Manual

PCI Manual for the Design of Hollow Core Slabs and Walls

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Third Edition - Electronic Version

PCI
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Notation

a = Depth of equivalent compression stress block
a_i = Dynamic coefficient for vibration
a_p = Amplification factor related to the response of a system or component as affected by the type of seismic attachment
a_θ = Depth of equivalent compression stress block under fire conditions
A = Cross–sectional area
A_{comp} = Cross-sectional area of equivalent rectangular stress block
A_{cr} = Area of crack face
A_e = Net effective slab bearing area
A_{ps}, A'_{ps} = Area of prestressed reinforcement in flexural tension zone and compression zone, respectively
A_s = Area of non-prestressed longitudinal tension reinforcement
A_{Tchord} = Transformed area of diaphragm chord
A_{trcomp} = Cross-sectional area of transformed composite section
A_{vf} = Area of shear friction reinforcement
b = Width of compression face
b = Width of slab
b_v = Width of cross section at contact surface being investigated for horizontal shear
b_w = Net web width of hollow core slab
c = Distance from extreme compression fiber to neutral axis
c_c = Clear cover of reinforcement
C = Confinement factor for horizontal joints
C = Compressive force (with subscripts to define specific locations)
C = Coefficient of thermal expansion
C = Seismic factor dependent on site and structure fundamental period
C = Factor for calculating steel relaxation losses as given in Table 2.2.3.2
C_d = Deflection amplification factor
C_m = Factor relating actual moment diagram to an equivalent uniform moment diagram
C_s = Seismic response coefficient
C_t = Building period coefficient
C_{vx} = Vertical distribution factor
C_w = Factor in calculation of n_a
CR = Prestress loss due to concrete creep
d = Distance from extreme compression fiber to centroid of non–prestressed tension reinforcement
d_b = Nominal diameter of reinforcement
d_p, d'_p = Distance from extreme compression fiber to centroid of prestressed reinforcement in tension and compression zones, respectively
D = Dead load
D_i = Length of shear wall i
D_s = Superimposed dead load
D_{sw} = Self-weight
D_t = Topping weight
D_u = Factored dead load for seismic calculations
DW = Width of effective resisting section
e = Eccentricity of design load or prestressing force parallel to axis measured from centroid of section or reaction
E_c = Modulus of elasticity of concrete
E_{ci} = Modulus of elasticity of concrete at the time of initial prestress
E_d = Dynamic modulus of elasticity
E_{ps} = Modulus of elasticity of prestressing reinforcement
ES = Prestress loss due to elastic shortening of concrete
E_s = Modulus of elasticity of reinforcement, excluding prestressing reinforcement, and structural steel
E_v = Vertical seismic force
f = Frequency of building
f_{bot} = Stress in bottom fiber of cross section
f'_c = Specified compressive strength of concrete
f'_{ci} = Compressive strength of concrete at the time of initial prestress
f_{cds} = Concrete stress at center of gravity of prestressing force due to all permanent (dead) loads not used in computing f_{cir}
f_{cir} = Concrete stress at center of gravity of prestressing force immediately after transfer
f_d = Stress at extreme tension fiber due to unfactored member self weight
f_f = Forcing frequency of rhythmic activity
f_n = Natural frequency of floor system

f_{pc} = Compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads

f_{pe} = Compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads

f_{pi} = Ratio of initial prestress force to area of prestressing steel

f_{ps} = Stress in prestressed reinforcement at nominal strength of component

$f_{ps\theta}$ = Stress in prestressing steel at nominal strength at elevated temperature

f'_{ps} = Stress in prestressed reinforcement in compression zone at nominal strength of component

f_{pu} = Specified tensile strength of prestressing steel

$f_{pu\theta}$ = Ultimate tensile strength of prestressing steel at elevated temperature

f_{px} = Maximum steel stress in partially developed strand

f_r = Modulus of rupture of concrete

f'_r = Allowable flexural tension computed using gross concrete section

f_{se} = Effective stress in prestressing steel (after allowance for all prestress losses)

f_t = Extreme fiber stress in tension in precompressed tensile zone

f_{top} = Stress in top fiber of cross section

f_u = Design compressive strength of wall or grout, whichever is less, when walls are reinforced against splitting and slab cores are filled

f_u = 80% of design compressive strength of wall or design compressive strength of grout, whichever is less, when walls are not reinforced against splitting or slab cores are not filled

f_y = Specified yield strength of reinforcement

F_a = Short-period site coefficient

F_i = Portion of base shear applied at level i

F_p = Seismic force acting on component of a building

F_{px} = Force applied to diaphragm at level under consideration

F_{roof}, F_{max}, F_{min}, F_u = Seismic design force

F_v = Long-period site coefficient

g = Acceleration due to gravity

GC_p = Product of internal pressure coefficient and gust-effect factor to be used in determination of wind loads on buildings

h = Overall height or thickness of component

h_n = Height above the base to the highest level of the building

h_{net} = Net height of grout in keyway between slab units

H = Load due to lateral earth pressure, ground water pressure, or pressure of bulk materials

I = Moment of inertia of section about centroidal axis (with subscripts)

I_{comp} = Moment of inertia of transformed composite section about centroidal axis

I_{cr} = Moment of inertia of cracked section transformed to concrete

I_e = Seismic importance factor

I_{eff} = Effective moment of inertia for second-order analysis

I_g = Moment of inertia of gross section about centroidal axis, neglecting reinforcement

I_p = Component importance factor

j = Height of internal couple expressed as fraction of depth

J = Factor for calculating steel relaxation losses as given in Table 2.2.3.1

k = Fraction of total load in a horizontal joint in a grout column

k = Effective length factor for compression members

k = Factor in vertical distribution of seismic forces

k_a = Amplification factor for diaphragm flexibility

K = Member stiffness

K_{cir} = Factor for calculating elastic shortening prestress losses

K_{cr} = Factor for calculating prestress losses due to concrete creep

K_d = Directionality factor for wind load

K_{es} = Factor for calculating prestress losses due to elastic shortening

K_{re} = Factor for calculating prestress losses due to steel relaxation as given in Table 2.2.3.1

K_{sh} = Factor for calculating prestress losses due to concrete shrinkage

K'_u = Factor from PCI Handbook Fig. 5.14.2 for calculating flexural design strength
K_{zt} = Topographic factor at mean roof height
ℓ = Span length
$\ell_{available}$ = Strand embedment length from member end to point of maximum stress
ℓ_d = Development length in tension of deformed bar, deformed wire, plain and deformed welded wire reinforcement, or pretensioned strand
ℓ_f = Flexural bond length
ℓ_j = Length of joint under consideration
ℓ_{pc} = Overall member length
ℓ_t = Strand transfer length
ℓ_u = Unbraced length
L = Live load
L_{eff} = Building length parallel to wind direction
L_f = Span of flexible diaphragm
L_r = Roof live load
M = Service load moment
M_{comp} = Moment applied to composite section
M_{cr} = Cracking moment
M_{cre} = Moment causing cracking at section due to externally applied loads
M_d = Moment due to service dead load
M_g = Unfactored moment due to weight of component
M_{max} = Maximum factored moment due to externally applied loads
= $M_u - M_d$
M_n = Nominal flexural strength at section
$M_{n\theta}$ = Nominal flexural strength at elevated temperature
$M_{non\text{-}comp}$ = Moment applied to non-composite section
M_{sd} = Unfactored moment due to superimposed dead load plus sustained live load
$M_{service}$ = Moment due to unfactored loads
$M_{superimposed}$ = Moment due to unfactored superimposed loads
$M_{sustained}$ = Moment due to sustained loads
M_u = Factored moment at section
M_θ = Applied moment for fire condition
n = Modular ratio
n_a = Approximate lower bound natural frequency
n_1 = Fundamental natural frequency
N = Axial force
N_u = Factored horizontal or axial force
p = Design pressure to be used for determination of wind loads on buildings
P_c = Critical buckling load
P_e = Effective prestress force after all losses
P_{ex} = Effective prestress force in partially developed strand
P_i = Initial prestress force (jacking force after anchorage loss)
P_n = Nominal strength of joint
P_n = Axial load nominal strength of compression component at given eccentricity
P_o = Prestress force after transfer losses
P_u = Factored point load
q = Wind velocity pressure
q_h = Wind velocity pressure evaluated at height $z = h$
q_i = Velocity pressure for internal pressure determination
q_z = Wind velocity pressure at height z above ground
Q = First moment of area
r = Radius of gyration of cross section of a compression member
R = Fire endurance rating in minutes
R = Thermal resistance
R = Response modification factor dependent on structural system type
R_e = Reduction factor for load eccentricity in horizontal joints
R_p = Component response modification factor
RE = Prestress loss due to steel relaxation
RH = Average ambient relative humidity
S_b = Section modulus with respect to bottom fiber of cross section
S_{D1} = Design spectral response acceleration at a 1-sec period
S_{DS} = Design spectral response acceleration at short periods
S_{MS} = Maximum considered earthquake, 5% damped, spectral response acceleration at short periods adjusted for site class effects
S_{M1} = Maximum considered earthquake, 5% damped, spectral response acceleration at a period of 1 sec adjusted for site class effects

S_s = Mapped maximum considered earthquake, 5% damped, spectral response acceleration at short periods
S_t = Section modulus with respect to the top fiber of a cross section
S_1 = Mapped maximum considered earthquake, 5% damped, spectral response acceleration at a period of 1 sec
SH = Prestress loss due to concrete shrinkage
t_{eq} = Area/width
t_g = Width of grout column in horizontal joint
T = Tensile force
T = Thermal thrust
T = Fundamental period of building
T_a = Approximate fundamental period of building
T_L = Long-period transition period between equations for the site-specific design spectral response acceleration
u = Distance from bottom of slab to a point within a member
V = Seismic base shear
V_c = Nominal shear strength provided by concrete
V_{ci} = Nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment
V_{cw} = Nominal shear strength provided by concrete when diagonal cracking results from high principal tensile stress in the web
V_d = Shear force at section due to unfactored self weight
V_h = Horizontal beam shear
V_i = Factored shear force at section due to externally applied loads occurring simultaneously with M_{max}
= V_u - V_d
V_n = Nominal shear strength
V_{nh} = Nominal horizontal shear strength
V_s = Nominal shear strength provided by shear reinforcement
V_u = Factored shear force at section
V_{uh} = Factored applied horizontal shear
V_x = Shear at location x
V/S = Volume to surface ratio
w = Unfactored load per unit length or per unit area
w = Bearing strip width
w_{cr} = Cracking load
$w_{equivalent}$ = Uniform load causing moment equivalent to actual loading
w_i = Portion of W at level i
w_n = load at nominal flexural strength
w_p = Component weight
w_{px} = Weight tributary to a diaphragm at level under consideration
w_u = Factored total load per unit length or per unit area
w_w = Weight of exterior wall
W = Total dead load plus other applicable loads for seismic design
W = Wind load
W_p = Component weight
x = Location in span
x_0 = Distance from simple support to inflection point
y' = Distance from top fiber to centroid of A_{comp}
y_b = Distance from bottom fiber to center of gravity of section
$y_{b,comp}$ = Distance from bottom fiber to center of gravity of transformed composite section
y_t = Used as either distance to top fiber or tension fiber from neutral axis
β_1 = Factor relating depth of equivalent rectangular compressive stress block to neutral axis depth
β_d = Ratio used to compute magnified moments in compression members due to sustained loads
β_m = Modal damping ratio
γ_p = Factor for type of prestressing steel
δ_{all} = Limit on free end slip for using ACI development length equations
δ_b = Braced moment magnifier
δ_s = Measured free end slip
δ_s = Unbraced moment magnifier
Δ = Deflection
Δ_T = Thermal bow
ε_{ps} = Strain in prestressing steel corresponding to f_{ps}
ε'_{ps} = Strain in prestressing steel in compression zone
ε_s = Strain in nonprestressed tension reinforcement
ε_{se} = Strain in prestressing steel after losses
λ = Stiffness modifier for moment magnification in slender compression members

λ = Modification factor related to unit weight of concrete

μ = Shear friction coefficient

μ_e = Effective shear friction coefficient

ρ = Redundancy load factor

ρ_p = Ratio of A_{ps} to bd (or bd_p) producing balanced strain conditions

ϕ = ACI strength reduction factor

ω_{pu} = $\dfrac{A_{ps} f_{pu}}{bd_p f'_c}$

Ω_o = Amplification factor to account for overstrength of the seismic force-resisting system

θ = Subscript denoting fire conditions

Chapter 1

HOLLOW CORE SLAB SYSTEMS

1.1 Methods of Manufacturing

A hollow core slab is a precast, prestressed concrete member with continuous voids provided to reduce weight and, therefore, cost. As a side benefit, voids can be used to conceal electrical or mechanical runs. Primarily used as floor or roof deck systems, hollow core slabs also have applications as both vertical and horizontal wall panels, spandrel members, and bridge deck slabs.

An understanding of the methods used to manufacture hollow core slabs will aid in the special considerations sometimes required in their use. This manual only addresses machine cast hollow core slabs. Custom cast voided slabs may behave in a similar manner. However, more design and detailing options may be available with such custom cast slabs. Those options are beyond the scope of this manual. The *PCI Design Handbook*[1] would be the proper design reference.

Machine cast hollow core slabs are cast using various methods in the seven major systems available today. Because each production system is patented, producers are usually set up on a franchise or license basis using the background, knowledge, and expertise provided with the machine development. Each producer then has the technical support of a large network of associated producers.

Two basic manufacturing methods are currently used for the production of hollow core slabs. The first is a dry-cast or extrusion system where a low-slump concrete is forced through the casting machine. The cores are formed with augers or tubes, and compaction and vibration are used to consolidate the concrete around the cores. The second (wet-cast) system uses a normal slump concrete. In this system, the sides of the slabs are formed either with stationary, fixed forms, or with forms attached to the machine (when the sides are slip formed). The cores in the wet-cast systems are formed with either lightweight aggregate fed through tubes attached to the casting machine, pneumatic tubes anchored in a fixed form, or long tubes attached to the casting machine that slip form the cores.

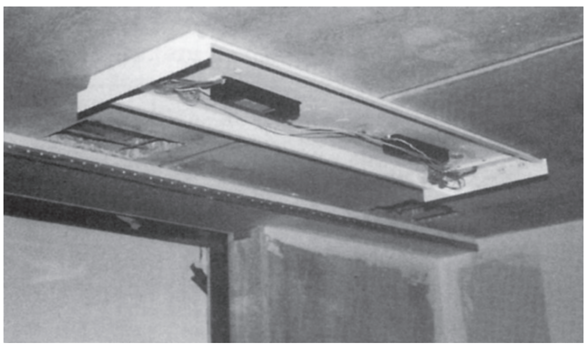
Table 1.1 lists the seven major hollow core slab systems available today along with the basic information on the casting technique. Various names may be used by local licensees to describe the same products. In most cases, the hollow core slabs are cast on long-line beds, normally 300 ft to 600 ft long. After curing, the slabs are sawcut to the appropriate length for the intended project.

The economy of the generalized hollow core slab system is in the quantity that can be produced at a given time with a minimum amount of labor required. Each slab on a given casting line will have the same number of prestressing strands. Therefore, the greatest efficiency is obtained by producing slabs with the same reinforcing requirements from several projects on a single casting line. The best efficiency for a single project is obtained if slab requirements are repetitive.

Table 1.1 Hollow core slab systems

Manufacturer	Machine type	Concrete type/slump	Core form
Dynaspan	Slip form	Dry/low	Tubes
Echo	Slip form	Dry/low	Tubes
Elematic	Extruder	Dry/low	Auger/tube
Flexicore	Fixed form	Wet/normal	Pneumatic tubes
Spancrete	Slip form	Dry/low	Tubes
SpanDeck	Slip form	Wet/normal	Filler aggregate
Ultra-Span	Extruder	Dry/low	Augers

The latex feathering is ready for direct carpet application.



Electrical and heating, ventilation, and air conditioning application.

1.2 Materials

As stated previously, hollow core slabs are produced with two basic types of concrete: low slump and normal slump. For the low-slump concretes, water content is limited to slightly more than that required for cement hydration. Water-cement ratios are typically about 0.30. Mixing of low-slump concrete is critical because the limited water available must be well dispersed. Water-reducing admixtures can be used to optimize a mixture by reducing water requirements while still maintaining adequate workability for proper compaction of the concrete by the machine. Air-entraining admixtures are not effective in the low-slump concrete. With the low water-cement ratios and compaction placing method, air is difficult to disperse well and maintain.

Wet-cast hollow core slabs (those cast with normal-slump concrete) have water-cement ratios in the range of 0.40 to 0.45. Depending on the slip-forming system used, a concrete slump of 2 in. to 5 in. is used. Producing concrete that will

Acoustical spray-on exposed slab ceiling.

consistently hold its shape with the forming technique used is dependent on mixture proportions and use of admixtures.

Aggregates vary in the manufacturing processes depending on what is locally available. Maximum aggregate size larger than pea gravel is rarely used because of the confined areas into which concrete must be placed. Lightweight aggregates are occasionally used to reduce the weight of the sections and to achieve a significant reduction in required equivalent thickness in a fire-rated application. Concrete densities ranging from 110 lb/ft^3 to 158 lb/ft^3 are used in the industry.

Almost every size and type of strand produced, depending on the availability to a particular producer, can be used in hollow core slabs. The industry is primarily using ½-in.-diameter, low-relaxation strand. The philosophy on strand use varies from using many strand sizes to optimize cost for a given project, to using only one or two strand sizes for simplicity of inventory and production.

Except for special situations, keyway grout is normally a mixture of sand and portland cement in proportions of approximately 3:1. The amount of water used is a function of the method used to place the grout, but will generally result in a wet mixture so keyways may be easily filled. Shrinkage cracks may occur in the keyways, but the configuration of the key is such that vertical load transfer can still occur with the presence of a shrinkage crack. Rarely is grout strength required in excess of 2000 psi for vertical load transfer.

In regions of the country subject to freezing temperatures, the guidelines of ACI 306, *Cold Weather Concreting*[2], should be followed. This report indicates that once concrete reaches a compressive strength of 500 psi, it will not be damaged by a single freezing cycle and will continue to cure to its design strength even with continued exposure to cold weather.

Although it is discouraged, non-shrink, non-staining grout is occasionally specified for use in keyways. The use of non-shrink grout can substantially increase the cost of a project because it must generally be mixed on site rather than delivered from a ready-mix plant, as can be done with standard 3:1 grout. In evaluating the potential benefits of non-shrink grout, the volume of grout must be compared with the overall volume of concrete in the slabs and support materials. Because the size of the keyway is small in relation to a floor or roof assembly of slabs, the total shrinkage of the grout will only affect the assembly to a minor degree. Shrinkage cracks can still occur in the keyways and there is little benefit to be gained by using non-shrink grout when compared with the additional cost.

1.3 Advantages of Hollow Core Slabs

Hollow core slabs are most widely known for providing economical, efficient floor and roof systems. Floor coverings can be installed on the top surface of a slab with proper preparation. The top surface can be prepared by feathering the joints between slabs with a latex cement; by installing a non-structural concrete topping ranging from $^1/_2$ in. to 2 in. thick (depending on the material used), or by casting a composite, structural concrete topping. The underside of the slab can serve as a finished ceiling as installed or paint or acoustical spray may be applied.

When properly coordinated for alignment, the voids in hollow core slabs may be used for electrical or mechanical runs. For example, routing a lighting circuit through the cores allows installation of fixtures in an exposed slab ceiling without an unsightly surface-mounted conduit. Slabs used as the thermal mass in a passive solar application can be detailed to distribute heated air through their cores.

Structurally, a hollow core slab provides the efficiency of a prestressed member for load capacity, span range, and deflection control. In addition, provided proper connections and details exist, the grouted slab assembly provides a basic diaphragm for resisting lateral loads. A detailed discussion of diaphragm capabilities is presented in Chapter 4.

Hollow core slabs also have excellent fire resistance. Depending on thickness and strand cover, fire ratings up to four hours can be achieved. The fire rating achieved depends on the equivalent thickness for heat transmission, concrete cover over the prestressing strands (for strength in a high temperature condition), and end restraint. Underwriters Laboratories publishes fire ratings for various structural assemblies. Many building codes also provide tables with empirical values for required equivalent slab thickness and concrete cover over the strand. In addition, most building codes allow a rational design procedure for strength in a fire. This procedure, described in detail in Chapter 7, considers the elevated strand temperature and the resulting loss of strength in the flexural capacity. Required fire ratings should be clearly specified in the contract documents, and the fire rating should be considered in the preliminary design when determining the slab thickness to be used.

Used as floor-ceiling assemblies, hollow core slabs have excellent sound-transmission characteristics. The sound transmission class rating ranges from about 47 to 57 (without topping) and the impact insulation class rating starts at about 23 for a plain slab, and may be increased to over 70 with the addition of carpeting and padding. Detailed information on the acoustical properties of hollow core slabs is presented in Chapter 8.

1.4 Framing Concepts

The primary consideration in developing a framing scheme using hollow core slabs is the span length. For a given loading and fire endurance rating, span length and slab thickness may be optimized by consulting a producer's published load tables. Section 1.6 presents sample load tables and instructions for the use of the tables. The *PCI Design Handbook* recommends limits on span-depth ratios for the hollow core slabs. For roof slabs, a span-depth ratio limit of 50 is suggested and for floor slabs, a limit of 40 is suggested. In practice, a span-depth ratio of 45 is common for floors and roofs when fire endurance, openings, or heavy or sustained live loads do not control a design.

The design professional must consider several factors that affect selection of hollow core slab thickness for a given span. Heavy superimposed loads, as required by the function of a system, would require a lower span-depth ratio. Similarly, heavy partitions or a large number of openings will result in greater load capacity requirements. The fire resistance rating required for the application will also affect the load capacity of a slab. As the required fire rating increases, prestressing strands can be raised for more protection from the heat and to satisfy the code-required concrete cover for the strand. The smaller effective strand depth will result in a lesser load capacity. Alternatively, if a rational design procedure is used, the strand may be raised to control the strand temperature, or the designer may choose to use the decreased strand strength due to elevated temperature in the design. Either of these approaches may control the slab design and result in a lesser load capacity.

Once the hollow core slab thicknesses and spans are selected, the economics of layout become important. While the ends of a slab can be designed to be cut at an angle, it is most efficient if square cut ends can be used (bearing perpendicular to the span).

It is also desirable to have the plan dimensions fit the hollow core slab module. This module depends on the slab systems available in the project area. Non-module plan dimensions can be accommodated using partial-width slabs. Some producers intentionally cast narrow widths as filler pieces, while others use a section split from a full width piece. Such a split section might be created by a longitudinal sawcut, or a break if the edge will not be exposed to view.

Construction tolerances must be accounted for in developing a plan layout. Tolerance on hollow core slab length may be taken up by allowing a gap at the ends in the bearing detail. On the non-bearing sides, clearance may be provided by using a detail where the slabs lap over a wall or beam. If the slab edge butts a wall or beam, a gap should be provided. Refer to local producers' information for recommendations of proper tolerances.

When a hollow core slab is exposed to weather for a long period of time during construction, water can accumulate in the cores. The primary source of water infiltration is at the butt joints. This fact, plus the camber of the slab, causes the water to pool at the ends of the slabs. In cold weather, this water can freeze and expand, causing localized damage. One remedy for this situation is to install weep holes at the slab ends under each core. The need for such weep holes is generally known only after a construction schedule is established. The specifier and the slab supplier are not usually in a position to know of such a need in advance.

Although some manufacturing methods may allow the holes to be cast into the hollow core slab, the most common means of creating weep holes is to drill into the cores through the bottom of the slab. Weep holes may be located in every core as a preventive measure or may be drilled in only when and where necessary after a heavy rain or snow melt. Careful observation of the underside of the deck will generally reveal water stains at cores with accumulated water. This can minimize the amount of overhead drilling required.

As with any other prestressed flexural member, hollow core slabs will be cambered. In the planning stages, consideration should be given to the causes of differential camber. For two slabs of identical length and prestressing, the camber may be different because of concrete and curing variations. This factor is independent of a framing scheme. However, joints between slabs of unequal spans or joints at which a change in the span direction occurs will cause a differential camber problem. This must be recognized and dealt with in the design layout. Wall locations may hide such a joint, but a door swing might be directed to the least variable side.

Camber must also be accommodated when a topping is to be provided. The quantity of topping required must consider the amount of camber and the function of the floor. In occupancies where flat floors are not a requirement, a constant topping thickness may be used to follow the curvature of the hollow core slabs. At the other extreme, if a flat floor is required in a structure consisting of multiple bays of varying length and changes in slab direction, the highest point will determine the top elevation of the topping. A greater amount of topping will then be required in low areas. These items must be considered in the project planning stages to control costs and minimize questions.

Camber, camber growth, and deflections must be considered when hollow core slabs run parallel to a stiff vertical element such as a wall (for example, slabs running parallel to the front wall of an elevator). The door rough opening should allow for camber to produce proper door installation. Alternatively, the slab span might be rearranged so the front wall is a bearing wall. This would alleviate door problems.

Camber, camber growth, and deflections must be taken into account in roofing details. Where changes in relative hollow core slab position can occur, counterflashings are suggested to accommodate such changes.

1.5 Design Responsibilities

It is customary in the hollow core slab industry for the precast concrete producer to perform the final engineering for the product to be supplied to the job. This includes design for vertical loads and lateral loads specified by the engineer of record, embedded items for specified connection forces, and handling and shipping. However, the engineer of record plays an important role in the design process. Prior to selection of the hollow core slab producer, enough preliminary planning should be done to insure that the specified floor and roof system is achievable. That is, the project should be one that can be engineered without requiring changes from the contract documents.

The contract documents must clearly indicate design criteria to which the hollow core slabs must conform. This is especially important when the slabs must interface with other construction materials. When connections are required, the forces to be transmitted through the connections must be specified in the contract documents. The precast concrete producer is best able to determine the most efficient connection element to be embedded in the slab. However, the balance of a connection that interfaces with another material should be detailed in the contract documents.

The engineer of record also has a responsibility in the review and approval of erection drawings prepared by the precast concrete producer. Review of these drawings is the last opportunity to ensure that the precast concrete producer's understanding of the project coincides with the intent of the design. Erection drawings should be checked for proper design loads, proper details and bearing conditions, conformance with specified fire ratings, and the location of openings.

1.6 Cross Sections and Load Tables

Each of the major hollow core slab systems has a standard set of cross sections that can be produced by its equipment. Available in thicknesses ranging from 4 in. to 20 in., core configurations make each system unique. Each precast concrete producer has additional production practices that may affect the capabilities of its product. Therefore, most precast concrete producers prepare and distribute load tables in their market area.

Precast concrete producer load tables define the allowable live load that a given hollow core slab can safely support in addition to the slab self-weight. The load capacity will be a function of the slab thickness, the amount of prestressing provided, and the location of the prestressing strands. Fire-rated slabs may require additional concrete cover below the strands, which will affect the load capacity.

The design criteria used to develop these load tables are defined by ACI 318 *Building Code Requirements for Structural Concrete and Commentary*[3] as outlined in chapter 2. Depending on the design criteria controlling a hollow core slab's load capacity, some advantage may be gained by understanding that in most applications, superimposed loads will consist of both dead and live loads. Where ultimate strength controls, an equivalent live load can be used to enter a load table. It is calculated as:

$$w_{equivalent} = \frac{1.2}{1.6} D_S + L$$

However, if bottom fiber tensile stresses control, no adjustment in superimposed loads may be used.

Similarly, many loading conditions consist of loads other than uniform loads. For preliminary design only, an equivalent uniform load may be calculated from the maximum moment caused by the actual loads.

$$w_{equivalent} = \frac{8\,M_{superimposed}}{\ell^2}$$

Shear will not be properly addressed in this situation. Thus, the final design must consider the actual load pattern.

Because of the uniqueness of each hollow core slab system and the many possibilities of strand patterns available from various producers, a generic hollow core slab has been developed to demonstrate design procedures. Figure 1.6.1 depicts the slab section and properties and illustrates a typical form for a producer's load tables. Throughout this manual, this section will be used to demonstrate various calculation procedures where any one of the proprietary cross sections could be substituted. *It must be emphasized that this cross section is not available for use and should not be specified.*

Figures 1.6.2 through 1.6.8 show the proprietary hollow core slab cross sections that are currently available. The section properties are as provided by the manufacturers, but weights are based on 150 lb/ft^3 concrete. The actual weights may vary slightly from those given. The availability of any particular section in a given area must be verified with the local producers. Figure 1.6.9 (a through e) presents charts of the general range of load capacities available in a given hollow core slab thickness. As with any chart of this nature, the chart should be approached carefully and verified with local producer load tables, especially for the longest, shortest, lightest, and heaviest conditions. Special care is also required when fire-rated hollow core slabs must be used on a project (see Chapter 7).

The following examples demonstrate the ways in which load tables may be used.

Example 1.6.1 Equivalent Uniform Load

From the load table in Fig. 1.6.1 select a strand pattern to carry a uniform superimposed dead load of 20 lb/ft^{2}and a uniform live load of 75 lb/ft^2 on a 24 ft span.

$$w = D_S + L$$
$$= 20 + 75 = 95 \text{ lb/ft}^2$$

Four $^7/_{16}$-in.-diameter strands required.

capacity = 132 lb/ft^2 → flexural strength controls

Because flexural strength controls, the analysis may be refined.

$$w_{equivalent} = \frac{1.2}{1.6}\ D_S + L$$

$$w_{equivalent} = \frac{1.2}{1.6}(20) + 75 = 90 \text{ lb/ft}^2$$

Use four $^3/_8$-in.-diameter strands.

capacity = 90 lb/ft^2 → flexural strength controls

As indicated, the load tables are intended primarily for preliminary design. Realistically, the presence of plumbing and mechanical openings may require the use of the more heavily stranded series. The designer should be especially careful when the preliminary analysis indicates that the highest stranded series is required.

Example 1.6.2 Non-uniform Loads

From the load table in Fig. 1.6.1, select a strand pattern to carry a superimposed uniform dead load of 20 lb/ft^2 plus live load of 40 lb/ft^2 and a continuous wall load of 600 lb/ft located perpendicular to the span and at midspan. The design span is 25 ft.

For preliminary design:

$$M_{superimposed} = \frac{25^2}{8}(20+40) + \frac{25}{4}(600)$$
$$= 8438 \text{ lb-ft/ft}$$

$$w_{equivalent} = \frac{8\,M_{superimposed}}{\ell^2}$$

$$w_{equivalent} = \frac{8(8438)}{25^2}$$
$$= 108 \text{ lb/ft}^2$$

Use four $^7/_{16}$-in.-diameter strands.

capacity = 118 lb/ft^2

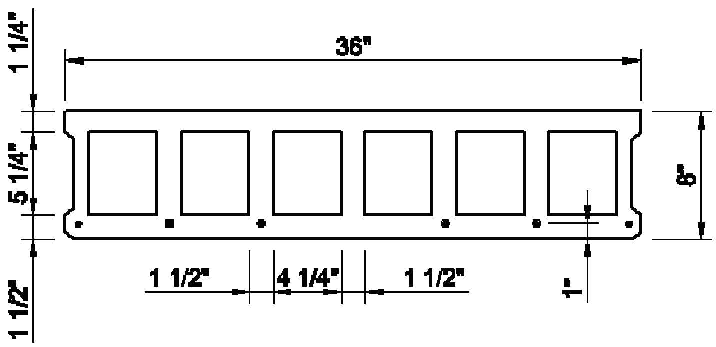
For final design, the methods found in chapter 2 must be used, particularly to check shear.

1.7 Tolerances

Figure 1.7.1 shows the dimensional tolerances for precast concrete hollow core slabs. These tolerances are guidelines only and each project must be considered individually to ensure that the tolerances shown are applicable.

Figure 1.7.2 shows erection tolerances for hollow core slabs. When establishing tolerances, the function of the slabs should be considered. For example, slabs covered by finish materials may not need the close tolerances required for those that are exposed.

Figure 1.6.1 Generic hollow core slab



Section Properties

A = 154 in.^{2}
I = 1224.5 in.^{4}
b_w = 10.5 in.
y_b = 3.89 in.
S_b = 314.8 in.^{3}
S_t = 297.9 in.^{3}
wt = 53.5 lb/ft^2

Sample Load Table*

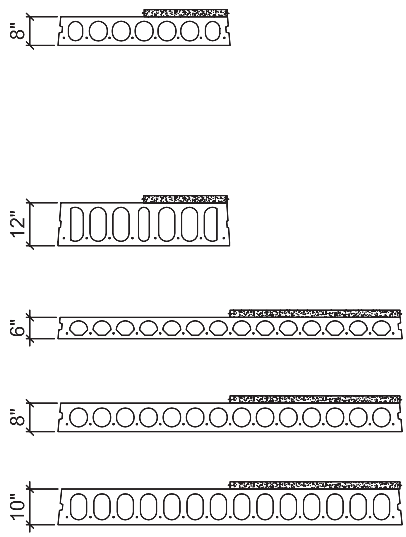
Allowable Superimposed Live Load, lb/ft^2

Strands, 270LR	ϕM_n, kip-ft	Span, ft									
		14	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**
4 - $^3/_8$ in.	45.1	343	294	253	220	192	168	148	130	115	102
6 - $^3/_8$ in.	65.4			386	337	296	262	232	207	185	166
4 - $^7/_{16}$ in.	59.4			347	302	265	234	207	184	164	147
6 - $^7/_{16}$ in.	85.0					341†	309†	283†	257†	237†	221†
4 - $^1/_2$ in.	76.7					321†	293†	267†	246†	224	202
6 - $^1/_2$ in.	105.3							331†	303†	278†	256†
Strands, 270LR	**ϕM_n, kip-ft**	**24**	**25**	**26**	**27**	**28**	**29**	**30**			
4 - $^3/_8$ in.	45.1	90	80	71	63	56	49	43			
6 - $^3/_8$ in.	65.4	149	134	121	109	99	89	81			
4 - $^7/_{16}$ in.	59.4	132	118	106	96	86	78	70			
6 - $^7/_{16}$ in.	85.0	201†	187	169	154	141	128	117			
4 - $^1/_2$ in.	76.7	182	164	149	135	123	112	101‡			
6 - $^1/_2$ in.	105.3	236†	220†	203†	190†	176†	167†	154†			

*Table based on 5000 psi concrete with allowable tension = $6\sqrt{f'c}$. Unless noted, values are governed by ultimate moment design. Prestress loss = 13.5% (assumed). This hollow core slab is for illustrative purposes only. Do not specify this slab for a project.
† Values are governed by shear strength.
‡ Values are governed by allowable tension.
Note: LR = low relaxation.

Figure 1.6.2

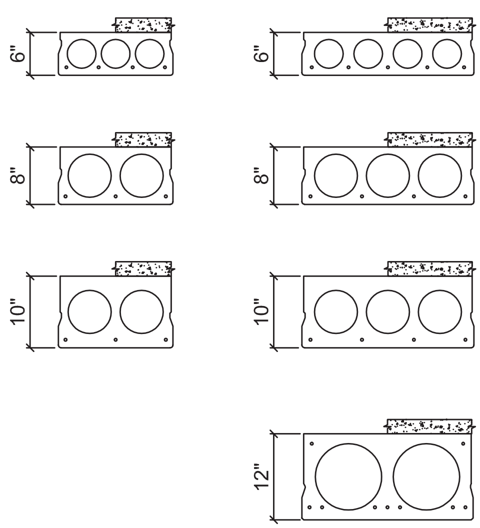
Trade name: Dynaspan®



Section		Untopped			with 2" topping		
depth x width	A in.^{2}	Y_b in.	I in.^{4}	w_t lb/ft^2	Y_{cb} in.	I_c in.^{4}	w_t lb/ft^2
4" x 4'-0"	133	2.00	235	35	3.08	689	60
6" x 4'-0"	165	3.02	706	43	4.25	1543	68
8" x 4'-0"	233	3.93	1731	61	5.16	3205	86
10" x 4'-0"	260	4.91	3145	68	6.26	5314	93
12" x 4'-0"	308	5.89	5215	80	7.58	8947	105
6" x 8'-0"	338	3.05	1445	44	4.26	3106	69
8" x 8'-0"	470	3.96	3525	61	5.17	6444	86
10" x 8'-0"	532	4.96	6422	69	6.28	10712	94
12" x 8'-0"	615	5.95	10505	80	7.32	16507	105

Figure 1.6.3

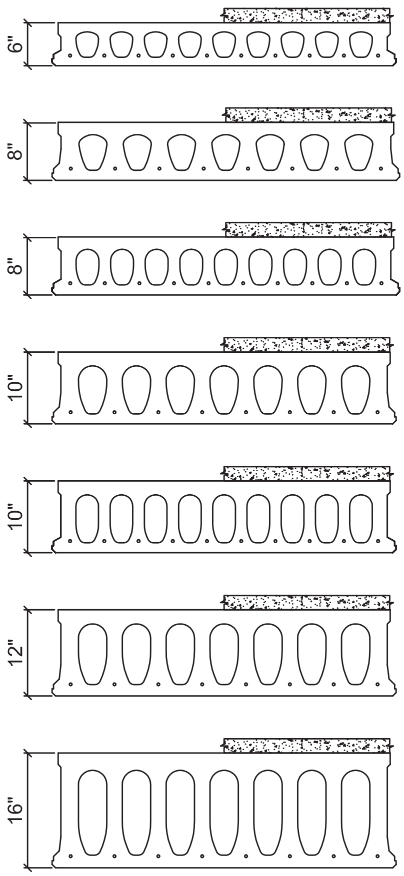
Trade name: Flexicore



Section		Untopped			with 2" topping		
depth x width	A in.^{2}	Y_b in.	I in.^{4}	w_t lb/ft^2	Y_{cb} in.	I_c in.^{4}	w_t lb/ft^2
6" x 1'-4"	55	3.00	243	43	4.23	523	68
6" x 2'-0"	86	3.00	366	45	4.20	793	70
8" x 1'-4"	73	4.00	560	57	5.26	1028	82
8" x 2'-0"	110	4.00	843	57	5.26	1547	82
10" x 1'-4"	105	5.00	1217	82	6.13	1936	107
10" x 2'-0"	157	5.00	1691	82	6.56	2604	107
12" x 2'-0"	141	6.00	2595	73	7.46	4049	98

Figure 1.6.4

Trade name: Echo®



Section		Untopped			with 2" topping		
depth x width	A in.^{2}	Y_b in.	I in.^{4}	w_t lb/ft^2	Y_{cb} in.	I_c in.^{4}	w_t lb/ft^2
6" x 4'-0"	187	3.09	757	47	4.19	2132	72
8" x 4'-0"	235	4.00	1761	59	5.19	3954	84
10" x 4'-0"	262	4.99	3196	66	6.31	6328	91
12" x 4'-0"	296	5.99	5192	75	7.38	9400	100
14" x 4'-0"	324	6.98	7764	82	8.46	13224	107
16" x 4'-0"	352	7.97	10986	89	9.18	15157	114
18" x 4'-0"	396	8.97	15139	100	10.54	23527	125
20" x 4'-0"	427	9.95	19661	107	11.57	30076	132
6" x 8'-0"	454	2.89	1629	57	3.90	4617	82
8" x 8'-0"	542	3.83	3657	68	4.93	8290	93
10" x 8'-0"	632	4.79	6769	80	5.96	13349	105
12" x 8'-0"	721	5.75	11149	91	6.98	20003	116

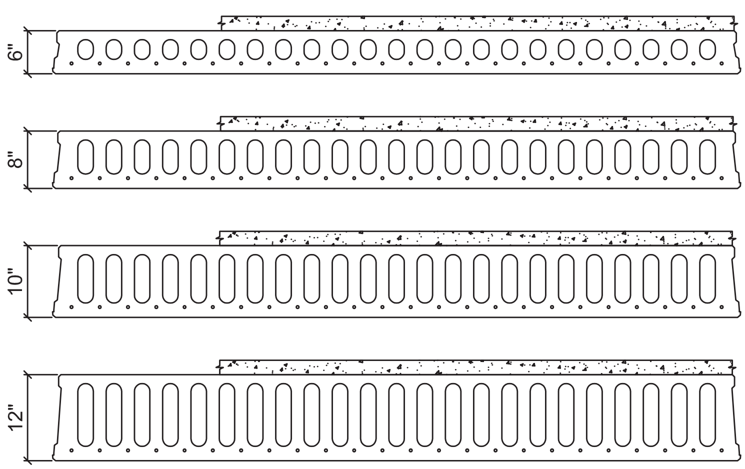
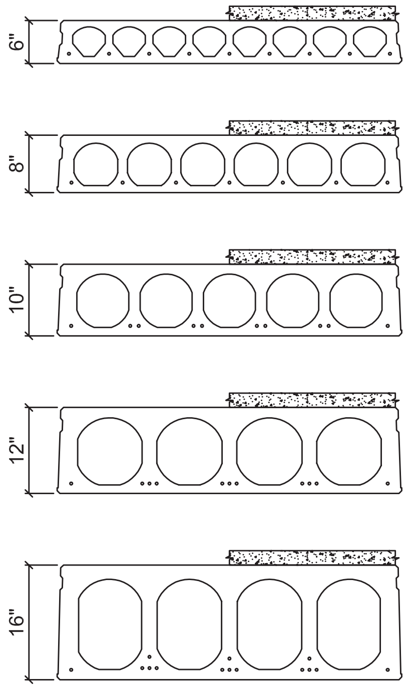


Figure 1.6.5

Trade name: Elematic®

Equipment manufacturer: Elematic Inc., Brookfield, Wisconsin



Section		Untopped				with 2" topping		
depth x width	A in.^{2}	Y_b in.	I in.^{4}	w_t lb/ft^2	Y_{cb} in.	I_c in.^{4}	w_t lb/ft^2
6" x 4'-0"	157	3.00	694	41	4.33	1557	66
8" x 4'-0"	196	3.97	1580	51	5.41	3024	76
10" x 4'-0" (5)	238	5.00	3042	62	6.49	5190	87
10" x 4'-0" (6)	249	5.00	3108	65	6.44	5280	90
12" x 4'-0" (4)	279	6.20	5104	74	7.90	8406	99
12" x 4'-0" (5)	274	6.00	5121	71	7.56	8134	96
16" x 4'-0"	346	8.30	11339	91	10.2	16883	116
20" x 4'-0"	501	10.3	24087	133	12.0	33073	158
H8" x 4'-0"	230	4.07	1667	60	5.34	3143	85
H12" x 4'-0"	307	6.00	5246	80	7.45	8393	105
8" x 8'-0"	404	4.00	3219	54	5.60	6475	79
10" x 8'-0"	549	5.00	6642	73	6.50	11827	98
12" x 8'-0"	620	6.10	10588	82	7.60	17915	107

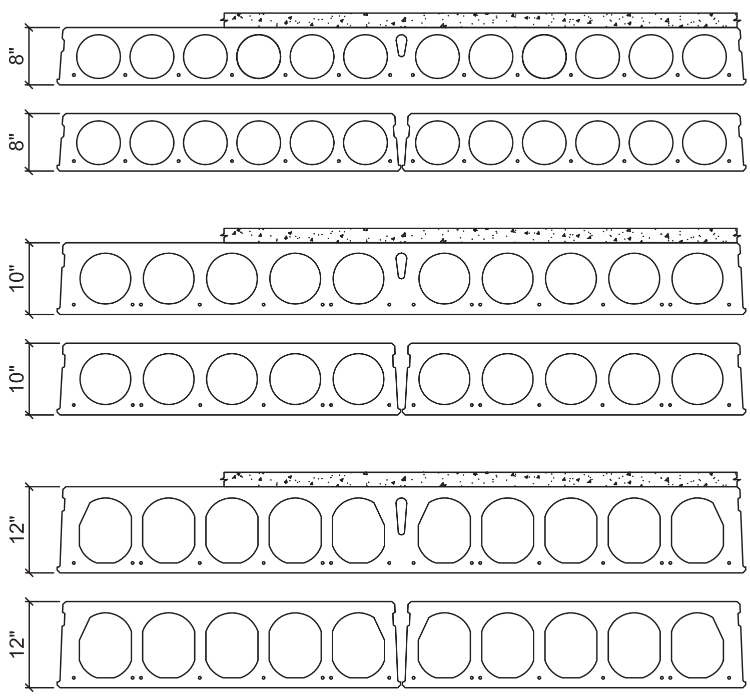
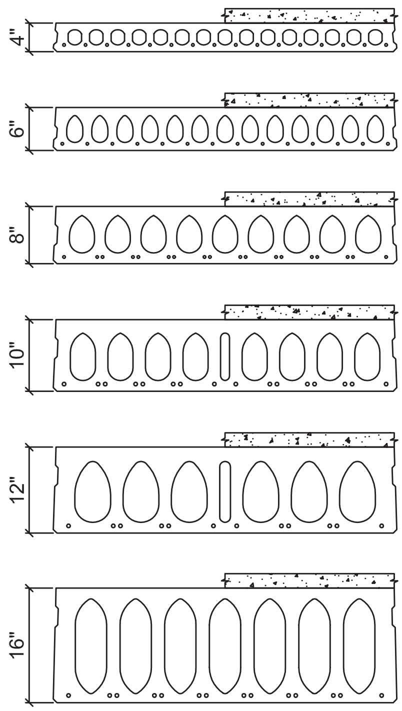


Figure 1.6.6a

Trade names: Spancrete®, Ultralight Spancrete®

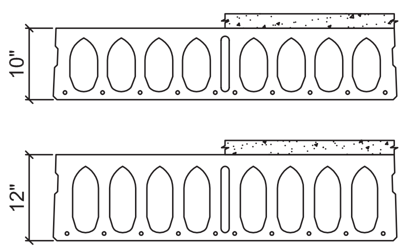
Licensing organization: Spancrete Machinery Corp., Waukesha, Wisconsin

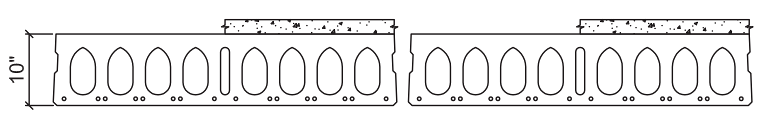
Standard Spancrete®



Section	Untopped				with 2" topping		
depth x width	A in.^{2}	Y_b in.	I in.^{4}	w_t lb/ft^2	Y_{cb} in.	I_c in.^{4}	w_t lb/ft^2
4" x 4'-0"	138	2.00	238	34	3.14	739	59
6" x 4'-0"	197	2.98	775	48	4.19	1766	73
8" x 4'-0"	258	3.98	1806	63	5.22	3443	88
10" x 4'-0"	312	5.16	3484	76	6.41	5787	101
12" x 4'-0"	355	6.28	5784	86	7.58	8904	111
15" x 4'-0"	417	7.45	10792	101	8.89	15774	126
16" x 4'-0"	401	8.14	12050	97	9.69	17575	122
U8" x 4'-0"	251	4.01	1817	61	5.27	3425	86
U10" x 4'-0"	277	5.22	3178	67	6.58	5376	92
U12" x 4'-0"	330	6.28	5442	80	7.65	8518	105

Ultralight Spancrete®





Note: All Spancrete® hollow core sections can be produced two at a time on Spancrete's 96" machine.

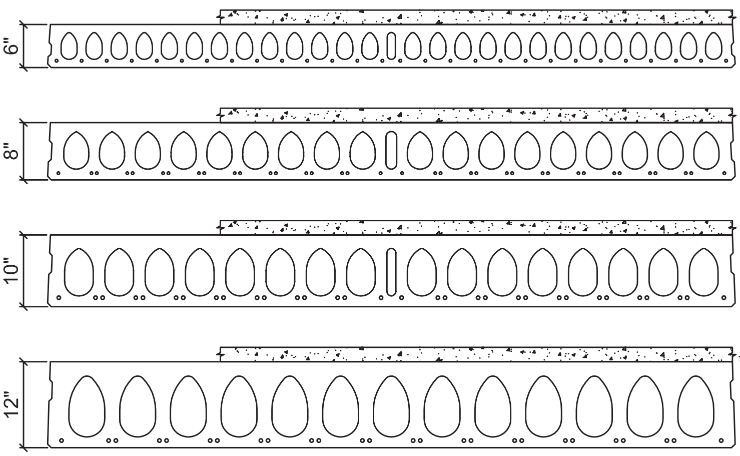
Figure 1.6.6b

Trade names: Spancrete®, Ultralight Spancrete®

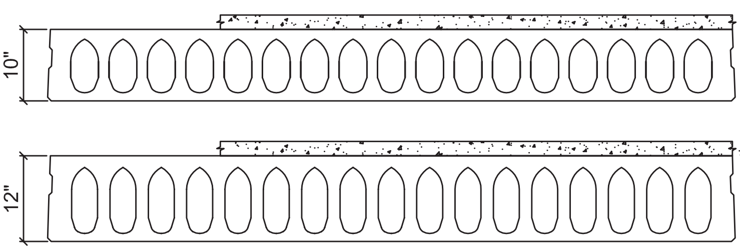
Licensing organization: Spancrete Machinery Corp., Waukesha, Wisconsin

Section		Untopped			with 2" topping		
depth x width	A in.^{2}	Y_b in.	I in.^{4}	w_t lb/ft^2	Y_{cb} in.	I_c in.^{4}	w_t lb/ft^2
6" x 8'-0"	447	2.96	1641	48	4.07	3705	73
8" x 8'-0"	511	4.01	3625	63	5.25	6857	88
10" x 8'-0"	602	5.21	6924	73	6.48	11418	98
12" x 8'-0"	706	6.31	11599	85	7.61	17778	110
U8" x 8'-0"	501	4.02	3641	61	5.28	6841	86
U10" x 8'-0"	567	5.10	6553	70	6.46	11610	95
U12" x 8'-0"	661	6.30	10915	80	7.67	17032	105

Standard Spancrete®



Ultralight Spancrete®



Note: Spancrete® hollow core is available as an Insulated or Non-insulated Wall Panel

Spancrete® Wall Panel

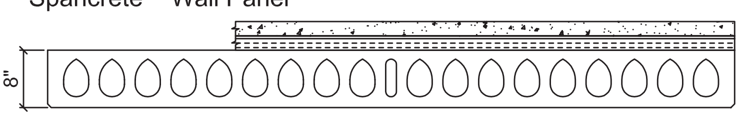
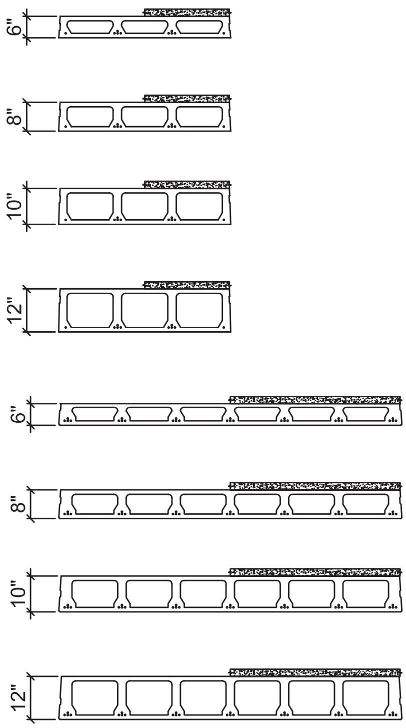


Figure 1.6.7

Trade name: SpanDeck

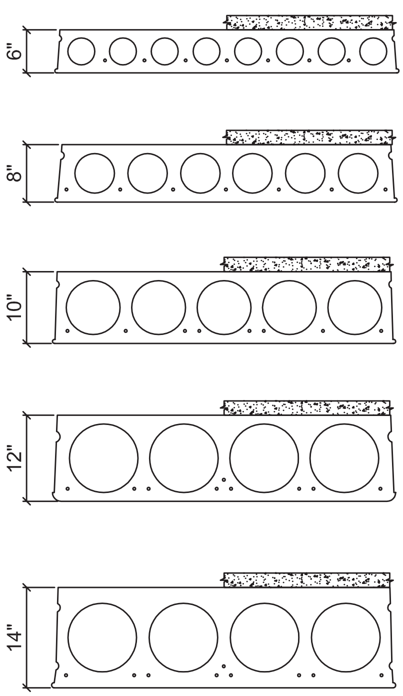


Section		Untopped			with 2" topping		
depth x width	A in.^{2}	Y_b in.	I in.^{4}	w_t lb/ft^2	Y_{cb} in.	I_c in.^{4}	w_t lb/ft^2
6" x 4'-0"	210	2.82	727	55	4.02	1755	80
8" x 4'-0"	246	3.75	1615	62	5.55	2791	87
10" x 4'-0"	261	4.81	2662	68	6.35	4993	93
12" x 4'-0"	298	5.87	5452	75	8.01	7856	100
6" x 8'-0"	407	2.81	1453	53	4.00	3529	78
8" x 8'-0"	477	3.73	3236	60	5.53	5643	85
10" x 8'-0"	507	4.82	5324	66	6.31	10036	91
12" x 8'-0"	578	5.86	10909	72	7.98	15709	97

Figure 1.6.8

Trade name: Ultra–Span

Licensing organization: Ultra–Span Technologies Inc., Winnipeg, Manitoba, Canada



Section		Untopped			with 2" topping		
depth x width	A in.^{2}	Y_b in.	I in.^{4}	w_t lb/ft^2	Y_{cb} in.	I_c in.^{4}	w_t lb/ft^2
4" x 4'-0"	154	2.00	247	40	2.98	723	65
6" x 4'-0"	188	3.00	764	49	4.13	1641	74
8" x 4'-0"	214	4.00	1666	56	5.29	3070	81
10" x 4'-0"	259	5.00	3223	67	6.34	5328	92
12" x 4'-0"	289	6.00	5272	75	7.43	8195	100
14" x 4'-0"	378	7.00	9185	98	8.22	12891	123
H8" x 4'-0"	241	4.00	1779	62	5.10	3126	87

Figure 1.6.9(a) Slab load ranges 6 in. Hollow core

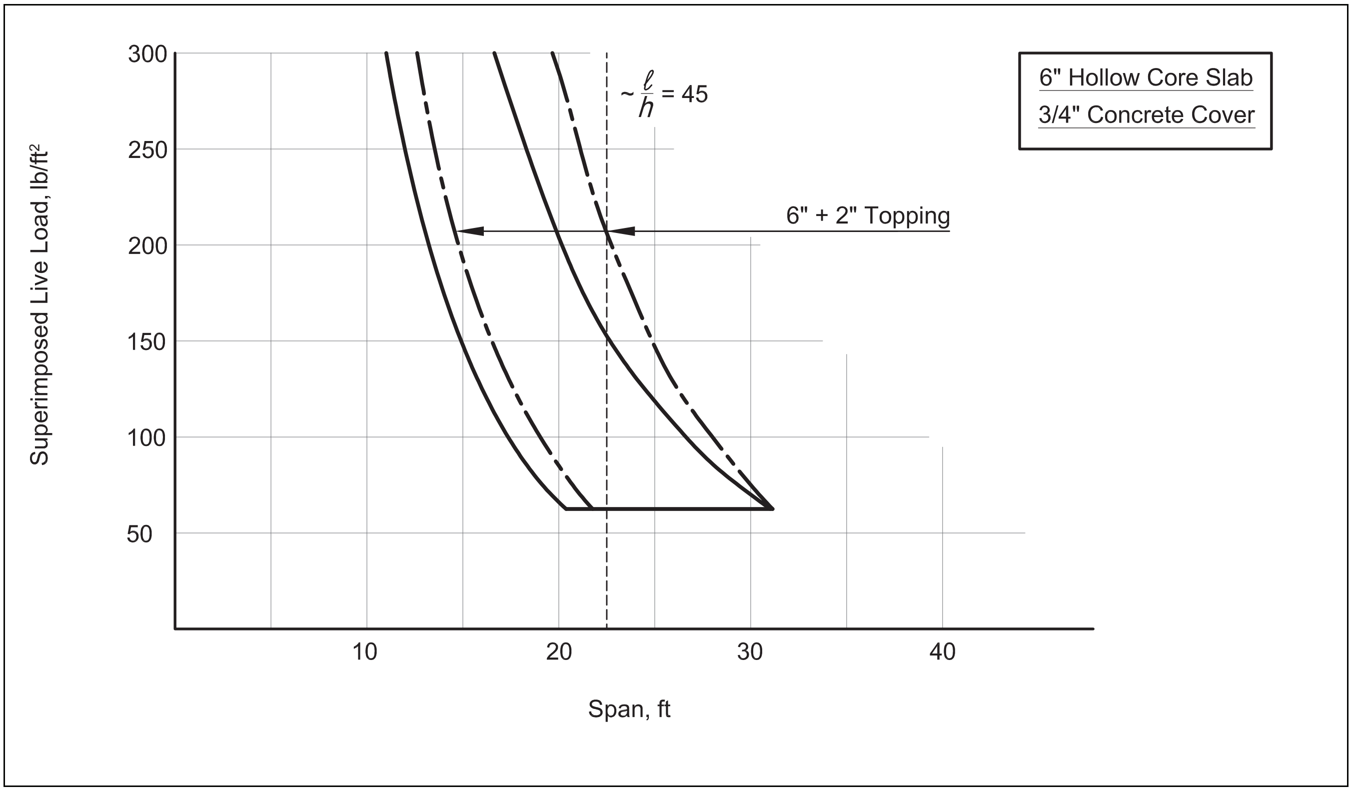


Figure 1.6.9(b) Slab load ranges 8 in. Hollow core

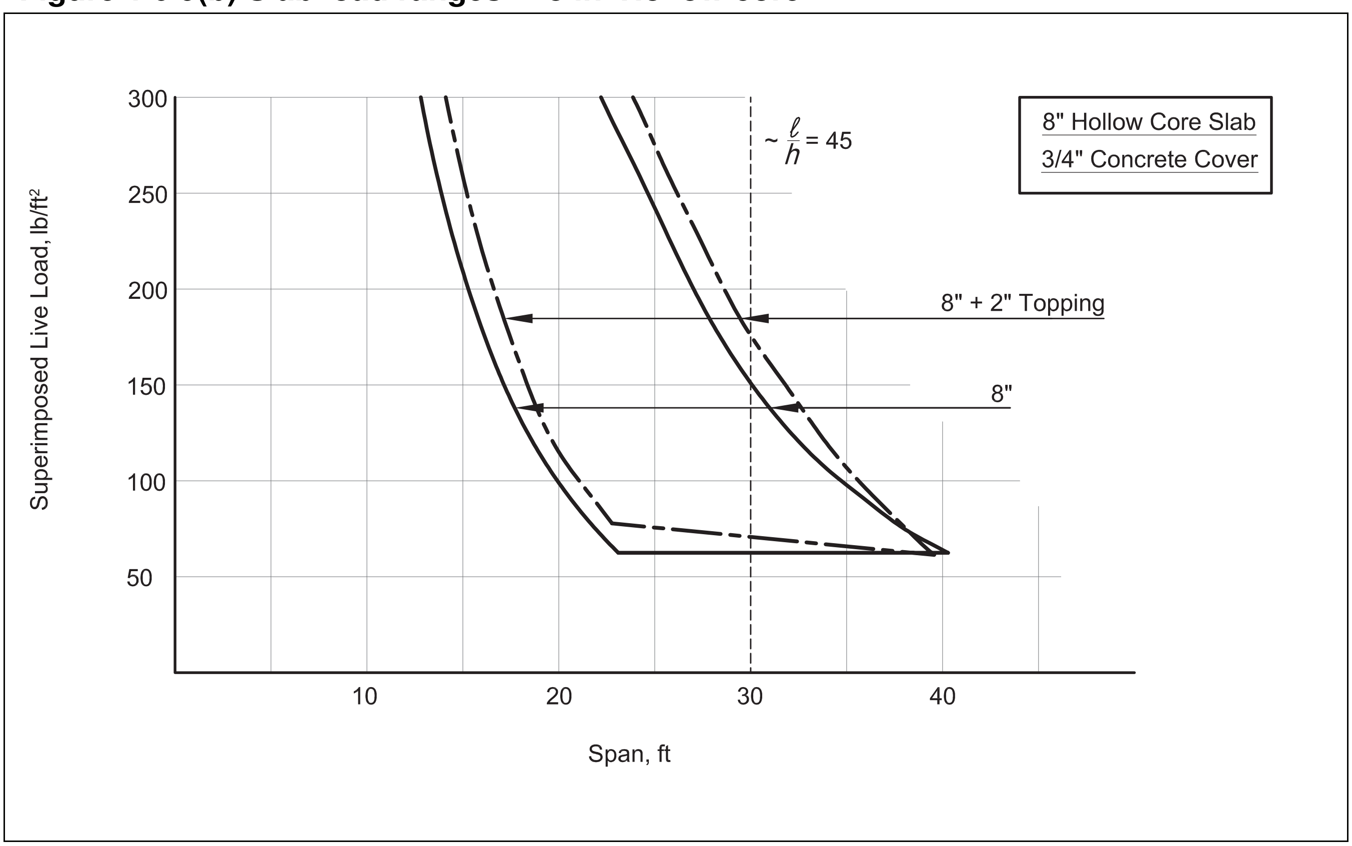


Figure 1.6.9(c) Slab load ranges 10 in. Hollow core

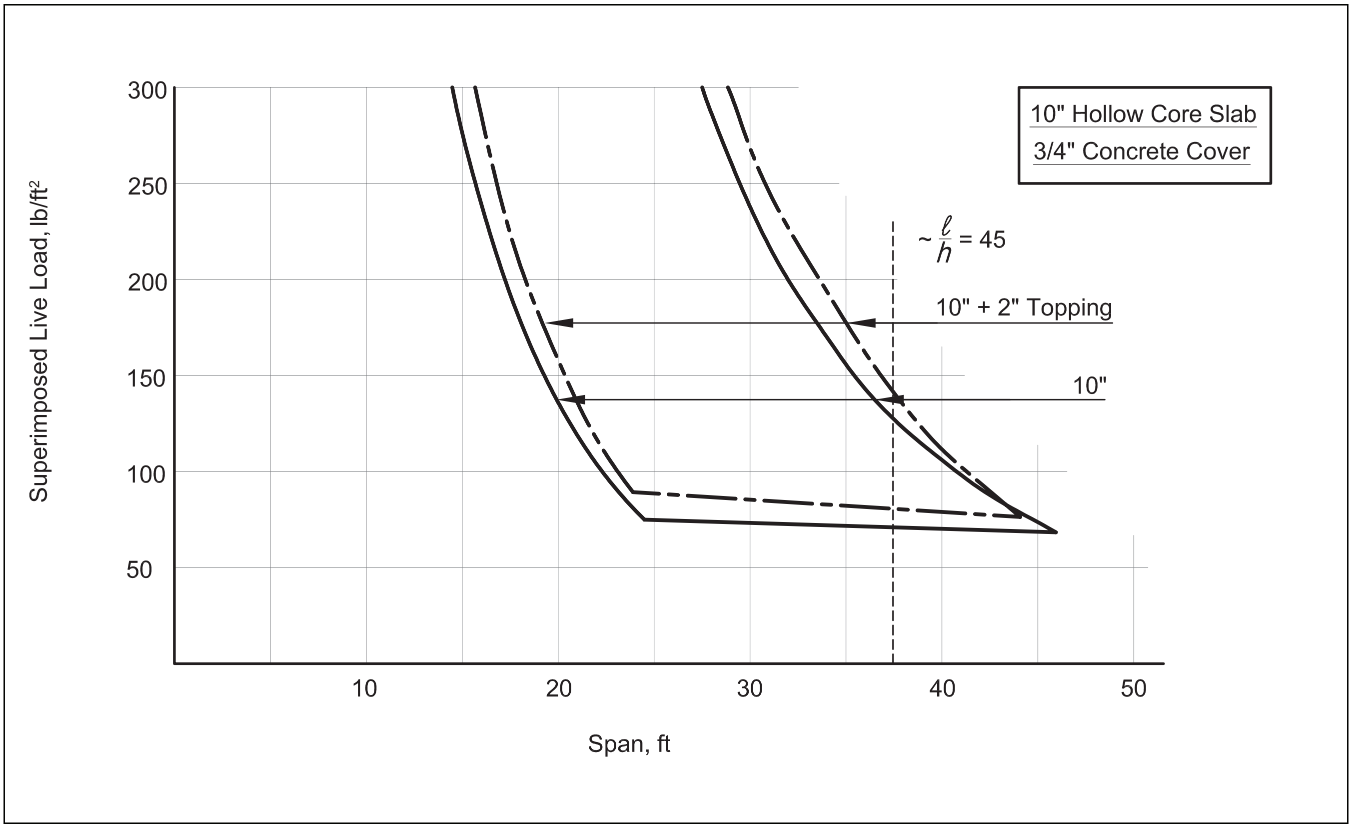


Figure 1.6.9(d) Slab load ranges 12 in. Hollow core

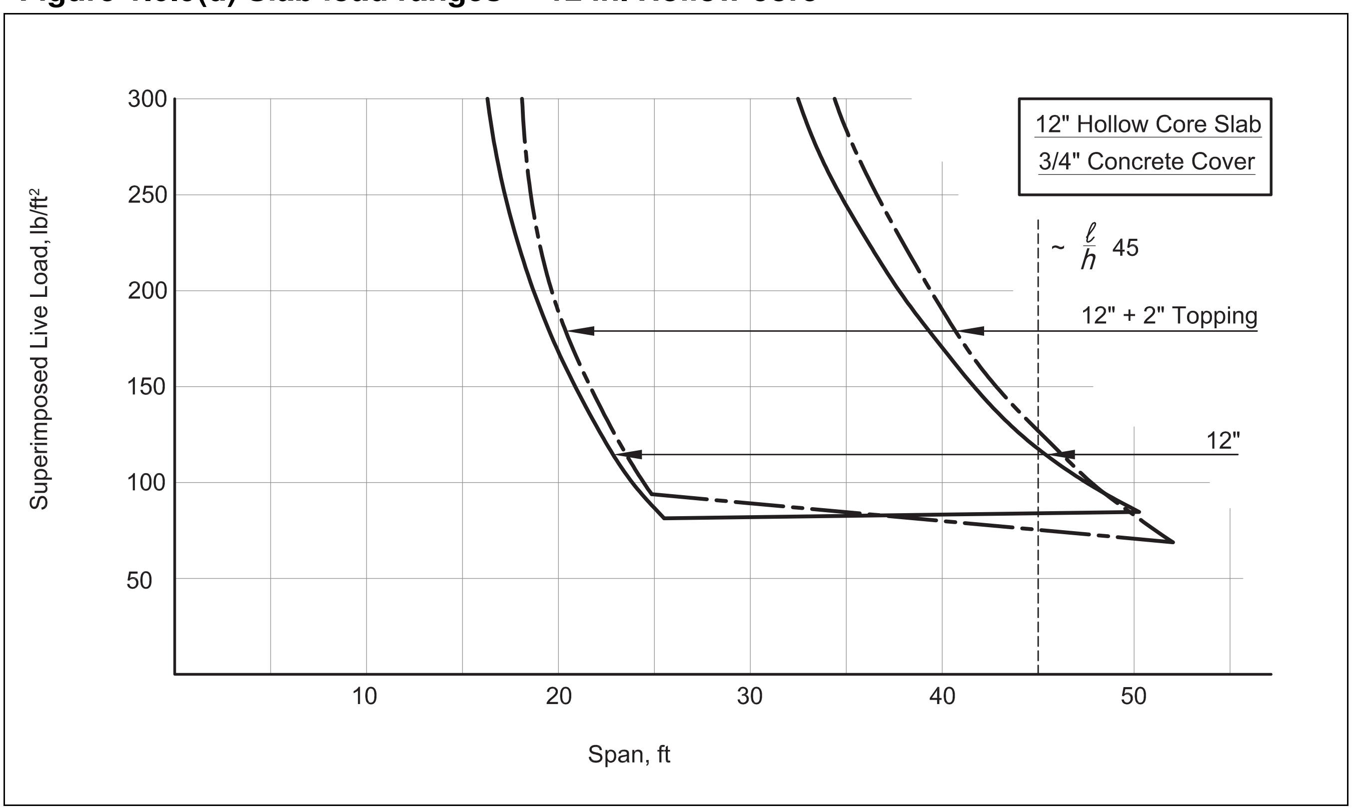
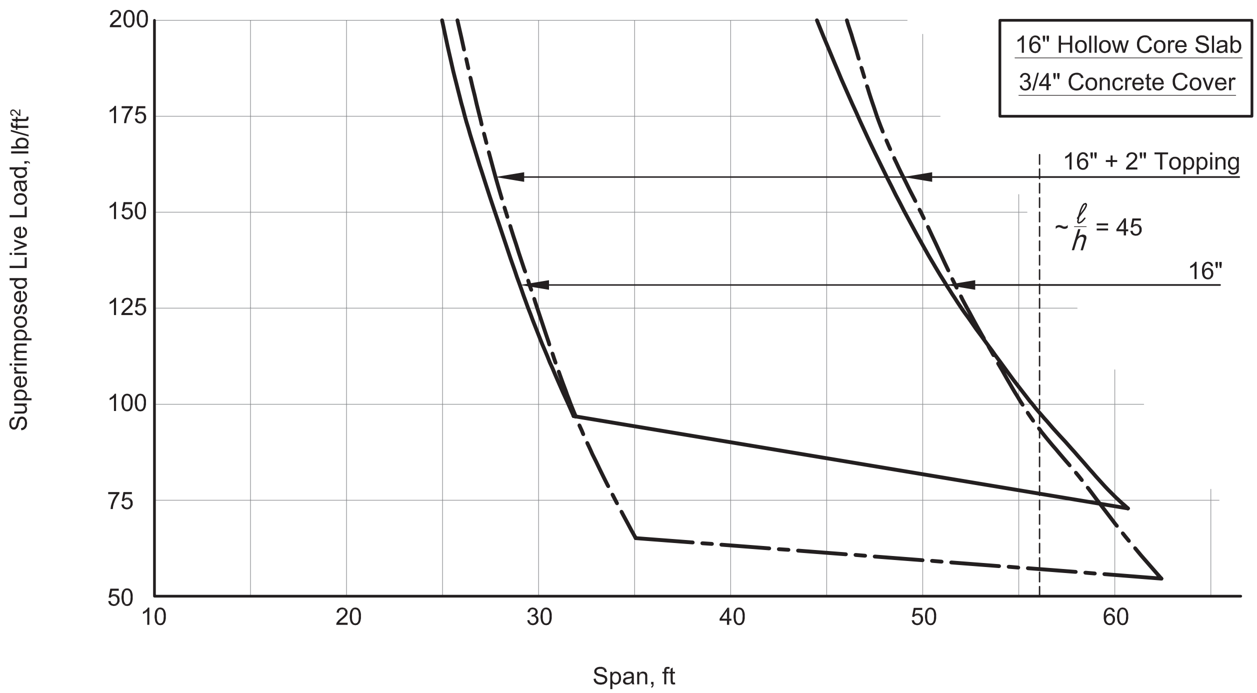


Figure 1.6.9(e) Slab load ranges 16 in. Hollow core



Note: The above data are based on flexural strength. Additional measures may be required for shear.

Figure 1.7.1 Product tolerances for hollow core slabs[4]

a = Length .... ± ½ in.
b = Width .... ± ¼ in.
c = Depth .... ± ¼ in.
d_t = Top flange thickness
Top flange area defined by the actual measured values of average d_t x *b* shall not be less than 85% of the nominal area calculated by d_t nominal x *b* nominal.
d_b = Bottom flange thickness
Bottom flange area defined by the actual measured values of average d_b x *b* shall not be less than 85% of the nominal area calculated by d_b nominal x *b* nominal.
e = Web thickness
The total cumulative web thickness defined by the actual measured value ∑*e* shall not be less than 85% of the nominal cumulative width calculated by ∑e nominal.
f = Blockout location... ± 2 in.
g = Flange angle .... ⅛ in. per 12 in., ½ in. max.
h = Variation from specified end squareness or skew .... ± ½ in.
i = Sweep (variation from straight line parallel to centerline of member) .... ± ⅜ in.
j = Center of gravity (CG) of strand group
The CG of the strand group relative to the top of the slab shall be within ± ¼ in. of the nominal strand group CG. The position of any individual strand shall be within ± ½ in. of nominal vertical position and shall have a minimum cover of ¾ in.
k = Position of plates.. ±2 in.
l = Tipping and flushness of plates .... ± ¼ in.
m = Local smoothness .... ± ¼ in. in 10 ft
(does not apply to top deck surface left rough to receive a topping or to visually concealed surfaces)
Slab weight
Excess concrete material in the slab's internal features is within tolerance as long as the measured weight of the individual slab does not exceed 110% of the nominal published slab weight used in the load capacity calculation.
n = Applications requiring close control of differential camber between adjacent members of the same design should be discussed in detail with the producer to determine applicable tolerances.

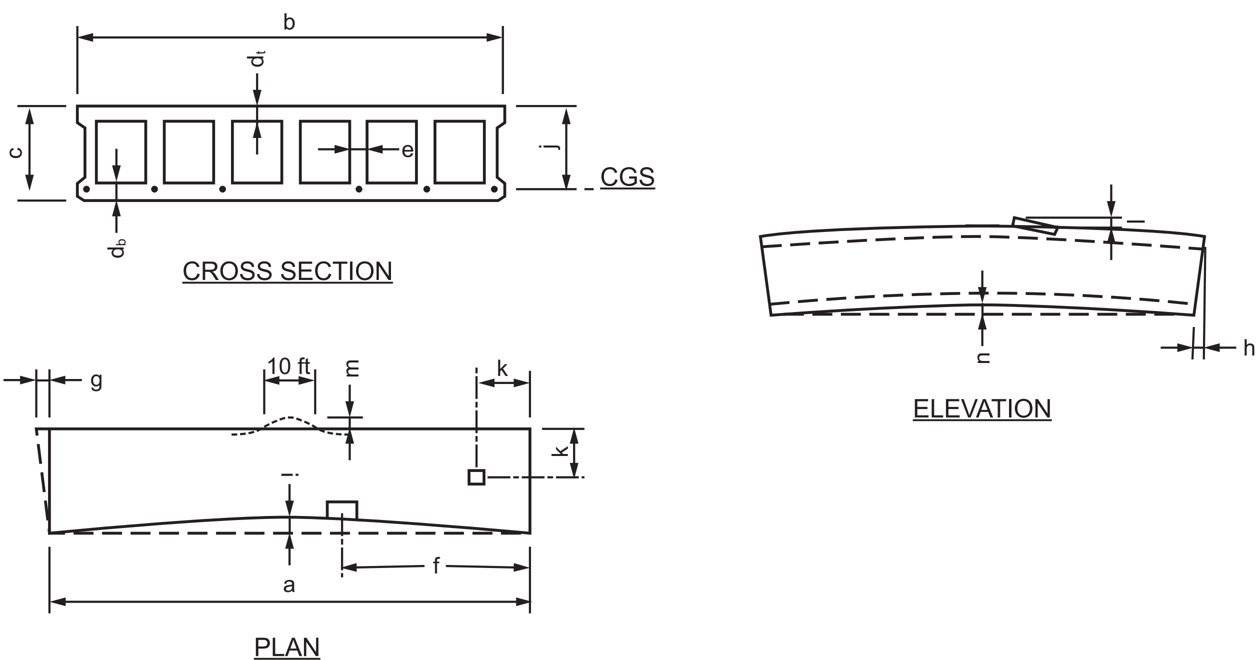
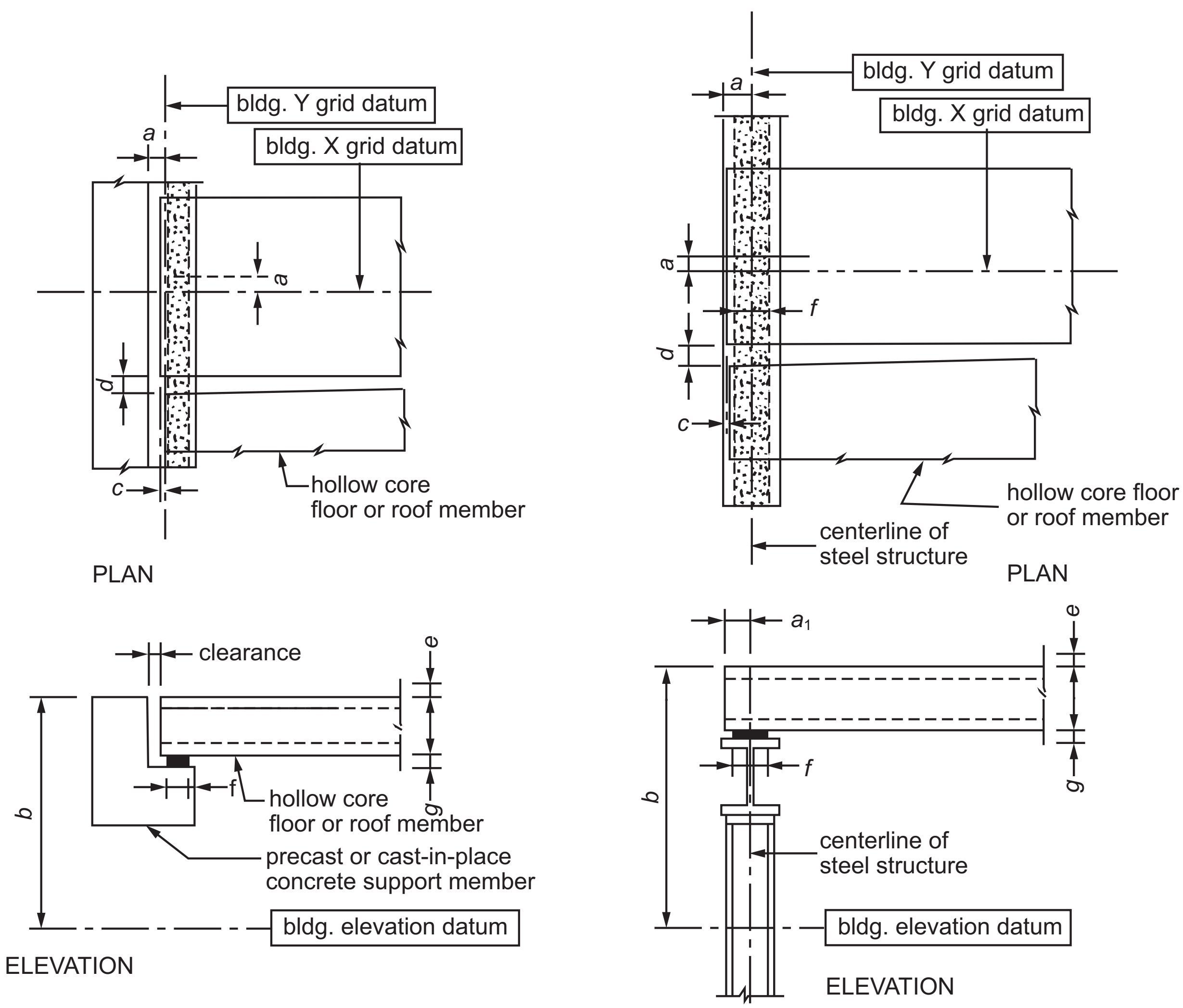


Figure 1.7.2 Erection tolerances for hollow core floor and roof members[4]

a =	Plan location from building datum	± 1 in.
a_1 =	Plan location from centerline of steel*	± 1 in.
b =	Top elevation from nominal top elevation at member ends	
	Covered with topping	± ¾ in.
	Untopped floor	± ¼ in.
	Untopped roof	± ¾ in.
c =	Maximum jog in alignment of matching edges (both topped and untopped construction)	1 in.
d =	Joint width	
	0 to 40 ft member length	± ½ in.
	41 to 60 ft member length	± ¾ in.
	61 ft plus	± 1 in.
e =	Differential top elevation as erected	
	Covered with topping	¾ in.
	Untopped floor	¼ in.
	Untopped roof†	¾ in.
f =	Bearing length‡ (span direction)	± ¾ in.
g =	Differential bottom elevation of exposed hollow core slabs§	¼ in.

* For precast concrete erected on a steel frame building, this tolerance takes precedence over tolerance on dimension *a*.
† It may be necessary to feather the edges to ± ¼ in. to properly apply some roof membranes.
‡ This is a setting tolerance and should not be confused with structural performance requirements set by the architect/engineer.
§ Untopped installation will require a larger tolerance here.



Precast element to precast or cast-in-place concrete or masonry

Precast element to structural steel

Chapter 2
DESIGN OF HOLLOW CORE SLABS

2.1 General

The design of hollow core slabs is in accordance with the American Concrete Institute (ACI) *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11)*[3]. As with prestressed concrete members in general, hollow core slabs are checked for prestressing force transfer stresses, handling stresses, service load stresses, deflections, and design (ultimate) strength in shear and bending. For uniform load cases, the manufacturer's load tables will take into account these various design considerations and list a load capacity based on the governing criteria. For conditions other than uniform loading, or for the development of load tables, the design steps presented in this section are used.

An excellent reference for prestressed member design exists in the *PCI Design Handbook: Precast and Prestressed Concrete*.[1] Charts and tables provide design aids to shorten the calculation procedures. Another excellent source for design information is the PCI Standard Design Practice, which reflects design practice in the precast, prestressed concrete industry.

The generic hollow core slab presented in Fig. 1.6.1 will be used for the calculations presented in this section. The cross section was selected to provide a means of demonstrating calculation procedures and does not represent any hollow core slabs currently in use. Therefore, this generic slab should never be specified for use on a project. Section 1.6 of this manual provides information regarding hollow core slabs currently available.

2.2 Flexural Design

2.2.1 ACI Requirements

Chapter 18 of ACI 318-11 presents provisions for the flexural design of prestressed concrete members. Section 18.3.3 of that Code states that prestressed flexural members shall be classified as Class U, Class T, or Class C based on f_t, the computed extreme fiber stress in tension in the precompressed tensile zone calculated at service loads, as follows:

Class U: $f_t \leq 7.5\sqrt{f_c'}$

Class T: $7.5\sqrt{f_c'} < f_t \leq 12\sqrt{f_c'}$

Class C: $f_t > 12\sqrt{f_c'}$

For Class U and Class T flexural members, stresses at service loads shall be permitted to be calculated using the uncracked section. For Class C flexural members, stresses at service loads shall be calculated using the cracked transformed section. The applicable requirements of ACI 318-11 are paraphrased as follows:

2.2.1.1 Permissible stresses at transfer (ACI 318-11, Section 18.4).

a) Extreme fiber stress in compression except as permitted in (b) is $0.6\,f_{ci}'$
b) Extreme fiber stress in compression at ends is $0.7\,f_{ci}'$
c) Extreme fiber stress in tension except as permitted in (d) is $3\sqrt{f_{ci}'}$
d) Extreme fiber stress in tension at ends of simply supported members is $6\sqrt{f_{ci}'}$

2.2.1.2 Permissible stresses at service loads (ACI 318-11, Section 18.4) for Class U and Class T members.

a) Extreme fiber stress in compression due to prestress plus sustained loads is $0.45\,f_c^{'}$

b) Extreme fiber stress in compression due to prestress plus total load is $0.60\,f_c^{'}$

2.2.1.3 Loss of prestress (ACI 318-11, Section 18.6)

Calculation of losses shall consider:

- seating loss;
- elastic shortening of concrete;
- creep of concrete;
- shrinkage of concrete; and
- steel relaxation.

2.2.1.4 Required strength

- Load combinations (ACI 318-11, Section 9.2).

If only gravity loads are considered, the applicable load combinations are:

$U = 1.4D$
$U = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 1.0L$

- Strength reduction factors (ACI 318-11, Section 9.3)

For flexure, the strength reduction factor ϕ is 0.9, assuming the sections are tension-controlled, which means the net tensile strain is greater than or equal to 0.005. Net tensile strain is the strain in the extreme tension reinforcement at nominal strength (that is, the strength corresponding to a strain value of 0.003 at the extreme compression fiber of concrete), exclusive of strains due to effective prestress, creep, shrinkage, and temperature.

- Flexural strength (ACI 318-11, Section 18.7)

To check for flexural strength, the following calculations are required:

$$M_u \leq \quad \phi M_n = \phi A_{ps} f_{ps} (d_p - \frac{a}{2})$$

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

Note: If a exceeds the top flange thickness, the compression block will encroach on the core area. For this situation, multiple compression forces are used for the internal couple, as is done with other flanged members.

$$f_{ps} \quad = \quad f_{pu}\left(1 - \frac{\gamma_p}{\beta_1}\rho_p \frac{f_{pu}}{f_c^{'}}\right)$$

Note: f_{ps} can be calculated using the strain compatibility and equilibrium method in lieu of the simplified equation.

2.2.1.5 Minimum reinforcement (ACI 318-11 Section 18.8.2)

Minimum reinforcement is defined in ACI 318-11, Section 18.8.2, as "Total amount of prestressed and non-prestressed reinforcement in members with bonded prestressed reinforcement shall be adequate to develop a factored load at least 1.2 times the cracking load computed on the basis of the modulus of rupture specified in 9.5.2.3." There has been some misunderstanding of this provision. It is not meant to require special end-region reinforcement where M_{cr} is very high and M_n is very low; the intent is to prevent brittle fracture at first flexural cracking, not at every section where the theoretical cracking moment is greater than the nominal moment strength. At the critical section, a simplification may be used as:

$\phi M_n > 1.2\ M_{cr}$

This requirement may be waived when the shear and flexural strength is twice that required by design.

2.2.2 Stresses at Transfer

At transfer of prestress, only the self weight of the hollow core slab is present to counteract the effects of eccentric prestress. A check of stresses is required at this stage to determine the concrete strength required to preclude cracking on the tension side. The concrete strength at the time of transfer may be only 50% to 60% of the 28-day specified compressive strength, but should be at least 3000 psi.

Example 2.2.2.1 Transfer Stresses

Using the generic cross section of the hollow core slab defined in Fig. 1.6.1, check stresses at transfer of prestress using the following criteria:

Prestressing steel:

4–1/2-in.-diameter, 270 ksi, low-relaxation strands.

$A_{ps} = 4(0.153) = 0.612 \text{ in.}^2$

assume 5% initial loss

$d_p = 7$ in.

$\ell_{pc} = 30$ ft-6 in.

$f_{pi} = 0.70 f_{pu}$

Self weight = 53.5 lb/ft^2

Solution:

Stresses will be checked at the transfer point and at midspan.

Prestress force at release is:

$P_o = f_{pi} A_{ps}\,(1 - \text{loss})$

$P_o = (0.70)(0.95)(0.612)(270) = 109.9$ kip

$$\text{Prestress effect} = \frac{P_o}{A} \mp \frac{P_o e}{S}$$

$$= \frac{109.9}{154} \mp \frac{109.9(2.89)}{\begin{cases}297.9\\314.8\end{cases}}$$

= -0.353 ksi top fiber

= +1.723 ksi bottom fiber

Stress due to self-weight at transfer point:

$$M_d = \left(\frac{\ell_{pc}}{2}\ell_t - \frac{\ell_t^2}{2}\right) D_{sw} b$$

$\ell_t = 50 d_b = 50(1/2) = 25$ in.

