

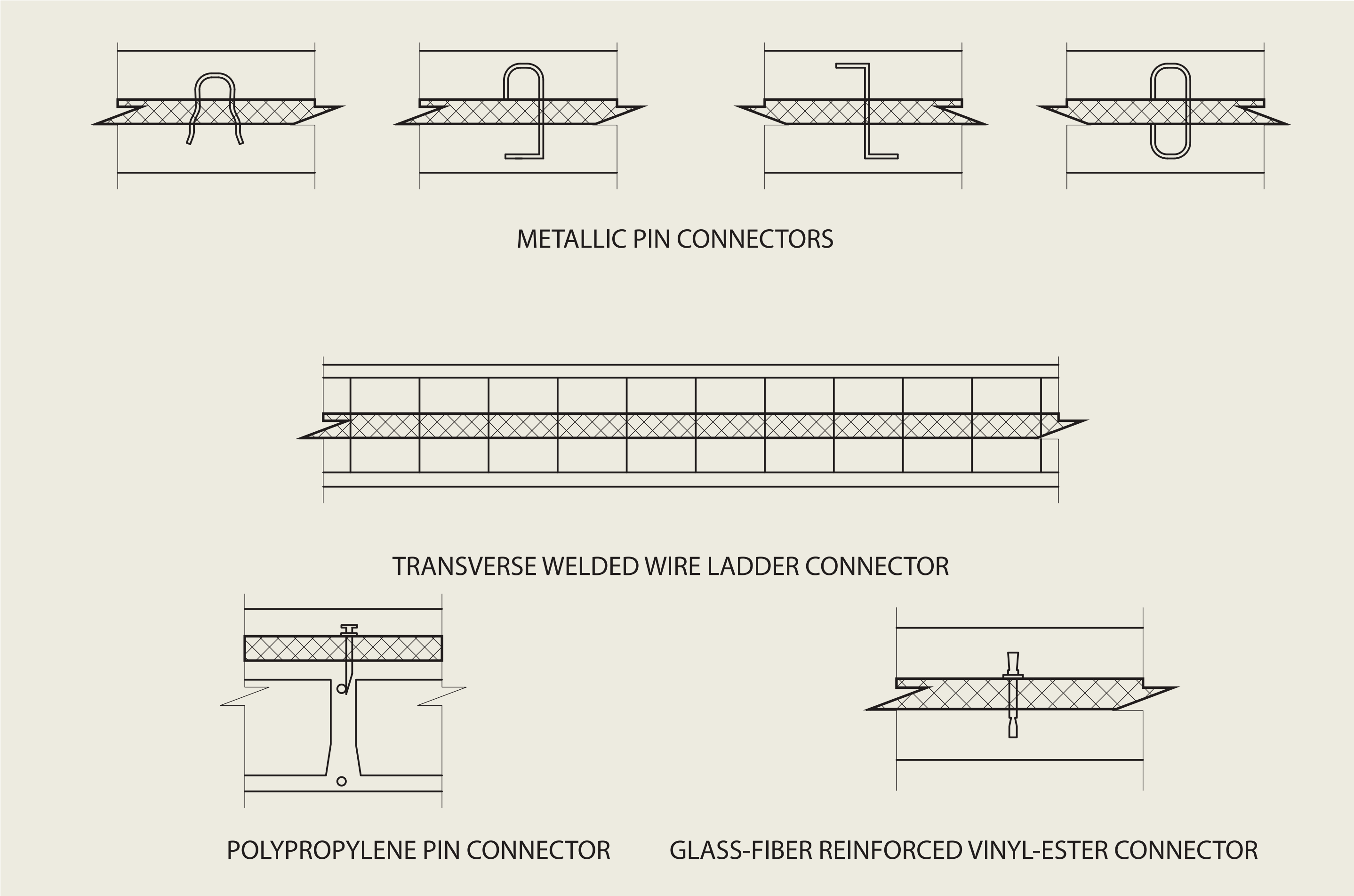


Figure 2.4.3. Noncomposite connectors.

is limited only by the handling capability of the plant, erection equipment, and transportation restrictions and the ability of the panel to resist the applied stresses. Local precasters should be contacted to verify optimal panel configurations and panel sizes.

Sandwich panels have been made as wide as 15 ft (4.6 m) and as tall as 75 ft (23 m), and overall thickness has varied from 5 in. (130 mm) to greater than 12 in. (305 mm) (wythe thickness is discussed in section 2.2). Insulation thicknesses have commonly varied from 1 in. (25.4 mm) to 4 in. (102 mm).

2.6 BOWING

Bowing in sandwich panels is a deflection caused by differential wythe shrinkage, eccentric prestressing force, thermal gradients through the panel thickness, differential modulus of elasticity between the wythes, or creep from storage of the panels in a deflected position. These actions cause one wythe to lengthen or shorten relative to the other. In panels with shear connectors, such differential wythe movement may result in curvature of the panel, or bowing. Because most sandwich panels exhibit some degree of composite action due to shear transfer by either bonded insulation or the wythe connectors, bowing can occur in all types of sandwich panels.

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 shapes are not precisely known.
- The degree of restraint provided by external connections is not precisely known.
- The degree of composite action in partially composite panels is generally not precisely known.
- An exact analytical model for each of the previous reasons, 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 partially composite panels. In this regard, it is similar to the imprecision in accurately predicting the camber of a double tee. It is important to realize that some bowing will occur and to establish a reasonable, allowable limit for the magnitude of bowing, often based on experience. Connections between the panels and the structural and nonstructural systems should then be designed so that distress is not 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 effect of temperature difference may be less than expected because actual thermal gradients are parabolic, not a straight line, as calculated. The design examples in the appendix reflect this approach. Detailing as related to bowing is discussed in section 2.11.

Some useful observations have been made by those experienced with the behavior of composite and partially composite sandwich panels:

- Panels bow outward most of the time.
- Panels exposed to the afternoon sun will bow more than those that are not; that is, 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 tend to 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 differing moduli of elasticity, such as panels with wythes containing different concrete strengths but with equal levels of prestress, will bow due to differential shortening and creep of the wythes after prestress transfer. The manufacturing process may contribute to this effect, particularly if the casting beds are heated. The bottom wythe will be covered with the panel's insulation layer as well as any curing blankets, so it will have greater maturity and a higher modulus than the top wythe at release of the panel prestressing. Because of this, the top wythe will immediately shorten more than the bottom wythe, resulting in a characteristic outward bowing of panels.

In addition, the possibility of differential bowing between adjacent panels must also be considered. Differential bowing occurs for multiple reasons:

- Adjacent panels may have different flexural stiffnesses, such as a flat panel adjacent to a stemmed panel.
- Adjacent panels may be connected to the structure at different points.
- Adjacent panels may be different lengths, commonly caused by steps in the footings.
- Adjacent panels may have different shrinkage, creep, or modulus-of-elasticity properties, especially if they were cast at different times.
- Adjacent panels may have different initial bows because of nonuniform storage conditions.
- Panels that meet at corners will not have identical bowing characteristics due to the different exposures to the sun. Panels bowing out at corners create a "fishmouth" effect at the joint. L-shaped panels at corners will also bow differently from adjacent flat panels.

To maintain integrity of the joint sealant between panels, 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. It is emphasized that architects and engineers should anticipate the potential for panel bowing and should check with the local precasters for expected bowing characteristics and methods to mitigate potential unwanted visual effects.

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 bowing cannot be determined by calculation. It is essential that all parties understand that there will be some bowing, that experience with similarly configured panels is the best method of predicting the magnitude of bow, and that the panel connections should be detailed accordingly.

For more information, read "Bowing of Insulated Precast Concrete Wall Panels" in the November–December 2003 issue of the *PCI Journal*.[3]

2.7 FLEXURAL DESIGN

2.7.1 Design

Sandwich 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 minimizing cracking in panels.

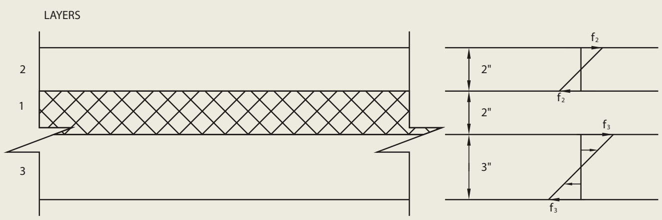
Restrained shrinkage due to solid areas of concrete and bond between insulation and concrete may result in cracking under service conditions if the panel is not prestressed. Thermal bowing and differential shrinkage bowing may cause cracking if the panel is restrained from bowing by intermediate connections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

Stresses for service loading are also checked. As stated previously, the stresses during handling are usually more critical than service load conditions.

Panels spanning horizontally are designed similarly to beams or spandrels.

2.7.2 Levels of Prestress

Many designers use a minimum level of effective prestress for each wythe, ranging from 225 psi (1550 kPa) to 600 psi (4136 kPa); these values have been successfully used to ensure that the panels will perform properly during stripping, handling, and erection and under service loads. According to the ACI code, transverse reinforcement is not required if the panel is prestressed to at least 225 psi.



$$I_2 = (1/12)(12)(2)^3 = 8\text{in.}^4$$

$$I_3 = (1/12)(12)(3)^3 = 27\text{in.}^4$$

$$S_2 = 8/1 = 8\text{in.}^3$$

$$S_3 = 27/1.5 = 18\text{in.}^3$$

$$M_2 = [I_2/(I_2+I_3)]M = (8/35)M = 0.23M$$

$$\therefore\ f_2 = (0.23M)/8 = 0.029M$$

$$M_3 = [I_3/(I_2+I_3)]M = (27/35)M = 0.77M$$

$$\therefore\ f_3 = (0.77M)/18 = 0.042M$$

Figure 2.7.3. Stress distribution in a noncomposite sandwich panel.

2.7.3 Noncomposite Panels

The flexural design of noncomposite sandwich panels is identical to that of solid panels that have the same sectional properties as the structural wythe(s) of the noncomposite 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 noncomposite panel would be as shown in Table 2.7.3.

As a practical matter, the 6 in. (150 mm) wythe in this case would probably be designed for 100% of the load. The deflection calculation would be based on a moment of inertia of 224 in.^{4}/ft (932 mm^4/m) width .

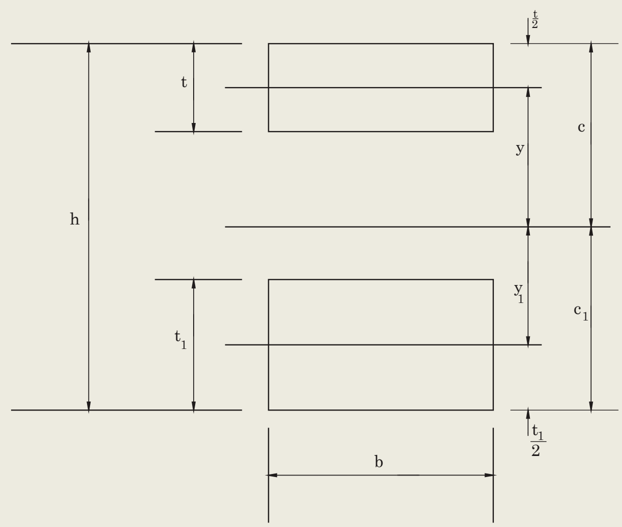
The theoretical stress distribution for a 2/2/3 sandwich panel is shown in Fig. 2.7.3. The 2 in. (51 mm) wythe resists 23% of the load and the 3 in. (76 mm) wythe resists 77% of the load.

2.7.4 Fully Composite Panels

The flexural design of fully composite panels is similar to a solid panel that has the same total thickness. The differences are that a mechanism for horizontal shear transfer between wythes must be provided and included in the analysis, and 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 (Fig. 2.7.4.a) and some comparisons between solid and sandwich flat panel section properties are listed in Table 2.7.4.

Table 2.7.3 Load distribution for 2/3/6 noncomposite panel

Wythe, in.	Moment of inertia, in.^{4}/ft width	Load, %
2	8	4
6	216	96
Total	224	100



$$A = b(t+t_1)$$

$$C = \frac{[1/2bt^2 + bt_1(h-1/2t_1)]}{A}$$

$$I = \frac{bt^3}{12} + bty^2 + \frac{bt_1^3}{12} + bt_1y_1^2$$

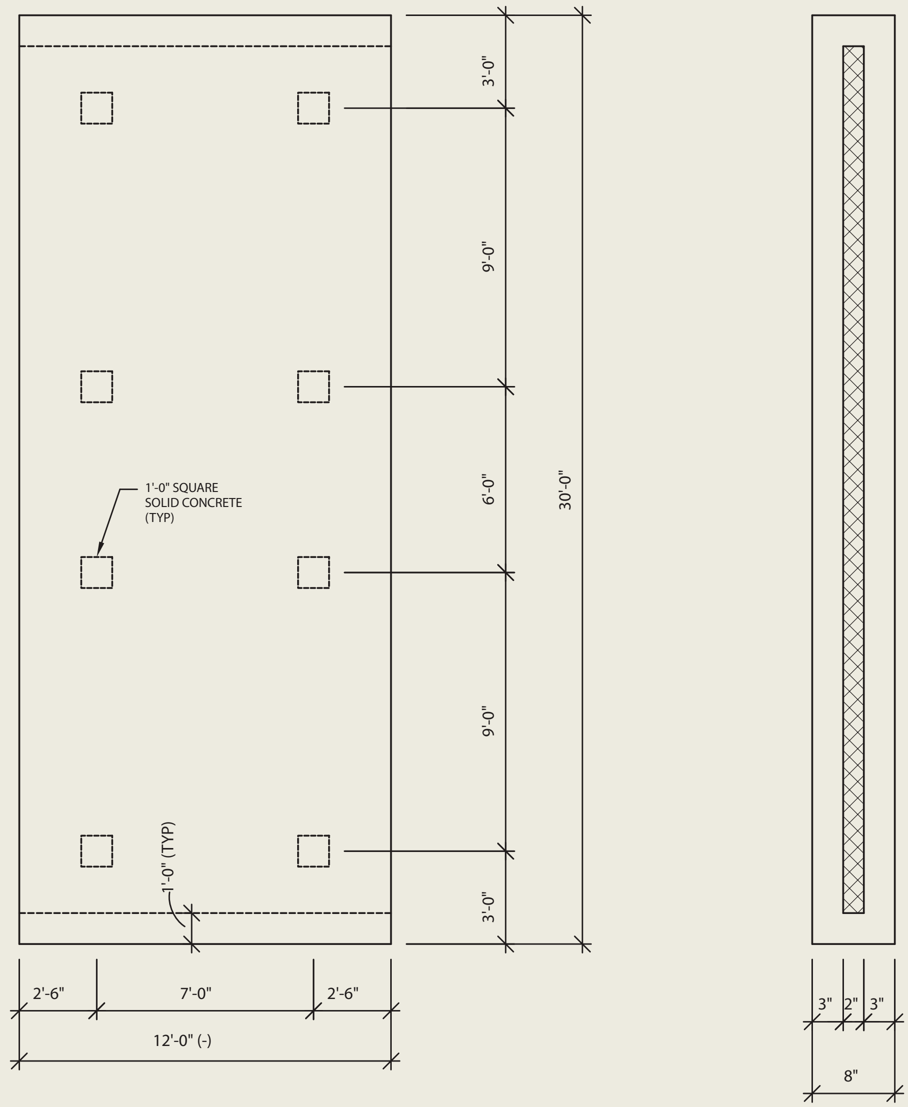
$$S = \frac{I}{C} \qquad S_1 = \frac{I}{C_1}$$

$$R = \left(\frac{I}{A}\right)^{\frac{1}{2}}$$

Figure 2.7.4.a. Section properties of composite sandwich panel.

Table 2.7.4 Example comparisons of solid versus sandwich panel section properties

Panel configuration	Total thickness	I_{solid}, in.^{4}/ft width	S_{solid}, in.^{3}/ft width	I_{comp}, in.^{4}/ft width	S_{comp}, in.^{3}/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



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:

$(4)(12\ \text{in.})(12\ \text{in.}) + (12\ \text{in.})(14\ \text{in.}) = 2304\ \text{in.}^2$

$V_{nh} = (0.080\ \text{ksi})(2304\ \text{in.}^2)$

$= 184\ \text{kips} > 178\ \text{kips}$

$\therefore$ Composite action is achieved.

Figure 2.7.4.b. Sample calculation of horizontal shear in 3/2/3 composite sandwich panel.

The calculation for horizontal shear transfer is made using the strength concept as given in the *PCI Design Handbook*, section 4.3.5.[4] In this approach, the maximum compressive capacity of the concrete is compared with 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 of the clear span length.

A sample calculation is shown in Fig. 2.7.4.b for a 3/2/3 composite panel. Each wythe is prestressed with eight 3/8-in.-diameter (9.53 mm), 270 ksi (1.86 MPa) strands, and the concrete strength is 5000 psi (35,000 kPa). The panel has a 1 ft (0.305 m) solid horizontal band of concrete at each end and eight 1-ft-square solid zones located at the lifting points. Although the calculations shown in Fig. 2.7.4.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 and panel ends.

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

2.7.5 Partially Composite Panels

Partially composite panels are panels in which the wythe ties provide only a portion of the horizontal shear transfer (less than needed for a fully composite panel). It is expected that only a percentage of the fully composite behavior (panel strength or stiffness) will be achieved. This percentage is determined by the designer and based on known panel behavior of similar existing panels using the same construction. A partially composite panel may be as stiff as a fully composite panel until ultimate failure occurs, or it may have much less stiffness and still attain a high moment capacity. Its behavior is dependent on the inherent ductility of the horizontal shear connectors.

2.7.6 Other Considerations

An important design check that should be made is the ACI 318-05 requirement in chapter 18.8.2 that states that the flexural strength of the panel exceeds 1.2 times the cracking strength. (Mild-steel-reinforced panels without prestressing need only to exceed 1.0 times the cracking strength, according to ACI 318-05, section 14.8.2.4.) In prestressed sandwich panels, the level of prestress, and thus the cracking strength, may be sufficiently high so that supplemental mild-steel reinforcement may need to be added to increase the flexural strength. This requirement does not apply for compression-controlled sections, which are not common. For tension-controlled sections, it only applies where the panel stresses are greatest, usually at midspan for simply supported panels. It would not usually be appropriate to apply this provision near the ends of a panel, where the reinforcement is not fully developed and stresses are low.

2.8 LOAD-BEARING DESIGN

2.8.1 Loading

Load-bearing sandwich panels are designed for various loads, including self-weight, roof (and floor) dead and live, wind, seismic, load from adjacent panels, soil (lateral pressure), temperature, and 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 5.8.5 of the *PCI Design Handbook* and section 2.6 of this report; each effect may create forces between adjacent panels.

The loadings listed previously are external; that is, they act on the entire structure. In the case of a sandwich panel, they relate to the panel in its erected position. Panels are also 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 8.2, 8.3, 8.5, and 8.6 of the *PCI Design Handbook*.

2.8.2 Design

The strength design of a load-bearing sandwich panel is the same as for compression members as described in section 5.9 of the *PCI Design Handbook*. Secondary moments associated with 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 plumbness 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 5.9.3 of the *PCI Design Handbook*. The stiffness reduction factor (φ-k) is assumed to be at least 0.85, considering the stringent dimensional accuracy of panel construction tolerances provided by a precasting plant. The maximum deflection due to service loads, including P-Δ effects, shall not exceed length/150, according to ACI 318-05 section 14.8.4. ACI 318-05 section 14.2.7 states that the maximum height/thickness ratio of 25:1 specified in section 14.5 can be waived when a structural analysis is done that includes secondary effects (P-Δ). Interaction curves for various wythe thicknesses and strand configurations can be determined using the methods described in section 5.9 of the *PCI Design Handbook* or from available specialized computer programs.

2.8.3 Noncomposite Panels

For flat noncomposite 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 stem is generally chosen to be the structural wythe. If the nonstructural wythe does not bear on the structure below, its weight must be transferred to the structural wythe. Connectors are used to support the nonstructural wythe as described in section 2.4.1. Examples of noncomposite designs are detailed in appendix A. Flexural design of noncomposite panels is described in section 2.7.3.

2.8.4 Fully Composite Panels

In a fully composite panel, the two wythes act together to resist handling and service loads. 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-wire trusses, or regions of solid concrete that join both wythes are used to achieve composite behavior. Loading and design are the same as described in section 2.8.1 and 2.8.2. Examples of composite panel designs are detailed in appendix A. Flexural design of fully composite panels is described in section 2.7.4.

2.8.5 Partially Composite Panels

Partially composite panels have enough shear-transfer capability between wythes to be stiffer than similar noncomposite panels but are not as stiff as fully composite panels. In addition, because there is some transfer of load between wythes, a tension/compression couple forms that increases the overall flexural resistance of the panel. This panel type is often proprietary and, if so, test reports, design charts, or software should be available from the system supplier to aid in design. The precast concrete engineer may assume a percentage of composite action that is justified by testing and experience.

2.9 SHEAR WALL CONSIDERATIONS

2.9.1 General

Sandwich 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 its relative stiffness. The design of shear wall buildings is performed in accordance with sections 4.3 and 4.5 of the *PCI Design Handbook*.

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 small groups of connected 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 those illustrated in 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 4.5 of the *PCI Design Handbook*.

2.9.2 Noncomposite Panels

Lateral load resistance of a noncomposite 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 wythe.

2.9.3 Fully Composite Panels

The overturning and shear resistance of a composite panel are based on the composite section of the sandwich panel if composite action is also provided in the transverse direction. Connections can be made to either wythe and are usually located at solid areas where the insulation is interrupted.

2.9.4 Partially Composite Panels

The in-plane lateral shear resistance of partially composite panels may be based on the wythe connected to the structure, in the same manner as for a noncomposite panel, when the shear connectors transmit shear force in the vertical direction only. Lateral shear stress within the panel is usually very small, so the connected wythe alone is normally more than adequate for the job. In some cases, the wythe connectors have weak-axis stiffness sufficient to transmit shear forces in the horizontal direction so that both wythes may be considered to resist lateral loads.

2.10 EXTERNAL CONNECTIONS

2.10.1 General

This section includes examples of connection details of sandwich panels to foundations, to other framing members of a structure, and to each other. The details are neither all-inclusive nor necessarily the best possible arrangement. Most of the details come from sandwich panel system brochures, precasters' standard connections, and actual construction projects. The details are included to present ideas and illustrate connections that are commonly used. The engineer of record should consult with local precasters when planning connection design. Precasters can advise which types of connections would be most appropriate and economical for a given sandwich panel application. The engineer of record usually provides the lateral force values to be transferred by the connections to the panels from the floor and roof diaphragms.