Moment at 25 in. from slab end

$$M_d = \left(\frac{30.5}{2}(2.08) - \frac{2.08^2}{2}\right)(0.0535)(3\,\text{ft})$$

= 4.74 kip-ft

$$\frac{M_d}{S} = \frac{(4.74)(12)}{\begin{cases}297.9\\314.8\end{cases}}$$

= +0.191 ksi top fiber

= -0.181 ksi bottom fiber

Net concrete stress at transfer point

= -0.353 + 0.191 = -0.162 ksi top fiber

= +1.723 – 0.181 = +1.542 ksi bottom fiber

Stresses due to self weight at midspan:

$$M_d = \frac{\ell_{pc}^2}{8} D_{sw} b = \frac{30.5^2}{8}\ (0.0535)(3\text{ ft})$$

= 18.66 kip-ft

$$\frac{M_d}{S} = \frac{(18.66)(12)}{\begin{cases}297.9\\314.8\end{cases}}$$

= +0.752 ksi top fiber

= -0.711 ksi bottom fiber

Net concrete stress at midspan:

= -0.353 + 0.752 = + 0.399 ksi top fiber

= +1.723 – 0.711 = +1.012 ksi bottom fiber

Allowable stresses using f'_{ci} = 3000 psi:

Tension at end = $6\sqrt{f'_{ci}} = 6\sqrt{3000}$ = 329 psi > 162 psi

Tension at midspan = $3\sqrt{f'_{ci}} = 3\sqrt{3000}$

= 164 psi does not control

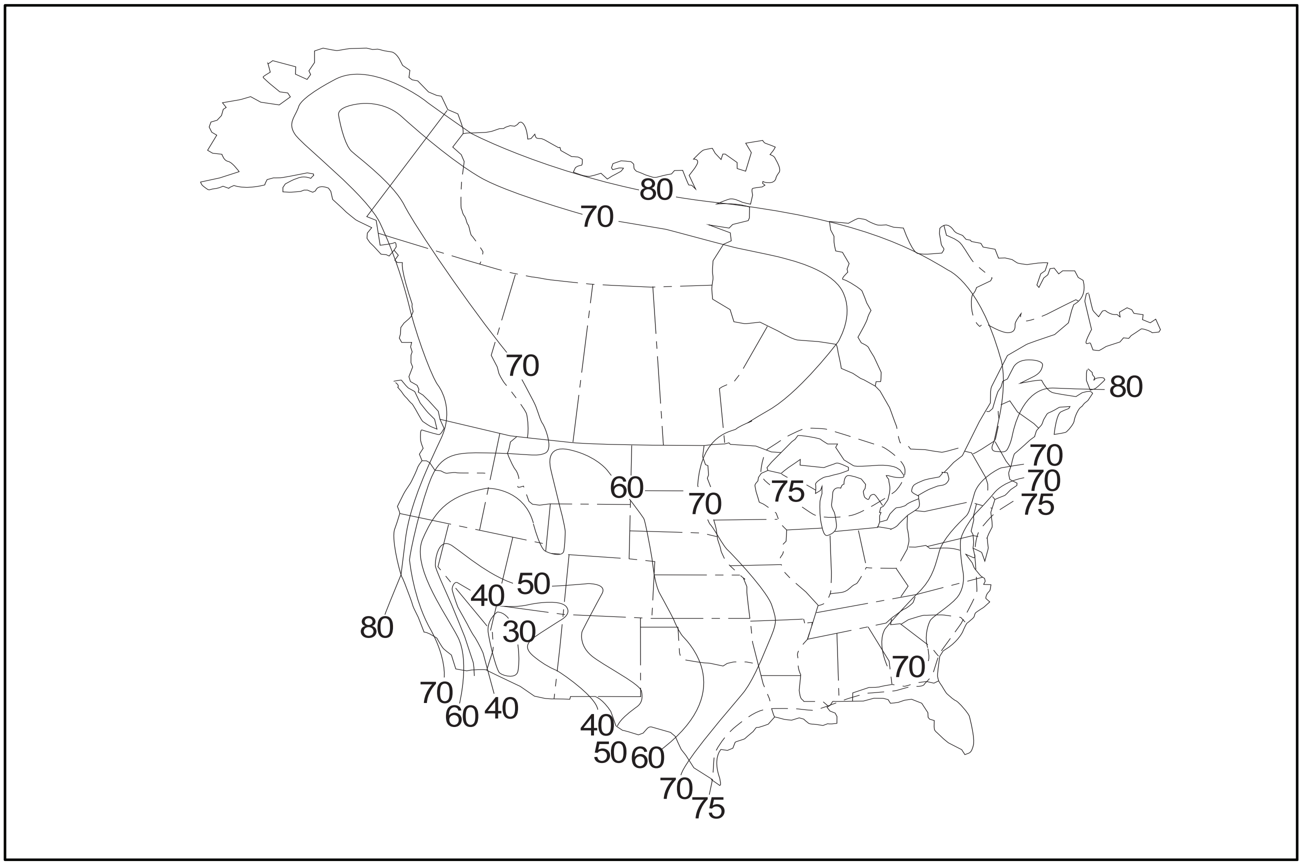
Compression at end = $0.7 f'_{ci}$ = 0.7 (3000)

= 2100 psi > 1542 psi

Compression at midspan = $0.6 f'_{ci}$ = 0.6(3000) = 1800 psi > 1012 psi

If the tension or compression in the end region exceeds the allowable values based on reasonable concrete release strength, strands may be debonded in some manufacturing systems. If the allowable tension is exceeded, top mild steel reinforcement may be used in some manufacturing systems to resist the total tension force.

Figure 2.2.3.1 Ambient relative humidity



If tension in the midspan region controls, either a high release strength must be used or mild steel reinforcement must be added to resist the total tension force. Nonprestressed reinforcement can be added in wetcast manufacturing systems or placed in concrete filled cores in drycast systems.

2.2.3 Prestress Losses

When calculating prestress losses, the calculation method is dependent on concrete and prestressing steel material properties and external factors, such as relative humidity. Prestress loss calculations affect the service-load behavior of a hollow core slab, but have little effect on the strength of a member.

Prestress loss calculations are required for prediction of camber and for service-load stress calculations. Hollow core producers should use a prestress loss calculation procedure that best predicts the behavior of the product as produced.

ACI 318-11 references several sources for prestress loss calculations. The method presented here was developed by Zia et al.[5] and considers the following:

1) Elastic shortening ES

$$ES = K_{es}\frac{E_{ps}}{E_{ci}}f_{cir}$$

K_{es} = 1.0 for pretensioned members

$$f_{cir} = K_{cir}\left(\frac{P_i}{A}+\frac{P_i e^2}{I}\right)-\frac{M_g e}{I}$$

K_{cir}= 0.9 for pretensioned members

2) Concrete creep CR

$$CR = K_{cr}\frac{E_{ps}}{E_c}(f_{cir} - f_{cds})$$

where

K_{cr} = 2.0 for normalweight pretensioned members

= 1.6 for sand lightweight pretensioned members

$$f_{cds} = \frac{M_{sd}e}{I}$$

3) Shrinkage of concrete SH

$$SH = 8.2 \times 10^{-6} K_{sh} E_{ps}\left(1 - 0.06\frac{V}{S}\right) \times (100 - RH)$$

K_{sh} = 1.0 for pretensioned members

RH = Ambient relative humidity from Fig 2.2.3.1

4) Steel relaxation RE

$RE = [K_{re} - J(SH + CR + ES)]C$

K_{re}, J, C = factors from Tables 2.2.3.1 and 2.2.3.2

5) Total loss = $ES + CR + SH + RE$

Observations and experience in a plant may provide modifications to prestress loss calculations to better predict the performance of hollow core slabs.

Example 2.2.3.1 Loss of Prestress

Using the generic cross section of the hollow core slab defined in Fig. 1.6.1, calculate the loss of prestress based on the following information:

Prestressing steel:

4–½-in.-dia., 270 ksi, low-relaxation strands

A_{ps} = 4(0.153) = 0.612 in^2

d_p = 7 in.

f_{pi} = $0.70 f_{pu}$

ℓ_{pc} = 30 ft-6 in.

Superimposed dead load D_S = 20 lb/ft^2

Solution:

1) Elastic shortening ES

$P_i = f_{pi}A_{ps}$

Table 2.2.3.1

Type of tendon	K_{re}, psi	J
270 ksi stress-relieved strand or wire	20,000	0.15
250 ksi stress-relieved strand or wire	18,500	0.14
240 or 235 ksi stress-relieved wire	17,600	0.13
270 ksi low-relaxation strand	5000	0.040
250 ksi low-relaxation wire	4630	0.037
240 or 235 ksi low-relaxation wire	4400	0.035
145 or 160 ksi stress-relieved bar	6000	0.05

P_i = 0.7(270)(0.612) = 115.6 kip

$$M_g = \frac{\ell^2_{pc}}{8} D_{sw} b$$

$$M_g = \frac{30.5^2}{8}(0.0535)(3\text{ ft})$$

= 18.66 kip-ft

= 224 kip-in.

$$f_{cir} = K_{cir}\left(\frac{P_i}{A} + \frac{P_i e^2}{I}\right) - \frac{M_g e}{I}$$

$$f_{cir} = 0.9\left(\frac{115.6}{154} + \frac{115.6(2.89)^2}{1224.5}\right) - \frac{(224)(2.89)}{1224.5}$$

= 0.857 ksi

E_{ps} = 28,800 ksi

$E_{ci} = 57\sqrt{f'_{ci}} = 57\sqrt{3000} = 3120$ ksi

$$ES = K_{es}\frac{E_{ps}}{E_{ci}} f_{cir}$$

$$= (1.0)\frac{28{,}800}{3120}(0.857)$$

= 7.91 ksi

Table 2.2.3.2 Values of C

f_{pi}/f_{pu}	Stress-relieved strand or wire	Stress-relieved bar or low-relaxation strand or wire
0.80		1.28
0.79		1.22
0.78		1.16
0.77		1.11
0.76		1.05
0.75	1.45	1.00
0.74	1.36	0.95
0.73	1.27	0.90
0.72	1.18	0.85
0.71	1.09	0.80
0.70	1.00	0.75
0.69	0.94	0.70
0.68	0.89	0.66
0.67	0.83	0.61
0.66	0.78	0.57
0.65	0.73	0.53
0.64	0.68	0.49
0.63	0.63	0.45
0.62	0.58	0.41
0.61	0.53	0.37
0.60	0.49	0.33

2) Concrete creep CR

$$M_{sd} = \frac{\ell^2_{pc}}{8} D_s b$$

$$M_{sd} = \frac{30.5^2}{8}(0.02)(3)(12) = 83.72 \text{ kip-in.}$$

$$f_{cds} = \frac{M_{sd}e}{I}$$

$$= \frac{83.72(2.89)}{1224.5}$$

$$= 0.198 \text{ ksi}$$

$$E_c = 57\sqrt{f'_c} = 57\sqrt{5000} = 4030 \text{ ksi}$$

$$CR = K_{cr}\frac{E_{ps}}{E_c}(f_{cir} - f_{cds})$$

$$= (2.0)\frac{28{,}800}{4030}(0.857 - 0.198)$$

$$= 9.42 \text{ ksi}$$

3) Shrinkage of concrete SH

$$\left(\frac{V}{S}\right) = \frac{\text{Volume}}{\text{Surface}} = \frac{\text{Area}}{\text{Perimeter}}$$

$$= \frac{154}{2(36+8)} = 1.75$$

Use $RH = 70\%$

$$SH = 8.2 \times 10^{-6} K_{sh} E_{ps}\left(1 - 0.06\frac{V}{S}\right) \times (100 - RH)$$

$$= 8.2 \times 10^{-6}(1.0)\,28{,}800 \times (1 - 0.06 \times 1.75)(100 - 70)$$

$$= 6.34 \text{ ksi}$$

4) Steel relaxation RE

From Table 2.2.3.1:

$K_{re} = 5000$ psi, $J = 0.04$

from Table 2.2.3.2

$C = 0.75$ for $f_{pi}/f_{pu} = 0.7$

$$RE = [K_{re} - J(SH + CR + ES)]C$$

$$= \left[\frac{5000}{1000} - 0.04 \times (6.34 + 9.42 + 7.91)\right]0.75$$

$$= 3.04 \text{ ksi}$$

5) Total loss at midspan

$$\text{Total loss} = ES + CR + SH + RE$$

$$= 7.91 + 9.42 + 6.34 + 3.04$$

$$= 26.7 \text{ ksi}$$

Percent loss of initial prestress:

$$\text{Percent} = \frac{\text{total loss}}{f_{pi}}(100)$$

$$= \frac{26.7}{(0.7)(270)}(100)$$

$$= 14.1\% \text{ of initial prestress}$$

2.2.4 Service Load Stresses

Service load concrete stresses are calculated as a measure of performance or serviceability. For the in-service state when deflections must be calculated, a stress check must first be made to determine the member classification (Class U, T, or C) and whether gross-section properties or cracked-transformed section properties are to be used.

In-service stresses are checked assuming that all prestress losses have occurred. The calculated stresses are compared to the permissible stresses noted in section 2.2.1. Hollow core slabs are normally designed to be Class U members. Tensile stress limits between $6\sqrt{f_c'}$ and $7.5\sqrt{f_c'}$ are commonly used. In special circumstances where deflections will not be a problem and where measures have been put in place to mitigate corrosion under severe exposure, Class T members may be used. Hollow core slabs are infrequently designed as Class C members. Caution should be exercised with the applicability of design provisions to Class C hollow core members, particularly with respect to shear, crack control, and deflections.

Example 2.2.4.1 Service Load Stresses

Using the generic cross section of the hollow core slab defined in Section 1.6, calculate the service load stresses given the following criteria:

Prestressing steel:

4–½ in. dia., 270 ksi, low-relaxation strands
$A_{ps} = 4(0.153) = 0.612\text{ in}^2$
$d_p = 7$ in.
$f_{pi} = 0.70 f_{pu}$
$f_c' = 5000$ psi
$\ell_{pc} = 30$ ft–6 in.
$\ell = 30$ ft–0 in.
Superimposed dead load $D_s = 20\text{ lb/ft}^2$
Live load $L = 50\text{ lb/ft}^2$

Solution:

Calculate the sustained moment:

$$M_{sustained} = \frac{\ell^2}{8}(D_{sw} + D_s)$$

$$M_{sustained} = \frac{30^2}{8}(0.0535 + 0.020)(3\text{ ft})$$
$$= 24.8\text{ kip-ft}$$
$$= 298\text{ kip-in}$$

Calculate the service moment:

$$M_{service} = \frac{\ell^2}{8}(D_{sw} + D_s + L)$$

$$M_{service} = \frac{30^2}{8}(0.0535 + 0.020 + 0.050)(3\text{ ft})$$
$$= 41.68\text{ kip-ft}$$
$$= 500\text{ kip-in.}$$

With losses = 14.1% from Example 2.2.3.1

$$P_e = A_{ps} f_{pi}(1 - \text{loss})$$
$$P_e = (0.612)(0.7)(270)\,(1 - 0.141)$$
$$= 99.4\text{ kip}$$

The stress in the precompressed tensile zone, which is the bottom fiber at midspan, is

$$f_{bot} = \frac{99.4}{154} + \frac{99.4(2.89)}{314.8} - \frac{500}{314.8}$$
$$= 0.645 + 0.913 - 1.588$$
$$= -0.030\text{ ksi (tension)}$$
$$< 7.5\sqrt{f_c'} = 7.5\frac{\sqrt{5000}}{1000} = 0.53\text{ ksi}$$

The member may be classified as Class U. Use gross section properties.

Top fiber compression with sustained loads

$$f_{top} = \frac{99.4}{154} - \frac{99.4(2.89)}{297.9} + \frac{298}{297.9}$$
$$= 0.645 - 0.964 + 1.000$$
$$= +0.681\text{ ksi}$$

Permissible compression

$$= 0.45 f_c'$$
$$= 0.45(5000)$$
$$= 2.25\text{ ksi} > 0.681\text{ ksi OK}$$

Top fiber compression with total load

$$f_{top} = \frac{99.4}{154} - \frac{99.4(2.89)}{297.9} + \frac{500}{297.9}$$
$$= 0.645 - 0.964 + 1.678$$
$$= 1.359\text{ ksi}$$

Permissible compression

$$= 0.60 f_c'$$
$$= 0.60(5000)$$
$$= 3.00\text{ ksi} > 1.359\text{ ksi OK}$$

2.2.5 Design Flexural Strength

The moment strength of a prestressed concrete member is a function of the maximum stress developed in the prestressing strands. Lower limits are placed on the amount of reinforcement to ensure that the member exhibits ductile behavior at first cracking. Nonprestressed members also have an upper limit of reinforcement. However, under the unified design procedure adopted in ACI 318-02[6] there is no upper limit of reinforcement for prestressed members. ACI 318-11 Section 18.8.1 requires that prestressed concrete members be classified as tension-controlled, transition, or compression-controlled based on the net tensile strain in the extreme tensile steel. For tension-controlled members, the net tensile strain must be at least 0.005 and $\phi = 0.9$. The addition of prestressed reinforcement beyond tension-controlled will result in diminishing return as ϕ decreases.

The lower limit on reinforcement requires that:

$\phi M_n \geq 1.2\ M_{cr}$

$$M_{cr} = \frac{I}{y_b}\left(\frac{P_e}{A} + \frac{P_e e}{S_b} + 7.5\sqrt{f'_c}\right)$$

This ensures that when the concrete first develops flexural cracks, the member will not fail in a brittle manner. However, ACI 318-11 Section 18.8.2 allows a waiver of this requirement for flexural members with shear and flexural strength at least twice that required by design. For additional information, refer to Section 2.2.1.5 of this manual.

The stress in the prestressing steel at nominal strength may be calculated in several ways. ACI 318-11 Eq. (18–1) may be used as an approximation, charts and tables from the *PCI Design Handbook* may be used, or a strain compatibility analysis may be made.

Example 2.2.5.1 Design Flexural Strength

Using the generic cross section of the hollow core slab defined in Fig. 1.6.1, check the design flexural strength, given the following criteria:

Prestressing steel:

4–½-in.-dia., 270 ksi, low-relaxation strands

$A_{ps} = 4(0.153) = 0.612$ in^2

$d_p = 7$ in.

$f_{pi} = 0.70 f_{pu}$

$f'_c = 5000$ psi

$\ell_{pc} = 30$ ft-6 in.

$\ell = 30$ ft-0 in.

Superimposed dead load $D_s = 20$ lb/ft^2

Live load $L = 50$ lb/ft^2

Solution:

Method 1: ACI Equation (18-1)

Use $\gamma_p = 0.28$ for low-relaxation strands

$$\beta_1 = 0.85 - \left(\frac{5000 - 4000}{1000}\right)0.05$$
$$= 0.80$$

$$\rho_p = \frac{A_{ps}}{bd_p} = \frac{0.612}{(36)(7)} = 0.0024$$

$$f_{ps} = f_{pu}\left(1 - \frac{\gamma_p}{\beta_1}\rho_p\frac{f_{pu}}{f'_c}\right)$$

$$f_{ps} = 270\left[1 - \frac{0.28}{0.80}\left(0.0024\frac{270}{5}\right)\right]$$
$$= 257.7 \text{ ksi}$$

$$a = \frac{A_{ps} f_{ps}}{0.85 f'_c b} = \frac{0.612(257.7)}{0.85(5)(36)}$$
$$= 1.03 \text{ in.}$$

The net tensile strain is

$$c = \frac{a}{\beta_1} = 1.03/0.8 = 1.29 \text{ in.}$$

$$\varepsilon_t = \frac{d_p - c}{c}(0.003) = \frac{7 - 1.29}{1.29}(0.003)$$
$$= 0.0133 > 0.005$$

Thus the section is tension-controlled, $\phi = 0.9$.

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

$$\phi M_n = 0.9\ (0.612)(257.7)\left(7 - \frac{1.03}{2}\right)$$
$$= 920 \text{ kip-in./slab}$$
$$= 76.7 \text{ kip-ft/slab}$$

Using ACI 318–11 Chapter 9 load combinations,

$$w_u = \text{the larger of } \begin{cases} 1.4D \\ 1.2D + 1.6L \end{cases}$$

$1.4D = 1.4(0.0535 + 0.02) = 0.103$ kip/ft^2

$1.2D + 1.6L = 1.2(0.0535 + 0.02) + 1.6(0.05)$
$= 0.168$ kip/ft^2 (controls)

Therefore, $w_u = 0.168$ kip/ft^2

$$M_u = \frac{30^2}{8}(0.168)(3\text{ ft})$$
$$= 56.7\text{ kip-ft} < 76.7\text{ OK}$$

Check minimum reinforcement

$$\phi M_n \geq 1.2\,M_{cr}$$

From Example 2.2.3.1

Loss = 14.1%

$$P_e = A_{ps}f_{pi}(1 - \text{loss})$$
$$= 0.7(270)(0.612)(1 - 0.141)$$
$$= 99.4\text{ kip}$$

Bottom compression

$$f_{bot} = \frac{99.4}{154} + \frac{99.4(2.89)}{314.8}$$
$$= 1.558\text{ ksi}$$

$$M_{cr} = \frac{1224.5}{3.89}\left(1.558 + \frac{7.5\sqrt{5000}}{1000}\right)$$
$$= 657\text{ kip-in.}$$

$$\frac{\phi M_n}{M_{cr}} = \frac{920}{657} = 1.4 > 1.2\text{ OK}$$

Please note that this check is necessary only at the critical section. Reference the discussion under section 2.2.1.5 in this manual.

Method 2: *PCI Design Handbook*
Using Fig. 5.14.2 from the seventh edition of the *PCI Design Handbook*

$$\omega_{pu} = \frac{A_{ps}f_{pu}}{bd_pf'_c}$$
$$= \frac{0.612(270)}{(36)(7)(5)}$$
$$= 0.131$$

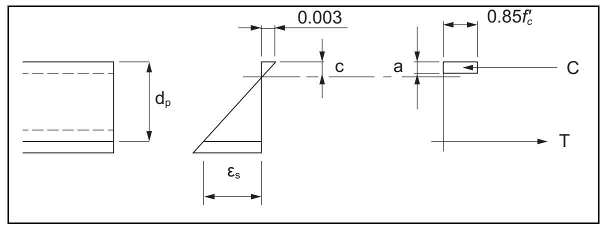
$$K'_u = 538$$

$$\phi M_n = K'_u\frac{bd_p^2}{12{,}000}$$
$$= 538\frac{36(7)^2}{12{,}000}$$
$$= 79.0\text{ kip-ft}$$

Method 3: Strain Compatibility

The stress-strain diagram from Fig. 15.3.3 of the *PCI Design Handbook*, shown in Fig. 2.2.5.1, will be used for this example.

The maximum usable strain at the extreme concrete compression fiber is assumed to be 0.003 in./in. The method involves a trial and error procedure to obtain equilibrium within the section, where the force in the compression block equals the tensile force in the steel. The equations are developed from the strain diagram shown.



$$a = \beta_1 c$$

Using 14.1% loss from Example 2.2.3.1

$$f_{se} = f_{pi}(1 - \text{loss})$$
$$f_{se} = 0.7(270)(1 - 0.141) = 162.4\text{ ksi}$$

$$\varepsilon_{se} = \frac{f_{se}}{E_{ps}} = \frac{162.4}{28{,}800} = 0.006$$

Assume $c = 1$ in., then $a = 0.80(1) = 0.8$ in.

$$\varepsilon_t = \frac{d_p - c}{c}(0.003)$$
$$= \frac{7-1}{1}(0.003) = 0.018$$

$$\varepsilon_{ps} = \varepsilon_{se} + \varepsilon_t$$
$$= 0.006 + 0.018 = 0.024$$

From stress-strain curve

$$f_{ps} = 268\text{ ksi}$$
$$T = (0.612)(268) = 164\text{ kip}$$
$$C = 0.85(5)(0.8)(36) = 122.4\text{ kip} < 164\text{ kip}$$

Try $c = 1.3$ in. then $a = 0.80(1.3) = 1.04$ in.

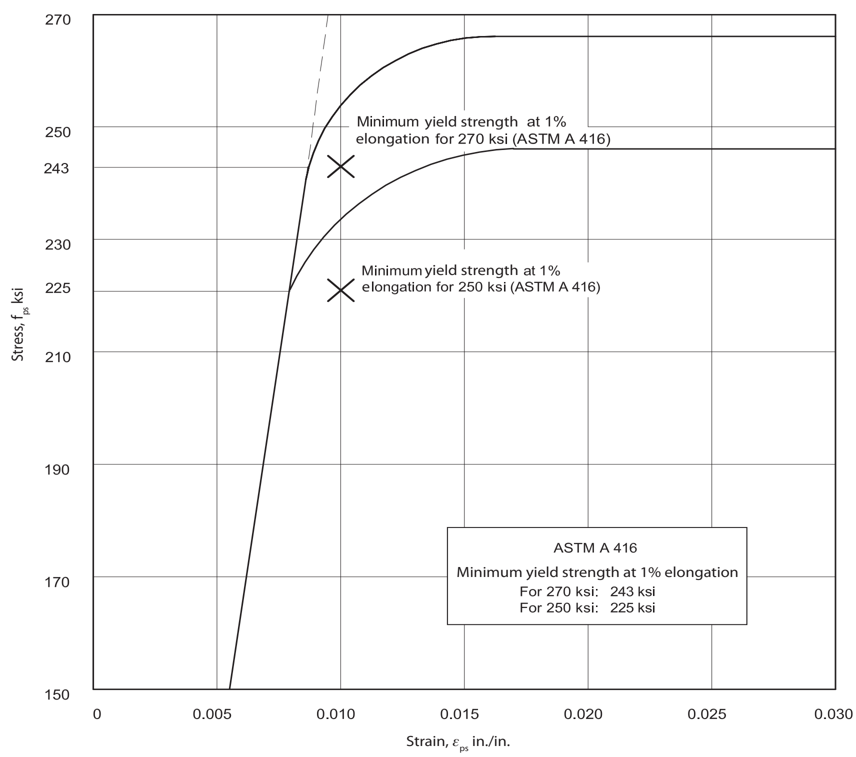
$$\varepsilon_t = \frac{7-1.3}{1.3}(0.003) = 0.0131$$

$$\varepsilon_{ps} = 0.0131 + 0.006 = 0.0191$$

From stress–strain curve

$$f_{ps} = 267\text{ ksi}$$
$$T = (0.612)(267) = 163\text{ kip}$$
$$C = 0.85(5)(1.04)(36)$$
$$= 159\text{ kip} \approx 163\text{ kip}$$

Figure 2.2.5.1 Stress-strain curves, prestressing strand



The curves can be approximated by the following equations:

250 ksi strand

$\varepsilon_{ps} \le 0.0076$: $f_{ps} = 28{,}800\ \varepsilon_{ps}$ (ksi)

$\varepsilon_{ps} > 0.0076$: $f_{ps} = 250 - \dfrac{0.04}{\varepsilon_{ps} - 0.0064}$ (ksi)

270 ksi strand

$\varepsilon_{ps} \le 0.0085$: $f_{ps} = 28{,}800\ \varepsilon_{ps}$ (ksi)

$\varepsilon_{ps} > 0.0085$: $f_{ps} = 270 - \dfrac{0.04}{\varepsilon_{ps} - 0.007}$ (ksi)

Because $\varepsilon_t = 0.0131 > 0.005$, $\phi = 0.9$.

$$\phi M_n = \phi A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)$$

$$\phi M_n = 0.9\ (0.612)(267)\left(7 - \frac{1.04}{2}\right)$$

= 953 kip-in.
= 79.4 kip-ft

On occasion, conventional mild steel reinforcement is added to a hollow core slab to locally provide added flexural strength. When required, the bars are placed in cores right after the slab is cast and concrete is added to fill the cores with the bars. The following example illustrates the flexural strength calculation.

Example 2.2.5.2 Flexural Strength with Bars

Repeat Example 2.2.5.1 but add two #4 bars in cores.

Solution:

Use strain compatibility for strength calculation with an effective depth of 5.5 in. for the #4 bars.

d = 5.5 in.
A_s = 2(0.2) = 0.4 in^2

Assume c = 1.53 in.

then $a = \beta_1 c = 0.80(1.53) = 1.22$ in.

For strands

$$\varepsilon_t = \frac{d_p - c}{c}(0.003) = \frac{7 - 1.53}{1.53}(0.003)$$
$= 0.0107$ in./in.

$\varepsilon_{ps} = 0.006 + 0.0107$
$= 0.0167$ in./in.

$f_{ps} = 266$ ksi

For bars

$$\varepsilon_s = \frac{d - c}{c}(0.003) = \frac{5.5 - 1.53}{1.53}(0.003)$$
$= 0.0078$ in./in.

$$\text{yield strain} = \frac{60}{29{,}000} = 0.002 \text{ in./in.}$$

$T = (0.612)(266) + (0.4)(60) = 162.8 + 24$
$= 186.8$ kip

$C = 0.85(5)(1.22)(36) = 186.7$ kip
$\cong 186.8$ kip ok

Since $\varepsilon_t > 0.005$, $\phi = 0.9$.

$$\phi M_n = \phi\left[A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_y\left(d - \frac{a}{2}\right)\right]$$

$$\phi M_n = 0.9\left[162.8\left(7 - \frac{1.22}{2}\right) + 24\left(5.5 - \frac{1.22}{2}\right)\right] = 1042 \text{ kip-in.}$$
$= 86.8$ kip-ft

2.3 Shear Design

2.3.1 ACI Requirements

Hollow core slabs are designed for shear according to the same ACI provisions used in general for prestressed concrete members. Stirrups are not normally provided when the applied shear exceeds the shear strength because of the difficulty with placing stirrups in most dry-cast production systems. The placement of stirrups in a wet-cast system is easier than in a dry-cast extruded system and is a viable shear enhancement method. An alternative to increase shear strength is to reduce the number of cores used in a given slab. This may be done by either leaving out a core for the entire length of a slab or by locally breaking into the cores and filling them so they are solid while the concrete is still in a somewhat fresh state.

The provisions for shear are found in Chapter 11 of ACI 318-11. The requirements are summarized as follows:

$V_u \le \phi V_n$

The strength reduction factor $\phi = 0.75$ for shear.

$V_n = V_c + V_s$

For the purpose of this discussion, the contribution of shear reinforcement V_s will be taken as zero. The nominal concrete shear strength V_c may be found using Eq. (11–9),

$$V_c = \left(0.6\lambda\sqrt{f_c'} + 700\frac{V_u d_p}{M_u}\right)b_w d \quad \text{(ACI 11–9)}$$

when the effective prestress force is not less than 40% of the tensile strength of the flexural reinforcement. The term $V_u d_p/M_u$ shall not be taken greater than 1.0. The minimum value for V_c may be used as $2\lambda\sqrt{f_c'}\, b_w d$ and the maximum value is the lesser of $5\lambda\sqrt{f_c'}\, b_w d$ or the value obtained from Eq. (11–12) considering reduced effective prestress in the transfer zone.

Alternatively, more refined shear calculations can be made using Eq. (11–10) or (11–12), with V_c taken as the lesser value given by those equations.

$$V_{ci} = 0.6\lambda\sqrt{f_c'}\, b_w d_p + V_d + \frac{V_i M_{cre}}{M_{max}} \quad \text{(ACI 11–10)}$$

$$V_{cw} = (3.5\lambda\sqrt{f_c'} + 0.3 f_{pc})\, b_w d_p \quad \text{(ACI 11–12)}$$

Equation (11–10) calculates shear strength for an inclined flexure-shear failure mode. For Eq. (11–10), the following relationships are used:

$$M_{cre} = \frac{I}{y_b}(6\lambda\sqrt{f_c'} + f_{pe} - f_d) \quad \text{(ACI 11–11)}$$

V_d = unfactored shear due to self-weight for non-composite sections

$V_i = V_u - V_d$

$M_{max} = M_u - M_d$

M_d = unfactored moment due to self-weight for non-composite sections

The minimum value for V_{ci} need not be less than $1.7\lambda\sqrt{f_c'}\,b_w d$ or $2\lambda\sqrt{f_c'}\,b_w d$ when the effective prestress force is not less than 40% of the tensile strength of the flexural reinforcement. The web width, b_w, is normally taken as the sum of the widths of the individual webs measured at their narrowest point. For Eq. (11–10), (11–11), and (11–12), the reduction in prestressing force at the member end due to transfer must be considered. ACI 318-11 allows an assumption that prestressing force increases linearly from zero at the member end to full effective prestress in a length equal to 50 strand diameters. If debonded strands are used, transfer of prestress for the debonded strands must also be considered.

ACI 318-11 Section 11.4.6.1 contains a list of members for which minimum shear reinforcement is not required where V_u exceeds $0.5\phi V_c$. The exemption from minimum shear reinforcement requirement in that section is for hollow core slabs with total untopped depth not greater than 12.5 in. and hollow core slabs of any depth where V_u is not greater than $0.50\phi V_{cw}$. The commentary to Section 11.4.6.1(b) points out that test results of hollow core slabs [7,8,9] with total depths of 12.5 in. or less have shown shear strengths greater than those calculated by Eq. (11-12) and (11-10). Test results of precast/prestressed concrete hollow core slabs with greater depths have shown that web shear strengths in end regions can be less than those strengths computed by Eq. (11-12). By contrast, flexure-shear strengths in these tests equaled or exceeded strengths computed by Eq. (11-10).

Example 2.3.1.1 Shear Design

Using the generic cross section of the hollow core slab defined in Section 1.6, check the slab for shear given the following information:

Prestressing steel:
4–½-in.-dia., 270 ksi, low-relaxation strands
$A_{ps} = 4(0.153) = 0.612$ in^2
$f_{pi} = 0.7 f_{pu}$

Loss = 15%

$f_c' = 5000$ psi (normal weight)
$\ell_{pc} = 25$ ft-6 in.
$\ell = 25$ ft-0 in.
Superimposed dead load $D_s = 20$ lb/ft^2

Live load $L = 50$ lb/ft^2

Masonry dead load = 800 lb/ft at 3 ft from one support

Solution:

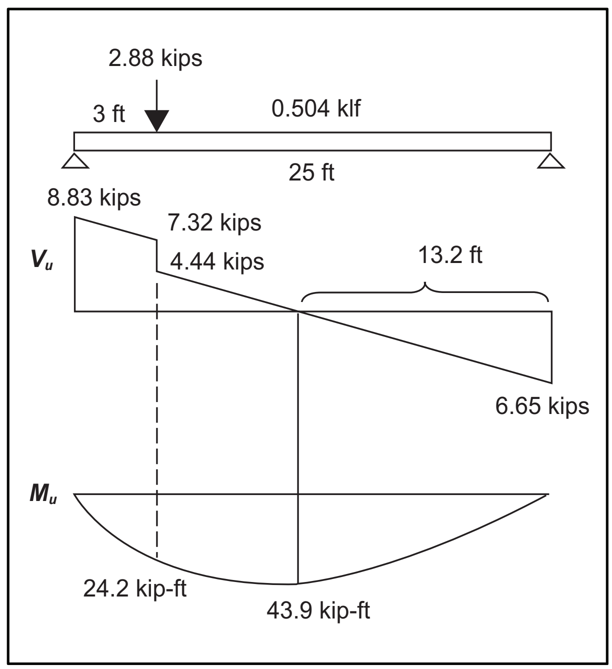
Uniform load:

$1.4D = 1.4(0.0535 + 0.020)(3\text{ ft}) = 0.309$ kip/ft
$1.2D+1.6L = [1.2(0.0535+0.020)+1.6(0.050)](3)$
$= 0.504$ kip/ft

Line load:
$1.4D = 1.4(0.800)(3\text{ ft}) = 3.36$ kip
$1.2D = 1.2(0.800)(3\text{ ft}) = 2.88$ kip

The combination of $w_u = 0.309$ kip/ft and $P_u = 3.36$ kip does not govern. The governing combination is $w_u = 0.504$ kip/ft and $P_u = 2.88$ kip.

Load, shear, and moment diagrams for a 3 ft hollow core slab width under the governing load combination are shown below:



Using the more refined approach according to ACI Eq. (11–10) or (11–12), ϕV_c is:

$$\phi V_{cw} = 0.75\left(\frac{3.5\sqrt{5000}}{1000} + 0.3 f_{pc}\right)(10.5)(7)$$

$$= 13.64 + 16.54 f_{pc}$$

f_{pc} is calculated as a function of the transfer of prestress into the section along the span.

$\ell_t = 50\, d_b = 50(^1/_2) = 25$ in.
With bearing length = 3 in.

Full prestress transfer is achieved 22 in. from the face of support

$$P_e = A_{ps} f_{pi}(1 - \text{loss})\left(\frac{x + \text{bearing}}{\ell_t}\right)$$

$$P_e = 0.612(0.70)(270)(1\text{–}0.150)\left(\frac{x+3}{25}\right)$$

$$= 98.318\left(\frac{x+3}{25}\right) \text{ to } x = 22 \text{ in.}$$

$$f_{pc} = \frac{P_e}{A} = \frac{98.318}{154}\left(\frac{x+3}{25}\right)$$

$$\phi V_{cw} = 13.64 + 16.54\,\frac{98.318}{154}\left(\frac{x+3}{25}\right)$$

$$= 13.64 + 10.56\left(\frac{x+3}{25}\right)$$

to $x = 22$ in.

$$\phi V_{ci} = 0.75\left(0.6\frac{\sqrt{5000}}{1000}(10.5)(7) + V_d + \frac{V_i M_{cre}}{M_{max}}\right) \qquad (11\text{–}10)$$

$$V_d = 3(0.0535)\left(\frac{25}{2} - x\right)$$

$$= 2.01 - 0.16x$$

V_i = shear due to factored loads minus V_d

$$M_{cre} = \frac{I}{y_b}(6\sqrt{f'_c} + f_{pe} - f_d)$$

$$f_{pe} = A_{ps} f_{se}\left(\frac{1}{A} + \frac{e y_b}{I}\right)\left(\frac{x+3}{25}\right)$$

$$f_{pe} = 98.318\left[\frac{1}{154} + \frac{(3.89-1)(3.89)}{1224.5}\right]\left(\frac{x+3}{25}\right)$$

$$= 1.541\left(\frac{x+3}{25}\right) \le 1.541 \text{ ksi}$$

to $x = 22$ in.

$$f_d = \frac{M_d}{S}$$

$$= \frac{\frac{3(0.0535)x}{2}(25-x)}{314.8} = \frac{2.01x - 0.08x^2}{314.8}$$

$$M_{cre} = \left(0.424 + f_{pe} - \frac{2.01x - 0.08x^2}{314.8}(12)\right)\left(\frac{314.8}{12}\right)$$

$$= 11.130 + 26.23 f_{pe} - 2.01x + 0.08x^2$$

Based on these definitions, ϕV_{cw}, ϕV_{ci}, and V_u are calculated at intervals across the span. A summary is presented in Table 2.3.1.1 and Fig. 2.3.1.1.

Alternatively, the simplified equation (11–9) might be used.

$$\phi V_c = 0.75\left[0.6\sqrt{5000} + 700\left(\frac{V_u}{M_u}\right)(7)\right]\left(\frac{10.5(7)}{1000}\right)$$

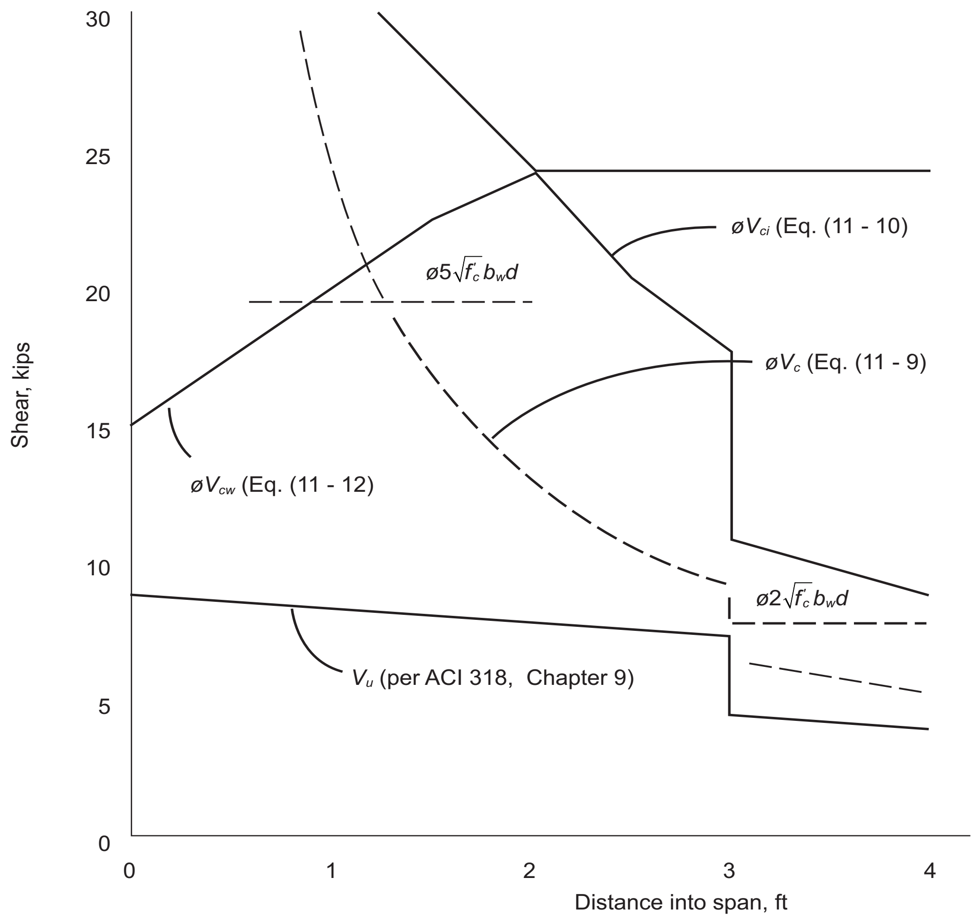
$$= 2.34 + 270.1\frac{V_u}{M_u} \quad (M_u \text{ in kip-in.})$$

The results of this equation are also shown in Fig. 2.3.1.1. At all points, $V_u < \phi V_c$ therefore shear strength is adequate and stirrups are not required.

Table 2.3.1.1 Allowable shear

***x*, ft**	***V***$_u$**, kip**	ϕV_{cw}**, kip**	ϕV_{ci}**, kip**
h/2 = 0.333	8.66	16.59	52.60
0.5	8.58	17.43	46.02
1.0	8.33	19.96	32.50
1.5	8.07	22.49	28.02
2.0	7.82	24.17	24.49
2.5	7.57	24.17	20.35
3.0	7.32	24.17	17.66
3.0	4.44	24.17	10.84
3.5	4.19	24.17	9.68
4.0	3.93	24.17	8.81

Figure 2.3.1.1 Shear for Example 2.3.1.1



2.4 Camber and Deflection

Camber is the upward deflection of a prestressed concrete member and results from the prestressing force being eccentric from the center of gravity of the cross section. Because both the prestressing force and the eccentricity are established by the required design loads and the span length, camber is a result of the design rather than a design parameter. Therefore, camber requirements should not be specified.

Deflections are also affected by the amount of prestressing because prestressing establishes the load at which a member will crack. If tensile stresses are kept below cracking stresses, deflections will be independent of the prestress level.

Cambers and deflections will change with time due to concrete creep, prestress loss, and other factors. The sustained compression due to the prestressing will cause camber growth. Balancing this is the effect of creep on deflections due to self-weight and other sustained loads. It is this time-dependent movement, in addition to instantaneous deflections, that must be considered in the development of framing schemes and detailing.

Instantaneous cambers and deflections are predictable as long as the material properties are known. The time-dependent cambers and deflections are difficult to predict with a high degree of accuracy and any calculation of long-term movements must be considered to be only estimates.

This section presents calculation procedures for determining long-term deflections. From the hollow core producer's standpoint, history and experience must be used to modify the procedures to fit the local product. From the standpoint of the specifier, these procedures will allow only approximate estimates of long-term effects and should be complemented with discussions with local producers.

2.4.1 Camber

Hollow core slabs are produced with straight strand patterns rather than draped or depressed strands. Using (+) to indicate upward movement and (–) to indicate downward movement, net camber can be calculated as:

$$\text{camber} = \frac{Pe\ell_{pc}^2}{8EI} - \frac{5w\ell_{pc}^4}{384EI}$$

To determine initial camber, the appropriate values for the prestress force and the modulus of elasticity of the concrete must be used. When strength rather than tensile stress governs a design, the initial strand stress may be reduced to modify the anticipated camber. Additionally, slab camber is sensitive to support point locations during storage. Camber will increase as these support points move in from the slab ends.

Example 2.4.1 Initial Camber

Using the generic cross section of the hollow core slab defined in Section 1.6, calculate the initial camber given the following:

Prestressing steel:

4 ½-in.-dia., 270 ksi, low-relaxation strands.

$A_{ps} = 4(0.153) = 0.612$ in^2
$f_{pi} = 0.7 f_{pu}$
$d_p = 7$ in.
$\ell_{pc} = 28$ ft-6 in.

Solution:

Estimate initial losses at 5% and use $E_{ci} = 3120$ ksi

$P_o = f_{pi}A_{ps}(1 - \text{loss})$
$P_o = 0.7(270)(0.612)(1 - 0.05) = 109.9$ kip

$$\text{Camber} = \frac{P_o e\ell_{pc}^2}{8E_{ci}I} - \frac{5w\ell_{pc}^4}{384E_{ci}I}$$

$$= \frac{109.9(3.89-1)[28.5(12)]^2}{(8)(3120)(1224.5)} - \frac{5(3)(0.0535)(28.5)^4(1728)}{(384)(3120)(1224.5)}$$

$= 1.22 - 0.62 = 0.60$ in.

Approximately ½ in. to ¾ in. initial camber.

Estimating long-term effects is complicated because, as time passes, the prestressing force decreases due to losses and the modulus of elasticity of the concrete increases with concrete strength gain. Traditionally, a creep factor of 2.0 has been applied to instantaneous deflections to estimate the additional deflection due to creep. This has been modified by Martin[10] for prestressed concrete. Table 2.4.1 presents suggested multipliers to determine both long-term final deflections and position at erection. It should be noted that in using these multipliers, a total deflection is calculated rather than the additional increment due to long-term effects.

Example 2.4.2 Long-Term Camber

For the slab of Example 2.4.1, determine the net camber at erection and the final camber.

Solution:

At erection,

Initial camber = 1.22 – 0.62 = 0.60 in. from Example 2.4.1

Erection camber = 1.22(1.80) – 0.62(1.85) = 1.05 in., approximately 1 in.

Final camber = 1.22(2.45) – 0.62(2.70) = 1.32 in., approximately 1¼ in.

2.4.2 Deflections

As with camber, concrete creep will also affect deflections due to sustained superimposed loads. These long-term effects must be considered for comparison with Table 9.5(b) of ACI 318-11 to determine acceptability. This table is reproduced here as Table 2.4.2. Engineering judgment should be used in comparing calculated deflections with the ACI limits. Live loads specified in building codes quite often exceed the actual loads in a structure. While it may be implied that the full live load be used for comparison to

Table 2.4.1 Long-term Multipliers

Condition	Without composite topping	With composite topping
At erection:		
1. Deflection (downward) component – apply to the elastic deflection due to the member weight at release of prestress	1.85	1.85
2. Camber (upward) component – apply to the elastic camber due to the prestress at the time of release of prestress	1.80	1.80
Final:		
3. Deflection (downward) component – apply to the elastic deflection due to the member weight at release of prestress	2.70	2.40
4. Camber (upward) component – apply to the elastic camber due to prestress at the time of release of prestress	2.45	2.2
5. Deflection (downward) – apply to elastic deflection due to superimposed dead load only	3.00	3.00
6. Deflection (downward) – apply to elastic deflection caused by the composite topping	——	2.30

Table 9.5(b), situations may arise where it is more reasonable to use actual anticipated live loads for deflection comparisons. A further complication for superimposed loads is that flexural cracking will reduce the effective moment of inertia of the section. Calculations using transformed cracked section analysis are required for Class C and Class T members when tension exceeds $7.5\sqrt{f_c'}$ and are covered extensively in references 1 and 11. Because hollow core slabs are normally designed to be uncracked under service loads, the effects of cracking will not be considered here.

Table 2.4.1 includes multipliers for determining the long-term effects of superimposed loads. Again, use of the multipliers gives an estimate of the total deflection rather than the additional long-term deflection.

Example 2.4.3

For the slab of Examples 2.4.1 and 2.4.2, compare deflections to the ACI 318 limits due to a superimposed dead load of 20 lb/ft^2 and live load of 50 lb/ft^2 on a clear span of 28 ft, including long-term effects. Use $E_c = 4030$ ksi.

Solution:

From Example 2.4.2:

Final camber = 1.32 in.

Instantaneous deflection due to superimposed dead load = $\dfrac{5(0.02)(3)(28)^4(1728)}{(384)(4030)(1224.5)} = 0.17$ in.

Final deflection = 0.17(3.0) = 0.51 in.

Instantaneous live load deflection

$$= \frac{5(0.05)(3)(28)^4(1728)}{(384)(4030)(1224.5)} = 0.42 \text{ in.}$$

Final position = Final camber + sustained dead load deflection + live load increment

Final camber = + 1.32 in.

Sustained dead load deflection = – 0.51 in.

Net camber = final camber + sustained dead load deflection
=1.32 – 0.51 = 0.81 in.

Live load increment = – 0.42 in.

Final position = 1.32 – 0.51 – 0.42 = + 0.39 in.

Table 2.4.2 Maximum permissible computed deflections

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load *L*	$\frac{\ell}{180}$ *
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load *L*	$\frac{\ell}{360}$
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements (sum of the long-term deflection due to all sustained loads and the immediate deflection due to any additional live load)†	$\frac{\ell}{480}$ ‡
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections		$\frac{\ell}{240}$ **

* Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.
†Long-term deflection shall be determined in accordance with 9.5.2.5 or 9.5.4.2, but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on the basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.
‡ Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.
** But not greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

For comparison to the provisions of Chapter 9 of ACI 318-11, when non-structural elements are attached to the slabs, the portion of deflection after erection may be used for comparison.

Change in camber = 1.32 – 1.05 = +0.27 in.

Sustained dead load deflection = –0.51 in.

Instantaneous live load deflection = <u>–0.42 in.</u>
–0.66 in.

Compare to ACI 318 limits from Table 2.4.2

As floor, live load deflection limit

$$= \frac{\ell}{360} = \frac{28(12)}{360} = 0.93\text{ in.} > 0.42\text{ in.}$$

Attached to non-structural elements likely to be damaged:

$$\text{deflection limit} = \frac{\ell}{480} = \frac{28(12)}{480} = 0.70\text{ in.}$$
$$> 0.66\text{ in.}$$

Not attached to non-structural elements likely to be damaged:

$$\text{deflection limit} = \frac{\ell}{240} = \frac{28(12)}{240} = 1.40\text{ in.}$$
$$> 0.66\text{ in.}$$

When a composite topping is used, it will be cast after a portion of the slab shrinkage has occurred. There will then be differential shrinkage between the topping and the hollow core slab.

This differential can cause additional deflection and bottom tensile stresses. These effects will generally be negligible.

Example 2.4.4 Composite Slab

Given the hollow core slab of Example 2.4.3, add a 2 in. composite topping and recalculate deflections.

Solution:

Using Table 2.4.1, final camber

$$= 1.22(2.20) - 0.62(2.40) = 1.2 \text{ in.}$$

Instantaneous deflection due to topping weight =

$$\frac{5(0.025)(3)(28)^4(1728)}{(384)(4030)(1224.5)} = 0.21 \text{ in.}$$

Long-term deflection due to topping weight

$$= 0.21(2.30) = 0.48 \text{ in.}$$

Deflection due to superimposed dead load

$$= \frac{5(0.02)(3)(28)^4(1728)}{(384)(4030)(2307)} = 0.09 \text{ in.}$$

(Note: 2307 in.^{4} = composite moment of inertia using a 3000 psi topping on a 5000 psi hollow core slab.)

Long-term deflection due to dead load

$$= 0.09(3.0) = 0.27 \text{ in.}$$

Instantaneous deflection due to live load

$$= \frac{5(0.05)(3)(28)^4(1728)}{(384)(4030)(2307)} = 0.22 \text{ in.}$$

Final position

$$= +1.2 - 0.48 - 0.27 - 0.22 = +0.23 \text{ in.}$$

including instantaneous live load deflection.

2.5 Composite Design

A composite, structural concrete topping is commonly used in floor construction with hollow core slabs. The composite action is desirable to add stiffness and strength for gravity loads and may also be required for force transfer within a diaphragm. When a composite topping is used, consideration must be given to its strength, detailing, and quality assurance.

The required compressive strength of the topping may be determined from the hollow core slab design requirements. Load tables provided by local producers will normally indicate that either 3000 psi or 4000 psi concrete is required. Diaphragm requirements may necessitate a higher strength concrete in the topping.

From a detailing standpoint, the primary consideration is that hollow core slabs will have camber. If the topping is finished as a level surface, the camber will reduce the topping thickness in the midspan region, which will affect the load capacity of the slabs. With significant topping thickness reduction, the integrity of the topping concrete may also be compromised. A preliminary slab design can provide an estimate of camber and the minimum topping thickness necessary to support the design loads. The first option is to provide the minimum thickness topping at midspan and allow the thickness to increase at the slab ends to maintain a flat floor. Finish and bearing elevations can then be set to this criterion.

A second option to minimize topping concrete volume is to allow the minimum topping thickness to follow the curvature of the hollow core slab. This will result in a finished floor with camber, which may be acceptable in some occupancy categories. In this option, it is important that all trades be made aware of the final camber, as it may affect their work. Partitions, doorways, and stairs will be particularly affected in this option.

When control joints are used in a structural topping, they should be located over the joints in the precast concrete slabs where cracks would most naturally occur in the topping. At the ends of hollow core slabs, where movement will occur due to camber changes, deflections, creep, or shrinkage, control joints are desirable.

Reinforcing of a topping may be required for structural design. If not, consideration should be given to using minimum shrinkage reinforcement for crack control.

Because the composite topping and hollow core slabs interact to create the final structural element, it is imperative that the topping bonds well with the slabs. While the building designer may only be interested in the final product, the process of achieving a well-bonded composite topping is very important. The hollow core producer is depending on a properly bonded topping, yet is not involved in specifying, designing, or installing the topping. The hollow core producer is responsible for supplying a slab that is capable of bonding with a topping. The installer of the topping is responsible for surface preparation, topping concrete design, and curing to ensure proper bond.

As a minimum, the hollow core slab surface must be clean and damp at the time of topping installation. It is recommended that the surface be thoroughly saturated prior to topping placement, but all standing water must be removed. ACI 301-10[12] specifies that a sand and cement grout be scrubbed into the slab surface ahead of topping placement. If this procedure is used, it is imperative that initial set not be allowed prior to topping placement. If initial set occurs, the grout can become a bond breaker. Similarly, bonding agents, which are rarely specified, will also act as a bond breaker if any initial set occurs prior to topping placement.

The topping concrete and curing techniques will also affect bond of a composite topping. Curling at topping edges or joints will cause local delamination. Curling is a result of differential shrinkage between the top and the bottom surfaces of the topping. Generally, water is lost more quickly from the top surface, causing additional drying shrinkage. This can be exacerbated by use of forced air heaters, but can be minimized by proper curing techniques and use of low-shrinkage concrete.

Design of hollow core slabs for composite action is usually limited to a horizontal shear strength of 80 psi according to Section 17.5.3.1 of ACI 318-11. Through limited published[13] and unpublished testing, the machine-finished surface has been found to meet the requirements of that section. The horizontal shear check should be based on the distribution of forces in the member, rather than using an average horizontal shear over the distance from zero moment to maximum moment, when checking compliance with the 80 psi limit.

Composite ties are not normally provided, given the difficulty and expense of installing the ties in a machine casting operation. When the horizontal shear exceeds 80 psi and composite ties are not used, the topping is considered to be superimposed dead load on a non-composite hollow core slab. In a wet-cast system, horizontal shear ties with $^1/_4$ in. amplitude roughening may be used to take advantage of the higher stresses allowed by ACI.

Design of a composite section is similar to that presented in sections 2.2 and 2.3. The following example demonstrates the additional considerations with a composite section.

Example 2.5.1 Composite Design

Using the generic cross section of the hollow core slab defined in Fig. 1.6.1, add a 2 in. structural topping and check for the following conditions:

Prestressing steel:

4 ½-in.-dia., 270 ksi low relaxation strands
A_{ps} = 4(0.153) = 0.612 in^2
f_{pi} = 0.7f_{pu}
d_p = 7 in.

Hollow core slab:

f'_c = 5000 psi
E_{ci} = 3120 ksi
E_c = 4030 ksi

Topping: f'_c = 3000 psi

E_c = 3120 ksi
ℓ_{pc} = 30 ft-6 in.
ℓ = 30 ft-0 in.
Loads: D_t = 25 lb/ft^2
D_s = 20 lb/ft^2 L = 50 lb/ft^2

Solution:

Calculate section properties:

Base section A = 154 in.^{2}
I = 224.5 in.^{4}
y_b = 3.89 in.

Topping

$$n = \frac{E_c \text{ of topping}}{E_c \text{ of slab}}$$

n = 3120/4030 = 0.77

Use topping width =0.77(36) = 27.7 in.

Composite section:

$A_{trcomp} = 154 + 2(27.7) = 209.4 \text{ in.}^2$

$$y_{b,comp} = \frac{154(3.89) + 2(27.7)(9)}{209.4} = 5.24 \text{ in.}$$

$$I_{comp} = 1224.5 + 154(5.24 - 3.89)^2 + \frac{2^3}{12}(27.7) + 2(27.7)(9 - 5.24)^2 = 2307 \text{ in.}^4$$

Calculate prestress losses:

$ES = 7.91$ ksi from Example 2.2.3.1

Concrete creep

$$M_{sd} = \frac{30^2}{8}(0.025 + 0.020)(3) = 15.19 \text{ kip-ft}$$

$$f_{cds} = \frac{15.19(12)(2.89)}{1224.5} = 0.430 \text{ ksi}$$

$f_{cir} = 0.857$ ksi from Example 2.2.3.1

$$CR = (2.0)\frac{28{,}800}{4030}(0.857 - 0.430) = 6.10 \text{ ksi}$$

$SH = 6.34$ ksi from Example 2.2.3.1

$$RE = \left[\frac{5000}{1000} - 0.04(6.34 + 6.10 + 7.91)\right]0.75$$
$$= 3.14 \text{ ksi}$$

Loss $= 7.91 + 6.10 + 6.34 + 3.14$
$= 23.49$ ksi $= 12.4\%$

Calculate service load stresses:

$P_e = 0.612(0.7)(270)(1 - 0.124) = 101.3$ kip

$$M_{non\text{-}comp} = \frac{30^2}{8}(0.0535 + 0.025)(3\text{ft})$$
$$= 26.5 \text{ kip-ft} = 318 \text{ kip-in.}$$

$$M_{comp} = \frac{30^2}{8}(0.020 + 0.050)(3 \text{ ft})$$
$$= 23.6 \text{ kip-ft} = 284 \text{ kip-in.}$$

At top of topping

$$f_{top} = \frac{284(10 - 5.24)}{2307}(0.77) = 0.450 \text{ ksi}$$

At the top of the hollow core slab

$$f_{top} = \frac{101.3}{154} - \frac{101.3(2.89)(4.11)}{1224.5} + \frac{318(4.11)}{1224.5} + \frac{284(8 - 5.24)}{2307} = 1.082 \text{ ksi}$$

At the bottom of the hollow core slab

$$f_{bot} = \frac{101.3}{154} + \frac{101.3(2.89)(3.89)}{1224.5} - \frac{318(3.89)}{1224.5} - \frac{284(5.24)}{2307} = \text{-}0.066 \text{ ksi}$$

Calculate flexural strength

$w_u = 1.2(0.0535 + 0.025 + 0.020)$
$+ 1.6(0.050) = 0.198 \text{ kip/ft}^2$

$$M_u = \frac{30^2}{8}(0.198)(3 \text{ ft}) = 66.9 \text{ kip-ft}$$

Using ACI Eq. (18–1)

$$\rho_p = \frac{A_{ps}}{bd_p} = \frac{0.612}{(36)(9)} = 0.0019$$

$$f_{ps} = f_{pu}\left(1 - \frac{\gamma_p}{\beta_1}\rho_p\frac{f_{pu}}{f'_c}\right)$$

$$f_{ps} = 270\left[1 - \frac{0.28}{0.80}(0.0019)\left(\frac{270}{5}\right)\right] = 260.3 \text{ ksi}$$

$$a = \frac{A_{ps}f_{ps}}{0.85f'_cb} = \frac{0.612(260.3)}{0.85(5)(36)} = 1.04 \text{ in.}$$

$$c = \frac{a}{\beta_1} = \frac{1.04}{0.80} = 1.3 \text{ in.}$$

$$\varepsilon_t = \frac{d_p - c}{c}0.003 = \frac{9 - 1.3}{1.3}0.003 = 0.0178$$
$$> 0.005$$

Thus, the section is tension-controlled,

$\phi = 0.9$

$$\phi M_n = 0.9(0.612)(260.3)\left(9 - \frac{1.04}{2}\right)$$
$$= 1216 \text{ kip-in.} = 101.3 \text{ kip-ft}$$

Check 1.2 M_{cr}

$$f_{bot} = \frac{101.3}{154} + \frac{101.3(2.89)(3.89)}{1224.5} = 1.588 \text{ ksi}$$

$$M_{cr} = \frac{2307}{5.24}\left(1.588 + \frac{7.5\sqrt{5000}}{1000}\right) = 933 \text{ kip-in.}$$

$$\frac{\phi M_n}{M_{cr}} = \frac{1216}{933} = 1.30 > 1.2 \text{ ok}$$

Please note that this check is necessary only at the critical section. For more information, refer to the discussion under section 2.2.1.5 in this manual.

Check horizontal shear:

$\phi V_{nh} = \phi 80 b_v d = 0.75(80)(36)(9)$
$= 19{,}440 \text{ lb} = 19.44 \text{ kip}$

At $h/2$,

$$V_u = \left(\frac{30}{2} - \frac{10}{2(12)}\right)(0.198)(3) = 8.66 \text{ kip}$$
< 19.44 kip ok

Section is composite.

Check web shear at $h/2$:

Transfer length = 50(0.5) = 25 in.

At $h/2$ plus 3 in. bearing

$$P_{ex} = 101.3\left(\frac{5+3}{25}\right) = 32.4 \text{ kip}$$

For composite section, f_{pc} is calculated at centroid of composite section

$$f_{pc} = \frac{32.4}{154} - \frac{32.4(2.89)(5.24-3.89)}{1224.5} = 0.107 \text{ ksi}$$

ϕV_{cw}

$$= 0.75\left[\frac{3.5\sqrt{5000}}{1000} + 0.3(0.107)\right](10.05)(9) =$$
19.81 kip > 8.66 kip ok

Check flexure-shear at 4 ft

$$V_u = \left(\frac{30}{2} - 4\right)(0.198)(3) = 6.53 \text{ kip}$$

$$V_d = \left(\frac{30}{2} - 4\right)(0.0535 + 0.025$$
$$+ 0.020)(3) = 3.25 \text{ kip}$$

$V_i = 6.53 - 3.25 = 3.28$ kip

$$M_u = 0.198(3)(4)\left(\frac{30}{2} - \frac{4}{2}\right) = 30.9 \text{ kip-ft}$$

$$M_d = (0.0535 + 0.025 + 0.020)(3)(4)\left(\frac{30}{2} - \frac{4}{2}\right)$$
$= 12.25 + 3.12 = 15.37$ kip-ft

$M_{max} = 30.9 - 15.37 = 15.52$ kip-ft

$$f_{pe} = \frac{101.3}{154} + \frac{101.3(2.89)(3.89)}{1224.5} = 1.588 \text{ ksi}$$

$$f_d = \frac{12.25(12)(3.89)}{1224.5} + \frac{3.12(12)(5.24)}{2307}$$
$= 0.552$ ksi

$$M_{cre} = \frac{2307}{5.24}\left(\frac{6\sqrt{5000}}{1000} + 1.588 - 0.552\right)$$
$= 643$ kip-in. $= 53.6$ kip-ft

$$\phi V_{ci} = 0.75\left[\frac{0.6\sqrt{5000}}{1000}(10.5)(9)\right] + 0.75\left[3.25 + \frac{3.28(53.6)}{15.52}\right]$$
$= 13.9$ kip > 6.53 kip ok

2.6 Strand Development

2.6.1 ACI Requirements

Section 12.9 of ACI 318-11 covers development length for prestressing strands. The topic has received considerable discussion in the literature.[14-21] The ACI expression currently is:

$$\ell_d = \left(\frac{f_{se}}{3000}\right)d_b + \left(\frac{f_{ps} - f_{se}}{1000}\right)d_b = (f_{ps} - 2/3 f_{se})d_b$$

A further requirement is that the development length shall be doubled when bonding of a strand does not extend to the end of the member and the precompressed tensile zone is allowed to be in tension at service loads.

The ACI expression for development length describes two bond mechanisms. The first is the transfer length, which is the bond length required to transfer the effective prestress after losses f_{se} to the concrete. This portion of the development length is:

$$\ell_t = \frac{f_{se}}{3000}d_b$$

With f_{se} equal to 150 ksi, the transfer length becomes $50d_b$, the length used for shear calculations. The second mechanism is for bond length after the steel stress increases above f_{se}. To develop the full stress in the strand corresponding to the nominal flexural strength f_{ps}, a bond length

Figure 2.6.1.1 Strand Development

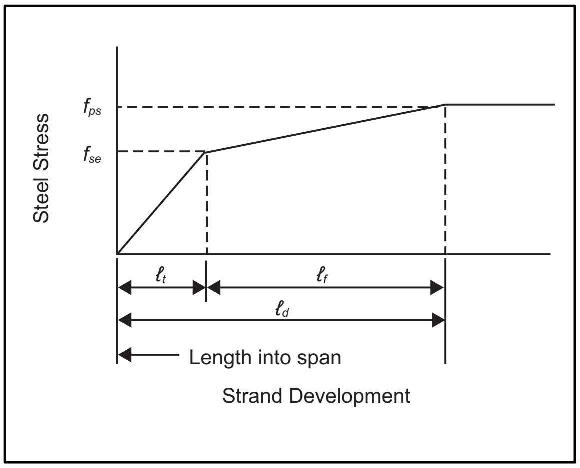
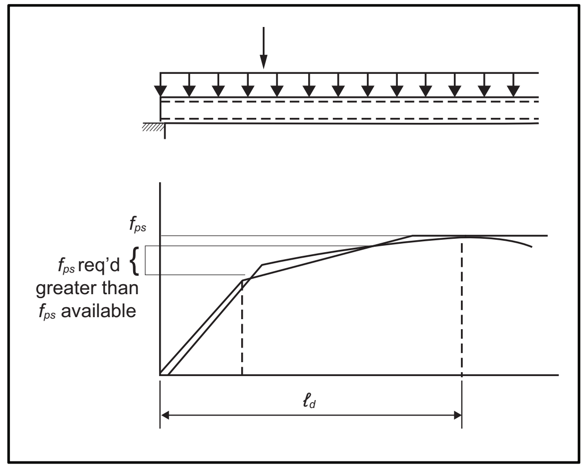


Figure 2.6.1.2 Critical Section in Development Length



in addition to the transfer length is required. The flexural bond length is expressed as:

$$\ell_f = \frac{f_{ps} - f_{se}}{1000} d_b$$

Figure 2.6.1.1 depicts the increase in steel stress along the development length of the strand.

Section 12.9.2 of ACI 318-11 limits investigation of development length to the section nearest to the end of the member where full design strength is required under the specified factored loads, except where bonding of one or more stands does not extend to the end of the member or where concentrated loads are applied within the strand development length. In nonprestressed concrete, the rate of moment increase must be considered in selecting reinforcing bar sizes. This consideration is also valid in prestressed concrete members. As shown in Fig. 2.6.1.2, with a steep rate of moment increase, critical sections may occur within the strand development length at less than maximum moment.

Demand on strand strength above f_{se} does not occur until after flexural cracking occurs. If flexural cracking occurs in the transfer length, the strand cannot accept additional stress, resulting in bond failure. Therefore, the limit on member flexural strength in the strand transfer length is the cracking moment. In the flexural bond length, strand stress can increase above f_{se}, but not to full f_{ps}. Therefore, there is additional flexural strength above the cracking moment but less than full nominal strength. If flexural cracking occurs at factored load in the flexural bond length, the maximum value for f_{ps} can be calculated as:

$$f_{px} = f_{se} + \frac{(x - \ell_t)}{\ell_f}(f_{ps} - f_{se}) = \frac{x}{d_b} + \frac{2}{3} f_{se}$$

where

x = the distance from the end of the member to the section of interest.

The nominal moment capacity is then calculated on the basis of this maximum strand stress.

Martin and Korkosz[21] suggest that with partially developed strand, the full concrete compressive failure strain will not be achieved. A strain compatibility analysis can be performed to determine the concrete strain that would be consistent with f_{px} and nominal strength can then be calculated using that strain.

When debonded strands are mixed with fully bonded strands, a similar strain compatibility analysis may be required in the flexural bond length for the debonded strands. In this case, nominal strength can be calculated in two ways:

1. Analyze section with all strands at the f_{px} for the debonded strands.
2. Analyze section with only fully bonded strands at their f_{ps} and ignore the debonded strands.

The greater of the two results would predict the nominal strength of the section.

For hollow core slabs, the strain compatibility analysis for partially developed strand will yield variable results as compared with a traditional approach where f_{px} is used with a full concrete strain of 0.003 in./in. If f_{px} is close to f_{se}, the strain compatibility analysis will predict moment capacity of about 85% of the traditional analysis. When f_{px} is 10% greater than f_{se}, the difference reduces to 5% or less. The additional complexity of the strain compatibility analysis would only seem warranted when flexural cracking is expected near the transfer point or when debonded strands are used.

If a critical section occurs in a region where strand is not fully developed, failure may be by bond slip. Such a failure resembles a brittle shear failure. ACI 318-11 prescribes reduced ϕ factors to be used in these situations. For flexural sections in pretensioned members where strand embedment is less than the development provided in ACI 318-11 Section 12.9.1.1, ACI prescribes a ϕ factor of 0.75 from the end of the member to the end of the transfer length (see ACI 318-11 Fig. R9.3.2.7[a], reproduced here as Fig. 2.6.1.3). From the end of the transfer length to the end of the development length, ϕ may be increased from 0.75 to 0.9 (Fig. 2.6.1.3). Where bonding of a strand does not extend to the end of the member, strand embedment is assumed to begin at the end of the debonded length. For more information, see section 12.9.3 of ACI 318-11 and Fig. 2.6.1.4 in this manual.

There are several aspects of a bond length discussion that are significant to hollow core slab design. In many framing schemes, there will be a requirement to use very short slabs to fill in an area. With fully developed strands, these slabs will normally have very large load capacities. However, capacity may be reduced because the strands might only be partially developed. For example, for a slab prestressed with 1/2 in. diameter, 270 ksi strands with f_{se} = 150 ksi and f_{ps} = 260 ksi:

$$\ell_d = \left(\frac{f_{se}}{3}\right)d_b + \left(f_{ps} - f_{se}\right)d_b$$

$$= \left(\frac{150}{3} + 110\right)(0.5) = 80 \text{ in.} = 6 \text{ ft-8 in.}$$

This hollow core slab would have to be two development lengths, or 13 ft-4 in. long in order to develop its full design strength. A shorter slab would have reduced capacity.

Hollow core slab systems are often required to carry concentrated or wall loads, which may affect the rate of moment increase near the member end. As discussed earlier in conjunction with Fig. 2.6.1.2, it is suggested that the transfer length and flexural bond length regions be investigated for reduced capacity when the moment gradient is high.

Figure 2.6.1.3 Variation of ϕ with distance from the free end of strand in pretensioned members with fully bonded strands

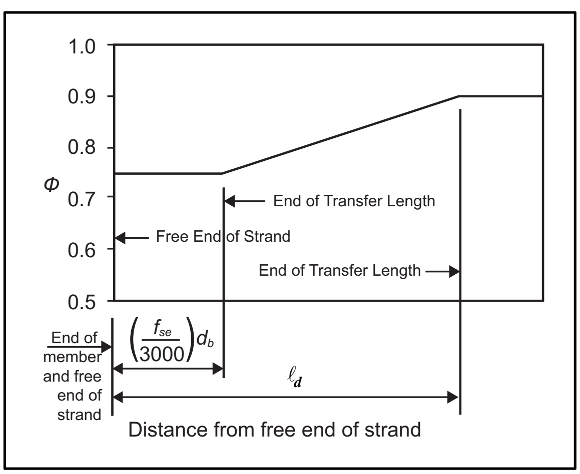
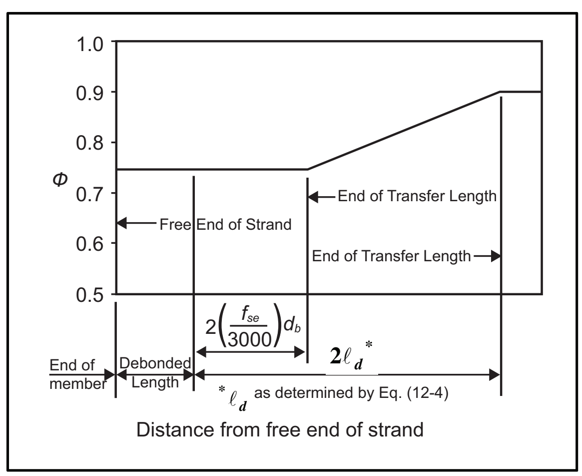


Figure 2.6.1.4 Variation of ϕ with distance from the free end of strand in pretensioned members with debonded strands where 12.9.3 applies



The development length equations in ACI 318 are based on testing conducted with members cast with concrete having normal water–cement ratios. As noted in the commentary to ACI 318, no-slump concrete requires extra precautions. Hollow core slabs produced with the extrusion process fall into this category. As originally presented by Anderson and Anderson[15] and reinforced by Brooks, Gerstle, and Logan[23], a measure of satisfactory bond is the free end strand slip in a member after it is cut to length. A limit on free end slip expressed as:

$$\delta_{all} = \frac{f_{se} f_{pi}}{6E_{ps}} d_b$$

has been suggested as a maximum free end strand slip for using the ACI development lengths. This expression approximates the strand shortening that would have to occur over the transfer length. For a ½-in.-diameter strand stressed initially to 189 ksi, the free end slip should not exceed about $^3/_{32}$ in. if the ACI transfer and development lengths are to be achieved.

When free end slip exceeds δ_{all}, the transfer length and the flexural bond length will exceed ACI values. Shear strength in the transfer length region and moment strength in the flexural bond length region will be reduced and the distance into the span where the full moment strength is developed will increase.

If the free end strand slip is known from quality control measurements, the member strength can be evaluated with consideration of extended transfer and flexural bond lengths. As a function of measured end slip, the transfer length and flexural bond length can be calculated for each strand as follows:

$$\ell_t = \frac{2\delta_s E_{ps}}{f_{pi}} \qquad \ell_f = \frac{6\delta_s E_{ps}\left(f_{ps} - f_{se}\right)}{f_{pi} f_{se}}$$

Shear strength can be evaluated by substituting the calculated transfer length for 50 d_b in evaluating the rate of increase of prestress. Flexural strength calculations are affected only by the extension of the strand development length and potential reduction of f_{px}. The strain compatibility analysis suggested by Martin and Korkosz for sections with partially developed strand becomes more complex, as there can be variations in development lengths within a given member.

Figure 2.6.1.5 illustrates the change in moment strength for the generic hollow core slab of Fig. 1.6.1 from normal slip to $^5/_{32}$ in. slip on all strands. In the upper diagram, the span length is 30 ft and there would be no change in slab strength for uniform load. In the lower diagram, the span is reduced to 25 ft and it is clear that the extended development length would result in reduced flexural strength, even with uniform load. End slip in excess of normal slip has a more significant effect in shorter slabs.

The following example demonstrates the use of the Martin and Korkosz strain compatibility analysis for partially developed strand and the use of free end slip for evaluating strength. The procedure is also valid with normal end slip by using the appropriate transfer and flexural bond lengths.

Example 2.6.1.1 Initial Strand Slip

Given the generic cross section of the hollow core slab defined in Fig. 1.6.1, calculate the design flexural strength, given the following:

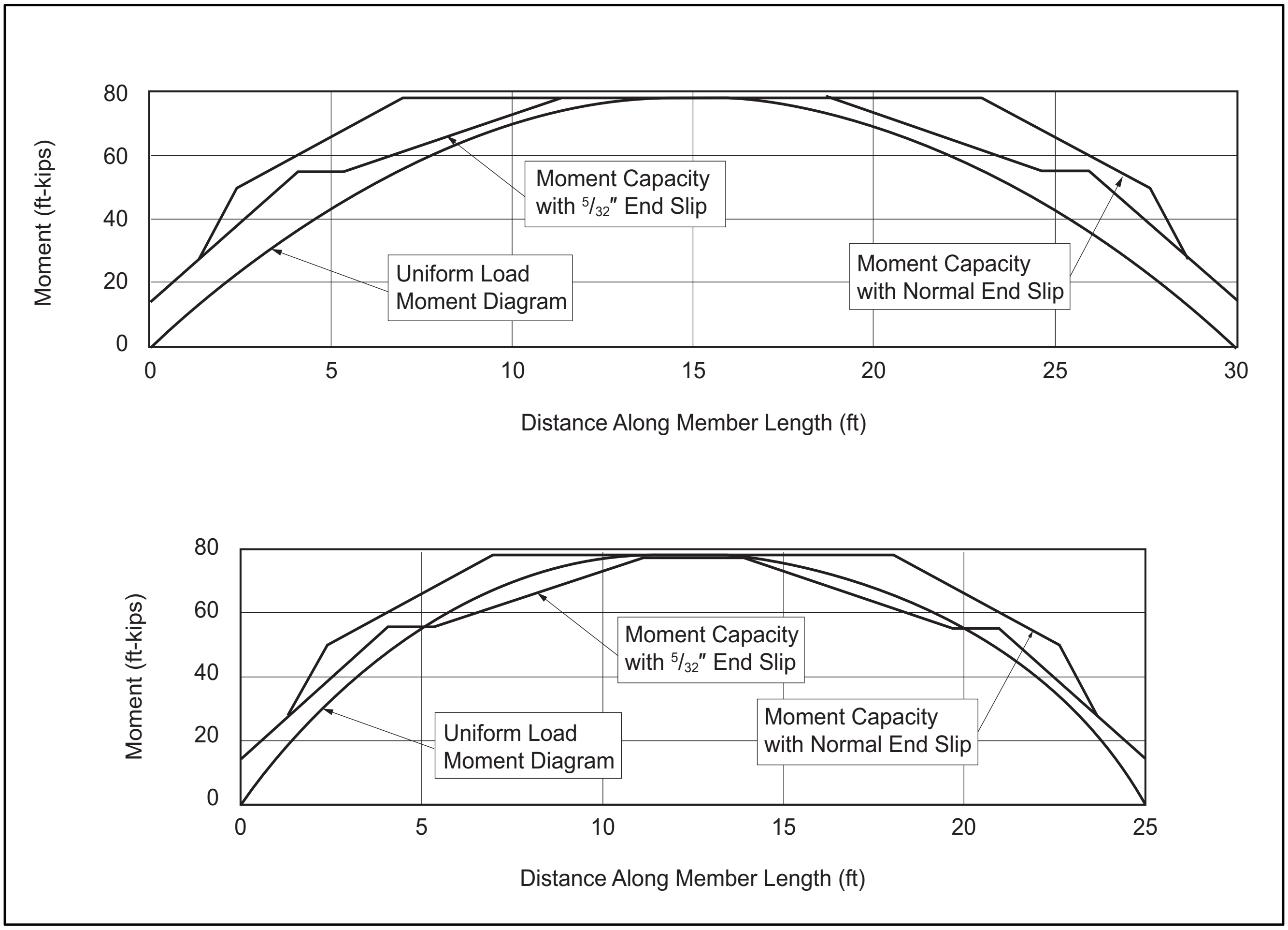
Prestressing steel:

4 ½-in.-dia., 270 ksi low-relaxation strands.
E_{ps} = 28,800 ksi
d_p = 7 in.
f'_c = 5000 psi
f_{pi} = 185 ksi
f_{se} = 163.4 ksi
f_{ps} = 267 ksi
δ_s = $^3/_{16}$ in. all strand

Solution:

$$\ell_t = \frac{2\delta_s E_{ps}}{f_{pi}}$$
$= 2(^3/_{16})(28,800)/185$
$= 58.4$ in.

Figure 2.6.1.5 Effect of end slip



$$\ell_f = \frac{6\delta_s E_{ps}\left(f_{ps} - f_{se}\right)}{f_{pi} f_{se}}$$

$$= \frac{6\left(^3/_{16}\right)(28{,}800)(267 - 163.4)}{185(163.4)}$$

$= 111$ in.

$\ell_d = 58.4 + 111 = 169.4$ in.

The minimum hollow core slab length required to achieve full flexural strength is 2(169.4)/12 or about 28 ft. Calculate flexural strength at 10 ft.

$$f_{px} = f_{se} + \frac{(x - \ell_t)}{\ell_f}(f_{ps} - f_{se})$$

$$= 163.4 + \frac{\left[10(12) - 58.4\right]}{111}(267 - 163.4)$$

$= 221$ksi

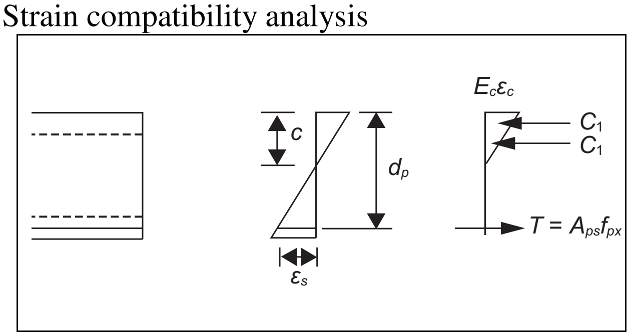
$A_{ps}f_{px} = 4(0.153)(221) = 135.3$ kip

Traditional analysis:

$$a = \frac{135.3}{0.85(5)(36)} = 0.88 \text{ in.}$$

$M_n = 135.3(7 - 0.88/2)/12 = 74$ kip-ft

Strain compatibility analysis



$\varepsilon_{ps} = \varepsilon_{se} + \varepsilon_t$

$\varepsilon_{se} = 163.4/28{,}800 = 0.00567$ in./in.

ε_{ps} = 222/28,800 = 0.00771 in./in.
ε_t = 0.00771 − 0.00567 = 0.00204 in./in.

Using trial and error for

$T = C$

Find

c = 2.25 in.
ε_c = 0.000966 in./in.

Concrete stress at top

f_{top} = 4030(0.000966) = 3.894 ksi

Concrete stress at top of core

$$= \frac{(2.25-1.25)}{2.25}(3.894) = 1.731\text{ksi}$$

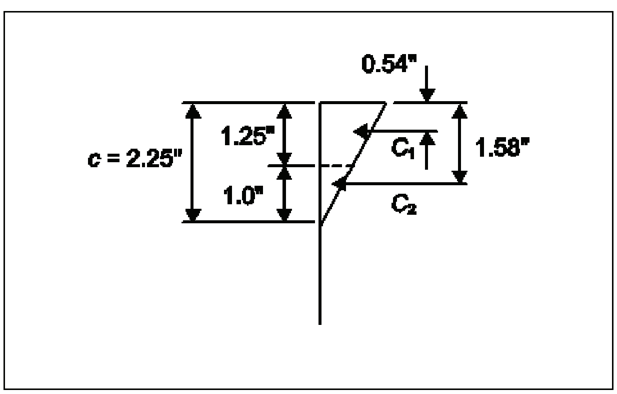
$$C_1 = \frac{3.894+1.731}{2}(1.25)(36) = 126.6 \text{ kip}$$

$$C_2 = \frac{1.731}{2}(10.5)(2.25-1.25) = 9.1 \text{ kip}$$

The locations of the concrete compressive forces C_1 and C_2 are determined assuming a triangular distribution of the concrete compressive stress.

$$d_{C1} = \frac{2(1.0/2.25)+1}{3(1+1.0/2.25)}(1.25) = 0.54 \text{ in.}$$

d_{C2} = 1.25 + 1.0/3 = 1.58 in.
M_n = (135.3(7 − 0.54) − 9.1(1.58 − 0.54)) /12
= 72 kip-ft



Chapter 3
SPECIAL DESIGN CONSIDERATIONS

3.1 General Information

The application of hollow core slabs as roof and floor deck members creates several situations for consideration in design that are either not completely covered by ACI 318-11[3] provisions or that involve consideration of production processes. This section presents information that may be used as a guideline for the situations described, but are not hard and fast rules. The criteria presented represent conservative practices and should be verified with local precast concrete producers. Published data relative to each situation are referenced. However, extensive in-plant testing has been conducted by hollow core slab producers, which may allow less conservative criteria to be used because of the unique characteristics of a particular slab.

3.2 Resistance for Non-uniform Loads

As demonstrated in Chapter 2 of this manual, hollow core slabs are designed as individual, one-way, simple-span slabs. When the hollow core slabs are installed and grouted together at the keyways, the individual slabs become a system that behaves similarly to a monolithic slab. A major benefit of slabs acting together is the ability to transfer forces from one to another. In most hollow core slab deck applications, non-uniform loading occurs in the form of line loads, concentrated loads, or load concentrations at openings. The ability of individual slabs to interact allows these load concentrations to be shared by several slabs. The ability to distribute loads among several slabs has been demonstrated in several published tests[24-30] and many unpublished tests.

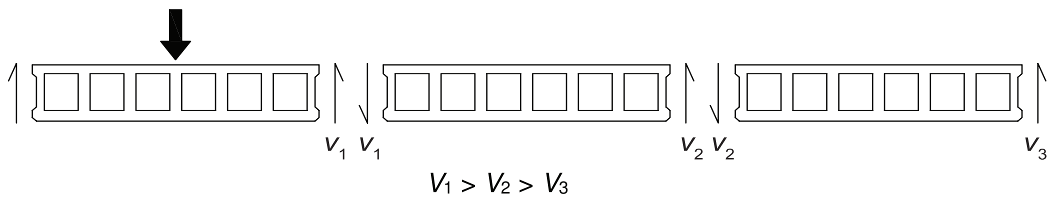
In many cases, load concentrations do not have to be carried by the hollow core slabs. For example, a header at a large opening may be supported directly by a foundation or vertical support element; a beam might be installed to directly carry a heavy concentrated load; or a heavy wall parallel to a slab span might be designed to carry its own weight or any load superimposed on the wall as a deep beam spanning between vertical supports. However, when such loads must be supported by the slab system, a method is required to decide how many slabs will contribute in carrying a given load in a given location. This section presents a design method that may be used when the hollow core slabs do have to support non-uniform loads.

3.2.1. Distribution Mechanisms

As load is applied to one hollow core slab in a system, the response of the slab system is to deflect and also twist if the load is not on the longitudinal centerline of the system. As the loaded slab edges try to move down, the interlock of the grout in the joints with the keyways formed in the slab edges causes adjacent slabs to deflect a similar amount. The flexural and torsional stiffnesses of the adjacent slabs reduce the deflection of the loaded slab from what might be expected if it acted alone. Shear forces are developed along the keyways and the loaded slab then gets some support from the adjacent slabs. Many times, shrinkage cracks will occur in the grouted joints at the interface between the grout and hollow core slab edge. This cracking does not impair the mechanism described previously because the configuration of the keyways in the slab edges still provides mechanical interlock even in the presence of a crack.

Shear forces transferred along keyways create two sets of forces that are normally not considered in hollow core slab design. The first is torsion, which develops because the shear on one edge of a given slab is different in magnitude than the shear on the opposite edge. As depicted in Fig. 3.2.1, the keyway shear reduces as the distance from the load increases. This torsion causes shear stress in the slabs that is additive to the direct shear stress.

Figure 3.2.1



The second set of forces is induced because the system tends to behave as a two-way slab. Transverse bending moments occur because of the edge support provided by adjacent slabs. The result is transverse tensile stress developed in the bottom of the slab and compressive stress in the top. Hollow core slabs do not contain transverse reinforcement. Therefore, transverse tensile stresses must be resisted by plain concrete. The magnitude of load concentration causing the transverse tension must be limited to preclude a splitting failure (see Section 3.2.2).

Several factors affect the ability of a hollow core slab system to distribute loads to adjacent slabs. As the width of an assembly of slabs gets narrower than the span length, a reduction in the number of slabs contributing to the support of a concentration of load occurs. This occurs because the freedom to deflect and twist at the unsupported edges of the system becomes more significant. A second factor is the spacing of the slab joints. With hollow core slabs available in widths ranging from 2 ft to 8 ft, some differences in load distribution behavior can be expected. Finally, the span length affects the number of slabs that contribute to load distribution. As span length changes for a wide system, the interaction of flexural and torsional stiffnesses changes. For longer spans, flexural stiffness reduces relative to torsional stiffness, resulting in relatively less slab rotation and less transverse curvature. The result is that more slabs can contribute to load resistance on longer spans as long as the system is wide relative to its length.

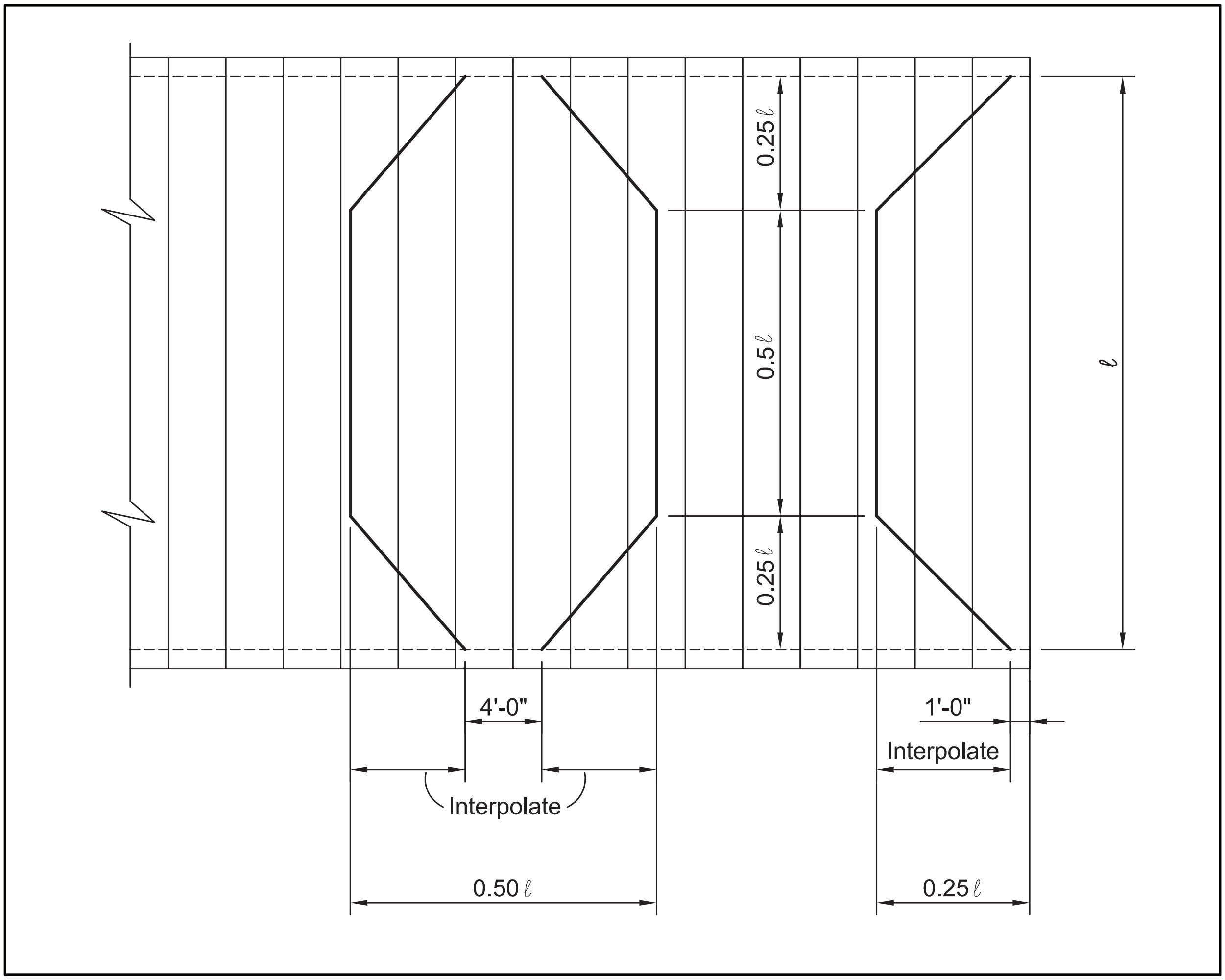
3.2.2 Design Guidelines

ACI 318-11 recognizes the load transfer capabilities of hollow core slabs in Section 16.3.1. That section requires that distribution of forces be established by analysis or test. The guidelines presented here are based on extensive, full-scale testing of a specific hollow core slab system. Additionally, a comparison of these guidelines with an analytical study has been done. Therefore, these guidelines should satisfy the requirement of ACI 318-11, Section 16.3.1.