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 in the *PCI Design Handbook* and PCI MNL-123, *Design and Typical Details of Connections for Precast and Prestressed Concrete,* second edition.[5] The requirements for structural integrity, as outlined in the *PCI Design Handbook* and ACI 318-05 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
- panel–to–intermediate floor connections
- corner panel connections

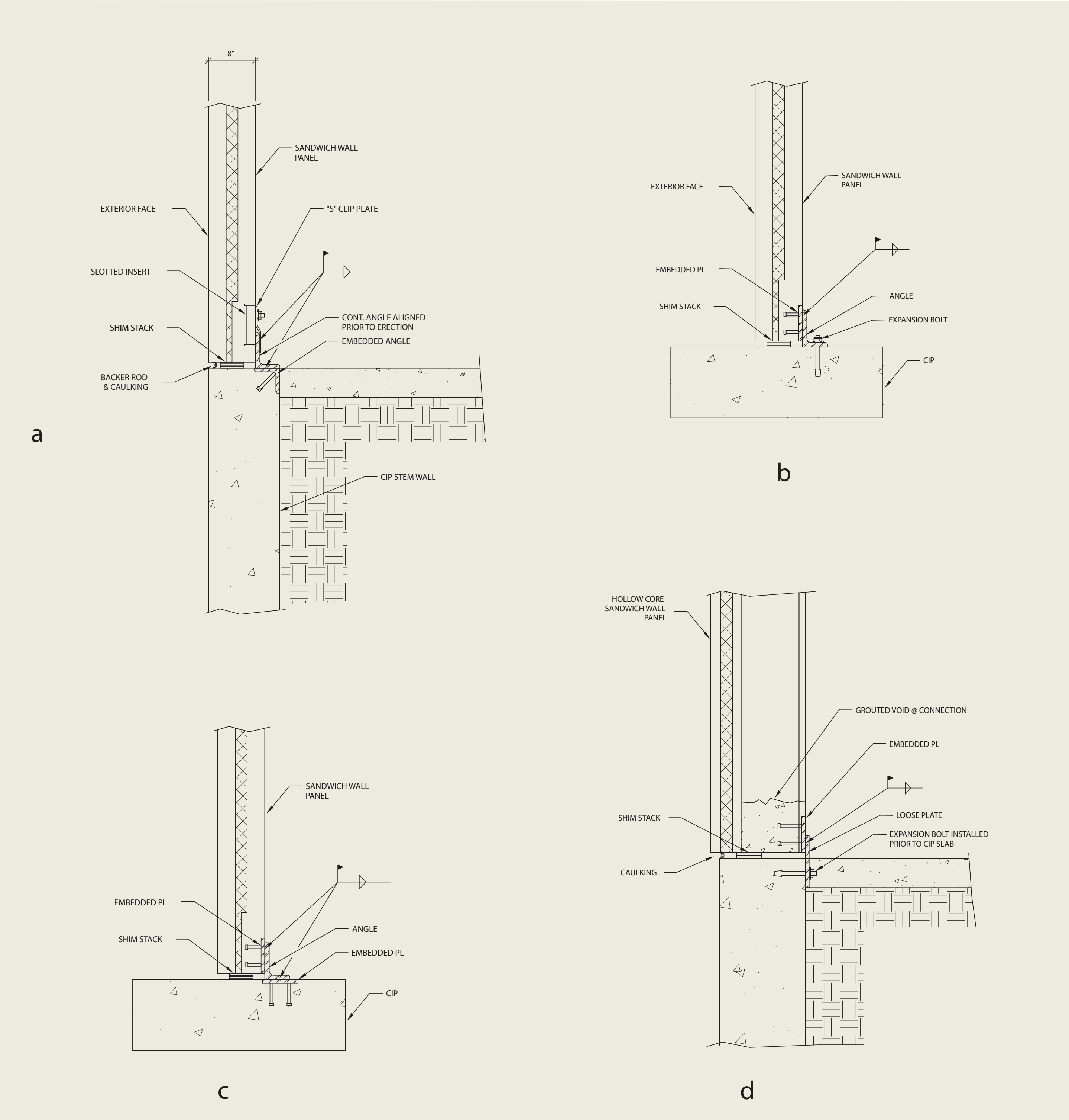


Figure 2.10.2.1. Non–shear wall to foundation connections.

- panel-to-panel connections

2.10.2 Panel Base–to–Foundation Connections

Five types of panel base–to–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

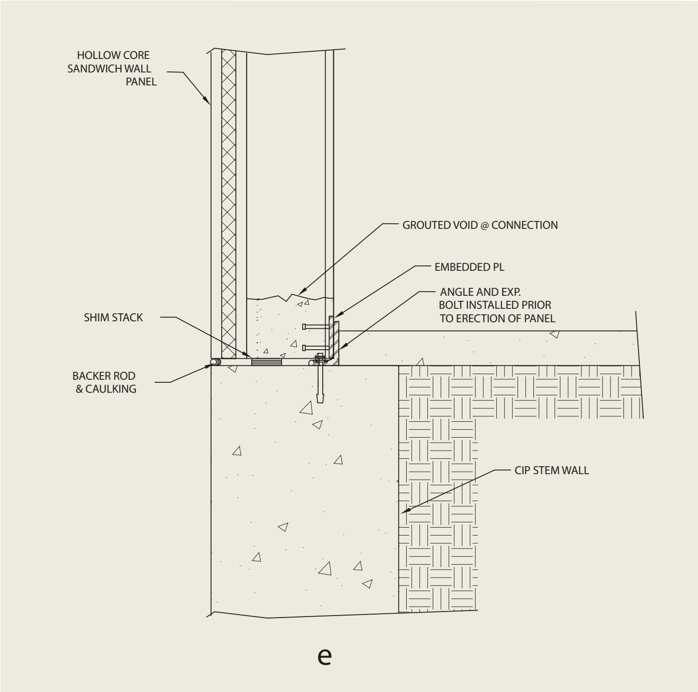


Figure 2.10.2.1 (cont.). Non–shear wall to foundation connections.

2.10.2.1 Non–Shear Wall Panels to Concrete Foundations (Fig. 2.10.2.1.a–e)

The panels are cast with either embedded weld plates or slotted inserts. The concrete foundation walls or footings have embedded weld plates or drilled-in expansion anchors. The sandwich 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-load-bearing panels or completely grouted for load-bearing panels. Temporary bracing or attachment to a braced structure provides panel support during erection.

Features:

- Drill-in expansion anchors eliminate tolerance problems.
- Extending panels to footing eliminates cost of forming and placing concrete for cast-in-place walls.

Considerations:

- Care needs to be taken concerning coordination and tolerance between embedded plates in cast-in-place walls or footing and embedded plates in precast concrete panels.
- Temporary bracing is needed if the panel is erected before the supporting structure.
- Connection may not be concealed, which may require corrosion protection.
- There is a need to grout under both wythes unless the panel is designed to support the outer wythe from the inner wythe.
- The panel needs to have a solid bottom section (eliminating insulation for 1 ft [0.305 m] or so may reduce the insulating properties at the base of panel).

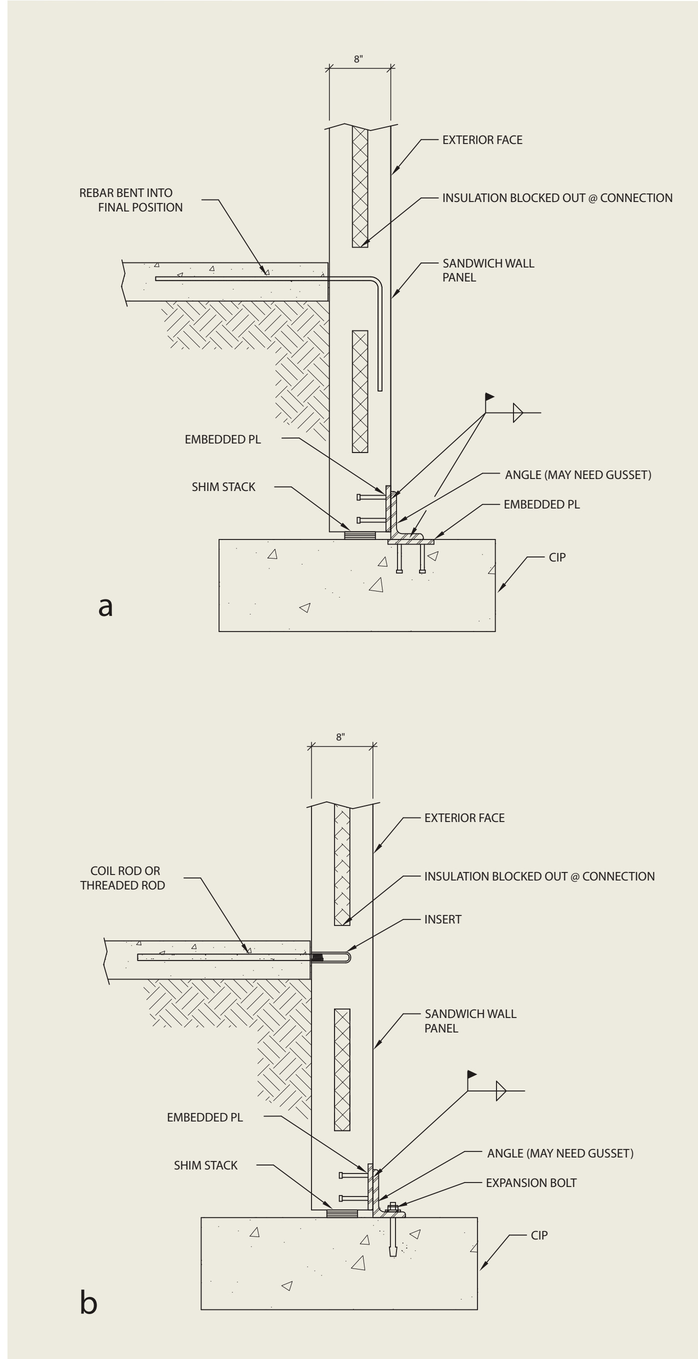


Figure 2.10.2.2. Sample connections for panel bases used for retaining wall.

- S-clip and expansion anchor connections may not provide sufficient uplift resistance.

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

The comments in section 2.10.2.1 apply to this section as well. 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:

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

- Connection to the floor reduces unsupported panel height.
- Extending panels to the footing eliminates the cost of forming and placing concrete for cast-in-place walls.

Considerations:

- The force in the floor slab connection may be large. Forces due to differential temperature and shrinkage between wythes should be accounted for, in addition to wind suction and axial load eccentricity.
- Panel moments at the floor slab are resisted by strands that may not be fully developed, and mild-steel reinforcement may be required.
- Temporary bracing is needed until concrete for the floor slab is placed.
- Backfill placed against the panel could force the panel out of plumb or induce a permanent bow before the connection is made.
- Care needs to be taken concerning coordination and tolerances between embedded plates in cast-in-place footings and embedded plates in precast concrete panels.
- The consequences of a future removal of the floor slab, thus eliminating the connection, should be considered.
- The solid section at the bottom of the panel may reduce the thermal efficiency at the panel's base.

2.10.2.3 Slotted Connections (Fig. 2.10.2.3.a and b)

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

Features:

- There are no embedments, thus no coordination or tolerance problems with embedments not lining up.
- Erection is sped up because there is no welding or bolting at the foundation.

Considerations:

- There is no positive connection to the footing, especially for uplift.
- Temporary stability is dependent on the panel being wedged in the 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 concrete panels is used. The panels and foundations and their connections are designed for the uplift forces and structural integrity requirements. Temporary bracing or attachment to a braced structure achieves panel support during erection.

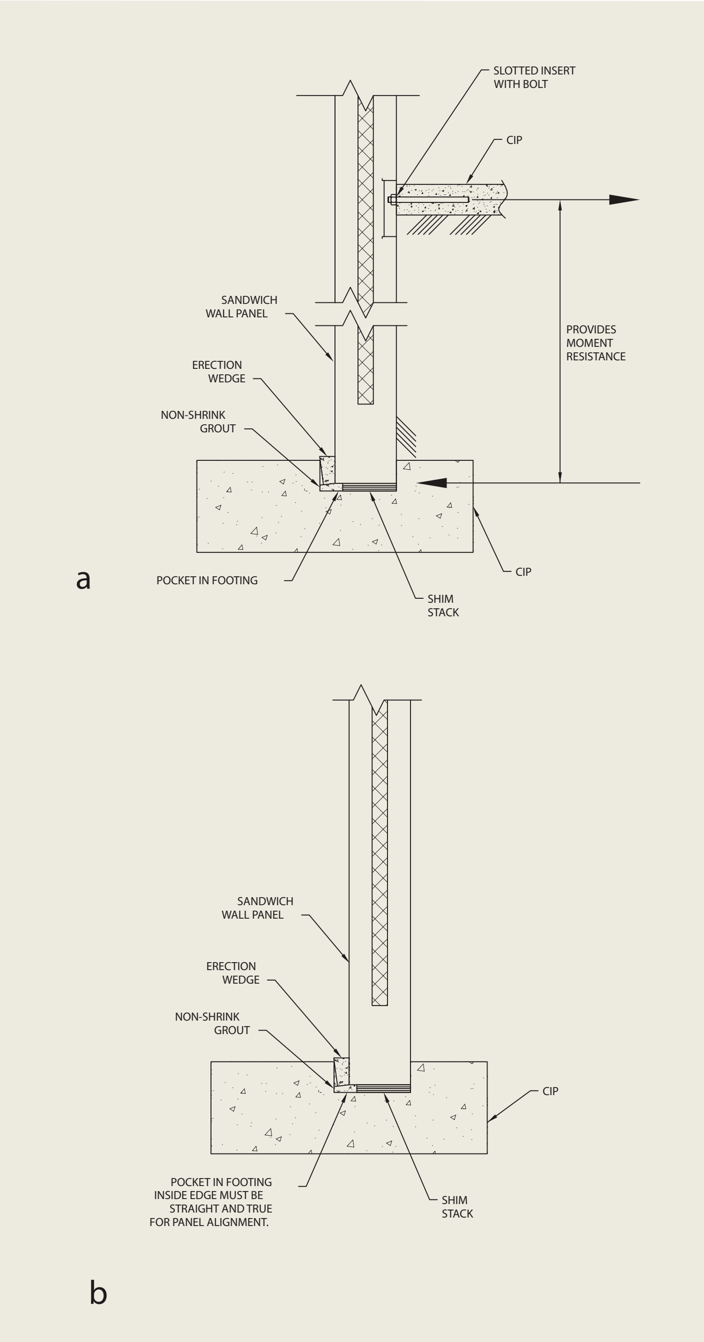


Figure 2.10.2.3. Slotted base connections.

Features:

- This is a simple, economical method that resists wind and seismic forces.
- Several panels can be connected together to increase the lateral load resistance and structural stiffness.

Considerations:

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

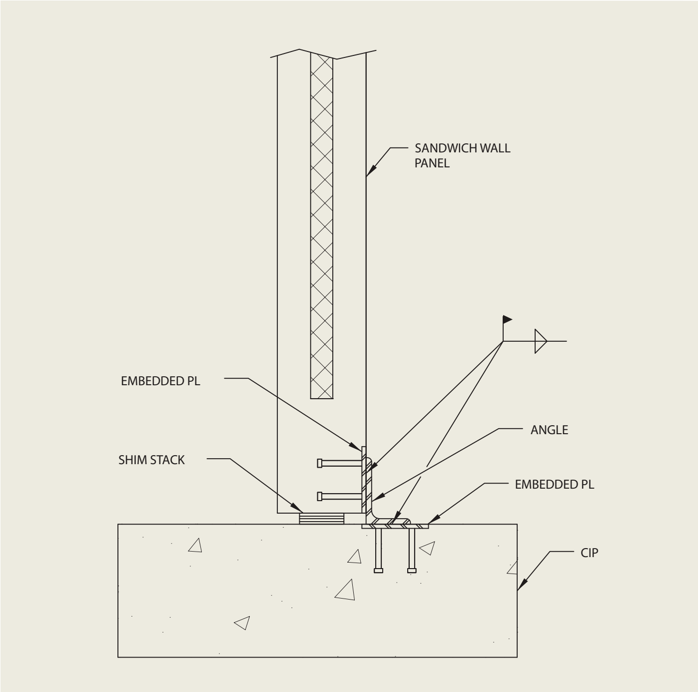


Figure 2.10.2.4. Shear wall panel–to–foundation connection (no uplift).

- Temporary bracing is needed if the panel is erected before the rest of the structure.

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

Load-bearing wall panels are usually erected before the support structure because the roof and floor members bear directly on haunches or ledges in the panels. A common way to temporarily brace the panels until the rest of the structure is constructed is to use pipe bracing to cast-in-place slabs or 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 the panels are erected. There are numerous alternative arrangements that are commonly used, including complete precast concrete framing with a variety of precast concrete roof and floor systems. This alternative requires temporary bracing for panels only on the leading edge of the construction.

Features:

- Bracing systems permit the sandwich walls to support the roof framing and eliminate permanent beams and columns adjacent to the panels.
- Deadmen, pipe bracing, and patching inserts are eliminated by the temporary column and beam system.

Considerations:

- Deadmen are often temporary. Buried deadmen, which may be permanent, need to be low enough so they do not act as unintended supports for the floor slab or adjacent parking lot.
- Inserts for the connection of pipe braces may need to be patched after the bracing is removed.
- Cable guys must be strong enough to resist the lateral forces of the building with the panels installed and not just the lightweight and open steel structure.
- Pipe-braced panels need to be erected before perimeter joists are installed.

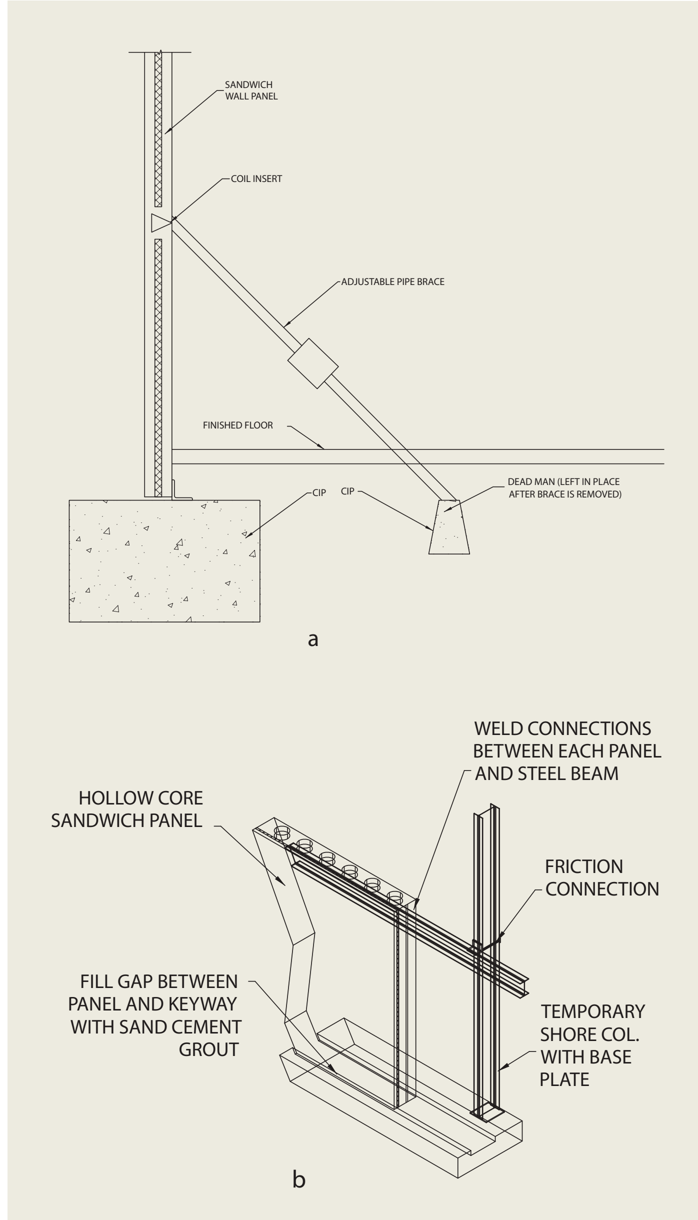


Figure 2.10.2.5. Temporary panel bracing.

2.10.3 Panel Top–to–Roof Connections

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

- non-load-bearing wall panels to beams or joists
- load-bearing wall panels to roof joists

- load-bearing wall panels to metal roof decks
- load-bearing wall panels to hollow-core slabs
- load-bearing wall panels to beams or joist girders
- load-bearing wall panels to double-tee roof decks

2.10.3.1 Non-Load-Bearing Wall Panels to Beams or Joists (Fig. 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, the connections are designed for the roof diaphragm forces.

Features:

- Embedded weld plates or slotted inserts together with steel roof members give latitude in erection tolerance.
- Connections are somewhat flexible and accommodate movements.

Considerations:

- Roof framing has to be erected before wall panels. If not, the panels need temporary bracing.
- Solid concrete at the embedded plates reduces the insulating properties of the sandwich panels.
- Often connections must be made to the bottom flanges of the beam or joist because roofing is already in place. In those cases, the panel designer usually supplies the engineer of record with reactions so he can design any kickers, if required.

2.10.3.2 Load-Bearing Wall Panels to Roof Joists (Fig. 2.10.3.2.a, b, and c)

Roof joists can bear either directly on 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:

- Load-bearing panels eliminate exterior beam and column framing.
- A continuous angle or beam ledge permits variable joist spacing.

Considerations:

- Walls have to be erected and temporarily braced before roof framing is erected. Alternatively, temporary columns can be used to support an edge beam, thus allowing the roof to be erected first and eliminating bracing of the wall panels.
- Solid concrete at the embedded plates reduces the insulating properties of the sandwich panel.
- Solid concrete at the embedded plates may hinder differential thermal movements.
- Continuous embedded angles in the tops of panels need to be slotted as required for prestressing strands to pass through.

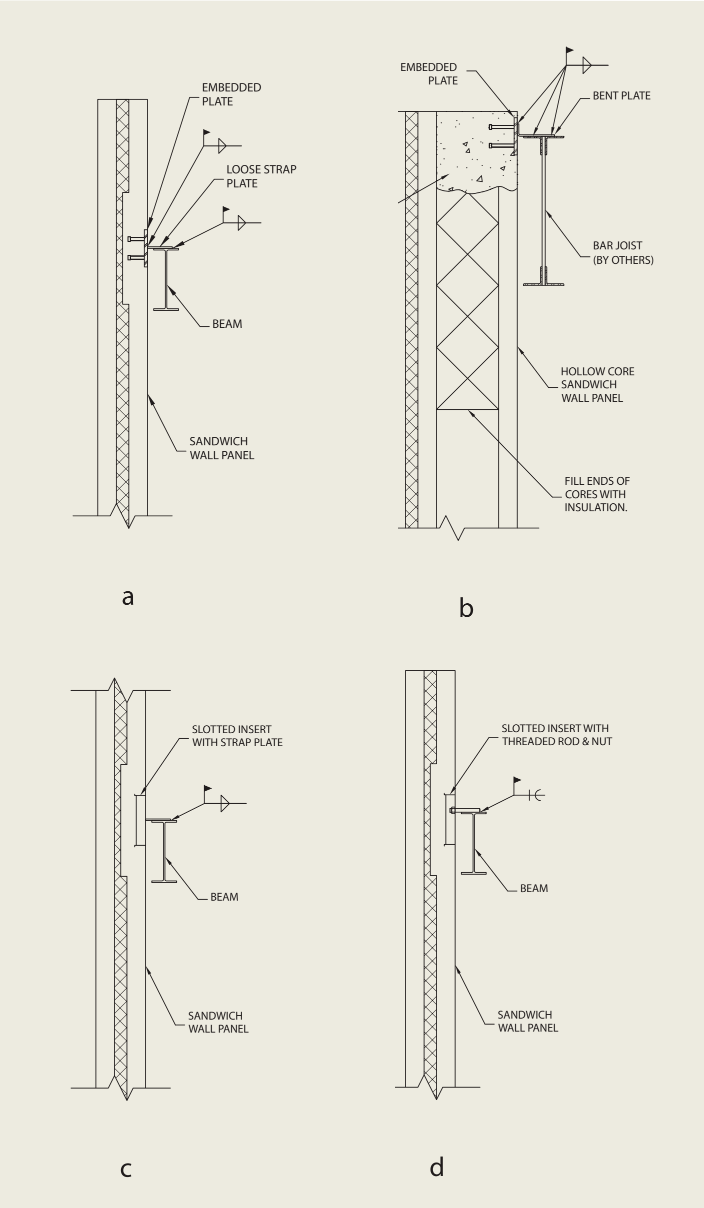


Figure 2.10.3.1. Non-load-bearing panels to beam or joist connections.

2.10.3.3 Load-Bearing Wall Panels to Metal Deck Roofs (Fig. 2.10.3.3.a and b)

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

Features:

- Load-bearing panels eliminate edge beams or joist.
- Ledge members can be installed to meet the roof slope.

Considerations:

- Walls have to be erected and temporarily braced before the roof deck supported by the wall can be installed.
- Solid concrete at the embedded plates reduces the insulating properties of the sandwich panel.