The two basic design parameters considered for hollow core slabs are flexure and shear. Design for flexure is straightforward, with the effective load-resisting width being a function of the span length. Conversely, shear design is complicated by torsions developed in the system. If torsion is not used as a design parameter, direct shear must be modified to reflect the increase in shear stress due to the torsion.

Figure 3.2.2 depicts a method of establishing an effective resisting section for any type of load to be distributed between hollow core slabs. In the midspan regions, the effective width is defined as a function of span length. At the supports, the effective width is defined as an absolute width. The width at the support is restricted to account for shear stresses due to torsion. Use of these resisting sections will result in prediction of peak values of moment and shear. That is, the effective width concept is simply a mechanism used to determine the maximum design moments and shears rather than a depiction of the actual load path through the system.

Figure 3.2.2 Effective resisting width of slab for load anywhere along span

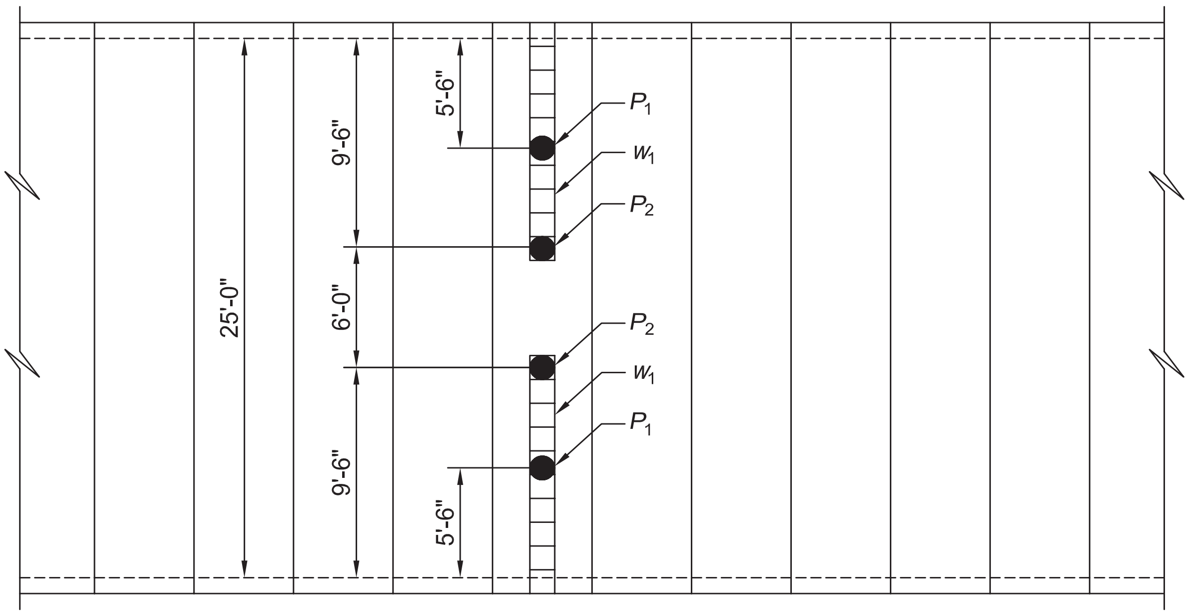


The performance of hollow core slab systems indicates that shear and moment might affect additional slabs. For example, for a load located some distance from a free edge, the peak moment due to that load can be predicted by assuming the load is resisted by a width equal to 0.50ℓ. In reality, in flexure, a total width equal to 85% to 90% of the span length might have some additional moment attributable to that load. In shear, the 1 ft effective section at the support at a free edge may be used to predict the peak shear, but, because of torsion, the total reaction due to an edge load will not actually be concentrated in the edge 1 ft.

Several limitations should be recognized for Fig. 3.2.2.

1) As the width of the system becomes narrower than the span length, the effective resisting widths will become narrower.
2) For extremely large span-depth ratios (in excess of approximately 50), the effective section at midspan may be reduced by 10 to 20%.
3) For spans less than about 10 ft, the effective width at the support may become narrower.
4) Local load concentrations can cause longitudinal splitting failures due to transverse bending in the system. Punching shear failures can also occur. The magnitude of concentrated loads must be limited to preclude such failures. These limits are best established by test for each hollow core slab system.

Figure 3.2.3



D = 10 lb/ft^2 w_{1D} = 650 lb/ft P_{1D} = 500 lb P_{2D} = 1000 lb

L = 40 lb/ft^2 w_{1L} = 1040 lb/ft P_{1L} = 1000 lb P_{2L} = 3000 lb

The concept of using an effective resisting section is subtly different from the traditional concept of load distribution width. Traditionally, loads have been divided by distribution widths for design. Using an effective resisting section means that a given load is resisted by a varying width depending on the location of the section being investigated in the span. Shears and moments are divided by the width of the effective resisting section rather than the loads. This is best illustrated by example.

Example 3.2.1 General Case

Using the generic hollow core slab shown in Fig. 1.6.1 and the plan shown in Fig. 3.2.3, determine the slab design loads. Hollow core slab weight is 53.5 lb/ft^2.

Solution:

See Table 3.2.1 for the first four steps.

Step 1: Evaluate the shear and moment diagrams for the non-distributable loads.

$$w_u = 1.2D + 1.6L$$

$$w_u = 1.2(53.5 + 10) + 1.6(40) = 140 \text{ lb/ft}^2$$

$$V_x = w(\ell/2 - x) = 0.140(25/2 - x)$$

$$M_x = w\,\frac{x}{2}\,(\ell - x) = \frac{0.140\,x}{2}(25 - x)$$

Step 2: Evaluate the shear and moment diagrams for the non-distributable loads.

w_{1u} = 1.2 (650) + 1.6 (1040) = 2444 lb/ft
P_{1u} = 1.2 (500) + 1.6 (1000) = 2200 lb
P_{2u} = 1.2 (1000) + 1.6 (3000) = 6000 lb

Step 3: Evaluate the effective width along the span.

At support

DW = 4.0 ft

Table 3.2.1 Shears and moments for Example 3.2.1

x, ft	Non-distributed loads		Distributable loads		Effective Width DW_x	Final	
	V_{ux}	M_{ux}	V_{ux}	M_{ux}		V_u (k/ft)	M_u (k-ft/ft)
0	1.75	0.00	31.42	0.00	4.00	9.60	0.00
h/2	1.70	0.58	30.60	10.34	4.45	8.58	2.90
1	1.61	1.68	28.97	30.20	5.36	7.02	7.31
2	1.47	3.22	26.53	57.95	6.72	5.42	11.84
3	1.33	4.62	24.09	83.26	8.08	4.31	14.92
4	1.19	5.88	21.64	106.12	9.44	3.48	17.12
5	1.05	7.00	19.20	126.54	10.80	2.83	18.72
6	0.91	7.98	14.55	143.42	12.16	2.11	19.77
7	0.77	8.82	12.11	156.75	12.50	1.74	21.36
10	0.35	10.50	0.00	179.39	12.50	0.35	24.85
11	0.21	10.78	0.00	179.39	12.50	0.21	25.13
12.5	0.00	10.94	0.00	179.39	12.50	0.00	25.29

At $0.25\ell = 0.25\,(25) = 6.25$ ft
$DW = 0.5\ell = 0.5\,(25) = 12.5$ ft

Between $x = 0$ and $x = 6.25$ ft

$$DW = 4 + \frac{x}{6.25}(12.5 - 4)$$
$$= 4 + 1.36x$$

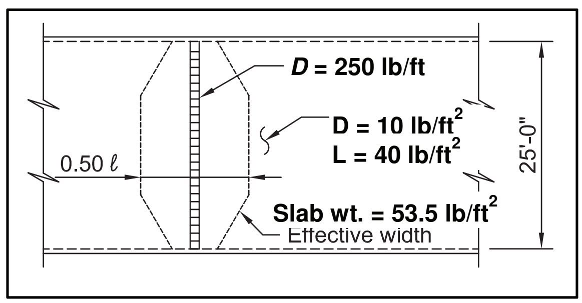
Step 4: Divide the shears and moments from step 2 by the effective width from step 3 and add to the shears and moments in step 1.

Step 5: Design the slabs for the web shear, inclined shear, and moments obtained from step 4.

The solution for the general case where the shears and moments are calculated at intervals along the span is best suited for use with a computer. The information could also be used to calculate shear strength at the same time.

For many cases, a general solution is not necessary. Simplifying shortcuts can be used to shorten the design process. Consider the case where shear is known not to be a problem.

Example 3.2.2



Given the system shown, determine the design load.

Solution:

Check flexure only as shear is judged not to be critical

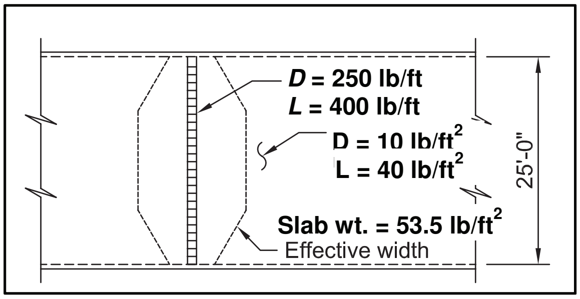
From Fig. 3.2.2, the effective width resisting the line load is $0.50\ell = 0.50\,(25) = 12.5$ ft. Determine the design superimposed load.

$$w = 40 + 10 + \frac{250}{12.5}$$
$$= 70 \text{ lb/ft}^2$$

Using the generic hollow core slab load table in Fig. 1.6.1, select an 8-in.-thick slab with 4 $^3/_8$-in.-diameter strands.

If it is not known whether shear is critical, simple iterative checks may be made.

Example 3.2.3



Given the system shown, select a generic hollow core slab from Fig. 1.6.1 to support the loads shown.

Solution:

Make preliminary selection based on flexure:

Superimposed w

$$= 10 + 40 + \frac{(250+400)}{(0.5)(25)} = 102\,\text{lb/ft}^2$$

Select $4^{7}/_{16}$-in.-diameter, 270 ksi low-relaxation strands from Fig. 1.6.1

First shear check

effective width at support, $DW = 4$ ft

$$w_u = 1.2(10+53.5) + 1.6(40) + \frac{[1.2(250)+1.6(400)]}{DW}$$

$$= 140 + \frac{940}{DW}$$

Using $DW = 4$ ft

$w_u = 140 + 940 / 4.0 = 375$ lb/ft^2

Check shear based on this load and find

at $h/2$ $V_u = 4.56$ kip/ft and

$\phi V_{cw} = 5.34$ kip/ft **ok**

at 4.0 ft $V_u = 3.19$ kip/ft and

$\phi V_{ci} = 2.95$ kip/ft **No Good**

Second shear check

Inclined shear did not check at 4 ft into the span, so determine the effective width at 4 ft, recalculate the distributed load, and check shear.

At $\ell/4$, $DW = 0.5\ell = 0.5\,(25) = 12.5$ ft

At support, $DW = 4$ ft

Interpolate at 4 ft into span

$$DW = \frac{4}{25/4}(12.5-4) + 4 = 9.44\,\text{ft}$$

$w_u = 140 + 940/DW$

$= 140 + 940/9.44$

$= 240$ lb/ft^2

Again check shear at 4 ft and beyond and find:

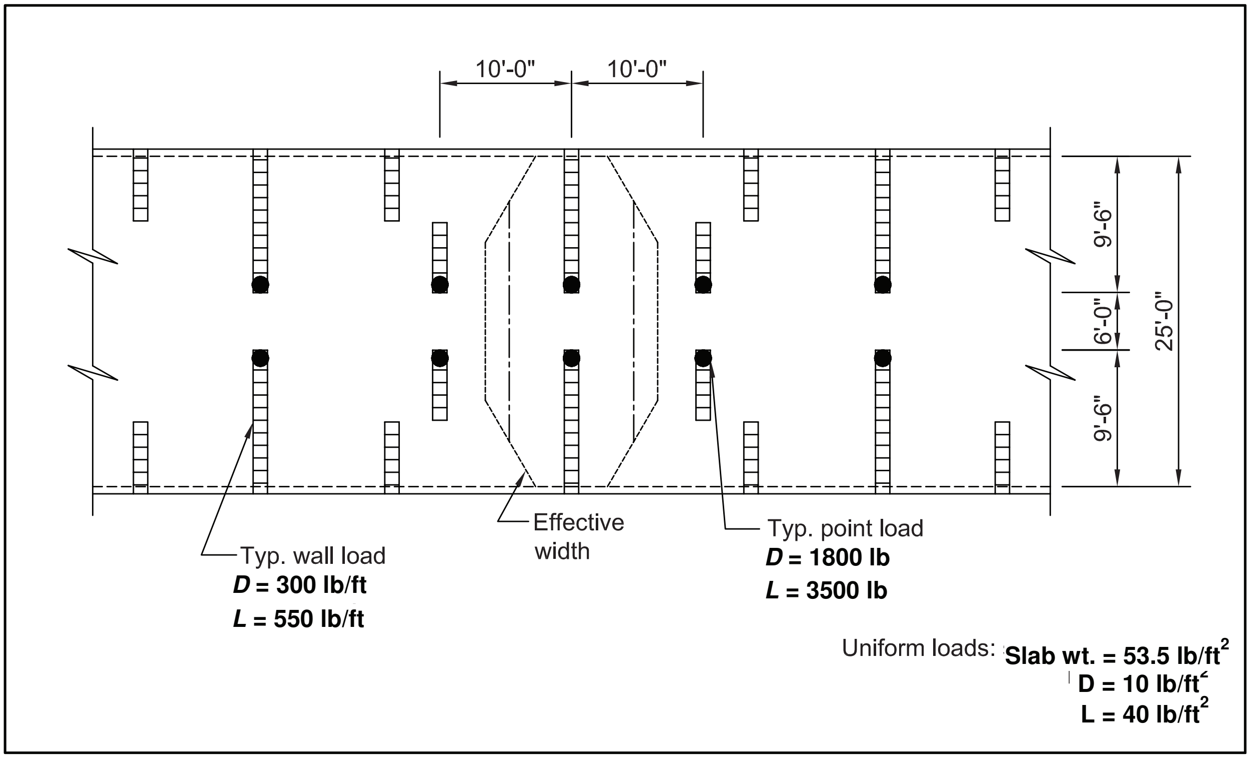
$\phi V_{ci} > V_u$ at all points.

Therefore, the shear check is complete and the slab is adequate.

To summarize the steps taken to check shear in Example 3.2.3, distributable loads were divided by the effective width at the support to make a conservative shear check. If shear along the span is found to be satisfactory, no additional steps are required and the shear check is complete. If shear in the span is found to be inadequate at some point, the effective width at that point is used to calculate a new load, which will then be conservative for points farther into the span. Shear is checked again. This iterative approach is used until all points farther into the span are adequate for shear. If shear capacity is sufficient for a given situation, generally no more than three iterative calculation cycles will be required.

A combination of loads will be used to demonstrate this method in the following example.

Example 3.2.4



Example 3.2.4

Given the center bay of an apartment building as shown, design for the applied loads using the generic hollow core slab shown in Fig. 1.6.1.

Solution:

Select a preliminary slab section based on flexure:

$DW = 0.50\ell = 0.5\ (25) = 12.5$ ft

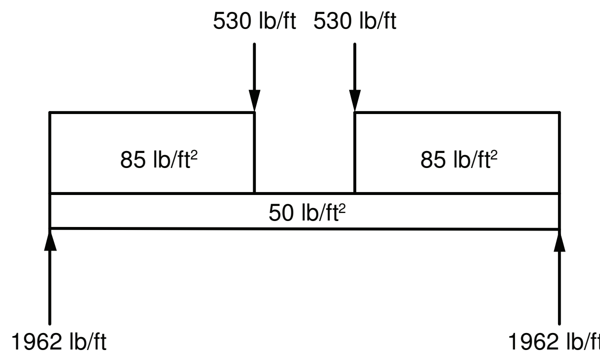
Because wall and point loads are spaced closer than 12.5 ft, conservatively use 10 ft spacing of loads as DW.

At design strip:

$$\text{Point load} = \frac{(1800+3500)}{10} = 530\ \text{lb/ft}$$

$$\text{Parallel wall} = \frac{(300+550)}{10} = 85\ \text{lb/ft}^2$$

Uniform load = 10 + 40 = 50 lb/ft^2



$M_{superimposed} = 12{,}777$ lb-ft/ft

$$w_{equivalent} = \frac{8M_{superimposed}}{\ell^2}$$

$$= \frac{8(12777)}{25^2}$$

$= 164$ lb/ft^2

Select $4^1/_2$-in.-diameter, 270 ksi, low-relaxation strands with a capacity of 164 lb/ft^2 at 25 ft.

Check shear

For design strip, including slab weight.

$$w_u = 1.2\ (\ 10\ +\ 53.5\ ) + 1.6\ (40)$$
$$+\ \frac{(1.2 \times 300 + 1.6 \times 550\ )}{DW}$$
$$= 140 + 1240\ /\ DW$$
$$P_u = \frac{(\ 1.2 \times 1800 + 1.6\ \times 3500\ )}{DW}$$
$$= 7760\ /\ DW$$

Start at support where effective width is 4 ft

w_u = 140 + 1240 / 4 = 450 lb/ft^2
P_u = 7760 / 4 = 1940 lb/ft

Obtain the following results:

x	$h/2$	1.25 ft	2.00 ft	2.75 ft	3.5 ft
V_u (kip/ft)	6.48	6.07	5.74	5.40	5.06
ø V_n (kip/ft)	5.46	6.88	6.83	5.06	4.02

Note that web shear at $h/2$ does not work. No other modifications can be made to adjust the shear calculation. Shear enhancement is required in the form of stirrups, solid cores, greater concrete strength, or using a deeper section.

Proceed to check inclined shear, which was not adequate at 2.75 ft.

Recalculate effective width at 2.75 ft as:

$$= \frac{2.75}{0.25\ell}(0.5\ell - 4) + 4$$
$$= \frac{2.75}{6.25}(12.5 - 4) + 4$$
$$= 7.74 \text{ ft}$$

w_u = 140 + 1240 / 7.74 = 300 lb/ft^2
P_u = 7760 / 7.74 = 1003 lb/ft

Obtain the following results:

x	2.75 ft	3.50 ft	4.25 ft	5.00 ft	5.25 ft
V_u (kip/ft)	3.45	3.23	3.00	2.78	2.70
ø V_n (kip/ft)	5.05	4.01	3.33	2.85	2.71

Inclined shear is now adequate to a distance of 5.25 ft into the span. Recalculate the effective width at 5.25 ft.

$$\frac{5.25}{0.25\ell}(0.5\ell - 4) + 4$$
$$= 11.14 \text{ ft}$$

Note that loads are located only 10 ft apart, which means that design strips would start to overlap. For this case, the maximum effective width might be used as the distance between loads, or 10 ft, rather than $0.5\,\ell$.

w_u = 140 + 1240 / 10 = 264 lb/ft^2

P_u = 7760 / 10 = 776 lb/ft

With these loads, it is found that $V_u < \phi V_n$ for the balance of the span. Therefore, the selected slab is adequate except for the shear enhancement required for web shear, as previously noted.

3.3 Effect of Openings

Openings may be provided in hollow core slab systems by saw cutting after a deck is installed and grouted, by shoring and saw cutting, by forming or sawing the openings in the plant, or by installing short slabs with steel headers. Some typical header configurations are shown in Section 6.7. In laying out openings for a project, the least structural effect will be obtained by orienting the longest dimension of an opening parallel to a span, coring small holes to cut the fewest prestressing strands, or, when several openings must be provided, aligning the openings parallel to the span to cut the fewest number of prestressing strands.

For hollow core slab design, openings cause load concentrations, which may be distributed over the slab system as discussed in Section 3.2. As with non-uniform loads, openings cause torsion in the slabs. Therefore, determining the shear adequacy of the slab must consider the effects of torsion as well as the direct shear on the section. In flexure, the primary consideration is the length of strand embedment available from the end of an opening to the point of maximum moment.

Figure 3.3.1 Effects of openings

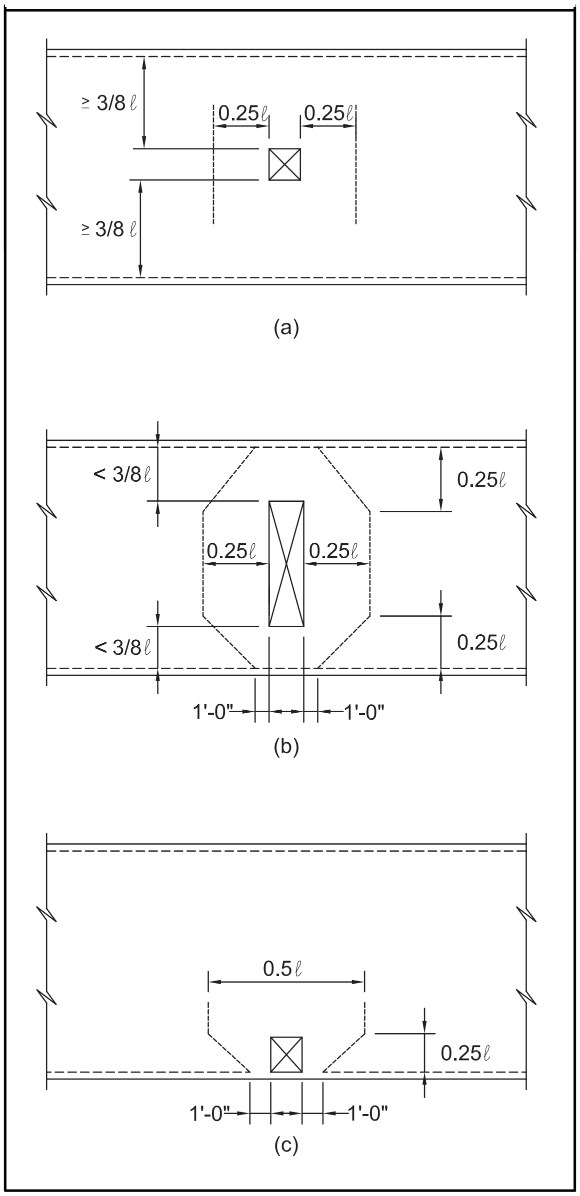


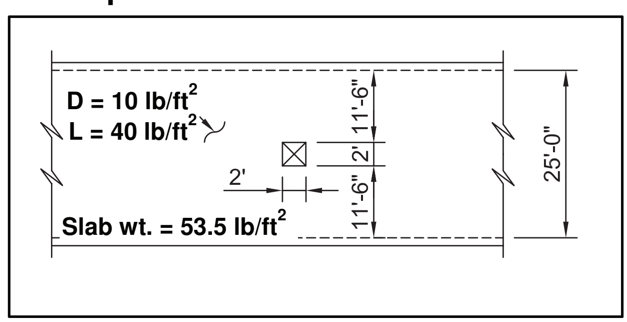
Fig. 3.3.1 shows some general opening locations with suggested interpretations of the effective resisting slab width described in Section 3.2. Local hollow core slab producers may have information that would allow different design approaches for their particular slab.

Figure 3.3.1(a) depicts a relatively small opening located at midspan. In flexure, slabs within 0.25ℓ on each side of the opening can resist the load from the short slabs. As a guideline, if an end of the opening shown is not closer to the support than $^3/_8\ell$, there will be no special considerations for shear design with only uniform loads. When non-uniform loads are superimposed near an opening, the effective resisting section shown in Fig. 3.2.2 would then be used for determining the resistance to those non-uniform loads.

Figure 3.3.1(b) shows a similar condition where an opening is located with an end closer to the support than $^3/_8\ell$. In this case, shear is considered as though the opening created a free edge. That is, load from the short slabs or opening will be transmitted as an edge load to the adjacent slabs. The resulting torsion on the adjacent slabs requires the use of a reduced effective width at the support if torsional shear stresses are not directly calculated.

Figure 3.3.1(c) depicts an extreme condition where an opening is located at the end of a span. Again, the reduced effective resisting section adjacent to the opening is required to reflect torsional shear stresses. When considering flexure, an opening that extends less than $0.125\,\ell$ and 4 ft from the slab end into the span may be neglected. However, a slab with an opening may have a reduced capacity when the strand embedment length is less than the full, required development length. When non-uniform loads are superimposed near an opening at the end of a slab, these loads should be considered as being at a free edge for shear calculations.

Example 3.3.1



Given the hollow core slab system shown, select a generic slab from Fig. 1.6.1 to resist the applied loads considering the opening.

Solution:

Check the proximity of the opening to the support.

$^3/_8\,\ell = 0.375\ (25) = 9.38$ ft

$11.5 \text{ ft} > 9.38 \text{ ft}$

Therefore, no special shear considerations are required.

Distribute load from strip with opening:

Superimposed w = 10 + 40 = 50 lb/ft^2

The load on the strip containing the opening is:

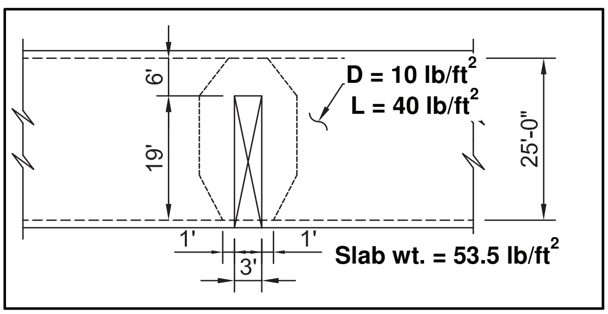
w = 2 (10 + 40 + 53.5) = 207 lb/ft

Distributing half of the strip load to each side of the opening:

$$w = 50 + \frac{207/2}{0.25\ell}$$
$$= 50 + \frac{207/2}{0.25(25)}$$
$$= 67 \text{ lb/ft}^2$$

From Fig. 1.6.1, select 4 $^3/_8$-in.-diameter, 270 ksi, low-relaxation strands.

Example 3.3.2



Given the floor system shown, select a generic hollow core slab from Fig. 1.6.1 to resist the given loads.

Solution:

The ends of the opening are closer than $^3/_8\,\ell$ to the support on both ends. Therefore, consider the opening as though it were a free edge.
The load on the strip containing the opening is:

w = (10 + 40 + 53.5)(3 ft)
= 311 lb/ft

For flexure and preliminary slab selection, use an effective width of 0.25 ℓ to each side of the opening.

$$w = 10 + 40 + \frac{311/2}{0.25(25)}$$
$$= 75 \text{ lb/ft}^2$$

Try 4 $^3/_8$-in.-diameter, 270 ksi, low-relaxation strands.

Check shear:

The effective width at support = 1 ft each side

$$w_u = 1.2(10+53.5)+1.6(40) + \frac{3[1.2(10+53.5)+1.6(40)]/2}{DW}$$
$$= 140 + 210/DW$$

where
DW = effective width on each side

w_u = 140 + 210 / 1 = 350 lb/ft^2

Using this load, obtain:

x	h/2	0.75 ft	1.50 ft	2.25 ft	3.00 ft
V_u (kip/ft)	4.25	4.12	3.86	3.59	3.33
$\phi\, V_n$ (kip/ft)	5.21	5.69	6.12	4.20	3.24

Shear resistance is adequate to 2.25 ft into span. Modify effective width at 2.25 ft.

$$DW = \frac{2.25}{0.25\ell}(0.25\ell - 1) + 1$$
$$= \frac{2.25}{6.25}(6.25 - 1) + 1$$
$$= 2.89 \text{ ft}$$

w_u = 140 + 210 / DW
= 140 + 210 / 2.89
= 213 lb/ft^2

Shear is found to be adequate at 2.25 ft and all points further into span. Use 4$^3/_8$-in.-diameter, 270 ksi, low-relaxation strands.

3.4 Continuity

Hollow core slabs are normally designed as part of a simple span system. However, continuity over supports can be achieved by placing reinforcing steel in the grouted keyways, in a composite structural topping, or in concrete-filled cores. Within limits, the result will be a better control of superimposed load deflections and a lower requirement for positive moment capacity.

With reinforcing steel located in either a composite topping or in cores, elastic moments with allowance for negative moment redistribution determine the amount of reinforcing required. Because of the relative efficiencies of positive prestressing steel and negative mild-steel reinforcement, it is difficult to economically justify a continuous system design.

When reinforcement such as structural integrity ties or diaphragm connections are required at supports, the reinforcement ratios are generally very low, and therefore, little moment capacity can develop. While this reinforcement may be considered when calculating service-load deflections, it is recommended that the full, simple-span, positive-moment capacity be provided for strength design unless moment-curvature relationships existing at the supports at ultimate loads are known.

One situation where the designer may consider a reduction in the positive moment requirements is when the rational design procedure is used to develop the required fire rating of the system. In this case, a limit analysis approach would be reasonable.

The negative moment reinforcement, which is unaffected by fire loads, can develop its full yield moment potential and effectively provide a plastic hinge at the support. As a result, the positive moment at midspan may be correspondingly reduced. A detailed discussion of this is presented in Section 7.6.4.

3.5 Cantilevers

Cantilever design for hollow core slabs differs from that for conventional precast concrete members because of the production procedures used for hollow core slabs. Guidelines noted here are conservative and may be exceeded depending on the specific product used.

Because long-line casting beds are frequently used for the production of hollow core slabs, top prestressing strands may be economical only when the full bed capacity is used. Even then, using substantial amounts of prestressing strand may be inefficient because of debonding requirements. The local precast concrete producer must determine the economics of using top strands.

When top strands are used, the length of the cantilever is usually not sufficient to fully develop a strand. A reduced value for f_{ps} is required and the design procedures given in section 2.6 should be used. In dry-cast systems, the bond of the top strands to the concrete may be less than desired, so a further reduction in f_{ps} is required. This reduction may be substantial and each precast concrete producer should be consulted for top strand bond performance.

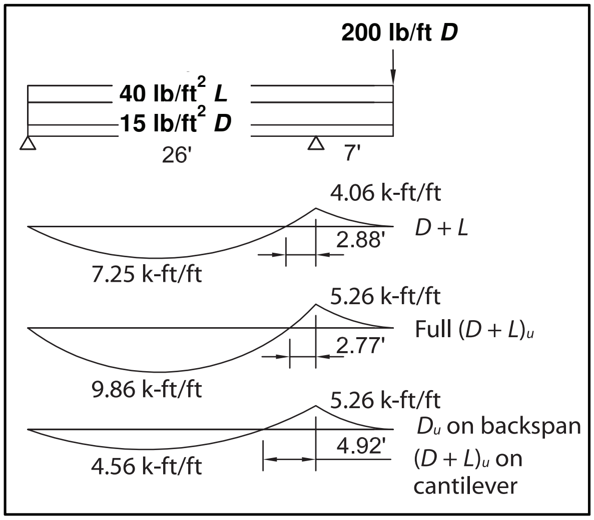
When top strands are not economical, non-prestressed reinforcement may be placed in the cores or, in the case of a wet-cast product, directly in the slab. This is generally done while the slab concrete is still fresh so that the fill concrete may bond with the slab. The reinforcement is selected based on conventional design with due consideration given to development length.

With either top strands or reinforcing bars, it may be necessary to debond portions of the bottom prestressing strand in the cantilever region to help minimize the top tension under service loads. However, because hollow core slabs are generally handled near their ends and not at the in-place support points, it is recommended that a portion of the bottom strands remain bonded for the full length of the slab. Not all producers have the ability to debond bottom strands. This could potentially limit the allowable cantilever length or the cantilever load capacity.

It is desirable to limit the service-level tensile stresses in cantilevers so that the uncracked section properties may be used to more accurately predict deflections. The tensile stress limit may vary for different hollow core slab systems used. For example, the practice with some dry-cast systems is to limit tensile stresses to 100 psi. In other dry-cast systems and in wet-cast systems, the limit may be raised to $6\sqrt{f'_c}$. The tension limit is basically a function of a precast concrete producer's past experience.

As a general rule, cantilever lengths between 6 and 12 times the slab thickness will be achievable, depending on the superimposed load and individual precast concrete producer's capabilities.

Figure 3.5.1



Example 3.5.1 Cantilever Design

Using the generic hollow core slab section defined in section 1.6, design for the conditions shown in Fig. 3.5.1.

Solution:

From the load table in Fig. 1.6.1, select 4 $^3/_8$-in.-diameter, 270 ksi strands as the primary reinforcement. Try 2 $^3/_8$-in.-diameter, 270 ksi strands at d_p = 7 in. as cantilever reinforcement. Assume 15% losses and 70% initial stress.

Check stresses at cantilever:

Bottom strands:

$$P_e = A_{ps}f_{pi}(1 - \text{loss})$$
$$= 4(0.085)(0.7)(270)(1 - 0.15)$$
$$= 54.6 \text{ kip}$$

$$f_{top} = 54.6\left(\frac{1}{154} - \frac{2.89(4.11)}{1224.5}\right)$$
$$= -0.176 \text{ ksi (tension)}$$

Top strands:

$$P_e = 2(0.085)(0.7)(270)(1 - 0.15)$$
$$= 27.3 \text{ kip}$$

$$f_{top} = 27.3\left(\frac{1}{154} + \frac{3.11(4.11)}{1224.5}\right)$$
$$= 0.463 \text{ ksi}$$

Applied moment:

$$M_{service} = 4.06(3 \text{ ft}) = 12.18 \text{ kip-ft}$$

$$f_{top} = \frac{12.18(12)(4.11)}{1224.5}$$
$$= -0.491 \text{ ksi (tension)}$$

Net tension with fully bonded bottom strands:

$$f_{top} = -0.176 + 0.463 - 0.491$$
$$= -0.204 \text{ ksi}$$

Allow $6\sqrt{5000} = 0.424\,\text{ksi}$ **ok**

Note that some of the bottom strands could have been debonded for the length of the cantilever if the top tensile stresses had exceeded a desirable level.

Stresses in backspan:

Because the backspan is long in this example, stresses will not be critical in the backspan region of the hollow core slab. When the backspan is short relative to the cantilever length, stresses may require a check in the backspan to determine the length of bonding of the top strands.

Ultimate strength:

At the cantilever, strain compatibility will generally show that the bottom strands may be ignored in determining the nominal moment capacity. When the bottom prestress is very heavy or the bottom strands are located high in the section, a strain compatibility analysis should be performed considering both strand layers.
For this example, assume the bottom strands may be ignored.

$$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1}\rho_p \frac{f_{pu}}{f'_c}\right]$$

$$f_{ps} = 270\left[1 - \left(\frac{0.28}{0.80}\right)\left(\frac{2\,(0.085)(270)}{(36)(7)(5)}\right)\right]$$
$$= 267 \text{ ksi}$$

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

$$a = \frac{2(0.085)(267)}{(0.85)(5)(36)}$$
$$= 0.296 \text{ in.}$$

$$\phi M_n = \phi A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)$$

$$\phi M_n = \frac{0.9}{12}(2)(0.085)(267)\left(7 - \frac{0.296}{2}\right)$$
$= 23.29$ kip-ft

$M_u = (5.26)(3 \text{ ft})$
$= 15.79$ kip-ft

Per Section 18.8.2 of ACI 318-11, the reinforcement must be adequate to develop a factored load of at least 1.2 times the cracking load. The minimum reinforcement check is required at the critical section, so based on $\phi M_n = 23.29$ kip-ft:
Find factored load:

$$\frac{\phi w_n \ell^2}{2} = \phi M_n$$

$$\frac{\phi w_n 7^2}{2} = 23.29$$

$$\phi w_n = 0.95 \text{ kip/ft}$$

Find cracking load:

$$-\frac{w_{cr}\ell^2}{2S_t} = -7.5\sqrt{f_c'} + f_{top}$$

$$-\frac{w_{cr}7^2(12)}{2(297.9)} = -\frac{7.5\sqrt{5000}}{1000} + 0.176 - 0.463$$

$w_{cr} = 0.83$ kip/ft

$$\frac{\phi w_n}{w_{cr}} = \frac{0.95}{0.83} = 1.15 < 1.20$$

Add one #4 top bar per slab

Check length of top strand to be bonded:

$\ell_{\,available} = (7)(12) = 84$ in.
$\ell_d = (f_{ps} - 2/3 f_{se})d_b$
$= [267 - 2(0.7)(0.85)(270)/3]0.375$
$= 60$ in. < 84 in.

Therefore, the strand is fully effective in the cantilever. If the development length is found to be greater than the length available, the moment capacity will have to be recalculated by the procedures discussed in section 2.6.

Bond of the top strands in the backspan must be long enough to develop the f_{ps} required in the cantilever design. The top strands should also be bonded for a distance of the greater of 12 strand diameters, $12d_b$, or the effective depth d past the inflection point under the worst load condition or $^1/_6$ of the backspan. For this example, a bonded length of 67 in. is required.

Alternate design: Provide mild-steel reinforcement instead of top prestressing strands.

Try 2 #5 grade 60 bars at $d = 7$ in.

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$$a = \frac{2(0.31)(60)}{(0.85)(5)(36)}$$
$= 0.243$ in.

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right)$$

$$\phi M_n = \frac{0.9}{12}(2)(0.31)(60)\left(7 - \frac{0.243}{2}\right)$$
$= 19.19$ kip-ft > 15.79 ok

Top stress = -0.176 – 0.491 = -0.667 ksi with fully bonded bottom strands

Note that a cracked section must be considered in calculating cantilever deflections because the top stress exceeds a tensile stress of $6\sqrt{f_c'}$.

3.5.1 Cantilever Load Distribution

Stair reactions, posts, or walls may apply non-uniform loads to cantilevered hollow core slabs, as shown in Fig. 3.5.2.

A finite element analysis was used to determine the shear and moment distribution widths for these loads. As shown in Fig. 3.5.3, two load cases were analyzed: a concentrated load at the end of the cantilever and a line load parallel to the span. In addition, these loads were both placed along the free edge and mid-width of a multi- slab system.

Similar to the load distribution discussed in Section 3.2, it was determined that the distribution width varies with the cantilever length and the location of the load along the length. The distribution width is significantly greater for the moment design than for the shear check. The slab thickness and the backspan length were found to have little or no effect. Figures 3.5.4 and 3.5.5 summarize the cantilever load distribution widths.

Figure 3.5.2

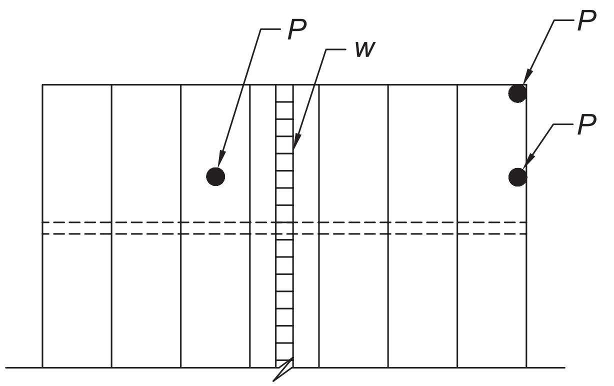
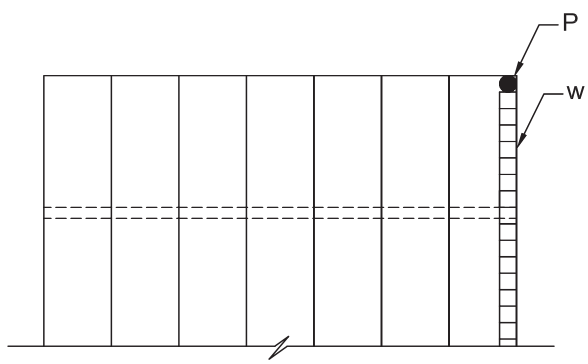
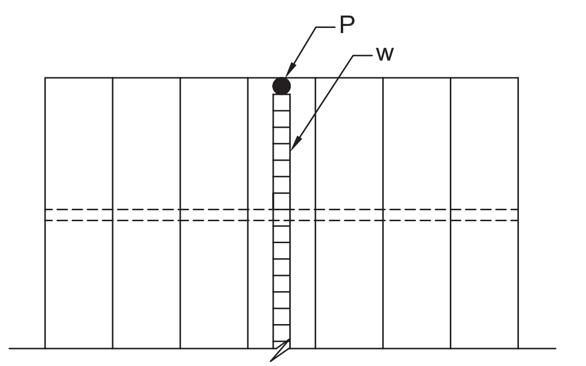


Figure 3.5.3

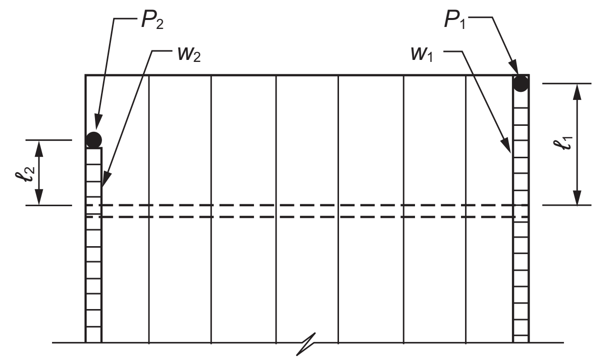


(a)



(b)

Figure 3.5.4

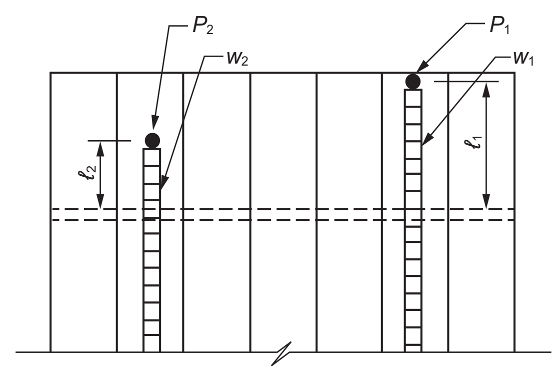


Edge Loads

Moment DW = 1 + 0.45ℓ (ft)
Shear DW = 1.5 ft

ℓ = distance from support to point load
or
ℓ = distance from support to end of parallel line load

Figure 3.5.5

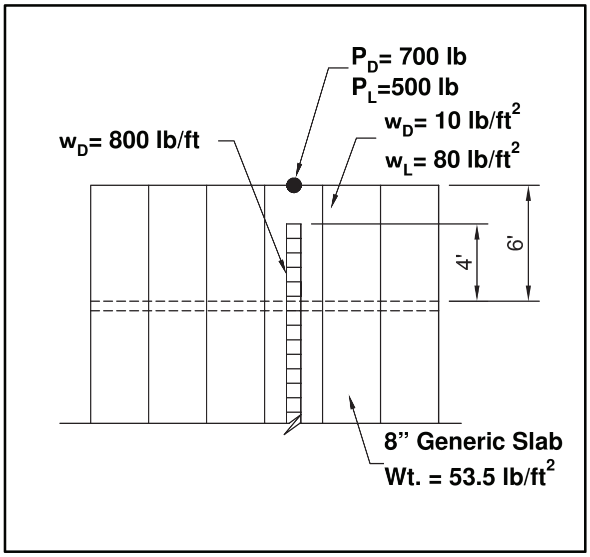


Interior Loads

Moment DW = 1.6 + 0.8ℓ (ft)
Shear DW = 1.5 + 0.2ℓ (ft)

ℓ = distance from support to point load
or
ℓ = distance from support to end of parallel line load

Example 3.5.2



Given the cantilever loading shown, determine the equivalent cantilever bending moment.

Solution:

For the end point load

P_u = 1.2(700) + 1.6(500)
= 1640 lb
DW = 1.6 + 0.8(6)
= 6.4 ft

$$M_u = \frac{1640(6)}{6.4}$$

= 1538 lb-ft/ft

For the parallel wall load

w_u = 1.2(800)
= 960 lb/ft
DW = 1.6 + 0.8(4)
= 4.8 ft

$$M_u = \frac{960(4^2)}{2(4.8)}$$

= 1600 lb-ft/ft

Uniform loads

$$w_u = 1.2(53.5 + 10) + 1.6(80)$$

= 204 lb/ft^2

$$M_u = \frac{204(6^2)}{2}$$

= 3676 lb-ft/ft

$$Total\ M_u = 1538 + 1600 + 3676$$

= 6814 lb-ft/ft

The cantilever should be designed for an ultimate moment of 6814 lb-ft/ft.

3.6 Horizontal Joints

Figure 3.6.1 depicts three conditions typically used in a multistory wall-bearing building where hollow core slabs are used in a platform detail. Several expressions[31-34] have been proposed to describe the transfer of axial load through this horizontal joint.

With hollow core slabs used for floors, the most efficient detail is to build the slab ends into the wall. Depending on the butt joint size, the strength of the joint for transfer of vertical loads can be enhanced with the addition of grout in the butt joint (Fig. 3.6.1b) or in both the butt joint and cores (Fig. 3.6.1c). Grout fill in the cores increases the net slab width and provides confinement for the grout column.

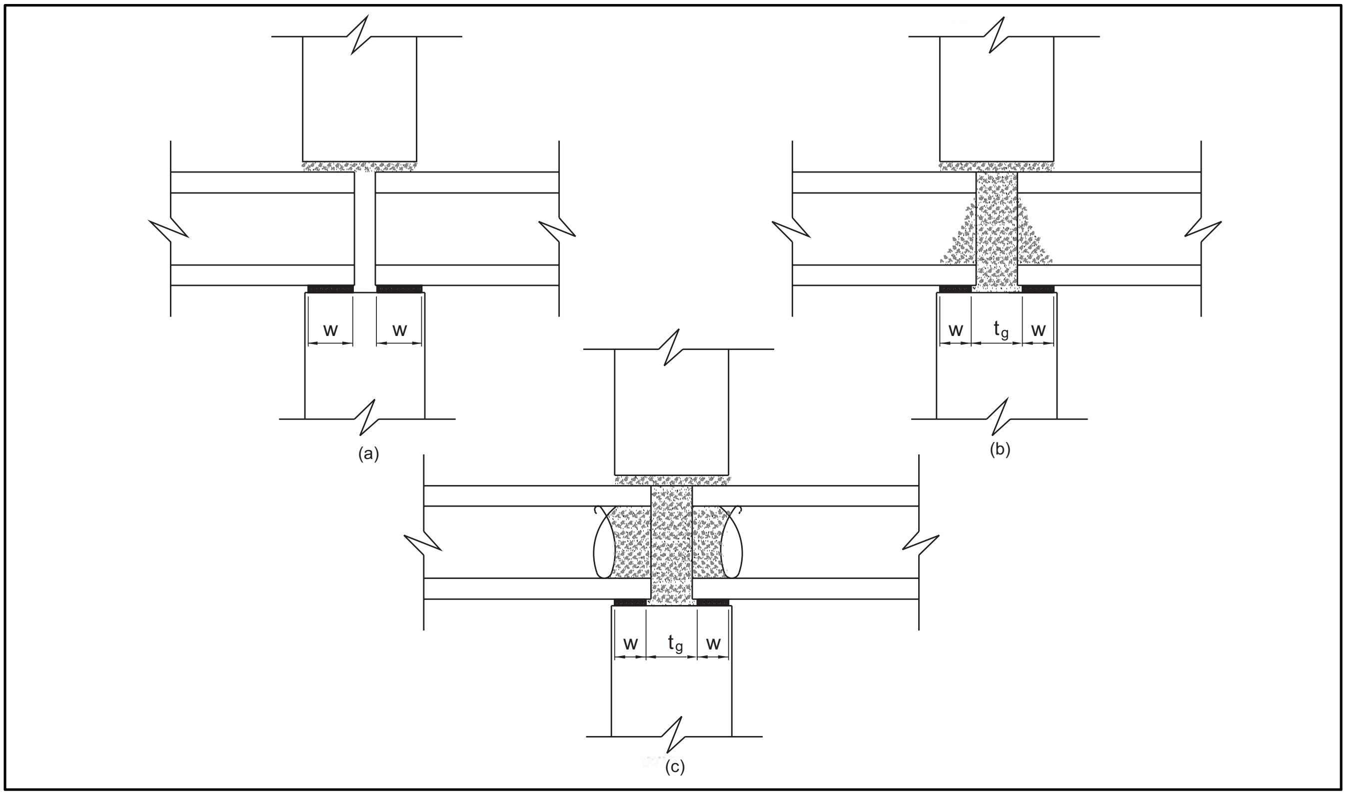
The strength of the joint for vertical load transfer can be predicted using Eq. 3.6.1 for an ungrouted joint (Fig. 3.6.1a). For a grouted joint (Fig. 3.6.1b or 3.6.1c), the greater of Eq. 3.6.1 and Eq. 3.6.2 can be used. Both grouted and ungrouted joints can have the slab cores either filled or not filled. Both equations include a capacity reduction term for load eccentric from the centerline of the joint. With single-story walls braced at the top and bottom, this eccentricity will be negligible.

$$\phi P_n = \phi 0.85 A_e f'_c R_e \qquad \text{(Eq. 3.6.1)}$$

$$\phi P_n = \phi t_g \ell f_u C R_e / k \qquad \text{(Eq. 3.6.2)}$$

where:
$A_e = 2wb_w$
w = bearing strip width
b_w = net web width of slab when cores are not filled
= slab width as solid slab when cores are filled
t_g = grout column thickness

Figure 3.6.1 Common platform details



ℓ = width of slab being considered

f_u = design compressive strength of wall or grout, whichever is less, when walls are reinforced against splitting and slab cores are filled

= 80% of design compressive strength of wall or design compressive strength of grout, whichever is less, when walls are not reinforced against splitting or slab cores are not filled

C = 1.0 when cores are not filled

$$= 1.4\sqrt{\frac{2500}{f_c'(\text{grout})}} \geq 1.0 \text{ when cores are filled}$$

k = $0.65 + [f'_c\,(\text{grout}) - 2500]/50{,}000$

R_e = $1 - 2e/h$

e = eccentricity of applied load measured from joint centerline

h = wall thickness

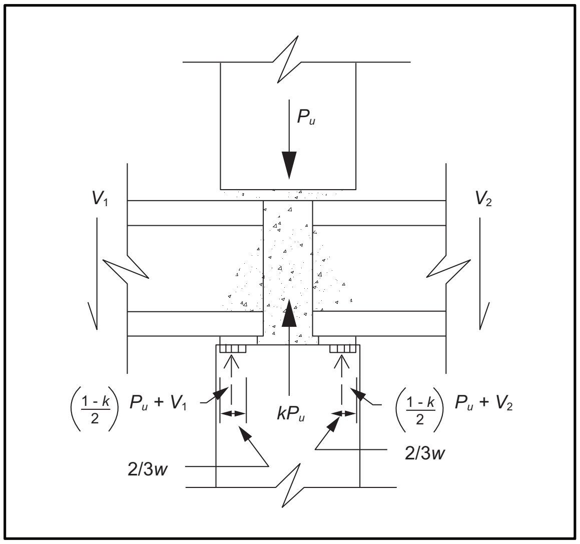
ϕ = 0.65

Where bearing strips with a modulus of elasticity other than 50,000 psi are used, the amount of force in the grout column will be altered. A theoretical approach presented in reference 34 considers pad stiffness, grout column strength as compared with grout strength, and confinement of the grout column. A comparison of this theoretical procedure with the values calculated using Eq. 3.6.1 and 3.6.2 indicates that a conservative capacity will be predicted by substituting the actual pad modulus of elasticity for 50,000 when calculating k.

The bearing strips also need to be checked against the manufacturer's recommended stress limits. Figure 3.6.2 summarizes the forces in the joint and the recommended effective bearing strip width.

Another set of forces acting on the horizontal joint develops from the negative moments induced in the floor slabs due to the clamping effect of a bearing wall on the slab ends. This results in two consequences: the splitting strength of the bearing wall is reduced when the normal force restraining slab end rotation is considered and the joint or slab may crack to relieve the frictional

Figure 3.6.2 Forces in joint



restraint. This condition is undesirable from the standpoint of either joint or slab integrity. Reinforcing located perpendicular to the slab butt joint is most effective for controlling this condition. To date, there are no published studies to evaluate effects of this rotational restraint. No adverse effects have been cited when nominal diaphragm or structural integrity reinforcement has been provided across the joint.

Example 3.6.1

Using the generic hollow core slab section defined in section 1.6, determine the grouting requirements for an interior butt joint as depicted in Fig. 3.6.1a given the following criteria:

slab span ℓ = 28 ft

18 story building with:
8 in. concrete bearing walls
f'_c (wall) = 5000 psi

Superimposed loads:

Roof D = 15 lb/ft^2
Roof L = 30 lb/ft^2
Floor D = 10 lb/ft^2
Floor L = 40 lb/ft^2
Walls D = 800 lb/ft/story
Live load reduction: None for example

Solution:

Loads

Roof:
$w_u = \ell\,[1.2\,(D) + 1.6\,(L)]$
$w_u = 28\,[1.2\,(53.5+15) + 1.6\,(30)]$
$= 3.65$ kip/ft

Floors:
$w_u = 28\,[1.2\,(53.5+10) + 1.6\,(40)]$
$= 3.93$ kip/ft

Walls:
$w_u = 1.2\,(800)$
$= 0.96$ kip/ft/story

Accumulate loads above floor noted.

Floor	w_u	$\sum w_u$
18	3.65 + 0.96	4.61
17	3.93 + 0.96	9.50
16	4.89	14.39
15	4.89	19.28
14	4.89	24.17
13	4.89	29.06
12	4.89	33.95
11	4.89	38.84
10	4.89	43.73
9	4.89	48.62
8	4.89	53.51
7	4.89	58.40
6	4.89	63.29
5	4.89	68.18
4	4.89	73.07
3	4.89	77.96
2	4.89	82.85

a) Evaluate capacity of ungrouted joint (Fig. 3.6.1a)

b_w = 10.5 in. for generic hollow core slab
= 3.5 in./ft of width
f'_c (slab) = 5000 psi

3 in. bearing strips

$\phi P_n = \phi 0.85\, A_e f'_c\, R_e$ (Eq. 3.6.1)

$$\phi P_n = 0.65(0.85)(2)(3)(3.5)(5)\left[1-\frac{2(0)}{8}\right]$$

= 58 kip/ft

Adequate for floors 8 through roof:

b) Evaluate strength of grouted joint using 3000 psi grout for

(1) A 2 in. butt joint with no filled cores (Fig. 3.6.1b)

$$\phi P_n = \phi 0.85 A_e f'_c R_e \qquad \text{(Eq. 3.6.1)}$$

$$= 0.65(0.85)(2)(3)(3.5)(5)\left[1 - \frac{2(0)}{8}\right]$$

= 58 kip/ft

or

$$\phi P_n = \phi t_g \ell f_u C R_e / k \qquad \text{(Eq. 3.6.2)}$$

f_u = 3000 psi
C = 1.0
k = 0.65 + (3000 − 2500)/50,000
= 0.66

$$\phi P_n = 0.65(2)(12)(3)(1.0)\left[1 - \frac{2(0)}{8}\right] / 0.66$$

= 70.9 kip/ft > 58 kip/ft

Therefore ϕP_n = 70.9 kip/ft

(2) A $^1/_2$ in. butt joint with cores filled (Fig. 3.6.1c)

$$\phi P_n = \phi 0.85 A_e f'_c R_e \qquad \text{(Eq. 3.6.1)}$$

$$= 0.65(0.85)(2)(3)(12)(3)\left[1 - \frac{2(0)}{8}\right]$$

= 119.3 kip/ft

or

$$\phi P_n = \phi t_g \ell f_u C R_e / k \qquad \text{(Eq. 3.6.2)}$$

f_u = 3000 psi

$$C = 1.4\sqrt{2500/3000} = 1.28$$

K = 0.65 + (3000 − 2500)/50,000
=0.66

$$\phi P_n = 0.65(0.5)(12)(3)(1.28)\left[1 - \frac{2(0)}{8}\right] / 0.66$$

= 22.7 kip/ft < 119.3 kip/ft

Therefore ϕP_n = 119.3 kip/ft

Use $^1/_2$ in. butt joint with cores filled below eighth floor.

This example may overstate the height of a building that can be supported on an ungrouted joint. Concentrated loads due to corridor lintels, wall openings, or exterior spandrels must also be considered in most buildings resulting in an increase in load to be transferred through the horizontal joint.

Chapter 4

DIAPHRAGM ACTION WITH HOLLOW CORE SLABS

4.1 General Information

When hollow core slabs are used to construct floor or roof decks that support vertical loads, the natural extension is to use the same decks as diaphragms to transmit lateral loads. Lateral loads act on building structures in the form of lateral earth pressures, wind loads, or seismic forces. The function of a diaphragm is to receive these loads from the building elements to which they have been applied or in which they originate and transmit the loads to the lateral force-resisting elements that carry the lateral loads to the foundation. The design issues in a diaphragm system comprised of hollow core slabs are the design of connections to get loads into the diaphragm (although most of the earthquake forces originate in the diaphragm itself), the strength and ductility of the system to transmit these loads to the lateral force-resisting elements, and the design of the connections required to direct the lateral forces from the diaphragm to the lateral force-resisting elements.

Clear communication is required between the building designer and the hollow core supplier when a deck comprised of hollow core slabs is to be used as a diaphragm. Some elements of the diaphragm design may be delegated to the hollow core supplier. However, only the building designer is in a position to know all of the parameters involved in generating the lateral loads. Because of the many design issues, only the building designer can determine the locations and relative stiffnesses of the lateral force-resisting elements. These parameters dictate the distribution of forces in the diaphragm. If any design responsibility will be delegated to the hollow core supplier, the location and magnitude of the lateral forces acting in the diaphragm and the location and magnitude of the forces to be transmitted to the lateral force-resisting elements must be specified. Where hollow core slabs must connect to building elements made of other building materials, or where demands on connections go beyond simple strength demands, the connection details should be shown in the contract documents.

An additional consideration in detailing diaphragms is the need for structural integrity. Section 16.5 of the American Concrete Institute's *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11)*[3] provides minimum requirements to satisfy Section 7.13 of ACI 318-11 for precast concrete structures. For large-panel bearing wall structures, minimum requirements are specified to provide ties throughout the structure. For other types of precast concrete structures, only general detailing philosophies are specified. In either case, the fundamental requirement is to provide a complete load path from any point in a structure to the foundation. Clearly, a diaphragm is a significant element in this load path. A tie system that satisfies the strength and force transfer demands on a diaphragm will generally satisfy the detailing requirements for structural integrity.

4.2 Design Loads

In plane forces in hollow core diaphragms can be due to lateral earth pressures, wind loads, or seismic forces. Lateral earth pressures are established by the characteristics of the soil being retained. Wind loads and seismic forces are dictated by the applicable building code for the structure. Soil and wind loads are actually applied to the structure. Seismic forces are generated within the structure as inertial forces due to lateral displacement from ground motions. While soil and wind loads can be safely treated as static loads, seismic forces must be considered as dynamic in nature. In all cases, the same elements will comprise a complete diaphragm, but the ductility demands on a seismic force-resisting system are significantly more important.

The balance of the discussion in this chapter will be concerned with lateral wind loads and earthquake forces. This is not intended to diminish the importance of considering unbalanced soil pressures, which can commonly be a significant consideration in many projects using hollow core slabs. The basic principles of hollow core diaphragms that will be discussed are equally applicable to diaphragms subject to in-plane forces due to lateral soil pressures.

There are many documents that cover design for wind and seismic forces. The reference used for this chapter is the *2012 International Building Code* (IBC).[35] The IBC refers to ASCE 7-10[36] for wind loads based on wind speeds that directly result in strength-level rather than service-level design wind pressures. A basic wind speed is selected based on the building location and the risk category (previously occupancy category) of the building, an exposure category is selected based on the surrounding terrain, modifying factors are determined for the geometry of the building and its site, and the design positive and negative wind pressures are calculated.

The 2012 IBC also refers to ASCE 7-10 for seismic design. ASCE 7-10 allows an equivalent lateral force approach for many buildings. For buildings assigned to high seismic design categories (D, E, or F), depending upon height or the presence of structural irregularities, modal response spectrum analysis or seismic response history procedures may be required. The equivalent lateral force approach allows design for a base shear of:

$$V = C_s W \qquad \text{(Eq. 4.2.1)}$$

The seismic response coefficient C_s is determined by

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \qquad \text{(Eq. 4.2.2)}$$

The value of C_s need not exceed

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \text{ for } T \le T_L \qquad \text{(Eq. 4.2.3)}$$

$$C_s = \frac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)} \text{ for } T > T_L \qquad \text{(Eq. 4.2.4)}$$

Also, C_s must not be less than

$$C_s = 0.044 S_{DS} I_e \ge 0.01 \qquad \text{(Eq. 4.2.5)}$$

For structures located where $S_1 \ge 0.6g$ (where g is the acceleration due to gravity), C_s must not be taken less than that determined from

$$C_s = \frac{0.5 S_1}{\left(\frac{R}{I_e}\right)} \qquad \text{(Eq. 4.2.6)}$$

The spectral response quantities are:

$$S_{DS} = \frac{2}{3} F_a S_s \qquad \text{(Eq. 4.2.7)}$$

$$S_{D1} = \frac{2}{3} F_v S_1 \qquad \text{(Eq. 4.2.8)}$$

S_s and S_1 for a particular site can be determined by interpolation between contours on maps included in ASCE 7-10 or in the 2012 IBC. They are preferably found on the website of the U.S. Geological Survey, based on the latitude and the longitude or the street address of the location within the United States.

The elastic fundamental period T may be taken equal to the approximate fundamental period T_a given by

$$T_a = 0.016(h_n)^{0.9} \qquad \text{(Eq. 4.2.9)}$$

for concrete moment frames resisting lateral forces, or

$$T_a = 0.020(h_n)^{0.75} \qquad \text{(Eq. 4.2.10)}$$

for other lateral force-resisting systems.

The base shear is distributed over the height of the structure in the manner prescribed by ASCE 7-10.

Where diaphragms are not flexible, provision must be made for the increased horizontal forces induced in vertical elements of the lateral force-resisting system, resulting from torsion due to eccentricity between the center of application of the lateral forces (center of mass) and the center of rigidity of the seismic force-resisting system (through which the resultant of the resistances to the lateral forces acts). Additionally, an accidental torsion must be considered. To compute the accidental torsion, the mass at each level must be assumed to be displaced from the calculated center of mass in each direction a distance equal to 5% of the building plan dimension at that level perpendicular to the direction of the force under consideration. Forces in the vertical elements of the seismic force-resisting system are not to be decreased when accidental torsion effects result in forces opposite to the direction being considered.

Every structure, depending upon its occupancy or use and the values of S_{DS} and S_{D1} at its site, is assigned to one of six seismic design categories (SDCs): A through F. Structures assigned to essentially non-seismic SDC A require only ordinary detailing by Chapters 1 through 18 of ACI 318-11. Structures assigned to (often referred to as low) SDC B also require only ordinary detailing as in the case of SDC A, but now Section 21.2 is additionally invoked. Structures assigned to (often referred to as moderate) SDC C require, as a minimum, intermediate detailing by Chapters 1 through 18 and Sections 21.3 and 21.4. Structures assigned to (typically described as high) SDC D, E, or F, require special detailing by Chapters 1 through 18, Sections 21.1.3 through 21.1.7, and Sections 21.5 through 21.12. Structural members that are not part of the seismic force-resisting system of a building assigned to SDC D, E, or F must satisfy Section 21.13 in addition to Chapters 1 through 18. Note that Chapter 21 at times supersedes the requirements of Chapters 1 through 18. Height limits on structural systems and other important code requirements (such as whether the equivalent lateral force procedure shall be permitted for design) also depend on the seismic design category to which a structure is assigned.

ASCE 7-10 requires that a floor or roof diaphragm resist a force equal to

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad \text{(Eq. 4.2.11)}$$

The magnitude of F_{px} need not exceed $0.4 S_{DS} I_e w_{px}$ and shall not be less than $0.2\ S_{DS} I_e w_{px}$. Other requirements are included in ASCE 7-10, which are not restated in this summary. It is important to note that ASCE 7-10 provisions yield forces that are already factored and are intended to be used with strength design methods with no additional load factors.

In light of the performance of some diaphragms in earthquakes, the seismic demand on diaphragms has been an area of focus in recent times. Many knowledgeable professionals feel that diaphragms should remain elastic during a design-level seismic event to ensure that post-elastic behavior can be achieved in the lateral force-resisting elements. By designing a diaphragm to remain elastic, several things are accomplished. Diaphragm flexibility, discussed in section 4.3 of this manual, will be less significant. The ductility requirements for connection details will be of less concern. The horizontal distribution of forces to lateral force-resisting elements will not change during the response of a structure to a design-level earthquake.

The building code provisions summarized previously are based on the expected post-elastic response of structures. To keep a diaphragm compatible with post-elastic seismic response of the lateral force-resisting system, an analysis can be done to evaluate the total potential post-elastic strength required in the lateral force-resisting elements. Providing a diaphragm with strength beyond this required level will achieve compatibility, but will involve significant analysis. Alternatively, the diaphragm design forces prescribed by the building codes can be increased by a factor to keep the diaphragm elastic and minimize required analysis.

The following recommendations have been made in the second edition of PCI's *Seismic Design Manual*,[37] based on the results of research:

For structures assigned to low and moderate seismic design categories (A, B, C), if every floor diaphragm is designed for the force at the uppermost level derived from the IBC, additional load factors are not required for elastic diaphragm response under the design earthquake.

For structures assigned to high seismic design categories (D, E, F), if lateral forces are resisted entirely by special moment frames, additional load factors are also not required if every floor diaphragm is designed for the force at the uppermost level derived from the IBC.

For structures assigned to high seismic design categories (D, E, F), if shear walls are part of the seismic force-resisting system, it is sufficient to apply a diaphragm load factor of 2 to the force at the uppermost level derived from the IBC and to design each floor for that force.

4.3 Distribution of Lateral Forces

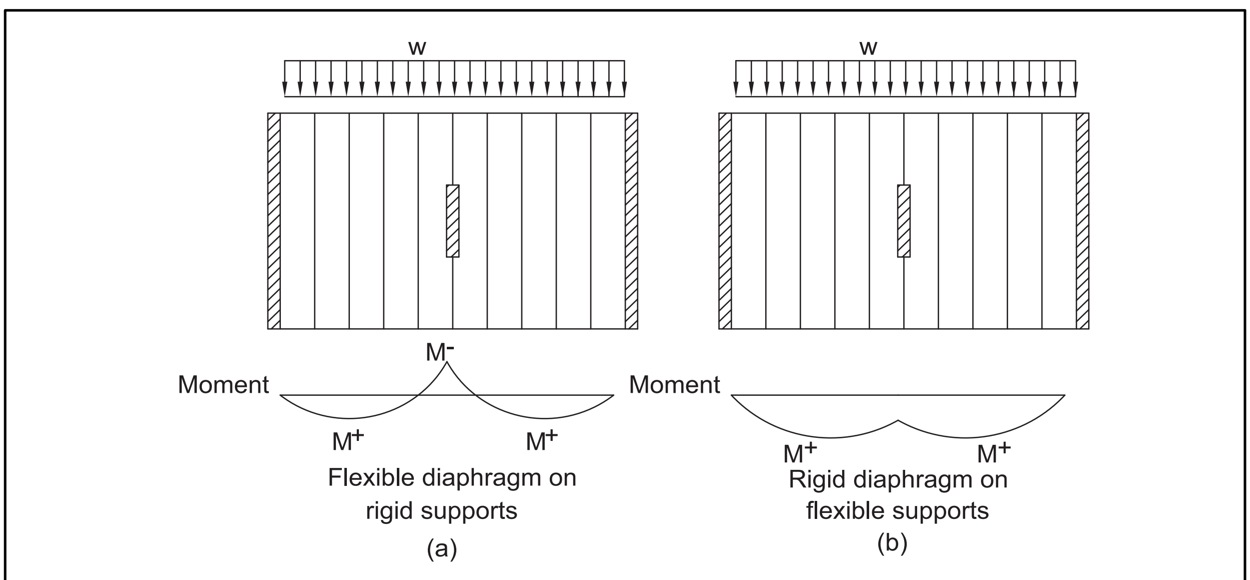
The design base shear given by Eq. 4.2.1, when distributed along the height of the structure in the manner described by ASCE 7-10, results in lateral forces (F_x at level x) acting at various floor levels, the story shear in any story is the sum of the lateral forces acting above the story. Non-seismic lateral forces can also be converted to story shears. The next issue is to determine the distribution of the story shears to the lateral force-resisting elements that will carry the forces down to the foundation. This problem is usually structurally indeterminate, which means that deformation compatibilities must be considered in addition to equilibrium. The stiffnesses to be considered are those of the diaphragm and the lateral force-resisting elements. Concrete diaphragms are normally considered to be rigid when compared with the lateral force-resisting elements. Depending on the type and magnitude of lateral forces applied, a hollow core slab diaphragm may need to be considered as a non-rigid diaphragm. For most low- and mid-rise structures in low-seismic-risk areas, an assumption of a rigid diaphragm would be reasonable.

The difference in behavior of flexible and rigid diaphragms is illustrated in Fig. 4.3.1. In (a), the flexible diaphragm with rigid supports behaves as a continuous beam. Shears and moments in the diaphragm are a function of the plan geometry. In (b), the deflections of the flexible supports must be equal because of the rigid diaphragm. The diaphragm shears and moments are a function of the relative stiffnesses of the supports. The differences between (a) and (b) can be considerable. Actual behavior will fall between the two cases, tending toward one or the other, depending on the diaphragm stiffness.

In seismic applications, the topic of diaphragm flexibility becomes a very significant issue. ASCE 7-10 requires consideration of diaphragm flexibility for the horizontal distribution of seismic story shears. A flexible diaphragm is defined by ASCE 7-10 as one having a maximum in-plane lateral deflection more than twice the average inter-story drift of the supporting lateral force-resisting elements. Section 12.3 of ASCE 7-10 also sets forth conditions under which diaphragms can be idealized as either flexible or rigid.

When diaphragm flexibility must be considered in determining the distribution of lateral

Figure 4.3.1 Diaphragm bending moments



forces because the diaphragms are neither flexible (in which case the lateral forces are distributed to the lateral force-resisting elements in proportion to the diaphragm areas tributary to those elements) nor rigid (in which case distribution can be in proportion to the rigidities of the lateral force-resisting elements), a cracked moment of inertia calculation is suggested in Reference 38 and a Virendeel truss model is suggested in Reference 39. Since the analysis of a structure with a semi-rigid diaphragm is dependent on so many factors beyond the diaphragm itself, such analysis is beyond the scope of this manual.

4.4 Structural Integrity

As noted in the introduction to this chapter, ACI 318-11 requires consideration of structural integrity for all precast concrete structures. While proper detailing for lateral loads will satisfy the complete load path requirement of structural integrity, there are some minimum provisions in section 16.5 of ACI 318-11 that must be met. With specific regard to diaphragms, the provisions to be aware of are the following.

For buildings other than bearing-wall structures, the connection to the diaphragm of members being laterally braced by the diaphragm shall have a minimum nominal tensile strength of 300 lb/ft. Note that in ASCE 7-10 and the 2012 IBC, this force is dependent on the seismic design category and the minimum will almost always be different from the above. In general, the 2012 IBC would supersede ACI 318-11. However, in cases where the code minimum is lower than the ACI 318-11 minimum, it may be advisable to comply with the ACI minimum.

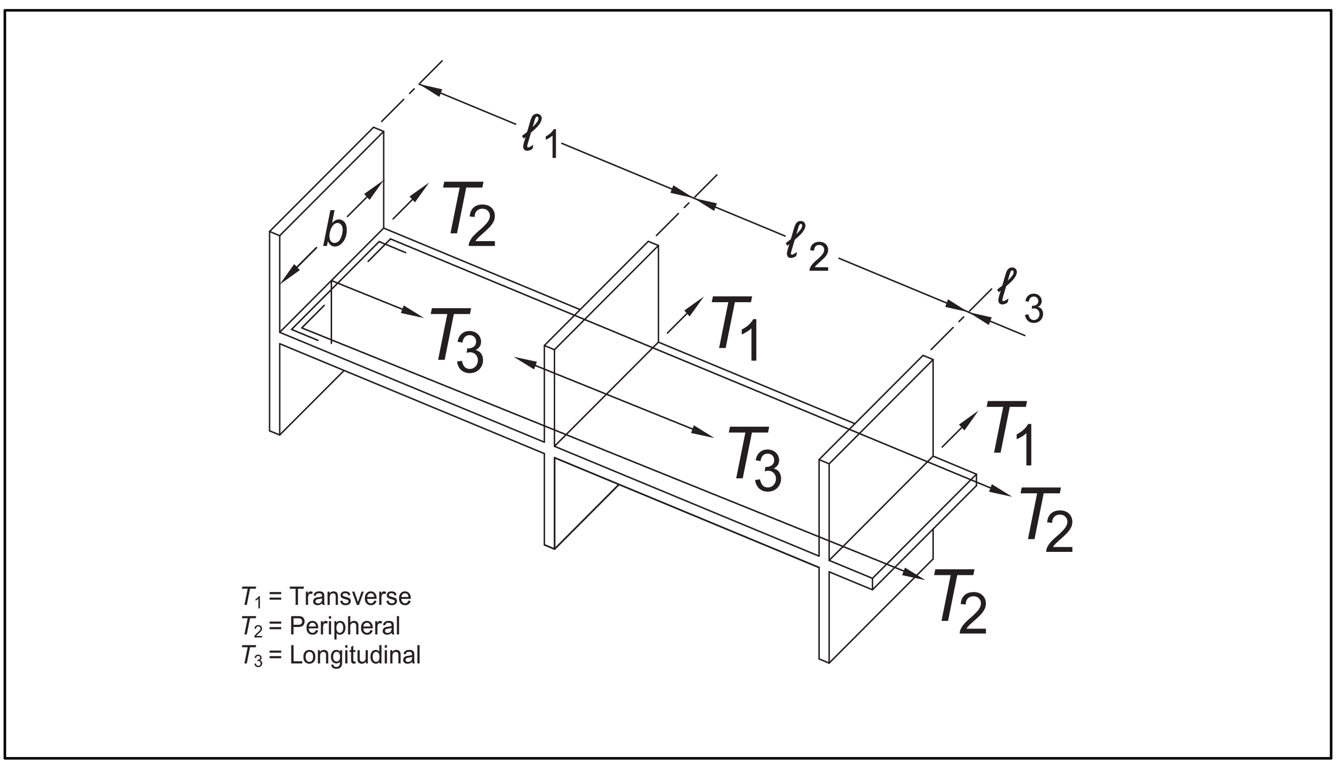
For large-panel bearing-wall structures, a summary of the tie forces is given in Fig. 4.4.1 and the ties are required to have the following minimum nominal strengths:

T_1 - 1500 lb/ft of floor or roof span
T_2 - 16,000 lb
T_3 - 1500 lb/ft of wall

These minimum strengths shall not control if the actual forces in the diaphragm are greater.

For seismic loading, it is preferable to use conventional reinforcing steel for these types of

Figure 4.4.1 Tie forces in bearing wall buildings



ties, to limit the elongations and deformations. When structural integrity requirements control in non-seismic areas, untensioned prestressing strands may be used to satisfy the strength requirements.

4.5 Elements of a Diaphragm

Figure 4.5.1 illustrates the various elements that comprise a complete diaphragm. The following definitions will be used to describe the various elements:

Boundary element: Edge member around the perimeter of a diaphragm or the perimeter of an opening in a diaphragm, which ties the diaphragm together. The boundary element may function as a chord or a collector.

Chord: A diaphragm boundary element perpendicular to the applied load that is assumed to take axial stresses due to the diaphragm moment.

Collector or Drag strut: A diaphragm boundary element parallel to the applied load that collects and transfers diaphragm shear forces to the vertical elements of the lateral force-resisting system or distributes forces within the diaphragm.

Longitudinal joint: Joint oriented parallel to the slab span.

Transverse joint: Joint oriented perpendicular to the slab span.

To satisfy structural integrity, all diaphragms should have boundary elements of some type. The boundary elements are essential to ensure that a diaphragm will have the strength to transfer lateral loads to the lateral force-resisting system. Tension reinforcement is placed in the boundary element to enable it to act as a chord, to allow the diaphragm to act as a deep horizontal beam or tied arch. This reinforcement can also provide shear friction steel for shear transfer along the longitudinal joints.

Collectors are required in all diaphragms to transfer shear forces from the diaphragm edges to the lateral force-resisting elements, unless the entire edge of a diaphragm is supported continuously on shear walls or frames resisting lateral forces. Such collectors are also required for structural integrity, to provide a complete load path for lateral forces to the foundation. Collectors may also function to get forces into a diaphragm, in which case, they are often referred to as distributors.

Figure 4.5.1 Diaphragm elements

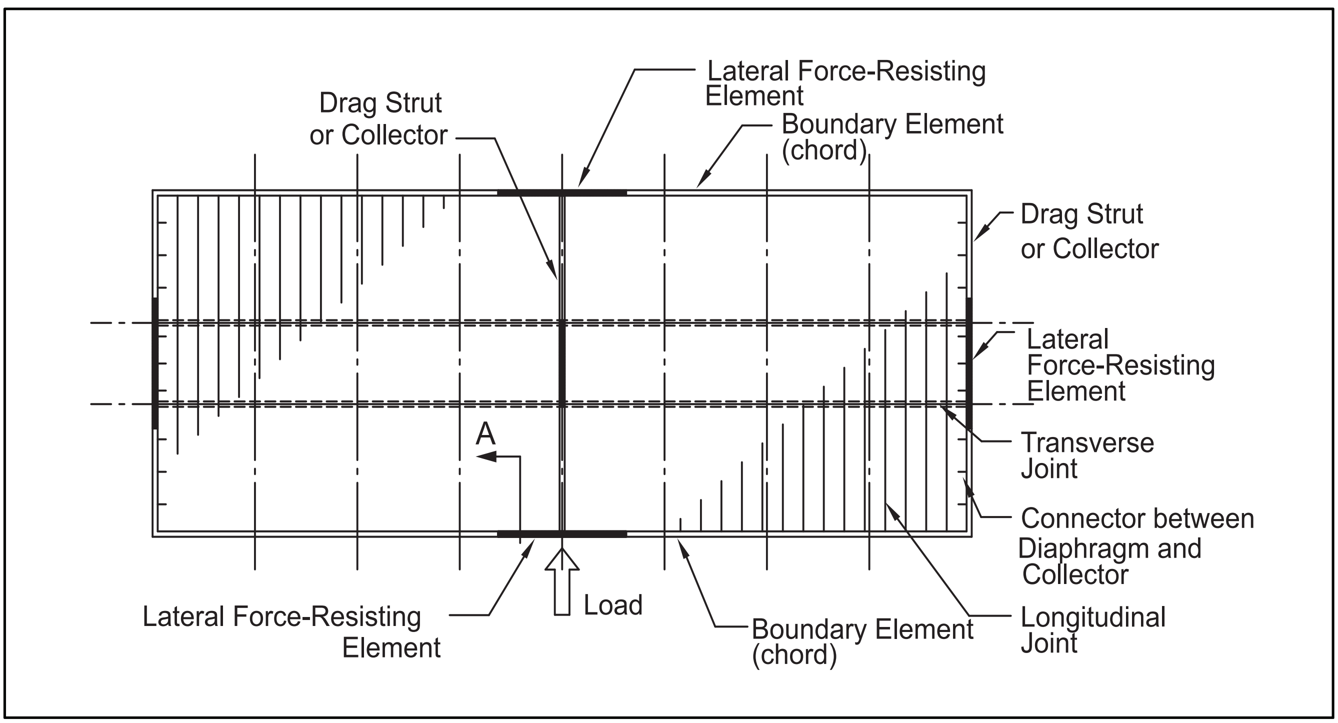
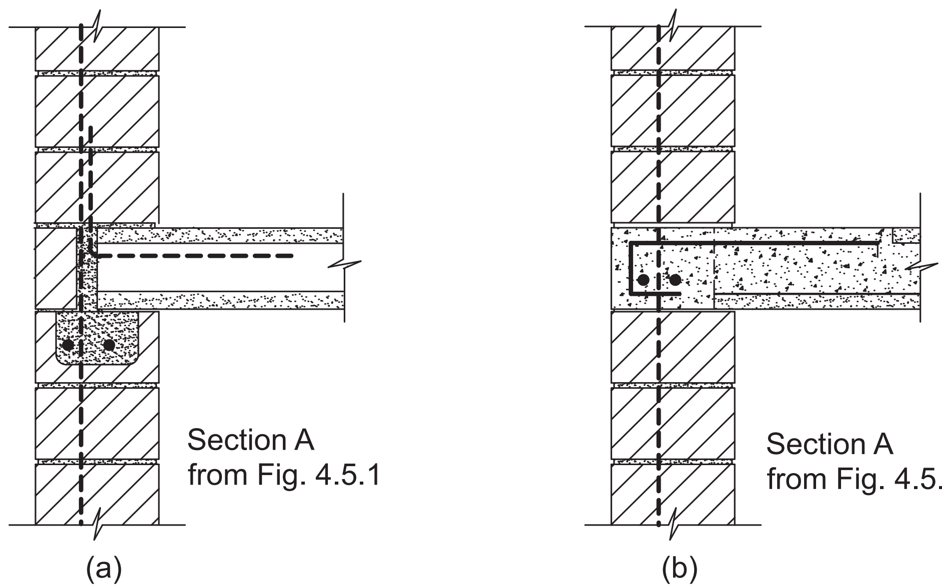


Figure 4.5.2 Boundary elements



Note that a chord or a collector may consist entirely of reinforcing bars accommodated within the thickness of the slab.

When a bonded structural topping is used with a hollow core slab diaphragm, boundary elements can be provided directly by reinforcement in the topping. When no topping is provided, these elements are developed as grouted or concrete elements external to the hollow core slabs. As a simple example, Fig. 4.5.2 depicts two common boundary conditions. In (a), the boundary reinforcement is placed in a masonry bond beam and the reinforcement connecting the shear wall or boundary element to the diaphragm is placed in the keyways between slabs. In (b), the boundary reinforcement is placed in a grouted or concrete-filled space at the end of the slabs. The reinforcement connecting the shear wall or boundary element to the diaphragm is again placed in the keyways between slabs. The primary difference between the details is that the boundary reinforcement in (a) is eccentric from the diaphragm web, while it is nearly concentric in (b). The concentric boundary element will exhibit better performance in a seismic situation and should be used in structures assigned to SDC C, D, E, or F.

4.6 Diaphragm Strength

The diaphragm must have the strength to transfer imposed lateral forces from the point of application or origin to the point of resistance. The diaphragm spans between lateral force-resisting elements as a deep beam or tied arch. Tensile and compressive stresses due to flexure and shear stresses will develop and must be resisted in the diaphragm.

4.6.1 Longitudinal Joints

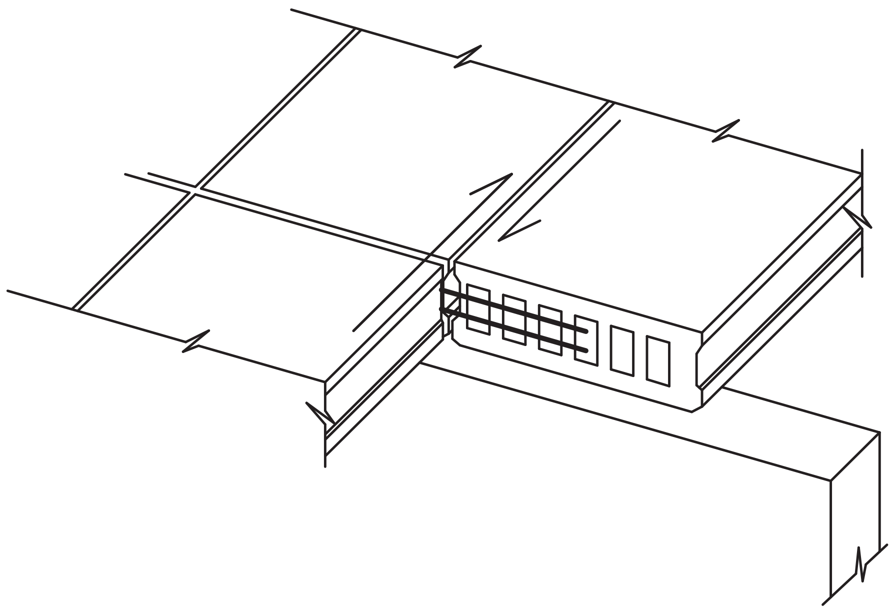
The grouted keyways between slabs do have the capacity to transfer longitudinal shear from one slab to the next. Using a shear stress of 80 psi, the useable (design) strength for longitudinal shear is:

$$\phi V_n = \phi(0.08) h_{net} \ell_j \qquad \text{(Eq. 4.6.1)}$$

where

ϕ = 0.75

Figure 4.6.1 Shear friction steel in butt joint



When the grout strength is exceeded or ductile behavior is required, shear friction principles may be used to design reinforcement to be placed perpendicular to the longitudinal joints.[40] This reinforcement may be placed in the transverse joints at the slab ends rather than being distributed along the length of the joints. Placed as shown in Fig. 4.6.1, the area of steel is calculated as:

$$A_{vf} = \frac{V_u}{\phi f_y \mu} \qquad \text{(Eq. 4.6.2)}$$

where
μ = 1.0 for shear parallel to longitudinal joints
ϕ = 0.75

While the detail shown in Fig. 4.6.1 is the most economical means of providing a mechanical connection across the longitudinal joints, alternate connections are available, which may be desirable in certain circumstances. Figure 4.6.2(a) shows reinforcing steel placed across the longitudinal joint and grouted into the cores. This detail might be considered when the amount of reinforcement required in the transverse joints is great enough to cause congestion. Figure 4.6.2(b) shows weld plates in the slabs and a loose plate welded across the longitudinal joint. Use of this detail should be carefully coordinated with the hollow core supplier to ensure that proper anchorage of the weld plates in the slabs can be accomplished.

Where the diaphragm must transfer shear into a lateral force-resisting element, boundary element, or interior drag strut, a condition similar to the longitudinal joint exists. For longitudinal shear, shear friction again can be used to design reinforcement to cross potential crack planes and transfer the shear. Figure 4.6.3 depicts an example of such a detail.

While drag struts and boundary elements may have a vertical stiffness similar to that of the deck, the lateral force-resisting elements will usually have a significantly higher vertical stiffness. The connections to the lateral force-resisting elements will tend to be vertically rigid. While strength and toughness of such connections are certainly important, it is equally important to consider everyday performance of the structure. At rigid vertical elements, it may be desirable to allow hollow core slab camber growth or deflection to occur without distress at the connection.

Figure 4.6.4 shows potential damage at the first interior longitudinal joint when a vertically rigid connection is used. The potential for distress is dependent on the span and the actual applied loads. Short, lightly loaded spans may experience no problems.

The effect of different vertical stiffnesses may be accounted for by:

- determining that distress will not affect the strength or performance of the system;
- locating vertically rigid connections near the hollow core slab supports where vertical movement is minimized; or
- providing allowance for vertical movement in the connection detail.

4.6.2 Transverse Joints

The transverse joints serve many functions. As described in Section 4.6.1 of this manual, reinforcement in the transverse joints may provide the shear friction reinforcement for shear in the longitudinal joints. The transverse joint may also have to act as part of a drag strut with axial tension or compression to carry diaphragm loads to the lateral force-resisting elements. A transverse joint may also be part of the chord member where flexural tension is resisted. Finally, an interior transverse joint disrupts the web of the horizontal beam where horizontal shear would have to be transferred to maintain the full effective depth of the diaphragm.