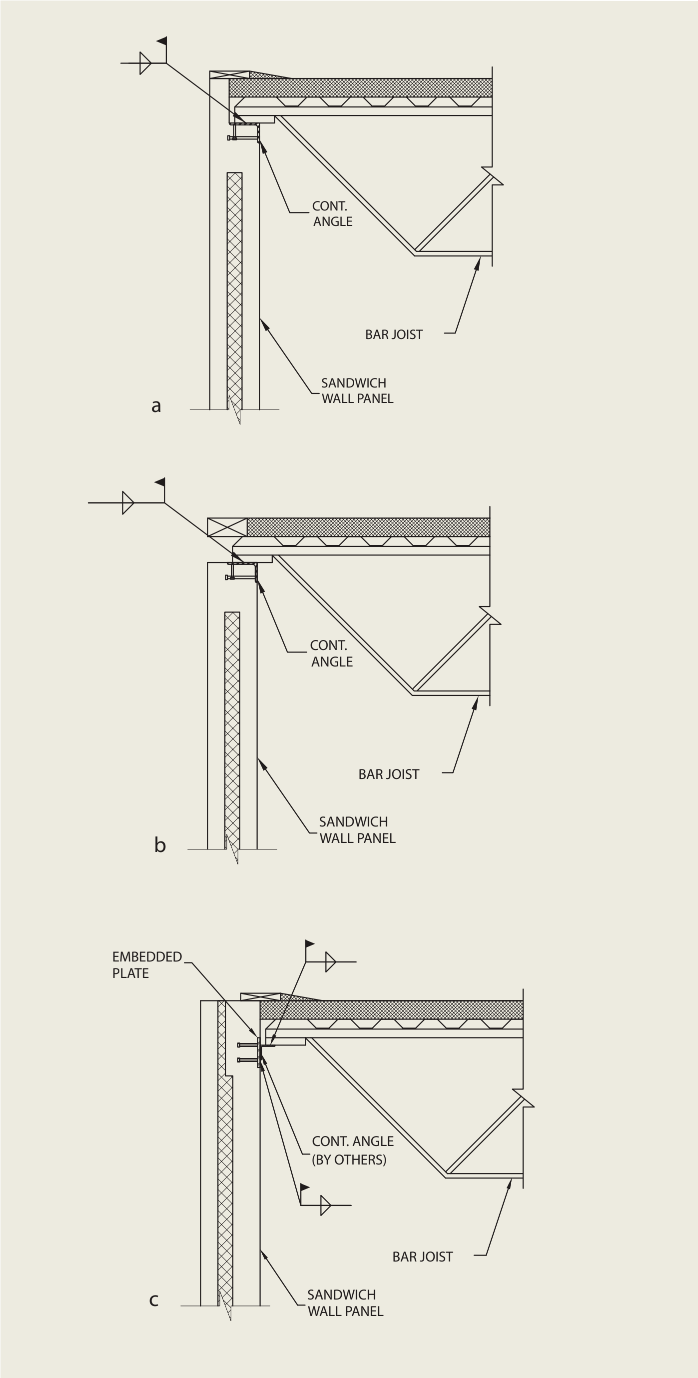


Figure 2.10.3.2. Load-bearing panels to beams or joist girder connections.

2.10.3.4 Load-Bearing Wall Panels to Hollow-Core Slabs (Fig. 2.10.3.4.a–c)

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

Features:

- Load-bearing panels eliminate edge beams or joists.
- Ledge members can be installed to meet the slope of hollow-core slabs.

Considerations:

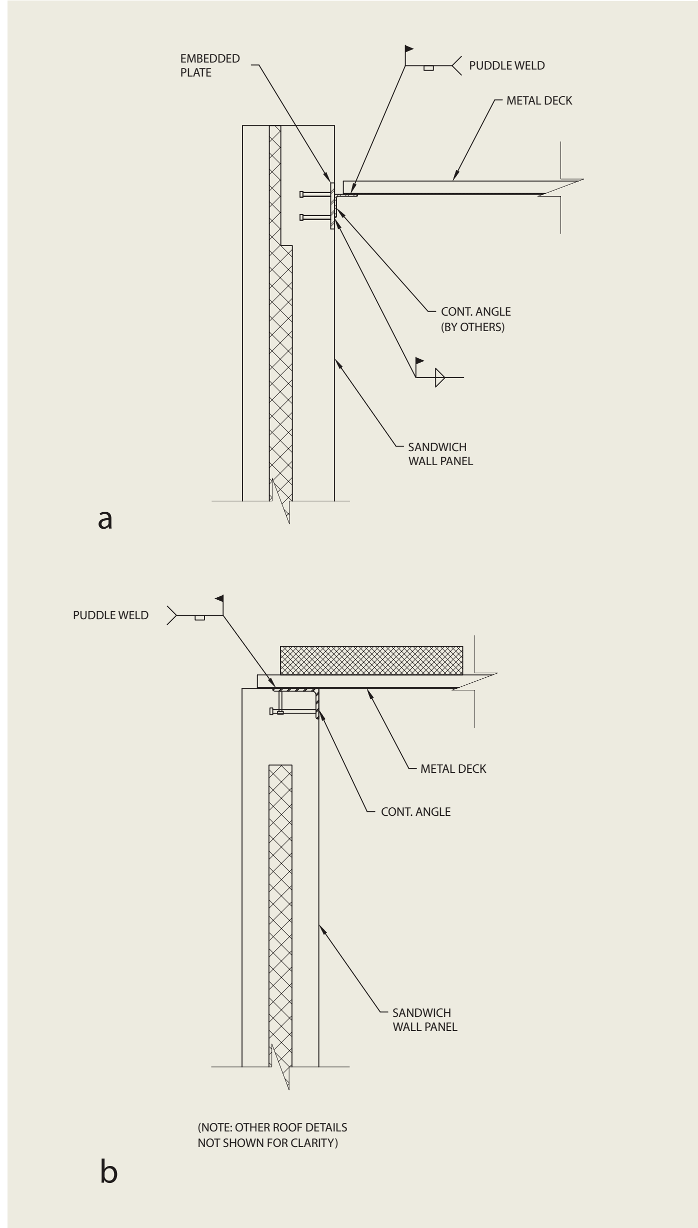


Figure 2.10.3.3. Load-bearing panels to metal deck roof connections.

- Walls have to be erected and temporarily braced before hollow-core slabs are erected.
- Solid concrete at the embedded plates reduces the insulating properties of the sandwich panels.
- Special embedded plates in hollow-core slabs are required.

2.10.3.5 Load-Bearing Wall Panels to Beams or Joist Girders (Fig. 2.10.3.5.a–c)

Beams or joist girders can bear either 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:

- Load-bearing panels eliminate exterior columns.

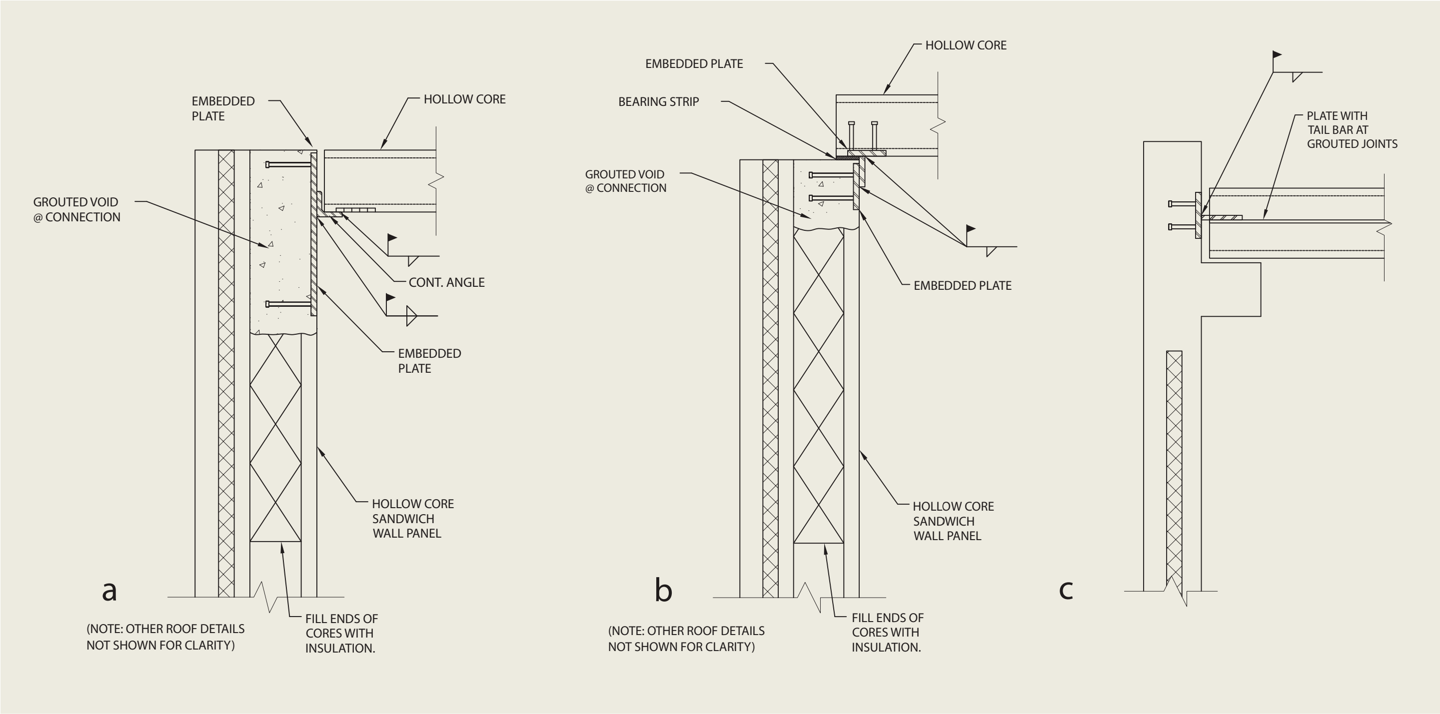


Figure 2.10.3.4. Load-bearing panels to hollow-core slabs.

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

Considerations:

- Walls have to be erected and temporarily braced before roof beams are erected.
- Solid concrete at the bearing area or at embedded plates reduces the insulating properties of the sandwich panels.

2.10.3.6 Load-Bearing Wall Panels to Double-Tee Roof Decks (Fig. 2.10.3.6)

Double-tee roof decks can bear either directly on top of the wall in pockets or on haunches. Panels with haunches should be designed for the eccentric loading from the double tees.

Features:

- Load-bearing panels eliminate exterior columns.
- A pocketed connection eliminates the need for haunches and eliminates eccentricity of the double tee reaction.
- Both types of connections allow for easy diaphragm connection to the wall panel.

Considerations:

- Walls have to be erected and temporarily braced before double tees are erected.
- Careful consideration should be given to the location toler-

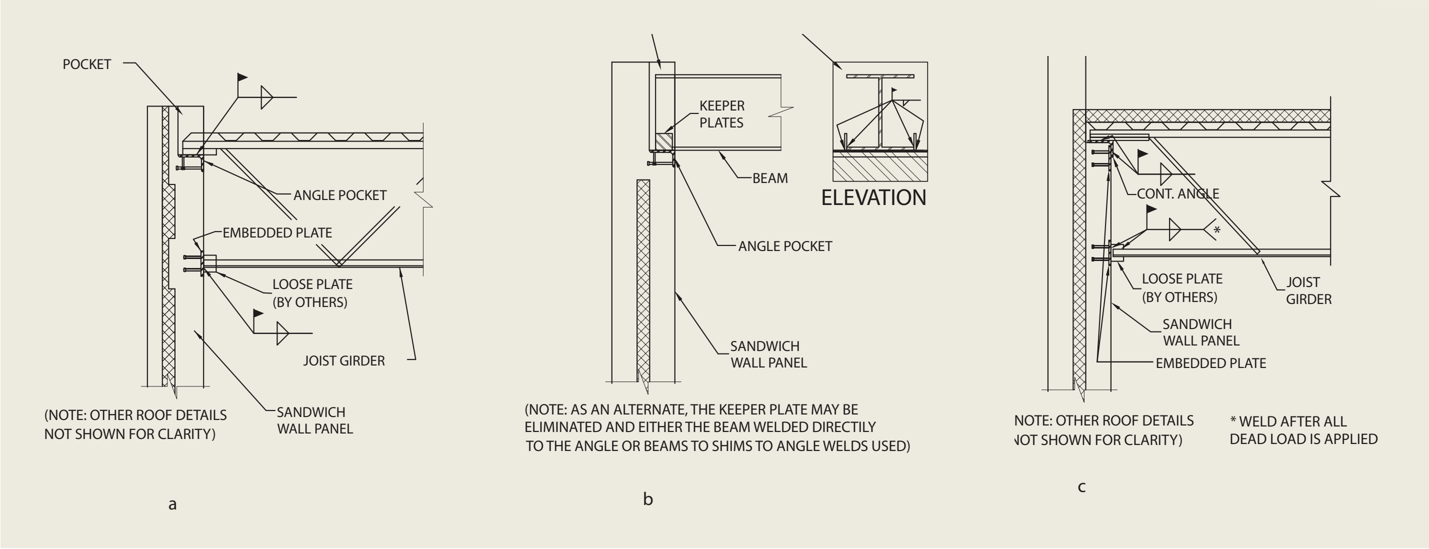


Figure 2.10.3.5. Load-bearing panels to beams or joist girder connections.

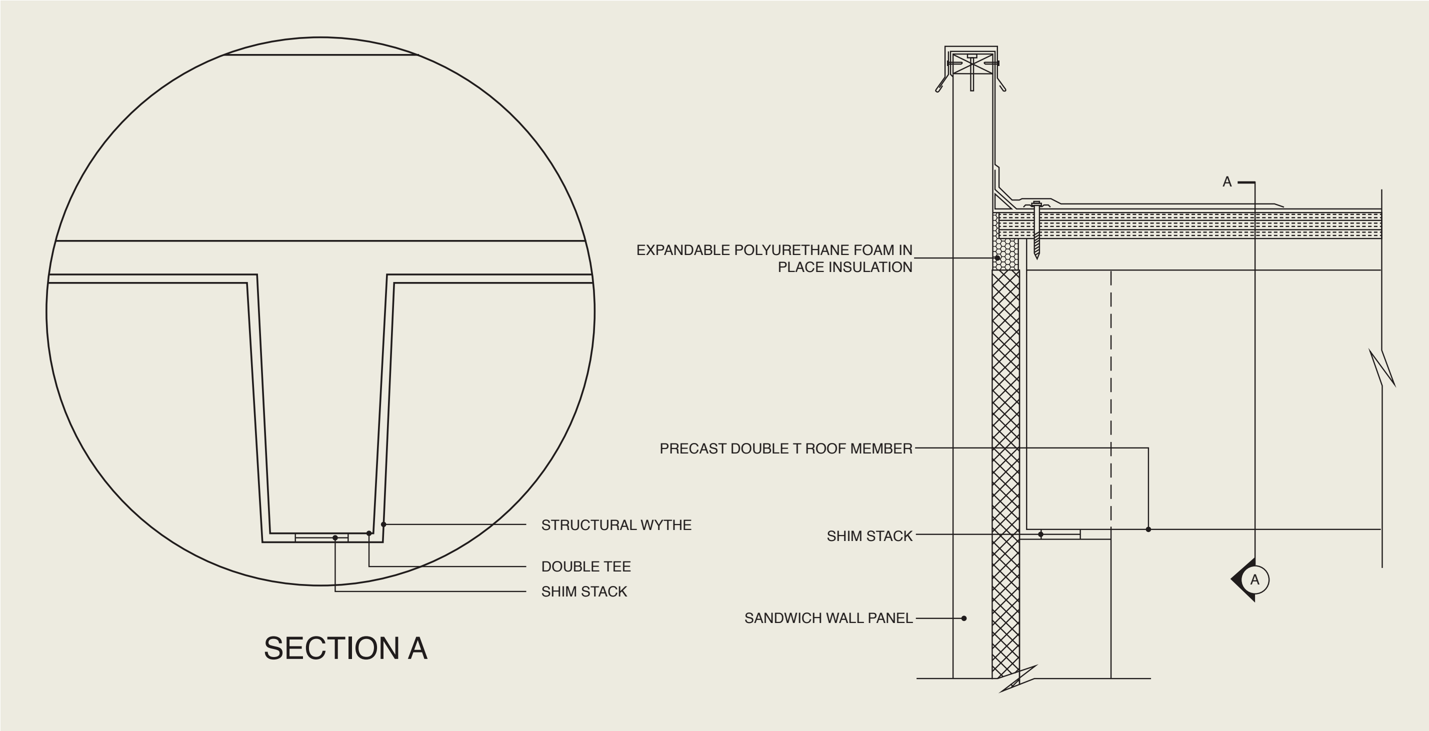


Figure 2.10.3.6. Load-bearing wall panels to double-tee roof decks.

ance of the pockets in the wall during the construction of wall panels because the double tee stems will have to fit in the pocket.

2.10.4 Panel–to–Lintel Beam Connections (Fig. 2.10.4.a–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 (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:

- Quick, easy erection is attained; tolerances are adequate.
- Wide openings can be achieved with economical framing and connections.

Considerations:

- Adjacent panels have to be erected before lintels and door panels. Temporary bracing of door panels and adjacent panels can be difficult.
- Torsion effects on lintel beams can cause rotations if not properly designed.
- Solid concrete at the embedded plates reduces panel-insulating properties.
- Careful thought needs to be given to the bowing and restraint of bowing of the door panel and adjacent panels.
- Adjacent panels may require more strands or reinforcement to support the additional wind and gravity loads.

2.10.5 Panel–to–Intermediate Floor Connections (Fig. 2.10.5.a–d)

Load-bearing panels with continuous ledge members can support intermediate floors or mezzanines (Fig. 2.10.5.a). Nominal or lateral connections to intermediate floors need to be designed for the forces that result from restraining panel bowing.

Features:

- Load-bearing panels eliminate exterior beam and column framing or mezzanines and intermediate floors.
- Intermediate connections reduce bowing and unsupported height.

Considerations:

- Large forces may develop due to restraint of panel bowing.
- Any eccentricity of nonbearing connections that will cause torsion on support beams needs to be considered.
- Solid concrete reduces insulating properties of the sandwich panels.

2.10.6 Corner Panel Connections (Fig. 2.10.6.a–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

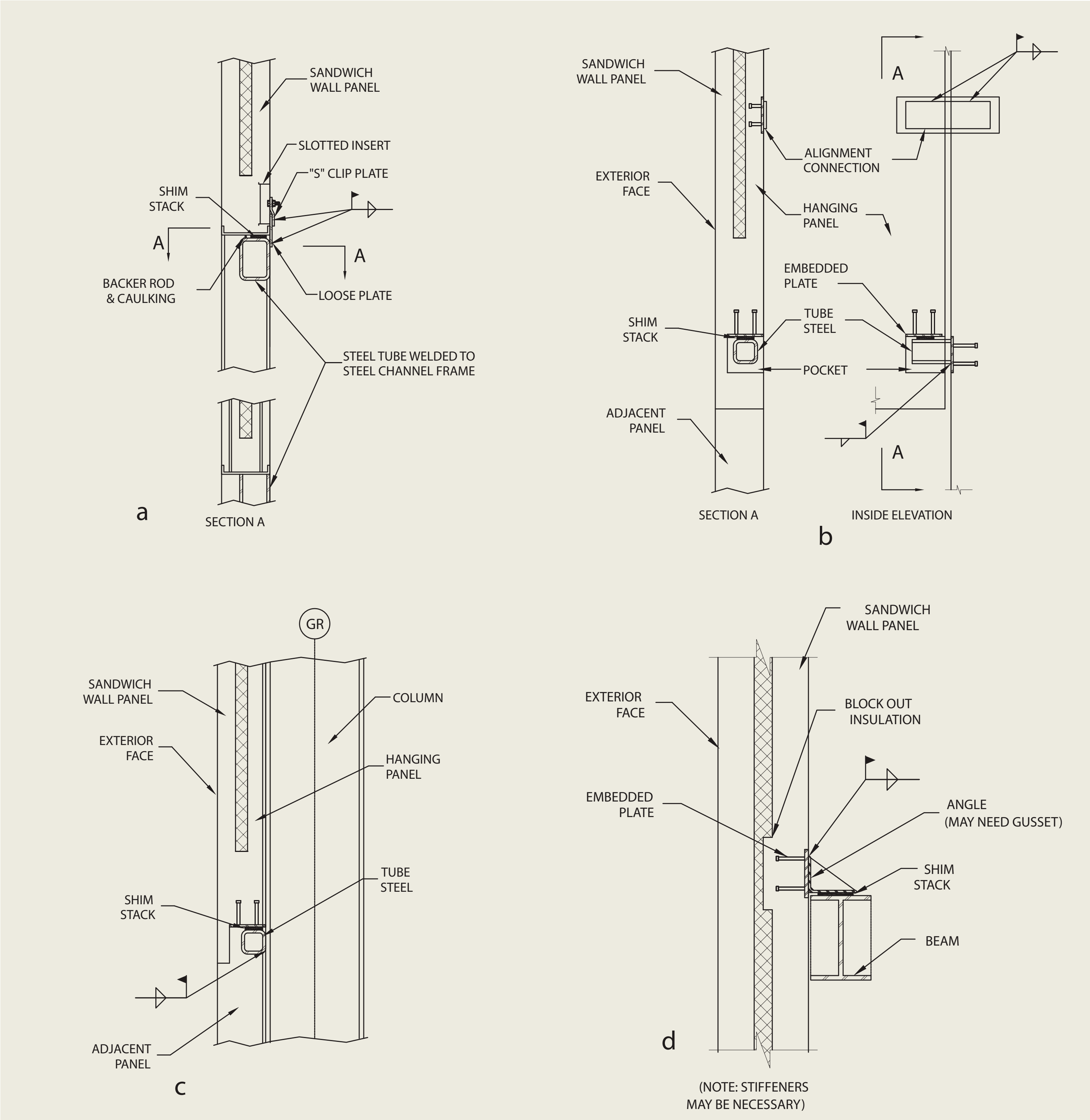


Figure 2.10.4. Panel–to–lintel beam connections.

(see *PCI Design Handbook*, section 5.8.5). Separate corner panels can also be used to help the bowing separation problem (Fig. 2.10.6.b and f). Mitered corners should have a quirk detail, usually 1 in. × 1 in. (25.4 mm × 25.4 mm) (Fig. 2.10.6.b, c, d, and g).

Features:

- Connecting panels together or using special corner panels eliminates separation and alignment problems.
- Corner columns can be eliminated if sandwich panels are load-bearing.

Considerations:

- Corner connection forces due to bowing can be high.
- Corner connections are more rigid than typical panel-to-panel connections.
- Solid areas reduce the insulating properties of sandwich panels.

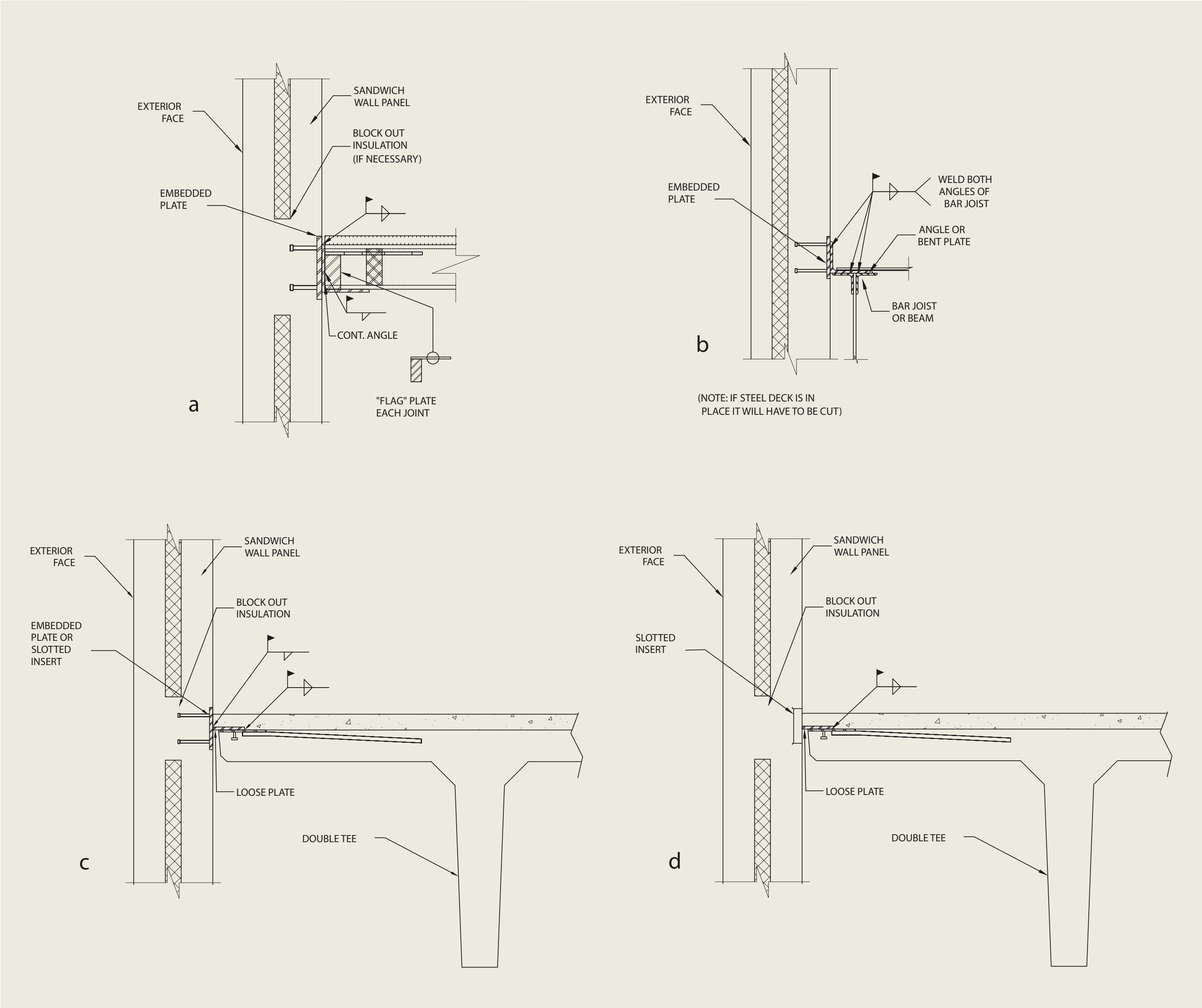


Figure 2.10.5. Panel to intermediate floor connections.

- Connecting corner panels to columns can cause lateral loads to act on the columns, which needs to be considered by the building designer.
- Strand tension can be adjusted between wythes to induce a slight inward bow to compensate for the tendency of these panels to bow out.

2.10.7 Panel-to-Panel Connections (Fig. 2.10.7.a–e)

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:

- Panels are a simple. , economical method to resist wind and seismic forces compared with cross-bracing or moment-resistant frames in the structure.
- Several panels can be connected together to increase the lateral load resistance.

Considerations:

- The connections are rigid and unyielding, thus volume-change effects need to be considered in the design.

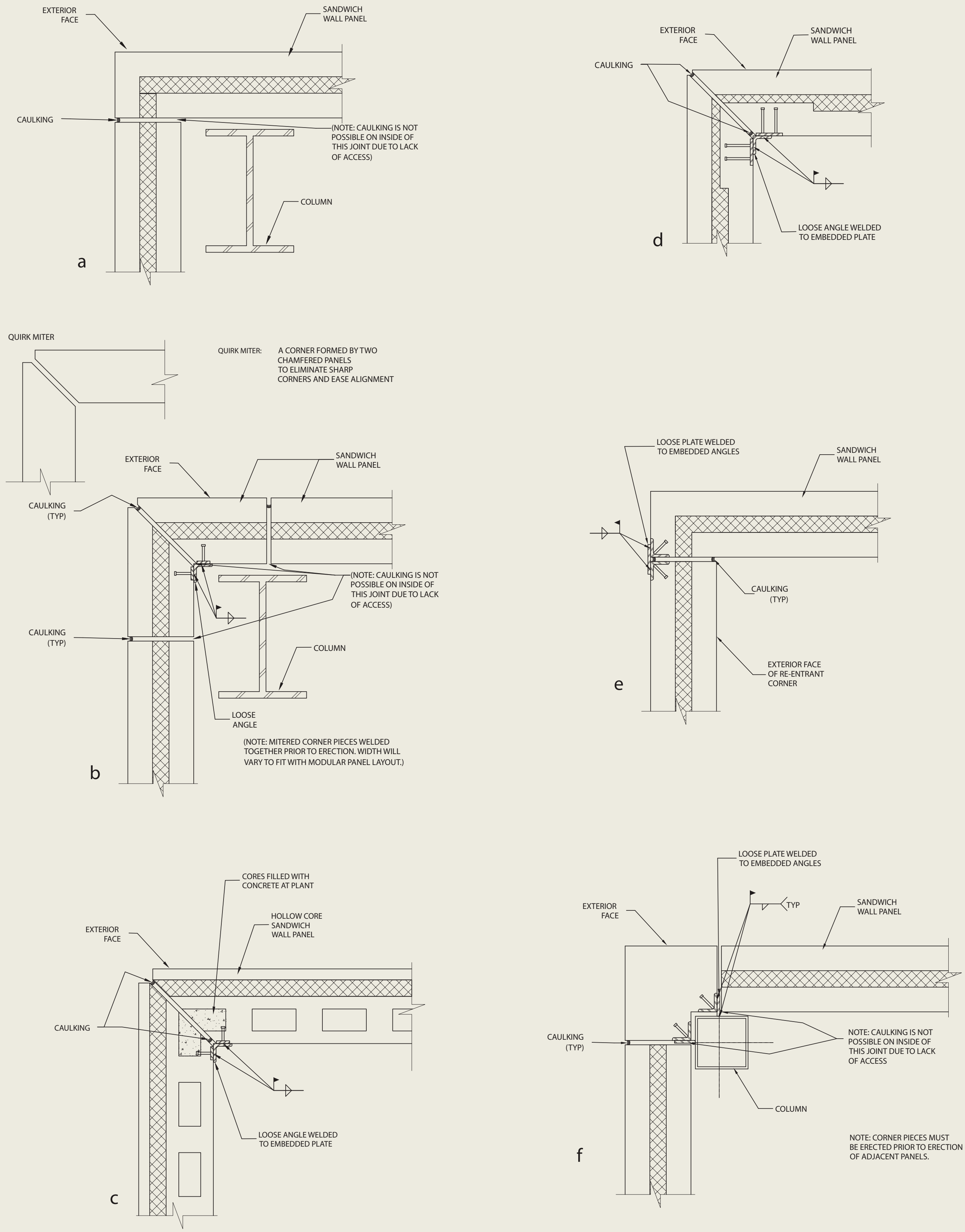


Figure 2.10.6. Corner panel connections.

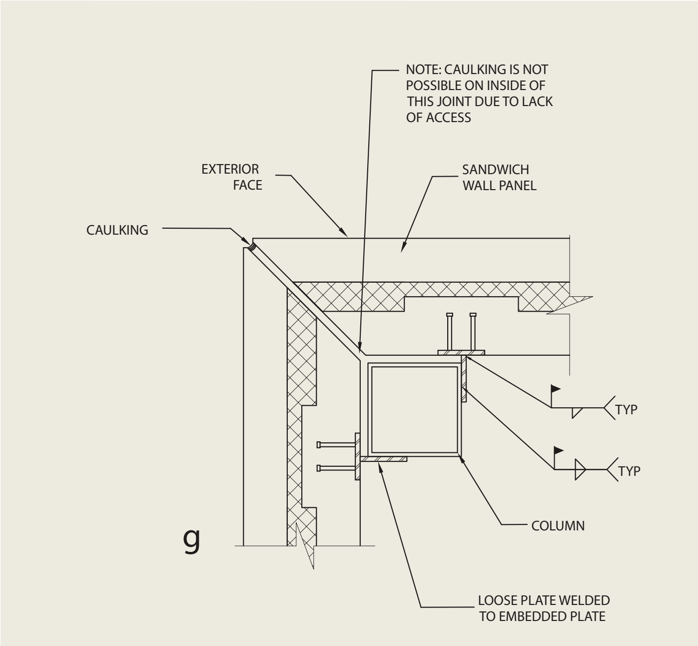


Figure 2.10.6 (cont.). Corner panel connections.

- Solid areas reduce the insulating properties of the sandwich panels.

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

Depending on their height, sandwich 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 connection should allow panels to move horizontally, parallel to the wall face.

Features:

- It allows the erector to efficiently align panels.
- The differential bowing between adjacent panels is reduced.

Considerations:

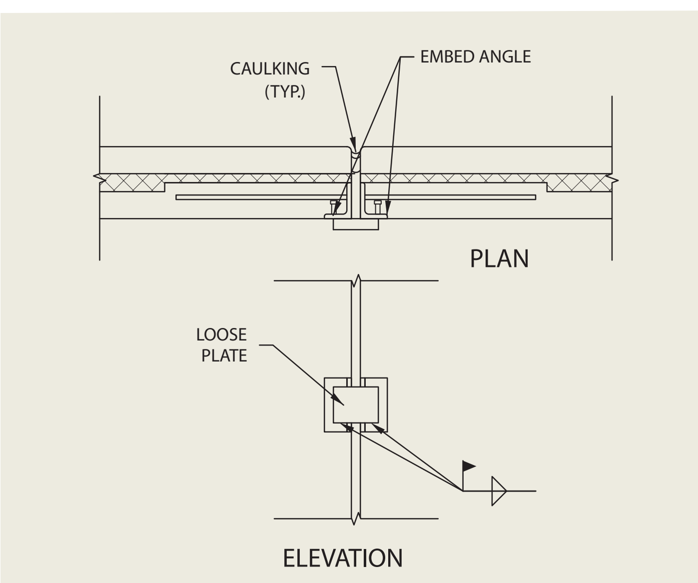


Figure 2.10.7.a. Panel-to-panel shear wall connections.

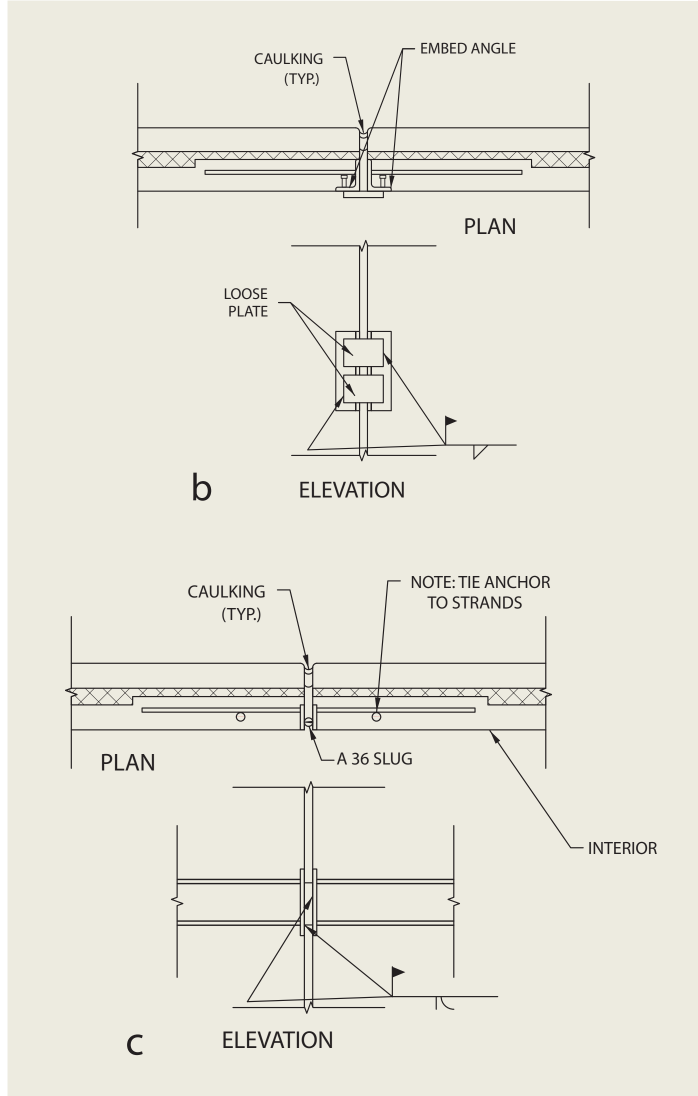


Figure 2.10.7.b–c. Panel-to-panel alignment connections.

- If connections are rigid, volume-change forces must be considered in connection design.
- If two panels have a differential bow, a large force may be required to pull them together, resulting in large connection forces that must be considered in design.

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

Sandwich panel joints are normally caulked on both the outside and the inside surfaces. 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*,[6] and in section 10.5.6 of the *PCI Design Handbook*. An example is presented in section 2.13 of this report.

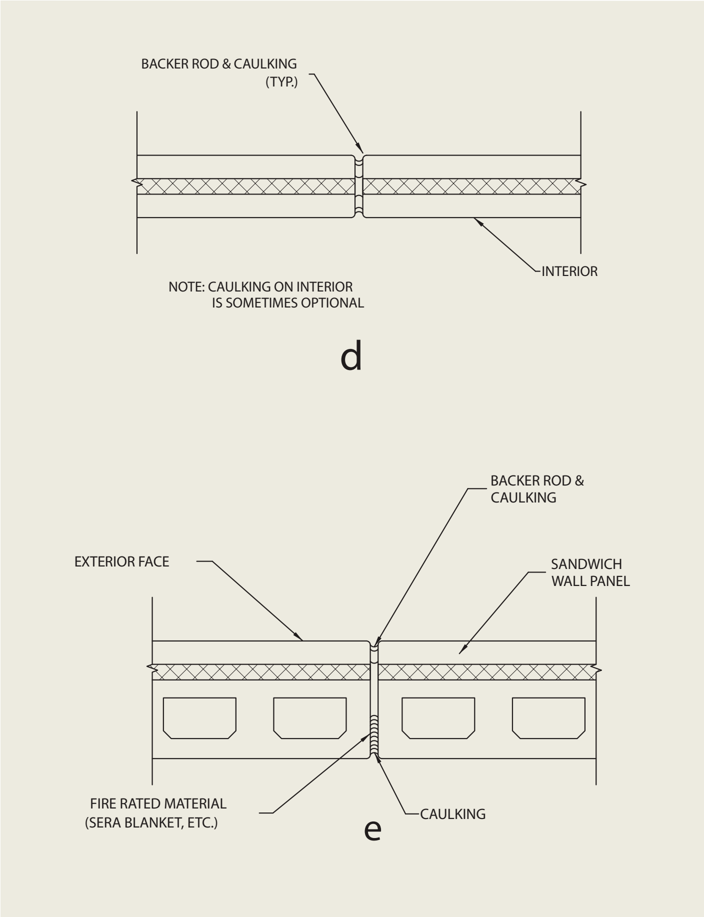


Figure 2.10.7.d–e. Panel joint caulking.

2.10.7.4 Panel-to-Panel, Horizontal Joint Connections (Fig. 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 load-bearing 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 common joint width between adjacent panels is ½ in. (12.7 mm), which allows for some fabrication and erection tolerance. A ½ in. joint is also a desired dimension for caulking. Although detailed as ½ in., actual in-place joint widths may vary from ¼ in. (6.4 mm) to ¾ in. (19 mm). Standard tolerances are given in the *PCI Design Handbook* (section 13.3.2).

Corner joints are normally detailed as either butt joints or mitered joints (refer to the corner configurations detailed in section 2.10.6). Butt joints are more easily fabricated because there is less formwork involved. When butt joints are used, it is important that the designer either hold the insulation back several inches from the edge of the panel or detail the insulation to turn the corner to meet with the insulation in the adjacent panel. Mitered corners are generally associated with projects involving architectural details and finishes. Miters must be formed, finished, and erected with care so that the final appearance is acceptable. Miter dimensions should incorporate standard quirk miter configurations found in the PCI *Architectural Precast Concrete* manual.[7] Other corner joint considerations are discussed in section 2.10.6.

At corners of buildings, composite and partially composite panels will bow in orthogonal directions (fishmouth) if not restrained. If the panel edges are not restrained, the corner caulking may fail. This phenomenon is described in the *PCI Design Handbook*, section 4.8.5. Connection details are discussed in section 2.10.6 of this report. Some caulking failure at the first joint from the corner has also been observed. The use of appropriate panel-to-panel connections that consider the volume-change effects on the last and second-to-last panel on an elevation has been successful in eliminating this problem. These panel-to-panel connections should be detailed to minimize significant in-plane, volume-change restraint forces, as discussed in section 2.10.7.2.

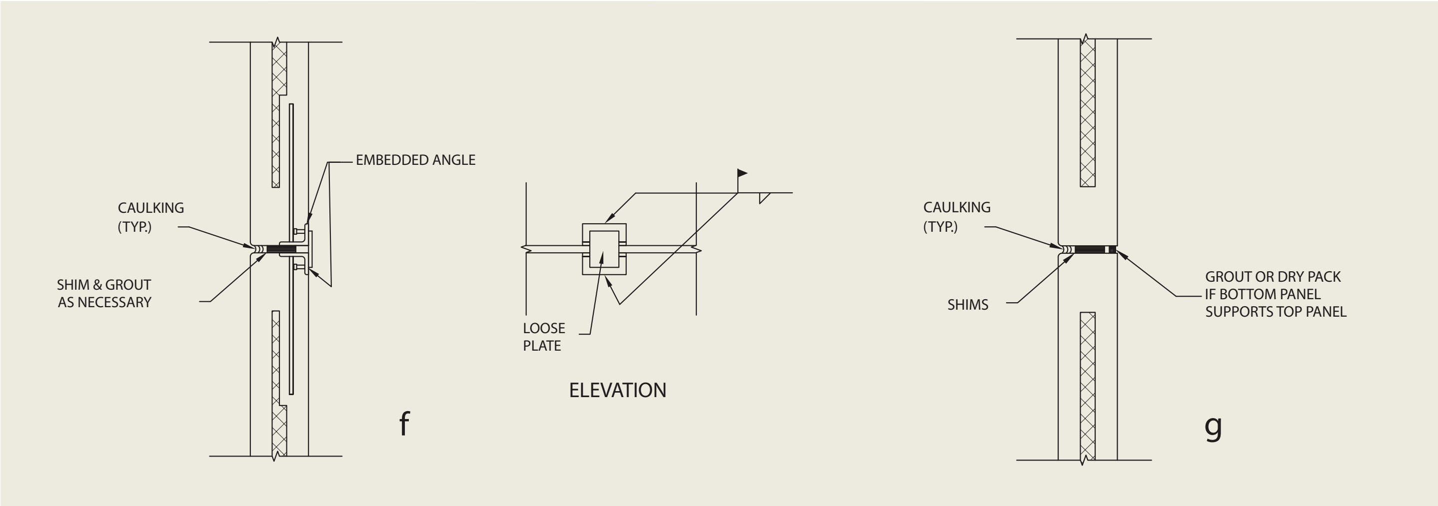


Figure 2.10.7.f–g. Horizontal panel-to-panel connections.

2.11.2 Clearances If the Orthogonal Bowing Is Not Restrained

As recommended in the *PCI Design Handbook*, the clearance detailed between sandwich panels and the structure should be 1 in. (25.4 mm); detailed clearance less than this may cause interference problems. Such problems are generally due to deviations in the plumbness or plan location of the structure, not because of the precast concrete panel dimensions. It is intended that panels be erected plumb and straight, but in some cases they must be erected to follow the existing structure. The 1 in. recommended clearance usually provides for the necessary tolerance. Clearances greater than 1 in. may allow for more tolerance in the structure but may cause greater load eccentricities that must be considered in the sandwich panel connection design.

2.11.3 Interface with Other Building Features

An important consideration relating to features of the building is the understanding that sandwich panels will move or bow during the life of the structure. Anything connected or adjacent to them must be able to accommodate the expected movement. It is important that the architect be aware of this phenomenon so that suitable details can be provided.

One example of this concern is a block wall constructed perpendicular to a composite sandwich panel. The panel may bow outward on a daily basis, whereas the block wall will not move. If the joint between the block wall and the sandwich panel is caulked, the caulking will eventually fail unless the two are mechanically tied together. Such a mechanical connection will generally need to be designed by the structural engineer. In lieu of a caulked joint, a continuous joint cover permitting horizontal movement could be used. Similar situations can occur at mezzanines that are not connected to the panels and joints between slabs-on-grade and panels that extend down past the slab to the footing.

Another example is suspended ceilings that may have their edge support attached to the wall panel. Panel bowing may be enough to cause ceiling tiles to fall out if not properly detailed.

The sandwich panel manufacturer is the most experienced with its panel behavior and can recommend details that appropriately accommodate expected panel movement. It is good practice that the precaster be consulted along with the general contractor and professional engineer early in the project regarding the most appropriate panel connection details for the various situations that will be encountered in the building.

2.11.4 Openings

Openings in panels may be detailed as being completely contained within a panel (punched) or as blockouts in the edges of the panels. Hanging a panel between two adjacent panels is commonly used to create large openings. These are commonly called hanging panels and are discussed in section 2.11.5.

Re-entrant corners at panel openings should be reinforced with diagonal bars to limit the width of potential corner cracks. This reinforcement should be placed in both wythes. ACI 318-05 calls for a diagonal no. 5 bar for each layer of wall reinforcement, but smaller-diameter bars are typically used in thin wythes due to clearance or cover considerations. Panels with punched openings located near one edge of a panel are susceptible to crack formation at the corners of the openings. It is sometimes necessary to eliminate the insulation in this area and add additional reinforcement.

Blockouts in the panel edges can complicate panel handling. Twisting of these panels during stripping or erection may cause corner cracks. Strongbacks may be attached to the panel to reduce the potential twisting effect.

In some cases, panel openings are so extensive that an insufficient section of panel is left to keep the applied stresses below the cracking limits specified in chapter 18 of ACI 318-05. In these cases, the panel is considered to be cracked. The designer is then mainly concerned with ultimate strength performance and crack control. Additional longitudinal mild-steel reinforcement can be added at each side of the opening. The addition of supplemental prestressed strand is generally not practical because long-line production methods are normally used to cast multiple panels each day. Deflections can also be an issue when panels have large openings. Those panels are often attached to adjacent panels when the adjacent panels have some residual capacity.

2.11.5 Hanging Panels

Hanging panels are panels hung from lintels (concrete or steel) or from adjacent panels. The strand layout for hanging panels wider than 12 ft (3.66 m) is usually parallel to the span of the panel. The connection details for these panels must consider the support of the adjacent panels and the bowing of the adjacent panels. The connections for these panels are typically classified as bearing, tieback, or alignment connections.

Designers should limit the number of bearing connections to two per panel to avoid creating an indeterminate load path. Bearing connections may be detailed to allow for larger-than-normal tolerances, thereby decreasing the potential need for a field retrofit if the bearing connection or its support is mislocated. Erection drawings should clearly indicate the bearing connections and the extent of connection welding required before the panel self-weight can be released from the crane.

Hanging panels that are rigidly attached to or supported from adjacent panels will move with them. This is generally the best configuration to maintain alignment of the exterior panel surfaces and to prevent differential bowing. If the hanging panel is also firmly attached to the structure at an elevation different from that of the adjacent panels, care must be taken in design to avoid the potential for structural distress that may occur in the connections. This condition can occur at either tieback connections or bearing connections. If the hanging panel is supported by a structural lintel and tied to the adjacent panels, the bowing of the adjacent panels will tend to pull the lintel outward. Sliding bearing connections have been successfully used to allow for this movement.

2.11.6 Cap Flashing

Precast sandwich walls should always have a continuous cap flashing. Occasionally, a designer will leave a precast sandwich wall exposed on top, without a flashing. Even though the insulation can be made to stop short of the top of panel, there will be a cold joint across the top panel edge, which could allow water penetration. In addition, prestressing strands extending out the top are typically burned off and patched. The patches could come loose over time if not protected by flashing.

2.12 REINFORCEMENT REQUIREMENTS

2.12.1 Minimum Transverse Reinforcement

Section 16.4.1 of ACI 318-05 permits precast, prestressed concrete wall panels, which are not mechanically connected so as to cause restraint in the transverse direction, to have the shrinkage and temperature reinforcement requirements of section 7.12 waived in the direction normal to the flexural reinforcement. Section 18.11.2.3 of ACI 318-05 permits panels with average prestress of 225 psi (1550 kPa) to waive all minimum requirements of section 14.3, provided structural analysis shows adequate strength and stability. The shear provisions of ACI 318-05 section 11.10.9 must also be checked, though sandwich panels seldom require any shear reinforcement.

Satisfactory results have been achieved for many sandwich panel projects with panels that have no welded-wire or other transverse reinforcement other than bars at the lift points or connection points proportioned based on actual demand requirements. For precast, nonprestressed concrete sandwich panels, section 8.3.4.3 of the *PCI Design Handbook* provides guidance in crack control. Section 16.4.2 of ACI 318-05 requires precast, nonprestressed concrete walls to be designed per the provisions in chapter 10, "Flexural and Axial Loads," or chapter 14, "Walls," except that the area of horizontal and vertical reinforcement each shall be not less than 0.001 times the gross cross-sectional area of the wall panel.