Drag strut reinforcement is calculated simply as:

$$A_s = \frac{N_u}{\phi f_y} \quad \text{(Eq. 4.6.3)}$$

Chord tension is resisted by reinforcement that provides flexural strength to the diaphragm. It is suggested[41] that the effective depth of the reinforcement from the compression edge of the diaphragm be limited to 0.8 times the depth of the diaphragm. Hence, the chord reinforcement is calculated as:

$$A_s = \frac{M_u}{\phi d f_y} \quad \text{(Eq. 4.6.4)}$$

Where d is taken as 0.8 times the depth of the diaphragm and

$\phi = 0.9$

Note that the seventh edition of the *PCI Design Handbook*[1] has made the following recommendation:

Since no additional factor on the diaphragm seismic design force of ASCE 7-10 is used in Seismic Design Categories A and B, it is recommended, based on earlier versions of the NEHRP Provisions, that perimeter diaphragm rein-

Figure 4.6.2 Alternate longitudinal shear connections

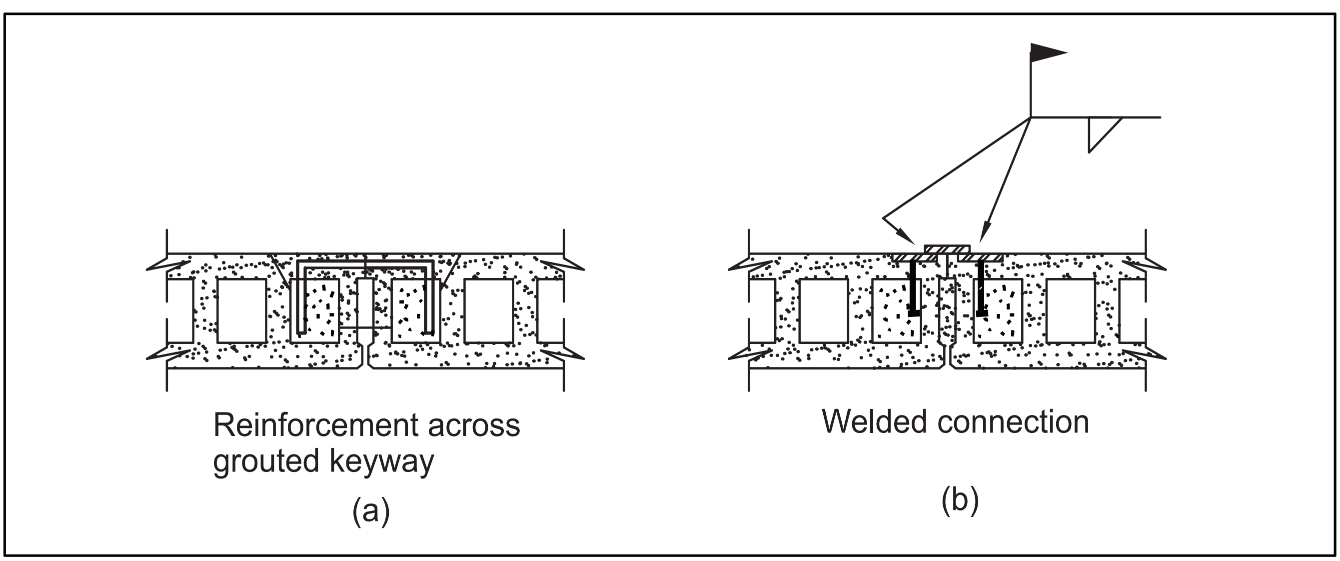
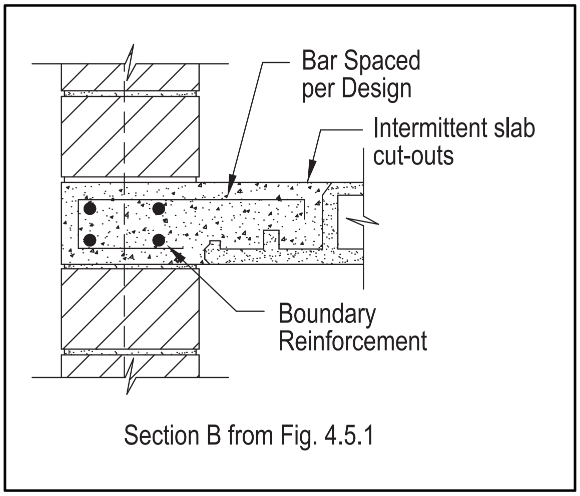


Figure 4.6.3 Collector detail



forcement be designed based on strength reduction factors, ϕ, as follows: For continuous bars, $\phi = 0.9$. For bars spliced with mechanical or welded connections, $\phi = 0.7$. For shear design of diaphragms, $\phi = 0.6$ for shear friction reinforcement and for mechanical connections in the joints.

Because diaphragms tend to act as tied arches rather than beams, tension in the chord reinforcement does not go to zero at the ends of the diaphragm. The chord reinforcement must be anchored at the ends of the diaphragm where standard hooks at the ends of the chords will suffice.

A shear parallel to the transverse joints is developed in the web of the diaphragm. Shear friction reinforcement perpendicular to the transverse joint and embedded in the hollow core slab keyways can be used to reinforce for this shear. The applied shear can be calculated as:

$$V_h = \frac{V_u Q}{I}$$

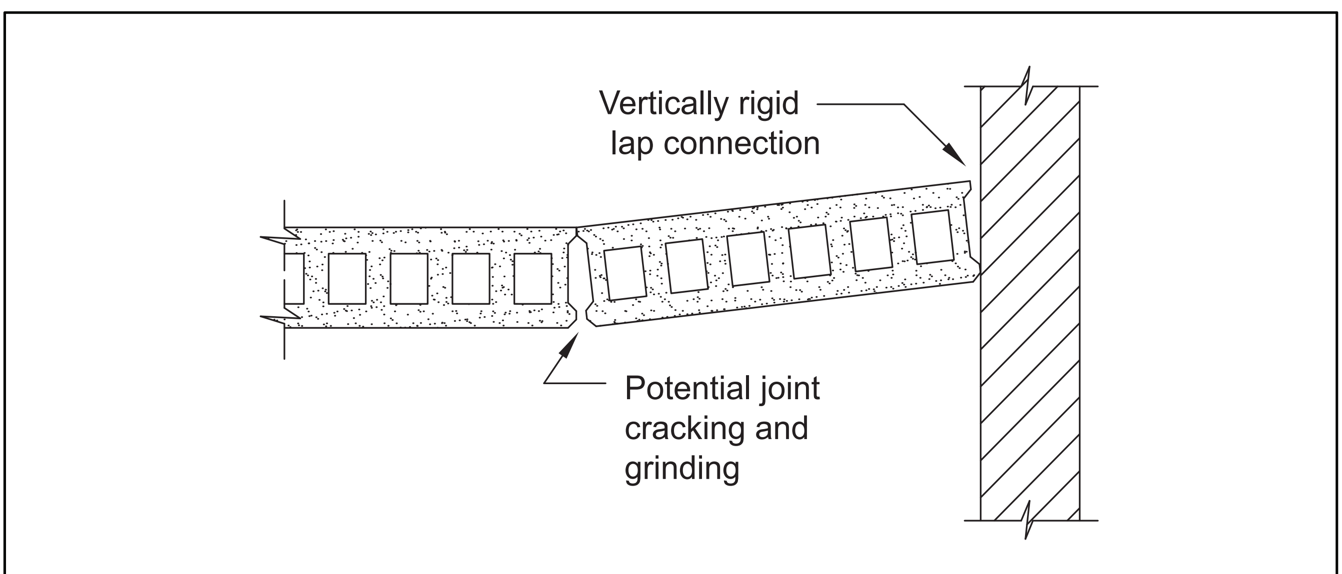
or

$$V_h = \frac{M_u}{d} \qquad \text{(Eq. 4.6.5)}$$

In the first case, a unit shear is calculated and shear friction reinforcement is distributed according to the shear diagram. In the second case, the total shear is calculated as the tension or compression of the internal couple. In this case, shear friction reinforcement is uniformly distributed over the length between zero moment and maximum moment. It is suggested that the shear friction reinforcement be distributed according to the shear diagram in structures assigned to SDC C, D, E, or F to minimize the force redistribution required with a uniform spacing.

Resistance for shear parallel to the transverse joints is provided by reinforcement placed perpendicular to the transverse joints. Design of this reinforcement is similar to that described in Section 4.6.1 for longitudinal joints. Equation 4.6.2

Figure 4.6.4 Potential effects of rigid lap connections



may be used for the calculation of shear friction reinforcement, except μ may be taken as 1.4 when the grout provided in the transverse joints is allowed to flow into the ends of the cores in the slabs. This forces the shear crack to propagate through monolithic grout rather than following a cold joint.

In view of of the orientation of the joints and the loading directions considered, the reinforcement in the transverse joint discussed previously is not all additive. Typically, the chord tension and longitudinal joint shear will be concurrent. The drag strut tension will typically occur with loads applied in the perpendicular direction.

4.7 Boundary Elements

The preceding discussion has indicated that reinforcing bars may be used to connect boundary elements to diaphragms using shear friction design procedures. As shear friction reinforcement, the steel is used in tension to resist a shear force. In detailing the steel, a crack plane is defined and the bars must be anchored for full strength on each side of the crack plane. For anchorage at a transverse boundary element, the bars may be grouted into the keyways or into hollow core slab cores where the top of the core is cut away. Concrete is then used to fill the cores for the length of the bar embedment. Based on a review of the literature, it is not clear when anchorage of the connector bars in keyways is sufficient and when the connector bars should be placed in hollow core slab cores. There is a concern that as the boundary element and keyway crack, anchorage for a connector bar in a keyway may be lost. Deformations and reversible loading in a seismic event would suggest that anchoring connector bars in hollow core slab cores would be preferable in more intense seismic situations. In keeping with code philosophy, it is suggested that bars be anchored in hollow core slab cores in structures assigned to SDC C and higher.

In non-seismic and low-seismic design situations, the connectors need not be reinforcing bars. Particularly for direct connections to lateral force-resisting elements, welded and bolted connections will suffice for the boundary element to diaphragm connections when they are compatible with the system used.

4.8 Topped versus Untopped Diaphragms

When a composite structural topping is provided, it should have a minimum thickness of 2 in. to $2^1/_2$ in. The topping can then be designed as the diaphragm without consideration of the hollow core slabs. When the topping provides the strength and the stiffness for the diaphragm but the connections are made in the hollow core slabs, shear stresses will be present at the interface of the topping and the hollow core slabs. These stresses will generally be well distributed throughout the interface, but may be more highly localized near the connections. As discussed in Chapter 2 of this manual, horizontal shear stresses should be kept below a nominal strength of 80 psi.

The primary benefits of a composite structural topping are to increase stiffness and to allow easier continuous ties in plans with irregular shapes or large openings. However, in seismic areas, the additional topping weight increases the seismic design forces. Topped diaphragms may be a necessity in buildings assigned to high SDC, and with plan irregularities or large diaphragm span-to-depth ratios.

Untopped hollow core slab diaphragms may be sufficient when the diaphragm force system is straightforward and the in-plane diaphragm deflections are acceptable. An example in Section 4.9 illustrates a procedure for determining diaphragm deflections.

Current practice is to generally use topped hollow core slab diaphragms in high seismic areas. Local codes may limit the use of untopped diaphragms in areas of high seismicity.

4.9 Design Example

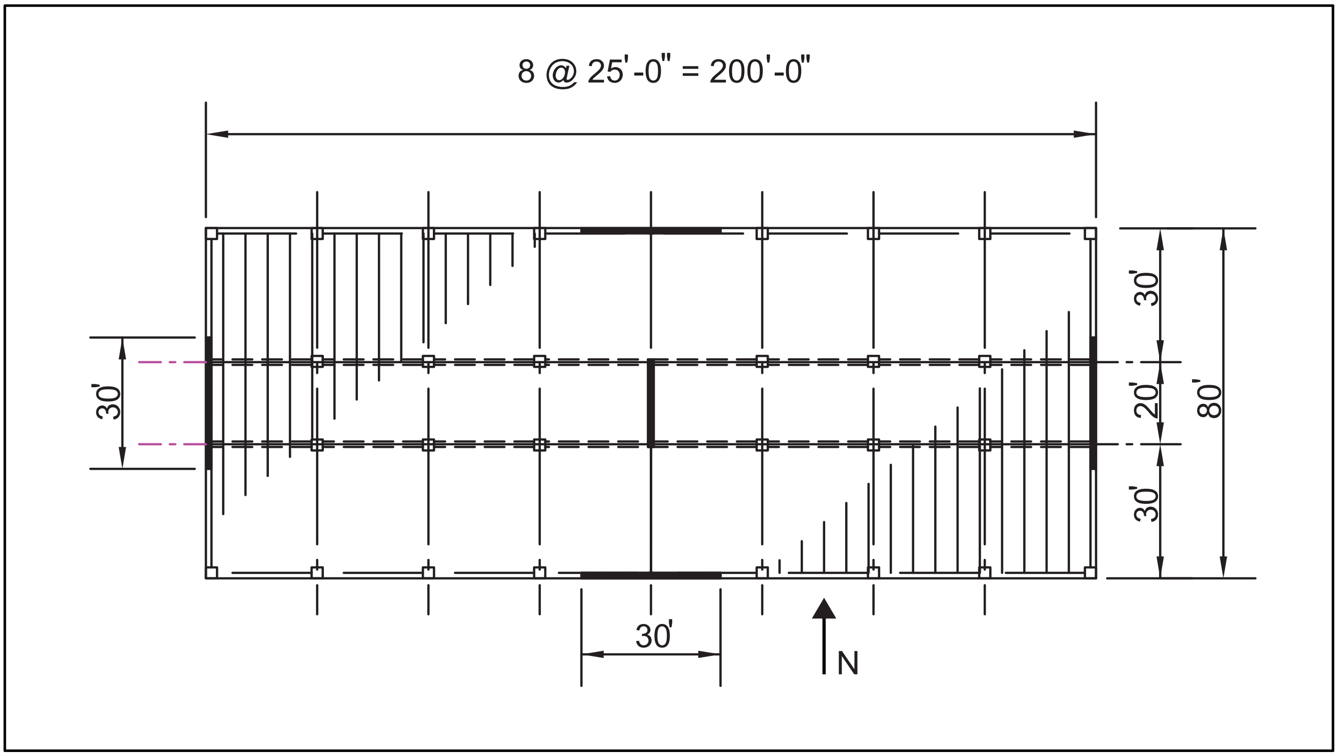
Given the building plan in Fig. 4.9.1, design and detail the untopped hollow core slab diaphragm assuming:

a. Wind design per ASCE 7-10.
b. Seismic design per ASCE 7-10 for SDC B.

Building data:

6 stories without parapet
Risk Category II
14 ft floor-to-floor

Figure 4.9.1 Example Problem



Weight of 8-in. hollow core slabs
= 53.5 lb/ft^2
Weight of partitions and mechanical equipment = 20 lb/ft^2
Weight of precast concrete framing system
= 32 lb/ft^2
Weight of exterior wall system (average)
= 35 lb/ft^2

Solutions:

a. Wind design; basic wind speed 130 mph

For wind design, it is assumed that the building is located in an area with a basic wind speed of 130 mph, per Fig. 26.5-1A of ASCE 7-10.

The mean roof height h is 14 ft × 6 = 84 ft > 60 ft. Thus use the Part 1 directional procedure in ASCE 7-10 Chapter 27 for the design of the main wind force-resisting system (MWFRS).

h = 84 ft < 300 ft
L_{eff} = 80 ft for wind in the N-S direction
= 200 ft for wind in the E-W direction
h/smaller L_{eff} =84/80 < 4

Therefore, approximate natural frequency, n_a, may be used for fundamental natural frequency, n_1.
The approximate lower-bound natural frequency of the building is:

$$n_a = \frac{385(C_w)^{0.5}}{h} \quad \text{(ASCE 7-10 Eq. 26.9-5)}$$

where

$$C_w = \frac{100}{A_B}\sum_{i=1}^{n}\left(\frac{h}{h_i}\right)^2 \frac{A_i}{\left[1+0.83\left(\frac{h_i}{D_i}\right)^2\right]}$$

In the N-S direction:

$$C_w = \frac{100}{16000}\left[\frac{2(30)}{1+0.83\left(\frac{84}{30}\right)^2} + \frac{20}{1+0.83\left(\frac{84}{20}\right)^2}\right]$$

$$= \frac{1}{160}[7.9923 + 1.2787] = 0.0579$$

Table 4.9.1 Calculation of velocity pressures, q_z, along height of building

Level	Height, z, ft	K_z[1,2]	K_{zt}	K_d	V, mph	q_z[3], lb/ft^2
6	84	1.220	1	0.85	130	44.8
5	70	1.174	1	0.85	130	43.2
4	56	1.120	1	0.85	130	41.2
3	42	1.054	1	0.85	130	38.7
2	28	0.968	1	0.85	130	35.6
1	14	0.849	1	0.85	130	31.2

[1] K_h = 1.220

[2] Combined height and exposure coefficient, $K_z = 2.01(z/z_g)^{2/\alpha}$ for $z \geq 15$ ft
$= 2.01(15/z_g)^{2/\alpha}$ for $z \leq 15$ ft
from ASCE 7-10 Table 27.3-1
Coefficient $\alpha = 9.5$ and gradient height, $z_g = 900$ lb for exposure C from ASCE 7-10 Table 26.9-1

[3] $q_h = 44.8$ lb/ft^2

$$n_a = \frac{385(C_w)^{0.5}}{h}$$

$$= \frac{385(0.0579)^{0.5}}{84} = 1.10\,Hz > 1.0\,Hz$$

In the E-W direction

$$C_w = \frac{100}{16000}\left[\frac{2(30)}{1+0.83\left(\frac{84}{30}\right)^2}\right]$$

$$= \frac{1}{160}[7.9923] = 0.05$$

$$n_a = \frac{385(0.05)^{0.5}}{84} = 1.02\,Hz > 1.0\,Hz$$

Thus, according to the definition in ASCE 7-10 Section 26.2, the building is rigid in both directions.

Directionality factor: $K_d = 0.85$
Exposure category: C
Topographic effect factor: $K_{zt} = 1.0$
Gust effect factor: taken as 0.85 because building is rigid.
Enclosure classification: enclosed
Internal pressure coefficient for MWFRS:

$GC_{pi} = \pm 0.18$

External pressure coefficients for MWFRS:

Windward wall: $C_p = 0.8$
Leeward wall: $C_p = -0.5$ for north-south wind, (L/B = 80/200 = 0.4)
$C_p = -0.3$ for east-west wind, (L/B = 200/80 = 2.5)
Side walls: $C_p = -0.7$

Design wind pressure for the MWFRS of rigid buildings of all heights is

$$p = qGC_p - q_i(GC_{pi})$$

where $q = q_z$ for windward walls at height z above the ground; $q = q_h$ for leeward walls, side walls, and roof evaluated at mean roof height h; and $q_i = q_h$ for windward, leeward, sidewalls, and roof of enclosed buildings. The values for q_z (calculated in Table 4.9.1), q_zGC_p (windward pressure), q_hGC_p (leeward pressure), and design wind forces in the north-south (NS) and east-west (EW) directions are summarized in Table 4.9.2.

- Consider load applied parallel to the hollow core spans (NS direction).

  The maximum load in the NS wind direction occurs at floor level 5 (from Table 4.9.2).

  $F_5 = 135.5$ kip

  The corresponding uniformly distributed wind load is:

  $w_5 = 135.5/200 = 0.678$ kip/ft

Table 4.9.2 Wind forces in the north-south and east-west directions

Level	Tributary Height, ft	q_z, lb/ft²	North-south direction				East-west direction			
			q_zGC_p, lb/ft²	q_hGC_p, lb/ft²	Total design wind pressure, lb/ft²	Total design wind forces, kip	q_zGC_p, lb/ft²	q_hGC_p, lb/ft²	Total design wind pressure, lb/ft²	Total design wind forces, kip
6	7.00	44.8	30.5	-19.1	49.6	69.4	30.5	-11.4	41.9	23.5
5	14.00	43.2	29.3	-19.1	48.5	135.5	29.3	-11.4	40.7	45.7
4	14.00	41.2	28.0	-19.1	47.1	131.8	28.0	-11.4	39.4	44.2
3	14.00	38.7	26.3	-19.1	45.4	127.1	26.3	-11.4	37.7	42.3
2	14.00	35.6	24.2	-19.1	43.3	121.1	24.2	-11.4	35.6	39.9
1	14.00	31.2	21.2	-19.1	40.3	112.8	21.2	-11.4	32.6	36.5
					∑	**697.7**				**232.1**

Assuming a rigid diaphragm, the shear distribution to the walls is based on the relative stiffnesses of the walls which can be approximated by flexural stiffness.

$$K_{30} = \frac{30^3(1)}{12} = 2250 \text{ and}$$

$$K_{20} = \frac{20^3(1)}{12} = 667$$

for a 12 in. thick wall.

The shear force distributed to each wall is:

30 ft walls:

$$V_{30} = \frac{K_{30}}{2(K_{30}) + K_{20}} F_5$$

$$= \frac{2250}{2(2250)+667}(135.5)$$

$$= 0.436(135.5) = 59.1 \text{ kip}$$

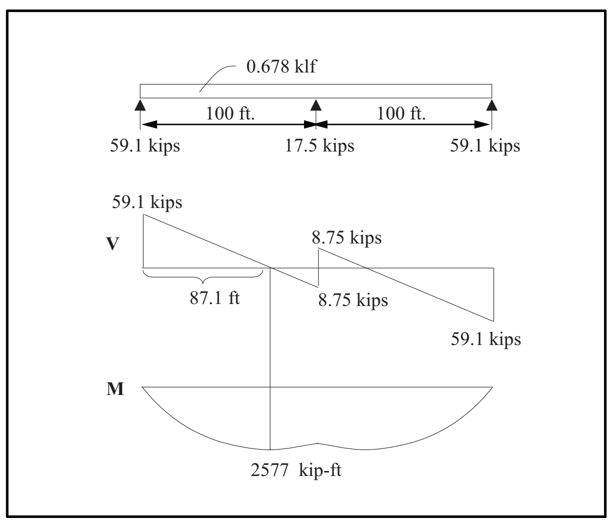
20 ft wall:

$$V_{20} = \frac{K_{20}}{2(K_{30}) + K_{20}} F_5$$

$$= \frac{667}{2(2250)+667}(135.5)$$

$$= 0.129(135.5) = 17.5 \text{ kip}$$

The diaphragm equilibrium is shown in the following illustration:



Location of maximum moment

$$= \frac{59.1}{(59.1+8.75)}(100) = 87.1 \text{ ft from}$$

left support

Maximum moment

$= 59.1(87.1) - 0.678(87.1^2/2)$
$= 5148 - 2571 = 2577$ kip-ft

- Chord forces:

Using the perimeter beams as chords:

$N_u = \dfrac{M_u}{\phi d}$ where d is taken as 0.8 times the depth of the diaphragm

$$= \frac{2577}{0.9(0.8)(80)} = 44.7 \text{ kip}$$

Connect beams to columns for this force plus forces due to volume change and gravity loads. (Fig. 4.9.2 detail C)

The chord must continue through the center wall.

$A_s = \dfrac{N_u}{f_y}$ (ϕ was included in N_u)

$$= \frac{44.7}{60}$$
$$= 0.75 \text{ in.}^2$$

Use two #6 bars (Fig. 4.9.2 detail F)

- Connect diaphragm web to chords

$$V_{uh} = \frac{M_u}{d}$$

where d is taken as 0.8 times the depth of the diaphragm

$$V_{uh} = \frac{2577}{0.8(80)}$$
$$= 40.3 \text{ kip}$$

Distribute over length from zero moment to maximum moment

$$V_{uh} = \frac{40.3}{87.1} = 0.46 \text{ kip/ft}$$

Additionally, this connection must resist the negative wind pressure from the exterior wall system, which is the sum of external and internal pressure on the exterior wall.

Internal pressure

$p_i = q_i GC_{pi} = 44.8(0.18) = 8.1$ lb/ft^2,
assuming $q_i = q_h$ and that building is enclosed

$w_u = (19.1 + 8.1)(14) = 381$ lb/ft
larger than 300 lb/ft for structural integrity, use 381lb/ft. (Fig. 4.9.2 detail A)

The same forces must be resisted at the transverse joints. Use shear friction for the shear with bars placed in the keyways perpendicular to the transverse joint. With keyways at 3 ft on center:

$$A_s = \frac{N_u}{\phi f_y} + \frac{V_u}{\phi f_y \mu}$$

$$A_s = \frac{3(0.381)}{0.9(60)} + \frac{3(0.46)}{0.75(60)(1.4)}$$
$$= 0.043 \text{ in.}^2\text{/keyway}$$

Use #3 bar at every second keyway (Fig. 4.9.2 detail F)

- Longitudinal shear (shear parallel to longitudinal joints)

The maximum longitudinal joint shear is at the first joint from the 30 ft shear wall. Since connections will be made directly from the center bay to the shear wall, only the center bay joint length should be considered.

$V_{u30} = 59.1$ kip
$\phi V_n = \phi(0.08) h_{net} \ell_j$
$= 0.75(0.08)(8 - 2)(20 \times 12)$
$= 86.4$ kip

With concerns for shrinkage cracking in joints, transverse shear friction reinforcement can be provided in the transverse joints at each end of the center bay.

$$A_{vf} = \frac{V_u}{\phi f_y \mu}$$
$$= \frac{59.1}{0.75(60)(1.0)}$$
$$= 1.3 \text{ in.}^2 / 2 \text{ transverse joints}$$
$$= 0.65 \text{ in.}^2 \text{ per joint}$$

Use one #8 bar in each transverse joint (Fig. 4.9.2 detail B)

- Shear connection to 30 ft wall

$V_{u30} = 59.1$ kip

Additionally, negative wind pressure must be resisted across this joint, but would not be concurrent with shear. Structural integrity ties will control for this case.

$N_u = (0.3)(20)$
$= 6$ kip for bay

Using shear friction reinforcement

$A_{vf} = 1.3$ in.^{2} (from above) or

$$A_{s\,\text{required}} = \frac{N_u}{\phi f_y} = \frac{6}{0.9(60)} = 0.11\text{ in.}^2$$

does not control

Use six #5 bars located near hollow core slab ends

(Fig. 4.9.2 detail D)

Alternatively, mechanical connections of hollow core slab to wall could be used to transfer the same forces.

- Shear at center 20 ft wall:

With the rigid diaphragm assumption:

$V_{u20} = 8.75$ kip on each side of wall

$$A_{vf} = \frac{V_u}{\phi f_y \mu}$$

$$A_{vf} = \frac{8.75}{0.75(60)(1.0)}$$
$= 0.19$ in.^{2}

Use two #3 bars located near hollow core slab ends or use mechanical connections

(Fig. 4.9.2 detail E)

- Consider load applied perpendicular to the hollow core spans (EW direction).

The maximum load in the EW wind direction occurs at floor level 5 (from Table 4.9.2).

$F_5 = 45.7$ kip

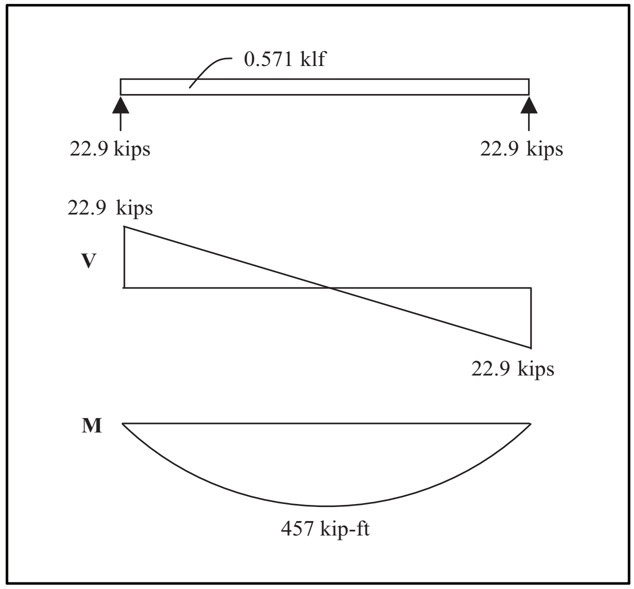
The corresponding uniformly distributed wind load is

$w = 45.7/80 = 0.571$ kip/ft

Shear distribution to 30 ft walls is

$V_{30} = 45.7/2 = 22.9$ kip

The diaphragm equilibrium is shown in the following illustration:



Maximum moment $= 0.571(80)^2/8$
$= 457$ kip-ft

- Chord force:

$N_u = \dfrac{M_u}{\phi d}$ where d is taken as 0.8 times the depth of the diaphragm

$$= \frac{457}{0.9(0.8)(200)}$$
$= 3.2$ kip

$$A_s = \frac{N_u}{f_y} = \frac{3.2}{60} = 0.053\text{ in.}^2$$

The #3 bars across the transverse joints will be adequate for the chord force. (Fig. 4.9.2 detail B)

Longitudinal shear (shear parallel to longitudinal joints)

$V_{uh} = \dfrac{M_u}{d}$ where d is taken as 0.8 times the depth of the diaphragm

$$= \frac{457}{0.8(200)}$$
$= 2.9$ kip will not control when compared to 59.1 kip applied in the NS direction

- Shear connection to walls

Using shear friction reinforcement

$$V_u = 22.9 \text{ kip/30 ft wall}$$
$$= 0.76 \text{ kip/ft}$$

With bars in keyways at 3 ft on center

$$A_{vf} = \frac{V_u}{\phi f_y \mu}$$

$$A_{vf} = \frac{3(0.76)}{0.75(60)(1.4)}$$
$$= 0.036 \text{ in.}^2 \text{ per keyway}$$

Use a #3 bar in every second keyway

(Fig. 4.9.2 detail F)

- Shear in transverse joint

$$V_u = 22.9 - 0.571(30)$$
$$= 5.77 \text{ kip}$$

$$A_{vf} = \frac{V_u}{\phi f_y \mu}$$

$$A_{vf} = \frac{5.77}{0.75(60)(1.0)}$$
$$= 0.13 \text{ in.}^2$$

A #3 bar at every second keyway will be adequate

(Fig. 4.9.2 detail B)

b. SDC B Seismic design

For seismic design, it is assumed that the building, with zip code 02110, is located in Boston, Mass.

Risk category: II

Importance factor: $I_e = 1.0$

Site class: D

- Seismic design category

The mapped spectral accelerations at this site (based on its latitude and longitude or postal address), corresponding to 0.2-second and 1-second periods, are:

$$S_s = 0.217g$$
$$S_1 = 0.069g$$

Site coefficients (from ASCE 7-10 Tables 11.4-1 and 11.4-2):

$$F_a = 1.6,\ F_v = 2.4$$
$$S_{MS} = F_a S_s$$
$$= 1.6(0.217g) = 0.35g$$
$$S_{M1} = F_v S_1$$
$$= 2.4(0.069g) = 0.17g$$
$$S_{DS} = \frac{2}{3g} S_{MS} = 0.23$$
$$S_{D1} = \frac{2}{3g} S_{M1} = 0.11$$

According to ASCE 7-10 Tables 11.6-1 and 11.6-2, the building is assigned to SDC B.

- Seismic weight

The building weight attributable to a typical floor is:

$$w_i = 80(200)[\,0.0535 \text{ (hollow core)} + 0.020 \text{ (partitions)} + 0.032 \text{ (precast framing)}] + 14(0.035)(200 + 80)(2) \text{ (exterior walls)}$$
$$= 1962 \text{ kip}$$

For the roof:

$$w_{roof} = 80(200)(0.0535 + 0.020 + 0.032) + 7(0.035)(200 + 80)(2)$$
$$= 1825 \text{ kip}$$

The total weight is

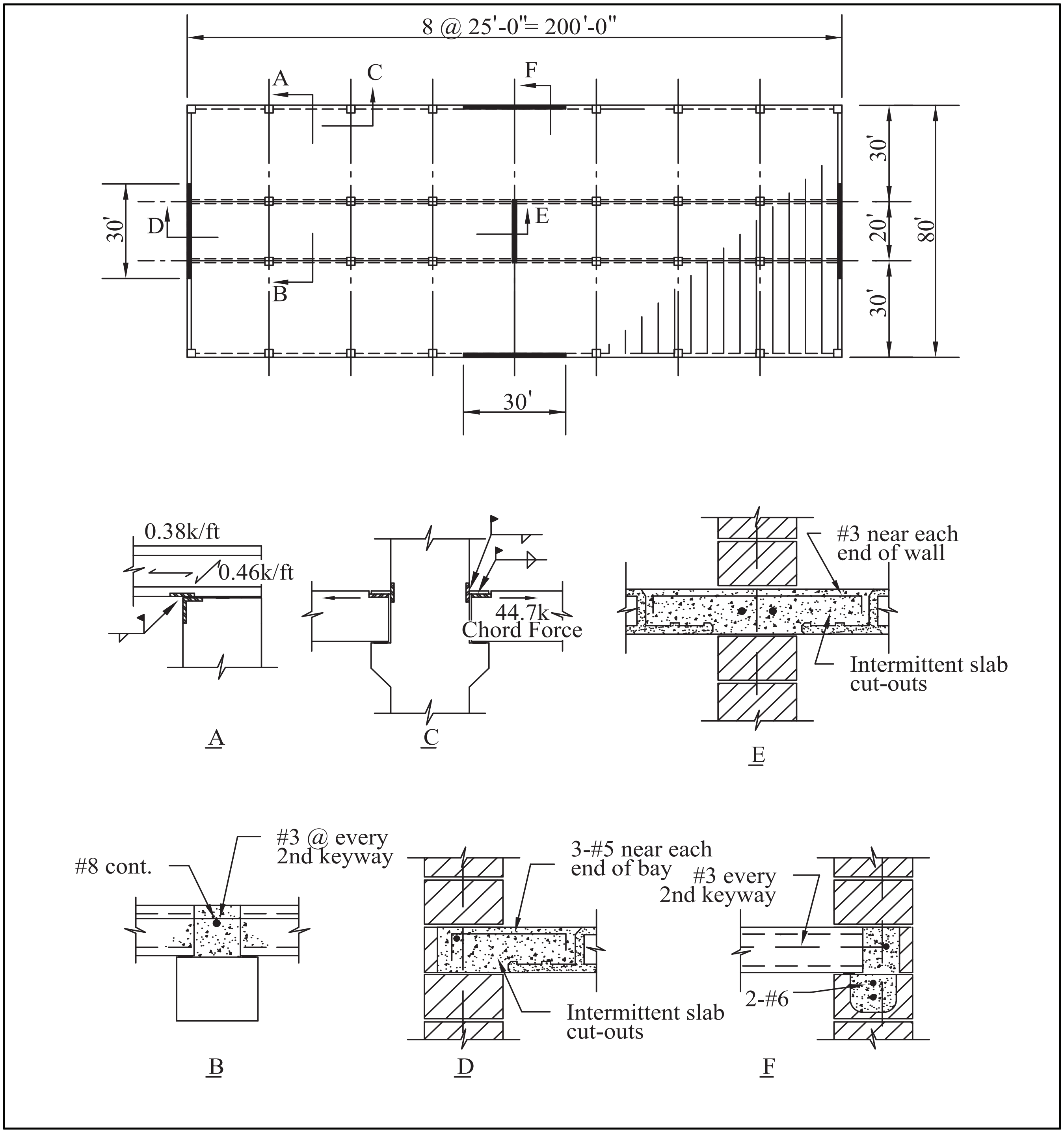
$$W = 5(1962) + 1825$$
$$= 11{,}635 \text{ kip}$$

- Base shear

The approximate building period is:

$$T_a = C_t h_n^x \qquad \text{(ASCE 7-10 Eq. 12.8-7)}$$
$$= 0.02(84)^{0.75}$$
$$= 0.55 \text{ sec}$$

Figure 4.9.2 Wind design summary



The seismic base shear is

$$V = C_s W$$

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$

$$= 0.23/5.0 = 0.046$$

where $R = 5$ for a building frame system with ordinary reinforced concrete shear walls (ASCE 7-10 Table 12.2-1).

C_s need not exceed

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \quad \text{For } T \leq T_L$$

$$= \frac{0.11}{0.55(5)} = 0.040$$

where $T_L = 6$ sec for Boston (from ASCE 7-10 Fig. 22-12)

C_s shall not be less than $0.044 S_{DS} I_e$ [$=0.044(0.23)(1) = 0.01$] nor less than 0.01.

Thus

$$C_s = 0.040$$
$$V = 0.040(11{,}635) = 465 \text{ kip}$$

- Vertical distribution

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

$k = 1.03$ for $T_a = 0.55$ sec (from ASCE 7-10 Section 12.8.3)

w_x, kip	h_x, ft	$w_x h_x^k$	F_x, kip
1825	84.0	175,094	128
1962	70.0	156,009	114
1962	56.0	123,975	90
1962	42.0	92,182	67
1962	28.0	60,712	44
1962	14.0	29,731	22
		∑= 637,703	∑= 465

- Diaphragm design force

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$$

w_{px}, kip	ΣF_i, kip	Σw_i, kip	F_{px}, kip
1825	128	1825	128
1962	242	3787	125
1962	332	5749	113
1962	399	7711	102
1962	443	9673	90
1962	465	11,635	78

The forces need not exceed

$0.4 S_{DS} I_e w_{px} = 0.4(0.23)(1.0)(1962)$

$= 181$ kip (168 kip for the roof diaphragm)

The forces must not be less than

$0.2 S_{DS} I_e w_{px} = 90$ kip

Thus, the design lateral force for the roof diaphragm is

$F_p = 128$ kip

Note that in the seventh edition of the *PCI Design Handbook*, low and moderate seismic design categories (SDC B and C) will be grouped together; no amplification of the code-specified seismic design force is considered necessary if the design force at the uppermost level is used for every floor diaphragm. The same recommendation will apply also to structures assigned to high seismic design categories (D, E, F) if lateral forces are resisted entirely by special moment frames. For SDC D, E, and F structures where shear walls are part of the lateral force-resisting system, a multiplier of 2 applies to the roof-level diaphragm design force; this amplified force is then to be kept constant down the height of the building.

- Consider diaphragm design force acting parallel to the hollow core spans (NS direction).

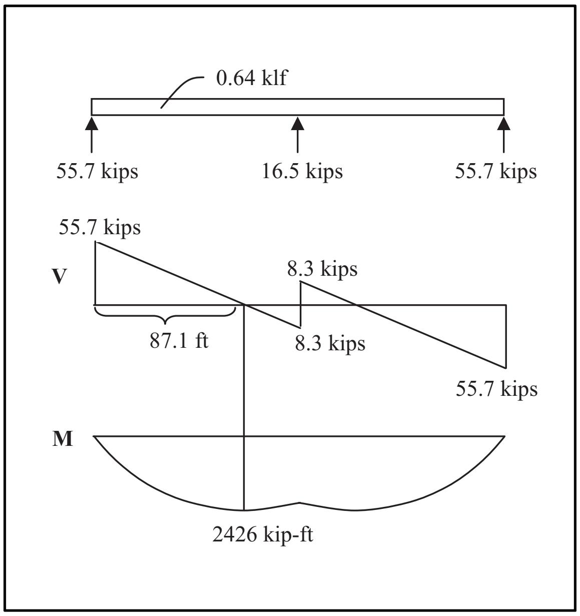
The equivalent uniformly distributed lateral load is

w_u = 128/200 = 0.64 kip/ft

Using a rigid diaphragm, the shear distribution to the walls is:

30 ft walls: V_{u30} = 55.7 kip
20 ft wall: V_{u20} = 16.5 kip

The diaphragm equilibrium is shown in the following illustration.



Location of maximum moment

$$= \frac{55.7}{(55.7+8.3)}(100) = 87.1 \text{ ft from left}$$
support

Maximum moment

$= 55.7(87.1) - 0.64(87.1)^2/2 = 2426$ kip-ft

- Chord forces:

Using reinforcement in a perimeter boundary element

$A_s = \dfrac{M_u}{\phi d f_y}$ where d is taken as 0.8 times the diaphragm depth

$$= \frac{2426}{0.9(0.8)(80)(60)}$$
$= 0.7$ in.^{2}

Four #4 bars would satisfy this requirement. (However, four #6 bars are provided because the force in the EW direction perpendicular to the spans controls, as shown later.)

(Fig. 4.9.3 detail A)

- Connect diaphragm web to chord

$V_{uh} = \dfrac{M_u}{d}$ where d is taken as 0.8 times the diaphragm depth

$$= \frac{2426}{0.8(80)}$$
$= 37.9$ kip

Distribute over length from zero moment to maximum moment

$$V_{uh} = \frac{37.9}{87.1} = 0.44 \text{ kip/ft}$$

Additionally, this connection must resist the outward force from the exterior wall system. Per section 12.11 of ASCE 7-10, the design force for wall anchorage N_u should be the greater of the following:

- $0.4 S_{DS} k_a I_e w_w$

where $k_a = 1.0$ for rigid diaphragms
$= 0.4(0.23)(1.0)(1.0)(0.035 \times 14)$
[Note that ½ the wall height could be used at the top level, but conservatively use the full wall height since the 5th level lateral force is about the same as the top level.]
$= 0.045$ kip/ft

- $0.2\, w_w = 0.2(0.035 \times 14)$
$= 0.098$ kip/ft

- Per ACI 318-11

$N_u = 0.300$ kip/ft (at nominal tensile strength)

$$A_s = \frac{N_u}{\phi f_y} + \frac{V_u}{\phi f_y \mu}$$
$$= \frac{0.300}{1.0(60)} + \frac{0.44}{0.75(60)(1.4)}$$
$= 0.012$ in.^{2}/ft

Use #3 bars at 3 ft on center, grouted into cores

(Fig. 4.9.3 detail A)

At the transverse joint, the same shear parallel to the transverse joint as at the chord must be transferred. However, the tension should consider the inertial force from the weight of the exterior bay, which is the largest of the following:

- $0.4S_{DS}k_aI_ew_p = 0.4(0.23)(1.0)(1.0)(3.66) = 0.34$ kip/ft

  where $w_p = 14(0.035) + 30(0.0535 + 0.020 + 0.032) = 3.66$ kip/ft

- 20% of $w_p = 0.2(3.66) = 0.73$ kip/ft (controls)

$$A_s = \frac{N_u}{\phi f_y} + \frac{V_u}{\phi f_y \mu}$$

$$A_s = \frac{0.73}{0.9(60)} + \frac{0.44}{0.75(60)(1.4)}$$
$$= 0.021 \text{ in.}^2/\text{ft}$$

Use #3 bars at 3 ft on center in keyways

(Fig. 4.9.3 detail B)

- Longitudinal shear (shear parallel to longitudinal joints)

  The maximum longitudinal shear is at the first joint away from the 30-ft wall. Provide shear friction reinforcement in the two transverse joints and the two boundary elements for shear resistance. Conservatively consider 5% minimum eccentricity being resisted only in end walls.

$$V_{u30} = 55.7 + (0.05 \times 200)(128)/200 = 62.1 \text{ kip}$$

$$A_{vf} = \frac{V_u}{\phi f_y \mu}$$

$$A_{vf} = \frac{62.1}{0.75(60)(1.0)}$$
$$= 1.38 \text{ in.}^2 / 4 \text{ joints}$$
$$= 0.35 \text{ in.}^2 \text{ per joint}$$

Note that in the seismic calculation, this shear friction reinforcement was distributed over four joints as opposed to two joints in the wind calculation. In the seismic detailing, a collector is provided so the shear can be distributed over the full width of the building and the outside bays are available for the shear transfer. No collector was used in the wind calculation so this shear had to be resisted in the center bay only. With shear friction reinforcement provided at the outside edges of the outer bays, the demand for chord reinforcement must also be considered as additive to the shear friction reinforcement since both cause tension in the reinforcement. In this example, chord reinforcement was checked at maximum moment without shear and is checked here for moment associated with maximum shear.

At first joint

$$M_u = 55.7(3) - 3^2(0.64)/2 = 164 \text{ kip-ft}$$

$$A_s = \frac{V_u}{\phi f_y \mu} + \frac{N_u}{\phi f_y}$$

$$A_s = \frac{164}{0.9(0.8)(80)(60)} + 0.35$$
$$= 0.4 \text{ in.}^2$$

Four #6 bars OK

(Fig. 4.9.3 detail A)

In transverse joints,

$A_s = 0.35$ in.^{2}

Use two #5 bars.

(Fig. 4.9.3 detail B)

- Shear connection to 30 ft wall:

  Transfer shear to wall and collector element

$$V_{u30} = \frac{62.1}{80} = 0.78 \text{ kip/ft}$$

$$A_{vf} = \frac{V_u}{\phi f_y \mu}$$

$$A_{vf} = \frac{0.78}{0.75(60)(1.0)}$$
$$= 0.017 \text{ in.}^2/\text{ft}$$

Use #4 hairpins at 8 ft on center

(Fig. 4.9.3 detail D)

Collector element reinforcement

$N = \dfrac{80-30}{2}(0.78)$

$= 19.5$ kip

Special Note: In SDC B, amplification of the collector design force is not required. It is included here only to demonstrate the procedure.

$N_u = \Omega_0 N = 2.5(19.5)$
$= 48.8$ kip

$A_s = \dfrac{48.8}{0.9(60)}$
$= 0.9$ in.^{2}

Use two #7 bars

(Fig. 4.9.3 detail C)

- Shear connection at 20 ft wall

$V_{u20} = 8.3$ kip

over building width

$V_{u20} = \dfrac{8.3}{80} = 0.104$ kip/ft

$A_{vf} = \dfrac{V_u}{\phi f_y \mu}$

$A_{vf} = \dfrac{0.104}{0.75(60)(1.0)}$
$= 0.002$ in.^{2}/ft

Use #4 bars at 8 ft on center.

(Fig. 4.9.3 detail F)

Collector element reinforcement

$N = \dfrac{80-20}{2}(0.104)(2)$
$= 6.2$ kip

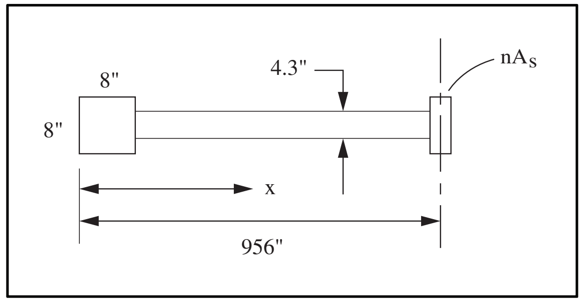
$N_u = \Omega_0 N = 2.5(6.2) = 15.5$ kip

$A_s = \dfrac{15.5}{0.9(60)}$
$= 0.29$ in.^{2}

Use two #4 bars

(Fig. 4.9.3 detail E)

- In-plane deflection of diaphragm
Idealize the diaphragm section as



With 4000 psi concrete in chord

$E_c = 57000\sqrt{f_c'}$

$= 57000\sqrt{4000} = 3605$ ksi

With 5000 psi concrete in hollow core slab

$E_c = 57000\sqrt{5000} = 4030$ ksi

Normalize with respect to concrete compressive strength in hollow core slabs

$n_{chord} = 3605/4030 = 0.89$
$A_{Tchord} = 0.89(8)(8)$
$= 57$ in.^{2}
$n_{steel} = 29{,}000/4030 = 7.2$
$nA_s = 7.2$(area of 4 #6 bars)
$= 7.2(1.76)$
$= 12.67$ in.^{2}

Solve for the neutral axis

$57(x-4) + 4.3(x-8)^2/2 = 12.67(956-x)$
find $x = 67.6$ in.

About the neutral axis

$I_{cr} = 57(67.6 - 4)^2 + 4.3(67.6 - 8)^3/3 + 12.67(956 - 67.6)^2$
$= 10{,}533{,}867$ in.^{4}
$= 508$ ft^4

If the diaphragm is considered rigid, the factored load deflection between end shear walls can be calculated as the deflection due to a uniform load spanning between the end shear walls minus the deflection due to the reaction at the center shear wall.

$$\Delta = \frac{5}{384}\frac{w\ell^4}{EI} - \frac{P\ell^3}{48EI}$$

$$\Delta = \frac{5}{384}\left[\frac{0.64(200^4)}{(4030)(508)(12)}\right] - \frac{16.5(200^3)}{48(4030)(508)(12)}$$

= 0.43 in. (ignoring shear deflections)

If the diaphragm is considered flexible with rigid supports, the deflection will be substantially smaller. The diaphragm deflection plus the deflection of the lateral force-resisting system is used to evaluate the gravity-load support members for integrity when deformed.

- Consider diaphragm design force acting perpendicular to the hollow core slabs (EW direction).

Total V_u = 128 kip

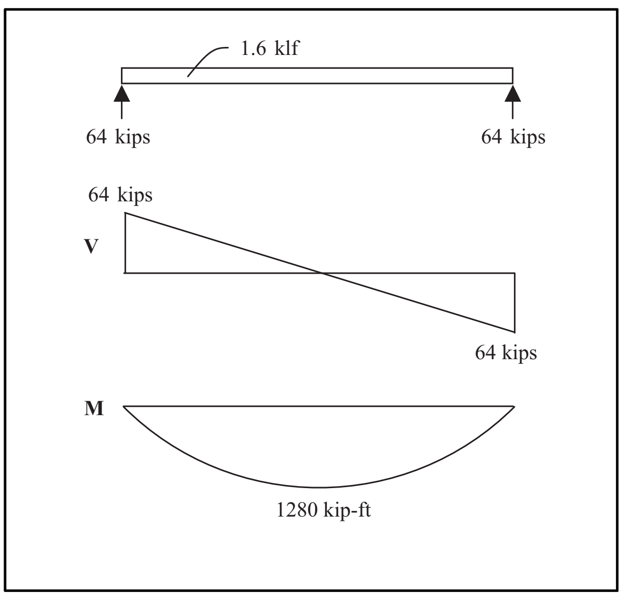
Distribution to walls is

V_u = 128/2 = 64 kip

The equivalent uniformly distributed lateral load is:

w = 128/80 = 1.6 kip/ft

The diaphragm equilibrium is shown in the following illustration



Maximum moment = $1.6(80)^2/8$ = 1280 kip-ft

- Chord force

$N_u = \dfrac{M_u}{\phi d}$ where d is taken as 0.8 times diaphragm depth

$= \dfrac{1280}{0.9(0.8)(200)}$

= 8.9 kip

$A_s = \dfrac{8.9}{60} = 0.15 \text{ in.}^2$

The #3 bars across the transverse joints at 3 ft on center will be adequate (Fig. 4.9.3 detail B)

Longitudinal shear

$V_{uh} = \dfrac{M_u}{jd}$

$\cong \dfrac{1280}{0.8(200)}$

= 8 kip will not control because it is much smaller than the longitudinal shear caused by the diaphragm design force acting in the orthogonal direction

- Shear connection to walls

With 5% eccentricity

V_u = 0.55(128) = 70.4 kip

Transfer shear to wall and collector element

V_u = 70.4/200 ft
= 0.35 kip/ft

The #3 bars at 3 ft on center in the grout key is adequate. (Fig. 4.9.3 detail G)

Collector reinforcement

$N = \dfrac{200-30}{2}(0.35)$

= 29.8 kip

$N_u = \Omega_0 N = 2.5(29.8) = 74.5$ kip

$A_s = \dfrac{74.5}{0.9(60)} = 1.38 \text{ in.}^2$

The chord reinforcement of four #4 bars is not enough. Use four #6 bars. (Fig. 4.9.3 detail A)

- Shear in transverse joint

Considering 5% eccentricity, the shear force resisted by one EW exterior wall is:

$$V_u = 64 + 0.05(80)(128)/80 = 70.4 \text{ kip}$$

The shear force at the transverse joint is:

$$V_u = 70.4 - 1.6(30) = 22.4 \text{ kip}$$

In center bay

$$w_p = 200(0.0535 + 0.020 + 0.032) + 14(0.035)(2) = 22.1 \text{ kip/ft}$$

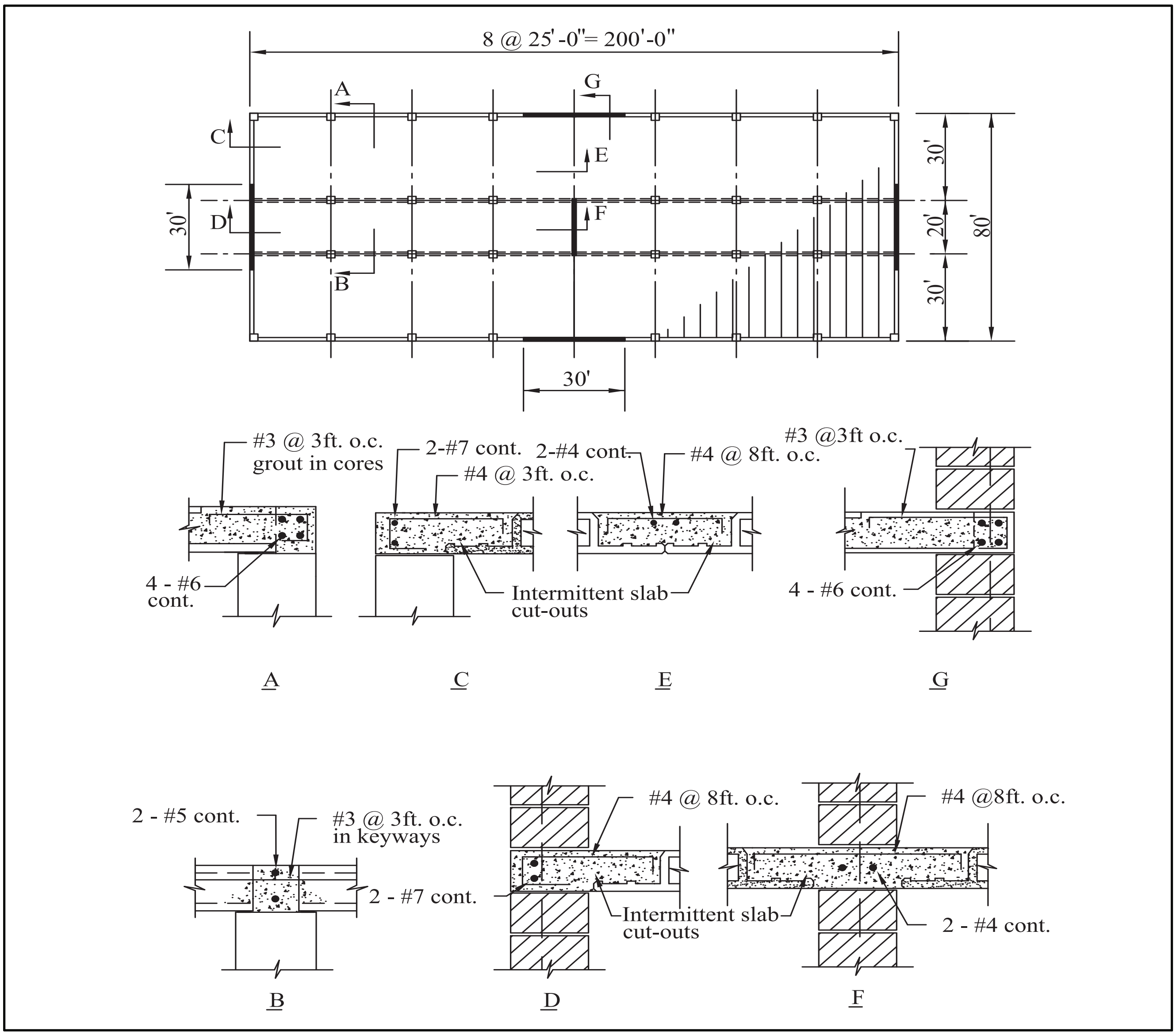
Conservatively use the largest of the following:

- $0.4S_{DS}\,k_a\,I_e w_p = 0.4(0.23)(1.0)(1.0)(22.1) = 2.0$ kip/ft
- 20% of $w_p = 0.2(22.1) = 4.4$ kip/ft (controls)

$V_u = 4.4(20)(0.55)$
= 48.4 kip per transverse joint including 5% eccentricity
= 48.4/200 = 0.24 kip/ft shear through transverse joint

Diaphragm design force in NS direction parallel to hollow core slabs at 0.44 kip/ft will control.

Figure 4.9.3 Seismic design summary



Chapter 5
HOLLOW CORE PANELS

5.1 Introduction

As an extension of their production of hollow core floor and roof slabs, many manufacturers also produce hollow core wall panels. Hollow core wall panels, floor slabs, and roof slabs are all produced in the same manner as discussed in Chapter 1 of this manual. In wall panels, prestressing strands are added to the top flange of the slab to produce uniform compression in the cross section and to provide flexural reinforcement for the reversible lateral loads imposed. Some systems allow for the use of reinforcing bars or welded-wire reinforcement as transverse reinforcement, while many panels are produced with no transverse reinforcement. The availability of hollow core wall panels varies throughout the country based on market demand and the production capabilities of the manufacturer.

Because appearance is more important for wall panels than for floor and roof slabs, the production of hollow core wall panels requires additional emphasis on such factors as finish appearance, uniformity of color, and consistency of joints. While acknowledging that the project will be judged on the basis of appearance as well as structural performance, the owner and architect should be made aware that hollow core wall panels are a machine-manufactured, structural product. As such, they are produced and erected to structural product tolerances rather than the more restrictive tolerances for architectural panels. If more stringent tolerances are required, they must be listed in the contract documents, and increased costs are likely to be incurred.

As with hollow core floor slabs, hollow core wall panels are most economical when certain guidelines are followed.

- Select building and opening dimensions to fit the standard panel module whenever possible.
- Minimize openings.
- Select repetitive connections and details.
- Choose a standard finish with few, if any, horizontal reveals or accents.

Although absolute adherence to these guidelines is not required, costs are likely to rise as the economy of repetition is lost.

While hollow core wall panels can be stacked horizontally, used as spandrels between horizontal ribbon windows, or stacked vertically for multistory buildings, the majority of panels are designed as single-piece panels spanning vertically in one- or two-story structures. Therefore, this manual will emphasize this type of building.

The structural design of hollow core wall panels is based on standard engineering principles; however, many aspects of the detailing and production are a function of the casting method. Items such as corner details, available finishes, insulation values, and connection details are often based on production limitations and past experience. Because of this, it is imperative that the hollow core wall panel supplier be consulted early in the design phase to assist in developing the most efficient and economical design.

5.2 Architectural Considerations

Hollow core wall panels are often considered to be architectural products by owners and architects. However, to maintain the economy of mass production, the wall panels are produced essentially the same as hollow core slabs; in essence, a structural product being used architecturally. As such, they will exhibit more variation in uniformity of color and finish and greater dimensional tolerances than a true architectural panel.

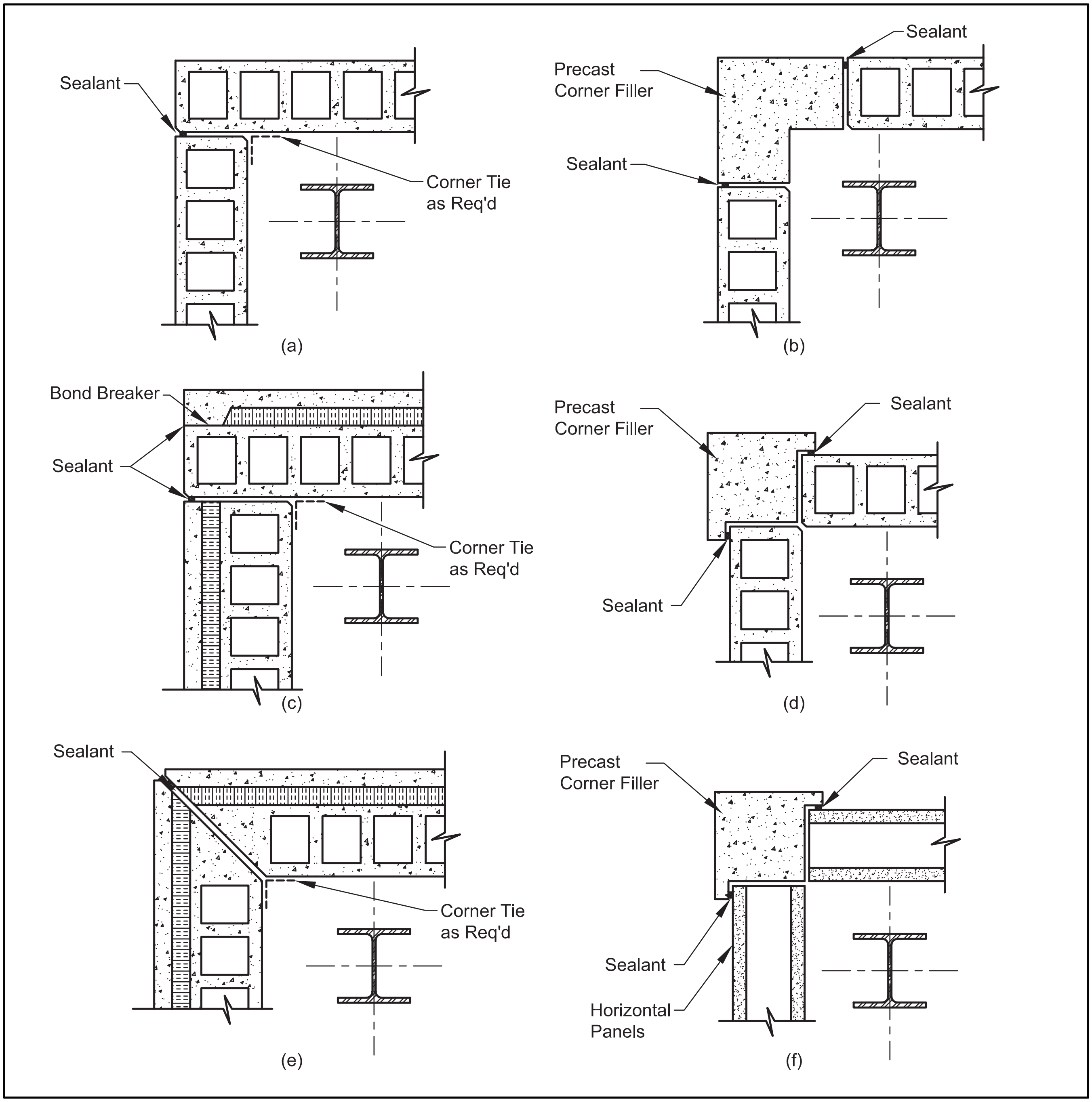
In sandwich wall panel construction, the structural wythe is generally designed to remain uncracked, but some cracking may occur in the finish wythe due to production methods, panel handling, or thermal gradients. For aesthetic reasons, steps should be taken to minimize or control this cracking. However, minor cracking of the finish face does not affect the structural integrity of the wall panel and should be considered acceptable.

5.2.1 Building Layout

Hollow core wall panels are produced in standard widths ranging from 2 ft to 8 ft, with 4 ft and 8 ft being the most common. To maximize the cost-efficiency of a hollow core wall panel building, the building should be laid out using standard width panels whenever possible. Casting special width panels or cutting standard panels to a special width will add to the cost of the project.

Figure 5.2.1 shows a number of different corner details. Factors such as insulated versus non-insulated panels, overall panel thickness, dimensions of corner filler (if any), and mitered versus butt-jointed corners will affect the overall building dimensions.

Figure 5.2.1 Corner details



5.2.2 Wall Openings

The size and location of openings will also affect the efficiency of a hollow core wall panel project. Again, the standard panel module should be the basis for establishing the opening layout.

There are three common methods to create openings:

- Formed on the casting bed
- Sawcut in the yard or field prior to erection
- Sawcut in-place after erection

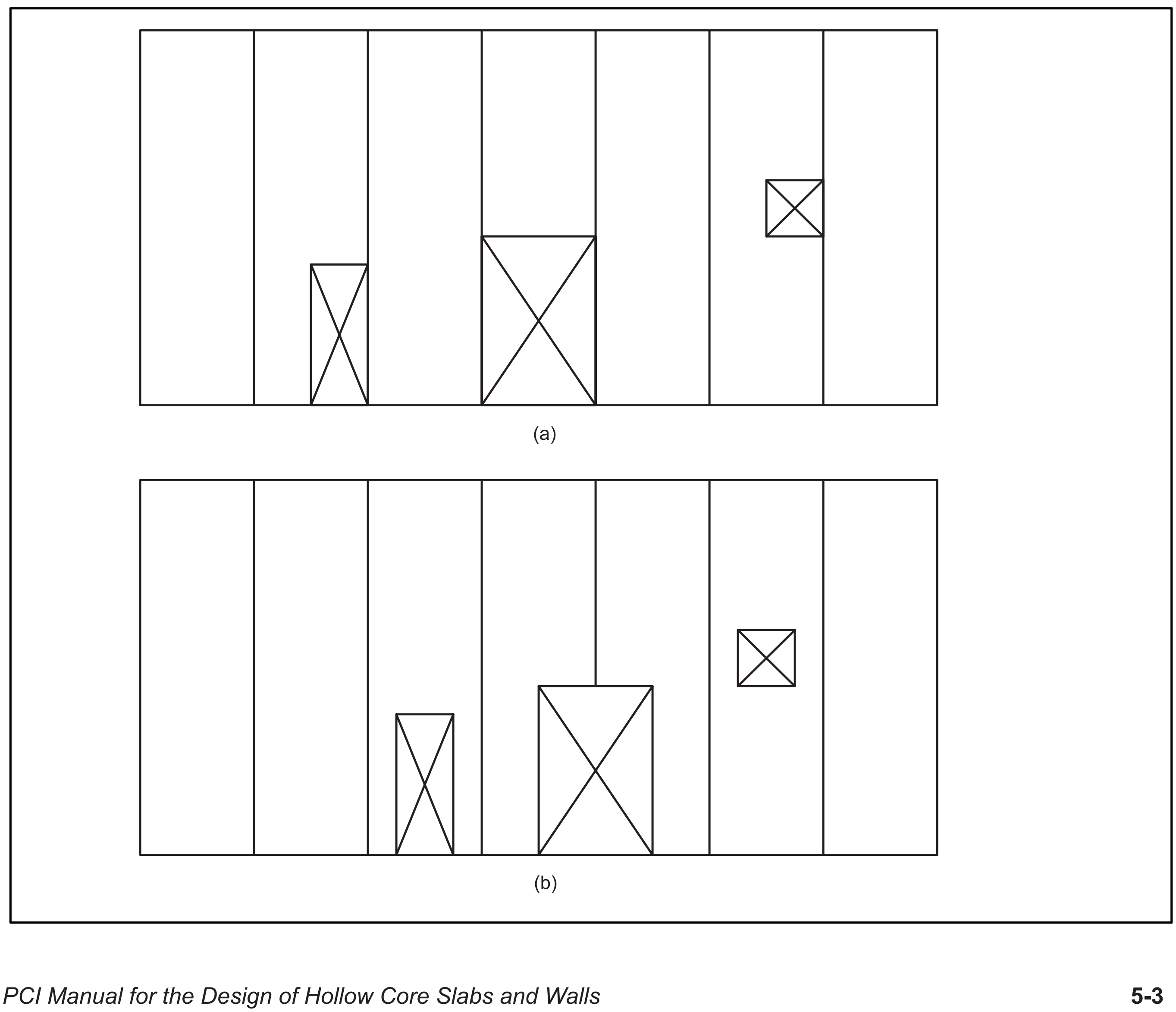
The method used will affect the desirable placement of the openings. Figure 5.2.2 shows two approaches to laying out a series of openings in a building elevation.

Hollow core producers who form the openings or cut them prior to erection will generally prefer the configuration of Fig. 5.2.2(a). The advantages include:

- Fewer linear feet of sawcutting;
- No concern about aligning door / window heads across the vertical joint; and
- The remaining panel is stocky enough to be handled and erected without cracking through the reduced section.

Producers who sawcut the openings after erection often prefer the layout shown in Fig. 5.2.2(b). Its advantages include:

Figure 5.2.2 Wall openings



(a)

(b)

- A structural frame may not be required at the overhead door opening; and
- Symmetric placement of openings within panels is often preferred architecturally.

The hollow core producer should be consulted early in the design process, because the preferred method is a function of the panel production system, the cost and availability of sawcutting equipment and subcontractors, and past experience.

No matter how the hollow core wall panels are laid out or the openings cut, there must be sufficient panel remaining adjacent to or between openings to resist the design loads. This will be addressed in more detail in section 5.3 of this chapter.

Framing of openings must also be considered with hollow core wall panels. As shown in Fig. 5.2.3, the jamb of a cut opening may expose a wall panel core. Similarly, cores will be exposed at the head of an opening. Although most manufacturing systems allow for the cores around openings to be provided as solid, this may require additional labor in the production process and, therefore, become an added expense. At service doors, a wide, hollow, metal door frame is often adequate to cover the exposed cores. At overhead doors, a precast concrete frame or a steel frame made of plates and structural steel sections is often used. This frame not only covers the exposed cores, it can be designed to support the panels above the opening, transfer lateral forces to adjacent panels, and provide armoring for the panel edges.

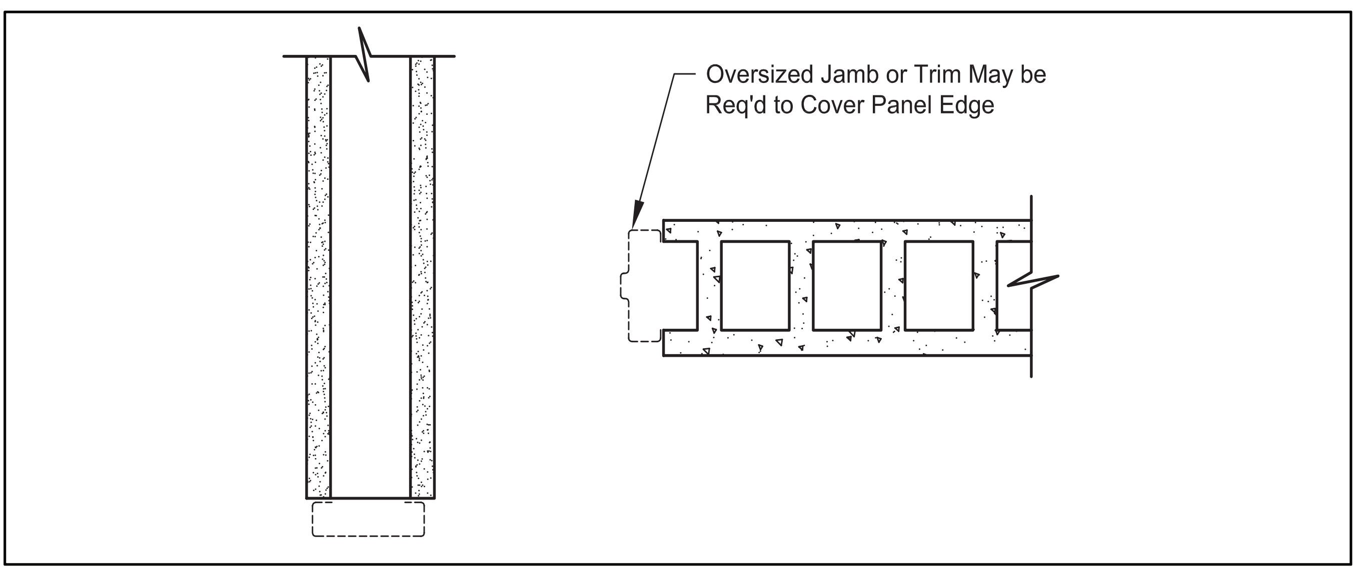
5.2.3 Finishes

Numerous finishes are available with hollow core wall panels. The most common are smooth, ribbed, raked, and exposed aggregate.

Smooth finishes include the plain, as-cast finish or a sand-blasted finish. The hollow core wall panels may be used as supplied or finished with a paint or stain. Ribbed finishes may include either a regular or a random spacing of flutes. Some hollow core producers are able to combine the ribbed finish with exposed aggregate. Raked finishes are created as the name implies. The surface is literally raked in a regular or random pattern to create a deeply textured surface. Exposed-aggregate finishes use chemical means to expose the aggregate to view. The color and texture of the finished panel will vary based on the size, shape, and color of the aggregate used.

Many variations of the above finishes are available; however, not all finishes may be avail able from all hollow core producers. Some finishes may only be available with hollow core sandwich wall panels. Therefore, final selections should be made after consultation with local hollow core producers.

Figure 5.2.3 Head and jam details



5.2.4 Insulation

Hollow core wall panels may be supplied as either insulated or non-insulated panels. The insulated wall panels may have their cores filled with an insulating material or may be cast as a sandwich panel. Sandwich wall panels are created by placing a layer of rigid insulation on the hollow core section, then casting a finish layer of concrete (Fig. 5.2.4).

Hollow core wall panels with insulation-filled cores have far less thermal resistance than hollow core sandwich wall panels; however, depending on the climate and the building usage, this may be an economical solution. Calculation of the thermal resistance of insulation-filled hollow core wall panels is complicated by the thermal bridges created by the concrete webs.

Various types of insulation are available to fill the hollow core wall panel cores. Table 5.2.1 lists the approximate thermal resistance properties for wall panel assemblies with and without filled cores.

The thermal resistance of hollow core sandwich wall panels can be calculated more accurately because the insulation layer will provide the majority of the thermal resistance. In addition, the thermal performance can be readily varied by the selection of insulation type and thickness. Table 5.2.2 lists approximate *U*-values for hollow core sandwich wall panels.

An additional thermal performance benefit of concrete wall panels is their ability to absorb and store large quantities of heat. This thermal storage effect means that concrete walls are slow to warm and cool, thereby reducing peak heating and cooling loads and delaying the time these peak loads occur by several hours.

For example, as the temperature of the exterior face of a wall rises, the temperature of the inside face rises much more slowly for a concrete wall than for a metal or wood stud wall. By delaying the temperature rise by several hours, the exterior temperature will have begun to moderate before the interior face reaches the peak temperature.

Figure 5.2.5 compares the heat flow through three walls made of different materials but each with the same *U*-value. The walls were exposed to temperatures simulating a typical spring day. Because of the heat storage effects, the peak loads for the concrete wall were approximately 13% lower for heating and 30% lower for cooling than for the stud walls.

Figure 5.2.4 Hollow core sandwich wall panel

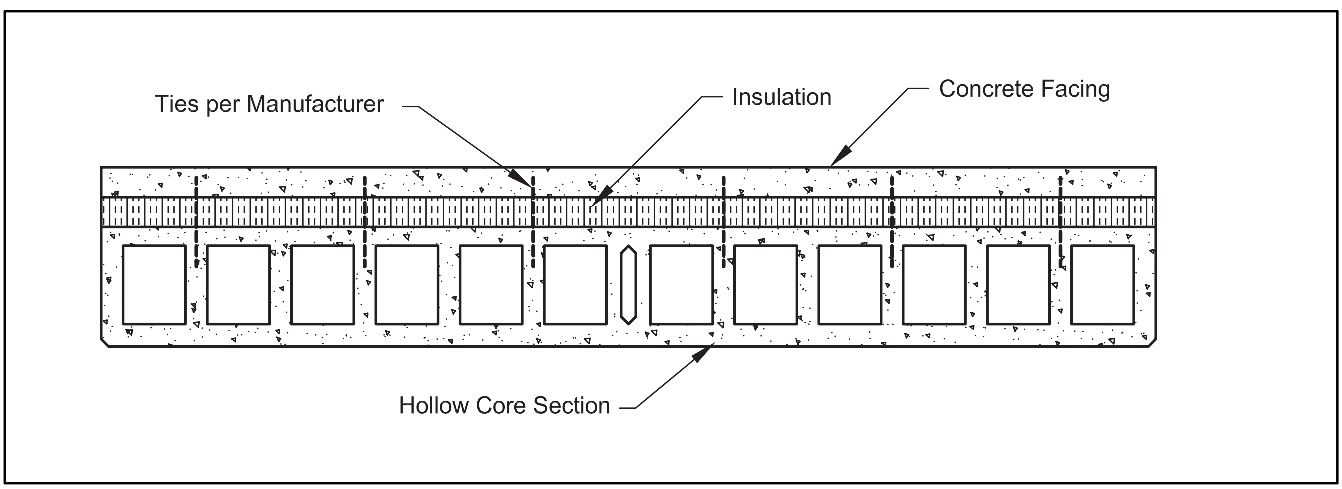
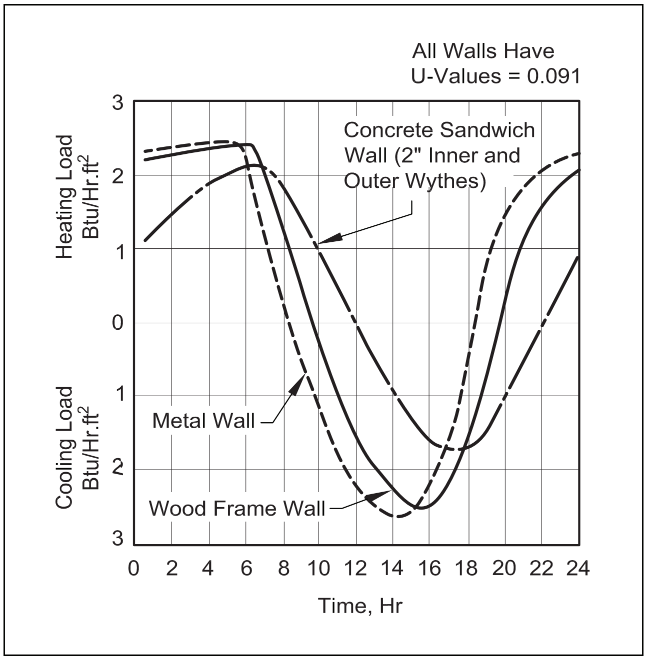


Figure 5.2.5 Heating and cooling load comparison



5.2.5 Thermal Bowing

A full discussion of thermal bowing is presented in Section 5.8.5 of the *PCI Design Handbook: Precast and Prestressed Concrete*[1], seventh edition. To summarize that section, as the warm side of a concrete wall expands relative to the cool side, the wall will bow. Differential shrinkage between the inside and outside faces of the concrete will also cause bowing. The thermal bowing can be reasonably estimated by the equations in Section 5.8.5; however, the calculation of shrinkage bowing is much more approximate and is seldom done.

For non-load-bearing wall panels, bowing is primarily an aesthetic issue. For vertical panels, bowing at the building corners may be visually noticeable and may result in joint sealant failure. If the building has a mid-height mezzanine, bowing may create a gap between the back face of the panel and the edge of the floor. Bowing in horizontal panels is most noticeable when a gap is created at a mezzanine floor, or waviness appears

Table 5.2.1 Wall assembly *U*-values, hollow core wall panels

Concrete density, lb/ft^3	Thickness, in.	Resistance, *R*, of concrete	*U*-value, Winter R_{fo} = 0.17 R_{fi} = 0.8	*U*-value, Summer R_{fo} = 0.25 R_{fi} = 0.68
145	6 (o)	1.07	0.52	0.50
	(f)	1.86	0.37	0.36
	8 (o)	1.34	0.46	0.44
	(f)	3.14	0.25	0.25
	10 (o)	1.73	0.39	0.38
	(f)	4.05	0.20	0.20
	12 (o)	1.91	0.36	0.35
	(f)	5.01	0.17	0.17
110	8 (o)	2.00	0.35	0.34
	(f)	4.41	0.19	0.19
	12 (o)	2.59	0.29	0.28
	(f)	6.85	0.13	0.13

Note: (o) and (f) designate cores open or cores filled with insulation.

Table 5.2.2 Wall assembly U-values, hollow core wall panels

Concrete density, lb/ft³	Thickness, in.	Resistance, *R*, of concrete	*U*-value, Winter R_{fo} = 0.17 R_{fi} = 0.68				*U*-value, Summer R_{fo} = 0.25 R_{fi} = 0.68			
			Insulation resistance, *R*							
			4	6	8	10	4	6	8	10
145	6 (o)	1.07	0.17	0.13	0.10	0.08	0.17	0.13	0.10	0.08
	(f)	1.86	0.15	0.11	0.09	0.08	0.15	0.11	0.09	0.08
	8 (o)	1.34	0.16	0.12	0.10	0.08	0.16	0.12	0.10	0.08
	(f)	3.14	0.13	0.10	0.08	0.07	0.12	0.10	0.08	0.07
	10 (o)	1.73	0.15	0.12	0.09	0.08	0.15	0.12	0.09	0.08
	(f)	4.05	0.11	0.09	0.08	0.07	0.11	0.09	0.08	0.07
	12 (o)	1.91	0.15	0.11	0.09	0.08	0.15	0.11	0.09	0.08
	(f)	5.01	0.10	0.08	0.07	0.06	0.10	0.08	0.07	0.06
110	8 (o)	2.00	0.15	0.11	0.09	0.08	0.14	0.11	0.09	0.08
	(f)	4.41	0.11	0.09	0.08	0.07	0.11	0.09	0.07	0.07
	12 (o)	2.59	0.13	0.11	0.09	0.07	0.13	0.11	0.09	0.07
	(f)	6.85	0.09	0.07	0.06	0.06	0.08	0.07	0.06	0.06

Note: (o) and (f) after thickness designates cores open or cores filled with insulation.

where the panels support horizontal ribbon windows.

At these conditions, panel-to-panel connections at the corner or connections to the mezzanine may be required to minimize the movement. Equations in Section 5.8.5 of the *PCI Design Handbook* can be used to calculate the restraint forces and resulting bending moments developed.

Bowing of tall, slender, load-bearing wall panels can significantly affect the factored design moment by increasing the curvature of the panel and, consequently, increasing the P-Δ effects. See Section 5.3.4.2 in this chapter for a more complete discussion of this analysis.

5.2.6 Architectural Details

Figures 5.2.6 through 5.2.8 present a selection of architectural details used in the precast concrete industry. These details are only a representative sample of the details available. Local precast concrete producers should be consulted for their input.

Figure 5.2.6 Wall opening details — Steel frame

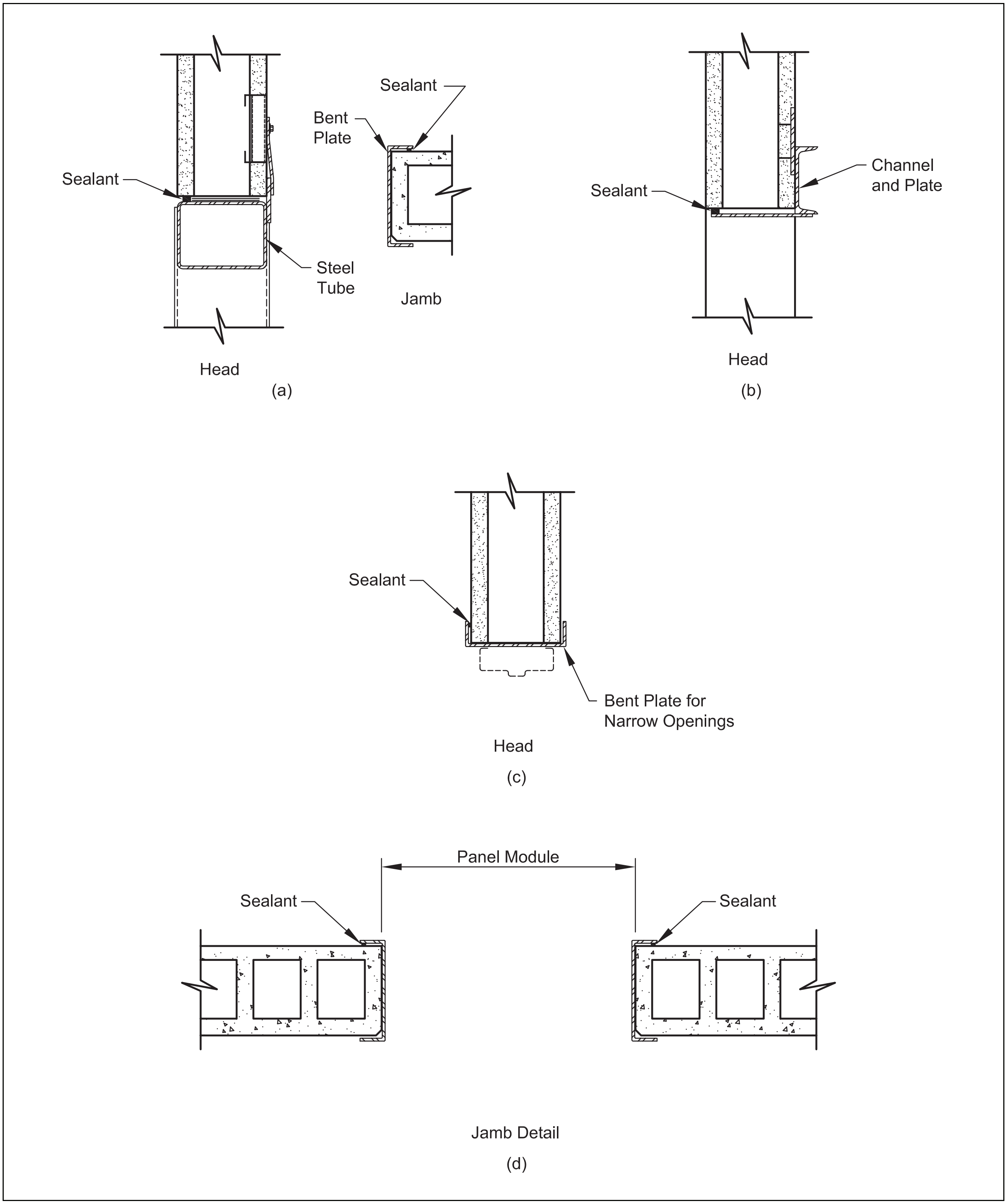


Figure 5.2.7 Wall opening details — Concrete frame

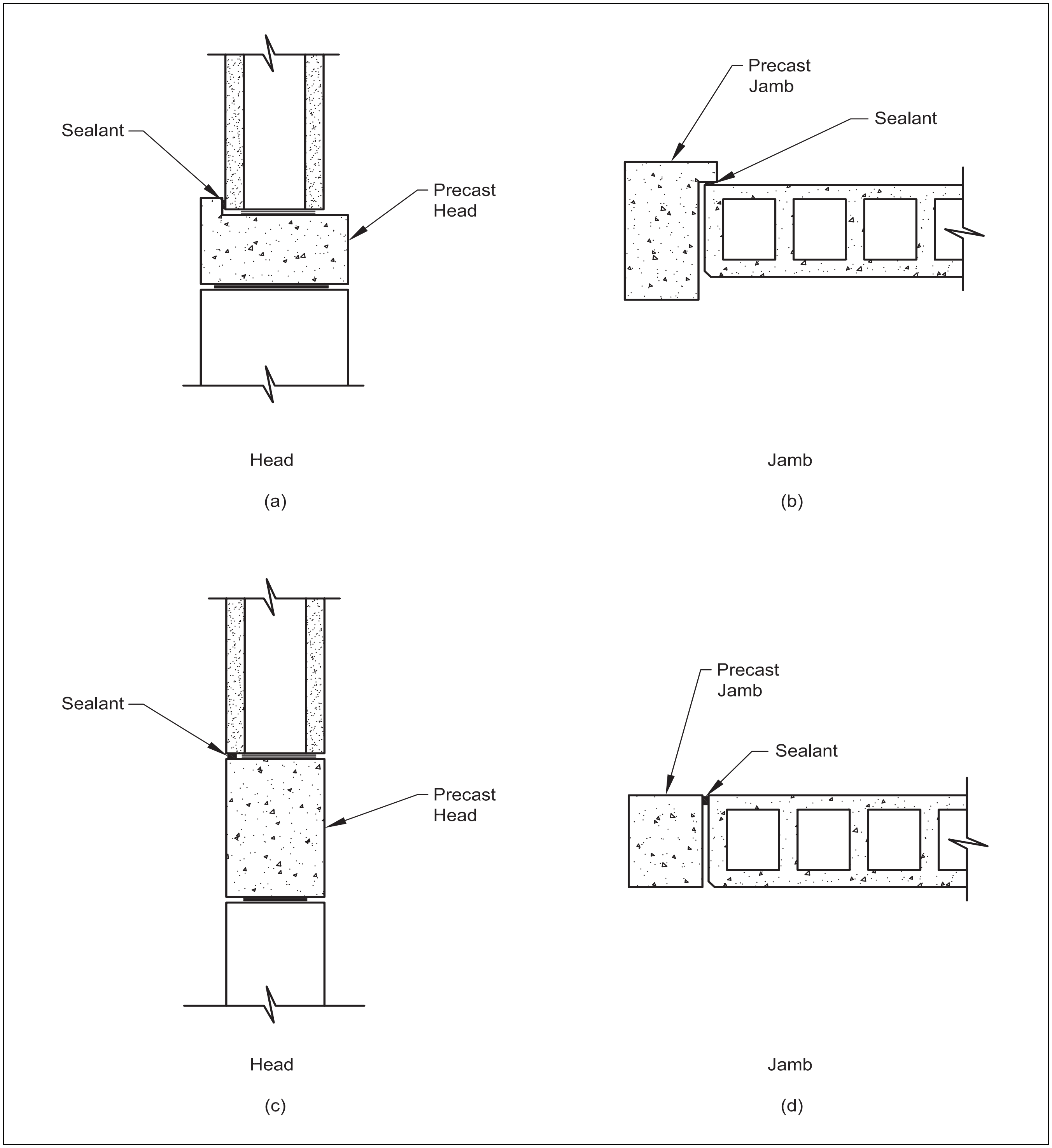
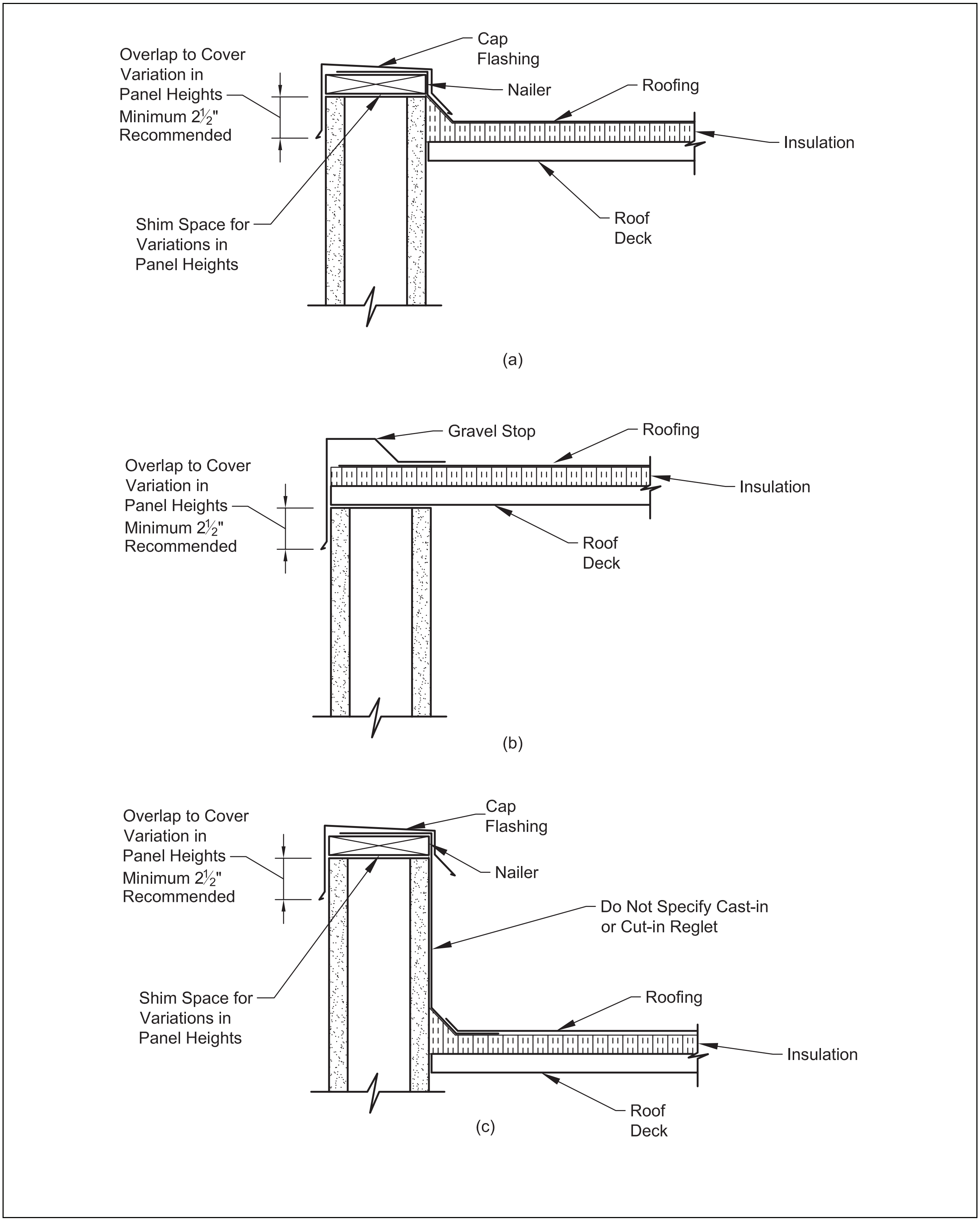


Figure 5.2.8 Roof Details



5.3 Structural Considerations

Hollow core wall panels may be used as the hollow core section only or may be cast as a sandwich panel with insulation and a concrete facing (Fig. 5.2.4).