2.12.2 Additional Reinforcement for Handling

The analysis and design procedures for wall panels contained in chapter 8 of the *PCI Design Handbook* are normally used for the determination of flexural stresses during panel handling. This analysis generally results in the requirement for 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 local bursting stresses when the prestressing force is transferred to the member. This reinforcement is generally most effective in controlling exposed cracks when it is placed between the strand and the outside face of each wythe.

2.13 FIRE RESISTANCE

The fire endurance of sandwich panels can be estimated by using the procedure outlined in the *PCI Design Handbook*. Some panel systems have been fire tested to provide this data. Once the fire endurance of each individual element is known, the estimated fire endurance of the assembly can be calculated using the formula

$$R^{0.59} = R_1^{0.59} + R_2^{0.59} + \ldots + R_n^{0.59}$$

where

R = fire endurance of composite assembly in minutes

$R_1, R_2, \ldots R_n$ = fire endurance of individual courses in minutes

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

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

From Fig. 9.3.6.3 of the *PCI Design Handbook:*

$R^{0.59}$ = $R^{0.59}$ (2 in. concrete) + $R^{0.59}$ (2 in. insulation) + $R^{0.59}$ (3 in. concrete)

$R^{0.59}$ = 6.8 + 2.57 + 9.5 = 18.87

By linear interpolation from the table in Fig. 9.3.6.3:

R = 120 + [(2.02/4.56) × 60] = 146 minutes = 2 hours 26 minutes

In general, the joints between sandwich panels do not require special treatment for fire resistance. PCI MNL-124, *Design for Fire Resistance of Precast Prestressed Concrete*, contains discussion related to the treatment of joints. Local building codes and PCI MNL-124 should be referenced for specific project requirements. Should special joint treatments be required, it is accepted practice to interpolate the graphic values found in PCI MNL-124.

Example 2. Determine the fire endurance of a ½-in.-wide (12.7 mm) butt joint between 6-in.-thick (150 mm) panels utilizing a 1-in.-thick (25.4 mm) ceramic fiber blanket.

From Fig. 8.5 of PCI MNL-124, the fire endurance of a 1 in. joint is approximately 1.7 hours and the fire endurance of a ³⁄₈ in. (9.53 mm) joint is approximately 3 hours.

By interpolation, the estimated fire endurance of a ½-in.-wide joint is:

$$R = 3\,hr - \left[\left(3\,hr - 1.7\,hr\right)\left(\frac{0.5\,in. - 0.375\,in.}{1\,in. - 0.375\,in.}\right)\right]$$

$$= 2.74 \text{ hours}$$

CHAPTER 3 – INSULATION AND THERMAL PERFORMANCE

3.1 GENERAL INFORMATION

Precast concrete sandwich 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.

While sandwich walls were being developed by the concrete industry, the need of the construction industry to develop procedures to calculate the actual performance characteristics of building envelopes has become more urgent. This move to focus

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

Physical properties	Polystyrene						Polyisocyanurate		Phenolic	Cellular glass
	Expanded			Extruded			Unfaced	Faced		
Density, lb/ft^3	0.4 to 0.9	1.1 to 1.4	1.8	1.3 to 1.6	1.8 to 2.2	3.0	2.0 to 6.0	2.0 to 6.0	2.0 to 3.0	6.7 to 9.2
Water absorption, % volume	<4.0	<3.0	<2.0	<0.3			<3.0	1.0 to 2.0	<3.0	<0.5
Compressive strength, psi	5 to 10	13 to 15	25	15 to 25	40 to 60	100	16 to 50	16	10 to 16	65
Tensile strength, psi	18 to 25			25	50	105	45 to 140	500	60	50
Linear coefficient of expansion, (in./in./°F)(10^{-6})	25 to 40			25 to 40			30 to 60		10 to 20	1.6 to 4.6
Shear strength, psi	20 to 35			—	35	70	20 to 100		12	50
Flexural strength, psi	10 to 25	30 to 40	50	40 to 50	60 to 75	100	50 to 210	40 to 50	25	60
Thermal conductivity, Btu-in./hr/ft^2/°F	0.30	0.26	0.23	0.20			0.18	0.10 to 0.15	0.16 to 0.23	0.35
Maximum use temperature, °F	165			165			250		300	900

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

Type of insulation	ASTM designation	ASTM type
Expanded polystyrene	ASTM C578	Types I, II, VIII, IX, XI
Extruded polystyrene	ASTM C578	Types IV, V, VI, VII, X
Polyurethane	ASTM C591	Types 1, 2, 3
Polyisocyanurate	ASTM C591	Types 1, 2, 3
Phenolic	ASTM C1126	Types I, II, III

on improvement of building energy performance is manifested in the most recent American Society of Heating, Refrigeration, and Air-Conditioning Engineers (ASHRAE) standard published in cooperation with the Department of Energy (DOE) and the Environmental Protection Agency (EPA). This document, *ASHRAE Standard 90.1,*[8] based on the previous *ASHRAE Standard 90A* and *ASHRAE Standard 90-75*, sets strict compliance guidelines for building design and calculation of energy efficiency performance for the entire constructed facility. This new standard focuses on all new construction, covering most building types. *ASHRAE Standard 90.1* has been adopted into the local building codes of all 50 U.S. states, as well as the recognized national building codes. The final form of this standard is a part of the 2006 *International Energy Conservation Code*.[9]

ASHRAE Standard 90.1 has been the adopted energy standard that contributes to the initial design, analysis of a design, construction, and operation of new facilities. Building codes based on these latest revisions to energy performance requirements now include more stringent minimum *U*-values for each assembly contributing to a building's envelope. This effect has mandated the improvement of energy efficiency calculation procedures, energy testing, and material production. Energy calculations have been adopted to benefit both the owner in selecting envelope assemblies and the producer in manufacturing and marketing a quality product. These calculation procedures are covered in section 3.4.

In addition to requiring more energy-efficient performance in every assembly contributing to the envelope, the energy codes also pertain to the manufactured materials used to produce the given assemblies. The relative effectiveness of different 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, sandwich panels use cellular (rigid) insulation because it provides the 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 condition, 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 known as molded expanded polystyrene (beadboard) and extruded expanded polystyrene (extruded board). Thermosetting insulations consist of polyurethane, polyisocyanurate, and phenolic. Table 3.2.a lists the physical properties of these thermosetting insulations.

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

A sandwich panel is a unique environment for insulating material. During manufacture of the panel, the insulation is exposed to high temperatures (140 °F to 150 °F [60 °C to 66 °C]) from concrete hydration and applied heat from accelerated curing. These high temperatures, as well as loading from worker foot traffic during production, exposure to high moisture levels from the curing of fresh concrete, and compressive forces and flexural stresses, all require that the insulation being used should exhibit appropriate and compatible performance. Once the panel is cured and erected into place, the insulation is then exposed to a continuous moisture and vapor gradient drive that continues to affect the physical and thermal behavior of the insulating material.

In some locations, freezing and thawing cycles during the building's lifetime induce forces on the insulation that can work to break or tear apart individual cells. For example, a molded polystyrene insulation has a high moisture-absorption rating. A building with usage that results in a high moisture gradient drive will cause this insulation to absorb potentially large amounts of moisture over time. When exposed to freezing and thawing, the relatively weak bond between the beads or cells of the insulation can break down and the insulation may begin to disintegrate. This process can be mitigated by using extruded polystyrene instead of expanded polystyrene.

In some special-use facilities, sandwich panels are exposed to very high interior operating temperatures. The physical property of the insulation must be selected to withstand these temperatures to avoid the potential for 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 of the insulation reach 160 °F (71 °C). For special building applications with high-temperature environments, 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 based on its compatibility with and resistance to the conditions to which it will be exposed, keeping in mind that higher-performance insulation material has higher material costs.

3.3 ENERGY PERFORMANCE

3.3.1 Thermal Transmission

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

To construct an insulated panel, structural elements (such as wythe ties) must often pass through the insulation layer or the insulation layer is fit into spaces provided between those structural elements. This construction practice interrupts the otherwise potentially continuous insulation layer, increasing the potential for conductance of energy. These interruptions are also known as thermal bridges. Thermal bridges can be created by materials such as steel, concrete, composites, and plastics. Thermal bridges conduct 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 overall effectiveness of the panel insulation.

During the past three decades, insulation systems have been developed to minimize or eliminate the solid zones of concrete, the steel connections, or both.[10,11] The selection of the insulation system for use in the sandwich panel should be based on the conditions of the building environment, the required structural performance, and the effect that the building codes have on the constructed assembly.

3.3.2 Performance Calculation

Thermal bridges in sandwich panels can create the potential for condensation and thermal inefficiency, which reduce the maximum designed performance of the panels. *ASHRAE Standard 90.1* addresses the calculation of the effects of the thermal bridge by mandating the use of two calculation procedures: isothermal (series-parallel) analysis and zonal method analysis. In addition to these two methods, a third method has been developed since the publication of the initial state-of-the-art report to specifically address precast concrete sandwich panels. This method, developed by Lee and Pessiki, is called the characteristic section method.[12,13] This method has been incorporated into the *PCI Design Handbook* and is subsequently being updated in the seventh edition of the handbook. All of these calculation methods are described in section 3.4 of this report.

Opinions vary concerning the most appropriate 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 or around the insulation has been common since the introduction of sandwich panels.

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, zonal method, and characteristic section method analyses.[13] Calculations using the isothermal and zonal methods are provided in chapter 23 of the 2005 *ASHRAE Handbook of Fundamentals*[14] and are included in the latest energy code adoption *ASHRAE 90.1*. Example calculations using the characteristic section method are provided in the *PCI Design Handbook*. Another useful publication is the *Thermal Mass Handbook, Concrete and Masonry Design Provisions Using ASHRAE/IES*

90.1-1989.[15]

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.[16]

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. Thermal testing to show the effect of the actual construction technique and the design on the in-place wall assembly supports the validity of this analysis.[17]

3.4.3 Characteristic Section Method (CSM)

This method is based on the zonal analysis method, but it further refines and clarifies the effects of solid concrete zones in sandwich panels. The CSM modifies the calculation method of the discrete zones associated with fully insulated areas and the panel areas or zones affected by thermal bridging. The original version of CSM addressed only the redefinition of the solid concrete areas and did not address the metal-wythe tie locations. The current version of CSM addresses both types of thermal bridging and allows for a simple, accurate calculation of the overall panel *R*-value. Full examples are included in Lee and Pessiki.[13]

3.4.4 Thermal Mass Calculation

Energy analysis by the three methods discussed previously is available in computer software or manual 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 *ASHRAE Standard 90.1*, 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 specifically evaluated for different panel assemblies.

A comparison of wall assemblies over the past decade by ASHRAE and the US Department of Energy (DOE) has demonstrated the additional benefits of using concrete in wall construction. Chapter 8 in *ASHRAE Standard 90.1* gives an equation for comparing the operating performance of walls with high mass to the performance of walls with low mass. Based on the specific geographic location of a building and the internal 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.

Evaluation of properly designed and constructed sandwich panels can take full advantage of these principles and provide significant energy efficiency improvements over other wall systems. The ASHRAE/DOE wall panel evaluation 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 those commonly used in the production of standard precast, prestressed products. The procedures are all described in PCI MNL-116-99, *Quality Control for Plants and Production of Structural Precast and Prestressed Concrete Products*[18] (see Fig. 4.1.a–p). Special manufacturing considerations are described in the remaining portion of this chapter.

4.2 STRESSING AND STRAND POSITIONING

The precasting plant must have the proper equipment for tensioning prestressing strand according to MNL-116-99 and the necessary equipment and form setup to ensure proper strand location in the final as-cast product. The tolerance on designed strand location is extremely important to maintain during production to prevent warping, bowing, and cracking in sandwich panels. 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 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 (Fig. 4.3.1.a). Vibration techniques vary. Some commonly used methods are standard spud vibrator, 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 (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 (Fig. 4.3.1.c). Because vibration tends to make the insulation float, some manufacturers use reinforcing bar slugs or other spacer elements placed between the top strand and the insulation to hold down the insulation.

Some manufacturers stress both the top and bottom strands in the initial production step. The remaining production procedures are then the same as described previously. In wet-cast panels, the bottom wythe (form wythe) is usually the exterior wythe on the

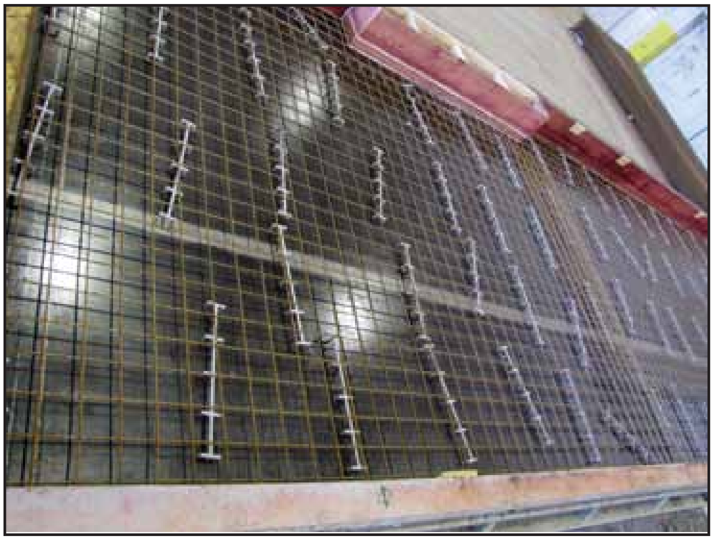


Figure 4.1.a. Basic form with reinforcement in place. Exterior side of panel is form surface. Typically, architectural panels are 3 in. (76 mm) of back-up concrete.

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

Figure 4.1.c. Vibrating face mixture using internal vibrator.

Figure 4.1.d. Placing 2-in.-thick (51 mm) insulation. Insulation is predrilled at 16 in. (406 mm) on center to accept pin-style connectors.

Figure 4.1.e. Placing 2-in.-thick (51 mm) insulation. Insulation can go outer edge to outer edge or can incorporate certain areas of panels that are uninsulated, depending on design criteria required.

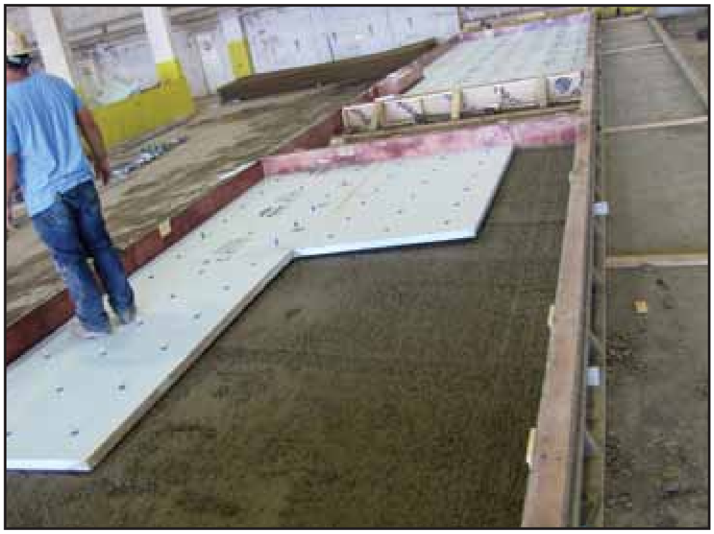


Figure 4.1.f. Insulation partially in place.

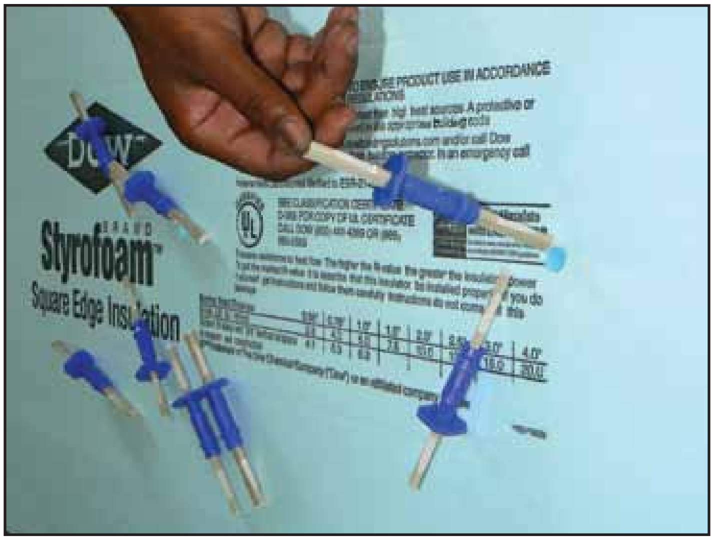


Figure 4.1.g. Placing pin connector through insulation.

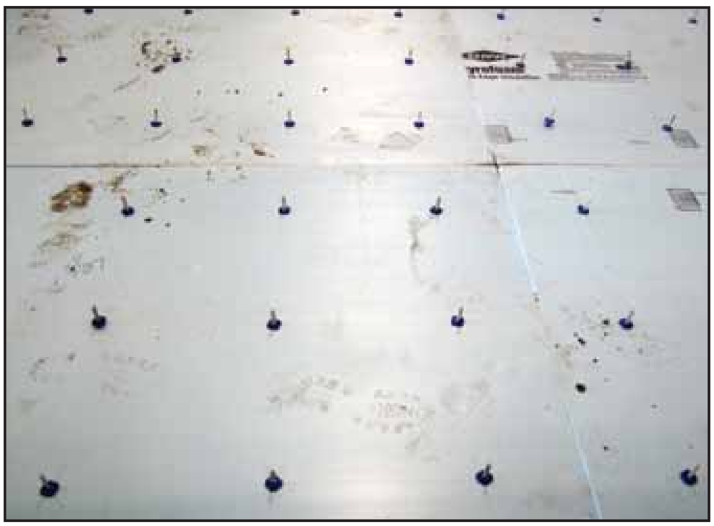


Figure 4.1.h. Pin connectors in place.

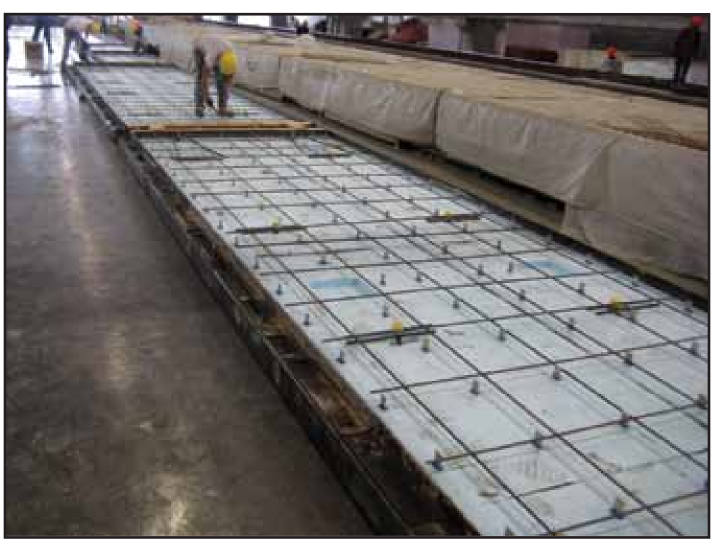


Figure 4.1.i. Pin connectors for composite action. Prestressing strand in place to prestress panel.

Figure 4.1.j. Stainless steel pin connectors for connecting wythes.

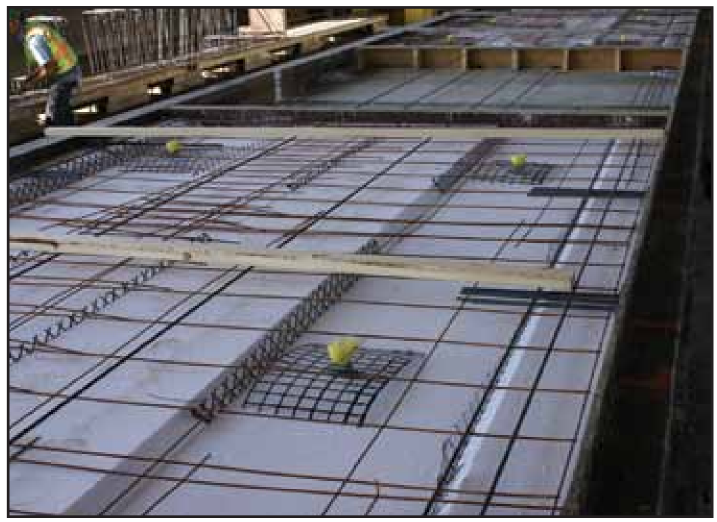


Figure 4.1.k. Epoxy-coated carbon-fiber grid to connect wythes for composite action. Prestressing strand in place.

Figure 4.1.l. Prestressing strand in place to prestress panel.

Figure 4.1.m. Electrical conduit and outlet box in place.

Figure 4.1.o. Placement of concrete backup over insulation. Use of pencil vibrator to consolidate backup mixture.

Figure 4.1.n. Blockout of foam around edge handling insert.

Figure 4.1.p. Vibration of backup mixture using pencil vibrator.

structure.

4.3.2 Dry Cast (Zero Slump)

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

4.3.3 Machine Cast

Manufacturers have developed various machine-casting systems to improve the quality and reduce the cost of manufacturing sandwich panels. These systems utilize the wet-cast system 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 uses machinery to produce panels with normal-slump concrete or by the wet-cast system (Fig. 4.3.3.a). 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 saw cut to length after curing.

Spandeck (Wet Cast) System

The Spandeck Wall Panel System uses machinery that produces hollow-core slabs (6 in., 8 in., 10 in., and 12 in. [152 mm, 208 mm, 254 mm, and 305 mm]) as the bottom wythe with normal-slump concrete by the wet-cast system (Fig. 4.3.3.b). 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

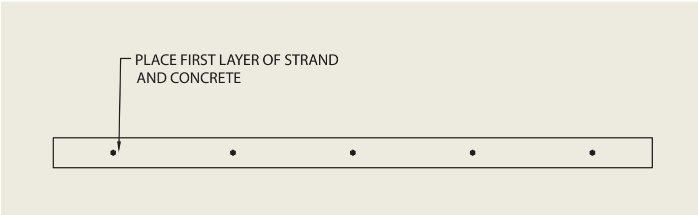


Figure 4.3.1a. First step: place first layer of strand and concrete.

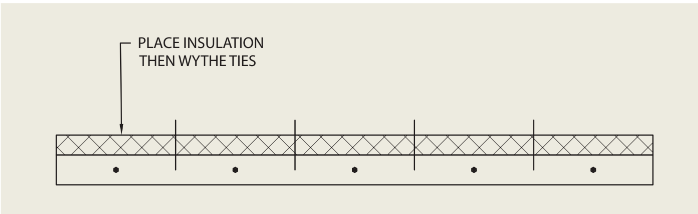


Figure 4.3.1b. Second step: place insulation and wythe ties.

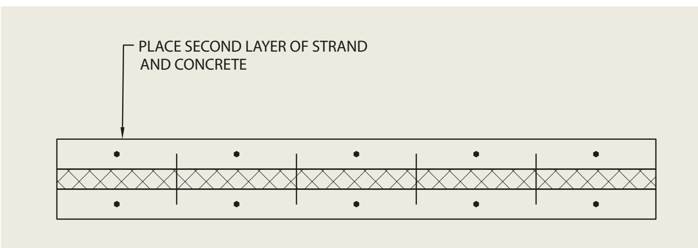


Figure 4.3.1c. Third step: place second layer of strand and concrete.

patterns in the concrete surface. After curing, the panels are saw cut to length.

Spancrete (Dry Cast) System

The Spancrete Wall Panel System also provides machinery that produces hollow-core slabs (6 in., 8 in., and 10 in. [152 mm, 203 mm, and 305 mm]) with zero-slump concrete or by the dry-cast system (Fig. 4.3.3.c). Insulation is then placed with ties, and the top-wythe concrete is placed by machine and finished. After curing, the panels are saw cut 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 (Fig. 4.3.4).