The facing wythe of hollow core sandwich wall panels must be attached to the hollow core panel to resist both handling and in-place forces. Most hollow core sandwich wall panels are designed as non-composite panels. To ensure non-composite behavior, flexible ties are used to allow the facing to expand and contract independently of the hollow core section.

There is generally adequate adhesion between the facing, insulation, and hollow core slab to prevent movement of the face during the initial handling and erection. However, this adhesion tends to break down through time and seasonal cycles. To keep the face wythe from creeping down, panel support details should provide full support for both the hollow core slab and the facing. If this is not possible and a rigid connection is required between the facing and the hollow core slab to support the face wythe, this connection should be made at only one location to avoid developing thermal restraint forces.

For non-load-bearing panels, there is no minimum prestress requirement beyond that required to limit stresses and provide the ultimate moment capacity. Load-bearing panels are typically stressed to provide a minimum prestress level greater than 225 psi. By doing so, Section 18.11.2 of ACI 318-11[3] allows the minimum wall reinforcement requirements of Section 14.3 to be waived. There is no structural requirement to prestress the face wythe of a hollow core sandwich wall panel. Some hollow core producers choose to do so to minimize or close any incidental cracks that may occur.

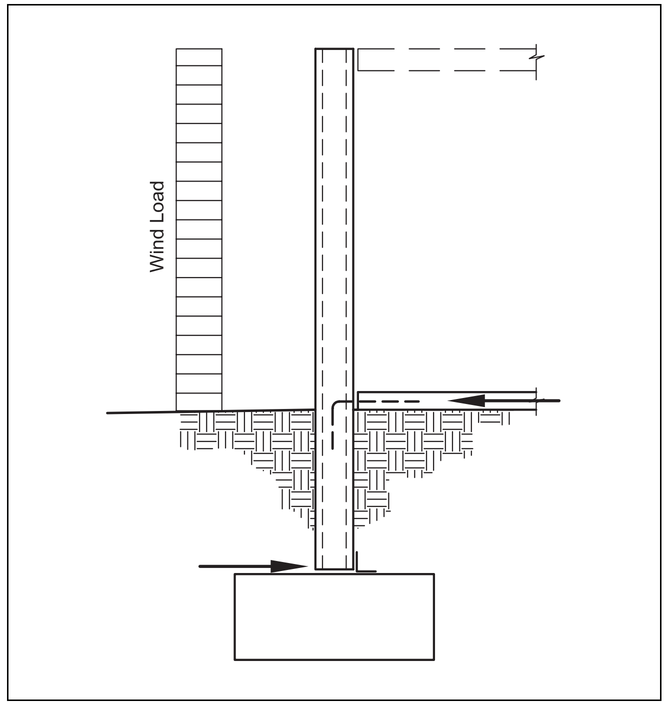
5.3.1 Lateral Bracing

The design concept for a building using hollow core wall panels is no different than that using other precast concrete walls. A load path must be provided to carry all design loads, both gravity and lateral, from their point of application to their final point of resistance.

Vertical hollow core wall panels are typically designed to span from the foundation to the roof. Less commonly, they may be tied into the floor slab and designed as a cantilever (Fig. 5.3.1).

Because it is the more common approach, this manual will focus on the simple-span, hollow core wall panel building system. The concepts presented can be extrapolated to other bracing systems by substituting the appropriate equations for flexure and deflection.

Figure 5.3.1 Cantilever wall system



While a continuous section of hollow core wall panel and reinforcement spanning from the foundation to the roof is required for flexural integrity, it is often interrupted by windows, doors, and mechanical openings. If the opening is small, the remaining wall panel may have adequate reserve capacity to compensate for the panel and strands that have been cut by the opening.

As the opening reaches and exceeds a full panel width, a frame is generally provided. This frame is designed both to support the gravity weight of the panel and to span horizontally to transfer the lateral loads on the cut wall panel to the adjacent wall panels. The lateral loads to be transferred may include the horizontal bracing force caused by eccentric gravity loads as well as wind forces normal to the panel face.

Continuing the load path, the wall panels adjacent to the opening must be designed for loads applied directly to them plus the added lateral loads from the frame. A finite element study indicates that the effective hollow core wall panel width for resisting edge loads can be predicted as:

$$\text{width} = 0.51\left(\frac{b}{\ell}\right)^{0.7} \geq 4\,\text{ft}$$

where b = panel width (ft)
ℓ = span length (ft)

At the top of the hollow core wall panels, lateral resistance must be provided by the roof system. Connections must be made between the walls and the roof to transfer wall panel reactions into the roof structure and, possibly, diaphragm forces out of the roof. If the roof system is not supplied by the wall panel producer, coordination is required to ensure that the roof has adequate integrity to transfer the panel reactions to the vertical bracing elements of the building.

At the hollow core wall panel base, connections are made to the footing, foundation wall, or interior slab on ground. These connections must resist the reactions normal to the face of the wall panel and, possibly, transverse shear and uplift forces.

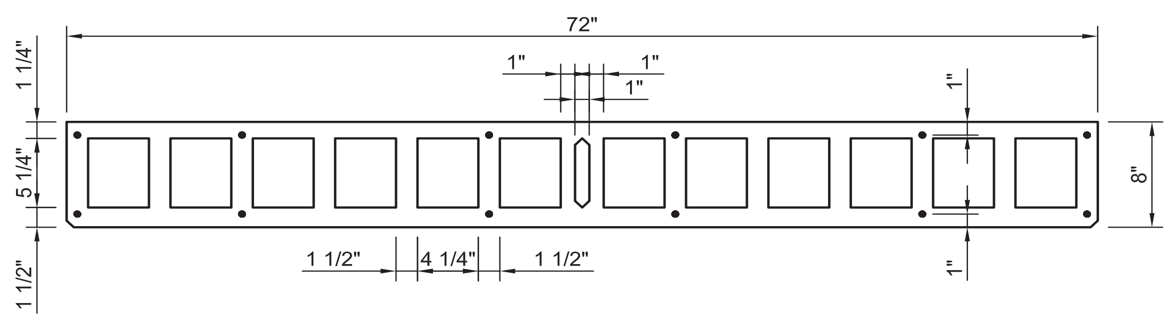
5.3.2 Load-Bearing versus Non-Load-Bearing Hollow Core Wall Panels

Because most hollow core wall panels have substantial axial load capacity, designing the panels to be load-bearing rather than only cladding may be economically beneficial. A load-bearing panel generally requires little or no additional reinforcement than a non-bearing panel. Using the wall panels to support the roof structure eliminates a beam and column line at each load-bearing elevation. These savings should be balanced against the likely need for additional or heavier connections and the possible increased erection costs due to temporary bracing requirements.

5.3.3 Non-Load-Bearing Hollow Core Wall Panel Design

Non-load-bearing hollow core wall panels are designed as flexural members as described in Chapter 2. In addition to checking the bending-moment capacity against the ultimate (factored)

Example 5.3.1 Interaction curve



A = 303.2 in.^{2} d'_p = 1 in. S_b = 652.2 in.^{3}

I = 2537.5 in.^{4} d_p = 7 in. S_t = 617.5 in.^{3}

b_w = 20.0 in. b = 72 in. wt = 52.6 lb/ft^2

y_b = 3.89 in.

Prestressing: Six ⅜ in., ϕ, 270 ksi, low relaxation strands each face

Initial prestress = 70% f_{pu} Prestress loss = 15%

E_{ps} = 28500 ksi

Concrete: f'_c = 5000 psi E_c = 4300 ksi loads, service load stresses are usually checked against the modulus of rupture. To avoid cracking a wall panel during handling, shipping, and erection, stresses should be limited to the modulus of rupture with a factor of safety of 1.5.

$$f_r' \leq \frac{7.5\,\lambda\sqrt{f_c'}}{1.5} = 5\lambda\sqrt{f_c'}$$

For transitory loads such as wind, using a limit of $7.5\lambda\sqrt{f_c'}$ is appropriate. The higher limit recognizes the small likelihood of the full load occurring, the short duration of the load, and the fact that the prestressing will readily close any cracks that may occur.

5.3.4 Load-Bearing Panel Design

The analysis of a load-bearing hollow core wall panel is similar to that of other prestressed compression members such as solid wall panels and columns. An interaction curve plotting ultimate axial load ϕP_n versus ultimate moment ϕM_n is developed using strain compatibility and internal equilibrium. The ultimate loads and moments, including magnification due to slenderness effects, are calculated at critical points in the panel and compared to capacities along the interaction curve.

5.3.4.1 Interaction Curves

An interaction curve showing the relationship of axial load versus moment capacity is developed by calculating points on the curve using strain compatibility and internal equilibrium. Unless more accurate curves are available, the stress-strain diagram shown in Fig. 2.2.5.1 can be used for the prestressing strand. For most hollow core wall panels, the axial load is relatively light and flexure is dominant; therefore, the lower portion of the interaction curve is most important.

The stress and equilibrium equations are as follows:

$$a = \beta_1 c$$
$$\varepsilon'_{ps} = f_{se} / E_{ps} - (0.003/c)(c - d'_p) \leq 0.035$$
$$\varepsilon_{ps} = f_{se} / E_{ps} + (0.003/c)(d_p - c) \leq 0.035$$

$$P_n = (A_{comp} - A'_{ps})0.85f'_c - A'_{ps}f'_{ps} - A_{ps}f_{ps}$$
$$M_n = P_n e$$

$$= (A_{comp} - A'_{ps})(y_t - y')(0.85f'_c) - A'_{ps}f'_{ps}(y_t - d'_p) + A_{ps}f_{ps}(d_p - y_t)$$

$$\varepsilon_t = \varepsilon_{ps} - f_{se}/E_{ps}$$

Using these equations, the interaction curve is developed by assuming a value of a, then calculating the remaining values including P_n and M_n. This is repeated several times to generate points to be plotted. Extra care must be taken with hollow core wall panels. If the compression area is deeper than the flange thickness, compression on the web area must be included.

Section 9.3 of ACI 318-11 prescribes the appropriate ϕ factors to apply to the calculated capacities. When the net tensile strain $\varepsilon_t \leq 0.002$, the section is compression-controlled and $\phi = 0.65$. When $\varepsilon_t \geq 0.005$, the section is tension-controlled and $\phi = 0.90$. For sections in which ε_t is between these limits,
$\phi = 0.48 + 83\varepsilon_t$.

Given the cross section shown, develop the tension portion of the interaction curve.

Solution:

Assume $a = 0.80$ in.

$$\beta_1 = 0.85 - 0.05(f'_c - 4000)/1000 = 0.85 - (1)(0.05) = 0.80$$
$$c = a / \beta_1 = 0.80 / 0.80 = 1.0 \text{ in.}$$
$$f_{se} = (\text{initial P/S})(1 - \text{loss})f_{pu} = (0.7)(0.85)(270) = 160.6 \text{ ksi}$$
$$\varepsilon'_{ps} = f_{se}/E_{ps} - (0.003/c)(c - d_p) = 160.6 / 28800 - (0.003 / 1.0)(1.0 - 1) = 0.00558 \text{ in./in.}$$
$$\varepsilon_{ps} = f_{se}/E_{ps} + (0.003/c)(d_p - c) = 160.6 / 28800 + (0.003 / 1.0)(7 - 1.0) = 0.02358 \text{ in./in.}$$
$$f'_{ps} = \varepsilon'_{ps}E_{ps} = (0.00558)(28800) = 160.6 \text{ ksi}$$
$$f_{ps} = f_{pu} - 0.04 / (\varepsilon_{ps} - 0.007) = 270 - 0.04 / (0.02358 - 0.007) = 267.6 \text{ ksi}$$
$$A_{comp} = (a)(b) = (0.8)(72) = 57.6 \text{ in.}^2$$
$$P_n = (A_{comp} - A'_{ps})(0.85f'_c) - A'_{ps}f'_{ps} - A_{ps}f_{ps}$$
$$= [57.6 - 6(0.085)](0.85)(5) - 6(0.085)(160.6) - 6(0.085)(267.6)$$
$$= 24 \text{ kip}$$

$$M_n = (A_{comp} - A'_{ps})(y_t - y')(0.85f'_c)$$
$$- A'_{ps}f'_{ps}(y_t - d'_p) + A_{ps}f_{ps}(d_p - y_t)$$
$$= [57.6 - 6(0.085)][(0.85)5](4.11 - 0.8/2)$$
$$- 6(0.085)(160.6)(4.11 - 1)$$
$$+ 6(0.085)(267.6)(7 - 4.11)$$
$$= 1039.8 \text{ kip-in.} = 86.6 \text{ kip-ft}$$

Determine ϕ

$$\varepsilon_t = 0.02358 - 160.6 / 28800$$
$$= 0.0180 > 0.005$$

Therefore: $\phi = 0.9$

$$\phi P_n = 0.9(24) = 22 \text{ kip}$$
$$\phi M_n = 0.9(86.6) = 78.3 \text{ kip-ft}$$

Additional points on the interaction curve are calculated by assuming values of a and computing the remaining terms. As the dimension a becomes larger than the flange thickness, the compression area must be adjusted accordingly. Continuing this example results in an interaction curve as shown in Fig. 5.3.2.

This interaction curve was developed assuming the strands are fully developed. This requires that the location at which M_u is being checked be a minimum of a full development length ℓ_d from the hollow core wall panel end. Situations may arise where the critical moment occurs near the end of the wall panel. A load-bearing wall panel with heavy loads or large eccentricities, or a cantilever wall panel as shown in Fig. 5.3.1, may have large bending moments within a distance less than the full development length.

Figure 5.3.2 Typical stress-strain curve, 7-Wire low relaxation prestressing strand

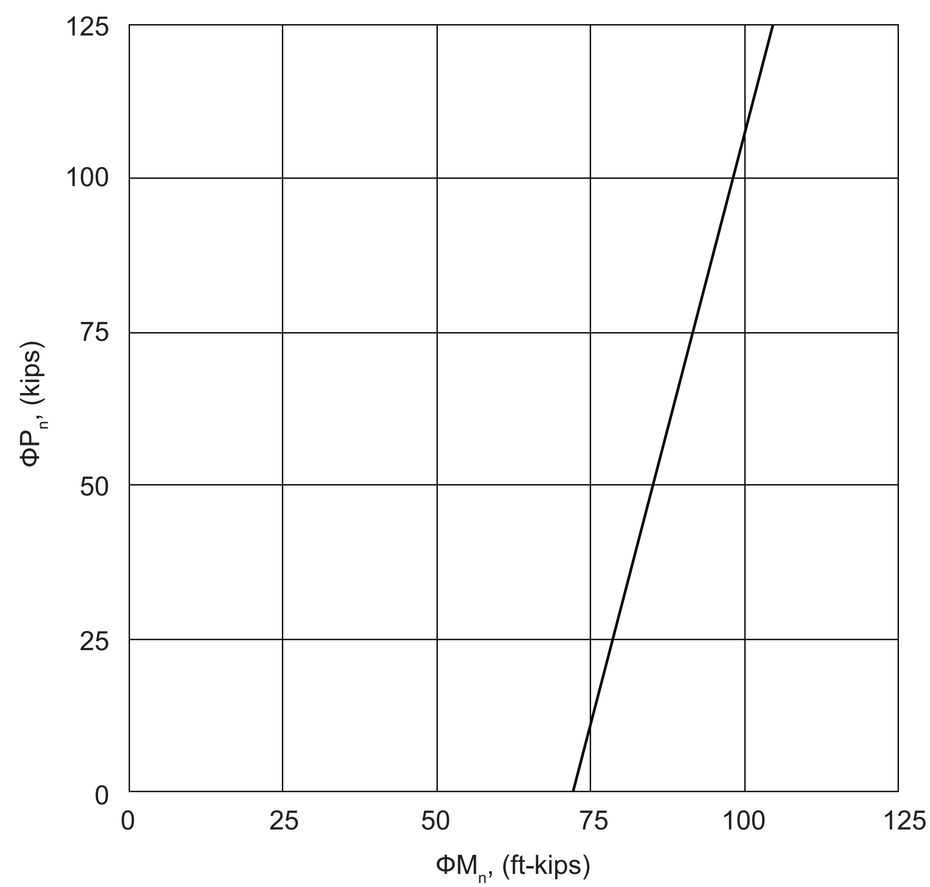
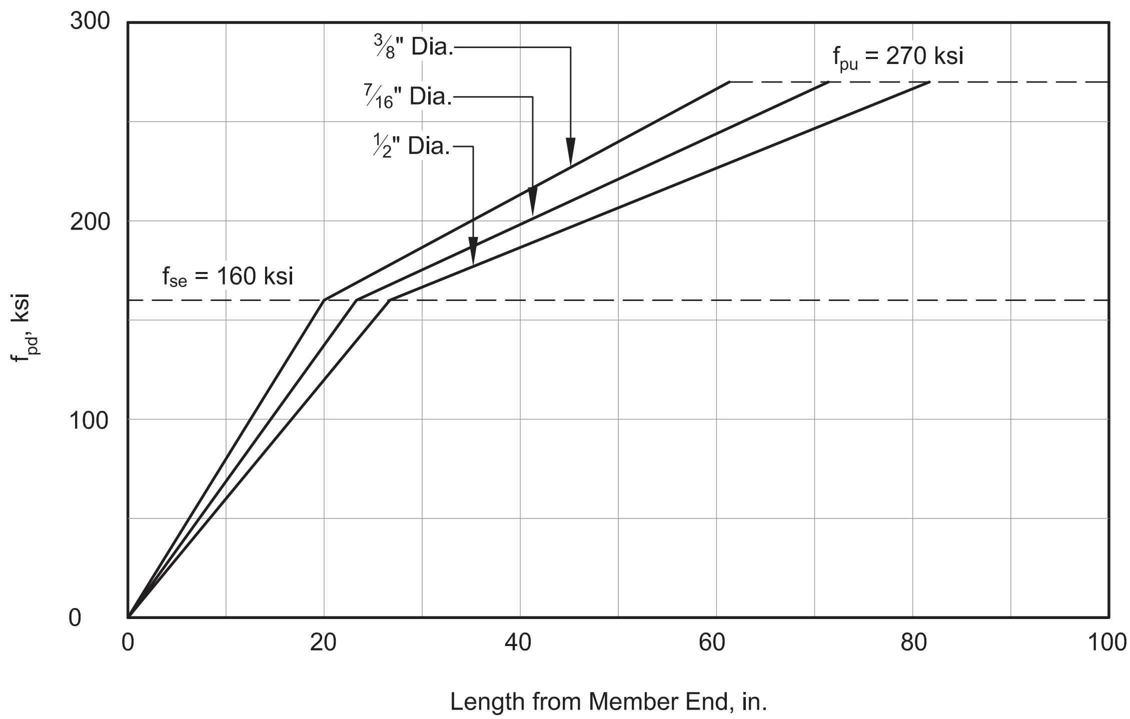


Figure 5.3.3 Design stress for underdeveloped strands



When the point of maximum moment occurs within the development length of the hollow core wall panel, Fig. 5.3.3 can be used to determine a maximum value for f_{ps}. A lower bound capacity can be established using f_{se} as a maximum value for f_{ps}. This requires a minimum bond length equal to the strand transfer length ℓ_t. If the point of maximum moment is less than the transfer length from the wall panel end, the only available moment capacity is the cracking moment. Refer to Section 2.6 for a more complete discussion of this topic. In this instance, reinforcing bars may be required.

If the strands are not fully developed, Section 9.3.2.7 of ACI 318-11 states that ϕ equals 0.75 from the end of the member to the transfer length with a linear increase to 0.9 at the development length. Therefore, when constructing the interaction diagram, this limit must be compared with the ϕ based on the steel strain and the lesser value used.

Example 5.3.2 Underdeveloped Strands

Given the data from Example 5.3.1, develop the interaction curve at 3 ft from the end of the hollow core wall panel.

Solution:

Referring to Fig. 5.3.3, the maximum available stress at 36 in. is 203 ksi. Repeating the strain compatibility analysis from the previous example:

Assume $a = 0.80$ in.

$$\beta_1 = 0.85 - 0.05(f'_c - 4000)/1000$$
$$= 0.85 - (1)(0.05) = 0.80$$
$$c = a / \beta_1 = 0.80 / 0.80 = 1.0 \text{ in.}$$
$$\varepsilon'_{ps} = f_{se}/E_{ps} - (0.003/c)(c - d_p)$$
$$= 160.6 / 28800 - (0.003 / 1.0)(1.0 - 1)$$
$$= 0.00558 \text{ in./in.}$$
$$\varepsilon_{ps} = f_{se}/E_{ps} + (0.003/c)(d_p - c)$$
$$= 160.6 / 28800 + (0.003 / 1.0)(7 - 1.0)$$
$$= 0.02358 \text{ in./in.}$$
$$f'_{ps} = \varepsilon'_{ps}E_{ps}$$
$$= (0.00558)(28800)$$
$$= 160.6 \text{ ksi} < 203 \text{ ksi}$$

$f_{ps} = f_{pu} - 0.04/(\varepsilon_{ps} - 0.007)$
$= 270 - 0.04 / (0.02358 - 0.007)$
$= 267.6 \text{ ksi} > 203 \text{ ksi}$
Use 203 ksi
$A_{comp} = (0.8)(72) = 57.6 \text{ in.}^2$
$P_n = (A_{comp} - A'_{ps})(0.85f'_c) - A'_{ps}f'_{ps} - A_{ps}f_{ps}$
$= [57.6 - 6(0.085)](0.85)(5)$
$- 6(0.085)(160.6) - 6(0.085)(203)$
$= 57 \text{ kip}$
$M_n = (A_{comp} - A'_{ps})(y_t - y')(0.85f'_c)$
$- A'_{ps}f'_{ps}(y_t - d'_p) + A_{ps}f_{ps}(d_p - y_t)$
$= [57.6 - 6(0.085)][(0.85)5](4.11$
$- 0.8/2) - 6(0.085)(160.6)(4.11 - 1)$
$+ 6(0.085)(203)(7 - 4.11)$
$= 944.6 \text{ kip-in.} = 78.7 \text{ kip-ft}$

Determine ϕ

$\varepsilon_t = 0.02358 - 160.6 / 28800$
$= 0.0180 > 0.005$

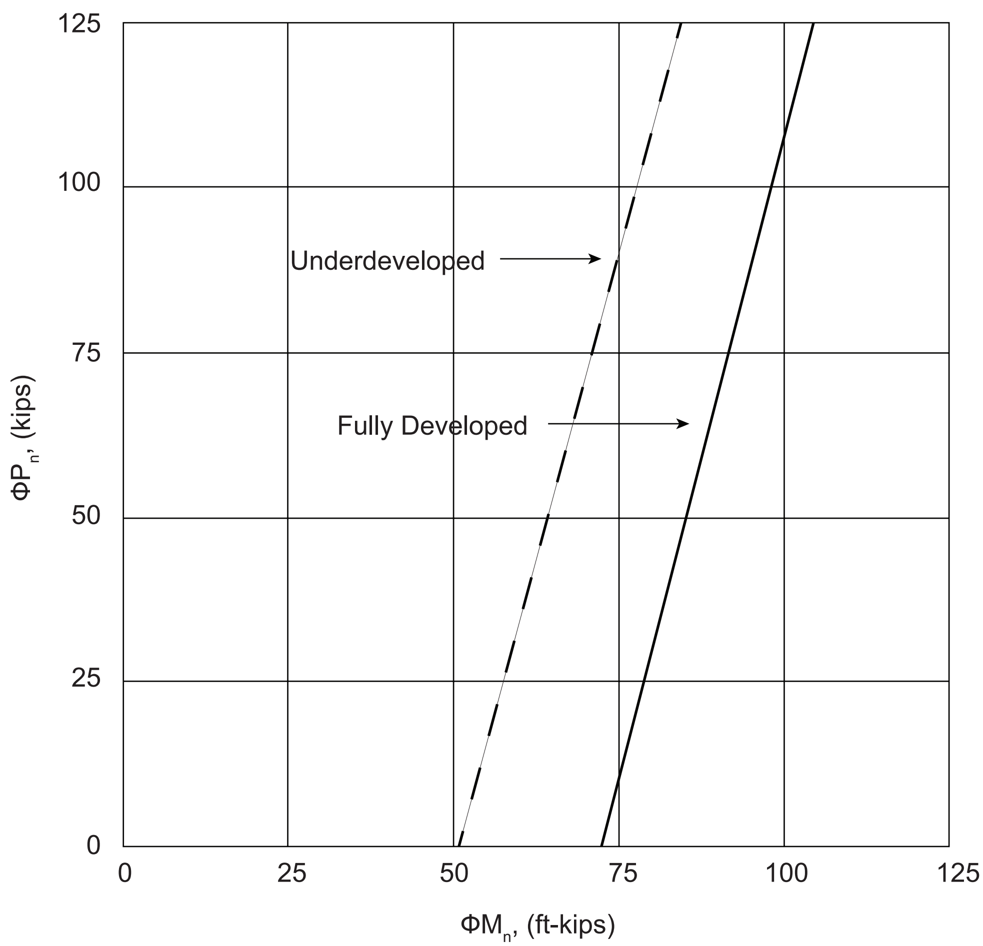
Therefore: $\phi_1 = 0.9$

With the available length falling between the transfer length and the development length, ϕ is limited to:

$$\phi_2 = 0.75 + \left(\frac{36-20}{61.25-20}\right)(0.9-0.75)$$
$$= 0.81$$

$\phi P_n = 0.81(57) = 46.2 \text{ kip}$
$\phi M_n = 0.81(78.7) = 63.7 \text{ kip-ft}$

Figure 5.3.4 Interaction curves with underdeveloped (ℓ available = 36 in.) and fully developed strands



By assuming additional values of a, the interaction curve shown in Fig. 5.3.4 (superimposed on the fully developed strand curve) is generated. It should be noted that this curve is only valid at 36 in. or more from the hollow core wall panel end. If the maximum moment occurs closer to the end of the hollow core wall panel, a new interaction curve must be developed.

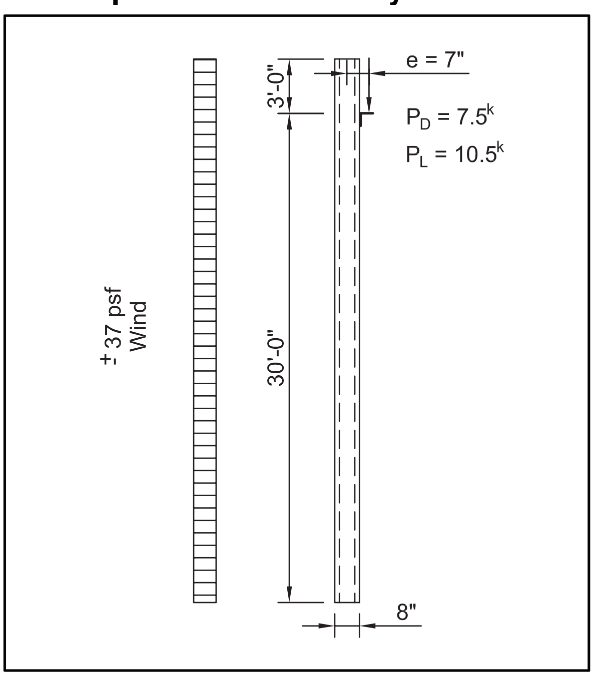
5.3.4.2 Slenderness Effects

The interaction curves developed in Section 5.3.4.1 provide the design strength of the hollow core wall panel section. For short walls, $k\,\ell_u/r \le 25$, the bending moments from a first order analysis using ultimate loads are computed and compared with values within the interaction curves.

As the hollow core wall panels get taller, the first order bending moments are magnified by the effect of the axial load acting through the deflected shape. This deflected shape may be caused by eccentric axial loads, applied lateral loads, thermal bowing, camber due to unsymmetrical prestressing, and manufacturing tolerance.

Section 10.10 of ACI 318-11 outlines a procedure for calculating magnified moments for panels with a slenderness ratio less than or equal to 100. Modifications developed by Nathan and summarized in the PCI manual *Recommended Practice for the Design of Prestressed Walls and Columns*[42] allow this procedure to be used for slenderness ratios up to 150.

Example 5.3.3 *P-Δ* Analysis



The most accurate method to account for slenderness effects in hollow core wall panels is the *P-Δ* analysis. This analysis method is recommended for all slender compression members and is required for all members with a slenderness ratio greater than 150.

The *P-Δ* analysis procedure involves calculating the deflected shape of the hollow core wall panel under factored loads, applying the axial load to this deflected shape to cause increased deflection, and continuing iterations until the panel stabilizes or fails. The final, stabilized wall panel and its moments are the magnified design forces for comparison with the interaction curve.

Example 5.3.3 *P-Δ* Analysis, Iterative Solution

Given the cross section and prestressing from Example 5.3.1, check the adequacy of the hollow core wall panel for the loading shown above.

Solution:

Although all relevant load cases must be checked, assume that ACI 318-11 (Eq 9-4) will control this example.

$$\begin{aligned} U &= 1.2D + 1.0W + 0.5S \\ P_u &= 1.2\,(7.5) + 0.5\,(10.5) \\ &= 14.25 \text{ kip} \\ w_u &= 1.0\,(0.037 \text{ kip/ft}^2)(6 \text{ ft}) \\ &= 0.221 \text{ kip/ft} \end{aligned}$$

Thermal bow due to 40 ˚F temperature differential

$$\Delta_T = \frac{C(T_1 - T_2)\ell^2}{8h} = \frac{6\times10^{-6}(40)(30\times12)^2}{8(8)}$$

$$= 0.49 \text{ in.}$$

Based on the hollow core producer's experience, assume initial out-of-straightness plus shrinkage bowing equals 0.75 in.

Per Section R10.10.3 in ACI 318-11, use a stiffness reduction factor ϕ_K of 0.80 to account for variability in properties and in the analysis. Dividing I by $(1 + \beta_d)$ accounts for the stiffness reduction caused by sustained loads.

$$EI_{eff} = \frac{0.80 E_c I_g}{1+\beta_d}$$
$$= \frac{0.80\,(4300)\,(2537.5)}{1+\beta_d}$$
$$= \frac{8.729\times10^6}{1+\beta_d}$$

For dead load plus live load:

β_d = 1.2(7.5) / 14.25
= 0.63

EI_{eff} = 8.729 × 10^6 / (1 + 0.63)
= 5.350 × 10^6

For wind load:

β_d = 0

EI_{eff} = 8.729 × 10^6

Deflection due to eccentric axial load:

$$\Delta_1 = \frac{P_u e \ell^2}{16EI}$$
$$= \frac{14.25(7)(30\times12)^2}{16(5.35\times10^6)}$$
$$= 0.15 \text{ in.}$$

Deflection due to wind suction:

$$\Delta_1 = \frac{5 w_u \ell^4}{384EI}$$
$$= \frac{5(0.221)(30)^4(1728)}{384(8.729\times10^6)}$$
$$= 0.46 \text{ in.}$$

Total first order deflection:

$\Delta_{Total\ 1}$ = 0.49 + 0.75 + 0.15 + 0.46
= 1.85 in.

Calculate the second order deflections.

$$\Delta_2 = \frac{P_u \Delta \ell^2}{16EI}$$
$$= \frac{14.25(30\times12)^2}{16(5.35\times10^6)}\Delta$$
$$= 0.0216\Delta$$

Inserting the first order deflection from above:

Δ_2 = 0.0216 (1.85)
= 0.040 in.

$\Delta_{Total\ 2}$ = 1.85 + 0.040
= 1.89 in.

With a second iteration:

Δ_2 = 0.0216 (1.89)
= 0.041 in.

$\Delta_{Total\ 3}$ = 1.85 + 0.041
= 1.891 in.

Because the final deflection is converging, the panel is stable. Use a final deflection of 1.891 in. and calculate the ultimate moment.

$$M_{D+L} = \frac{14.25(7)}{2(12)}$$
= 4.16 kip-ft at midheight

$$M_W = \frac{0.221(30)^2}{8}$$
= 24.86 kip-ft

$$M_{P-\Delta} = \frac{14.25(1.891)}{12}$$
= 2.25 kip-ft

The ultimate design moment equals

M_u = 4.16 + 24.86 + 2.25
= 31.27 kip-ft

Because all of the preceding deflection calculations were made based on the assumption of an uncracked section, this assumption must be checked. Unlike most stress checks, this calculation must be done using factored loads and moments to verify that the hollow core wall panel remains uncracked at the design level used in the analysis.

$$f_t = \frac{P}{A} - \frac{M_u}{S}$$
$$= \frac{12\,(0.085)\,(270)\,(0.7)\,(0.85)}{303.2} + \frac{14.25}{303.2} - \frac{31.27\,(12)}{617.5}$$
$$= 0.540 + 0.047 - 0.608$$
$$= -0.021 \text{ ksi} \qquad \text{Net tension}$$

$$f_r = \frac{7.5\sqrt{5000}}{1000} = 0.530 \text{ ksi} > 0.021 \text{ ok}$$

The section is not cracked; therefore, the analysis is valid.

Using the interaction curve of Fig. 5.3.2, it can be seen that the panel is adequate for P_u = 14.25 kip and M_u = 31.27 kip-ft.

The section at the top of the hollow core wall panel should also be checked due to the underdeveloped strands. The full dead load plus full live load combination will be critical at this location. This moment is not required to be magnified.

$$M_u = \frac{[1.2(7.5) + 1.6(10.5)](7)}{12}$$

$$= 15.1 \text{ kip-ft}$$

Comparison with the interaction curve of Fig. 5.3.4 verifies that the hollow core wall panel is adequate at this location also.

For the particular case of a hollow core wall panel braced at the top and bottom with pinned connections, there is a closed form solution to simplify the P-Δ analysis.

The deflected shape of the hollow core wall panel is a sine curve. Therefore, the total deflection can be expressed as

$$\Delta = \frac{P\Delta\ell^2}{\pi^2 EI} + \Delta_1$$

where Δ_1 = first order deflection

Solving for Δ yields:

$$\Delta = \frac{\Delta_1}{1 - \dfrac{P\ell^2}{\pi^2 EI}}$$

And the final moment equals:

$$M_u = M_1 + P\Delta$$

$$= M_1 + \frac{P\Delta_1}{1 - \dfrac{P\ell^2}{\pi^2 EI}}$$

where M_1 = first order moment

Example 5.3.4 P-Δ Analysis, Direct Solution

Solve Example 5.3.3 using the direct solution equation.

Solution:

From Example 5.3.3

Δ_1 = 1.85 in.

$$\Delta = \frac{1.85}{1 - \dfrac{14.25(30\times12)^2}{\pi^2(5.35\times10^6)}}$$

$$= 1.917 \text{ in.}$$

M_1 = 4.16 + 24.86 = 29.02 kip-ft

$$M_u = 29.02 + \frac{14.25[1.917]}{12}$$

$$= 31.30 \text{ kip-ft}$$

These values are approximately equal to the values computed by the iterative process.

Although both ACI and PCI recommend the P-Δ analysis method, moment magnification is an acceptable design procedure. Using this approach for hollow core wall panels will generally result in overly conservative designs. For comparison's sake, the Nathan moment magnification design procedure, as modified by the PCI *Recommended Practice*, will be presented.

The basic equations are as follows:

$$M_c = \delta_b M_{2b} + \delta_s M_{2s}$$

$$\delta_b = \frac{C_m}{1 - \dfrac{P_u}{\phi P_c}} \ge 1.0$$

$$\delta_s = \frac{1}{1 - \dfrac{\sum P_u}{\phi \sum P_c}} \ge 1.0$$

$$P_c = \frac{\pi^2 EI}{(k\ell_u)^2}$$

where:

$C_m = 0.7 + 0.3(M_{1b} / M_{2b})$

if braced against sidesway and without transverse loads

C_m = 1.0 for all other cases

$$EI = \frac{E_c I_g / \lambda}{(1+\beta_d)}$$

$$\lambda = \eta\theta \geq 3.0$$

$$\eta = 2.5 + \frac{1.6}{P_u / P_o}$$

$$6 \leq \eta \leq 70$$

For cross sections without a compression flange:

$$\theta = \frac{27}{(k\ell_u / r)} - 0.05$$

For cross sections with a compression flange:

$$\theta = \frac{35}{(k\ell_u / r)} - 0.09$$

For the typical hollow core wall panel, spanning vertically, braced at the top and bottom, and resisting wind loads, δ_s is not used and C_m is equal to 1.0.

Example 5.3.5 Moment Magnifier Analysis

Given the information from Example 5.3.3, solve for the magnified design moment.

Solution:

Calculate *EI*

$$P_o = 0.85 f_c' \left(A - A_{ps}' - A_{ps}\right) - \left(A_{ps}' + A_{ps}\right)\left(f_{se} - 0.003E_{ps}\right)$$
$$= 0.85(5)[303.2 - 6(0.085) - 6(0.085)]$$
$$- [6(0.085) + 6(0.085)][160.6 - 0.003(28800)]$$
$$= 1208 \text{ kip}$$

$$\eta = 2.5 + \frac{1.6}{14.25/1208}$$
$$= 138.1 \qquad \text{Use } 70$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{2537.5}{303.2}}$$
$$= 2.893 \text{ in.}$$

$$\theta = \frac{35}{(30)(12)/2.893} - 0.09$$
$$= 0.191$$

$$\lambda = \eta\theta = 70(0.191)$$
$$= 13.39$$

$$M_u = 29.02 \text{ kip-ft}$$

$$M_{uD} = \frac{1.2(7.5)(7)}{2(12)}$$
$$= 2.62 \text{ kip-ft}$$

$$\beta_d = \frac{M_{uD}}{M_u} = \frac{2.62}{29.02} = 0.090$$

$$EI = \frac{E_c I_g / \lambda}{1+\beta_d}$$

$$EI = \frac{4300(2537.5)/13.39}{1+0.090}$$
$$= 7.48 \times 10^5$$

$$P_c = \frac{\pi^2 (7.48\times 10^5)}{(30\times 12)^2}$$
$$= 56.9 \text{ kip}$$

Using ϕ_K = 0.75 as recommended in Section R10.10.6 of ACI 318-11 results in:

$$\delta_b = \frac{1.0}{1 - \dfrac{14.25}{0.75(56.9)}}$$
$$= 1.50$$

This is greater than the 1.4 limit in Section 10.10.2.1 of ACI 318-11 so the panel would be judged too slender to be used.

The primary reason for this is that the moment magnifier method assumes the panel will crack, resulting in a drastically reduced moment of inertia and increased deflections and moments. The *P-Δ* method requires a check for flexural cracking. Because most wall panels are prestressed sufficiently to prevent cracking, the resulting deflections and moments are not arbitrarily and unnecessarily increased.

The *P-Δ* analysis may show extremely slender hollow core wall panels to be structurally adequate even while indicating very large and unreasonable deflections. Therefore, Reference 42 limits the lateral deflection under factored loads to ℓ_u / 100.

5.4 Hollow core Wall Panels as Shear Walls

For the most common hollow core wall panel-clad building–one- and two-story structures with vertically spanning wall panels–the panels are often designed to act as shear walls. This approach requires that the roof be designed to brace the top of the wall panels and then act as a diaphragm to deliver the lateral forces to perpendicular shear walls. If the wall panel supplier does not provide the roof, there must be coordination to ensure that there is a mechanism and adequate connection capacity to provide this force transfer.

Ideally, the resisting elevation is designed using individual shear walls, with the weight of each panel serving to resist the overturning moment. This procedure minimizes the connection requirements. However, depending on the lateral forces, building dimensions, and available walls, this approach may not provide adequate resistance against overturning. If this is the case, additional resistance can be gained by providing tie-down connections near the wall panel edges or by connecting panels along the vertical joint to create multi-panel units.

Section 16.5.1.3(b) of ACI 318-11 requires all vertical, structural wall panels, except cladding, to be connected to the foundation with a minimum of two connections, each having a minimum nominal strength of 10 kip. Because the structural integrity requirement is not additive to the actual design forces, this connection capacity is available to provide additional overturning resistance. Because some casting systems allow only shallow anchors with limited capacities, the local hollow core producer should be consulted before designing for larger tie-down forces.

Alternatively, shear connections can be used along the vertical joint between hollow core wall panels to create a wider, stiffer shear wall element. If this technique is used, the minimum number of hollow core wall panels necessary to resist the design loads should be connected. Concrete shrinkage and temperature change will cause transverse movement of the hollow core wall panels. As more hollow core wall panels are connected, restraint forces will develop in the panel-to-panel connections. Minimizing the number of hollow core wall panels connected will minimize these restraint forces.

Example 5.4.1 Shear Walls

Given the building shown in Figure 5.4.1, design the hollow core wall panels to act as shear walls. The panels are 6-ft-wide panels, as shown in Example 5.3.1.

Solution:

Determine the forces applied to the shear walls.

Force applied to roof diaphragm

$$w_u = \frac{0.029(33)^2}{2(30)}$$
$$= 0.526 \text{ kip/ft}$$

At north and south elevations

$$V_u = \frac{0.526(150)}{2} = 39.4 \text{ kip}$$

Resist this shear with forty-six 6-ft-wide hollow core wall panels.

The critical load case is $U = 0.9D + 1.0W$

$$M_u = \frac{39.4(30)}{46 \text{ panels}} = 25.6 \text{ kip-ft / panel}$$

Considering self-weight as tiedown:

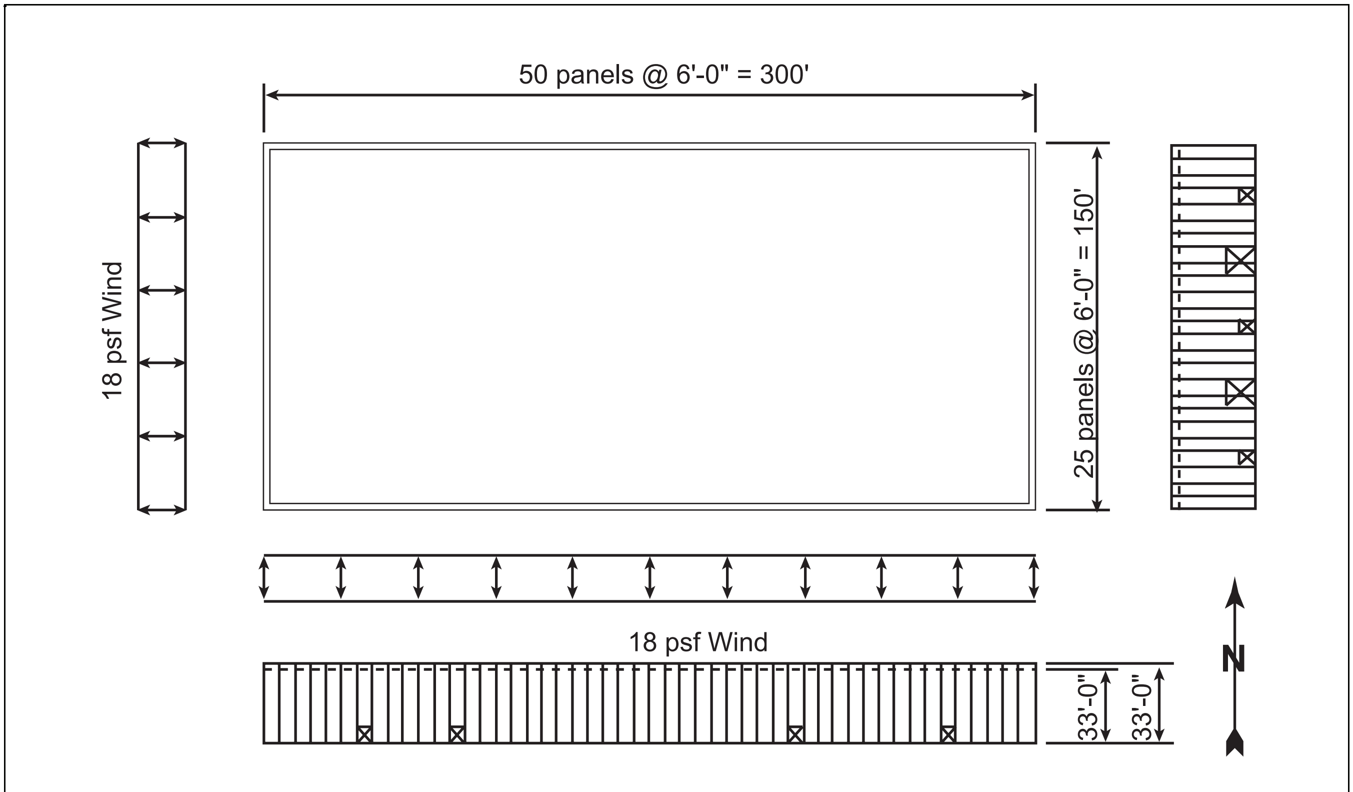
$$T_u = 0.9\,(0.0526)(6)(33)$$
$$= 9.37 \text{ kip per panel}$$

$$a = \frac{9.37}{0.85(5)(8)}$$
$$= 0.28 \text{ in.} = 0.023 \text{ ft}$$

$$M_n = 0.9(9.37)\left(\frac{6}{2} - \frac{0.023}{2}\right)$$
$$= 25.2 \text{ kip-ft} \approx M_u$$

Therefore, these elevations are adequate with minimal tie-down connections.

Figure 5.4.1 Hollow Core Panel Shear Walls



At east and west elevations

$$V_u = \frac{0.526(300)}{2} = 78.9 \text{ kip}$$

Resist this shear with eighteen 6-ft-wide hollow core wall panels.

$$M_u = \frac{78.9(30)}{18 \text{ panels}}$$
$$= 131.5 \text{ kip-ft / panel}$$

Because $M_u > \phi M_n$ with self-weight alone, a tie-down connection is required.

Estimate the tie-down force as 24 kip.

$$a = \frac{9.37 + 24}{0.85(5)(8)}$$
$$= 0.98 \text{ in.} = 0.082 \text{ ft}$$

$$\phi M_n = 0.9\left[9.37\left(\frac{6}{2} - \frac{0.082}{2}\right) + 24\left(5 - \frac{0.082}{2}\right)\right]$$
$$= 132.1 \text{ kip-ft} > M_u$$

Therefore, one solution is to provide a tie-down connection designed for 24 kip. Because this is a fairly large force, try connecting the hollow core wall panels along the vertical joint to form six 18-ft-wide shear wall units. These two options are illustrated in Fig. 5.4.2.

$$M_u = \frac{78.9(30)}{6 \text{ units}} = 395 \text{ kip-ft / unit}$$

Force due to self-weight:

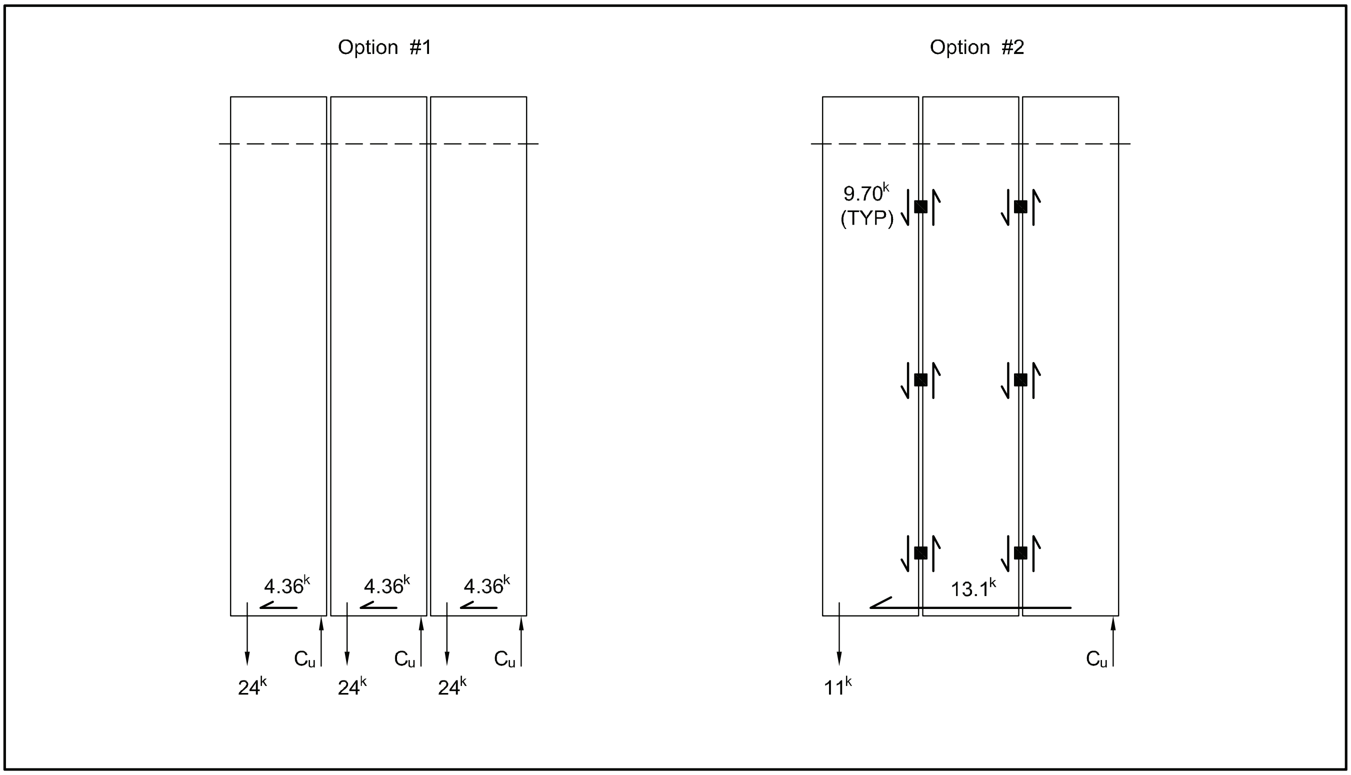
$$T_u = 0.9(0.0526)(18)(33)$$
$$= 28.12 \text{ kip per unit}$$

Estimate the tie-down force as 12 kip.

$$a = \frac{28.12 + 12}{0.85(5)(8)}$$
$$= 1.18 \text{ in.} = 0.098 \text{ ft}$$

$$\phi M_n = 0.9\left[28.12\left(\frac{18}{2} - \frac{0.098}{2}\right) + 12\left(17 - \frac{0.098}{2}\right)\right]$$
$$= 409.62 \text{ kip-ft} > M_u$$

Figure 5.4.2 Shear wall connection forces at east and west elevations



Further iterations calculate the required tie-down force to be approximately 11 kip.

To achieve this resistance, the panels must be adequately connected along the vertical joints to create a composite shear wall unit.

$$V_{uh} = \frac{V_u Q}{I}$$

$$V_u = \frac{78.9}{6} = 13.2 \text{ kip / unit}$$

$$Q = 6\left(\frac{18}{2} - \frac{6}{2}\right) = 36 \text{ ft}^2$$

$$I = \frac{18^3}{12} = 486 \text{ ft}^3$$

$$V_{uh} = \frac{13.2(36)}{486}$$

= 0.98 kip/ft along vertical joint

Using three connections along the panel height:

$$V_u = \frac{0.98(30)}{3} = 9.8 \text{ kip / connection}$$

The panel-to-panel connections along the vertical joints must be designed for this shear force.

5.5 Seismic Design of Hollow core Wall Panels

Table 12.6-1 of ASCE 7-10[36] summarizes the permissible seismic analysis procedures and the criteria for their use. Nearly all buildings that use hollow core wall panels will meet the criteria for analysis by the equivalent lateral force procedure as outlined in Chapters 12 and 13 of ASCE 7-10. This procedure requires that the building and its components resist a percentage of their weight acting horizontally. The percentage varies based on factors such as the seismic design category (SDC), the importance factor, and the response design coefficients, which reflect strength, toughness, ductility, and redundancy.

Once the lateral forces are determined, hollow core wall panels and their connections are designed to resist seismic forces in a manner similar to the design for wind loads. It is important to note that seismic forces calculated in this manner are considered factored design loads.

Because it is nearly impossible for hollow core wall panels to satisfy the requirements of a special reinforced concrete shear wall, the use of hollow core wall panels was effectively limited to buildings in SDC A and B (plus SDC C if transverse steel was provided) prior to the adoption of ASCE 7-05 and ACI 318-05. With the addition of the intermediate precast concrete shear wall as a system category, ASCE 7-05 and ACI 318-05 did not automatically preclude the use of hollow core wall panels in any SDC. However, the design loads increase and the detailing requirements become more stringent in regions of high seismic risk. Combined with the requirements for transverse reinforcement and the shallow embedment depths of most embedments in hollow core slabs, there may be practical or economic limitations to the use of hollow core wall panels in moderate or high seismic areas.

Because the intermediate precast concrete shear wall is a new system category, many of its design and detailing requirements are only now being interpreted and clarified by designers and building officials. Therefore, it is essential that the building designer, the precast concrete producer, and the local building official work together in establishing whether hollow core wall panels are an efficient selection as the lateral force resisting system in SDC C through F.

The fundamentals of seismic design and the determination of the SDC have been covered in Chapter 4 of this manual and will not be reviewed here. Having determined the SDC, the design forces to be applied to the panel and its connections can be calculated.

5.5.1 Designing for Out-of-Plane Forces

Designing a hollow core wall panel for out-of-plane seismic loads is seldom a problem. The lateral seismic forces are generally of a smaller magnitude than the design wind loads. However, because the cyclic, reversible nature of a seismic event requires increased ductility and toughness, the seismic connection forces are often substantially greater than the wind loads.

5.5.1.1 Design Forces

Table 5.5.1 summarizes the out-of-plane seismic design forces. If more than one force is listed for an element, the largest of the forces must be used. The referenced sections should always be reviewed for a more complete explanation of terms and possible footnotes or exceptions.

One notable exemption from these requirements is Section 13.1.4(3) of ACSE 7-10, which allows architectural wall panels (cladding panels) in SDC B to be exempt from seismic design requirements, provided the component importance factor I_p is equal to 1.0. The component importance factor is equal to 1.0 unless any of the following conditions apply.

- The component is required for life-safety purposes after an earthquake.
- The component supports or contains toxic, explosive, or hazardous material.
- The component is in or attached to an occupancy category IV structure and is needed for continued operation of the facility.

It should be noted that seismic design criteria are cumulative. That is, an element in SDC D must satisfy all of the requirements of SDC A through C in addition to the requirements for SDC D.

5.5.2 Types of Shear Walls

ACI 318-11 and ASCE 7-10 recognize three types of precast concrete shear walls. Note that ACI uses the term *structural wall*. For the purposes of lateral load resistance, this is the same as a shear wall. Note also that there is some discrepancy in the types of walls that are given formal definitions.

The intent of the analysis and detailing requirements is the same in both documents and the terminology will likely be coordinated in future editions. There are three types of precast concrete shear walls.

- **Ordinary precast concrete shear walls** are walls that comply with the requirements of ACI 318-11, Chapters 1 through 18, and Chapter 22. This wall type requires no special seismic detailing and is used in SDC A and B.

Table 5.5.1 Out-of-plane seismic design forces

Seismic Design Category	Element	ASCE 7-10 reference section
A	Structural Walls	
	0.01 W_p	1.4.3
	Connections	
	0.2 W_p 5 lb/ft^2	1.4.5 1.4.5
	Non-structural walls	
	Exempt from seismic design requirements	11.7
B Through F	Structural walls – Hollow core panels	
	$0.10\ W_p \le 0.4\ S_{DS}\ I_e\ W_p$	12.11.1
	Connections	
	$0.2\ k_a\ I_e\ W_p \le 0.4\ S_{DS}\ k_a\ I_e\ W_p$ where $k_a = 1.0$ for a rigid diaphragm or $k_a = 1.0 + \frac{L_f}{100} \le 2.0$ for a flexible diaphragm These forces are multiplied by 1.4 for steel elements of the connections for SDC C through F	12.11.2.1 12.11.2.2.2
	Non-structural walls – Hollow core panels*	
	$0.3\ S_{DS}\ I_p\ W_p \le \frac{0.4 S_{DS} I_p W_p}{2.5}\left(1+2\frac{z}{h}\right) \le 1.6\ S_{DS}\ I_p\ W_p$	13.3.1**
	Connections (rigid diaphragm)	
	$\frac{0.4 S_{DS} I_p W_p}{2.5}\left(1+2\frac{z}{h}\right)$ Body of connection $0.5 S_{DS} I_p W_p\left(1+2\frac{z}{h}\right)$ Fasteners of connection	13.4.1 13.3.1
	Connections (flexible diaphragm)	
	$0.2\ k_a\ I_e\ W_p \le 0.4\ S_{DS}\ k_a\ I_e\ W_p$	12.11.2.1

*Per Section 13.1.4(3) of ASCE 7-10, non-structural panels in SDC B are exempt from seismic design requirements if I_p is 1.0
**R_p = 2.5 and a_p =1.0 from Table 13.5-1

- **Intermediate precast concrete shear walls** satisfy the requirements of an ordinary precast concrete shear wall, but must also meet the criteria of Section 21.4 of ACI 318-11. This section requires that connections fail by yielding of steel elements or reinforcement and those portions of the connection that are not designed to yield must develop 150% of the yield strength of the connection. These walls may be used for buildings in SDC C through F (with height restrictions in categories D through F).
- **Special precast concrete shear walls** must comply with Sections 21.1.3 through 21.1.7, 21.9, and 21.10 of ACI 318-11. Commonly referred to as emulative design, this approach requires that the precast concrete wall needs to be designed and perform the same as a cast-in-place concrete wall. With its requirements for full strength reinforcement splices and joints that are (or behave as if) cast monolithically, emulation design is not well suited for use with hollow core wall panels.

Section 21.1.1.8 of ACI 318-11 does allow an alternative to emulation if "it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying this chapter." Hollow core wall panel systems have been or are being developed that use post-tensioning strand or self-limiting ductile base connections to satisfy this requirement. However, these systems are experimental or proprietary and, as such, are outside the scope of this manual.

5.5.2.1 Designing for Shear Wall Forces

The analysis of hollow core wall panels to act as shear walls for resisting seismic forces is similar to the analysis for wind loads. The panels are typically braced at the foundation and the roof. The roof diaphragm braces the top of the hollow core wall panels and delivers the lateral forces to the available shear walls.

One significant difference is that the lateral force acting on the shear walls is not only a portion of the weight of the braced hollow core wall panels but also a portion of the roof weight and the weight of the shear wall itself. Consequently, seismic forces will often control the shear wall analysis, particularly if there is a precast concrete roof.

In addition to the increased forces required in SDC C through F, the detailing requirements for anything other than an ordinary precast concrete shear wall may prevent, or make cost-prohibitive, the use of hollow core wall panels as shear walls.

5.5.2.2 Design Forces

Having determined the spectral response accelerations S_{DS} and S_{D1}, the importance factor I_e, and the elastic fundamental period T as outlined in chapter 4, the seismic response coefficient C_S is calculated as the lesser of:

$$C_s = \frac{S_{DS}}{\frac{R}{I_e}}$$

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \text{ for } T \le T_L$$

But not less than:

$$C_S = 0.044 S_{DS} I_e \ge 0.01$$

In addition, if S_1 is equal to or greater than 0.6g, where g is the acceleration of gravity:

$$C_s \ge \frac{0.5 S_1}{\left(\frac{R}{I_e}\right)}$$

The response modification factor R is based on the seismic force-resisting system. The values of R are listed in Table 12.2-1 of ASCE 7-10 and partially summarized in Table 5.5.2.

Using the equivalent lateral force procedure, the seismic base shear is calculated as:

$$V = C_s W$$

If the structure is a single story, as are the majority of hollow core wall panel buildings, this base shear is applied at the roof and distributed to the available shear walls using the guidelines of Section 4.3 and Example 4.9.1 of this manual. Factors to consider in the distribution include flexible versus rigid diaphragm, configuration and stiffness of the shear walls, and inherent and accidental eccentricity of the lateral force.

Although the design of the diaphragm and the shear walls are inherently related, in a multistory building they may not be designed for the same lateral force. Section 12.8.3 of ASCE 7-10 specifies the vertical distribution requirements for the shear wall analysis, while Section 12.10 lists special requirements for the design of the diaphragm. In this case, the diaphragm and its connections to the shear walls should be designed for the greater of the calculated forces, but the design of the shear walls is based on the loads and vertical distribution of Section 12.8.3.

5.5.2.3 Load Combinations

For buildings in SDC D through F, Section 12.3.4 of ASCE 7-10 requires that a redundancy load factor ρ equal to 1.3 be applied to the seismic force-resisting system unless it can be demonstrated by the criteria of Section 12.3.4.2 that the existing shear wall layout provides redundant load paths. If the redundancy factor is required, it does not need to be used for design of the diaphragm or for any elements to which the overstrength factor Ω_0 has been applied.

Because a seismic event may include both vertical and horizontal ground motion, the ACI load combinations that include seismic forces must be modified to include the potentially damaging effects of vertical movements. The vertical seismic force is defined as

$$E_v = 0.2\ S_{DS}D$$

and is assumed to act in the direction that creates the most critical load combination. Therefore, when designing for maximum axial effects, E_v is added to the factored gravity loads. However, when checking overturning, E_v is subtracted from the resisting loads. With Q_E defined as the effects of horizontal seismic forces, the seismic load combinations become:

$$U = (1.2 + 0.2S_{DS})D + \rho Q_E + L + 0.2S$$
$$U = (0.9 - 0.2S_{DS})D + \rho Q_E + 1.6H$$

For buildings in SDC C through F, some elements of the seismic force-resisting system must be designed for an overstrength factor Ω_0 (Table 5.5.2). Chapter 4 of this manual discusses the use of Ω_0 when designing diaphragms and collectors. Section 12.10.2.1 of ASCE 7-10 also requires that collectors and their connections to the shear walls include the overstrength factor. If the connection supports gravity loads in addition to unloading the diaphragm, the overstrength factor is applied only to the horizontal seismic forces and is not required for the vertical component E_v.

The following examples demonstrate the calculation of seismic forces required for the design of a warehouse with hollow core wall panels. These forces must be compared with those generated by the wind load analysis, with the greater value used in the final design. Because embedments and connection types vary widely among hollow core producers due to the various production methods, the design of the connection itself is not covered. Chapter 6 of this manual illustrates some connection types and their usage.

Table 5.5.2 Design coefficients and factors

Seismic-force-resisting system	R	Ω_0	C_d
Bearing wall systems			
Special reinforced concrete shear walls	5	2½	5
Intermediate precast concrete shear walls	4	2½	4
Ordinary precast concrete shear walls	3	2½	3
Building frame systems			
Special reinforced concrete shear walls	6	2½	5
Intermediate precast concrete shear walls	5	2½	4½
Ordinary precast concrete shear walls	4	2½	4

Example 5.5.1(a)

Given the building shown and the information below, determine the seismic design forces for the wall panels. Note that wind would also have to be checked for a complete design.

Cleveland, Ohio	Zip code 44110
Occupancy category:	II
Importance factor	$I_e = 1.0$
Site class	D

Steel joist / Metal deck roof 20 lb/ft^2
8 in. thick, 6-ft-wide hollow core wall panels with a weight of 52.6 lb/ft^2

Solution:

From USGS $S_S = 0.173g$
$S_1 = 0.058g$
$T_L = 12$ sec

ASCE 7-10, Tables 11.4-1 and 11.4-2
$F_a = 1.6$
$F_v = 2.4$

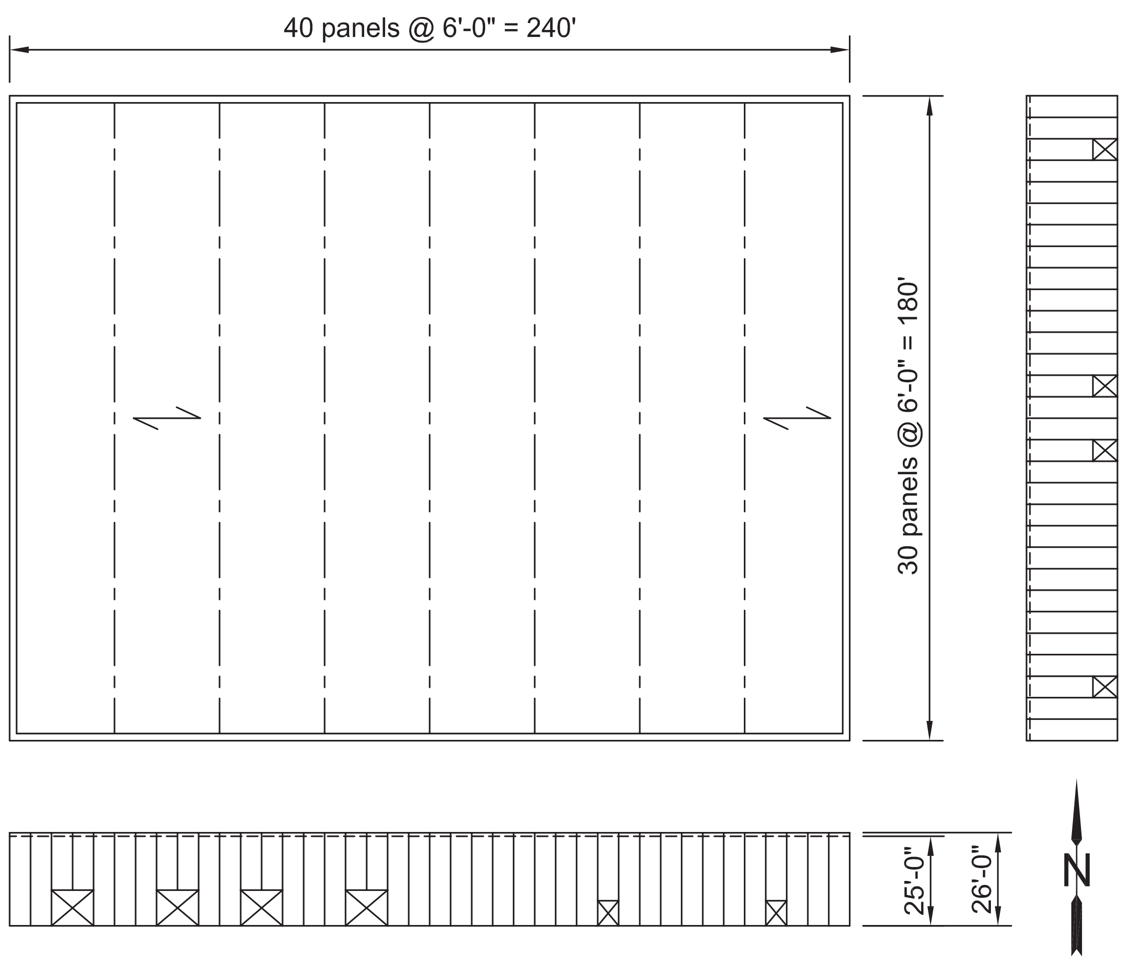
$S_{MS} = 1.6 \times 0.173 = 0.277g$
$S_{M1} = 2.4 \times 0.058 = 0.138g$

$$S_{DS} = \frac{2}{3}(0.277) = 0.185g$$

$$S_{D1} = \frac{2}{3}(0.138) = 0.092g$$

Per Tables 11.6-1 and 11.6-2 of ASCE 7-10, the building is in SDC B.

Example 5.5.1 Seismic forces on hollow core wall panels



Building period:

$$T = C_t h_n^x$$
$$= 0.020(26)^{0.75}$$
$$= 0.23 \text{ sec} < T_L$$

Building weights:

w_{roof} = (240 × 180) 0.020 = 864 kip
w_{walls} = 0.0526 × 26 = 1.37 kip/ft

Design for seismic motion acting north-south:

The east and west elevations will be designed as ordinary precast concrete shear walls. Because these elevations are load-bearing, R = 3. If the hollow core wall panels and their connections are designed to satisfy the criteria of intermediate or special reinforced shear walls, a larger value of R may be used. A judgment must be made regarding the economics of possible reduced forces versus increased detailing requirements.

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$
$$= \frac{0.185}{\left(\frac{3}{1}\right)} = 0.062$$

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$$
$$= \frac{0.092}{0.23\left(\frac{3}{1}\right)} = 0.133$$

Use C_s = 0.062

Load to roof diaphragm

$$W = 864 + 2(240)\frac{1.37}{2}$$
$$= 1192 \text{ kip}$$

Diaphragm to shear wall forces

F_{roof} = C_sW
= 0.062(1192)
= 73.9 kip

F_{max} = $0.4S_{DS}I_eW_p$
= 0.4(0.185)(1.0)(1192)
= 88.2 kip

F_{min} = $0.2S_{DS}I_eW_p$
= 0.2 (0.185)(1.0)(1192)
= 44.1 kip

Therefore, use V = 73.9 kip

Applying the required 5% accidental torsion, the force applied to the top of the shear wall is:

F_u = 0.55 × 73.9
= 40.6 kip

With twenty-six 6-ft-wide panels available, the top connections must be designed for a shear force of:

$$V_u = \frac{40.6}{26}$$
$$= 1.56 \text{ kip/panel}$$

Shear wall design forces

The analysis of the shear walls is similar to the analysis for wind presented in section 5.4.

F_{roof} = 40.6 kip applied at h = 25 ft
F_{walls} = 0.062(1.37)(180)
= 15.3 kip applied at h = 13 ft

$$M_u = \frac{40.6(25) + 15.3(13)}{26 \text{ panels}}$$
$$= 46.7 \text{ kip-ft/panel}$$

The resisting dead load is:

$$D = \left(26(0.0526) + \frac{30}{2}(0.020)\right)6$$
$$= 10.0 \text{ kip/panel}$$

$$D_u = (0.9 - 0.2S_{DS})D$$
$$= (0.9 - 0.2(0.185))10.0$$
$$= 8.63 \text{ kip}$$

Assuming the tie-down T_u equals 10 kip:

$$a = \frac{8.63 + 10}{0.85(5)(8)} = 0.55 \text{ in.} = 0.046 \text{ ft}$$

$$46.7 = 0.9\left[8.63\left(\frac{6}{2} - \frac{0.046}{2}\right) + T_u\left(5 - \frac{0.046}{2}\right)\right]$$

Solving for T_u yields:

T_u = 5.3 kip

And $V_u = \dfrac{40.6+15.3}{26} = 21.5 \text{ kip / panel}$

Therefore, the hollow core wall panel and its base connections must be designed for an in-plane shear of 2.15 kip and a tie-down force of 5.3 kip.

Tie-back forces on the north and south

Since the panels on the north and south elevations are non-load-bearing and do not act as shear walls for north-south forces, they may be designed as nonstructural walls. Applying the exemption from Section 13.1.4(3) of ASCE 7-10, these panels are exempt from seismic design requirements.

Design for seismic motion acting east-west:

As non-load-bearing, ordinary precast concrete shear walls, the north and south elevations will be designed using R equal to 4.

$$C_s = \frac{S_{DS}}{\left(\dfrac{R}{I_e}\right)} = \frac{0.185}{\left(\dfrac{4}{1}\right)} = 0.046$$

$$C_s = \frac{S_{D1}}{T\left(\dfrac{R}{I_e}\right)} = \frac{0.092}{0.23\left(\dfrac{4}{1}\right)} = 0.10$$

Use $C_s = 0.046$

Load to roof diaphragm

$$w = 864 + 2(180)\frac{1.37}{2}$$
$$= 1111 \text{ kip}$$

Diaphragm to shear wall forces

$F_{roof} = C_s W$
$= 0.046(1111)$
$= 51.1$ kip
$F_{min} = 0.2 S_{DS} I_e W_p$
$= 0.2(0.185)(1.0)(1111)$
$= 41.1$ kip

Therefore, use $V = 51.1$ kip

Including 5% accidental torsion, the force applied to the top of the shear wall is:

$F_u = 0.55(51.1)$
$= 28.1$ kip

With thirty 6-ft-wide hollow core wall panels available, the top connections must be designed for a shear force of:

$$V_u = \frac{28.1}{30} = 0.94 \text{ kip/panel}$$

Shear wall design forces

$F_{roof} = 28.1$ kip applied at $h = 25$ ft
$F_{walls} = 0.046(1.37)(240)$
$= 15.1$ kip applied at $h = 13$ ft

$$M_u = \frac{28.1(25) + 15.1(13)}{30 \text{ panels}}$$
$$= 30.0 \text{ kip-ft/panel}$$

The resisting dead load is:

$$D = 26(0.0526)(6)$$
$$= 8.21 \text{ kip/panel}$$
$$D_u = (0.9 - 0.2S_{DS})D$$
$$= (0.9 - 0.2(0.185))8.21$$
$$= 7.1 \text{ kip}$$

Assuming the tie-down T_u equals 5 kip:

$$a = \frac{7.1+5}{0.85(5)(8)} = 0.36 \text{ in.} = 0.030 \text{ ft}$$

$$30.0 = 0.9\left[7.1\left(\frac{6}{2} - \frac{0.030}{2}\right) + T_u\left(5 - \frac{0.030}{2}\right)\right]$$

Solving for T_u yields:

$T_u = 2.4$ kip

and $V_u = \dfrac{28.1+15.1}{30} = 1.44 \text{ kip / panel}$

Therefore, the hollow core wall panel and its base connections must be designed for an in-plane shear of 1.44 kip and a tie-down force of 2.4 kip.

Tie-back forces on the east and west

Because the panels on the east and west elevations are load-bearing, tie-backs must satisfy Sections 1.4.5 and 12.11 of ASCE 7-10.

The out-of-plane tie-back force is the greatest of the following:

$$V_u = 0.4 S_{DS} I_e W_p$$
$$= 0.4(0.185)(1.0)(0.0526)(6)\left(\frac{26}{2}\right)$$
$$= 0.3 \text{ kip}$$

$$k_a = 1.0 + \frac{L_f}{100} = 1.0 + \frac{180}{100} = 2.8 \text{ Use } 2.0$$

$$V_u = 0.2 k_a I_e W_p$$
$$= 0.2(2.0)(1.0)(0.0526)(6)\left(\frac{26}{2}\right)$$
$$= 1.64 \text{ kip}$$

but not greater than

$$V_u = 0.4 S_{DS} k_a I_e W_p$$
$$= 0.4(0.185)(2.0)(1.0)(0.0526)(6)\left(\frac{26}{2}\right)$$
$$= 0.61 \text{ kip}$$

$$V_u = 0.2 W_p$$
$$= 0.2(0.0526)(6)\left(\frac{26}{2}\right)$$
$$= 0.82 \text{ kip}$$

$$V_u = 0.005(6)\left(\frac{26}{2}\right) = 0.39 \text{ kip}$$

The connections at the top and bottom of the hollow core wall panel should be designed for a tie-back force of 0.82 kip per panel.

Example 5.5.1(b)

Given the same building, but located in San Francisco, Calif., determine the seismic design forces for the wall panels.

Note: This example is included to demonstrate the analysis procedure and the magnitude of design forces that may be encountered in a high-seismic-risk area. Careful consideration should be given as to whether hollow core wall panels are suitable for this application.

A complete design must include designing the wall panel for bending and shear and will require transverse steel equal to the greater of the minimum horizontal wall reinforcement or the calculated shear reinforcement. The precast designer should evaluate if the required connection ductility and overload factors can be accomplished in their product.

San Francisco, Calif. Zip 94110
Occupancy category: II
Importance factor $I_e = 1.0$
Site class D

Steel joist / Metal deck roof 20 lb/ft^2
8 in. thick, 6-ft-wide hollow core wall panels weighing 52.6 lb/ft^2

Solution:

From USGS $S_S = 1.500g$
$S_1 = 0.600g$
$T_L = 12$ sec

ASCE 7-10, Tables 11.4-1 and 11.4-2

$F_a = 1.0$
$F_v = 1.5$
$S_{MS} = 1.0\ (1.500) = 1.500$g
$S_{M1} = 1.5(0.600) = 0.900$g

$$S_{DS} = \frac{2}{3}(1.500) = 1.000$$

$$S_{D1} = \frac{2}{3}(0.900) = 0.600$$

Per Tables 11.6-1 and 11.6-2, the building is in SDC D.

Building period:

$$T = C_t h_n^x$$
$$= 0.020(26)^{0.75}$$
$$= 0.23 \sec < T_L$$

Building weights:

$w_{roof} = (240 \times 180)0.020 = 864$ kip
$w_{walls} = 0.0526 \times 26 = 1.37$ kip/ft

Design for seismic motion acting north-south:

The east and west elevations will be designed as intermediate precast concrete shear walls in a bearing wall system. Therefore, R is equal to 4.

$$C_S = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$

$$= \frac{1.000}{\left(\frac{4}{1}\right)} = 0.25$$

$$C_S = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$$

$$= \frac{0.600}{0.23\left(\frac{4}{1}\right)} = 0.65$$

Use $C_s = 0.25$

Load to roof diaphragm

$$w = 864 + 2(240)\frac{1.37}{2}$$
$$= 1192 \text{ kip}$$

Diaphragm to shear wall forces

$F_{roof} = C_sW$
$= 0.25(1192)$
$= 298$ kip
$F_{max} = 0.4S_{DS}I_eW_p$
$= 0.4(1.000)(1.0)(1192)$
$= 477$ kip
$F_{min} = 0.2S_{DS}I_eW_p$
$= 0.2(1.000)(1.0)(1192)$
$= 238$ kip

Therefore, use $V = 298$ kip

Applying the required 5% accidental torsion, the force applied to the top of the shear wall is

$F_u = 0.55 \times 298$
$= 164$ kip

With twenty-six 6-ft-wide hollow core wall panels available the top connections must be designed for a shear force of

$$V_u = \frac{164}{26}$$
$$= 6.3 \text{ kip/panel}$$

Shear wall design forces

$F_{roof} = 164$ kip applied at $h = 25$ ft
$F_{walls} = 0.25(1.37)(180)$
$= 61.6$ kip applied at $h = 13$ ft

$$M_u = \frac{164(25) + 61.6(13)}{26 \text{ panels}}$$

$$= 189 \text{ kip-ft/panel}$$

The resisting dead load is:

$$D = \left[26(0.0526) + \frac{30}{2}(0.020)\right]6$$
$$= 10.0 \text{ kip/panel}$$
$$D_u = (0.9 - 0.2S_{DS})D$$
$$= (0.9 - 0.2(1.000))10.0$$
$$= 7.0 \text{ kip}$$

Because using single panels will result in a tie-down of over 38 kip, try connecting the panels to form thirteen 12-ft-wide hollow core units. This results in $M_u = 378$ kip-ft per unit and $D_u = 14.0$ kip.

Assuming T_u equals 32 kip:

$$a = \frac{14.0 + 32}{(.85)(5)(8)} = 1.35 \text{ in.} = 0.113 \text{ ft}$$

$$378 = 0.9\left[14.0\left(\frac{12}{2} - \frac{0.113}{2}\right) + T_u\left(11 - \frac{0.113}{2}\right)\right]$$

Solving for T_u yields:

$$T_u = 30.8 \text{ kip}$$

and $$V_u = \frac{164 + 61.6}{13}$$
$$= 17.4 \text{ kip / unit}$$

Therefore, the two-panel hollow core unit and its base connections must be designed for an in-plane shear of 17.4 kip and a tie-down force of 30.8 kip.

As an intermediate precast concrete shear wall, special care must be taken in detailing the base connections and panel-to-panel connections. Section 21.4 of ACI 318-11 requires that the weak link in the connection be yielding of steel elements or reinforcement. All other components of the connection must be designed for 150% of their yield strength.

Tie-back forces on the north and south

The panels on the north and south elevations may be designed as nonstructural walls using ASCE 7-10, chapter 13.

From Table 5.5.1 with a flexible diaphragm, flexural design of the panels is based on:

$$w = \frac{0.4 S_{DS} w_p}{2.5}\left(1+2\frac{z}{h}\right)$$

$$= \frac{0.4(1.000)(52.6)}{2.5}\left(1+\frac{2(13)}{26}\right)$$

$$= 16.8\,\text{psf} \qquad \text{controls}$$

$$w_{\min} = 0.3 S_{DS} I_p w_p$$

$$= 0.3(1.000)(1.0)(52.6)$$

$$= 15.8\,\text{psf}$$

$$w_{\max} = 1.6 S_{DS} I_p w_p$$

$$= 1.6(1.000)(1.0)(52.6)$$

$$= 84.2\,\text{psf}$$

Connections:

$$F_p = 0.20 W_p = 0.20(0.0526)(6)\left(\frac{26}{2}\right)$$

$$= 0.82\,\text{kip}$$

$$F_p = 0.4 S_{DS} k_a I_e W_p$$

$$= 0.4(1.000)(2.0)(1.0)(0.0526)(6)\left(\frac{26}{2}\right)$$

$$= 3.28\,\text{kip/panel}$$

$$F_{p\min} = 0.2 k_a I_e W_p$$

$$= 0.2(2.0)(1.0)(0.0526)(6)\left(\frac{26}{2}\right)$$

$$= 1.64 \text{ kip / panel}$$

These connection forces would be per panel at the top and bottom of the panel.

Design for seismic motion acting east-west:

The north and south elevations are non-load-bearing, intermediate precast concrete shear walls and will be designed using R equal to 5.

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$

$$= \frac{1.000}{\left(\frac{5}{1}\right)} = 0.20$$

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$$

$$= \frac{0.600}{0.23\left(\frac{5}{1}\right)} = 0.52$$

Use $C_s = 0.20$

Load to roof diaphragm

$$w = 864 + 2(180)\frac{1.37}{2}$$

$$= 1111 \text{ kip}$$

Diaphragm to shear wall forces

$$F_{roof} = C_s W$$
$$= 0.20(1111)$$
$$= 222 \text{ kip}$$
$$F_{min} = 0.2 S_{DS} I_e w_p$$
$$= 0.2(1.000)(1.0)(1111)$$
$$= 222 \text{ kip}$$

Therefore, use $V = 222$ kip

Including 5% accidental torsion, the force applied to the top of the shear wall is:

$$F_u = 0.55(222)$$
$$= 122 \text{ kip}$$

With thirty 6-ft-wide hollow core wall panels available, the top connections must be designed for a shear force of

$$V_u = \frac{122}{30}$$

$$= 4 \text{ kip / panel}$$

Shear wall design forces

F_{roof} = 122 kip applied at h = 25 ft
F_{walls} = 0.20(1.37)(240)
= 65.8 kip applied at h = 13 ft

$$M_u = \frac{122(25)+65.8(13)}{30 \text{ panels}}$$

$$= 130 \text{ kip-ft / panel}$$

The resisting dead load is:

D = 26(0.0526)(6)
= 8.21 kip / panel
$D_u = (0.9 - 0.2S_{DS})D$
= (0.9 – 0.2(1.000))8.21
= 5.74 kip

Assuming the tie-down force equals 26 kip:

$$a = \frac{5.74+26}{(0.85)(5)(8)} = 0.93 \text{in.} = 0.078 \text{ft}$$

$$130 = 0.9\left[5.74\left(\frac{6}{2} - \frac{0.078}{2}\right) + T_u\left(5 - \frac{0.078}{2}\right)\right]$$

Solving for T_u yields:

T_u = 25.7 kip

and $V_u = \dfrac{122+65.8}{30}$ = 6.26 kip/panel

The hollow core wall panel and its base connections must be designed for an in-plane shear of 6.26 kip and a tie-down force of 25.7 kip, with detailing that satisfies the requirements of an intermediate precast concrete shear wall.

Tie-back forces on the east and west

The hollow core wall panels on the east and west elevations are load-bearing structural walls. As such, they must meet the requirements of Sections 1.4.5 and 12.11 of ASCE 7-10.

The out-of-plane tie-back force is the greatest of the following:

$$V_u = 0.4 S_{DS} I_e W_p$$

$$= 0.4(1.000)(1.0)(0.0526)(6)\left(\frac{26}{2}\right)$$

$$= 1.64 \text{kip}$$

$$V_u = 0.4 S_{DS} k_a I_e W_p$$

$$= 0.4(1.000)(2.0)(1.0)(0.0526)(6)\left(\frac{26}{2}\right)$$

$$= 3.28 \text{ kip}$$

$$V_{u\min} = 0.2 k_a I_e W_p$$

$$= 0.2(2.0)(1.0)(0.0526)(6)\left(\frac{26}{2}\right)$$

$$= 1.64 \text{kip}$$

The controlling out-of-plane force is 3.28 kip per panel at the top and the bottom.

Chapter 6

CONNECTIONS IN HOLLOW CORE SLABS AND PANELS

6.1 General

Connections will be required in hollow core slab systems for a variety of reasons. If the slabs are required to perform as a diaphragm, internal connections and connections to the lateral force-resisting elements will be required as outlined in Chapter 4. For an untopped system, minimal connections may be required to satisfy the structural integrity requirements of ACI 318[3]. In addition, localized forces such as bracing an interior or exterior wall, laterally bracing the top flange of a beam, or hanging mechanical equipment or a ceiling may require connections. Hollow core wall panels will require connections that may include tie-backs to the structure, shear wall connections, and gravity support of floors or roofs. In many instances, a connection will be designed to resist forces in multiple directions, either separately or simultaneously. Other connections are specifically designed to provide resistance in only one direction to avoid generating unexpected loads to either the panel or the structure.

Connections are an expense to a project and, if used improperly, may have detrimental effects by not accommodating volume change movements that occur in a precast structure. Forces may develop in connections as they restrain these movements. In specifying connection requirements, the actual forces in the connection must be addressed. If no force can be shown to exist, the connection should not be used. When a connection is determined to be necessary, the force in the connection should be specified, particularly at an interface between the hollow core and another material. The hollow core supplier is generally responsible for designing and detailing only those items that will be supplied with the hollow core product.

6.2 Details

Sections 6.3 through 6.8 show a number of conditions where forces are likely to exist that need to be transmitted into or through a hollow core slab or wall panel. Common details are illustrated to give the designer an idea of the connection possibilities that exist. The commentary provided with each detail is intended to give a better understanding of the merits of each detail. The details are only conceptual and are intended as a guide that can be used for further discussions with local producers. A final design requires knowledge of both the project design forces and the hollow core producer's capabilities.

Differences between wet cast and dry cast hollow core slabs will be evident in the embedded anchors that can be provided. Without forms to which anchors might be secured, dry cast systems may be limited to shallow anchors that can be tied directly to strands or to inserts that can be placed after casting. Wet cast systems can accommodate a wider variety of anchors placed directly in the form prior to casting. Therefore, details of the embedment in the hollow core slabs are not shown. Other connection details perform functions similar to those shown. Consult a local PCI producer for information on relative economy and design capabilities.