4.4 CURING

Proper curing of sandwich panels is critical to product quality. It is sometimes necessary to have a curing system capable of providing external heat to accelerate curing by supplementing the concrete heat of hydration so as to permit daily casting. The curing system usually incorporates controls to provide and maintain the curing temperature of the freshly placed concrete according to the procedures contained in PCI MNL-116-99.

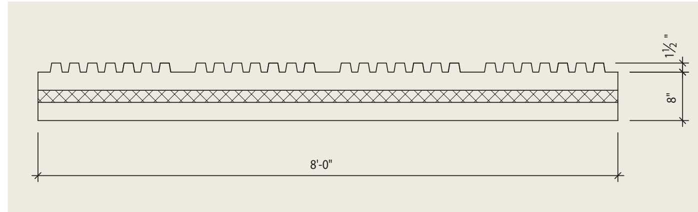


Figure 4.3.3.a. Corewall panel system (wet cast).

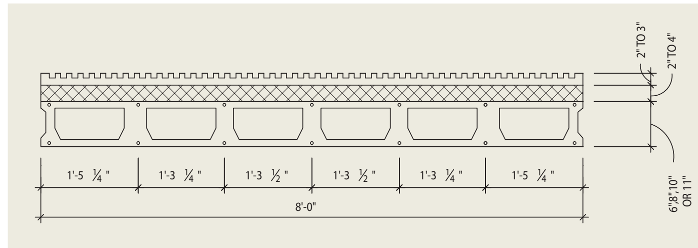


Figure 4.3.3.b. Spandeck wall panel system (wet cast).

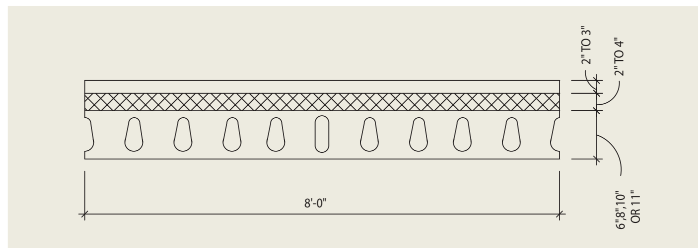


Figure 4.3.3.c. Spancrete wall panel system (dry cast).

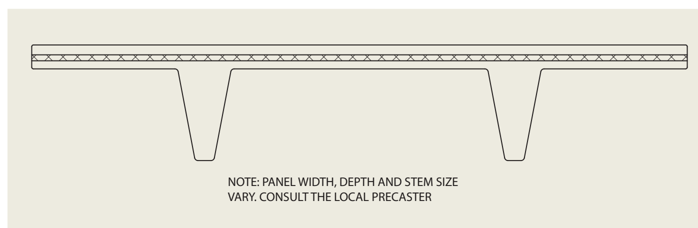


Figure 4.3.4. Insulated double-tee wall panel system.

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%, causing a blowout in the uncured concrete, and expanded polystyrene may shrink as the result of excessive temperature, causing gaps between the insulation and concrete.

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

4.5 FINISHES

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

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 formliners 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, as well as broomed or hard steel troweled. Designers should 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 their customers find acceptable to a wide range of products. Variations in finish from standard practice may significantly increase the cost of the panels.

4.6 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 release 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-116-99[18] for insulated wall panels in 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 more stringent manufacturing tolerances in PCI MNL 117-11, *Quality Control for Plants and Production of Architectural Precast Concrete Products,*[19] are not usually applied to sandwich panels. If project requirements mandate an architectural specification, then panel costs greatly increase and the stricter tolerances may not be achievable.

5.1.2 Erection Tolerances

The erection tolerances as stated for structural wall panels in the *PCI Design Handbook* and PCI MNL-127-99, *Standards and Guidelines for the Erection of Precast Concrete Products*,[20] or PCI MNL-135-00, *Tolerance Manual for Precast and Prestressed Concrete Construction*,[21] are normally used.

5.1.3 Relationships among 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 13.1.5 of the *PCI Design Handbook* 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 may crack in ways similar to other precast concrete wall panels. Some of the cracks are those that may occur at re-entrant 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 is normally cause for rejection, but the designer, manufacturer, shipper, and erector can and should take measures to reduce the occurrence of these cracks to a practical minimum.

5.3 REPAIRS

Repairs to cracks in sandwich panels are generally not structural repairs. In other words, the repairs normally are not required to return the cracked panels to their original uncracked stiffness. The use of epoxy injection of cracks is not applicable in areas where insulation is located because once the epoxy penetrates the injected wythe, there exists an unlimited path of travel for the epoxy at the concrete-to-insulation interface.

Painting of the cracks at the surface 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 nonstructural 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 lengths and widths vary due to project requirements, form size, handling-equipment capabilities, transportation limitations, jobsite restrictions, and design limitations. In some parts of the United States, shipping laws allow unescorted, permitted transportation of 12-ft-wide (3.7 m) panels, but in other areas the maximum width is only 10 ft (3.0 m). Panels as wide as 15 ft (4.6 m) and as long as 75 ft (23 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 may be 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 for handling are done in accordance with chapter 8 of the *PCI Design Handbook*.

6.2 PANEL STRIPPING

Stripping design depends on panel architectural features. Panel geometric features allowing panels to be rotated on their sides (edge lift) generally will minimize cracking and allow panels to be stored on their edge in the yard. Edge storage will more easily allow exposed treatments to be administered.

Flat stripping may be necessary due to delicate contours of architectural features, thereby significantly reducing the potential for spalls and fracturing. Tilt tables are utilized to rotate panels from the flat as-cast position to a side lift and storage position, or yard cranes may have two part lines and travel lifts may have two lift-hoist positions for panel rotation.

The fewer times a panel is handled, the lower the possibility that damage will occur. Side lifts eliminate back face stripping anchors and are generally more economical.

6.3 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. Prestressed panels will permit flat-position shipping more readily than nonprestressed panels. When panels are shipped on their edge, consideration of localized bearing stresses must be given to prevent chipping and spalling. Panels receiving special finish are often shipped on edge to prevent damage to the finish.

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

- The length of panel versus the length of trailer must be considered. If the trailer is structurally flexible (such as a stretch trailer), overstressing of panel may result from trailer twisting and deflection during transit.
- Dunnage is usually positioned at the lifting points.
- Jobsite access needs to be such that torsional twist of the trailer and panel is minimized.
- Care should be taken to ensure that the dunnage does not stain the panels.

6.4 PANEL STORAGE

Vertical panel storage is generally preferable for yard space, less damage, and positioning panels for loading. Care must be taken in locating storage points. Generally, storage points coincide with lifting points designated during panel design.

Storage lateral supports must include considerations to minimize or, hopefully, eliminate panel bowing in storage.

Bearing supports must be detailed to ensure that there is no crushing of the thin wythe of the sandwich panel.

CHAPTER 7 – ERECTION OF SANDWICH PANELS

7.1 PANEL HANDLING AND JOBSITE STORAGE

Panel handling at the jobsite is generally the same as yard handling. Jobsite equipment availability and rigging must be considered during the panel design process.

Jobsite conditions, roadways, excavations, and the like must be considered to eliminate torsion, which is twisting of the panels during access to the erection area. Handling diagrams are critical to safe panel handling and should be incorporated in the manufacturer's shop drawings and shop tickets by the precast concrete designer and made available to the erector.

7.2. PANEL ERECTION

The panel manufacturer should provide shop drawing details and must have preplanned erection techniques that are unique to the project. Consideration of rigging, cranes, or handling equipment is a must. The *PCI Design Handbook* and MNL-127-99 should be referenced by the precast concrete designer.

7.3 PANEL BRACING

Temporary bracing is common in load-bearing sandwich panel erection and construction. Bracing must be a part of the sandwich panel analysis. Suppliers of bracing will assist in the availability of bracing design criteria.

Building foundations, partial building slab placement, temporary strip footings, helical piers, or deadmen are used as foundations for the elements used to brace the precast concrete panels.

CHAPTER 8 – INSPECTION OF SANDWICH PANELS

8.1 PLANT INSPECTION

Inspection of sandwich panels at the manufacturing facility follows normal prepour and postpour quality control procedures as outlined in PCI MNL-116. 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. The loading crew usually conducts a final general inspection during the precasting plant loading process.

8.2 JOBSITE INSPECTION

Any required 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.

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APPENDIX – DESIGN EXAMPLES

EXAMPLE A1. NONCOMPOSITE CLADDING PANEL

Design Criteria (see Fig. A.1)

Wind load:

Direct pressure = 10 lb/ft^2

Suction pressure = 15 lb/ft^2

a = 8 ft = 96 in. (panel width)

b_p = 1.5 ft (height of parapet)

f'_c = 5000 psi

f'_{ci} = 3500 psi

f_{pu} = 270 ksi l = 23 ft = 276 in. (clear span)

t_s = 4 in. (inner wythe thickness)

t_{ns} = 2 in. (outer wythe thickness)

t = $t_s + t_{ns}$ = 6 in. = 0.5 ft

w_{dead} = (150 lb/ft^3) = 0.15 kip/ft^3

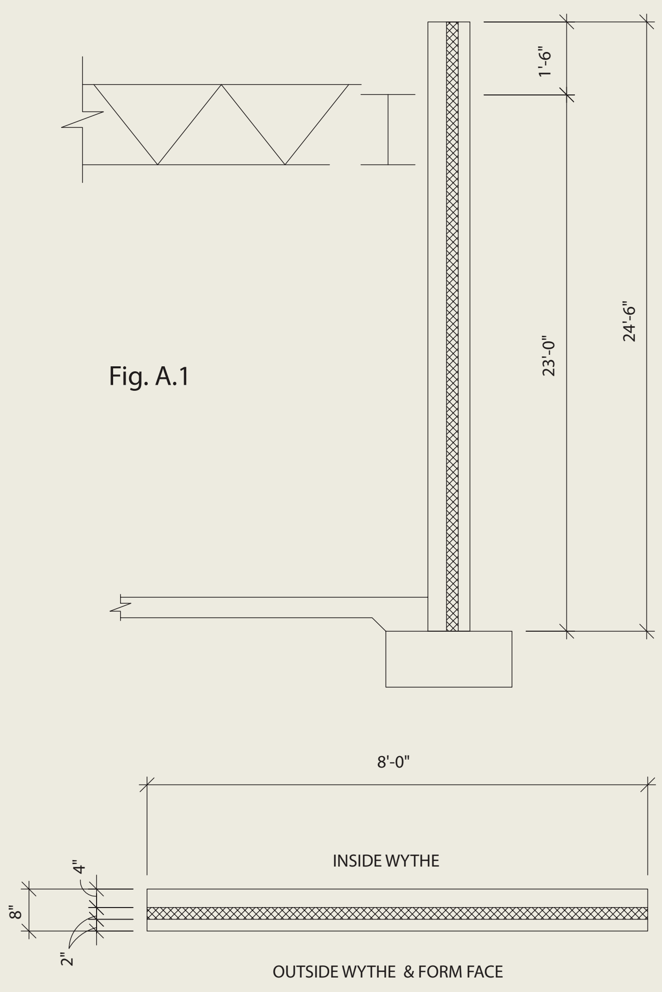


Fig. A.1

Noncomposite behavior.

Note: The inner wythe t_s is 4 in. thick and is used as the structural wythe. It is assumed to resist 100% of the service and handling loads.

Inner Wythe Section Properties

$A_g = ta = 4(96) = 384$ in.^{2}

$I = a(t_s^3)/12 = 96\ (4^3)/12 = 512$ in.^{4}

$S = a(t_s^2)/6 = 96\ (4^2)/6 = 256$ in.^{3}

Structural Analysis

Analyze the section for handling and service load conditions before determining the prestress required.

$U = 1.2D + 1.6W + 1.0L + 0.5\ (L_r \text{ or } S \text{ or } R)$

Suction pressure governs because it is larger than direct pressure.

Note: In calculating wind moment, reduction caused by 1 ft 6 in. cantilevered parapet is neglected. (A separate design check of the parapet should be done.)

w_{wind} = (15 lb/ft^2)(8 ft) = 120 lb/ft = 0.12 kip/ft = 0.01 kip/in.

$$\frac{w_{wind}ab^2}{8} = \frac{(120)(23)^2}{8}$$

$$M_{service} = \frac{w_{wind}l^2}{8} = \frac{(120)(23)^2}{8}$$

$= 7935$ lb-ft $= 7.94$ kip-ft

$M_{u,wind} = (7.94)(1.6)$

$= 12.7$ kip-ft

In calculating the deflection due to wind, assume $\phi_k = 0.85$ and $\beta_d = 0$.

(Stiffness-reduction factor is assumed to be at least 0.85 considering the stringent dimensional accuracy provided by a precasting plant.)

$$\Delta = \frac{5}{384}\left(\frac{w_{wind}l^4}{EI}\right)$$

$$\frac{5}{384}\left(\frac{w_{wind}b^4}{EI}\right)$$

$$E_c = 57\sqrt{f'_c} = 57\sqrt{5000}$$

$= 4030$ ksi

$$EI = \frac{\phi_k E_c I}{1+\beta_d} = \frac{0.85(4030)(512)}{1+0}$$

$= 1.75 \times 10^6$ kip-in.^{2}

$$\Delta = \frac{5}{384}\left(\frac{0.00\left(276\right)^4}{1,750,000}\right)$$

$= 0.43$ in.

To continue the analysis, a load-deflection (P-Δ) analysis is carried out to demonstrate the P-Δ effects due to the relatively high flexibility of the noncomposite panel.

$P_D = (t)(a)(w_{dead})[(l/2) + b_p]$

$P_D = (0.5)(8)(0.15)[(23/2) + 1.5] = 7.8$ kip

$P_u = 1.2\,P_D = 1.2\,(7.8)$

$= 9.36$ kip at mid-height

$\beta_d = 1.0$

$$EI = \frac{\phi_k E_c I}{1+\beta_d} = \frac{0.85\left(4030\right)\left(512\right)}{1+1}$$

$= 8.77 \times 10^5$ kip-in.^{2}

Assume that the initial bow of the panel is ±0.77 in. (l/360) due to differential shrinkage during storage and is additive to the deflection due to the wind.

Because this panel is noncomposite, thermal bowing will be minimal. The deflection of the panel at midspan is the sum of the initial bow and the wind deflection.

$\Delta_{midspan} = 0.77 + 0.43$

$= 1.20$ in.

Secondary deflection from self-weight:

$\Delta = {}^2P_u e l^2/8EI$
$= (9.36)\,e\,(23 \times 12)^2/[(8)(8.77 \text{ x } 10^5)]$
$= 0.102e$

First iteration:
$\Delta = 0.102\,(1.20) = 0.12$ in.

Second iteration:
$e = 1.20 + 0.12 = 1.32$ in.

$\Delta = 0.102\,(1.32) = 0.14$ in.

Third iteration:
$e = 1.20 + 0.14 = 1.34$ in.

$\Delta = 0.102\,(1.34) = 0.14$ in.
resulting in convergence.

The ultimate moment due to wind and P-Δ effects is:

$M_u = M_{u,wind} + M_{u,P\text{-}\Delta}$
$= (12.7 \text{ kip})(12 \text{ in.}) + (9.36 \text{ kip})(1.34 \text{ in.})$
$= 164.9$ kip-in. $= 13.7$ kip-ft

Note that the P-Δ calculations were based on the uncracked moment of inertia, so M_u should be compared with M_{cr}. Handling stresses often dictate the required amount of prestress, so this will be checked later.

Stripping and Yard Handling (see Fig. A.2)

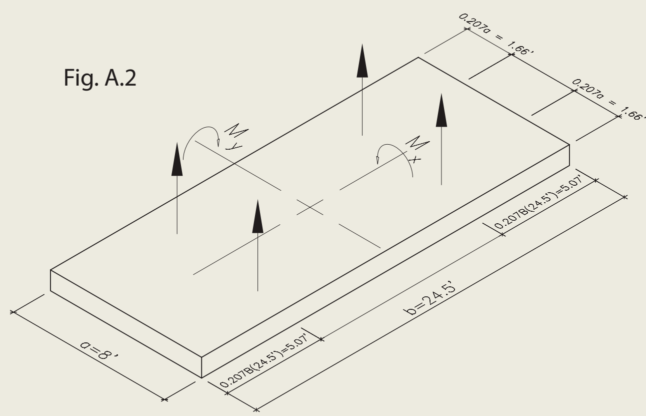
$f'_{ci} = 3500$ psi

Stripping equivalent static multiplier = 1.3

Yard handling equivalent static multiplier = 1.2

Therefore, stripping governs.

Fig. A.2



$w = 1.3\,(0.075)$
$= 0.098$ kip/ft^2

From *PCI Design Handbook*, Fig. 8.3.2:

$+M_x = -M_x = (0.0107)wa^2b$
$= (0.0107)(0.098)(8)^2(24.5)$
$= 1.6$ kip-ft

M_x is resisted by an effective width of

$15t_s = (15)(4) = 60$ in. (governs)

or $b/2 = (24.5)(12)/2 = 147$ in. (does not govern)

$S_x = (1/6)(4^2)(60)$
$= 160$ in.^{3}

$M_x/S_x = (1.6)(12000)/160$
$= 120$ psi (0.83 MPa)

which is less than $5\sqrt{3500} = 296$ psi (OK)(see *PCI Design Handbook*, 7th ed., Eq. 8.3.4.2)

$+M_y = -M_y = (0.0107)\,wab^2$
$= (0.0107)(0.098)(8)(24.5^2)$
$= 5.04$ kip-ft

M_y is resisted by an effective width of:

$a/2 = (8)(12)/2 = 48$ in.

Stripping and yard handling moment required for total panel width:

$M_{u,strip} = (2)(5.05)$
$= 10.1$ kip-ft

Check stresses after prestress level is determined.

Shipping

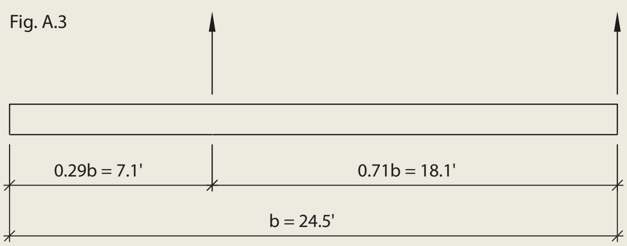
Because the panel will be supported at the same locations as at stripping, shipping stresses will not govern. The ratio of the shipping equivalent static multiplier of 1.5 to the stripping multiplier of 1.3 is 1.5/1.3 = 1.15. The ratio of the square root shipping concrete strength of 5000 psi to stripping concrete strength of 3500 psi is $\sqrt{5000}/\sqrt{3500}$ = 1.19. This means that the concrete strength increase is greater than the higher shipping stresses.

Condition	Multiplier on Static Load
Stripping	1.3
Shipping	1.5
Erection	1.2

Erection

Try a two-point pick (see Fig. A.3) with equal $+M$ and $-M$: $M = 0.044wb^2$.

Fig. A.3



Equivalent static multiplier = 1.2 for erection.

w = 1.2 (0.075)(8)
= 0.72 kip/ft

M = $(0.044)wb^2$ (see *PCI Design Handbook*, 7th ed., Fig. 8.6.1)

M = $(0.044)(0.72)(24.5^2)$
= 19.0 kip-ft

f_{bv} =(19.0)(12,000)/256
= 890 psi

$f_{pc,req.}$ = $890 - 5\sqrt{5000}$
= 537 psi

This prestress value is too high; therefore, revise the handling scheme. Try a three-point pick (see Fig. A.4):

$+M_y$ = $0.034\ wb^2$

$-M_y$ = $0.011\ wb^2$ (does not govern)

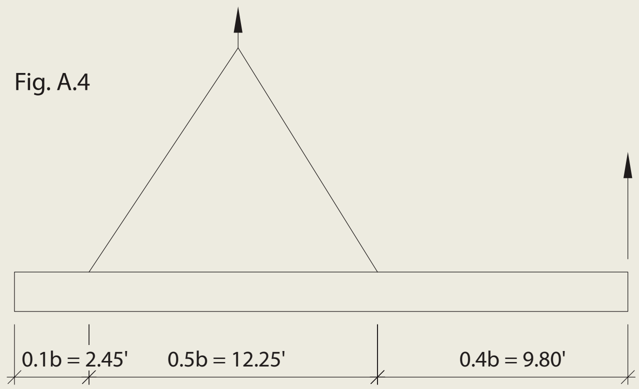
$+M_y$ = 14.7 kip-ft
governs over

$M_{u,wind}$ = 12.7 kip-ft

M_u = $M_{u,wind} + M_{u,P\text{-}\Delta}$ = 13.7 kip-ft and $M_{y\ handling}$ = 10.1 kip-ft

$f_{pc,req.}$ = $M/S - 5\sqrt{f'_c}$

Fig. A.4



= $(14.7)(12{,}000)/256 - 5\sqrt{5000}$
= 689 – 354 = 335 psi

Determination of Prestress Levels

Assume 15% prestress losses. For a 4 in. wythe, use ½-in.-diameter, low-relaxation, 270 ksi strands.

A_{ps} = 0.153 in.^{2} per strand. Determine number of strands required:

Number of strands required:

$$= \frac{f_{pc}A_{wythe}}{\left(0.7A_{ps}f_{pu}\right)\left(1-loss\right)}$$

$$= \frac{\left(0.335\right)\left(384\right)}{\left[0.7\left(0.153\right)\left(270\right)\right]\left(1-0.15\right)}$$

= 5.3 (Use six $^1/_2$ in. strands)

For a 2 in. wythe, use $^1/_2$-in.-diameter, low-relaxation, 270 ksi strands.

A_{ps} = 0.085 in.^{2}

To satisfy the minimum prestress requirements of ACI 318-05, section 18.11.2.3,

$f_{pc,min.}$ = 225 psi

Number of strands required

$$= \frac{f_{pc,min}A_{wythe}}{\left(0.7A_{ps}f_{pu}\right)\left(1-loss\right)}$$

$$= \frac{\left(0.225\right)\left(2\right)\left(96\right)}{\left[0.7\left(0.085\right)\left(270\right)\right]\left(1-0.15\right)}$$

= 3.2

(Use four $^3/_8$ in. strands)

Evaluate $\phi M_{n\ structural\ wythe}$:

The prestressing steel stress f_{ps} = 264 ksi is determined from methods given in the *PCI Design Handbook*.

$$a = \frac{A_{ps} f_{ps}}{0.85 f'_c b}$$

$$= \frac{(6 \times 0.153)264}{0.85(5)(96)} = 0.59 \text{ in.}$$

$$\phi M_n = \phi A_{ps} f_{ps}(d - a/2)$$
$$= 0.9\ (6 \times 0.153)(264)(2 - 0.59/2)/12 = 31.0 \text{ kip-ft}$$

This moment is greater than M_u = 13.5 kip-ft (wind + P-Δ)

Therefore:

$$M_{cr} = (f_{pc} + f_r)S$$
$$= (0.384 + 0.53)(256)/12$$
$$= 19.5 \text{ kip-ft}$$

$\phi M_n/M_{cr}$ = 31.0/19.5 = 1.6 > 1.2

Therefore, the section is satisfactory and basing the P-Δ calculations on uncracked section properties is valid.

EXAMPLE A2. NONCOMPOSITE LOAD-BEARING PANEL

Design Criteria (see Fig. A.5)

Wind load:

Direct pressure = 17 lb/ft^2
Suction pressure = 24 lb/ft^2

Panel dead load D_p = 30 kip
Roof dead load D_r = 20 kip
Roof live load L_r = 6.2 kip

f'_c = 5000 psi

f'_{ci} = 3500 psi

f_{pu} = 270 ksi

t_s = 6 in.

a = 10 ft = 120 in.

l = 28.5 ft = 342 in.

Differential temperature between inside and outside faces = 35 °F.

As panel is noncomposite, the thermal bowing effect is negligible and is not considered in this design.

The inner 6-in.-thick wythe is considered to be the structural wythe and will be designed to resist 100% of all loadings.

Allowable flexural tension stress at ultimate = $7.5\sqrt{f'_c}$

Inner Wythe Section Properties

$A = t_s a = 6(120) = 720$ in.^{2}
$I = a(t_s^3)/12 = (1/12 \times 120)(6^3) = 2160$ in.^{4}

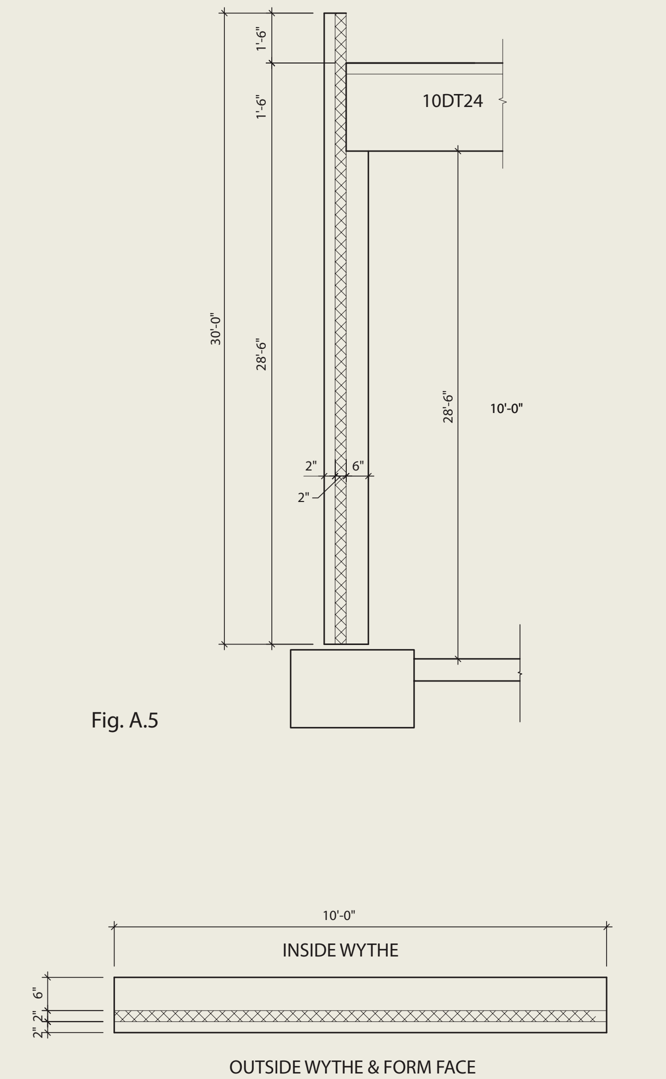


Fig. A.5

$S = a(t_s^2)/6 = 120(6^2)/6 = 720$ in.^{3}

Selection of Prestressing

Assume f_{pc} minimum = 225 psi per ACI 318-05 Section 18.11.2.3 so that minimum requirements of ACI 318-05 Section 14.3 may be waived.

6 in. structural wythe, try seven 1/2-in.-diameter strands located at the center of gravity of the wythe.

A_{ps} = 0.153 in.^{2} / strand
Assume 15% prestress losses.

$$f_{pc} = (1 - 0.15)\frac{f_{py}\phi n A_{ps}}{A_g}$$

$$f_{pc} = (1 - 0.15)\frac{270(0.7)(7)(0.153)}{720}$$

$$f_{pc} = 0.239 \text{ ksi}$$

f_{pc} > 0.225 ksi

Satisfies ACI 318-05 section 18.11.2.3 and minimum reinforcement requirements of ACI 318-05 section 14.3 may be waived.

For 2 in. wythe, try four 3/8-in.-diameter strands located at the center of gravity of the wythe.

A_{ps} = 0.085 in.^{2} / strand

Assume 15% prestress losses.

$$f_{pc} = (1-0.15)\frac{f_{py}\phi n A_{ps}}{A_g}$$

$$= (1-0.15)\frac{270(0.7)(4)(0.085)}{240}$$

f_{pc} = 0.228 ksi

f_{pc} > 0.225 ksi

Satisfies ACI 318-05 section 18.11.2.3 and minimum reinforcement requirements of ACI 318-05 section 14.3 may be waived.

It could be argued that the face wythe of a noncomposite panel is a nonstructural veneer, so ACI 318-05 requirements would not apply. Therefore, the prestress level in the face wythe would be governed only by crack control requirements (under wind and handling effects).

Stripping, Handling, Shipping, and Erection

Noncomposite properties of the panel are used to resist forces generated by stripping, handling, shipping, and erection procedures. Analysis in accordance with the *PCI Design Handbook* results in the following:

Stripping – Use two-point pick at about the 1/5 points from ends and sides of panel (Example A1 and Fig. A.3).

Erection – Use three-point pick with lower points at 4 ft and 14 ft from the lower end and inserts in the ledge area near the top of the panel (Example A1 and Fig. A.4).

Analysis:

All of the load cases of ACI 318-05 section 9.2 must be considered. The controlling load case for this example, Eq. (9-4), was found to govern.

$U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R)$

$W_{suction} > W_{pressure}$

$w_{suction}$ = (10)(0.024)

= 0.240 kip/ft

Determine axial load:

P_{u1} = 1.2[(30/2)+20]+0.5[6.2] = 45.1 kip
Includes half of the panel self-weight.

P_{u2} = 1.2[20]+0.5[6.2] = 27.1 kip
Externally applied loads only.

Calculate stiffness for deflection computation:

ϕ_k = 0.85

Stiffness-reduction factor is assumed to be at least 0.85 considering the stringent dimensional accuracy provided by a precasting plant.

Ultimate dead load at midheight = (30 kip /2 + 20 kip)1.2 = 42 kip

Total load at midheight = 42 kip + (6.2 kip)0.5 = 45.1 kip

β_d = 42/45.1 = 0.93

$$EI = \frac{\phi_k E_c I}{1+\beta_d} = \frac{0.85(4030)(2160)}{1+0.93}$$

$= 3.83 \times 10^6$ ksi

Assume a minimum eccentricity of 1 in. for construction tolerances.

Deflection at midheight of panel due to externally applied loads at e = 1 in.:

$$\Delta = \frac{Pel^2}{16EI}$$

$$\Delta = \frac{(27.1)(1)(342)^2}{16(3.83\times10^6)} = 0.05 \text{ in.}$$

Note that the eccentricity is applied in the direction that achieves maximum total moment and deflection. In this case, the wind suction load governs and the eccentricity should be applied toward the inside face of the panel.

Based on the *PCI Design Handbook* maximum bowing tolerance (section 13.2.8), assume an initial panel bow of l/360 = 0.95 in.

Wind deflection (suction)

Use β_d = 0, ϕ_k = 0.85

$$EI = \frac{\phi_k E_c I}{1+\beta_d} = \frac{0.85(4030)(2160)}{1+0}$$

$= 7.40 \times 10^6$ ksi

$$\Delta = \frac{5}{384}\left(\frac{wl^4}{EI}\right)$$

$$= \frac{5}{384}\left(\frac{0.02(342)^4}{7,400,000}\right)$$

= 0.48 in.

Δ_{total} = 0.05 + 0.95 + 0.48
= 1.48 in.

A P-Δ analysis is performed to account for secondary moments.

P-Δ analysis at mid-height of wall

$$\Delta = \frac{Pel^2}{8EI}$$

$$= \frac{(45.1)e(342)^2}{8(3.83\times 10^6)}$$

$= 0.17e$

First iteration:
Δ = 0.17 (1.48) = 0.25

Second iteration:
e = 1.48 + 0.25 = 1.73
Δ = 0.17 (1.73) = 0.29

Third iteration:
e = 1.48 + 0.29 = 1.77
Δ = 0.17(1.77) = 0.30

Fourth iteration:
e = 1.48 + 0.30 = 1.78 in.
Δ = 0.17(1.78) = 0.30 (converges)

M_u = total factored moment

$M_u = 0.5P_{u,applied}\,(e_{applied}) + M_{u,wind} + P_{u,total}\,(e_{p\text{-}\Delta})$

$M_u = 0.5(27.1)(1) + 1.6(0.24)(28.5)^2(1.5) + (45.1)(1.78)$
= 562 kip-in.

Verify that the panel remains uncracked and that gross section properties apply.

Determine cracking moment

$$M_{cr} = \left(\frac{P_{u,total}}{A} + f_{pc} + f_r\right)S$$

$$= \left(\frac{45.1}{720} + 0.239 + 0.53\right)(720)$$

= 599 kip-in.

$M_{cr} > M_u$ Therefore, analysis is valid.

Check flexural strength of structural wythe

f_{ps} = 255 ksi ACI 318-05 Eq. (18-3)

$$a = \frac{(nA_{ps}f_{ps}) + P_{u,total}}{0.85f'_c b}$$

$$= \frac{[(7)(0.153)(255) + P_{u,total}]}{0.85(5)(120)}$$

= 0.62 in.

$c = a/\beta_1$
= 0.62/0.8
= 0.77 in.

$\varepsilon_t = [(\varepsilon_c)\,(d_p - c)]\,/\,(c)$

ε_t = [(0.003)(3–0.77)]/(0.77)
= 0.009

Since $\varepsilon_t > 0.005$, section is tension controlled and ø = 0.9.

$$\phi M_n = \phi\left[\left(nA_{ps}f_{ps}\right) + P_{u,total}\right]\left(d - \frac{a}{2}\right)$$

$$= 0.9\left[\left((7)(0.153)(255)\right) + 45.1\right]\left(3 - \frac{0.62}{2}\right)$$

= 770 kip-in. > 562 kip-in.

$\phi M_n > M_u$ (OK)

$\phi M_n/M_{cr}$ = 770/599 = 1.29 > 1.2 (OK)

The upper row of sandwich panel connectors will be about 2 ft below the top of the parapet. The controlling moment in the 2-in.-thick parapet will be governed by handling forces. Unit weight of the 2 in. wythe = 0.025 psf

$W = 1.3(0.025) = 0.033$ kip/ft^2 where 1.3 is the PCI load factor for handling

$M_y = wal^2/2$. For l = 2 ft, and a = 1 ft (a unit strip of the face wythe),

$M_y = 0.0331(2)^2/2$
= 0.066 kip-ft/ft width

The critical section for the parapet is within the transfer length for the strand, so the prestressing steel stress must be reduced. The transfer length for 3/8 in. strand is about 19 in., so the cracking moment for the parapet can include f_{pc}:

$S = 12(2)^2/6 = 8$ in.^{3}

$M_{cr} = [f_{pc} + f_r]S$
= [0.228+0.530](8)/12
= 0.51 kip-ft/ft width (OK)

The sandwich panel connectors should be checked to ensure that the parapet is adequately anchored. The connector reaction can be calculated by making the simplifying assumption that the parapet is a cantilever with a short backspan.

EXAMPLE A3. NONCOMPOSITE SHEAR WALL PANEL

Design criteria

Same panel as in Example A2. Each panel is to be subjected to a seismic force Q_E of 16.1 kip. The seismic design category of the structure is B, with S_{DS} = 0.25.

Individual panel resists shear (see Fig. A.6)

From ASCE 7-05, section 12.4.2.3:

$U = (0.9\text{-}0.2S_{DS}DS)D + \rho Q_E E$ where: ρ = 1.0 for *SDC B*

$U = 0.85D + 1.0Q_E$

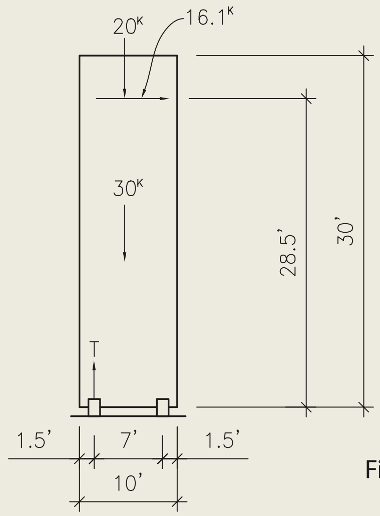


Fig. A.6

Total panel weight = (100 lb/ft^2 / 1000 lb/k)(10)(30)
= 30 kip

Panel base assumed to be fully grouted, sum moments about toe:

Total uplift (single panel) = [(16.1)(1.0)(28.5 ft) – (30 + 20)(0.85)(10/2)]/8.5 ft
= 29.0 kip uplift

Try connecting panels together to eliminate uplift.

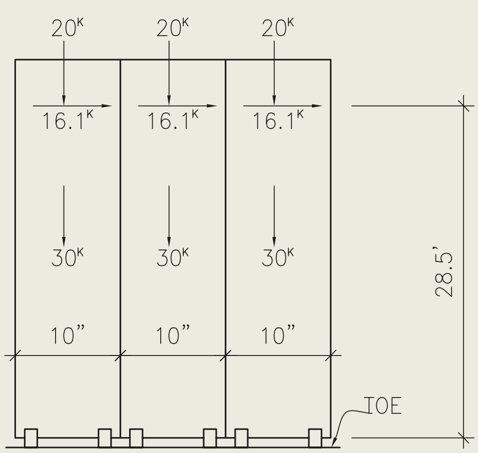


Fig. A.7

Panels connected to resist shear (see Fig. A.7)

Try connecting three panels:

Summation of moments at toe:

Due to dead load (30 + 20)(5 + 15 + 25) = 2250 kip-ft

Due to seismic load (16.1)(3)(28.5) = 1376 kip-ft

Strength design: U = 0.85D + 1.0*QE*

T_u (3 panels) = 0.85(2250) – 1.0(1376)
= 536.5 kip-ft
(+) No uplift present

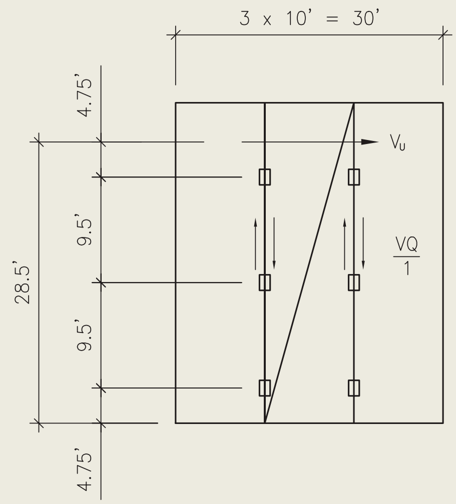


Fig. A.8

Panel-to-panel connections (see Fig. A.8)

V_u = $0.85D + 1.0Q_E$ (No D in direction of Q_E, therefore $V_u = 1.0Q_E$)

V_u = (16.1)(3)(1.0)
= 48.3 kip

Shear flow equation: $v = VQ/It$, $Q = A\bar{y}$

Q = (10)(10) (per unit width)
= 100 ft^2

I = $th^3/12$
I = $(30)^3/12$
= 2250 ft^3

V_uQ/I = (48.3)(100)/2250
= 2.15 kip/ft

V_u (joint) = (2.15 kip/ft)(28.5 ft) = 61.3 kip

Check dead load demand (determine portion of dead load that must be mobilized to resist overturning):

Dead load demand = 1376/{0.85(2250)} = 0.72

Therefore, max joint shear = 0.72(30+20)(2 panels) = 72 kip (controls)

If three connections per panel are used:

$V_{u\ Connx}$ = (72 kip)/3
= 24.0 kip each

Each connection is required to resist this shear force.

A connection between panels similar to that shown in section 2.10.7 can be designed for the 24.0 kip load using design methods in chapter 6 of the *PCI Design Handbook*.

Panel-to-Foundation Connections and Panel-to-Roof Connections

Since there is no uplift tension, the panel connection to foundation must transfer the shear force either by individual connection shear or shear friction, with the panel connectors serving as the shear-friction steel.

Use two connections per panel.

V_u = 16.1(1.0)/(2 connections)
= 8.05 kip each

A connection between the panel and the foundation similar to that shown in section 2.10.2 can be designed for the 8.05 kip load using design methods in chapter 6 of the *PCI Design Handbook*. Connections similar to that shown in section 2.10.3 can be designed for the panel-to-roof connections. See Example A2 for prestress design and handling design.

EXAMPLE A4. FULLY COMPOSITE CLADDING PANEL

Design criteria (see Fig. A.9)

Wind load = 30 lb/ft^2
Differential temperature = 30 ºF

f'_c = 5000 psi

f_{pu} = 270 ksi

Normalweight concrete

Strands:

Use $^3/_8$-in.-diameter, 270 ksi, low-relaxation strands.
Minimum prestress = 225 psi on gross section.
Try a composite insulated wall panel, 3/2/3. Assume 100% composite action for ultimate strength.

Section properties

$A = t_s a$ = (3 in.)(96 in.)(2)
= 576 in.^{2}

I = [(96)(3)^3/12 + (96)(3)(2.5)^2](2)
= 4032 in.^{4}

w = 2(3 ft)/12 ft (150 lb/ft^3)(8 ft/ft)

w = 600 lb/ft (self-weight)

Prestress

Find A_{ps} by using the minimum prestress stress of 225 psi (according to the ACI code, transverse reinforcement is not required if the panel is prestressed to at least 225 psi):

$$A_{ps,\,min.} = \frac{0.225A}{\phi P}$$

Note: To calculate P, 0.75 is the percent strand pull, 270 is the ultimate strength in ksi, and the 0.85 coefficient approximates the strand stress after all losses.

$$= \frac{0.225(576)}{0.75(0.85 \times 270)}$$
$$= 0.752 \text{ in.}^2$$

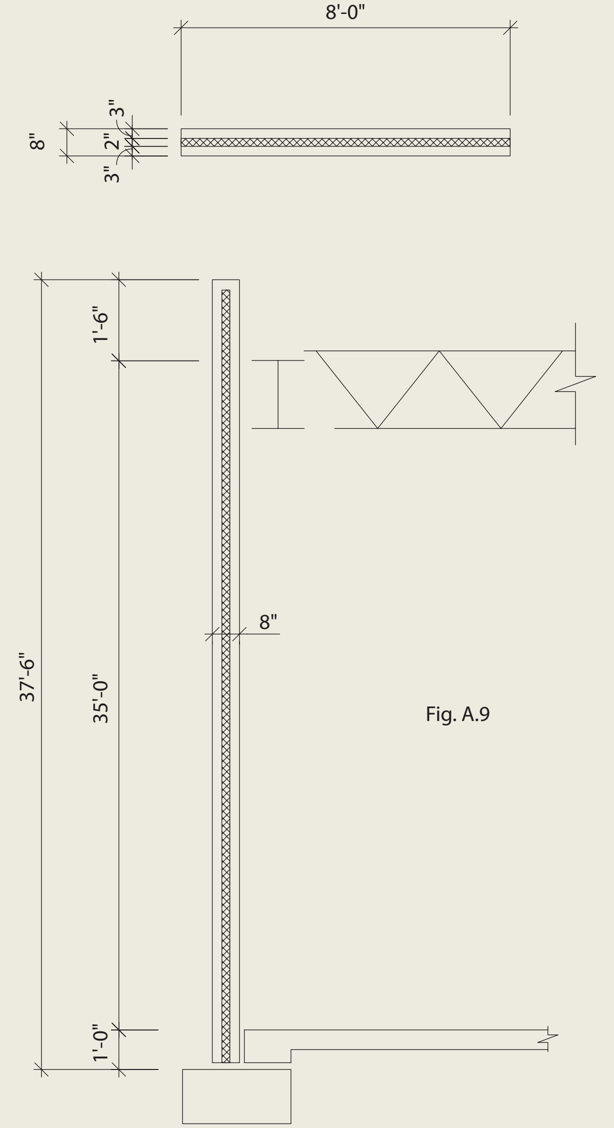


Fig. A.9

Number of strands (minimum) = 0.752/0.085 = 8.85 ($^3/_8$ in. diameter)

Try ten $^3/_8$-in.-diameter strands = 0.85 in.^{2}.

Use five $^3/_8$-in.-diameter strands in each wythe.

Normally, a P-Δ check is not necessary for a non-load-bearing panel. The calculation is included here to demonstrate that P-Δ does not control in this case.

Analysis

- Check conditions at mid-height
- Consider P-Δ effect

At mid-height:

Dead load and lateral wind load:

Dead load (self-weight):
P_D = (0.600)(36/2 + 1.5)

$= 11.7$ kip

Wind load:
$M_w = (0.030)(8)(35)^2/8$
$= 36.8$ kip-ft

Consider thermal bow because the panel is composite:

$\Delta_T = C(T_1 - T_2)\ l^2/8h$ (T_1 = outside temperature, T_2 = inside temperature)
$= (6 \times 10^{-6})(30)(35 \times 12)^2/(8 \times 8)$
$= 0.5$ in.

Initial bow:

Assume an initial outward bow of $l/360$, based on the PCI specified maximum bow tolerance, $\Delta_i = 1.17$ in.

Determine EI from *PCI Design Handbook*:

$\phi_k = 0.85$

(The stiffness-reduction factor is assumed to be at least 0.85 considering the stringent dimensional accuracy provided by a precasting plant.)

$$E_c = 57\sqrt{f_c'} = 57\sqrt{5000}$$

$= 4030$ ksi

$$EI = \frac{\phi_k E_c I}{1+\beta_d} = \frac{0.85(4030)(4032)}{1+0}$$

$= 1.38 \times 10^7$ kip-in.^{2}

in which $\beta_d = 0$ because sustained self-weight is small.

Wind deflection:

$$\Delta = \frac{5}{384}\left(\frac{W_{wind}\,l^4}{EI}\right)$$

$w = 30$ lb/ft^2 x 8 ft / 1000 lb/kip = 0.24 kip/ft

$$\Delta_w = \frac{5}{384}\left[\frac{(1.6\times 0.24)(420)^4}{(1.38\times 10^7)}\right]\left(\frac{1}{12}\right)$$

$= 0.94$ in.

Case 1, Wind Load: $U = 1.2D + 1.6W + 1.0L + 0.5$ (L_r or S or R)

$L = L_r = S = R = 0$ so $= 1.2D + 1.6W$

$P_u = 1.2(11.7)$
$= 14.0$ kip

$EI = \phi_k EI/(1 + \beta_d)$

$\beta_d = 1$ (for wind load) and $EI = 6.90 \times 10^6$ kip-in.^{2}

$M_u = (1.6)(36.8) + P\Delta$
$= 58.9 + P\Delta$

$\Delta_i + \Delta_w = 1.17 + 0.94$
$= 2.11$ in.

***P*-Δ Analysis**

$$\Delta = \frac{Pel^2}{8EI}$$

$$= \frac{(14)e(420)^2}{8(6.9\times 10^6)}$$

$= 0.045e$

First iteration:
$\Delta = 0.045(2.11)$
$= 0.095$ in.

Second iteration:
$\Delta = 0.045(2.11 + 0.095)$
$= 0.099$ in.

Third iteration:
$\Delta = 0.045(2.11 + 0.099)$
$= 0.099$ in.
(converges)

$M_u = 58.9 + 14(2.11 + 0.099)/12$
$= 61.5$ kip-ft

Case 2: $U = 1.2(D+F+T) + 1.6\ (L+H) +$

0.5 (L_r or S or R)

$P_u = 11.7(1.2)$
$= 14.0$ kip

where $\beta_d = 1$ due to sustained dead load and $EI = 6.90 \times 10^6$ k-in.^{2}

$\Delta_T + \Delta_i = 0.5 + 1.17$

$$\Delta = \frac{Pel^2}{8EI} = 1.67 \text{ in.}$$

$$= \frac{(14)e(420)^2}{8(6.9\times 10^6)}$$

$= 0.045e$

First iteration:
$\Delta = 0.045(1.67) = 0.075$ in.

Second iteration:
$\Delta = 0.045(1.67 + 0.075)$
$= 0.08$ in. (converges)

$M_u = (14)(1.75)/12$
$= 2.04$ kip-ft

Therefore, the critical case is Case 1: $1.2D + 1.6W + 1.0L + 0.5$ (L_r or S or R)

$P_u = 14.0$ kip

$M_u = 61.5$ kip-ft

From Fig. 3.12.5, *PCI Design Handbook*, the interaction capacities for an 8 in. composite panel section are adequate.

Check for cracking at mid-height of panel (use ultimate loads):

P_u = 14.0 kip

M_u = 61.5 kip-ft

$f_{DL+wind} = P/A + MC/I$:

$$f_{DL+wind} = \frac{(10)(0.085)(0.75\times 270)(10^3)(0.85)}{576}$$

= -708 psi (tension)

$f_{ps} = P/A$:

$$f_{ps} = \frac{(10)(0.085)(0.75\times 270)(10^3)(0.85)}{576}$$

= +254 psi (compression)

Net stress = -708 + 254 = 454 psi tension

$$f_r = 7.5\sqrt{f_c'}$$

$$= 7.5\sqrt{5000}$$

= 530 psi max tension
OK

Therefore, section is uncracked.