Chapter 6 Figure List

6.3 Typical Details with Concrete Beams

Design Considerations:

- Can transfer internal diaphragm forces
- Can be designed as structural integrity tie

Fabrication Considerations:

- Advantageous to not have hardware in slab
- Beam embedments must line up with slab joints
- Accommodates variations in slab length

Erection Considerations:

- Advantageous to have connection completed by follow-up crew
- Difficult for welder to hold loose plate in position

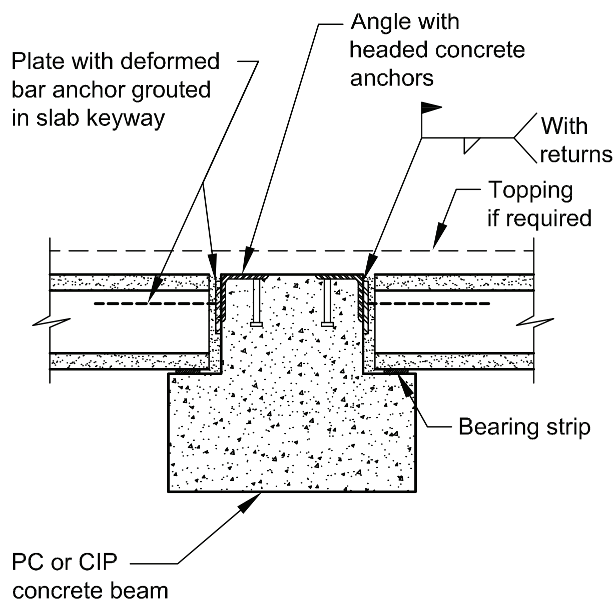


Figure 6.3.1

Design Considerations:

- Can transfer internal diaphragm forces
- Can be designed as structural integrity tie

Fabrication Considerations:

- May increase beam reinforcement for shallower beam
- Layout must have opposing slab joints lined up

Erection Considerations:

- Clean and simple

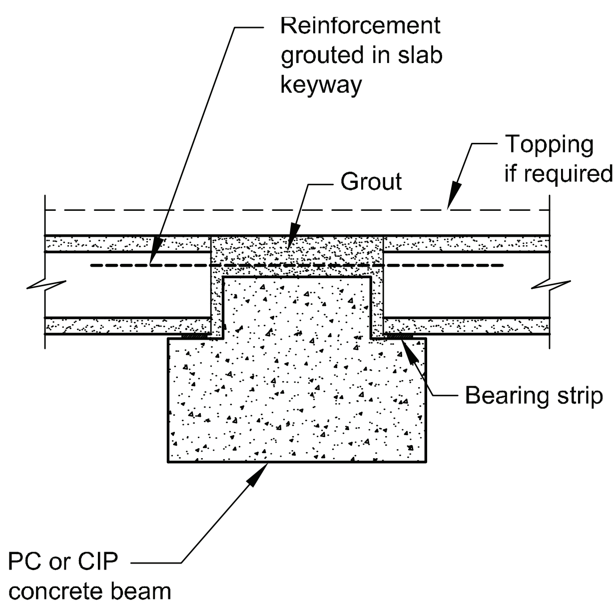


Figure 6.3.2

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.3 Typical Details with Concrete Beams (Continued)

<u>Design Considerations:</u>

- Should not be relied on to transfer lateral or diaphragm forces
- Structural integrity ties may be required if no topping is provided

<u>Fabrication Considerations:</u>

- Clean and simple

<u>Erection Considerations:</u>

- Clean and simple

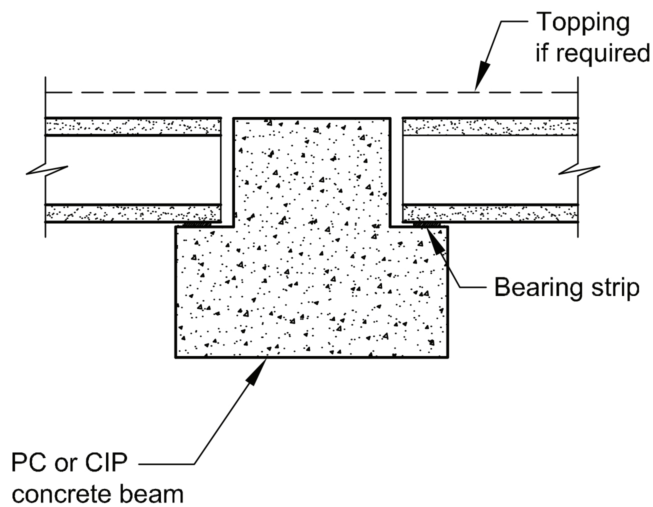


Figure 6.3.3

<u>Design Considerations:</u>

- Can transfer internal diaphragm forces
- Can be designed as structural integrity tie
- Consider concrete cover on reinforcement over beam

<u>Fabrication Considerations:</u>

- Slab layout must have opposing joints lined up

<u>Erection Considerations:</u>

- Clean and simple

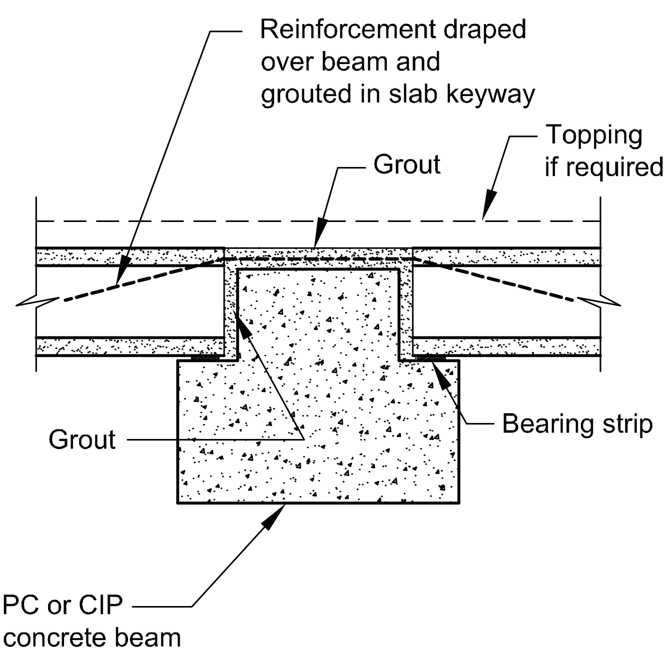


Figure 6.3.4

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.3 Typical Details with Concrete Beams (Continued)

Design Considerations:

- Can transfer internal diaphragm forces
- Will develop volume change restraint forces that must be considered in design of connections

Fabrication Considerations:

- Slab manufacturing system must allow bottom weld anchors
- Beam inserts must align with slab inserts allowing fabrication tolerances

Erection Considerations:

- Connections can be completed by follow up crew
- Access for welding may require ladders or scaffold
- Spacer may be required to make weld

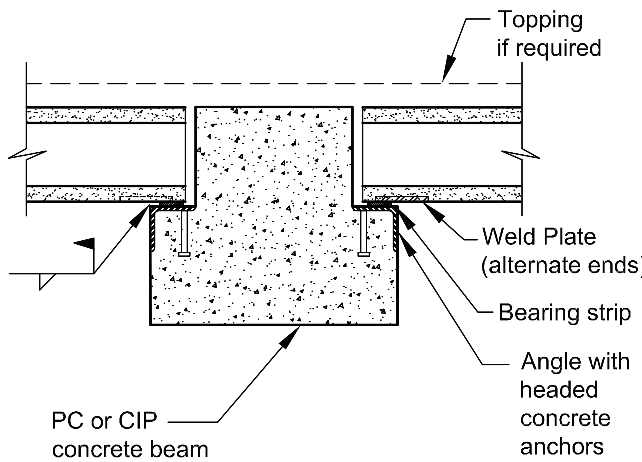


Figure 6.3.5

Design Considerations:

- Can transfer internal diaphragm forces
- Can be designed as structural integrity tie
- Horizontal shear from beam cap must be transferred
- Opposing slab joints must line up

Fabrication Considerations:

- Clean and simple for slabs

Erection Considerations:

- Beam may have to be shored until cap is cured
- Horizontal shear reinforcement may present safety hazard for erector
- Core dams must be placed

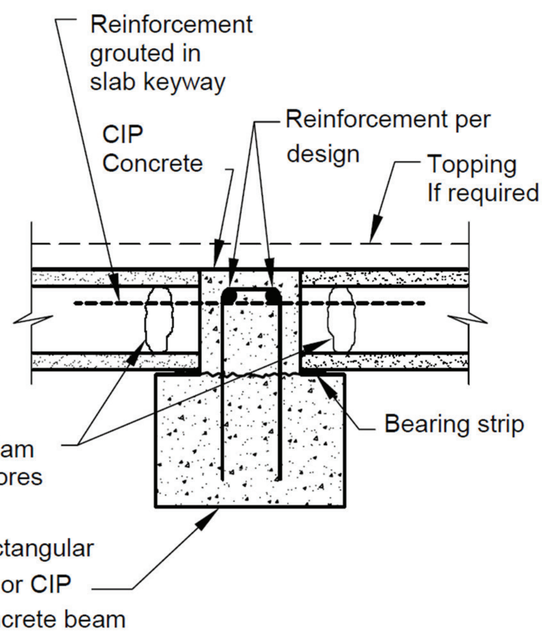


Figure 6.3.6

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.3 Typical Details with Concrete Beams (Continued)

Design Considerations:

- Can transfer internal diaphragm forces
- Can be designed as structural integrity tie
- Horizontal shear from beam cap must be transferred
- Opposing slab joints must line up

Fabrication Considerations:

- Clean and simple for slabs

Erection Considerations:

- Beam may have to be shored until topping is cured
- Horizontal shear reinforcement may present safety hazard for erector
- Core dams must be placed

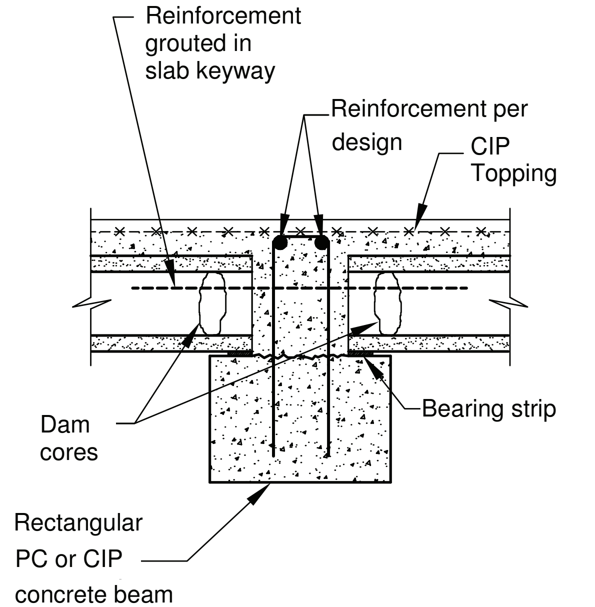


Figure 6.3.7

Design Considerations:

- Can transfer diaphragm shear
- Can provide lateral brace for beam
- Potential for negative moment in slabs

Fabrication Considerations:

- Slab insert difficult to install. Because of tolerance on sawcut ends, the insert should be installed after slabs are cut to length
- Beam and slab inserts must align

Erection Considerations:

- If required for lateral beam stability, welding may have to be completed as slabs are set

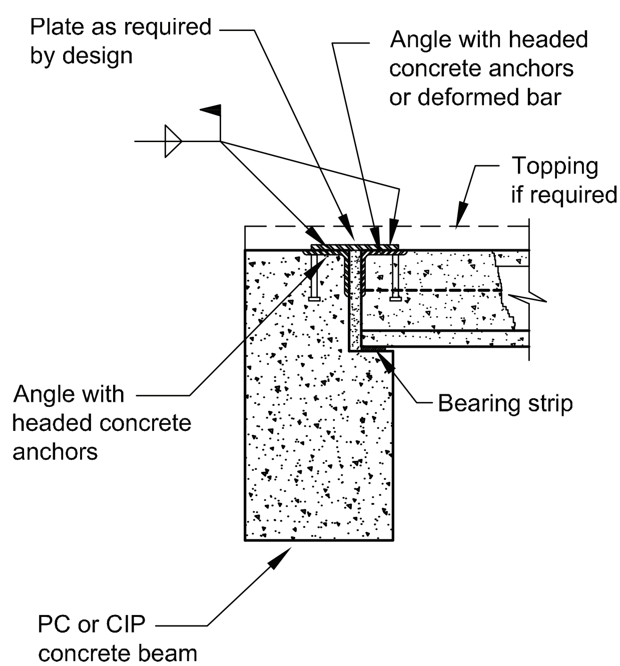


Figure 6.3.8

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.3 Typical Details with Concrete Beams (Continued)

Design Considerations:

- Can transfer diaphragm shear
- Can provide lateral brace for beam
- Potential for negative moment in slabs

Fabrication Considerations:

- Plates in beams must align with slab joints allowing tolerance

Erection Considerations:

- Connection can be completed by follow up crew
- Lateral bracing for beam will not be provided until keyway grout cures

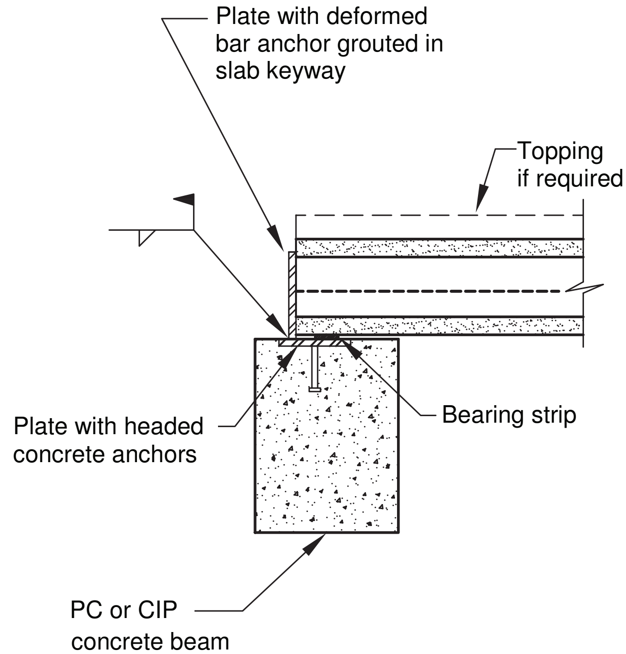


Figure 6.3.9

Design Considerations:

- Can transfer internal diaphragm forces
- Can be designed as structural integrity tie

Fabrication Considerations:

- Clean and simple

Erection Considerations:

- Clean and simple
- Keyway dimensions may limit the reinforcement diameter

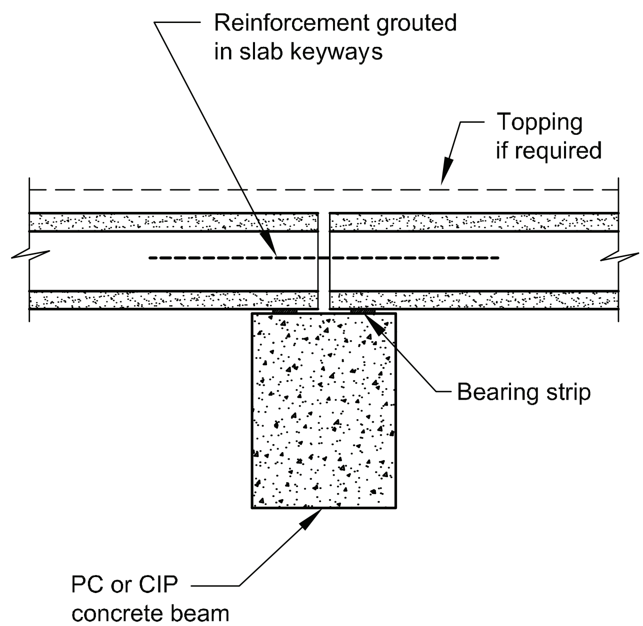


Figure 6.3.10

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.3 Typical Details with Concrete Beams (Continued)

Design Considerations:

- Can transfer diaphragm shear
- Can be designed as structural integrity tie

Fabrication Considerations:

- Clean and simple for both beam and slabs

Erection Considerations:

- Reinforcement must be tied in place
- Concrete must be cast around reinforcement
- Edge form is required for cast-in-place concrete
- Dowels from beam may present safety hazard

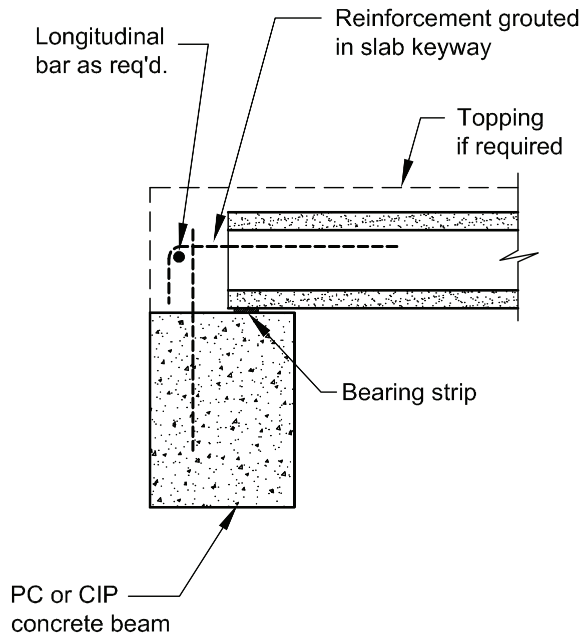


Figure 6.3.11

Design Considerations:

- Can transfer internal diaphragm forces
- Will develop volume change restraint forces that must be considered in design of connection

Fabrication Considerations:

- Slab manufacturing system must allow bottom weld inserts
- Beam and slab inserts must align with allowance for tolerance

Erection Considerations:

- Connections can be completed by follow-up crew
- Access for welding may require ladders or scaffold
- Spacer may be required to make weld

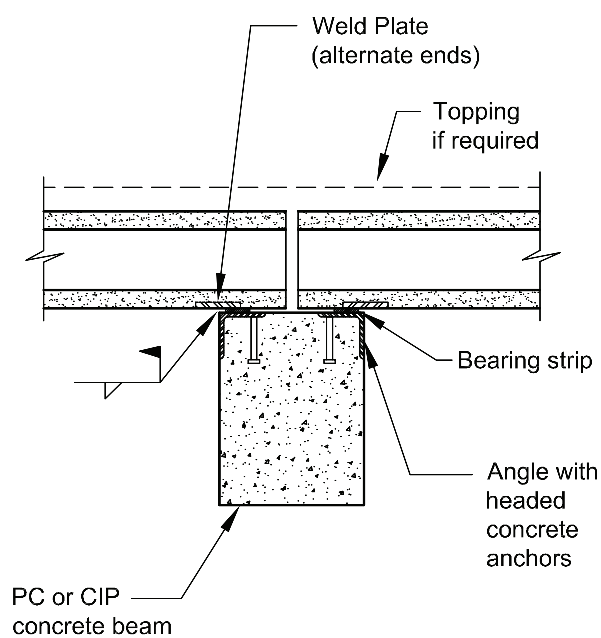


Figure 6.3.12

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.3 Typical Details with Concrete Beams (Continued)

Design Considerations:

- Can transfer diaphragm shear
- Torsional and lateral beam restraint can be provided
- Will develop volume change restraint forces that must be considered in design

Fabrication Considerations:

- Slab manufacturing system must allow bottom weld inserts
- Beam and slab inserts must align with allowance for tolerance

Erection Considerations:

- Connections can be completed by follow-up crew
- Access for welding may require ladders or scaffold
- If required for lateral or torsional stability, welding may have to be completed as slabs are set
- Spacer may be required to make weld

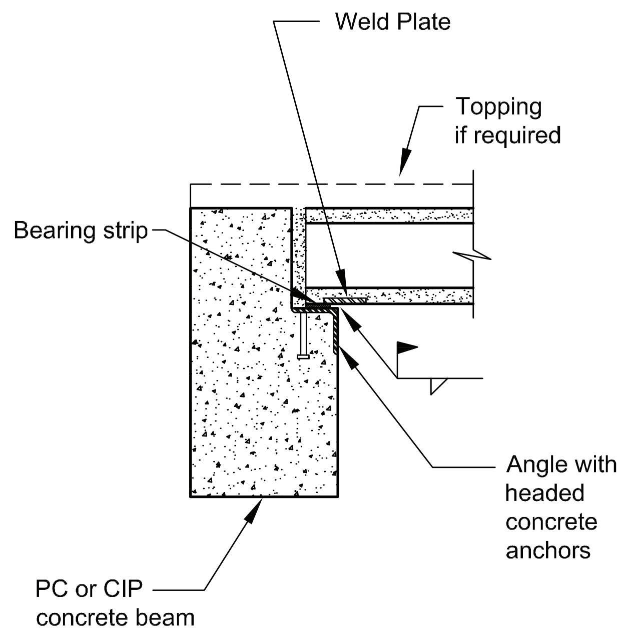


Figure 6.3.13

Design Considerations:

- This detail is not recommended because of installation difficulties which may result in an unreliable connection

Fabrication Considerations:

- Great difficulty aligning bars with keyways

Erection Considerations:

- Potential difficulties in bending bars
- Possible fracture of bent bars
- Second rebar bend may be required to align with slab joints
- Cast-in-place concrete required around reinforcement
- Edge forming required

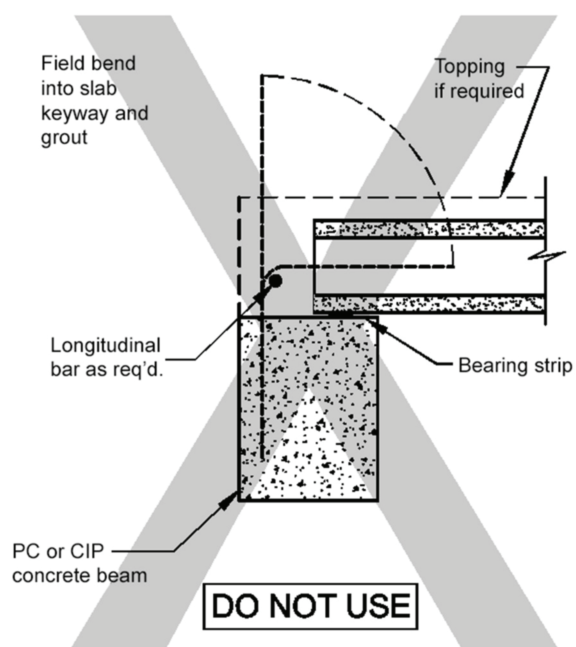


Figure 6.3.14

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.4 Typical Details with Walls

Design Considerations:

- Can transfer diaphragm shear
- Can be designed as structural integrity tie
- Can provide lateral brace for wall
- Consider axial force path through slab ends
- Opposing slab joints must line up

Fabrication Considerations:

- Clean and simple for slabs
- Small tolerance for placement of bars in walls
- Small tolerance for length of slabs to accommodate bars in joint

Erection Considerations:

- With longitudinal bar, have potential for congestion
- Slab erection must consider tight tolerance on butt slab joint gap
- With precast walls, consider method of installing vertical dowel

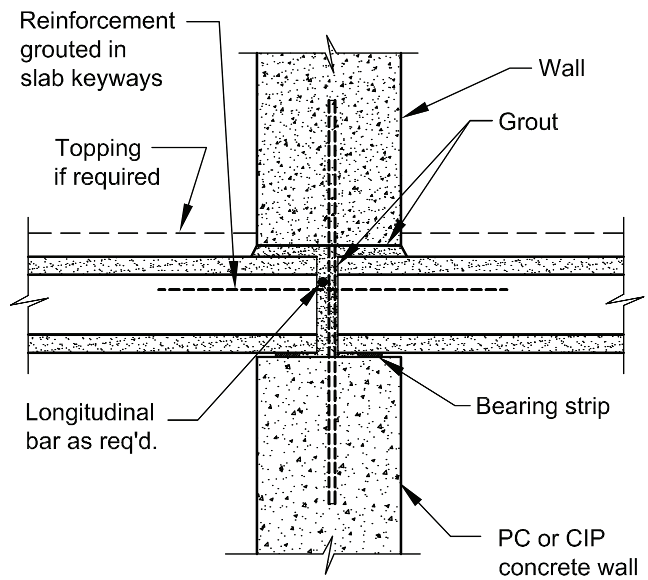


Figure 6.4.1

Design Considerations:

- Can transfer diaphragm shear
- Can be designed as structural integrity tie
- Can provide lateral brace for wall
- Opposing slab joints must line up

Fabrication Considerations:

- Clean and simple for slabs

Erection Considerations:

- Clean and simple
- Wall is not braced until grout is placed and cured

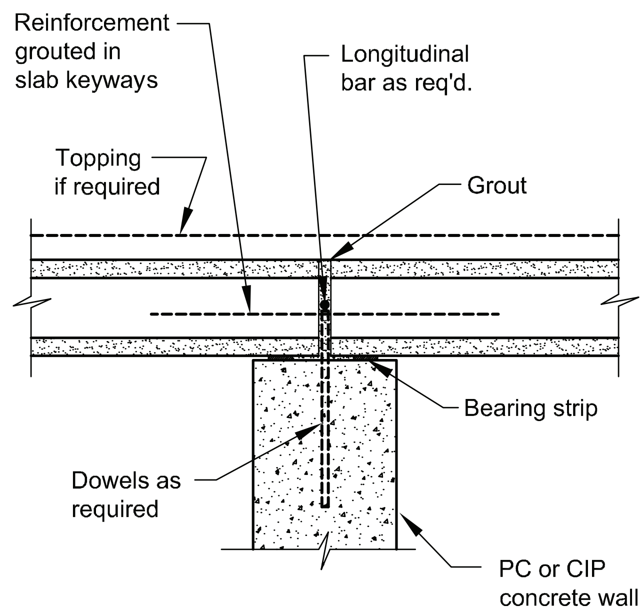


Figure 6.4.2

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.4 Typical Details with Walls (Continued)

Design Considerations:

- Can transfer diaphragm shear
- With proper bar detailing, can provide lateral brace for wall
- Consideration should be given to forces developed as slab ends rotate

Fabrication Considerations:

- Clean and simple
- Small tolerance for placement of bars in walls

Erection Considerations:

- Simple for slab erection
- Mason can set bars independent of the slab joints
- Some block cutting may be required for bars from keyways

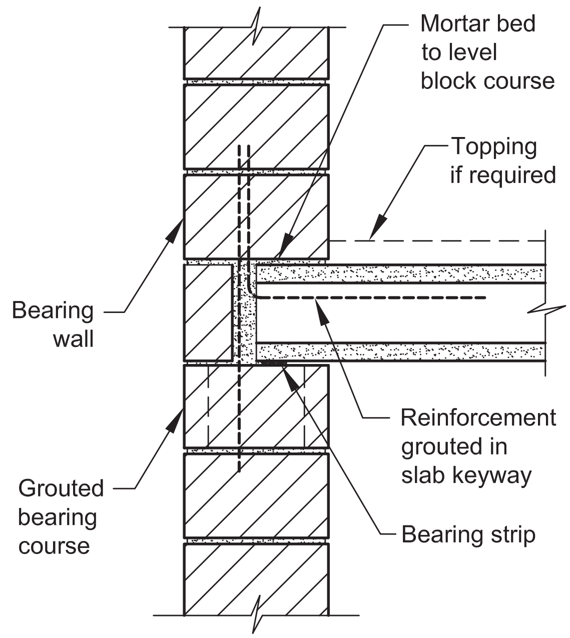


Figure 6.4.3

Design Considerations:

- Can transfer diaphragm shear
- With proper bar detailing, can provide lateral brace for wall
- Consideration should be given to forces developed as slab ends rotate

Fabrication Considerations:

- Clean and simple
- Small tolerance for placement of bars in walls

Erection Considerations:

- Simple for slab erection
- The mason can set bars independent of the slab joints
- Grout at slab end may be difficult to place

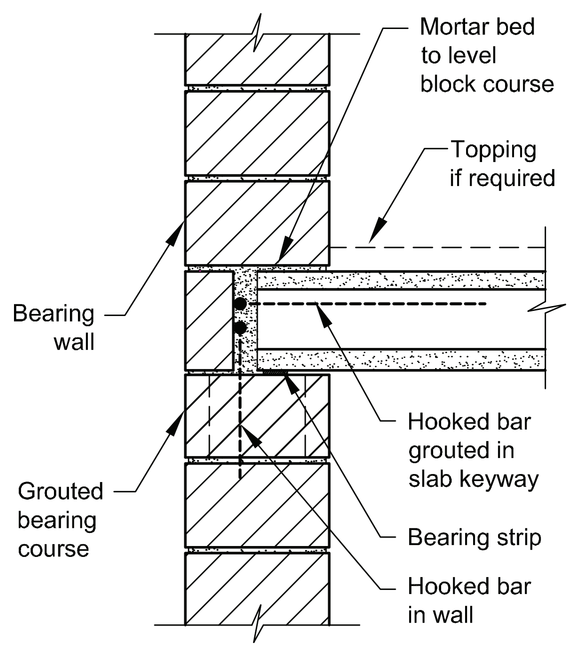


Figure 6.4.4

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.4 Typical Details with Walls (Continued)

Design Considerations:

- This detail is not recommended because of installation difficulties which may result in an unreliable connection

Fabrication Considerations:

Erection Considerations:

- Mason will have great difficulty locating bars at slab joints
- Potential difficulties to field bend bars including fracture
- Second bend may be required to align bars with joints

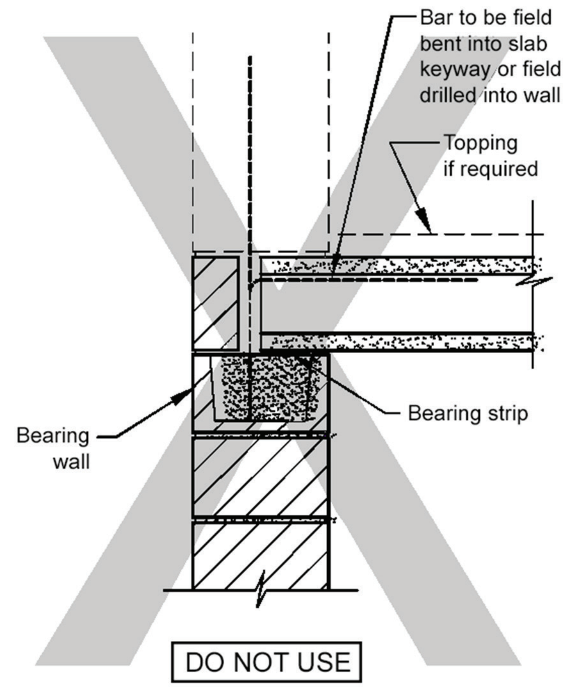


Figure 6.4.5

Design Considerations:

- Wall will not be braced at this level
- No lateral or diaphragm forces can be transferred

Fabrication Considerations:

- Clean and simple

Erection Considerations:

- Small tolerance in slab layout

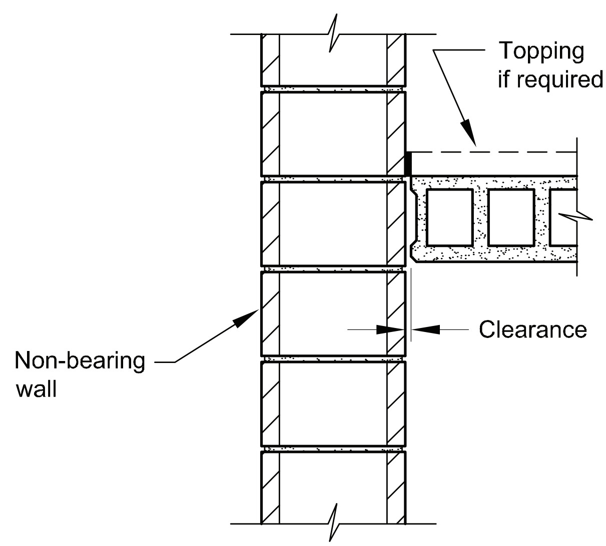


Figure 6.4.6

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.4 Typical Details with Walls (Continued)

Design Considerations:

- Can transfer diaphragm shear
- Can provide lateral brace for wall
- Allows for vertical deflection and camber growth

Fabrication Considerations:

- Slab manufacturing system must allow bottom weld anchors

Erection Considerations:

- Small tolerance in slab layout
- Connections can be completed by follow up crew
- Access for welding may require ladders or scaffold
- Wall is not braced until connection is made
- After setting bolt, nut should be backed off to “finger-tight”

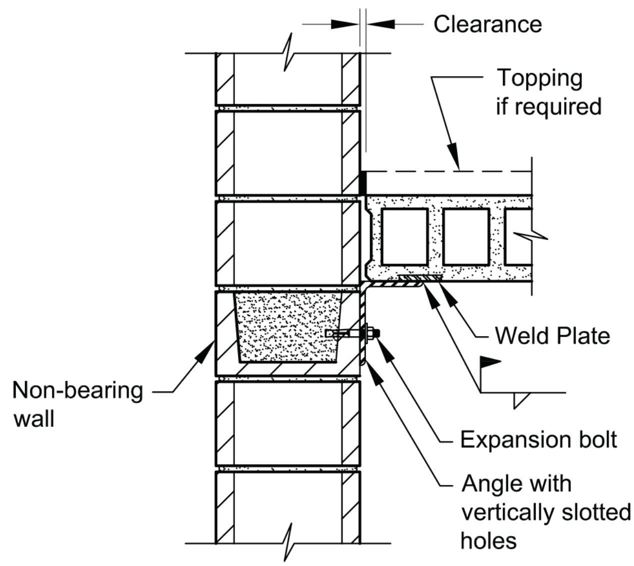


Figure 6.4.7

Design Considerations:

- Walls will not be laterally braced at this level
- Consideration should be given to forces developed from deflections or camber growth
- Dry pack may be required under slab for transferring axial wall load

Fabrication Considerations:

- Clean and simple

Erection Considerations:

- Allowance must be made for slab camber
- Wall will not be laterally braced at this level
- Small tolerance in slab layout

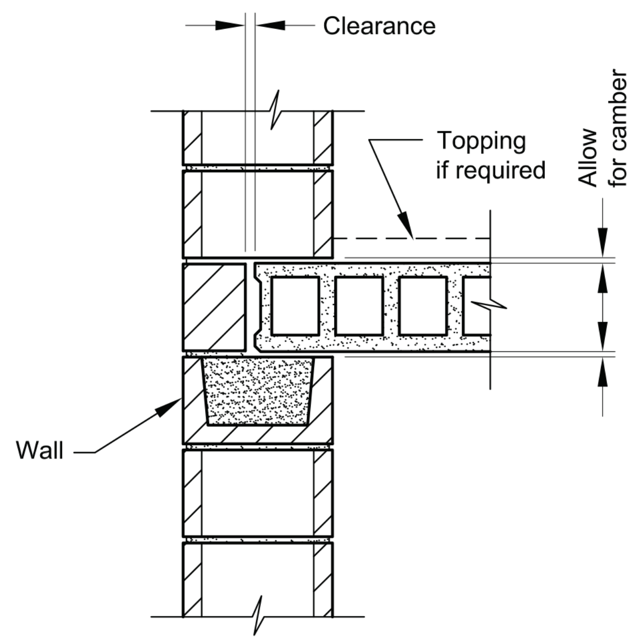


Figure 6.4.8

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.4 Typical Details with Walls (Continued)

Design Considerations:

- Can transfer diaphragm shear
- Can provide lateral brace for wall
- Consideration should be given to forces developed from deflection or camber growth
- Consider axial load path

Fabrication Considerations:

- If not done in field, slots and holes must be cut for steel
- In stack casting system, slots and holes might not be practically cut in plant

Erection Considerations:

- Allowance must be made for slab camber
- If not done in plant, holes and slots must be cut for steel
- Wall is not braced until steel is grouted

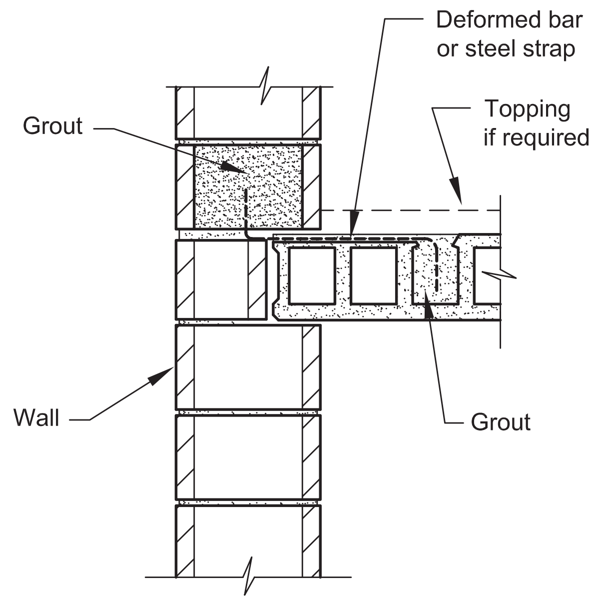


Figure 6.4.9

Design Considerations:

- Wall thrust from earth pressure can be resisted
- Can transfer diaphragm shear only with special detailing of keyway and reinforcement
- For long spans consider effects of restraint of vertical movement

Fabrication Considerations:

- Clean and simple

Erection Considerations:

- Edge joint must be grouted which may not be standard practice

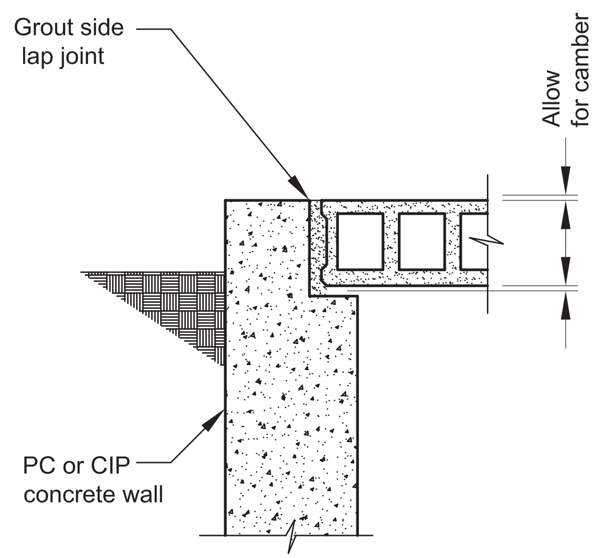


Figure 6.4.10

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.4 Typical Details with Walls (Continued)

Design Considerations:

- Can transfer diaphragm shear
- Can provide lateral brace for wall
- Consideration should be given to forces developed from deflections or camber growth

Fabrication Considerations:

- If not done in field, edge core must be cut open
- In stack casting operation, holes might not be practical to cut in plant

Erection Considerations:

- If not done in plant, holes must be field cut into edge core
- Mason may have to cut block to install reinforcement

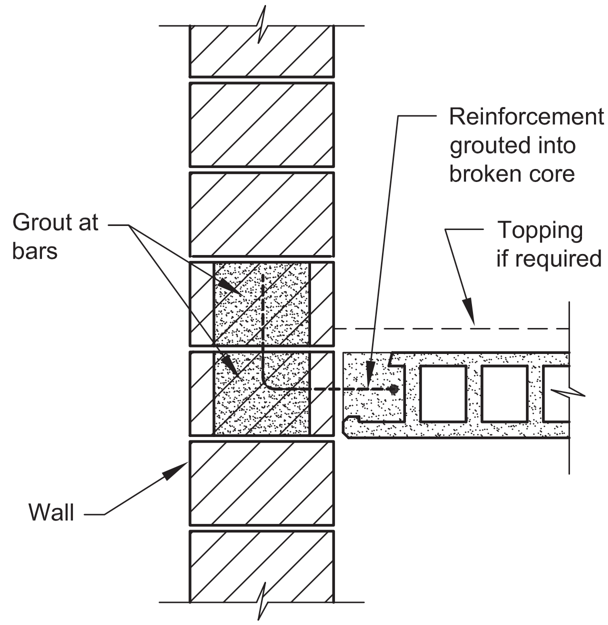


Figure 6.4.11

Design Considerations:

- Can transfer diaphragm shear
- Can provide lateral brace for wall
- Connection capacity must be verified by test

Fabrication Considerations:

- Clean and simple

Erection Considerations:

- Minimum edge distances must be maintained
- Holes must be drilled through slabs into masonry

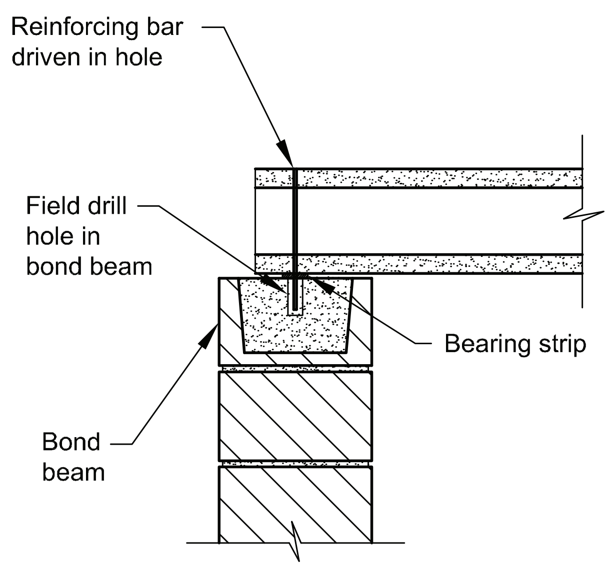


Figure 6.4.12

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.4 Typical Details with Walls (Continued)

Design Considerations:

- Can transfer diaphragm shear
- Can provide lateral brace for wall
- Consider effects of vertical restraint
- Connection capacity must be verified by test

Fabrication Considerations:

- Clean and simple

Erection Considerations:

- Minimum edge distances must be maintained
- Holes must be drilled through slabs into masonry

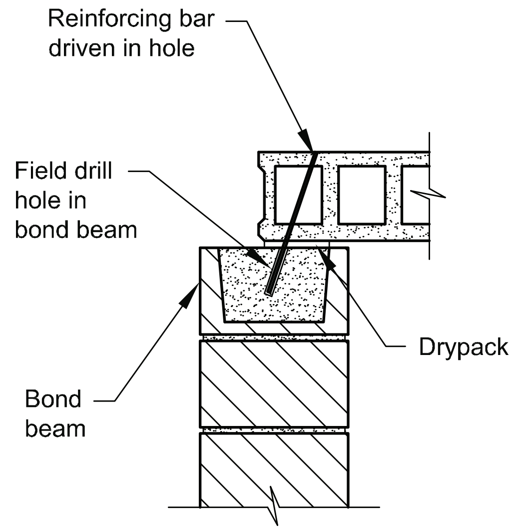


Figure 6.4.13

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.5 Typical Details with Steel Beams

Design Considerations:

- Top beam flange should be considered unbraced,
- Select beam flange width to allow minimum bearing considering tolerances

Fabrication Considerations:

- Clean and simple for slabs
- Beam flange width must be sufficient for slab bearing length

Erection Considerations:

- Unsymmetrical loading may cause beam instability
- Beam may need to be braced against rotation during erection

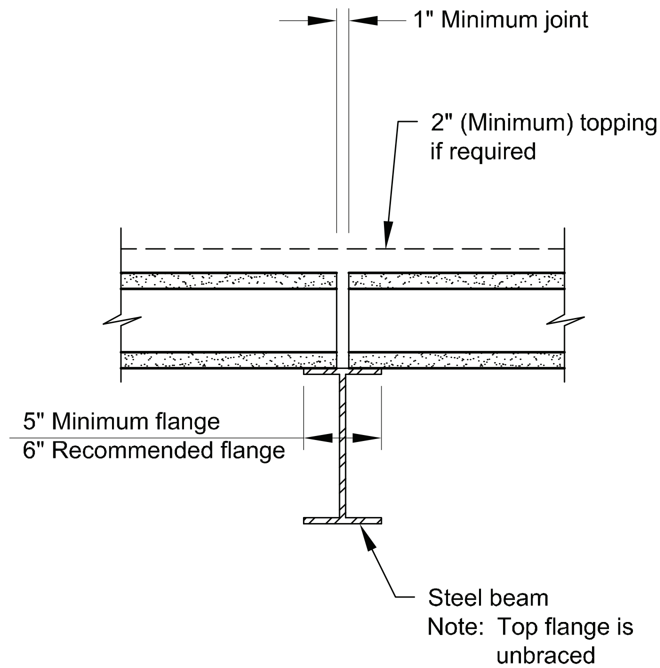


Figure 6.5.1

Design Considerations:

- Can transfer internal diaphragm forces
- Provides lateral brace for steel beam
- Select beam flange width to allow minimum bearing considering tolerances

Fabrication Considerations:

- Slab layout must align slab joints
- Stabilizer bars might be installed in the field or steel plant depending on local regulations or agreements
- Beam flange width must be sufficient for minimum slab bearing

Erection Considerations:

- Grouting of slabs must include the butt joint
- Steel erection may require that stabilizer bars be field installed
- Steel beam will not be laterally braced until grout cures
- Unsymmetrical loading may cause beam instability

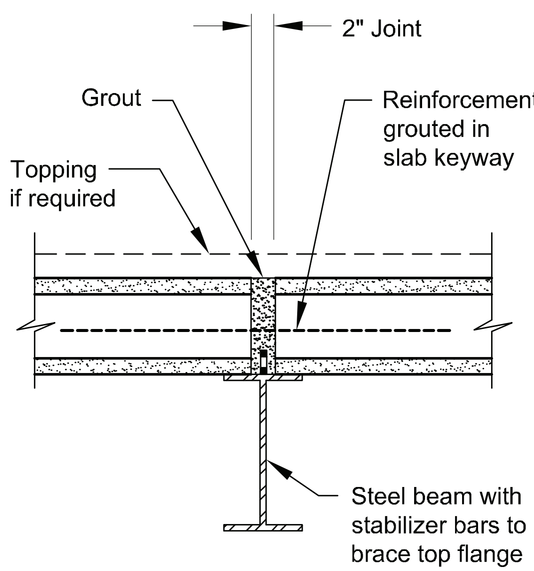


Figure 6.5.2

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.5 Typical Details with Steel Beams (Continued)

Design Considerations:

- Can transfer internal diaphragm forces
- Provides lateral brace for steel beam
- Will develop volume change restraint forces that must be considered in design of connection
- Select beam flange width to allow minimum bearing considering tolerances

Fabrication Considerations:

- Slab manufacturing system must allow for installation of bottom weld anchors

Erection Considerations:

- Welding of slabs to beam should be done as erection proceeds to laterally brace beams

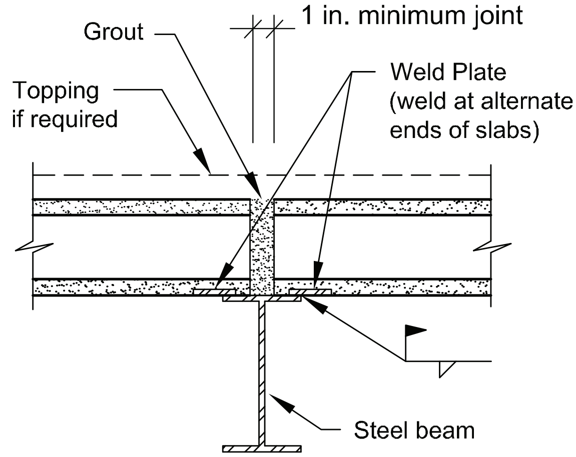


Figure 6.5.3

Design Considerations:

- Can transfer diaphragm shear
- Provides lateral brace for steel beam
- Potential torsion on steel beam should be considered
- Will develop volume change restraint forces that must be considered in design of connection

Fabrication Considerations:

- Slab manufacturing system must allow for installation of bottom weld anchors

Erection Considerations:

- Welding of slabs to beam should be done as erection proceeds to brace beam
- Spacer may be required to make weld

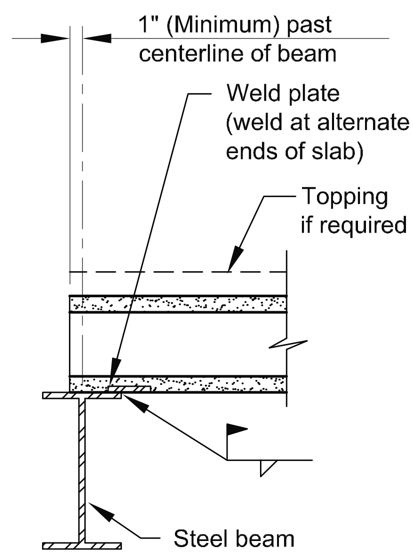


Figure 6.5.4

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.5 Typical Details with Steel Beams (Continued)

Design Considerations:

- Can transfer diaphragm shear
- Provides lateral brace for steel beam

Fabrication Considerations:

- Clean and simple

Erection Considerations:

- Welding of bars must be coordinated with slab erection for alignment
- Depending on forces to be transferred, concrete may have to be cast along edge
- Beam will not be braced until keyway grout cures

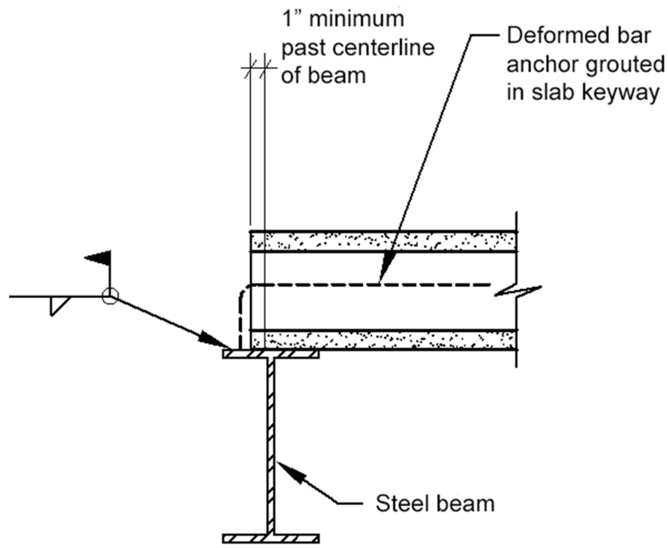


Figure 6.5.5

Design Considerations:

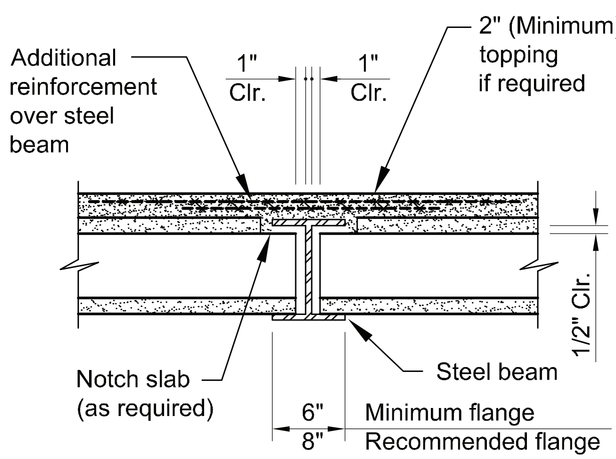
- Internal diaphragm forces can be transferred only through topping
- Provides lateral brace for steel beam
- Consider potential torsion on beam during slab erection

Fabrication Considerations:

- Beam flange width must be sufficient for minimum slab bearing
- Slab notching will require a hand operation in field or, preferably, in plant

Erection Considerations:

- Slab erection will be very difficult with this detail on both ends. Slabs must be slid into beams, possibly through access holes in flanges
- Beams will not be braced during slab erection



Note: Difficult erection if this detail occurs at both ends of slab

Figure 6.5.6

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.5 Typical Details with Steel Beams (Continued)

Design Considerations:

- Internal diaphragm forces can be transferred only through topping
- Provides lateral brace for steel beam
- Consider potential torsion on beam during slab erection

Fabrication Considerations:

- Angle legs must be sufficient for minimum slab bearing
- Beam depth must be sufficient for clearance under top flange

Erection Considerations:

- Slab erection will be very difficult with this detail on both ends. Slabs must be slid into beams, possibly through access holes in flanges
- Beams will not be braced during slab erection

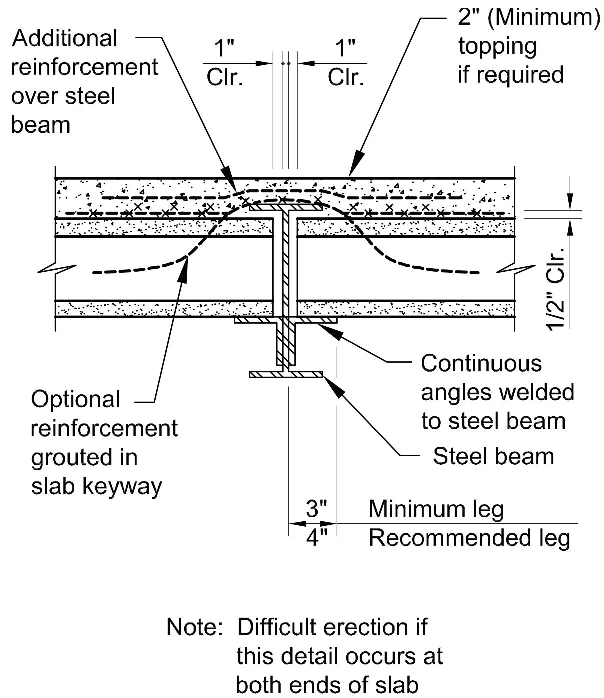


Figure 6.5.7

Design Considerations:

- Torsion design must consider erection tolerance
- Lintel must be securely anchored at span ends
- Connection to slab may be required to brace lintel

Fabrication Considerations:

- Clean and simple

Erection Considerations:

- Watch for stability of lintel prior to slab erection

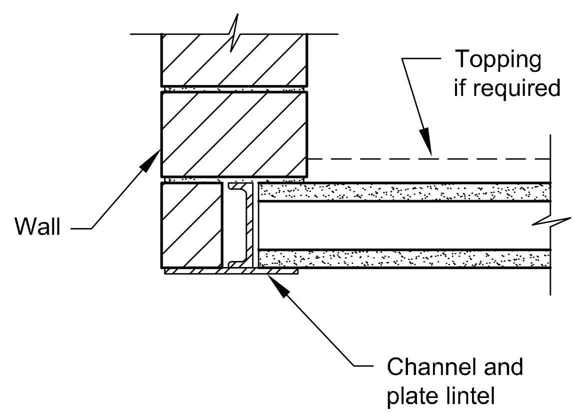


Figure 6.5.8

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.5 Typical Details with Steel Beams (Continued)

Design Considerations:

- Butt joint must be grouted to brace vertical angle legs
- Lintel must be securely anchored at span ends

Fabrication Considerations:

- Clean and simple

Erection Considerations:

- Lintel must be securely anchored prior to setting slabs

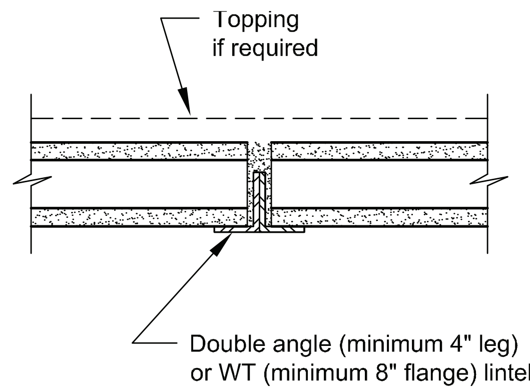


Figure 6.5.9

Design Considerations:

- Clearance must be allowed for slab camber
- Beam will not be braced until topping is cast

Fabrication Considerations:

- Camber must be monitored to stay within clearance

Erection Considerations:

- Erection may be difficult if slab support beams are also raised

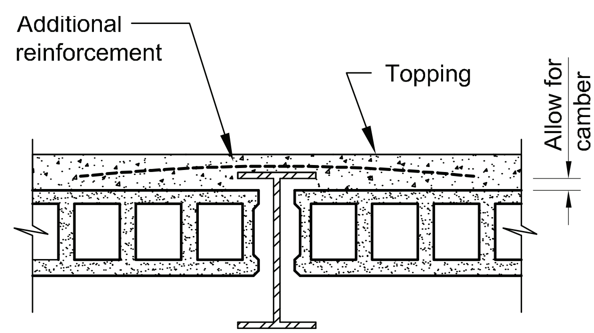


Figure 6.5.10

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.6 Typical Cantilever Details

Design Considerations:

- Should not be relied on to transfer lateral or diaphragm forces
- Wall bracing would only be accomplished by questionable friction
- Additional structural integrity ties may be required

Fabrication Considerations:

- None other than top reinforcement required for cantilever

Erection Considerations:

- Clean and simple

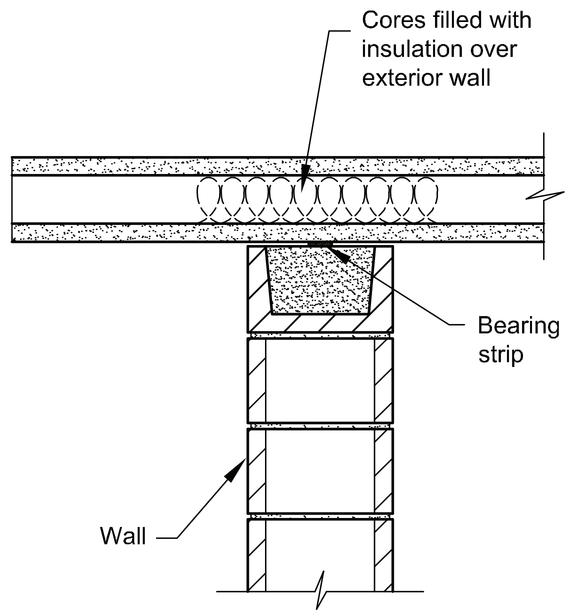


Figure 6.6.1

Design Considerations:

- Can transfer diaphragm shear
- Provides lateral brace for wall

Fabrication Considerations:

- If not field drilled, slots in keyways and aligning holes in masonry are required
- If not field drilled, alignment will be difficult

Erection Considerations:

- If not preformed, holes must be drilled through slabs into masonry
- Wall may not be braced until grout cures
- Grout placement may be difficult

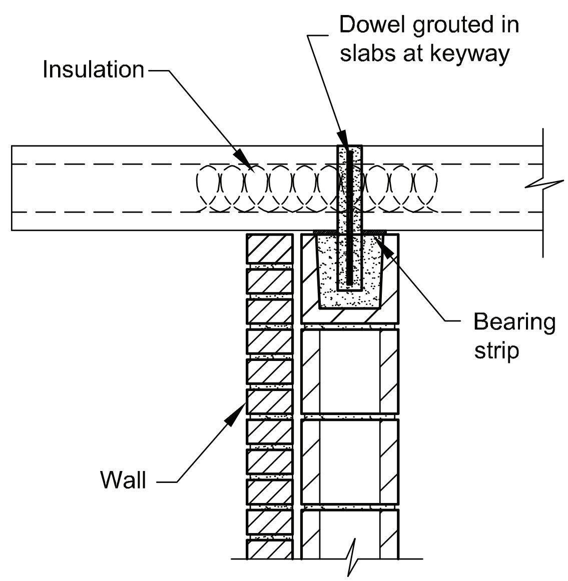


Figure 6.6.2

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.6 Typical Cantilever Details (Continued)

Design Considerations:

- This detail is not recommended because of installation difficulties which may result in an unreliable connection

Fabrication Considerations:

Erection Considerations:

- Mason will have great difficulty aligning dowels with slab joints
- Most keyway configurations will require notches for dowels
- Field bending of dowels into keyways will be very difficult

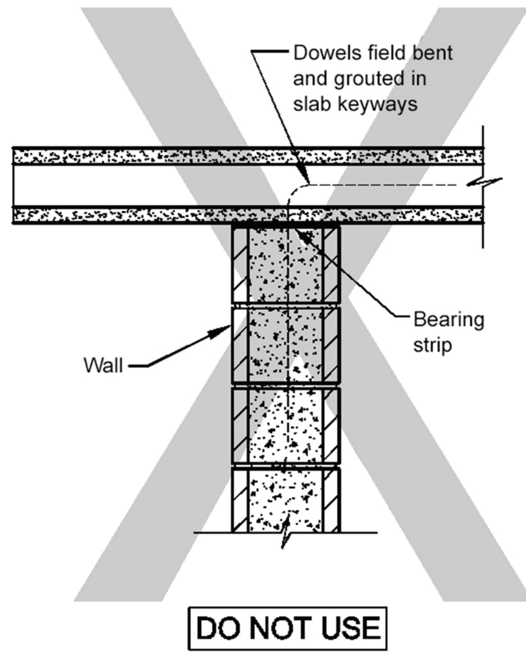


Figure 6.6.3

Design Considerations:

- Wall will not be braced by slabs
- Depending on end support conditions, wall may have to support edge slab
- No thermal break provided between interior and exterior

Fabrication Considerations:

- Depending on bearing conditions, the overhang dimension may be limited by the producer's ability to install transverse reinforcement

Erection Considerations:

- None

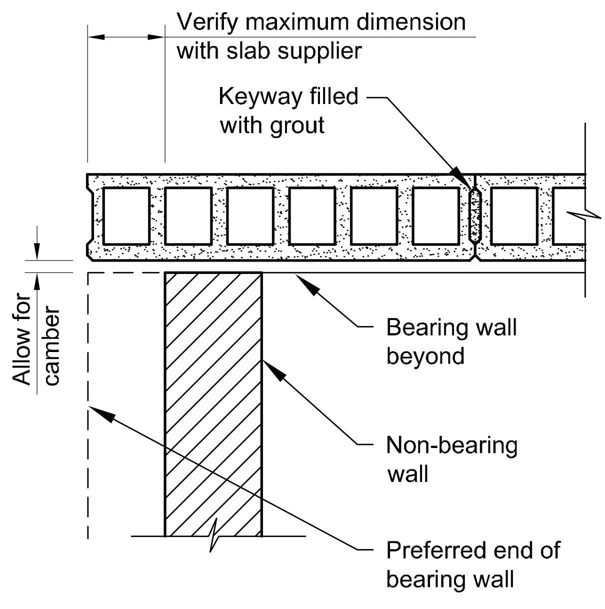


Figure 6.6.4

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.6 Typical Cantilever Details (Continued)

Design Considerations:

- Wall will not be braced by slabs
- Depending on end support conditions, wall may have to support edge slab
- No thermal break provided between interior and exterior

Fabrication Considerations:

- When transverse reinforcement cannot be installed, steel strap must serve as external reinforcement
- Anchorage of a steel strap to the slabs will depend on the producer's ability to install top weld anchors

Erection Considerations:

- Depending on end support conditions, temporary shoring may be required until steel strap is installed and keyways are grouted

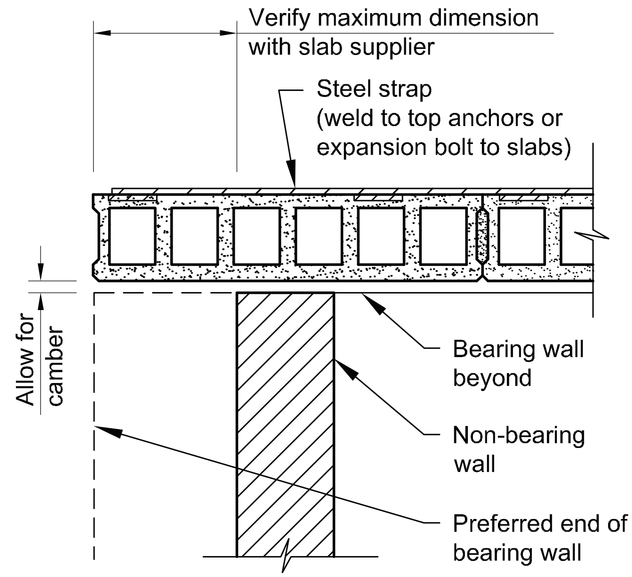


Figure 6.6.5

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.7 Miscellaneous Details

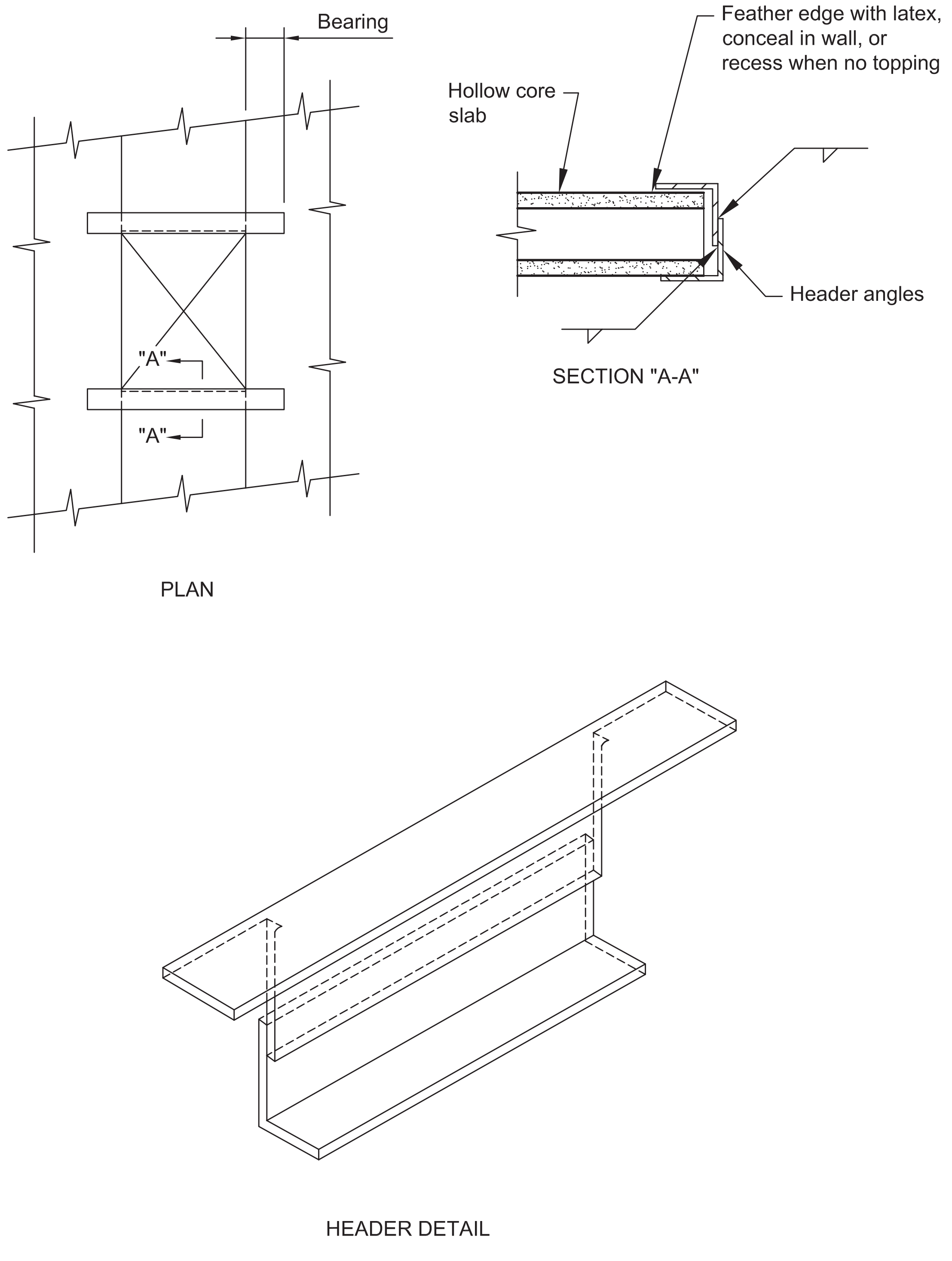


Figure 6.7.1

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.7 Miscellaneous Details (Continued)

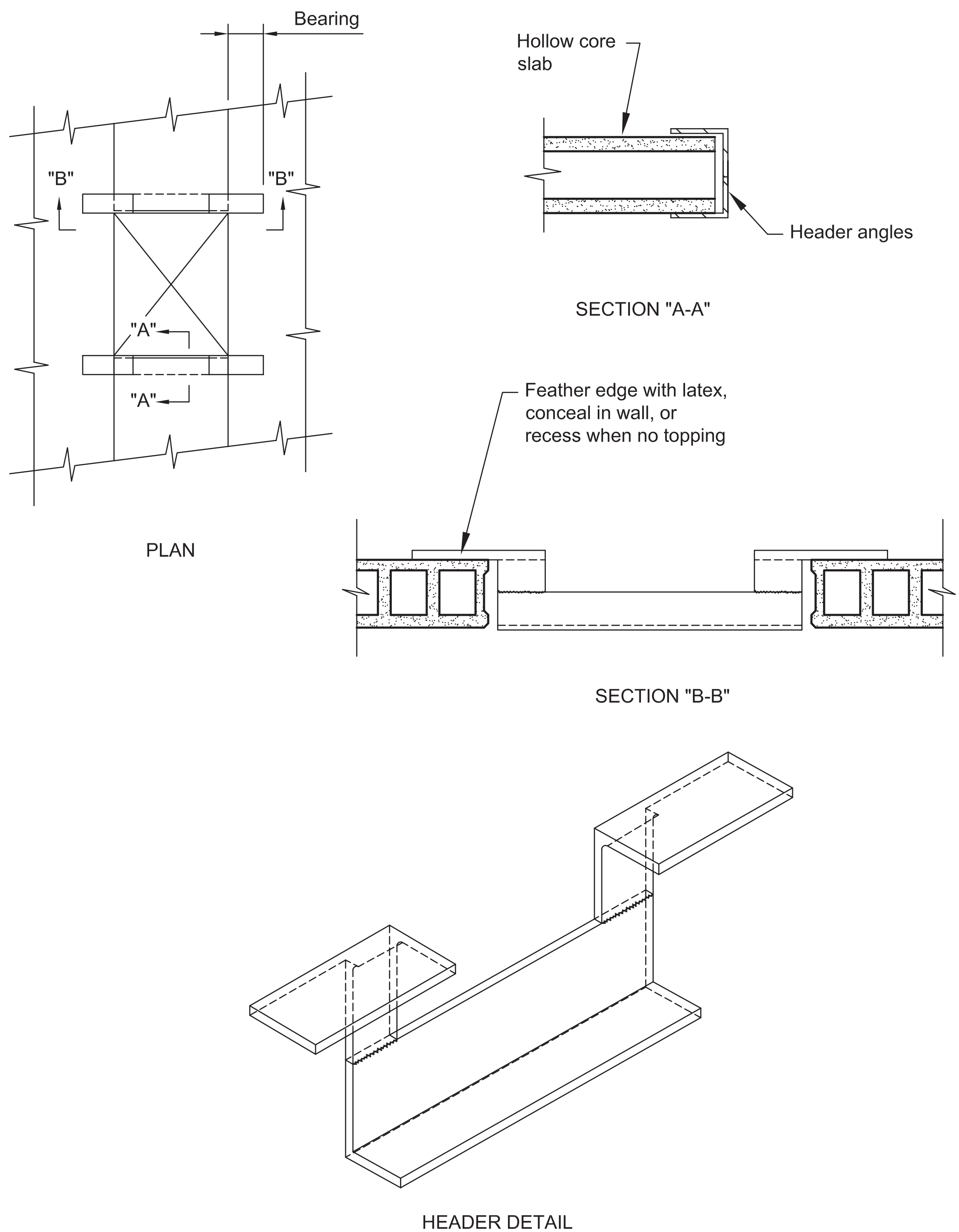


Figure 6.7.2

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.7 Miscellaneous Details (Continued)

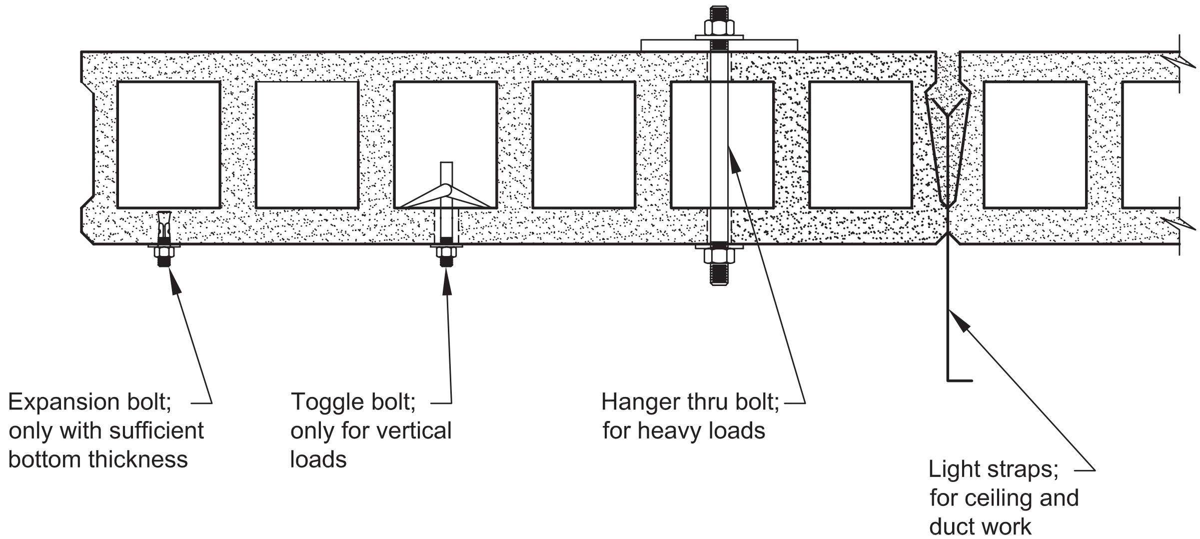


Figure 6.7.3

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.8 Typical Hollow Core Wall Panel Details

Design Considerations:

- Detail resists force normal to the face of panel only
- May need additional structural integrity connections

Fabrication Considerations:

- Clean and simple

Erection Considerations:

- Clean and simple
- Wall is not fully braced until grout cures

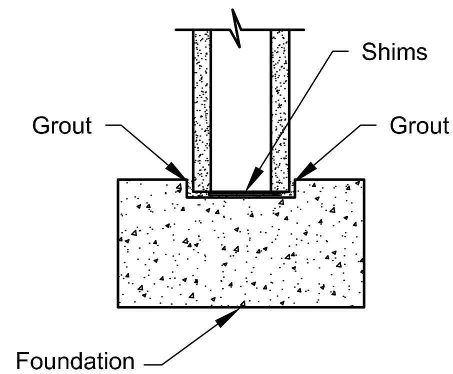


Figure 6.8.1

Design Considerations:

- Detail can transfer forces in any direction
- Angle may be continuous or intermittent
- Continuous angle may need to be designed for load applied between embedded plates in foundation

Fabrication Considerations:

- If intermittent angle, embedded plates in foundation and weld plates in wall panel must be aligned

Erection Considerations:

- Depending on forces to be transferred, angle may need to be welded to embedded plate prior to setting panel
- Continuous angle aids erection by acting as a setting guide

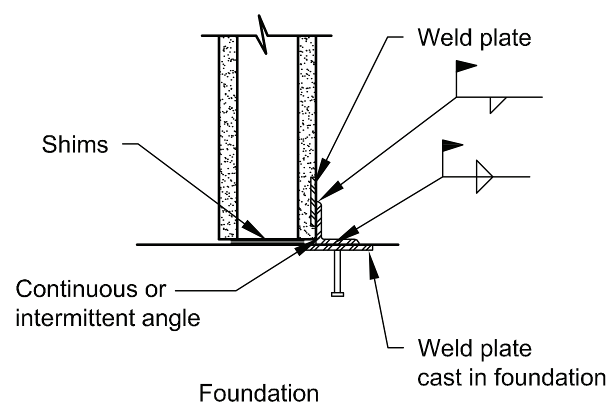


Figure 6.8.2

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.8 Typical Hollow Core Wall Panel Details (Continued)

Design Considerations:

- Forces that can be transferred depend on anchorage of bar

Fabrication Considerations:

- Access holes must be cast or drilled into panel for installation of grout

Erection Considerations:

- Casting reinforcing bar into foundation may create safety hazard for erector
- Alternate installation is to grout or epoxy reinforcing bar into field drilled holes in foundation
- Base of panel may need to be dammed to prevent grout from flowing out

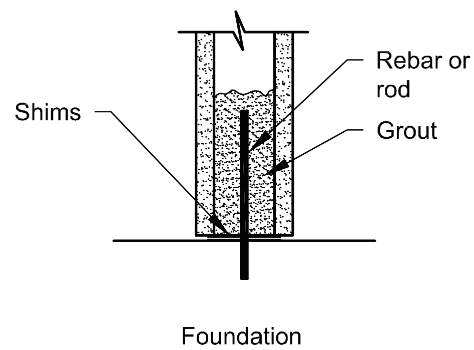


Figure 6.8.3

Design Considerations:

- Detail can transfer forces in any direction
- Angle may be continuous or intermittent
- Continuous angle may need to be designed for load applied between expansion anchors

Fabrication Considerations:

- Clean and simple

Erection Considerations:

- Use of expansion anchors eliminates need for accurate placement of embedded plates in foundation
- Allows foundation to be cast prior to finalizing wall panel layout

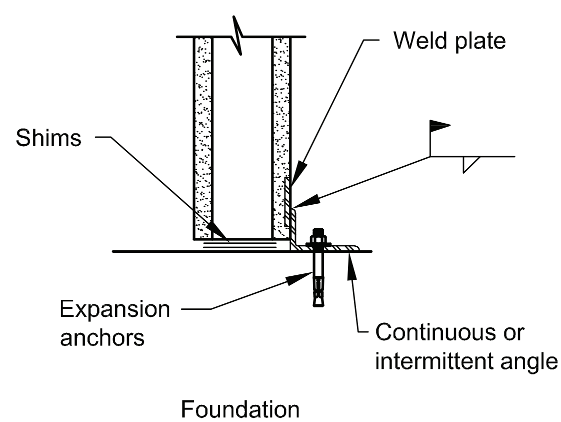


Figure 6.8.4

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.8 Typical Hollow Core Wall Panel Details (Continued)

Design Considerations:

- If bent plate is not welded to angle, detail resists force normal to the face of panel only
- If bent plate is welded to angle, detail can also resist force in the plane of the panel
- Angle may be continuous or intermittent
- Continuous angle may need to be designed for load applied between embedded plates in foundation

Fabrication Considerations:

- If intermittent angle, embedded plates in foundation and weld plates in wall panel must be aligned
- Slab manufacturing systems must allow installation of deep slotted insert

Erection Considerations:

- Depending on forces to be transferred, angle may need to be welded to embedded plate prior to setting panel
- Continuous angle aids erection by acting as a setting guide

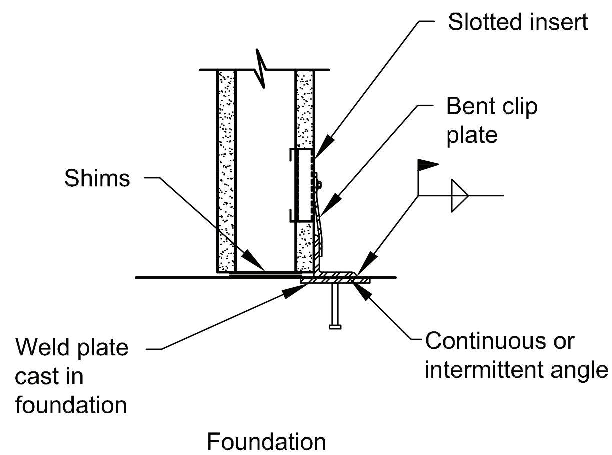


Figure 6.8.5

Design Considerations:

- Detail can transfer forces in any direction

Fabrication Considerations:

- Embedded plates in foundation and weld plates in wall panel must be aligned

Erection Considerations:

- Little tolerance for misalignment of face of foundation wall relative to inside face of wall panel

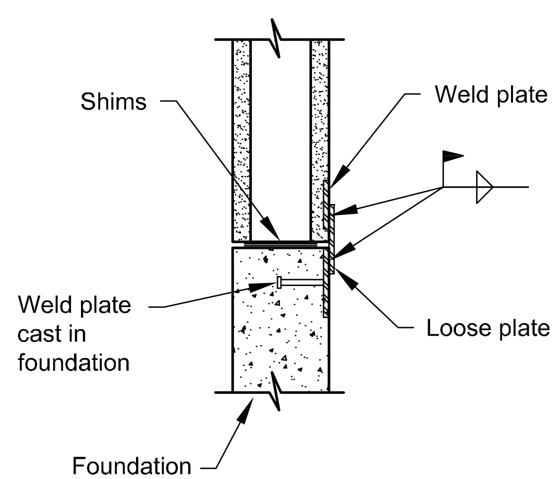


Figure 6.8.6

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.8 Typical Hollow Core Wall Panel Details (Continued)

Design Considerations:

- Detail can transfer forces in any direction
- Angle may be continuous or intermittent
- Continuous angle may need to be designed for load applied between embedded plates in foundation

Fabrication Considerations:

- If intermittent angle, embedded plates in foundation and weld plates in wall panel must be aligned

Erection Considerations:

- Horizontal leg of angle allows for tolerance in alignment of faces of foundation wall and wall panel
- Depending on forces to be transferred, angle may need to be welded to embedded plate prior to setting panel
- Continuous angle aids erection by acting as a setting guide

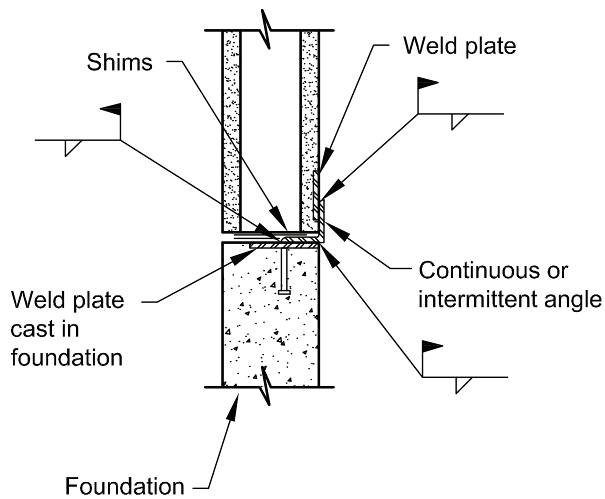


Figure 6.8.7

Design Considerations:

- Detail can transfer force normal to face of panel and in the plane of the panel
- If no axial tension exists at base of wall, detail can be used to satisfy structural integrity requirements

Fabrication Considerations:

- Slab manufacturing system must allow installation of deep slotted insert

Erection Considerations:

- Panel must be temporarily braced against backfill soil pressures
- Panel base is not braced until floor slab is cast and cured

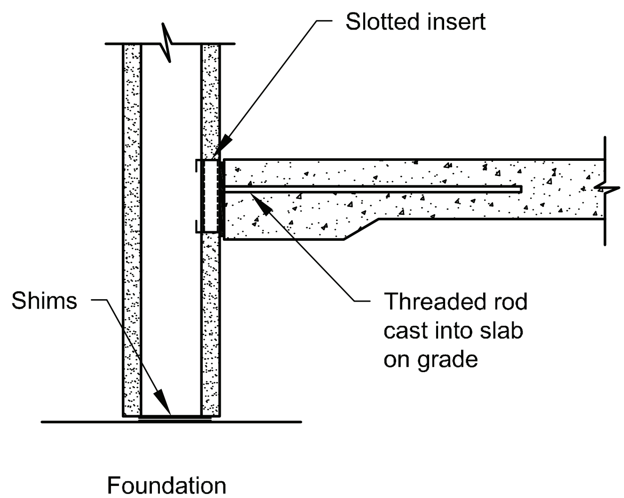


Figure 6.8.8

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.8 Typical Hollow Core Wall Panel Details (Continued)

Design Considerations:

- Applying gravity loads concentrically on the panel allows heavier gravity loads on slender panels
- With proper detailing by joist designer, detail can resist lateral forces normal to face of panel or in the plane of the panel
- Additional connections to deck may be required for large diaphragm forces

Fabrication Considerations:

- Depending on manufacturing system, bearing plate may need to be grouted into panel after casting

Erection Considerations:

- Panel must be braced until joists and decking are installed

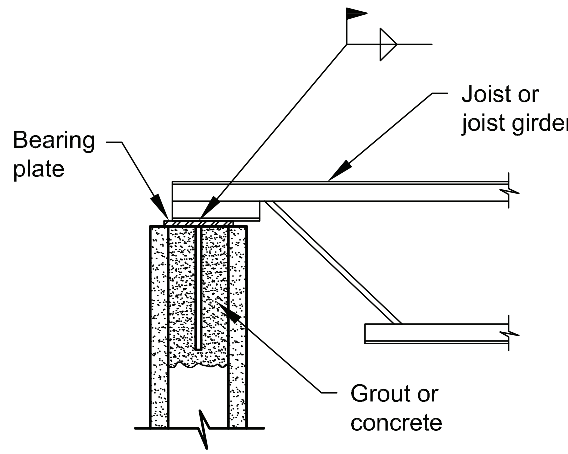


Figure 6.8.9

Design Considerations:

- Applying gravity loads concentrically on the panel allows heavier gravity loads on slender panels
- With proper detailing by joist designer, detail can resist lateral forces normal to face of panel or in the plane of the panel
- Additional connections to deck may be required for large diaphragm forces

Fabrication Considerations:

- Depending on manufacturing system, bearing plate may need to be grouted into panel after casting
- Intermittent pockets allow for a parapet

Erection Considerations:

- Panel must be braced until joists and decking are installed

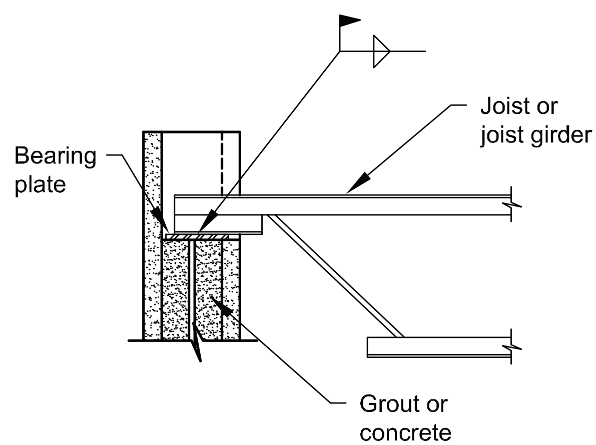


Figure 6.8.10

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.8 Typical Hollow Core Wall Panel Details (Continued)

Design Considerations:

- With proper detailing by joist designer, detail can resist lateral forces normal to face of panel or in the plane of the panel
- Additional connections to deck may be required for large diaphragm forces
- Weld plate should be designed for greater of uniform load or maximum joist reaction.
- Angle may be continuous or intermittent
- Continuous angle allows for greater flexibility of joist spacing

Fabrication Considerations:

- With some manufacturing systems, weld plates may not have capacity for heavy eccentric gravity loads

Erection Considerations:

- Panel must be braced until joists and decking are installed

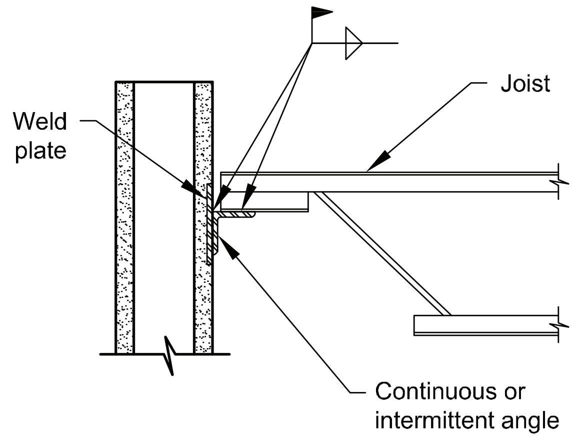


Figure 6.8.11

Design Considerations:

- With proper detailing by joist designer, detail can resist lateral forces normal to face of panel or in the plane of the panel
- Additional connections to deck may be required for large diaphragm forces
- Stiffener plate can be located above or below continuous steel beam
- Continuous beam allows for greater flexibility of joist spacing

Fabrication Considerations:

- With some manufacturing systems, weld plates may not have capacity for heavy eccentric gravity loads

Erection Considerations:

- By using temporary cantilevered columns, the joists and deck can be in place at time of panel erection, minimizing or eliminating the need for bracing

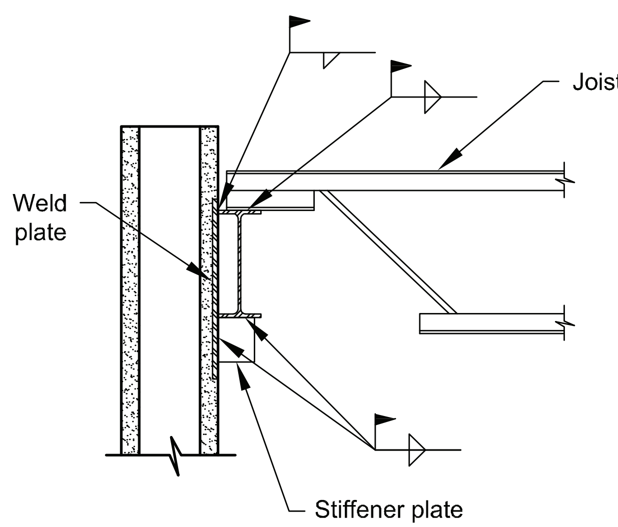


Figure 6.8.12

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.8 Typical Hollow Core Wall Panel Details (Continued)

Design Considerations:

- With proper weld design, detail can resist lateral forces normal to face of panel or in the plane of the panel
- Precast supplier may be responsible for designing and providing embedments in structure
- Slotted insert allows vertical movement of structure without generating unintended vertical forces

Fabrication Considerations:

- If intermittent plates in structure, insert in wall panel must be aligned with weld plates
- Panel manufacturing system must allow installation of deep slotted insert

Erection Considerations:

- Clean and simple

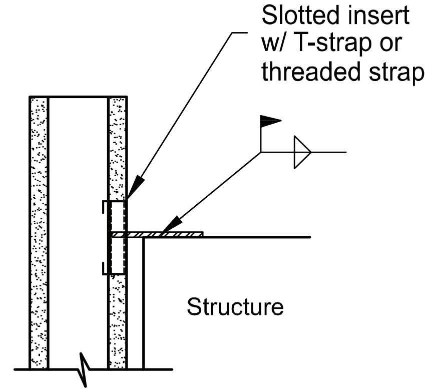


Figure 6.8.13

Design Considerations:

- With proper weld design, detail can resist lateral forces normal to face of panel or in the plane of the panel
- Precast supplier may be responsible for designing and providing embedments in structure
- Vertical movement of wall panel or structure may generate unintended vertical forces

Fabrication Considerations:

- If intermittent plates in structure, embedded plates and weld plates in wall panel must be aligned

Erection Considerations:

- Clean and simple

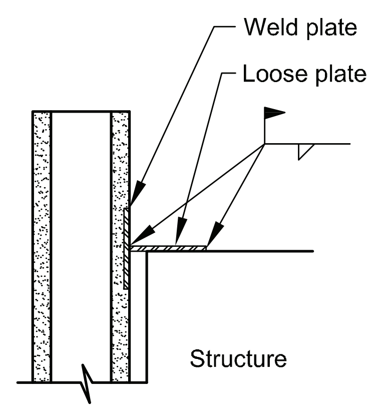


Figure 6.8.14

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.8 Typical Hollow Core Wall Panel Details (Continued)

Design Considerations:

- If bent plate is not welded to angle, detail resists force normal to the face of panel only
- If bent plate is welded to angle, detail can also resist force in the plane of the panel
- Precast supplier may be responsible for designing and providing embedments in structure

Fabrication Considerations:

- If intermittent plates in structure, embedded plates and slotted inserts in wall panel must be aligned
- Requires additional length of panel above structure in comparison to similar details
- Panel manufacturing system must allow installation of deep slotted insert

Erection Considerations:

- Weld of clip plate to angle, if required, can be completed by follow-up crew

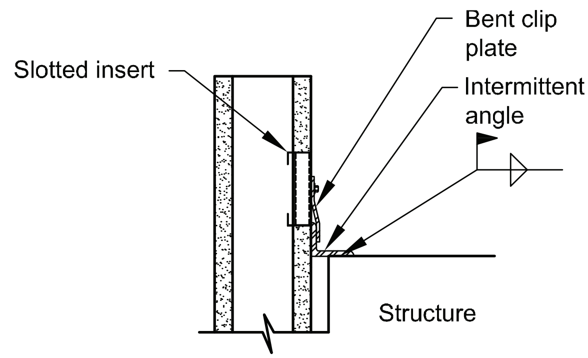


Figure 6.8.15

Design Considerations:

- With proper weld and angle design, detail can resist lateral forces normal to face of panel or in the plane of the panel
- Precast supplier may be responsible for designing and providing embedments in structure
- Vertical movement of wall panel or structure may generate unintended vertical forces

Fabrication Considerations:

- If intermittent plates in structure, embedded plates and weld plates in wall panel must be aligned

Erection Considerations:

- Clean and simple

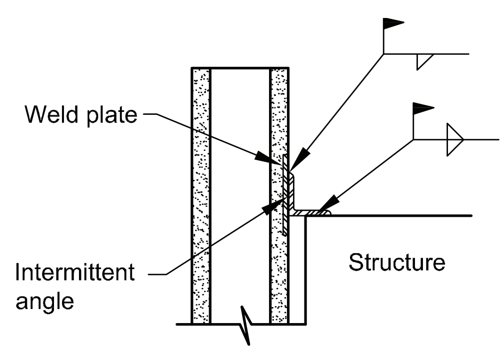


Figure 6.8.16

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.8 Typical Hollow Core Wall Panel Details (Continued)

Design Considerations:

- With proper weld and angle design, detail can resist lateral forces normal to face of panel or in the plane of the panel
- Angles should be intermittent
- Precast supplier may be responsible for designing and providing embedments in structure
- With threaded insert, angle should be slotted vertically for erection tolerance and to allow for vertical movement of structure

Fabrication Considerations:

- If intermittent plates in structure, inserts in wall panel must be aligned

Erection Considerations:

- If only forces normal to panel are resisted, shim space may be provided between panel and angle allowing for future adjustment

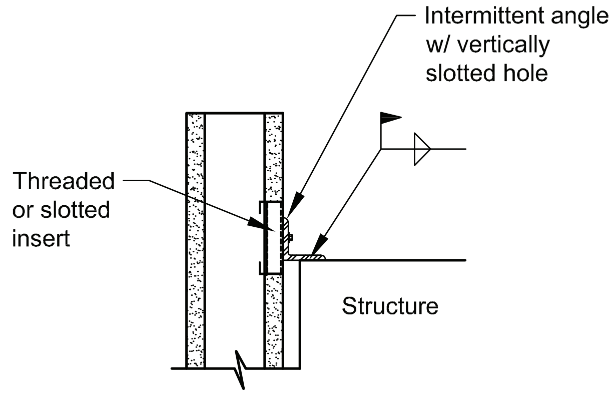


Figure 6.8.17

Design Considerations:

- Used primarily to resist vertical shear between panels
- Use in multiple, consecutive panels may generate volume change forces that should be considered in design

Fabrication Considerations:

- Capacity of insert at edge of panel may be limited

Erection Considerations:

- Visible loose plate may be objectionable in some usages

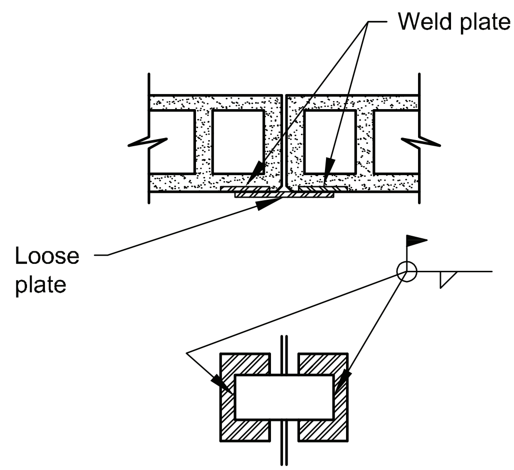


Figure 6.8.18

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.8 Typical Hollow Core Wall Panel Details (Continued)

Design Considerations:

- Used primarily to resist vertical shear between panels
- Use in multiple, consecutive panels may generate volume change forces that should be considered in design

Fabrication Considerations:

- Some manufacturing systems will not allow installation of a corner angle

Erection Considerations:

- Rod may be difficult for welder to hold in position

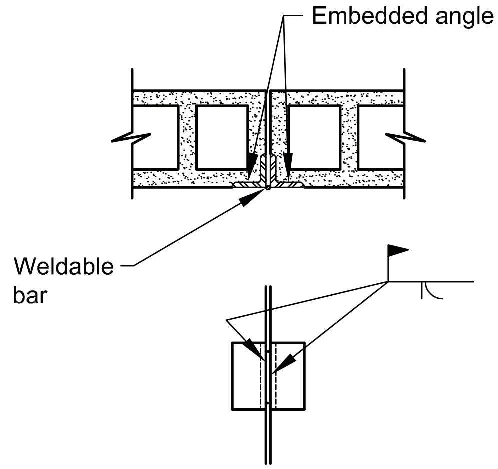


Figure 6.8.19

Design Considerations:

- Used primarily to resist vertical shear between panels
- Use in multiple, consecutive panels may generate volume change forces that should be considered in design

Fabrication Considerations:

- Some manufacturing systems will not allow installation of an edge plate

Erection Considerations:

- Loose plate may be difficult for welder to hold in position
- Erector may need loose plates of varying thickness to accommodate variations in joint width
- Hidden detail is preferable in some usages

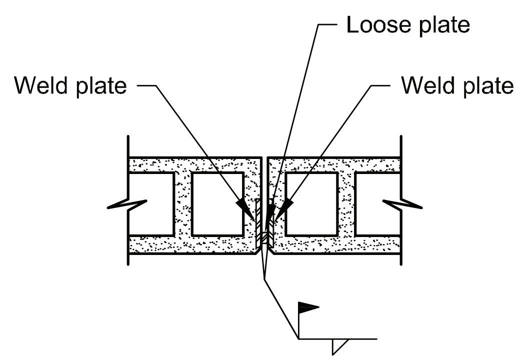


Figure 6.8.20

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

6.8 Typical Hollow Core Wall Panel Details (Continued)

Design Considerations:

- Used primarily to control panel bowing at corners

Fabrication Considerations:

- May be difficult to match finish on edge of wall panel with finish on face

Erection Considerations:

- Difficult to install if corner column is present

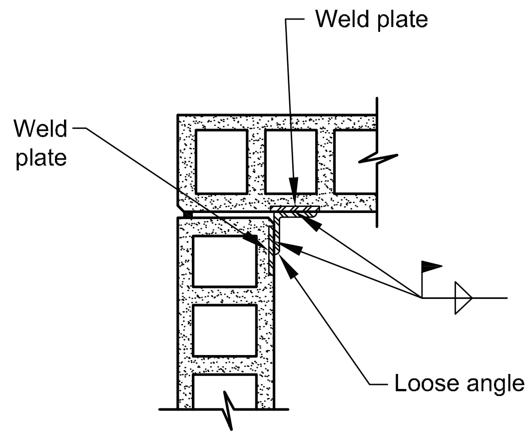


Figure 6.8.21

Design Considerations:

- Used primarily to control panel bowing at corners

Fabrication Considerations:

- Mitered corner eliminates concern about matching finishes
- Cutting or forming mitered edge may be difficult for some manufacturing systems

Erection Considerations:

- Difficult to install if corner column is present

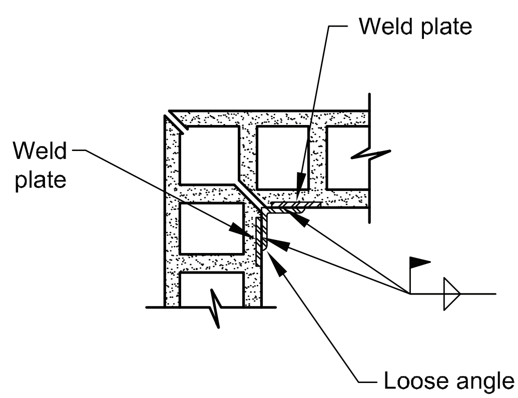


Figure 6.8.22

Other connection details perform functions similar to those shown. Consult the local PCI producer for information on relative economy and design capabilities.