Check to see whether $\phi M_n > 1.2\ M_{cr}$.

Determine M_n:

Assume the strand stress at ultimate equals the release stress:

f_{ps} = 269 ksi

$$a = \frac{A_{ps}f_{ps}}{0.85f_c'b}$$

$$= \frac{(5\times 0.085)269}{0.85(5)(96)}$$ (Strands in compression wythe are ignored – contribution is small)

= 0.28 in.

d = 8 – 3/2 = 6.50 in.

$\phi M_n = \phi A_{ps}f_{ps}(d - a/2)$

ϕM_n = (0.9)(114.3)[6.5 – (0.28/2)]/12
= 54.5 kip-ft

$$1.2\ M_{cr} = \frac{(1.2)(0.530+0.254)(4032)}{(4)(12)}$$

= 79.0 kip-ft
> ϕM_n = 55 kip-ft

Add mild reinforcing steel to increase ϕM_n

$$A_{s,\ req.} = \frac{(1.2M_{cr} - \phi M_n)}{df_y}$$

$$= \frac{(79.0 - 54.5)(12\ in./ft)}{(6.5)(60)} = 0.75\ \text{in.}^2$$

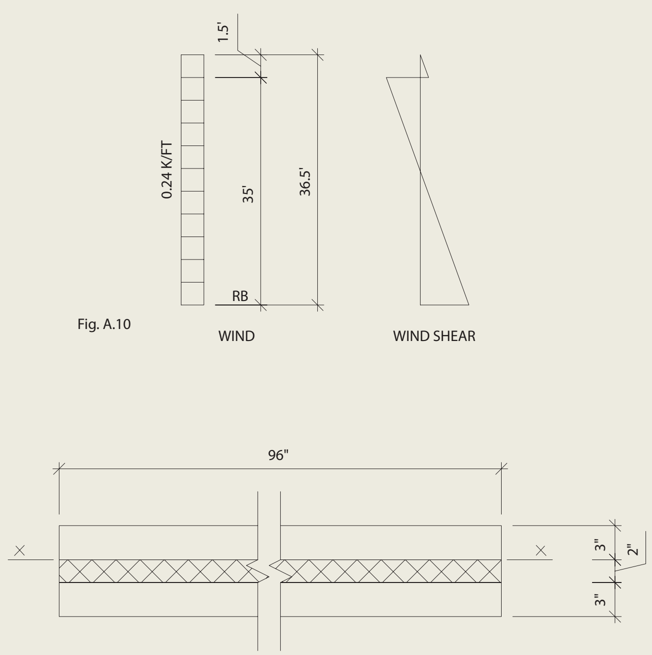


Fig. A.10

Add no. 4 bars in each wythe at strand level:

n = $A_{s,\ req}$/0.2 in.^{2} = 3.4, add four no. 4 bars

a = 0.40 in., ϕM_n = 76.7 kip-ft vs 79 kip-ft, (OK)(strands in compression wythe ignored)

Composite action of concrete and insulation (see Fig. A.10):

Strand force transfer:

$T_u = nA_{ps}f_{ps}$ = (5)(0.085)(270)
= 114.75 kip [use this value—see following]

C_u = (0.85)(5)(3)(96)
= 1224 kip

Provide wythe interconnectors to resist 114.75 kip ultimate in each half height of panel.

Stripping (see Fig. A.11)

f'_{ci} = 3500 psi

w = 75 lb/ft^2

a = 8 ft

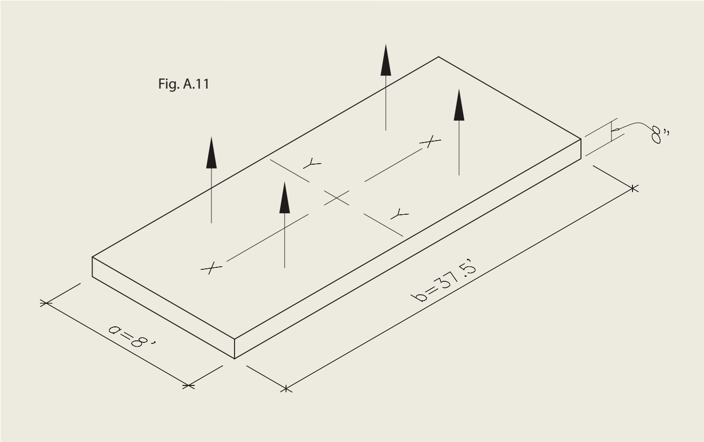
b = 37.5 ft

Static load multiplier = 1.3

w_i = (75)(1.3)
= 98 lb/ft^2

x-x Direction

$M_x = 0.0107wa^2b$
= (0.0107)(0.098)(8)^2(37.5)



$= 2.52$ kip-ft

over a width of $15a = 15(8) = 120$ in.

$I_x = [(120)(3)^3/12 + (120)(3)(2.5)^2](2)$
$= 5040 \text{ in.}^4$

$f_b = MC/I$

$f_b = (2520)(12)(4)/5040$
$= 24$ psi

$f_{pc} = 0$, transverse direction – no prestress

Net stress = 24 – 0 = 24 psi

$f'_r = 5\sqrt{f'_c}$

$= 5\sqrt{3500}$

$= 296$ psi (OK)

y-y Direction

$M_y = 0.0107\, wab^2$
$= (0.0107)(0.098)(8)(37.5)^2$
$= 11.80$ kip-ft

over a width equal to $0.50a = 0.50(8)(12) = 48$ in.

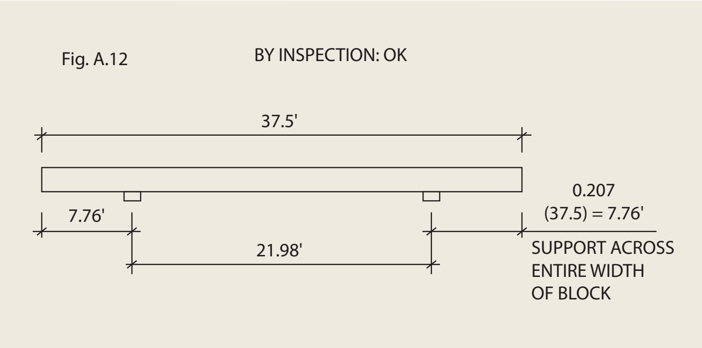
$I_{x,48} = [(48)(3)^3/12 + (48)(3)(2.5)^2](2)$
$= 2016 \text{ in.}^4$

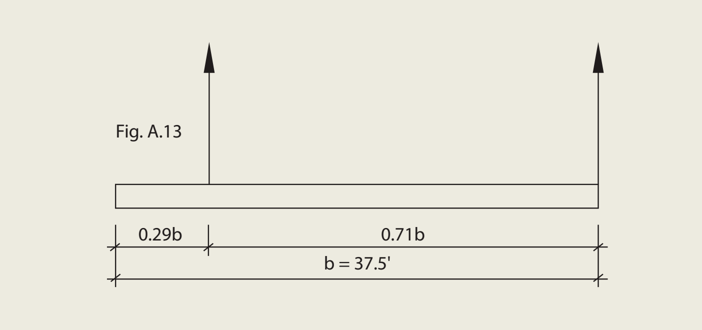
$f_{by} = -(11800)(12)(4)/2016$
$= -281$ psi

$f_{pc} = (254)(0.9)/0.85$ (final prestress stress converted to initial stress by ratio of 0.9/0.85)
$= +269$ psi

Net stress = -281 + 269 = -12 psi

Note that only 10% stress loss occurred at this stage.





Yard Handling

Use the same arrangement as shown previously.

Static load multiplier = 1.2

Shipping (see Fig. A.12)

Static load multiplier = 1.5

$w = 75\,(1.5) = 113 \text{ lb/ft}^2$

$f_{by} = (-281)(113)/98$
$= -324$ psi

$f_{pc} = +254$ psi (use final prestress stress for this stage)

Net stress = -324 + 254

= -70 psi (OK)

The optimal balance between positive and negative moments is achieved by bunking at 0.207 times the panel length:

Erection (see Fig. A.13)

Refer to *PCI Design Handbook*.

Try two-point pick-up.

Static load multiplier = 1.2

$w = 75(1.2) = 90 \text{ lb/ft}^2$

or $(90)(8)/1000 = 0.72$ kip/ft

$M^{\pm} = 0.044wb^2$
$= (0.044)(0.72)(37.5)^2$
$= 44.6$ kip-ft

Effective width = 96 in.

$f_{by} = (44.6)(12000)(4)/4032$

= -530 psi

f_{pc} = +254 psi

Net stress = -530 + 254 = -276 psi

which is less than $5\sqrt{5000}$ = 354 psi (OK, $7.5\sqrt{5000}$ cracking stress with a 1.5 factor)

EXAMPLE A5. FULLY COMPOSITE LOAD-BEARING PANEL

Design criteria (see Fig. A.14)

Wind load:
Direct pressure = 20 lb/ft^2
Suction pressure = 10 lb/ft^2
Differential temperature = 30 ºF
Service roof dead loads:
8-ft-wide double tee (8DT18) spanning 40 ft = 45 lb/ft^2
Roofing and mechanical = 10 lb/ft^2
Service roof snow load = 30 lb/ft^2

Roof reaction:

Dead load = (0.045+0.010)(20)(8)
= 8.8 kip

Snow load = (0.030)(20)(8)
= 4.8 kip

f'_c = 5000 psi

E_c = 4030 ksi

f_{pu} = 270 ksi

The roof acts as a diaphragm supported for lateral loads by shear walls at the ends of the building. Therefore, the wall panel is braced at the top and bottom. The wall panels are connected to the double tee flanges at 1 ft 6 in. below the top of the panel and are connected to the top of the floor slab at 1 ft 0 in. above the panel base. Therefore, the unbraced length of the panels is assumed to be 35 ft 0 in.

Section properties

$A = at = (6)(96) = 576$ in.^{2}

$I = b(h_1^3 - h_2^3)/12 = 4032$ in.^{4}

Note: The value of I for deflection should be verified by full-scale testing of panels with a similar configuration to the design panel. Overestimating the effective moment of inertia I is unconservative, as the stability calculations are directly affected by the value of EI.

S = 1008 in.^{3}

w = (6 in./12 in./ft)150 lb/ft^3(8 ft) = 600 lb/ft

Strands

Use 3/8-in.-diameter, low-relaxation, 270 ksi strands.

Minimum prestress = 225 psi = f_{pc}

Assume prestress losses = 15%

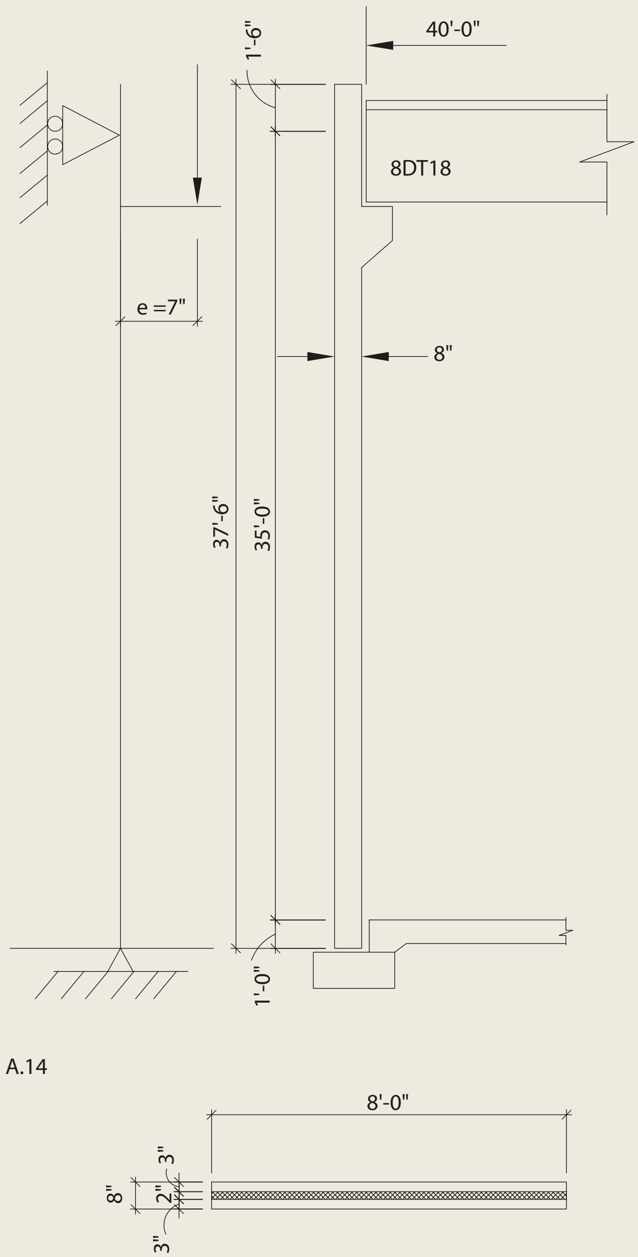


Fig. A.14

Prestress

$A_{ps,min.} = 0.225\ (A/f_{ps})$

Percent pull = 75%, final losses estimated at 15%:

$A_{ps,min.}$ = (0.225)(576)/[(0.75)(270)(0.85)]
= 0.753 in.^{2}

Number of strands = 0.753/0.085
= 8.86 (use 10 strands)

f_{pc} = [(0.75)(270)(0.85)(10)(0.085)(10^3)]/576 = 254 psi with 10 strands > 225 psi (OK)

Use five 3/8-in.-diameter strands in each wythe.

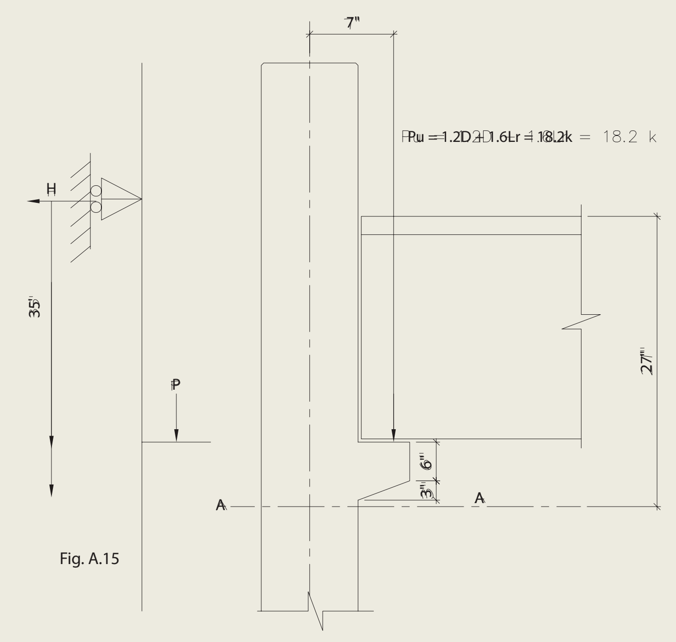


Fig. A.15

Analysis

Case 1 (no wind): $U = 1.2D + 1.6\ (L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W)$

$(L_r = R = L = W = 0)$

Load at top of panel:

$P_{u\text{-}top} = 1.2(8.8) + 1.6(4.8)$
$= 18.2$ kip

Panel weight at midheight:
$P_{u\text{-}mid(self\text{-}wt)} = 1.2[0.600(37.5/2)]$
$= 13.5$ kip

Total factored load at midheight $= P_{u\text{-}mid} = 18.2 + 13.5 = 31.7$ kip

Note: Midheight may not necessarily be the point of maximum bending moment because of the large moment applied at the corbel. It may be necessary to check several sections along the panel height to confirm where the maximum moment in the panel occurs.

$$EI = \frac{\phi_k E_c I}{1 + \beta_d}$$

where

$\beta_d = [13.5 + 1.2(8.8)]/31.7$
$= 0.76$

$\phi_k = 0.85$

Stiffness-reduction factor is assumed to be at least 0.85, considering the stringent dimensional accuracy provided by a precasting plant.

Therefore:

$$EI = \frac{0.85(4030)(4032)}{1 + 0.76}$$

$= 7.85 \times 10^6$ kip-in.^{2}

Deflection at midheight due to factored load on corbel of panel:

$\Delta = P_{u\text{-}top} e_p l^2/16EI$

See Fig. A.14 for 7 in. eccentricity $= e_p$

$$\Delta = \frac{(18.2)(7)(420)^2}{16(7.85 \times 10^6)}$$ (The unbraced span length is 35 ft = (35)(12) = 420 in.)

$= 0.18$ in.

Assume initial bow = 1.17 in. (b/360)

Total deflection $= 0.18 + 1.17$
$= 1.35$ in.

P-Δ Analysis

$\Delta = P_{u\text{-}mid} e l^2/8EI$

$$= \frac{(31.7)e(420)^2}{8(7.85 \times 10^6)}$$

$= 0.09e$

First iteration:
$e = 1.35$ in.
$\Delta = (0.09)(1.35) = 0.12$ in.

Second iteration:
$e = 1.35 + 0.12 = 1.47$ in.
$\Delta = (0.09)(1.47) = 0.13$ in.

Third iteration:
$e = 1.35 + 0.13 = 1.48$ in.
$\Delta = (0.09)(1.48) = 0.13$ in.
(converges)

Moment at midheight:

$M_u = P_{u\text{-}top}(e_P)/2 + P_{u\text{-}mid}\, e = (18.2)(7)/2 + (31.7)(1.48)$
$= 110.6$ kip-in.

Load at midheight:

$P_{u\text{-}mid} = 31.7$ kip

Check for cracking:

Case 1: $1.2D + 1.6S$, at midheight:
$f = (31.7)(10^3)/576 - (110.6)(10^3)/1008$
$= -55$ psi

$f_{pc} = 254$ psi

Net stress $= -55 + 254$
$= +199$ psi (compression)

Therefore, stresses are satisfactory; that is, the panel is uncracked and the P-Δ analysis is valid.

Check interaction curve:

$M_u = 110.6/(8\times12) = 1.15$ ft kip/ft

$P_{u\text{-}mid} = 31.7/8 = 3.96$ kip/ft

From *PCI Design Handbook*, Fig. 3.12.5, the interaction diagram shows that the section is satisfactory.

Case 2 (include wind): $U = 1.2D + 1.6W + 1.0L + 0.5\ (L_r \text{ or } S \text{ or } R)$

$(L = L_r = R = 0)$

For wind, use 10 lb/ft^2 suction, which causes moments additive to moments due to dead and snow load and eccentricity.

Load at top of panel:

$P_{u\text{-}top} = [1.2(8.8) + 0.5(4.8)]$
$= 13$ kip

Panel weight at midheight:

$P_{u\text{-}mid(self\text{-}wt)} = 13.5$ kip

Total factored load at midheight

$P_{u\text{-}mid} = 13 + 13.5 = 26.5$ kip

$$EI = \frac{\phi_k E_c I}{1 + \beta_d}$$

Where

$\beta_d = [13.5 + (1.2)(8.8)]/26.5$
$= 0.91$

Therefore, for $\beta_d = 0.91$, $EI = 7.23 \times 10^6$ kip-in.^{2}

for $\beta_d = 0$, $EI = 1.38 \times 10^7$ kip-in.^{2}

Deflection at midheight due to load at top:

$\Delta = 0.14$ in.

Deflection due to wind:

$$\Delta = \frac{5}{384}\left(\frac{w_{wind} l^4}{EI}\right)$$

$$= \frac{5}{384}\left(\frac{\left(0.01 \times 1.6 \times 8\right)\left(420\right)^4}{12\left(1.38 \times 10^7\right)}\right)$$

$= 0.31$ in.

Assume initial bow = 1.17 in.

Total deflection $= 0.14 + 0.31 + 1.17$
$= 1.62$ in.

P-Δ Analysis

$\Delta = P_{u\text{-}mid} e l^2/8EI$

$$= \frac{\left(26.5\right)e\left(420\right)^2}{8\left(7.23 \times 10^6\right)}$$

$= 0.081e$

First iteration:
$e = 1.62$ in.

$\Delta = (0.081)(1.62) = 0.13$ in.

Second iteration:
$e = 1.75$ in.

$\Delta = (0.081)(1.75) = 0.14$ in.

Third iteration:
$e = 1.76$ in.

$\Delta = (0.081)(1.76) = 0.14$ in.
(converges)

$M_w = wl^2/8 = (0.01)(8)(35 \times 12)^2/(12 \times 8)$
$= 147$ kip-in.

$M_{uw} = 147 \times 1.6$
$= 235.2$ kip-in.

$M_{uPu} = P_{u\text{-}top}(e_P)/2 + P_{u\text{-}mid}\, e = (13.0 \text{ x } 7)/2 + (26.5)(1.76)$
$= 92.1$ kip-in.

Total moment at midheight:

$M_u = 235.2 + 92.1 = 327.3$ kip-in.

Load at midheight:

$P_{u\text{-}mid} = 26.5$ kip

Check for cracking:

Case 2: $1.2D + 1.6W + 0.5S$, at midheight:

$f = (26.5)(10^3)/576 - (327.3)(10^3)/1008$
$= -279$ psi

$f_{pc} = 254$ psi

Net stress $= -279 + 254$
$= -25$ psi

$fr = 7.5\sqrt{5000} = 530 \text{ psi} > 25$ psi

Therefore, stresses are satisfactory; that is, the panel is uncracked and the *P-Δ* analysis is valid.

Check interaction curve:

$M_u = 327.3/(8 \times 12) = 3.41$ kip-ft/ft

$P_{u\text{-}mid} = 26.5/8 = 3.31$ kip/ft

From *PCI Design Handbook*, Fig. 3.12.5, the interaction diagram is satisfactory.

Check for cracking at the base of the corbel, where the moments due to factored gravity loads are highest. Note: Moment due to wind is negligible at this location; therefore, ignore wind load effects in this case. Use 35 ft as the unbraced length.

Case 1 (no wind): $U = 1.2D + 1.6\ (L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W)$

$(L_r = R = L = W = 0)$

$U = 1.2D + 1.6S$ at section A-A (Fig. A.19)

$P_{u\text{-}top} = 1.2(8.8) + 1.6(4.8)$
$= 18.2$ kip

Calculate the horizontal reaction at the double tee–to–wall panel connection:

$H_u = P_{u\text{-}top}(e_P)/l = (18.2 \times 7)/(35 \times 12)$
$= 0.30$ kip

Moment at section A-A:

$M_u = 18.2 \text{ kip} \times 7 \text{ in.} - 0.30 \text{ kip} \times 27 \text{ in.}$
$= 127.4 - 8.1 = 119.3$ kip-in.

$f = (18.2 \times 10^3)/576 - (119.3 \times 10^3)/1008$
$= -87$ psi

Note: Because the base of the corbel is beyond the transfer point, use the full value of f_{pc}.

$f_{pc} = +254$ psi

Net stress = 254 – 87
= 167 psi (compression)

Therefore, stresses are satisfactory; that is, the panel is uncracked and the P-Δ analysis is valid.

Check interaction curve:

$M_u = 119.3/(8)(12) = 1.24$ kip-ft/ft

$P_{u\text{-}top} = 18.2/8 = 2.28$ kip/ft

From *PCI Design Handbook*, Fig. 3.12.5, the interaction diagram is satisfactory.

Case 3 (include wind): $U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.8W$

By inspection, the P-Δ analysis will converge and the moments will be less than in Case 2. Therefore, the section is satisfactory for ultimate moment capacity.

Horizontal Shear at Interface of Concrete and Insulation

Strand force transfer will be critical.

$T_u = (5)(0.085)(270)$
$= 114.8$ kip

$C_u = (0.85)(5)(3)(96)$
$= 1224$ kip

Provide wythe connectors to resist 114.8 kip (ultimate) in each half height of panel.

Note: Wythe connectors should be stiff enough to ensure that the relative end slip between wythes is close to zero at ultimate loads. Solid zones of concrete may be necessary to achieve full composite action for panel stiffness.

Check to see if ϕM_n is greater than $1.2M_{cr}$.

See Example A4. Add four no. 4 reinforcing bars in each wythe at strand level.

Handling and Erection

See composite cladding panel, Example A4.

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