Chapter 7

FIRE RESISTANCE OF ASSEMBLIES MADE WITH HOLLOW CORE SLABS

7.1 Introduction

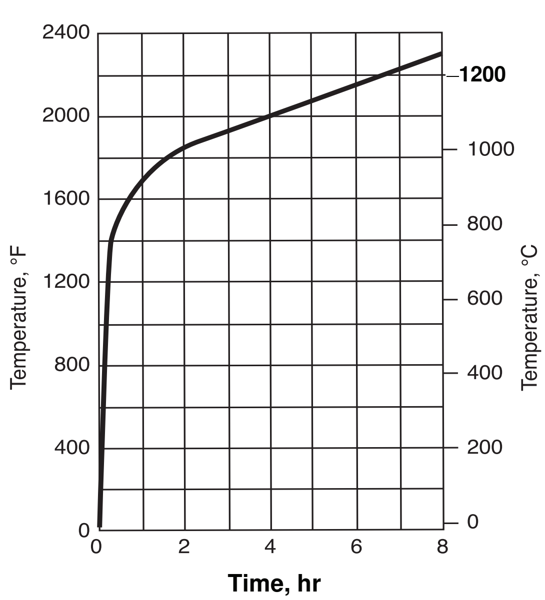
To provide for public safety and limit property damage, building codes require that resistance to fire be considered in the design of buildings. The fire rating required depends on the size of the building, its occupancy, the proximity of other structures, and the presence of other fire-detecting or extinguishing equipment.

The inherent insulating capacity of concrete makes hollow core slabs an efficient component in fire-rated assemblies. With proper design and detailing, fire-resistance ratings up to four hours can be achieved for hollow core floors, roofs, and walls.

7.2 Fire-Resistance Criteria

A time-temperature relationship (Fig. 7.2.1) as defined by ASTM E119[43] provides a standard fire exposure for measuring the fire resistance of a component or assembly. While a fire of this intensity and duration bears little resemblance to most actual fires, it does provide a benchmark for comparing the fire performance of similar products and materials. The fire resistance of an assembly is defined as the length of exposure to this standard fire before any of the "end point" criteria are reached.

Figure 7.2.1 Standard time-temperature relationship (ASTM E119)



The conditions of acceptance or end point criteria are also defined by ASTM E119. For typical floor and roof slabs, there are three potential end points.

- Structural endpoint – the assembly must support the applied load for the length of the rating exposure.
- Flame passage end point – holes or cracks may not develop that allow the escape of gases that are hot enough to ignite cotton waste.
- Heat transmission end point – the average temperature of the unexposed surface may not increase more than 250 °F or may not at any one point increase more than 325 °F.

In addition to the three conditions listed previously, wall panels must withstand a hose stream test (simulating a firefighter's hose).

Because the concept of restrained versus unrestrained fire ratings was not introduced until 1970, long after many of the original fire tests were conducted, an additional criterion for converting restrained fire tests to unrestrained ratings is required. If a slab was tested in a restrained condition, the unrestrained fire test endpoint occurs when the temperature of the tension steel reaches 800 °F for prestressing steel and 1100 °F for mild steel reinforcement. It should be noted that there are no limitations on the steel temperatures for restrained slab assemblies.

7.3 Determination of Fire Ratings

Historically, fire ratings were established by actual tests of the structural assembly. For a floor system, the floor was built over a test furnace, the maximum permissible gravity load was applied, and fuel was burned to match the standard ASTM E119 time-temperature curve. The fire rating was the actual time measured before reaching one of the end point criteria.

Through the years, more than 30 standard fire tests have been conducted on hollow core floor

Table 7.3.1

Minimum Protection of Structural Parts Based on Time Periods

Structural parts to be protected	**Item number**	**Insulating material used**	**Minimum thickness of insulating material for the following fire-resistance period, in.**			
			4 hour	**3 hour**	**2 hour**	**1 hour**
Bonded, pretensioned reinforcement in prestressed concrete	3-1.1	Solid slabs — carbonate, lightweight, and sand-lightweight aggregate concrete	—	2	1.5	1
		Solid slabs — siliceous aggregate concrete	—	2.4	1.8	1.3

Rated Fire-Resistance Periods for Various Walls

Material	**Item number**	**Construction**	**Minimum finished thickness face-to-face**			
			4 hour	3 hour	2 hour	1 hour
Solid concrete	4-1.1	Siliceous aggregate concrete	7.0	6.2	5.0	3.5
		Carbonate aggregate concrete	6.6	5.7	4.6	3.2
		Sand-lightweight concrete	5.4	4.6	3.8	2.7
		Lightweight concrete	5.1	4.4	3.6	2.5

Minimum Protection for Floor and Roof Systems

Floor or roof construction	**Item number**	**Ceiling construction**	**Thickness of floor or roof slab, in.**			
			4 hour	3 hour	2 hour	1 hour
Siliceous aggregate concrete	1-1.1	Slab (no ceiling required)	7.0	6.2	5.0	3.5
Carbonate aggregate concrete	2-1.1		6.6	5.7	4.6	3.2
Sand-lightweight concrete	3-1.1		5.4	4.6	3.8	2.7
Lightweight concrete	4-1.1		5.1	4.4	3.6	2.5

assemblies. The Underwriters Laboratories Inc. (UL) Fire Resistance Directory[44] includes more than 50 design numbers for hollow core slabs that qualify for ratings of 1, 2, 3, or 4 hours. The 2012 International Building Code[35] (IBC) allows the building official to accept a UL-listed design number in lieu of the original test report.

As an alternative to UL ratings, the 2012 IBC also allows prescriptive and empirical designs and analytical methods to determine fire ratings. These alternate methods are all based on the ASTM E119 fire criteria, but make use of existing test data and basic engineering principles to determine the fire resistance of the concrete members.

Section 721 of the 2012 IBC contains prescriptive details for fire-rated building elements. Those requirements that apply to hollow core slabs are reproduced in Table 7.3.1. Although the section refers to solid slabs and walls, the criteria are equally valid for hollow core slabs provided equivalent thickness is substituted for the actual thickness.

IBC Section 722 contains a series of tables that can be used to determine the fire-resistance rating of hollow core slabs. For each fire-endurance rating, the equivalent thickness and required strand cover are designated. If these provisions are met, the heat transmission and structural end point criteria are considered to be satisfied and neither a UL rating nor rational design calculation is required. The relevant tables are summarized in Tables 7.3.2 and 7.3.3.

In addition, the 2012 IBC recognizes the fire-resistance ratings calculated in accordance with the procedures of the PCI manual, *Design for Fire Resistance of Precast, Prestressed Concrete*[45]. Commonly referred to as the rational design method, the method used in that manual uses information gathered from previous fire tests to determine the strand temperature and the resulting reduced steel strength at the required fire endurance. Basic structural engineering principles are then applied to compute the reduced load-carrying capacity of the member.

The National Building Code of Canada[46] requires that fire-resistance ratings be determined either on the basis of Appendix D of the code, "Fire Performance Ratings," or on results of tests conducted in accordance with CAN/ULC-S101, *Standard Methods of Fire Endurance Tests of Building Construction on Materials*[47]. While the general principals set forth in this manual are fully valid in that they are based on material properties and structural engineering procedures, manual users are cautioned that in Canada, fire-resistance ratings should be determined strictly in accordance with applicable building code requirements.

7.4 Restrained versus Unrestrained

Many of the early fire tests were conducted in furnaces in which the specimens were locked into the test fixtures such that they were restrained against rotation and thermal expansion. It was recognized early on that testing in this manner yielded fire-resistance durations significantly longer than those achieved with supports that allowed the member to freely expand and rotate. However, it wasn't until 1970 that a dual system of ratings for restrained and unrestrained members was developed.

Having officially recognized the beneficial effects of restrained construction, the ASTM E5 Committee developed a guide for determining if a component or assembly was restrained or not. Table X3.1 of ASTM E119 is reproduced here as Table 7.4.1.

Table 7.3.2

Minimum Equivalent Thickness of Precast Concrete Walls and Slabs, in.

Concrete Aggregate type	Fire-Resistance Rating, hr				
	1	1.5	2	3	4
Siliceous	3.5	4.3	5.0	6.2	7.0
Carbonate	3.2	4.0	4.6	5.7	6.6
Sand-lightweight	2.7	3.3	3.8	4.6	5.4
Lightweight	2.5	3.1	3.6	4.4	5.1

Table 7.3.3

Cover Thickness for Prestressed Concrete Floor or Roof Slabs, in.

Concrete Aggregate type	Fire-Resistance Rating, hr									
	Restrained					Unrestrained				
	1	1.5	2	3	4	1	1.5	2	3	4
Siliceous	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{3}{4}$	$1\frac{1}{8}$	$1\frac{1}{2}$	$1\frac{3}{4}$	$2\frac{3}{8}$	$2\frac{3}{4}$
Carbonate	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{3}{4}$	1	$1\frac{3}{8}$	$1\frac{5}{8}$	$2\frac{1}{8}$	$2\frac{1}{4}$
Sand-lightweight or Lightweight	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{3}{4}$	1	$1\frac{3}{8}$	$1\frac{1}{2}$	2	$2\frac{1}{4}$

Table 7.4.1 Construction Classification, Restrained and Unrestrained

I. Wall bearing:	
Single span and simply-supported end spans of multiple bays:*	
(1) Open-web steel joists or steel beams, supporting concrete slab, precast units, or metal decking	unrestrained
(2) Concrete slabs, precast units, or metal decking	unrestrained
Interior spans of multiple bays:	
(1) Open-web steel joists, steel beams, or metal decking, supporting continuous concrete slab	restrained
(2) Open-web steel joists or steel beams, supporting precast units or metal decking	unrestrained
(3) Cast-in-place concrete slab systems	restrained
(4) Precast concrete where the potential thermal expansion is resisted by adjacent construction†	restrained
II. Steel framing:	
(1) Steel beams welded, riveted, or bolted to the framing members	restrained
(2) All types of cast-in-place floor and roof systems (such as beam-and-slabs, flat slabs, pan joists, and waffle slabs) where the floor or roof system is secured to the framing members	restrained
(3) All types of prefabricated floor and roof systems where the structural members are secured to the framing members and the potential thermal expansion of the floor or roof system is resisted by the framing system or the adjoining floor or roof construction†	restrained
III. Concrete framing:	
(1) Beams securely fastened to the framing members	restrained
(2) All types of cast-in-place floor or roof systems (such as beams-and-slabs, flat slabs, pan joists, and waffle slabs) where the floor system is cast with the framing members	restrained
(3) Interior and exterior spans of precast concrete systems with cast-in-place joints resulting in restraint equivalent to that which would exist in condition III (1)	restrained
(4) All types of prefabricated floor or roof systems where the structural members are secured to such systems and the potential thermal expansion of the floor or roof systems is resisted by the framing system or the adjoining floor or roof construction†	restrained
IV. Wood construction: All types	unrestrained

* Floor and roof systems can be considered restrained when they are tied into walls with or without tie beams, the walls being designed and detailed to resist thermal thrust from the floor or roof system

† For example, resistance to potential thermal expansion is considered to be achieved when:

(1) Continuous structural topping is used

(2) The space between the ends of precast concrete units or between the ends of units and the vertical face of supports is filled with concrete or mortar

(3) The space between the ends of precast concrete units and the vertical faces of supports, or between the ends of solid or hollow core slab units does not exceed 0.25% of the length for normal weight concrete members or 0.10% of the length for structural lightweight concrete members

For purposes of this analysis method, a floor or roof assembly is considered restrained when the adjacent structure is capable of resisting substantial thermal expansion. Construction that does not satisfy this definition is assumed to be free to rotate and expand and should be considered unrestrained.

Even with these guidelines, restraint remains one of the most controversial topics in precast concrete design. Appendix X3 of ASTM E119 recognizes the difficulty in determining whether a system is restrained or not with paragraph X3.5:

> This definition requires the exercise of engineering judgment to determine what constitutes restraint to "substantial thermal expansion." Restraint may be provided by the lateral stiffness of supports for floor or roof assemblies and intermediate beams forming part of the assembly. In order to develop restraint, the connections must adequately transfer thermal thrusts to such supports. The rigidity of adjoining panels or structures should be considered in assessing the capability of a structure to resist thermal expansion.

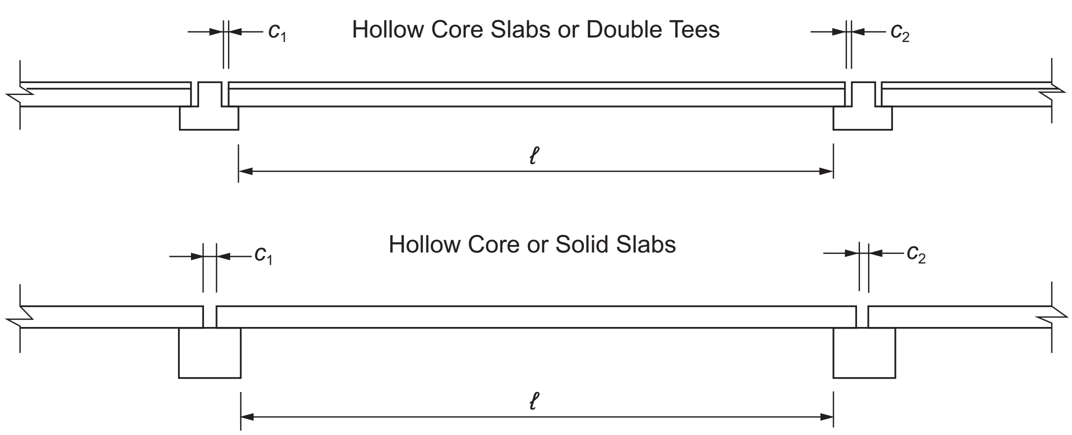
Section 703.2.3 of the 2012 IBC goes so far as to state, "Fire-resistance-rated assemblies tested under ASTM E119 or UL263[48] shall not be considered to be restrained unless evidence satisfactory to the building official is furnished by the registered design professional showing that the construction qualifies for a restrained classification in accordance with ASTM E119 or UL 263."

Often, the precast concrete engineer must determine (and perhaps to convince a building official) whether the floor or roof may be considered restrained using calculations, standard engineering practice, and historical precedent.

Typically, the interior bays of multibay floors and roofs can be considered restrained provided one of the conditions listed in the second footnote of Table 7.4.1 is satisfied. Note that in addition to a continuous structural topping or filling the butt joints with grout, adequate restraint can be developed if the end gap is kept sufficiently small, as illustrated in Fig. 7.4.1.

Single spans or end bays of wall-bearing structures may be considered restrained if the slab is adequately connected to walls that are designed and detailed to resist the thermal thrust from the floor or roof system. Similarly, end bays of steel- or concrete-framed buildings may be considered restrained if thermal expansion of the deck is resisted by the framing system or by the adjoining floor or roof construction.

Figure 7.4.1 Typical examples of restrained floors and roofs of precast construction



To be considered as restrained:

$c_1 + c_2 < 0.0025\ell$ for normal weight concrete

$c_1 + c_2 < 0.0010\ell$ for lightweight concrete

Example: Determine maximum value of $c_1 + c_2$ for normal weight hollow core span of 30 ft

Solution: $c_1 + c_2 = 0.0025\ (30 \times 12) = 0.90$ in.

It is in defining and providing this restraint to thermal expansion that the difficulties lie. In addition to direct axial thrust provided by the supporting structure, alternate load paths for resistance to thermal expansion are often inherently present in a hollow core deck. If the fire is localized, the grouted keyways will provide substantial longitudinal shear capacity for the cooler areas of the deck to restrain the fire-affected areas. Perimeter reinforcement, such as that provided to satisfy structural integrity requirements, will act as shear-friction reinforcement to further tie the heated slab to adjacent cooler slabs.

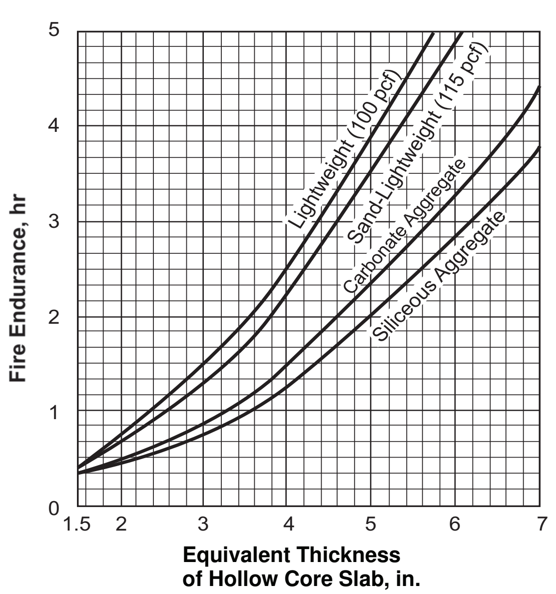
Past performance of precast concrete structures subject to fire would indicate that typical design practices and the inclusion of structural integrity reinforcement provide fire resistance equivalent to that provided by a restrained system. However, if the designated restraint mechanism does not fully comply with Table 7.4.1, it is recommended that the governing building official be consulted.

7.5 Heat Transmission through Floors or Roofs

The standard fire test method, ASTM E119, limits the average temperature rise of the unexposed surface (the surface of floor or roof not exposed to fire) to 250 °F during a fire test. This criterion is often called the heat transmission end point.

For solid concrete slabs, the temperature rise of the unexposed surfaces depends mainly on the slab thickness and aggregate type. Figure 7.5.1 shows the relationship between slab thickness and fire endurance as determined by the heat transmission end point criterion.

Figure 7.5.1 Fire endurance (heat transmission) of hollow core slabs



7.5.1 Equivalent Thickness

The information in Fig. 7.5.1 is applicable to hollow core slabs by entering the graph with the "equivalent thickness" of the slab instead of the thickness. Equivalent thickness can be calculated by dividing the net area of the cross section of a hollow core slab by its width.

In Fig. 7.5.1, concrete aggregates are designated as lightweight, sand-lightweight, carbonate, or siliceous. Lightweight aggregates include expanded clay, shale, slate, and slag that produce concretes having densities between about 95 lb/ft^3 and 105 lb/ft^3 without sand replacement. Lightweight concretes in which sand is used as part or all of the fine aggregate and weigh less than about 120 lb/ft^3 are designated as sand-lightweight. For normalweight concrete, the type of coarse aggregate influences the fire endurance; the type of fine aggregate has only a minor effect. Carbonate aggregates include limestone, dolomite, and limerock (those consisting mainly of calcium or magnesium carbonate). Siliceous aggregates include quartzite, granite, basalt, and most hard rocks other than limestone or dolomite.

7.5.2 Toppings, Undercoatings, or Roof Insulation

All 8-in.-deep hollow core slabs that are currently manufactured in North America qualify for at least a one-hour fire-endurance rating as determined by heat transmission, and some qualify for two hours or more. The addition of toppings, undercoatings, fire-resistive ceilings, roof insulation, or filling the cores with dry aggregates will increase the heat transmission fire endurance. Figure 7.5.2.1 shows the thickness of spray-applied undercoating required for heat transmission fire endurances of 2, 3, and 4 hours. Figure 7.5.2.2 shows the thickness of sand-lightweight concrete, insulating concrete, and high-strength gypsum concrete overlays required for 2, 3, and 4 hours. Figure 7.5.2.3 shows data for 2- and 3-hr roofs with mineral board or glass-fiber board insulation with 3-ply built-up roofing. Data shown in Fig. 7.5.2.1, 7.5.2.2, and 7.5.2.3 apply directly to hollow core slabs made with siliceous aggregates and are conservative for slabs made with carbonate or lightweight aggregates.

Example 7.5.1 Equivalent Thickness

Determine the thickness of topping required to provide 3-hr fire endurance (heat transmission) for the generic hollow core slab shown in Fig. 1.6.1. Both the slab and the topping are made with carbonate aggregate concrete.

Solution:
Equivalent thickness

$$t_{eq} = \frac{\text{Area}}{\text{width}}$$

$$= \frac{154}{36} = 4.28 \text{ in.}$$

From Fig. 7.5.1, the thickness of carbonate aggregate concrete required for 3 hours is 5.75 in. Thus, the thickness of topping needed is:

$$5.75 - 4.28 = 1.47 \text{ in.}$$

Figure 7.5.2.1 Hollow core slabs undercoated with spray applied materials (Heat transmission fire endurance)

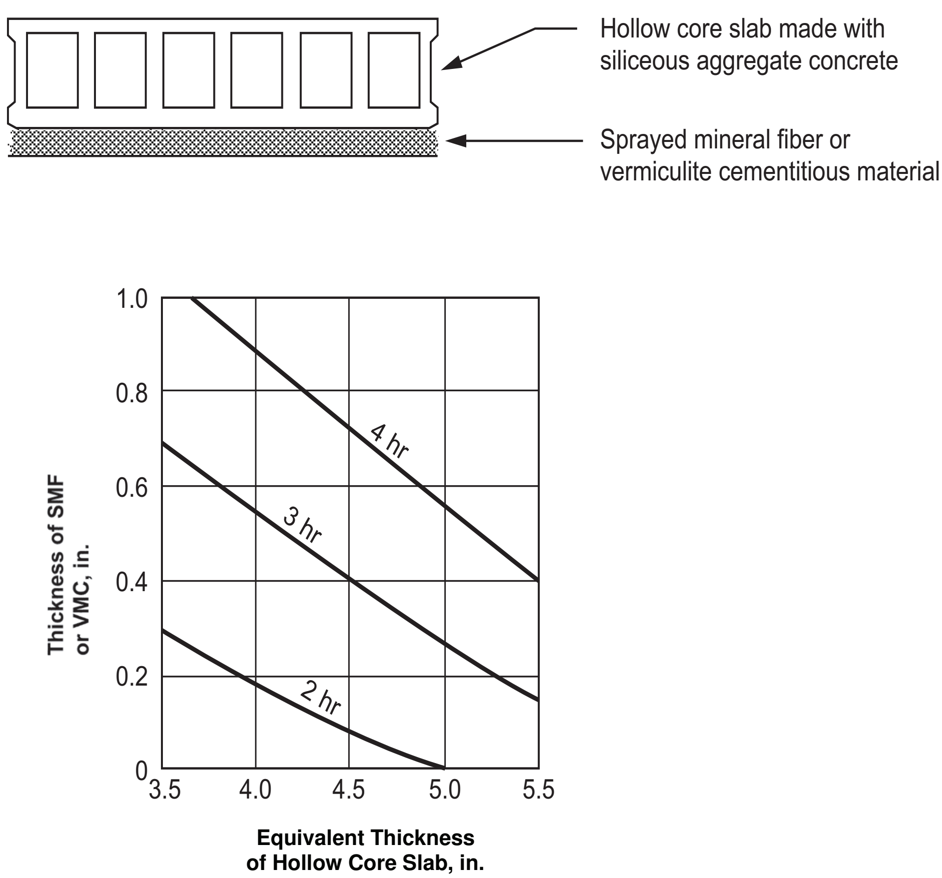


Figure 7.5.2.2 Floors with overlays of sand-lightweight concrete (120 lb/ft^3 maximum), insulating concrete (35 lb/ft^3 maximum), and high strength gypsum concrete

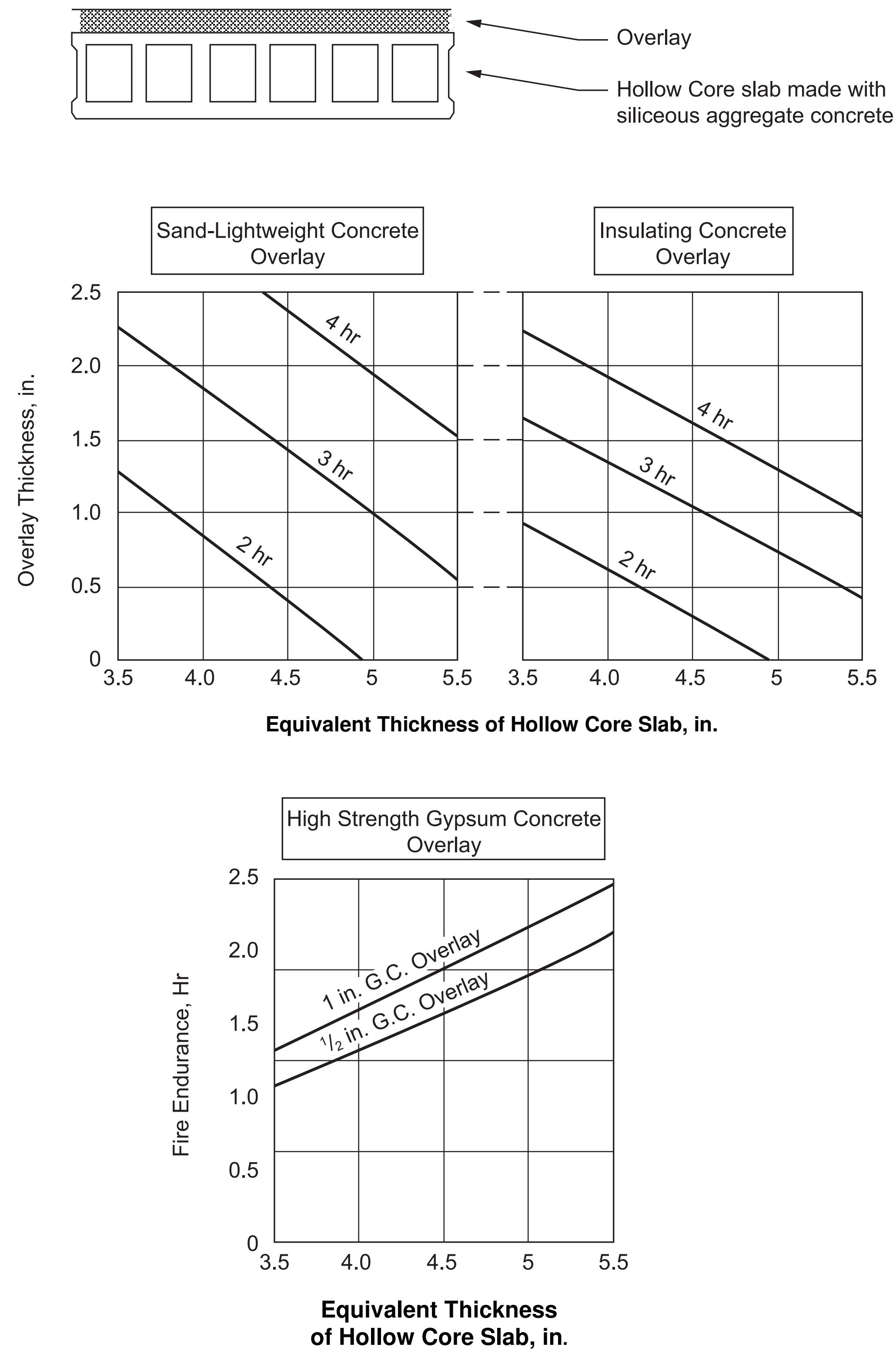
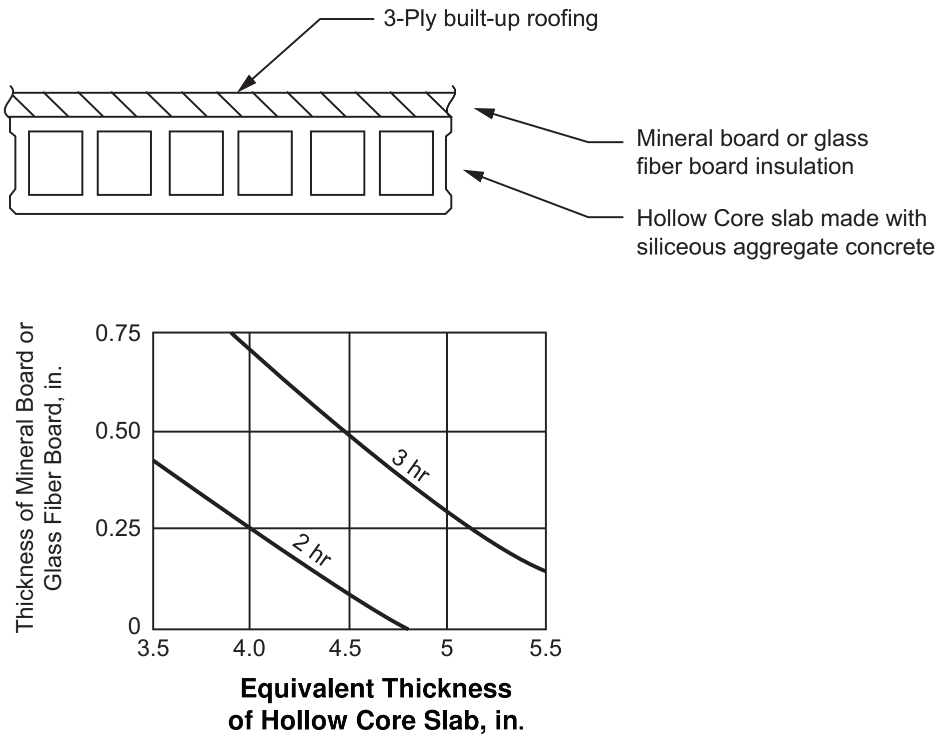


Figure 7.5.2.3 Roofs with insulation board and 3-ply built-up roofing (Heat transmission fire endurance)



Example 7.5.2

Determine if a hollow core slab roof will qualify for a 2-hr fire endurance (heat transmission) if the slabs are made with carbonate aggregate concrete, have an equivalent thickness of 4.28 in., and the roof insulation consists of a layer of $^3/_4$-in.-thick mineral board. The roofing is a standard 3-ply built-up roof.

Solution:

From Fig. 7.5.2.3, it can be seen that with an equivalent thickness of 4.28 in., a layer of mineral board 0.16 in. thick with 3-ply roofing qualifies for a 2-hr heat-transmission rating when made with siliceous aggregates. Because this curve is conservative when used with carbonate aggregates, the roof assembly will qualify for a fire endurance significantly longer than 2 hours.

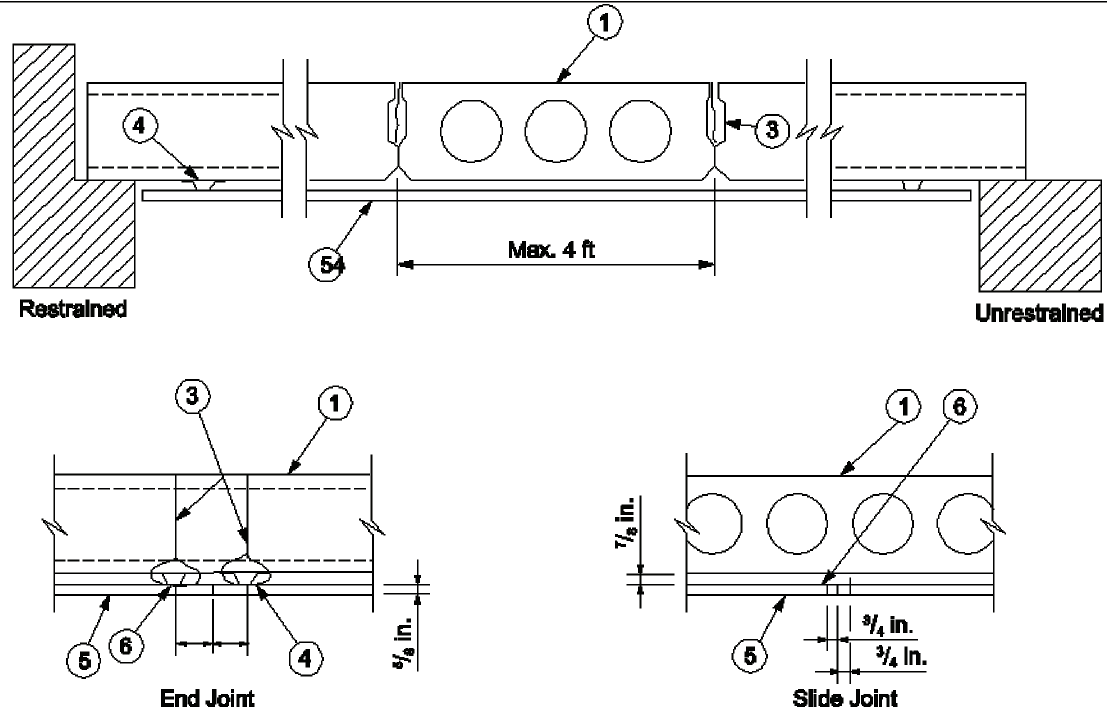
7.5.3 Ceilings

Gypsum wallboard used in a ceiling increases the fire endurance of the assembly. Few fire tests have been conducted using concrete floors with gypsum wallboard ceilings, and no such tests have been conducted using hollow core slabs. To be effective, gypsum wallboard must remain in place throughout most of the fire-endurance period. Because most hollow core slabs by themselves have heat-transmission fire endurances of 1 hr to 2 hr and longer, the wallboard must remain in place during fire exposure for long periods of time. For a fire endurance of 3 hr, a layer of $^5/_8$ in., type X gypsum wallboard can be used. The wallboard should be installed as shown in Fig. 7.5.3.1.

7.6 Structural Fire Endurance of Floor or Roof Assemblies

During standard fire tests, specimens must support the anticipated superimposed loads throughout the fire-endurance period. Failure to support the loads is called the structural end point.

Figure 7.5.3.1 Details of three hour assembly consisting of hollow core slabs with a gypsum wall board ceiling



1. Precast concrete hollow slabs – Minimum equivalent thickness = 2.75 in.
2. Grout – (Not Shown) – Sand-cement grout along full length of joint.
3. Hanger Wire – No. 18 SWG galvanized steel wire. Hanger wire used to attach wallboard furring channels to precast concrete units. Wire to be located at each intersection of furring channels and joints between hollow core slabs, but not to exceed 4 ft o.c.
4. Wallboard Furring Channels – No. 26 ga. Galvanized steel, $^7/_8$ in. high, $2^3/_4$ in. base width, $1^3/_8$ in. face width and 12 ft long. Channels to be installed perpendicular to hollow core slabs and spaced 24 in. o.c., except at wallboard butt joints where they are spaced $6^1/_2$ in. o.c. Channels secured to concrete units with double strand of hanger wire looped through fasteners. At furring channel splices, channels to be overlapped 6 in. and tied together with hanger wire at each end of splice.
5. Wallboard – $^5/_8$ in. thick, 4 ft wide. Type X, installed with long dimension perpendicular to furring channels. Over butt joints, a 3 in. wide piece of wallboard to be inserted with end extending a minimum 6 in. beyond board width.
6. Wallboard Fasteners – 1 in. long. Type S, bugle head screws. Fasteners spaced 12 in. o.c. along each furring channel except at butt joints where fasteners spaced at 6 in. o.c. At butt joints, fasteners located $3^1/_4$ in. from board edge. Along side joints, fasteners located $^3/_4$ in. from board edge.
7. Joint System – (Not Shown) – Paper tape embedded in cementitious compound joint, and covered with two layers of cementitious compound with edges feathered out. Wallboard fastener heads covered with two layers of cementitious compound.

The most important factor affecting the structural fire endurance of a floor or roof assembly is the method of support, as in, whether the assembly is simply supported and free to expand (unrestrained) or if the assembly is continuous or thermal expansion is restricted (restrained).

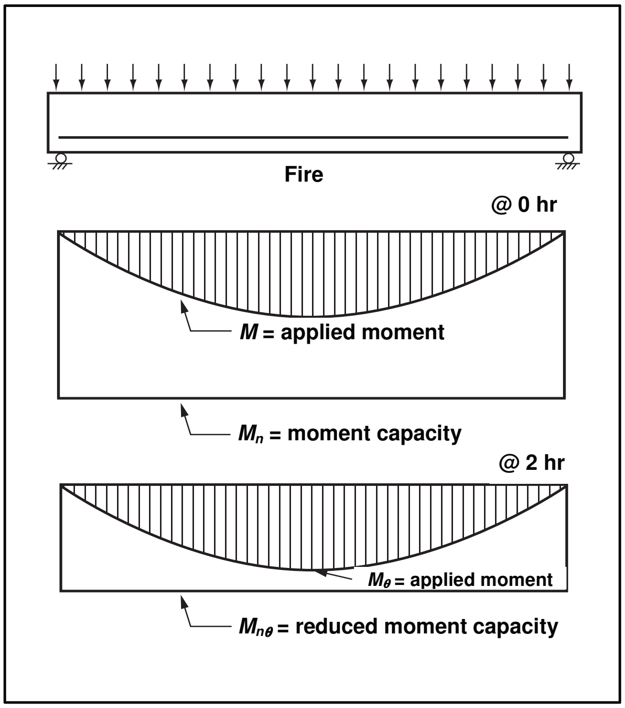
7.6.1 Simply Supported Slabs—Unrestrained

Figure 7.6.1.1 illustrates the behavior of a simply supported slab exposed to fire from beneath. Because strands are parallel to the axis of the slab, the ultimate moment capacity is constant throughout the length:

$$\phi M_n = \phi A_{ps} f_{ps}(d_p - a/2) \qquad \text{(Eq. 7.6.1)}$$

See chapter 2 for evaluating f_{ps}.

Figure 7.6.1.1 Moment diagrams for simply supported slab before and during fire exposure



If the slab is uniformly loaded, the moment diagram will be parabolic with a maximum value at midspan of:

$$M_\theta = \frac{w\ell^2}{8} \qquad \text{(Eq. 7.6.2)}$$

As the material strengths diminish with elevated temperatures, the retained moment capacity becomes:

$$M_{n\theta} = A_{ps} f_{ps\theta} \, (d_p - a_\theta /2) \qquad \text{(Eq. 7.6.3)}$$

where θ signifies the effects of high temperatures. Note that A_{ps} and d_p are not affected, but f_{ps} is reduced. Similarly, a is reduced, but the concrete strength at the top of the slab f'_c is generally not reduced significantly because of its lower temperature.

Flexural failure can be assumed to occur when $M_{n\theta}$ is reduced to M_θ. From this expression, it can be seen that the fire endurance depends on the applied loading and on the strength-temperature characteristics of the steel. In turn, the duration of the fire before the "critical" steel temperature is reached depends on the protection afforded to the reinforcement.

Note that, when calculating the fire-rated capacity, the load factors and strength reduction factor are taken as 1.0. The factors of safety are built into the ASTM E119 conditions of acceptance. If a greater safety factor is desired than is inherent in the load tests, a proportional increase in the fire-rating time period should be specified.

Figure 7.6.1.2 shows the relationship between temperature and strength of various types of steel. Figures 7.6.1.3, 7.6.1.4, and 7.6.1.5 show temperatures within concrete slabs during standard fire tests. The data in those figures are applicable to hollow core slabs as well as solid slabs. By using the equations given above and the data in Fig. 7.6.1.2 through 7.6.1.5, the moment capacity of slabs can be calculated for various fire endurance periods.

Table 7.6.1 shows values of u for simply supported, unrestrained hollow core slabs. For a given moment ratio and fire endurance, a value of u can be selected to satisfy the structural end point. The values shown are based on $\omega_{pu} = 0.05$ and can be reduced by $^1/_{16}$ in. for $\omega_{pu} = 0.10$.

Example 7.6.1

Determine the maximum safe superimposed load that can be supported by an 8-in.-deep hollow core slab with a simply supported unrestrained span of 25 ft and a required fire endurance of 3 hours.

Given:

$h = 8$ in. $u = 1.75$ in. $b = 36$ in.
6 $^1/_2$- in.-dia., 270 ksi strands
$A_{ps} = (6)(0.153) = 0.918$ in.^{2}
$d_p = 8 - 1.75 = 6.25$ in.
$D_{sw} = 54$ lb/ft^2
$\ell = 25$ ft

carbonate aggregate

Solution:

(a) From Fig. 7.6.1.3, estimate strand temperature at 3 hours. With $u = 1.75$ in. above fire-exposed surface, $\theta_s = 925$ °F.

(b) Determine $f_{pu\theta}$ from Fig. 7.6.1.2.
For cold-drawn steel at 925 °F:
$f_{pu\theta} = 33\% \, (f_{pu}) = (0.33)(270) = 89.1$ ksi

Figure 7.6.1.2 Temperature-strength relationships for hot-rolled and cold-drawn steels

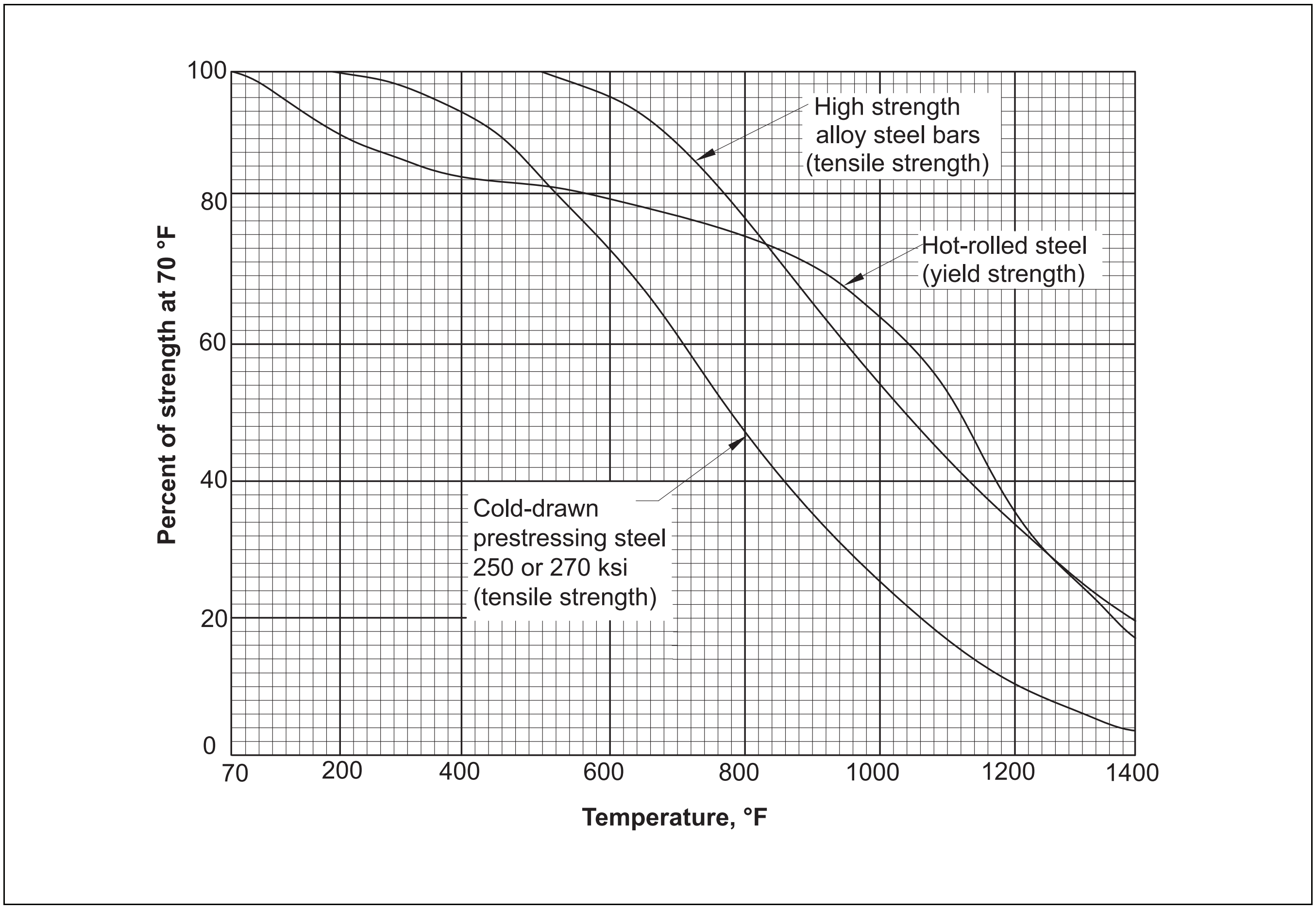


Figure 7.6.1.3 Temperatures within carbonate aggregate concrete slabs during fire tests

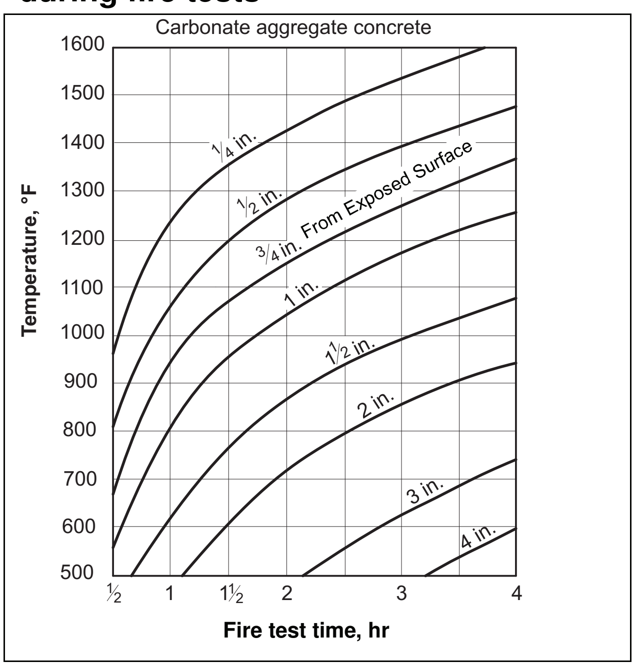


Figure 7.6.1.4 Temperatures within siliceous aggregate concrete slabs during fire tests

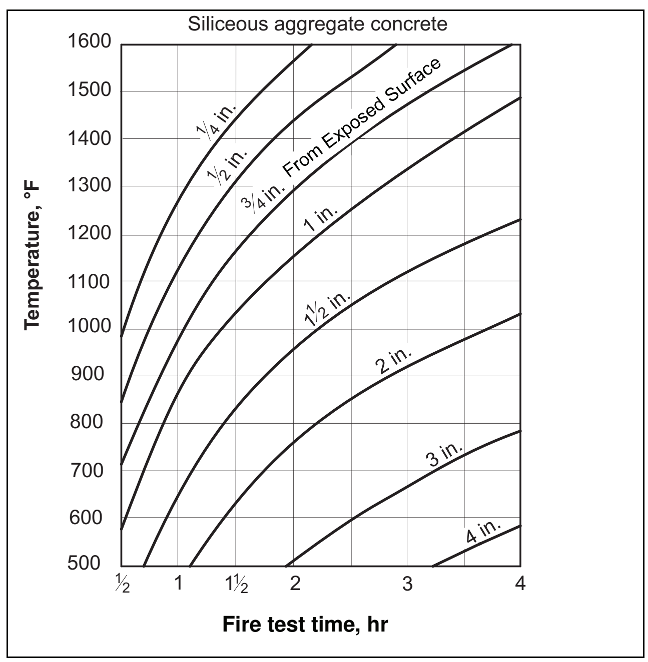
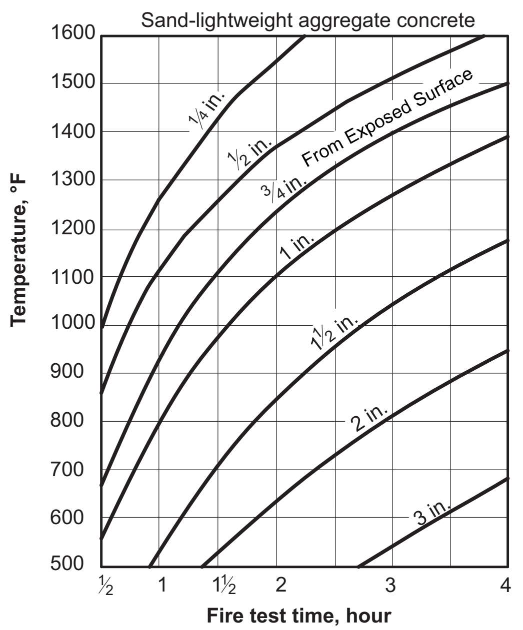


Figure 7.6.1.5 Temperatures within lightweight aggregate concrete slabs during fire tests



(c) Determine $M_{n\theta}$ and w.

$$f_{ps\theta} = 89.1\left[1 - \frac{0.28}{0.80}\left(\frac{0.918}{36(6.25)}\frac{89.1}{5}\right)\right]$$
$$= 86.8 \text{ ksi}$$

$$a_\theta = \frac{0.918(86.8)}{0.85(5)(36)} = 0.52 \text{ in.}$$

$$M_{n\theta} = 0.918(86.8)(6.25 - 0.52/2)/12$$
$$= 39.8 \text{ kip-ft}$$

$$w = \frac{8(39.8)1000}{(25^2)(3 \text{ ft})} = 170 \text{ lb/ft}^2$$

$$L = w - D_{sw} = 170 - 54 = 116 \text{ lb/ft}^2$$

(d) Calculate maximum allowable live load at room temperature.

$$f_{ps} = 270\left[1 - \frac{0.28}{0.80}\left(\frac{0.918}{36(6.25)}\frac{270}{5}\right)\right]$$
$$= 249 \text{ ksi}$$

$$a = \frac{0.918(249)}{0.85(5)(36)} = 1.49 \text{ in.}$$

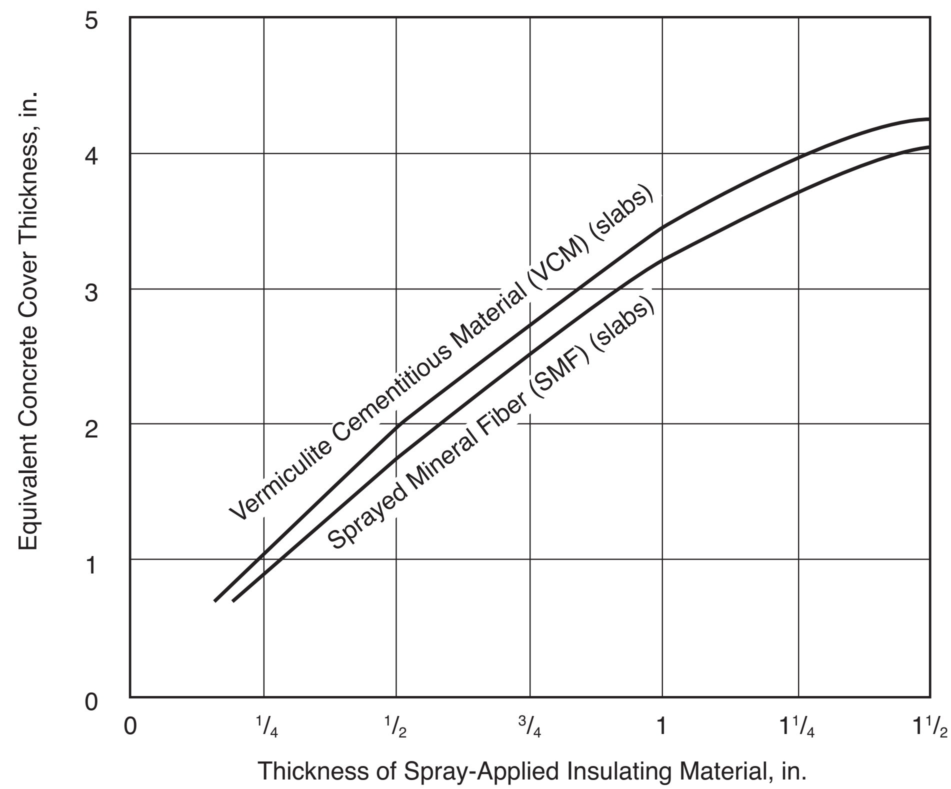
$$\phi M_n = 0.9(0.918)(249)(6.25 - 1.49/2)/12$$
$$= 94.4 \text{ kip-ft}$$

Table 7.6.1 *u* for simply supported unrestrained hollow core slabs*

Fire endurance, hours	M / M_n	Aggregate Type		
		Siliceous, in.	Carbonate, in.	Sand-lightweight, in.
1	0.50	$1^1/_4$	$1^1/_{16}$	$1^1/_{16}$
1	0.40	$1^1/_{16}$	$^{15}/_{16}$	$^{15}/_{16}$
1	0.30	$^{15}/_{16}$	$^{13}/_{16}$	$^{13}/_{16}$
2	0.50	$1^{15}/_{16}$	$1^{13}/_{16}$	$1^{13}/_{16}$
2	0.40	$1^3/_4$	$1^9/_{16}$	$1^9/_{16}$
2	0.30	$1^9/_{16}$	$1^5/_{16}$	$1^5/_{16}$
3	0.50	$2^1/_2$	$2^5/_{16}$	$2^1/_8$
3	0.40	$2^3/_{16}$	2	$1^{15}/_{16}$
3	0.30	$1^{15}/_{16}$	$1^{11}/_{16}$	$1^{11}/_{16}$

* *u* is the distance from bottom of slab to center of strands with all strands having the same *u*.
Based on $A_{ps}f_{pu}/bd_pf'_c = 0.05$; conservative for values greater than 0.05.

Figure 7.6.2.1 Equivalent concrete cover thickness for spray-applied coatings



$$w_u = \frac{8(94.4)(1000)}{(25^2)(3\text{ ft})} = 403\ \text{lb/ft}^2$$

With load factors of $1.2D + 1.6L$

$$L = \frac{403 - 1.2(54)}{1.6} = 211\,\text{lb/ft}^2$$

Conclusion: $L = 116 < 211$; 116 lb/ft^2 governs

(e) To complete the analysis, heat transmission should also be checked. As calculated in Example 7.5.1, t_{eq} = 4.28 in. Figure 7.2.1 indicates that, for a 3-hr fire rating,the required equivalent thickness is 5.65 in. Therefore, some type of undercoat or overlay is required to satisfy the 3-hr heat transmission.

7.6.2 Effect of Spray-Applied Coatings

The addition of a spray-applied coating of vermiculite cementitious material (VCM) or sprayed mineral fiber (SMF) can be used to increase the equivalent strand cover. Figure 7.6.2.1 shows the relationship between thickness of spray-applied coatings and equivalent concrete cover. Thus, if strands are centered 3/4 in. above the bottom of a hollow core slab and 1/4 in. of sprayed mineral fiber is applied, the *u* distance to be used in Figures 7.6.1.3, 7.6.1.4, or 7.6.1.5 is 3/4 in. plus the equivalent cover of 0.9 in. obtained from Fig. 7.6.2.1.

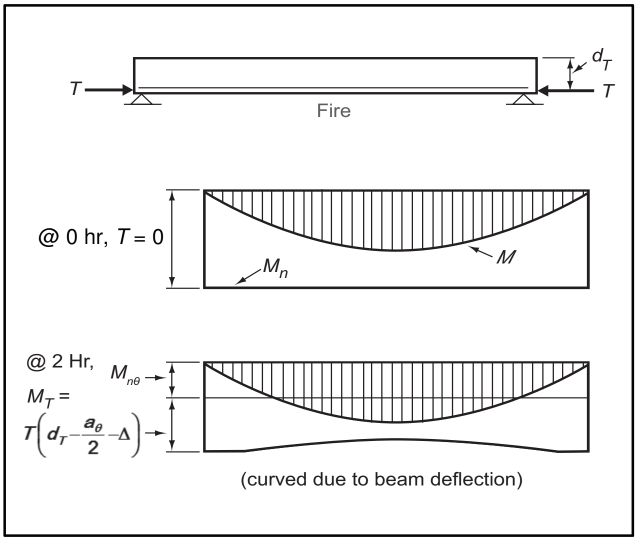
7.6.3 Simply Supported Slabs—Restrained

As stated earlier, a restrained hollow core floor will have a significantly longer fire-resistance rating than an unrestrained floor. Stated conversely, a restrained floor will support significantly more load for a given fire duration than an unrestrained floor.

As the heated portion of the floor expands and pushes against the surrounding unheated area, compressive forces, or thrusts, develop. These thrusts act near the bottom of the slab when the fire starts and rise as the fire progresses. Nevertheless, the thrust is generally great enough to increase the fire endurance significantly, in some instances by more than 2 hours. In most fire tests of restrained assemblies, the fire endurance is determined by the heat transmission end point rather than the structural end point.

The effects of restraint to thermal expansion can be characterized as shown in Fig. 7.6.3.1. The thermal thrust T acts in a manner similar to an external prestressing force inducing negative end moments, which increase the positive moment capacity.

Figure 7.6.3.1 Moment diagrams for axially restrained slab during fire exposure. Note that at 2 hours $M_{n\theta}$ is less than M, but axial restraint permits slab to continue to support load



If a hollow core floor is determined to be restrained, there are three methods for evaluating the fire endurance of a restrained slab.

Appendix C of PCI Manual MNL 124-11 outlines calculation methods where either a required thrust is calculated or the effect of thrust on the moment capacity is calculated. These procedures are seldom used except as an academic exercise.

If the slab has been tested and received a UL restrained fire rating, this design number can be used to justify the fire endurance and no rational fire design is required.

The simplest and most common method is to use the prescriptive requirements given in Table 7.3.3. A review of this table shows that a restrained slab with $^3/_4$ in. strand cover will develop a 4-hr fire rating. Because there is no limitation on strand temperature for restrained slabs, the slab capacity will be controlled by typical (non-fire) design criteria and no rational fire design is required.

For all methods, the heat transmission requirements must still be checked. Heat transmission is not affected by restraint and must be checked for the full required rating.

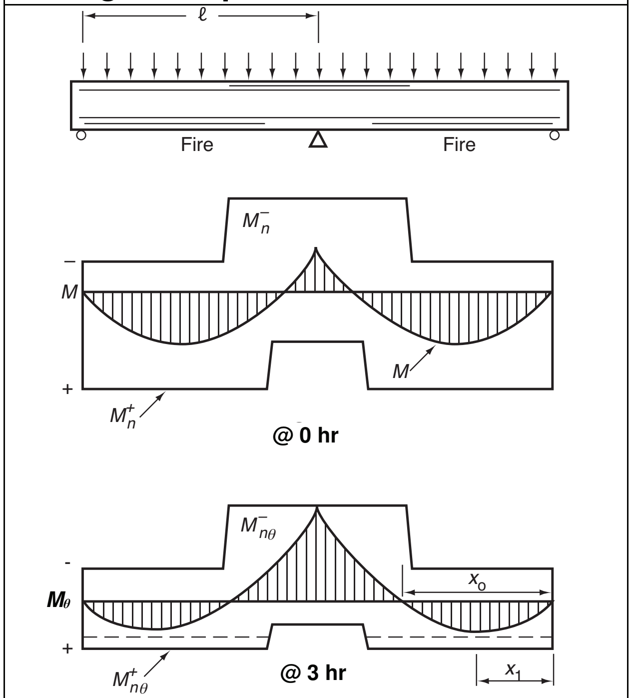
7.6.4 Structurally Continuous Slabs

Continuous members undergo changes in stresses when subjected to fire, resulting from temperature gradients within the structural members, or changes in strength of the materials at high temperatures, or both.

Figure 7.6.4.1 shows a continuous beam whose underside is exposed to fire. The bottom of the beam becomes hotter than the top and tends to expand more than the top. This differential temperature causes the ends of the beam to tend to lift from their supports, thereby increasing the reaction at the interior support. This action results in a redistribution of moments, that is, the negative moment at the interior support increases while the positive moments decrease.

During the course of a fire, the negative moment reinforcement remains cooler than the positive moment reinforcement because it is better protected from the fire. Thus, the increase in ly, the redistribution that occurs is sufficient to cause yielding of the negative moment reinforcement. The resulting decrease in positive moment means that the positive moment reinforcement can be heated to a higher temperature before a failure will occur. Therefore, the fire endurance of a continuous concrete beam is generally significantly longer than that of a simply supported beam having the same cover and loaded to the same moment intensity.

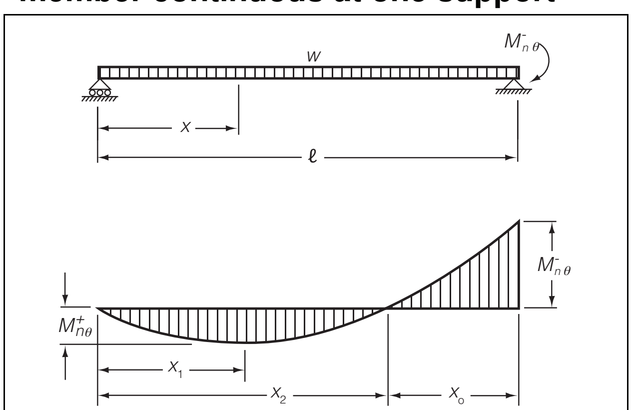
Figure 7.6.4.1 Moment diagrams for continuous 2-span beam before and during fire exposure



It is possible to design the reinforcement in a continuous beam or slab for a particular fire endurance period. From Fig. 7.6.4.1, the beam can be expected to collapse when the positive moment capacity $M_{n\theta}^+$ is reduced to the value indicated by the dashed horizontal line (when the redistributed moment at point x_1, from the outer support, $M_{\theta x1} = M_{n\theta}^+$).

Figure 7.6.4.2 shows a uniformly loaded beam or slab continuous (or fixed) at one support and simply supported at the other. Also shown is the redistributed applied-moment diagram at failure.

Figure 7.6.4.2 Uniformly loaded member continuous at one support



Values for $M_{n\theta}^+$, can be calculated by the procedures given in section 7.6.1.

Values for $M_{n\theta}^-$, and x_o can be calculated as:

$$M_{n\theta}^- = \frac{w\ell^2}{2} \pm w\ell^2\sqrt{\frac{2M_{n\theta}^+}{w\ell^2}} \qquad \text{(Eq. 7.6.4)}$$

$$x_o = 2\frac{M_{n\theta}^-}{w\ell} \qquad \text{(Eq. 7.6.5)}$$

In most cases, redistribution of moments occurs early during the course of a fire before the negative moment capacity has been reduced by the effects of fire. In such cases, the length of x_o is increased (the inflection point moves toward the simple support.) For such cases:

$$x_o = \frac{2M_n^-}{w\ell} \qquad \text{(Eq. 7.6.6)}$$

Figure 7.6.4.3 shows a symmetrical beam or slab in which the end moments are equal. In that case:

$$M_{n\theta}^- = \frac{w\ell^2}{8} - M_{n\theta}^+ \qquad \text{(Eq. 7.6.7)}$$

and $$\frac{wx_2^2}{8} = M_{n\theta}^+ \qquad \text{(Eq. 7.6.8)}$$

In negative moment regions, the compressive zone is directly exposed to fire, so calculations for d_θ and a_θ must be modified by using $f'_{c\theta}$ from Fig. 7.6.4.4 and neglecting concrete hotter than 1400 °F.

Figure 7.6.4.3 Symmetric uniformly loaded member continuous at both supports

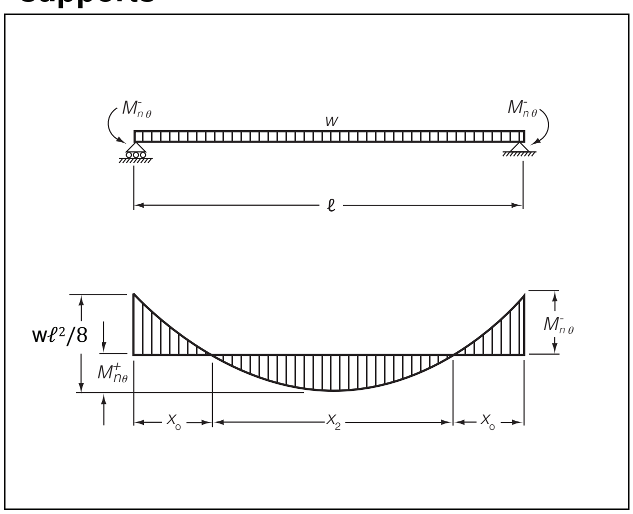
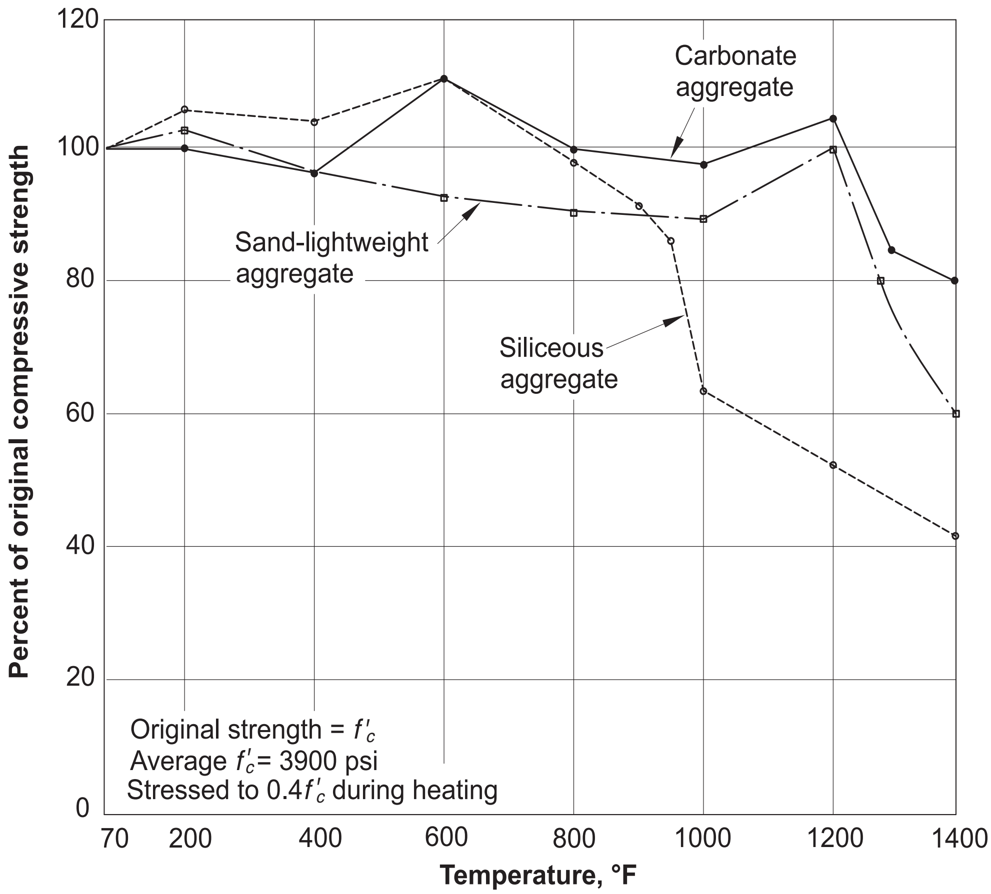


Figure 7.6.4.4 Compressive strength of concrete at high temperatures



7.6.5 Detailing Precautions

It should be noted that the amount of moment redistribution that can occur depends on the amount of negative reinforcement. Tests have clearly demonstrated that the negative moment reinforcement will yield, so the negative moment capacity is reached early during a fire test, regardless of the applied loading. The designer must exercise care to ensure that a secondary type of failure will not occur. To avoid a compression failure in the negative moment region, the amount of negative moment reinforcement should be small enough so that ω_θ $(A_s f_{y\theta}/b_\theta d_\theta f'_{c\theta})$ is less than 0.30, before and after reductions in f_y, b, d, and f'_c are taken into account. Furthermore, the negative moment bars or welded-wire reinforcement (WWR) must be long enough to accommodate the complete redistributed moment and change in the inflection points. It should be noted that the worst condition occurs when the applied loading is smallest, such as dead load plus partial or no live load. It is recommended that at least 20% of the maximum negative moment reinforcement be extended throughout the span.

Example 7.6.4

Determine the amount of negative moment reinforcement needed to provide a 3-hr fire endurance for sand-lightweight hollow core slabs, 8 in. deep, 5000 psi concrete, 48 in. wide, with 6-$^7/_{16}$ in. dia., 270 ksi strands and 2 in. of 4000 psi composite topping. Slabs span 25 ft of an exterior bay (no restraint to thermal expansion). D = 65 lb/ft^2; L = 100 lb/ft^2. Center of gravity of strands = 1$^3/_4$ in.

Solution:

The value for $M^{+}_{n\theta}$ can be calculated (using the procedure discussed in section 7.6.1) to be 40.8 kip-ft. Solving Eq. 7.6.4.

$$w\ell^2 = \frac{4(65+100)(25^2)}{1000}$$
$$= 412.5 \text{ kip-ft}$$

$$M^{-}_{n\theta} = \frac{412.5}{2} - 412.5\sqrt{2\left(\frac{40.8}{412.5}\right)}$$
$$= 22.8 \text{ kip-ft}$$

Determine A_s^- neglecting concrete above 1400 °F in negative moment region. From Fig. 7.6.1.5, neglect $^3/_4$ in. above bottom and assume steel centered in topping.

$$d = 10 - {}^3/_4 - 1 = 8.25 \text{ in.}$$

Assume $f'_{c\theta}$ in compressive zone = 0.8 f'_c = 4 ksi

Assume $d^- - \dfrac{a^-_\theta}{2} = 8.1 \text{in.}$

$$A^-_s = \frac{22.8(12)}{60(8.1)} = 0.563 \text{ in.}^2$$

Check $a^-_\theta = \dfrac{0.56(60)}{0.85(4)(48)} = 0.21 \text{in.}$

$$d^- - \frac{a^-_\theta}{2} = 8.25 - \frac{0.21}{2} = 8.15 \cong 8.1 \text{in. ok}$$

Use 6 × 6 - W2.1 × W2.1 WWR throughout plus grade 60, #4 bars at 18 in. on center in the negative moment region.

$$A^-_s = 4(0.041) + \frac{48}{18}(0.20) = 0.697 \text{ in.}^2$$

Calculate x_o for fully loaded condition and for dead load plus one-half live load.

$$M^-_{n\theta} = \frac{0.697}{0.563}(22.8) = 28.2 \text{ kip-ft}$$

full load = 4(0.065 + 0.100) = 0.66 kip/ft
pattern load = 4(0.065 + 0.050) = 0.46 kip/ft
$M^-_n = 31.0$ kip-ft (calculated at room temperatures)

From Eq. 7.6.6
With full load

$$x_o = \frac{2M^-_n}{w\ell} = \frac{2(31.0)}{0.66(25)} = 3.76 \text{ ft}$$

With half live load

$$x_o = \frac{2M^-_n}{w\ell} = \frac{2(31.0)}{0.46(25)} = 5.39 \text{ ft}$$

Place the WWR throughout to satisfy the 20% requirement. Half of the #4 bars should extend 6 ft on each side of the interior support and half should extend 4 ft.

Use grade 60, #4 bars 10 ft long at 18 in. on center and stagger their placement across the joint.

7.6.6 Existing Construction

Due to changes in occupancy, fire endurance of existing construction may have to be evaluated. The following example presents some options that may be helpful when fire endurance of existing construction must be improved.

Example 7.6.6

Hollow core floor slabs were installed in a building several years ago when a 1-hr fire endurance was required. The occupancy of the building will be changed and the floors must qualify for a 3-hr fire endurance. What can be done to upgrade the fire endurance?

Given:

Slabs are 4 ft wide, 8 in. deep, prestressed with five $^3/_8$ in. dia., 270 ksi strands located 1 in. above the bottom of the slab, and span 24 ft. Slabs are made with 5000 psi siliceous aggregate concrete, have an equivalent thickness of 3.75 in., and weigh 47 lb/ft^2. The slabs are untopped and the superimposed load will be 50 lb/ft^2.

Solution:

There are a number of possible solutions. The appropriate solution will depend on architectural or functional requirements and economics.

For some parts of the building, the slabs might be made to qualify as restrained in accordance with Table 7.4.1 and Fig. 7.4.1, in which case those slabs would qualify structurally for 3 hours, but would still have to be upgraded to qualify for 3 hours by heat transmission.

A gypsum wallboard ceiling installed as shown in Fig. 7.5.3.1 would provide 3 hours both structurally and for heat transmission. Calculations of the ultimate capacity and stresses should be made to ensure that the added weight of the ceiling can be adequately supported.

A spray-applied undercoating of VCM or SMF can also be used. For heat transmission, the required thickness of undercoating for 3 hours is 0.6 in. (Fig. 7.5.2.1). From Fig. 7.6.2.1, it can be seen that with a thickness of 0.6 in. of VCM or SMF, the equivalent thickness of concrete cover is approximately 2 in. Thus, the equivalent u distance is 2 in. plus 1 in. or approximately 3 in. From Fig. 7.6.1.4, with u equal to 3 in., the strand temperature will be 690 °F at 3 hours, resulting in the strength of the prestressing steel being 63% of its 70 °F strength (Fig. 7.3.1.5) or 0.63×270 ksi = 170 ksi. Calculations can be made in accordance with the procedures of section 7.6.1, but if the strand strength exceeds about 50% of its room temperature strength, the assembly will generally be satisfactory structurally.

7.7 Wall Panels

The fire endurance of hollow core wall panels is nearly always governed by the heat transmission criteria of ASTM E119, rather than by structural requirements during fire tests. This is likely due to the fact that gravity load bending moments are so small that even fire-reduced steel strengths are more than adequate. In fact, the 2012 IBC does not list any cover requirements for reinforcing steel (mild or prestressed) in walls. For protection against weather and other effects, it is recommended that the cover requirements of ACI 318-11 Section 7.7.3 be satisfied.

The calculation of equivalent thickness for hollow core wall panels and the required minimum thickness for 1- through 4-hour fire ratings is the same as for a hollow core floor or roof slab. These requirements are addressed in Section 7.5.1 and Table 7.3.2 of this chapter. In addition, IBC section 722.2.1.1.2 states that if all of the cores of a hollow core wall panel are filled with loose-fill material, such as expanded shale, clay, slag, vermiculite, or perlite, the fire-resistance rating is the same as for a solid wall of the same concrete type and the same overall thickness.

7.7.1 Sandwich Panels

Hollow core wall panels are often made with a hollow core structural wythe, a layer of rigid insulation, and a 2 in. to 3 in. facing wythe. When the insulation is sandwiched between two layers of concrete in this fashion, the IBC limits the flame-spread index to 100, except that foam plastic insulation may not exceed 75.

While the facing wythe is considered non-composite for structural purposes, it does improve the fire resistance of the panel. The fire endurance of such a panel can be calculated as:

$$R = (R_1^{0.59} + R_2^{0.59} + R_3^{0.59})^{1.7} \qquad \text{(Eq. 7.7.1)}$$

where R^n is the fire endurance of each layer in minutes. Limited testing indicates that R equals 5 minutes for a 1-in.-thick layer of cellular polystyrene. Based on this, IBC Section 722.2.1.2.2 allows the use of 5 minutes as the value of R for any foam plastic insulation equal to or greater than 1 in. thick.

Example 7.7.1

Calculate the fire endurance of an assembly consisting of an 8-in.-thick hollow core panel (carbonate aggregate, t_{eq} = 4.28 in.), 2 in. of polyurethane insulation, and a 2 in. facing wythe.

Solution:

From Fig. 7.5.1, the fire endurance of the hollow core panel is 102 minutes and the 2 in. wythe is 30 minutes.

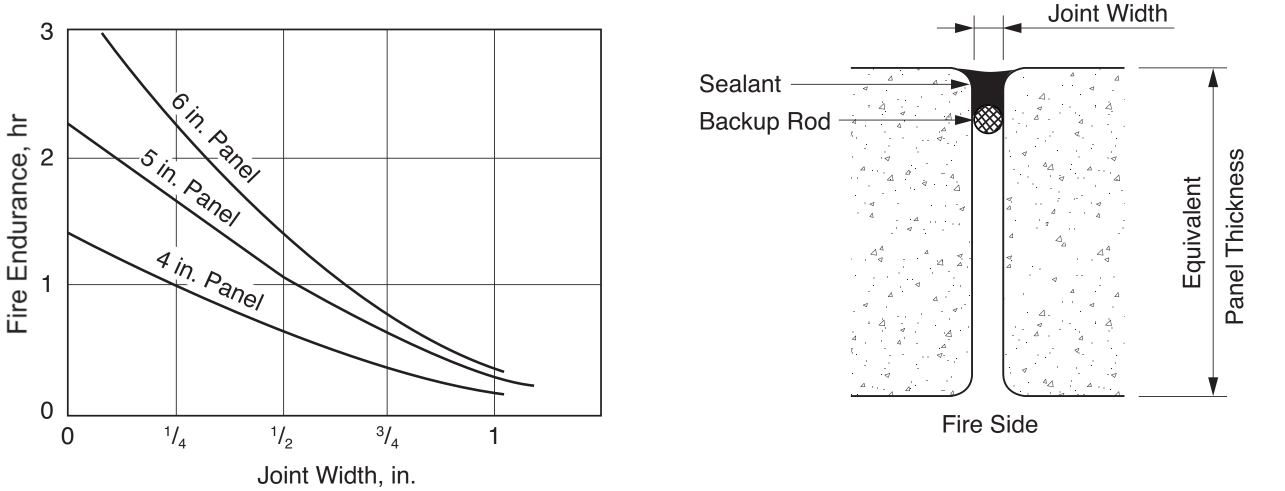
Therefore, the fire endurance of the assembly is:

$$\begin{aligned} R &= (102^{0.59} + 5^{0.59} + 30^{0.59})^{1.7} \\ &= (15.3 + 2.6 + 7.4)^{1.7} \\ &= 243 \text{ minutes} = 4 \text{ hr } 3 \text{ min} \end{aligned}$$

7.7.2 Joint Treatments

Section 715.1 of the IBC requires joints between wall panels to have the same fire-resistance rating as the wall itself. An exception is made for walls that are allowed to have unprotected openings. Table 705.8 of the IBC summarizes the permitted percentages of protected and unprotected openings in a wall. These percentages are based on the building occupancy and the fire-separation distance.

Figure 7.7.1 Fire endurance of one-stage butt joint with joint treatment of backer rod and sealant

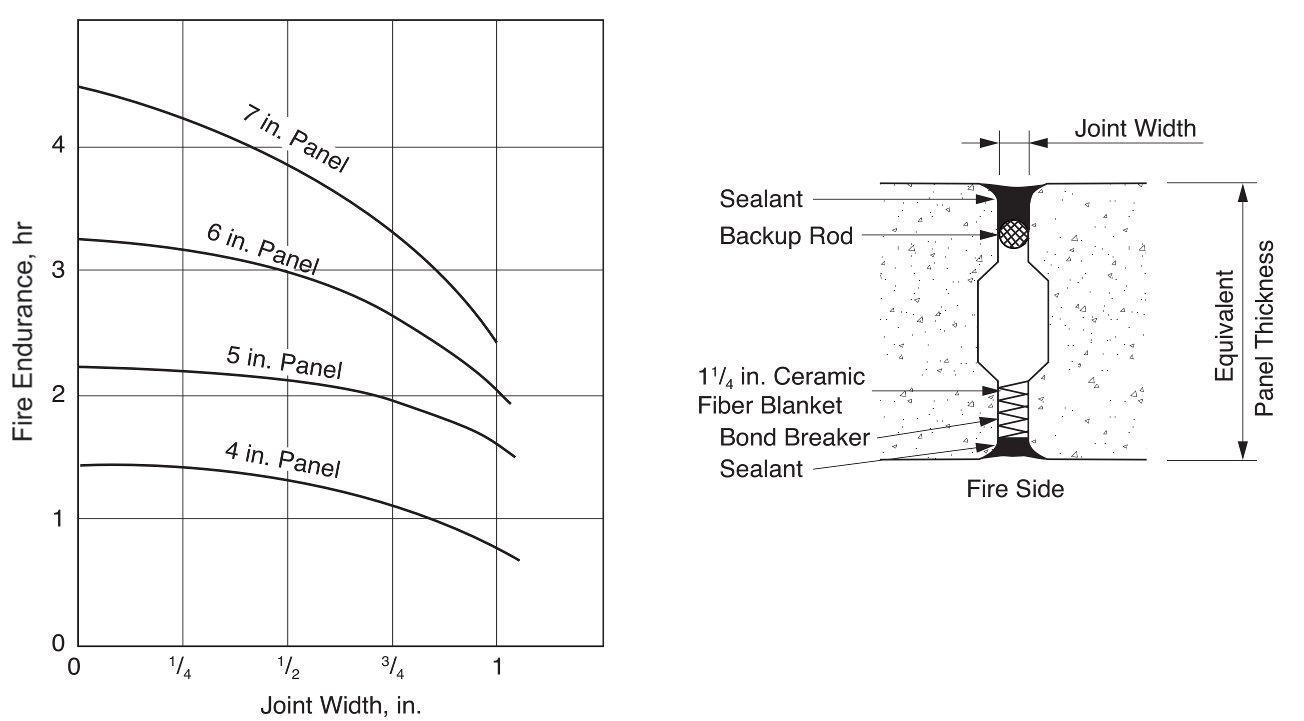


If unprotected openings are allowed, the unprotected joints must be included as openings in calculating the opening percentage for comparison with the permitted percentages from Table 705.8. If unprotected openings are not permitted, or the percentage of openings exceeds that which is allowed, the joints must be protected to satisfy the same fire endurance as the wall.

Fire tests of wall panels indicate that the fire endurance, as limited by heat transmission requirements, is influenced by the equivalent panel thickness, joint size, and type and thickness of the insulating material. Figures 7.7.1, 7.7.2, and 7.7.3 can be used to determine the joint treatment needed to achieve the required fire-resistance rating.

Figure 7.7.1 shows the fire endurance of a one-stage butt joint treatment of polyethylene backer rod and sealant. Figure 7.7.2 provides data for a two-stage cavity joint incorporating a $1^1/_4$-in.-

Figure 7.7.2 Fire endurance of two-stage cavity joint with joint treatment of $1^1/_4$ in. ceramic fiber blanket, backer rod, and sealant



thick ceramic blanket. Figure 7.7.3 provides a design aid to estimate the thickness of ceramic fiber blanket required for a one-stage butt joint of a specific joint width, panel thickness, and fire endurance.

Example 7.7.2

Given a hollow core wall panel with an equivalent thickness of 5 in. and a joint width of $^3/_4$ in., determine the thickness of ceramic fiber blanket required to achieve a 2-hr fire rating.

Solution:

From Fig. 7.7.3, with an equivalent thickness of 5 in. and a 2-hr fire endurance, a 1 in. joint requires 2.1 in. and a $^3/_8$ in. joint requires 0.7 in. of ceramic fiber blanket. Interpolating between these values gives a required thickness of approximately 1.6 in.

7.8 Protection of Connections

Bearing in mind that the purpose of fire-resistance design is not to eliminate all damage, but only to maintain stability and prevent collapse, many types of connections do not need protection. Gravity connections where the slabs bear directly on a ledge or wall generally do not require special treatment. In the event of a fire, the bearing pad or strip may melt or burn, but no collapse will occur. A bearing angle or bracket welded to the inside face of a wall panel may require protection if weakening of this support steel will cause the collapse of the floor or roof. Critical connections should be protected to the same extent as the supported structure.

Figure 7.7.3 Design aid for estimating the thickness of ceramic fiber blanket required in one-stage butt joints for various fire endurances

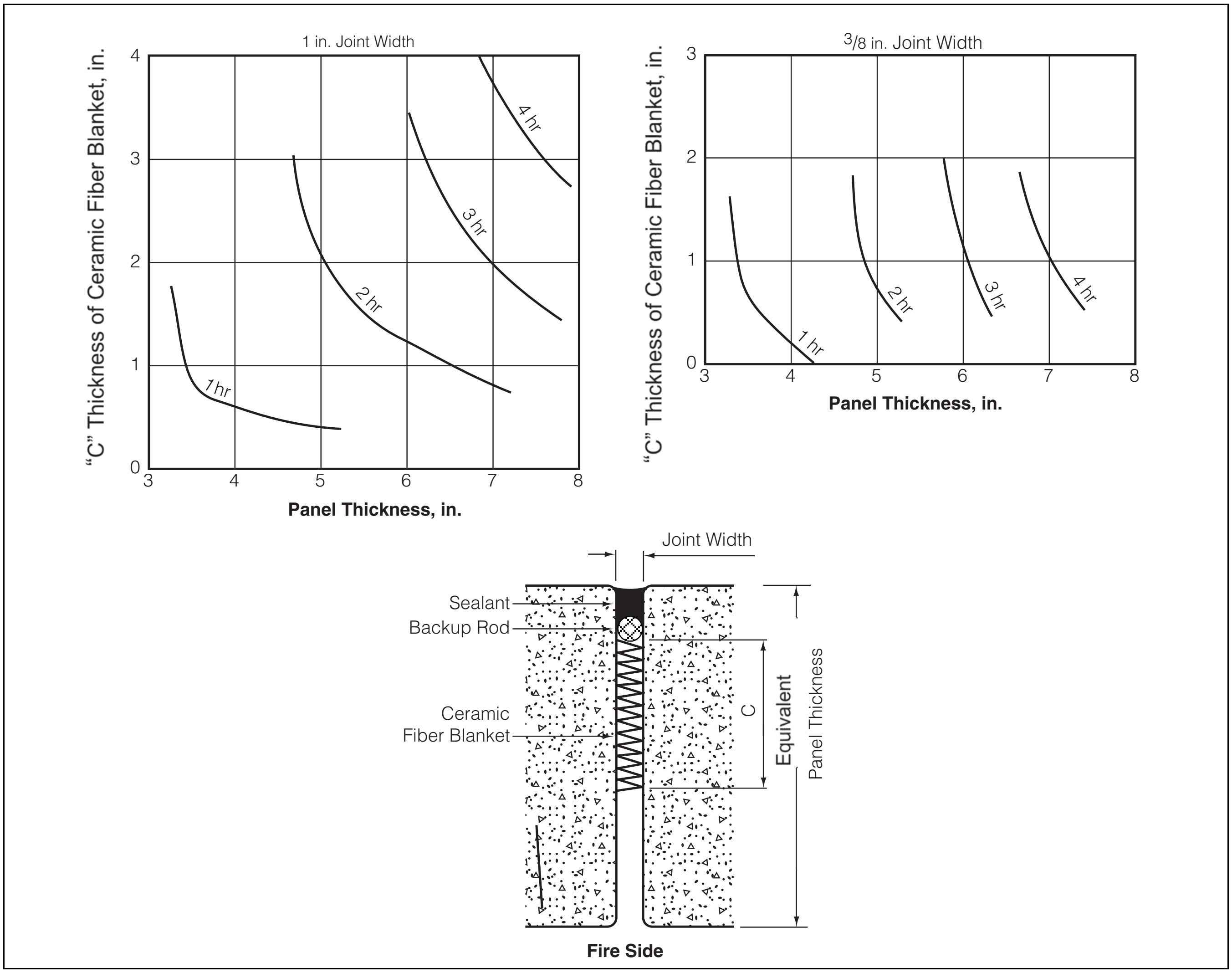
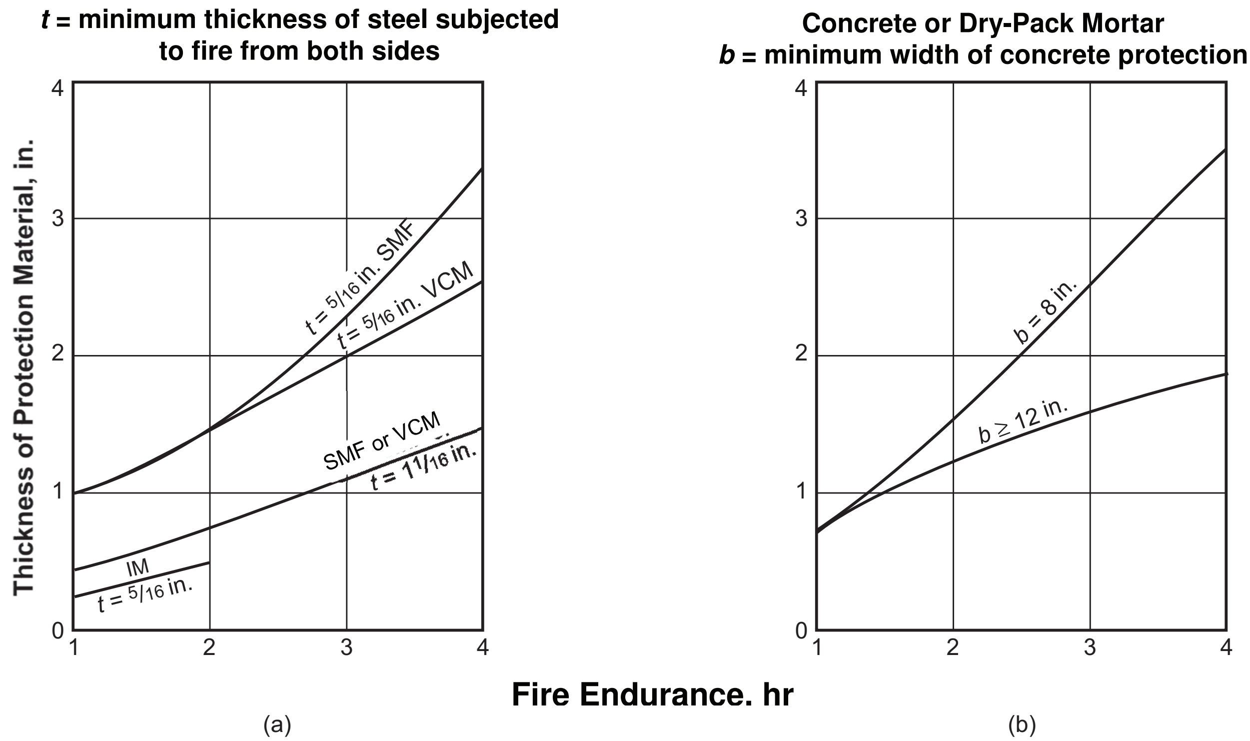


Figure 7.8.1 Thickness of protection materials applied to connection consisting of structural steel shapes



IM = intumescent mastic   SMF = sprayed mineral fiber   VCM = vermiculite cementitious material

The amount of protection required is a function of the stress-strength ratio in the steel at the time of the fire and the intensity and duration of the fire. The required thickness of protection material increases as the stress level and fire intensity increase.

Figure 7.8.1 shows the thicknesses of common fire-protection materials required for fire endurances of 1 hr to 4 hr. The values shown correspond to a critical steel temperature of 1000 °F (a stress-strength ratio of 0.65). Values in Fig. 7.8.1b apply to concrete encasement of structural steel shapes used as brackets or lintels.

Chapter 8

ACOUSTICAL PROPERTIES AND VIBRATIONS OF HOLLOW CORE SLABS

8.1 Glossary

Airborne Sound – sound that reaches the point of interest by propagation through air.

Background Level – the ambient sound pressure level existing in a space.

Decibel (dB) – a logarithmic unit of measure of sound pressure or sound power. Zero on the decibel scale corresponds to a standardized reference pressure (20 μPa) or sound power (10^{-12} watt).

Flanking Transmission – transmission of sound by indirect paths other than through the primary barrier.

Frequency – the number of complete vibration cycles per second (in units of Hz).

Impact Insulation Class (IIC) – a single figure rating of the overall impact sound insulation merits of floor-ceiling assemblies in terms of a reference contour (ASTM E989[49]).

Impact Noise – the sound produced by one object striking another.

Noise – unwanted sound.

Noise Criteria (NC) – a series of curves, used as design goals, to specify satisfactory background sound levels as they relate to particular use functions.

Noise Reduction (NR) – the difference in decibels between the space-time average sound pressure levels produced in two enclosed spaces by one or more sound sources in one of them.

Noise Reduction Coefficient (NRC) – the arithmetic average of the sound absorption coefficients at 250, 500, 1000, and 2000 Hz expressed to the nearest multiple of 0.05 (ASTM C423[50]).

Reverberation – the persistence of sound in an enclosed or partially enclosed space after the source of sound has stopped.

Room Criteria (RC) Curves – a revision of the NC curves based on empirical studies of background sounds.

Sabin – the unit of measure of sound absorption (ASTM C423).

Sound Absorption Coefficient α – the fraction of randomly incident sound energy absorbed or otherwise not reflected off of a surface (ASTM C423).

Sound Pressure Level (SPL) – ten times the common logarithm of the ratio of the square of the sound pressure to the square of the standard reference pressure of 20 μPa. Commonly measured with a sound level meter and microphone, this quantity is expressed in decibels.

Sound Transmission Class (STC) – the single number rating system used to give a preliminary estimate of the sound insulation properties of a partition system. This rating is derived from measured values of transmission loss (ASTM E413[51]).

Sound Transmission Loss (TL) – ten times the common logarithm of the ratio, expressed in decibels, of the airborne sound power incident on the partition that is transmitted by the partition and radiated on the other side (ASTM E90[52]).

Structure borne Sound – sound that reaches the point of interest over at least part of its path by vibration of a solid structure.

8.2 General

The basic purpose of architectural acoustics is to provide a satisfactory environment in which desired sounds are clearly heard by the intended listeners and unwanted sounds (noise) are isolated or absorbed.

Under most conditions, the architect/engineer can determine the acoustical needs of the space and then design the building to satisfy those needs. Good acoustical design uses both absorptive and reflective surfaces, sound barriers, and vibration isolators. Some surfaces must reflect sound so that the loudness will be adequate in all areas where listeners are located. Other surfaces absorb sound to avoid echoes, sound distortion, and long reverberation times. Sound is isolated from rooms where it is not wanted by selected wall and floor-ceiling constructions. Vibration generated by mechanical equipment must be isolated from the structural frame of the building.

Most acoustical situations can be described in terms of sound source, sound transmission path, and sound receiver. Sometimes the source strength and path can be controlled and the receiver made more attentive by removing distraction or made more tolerant of disturbance. Acoustical design must include consideration of these three elements.

8.3 Approaching the Design Process

Criteria must be established before the acoustical design of a building can begin. Basically, a satisfactory acoustical environment is one in which the character and magnitude of all sounds are compatible with the intended space function.

Although a reasonable objective, it is not always easy to express these intentions in quantitative terms. In addition to the amplitude of sound, properties such as spectral characteristics, continuity, reverberation, and intelligibility must be specified.

People are highly adaptable to the sensations of heat, light, odor, and sound, with sensitivities varying widely. The human ear can detect a sound intensity of rustling leaves, 10 dB, and can tolerate, if even briefly, the powerful exhaust of a jet engine at 120 dB, 10^{12} times the intensity of the rustling sound.

8.3.1 Dealing with Sound Levels

The problems of sound insulation are considerably more complicated than those of sound absorption. The former involves reductions of sound level, which are of the greater orders of magnitude than can be achieved by absorption. These reductions of sound level from space to space can be achieved only by continuous, impervious barriers. If the problem also involves structure borne sound, it may be necessary to introduce resilient layers or discontinuities into the barrier.

Sound-absorbing materials and sound-insulating materials are used for different purposes. There is not much sound absorption from an 8-in.-thick hollow core slab; similarly, high-sound insulation is not available from a porous, lightweight material that may be applied to room surfaces. It is important to recognize that the basic mechanisms of sound absorption and sound insulation are quite different.

8.4 Sound Transmission Loss

Sound-transmission loss measurements are made at 16 frequencies at one-third octave intervals covering the range from 125 to 4000 Hz. The testing procedure is ASTM E90. To simplify specification of desired performance characteristics, the single number sound transmission class (STC) was developed.

Airborne sound reaching a wall, floor, or ceiling produces vibration in the element and is radiated with reduced intensity on the other side. Airborne sound-transmission losses of walls and floor-ceiling assemblies are a function of their weight, stiffness, and vibration-damping characteristics.

Weight is the greatest asset of concrete when it is used as a sound insulator. For sections of similar design, but different weights, the STC increases approximately six units for each doubling of weight, as shown in Fig. 8.4.1.

Precast concrete walls, floors, and roofs usually do not need additional treatments in order to provide adequate sound insulation. If desired, greater sound insulation can be obtained by using a resiliently attached layer(s) of gypsum board or other building material. The increased transmis-

Figure 8.4.1 Sound Transmission Class as a function of weight of floor

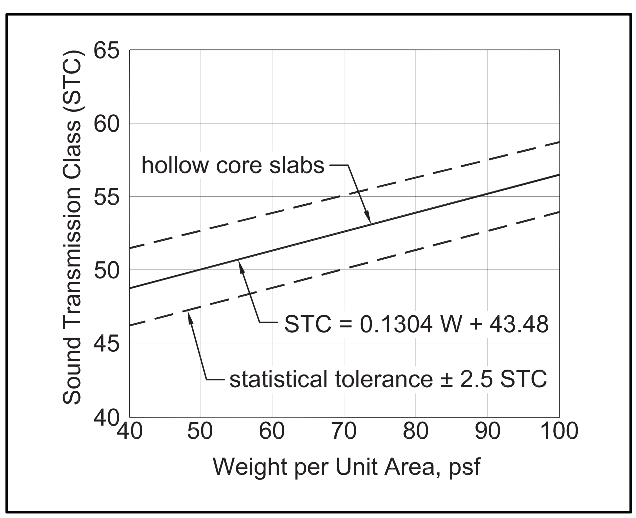
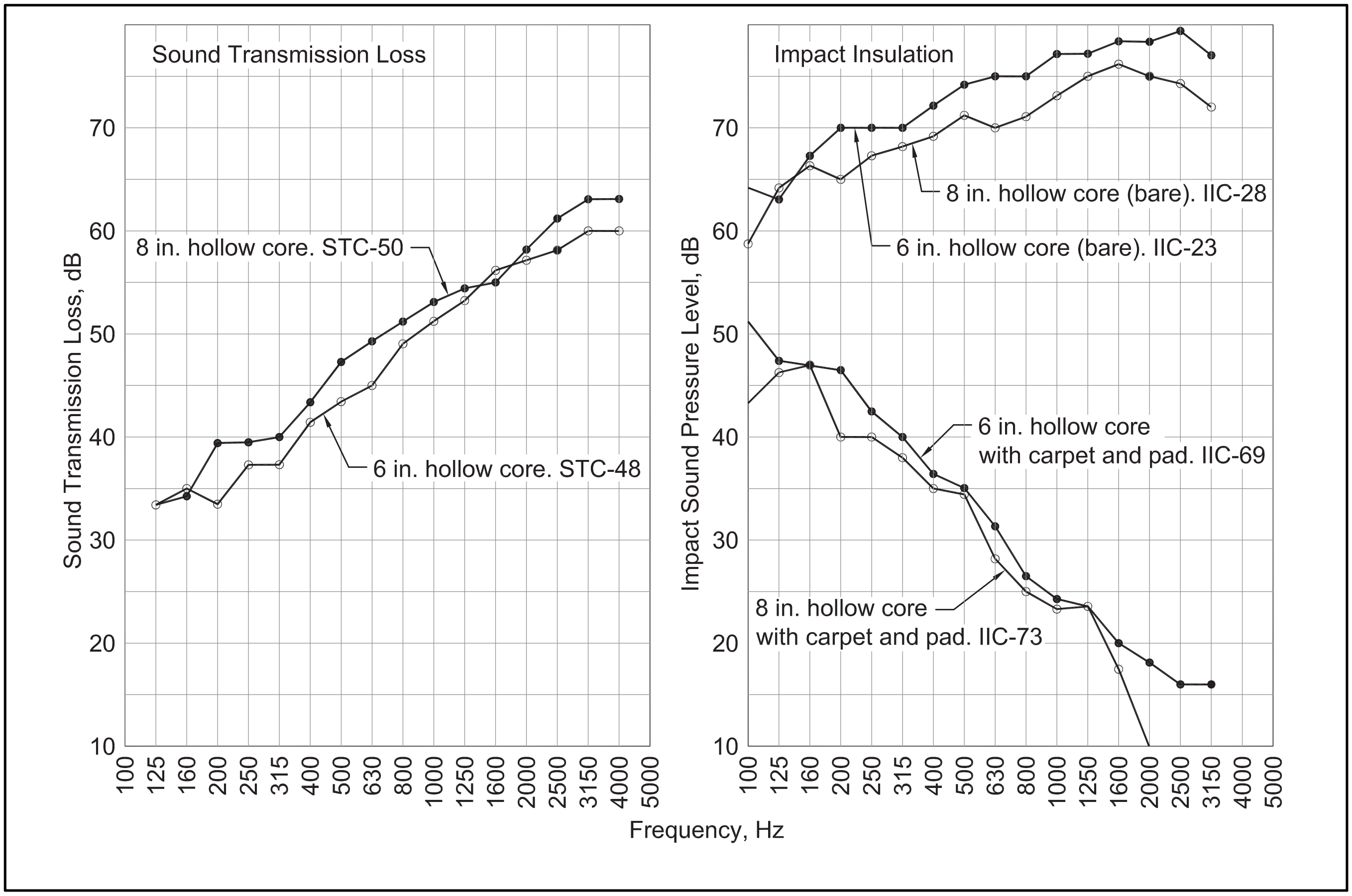


Figure 8.4.2 Acoustical test data of hollow core slabs (normal weight concrete)



sion loss occurs because the energy flow path is increased to include a dissipative air column and additional mass.

The acoustical test results of both airborne sound-transmission loss and impact insulation of 6 in. and 8 in. hollow core slabs are shown in Fig. 8.4.2. Tables 8.4.1 and 8.4.2 present the ratings for various floor-ceiling assemblies and precast concrete walls.

8.5 Impact Noise Reduction

Footsteps, dragged chairs, dropped objects, slammed doors, and plumbing generate impact noise. Even when airborne sounds are adequately controlled, there can be severe impact noise problems. The test method used to evaluate systems for impact-sound insulation is described in ASTM E492[53]. As with the airborne standard, measurements are made at 16 one-third octave intervals, but in the range from 100 to 3150 Hz. For performance specification purposes, the single number impact insulation class (IIC) is used.

Hollow core floors used in combination with resilient materials effectively control impact sound. One simple solution consists of good carpeting on resilient padding. Table 8.4.1 shows that a carpet and pad over a bare slab will significantly increase the impact noise reduction. The overall efficiency varies according to the characteristics of the carpeting and padding, such as resilience, thickness, and weight. So-called resilient flooring materials, such as linoleum, rubber, asphalt, vinyl, and the like, are not entirely satisfactory when applied directly on concrete, nor are parquet and strip-wood floors. Impact sound may also be controlled by providing a discontinuity in the structure such as would be obtained by adding a resilient-mounted plaster or drywall suspended ceiling.

Table 8.4.1 Airborne sound transmission and impact insulation class ratings from laboratory tests of hollow core floor-ceiling assemblies

Assembly No.	Description	STC	IIC
1	6 in. hollow core slabs	48	23
2	Assembly 1 with carpet and pad	48	69
3	Assembly 1 with 1/2 in. wood-block flooring adhered directly	48	48
4	Assembly 1 with 1/2 in. wood-block flooring adhered to 1/2 in. sound-deadening board underlayment adhered to concrete	49	49
5	Assembly 1 with 1/2 in. gypsum concrete	50	41
6	Assembly 1 with 3/4 in. gypsum concrete on 1/2 in. sound-deadening board underlayment adhered to concrete	50	50
7	Assembly 1 with carpet and pad on 3/4 in. gypsum concrete on 1/2 in. sound-deadening board underlayment adhered to concrete	50	72
8	8 in. hollow core slabs	50	28
9	Assembly 8 with carpet and pad	50	73
10	Assembly 8 with 1/2 in. wood-block flooring adhered directly	51	47
11	Assembly 8 with 1/2 in. wood-block flooring adhered to 1/2 in. sound-deadening board underlayment adhered to concrete	52	54
12	Assembly 8 with 1/2 in. wood-block flooring adhered to 1/2 in. plywood adhered to 7/16 in. sound-deadening board underlayment adhered to concrete	52	55
13	Assembly 8 with 5/16 in. wood-block flooring adhered to 1/4 in. polystyrene underlayment adhered to concrete	50	51
14	Assembly 8 with vinyl tile adhered to 1/2 in. plywood adhered to 7/16 in. sound-deadening board underlayment adhered to concrete	50	55
15	Assembly 8 with vinyl tile adhered to 1/4 in. inorganic felt-supported, cushion underlayment adhered to concrete	50	51
16	Assembly 8 with vinyl tile adhered to 1/8 in. polyethylene foam underlayment adhered to concrete	50	58
17	Assembly 8 with 1 1/2 in. concrete topping with carpet and pad	50	76
18	Assembly 8 with 1 1/2 in. concrete topping with vinyl tile adhered to concrete	50	44
19	Assembly 8 with 1 1/2 in. concrete topping with vinyl tile adhered to 3/8 in. plywood adhered to 1/2 in. sound-deadening board adhered to concrete	52	55
20	Assembly 8 with 1 1/2 in. concrete with 1 1/2 in. wood-block flooring adhered to 1/2 inch in. sound-deadening board adhered to concrete	51	53
21	Assembly 8 with 1 1/2 in. concrete with 5/16 in. wood-block flooring adhered to foam backing adhered to concrete	51	54
22	Assembly 8 with 3/4 in. gypsum concrete with 5/16 in. wood-block flooring adhered to foam backing adhered to concrete	50	53
23	Assembly 11 with acoustical tile	59	61
24	Assembly 8 with quarry tile, 1 1/4 in. reinforced mortar bed with 0.4 in. nylon and carbon black spinnerette matting	60	54
25	Assembly 24 with suspended 5/8 in. gypsum board ceiling with 3 1/2 in. insulation	61	62

Note: STC = sound transmission class; IIC = impact insulation class.

8.6 Absorption of Sound

A sound wave always loses part of its energy as it is reflected by a surface. This loss of energy is termed sound absorption. It appears as a decrease in sound pressure of the reflected wave. The sound absorption coefficient is the fraction of energy incident, but not reflected, per unit of surface area. Sound absorption can be specified at individual frequencies or as an average of absorption coefficients (NRC).

A dense, nonporous concrete surface typically absorbs 1% to 2% of incident sound and has an NRC of 0.015. In the case where additional sound

Table 8.4.2 Airborne sound transmission class ratings from laboratory tests of hollow core wall systems

Assembly No.	Description	STC
1	6 in. hollow core walls	48
2	Assembly 1 with 2 in. rigid insulation and 2 in. concrete facing	51
3	Assembly 1 with wood furring, $^3/_4$ in. insulation and $^1/_2$ in. gypsum board	51
4	Assembly 1 with $^1/_2$ in. air space, $1^5/_8$ in. metal stud row, $1^1/_2$ in. insulation, and $^1/_2$ in. gypsum board	56
5	8 in. hollow core walls	50
6	Assembly 5 with 2 in. rigid insulation and 2 in. concrete facing	53
7	Assembly 5 with wood furring, $^3/_4$. insulation and $^1/_2$ in. gypsum board	53

Note: STC = sound transmission class.

absorption of precast concrete is desired, a coating of acoustical material can be spray-applied, acoustical tile can be applied with adhesive, or an acoustical ceiling can be suspended. Most of the spray-applied fire-retardant materials used to increase the fire resistance of precast concrete and other floor-ceiling systems can also be used to absorb sound. The NRC of the sprayed fiber types range from 0.25 to 0.75. Most cementitious types have an NRC from 0.25 to 0.50.

If an acoustical ceiling were added to assembly 11 of Table 8.4.1 (as in assembly 23), the sound entry through a floor or roof would be reduced by 7 dB. In addition, the acoustical ceiling would absorb a portion of the sound after entry and provide a few more decibels of quieting. The following expression can be used to determine the intra-room noise or loudness reduction due to the absorption of sound.

$$NR = 10 \log \frac{A_o + A_a}{A_a} \qquad \text{(Eq. 8.6.1)}$$

where

NR = sound-pressure level reduction, dB
A_o = original absorption, sabins
A_a = added absorption, sabins

Values for A_o and A_a are the products of the absorption coefficients of the various room materials and their surface areas.

A plot of this equation is shown in Fig. 8.6.1. For an absorption ratio of 5, the decibel reduction is 7 dB. Note that the decibel reduction is the same, regardless of the original sound-pressure level and depends only on the absorption ratio. This is due to the fact that the decibel scale is

Figure 8.6.1 Relation of decibel reduction of reflected sound to absorption ratio

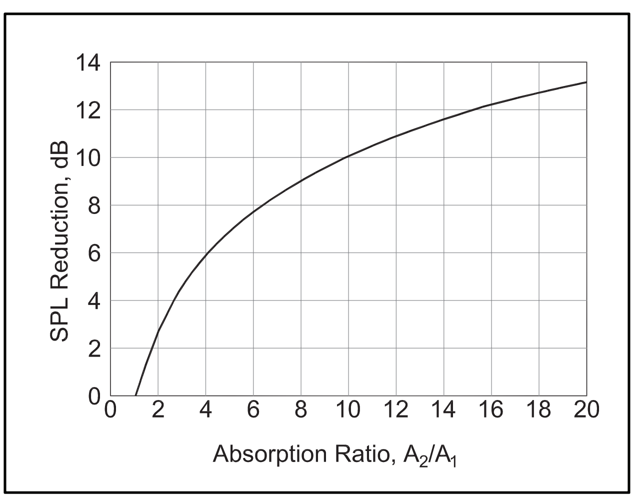


Figure 8.6.2 Relation of percent loudness reduction of reflected sound to absorption ratio

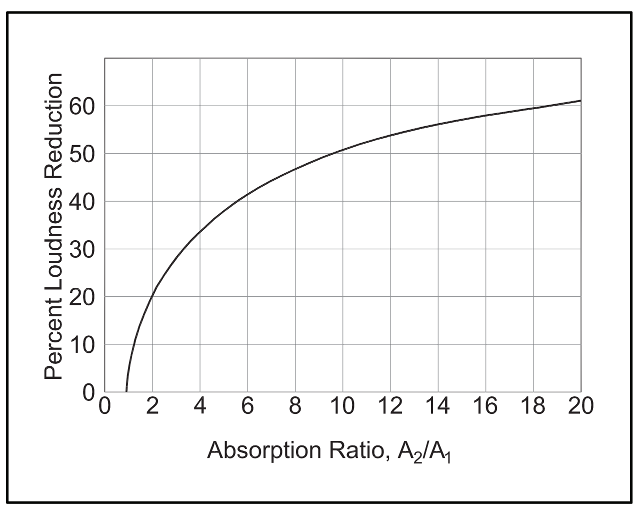


Figure 8.7.1a NC (Noise Criterion)

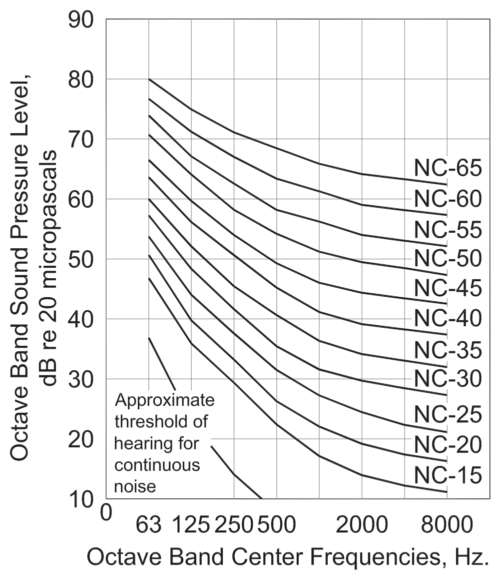
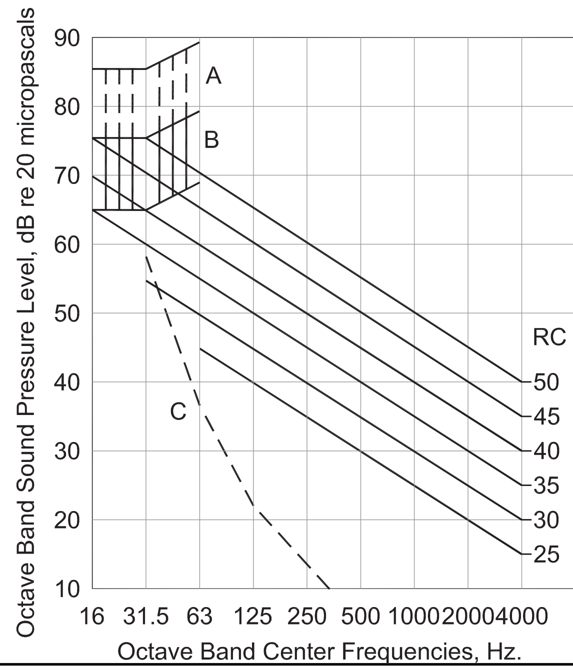


Figure 8.7.1b RC (Room Criterion) Curves



Region A: High probability that noise-induced vibration levels in lightweight wall/ceiling construction will be clearly felt; anticipate audible rattles in light fixtures, doors, windows, etc.
Region B: Noise-induced vibration in lightweight wall/ceiling construction may be moderately felt; slight possibility of rattles in light fixtures, doors, windows, etc.
Region C: Below threshold of hearing for continuous noise

itself a scale of ratios, rather than a difference in sound energy.

While a decibel difference is an engineering quantity that can be physically measured, it is also important to know how the ear judges the change in sound energy due to sound conditioning. Apart from the subjective annoyance factors associated with excessive sound reflection, the ear can make accurate judgments of the relative loudness between sounds. An approximate relation between percentage loudness, reduction of reflected sound, and absorption ratio is plotted in Fig. 8.6.2

The percentage loudness reduction does not depend on the original loudness, but only on the absorption ratio. (The curve is drawn for loudness within the normal range of hearing and does not apply to extremely faint sounds.) Referring again to the absorption ratio of 5, the loudness reduction from Fig. 8.6.2 is approximately 40%.

8.7 Acceptable Noise Criteria

As a rule, a certain amount of continuous sound can be tolerated before it becomes noise. An "acceptable" level neither disturbs room occupants nor interferes with the communication of wanted sound.

The most widely accepted and used noise criteria are expressed as the noise criterion (NC) curves (Fig. 8.7.1a). The values in Table 8.7.1 represent general acoustical goals. They can also be compared with anticipated noise levels in specific rooms to assist in evaluating noise reduction problems.

The main criticism of NC curves is that they are too permissive when the control of low- or high-frequency noise is of concern. For this reason, room criterion (RC) curves were developed (Fig. 8.7.1b). These curves are the result of extensive studies based on the human response to both sound-pressure levels and frequency and take into account the requirements for speech intelligibility.

A low background level is necessary where listening and speech intelligibility is important. Conversely, higher levels can exist in large business offices or factories where speech communication is limited to short distances. Often, it is just as important to be interested in the minimum as in the maximum permissible levels of Table 8.7.1. In an office or residence, it is desirable to have a certain ambient sound level to ensure adequate acoustical privacy between spaces, thus minimizing the transmission loss requirements of unwanted sound (noise).

Table 8.7.1 Recommended category classification and suggested noise criteria range for steady background noise as heard in various indoor functional activity areas*

Type of Space	NC or RC Curve
Private residences	25–30
Apartments	30–35
Hotels/motels	
Individual rooms or suites	30–35
Meeting/banquet rooms	30–35
Halls, corridors, lobbies	35–40
Service/support areas	40–45
Offices	
Executive	25–30
Conference rooms	25–30
Private	30–35
Open-plan areas	35–40
Computer/business machine areas	40–45
Public circulation	40–45
Hospitals and clinics	
Private rooms	25–30
Wards	30–35
Operating rooms	25–30
Laboratories	30–35
Corridors	30–35
Public areas	35–40
Churches	25–30†
Schools	
Lecture and classrooms	25–30
Open-plan classrooms	30–35†
Libraries	30–35
Concert halls	†
Legitimate theaters	†
Recording studios	†
Movie theaters	30–35

Note: NC = Noise criterion curve; RC = room criterion curve.

* Design goals can be increased by 5 dB when dictated by budget constraints or when noise intrusion from other sources represents a limiting condition.

† An acoustical expert should be consulted for guidance on these critical spaces

These undesirable sounds may be from an exterior source such as automobiles or aircraft, or they may be generated as speech in an adjacent classroom or music in an adjacent apartment.

They may be direct impact-induced sound such as footfalls on the floor above, rain impact on lightweight roof construction, or vibrating mechanical equipment.

Thus, the designer must always be ready to accept the task of analyzing the many potential sources of intruding sound as related to their frequency characteristics and the rates at which they occur. The level of toleration that is to be expected by those who will occupy the space must also be established. Figures 8.7.2 and 8.7.3 are the spectral characteristics of common noise sources.

With these criteria, the problem of sound isolation now must be solved, namely, the reduction process between the high noise source and the desired ambient level. For this solution, two related, yet mutually exclusive, processes must be incorporated, that is, sound transmission loss and sound absorption.

8.8 Establishment of Noise Insulation Objectives

Acoustical control is often specified as to the minimum insulation values of the dividing partition system. Building codes, lending institutions, and the Department of Housing and Urban Development (HUD) list both airborne STC and IIC values for different living environments. For example, the International Building Code[35] requires both an STC and an IIC of 50 (45 if field tested), while the HUD minimum property standards are:

Location	STC	IIC
Between living units	45	45
Between living units and public space	50	50

Once the objectives are established, the designer should refer to available data, such as those in Fig. 8.4.2 or Table 8.4.1, and select the system that best meets these requirements. In this respect, concrete systems have superior properties and

Figure 8.7.2 Sound pressure levels — exterior noise sources

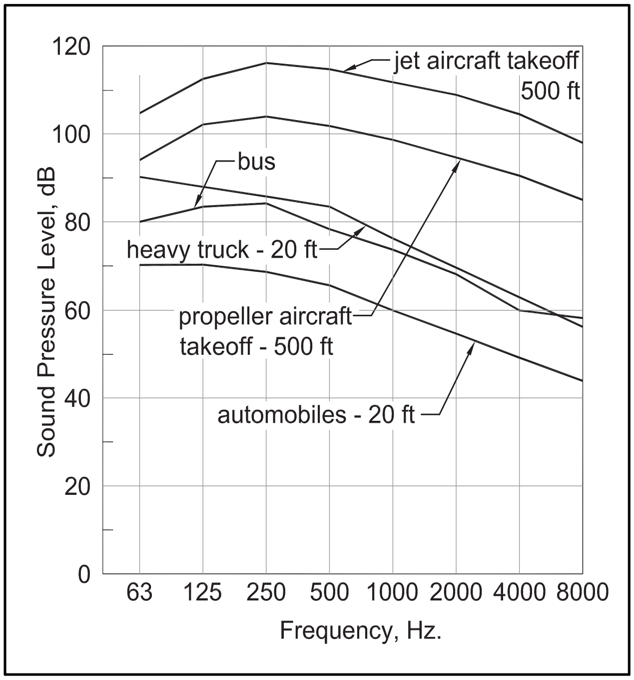
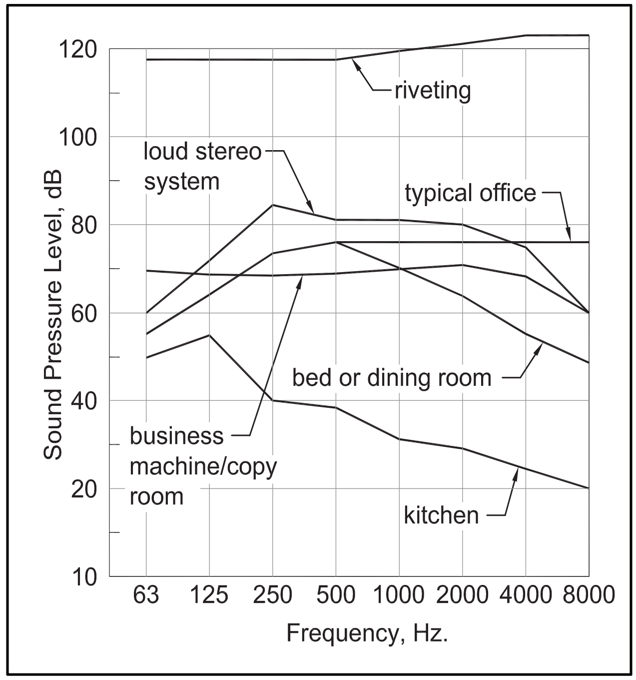


Figure 8.7.3 Sound pressure levels — interior noise sources



can, with minimal effort, comply with these criteria. When the insulation value has not been specified, selection of the necessary barrier can be determined analytically by identifying exterior and/or interior noise sources and by establishing acceptable interior noise criteria.

Example 8.8.1

Given an apartment building with hollow core floor slabs, use Fig. 8.4.2, 8.7.1b, and 8.7.3 to determine the degree of acoustical insulation required of the floor-ceiling assembly.

Table 8.8.1 contrasts the sound-pressure level generated (stereo source noise) with the acceptable threshold (RC). Using the 500 Hz requirement (47 dB) as the first approximation of the floor STC category, an 8-in.-thick hollow core slab is selected. Table 8.8.2 summarizes the comparison of the required insulation to the acoustical values of this size slab.

Ideally, the selected floor should meet or exceed the insulation needs at all frequencies. However, experience has shown that deficiencies of 3 dB at two frequencies or 5 dB at one frequency point can be tolerated.

8.9 Wall Considerations

An acoustically composite wall is made up of elements of varying acoustical properties. Doors and windows are often the weak link in an otherwise effective sound barrier. The sound transmission loss of windows will be affected by the type of glass assembly specified, as seen in Table 8.9.1. Mounting of the glass in its frame should be done with care to minimize noise leaks and reduce the glass-plate vibrations.

Sound transmission loss of a door is dependent on its material and construction and the sealing between the door and frame. Gaskets, weatherstripping, and raised thresholds serve as excellent thermal and acoustic barriers and are recommended.

For best results, the distances between adjacent door and/or window openings should be maximized, staggered when possible, and held to a minimum area. Minimizing the opening area retains the acoustical properties of the precast concrete.

Figure 8.9.1 can be used to calculate the acoustic properties of a wall system that consists of a composite of elements, each with known transmission-loss values.

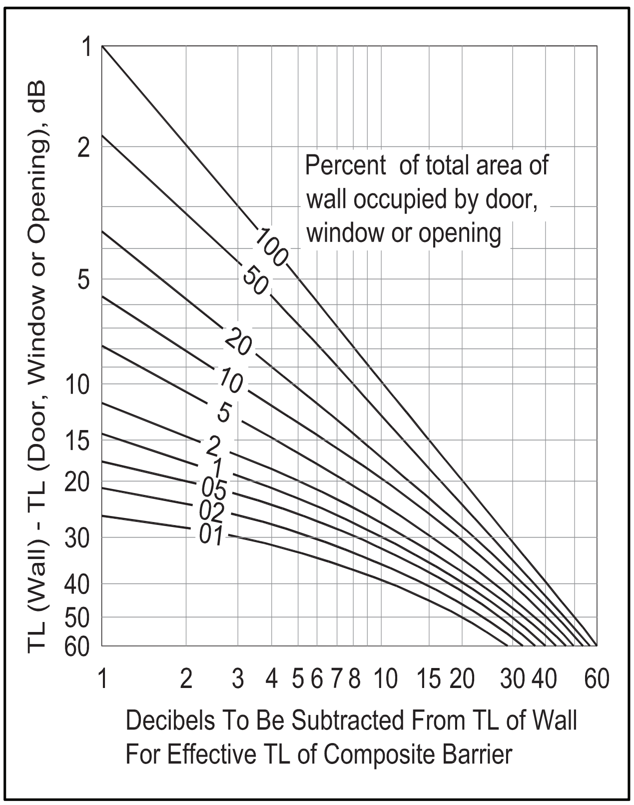
Table 8.8.1 Example 8.8.1 — Acoustical insulation demand

Sound Pressure Level (dB)								
Frequency (Hz)	**63**	**125**	**250**	**500**	**1000**	**2000**	**4000**	**8000**
Stereo source noise (Fig. 8.7.3)	60	72	84	82	82	80	75	60
Bedroom room criteria RC30 (Fig. 8.7.1b)	50	45	40	35	30	25	20	15
Required insulation	10	27	44	47	52	55	55	45

Table 8.8.2 Example 8.8.1 — Acoustical insulation provided

Sound Pressure Level (dB)						
Frequency (Hz)	**125**	**250**	**500**	**1000**	**2000**	**4000**
Required insulation	27	44	47	52	55	55
8-in.-thick hollow core slab (Fig. 8.4.2)	34	39	47	53	58	63
Deficiencies	—	5	—	—	—	—

Figure 8.9.1 Chart for calculating the effective transmission loss of a composite barrier



Example 8.9.1

Given a precast concrete office building adjacent to a major highway, with private offices along the perimeter, determine the degree of insulation required for the exterior wall system. Considering the precast concrete wall without any openings, Table 8.9.2 compares the generated sound levels with the acceptable threshold to determine the required acoustical insulation.

The 500 Hz requirement, 38 dB, is used as the first approximation of the wall STC category. Recognizing that there will be windows in the wall, a system with an STC of 50 is selected. Table 8.9.3 summarizes the comparison of a windowless, 8-in.-thick, hollow core wall (STC of 50) with the insulation requirements.

To complete the design, the effect of the window openings must be considered. Assuming the following:

1. The glazing area represents 10% of the exterior wall area.
2. The windows are double glazed with a 38 STC acoustical rating.

Table 8.9.4 includes the effects of windows on the acoustical performance of the wall.

Table 8.9.1 Acoustical properties of glass

(a) Sound Transmission Class (STC)

Type and Overall Thickness	Inside Light	Construction Space	Outside Light	STC
1/8 in. plate or float	—	—	1/8 in.	23
1/4 in. plate or float	—	—	1/4 in.	28
1/2 in. plate or float	—	—	1/2 in.	31
1 in. insulated glass	1/4 in.	1/2 in. air space	1/4 in.	31
1/4 in. laminated	1/8 in.	0.030 in. vinyl	1/8 in.	34
1 1/2 in. insulated glass	1/4 in.	1 in. air space	1/4 in.	35
3/4 in. plate or float	—	—	3/4 in.	36
1 in. insulated glass	1/4 in.	1/2 in. air space	1/4 in. laminated	38
1 in. plate or float	—	—	1 in.	37
2 3/4 in. insulated glass	1/4 in.	2 in. air space	1/2 in.	39
4 3/4 in. insulated glass	1/4 in.	4 in. air space	1/2 in.	40
6 3/4 in. insulated glass	1/4 in.	6 in. air space	1/4 in. laminated	42

(b) Transmission Loss, dB

Frequency, Hz															
125	160	200	250	315	400	500	630	800	1000	1250	1600	2000	2500	3150	4000
1/4 in. plate glass — 28 STC															
24	22	24	24	21	23	21	23	26	27	33	36	37	39	40	40
1 in. insulating glass with 1/2 in. air space — 31 STC															
25	25	22	20	24	27	27	30	32	33	35	34	29	31	33	36
1 in. insulating glass laminated with 1/2 inch air space — 38 STC															
30	29	26	28	31	34	35	37	37	38	38	40	41	40	41	41

Table 8.9.2 Example 8.9.1 — Acoustical insulation demand

Sound-Pressure Level, dB							
Frequency, Hz	**63**	**125**	**250**	**500**	**1000**	**2000**	**4000**
Bus traffic Source noise (Fig. 8.7.2)	80	83	85	78	74	68	62
Private office Room criteria curve 35 (Fig. 8.7.1b)	55	50	45	40	35	30	25
Required insulation	25	33	40	38	39	38	37

Table 8.9.3 Example 8.9.1 — Acoustical insulation provided (no windows)

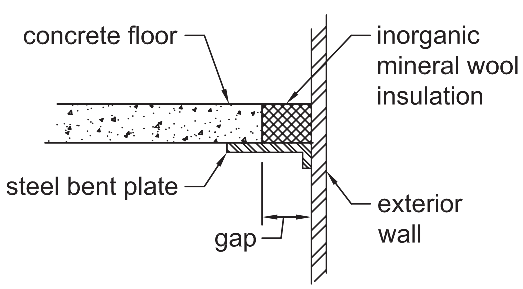
Sound-Pressure Level, dB						
Frequency, Hz	**125**	**250**	**500**	**1000**	**2000**	**4000**
Required insulation	33	40	38	39	38	37
8-in. thick hollow core wall (Fig. 8.4.2)	33	39	47	53	58	63
Deficiencies	—	1	—	—	—	—

Table 8.9.4 Example 8.9.1 — Acoustical insulation provided (10% windows)

Sound-Pressure Level, dB						
Frequency, Hz	**125**	**250**	**500**	**1000**	**2000**	**4000**
8 in. hollow core wall (Fig. 8.4.2)	33	39	47	53	58	63
Double glazed windows (Table 8.9.1[b])	30	28	35	38	41	44
Correction (Fig. 8.9.1)	0	-4	-4	-6	-8	-9
Combined transmission loss	33	35	43	47	50	54
Insulation requirements	33	40	38	39	38	37
Deficiencies	—	5	—	—	—	—

The maximum deficiency is 5 dB and occurs at only one frequency point. This is generally considered acceptable.

Figure 8.10.1 Effect of safing insulation seals



	Combined Transmission Loss
No closure	14 STC
With steel vent plate closure	28 STC
With 4 in.- thick safing insulation	30 STC
steel bent plate added	42 STC
With 6 in.- thick safing insulation	38 STC
steel bent plate added	45 STC

8.10 Leaks and Flanking

The performance of a building section with an otherwise adequate STC can be seriously reduced by a relatively small hole or any other path that allows sound to bypass the acoustical barrier. All noise that reaches a space by paths other than through the primary barrier is called flanking. Common flanking paths are openings around doors or windows, at electrical outlets, telephone and television connections, and pipe and duct penetrations. Suspended ceilings in rooms where walls do not extend from the ceiling to the roof or floor above allow sound to travel to adjacent rooms.

Anticipation and prevention of leaks begins at the design stage. Flanking paths (gaps) at the perimeters of interior precast concrete walls and floors are generally sealed during construction with grout or drypack. In addition, all openings around penetrations through walls or floors should be as small as possible and must be sealed air-tight. The higher the STC of the barrier, the greater the effect of an unsealed opening.

Perimeter leakage more commonly occurs at the intersection between an exterior curtain wall and floor slab. It is of vital importance to seal this gap to retain the acoustical integrity of the system, as well as provide the required fire stop between floors. One way to achieve this seal is to place a 4 lb/ft^3 density mineral wool blanket between the floor slab and the exterior wall. Figure 8.10.1 demonstrates the acoustical isolation effects of this treatment.

In exterior walls, the proper application of sealant and backup materials in the joints between walls will not allow sound to flank the wall.

If the acoustical design is balanced, the maximum amount of acoustic energy reaching a space via flanking should not equal the energy transmitted through the primary barriers.

Although not easily quantified, an inverse relationship exists between the performance of an element as a primary barrier and its propensity to transmit flanking sound. In other words, the probability of existing flanking paths in a concrete structure is much less than in one with a steel or wood frame.

In addition to using basic structural materials, flanking paths can be minimized by interrupting the continuous flow of energy with dissimilar material, such as expansion or control joints or air gaps; or increasing the resistance to energy flow with floating floor systems, full height and/or double partitions, and suspended ceilings.

8.11 Human Response to Building Vibrations

Modern buildings often use components with low weight-to-strength ratios, which allow longer spans with less mass. This trend increasingly results in transient vibrations that may be annoying to the occupants. These vibrations often went unnoticed in older structures with heavier framing and more numerous and heavier partitions, which provided greater damping and other beneficial dynamic characteristics.

While vibration analysis has progressed greatly, there are still many aspects that require engineering judgment rather than pure calculations. Human perception and degree of damping can only be estimated and are subject to substantial variation. Therefore, vibration calculations should only be considered guidelines and not strict limits. When the acceptability of a floor system is in doubt, the best solution is to compare it, using the same system of analysis, with similar structures that are known to be acceptable.

Present analysis is based on a resonant vibration model[54]. That is, when the natural frequency of the floor system is close to a forcing frequency and the deflection is significant, motion will be perceptible and perhaps disturbing. Whether a motion is considered disturbing is a function of the activity of the occupant and the damping characteristics of the floor system. An occupant who is seated or lying down is much more sensitive to vibrations than one who is standing or moving. If the floor system dissipates the forcing motion quickly, the occupant is less likely to find the vibrations objectionable.

Much of the vibration theory has been derived from experience with wood and steel floors. While the theories are still valid, the mass and stiffness of hollow core concrete floors make them less susceptible to vibration problems. However, certain circumstances, such as long, shallow spans, and isolated systems, such as walkways and footbridges, may have problems and should be investigated.

8.11.1 Types of Vibration Analysis

Three types of vibrations may need to be analyzed. The analyses differ because the inputs causing the vibration differ.

Walking

As a person walks, the footsteps create vibration of the floor system. In a quiet area such as an office, church, or residence, this vibration may be annoying to other persons sitting or lying down in the same area. Although more than one person may be walking in the same area at the same time, their footsteps are normally not synchronized. Therefore, the analysis is based on the effect of the impact of the steps of one individual walker.

Rhythmic activities

There are many activities in which a large or small group of people participate in more or less synchronized motion. Spectators at sporting events, rock concerts, and other entertainment events often move in unison in response to a cheer, music, or other stimuli. Dance and exercise classes may involve rhythmic, synchronized movements.

In these instances, both the people involved in the activity, as well as those nearby engaged in a quieter activity, may be affected by the vibrations. However, the people engaged in the rhythmic activity have a higher level of tolerance for the induced vibrations than those working nearby.

Mechanical equipment

Mechanical equipment may produce a constant impulse at a fixed frequency, causing the structure to vibrate.

Analysis methods

Because the nature of the input varies, each of the three input types described requires a different solution. However, all cases require calculation of an important response parameter of the floor system, its natural frequency of vibration.

Using consistent units

Because these calculations involve equations and measures for which most engineers have no "feel," it is important to be careful about maintaining consistent units.

All of the equations in this section are dimensionally correct when using units of kips, inches, and seconds. When quantities using other dimensions (such as span in feet, weight in pounds per square foot) are used, they must be converted to kips, inches, and seconds.

8.11.2 Natural Frequency of Vibration

The natural frequency of a floor system is important both for determining how the floor system will respond to forces causing vibrations and for determining how human occupants will perceive these vibrations. It has been found that certain frequencies seem to set up resonance with internal organs of the human body, making these frequencies more annoying to people.

The human body is most sensitive to frequencies in the range of 4 Hz to 8 Hz (cycles per second). This range of natural frequencies is common for typical floor systems.

The natural frequency of a vibrating slab is determined by the ratio of its mass (or weight) to its stiffness. The deflection of a simple span slab also depends on its weight and stiffness. A simple relationship exists between deflection and the natural frequency of a uniformly loaded simple-span slab on rigid supports:

$$f_n = 0.18\sqrt{\frac{g}{\Delta_j}} \qquad \text{(Eq. 8.11.2.1)}$$

$$\Delta_j = \frac{5w\ell^4}{384EI} \qquad \text{(Eq. 8.11.2.2)}$$

Because many vibration problems are more critical when the mass (or weight) is low, w should include the dead load plus a minimum realistic live load, not the building code specified live load.

The dynamic modulus of elasticity, as measured by the natural frequency, is higher than the static modulus given in ACI 318-11. Therefore, it is recommended that the modulus in ACI 318-11 be multiplied by 1.2 when computing Δ_j for use in determining f_n.

The deflection of beams or girders supporting the floor system also affects the natural frequency of the floor system. The simple-span deflection Δ_g of the floor girder may be calculated in the same manner as Δ_j. The natural frequency of the floor system may then be estimated by the following equation:

$$f_n = 0.18\sqrt{\frac{g}{\Delta_j + \Delta_g}} \qquad \text{(Eq. 8.11.2.3)}$$

For concrete floor systems supported on walls, Δ_g may be assumed to be zero. For concrete floor systems supported by concrete girders, Δ_g is normally small and often neglected, unless the girders are unusually long or flexible. For concrete floor slabs supported on steel beams, the beam deflection can have a significant effect, and should usually be included when computing f_n.

8.11.3 Minimum Natural Frequency

Floors and floor systems with natural frequencies lower than 3 Hz are not recommended because people may more readily synchronize their actions at lower frequencies.

8.11.4 Graphs of Natural Frequency

Equations 8.11.2.1 and 8.11.2.2 may be combined to produce the following equation for a floor slab on stiff supports:

$$f_n = \left(\frac{1.58}{\ell^2}\right)\sqrt{\frac{E_d Ig}{w}} \qquad \text{(Eq. 8.11.4.1)}$$

Figure 8.11.1 shows the relationship between span and expected natural frequency for various hollow core floor slabs.

8.11.5 Damping

Damping determines how quickly a vibration will decay and die out. This is important because humans will more readily tolerate a vibration of short duration than one that is long lasting.

Damping is usually expressed as a fraction or percent of critical damping. A critically damped system is one where the motion slowly returns to zero without ever completing a cycle of motion in the opposite direction. Real building structures have damping from 1% to a few percent of critical.

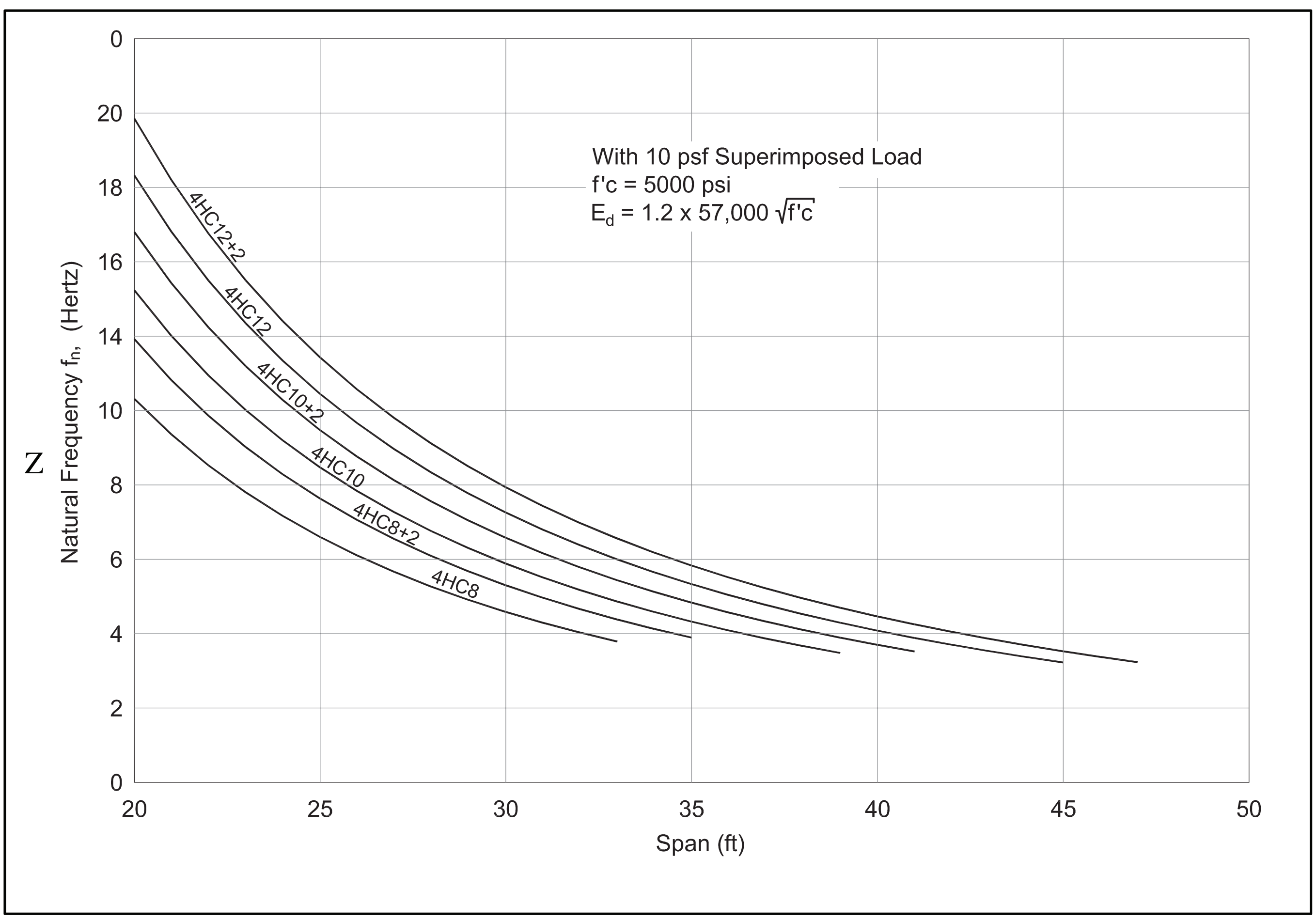
Types of damping

There are two types of damping used in the literature on building vibration, modal damping, and log-decrement damping. All equations and references in this section are based on modal damping. Because the two damping types are not interchangeable, if damping values are obtained from other sources, it is imperative that they are verified to be modal damping values.

Estimation of damping

Damping of a floor system is highly dependent on the non-structural items (partitions, ceilings, furniture) present. The modal damping ratio

Figure 8.11.1 Natural frequency of hollow core floor slabs



of a bare structure can be very low, on the order of 0.01. Non-structural elements may increase this up to 0.05.

The results of a vibration analysis are greatly influenced by the choice of the assumed damping, which can vary widely. Yet this choice is based more on judgment than science.

8.11.6 Vibrations Caused by Walking

Vibrations caused by walking are seldom a problem in hollow core floor systems because of the mass and stiffness of the slab. When using hollow core floor systems of ordinary proportions, it is usually not necessary to check for vibrations caused by walking.

When designing long-span or isolated hollow core floor systems, this section may be used to evaluate their serviceability with respect to vibrations.

Minimum natural frequency

An empirical formula, based on resonant effects of walking, has been developed to determine the minimum natural frequency of a floor system needed to prevent disturbing vibrations caused by walking:

$$f_n \geq 2.86\left[\ln\left(\frac{K}{\beta_m W_f}\right)\right] \qquad \text{(Eq. 8.11.6.1)}$$

where:

K = a constant given in Table 8.11.1

β_m = modal damping ratio

W_f = weight of floor area affected by a point load

The constant 2.86 has units of 1/sec.

Effective weight W_f

The effect of an impact such as a footfall is strongly influenced by the mass (or weight) of the structure affected by the impact. This weight W_f is normally taken as the unfactored unit dead load (per square foot) of the floor slabs plus some (not full code) live load, multiplied by the span and by a width B. For hollow core slabs, which are stiff in torsion, it is recommended that B equal the lesser of the span or the actual width available.

Table 8.11.1

Occupancies Affected by the Vibrations	Constant K, kip	Modal damping ratio β_m
Offices. residences, churches	13	0.02*
		0.03†
		0.05§
Shopping malls	4.5	0.02
Outdoor foot-bridges	1.8	0.01

* For floors with few non-structural components and furnishings, open work areas, and churches
† For floors with non-structural components and furnishings, cubicles
§ For floors with full-height partitions

Recommended values

The recommended values of K and β_m for use in Eq. 8.11.6.1 are given in Table 8.11.1.

Example 8.11.1 Vibrations Caused by Walking

Given: 6-ft-wide by 30-ft-span footbridge made from the generic slab in Fig. 1.6.1

Check for vibration from walking.

Use: $K = 1.8$ kip — Table 8.11.1
$\beta_m = 0.01$

Effective weight:

$W = 53.5$ lb/ft^2 (self weight)
+ 25 lb/ft^2 (non-composite hollow core slab)
+ 15 lb/ft^2 (assumed live load)
= 93.5 lb/ft^2 Total

Converting to units of kips and inches yields:

$$w = \frac{3\text{ ft}\left(93.5\text{ lb/ft}^2\right)}{12\text{ in./ft}}\,\frac{1\text{ kip}}{1000\text{ lb}} = 0.0234\text{ kip/in.}$$

$$W_f = 0.0935(6)(30) = 16.8\text{ kip}$$

From Eq. 8.11.6.1, find the minimum required f_n:

$$f_n = 2.86\left[\ln\left(\frac{1.8}{0.01(16.8)}\right)\right]$$
$$= 6.78 \text{ Hz}$$

Using Eq. 8.11.4.1, calculate the fundamental frequency of this floor system:

$$f_n = \left(\frac{1.58}{360^2}\right)\sqrt{\frac{(1.2)(4030)(1224.5)(386)}{0.0234}}$$
$$= 3.81 \text{ Hz}$$

Note that the dynamic modulus of elasticity, rather than the static modulus, is used, as explained in section 8.11.2.

Because the fundamental frequency is less than the minimum required frequency, the walkway should be stiffened by shortening the span (if possible) or selecting a stiffer/deeper section.

8.11.7 Vibrations Caused by Rhythmic Activities

Rhythmic excitation occurs when a group of people move in unison such as during exercises, coordinated cheers, or in response to a musical beat. Because much more mass (weight) is put into motion than when an individual is walking, the input forces are far more powerful. A resonance can occur when the input frequency is at or near the fundamental frequency of the floor system. Therefore, the fundamental frequency of the floor system must be sufficiently higher than the input frequency to prevent resonance.

Harmonics

A harmonic of a frequency is any higher frequency that is equal to the fundamental frequency multiplied by an integer. If the fundamental frequency of a floor system is equal to a harmonic of the exciting frequency, resonance may occur.

Recommended minimum natural frequency

The following design criterion for the minimum natural frequency for a floor system subjected to rhythmic excitation is based on dynamic response of the floor system to dynamic loading:

$$f_n \geq f_f\sqrt{1+\left(\frac{k_r}{a_o/g}\right)\left(\frac{\alpha_i\, w_p}{w_t}\right)} \quad \text{(Eq. 8.11.7.1)}$$

where:

f_f = forcing frequency
k_r = a dimensionless constant
= 1.3 for dancing
= 1.7 for a lively concert or sporting event
= 2.0 for aerobics
α_i = dynamic coefficient
a_o/g = ratio of peak acceleration limit to the acceleration due to gravity
w_p = effective distributed weight of participants per unit area
w_t = effective total distributed weight per unit area (weight of participants plus floor system)

See Table 8.11.2 for limiting values of a_o/g and Table 8.11.3 for α_i and f_f.

The natural frequency of the floor system f_n is computed as discussed in section 8.11.2.

Recommended values for all of the parameters on the right side of Eq. 8.11.7.1 are given in Tables 8.11.2 and 8.11.3, except for w_t, which includes the actual distributed dead weight of the floor system. Note that Eq. 8.11.7.1 uses the distributed weight w_t not the total weight of a panel W_f that was used in Eq. 8.11.6.1.

Higher harmonics

Equation 8.11.7.1 will always require a higher natural frequency f_n than the forcing frequency f_f.

Table 8.11.2 Recommended Acceleration Limits for Rhythmic Activities

Occupancies Affected by the Vibration	Acceleration Limit, Fraction of Gravity, a_o/g
Office or residential	0.004 — 0.007
Dining	0.015 — 0.025
Weightlifting	0.015 — 0.025
Rhythmic activity only	0.040 — 0.070

Table 8.11.3 Estimated Loading during Rhythmic Events

Activity	Forcing Frequency f_f, Hz	Weight of Participants* w_p, lb/ft^2	Dynamic Coefficient α_i	Dynamic Load $\alpha_i w_p$
Dancing				
First harmonic	1.5 – 3	12	0.50	6.0
Lively concert or sports event				
First harmonic	1.5 – 3	30	0.25	7.5
Second harmonic	3 – 5	30	0.05	1.5
Jumping exercises				
First harmonic	2 – 2.75	4	1.50	6.0
Second harmonic	4 – 5.5	4	0.60	2.4
Third harmonic	6 – 8.25	4	0.10	0.4

* Based on maximum density of participants on the occupied area of the floor for commonly encountered conditions. For special events, the density of participants may be greater.

Thus, a critical decision is the determination of whether the forcing frequencies for higher harmonics need to be considered. Equation 8.11.7.2 gives the peak acceleration a_{peak}/g for a condition of resonance.

$$\frac{a_{peak}}{g} = \left(\frac{1.3}{2\beta_m}\right)\left(\frac{\alpha_i w_p}{w_t}\right) \qquad \text{(Eq. 8.11.7.2)}$$

In applying Eq. 8.11.7.2, the literature[55] recommends a value for the damping ratio β_m as follows: "Because participants contribute to the damping, a value of approximately 0.06 may be used, which is higher than … for walking vibration."

If β_m or the total distributed weight w_t is great enough, the dynamic load $\alpha_i w_p$ from Table 8.11.3 for higher harmonics may result in a peak acceleration a_{peak}/g within the acceleration limits a_o/g given in Table 8.11.2. If this is true, that harmonic need not be considered.

Many untopped and most topped hollow core floors weigh 75 lb/ft^2 or more. For these floors, the weight w_t is such that the resonant acceleration at the third harmonic frequency will usually be within limits. Generally, only the first and second harmonics need be considered for hollow core floors.

Adjacent activities

A space with a quiet activity may be located next to a space with rhythmic activity. In such cases, it is desirable to have a rigid wall between the two spaces, supporting the floor system in each space. If this is not practical, the acceleration limits for the quiet activity should be used in combination with the rhythmic loading for the rhythmic activity. This combination can often be critical for hollow core floor systems, requiring a stiffer floor than needed for supporting gravity loads.

Example 8.11.2 Vibrations Caused by Rhythmic Activities

Given: Span = 20 ft
8 in. generic hollow core slab (Fig. 1.6.1)
with 2 in. composite topping
Aerobics gym adjacent to a restaurant

Determine the suitability of this design.

From Eq. 8.11.4.1, the natural frequency of the floor system is calculated as:

$$f_n = \left(\frac{1.58}{\ell^2}\right)\sqrt{\frac{E_d I g}{w}}$$

where: ℓ = 20 ft = 240 in.
E_d = (1.2)(4030) = 4837 ksi

I = 2529.6 in.^{4} (composite section)
g = acceleration due to gravity
= 386 in./sec^2

$$w = \frac{3\,\text{ft}\left(82.5\,\text{lb/ft}^2\right)}{12\ \text{in./ft}}\frac{1\ \text{kip}}{1000\,\text{lb}}$$
= 0.0206 kip/in.

$$f_n = \left(\frac{1.58}{240^2}\right)\sqrt{\frac{(4837)(2529.6)(386)}{0.0206}}$$
= 13.1 Hz

Using Eq. 8.11.7.1 to find the minimum natural frequency for the first harmonic:

$$f_n \geq f_f \sqrt{1+\left(\frac{k_r}{a_o / g}\right)\left(\frac{\alpha_1 w_p}{w_t}\right)}$$

where: k_r = 2.0 for aerobics
f_f = 2.5 Table 8.11.3
a_o/g = 0.020 Table 8.11.2 (note that the dining limit is used)
$\alpha_i w_p$ = 6 Table 8.11.3
w_t = 53.5 + 25 + 4 = 82.5 lb/ft^2

Note that unit conversion is not necessary provided that the units of w_p and w_t are consistent.

$$f_n = 2.5\sqrt{1+\left(\frac{2.0}{0.020}\right)\left(\frac{6}{82.5}\right)}$$
= 7.19 Hz

Because the natural frequency is greater than the required minimum, the floor system is satisfactory for the first harmonic.

Check the minimum natural frequency of the second harmonic in a similar fashion.

$$f_n = 5.0\sqrt{1+\left(\frac{2.0}{0.020}\right)\left(\frac{2.4}{82.5}\right)} = 9.89\ \text{Hz}$$

The natural frequency is also greater than the second harmonic. Therefore, the floor system is acceptable for this usage. If more stringent vibration limits were required, the performance could be improved by increasing the stiffness of the floor system, decreasing the span, or isolating the aerobics gym from the quiet space.

8.11.8 Vibration Isolation for Mechanical Equipment

Vibration produced by equipment with unbalanced operating or starting forces can usually be isolated from the structure by mounting the equipment on a heavy concrete slab placed on resilient supports. This type of slab, called an inertia block, provides a low center of gravity to compensate for thrusts such as those generated by large fans.

For equipment with less unbalanced weight, a "housekeeping" slab is sometimes used below the resilient mounts to provide a rigid support for the mounts and to keep them above the floor so they are easier to clean and inspect. This slab may also be mounted on pads of precompressed glass fiber or neoprene.

The natural frequency of the total load on resilient mounts must be well below the frequency generated by the equipment. The required weight of an inertia block depends on the total weight of the machine and the unbalanced force. For a long-stroke compressor, five to seven times its weight might be needed. For high pressure fans, one to five times the fan weight is usually sufficient.

A floor supporting resiliently mounted equipment must be much stiffer than the isolation system. If the static deflection of the floor approaches the static deflection of the mounts, the floor becomes a part of the vibrating system, and little vibration isolation is achieved. In general, the floor deflection should be limited to about 15% of the deflection of the mounts.

Simplified theory shows that for 90% vibration isolation, a single resilient supported mass (isolator) should have a natural frequency of about one-third the driving frequency of the equipment. The natural frequency of this mass can be calculated by:

$$f_n = 188\sqrt{\frac{1}{\Delta_i}} \qquad \text{(Eq. 8.11.8.1)}$$

where:
f_n = natural frequency of the isolator, cycles per minute (CPM)
Δ_i = static deflection of the isolator, in.

The required static deflection of an isolator can be determined as follows:

$$f_n = \frac{f_d}{3} = 188\sqrt{\frac{1}{\Delta_i}} \quad \text{or}$$

$$\Delta_i = \left(\frac{564}{f_d}\right)^2 \qquad \text{(Eq. 8.11.8.2)}$$

and:

$$\Delta_f \leq 0.15\,\Delta_i \qquad \text{(Eq. 8.11.8.3)}$$

where:

f_d = driving frequency of the equipment

Δ_f = static deflection of the floor system caused by the weight of the equipment, including the inertia block, at the location of the equipment

Example 8.11.3 - Vibration Isolation

Given:

A piece of mechanical equipment has a driving frequency of 800 CPM.

Determine the approximate minimum deflection of the isolator and the maximum deflection of the floor system that should be allowed.

Isolator, $\Delta_i = \left(\frac{564}{800}\right)^2 = 0.50\,\text{in.}$

Floor, $\Delta_f = 0.15(0.50) = 0.07$ in.

Chapter 9

GUIDE SPECIFICATION FOR PRECAST, PRESTRESSED HOLLOW CORE SLABS

This Guide Specification is intended to be used as a basis for the development of an office master specification or in the preparation of specifications for a particular project. In either case, this Guide Specification must be edited to fit the conditions of use.

Particular attention should be given to the deletion of inapplicable provisions. Necessary items related to a particular project should be included. Also, appropriate requirements should be added where blank spaces have been provided. Coordinate the specification with the information shown in the Contract Drawings to avoid duplication or conflicts.

The Guide Specifications are on the left. *Notes to Specifiers are on the right.*

GUIDE SPECIFICATIONS

1. GENERAL

1.01 Description

A. Work Included:

1. These specifications cover manufacture, transportation, and erection of precast, prestressed concrete hollow core slabs including grouting of joints between adjacent slab units.

B. Related Work Specified Elsewhere:

1. Cast-in-Place Concrete: Section ____.
2. Architectural Precast Concrete: Section ____.
3. Precast Structural Concrete: Section ____.
4. Structural Metal Framing: Section ____.
5. Masonry Bearing Walls: Section ____.
6. Underlayments: Section ____.
7. Caulking and Sealants: Section ____.

NOTES TO SPECIFIERS

1.01.A This Section is to be in Division 3 of Construction Specifications Institute format.

1.01.B.l Includes structural or non-structural topping. See Section 2.5 for discussion of composite, structural topping.

1.01.B.3 Beams, columns, etc.
Prestressed concrete may be specified in Section

1.01.B.4 Includes support framework not supplied by Hollow Core Slab Manufacturer.

1.01.B.5 Include any inserts or anchoring devices required for slab connections.

1.01.B.6 Underlayment may be any of the following general types: asphaltic concrete, gypsum concrete, latex concrete, mastic underlayment.

1.01.B.7 Caulking between slab edges at exposed underside of floor members and/or perimeter caulking may be included in this section.

GUIDE SPECIFICATIONS

8. Holes for Mechanical Equipment: Section ____.

9. Painting: Section ____.

10. Carpet and Pad: Section ____.

11. Roofing and Roof Insulation: Section ____.

1.02 Quality Assurance

A. The *precast concrete manufacturing plant* shall be certified by the Precast/Prestressed Concrete Institute (PCI) Plant Certification Program. Manufacturer shall be certified at the time of bidding in Category C2.

B. Erector Qualifications: Regularly engaged for at least _________ years in the erection of *precast structural concrete* similar to the requirements of this project.

1. Erector Certification: A precast concrete erector with erecting organization and all erecting crews Certified and designated, prior to beginning work at project site, by PCI's Certified Erectors Certificate to erect [Category S1 (Simple Structural Systems) for horizontal decking members and single-lift wall panels]

NOTES TO SPECIFIERS

1.01.B.8 Holes may be drilled or cut and trimmed with a chisel. Cut outline of hole through lower portion of slab from underside, after which the top side may be removed from above. Do not cut prestressing strand without permission of Specialty Structural Engineer.

1.01.B.9 Prime coat should be a latex base paint. Finish coat may be an oil base, flat wall or emulsified finish.

1.01.B.10 Specify minimum 55 oz. pad when no cast-in-place topping is used.

1.01.B.11 Non-absorbent rigid board insulation 1" or more in thickness should be used on roofs. Check local energy code for exact requirements.

1.02.A Structural Precast Products must meet the requirements of PCI Manual, MNL-116.

In Canada, the manufacture, transportation and erection of precast prestressed hollow core slabs is governed by the Canadian Standards Association Standard A23.4-94, "Precast Concrete - Materials and Construction".

Assurance of plant capability to produce quality precast concrete products is set by the CSA Standard A23.4-94.(2014) This Standard forms the basis of a certification program which sets rigid capability criteria for precast manufacturers, their personnel and operations.

GUIDE SPECIFICATIONS

C. Qualifications of Welders: Qualify procedures and personnel according to AWS Dl.l/D1.1M.

D. Testing: In general compliance with applicable provisions of Precast/Prestressed Concrete Institute MNL-116, Manual for Quality Control for Plants and Production of Precast Prestressed Concrete Products.

E. Requirements of Regulatory Agencies: All local codes plus the following specifications, standards and codes are a part of these specifications:

1. ACI 318-Building Code Requirements for Structural Concrete.
2. AWS D1.1/D1.1M-Structural Welding Code - Steel.
3. AWS D1.4/D1.4M-Structural Welding Code Reinforcing Steel.
4. ASTM Specifications- As referred to in Part 2 - Products, of this Specification

1.03 Submittals

A. Shop Drawings

1. Erection Drawings

a. Plans locating and defining all hollow core slab units furnished by the manufacturer, with all openings larger than 10 in (250 mm) shown and located.

b. Sections and details showing connections, edge conditions and support conditions of the hollow core slab units.

c. All dead, live and other applicable loads used in the design.

d. Estimated cambers.

NOTES TO SPECIFIERS

1.02.C Qualified within the past year.

1.02.E Always include the specific year or edition of the specifications, codes and standards used in the design of the project and made part of the specifications. Fire safety and resistance requirements are specified in local or model codes. When required, fire rated products shall be clearly identified on the design drawings.

For projects in Canada, the National Building Code of Canada governs design. Canadian Standards Association Standards A23.3-14, "Design of Concrete Structures" and A23.4-94,(2014) "Precast Concrete - Materials and Construction" also apply. Fire resistance is specified in the National Building Code and the National Fire Code.

1.03.A.1.a Openings shown on erection drawings are considered in the slab design. Verify slab adequacy for any other openings with the Structural Engineer of Record.

1.03.A.1.d Floor slabs receiving cast-in-place topping. The elevation of top of floor and amount of concrete topping must allow for camber of prestressed concrete members.

GUIDE SPECIFICATIONS

2. Production Drawings
   a. Plan view of each hollow core slab unit type.
   b. Sections and details to indicate quantities, location and type of reinforcing steel and prestressing strands.
   c. Lifting and erection inserts.
   d. Dimensions and finishes.
   e. Prestress for strand and concrete strength.
   f. Estimated camber at release.
   g. Method of transportation.

B. Product Design Criteria
1. Loadings for design
   a. Initial handling and erection stresses.
   b. All dead and live loads as specified on the contract drawings.
   c. All other loads specified for hollow core slab units where applicable.
2. Design calculations of *products* not completed on the contract drawings shall be performed by a registered engineer experienced in *precast prestressed concrete design* and submitted for approval upon request.
3. Design shall be in accordance with ACI 318 or applicable codes.

C. Permissible Design Deviations
1. Design deviations will be permitted only after the Architect/Engineer of Record's written approval of the manufacturer's proposed design supported by complete design calculations and drawings.
2. Design deviation shall provide an installation equivalent to the basic intent without incurring additional cost to the owner.

D. Test Report: Reports of tests on concrete and other materials upon request.

NOTES TO SPECIFIERS

1.03.A.2 Production drawings are normally submitted only upon request

1.03.B and C Contract drawings normally will be prepared using a local precast prestressed concrete hollow core slab manufacturer's design data and load tables. Dimensional changes which would not materially affect architectural and structural properties or details usually are permissible.

Be sure that loads shown on the contract draw-ings are easily interpreted. For instance, on members which are to receive concrete topping, be sure to state whether all superimposed dead and live loads on precast prestressed members do or do not include the weight of the concrete topping. It is best to list the live load, superimposed dead load, topping weight, and weight of the member, all as separate loads. Where there are two different live loads (e.g., roof level of a parking structure) indicate how they are to be combined. Where additional structural support is required for openings, design headers in accordance with hollow core slab manufacturer's recommendations.

GUIDE SPECIFICATIONS

2. PRODUCTS

2.01 Materials

A. Portland Cement:
1. ASTM C150 — Type I or III

B. Admixtures:
1. Air-Entraining Admixtures: ASTM C260.
2. Water Reducing, Retarding, Accelerating, High Range Water Reducing Admixtures: ASTM C494

C. Aggregates:
1. ASTM C33 or C330.

D. Water:

Potable or free from foreign materials in amounts harmful to concrete and embedded steel.

E. Reinforcing Steel:
1.Bars:
Deformed Billet Steel: ASTM A615/A615M.
Deformed Low Alloy Steel: ASTM A706/A706M.
2.Wire:
Cold Drawn Steel: ASTM A1064/A1064M.

F. Prestressing Strand:
1. Uncoated, 7-Wire, Stress-Relieved Strand: ASTM A416/A416M (including supplement) - Grade 250K or 270K.
2. Uncoated, Weldless 2- and 3-Wire Strand: ASTMA910/A910M
3. Indented, 7-Wire, Stress-Relieved Strand: ASTM A886/A886M (including supplement)

G. Welded Studs: In accordance with AWS Dl.l.

H. Structural Steel Plates and Shapes: ASTM A36/A36M.

NOTES TO SPECIFIERS

2.01 Delete or add materials that may be required for the particular job.

2.01.B Verify ability of local producer to use admixtures.

2.01.E.1 When welding of bars is required, weldability must be established to conform to AWS D1.4/D1.4M.

2.01.F Low-relaxation strand is the predominant strand in use. References to stress-relieved strand are from the ASTM titles.

2.01.H When required for anchorage or lateral bracing to structural steel members, some methods of manufacturing hollow core slabs preclude the use of anchors and inserts

GUIDE SPECIFICATIONS

I. Grout:

1. Cement grout: Grout shall be a mixture of not less than one part portland cement to 2 ½ to 3 parts fine sand, and the consistency shall be such that joints can be completely filled but without seepage over adjacent surfaces. Any grout that seeps from the joint shall be completely removed before it hardens.

J. Bearing Strips:

1. Random Oriented Fiber Reinforced: Shall support a compressive stress of 3000 psi (20.7 MPa) with no cracking, splitting or delaminating in the internal portions of the pad. One specimen shall be tested for each 200 pads used in the project.

2. High Density Plastic: Multi-monomer plastic strips shall be non-leaching and support construction loads with no visible overall expansion.

3. Tempered Hardboard: AHA A135.4 Class 1, smooth on both sides.

4. Untempered Hardboard

2.02 Concrete Mixes

A. 28-day compressive strength: Minimum of _____ psi.

B. Release strength: Minimum of ______psi.

C. Use of calcium chloride, chloride ions or other salts is not permitted.

2.03 Manufacture

A. Manufacturing procedures shall be in compliance with PCI MNL-116.

B. Manufacturing Tolerances: Manufacturing tolerances shall comply with PCI MNL-135.

NOTES TO SPECIFIERS

2.01.I Grout strengths of 2000 psi to 3000 psi (13.8- 20.7 MPa) can generally be achieved with the proportions noted. Rarely is higher strength grout required. Non-shrink grout is not required for satisfactory performance of hollow core slab systems.

2.01.J.1 Standard guide specifications are not available for random-oriented, fiber-reinforced pads. Proof testing of a sample from each group of 200 pads is suggested. Normal design working stresses are 1500 psi (10.3 MPa), so the 3000 psi (20.7 MPa) test load provides a factor of 2 over design stress. The shape factor for the test specimens should not be less than 2.

2.01.J.2 Plastic pads are widely used with hollow core slabs. Compression stress in use is not normally over a few hundred psi and proof testing is not considered necessary. No standard guide specifications are available.

2.01.J.3 Hardboard bearing strips should not be used in areas where undesirable staining is possible or where bearing strips may be continually wet.

2.02.A and B Verify with local manufacturer. 5000(35MPa) psi for prestressed products is nor- mal practice, with release strength of 3000 psi (20.7 MPa).

GUIDE SPECIFICATIONS

C. Openings: Manufacturer shall provide for those openings 10 in (250 mm) round or square or larger as shown on the structural drawings. Other openings shall be located and field drilled or cut by the trade requiring them after the hollow core slab units have been erected. Openings and/or cutting of prestressing strand shall be approved by Architect/Engineer of Record and manufacturer before drilling or cutting.

D. Patching: Will be acceptable providing the structural adequacy of the hollow core unit is not impaired.

3. EXECUTION

3.01 Product Delivery, Storage, and Handling

A. Delivery and Handling:

1. Hollow core slab units shall be lifted and supported during manufacturing, stockpiling, transporting and erection operations only at the lifting or supporting point, or both, as shown on the shop drawings, and with approved lifting devices. Lifting inserts shall have a minimum safety factor of 4. Exterior lifting hardware shall have a minimum safety factor of 5.
2. Transportation, site handling, and erection shall be performed with acceptable equipment and methods, and by qualified personnel.

B. Storage:

1. Store all units off ground.
2. Place stored units so that identification marks are discernible.
3. Separate stacked members by battens across full width of each slab unit.
4. Stack so that lifting devices are accessible and undamaged.
5. Do not use upper member of stacked tier as storage area for shorter member or heavy equipment.

NOTES TO SPECIFIERS

2.03.C This paragraph requires other trades to field drill holes needed for their work, and such trades should be alerted to this requirement through proper notation in their sections of the specifications. Some manufacturers prefer to install openings smaller than 10 in (250 mm) which is acceptable if their locations are properly identified on the contract drawings

GUIDE SPECIFICATIONS

3.02 Erection

A. Site Access: The General Contractor shall be responsible for providing suitable access to the building, proper drainage and firm level bearing for the hauling and erection equipment to operate under their own power.

B. Preparation: The General Contractor shall be responsible for:

1. Providing true, level bearing surfaces on all field placed bearing walls and other field placed supporting members.
2. All pipes, stacks, conduits and other such items shall be stubbed off at a level lower than the bearing plane of the prestressed concrete products until after the latter are set.

C. Installation: Installation of hollow core slab units shall be performed by the manufacturer or a certified erector. Members shall be lifted by means of suitable lifting devices at points provided by the manufacturer. Bearing strips shall be set, where required. Temporary shoring and bracing, if necessary, shall comply with manufacturer's recommendations. Grout keys shall be filled.

D. At Slab Ends (where shown on Drawings): Provide suitable end cap or dam in voids as required.

E. For areas where slab voids are to be used as electrical raceways or mechanical ducts provide a taped butt joint at end of slabs, making sure the voids are aligned.

F. Alignment: Members shall be properly aligned and leveled as required by the approved shop drawings. Variations between adjacent members shall be reasonably leveled out by jacking, loading, or any other feasible method as recommended by the manufacturer and acceptable to the

NOTES TO SPECIFIERS

3.02.B Construction tolerances for cast-in-place concrete, masonry, etc., should be specified in those sections of the specifications.

3.02.B.2 Should be in Electrical, Mechanical, and Plumbing sections of project specifications.

3.02.D If a bearing wall building, special care must be taken. Delete when end grouting is not required.

3.02.E Delete when voids not used for electrical or mechanical.

3.02.F Tolerances should comply with industry tolerances published in Tolerance Manual for Precast and Prestressed Concrete Construction (PCI MNL 135-00), Precast/Prestressed Concrete Institute,2000.

GUIDE SPECIFICATIONS

Architect/Engineer.

3.03 Field Welding

A. Field welding is to be done by qualified welders using equipment and materials compatible with the base material.

3.04 Attachments

A. Subject to approval of the Architect/Engineer of Record, hollow core slab units may be drilled or "shot" provided no contact is made with the prestressing steel. Should spalling occur, it shall be repaired by the trade doing the drilling or the shooting.

3.05 Inspection and Acceptance

A. Final observation of erected hollow core slab units shall be made by Architect/Engineer of Record for purposes of final payment.

NOTES TO SPECIFIERS

1. Precast/Prestressed Concrete Institute. 2010. *PCI Design Handbook*: *Precast and Prestressed Concrete*. MNL-120-10, 7th Edition, Chicago, IL (PCI).
2. ACI Committee 306. 2010. *Guide to Cold Weather Concreting* (ACI 306R-10), American Concrete Institute, Farmington Hills, MI.
3. ACI Committee 318. 2011. *Building Code Requirements for Structural Concrete* (ACI 318–11) and *Commentary* (ACI 318R–11), American Concrete Institute, Farmington Hills, MI.
4. PCI Committee on Tolerances. 2000. *Tolerance Manual for Precast and Prestressed Concrete Construction*. (PCI MNL 135-00) Chicago, IL.
5. Zia, Paul, Preston, H. Kent, Scott, Norman L, and Workman, Edwin B., 1979. “Estimating Prestress Losses”, *Concrete International,* (June) pp. 32–38.
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